Expeditious methods for seismic assessment of pre-code masonry buildings in portugal

Métodos expeditos para avaliação sísmica de edifícios existentes de alvenaria em portugal

Vasco Bernardo
Alfredo Campos Costa
Paulo Candeias
Aníbal Costa

Abstract

The seismic assessment of existing buildings became mandatory and based on the procedures and requirements included in NP EN 1998-3: 2017 (Portuguese version of Eurocode 8 – part 3), which establishes the performance requirements and compliance criteria for existing buildings subjected to a certain level of seismic action. According to this normative requirements, analytical seismic vulnerability assessment and reliability-based analyses were carried out on a large set of masonry buildings representative of the Portuguese housing stock, leading to the development of surrogate and expeditious method for seismic assessment in compliance with the reference method defined in the European standard. The method allows the seismic assessment of masonry buildings with rigid and flexible floors, without explicit numerical analyses and using only geometric parameters and the material properties.

Resumo

A avaliação da segurança sísmica de edifícios existentes de alvenaria tem como referência os procedimentos dispostos na NP EN 19983:2017 (Anexo C) e o respetivo Anexo Nacional, que estabelecem os requisitos de desempenho e os critérios de conformidade para edifícios existentes sujeitos a um determinado nível de ação sísmica. No seguimento das exigências regulamentares, e à luz da verificação da segurança à ação sísmica preconizada na norma, realizaram-se análises probabilísticas de vulnerabilidade sísmica e fiabilidade estrutural a um grande conjunto de edifícios de alvenaria representativos do parque habitacional, que conduziram ao desenvolvimento de métodos expeditos para a avaliação sísmica em alternativa à verificação pelo método de referência. Os métodos propostos permitem avaliar a resistência sísmica de edifícios de alvenaria com pavimentos rígidos e flexíveis, sem recurso a análises numéricas e recorrendo apenas a parâmetros geométricos e propriedades mecânicas dos materiais.

Keywords: Pre-code masonry buildings / Seismic safety / Seismic vulnerability assessment / Probabilistic methods / Expeditious methods

Palavras-chave: Edifícios existentes de alvenaria / Segurança sísmica / Avaliação da vulnerabilidade sísmica / Métodos probabilísticos / Métodos expeditos

Vasco Bernardo

Bolseiro doutoramento
Laboratório Nacional de Engenharia Civil
Lisboa, Portugal
vbernardo@lnec.pt

Alfredo Campos Costa

Investigador principal
Laboratório Nacional de Engenharia Civil
Lisboa, Portugal
alf@lnec.pt

Paulo Candeias

Investigador principal
Laboratório Nacional de Engenharia Civil
Lisboa, Portugal
pcandeias@lnec.pt

Aníbal Costa

Professor Catedrático
Universidade de Aveiro
Aveiro, Portugal
agc@ua.pt

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1 Introduction

In the last decades, the performance of masonry buildings under seismic actions deserved special attention due to the increasing public awareness related to the protection of human life and architectural heritage, and the use of this material particularly unsuitable in seismic zones, as has been investigated by several authors[1]–[6].

According to the latest national housing census[7], the Portuguese building stock is constituted by approximately 45% of masonry residential buildings. Moreover, the vast majority have been built mostly to withstand gravity loads and the impact of earthquakes has not been considered in their design, before the first Portuguese seismic design regulation (RSCCS[8]) enforced in 1958. It is also worth pointing out that many of these buildings need maintenance or deeper interventions due to the state of degradation or adulteration of the original structure which contributed to the reduction of the building strength. A considerable amount of research has been published addressing their seismic vulnerability in the last few years, indicating an unsatisfactory structural behavior[5], [6], [9]–[12]. In order to regulate the rehabilitation of buildings, a new law (Decree-law no. 95/2019 of 18 July) was approved, adopting specific measures to be taken in several aspects, including seismic safety. In this context, the seismic assessment of existing buildings became mandatory with the ministerial order no. 302/2019 of 12 September. In Portugal, the NP EN 1998-3:2017 (Portuguese version of Eurocode 8 – part 3: Assessment and retrofitting of buildings) and the respective Annex C, hereinafter EC8-3[13], regulates the seismic assessment of masonry buildings with different approaches, as summarized in Candeias *et al.*[14].

In this framework, the present work aims to develop an expeditious method for the seismic assessment of pre-code masonry buildings. The purpose of the expeditious method is to provide a relatively fast approach for the seismic assessment of masonry buildings, in compliance with the European standard (EC8-3), that can be easily applied by the technical community. The method proposed does not require explicit numerical analyses to be carried out, only simple characteristics like the building geometry and the material properties. The method was developed for masonry residential buildings with rigid and flexible floors and up to five stories high, which represent the vast majority of masonry buildings in Portugal[7]. Furthermore, the method is also restrained to buildings with regularity characteristics in plan and in elevation, as specified in the standard.

The methodology follows the SAC/FEMA probabilistic approach (Cornell *et al.*[15]; FEMA-350[16]) properly adapted to the context of the performance-based seismic assessment adopted in EC8-3. The methodology was employed to a synthetic database of 18.000 buildings generated to represent the variability in the Portuguese housing stock, considering different archetypes and to account for the material properties uncertainty. Based on these results the seismic demand was estimated and calibrated using the reference method of EC8-3 for an in-plane global seismic verification. The capacity of the buildings in the database was then estimated by the expeditious method, employing empirical expressions that quantify the in-plane strength of the masonry structural elements, walls and piers, and corrected through a surrogate model developed to account for other nonlinear effects. Finally, the assertiveness of the method and its code compliance was verified using confidence tests, comparing the assessment results of the entire database computed by the expeditious method and the ones obtained applying the reference method EC8-3.

Although the methodology has been developed and presented for the Portuguese building stock and geared to the national seismic hazard, it can be adapted for other scenarios and other countries.

Further information regarding the development of expeditious methods for seismic assessment of pre-code masonry buildings in Portugal can be consulted in [17].

2 Representative geometry of the buildings

The structural typologies used in the current study were based on the geometrical information collected from the original detailed drawings (blueprints) and design notes, consulted in the municipal archives. The data refers essentially to masonry buildings built between the end of 19th century and the enforcement of the first Portuguese seismic code in 1958. The buildings surveyed were randomly selected for a population of 100 masonry buildings up to 5 stories high, which represent the vast majority of the buildings in that period, according to the latest national housing census[7]. The geometric parameters collected, such as the plan dimensions, stories height, wall openings ratio, interior walls density, walls thickness and type/thickness of floors, were statistically characterized and described in Bernardo *et al.*[18]. Based on the statistics reported in that study, nine representative archetypes were defined (A1, A2, A3, B1, B2, B3, C1, C2, C3) to cover the geometrical variability of a common urban building blocks, as shown in Figure 1.

The plan dimensions of archetypes are represented by the 16th, 50th and 84th quantiles, wherein the "B2" archetype corresponds to the mean size (12.6 × 12.1 m). The total area in plan ranges between 60.0 m^2 to 285.0 m^2. The layout for the interior walls respects the corresponding mean walls density, equal to 0.054, and the typical size of the compartments observed in these buildings (3 × 3 m up

Table 1 Statistical properties for the geometric parameters [18]

Moments	L_x [m]	L_y [m]	IWD [-]	H_0 [m]	H_i [m]	OR_F [-]	OR_B [-]	Th_1 [m]	Th_2 [m]	Th_3 [m]	Th_4 [m]	AWTR [-]
Mean	12.6	12.1	0.054	3.23	3.01	0.23	0.21	0.47	0.34	0.21	0.14	0.11
Std. deviation	5.00	4.1	0.01	0.42	0.24	0.08	0.08	0.14	0.11	0.05	0.02	0.06
mode	–	–	–	–	–	–	–	–	–	0.25	0.15	0.10

L_x and L_y size; H_0 and H_i – ground and upper floor stories high;
OR openings ratio: front (OR_F) and back (OR_B) facade;
WD interior walls density;
Th walls thickness: facades (1), lateral side (2), interior (3), partition (4);
AWTR average walls thickness reduction on the facade in height.

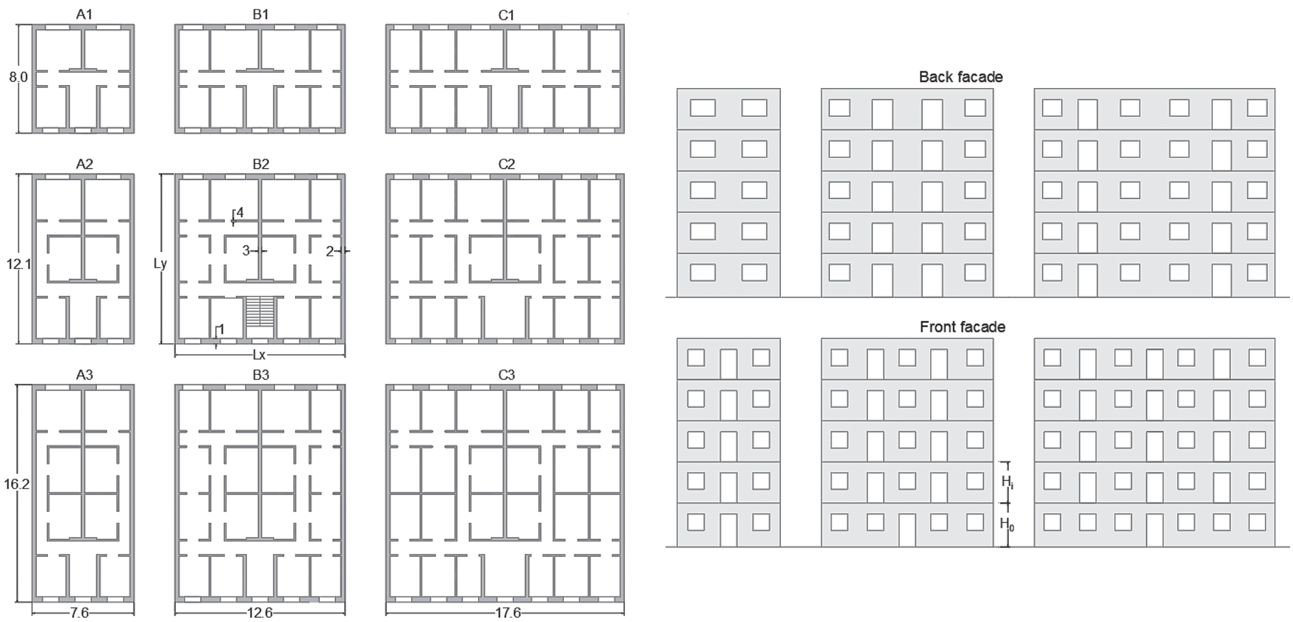

Figure 1 Archetypes adopted to represent the population of pre-code masonry buildings in Portugal

Table 2 Characterization of random variables for masonry mechanical properties

Random variable	Distribution	COV	Mean value			
			Type I-	1Type I-2	Type II-1	Type II-2
Compressive strength fc [MPa]	LogNormal	0.40	5.00	2.00	2.50	1.25
Factor K* [-]	Truncated Normal 0.25		800 (250 - 1100)			
Young's modulus E [MPa]	–	–	4000	1600	2000	1000
Shear modulus G [MPa]	LogNormal	0.40	1700	650	850	450
Density ρ [kg/m^3]	Normal	0.10	1800	1200	1800	1200
Cohesion T_0 [MPa]	LogNormal	0.40	0.15	0.15	0.07	0.07
Friction coefficient μ** [-]	LogNormal	0.40	0.40	0.40	0.40	0.40

* factor K correlates the Young's modulus and compressive strength: $E = K \cdot f_c$
** according to EC8-3

to 4 × 5 m), as defined by Bernardo *et al.*[18]. Regarding the walls thickness, a mean value was adopted for the facades and side walls while for the interior/partition walls was considered the modal value, which represents more than 60% of the buildings surveyed. The uncertainty in the walls thickness was tackled through the uncertainty in the material properties (see Section 3). This option is due to the lack of information in the original drawings related to the type of masonry which did not allow to stratify the sample, as discussed in Bernardo *et al.*[18].

Table 1 summarizes the geometrical parameters adopted (underlined) to characterize the archetypes.

3 Material properties definition

The masonry buildings present an enormous variability in the mechanical properties which reflects the differences between type of material, arrangement and state of conservation. Several works can be found in the literature regarding the mechanical properties of Portuguese masonry building stock (e.g., irregular limeston [19]-[24]; granite[25], [26]; brick masonry with lime mortar[23], [24], [27]-[29]; brick masonry with cement mortar[5], [30]). Moreover, the latest version of EC8-3 also suggests the mechanical properties for different types of masonry (see Candeias *et al.*[14]).

To cover the wide range of masonry mechanical properties found in the literature review, two main classes of typologies were considered: *Type I* – buildings with good quality masonry (e.g., regular and squared masonry, brick masonry with cement lime mortar) or in good state of conservation; *Type II* – buildings with poor quality masonry (e.g., rubble stone masonry, brick masonry with lime mortar) or in poor state of conservation. Given the differences found between interior walls (e.g., *tabique*[1], *frontal walls*[2], perforated brick masonry) and exterior walls (e.g., solid masonry bricks, stone masonry or concrete blocks), two subcategories were defined: *Type I-1* and *Type II-1*, to represent the properties of exterior walls; Type I-2 and *Type II-2* for the interior/partition walls.

Table 2 summarizes the distributions adopted related with the four classes of random variables (r.v.), associated to each mechanical property. The uncertainty was propagated through Monte Carlo simulations considering independent r.v. constituted by only six sets of 100 samples. The same set of r.v. was applied to each class and for all archetypes previously defined, meaning that each archetype contains 200 different structures.

4 Numerical modelling strategy

In order to simulate the nonlinear response of the buildings, tridimensional multi-degree-of-freedom models were built (see Figure 2), based on an equivalent frame modeling strategy available in the research version of TREMURI software[31]. The software was developed to analyze the global behavior of unreinforced masonry buildings considering only the in-plane behavior of the walls. On the other hand, the current version of EC8-3 does not include the out-of-plane mechanisms, assuming that these mechanisms are prevented from occurring (i.e. tie rods).

For the in-plane response of the walls that software adopts a frame-type representation, wherein each wall with openings is modeled by assembling the piers and the spandrel beams (macroelements), connected by rigid area (nodes), as shown in Figure 2. The nodes are non-deformable, while the inelastic response is governed by the nonlinear deformations of the macroelements. The shear response of panels is controlled by an equivalent shear model that use the Mohr-Coulomb criterion, while the flexural-rocking of the panels are represented by a unilateral contact model with zero-length springs, located at the interface of the element, that follows a bilinear constitutive model in compression and no strength capacity in tension. Further details related to the formulation can be consulted in Lagomarsino *et al.* 2013[31].

Regarding the floors, two types of horizontal diaphragms were considered: rigid[5] and flexible[6]. The diaphragms were modelled through a two-dimensional orthotropic membrane element,

1 Set of vertical long boards connected by horizontal small wood stripes, normally filed with pieces of bricks and lime mortar

2 Set of plane wood trusses

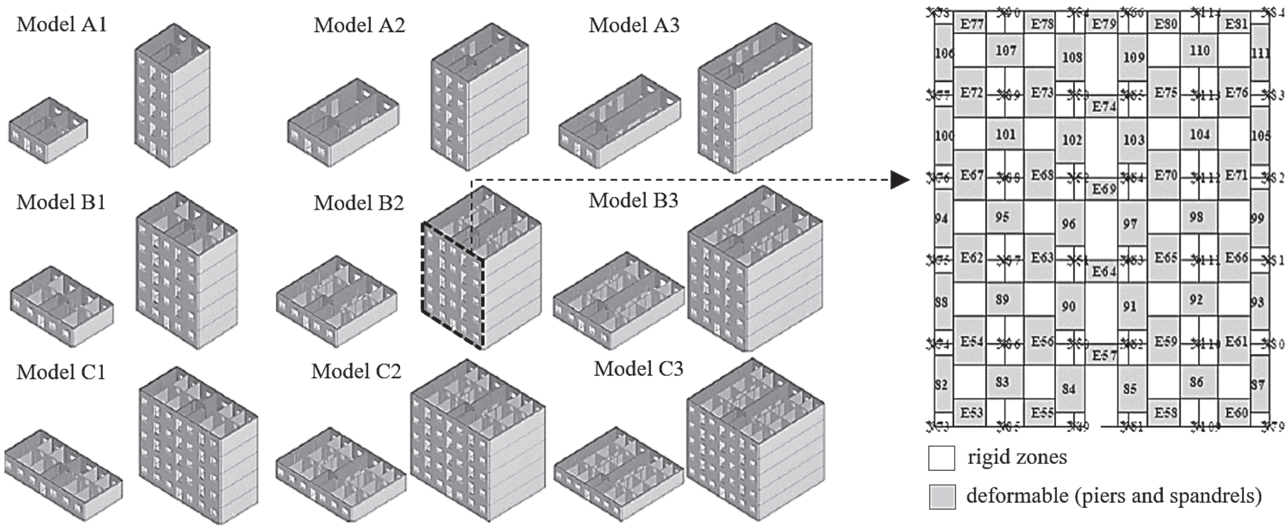

Figure 2 Macroelement models of archetypes (left) and example of equivalent frame model for the facade wall (right)

defined by equivalent mechanical properties: equivalent thickness (*t*), modulus of elasticity of the diaphragm parallel to the principal direction (E1) and perpendicular (E_2), shear modulus (*G*) and Poisson coefficient (ν). In Table 3 are listed the mechanical properties adopted in this study.

Table 3 Table 3 Mechanical properties for horizontal diaphragms

Diaphragm	t [m]	E1 [GPa]	E2 [GPa]	G [GPa]	ν [-]	Reference
Rigid	0.20	30.0	30.0	13.0	0.20	Milosevic[5]
Flexible	0.022	29.0	12.0	0.011	-	Simões[6]

5 Definition of Seismic action

The seismicity in Portugal is controlled by two main seismic scenarios characterized by their magnitude, epicenter location, event duration and frequency content: (i) interplate scenario – offshore epicenters, high magnitude, long duration and lower frequency content; (ii) intraplate scenario – mainly occurring inland, moderate magnitudes, short duration and higher frequency content.

In order to provide an expeditious method fully compatible with the current national standards, the seismic action at the ground surface was modelled through the elastic response spectrum defined in EC8-1. Considering both scenarios, two main shapes for the elastic spectrum are defined in the Portuguese National Annex of EC8-1, resulting in the two seismic zonings (Type 1 – interplate/offshore and Type 2 – intraplate/onshore). The seismic zones 1.1 to 1.6 and 2.3 to 2.5 corresponds to the Portugal mainland, respectively offshore and onshore scenario; 2.1 and 2.2 the Azores islands (onshore).

6 Performance-based assessment

The seismic assessment methodology follows the performance-based approach specified in the EC8-3 towards a global safety verification in-plane. The capacity of the structures was assessed from nonlinear static analyses (*pushover*), considering an inverted pseudo-triangular load-pattern for buildings with flexible diaphragms, and an adaptive pushover with inverse triangular first mode pattern for rigid floors. These assumptions revealed to be the most adequate for these building typologies as investigated by Galasco *et al.*[32] and Lagomarsino *et al.*[33]. The control node was selected at the top level and the shear force was measured on the base up to reaching 20% decay of the maximum shear strength, as recommend by the EC8-3. Figure 3 show the capacity curves in spectral acceleration (Sa) and spectral displacement (Sd) for the archetypes B2; three to five stories high; typology I and II; rigid and flexible diaphragms.

The seismic demand was estimated based on the N2 Method[34] iterative procedure recommended in the Appendix B of EC8-1. In short, the response of the structures is obtained from the intersection of the capacity curve with the elastic response spectra, in acceleration-displacement response spectrum (ADRS) format. The dynamic properties of the structures are used to convert the multi degree of freedom (MDOF) capacity curves into a bilinear equivalent single degree of freedom (SDOF) system[35], assuming an elastic-perfectly plastic force-displacement relationship and incorporates the inelastic response spectrum based on structure's ductility[36].

The methodology was applied to a synthetic database of 18.000 buildings: 9 archetypes of buildings up to 5 stories high with rigid and flexible diaphragms with 200 random variables. The structures were analyzed considering different seismic zones and a wide range of return periods up to 5000 years, which allow to compute the hazard function that will be derived in section 8. The spectral ground acceleration a_g for a given return period Tr was obtained as suggested in the Portuguese version of EC8-1: $a_g = a_{gr}\ (T_{ref}/T_r)^{-1/k}$, wherein a_{gr} corresponds to the ground acceleration for the reference return period T_{ref} and k takes the values of 1.5 (Type 1), 2.5 (Type 2) and 3.6 for Azores islands (Type 2 – 2.1 and 2.2).

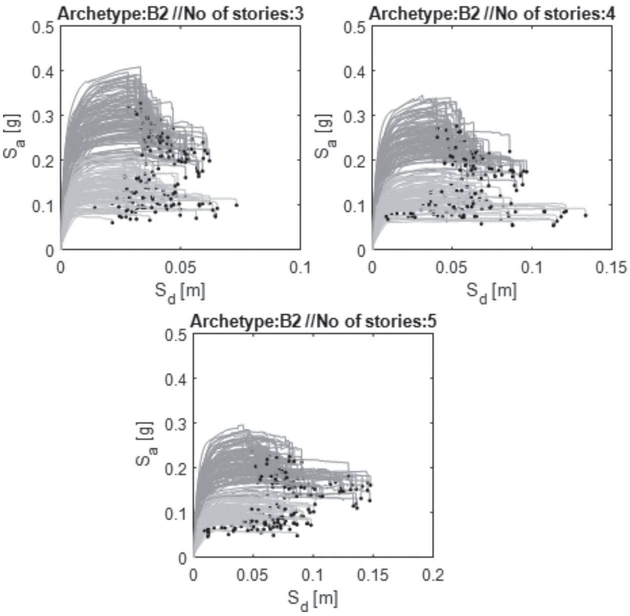

Figure 3 Example of capacity curves for buildings with rigid diaphragm – archetypes B2

7 Limit state and fragility curves

To develop an expeditious method for seismic assessment under the current framework, the limit state assumed corresponds to the maximum strength of the structures, i.e., the yielding point (S_{ay}, S_{dy}) of the bilinear elastic-perfectly plastic capacity curve, defined according to the N2 method bilinearization process of the original nonlinear capacity curves.

This option is based on the need to develop an expeditious method that relates the global strength of the building to the strength capacity of each wall in the seismic direction. Moreover, this limit state is associated to the "damage limitation" requirements[13], corresponding to "immediate occupancy", that makes more sense in the framework of expeditious assessment.

The fragility curves presented in Figure 4 and expressed as a function of maximum spectral acceleration, were best fitted to the empirical cumulative distribution function, based on Kolmogorov-Smirnov tests[37]. The analytical fragility functions and the computed dispersion were used to derive the building's capacity distribution on the seismic reliability analysis carried out in the following section.

The archetypes were grouped given the similarities found in the maximum spectral acceleration between different archetypes, which reflects the lower discrepancies in the ratio of masonry walls area to the area of floors in plan, as already mentioned in Bernardo *et al.*[18]

The fitted analytical fragility curves follow a log-normal cumulative distribution function, with standard deviation of the natural logarithm between 0.13 to 0.18 (Typology I-rigid); 0.22 to 0.26 (Typology II-rigid); 0.14 to 0.17 (Typology I-flexible); 0.18 to 0.24 (Typology II-flexible). Figure 4 shows that, as the buildings become higher, median values of Sa tend to decrease, as expected, and the efficiency of rigid to flexible floors are less pronounced.

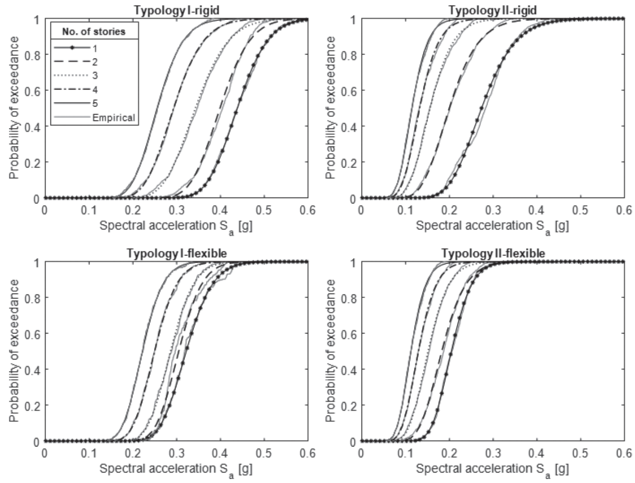

Figure 4 Fragility curves by number of floors, typology and type of floor (rigid and flexible)

8 Seismic reliability analysis

The reliability analysis was performed individually for the entire population of buildings generated. The mean annual frequency λ, see Equation (1) adapted from Cornell *et al.*[15], of reaching or exceeding a given limit-state was computed assuming that the maximum capacity Sa_{max}, corresponding to the yielding point of the elastic-perfectly plastic idealized capacity curve, cannot be exceeded. Thus, the upper bound of the convolution integral is constrained to Sa_{max} and C is the normalizing constant of the truncated probability density function of capacity lognormally distributed, expressed as a function of the spectral acceleration with mean μ and standard deviation σ of the logarithmic values.

$$\lambda = \int_0^{Sa_{max}} \frac{\overbrace{\phi_{LN}(x,\mu,\sigma)}^{capacity}}{C} \overbrace{H(x)}^{azard}\, dx = \int_0^{Sa_{max}} \frac{1}{C\sqrt{2\pi}x\sigma} e^{-\frac{1}{2}\left(\frac{\ln(x)-\mu}{\sigma}\right)^2} \left(k_0 x^{-k}\right) dx$$

where (1)

$$C = \Phi\left[\frac{\ln(Sa_{max}) - \mu}{\sigma}\right]$$

Note that the intercept k_0 and the slope k are the parameters to define the shape of the hazard curve $H(x)$ in the bi-logarithmic scales and for a first order power law approximation according to the code. In fact, assuming an idealized capacity curve with linear elastic branch up to the yielding point, the intercept k_0 of the hazard function can be easily obtained from the structure's performance, considering for example, a low return period (e.g., 1-year return period). Figure 5 shows the hazard functions and the capacity curve for a given building sample of the database, with three stories high, equivalent period T_{eq} 0.31s and Sa_{max} = 0.44g. For convenience, the hazard curves are presented in linear scale. The convolution to mean annual frequency λ is also presented for all seismic zones. As can been seen, the hazard functions distributions of both seismic scenarios considered for Portugal are quite different. The mean annual frequency is higher for the interplate seismicity of Azores (seismic zone 2.1 and 2.2) and for south of Portugal mainland (seismic zone 1.1 and 1.2). For the south to north of mainland Portugal the λ decreases one order of magnitude, which completely reflets, in general, the geographical distribution seismicity in Portugal.

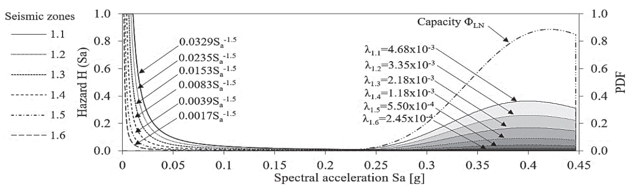

Figure 5 Example of mean annual frequency for a three stories high building, seismic zones 1 and ground type A

9 Derivation of THE expeditious method

The expeditious method proposed is based on the comparison between the seismic demand and the buildings capacity, both evaluated in terms of horizontal seismic coefficient C_S, to be easily estimated combining the material properties and the building's geometry with the proposed expressions provided in section 9.2.

9.1 Seismic demand and calibration of the method

Based on the previous analyses, the reliability index β for all buildings was computed using the inverse standard normal cumulative density function $\beta = -\Phi^{-1}(\lambda)$ and then related with the maximum base level seismic coefficient C_S. Figure 6 present the relationship between β value for each structure against C_S for Portugal mainland – offshore (k = 1.5) and onshore (k = 2.5) earthquake scenarios, respectively, and for the specific cases of buildings with 3 to 5 stories high, rigid and flexible floor and ground type A. As shown in both figures, a power law function was fitted to describe the relation between the C_S and β, reaching a reasonable range of values for the coefficient of determination between 0.91 and 0.99. Note that, onshore seismic source leads to higher dispersion due to the response spectrum shape, wherein the horizontal acceleration spectral branch is shorter, compared to the offshore scenario.

To develop an expeditious method compatible with EC8-3, the 18.000 buildings in synthetic database were assessed, applying the global safety verification preconized by the code (EC8-3 – section C.4.1), and the reliability index for each one was obtained. More explicitly, the global seismic assessment by the EC8-3 (C.4.1) consists in comparing the structure's performance point for a 308-years return period (15% probability of exceedance in 50 years) to the global displacement for the Significant Damage (SD) limit state that corresponds to 75% of the Near Collapse (NC) displacement, i.e., the displacement (recommended at roof level) for a 20% decay of the maximum base shear strength. The adoption of the SD limit state refers to importance class II (ordinary buildings) in line with the global assessment procedures of EC8-3 (Annex C). A gray circle in Figure 6 represents the result of the reliability-based analyses and the relation with the seismic coefficient, for a given structure in the database generated. On the other hand, the same database was assessed by the reference method (EC8-3), thus a black dot in the center of a gray circle corresponds to a given structure that does not verify the seismic safety according to the code (EC8-3).

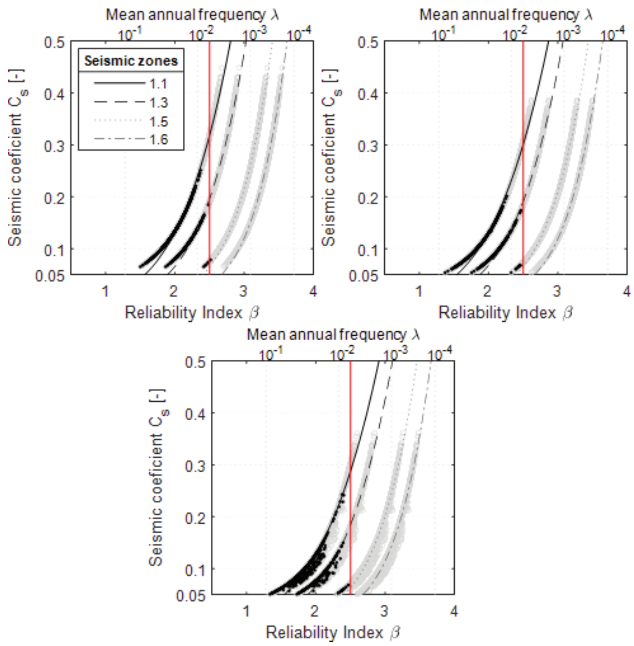

Figure 6 Relation between seismic coefficient and reliability index for offshore scenario (seismic zone 1.1, 1.3, 1.5 and 1.6) and ground type A: buildings with rigid (3, 4 and 5 floors)

Based on these results, a global uniform reliability index of 2.5 (vertical line in Figure 6) was adopted to provide the minimum seismic coefficient required to verify the seismic safety for a given building located in a specified seismic zone and ground type. Naturally, the expeditious method should guarantee a certain level of safety and therefore the 2.5 reliability index and the corresponding C_S, tackled as a demand $C_{S,D}$, must reflect a lower bound exceedance probability for all the buildings. The selection of β = 2.5 results in a minimum 0.95-quantile, meaning that only a maximum of 5% of the structures in the synthetic database do not verify the seismic safety for the seismic coefficient demand at the 308-years return period performance point. In fact, EC8-3 noncompliance black dots situations, depicted in those figures, shows that the same return period adopted for all seismic zones (and ground type) do not assure a uniform reliability index for all seismicity levels of the code, being more noticed for offshore seismic zones.

Table 4 Proposed seismic coefficient demand $C_{S,D}$ for Portugal

Seismic zone	Number of stories - rigid diaphragms														
	1			2			3			4			4		
	Soil A	Soil B	Soil C	Soil A	Soil B	Soil C	Soil A	Soil B	Soil C	Soil A	Soil B	Soil C	Soil A	Soil B	Soil C
1.1	0.36	4.48	0.52	0.33	0.42	0.48	0.31	0.39	0.44	0.29	0.36	0.40	0.28	0.34	0.38
1.2	0.29	0.39	0.46	0.26	0.35	0.40	0.25	0.33	0.37	0.24	0.31	0.35	0.23	0.30	0.33
1.3	0.21	0.29	0.35	0.19	0.27	0.31	0.19	0.25	0.30	0.18	0.24	0.28	0.18	0.23	0.27
1.4	0.12	0.18	0.22	0.12	0.17	0.21	0.12	0.17	0.20	0.12	0.16	0.20	0.11	0.16	0.19
1.5	0.06	0.09	0.12	0.06	0.09	0.11	0.06	0.09	0.11	0.06	0.09	0.11	0.06	0.09	0.11
1.6	0.03	0.05	0.06	0.03	0.05	0.06	0.03	0.05	0.06	0.03	0.05	0.06	0.03	0.05	0.06
2.1	0.33	0.44	0.51	0.29	0.38	0.44	0.23	0.30	0.34	0.17	0.23	0.27	0.14	0.19	0.22
2.2	0.29	0.39	0.45	0.25	0.34	0.39	0.20	0.27	0.31	0.15	0.21	0.24	0.12	0.16	0.19
2.3	0.34	0.45	0.51	0.30	0.38	0.42	0.24	0.30	0.34	0.18	0.24	0.27	0.14	0.19	0.22
2.4	0.20	0.29	0.35	0.18	0.26	0.31	0.15	0.21	0.25	0.11	0.16	0.19	0.18	0.12	0.15
2.5	0.13	0.20	0.25	0.12	0.18	0.22	0.10	0.15	0.18	0.07	0.11	0.13	0.15	0.08	0.10

Seismic zone	Number of stories - flexible diaphragms														
	1			2			3			4			4		
	Soil A	Soil B	Soil C	Soil A	Soil B	Soil C	Soil A	Soil B	Soil C	Soil A	Soil B	Soil C	Soil A	Soil B	Soil C
1.1	0.31	0.41	0.47	0.28	0.36	0.40	0.24	0.31	0.35	0.21	0.27	0.30	0.18	0.24	0.26
1.2	0.27	0.37	0.42	0.24	0.32	0.37	0.22	0.28	0.32	0.18	0.24	0.27	0.16	0.21	0.24
1.3	0.20	0.28	0.33	0.18	0.25	0.29	0.16	0.22	0.26	0.13	0.18	0.22	0.11	0.16	0.19
1.4	0.12	0.18	0.21	0.11	0.16	0.20	0.10	0.14	0.17	0.08	0.12	0.14	0.07	0.10	0.12
1.5	0.06	0.10	0.12	0.06	0.09	0.11	0.05	0.08	0.10	0.04	0.06	0.08	0.04	0.06	0.07
1.6	0.03	0.05	0.06	0.03	0.05	0.06	0.03	0.04	0.05	0.02	0.03	0.04	0.02	0.03	0.04
2.1	0.21	0.29	0.33	0.14	0.19	0.23	0.10	0.15	0.18	0.08	0.12	0.15	0.07	0.10	0.13
2.2	0.18	0.25	0.29	0.11	0.16	0.20	0.08	0.12	0.15	0.07	0.10	0.12	0.06	0.09	0.11
2.3	0.21	0.28	0.32	0.13	0.19	0.22	0.10	0.14	0.17	0.08	0.12	0.14	0.07	0.10	0.12
2.4	0.12	0.18	0.21	0.07	0.11	0.14	0.05	0.18	0.10	0.04	0.06	0.08	0.03	0.05	0.07
2.5	0.08	0.13	0.16	0.05	0.08	0.10	0.03	0.05	0.07	0.03	0.04	0.06	0.02	0.04	0.05

Table 4 summarizes the proposed seismic demand in terms of seismic coefficient $C_{S,D}$ for both rigid and flexible diaphragms of buildings up to 5 stories high, all seismic zones, ground type A, B and C. The seismic demand coefficient $C_{S,D}$ values in Table 4 display the same trend stated before regarding the seismic intensities' levels for different regions considering the assumption of a reliability index of 2.5, independently of all seismic zonation and ground type. Therefore, levels of seismicity controls to a large amount differences of $C_{S,D}$ values proposed. Another interesting point, which apparently does not make sense, is the lower $C_{S,D}$ demand values observed for structures with flexible diaphragms comparing to the correspondent demand values of rigid floor. Ultimately, these lower values result from the in-plane behavior of those diaphragms, which does not allow a full exploitation of the total shear capacity of walls.

9.2 Surrogate model for the structural capacity

For the development of an expeditious surrogate procedure to evaluate the seismic safety of a given masonry building, the seismic coefficient demand $C_{S,D}$ (see section 9.1), must be lower or equal than the estimated capacity value, also expressed as a seismic coefficient $C_{S,C}$, in the framework of this study.

The $C_{S,C}$ values should be computed according to the empirical Equations (2), (3) and (4). These equations account the most common in-plane failure mechanisms of masonry walls[38]-[40]: flexural V_{flk}, diagonal shear V_{cdk} and sliding V_{clk}. Furthermore, they are also on the basis of the safety verification in local terms proposed by the EC8-3 considering the different types of elements failing. Thus, the horizontal capacity $V_{H,i}$ of a given masonry wall i is the minimum value obtained from the expressions:

$$V_{flk,i} = \frac{\sigma_0 \cdot t \cdot l^2 \left(1-1.15\cdot\frac{\sigma_0}{f_k}\right)}{2\cdot\alpha\cdot h} \tag{2}$$

$$V_{cdk,i} = l\cdot t\cdot \frac{f_{tk}}{b}\cdot\sqrt{\frac{\sigma_0}{f_{tk}}+1} \tag{3}$$

$$V_{clk,i} = l \cdot t \cdot \left(f_{v0k} + \mu \cdot \sigma_0 \right) \tag{4}$$

where t, l, h are the geometric parameters that correspond to the thickness, length and height of the wall, respectively; the parameter α takes into account the boundary conditions; b is a factor that accounts for the strength distribution on the wall[40]; σ_0 the axial stress; f_k, f_{v0k}, f_{tk} and μ are strength parameters of the masonry, respectively, compression, cohesion, diagonal tensile and friction coefficient.

Finally, the seismic coefficient capacity $C_{S,C}$ for a given building should be estimated considering the capacity of the walls in the seismic load direction, divided by the total mass of the building W_E (nominal values of permanent loads combined with the quasi-permanent live loads). The measurement criteria adopted to determine the seismic capacity coefficient $C_{S,C}$ at the base level only accounts the contribution of the walls on the seismic direction, excluding the walls segments with doors openings. In case of windows openings, is only accounted the contribution of the initial shear strength (cohesion) on the portion of the walls below the windows at base level.

$$C_{S,C} = \frac{\sum_{i=1}^{n} V_{H,i}}{W_E} = \frac{\sum_{i=1}^{n} \min\left(V_{flk,i}, V_{cdk,i}, V_{clk,i} \right)}{W_E} \tag{5}$$

Note that the capacity of buildings evaluated by the previous equations are not feasible and overestimated. In fact, these empirical expressions assume that the maximum capacity of the walls could be reached at the same time, and do not consider the re-distribution of the seismic loads, or even, consider the nonlinear strength degradation of the walls.

To overcome those limitations, surrogate models (for rigid and flexible floor diaphragms) were developed to consider the simple practical assessment through Equation (5). The models were derived based on empirical relationship between values of seismic coefficients achieved in the numerical predictions C_S of the entire population of buildings in the synthetic database and the corresponding ones obtained from the above equations for $C_{S,C}$, as depicted in Figure 7. A surrogate model, defined as a power law function with a shape given by $y = a \cdot x^{(b-c \cdot x)}$, was capable to reproduce a similar trend observed in that figure. A median curve was best fitted to the data using a nonlinear least squares method (Levenberg–Marquardt algorithm[41]) to describes the empirical relation between $C_{S,C}$ and C_S for rigid and flexible floor diaphragms.

Thus, the seismic coefficient $C_{S,C}$, obtained by means of Equation (5), must be corrected ($C^*_{S,C}$) and only then should be compared with the seismic demand $C_{S,D}$. The epistemic nature of the uncertainties related to this surrogate model can also be tackled by considering lower quantile rather than median curve, as shown in Figure 7. Notice that lower quantiles in figure should not be reinterpreted as factors associated to the confident levels of EC8-3. As a matter of fact, they only reflect the epistemic uncertainties associated to the surrogate expeditious model developed in this study.

In order to assess the accuracy and applicability of the proposed expeditious method, the buildings generated in the scope of the reliability analyses were assessed according to different methods, namely the method corresponding to the procedures defined in EC8-3 and the expeditious method above presented in last section considering the median curve of the surrogate model. Further details can be consulted in Bernardo *et al.* 2022.

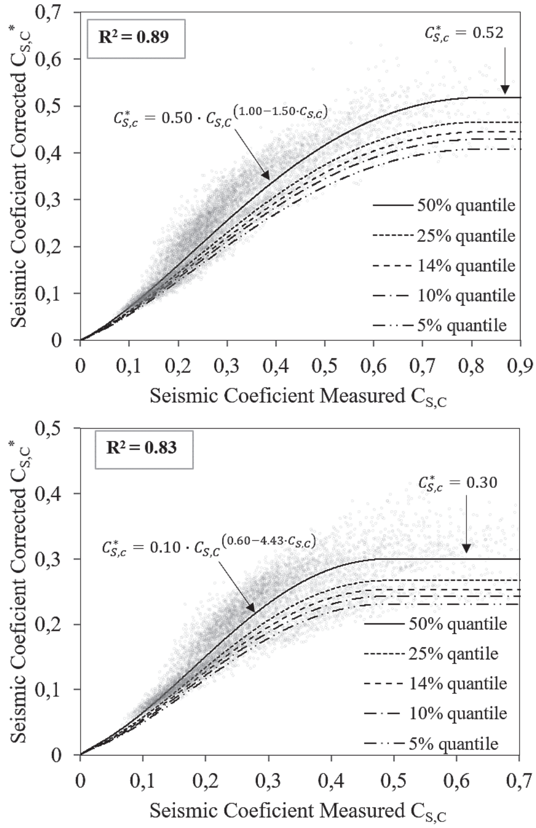

Figure 7 Correction of seismic capacity $C_{S,C}$ to account nonlinear effects: rigid floors(left) and flexible floors (right)

Finally, the practical application of the proposed expeditious method is summarized in the following main steps: (i) building survey (e.g., building geometry, wall's thickness, material properties, building mass); (ii) estimate the seismic capacity coefficient $C_{S,C}$ at base level through the application of Equation (5), considering only the in-plane behavior of masonry walls in the seismic direction analyzed; (iii) correction of the $C_{S,C}$ *to* ($C^*_{S,C}$) according to the type of floor diaphragms (see Figure 11), to account for the nonlinear behavior of masonry and seismic load re-distribution; (iv) definition of seismic demand coefficient $C_{S,D}$ (function of seismic zone, number of stories high and ground type – see Table 5); (v) compare the coefficients computed in (iii) and (iv) in order to conclude the seismic safety assessment procedure of the building ($C^*_{S,C}) \geq C_{S,D}$).

10 Final comments and Conclusions

The aim of the present paper was to derive an expeditious method for seismic assessment of pre-code masonry buildings with rigid and flexible diaphragms. The method has been developed to assess, in practice, the seismic coefficient capacity of a given masonry building, in a relatively simple way and in compliance with the reference method (EC8-3). However, its application is restricted to buildings that observe certain conditions of regularity in height and in plan as specified in the standard (EC8-3). The methodology was developed in order to achieve a full compatibility with the standard specifications though out the entire seismicity for Portugal.

In this framework, a synthetic database of 18.000 buildings up to 5 stories high was generated, which include a set of archetypes defined based on previous statistical information[18] collected and different material properties to cover the variability found in literature. The buildings were analyzed following a global in-plane safety verification proposed by the code. The nonlinear response of the buildings and their performance was assessed in all seismic zones for Portugal, considering a wide range of return periods up to 5000 years and foundations on different ground types. These results combined with reliability-based analyses allowed to estimate the mean annual frequency of exceedance of a specific value of the horizontal seismic coefficient, obtained by the maximum base shear divided by the building's weight.

The expeditious method proposed follow the empirical expressions that consider the most common in-plane failure mechanisms for masonry walls. Thus, the capacity of the buildings, also in terms of seismic coefficient, can be easily estimated using these expressions and considering only the walls in the direction of the seismic action. To account for the nonlinear structural behavior, the estimated seismic coefficient capacity must be corrected to provide more realistic values employing a simple surrogate model developed for this purpose. The methods discussed in this study can also be useful for code calibration in the framework of the elaboration of national application documents for Eurocode 8 concerning the definition of return periods in moderate to high seismic regions in Europe through a performance-based design approach of buildings.

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