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1 Static and seismic design of Dry Stone Retaining Walls (DSRWs) following Eurocode
2 standards

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14 **Abstract**

15 Dry Stone Retaining Walls are structures made of rubble stones assembled without mortar and have been
16 present worldwide for centuries. Today, they still constitute an attractive alternative to building techniques
17 involving higher embodied energy, such as reinforced concrete walls. This study uses a pseudo-static approach
18 to give design recommendations to maintain this built heritage and allow its modern construction. Both non-
19 seismic (Eurocode 7) and seismic (Eurocode 8) cases are addressed. The present work confirms that a seismic
20 design is not critical and is therefore not required for zones with a design acceleration below 0.05g. In addition,
21 this work highlights the significant positive effect of the stone bed inclination and the internal wall face batter.
22 Finally, depending on the wall site conditions and the seismic zone associated with the project, general design
23 recommendations are given to optimise the volume of stones used, which are illustrated in the case of France.
24 These recommendations based on pseudo-static analyses are already usable in practice for low to moderate
25 seismic areas as the required retaining wall dimensions can be easily implemented on-site. In addition, it is also
26 shown that the actual French recommendations for these walls fully comply with Eurocode 7.

27 **Keywords**

28 Masonry, Dry stone, Retaining walls, Pseudo-static, Earthquakes, Coulomb's wedge, Standards

29 [Introduction](#)

30 Dry stone structures have been built in most regions of the world, sometimes shaping typical and valuable
31 landscapes. These vernacular structures are made of rubble stones carefully assembled by hand and without
32 mortar. Dry Stone Retaining Walls (DSRWs) are likely to constitute the most representative part of this built
33 heritage, allowing agricultural activities on terraces and traffic on rural roads in mountainous or sloped areas.
34 Therefore, DSRWs play an essential economic role in these regions that benefit less from globalisation and
35 major investments. In addition, they also hold a high cultural value, sometimes labelled by UNESCO (e.g.,
36 Douro's Valley in Portugal or the Lavaux's Terraces in Switzerland). In fact, the art of dry stone walling,
37 knowledge and techniques were designated as Intangible Cultural Heritage of Humanity by UNESCO in 2018.
38 However, these structures have often faced a lack of maintenance in recent decades and require urgent repair.

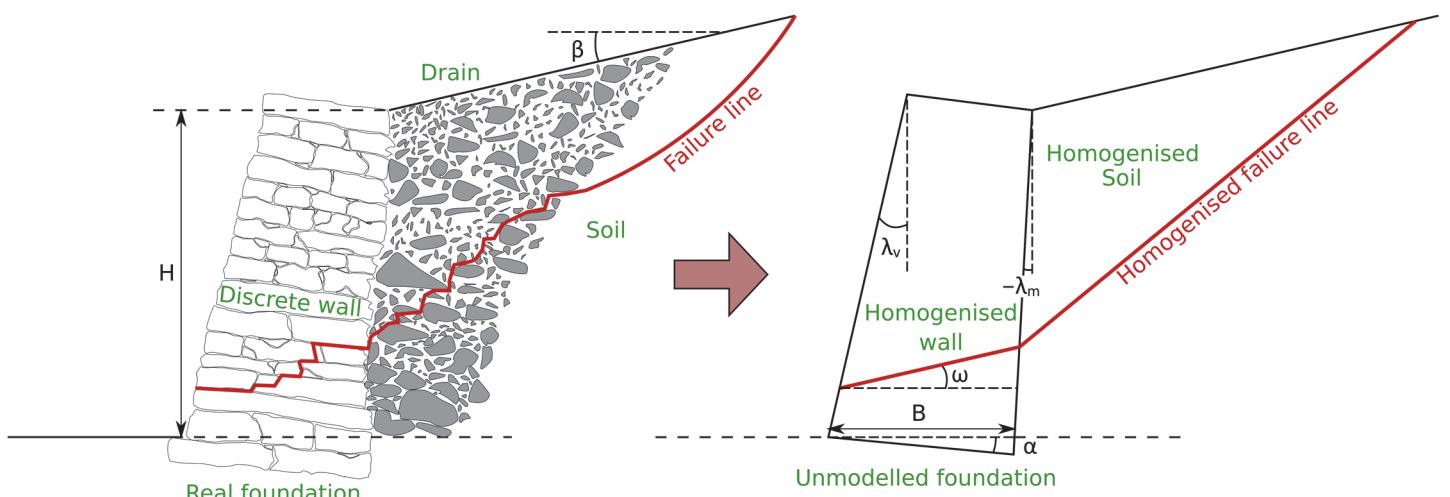
39 Given the need to preserve and repair old DSRWs, several research studies have been conducted mainly in
40 Europe. Experimental works [1]–[3], analytical [4]–[7] and numerical studies [8]–[15] focused on the static
41 mechanical behaviour of 2D sloped DSRWs, while other studies investigated the 3D mechanical behaviour of
42 these walls in case of a concentrated traffic load [16]–[19]. In France, these researches led to two practical
43 handbooks that include design rules for DSRWs retaining slopes. These are valid for any country with similar
44 building techniques, which can be found worldwide [20], [21]. However, even though the recommendations
45 are used in practice and recognised by the drystone masonry and civil engineering communities, they do not
46 consider seismic action. Only a few study cases have been investigated according to the past French seismic
47 recommendations [22], [23]. Moreover, the validation of the recommendations according to Eurocode 7 [24]–
48 [26] has not been investigated exhaustively, even if partly considered in the latest DSRWs French handbook
49 [21].

50 To address the seismic design of DSRWs in slopes, the authors developed a pseudo-static analytical tool based
51 on Coulomb's wedge theory, which was validated by pseudo-static scaled-down laboratory experiments [27].
52 The first section of this paper revised the analytical method, while the second section provides a comparison

53 with the current standards on geotechnical engineering and seismic engineering, respectively, Eurocode 7 & 8
 54 [24]–[26], [28], [29]. Finally, following the Eurocodes, recommendations are given for designing DSRWs in areas
 55 ranging from very low to high seismicity.

56 Analytical method

57 The analytical method relies on the limit-equilibrium theory under plane strain conditions (Figure 1). The DRSW
 58 is characterised by a height H , a base width B , an external slope to the vertical of λ_v and an internal slope to the
 59 vertical of λ_m . The bed inclination of the wall is referred to as α and the backfill slope as β . Contrary to the static
 60 limit-equilibrium approach of Villemus [30], the analytical method includes seismic forces modelled as
 61 equivalent horizontal pseudo-static actions. Briefly, the pseudo-static equilibrium of a Coulomb's wedge of soil
 62 is first computed to obtain the pseudo-static active earth pressure [31]–[33]. The wall's equilibrium is then
 63 computed, stating the possible types of failure: an internal sliding or toppling mode [34].



64
 65 Figure 1: DSRW with its geometric parameterisation a): actual backfill-wall system; b): modelled backfill-wall system. The present sign convention
 66 makes the internal batter (λ_m) negative.

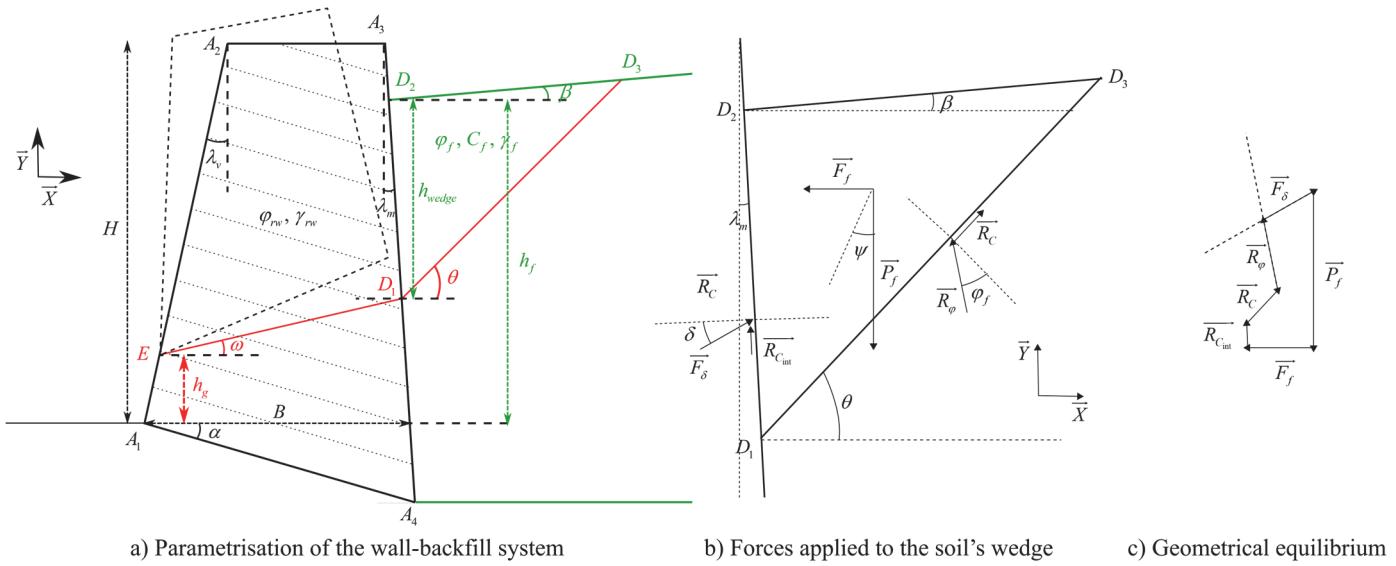
67 Figure 2 describes the backfill-wall system, with all geometrical and mechanical parameters clarified. The wall
 68 is characterised by a homogeneous medium (Figure 1) with its homogeneous unit weight γ_{rw} (accounting for
 69 voids between stones) and joint frictional angle φ_{rw} . Similarly, the backfill is modelled as a homogeneous
 70 medium described by its unit weight γ_f , cohesion C_f and friction angle φ_f . Compared to other retaining

71 structures, the particularity of DSRWs lies in a failure line developing through the dry joints, which is modelled
72 as an equivalent straight failure line with an inclination ω (Figure 1). In practice, the inclination of this failure
73 line from the bed joints is limited by a maximum value of 20° , see [27]. This inclination is also different from
74 the homogenised inclination θ of the failure line crossing the backfill, which mainly depends on material (soil
75 friction and cohesion), geometrical (slope of the backfill) and seismic (pseudo-static accelerations) parameters,
76 as explained below.

77 The mechanical system has three unknowns (θ , ω and h_g) that should be determined to compute the earth
78 pressure F_δ . According to the Coulomb soil's wedge theory, for each combination of these parameters, the limit
79 equilibrium of the soil's wedge $D_1D_2D_3$ can be calculated (Figure 2b-c) to evaluate the active earth pressure F_δ .
80 In particular, the weight P_f of the soil is proportional to the wedge area (triangle $D_1D_2D_3$) and is applied at the
81 gravity centre of the triangle $D_1D_2D_3$. Similarly, the pseudo-static action (inertial force due to the seismic
82 motion) F_f is also proportional to the wedge area and applied at its gravity centre. The backfill frictional reaction
83 R_φ application point and intensity are unknown, but its orientation is given by the backfill friction angle φ_f since
84 the limit-equilibrium is assumed. Similarly, the orientation, yet not the application point, of the backfill cohesive
85 reaction R_c is known. Regarding the backfill-wall interface, the interface cohesive reaction R_{Cint} has an unknown
86 point of application and a known orientation. The intensities of the cohesive forces (R_c and R_{Cint}) are
87 proportional to the cohesive strength and the length of the interface (D_1D_3 and D_1D_2). Hereafter, the interface
88 cohesive strength (R_{Cint}) always equals zero, as the drain directly behind a DSRW is usually made of dry
89 cohesiveless gravel.

90 Finally, the earth pressure F_δ has a known orientation δ (internal face of the wall) and application point but
91 unknown intensity. The intensity is deduced from the mechanical equilibrium of the soil's wedge (Figure 2c). In
92 the absence of pseudo-static action F_f and cohesive resistance, the application point of the earth pressure is
93 located at one-third of the height of the retaining structure. Then, adding cohesive effects decrease the

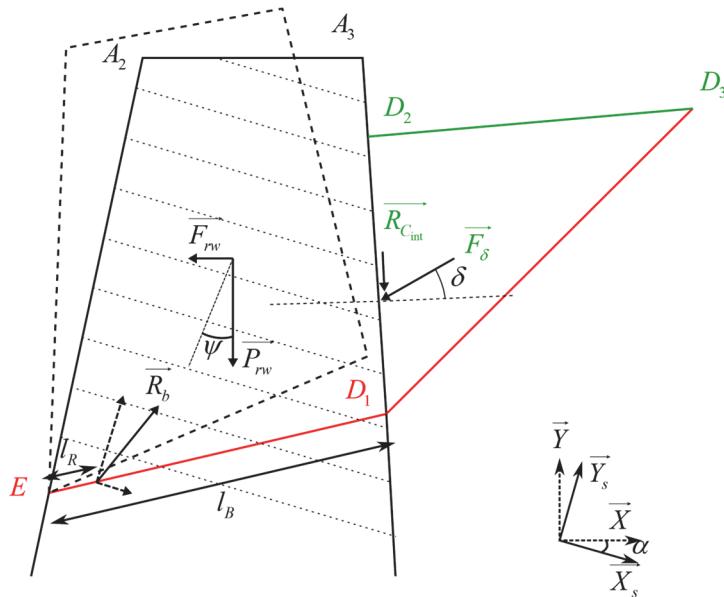
94 application point height while adding a pseudo-static action increases its height. The reader can refer to the
 95 literature for deeper insights into the location of the application point of earth pressure in this case [34]–[36].
 96 The analytical method also accounts for the tensile cracks that classically occur at the top of cohesive backfill
 97 and reduce the cohesive forces (R_c and R_{Cint}). In addition, in the present work, the presence of dead loads on
 98 top of the backfill and saturated retained backfill [34] can be accounted for, yet not described here for brevity.



99 **Figure 2: Parametrisation of the mechanical system and geometrical equilibrium of the soil's wedge in order to compute the earth pressure F_δ .** [34]

100 In a second stage of the calculation, the equilibrium of the wall itself is computed (Figure 3) including its own
 101 weight and the pseudo-static horizontal action, applied at the centre of gravity of the studied portion of wall
 102 $A_2A_3D_1E$, corresponding to the part above the failure line D_1E . The interface actions F_δ and R_{Cint} (as well as their
 103 line of actions) are derived from the previous stage of calculation. Subsequently, the wall equilibrium is
 104 computed in the X and Y directions to evaluate the base reaction R_b . The equilibrium in terms of momentum
 105 gives the application point of R_b . Finally, the stability of wall portion $A_2A_3D_1E$ is computed considering a toppling
 106 mode of failure (e.g. application point inside the wall, i.e. $l_b > 0$, or any other criteria defined in the following
 107 sections) or a sliding mode of failure. For this last point, the base reaction R_b is projected on the plane defined
 108 by the orientation of the bed joints (axes X_s and Y_s in Figure 3), and then Mohr-Coulomb criterion is checked.

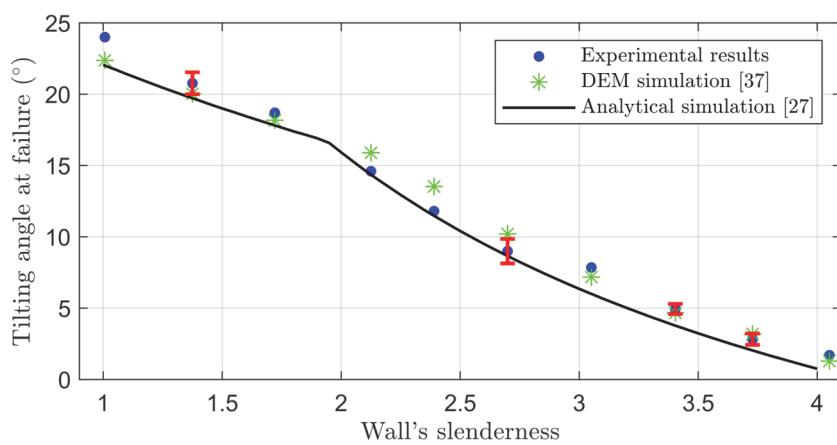
110 Note that the bed joints' orientation is updated because of the possible internal rotation of stones inside DSRWs
 111 (see [34] for more details). Finally, the unknowns of the system (θ , ω and h_g) are optimised for each failure
 112 mode to find the most critical situation for the criterion checked. Several iterations involving the base width B
 113 of the wall allows to identify the minimum B value that barely satisfies the stability criteria.



114

115 *Figure 3: Equilibrium of the dry stone retaining wall [34]*

116 The method has been validated on scaled pseudo-static experiments on a tilting table, using dry joint brick
 117 retaining walls retaining a sandy backfill [27]. Figure 4 gives the results, showing that the developed pseudo-
 118 static approach is as precise as more sophisticated Discrete Element Modelling (DEM) simulations [37].



119

120 *Figure 4: Comparison of DEM simulation, analytical simulation and experimental tilting tests [37]*

121 As additional validation, the analytical method was used to model two sets of experimental campaigns carried
 122 out on full-scale DSRWs with 1) a hydrostatic load [1]; 2) a dry backfill load, as displayed in Figure 5 [2], [3].
 123 Table 1 describes the geometric and mechanical parameters of the experiments, along with the analytical
 124 results, which are in excellent agreement for both campaigns. Moreover, the developed analytical approach
 125 provides a similar level of accuracy to the results of Villemus *et al.* [1] and Colas *et al.* [6], [38].



126

Figure 5: Experimental toppling failure obtained by Colas [39]

127
 128 Table 1: Parameters of the full-scale DSRWs experimental campaigns from [1]–[3]. Experimental and analytical results are also given.

Name*	V1I	V2I	V3I	V4I	V5s	C1g†	C2s	C3s	C4I
Geometrical parameters									
Height H (m)	2	1.95	4	2	4.25	2.5	2.5	2.5	2.5
Base width B (m)	0.9	0.91	1.8	0.9	1.8	0.6	0.6	0.7	0.65
External batter λ_v (%)	15	0	15	12	15	6	6	6	6
Beds inclination α (°)	0	0	0	4	8.5	3.4	3.4	9.1	9.1
Backfill slope β (°)	NA	NA	NA	NA	NA	26.4	31.7	32.6	34.9
Mechanical parameters									
Wall weight (kN/m ³)	15.4	14.9	15.7	15.7	18.0	21.0	20.0	20.0	21.8
Stone friction (°)	36	36	36	36	28.5	27	25	25	35
Internal rotation (°) ‡	5	5	5	5	5	5	5	5	5
Soil weight γ_r (kN/m ³)	NA	NA	NA	NA	NA	14.9	14.9	14.9	14.9
Soil friction φ_r (°)	NA	NA	NA	NA	NA	37.7	37.7	37.7	37.7
Experimental results									
Critical height (m)	1.74	1.9	3.37	1.94	3.62	>2.1 7	2.41	2.96	2.95
Failure mode (S/T)	S	S/T	S	S/T	S	NA	S/T	T	T
Analytical results									
Critical height (m)	1.73	1.84	3.48	1.86	3.68	2.86	2.69	3.02	2.82
Failure mode (S/T) §	S	S/T	S	S/T	S	T	S/T	T	T

Difference to exp.	-1%	-3%	+3%	-4%	+2%	NA	+11 %	+2%	-4%
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* V refers to Villemus [1] with hydrostatic loading and C to Colas [2], [3] with backfill loading, while / refers to limestone blocks, s to schist blocks and g to granite blocks.

† This experiment failed. However, the wall resisted at least a loading corresponding to a backfill height of 2.17m.

‡ This information is based on experimental results, using a default value. Details can be found in [1]–[3] for the experiments and in [34], [40] for the analytical method.

§ A combined sliding-overturning failure has been defined if the critical theoretical heights of the two failure modes were within a range of ±5%.

129 Design of DSRWs following Eurocodes

130 The present section aims at designing DSRWs using the analytical method and the partial safety factors (actions,
 131 material properties and resistance) from Eurocode 7 and 8 [24]–[26], [28]. First, it is emphasised that in a
 132 seismic context, a pseudo-static method (like the one presented above) will never accurately predict the true
 133 time evolution of the dynamic response or resistance of a real DSRW during an earthquake. Therefore, it is only
 134 used as a simplified design method proposed by Eurocode 8 to give fast seismic assessment of retaining walls.
 135 No partial safety factor related to the method is considered since no constant bias has been found in the
 136 validation processes between theoretical and experimental results. Moreover, only the wall's internal sliding
 137 and toppling failures are considered: the bearing capacity of the foundation soil at ultimate or serviceability
 138 limit states are assumed not to be reached. Similarly, the passive soil is considered infinitely rigid, which is
 139 reasonable according to Alejano *et al.* [5]. Finally, liquefaction and the failure of the entire soil slope are
 140 disregarded. Table 2 presents the safety factors used for the computations as well as those from the French
 141 professional rules, which are similar [21].

142 Regarding non-seismic verifications, the Ultimate Limit State Equilibrium (ULS EQU), the
 143 Structural/Geotechnical Ultimate Limit State with the second approach (ULS STR/GEO), see [41], and the
 144 Serviceability Limit State (SLS) are examined. The Ultimate Limit State (ULS SEISM) is applied for the seismic
 145 verification.

146 *Table 2: Partial safety factors for non-seismic and seismic safety verifications; Annex A of the Eurocode 7 [24] and ENTPE (Eds.) et al. [21].*

	Eurocode 7 [24]				French professional rules [21]	
	ULS			SLS		
	EQU	STR/GEO	SEISM			
Safety factors for actions						
Favourable weight actions factor ($\gamma_{G, \text{fav}}$)	0.9	1	1	1	1	
Unfavourable weight actions factor ($\gamma_{G, \text{unfav}}$)	1.1	1.35	1	1	1.35	
Safety factors for material properties						
Drained soil friction angle factor (γ_ϕ)	1.25	1	1.25	1	1	
Drained soil cohesion factor (γ_c)	1.25	1	1.25	1	1.25	
Safety factors for resistances						
Sliding factor ($\gamma_{R, h}$)	1	1.1	1	NA	1	
Toppling factor ($\gamma_{R, v}$)	1	NA	1	NA	1	
Eccentricity factor ($1 - 2e/B$)	NA	1/15	NA	1/2	1	
Model resistant factor ($\gamma_{R, d}$)	1	1	1	1	1.2	

147

148 For the toppling verification of the SLS and the ULS STR/GEO, the design criterion to satisfy corresponds to the
 149 maximum eccentricity (noted e) of the transmitted load through the wall, as stated in Eurocode 7 [25], [26]. In
 150 the context of the ULS EQU and SEISM verifications, the partial safety factors γ_m (γ_ϕ' and γ_c') for the materials
 151 are only applied to the backfill soil properties and not to the friction between blocks (as mentioned in [41]–
 152 [43]). The friction between blocks is linked to the wall resistance, thus to the resistance safety factor γ_R . As a
 153 consequence, applying a safety factor for the block material (block-block friction) would penalise twice the
 154 same parameters, which is not in agreement with the framework of Eurocodes. Moreover, in the seismic
 155 verification (ULS SEISM), the safety factors for materials are linked to the degradation of the shear strength of
 156 soils at high strain and/or in the presence of pore pressures. These are unlikely to occur for dry block-block
 157 joints [42]–[44].

158 *Non-seismic case*

159 Computations discarding the seismic action are carried out on five walls, whose main characteristics are
 160 presented in Table 3. The walls are built of stones with geological natures representative of European and, in
 161 particular French, geology (molasse or marl sandstone, schist, limestone and granite). They include typical

162 variations of essential DSRW properties (geometrical shape and soil resistance). Note that their height is typical
 163 of relatively high DSRWs. The backfill-wall interface friction angle δ is equal to the soil friction angle, given the
 164 dry gravel drain placed behind each DSRW [2], [3]. Both the internal batter λ_m and the stone bed inclination α
 165 are initially equal to zero. The minimum required (bottom) widths for the walls given by the Eurocodes are then
 166 computed and compared to the recommended widths from ENTPE (Eds) *et al.* [21].

167 *Table 3: Main characteristics and required widths of five typical DSRWs according to Eurocode 7 (non-seismic case) criteria; maximum values are*
 168 *given in bold. Comparisons with the recommendations of ENTPE (Eds) *et al.* [21] are also given. Each column heading displays the critical failure mode*
 169 *(T: toppling; S sliding). Granite DSRWs have similar properties as schist walls, and the values shown for schist can be used.*

	Wall 1 (T) Molasse	Wall 2 (S) Schist	Wall 3 (T) Limestone	Wall 4 (S) Schist	Wall 5 (T) Limestone
Mechanical & geometric properties					
Height H (m)	2.5	2.5	2.5	2.5	2.5
External batter λ_v (%)	0	10	20	10	0
Backfill slope β^* ($^{\circ}$)	0	0	0	0	10
Wall unit weight γ_{rw} (kN/m ³)	16	20	20	20	20
Blocks friction ϕ_{rw} ($^{\circ}$)	36	28	36	28	36
Backfill unit weight γ_f (kN/m ³)	20	20	20	20	20
Backfill cohesion C_f (kPa)	0	0	0	5	0
Backfill friction ϕ_f ($^{\circ}$)	30	30	25	25	30
Interface friction δ ($^{\circ}$)	30	30	25	25	30
Non-seismic design: wall base widths					
ULS EQU (Sliding)	0.57m	0.83m	0.85m	0.68m	0.54m
ULS EQU (Toppling)	0.83m	0.77m	0.90m	0.51m	0.81m
ULS STR-GEO (Sliding)	0.51m	0.77m	0.82m	0.63m	0.47m
ULS STR-GEO (Toppling)	0.76m	0.71m	0.84m	0.30m	0.74m
SLS (Eccentricity)	0.89m	0.78m	0.91m	0.30m	0.86m
ENTPE (Eds) <i>et al.</i> [21]	0.91m	0.85m	1.00m	0.75m	0.88m
Absolute difference	2%	2%	9%	9%	2%

170

171 * The slope inclination refers to the backfill directly retained by the wall and not the global slope of the site.
 172 The recommendations from ENTPE (Eds) *et al.* [21] are very close to the maximum value recommended by the
 173 Eurocode standards with slightly larger and conservative values: on average +5% and maximum +10%
 174 difference. The most critical limit state and the corresponding failure mode differ according to the studied walls,
 175 justifying the consideration of all possible limit states. Finally, even if only five case studies are shown for
 176 brevity, the authors carried out a more comprehensive set of typical walls with identical conclusions.

177 Seismic case
178 *Global analysis*
179 The same walls are assessed according to Eurocode 8 [29], [44], assuming their design is provided by the most
180 non-seismic critical case mentioned above. Keeping all other geometrical parameters constant, the extra-width
181 required to withstand the seismic loading is computed (see Table 2 for the associated partial safety factors).
182 Here, the authors recall that this section does not aim to describe precisely the dynamic behaviour of DSRWs
183 against earthquakes but aims to provide a safe design width for DSRWs built in a seismic context. For this
184 reason, the simplified pseudo-static modelling strategy is considered. The following equations give the pseudo-
185 static design accelerations (horizontal a_h and vertical a_v):

$$(1) \quad a_h = \frac{a_{gR} * S * S_T * \gamma_I}{r}, \quad a_v = 0.5 * a_h$$

186 where a_{gR} is the reference acceleration for the considered seismic zone; S is the soil amplification factor (the
187 maximum value being $S = 1.8$); S_T is the topographic amplification coefficient; γ_I is the importance coefficient
188 of the structure; and r is the seismic behaviour factor [29]. The latter parameter considers the wall's ability to
189 move during the seismic motion and dissipate seismic energy before collapsing [28], [44].

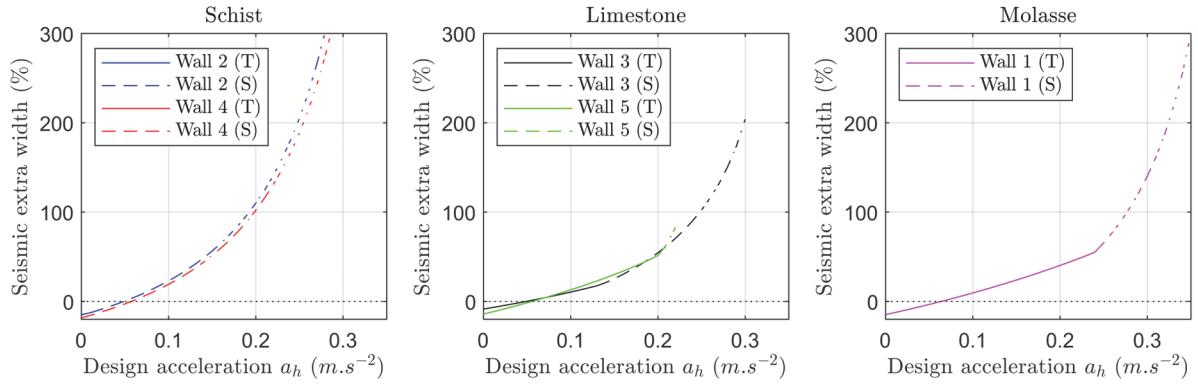
190 Scaled-down experiments on a shaking table suggested a conservative value of $r = 1.5$ [45], corresponding to
191 walls able to handle small movements before collapsing according to Eurocode 8 [28]. In addition, numerical
192 simulations using DEM approach have concluded that $r = 1.5$ was indeed conservative enough for the French
193 seismicity [46]. In fact, after a numerical validation step using scaled-down experiments with dynamic time-
194 history analyses, the numerical model was applied to full-scale structures with various dynamic time-history
195 analyses (signal and backfill-wall parameters). Then, comparing the obtained numerical resistance and the
196 estimated (analytical) pseudo-static resistance, the seismic behaviour factor r for each configuration was
197 determined. Combining all the results, a mean value of $r = 2.0$ and a minimum value of $r = 1.8$ have been found.
198 Therefore, a conservative recommended value of $r = 1.5$ was proposed since Eurocode only advices three values
199 for r namely 1, 1.5 and 2. Even though this study has focused on the French case with moderate seismicity, the

200 obtained outcome is considered acceptable elsewhere in Europe, especially given the safety margin taken at
201 each step in [46]. In all analyses, the studied DSRWs are assumed to belong to the normal importance class.
202 Particular conditions that rarely occur on-site are excluded (e.g., walls near very high buildings, highways,
203 hospitals or energy facilities). It means that the importance coefficient γ_i is lower than (or equal to) 1, which
204 also implies no topographic amplification ($S_T = 1$). Depending on the reference acceleration a_{gR} , the most critical
205 design acceleration reads:

$$(2) \quad a_{h,max} = \frac{a_{gR} * 1.8}{1.5} = a_{gR} * 1.2, \quad a_{v,max} = 0.5 * a_{h,max} = a_{gR} * 0.6$$

206

207 Figure 6 depicts the extra-width required to fulfil a seismic design compared to the previously non-seismic
208 design for each of the five walls (Table 3). Eurocode 8 states that seismic design is not required below a
209 threshold of $a_h = 0.05g$ [29]: actually, the current calculations show that even if accounted for, the seismic case
210 is not critical for the wall stability (Figure 6). For higher design accelerations, walls sensitive to a sliding failure
211 (case of schist wall, Figure 6a) tend to have larger required extra-widths than those sensitive to a toppling
212 failure (Figure 6b-c). Finally, for high seismic hazard regions in Europe and France, see Figure 7 ($a_h = 0.25 - 0.4g$),
213 extra-widths reach very high values (+200%), reflecting the inaccuracy of the pseudo-static method for high
214 reference accelerations, which provides unpractical recommendations for on-site works. Here, it is noted that
215 wall disintegration is not considered, meaning that in high seismicity areas, additional prevention actions may
216 be required.



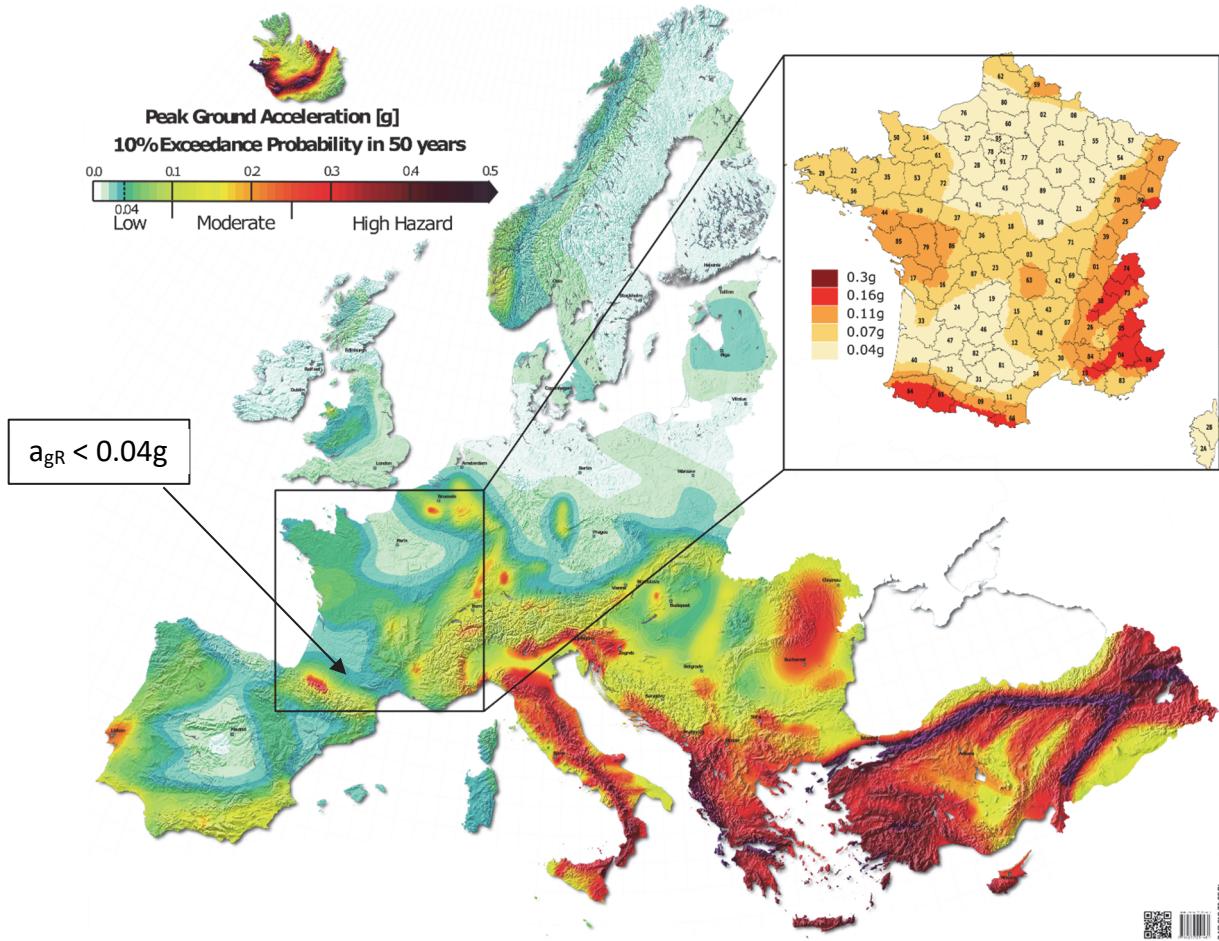
217

218 *Figure 6: Extra-width required by a seismic design compared to a non-seismic design for the studied walls. Walls are presented according to the type*
 219 *of stone used; a) schist; b) limestone; c) molasse (marl sandstone).*

220 A larger set of walls was analysed to obtain more general results. Walls geometries are still given in Table 3, but
 221 each geometry uses three different types of stone (schist, limestone, and molasse). Additionally, the analysis
 222 considers three typical bed inclinations α (0° , $10\% = 5.7^\circ$ and $20\% = 11.3^\circ$) for each wall to improve the sliding
 223 resistance of DSRWs [2], [30], [47], [48]. Figure 8 gives the maximum (and mean) required extra-widths for
 224 DSRWs built with different kinds of stone depending on the stone bed inclination α . Here, the local peaks are
 225 related to vertical asymptotes for a peculiar response: when the design acceleration a_h equals $a_{h,critical}$ (Eq. 3),
 226 the whole retained soil slope loses its static equilibrium leading to infinite forces impossible to be sustained by
 227 the wall. Hence, the pseudo-static design requires huge wall widths close to this critical acceleration.

(3)

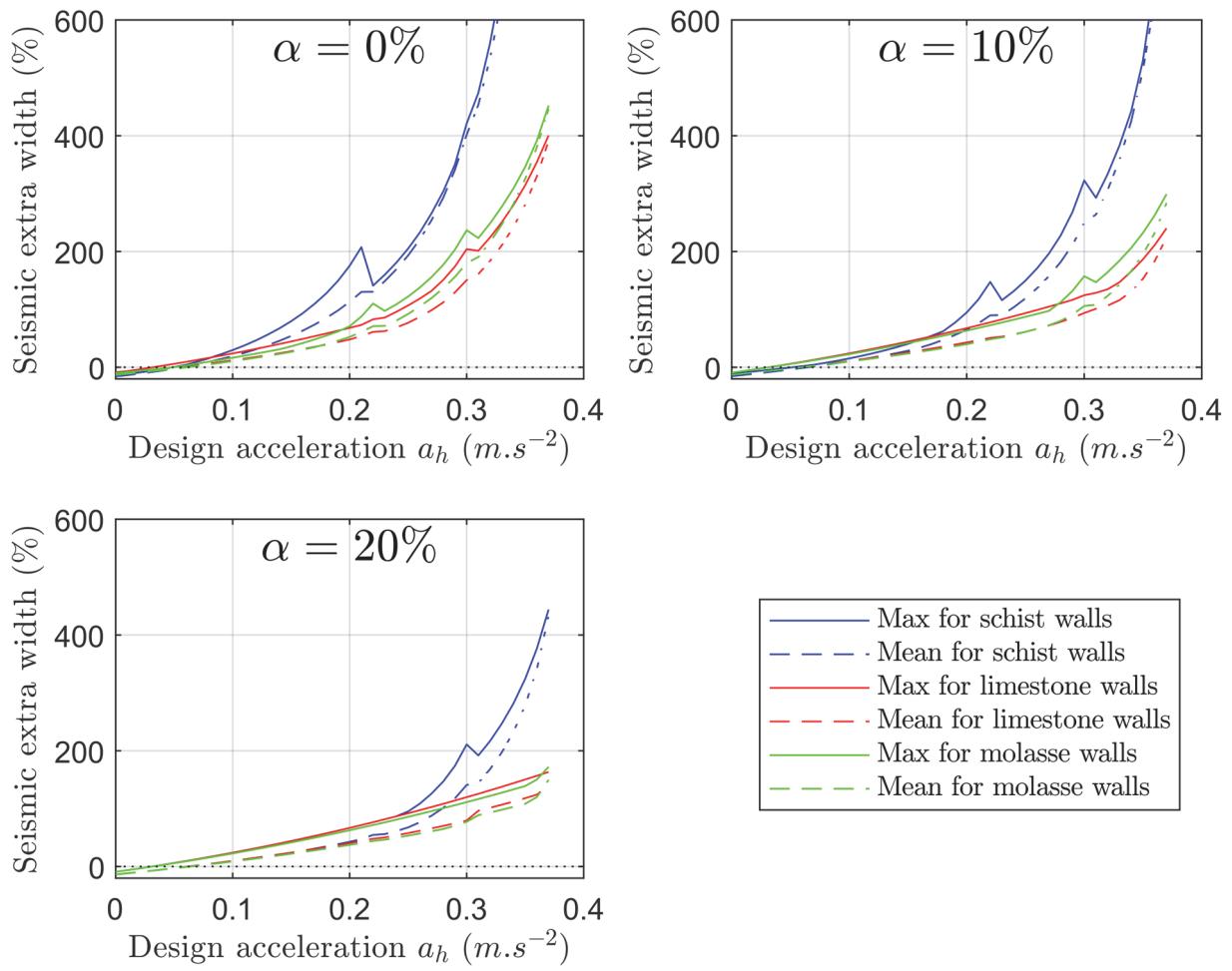
$$a_{h,critical} = \frac{\sin(\varphi_r - \beta) + \frac{c_r \cos \varphi}{\gamma_r * H \cos \beta}}{\cos(\varphi_r - \beta) + 0.5 * \sin(\varphi_r - \beta)}$$



228

229 *Figure 7: European map of reference acceleration a_{gR} (return period of 475 years) according to Giardini et al. [49]. The threshold $a_{gR} = 0.04g$ is also*
 230 *highlighted in the legend. National seismic zonation maps in Eurocode 8 currently differ from this map.*

231 For clarity, the maximum and mean values displayed in Figure 8 ignore walls around and after their critical
 232 acceleration $a_{h, \text{critical}}$, yet still resulting in small visible local peaks. For example, the first peaks for a design
 233 acceleration of about 0.22g observed in Figure 8 correspond to Wall 5, the only case with a non-zero backfill
 234 slope θ . Therefore, in practice, one should prefer to build a higher wall with a flat retained backfill than a smaller
 235 wall with an inclined backfill, especially in high seismic hazard regions. Finally, Eq. 3 highlights the inherent
 236 limitation of the design approach proposed by Eurocodes, which does not apply for high design acceleration a_h ,
 237 basically larger than 0.25-0.4g depending on the parameters of the backfill.



238

239 *Figure 8: Maximum (continuous line) and mean (dashed line) required extra-widths for the studied DSRWs depending on the type of stone and the*
 240 *stone beds inclination α : a) $\alpha=0\%$; b) $\alpha=10\%$; c) $\alpha=20\%$.*

241 Analysing Figure 8, it is noted that increasing the stone bed inclination is very efficient in reducing the extra-
 242 widths according to a pseudo-static design, although this parameter was not integrated into the non-seismic
 243 French recommendations [20], [21]. Table 4 details the maximum required extra-widths for each bed inclination
 244 according to different reference accelerations (a_{gR}) representative of Europe. A non-exhaustive description of
 245 the concerned regions is also given for each reference acceleration. For most western, northern, and eastern
 246 Europe, where the reference acceleration is smaller than $a_{gR} = 0.11g$ (low to medium seismicity), the maximum
 247 required extra-width reaches about 50%. If the wall is more resistant to sliding (not built with schist or with a
 248 non-zero bed inclination), the maximum extra-width decreases to about 35%. Finally, for reference

249 accelerations larger than 0.25g, due to the inherent approximations of the pseudo-static approach, the
 250 maximum extra-widths exceed 100% of the non-seismic design which seems non reasonable in practice. For
 251 these cases, the pseudo-static modelling approach may also be inadequate, given the weak consideration of
 252 dynamics and the fact that wall disintegration is ignored.

253 *Table 4: Maximum extra-widths for DSRWs required by a pseudo-static seismic design compared to a non-seismic design depending on the reference*
 254 *acceleration (a_{gR}). The analysis only covers DSRWs belonging to the normal class of importance ($\gamma_I \leq 1.0$). According to the seismic hazard map of*
 255 *European countries, the second column gives the corresponding regions of each reference acceleration. The reader is referred to Figure 7 and Figure*
 256 *8.*

Reference acceleration a_{gR} (design acceleration $a_{h,max}$)	Corresponding regions in Europe	Max (and mean) extra widths		
		$\alpha = 0\%$	$\alpha = 10\%$	$\alpha = 20\%$
0.04g (0.05g)	Most Northern Europe (up to France, Germany and Poland) + Mainland Spain	6% (0%)	6% (0%)	6% (0%)
0.06g (0.07g)	Most North-Eastern Europe; first seismic zone of many countries	13% (4%)	13% (2%)	13% (2%)
0.11g (0.13g)	Most Iberia; most Northern Europe up to the north of Italy and the north of Croatia	56% (26%)	36% (19%)	36% (18%)
0.16g (0.19g)	All Northern Europe; most Eastern Europe; first seismic zones of all European countries	>100% (63%)	77% (44%)	62% (37%)
0.20g (0.24g)	All Europe except localised areas (Iceland, Portugal, Spain) and high seismic regions (Croatia-Italy-Slovenia; Bosnia-Croatia-Greece-Serbia; Bulgaria-Romania; Cyprus-Turkey)	>100% (≈100%)	>100% (72%)	87% (55%)
0.25g (0.29g)	All Cyprus, Slovenia; almost all Bosnia, Bulgaria, Croatia, Greece, Italy, Romania	>100% (>100%)	>100% (>100%)	>100% (88%)
>0.25g (>0.29g)	Localised zones in Iceland; rest of Bosnia, Bulgaria, Croatia, Greece, Italy, Romania, and Turkey	>100% (>100%)	>100% (>100%)	>100% (>100%)

257 conclusion, the systematic use of a stone bed inclination α is recommended. Whenever possible, an external
 258 batter equal to the stone bed inclination to facilitate the construction process is also suggested. For low to
 259 medium seismic regions ($a_{gR} < 0.11g$), a stone bed inclination α of 10% is suitable, whereas, for larger seismic
 260 hazards ($a_{gR} \approx 0.16-0.2g$), a value of 20% can help to reduce the required extra-widths significantly.

262 French case study

263 This section provides a more detailed case study for France, chosen as an illustrating example of low to medium
 264 seismicity European countries. The section helps to understand the trends and practical seismic design for
 265 DSRWs, depending on three different but typical cases for the backfill-wall condition. The first case (MAX)
 266 corresponds to the previous European study, i.e., large amplification for the soil ($S = 1.8$) and a standard
 267 construction (importance factor $\gamma_I = 1.0$). The second (ROCK) represents the most critical case when the DSRW
 268 is directly founded on the bedrock ($S = 1.0$), considered a reference case for foundation conditions. The last one
 269 (RURAL) corresponds to walls of less importance ($\gamma_I = 0.8$), built far from any road or building. In practice,

according to the French regulations, these specific cases (RURAL) are not subjected to seismic regulations. However, this work gives reference values, which are helpful for DSRWs stakeholders. These three configurations are analysed according to the four seismic zones of metropolitan France ($a_{gR} = 0.04, 0.07, 0.11$ and $0.16g$), leading to a total of 12 case studies (Table 5).

*Table 5: Horizontal seismic design accelerations a_h for different critical cases (as a proportion of $g = 9.81m.s^{-2}$). The vertical acceleration a_v is systematically taken equal to $0.5*a_h$.*

	a_{gR}	MAX	RURAL	ROCK
Formulas used to compute a_h	-	$a_{h, max} = a_{gR} * 1.20$	$a_{h, rural} = a_{gR} * 0.96$	$a_{h, rock} = a_{gR} * 0.67$
Very Low Seismic zone (S1)	0.04g	0.05g	0.04g	0.03g
Low Seismic zone (S2)	0.07g	0.09g	0.07g	0.05g
Moderate Seismic zone (S3)	0.11g	0.13g	0.11g	0.07g
Mean Seismic zone (S4)	0.16g	0.20g	0.16g	0.11g

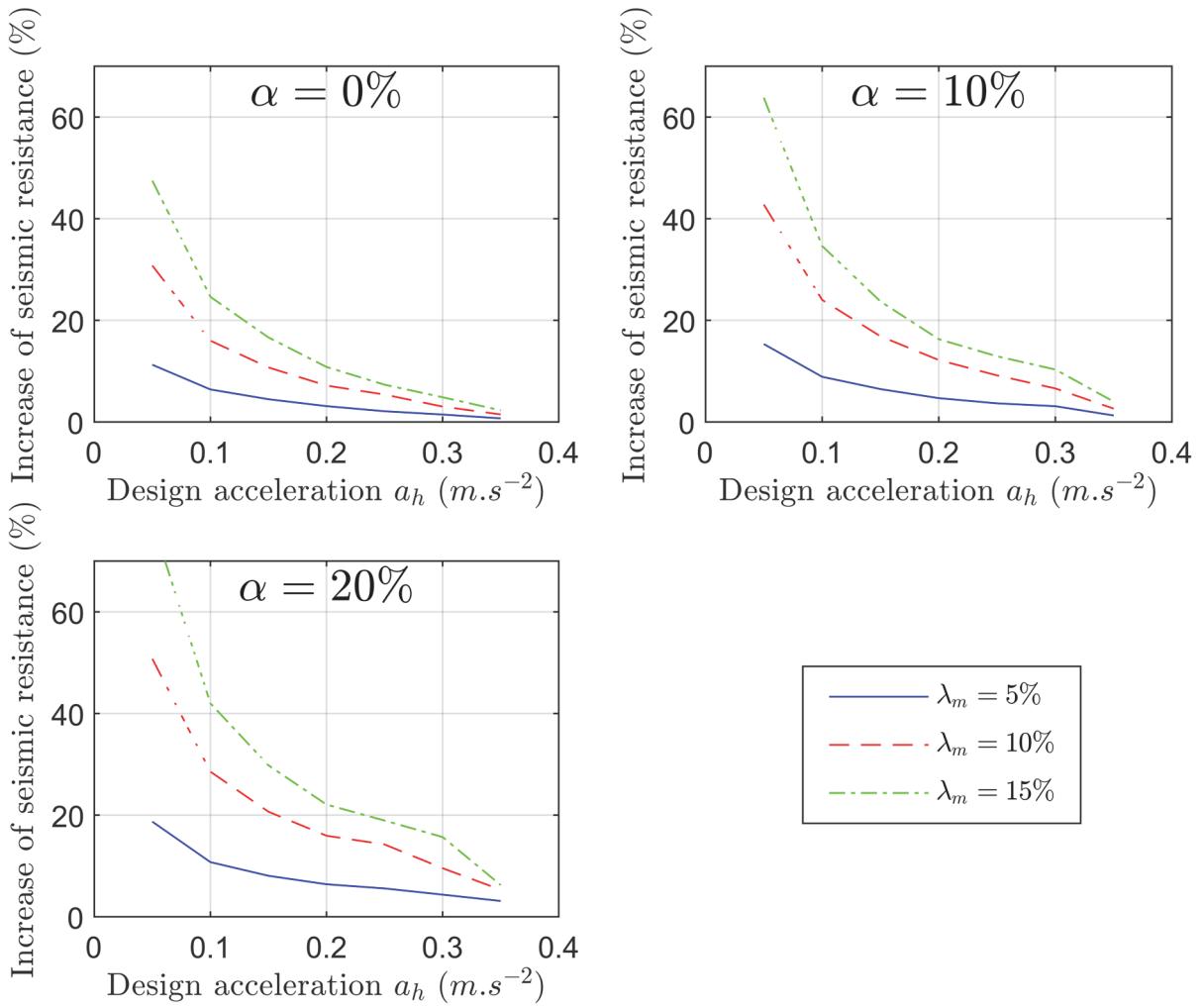
Table 6 sums up the maximum (and mean) extra-widths (among the different walls and stones type) obtained for each seismic situation depending on the stone bed inclinations. In seismic zones S1 to S3, one can again note that bed inclination α dramatically impacts the results; however, a bed inclination of 20% does not provide a significant increase in resistance compared with an inclination of 10% (see also Figure 8b-c). Therefore, as usual in the South of France, an inclination of the stone bed of 10% is recommended: seismic design requires no more than 40% extra-width (compared to a non-seismic design). In seismic zone S4, walls built with a stone bed inclination of 10% require an extra-width of 90%, which induces substantial extra costs for the wall construction. In this case, one should either use a steeper bed inclination or conduct a specific analytical computation to optimise the geometry of the DSRW. However, if the wall is built far away from roads and buildings (RURAL) and with an inclination bed of 10%, the maximum expected extra-width only reaches 50%. On the contrary, if the wall is directly founded on the bedrock (ROCK), the seismic required extra-width drops to a maximum of 30%. If both conditions are fulfilled (RURAL & ROCK), the required extra-widths do not exceed 20% (case not addressed in Table 6). Finally, the general recommendations of Table 6 (maximum values) can

290 be readily used for practical non-seismic and seismic design of DSRWs without requiring more detailed
291 computations.

292 *Table 6: Influence of the stone bed inclination α on the extra width required for seismic design. Maximum values, together with average values in*
293 *parentheses, are given.*

	$\alpha = 0\%$	$\alpha = 10\%$	$\alpha = 20\%$
ROCK-S1 (0.03g)	0% (0%)	0% (0%)	0% (0%)
RURAL-S1 (0.04g)	2% (0%)	2% (0%)	3% (0%)
ROCK-S2 / MAX-S1 (0.05g)	6% (0%)	6% (0%)	6% (0%)
RURAL-S2 (0.07g)	12% (4%)	12% (3%)	12% (3%)
ROCK-S3 (0.07g)	15% (6%)	15% (4%)	15% (4%)
MAX-S2 (0.09g)	20% (9%)	19% (7%)	19% (7%)
ROCK-S3 / ROCK-S4 (0.11g)	36% (17%)	27% (13%)	27% (12%)
MAX-S3 (0.13g)	61% (28%)	39% (21%)	39% (19%)
RURAL-S4 (0.16g)	89% (40%)	49% (29%)	48% (25%)
MAX-S4 (0.20g)	>100% (67%)	87% (47%)	67% (39%)

294 As already noted, specific analytical computations should be carried out for the situation MAX-S4 or more
295 critical seismic implantations instead of using the general approach with the maximum values displayed in Table
296 6. More specifically, one should pay attention to specific parameters of the DSRW in the design that plays a
297 critical role in the seismic computation (see full details in [40]). Apart from the positive influence of the stone
298 bed inclination and the negative effect of the retained slope angle already highlighted, a positive internal wall
299 batter is recommended (this is the case of a self-stable wall). To support this recommendation, Walls 1 to 5
300 (with various stone types and bed inclinations) have been designed to withstand a specific seismic acceleration
301 (0.05g, 0.10g ... and 0.35g). In a second step, each wall section geometry has been modified, adding an internal
302 batter ($\lambda_m = 5\%, 10\%$ and 15%) but keeping the same surface area as before. It means that the total volume of
303 stones used in that case is the same but that the geometry of the wall section is different (i.e., with a larger
304 width at the base). Finally, the maximum acceleration withstood by the walls with an internal batter is
305 compared to the maximum acceleration found for those without the batter (Figure 9). The curves correspond
306 to the average values found throughout the different walls and stones. Only positive values have been found,
307 meaning a systematic improvement of the seismic resistance when adding an internal batter. This improvement
308 is particularly significant for walls with non-zero stone bed inclination and low (S2) to moderate (S3) seismic
309 hazard regions.



311

312

Figure 9: Effect of the internal batter λ_m (keeping the same area for the wall section) on the seismic resistance of a DSRW for different bed inclinations

313

a) $\alpha=0\%$; b) $\alpha=10\%$; c) $\alpha=20\%$.

314

To conclude, in zone S2, no DSRW extra-width is required to fulfil a seismic design if both an inclination bed of

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10% and an internal batter of at least 10% are used. This means that an adequate choice for the wall geometry

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compensates for the required extra resistance required to satisfy a seismic design in case of low seismicity.

317

Conclusions

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The present study addresses non-seismic and seismic designs of Dry Stone Retaining Walls (DSRWs) according

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to the European standards (Eurocodes) used for conventional retaining walls while proposing an adapted

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methodology (e.g. including internal failure of DSRW). It has been shown that the current French

321

recommendations for the non-seismic design of DSRWs comply with Eurocode 7 (geotechnical engineering)

322 standards being slightly more conservative than the latter. Moreover, in very low seismic zones ($a_h < 0.05g$),
323 non-seismic limit states are the most critical states for the design of DSRWs, which confirms that a seismic
324 design is not required in these regions [29]. This is generally not the case in zones of higher seismicity where
325 the seismic design according to Eurocode 8 (seismic engineering) standard is almost always the most critical.

326 The study revealed different geometrical optimisation options for DSRWs. It is highly recommended to use
327 systematically: i) a bed inclination of at least 10%; ii) a flat retained backfill; iii) and a positive internal batter of
328 at least 10%. These three geometric parameters have a significant impact on seismic design. If these
329 recommendations are followed in low seismic zones (up to $a_h = 0.08g$), there is also no increase in dimensions
330 to fulfil a seismic design.

331 The pseudo-static approach generally gives practical global recommendations for low to moderate seismic
332 hazard zones (up to $a_h = 0.2g$). In addition, a specific geometrically optimised (as stated above) pseudo-static
333 design can still produce affordable recommendations in more critical cases (up to $a_h = 0.3g$). However, for
334 higher design acceleration or particularly critical cases, the pseudo-static approach for the seismic design leads
335 to values higher than 50% for the extra-widths. In this case, dynamic time history computations are
336 recommended to obtain more accurate results that account for wall disintegration failure. Indeed, this failure
337 mode may be critical for high seismicity areas, particularly if combined with poor execution conditions.

338 Finally, as an example illustrating European countries, France is used as a case study of seismic design applied
339 to DSRWs in low to moderate seismicity areas. In metropolitan France, where many DSRWs can be found, the
340 expected extra-width provided by seismic design does not exceed 40% for walls located in low seismic zones
341 presenting a stone bed inclination of 10%, in case the foundation is not on the bedrock. In the same conditions,
342 walls built in moderate seismic hazard zones need either a stone bed inclination of 20% or a specific analytical
343 computation to optimise the section. Finally, walls directly founded on the bedrock, in the case of low and
344 moderate seismicity areas, require a maximum of 30% extra width to fulfil a seismic design.

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355 [Competing interest](#)

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357 [Authors' contribution](#)

358 NS: Investigation, Formal Analysis, Visualisation, Writing – original draft; CM: Methodology, Resources,
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361 [Data availability](#)

362 The data arising from the analytical simulations presented in the paper is not available publicly.

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