CALIBRATION OF A NUMERICAL MODEL FOR MASONRY WITH APPLICATION TO EXPERIMENTAL RESULTS

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A b s t r a c t
A calibration of a numerical model for analysis of masonry walls with application to experimental results is presented in this paper. The experimental results used for calibration are derived from the research project "Optimization of shape of masonry units and technology of construction for earthquake resistant masonry buildings" conducted by Bosiljkov and Tomaževič in 2005 for ZAG Ljubljana. This paper adopts micro-modelling strategy for analysis of masonry specimen by discrete element method and application of different nonlinear material models both for blocks and mortar. In order to determine the values and to calibrate the necessary material data for the used materials that were not obtained experimentally, several numerical investigations and simulations were performed. Numerical analysis of masonry walls was performed with the use of UDEC software. A comparison of numerical and experimental results as well as a comparison of the failure mechanisms is presented. With the assumed modelling strategy and numerical method, a satisfactory compliance with the experimental results regarding limit state and developed failure mechanisms is obtained.
With the results from this research and literature survey several recommendations regarding material properties necessary for numerical analysis are provided in the end of this paper.

S t r e s z c z e n i e
W artykule przedstawiono opartą na wynikach badań kalibrację modelu numerycznego do analizy ścian murowych. Wyniki badań wykorzystywane do kalibracji pochodzą z projektu „Optymalizacja kształtu jednostek murowych oraz technologia budowy budynków murowych odpornych na trzęsienia ziemi" przeprowadzonych przez Bosiljkova i Tomaževića w 2005 roku dla ZAG Ljubljana. W artykule przyjęto strategię mikro modelowania do analizy próbki muru poprzez metodę elementu dyskretnego oraz zastosowanie różnych, nieliniowych modeli materiałowych zarówno dla bloczków, jak i zaprawy. W celu określenia wartości liczbowych oraz wykalibrowania koniecznych danych materiałowych dla zastosowanych materiałów, których nie uzyskano eksperymentalnie, przeprowadzono kilkanaście badań oraz symulacji numerycznych. Analiza numeryczna ścian murowych została wykonana przy użyciu programu UDEC. Zaprezentowano porównanie wyników analiz numerycznych z wynikami z badań, jak i mechanizmów zniszczenia. Przy założonej strategii modelowania oraz metodzie numerycznej uzyskano zadowalającą zgodność z wynikami z badań, co do stanu granicznego oraz rozbudowanych mechanizmów zniszczenia.
Na podstawie wyników z tych badań oraz studiów literaturowych, na końcu artykułu przedstawiono kilka zaleceń dotyczących właściwości materiałowych koniecznych w analizach numerycznych.

K e y w o r d s: **Discrete Element Method; Failure Mechanisms; Masonry; Micro-Modelling; Nonlinear Analysis; UDEC.**

1. INTRODUCTION

This paper presents numerical simulation and identification of the dominant failure mechanisms of masonry walls with different geometrical, load and boundary conditions. Next, a numerical model for calculation of the ultimate bearing capacity by development of capacity curve of masonry walls by application of discrete element method is shown. Calculations presented in this paper are obtained by means of performing two-dimensional analysis with discrete element method and the UDEC program [1].

2. MODELLING STRATEGY

In this research the main emphasis is given to the simulation of the behaviour of masonry walls based on numerical models with the use of micro-modelling approach and use of the discrete element method. Micro-modelling of masonry is probably the most important approach to understand the behaviour and performance of masonry walls. In this paper a simplified micro-modelling strategy is adopted where expanded units are represented by continuum elements whereas the behaviour of the mortar joints and unit-mortar interface is lumped in discontinuous elements.

An accurate and precise micro-model should include all basic masonry failure mechanisms, i.e. (a) bed joint failure, (b) head and bead joint failure at low levels of the vertical stress, (c) block failure at direct tension, (d) block failure by overturning of the sample and (e) crushing of masonry by opening of gapping joints as a result of mortar dilatancy when high levels of vertical stress are present, as shown in Fig. 1 [2]. With the previously described actions, several actions can be distinguished as follows: occurrence of joint failure mechanisms (a, b), block failure mechanisms (c) and combined failure mechanisms that include joint and block failure (d, e). But still, this approach yields one open question and that is how to investigate all those actions in a single model. The assumed approach suggests that the numerical model should concentrate on all joint failure mechanisms when relatively weak joints are present and, if necessary, on potential cracks in the blocks due to tension stress in the blocks in the middle section of each block, Fig 2.

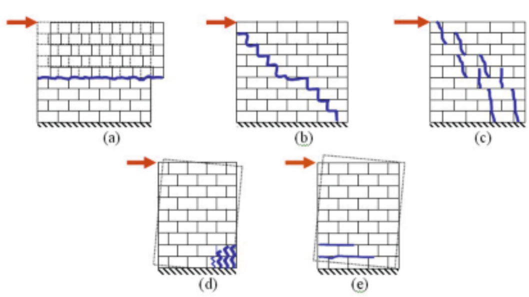

Figure 1.
Masonry failure mechanisms: Shear failure (a, b, c); Compression failure (d); Tension failure (e)

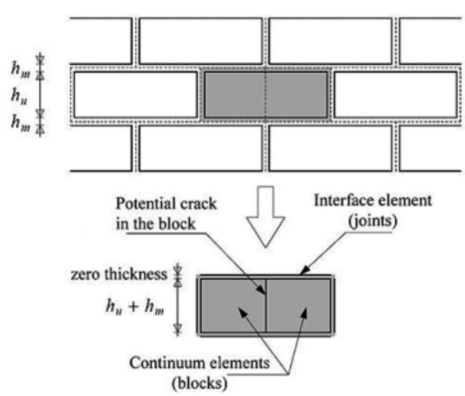

Figure 2.
Adopted modelling strategy. Blocks (u) are expanded in both directions by the thickness of the filled joint and modelled by continuum elements. Filled joints (m) and potential cracks are presented by zero thickness and interface or contact elements

3. DISCRETE ELEMENT METHOD

Discrete element method is a numerical method used for simulation of the mechanical behaviour of structures composed of particles or blocks and it is well suited for solving problems in which significant part of the deformation happens in the joints or in the contact points. This method treats the structures made of blocks that mutually interpenetrate each other through contacts. This assumption eliminates the main two difficulties naturally connected with finite element method, i.e. creation of compatible finite element mesh between blocks and joints as well as the inability of the method for remeshing in order to update the contact size or to create new contacts if relatively large movements are present. During the analysis with the use of the discrete element method, different contact types can be expected depending on the initial geometry and displacement history.

Discrete element models use explicit solving algorithm, respectively dynamic and quasi- static analyses are conducted by means of dynamic relaxation. This method uses large viscous damping for solving the differential equations of movement in order to achieve convergence to the static solution or steady failure mechanism. The method was introduced to model the arrangement of free or unattached blocks when it is not possible to compile the stiffness matrix. The main advantages of the discrete element method are: general and universal; nonlinear material behaviour and large displacements (change in system interconnections); low data storage requirements; simple programming; same algorithm for static and dynamic analysis; appropriate for parallel data processing.

4. DESCRIPTION OF EXPERIMENTAL DATA

For the requirements of the research [2] validation and confirmation of the adopted strategy for micro-modelling of masonry walls was performed by comparison and calibration of the numerical model with the available experimental results [3], [4] and [5]. In this paper special attention is given to experimental results performed by Slovenian National Building and Civil Engineering Institute (ZAG-Zavod za gradbenistvo Slovenije) from Ljubljana for the research project “Optimization of shape of masonry units and technology of construction for earthquake resistant masonry buildings”.

In the first part of the research project, the failure of the masonry specimens of the size of 1.0 × 1.5 m, (h/b=1.5) made from Group 2 units was attributed to the local brittle collapse of the units with almost no influence of the type of the execution of perpend joints on the mechanical properties of masonry. In almost all cases for laterally loaded specimens, before the formation of characteristic diagonal cracks, in the very beginning of the tests, cracks in the units occurred due to compression in the compressed toes of the walls as a result of rocking. However, the failure of the specimens was after the occurrence of shear cracks in diagonal direction. Due to suspicion that the size of the specimen may lead to high compressive stresses and thus to premature failure of the units, the work has been focused both on the investigation of the influence of the size of the specimens on the failure mechanism of shear loaded masonry walls as well as the influence of type of the head joints on the behaviour of laterally loaded masonry.

In order to determine the influence of the size of the specimens (geometry aspect ratio) on the mechanism of seismic behaviour and subsequent evaluation of seismic resistance parameters, three specimens with dimensions of 250 × 175 × 30 cm (b/h~1.5), made of the units of series BN walls with fully filled vertical joints, were first tested. Class M5 mortar was used for the construction of specimens. Each specimen was tested under different level of precompression (σ_0 = 0.6, 0.9 and 1.2 MPa, respectively) in a modified test set-up, presented in Fig. 3. Units were delivered by Wienerberger – Opekarne Ormoz, whereas the new foundation blocks and the construction of the walls were carried out by Goriske opekarne [5].

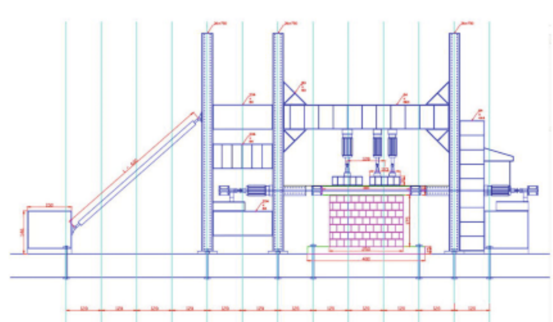

Figure 3.
Test set-up for testing masonry walls

All masonry walls were tested as vertical cantilevers fixed in the foundation blocks. The walls were subjected to constant vertical load and cyclically applied horizontal load acting on the reinforced concrete bond beams on the top of each specimen. Horizontal load was applied in the form of programmed displacements, cyclically in both directions by gradually increasing amplitudes until specimen failure. At each displacement amplitude, the loading was repeated three times. During the experiment, forces, displacements and rotations of the specimen were measured. Mechanical properties of the component materials, bricks and mortar, were determined by standardized procedures in the first part of the research project. Bricks are made from specially shaped hollow units of dimensions 24.5 × 30 × 24 cm, as shown in Fig. 4 (a).

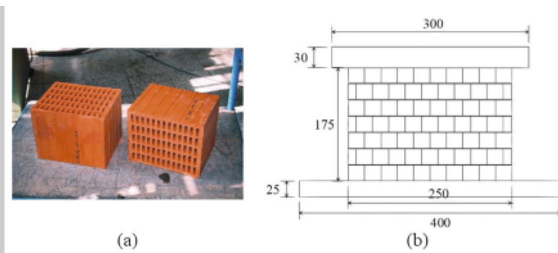

Figure 4.
(a) Masonry units for construction of walls; (b) Shape and dimensions of wall specimen BNW

The vertical load was applied by hydraulic actuators with total capacity of 2000 kN. The actuators were attached to the beam from the experimental frame and applied resultant force in the middle length of the walls. The horizontal load was applied by means of two horizontal hydraulic actuators with static capacity of 1000 kN fixed on two opposite columns from the testing frame on both sides of the wall. Experimental hysteretic force-displacement curve is shown in Fig. 10.

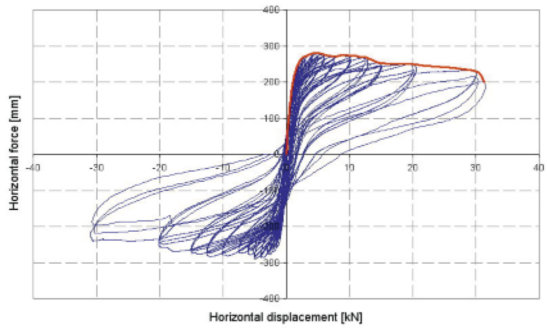

Figure 5.
Experimental force-displacement diagram for cyclic displacement controlled test

5. NUMERICAL SIMULATIONS OF MASONRY WALLS

Numerical simulation on walls denoted as BNW1, BNW2 and BNW3 was performed by first calibrating some of unknown material parameters needed in the material laws. Therefore, a numerical analysis with monotonous loading was carried out first. The geometry of the walls is shown in Fig. 6(a) while the internal element mesh with constant dilatations is shown in Fig. 6(b). In the simulations, blocks were considered as fully deformable, while the joints were presented as point contacts which mean that the blocks are connected by means of contact points between each other.

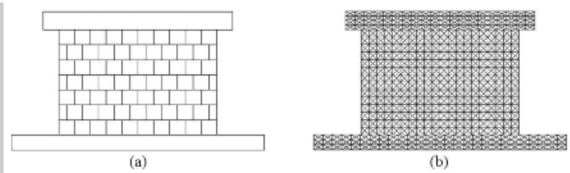

Figure 6.
(a) Wall geometry in UDEC; (b) Triangular element mesh of deformable blocks

Table 1.
Material properties for wall from BNW series

Blocks:

Properties	Unit	Elasto/Plastic with Mohr-Coulomb failure	Linear elastic, Isotropic material
		Brick	Concrete
Unit weight	(t/m^3)	1.756	2.5
Modulus of elasticity	(kN/m^2)	$5.0 \cdot 10^6$	$3.15 \cdot 10^7$
Poisson's ratio		0.2	0.2
Bulk modulus	(kN/m^2)	$2.7778 \cdot 10^6$	$1.75 \cdot 10^7$
Shear modulus	(kN/m^2)	$2.0833 \cdot 10^6$	$1.3125 \cdot 10^7$
Tension strength	(kN/m^2)	$1.150 \cdot 10^3$	/
Cohesion	(kN/m^2)	$2.300 \cdot 10^3$	/
Friction angle	$(^0)$	35.0	/
Dilatancy angle	$(^0)$	12	/

Mortar:

Properties	Unit	Elasto/Plastic with Mohr-Coulomb slip failure	
		Bed joints	Head joints
Joint thickness	(m)	0.015	0.015
Modulus of elasticity	(kN/m^2)	$0.513 \cdot 10^6$	$0.513 \cdot 10^6$
Poisson's ratio		0.1	0.1
Normal stiffness:	(kN/m^3)	$3.811 \cdot 10^7$	$3.811 \cdot 10^7$
Shear stiffness:	(kN/m^3)	$1.750 \cdot 10^7$	$1.750 \cdot 10^7$
Compressive strength:	(kN/m^2)	$5.500 \cdot 10^2$	$5.500 \cdot 10^2$
Tensile strength:	(kN/m^2)	$4.400 \cdot 10^2$	$4.400 \cdot 10^2$
Cohesion:	$(^0)$	36.87	36.87
Friction angle:	$(^0)$	6.87	6.87
Residual cohesion	(kN/m^2)	$1.540 \cdot 10^2$	$1.540 \cdot 10^2$
Residual friction angle	$(^0)$	30.00	30.00
Residual tensile strength	(kN/m^2)	0	0

The numerical model uses Mohr-Coulomb elasto/plastic model with tension cutoff. The failure envelope for this model corresponds to a Mohr-Coulomb criterion (shear yield function) with tension cutoff (tensile yield function). The shear flow rule is non-associated and the tensile flow rule is associated. Joints follow elasto/plastic model governed by Coulomb slip failure with residual strength. This law simulates joint softening behaviour while joint displacement with loss of friction, cohesion and/or tension strength in failure beginning due to shear or tension strength limit is reached. Material properties are given in Table 1. Boundary conditions of the DE model try to simulate, in the best possible way, the experimental conditions as shown in Fig. 3.

Numerical simulations were performed by:

- Variation of mechanical properties of constituent components due to limited data from experimental tests on each component;
- Variation of numerical models and nonlinear material laws in order to determine the appropriate wall behaviour and failure during the experimental test.

5.1. Wall BNW1

The wall BNW1 was loaded with constant vertical load with precompression level of 0.59 MPa. The only difference between the walls is the precompression load applied. All geometry and boundary conditions, as well as the application of the horizontal load are equal for all walls. In order to avoid any dynamic effects at the time of application of the load, the horizontal loading was applied in small steps. First, vertical precompression load is applied on the walls and then horizontal loading in several steps.

The deformed state of the DE model with ultimate limit horizontal force of 282 kN is shown in Fig. 7. Experimental and numerical failure mechanisms are shown in Fig. 8, 9 and 10. It can be seen that the predicted failure mechanism suits well with the failure obtained experimentally.

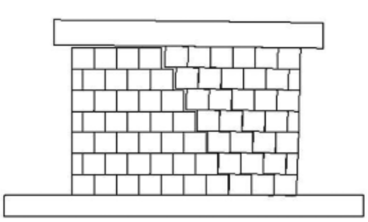

Figure 7.
Deformed block structure in DE simulation, for an imposed force of 282 kN (deformation magnified 3 times)

Fig. 11 presents comparison between experimental envelope curve and numerical capacity curve. It can be concluded that curves correspond very well and

Figure 8.
Disintegration of compressed toes in first load cycles

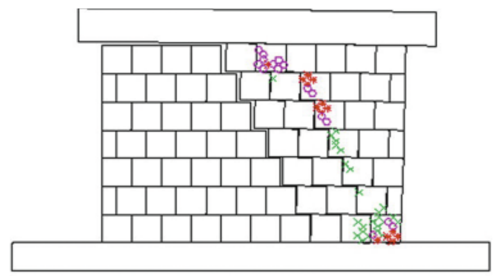

Figure 9.
Numerical failure mechanism for block. * = at yield surface; x = yielded in past; o = tensile failure

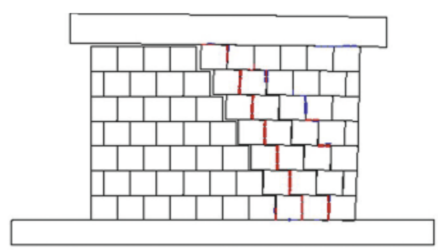

Figure 10.
Numerical failure mechanism for joint. Joint shear and slip failure

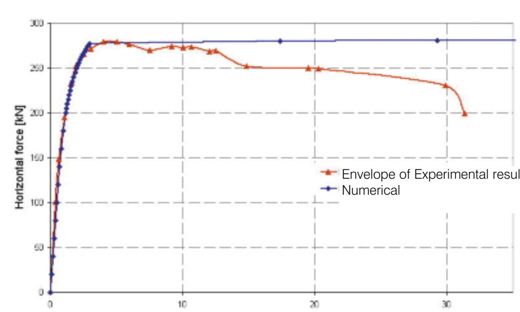

Figure 11.
Comparison of experimental and numerical force-displacement diagrams for wall BNW1

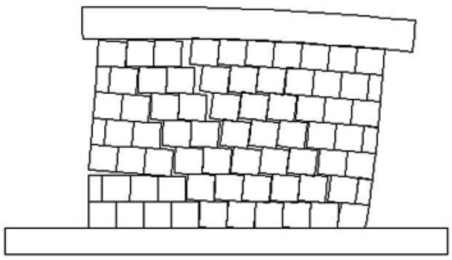

Figure 12.
Deformed block structure in DE simulation, for an imposed force of 470 kN (deformation magnified 10 times)

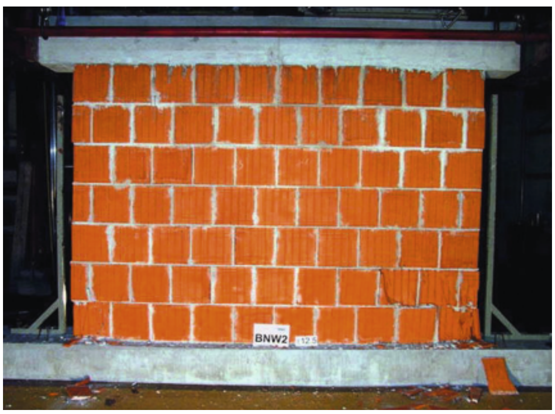

Figure 13.
Falling-off of shells in compressed toes of the specimen

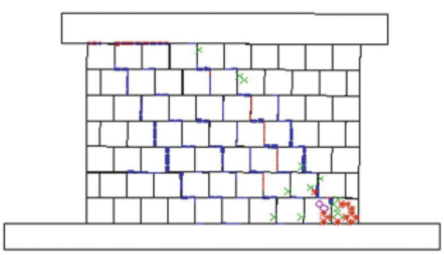

Figure 14.
Numerical failure mechanism for block and joints. * = at yield surface; x = yielded in past; o = tensile failure

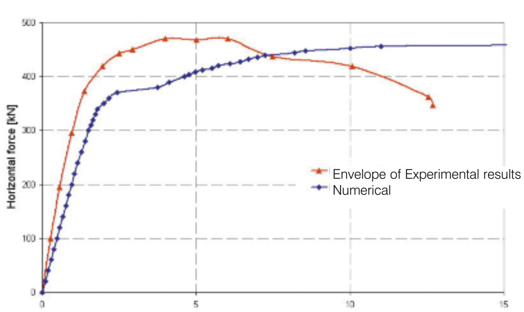

Figure 15.
Comparison of experimental and numerical force-displacement diagrams for wall BNW2

the deviation happens after reaching the ultimate capacity due to the loading method used in simulations which applies forces instead of displacements. This method implies plastic behaviour without possibility to obtain material softening after reaching peak values.

5.2. Wall BNW2

Wall specimen denoted as BNW2 is calculated using the same material and geometry properties as well as boundary conditions as in wall BNW1. The only difference between them is the vertical precompression load on the top RC beam. Fig. 12 shows magnified deformed state of the specimen for vertical load of 1.19 MPa. The ultimate limit horizontal force is 470 kN.

The experimental failure mechanism is shown in Fig. 13. In positive direction of loading, the shear cracking occurred first followed by cracking of the specimen in compressed toe. Shear cracks were passing mainly through the units. The predicted failure mechanisms for block and joint failure correspond well with the crack distribution found through the experiment. This is shown in Fig. 14.

In Fig. 15 the hysteresis envelope with limit states is compared to the calculated force – displacement diagram obtained with numerical simulation. There is not good agreement established between experiment and simulation. It is obvious that some material parameters are underestimated. A calibration of the properties is necessary in order to obtain good correlation with the experimental curve.

5.3. Wall BNW3

The wall specimen BNW3 is numerically calculated using the same material and geometry properties as well as the same boundary conditions as wall BNW1. Fig. 16 shows the magnified deformation obtained in the DE model for an imposed horizontal force of 375 kN with a precompression load of 0.89 MPa. The

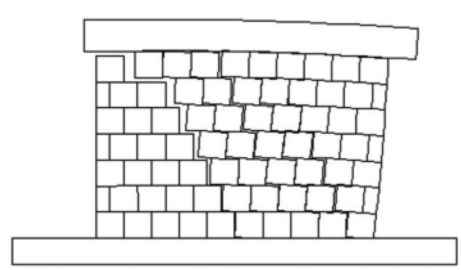

Figure 16.
Deformed block structure in DE simulation, for an imposed force of 375 kN (deformation magnified 10 times)

vertical load was applied first and then the horizontal force was applied in several load steps.

The experimental failure mechanism obtained in this test is presented in Fig. 17. First shear cracks were oriented partly along head joints and partly through the units. Cracks in compressed toes occurred almost at the attainment of maximum resistance of the wall or shortly after that. Failure of the specimen was due to shear with extensive falling off of the shells.

The predicted failure mechanisms by numerical simulation for block and joint failure correspond well with the crack distribution found through the experiment also. This is shown in Figure 18.

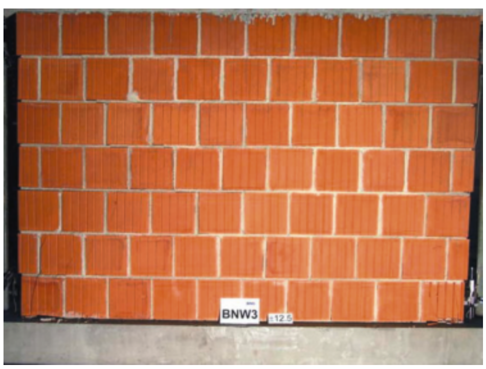

Figure 17.
Development of crack pattern

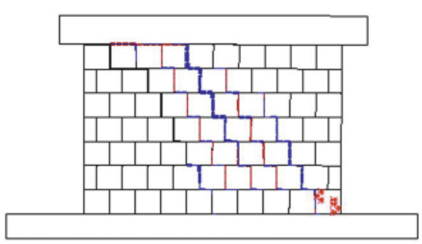

Figure 18.
Numerical failure mechanism for block and joint. * = at yield surface; - - - = joint opening and shear displacement

In Fig. 19 the hysteresis envelopes with limit states are compared to the calculated force – displacement diagram obtained with numerical simulation. There is not good agreement between experiment and calculation. It is obvious that some material parameters are underestimated.

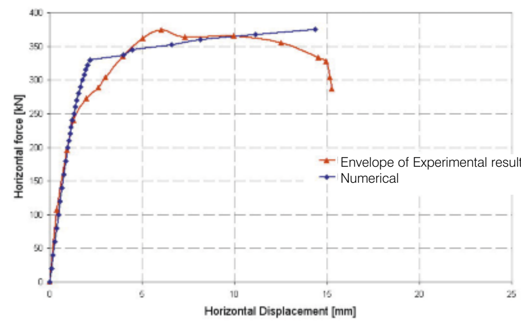

Figure 19.
Comparison of experimental and numerical force-displacement diagrams for wall BNW3

6. CONCLUSIONS

This paper presents calibration of a numerical model and calculation of capacity curves of masonry walls by using discrete element method implemented in UDEC. For the sake of controlling the possibilities of the used software and interpretation of results, a verification of the software in regards to experimental test was performed. Also, a calibration of some material parameters included in the numerical model is completed. According to the numerical simulation data one can make conclusion that UDEC software as well as the discrete element method is capable to simulate the behaviour of masonry walls under monotonous loading conditions. In addition, failure mechanism of masonry specimen walls is achieved very precisely.

In this paper only monotonous loading has been considered. Due to simplification reasons in the input data monotonous loading in several steps was applied after application of constant vertical load. This

Table 2.
Suggested material properties

Properties	Clay bricks	General purpose mortar
Tension strength	(0.03-0.1) · compression strength	(0.0-0.1) · compression strength
Cohesion	(0.0-2.0) · tension strength	(0.1-0.4) MPa
Friction angle	35-45^0	13-40^0
Dilatancy angle	0-12^0	0^0

method is good enough when loading values are close to the ultimate load which generates structure failure. Numerical analysis involving cyclic loading should present more accurate results for masonry behaviour and should be able to present softening behaviour or stiffness degradation in more detail. In order to load the specimen in cycles, one has to create numerical model that is able to include stiffness degradation explicitly. Also, it is recommended to generate a model which will be able to predict unit failure more precisely.

It has to be emphasized here that in order to determine the behaviour of masonry walls more precisely by using discrete element method it is necessary to know all material properties of the components. Recommended values for some material parameters are hard to obtain through literature survey and it is proposed to acquire them experimentally. According to the results from the numerical simulations, several suggestions for masonry clay bricks and general purpose mortar are given in Table 2.

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