Simplified Out-of-plane Resistance Verification for Slender Clay Masonry Infills in RC Frames

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ABSTRACT

Field observations from recent earthquakes in Italy have shown numerous examples of unsatisfactory out-of-plane performance of masonry infills and façade veneers. Extensive damage in some cases has occurred also in recently constructed buildings, indicating the need for improved detailing and a more effective design procedure.

The design recommendations related to the verification of the out-of-plane stability in currently adopted code regulations are rather general, providing a simplified approach for the approximation of seismic action effects. However, no recommendations related to the evaluation of the infill resistance are given. In principle, resistance models based on arching action can be used, but they may be considered appropriate only for undamaged infills built in contact with the frame. Due to the directionally combined infill degradation in the case of horizontal loading induced by earthquakes, the out-of-plane strength verification of infills damaged due to excitations in the in-plane direction appears to be governing the design. In this work, the application of the current design procedure for non-structural masonry is studied, and the relation between in-plane and out-of-plane damage is examined, starting from existing experimental results. Consequently, proposals for a simplified design approach are provided, accounting for expected levels of in-plane damage in the out-of-plane safety verification.

1 INTRODUCTION

The out-of-plane performance of masonry infills in RC frame structures has been addressed in several previous analytical and experimental studies, *e.g.* (Angel *et al.* 1994), (Calvi and Bolognini 1999), (da Porto *et al.* 2012), but a significant number of problems related to the prevention of out-of-plane failure has remained unsolved. Following major earthquake events, unsatisfactory performance of masonry infill and brick veneer in the out-of-plane direction has been repeatedly reported, *e.g.* (Braga *et al.* 2011), (Ricci *et al.* 2011). Extensive damage has been observed also in newly constructed buildings designed following modern seismic code regulations, indicating the need for improved detailing and a more effective design procedure.

The presence of a significant correlation between the in-plane and out-of-plane infill response and damage propagation has been stressed by several authors, *e.g.* (Morandi *et al.* 2011a), (Hak *et al.* 2012a), (Vicente *et al.* 2012). Referring to infill resistance verifications, the importance to account for the possible out-of-plane strength reduction due to previous in-plane damage has been to some extent recognised and is included in a number of current recommendations for the assessment of existing buildings, see *e.g.* (FEMA-306, 1998), (Al-Chaar 2002). Nevertheless, in particular according to European and Italian National Code provisions, such effect is commonly not accounted for in the seismic design of new masonry infilled RC structures, possibly inducing misleading conclusions in the design and detailing procedure that may result in unsatisfactory infill response.

2 FIELD OBSERVATIONS

In recent years, several strong earthquakes have occurred on the Italian territory, providing numerous reported examples of serious out-of-plane damage and loss of stability of many masonry infill and veneer typologies. In particular, observations from the latest two major events, *i.e.* in Abruzzo (L'Aquila), 2009, and in Emilia, 2012 (see *e.g.* Figure 1.a and Figure 1.b, respectively) point to a series of problems in the seismic response of typical non-structural masonry in buildings of recent construction, indicating possible deficiencies in the current, commonly applied, design and detailing approach.

Figure 1. In-plane masonry infill/veneer damage and out-of-plane expulsion: (a) Abruzzo, Italy, 2009; (b) Emilia, Italy, 2012 (Magenes *et al.* 2012)

Generally, higher values of out-of-plane seismic actions are imposed for elements in upper parts of the building, where the occurrence of out-of-plane failure is more likely to be expected, as illustrated *e.g.* by Figure 2.a, showing the example of a masonry veneer at the top of the structure that has detached and fallen out-of-plane.

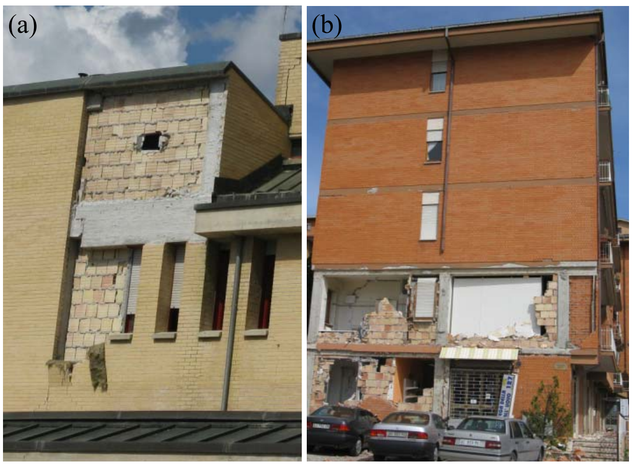

Figure 2. L'Aquila, Italy, 2009: (a) Out-of-plane masonry veneer expulsion; (b) Out-of-plane masonry veneer/infill expulsion

On the other hand, the failure of masonry veneers and/or infills in intermediate and/or lower storeys (see *e.g.* Figure 2.b, Figure 3.a and Figure 3.b), where lower out-of-plane seismic demands are imposed than at the top, has often been reported, see also (Braga *et al.* 2011), (Vicente *et al.* 2012). This fact also indicates that infills, having previously sustained larger in-plane drift demands, possess a significantly reduced out-of-plane resistance that has to be accounted for in the verifications required for non-structural elements at the ultimate limit state.

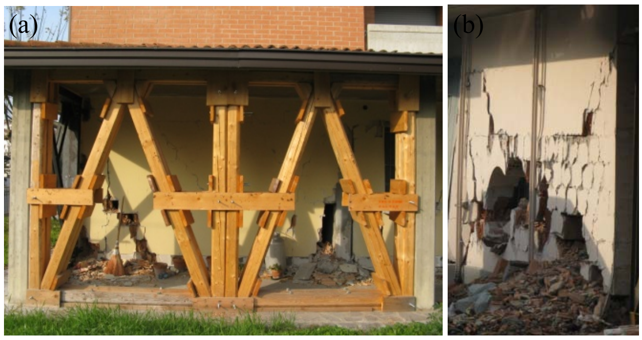

Figure 3. Emilia, Italy, 2012 (Manzini and Morandi 2012): (a) In-plane masonry infill damage and onset of out-of-plane expulsion; (b) In-plane masonry infill damage and out-of-plane expulsion

3 CURRENT DESIGN PROVISIONS

3.1 Simplified Seismic Analysis of Non-structural Elements

According to European design provisions, *i.e.* Eurocode 8 – Part 1 (CEN, 2004) and the Italian Norms (D.M. 14/01/08, 2008), non-structural elements of buildings that might, in case of failure, threaten human lives or affect the main bearing structure or services of critical facilities, should be verified to resist the design seismic action. For non-structural elements of great importance or of a particularly dangerous nature, the seismic analysis should be based on a realistic model of the relevant structures and on the use of appropriate response spectra derived from the response of the supporting structural elements of the main seismic resisting system. In all other cases, properly justified simplifications are allowed. This code requirement is usually applied for the out-of-plane response verification of masonry infills in RC frame structures and the simplified verification procedure suggested in the code is regularly adopted in practise.

Thus, the effects of the seismic action are commonly determined applying a horizontal force F_a, acting at the centre of mass of the non-structural element in the most unfavourable direction, as given in Equation (1), where S_a is the seismic coefficient applicable to non-structural elements, W_a is the corresponding weight of the element, γ_a is the importance factor of the element and q_a the behaviour factor of the element; the Italian norms report the same expression (1), considering implicitly the importance factor equal to 1.

$$F_a = \frac{S_a W_a \gamma_a}{q_a} \tag{1}$$

The seismic coefficient S_a may be calculated according to the expression given in Equation (2), where z represents the distance of the centre of mass of the non-structural element from the level of application of the seismic action (*i.e.*, the top of the foundation or a rigid basement) and H is the corresponding building height, see Figure 4.

$$S_a = \alpha S\left[\frac{3(1+z/H)}{1+(1-T_a/T_1)^2} - 0.5\right] \tag{2}$$

The ratio of the design ground acceleration a_g on type A ground, to the acceleration of gravity g is denoted by α, S is the soil factor, T_a the fundamental vibration period of the non-structural element and T_1 the fundamental vibration period of the building in the relevant direction. The seismic coefficient S_a should not be taken less than αS.

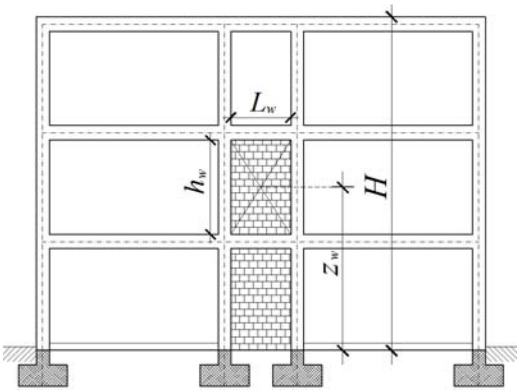

Figure 4. Position of a masonry infill in the RC structure

Note that the seismic coefficient S_a accounts contemporarily for the dynamic amplification of accelerations along the storey height and for the influence of the initial vibration characteristics of the non-structural element with respect to the structure. For a very rigid non-structural element (*i.e.*, $T_a \approx 0$), the expression multiplying αS in Equation (2) results to be equal to $(1+1.5z/H)$. This coefficient accounts for the dynamic amplification of accelerations along the height of the building, which in this case are assumed to increase linearly, resulting in top floor accelerations being 2.5 times larger with respect to those at the bottom.

Similarly, as illustrated in Figure 5, a linear amplification of floor accelerations is assumed in the American (ASCE/SEI 7-10, 2010) and the Canadian (NBCC, 2010) code provisions, with a ratio of top and bottom acceleration equal to 3.0. Such approach, however, has received some criticism in the past and alternative procedures have been suggested (*e.g.*, Drake and Bachman 1995, Taghavi and Miranda 2004). In the New Zealand standard dedicated to the design of non-structural elements and their connections (NZS 1170.5:2004 2004), a bilinear increase of dynamic accelerations along the building height has been introduced instead, considered to represent more realistically the nonlinear building response than the assumption of a linear, first-mode proportional amplification.

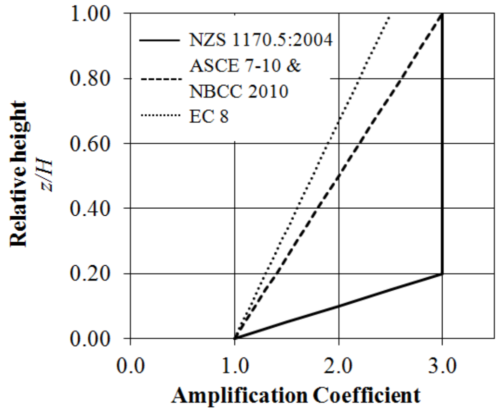

Figure 5. Amplification of accelerations along the building height

3.2 *Safety Verification*

The importance factor for non-structural elements γ_a in the case of masonry infills, according to Eurocode 8 – Part 1, may be taken equal to 1.0, while the upper limit value of the behaviour factor q_a for exterior and interior walls, partitions and façades is defined to be equal to 2.0. The seismic force may be assumed as a distributed load per unit area (kN/m^2) on the surface of a single infill panel of height h_w and length L_w, as given in Equation (3) and illustrated in Figure 6.a.

$$w_a = \frac{F_a}{h_w L_w} \tag{3}$$

The fundamental period T_a of the masonry infill in the out-of-plane direction can be calculated according to the expression for the case of single vertical bending response with hinged ends given in Equation (4), where m_w is the mass of the infill per unit height, E_{wv} is the vertical modulus of elasticity of masonry and I_{wy} the moment of inertia about the longitudinal axis of the horizontal cross section of the panel.

$$T_a = \frac{2h_w^{\,2}}{\pi}\sqrt{\frac{m_w}{E_{wv} I_{wy}}} \tag{4}$$

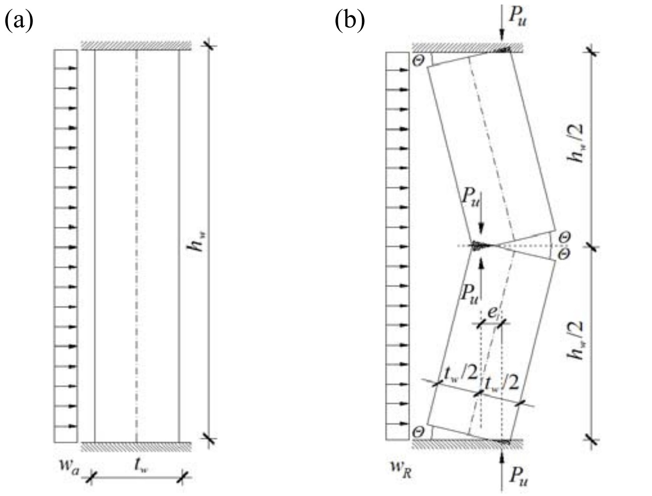

Figure 6. (a) Uniformly distributed out-of-plane action; (b) Formation of the out-of-plane arching mechanism

Clearly, this simplified approach for the approximation of the seismic action, in function of the seismic coefficient S_a given in Equation (2), depends also on the estimation of the fundamental elastic period of the structure T_1, in the direction orthogonal to the plane of the infill. For the evaluation of T_1, Eurocode 8 – Part 1 allows the application of expressions based on methods of structural dynamics (*e.g.* the Rayleigh method) or simplified empirical formulae. For structures up to 40 *m* height, the expression given in Equation (5) is suggested, where H denotes the total height of the building and C_t is defined equal to 0.085 for moment resistant space steel frames, 0.075 for moment resistant spatial RC frames or eccentrically braced steel frames and 0.050 for all other structures.

$$T_1 = C_t H^{3/4} \tag{5}$$

The normalised seismic coefficient $S_a/\alpha S$ as a function of the ratio of periods T_a/T_1 is shown in Figure 7, illustrating that for structures having a higher fundamental period, normally lower force demands are obtained, since T_a/T_1 is expected to be smaller than 1.0.

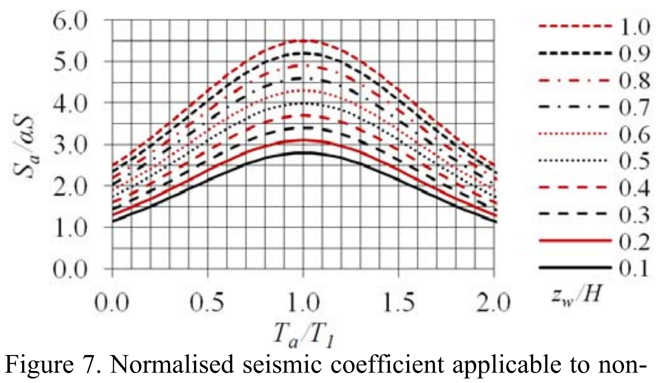

Figure 7. Normalised seismic coefficient applicable to non-structural elements (Eurocode 8)

Studying relationships for periods of vibration and their employment in linear seismic analysis, (Pinho and Crowley, 2009) have concluded that the application of Equation (5) may be reasonable for the period of vibration of RC frames with rigid infills, using the value of C_t recommended for other structures (*i.e.* $C_t = 0.050$). In fact, even though the out-of-plane verification is performed at the ultimate limit state, when masonry infills are expected to achieve a certain extent of damage, without questioning requirements related to life safety, the assumption of the bare frame fundamental period may not be safe-sided, since a reduced but still important contribution to the structural stiffness is expected to be induced by the infills. The fundamental vibration period of a confined infill in the out-of-plane direction T_a is expected to be rather low, with exception of very slender infills. Subsequently, for the majority of practical design situations, the seismic coefficient S_a is not substantially influenced by the approximate evaluation of the fundamental period of the structure.

To satisfy the safety verification, as given in Equation (6), the seismic force F_a, expressed as equivalent pressure w_a in Equation (3) acting on the masonry infill, needs to be smaller than the corresponding out-of-plane resistance w_R.

$$w_a < w_R \tag{6}$$

Nevertheless, no specific recommendations for the calculation of the infill resistance are provided in Eurocode 8 and in the Italian regulations. In principle, resistance models based on full vertical arching action, as for instance suggested in Eurocode 6 – Part 1-1 (CEN, 2005), may be assumed, although they are typically applied to elements subjected to non-seismic actions (*i.e.*, wind loads), and they may be considered appropriate only for undamaged infills.

3.3 *Arching Action Resistance Mechanism*

For unreinforced masonry walls built between rigid supports that restrain outward movement of any part of the wall in its plane, axial compressive forces are induced as the wall bends. These in-plane compression forces can delay cracking and, subsequently, can produce an arching action that in many cases has a capacity that exceeds several times the capacity of masonry in pure flexure (Drysdale *et al.* 1999). Hence, as shown also experimentally (*e.g.* McDowall *et al.* 1956), under certain conditions, significantly larger loads can be sustained than predicted based on conventional bending analysis. The application of such resistance mechanism for the evaluation of the out-of-plane capacity of undamaged unreinforced masonry infills appears to be appropriate when the panel is built in full contact with the surrounding frame. As demonstrated by (Angel *et al.* 1994), unreinforced masonry infills restrained by bounding frames can develop a significant out-of-plane resistance due to the formation of an arching mechanism, depending in particular on the slenderness ratio of the panel and on the masonry compressive strength.

According to Eurocode 6 – Part 1-1, at the ultimate limit state the verification of masonry walls constructed solidly between supports capable of resisting an arch thrust may be carried out assuming that an arch in the relevant direction develops within the thickness of the wall. The analysis may be based on a three-pin arch with a bearing of the arch thrust at the supports and at the central hinge assumed equal to 10% of the wall thickness t_w. The vertical component of the maximum compressive force P_u acting at each support and at the top of the arch (see Figure 6.b) can be calculated from Equation (7), where L_w is the length of the wall and $f_d = f_{wv}$ the compressive strength of masonry in the direction of the arch thrust, *i.e.*, the vertical compressive strength.

$$P_u = 0.10\, t_w\, L_w\, f_d \tag{7}$$

The corresponding arch rise e, or the lever arm of the couple of forces providing arching action, can be found from Equation (8), where Θ is the inclination angle of each half-height panel and h_w is the height of the panel. Assuming that the deflection of the arch under lateral loads is close to zero (for $\Theta \approx 0$, $\sin\Theta \approx 0$ and $\cos\Theta \approx 1.0$), even in the case of slender infills, the simplified moment of resistance M_R can be determined from Equation (9), resulting in an out-of-plane strength in terms of lateral pressure w_R expressed by Equation (10). Apparently, in this procedure the two significant parameters for the out-of-plane resistance verification of an undamaged masonry infill wall are the slenderness ratio h_w/t_w and the masonry vertical compressive strength $f_d = f_{wv}$.

$$e = 0.90\, t_w \cos\Theta - 0.5\, h_w \sin\Theta \tag{8}$$

$$M_R = P_u e = 0.09\, t_w^{\,2} L_w f_d \tag{9}$$

$$w_R = \frac{8M_R}{L_w h_w^{\,2}} = 0.72\left(\frac{t_w}{h_w}\right)^2 f_d \tag{10}$$

Observations on previous experimental test results, see (Calvi and Bolognini 1999, 2001), addressed also in Section 4.1 of this work, imply the conclusion that in the case of weak masonry infills with mesh reinforcement in the plaster the out-of-plane resistance $w_{R,\chi}$ for undamaged panels may be evaluated accounting for the contribution of the reinforcement to the total bending moment resistance. Assuming the moment resistance due to the presence of vertical reinforcement as given in Equation (11), where A_s is the total cross sectional area of the vertical reinforcement in tension and f_y is the reinforcement yield strength, the combined out-of-plane resistance due to arching action and vertical reinforcement $w_{R,\chi}$ can be expressed by Equation (12).

$$M_{R,\chi} = 0.9 t_w A_s f_y \tag{11}$$

$$w_{R,\chi} = 0.72\left(\frac{t_w}{h_w}\right)^2 f_d + 7.2\frac{t_w}{L_w h_w^{\,2}} A_s f_y \tag{12}$$

For out-of-plane verifications in the design of new RC structures with masonry infills the direct application of the presented expressions for the calculation of the masonry infill resistance, *i.e.* Equation (10) and Equation (12), respectively, for unreinforced and reinforced infill, entails that the masonry is principally expected to be undamaged when subjected to out-of-plane actions. For the safety verification of buildings subjected to seismic actions carried out at the ultimate limit state, commonly required by modern code regulations, such assumption is however not suitable, since simultaneous actions in two orthogonal directions are usually imposed on the structure. Consequently, in order to control efficiently the out-of-plane performance and achieve a satisfactory response in the case of earthquake actions, the potential reduction of the out-of-plane infill strength in proportion to the expected damage due to previous in-plane excitations should possibly be accounted for in codified procedures.

In effect, the application of the presented European code approach for the evaluation of force demands and the simplified method defined above for the estimation of corresponding levels of resistance to a number of case study configurations (Hak, 2010) has shown that the out-of-plane verification for unreinforced undamaged masonry infills does not present a critical issue in the design of infilled RC structures, with the exception of very weak or very slender masonry infills. Therefore, a reasonable approximation of the out-of-plane capacity for masonry infills, previously damaged up to a certain extent in-plane, is considered to be of major importance, not only for the assessment of existing buildings, but also in the design of new RC structures with masonry infills.

4 RESISTANCE REDUCTION

4.1 Available Experimental Results

At present, relatively few test results related to a possible correlation of the out-of-plane resistance and previous in-plane damage are available for current masonry infill typologies, see *e.g.* (Calvi and Bolognini, 1999). With the aim to gather further relevant data, in particular for currently widely adopted infill typologies and innovative techniques, a recent experimental study has been accomplished at the University of Padova (da Porto *et al.*, 2012), and further related results are expected to be obtained shortly at the University of Pavia.

In the experimental study previously carried out at the University of Pavia (Calvi and Bolognini, 1999), related to the in-plane and out-of-plane behaviour of RC frames with clay masonry infills, static tests on full-scale single-story single-bay frame specimens (with a height of 2.875 *m* and a span of 4.50 *m*), designed according to modern seismic code provisions, were performed. Three different types of slender masonry infill were considered; specifically, a traditional unreinforced 13.5 *cm* thick infill typology consisting of horizontally hollow clay brick units with 1.0 *cm* plaster on each side, as well as two corresponding lightly reinforced types of infill, respectively, with rebars in the bed joints and mesh reinforcement in the plaster.

One of the major goals of the study was related to the assessment of the potential for expulsion in traditional and slightly reinforced masonry infill panels, at different levels of previously induced damage, to be achieved based on cyclic in-plane static tests followed by monotonic out-of-plane loading. The tests were performed applying three cycles of horizontal displacements at each target level of in-plane drift, while in the out-of-plane direction a four-point load was imposed on the infill, firstly in the case of an undamaged panel and subsequently on specimens that have previously sustained selected levels of increasing drift and corresponding masonry damage.

4.2 Out-of-plane Strength in Function of In-plane Damage

The associated experimental values of out-of-plane resistance obtained following the in-plane tests (*i.e.*, at 0.0%, 0.40% and 1.20% drift) are summarised in Table 1, expressed as the corresponding equivalent pressure $w_{R,exp}$ acting on the surface of the masonry infill. Alternatively, the out-of-plane strength can be expressed as a fraction of the corresponding value obtained for the undamaged panel, resulting in the experimentally evaluated out-of-plane strength reduction coefficient $\beta_{a,exp}$, summarised in Table 2 and illustrated in Figure 8.a. Table 1 shows that the contribution of the reinforcement in the bed joints in enhancing the out-of-plane resistance is limited, above all in the case of infills undamaged in-plane; a further comment is that the masonry panel with mesh in the plaster is substantially undamaged at in-plane drifts of 0.4% and the out-of-plane resistance can be assumed as if the specimen was not subjected to any in-plane action, justifying the use of a strength reduction coefficient $\beta_{a,exp}$ equal to 1.0, as reported in Table 2.

Table 1. Experimental out-of plane resistance $w_{R,exp}$ in function of previous in-plane drift (Calvi and Bolognini, 1999).

	Unreinforced	Lightly reinforced	
		Rebars in the bed joint	Mesh in the plaster
In-plane drift		$w_{R,exp}$ $[kN/m^2]$	
0.0%	5.62	6.13	-
0.4%	1.50	2.92	7.77
1.2%	1.00	1.80	3.57

Table 2. Experimental out-of-plane strength reduction coefficient $\beta_{a,exp}$

	Unreinforced	Lightly reinforced	
		Rebars in the bed joint	Mesh in the plaster
In-plane drift		$\beta_{a,exp}$	
0.0%	1.00	1.00	-
0.4%	0.27	0.48	1.00
1.2%	0.18	0.29	0.46

Even though only limited data is currently available, resulting in two values of reduced strength at two levels of previous damage for each infill typology, observations on the existing test results indicate that for the estimation of the out-of-plane resistance an experimental reduction may be assumed, descending for increasing levels of previously imposed in-plane drift (see Figure 8.a). Subsequently, considering the need to adopt for possible design applications a simple but effective approach, the out-of-plane strength reduction coefficient β_a may be defined in function of the expected in-plane drift demand δ_w of the infilled frame, expressed by a simplified relation depending on the drift limits δ_m' and δ_u, corresponding to the attainment of damage limitation and ultimate limit state conditions, respectively.

In particular, as illustrated in Figure 8.b, possible approximations of the out-of-plane resistance in function of increasing in-plane drift demands δ_w may be represented by a stepwise decrease given in Equation (13), or more conservatively, a linear reduction by parts given in Equation (14). Values of the parameters δ_m' and δ_u have been evaluated through the calibration of a numerical model on the existing experimental results (Hak *et al.*, 2012b), and are summarised in Table 3.

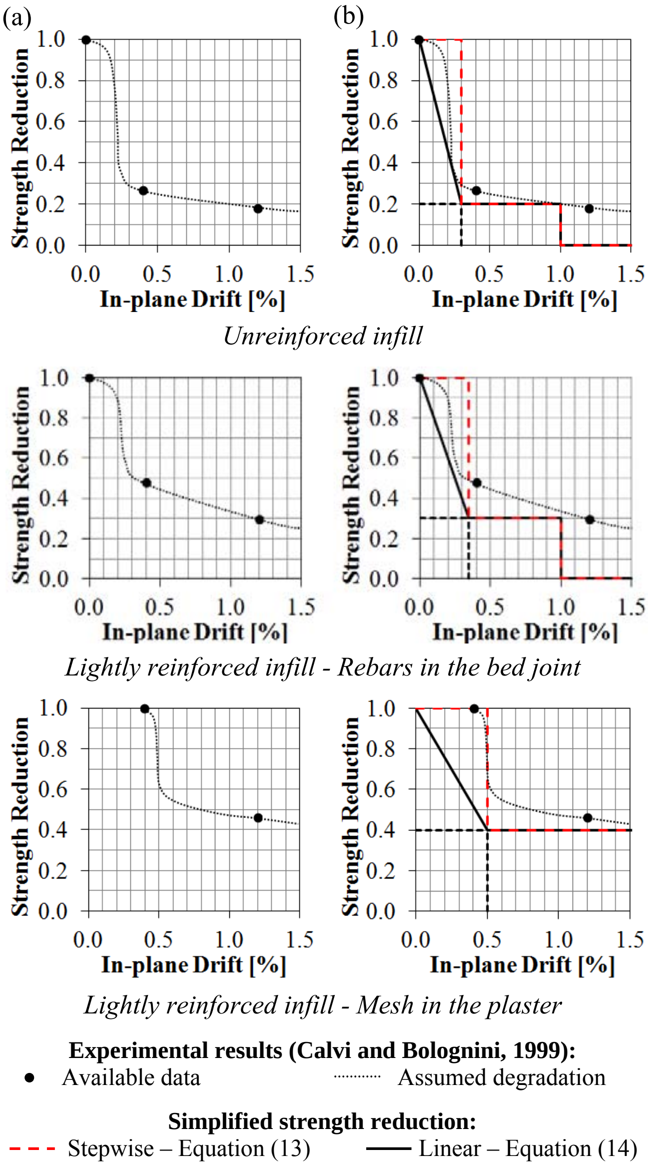

Figure 8. (a) Experimental out-of-plane strength reduction coefficient $\beta_{a,exp}$; (b) Simplified out-of-plane resistance reduction coefficient β_a

$$\beta_a = \begin{cases} 1.0, & \delta_w \le \delta_m' \\ r_a, & \delta_m' < \delta_w \le \delta_u \\ 0.0, & \delta_w > \delta_u \end{cases} \tag{13}$$

$$\beta_a = \begin{cases} (r_a - 1)\delta_w / \delta_m' + 1, & \delta_w \le \delta_m' \\ r_a, & \delta_m' < \delta_w \le \delta_u \\ 0.0 & \delta_w > \delta_u \end{cases} \tag{14}$$

The fraction of remaining out-of-plane strength r_a assumed to correspond to the attainment of the peak in-plane resistance of the infill, and hence, to the drift δ_m', has also been estimated based on experimental observations for the considered types of infill and is presented in Table 4. Note that after exceeding the drift corresponding to the achievement of infill ultimate limit state conditions δ_u, zero out-of-plane strength is assumed.

Table 3. Estimated inter-storey drifts δ_m' and δ_u for RC frames corresponding to damage limitation and ultimate limit state infill performance levels (Hak *et al.*, 2012b)

Limit State	Unreinforced	Lightly reinforced	
		Rebars in the bed joint	Mesh in the plaster
Damage Limitation δ_m'	0.30%	0.35%	0.50%
Ultimate δ_u	1.00%	1.00%	2.20%

Table 4. Assumed fraction of out-of-plane resistance r_a corresponding to peak in-plane infill resistance

Unreinforced	Lightly reinforced	
	Rebars in the bed joint	Mesh in the plaster
	r_a	
0.20	0.30	0.40

5 IMPLICATIONS FOR DESIGN

5.1 *Proposed Design Approach*

In order to carry out out-of-plane safety verification for masonry infills in the design of new RC structures, complying with European seismic code regulations, the proposed out-of-plane strength reduction coefficient β_a may be applied to estimate the reduced out-of-plane resistance, accounting for a certain level of previous in-plane damage that is likely to be sustained by the infill. Given that the infill resistance verification is commonly carried out at the ultimate limit state, the corresponding expected in-plane drift consequently needs to be evaluated.

Assuming that the full contact between the infill and the surrounding structure can be preserved and the arching action remains active, the out-of-plane resistance of a damaged infill $w_{R,\chi,\beta}$ can be found in a simplified manner from Equation (15), reducing the strength of the undamaged panel, evaluated according to Equation (12), applying the corresponding reduction coefficient β_a. Thus, in order to satisfy the out-of-plane resistance verification for a masonry panel that is expected to sustain a certain level of in-plane damage, as given in Equation (16), the seismic force F_a, expressed as equivalent pressure w_a in Equation (3), acting on the masonry infill, needs to be smaller than the corresponding out-of-plane resistance $w_{R,\chi,\beta}$.

$$w_{R,\chi,\beta} = \left[0.72 \left(\frac{t_w}{h_w} \right)^2 f_d + 7.2 \frac{t_w}{L_w h_w^2} A_s f_y \right] \beta_a \tag{15}$$

$$w_a < w_{R,\chi,\beta} \tag{16}$$

The evaluation of the reduction coefficient β_a given by Equation (14), expressed in a conservative manner as a linear function by parts of the in-plane drift of the infilled structure, can be illustrated in a general form, as shown in Figure 9.a depending on the drifts δ_m' and δ_u and the out-of-plane strength coefficient r_a, for the infill typology of interest.

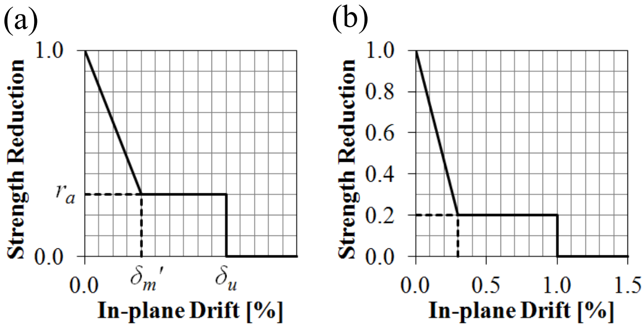

Figure 9. (a) Evaluation of out-of-plane strength reduction coefficient β_a; (b) Out-of-plane strength reduction coefficient β_a for slender unreinforced infill typology

Note that as a result, in order to carry out the verification of each infill panel, accounting for the possible strength reduction due to in-plane damage, essential properties assigned to the considered type of infill have to be known, in particular, the drift limits δ_m' and δ_u, corresponding to the attainment of damage limitation and ultimate limit state conditions, respectively, as well as the remaining portion of out-of-plane strength at the attainment of the drift δ_m', expressed by the out-of-plane strength coefficient r_a.

In this work the parameters of interest have been summarised for three different types of infill, see Table 3 and Table 4; the presented procedure may, however, be extended to any type of infill, if the relevant properties are available. Additionally, in order to estimate the out-of-plane strength, the expected level of in-plane drift to be sustained by the infill at the ultimate limit state has to be evaluated. Clearly, the in-plane drift demand and the corresponding infill damage may vary along the height of the building, and hence, the evaluation of an approximate in-plane drift profile of the infilled frame configuration corresponding to the seismic demand at the ultimate limit state is required.

Given the fact that the design of masonry infilled RC structures is commonly carried out on bare frame structural configurations, see *e.g.* (Morandi *et al.*, 2011b) and (Hak *et al.*, 2012a), the assessment of the related drift demands for the infilled structure may not be a straightforward task. In everyday design practise, the requirement to carry out detailed analyses on the infilled configuration may be rather demanding due to a series of complex issues, such as the nonlinear behaviour of the masonry and uncertainties related to the relevant material properties. However, if for the evaluation of the out-of-plane infill strength reduction the design drift of the bare frame configuration is assumed, the given procedure may be overly conservative, since reduced drift demands are expected for the corresponding infilled frame. Hence, future improvements may be achieved through the introduction of simplified procedures for the evaluation of the expected drift demands of the infilled frame based on the response of the corresponding bare configuration, assuming the material properties of the masonry infills to be adopted, as well as the related infill distribution in plan and along the height of the building.

5.2 Design Example

The proposed approach for the out-of-plane verification of masonry infills has been applied to a simple 6-storey frame configuration, regular in plan and elevation, shown in Figure 10. The structure has been designed following Eurocode 8 design provisions of high ductility class for a design PGA equal to 0.35*g* on ground type B.

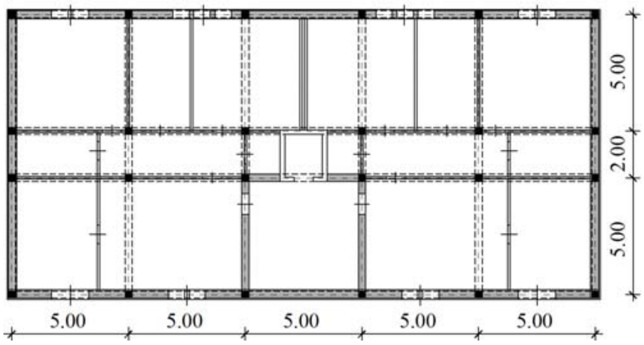

Figure 10. Structural layout of 6-storey RC frame example

Two different types of infill are assumed in the building layout, being in contact with surrounding structural elements, *i.e.*, a stronger (external) and a weaker (internal) type of infill. Herein, the non-structural verifications are presented for the weaker infill typology foreseen in the given configuration, corresponding to a t_w = 10.0 *cm* thick unreinforced type of infill, including a 1.0 *cm* thick plaster on each side, resulting in a slenderness ratio of $h_w/t_w \approx 26$, with a vertical compressive strength of $f_d = 2.00$ *MPa*. The inter-storey drift parameters have been assumed according to Table 3 ($\delta_m{}' = 0.30\%$ and $\delta_u = 1.0\%$) and the out-of-plane strength coefficient as given in Table 4 ($r_a = 0.20$), resulting in the reduction coefficient β_a shown in Figure 9.b.

The expected in-plane inter-storey drift demands $\delta_{w,j}$ for each storey (denoted by j) of the infilled frame configuration at the ultimate limit state design seismic action have been estimated for both, the longitudinal and the transversal direction, as summarised in Table 5, along with the corresponding values of the strength reduction coefficient $\beta_{a,j}$, evaluated from Figure 9.b, *i.e.* Equation (14).

Table 5. Estimated in-plane inter-storey drift demands $\delta_{w,j}$ of infilled structure and corresponding out-of-plane strength reduction coefficients $\beta_{a,j}$ for each storey

Direction	Longitudinal		Transversal	
Storey j	$\delta_{w,j}$	$\beta_{a,j}$	$\delta_{w,j}$	$\beta_{a,j}$
1	0.84%	0.20	0.98%	0.20
2	0.84%	0.20	0.98%	0.20
3	0.80%	0.20	0.93%	0.20
4	0.75%	0.20	0.90%	0.20
5	0.54%	0.20	0.66%	0.20
6	0.24%	0.35	0.27%	0.27

The seismic out-of-plane action has been calculated according to the provisions given in Eurocode 8 – Part 1 (CEN, 2004), from Equation (1) and Equation (2), to obtain the distributed load per unit area w_a (kN/m^2), given in Equation (3), assuming a behaviour factor of the infills $q_a = 2.0$ in line with the code recommendations. The vibration period T_a of the masonry infill in the out-of-plane direction has been calculated according to Equation (4), while the initial fundamental period of the structure configuration T_1 has been estimated from Equation (5), assuming conservatively $C_t = 0.050$. The out-of-plane resistance $w_{R,\chi,\beta}$ of the damaged infills for each storey has been found from Equation (15) and compared to the seismic demand evaluated according to Equation (3); the safety verification according to Equation (16) is summarised in Table 6. The indicated directions (longitudinal and transversal) are related to the previous in-plane demands and hence, the given values of out-of-plane strength refer to the corresponding orthogonal actions. For comparison, the strength of the undamaged infill panels w_R evaluated from Equation (10) is also presented.

Table 6. Verification of out-of-plane resistance

	Storey j	h_w [m]	z_w/H	T_a/T_1	w_a [kN/m^2]	$w_{R,\chi,\beta}$ [kN/m^2]	w_R [kN/m^2]
Longitudinal	1	2.60	0.07	0.204	0.17	0.43	2.13
	2	2.60	0.24	0.204	0.20	0.43	2.13
	3	2.60	0.40	0.204	0.24	0.43	2.13
	4	2.65	0.57	0.211	0.28	0.41	2.05
	5	2.65	0.74	0.211	0.31	0.41	2.05
	6	2.65	0.90	0.211	0.35	0.72	2.05
Transversal	1	2.60	0.07	0.204	0.17	0.43	2.13
	2	2.60	0.24	0.204	0.20	0.43	2.13
	3	2.60	0.40	0.204	0.24	0.43	2.13
	4	2.65	0.57	0.211	0.28	0.41	2.05
	5	2.65	0.74	0.211	0.31	0.41	2.05
	6	2.65	0.90	0.211	0.35	0.55	2.05

Note that the out-of-plane strength of the infills is significantly lower when expected previous in-plane damage is accounted for and the simple proposed design approach may, in critical cases, ensure that a safe-sided verification procedure is carried out, and the occurrence of unexpected out-of-plane expulsion is prevented.

6 CONCLUSIONS

Based on a series of field observations from recent earthquake events the occurrence of common out-of-plane failure mechanisms on both masonry infill walls and façade veneers has been identified, indicating the need to introduce improved procedures related to the out-of-plane resistance verification of non-structural masonry elements in the design of infilled RC structures.

In particular, current design recommendations according to European seismic codes have been discussed in this work and improvements in the safety verification procedure for slender clay masonry infills in RC frames have been introduced. Based on existing experimental results from a previous test campaign on different types of masonry infills, the reduction of out-of-plane strength in function of previous in-plane damage has been studied. Hence, following the proposed simplified approach, a reduction of the out-of-plane strength due to a certain extent of in-plane damage, estimated based on expected inter-storey drift demands at the design seismic action, can be accounted for with the aim to prevent the occurrence of unexpected out-of-plane expulsion of non-structural masonry elements. The application of the design approach has been demonstrated on the example of a simple case study building configuration, showing its practical simplicity and effectiveness.

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