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1 **IN-PLANE DRIFT CAPACITY AT NEAR COLLAPSE OF ROCKING UNREINFORCED CALCIUM**
2 **SILICATE AND CLAY MASONRY PIERS**

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ABSTRACT

12 In recent years, seismic assessment of existing unreinforced masonry (URM) structures is being increasingly
13 based on nonlinear methods. The in-plane displacement capacity represents one of the most crucial yet still
14 debated features of the nonlinear behaviour of URM piers. International codes often employ empirical models to
15 estimate the pier ultimate drift. These models usually depends on the failure mode (flexure or shear) and on the
16 properties of the pier (such as geometry, material properties, boundary or loading conditions).

17 The present work focuses on the displacement capacity of Dutch masonry piers, or walls comparable to those,
18 failing after the activation of a rocking mechanism. As a consequence, a dataset of 38 quasi-static tests on URM
19 piers representative of the Dutch masonry is constructed and statistically analysed. The dataset, that includes also
20 new laboratory tests recently performed at Delft University of Technology, consists of both calcium silicate and
21 clay brick masonry piers characterised by low axial compressive loads and limited thickness. The displacement
22 capacity of calcium silicate masonry is of special interest because it was not investigated in the past as
23 extensively as for clay brick masonry. The analysis of the dataset highlights the influence of axial load ratio,
24 aspect ratio and pier height on the drift capacity of Dutch rocking URM piers, whereas the other parameters do
25 not appear to have a remarkable impact. Subsequently, a new empirical equation is derived and calibrated
26 against the dataset. The accuracy of the proposed equation is assessed by comparing it to empirical models
27 recommended in international standards and in the literature. For the considered dataset, representative of Dutch
28 rocking URM piers, the proposed equation improves the accuracy of the predictions and fairly reproduces the
29 dependence of the experimental drift capacity on the principal wall parameters.

30

31 **KEYWORDS:** Unreinforced masonry, Pier, In-plane test, Quasi-static cyclic test, Drift capacity, Calcium
32 Silicate masonry, Clay brick masonry, Dutch masonry, Empirical model

33

34 1. INTRODUCTION

35 In the past years, the assessment of existing unreinforced masonry (URM) buildings has been more and more
36 oriented towards the use of nonlinear analysis methods and displacement-based procedures. At component level,
37 different failure modes of masonry piers subjected to axial and shear loads can be observed. Rocking, shear
38 (sliding or diagonal cracking) and crushing are traditionally distinguished, but a combination of them (hybrid
39 failure mode) is often observed. For each mechanism, the in-plane drift capacity of URM piers represents a
40 crucial parameter, which needs to be properly identified to assess the performance of the building close to
41 structural collapse. The in-plane drift capacity is explicitly and directly required in equivalent-frame based
42 models [1-3] and its estimate is also provided by international standards, which usually differentiate between
43 flexural (rocking) and shear failure of the piers. For rocking piers, standards often recommend the use of
44 empirical equations that include several physical parameters and are based on the results obtained from quasi-
45 static tests. However, the background of these equations is not always properly described and sufficiently
46 transparent [4]. Even though some progress to refine the current empirical equations have been made for hollow
47 clay brick URM [5-6], there is a lack of information for other masonry types. Analytical mechanics-based
48 formulations of the load-displacement behaviour of URM piers [7-9] may represent an efficient alternative
49 approach to empirical formula, able to relate the local deformation of the material to the global displacements of
50 the structural element. However, the analytical formulations in these models are complex and further validation
51 for many specific masonry typologies is required, so that in the coming years they may be included in the
52 standard recommendations along with simple empirical equations.

53 The Dutch URM buildings have specific material [10] and structural [11] characteristics. A series of quasi-static
54 in-plane tests on URM piers was performed at the Stevin laboratory of Delft University of Technology in
55 2015-17 [12-13]. These tests were part of a comprehensive testing program that aimed at the characterization of
56 the structural behaviour of Dutch masonry, part of which was also developed at the European Centre for
57 Training and Research in Earthquake (EUCentre) and reported in [14-15]. In this paper, the findings of that
58 experimental campaign are used to produce an integrated dataset of tests performed on piers consisting of
59 masonry typologies typical for Dutch URM buildings. Hence, the work considers solid or perforated clay

60 masonry, with brick format and general purpose mortar, and calcium silicate masonry, with either brick format
61 and general purpose mortar or block/element format with thin layer mortar.

62 Based on the selected and examined dataset, the current work presents an evaluation of the displacement
63 capacity of rocking URM piers typical for Dutch or similar buildings. The paper evaluates walls whose collapse
64 is subsequent to the activation of a rocking mechanism. Rocking is intended in this work as a working
65 mechanism, with piers showing clear cracks in the bed-joints at the bottom-end of the specimen (and also at the
66 top-end in fixed-fixed conditions) and having narrow dissipative hysteresis loops (thin *S*-shaped cycles). The
67 focus is on the Near Collapse (NC) limit state, in line with the recent Dutch guidelines [16] for the assessment of
68 existing structures. The relevance of geometry, material properties, and boundary and loading conditions on the
69 drift capacity of the considered specimens is studied and discussed. Subsequently, a new empirical drift limit
70 equation for rocking piers is derived. The accuracy of the proposed equation is compared to that of the empirical
71 equations recommended in international standards or guidelines and in the literature.

72

73 **2. DRIFT CAPACITY MODELS IN INTERNATIONAL STANDARDS, GUIDELINES AND** 74 **LITERATURE FOR ROCKING URM PIERS**

75 As introduced in Section 1, several international standards and guidelines include equations based on empirical
76 and physical models that aim at estimating the displacement capacity of rocking URM piers. This section
77 presents the available formulations. It should be noted that the definition of rocking URM walls provided in
78 standards or guidelines may not correspond to the definition suggested in Section 1. Hence, the different
79 definitions will be mentioned.

80 Eurocode 8 – part 3 (EC8-3) [17] provides an estimate of drift capacity at the NC limit state (δ_u) based on the
81 ratio between the shear span (the distance between the point of zero moment and the base of the wall, H_0) and
82 the pier length (L), as shown in Eq. (1). This ratio (H_0/L) is commonly referred to as ‘shear ratio’.

$$\delta_{u,EC8} = \frac{4}{3} \cdot \left[0.8\% \left(\frac{H_0}{L} \right) \right] \quad (1)$$

83 Eq. (1) should be used to estimate the ultimate drift of the wall when its capacity is controlled by flexure, i.e.
84 when the flexural strength (computed at sectional level with a stress-block model, assuming the critical section at
85 the base of the pier) is smaller than the shear strength (determined according to a modified Mohr-Coulomb
86 criterion).

87 The American standard ASCE 41-13 [18] estimates the drift capacity at the Collapse Prevention limit state
 88 (equivalent to NC) equal to $\Delta t_{c,r}/H$, where $\Delta t_{c,r}$ is the lateral displacement associated with the onset of toe
 89 crushing that should be calculated using a moment-curvature or similar analytical approach. The moment-
 90 curvature analysis may be based on the plastic hinge length approach proposed in [19]. The drift is limited by a
 91 cap equal to 2.5%, as reported in Eq. (2).

$$\delta_{u,ASCE} = \min \left\{ \frac{\varepsilon_{cm}}{2} \left[\frac{\alpha\beta}{(\sigma_0/f_c)} - 1 \right]; 2.5\% \right\} \quad (2)$$

92 where $\varepsilon_{cm} = 0.4\%$ is the strain capacity of the masonry suggested in [19], α and β are parameters used to compute
 93 the neutral axis depth with the Whitney Stress Block for unconfined masonry (the value 0.85 is suggested [19]),
 94 and σ_0/f_c is the axial load ratio, computed as the ratio between the axial stress (σ_0) and the ultimate compressive
 95 strength of the masonry (f_c). ASCE 41-13 also limits the minimum thickness of the walls to 6 inches (≈ 152 mm)
 96 or, for solid brick masonry, two wythes. Eq. (2) refers to walls whose lateral strength is governed either by the
 97 expected in-plane rocking strength or by the lower-bound in-plane toe-crushing strength, both described in [18].
 98 The New Zealand 2017 NZSEE guidelines [20] estimate the drift capacity at the Life Safety (LS) limit state,
 99 which is equivalent to Significant Damage (SD), proportionally to the aspect ratio (H/L) of the pier. The ultimate
 100 drift at NC can be estimate by applying a coefficient equal to 4/3 (Eq.(3)).

$$\delta_{u,NZSEE} = \frac{4}{3} \cdot \min \left\{ 0.3\% \cdot \left(\frac{H}{L} \right); 1.1\% \right\} \quad (3)$$

101 Eq. (3) refers to piers generally characterised by a stable post-yield slope, limited by toe crushing that typically
 102 occurs at large rotations.

103 The Italian building code NTC [21] and its commentary [22] differentiate between new and existing URM
 104 structures. In the latter case, a constant value equal to 0.6% is recommended for walls whose failure is governed
 105 by flexure for the drift capacity at a limit state equivalent to Significant Damage (SD); the value should be
 106 doubled in case of cantilever behaviour (this last recommendation does not apply to new buildings, for which a
 107 unique value 0.8% is suggested). A coefficient equal to 4/3 is therefore applied to compute the ultimate drift at
 108 NC limit state. The estimate may be expressed as proposed in Eq. (4).

$$\delta_{u,NTC} = \begin{cases} 1.6\% & \text{for cantilever walls} \\ 0.8\% & \text{for double clamped walls} \end{cases} \quad (4)$$

109 Unlike the aforementioned standards, the Swiss guideline SIA D0237 [23] does not recommend different
 110 ultimate drifts for each failure mode, since the pier capacity is determined via stress-fields. This approach may
 111 be convenient since it limits substantial errors related to the wrong estimate of the correct failure mechanism

112 when shear and rocking strengths are similar. The provided formulation is expressed as function of the axial
 113 stress (σ_0), normalized with respect to the design compressive strength of the masonry (f_d), and of the boundary
 114 conditions. As proposed in the Swiss masonry standard SIA 266 [24], the mean compressive strength may be
 115 taken equal to $f_c = 2.4 f_d$. Similar to the Italian code, SIA D0237 estimates the drift capacity at a limit state
 116 equivalent to the SD limit state, and the factor 4/3 is again adopted to estimate the drift at NC. The final
 117 expression is reported in Eq. (5).

$$\delta_{u,SIA} = \begin{cases} \frac{4}{3} \cdot \left[0.8\% \cdot \left(1 - 2.4 \frac{\sigma_0}{f_c} \right) \right] & \text{for cantilever walls} \\ \frac{4}{3} \cdot \left[0.4\% \cdot \left(1 - 2.4 \frac{\sigma_0}{f_c} \right) \right] & \text{for double clamped walls} \end{cases} \quad (5)$$

118 In recent years, alternative models have been proposed to estimate the ultimate drift capacity of rocking URM
 119 piers, based on either empirical [5-6] or analytical models [7-9]. In the former category, Petry and Beyer [5]
 120 provide a remarkable analysis based on a dataset composed of 64 quasi-static tests performed on URM piers
 121 constructed with clay brick units and normal cementitious mortar. In accordance with the approach proposed in
 122 SIA D0237, both flexural and shear failure are evaluated by a single expression. The analysed data show that the
 123 drift capacity is also dependent on the height of the wall. For this reason, with respect to Eq. (5), an additional
 124 term that takes into account the size effect is included and the constant coefficients are returned to best-fit the
 125 experimental results. Two equations are proposed to estimate the pier capacity at NC (Eq. (6)) and SD limit state
 126 (Eq. (7)).

$$\delta_{u,Petry\&Beyer,NC} = 1.3\% \cdot \left(1 - 2.2 \frac{\sigma_0}{f_c} \right) \cdot \frac{H_0}{H} \cdot \left(\frac{H_{ref}}{H} \right)^{0.5} \quad (6)$$

$$\delta_{u,Petry\&Beyer,SD} = (0.7\% \div 1.0\%) \cdot \left(1 - 0.9 \frac{\sigma_0}{f_d} \right) \cdot \frac{H_0}{H} \cdot \left(\frac{H_{ref}}{H} \right)^{0.5} \quad (7)$$

127 H_{ref} is a reference height set equal to a typical storey height (2400 mm). In the analysis reported in Section 4, the
 128 range of values (0.7% ÷ 1.0%) included in Equation 7 will be substituted by the average value 0.85%, which
 129 provided the best estimate of the displacement capacity of the piers investigated in this paper.

130 Also Salmanpour et al. [6] propose a modification of the formulation of the Swiss guidelines (Eq. (5)), with the
 131 introduction of a coefficient δ_0 in place of a constant value and of the H_0/H variable, that takes into account the
 132 dependence on boundary conditions other than fixed-free and fixed-fixed conditions, as reported in Eq. (8). The
 133 value of δ_0 should be calibrated against experimental tests for piers with similar constituent materials and,
 134 possibly, geometry and pre-compression level, whereas analytical models are advised to derive a general

135 formulation. For Swiss clay masonry walls, the characteristic value of 0.7% is proposed, based on a dataset of 11
 136 tests included in [5] and [6].

$$\delta_{u,Salmanpour\&al.} = \delta_0 \cdot \left(1 - 2.4 \frac{\sigma_0}{f_c}\right) \cdot \left(\frac{H_0}{H}\right) \quad (8)$$

137 Table 1 summarises the parameters taken into account by the equations proposed in the international standards or
 138 in the literature, and reported in this section (Eq. (1)-(8)). Besides, it should be remarked that the models
 139 presented in Eurocode 8-Part 3 and in the American, New Zealand and Italian standards depend on the predicted
 140 failure mode (different drift limitations are provided for rocking/flexure and shear failure).

141
 142 **Table 1. Parameters evaluated in standards, guidelines and literature models estimating the drift capacity**
 143 **of rocking URM piers.**

	EC8-3	ASCE 41-13	NZSEE	NTC	SIA D0237	Petry & Beyer [5]	Salmanpour et al. [6]
Aspect ratio (H/L)	✓		✓				
Boundary conditions (H_0/H)	✓			✓	✓	✓	✓
Axial load ratio (σ_0/f_c)		✓			✓	✓	✓
Size effect (H_{ref}/H)						✓	

144
 145 **3. INFLUENCE OF DIFFERENT PARAMETERS ON THE DISPLACEMENT CAPACITY OF**
 146 **ROCKING URM PIERS**

147 The present work focuses specifically on the displacement capacity of Dutch masonry piers, or walls comparable
 148 to those.

149 Residential URM buildings in the Netherlands, and especially in the province of Groningen (where induced
 150 seismicity occurs), can be classified in four main typologies: terraced houses, detached houses, semi-detached
 151 houses and apartments [25]. The former three typologies dominate and generally involve low-rise buildings (one
 152 or two stories) characterised by low axial compressive actions in the load-bearing walls. The detached houses
 153 have usually relatively small openings and are often irregular in plan. The terraced houses are characterised by
 154 the presence of large daylight opening in the façades, so that the loadbearing structure is composed of slender
 155 piers in the longitudinal direction and by long and massive transversal walls, without openings. The semi-
 156 detached houses have intermediate characteristics between the other two typologies.

157 Until the Second World War (WW2), Dutch houses used to have internal and perimeter loadbearing walls made
 158 of single and double wythe solid clay brick masonry, respectively. After WW2, the use of cavity walls became

159 more and more common, with the load-bearing inner leaf made of CS masonry (CS bricks, blocks, and elements)
160 and the outer veneer of clay brick masonry.

161 A more detailed description of the Dutch masonry is provided in [10] and [26].

162

163 **3.1 DATASET OF QUASI-STATIC TESTS ON ROCKING PIER**

164 A database of quasi-static tests on URM piers is considered in this section and listed in Table 2. The database
165 consists of 38 specimens, from 14 series of tests performed in 9 different structural engineering laboratories, and
166 it has been selected with special focus on the properties of Dutch masonry piers as described above. Hence, the
167 dataset includes both calcium silicate and clay brick masonry walls, characterised by low axial compressive
168 loads (the axial load ratio σ_0/f_c shows a normal distribution with the average slightly smaller than 10%, as shown
169 in Figure 1a) and limited thickness (often 100 mm or 210 mm for single and double wythe clay brick masonry
170 walls, respectively; 100÷175 mm for CS masonry walls). Besides, the aspect ratio of the specimens is related to
171 the geometry of the piers in real buildings: calcium silicate masonry walls (typical of terraced houses) are
172 characterised by large aspect ratios, while clay brick masonry walls (used both in terraced and detached houses)
173 can be both slender and squat (Figure 1b). Only piers whose collapse is subsequent to the activation of a rocking
174 mechanism are considered. Tests 29-31 and 35-38 belong to the experimental campaign performed at TU Delft
175 in 2015/17. Tests 28, 32-34 were performed for the same campaign at the EUCentre. Several tests were already
176 included in the dataset analysed by Petry and Beyer [5], whose empirical model has been summarised in section
177 2. Differently from the dataset selected and analysed in this paper, Petry and Beyer included also specimens
178 failing in shear to derive an estimate of the pier drift capacity independent on the failure mode, as well as
179 specimens whose characteristics do not correspond with those described above for the Dutch masonry.

180 The considered specimens differ in employed material, geometry, boundary conditions, pre-compression level,
181 and bed- and head- mortar joints. The dataset includes a larger number of clay brick masonry piers, which are
182 more common in seismic countries and whose performance under lateral has been investigated over many
183 decades. Also piers with various sizes of calcium silicate units (bricks, blocks, or elements) are included; the
184 bricks with general purpose mortar (GPM) joints and the blocks and elements with thin layer mortar (TLM)
185 joints.

186 Table 2 reports the main properties of the considered specimens: the employed masonry units, the head- and bed-
187 joint typologies, the dimensions (length L , height H , thickness t), the ratio H_0/H , the aspect ratio H/L and the

188 shear ratio H_0/H , the applied confining vertical pressure σ_0 , the mean masonry compressive strength f_c , the axial
189 load ratio σ_0/f_c , the failure mode type, and the ultimate drift δ_u . Since the present work focuses on the Near
190 Collapse limit state, the ultimate drift δ_u is estimated as the drift corresponding to 20% strength degradation, as
191 adopted previously in [5] and [6]. In the case of cyclic tests, the average between the envelope of the hysteresis
192 loops is considered, as suggested in [18]. The hybrid failure mode is defined by the contribution of different
193 mechanisms, such as flexure and diagonal cracking or sliding.

194 A summary of the main features of the tests grouped for different unit type is presented in Table 3. The average
195 performances of clay brick piers and calcium silicate unit piers are similar, even though a large variation of
196 ultimate drifts is measured, with coefficient of variation larger than 40%. This mainly depends on the variability
197 of the specimen properties, since the ultimate drift is not an independent variable: for this reason, constant drift
198 limits may be estimated only for specific categories of piers having similar properties, such as the material
199 constituents, the aspect or the axial load ratio. The dependence of δ_u on each parameter is discussed in the next
200 section.

201 The results of the tests performed at TU Delft are overall not considerably different from those obtained in past
202 experiments: their drift capacity is larger than the average value (+21%), but still fully included in the standard
203 deviation of the sample set. The difference mainly depends on the higher average aspect ratio (+67%) with
204 respect to the overall dataset. The tests performed in 2016 (35-38 [13]) are characterized by larger ultimate drifts
205 that could be achieved after improvements of the setup, which allowed more stability of the system for large
206 drifts and consequently the application of larger imposed displacements.

207

Table 2. Database of quasi-static in-plane tests on rocking URM piers.

No.	Name	Ref.	Units	Head-Joints	Bed-Joints	L	H	t	H_0/H	H/L	H_0/L	σ_0	f_c	σ_0/f_c	Failure mode	δ_u
						(mm)	(mm)	(mm)	(-)	(-)	(-)	(MPa)	(MPa)	(-)		(%)
1	W3	[28]	SC	F	GPM	1625	1625	198	1.12	1.00	1.12	0.31	6.2	0.05	F	0.78
2	18-1	[29]	PC	F	GPM	2500	1750	300	0.5	0.70	0.35	0.60	6.0	0.10	H	1.00
3	18-2	[29]	PC	F	GPM	2500	1750	300	0.5	0.70	0.35	0.60	6.0	0.10	H	1.00
4	18-3	[29]	PC	F	GPM	2500	1750	300	0.5	0.70	0.35	0.60	6.0	0.10	H	2.00
5	18-4	[29]	PC	F	TLM	2500	1750	300	0.5	0.70	0.35	0.60	6.0	0.10	H	1.22
6	15-1	[29]	PC	U	TLM	984	1250	300	1.18	1.27	1.50	1.18	7.0	0.17	H	1.64
7	15-5	[29]	PC	U	GPM	992	1170	300	1.18	1.18	1.39	0.94	5.5	0.17	F	2.04
8	15-8	[29]	PC	F	GPM	992	1170	300	1.18	1.18	1.39	0.89	5.2	0.17	H	3.32
9	10-1	[29]	PC	F	GPM	1028	1510	300	1.06	1.47	1.56	0.60	4.0	0.15	F	1.71
10	10-3	[29]	PC	F	GPM	1033	1510	300	1.06	1.46	1.55	0.60	4.0	0.15	H	1.31
11	10-6	[29]	PC	F	GPM	1026	1510	300	1.06	1.47	1.56	0.60	4.0	0.15	F	2.32
12	14-1	[29]	PC	F	GPM	2567	1750	297	1.1	0.68	0.75	0.56	4.0	0.14	H	1.37
13	CL01	[30]	PC	U	TLM	1500	2500	175	0.5	1.67	0.83	0.32	4.0	0.08	F	2.97
14	CL03	[30]	PC	U	TLM	1000	2500	365	0.5	2.50	1.25	0.15	3.9	0.04	F	1.48
15	CL06	[30]	PC	F	GPM	1250	2600	300	0.5	2.08	1.04	0.50	10.0	0.05	F	1.97
16	CS05	[31]	CS-BL	F	TLM	1250	2500	175	0.5	2.00	1.00	1.04	13.0	0.08	H	1.61
17	CS06	[31]	CS-BL	U	TLM	1250	2500	175	1.05	2.00	2.10	1.04	13.0	0.08	F	1.73
18	CS07	[31]	CS-BL	U	TLM	2500	2500	175	0.5	1.00	0.50	1.04	13.0	0.08	H	1.20
19	CS08	[31]	CS-BL	U	TLM	2500	2500	175	1.05	1.00	1.05	1.04	13.0	0.08	H	0.85
20	W-2.7-L1-a	[32]	SC	F	GPM	2700	2700	190	0.5	1.00	0.50	0.09	6.2	0.01	F	1.90
21	W-2.7-L2-a	[32]	SC	F	GPM	2700	2700	190	0.5	1.00	0.50	0.25	6.2	0.04	F	1.60
22	W-2.7-L2-b	[32]	SC	F	GPM	2700	2700	90	0.5	1.00	0.50	0.25	6.2	0.04	F	1.10
23	W-1.2-L2-a	[32]	SC	F	GPM	1200	2700	190	0.5	2.25	1.13	0.37	6.2	0.06	F	2.92
24	W-1.8-L2-a	[32]	SC	F	GPM	1800	2700	190	0.5	1.50	0.75	0.37	6.2	0.06	F	1.98
25	W-3.6-L2-a	[32]	SC	F	GPM	3600	2700	190	0.5	0.75	0.38	0.37	6.2	0.06	H	1.49
26	PUP3	[5]	PC	F	GPM	2010	2250	200	1.5	1.12	1.68	1.05	5.9	0.18	F	0.83
27	T7	[6]	PC	F	GPM	2700	2600	150	1.0	0.96	0.96	0.64	6.4	0.1	F	0.62
28	COMP-1	[14]	CS-BR	F	GPM	1100	2750	100	0.5	2.50	1.25	0.52	6.2	0.08	F	2.00
29	COMP-0a	[12]	CS-BR	F	GPM	1100	2750	100	0.5	2.50	1.25	0.71	5.9	0.12	H	0.82
30	COMP-2	[12]	CS-BR	F	GPM	1100	2750	100	1.15	2.50	2.88	0.51	5.9	0.09	F	1.60
31	COMP-3	[12]	CS-BR	F	GPM	1100	2750	100	0.5	2.50	1.25	0.40	5.9	0.07	F	1.30
32	COMP2-1	[15]	SC	F	GPM	1200	2710	210	0.5	2.26	1.13	0.52	11.2	0.05	F	3.55
33	COMP2-2	[15]	SC	F	GPM	1200	2710	210	0.5	2.26	1.13	1.20	11.2	0.11	H	1.15
34	COMP2-3	[15]	SC	F	GPM	1200	2710	210	0.5	2.26	1.13	0.86	11.2	0.08	H	1.30
35	COMP-20	[13]	CS-BR	F	GPM	1110	2778	100	1.1	2.50	2.75	0.65	6.4	0.09	F	2.26
36	COMP-22	[13]	SC	F	GPM	2960	2710	210	1.1	0.92	1.01	0.36	9.24	0.04	H	1.81
37	COMP-24	[13]	CS-EL	F	TLM	977	2743	100	0.5	2.81	1.40	0.60	13.9	0.04	F	2.43
38	COMP-25	[13]	CS-EL	F	TLM	977	2743	100	1.1	2.81	3.09	0.60	13.9	0.04	F	3.10
Min						977	1170	90	0.5	0.7	0.4	0.09	3.9	0.01	-	0.72
Max						3600	2778	365	1.5	2.8	2.8	1.2	13.0	0.18	-	4.00
Average						1683	2232	220	0.7	1.6	1.1	0.651	7.3	0.10	-	1.73

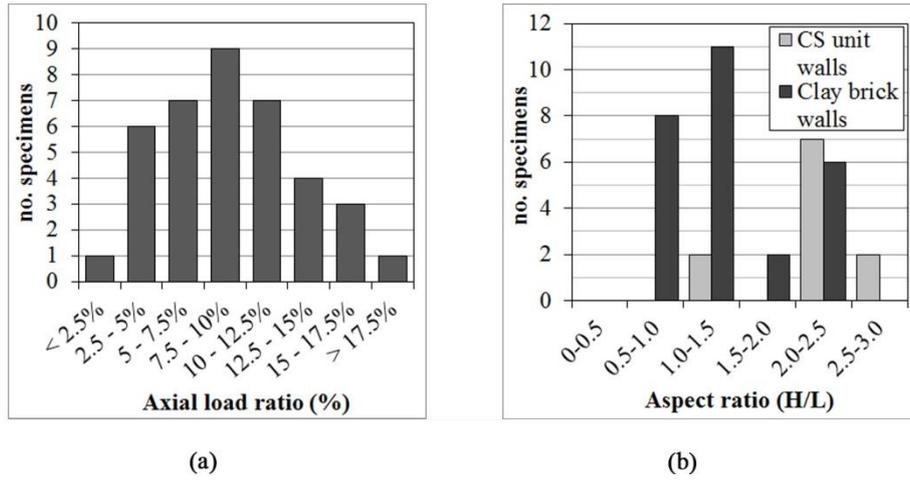
Legend:

Material: PC = Perforated clay bricks; SC = Solid clay bricks; CS-BR = Calcium silicate bricks; CS-BL = Calcium silicate blocks; CS-EL = Calcium silicate elements.

Head-joints: F = Filled head-joints; U = Unfilled head-joints.

Bed-joints: GPM = General purpose mortar bed-joints; TLM = Thin layer mortar bed-joints.

Failure modes: F = Flexural failure; H = Hybrid failure.



210

(a)

(b)

211 **Figure 1. Distribution of specimens depending on their axial load ratio (a) or aspect ratio (b).**

212

213 **Table 3. Main features of the tests grouped for different unit types**

	No. of tests	H/L	σ_0/f_c	δ_u (%)
Clay bricks	29	1.32 ± 0.58	0.10 ± 0.05	1.70 ± 0.73 (CV = 43%)
Solid bricks	10	1.53 ± 0.65	0.06 ± 0.02	1.95 ± 0.90 (CV = 46%)
Perforated bricks	19	1.22 ± 0.52	0.12 ± 0.04	1.57 ± 0.61 (CV = 43%)
Calcium silicate units	11	2.19 ± 0.64	0.08 ± 0.02	1.82 ± 0.82 (CV = 45%)
Calcium silicate bricks	5	2.50 ± 0.00	0.09 ± 0.02	1.80 ± 0.88 (CV = 44%)
Calcium silicate blocks	4	1.50 ± 0.58	0.08 ± 0.00	1.37 ± 0.49 (CV = 36%)
Calcium silicate elements	2	2.81 ± 0.00	0.05 ± 0.00	2.78 ± 0.46 (CV = 17%)
Every specimen	40	1.56 ± 0.22	0.09 ± 0.04	1.73 ± 0.75 (CV = 43%)
Tests performed at TU Delft	6	2.60 ± 0.16	0.08 ± 0.03	2.09 ± 0.97 (CV = 46%)

214

215 3.2 ANALYSIS OF THE PARAMETERS INFLUENCING THE DRIFT CAPACITY

216 This section investigates the influence of a number of parameters on the ultimate drift capacity of the specimens
 217 listed in Table 2. Those parameters are related to the geometry of the piers (the aspect ratio, and the wall height
 218 and thickness), to the boundary and loading conditions (the shear span over height ratio and the axial load ratio),
 219 or to a combination of the two (the shear ratio). Also different mortar bed- and head-joint typologies are
 220 evaluated. Other parameters related to the applied loading protocol (such as the number of cycles, in case of
 221 cyclic loading) may also influence the displacement capacity of URM piers. However, as far the author is aware,
 222 no performed testing campaign focused on the investigation of this specific aspect, and further research would be
 223 needed (in [5] a correction factor equal to 2-3 is suggested, but it is based on a very limited number of tests).
 224 Hence, despite the relevance of the factor, any evaluation of the influence of the loading protocol is excluded by
 225 this paper and will be hereinafter neglected.

226 Figure 2 and Figure 3 present the ultimate drifts measured in the experiments listed in Table 2 as function of the
 227 analysed parameters. In order to identify potential trends, the average and median values of the experimental

228 drifts are computed for specific ranges of the investigated parameter and shown in the diagrams by red or pink
229 lines (each range for which a value of average or median is computed is identified by the principal grid on the
230 horizontal axis). Red markers identifies the tests performed at the laboratory of TU Delft. It should be noted that
231 the ultimate drift capacity of URM piers is a complex phenomenon, which depends on several parameters, whose
232 effects, however, may be statistically correlated. As a consequence, the performed analysis, which considers
233 each parameter separately, can be useful to identify possible trends, but has a limited statistical value.

234 The investigated parameters were selected on the basis of their presence in the empirical models presented in
235 section 2. International standards and guidelines account either for the axial load ratio (SIA D0237 [23]), or the
236 shear ratio (EC8-3 [16]), or the aspect ratio (NZSEE [20]), or the shear span over height ratio (NTC [21], SIA
237 D0237 [23]). Additionally, Petry and Beyer [5] propose a correction coefficient to evaluate the influence of the
238 wall height (size effect).

239 The influence of the axial load ratio is accounted for only in the Swiss guidelines, but it is taken into account in
240 both empirical [5-6] and analytical [7-9] models available in the literature, since high levels of normal stresses
241 may lead to faster strength degradation of the specimens, due to crushing and splitting of the units. The trend is
242 confirmed by the present work (Figure 2a), having the specimens with low axial load ratio (≤ 0.05) a
243 displacement capacity on average about 30% larger than those with large axial load ratio (≥ 0.15).

244 In EC8 – part 3 a linear dependence of the ultimate drift on the shear ratio (H_0/L) is suggested. In the considered
245 database, higher drifts are in effect observed for larger shear ratios: the specimens with large shear ratios (≥ 2)
246 attain average ultimate drifts 40% larger than those with low shear ratios (≤ 1). However, the trend is not
247 homogeneous, and the relationship is far from being linear (Figure 2b). Besides, the shear ratio can be computed
248 as the product of the aspect ratio (H/L , related to the geometry of the pier) and of the ratio between the shear
249 span and the wall height (H_0/H , related to the boundary conditions): $H_0/L = H/L \cdot H_0/H$. The contribution of the
250 former can clearly be identified in Figure 2c, whereas the role of the shear span appears negligible (Figure 2d).
251 As a consequence, the displacement capacity of rocking piers may be related to the aspect ratio, such as in ASCE
252 41-13 and NZSEE, rather than to the shear ratio (EC8-3) or to the shear span (NTC, SIA D0237). From a
253 physical point of view, the local ductility of pier toes (that is related to the ultimate drift capacity [9-10]) is
254 approximately the same for double clamped and cantilever walls. This is consistent with the previous statement.

255 In past works [5, 9, 25] the drift capacity of rocking URM piers was linked to the height of the specimens,
256 suggesting that the displacement capacity is higher in piers of smaller heights. The specimens included in the

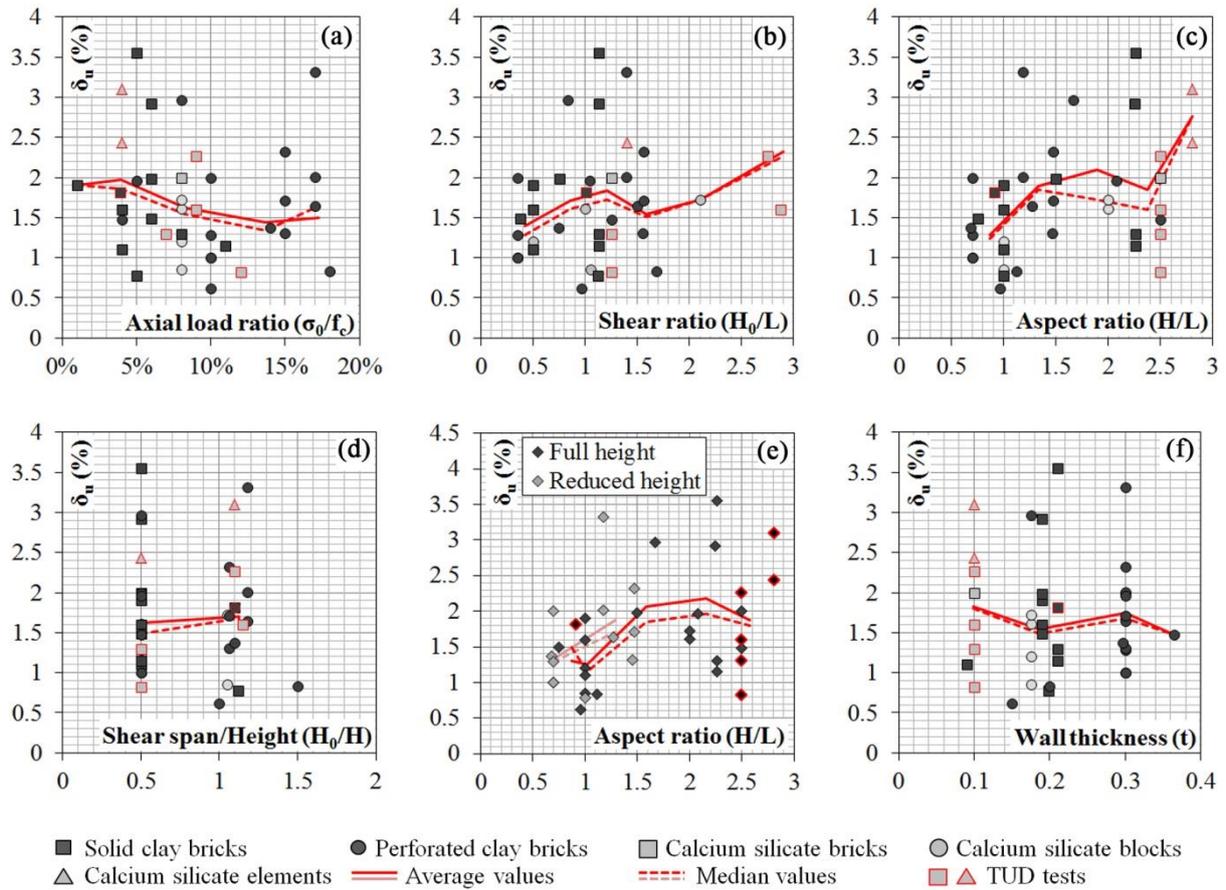
257 dataset have been divided into piers whose height is lower or higher than 2 m. The piers belonging to the former
258 group are characterised by smaller aspect ratios and larger axial load ratios; nevertheless, the average
259 performances of the two groups of specimens are similar (Figure 2e), suggesting that piers with reduced size
260 may in fact have larger capacity than comparable full height piers. However, the dispersion of the results does
261 not allow to derive stronger conclusions, and specific mechanics based models [9] or finite element analyses [25]
262 are believed to be more appropriate to adequately identify the effect of the relative dimensions of the single units
263 with respect to the size of the pier.

264 Other parameters related to the geometry of the specimens, such as the wall thickness (that, for thick walls, is
265 also related to different bond patterns), do not show any remarkable influence on the drift capacity of the piers
266 (Figure 2f).

267 Specific attention has also been devoted to the typologies of bed- and head-joints, respectively, as shown in
268 Figure 3. Small differences are observed between the specimens constructed with general purpose mortar or thin
269 layer mortar, and between those with filled or unfilled head-joints. These results may confirm the relatively little
270 influence of the type of bed- and head-joints on the displacement capacity of URM piers, as already remarked in
271 [5]. However, given the rather small number of specimens with unfilled head-joints, further research would be
272 needed to properly define the effect of filling the head-joints on the ultimate drift of rocking URM piers.

273 Overall, the axial load ratio (Figure 2a) and the aspect ratio (Figure 2c) are the only two parameters for which a
274 clear trend relating the ultimate drift to the parameter was observed. Additionally, Figure 2e suggests that piers
275 with reduced height may have larger capacity than comparable full height piers, but the small number of piers
276 with reduced height and the dispersion of the results do not allow to derive stronger conclusions. Other factors
277 do not show any remarkable impact on the displacement capacity of the piers, even though further research may
278 be needed, especially regarding the use of thin layer mortar and the filling of head-joints.

279

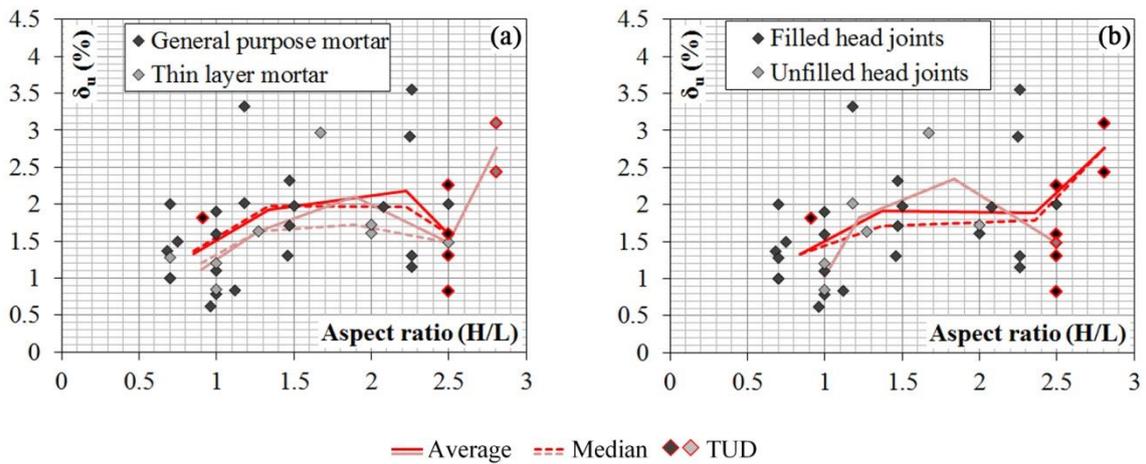


280

281 **Figure 2. Ultimate drift δ_u in function of: the axial load ratio (a), the shear ratio (b), the aspect ratio (c),**

282 **the shear span/height ratio (d), the aspect ratio with two groups of wall height (e), the wall thickness (f).**

283



284

285 **Figure 3. Ultimate drift δ_u in function of the aspect ratio: specimens with general purpose/thin layer**

286 **mortar bed-joints (a) and with filled/unfilled head-joints (b) are identified by different markers.**

287

288

289 **4. A NEW EMPIRICAL DRIFT CAPACITY MODEL**

290 As discussed in Section 2, several empirical models that aim at estimating the displacement capacity of rocking
 291 URM piers have already been included in international standards and guidelines. In this work a new empirical
 292 equation is presented, and the parameters included in the equation are calibrated to provide the best estimate of
 293 the displacement capacity of the piers. This section aims at assessing the efficiency of the proposed model in
 294 comparison with the existing models, by means of the experimental dataset presented in Section 3.1.

295 The analysis reported in Section 3.2 highlights that three parameters (the axial load ratio and the aspect ratio; the
 296 height of the pier) appear to affect the displacement capacity of rocking URM piers. However, in order to further
 297 investigate possible correlations between the parameters and the displacement capacity of the piers, the
 298 following general equation is considered:

$$\delta_u = A \left(1 - B \left(\frac{\sigma_0}{f_c} \right)^{1/C} \right) \left(\frac{H}{L} \right)^{1/D} \left(\frac{H_0}{H} \right)^{1/E} \left(\frac{H_{ref}}{H} \right)^{1/F} \kappa_{BJ} \kappa_{HJ} \quad (9)$$

299 Eq. (9) can be subdivided in several parts: a constant part, identified by the coefficient A ; the dependence on the
 300 axial load ratio, which reduces the displacement capacity with increasing axial load ratios (via B and C); the
 301 dependence either on the aspect ratio or on the shear ratio, with larger ultimate drifts for more slender piers (via
 302 D or E , respectively); correction factors for specimens with TLM bed-joints or unfilled head-joints (κ_{BJ} and κ_{HJ} ,
 303 respectively, whose value is equal to 1 in case of GPM bed-joints and filled head-joints). The value $H_{ref} = 2.4$ m
 304 is selected, as suggested in [5].

305 In the absence of an analytical model that may be able to justify the influence of each single parameter (which is
 306 out of the scope of the current work), the value of each of the coefficients A , B , C , D , E , F , κ_{BJ} and κ_{HJ} is derived
 307 in order to minimize the mean absolute error (MAE) and the modified mean relative error (MRE^{*}) for each of the
 308 three material types included in the considered dataset (solid [i] or perforated [ii] clay brick masonry; calcium
 309 silicate [iii] masonry). The absolute and modified mean errors are computed according to Eq. (10) and (11),
 310 respectively; the latter allows to weigh in the same way underpredictions and overpredictions (e.g. if the
 311 predicted ultimate drift is twice or half the experimental value, the same error, I , is computed).

$$MAE = \frac{1}{n} \sum_{i=1}^n |\delta_{u,p} - \delta_{u,e}| \quad (10)$$

$$\text{MRE}^* = \begin{cases} \frac{1}{n} \sum_{i=1}^n \left| 1 - \frac{\delta_{u,e}}{\delta_{u,p}} \right| & \text{if } \frac{\delta_{u,p}}{\delta_{u,e}} < 1 \\ \frac{1}{n} \sum_{i=1}^n \left| \frac{\delta_{u,p}}{\delta_{u,e}} - 1 \right| & \text{if } \frac{\delta_{u,p}}{\delta_{u,e}} \geq 1 \end{cases} \quad (11)$$

312 where n is the number of specimens considered.

313 The calibration procedure was divided in two phases. First, MAE and MRE* are computed for each single
 314 coefficient to identify clear trends. The procedure is run iteratively, so that the other coefficients are selected on
 315 the basis of the previous iterations. Then, the possible correlation between parameters A , B , D , F (for which the
 316 changes of the parameter produce small variations of the errors) is analysed, varying systematically each of these
 317 four parameters. Subsequently, the combination of values which minimize both MAE and MRE* is considered.

318 The procedure is repeated for each of the three main groups of evaluated materials in order to further minimize
 319 the effect of possible correlations between the parameters, since each of the three groups is characterized by
 320 different geometry of the piers and constituent materials.

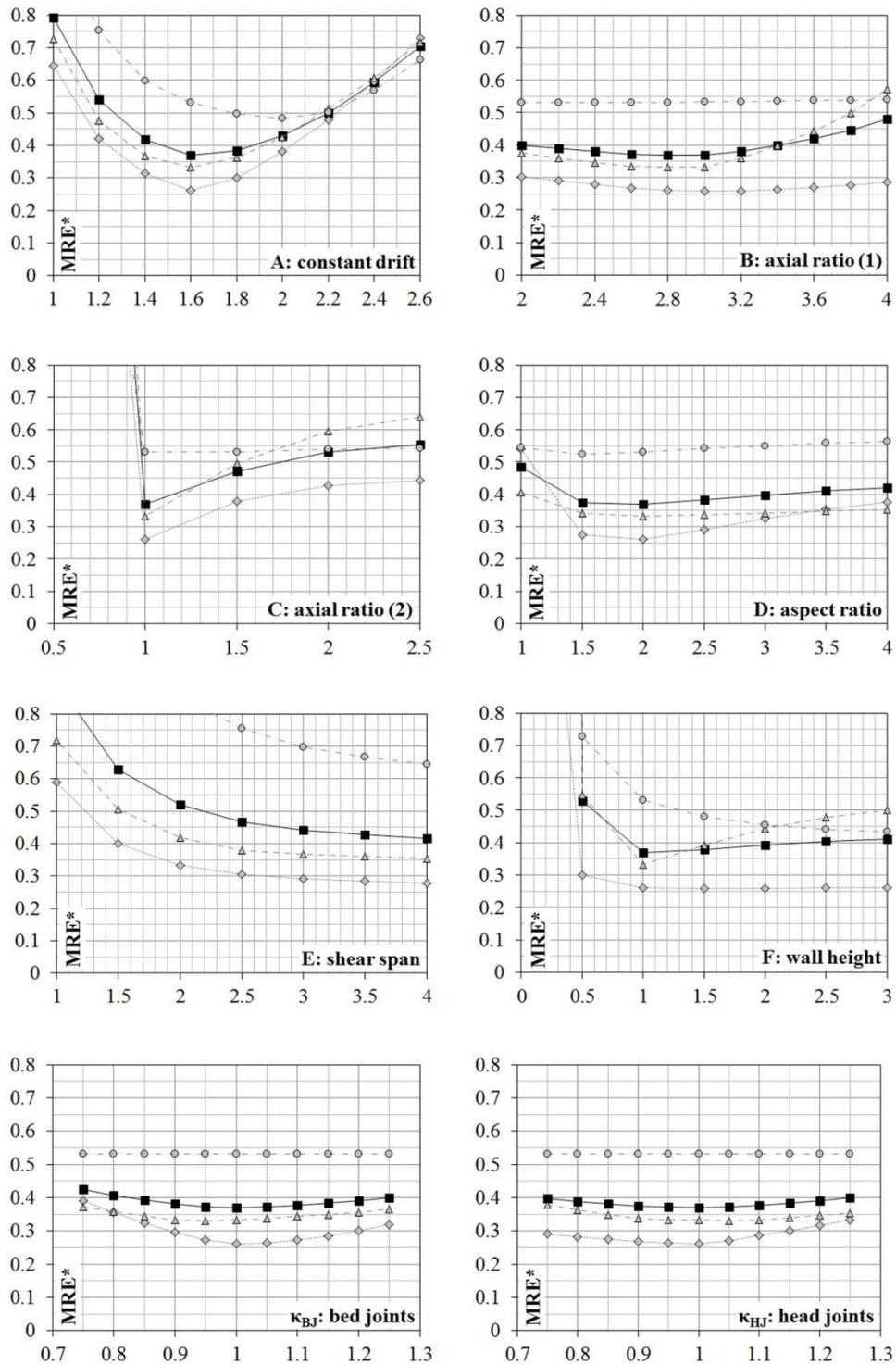
321 A graphical interpretation of the first phase of the applied procedure is shown in Figure 4.

322 The procedure finally results in the following coefficients: $A = 1.6\%$; $B = 2.6$; $C = 1$; $D = 2$; $E \rightarrow +\infty$; $F = 1$;
 323 $\kappa_{BJ} = 1$; $\kappa_{HJ} = 1$. This concludes towards the following equation for the estimate of the mean near-collapse drift
 324 limit of Dutch -or comparable- rocking URM piers:

$$\delta_u = 1.6\% \cdot \left(1 - 2.6 \left(\frac{\sigma_0}{f_c} \right) \right) \cdot \sqrt{\frac{H}{L}} \cdot \left(\frac{H_{ref}}{H} \right) \quad (12)$$

325

326



■ Any masonry ◇ Calcium silicate unit masonry
 ▲ Solid clay brick masonry ○ Perforated clay brick masonry

327

328 **Figure 4. Calibration of the coefficients A, B, C, D, E, F, κ_B , and κ_H : modified mean relative error**

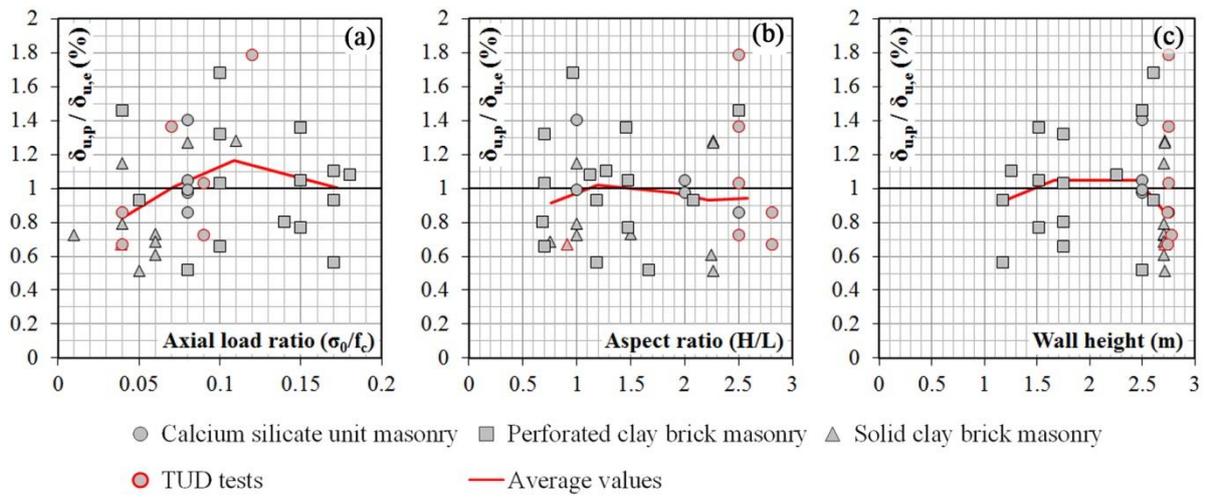
329 **(MRE*) at varying each single coefficient. The values of the coefficients not analysed in each figure should**

330 **be assumed as in Eq. (12).**

331

332 In the following, the accuracy of Eq. (12) is assessed by plotting the ratio between the predicted ($\delta_{u,p}$) and the
 333 experimental ($\delta_{u,e}$) ultimate drift at varying the different parameters. Figure 5 shows the distribution of the
 334 $\delta_{u,p}/\delta_{u,e}$ ratio at varying aspect ratio, axial load ratio, and height of the specimens. Similarly to the analysis
 335 presented in Section 3.2, the average value of the $\delta_{u,p}/\delta_{u,e}$ ratio is computed for small ranges of the investigated
 336 parameter and shown in the diagrams by a red line (the range for each parameter is identified by the principal
 337 grid on the horizontal axis). The ratios show distributions close to one and without significant trends. This entails
 338 a correct estimate of the dependency of the ultimate drift on the evaluated parameter.

339



340

341 **Figure 5. Predicted ($\delta_{u,p}$) over experimental ($\delta_{u,e}$) ultimate drift at varying the axial load ratio (a), the**
 342 **aspect ratio (b), and the shear ratio (c).**

343

344 Eq. (12) is also compared to the models reported in Section 2 (Eq. (1)-(6)) by plotting the ratio between the
 345 predicted ($\delta_{u,p}$) and the experimental ($\delta_{u,e}$) ultimate drift at varying the axial load ratio. This parameter has been
 346 selected since it visibly affects the ultimate drift of a pier, as discussed in section 3.2. Eq. (8) was not included in
 347 the discussion because it was derived for a specific masonry type with well-defined properties [7], significantly
 348 different from the Dutch masonry studied in the present work. The results are plotted in Figure 6, along with the
 349 distribution curves computed assuming a log-normal distribution of the sample. Figure 7 plots the cumulative
 350 functions derived for both the original data and equivalent distributions (each equivalent distribution has the
 351 same average and standard deviation of the original distribution but a larger population to allow a neater
 352 representation of the results). Finally, Table 4 summarises the values of MRE*, MAE, and the main statistics of
 353 the predicted over experimental ultimate drift ratios.

354 The analysis of the diagrams reported in Figure 6 allows to evaluate the capability of the various equations to
355 provide a good estimate of the experimental capacity. It is important to assess whether the distribution is
356 distributed around one and does not show peculiar trends. Besides, a limited dispersion of the results (i.e. a small
357 coefficient of variation) is also advisable. Graphically, the dispersion of the results can be easily visualized by
358 the distance between the lines of 50% and 95% fractiles (red lines in Figure 6). It should be noted that most of
359 the considered standards, guidelines, and papers relate the displacement capacity to the pier failure mode. Since
360 each empirical model targets different types of response, several specimens included in the dataset presented in
361 Section 3.1 would not be recognised as rocking piers. As a consequence, different equations from those
362 discussed in Section 2 (Eq. (1)-(6)) would be used. However, the proposed comparison focuses specifically on
363 rocking URM piers and attempts to assess the suitability of different equations, as those discussed in Section 2,
364 to predict the ultimate drift of this type of piers. Therefore, it was not conceived as a general criticism to the
365 assessment procedures suggested in standards, guidelines, or papers.

366 Among the models discussed in Section 2, a good performance is achieved by applying the moment-curvature
367 based model suggested in [19] and in agreement with requirements of ASCE 41-13, which presents small values
368 of both MRE* and MAE. However, a trend in the distribution of the predicted over experimental ultimate drift
369 ratios seems to be identified (Figure 6c), suggesting that the suggested hyperbolic dependence on the axial load
370 ratio may lead to underestimate the displacement capacity of walls with large axial load ratio.

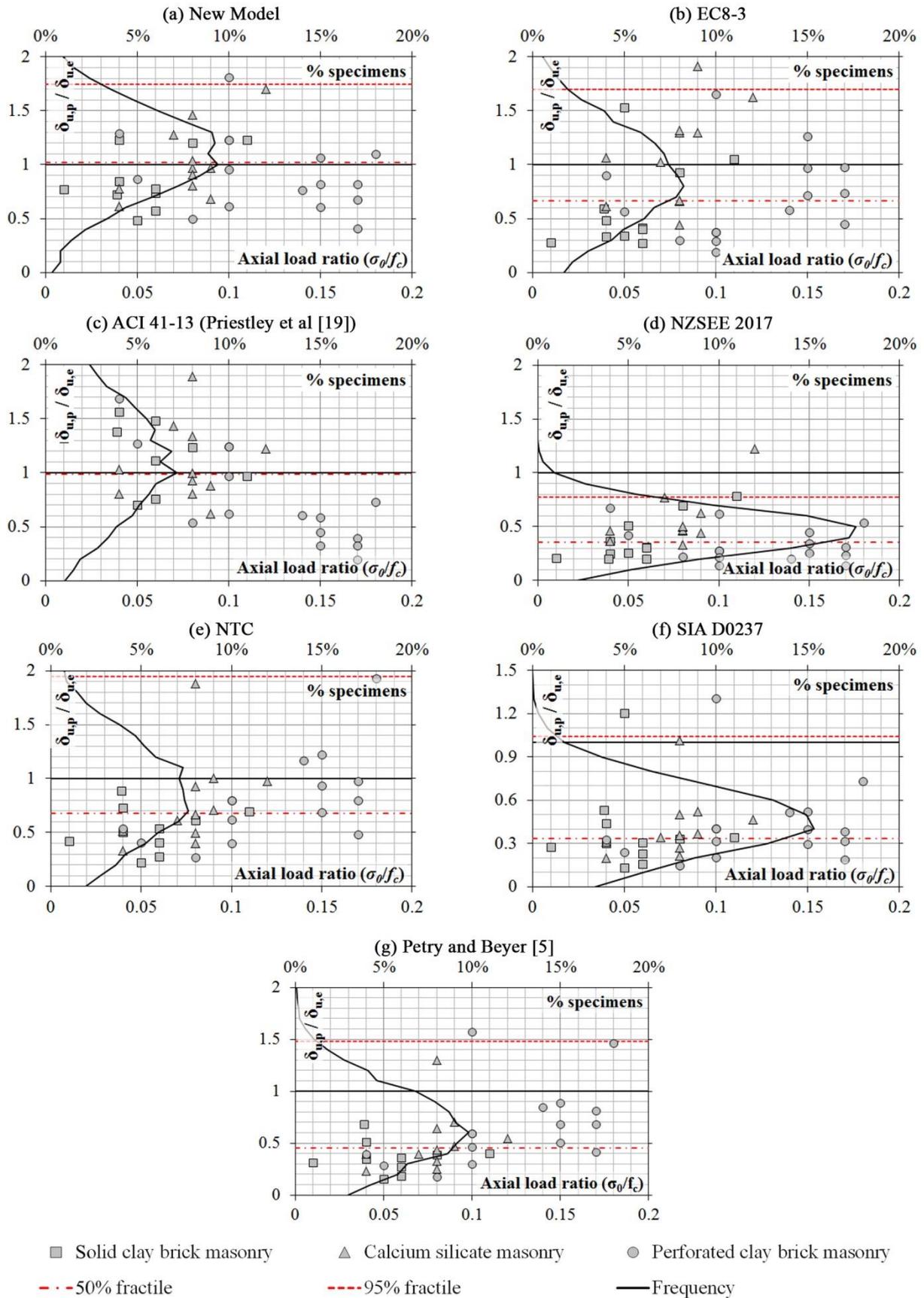
371 The equations presented on the basis of Eurocode 8-Part 3 or NTC have overall similar performance. In both
372 cases the obtained ratios are more scattered and overall slightly conservative, since only 30% and 20%,
373 respectively, of the experimental ultimate drift values are overestimated, as shown in Figure 7a. Criticisms to the
374 accuracy of the current formulation of the EC8-Part3 equation had already been presented in [4], where the
375 influence of vertical stresses, element aspect ratio and boundary conditions was advised to be evaluated for
376 different types of masonry. As for NTC, a small trend is shown in Figure 6e, possibly due to the neglect of the
377 influence of the axial load ratio.

378 In a similar way, also the ratios obtained for both the equations provided in NZSEE and SIA D0237 show a
379 similar distribution, and no trends can be identified. However, the two equations strongly underestimate on
380 average the experimental values. As for the Swiss guidelines, the large underestimation may be partially due to
381 the large differences in geometry and material constituents between typical Swiss hollow clay brick masonry
382 piers and the piers studied in this work. Similar evaluations are valid for the model proposed by Petry and Beyer
383 [5], which, however, presents smaller values of MRE* and MAE, showing that accounting for the shear ratio and

384 the size effect introduce small improvements for the evaluated dataset.

385 It should be remarked that different couples of equations (EC8-Part 3 and NTC; NZSEE and SIA D0237) have
386 rather similar cumulative curves, even though certain equations contain parameters that are neglected in the other
387 ones (as summarised in Table 1). Consequently, the various approaches can provide a similar overall estimate of
388 the displacement capacity of rocking URM piers, but the individual estimates for specific walls may be very
389 different. The specific limitations of each of these models can be overcome by considering a different
390 combination of them, such as in the proposed Eq. (12).

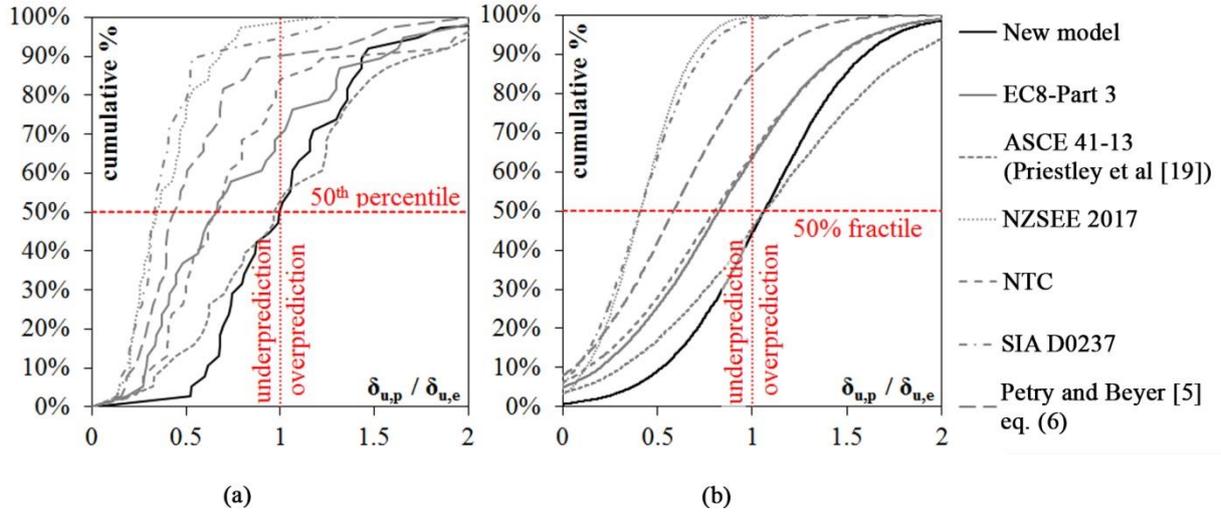
391



392

393 **Figure 6. Predicted ($\delta_{u,p}$) over experimental ($\delta_{u,e}$) ultimate drifts at varying of the axial load ratio for the**

394 **proposed model and the equations available in standards and in the literature.**



395

396 **Figure 7. Cumulative functions for the predicted ($\delta_{u,p}$) over experimental ($\delta_{u,e}$) ultimate drift ratio**
 397 **obtained for: (a) the original data and (b) an equivalent normal distributions of the evaluated dataset.**

398

399 **Table 4. Main values for the predicted ($\delta_{u,p}$) over experimental ($\delta_{u,e}$) ultimate drift ratios for the evaluated**
 400 **models.**

Standard/model	MRE*	MAE	$\delta_{u,p} / \delta_{u,e}$		$\delta_{u,p} / \delta_{u,e}$		CoV
					min	Max	
New Model, Eq. (12)	0.38	0.49	0.46	2.44	1.09	0.43	39%
EC8-Part 3	1.00	0.77	0.19	2.16	0.82	0.50	62%
ASCE 41-13	0.69	0.69	0.20	3.21	1.08	0.60	55%
NZSEE 2017	2.12	1.09	0.14	1.22	0.41	0.22	54%
NTC	0.89	0.78	0.23	2.58	0.80	0.52	65%
SIA D0237	2.23	1.16	0.13	1.31	0.41	0.26	64%
Petry and Beyer [5], Eq. (6)	1.54	0.99	0.15	2.02	0.58	0.41	70%

MRE* = modified mean relative error; MAE = mean absolute error; min = minimum $\delta_{u,p} / \delta_{u,e}$; Max = maximum $\delta_{u,p} / \delta_{u,e}$; μ = mean value; σ = standard deviation; CoV = coefficient of variation

401

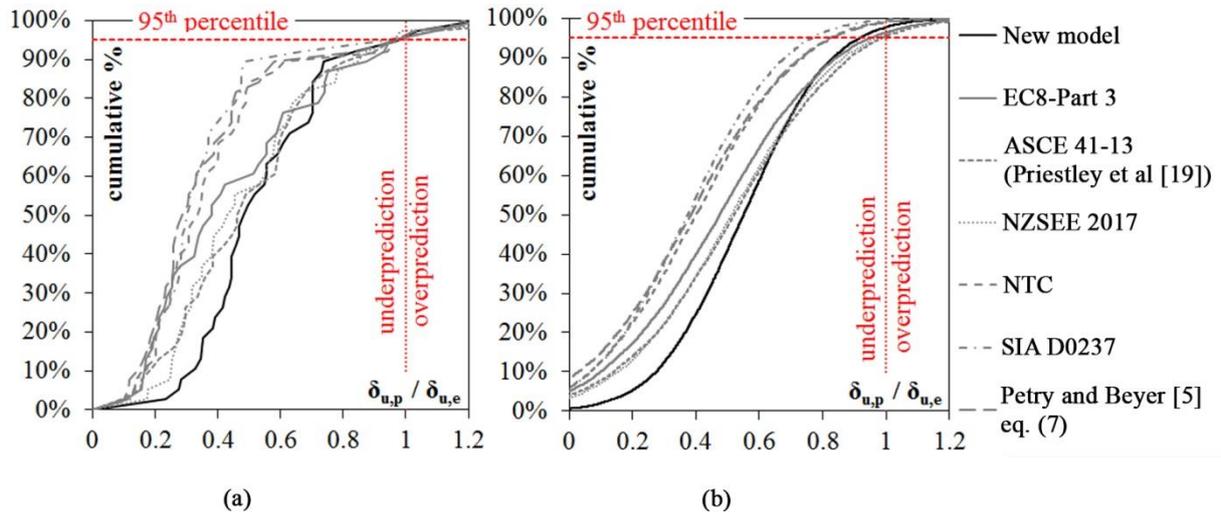
402 The Dutch guidelines for the assessment of existing buildings [16] do not refer to mean values of the drift
 403 capacity (such as in Eq. (12)) but they are based on a specific semi-probabilistic safety philosophy. Hence, to
 404 make the proposed equation suitable for those guidelines, the 5th fractile value of the dataset distribution is
 405 evaluated. The process of minimization of the error is repeated to have the 95% of the predicted ultimate drifts
 406 lower than the corresponding experimental value (i.e. 95% of the specimens in the dataset satisfies the relation
 407 $\delta_{u,p} / \delta_{u,e} < 1$). The process results in a new value for A (A=0.9%) while the other parameters do not change, as
 408 reported in Eq. (13):

$$\delta_u = 0.9\% \cdot \left(1 - 2.6 \left(\frac{\sigma_0}{f_c}\right)\right) \cdot \sqrt{\frac{H}{L}} \cdot \left(\frac{H_{ref}}{H}\right) \quad (13)$$

409 To assess the suitability of the proposed formulation for the considered dataset of tests, the accuracy of Eq. (13)
 410 is compared again to that of the models above discussed. However, since standards usually refer to the average
 411 structural performance for the assessment of existing buildings, the original equations are divided by a
 412 coefficient λ , so that the 95th percentile of the distribution of the predicted over experimental ultimate drift ratios
 413 ($\delta_{u,p}/\delta_{u,e}$) is slightly smaller than 1. The cumulative curves for each of the considered equations are plotted in
 414 Figure 8, and Table 5 summarises the value of the coefficient λ and the main values for the predicted ($\delta_{u,p}$) over
 415 experimental ($\delta_{u,e}$) ultimate drift ratios.

416 Once more, even though the dispersion of the results is high also for the new model, the coefficient of variation
 417 is reduced considerably. Therefore, for the considered dataset representative of the Dutch clay and calcium
 418 silicate masonry piers, the proposed equation allows to improve the average estimate of the drift capacity of the
 419 piers with respect to the existing empirical formulations. The analysis represents the basis for the refinement of
 420 the estimate of the displacement capacity of rocking URM piers that will be included in the updated version of
 421 the Dutch guidelines NEN-NPR 9998.

422



423

424 **Figure 8. Cumulative functions for the predicted ($\delta_{u,p}$) over experimental ($\delta_{u,e}$) ultimate drift ratio**
 425 **obtained for: (a) the original data and (b) an equivalent normal distributions of the evaluated dataset.**

426

427 **Table 5. Main values for the predicted ($\delta_{u,p}$) over experimental ($\delta_{u,e}$) ultimate drift ratios for the evaluated**
 428 **models.**

Standard/model	λ			$\delta_{u,p} / \delta_{u,e}$		CoV	95 th perc.
		Min	Max	μ	σ		
New Model, Eq. (13)	-	0.23	1.24	0.55	0.22	39%	0.98
EC8-Part 3	1.75	0.11	1.23	0.47	0.29	62%	0.97
ASCE 41-13	2.10	0.09	1.53	0.52	0.29	55%	0.98
NZSEE 2017	0.80	0.18	1.52	0.51	0.28	54%	0.96
NTC	2.00	0.11	1.29	0.40	0.26	65%	0.98
SIA D0237	1.10	0.12	1.19	0.37	0.24	64%	0.95
Petry and Beyer [5], Eq. (7)	-	0.10	1.32	0.38	0.27	70%	0.97

λ = reduction factor (the original equations are divided by the factor λ); min = minimum value of $\delta_{u,p} / \delta_{u,e}$; Max = maximum value of $\delta_{u,p} / \delta_{u,e}$; μ = mean value; σ = standard deviation; CoV = coefficient of variation

429

430 5. CONCLUDING REMARKS

431 In the present work, the results of a comprehensive experimental campaign performed in 2015-17 at the
 432 laboratory of Delft University of Technology [12-13] and at the European Centre for Training and Research in
 433 Earthquake (EUCentre) [14-15] are used to integrate the outcomes of past quasi-static cyclic tests performed on
 434 URM masonry piers to create a dataset representative of Dutch clay and calcium silicate masonry URM piers.
 435 The ultimate displacement capacity of piers failing after the activation of a rocking mechanism is investigated,
 436 and the influence of the following parameters discussed: the axial load ratio, the aspect ratio, the shear ratio, the
 437 shear span, the wall height and thickness, the features of the mortar bed- and head-joints. An alternative new
 438 empirical model is then derived and the accuracy of its predictions is compared to that of the empirical equations
 439 recommended in international standards and guidelines or in the literature.

440 The analysis of the selected dataset shows that the average performances of clay brick masonry piers and
 441 calcium silicate masonry piers are similar, even though a large variation of ultimate drifts is measured. Among
 442 the evaluated parameters, the axial load ratio and the aspect ratio are the most important parameters that affect
 443 the ultimate drifts. Additionally, piers with reduced height may have larger capacity than comparable full height
 444 piers, but the small number of piers with reduced height and the large dispersion of the results do not allow to
 445 derive strong conclusions. Other properties of the specimens (the boundary conditions, the wall height and
 446 thickness, the head- and bed-joint typologies) do not show remarkable impact on the displacement capacity of
 447 the piers, even though further research may be needed, especially with regard to the bed- and head-joint
 448 typologies.

449 Based upon the analysis, a new empirical drift limit equation for rocking piers is proposed. The model takes into
 450 account the effects of the axial load ratio, the aspect ratio, and the height of the specimen. The parameters of this

451 model have been calibrated against the dataset. With respect to the existing empirical models recommended in
452 international standards and guidelines as well as in the literature, the proposed model improves the average
453 accuracy of the results and fairly reproduces the dependence of the experimental drift capacity on the principal
454 wall parameters. The proposed equation has been also recomputed to suit the specific safety philosophy of the
455 Dutch guidelines. Once more, for the considered dataset representative of the Dutch masonry typologies, the
456 proposed equation allows improving significantly the average estimate of the drift capacity of rocking URM
457 piers. It will therefore be included in the updated version of the Dutch guidelines NEN-NPR 9998.

458

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