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AN ATTEMPT TO DESCRIBE THE STIFFNESS DEGRADATION OF BRICK MASONRY SUBJECTED TO UNIAXIALLY CYCLIC COMPRESSIVE LOADS

PROPOZYCJA OPISU PROCESU DEGRADACJI MURU PODDANEGO CYKLICZNEMU, OSIOWEMU ŚCISKANIU

A b s t r a c t

The paper presents the results of laboratory experiments carried out on eight clay brick masonry wallettes of two types under cyclic compressive loading. Based on the results, the failure envelopes were determined, presented and discussed, as were common points stress-strain relationships for both series of specimens. The analytical description of the kinetic of stiffness degradation (with proposed appropriate formulae and experimentally determined parameters) was elaborated and proposed.

Keywords: clay brick masonry; compression; cyclic load; stiffness degradation; envelope curve; common-point curve

S t r e s z c z e n i e

W pracy zaprezentowano wyniki badań 8 murów (dwóch serii) z cegły ceramicznej pełnej poddanej cyklicznej sile ściskającej. Zależność naprężenie – odkształcenie zostało omówione dzięki znajomości krzywej punktów wspólnych i obwiedni z badań cyklicznych. Podjęto próbę analitycznego opisu procesu degradacji sztywności muru.

Słowa kluczowe: ściany murowe; ściskanie; obciążenie cykliczne; degradacja sztywności; krzywa punktów wspólnych, obwiednia

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Notation

$[D_{cr}]$	–	stiffness matrix of material after cracking;
$[D_c]$	–	stiffness matrix of material for linear-elastic behaviour;
ω	–	damage parameter, (scalar value from the range $<0;1>$);
ω_{cc}	–	damage parameter for masonry under uniaxially cyclic compression;
ε_{cr}	–	strain corresponding to first crack appearance;
ε_{pl}	–	strain corresponding to the beginning of the quasi-plastic behaviour of the cyclically loaded masonry (rapidly growth of deformations);
E_i	–	values of secant modulus of elasticity determined for *i*-th cycle;
$E_{0,1}$	–	values of secant modulus of elasticity determined for first cycle;
a_i, b_i, c_i, d_i	–	empirically determined coefficients (table values).

1. Introduction

When analysing any material in any stress state, it is necessary to know the limit values of certain parameters related to its cracking and/or failure. The characteristics of stresses (compressive, tensile or shear) as a function of strains which limit the elastic and plastic behaviour of the material, with plastic-brittle or perfectly-plastic failure mode, allows the defining of boundary or failure curves (in plane state) or surfaces (in spatial state). These functions must not only represent the parameters of the masonry components but they must also be representative of failure modes. For many years, attempts have been made to develop more and more complex failure criteria and material models dedicated mainly to the analysis of the behaviour of masonry. There is vast available literature in this topic. It is worth mentioning that a well-known and commonly used model developed by Lourenço [1] and Lourenço & Rots [2]. Generally, the model is based on the assumption of the two well-known failure criteria: Rankine criterion in tensile stresses range and Hill's criterion in compressive stresses range. Another interesting model was developed by Lubliner, Olivier, Oller & Oñate [3] which, although being developed and used for the analysis of concrete structures, has recently been more often used for the numerical analysis of masonry – also in Silesian University of Technology [4]. It is an elastic-plastic-damage model (e-p-d), commonly referred to as the Barcelona model.

In more precise numerical analyses of masonry walls subjected to cyclic loading, including cyclic compression, it is necessary to apply a material model taking into account the phenomenon of material degradation due to increasing stresses and strains. The process of material degradation is usually accounted for by the introduction of specific parameters (coefficients of mostly constant values) modifying certain entries of the stiffness matrix. As the material degradation progresses, the stiffness of masonry changes after cracking (represented by matrix $[D_{cr}]$). According to continuum fracture mechanics (equivalence of strain in cracked masonry and equivalent uncracked masonry with elastic characteristics) the changes in material can be represented in the form of a modified elasticity matrix describing the behaviour of the material in an elastic phase ($[D_c]$). A general form for the calculation of a modified stiffness matrix after the appearance of cracks in the material is as follows:

$$[D_{cr}] = (1-\omega)[D_c] \tag{1}$$

The scalar parameter (coefficient) ω takes values between 0 and 1. In cases where there is no fracture in the material, the value of ω is 0. When a fracture develops, the value of the coefficient asymptotically reaches a maximum of ω = 1 when the fracture reaches the point of failure.

The process of fracture development as a function of strains ω(ε) relationship for a given material is usually not easy to define. It depends not only on the stress and strain states and corresponding limit values but also on the parameters characterising material properties (which have a random nature) and on the loading history.

In the case of masonry walls subjected mainly to compressive loads, the modulus of elasticity is the basic property characterising the material. When compressive loads act in a cyclic manner, based on the values of the modulus of elasticity (secant modulus from the stresses range from 0 to 1/3 σ_{max}) determined for subsequent loading cycles, it is possible to determine the parameter ω_{cc} (for masonry under uniaxial cyclic compression) from the following formulae:

$$E_i = E_{0.1}(1-\omega_{cc}) \tag{2}$$

$$\omega_{cc} = \frac{E_{0.1} - E_i}{E_{0.1}} \tag{3}$$

Thus, laboratory tests of cyclic compressive loading of masonry are performed. They allow to define the behaviour of walls beyond the elastic behaviour, the characteristics of hardening (softening) laws at compression and material degradation (changes of the modulus of elasticity during loading cycles).

Cyclic loads are not only loads of seismic or paraseismic (mining) origin but also loads induced by heavy vehicular and railway traffic as well as being due to tunnel works and different types of machines and equipment located in industrial buildings. Knowledge about the behaviour of masonry under cyclic loading will allow for better protection against such effects for both newly-designed and existing buildings. The problem of the influence of repetitive loads on the behaviour of masonry has frequently been discussed in Poland by Ciesielski et al. [5]. However, as there are still no definite specifications for the selection of the mechanical parameters of masonry under complex dynamic loading, further investigation and tests in this area should be performed.

The issue of the behaviour of masonry walls subjected to cyclic compressive loading has been investigated for 20 years by Sinha et al. [6–9], AlShebani [10] and Tiwari [11]. Researchers defined the boundary curves and common-point curves with exponential or polynomial functions with variable empirical coefficients. Information from these analyses is interesting from a qualitative point of view. However, as the tests were conducted on a different type of masonry to those which are used in Central Europe, the results have no practical use, not only in Poland. Moreover, the results of these investigations were often contradictory to each other; therefore, it was considered necessary to analyse the issue based on the own tests of masonry walls made of the most popular components: ceramic bricks with cement-lime mortar.

2. Experimental tests

The experimental investigations we performed on two types of test specimens made of clay brick of class ‘15’ (f_b = 18.7 N/mm^2) and cement-lime mortar (1 : 1 : 6) class M5 (f_m = 6.8 N/mm^2). Elements of CV type were used to determine the compressive strength of masonry according to the method given in EN 1052-1:2001 [12]. Masonry specimens of an MW type were used for the testing of masonry with higher overall dimensions (according to requirements specified in standard [12]) and with most popular thicknesses used in the construction of load-bearing walls in Poland (1 brick, i.e. 250 mm). English bond (also very popular in Poland) was applied so the longitudinal joint was formed in every second layer.

Measuring frames with inductive sensors to measure deformations to an accuracy of 0.0002 mm were located on the both sides of the masonry wallette. The measurement base recording vertical and horizontal deformations was equal to 300 mm in the case of CV models and 600 mm in the case of MW series wallettes.

The tests of the CV type masonry wall specimens (smaller specimens) were performed using a hydraulic press machine with a 2000 kN range capacity while the tests of the MW masonry wallettes were carried out with using a hydraulic press machine with a maximal range of 6000 kN. Before placing the specimen in the press machine, both the top and the bottom surfaces of each specimen were levelled with a cement mortar. To eliminate friction between the surfaces of the steel heads of the machine and specimen’s surfaces special pads were used; these pads were made of Teflon (thickness of pad – 10 mm) for the CV series specimen and a double layer of polyurethane foil with graphite grease between them for the MW series specimens. The shape, dimensions of the testing specimens and the view of both types of test specimens prepared for testing is shown below in Fig. 1.

a) b) c) d)

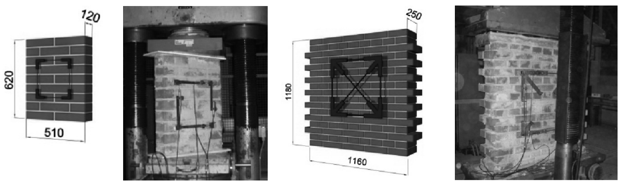

Fig. 1. The shape, dimensions and view of both types of specimens prepared for testing: a) CV type; b) MW type

The tested elements were cyclically loaded with the load increasing in each cycle. The loading velocity was equal to 2 kN/s. The first level of load for the CV series masonry specimens was equal to 50 kN and then increased by 50 kN in each cycle. The first level of load for the MW series wallettes was equal to 300 kN, followed by 600, 900 and 1200 kN, and the next cycles – until failure of the element – were increased by steps of 150 kN. During each cycle, the load was sustained for approx. 3 minutes to stabilise the state of deformations. The loading history for cyclically compressed masonry is graphically presented in Fig. 2. In total, eight specimens were tested (3 – CV type and 5 – MW type).

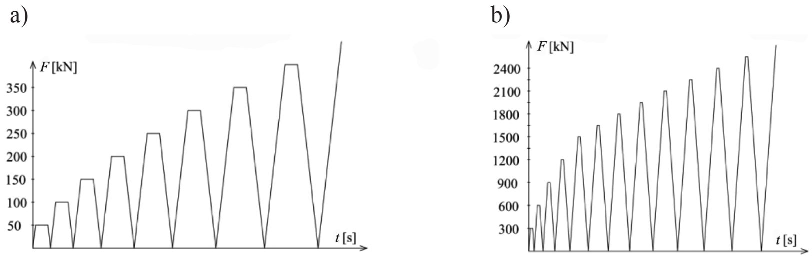

Fig. 2. Loading history for testing of specimens: a) CV series; b) MV series

3. Results and discussion

Cyclic tests allow the definition of a failure envelope of the stress–strain relationship (σ–ε) and determining the common points, i.e. the locations of the intersection of the loading curve in a given cycle with an unloading curve from the previous cycle. Each common point provides information about transformation of the material from the initial to secondary deformation states connected with the process of failure development (progressive degradation). Typically, the σ–ε relationship for masonry subjected to cyclic loading with depicted characteristic curves is presented in Fig. 3a. To eliminate small changes of the material and strength differences, further comparison of results was conducted on normalised relationships σ_i/σ_{max},–$\varepsilon_i/\varepsilon(\sigma_{max})$. Averaged, normalised failure envelopes and common-points curves for the CV and MW masonry wallettes are shown in Fig. 3b.

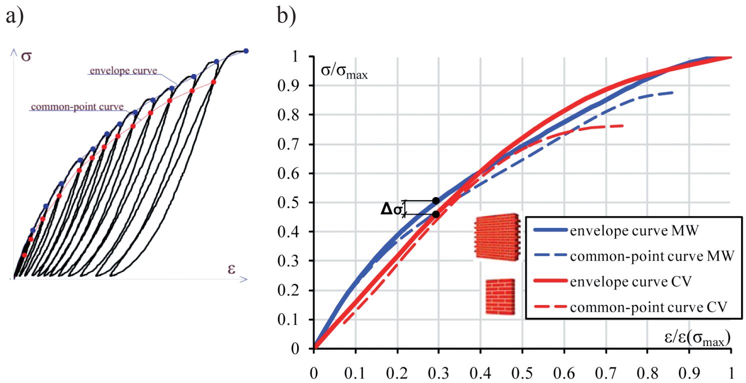

Fig. 3 σ–ε relationship for masonry subjected to cyclic loading: a) typically with characteristic curves: envelope curve and common-points curve; b) normalised for MW and CV series models

In analysing the above-mentioned relationships, it can be noticed that in the initial range of compressive stresses, i.e. up to approx. $0.6\sigma_{max}$, in case of masonry with a thickness of ½ brick (CV series), the resultant relationships are linear which signifies close-to-elastic behaviour of the masonry. On the other hand, from analysis of the relationships determined in the tests of specimens with a thickness of 1 brick (MW series), it appears that the diagrams from the very beginning up to the level of approx. 30% of σ_{max} (first visible crack appearance) have a curvilinear character (linear elastic behaviour of the material). Then, in the range from 30% to approx. 75% of σ_{max} (when this are rapidly increasing plastic-brittle damage deformations) fracture development stabilises at a similar level and the diagram has a more or less linear character. This observation can be explained by the fact that in masonry walls with greater thickness, there is an unbound longitudinal joint located in the axis of the wall which has an effect on the behaviour of that wall – this is because of the applied bonding of the elements in subsequent layers of the walls. When stresses exceed the value of 75% of σ_{max}, the process of fracture and disintegration of the material is very quick (rapid) leading to the state of.

In the presented cases, failure envelopes and common-points curves run parallel up to the level of cracking stresses (σ_{cr}). The occurrence of further plastic deformations, fractures and cracks causes visible splitting along their trajectories – the common-points curve descends rapidly.

Describing the curves with fourth-order polynomials, the following formula was used:

$$\frac{\sigma}{\sigma_{max}} = a \cdot \left(\frac{\varepsilon}{\varepsilon(\sigma_{max})}\right)^4 + b \cdot \left(\frac{\varepsilon}{\varepsilon(\sigma_{max})}\right)^3 + c \cdot \left(\frac{\varepsilon}{\varepsilon(\sigma_{max})}\right)^2 + d \cdot \frac{\varepsilon}{\varepsilon(\sigma_{max})} \tag{4}$$

Table 1

Constant coefficients of polynomial function $\sigma/\sigma_{max,i} - \varepsilon/\varepsilon(\sigma_{max,i})$ for eq. (4)

series	**curve**	**values of constant coefficients**			
		a	***b***	***c***	***d***
MW	envelope curve	–2.17	5.00	–4.47	2.64
	common-points curve	–4.19	8.14	5.57	2.72
CV	envelope curve	1.16	–2.52	0.87	1.49
	common-points curve	4.75	–8.96	4.19	0.92

As the $\Delta\sigma$ parameter characterises the kinetics of the degradation process developing the difference in compressive stresses between failure envelope and common-point curve, it was used and determined according to the scheme presented in Fig. This allows the graphical representation of the process of degradation, which is shown in Fig. 5.

Except the initial phase of loading in the case of both MW and CV masonry wallettes, the $\Delta\sigma - \varepsilon/\varepsilon(\sigma_{max})$ relationship can be described with an exponential function:

$$\Delta\sigma = a_1 \cdot e^{b_1 \cdot \frac{\varepsilon}{\varepsilon(\sigma_{max})}} \tag{5}$$

The relationship ($\Delta\sigma - \varepsilon_i/\varepsilon(\sigma_{max})$) presented in Fig. 4 indicates that in the case of small masonry specimens with thickness of ½ brick (CV series models), the chain process of material degradation begins for strains of a magnitude over approx. 35% of ultimate strains

($\varepsilon(\sigma_{max})$). This corresponds to the situation of the appearance of the first cracks. It is slightly different in the case of masonry walls with a thickness of 1 brick (masonry wallettes of MW series) with a longitudinal joint in every second layer. The first zone of visible fracture development and stiffness degradation was observed in the range of strains from almost the beginning until approx. 35% of ultimate strains ($\varepsilon(\sigma_{max})$). The degradation process then stabilises up to a level of strains of approx. 0.75 ($\varepsilon(\sigma_{max})$) when a sudden increase of strains is observed. Probably at this moment, the plastic zone with some fractures and material internal damage is reached. Intensive strain development begins, this is caused by both the development of unrecoverable plastic strains and the progressive development of cracking (fracture). This situation took place up to the state of failure of the masonry.

Table 2

Constant coefficients of exponential function $\Delta\sigma - \varepsilon/\varepsilon(\sigma_{max})$ for eq. (5)

series	constant coefficient values	
	a_1	b_1
MW	0.0041	4.91
CV	0.0037	3.53

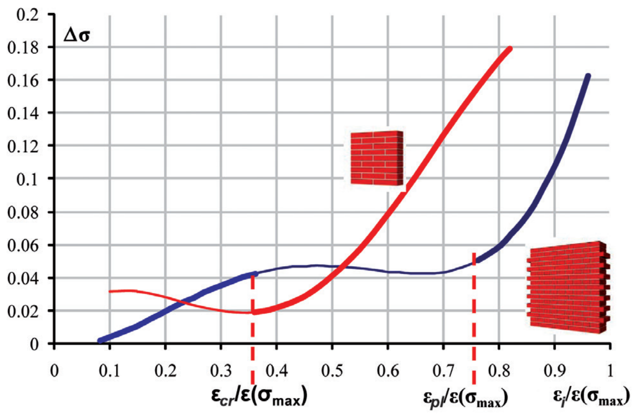

Fig. 4. Resultant $\Delta\sigma - \varepsilon_i/\varepsilon(\sigma_{max})$ relationships for **MW** and **CV** masonry specimens

In the case of smaller specimens (CV series) based on the equations (3) and (5), the process of stiffness degradation as a function depending on stress (σ) and strains (ε) state, can be expressed as:

$$\omega_{cc} = F(\sigma, \varepsilon) = c_1 \cdot \frac{\varepsilon}{\varepsilon(\sigma_{max})} \tag{6}$$

where value of coefficient c_1 is presented in Table 3.

The function of the failure progress during cyclic loading $\omega_{cc} - \varepsilon/\varepsilon(\sigma_{max})$ of CV masonry wallettes is shown in Fig. 5a; for the MW series masonry wallettes, refer to Fig. 5b.

The situation becomes more complicated in the case of masonry wallettes with a longitudinal joint in every second layer (MW series specimens). As is shown in Fig. 5b, the $\omega_{cc} - \varepsilon/\varepsilon(\sigma_{max})$ relationship has a similar to linear character only in the initial phase of loading. After exceeding the specific value of strain (ε_{cr}) which corresponds to the cracking strain (in the presented tests, this corresponded to a level of strain of approx. 0.35 $\varepsilon/\varepsilon(\sigma_{max})$), the curvature of the function changes. From that point up to about 0.75 $\varepsilon/\varepsilon(\sigma_{max})$, stabilisation of the fracture development was observed. A sudden increase in a fracture parameter was observed after exceeding the value of strains – this can be taken as corresponding with plastic strains (here, this level was equal to approx. 0.75$\varepsilon/\varepsilon(\sigma_{max})$). To better visualise the $\omega_{cc} - \varepsilon/\varepsilon(\sigma_{max})$ relationship, characteristic points up to the appearance of cracking are marked as filled points while they are marked as empty points after cracking.

a) b)

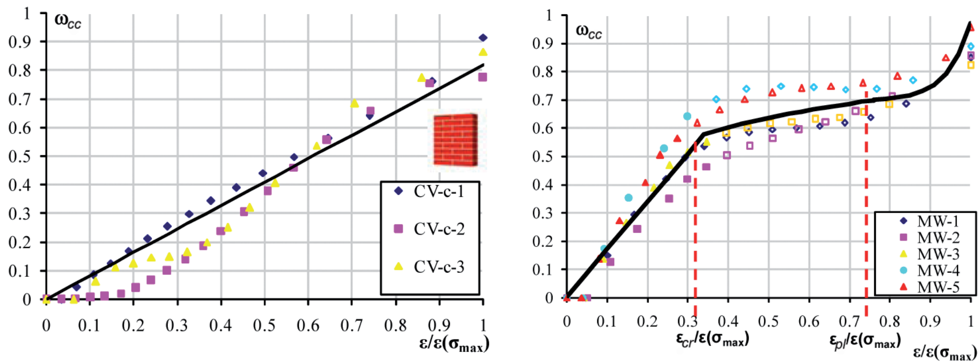

Fig. 5 $\omega_{cc} - \varepsilon/\varepsilon(\sigma_{max})$ relationship: a) for **CV** masonry specimens; b) for **MW** masonry specimens

The proposed function describing the material degradation process in the case of the masonry wallettes with a longitudinal joint (models of MW series) was defined depending on the value of strains as a continuous interval relationship – a combination of linear, logarithmic and power functions:

$$\omega_{cc} = F(\sigma,\varepsilon) = \begin{cases} b_2 \cdot \dfrac{\varepsilon}{\varepsilon(\sigma_{max})} & \varepsilon \le \varepsilon_{cr} \\ \sigma(\varepsilon_{cr}) + b_3 \cdot \ln\dfrac{\varepsilon}{\varepsilon(\sigma_{max})} + d_2 & \varepsilon_{cr} < \varepsilon \le \varepsilon_{pl} \\ \sigma(\varepsilon_{pl}) + b_4 \cdot \left(\dfrac{\varepsilon}{\varepsilon(\sigma_{max})}\right)^{d_3} & \varepsilon < \varepsilon_{pl} \end{cases} \tag{7}$$

Table 3

Constant coefficients used in eq. (6) and (7)

constant coefficient values					
c_1	b_2	b_3	b_4	d_2	d_3
0.82	1.70	0.15	0.27	0.16	17.00

The proposed functions [6 & 7] were graphically presented with continuous lines in Fig. 5. The differences in the character of the $\omega_{cc} - \varepsilon/\varepsilon(\sigma_{\max})$ relationship between the CV and MW masonry wallettes correlate to the differences observed in $\Delta\sigma - \varepsilon_i/\varepsilon(\sigma_{\max})$ relationships.

4. Summary and conclusions

The progressive degradation process of masonry walls occurs as a result of low frequency cyclic compression. Investigations into the relationship between the failure envelope and common-points curve is necessary and useful for the description of the yield surface changes during the loading process. Stresses below the common-points curve cause only reduced plastic strains in masonry while stresses over that curve lead to the formation of unrecoverable plastic strains. Common-points curve up to the level of σ_{cr} (stresses corresponding to the first crack appearance) run parallel below the envelope curve for cyclic tests. After exceeding cracking stresses, a sudden change in the curvature of the common-point curve is observed due to the degradation of masonry caused by cracking and fracture. Based on the result of the presented tests and analytical analysis, the following conclusions may be formulated:

- the differences in compressive stresses between the failure envelope and the common-points curve characterises the process of fracture development of masonry elements subjected to uniaxial cyclic compressive loads in a vertical direction;
- knowledge of the values of the modulus of elasticity in subsequent cycles allows determining the fracture (damage) coefficient (ω) described as a function dependent on the stresses and strains in a masonry wall as opposed to typically taking this parameter as a scalar value from the range <0;1>. This approach may be useful in the case of the numerical modelling of masonry walls with a internal longitudinal joint (e.g. when the thickness of the wall is equal to or higher than the length of the masonry unit);
- fracture (damage) coefficient in case of the masonry with longitudinal joints (e.g. the most popular English bond or Flemish bond) subjected to cyclic compressive loads in direction perpendicular to bed joints may be expressed using the continuous interval function proposed in eq. (7). In the first range from 0 to achieving the cracking strain value ε_{cr} (in presented tests corresponding with 0.35 $\varepsilon/\varepsilon(\sigma_{\max})$), the fracture coefficient was described with a linear function. Then, in the range from ε_{cr} to $\varepsilon_{\mathrm{pl}}$ (0.35 $\varepsilon/\varepsilon(\sigma_{\max})$ to 0.75 $\varepsilon/\varepsilon(\sigma_{\max})$ it is described with a logarithmic function. In the third range, from ε_{pl} (0.75 $\varepsilon/\varepsilon(\sigma_{\max})$), up to failure, the degradation process was described by the power function;
- the proposed mathematical description of the fracture (damage) coefficient function $\omega_{cc} = F(\sigma,\varepsilon)$ has not too universal character; this is due to the limited number of tested specimens and using only one type of masonry units and mortar. Further investigations, both experimental and analytical, are necessary.

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