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AKADÉMIAI KIADÓ

On the Buckling of the No-tension Material Masonry Column

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ORIGINAL RESEARCH ARTICLE

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SUMMARY

Masonry columns, subjected to eccentric compression, crack due to tension if the eccentricity is larger than the size of the core of the section. Previous studies have assumed that the cracks have so small spacing that the cracked tension side can be neglected during the analysis.

The critical load can be determined using this assumption. However, experimental experience has shown that the cracks have large spacing, approximately equal to one and a half times the cross-section height. Therefore, the crack-free parts between the cracks influence the lateral deflection and the critical load. Considering the above-mentioned phenomenon, we determined the elastic critical buckling load of the cracked masonry column.

KEYWORDS

masonry column, buckling, no-tension material

INTRODUCTION

Masonry is one of the oldest building methods, and its application possibilities are controlled by thousands of years of experience. In Hungary, the masonry walls were not yet calculated in the 19th century. The design was based on the rules of thumb. The detailed analysis of masonry columns became possible after Bloch (1930), Czakó (1916) accomplished the masonry element compressive strength experiments in 1916. Until 1956 masonry structures were calculated only for centric compression according to the Hungarian masonry structure standard (Csonka et al. 1950). Design for eccentric compression of masonry walls and columns was first introduced in the MSZ standard of 1957 (MSZ 15023 1957), following the experimental and theoretical

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models of Andrejev's book (Andrejev 1953). It did not even arise at the time that the behaviour of an inhomogeneous no-tension material differs from a homogeneous one.

The question was examined experimentally by Haller in 1949 (Haller 1949), considering non-linear stress distribution over the cross-section. However, Yokel's 1971 study (Yokel 1971), based on Angervo's (Angervo 1954) and Royen's (Royen 1937) papers, brought the breakthrough. Yokel neglected the tensile stress on the cross-section due to the possibility of cracking. He showed, based on experimentally verified, iteratively solved differential equation solution, that the eccentricity reduces the resistance of the masonry column. Yokel assumed that the cracks are developing in the weak, closely spaced mortar bed joints, which is why the effect of tension can be neglected. Based on these results, simplifying Royen's model, Dulácska (Dulácska 1972; Bölcskei – Dulácska 1974; Dulácska 1979, 1983, 1990) was able to derive a closed-form solution for the linear buckling load of the linear elastic, no-tension masonry column, using the state properties of the mid-cross-section along the column height.

Recently many researchers examined the problem. It is worth mentioning the works of Mojsilovic (Mojsilovic – Marti 1994), Sabha (Sabha 1999), Schlegel (Schlegel 2004; Schlegel – Rautenstrauch 2000, 2005), Bakeer (Bakeer 2014, 2016; Bakeer – Christiansen 2017) and Kirtschig (Kirtschig 1976). In addition to these references, we must also cite the following literature: Andrejev (1953), *Beanspruchungen und Tragmodelle* (2018), Fódi (2011), Hendry (1998), Turkstra (1971), Yokel (1971).

Hendry provides an extensive, thorough, but not complete, bibliography on the subject in the book chapter titled "The Strength of Masonry Compression Element" (Hendry 1998).

A good review of the problem can be found on the Internet in *Beanspruchungen…* (2018) and in Como's book (Como 2015).

NOTATIONS AND ASSUMPTIONS, USED IN THE PAPER

Notations

b: the thickness of the column cross-section
h_t: the height of the column cross-section
A_0, A_c: cross-section area of the column, compressed area of the column, respectively
e_0: eccentricity of the compressive force
e_d: calculated eccentricity
e_{init}: initial eccentricity (imperfection)
a: the distance between the point of action of the eccentric force and the compressed extreme fiber
w: displacement of the column mid-cross-section
l_0: buckling length of the column (he, according to Eurocode-6)
H: width of the experimental specimen
D: height of the experimental specimen
N: compressive force acting on the column
N_t: the characteristic value of the plastic resistance of an eccentrically compressed cross-section
$N_{E,kr}$: critical, Euler buckling load of an elastic column
$N_{1,kr}$: critical buckling load of a continuously cracked, elastic column
$N_{h,kr}$: critical buckling load of a discretely cracked, elastic column

$N_{m,k}$: critical buckling load of the elastic-plastic column
M: the bending moment
E_{lt}: the modulus of elasticity
v: the Poisson ratio
K: the bending stiffness
R: the radius of curvature
ρ: the curvature M/K = 1/R
ρ_1: the curvature of the continuously cracked elastic column
ρ_2: the curvature of the crack-free elastic column
1, 2: the index of continuously cracked and the crack-free column
h: index of the discretely cracked column

Assumptions
- The material of the column is linear-elastic and perfectly plastic or rigid-plastic to determine the failure load.
- The buckling length of the cracked column is equal to the buckling length of the crack-free column.
- According to the rigid-plastic material model, the real, non-linear plastic stress distribution for compression is considered constant.
- The variable eccentricity of the column is considered to be constant, corresponding to the mid-cross-section, assuming in favour of safety.

EXPERIMENTAL EXPERIENCES

All the methods mentioned above assumed that the bed joint mortar has smaller tensile strength than the masonry unit or stone block, which is why the masonry column cracks for tension due to eccentric compression. In reality, the adhesive strength between the mortar and the masonry unit is much lower than the tensile strength of the mortar. Therefore, the crack develops between the mortar and the masonry unit, as shown in *Figure 1.*

Figure 1. Opening crack between mortar and brick

The researchers also assumed that the crack develops closely spacing, i.e., the column is continuously cracked; therefore, the cracked tension side of the column is like not being there. This model is shown in *Figure 2.a.*

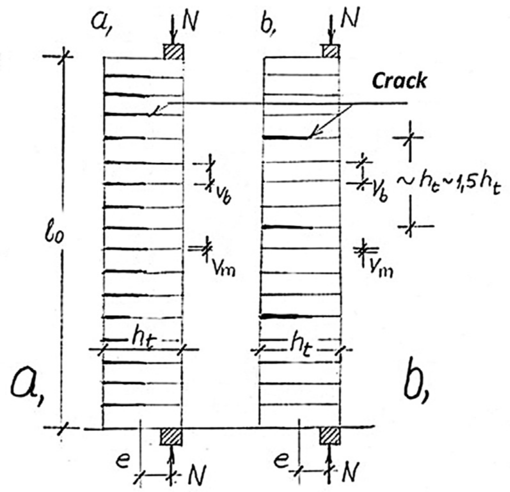

Figure 2. a) The continuously cracked masonry column (assumed)
b) The discretely cracked masonry column (according to experiments)

However, the experimental experience shows that the design according to the continuously cracked model is safe but does not represent reality. That could be observed in experiments that the crack spacing is about one and a half times the cross-section height (1.5 h_t). This is illustrated in *Figure 2.b.*

In our opinion, this occurs because a crack appearing changes the stress state around the crack. If the crack spacing is one and a half times the cross-section height or less, then the tensile stress is far less than the tensile strength; therefore, no crack can develop.

The consequence is that the cracked cross-section defines the bending rigidity in the vicinity of the crack. In between the cracks, the bending rigidity is according to the crack-free cross-section.

We proved experimentally and by finite element calculation that if the crack spacing is one and a half times the cross-section height or less, no tensile stress causing cracking may occur on the tension side between the cracks. This is shown in the next section

INVESTIGATION OF THE STRESS DISTRIBUTION OF THE BLOCKS BETWEEN CRACKS

It would have been best to do the experimental tests on masonry units. However, the large scatter of the masonry unit's strength and modulus of elasticity would make it impossible to get good results. Therefore, we chose to make the specimens (cube and prism) from steel to approximate the material's assumed linear properties with a load below the elastic limit. By rotating the block, we could also measure the stress distribution for large and small crack spacing of masonry blocks. The experiments on the cube are for the mid-size block. The experimental elements are shown in *Figure 3.*

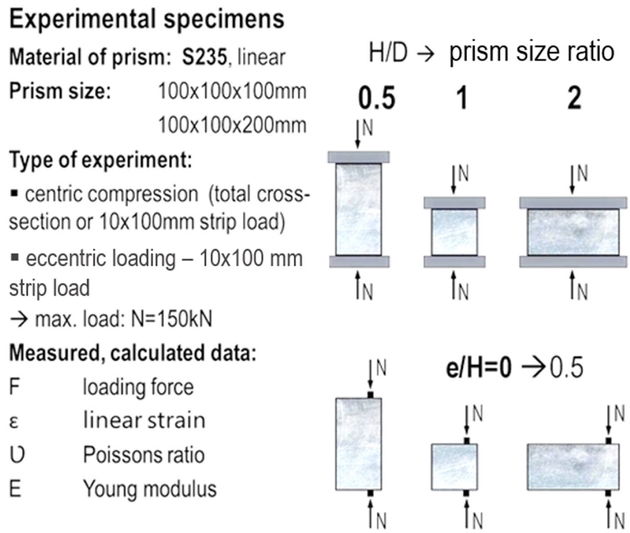

Figure 3. Steel specimens to measure stress distribution due to eccentric load

The experimental setups are shown in *Figure 4*. The width of the steel specimens is denoted by *H* and the height by *D*. The specimen sizes are as follows: 100×100 mm cube and 100×100×200 mm prism. The eccentric line load was applied to the specimens through a 10×10 mm steel strip, which was welded to the loading platen. The compressive force was N = 150 kN during the test. Based on the experiments, the steel modulus of elasticity is E = 20600 kN/cm^2, and the Poisson's ratio is υ = 0,34.

In the experiments, the strain at extreme fibres was measured by strain gauges to calculate the stresses. The stresses were also calculated by finite element method using AXIS VM software (AXIS VM 2020).

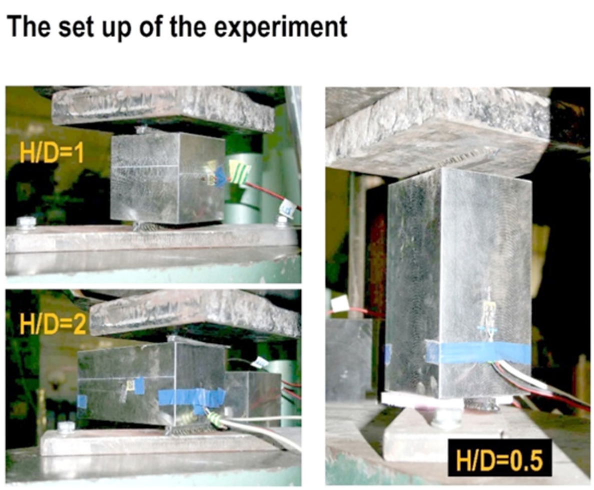

Figure 4. The tested steel specimens between the loading plates

The results of the calculation for different eccentricities are shown in *Figures 5–6*, to see how the stress diagrams are changing.

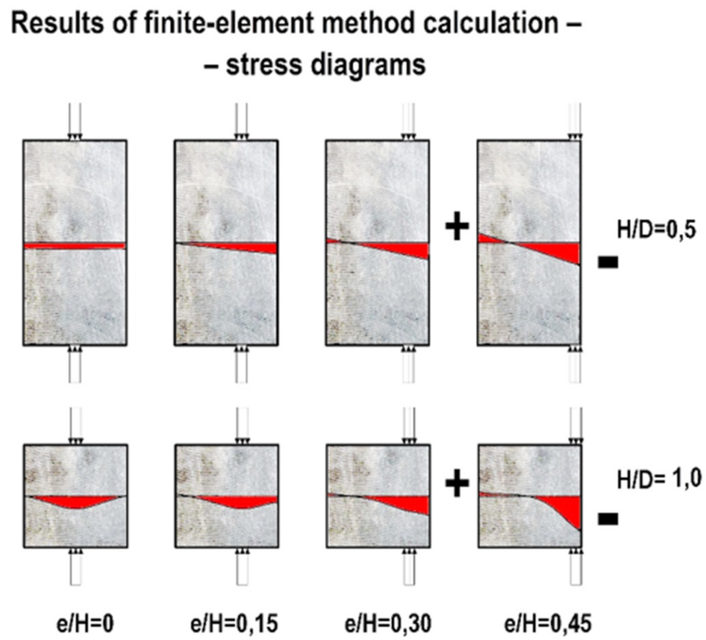

Figure 5. The standing prism and the cube normal stress for different e/H eccentricities

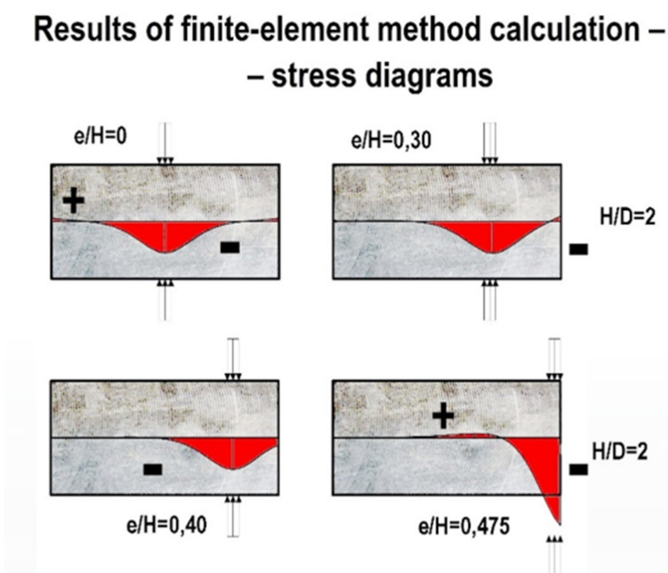

Figure 6. The horizontal prism normal stress at different e/H eccentricities

A comparison of the calculated values with AXIS software (AXIS VM 2020) and measured strains is shown in *Figure 7*. The solid lines are for compressive strain and the dashed lines are for tensile strain, or the latter shows the strain at the less compressed side. The measured and calculated results on the compressed side are slightly different but have the same trend. There may be several reasons for the difference (e.g., strain gauge, experimental setting, etc.), but this is not addressed here because it is unimportant.

The calculated and measured strain and stress results show clearly that the eccentric force does not cause tension at all in the case of the 2/1 ratio horizontal prism and in the case of the 1/1 ratio cube (i.e., there is no possibility of cracking). However, in the case of a 2/1 standing prism, considerable tensile stress develops, so there is a possibility for cracking. These facts explain why the experimentally observed cracks of eccentrically compressed masonry columns do not show the assumed continuous or near continuous crack distribution (i.e., crack at each

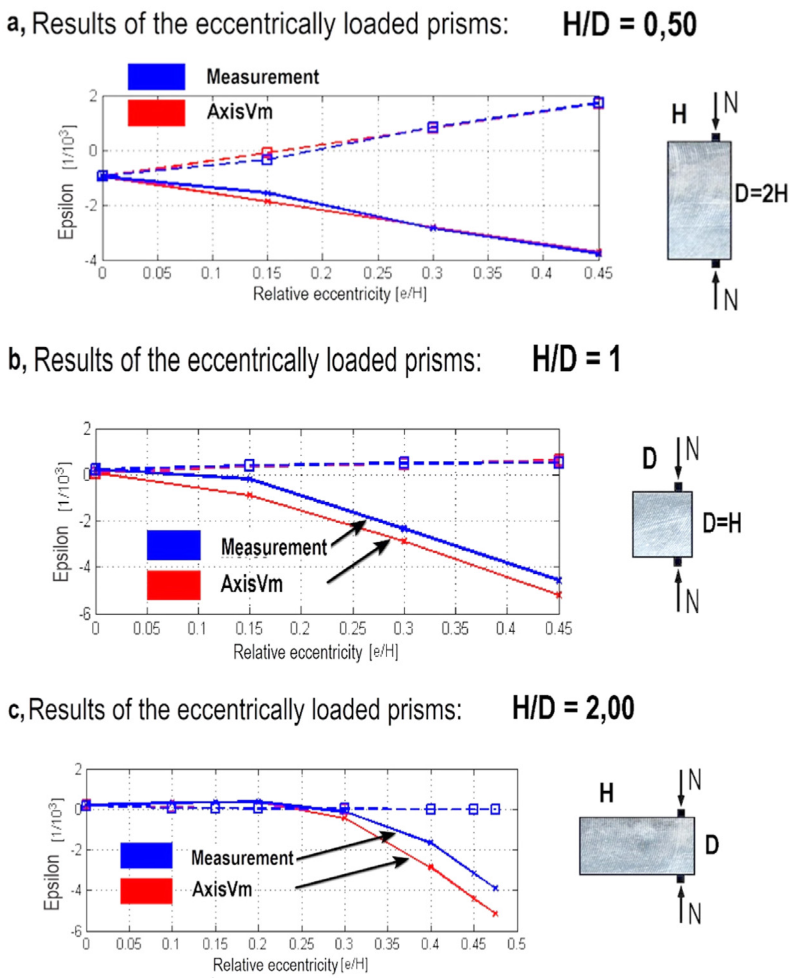

Figure 7. Comparison of the calculated and the measured strains

mortar bed joint). However, the observed discrete crack spacing is between the 1/1 ratio cube and the 1/2 ratio prism, approximately around at crack spacing to cross-section ratio 1/1.5.

That is, the masonry column has cross-sections, considerably far away from each other, that are cracked and have parts that are not cracked.

It is a consequence of all of these that the assumption of the continuous cracking results lower limit for the bending stiffness and so for the buckling strength of cracked masonry column, and its real buckling load *must be larger*. We discuss that in the following.

ANALYSIS OF THE CRACKED MASONRY COLUMN

Buckling of a perfectly elastic, no-tension material column is due to the continuous reduction of the compressed area of the cross-section during the increase of the lateral deflection, and a limit point type buckling defines the load-bearing capacity, although the stress maximum is far less than the strength of the material. If the material of the column is elastic-perfectly plastic, then there may be such eccentricities when the load-bearing capacity is defined by material failure instead of elastic buckling. Therefore, $\sigma \leq 1.3–1.5\ \sigma_{\text{rupture}}$, condition must also be satisfied in the case of a compressed column because the stress, 30–50% larger than σ_{rupture}, causes material failure in the cross-section. When the stress σ for elastic buckling and material failure is equal, the column's failure mode changes from buckling to material failure. The buckling resistance of the column, as a function of the slenderness, is a hyperbola, while the material failure is characterized approximately by a shallow parabola. In order to study the behaviour of the masonry column, we must know the linear elastic buckling load capacity. As it was pointed out in the Introduction, it is possible to calculate the elastic buckling resistance $N_{1,\text{kr}}$ of the continuously cracked masonry column, neglecting the effect of the cracked tension part of the cross-section, similarly to Bölcskey – Dulácska (1974), Dulácska (1979, 1983), Royen (1937), Yokel (1971), Dulácska – Tajta (2009). However, the column resistance may also be determined considering the crack-free, total cross-section, $N_{2,\text{kr}}$.

Assuming that the crack spacing, according to the experiments, is equal to one and a half times the height of the cross-section, then the average curvature of the column can be determined as the average stiffness K of the continuously cracked and the crack-free column.

THE CRITICAL LOAD OF THE CONTINUOUSLY CRACKED COLUMN, $N_{1,KR}$

The curvature ρ1 of the mid cross-section and the stiffness K_1 and the load N_1 according to Bölcskey – Dulácska (1974) are: $\rho_1 = \sigma/3E(a–w)$, and $K_1 = 4{,}5Eb\ e \cdot (0{,}5h–e–w)^2$, and the compressive force in the function of the lateral displacement is given by $N_1 = 1{,}5b \cdot (a–w)^2$. The notations are shown in *Figure 8*.

The lateral displacement is assumed to vary according to the sine function $w = \rho_1\ (l_0/\pi)^2$. Using these equations, the N_1 compressive force can be determined as a function of lateral displacement w: $N_1 = (4{,}5\pi^2\ ba^3\ [(w/a)–2(w/a)^2+(w/a)^3]/(l_0^2)$. The function has a maximum at $w = a/3$. This limit point defines the critical load of a continuously cracked column: $N_{1,kr} = 2\pi^2\ Eba^3) / (3l_0^2\ )$. The equation can be written as a function of the eccentricity e_0 instead of using variable a.

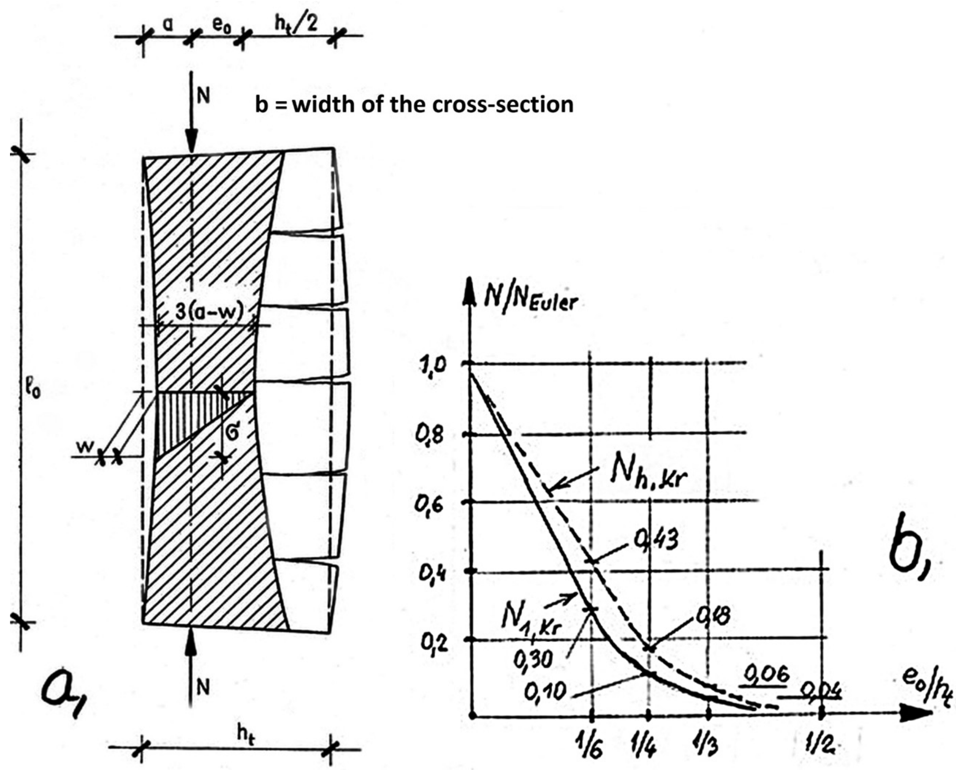

Figure 8. The eccentrically compressed column, cracked on the tension side:
a) Model of the column; *b)* The critical load as a function of the eccentricity e_0

Figure 8.a shows the model of the cracked column. In *Figure 8.b*, the solid line shows how the critical load of the continuously cracked column $N_{1,kr}$ varies as a function of the eccentricity e_0. The figure clearly shows that the increase of eccentricity significantly reduces the critical load.

THE CRITICAL LOAD OF THE DISCRETELY CRACKED COLUMN, $N_{H,KR}$

According to the experiments, the eccentrically compressed masonry column cracks discretely, and the spacing between the cracks is approximately one and a half times the cross-section height.

In the vicinity of the crack, the stiffness is K_1, which is the stiffness of the continuously cracked column. The crack-free column stiffness in between the cracks is $K_2 = E_{lt}\, bht^3/12$.

Using the two stiffness parts, a replacement stiffness K_h must be calculated to determine the Nh load resistance of the column. The replacement stiffness is $K_\mathrm{h} = 2K_1K_2/(K_1+K_2)$.

(Do not forget that K_1 is a function of both the eccentricity e_0 and the displacement w.)

The load resistance of the discretely cracked column N_h is: $N_h = (\pi^2\, wK_h)/(e_0 l_0^2)$.

(The lateral displacement defining the limit point is shifted slightly from $w = a/3$. Since this difference is small, it can be neglected as an approximation.)

The load resistance of column N_h is the function of the displacement w and the eccentricity e_0.

Figure 9 shows the variation of the N_h load resistance of the column, scaled by Euler's critical load, as a function of lateral displacement w for three different eccentricities e_0.

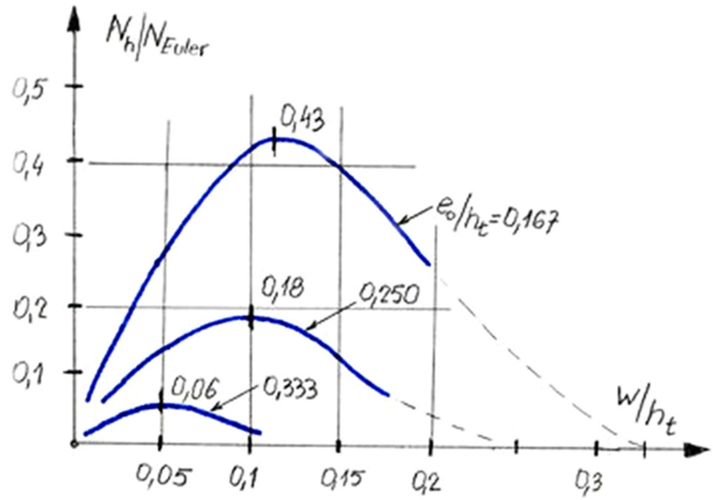

Figure 9. Variation of $N_h/N_{E,kr}$ as a function of w/h_t

The dashed line in *Figure 8.b* shows the critical buckling load $N_{h,kr}$. It is calculated by the replacement stiffness K_h, and with the exact limit point, as it is shown in *Figure 9*.

Observing the variation of N_{kr1}, and $N_{h,kr}$, in *Figure 8*, it can be concluded that in the case of the discretely cracked column, the critical buckling load of the masonry column increases about one and a half times compared to the continuously cracked case.

SUMMARY

Masonry columns, subjected to eccentric compression, crack due to tension if the eccentricity is larger than the size of the core of the section. Previous studies have assumed that the cracks have so small spacing that the cracked tension side can be neglected during the analysis.

The critical load can be determined using this assumption. However, experimental experience has shown that the cracks have large spacing, approximately equal to one and a half times the cross-section height. Therefore, the crack-free parts between the cracks influence the lateral deflection and the critical load. Considering the above mentioned phenomenon, we determined the elastic critical buckling load of the cracked masonry column.

NUMERICAL EXAMPLE

We examine the characteristic load-bearing capacity of an eccentrically compressed masonry column, comparing the EC6 (MSZ EN 1996-1-1 Eurocode 6 2009) standard and the results of the present paper.

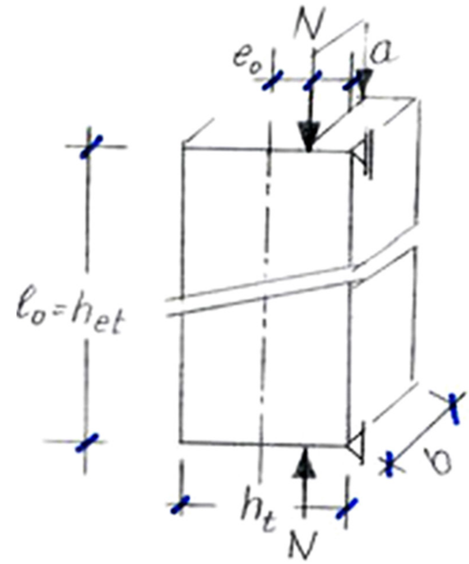

Figure 10. Numerical example

Data according to EC6, Figure 10:

h_t = 380 mm (38cm), b = 1000 mm (100 cm), h_e = 5700 mm (570 cm). $a = e_0$ = 95 mm (9.5 cm).
$A_0 = 380 \cdot 1000 = 3.8 \cdot 10^5$ mm² (3800 cm²), $A_c = 1.9 \cdot 10^5$ mm² (1900 cm²).
Masonry unit: f_b = 10N/mm² (100 kp/cm²). Mortar: f_m = 1.0 N/mm² (10 kp/cm²).
Masonry element: $f_k = 0.5\ (f_b^{0.7}) \cdot (f_m^{0.3}) = 0.5 \cdot 10.0^{0.7} \cdot 1.0^{0.3} = 2.5$ N/mm² (25 kp/cm²).
Modulus of elasticity: $E_{lt} = 1000 \cdot f_k = 2500$ N/mm² (25000 kp/cm²).
Eccentricity: $e_0 = M/N + e_{init} = e_d + h_e/450 = (82 + 13) = 95.0$ mm. (9.5 cm).

CHARACTERISTIC VALUE OF THE LOAD CAPACITY ACCORDING TO EC6

In this case, the eccentricity and slenderness reduction factor according to EC6 (MSZ EN 1996-1-1 Eurocode 6 2009): $\Phi = 0.32$. The load-bearing capacity: Nm, k=$\Phi \cdot b \cdot h_t \cdot f_k = 0.32 \cdot 380 \cdot 1000 \cdot 2.5 = 3.04 \cdot 10^5$ N. (3.04 Mp).

THE CHARACTERISTIC VALUE OF THE LOAD CAPACITY ACCORDING TO THE PRESENT PAPER

Euler critical load:
$N_{E,\,kr} = \pi^2\, E_{lt} I/h_e^{\,2} = 9.86{\cdot}2500{\cdot}1000{\cdot}380^3/12 \,/\, 5700^2 = 3.47{\cdot}10^6$ N (34.7 MP).
The buckling load of the discretely cracked elastic masonry column:
$N_{h,kr} = 0.18\, N_{E,kr} = 0.18{\cdot}3.47{\cdot}10^6 = 6.25{\cdot}10^5$ N (6.25 MP).
Area whose centroid is the point of action of the compressive force:
$A_c = 190{\cdot}1000 = 1.9{\cdot}10^5$ mm^2 (1900 cm^2).
The plastic load-bearing capacity of the cross-section for eccentric loading:
$N_t = A_c{\cdot}f_k = 1.9{\cdot}10^5{\cdot}2.5 = 4.75{\cdot}10^5$ N (4.75 MP).
The non-linear critical buckling load Nm, k can be calculated by the Ritter–Mörsch, (and Renkine) equation: $N_{m,k} = N_t\, /(1{+}N_t/N_{h,kr})$, which was originally derived by Navier, according to Bach – Baumann (1924).
The load-bearing capacity of the cracked masonry column:
$N_{m,\,k} = N_t\, /(1{+}\, N_t/N_{h,\,kr}) = 4.75{\cdot}10^5 \,/\, (1 + 4.75/6.25) = 2.70{\cdot}10^5$ N (2.7 Mp).

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A húzószilárdság nélküli téglapillér statikai modelljéről

ÖSSZEFOGLALÓ

A külpontosan nyomott téglapillér a maghatárt meghaladó külpontosságnál a húzott oldalon bereped. A korábbi vizsgálatok feltételezték, hogy a berepedés olyan sűrűn helyezkedik el, hogy a vizsgálat során a húzott oldalt teljesen el lehet hanyagolni, és a kritikus erőt e feltevés alapján lehet meghatározni. A kísérleti tapasztalatok azonban azt mutatták, hogy a repedés nem sűrűn jelentkezik, hanem megközelítően a másfélszeres keresztmetszeti méretnek megfelelő távolságban. Ezért a repedések közötti tömb belejátszik az alakváltozásba, és így a kritikus teher meghatározásába is. E tulajdonság figyelembevételével meghatároztuk a téglapillér rugalmas kritikus erejét.

KULCSSZAVAK

falazott szerkezet, stabilitásvesztés, húzószilárdság nélküli anyag