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1 ORIGINAL ARTICLE

2 **Assessing the three-dimensional behaviour of dry stone retaining**
3 **walls by full-scale experiments**

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12 **ARTICLE HISTORY**

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14 **Abstract**

15 Dry stone masonry is a widespread building technique, which has been used in
16 Europe and all around the world in both monumental and vernacular architecture.
17 Amongst them, dry stone structures retaining slopes have received growing attention
18 over the past two decades, but only few studies concentrate on the influence of
19 localised loading upon the backfill. This paper describes an experimental campaign,
20 comprising two tests on full-scale structures, which has been undertaken in France
21 in order to investigate the behaviour of dry stone road retaining walls. The results
22 of these tests are compared with those of a previous experimental campaign, and of
23 a theoretical approach.

24 **KEYWORDS**

25 Dry stone masonry; retaining walls; full-scale tests; 3D behaviour; upper-bound
26 limit analysis

27 1. Introduction

28 Dry stone walling is an ancient, vernacular building technique that can be found world-
29 wide, in areas where the supply of stones is significant and planning difficult. In Europe,
30 dry stone accounts in particular for a large part of the retaining walls along road net-
31 works. Although dry stone structures have proven their robustness and sustainability,
32 when they experience important pathologies or collapse, they are most often replaced
33 with concrete or gabion alternatives, because there is a lack of knowledge of this tech-
34 nology.

35 Research on dry stone masonry is mainly dedicated to building construction (Bui,
36 Limam, Sarhosis, and Hjjaj, 2017; Lourenço, Oliveira, Roca, and Orduña, 2005; Smol-
37 janović, Živaljić, Nikolić, and Munjiza, 2018; Walker and Dickens, 1995). Nevertheless,
38 over the past two decades, research studies in France and in UK have been dedicated
39 to dry stone earth retaining walls. Distinct element modelling has been developed to
40 understand the bulging pathologies in existing walls (Claxton, Hart, McCombie, and
41 Walker, 2005; Harkness, Powrie, Zhang, Brady, and O'Reilly, 2000; Powrie, Harkness,
42 Zhang, and Bush, 2002; Walker, McCombie, and Claxton, 2007), based on the exper-
43 imental data from (Burgoyne, 1853). On the other hand, limit equilibrium (Mundell,
44 McCombie, Bailey, Heath, and Walker, 2009; Villemus, Morel, and Boutin, 2007), yield
45 design (Colas, Morel, and Garnier, 2010, 2013), and distinct element (Oetomo, Vin-
46 cens, Dedecker, and Morel, 2015) analyses have been performed to assess the stability
47 of new or existing structures, calibrated and validated on dedicated full-scale tests (Co-
48 las et al., 2010; McCombie, Mundell, Heath, and Walker, 2012; Mundell, McCombie,
49 Heath, Harkness, and Walker, 2010; Villemus et al., 2007). Numerical and theoretic-
50 al developments have been exclusively performed in 2D on wall cross-sections. Only
51 one full-scale (McCombie et al., 2012; Mundell et al., 2010) and one recent scale-down
52 (Quezada, Vincens, Mouterde, and Morel, 2016) experimental campaign have been
53 performed in 3D, to assess the effect of traffic on dry stone walls.

54 Recently, Le, Garnier, Colas, Terrade, and Morel (2016) have developed a 3D model,
55 using yield design theory, in order to take into account the effect of axle loads on dry
56 stone retaining structures. On-going on this study, this paper presents an experimental

57 campaign on full-scale point loaded retaining walls. First, the experimental configura-
58 tion and protocol is detailed. The results are analysed, and compared with those of
59 a previous experimental campaign and of a theoretical approach. Perspectives on this
60 work include a new experimental campaign, which will be introduced in the conclusion.

61 **2. Configuration of the experimental campaign**

62 The experimental campaign aims at a better understanding of the three-dimensional
63 behaviour of dry stone retaining walls. It comprises two on-site tests, undertaken in
64 Vaucluse (France), consisting in loading until failure full-scale dry stone retaining walls
65 with a loading plate placed on the top of the backfill (Fig. 1).

66 **2.1. Wall construction**

67 The experimental walls were constructed by professional dry stone wallers, following
68 dry stone state-of-the-art practice, as detailed in (CAPEB, 2008) and included in (Mc-
69 Combie, Morel, and Garnier, 2015), except for through-stones, which have not been
70 introduced in the wall in order to encourage its deformation. Nevertheless, the stones
71 on each face overlapped and voids have been filled with small pieces of stone (Fig. 2).

72 The two test walls are made of limestone. They are 2.3 m high and more than 10 m
73 long, to limit side effects during the experiment. Wall thickness has been selected based
74 on the 2D yield design model developed by the authors (Colas et al., 2010, 2013), and
75 on 3D full-scale experiments undertaken in the University of Bath (McCombie et al.,
76 2012; Mundell et al., 2010). Self-standing walls are built and backfilled to the top of
77 the wall. Walls are made of limestone, characterised by its inter-block friction angle
78 $\phi = 36.0 \pm 0.5^\circ$, measured by shear box testing. The backfill is made of rolled gravel. It
79 has already been used and characterised in a previous experimental campaign (Colas
80 et al., 2010) using triaxial test, providing an internal friction angle $\phi_s = 37.7^\circ$. The
81 backfill is placed without compaction, and its surface is finally levelled. The geometrical
82 and physical characteristics of the wall, the backfill and the loading system are recorded
83 in Table 1.

84 Considering this experimental campaign was the first of its kind in France, the

85 first experiment (wall 1) has been disrupted by several technical problems. Yet, this
86 experiment has provided a first estimate of the ultimate load and has enabled the
87 improvement of the experimental protocol and device. This paper will focus on the
88 second experiment (wall 2).

89 *2.2. Loading device and protocol*

90 In order to simulate the action of the traffic on the retaining wall, a load was applied to
91 the surface of the backfill via a 60×60 cm steel plate. For simplicity reasons, it has been
92 decided to use a 34 ton excavator to apply the load (Fig. 3a). The load is measured by
93 a load cell on which is fixed a hemi-spherical head (Fig. 3b). The load measured by the
94 sensor is thus the vertical component of the excavator load, the horizontal component
95 being considered as negligible, given the vertical orientation of the excavator pressure.

96 The loading protocol has been undertaken in 5 steps (Tab. 2). The first experiment
97 showed that the load have no influence on the wall when applied on the surface of the
98 backfill over 2 m from the top of the wall. This is consistent with the standard practice
99 considering that concentrated loads irradiate with a maximal angle of 45° (EN 1991).
100 For the second testing wall, the load was first applied at a distance $d = 1.70$ m from
101 the top of the wall, in 2 steps. Then, the load has been moved closer, at $d = 0.50$ m
102 from the top of the wall, and the loading process has been carried out in 3 steps. The
103 steps are due to the steel plate, which sinks in the gravel backfill. In order to keep
104 constant loading conditions, the loading process is stopped to remove the steel plate,
105 add gravel and ensure a level surface of the backfill.

106 The wall displacements have been recorded throughout the experiment by cable
107 sensors and at key points of the loading process using a laser measurement device.

108 *2.3. Measurement device*

109 *2.3.1. Cable measurement device*

110 Displacements of the wall are measured by 12 cable sensors, already used by Villemus
111 et al. (2007) and Colas et al. (2010) in their experimental campaign. Yet, unlike the
112 previous plane strain experimental campaigns, the sensors are disposed crosswisely, 8

113 on a vertical line at the middle of the wall and 5 on a horizontal line (Fig. 4), in order
114 to measure displacements of the whole front face. They are fixed on a steel structure
115 at one end, and on wood target stuck on blocks of the front face on the other end. The
116 data acquisition system records the displacements every 2 seconds.

117 *2.3.2. Laser measurement device*

118 Along with the cable sensor system, it has been decided to use a laser measurement
119 device to measure the wall displacements. The laser system relies on a tracker (Fig. 5a)
120 measuring the position of 60×60 mm targets (Fig. 5b) with an accuracy of 1 mm. 20
121 targets are disposed on half of the front face of the wall, 5 of them being paired with
122 cable sensor targets in order to assess the correspondence between the two measurement
123 systems (Fig. 4). It has to be noted that laser is a one-time measurement system
124 whereas cable system performs a continuous measurement of the wall displacements.

125 **3. Testing results and analysis**

126 *3.1. Measurement device*

127 This experiment provides an opportunity to test the displacement sensor device. Cable
128 sensors have already been used by the authors for plane strain full-scale experiments
129 and proved consistent for the measurement of the wall displacements. In this experi-
130 mental campaign, it has been decided to use also laser sensors. The repartition of cable
131 sensor and laser sensor targets are displayed in Figure 4. 10 targets (cable targets C11,
132 C7, C2, C13, C14, and laser targets L1, L2, L3, L5, L10) have been paired in order to
133 assess the differences between the two systems.

134 Figure 6 shows the displacements measured by the 5 couples of sensors for stage
135 1.7mII of the second experiment. This graph shows that the displacements given by
136 the two systems have the same order of magnitude. Results are even closer for large
137 displacements. These two systems are complementary, with the cable sensors providing
138 continuous measurement on a limited number of targets, and the laser sensors providing
139 a one-time measurement of a larger area.

140 *3.2. Elasto-plastic behaviour of the structure*

141 Figures 7 and 8 represent the load-displacement response, generated from the force
142 measured by the force sensor, and the displacements measured by the sensors situated
143 on the vertical axis of symmetry of the wall face (Fig. 4). Figure 7 shows the results of
144 sensor C7, when the distance d between the load and the top of the wall equals 1.7 m,
145 and Figure 8 shows the results of sensors C2, C4, C5, C8 and C10, for $d = 0.5$ m.
146 The results of the other sensors were also plotted, with similar responses; they are not
147 reported on the figures for the sake of clarity.

148 Considering the testing protocol (Tab. 2), which alternates loading and unloading
149 stages of the structure, Fig. 7 shows a perfect elasto-plastic behaviour of the retaining
150 system, comprising the wall and its backfill, during the experiment, when the load is
151 1.7 m distant from the wall. Figure 8 proves that the hypothesis of perfect plasticity
152 no longer applies, when the load get closer to the top of the wall. Yet, the wall-backfill
153 system experiences a post-peak behaviour which can be assimilated to softening.

154 *3.3. 3D deformation of the wall*

155 Figure 9 shows the displacement of the front face in the central cross-section of the
156 wall, during the whole experiment. It can be noticed that, when the localised loading
157 is far from the wall ($d = 1.7$ m), the displacements are quite small, even if the load
158 reaches 90 kN. Displacements are much larger when the load is close to the wall, even
159 for small loading. In a first approximation, the wall seems to experience two modes of
160 deformation during the experiment:

- 161 • a translation of the upper two-thirds of the wall;
- 162 • a shearing of beds in the lower third of the wall.

163 Figures 10 and 11 show that the deformation of the wall is actually in 3D, with little
164 displacements for side cross-sections and larger displacements in the central cross-
165 section. These results have to be compared to Villemus et al. (2007) and Colas et al.
166 (2013), where the deformation of a dry stone retaining wall solely loaded by its backfill
167 is the same for every cross-section of the wall. The experimental campaign presented
168 hereby confirms the 3D behaviour of a retaining wall subjected to a localised loading

169 Figure 12 shows the wall condition after failure. This confirms the 3D behaviour of
170 the wall, as the breach is included in a 4×1.5 m rectangle located in the central part of
171 the wall. The ends of the wall have not experienced any deformation. These observations
172 confirm the possibility of repairing dry stone breaches with a careful reconstruction of
173 the collapsed part of the wall.

174 4. Comparative analysis and discussion

175 *4.1. Comparison with 3D full-scale testing at the University of Bath*

176 Only two full-scale experimental campaigns have been performed in order to assess the
177 3D behaviour of dry stone retaining walls. The first one took place at the University
178 of Bath (UK), comprising 5 tests on 2.5 m high walls (McCombie et al., 2012; Mundell
179 et al., 2010). Walls were built in limestone or slate, backfilled with crushed gravel, and
180 loaded with a hydraulic jack applied on a plate placed on the surface of the backfill.
181 The characteristics of the walls and the tests are given in Table 3.

182 The two experimental campaigns are quite similar, as they both consists in building
183 dry stone walls, backfilled with a granular material, and loading them until failure with
184 a load applied on the backfill. Yet, the experiments differs in their objectives: the tests
185 undertook at the University of Bath aimed at a better understanding of the pathologies
186 of existing walls, and more especially the origin of bulge deformations. Assuming that
187 bulges are due to the settlement of the soil of foundation, the experimental walls have
188 been built on a steel platform which could be raised, lowered or tilted in order to
189 simulate ground motions. The first experimental wall has been subjected to raising
190 and tilting stages, whereas the other four walls where only raised of a few centimetres.
191 Besides, walls 2 to 5 were deliberately built with a poorer quality of construction in
192 order to make more realistic experiments, as most of the existing walls were not been
193 built by skilled masons.

194 Both experimental campaigns prove that dry stone retaining walls subjected to a
195 load on the surface of the backfill experience 3D behaviour. Moreover, the comparative
196 analysis shows that dry stone walls can withstand significant deformation prior failure,
197 showing a certain ductility, even poorly constructed. This property is a valuable asset

198 for monitoring and maintenance of existing walls as significant deformations provide a
 199 visual indication of the potential weakness of the structure.

200 *4.2. Comparison with a 3D yield design model*

201 The results of the experimental campaign have also been used to test yield design
 202 models. Comparisons have been drawn at two key-stages of the experiment: when the
 203 localised loading is placed at 1.70 m of the top of wall (stage 1.7mII), and at 0.50 m
 204 (stage 0.5mI).

205 For stage 0.5mI, comparisons have been performed with a 3D model developed by
 206 Le et al. (2016) to assess dry stone road retaining structures. In this model, the effects
 207 of traffic loadings have been represented by a single pointed load applied on a rigid
 208 plate on the backfill, figuring the action of an axle (Fig. 13). The model is based on
 209 the knowledge of the geometry of the structure, the loading mode and the strength
 210 criterion of the constituent materials. The 3D strength criterion of dry stone is obtained
 211 using periodic homogenisation in the framework of yield design. The strength domain
 212 of the homogenised masonry is given by:

$$G^{\text{hom}} = \left\{ \underline{\underline{\Sigma}} / \underline{\underline{\Sigma}} : \underline{\underline{D}} \leq \pi^{\text{hom}}(\underline{\underline{D}}) = \frac{C}{\tan \phi} \text{tr}(\underline{\underline{D}}) \right\} \quad (1)$$

213 where :

- 214 • $\underline{\underline{\Sigma}}$ is the macroscopic stress field;
- 215 • $\underline{\underline{D}}$ is the macroscopic strain rate field;
- 216 • $\pi^{\text{hom}}(\underline{\underline{D}})$ is the support function of G^{hom} depending on the cohesion C , the fric-
 217 tion angle ϕ of the joints, and conditions on the relevant velocity fields, exposed
 218 in Le et al. (2016).

219 Then, yield design upper-bound approach is undertaken to assess the stability of
 220 the whole structure, comprising the dry stone wall, the backfill, and the load upon the
 221 backfill. The virtual failure mechanism families explored in this model is defined on a
 222 half-system (Fig. 13), and comprises:

- 223 • a translation $\underline{v} = v \cos \theta \underline{e}_1 + v \sin \theta \underline{e}_3$ of a polyhedron $JRSTMOPN$ of wall;

224 • a translation $\underline{v}_s = v_s \cos(\psi_s - \phi_s) \underline{e}_1 - v_s \sin(\psi_s - \phi_s) \underline{e}_3$ of a polyhedron
 225 *MOIKRJ* of soil.

226 The kinematic approach of yield design is based on the application of the principle
 227 of virtual works and states that the work of the external forces W_e has to remain lower
 228 than the maximal resisting work W^{mr} :

$$\forall \underline{v} \text{ kinematically admissible, } W^e \leq W^{\text{mr}} \quad (2)$$

229 Considering there is no cohesion in the dry stone wall joints and within the gravel
 230 backfill, the maximum resisting work vanishes to 0. The work of the external forces is
 231 given by the sum of the work of the body forces in the wall and the backfill, and the
 232 work of the point load, so that:

$$W_e = \underbrace{\int_V \underline{\gamma} \cdot \underline{v} \, dV}_{W_{\text{wall}}^e} + \underbrace{\int_V \underline{\gamma}_s \cdot \underline{v}_s \, dV}_{W_{\text{soil}}^e} + \underline{F} \cdot \underline{v}_F \quad (3)$$

233 The inequality exposed in (2) finally provides an upper-bound estimation of the
 234 ultimate load F^+ that the retaining system can bear :

$$F_{\text{system}}^+ = \min_{\substack{\text{kinematic} \\ \text{parameters}}} \left\{ - \frac{W_{\text{wall}}^e + W_{\text{soil}}^e}{v_s \sin(\psi_s - \phi_s)} \right\} \quad (4)$$

235 Using parameters of Tab. 1, the ultimate load is $F^+ = 189$ kN.

236 For stage 1.7mII, the experiment proved that the results can merely be considered as
 237 the test of the bearing capacities of the backfill soil (Fig. 14), as the wall has experienced
 238 very small displacements. It has thus been decided to use a 3D yield design model of a
 239 semi-infinite soil loaded by a localised loading (Garnier, 1995; Maghous and Garnier,
 240 1995).

241 The virtual failure mechanism family is a presented at Fig. 14 for a quarter-system,
 242 given the double-symmetry of the problem:

243 • the volume $AA'B'BCC'$ is given a translation \underline{v}_{s1} ;

- 244 • the volumes $AA'C'CDD'$ and $ABCK$ are given a deformation $\underline{v}_{s2} = v_{s2}e^{\theta \tan \phi} \underline{e}_\theta$
245 and \underline{v}_{s20} , respectively;
- 246 • the volumes $AA'D'DEE'$ and $ABKG$ are given a translation \underline{v}_{s3} and \underline{v}_{s30} , re-
247 spectively.

248 The maximum resisting work W^{mr} still equals 0. The work of the external forces is
249 given by the work due to the soil unit weight and that due to the point load, so that:

$$W_e = \underbrace{\int_V \gamma_s \cdot \underline{v}_s \, dV}_{W_{\text{soil}}^e} + \underline{F} \cdot \underline{v}_F \quad (5)$$

250 The yield design upper-bound approach (2) finally provides an estimation of the
251 ultimate load F^+ the soil can bear:

$$F_{\text{soil}}^+ = \min_{\substack{\text{kinematic} \\ \text{parameters}}} \left\{ - \frac{W_{\text{soil}}^e}{v_{s1}} \right\} \quad (6)$$

252 For the rounded gravel used in the experiment, the model provides an ultimate load
253 of $F^+ = 339$ kN.

254 These results show that the model happens to overestimate the experimental results
255 with a factor of 3. This can be accounted by the upper-bound nature of the theo-
256 retical approach chosen, which necessarily overestimates the actual solution, and the
257 complexity of the 3D homogenisation and calculation processes, which prevents from
258 computing the optimal solution.

259 The difference can also be due to the experimental protocol. Actually, this experi-
260 mental wall does not conform to the whole standards of dry stone construction, and
261 have intentionally been built with weaknesses. Furthermore, the vertical direction of
262 the load upon the backfill has proven difficult to control during the test, as the steel
263 plate sink into the gravel, despite of the use of a hemi-spherical head. This can lead to
264 an underestimation of the load which has been really applied on the plate.

265 To go further in this comparison, it has been decided to introduce a reduction coef-
266 ficient of the backfill soil bearing capacities, defined as the ratio between the ultimate
267 bearing capacities of the retaining system and those of the backfill soil, considered as

268 a semi-infinite medium:

$$r = \frac{F_{\text{system}}^+}{F_{\text{soil}}^+} \quad (7)$$

269 The theoretical reduction coefficient is given by the the ratio between the ultimate loads
270 given by equations (4) and (6). The experimental reduction coefficient is defined as the
271 ratio between the ultimate load at stage 0.5mI (with the concentrated load at 0.5 m
272 of the top of the wall) and that of stage 1.7mII (with the concentrated load at 1.7 m
273 of the wall). The results are given in Tab. 4. It can be seen that the experimental and
274 theoretical reduction coefficient are similar. This would tend to support the hypothesis
275 of an overestimation of the 3D yield design modelling. More experimental data are
276 needed to confirm this result.

277 5. Conclusions

278 dry stone masonry is widespread in Europe and all around the world, where it is mainly
279 used to build road retaining walls. These earth retaining structures have received grow-
280 ing attention over the past two decades, to better understand their behaviour, but only
281 few studies concentrate on the influence of localised loading on the backfill.

282 This paper describes an experimental campaign, which has been undertaken in
283 France in order to investigate the behaviour of dry stone road retaining walls. Two ex-
284 perimental dry stone retaining walls have been built and localised loading has been ap-
285 plied to the surface of the backfill until failure. Tests have proven the three-dimensional
286 behaviour of dry stone walls subjected to an axle load. Walls have experienced a de-
287 formation located around the point of load application, leading to the collapse of the
288 central part of the wall. Comparisons with a yield design model shows a gap between
289 theoretical and experimental results, which can be accounted for by the differences
290 between the experimental protocol and the modelling hypotheses.

291 Perspectives on this work include a new experimental campaign, with different ma-
292 terials and geometry, complying with the modelling hypotheses. Besides, the model will
293 also be enhanced, using numerical developments of the yield design approach exposed

294 here, and thus enabling the test of all the kinematically admissible virtual velocity
295 fields. These parallel developments can lead to go further in the analysis of dry stone
296 retaining structures, and propose a practical calculation tool dedicated to these struc-
297 tures.

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305 **References**

- 306 T.T. Bui, A. Limam, V. Sarhosis, and M. Hjiiaj. Discrete element modelling of the in-plane
307 and out-of-plane behaviour of dry-joint masonry wall constructions. *Engineering Structures*,
308 136:277–294, 2017.
- 309 J. Burgoyne. Revetments or retaining walls. *Corps of Royal Engineering Papers*, 3:154–159,
310 1853.
- 311 CAPEB. *Guide des bonnes pratiques de construction de murs de soutènement en pierre sèche*.
312 ENTPE, Vaulx-en-Velin, 2008.
- 313 M. Claxton, R.A. Hart, P.F. McCombie, and P.J. Walker. Rigid block distinct-element mod-
314 elling of dry-stone retaining walls in plane strain. *Journal of Geotechnical and Geoenviron-*
315 *mental Engineering*, 131(3):381–389, 2005.
- 316 A. S. Colas, J. C. Morel, and D. Garnier. Full-scale fields to assess dry-stone retaining wall
317 stability. *Engineering Structures*, 32:1215–1222, 2010.
- 318 A. S. Colas, J. C. Morel, and D. Garnier. Assessing the two-dimensional behaviour of drystone
319 retaining walls by full-scale experiments and yield design simulation. *Géotechnique*, 63(2):
320 107–117, 2013.
- 321 D. Garnier. *Analyse par la théorie du calcul à la rupture des facteurs de réduction de la capacité*
322 *portante de fondations superficielles*. PhD thesis, ENPC, 1995.

323 R. M. Harkness, W. Powrie, X. Zhang, K. Brady, and M. O'Reilly. Numerical modelling of
324 full-scale tests on drystone masonry retaining wall. *Géotechnique*, 50(2):165–179, 2000.

325 H. H. Le, D. Garnier, A-S. Colas, B. Terrade, and J-C. Morel. 3D homogenised strength
326 criterion for masonry: application to drystone retaining walls. *Journal of the mechanics and*
327 *physics of solids*, 95:239–253, 2016.

328 P.B. Lourenço, D.V. Oliveira, P. Roca, and A. Orduña. Dry joint stone masonry walls subjected
329 to in-plane combined loading. *Journal of Structural Engineering*, 131(11):1665–1673, 2005.

330 S. Maghous and D. Garnier. Feasibility of a numerical method for computing the ultimate
331 loads of three dimensional structures. *Mechanics Research Communications*, 22(3):279–288,
332 1995.

333 P. McCombie, C. Mundell, A. Heath, and P. Walker. Drystone retaining walls: ductile engi-
334 neering structures with tensile strength. *Engineering Structures*, 45:238–243, 2012.

335 P. F. McCombie, J-C. Morel, and D. Garnier. *Drystone Retaining Walls: Design, Construction*
336 *and Assessment*. CRC Press, London, 2015.

337 C. Mundell, P. McCombie, C. Bailey, A. Heath, and P. Walker. Limit-equilibrium assesement
338 of drystone retaining structures. *Geotechnical Engineering*, 162(4):203–212, 2009.

339 C. Mundell, P. McCombie, A. Heath, J. Harkness, and P. Walker. Behaviour of drystone
340 retaining structures. *Structures and Buildings*, 163 (1):3–12, 2010.

341 J. J. Oetomo, E. Vincens, F. Dedecker, and J-C. Morel. Modeling the 2D behavior of dry-stone
342 retaining walls by a fully discrete element method. *International Journal for Numerical and*
343 *Analytical Methods in Geomechanics*, 40(7):1099–1120, 2015.

344 W. Powrie, R. Harkness, X. Zhang, and D. I. Bush. Deformation and failure modes of drystone
345 retaining walls. *Géotechnique*, 52(6):435–446, 2002.

346 J-C. Quezada, E. Vincens, R. Mouterde, and J-C. Morel. 3D failure of a scale-down dry stone
347 retaining wall: a DEM modeling. *Engineering Structures*, 117:506–517, 2016.

348 H. Smoljanović, N. Živaljić, Ž. Nikolić, and A. Munjiza. Numerical analysis of 3D dry-stone
349 masonry structures by combined finite-discrete element method. *International Journal of*
350 *Solids and Structures*, 136-137:150–167, 2018.

351 B. Villemus, J. C. Morel, and C. Boutin. Experimental assessment of dry stone retaining wall
352 stability on a rigid foundation. *Engineering Structures*, 29(9):2124–2132, 2007.

353 P. Walker, P. McCombie, and M. Claxton. Plane strain numerical model for drystone retaining
354 walls. *Geotechnical Engineering*, 160(2):97–103, 2007.

355 P.J. Walker and J. Dickens. Stability of medieval dry stone walls in zimbabwe. *Géotechnique*,

356 45:141–147, 1995.



Figure 1. Overview of the experimental site in Vacluse (Photo credit: J. Blanc-Gonnet).



(a)



(b)

Figure 2. Testing wall 2 : elevation (a) and top view (b).



Figure 3. Experimental loading system: 34 tonne excavator (a) acting on a hemi-spherical head linked to the force sensor (b).

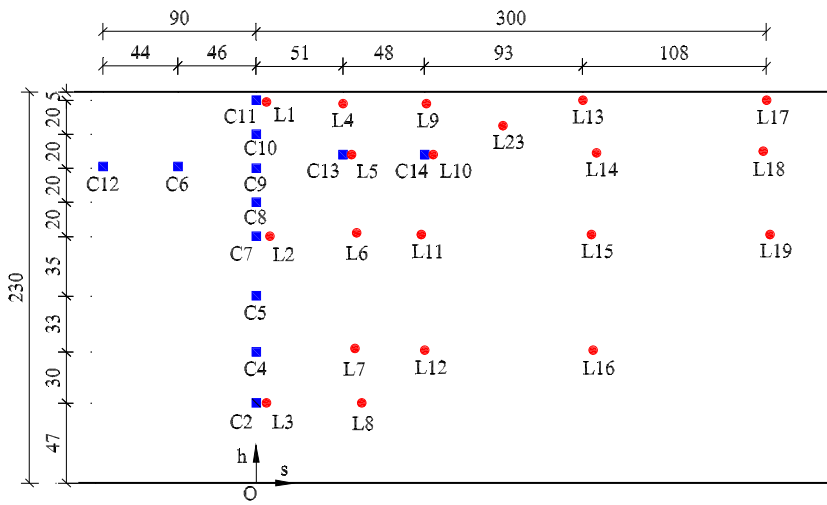
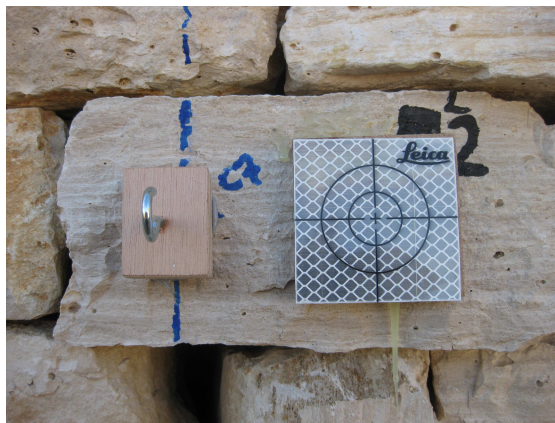


Figure 4. Distribution of the sensor targets: Ci - cable sensor target; Li - laser target (distances in cm).



(a)



(b)

Figure 5. Laser measurement device : tracker (a) and 60 × 60 mm target (b).

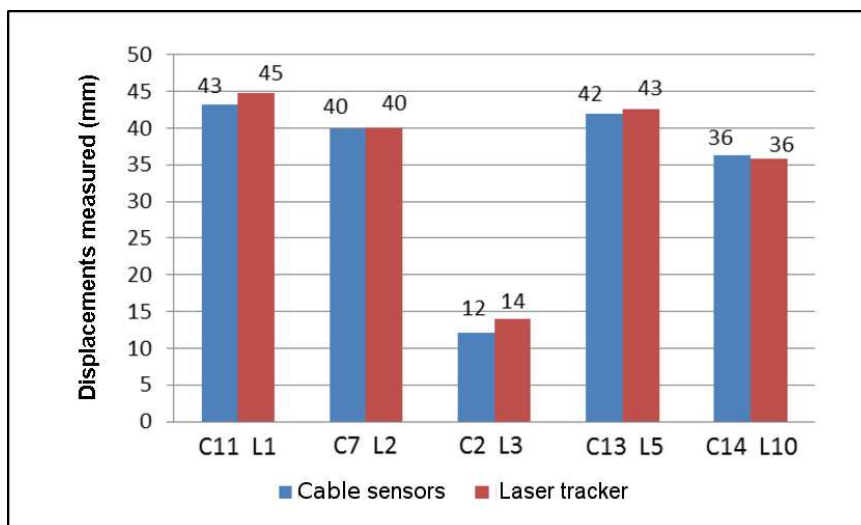


Figure 6. Comparison between the displacements measured by cable sensors and laser sensors for stage 1.7mII.

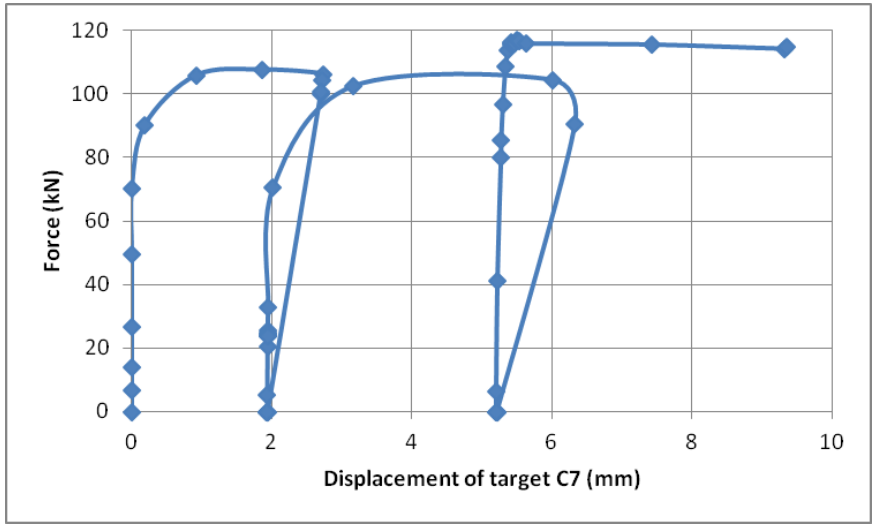


Figure 7. Force against sensor C7 displacements during stage 1.7II.

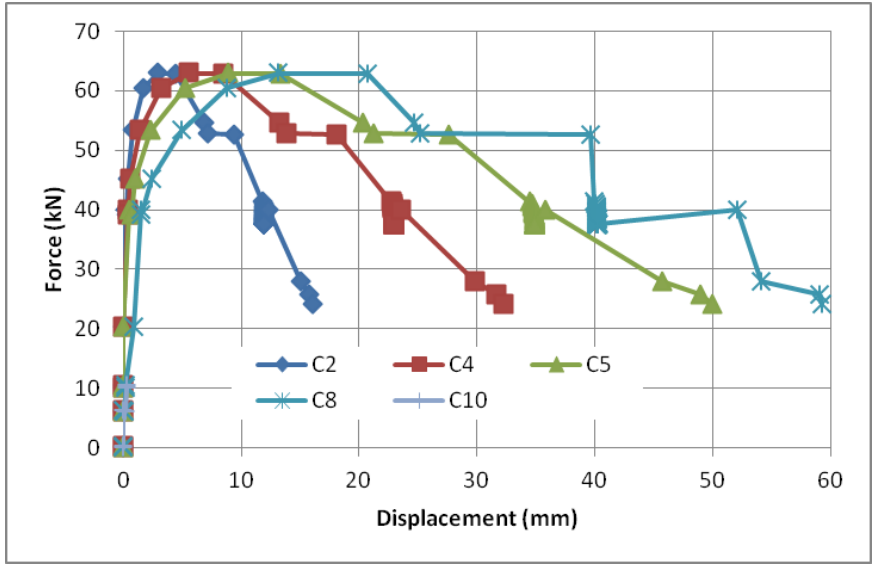


Figure 8. Force against sensors C2, C4, C5, C8 and C10 displacements during stage 0.5I.

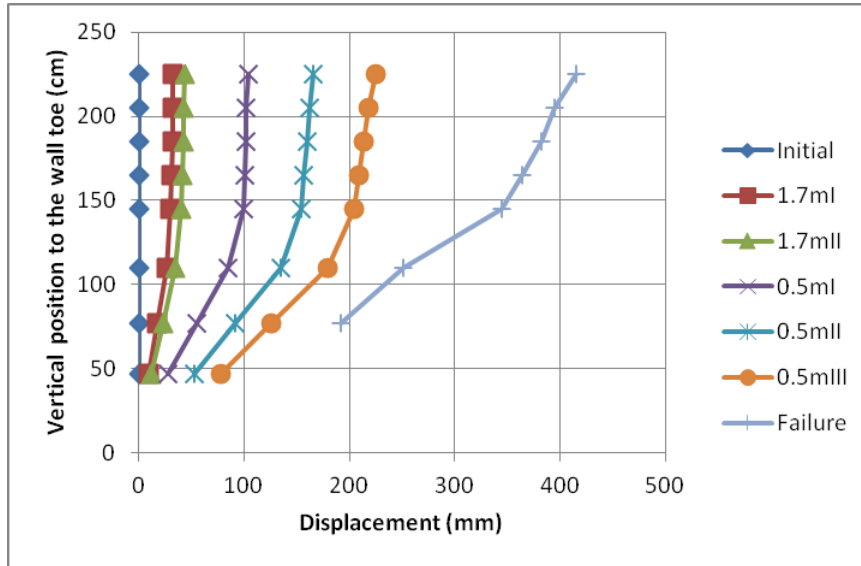


Figure 9. Evolution of the vertical axis of symmetry of the wall facing during the tests: results from sensors C2, C4, C5, C7, C8, C9, C10, C11.

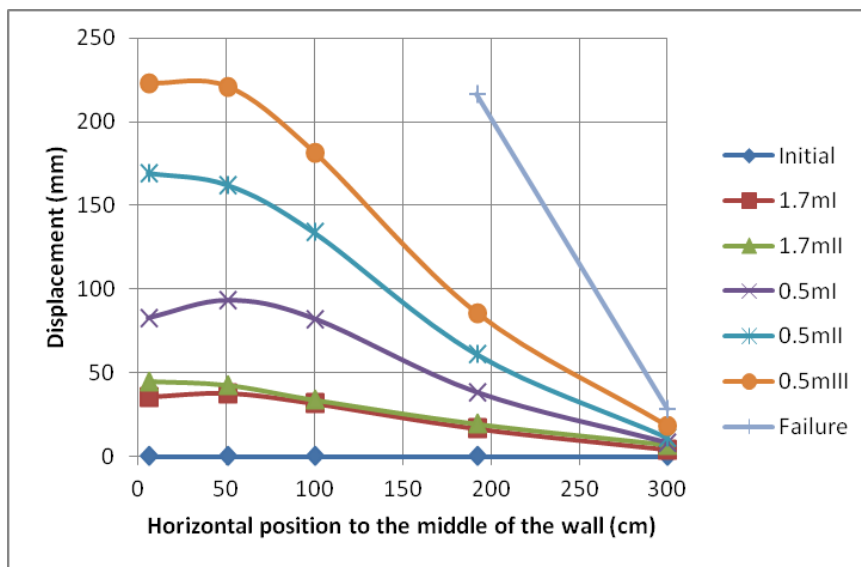


Figure 10. Evolution of a horizontal line of the wall facing during the tests: results from sensors C11, L4, L9, L13, L17



Figure 11. Deformation of the wall after stage 0.5III: top view.



Figure 12. Test wall 2 after collapse.

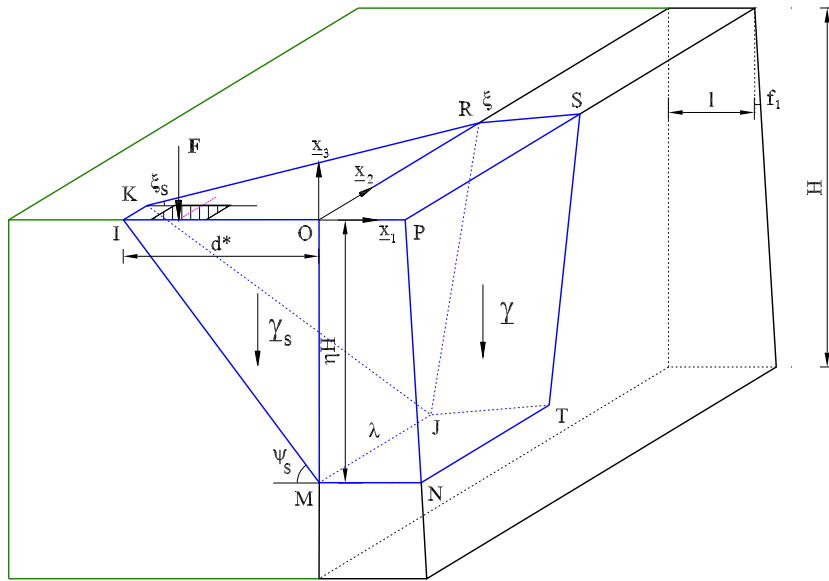


Figure 13. Half-system cross-section definition for 3D yield design analysis of the retaining wall by Le *et al.* (Le *et al.*, 2016).

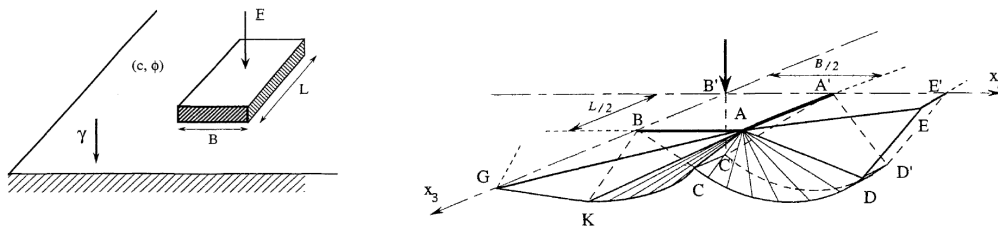


Figure 14. Test of a semi-infinite soil bearing capacities (left) and quarter-system cross-section definition for 3D yield design analysis (right).

Table 1. Experimental tests in Vaucluse: geometrical and physical characteristics, and experimental results.

	Wall 1	Wall 2
Type of stone	Limestone	Limestone
Wall height (m)	2.3	2.3
Backfill height(m)	2.3	2.3
Wall top thickness (m)	0.55	0.52
Wall length (m)	10.55	10.55
Wall batter (%)	6	6
Wall unit weight (kN/m ³)	20.3	20.3
Backfill unit weight (kN/m ³)	14.9	15.4
Block friction angle (°)	36	36
Backfill friction angle (°)	37.7	37.7
Distance load/wall (m)	0.5	0.5
Experimental load (kN)	55	63

Table 2. Testing protocol on wall 2.

Stage	Distance (m)	Maximum (kN)	Comments
1.7mI	1.70	90	Unloading, backfill levelling
1.7mII	1.70	117(143)*	Unloading
0.5mI	0.50	63	Pause
0.5mII	0.50	43	Pause
0.5mIII	0.50	37	Unloading, backfill levelling
Failure	0.50	35	Loading until failure

*117 kN: ultimate loading causing large displacements;
143 kN: maximum loading measured by the force sensor.

Table 3. Experimental tests at the University of Bath: geometrical and physical characteristics, and experimental results.

	Wall 1	Wall 2	Wall 3	Wall 4	Wall 5
Type of stone	limestone	limestone	limestone	limestone	slate
Wall height (m)	2.5	2.5	2.5	2.5	2.5
Backfill height(m)	2.2	2.2	2.2	2.2	2.2
Wall top thickness (m)	0.4	0.3	0.4	0.4	0.4
Wall batter (%)	8	8	8	12	11
Wall unit weight (kN/m ³)	19.2	19.9	16.8	17	19.7
Backfill unit weight (kN/m ³)	16.1	13.7	13.7	13.7	13.7
Block friction angle (°)	37.4	37.4	37.4	37.4	17.5
Backfill friction angle (°)	51	39	39	39	39
Distance load/wall (m)	1	1	1	1	1
Experimental load (kN)	110	75	80	85	60

Table 4. Comparison between experimental and theoretical reduction coefficient of the backfill soil bearing capacities, defined as the ratio between the ultimate bearing capacities of the retaining system F_{system}^+ and those of the soil F_{soil}^+ .

	Experiment	Yield design
Reduction coefficient r	0.57	0.56