16TH EUROPEAN CONFERENCE ON
EARTHQUAKE ENGINEERING
THESSALONIKI 18 - 21 JUNE 2018

FINITE ELEMENT MODELING OF EXPERIMENTALLY TESTED SOLID BRICK MASONRY WALLS

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ABSTRACT

Unreinforced unconfined solid brick masonry walls were experimentally tested in full scale (233x241x25cm) and reduced scale (100x100x25cm) at the laboratory of the Institute for materials and structures, Faculty of Civil Engineering in Sarajevo. Cantilever walls were loaded in cyclic shear or pushed monotonically. In order to study the nonlinear behavior in a detailed and global manner, finite element meso- and macro-models of the tested walls were created using finite element software Diana 10.1.

Brick units are discretized by continuum elements in a meso-model and discontinuity in displacement field is introduced by interface elements between units. In order to account for brick cracking, an additional interface element was added in the unit middle. Continuum macro-models approximate heterogeneous masonry wall by a single material and discretization is independent of brick layout, i.e. bricks, mortar and unit-mortar interface are smeared out in the continuum. Recently developed engineering masonry model is an orthotropic total-strain continuum model with smeared cracking and it was used with shell elements. Numerical results are verified against the data obtained from experimental research program. In case of low precompression, the walls exhibit rocking failure mode while for higher vertical stresses diagonal cracking occurs. The results show good matching with the experimentally obtained curves regarding the ultimate load and ductility. Macro-modeling approach was also employed for analysis of an existing masonry house in Sarajevo.

Keywords: masonry walls; finite elements; meso- and macro-models

1. INTRODUCTION

Bosnia-Herzegovina is situated in a seismically active region of South-East Europe and it is divided into seismic zones with peak ground acceleration (PGA) ranging from 0.1-0.2g for 475 years return period in the most parts of the country up to PGA of 0.30-0.35g in some regions. The majority of multi-story residential buildings erected in the years following World War II were unreinforced unconfined masonry buildings with 4-6 floors. With respect to seismic vulnerability classification (EMS), masonry structures belong to classes B and C which means that heavy and very heavy damages, including partial collapse, could occur in the case of stronger earthquake motions. This was unfortunately proven during several earthquakes in the region, Skopje 1963, Banjaluka 1969, Montenegro Coast 1979 (Hrasnica 2009, Hrasnica and Medic 2012).

Increased seismic demand according to EC 8 compared to the old national standard poses new challenges in verification of load bearing capacity of existing buildings. Faculty of Civil Engineering in Sarajevo initiated a specific research program with experimental testing and computational modeling in order to assess masonry behavior and risk (Medic 2018). Masonry is a composite material which consists of units and joints which are usually filled with mortar. It is a complex material with 3D internal arrangement (bond). Not only varying material characteristics, but also building technology can have significant influence on masonry response, which makes the modeling a demanding task for structural engineers. Low tensile strength of masonry imposes the use of nonlinear constitutive laws in assessment of existing structures and in seismic analysis of new buildings (Lourenço 2008).

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Experimental testing of masonry is essential in understanding structural behavior, however, numerical modeling can complement experimental research and provide new insights. Masonry structures are usually analyzed by finite elements (FEM) and, based on the level of detail, computational strategies are traditionally divided into following categories: micro-, meso- or macro-modeling techniques. One modeling strategy cannot be preferred over the other since different application fields exist for each model type. Linear elastic methods like lateral force method or response spectrum analysis do not seem adequate to describe quite nonlinear response of masonry and estimate global ductility.
The focus of this paper is on nonlinear quasi-static cyclic analysis of individual masonry walls using smeared continuum models discretized by shell elements. Additionally, several meso-models were created, where nonlinear behavior was lumped into traction-displacement relation in interface elements at joints. The numerical models were verified against experimental data obtained from the tests performed at the laboratory of the Institute for materials and structures in Sarajevo. Unreinforced unconfined masonry walls were built in full scale (233x241x25cm) and reduced scale cca. 1:2 (100x100x25cm). The walls were loaded in cyclic shear under constant vertical pressure or pushed monotonically. Mechanical properties of masonry components (brick, mortar, and interface) and homogenized masonry (compressive strength and elastic modulus) were determined using appropriate specimens (Hrasnica et al. 2014).
Nonlinear static pushover analysis of an old masonry building carried out under constant gravitational load and monotonous horizontal loads in the form of displacement increments is also presented. The building was heavily damaged during war in Sarajevo and the floors were completely destroyed. Pushover analysis is used to verify the nonlinear behavior of newly-designed, and in particular, of existing structures. Two numerical macro-models were created. The first model represents the existing damaged structure. In the second model, which represents the rehabilitated structure, R.C. floors and internal walls were added to the building in order to increase the load bearing capacity.

2. EXPERIMENTAL TESTS OF WALLETS AND WALLS

In the first step, a testing program was designed to identify the mechanical properties of masonry and its components. Compressive and tensile strength, elastic modules of brick and mortar, as well as the properties of their mutual contact were separately investigated. The compressive strength and the modulus of elasticity of the solid brick masonry were determined on the reduced wall samples – wallets (Hrasnica et al. 2016). In the next step, physical models of plain (unconfined unstrengthened) masonry walls were constructed in the full scale (233x241x25cm) (Figure 1) and in the reduced scale (100x100x25cm) (Hrasnica et al. 2017).

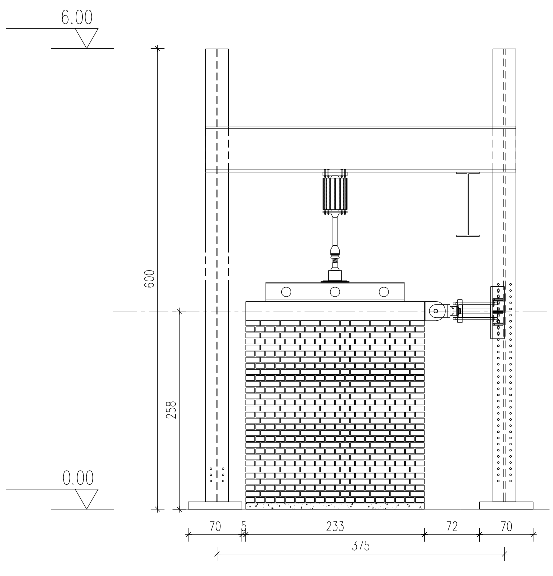


Figure 1. Test set-up.

The chosen wall geometry introduces complex response to seismic action because the walls cannot be classified either as slender or squat. Two plain walls statically modeled as cantilevers were tested for the horizontal static cyclic in-plane load under a vertical pressure of 0.4 N/mm^2. A vertical load was applied to the wall centerline via steel loading beam and then displacements were incrementally and cyclically imposed at the RC top tie beam. The walls failed at almost equal horizontal load level with the appearance of characteristic diagonal crack pattern (Figure 2). The measured force at failure agrees well with the ultimate load obtained by using the expression from EC 6. Unexpectedly, plain masonry walls have considerably greater ductility than recommended in seismic codes using behavior factors.

Reduced model walls were exposed to a cyclic loading program under variable values of the vertical pressure. For a compressive stress of 0.4 N/mm^2, the wall rotates in a rigid body mode without cracking. On the other hand, under 0.6 N/mm^2 pressure the wall rotates, but with the occurrence of cracks that develop on the compressed toe and extend diagonally through the wall. For a stress level of 1.0 N/mm^2, a crack appears in the middle of the wall which is characteristic for shear failure.

a) b)

Figure 2. Diagonal failure pattern of the tested walls: a) W1, b) W2

3. NONLINEAR FINITE ELEMENT MODELING AND RESULTS

3.1 Macro-model of the tested wall

Masonry is homogenized in a macro-model employing recently developed Engineering Masonry model (DIANA FEA 2016). Engineering Masonry model is an orthotropic total – strain continuum model with smeared cracking and it can be used with membrane or shell elements. The model is capable of simulating compression, tensile and shear failure modes and it can crack in bed-joints, head-joints as well as diagonally. Crushing, shear and tensile behavior of Engineering Masonry constitutive model is illustrated in Figure 3 (Schreppers et al. 2016). Parameters of the constitutive model for the wall analysis are given in Table 1. All nodes on the upper edge were rigidly connected to have the same horizontal displacement and free rotation. The walls were discretized using 2D plane stress elements with 8 nodes of an average size of 0.1 m. A quasi-static implicit nonlinear analysis was performed with the Newton-Raphson iterative scheme involving both material and geometric nonlinearity. The loading program is shown in Figure 4 and each cycle consists of three runs. The model was run until the model ceased to converge, or the maximum displacement of the actual experimental test was attained.

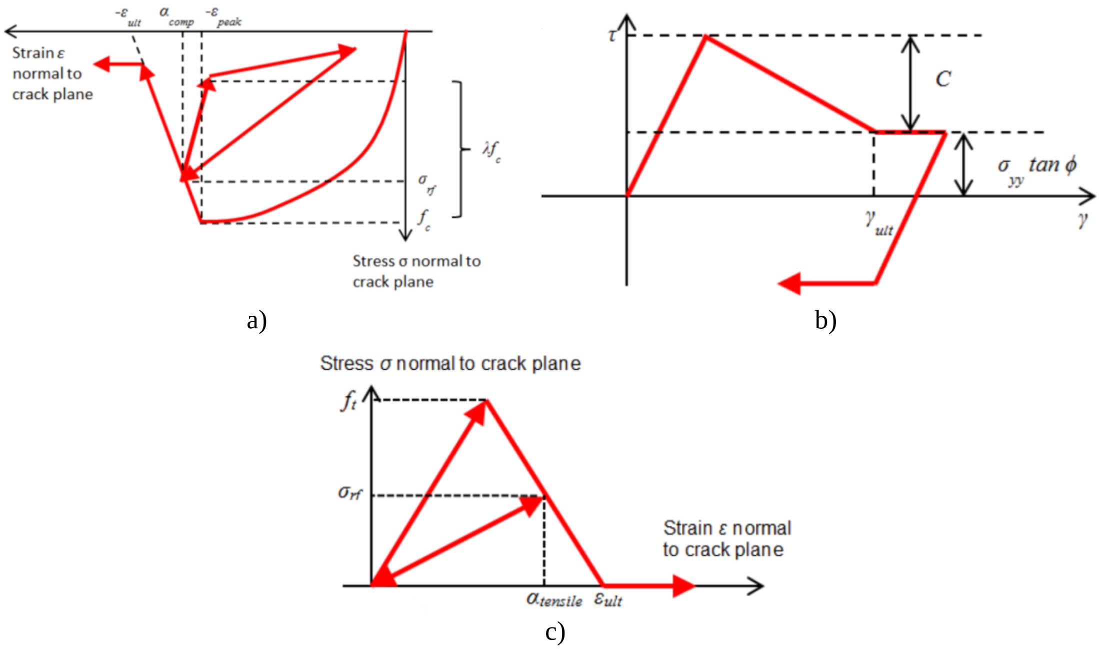


Figure 3. Engineering masonry model: a) crushing behavior, b) shear behavior, c) cracking behavior

Table 1. Parameters of engineering masonry model used for the wall analysis.

Parameter	Value	Parameter	Value	Parameter	Value
E_x	4e+09 N/m^2	G_{ft}	10 N/m	G_{fs}	20 N/m
E_y	4e+09 N/m^2	HEADTP	NO	f_c	6.48e+06 N/m^2
G_{xy}	1.6e+09 N/m^2	h	Rots	n	4
ρ	1850 kg/m^3	c	90000 N/m^2	G_c	40000 N/m
f_{tx}	90000 N/m^2	ϕ	0.78 rad	λ	1

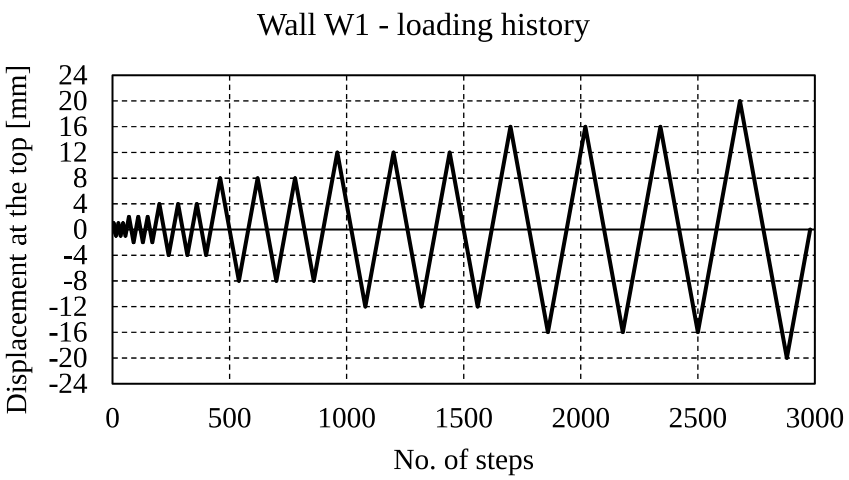


Figure 4. Loading program for wall W1

Comparison of hysteretic curves of the macro-model and the results of previously described experiment are shown in Figure 5 where one can notice quite good matching. Wall displacement pattern as well as accumulated shear strains that pertain to major cracks are shown in Figure 6. The damage pattern is diagonal which complies with the experimental observations.

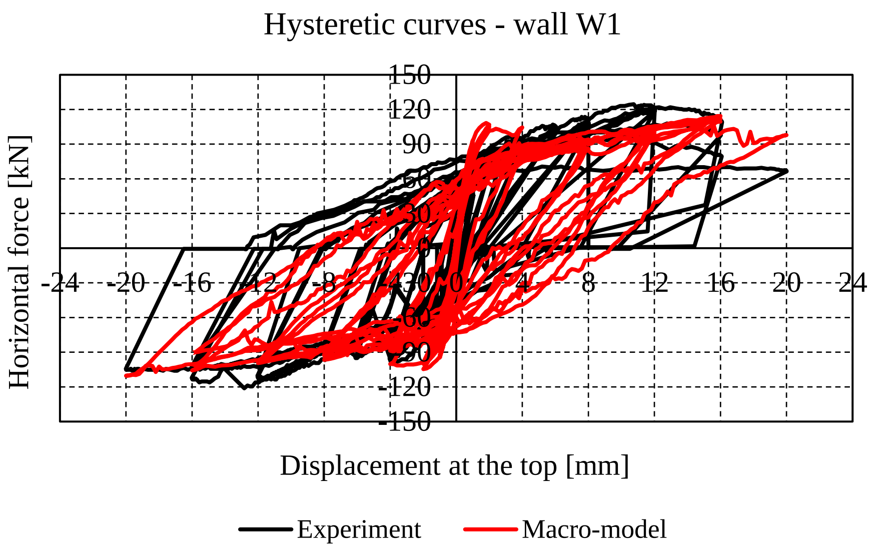
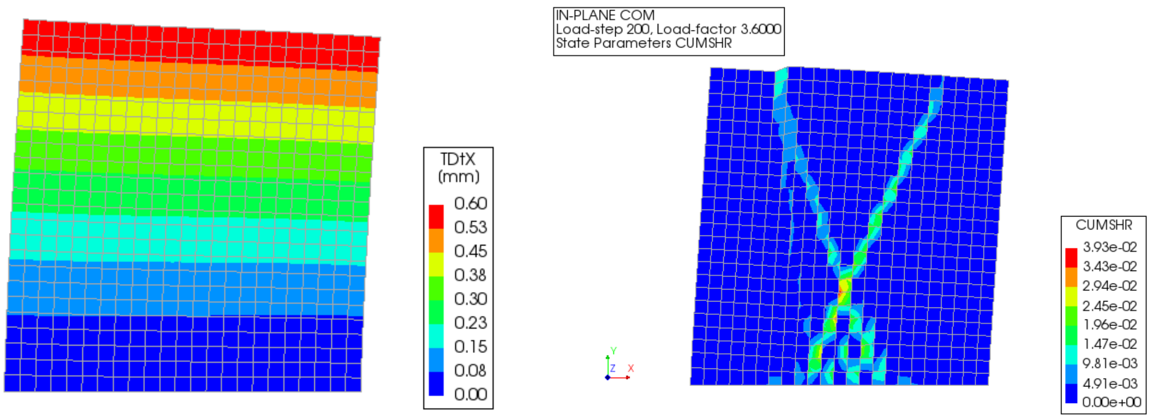


Figure 5. Comparison of hysteretic curves of the macro-model and the experiment for W1



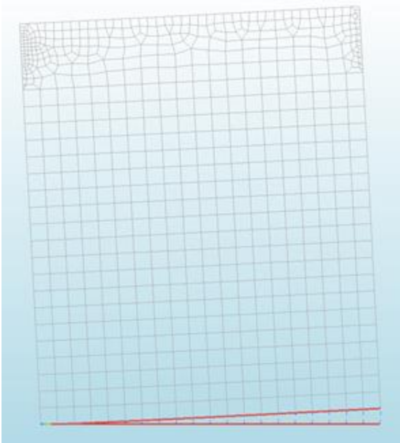
a) b)

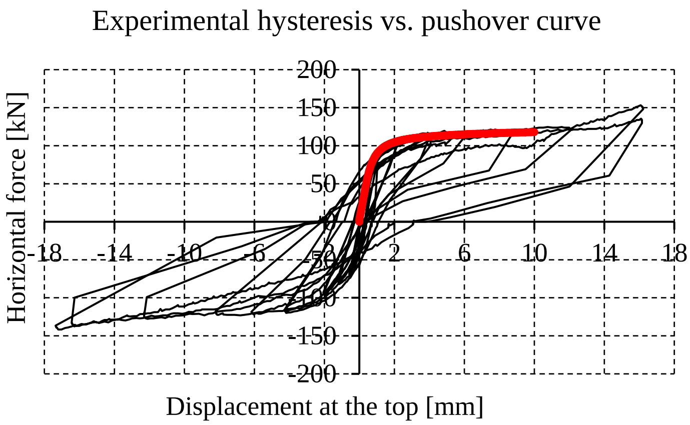
Figure 6. Results of the numerical analysis: a) wall displacements, b) accumulated shear strains

In case of low precompression, rocking failure mode was detected on reduced scale walls. Since the wall behaves as a rigid body, in this case masonry was modeled as a linear elastic material. Nonlinearity is concentrated in the interface element at the contact between the wall and the foundation, with the properties listed in Table 2. An opening mode (gap) was set for the interface. Wall displacements and comparison of experimentally obtained hysteresis and numerical pushover curve are given in Figure 7.

Table 2. Parameters of Mohr-Coulomb interface for rocking failure mode.

Parameter	**Value**
k_n	1e+06 N/mm^3
k_s	1e+06 N/mm^3
c	0 N/mm^2
ϕ	0.38 rad
f_t	0 N/mm^2



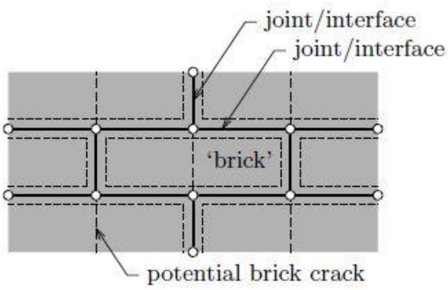


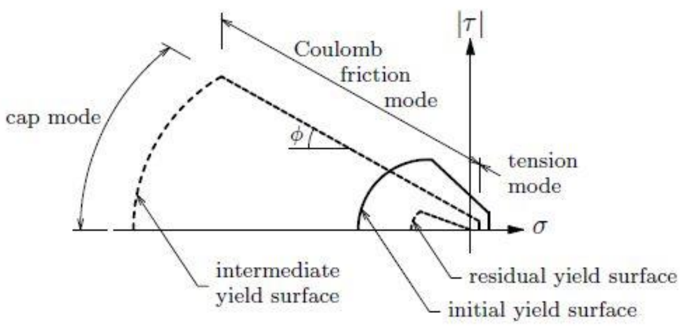
a) b)

Figure 7. Results of the numerical analysis in case of rocking: a) wall displacements, b) comparison of experimentally obtained hysteresis and numerical pushover curve

3.2 Meso-model of the tested reduced scale wall

The contact material model, also known as the "Composite Interface model", is appropriate for simulation of fracture, frictional slip as well as crushing along material interfaces, for instance at joints in masonry. Usually, the brick units are modeled as linear elastic continua, while the mortar joints are modeled with interface elements, which obey the nonlinear behavior described by this combined cracking-shearing-crushing model (acronym CCSC). Figure 8a shows the elements of a meso-model, with an additional interface element in the unit middle (DIANA FEA 2016). This interface was modeled in order to enable cracking of bricks which was experimentally observed. A plane stress interface model was formulated by Lourenço (Lourenço 1996). It is based on multi-surface plasticity, comprising a Coulomb friction model combined with a tension cut-off and an elliptical compression cap (Figure 8b). Softening acts in all three modes and it is preceded by hardening in the case of the cap mode.
Parameters of CCSC interface (horizontal and vertical joints) and Coulomb interface with zero-tension gapping mode (brick contact) are listed in Table 3. The brick behavior was assumed linear elastic with Young's modulus equal to 20 000 N/mm^2 and Poisson ratio of 0.15. The input parameters were based on measured material properties (Medic 2018) or previous numerical and experimental studies (Lourenço 1996) (Allen et al. 2017). Mesh, loading and boundary conditions for the meso-model of reduced-scale wall are shown in Figure 9a. The resulting normal interface tractions are given in Figure 9b. It can be noticed that the mortar joint opens at the tension side and that rocking governs the failure mechanism. The comparison of experimentally and numerically obtained pushover curves is given in Figure 10 and it can be concluded that the curves match quite well.



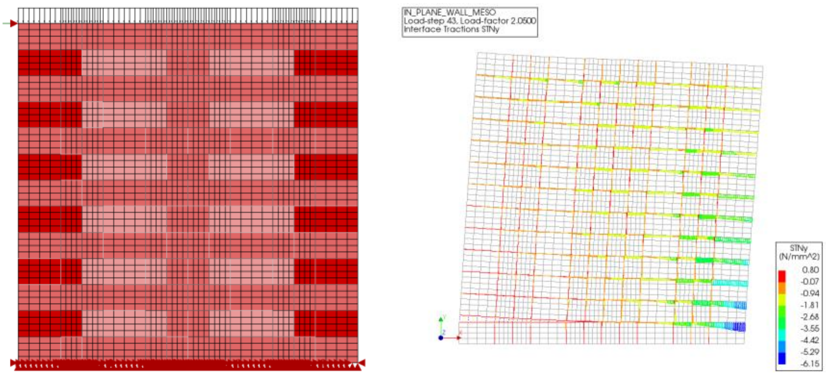


a) b)

Figure 8. Meso-model of masonry: a) locations of interfaces, b) multi-surface plasticity for CCSC model

Table 3. Parameters of CCSC interface (joints) and Coulomb interface (brick contact).

CCSC	Value	CCSC	Value	Coulomb	Value
k_n	50 N/mm^3	σ_u	- 0.75N/mm^2	k_n	1000 N/mm^3
k_s	10 N/mm^3	δ	1.8	k_s	500 N/mm^3
f_t	0.1 N/mm^2	a	- 0.8	f_t	3.6 N/mm^2
G_t	0.003 N/mm	b	0.05	G_t	0 N/mm
c	0.09 N/mm^2	f_c	6.5 N/mm^2	c	1.2 N/mm^2
ϕ	0.785 rad	C_s	9	ϕ	0 rad
Ψ	0.540 rad	G_c	10 N/mm	Ψ	0
ϕ_r	0.785 rad	κ_p	0.015	ϕ_r	0



a) b)

Figure 9. Meso-model of reduced-scale masonry wall: a) mesh, loading and boundary conditions, b) normal interface tractions

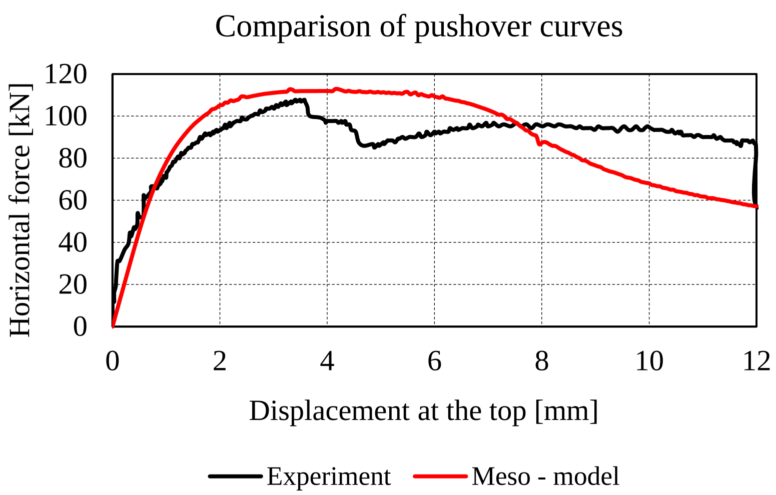


Figure 10. Comparison of pushover curves obtained experimentally and numerically

3.3 Pushover analysis of an existing building

The paper analyzes the response of an old masonry building in Sarajevo, Trnovska 9 (Figure 11), where the floors suffered significant damage during the war. Generally, older buildings have a stiff structure, but also limited ductility. Two numerical macro-models were constructed with engineering masonry material model, the first of which is without floors and the other with reinforced concrete floors and additional internal walls. The EC8 acceleration spectrum with the following properties was used: type 1, behavior coefficient q = 1, damping of 5%, PGA 0.1g and soil type B.

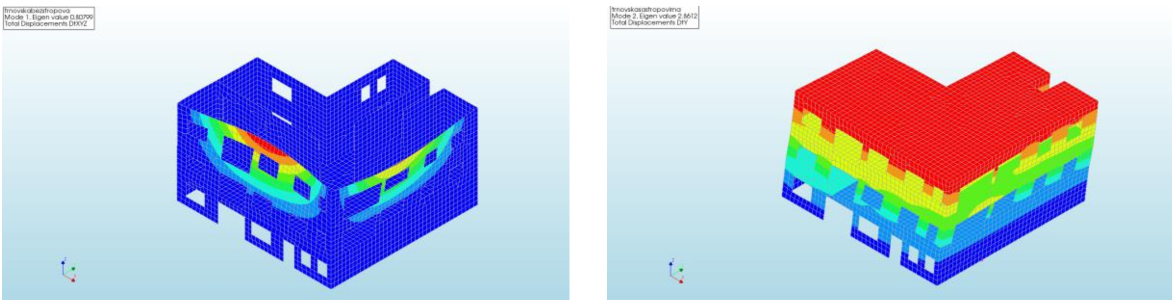
a) b)

Figure 11. Existing structure at Trnovska 9, Sarajevo: a) view from outside, b) view from inside

The properties of the masonry model used for the analysis of the building in its original state and with the reinforced concrete floors are shown in the following table (Table 4). The assumed behavior of R.C. floors is linear elastic. Finite element types CQ40S and CT30S (walls) and CQ40F and CT30F (floors) were used in the analysis. An integration scheme with higher number of Gauss points was used, i.e. 3 points for CQ40S, 2 for CQ 40F, 6 for CT30S, and 3 points for CT30F. Fundamental eigen modes that pertain to damaged building and the building with R.C. floors are shown in Figure 12a and Figure 12b. Cracking in the damaged state is illustrated in Figure 13.

Table 4. Parameters of engineering masonry model used for analysis.

Parameter	**Value**	**Parameter**	**Value**	**Parameter**	**Value**
E_x	600 N/mm^2	G_{ft}	0.005 N/mm	G_{fs}	0.02 N/mm
E_y	300 N/mm^2	HEADTP	DIAG	f_c	2 N/mm^2
G_{xy}	190 N/mm^2	θ	30°	n	4
ρ	1850 kg/m^3	h	Rots	G_c	15 N/mm
f_{tx}	0.10 N/mm^2	c	0.10 N/mm^2	λ	1
f_{ty}	0.05 N/mm^2	ϕ	32°		



a) b)

Figure 12. Analysis results: a) the first eigen mode of the building in a damaged state (T_1 = 1.25 s), b) the first eigen mode with R.C. floors added (T_1 = 0.35 s)

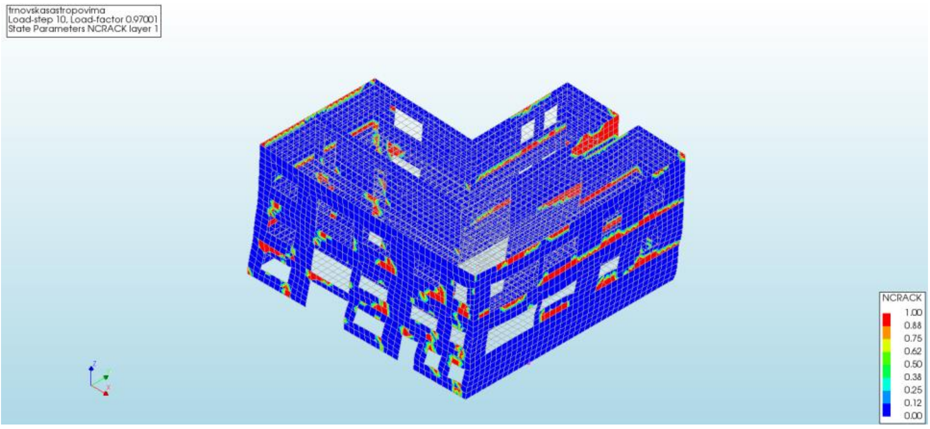


Figure 13. Analysis results – cracking in the damaged state

In order to describe the building response, the displacements of the highest floor are monitored and plotted against the total applied horizontal force (Hrasnica 2012). The resulting graph F-Δ is called the capacity curve which is directly compared with the demand that the observed structure needs to fulfill (i.e. the spectrum of the selected earthquake). Both graphs were plotted in the Acceleration Displacement Response Spectra Format (ADRS) described by spectral acceleration and spectral displacement.

After the analysis of the damaged building and the building with reinforced concrete floors, it can be concluded that the strengthened structure has a much better response to the earthquake action (Figure 14). The capacity curve of the building with R.C. floors passes through the spectrum in the elastic section at the horizontal displacement of 1 cm so that the observed structure will withstand the assumed earthquake. The capacity curve of the damaged structure also passes through the spectrum. However, the spectral displacement is significant and amounts to 5 cm. The observed structure will theoretically withstand the assumed earthquake intensity, but with the presence of considerable damage.

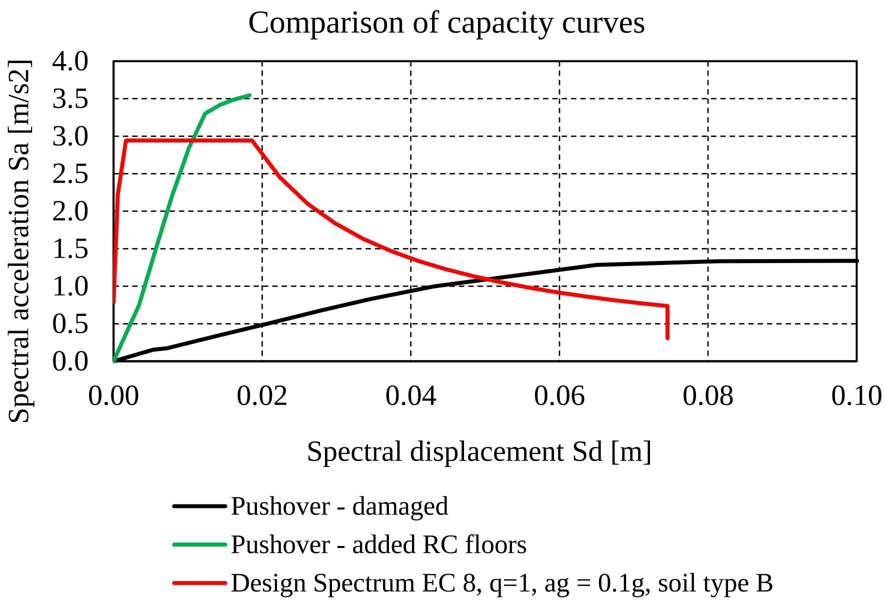


Figure 14. Comparison of capacity curves of existing damaged building and the building with R.C. floors

4. CONCLUSIONS AND FURTHER WORK

Different classes of numerical models are presented in the paper. Two modeling approaches were used to simulate the tested walls: the macro-model where the structure is a homogeneous continuum whose behavior is described with the engineering masonry model and the meso-model where the structure is discretized using the continuum elements (bricks) and contact elements (joints, cracks). In the discrete model, the bricks are linear elastic, and nonlinear behavior is possible on the contact elements where the composite CCSC (combined cracking-shearing-crushing) model is employed for the joints and Mohr-Coulomb for cracks in the bricks. The models are planar so that the 3D character of masonry structure is neglected (longitudinal and transverse layout of brick elements).

The advantage of the macro-model lies in the simplicity of the finite element mesh and the construction of the model, so the requirements for computer time are much lower than in the meso-model. It is possible to apply the complete loading program without divergence at large displacements, which ultimately results in a failure mode with cross diagonal cracks. On the other hand, the macro-crack is most often diagonally localized, while the cracks in the model are smeared out along certain width or the cracking pattern is diffuse. When using macro models, it is tacitly assumed that the structure, load and boundary conditions are such that the mortar and the brick do not have to be separately discretized, which is also the lack of a macro-model. The element and the joint are no longer distinguished, so it is impossible to determine the degree of stress of the brick or mortar.

Generally, different failure modes can be obtained using macro-models. In case of low precompression, rocking and local crushing occurs. Shear failure with diagonal cracking pattern develops for higher precompression levels. Depending on the loading program, force-displacement relationship shows a pushover or a hysteretic curve. Linear increase of force with displacement increment is typical for the initial phase of the obtained pushover curves. The other part of the curve is almost horizontal which implies stiffness degradation and yielding. However, in case of rocking, yielding is not caused by material degradation but the reduction of the compressed zone as imposed displacement increases. Hysteretic curves are full and significant energy is dissipated when shear governs failure. The slope of hysteretic curve decreases when unloading, which means that, aside from plastic deformations typical for joint failure, bricks fail in tension and damage occurs. The numerically obtained hysteretic curve agrees reasonably well with the one determined by the experimental program. The initial elastic stiffness, ultimate resistance, failure mode and cracking pattern were predicted quite well by the nonlinear finite element analysis. However, further numerical investigations related to mesh sensitivity and variability of material properties are necessary.

Bending failure mode of reduced-scale masonry walls loaded with horizontal force and low vertical precompression can be modeled with meso-model quite well. When tensile strength is reached, the joint opens and the wall rotates around the compressed toe. Generally, this failure pattern is obtained regardless of the changes in model parameters, and the reason lies in the fact that the wall geometry and the boundary conditions between the bricks are correctly modeled, so that the weak spots can easily be identified. The level of details of a discrete model cannot be achieved by using a homogenized continuum, such as in the case of separating individual bricks from the rest of the structure. Also, the stress state in bricks and contacts is well described. Regardless of the many advantages of the discrete model, there are several important shortcomings. It is necessary to invest considerably more time in modeling geometry and analyzing the results than with the continuum model. The analysis is more demanding ("expensive") due to additional contact elements.

Macro-models can be employed for assessment of engineering structures. Analyses of a heavily damaged building located in Sarajevo was performed using engineering masonry material model with shell elements. The first model represents the existing damaged structure. In the second model, which represents the rehabilitated structure, R.C. floors and internal walls were added to the building in order to increase the load bearing capacity. Results indicate that significant cracking occurs in the existing structure and that collapse is expected for an earthquake with PGA of 0.1 g. Seismic response of the upgraded building characterized by limited nonlinear deformations was more favorable considering that the same material properties were assumed for masonry.

5. ACKNOWLEDGMENTS

The authors wish to acknowledge the support of this research effort by the Federal Ministry of Education and Science under "Seismic Resistance of Existing Masonry Buildings" award no. 05-39-3087-5/16.

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