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Mechanical characterization of autoclaved aerated concrete masonry subjected to in-plane loading: Experimental investigation and FE modeling

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1	Mechanical characterization of autoclaved aerated
2	concrete masonry subjected to in-plane loading:
3	experimental investigation and FE modeling
4	
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13	
14	ABSTRACT
15	This paper aims to provide a mechanical characterization of autoclaved aerated
16	concrete (AAC) masonry with thin bed joints subjected to in-plane loading. To this
17	purpose, a detailed experimental program has been carried out on masonry beams
18	subjected to bending and masonry panels subjected to uniaxial and biaxial loads. The
19	obtained results have highlighted an almost isotropic behavior of the material. The
20	collected data have been applied to calibrate a well-known numerical macro-model
21	available in the technical literature for the analysis of classical masonry structures. The
22	effectiveness of the proposed procedure has been finally verified by simulating the

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- *experimental behavior of a full-scale AAC bearing wall, through nonlinear finite*
- *element analysis.*

# **KEYWORDS**

- 27 AAC; thin bed masonry; experimental tests; constitutive model; mechanical properties;
- 28 finite element analysis.

#### **30 1. INTRODUCTION**

In recent years, the increasing demand for flexibility, comfort and energy saving in residential and industrial buildings has led both designers and manufactures to adopt new constructive systems, like Autoclaved Aerated Concrete (AAC) masonry.

34 AAC is a lightweight cementitious product of calcium silicate hydrates, whose 35 low density is obtained by the presence of air bubbles in the matrix – thanks to the 36 addition of aluminum powder in the mixture during the liquid or plastic phase - to 37 produce a cellular structure [1]. Therefore, it offers excellent sound and thermal 38 insulation properties [2, 3], which have led in the past decades to an increasing use of 39 this material for non-structural applications, especially cladding and infill panels. 40 Anyway, AAC is also characterized by a good mechanical strength and fire-resistance 41 (due to its incombustible nature) that make it suitable for the realization of masonry 42 bearing walls of low-to-medium rise buildings, even in seismic zones [3-10]. Compared 43 with conventional concrete (including concrete made with lightweight aggregates), 44 AAC has typically a lower density (which in turn reduces the seismic inertial forces 45 acting on the structure), ranging from one-sixth to one-third, and by a lower 46 compressive strength, which is almost reduced in the same ratio. The tendency to absorb 47 water, related to the porous structure of the material, can further reduce its structural 48 performances [11]; consequently, specific construction details can be required in order 49 to achieve a satisfactory static behavior.

50 Since the aforementioned peculiarities make AAC different from conventional 51 masonry materials, it is of fundamental importance to provide a detailed experimental 52 characterization of its mechanical behavior. Test results allow indeed to calibrate 53 sophisticated numerical models to be used in structural analyses, both for the design of

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new structures and for the assessment/retrofit of existing ones [12].

55 As known, masonry assemblages such as shear walls, infill walls in framed 56 construction or walls supported on beams are generally subjected to a biaxial state of 57 stress, due to the presence of normal stresses parallel and perpendicular to the bed 58 joints, as well as of shear stresses along the joints themselves. Moreover, unreinforced 59 conventional masonry generally exhibits anisotropic properties due to its composite 60 structure, with mortar joints acting as planes of weakness. Therefore, its failure cannot 61 be described solely in terms of the two principal stresses, but a third variable – related to 62 bed joint orientation - must be also considered. For these reasons, several researches in 63 the past focused their attention on the experimental determination of reliable parameters 64 of masonry strength, as well as on the development of failure criteria for masonry 65 elements subjected to in-plane loading (e.g. [13-16]). One of the most complete 66 experimental campaigns relative to masonry subjected to proportional biaxial loading 67 was carried out by Page [17, 18]. These tests were performed on half-scale square 68 panels made of solid clay units to investigate the influence exerted on failure mode and 69 strength by bed joint orientation (with respect to the vertical principal stress direction), 70 as well as by the applied principal stress ratio. Based on these experimental data, biaxial 71 failure surfaces were first derived in terms of the two principal stresses and their 72 orientation to bed joints ([17], [18]) and subsequently in terms of the stress system 73 related to the direction of the joints [19], which is better suited for finite element 74 modeling. Moreover, the same test results were also used in [20] to determine 75 macroscopic elastic and non-linear stress-strain relations. However, it is worth noticing 76 that the strength envelope obtained by Page is of limited applicability for other types of 77 masonry, characterized by different materials, unit shapes and/or geometry. For

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example, the influence of joint orientation was found to be less significant for grouted
concrete masonry, whose experimental behavior under biaxial stresses seems to be
essentially isotropic [21]. Further experimental investigations were also carried out on
masonry panels subjected to in-plane forces, by considering different unit geometries
and materials (concrete blocks, calcium-silicate blocks and clay bricks [22], or grouted
unreinforced brick masonry [23]), so as to define suitable failure criteria.

84 It should be also remarked that the results provided from the abovementioned 85 experimental programs could be hardly extended to AAC masonry, also because this 86 latter belongs to "thin bed masonry" typology. The units are indeed connected together 87 through thin glue layers, with thickness usually ranging between 0.5 and 3 mm. 88 Researches carried out on thin bed masonry (among others, e.g. [24-27]) have shown 89 that joint thickness significantly affects masonry behavior. As an example, the 90 compressive strength of thin bed masonry is higher with respect to conventional 91 masonry, since it tends to approach the strength of the blocks. Moreover, its shear and 92 flexural strengths are not significantly affected by the interface bond behavior and 93 therefore thin bed masonry performs more similarly to a continuum under loading, 94 without excessive localization of the failure path along the joints. Biaxial compression 95 tests carried out by Vermeltfoort [28] on thin bed masonry panels with different joint 96 orientations showed that their failure mechanism was characterized by the three 97 following phenomena: spalling of the units with fragments of approximately 20 mm, 98 vertical splitting, and bending of the sample.

99 It is not clear if AAC masonry displays the same behavior. The most of the 100 experimental programs carried out in recent years on this specific type of masonry were 101 indeed mainly focused on the assessment of its seismic performances and were devoted

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to the development of seismic design provisions to be included in Design Codes (among
others, e.g. [8]-[10], [29]-[31]).

104 Aim of this research work is to provide a complete description of AAC masonry 105 behavior under in-plane static loading, with particular attention to the softening regime. 106 To the purpose, several experimental tests are performed on both AAC masonry panels 107 and beams, so as to characterize masonry behavior under uniaxial and biaxial 108 compression, flexure and shear, determining not only its elastic parameters and strength 109 values, but also the fracture energies in tension and compression. The collected 110 experimental data can be useful in the calibration of suitable numerical models; as an 111 example, in this work an anisotropic nonlinear constitutive model, well-known in the 112 technical literature for FE analysis of ordinary masonry structures [32-34], is adapted to 113 AAC masonry elements. The so calibrated model is subsequently validated by 114 performing a nonlinear FE analysis on a full-scale AAC masonry wall subjected to a 115 pushover test [31].

116

## 117 2. EXPERIMENTAL PROGRAM ON AAC MASONRY ELEMENTS

118 The performed experimental program consisted in 33 tests on AAC masonry 119 elements with thin bed joints. A general view of some of the assembled specimens 120 before testing is shown in Figure 1.

121 In order to characterize the material behavior in compression, several monotonic 122 uniaxial and biaxial tests were performed on small-scale masonry panels, by varying the 123 bed joint orientation with respect to the horizontal axis. Some of the uniaxial tests were 124 carried out under displacement control to obtain the complete stress-strain curve and the

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## 125 corresponding fracture energy in compression.

# 126

128 Due to the limited features of the universal testing machines at our disposal, 129 tensile behavior of AAC masonry was instead investigated indirectly, by performing 130 three-point bending tests on small-scale masonry beams. In this case, only two angles of 131 inclination between the bed joints and the horizontal axis were considered (namely 0° 132 and 90°). For each examined typology, two beams were provided of a central notch to 133 guide crack formation and were tested under crack mouth opening displacement 134 (CMOD) control, so obtaining the complete load-deflection response and the 135 corresponding fracture energy in tension. Finally, two small masonry panels were 136 subjected to diagonal compression tests.

# 137 2.1 Description of test specimens

All the specimens were prepared by using masonry-type AAC units directly provided by the Manufacturer. It is worth noticing that AAC units are commonly produced in different sizes that may reach 625×250×200 mm; in this last case, masonry panels including a representative number of head and bed joints would be huge and some problems may arise to test them into the frame of a universal testing machine.

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<sup>127</sup> *Figure 1.* General view of some of the AAC masonry specimens tested during the experimental program.

143 However, considering that the dimensions of the units available on the market are 144 variable, in the present work it has been preferred to employ non-standard small size 145 bricks, with nominal dimensions equal to 250×50×100 mm. In this way, it has been 146 possible to keep the specimen size small, while having at the same time an adequate 147 number of head and bed joints, so emphasizing their influence on masonry global 148 behavior and increasing possible anisotropic effects. Therefore, the behavior of 149 structural elements realized with larger units and a more limited number of joints should 150 lie between the two "limit cases" of homogeneous material (previously investigated by 151 the same Authors in [35]) and the here investigated masonry formed by small units. In 152 any case, experimental evidences have shown that the use of scaled bricks does not 153 seem to alter substantially the results, as will be discussed in more details in the 154 forthcoming Sections.

A deep characterization of the raw autoclaved aerated concrete adopted for the realization of units can be found in [35]. Its main mechanical characteristics were: average density  $\rho_b = 550 \text{ kg/m}^3$ , average compressive strength  $f_b = 3.1 \text{ MPa}$  (as determined on cubes with an edge length of 100 mm), average modulus of rupture  $f_{t,b} = 0.6 \text{ MPa}$ , and elastic modulus  $E_b = 1320 \text{ MPa}$ .

160 Units were assembled by using a specific cementitious grey glue produced by the 161 same Manufacturer, mainly composed of Portland cement, silica sand and specific 162 additives, with a water dosage equal to 24% in weight. This grey glue was a guaranteed 163 performance mortar characterized by the following main properties: average density 164  $\rho_g = 1300 \text{ kg/m}^3$ , average compressive strength of cylindrical specimens  $f_g = 7 \text{ MPa}$  (it 165 was instead equal to 6 MPa when determined on  $40 \times 40 \times 160 \text{ mm prisms}$ ), average 166 modulus of rupture  $f_{t,g} = 2.7 \text{ MPa}$ , elastic modulus  $E_g = 5300 \text{ MPa}$ . The nominal

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167 thickness of head and bed joints was equal to 1.5 mm.

To allow a complete drying of the glue and the reaching of stationary moisture conditions, all the specimens were stored in laboratory for more than three months. At time of test execution, all samples were characterized by an average moisture content approximately equal to 2.4% and by an average density  $\rho = 550 \text{ kg/m}^3$ .

172 As already mentioned, uniaxial and biaxial compression tests were performed on 173 23 small-scale masonry panels, with length of 625 mm, thickness equal to 100 mm and 174 height equal to 750 mm. Two additional uniaxial tests were performed on smaller 175 square panels with an edge length of 250 mm, characterized by the same thickness. This 176 last specimen geometry was also adopted for the two diagonal compression tests. 177 Flexural tests were instead carried out on six small-scale masonry beams with length of 178 625 mm, thickness equal to 100 mm and height equal to 250 mm. Further details about 179 sample characteristics and adopted test arrangements are reported in the following 180 Sections.

## 181 2.2 Characterization of AAC masonry panels in compression

# 182 2.2.1 Uniaxial compression tests

Uniaxial compression tests were performed on 17 small-scale masonry panels (Figure 2a). In order to study the influence exerted by the geometrical arrangement of units and joints on masonry compressive strength, 5 specimen typologies, characterized by a different inclination  $\theta$  of glue beds – equal to 0°, 22°, 45°, 68° and 90° with respect to the horizontal direction – were considered, as depicted in Figure 2a. The same Figure also provides specimen nominal dimensions, as well as their denomination, which is composed by the acronym PMC (which stands for Panel Masonry Compression),

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190 followed by two digits, the first one representing the angle of inclination of glue beds

and the second one (in brackets) the total number of tested samples belonging to a



192 considered typology.

194 Figure 2. Sketch of uniaxial compression tests on masonry panels (a) PMC and (b) PMCs (characterized
195 by reduced dimensions).

196 A larger part of this experimental program (15 samples) was carried out at the 197 Laboratory of the AAC Manufacturer Company, by using a Metrocom PV50 press 198 working under loading control, with a capacity of the hydraulic actuator equal to 199 5000 kN [36]. The adopted test arrangement is shown in Figure 3; in order to apply a 200 distributed load, a 650 mm long steel rigid beam with I-section was placed on the top of 201 the sample. Panel surfaces were preliminary flattened by sandpaper to eliminate any 202 irregularity and thereby ensure a complete contact between the specimen itself and the 203 testing apparatus. Furthermore, two overlapping Teflon sheets were interposed to 204 minimize the confinement effect due to friction and apply a uniform state of stress. The 205 most of these tests simply provided the uniaxial compressive strengths  $f_{mx}$ ,  $f_{my}$  in the two 206 masonry principal directions (respectively parallel, x, and perpendicular, y, to glue 207 beds). Three of these 15 samples – denoted as PMC0-2, PMC0-3, and PMC90-1 – were 208 also instrumented with 6 linear variable displacement transducers (LVDTs) aligned 209 along x and y directions on the two opposite panel faces, as shown in Figure 3b. The 210 LVDTs were installed to measure vertical and horizontal strains ( $\varepsilon_v$  and  $\varepsilon_h$ ), so allowing 211 the evaluation of the elastic moduli  $E_x$  and  $E_y$  in the two masonry principal directions, as 212 well as the Poisson coefficient v. Moreover, it was also possible to follow the initial part 213 of the softening branch by performing a gradual unloading of the specimen after the 214 reaching of the peak load.



Figure 3. General setup of uniaxial compression tests performed under loading control: (a) not
instrumented and (b) instrumented small-scale masonry panels (PMC).

215 216

The remaining two samples (respectively indicated as PMC0-7 and PMC90-2 according to Figure 2a) were tested at the Materials and Structures Laboratory of Milan Polytechnic University, by using a 1000 kN Schenck press working under displacement control (Figure 4a). Two 650 mm long steel rigid beams with I-section were placed on the top and bottom bases of the specimen, by interposing two thin Teflon sheets in order to reduce friction. All samples were instrumented with 6 LVDTs aligned along *x* and *y* 

- directions on the two opposite panel faces, as shown in Figure 4a. These tests providedthe complete stress-strain curve for the material in compression.
- 227 The so obtained results were subsequently integrated by testing two additional smaller
- 228 panels (Figure 2b), respectively characterized by horizontal (PMCs0) and vertical
- (PMCs90) glue beds.





Figure 4. General setup of uniaxial compression tests performed under displacement control: (a) smallscale masonry panels (PMC) and (b) square specimens with reduced dimensions (PMCs).

In this last case, a 100 kN Instron 8862 press working under displacement control was

used and the samples were still instrumented with 6 LVDTs aligned along x and y

directions on the two opposite panel faces, with two further vertical LVDTs in the

thickness (Figure 4b). These two additional specimens were realized in order to achieve

- a better description of the post-peak response, since bigger panels displayed local
- 239 failures that determined in some cases sudden jumps in the softening branch of the
- stress-strain curve.



The main results of the above-described tests are summarized in Table 1 in terms

242 of experimental failure load  $P_{v,u}$  and corresponding vertical compressive stress  $\sigma_v$  for

Sampla	#	Test	L	t	Н	P <sub>v,u</sub>	σ	σ	σ	$\tau_{xy}$
Sample	#	control	(mm)	(mm)	(mm)	(kN)	(MPa)	(MPa)	(MPa)	(MPa)
PMC0	1	LC	622.00	100.90	740.00	160.82	-2.56	0.00	-2.56	0.00
PMC0	2	LC	621.00	99.50	746.00	151.27	-2.45	0.00	-2.45	0.00
PMC0	3	LC	621.00	99.11	745.00	167.50	-2.72	0.00	-2.72	0.00
PMC0	4	LC	622.00	99.86	744.00	140.75	-2.27	0.00	-2.27	0.00
PMC0	5	LC	623.00	99.71	745.00	171.33	-2.76	0.00	-2.76	0.00
PMC0	6	LC	623.00	99.87	747.00	151.27	-2.43	0.00	-2.43	0.00
PMC0	7	DC	623.00	100.00	756.00	172.86	-2.77	0.00	-2.77	0.00
PMCs0	1	DC	240.00	100.00	250.00	67.14	-2.80	0.00	-2.80	0.00
PMC90	1	LC	610.00	100.00	741.00	156.00	-2.56	-2.56	0.00	0.00
PMC90	2	DC	610.00	100.00	740.00	143.57	-2.35	-2.35	0.00	0.00
PMCs90	1	DC	250.00	100.00	246.00	72.03	-2.88	-2.88	0.00	0.00
PMC22	1	LC	627.00	100.00	751.00	141.84	-2.26	-0.32	-1.94	0.79
PMC22	2	LC	625.00	100.00	749.00	143.59	-2.30	-0.32	-1.98	0.80
PMC45	1	LC	627.00	100.00	747.00	133.08	-2.12	-1.06	-1.06	1.06
PMC45	2	LC	625.00	100.00	748.00	137.46	-2.20	-1.10	-1.10	1.10
PMC45	3	LC	623.00	99.50	749.00	139.21	-2.25	-1.12	-1.12	1.12
PMC45	4	LC	625.00	100.00	748.00	139.21	-2.23	-1.11	-1.11	1.11
PMC68	1	LC	624.00	100.00	748.00	134.83	-2.16	-1.86	-0.30	0.75
PMC68	2	LC	625.00	100.00	748.00	128.70	-2.06	-1.77	-0.29	0.72

each tested panel.

244 LC = loading control

245 DC = displacement control

246 Table 1. Uniaxial compression tests on AAC masonry panels (PMC and PMCs): effective dimensions of

247 *the specimens and experimental failure loads.* 

- 248 The same Table also reports the total stress state related to bed joints  $\sigma_x$ ,  $\sigma_y$  and  $\tau_{xy}$
- 249 (being x, y the directions respectively parallel and perpendicular to bed joints, as
- 250 depicted in Figure 2a), which has been subsequently deduced by using the following
- standard relations [19]:

252 
$$\sigma_x = \frac{\sigma_v + \sigma_h}{2} - \frac{\sigma_v - \sigma_h}{2} \cos(2\theta)$$

253 
$$\sigma_{y} = \frac{\sigma_{v} + \sigma_{h}}{2} + \frac{\sigma_{v} - \sigma_{h}}{2} \cos(2\theta)$$
(1)

254  $\tau_{xy} = \frac{\sigma_v - \sigma_h}{2} \sin(2\theta) \,.$ 

255 These equations, that are valid for a general biaxial state of stress (being  $\sigma_v$  and  $\sigma_h$  the

vertical and horizontal principal stresses, respectively, and  $\theta$  the angle between bed joints and the horizontal axis), have been here applied to the uniaxial case by simply posing  $\sigma_h = 0$ . Table 1 also indicates the effective specimen dimensions (*L*, *t*, *H*), as well as the adopted type of testing, that is to say under displacement or loading control (respectively indicated as DC and LC).

Experimental results highlighted that the compressive strengths in the two principal directions *x-y* – respectively obtained as the average peak stress values  $\sigma_v$  for PMC90 and PMC0 samples – were almost equal to each other ( $f_{mx} \approx f_{my} \approx 2.60$  MPa). A slight reduction of masonry compressive strength - ranging between 13 and 24% - was instead observed for other angles of bed joints (that is  $\theta = 22^\circ$ , 45°, 68°).

Based on the results of this investigation, it seems that bed joint orientation exerts only a limited influence on the compressive behavior of tested panels, as also reported in the literature for other types of thin bed masonry [25]. This is mainly due to the isotropic behavior of AAC units (which do not have internal perforations), as well as to the presence of thin glue joints with a relatively high strength.

271 Moreover, AAC masonry compressive strength in the two principal directions x-y272 appears to be only slightly lower with respect to that of the raw material, as determined 273 on AAC prisms and cubes sawed from some of the tested masonry specimens (Figure 274 5a). The first ones, characterized by a 40 mm square basis and 80 mm height, provided 275 indeed an average uniaxial compressive strength approximately equal to  $f_{AAC} = 2.8$  MPa, 276 while for 40 mm side cubes a value of  $f_{AAC,cube} = 3.1$  MPa was found. These values were 277 almost coincident with those provided from standard tests previously carried out on the 278 adopted raw material (see [35] for further details), as well as with those available in the 279 technical literature for a material with similar density and moisture content [5-6].

Anyway, it should be kept in mind that AAC compressive strength depends on the geometry and dimensions of tested blocks. Compression tests performed on raw-AAC panels with nominal dimensions equal to those of masonry ones (625 x 750 x 100 mm, Figure 5b) and produced by the same Manufacturer provided indeed an average compressive strength  $f_c = 2.4$  MPa, which is comparable to that obtained from masonry

285 specimens.

286



Figure 5. Uniaxial compression tests on homogeneous AAC samples: (a) cubes with an edge length of 40
mm and (b) panels with the same geometry and dimensions of PMC masonry ones.

289 This result is not surprising when considering both the high strength of the 290 cementitious glue and the relatively small size of adopted units, which allowed a 291 random redistribution of the defects in the tested panel. In homogeneous samples, 292 failure was instead mainly localized near the weaker edge, related to the expansion 293 process. Moreover, at time of test execution, masonry samples were characterized by a 294 slightly lower moisture content with respect to homogeneous ones ([35]). 295 Masonry elastic properties are summarized in Table 2. Elastic moduli were 296 determined as the chord slope of the stress - vertical strain curve (within a stress interval 297 ranging from 0.05  $\sigma_v$  and 0.33  $\sigma_v$ ), so obtaining an average value respectively equal to 298  $E_x = 1700$  MPa for PMC90 samples (with vertical glue beds) and  $E_y = 1400$  MPa for

299	PMC0 samples (with horizontal glue beds). This last value was calculated by discarding
300	the result relative to PMCs0 sample, which was characterized by an anomalous more
301	rigid behavior compared to the other ones belonging to the same typology. The obtained
302	values are of the same order of magnitude as the ones provided by Manufacturer
303	certifications and technical sheets, which range between 1400 MPa and 1750 MPa
304	(referred to a masonry with horizontal glue beds). Poisson coefficient was instead
305	evaluated as the ratio between horizontal and vertical strains ( $\varepsilon_h / \varepsilon_v$ ), obtaining an
306	average value almost equal to $\nu \simeq 0.30$ in both the two main directions. It can be
307	observed that the so determined elastic properties are similar, even if not identical, to
308	those previously derived on homogeneous panels, which were respectively equal to
309	$E = 1352$ MPa and $\nu = 0.38$ , as reported in [35].

310

Table 2 also summarizes the strains values corresponding to peak stresses, which

Sample	#	Test control	E <sub>x</sub> (MPa)	E <sub>y</sub> (MPa)	v (MPa)	ε <sub>рх</sub> (‰)	€ру (‰)
PMC0	2	LC		1417	0.32		2.00
PMC0	3	LC		1392	0.38		2.20
PMC0	7	DC		1346	0.29		2.40
PMCs0	1	DC		1737	0.35		1.68
PMC90	1	LC	1653		0.29	1.90	
PMC90	2	DC	1713		0.26	1.50	
PMCs90	1	DC	1719		0.25	2.10	

311 were respectively equal, on average, to  $\varepsilon_{px} = 1.8\%$  and  $\varepsilon_{py} = 2.2\%$ .

312 LC = loading control

313 DC = displacement control

314 *Table 2.* Uniaxial compression tests on AAC masonry panels (PMC and PMCs): elastic moduli  $E_x$ ,  $E_y$ , 315 Poisson ratio v and compressive peak strains  $\varepsilon_{px}$  and  $\varepsilon_{py}$ .

316 Finally, the observed crack patterns at failure are shown in Figure 6. Depending

317 on the orientation of the bed joints to the applied load, failure mainly occurred by

318 cracking and sliding in the bed and/or head joints, or in a combined mechanisms

involving cracking in both units and joints. As can be seen from Figure 6a-b, for both
PMC0 and PMC22 panels a major crack developed in the direction perpendicular to bed
joints, alternatively crossing AAC blocks and thin glue layers. On the contrary, in
PMC90 and PMC68 samples (Figure 6d-e), cracks mainly developed along glue beds,
while for PMC45 ones (Figure 6c) the observed failure mode was less defined, with
diagonal cracks spreading at the same time both in glue beds and AAC blocks.



326 *Figure 6.* Uniaxial compression tests on AAC masonry panels (PMC): observed crack patterns at failure

327 *as function of bed joint inclination.* 

# 328 2.2.2 Biaxial compression tests

329 Biaxial compression tests were carried out on the same panel typologies already

330 subjected to uniaxial compression, as depicted in Figure 7a. As can be seen from the 331 same Figure, these panels were named PMB (which stands for Panel Masonry Biaxial 332 compression), followed by two digits, the first one representing the angle of inclination 333 of glue beds with respect to the horizontal axis and the second one (in brackets) the total 334 number of tested samples belonging to a considered typology. In this case, three 335 different inclinations of bed joints were considered  $(0^\circ, 22^\circ, and 45^\circ)$  for a total of 6 336 samples, two for each typology. Since equal compressive forces were applied in the two 337 principal directions, 68° and 90° inclinations were indeed coincident with 22° and 0°.



Figure 7. (a) Sketch of biaxial compression tests on masonry panels (PMB); (b) sketch and (c) general
view of the adopted setup.

342 Vertical load  $P_v$  was applied by using the same Metrocom PV50 press already 343 used for uniaxial compression tests, by adopting a similar loading arrangement (with a 344 steel rigid beam on the top of the sample and thin Teflon layers interposed between the

345	sample and the loading apparatus). Lateral confinement $P_h$ was instead applied by
346	adopting the device depicted in Figure 7b-c, formed by a system of steel rigid beams
347	connected together through steel ribbed bars. These beams were placed on the sides of
348	the AAC sample (by interposing the usual thin Teflon layers) and were used as contrast
349	for two oil-pressure jacks, aligned in the horizontal direction. Biaxial tests were
350	performed under loading control by monitoring that the same value of vertical and
351	horizontal pressure was simultaneously applied on sample surfaces (the two loading
352	devices were indeed not directly connected to each other). In this case, none of the
353	specimen was instrumented.
354	The main results of the above described tests are summarized in Table 3, in terms
355	of experimental failure loads $P_{v,u}$ and $P_{h,u}$ , as well as corresponding stresses $\sigma_v$ and $\sigma_h$ .
356	From these values, the total stress state related to bed joints $\sigma_x$ , $\sigma_y$ and $\tau_{xy}$ has been also
357	determined (being $x$ , $y$ the directions respectively parallel and perpendicular to bed
358	joints, as depicted in Figure 7a), by still applying Equations 1. The obtained results
359	confirm that the bed joint angle exerts only a limited influence on the strength of
360	masonry; moreover, the biaxial strength value is substantially comparable with the
361	uniaxial one. Table 3 also reports the effective specimen dimensions $(L, t, H)$ .

Sampla	#	Test	L	t	Н	Pv,u	$\sigma_{v}$	P <sub>h,u</sub>	$\sigma_{h}$	σ	σ	$\tau_{xy}$
Sample	#	control	(mm)	(mm)	(mm)	(kN)	(MPa)	(kN)	(MPa)	(MPa)	(MPa)	(MPa)
PMB0	1	LC	624	100	761	138.09	-2.21	165.52	-2.18	-2.18	-2.21	0.00
PMB0	2	LC	623	100	760	143.10	-2.30	171.61	-2.26	-2.26	-2.30	0.00
PMB22	1	LC	626	100	749	137.66	-2.20	159.54	-2.13	-2.14	-2.19	0.02
PMB22	2	LC	625	100	750	146.19	-2.34	173.48	-2.31	-2.32	-2.34	0.01
PMB45	1	LC	626	100	748	124.51	-1.99	142.12	-1.90	-1.95	-1.95	0.04
PMB45	2	LC	624	100	748	137.22	-2.20	161.57	-2.16	-2.18	-2.18	0.02

362 363 LC = loading control

DC = displacement control

364 Table 3. Biaxial compression tests on AAC masonry panels (PMB): effective dimensions of the specimens

365 and experimental failure loads.

- 366 The observed crack patterns at failure are depicted in Figure 8. As can be seen, all
- 367 panels showed the same failure mode, characterized by an out-of-plane expansion in the
- 368 unconfined direction, regardless of bed joint orientation. Spalling of small masonry
- 369 fragments was also observed.



370

371 Figure 8. Biaxial compression tests on AAC masonry panels (PMB): observed crack patterns at failure as

372 *function of bed joint inclination.* 

# 373 2.3 Characterization of AAC masonry beams in flexure

- 374 Three-point bending tests were performed on six AAC small-scale masonry
- beams, with nominal dimensions equal to 625 x 250 x 100 mm (Figure 9).



376

**Figure 9.** Three-point bending tests on AAC masonry beams BMF (a) without and (b) with notch:

378 *adopted setup.* 

379 The main characteristics of the tested samples, as well as the adopted 380 nomenclature are summarized in Figure 10. In this case, the acronym BMF (which 381 stands for Beam Masonry Flexure) has been adopted, followed by a number 382 representing the angle of inclination of bed joints with respect to the horizontal axis; a 383 further digit (in brackets) indicates the number of tested specimens belonging to the 384 same typology. In more detail, 4 specimens BMF0 (with horizontal bed joints) and 2 385 specimens BMF90 (with vertical bed joints) were realized; for each typology, 2 beams 386 were provided of a central notch, so as to guide crack formation. It should be here 387 observed that the two BMF90 specimens were characterized by different effective 388 lengths and net spans, as highlighted in Table 4. The first sample was indeed formed by 389 an odd number of units (13), so having the notch placed at half-width of the central 390 brick line; however, the experimental failure was localized in correspondence of one of 391 the adjacent glue joints. For this reason, the other specimen was realized with an even 392 number of units (one less than sample BMF90-1), to place the notch exactly in 393 correspondence of the glue joint.



394



397 The two unnotched specimens were tested at the Laboratory of the AAC398 Manufacturer Company, by using an Instron 5882 press working under loading control,

- 399 with a loading rate of about 1 kN/min (Figure 9a). The 4 notched samples were instead
- 400 tested at the Materials and Structures Laboratory of Milan Polytechnic University, by
- 401 using an Instron 8862 press working under CMOD control, with a loading rate of
- 402 1 μm/min (Figure 9b). All the specimens were instrumented with a LVDT properly
- 403 fixed on a bar installed over the two supports, in order to monitor the true midspan
- 404 deflection during test execution (Figure 9 a-b).
- 405 The performed tests provided the failure load in flexure  $P_{u,fl}$  for the two

406	investigated bed joint angles, as reported in Table 4.
100	mi estiguieu seu Joint angles, as reported in ruore in

Sampla	#	Test	L	t	Н	a	1	Pu,fl	<b>f</b> ' <sub>tx</sub>	Gfx	f' <sub>ty</sub>	Gfy
Sample	#	control	(mm)	(mm)	(mm)	(mm)	(mm)	(kN)	(MPa)	(N/mm)	(MPa)	(N/mm)
BMF0	1	LC	625.4	100.6	251.2	0.0	545.4	2.67	0.34	-		-
BMF0	2	LC	624.2	100.1	251.7	0.0	544.2	2.94	0.38	-		-
BMF0	3	DC	625.0	100.0	250.0	37.5	545.0	2.34	0.42	6.4E-03		-
BMF0	4	DC	626.0	100.0	258.0	25.0	546.0	2.27	0.34	7.8E-03		-
BMF90	1	DC	660.0	100.0	250.0	12.5	600.0	2.56		-	0.37	4.5E-03
BMF90	2	DC	610.0	100.0	240.0	24.0	550.0	1.31		-	0.23	5.8E-03

407 LC = loading control

408 DC = displacement control

409 *Table 4.* Three-point bending tests on AAC masonry beams (BMF): effective dimensions of the specimens,

410 *experimental failure loads, indirect tensile strengths and fracture energies in tension.* 

411 The corresponding flexural tensile strengths (moduli of rupture) in the two

412 examined directions were subsequently determined through a liner elastic FE inverse

413 analysis, so obtaining an average value respectively equal to  $f'_{tx} = 0.37$  MPa and

414  $f'_{ty} = 0.30$  MPa. Moreover, tests carried out under CMOD control allowed the

- 415 determination of the complete post-peak load-deflection response and consequently of
- 416 the two average fracture energies in tension, respectively equal to  $G_{fx} = 7.10 \cdot 10^{-3}$  N/mm
- 417 and  $G_{fy} = 5.15 \cdot 10^{-3}$  N/mm. The obtained results, as well as the effective sample
- 418 dimensions, are reported in detail in Table 4. As can be seen, the first BMF90 sample
- 419 provided a tensile strength comparable to that obtained for BMF0 samples, while the

second one presented a value approximately 40% lower, probably due to a not properfilling of the central glue joint.

It can be also observed that both the strength and the fracture energy of masonry
samples in the two examined directions are comparable to those of the homogeneous
material, as reported in the technical literature [5, 35, 37, 38].

# 425 2.4 Characterization of AAC masonry panels in diagonal compression

426 The characterization of AAC masonry behavior was completed through the testing 427 of two small panels in diagonal compression, as depicted in Figure 11a-b. Those 428 samples, named PMS (Panel Masonry Shear), were characterized by nominal 429 dimensions equal to 250 x 250 x100 mm. The tests were still carried out at the Materials 430 and Structures Laboratory of Milan Polytechnic University, by using an Instron 8862 431 press to apply the vertical load  $P_{\nu}$ . In order to avoid the crushing of loaded angles and to 432 obtain a uniform state of stress, thin cardboard layers were interposed between the 433 loading platens and the sample, as shown in Figure 11b.





438 As can be seen from the same Figure, each sample was instrumented with 2

	$\overline{\tau}$ . Test L t H Py fy $\sigma_x$ $\sigma_y$ $\tau_{xy}$
447	strength [40].
446	common empirical relations, such as $f_{v0} \approx 0.15 f_{\rm m}$ , being $f_m$ the masonry compressive
445	high if compared to those suggested in technical Codes (e.g., [39]) or determinable with
444	the panel and $t$ is the thickness. As can be observed from Table 5, these values are rather
443	panel. The latter has been evaluated as $A_n = t (H+L)/2$ , where H and L are the sides of
442	strength (under zero compressive stresses) $f_{v0} = 0.7P_v / A_n$ , being $A_n$ the net area of the
441	in Table 5, in terms of vertical load at failure $P_v$ and corresponding masonry shear
440	The obtained results, as well as the effective sample dimensions, are summarized
439	vertical and 2 horizontal LVDTs, so as to record panel strains with increasing loads.

Sampla	#	Test	L	t	Н	Pv	fv0	σ	$\sigma_{y}$	$\tau_{xy}$
Sample	#	control	(mm)	(mm)	(mm)	(kN)	(MPa)	(MPa)	(MPa)	(MPa)
PMS	1	DC	240	100	240	19.91	0.59	-0.49	-0.49	0.92
PMS	2	DC	250	100	250	25.74	0.73	-0.60	-0.60	1.14

448 LC = loading control 449 DC = displacement control

452 The same Table also reports the total stress state related to bed joints  $\sigma_x$ ,  $\sigma_y$  and

453  $\tau_{xy}$ . As concerns the observed failure modes, both the specimen presented the spreading

454 of a main sub-vertical crack, alternatively crossing AAC blocks and glue joints (Figure

455 11c).

456

# 457 3. CALIBRATION OF A NUMERICAL MACRO-MODEL FOR AAC

## 458 MASONRY BASED ON THE EXPERIMENTAL RESULTS

- 459 The described experimental results have been used for the calibration of the well-
- 460 known macro-model proposed by Lourenço et al. [32-34] for the analysis of masonry
- 461 structures. This model, which treats masonry as an anisotropic continuum and describes

<sup>450</sup> *Table 5.* Diagonal compression tests on small AAC masonry panels (PMS): effective dimensions of the
451 specimens and experimental failure loads.

462 its behavior in terms of average stresses and strains, seems an appropriate tool for 463 simulating the behavior of AAC walls, as well as that of other types of thin bed masonry 464 [25], due to the quite limited influence of the interface. For finite element analyses, the 465 material model has been implemented in [41] into a computer algorithm that handles 466 plasticity features like return mapping, corners, apex, etc. This algorithm has been here 467 converted into a user material subroutine (UMAT) and implemented into the general 468 purpose finite element code ABAQUS [42] to perform numerical simulations. 469 In the following Sections, the adopted model will be first briefly described and its 470 calibration to the case of AAC masonry will be subsequently discussed.

### 471 3.1 Main features of the adopted macro-model

472 The considered model [32-34], which has been developed with reference to 473 general plane stress conditions, represents an extension of conventional formulations for 474 isotropic quasi-brittle materials to describe orthotropic behavior. Two different failure 475 criteria are adopted for tension and compression, respectively "Rankine-type" and "Hill-476 type", whose equations [33] are recalled in Figure 12a. As can be seen in Figure 12b, 477 the non-linear behavior of masonry in compression  $\overline{\sigma}_{ci} - \kappa_c$  (where the subscript *i* refers 478 to the material axis x or y) is described through a parabolic plastic stress-strain relationship ( $\overline{\sigma}_{ai} - \kappa_c$ , according to Fig. 12b) until the reaching of the peak value, 479 480 which is usually different in the two principal orthotropic directions x-y - namely  $\overline{\sigma}_{px} = f_{mx}$  and  $\overline{\sigma}_{py} = f_{my}$ . After the peak, a parabolic/exponential softening branch ( 481 482  $\overline{\sigma}_{bi} - \kappa_c$  and  $\overline{\sigma}_{di} - \kappa_c$ , respectively) is adopted in both directions, characterized by 483 different fracture energies  $G_{fcx}$  and  $G_{fcy}$ . The inelastic work  $g_{fci} = G_{fci}/h$  in Figure 12 is

-25-

related to the fracture energy  $G_{fci}$  through the equivalent length h, which corresponds to 484 485 a representative dimension of the mesh size (so that the obtained results are objective 486 with regard to mesh refinement). According to [32], the compressive law shown in 487 Figure 12b can be defined on the basis of three stresses (that is initial, mean and residual 488 ones), which are determined as a fraction of the peak value  $\overline{\sigma}_{pi} = f_{mi}$  through the following relations:  $\overline{\sigma}_{ii} = 1/3 f_{mi}$ ,  $\overline{\sigma}_{mi} = 1/2 f_{mi}$ ,  $\overline{\sigma}_{ri} = 1/10 f_{mi}$ . The equivalent plastic 489 strain  $\kappa_p$ , corresponding to the peak compressive strength, is assumed to be an 490 491 additional material parameter. Furthermore, in order to obtain a mesh independent 492 energy dissipation, it should be posed [32]:

493 
$$\kappa_{mi} = \frac{75}{67} \frac{G_{fci}}{h f_{mi}} + \kappa_p,$$
 (2)

494 with the limitations reported in [32, 34].

495 The nonlinear behavior of masonry in tension is instead described through an 496 exponential softening plastic stress-strain relationship  $\bar{\sigma}_{ti} - \kappa_t$ , with different tensile strengths ( $f_{tx}$  and  $f_{ty}$ ) and fracture energies ( $G_{fx}$  and  $G_{fy}$ ) in the two principal orthotropic 497 498 directions (Figure 12c). The exhaustive theoretical formulation of the adopted model 499 can be found in [32-34], to which reference is made for further details. 500 To be correctly calibrated, the considered model requires the knowledge of seven 501 parameters governing material strength (respectively indicated as  $f_{tx}$ ,  $f_{ty}$ ,  $f_{mx}$ ,  $f_{my}$ ,  $\alpha$ ,  $\beta$  and 502  $\gamma$ ), as well as five inelastic parameters governing the plastic stress-strain relationships 503  $(G_{fx}, G_{fy}, G_{fcx}, G_{fcy} \text{ and } \kappa_p)$ . The first four strength properties (which represent uniaxial 504 tensile  $f_{ii}$  and compressive  $f_{mi}$  strengths along the material axes i = x, y) can be 505 determined by performing uniaxial tension and compression tests on masonry

506 specimens, along the two main directions respectively parallel and perpendicular to

#### 507 mortar beds.

508



510 Figure 12. (a) Biaxial strength envelope for masonry; stress-strain laws adopted for the material in (b)
511 uniaxial compression and (c) uniaxial tension [32-34].

512 If these experimental tests are performed under displacement control, they also 513 provide the five required inelastic parameters, that is to say fracture energies in tension 514  $G_{fi}$  and compression  $G_{fci}$  in both directions *x-y*, as well as the plastic strain 515 corresponding to the peak compressive strength,  $\kappa_p$ . The complete calibration of the 516 model also requires additional non-standard tests [34] in order to determine the three 517 remaining parameters  $\alpha$ ,  $\beta$  and  $\gamma$  (Figure 12a); in more detail,  $\alpha$  weights the shear stress 518 contribution to tensile failure,  $\beta$  controls the coupling between normal stress values in 519 case of compressive failure and  $\gamma$  weights the shear stress contribution to compressive 520 failure.

#### 521 3.2 Calibration of the model for AAC masonry

#### 522 3.2.1 Biaxial failure envelope

523 The parameters required for the construction of the failure envelope proposed by 524 Lourenço et al. have been derived on the basis of the results provided by the previously 525 described experimental tests on AAC masonry elements. The compressive strengths in 526 the two main directions x, y - respectively obtained as the average peak stress values  $\sigma_v$ 527 for PMC90 and PMC0 samples (see Table 1) - have been assumed equal to  $f_{mx} = f_{my} = 2.60$  MPa. The direct tensile strengths, respectively equal to  $f_{tx} = 0.29$  MPa and 528  $f_{ty} = 0.24$  MPa, have been instead determined numerically, by simulating the three-point 529 530 bending tests carried out on samples BMF0 and BMF90 (Table 4), as better described in 531 Section 3.2.2.

The parameter  $\alpha$  has been calculated through the least squares method, by minimizing the function  $\sum f_{1,j}^2(\alpha)$  with the non-linear optimization algorithm of Levemberg-Marquard, being  $f_{1,j}$  the plastic potential in the *j*-th experimental point (Figure 12a); in this way, the value of  $\alpha = 0.5$  has been obtained. Similarly, the parameters  $\beta$  and  $\gamma$  have been calculated by minimizing the function  $\sum f_{2,j}^2(\beta, \gamma)$ , so obtaining  $\beta = -0.6$ and  $\gamma = 5$ .

538 The so calibrated failure envelope is depicted in Figure 13 in the  $\sigma_x - \sigma_y$  plane, 539 through level curves corresponding to different values of the applied shear stress  $\tau_{xy}$ , 540 respectively equal to 0, 0.76, and 1.09 MPa. On the same graph, the corresponding experimental values obtained from uniaxial and biaxial compression tests (Section 2.2), as well as from three-point bending (Section 2.3) and diagonal compression ones (Section 2.4), are also reported; as can be seen, the model error appears to be comparable with the experimental scattering. Furthermore, the obtained curves show that the behavior of AAC masonry is characterized by a weak anisotropy, with similar strength values in the two main directions *x*, *y*.



548 *Figure 13.* Comparison between the adopted failure envelope [32-34] and the experimental results 549 obtained for AAC masonry for different values of the applied shear stress  $\tau_{xy}$ .

#### 550 3.2.2 Stress-strain laws in uniaxial compression and tension

547

Based on the experimental results, the plastic stress-strain laws for uniaxial compression in the two main directions – whose equations are reported for reading convenience in Figure 12b – have been calibrated by adopting the average values of compressive strengths ( $f_{mx} = f_{my} = 2.60$  MPa), and elastic moduli ( $E_x = 1700$  MPa,

555  $E_y = 1400$  MPa). The plastic strain corresponding to the peak compressive strength has 556 been assumed equal to  $\kappa_p = 0.3\%$ . The two fracture energies in compression have been 557 deduced numerically, through an inverse procedure based on the fitting of all the available 558 experimental data (Figure 14 a-b), so as to take into account the important scatter of the 559 softening branches. Since the experimental response was quite similar in the two main directions (Figure 14c), also in this case a unique value of  $g_{fcx} = g_{fcy} = 2.26 \cdot 10^{-3} \text{ N/mm}^2$ 560 has been adopted. This value corresponds only to the local contribution of the  $\overline{\sigma}_{ci} - \kappa_c$ 561 562 diagram (where the subscript *i* refers to the material axis) and therefore the basis for its 563 definition is only numerical, in order to obtain objective results with respect to mesh 564 refinement [32].



569 *Figure 14.* Comparisons between the adopted inelastic law in compression for AAC masonry and the

571 and y; (d) definition of a unique inelastic law and comparison with all the experimental results.

572 The complete inelastic law adopted for compression in both the considered 573 directions, respectively parallel and perpendicular to bed joints, is reported in Figure 14d in the dimensionless plane  $\overline{\sigma}_{ci}/f_{mi}-\kappa_{c}/\kappa_{p}$ , where it is compared with the available 574 575 experimental data (specimens PMC0, PMCs0, PMC90, and PMCs90). On the same 576 Figure, the new parameters used herein for the definition of the shape of the curve are also reported, that is to say  $\overline{\sigma}_{ii} / f_{mi} = 0.5$ ,  $\overline{\sigma}_{mi} / f_{mi} = 0.85$ ,  $\overline{\sigma}_{ri} / f_{mi} = 0.33$ , with reference 577 578 to the nomenclature adopted in Figure 12b and according to [32]. Moreover, with the 579 previous assumptions, parameter  $\kappa_{mi}$  becomes:

580 
$$\kappa_{mi} = \frac{1080}{1627} \frac{G_{fci}}{h f_{mi}} + \kappa_p.$$
 (3)

581 The stress-strain laws in uniaxial tension – whose equations are reported in 582 Figure 12c – have been instead calibrated by hypothesizing the same elastic moduli 583 obtained for compression and by adopting the average fracture energies determined 584 from three-point bending tests on BMF0 and BMF90 samples (respectively equal to  $G_{fx} = 7.10 \cdot 10^{-3}$  N/mm and  $G_{fy} = 5.15 \cdot 10^{-3}$  N/mm, see Table 4). Direct tensile strengths 585 586 have been obtained from inverse analysis by using the general-purpose FE code 587 ABAQUS to simulate the above-mentioned three-point bending tests. To this scope, 588 each beam has been modeled by adopting a regular mesh formed by square 4-nodes 589 plane stress elements (CPS4 in the adopted FE code library), with 5 mm side, and this 590 discretization has been further refined in correspondence of the supports and of the 591 loading plate (Figure 15d). The problem of mesh dependence has been overcome by scaling the fracture energies through the equivalent length  $h = \sqrt{A_e}$ , where  $A_e$  is the 592

-31-

593 area of the adopted element [33].

594	The interaction between the supports and the masonry sample has been taken into
595	account by introducing contact elements and by considering friction between AAC and
596	steel. The analyses have been performed by adopting quasi-static loading control to
597	overcome convergence problems, so obtaining the complete load-deflection curve. The
598	computed tensile strength average values in the two main masonry directions are
599	respectively equal to $f_{tx} = 0.29$ MPa and $f_{ty} = 0.24$ MPa.



601 *Figure 15.* Comparisons between numerical and experimental curves obtained for AAC masonry beams 602 (BMF) in terms of applied load P vs. midspan deflection  $\delta$  in (a) x and (b) y directions. (c) Adopted 603 inelastic law in tension for AAC masonry in x and y directions; (d) FE mesh of AAC masonry beams.



607	experimental results are almost superimposed with each other and with the numerical
608	curve. On the contrary, experimental results relative to $y$ direction present a larger
609	scatter, which is due to the different characteristics of the examined samples. As already
610	mentioned, specimen BF90-1 was indeed formed by an odd number of units, with the
611	notch placed at half-width of the central brick line, while specimen BMF90-2 was
612	realized with an even number of units, with the notch placed exactly in correspondence
613	of the glue joint. As regards numerical modeling, the geometry of specimen BMF90-2
614	has been considered, while the adopted mechanical properties have been set equal to the
615	average values deduced from tests, so obtaining an intermediate response.
616	Finally, Figure 15c reports the corresponding exponential softening plastic stress-

strain relations  $\overline{\sigma}_{ti} - \kappa_t$  in x and y directions. 617

618

#### 619 4. SIMULATION OF THE BEHAVIOR OF A FULL-SCALE AAC MASONRY 620 WALL

621 The effectiveness of the so calibrated plastic model and its ability of correctly 622 describe the behavior of AAC masonry structures has been subsequently verified 623 through the simulation of an experimental test carried out at the University of Pavia on a 624 full-scale AAC masonry wall subjected to a vertical load and an increasing in-plane 625 horizontal force [31]. The geometry and the adopted test arrangement are schematized 626 in Figure 16a. As can be seen, the considered wall was 1.5 m long, 2.75 m high and 627 0.3 m wide, and it was assembled by using  $625 \times 300 \times 250$  mm AAC blocks, which are 628 bigger than those used to calibrate the adopted constitutive model. The specimen was 629 built with thin mortar layers (2-3 mm thick) and filled head joints. Reinforced concrete

-33-

630 beams – whose dimensions can be found in [31] – were built at the top of the wall, to 631 guarantee a better load distribution, and at the bottom, acting as a foundation. The 632 chosen test setup was indeed a cantilever system (fixed at the base and free at the top) 633 with a constant vertical load of 200 kN applied on the top beam through hydraulic jacks. 634 The horizontal load was instead applied through a displacement-controlled horizontal 635 hydraulic actuator, performing three fully reversed cycles for the chosen target 636 displacement level, until the reaching of a horizontal displacement equal to 0.6% of the 637 wall height [31].



639 Figure 16. Tests on a full scale AAC masonry wall: (a) sketch of the experimental setup [31]; (b) adopted
640 FE mesh.

The considered wall has been modeled by adopting a uniform mesh, formed by
square 4-node plane stress elements (CPS4) with 50 mm side, as shown in Figure 16b.
For sake of simplicity, the two reinforced concrete beams have not been included in the
FE model, by simply introducing contact elements at the base and considering friction
between the AAC wall and the support. The horizontal force has been replaced by a
uniform distribution of prescribed displacements δ. The mechanical behavior of AAC

647 masonry has been simulated through the constitutive model described in the previous 648 Section. Since the mechanical properties of AAC blocks used for the realization of the 649 examined wall were not exactly the same of those determined in the experimental 650 program illustrated herein, both the compressive strengths as well as the elastic moduli 651 were properly updated, according to the corresponding values reported in [31]. In more 652 detail, the following values have been assumed:  $f_{mx} = 1.90$  MPa,  $f_{my} = 2.20$  MPa,  $E_x = E_y = 1498$  MPa. On the contrary, for the other required elastic and inelastic 653 654 properties – which were not available in [31] - the values determined in Section 3 have 655 been assumed. This modeling choice seems to be reasonable, since the adopted model is 656 based on an orthotropic formulation and also its calibration to the case of AAC masonry 657 is based on weakly anisotropic data (e.g. the elastic moduli in the two principal 658 directions are different). As a consequence, the adoption of slightly different strength 659 values in the two principal direction (which are anyway comparable with those adopted 660 in Section 3), should not alter the effectiveness of the proposed approach. 661 The NLFE analysis has been carried out under displacement control, by simply 662 modeling the last loading cycle. Also in this case, a quasi-static analysis has been 663 performed to mitigate convergence problems, and the controlled displacement has been 664 increased monotonically up to failure. The so obtained results have been reported in 665 terms of applied load F vs. top horizontal displacement  $\delta$  in Figure 17, where they are 666 compared with the experimental response. The latter represents the envelope curve of 667 the three fully reversed cycles performed during the test. As can be observed, the model 668 is able to describe with sufficient accuracy the behavior of the examined full-scale wall 669 until the reaching of the ultimate load, whose value is also predicted.

-35-



671 *Figure 17.* Comparison between numerical and experimental results in terms of applied load F vs.

672 *horizontal displacement*  $\delta$ .

Anyhow, it should be underlined that the numerical response appears to be significantly less ductile than the experimental one. The underestimated ductility is due to important convergence problems that have been reported also in other works that adopt the same algorithm [43, 44]. In particular, van der Meer [44] analyzed the reasons of the convergence weakness of the subroutine (apex, return-mapping, multi-surface plasticity algorithm) and proposed some promising improvements that are, however, out of the scope of this work.

680

### 681 5. CONCLUSIONS

The present work illustrates the main results of an experimental program focused
on the mechanical characterization of AAC masonry. To this aim, the following test
typologies have been performed:

685 – uniaxial compression tests on masonry panels under force/displacement control;

686 – biaxial compression tests on masonry panels;

687 – diagonal compression tests on small masonry panels;

688 - three-point bending tests on masonry beams under force/CMOD control.

689 The results have been used to calibrate a well-known macroscopic anisotropic

690 constitutive model already developed for ordinary masonry and available in the

technical literature [32-34]. This model has been subsequently applied to simulate

numerically the behavior of a full-scale AAC masonry wall subjected to a pushover test

693 [31].

694 The main conclusions of this research are summarized herein:

695 – if properly calibrated, numerical anisotropic models proposed for traditional masonry

696 can be also used for AAC masonry;

697 - AAC masonry is characterized by a very weak anisotropy due to the particular

698 cementitious glue adopted for the realization of thin joints;

699 – observed failure modes reveal that if joints are correctly realized by skilled labor,

they do not represent a significant weakness in masonry behavior.

701 For these reasons, as a first approximation, the mechanical properties of the raw AAC

702 material (with a similar density and moisture content) can be used for mechanical

703 models and finite element simulations on masonry elements.

704

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