

## CHARACTERIZATION OF THE TENSILE AND SHEAR BOND STRENGTH OF CONCRETE BLOCK MASONRY

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### ABSTRACT

*Masonry buildings are frequently subjected to lateral loads such as wind and inertial forces from earthquakes. These lateral loadings generate flexure and shear in masonry walls. Masonry is an anisotropic material where units are embedded in mortar joints which generate weakness planes in masonry walls. These planes are subjected to high stresses due to the flexure and shear generated from lateral loadings and in general they condition the failure of masonry walls. Thus, the knowledge about the mechanical properties of the bond between units and mortar is fundamental to design and evaluation of masonry structures. In this way, an experimental program is carried out to evaluate the behaviour of unit-mortar interface under tensile and shear stresses. Tensile bond strength is indirectly measured through four point loads tests whereas shear bond strength is measured through tests in triplet specimens. Results indicated that Eurocode 6 (2005) provide very safe values for flexural and shear strength of masonry. Related to shear tests, it was possible to note and quantify a residual strength of unit-mortar interface after the failure and the dilatancy behaviour.*

### 1- INTRODUCTION

Structures are often subjected to lateral loads from wind or, in zones of moderate or high seismicity, from seismic actions, meaning that structural systems have to be designed to resist these types of loading. In masonry buildings walls are the main structural elements that assure the structural stability for in-plane loads and out-of-plane loads. Masonry walls are the main elements that resist the in-plane loads and act in conjunction with beams over doors or windows connecting the masonry piers. Besides lateral loads, the walls are submitted to vertical loads since they constitute the main supports of slabs,

vaults and domes, meaning that a complex stress state develops in masonry walls.

Masonry behaves reasonably well under compressive loads. However, its tensile strength is much reduced, leading to early cracking due to shear and tensile stresses. The mortar joints in masonry structures are weakness planes where in general is concentrated the failure mode mainly due to the low resistance of interface unit-mortar. Therefore, the knowledge about the properties of interface unit-mortar in masonry structures is fundamental to design.

Thus, this paper aims to present results of an experimental program of characterization where masonry specimens

were tested under shear and tensile efforts in order to evaluate the interface unit-mortar properties. A discussion and comparison of these results with suggested values by Eurocode 6 (2005) is presented.

## 2- EXPERIMENTAL PROGRAM

### 2.1 - Units and mortar

Given the traditional use of concrete blocks for non-loadbearing walls such enclosure and partition walls in Portugal, it was decided to develop two new structural concrete masonry blocks in order to make the use of distinct masonry bond patterns possible and also to enable the introduction of horizontal and vertical reinforcements, as presented in Haach (2009). Two (2C-units) and three cell (3C-units) concrete blocks were designed according to the shape and geometry indicated in Fig. 1.

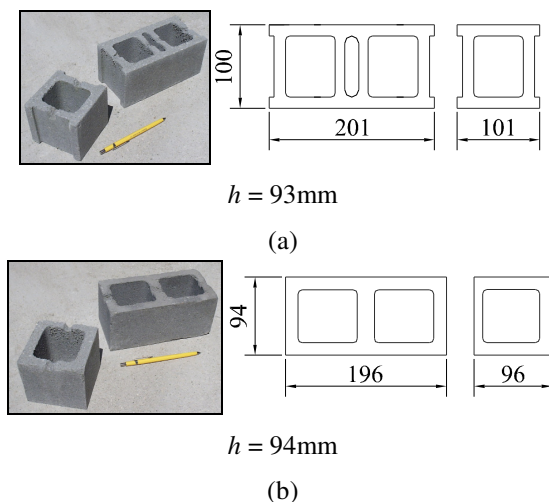


Fig. 1 – Geometry and shape of the units; (a) three cell blocks (3C-units); (b) two cell blocks (2C-units). Dimensions are in mm.

The concrete units were produced in reduced scale (1:2) in order to comply with technical limitations at the structural laboratory of University of Minho to perform real scale tests on masonry walls tested by Haach (2009). The idea of using frogged ends in 3C-units is the placement of vertical reinforcements in a continuous vertical joint in order to simplify the construction technology of reinforced masonry. The 2C-units has a geometry very similar to non structural concrete blocks existing in the Portuguese market

and are a real possibility for the traditional masonry bond.

In Eurocode 6 (2005), units are classified in four groups according to geometrical requirements such as percentage of voids and thickness of webs and shells. According to the classification proposed in Eurocode 6 (2005) both units belong to group 2.

Mechanical properties of units are fundamental to the design. The mechanical properties of concrete units evaluated include the tensile strength,  $f_{bt}$ , and compressive strength,  $f_{bc}$ , and Elastic modulus,  $E_b$ . A summary of the results on the mechanical properties of concrete blocks under tension and compression is indicated in Table 1. All mechanical properties were calculated in relation to gross area of the specimens. Compressive tests were performed according to EN 772-1 (2000).

Table 1 – Mechanical properties of units.

	$f_{bt}$ (MPa)	$f_{bc}$ (MPa)	$E_b$ (GPa)
Block (2C-units)	3.13	9.38	8.80
Half block (2C-units)		9.27	8.21
Block (3C-units)	3.19	12.13	9.57
Half block (3C-units)		10.33	9.44

A general purpose mortar was adopted, being composed of cement and sand in the proportion of 1:3 (cement/sand) with water/cement ratio equal to 0.9. The cement used was CEM II/B-L 32.5N, according to EN 197-1 (2000). The sand had a fineness modulus of 1.8 and a maximum diameter of 2.35mm.

Mechanical behaviour of mortar was defined through compressive and flexural strength ( $f_{mc}$  and  $f_{mf}$  respectively). Compressive and flexural tests were carried out on prismatic specimens 40mmx40mmx160mm according to EN 1015-11(1999). Three specimens of mortar were cast for each type of test.

Table 2 – Mechanical properties of mortar

	$f_{mc}$ (MPa)	$f_{mf}$ (MPa)
Flexural tests	10.15	2.50
Shear tests	8.35	2.15

## 2.2 - Flexural tests

Flexural tests on masonry were carried out according to EN1052-2 (1999). Six masonry wallets were built with the two geometries of concrete blocks. Specimens had two units of length and seven courses height with an 8mm-joint, see Fig. 2a. Three LVDTs were used to measure the vertical displacements, see Fig. 2b.

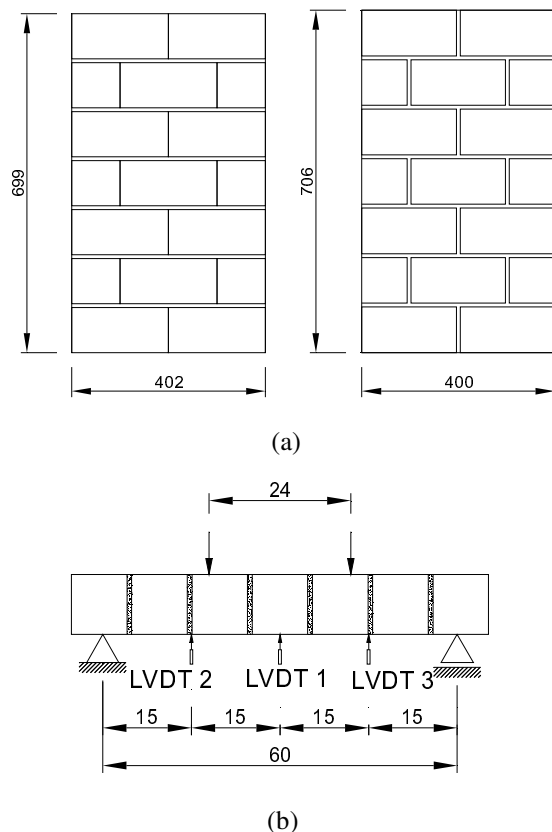


Fig. 2 – Flexural tests: (a) Specimens and (b) Test setup.

## 2.3 - Shear tests

Initial shear tests were carried out according to EN1052-3 (2002). Three distinct pre-compression levels ( $\sigma$ ) were applied in specimens (4 kN, 12 kN and 20 kN equivalent to 0.22 MPa, 0.66 MPa and

1.10 MPa in case of 2C-units and 0.2 MPa, 0.6 MPa and 1.0 MPa in case of 3C-units). Six specimens were built for each pre-compression level, totalizing 18 samples for each type of unit used in this study. Specimens were built with one unit of length and three courses with a 8mm joint, see Fig. 3a. The pre-compression was applied through four steel cables forming a self equilibrated system, see Fig. 3b.

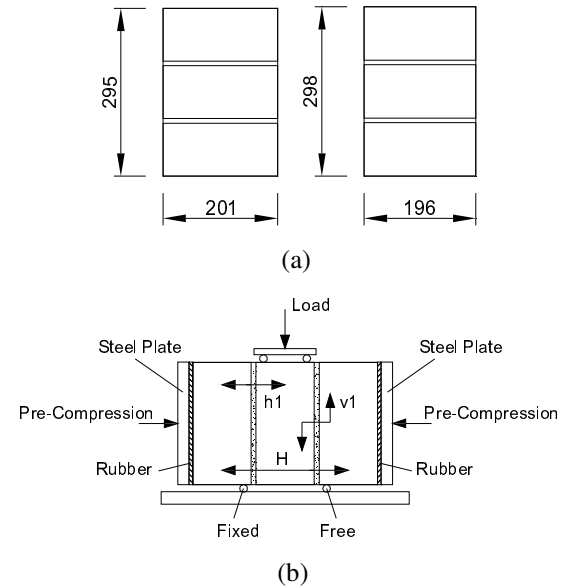


Fig. 3 – Initial shear tests: (a) Geometry of specimens and (b) Test setup.

Rubber pieces were used at the extremities of the samples to avoid concentration of stresses. A set of LVDTs was used in both sides of specimens to analyse the behaviour of unit-mortar interface under shear stresses. Three LVDTs were used to evaluate horizontal displacements of the joints: two LVDTs measuring horizontal displacements of each individual joint and one LVDT, indicating measuring the global horizontal displacement. Besides, two LVDTs were attached to the specimen to record the shear displacement of the joints.

## 3- RESULTS AND DISCUSSION

### 3.1 - Flexural behavior

According to EN 1052-2 (1999) flexural strength is calculated following Eq. (1). This value corresponds to the

tensile stress in the middle section of the specimen caused by the flexure.

$$f_x = \frac{3P(l_1 - l_2)}{2bt^2} \quad (1)$$

where  $f_x$  is the flexural strength,  $P$  is the maximum load applied,  $l_1$  is the spacing of the outer bearings,  $l_2$  is the spacing of the inner bearings,  $b$  is the length of specimen and  $t$  is the width of specimen.

The flexural stiffness of masonry can be calculated based on the moment vs. curvature diagram. From the displacements measured by the LVDTs it is possible to define the curvature in the zone of pure flexure, see Fig. 4. The curvature is given by Eq. (2).

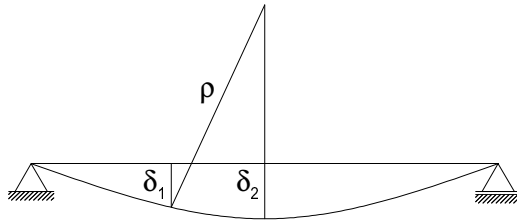


Fig. 4 – Curvature through the displacements in flexural test.

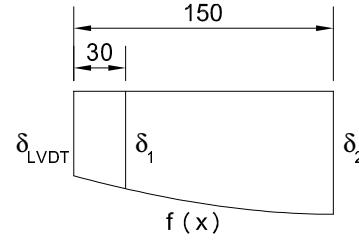
$$\frac{1}{\rho} = \kappa = \frac{2(\delta_2 - \delta_1)}{(\delta_2 - \delta_1)^2 + \left(\frac{l_2}{2}\right)^2} \quad (2)$$

where  $\delta_1$  is the displacement of the point corresponding to one third length of the specimen and  $\delta_2$  is the displacement measured by LVDT1.

LVDTs 2 and 3 were not positioned in the region with constant bending moment and null shear force. So, values of these displacements should be corrected in order to calculate the curvature. Thus,  $\delta_1$  was calculated using the mean of displacements of LVDT2 and LVDT3 and using an equation of second order to represent the deformed shape of specimen, see Fig. 5.

The value of stiffness was transformed to the gross area to keep consistency with the other results, see

Fig. 6. It is observed that both types of specimens, built with 3C-units and 2C-units, presented negative curvatures at the beginning of the test. It means that displacements of the thirds were higher than displacements in the center of the panel. This behaviour can be explained by the asymmetric deformation of the rubbers, used in supports to better distribute the stresses during the accommodation of the structure.



$$f(x) = Ax^2 + Bx + C$$

$$f(0) = \delta_{LVDT}, \quad f(150) = \delta_2, \quad f'(150) = 0$$

$$\delta_1 = 0.64\delta_{LVDT} + 0.36\delta_2$$

$$\delta_2 = \delta_{LVDT1} \quad \delta_{LVDT} = \frac{\delta_{LVDT2} + \delta_{LVDT3}}{2}$$

Fig. 5 – Correction of displacements in flexural test.

The flexural mechanical properties of masonry are presented in Table 3, namely the mean,  $f_{xm}$ , and the characteristic,  $f_{xk}$ , flexural strength and the flexural stiffness, EI.

Table 3 – Mechanical flexural properties.

Type of unit	$f_{xm}$ (MPa)	$f_{xk}$ (MPa)	EI (kNm <sup>2</sup> )
2C-units	0.31 (13%)	0.24	141.80 (7%)
3C-units	0.41 (4%)	0.38	123.44 (16%)

It is seen that specimens built with 3C-units had a higher flexural strength and lower flexural stiffness than the masonry built with 2C-units. The proximity of the internal webs in 3C-units promoted the union of the excess mortar during the laying, providing a better tensile bond strength at the unit-mortar interfaces. On

the other hand, the filling of vertical joints in wallets built with 2C-units probably was the responsible by the higher stiffness.

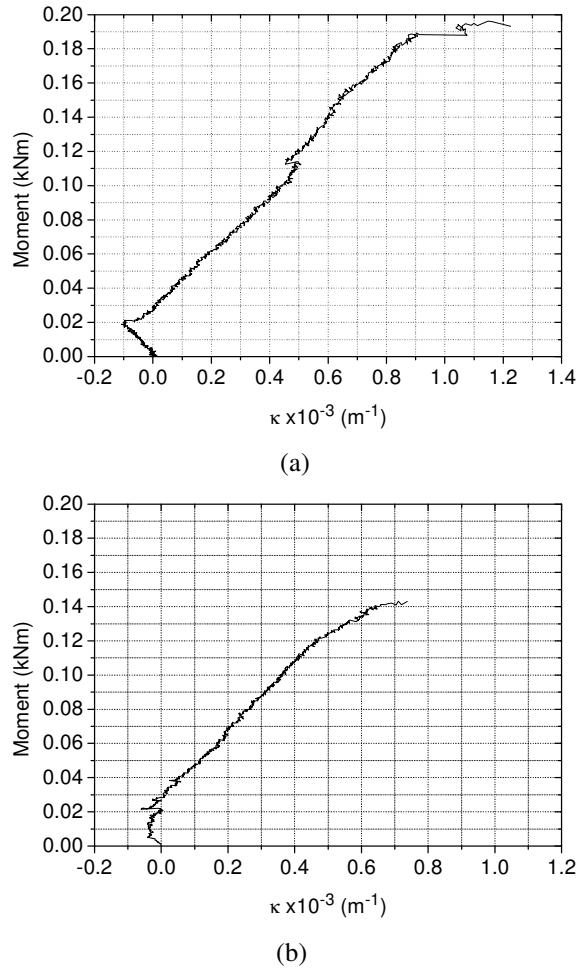


Fig. 6 – Moment vs. curvature diagrams in flexural tests: (a) 3C-units and (b) 2C-units.

Comparing experimental results with value of flexure strength equal to 0.10 MPa suggested by Eurocode 6 (2005), it can be noticed that the standard underestimates the capacity of masonry under flexure.

### 3.2 - Shear tests

According to EN 1052-3 (2002), test specimens should have one of four different types of failure.

a) Shear failure in the unit/mortar bond area either on one or divided between two units face;

b) Shear failure of the mortar;

c) Shear failure of the unit;

d) Crushing and/or splitting failure of the units.

All tested specimens presented the failure at the unit-mortar bond area either on only one or divided between two units face. In some specimens, horizontal cracks on the units appeared at the end of test, after the slide of the central unit, see Fig. 7. This behaviour can be caused by some expansion of the interface as observed by horizontal LVDTs.

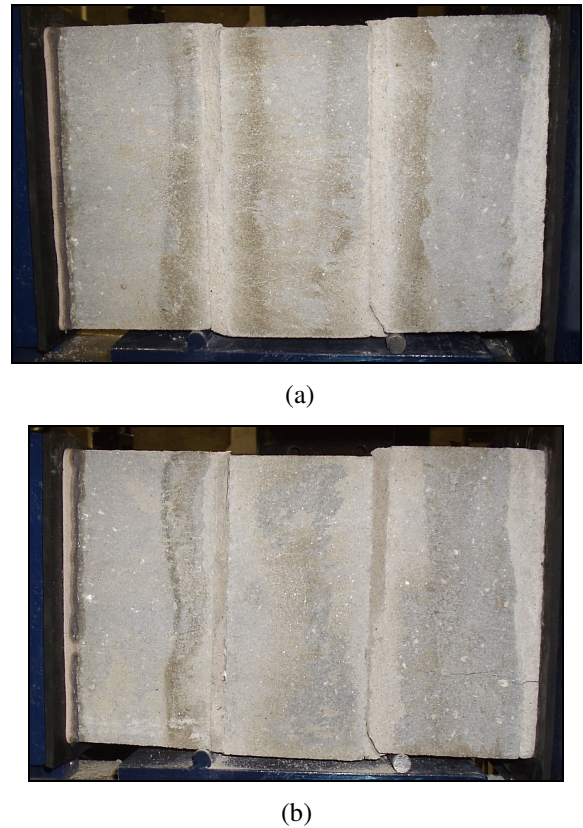


Fig. 7 – Failure mode of shear tests: (a) sliding and (b) sliding with horizontal crack.

A summary of the shear strength properties obtained in shear tests, namely the initial cohesion,  $f_{vo}$  ( $f_{vok}$  is the characteristic value), and the coefficient of friction  $\mu_k$  ( $\mu_k$  is the characteristic value), is shown in see Table 4. All properties were calculated in relation to gross area of the specimen. The cohesion is practically the double in case of 3C-units. On the other hand, it is seen that the coefficient of friction is similar in both geometries of the concrete blocks, see Table 4 and Fig. 8. The friction coefficient depends only on the concrete-mortar surface contact. As the concrete and mortar

was the same for both geometries of the blocks the friction coefficient was expected to be similar. The higher cohesion recorded in 3C-units can probably be attributed to the proximity of the internal webs leading to the union of the excess of mortar during the laying providing a better adherence between the courses.

Table 4 – Results of peak shear strength properties.

Type of unit	$f_{vo}$ (MPa)	$f_{vok}$ (MPa)	$\mu$	$\mu_k$
2C-units	0.21	0.17	0.49	0.39
3C-units	0.42	0.34	0.49	0.40

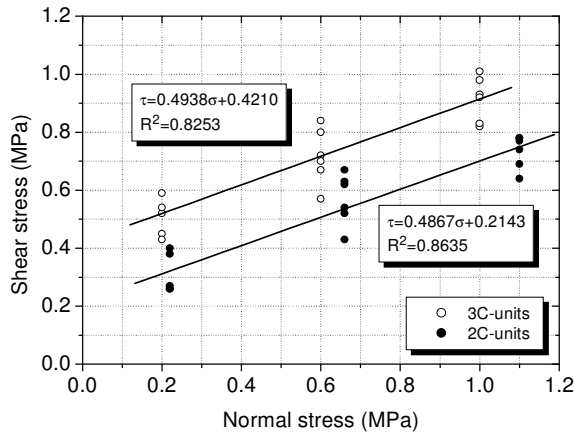


Fig. 8 – Shear stress vs. normal stress diagram.

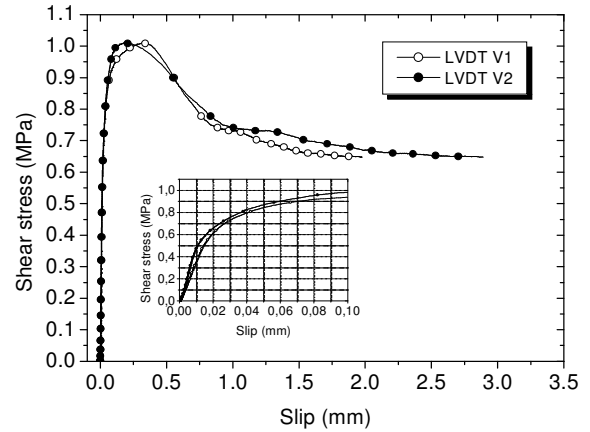
As observed by other authors (Vasconcelos, 2005; Abdou et al., 2006), after the peak the shear stress had a gradual decrease and stabilizes in a residual value, see Fig. 9. The residual shear strength properties are presented in Table 5.

Table 5 – Results of residual values of initial-shear tests.

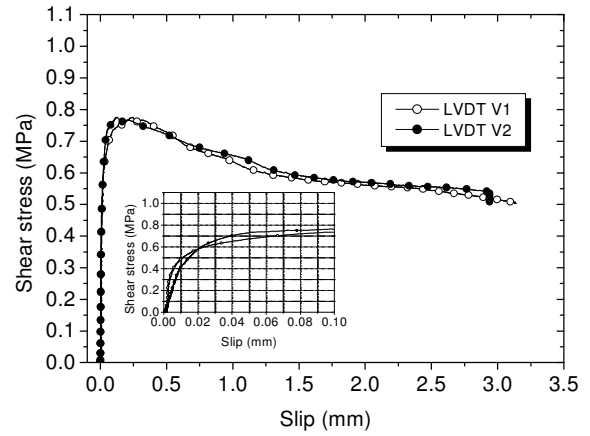
Type of unit	$f_{vo, res}$ (MPa)	$f_{vok, res}$ (MPa)	$\mu_{res}$	$\mu_{k, res}$
2C-units	0.14	0.12	0.32	0.25
3C-units	0.16	0.13	0.43	0.34

In the Table  $f_{vo, res}$  is the residual cohesion,  $f_{vok, res}$  is the characteristic value of residual cohesion,  $\mu_{res}$  is the residual coefficient of friction and  $\mu_{k, res}$  is the characteristic value of the residual coefficient of friction. The relation between normal and shear stresses fit also

reasonably well a linear function, see Fig. 10.



(a)



(b)

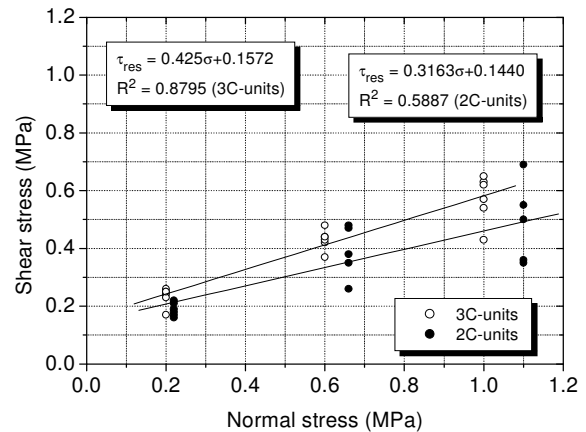
 Fig. 9 – Shear-slipping diagrams: (a) 3C-units –  $\sigma = 1.00$  MPa and (b) 2C-units –  $\sigma = 1.10$  MPa.


Fig. 10 – Residual shear stress vs. normal stress diagram.

The cohesion reduced approximately 62% and 33% for specimens built with 3C-units and 2C-units respectively, meaning that if cohesion is neglected in design of masonry walls underestimated shear

strength can be achieved. According to Abdou et al. (2006), the existence of this residual cohesion can be explained by the penetration of mortar into the holes, which avoids the separation of the blocks. Thus, this value can be used for evaluation of the shear sliding resistance of walls or piers submitted to seismic action failing along horizontal sliding joints (Calvi et al., 1996). The residual friction coefficient is 12% and 35% lower than peak friction coefficient in specimens built with 3C-units and 2C-units respectively. In this case the higher reduction occurs for 2C-Units.

Mode II fracture energy was calculated according to Pluijm (1999), see Fig. 11. A high variation was observed in results. However, it was clear that  $G_f^{II}$  depends of the normal stress applied on the mortar joints, see Table 6 and Fig. 12. All properties were calculated in relation to gross area of the specimen. From the linear fitting to the experimental results it is possible to conclude that the mode II fracture energy can be calculated from Eq. (3):

$$G_f^{II} = A + B\sigma \quad (3)$$

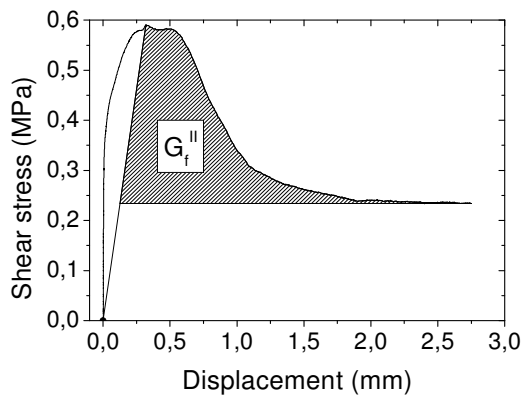


Fig. 11 – Mode II fracture energy.

Table 6 – Results of mode II fracture energy obtained from initial-shear tests.

Type of unit	A	B
2C-units	0.02	0.30
3C-units	0.19	0.13

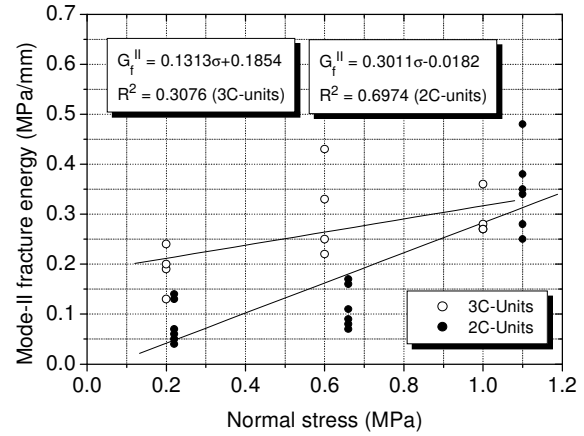


Fig. 12 – Variation of mode II fracture energy with normal stresses.

Specimens built with 3C-units exhibited higher values of mode II fracture energy than specimens built with 2C-units. This behavior is probably due to the higher penetration of mortar into the holes blocks in 3C-units.

The shear stiffness of the unit-mortar interface was also calculated from the shear stress vs. shear slipping. It seems not to be influenced by the normal stresses, see Fig. 13. However, the scatter of the results was very high, which means that more tests should be carried out to evaluate this mechanical property.

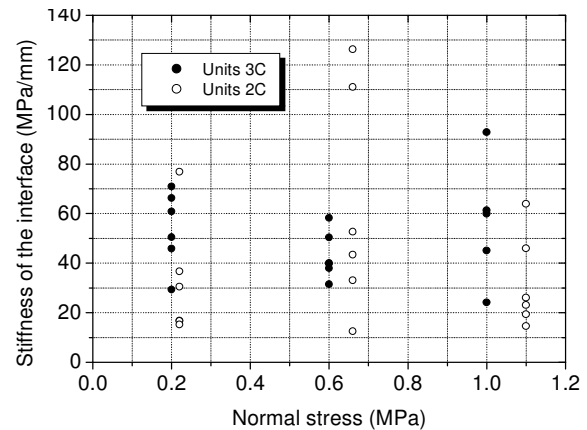


Fig. 13 – Relation between shear stiffness and normal stress.

Horizontal deformation of the specimens was also observed during the test. All horizontal LVDTs exhibited similar behaviour at the beginning of the test and presented small values. With no symmetrical damages of the interface some rotations appeared in specimen and the

horizontal displacements became quite different, see Fig. 14.

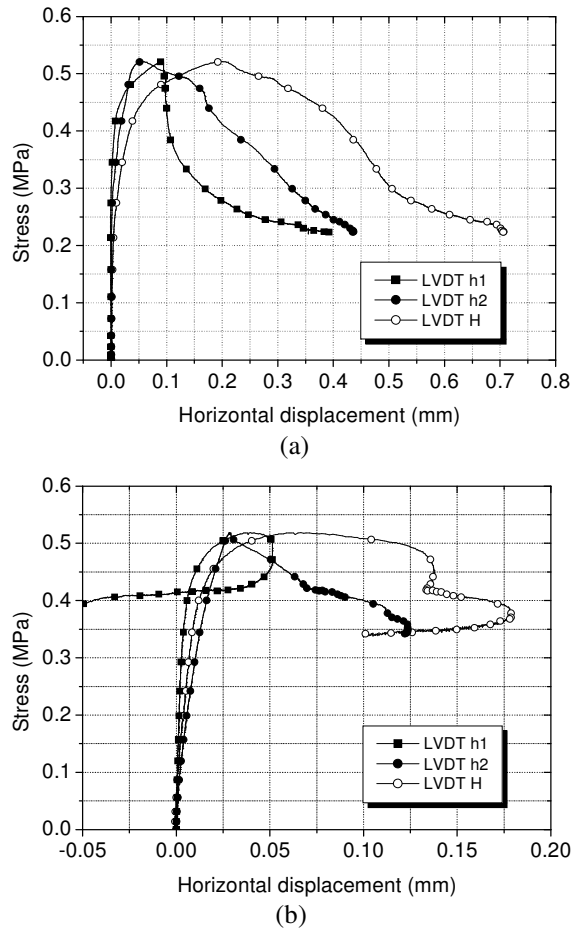


Fig. 14 – Horizontal behaviour of the specimens in initial shear tests: (a) 3C-units –  $\sigma = 0.20$  MPa and (b) 2C-units –  $\sigma = 0.66$  MPa.

The relation between the horizontal displacement and vertical displacement define the tangent of the dilatancy angle ( $\tan \psi$ ). As after the beginning of damage there were some rotations, LVDT H was used to define the dilatancy. Half of the values recorded in LVDT H were considered since there were two joints in the measured distance. Thus, dilatancy was calculated as the tangent of the horizontal displacements vs. vertical displacements diagrams, see Fig. 15. As observed by Pluijm (1999) and Vasconcelos (2005), dilatancy decreases with the increasing of pre-compression, see Fig. 16. Dilatancy seems to have a linear variation in relation to normal stress and can be given by Eq. (4):

$$\tan \psi = A + B\sigma \quad (4)$$

The values of the variables A and B in the equation are defined in Table 7.

Table 7 – Results of dilatancy of initial shear tests.

Type of unit	A	B
2C-units	0.41	-0.19
3C-units	0.52	-0.38

Specimens built with 3C-units had a high coefficient of correlation equal to 0.91. On the other hand, specimens built with 2C-units had a high scatter on the results.

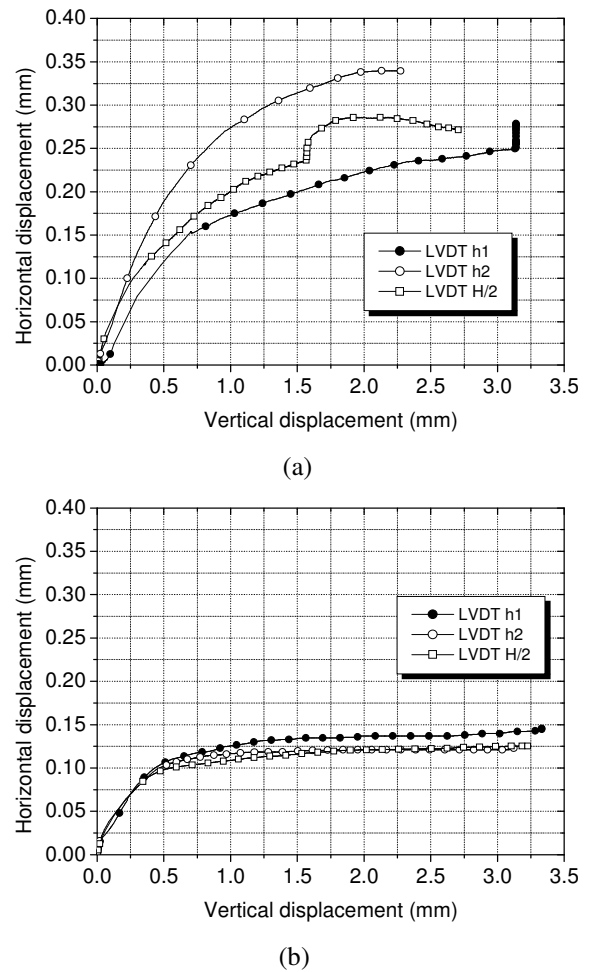


Fig. 15 – Horizontal displacements vs. vertical displacements of the specimens in initial-shear tests: (a) 3C-units –  $\sigma = 0.20$  MPa and (b) 2C-units –  $\sigma = 0.66$  MPa.



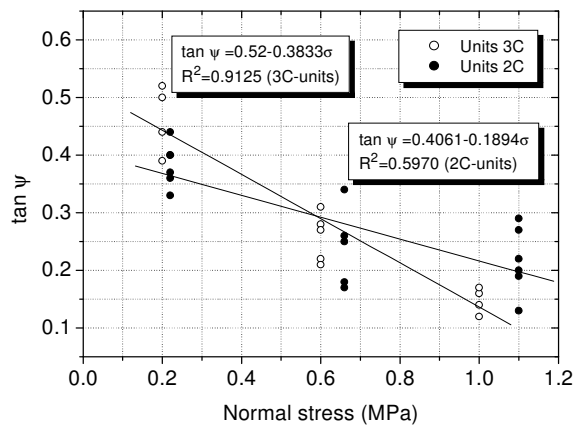


Fig. 16 – Relation between dilatancy and normal stress.

#### 4- CONCLUSIONS

In general could be observed that properties of interface unit-mortar can be influenced by several variables. The absence of research in this field should encourage the researchers to perform more studies in order to better understand this specific region in masonry structures since the interface has a keypoint in this type of structure.

Results of flexural tests indicated that flexure strength suggested by Eurocode 6 (2005) is very low underestimating the capacity of masonry under flexure. It means that more study should be performed in order to obtain more realistic values to flexural properties of masonry.

The results of shear tests showed that shear strength followed the Coulomb's law with a linear relation between shear and normal stresses. The shear strength of unit-mortar interface is higher in 3C-units, possibly due to connection of mortar of the internal webs, avoiding the separation of the blocks. Besides, it was observed that dilatancy depends on the normal stresses.

Finally, it should be stressed that the knowledge of the mechanical properties and the understanding of the behaviour of masonry and masonry materials is fundamental to analyse the experimental and numerical results of the masonry structural elements. Besides, the

mechanical properties are essential for the design of masonry structures.

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