

Numerical Simulation of Dissipative Anchor Devices in Historic Masonry



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SUMMARY:

Traditional cross-ties are able to restore the box-like behaviour of historic buildings, otherwise affected by the poor performance or complete lack of connections between sets of perpendicular walls. However, the localised increase of stiffness resulting from the insertion of ties in a weak substratum can cause damage such as pull-out or punching. To address this problem, the authors developed a dissipative device that is installed in series with an anchor at the joint of perpendicular walls and allows for controlled displacements, thus reducing the acceleration and the concentration of stresses in the parent material. Finite Element models of the device have been developed on the basis of experimental work. The paper focuses on the dynamic analysis performed on a half-scale model of a masonry building to assess the change in the seismic performance of the structure as consequence of the use of the dissipative devices in respect to the unreinforced and traditionally reinforced set-ups. Results are compared with the preliminary output of shaking table tests.

Keywords: grouted anchors, dissipative devices, historic masonry, shaking table tests, seismic protection

1. INTRODUCTION

Notwithstanding the consolidated implementation of performance based–design for new structures, current codes still acquiesce to the use of traditional stiffness-based techniques for the retrofit of historic buildings (EN 1998 Eurocode 8; Italian Ministry of Cultural Heritage and Activities, 2006). The application of techniques involving ductility and energy dissipation, despite being recommended in principle, is in fact limited since innovative systems rarely meet some of the constraints – reversibility, low impact – required for interventions on historic structures. Indeed, few high-profile case studies appear in the literature (Indirli and Castellano, 2008; Benedetti, 2004; Mandara and Mazzolani, 1994). However, strength-based techniques are unsuitable for historic low shear capacity masonry walls: L'Aquila earthquake, Italy, April 2009 proved once more that elements such as concrete ring beams, due to the elevated mass and stiffness, often aggravated by inadequate connections, concur to cause tragic collapses (D'Ayala and Paganoni, 2011a). Conversely unreinforced masonry buildings benefit from the presence of cross-ties, despite some drawbacks connected to the difference in stiffness between steel and parent material.

Drawing on the above observations, the authors developed, within the framework of a Knowledge Transfer Partnership (KTP) between the University of Bath and Cintec International Ltd, a dissipative device specifically designed to address the lack of passive systems for the seismic protection of heritage buildings. The device is conceived as add-on for Cintec's stainless steel ties © and is designed to be placed at locations where cracking is most likely to occur or is already present, so that in case of relative movements the devices can be activated. Thanks to either the hysteretic properties of a stainless steel element, shaped to optimise its post-elastic behaviour, or a friction mechanism set to be triggered for a certain level of pulling force, the device allows small relative displacements, dissipating energy and hence reducing the impact of seismic force on the walls, thus controlling damage.

Initial experimental work reported elsewhere (Paganoni and D’Ayala, 2009) included cyclic tests of the isolated devices over a range of frequencies relevant to the typical frequency content of European earthquakes. A target displacement of ± 10 mm, comparable to the allowable inter-storey drift required by current guidelines (OPCM 2005) was achieved; the design of the devices was fine-tuned so as to obtain stable and repeatable behaviour. Furthermore, pseudo-static monotonic and cyclic tests of the devices embedded in brickwork masonry specimens were carried out (D’Ayala and Paganoni, 2011b), proving that the addition of the dissipative devices to strength-only anchors limits the damage to the substratum and reduces scattering in the performance of the strengthening, with considerable advantages in respect to a standard system.

As final step of the process of validation of the dissipative devices, shaking table tests are being performed within the framework of the FP7-SERIES project: a prototype of a residential masonry building is being tested in different strengthened set-ups so as to compare the influence of standard and dissipative ties on the overall structural behaviour. Tests are still at the initial stages; therefore, the paper focuses on the Finite Element (FE) modelling that is being carried out throughout the experimental phases. It is herein shown how models were calibrated to match the preliminary output of tests and are used to predict the structural response in different scenarios, to guide future experimental activities and to fulfil the computational validation of the dissipative devices.

2. EXPERIMENTAL FRAMEWORK

Shaking table tests are being performed as part of the FP7-SERIES project at the facilities of the Laboratório Nacional de Engenharia Civil (LNEC) in Lisbon, Portugal. Tests aim to validate the applicability of patented dissipative devices jointly developed by the University of Bath and CINTEC International Ltd and assess their effectiveness for the improvement of the seismic behaviour of a masonry structure in comparison to an unreinforced model and other reinforcement solutions. In order to carry out such comparison, a half scale masonry structure is tested in different configurations (unreinforced, reinforced with standard metallic ties and reinforced with the innovative devices) under an uniaxial input signal, scaled to provoke structural damage to the unreinforced structure and activate the devices. Samples (Fig. 2.1) are representative of multi-storey heritage masonry buildings, which are both of architectural interest and in need of seismic upgrade, not only for their historical value, but also because of the cultural and strategic activities (museums, council offices, etc.) they generally host, which need protection from damage and vibrations.

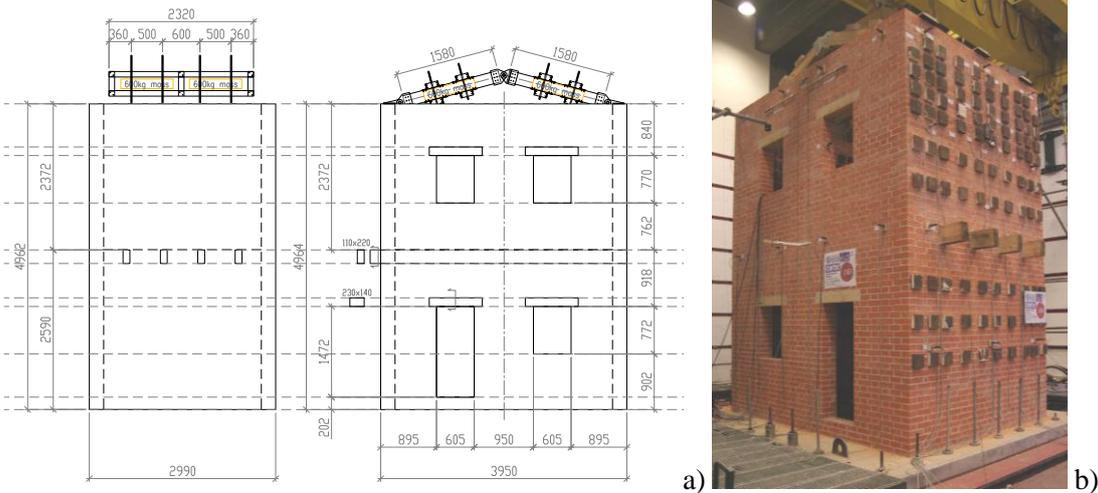


Figure 2.1 Geometry of tested specimen: a) sketch; b) mock-up in final configuration

The mock-up is made of double-bond walls of bricks with holes and lime mortar. Horizontal structures are: simply supported timber beams with a single layer of plywood sheets nailed on top at the first

level, and a timber truss at the roof level; both structures are laid perpendicularly to the solid walls. Lintels above the openings are also made of timber. Results of characterisation tests performed to date are summarised in Table 2.1.

Table 2.1 Summary of material characterization tests

Specimen type	Test type	Average value	Curing time
Bricks	Compressive strength [MPa]	30.1	-
Mortar	Flexural strength [MPa]	0.92	28 dd
		1.18	60 dd
	Compressive strength [MPa]	2.33	28 dd
		2.80	60 dd
Masonry	Compressive strength [MPa]	6.34	At time of prototype testing
	Young's modulus [MPa]	4570	At time of prototype testing
	Poisson's ratio [-]	0.25	At time of prototype testing

Extra masses are placed on the horizontal structures as well as distributed on the solid walls so as to respect a Cauchy similitude (Carvalho, 1998). Because the mock-up doesn't represent a specific prototype building, the distribution and quantity of masses doesn't need to adhere to a precise lay-out, which would be otherwise obtained by a stricter application of the similitude law. On the other hand, it is important that out-of-plane damage is achieved, this being the sought-after failure mode that motivates strengthening by metallic ties. As overturning is commonly surveyed in unreinforced masonry buildings in the aftermaths of a seismic event, it is not unrealistic to the design the specimen so as to obtain such mechanism, as long as the mock-up is representative of a realistic category of structures. Accordingly, the distribution of masses, the design of horizontal structures and the input signal are chosen to facilitate the occurrence of the out-of-plane of the solid walls; at the same time, it is verified that similitudes of both distribution of masses and failure mechanism in respect to a real structure are obtained. This is done by checking that the ratio floor/wall mass ranges between 1.25-3 and the ratio working stress/compressive strength of masonry is within the interval 1.3-10% according to recommendations provided in Tomažević et al. (2009)

The chosen input signal is L'Aquila 2009 mainshock, as recorded at the station of L'Aquila - Valle Aterno – Centro Valle, station code AQV (ITACA); the signal is scaled according to the Cauchy similitude, smoothed according to the Eurocode 8 (Fig. 2.2a) so as to have a smoother and wider spectrum of exciting frequencies (Fig. 2.2b), and is applied in the direction parallel to the walls with openings.

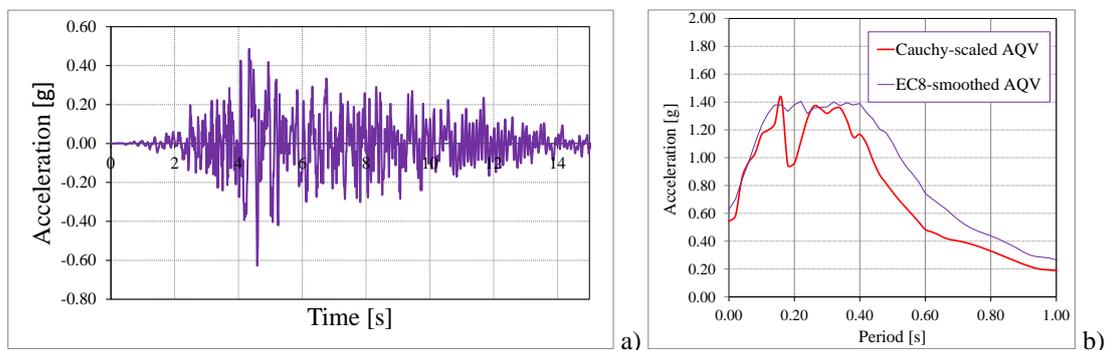


Figure 2.2 Input signal: a) Acceleration time-history; b) elastic spectrum

As tests are at the initial stage, namely only an unreinforced specimen has been tested to date, the extensive discussion of experimental results is postponed to future publications. In the following, the preliminary output of tests is only described to give evidence of the correctness of the FEM.

3. CALIBRATION OF FE MODEL BY TEST PRELIMINARY RESULTS

3.1 Linear analysis

Finite Element Analysis is carried out with the commercial software Algor ©. The structure is modelled with tetrahedral brick elements, except for the roof structure: rafters and purlins are modelled by beam elements, the L-shaped plates that join the roof structure to the wall are shell elements. All the parts are bonded together to simulate the initial fully-connected undamaged system; the joints between the rafters and between the rafters and the wall plates are designed as hinges and modelled as such. A first set of linear analyses is carried out only considering the elastic behaviour of materials. Values chosen for materials on the basis of characterisation tests or of producers' specifications are reported in Table 3.1.

A static analysis with dead load only is carried out to verify that the thrusting action of the roof doesn't provoke any damage to the top spandrel: the maximum strain of 1.26×10^{-4} shown in Fig. 3.1a is compatible with the masonry. Indeed, assuming a value of tensile strength of 1 MPa, this gives an upper-bound limit value of strain equal to 2.2×10^{-4} . Through the static analysis, it is also verified that the average level of vertical stress calculated at the bottom of the structure on the basis of the average material weight and geometry is achieved. The calculated value of 0.16 MPa roughly corresponds to the value of 0.14 MPa shown in Fig. 3.1b. Furthermore, it is also verified that the stresses in the roof elements are within an acceptable range (Fig. 3.1c).

Table 3.1 Summary of mechanical properties used for linear analysis

Part	Young modulus [MPa]	Density [kg/m^3]
Masonry walls	4,500	1736
Lintels	10,000	800
Roof rafters and purlins	8,000	490
Roof plates	20,000	7705

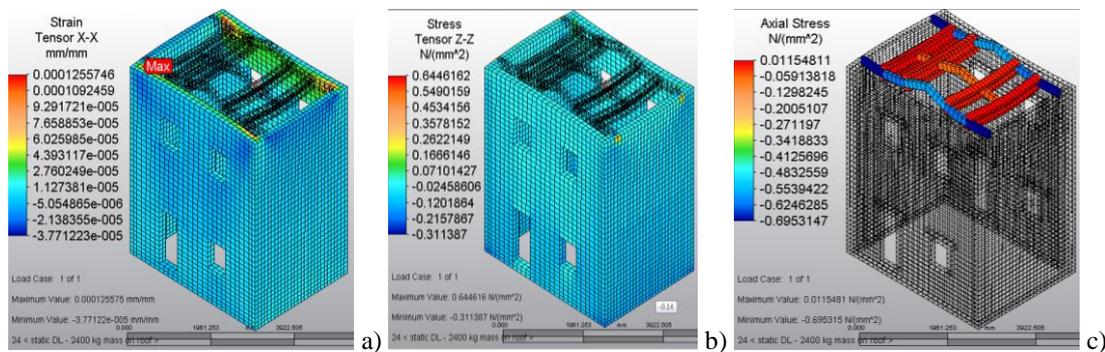


Figure 3.1 Linear static analysis: a) strain in direction parallel to side walls; b) stress vertical component; c) axial stress in beam elements

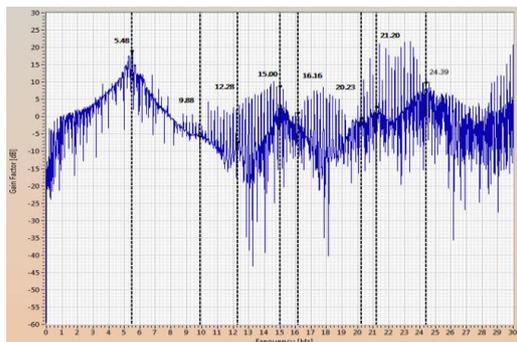


Figure 3.2 Gain factor and natural frequencies of mock-up

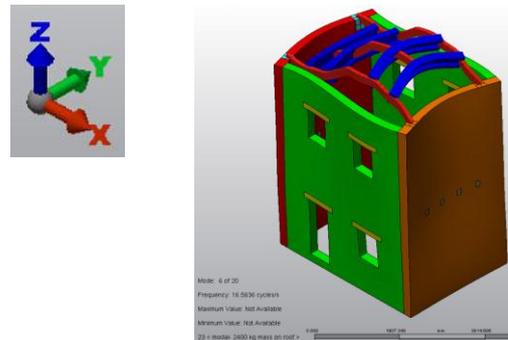


Figure 3.3 Mode shape No 6

A first tuning of the numerical model is carried out by matching the results of the modal analysis with the output of the dynamic identification performed on the mock-up before the beginning of the shaking table tests. Natural frequencies of the mock-up are identified by looking at the peaks of the gain function calculated over the acceleration of the shaking table and of one of the measuring points at the top of the building (Fig. 3.2). Computational results are instead shown in Table 3.2: the main objective of tests is to excite the 6th mode, this having a high participation factor in the X-direction, namely the direction parallel to the side walls. Indeed, the mode shape shows an out-of-plane deformation of the solid walls (Fig. 3.3). Considering that stiffness and mass of the mock-up are known, the main parameter that can be varied to obtain the alignment between numerical and experimental results is the boundary conditions. A sufficiently good match is achieved (average 5.6% difference between first 8 calculated and recorded natural frequency values) when the initial hypothesis of full translational constraint at the bottom of the model is changed to account for a hairline crack that was observed between the foot of the solid wall and the base plate of the mock-up, which is supposed to give translational freedom along the axis of application of the input signal.

Table 3.2 Summary of FE modal analysis

Mode No.	Frequency [Hz]	Modal mass [%]			Cumulative mass [%]		
		X-dir.	Y-dir.	Z-dir.	X-dir.	Y-dir.	Z-dir.
1	5.30	0	8.92	0	0	8.92	0
2	9.11	0	0	0	0	8.92	0
3	11.42	0	0	6.32	0	8.92	6.32
4	13.70	6.22	0	0	6.22	8.92	6.32
5	16.16	0.14	0	1.67	6.35	8.93	7.99
6	16.56	54.15	0	0.01	60.5	8.93	8
7	17.34	0	45.23	0	60.5	54.15	8
8	20.89	0	0	0	60.5	54.16	8
9	23.02	0	2.09	0	60.5	56.24	8
10	23.36	10.94	0	0	71.45	56.24	8



Figure 3.4 Hairline crack between the solid walls and the base plate of the mock-up

3.2 Non-linear analysis

After the initial tuning performed through linear analysis, the output from the shaking table runs is used as means of comparison to calibrate the values of the non-linear parameters chosen for the FEM.

Actuator elements are used to simulate the motion of the shaking table, in terms of the displacement time-history acquired during tests. It is assumed for the sake of limiting the computational burden, that timber and metallic elements remain in the elastic range; the correctness of such hypothesis is verified “a posteriori” by checking that stresses remain indeed within such range. A Drucker-Prager yield criterion is used to define the masonry behaviour in the non-linear field; used values are:

Angle of friction: 0.9 rad
 Cohesion: 1.5 MPa
 Tension cut-off: 1 MPa

Values were initially chosen as “estimated guess” on the basis of data from available characterisation tests and then corrected accordingly to the output of the shaking table test.

A first session of tests with the mock-up undergoing increasing levels of acceleration (10%, 20%, 50% and 75% of the intensity of the input signal) was terminated when diagonal cracks appeared at the corners of the opening of the ground floor (Fig. 3.5). Such failure is not desirable inasmuch it can affect the response of the mock-up, shifting its natural frequencies and preventing the formation of the required out-of-plane mechanism. The FE model correctly predicts such behaviour: looking at the model when it is undergoing gravity loads only, it is possible to see that the strain field in the X direction (axis parallel to side walls) features higher values according to a pattern that is the same as the cracking pattern (Fig. 3.6a). The authors believe that such strain field is due partly to the lack of

full translational restraint at the bottom of the mock-up, and partly to low compression in the direction perpendicular to the bed joints. Whereas the solid walls bear the loads of vertical and horizontal structures as well as the weight of the additional masses, the side walls only bear their own weight; this doesn't create sufficient overburden to prevent the opening of cracks, which follow the weak pattern created by the concentration of tensile strains. Even though the maximum values of X strain are within acceptable values, and therefore do not create cracking, they overall contribute to the damage of the mock-up when this is excited by dynamic forces. This effect is confirmed by the fact that the mock-up shows compressive stresses in the lower boundary of the range indicated by Tomažević et al. (2009) due to the scale reduction, which would not occur in reality. To readdress this situation pressure is applied by means of metallic plates, which "clamp" the side walls through the action of a set of threaded bars that connect the plates to the concrete slab upon which the mock-up rests; the tensioning of the bars controls the level of compression acting on the side walls. The FE model (Fig. 3.6b) proves that the strain field in the X direction is more homogeneous when vertical pressure is applied to side walls. The model is not modified to include diagonal cracks as it is assumed that the applied vertical compression is sufficient to close the hairline gaps and therefore the mock-up is restored to an undamaged situation.

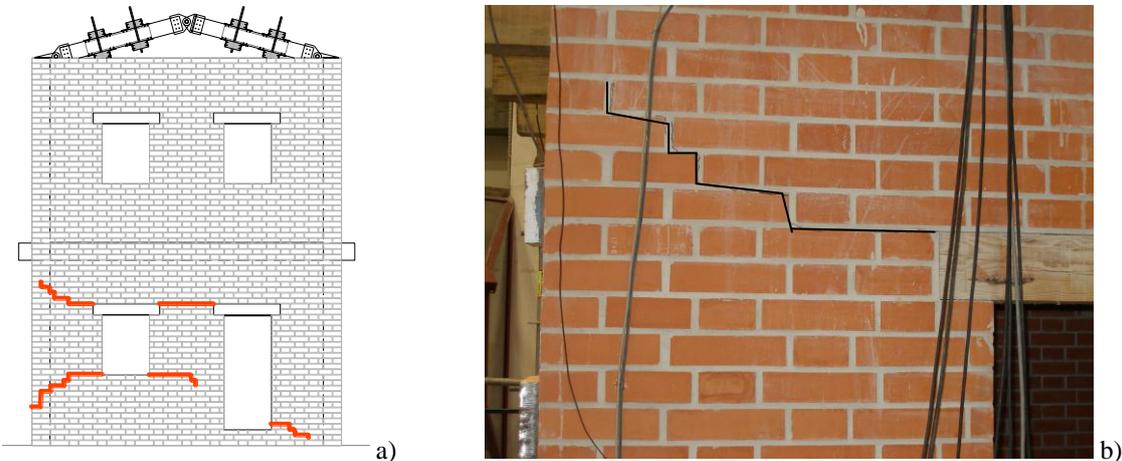


Figure 3.5 Diagonal cracking occurred in first testing session: a) overall crack pattern; b) detail of crack

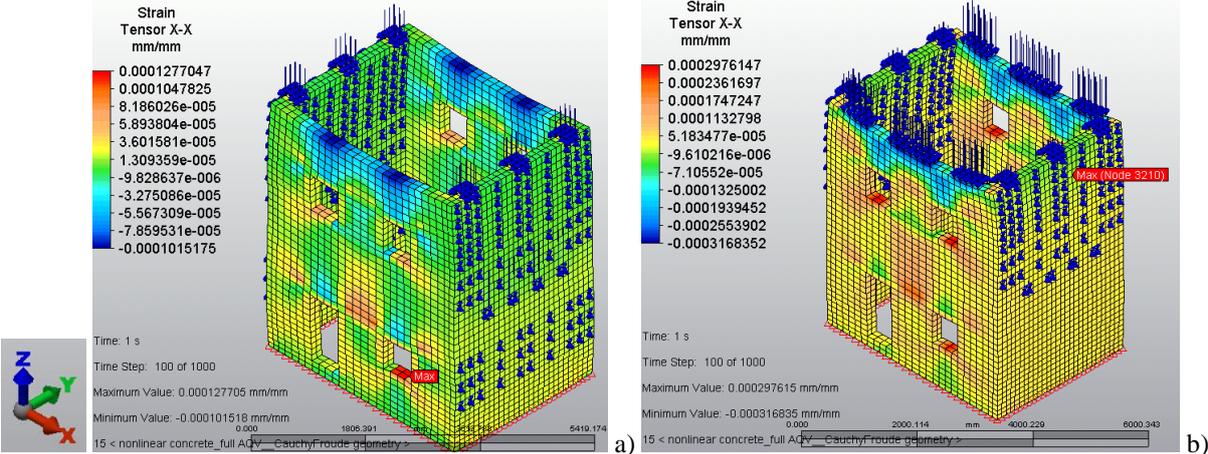


Figure 3.6 FEM showing the difference in X strains in case of a) model with no weight on side walls and b) model with additional pressure on side walls

After setting up the mock-up with the plates for the additional vertical load, the test sequence is repeated. Damage occurs for the 40% intensity signal at the top corners of the mock-up; the computational model well reflects the experimental evidence both from a point of view of distribution and extension of damage (Fig. 3.7). The maximum principal strains in the model ($0.4e-4$) are above

the limit set through the tension cut-off, this being estimated as $\epsilon = \sigma_v/E = 1 \text{ MPa}/4500 \text{ MPa} = 2.2e-4$. Maximum values are in the corner, where cracks opened during tests, and extend down three rows of elements, which are roughly the same dimension of the top 6 courses of bricks of the mock-up. In terms of displacements, a good agreement is observed (Fig. 3.8) up to about 8 seconds: after this moment, the brittle behaviour of the mock-up, i.e. the opening of the crack, produces large movement that cannot be reproduced by the model, as the damaged parts are still fully connected.

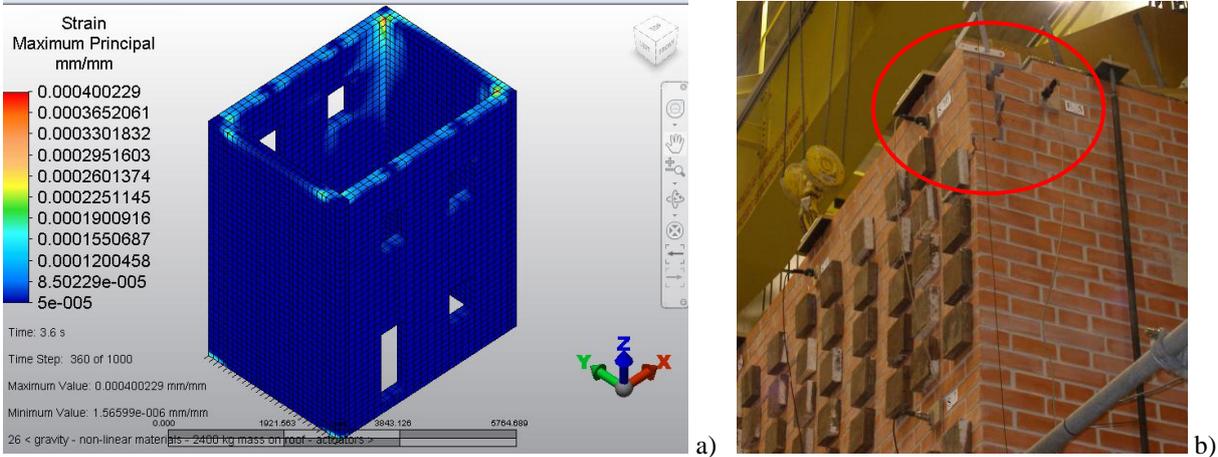


Figure 3.7 Damage at top corners of structure in a) computational model and b) tested mock-up

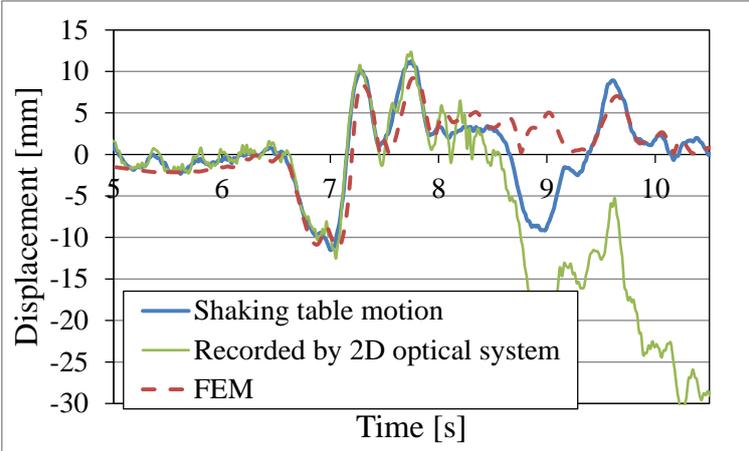


Figure 3.8 Displacement time-histories at the top corner of the structure as calculated and recorded

4. PREDICTION OF FURTHER EXPERIMENTAL SCENARIOS BY FEA

On the basis of the results of the calibration described above, the model is deemed suitable for predicting the response of the structure in other case scenarios, for instance by introducing in the structure the same strengthening elements that will be installed in the damaged mock-up. This will provide guidance for future experimental activities and will contribute to the computational validation of the dissipative anchoring devices. Damage experienced by the structure is accounted for by removing the contact between the top part of solid and side walls, so that the two parts can move freely one from the other. Material properties are the same as for the undamaged model, as cracking is localised and doesn't affect other parts of the structure; thus, it is not necessary to reduce the stiffness of the parent material. The same input signal, scaled for increasing intensities, is used.

A first FEA is performed on a model including standard stainless steel ties. Ties that will be adopted for future tests are grouted within the masonry inside a drilled hole and are normally used to restore

the box-like behaviour of masonry structures. Therefore, the metallic elements run along the whole length of the side walls, reconnecting the sets of perpendicular walls. Truss elements are used to model the ties; as anchors are full grouted, a bonded contact is used at the interface between truss and brick elements. The model shows a punching/pull-out failure at the head of the anchorage (Fig. 4.1a), this being commonly observed in the aftermath of major seismic events (Fig. 4.1b) as well as in previous experimental campaigns (Fig. 4.1c) reported elsewhere. The input of the non-linear analysis is the 100% intensity L'Aquila signal, scaled as explained in § 2; therefore, the insertion of metallic ties does improve the overall behaviour of the structure. Indeed, apart from the area surrounding the truss elements, strains are very low. However, such improvement has its shortfalls as it can lead to localised failures, with negative effects on the substratum, which in historic structures should be preserved as much as possible.

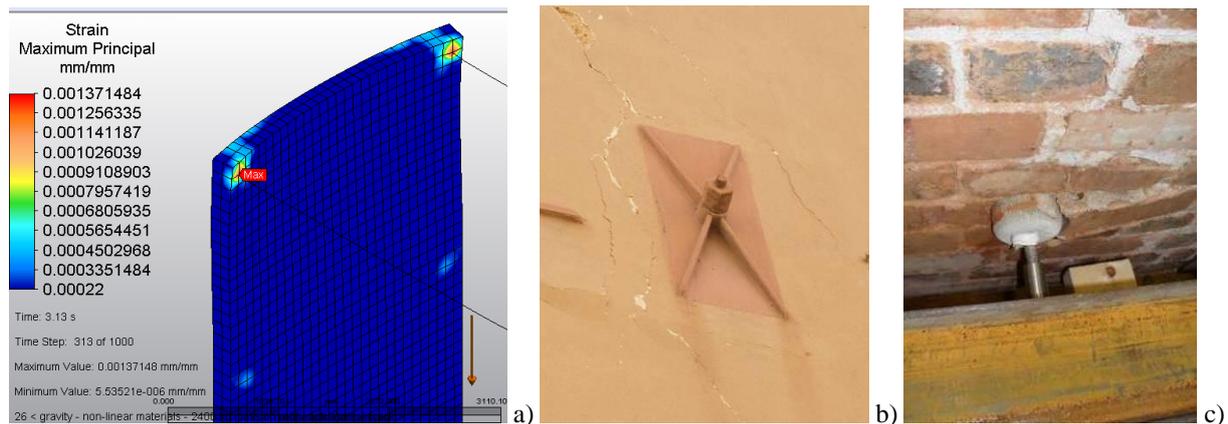


Figure 4.1 Punching failure of standard metallic ties: a) FEM, b) on-site survey (D'Ayala Paganoni, 2011a), c) laboratory tests (Paganoni et al., 2010)

A second model including a hysteretic device inserted at the joint between solid and side walls undergoes the same input as the model with standard steel ties. The hysteretic device is modelled with truss elements with a lower capacity in respect to the rest of the anchor; this is obtained by reducing the cross-section area of the element according to the design of the dissipative element. In reality the hysteretic prototype has also a lower yielding capacity, so as to fully exploit the plastic field of steel. However, truss elements are modelled as linear since it is observed from the models that loads experienced by the anchors are within the elastic range of the anchors.

The model fails for a 150% intensity input. An improvement in respect to the performance of standard ties is visible from the model: thanks to the higher deformability of the hysteretic element in respect to the rest of the anchor, the model is able to withstand a larger intensity input. However, failure eventually occurs. This is in line with other experimental evidence: tests performed on masonry specimens show that the yielding threshold of the hysteretic device, as designed and tested so far, might be too high in a particularly weak substratum (Paganoni et al., 2012). When this occurs, bond failure between the grouted anchor and the masonry occurs before the plastic field of the device is reached (Fig. 4.2b). Conversely, the device performs well in higher capacity masonry (D'Ayala Paganoni, 2011b): damage in the substratum is reduced as deformation is localised in the hysteretic device (Fig. 4.2c). The FE model is able to reproduce such response in relation to the capacity of the hysteretic device and of the parent material. Accordingly, the design of the dissipative element will be modified before further shaking table tests are carried out, so as to achieve yielding for lower level of axial loads acting on the tie and improve the performance of the strengthening system.

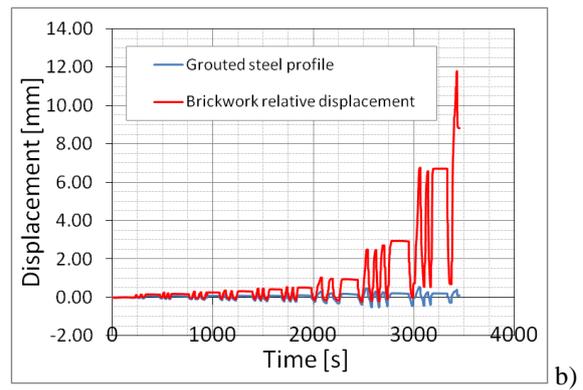
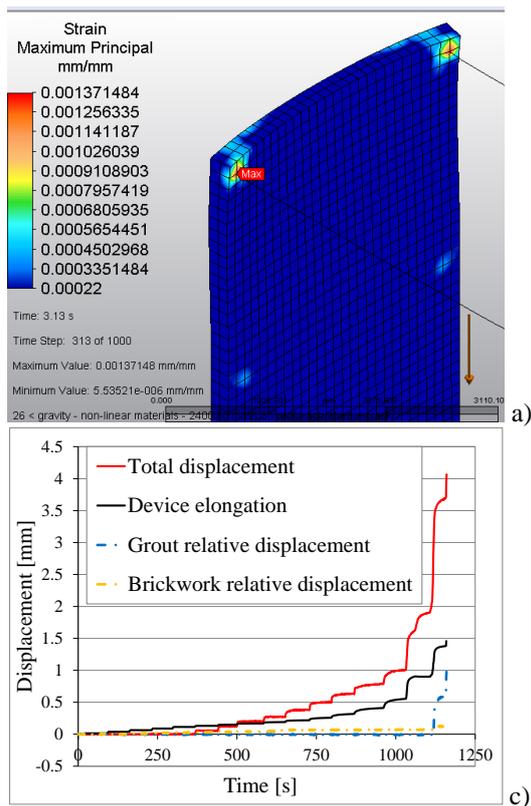


Figure 4.2 Performance of hysteretic dissipative device in: a) FE model of shaking table mock-up, b) weak substratum as tested by Paganoni et al. (2012) and c) higher capacity substratum as tested by D’Ayala and Paganoni (2011b)

5. CONCLUSIONS

Drawing on on-site evidence and on the current shift from a stiffness to a ductility based approach to the design of structures, the authors developed and patented an innovative anchoring device. Such device is conceived to be installed in series with standard metallic anchors at the joint between perpendicular sets of walls of historic structures. The purpose of the strengthening system is to reconnect structural elements that might have become detached as result of seismic events, but also to control relative displacements and reduce accelerations in case of further earthquakes, so that damage to the precious masonry substratum can be reduced.

As final step in the process of experimental validation, the dissipative devices will be installed and tested in a half-scale masonry structure. Tests, which are being performed at the facilities of LNEC in Lisbon, will allow comparing the response of the same structure in different set-ups: unreinforced, reinforced by standard metallic ties and reinforced by dissipative devices.

The paper focuses on the Finite Element Modelling that is being carried out in parallel to shaking table tests. The model is calibrated on the basis of the preliminary experimental output through both linear and non-linear analysis. Significant parameters, such as mechanical properties and boundary conditions, are varied so as to achieve a good match between computational results and experimental evidence. Natural frequencies derived from the modal analysis are satisfactorily matched to the frequencies of the mock-up (6% error); moreover, the model is able to reproduce the damage experienced by the masonry specimen both in terms of distribution and extension of damage, and from the point of view of the overall displacements of the structure.

The model is also used to predict the behaviour of the mock-up in further case scenarios, i.e. taking into account the different types of strengthening that will be tested in the near future. FEA performed on a model with standard steel ties show that this type of strengthening does improve the response of the structure (failure for 100% intensity input against 40% of the unreinforced specimen). However, in agreement with on-site and laboratory evidence, the model fails for punching-pull-out at the head of

the anchorage, thus proving that standard ties, although useful, can be detrimental when one is pursuing the protection of heritage assets.

The insertion a hysteretic anchor device in the model delays the local failure of the anchorage, allowing the model to withstand a larger intensity input. However, due to the low capacity of the parent material, cracking is still likely to occur. Such result is in line with other experimental results collected by the authors by testing the dissipative devices embedded in masonry samples with different mechanical properties: the application of the device, as designed and tested so far, has a better outcome in higher capacity substrata. The model successfully reflects this phenomenon: indeed, the plastic threshold of the device needs to be carefully chosen depending on the capacity of the parent material. Nevertheless, it is of paramount importance that a robust and precise FE model has been developed and so that it can be relied upon to guide the experimental process.

Drawing on results above, further analysis will be carried out in order to fine-tune the performance of the strengthening system; the use of a frictional dissipative device as alternative to the hysteretic ones will be also perused.

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