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ORIGINAL RESEARCH PAPER

A simplified methodology for the seismic assessment of masonry buildings with RC slabs

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Abstract The present paper aims to contribute to the knowledge concerning the seismic assessment of load bearing masonry buildings with reinforced concrete slabs. The final goal of the present research was to propose a simple, yet accurate, methodology to assess the seismic safety of existing masonry buildings. The methodology here presented was based on the so-called ICIST/ACSS methodology with major improvements such as the extension to load bearing masonry wall buildings and the consideration of the effects of one of the most common strengthening solutions for masonry walls, here referred to as reinforced plastering mortar, as well as the possibility of considering four levels of increasing refinement: global, by alignment, by wall panel and by wall element. An extended research was performed on the existing methodologies to evaluate the seismic structural risk of load bearing masonry buildings, briefly describing methodologies similar to the one proposed, namely all of those that have in common the fact that they are based in the physical comparison between the resisting and acting shear forces at all storeys and along the two orthogonal horizontal directions. A case study is presented to check the applicability of the proposed methodology. The case study showed that the proposed methodology is relatively simple to apply and has a sufficiently good accuracy when compared with alternative methodologies. The degree of refinement of the analysis (global, by alignment, by wall panel and by wall element) must be taken into consideration and successively more complex analyses may be required when the results of simpler analyses are inconclusive.

Keywords Masonry buildings · Seismic behaviour · Safety assessment · Simplified approach

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1 Introduction

Typically the historic centres of cities are composed of old buildings with masonry load bearing walls, presenting, in many cases, an important architectural heritage value. On the other hand, in old cities a significant part of the fundamental public services with more stringent requirements in terms of structural safety, namely health and civil protection services (hospitals, fire and police stations, school and military facilities, government departments and others) are located in old masonry buildings. Thus, either due to the architectural value or to the importance of the hosted services, old (pre-code) masonry buildings are a priority in terms of seismic safety assessment and strengthening. This priority is further exacerbated by the fact that in view of recent earthquake events (Spacone et al. 2012), these buildings are likely to present inadequate seismic behaviour (Magenes and Penna 2009).

The dynamic behaviour of old buildings is dependent on the materials and techniques used in their structural components as well as on their state of conservation. Thus, the prediction of the structural behaviour of old buildings is a complex task, as a result of the widely varying state of conservation, the existence of discontinuities between masonry elements and, mainly, due to the non-linear mechanical behaviour of the masonry, which is characterized by a low tensile strength (Magenes and Della Fontana 1998; Lourenço 2002).

Due to the large concentration of masonry buildings in seismically-prone regions and due to the difficulties required by a detailed structural analysis, particularly from the numerical point of view, simple to use (yet reasonably accurate and conservative) tools would help structural engineers in the seismic safety assessment of old buildings. In fact, a first screening of the buildings' seismic safety through a simplified method should allow the identification of the buildings with striking structural defects which could subsequently be selected for further studies conducted with more sophisticated models.

Current existing European seismic vulnerability procedures may present a gap in terms of the required information, complexity, detail and reliability. Some of these procedures, such as those related to the EMS-98, are based on a limited number of constructional typologies and may consequently be too generalist, not taking into due account the individual sizes of the resisting structural elements and other detrimental (or beneficial effects) in a specific building. Moreover, these procedures are empirically-based, making it difficult to extrapolate to conditions other than those observed (earthquake severity and local constructional characteristics, as observed). Others, such as EN 1998-3:2005, are physically-based, but may require large amounts of information making these inadequate for survey and screening purposes.

The main goal of the present paper is to propose a simplified method for the seismic safety assessment of masonry buildings. The presented method allows for the assessment of the buildings in their original condition, but can also be used to assess the buildings after strengthening, provided that the strengthening technique falls within what is commonly referred to by "reinforced plastering mortar".

The proposed method is an adjustment to load bearing masonry buildings (before and after strengthening) of the so called ICIST/ACSS seismic risk assessment methodology, originally developed for reinforced concrete structures based on a rational interpretation of the multi-level methodology proposed for the same type of buildings by the Japanese construction ministry.

The method is of simple application (adapted to the most current practical engineering situations), enabling the seismic assessment of buildings based on the geometry of the structural system and on the mechanical properties of the structural materials, information that may be obtained in documents (project, plans, etc.) or during on-site surveys.

The development of the proposed methodology was initially based on a comprehensive review of the most relevant simplified methods for the seismic vulnerability assessment of masonry buildings, some of which are herein presented. The proposed method was further applied to an existent masonry building, the "Tomás Cabreira" Secondary School, in Faro, Portugal, in order to identify the major difficulties and the weaknesses of the methodology.

2 Seismic behaviour of masonry walls

The seismic resistance of old buildings is mainly due to the load bearing masonry walls, which present a significantly different behaviour when the direction of the earthquake action-effects is aligned with the wall plane or perpendicular to it.

Out-of-plane collapse occurs on walls orientated perpendicularly to the predominant direction of the seismic action. Depending on the boundary conditions (namely to the connection to the adjacent orthogonal walls and to the floor structures) several out-of-plane collapse mechanisms may occur, such as rocking, articulated rocking, vertical flexure or horizontal flexure (Calderini et al. 2009).

Walls parallel to the direction of the seismic action tend to collapse through in-plane mechanisms. As it is well known, the in-plane behaviour is the one where the masonry walls present more shear strength. For in-plane loaded walls, failure may occur by sliding, diagonal cracking, or by rocking, associated to articulated flexure (rocking or toe-crushing). The occurrence of one in-plane failure mode over another is influenced by the slenderness of the wall; boundary conditions; applied vertical force; and the mechanical properties of the wall. Out-of-plane and in-plane collapse modes occur for different loading levels, being the out-of-plane the one that presents less strength. However, out of plane collapse is usually local and is not prevailing when the floor structures behave like rigid diaphragms with effective connections to the masonry walls (Tomazevic et al. 1991; Mendes and Lourenço 2010, 2013). For that reason, most of the models to assess the seismic strength of masonry buildings are based on the masonry walls in-plane behaviour (Corradi et al. 2003; Tomaževič 1999; Anthoine 1991), with a local analysis of its out-of-plane behaviour.

The strength of a masonry wall subjected to in-plane shear is computed differently for each of the considered in-plane collapse mechanisms: sliding, diagonal cracking, and rocking or toe-crushing. Equations (1), (2) and (3) allow for the computation of the resisting shear stress τ_{Rd} of a collapse mechanism characterized by sliding shear failure, diagonal cracking failure and rocking or toe-crushing failure, respectively, according to the safety verifications proposed by some authors and codes (Turnašek and Cacovic 1971; Decreto Ministeriale del 2008; Magenes and Calvi 1997).

$$\tau_{Rd} = \frac{1.5c_u + \sigma_0 \tan\phi'}{1 + \frac{3h_0 \cdot c_u}{\sigma_0 \cdot b}} \tag{1}$$

$$\tau_{Rd} = \frac{1.5c_u}{\beta}\sqrt{1 + \frac{\sigma_0}{1.5c_u}} \tag{2}$$

$$\tau_{Rd} = \frac{\sigma_0 \cdot b}{2h_0}\left(1 - \frac{\sigma_0}{k \cdot f_d}\right) \tag{3}$$

where c_u, cohesion of the masonry material; $\sigma_0 = N/bt$, compressive stress on the wall; ϕ', internal friction angle of the masonry material; b, length of the wall; t, thickness of the wall; h_0, distance between the section of the null moment and the base of the wall (which can be considered equal to the distance between the base and the top of the wall for walls located at the last storey level and half of that length for walls located at intermediate storey levels); β, ratio between the distance from the base section of the wall and the top section (h) and the length (b). This parameter is inferiorly limited by 1.0 and superiorly by 1.5; k, factor that converts the linear distribution of the compressive stress into a rectangular diagram (assumed to be equal to 0.85); f_d, compressive strength of the masonry material.

Despite the diversity of the mechanical properties of masonry, resulting from the wide variety of construction methods and of materials used, as well as from the widely varying state of conservation, it is possible to identify some common characteristics, such as very low tensile strength, a weak shear strength (increased when subjected to compressive stresses) and a reasonable compressive strength. The best way to quantify the mechanical properties of masonry walls is through experimental tests (preferably in situ), which may be destructive or non-destructive. In the impossibility to perform experimental tests, reference values should be considered, using literature data such as Calderini et al. (2009), Anthoine (1991), Magenes and Calvi (1997) and Milosevic et al. (2013).

3 Simplified methods for the seismic safety assessment of masonry buildings

Current existing European seismic vulnerability procedures may present a gap in terms of the required information, complexity/detail and reliability. Some of these procedures are based on a limited number of constructional typologies and may consequently be too generalist, not taking into due account the individual sizes of the resisting structural elements and other detrimental (or beneficial effects) in a specific building. Others, such as EN 1998-3 (EN 2005), may require large amounts of information making these inadequate for survey and screening purposes.

Several simplified methods to assess the seismic vulnerability of old masonry buildings are available in the literature (Kappos et al. 2002; Lourenço and Roque 2006; Angeletti et al. 1997; LESSLOSS Report 2007a, b; Mouroux and Le Brun 2006); however, the Hirosawa (or Japanese Ministry of Construction) method (Hirosawa 1992) and its variants [PAHO (2000); ICIST/ACSS (2011) methods], the Italian Method *Vulnerabilità Muratura* (Dolce and Moroni 2005), developed by Dolce and Moroni, and the Index of Vulnerability Method (Vicente 2008), proposed by Vicente, are those that most influenced the development of the proposed method.

Since 1977, the Japanese Ministry of Construction has promoted the development of a multi-level procedure for the seismic vulnerability assessment of RC buildings. This procedure initially termed as "Hirosawa Method", has been upgraded along time and was published by Japan Building Disaster Prevention Association (Hirosawa 1992; BRI 2001). The procedure, initially developed for reinforced concrete buildings with less than six floors, has been extensively validated and calibrated with earthquake occurrences (Umemura 1980; Watanabe 1997).

The ICIST/ACSS methodology (ICIST/ACSS 2011; Proença et al. 2010) is the adjustment of the Hirosawa method to the Portuguese context and European code practices. The seismic vulnerability survey for a given building is expressed through the indirect comparisons of storey shear demands with storey shear strengths. These comparisons are performed for each above ground level storey, and are checked (separately) in two orthogonal horizontal directions. The comparisons are carried out through two indices, I_S, shear strength index, and I_{S0}, shear demand index—so that the survey is positive in a given storey i (and horizontal direction, here omitted) whenever $I_{Si} \geq I_{S0}$. The shear strength index relates to a specific storey, whereas the shear demand index is considered equal for all storeys.

The shear strength index (I_S) is calculated for each storey level and for each horizontal direction of the building (Proença et al. 2010) through the following equation:

$$I_S = E_0 \cdot S_D \cdot T \tag{4}$$

where E_0, basic shear strength index, i.e., the seismic performance index, shear strength in the absence of beneficial or detrimental effects induced either by the structural configuration or by usage deterioration, considers the resistance of the different types of RC resistant elements: columns, short columns and walls; S_D, index of structure irregularity, which accounts for a wide range of structural irregularities: plane irregularities, vertical irregularities and others; T, index of structural deterioration, which considers the effects on strength induced by usage. Apart from building age this index also considers structural damage, storage of hazardous materials, and previous exposure to hazards, such as earthquakes or fires.

The shear demand index of the structure (I_{S0}) is computed for the whole building (being possibly different for the two orthogonal horizontal directions) through the following equation (Proença et al. 2010) based on the lateral force method of analysis of Eurocode 8 (CEN 2004):

$$I_{S0} = \frac{S_d(T_1) \cdot \lambda_1 \cdot \chi}{g} \tag{5}$$

in which T_1, period of the structure for the fundamental translational mode (Silva 2011) in the considered horizontal direction; $S_d(T_1)$, spectral acceleration at period T_1, calculated according to Eurocode 8 (CEN 2004) for the no-collapse requirement and considering the importance of the building; λ_1, correction factor to account for the percentage of the mobilized mass for the fundamental mode in the horizontal direction of analysis; χ, reduction coefficient to apply in case the design life of the structure is different from 50 years (reference design life); g, acceleration of gravity (9.8 m/s^2).

The Vulnerabilità Muratura (VM) method (Dolce and Moroni 2005) estimates the intensity of earthquakes correspondent to two levels of damage in the building: light damage and terminal damage that correspond, in terms of performance, to the limit conditions of operation of the building and that of collapse, respectively, of the Italian Code NTC2008 (Decreto Ministeriale del 2008).

For the masonry walls, loaded in shear, the VM method takes into account the diagonal cracking and the rocking/toe-crushing collapse modes. Thus, the wall strength to seismic forces is computed by the Turnsek–Cacovic formula, which expresses the ultimate capacity of a wall when collapse occurs by diagonal cracking. To take into account the rocking/toe-crushing collapse mode when the wall is slender or is subjected to a low compressive stress, the VM method considers a corrected resisting shear stress (τ_{kcorr}), by a reduction factor function of the wall slenderness and of the compressive stress that the wall is subjected to.

The safety index of the masonry structure (I_{Seg}) obtained by the application of the VM method results from the comparison of the maximum acceleration (PGA) that the structure is able to sustain with the earthquake acceleration, for the limit condition considered, provided through the Italian Code NTC2008 (Decreto Ministeriale del 2008). Thus, the application of the VM method yields two safety indexes for the same masonry structure in evaluation, corresponding to the limit operation and collapse conditions for the structure under study.

The maximum peak ground acceleration (PGA) that the structure may withstand is calculated for each storey level and in each main horizontal direction, through the relation between the strength to seismic force and the seismic solicitation for the required limit conditions (of operation and/or of collapse), being affected by coefficients that take into account the participation of the first mode of vibration (in function of the number of storeys of the building), the spectral amplification, the dissipative capacity of the building and the structural ductility.

In the method proposed by Vicente (2008) and Vicente et al. (2011), the classification of a masonry building is obtained by a vulnerability index, which is the weighted sum of the parameters intended to address features that influence the seismic response of the building. Those parameters are related to 4 classes of increasing vulnerability and are arranged into four groups, in order to emphasize their differences and relative importance. The first group includes parameters characterizing the building resisting system, the type and quality of masonry, the quality of connections between walls, the shear strength capacity of the building, the potential out-of-plane collapse risk, the height and the soil foundation conditions of the building. The second group of parameters is mainly focused on the relative location of a building in the area as a whole and on its interaction with other buildings, on the irregularity in plan and height, and on the relative location of openings, The third group of parameters evaluates horizontal structural systems, namely the type of connection of the timber floors and the impulsive nature of pitched roofing systems. Finally, the fourth group evaluates structural fragilities, the conservation level of the building and the negative influence of non-structural elements with poor connections to the main structural system.

A weight is assigned to each parameter, ranging from 0.50 for the less important parameters (in terms of structural vulnerability) up to 1.5 for the most important. The definition of each parameter weight is a major source of uncertainty, despite being based on expert opinion (Vicente et al. 2011).

The calculated vulnerability index can then be used to estimate building damage after a specified intensity of seismic event.

4 Methodology proposed for seismic evaluation of masonry buildings

The method for Seismic Evaluation of Masonry Buildings that is proposed in the present paper is a development of the ICIST/ACSS methodology (Proença et al. 2010). In this proposal the assessment of the seismic vulnerability of masonry buildings (with RC slabs) is made through the comparison between the resisting and the acting shear forces (F_{Rd} and F_{Sd}), computed at storey levels and in each of the main orthogonal lateral directions. If all the resisting shear forces (F_{Rd}) are reasonably higher than the corresponding acting shear forces (F_{Sd}), the building is considered in a safe condition.

This approach takes into account three levels of seismic vulnerability, depending on the ration between the shear forces F_{Rd} and F_{Sd}. Seismic safety is confirmed at a given storey and along a given lateral direction when the ratio between the resisting shear force (F_{Rd}) and the acting shear force (F_{Sd}) is higher than 1.2. When the ratio between F_{Rd} and F_{Sd} is smaller than 0.8, the building is considered not to be seismically safe. When the ratio between the forces F_{Rd} and F_{Sd} is between 0.8 and 1.2, the seismic assessment of the building is considered inconclusive and a more complex analysis (based on more refined models) is required.

Although the method was initially developed for a global analysis of the building under study, it also allows for a more refined analysis through the study of particular wall alignments (a wall alignment is considered as a group of wall panels, albeit with different thicknesses and constitutions within the same plan structural alignment), wall panels (wall panel is an interrupted combination of wall elements with the same thicknesses and constitutions) or individual wall elements (wall elements between successive openings). Thus, the method may be applied to four different levels of analysis: *global* (complete building), *alignments'*, *wall panels'* and *individual wall elements'*.

The assessment of the seismic vulnerability of the building by global analysis is made individually for each storey level, in each of the main horizontal directions.

It must be mentioned that the proposed methodology was developed for masonry buildings with floors behaving like rigid diaphragms and with an efficient connection between orthogonal walls and between walls and floors. In that case, and if the building has enough force redistribution capacity, it can be assumed that the seismic inertial forces are distributed by the walls proportionally to their stiffness. In addition, the models to assess the shear strength of the walls can be based in in-plane collapse mechanisms.

The proposed method does not attest the safety of the spandrels (wall areas in between superimposed openings) as these elements were considered secondary in terms of the vertical load bearing capacity of the building structure. The underlying assumption is that the damage suffered by the spandrels is not of critical importance for the global stability of the structure since these elements are not directly connected to the foundation, meaning that in an ultimate limit state scenario the vertical loads may be transferred to the adjoining pier elements. The other beneficial effect of the spandrels—that of the lateral interconnection of the vertical piers in such a way that these behave jointly—can be taken over by the diaphragm effect provided by the RC slab structure.

The resisting shear force of the building at a given storey level i is calculated through Eq. (6), in each main horizontal direction of the building (X or Y). That force is the combination (approximate sum) of the resisting shear forces of all the lateral load resisting elements (including masonry walls) that exist in that direction (and of the storey level under analysis).

$$F^{X}_{Rd_i} = \sum_{\mathrm{j}}^{n} \alpha_j \cdot A_{\mathrm{j}} \cdot \tau_j \tag{6}$$

where n, number of wall elements at the storey level i, aligned with the horizontal direction X; α_j, is a reduction factor for the strength capacity of the element j, accounting for the possibility that the element under consideration may not have reached (or may have surpassed) the maximum capacity for the prevailing interstorey drift mechanism. In either case this factor should not be greater than 1. A_j, transversal cross section area of the wall element; τ_j, shear strength (stress) of the wall element.

The α factors indicate the proportion of the total shear strength index attributable to a given class of elements—masonry walls, short columns, columns or walls—for the prevailing storey shear failure mode. Failure modes can be classified into three categories—A to C, from the more brittle to the more ductile—depending on which of the three deformation capacity categories (masonry walls/short columns, columns or walls, in the same order) attains its peak strength. The extension of the original Japanese Ministry of Construction method (Hirosawa 1992) to account for the contribution of the masonry walls to the lateral load resisting capacity was performed by PAHO (2000), assuming that walls belonged to the more brittle category with no differentiation on the prevailing (in-plane) collapse mechanism (sliding, diagonal cracking, and rocking or toe-crushing) of these walls. In this study and considering that there are no clear indications that those in-plane wall collapse mechanisms tend to occur for significantly different levels of interstorey drift, unitary values were considered for the α factors. However, considering that the lateral load capacity of these type of buildings is solely dependent on walls subjected to in-plane loads, this factor should be refined in further studies.

The wall ultimate shear strength is the lowest value of the ones obtained for each of the three considered collapse mechanisms: sliding shear failure, diagonal cracking and rocking or toe-crushing. Those values correspond to the three possible collapse mechanisms of the wall element and are computed by Eqs. (1), (2) and (3).

The acting shear force (F_{Sd}) is computed for each storey level of the building according to Eq. (7), consisting again in a derivation of the base shear force expression of the lateral force method of analysis of Eurocode 8 (CEN 2004). For the buildings under study (where the fundamental periods lie in the constant acceleration branch of the response spectrum and the masses associated with the fundamental modes are similar in both horizontal directions) that value can be used in the two main horizontal directions.

$$F_{Sd} = \frac{S_d(T_1) \cdot \lambda_1 \cdot \chi}{g} \cdot \frac{W}{\phi \cdot S_D \cdot T} \tag{7}$$

in which: T_1, period of the structure for the fundamental translational mode in the considered horizontal direction; $S_d(T_1)$, spectral acceleration at period T_1, calculated according to Eurocode 8 for the no-collapse requirement and considering the importance of the building; λ_1, correction factor to account for the percentage of the mobilized mass for the fundamental mode in the horizontal direction of analysis; χ, reduction coefficient to apply in case the design life of the structure is different from 50 years; g, acceleration of gravity (9.8 m/s^2). W, total weight of the building above the storey level in analysis; ϕ, modification factor given by $\frac{n+1}{n+i}$ where n corresponds to the total number of storey levels of the building under evaluation and i corresponds to the storey level under analysis; S_D, sub-index of structural irregularity (in plan and in height); T, sub-index structural deterioration (0.7 for buildings with settlements at the foundations, 0.9 for buildings that present cracks on the walls and 1.0 for buildings without signs of deterioration).

The correction factor λ_1 should be considered equal to 0.85 if the building has more than two above ground storeys and a fundamental period lower than two times the period that defines the beginning of the constant spectral acceleration ($T_1 \leq 2T_C$). In all other cases λ_1 must be considered with a unitary value.

Following the "Hirosawa Method" (BRI 2001), the sub-index of structural irregularity S_D is calculated according to the following equation:

$$\mathrm{S_D = q_a \cdot q_b \cdot q_c \cdot q_e \cdot q_f \cdot q_i \cdot q_j} \tag{8}$$

where

$$\mathrm{q_i = 1 - (1 - G_i) \cdot R_i, \quad i = a, b, c, e, f, i, j} \tag{9}$$

in which the parameters $\mathrm{G_i}$ and $\mathrm{R_i}$ are obtained from Table 1.

Depending on the existence (and extent) of detrimental irregularity effects the sub-index of structural irregularity $\mathrm{S_D}$ varies between 0.4 and 1.0.

As in the original method (Hirosawa 1992; BRI 2001), the possible existence of detrimental torsion effects is accounted for not at the distribution of the storey forces between the different lateral load resisting elements but at the computation of the structural irregularity sub-index. For that reason that sub-index also reflects that source of irregularity, albeit with less discrimination.

For a more detailed analysis (by *alignment*, by *wall panel* or by *wall element*) the global seismic shear force (at each storey level and on both of the main horizontal directions) should be distributed along the vertical load resisting elements. For buildings with rigid diaphragms and regular storey plans, this distribution can be based on the relation of the transversal cross section areas of the wall elements. However, the accuracy of this distribution can further be improved if it is based, instead, on the global stiffness (shear + flexure) relation.

The extension of the proposed methodology to strengthened buildings requires a modification of Eqs. 1 to 3 to account for the strengthening effects.

Table 1 Parameters G_i and R_i

Parameters			G_i			R_i
			1.0	0.9	0.8	
Regularity in plan	*a*	Regularity	*Regular*(a_1)	*NearlyRegular*(a_2)	*Irregular*(a_3)	1.00
	b	Aspect ratio (length/width)	$b \leq 5$	$5 < b \leq 8$	$b > 8$	0.50
	c	Setback in plan (narrow part)	$c \geq 0.8$	$0.5 \leq c < 0.8$	$c < 0.5$	0.50
	e	Inner courtyard (well-style area)	$e \leq 0.1$	$0.1 < e \leq 0.3$	$e > 0.3$	0.50
	f	Eccentricity of the inner courtyard (well-style area)	$f_1 \leq 0.4$ $f_2 \leq 0.1$	$f_1 \leq 0.4$ $0.1 < f_2 \leq 0.3$	$f_1 > 0.4$ $f_2 > 0.3$	0.25
Regularity in elevation	*i*	Storey height uniformity	$i \geq 0.8$	$0.7 \leq i < 0.8$	$i < 0.7$	0.50
	j	Soft story	No soft storey	Soft storey	Eccentric soft storey	1.00

a_1, structural balance is good, and the area of a projecting part is not more than 10% of the floor area; a_2, structural balance is worse than a_1, or the area of a projecting part is not more than 30% of the floor area with L, T or U shaped plan; a_3, structural balance is worse than a_2, or the area of a projecting part is larger than 30% of the floor area with L, T or U shaped plan; b, length ratio (=length of the long side/length of the short side); c, ratio between the narrow part length and the main length of the building; e, well-style area ratio = well-style area/total floor area. The well-style area is the room or the space stretching over two stories or more; f_1, distance-length ratio = (distance between the centre of the floor area and the centre of the well-style area/the length of the short side of the building); f_2, distance-length ratio = (distance between the centre of the floor area and the centre of the well-style area/the length of the long side of the building); i, height ratio = (height of above story/the height of the storey concerned). In case of the top storey, the height of the storey below is taken instead of the above storey height

One of the most commonly used seismic strengthening techniques for masonry wall buildings is the application of the so-called reinforced plastering mortar. This strengthening technique consist in the addition of outer leafs (preferably on both faces of the existing walls) made of premixed structural mortar or sprayed concrete, reinforced with strengthening meshes (steel, fibreglass or carbon fibre).

The main effect of this strengthening technique is the increase of the wall tensile and shear strength. Thus, the equations considered to calculate the wall shear strength by sliding shear failure and by rocking/toe-crushing must be reformulated, as the expressions (1) and (3) do not consider the tensile strength of the wall elements.

In unreinforced masonry walls, the sliding shear failure mode is characterized by the assumption that friction is active only in the compressed length of the wall base. However, due to the effect of the strengthening mesh, the shear strength without compression (i.e. cohesion) is effective in all length of the base wall. Thus, to take into account the shear strength due to the strengthening mesh, an equivalent cohesion c_u can be considered.

The ultimate shear stress of a wall panel for the sliding shear failure mode can then be computed through Eq. (10), which was obtained by summing the shear strengths in the tensile (cohesion effect) and in the compressed zones (friction and cohesion effects).

$$\tau_{Rd} = \frac{-c_u - \frac{\sigma_0 \tan\phi'}{2} - \sqrt{-c_u^2 + c_u \sigma_0 \tan\phi' + \frac{b\sigma_0^2 \tan\phi'}{6h_0}}}{\frac{3h_0 \tan\phi'}{b} - 2} \tag{10}$$

In Eq. (10) the ratio h_0/b, is superiorly bounded by the value of $2/(3\tan\phi')$, considering that for strengthened elements with reinforced plastering mortar and with ratios h_0/b of higher values than that the bounding limit, the collapse by sliding shear failure does not prevail over the other failure modes (diagonal cracking or rocking/toe-crushing failures).

It should be mentioned that the previous expression assumed, for simplicity, similar values for the modulus of elasticity in tension and in compression.

In the case of the collapse by diagonal cracking, Eq. (2) can be extended to such cases if the values of the mechanical parameters are increased, namely the cohesion (shear strength without compression stresses) of the masonry.

To quantify the shear strength of a masonry wall strengthened with reinforced plastering mortar for the collapse mode corresponding to rocking/toe-crushing failure, a non-linear distribution for the compressive stresses on the base of the element and a constant distribution for the tensile stresses can be considered. In a masonry element strengthened with reinforced plastering mortar, it may be considered that the reinforcing mesh resists solely to tensile stresses whereas masonry resists solely to compressive stresses. Thus, in the rocking/toe-crushing failure mode the ultimate state occurs by toe-crushing of the compressed masonry and, simultaneously, by yielding of the reinforcing mesh. Equation (11) can be show to reflect the previous assumptions, allowing for the computation of the shear strength of a strengthened masonry element with the prevailing failure mechanism of rocking/toe crushing.

$$\tau_{Rd} = \frac{b(\sigma_0 t + 2f_{tu})[0.8^2 k f_d (k f_d t + 3f_{tu} - \sigma_0 t) - 2\sigma_0 f_{tu}]}{2h_0(0.8 k f_d t + 2f_{tu})^2} \tag{11}$$

where f_{tu}, tensile strength of the strengthened masonry wall; t, thickness of the wall.

The factor $k = 0.85$ takes into account the non-linear distribution of the compressive stresses.

5 Case study: a secondary school in Portugal

The main building of Tomás Cabreira Secondary School, in Faro, Portugal, was used to test the applicability of the proposed methodology for the assessment of seismic vulnerability of masonry buildings. The building was one of the Secondary School buildings functionally restored and structurally reinforced within the scope of the *Parque Escolar* Government Programme (Proença and Gago 2011). The building was previously studied and modelled using linear elastic finite element models (Silva 2011; Proença and Gago 2011) and the corresponding numerical results were used to validate the results obtained with the present modified version of the ICIST/ACSS methodology. The structure was modelled considering discretized shell-thick elements for the individual wall elements (walls of uniform thickness and constitution between consecutive wall openings) and floor slabs and frame elements for the beams and columns (only existing at the basement level).

The proposed methodology was applied to the study of the vulnerability of the main school building, before and after the intervention of seismic strengthening that the same building underwent in 2009. The structural safety assessment was made for all the four levels of the proposed methodology: global, alignment, wall panel and wall element.

The main building of the school was erected in the early twentieth century, but lost some of its original constructional and structural characteristics due to several modifications and extensions that were carried out during the first half of the same century. The most relevant works from the seismic point of view were the replacement of the wooden floors by reinforced concrete slabs, the addition of reinforced concrete beams and the extension of the second floor to the whole plan. Seismic resistance was not taken into account in the original design, as well as, in the subsequent interventions, which suggests that the vertical elements (masonry walls) may have inadequate strength.

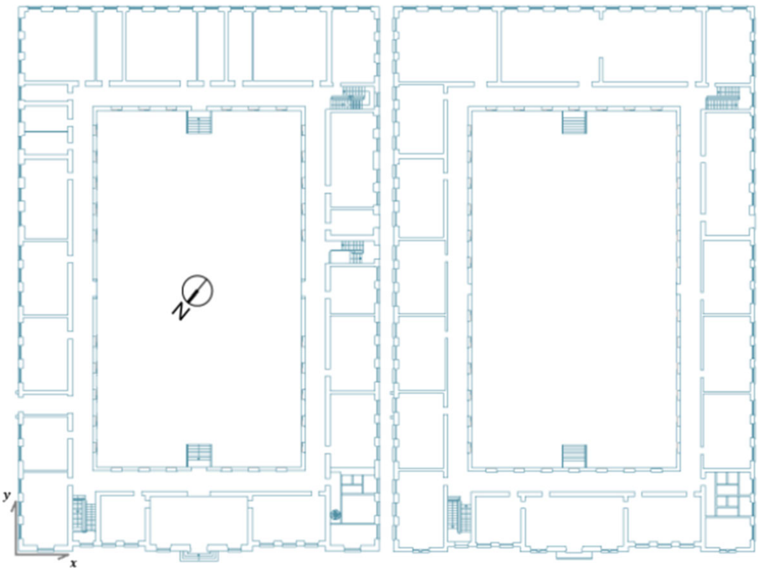

Fig. 1 Main building of the Tomás Cabreira Secondary School—ground floor level (*left*); first floor level (*right*)

The building has two floor levels and a rectangular plan configuration with a central inner courtyard (Fig. 1), also rectangular in plan shape. The outside plan dimensions of the building are 44.3 m (X-direction, roughly coincident with NE–SW geographical orientation) and 66.9 m (Y-direction, roughly coincident with NW–SE geographical orientation), whereas the central courtyard measures 25.2 m and 43.8 m in plan, in the same order (X and Y). Each floor has a height of 4.3 metres and the basement (under parts of the southeast and southwest wings) has a height of 2.6 metres. Previous numerical studies (Silva 2011) did not indicate structural insufficiencies in the masonry panels located in basement of the building. For that reason, in the present study the vulnerability assessment was carried out not considering the basement.

The structural elements of the building are the masonry walls (rubble stone and brick masonry), reinforced concrete slabs and some reinforced concrete beams, supporting the stairs slabs. To simplify the generation of the finite element model and to make the results easier to understand (avoiding the occurrence of spurious stress concentrations), similar mechanical characteristics were adopted for both types of masonry—stone and brick masonry (Table 2). The values adopted (and presented in Table 2) are typical values for Portuguese rubble lime stone masonry walls, which are not far from the values that characterize traditional Portuguese brick masonry walls from the first half of the twentieth century.

In the intervention carried out in 2009 a strengthening technique by reinforced plastering mortar was applied on main load bearing masonry walls. The reinforced plastering mortar was applied on both sides of the walls, improving their resistance and promoting their transversal confinement. The strength increase was quantified by experimental loading tests in prototypes (Proença et al. 2012) leading to the values presented in Table 3 for the mechanical parameters of strengthened masonry walls. It should be noted that the Italian Standard (Decreto Ministeriale del 2008) suggests that irregular masonry stone walls strengthening with reinforced plastering mortar present an increase of strength (cohesion and compressive strength) of about 250%. The values of cohesion and friction coefficient presented on Tables 2 and 3 were further reduced by a safety factor, according with Eurocode 8—Part 3 (CEN 2005) and with the Italian Standard (Decreto Ministeriale del 2008). Considering for the present building a level of knowledge of “limited on-site surveys”, the additional safety factor was taken as 1.35.

First, the seismic safety assessment of the Tomás Cabreira School Building was based on a three-dimensional (linear-elastic) finite element model, where the masonry walls and the reinforced concrete slabs were modelled by 8-node shell elements. Then, the proposed methodology (i.e. the development of the ICIST/ACSS methodology) was applied and the results compared with the previous safety assessment conducted with the results of the numerical model. In both analyses the non-linear effects were considered by a behaviour factor q = 1.50, according to Eurocode 8 (CEN 2004).

The shear demands, namely the shear stresses due to the seismic load, were computed based on the numerical results. The computation of the shear strength was performed using

Table 2 Mechanical properties of the Stone Masonry in its original state

Specific weight γ	23 kN/m^3
Young modulus E	2.5 GPa
Cohesion Cu	0.06 MPa
Compressive strength (f_d)	1.8 MPa
Friction coefficient ($\tan \phi'$)	0.4

Table 3 Mechanical properties of the strengthened stone masonry

Cohesion (c_u)	0.25 MPa
Compressive strength (f_d)	1.8 MPa
Tensile strength of the steel mesh (f_{tu})	0.04 MPa
Friction coefficient ($\tan \phi'$)	0.4

Eqs. (1), (2), (3), (10) and (11), and considering an average compressive stress (for permanent loads), $\overline{\sigma_0}$, obtained by the results of the numerical model. Table 4 summarizes those results, considering the cases of the building in the original state and after seismic strengthening.

The abbreviation VS summarizes the safety check, provided by the ratio between the resisting shear force (F_{Rd}) and the acting shear force (F_{Sd}).

The relatively small attenuation of the storey shear forces along height can be attributed to the combined effect of the inverted triangular pattern (along height) of the inertia forces (due to the fundamental mode shape configuration) with the fact that there is some mass concentration at roof level.

The same safety analysis can be performed computing the acting shear force through the Eq. (7) of the proposed ICIST/ACSS methodology.

To compute the period of the structure for the fundamental translational mode in the considered horizontal direction, needed to compute the spectral acceleration, empirical rules or experimental data can be used. However, in the present case the results of the finite element model were used, namely, the periods of vibration of 10th and 23rd modes, which are the main translational global vibration modes, in X and Y directions, respectively ($T1x = 0.211$ s and $T1y = 0.176$ s). Therefore, the same values for $S_d(T_1)$ of 5.90 m/s^2 were computed for both horizontal directions.

For the correction factor λ_1, a value of 0.60 was adopted based in the observation (confirmed by the finite element model results), that the mobilised mass in the fundamental mode of vibration is about 60% of the total mass of the building.

Although some masonry walls presented cracks before strengthening (which, normally corresponds to a sub-index of temporal deterioration $T = 0.9$), that sub-index was considered unitary (also because that deterioration was also not considered in the finite element model).

In what concerns the sub-index of structural irregularity S_D, this was also taken as unitary (corresponding to a regular building) since the irregularities were also not considered in the behaviour factor used in the numerical dynamic analysis (finite element model).

Table 4 Building safety check—global analysis based on the numerical model results

Global analysis			Before strengthening		After strengthening	
Storey level	F_{Sd} (kN)	$\overline{\sigma_0}$ (kN/m^2)	F_{Rd} (kN)	VS	F_{Rd} (kN)	VS
1x	25,015	329.3	13,564	0.54	21,028	0.84
1y	25,176	332.5	17,227	0.68	25,934	1.03
2x	19,362	193.4	6228	0.32	12,795	0.66
2y	21,026	195.2	7157	0.34	13,365	0.64

Table 5 shows the weight supported (W) by the walls and the acting shear forces, by storey levels, obtained by the proposed modified version of the ICIST/ACSS methodology, closely matching the results of the numerical model.

The shear strength (F_{Rd}) can be computed considering the average compressive stress and following the Eqs. 1, 2, 3, 10 and 11. Comparing the shear strength with the shear demand it is possible to assess the safety of the structure by the VS factor (Table 6).

Comparing Tables 4 and 6, which present the results of the global analysis using, respectively, results from the finite element model and from the proposed modified version of the ICIST/ACSS methodology, it may be concluded that the safety verification checks (VS) are very similar in both cases, leading to similar conclusions.

In both cases the values of VS before strengthening do not comply with safety requirements. After strengthening, storey level 2 still does not comply with safety requirements, whereas storey level 1 is at an inconclusive level of the assessment (between 0.8 and 1.2), requiring a more detailed analysis.

However, it must be mentioned that for the strengthened walls some overly conservative values for the mechanical parameters were considered, namely the friction coefficient and the compressive strength that were taken with the same value as that for the non-strengthened walls. In this way, the conclusions for the seismic performance of the building may be conditioned by those conservative hypotheses. It should also be mentioned that the value adopted for the behaviour factor ($q = 1.50$) for the building in both (original and strengthened) conditions, also may be regarded as conservative. Note that the Italian Standard NTC2008 (Mendes and Lourenço 2013) states for old masonry buildings behaviour factors with values higher than 1.50.

The proposed development of the ICIST/ACSS methodology can also be extended to increasingly detailed levels—alignment, wall panel or element—allowing for the identification of the components that more stringently require strengthening.

Firstly, it is required a methodology for allocating the global (acting) shear force through the various structural elements (walls). The generalized distribution rule herein proposed has two steps. A first step where the shear force distribution is performed proportionally to the cross section area, to the shear stiffness or to a combined shear + flexure stiffness, from the least effective, yet simpler, to the more refined and complex (Eq. 12b). The second step is based in the lateral deformation pattern of the fundamental mode. If this information is available, a better distribution can be obtained by a second correction step (Eq. 12b).

$$F_{Sd0_i} = \kappa_i \frac{F_{Sd}}{\sum \kappa_i} \tag{12a}$$

$$F_{Sd_i} = F_{Sd0_i} \delta_i \frac{\sum F_{Sd0_i}}{\sum F_{Sd0_i} \delta_i} \tag{12b}$$

Table 5 Modification factor (ϕ), weight supported (W) and acting shear forces by storey levels for the proposed modified version of the ICIST/ACSS methodology

Storey level	ϕ	W (kN)	F_{Sd} (kN)
1	1.00	83,903	25,242
2	0.75	39,659	19,090

Table 6 Global analysis of the building, before and after seismic strengthening, by the application of the proposed modified version of the ICIST/ACSS methodology

Global analysis			Before strengthening		After strengthening	
Storey level	F_{Sd} (kN)	$\overline{\sigma_0}$ (kN/m^2)	F_{Rd} (kN)	VS	F_{Rd} (kN)	VS
1x	25,242	323.2	13,354	0.53	20,663	0.82
1y	25,242	334.3	17,213	0.68	25,868	1.02
2x	19,090	177.2	5715	0.30	12,429	0.65
2y	19,090	186.6	6560	0.34	12,739	0.67

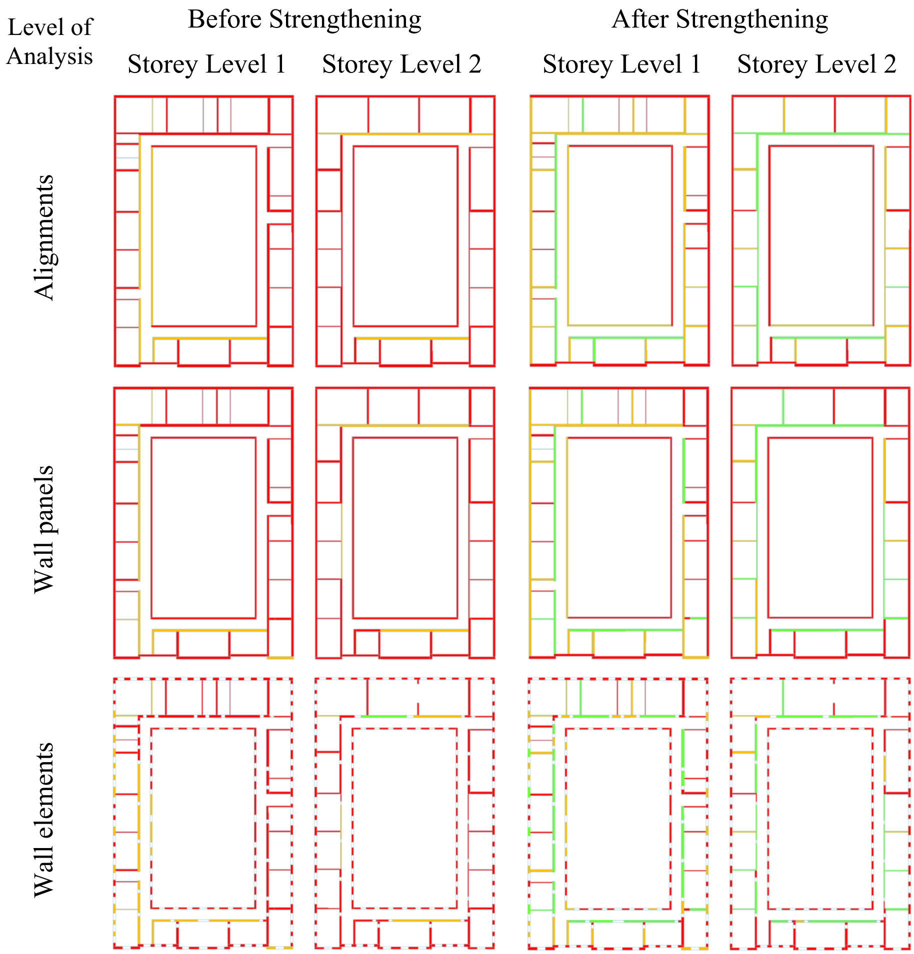

Fig. 2 Safety assessment through the application of the proposed variation of the ICIST/ACSS methodology, considering different levels of analysis: by alignment, by wall panel and by wall element

where F_{Sd}, global acting shear force at the storey level; $F_{Sd0,i}$, initial acting shear force on the component (alignment, panel or element); F_{Sd_i}, acting shear force on the component (considering the fundamental mode shape); κ_i, initial weighing factor (based on the cross section area or stiffness of the component); δ_i, estimated lateral displacement in the wall component at the fundamental mode shape.

Figures 2 graphically show the results of the seismic vulnerability assessment of the main school building, before and after strengthening. The assessment was performed through the application of the proposed variation of the ICIST/ACSS methodology, for different levels of analysis, namely global analysis, analysis by alignment, analysis by wall panel and analysis by wall element. The distribution of the acting shear force through the components was performed considering, at first, the cross section area proportionality rule and, then, further refining it through the consideration of the fundamental mode shape (using the FEM model, normally inexistent). In those figures the load bearing masonry walls were identified with colours, according to the result of their seismic vulnerability assessment. Red, orange and red, respectively, are used to identify the elements that do not fulfil, partially fulfil (inconclusive) or fulfil the safety requirements.

Figures 2 show, as expected, that lower levels of refinement in the analysis tend to be more conservative. These more refined analyses allow for the identification of the elements or panels where strengthening is really necessary as well as of others where strengthening may be waived.

6 Conclusions

The confrontation of the assessment results on the seismic vulnerability resulting from the applied methodologies (finite element numerical model and the modified version of the ICIST/ACSS methodology), indicates that the proposed methodology (the modified version of the ICIST/ACSS methodology) provides a good assessment for global analysis of the structure.

The computation of the acting shear forces (F_{Sd}), generically based on the lateral force method of analysis of Eurocode 8, at each storey level and along each of the main horizontal directions yields results that extremely close to those of more refined finite element analysis. As to the acting shear forces (F_{Sd}), a set of rational expressions is proposed to account for the prevailing collapse mechanisms of masonry walls subjected to in-plane forces. These expressions can be adapted to account for the beneficial effects of one of the most common strengthening techniques (here termed as reinforced plastering mortar).

The proposed methodology has two different levels of study, global analysis and local (or more refined) analysis. The local analysis can be done with several levels of detail, namely by *alignment*, by *wall panels* and *wall elements*. This strategy proved to be effective when the conclusions of the global analysis are inconclusive, allowing a gradual deepening of the safety study and providing a good overview of the building seismic behaviour.

The quality of the safety assessment by the proposed modified version for masonry buildings of the ICIST/ACSS methodology, increases with the value of the information available, namely about the geometry of the walls and mechanical properties of the masonry. Under optimal conditions, the methodology would provide reliable assessments of seismic vulnerability, in any of the four levels of analysis presented (global, by alignment, by wall panel and by wall element).

In specific applications of the proposed methodology, such as those in the case study, some difficulties may arise in the assignment of the masonry mechanical properties, as well as, while trying to adapt the Codes rules to existing buildings (e.g. the value of the behaviour factor for old masonry buildings). In fact, further experimental in situ tests and numerical studies are required to characterize building typologies, materials and seismic strengthening techniques.

The seismic safety assessment of Tomás Cabreira Secondary School classroom building, for both original and strengthened states, led to the conclusion that the load bearing walls strengthening improved the seismic safety of the building, although, not completely. As mentioned, this conclusion may be excessively conservative, since for the mechanical properties of the strengthened stone masonry conservative values were adopted.

However, the goal of the study was reached, since it was to test and exemplify the present modified version of the ICIST/ACSS methodology (for masonry buildings).

However, the building may collapse partially before all the wall reach their ultimate shear stress, since the redistribution capacity of masonry walls is limited. For that reason the reduction factor for the resistance capacity α_i, which takes into account the limited capacity of the structure in terms of force redistribution, must be calibrated to represent the contribution of each collapse mechanism (sliding shear failure, diagonal cracking, and rocking or toe-crushing) on the global behaviour. In the case study presented in Sect. 5 this reduction factor was assumed unitary (for the three considered collapse mechanisms—sliding shear failure, diagonal cracking and rocking or toe-crushing).

The sub-indexes of structural irregularity S_D and of temporal deterioration T considered for the seismic demand computation should be calibrated, to better adapt the proposed methodology to the behaviour of real masonry buildings.

At a more advanced level of research, it can be added equations and/or strategies to the present methodology in order to take into account other issues, such as the consideration of out-of-plane collapse mechanisms.

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