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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*

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

25

26 **KEYWORDS**

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

29

30 1. INTRODUCTION

31 In recent years, the increasing demand for flexibility, comfort and energy saving
32 in residential and industrial buildings has led both designers and manufactures to adopt
33 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

54 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

78 example, the influence of joint orientation was found to be less significant for grouted
79 concrete masonry, whose experimental behavior under biaxial stresses seems to be
80 essentially isotropic [21]. Further experimental investigations were also carried out on
81 masonry panels subjected to in-plane forces, by considering different unit geometries
82 and materials (concrete blocks, calcium-silicate blocks and clay bricks [22], or grouted
83 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

102 to the development of seismic design provisions to be included in Design Codes (among
103 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

125 corresponding fracture energy in compression.



126

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

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**

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

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.

155 A deep characterization of the raw autoclaved aerated concrete adopted for the
156 realization of units can be found in [35]. Its main mechanical characteristics were:
157 average density $\rho_b = 550 \text{ kg/m}^3$, average compressive strength $f_b = 3.1 \text{ MPa}$ (as
158 determined on cubes with an edge length of 100 mm), average modulus of rupture
159 $f_{t,b} = 0.6 \text{ MPa}$, 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 40×40×160 mm prisms), average
166 modulus of rupture $f_{t,g} = 2.7 \text{ MPa}$, elastic modulus $E_g = 5300 \text{ MPa}$. The nominal

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

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

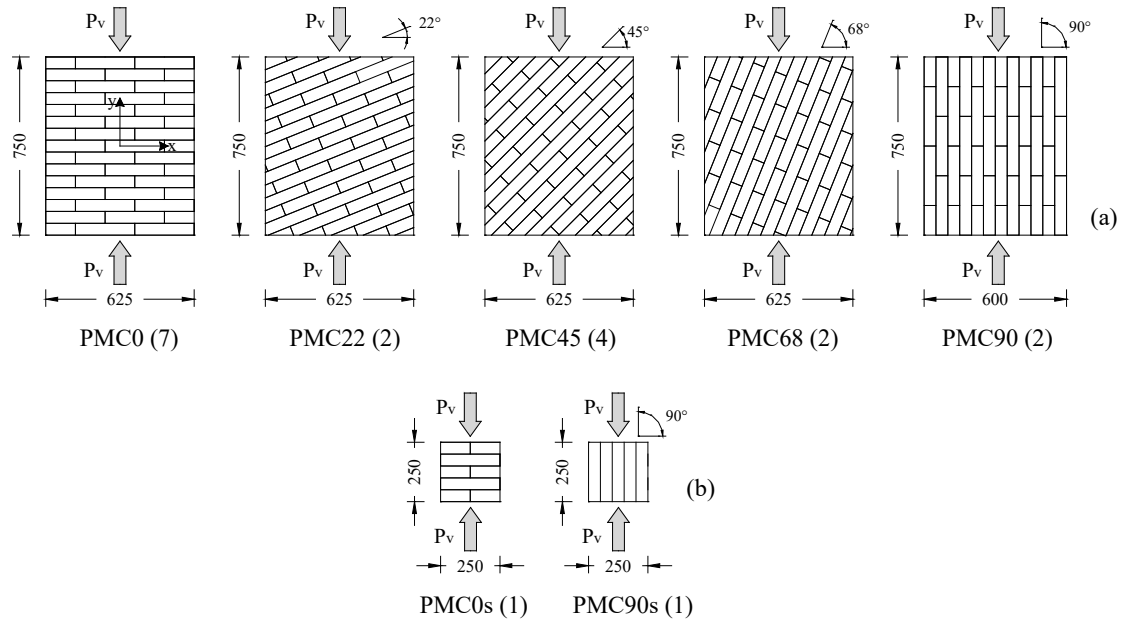
172 As already mentioned, uniaxial and biaxial compression tests were performed on
173 23 small-scale masonry panels, with length of 625 mm, thickness equal to 100 mm and
174 height equal to 750 mm. Two additional uniaxial tests were performed on smaller
175 square panels with an edge length of 250 mm, 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***

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

190 followed by two digits, the first one representing the angle of inclination of glue beds
 191 and the second one (in brackets) the total number of tested samples belonging to a
 192 considered typology.



193
 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 (ε_v and ε_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 ν . 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.

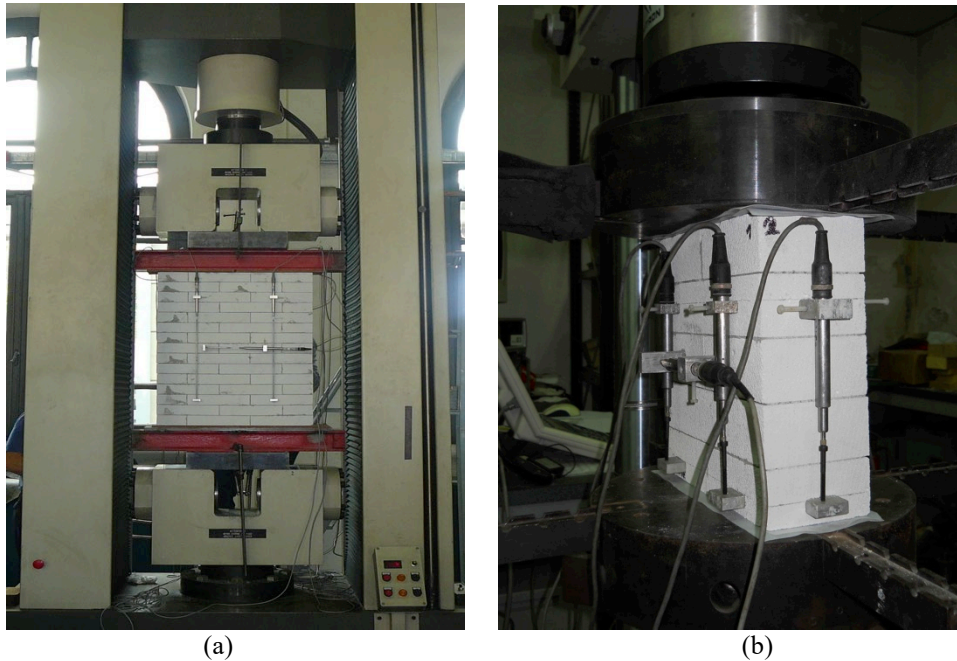


215
216

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

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

225 directions on the two opposite panel faces, as shown in Figure 4a. These tests provided
226 the 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
229 (PMCs90) glue beds.



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

234 In this last case, a 100 kN Instron 8862 press working under displacement control was
235 used and the samples were still instrumented with 6 LVDTs aligned along x and y
236 directions on the two opposite panel faces, with two further vertical LVDTs in the
237 thickness (Figure 4b). These two additional specimens were realized in order to achieve
238 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
240 stress-strain curve.

241 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 σ_v for
 243 each tested panel.

Sample	#	Test control	L (mm)	t (mm)	H (mm)	$P_{v,u}$ (kN)	σ_v (MPa)	σ_x (MPa)	σ_y (MPa)	τ_{xy} (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

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 σ_x , σ_y and τ_{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
 251 standard relations [19]:

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

$$253 \quad \sigma_y = \frac{\sigma_v + \sigma_h}{2} + \frac{\sigma_v - \sigma_h}{2} \cos(2\theta) \quad (1)$$

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

255 These equations, that are valid for a general biaxial state of stress (being σ_v and σ_h the

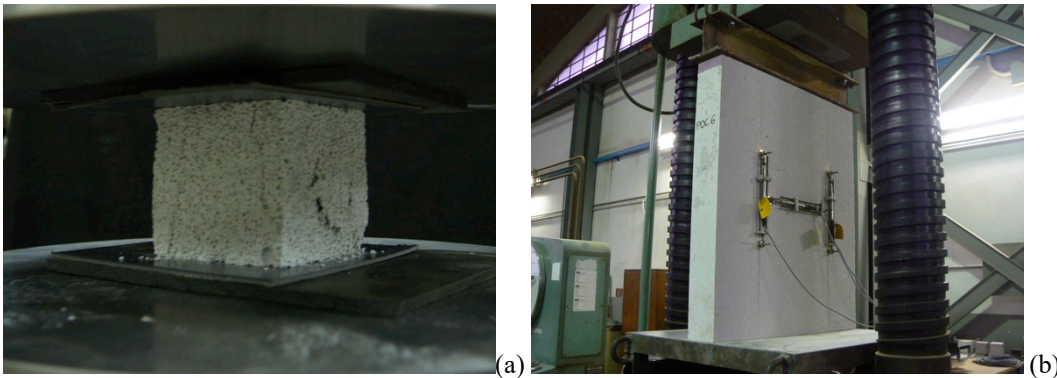
256 vertical and horizontal principal stresses, respectively, and θ the angle between bed
257 joints and the horizontal axis), have been here applied to the uniaxial case by simply
258 posing $\sigma_h = 0$. Table 1 also indicates the effective specimen dimensions (L, t, H), as
259 well as the adopted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

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

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

271 Moreover, AAC masonry compressive strength in the two principal directions x - y
272 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].

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



286 (a) (b)
287 **Figure 5.** Uniaxial compression tests on homogeneous AAC samples: (a) cubes with an edge length of 40
288 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

309 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 $\nu \cong 0.30$ 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
 311 were respectively equal, on average, to $\varepsilon_{px} = 1.8\text{‰}$ and $\varepsilon_{py} = 2.2\text{‰}$.

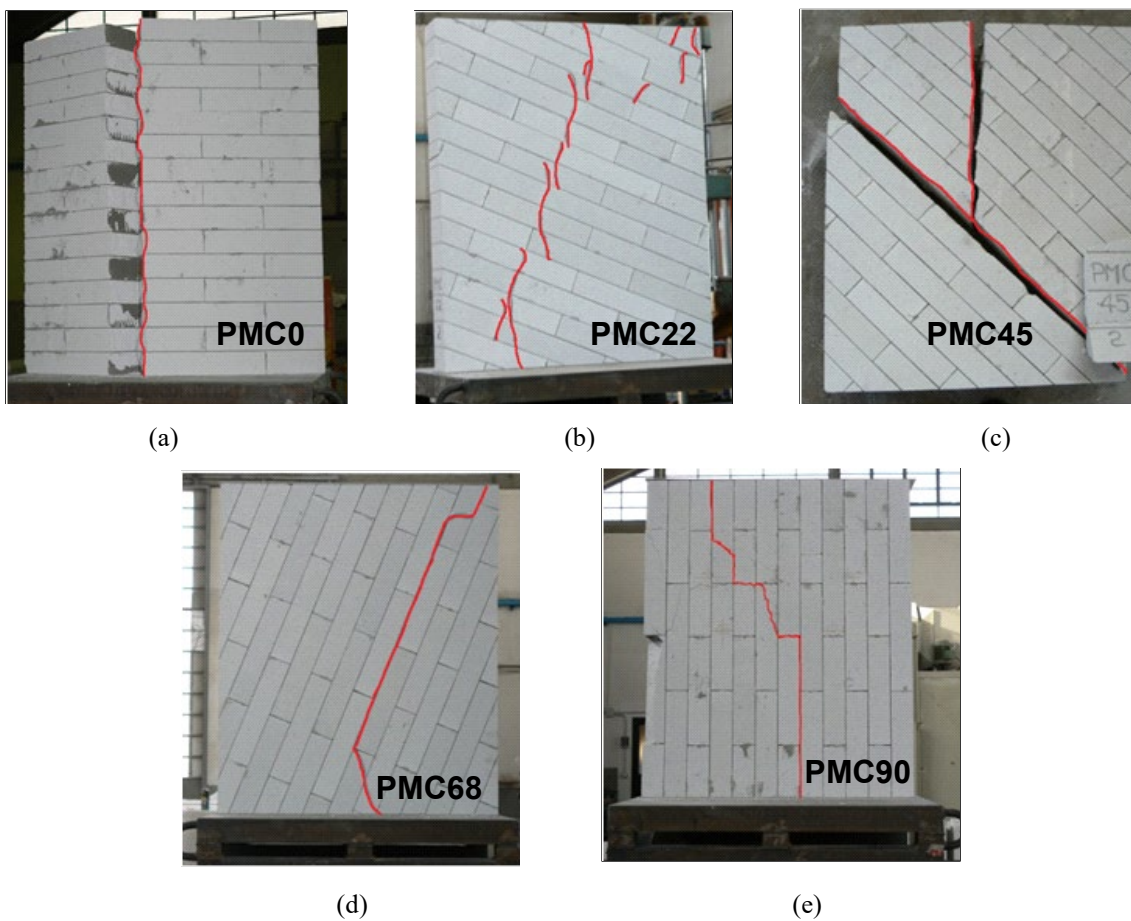
Sample	#	Test control	E_x (MPa)	E_y (MPa)	ν (MPa)	ε_{px} (‰)	ε_{py} (‰)
PMC0	2	LC		1417	0.32		2.00
PMC0	3	LC		1392	0.38		2.20
PMC0	7	DC		1346	0.29		2.40
PMCs0	1	DC		1737	0.35		1.68
PMC90	1	LC	1653		0.29	1.90	
PMC90	2	DC	1713		0.26	1.50	
PMCs90	1	DC	1719		0.25	2.10	

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 ν and compressive peak strains ε_{px} and ε_{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

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

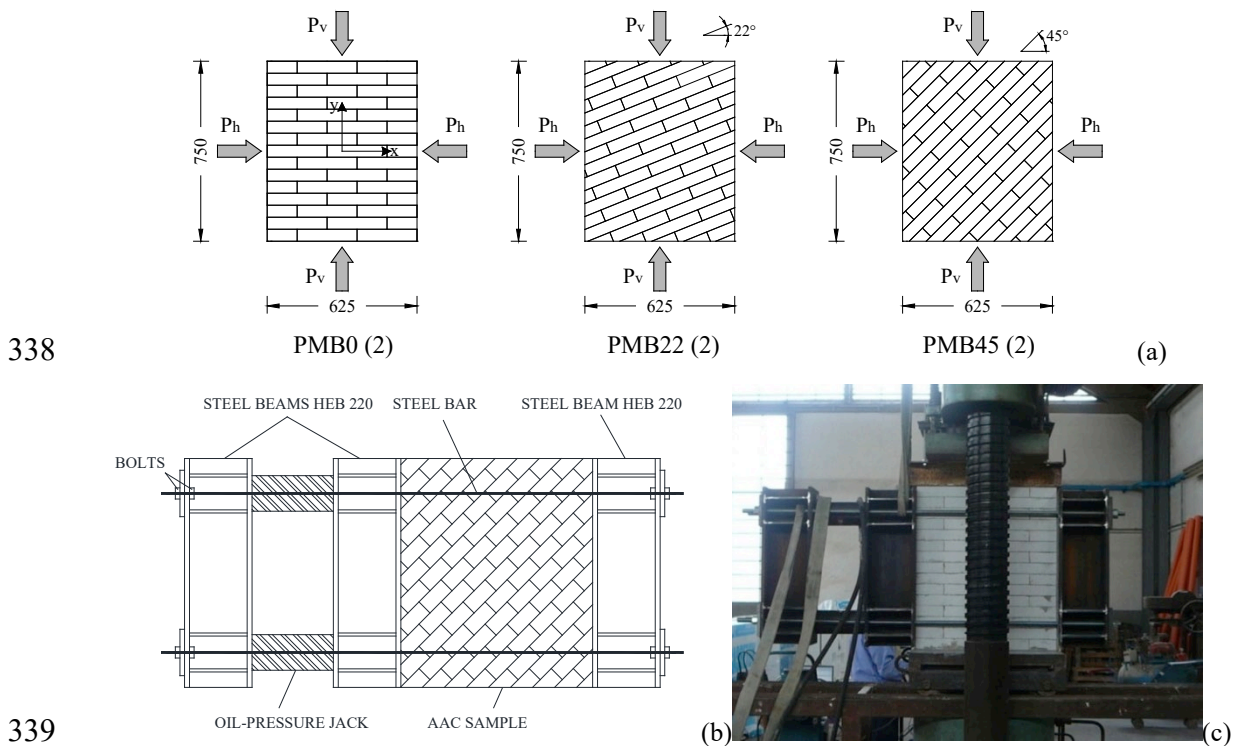


325
326 **Figure 6.** Uniaxial compression tests on AAC masonry panels (PMC): observed crack patterns at failure
327 as function of bed joint inclination.

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° , 22° , and 45°) 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° .



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 σ_v and σ_h .
 356 From these values, the total stress state related to bed joints σ_x , σ_y and τ_{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).

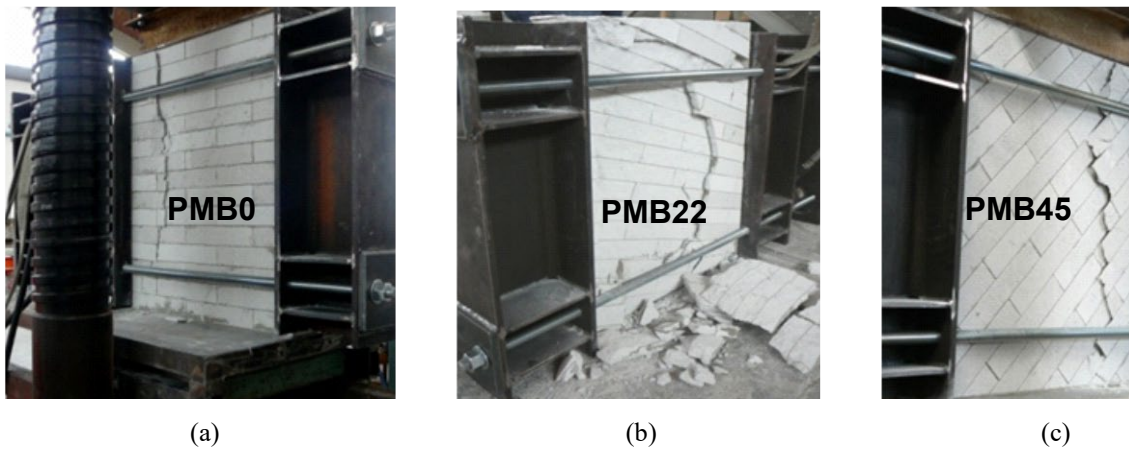
Sample #	Test control	L (mm)	t (mm)	H (mm)	$P_{v,u}$ (kN)	σ_v (MPa)	$P_{h,u}$ (kN)	σ_h (MPa)	σ_x (MPa)	σ_y (MPa)	τ_{xy} (MPa)
PMB0 1	LC	624	100	761	138.09	-2.21	165.52	-2.18	-2.18	-2.21	0.00
PMB0 2	LC	623	100	760	143.10	-2.30	171.61	-2.26	-2.26	-2.30	0.00
PMB22 1	LC	626	100	749	137.66	-2.20	159.54	-2.13	-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 LC = loading control

363 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.

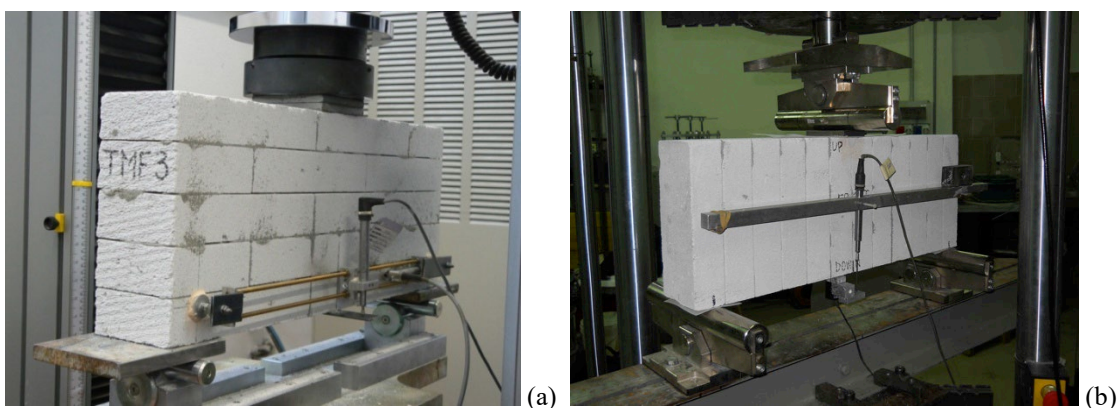


370

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

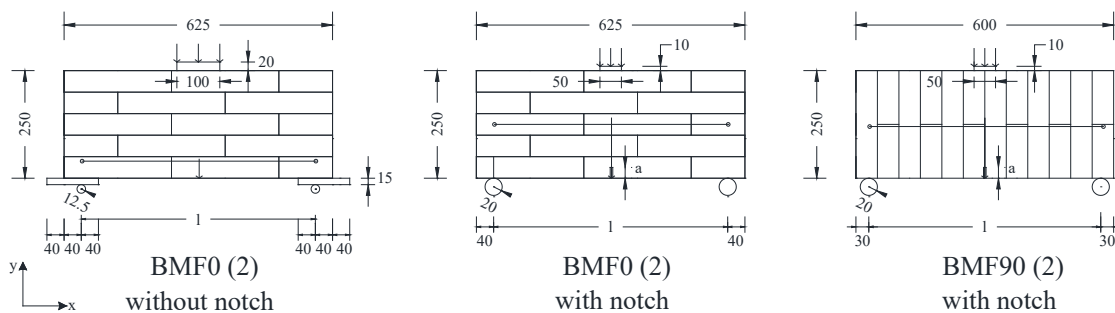
374 Three-point bending tests were performed on six AAC small-scale masonry
375 beams, with nominal dimensions equal to 625 x 250 x 100 mm (Figure 9).



376

377 **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
 395 **Figure 10.** Sketch of three-point bending tests on AAC masonry beams BMF: (a) unnotched specimens
 396 BMF0; specimens (b) BMF0 and (c) BMF90 with a central notch.

397 The two unnotched specimens were tested at the Laboratory of the AAC
 398 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 $\mu\text{m}/\text{min}$ (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 $P_{u,\beta}$ for the two
 406 investigated bed joint angles, as reported in Table 4.

Sample #	Test control	L (mm)	t (mm)	H (mm)	a (mm)	l (mm)	$P_{u,\beta}$ (kN)	f'_{tx} (MPa)	G_{fx} (N/mm)	f'_{ty} (MPa)	G_{fy} (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.

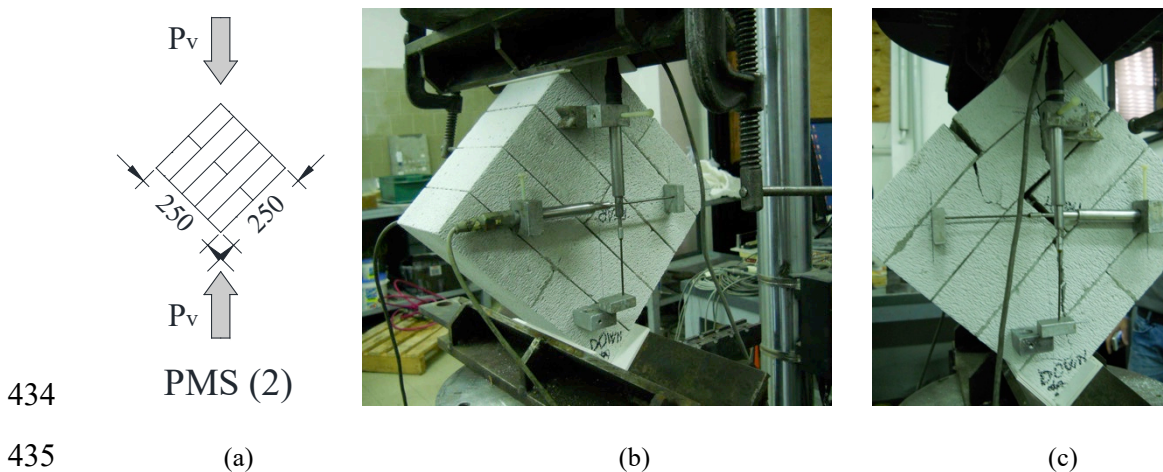
411 The corresponding flexural tensile strengths (moduli of rupture) in the two
 412 examined directions were subsequently determined through a linear elastic FE inverse
 413 analysis, so obtaining an average value respectively equal to $f'_{tx} = 0.37$ MPa and
 414 $f'_{ty} = 0.30$ MPa. Moreover, tests 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

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

422 It can be also observed that both the strength and the fracture energy of masonry
423 samples in the two examined directions are comparable to those of the homogeneous
424 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 11 a-b. Those
428 samples, named PMS (Panel Masonry Shear), were characterized by nominal
429 dimensions equal to 250 x 250 x 100 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_v . 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.



436 **Figure 11.** (a) Sketch and (b) adopted arrangement of diagonal compression tests on small AAC masonry
437 panels PMS; (c) observed failure mode.

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

439 vertical and 2 horizontal LVDTs, so as to record panel strains with increasing loads.

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

Sample #	Test control	L (mm)	t (mm)	H (mm)	P_v (kN)	f_{v0} (MPa)	σ_x (MPa)	σ_y (MPa)	τ_{xy} (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

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

452 The same Table also reports the total stress state related to bed joints σ_x , σ_y and
 453 τ_{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

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 $\bar{\sigma}_{ci} - \kappa_c$ (where the subscript i refers
 478 to the material axis x or y) is described through a parabolic plastic stress-strain
 479 relationship ($\bar{\sigma}_{ai} - \kappa_c$, according to Fig. 12b) until the reaching of the peak value,
 480 which is usually different in the two principal orthotropic directions x - y - namely
 481 $\bar{\sigma}_{px} = f_{mx}$ and $\bar{\sigma}_{py} = f_{my}$. After the peak, a parabolic/exponential softening branch (
 482 $\bar{\sigma}_{bi} - \kappa_c$ and $\bar{\sigma}_{di} - \kappa_c$, respectively) is adopted in both directions, characterized by
 483 different fracture energies G_{fex} and G_{fey} . The inelastic work $g_{fci} = G_{fci}/h$ in Figure 12 is

484 related to the fracture energy G_{fci} through the equivalent length h , which corresponds to
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 $\bar{\sigma}_{pi} = f_{mi}$ through the
489 following relations: $\bar{\sigma}_{ii} = 1/3 f_{mi}$, $\bar{\sigma}_{mi} = 1/2 f_{mi}$, $\bar{\sigma}_{ri} = 1/10 f_{mi}$. The equivalent plastic
490 strain κ_p , corresponding to the peak compressive strength, is assumed to be an
491 additional material parameter. Furthermore, in order to obtain a mesh independent
492 energy dissipation, it should be posed [32]:

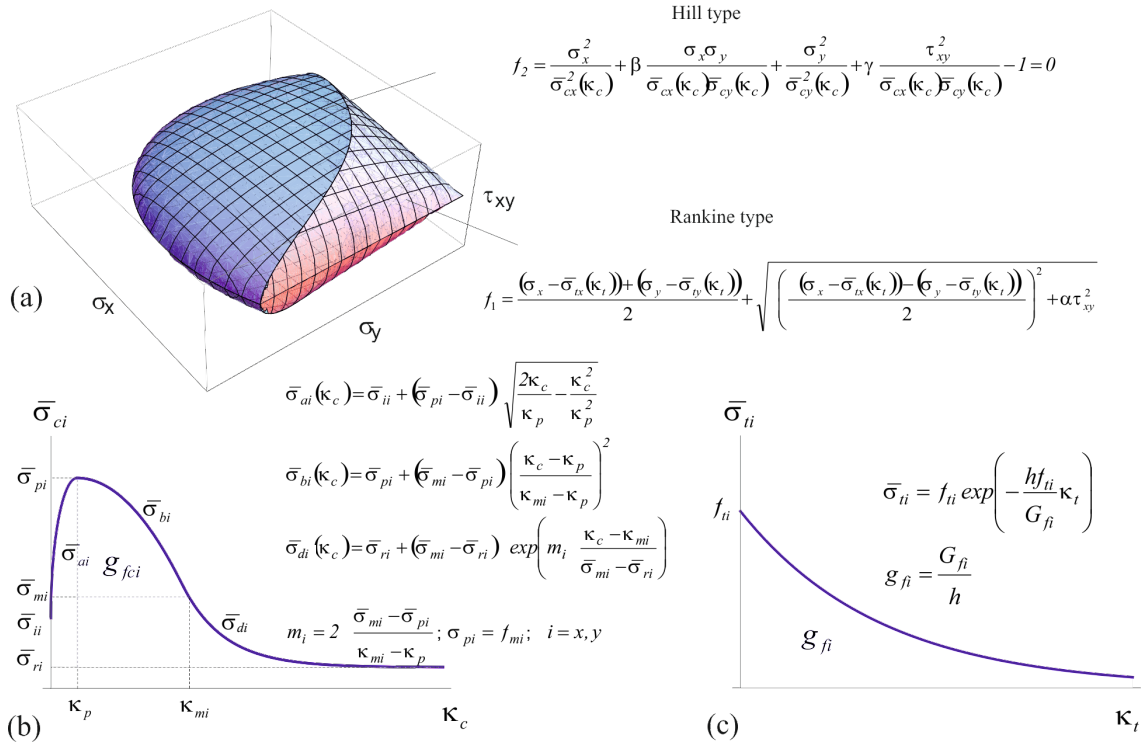
$$493 \quad \kappa_{mi} = \frac{75}{67} \frac{G_{fci}}{h f_{mi}} + \kappa_p, \quad (2)$$

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 $\bar{\sigma}_{ii} - \kappa_i$, with different tensile
497 strengths (f_{tx} and f_{ty}) and fracture energies (G_{fx} and G_{fy}) in the two principal orthotropic
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 γ), as well as five inelastic parameters governing the plastic stress-strain relationships
503 ($G_{fx}, G_{fy}, G_{fex}, G_{fey}$ and κ_p). The first four strength properties (which represent uniaxial
504 tensile f_{ti} 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



509
 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, κ_p . The complete calibration of the
 516 model also requires additional non-standard tests [34] in order to determine the three
 517 remaining parameters α , β and γ (Figure 12a); in more detail, α weights the shear stress
 518 contribution to tensile failure, β controls the coupling between normal stress values in

519 case of compressive failure and γ weights the shear stress contribution to compressive
520 failure.

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

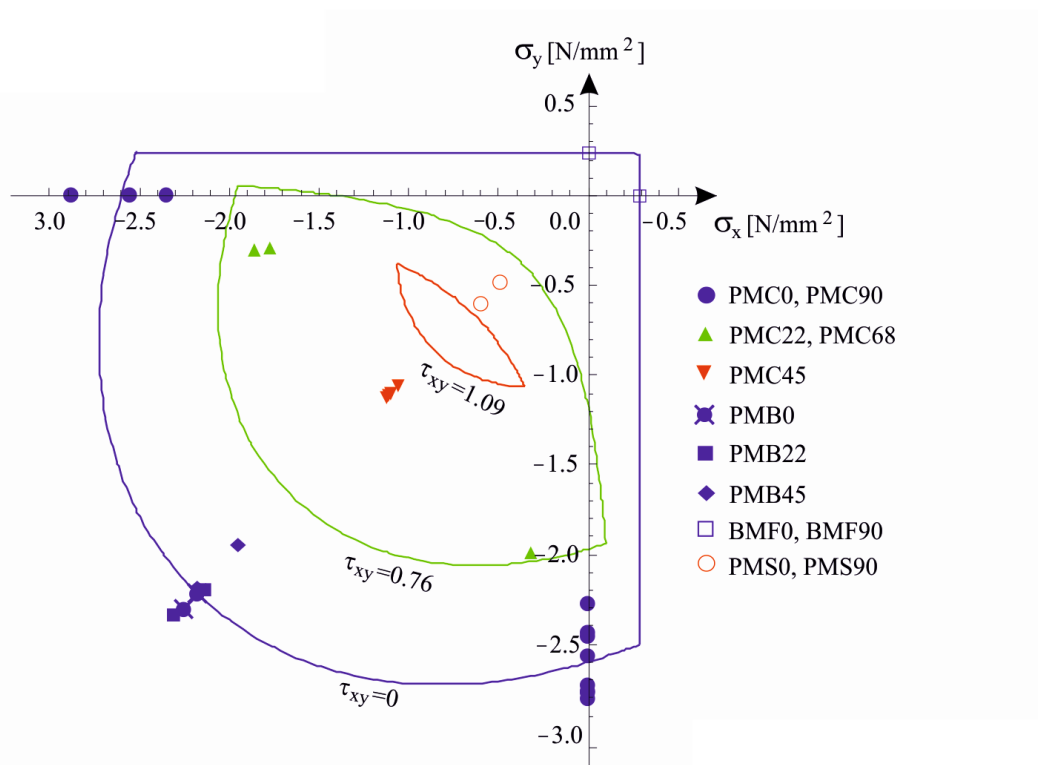
522 **3.2.1 Biaxial failure envelope**

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 σ_v
527 for PMC90 and PMC0 samples (see Table 1) - have been assumed equal to
528 $f_{mx} = f_{my} = 2.60$ MPa. The direct tensile strengths, respectively equal to $f_{tx} = 0.29$ MPa and
529 $f_{ty} = 0.24$ MPa, have been instead determined numerically, by simulating the three-point
530 bending tests carried out on samples BMF0 and BMF90 (Table 4), as better described in
531 Section 3.2.2.

532 The parameter α has been calculated through the least squares method, by
533 minimizing the function $\sum f_{1,j}^2(\alpha)$ with the non-linear optimization algorithm of
534 Levenberg-Marquard, being $f_{1,j}$ the plastic potential in the j -th experimental point (Figure
535 12a); in this way, the value of $\alpha = 0.5$ has been obtained. Similarly, the parameters β and
536 γ have been calculated by minimizing the function $\sum f_{2,j}^2(\beta, \gamma)$, so obtaining $\beta = -0.6$
537 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 τ_{xy} ,
540 respectively equal to 0, 0.76, and 1.09 MPa. On the same graph, the corresponding

541 experimental values obtained from uniaxial and biaxial compression tests (Section 2.2),
 542 as well as from three-point bending (Section 2.3) and diagonal compression ones (Section
 543 2.4), are also reported; as can be seen, the model error appears to be comparable with the
 544 experimental scattering. Furthermore, the obtained curves show that the behavior of AAC
 545 masonry is characterized by a weak anisotropy, with similar strength values in the two
 546 main directions x, y .

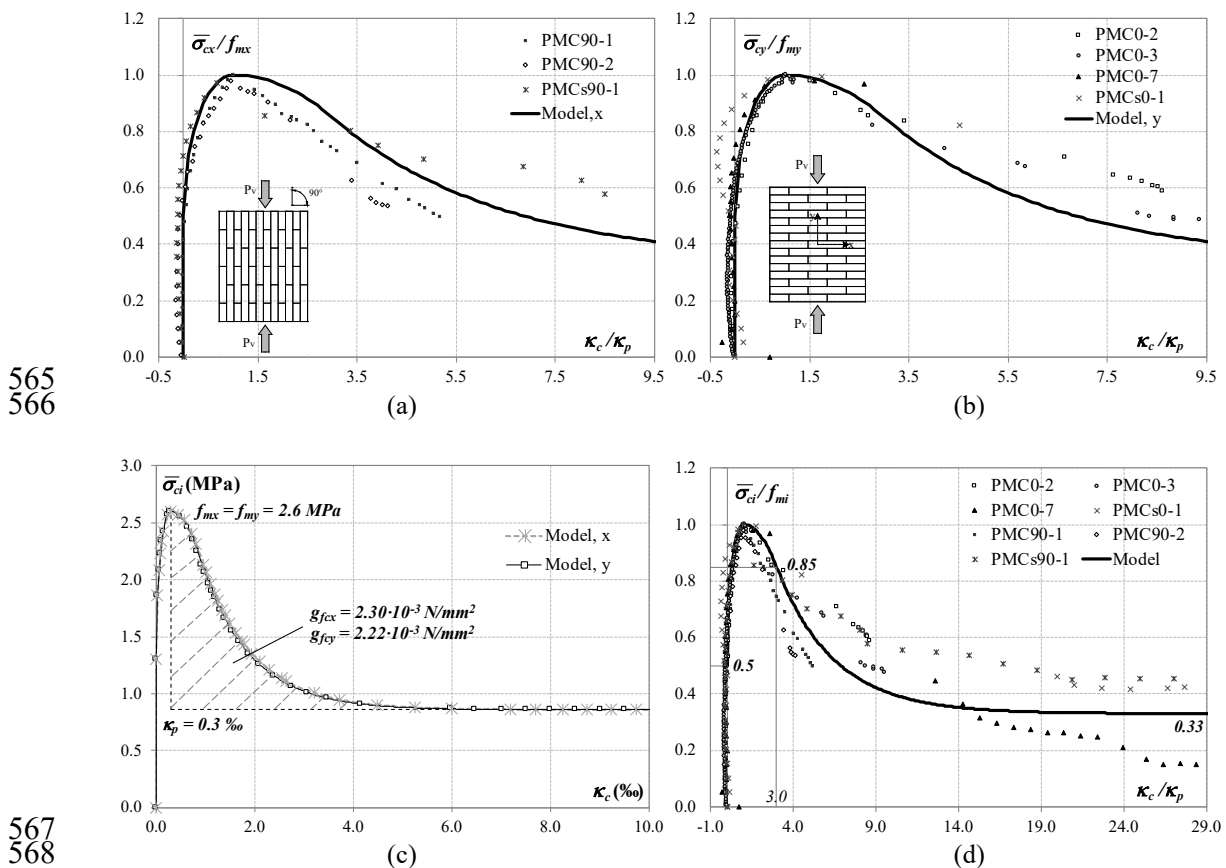


547
 548 **Figure 13.** Comparison between the adopted failure envelope [32-34] and the experimental results
 549 obtained for AAC masonry for different values of the applied shear stress τ_{xy} .

550 3.2.2 Stress-strain laws in uniaxial compression and tension

551 Based on the experimental results, the plastic stress-strain laws for uniaxial
 552 compression in the two main directions – whose equations are reported for reading
 553 convenience in Figure 12b – have been calibrated by adopting the average values of
 554 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
 560 directions (Figure 14c), also in this case a unique value of $g_{fcx} = g_{fcy} = 2.26 \cdot 10^{-3}$ N/mm²
 561 has been adopted. This value corresponds only to the local contribution of the $\bar{\sigma}_{ci} - \kappa_c$
 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
 570 available experimental data in (a) x and (b) y directions; (c) main parameters of the adopted laws in x

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

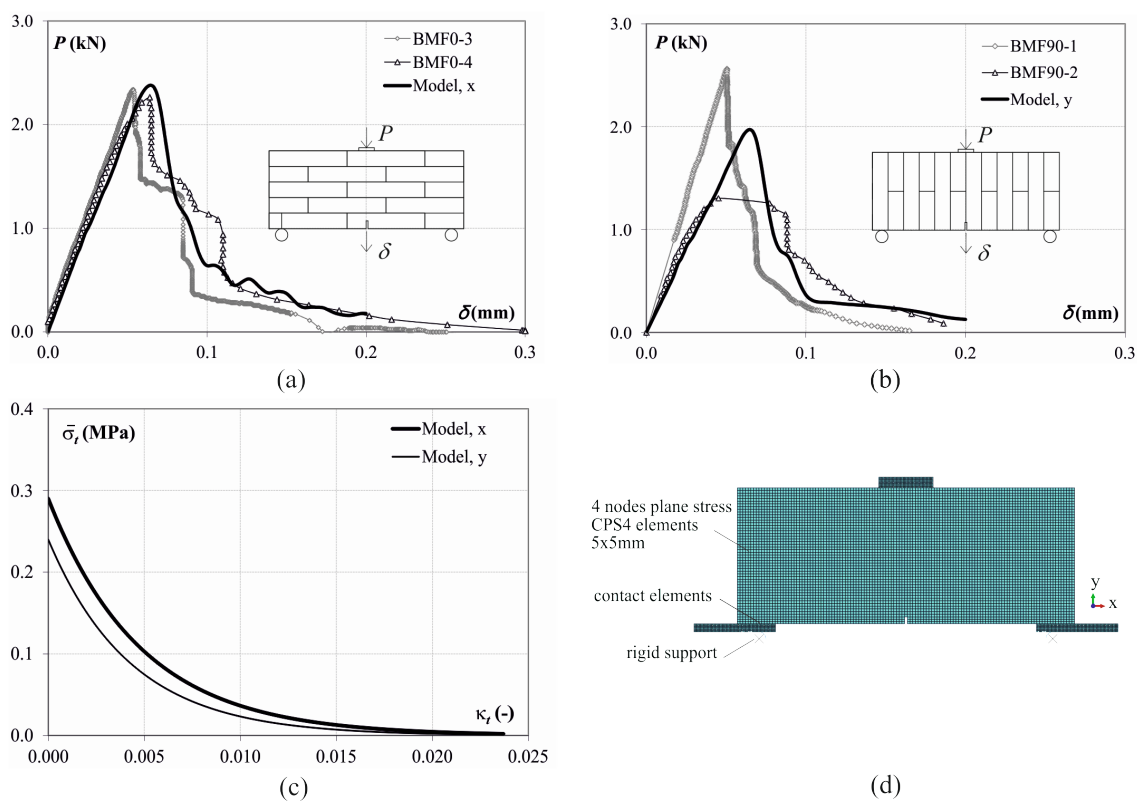
572 The complete inelastic law adopted for compression in both the considered
573 directions, respectively parallel and perpendicular to bed joints, is reported in Figure 14d
574 in the dimensionless plane $\bar{\sigma}_{ci} / f_{mi} - \kappa_c / \kappa_p$, where it is compared with the available
575 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
577 also reported, that is to say $\bar{\sigma}_{ii} / f_{mi} = 0.5$, $\bar{\sigma}_{mi} / f_{mi} = 0.85$, $\bar{\sigma}_{ri} / f_{mi} = 0.33$, with reference
578 to the nomenclature adopted in Figure 12b and according to [32]. Moreover, with the
579 previous assumptions, parameter κ_{mi} becomes:

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

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
585 $G_{fx} = 7.10 \cdot 10^{-3}$ N/mm and $G_{fy} = 5.15 \cdot 10^{-3}$ N/mm, see Table 4). Direct tensile strengths
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
592 scaling the fracture energies through the equivalent length $h = \sqrt{A_e}$, where A_e is the

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.



600
 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 δ 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.

604 Figure 15a-b shows the comparison between numerical and experimental results
 605 in terms of applied load P vs. midspan deflection δ , in case of BMF0 (Figure 15a) and
 606 BMF90 samples (Figure 15b). As can be observed, in the first case (x direction)

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.

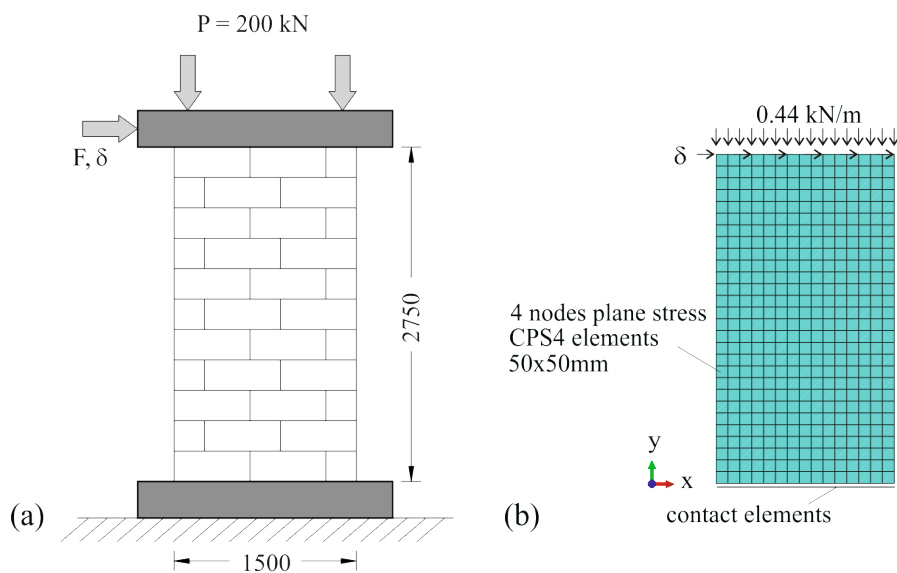
616 Finally, Figure 15c reports the corresponding exponential softening plastic stress-
617 strain relations $\bar{\sigma}_{ii}-\kappa_i$ in x and y directions.

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

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].

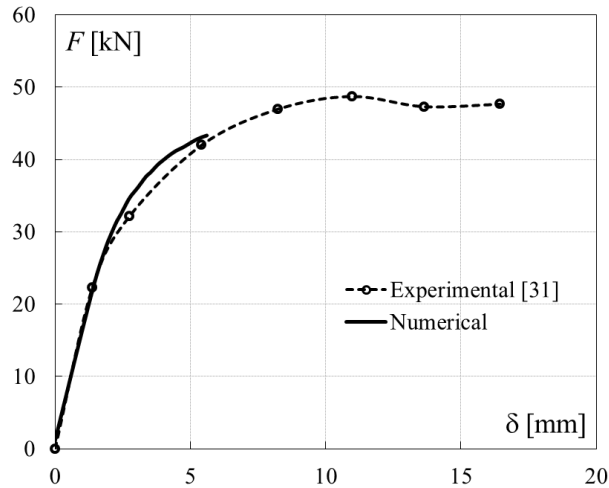


638
 639 **Figure 16.** Tests on a full scale AAC masonry wall: (a) sketch of the experimental setup [31]; (b) adopted
 640 FE mesh.

641 The considered wall has been modeled by adopting a uniform mesh, formed by
 642 square 4-node plane stress elements (CPS4) with 50 mm side, as shown in Figure 16b.
 643 For sake of simplicity, the two reinforced concrete beams have not been included in the
 644 FE model, by simply introducing contact elements at the base and considering friction
 645 between the AAC wall and the support. The horizontal force has been replaced by a
 646 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,
653 $E_x = E_y = 1498$ MPa. On the contrary, for the other required elastic and inelastic
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 δ 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.



670

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

672 horizontal displacement δ .

673

Anyhow, it should be underlined that the numerical response appears to be

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significantly less ductile than the experimental one. The underestimated ductility is due

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to important convergence problems that have been reported also in other works that

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adopt the same algorithm [43, 44]. In particular, van der Meer [44] analyzed the reasons

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of the convergence weakness of the subroutine (apex, return-mapping, multi-surface

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plasticity algorithm) and proposed some promising improvements that are, however, out

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of the scope of this work.

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681

5. CONCLUSIONS

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The present work illustrates the main results of an experimental program focused

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on the mechanical characterization of AAC masonry. To this aim, the following test

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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
691 technical literature [32-34]. This model has been subsequently applied to simulate
692 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,
700 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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709

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820

821 **LIST OF FIGURES**

822 **Figure 1.** General view of some of the AAC masonry specimens tested during the experimental program.

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

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

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

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

831 **Figure 6.** Uniaxial compression tests on AAC masonry panels (PMC): observed crack patterns at failure
832 as function of bed joint inclination.

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

835 **Figure 8.** Biaxial compression tests on AAC masonry panels (PMB): observed crack patterns at failure as
836 function of bed joint inclination.

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

839 **Figure 10.** Sketch of three-point bending tests on AAC masonry beams BMF: (a) unnotched specimens
840 BMF0; specimens (b) BMF0 and (c) BMF90 with a central notch.

841 **Figure 11.** (a) Sketch and (b) adopted arrangement of diagonal compression tests on small AAC masonry
842 panels PMS; (c) observed failure mode.

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

845 **Figure 13.** Comparison between the adopted failure envelope [32-34] and the experimental results

846 obtained for AAC masonry for different values of the applied shear stress τ_{xy} .

847 **Figure 14.** Comparisons between the adopted inelastic law in compression for AAC masonry and the
848 available experimental data in (a) x and (b) y directions; (c) main parameters of the adopted laws in x
849 and y; (d) definition of a unique inelastic law and comparison with all the experimental results.

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

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

855 **Figure 17.** Comparison between numerical and experimental results in terms of applied load F vs.
856 horizontal displacement δ .

857

858 LIST OF TABLES

859 **Table 1.** Uniaxial compression tests on AAC masonry panels (PMC and PMCs): effective dimensions of
860 the specimens and experimental failure loads.

861 **Table 2.** Uniaxial compression tests on AAC masonry panels (PMC and PMCs): elastic moduli E_x , E_y ,
862 Poisson ratio ν and compressive peak strains ϵ_{px} and ϵ_{py} .

863 **Table 3.** Biaxial compression tests on AAC masonry panels (PMB): effective dimensions of the specimens
864 and experimental failure loads.

865 **Table 4.** Three-point bending tests on AAC masonry beams (BMF): effective dimensions of the specimens,
866 experimental failure loads, indirect tensile strengths and fracture energies in tension.

867 **Table 5.** Diagonal compression tests on small AAC masonry panels (PMS): effective dimensions of the
868 specimens and experimental failure loads.

869