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In-plane drift capacity at near collapse of rocking unreinforced calcium silicate and clay masonry piers

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ABSTRACT

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

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

1. Introduction

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

The Dutch URM buildings have specific material [10] and structural [11] characteristics. A series of quasi-static in-plane tests on URM piers was performed at the Stevin laboratory of Delft University of Technology in 2015–17 [12,13]. These tests were part of a comprehensive testing program that aimed at the characterization of the structural behaviour of Dutch masonry, part of which was also developed at the

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European Centre for Training and Research in Earthquake (EUCentre) and reported in [14,15]. In this paper, the findings of that experimental campaign are used to produce an integrated dataset of tests performed on piers consisting of masonry typologies typical for Dutch URM buildings. Hence, the work considers solid or perforated clay masonry, with brick format and general purpose mortar, and calcium silicate masonry, with either brick format and general purpose mortar or block/element format with thin layer mortar.

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

2. Drift capacity models in international standards, guidelines and literature for rocking urm piers

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

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

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

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

The American standard ASCE 41-13 [18] estimates the drift capacity at the Collapse Prevention limit state (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 crushing that should be calculated using a moment-curvature or similar analytical approach. The moment-curvature analysis may be based on the plastic hinge length approach proposed in [19]. The drift is limited by a cap equal to 2.5%, as reported in Eq. (2).

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

where $\varepsilon_{cm} = 0.4\%$ is the strain capacity of the masonry suggested in [19], α and β are parameters used to compute the neutral axis depth with the Whitney Stress Block for unconfined masonry (the value 0.85 is suggested [19]), and σ_0/f_c is the axial load ratio, computed as the ratio between the axial stress (σ_0) and the ultimate compressive strength of the masonry (f_c). ASCE 41-13 also limits the minimum thickness of the walls to 6 in. ($\approx$152 mm) or, for solid brick masonry, two wythes. Eq. (2) refers to walls whose lateral strength is governed either by the expected in-plane rocking strength or by the lower-bound in-plane toe-crushing strength, both described in [18].

The New Zealand 2017 NZSEE guidelines [20] estimate the drift capacity at the Life Safety (LS) limit state, which is equivalent to Significant Damage (SD), proportionally to the aspect ratio (H/L) of the pier. The ultimate drift at NC can be estimate by applying a coefficient equal to 4/3 (Eq. (3)).

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

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

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

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

Unlike the aforementioned standards, the Swiss guideline SIA D0237 [23] does not recommend different ultimate drifts for each failure mode, since the pier capacity is determined via stress-fields. This approach may be convenient since it limits substantial errors related to the wrong estimate of the correct failure mechanism when shear and rocking strengths are similar. The provided formulation is expressed as function of the axial stress (σ_0), normalized with respect to the design compressive strength of the masonry (f_d), and of the boundary conditions. As proposed in the Swiss masonry standard SIA 266 [24], the mean compressive strength may be taken equal to $f_c = 2.4\, f_d$. Similar to the Italian code, SIA D0237 estimates the drift capacity at a limit state equivalent to the SD limit state, and the factor 4/3 is again adopted to estimate the drift at NC. The final expression is reported in Eq. (5).

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

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

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

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

H_{ref} is a reference height set equal to a typical storey height (2400 mm). In the analysis reported in Section 4, the range of values (0.7% ÷ 1.0%)