Methods and approaches for blind test predictions of out-of-plane behavior of masonry walls: A numerical comparative study

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Short Title: Blind test predictions

Abstract

Earthquakes cause severe damages to masonry structures due to inertial forces acting in the normal direction to the plane of the walls. The out-of-plane behaviour of masonry walls is complex and depends on several parameters, such as material and geometric properties of walls, connections between structural elements and the characteristics of the input motions, among others. Different analytical methods and advanced numerical modelling are usually used for evaluating the out-of-plane behaviour of masonry structures. Furthermore, different types of structural analysis can be adopted for this complex behaviour, such as limit analysis, pushover or non-linear dynamic analysis.

Aiming to evaluate the capabilities of different approaches to similar problems, blind predictions were made using different approaches. For this purpose, two idealized structures were tested on a shaking table and several experts on masonry structures were invited to present blind predictions on the response of the structures, aiming at evaluating the available tools for the out-of-plane assessment of masonry structures. This paper presents the results of the blind test predictions and the comparison with the experimental results, namely in terms of formed collapsed mechanisms and control outputs (PGA or maximum displacements), taking into account the selected tools to perform the analysis.

Keywords: Masonry, out-of-plane, seismic performance, numerical analysis, predictions.

1. Introduction

Natural hazards have caused a considerable number of disasters in the last decades. According to the World Bank, from 1975 to 2005 the number of natural disasters increased from approximately 100 to more than 400 (Parker et al., 2007). These events lead to important economic impacts (Noy, 2009), deaths and irrecoverable losses due to the collapse of existing masonry buildings. Consequently, earthquakes contribute significantly to these natural hazard disasters. It is predicted that in the current century the total fatalities caused by earthquakes will increase to about 2.57±0.64 million (Holzer and Savage, 2013). Recent seismic events caused severe damages to a considerable number of existing masonry constructions, such as the earthquakes in L'Aquila (Italy, 2009) (Augenti and Parisi, 2010; D'Ayala and Paganoni, 2011), in Canterbury (New Zealand, 2010 and 2011) (Leite et al., 2013; Moon et al., 2014), in Emilia (Italy, 2012) (Milani, 2013; Penna et al., 2014) and in City of Napa (USA, 2014) (Galloway and Ingham, 2015).

Existing masonry constructions present high seismic vulnerability, which is mainly related to the following aspects: (a) low tensile strength and ductility of masonry; (b) weak connections between orthogonal walls and between walls and horizontal diaphragms; (c) high mass of the masonry structural elements; (d) flexible horizontal diaphragms; (e) absence of seismic requirements at the time of their construction (Lagomarsino, 2006, Lourenço et al., 2011).

Regarding the out-of-plane behaviour of these structures, the low strength/mass ratio of common masonry structures increases their vulnerability in the out-of-plane direction because inertia forces are not restrained due to reduced stiffness and strength of the masonry walls in that direction (Ferreira et al., 2014).

For this reason, the seismic performance of masonry structures has received great attention in the last decade, mainly for masonry buildings without box-behaviour (Costa et al., 2013b; Lourenço et al., 2011; Shawa et al., 2012; Mendes et al., 2014). However, little consensus exists on the most appropriated assumptions and approaches for modelling unreinforced masonry buildings without box-behaviour, where the out-of-plane performance still needs further research (Ferreira et al. 2014).

Motivated by the previous aspects, approximately 25 world experts on masonry structures met in Guimarães (Portugal) prior to the 9th International Masonry Conference (9IMC, July 2014) for a one-day Workshop focused on the out-of-plane assessment of existing masonry buildings.

The experts were invited to present blind-predictions for two idealized one-story masonry structures, which were previously tested on a shaking table subjected to unidirectional ground motions (out-of-plane excitation). Thus, this paper presents the comparison between the seismic response obtained from the blind predictions as well as between the blind predictions and the experimental response. The comparison of seismic response was evaluated mainly in terms of collapse mechanisms and maximum load capacity.

2. Seismic analysis of masonry structures

Masonry is a heterogeneous material that consists of units and joints, and presents distinct directional properties, being the mortar joints, in the case of regularly dressed masonry, planes of weakness (Lourenço, 1996). The geometrical parameters, such as unit and joint dimensions or the arrangements of bed and head joints, increase the complexity of masonry structural analysis. According to Lourenço (2002), three main modelling approaches can be adopted for masonry, namely: (a) detailed micro-modelling, in which the units and mortar of joints are represented by continuum elements whereas the unit/mortar interface is represented by discontinuous elements; (b) simplified micro-modelling, in which the expanded units are represented by continuum elements whereas the behaviour of the mortar joints and unit/mortar interface is lumped in discontinuous elements; (c) macro-modelling, in which units, mortar and the unit/mortar interface are smeared out as a homogeneous continuum material. Besides the different modelling approaches in terms of material, several structural analysis techniques can be adopted for masonry structures, such as limit analysis, kinematic analysis with rigid macroblocks, pushover analysis and non-linear dynamic analysis with time integration. Furthermore, two main methods for advanced numerical modelling can be adopted, namely: (a) Finite Element Method (b) Discrete Element Method.

Since the inverted catenary principle, published by Robert Hooke in the 17^{th} century, to the development of graphical catenary-based methods by La Hire in the 18^{th} century, and Rankine and Moseley in the 19^{th} century, rational approaches started being considered. After classic limit analysis and the relation between limit analysis and thrust line stated by Kooharian in 1952 (Kooharian, 1952), several simplified but more sophisticated analysis methods were developed (Nielsen, 1999). For the assessment of arches two main approaches were developed (Kooharian, 1952; Heyman, 1969), which were later applied to other types of masonry elements, namely: (a) static approach, based on the principles of thrust lines; (b) kinematic approach, based on the analysis of failure mechanisms with rigid macroblocks. In general, these approaches correspond to simplified analyses assuming that the masonry has no tensile strength along the block interfaces, has infinite compressive strength and that sliding failure is not permitted. Different assumptions have been implemented for limit analysis with macroblocks, see for example (Gilbert et al., 2006). The use of graphic methods became outdated due to advances in computer technology (Lourenço, 2002) and the analysis of historical construction using the thrust line approach is difficult to solve, such that the kinematic approaches are more practical and effective. Models based on the rocking motion of monolithic walls, i.e. kinematic approach with macroblocks, allow good estimations of the collapse load factor and of the displacement capacity (Lagomarsino, 2015, Doherty et al., 2002; Orduña and Lourenço, 2005, Mendes, 2014). Non-linear dynamic analyses of rocking systems has been proposed as well for the interpretation of field and laboratory observations (Papantonopoulos et al., 2002; DeJong 2012; Sorrentino et al., 2014a-b).

Mechanisms can be proposed on the basis of the knowledge obtained from post-earthquake surveys of similar buildings, using the crack patterns obtained from experimental research and on the basis of practitioner experience. Thus, a bad evaluation of the possible mechanisms can lead to the non-consideration of the mechanism with the lowest load factor and, consequently, can lead to a failure load higher than the real maximum capacity of the structure (Mendes, 2014; Mauro et al., 2015).

The numerical models based on the Finite Element Method (FEM) allow several materials and types of elements (beam, shell, solid, etc.) to be easily combined. The non-linear seismic analysis of masonry buildings through FEM numerical models has been performed using discrete models (simplified micro-modelling approach) (Lourenço, 1996), continuous and anisotropic models (macro-modelling approach) (Lourenço et al., 1997; Lourenço, 2000) and, mainly, continuous and isotropic models (macro-modelling approach) (Mendes and Lourenço, 2014; Roca et al., 2013; Peña et al., 2010). The detailed micro-modelling approach has not often been used for masonry buildings, mainly due to the difficulty of mesh preparation using FEM software solutions and long-time consumed to run the non-linear analyses. FEM numerical models based on the macro-modelling approach present in general several simplifications, in terms of geometry and material properties, with respect to the real non-linear dynamic behaviour mainly for complex masonry buildings.

The Discrete Element Method (DEM) presents two types of formulation for masonry structures: (a) discrete models in which the blocks with polyhedral shapes can be assumed as rigid or deformable, and the discontinuities are treated as boundary conditions between blocks; (b) discrete models with spherical particles, which are not yet practical for larger structures. The DEM models allow realistic representations of complex structures (e.g. monuments composed by domes, vaults, arches and columns), including detailed block arrangements (Azevedo et al., 2000; de Felice and Giannini, 2001; Lemos et al., 2011) and typical dynamic rocking motion (Peña et al., 2007). DEM is also appropriate for modelling the out-of-plane collapse of multi-leaf masonry walls taking into account the real unit arrangement (de Felice, 2011). Although most FEM codes allow the development of models using the micro-modelling approach, only the general contact formulations implemented in DEM and combined FEM-DEM (Munjiza, 2004) allow the development of analyses in the large displacement range.

The recent work presented by Ferreira et al. (2014) presents a global overview of the state-of-art regarding the out-of-plane analysis of masonry structures and possible approaches.

As presented in the cited work, the assessment of the stability of masonry structures may be carried out by using three types of approaches, namely force-based (FBA), displacement-based (DBA) and energy-based (EBA or rigid body-based as named in Ferreira et al. (2014)) approaches. According these approaches the stability is evaluated comparing the demand and capacity of the structure in terms of maximum load capacity/strength (FBA), maximum displacement/deformation (DBA) and energy balance (EBA). For more information on seismic assessment of masonry structures, see Sorrentino et al. (2015) and Penna (2015), for the case of stone masonry buildings.

3. Shaking table tests

3.1 Description of experiments

A blind prediction challenge was carried out in which experts on masonry structures were invited to present their conjectures on the dynamic response of two idealized masonry structures tested on a shaking table and subjected to unidirectional ground motion. One structure was constructed of irregular stone and the other of clay-unit masonry with English bond (Figure 1 and Figure 2). The walls of the brick structure were built with perforated brick, and cement-based mortar, whereas the walls of the stone specimen were built with granite stone and lime-based mortar. The configuration of each structure included a single perforated unreinforced wall with a gable, and return walls on both ends. In each structure, an opening was placed in one of the returning walls, resulting in an asymmetry, and consequently, inducing torsional movements. The thickness of the walls was equal to 0.500 m and 0.235 m for the stone and brick structure, respectively.

Each structure was tested on the LNEC shaking table in Lisbon (Portugal). A unidirectional seismic action was applied perpendicular to the gable wall and by stages of increasing amplitude. As identified by Costa et al. (2013a), the characteristics of the ground motions may significantly influence the out-of-plane behavior of the specimen if near-field inputs are used. In the present case, the selection was made resorting to a near-source ground motions with significant acceleration input. For more information regarding the input see Campos Costa et al. (2015). Wallettes constructed with corresponding types of masonry were tested under vertical and diagonal compression to provide baseline values of Young´s modulus, tensile and compressive strength and the specific mass of masonry. These parameters were sent to the experts for consideration in their blind predictions.

3.2 Results of tests

Accelerations and displacements were measured at several locations across the height and length of the gable and return walls. Maximum relative displacements near the top of the gable walls are present in Figure 3 for each test and structure. For the penultimate test run of the stone structure (Figure 3a), the peak out-of-plane displacement was 25.4 mm, and occurred at the center of the gable wall (LVDT 2) and at the corner of the return wall with opening (LVDT 3). The peak relative displacement was significantly lower (12.4 mm) at the corner of the return wall without openings (LVDT 1). In the last test run, the stone structure presented a maximum displacement equal to 218.5 mm (center of the gable wall).

Concerning the brick structure (Figure 3b), in the penultimate test run the largest out-of-plane displacements were equal at the center of the gable wall and at the corner of the return wall with openings (5.4 mm). However, the out-of-plane was much less at the corner of the return wall without opening (0.7 mm), which is in agreement with the commonly expected response for asymmetrical twisting of a system. In the last test, the maximum out-of-plane displacement was about 136.5 mm. The opening in one of the return walls resulted in an asymmetric stiffness, and thus different response of the left and right corners. In the last test run, the maximum displacement increased significantly, particularly for the brick structure, due to severe damage and/or partial collapse of the gable and return walls.

Although full collapse of the stone structure did not occur despite peak ground accelerations as high as 1.07 g, the return wall with opening incurred severe damage due to in-plane shear and flexure. Stones at the right top corner fell out (Figure 4b and 4d). However, the return wall without openings incurred little damage. If the test structure was subjected to a further test of increasing amplitude, the gable wall would have likely fallen out and substantial damage to the return wall with opening would have occurred. Out-of-plane collapse of the gable wall would likely be a result of diagonal cracking of the left pier, and horizontal cracking at mid-height of the right pier. In the last seismic test of the brick test structure (PGA = 1.27 g) the tympanum of the gable wall fell outwards (Figure 5a) as is commonly observed in many earthquakes. Furthermore, the lintel and pier of the return wall collapsed due to in-plane behavior, and the corner rotated due to torsional effects. It is noted that Figure 4 and Figure 5 only present the moderate and severe damage caused by the last seismic tests. For more information on the shaking table tests see Campos Costa et al. (2015).

4. Expert predictions

The geometry of the structures, the material properties (specific mass, Young´s modulus, tensile and compressive strength), the normalized accelerogram envelopes of the seismic action applied at the base, and the corresponding response spectra were provided to the experts. No specific requirements were given to experts in terms of the types of computed results they needed to provide.

The experts presented several modelling approaches, type of structural analysis and assessment criteria for predicting the dynamic behaviour of the structures. It is noted that the predictions were made for either or both test structures depending on the expertise of the expert. Three modelling approaches were adopted:

- Modelling approach based on rigid blocks defined according to the expected collapse mechanisms. A total of 23 models with rigid blocks were prepared.
- Modelling approach based on the Finite Element Method (FEM). Seven FEM models were prepared using the macro-modelling approach and three FEM models were developed using simplified micro-modelling. One of the latter resorted to a combined FEM-DEM strategy.
- Modelling approach based on the Discrete Element Method (DEM). Three DEM models were prepared using rigid elements, for simulating portions of the masonry walls (not the units), and interface elements with Mohr-Coulomb law, for simulating the connection between the rigid elements.

Concerning the type of structural analysis, three techniques were used:

- Limit analysis based on the kinematic approach;
- Static non-linear analysis (pushover). In general a horizontal load distribution proportional to the mass was adopted. However, in some analyses a load proportional to the first mode shape was applied.
- Non-linear dynamic analysis with integration. The artificial accelerograms applied at the base of structures were generated by the experts, taking into account the normalized response spectra and the accelerogram envelope of the seismic action measured in the shaking table tests.

The collapse of structures was evaluated through force-based, displacement-based and energy-based criteria. Most experts adopted limit analysis with rigid blocks to predict the Peak Ground Acceleration (PGA) that caused collapse of the structure, taking into account the force-based and displacement-based criteria. The collapse mechanisms were defined based on the FEM and DEM (pushover analysis and mode shapes) and personal judgment.

Several tools of structural analysis were used, namely the 3DEC™, Abaqus Unified FEA™, ANSYS®, DIANA™, LS-DYNA®, Strand7® as well as tools developed by the experts for limit analysis. Figure 6 presents examples of models prepared by the experts using different modelling approaches and different tools of structural analysis.

The comparison of predictions was carried out mainly in terms of collapse mechanisms idealized from models proposed by the experts and respective PGAs. Furthermore, some experts used different assessment methods for the same collapse mechanism, which resulted in several PGAs for the same collapse mechanism. In the following sections the comparison of the blind predictions is presented.

4.1 Predictions for the stone structure

For the blind predictions of the stone structure, 13 different idealized collapse mechanisms were proposed by the experts (Figure 7). The proposed collapse mechanisms can be divided into the following sets:

- Partial collapse of the gable wall (Mechanisms 1-4): Out-of-plane overturning of the lintel of the door with inclined cracks from the top corners of the door to the top of the tympanum. Out-of-plane overturning of tympanum with horizontal cracks at the top of the door. Partial out-of-plane overturning of the gable wall with diagonal cracks from the top corners of the gable wall to the base of the door.

- Total collapse of the gable wall (Mechanism 5): Out-of-plane overturning of the gable wall with vertical cracks between orthogonal walls and without any collapse of the return walls.
- Total collapse of the gable wall and partial collapse of the return walls (Mechanisms 6 and 7): Out-of-plane overturning of the gable wall with partial in-plane collapse of one or both return walls.
- Collapse of the tympanum and partial collapse of the return walls (Mechanism 8): Out-of-plane overturning of tympanum with horizontal cracks and partial in-plane collapse of both return walls.
- Partial collapse of the gable and return walls (Mechanisms 9-13): Partial out-of-plane overturning of the gable wall, involving diagonal and horizontal cracks, and partial in-plane collapse of the return wall with opening.

The estimated PGA at collapse of the blind predictions for the stone structure ranged from 0.22 g to 2.50 g (Figure 8), with a wide variance between the predictions (COV=63%). The average of the estimated PGA for the stone structure was equal to 0.91 g. It is noted that the experimental PGA near the collapse was equal to 1.07 g. The large variance was mainly attributable to predicting the correct collapse mechanism. When considering only the good predictions in terms of collapse mechanisms, i.e. mechanisms similar to the damage observed in the shaking table tests (Mechanisms 9-13), the estimated PGA ranged from 0.53 g to 1.42 g and the coefficient of variation decreased to 31% (Figure 9). The average error between the experimental PGA and the PGA of the good predictions was equal to 28%, which corresponds to an acceptable error taking into account the complexity of the challenge. Most of the good predictions (67%) presented an estimated PGA of collapse lower than the experimental PGA. Within good predictions only two results were presented for the displacement of collapse at the top of the tympanum, namely 0.16 m and 0.25 m. According to the experimental results (Figure

3a) the displacement of collapse at this control point should be higher than 0.22 m (about half of the wall thickness).

The good predictions more close to the experimental results presented an estimated PGA equal to 1.11 g (Mechanism 11), which corresponds to an error of about 4%. This prediction corresponds to the collapse mechanism with lower error also in terms of displacement at collapse (0.25 m). The collapse mechanism proposed by this prediction was based on the damage obtained from a pushover analysis carried out with a FEM model (simplified micro-modeling approach). Its response was based on two points, namely the point of maximum load capacity and the point of collapse. The point of maximum load capacity was estimated through limit analysis (kinematic approach) and linear stiffness of the FEM model. The limit analysis was carried out by applying the principle of virtual work. For computing the internal work a flexural tensile strength of masonry parallel and orthogonal to the bed-joints equal to 0.10 MPa and 0.20 MPa was adopted, respectively. Furthermore, the tensile strength of the masonry obtained from the diagonal compression tests (0.22 MPa) was used for the cracks at the return wall with opening. Dynamic effects were also taken into account through the modal properties of the 1st mode of vibration of the structure and respective linear spectral acceleration. Finally, the point at collapse was defined based on the equilibrium for vertical loads, where it was assumed that displacement at collapse is equal to half the wall thickness.

4.2 Predictions for the brick structure

The collapse mechanisms predicted by the experts for the brick structure can be organized into the following types of mechanisms (Figure 10):

- Partial collapse of the gable wall (Mechanisms 1-4): Out-of-plane overturning of the lintel of the door with diagonal cracks from the top corners of the door to the top of the tympanum or to the top corners of the gable wall. Out-of-plane overturning of the tympanum with a horizontal crack. Partial out-of-plane overturning of the gable wall with diagonal cracks from the top corners of the gable wall to the base of the window.

- Collapse of the gable wall (Mechanisms 5 and 6): Total or partial out-of-plane overturning of the gable wall with vertical cracks between orthogonal walls and without any collapse of the return walls.
- Partial collapse of the gable wall and returns walls (Mechanisms 7 and 8): Partial out-of-plane overturning of the gable wall, involving diagonal cracks, and partial in-plane collapse of the return wall with opening. Partial out-of-plane overturning of the gable wall, involving a vertical crack at the connection between the gable wall and the return wall without openings, and partial in-plane collapse of the return wall with opening.

The experts presented 17 predictions for the brick structure. The estimated PGA at collapse for the brick structure (Figure 11) ranged from 0.30 g to 1.00 g (COV=39%), which means that the all predictions were lower than the experimental result (1.27 g). The average PGA of the predictions for the brick structure was equal to 0.64 g.

The blind predictions for the brick structure presented greater difficulties, which can be related to the slenderness of the structure and to the torsional effects clearly observed during the shaking table tests. As a consequence, only fair predictions in terms of collapse mechanism were obtained. The collapse mechanisms considered as fair predictions are related to the damage due to the out-of-plane behaviour observed at the tympanum and to the damage caused by in-plane behaviour at the return wall with opening (Mechanisms 2 and 7). The average error of the PGA for these predictions was equal to 63% and the minimum error was equal to 21% (Figure 12). The collapse displacement at the top of the tympanum for the fair predictions ranged from 0.12 m to 0.31 m, which according to the experimental results should be less than or equal to 0.14 m (Figure 3b).

5. Conclusions

The assessment of the out-of-plane behavior of masonry structures is still a challenge. Thus, two idealized masonry structures were built and tested on a shaking table, aiming to obtain the out-of-plane dynamic response of the structures. One structure was constructed of clay-unit masonry with English bond and the other of irregular stone. Several experts on masonry structures presented blind predictions on the response of the structures. The blind predictions were evaluated and compared with respect to the experimental results obtained from the shaking table tests.

Several types of analysis, numerical modelling and approaches for assessment were adopted by the experts. In general, the predictions were carried out using limit analysis based on the kinematic approach, and the assessment of the collapse was based on the force-based and displacement-based approaches. The collapse mechanisms were defined through numerical models, based on the Finite and Discrete Element Methods, and personal judgment.

Good predictions were obtained for the stone structure, either in terms of collapse mechanism or PGA at collapse. For the predictions assumed as good, the average and the minimum error of PGA at collapse was equal to 28% and 4%, respectively. The prediction having the least error was performed by using limit analysis based on the kinematic approach and the collapse mechanism was defined based on the pushover analysis with a FEM model.

In the blind predictions of the brick structure only fair results were obtained, which can be related to the difficulty of predicting the correct collapse mechanism taking into account the torsional effects. Furthermore, the PGA at collapse for the fair predictions presented a high average error (63%). The minimum error was equal to 21%.

Finally, it is concluded that most of the predictions present a PGA at collapse lower than the respective PGA obtained from the shaking table tests, i.e. both test structures resisted higher intensities before collapse than estimated by nearly all predictions. However, more efforts on the out-of-plane behaviour of masonry structures should be conducted, aiming at improving knowledge on this type of behaviour and, consequently, presenting methodologies that allow results to be obtained that more closely match the real behaviour of masonry structures. Moreover, from the obtained collapse mechanisms, several were estimated based on expert judgement, which means that further research is needed within this topic.

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References

Augenti, N. and Parisi, F. 2010. Learning from Construction Failures due to the 2009 L'Aquila, Italy, Earthquake. Journal of Performance of Constructed Facilities, 24:6, 536–555. doi: http://dx.doi.org/10.1061/(ASCE)CF.1943-5509.0000122

Azevedo, J., Sincraian, G., Lemos, J.V. 2000. Seismic Behavior of Blocky Masonry Structures. Earthquake Spectra, 16:2, 337–365. doi: http://dx.doi.org/10.1193/1.1586116

Campos Costa et al. (2015). Add reference of the Paper 2.

Costa, A.A., Arêde, A., Campos Costa, A., Penna, A. and Costa, A. 2013a. Out-of-plane behaviour of a full scale stone masonry façade. Part 1: specimen and ground motion selection. Earthquake Engineering and Structural Dynamics, 42:14, 2081-2095. doi: 10.1002/eqe.2313

Costa, A.A., Arêde, A., Campos Costa, A., Penna, A. and Costa, A. 2013b. Out-of-plane behaviour of a full scale stone masonry façade. Part 2: shaking table tests. Earthquake Engineering & Structural Dynamics, 42:14, 2097–2111. doi: 10.1002/eqe.2314

D'Ayala, D.F., Paganoni, S. 2011. Assessment and analysis of damage in L'Aquila historic city centre after 6th April 2009. Bulletin of Earthquake Engineering, 9(1):81-104. doi: 10.1007/s10518-010-9224-4

de Felice, G. 2011. Out-of-Plane Seismic Capacity of Masonry Depending on Wall Section Morphology. International Journal of Architectural Heritage, 5:4-5, 466–482. doi: 10.1080/15583058.2010.530339

de Felice, G., Giannini, R. 2001. Out-of-plane seismic resistance of masonry walls, Journal of Earthquake Engineering, 5(2): 253-271. doi: 10.1080/13632460109350394

DeJong, M. J. 2012. Seismic response of stone masonry spires: Analytical modeling. Engineering Structures, 40:556-565. doi: 10.1016/j.engstruct.2012.03.010.

Doherty, K., Griffith, M. C., Lam, N., and Wilson, J. 2002. Displacement-based seismic analysis for out-of-plane bending of unreinforced masonry walls. Earthquake Engineering & Structural Dynamics, 31(4):833–850. doi: 10.1002/eqe.126

Ferreira T.M., Costa A.A. and Costa A. 2014. Analysis of the out-of-plane seismic behaviour of unreinforced masonry: A literature review. International Journal of Architectural Heritage, doi: 10.1080/15583058.2014.885996

Galloway, B., Ingham, J.M. 2015. The 2014 South Napa earthquake and its relevance for New Zealand. Bulletin of the New Zealand Society for Earthquake Engineering, 28(1).

Gilbert, M., Casapulla, C., Ahmed, H.M. 2006. Limit analysis of masonry block structures with non-associative frictional joints using linear programming. Computers & Structures, 84:13-14), 873–887. doi: 10.1016/j.compstruc.2006.02.005

Heyman, J. 1969. The safety of masonry arches. International Journal of Mechanical Sciences. 11(4), 363–385. doi: 10.1016/0020-7403(69)90070-8

Holzer, T.L. and Savage, J.C. 2013. Global Earthquake Fatalities and Population. Earthquake Spectra, 29:1, 155–175. doi: http://dx.doi.org/10.1193/1.4000106

Kooharian, A. 1952. Limit analysis of voussoir (segmental) and concrete arches. Journal of the American Concrete Institute, 24:4, 317–328. doi: 10.14359/11822

Lagomarsino, S. 2006. On the vulnerability assessment of monumental buildings. Bulletin of Earthquake Engineering, 4:4, 445–463. doi: 10.1007/s10518-006-9025-y

Lagomarsino, S. 2015. Seismic assessment of rocking masonry structures. Bulletin of Earthquake Engineering, 13:1, 97-128. doi: 10.1007/s10518-014-9609-x

Leite, J., Lourenço, P.B. and Ingham, J.M. 2013. Statistical Assessment of Damage to Churches Affected by the 2010–2011 Canterbury (New Zealand) Earthquake Sequence. Journal of Earthquake Engineering, 17:1, 73–97. doi: 10.1080/13632469.2012.713562

Lemos, J.V., Campos Costa, A., Bretas, E.M. 2011. Assessment of the seismic capacity of stone masonry walls with block models, Computational Methods in Earthquake Engineering, M. Papadrakakis, M. Fragiadakis, N.D. Lagaros (eds.), Springer, 221-235, 2011. ISBN 978-94-007-0053-6

Lourenço, P.B., 1996. Computational strategies for masonry structures. PhD Thesis. Delft University of Technology, Delft, The Netherlands.

Lourenço, P.B., 2000. Anisotropic Softening Model for Masonry Plates and Shells. Journal of Structural Engineering, 126:9, 1008–1016. doi: 10.1061/(ASCE)0733-9445(2000)126:9(1008)

Lourenço, P.B., 2002. Computations on historic masonry structures. Progress in Structural Engineering and Materials, 4:3, 301–319. doi: 10.1002/pse.120

Lourenço, P.B., De Borst, R., Rots, J.G. 1997. A plane stress softening plasticity model for orthotropic materials. International Journal for Numerical Methods in Engineering, 40:21, 4033–4057.

Lourenço P.B., Mendes N., Ramos L.F. and Oliveira, D.V. 2011. Analysis of masonry structures without box behaviour. International Journal of Architectural Heritage, 5, 369-382. doi: 10.1080/15583058.2010.528824

Mauro, A., de Felice, G., DeJong, M. 2015. The relative dynamic resilience of masonry collapse mechanisms, Engineering Structures, 85: 182-194. doi:10.1016/j.engstruct.2014.11.021

Mendes N. 2014. Masonry Macro-block Analysis. Encyclopedia of Earthquake Engineering, Springer-Verlag Berlin Heidelberg 2014. Editors: Michael Beer, Ioannis A. Kougioumtzoglou, Edoardo Patelli, Ivan Siu-Kui Au. doi: 10.1007/978-3-642-36197-5_154-1. ISBN: 978-3-642-36197-5 (Online)

Mendes, N., Lourenço, P.B. 2014. Sensitivity analysis of the seismic performance of existing masonry buildings. Engineering Structures, 80:1, 137–146. doi: 10.1016/j.engstruct.2014.09.005

Mendes, N., Lourenço, P. B. and Campos-Costa, A. 2014. Shaking table testing of an existing masonry building: assessment and improvement of the seismic performance. Earthquake Engineering & Structural Dynamics, 43:2, 247–266. doi: 10.1002/eqe.2342

Milani, G. 2013. Lesson learned after the Emilia-Romagna, Italy, 20–29 May 2012 earthquakes: A limit analysis insight on three masonry churches. Engineering Failure Analysis, 34, 761–778. doi: 10.1016/j.engfailanal.2013.01.001

Nielsen, M. 1999. Limit Analysis and Concrete Plasticity. CRC Press LLC. Second Edition. ISBN 9780849391262

Noy, I. 2009. The macroeconomic consequences of disasters. Journal of Development Economics, 88:2, 221–231. doi:10.1016/j.jdeveco.2008.02.005

Moon, L., Dizhur, D., Senaldi, I., Derakhshan, H., Griffith, M., Magenes, G., Ingham, J. 2014. The demise of the URM building stock in Christchurch during the 2010-2011 Canterbury earthquake sequence. Earthquake Spectra, 30(1):253-276. doi: 10.1193/022113EQS044M

Munjiza, A. 2004. The combined finite-discrete element method. Chichester: John Wiley and Sons. ISBN 0470841990

Orduña, A., Lourenço, P.B. 2005. Three-dimensional limit analysis of rigid blocks assemblages. Part I: Torsion failure on frictional interfaces and limit analysis formulation. International Journal of Solids and Structures, 42:18-19, 5140–5160. doi: 10.1016/j.ijsolstr.2005.02.010

Parker, R., Little, K. and Heuser, S. 2007. Development actions and the rising incidence of disasters. IEG evaluation brief, no. 4. Washington, DC: World Bank. Available at: http://documents.worldbank.org/curated/en/2007/06/11918676/development-actions-rising-incidence-disasters

Peña F., Lourenço P.B., Mendes N. and Oliveira D.V. 2010. Numerical models for the seismic assessment of an old masonry tower. Engineering Structures, 32:5, 1466-1478. doi: 10.1016/j.engstruct.2010.01.027.

Peña, F., Prieto, F., Lourenço, P.B., Campos Costa, A., Lemos, J.V. 2007. On the Dynamics of Rocking Motion of Single Rigid-Block Structures, Earthquake Engineering & Structural Dynamics, 36:15, 2383-2399. doi: 10.1002/eqe.739

Penna, A. 2015. Seismic assessment of existing and strengthened stone-masonry buildings: critical issues and possible strategies. Bulletin of Earthquake Engineering, 13(4): 1051-1071. doi: 10.1007/s10518-014-9659-0

Penna, A., Morandi, P., Rota, M., Manzini, C. F., da Porto, F., and Magenes, G. 2014. Performance of masonry buildings during the Emilia 2012 earthquake. Bulletin of Earthquake Engineering, 12(5): 2255-2273. doi: 10.1007/s10518-013-9496-6

Papantonopoulos, C., Psycharis, I. N., Papastamatiou, D. Y., Lemos, J. V., and Mouzakis, H. P. 2002. Numerical prediction of the earthquake response of classical columns using the distinct element method. Earthquake Engineering & Structural Dynamics, 31(9):1699-1717. doi: 10.1002/eqe.185

Roca, P., Cervera, M., Pelà, L., Clemente, R., Chiumenti, M. 2013. Continuum FE models for the analysis of Mallorca Cathedral. Engineering Structures, 46, 653–670. doi: 10.1016/j.engstruct.2012.08.005

Shawa, O. A., de Felice, G., Mauro, A., and Sorrentino, L. 2012. Out-of-plane seismic behaviour of rocking masonry walls. Earthquake Engineering & Structural Dynamics, 41(5):949-968. doi: 10.1002/eqe.1168

Sorrentino et al. (2015). Add reference of the Paper 1.

Sorrentino, L., Liberatore, L., Decanini, L.D. and Liberatore, D. 2014a. The performance of churches in the 2012 Emilia earthquakes. Bulletin of Earthquake Engineering, 12(5):2299-2331. DOI: 10.1007/s10518-013-9519-3.

Sorrentino, L., Liberatore, L., Liberatore, D., and Masiani, R. 2014b. The behaviour of vernacular buildings in the 2012 Emilia earthquakes. Bulletin of Earthquake Engineering, 12(5):2367-2382. doi: 10.1007/s10518-013-9455-2

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(a) (b)

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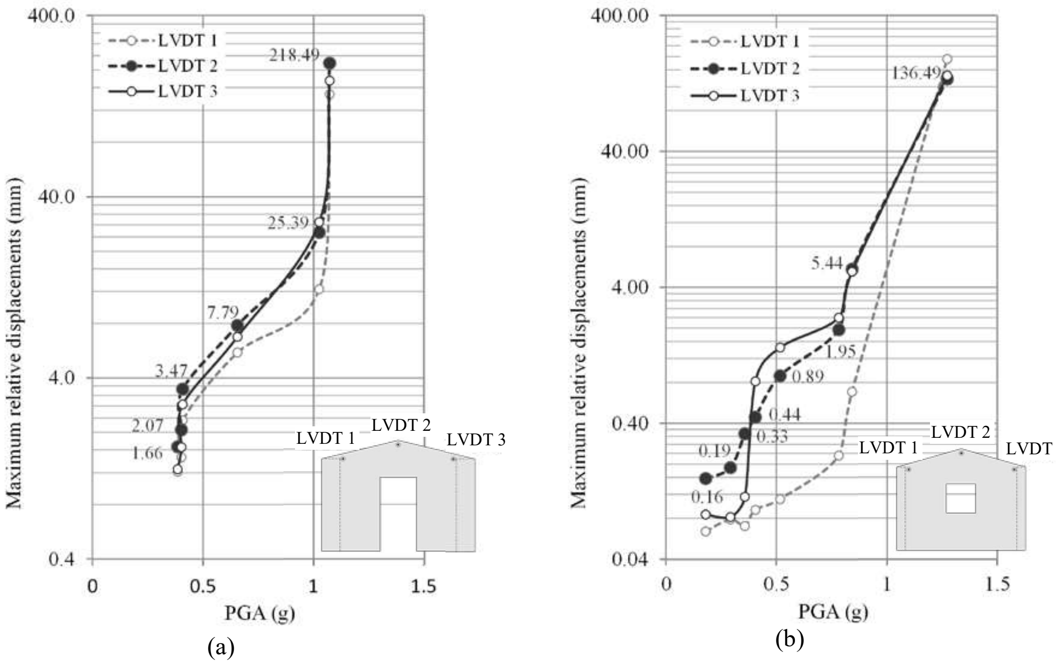
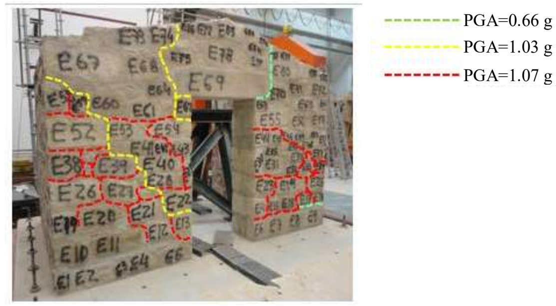
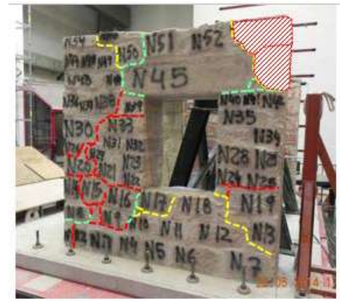


Figure 3 – Maximum relative displacement: (a) stone structure; (b) brick structure.



(a)

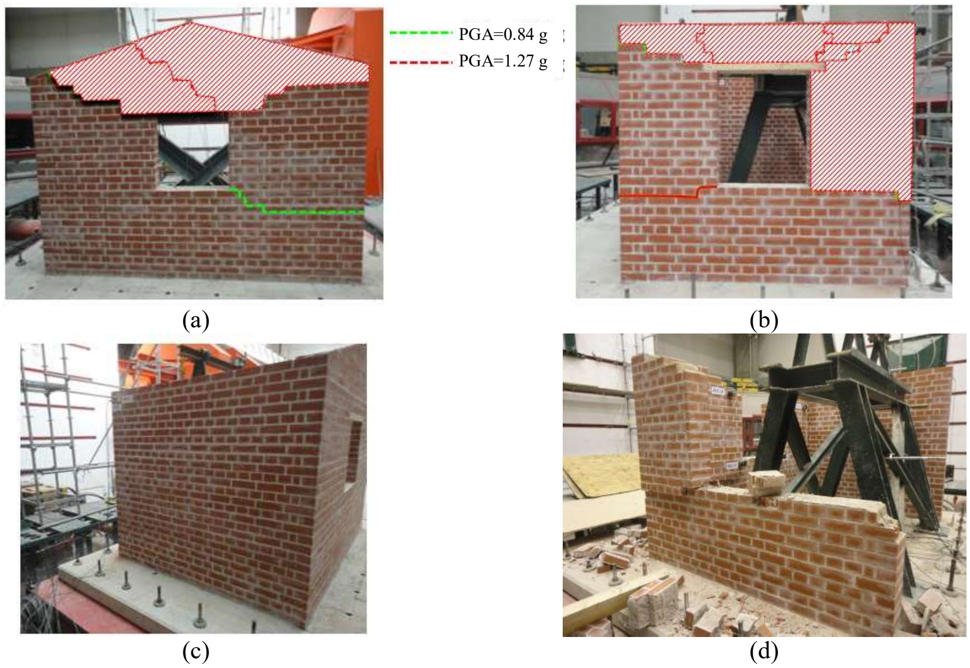


(b)

(c)

(d)

Figure 4 – Crack pattern of the stone structure: (a) gable wall; (b) return wall with opening; (c) return wall without openings; (d) general view of the damage.



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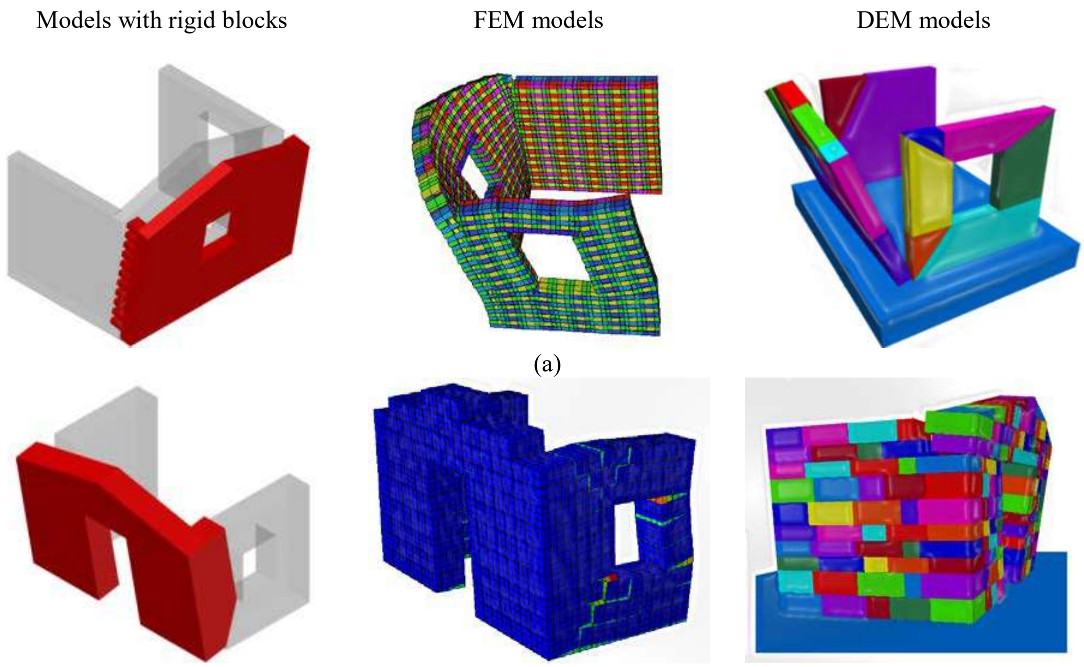

(b)

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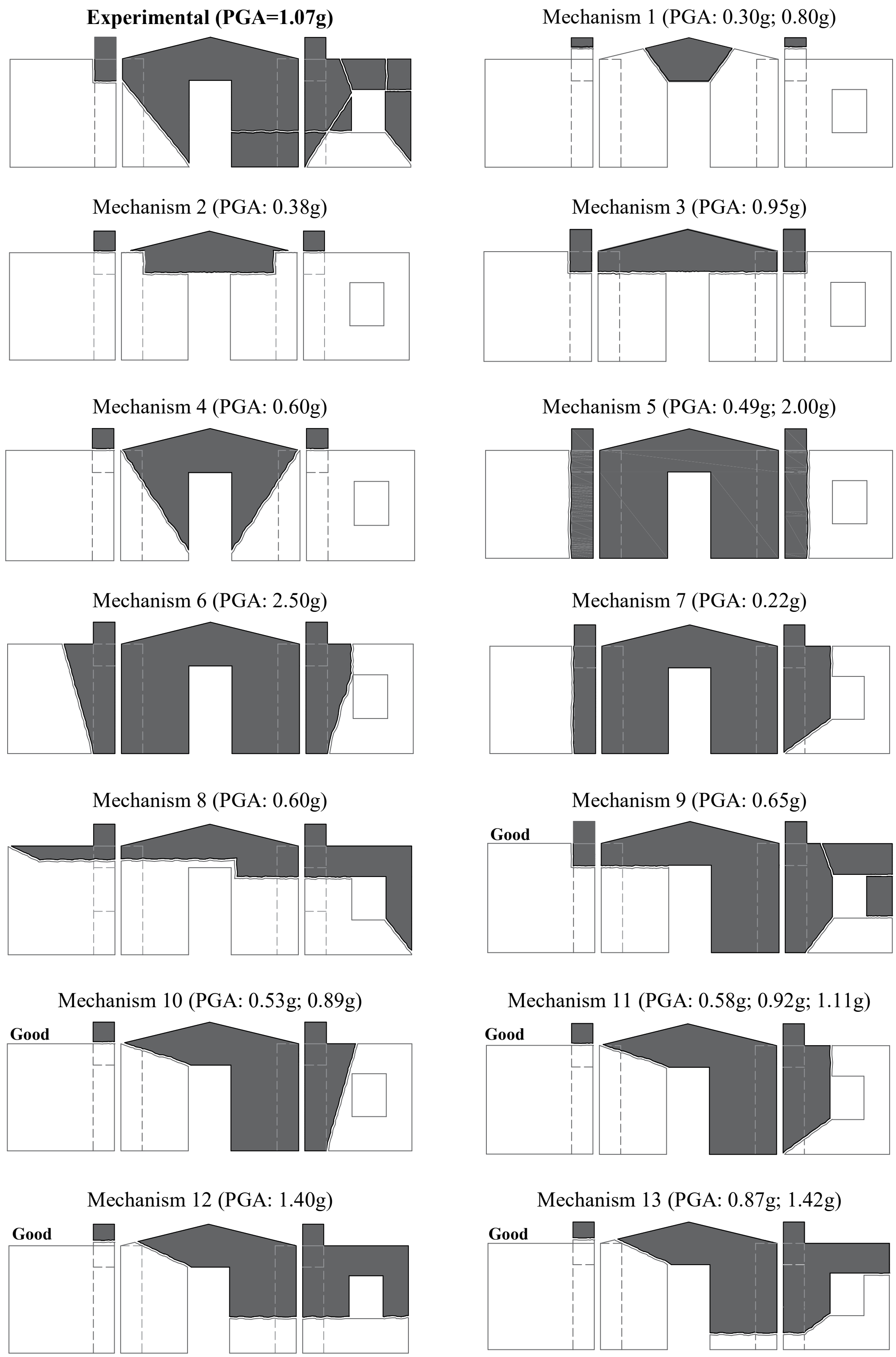

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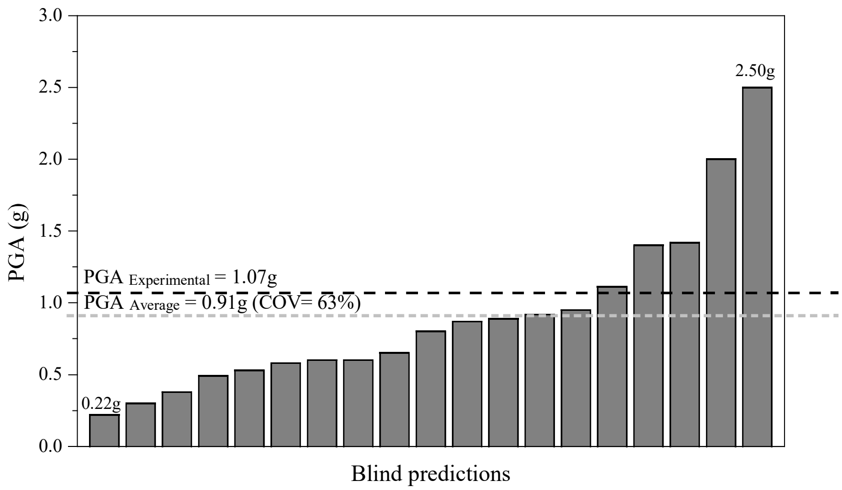
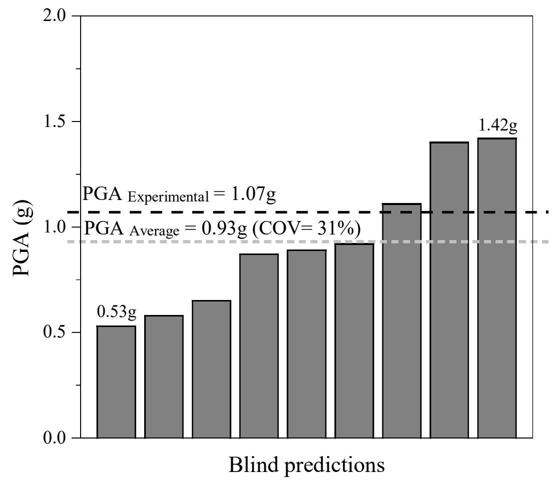


Figure 8 – PGA for all blind predictions of the stone structure.



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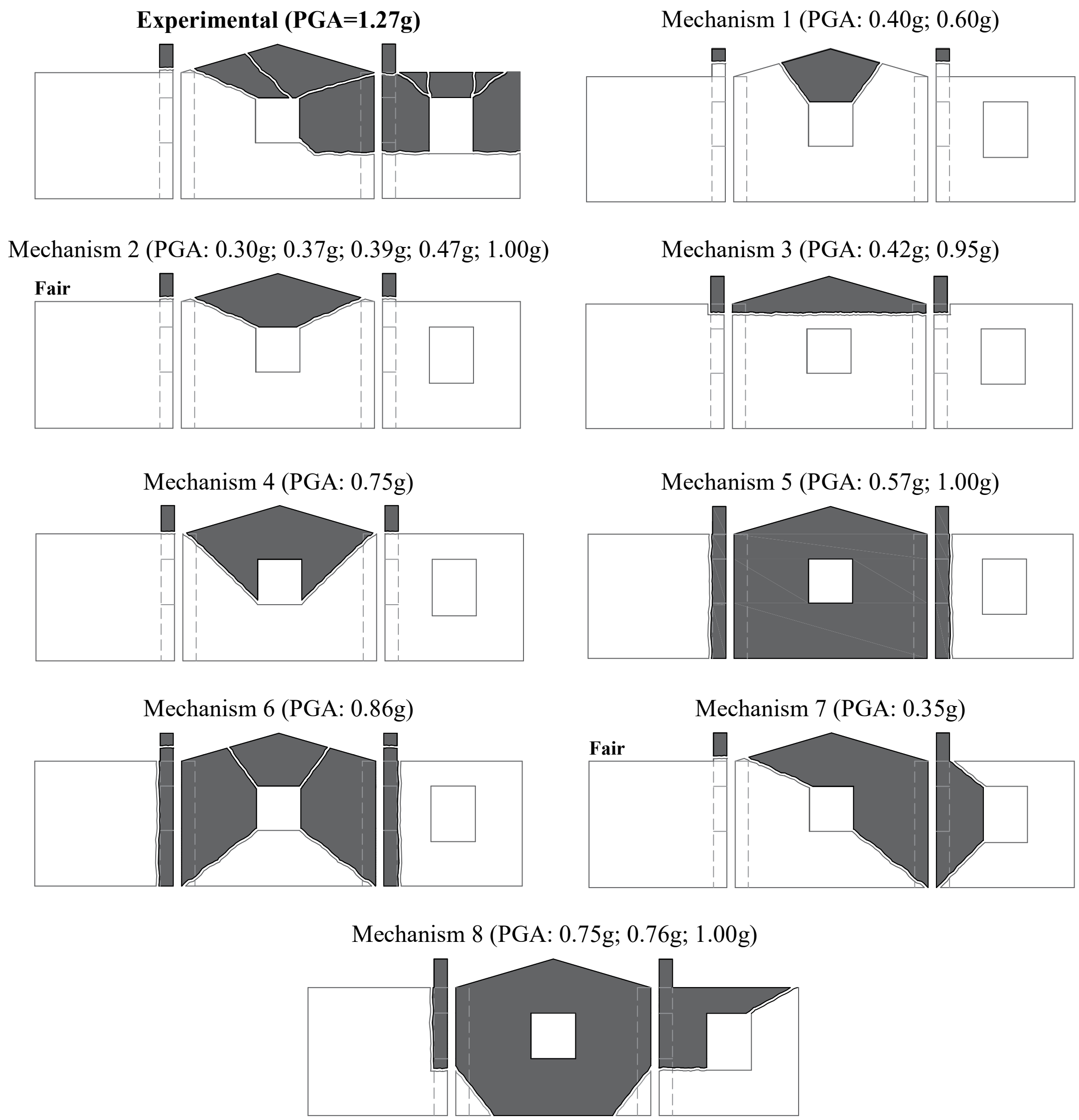

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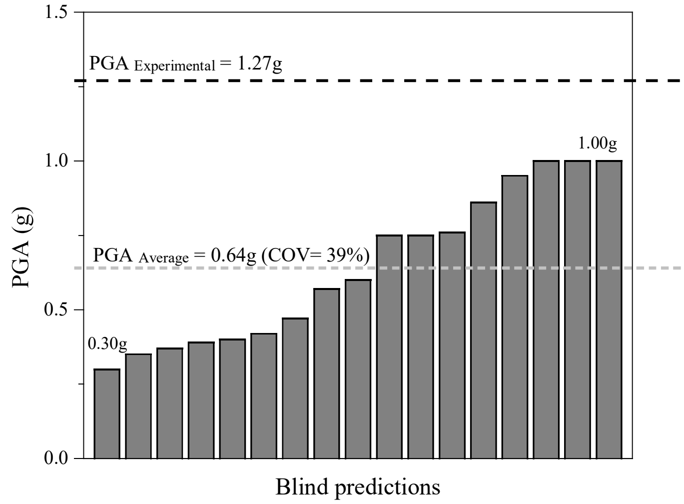

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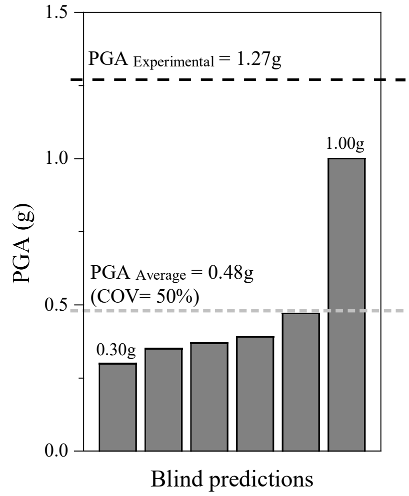

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