2CroCEE
22 - 24 March 2023

Proceedings of the 2nd Croatian Conference on Earthquake Engineering - 2CroCEE
Zagreb, Croatia - March 22 to 24, 2023

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.102

NUMERICAL MODELING OF THE IN-PLANE BEHAVIOR OF EXPERIMENTALLY TESTED SOLID BRICK MASONRY WALLS

Senad Medić (1), Mustafa Hrasnica (2)

(1) *Assistant professor, University of Sarajevo-Faculty of Civil Engineering, senad.medic@gf.unsa.ba*
(2) *Full professor, University of Sarajevo-Faculty of Civil Engineering, hrasnica@bih.net.ba*

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 the finite element software Diana FEA. 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. The 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 the experimental research program. The walls exhibit rocking failure mode in low precompression, while diagonal cracking occurs for higher vertical stresses. The results show good matching with the experimentally obtained curves regarding the ultimate load and ductility.

Keywords: masonry walls, finite elements, meso- and macro-models, experimental testing.

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 most parts of the country up to a 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. Concerning seismic vulnerability classification (EMS), masonry structures belong to classes B and C, which means that heavy and hefty damages, including partial collapse, could occur in the case of stronger earthquake motions. Unfortunately, this was proven during several regional earthquakes, Skopje 1963, Banjaluka 1969, and Montenegro Coast 1979 [1].

According to EC 8 compared to the old national standard, increased seismic demand poses new challenges in verifying existing buildings' load-bearing capacity. Faculty of Civil Engineering in Sarajevo initiated a specific research program with experimental testing and computational modeling to assess masonry behavior and risk [2]. Masonry is a composite material consisting of units and joints usually filled with mortar, and it is a complex material with a 3D internal arrangement (bond). Not only varying material characteristics but also building technology can significantly influence masonry response, making modeling a demanding task for structural engineers. The low tensile strength of masonry imposes nonlinear constitutive laws in assessment of existing structures and seismic analysis of new buildings [3].

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 the following categories: micro-, meso- or macro-modeling techniques. One modeling strategy cannot be preferred since different application fields exist for each model type.

Proceedings of the 2nd Croatian Conference on Earthquake Engineering - 2CroCEE
Zagreb, Croatia - March 22 to 24, 2023

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.102

Linear elastic methods like the lateral force method or response spectrum analysis do not seem adequate to describe the quite nonlinear response of masonry and estimate global ductility.

This paper focuses on the 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 Institute for materials and structures laboratory in Sarajevo. Unreinforced unconfined masonry walls were built in full scale (233x241x25cm) and reduced scale ca. 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 [4]. Dispersion of material properties that can be found in literature is considerable [5-8]

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 the war in Sarajevo, and the floors were destroyed. Pushover analysis verifies the nonlinear behavior of newly-designed structures and the existing ones. 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, and the properties of their mutual contact were investigated separately. The compressive strength and the modulus of elasticity of the solid brick masonry were determined on the reduced wall samples – wallets. In the next step, physical models of plain (unconfined unstrengthened) masonry walls were constructed in the full scale (233x241x25cm) (Fig. 1) and the reduced scale (100x100x25cm) [2, 4].

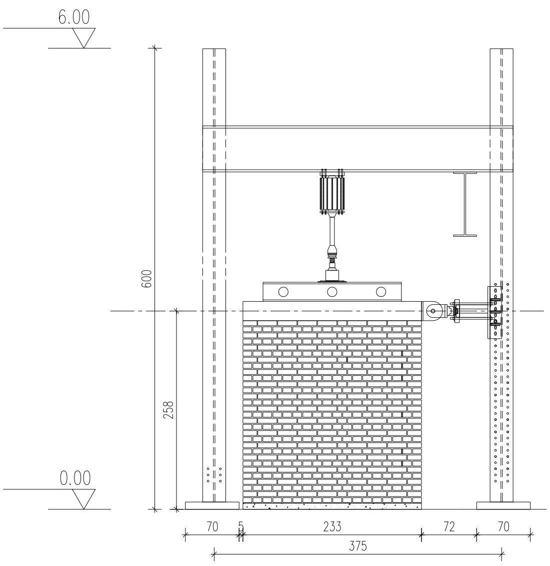


Figure 1. Test set-up.

The chosen wall geometry introduces a complex response to seismic action because the walls cannot be classified 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 a 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 levels with the

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.102

appearance of the characteristic diagonal crack pattern (Fig. 2). The measured force at failure agrees well with the ultimate load obtained 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 vertical pressure. For 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 of 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 the recently developed Engineering Masonry model [9]. The 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 can simulate compression, tensile and shear failure modes and crack in bed joints, head joints, and diagonally. The crushing, shear, and tensile behavior of the Engineering Masonry constitutive model is illustrated in Fig. 3 [10]. 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 eight 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 Fig. 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.

A comparison of hysteretic curves of the macro-model and the results of the previously described experiment are shown in Fig. 5 where one can notice a pretty good matching. Wall displacement patterns and accumulated shear strains that pertain to the major cracks are shown in Fig. 6. The damage pattern is diagonal, which complies with the experimental observations.

Proceedings of the 2nd Croatian Conference on Earthquake Engineering - 2CroCEE
Zagreb, Croatia - March 22 to 24, 2023

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.102

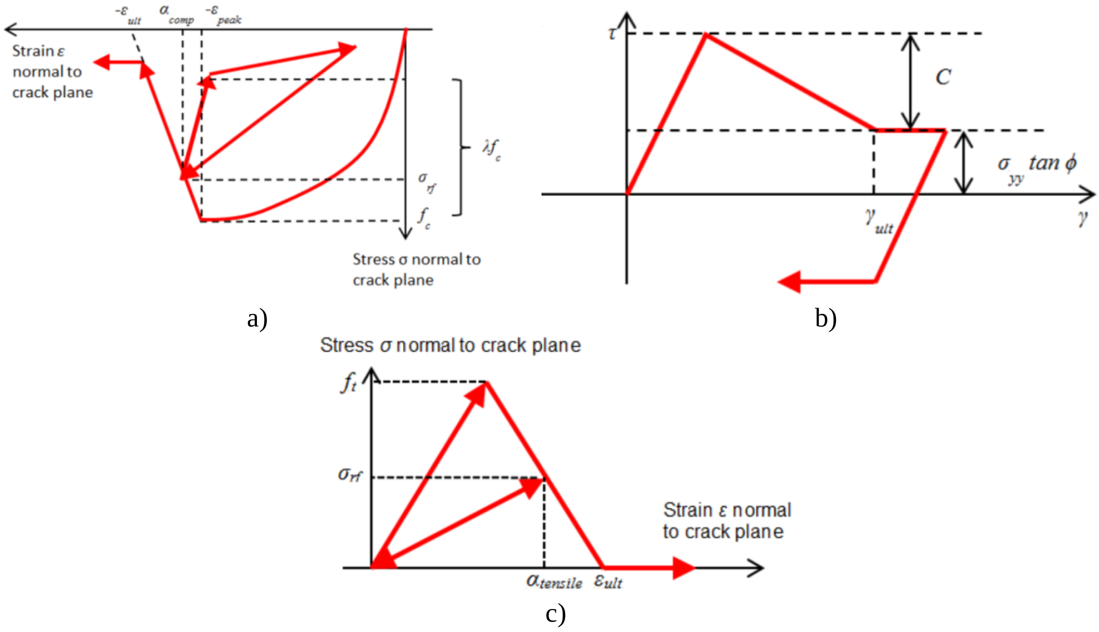


Figure 3. Engineering masonry model: a) crushing behavior, b) shear behavior, c) cracking behavior.

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

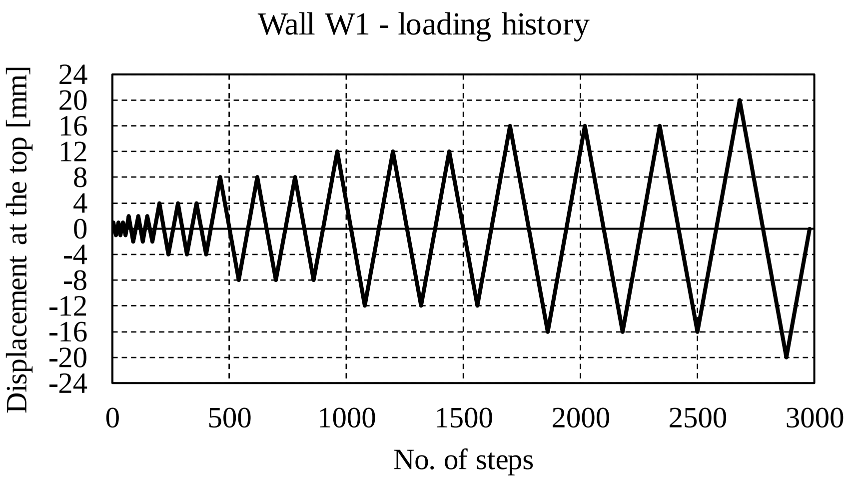


Figure 4. Loading program for wall W1.

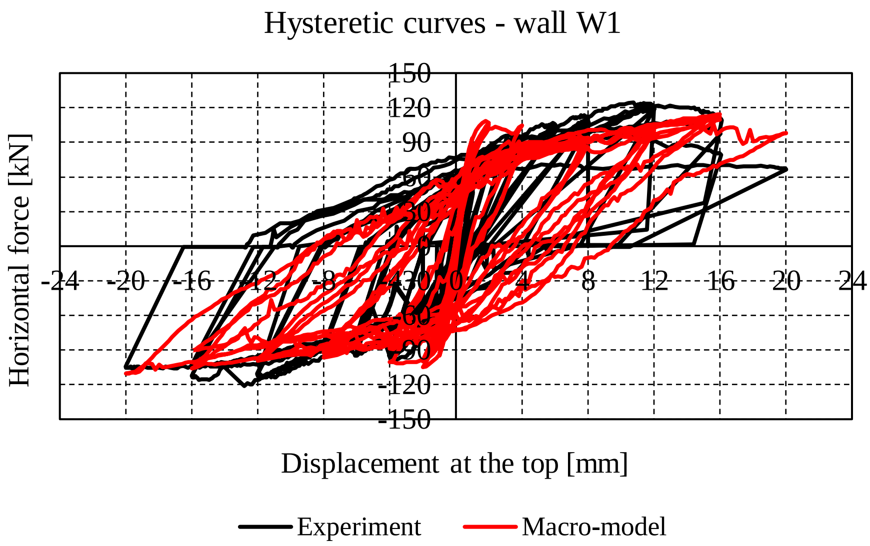
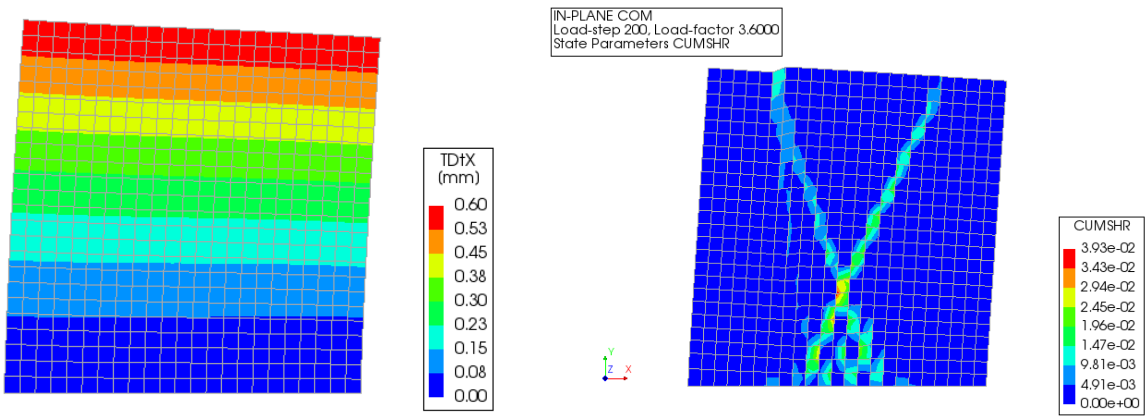


Figure 5. Comparison of hysteretic curves of the macro-model and the experiment for W1.



a) b)

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

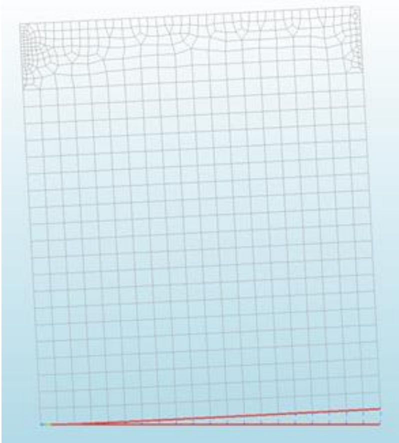
In the case of low precompression, rocking failure mode was detected on reduced-scale walls. Since the wall behaves as a rigid body, masonry was modeled as a linear elastic material in this case. Nonlinearity is concentrated in the interface element at the contact between the wall and the foundation, 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 Fig. 7.

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

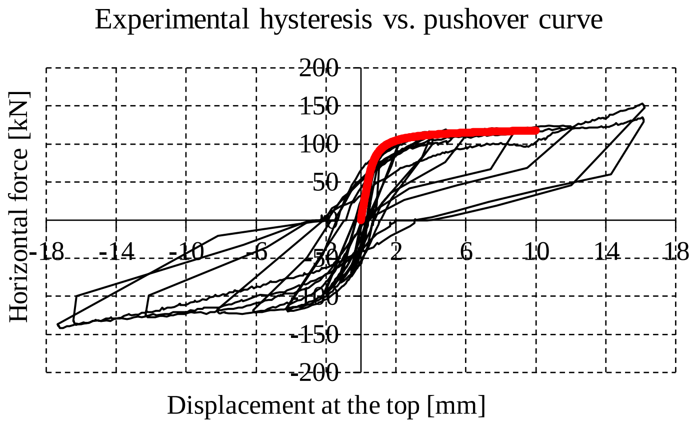
Parameter	Value
k_n	1e+06 N/mm^3
k_s	1e+06 N/mm^3
c	0 N/mm^2
ϕ	0.38 rad
f_t	0 N/mm^2

Proceedings of the 2nd Croatian Conference on Earthquake Engineering - 2CroCEE
Zagreb, Croatia - March 22 to 24, 2023

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.102



a)

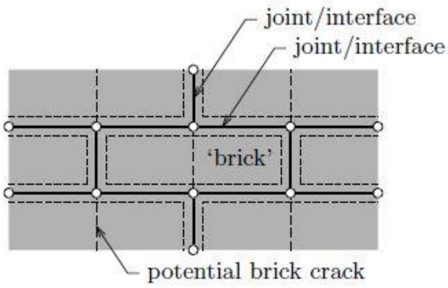


b)

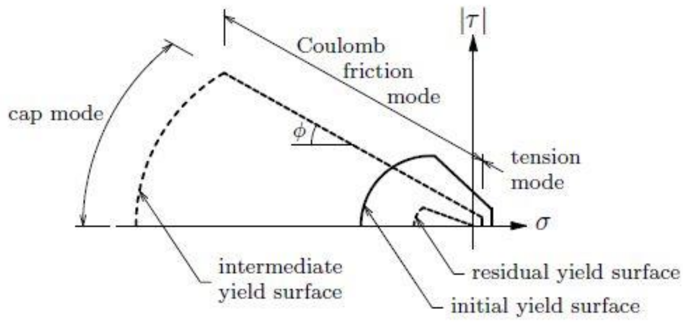
Figure 7. Results of the numerical analysis in case of rocking: a) wall displacements, b) comparison of experimentally obtained hysteresis and numerical pushover curve.

3.2 Meso-model of the tested reduced scale wall

The contact material model, also known as the “Composite Interface model,” is appropriate for the simulation of fracture, frictional slip, and 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). Fig. 8a shows the elements of a meso-model, with an additional interface element in the unit middle [9]. This interface was modeled in order to enable the cracking of bricks which was experimentally observed. A plane stress interface model was formulated by Lourenço [11]. It is based on multi-surface plasticity, comprising a Coulomb friction model combined with a tension cut-off and an elliptical compression cap (Fig. 8b). Softening acts in all three modes, and it is preceded by hardening in the case of the cap mode.



a)



b)

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

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/mm2 and Poisson ratio of 0.15. The input parameters were based on measured material properties [2] or previous numerical and experimental studies [11, 12]. Mesh, loading, and boundary conditions for the meso-model of the reduced-scale wall are shown in Fig. 9a. The resulting normal interface tractions are given in Fig. 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 Fig. 10, and it can be concluded that the curves match pretty well.

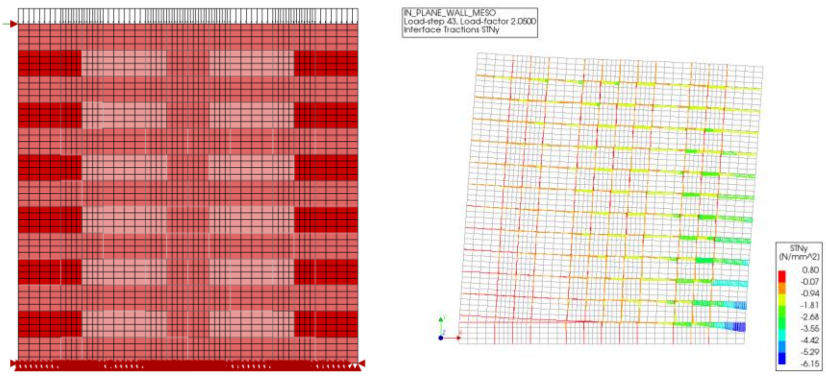
2CroCEE
22 - 24 March 2023

Proceedings of the 2nd Croatian Conference on Earthquake Engineering - 2CroCEE
Zagreb, Croatia - March 22 to 24, 2023

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.102

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.

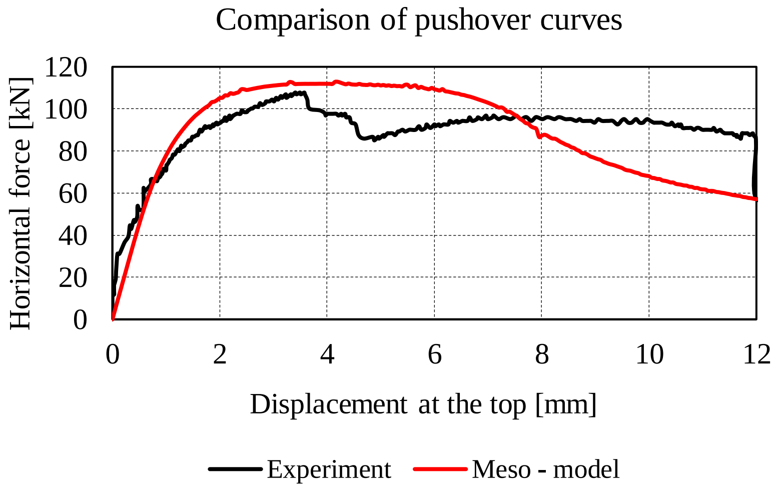


Figure 10. Comparison of pushover curves obtained experimentally and numerically.

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.102

4. Conclusion

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 the 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, resulting in a failure mode with cross-diagonal cracks. On the other hand, the macro-crack is most often diagonally localized, while the models' cracks are smeared along a specific 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 a diagonal cracking pattern develops for higher precompression levels. Depending on the loading program, the force-displacement relationship shows a pushover or a hysteretic curve. Linear force increase with displacement increment is typical for the initial phase of the obtained pushover curves. The other part of the curve is almost horizontal, implying stiffness degradation and yielding. However, in the case of rocking, yielding is not caused by material degradation but by reducing the compressed zone as imposed displacement increases. Hysteretic curves are full and significant energy is dissipated when shear governs failure. The slope of the 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.

The 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 is that the wall geometry and the boundary conditions between the bricks are correctly modeled, so the weak spots can easily be identified. The level of detail 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 significant 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.

References

[1] Hrasnica, M., Medic, S. (2012): Structural challenges of historical stone masonry in Bosnia Herzegovina. In: Ibrahimbegovic A, Hrasnica M, Zlatar M, Dolarevic S, Madzarevic M (eds) *IASS-IACM 7th International Conference on Computational Mechanics for Spatial Structures*. Faculty of Civil Engineering, University of Sarajevo, pp. 144-147.

DOI: https://doi.org/10.5592/CO/2CroCEE.2023.102

[2] Medić, S., Hrasnica, M. (2021): In-Plane Seismic Response of Unreinforced and Jacketed Masonry Walls. *Buildings*, 11(10), 472, doi: https://doi.org/10.3390/buildings11100472

[3] Lourenço, P.B. (2008): Structural masonry analysis: Recent developments and prospects, *Proceedings of the 14th international brick and block masonry conference,* 13-20 February, Sydney, Australia.

[4] Hrasnica, M., Biberkic, F., Medic, S. (2017): In-plane behavior of plain and strengthened solid brick masonry walls. *Key Engineering Materials* 747, 694-701. https://doi.org/10.4028/www.scientific.net/kem.747.694

[5] Berto, L., Saetta, A., Scotta, R., Vitaliani, R. (2004): Shear behaviour of masonry panel: parametric FE analyses. *International journal of solids and structures*, *41*(16-17), 4383-4405. doi: https://doi.org/10.1016/j.ijsolstr.2004.02.046

[6] Magenes, G., Calvi, G. M. (1997): In-plane seismic response of brick masonry walls. *Earthquake engineering & structural dynamics*, *26*(11), 1091-1112. doi: https://doi.org/10.1002/(SICI)1096-9845(199711)26:11<1091::AID-EQE693>3.0.CO;2-6

[7] Chaimoon, K., Attard, M. M. (2007): Modeling of unreinforced masonry walls under shear and compression. *Engineering structures*, *29*(9), 2056-2068. doi: https://doi.org/10.1016/j.engstruct.2006.10.019

[8] Burnett, S., Gilbert, M., Molyneaux, T., Beattie, G., Hobbs, B. (2007): The performance of unreinforced masonry walls subjected to low-velocity impacts: Finite element analysis. *International Journal of Impact Engineering*, *34*(8), 1433-1450. doi: https://doi.org/10.1016/j.ijimpeng.2006.08.004

[9] DIANA FEA (2016): *DIANA - User´s Manual*, *Material library*, Delft, The Netherlands.

[10] Schreppers, G.J., Garofano, A., Messali, F., Rots, J.G. (2016): *DIANA Validation report for Masonry modelling*, DIANA FEA BV and TU Delft, Delft, The Netherlands.

[11] Lourenço, P.B. (1996): Computational Strategies for Masonry Structures, *Ph.D. Thesis*, Faculty of Civil Engineering and Geosciences, Delft University of Technology, The Netherlands.

[12] Allen, C., Masia, M., Page, A., Griffith, M., Ingham, J. (2017): Nonlinear finite element modelling of unreinforced masonry walls with openings subjected to in-plane shear, *13th Canadian Masonry Symposium*, 4-7 June 2017, Halifax, Canada.