Advanced Materials Research Vols. 133-134 (2010) pp 831-836

doi:10.4028/www.scientific.net/AMR.133-134.831

Experimental and Computational Validation of Dissipative Prototype for the Seismic Protection of Heritage Buildings

PAGANONI Sara[1, a] and D'AYALA Dina[2, b]

[1, 2]University of Bath, Bath, UK

[a]S.Paganoni@bath.ac.uk, [b]D.F.D'Ayala@bath.ac.uk

Abstract Since earthquakes such as Northbridge (1994) and Kobe (1995) gave the impetuous for the development of performance-based design methods, engineers have been strenuously working to the improvement of the seismic behaviour of structures; in fact, high ductility frames, as well as damping and isolation systems, are nowadays common practice in seismic prone areas. Heritage buildings constitute an odd case: many historic centres are still considerably affected by seismic events (L'Aquila, 2009) due to the lack of a methodical retrofit and this, where applied, is still largely based on the increase of stiffness and capacity, without the due care for precious finishings. In order to address the lack of specific passive systems for heritage buildings, the authors have developed two typologies of dissipative devices that can be integrated in traditional steel anchors and installed within the masonry at the joints of perpendicular walls, where out-of-plane mechanisms are likely to form due to poor quality connections. Both prototypes, one based on the plasticity of steel, the other relying on friction, were tested as isolated elements in pseudo-static regime for proofing and fine tuning, and in a dynamic range typical of the seismic frequency content to validate the stability of dissipative loops. The paper focuses on pull-out tests aimed to analyse the behaviour of the hysteretic prototype in respect to traditional steel anchors in masonry panels with low shear capacity. Finite Element (FE) models were also developed and calibrated applying the data from tests. Experimental and computational results are discussed in the following; the need for further theoretical work concludes the paper.

Keywords: Seismic passive protection, heritage building, masonry structure, dissipative, steel anchor

Introduction

Although earthquakes such as Kobe, Japan, 1995, highlighted the unreliability of highly stiff structures and shifted engineers' attention towards performance design methods (Priestley 2000), the traditional stiffness-based approach is still accepted and widely used in the field of retrofit of heritage buildings (EN 1998 Eurocode 8; Italian Ministry of Cultural Heritage and Activities 2006). The application of techniques involving ductility and energy dissipation, excluding few high-profile case studies (e.g. Indirli and Castellano 2008), are limited, as innovative systems often do not meet some of the requirements – reversibility, low impact – required for interventions on historic structures.
Drawing on this last observation, 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-ons for metallic ties, these being commonly applied in rehabilitation practice all over Europe (Tomaževič 1999) to prevent out-of-plane collapse of masonry panels, as recently confirmed by a survey in L'Aquila, Italy, after the April 2009 earthquake (D'Ayala and Paganoni 2010). Traditional cross ties consist of steel profiles, which are inserted longitudinally within the main walls to improve the box-like behaviour of buildings providing a connection at the joints of perpendicular sets of walls, where the poor quality and wear and tear of historic masonry facilitate crack onset and eventually failure of the connection. A shortcoming of this retrofitting technique is the localised increase in stiffness and the punching failure that might occur during an earthquake, often causing substantial non-structural damage to precious finishes, despite the successful prevention of collapse.

The dissipative device developed within this research allows small relative displacements reducing the transmitted energy, and hence damage, thanks to the hysteretic properties of a stainless steel element, shaped to optimise its post-elastic behaviour.

Initial experimental work, reported elsewhere (Paganoni and D'Ayala 2009) included tests in pseudo-static and dynamic regime of the dissipating device on its own. A target displacement of ± 10 mm, comparable to the allowable inter-storey drift required by current guidelines, was achieved.

Hereafter results of pull-outs tests, aiming to characterise the behaviour of anchors in low shear capacity walls and further validate the hysteretic device, are reported and compared with strength-only anchors. Experimental data are applied to calibrate a Finite Element (FE) model and investigate further scenarios.

Pull-out Tests

The purpose of the pull-out test is to analyse the behaviour of the dissipative anchors in masonry panels with low shear capacity and to compare the results with the performance of standard grouted anchors. This represents the lower limits of possible application and hence an upper limit for the strength capacity of the device, if it is to fail before the masonry.

The test set-up reproduces the portion of the tie anchored into the wall subjected to overturning by way of a grouted socket, while the dissipative devices attached to it would be positioned at the interface with a wall parallel to the main shock direction and free to move relatively to the wall. The pulling action of the testing apparatus represents the reaction of the longitudinal anchor to the thrust of the overturning wall due to seismic acceleration (Figure 1).

Following the prescriptions of BS EN 846-2, five anchors were positioned in each of two panels at a sufficient distance to avoid interaction effects between adjacent anchors or between anchor and wall edges; five were tested as strength only anchors, while hysteretic devices were connected in series to the others. A vertical load was applied throughout the tests, providing a stress between 0.05 and 0.1 MPa to simulate the standard compression perpendicular to bed joints. Characterisation tests were carried out according to relevant Eurocodes (BS EN 772 - masonry units, 1015 –mortar, 1052– masonry); samples were tested after a curing period of 60 days, this including the curing of masonry, the installation of anchors – drilling and grouting – and curing of grout.

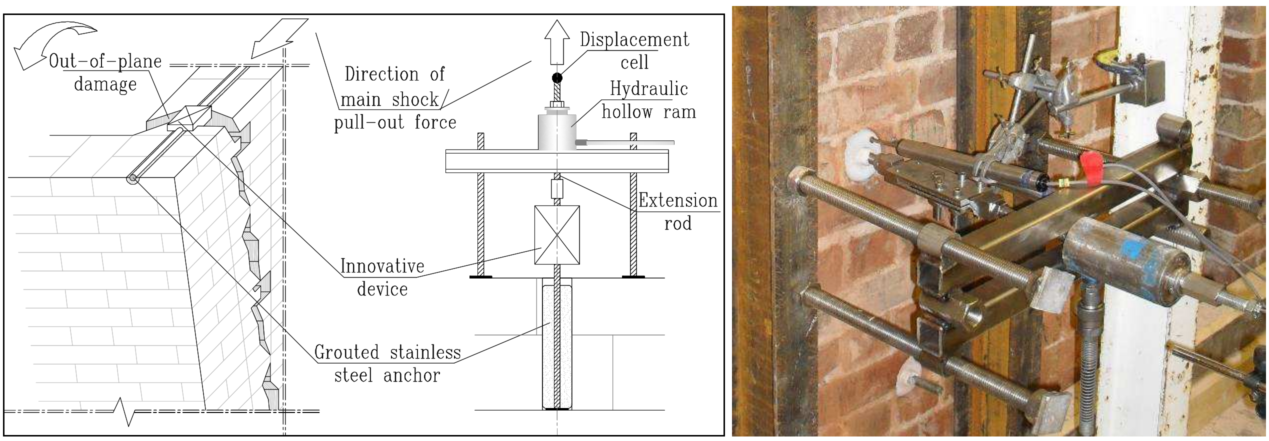


Figure 1: Real case application of cross tie and experimental set-up of pull-outs

Materials and Specimens Two masonry panels with a base of 1.4 x 0.35 m and 1.5 m high were built in English bond, using recycled Victorian bricks and natural hydraulic lime mortar. Masonry units were fired bricks, sized 220 x 110 x 70 mm, with 20 vertical holes; average compressive strength was 27.3 MPa (CoV = 0.19). The mortar was a NHL 5, as characterised in Table 1.

Table 1: Properties of mortar

Mix proportions lime to sand	1:2.13	b.v.	Flow value	166	[-]
	1:4	b.m.	Flexural strength	0.3 (CoV 0.29)	[MPa]
Water to lime ratio	1.82	[-]	Compressive strength	1.0 (CoV 0.13)	[MPa]

Masonry panels had a compressive strength of 6.7 MPa (CoV 0.01), while bond strength measured by wrench test resulted equal to 0.67 MPa (CoV 0.15), which correlates to a value of characteristic initial shear strength equal to 0.42 MPa; given the applied vertical load, the shear strength at the anchor level is 0.45 MPa (Zhou et al. 2008).

Anchors were made of a threaded M16 bar, AISI 304 stainless steel (UNI 14,301) class 70 (yield proof stress 300 MPa, ultimate tensile strength 700 MPa), 400 mm long, with a 60 mm end plate and a 350 mm long fabric sleeve for grouting, and were post-installed in 80 mm diameter drilling holes passing through the wall.

Results For such testing set-up, failure may occur: 1) at the bond between grout and adjoining bricks, or between bricks themselves, 2) in the masonry units and 3) in the steel profile. The tests aimed to obtain the latter typology of failure, this being ductile and therefore preferable to the other two, which are fragile. Loading was applied until appearance of cracks and displayed damage indeed reflected the three categories listed above. Given that the total displacement was measured at the end of the assembly, which is combined in series of pulling apparatus, free anchor, and anchor embedded in the grout and connected to the masonry, the resulting load-displacement curves are affected by the stiffness of each of these parts.

Standard anchors, as expected, showed an elevated stiffness, with maximum loads (61 kN, CoV 0.22) corresponding to ultimate displacements (2÷5 mm – Figure 2a) lower than the set target of 10 mm, which is used inasmuch comparable with allowable inter-storey drift.

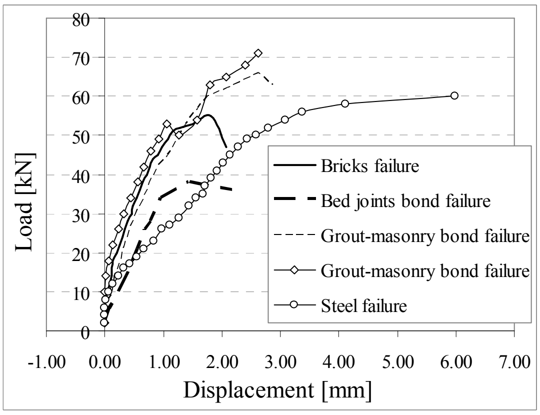


Figure 2: (a) Load-deflection curves of standard anchors and (b) example of pull-out failure

Ultimate loads for bond failure at the interface between grout and masonry are higher than what calculated (45 kN) by characteristic shear strength on the cylindrical surface of the grouted sleeve. However, the enhancement of the mechanical bond due to the presence of holes in the bricks is not taken into account in the calculation, while the overall capacity of the anchor is considerably increased in respect to a smooth drilled hole. A lower ultimate load was instead recorded when masonry units were affected by cracking; the holes in the bricks, despite enhancing the mechanical bond, affect the material thickness, creating whytes vulnerable to cracking.

In one case the anchorage featured a higher deformation, probably due to a weaker steel section that reached the field of non linear deformations at the same time as micro-cracking began, providing ductility to the assembly.

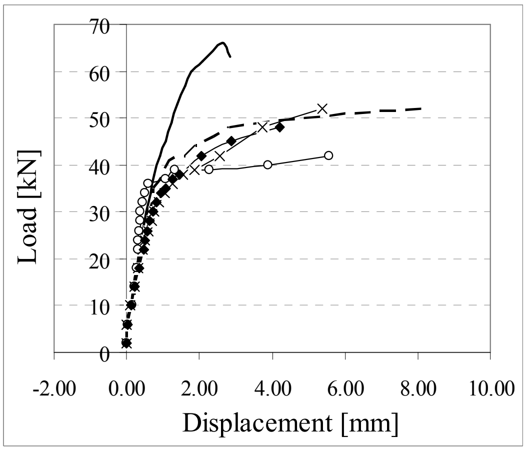


Figure 3: Comparison between traditional (continuous line) and hysteretic anchors

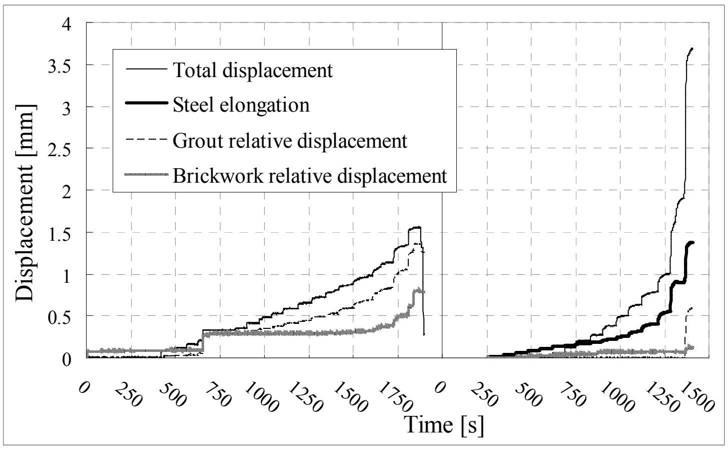


Figure 4: Relative displacements within the assembly for standard (left) and hysteretic (right) anchor

In the second set of tests, the applied hysteretic devices allowed larger displacements without damage occurring to either the grouted element or the parent material, as they were designed to reach the yielding point, namely 30 kN, before the load increased up to the pull-out limit value (Figure 3).

During tests, relative displacements between the outer surface of the grouted sleeve and brickwork were measure by transducers and the elongation of the hysteretic element was calculated via strain-gauges positioned in the central section of the element and at the connections. In Figure 4 comparison between a standard and a hysteretic anchor is showed; for the first, values are plotted until failure, while in the second case strain gauges reached the limit before completion of the tests, which was continued up to a total displacement of 5.7 mm. The graph clearly shows that, while a standard anchor fails at 1.5 mm, the hysteretic element is able to provide ductility so that the grouted part of the anchor is not affected by the pull-out. Even if part of the recorded total displacement is due to deformations and tolerances of the testing apparatus due to the position of the dial gauge, yet the hysteretic device displayed a permanent elongation of 4 mm, without any detectable damage to the anchorage.

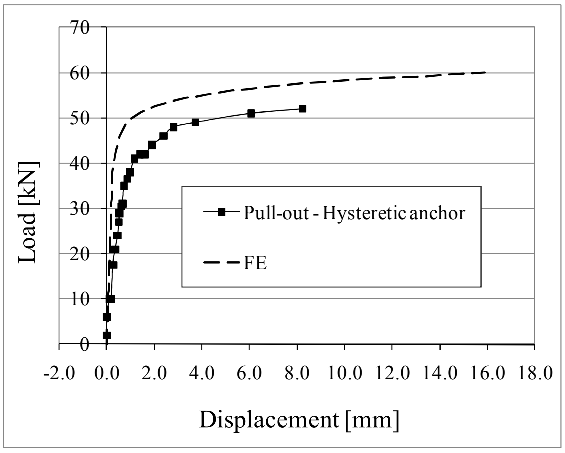
Finite Elements Analysis

A FE model was developed using Algor commercial package to investigate the behaviour of the dissipative element of the hysteretic tie. The model reproduces the hysteretic element without accounting for either the grouted anchor or the brickwork, as it is proved by the tests that the post-elastic behaviour of the lower capacity steel is predominant within the assembly; thus one end of the device is fully constrained and the other is subjected to increasing positive load. The behaviour of steel is modelled using a von Mises stress-strain curve derived by the application of the Ramberg-Osgood model, as modified by Rasmussen and reported by the Eurocode 3. Standard material parameters for the applied type of steel were used (Table 2); only the yielding strength was increased in respect to the standard value, which is indeed a guaranteed minimum, to match experimental results.

Table 2: Mechanical properties of steel used for the FEs model

Yielding strength	300	[MPa]	Young's modulus	193	[GPa]
Ultimate tensile strength	500	[MPa]	Poisson coefficient	0.28	[-]

The model showed good agreement with the data collected from pull-outs: the post-elastic branch of the load-deflection curves deriving from tests and FEs are consistent, even if the experimental graph features lower stiffness due to the presence of the grouted element and brickwork, which were not considered in the model. However, the simplified model reproduces satisfactorily the overall behaviour and was therefore used to perform cyclic analysis, which aimed to correlate the pull-out tests with previous sessions carried out on the isolated anchor devices (Figure 5).



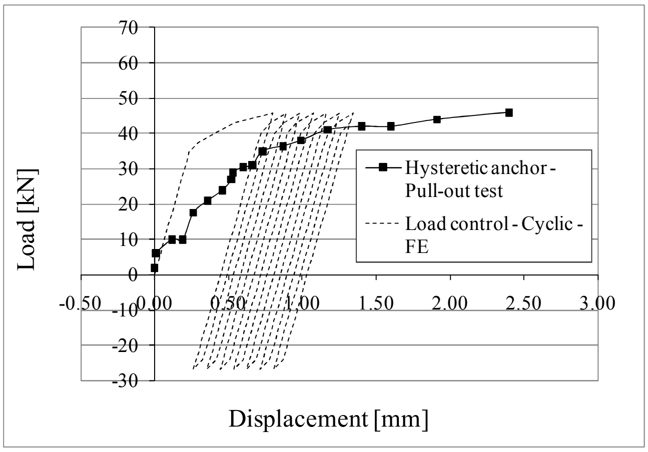


Figure 5: Load deflection curve - Comparison between pull-out tests and FEs

Figure 6: Load-deflection curves - Comparison between pull-out tests and cyclic FEs

Loading was applied on a FE model with same characteristic as before, imposing a sinusoidal curve with maximum equal to 50.5 kN and minimum -30.5 kN to reproduce the testing condition and avoid buckling; kinematic hardening was imposed. The obtained load-deflection curve is compared again with pull-out results and confirms observations drawn on the first computational analysis (Figure 6). Moreover the comparison with stress-strain curves of pseudo-static and dynamic tests on the single device shows good agreement and proves that plastic deformation is localised in the expected section, while connections with the rest of the assembly remain in the elastic field, being the hierarchy of capacities important to control the damage mechanism of the hysteretic anchor (Figure 7).

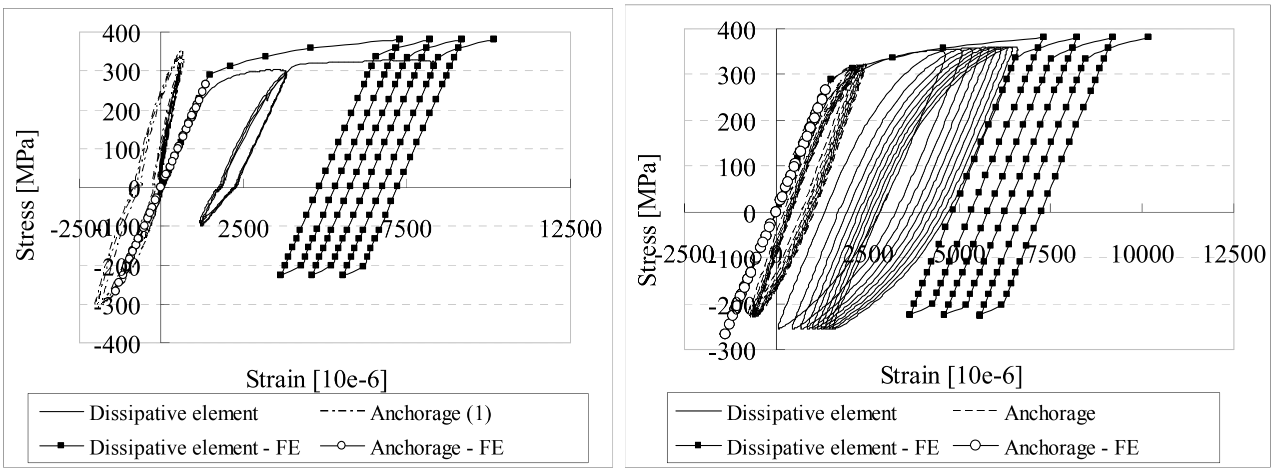


Figure 7: Stress-strain curves - FE models compared with static (left), and dynamic (right) tests

Conclusions

The paper analyses the results of a series of pull-outs tests performed to investigate the behaviour of stainless steel anchors in walls with a low shear capacity. Two typologies of anchors have been considered: the first is a traditional grouted anchor, the other is an anchor provided with a steel element, shaped to reach the non-linear field at a lower level than the rest of the assembly; such anchor device aims to tackle the problem of the localised increase of stiffness and damage that affects cross ties anchors commonly used in Europe for the seismic protection of historic buildings.

Experimental results confirmed that the yielding element is able to provide the anchorage with ductility and prevent damage to occur in either the grouted sleeve or the masonry, while traditional anchors display a high stiffness and fail at the interface between grout and parent material or in the masonry units, with reduced displacements. A simple finite element model was developed and calibrated with experimental data through a monotonic non-linear analysis; further cyclic analysis were performed and showed consistency with the pull-outs as well as with previous dynamic tests.

In conclusion, the hysteretic anchor device was able to reach a displacement comparable with prescribed inter-storey drifts and prevent pull-out failure; it is therefore consistent from a point of view of performance design, although further tests will be needed to validate the dynamic behaviour of the assembly in masonry panels. The behaviour of the device is predictable by FE modelling, this allowing for future refinements on the model and the simulation of other applications of the device.

Hence, cyclic full-scale pull-out tests on large-scale masonry specimens will be performed in the near future while a refined computational model will be developed to obtain full agreement with tests and simulate additional scenarios.

Acknowledgements

The authors acknowledge Cintec International Ltd, who supplied and installed the stainless steel anchors, supported the experimental sessions providing equipment and staff, and collaborates to the research project within the framework of a Knowledge Transfer Partnership (KTP).

References

[1] *BS EN 1015:1999 – Methods of Test for Mortar for Masonry*, 1999.

[2] *BS EN 1052:1999 – Methods of Test for Masonry*, 1999.

[3] *BS EN 1996-1-1:2005 Eurocode 6 – Design of Masonry Structures*, 2000.

[4] *BS EN 772:2000 – Methods of Test for Masonry Units*, 2000.

[5] *BS EN 846-2:2000 - Methods of Test for Ancillary Components for Masonry*, Part 2: Determination of Bond Strength of Prefabricated Bed Joint Reinforcement in Mortar Joints, 2000.

[6] D'Ayala, D. and Paganoni, S (2010). "Assessment and Analysis of Damage in L'Aquila after 6th April 2009." Submitted for publication in special issue of the *Bulletin of Earthquake Engineering*.

[7] *EN 1993-1-4:2006 Eurocode 3 – Design of steel structures*, Annex C, 1996.

[8] *EN 1998 Eurocode 8 – Design of Structures for Earthquake Resistance*, 1998.

[9] Indirli, M, and Castellano, M G (2008). "Shape Memory Alloy Device for the Structural Improvement of Masonry Heritage Structures." *International Journal of Architectural Heritage*, 2: 93-119.

[10] Italian Ministry of Cultural Heritage and Activities (2006). *"Guidelines for Evaluation and Mitigation of Seismic Risk to Cultural Heritage"*.

[11] Paganoni, S, and D'Ayala, D (2009). "Development and Testing of Dissipative Anchor Devices for the Seismic Protection of Heritage Buildings," in *Proc. of ANCER Workshop 2009*

[12] Priestley, M J N (2000). "Performance Based Seismic Design," in *Proc. of 12th World Conference of Earthquake Engineering,* paper No. 2831.

[13] Tomaževič, M (1999). *"Earthquake-resistant Design of Masonry Buildings."* A. S. Elnashai & P. J. Dowling Ed., London, UK: Imperial College Press.

[14] Zhou, Z , Walker, P , and D'Ayala, D (2008). "Strength Characteristics of Hydraulic Lime Mortared Brickwork," in *Proc. of the Institution of Civil Engineering – Construction Materials*, 161(4)，139-146.