LOCAL EFFECTS IN THE SEISMIC DESIGN OF RC FRAME STRUCTURES WITH MASONRY INFILLS

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Abstract. *For the prevention of local effects that may occur on the load-bearing elements of newly designed masonry infilled RC frames, additional measures are foreseen in modern seismic design codes. In particular, European code regulations impose special detailing and confinement requirements for structural elements in RC frames with adherent masonry infills. Moreover, the verification of adjacent columns for increased shear demands is required, estimated based on approximate values of horizontal shear strength of the infill, and through the application of capacity design principles along the contact length. With the aim to verify the efficiency of the current design approach and provide additional recommendations related to local effects due to the presence of masonry infills in RC frames, in this study a series of nonlinear dynamic analyses on typical newly designed building configurations with different masonry infill typologies of varying strength and stiffness properties has been performed at increasing levels of seismicity. The analyses are carried out using a simple numerical model representing the masonry infill as a single diagonal strut. Although such model cannot directly capture the local effects in the contact region, it allows, a posteriori, the calculation of effective shear demands on the columns. For a number of considered prototype buildings, a comparison of effective shear demands obtained from analyses and shear design forces required by the code has been carried out. Furthermore, a simplified procedure for the verification of columns in RC frames exposed to increased shear demands due to the presence of masonry infills as a function of expected inter-storey drift demands has been proposed. Hence, starting from existing code recommendations, the given method allows the evaluation of shear design forces in compliance with effective shear demands, accounting for different masonry infill typologies, the position of the infill along the height of the building and the seismic ground motion intensity.*

1   INTRODUCTION

The possible occurrence of local effects on structural elements of RC frames with masonry infills is widely recognized and has repeatedly been supported by field evidence from recent earthquake events in Italy. After the April 2009 earthquake in Abruzzo (L'Aquila), extensive damage has been reported on non-structural masonry infills, see *e.g.* [1]; moreover, in many cases the poor behaviour or even failure of structural elements in RC frames has been identified [2], often caused by adverse effects due to the presence of masonry infills. Also following the most recent seismic event that has stroked the region of Emilia in May 2012, examples of local effects on the structural elements of infilled frames have been reported [3, 4, 5], as illustrated in Figure 1.

Figure 1: Examples of local effects reported after the May 2012 Emilia earthquake
a) Magenes *et al.* 2012 [3]; b) Manzini and Morandi 2012 [4].

In particular, local damage and eventually brittle failure can commonly be caused on columns that are in partial contact with masonry infill walls, causing a reduction of the clear height of the column and, hence, inducing increased shear and displacement demands. As discussed in previous studies [6], the problem of laterally restrained captive columns, particularly vulnerable to seismic actions, should possibly be pursued through adequate measures in the conceptual design of new buildings. However, detrimental effects on columns of RC frame structures can be caused by the interaction with masonry infills also in the case of full contact along the height of the column, in particular when masonry infill typologies of high strength and stiffness properties are used in construction, and/or when the infill is located only on one side of the column. Specifically, additional concentrated shear demands may be imposed on the column at its ends in the region of contact with the masonry infill due to the activation of compressive diagonal strut forces. For the prevention of such local effects in newly designed structures, with infills adherent to the RC frame, additional measures are foreseen in modern seismic design codes, such as Eurocode 8 – Part 1 [7].

The application of the recommended simplified approaches to practical design cases reveals, however, the fact that increased shear design forces are imposed independently of the design conditions and of the position of the structural element in the building, even though the corresponding column shear may vary significantly. In this study the current European design provisions for RC frames have been applied to a number of prototype building configurations, including the additional measures for infilled structures by means of increased shear design forces, capacity design principles and detailing rules, in order to detect the governing design situations for different peak ground accelerations and design ductility classes. Subsequently, a

number of nonlinear time-history analyses of the RC frames with masonry infills of increasing strength and stiffness properties has been carried out, with the aim of determining effective shear demands imposed on the columns and to propose improved shear design verifications as a function of in-plane drift demands imposed on the structure.

2 DESIGN FOR THE PREVENTION OF ADVERSE LOCAL EFFECTS

2.1 Current European Design Provisions

According to current European provisions for the design of structures for earthquake resistance Eurocode 8 – Part 1 [7], besides irregularities in plan and elevation due to the presence of infills that may adversely influence the global behaviour of RC frames, possible local effects due to the frame-infill interaction need to be taken into account. Therefore, special detailing and confinement requirements for structural elements in RC frames with adherent masonry infills, as well as a verification of the adjacent columns for approximate increased shear demands are imposed. Specifically, due to the particular vulnerability of infills in the ground floor, where seismically induced irregularity may be expected, if a more precise method is not used, the entire length of all columns should be considered as critical region (Figure 2a) and consequently, the relevant confinement and detailing rules have to be satisfied. The same requirement applies in the case when infills extend over the entire column height l_t, but only on one side of the column (Figure 2b), as well as for columns which are in contact with adjacent infills along just a part of their total height l_t (Figure 2c), inducing the presence of a clear portion of the column height $l_{cl} < l_t$. In this latter case, the consequences of the decreased shear span of the column due to the horizontal restraint by the infills (Figure 3) should be covered in addition. Therefore, the shear demand has to be determined from the provided column resistance $M_{C,Rd}$ following the capacity design principle applied over the clear height l_{cl} (Equation 1), adopting an overstrength coefficient $\gamma_{Rd} = 1.1$ for ductility class medium (DCM) and $\gamma_{Rd} = 1.3$ for ductility class high (DCH).
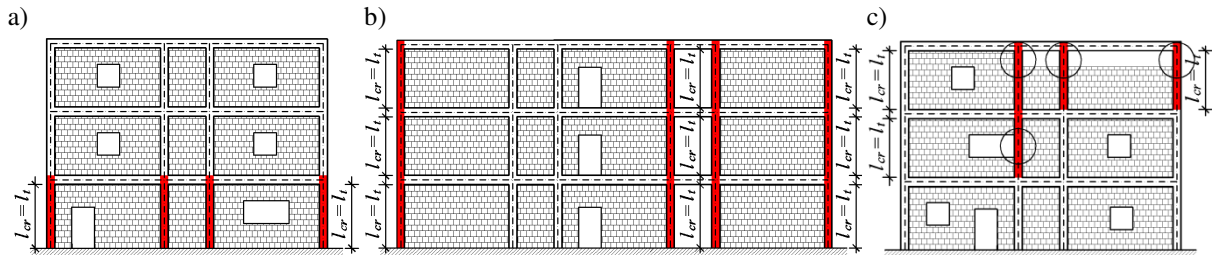

Figure 2: a) Critical length of columns in the ground floor;
b) Critical length of columns in full contact with the infill on one side;
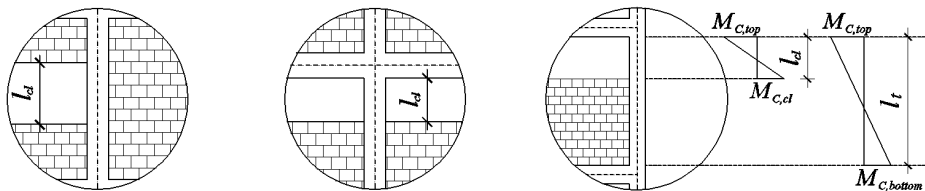c) Critical length of columns in partial contact with the infill.

Figure 3: Decreased column shear span ratio due to partial contact with the infill

$$V_{Ed,cl} = \gamma_{Rd} \frac{2M_{C,Rd}}{l_{cl}} \tag{1}$$

Further safety verifications present in the European code regulations rely upon the common simplification of the masonry infill action based on an equivalent strut model, representing the diagonal portion of the infill by a compressive strut. The requirement that is of particular interest for this study indicates that the contact length l_c of the column, over which the diagonal strut force of the infill acts on the column causing a local pressure f_s and introducing a concentration of forces (Figure 4a), should be verified in shear for the smaller of the following two shear forces:

a) the horizontal component $F_{w,s,hor}$ of the diagonal strut force $F_{w,s}$ of the infill, that can be assumed to be equal to the shear strength of the panel;
b) the shear force $V_{C,Ed}$ determined following the capacity design principle (Equation 2), assuming that the flexural column capacity develops at the ends of the contact length l_c (Figure 4b).

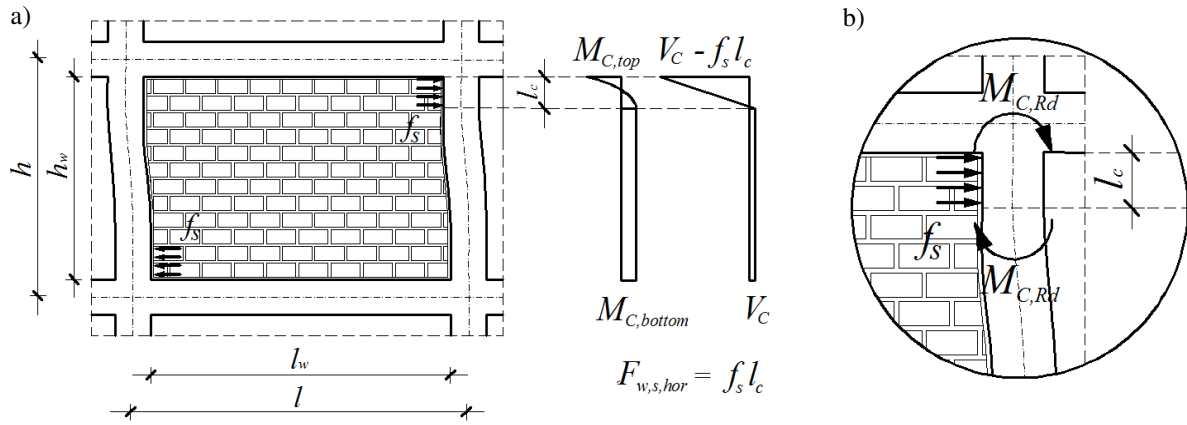$$V_{C,Ed} = \gamma_{Rd} \frac{2M_{C,Rd}}{l_c} \tag{2}$$

Figure 4: a) Application of the diagonal strut action on the column within the contact length; b) Flexural column capacity developed at the ends of the contact length

2.2 Interpretation of Design Requirements

According to the code, the horizontal component $F_{w,s,hor}$ of the diagonal strut force $F_{w,s}$ can be assumed to be equal to the horizontal shear strength of the panel, estimated on the basis of the shear strength of bed joints. Since no explicit recommendations related to the evaluation of the shear strength of the bed joints are specified in the code, it could be reasonable to use the expression included in the Eurocode 6 – Part 1 [8], commonly assumed for load-bearing masonry walls, considering the shear strength of the bed joints equal to the initial shear strength under zero compressive stress f_{v0}. Hence, for a masonry infill of thickness t_w, length l_w and shear strength $f_v = f_{v0}$ the corresponding horizontal strength $F_{w,s,hor,1}$ may be estimated as given in Equation 3.

$$F_{w,s,hor,1} = f_{v0} t_w l_w \tag{3}$$

A number of more refined models for the evaluation of the masonry infill strength and corresponding equivalent diagonal strut forces is available. Some of these models are able to account for different failure mechanisms typically observed on masonry panels and usually supported by experimental studies, *e.g.* as proposed by Decanini *et al.* [9] for compression at

the centre, compression at the corners, shear sliding and diagonal tension. The horizontal component of the corresponding equivalent strut strength $F_{w,s,hor,2}$ can be expressed by Equation 4, where $f_m{'}$ indicates the strength associated with the critical failure mode, t_w the thickness of the infill, b_w the width and θ the inclination of the equivalent strut.

$$F_{w,s,hor,2} = f_m{'}t_w b_w \cos\theta \qquad (4)$$

The evaluation of local shear design forces can eventually be influenced by the choice of the contact length l_c, over which the diagonal strut force of the infill acts on the column, since applying the capacity design principle given in Equation 2, for larger values of contact length lower shear demands $V_{C,Ed}$ are obtained and may be governing with respect to the horizontal strength of the panel $F_{w,s,hor}$. As specified in the code, unless a more accurate estimation is made taking into account the elastic properties and the geometry of the infill and the column, the strut width may be assumed to be a fixed fraction of the length of the panel diagonal. Paulay and Priestley have suggested that a conservatively high value of the diagonal strut width b_w equal to one quarter of the panel diagonal d_w may be assumed [10]. However, for the definition of the equivalent strut width a series of more refined approaches are available in literature. For instance, according to the model proposed by Decanini *et al.* [9], the equivalent strut width is evaluated as a function of a relative stiffness parameter to account for the interaction between the frame and the infill. A similar approach is adopted in Mainstone's model [11], suggested also in other studies, and recommended *e.g.* in the work by Fardis for the application of design provisions according to Eurocode 8 [6]. Even though it is stated in the code that the contact length should be assumed to be equal to the full vertical width of the diagonal strut of the infill b_w, this requirement should be interpreted depending on the strut model that is being adopted and the corresponding failure modes considered. Herein, the contact length has been evaluated following the model by Paulay and Priestley [10] and Decanini *et al.* [9], assuming the contact length equal to $0.5b_w/\cos\theta$ (see Figure 5a) and according to the model by Mainstone [11], assuming the contact length equal to $b_w/\cos\theta$ (see Figure 5b).
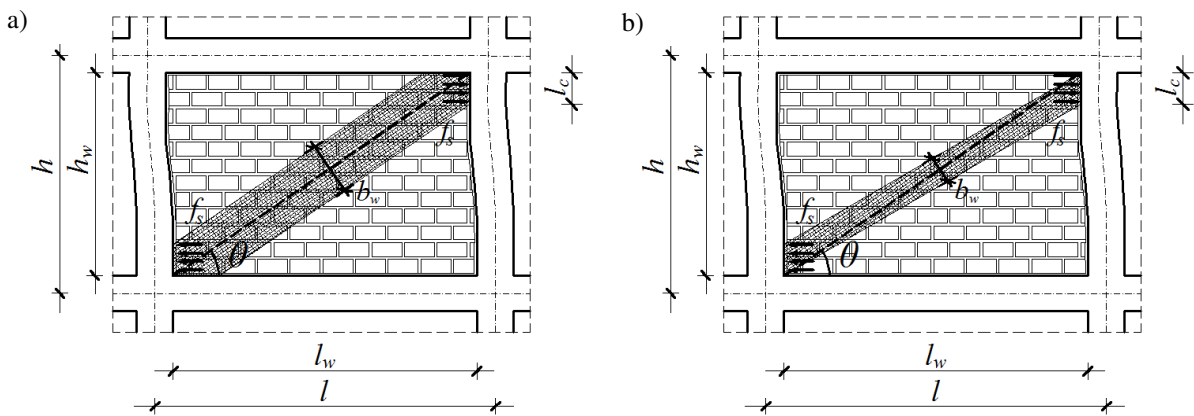

Figure 5: Evaluation of the contact length: a) $l_c = 0.5b_w/\cos\theta$; b) $l_c = b_w/\cos\theta$

3 NUMERICAL STUDY

3.1 Design and Modelling Assumptions

For the needs of this study, as part of a wider analytical campaign, a number of numerical analyses has been carried out on different typologies of RC frames, newly designed according to current European (Eurocode 8 – Part 1 [7]) and Italian national code provisions (NTC08

[12]). Specifically, 3-storey, 6-storey and 9-storey frames have been considered, designed for two different ductility classes, namely medium (DCM) and high (DCH), and five different design peak ground accelerations, *i.e.* 0.05*g*, 0.10*g*, 0.15*g*, 0.25*g* and 0.35*g*, resulting in 30 different bare frame configurations. A more detailed description of the frame design procedure is given in the work by Morandi *et al.* [13].

In summary, as recommended in the code provisions for the design of regular structures, the design has been carried out for bare frame configurations, based on linear analyses of elastic models, satisfying capacity design principles, strength and ductility requirements as well as displacement limitations. Subsequently, time-history analyses have been carried out on nonlinear models of the infilled frame typologies, using the structural analysis program Ruaumoko [14], adopting the concentrated plasticity modelling approach for structural elements and representing the masonry infills by a simple single-strut model. Even though in such simplified model the local effects due to the presence of masonry infills cannot be captured directly, induced effective column shear demands have been evaluated a posteriori from the diagonal strut force. In order to evaluate the strength and stiffness properties of the equivalent strut, the model proposed by Decanini *et al.* [9] has been adopted, while the hysteretic rule introduced by Crisafulli [15] has been calibrated on experimental test results and used to model the nonlinear cyclic behaviour of the infill. The local effects have been studied on the external columns of fully infilled frame typologies for three different types of masonry infills with increasing strength and stiffness properties, representing weak (T1), medium (T2) and strong infill (T3). Further details on the modelling assumptions and the considered masonry infill typologies are given in the work by Hak *et al.* [16].

3.2 Column Shear Design Forces

The shear design procedure complying with the provisions given in Eurocode 8 [7] has been applied to the columns of the considered case study frames, showing that in some cases the detailing rules, imposed to attain satisfactory local ductility in critical regions of primary seismic elements, may require transversal reinforcement amounts resulting in values of shear resistance high enough to satisfy also the recommended shear demands imposed due to local effects, evaluated from Equation 2 or Equation 3, whichever is governing. In order to represent a typical set of results, the shear demands evaluated for the external columns at the first and at the last storey of the 6-storey case study frames are summarised in Table 1. In particular, the shear demands indicated as *w/o* local effects have been evaluated for the case when only action effects obtained from linear analyses on an elastic model of the bare frame and global capacity design principles are considered, while local effects due to the presence of infills are neglected. The shear demands obtained for the case when local effects are taken into consideration following the code recommendation, determined as the minimum of the values obtained from Equation 2 and Equation 3, is indicated as *w/* local effects. The provided column shear resistance has been calculated based on shear design for obtained action effects, global capacity design verifications and local ductility requirements in the critical region of the column, as commonly performed for bare frames. The shear demand imposed due to local capacity design requirements (Equation 2) has been verified assuming the simplified approach for the evaluation of the contact length based on the strut width recommended by Paulay and Priestley [10]. The strut force of the external masonry panel has been evaluated from Equation 3, based on the values of initial shear strength f_{v0} and thickness t_w, as summarised in Table 2 for the considered masonry typologies. The length of the masonry panel l_w varies depending on the dimensions of the adjacent column and can be evaluated subtracting the column width h_c from the centreline span of the external bay of the prototype frame, *i.e.*, $l = 5.0\ m$.

	6-storey		Shear demand				Column shear resistance			Shear demand				Column shear resistance
				Local effects							Local effects			
		h_c	w/o		w/				h_c	w/o		w/		
				T1	T2	T3					T1	T2	T3	
PGA	DCM	[m]	[kN]	[kN]	[kN]	[kN]	[kN]	DCM	[m]	[kN]	[kN]	[kN]	[kN]	[kN]
0.05g		0.45	202.8	200.2	295.8	409.5	422.6		0.35	96.9	204.6	302.3	331.1	230.0
0.10g		0.45	202.8	200.2	295.8	409.5	422.6		0.35	96.9	204.6	302.3	331.1	230.0
0.15g	1st st.	0.45	202.8	200.2	295.8	409.5	422.6	6th st.	0.35	96.9	204.6	302.3	331.1	230.0
0.25g		0.45	202.8	200.2	295.8	409.5	422.6		0.35	96.9	204.6	302.3	331.1	230.0
0.35g		0.50	300.0	198.0	292.5	405.0	535.0		0.40	116.9	202.4	299.0	397.5	323.6
	DCH							DCH						
0.05g		0.45	249.1	200.2	295.8	409.5	422.6		0.35	123.7	204.6	302.3	406.8	237.7
0.10g		0.45	249.1	200.2	295.8	409.5	422.6		0.35	123.7	204.6	302.3	406.8	237.7
0.15g	1st st.	0.45	249.1	200.2	295.8	409.5	422.6	6th st.	0.35	123.7	204.6	302.3	406.8	237.7
0.25g		0.45	249.1	200.2	295.8	409.5	422.6		0.35	123.7	204.6	302.3	406.8	237.7
0.35g		0.45	249.1	200.2	295.8	409.5	422.6		0.35	123.7	204.6	302.3	406.8	237.7

Table 1: Column shear demands (*w/o* and *w/* local effects) and shear resistance of 6-storey prototype frames

	T1	T2	T3
f_{v0} [MPa]	0.44	0.25	0.30
t_w [mm]	0.10	0.26	0.30

Table 2: Masonry infill initial shear strength and thickness

In the case of external columns, the EC8 recommendations state that the critical region should be taken equal to the full height of the column since the infill is present only on one side. It can be noticed that in some cases a high shear resistance may be achieved due to the given detailing requirements in order to obtain the needed ductility in critical regions, or due to the choice of large cross sections required by other design issues, such as the limitation of in-plane displacement demands. Hence, the additional shear demand due to the presence of infills may not always govern the shear design, above all at the bottom storeys of taller buildings and/or in the case of high seismicity. At the same time, however, it can be observed that in the upper storeys, in particular in buildings of lower height and/or in the case of lower or intermediate seismicity, shear demands imposed due to the presence of infills may exceed significantly the values obtained from other design criteria. Following the current design procedure, shear demands induced by local effects due to the presence of infills are determined independently of different design conditions and the position of the infill along the height of the building, although the assumption of equal maximum strut forces for all verifications may not always be rational.

3.3 Nonlinear Analyses Results

In order to determine the variation of possible shear effects, imposed under seismic actions, along the height of a building as a function of the considered design parameters, *i.e.,* design peak ground acceleration, ductility class and building height, nonlinear time-history analyses of the infilled frame models have been performed at different levels of seismicity, corresponding to the considered levels of design peak ground acceleration. The analyses have been carried out for a set of ten scaled natural earthquake records selected matching the average earthquake response spectrum with the design spectrum [13]. Hence, maximum response quantities have been determined for each record and subsequently a corresponding average value has been evaluated. In particular, the maximum horizontal strut actions achieved in the external masonry panels and the maximum effective shear forces on the external columns of each storey of the fully infilled case study frames have been evaluated.

Activated Strut Forces. The average maximum horizontal component of the activated strut force $F_{w,s,hor,act}$, see Equation 5 where i indicates the i-th acceleration record, obtained from analyses for each storey of each prototype frame, for the three considered infill typologies, has been normalised, as given by Equation 6, to the corresponding horizontal resistance of the strut $F_{w,s,hor,2}$, determined from the governing failure mode according to the model by Decanini *et al.* [9] (Equation 4). The percentage of strut force activation a_w and its variation along the building height, for infills in the 6-storey frame typologies, designed for ductility class medium, is shown in Figure 6. The position of the infill along the height of the building is represented by the ratio h_n of the distance from the foundation of the building to the centre of mass of the infill h_w and the total height of the building H as given by Equation 7.

$$F_{w,s,hor,act} = \frac{1}{10} \sum_{i=1}^{10} F_{w,s,hor,\max,i} \tag{5}$$

$$a_w = \frac{F_{w,s,hor,act}}{F_{w,s,hor,2}} \cdot 100 \tag{6}$$

$$h_n = \frac{h_w}{H} \tag{7}$$

A significant reduction of the strut force amplitude can be observed in the upper quarter of the building height. Moreover, the results show clearly the considerable increase of activated strut forces with increasing ground motion intensity. Additionally, a reduction of the strut force activation can be noticed for increasing strength and stiffness properties of the considered masonry infills.
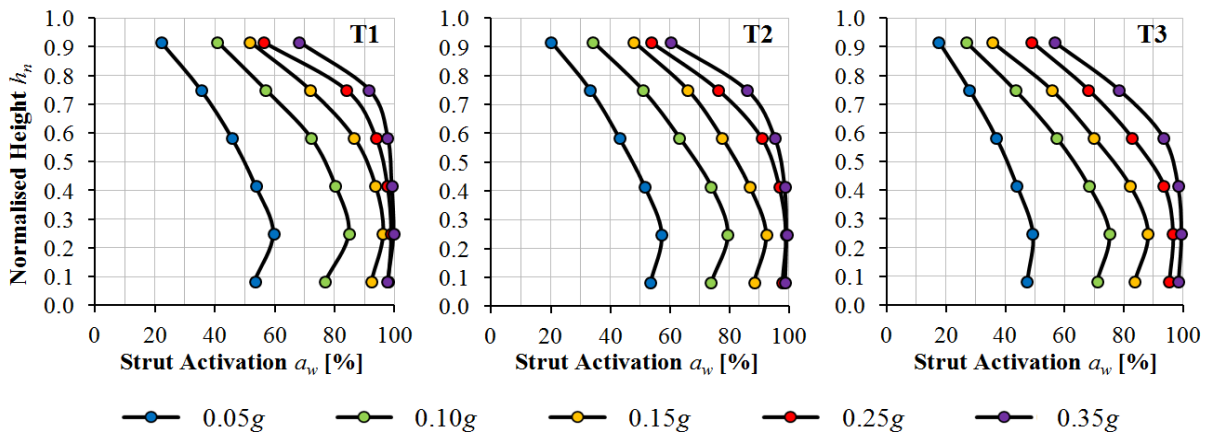

Figure 6: Strut force activation in 6-storey DCM frames

Effective Column Shear Demands. The effective shear demand $V_{w,C}$ imposed on a column due to the strut action (see Figure 4a) can be approximately evaluated a posteriori as the difference between the horizontal component of the activated strut force $F_{w,s,hor,act}$ and the corresponding shear in the column V_C, as given in Equation 8. For the considered prototype frames, the average maximum effective shear demand $V_{w,C,ref}$ imposed on each column has been evaluated from Equation 9.

$$V_{w,C} = \left| F_{w,s,hor,act} - V_C \right| \tag{8}$$

$$V_{w,C,ref} = \frac{1}{10}\sum_{i=1}^{10} V_{w,C,\max,i} \tag{9}$$

The obtained results indicate that the effective column shear demands are regularly slightly lower than the activated strut forces; hence the verification of the column for the activated strut force rather than the effective column shear demand can be considered safe-sided, but not too conservative. In fact, also the code provisions rationally require the verification of the column for the strut force action neglecting the eventual presence of shear forces in the columns, considering the strut, however, always fully activated.

Comparison: Design Shear Forces, Activated Strut Forces, Effective Shear Demands. Summarising the presented results, a comparison of the column design shear forces obtained following the code recommendations, and the shear forces obtained from nonlinear time-history analyses, for two selected case studies, namely the 6-storey fame designed for peak ground accelerations of 0.10g and 0.25g in ductility class medium, are shown in Figure 7. In particular, as summarised in Table 3, the graphs include shear design forces obtained from locally applied capacity design principles using three different models for the evaluation of the contact length, horizontal strut forces calculated following two different approaches, activated strut forces and effective shear demands from nonlinear analyses and design shear forces that would be imposed due to action effects evaluated from linear analyses and from capacity design principles, without considering any local effects.

		Column shear demand	Description
1	$V_{C,Ed}$	Capacity design shear force	Equation 2
		Contact length	l_c - Mainstone [11], Figure 5b
2	$V_{C,Ed}$	Capacity design shear force	Equation 2
		Contact length	l_c - Paulay and Priestley [10], Figure 5a
3	$V_{C,Ed}$	Capacity design shear force	Equation 2
		Contact length	l_c - Decanini *et al.* [9], Figure 5a
4	$F_{w,s,hor,1}$	Horizontal strut resistance (simplified)	Equation 3, Initial shear strength criterion
5	$F_{w,s,hor,2}$	Horizontal strut resistance (refined)	Equation 4, Decanini *et al.* [9]
6	$F_{w,s,hor,act}$	Activated horizontal strut force	Equation 5, from nonlinear analyses
7	$V_{w,C,ref}$	Effective shear demand	Equation 9, from nonlinear analyses
8	$V_{C,Ed,0}$	Design shear demand *w/o* local effects	From linear analysis and global capacity design

Table 3: Design and effective column shear demands

The design shear forces obtained applying the capacity design along the contact length evaluated following the simplified model proposed by Paulay and Priestley [10] vary only due to the difference in moment resistance of adjacent column elements, while the other two models considered for the evaluation of the strut width result in increasing values of shear demands for increasing masonry strength and stiffness. Due to the significant influence of the relative stiffness of frame and infill on the contact length evaluated following the model proposed by Decanini *et al.* [9], the corresponding shear forces are considerably lower compared to the other models, especially where the relative stiffness of the masonry is low compared to that of the frame, as in the case for weak masonry infills and/or in lower storeys, due to larger column dimensions. However, independently of the strut model, the capacity design shear force from Equation 2 in comparison with the simplified horizontal resistance of the strut

from Equation 3, results to be governing only for the strong infill (typology T3) in some of the upper storeys, where columns have smaller dimensions and a lower bending capacity.
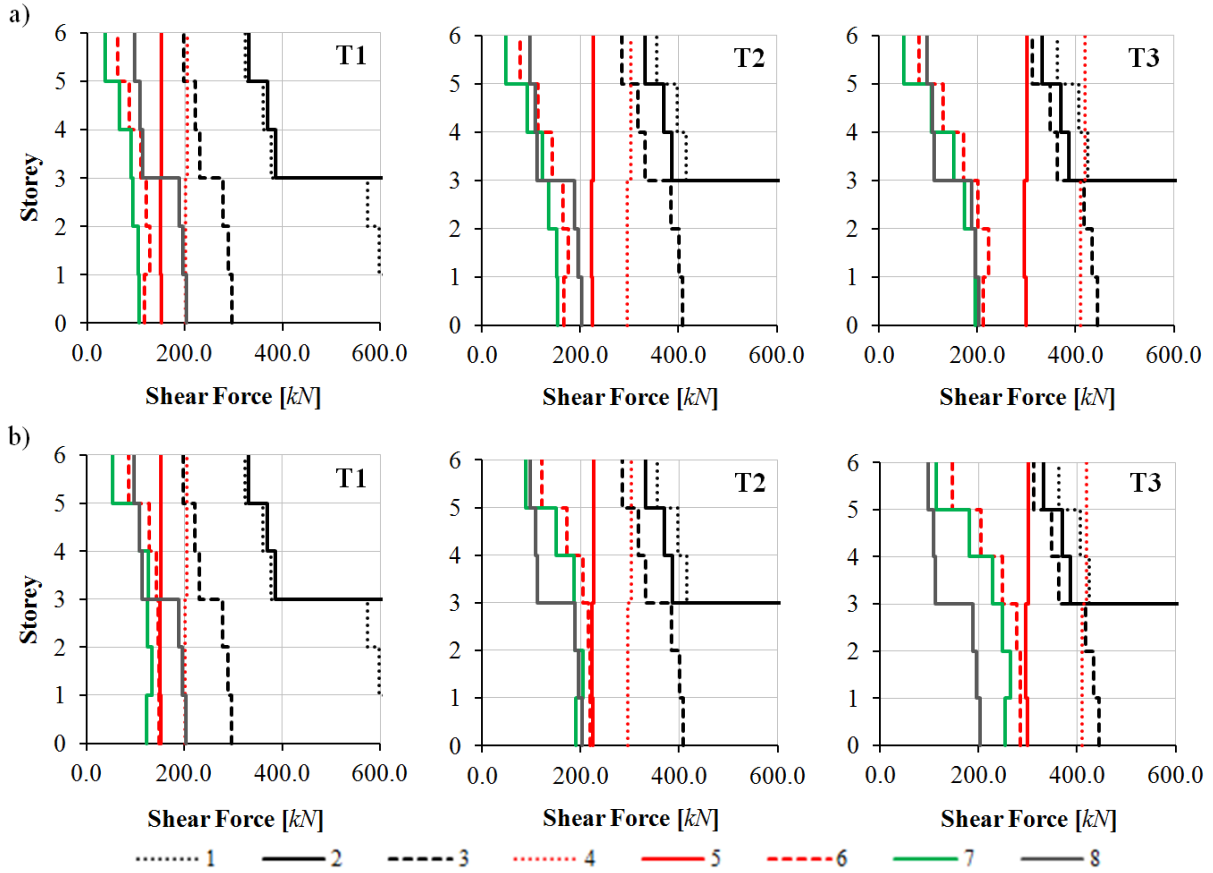

Figure 7: Design shear forces, activated strut forces and effective shear demands for 6-storey frames
a) 0.10g DCM; b) 0.25g DCM

Considering a refined evaluation of the strut resistance, including the possible occurrence of different failure modes, as for example based on the model by Decanini *et al.* [9] (Equation 4), the local shear imposed due to the strut action can be identified as the governing value for all considered case studies. Besides that, such model results in 30-40 % lower values of masonry infill strength compared to the values obtained from the initial strength criterion (Equation 3), depending on the typology. Therefore, it can also be noticed that for all considered infill strength and stiffness properties, the resistance based on the initial shear strength criterion exceeds significantly the effective shear demands on the columns evaluated from time-history analyses based on the more refined infill model. This fact is particularly pronounced if the maximum strut force has been only partially activated, such as in the case of lower earthquake actions, stronger infills and in upper storeys.

4 CONCLUSIONS AND IMPLICATIONS FOR DESIGN

4.1 Shear Demands in Function of Inter-storey Drifts

Having in mind the considerable variation of shear demands imposed due to the presence of masonry infills, the application of a refined procedure for the evaluation of design forces accounting for local effects is envisaged. It can be shown that the activation of the strut force a_w in each storey of the infilled frame, as presented in Figure 6 *e.g.* for varying peak ground

accelerations and infill typologies in the case of DCM 6-storey building configurations, can be expressed as a function of a single parameter, namely the average inter-storey drift demand δ_w imposed on the infilled frame. Accordingly, the complete set of results for all considered prototype frames showing the strut activation obtained from nonlinear time-history analyses can be summarised as given in Figure 8a. The obtained relation can be represented approximately, for example, by polynomial and linear or linear segments, as shown in Figure 8b, and expressed by Equation 10 and Equation 11, respectively. Such simplified expressions may be used in the design of infilled RC frames in order to determine the expected activation of the strut action.
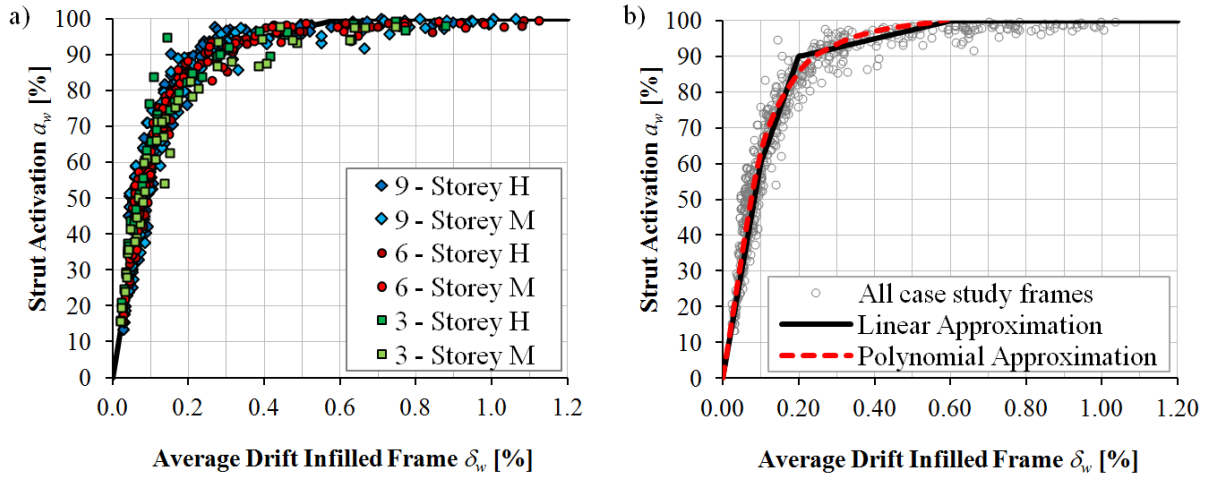

Figure 8: Strut force activation vs. average drift of the infilled frame: a) TH results; b) Possible approximations

$$a_w = \begin{cases} 62.5\delta_w^5 - 126.0\delta_w^4 + 102.5\delta_w^3 - 43.2\delta_w^2 + 9.7\delta_w, & \delta_w < 0.006 \\ 1.0, & \delta_w > 0.006 \end{cases} \tag{10}$$

$$a_w = \begin{cases} 600\,\delta_w, & \delta_w \le 0.001 \\ 300\,\delta_w + 0.30, & 0.001 < \delta_w \le 0.002 \\ 25\,\delta_w + 0.85, & 0.002 < \delta_w \le 0.006 \\ 1.0, & \delta_w > 0.006 \end{cases} \tag{11}$$

4.2 Refined Shear Design for Local Effects

Based on the presented results, a possible refinement of the current design procedure for the evaluation of local effects due to the presence of masonry infills in RC frames may be introduced, assuming that the inter-storey drifts of the infilled frame δ_w can be predicted. Hence, the corresponding expected strut force activation a_w can be evaluated from a simplified expression, such as Equation 10 or Equation 11. Given that the design of regular infilled frame structures is commonly carried out on bare frames, the evaluation of expected drifts of the infilled frame is not a straightforward task and clearly depends, besides on the design properties of the bare frame, such as the building configuration, design seismic action and ductility class, also on the typology and distribution of the masonry infill. Assuming conservatively that the maximum shear demand on the column equals the activated horizontal component of the strut force rather than the effective column shear force (see Equation 8 and Equation 9), the shear demand can be estimated as given in Equation 12.

$$V_{C,Ed} = a_w F_{w,hor} \tag{12}$$

The maximum horizontal strut force $F_{w,hor}$ should possibly be evaluated from a model accounting for different infill failure modes (Equation 4) or may be estimated following other approaches (*e.g.* such given by Equation 3). Finally, the capacity design verification has to be accomplished along the contact length in order to determine the governing shear demand. Even though different values of the contact length l_c can be obtained from different strut models, the design is not significantly influenced by this choice and the application of a simple model, such as assuming the diagonal strut width b_w equal to one quarter of the panel diagonal d_w, as suggested by Paulay and Priestley [10], can be considered satisfactory.

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