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Seismic Analysis of Traditional Stone Rural Buildings: Case study of a one-storey building

Análisis del comportamiento sísmico de construcciones rurales tradicionales de piedra: estudio de caso de una estructura de un piso

C. Arcidiacono [(*)], S. M. C. Porto [(*)], G. Cascone [(*)]

ABSTRACT

Specific features of traditional rural buildings can influence the assessment of their seismic behaviour. When a change in intended use of traditional rural buildings is necessary, restoration work must comply with specific seismic norms and should preserve their original features. In this paper, a model for the seismic safety verification of masonry walls for in-plane actions was applied to investigate the structural behaviour of one-storey stone-masonry traditional rural buildings, in relation to standards application and possible retrofitting interventions.

The results showed that pier-panel collapse mechanisms and the simulation method of masonry spandrel behaviour are of importance and affect the need to provide for strengthening interventions.

Keywords: Seismic assessment; in-plane forces; shear-type pier panels; stone masonry; seismic standards; modelling; reuse.

RESUMEN

Las características específicas de las construcciones rurales tradicionales pueden tener influencia en la verificación de su comportamiento sísmico. Cuando se hace necesario cambiar el uso de las construcciones rurales tradicionales, los trabajos de reestructuración deben satisfacer las normas sísmicas específicas y, al mismo tiempo, deberían preservar sus peculiaridades originales. En este artículo se ha aplicado un modelo para verificar la seguridad sísmica de mamposterías sujetas a acciones coplanarias, con el objetivo de ahondar en el comportamiento estructural de construcciones rurales tradicionales de una sola planta, hechas con mampostería de piedra, con relación a la aplicación de normativas y posibles trabajos de mejora antisísmica. Los resultados han demostrado que los mecanismos de colapso de los elementos resistentes y el método de simulación del comportamiento de los tímpanos de mampostería tienen influencia en la verificación sísmica, así como la necesidad de efectuar obras de refuerzo.

Palabras clave: *Verificación sísmica; fuerzas coplanarias; estructura tipo pórtico (shear-type); mampostería de piedra; normas sísmicas; modelación; reutilización.*

[(*)] University of Catania. Catania (Italy).
Persona de contacto/*Corresponding author:* carcidi@unict.it (C. Arcidiacono)

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1. INTRODUCTION

In many European countries, the abandonment of traditional rural buildings (TRBs) is the result of many driving forces such as technological innovation and past agricultural policies (1). TRBs were prevented from being adapted to new production systems, due to their morphologic, technical and constructive characteristics (2) (3).

Considering that TRBs are cultural heritage sites as well as important territorial resources which possess significant potential that can be utilised by carefully considering their adaptive reuse (4) (5) (6) (7) (8), today, specific measures contained in the European Union (EU) Rural Development Policy (RDP) 2007-2013 (9) and specific national laws and regulations contribute towards their conversion. However, since the reuse of TRBs could determine a load increase and affect the overall structural behaviour, structural modifications may be required.

In earthquake-prone areas, such as the majority of Southern European areas, changes in the intended use or structural modifications of existing TRBs should comply with European Codes or specific National Seismic Standards which require structural seismic safety assessment (10) (11) (12) (13).

The mechanisms involved in assessing masonry buildings are 'first damage mode' mechanisms, usually concerning out-of-plane mechanisms, and 'second damage mode' mechanisms related to the in-plane response of walls. It is generally recognized that acceptable seismic behaviour is attained only if out-of-plane collapse is prevented and the in-plane strength and deformation capacity of walls can be fully exploited (14). Once it has been verified that 'first damage mode' mechanisms have been prevented, 'second damage mode' mechanisms are taken into consideration. Generally, to analyse these two mechanisms, a global structural model which requires that structural connections guarantee a global box-type behaviour by performing specific building interventions (e.g., construction of rigid horizontal diaphragms and construction of reinforced concrete belts) should be considered. However, in stone masonry TRBs, horizontal structural elements are often flexible, providing a lower degree of coupling to the walls which tend to vibrate more independently (14) (15). Therefore, for these types of buildings, the hypothesis of equal storey horizontal displacement used in the most common seismic assessment methods cannot be applied. This is the case of several one-storey buildings, usually characterized by the absence of inter-storey floors and internal partitions which would confer stiffness to TRBs' structure. Therefore, these kinds of buildings cannot be considered 'simple buildings' for which no explicit safety verification is required (11). Furthermore, the condition (11) (13) that "for unreinforced masonry buildings, walls in one direction should be connected with walls in the orthogonal direction at a maximum spacing of 7 m" is often not met by several existing TRBs.

Since the global box-type behaviour required by the European Codes could lead to invasive building rehabilitation work which may cause the loss of TRBs' original character and cultural value (e.g., construction of concrete floors and construction of connected tie beams and lintels), in previous studies (16) (17) two models based on simplified methods for stone masonry walls were proposed. In these two models, which were related to 'first damage mode' and 'second damage mode' mechanisms, respectively, charts were presented to provide simple tools for the safety assessment of TRBs subject to seismic loads, as a function of loading conditions and wall dimensions.

In this paper, the model for the safety assessment of buildings for in-plane seismic actions and vertical loads ('second damage mode') was applied to a case study in order to examine the structural behaviour of one-storey TRBs characterized by the absence of stiffening structures and openings of various sizes with misaligned lintels. As was found in a previous study (18), this horizontal irregularity could affect in-plane seismic capacity of masonry walls with openings. The TRBs considered in this study are farm buildings originally used as warehouses (Figure 1) where lintels misalignment may have been caused by indoor environmental requirements for agricultural products stored or previous building interventions.

The TRBs' seismic behaviour was analyzed by varying some of the most significant model parameters, in relation to the application of two different standards and possible retrofitting interventions. These parameters were employed in the computation of the seismic load and the ultimate capacities of the pier panels. With regard to the retrofitting interventions, an hypothesized adaptive reuse was selected among those suitable to improve the horizontal regularity of the openings.

In the field of agricultural engineering research, the analysis of the seismic behaviour of TRBs is of relevant importance, particularly in earthquake-prone areas, due to the risk of loss of human lives, agricultural production, and livestock. Therefore, this research can help bridge the gap between studies concerning the structural analysis of rural buildings, which,

a) b)

Figure 1. Examples of traditional rural masonry buildings located in Eastern Sicily.

at present, are mainly focused on a few typologies of farm buildings such as silos (19) (20) and structures for protected cultivation (21) (22).

2. REGULATORY FRAMEWORK

The Eurocode 8 (EC8 hereafter) and, specifically, part 1 and part 3 (EC8-1 and EC8-3 hereafter) (11) (12) have been structured to provide a common approach for the design of masonry buildings in seismic areas. Nevertheless, when dealing with safety levels and classification of retrofitting interventions of existing masonry buildings, the EC8 shows some lacking aspects because some concepts are not fully compatible with real applications (14) (23), especially for architectural heritage having historical and cultural values (24) (25).

Italian standards have been improved towards a more suitable safety assessment for existing masonry buildings through transposing Eurocodes to the Italian experience. In detail, Ministerial Decree D.M. 14/09/2005 (NTC05 hereafter) (26) was the first Italian Standard containing the principles of EC8.The latest Italian Standard for buildings in seismic areas is the New Technical Code introduced by the Ministerial Decree D.M. 14/01/2008 (NTC08 hereafter) (13), which substitutes NTC05. NTC08 became mandatory on 1st July 2009 to shorten its transitional phase (due to expire on 1st July 2010) with the aim to apply its recommendations to the Abruzzi reconstruction after the earthquake. During the transitional phase of applying NTC08, other decree explanatory notes (DEN09 hereafter) (27) were issued.

The calculations proposed in this work are based on NTC08 and compared to those in compliance with NTC05 (which adopts the 'significant damage' limit state of EC8), integrated with their explanatory notes.

3. MATERIALS AND METHODS

3.1. The Building under Study

The TRB analysed in this paper appertains to a building typology highly recurrent in Eastern Sicily (Italy) and devoted to wine storage and production. It is a one-storey TRB (Figure 2) located at an altitude of 350 m above sea level in the Etnean area which is one of the most earthquake-prone Italian areas. To date, it is used as warehouse for agricultural products. The walls are made of well-structured rough-shaped basaltic stone masonry which is highly frequent in the considered area (28). The double-pitched roof has a wooden bearing structure and is covered by traditional curved tiles made of baked-clay.

Some alterations of the original character of the facades, probably due to a variation of its original intended use, resulted in the misalignment of the external wall openings. The lintels of the openings are built by using materials having different mechanical characteristics. This is the case of many TRBs located in the Etnean area, where previous studies revealed that openings have an internal and an external flat-arch lintel: the external one is made of two blocks of lavastone, the internal one is made of wooden beams (28).

In this paper a further geometrical configuration of the wall openings was considered by assuming the building to be reused for rural tourism activities (e.g., tourist accommodation and agriculturally-based activities). This involved the lowering of some window openings down in order to assure a view from the inner rooms to the outside, and widening some door openings to allow vehicle passage.

3.2. Modelling the Seismic Behaviour of One-Storey TRBs

In this study, the structural scheme used to simulate the behaviour of the building under study considered each wall independently, as set out in Section C8.7.1.1 of DEN09 and adopted in other research (29). In detail, the building was subdivided into four perimeter walls which were separately studied, by modelling each shear masonry wall with openings as a shear-type structural system. In such a system the wall is composed of pier panels and masonry spandrels.

By applying the seismic safety assessment model described in a previous work (17) and summarized in Figure 3, the in-plane structural behaviour of the East longitudinal wall, analyzed in the current functional destination (Figure 4a) and in the hypothesized adaptive reuse (Figure 4b), is discussed in the following of this paper. This longitudinal wall was selected among the others as it had the lowest wall-to-opening ratio, and showed the absence of connected crossing walls which would have improved its overall resistance.

The mechanical resistance parameters of this masonry, classified as 'well-structured rough-shaped stone masonry', are re-

(a) (b)

Figure 2. The building under study, located in the Etnean area.

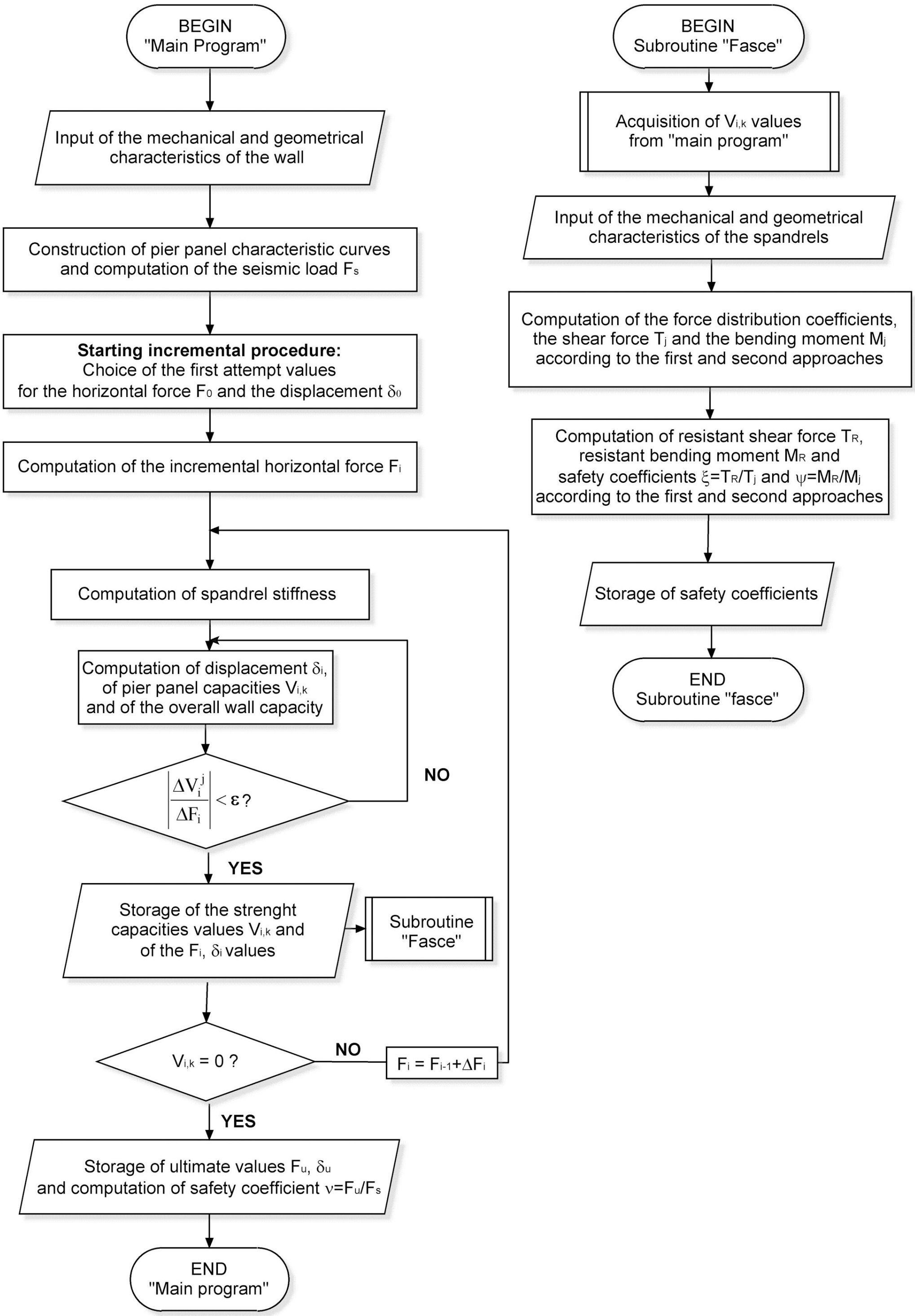

Figure 3. Flowchart of the seismic assessment model for masonry walls.

ported in Table 1 according to the considered standards. The values of the masonry compression resistance (f_m) in NTC08 are comparable with those adopted in studies on stone masonry (30) or resulting from tests using flat jacks (31). Studies on lava-stone masonry (32) (33) recommend a masonry specific weight value (γ_m) in the range 18 ÷ 25 10^{-6} Nmm^{-3}. With regard to these parameters, it should be underlined that many factors affect masonry in-plane behaviour, such as the geometry of the wall (i.e., height, width, thickness), the mechanical and geometrical properties of the masonry constituents (stones and mortar), the building techniques adopted, the geometrical characteristics of the openings.

As uncertainty is typical of structural evaluation of existing building, EC8-3 and NTC08 introduced different sets of materials, structural safety factors and associated analysis procedures, defining three knowledge levels (KLs) related to the mechanical properties of the masonry constituents, and the geometrical properties and building details (i.e. connection type and reinforcing components) of the structure to be evaluated. In the absence of experimental values of resistance for masonry walls, the knowledge level could be KL1 or KL2 type (namely 'limited' and 'normal' knowledge level in EC8-3). In Table 1 the confidence factors (CF) related to the KLs were reported.

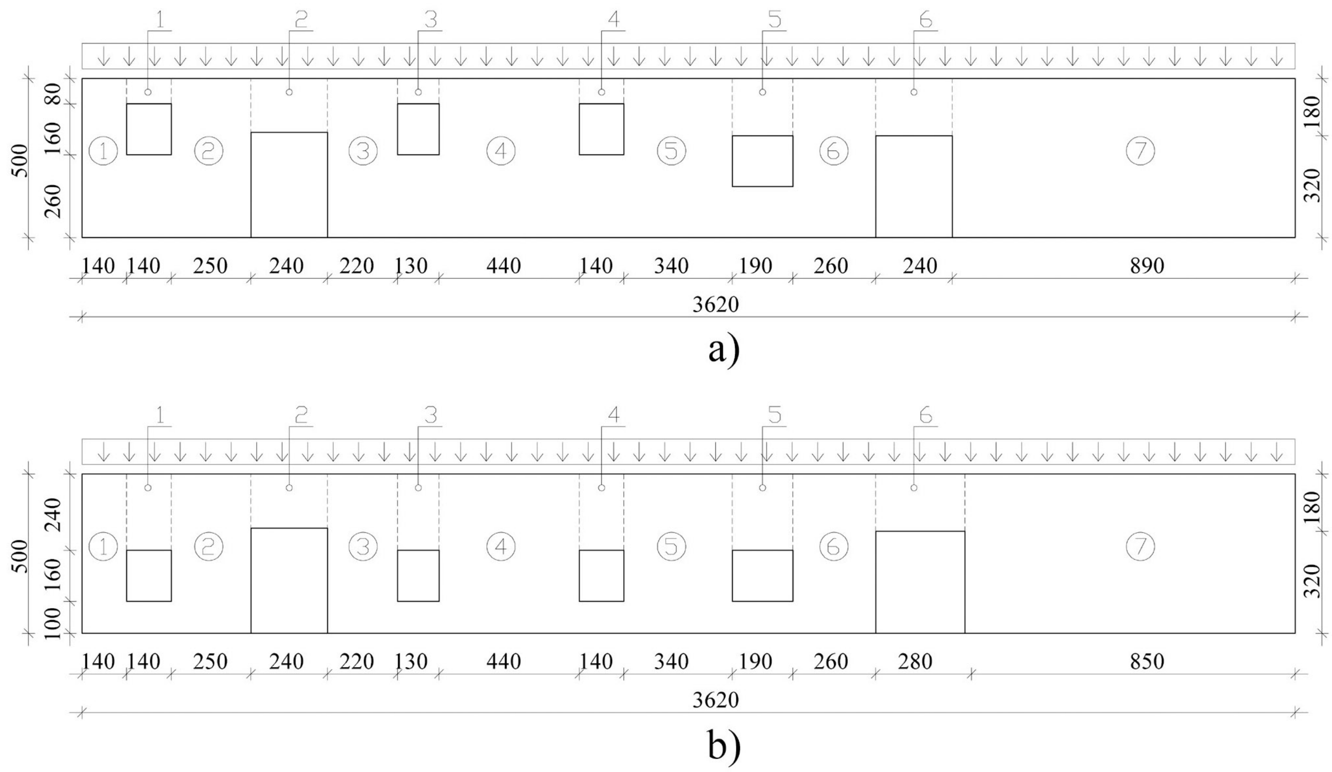

Figure 4. Scheme of the wall analysed in the simulations.

Tri-linear characteristic (i.e., force-displacement) curves of the pier panels were constructed by using a non-linear algorithm applied in previous studies (17) (34). In the adopted model (Figure 3), the principal failure mechanisms of masonry pier panels due to shear and flexure were considered: diagonal-cracking shear failure, bed-joint-sliding shear failure, and flexural failure. For the analysed masonry wall, the shear capacities corresponding to diagonal-cracking shear collapse ($V_{u,diag}$) and bed-joint-sliding shear collapse ($V_{u,oriz}$) were calculated in compliance with NTC08.

Since the use of different relations to compute the ultimate capacities V_u of the masonry pier panels may influence the overall performance of the wall, in this paper the flexural capacity $V_{u,flex}$ was calculated in compliance with NTC08 ($V_{u,flexNTC}$) as well as according to some Authors (34) ($V_{u,flexAU}$). For the computation of V_u, in the absence of specific experimental data, some masonry strength characteristic values (e.g., flexural resistance of the spandrel) were conservatively set according to Fusier and Vignoli (34). It is widely recognized, in fact, that in this field there are few experimental studies due to the difficulty in obtaining these values (35).

With regard to the computational height of the pier panels (*h*), an univocal relation was not found in literature (35) (36) and in specific standards. In Italy a specific technical norm (37) suggests that *h* should be "equal to the interstorey height though, strictly speaking, it would be more correct to consider the height of the openings". Therefore, the model was applied in order to evaluate wall behaviour in relation to three different scenarios of pier-panel height: the first is the height of the openings (hp_1); the second is a function of the 'effective height' as reported by Dolce (38) (hp_2); the third (hp_3) is the distance between the wall base and the axes of masonry spandrels.

The graphical symbols for the collapse mechanisms of pier panels and masonry spandrels were reported in Figure 5. Here, condition $\delta_1 \leq \delta \leq \delta_2$ refers to the displacement related to the elasto-plastic phase of the characteristic curve of the

Table 1. Mechanical characteristics of the wall masonry (i.e., ranges of shear and compression resistance, ranges of shear and elastic modulus, and specific weight), and knowledge levels for the different standards considered.

'WELL-STRUCTURED ROUGH-SHAPED STONE MASONRY'	SHEAR RESISTANCE (τ_m)	COMPRESSION RESISTANCE (f_m)	SHEAR MODULUS (G)	ELASTIC MODULUS (E)	SPECIFIC WEIGHT (γ_m)	KL1	KL2
	Nmm^{-2}	Nmm^{-2}	Nmm^{-2}	Nmm^{-2}	Nmm^{-3}		
NTC05	0.056 ÷ 0.074	1.50 ÷ 2.00	250 ÷ 330	1,500 ÷ 1,980	22.0 10^{-6}	Min E Min f_m Min τ_m CF = 1.35	Avg E Avg f_m Avg τ_m CF = 1.20
NTC08	0.056 ÷ 0.074	1.50÷2.00	500 ÷ 660	1,500 ÷ 1,980	22.0 10^{-6}	Avg E Min f_m Min τ_m CF = 1.35	Avg E Avg f_m Avg τ_m CF = 1.20

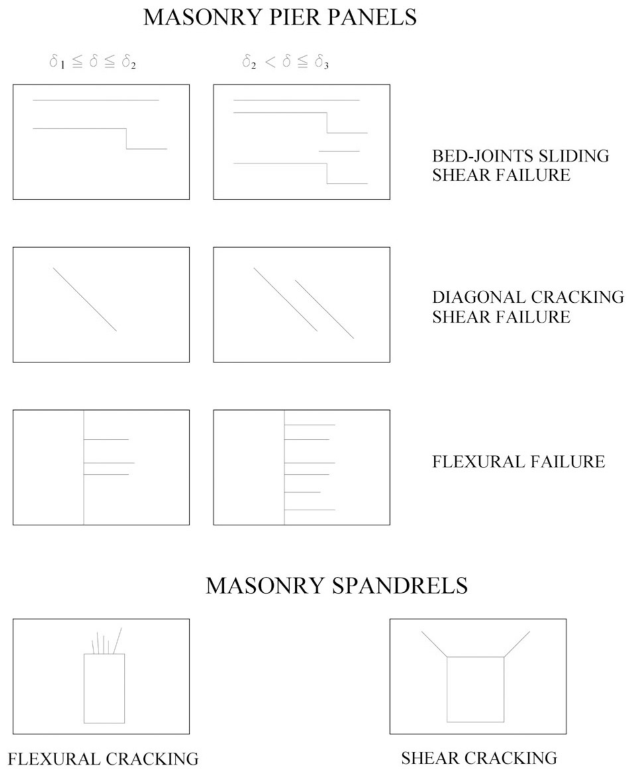

Figure 5. Graphical symbols of the considered failure modes of pier panels and masonry spandrels.

pier panel, whereas condition $\delta_2 \leq \delta \leq \delta_3$ indicates the displacement related to the plastic phase (17) (34).

The conventional seismic loads (F_s) for the analysed masonry wall were computed by applying the two static methods described in NTC08 and NTC05. In general, the limit state of reference for masonry structures is the Ultimate Limit State (ULS). In NTC05 the ULS associated to a reference probability of exceedance (RPE) of 10 % in 50 years, corresponds to the significant damage state (SD) of EC8 and to the life safety limit state (LSLS) of NTC08. The collapse limit state (CLS) defined in NTC08 refers to a RPE of 5 % in 50 years. Moreover, the relation for computing the seismic action according to NTC05 is straight forwardly derived from EC8, whereas its definition in NTC08 for that action is site specific depending on the location of the buildings. Therefore, in the analysed case study the coefficients required for the F_s computation were related to the second category seismic areas (medium seismicity) as per NTC05, whereas for CLS and LSLS they were derived from tab.1-Annex B of NTC08 that provides values related to TRB's geographical coordinates.

The force-based incremental iterative procedure shown in Figure 3 allowed the computation of the capacity curve of the masonry shear wall as well as the safety verification of masonry spandrels.

Exhaustive experimental trials are necessary to find out the real flexural strength capacity of masonry spandrels of existing buildings, taking into account their different typologies. The literature lacks experimental studies which can thoroughly evaluate resistance, strain behaviour and dissipative capacity of masonry spandrels subject to seismic forces (33).

EC8-1 states that: "In the structural model masonry spandrels may be taken into account as coupling beams between two wall elements, if they are regularly bonded to the adjoining walls and connected both to the floor tie beam and to the lintel below". The NTC05 considers, instead, that a certain flexural resistance could develop in masonry spandrels if there is at least one horizontal tensile-resistant element (e.g., chains, lintels, tie beams, steel ties, and other strengthening elements like fibre reinforced polymer bands). When an axial force comes about thanks to these other structural elements, masonry spandrels can be simulated as:

a) resistant masonry elements, if the hypothesis of connection between them and other structural elements is not completely effective yet there is some mechanism that provides axial compression in the masonry spandrels (39). In this study this hypothesis of spandrel modelling (SM1 hereafter) is fulfilled by supposing that axial compression is provided by a steel tie acting as a chain along the spandrel;
b) low-slenderness reinforced masonry beams if the connection can be considered as completely effective. In this study this hypothesis of spandrel modelling (SM2 hereafter) is fulfilled by assuming that the effectiveness of connections is guaranteed by a tie beam built in the inner side of the wall and fixed to the masonry by the insertion of steel bars and grouting (39) or by a tie beam built within the wall thickness (40).

4. RESULTS AND DISCUSSION

4.1. Influence of the Standard on the Seismic Assessment Simulations

Figures 6a, 6b, and 6c show the results of the seismic assessment simulations to evaluate the wall behaviour in relation to the different scenarios of pier-panel height and masonry spandrel modelling. In detail, the figures report the crack development of pier panels and masonry spandrels at the increasing of the horizontal in-plane forces (F_h), which are expressed as a function of F_s, up to wall collapse. Moreover, spandrel cracking is depicted in thin black lines for SM1 and in thick black lines for SM2. The graphical symbols related to the collapse mechanisms reported in Figure 6 are clarified in Figure 5.

In each figure, the bottom scheme shows the first pier-panel cracking, meaning that the elastic phase of the panel was over and the elasto-plastic phase had begun, and/or the first failure in masonry spandrels. The top scheme shows wall collapse at ultimate in-plane strength capacity (F_u) and the ultimate displacement of the *i-th* pier panel ($\delta_u(i)$). The other schemes show intermediate damage levels, e.g., the elasto-plastic limit of all the pier panels, when the first cracking takes place in all the pier panels; plasticity, when one of the pier panels reaches the plastic phase of its characteristic curve; seismic performance state, when F_h about equals the seismic force F_s; the shear or flexural cracking of all the masonry spandrels when the shear or flexural verification of all masonry spandrels fails.

The results of the simulations as per NTC08 showed some differences compared to those obtained using NTC05. Though the crack development of pier panels and masonry spandrels was similar yet F_u varied since some pier panels exhibited different collapse mechanisms and thus their characteristic curves differed. In the scenario hp_3, for instance, the collapse

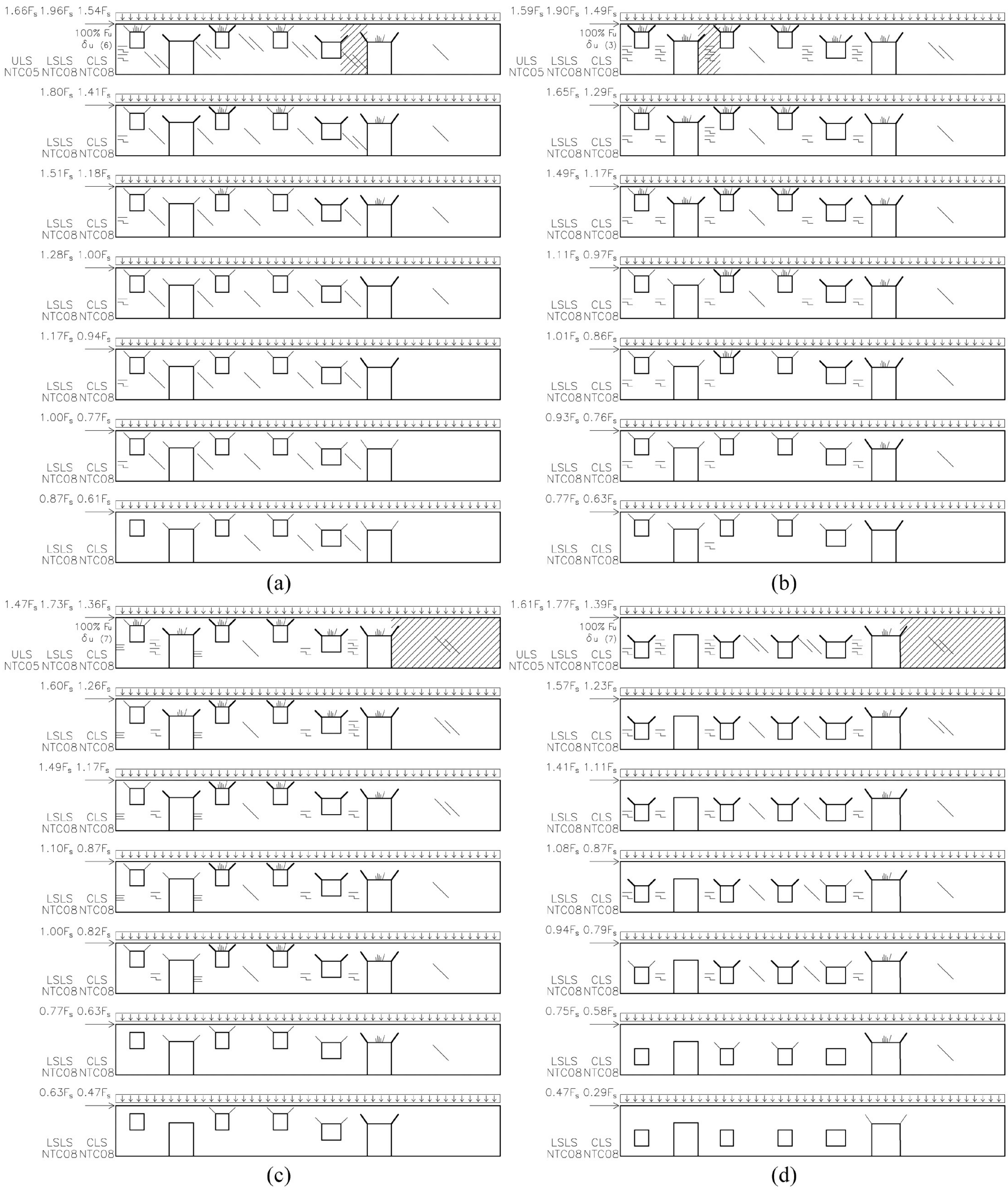

Figure 6. Crack development patterns of the considered wall for the different pier-panel heights: a) first scenario; b) second scenario; c) third scenario; d) hypothesized adaptive reuse.

mechanism of the first pier panel changed from flexural failure as per NTC05 to bed-joint-sliding shear failure according to NTC08. Moreover, the ULS of NTC05 always led to lower wall seismic safety coefficients ν (defined as the ratio between F_u and F_s) than the LSLS of NTC08 while always producing higher ones than the CLS of NTC08.

The results showed that the simulations were affected by the application of both the different values of the mechanical characteristic recommended by the two standards and the different computation of F_s as per the two standards. In fact, according to NTC08 the values of f_m and the G modulus increased nearly twofold if compared to those recommended by NTC05 (Table 1). As regards the computation of F_s, the NTC08 gave more geographically differentiated values for seismic load as its computation parameters were related to geographic coordinates rather than to uniform values in each of the seismic zones defined by NTC05.

The KL variation from KL2 to KL1 highlighted once more a different mechanical behaviour for some of the pier panels. According to NTC05, the reduction of ν due to the considered KL variation was lower than that obtained as per NTC08. This was caused by the different values of the elastic modulus provided by the standards (Table 1). Moreover, the increase in CF values due to the variation of KL produced a reduction in strength capacity causing a change in pier-panel collapse mechanisms. Therefore, the building characterization of TRBs as well as the analysis of their current conservation state are crucial to define the appropriate CF value in the norms, especially if the building had been subject to different rehabilitation works over the years (25).

4.2. Influence of Pier-Panel Height on Wall Seismic Behaviour

By considering the wall seismic safety coefficients ν, Figures 6a, 6b, and 6c show that the scenario hp_1 was less conservative than the others since it provided higher strengths and safety coefficients, while the scenario hp_2 led to intermediate results of hp_1 and hp_3.

Furthermore, the different scenarios of *h* resulted in different wall damage conditions. In fact, the first cracking of masonry occurred in different pier panels: the sixth one in hp_1, the third one in hp_2 and the seventh one in hp_3. When the strength capacity of the wall exceeded F_s, the scenario hp_1 always determined pier panels to reach the plastic phase for F_h/F_s values higher than those related to hp_2 and hp_3 (for instance, in the LSLS state the F_h/F_s values were 1.80, 1.65, and 1.49 in the three scenarios of *h*, respectively, as shown in Figures 6a, 6b, and 6c).

These results are also interrelated to pier-panel collapse mechanisms which in turn depended on *h*. In fact, the collapse mechanisms of almost all the pier panels changed from diagonal cracking failure in the scenario hp_1 to bed-joint-sliding shear failure and flexural failure in hp_2 and hp_3.

By analysing the pier-panel characteristic curves corresponding to the three different scenarios of *h*, the most frequent pier-panel failure mechanism was diagonal cracking shear failure though there was also some bed-joint-sliding shear failure. In general, for low constructions with wide pier panels, the most frequent condition determining collapse is diagonal cracking shear failure. Yet, when normal stress decreases, the failure is not due to exceeding the tensile resistance governing the collapse mechanism of shear with diagonal cracking, but rather by exceeding the bed-joint-sliding shear resistance (Figure 7a). These changes in collapse mechanisms affected wall collapse as they caused different pier panel to reach failure (top schemes in Figures 6a, 6b, and 6c). Yet, collapse mechanisms depended also on pier-panel slenderness. In fact, when analyzing the variation of pier-panel failure mechanisms in relation to both their slenderness and normal stress (σ_o), it resulted that:

- walls subject to low values of σ_o, as occurred in the analysed one-storey TRB, showed bed-joint-sliding shear failure until a threshold value where it turned to diagonal cracking shear failure, as exemplified in Figure 7a for pier-panels having a slenderness value of 1.6, and wall thickness equal to 0.6 m, in the KL2 case;
- squat masonry pier panels experienced diagonal cracking shear failure for low values of slenderness, bed-joint-sliding shear failure for higher values of slenderness, whereas flexural failure was attained for the highest values. This variation of pier-panel failure mechanisms in relation to their slenderness is shown in Figure 7b which exemplifies pier-panel strength capacity V_u in relation to slenderness for pier panels of 3 m height and subject to normal stress σ_o of 0.14 MPa, in the KL2 case.

Therefore, in the simulations it was found that, when σ_o kept under the threshold value, failure varied from diagonal cracking shear failure to bed-joint-sliding shear failure at increasing of the slenderness. For instance, this occurred for the second pier panel. When pier panels were simulated as slender (i.e., slenderness ≥ 1.5), for instance the third one in hp_3, failure was due to flexure whereas diagonal cracking shear failure and bed-joint-sliding shear failure were the failure modes in hp_1 and hp_2, respectively (Figures 6a, 6b, and 6c). These different predictions of failure mechanisms would modify the pier-panel characteristic curves as well as the capacity curve of the whole structure.

Moreover, when different relations were used to account for the flexural capacity of masonry pier panels (34) (41), results showed that in some cases the condition $V_{u,flexAU}<V_{u,oriz}$ occurred for low axial-force values typical of masonry build-

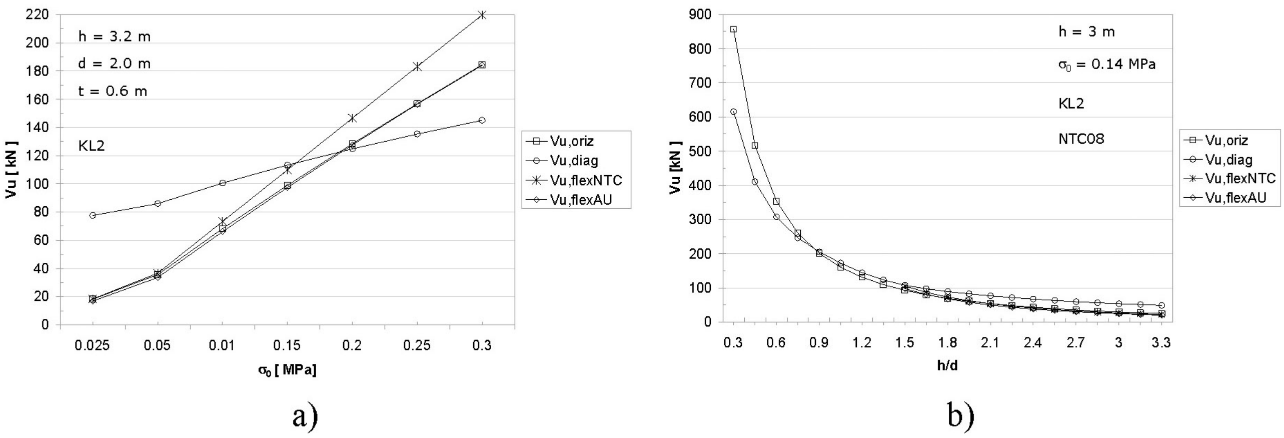

Figure 7. Pier-panel strength capacities V_u in relation to: a) the normal stress σ_o exemplified for a pier panel with slenderness equal to 1.6 in the KL2 case; b) pier-panel slenderness, for pier-panels having a 3 m height and subject to a normal stress σ_o equal to 0.14 MPa in the KL2 case.

ings and NTC08 overestimates the ultimate force ($V_{u,flexNTC}$) (Figure 7a), resulting in wrong predictions of failure mechanisms for some values of the applied axial force (41). Also this overestimation could affect pier-panel characteristic curves as well as the capacity curve of the whole structure.

4.3. Improvements of the Wall Seismic Behaviour through Retrofitting Interventions

In SM1, diagonal-cracking shear failure for well-sized spandrels, was more limiting than other failure mechanisms, which in turn would affect the spandrels having a higher length/height ratio (*r*). In fact, according to SM1, all the cases of spandrel failure due to shear occurred before flexural failure (Figures 6a, 6b, and 6c). Generally, the spandrels having a more uniform *r* (fixed shear force transmitted by the masonry piers, fixed *h*), show failure mainly due to shear while spandrels having a higher and more differentiated *r* frequently exhibit flexural failure (data not reported). Under SM2, flexural failure did not occur in the examined cases.

The results related to the reuse simulation (Figure 6d), computed for hp_3 which showed the most diffuse spandrel cracking, demonstrated that the considered geometrical modifications of wall openings position and width improved both pier panel and masonry spandrel resistances. In fact, the first cracking and the beginning of the plastic phase of the pier panels occurred earlier in the simulation of the original building condition than in the reuse one, and also masonry spandrels cracking occurred later in the reuse simulation (Figures 6c and 6d). Therefore, enhancing the horizontal regularity of the wall openings improved the overall wall resistance. This result confirmed what observed in previous research (18) (42).

Masonry spandrels, however, often exhibit shear failure and thus need to be strengthened in order to make it possible to use the shear-type modelling. By applying the model in the case of a masonry wall strengthened by mortar injections, a higher increase of ν and a higher resistance of spandrels were achieved in comparison to the case of unstrengthened masonry. In the LSLS limit state, for instance, ν increased from 1.77 (Figure 6d) to 2.07 and spandrel cracking was prevented for $F=F_s$ in the SM2 case.

On the basis of these results it appears to be very important to perform experimental tests and refine models to achieve simulations of masonry building responses to earthquakes closer to their real behaviour, also taking into account the characteristics peculiar to specific typologies of buildings such as those considered in this study (25) (43) (44) (45).

5. CONCLUSIONS

In this paper, a number of issues concerning the seismic behaviour and the modelling of one-storey stone-masonry traditional rural buildings (TRBs) arose from the results obtained, by modifying a number of parameters of the seismic safety assessment model and defining two types of spandrel modelling related to different retrofitting interventions.

The change in the Italian Standards from NTC05, which shares common aspects with EC8, to NTC08, has produced various results in terms of wall safety coefficients and pier-panel failure mechanisms due to both the different mechanical characteristics recommended by the two standards and the different computation methods of seismic load.

The results showed that pier-panel height plays a significant role in the seismic response of the entire wall, given that it affected the seismic behaviour of both pier panels and masonry spandrels. When pier-panel height varies, the panel failure mechanism may change, causing a different overall wall strength capacity which is not always determined by the collapse of the same pier panel.

For walls subject to low normal stress, as in the case of TRBs, the prevailing failure mechanism is due to shear and, if it is due to bed-joint-sliding or diagonal cracking, it depends on both normal stress and pier-panel slenderness which, in turn, is related to the pier-panel height scenario.

The need for and extent of masonry-spandrel strengthening interventions, depending on the shear force transmitted by the pier panels and their own resistance, is strictly related to the definition of the pier-panel height. The collapse of masonry spandrels due to diagonal cracking shear is associated most frequently to low length/height ratios and generally occurs before collapse due to flexure, also depending on spandrel modelling.

The knowledge level of masonry mechanical characteristics produces effects on both the ultimate capacity of the wall and the pier-panel collapse mechanisms, causing a different seismic behaviour of the wall.

Finally, interventions aimed at the improvement of horizontal regularity of wall openings and the strengthening of wall masonry are suitable to increase wall resistance.

AUTHORS' CONTRIBUTION

Prof. Claudia Arcidiacono elaborated the model, carried out the simulations and contributed to the writing of the text.

Ph.D. Eng. Simona M.C. Porto carried out the study on TRBs typologies and contributed to the writing of the text.

Prof. Giovanni Cascone coordinated the research.

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NOMENCLATURE

CF	Confidence factor
CLS	Collapse limit state
DEN09	Explanatory Notes (Newsletter of 02/02/2009)
E	Elastic modulus
EC8, EC8-1, EC8-3	Eurocode 8, Eurocode 8 part 1, Eurocode 8 part 3
F_h	Wall in-plane strength capacity
f_m	Masonry compression resistance
F_s	Conventional seismic load
F_u	Wall ultimate in-plane strength capacity
G	Shear modulus
h	Pier-panel height
hp_1	First hypotheses of pier-panel height
hp_2	Second hypotheses of pier-panel height
hp_3	Third hypotheses of pier-panel height
KL	Masonry knowledge level
LSLS	Life safety limit state
NTC05	Technical Code (Italian Ministry Decree D.M. 14/09/2005)

NTC08	Technical Code (Italian Ministry Decree D.M. 14/01/2008)
r	Spandrel length/height ratio
RPE	Reference probability of exceedance
SD	Significant Damage state
SM1	First approach for masonry spandrel verification
SM2	Second approach for masonry spandrel verification
TRB	Traditional rural building
V_u	Pier-panel shear capacity
$V_{u,diag}$	Pier-panel diagonal-cracking shear collapse
$V_{u,flex}$	Pier-panel flexural capacity
$V_{u,flexAU}$	Pier-panel flexural capacity computed in compliance with Fusier and Vignoli (1993)
$V_{u,flexNTC}$	Pier-panel flexural capacity computed in compliance with NTC08
$V_{u,oriz}$	Pier-panel bed-joint-sliding shear collapse
ULS	Ultimate limit state
δ	Wall displacement
γ_m	Masonry specific weight
ν	Seismic safety coefficient
σ_o	Normal stress
τ_m	Masonry shear resistance

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