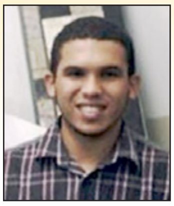
Volume 11, Number 4 (August 2018) p. 673 – 685 • ISSN 1983-4195
http://dx.doi.org/10.1590/S1983-41952018000400003

REVISTA IBRACON DE ESTRUTURAS E MATERIAIS
IBRACON STRUCTURES AND MATERIALS JOURNAL

Verification of the flexion and shear behavior in masonry panels accordance to ABNT NBR 15961-1 (2011), EN 1996-1-1 (2005) and ACI TMS 530 (2013)

Verificação do comportamento à flexão e cisalhamento em painéis de alvenaria conforme a ABNT NBR 15961-1 (2011), EN 1996-1-1 (2005) e ACI TMS 530 (2013)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Abstract

This paper presents a numerical analysis of the mechanical behavior of structural masonry panels submitted to horizontal and vertical stresses. To evaluate the design process of these structures, the results obtained by the computer simulations were compared with the results determined by the design criteria of ABNT NBR 15961-1 (2011), ACI TMS 530 (2013) and EN 1996-1-1 (2005). The finite element software DIANA v.9.3 was used to simulate two-dimensional models with the simplified micro modelling procedure. The results obtained by the normative standards were more conservative than the results of the numerical model, as expected. With the increase of the pre-compression level, the computer simulation has demonstrated the increasing trend of the values of resistant forces, besides the change of the way of rupture of the panels. Among the three standards evaluated, the American Standard was the most conservative.

Keywords: structural masonry, numerical analysis, FEM, standards criteria.

Resumo

O presente trabalho apresenta uma análise numérica do comportamento mecânico de painéis de alvenaria estrutural submetidos a esforços horizontais e verticais. Para avaliar o processo de dimensionamento destas estruturas, foram confrontados os resultados obtidos pelas simulações computacionais com os resultados determinados pelos critérios de dimensionamento da ABNT NBR 15961-1 (2011), do ACI TMS 530 (2013) e do EN 1996-1-1 (2005). Foi utilizado o software de elementos finitos DIANA v.9.3 para simular os modelos bidimensionais com o procedimento de micromodelagem simplificada. Os resultados obtidos pelos padrões normativos foram mais conservadores que os resultados do modelo numérico, conforme o esperado. Com o aumento do nível de pré-compressão, a simulação demonstrou a tendência de aumento dos valores dos esforços resistentes, além da mudança do modo de ruptura dos painéis. Dentre as três normas avaliadas, a norma americana foi a mais conservadora.

Palavras-chave: alvenaria estrutural, análise numérica, MEF, critérios normativos.

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Received: 23 Mar 2017 • Accepted: 16 Jan 2018 • Available Online: 20 Jul 2018

1. Introduction

According to Parsekian [10], the structural masonry system was based on empirical methods until the 1950s, requiring technical formulations that established more efficient constructive guidelines as well as rational calculation methods.

In this sense, in the Brazil, until the year 2011 the design of structures in masonry was regulated by ABNT NBR 10837 [2], whose security criterion was based on the Admissible Tensions Method. According to reports by Reboredo [14], the current normalization was updated with the launch of ABNT NBR 15961 [3], whose basic premises that guided its elaboration were: the adoption of the Last Limit State Method as a security concept, the use of characteristic values rather than the use of mean values in the design and the use of the gross area of the block, prism and wall as a reference for calculations during structural analysis - although the net area is more accurate.

Moreover, at international level, the use of masonry as a structure differs according to the physical aspects of each region. In Europe, most of the units used in the construction of masonry panels are ceramic bricks, whereas in the United States, due to the occurrence of earthquakes, it is commonly adopted the reinforced masonry, unlike Brazil, where the partially reinforced masonry is widely used with concrete or ceramic blocks.

However, according to Lopes [6], until the 1970s the American Standard used the same expression as in the Brazilian Standard [3] to calculate the load capacity reducing factor due to the slenderness of the masonry. However, this factor was updated, where the power was corrected for the "square" and not the cube, besides the limitation of its application to values of slenderness index lower less than 99 - for larger values, a new expression was deduced. Lopes [6] also states that the new factor used by the American Standard [1], based on the classical theory of material strength, takes into account the problem of buckling, in which the lateral displacement during compression is considered. In this sense, the adoption of the Limit State Method in the Brazilian Standard [3] without the correction of the reducing coefficient generates a contradiction in its updating.

In this context, the main international normative codes - [1] and [5] - differ in the criteria adopted during the structural stability analysis of the masonry, and there is a need for research and studies that validate the verification.

Ramalho et al. [13] explains that the main structural concept in the transmission of actions in the masonry is the distribution by compression stress. Therefore, the traction stresses should be restricted to some elements and should not have significant values. Also, the influence of horizontal actions under the stability of masonry walls, especially in tall buildings, is notable. Such actions - usually from the wind or from the building's rubbish - subject the masonry to flexural and shear loads, generating stress that compromise the wall, especially if they are unreinforced.

According to Camacho [4], the horizontal actions that act along a façade are transmitted to the slabs, which in turn distribute them between the parallel walls in the direction of these actions. Such walls, called bracing walls, transmit the load to the foundations and prevent possible cracks. In this sense, such loads are transmitted to the walls proportionally to their rigidity, since all are subjected to the same horizontal displacement. Thus, a wall when subjected to such loads may present various types of cracks that depend on the loading direction, the geometric shape of the panel and the strength of its constituent elements.

As reported by Lourenço [7], in the shear rupture, diagonal cracks occur on the panel, with sliding of the horizontal joints and the consequent separation of the vertical joints. The simple compression rupture is characterized by the crushing of the compressed units, in which the separation of the units occurs with the traction regime due to the expansion of the mortar of the horizontal joints under high normal stresses. However, when analyzing the behavior of the panel under actions of bi-axial flexural, the influence of the geometric shape of the wall is preponderant. Thus, in the flexotraction - action of flexion and traction forces in the units - the rupture of the blocks occur by traction of the horizontal joints, while in the flexocompression - the action of the flexion with the compression - the rupture verified is due to the crushing of the compressed units during bending of the panel. Therefore, the sizing of masonry walls must incorporate the checks of the rupture modes described, as well as their strength characteristics.

The objective of this work is to perform a comparative analysis of the design of unreinforced structural masonry panels according to the American [1], Brazilian [3] and European [5] Standards, using computational simulations as a parameter in the validation of hypotheses. Also, for greater representativeness of the results, several types of panels were used in different loads levels. In this way, it is possible to evaluate the structural behavior of masonry walls submitted to vertical and horizontal loads, parallel to the study of ABNT NBR 15961-1 [3], ACI TMS 530 [1] and EN 1996-1-1 [5].

2. Standards verification criteria

Besides resisting to horizontal actions, the bracing panels give strength to the loads resulting from vertical actions - self-weight, permanent actions, overloads and accidental loads. In this context, it can be stated that the mechanical behavior of masonry panels follows the main concepts of classical mechanics of materials, although each norm presents different study strands from these basic concepts.

Therefore, the design of the panels must meet three normative criteria: simple compression verification; shear verification and flexural verification.

2.1 Compression design

According to Ramalho et al. [13], the main structural concept in the transmission of actions in the masonry is the distribution by compression tensions. This fact results from the influence of the blocks on the strength capacity of the wall, since they promote considerable gains of strength. In fact, for this work, it is consistent that the verification of the structural stability of the masonry is restricted in the analysis of the flexural and shear stresses, since during the simulations the maximum value for the vertical load to which the panels were submitted was equivalent to the compressive strength of the wall.

In this sense, the compression verification procedure according to the Brazilian Standard [3] was used to define the vertical load limit to which the panels were submitted during the numerical simulations.

According to the Brazilian Standard [3], the process of checking the compressive strength of masonry in the ultimate limit state is defined by the following equation:

$$N_{sd} \leq N_{rd} \quad (1)$$

Where, “N_{sd}” is the design value of the axial load and “N_{rd}” is the design value of the strength axial load, given by:

$$N_{rd} = f_d \cdot A \cdot R \quad (2)$$

Where, “A” is the gross cross-sectional area of the wall, “f_d” is the design value of the compressive strength of the masonry and “R” is the reducing coefficient due to slenderness of the panel, which is given by:

$$R = 1 - \left(\frac{\lambda}{40}\right)^3 \quad (3)$$

In which, “λ” is the slenderness index of the wall.

2.2 Shear design

Considering its importance for the design of high-rise buildings - where the wind action is preponderant - the shear can act in conjunction with the bending and is directly linked to the adhesion between the blocks and the mortar.

Tomaževic [17] states that the actions that affect the plane of the wall can generate rupture modes the panel due to sliding by cutting. Therefore, the strength shear is a determining factor to guarantee the security of parts subjected to lateral loads.

According to the American Standard [1], for the design of panels subjected to loads shear, the shear stress (f_v) is calculated according to the following expression:

$$f_v = \frac{V \cdot Q}{I_n \cdot b} \quad (4)$$

In which, “V” is the shear force acting on the prism, “Q” is the value of the first order static moment, “b” the width of the strength section and “I_n” is the moment of inertia of the net cross section.

Also, the maximum values that limit the shear stress in the masonry are:

1) $0{,}125 \cdot \sqrt{f'_m}$ **(5)**

2) 0,827 MPa;

3) $0{,}255\ \mathrm{MPa} + 0{,}45 \cdot \frac{N_v}{A_n}$ **(6)**

Where, “f'_m” is the compressive strength of the wall, “N_v” is the value of the axial load and “A_n” is the net cross-sectional area. Considering masonry walls with blocks connected by direct lashing and not grating in their totality.

According to the Brazilian Standard [3], in the shear verification of the panels in the ultimate limit state, the design value of the shear strength must be greater than or equal to the shear stress.

The characteristic value of shear strength is tabulated and defined according to NBR 15961 [3], depending on the medium compressive strength of the mortar used. In this way, one has:

$$\tau_{vd} = \frac{V_d}{A} \quad (7)$$

Where, “τ_{vd}” is the design value of the shear stress and “V_d” is the design value of the shear force.

According to the European Standard [5], the design value of the shear stress applied for unreinforced walls should be less than or equal to the design value of the shear strength of the wall, and, in the absence of tests that define it, the characteristic value of the shear strength is given by:

$$f_{vk} = f_{vk0} + 0{,}4 \cdot \sigma_d \quad (8)$$

Thus, “f_{vk}” is the characteristic value of the shear strength and “f_{vk0}” represents its initial value, which is determined from the test or with the use of specific tables present in EN 1996-1-1 [5]. The design value of strength load of the panel is defined as follows:

$$V_{Rd} = f_{vd} \cdot t \cdot l_c \quad (9)$$

Where, “V_{Rd}” is the design value of the strength shear force, “V_{Rd}” is the design value of strength shear stress, “t” is the wall thickness and “l_c” is the width of the compressed part of the panel.

2.3 Flexural design

According to Reboredo [14], the horizontal actions to which the panels are submitted cause bending stresses on the wall. Thus, compression stresses can be generated in the units, characteristic of flexo-compression, or traction stresses in the blocks, typical of flexotraction.

According to the American Standard [1], for the sizing of panels when subjected to a combination of bending, due to the occurrence of earthquakes or winds, and axial compression, the following equations must be satisfied:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1{,}33 \quad (10)$$

Where, “f_a” is the design value of the compression stress due only to the axial load, “f_b” the design value of the compression stress due only to bending, “F_a” is the value of the admissible stress of compression strength and “F_b” is the value of the admissible stress of flexural strength.

These calculation parameters are obtained according to the following equations:

$$F_a = \frac{1}{4} \cdot f'_m \cdot \left[1 - \left(\frac{h}{140 \cdot r}\right)^2\right] \quad (11)$$

Being,

$$F_b = \frac{1}{3} \cdot f'_m \quad (12)$$

Where, “f'_m” is the compressive strength of the prism, “r” is the spinning radius of the net cross-section and “h” is the height of the wall.

The Brazilian Standard [3] establishes that the normal stresses in the cross section must be obtained by superposing of the uniform stresses arising from the compression load and of the linear stresses due to the action of the flexion. Thus, for the verification of panels submitted to biaxial flexural, the following equation must be satisfied:

$$f_d \geq \frac{N_d}{A \cdot R} + \frac{M_d}{W \cdot K} \quad (13)$$

Where, "f_d" is the design value of the compressive strength of the masonry, "N_d" is the design value of the normal stress, "M_d" is the design value of the bending moment, "A" is the gross cross-sectional area, "K" is the adjustment factor of the flexural compression strength (equivalent to 1,5), "W" is the flexural strength module and "R" is the reducing coefficient due to the slenderness of the panel given by equation 3. If the load is flexion, the factor "K" is neglected and the normal force takes negative value.
However, according to the european normative prescriptions [5], for the verification of the flexing of unreinforced masonry the following requirement is met:

$$M_{Ed} \leq M_{Rd} \tag{14}$$

Where, "M_{Ed}" is the value of the load moment and "M_{Rd}" is the design value of the strength moment. The value of "M_{Rd}" is given by the equation given below.

$$M_{Rd} = (f_{xd1} + \sigma_d) \cdot Z \tag{15}$$

Where, "f_{xd1}" is the design value of the flexural strength having the failure plane parallel to the horizontal joints, "σ_d" is the design value of the compression stress and "Z" is the flexural strength module.

3. Numerical simulations in masonry panels

According to Peleteiro [11], the present challenge in the numerical simulation of masonry walls refers to the succinct representation of its heterogeneous and anisotropic nature, in addition to being constituted by three regions with different physical behaviors: the unit, the mortar and the interface mortar-block. Thus, factors such as the geometry of the units, the arrangements of the horizontal and vertical joint and the properties of the units and the mortar, directly interfere in the mechanical characteristics of the assembly.
According to Lourenço [7], a numerical analysis model is defined according to the objective of the study, there are three modeling strategies for masonry validated by the technical literature: detailed micro-modeling, simplified micro-modeling and macromodeling. In the detailed micromolding, the blocks and the mortar are represented by continuous elements and the interface between them as discontinuous elements. In the simplified modeling, the units are represented by continuous elastoplastic elements and their dimensions are expanded, with the joints and interface being uniquely seen as an average interface. For macromodeling all masonry is considered as a continuous medium with homogeneous properties. It is notable that the processing time of the macromodeling is smaller compared to the other models, but this modeling is best applied for general analyzes of structures where the dimensions of the walls are large enough to guarantee the uniform distribution of the stresses. The micromolding allows a good understanding of the local behavior of the structures in masonry, allowing the analysis of structural details.
The simplified micromolding meets the assumptions of this research because it presents less processing time when compared to the detailed micromolding and allows the consideration of the mortar-block interface, making it possible to contemplate the basic mechanisms of rupture described by Lourenço [7] - cracking of the joint by direct traction ; slip along joints; cracking of the units by direct traction; cracking of the units by diagonal traction when the normal stress enables friction formation at the joints, and the crushing of the units.
In this sense, during the modeling, the region delimited by the mortar/block interface must be evaluated more accurately, since it is the most fragile of the structure, especially the rupture mechanism by direct traction in the joint, which, according to tests performed by Pluijm [12]], it is proven that the area of adhesion is less than the cross-sectional area of the specimen.

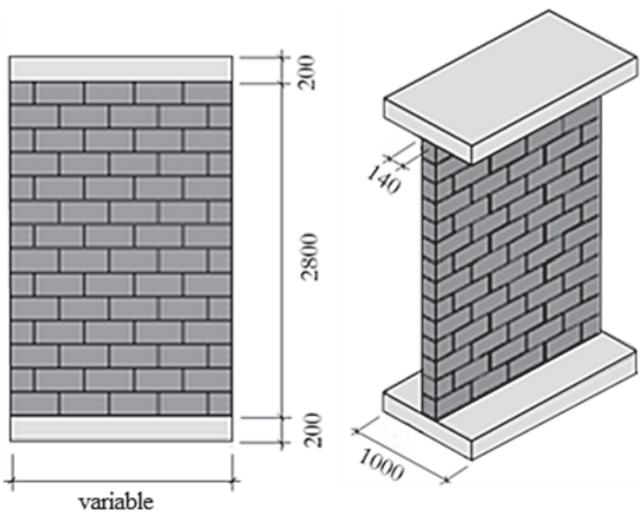

Figure 1
Geometry of masonry panels simulated in numerical modeling

4. Models studied

4.1 Geometry of panels

The unreinforced masonry panels studied have height equal to 280 cm and thickness equal to 14 cm, being variable its length - 120 cm and 420 cm - to be able to evaluate the height/length ratio in checking the rupture mode of the wall, moreover, one centimeter relative to the length of the wall relative to the panel coating was disregarded. Thus, two groups of panels were obtained: the PAR120 group (model 01 to model 09, with a length equal to 120 cm) whose height/length ratio was 2,33 and the PAR420 group (model 10 to model 18, with a length equal to 420 cm) whose height/length ratio was 0,67.
Also, in order to represent the connection between the masonry and the floor slab, at the top and bottom of the bracing panel were modeled concrete slabs with a height equal to 0,20 m, thick equal to 1,00 m and variable length, with restrictions on vertical and horizontal translations applied as boundary conditions. The units were defined as solid blocks and modeled in the two-dimensional plane, the geometric model of the panels can be seen in Figure 1.

4.2 Finite element mesh

For the generation of finite element mesh, as done by Mata [9], was defined the two-dimensional isoparametric plane finite

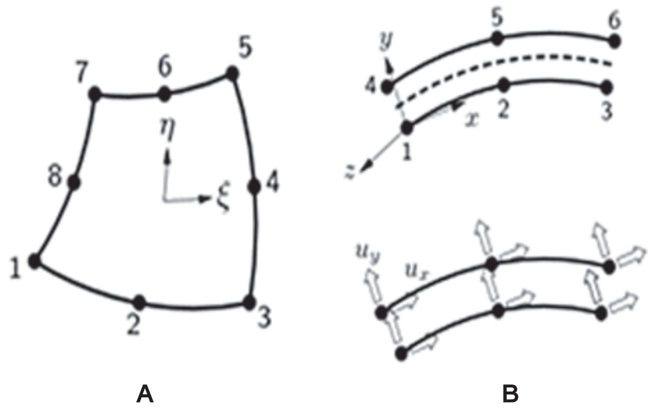

Figure 2
Finite elements adopted in numerical modeling: a - CQ16M, two-dimensional isoparametric plane element [15]; b - CL12I, two-dimensional interface element [16]

element of type CQ16M to represent the units. This element, which can be seen in Figure 2-a, has 8 nodes and 2 degrees of nodal freedom (representing the translations in the x and y directions) with quadratic interpolation function. For the modeling of vertical and horizontal mortar joints, the quadratic interface element of type CL12I was used, which has 6 nodes, each with 2 degrees of freedom (representing the translations in x and y), quadratic interpolation function and thickness equal to zero, which can be seen in Figure 2-b.
For the distribution of the finite elements, it was adopted the same one defined by Mata [9], since it carried out all the inherent study to the refinement of the mesh with the intention of discovering for which distribution the results became constant. Thus, the unit that compose the wall was discretized by three elements two-dimensional in length and three elements in two-dimensional in the height, where between these units, three interface elements were applied.
Also, vertical fracture planes were defined in the middle of the units according to Lourenço's recommendations [7].

4.3 Properties of materials

The concrete slabs, located at the top and at the base of the wall, acted only as a contour condition of the panel analyzed. Thus, only elastic and isotropic properties were defined to model these elements, with the modulus of longitudinal elasticity equal to 20,0 GPa and the Poisson coefficient equal to 0,20.
The Table 1, 2 and 3 present the input data in the DIANA® software concerning the materials properties of the panel, experimentally raised by Mata [9] - in the Structures Laboratory of the São Carlos Engineering School, University of São Paulo - which were parameterized for the numerical modeling.
Since, in Table 1, the "f_{bk}" is the characteristic value of the block strength, "f_{pk}" is the characteristic value compressive strength of the prism, "f_a" is the average compressive strength of the mortar and the other data related to the physical properties of the panels are tabulated and derived from ABNT NBR 15961 [3].
In Table 2, "f_b" is the average axial compressive strength of the block, "E" is the modulus of longitudinal elasticity, "v" is the Poisson coefficient, "G_c" is the compression fracturing energy, "f_{bt}" is the average traction strength calculated in relation to the gross area of the block, "G_f" is the energy of traction fracture and "β" is the shear retention factor.
Also, in Table 3, "f_t" is the traction strength of the joint, "k_n" and "k_s" refer to the module of the normal and transverse elastic stiffness, respectively, and the "G^I_f" fracturing energy of mode I.
The Table 4 lists the mechanical data of the materials used for the algebraic sizing according to ABNT NBR 15961 [3], in which "f_c" is the average compression strength of prism with three blocks, "G_c" is the compression fracture energy of prism with three blocks, "ε_c" is the deformation corresponding to the peak of the compression stress of the prism, "C_{ss}" is the contribution control of shear stresses at rupture (Lourenço et al., apud Mata [9]), "f_{v0}" is the shear stress in the absence of compression, "tanφ" is the friction coefficient, and "G^{II}_f" is the fracture energy of mode II.

Table 1
Physical properties of the materials studied by Mata [9]

Property	Value (MPa)
f_{bk}	9,77
f_{pk}	8,45
f_a	6,28

Table 2
Mechanical properties of the blocks studied by Mata [9]

Property	Value
f_b (MPa)	12,86
E (MPa)	7586,00
v	0,37
G_c (MPa.mm)	19,94
f_{bt} (MPa)	1,20
G_f (MPa.mm)	0,05
β	0,03
Fissure band width (mm)	1,00

Table 3
Mechanical properties of the vertical fracture plane used by Mata [9]

Property	Value
f_t (MPa)	1,200
k_n (N/mm³)	106
k_s (N/mm³)	106
G^I_f (MPa.mm)	0,047

Table 4
Mechanical properties of joints used by Mata [9]

Property	Value for horizontal joints	Value for vertical joints
f_t (MPa)	0,093	0,085
k_n (N/mm³)	0	34,380
k_s (N/mm³)	118,710	161,820
G^I_f (MPa.mm)	0,005	0,005
f_c (MPa)	7,920	6,280
G_c (MPa.mm)	11,640	14,410
ε_c (10^{-3})	5,400	3,700
C_{ss}	2	2
f_{v0} (MPa)	0,208	0,235
tan ϕ	0,612	0,624
G^{II}_f(MPa.mm)	0,090	0,020

4.4 Calculation procedure

In the initial stage of the numerical test, the two geometric models evaluated were submitted to nine pre-compression levels, whose values ranged from 0,20 MPa to 1,85 MPa. The maximum pre-compression applied pressure (1,85 MPa) was the compressive strength of the panel, determined according to ABNT NBR 15961 [3]. The load data to which the panels were subjected are shown in the Table 5 and 6.

The loads subjected to the panel models were applied in steps, where the pre-compression stress was imposed in two successive steps, each with half loading. Then, at the top of the panel, successive deformations were applied in the amount of steps required to bring the model to rupture, ranging from 500 to 800 steps.

Table 5
Geometric characteristics and vertical loading of panels of the PAR120 group

Panel	Height (mm)	Length (mm)	Pre-compression level (MPa)	Distributed load (N/mm)	Vertical force (kN)
Model 1	2800,00	1190,00	0,20	28,00	33,32
Model 2	2800,00	1190,00	0,40	56,00	66,64
Model 3	2800,00	1190,00	0,60	84,00	99,96
Model 4	2800,00	1190,00	0,80	112,00	133,28
Model 5	2800,00	1190,00	1,00	140,00	166,60
Model 6	2800,00	1190,00	1,20	168,00	199,92
Model 7	2800,00	1190,00	1,40	196,00	233,24
Model 8	2800,00	1190,00	1,60	224,00	266,56
Model 9	2800,00	1190,00	1,85	258,78	307,95

Table 6
Geometric characteristics and vertical loading of panels of the PAR420 group

Panel	Height (mm)	Length (mm)	Pre-compression level (MPa)	Distributed load (N/mm)	Vertical force (kN)
Model 10	2800,00	4190,00	0,20	28,00	117,32
Model 11	2800,00	4190,00	0,40	56,00	234,64
Model 12	2800,00	4190,00	0,60	84,00	351,96
Model 13	2800,00	4190,00	0,80	112,00	469,28
Model 14	2800,00	4190,00	1,00	140,00	586,60
Model 15	2800,00	4190,00	1,20	168,00	703,92
Model 16	2800,00	4190,00	1,40	196,00	821,24
Model 17	2800,00	4190,00	1,60	224,00	938,56
Model 18	2800,00	4190,00	1,85	258,78	1084,29

A displacement of 0,01 mm was applied at each step. The result of the modeling was the critical shear force at the moment of peak deformation. For the algebraic sizing by standards, the product between the pre-compression value at which the wall was subjected and its thickness resulted in the load distributed along the length of the panel, which when multiplied by the length of the wall results in the vertical force considered in the calculations .

In parallel to the numerical simulation was performed the verification of the maximum value of strength shear forces for the same panels models.

The equations used in the algebraic sizing were defined by adaptations of the equations present in each norm, respecting the conditions imposed by the codes. For the shear verification process, the adaptations made to ABNT NBR 15961-1 [3] are represented in equations 16 and 17, and for EN 1996-1-1 [5] the adaptations used are shown in equations 18, 19 and 20, considering that when the condition imposed by equation 18 is not satisfied, the sizing is done by means of equation 20. Finally, in the equation 21, the calculation model used for the algebraic sizing according to the criteria of ACI TMS 530 [1] is defined.

$$f_{vk}=0{,}15+0{,}5\cdot\left(\frac{0{,}9\cdot N_k}{t\cdot L}\right)\leq 1{,}4 \quad \textbf{(16)}$$

where, "f_{vk}" is the characteristic value of the shear strength, "Nk" is the characteristic value of the compression load and "L" is the length of the wall.

$$V_k=\frac{f_{vk}\cdot t\cdot L}{\gamma_m\cdot\gamma_Q} \quad \textbf{(17)}$$

Where "V_k" is the characteristic value of the shear load, "γ_m" is the strength reduction coefficient (equivalent to 2,0), "γ_Q" is the coefficient of increase of the variable shares (equivalent to 1,4).

$$f_{vk0}+\frac{0{,}4\cdot\gamma_G\cdot N_k}{t\cdot L}\leq 0{,}065\cdot f_b \quad \textbf{(18)}$$

Since, "γ_G" is the coefficient of increase of the permanent actions (equivalent to 1,4), "f_b" is the compressive strength of the unit.

$$V_k=\frac{f_{vk0}\cdot t\cdot L+0{,}4\cdot N_k}{\gamma_m\cdot\gamma_Q} \quad \textbf{(19)}$$

$$V_k=\frac{0{,}065\cdot f_b\cdot t\cdot L}{\gamma_m\cdot\gamma_Q} \quad \textbf{(20)}$$

$$V_k=\frac{2\cdot\tau_v\cdot t\cdot L}{3} \quad \textbf{(21)}$$

Where, "τ_v" is the minimum shear stress.

In this context, it is shown in equations 22, 23 and 24, the adaptations made to the verification of the flexo-compression of the calculation models presented in ABNT NBR 15961-1 [3], EN 1996-1 [5] and ACI TMS 530 [1], respectively. Also, in equations 25 and 26, we have the calculation model adopted to verify the critical flexotration load based on ABNT NBR 15961-1 [3] and ACI TMS 530 [1], respectively.

Table 7
Comparison between the results of the numerical models and those coming from the American Standard

Panel	Shear verification (kN)	Flexo-traction verification (kN)	Flexo-compression verification (kN)	Predominant rupture mode	Numerical result (kN)
Model 1	14,83	3,42	39,64	Flexo-traction	13,33
Model 2	20,83	5,78	35,51	Flexo-traction	20,86
Model 3	26,83	8,14	31,39	Flexo-traction	27,11
Model 4	32,83	10,50	27,26	Flexo-traction	33,85
Model 5	38,82	12,86	23,14	Flexo-traction	40,57
Model 6	44,82	15,22	19,01	Flexo-traction	50,02
Model 7	47,76	17,58	14,89	Flexo-compression	56,91
Model 8	47,76	19,94	10,76	Flexo-compression	63,79
Model 9	47,76	22,87	5,63	Flexo-compression	71,80
Model 10	52,23	42,35	491,44	Flexo-traction	71,35
Model 11	73,35	71,61	440,29	Flexo-traction	99,64
Model 12	94,47	100,87	389,14	Shear	126,70
Model 13	115,58	130,13	337,99	Shear	152,50
Model 14	136,70	159,39	286,84	Shear	177,70
Model 15	157,82	188,65	235,69	Shear	202,80
Model 16	168,17	217,91	184,54	Shear	227,90
Model 17	168,17	247,17	133,39	Flexo-compression	253,10
Model 18	168,17	283,51	69,85	Flexo-compression	283,80

$$V_k=\frac{K\cdot L}{6\cdot h\cdot \gamma_Q}\cdot\left(f_d\cdot L\cdot t+\frac{\gamma_G\cdot N_k}{R}\right) \quad (22)$$

$$V_k=\frac{t\cdot L^2}{6\cdot \gamma_Q\cdot h}\cdot\left(\frac{f_{xk1}}{\gamma_m}+\frac{\gamma_G\cdot N_k}{t\cdot L}\right) \quad (23)$$

$$V_k=\frac{t\cdot L^2\cdot F_b}{6\cdot h}\cdot\left(1{,}33-\frac{N_k}{t\cdot L\cdot F_a}\right) \quad (24)$$

$$V_k=\frac{L}{6\cdot h\cdot \gamma_Q}\cdot\left(f_{td}\cdot L\cdot t+\frac{\gamma_G\cdot N_k}{R}\right) \quad (25)$$

Where, "f_{td}" is the design value of the traction strength of masonry.

$$V_k=\frac{t\cdot L^2}{6\cdot h}\cdot\left(f_{td}+\frac{N_k}{t\cdot L}\right) \quad (26)$$

5. Results and discussion

The behavior of 18 panel models was verified for the analysis of the rupture form presented, either by shear or biaxial flexural.
Below are the graphs that compare the results obtained by the normative criteria with the results of the computer simulation for the American [1], Brazilian [3] and European [5] Standards. The axis of the ordinates represents the strength values obtained by each norm, while the axis of the abscissa refers to the levels of pre-compression to which the panels were submitted. The smallest value between the standards verification criteria corresponds to the critical force (V_k) of the panel, defining the load capacity of the wall under the conditions evaluated. The Tables 7, 8 and 9 illustrate the results obtained from the perspective of standards and computational modeling. The type of rupture presented by the panels is differentiated according to their geometry and load levels.

5.1 Group PAR120 (280 cm x 120 cm)

By analyzing the graph present in figure 3 relative to the American Standard [1], it can be seen that the results of the critical rupture load were lower than the results of the numerical models, as expected. Also, for pre-compression levels ranging from 0,20 MPa to 1,20 MPa, the rupture form was by flexo-tration, characteristic of the panel evaluated due to its high height/length ratio. However, for higher levels of load, the form of rupture verified was to flexo-compression, with an increase in the difference between the result of the computational model and the result presented by the American Standard [1].
Further, it is observed that, from the load level equivalent to 1,40 MPa, the values referring to the strength shear force of the panel remain constant with the growth of the load. This is due to the limitation to the minimum shear stress (τ_v) by the American Standard [1] that, for values of such pre-compression loads, equating to 0,827 MPa.
The analysis of the graph of Figure 4 referring to the Brazilian Standard [3] denotes a tendency of growth of the strength loads

Table 8
Comparison between the results of the numerical models and those coming from the Brazilian Standard

Panel	Shear verification (kN)	Flexo-traction verification (kN)	Flexo-compression verification (kN)	Predominant rupture mode	Numerical result (kN)
Model 1	13,33	4,09	38,68	Flexo-compression	13,33
Model 2	18,33	6,61	42,45	Flexo-compression	20,86
Model 3	23,32	9,13	46,23	Flexo-compression	27,11
Model 4	28,32	11,64	50,01	Flexo-compression	33,85
Model 5	33,32	14,16	53,78	Flexo-compression	40,57
Model 6	38,32	16,68	57,56	Flexo-compression	50,02
Model 7	43,32	19,20	61,33	Flexo-compression	56,91
Model 8	48,31	21,71	65,11	Flexo-compression	63,79
Model 9	54,52	24,84	69,80	Flexo-compression	71,80
Model 10	46,93	50,72	479,50	Shear	71,35
Model 11	64,53	81,93	526,32	Shear	99,64
Model 12	82,12	113,14	573,13	Shear	126,70
Model 13	99,72	144,35	619,95	Shear	152,50
Model 14	117,32	175,56	666,77	Shear	177,70
Model 15	134,92	206,77	713,58	Shear	202,80
Model 16	152,52	237,98	760,40	Shear	227,90
Model 17	170,11	269,19	807,21	Shear	253,10
Model 18	191,97	307,96	865,37	Shear	283,80

Table 9
Comparison between the results of the numerical models and those coming from the European Standard

Panel	Shear verification (kN)	Flexo-compression verification (kN)	Predominant rupture mode	Numerical result (kN)
Model 1	11,91	2,67	Flexo-compression	13,33
Model 2	15,46	5,04	Flexo-compression	20,86
Model 3	19,01	7,40	Flexo-compression	27,11
Model 4	22,57	9,76	Flexo-compression	33,85
Model 5	29,83	12,12	Flexo-compression	40,57
Model 6	29,83	14,48	Flexo-compression	50,02
Model 7	29,83	16,84	Flexo-compression	56,91
Model 8	29,83	19,20	Flexo-compression	63,79
Model 9	29,83	22,13	Flexo-compression	71,80
Model 10	41,92	33,16	Flexo-compression	71,35
Model 11	54,44	62,42	Shear	99,64
Model 12	66,95	91,68	Shear	126,70
Model 13	79,46	120,94	Shear	152,50
Model 14	105,03	150,20	Shear	177,70
Model 15	105,03	179,46	Shear	202,80
Model 16	105,03	208,72	Shear	227,90
Model 17	105,03	237,98	Shear	253,10
Model 18	105,03	274,33	Shear	283,80

with the increase of the level of pre-compression, as a result of the linear relationship between the ultimate load of rupture (V_k) of the panels and the pre-compression stress (σ) to which they are submitted. Also, it is verified that, for all the models strengthed by the Brazilian Standard [3], the predominant rupture mode was flexo-tration, being coherent with the geometric form of the panel. In the analysis of the graph of Figure 5 referring to the European Standard [5], the level of conservatism adopted in the verification is notable, since for all the analyzed walls the values of the strength loads were below the results of the numerical models. There is

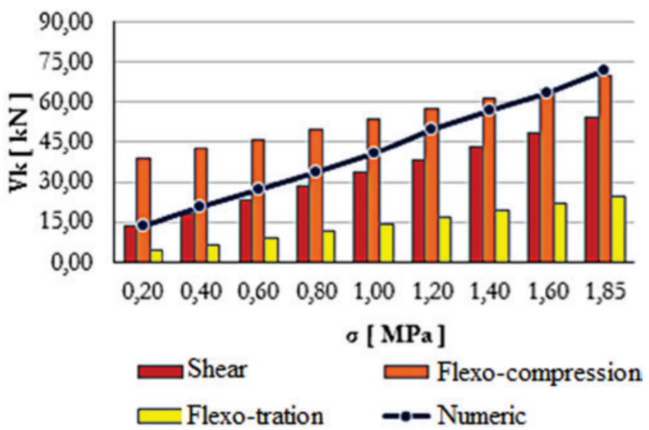

Figure 4
Comparison between the results of the Brazilian Standard and those of the numerical models, referring to the models of PAR120 (280 cm x 120 cm) group

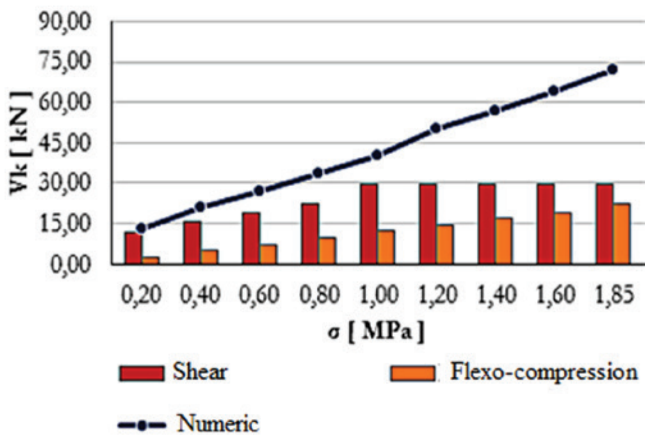

Figure 5
Comparison between the results of the European Standard and those of the numerical models, referring to the models of PAR120 (280 cm x 120 cm) group

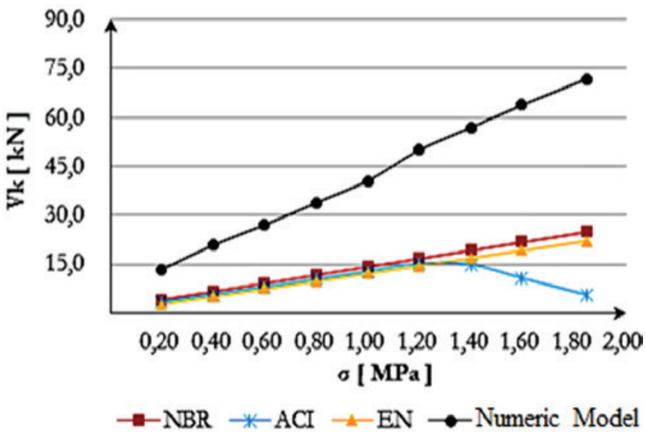

Figure 6
Comparison between strength shear forces obtained by the three normative codes and the numerical simulation, referring to the models of PAR120 (280 cm x 120 cm) group

also a trend of increasing strength loads with increasing pre-compression level.
However, for higher loading levels - more precisely from loads whose strength loads is equivalent to a percentage of 17,91% - the linear relationship between shear strength and pre-compression stress ceases to exist, providing lower resistance to the shear stress and the origin of this model of cracking in the panel - a fact that can be noticed in the analysis of equations 18, 19 and 20, within the condition imposed by equation 18.
With the results, the data obtained by the three normative models and those from the numerical model were compared, obtaining the graph of curves presented in Figure 6. By its analysis, it is noticed that in all panels evaluated, for the three normative models, the results obtained by the standards were smaller than the results derived from the numerical model, verifying the conservatism adopted in the sizing. Also, for all levels of loads, the similarity between the behaviors of panels evaluated according to the Brazilian [3] and European Standards [5] is verified, which have the same security criterion in the checks - Ultimate Limit State - although the European Standard is more conservative compared to the national standard.
Further, it is noted that, for high levels of load, there is a significant drop in the load capacity of the panels according to the American Standard [1], different from the results presented by the other two normative models. This fact denotes the preponderance of this normative model in the consideration of compression loads.
As an illustration of the results presented, below are the images of the numerical models, with the definition of the mesh and stress spectrum, at the moment of panel rupture. The most representative models were chosen, corresponding to the pre-compression levels equivalent to 0,20 MPa and 1,85 MPa. It was decided to maintain the same amplitude between the values in the stress distribution of the panels in order to allow conditions of comparison between the numerical models.
In the image of the numerical model present in Figure 7 it is possible to observe the rupture configuration typical of flexion, characterized by the appearance of horizontal cracks in the base and top of the wall that propagate along its length, besides the absence of diagonal cracks and the crushing of the units in the compressed region (lower right end). It is also observed the concentration of traction stress at the ends of the base and top of the panel, a factor responsible for the presence of cracks in the region, as observed by Mata [9].
For the numerical model present in Figure 8, subjected to a high pre-compression load, it is noticed the rupture mode typical of

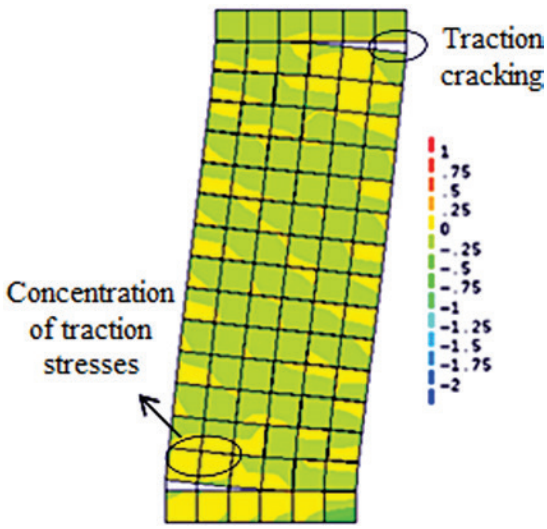

Figure 7
Deformation and principal stress distribution in the model panel of the PAR120 (280 cm x 120 cm) group with of pre-compression level equal to 0,20 MPa

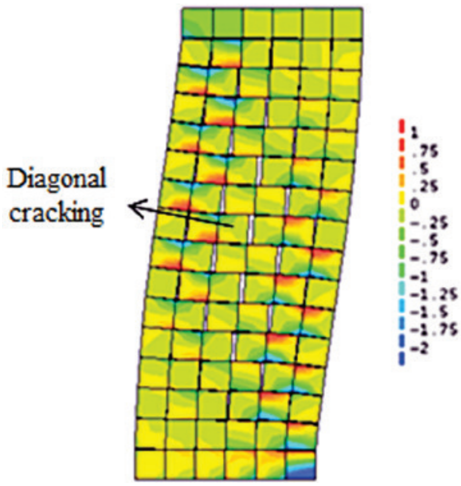

Figure 8
Deformation and principal stress distribution in the model panel of the PAR120 (280 cm x 120 cm) group with of pre-compression level equal to 1,85 MPa

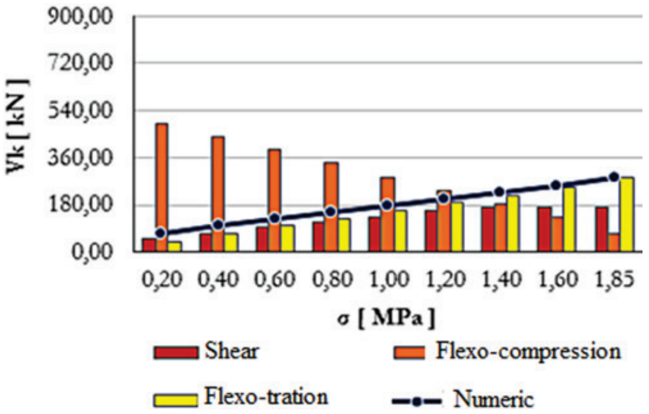

Figure 9
Comparison between the results of the American Standard and those of the numerical models, referring to the models of PAR420 (280 cm x 420 cm) group

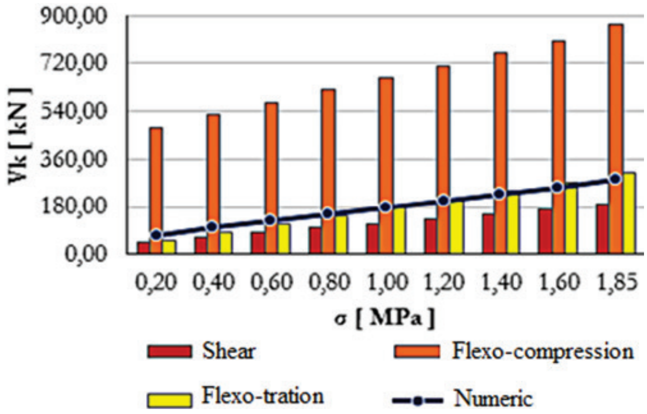

Figure 10
Comparison between the results of the Brazilian Standard and those of the numerical models, referring to the models of PAR420 (280 cm x 420 cm) group

the shear with the presence of diagonal cracks in the wall. Also, it should be noted that although panel dimensions (high height/length ratio) preponderate flexural rupture, high stress values caused the confinement effect of the units, which reduced the deformation capacity of the panel and led to a rupture mode with shear action.

5.2 Group PAR420 (280 cm x 420 cm)

The analysis of the graph present in Figure 9 referring to the American Standard [1] allows to conclude that, for most of the dimensioned models, the rupture mode has shear action, whereas for low loading levels – 0,20 MPa and 0, 40 MPa - The rupture is characterized by flexo-tration. Also, it is observed that at high loading levels (1,60 MPa and 1,85 MPa), the rupture occurs by flexo-compression, being incoherent with the geometric shape of the panels (low height/length ratio) and the high level of solicitation.
However, it is possible to notice the proximity of the results of the computational simulation with the results obtained from the algebraic sizing of the norm, mainly with the values related to shear strength. This fact demonstrates that, for average load levels - less than 1,60 MPa - the American Standard [1] achieves a good prediction of the rupture mode of the panel, validated by its proximity to the numerical model.
By analyzing the data presented in the graph of Figure 10 referring to the Brazilian Standard [3], it can be seen that the values of all the strength loads increased with the increase of the pre-compression level. Also, it was noted that the shape of rupture of the walls, for all evaluated loads, was by shear, being consistent with the geometry of the panel (low height/length ratio).
However, the mode of rupture of the model with 0,20 MPa of pre-compression is incoherent, since the Brazilian Standard [3] indicates predominance of shear, being different from the results of the other two normative models - flexion - and not validated by the analysis of the rupture of the numerical model present in Figure 12.
The analysis of the graph in Figure 11, that refers to the European Standard, shows that, as in the data referring to the model with the highest height/length ratio (Figure 5), all the strength values were below the results from the numerical model. It is also verified that, there is the preponderance of the shear in the rupture mode of the panel for most of the models evaluated.
Further, it can be seen that, from the pre-compression load level equivalent to 1,20 MPa, the critical force of the panel validated by the European Standard [5] is constant (105,03 kN) and corresponds to the shear rupture. This is due to the growth of the load and the consequent breakdown of the linear relationship with the pre-compression level.
The Figure 12 shows the graph that confronts the results obtained by the three normative models with the results coming from the numerical models for the panels of the PAR420 group. It can be noticed the conservatism adopted by the norms during the sizing,

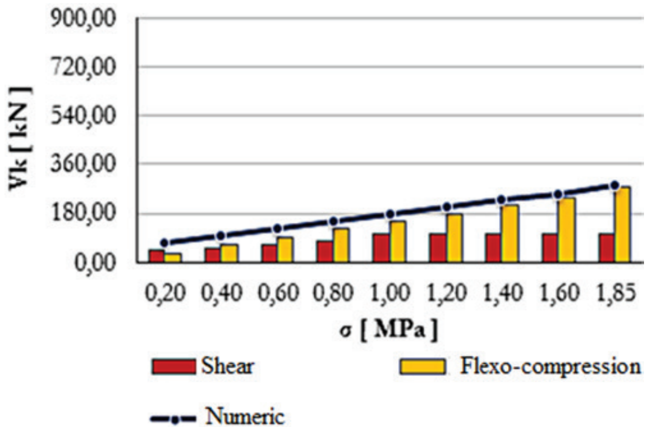

Figure 11
Comparison between the results of the European Standard and those of the numerical models, referring to the models of PAR420 (280 cm x 420 cm) group

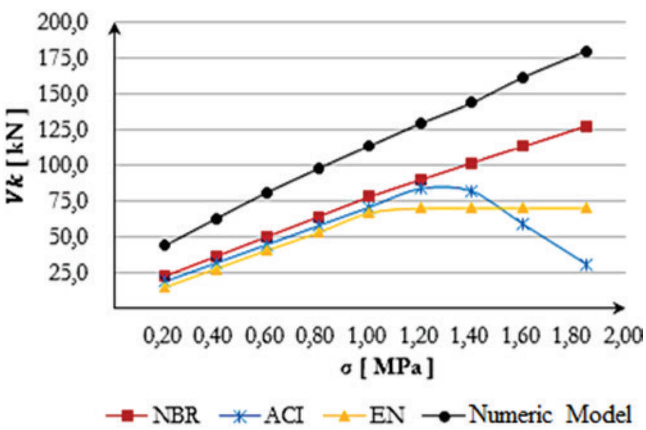

Figure 12
Comparison between strength shear forces obtained by the three normative codes and the numerical simulation, referring to the models of PAR420 (280 cm x 420 cm) group

since the results coming from the norms were inferior to the values obtained by the computational simulation.
Also, there is a difference in the behavior of the three normative codes for load levels greater than 1,20 MPa, where the European Standard [5] presents constant results, the Brazilian Standard [3] denotes a gain in the strength capacity, whereas the american norm [1] shows a decrease in the values of the strength loads.
As an illustration of the results presented, below are the images of the numerical models, with the definition of the mesh and the stress distribution, at the eminence of panel rupture and corresponding to the pre-compression levels equivalent to 0,20 MPa and 1,85 MPa.
In the image of the numerical model present in Figure 13 it is possible to observe the rupture configuration typical of flexion, characterized by the appearance of horizontal cracks in the base and top of the wall (highlighted in the image) that correspond to low displacements - compared to a model whose height/length ratio is higher (Figure 7) - due to its geometry and the low load level. Also, a concentration of compression stresses in the lower right end of the panel is verified, factor responsible for the crushing of the units in this region.
For the numerical model present in Figure 14, a greater negative stresses distribution on its surface is observed, and the preponderance of the rupture mode governed by the combination of the flexural and the shear, being in accordance with the high load level of the panel and its low height/length ratio.

6. Conclusions

The calculation procedures described by the Brazilian [3], American [1] and European [5] Standards were presented, as well as the numerical modeling routine adopted. During the computational simulation two geometric models of masonry panels were evaluated, which were submitted to nine pre-compression levels and to successive displacements until the moment of rupture, resulting in the critical shear force of the wall and in 18 numerical models studied. Also, the results of the three normative codes and the computer simulation were exposed and confronted.
Therefore, in view of the results and analyzes presented, the conclusions are presented below:

- The modeling of the panels ensured a good representation of the mechanical behavior of the masonry, a fact justified by the rupture modes presented by the numerical models, being coherent with their geometric characteristics and the loading levels to which they were subjected;
- Through the analysis of the curve graphs, it is noticed that, among the three normative models, the american code was the most conservative, since it presented the lowest value of strength capacity for the maximum compression stress, while the Brazilian Standard [3] was the most daring because it was closer to the values coming from the computational simulation. This fact indicates that the verification process adopted by the national standard is coherent and indicates the search for a balance between security in design and the economic viability of results;
- Another conclusion refers to the behavior of the curve referring to the American Standard [1], which prioritizes, for higher loads levels, the compression effect of the panels. This fact is

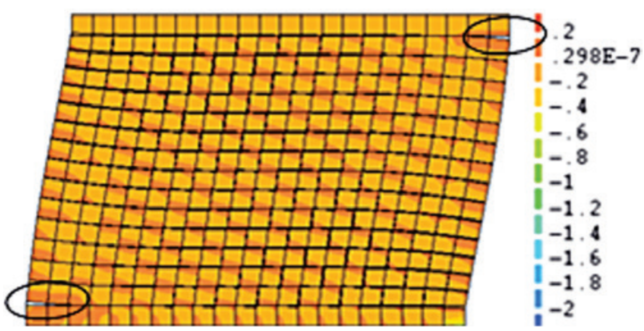

Figure 13
Deformation and principal stress distribution in the model panel of the PAR420 (280 cm x 420 cm) group with of pre-compression level equal to 0,20 MPa

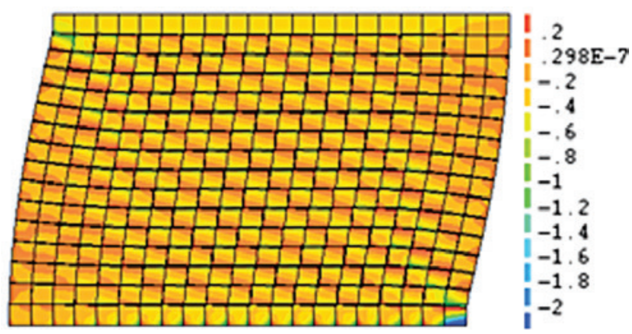

Figure 14
Deformation and principal stress distribution in the model panel of the PAR420 (280 cm x 420 cm) group with of pre-compression level equal to 1,85 MPa

responsible for the considerable loss of strength of the walls, since, as the axial stress level increases, the compressive stresses generated only by flexion of the panel are reduced - as can be seen in equation 4 - which, consequently, decreases the value of the lateral load acting on the panel;

- Still referring to the American Standard [1], it is noticed that its verification procedure is uneconomical, because in determining the rupture of the panel by flexo-compression it is compromising the whole wall to this effort, which is incoherent. This fact is validated, from the analysis of Figures 08 and 14, referring to numerical models with a pre-compression level equivalent to 1,85 MPa, in which only the localized treatment of the compressed regions would be sufficient to increase their strength capacity to lateral stress.

7. Acknowledgements

The authors gratefully acknowledge the support of the Pro-Rectory of Research and Extension (PROPE) of the Pontifical Catholic University of Goiás - PUC Goiás and of the National Council for Scientific and Technological Development (CNPq).

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