OUT-OF-PLANE EXPERIMENTAL TESTS ON MASONRY PANELS

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ABSTRACT

Out-of-plane mechanisms induced by earthquakes are typical of many masonry structures such as standing out elements (parapets, battlement of fortresses, soaring portion of church façades) or portions of masonry facades badly connected to the building. The displacement-based assessment procedures proposed in codes are mainly based on the hypothesis of rigid block, which sometimes does not properly represent the actual behaviour of these structures.

In this context, in order to verify the reliability of the rigid block hypothesis and to analyse the equivalent damping of stone masonry panels under rocking, an experimental campaign on three mock-ups subjected to out-of-plane actions has been performed in the Laboratory of Structural Engineering of the University of Genoa (Italy). Panels are made of a two-leaves irregular stone masonry, characterized by a quite poor quality, like that one usually present in the Italian historical centres.

Two types of test have been performed: a) static out-of-plane test under controlled displacements conditions, in order to evaluate the pushover curve; b) free vibration test, in order to measure rocking in the nonlinear range and evaluate the equivalent viscous damping, due to impacts and other sources of dissipation. The above-mentioned tests have been repeated for different levels of displacement.

In general, experimental results have shown that rigid block model is able to properly describe the seismic out-of-plane response, by introducing proper modelling strategies in order to take into account the real behaviour of this kind of structures.

INTRODUCTION

The observation of the seismic damage after recent and past earthquakes highlights the vulnerability of masonry structures, particularly referring to the activation of out-of-plane mechanisms.

In fact, both in monumental and ordinary buildings, the subsequent transformations in the course of time have often determined the presence of masonry walls non-effectively connected with the previous elements of the building, which make them particularly susceptible to the activation of local mechanisms, involving their out-of-plane response. Furthermore, standing out elements (such as parapets in ordinary buildings, battlements in fortresses, soaring portion of church façades) are usually present in all these structures; they are different for geometrical shapes and dimensions, but all interested by high slenderness and subjected to a rocking motion induced by earthquakes.

Despite the dynamic nature of this problem, displacement-based assessment procedures are generally proposed in the codes (NTC, 2008; MIT, 2009), mainly based on the hypothesis of rigid block; however, this assumption sometimes is not able to properly represent the actual behaviour of

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structures, especially if characterized by a quite poor masonry quality, which traditionally can be found in ordinary historical buildings.

In this context, the paper focuses on the results of an experimental campaign on three masonry panels subjected to out-of-plane actions, performed in the Laboratory of Structural Engineering of the University of Genoa (Italy). In particular, two types of tests have been performed on each panel: a) static out-of-plane test under controlled displacements conditions, by applying a horizontal action; b) free vibration test, by releasing the panel after the static pulling. The above-mentioned tests have been repeated for different levels of displacement.

The experimental campaign has been different aims: first of all, it intends to contribute to the comprehension of dynamic rocking response of slender structures, built with irregular multi-leaves stone masonry, since most of the contributions provided in literature refer to monolithic stone blocks (Liberatore and Spera, 2001; Pena et al., 2007) or single-leaf masonry walls (Lam et al., 1995); then, to verify the reliability of the rigid block hypothesis and to analyse the damping characteristic of a quite poor quality masonry, present in a significant percentage of Italian historical centres.

SPECIMENS CONSTRUCTION AND MATERIALS CHARACTERIZATION

In order to investigate the seismic out-of-plane response of different types of standing structures, three specimens were built with different dimensions (height and thickness) and slenderness. In particular, each couple of panels has one parameter in common: different size but the same slenderness, the same height or the same thickness. Table 1 summarizes the size dimensions and slenderness of each panel. The specimens can be considered representative of a 1:1.5 scale.

Table 1. Mock-up features

Panels	Dimensions (width, height, thickness) [cm]	Slenderness
1	90x110x22	5
2	90x90x30	3
3	90x150x30	5

All panels were built with irregular stones and air lime mortar, trying to reproduce as far as possible the masonry of traditional stone buildings usually present in the Italian historical centres.

The cross-section was formed by two external leaves made of medium size stones (standard dimensions of blocks are about 18x12x12 cm, but it has to be considered the specimen are representative of 1:1.5 scale) and a central leaf made of smaller stones and mortar infill (Fig. 1a). The two external leaves were not connected systematically through headers; however, some randomly set bigger stones ensured a quite good cross-section connection, as it was possible to deduce by the results of the performed sonic tests (Fig. 1c). The face of the masonry panel was characterized by the presence of rough-hewn stones, quite small and not very stretched, while mortar joints were quite thick (Fig. 1b).

Blocks are sedimentary, carbonaceous and monomineralogic stone, while mortar is a non magnesiac air lime mortar, made with a normally seasoned non hydraulic putty, classified as CL 90-S according to UNI EN-459-1 (2002) and aggregate obtained by crushing and grinding the same limestone of blocks. The aggregate is characterized by a grading which was variable from very fine to rough. The binder/aggregate ratio varied from 1:3 to 1:2.5. The stone/mortar volume ratio was about 1:1.5.

Flexural and compressive strengths of mortar were measured by testing prismatic specimens after different maturation periods. In particular, 24 prismatic 160x40x40 mm^3 specimens were realized, following the indications of UNI EN 1015-11 (2007). The obtained results show that in general strength values are on the lower bound according to literature (Bokan-Bosiljkov and Valek, 2005; Lourenço et al., 2014). In particular, the mean compressive strength measured at the time of the full-scale tests (300 days old) was 1.44 MPa (with 0.09 MPa of standard deviation), while the flexural strength was 0.40 MPa (with 0.033 MPa of standard deviation). A constant increase of these values was observed during the seasoning time. Preliminary tests show that carbonisation of mortar is, still

today, very low (approximately 37%). This may be due to the use of putty as well as to the conditions of building of the walls (heated environment). By ultrasonic tests, it was also possible to evaluate that the Young Modulus was approximately 1189 MPa (if measured on specimens taken from the wall) or 2177 MPa (if measured on the prismatic specimens).
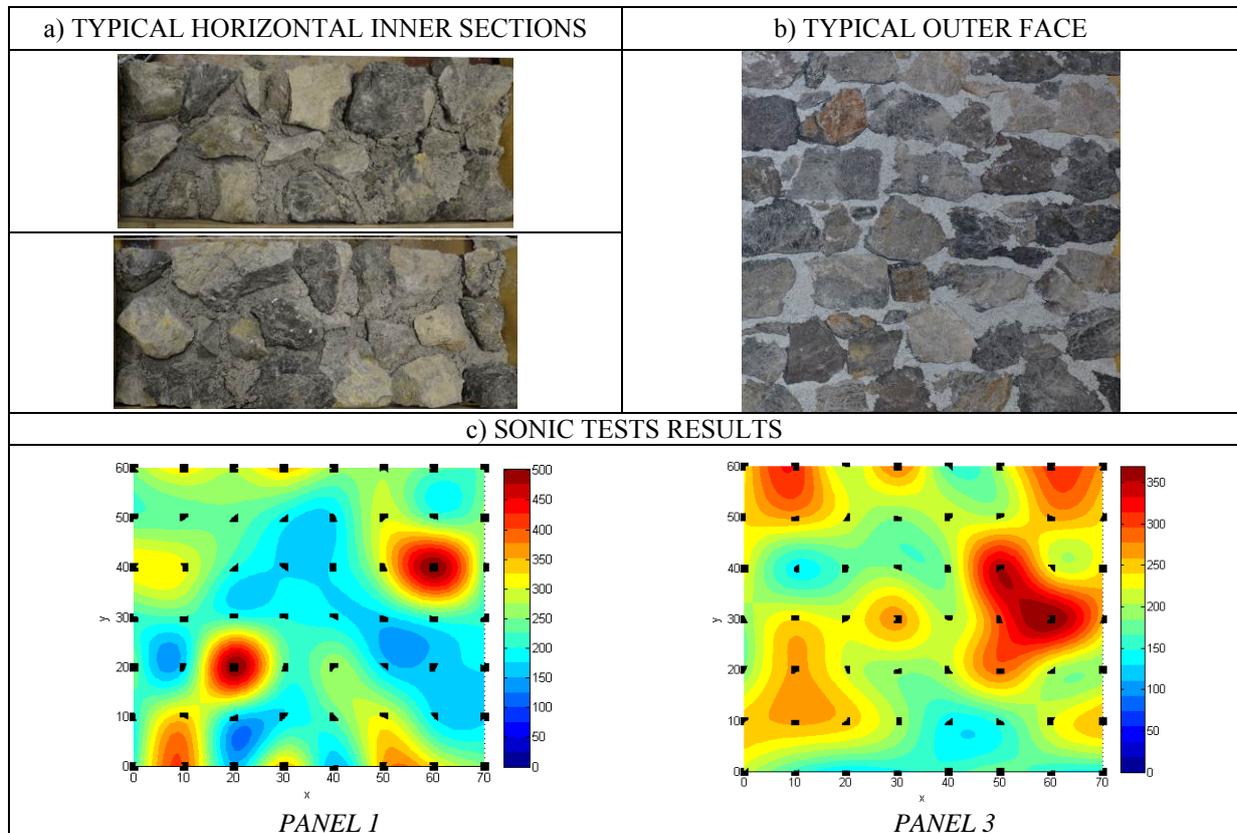
a) TYPICAL HORIZONTAL INNER SECTIONS	b) TYPICAL OUTER FACE
c) SONIC TESTS RESULTS	
PANEL 1	*PANEL 3*

Figure 1. Typical horizontal inner sections (a) and outer face (b) of tested panels; sonic tests results (c)

Table 2. Mechanical parameters obtained by tests for mortar

Mortar	Compressive Strength – 300 days old		1.44 MPa
	Flexural Strength – 300 days old		0.40 MPa
	Current Mortar Carbonisation		37%
	Young Modulus	Specimens from the wall	1189 MPa
		Prismatic specimens	2177 MPa

TEST SET-UP

Out-of-plane tests on three full-scale specimens have been performed in the Laboratory of Structural Engineering of the University of Genoa (Italy).

As above introduced, two kinds of test have been performed: static out-of-plane test and free vibration one. For both of them, a special testing set-up has been adopted and following illustrated.

Each panel was built on a base, made by a first layer of concrete followed by a layer of masonry; this base has a slightly higher thickness in comparison to the panel above, in order to be the weaker section and localize the position of the overturning hinge, by guaranteeing at the same time that the interface in which rocking takes place is masonry-masonry. In order to avoid the rocking of the whole specimen during the test, the base of the panel was welded to the floor by using steel L-shaped profiles all around it (Fig. 2).
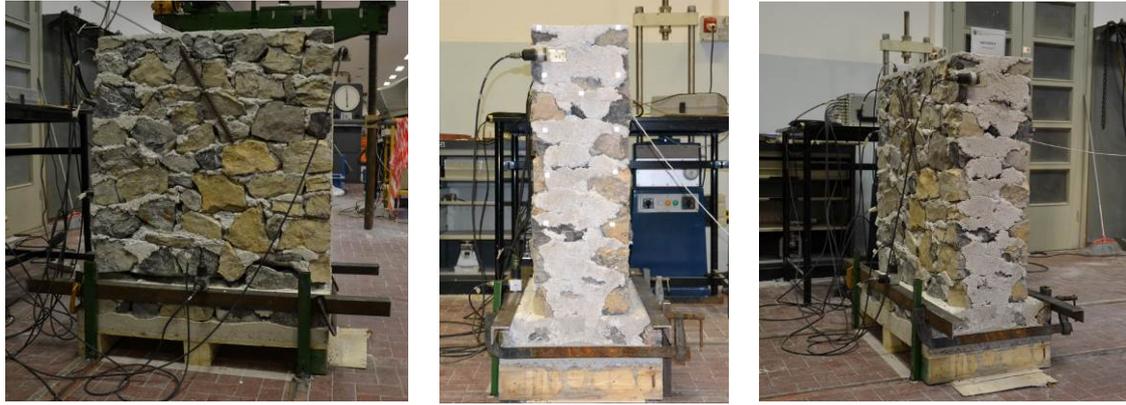
Figure 2. Details of the masonry base, realized to localise the plastic hinge position

Before the test starting, each panel has been placed in front of a restraint frame and linked to it through a cable, fixed at about 2/3 of the panel height by means of a steel bar (Fig. 3). Then, the panel has been first pulled by means of the above-mentioned cable until the formation of the plastic hinge (test a) and then released (test b), in order to analyse rocking in the nonlinear range. This procedure has been repeated for different levels of imposed displacement and by considering (for panels 2 and 3) the effects connected to the presence of an added load (about equal to 120 kg), placed on the top of each panel after a certain number of imposed displacements.
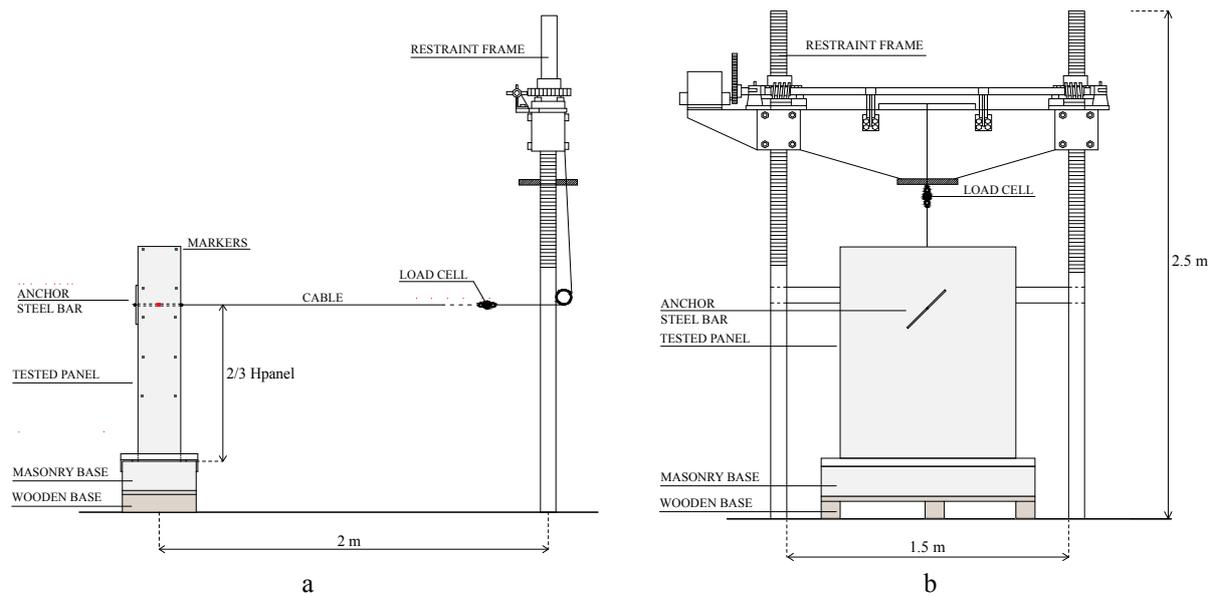

Figure 3. Test set up: lateral (a) and frontal (b) view

Table 3 summarizes, for each panel, the number of performed tests (*TS*) and the values of maximum imposed displacements in each one (*D_i*). Displacement values are at the point in which the cable was fixed, placed at 2/3 of the panel height in the centre of the transversal cross section (marked in red in Fig. 3a).

Table 3. Number and values of the imposed displacements for each panel (in mm)

Panel	TS	D_1	D_2	D_3	D_4	D_5	D_6	D_7	D_8	D_9	D_{10}	D_{11}	D_{12}
1	9	15	30	55	74	81	96	124	146	150	-	-	-
2	10	38	59	91	31^	67^	98^	112^	115	151	205	-	-
3	12	8	9	20	28	12^	24^	29^	30^	32	41	54	54
^ Test performed by considering 120 kg added on the top of the panel													

Regarding the acquisition system, both standard instruments and more innovative ones have been used. In particular, a standard load cell linked to the cable (Fig. 4a) has been used to measure the applied horizontal force during the static out-of-plane test (test a), while standard accelerometers (Fig. 4b) located at different levels of the panel have been used in the free vibration test (test b).

Figure 4. Details of the test set-up: a) link between cable and load cell; b) detail of an accelerometer

Furthermore, in both cases, the displacements on the two leaves of the panel were measured by 2-D optical movement detection technique, based on infrared optical acquisition system (Lunghi et al. 2012). The acquisition was made at 20 Hz with a precision of 0.05 mm. A grid made by 11 or 15 reflecting markers was adopted respectively for the panels 1 and 2 and for the panel 3 (the tallest one); in particular, one marker has been always placed at about 2/3 of the panel on the panel lateral surface, while the other ones were located at different levels on each leaf. This latter choice has been determined by the will to monitor the relative displacements between the two leaves.

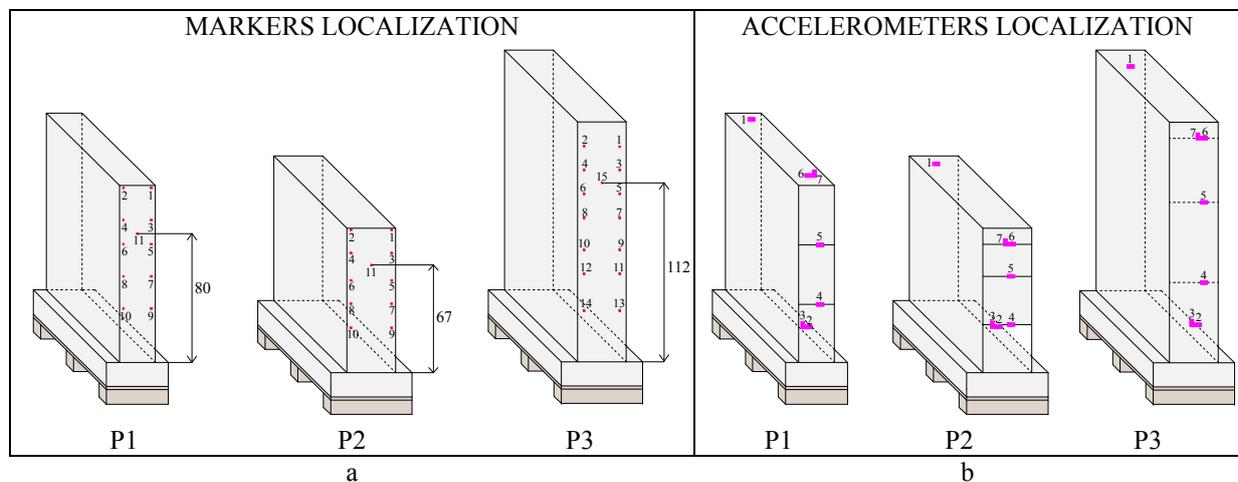Fig. 5 illustrates the localization of markers and accelerometers in each panel.

Figure 5. Localization of the accelerometers and of the reflecting markers for panels 1, 2 and 3

RESULTS OF THE STATIC OUT-OF-PLANE TEST

Generally, models based on rigid-block assemblies provide a suitable framework for understanding the dynamical behaviour under seismic actions of free-standing existing masonry structures. In particular, these types of models can be alternatively applied both in the dynamic and in the static field. Although the actual problem is dynamic, the static approach (usually proposed in the codes in the framework of the displacement-based assessment procedures) can be a reliable alternative.

Starting from the limit analysis, by considering the classical hypotheses introduced by Heyman (1966) for modeling a rigid block model (infinite compressive strength, no tensile strength and no sliding permitted), it is possible to obtain the curve α-d_C (Fig. 6), which represents the collapse

multiplier α (that quantify the limit equilibrium of the system) for increasing finite values of the generalized displacement d, up to the value for which $\alpha(d_u)=0$. The collapse multiplier α is the ratio between the horizontal applied force and the weight of the panel. This curve corresponds to the capacity curve of the system in the case of a single block (Lagomarsino, 2014).
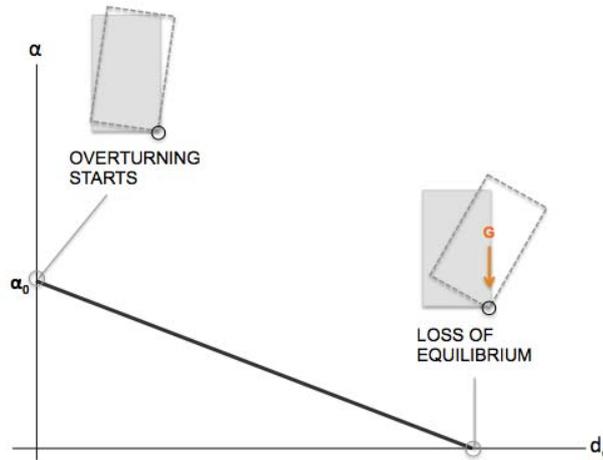

Figure 6. Curve α-d obtained through a static approach

The results of the static out-of-plane tests (test a) are illustrated by comparing the experimental curves obtained in the two configurations with and without added masses. Table 4 summarizes data of each panel in terms of: assumed specific gravity (γ; height of the plastic hinge (H_{HINGE}); height of the monitored marker (H_{MARKER}); level where the pulling force has been applied (H_F), height of the center of gravity of the overturning part of the panel, in the configuration without (H_G) and with ($H_{G, MASS}$) added loads.

Table 4. Obtained data for each panel

Panel	γ kN/m^3]	H_{HINGE} [m]	H_{MARKER} [m]	H_F [m]	H_G [m]	$H_{G, MASS}$ [m]
1		0	0.80	0.73	0.55	-
2	22000	0	0.67	0.60	0.45	0.56
3		0.3	1.12	0.70	0.60	0.71

Fig. 7 shows the comparison between the experimental curves for each levels of imposed displacements D_i, in both the configurations without and with added loads; experimental results are expressed in terms of spectral accelerations $a*$ and spectral displacements $d*$, according to the procedure described in Lagomarsino (2014). These curves, compared with the ideal one describing the rigid block behaviour (identified with the continuous black line in figure), shows a good correspondence for panels 1 and 2, highlighting that this assumption seems to be reliable even for poor masonry.

In particular, it is interesting to observe that, while for panels 1 and 2 the experimental curves fit the theoretical one, in the case of panel 3, the curve is lower both in terms of displacements and strength, due mainly to the position of the plastic hinge. In fact, while in panels 1 and 2 the hinge appeared at the bottom of the panel, at the interface with the base, in panel 3 the hinge is formed at 30 cm from the base, where the stone blocks were badly interlocked (Fig. 8). As a consequence, the first two panels behaved almost as rigid blocks, while the third panel did not benefit of the confinement effect determined by the base below and the section is gradually considerably deteriorating. This implies a significant shift of the plastic hinge and reduction of the capacity, both in terms of strength and displacement.

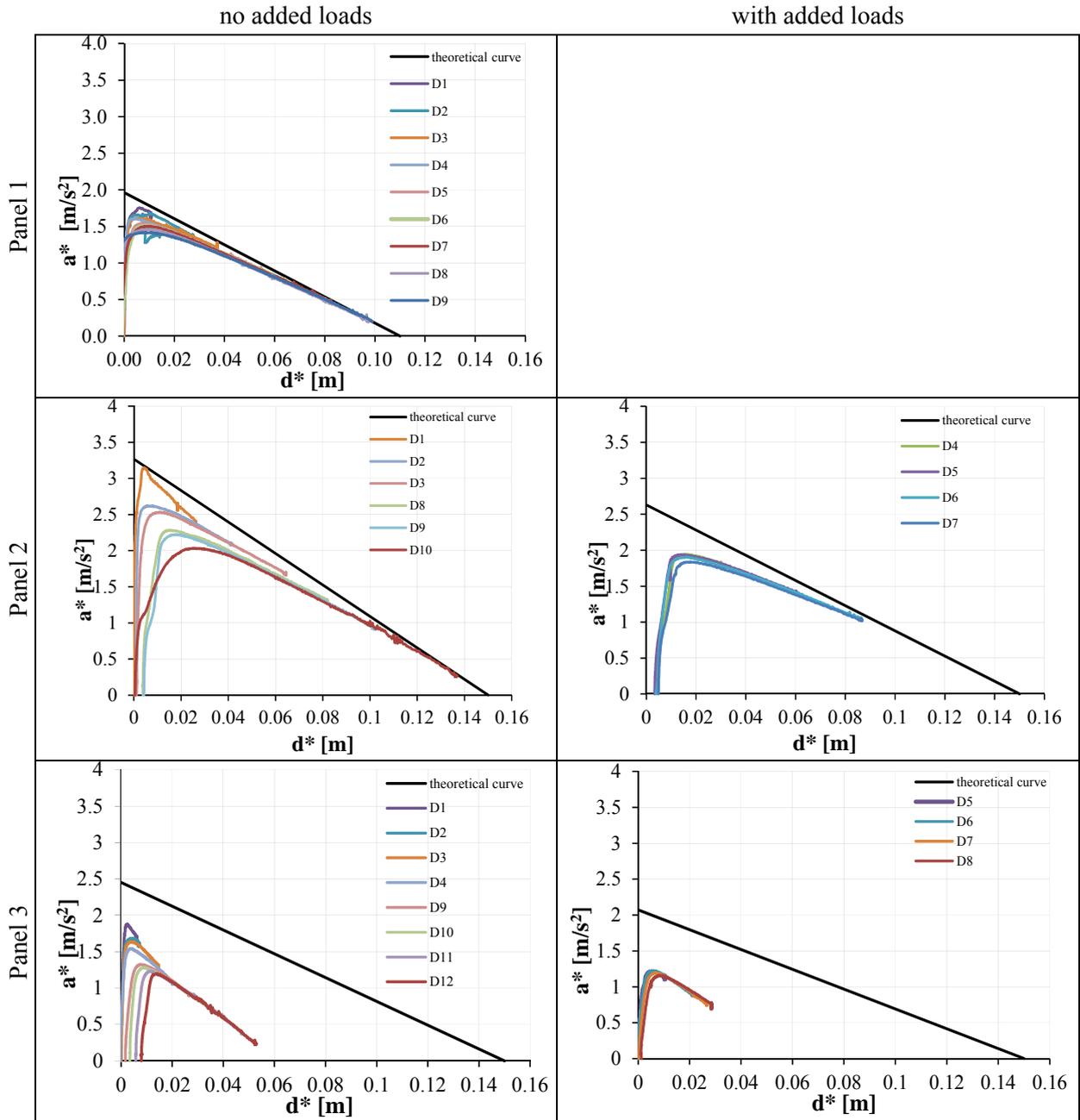no added loads   with added loads

Figure 7. Experimental results in terms of capacity curve for panels 1, 2 and 3

However, in general for the all panels, it was possible to observe that (Fig. 9): firstly, there is an initial elastic stiffness, due to the deformability which interested from the outset the panel behaviour, unlike the rigid block assumption; then, the initial stiffness of the panel tends to deteriorate, while the imposed displacement is increasing. Hence, the experimental curves tend to the ideal one, describing the rigid block assumption, without reaching the maximum value of spectral acceleration a_0* correspondent to the mechanism activation. Finally, the value of the spectral displacement corresponding to the panel complete collapse is very close to the rigid block one; so the assumption of infinite compressive strength seems to be reliable, even for a rather poor masonry.

Figure 8. Formation of the plastic hinge in panels 1, 2 and 3

For all these reasons, the experimental results show that the behaviour seems to be better represented by a dynamic bi or tri-linear model, rather than the classic rigid block model, characterized by the rigid-nonlinear behaviour (represented by the continuous black line).

Furthermore, it is interesting to observe that all experimental curves, for the different levels of imposed displacement, do not start from a value of the spectral displacements equal to zero, but present a residual displacement, due to the fact that close to the plastic hinge masonry is gradually deteriorating.

Figure 9. Revision of the rigid block assumption

RESULTS OF FREE VIBRATION TEST: EVALUATION OF DAMPING

As above-mentioned, after the static out-of-plane tests, free vibration tests have been performed on each panel, in order to study the rocking response in the nonlinear range. In particular, the nature of inherent damping mechanism, due to impacts and any sort of dissipation, has been investigated, by obtaining the damping ratio ξ the decay of free vibrations technique and by considering it as an equivalent viscous damping, as commonly assumed in the dynamic analyses. More in detail, the damping ratio has been estimated by measuring the decay in the displacements amplitudes at each cycle, as following described more in detail.

The equation of motion for an underdamped vibrating single degree of freedom (SDOF) can be evaluated as

$$u(t) = e^{-\alpha t}(A_1 \cos \omega_d t + A_2 sen \omega_d t) \tag{1}$$

where A_1 and A_2 are arbitrary constants depending on the initial conditions; $\omega_d = \omega \sqrt{1 - \xi^2}$ is the damped circular frequency in rad/s; ω is the undamped circular frequency; ξ is the viscous damping ratio. Eq. (1) may be represented as a multiplication of two functions

$$u(t) = f_d(t) \cdot f_0(t) \tag{2}$$

where $f_d(t)$ is a damping function and $f_0(t)$ is a non-decreasing oscillating function (which is a harmonic function, for a discrete SDOF system). The damping function is an exponentially decreasing function and represents the envelope of the free vibration response and it takes the following form:

$$f_d(t) = \rho e^{-\alpha t} \tag{3}$$

where ρ is the amplitude of the function and $\alpha = \omega \xi$. In the following, the displacement decay has been evaluated. The logarithmic decrement δ (which allows to determine the viscous damping ratio) is defined as:

$$\delta = \ln \left[\frac{e^{-\alpha t_i}}{e^{-\alpha\left(t_i + \frac{2\pi}{\omega_d}\right)}} \right] = 2\pi \frac{\xi}{\sqrt{1 - \xi^2}} \tag{4}$$

Eq. (4) is simplified in the case of slightly damped system (with $\xi \ll 1$); so, viscous damping ratio can be determined by means of:

$$\xi = \frac{\delta}{2\pi} \tag{5}$$

Fig. 10 illustrates the displacement amplitudes of the monitored marker (Fig. 3a) for the different levels of imposed displacements D_i for the three panels, considering both the testing configurations (with and without added loads). It is worth noting that, for panel 1, the test has been performed until the panel collapsed; for this reason the last test, corresponding to the imposed displacement D_9, has not been recorded.

Fig. 12 shows the damping evaluation for the most significant values of imposed displacements D_i and for each panel tested without added loads. These values have been determined by considering separately the positive and negative peaks of displacements, as sketched in Fig. 11.

As it is possible to deduce from the diagram of Fig. 12, the damping increases with the displacement from the initial value of approximately 4% (very close to the classical assumption of 5% proposed by the Codes) till to around 10%. Damping increase occurs in the pseudo-elastic phase, that is the ascending branch of the capacity curve; this is because of a progressive deterioration of masonry, at the interface. After that, during the rocking phase, equivalent damping remains almost constant; this is coherent with the behaviour of Housner model (Housner, 1963), in which dissipation is directly related to the coefficient of restitution of the rigid block (Makris and Konstantinidis, 2003).

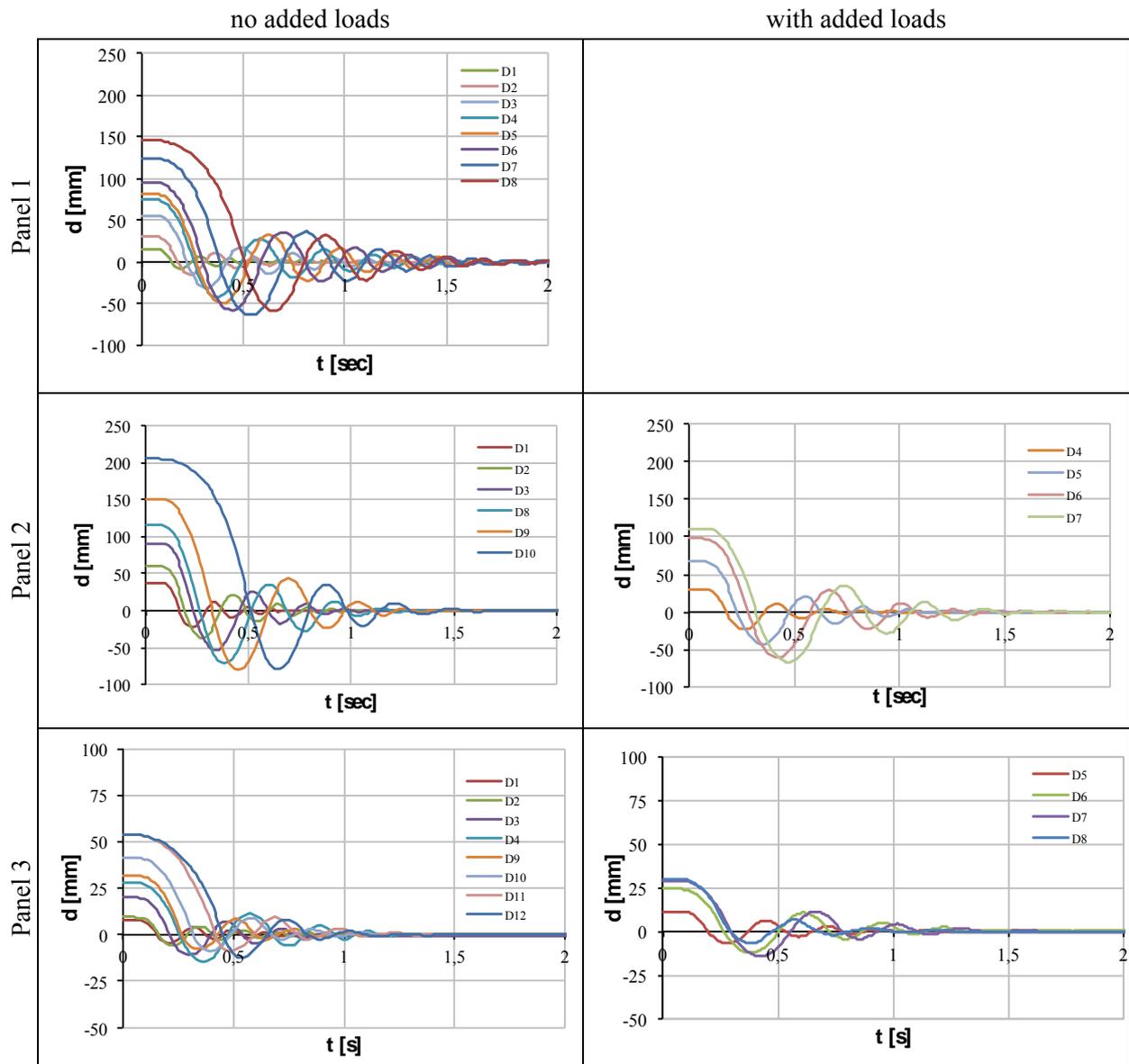no added loads   with added loads

Figure 10. Free vibration test results in terms of displacement amplitudes for panels 1, 2 and 3
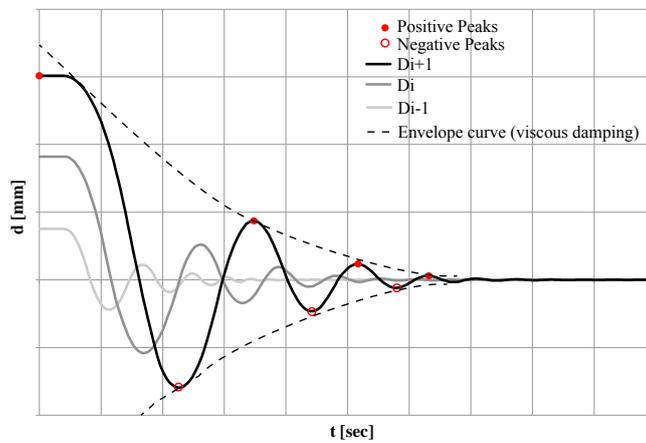
Figure 11. Evaluation of viscous damping from the decay of free vibrations
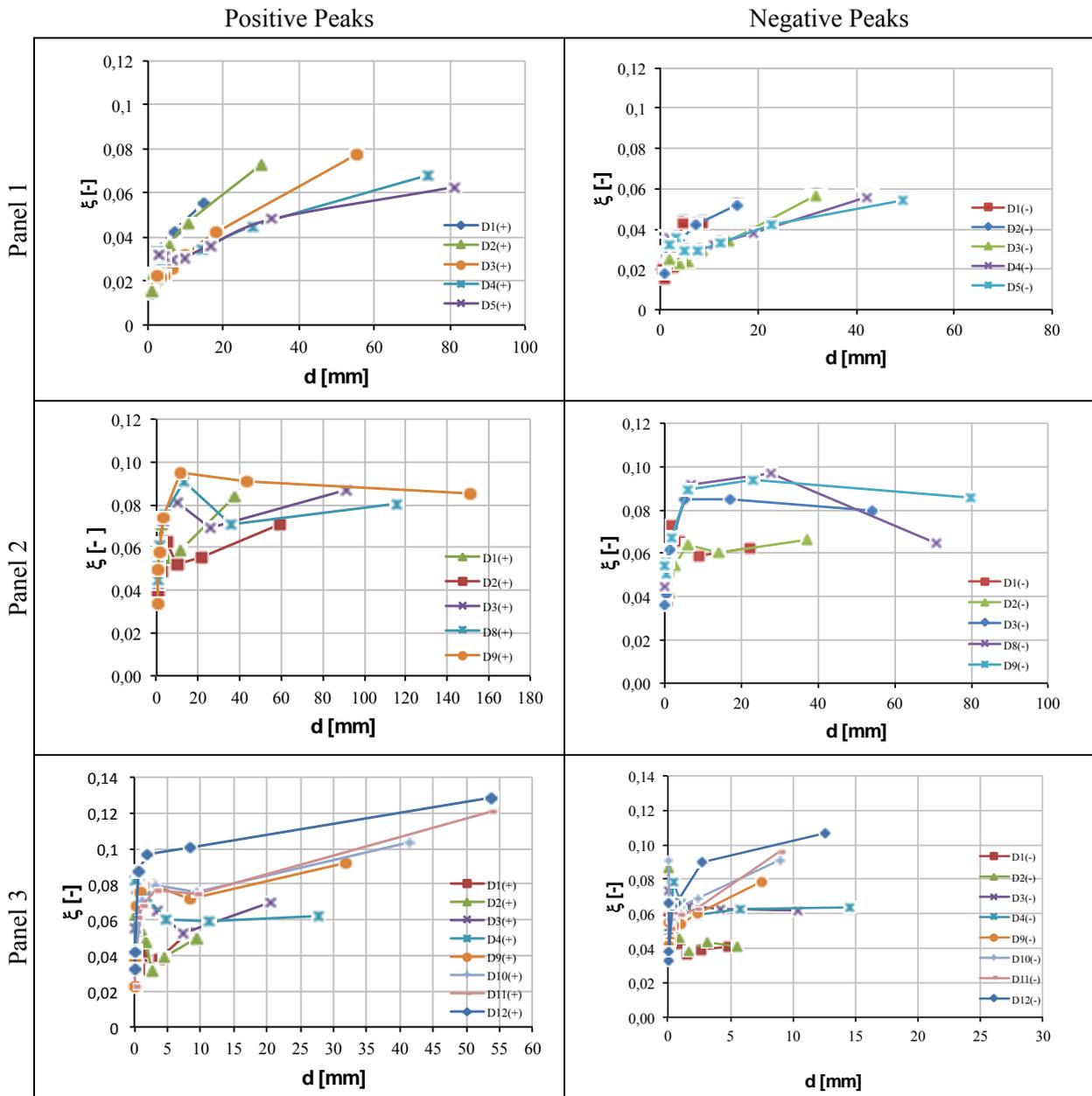
Positive Peaks   Negative Peaks

Figure 12. Damping values obtained from free vibrations as a function of displacement amplitude

CONCLUSIONS

In general, experimental results have shown that the rigid block model is able to properly describe the seismic out-of-plane response, by introducing proper modelling strategies in order to take into account the real behaviour of this kind of structures.

In particular, results have shown: a) the necessity to consider an initial elastic stiffness (so, the behaviour is better represented by a dynamic bi or tri-linear model, rather than the classic rigid block model); b) the sudden damping increase with the displacement, from the initial value of 4% till to around 10%; c) the panel initial stiffness deterioration with the increasing imposed displacement; d) a residual displacement, due to the fact that the section where the plastic hinge has developed is gradually deteriorating.

ACKNOWLEDGEMENTS

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