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Experimental analysis of timber inclusions effect on paraseismic behavior of earth masonry walls

Jairo Aranguren¹, Florent Vieux-Champagne², Maïa Duriez³ and Jean-Emmanuel Aubert⁴

Abstract. This research aims at the seismic assessment and understanding of a traditional loadbearing system incorporating horizontal timber elements into earth brick masonry walls. In order to characterize the global behavior of the loadbearing device, an experimental campaign was performed on the constituents (i.e. bricks and mortar) and on masonry samples. Compression tests were carried out on four geometries of earth brick samples and three geometries of earth mortar samples. As the methods to obtain the compressive strength of earth material are still discussed within the scientific community, a focus was made on different configurations tested. Uniaxial compression tests were performed on two earth brick wallets. Then, two brick walls (with and without horizontal timber reinforcement) were submitted to lateral quasi-static cyclic load. A LVDTs system and a stereo correlation image system were coupled to study the experimental response of the reinforced and unreinforced walls. This investigation led to estimate the lateral strength, the stiffness degradation and the dissipated energy of both reinforced and unreinforced masonry walls. Moreover, it allowed the comparison between the failure modes of the two types of wall. This analysis brings light on the mechanical impact of the horizontal reinforcement.

Keywords: Seismic behavior, traditional loadbearing system, earth brick masonry, timber insertions, stereo correlation image system, timber laced masonry.

1 Introduction

Traditional masonry structures represent an important part of the buildings around the world. This can be explained in particular by the constructive advantages of this system against other structure typologies such as reinforced concrete (RC) in terms of: construction costs, execution complexity and speed, materials accessibility and workmanship qualification requirements. Despite being commonly present in areas where the seismic risk is high (e.g. Nepal, Pakistan, Turkey, Italy, Haiti, etc.), traditional masonry structures are especially vulnerable when subjected to lateral loading such as seismic stresses. This was particularly clear after 2015's earthquake in Gorkha, Nepal, where thousands of non-engineered masonry dwellings suffered collapse and/or severe damage, affecting thousands of families and households. More details about the post-seismic construction in Nepal are available in [1].

1 Laboratoire Matériaux et Durabilité des Constructions (LMDC), Université de Toulouse, UPS, INSA, 135 avenue de Rangueil, 31077 Toulouse Cedex 4, France, e-mail : jairodavidbz@gmail.com

2 Univ. Grenoble Alpes, CNRS, Grenoble INP, 3SR, 38000 Grenoble, France, e-mail : florent.vieux-champagne@univ-grenoble-alpes.fr

3 Laboratoire Matériaux et Durabilité des Constructions (LMDC), Université de Toulouse, UPS, INSA, 135 avenue de Rangueil, 31077 Toulouse Cedex 4, France, e-mail : maia.duriez@insa-toulouse.fr

4 Laboratoire Matériaux et Durabilité des Constructions (LMDC), Université de Toulouse, UPS, INSA, 135 avenue de Rangueil, 31077 Toulouse Cedex 4, France, e-mail : aubert@insa-toulouse.fr

29 As remarked by different authors [2]-[3], the collapse of masonry structures is mainly associated with material
30 and construction deficiencies, as well as the lack of wall-to-wall and roof-to-wall connections and the
31 presence of heavy roofs. However, replacement of existing dwellings with new structures built according to
32 paraseismic standards used in developed countries is not appropriate because of the high cost and embodied
33 energy. On the contrary, the challenge today is either to build new structures based on the improvement of
34 vernacular constructions, or to be able to reinforce existing structures by using local resources according to the
35 local building culture.

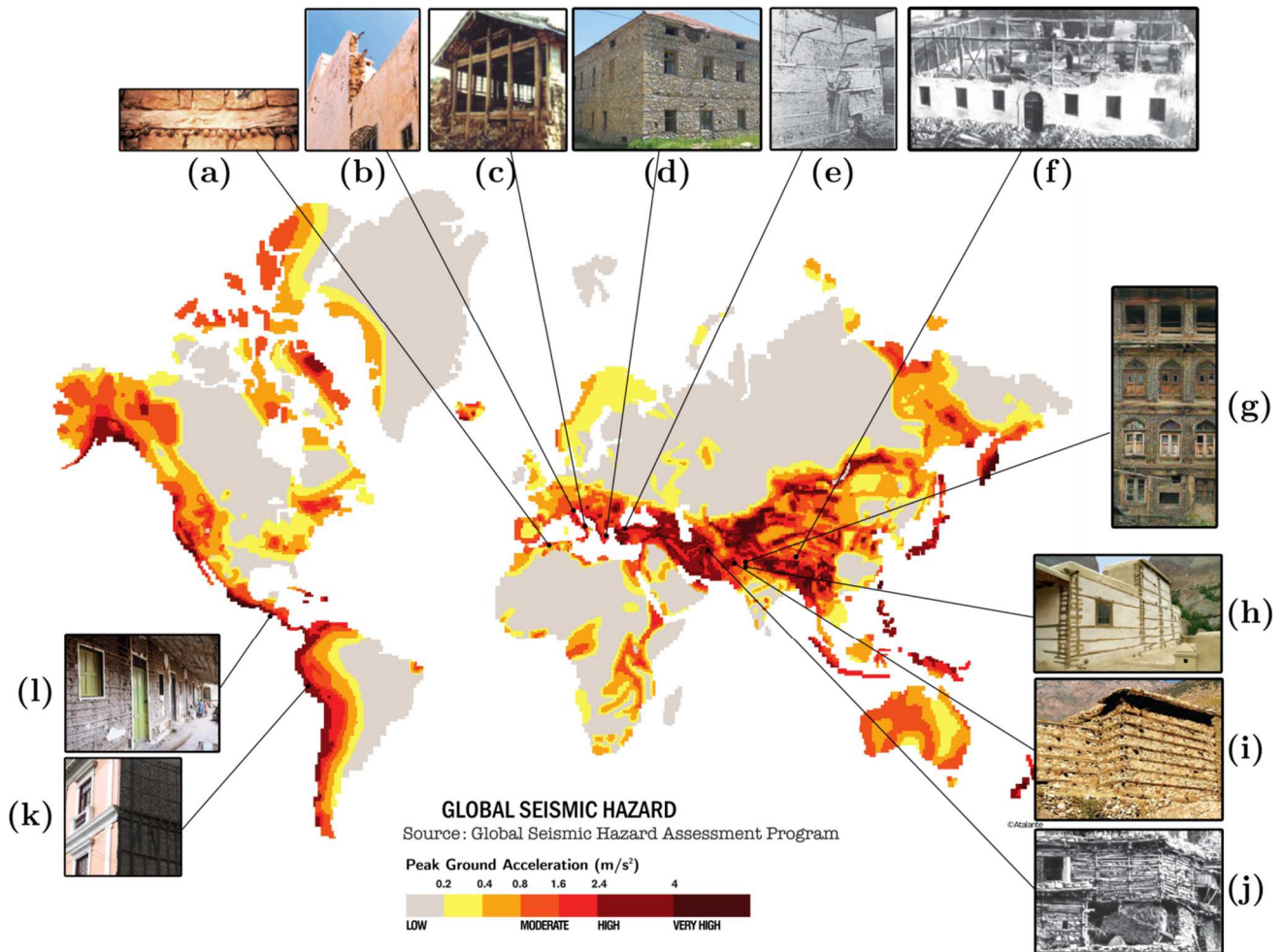
36 This article provides a structural analysis regarding the paraseismic behavior of the interesting system of
37 timber laced masonry since it is a traditional technique that seems to be specifically developed in seismic
38 prone areas such as Nepal, as well as in Greece, Turkey and India (see [3]-[6]). This system is characterized
39 by stone or earth masonry reinforced with horizontal timber beams in most cases (cf. **Fig. 1**).

40 By combining a rigid material (masonry) with a flexible material (timber) it is possible to modify the behavior
41 of masonry making it more flexible and more ductile. This composite system allows increasing the energy
42 dissipation capacity of walls subjected to lateral forces thanks to the combined effect of both the friction on
43 the joints, and on the timber and the masonry elements interface. The absorption of large amounts of energy is
44 also favored as inclusions interrupt structural homogeneity allowing relative displacements of the sub-divided
45 elements [3]. The inclusions also play a fundamental role in controlling the crack propagation, since they
46 allow holding masonry together when subjected to seismic forces, facilitating the formation of numerous
47 micro and macrocracks before rupture [5]. Moreover, it has been found that inclusions may increase the
48 masonry compressive strength as they act as confinement reinforcement for masonry [6].

49 Despite the popularity of masonry reinforced with timber, the lack of scientific knowledge regarding this
50 system is an important hindrance to its use and more widely to its sustainability. Indeed, very little literature is
51 available [6] despite the complexity of the behavior of this kind of structure. Moreover, the great variability of
52 masonry structures makes elaboration of a representative constitutive law difficult. Therefore, research
53 considers the study of structures with very specific materials and constructive arrangements for which
54 different theoretical, experimental and numerical approaches are proposed.

55 This research follows two main objectives; first, to increase the scientific knowledge on seismic behavior of
56 traditional adobe masonry structures and secondly, to assess the influence of a specific masonry loadbearing
57 system adapted to traditional construction techniques and vernacular materials, on the masonry's seismic
58 behavior.

59 The purpose of this research is to study the influence of the timber inclusion in a part of masonry. The
60 influence of specific vernacular earthquake resistant technologies at the scale of masonry structures (confining
61 effect, tie effect, out-of-plane behavior and geometrical aspects) have been detailed in other studies (cf. [6]-
62 [7], [11]-[13]).



63

64 **Fig. 1** Timber frame structures in the world (Adapted from [7]) : (a-b) Media Corpus (seen in [8]), (c) Tsakanika-Theohari (seen in
65 [8]), (d) Hoffman (seen in [8]) (e) Aytun (seen in [8]) (f) Liu [9], (g) Langenbach [5], (h) Hoffman (seen in [8]), (i) Schacher
66 (seen in [8]), (j) Wutt (seen in [8]), (k) Langenbach [10], (l) T. Joffroy (seen in [7])

67

68 The studied structure is made of earth masonry walls reinforced with horizontal ladder shaped timber ties. An
69 experimental campaign was performed from the scale of the masonry elements (bricks and mortar) to the scale
70 of a shear wall of about $1.3m \times 1.3m$. On the first scale, a focus is made on different compression tests since
71 there is still an important discussion on earth material as no consensus exists regarding the procedure to
72 determine the mechanical characteristics of earth bricks. On a second scale, the masonry compression
73 behavior is studied through compression tests carried on masonry wallets. On the scale of the wall, results of
74 lateral quasi-static load tests on two adobe walls (with and without timber bands) and the efficiency of
75 reinforcement are discussed.

76 In order to characterize the traditional Nepalese masonry system the experimental program includes; bending
77 and uniaxial compression tests on the constituents (bricks and mortar), compression tests on masonry wallets
78 and quasi-static lateral loading tests on masonry walls. The current paper presents the materials, the test
79 procedures and the experimental results for each test achieved on masonry elements (bricks and mortar), scale
80 wallets and scale walls.

81 The dimensions of the materials and elements adopted within the framework of this project were chosen to be
82 representative of the *Design catalogue for reconstruction of earthquake resistant houses* in Nepal [14]. This
83 document, prepared by the government of Nepal as a response to the earthquake on 15th April 2015, aims at
84 presenting the minimal specifications necessary to reconstruct masonry dwellings using traditional
85 paraseismic techniques, which guarantee the vernacular architecture conservation. Variability of materials
86 used to fabricate earth bricks and timber elements of vernacular buildings is significant since it is common
87 practice to dispose of materials locally available. However, all earthen materials show similar mechanical
88 characteristics because of their capillarity properties. In this respect, earth and wood used for this project were
89 local products. It is worth to highlight that the indications provided in the catalogue were completed with
90 other documents dealing with construction practices in Nepal (see [8], [15]).

91 **2 Constituents of masonry**

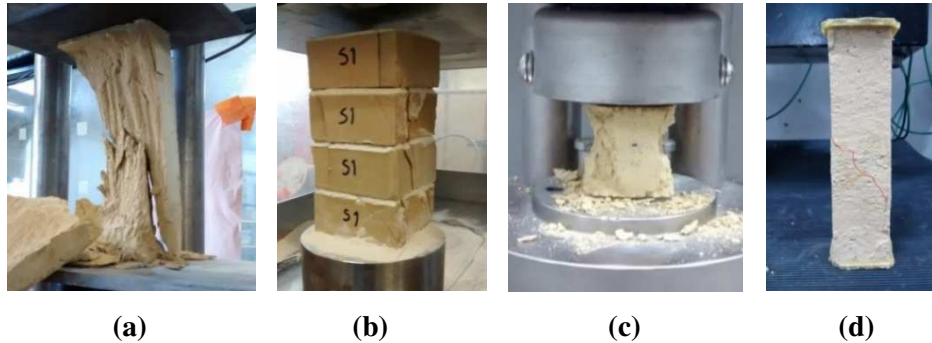
92 Masonry is a complex composite material. In order to properly understand its behavior, it is necessary to
93 characterize the main mechanical properties of its constituents (bricks and mortar). This paragraph focuses on
94 the determination of compressive and tensile strengths of both mortar and units. These properties will be
95 useful to understand the behavior of the masonry at the scale of the wallet (cf. part 3).

96 **2.1 Materials and experimental setup**

97 Bricks and mortar used for the experiments were supplied by a local manufacturer of earthen products. Earth
98 bricks were manufactured by extrusion (without compression) and by air drying with a size of $50 \times 105 \times$
99 220 mm^3 . Mortar was composed of a mixture of earth laminated at $0.7 / 0.1 \text{ mm}$ and white sand $0 / 4 \text{ mm}$.
100 Earth was composed of quartz, calcite, goethite, feldspars, illite and montmorillonite [16]. The binder was
101 prepared from a mixture of water and earth in proportions of approximately 1:6. Timber used as
102 reinforcement is made of raw unplanted fir tree, untreated, class C18 according to the standard NF EN 338
103 [17].

104 Since the compressive characterization of earth bricks is still under discussion (see [18]-[19]), four protocols
105 were implemented to determine the compressive strength of masonry units. Two tests were performed on
106 entire bricks according to standard NF EN 772-1 [20]; for the first panel, entire bricks were tested in
107 horizontal position (load applied on bed joints) (cf. **Fig. 2(a)**); for the second panel, entire bricks were tested
108 in vertical position (load applied on head joints). Additionally, a third type of tests was performed on stacked
109 half-bricks considering the prescriptions of the experimental standard XP P13-901 (2001) [21]. However,
110 unlike standard, a fine layer of sand was laid out between the half-bricks, and on the top and bottom surface of
111 each specimen to ensure a uniform load distribution. A fourth protocol was used on samples formed by
112 superimposing four half-bricks, in order to guarantee an aspect ratio of 2 (cf. **Fig. 2(b)**). The tensile strength
113 was determined indirectly through 3-point bending tests on entire bricks. The Young's modulus and Poisson's
114 ratio were obtained through a series of uniaxial compression tests performed on entire bricks vertically

115 positioned. Loads were applied according to prescriptions in NF EN 12390-13 [22]. Additionally,
 116 compression tests on three types of earth mortar samples with different aspect ratios were carried out. For the
 117 first panel (aspect ratio of 1), mortar specimens with a size of $40 \times 40 \times 160 \text{ mm}^3$ were tested according to
 118 EN 1015-11 [23] (cf. **Fig. 2(c)**). For the second panel (aspect ratio of 1.7) samples with initial dimensions of
 119 $40 \times 40 \times 160 \text{ mm}^3$ were cut in two halves. Then, the cut specimens were tested along the largest direction.
 120 The last panel (aspect ratio of 4) consisted of specimens of size $40 \times 40 \times 160 \text{ mm}^3$ tested along the largest
 121 dimension (cf. **Fig. 2(d)**). Furthermore, mortar tensile strength was obtained through 3-point bending tests
 122 according to EN 1015-11.



123
 124

125 **Fig. 2** Compression tests on: (a) vertical entire bricks, (b) 4 stacked half-bricks, (c) mortar aspect ratio 1, (d) mortar aspect ratio 4.

126 2.2 Results

127 Tests results are shown in **Table 1**. For each configuration, mean values and standard deviation SD (%) are
 128 presented. Before each test, samples were measured and weighted, which allows to calculate the brick bulk
 129 density $\rho = 2098 \text{ kg/m}^3$ ($SD = 2.4 \%$) and the mortar bulk density $\rho = 1918 \text{ kg/m}^3$ ($SD = 0.6 \%$).

130 Observations carried out after compression tests on stacked half-bricks revealed the presence of an unconfined
 131 zone in the sand layers, where the load was only partially transferred between bricks. Therefore, the surface
 132 used to determine brick compression strength of stacked half-bricks is reduced by 10 % on each side to obtain
 133 an equivalent confined area. This reduction percentage was determined empirically.

134 The higher compressive strength corresponds to entire bricks tested horizontally. This is essentially due to the
 135 specimen geometrical conditions. As exposed by different authors [18]-[19], [26]-[28], compressive strength
 136 increases with decreasing aspect ratio because of the confinement produced by the friction between the
 137 surfaces of the samples and the press plates. This friction can induce inconsistent results as shown by [29].
 138 The compressive strength obtained from tests on 2 and 4 stacked half-bricks is quite similar. However, these
 139 values are substantially lower than the strength found in tests on entire bricks. This difference can probably be
 140 explained by the fact that the sand has a higher Poisson's ratio, which caused high lateral strain under
 141 compression inducing some tensile stress on the bricks. This fact could be at the origin of the failure mode
 142 observed on stacked specimens, which is characterized by vertical cracks passing through bricks and vertical
 143 decohesion. The ones tested vertically (cf. **Fig. 2 (a)**) present the typical compression failure mode (two
 144 opposite confinement cones) and a compressive strength between that of bricks tested horizontally and

145 stacked samples. The variability of compressive strength is significantly higher when testing 2 stacked half-
 146 bricks ($SD = 25.3\%$). Conversely, tests on entire vertical bricks are considerably more consistent as they
 147 report the lowest standard deviation (10.9%). Furthermore, taking the results of the tests on entire vertical
 148 bricks as a reference, it appears that the mean value of the bricks tensile strength (measured in flexion)
 149 corresponds to 17% of the compressive strength.

150 **Table 1.** Mechanical parameters of the constituents

Material	Mechanical parameter	Number of tests	Mean	SD (%)	
Brick	Compressive strength (MPa)	Entire block horizontal	6	10.9	14.4
		Entire block vertical	6	7.7	10.9
		2 stacked half-bricks	7	4.9	25.3
		4 stacked half-bricks	6	4.8	13.6
	Tensile strength (measured in flexion) (MPa)	6	1.3	16.1	
	Young's modulus (MPa)	3	4362	9.3	
	Poisson's ratio (-)	2	0.12	-	
Mortar	Compressive strength (MPa)	Aspect ratio 1	5	2.2	3.9
		Aspect ratio 1.7	5	1.9	6.4
		Aspect ratio 4	6	1.8	3.3
	Tensile strength (measured in flexion) (MPa)	5	0.9	9.5	

151
 152 Results in **Table 1** show that the mortar compressive strength is lower for higher aspect ratios. However,
 153 specimens with an aspect ratio of 1.7 and 4 exhibit similar mechanical strengths due to the fact that both
 154 samples present shear failure, as brought to light by the presence of a characteristic shear fracture surface,
 155 with sliding planes inclined 45° with respect to the vertical plane (cf. **Fig. 2(d)**). In fact, when calculating the
 156 shear strength analytically with the hypothesis of a shear fracture plane at 45° we obtain a shear strength of
 157 0.9 MPa , which is similar to tensile strength. On the contrary, the samples with an aspect ratio of 1 revealed a
 158 typical compressive failure mode (cf. **Fig. 2(c)**). The tensile strength, measured in flexion, is about 0.9 MPa .
 159 This value corresponds to 41% of the compressive strength found for the cubic sample.

160 One can suppose from **Table 1** that the behavior of the current masonry has a compression strength governed
 161 by the mortar behavior since this material has a low resistance compared to the bricks. However, this might
 162 not be the only determinant parameter as will be discussed on the masonry wallets study.

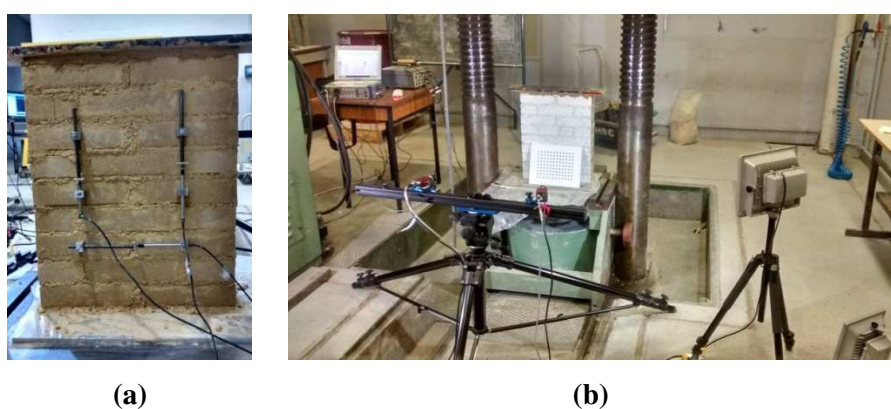
163 3 Masonry wallets

164 3.1 Materials and experimental setup

165 Two masonry wallets of size $525 \times 445 \times 105\text{ mm}^3$ (MC-1) and $570 \times 450 \times 105\text{ mm}^3$ (MC-2) were
 166 constructed to be tested under quasi-static uniaxial compression loading in order to analyze the behavior of the
 167 structure (rigidity, plasticity, strength, crack pattern) and to estimate the vertical load that should be applied on
 168 the elements of shear wall during the quasi-static test. The wallets were built on steel plates following a

169 "stretcher" bond (cf. **Fig. 3(a)**). After construction, both specimens were stored in a room under ambient
 170 temperature and humidity conditions (approximately 22°C and 40% RH).

171 The compressive strength and the Young's modulus of the masonry were determined according to EN 1052-1.
 172 To determine the Young's modulus, the samples were subjected to three charge-discharge cycles. For each
 173 cycle, the vertical load was gradually varied between two values corresponding to a minimum pre-load of
 174 15 kN and a load equal to 50 % of the expected masonry strength, this value was obtained from the literature
 175 [27]. Vertical and horizontal displacements were measured using Linear Variable Displacement Transformers
 176 (LVDTs) glued on one side of each wallet (cf. **Fig. 3(a)**). Once the charge-discharge cycles completed, each
 177 specimen was subjected to an increasing monotonic compression load until failure in order to determine the
 178 compressive strength.



179
 180
 181 **Fig. 3** Compression test on masonry. (a) Position of LVDT captors, (b) Stereoscopique system disposition.

182 A stereoscopic system was set up to visualize the compression test in order to obtain the displacement field of
 183 the MC-2 sample (cf. **Fig. 3(b)**). A system of two AVT Manta 504B cameras with 8 mm lenses was used to
 184 record the sample. Because of the uniform color of the earth bricks, a mouchetis was made on one of the faces
 185 of the wall to generate a very contrasting pattern to improve the results of the Digital Image Correlation (DIC)
 186 process. The observed face of the specimen was first painted white and then small black spots were applied
 187 using a spray gun. A LED continuous lighting system was installed to ensure a uniform light intensity on the
 188 sample. Images were taken every 10 seconds for a total of 256 images recorded during the cyclic test and 71
 189 images obtained during the monotonic test. In a post-processing phase the recorded images were analyzed
 190 using the VIC-3D software. Displacement and deformation fields of the wallet were obtained by comparing
 191 the images of its surface acquired before and after deformation. The value of the displacement of a point
 192 corresponds to the average value of the displacement of a subset (a portion of a reference image) centered at
 193 the considered point. To determine the displacement suffered by the subset after deformation, the subset in the
 194 reference image "moves" to find a pattern that fit the best in the distorted image. The evaluation of the
 195 correspondence between the two images (normal and deformed) is done by minimizing a function that
 196 represents the intensity of gray level of the pixels contained in the subsets [30]. In order to have a confidence
 197 interval of maximum 0.01 pixel (spatial resolution 0,28 mm²), the correlation criterion "normalized sum of

198 squared differences" was used for image processing. Then, different quantities such as displacements and
 199 deformations were evaluated at different points of the wall for different stages of the test.

200 3.2 Results

201 The results of the compression tests carried out on the masonry wallets are reported in **Table 2**. The vertical
 202 strain and stress variation during the loading protocol is illustrated in **Fig. 4 (a)**. When compared to the
 203 constituents ($E_{\text{brick}} = 4362 \text{ MPa}$), both specimens showed low values of Young's modulus and Poisson's
 204 ratio. This difference of rigidity between the constituents and masonry may be explained by the influence of
 205 the interface as masonry resistance is strongly related to the tensile bond strength between the joints and the
 206 units [31]. In turn, compressive strength of both specimens was approximately the strength of the mortar
 207 sample with an aspect ratio of 1 (2.2 MPa). However, it was substantially different from bricks' compressive
 208 strength.

209 Both wallets presented a failure mode characterized by vertical cracks crossing the bricks and the joints. In the
 210 MC-1 sample, cracks appeared on one flank of the wall where the bricks eventually detached from the
 211 structure (cf. **Fig. 4 (d)**). In MC-2, the main cracks occurred on both sides of the specimen. Besides, a
 212 longitudinal and transverse expansion at the top of the wall was noticed. Similar behavior of the masonry was
 213 observed by Miccoli et al. [27] after completing uniaxial compression tests on earth masonry wallets,
 214 obtaining compression strength values between 2.7 and 3.8 MPa and a Young's modulus of 803 MPa.

215 **Table 2.** Mechanical parameters of masonry according to EN 1052-1

Specimens	Dimensions (mm)	F_{max} (kN)	f (MPa)	$\epsilon_{yy}^{1/3}$ ($\mu\text{m/m}$)	$E_{1/3}$ (MPa)	$\nu^{1/3}$ (-)
MC-1	525 x 445 x 105	116.0	2.5	2153.7	384.4	0.05
MC-2	570 x 450 x 105	96.4	2.1	1261.4	538.9	0.04

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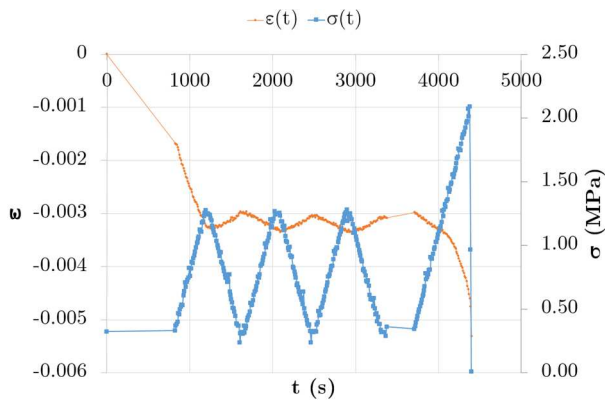
217 **Table 3.** Mechanical parameters of masonry according to **Fig. 4**

Specimen	E (MPa)	E_h (MPa)
MC-2	≈ 700	2770

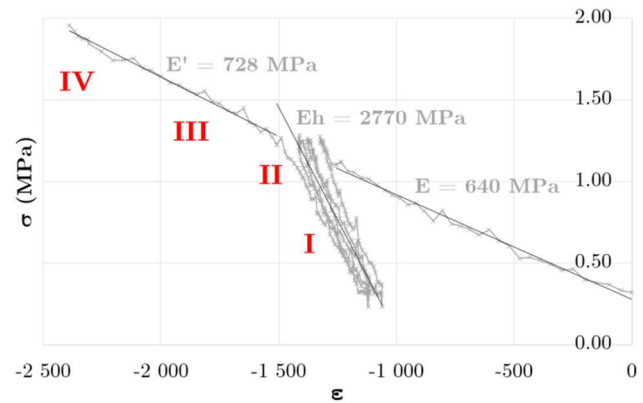
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219 **Fig. 4 (b)** shows the vertical strain stress curve obtained from the stereo correlation data, with the curves
 220 obtained during the load-discharge cycles (LVDT, load cell) used to determine the modulus of elasticity of the
 221 MC-2 wall (cf. **Table 3**). In the elastic domain of masonry, the slope of the curves representing the last two
 222 charge-discharge cycles (cycles 2 and 3) and the slope of the initial part of the curve calculated through the
 223 stereo data correlation are similar. This slope, of approximately 2770 MPa, represents the initial tangent
 224 modulus of elasticity of the masonry. This is typical behavior of a plastic soil. Indeed, one can observe the
 225 residual strain during the cycles 2 and 3 and the increase of the modulus due to pore closure. **Fig. 4 (c)** shows
 226 the evolution of MC-2 vertical deformation field for different compression stress values (after the cyclic
 227 modulus loading). During the first three measuring zones, the deformations evolved almost uniformly along

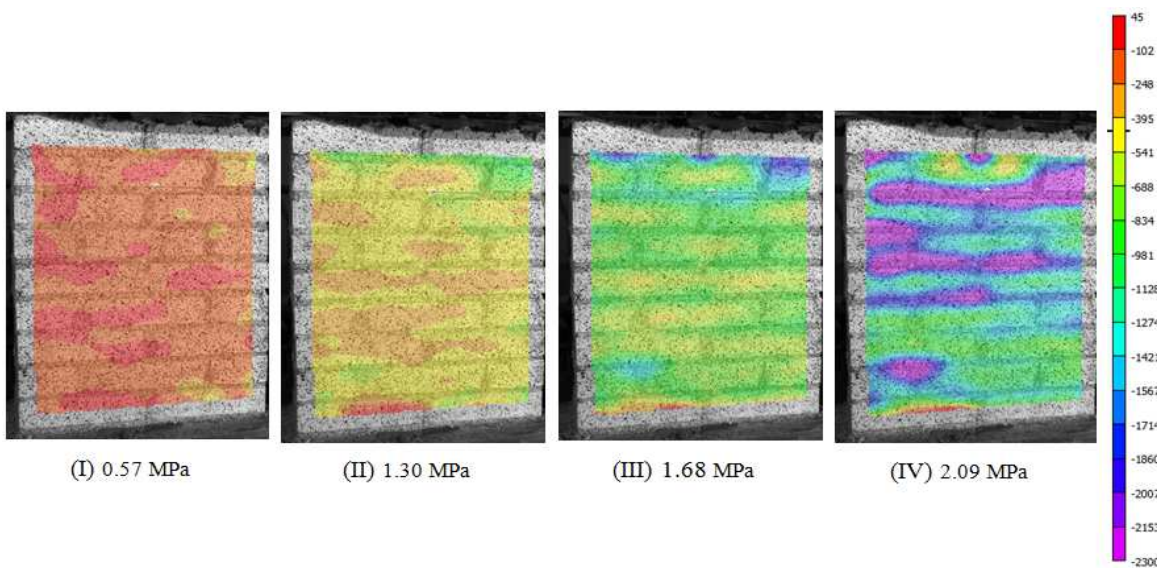
228 the surface of the wall (cf. **Fig. 4 (c)**). For the last measurement point (IV) the most important deformations
 229 were concentrated at the bed joints near to the top of the wall and on the left side of the image. Besides, for
 230 this stress level, a compressed zone at the bottom left of the wallette can be perceived. As shown in **Fig. 4 (c)**,
 231 results can be associated with the failure zones observed at the end of the test. It is interesting to note that the
 232 successive appearance of these damaged zones is reflected by an elastoplastic behavior; the failure is
 233 concentrated on the left side of the specimen due to a slight asymmetry in the loading between the left and
 234 right side. Indeed, for the maximal vertical stress, deformation on the left side was near to one to three times
 235 more important than deformation on the right side.



(a)



(b)



(c)



(d)

236 **Fig. 4** MC-2 test description; (a) loading protocol, (b) vertical strain stress curve obtain throw DIC (c) Evolution of deformation
 237 field ϵ_{yy} [$\mu\text{m}/\text{m}$] for different compressive stress (d) Compression test on masonry.

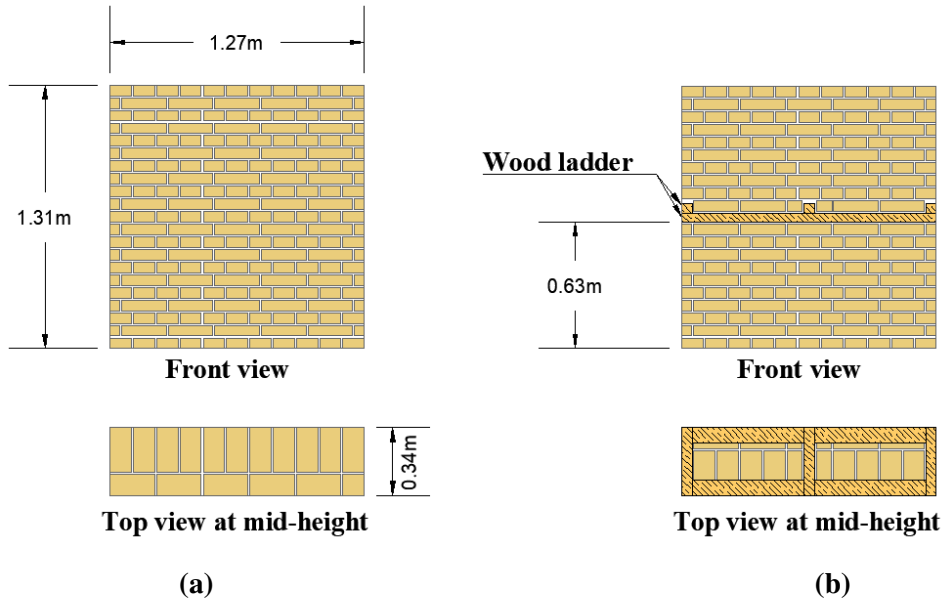
238 4 Element of shear wall

239 Previous section allowed us to evaluate an elementary assembly of the components studied in the first part.
 240 Thus, we highlighted that the monotonous uniaxial loading behavior of masonry depends on the behavior of
 241 the elements interface. The subsequent study of reinforced and unreinforced shear walls submitted to lateral,
 242 cyclic, quasi-static loading will allow studying the crack patterns on a scale representative of the structure.

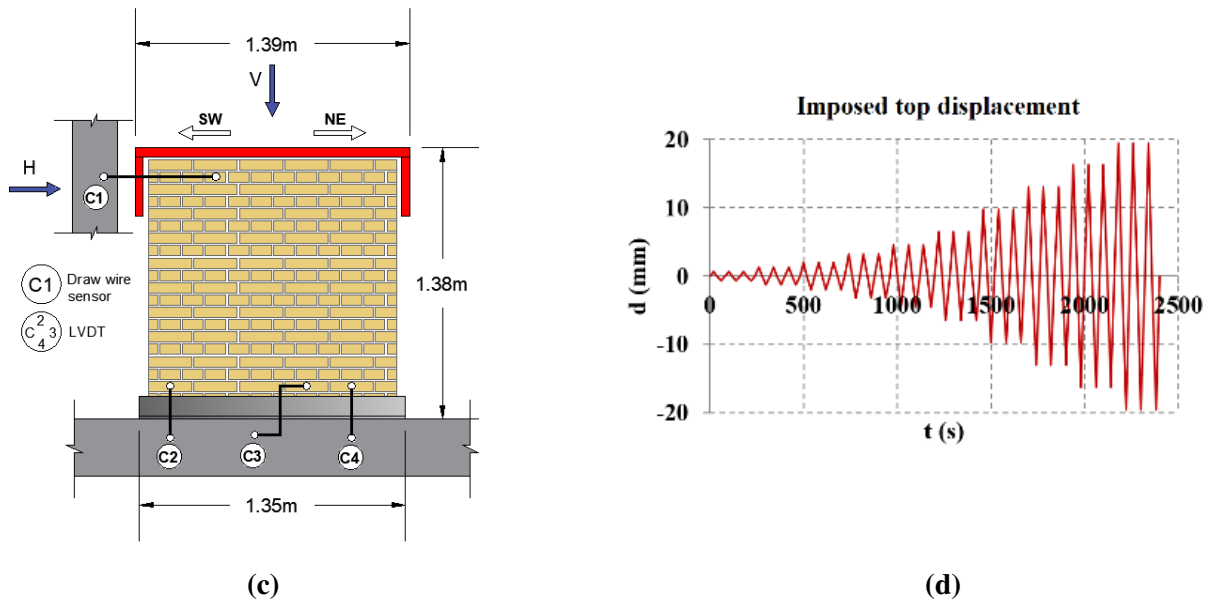
243 4.1 Experimental Setup

244 Two adobe masonry walls were built as depicted in **Fig. 5 (a)** and **(b)**: one was not reinforced (UW, $1.27 \times$
 245 $1.31 \times 0.34 \text{ m}^3$), the other one (RW, $1.33 \times 1.29 \times 0.34 \text{ m}^3$) was reinforced with a ladder-shaped, horizontal
 246 timber insertion. The UW and RW walls were built on welded steel beams. Each wall was built following an
 247 "English" masonry building technique. This bond was chosen to be representative of one of the multiple
 248 techniques used to lay units on earth masonry buildings [24]-[25]. The longitudinal sections of timber beams
 249 had dimensions of $75 \times 45 \text{ mm}^2$, and the transversal ones of $50 \times 45 \text{ mm}^2$, according to the Nepalese design
 250 catalogue DUDBC [14]. The pine timber had a C18 mechanical class according to [17]. Longitudinal and
 251 transversal elements were connected with screws (diameter 5 mm, length 70 mm) to limit the energy
 252 dissipation in the timber-timber connections in order to ease the analysis of the global energy dissipation
 253 mechanism in the wall.

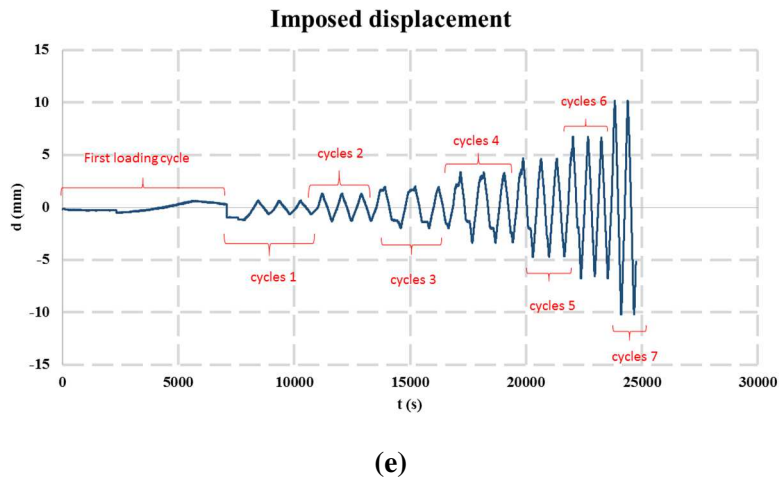
254 A speckle was created on one side of each specimen to prepare the surface of the walls for image correlation
 255 analysis. In order to obtain a uniform distribution of the charges applied to the samples, a layer of mortar was
 256 extended at the top and on the sides of the walls. Then, a steel piece (in red in **Fig. 5 (c)**) was placed on the top
 257 of the wall to ensure the interface between the structure and the machine. Finally, the walls were positioned on
 258 the testing machine where adjusting shims were installed to avoid rigid body movement and reduce the
 259 duration of the test.



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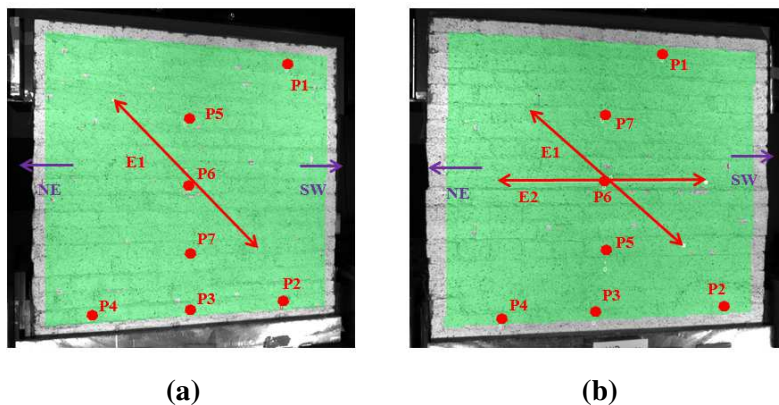


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Fig. 5 Front view and top view at mid-height of unreinforced and reinforced specimens; (a) UW, (b) RW (c) Test setup (d) Imposed top displacement (e) Total imposed displacement.

267 The tests were performed using a press equipped with a vertical hydraulic actuator to apply compressive loads
 268 and a horizontal hydraulic cylinder to impose cyclic loads. Displacements were measured through digital
 269 image correlation (DIC) thanks to a stereo-camera system, completed by three LVDT sensors, and a draw
 270 wire sensor (cf. **Fig. 5 (c)**). To simulate permanent loads and the presence of stories in such structures, a
 271 constant vertical load of 0.2 MPa corresponding to 10% of the compressive strength of the masonry
 272 (determined from wallet scale) was applied on top of the walls. The compression is equivalent to the sum of
 273 the load of two wooden floors and a distributed occupancy load calculated following the Nepal National
 274 Building Code [32]. Vertical load was kept constant during the test by means of the vertical hydraulic actuator
 275 piloted in force. The lateral load was applied under imposed displacement designed according to the standard
 276 ASTM E2126-05 [33] (cf. **Fig. 5 (d)**). The control displacement curve was made of series of three cycles at a
 277 constant frequency (0.013 Hz), with a constant magnitude inside a series, but with increasing magnitude
 278 between two series. The real displacement imposed to the wall was actually different due to a problem during
 279 the first part of the loading path (cf. **Fig. 5 (e)**).

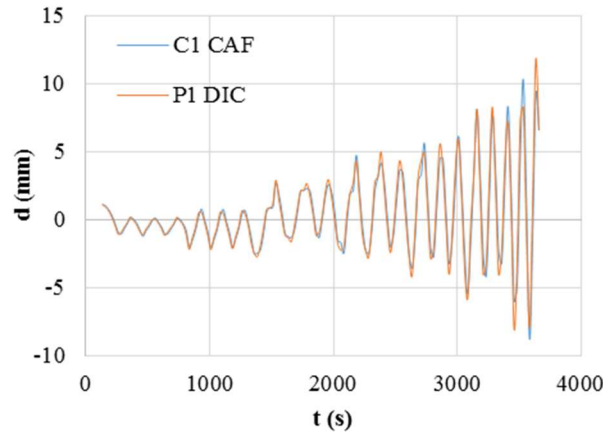
280 The displacement measurements of the sensors integrated in the walls (C1, C2, C3 and C4) were compared
 281 with the corresponding displacements obtained through the CIN system. To this end, seven fictitious study
 282 points (P1 to P7) were defined at several points of the walls. Moreover, two virtual extensometers (E1)
 283 were used to measure the deformation along a diagonal reference line (cf. **Fig. 6**). The positioning of the virtual
 284 sensors has been chosen so as to obtain representative information for the critical zones, i.e. the diagonals, the
 285 center and the base of the walls.



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 287
 288 **Fig. 6** Reference points and virtual extensometers for; (a) UW, Left, (b) RW, Right.

289 4.2 Results

290 **Fig. 7** compares the DIC horizontal displacement measurements at point P1 of the UW with the corresponding
 291 displacements measured by the physical sensor C1. A noticeable concordance between the two displacement
 292 measurements of two systems is obvious. Because the physical sensor and the results of the DIC are not
 293 obtained on the same face of the wall, slight differences exist.



294
295 **Fig. 7** Virtual CIN and C1 sensor displacement comparison.

296 **Fig. 8** shows the horizontal displacement field obtained by DIC with VIC3D software in the reinforced and
 297 unreinforced masonry walls before failure. In the case of the unreinforced masonry adobe wall (UW) (**Fig. 8**
 298 **(a)**), contours show a typical diagonal crack pattern where negative displacements indicate a left displacement
 299 whereas positive values refer to right one. The contours of the reinforced shear wall (RW) (**Fig. 8** **(b)**) show
 300 the apparition of a friction plane along the bed joints. Two types of failure can be observed here. The
 301 unreinforced wall experienced shear failure, according to the presence of diagonal cracks in the joints and
 302 through some adobe units. During the first cycles of the quasi-static test, the horizontally reinforced wall
 303 showed a behavior similar to the first wall: diagonal cracks appeared at the corners of the wall. However,
 304 when the lateral force magnitude increased, a horizontal failure plane developed two beds of bricks below the
 305 timber insertion and sliding occurred along this plane. Additionally, very few cracks appeared above the
 306 reinforcement.

307 The inclusion creates a supplementary interface that limits the displacements of the adjacent elements; this
 308 apparently reduces the total number of cracks on the element and restrains the crack propagation to certain
 309 zones of the wall. Furthermore, the absence of cracks on the first bed of bricks below the timber insertion
 310 could be caused by a prestressed state of these elements due to shrinkage of timber during the drying process.

311 The hysteresis curves lateral force against displacement, corresponding to the quasi-static cyclic tests on UW
 312 and RW, are presented in **Fig. 9**. The envelope corresponds to the points of maximal force and maximal
 313 displacement for each series of three cycles of same magnitude. One can see that UW depicts an asymmetrical
 314 behavior due to the initial monotonic loading before the cycles (cf. **Fig. 5** **(e)**). During this step, the strength of
 315 the wall in the pulling state was reach and it explains why the behavior is close to perfect elasto-plastic.

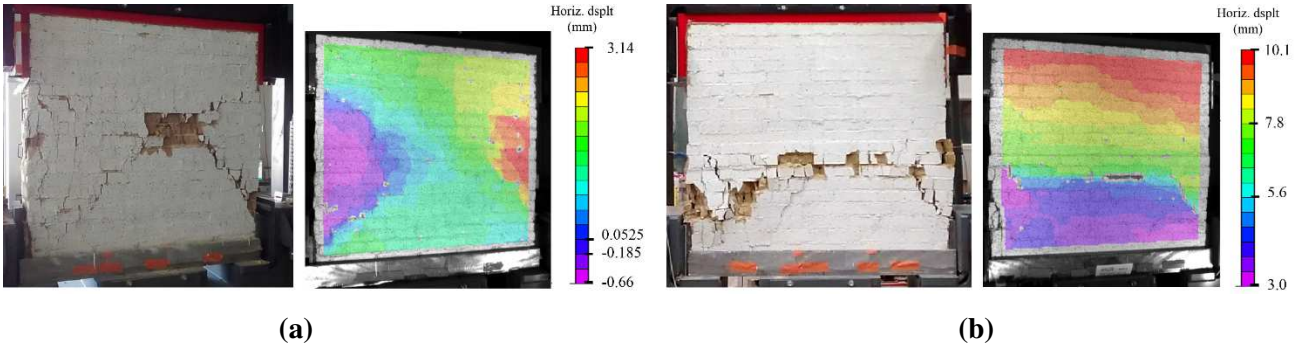


Fig. 8 Horizontal displacement field obtained with DIC and failure pattern: (a) UW, (b) RW.

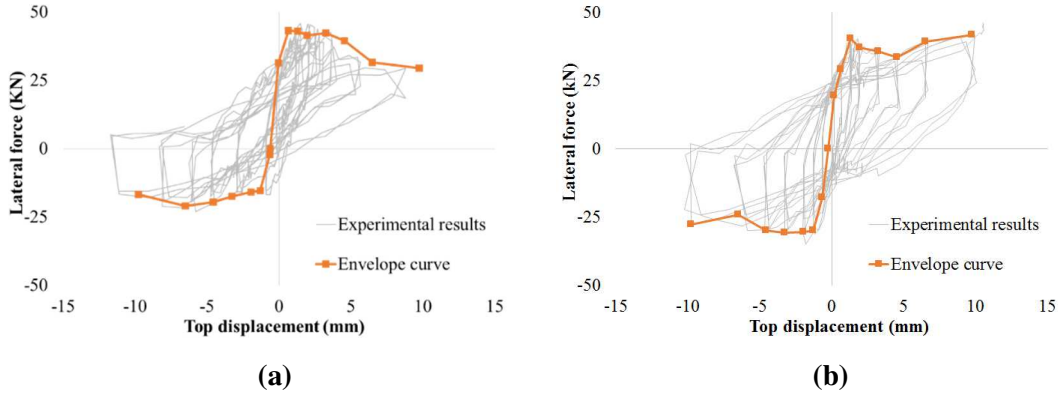


Fig. 9 Hysteresis curves lateral force against displacement for quasi static cyclic tests on: (a) unreinforced masonry shear wall, (b) timber reinforced masonry shear wall.

The use of force-displacement curves is often based on an idealization that transforms these graphs into bilinear envelopes representative of perfectly elastic linear behavior (cf. **Fig. 10 (a)**). In the literature it is possible to find many examples of the application of this method for the analysis of structures seismic response [34]-[36]. From the bilinear representations in push and pull states, the average bilinear idealization for each of the walls was determined. The results are shown in **Fig. 10 (b)**. The RW shows a higher maximum resistance (32.6 kN) and ultimate displacement (9.8 mm).

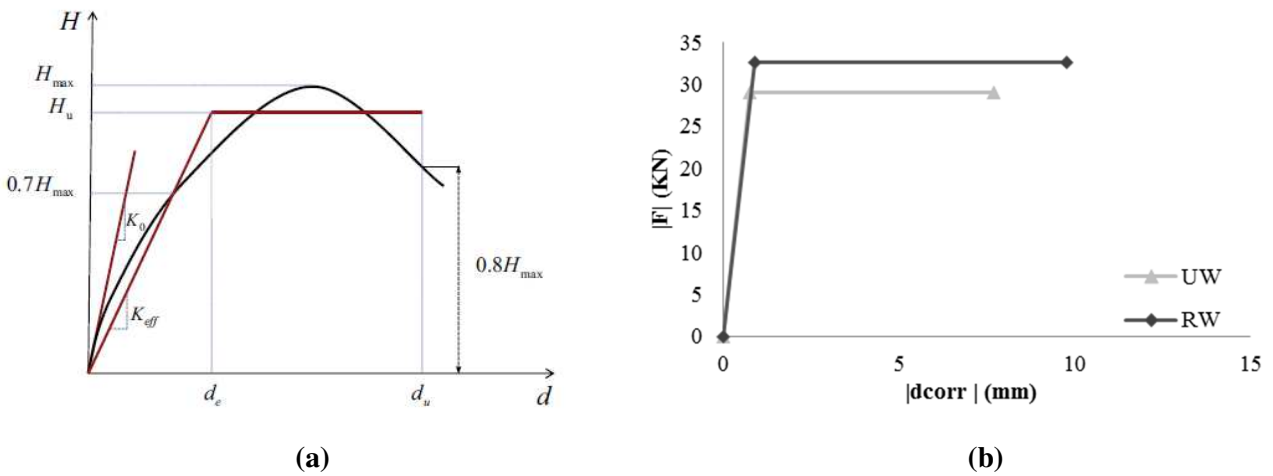


Fig. 10 Bilinear idealization of the hysteresis envelope: (a) critical parameters on seismic behavior analysis [36] (b) idealization for UW and RW.

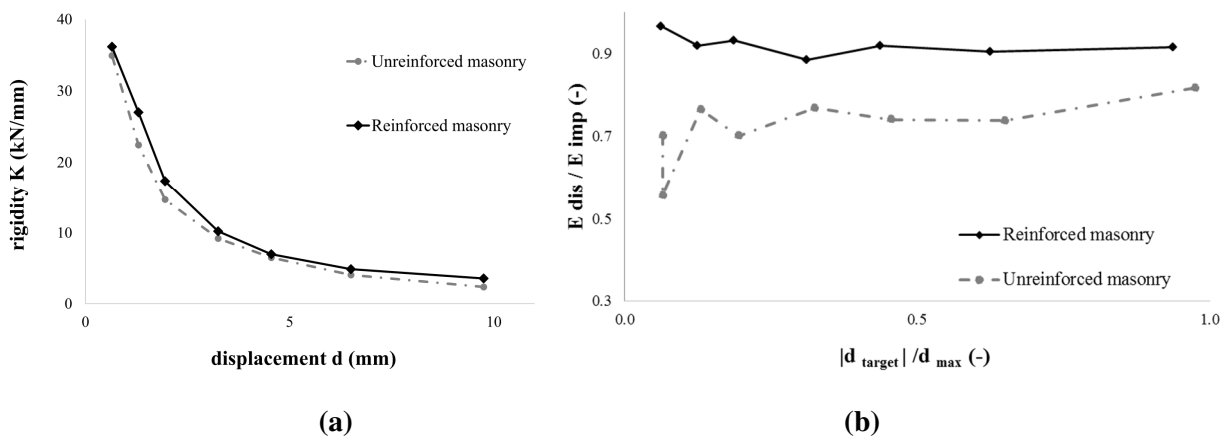
332 **Table 4** lists the different critical parameters evaluated for each wall. The most notable difference between the
 333 two specimens is found in the displacement corresponding to the maximum resistance d_{Hmax} . Indeed, in the
 334 case of the UW, H_{max} was reached for a much smaller displacement (1.26 mm compared to 10 mm of the
 335 RW wall). However, in general the behavior of the two walls is similar with higher resistances in push state
 336 than in pull state and effective stiffness rounding 40 kN / mm. In addition, the RW shows a slight increase in
 337 ductility against the UW.

338 **Table 4.** Lateral resistance and deformability parameters of UW and RW walls

Test	H_{max} (kN)	d_{Hmax} (mm)	K_{Hmax} (kN/mm)	H_{cr} (kN)	d_{cr} (mm)	K_{eff} (kN/mm)	H_u (kN)	d_u (mm)	d_e (mm)	μ (-)
UW-push	43.32	1.26	34.37	30.32	0.55	55.13	38.99	6.20	0.71	8.77
UW-pull	20.99	5.89	3.56	14.69	0.60	24.49	18.89	9.16	0.77	11.87
Average	32.15	3.58	18.96	22.51	0.58	39.81	28.94	7.68	0.74	10.32
RW-push	41.86	10.00	4.18	29.30	0.90	32.56	37.67	10.00	1.16	8.64
RW-pull	30.51	3.00	10.18	21.36	0.55	38.83	27.46	9.50	0.71	13.43
Average	36.19	6.50	7.18	25.33	0.73	35.69	32.57	9.75	0.93	11.04

339

340 Stiffness degradation of the walls was evaluated by computing the slope between two points of maximal force
 341 and displacement of two loading series, which corresponds to the slope between two points of the hysteresis
 342 curves. Loss of rigidity is widely used as an indicator of the seismic performance of a structure. This
 343 parameter depends on characteristics of the structure as well as on the loading history. The stiffness
 344 degradation curves (cf. **Fig. 11 (a)**) reveal similar responses for the two walls. Most of the degradation
 345 happens during the three first loading cycles, and then the stiffness stabilizes. It can be noticed that the
 346 stiffness of the reinforced shear wall degraded slightly more progressively, which fosters the hypothesis of a
 347 more ductile behavior of reinforced masonry pointed out by the hysteresis curves.



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Fig. 11 (a) Stiffness degradation, (b) Energy dissipation.

352 The hysteresis curves also reveal some information about dissipated energy in the system. The dissipated
 353 energy E_{dis} is defined as the area inside a cycle of hysteresis. The input energy E_{inp} is defined as the area
 354 under the curve, down to the x-axis [37]. **Fig. 11 (b)** presents the variation of the ratio E_{dis} / E_{inp} according to
 355 the displacement, normalized with the value of maximum displacement during the test d_{max} . The plot shows
 356 more significant energy dissipation in the case of the timber reinforced masonry wall (90 %) compared to the
 357 unreinforced wall (80 %). This could be explained by the friction phenomenon which is probably more
 358 important for the crack pattern depicted in **Fig. 8 (b)**.

359 **Table 5** records the energies E_{inp} and E_{dis} for the first charge cycles of the series of three cycles applied for
 360 each target displacement. The supplied energy values and the energy dissipation ratio E_{dis}/E_{inp} are also
 361 presented. The first UW cycle (cycle 0) represents the energy values of a preliminary load phase achieved
 362 previously to the test. The results show that, for all displacement cycles, the energy dissipated by the RW
 363 (E_{dis}) is greater than that of the UW, with a final dissipated energy of 403 kNmm for UW against 627 kNmm
 364 for RW. Likewise, it is observed that the energy supplied by the jack to reach the target displacements (E_{inp})
 365 is higher in the case of the RW, with a final supplied energy of 685 kNmm compared to 493 kNmm for the
 366 UW specimen.

367 Results show that the inclusion effect on the stiffness degradation is not significant; however it makes
 368 masonry more ductile, giving it a greater energy dissipation capacity, which highlights the importance of
 369 using this kind of reinforcement system in structures located in seismic zones.

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Table 5. Energy supplied and energy dissipated for each charge cycle

Specimen	Cycle	d target (mm)	d target / d max	E dis (kN mm)	E inp (kN mm)	Edis/Eimp
UW	0	0.65	0.07	24.02	34.22	0.70
	1	0.65	0.07	37.18	66.71	0.56
	2	1.30	0.13	94.91	124.14	0.76
	3	1.95	0.20	107.19	152.93	0.70
	4	3.25	0.33	174.29	226.92	0.77
	5	4.55	0.46	204.52	276.31	0.74
	6	6.50	0.65	244.95	332.15	0.74
	7	9.75	0.98	402.85	492.95	0.82
RW	1	0.65	0.06	31.63	32.74	0.97
	2	1.30	0.13	80.38	87.36	0.92
	3	1.95	0.19	140.60	150.88	0.93
	4	3.25	0.31	179.71	202.83	0.89
	5	4.55	0.44	264.63	287.99	0.92
	6	6.50	0.63	365.77	403.93	0.91
	7	9.75	0.94	626.96	684.68	0.92

372 **5 Conclusions**

373 The results discussed on the scale of the materials highlight the inherent difficulties to characterize the
374 compression strength of earth material. Indeed, this mechanical parameter is highly influenced by the
375 geometry of the samples and by the different properties of the materials used during the tests. Results show
376 that the compressive strength of the bricks increases for a decreasing aspect ratio. Furthermore, it seems that
377 the failure mode depends on the specimen preparation, since only entire bricks tested vertically exhibited the
378 characteristic compression failure mode (two opposite confinement cones), with a relatively low standard
379 deviation that let one think that these tests are the more relevant in the current case. The main limitation for
380 using this test is the geometry of the brick. Regarding the test on stacked bricks, the greater lateral strain of the
381 sand under compression might be the cause of the reduced strength obtained. For the earth mortar, it seems
382 possible to estimate a representative compressive strength, since an aspect ratio of 1 allows achieving the
383 typical compression failure mode. Complementary tests might be performed regarding the stacked bricks
384 configuration since it gives results independent from bricks dimensions.

385 The compressive strength test performed on the masonry revealed a typical compression failure mode,
386 characterized by the appearance of vertical cracks through bricks and joints. Rigidity difference between
387 elements and masonry highlighted that the monotonous uniaxial loading behavior of masonry depends on the
388 behavior of the elements interface. In turn, the use of a stereovision system completes the understanding of
389 masonry mechanical behavior as it allows the tracking of deformation and strain fields through time, and the
390 tracing of the first cracks formation and its propagation. Moreover, it provides complementary data to the
391 more classical measurement systems.

392 Experimental results show significant differences concerning the behavior of the walls. The unreinforced wall
393 presents a shear failure, with the development of a diagonal crack pattern following the joints and going
394 through some units. The reinforced wall failure was characterized by the propagation of diagonal cracks at the
395 bottom corners in the beginning of the test, followed by the formation of a horizontal crack below the
396 insertion and the appearance of a sliding surface between the top and the bottom part of the wall. It was
397 noticed that very few cracks developed in the top part of the wall located above the timber reinforcement.
398 Furthermore, the reinforced wall exhibited a more ductile behavior, characterized by greater energy
399 dissipation for higher displacement levels. However, similarly to the unreinforced system, the wall stiffness
400 degrades considerably for the first levels of lateral solicitation. Concerning lateral resistance, reinforced and
401 unreinforced systems revealed similar results.

402 The inclusion allows limiting the displacements of the adjacent elements which apparently reduces the total
403 number of cracks in the masonry when subjected to lateral forces. Furthermore, timber seems to create a
404 prestressed state on the bed of bricks below the timber insertion, which restrains the crack propagation to
405 certain zones of the wall subjected to ordinary stress conditions. This last feature may limit the risk of collapse

406 of the entire structure when subjected to lateral loads. The overall behavior of reinforced wall highlights the
407 role and the importance of using horizontal timber insertions in masonry structures in seismic zones.

408 The digital images stereo-correlation is a powerful tool which allows identifying some critical parameters in
409 the response evaluation of the walls under horizontal quasi-static loading. In particular, this system provides
410 information on the moment of first cracks formation, the cracks propagation during the test, and the history of
411 deformations and strain fields. Moreover, the DIC facilitates analysis of results as data is presented through
412 images. However, the accuracy of the results obtained through this technique depends directly on the quality
413 of the recorded images, therefore, a correct acquisition system calibration is critical.

414 A perspective of the present experimental results is the study of the development of a computational model of
415 the timber reinforced shear wall (see [38]). The current knowledge of the mechanical properties of the
416 elements and of the masonry is a solid base to determine the parameters of a homogenized constitutive law of
417 the earth masonry. A further study of the interface between timber and masonry is also needed to have a
418 proper understanding of the reinforced wall behavior.

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