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## Research Article

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# THE EFFECT OF TRANSVERSAL CONNECTION IN THE IN-PLANE RESPONSE OF DOUBLE-LEAF BRICK MASONRY WALLS

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**Abstract.** The evaluation of structural safety derives from the knowledge of material properties. In case of existent masonry building, the definition of reliable mechanical parameters could be a very difficult task to be achieved. For this reason, an estimation of these values is useful, for example it is the first phase of the knowledge process, for simplified mechanical model or when NTD test is the only possibility.

The transversal connection in masonry panels is a technological detail that affects the static and seismic behavior and could significantly increase the strength of the element.

In this paper the effect of transversal connection in double-leaf brickwork masonry panels is evaluated by diagonal compression tests. To achieve this goal, a new set-up was designed to load each leaf independently.

The results have shown an increment of about 20% in strength if transversal connection is present.

If the leaves have very different mechanical parameters, the tests highlight an unexpected behavior.

**Keywords:** shear strength, diagonal compression test, masonry panel, sonic tests, in-plane response, diatoni

## 1 Introduction

The seismic assessment of existent masonry structures is still a challenge in the retrofit design process (Mendes et al. 2017). A bad evaluation of the structural behavior can lead to an unexpected structural response with the possibility of high damage level. Three different cases can usually occur: i) identification of the correct response but an underestimation of the safety level; ii) identification of the correct response but an overestimation of the safety level; iii) wrong identification of the structural behavior. In the first case, invasive and extensive retrofit interventions are required to improve the structural seismic capacity. A high number of strengthening solution can significantly change the intrinsic behavior, bringing sometimes to new failure modes as the local mechanisms. In the second case, very soft interventions are needed, and only local solution are applied. The result is a vulnerable structure with a potentially high seismic risk. If a wrong identification of structural response occurs, the retrofit interventions distance further the structural behavior. Strengthening are located in the worst areas of the structure.

The correct knowledge help to reduce the possibility of errors and increase the confidence of seismic assessment (Kržan et al. 2015).

In the Italian Building Code, the mechanical properties of masonry could be evaluated starting from a min-max range defined on masonry typology, increased by modifying coefficients able to take into account the variability of this type of construction material (i.e. mortar quality, joints thickness, effect of transversal connection, etc). Sometimes it is necessary to use mechanical parameters without in-situ tests or to have a comparison with the experimental ones. The aim of this paper is to provide a quantitative estimation of the in- plane strength benefit of transversal connection in masonry panels.

As is of common knowledge, the morphology plays an important role in static and seismic behavior of masonry structures and for this reason should it be considered properly. The most important parameters in knowledge process are related to the masonry quality, in particular mechanical characteristics and technological details. These two aspects influence each other, both in the presence of static actions alone and combined with seismic actions. In both of them, the masonry quality plays an important role, especially if the walls have a multi-leaf section (Binda et al. 2006; Casolo et al. 2013; Egermann and Newald-Burg 1994; de Felice 2011; Vintzileou and Miltiadou-Fezans 2008). The presence of

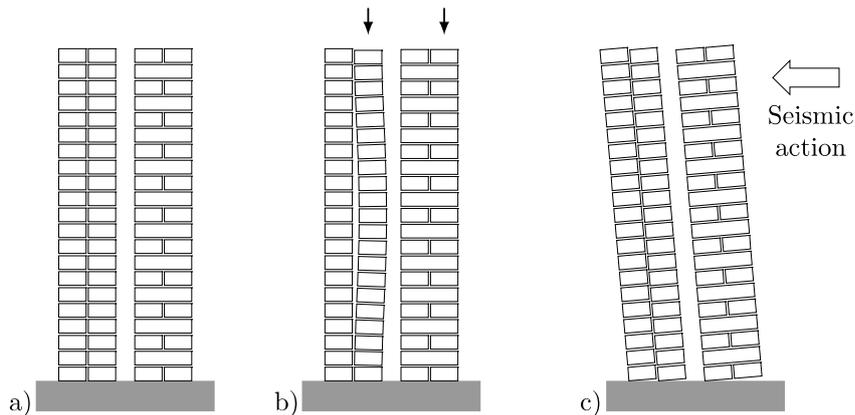
transversal elements that links the leaves, also called in Italian "diatoni" (this term will be used in the paper thanks to its conciseness), allows a better response of the panel. For this reason, the identification of *diatoni* is an important task. Sonic test and tomography are usually the best non-destructive test (NDT) to evaluate the leaves connection (Miranda et al. 2013; Miranda et al. 2016; Silva et al. 2014). In absence of this technological detail, with the aim to prevent local failure and to increase compression and shear strength, a lot of research is being done on injections and "artificial" *diatoni* (Corradi et al. 2017; Silva et al. 2014; Valluzzi et al. 2004; Vintzileou 2011; Vintzileou and Tassios 1995). In this paper only brickwork masonry panels have been considered. The selection of this type of masonry was been led by two main aspects: less uncertainties due to materials and assembly and the damage occurred during 2012 Emilia-Romagna earthquake (Figure 1). To achieve this goal an experimental campaign was carried out analyzing the response of 12 masonry panels.



**Figure 1.** Damage due to separation of multi-leaf brickwork masonry wall after 2011 Emilia earthquake. Finale Emilia: a) Church of SS. Filippo e Giacomo; b) Modenesi's Tower.

Considering Figure 2a, two extreme configurations could be identified for a double-leaf brickwork masonry wall, depending on the orderly presence of connections. From a static point of view, the lack of bonding between the leaves, can lead an instability phenomenon before the ultimate compression

load of the masonry is reached (Figure 2b). In particular, the Eulerian instability load of the wall can be up to eight times lower if there is no connection between leaves and it can condition the in-plane behavior. Regarding seismic load, if a simple overturning mechanism is considered, the activation multiplier is double in the case of efficient *diatoni* since only one rotation point for the system exists (Figure 2c).



**Figure 2.** Two leaves walls under static and seismic action: a) transversal sections; b) instability due to vertical loads if no transversal connection exists; c) seismic behavior: simple overturning mechanism.

The surveys after earthquakes have shown, in some case, the overturning of the external leaf only, due to different boundary conditions and stress distribution of the leaves (i.e. slab or roof).

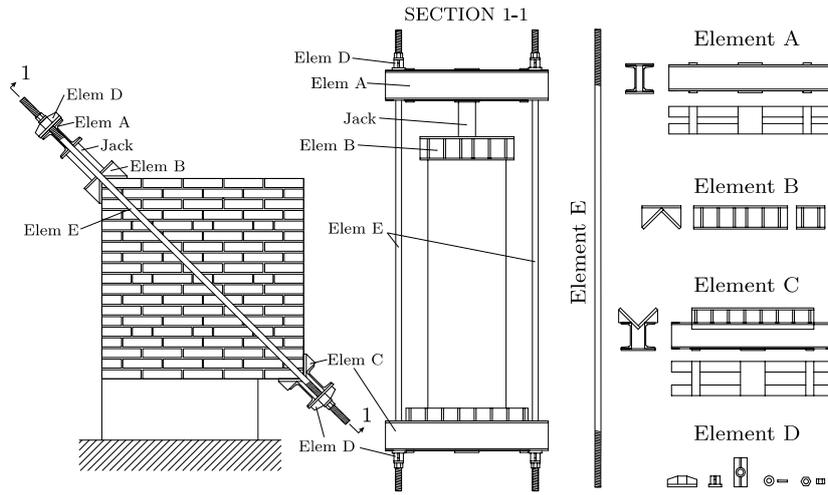
The assessment of in-plane behavior of masonry wall is usually based on simplified models able to describe the global response of the element with regards to different failure modes (flexural, sliding, diagonal cracking). Nevertheless, in many cases the real response is a mixture of the single modes. Few parameters are required to estimate the mechanical parameters of the masonry, that can be evaluated through laboratory tests. For different reasons, the identification of mechanical parameters in existing masonry building is not a simple task. The non-destructive or semi-destructive tests (rebound hammer, sonic test, tomography, double flat-jack) are not able to quantify strength values. The destructive tests (compression test, Sheppard test, diagonal compression test, shear-compression test) provide useful parameters but have to be contextualized in a mechanical model. Moreover, they cannot be extended to the whole structure and, for this reason, they are specific of a local condition of a defined masonry typology. The boundary conditions that affect the in-situ test (damage during cutting operation,

eccentricity of loads, unknown of vertical load), can lead to non-conservative results. Despite uncertainty, a quantitative estimation of mechanical parameters is necessary to avoid banal errors of an expert evaluation or wrong identification of the masonry typology (referred to the code). Regarding the seismic action, diagonal cracking is the main failure mode. Moreover, the most important parameter is the shear strength of the masonry and the diagonal compression test is, probably, the best way to investigate it.

## **2 Diagonal compression test**

### **2.1 Standard test**

This test was developed to indirectly estimate the shear strength of masonry walls and it is more accurate with respect to other available methods. The applicability of the set-up at laboratory and in-situ, provides a useful tool to compare the diagonal tensile strength for new and existent masonry walls. An example of the loading scheme of the test is represented in Figure 3. Today a wide knowledge already exists, and a lot of applications were made (Almeida et al. 2015; Borri et al. 2011; Brignola et al. 2008; Corradi and Borri 2018; Knox et al. 2018; Milosevic et al. 2013), permitting a good reliability of structural assessment. Even if international technical documents exist, such as RILEM (TC 76 LUM, 1976), ASTM (E519-E519M, 2015), interpretation of the test results and definition of mechanical parameters, is not so easy (Calderini et al. 2010). Indeed, if a Turnšek and Čačovič model is used (Turnšek and Čačovič 1971), the diagonal cracking occurs when the maximum principal stress matches the diagonal tensile strength of the masonry ft. If a Mann and Müller model is adopted (Mann and Müller 1982), the definition of cohesion and friction angle requires two tests that must be defined properly.



**Figure 3.** Loading scheme example with steel frame elements required for standard diagonal compression test.

In this work, a Turnšek and Čačovič model is assumed since the main aim is to quantify the influence of transversal connection. Moreover, if the quality of mortar and block are not so different, it is possible to assume that, in the safe side of the analysis, the mechanism could be well represented by Turnšek and Čačovič model.

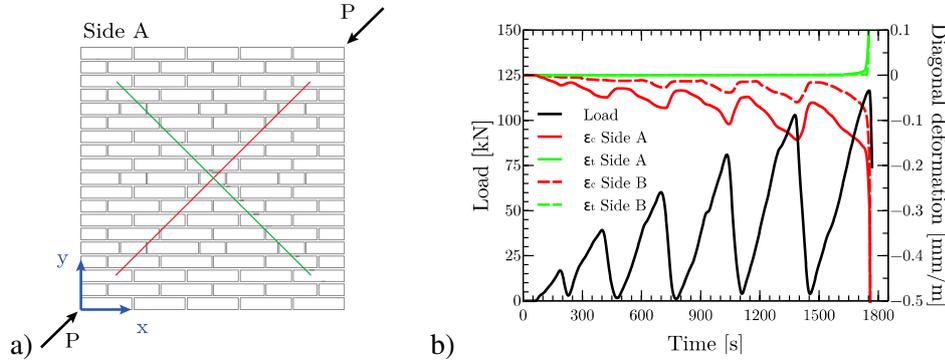
With reference to a diagonal compression test, Frocht demonstrated that the stress state at the center of the panel is very different from a pure shear stress state, by using photo-elasticity and theoretical approaches (Frocht 1931). The good approximation of Frocht's solution was confirmed by numerical elastic finite element analyses (Brignola et al. 2008; Yokel and Fattal 1976), where the stresses  $\sigma_x = 0.56P_d/A$ ,  $\sigma_y = 0.56P_d/A$ ,  $\sigma_z = 1.05P_d/A$ ,  $\tau_{xy} = 0.5P_d/A$  and  $\tau_{yz} = 1.62P_d/A$  were reached at the center of the panel (being  $P_d$  the maximum diagonal load and  $A$  the transversal area of the specimen).

## 2.2 Post-processing of diagonal compression results

The applied load and the diagonal displacements on each sides of the panels are usually recorded during the test (Figure 4). According to Turnšek and Čačovič, only one parameter is necessary to define the failure domain. The tensile stress reached at the center of the panel could be directly derived from  $f_t = \tau_{xy} = 0.5P_d/A$ . The shear strength design parameter can be easily derived dividing the maximum tensile stress by 1.5 obtaining  $\tau_{norm} = f_t/1.5 = 0.333P_d/A$ . The estimation of other mechanical parameters, as shear deformation  $\gamma$  or shear modulus  $G$ , require the definition of shear deformation vs shear stress curve. The tangential stress is evaluated starting from the "exact" solution as  $\tau_{xy} = 1.05P_d/A$

while the tangential deformation  $\gamma$  is obtained as the difference between the compressive strain and tensile strain along the diagonals, averaging the two sides of the panel  $\gamma = \epsilon_c - \epsilon_t$  where, for example,

$$\epsilon_c = (\epsilon_c^A + \epsilon_c^B) / 2.$$



**Figure 4.** Example of recorded data: different behavior between sides A and B of the panel.

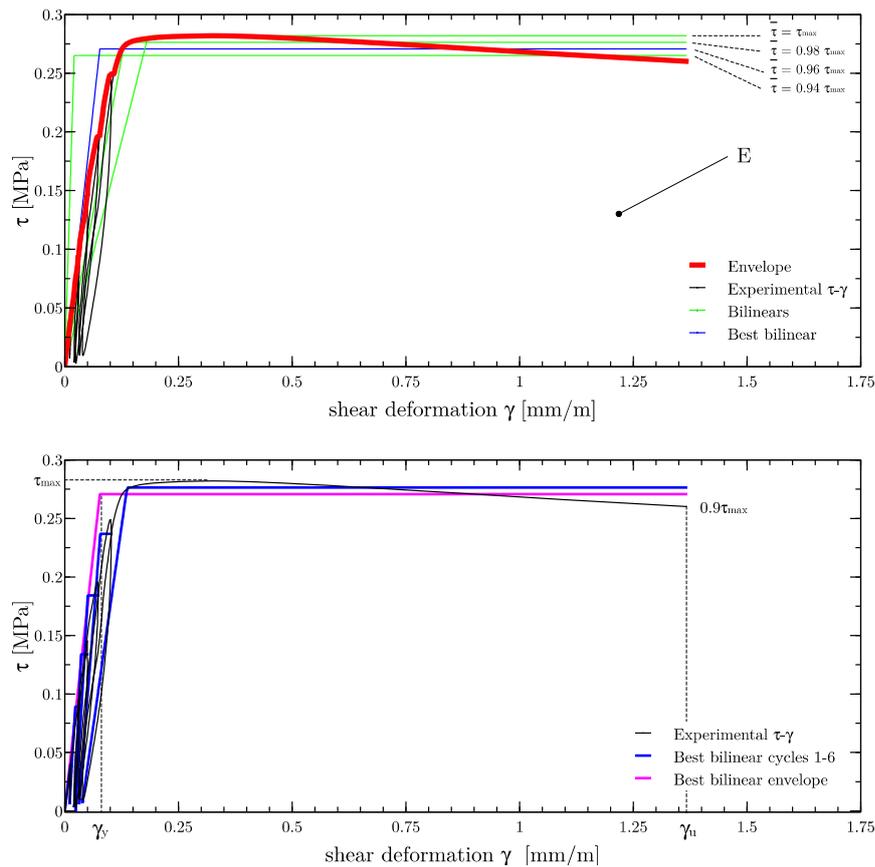
The curve could be retrieved for each loading cycle or for the envelope of the test. The procedure (Figure 5) usually adopted by the author is to define a bilinear curve with the following criteria: the equivalence of underneath area for a fixed value of shear stress  $\bar{\tau}$  and the maximum shear deformation is defined when a reduction of  $0.1 \times \tau_{max}$  is reached. With these assumptions the yielding deformation  $\gamma_y$  could be obtained from Eq. 1, being E the area under the envelope curve.

$$\gamma_y = 2 \left( u \frac{E}{\bar{\tau}} \right) \quad (1)$$

The shear stress  $\bar{\tau}$  is varied starting from  $\tau_{max}$  and decreasing it up to  $0.9 \times \tau_{max}$ , choosing the solution with the minimum standard deviation ( $\bar{\tau} = \tau_{max} \times 0.9$ ).

It is possible to evaluate directly from the recorded data, an instantaneous shear modulus as  $G = \tau/\gamma = 1.05P/A/\gamma$  but a useful estimation of elastic shear modulus could be derived from the bilinear curve, evaluated for the envelope curve (but the same operation could be done for each cycle) obtaining a value representative of a cracked behavior, as stated in Eq. 2:

$$G_{crack} = \frac{\bar{\tau}}{\gamma} = \frac{1.05 \times P_d}{A_y} \quad (2)$$



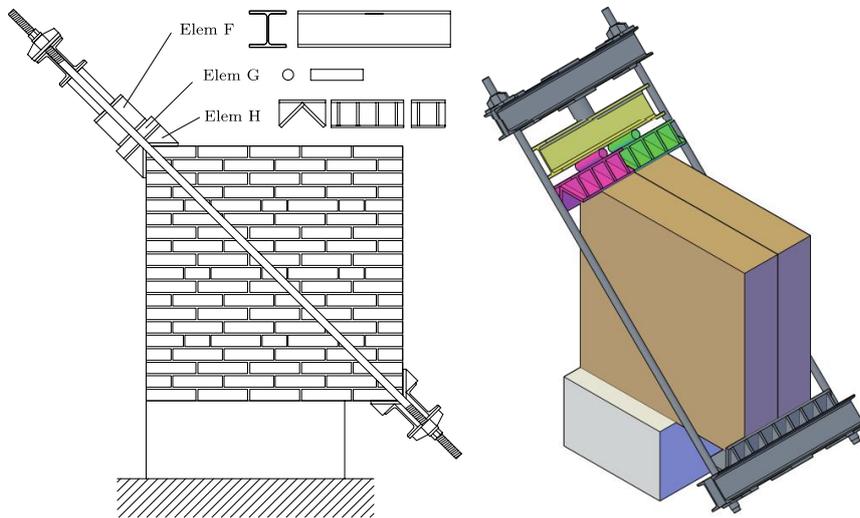
**Figure 5.** Graphical representation of post-processing of recorded data.

The analysis of shear (or axial) deformations usually shows a different behavior between the two sides of the panel (Brignola et al. 2008), an example is illustrated in Figure 4b. This response could be ascribed to a non-centered diagonal load with respect to the wall thickness, different stiffness of steel loading frame and different response of masonry leaves. The result is a rotation of element B (Figure 3) with a different load transferred to each leaf. To avoid this behavior, a modified set-up was designed to separately load the leaves and understand the role of *diatoni* in the in-plane response.

### 2.3 New set-up

With the traditional diagonal compression set-up, it is not possible to analyze the influence of transversal connection since the applied load is uniformly transferred to the panel. For this reason, a new set-up was developed to evaluate the different response of the leaves in double-leaf masonry panels. The goal is to obtain a loading scheme that allows to subdivide, as needed, the force applied to each leaf. Element B of traditional scheme was split in two independent parts (Element H) and two steel

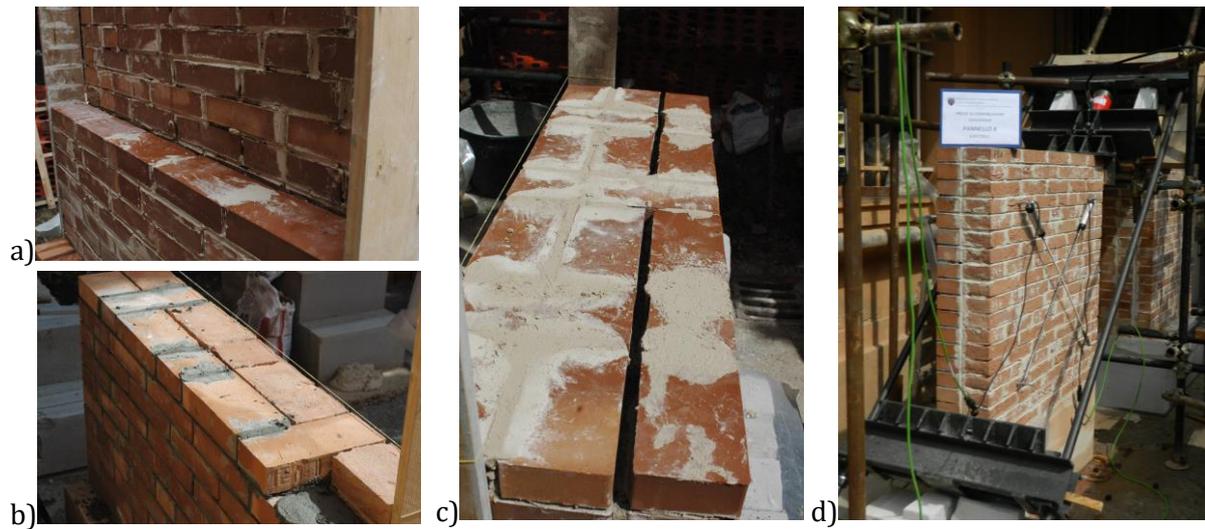
cylinders were added upon them (Element G). The latter elements ensure to change the load transferred to each leaf modifying their position. In particular, the cylinders were always arranged in the middle of each leaf to proportionally transfer the overall load to the thickness of the leaves. In Figure 6 the new steel elements are identified and a tridimensional representation of the set-up is shown.



**Figure 6.** New set-up: new elements added or modified and a 3D sketch of the whole steel frame.

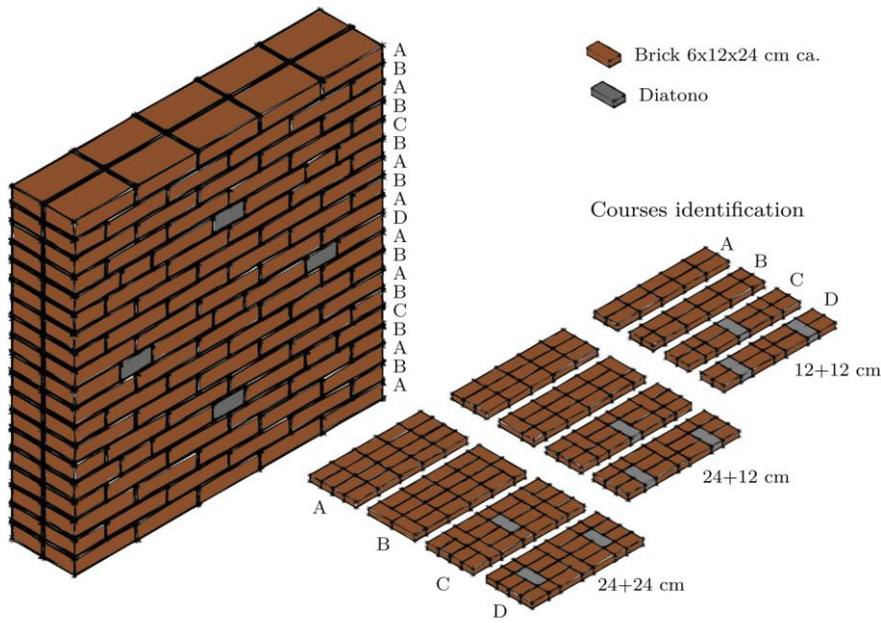
### 3 Experimental campaign

The goal of this research is understanding the role of transversal connection in multi-leaf brick masonry panel. To achieve this result, 12 different double-leaf specimens were realized, 6 with *diatoni* and 6 without them. The courses organization and the position of transversal elements was defined to preserve the same texture for each panel as shown in Figure 8. If no *diatoni* are present, the two leaves have been built independently, with no connections. In all the cases, a gap of 1 cm was left to avoid inadvertent contact, filled step by step with gravel (Figure 7). The role of leaf thickness was taken into account. Different configurations were defined with equal leaf thickness 12+12 cm (one-head leaf) or 24+24 cm (two-head leaf) and with different thickness 12+24 cm. For some of 12+12 cm specimens, material properties of one leaf were modified with the purpose of introducing a different response of the leaves and understand how *diatoni* could operate.



**Figure 7.** Some images of specimens and test setup: leaves construction in the case a) without *diatoni* (panel P5), and b) with *diatoni* (panel P8); c) transversal connection in non-uniform thickness leaves (panel P6) and d) just before the start of compression test on P8 element.

Two types of bricks (B) were used: B\_A is a standard brick while B\_B is a un-plastered brick with sandblasted surfaces. The first one has 50 MPa of compressive strength while the second one 30 MPa. Analogously, two types of mortar (M) were employed: M\_A is a lime mortar while M\_B is a cement mortar. The first one has 0.75 MPa of flexural strength and 1.55 MPa of compressive strength while the second one 2.85 MPa and 9.00 MPa, respectively. Bricks and plasters were tested according to UNI EN 1015-11:2007. Moreover, additional tests were made on small masonry specimens of one-brick length and thick, with three and five courses, composed with B\_A and M\_A, according to UNI EN 1052-1:2001. The results provide a compressive strength  $f_m$  of 12.6 MPa (7 tests) and 11.0 MPa (5 tests) respectively, with no scattered values. Some test on small specimens of half-brick length with three courses and two-brick length with five courses were also carried out. The outcomes confirmed the previous results: 10.3 MPa (4 tests) and 9.5 MPa (2 tests) of compressive strength.



**Figure 8.** 3D sketch of 24 cm panel and courses identification of double-leaf masonry walls with transversal connections.

In Table 1 specifications of the 12 panels are shown where the different configurations are able to investigate 3 different parameters: materials, leaf thickness and transversal connections. The specimens ID has an alternation between presence or not of *diatoni*, in order to have an identifier  $P_i$  for the panel without transversal connection and  $P_{i+}$  for the same panel configuration but with *diatoni*.

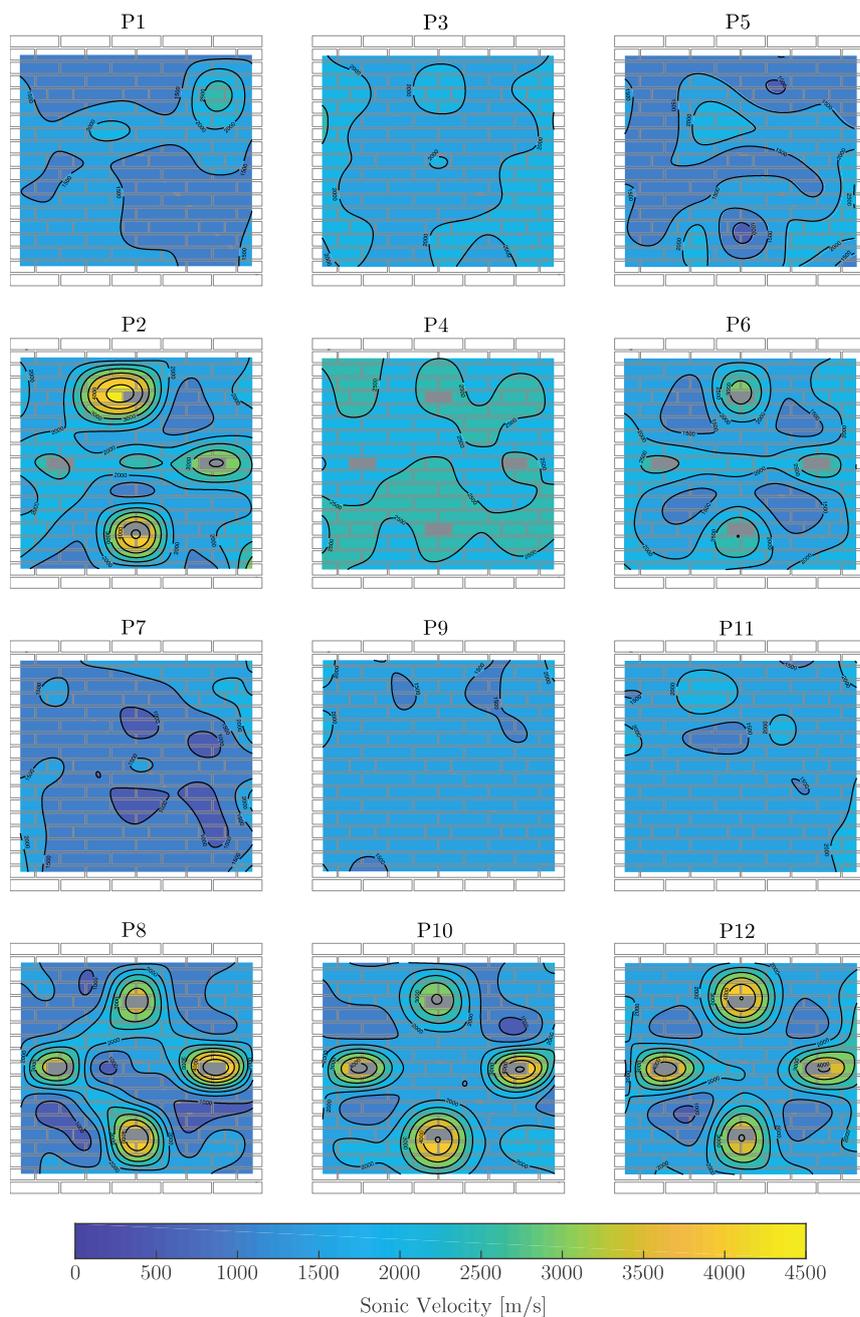
**Table 1.** Specimen details.

ID [-]	leaf A [brick and mortar types]	leaf B [brick and mortar types]	thickness [cm]	<i>diatoni</i> [Y/N]
P1	B_A - M_A	B_A - M_A	12+12	N
P2	B_A - M_A	B_A - M_A	12+12	Y
P3	B_A - M_A	B_A - M_A	24+24	N
P4	B_A - M_A	B_A - M_A	24+24	Y
P5	B_A - M_A	B_A - M_A	24+12	N
P6	B_A - M_A	B_A - M_A	24+12	Y
P7	B_B - M_A	B_A - M_B	12+12	N
P8	B_B - M_A	B_A - M_B	12+12	Y
P9	B_A - M_B	B_B - M_A	12+12	N
P10	B_A - M_B	B_B - M_A	12+12	Y
P11	B_A - M_A	B_A - M_A	12+12	N
P12	B_A - M_A	B_A - M_A	12+12	Y

For each panel, compression tests were performed with the new set-up described in §2.3. The load was applied through a hydraulic jack with 300 kN of capacity (for panels P3-P4 a jack with maximum capacity of 1000 kN was used instead). The diagonal elongations were measured with traditional potentiometric transducers with 50 mm gauge length. The acquiring system recorded simultaneously the displacements of the 4 diagonal transducers and the pressure applied to the hydraulic jack. Some loading cycle was made to remove mechanical play, to train the masonry and to verify the load-unload quasi-elastic path. After that, at least 4-5 cycles incrementing the maximum load were done before the collapse of the specimen.

### **3.1 Indirect identification of diatoni**

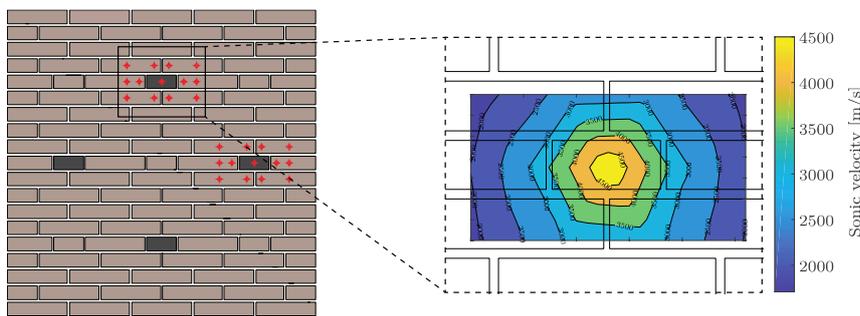
The presence of transversal connection in existing masonry walls is very difficult to check since in many cases non-destructive test are the only ones allowed. In this cases, sonic test or tomographic test are very useful techniques to investigate into the thickness of masonry elements. It is important to highlight that transversal connection can be clearly investigated for about 60 cm wall thick with thin outer plaster. The external layers could smooth the pick velocity and limit the *diatoni* identification. On the entire set of samples, sonic tests were made on a grid with 20 cm of spacing. The results are shown in Figure 9, where the panels were organized by row with and without *diatoni*, to quickly identify, by column, the differences among the specimens. Panel with transversal connections and low thickness (24 cm) show a high peak velocity. On the contrary, on the Panels P4, with 48 cm thickness, the identification of *diatoni* exact position is quite unattainable.



**Figure 9.** Sonic test.

Panel P6, with thickness of 36 cm, shows an intermediate behavior and the locationing of transversal bricks is possible. it is worth to mention that any panel without *diatoni*, have pointed out sonic velocity higher than 2000 m/s (except for a measure in P1). Even if P4 does not exhibit a clear map, the velocity is high enough to hypothesize a good transversal connection among the thickness. This smoothed map is due to the path of sonic waves that is not so dissimilar among *diatoni* and the other points. In the first case, the waves need to go through a brick- mortar-*diatono*-mortar-brick sequence while, in the second case, for example, brick-gravel-brick. The difference in waves traveling time is not so different to

identify the correct position of *diatoni*. Subsequently, further investigation was carried out to understand the influence of *diatoni* near their boundaries. Several sonic measurements were performed on these elements and the results are contained in Figure 10. Velocity decreasing is uniform and a value higher than 2500 m/s could be also observed 15 cm far from the center. For this reason, a grid 20x20 cm is enough dense. If the thickness is very high, the presence of transversal elements could be not so clear. Nevertheless, for field- stone masonry, the sonic velocity difference between lithotype and mortar is high enough to compensate the thickness of the panel and to make possible the detection of *diatoni*.



**Figure 10.** Sonic identification of *diatoni*. Test points (in red) and distribution of velocity around transversal element of P12 specimen.

#### 4 Tests results

For each specimen in Table 1 diagonal compression test with the new set-up was carried out and the recorded data were analyzed with the procedure described in Section 2. In Table 2 the results of the tests are shown. At the end of the tests on the 12 panels, 3 more investigations were carried out on the single leaf survived after such tests. The capitol letter after the specimen ID is referred to the tested leaf. In the table, the maximum reached load, the shear strength according to Turnšek and Čačovič model ( $\tau_{\text{norm}} = f_t/1.5$ ) and the cracked shear modulus  $G_{\text{crack}}$  are summarized.

Thanks to these results, several information and conclusion could be carried out. Considering the strength of P3A specimen, it is representative of a two- leaf panel with an infinite transversal connection. The strength of P2 (without *diatoni*) panel is exactly the double of the P3A ones, highlighting that the system works in a parallel way and the maximum load reachable is twice the value of the single leaf. With transversal elements the strength increases by 20% (270 kN  $\rightarrow$  318 kN). Similar behavior can also be found in panels P1-P2 (99 kN  $\rightarrow$  116 kN).  $P_u$  of P3A is the maximum value that

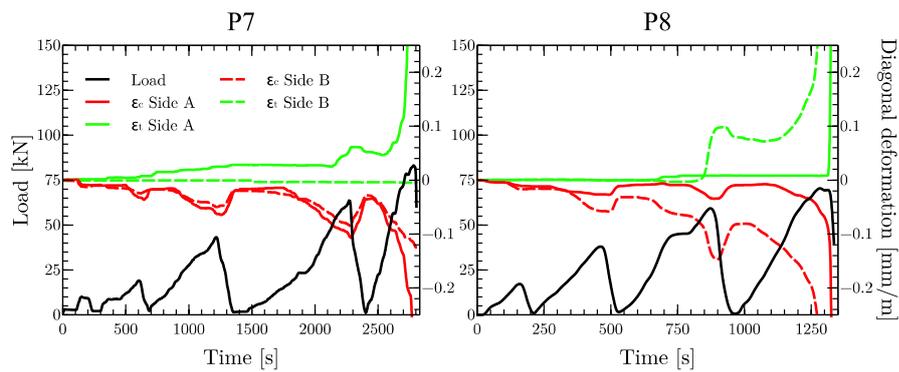
could be obtained by a 12+12 cm double-leaf panel if high number of diatoni is present (about 35%). If the leaves are different in materials (P7-P8), the presence of *diatoni* has caused a lower maximum load, roughly the 90% with respect to the absence of transversal connection. Analyzing leaf deformations (Figure 11) it is possible understand this unexpected behavior. In absence of *diatoni*, the two leaves are loaded in the same way and the collapse involved the weakest leaf (P7 side A). This behavior is also justified by the single leaf remained intact after the tests which have an odd ID (P3A-P7B-P9B). In presence of transversal connections, the different stiffness leads first to the collapse of the strongest leaf. Since it is less deformable, it absorbs also a ratio of the other leaf transferred by *diatoni*, and subsequently the carrying load is moved to the other leaf in an impulsive way.

**Table 2.** Specimen details.

ID [-]	thickness [cm]	diatoni [Y/N]	Pu [kN]	$\tau_{norm}$ [MPa]	G [MPa]
P1	12+12	N	99	0.108	2318
P2	12+12	Y	116	0.128	4715
P3	24+24	N	270	0.148	2808
P4	24+24	Y	318	0.174	2230
P5	24+12	N	127	0.097	2492
P6	24+12	Y	117	0.090	3507
P7	12+12	N	83.2	0.094	1592
P8	12+12	Y	70.4	0.080	2899
P9	12+12	N	34.0	0.038	1461
P10	12+12	Y	112	0.127	1389
P11	12+12	N	129	0.146	4736
P12	12+12	Y	112	0.128	6664
P3A	24	-	136	0.153	7273
P7A	12	-	80.8	0.183	7394
P9A	12	-	108	0.251	23459

Obviously, the weaker leaf is not able to support all the load, and for these reasons, the failure of the entire panel occurs (Figure 11). Unfortunately, P9 panel (P9-P10 panels are a reply of P7-P8) has shown a maximum load of only 34 kN with the collapse of the weaker leaf (side B) in the case of absence of diatoni. Since the single leaf P9A collapsed for 108 kN, probably the panel P9 could have reached a higher value with respect P10, confirming the result of the previous panels (P7-P8). These results seem lead to the following assumption: if the masonry panel is made with two or more leaf with non-homogeneous properties, but couple together, the weak leaf is able to lead the entire panel to the

collapse. The behavior must be ascribed to the different stiffness of masonry leaves. Indeed, this deduction could find a support also in P5-P6 tests. Even if the two leaves were been loaded in their barycentre, the stiffness of the 24 cm leaf is higher than the twice of 12 cm ones, due to the effect of an infinity transversal connection, as already stated in the case of P1-P2-P3A. The load-deformation behavior of these panels was exactly analogous to P5- P6 cases, as shown in Figure 11. It is correct to highlight that, despite the P11-P12 specimens were a reply of P1-P2 panels, the results did not confirm the previous ones.



**Figure 11.** Test results for P7 and P8 specimens. For clarity, the continuous lines is referred to the weaker leaf while the dashed line to the stronger one.

The effect of transversal connection could be found analyzing the damage pattern occurred after the tests (Figure 12). The even specimens have always a specular damage path, while in the odd panels the absence of transversal connection allows a different behavior between the leaves. In most of the cases, the cracks affected the mortar joints with the classic pattern and without relation with the presence of diatoni. Sometimes a horizontal sliding mechanism occurred, probably due to a non-perfect-diagonal load but also to the effect of transversal element as stated in case P12. This failure mechanism, that involved from P9 to P12 panels, could be the reason for the unexpected results of this specimen. It is worth noting that, the disassembly of the panels after the test was made carefully for even specimens to check the status of the transversal elements. In all the analyzed cases, diatoni remained intact (Figure 13) highlighting that only few elements can be sufficient to transfer the carrying load among the leaves. On the other hand, a high number of transversal elements can modify the crack path and increase the shear strength of the panel.





**Figure 13.** Transversal elements after tests in P8 (left) and P6 (right) panels.

## 5 Conclusions

In this paper the evaluation of transversal connection in double-leaf masonry panels was analyzed through experimental tests with an innovative set-up for the diagonal compression test. 12 specimens were built with different blocks and mortar materials and were tested applying the loading forces to each leaf independently. The results highlight that, less the homogeneous of leaves is, less the maximum load that could be reached is. Moreover, according to the Italian National Code, a masonry with transversal connection could exhibit a maximum shear strength of about 130% with respect to a double-leaf masonry without *diatoni*. Moreover, the sonic tests made on the specimens, confirm the grid spacing of 20 cm. With this geometry, the presence of *diatoni*, could be defined very well. On the contrary, if a good transversal texture is it present, the single transversal element could not be so clearly identified. Nevertheless, the overall sonic velocity is high enough to ensure a "monolith" response.

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**Availability of data and material** Data of experimental tests could be obtained by writing to the corresponding author.

**Code availability** Not applicable.

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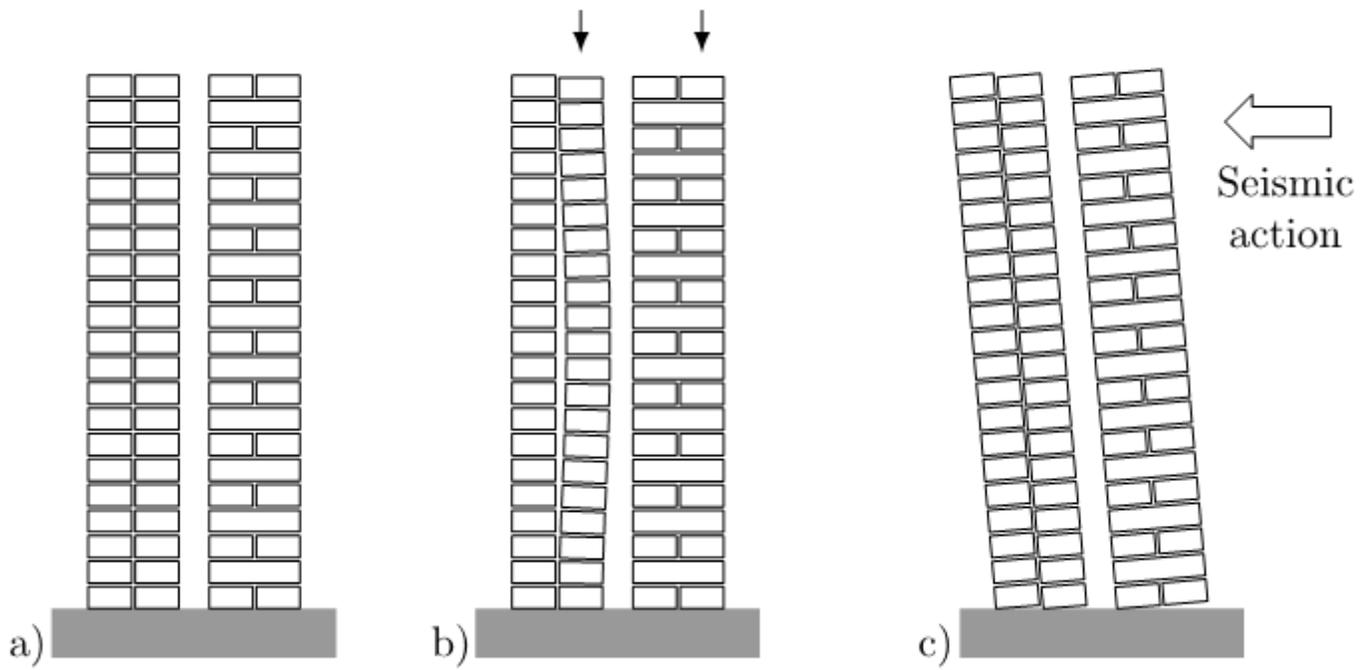
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# Figures



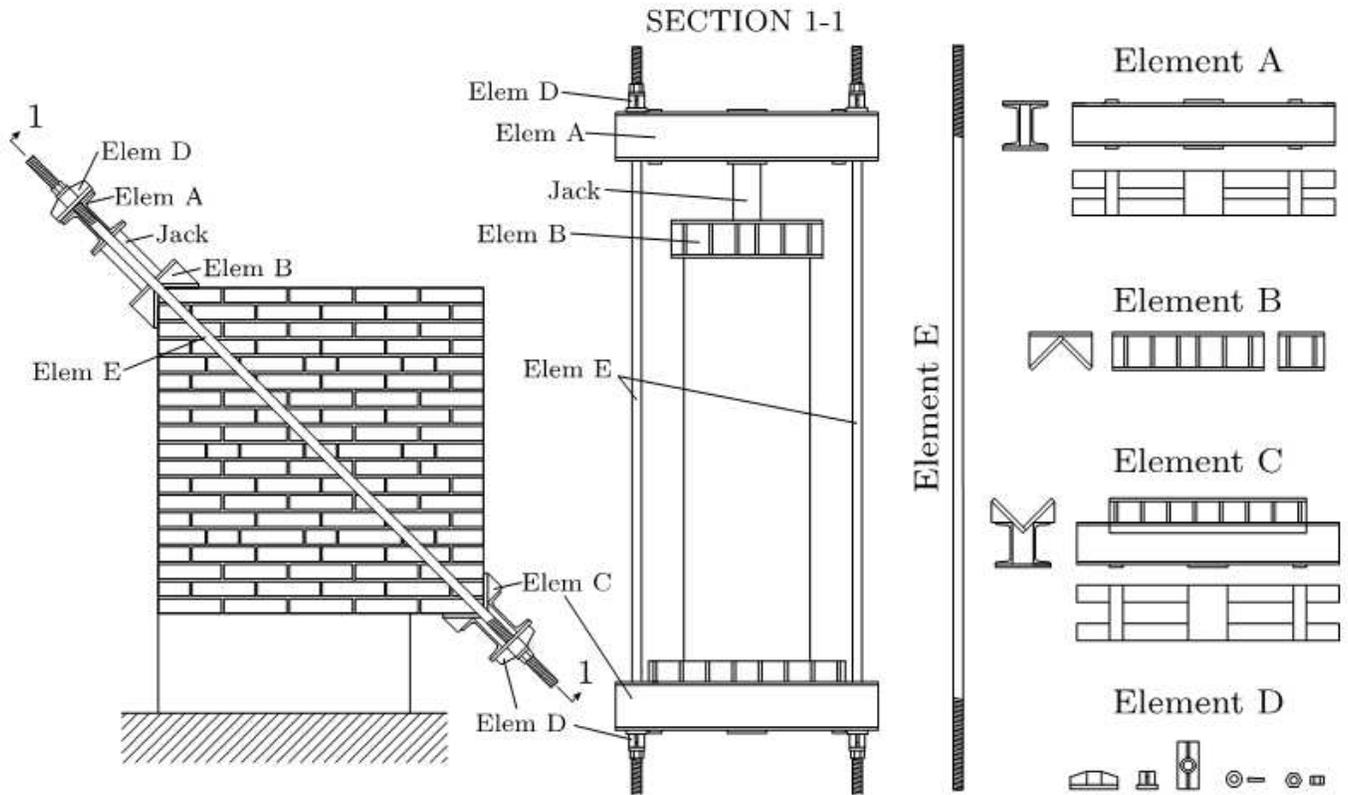
**Figure 1**

Damage due to separation of multi-leaf brickwork masonry wall after 2011 Emilia earthquake. Finale Emilia: a) Church of SS. Filippo e Giacomo; b) Modenesi's Tower.



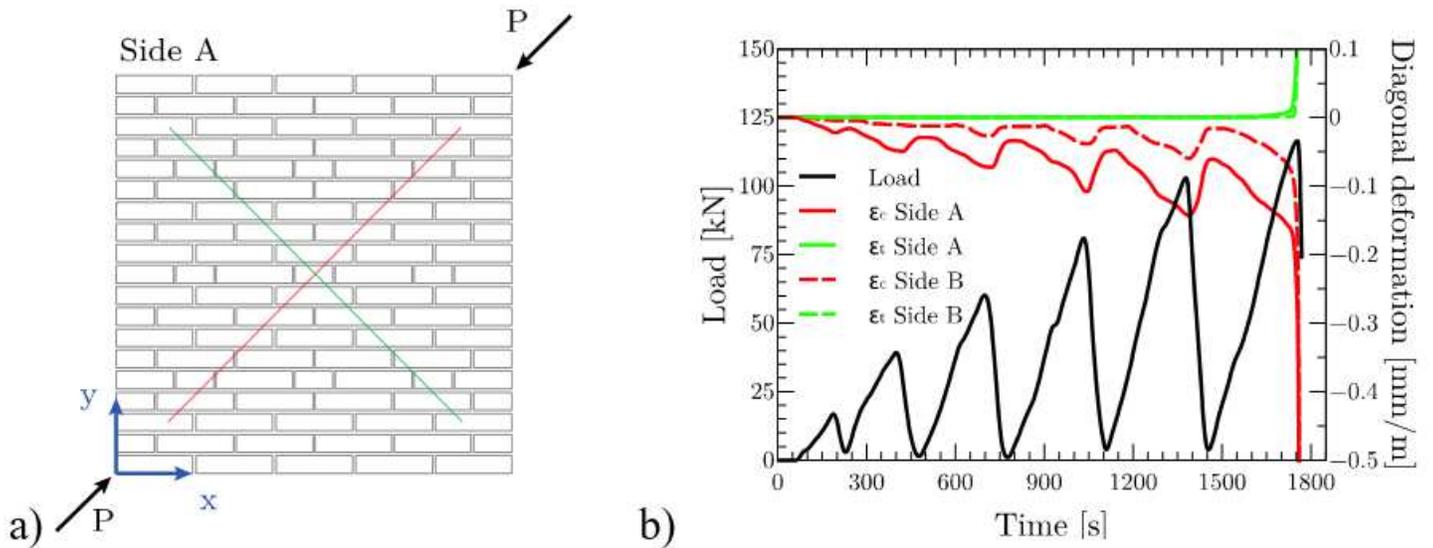
**Figure 2**

Two leaves walls under static and seismic action: a) transversal sections; b) instability due to vertical loads if no transversal connection exists; c) seismic behavior: simple overturning mechanism.



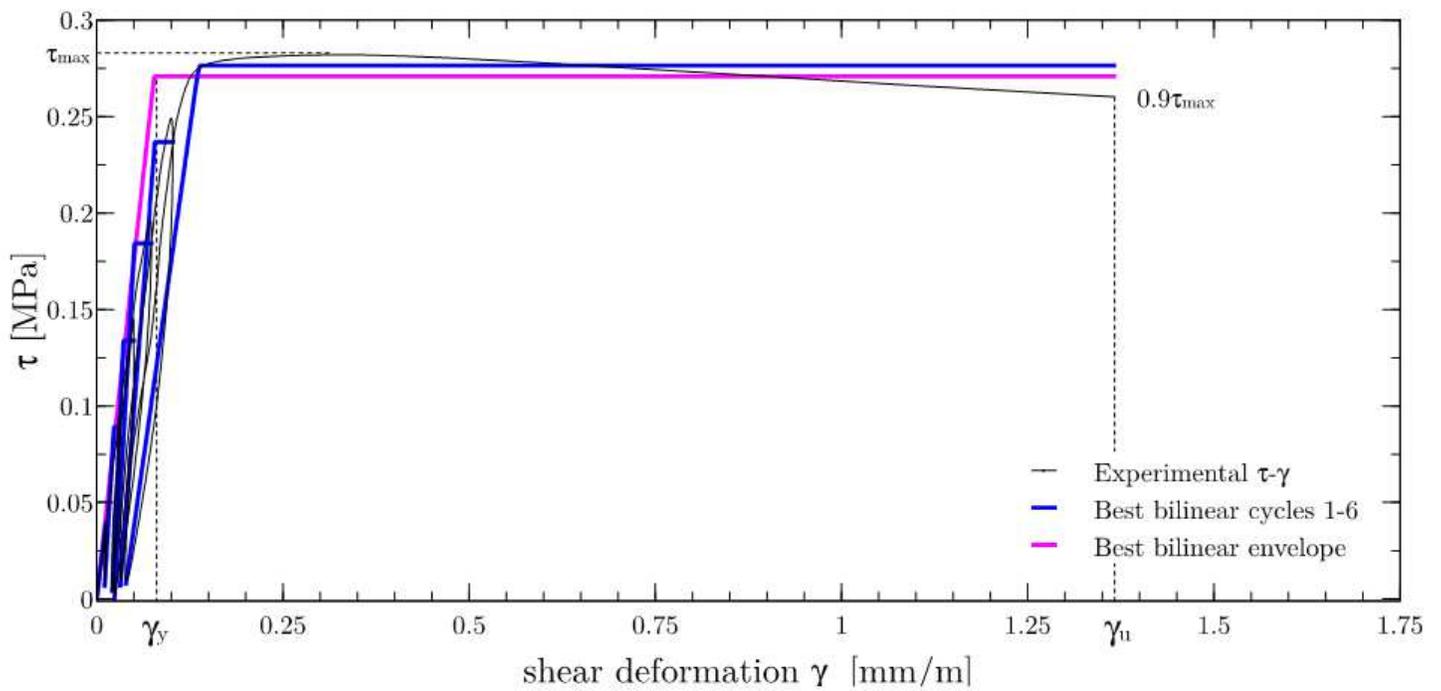
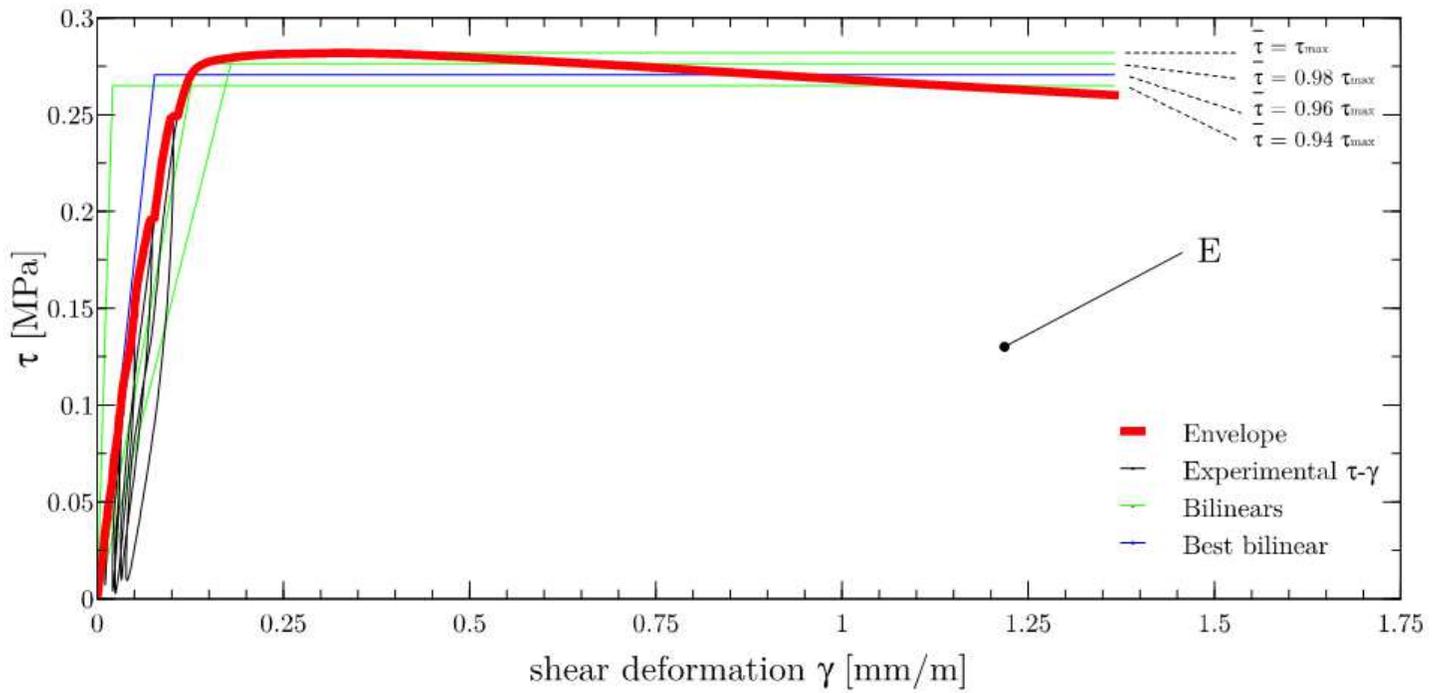
**Figure 3**

Loading scheme example with steel frame elements required for standard diagonal compression test.



**Figure 4**

Example of recorded data: different behavior between sides A and B of the panel.



**Figure 5**

Graphical representation of post-processing of recorded data.

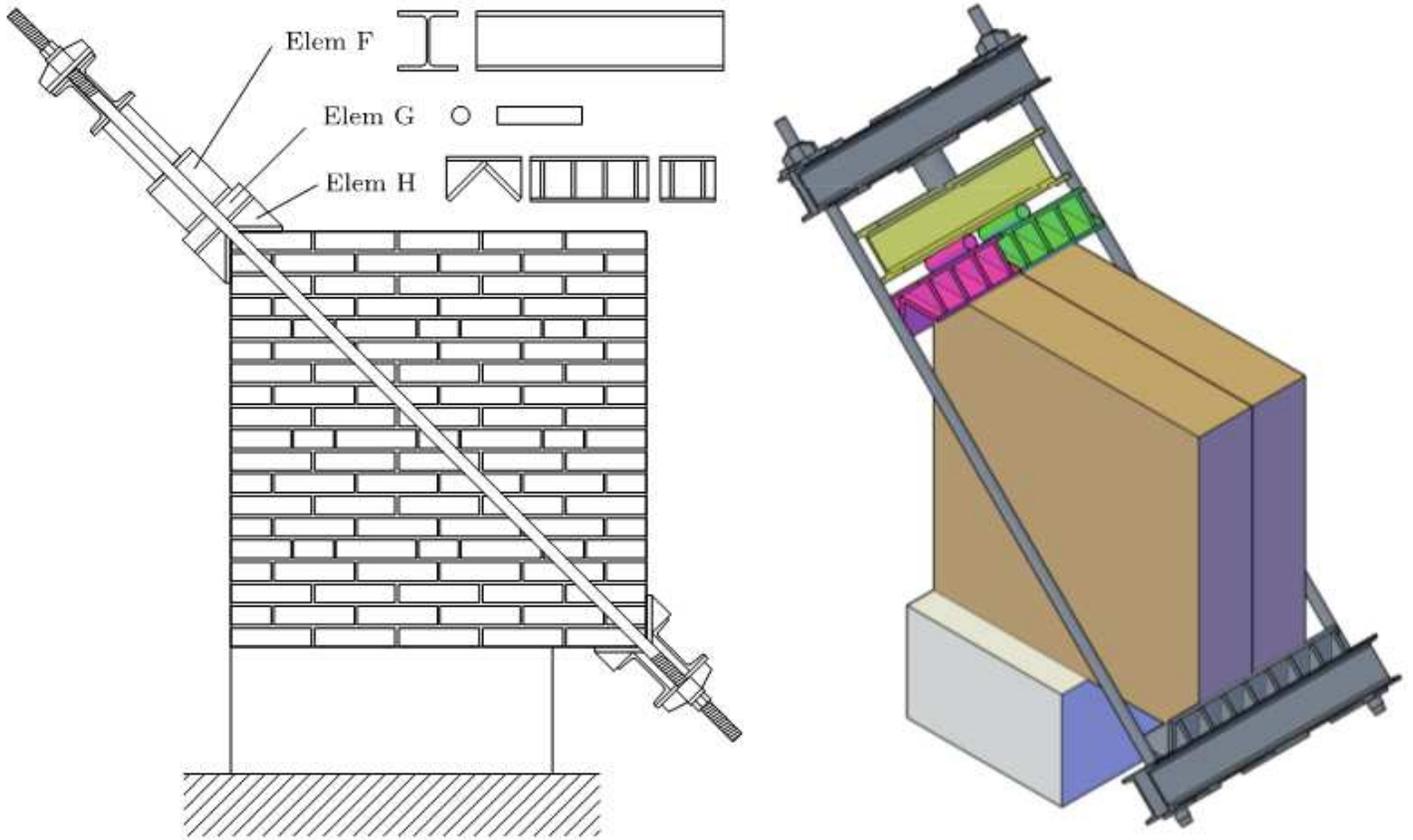


Figure 6

New set-up: new elements added or modified and a 3D sketch of the whole steel frame.

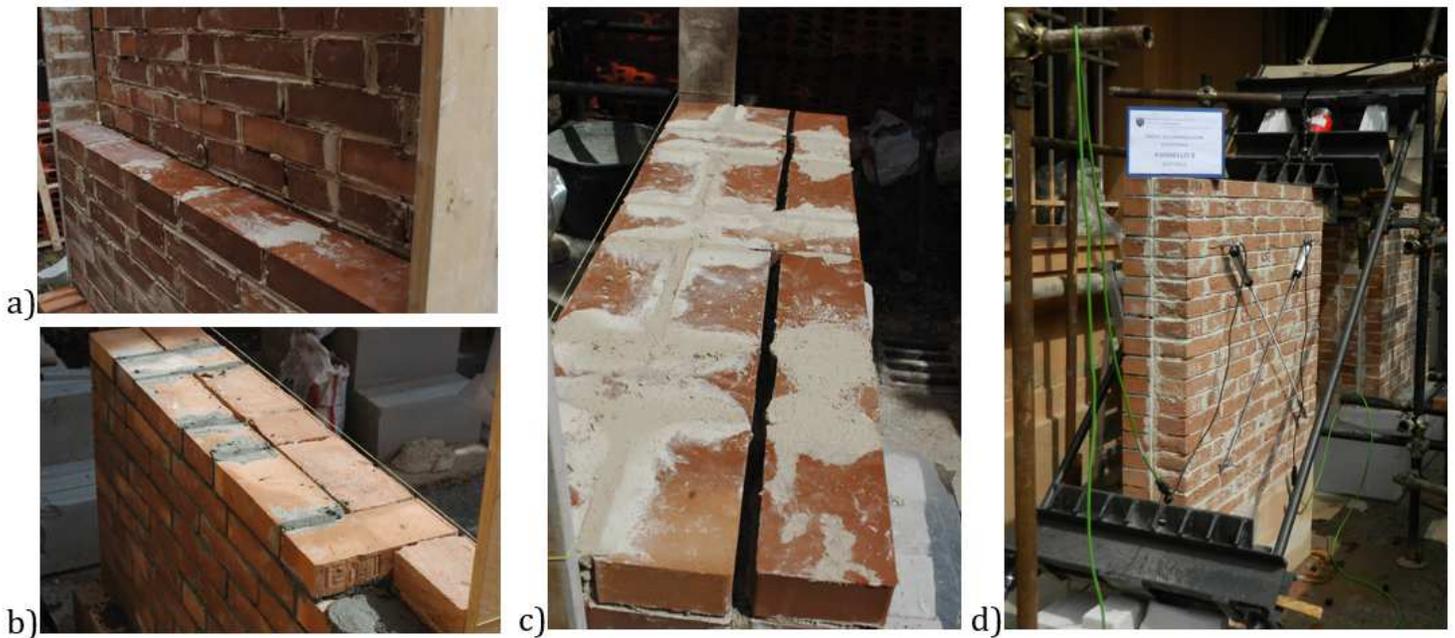
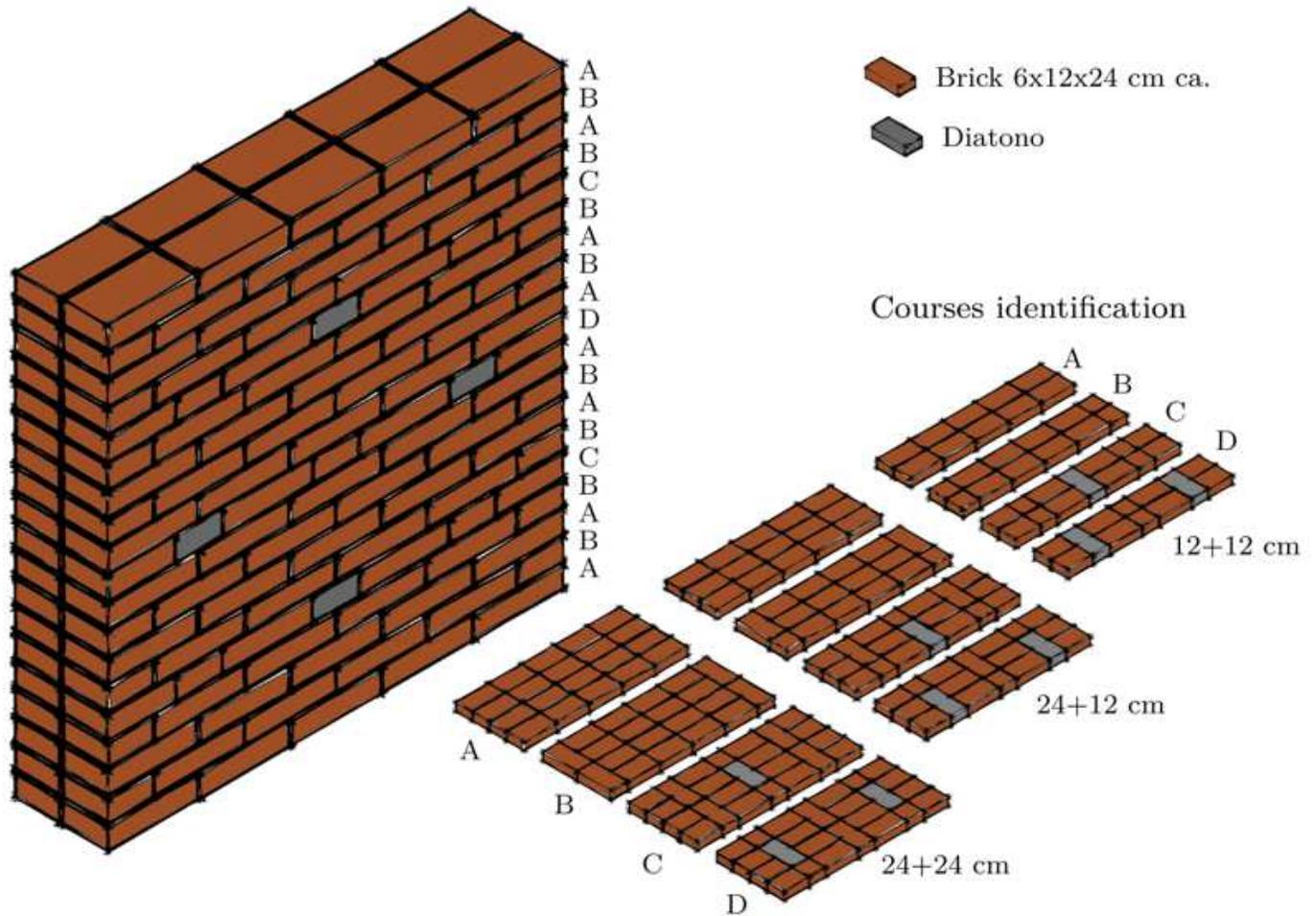


Figure 7

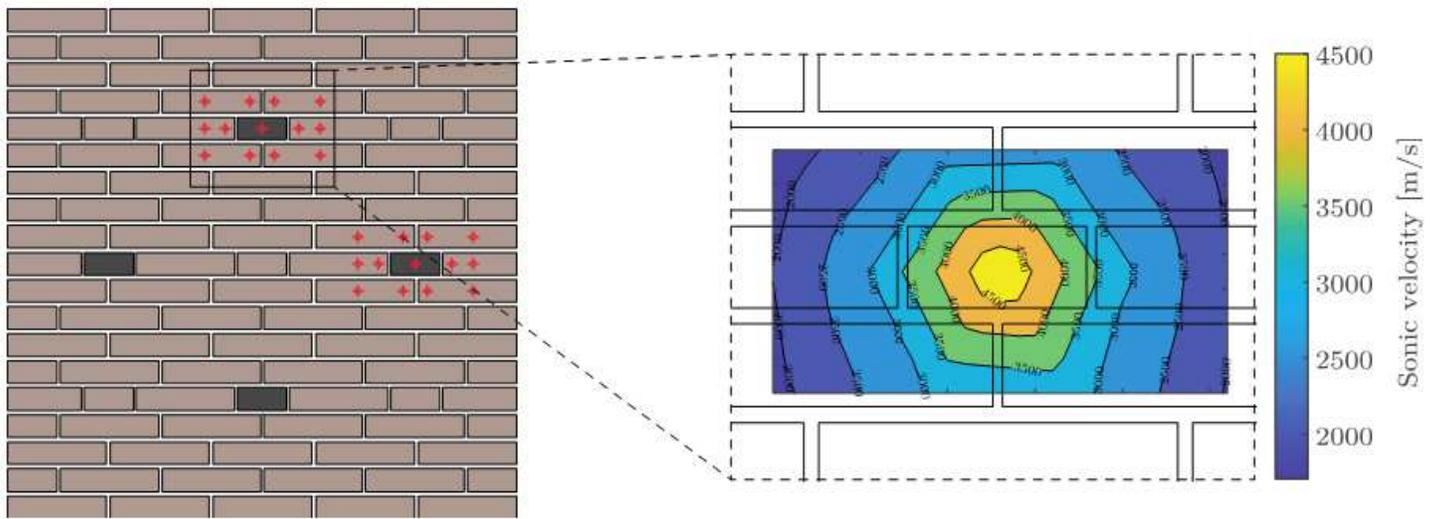
Some images of specimens and test setup: leaves construction in the case a) without diatoni (panel P5), and b) with diatoni (panel P8); c) transversal connection in non-uniform thickness leaves (panel P6) and d) just before the start of compression test on P8 element.



**Figure 8**

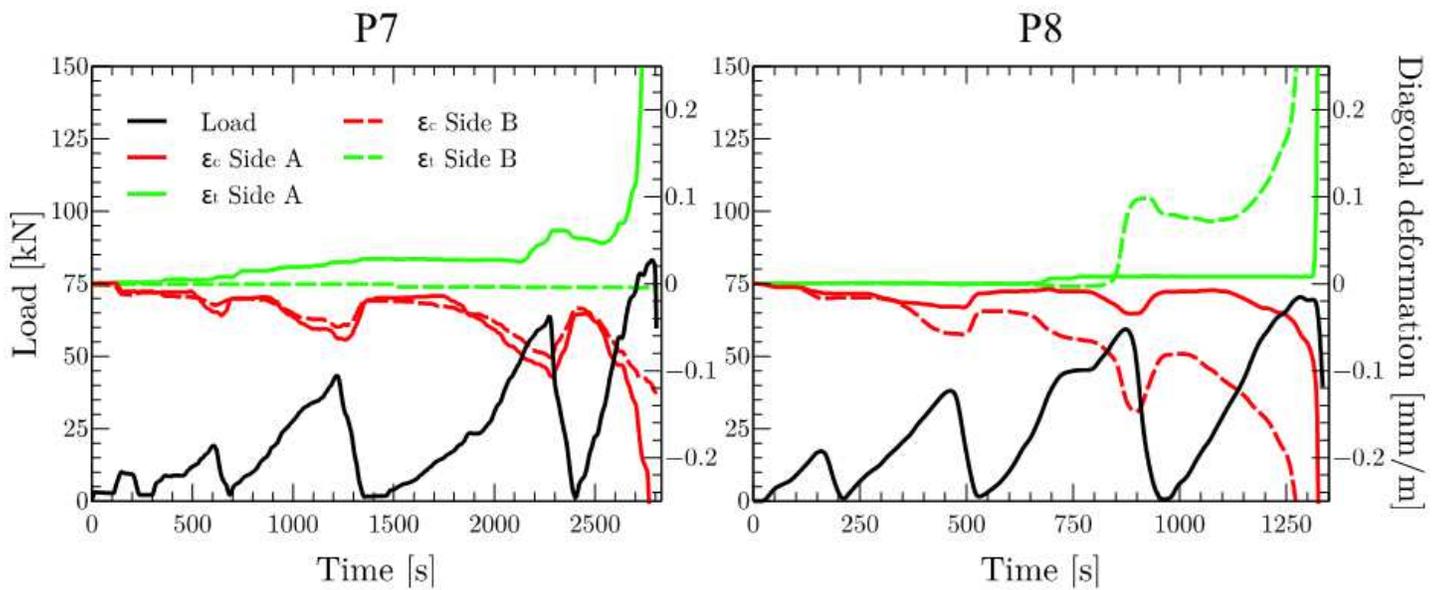
3D sketch of 24 cm panel and courses identification of double-leaf masonry walls with transversal connections.





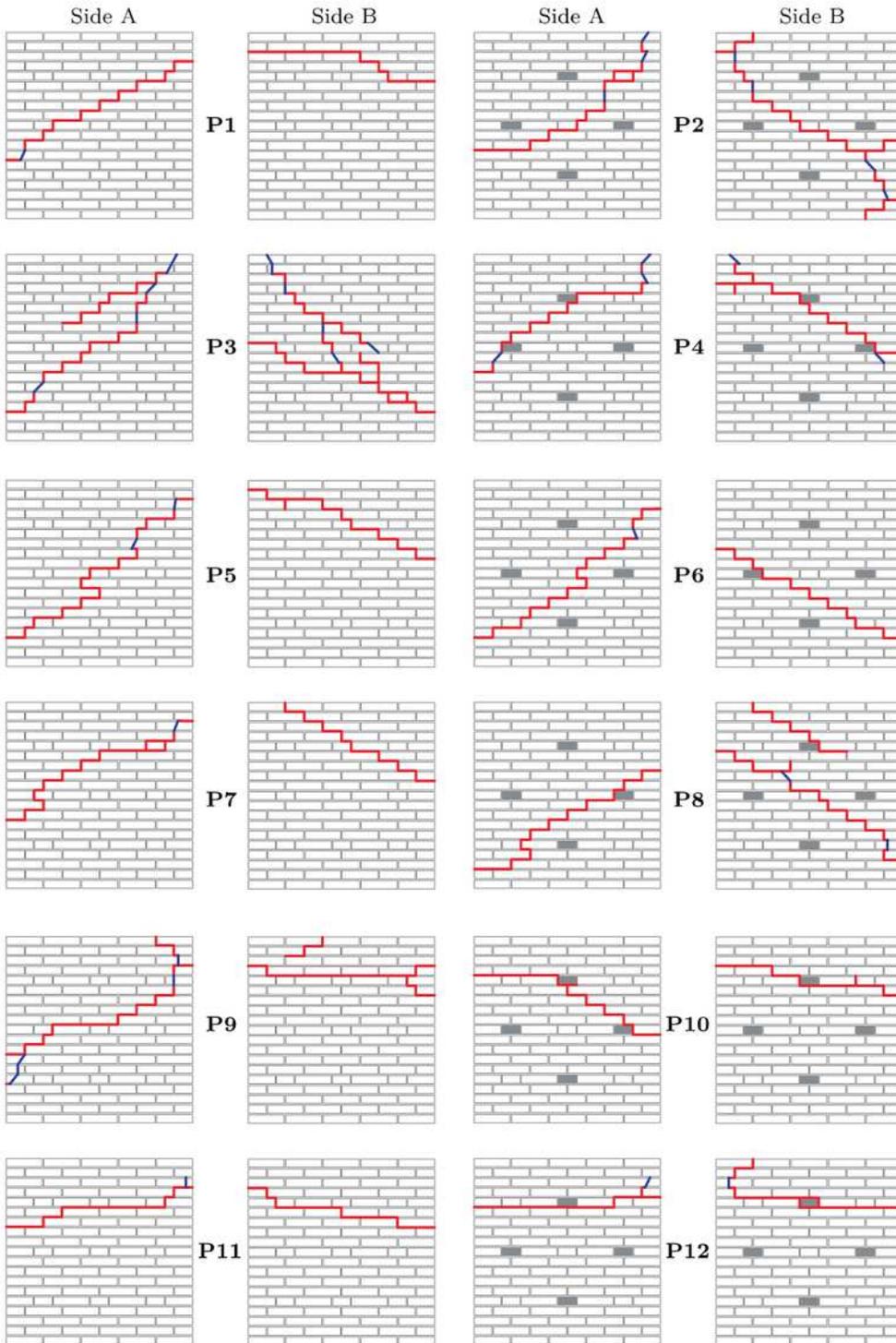
**Figure 10**

Sonic identification of diatoni. Test points (in red) and distribution of velocity around transversal element of P12 specimen.



**Figure 11**

Test results for P7 and P8 specimens. For clarity, the continuous lines is referred to the weaker leaf while the dashed line to the stronger one.



**Figure 12**

Damage pattern. The blue line indicates the crack of the brick.



**Figure 13**

Transversal elements after tests in P8 (left) and P6 (right) panels.