Engineering Structures 164 (2018) 195–213

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Fragility curves for free and restrained rocking masonry façades in one-sided motion

Linda Giresini[a,*], Claudia Casapulla[b], Roman Denysiuk[c], Jose Matos[c], Mauro Sassu[d]

[a] Department of Energy, Systems, Territory and Constructions Engineering, University of Pisa, Largo Lucio Lazzarino, 1, 56100 Pisa, Italy
[b] Department of Structures for Engineering and Architecture, University of Naples Federico II Naples, via Forno Vecchio 36, 80134 Naples, Italy
[c] ISISE, Department of Civil Engineering, University of Minho, Guimarães, Portugal
[d] Department of Civil, Environmental Engineering and Architecture, University of Cagliari, Via Marengo, 2, 09123 Cagliari, Italy

ARTICLE INFO

Keywords:
Rocking
One-sided motion
Fragility curves
Horizontal restraints
Masonry façades
Intensity measures

ABSTRACT

This paper deals with the need of extending results of deterministic rocking analyses to stochastic analyses on restrained masonry façades in one-sided motion. The purpose is to define the level of improvement achieved with any anti-seismic device of a given stiffness and strength, in terms of reduction of probability of exceedance of a certain limit state. The most efficient intensity measures (IMs) are identified for three masonry façades of churches in free and restrained conditions. A reliability analysis is carried out by considering over 70 earthquakes, of which 50 recorded during the recent 2016–2017 Central Italy Earthquake. Four limit states are taken into account: rocking initiation, limited rocking, moderate rocking and near-collapse condition. The yielding limit state is considered for the analysis with anti-seismic devices. Univariate and bivariate fragility curves (FCs) are compared in free and restrained configurations, to discuss the reduction of probability of exceedance depending on 15 intensity measures. The results show that the best IMs are velocity-based parameters, in particular the Fajfar Index and Peak Ground Velocity, together with Peak Ground Acceleration. In one-sided motion without restraints, the higher the compression stiffness of the sidewalls, the more unstable the wall is in probabilistic terms. Practical curves show, for each IM, the reduction of probability of exceedance obtained thanks to assumed horizontal restraints. These help to understand, in a stochastic perspective, to what extent the anti-seismic device can be beneficial or detrimental (in case of amplifications of motion) for given earthquake intensities. The comparison of univariate and bivariate FCs confirms the superiority of bivariate FCs. Indeed, often the univariate curves sensitively underestimate the probability of exceedance, especially for low-medium intensity earthquakes, and are not able to offer any information regarding the influence of other IMs.

1. Introduction

The out-of-plane behavior of masonry walls involves the rocking motion of rigid blocks, considered as single (SDOF) or multi (MDOF) degree-of-freedom systems. The possible tools currently used to assess their seismic vulnerability are deeply discussed in [1,2], covering force-based, displacement-based, standard and rocking approaches. These procedures can be adopted to evaluate the behavior of many types of structural and non-structural elements such as parapets, gable ends, chimneys and masonry walls, often struck by earthquakes [3,4]. Observations on past earthquakes occurred in New Zealand [5] showed that the majority of parapets that exhibited no damage were properly restrained to limit out-of-plane. The risk associated to these mechanism is relevant, not only for the safeguard of human life, but also for adjacent structures. Indeed, from the observations sometimes the only damage to a structure was the fall of chimneys or parapets onto or through the roof of a parapet or neighboring buildings [5]. Other observations regarded masonry façades of churches damaged by the 2011 Christchurch Earthquake [6] and by the 2012 Emilia Romagna Earthquake [7]. Numerous uncertainties affect the MDOF models, such as the energy dissipation during each impact or the boundary conditions to assume. In order to stochastically assess the influence of anti-seismic devices, it is necessary to simplify the topic as much as possible. For instance, the incipient overturning of a masonry wall may be considered by assuming a SDOF rigid block rotating around a base hinge [8]. However, also the rocking motion of a SDOF block is strongly influenced by the restitution coefficient, related to the dissipative properties, and by the boundary conditions [9,10]. The first issue, although,

* Corresponding author.
E-mail addresses: linda.giresini@unipi.it (L. Giresini), casacla@unina.it (C. Casapulla), roman.denysiuk@algoritmi.uminho.pt (R. Denysiuk), jmatos@civil.uminho.pt (J. Matos), msassu@unica.it (M. Sassu).

https://doi.org/10.1016/j.engstruct.2018.03.003
Received 10 July 2017; Received in revised form 27 February 2018; Accepted 5 March 2018
0141-0296/

in principle, extremely relevant, is here neglected since the comparison is made between free and restrained conditions assuming for them the same restitution coefficients, the analytical one [11]. In a more realistic analysis, experimental values of restitution coefficients should be taken into account. For this purpose, several experimental tests on masonry walls confirm that generally the real value is about 90% of that analytical [12,13]. In this sense, the performed analyses are on the safe side, as a higher restitution coefficient generally implies larger rotation amplitudes. Moreover, the type of input motion plays a crucial role in the dynamic response. The earthquake that causes motion is characterized by many parameters, all of them affecting the output in a combined way. As a consequence, it is relevant to identify the most meaningful parameters to consider when the stability of the block has to be assessed.

These issues are commonly addressed by using deterministic or probabilistic approaches [1]. Based on the integration of the equation of motion, the former is generally aimed at defining the peak value of the motion amplitude that defines the stability of the block, given its deterministic geometric parameters and the type of excitation. Traditional deterministic approaches are those proposed by Makris and Konstantinidis [14], that introduced rocking spectrum as a distinct and valuable intensity measure of earthquakes. Other deterministic approaches are based on the identification of the worst input scenario corresponding to the resonant response of rigid blocks [15–21]. In this line, Casapulla et al. [15,21] and Casapulla [17] proposed a simplified representation of the seismic input as a superposition of two sequences of instantaneous Dirac impulses of acceleration and identified the resonant condition with a time interval between the main impulses coincident with the amplitude-dependent durations of the half-cycles of the motion. DeJong [22] defined a theoretical accelerogram with the condition of maximization of the input energy, dealing with a step function with alternate sign, always opposite to the current rotational velocity of the block. Alongside these studies, the probabilistic approaches attracted increasing attention over the last years. These procedures started to be applied in seismic field almost twenty years ago, with the introduction of the concept of fragility, or conditional probability of failure [23]. Sorrentino et al. [24] numerically investigated how the parameters of real seismic records (Peak Ground Acceleration, PGA, Peak Ground Velocity, PGV, Peak Ground Displacement, PGD, mechanical energy, etc.) are meaningful in defining the seismic hazard of a free rocking block. They stated that PGV is the most significant parameter as it takes into account both amplitude and frequency. Referring to fragility applied to rocking, Dimitrakopoulos and Paraskeva [25] investigated the response of rocking structures to near-fault seismic actions and highlighted the most important Intensity Measures (IMs) characterizing the rocking response and the relevance of bivariate fragility curves. PGA and Peak Total Roof Velocity (PTRV) were assumed as intensity measures in the determination of damage fragility functions for parapets and rooftop chimneys [26]. Other authors performed a seismic reliability assessment of classical columns, by using synthetic or recorded ground motions [27,28]. Generally, the adopted Engineering Demand Parameter (EDP) is the maximum amplitude ratio, whereas the peak ground velocity is one of the most reliable IMs [25]. Performance levels can then be assigned to each EDP, together with the values of the corresponding thresholds, to generate fragility curves. Therefore, these tools are more reliable than a simple deterministic analysis, mainly for the extremely high sensitivity of the response to the input motion.

In this paper, the stochastic approach is adopted to find fragility curves of masonry walls regarded as rigid blocks under one-sided rocking motion. The probability of exceedance of specific limit states in the rocking response is calculated when the walls are both in free conditions and in restrained conditions. The latter situation refers to the state of the walls strengthened by proper anti-seismic devices simulated by horizontal springs. These springs can reproduce the common steel tie-rods usually adopted as strengthening techniques for impeding or limiting the out-of-plane behavior of masonry structures. Three masonry façades of churches, struck by the 2016–17 Central Italy Earthquake, are considered as case studies. To calculate the structural reliability, there is the need of computing first the structural failure and then the failure probability. The structural failure is estimated by performing non-linear transient analyses that evaluate whether the EDP has been exceeded. The calculation is made for over 70 earthquakes, each of them defined by 15 IMs. On the other hand, the structural reliability is estimated by determining the failure probability associated to limit states that define safe and unsafe domains. Four limit states are taken into account for one-sided rocking without restraints: rocking initiation, limited rocking, moderate rocking and near-collapse condition. For one-sided rocking with restraints, only rocking initiation and limited rocking corresponding to yielding of the steel tie-rod are significant and therefore considered. The basic scheme of the structural rocking system and the three façades are presented in Section 2. The reliability analysis is illustrated in Section 3, introducing the seismic input parameters and the IMs. Section 4 discusses the correlation between EDP and IMs by analyzing Pearson's and Spearman's coefficients. Afterwards, univariate and bivariate fragility curves are obtained for the case with and without horizontal restraints acting as anti-seismic devices, and the resulting curves are compared (Sections 5 and 6). Such comparison allows quantitatively assessing the seismic improvement achieved with the anti-seismic devices in a stochastic perspective.

2. Structural assessment

2.1. Rocking analysis for one-sided motion of free and restrained walls

The considered model is a rectangular block of mass m rocking around O or O' (Fig. 1), defined in size by a semi-diagonal R that connects the center of mass to the pivot point, and in shape by the slenderness ratio α, arctangent of the ratio thickness s to height h (Fig. 1a). The boundary conditions in the rocking motion are relevant as they strongly influence the dynamic response [29–31]. The sidewalls (or transverse walls) can be modelled as spring bed with specific stiffness per unit of length. When the masonry façade rotates inward, it impacts the sidewalls (Fig. 1a). In this case, a compression stiffness has to be considered. When the wall rocks outward and is not restrained by any device, a spring bed in tension should be taken into account. Being masonry poorly resistant in tension, in this work the spring bed is supposed to be active only in compression, and its compression stiffness is [9]:

$$K_{compr} = \frac{\mathrm{E_x A}}{\mathrm{L}\bar{h}} = \mathrm{E_x}\frac{\mathrm{t}}{\mathrm{L}} \tag{1}$$

where $\mathrm{E_x}$ is the masonry elastic modulus in horizontal direction, t and $\bar{h}$ respectively thickness and effective depth of the transverse walls, whereas $\mathrm{A} = \mathrm{t}\,\bar{h}$ is the side walls cross section. This condition is called one-sided (1S) motion. When the spring bed can be assumed to act in tension, the stiffness to consider is related to the dissipated energy due to friction in the sidewalls [32]. The restrained configuration is that where steel-tie rods tend to recenter the wall in outward motion. The steel tie-rods, modelled as individual springs (b), are supposed to be active only in tension (Fig. 1b).

The full equation of motion of the non-smooth contact problem is [9]:

$$I_0\ddot{\vartheta} + sgn(\vartheta)mgR\sin A_\vartheta + sgn(\vartheta)K\beta^2R^2\cos A_{r,\vartheta}[\sin\alpha_r - \sin A_{r,\vartheta}] + sgn(\vartheta)K_{compr}\bar{h}\left(A + \frac{B\bar{h}}{2} + \frac{C\bar{h}^2}{3}\right) - m\ddot{u}_g R\cos A_\vartheta = 0 \tag{2}$$

where $A_\vartheta = \alpha - sgn(\vartheta)\vartheta$ and $A_{r,\vartheta} = \alpha_r - sgn(\vartheta)\vartheta$. I_0 is the polar inertia moment with respect to O (or O′), $I_0 = \frac{4}{3}\mathrm{m}(\mathrm{h}^2 + \mathrm{s}^2) = \frac{4}{3}\mathrm{mR}^2$, for perpendicular blocks. However, inertia moments have to be calculated from the actual geometry of the masonry façade as it is. α_r is the single