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Experimental and numerical investigation of walls strengthened with fiber plaster

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Abstract. The topic of this study is to investigate behaviors of masonry walls strengthened with reinforced fiber plaster under diagonal tensile loads. Full blend brick 100×50×30 mm in dimensions were used to make masonry walls with dimensions of 400×400×100 mm. Three different samples were manufactured by plastering masonry walls with traditional style, with 3% polypropylene or with 5% steel fiber. All the samples were tested using ASTM 1391- 81 standards. The propagation of damage on samples caused by diagonal tensile load was observed and load-displacement graphs were plotted for each sample. A finite element software (ABAQUS) was used to obtain numerical values for all samples and crack patterns and load-displacement responses were obtained. Experimental and numerical results were compared.

Keywords: composite materials; masonry walls; polypropylene and steel fiber; reinforced plaster mortar; finite element method

1. Introduction

Masonry buildings are the majority of buildings throughout the world. More than 50% of the all buildings in our country are masonry buildings (Korkmaz *et al.* 2010). Little, if any, engineering support has been used in the construction of these masonry buildings. Most of Turkish land is part of Seismic 1 zone. Moreover, most of the earthquakes in Turkey have caused more damages and loss of lives than what they are supposed to cause considering their relatively low magnitude. In masonry buildings, collateral damages occur even in earthquakes having considerably low magnitude. The majority of lives lost in earthquakes can be linked to these masonry buildings.

Forces occurring in masonry building are carried by the walls. Horizontal loads occurring during earthquakes cause powerful dimensional or non-dimensional stresses in load carrying walls. Due to these stresses, damages occurring in the walls of building make the structure unsafe. Under earthquake forces, damages are more pronounced in masonry buildings than those in other buildings. Therefore, it is of high significance to investigate earthquake behavior of masonry

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buildings from the safety point of view. Several experimental and theoretical studies exist in the literature for masonry buildings (Döndüren 2008, Valluzzi *et al.* 2005, Lucıano and Sacco 1998, Hendry 2001, Laurence and Rots 1997, Lopez *et al.* 1999). Before strengthening the walls of a masonry building, it is a must to know what the earthquake behavior of a masonry building is and how the structure collapses during an earthquake. Masonry buildings and concrete buildings have a lot in common, but, they quite differ with respect to their earthquake behavior (Kanit *et al.* 2005). The complexity in the mechanical behavior of a masonry building can be attributed to non-homogeneous material composition, its fragile behavior and uneven geometry (Daryan *et al.* 2009). Masonry walls are resistant to pressure effect, but are very weak against tensile forces. To overcome this weakness, ways to increase adherence between the brick and plaster have been investigated. Several additives were used in the plaster formula to increase tensile strength of masonry walls and strength, rigidity and ductility of masonry walls were experimentally and numerically studied (Guinea *et al.* 2000, Gabor *et al.* 2005, Fathy *et al.* 2008, Sinha *et al.* 1979, Marfia and Sacco 2001, Lee *et al.* 1996, Corradi *et al.* 2003, Elvin and Uzoegbo 2011, Griffith *et al.* 2004, Bayülke *et al.* 1993, Magenes and Calvi 1998, Benedetti *et al.* 1998, Blondet *et al.* 2006). Masonry buildings having weak load bearing capacity were strengthened using methods applied externally to a building. By strengthening wall surfaces with various materials (FRP, wires, fiber materials, steel mesh, steel and used tires), rigidity, strength and ductility of masonry buildings were studied (Gülkan and Gürdil 1998, Krevaikas and Triantafillou 2005, Paulay and Priestley 1992, Stratford *et al.*2004, Kolsch 1998, Ehsani *et al.* 1997, Gabor *et al.* 2005).

Thanks to advances in computer modeling in recent years, it is made possible to simulate real time behavior of masonry buildings, which is otherwise quite complex in building. In revealing real time behavior of a masonry building during an earthquake, non-linear dimensional analysis using FE methods has utmost significance. Studies using FE methods have addressed not only how the damage occurs, but also what the extent of damage is (Fathy *et al.* 2008, Abruzzese *et al.* 2009, Köksal *et al.* 2009).

In this study, shear behavior of masonry walls having traditional and fiber added plasters was investigated in their plane. To achieve this, 3 different samples rotated 45° were subjected to vertical loading and their damage level and load-displacement graphs were compared. Additionally, finite element (FE) analysis was carried out using ABAQUS software. Crack patterns and load-displacement graphs were obtained by ABAQUS. Experimental and numerical results were compared.

2. Experimental program

2.1 Test setup

Vertical load of 0.1 mm/s was applied to samples using a 250 kN hydraulic piston. Load was applied to samples with 45° steel caps. A load cell positioned between hydraulic piston and steel cap was used to measure the loads. To measure displacement of samples, 5 displacement meters were horizontally and vertically positioned on hydraulic piston or on either side of plaster. Data were transferred to a computer using an 8-channel static data acquisition system. Test setup is given in Fig. 1.

Fig. 1 Test setup and data acquisition system

Fig. 2 Brick producing

Fig. 3 Material tests

Table 1 Mechanical properties of masonry walls and plasters

Material Type	Mixing Ratio Sand / lime / cement / water	Fibers Additive	Compressive Strength (Mpa)	Tensile Strength (Mpa)	Modulus of Elasticity (Mpa)
n	20 / 2 / 3.6 / 1.7	-	2.68	0.325	2100
p	20 / 2 / 3.6 / 1.7	3%	8.95	0.573	9017
s	20 / 2 / 3.6 / 1.7	5%	4.82	0.867	10450
Brick	-	-	2.65	0.5	125

2.2 Materials

Masonry bricks of 100×50×30 mm were produced using the traditional method in order to represent properties of masonry buildings (Fig. 2).

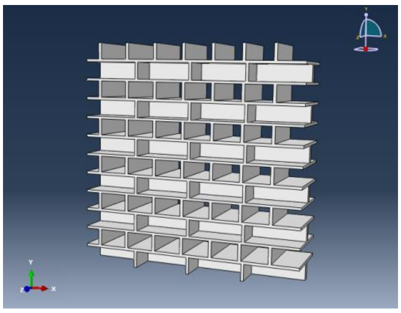
Three types of plaster were used in the producing of wall samples. These were the traditional plaster (mixture of sand, lime, mortar and water), the traditional plaster +3% polypropylene fiber and the traditional plaster +5% steel fiber. Mechanical properties of masonry walls and plasters were determined with uni-axial compression test and three point bending test (Fig. 3) and these mechanical properties are given in Table 1.

2.3 Producing of the samples

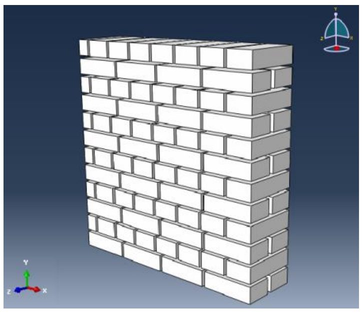
The dimensions of the samples were 400×400×100 mm. Horizontal and vertical joint gap between brick and plaster thickness measured 10 mm. After plastering, samples were cured under laboratory conditions for at least 7 days. Samples were produced as seen in Fig. 4.

Samples shown in Fig. 3 were plastered with mortars given in Table 2. Samples having the traditional plaster were coded as N, those having 3% polypropylene as P3 and those with 5% steel fiber as S5. Three numbers were produced from each type sample.

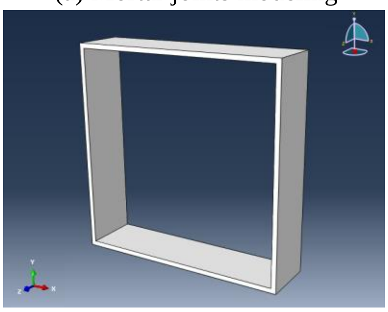
Fig. 4 Producing of the samples



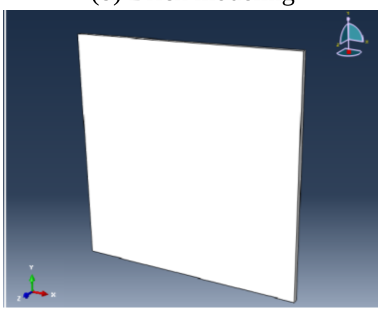
(a) Mortar joints modeling



(b) Brick modeling

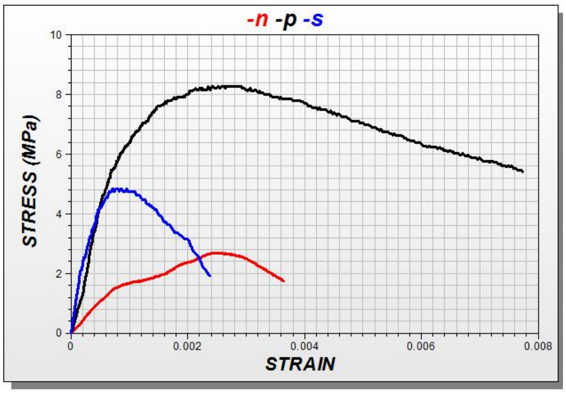


(c) Side surface plaster modeling

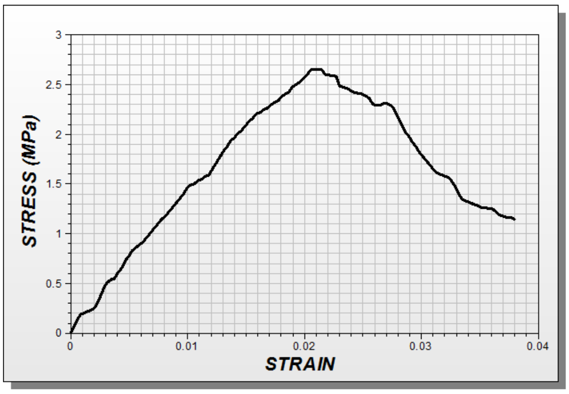


(d) Front and back surface plaster modeling

Fig. 5 Parts of walls in FE model



(a) Mortar



(b) Brick

Fig. 6 Stress-strain relationship of wall parts used in FE model

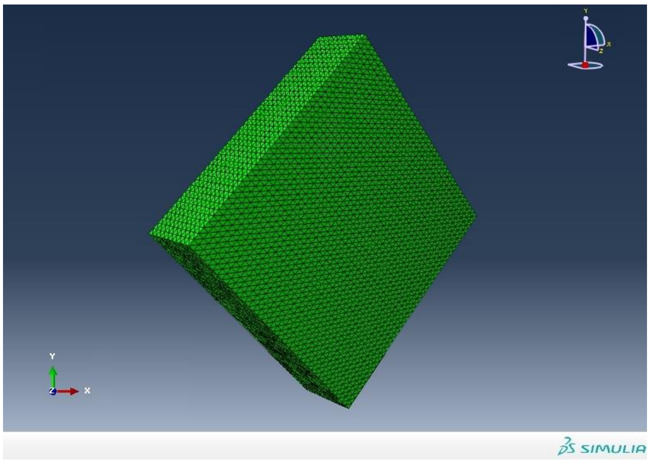


Fig. 7 FE model for the combined wall

3. FE Method

ABAQUS/CAE interface of ABAQUS finite element software was used in numerical modeling of samples. The type of analysis was ABAQUS/Explicit. ABAQUS/Explicit is a solver for time dependent problems. In the numerical analysis of the masonry wall, brick, plaster and joint were separately modeled (Fig. 5).

Test results of wall parts were used in FE modeling. Stress-strain relationships of masonry brick and plaster are given in Fig. 6.

As shown in Fig. 7, separate wall parts were combined and the probable interaction problem among wall parts was overcome. Thus, modeling time was shortened and interaction errors during modeling were eliminated.

In FE analysis, elements of three dimensional (C3), ten nodes (D10), modified (M) and four 3-corner tetrahedral types were used. This mesh (type) has 4 corner nodes (A, B, C, D) and 6 side nodes. The ten node tetrahedral element is better suited for and more accurate in modeling problems. Fig. 8 shows type of elements used and their local positions.

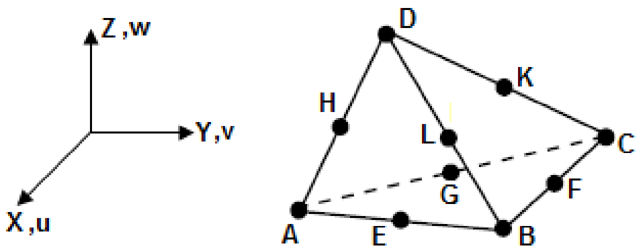
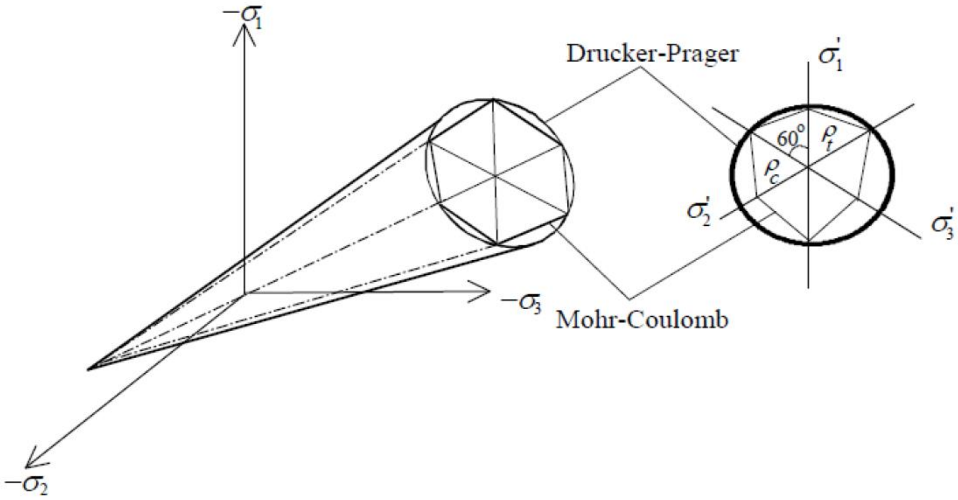


Fig. 8 Ten-point modified tetrahedral element (C3D10M)



(a) in principal plane of stress (b) in deviator plane

Fig. 9 Comparison of Drucker-Prager and Mohr Coulom criterion

C3D10M element type has three degrees of freedom in *x*, *y* and *z* direction. *u*, *v* and *w* represent displacements in *X*, *Y* and *Z* directions, respectively.

In this study, Drucker-Prager yield criterion which is suitable for semi-brittle and elasto plastic material was applied to brick and plaster. FE applications using Drucker-Prager yield criterion have been topic of different studies (Doran *et al.* 2009, Doran 2009). This criterion has successfully been applied in concrete and metals and is the modification of Von-Mises criterion, which is a practical mathematical form of strain criterion. This criterion takes hydrostatic pressure and deviator strain effects into account at the maximum stress. Yield surface

$$f\,(I_1 , J_2) = \alpha\, I_1 + \sqrt{J_2} - k = 0 \qquad (1)$$

was expressed as follows. α and k are material constants. When α is zero, the criterion becomes the same as Von-Mises criterion. As shown in Eq. (1), yield surface is a circular conic in principal plane of stress. Drucker-Prager and Mohr Coulomb criterion are compared in Fig. 9.

Additionally, α and k values can be calculated using internal angle of friction (ϕ) and cohesion (c) values (Chen and Han 1988), (ϕ) and (c) values in Eq. (2) were taken from (Basaran *et al.* 2013).

Table 2 Friction (ϕ) and cohesion (c) values of the samples

Specimen	ϕ	c (MPa)
N	27.40°	0.546
P3	26.51°	1.186
S5	18.64°	1.166

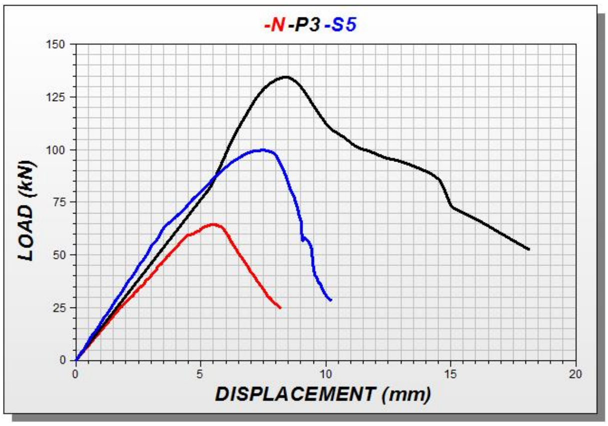


Fig. 10 Load-displacement curves of experimental study

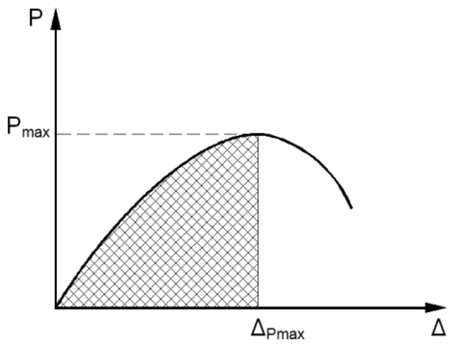


Fig. 11 Calculation of EDC

$$\alpha = \frac{2\sin\phi}{\sqrt{3}(3-\sin\phi)} \quad ; \quad k = \frac{6c\cos\phi}{\sqrt{3}(3-\sin\phi)} \qquad (2)$$

Input parameters of numerical model in Eq. (2) are given Table 2.

4. Experimental and FE method results

Experimental and numerical results were compared in terms of load carrying capacity (LCC), energy dissipation capacity (EDC) and propagation of cracks.

Table 3 LCC and EDC of the samples in test

Specimen	LCC (kN)	Increasing %	EDC (kNmm)	Increasing %
N	64	-	197	-
P3	134	109	590	200
S5	100	56	450	128

(a) N sample (b) P3 Sample (c) S5 Sample

Fig. 12 Diagonal tension cracks in experiments

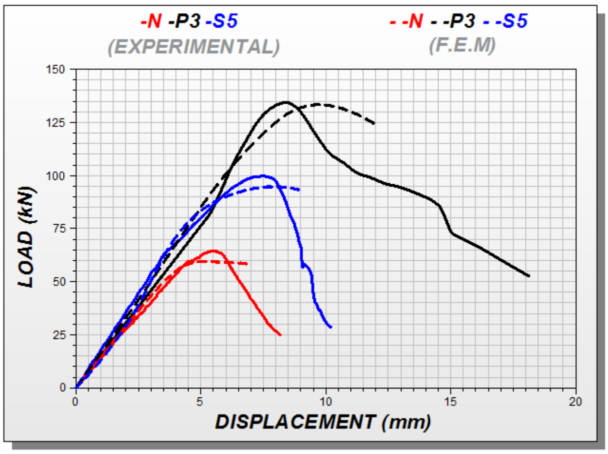


Fig. 13 Load- displacement curves obtained experimentally and numerically

4.1 Test results

Displacements caused by loads applied to samples were measured and curves in Fig. 10 were obtained.

LCC was lost at 64 kN and 5.5 mm for sample N, at 134 kN and 8.6 mm for sample P3 and at 100 kN and 7.5 mm for sample S5. To determine EDC, the curves in Fig. 11 were used. Area up to maximum load under the curve was calculated (Fig. 11).

Table 3 gives not only LCC and EDC but also percent increases in these parameters.

Increase in LCC was 109% for sample P3 whereas it was only 56% for sample S5. Increase in EDC was 200% for sample P3 and 128% for sample S5.The cracks in all samples occurred in the loading direction due to diagonal tensile forces. Damages in the samples are shown in Fig. 12.

Table 4 LCC and EDCs of the samples in FE analysis

Specimen	LCC (kN)	Increasing %	EDC (kNmm)	Increasing %
N	59	-	191	-
P3	125	112	603	215
S5	93	58	443	131

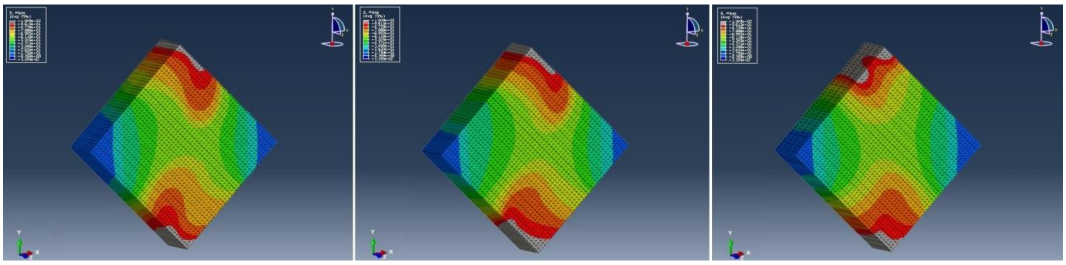


Fig. 14 Stress distributions in FE Method

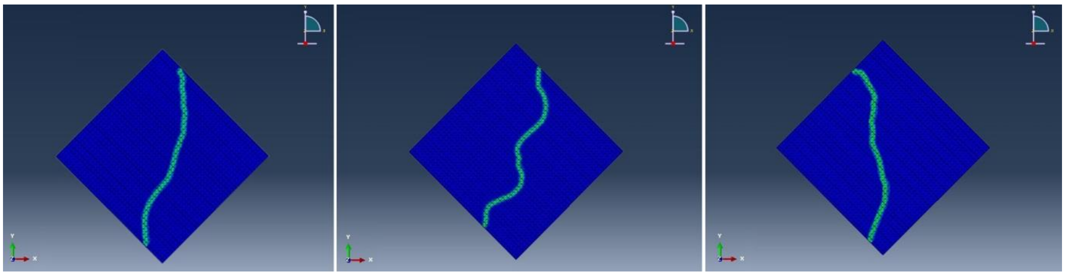


Fig. 15 Diagonal tension cracks in FE Method

4.2 FE analysis results

Load-displacement curves obtained from FE analyses are shown in Fig. 13. The LCC, EDC and increasing in these parameters are tabulated in Table 4.

Increase in LCC was about 112% for sample P3, while it was about 58% in sample S5. The increase in EDC was 215% increase in sample P3 and 131% increase in sample S5.Stress distributions and diagonal tension cracks obtained from FE Method are given in Fig. 14 and Fig. 15, respectively.

5. Discussion

In this study, strengthening of masonry walls with fiber added plasters were investigated. The experimental and numerical studies were carried out. LCC and EDC of strengthened samples were much higher than those of N samples and these values are presented in Table 5.

Experimentally, increase in LCC was about 2.09-times for sample P3 and 1.56-times for sample S5. Similarly, EDC of sample P3 increased 3-times while that of sample S5 increased 2.28-times. In the FE analyses, increase in LCC was more than 2.12-times for sample P3, while that was more than 1.58-times for sample S5. As for EDC, the increase was 3.15-times for sample P3

Table 5 Comparison of experimental and numerical results

Specimen	Experiment		FEM		Difference	
	LCC (Kn)	EDC (kNmm)	LCC (Kn)	EDC (kNmm)	LCC %	EDC %
N	64	197	59	191	7.81	3.04
P3	134	590	125	603	6.71	2.15
S5	100	450	93	443	7.00	1.55

whereas it was 2.31-times for sample S5.

When experimental and numerical results were compared, the difference in LCC increase was 7.81% for sample N, 6.71% for sample P3, and 7% for sample S5. As for EDC, 3.04% difference in sample N, 2.15% difference in sample P3 and 1.55% difference in sample S5 were observed. Experimental and numerical results were in good agreement. When the type of damage was examined in sample N, crack starting from the right side of upper steel caps propagated downward and formed two separate cracks. These cracks stretched out to both sides of steel caps. Cracks in sample P3 started from the right side of steel caps and propagated downward making a curve and ended on the left side of steel caps. However, cracks in sample S5 started from the left side of steel caps and turned into two separate cracks and stretched out to the sides of lower steel caps. Additionally, thin cracks formed on the periphery of lower steel caps. Generally, damages occurred in the sample were seen in the direction of applied force and were under the effect of diagonal tensile forces. Damages in FE analysis were in accord with experimental results. However, secondary cracks were not seen in FE analysis.

6. Conclusions

Nine masonry wall samples of 400×400×10 mm were produced in this study by using full blend bricks. Samples were obtained by plastering masonry walls with traditional style, with 3% polypropylene or with 5% steel fiber. Shear behavior of samples, which were subjected to loads in their plane were experimentally and numerically studied.

The maximum increases in LCC and EDC were observed in sample P3. Traditional plastered samples failed as brittle and sudden. On the contrary, samples plastered with added fiber exhibited ductile behavior due to confinement effect.

Thus, these samples did not rupture. Experimental and numerical results were in agreement for LCC and EDC and the propagation of cracks. Determining material behavior experimentally, the chosen model, the type of support and failure criterion chosen may have contributed to the good agreement between experimental and numerical results.

In conclusion, masonry buildings in rural areas may be economically and safely strengthened against earthquakes by the developed method in this study. This strengthening method can be easily and quickly applied to new masonry buildings or can be used to strengthen existing ones.

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