Gómez Martínez, F.; Perez Garcia, A.; Verderame, G.; De Luca, F. (2013). FAST simplified approach for seismic assessment of infilled RC MRF buildings: application to the 2011 Lorca earthquake. 9th International Conference on Earthquake Resistant Engineering Structures (ERES 2013).

FAST simplified approach for seismic assessment of infilled RC MRF buildings: application to the 2011 Lorca earthquake

F. Gómez-Martínez, A. Pérez-García
Department of Mechanics of the Continuum Media and Structural Theory, Polytechnic University of Valencia, Spain

F. De Luca, G. M. Verderame
Department of Structures for Engineering and Architecture, DIST, University of Naples Federico II, Italy

Abstract

A generalized formulation of the FAST vulnerability method for Reinforced Concrete Moment Resisting Frame (RC-MRF) buildings with irregular distribution of infills in elevation is presented. This method, which belongs to the wider family of spectral-based methodologies for the simplified assessment of infilled RC MRF buildings, has been already proposed for uniformly infilled frames and applied in the case of post-earthquake damage survey results for benchmarking purposes. The generalized approach allows to consider a reduction of the amount of infills at the ground floor. Thus, this new form of FAST is capable of computing all intermediate situations between the uniformly infilled and pilotis (no infills at ground storey) cases. Finally, the approach provided is applied to the case of Lorca (Spain) 2011 earthquake damage survey data.

Keywords: Infilled RC-MRF, pilotis, period, damage states, Lorca earthquake, FAST.

1 Introduction

In previous works [1,2], a simplified approach (FAST) for the estimation of large scale vulnerability of uniformly infilled Reinforced Concrete Moment Resisting Frames (RC-MRFs) has been proposed and tested by using real post-earthquake damage scenarios as a benchmark. The aim of this paper is to provide a generalized form of the method for non-uniformly infilled frames, in particular infilled frames with a certain reduction of the amount of infills at the ground floor, which is a common practice in residential building. In section 2, basic principles of the method are described and the extension of the method is presented in its details. Finally, in section 3 the new approach is applied to the case of Lorca (Spain) 2011 earthquake, and comparisons are provided with respect to post-earthquake damage data collected in-field.

2 FAST vulnerability approach

FAST approach provides information about the vulnerability in terms of damage state (DS) of infilled RC-MRF, allowing a preliminary comparison with observed damage. Necessary data for its implementation are: (i) number of storeys, (ii) age of construction, and (iii) localization of the building or set of buildings considered; they are all necessary for the definition of the capacity curve of the building or class of buildings, while information (iii) is also necessary for demand characterization (e.g., PGA demand).

2.1 Basic concepts

FAST is based on the definition of an approximate capacity curve (CC) and its correspondent IN2 curve [3] in terms of PGA for the equivalent Single Degree of Freedom (SDOF) representing the building, or the class of buildings, considered. Then, thanks to mechanical interpretation of non-structural damage states (DS) in terms of interstorey drift (IDR), the thresholds of the DS in terms of top displacement of the SDOF are carried out. The IN2 curve allows the switch from SDOF displacement thresholds to PGA thresholds for each DS. Thus, the approach allows determining the PGA of the exceedance of each single DS.

2.1.1 Capacity curve

The simplified quadrilinear CC [3] (Figure 1) asks for the definition of:

- $C_{s,max}$ and $C_{s,min}$: maximum and residual (minimum) inelastic spectral acceleration capacity, both obtained (eqn (1)) as weighted addition of the respective capacities of bare frame $C_{s,RC}$ and infills $C_{s,w}$ (eqn (2)).
- μ_s: available ductility up to the beginning of the degradation of the infills, assumed to be 2.5 [4].
- $T_{eff,inf}$: equivalent elastic period of the infilled frame (eqn (3)), obtained from the elastic one through factor κ, assumed to be 1.4 [5].

The rest of the necessary variables can be divided in four groups:

1) Assessment parameters: α=0.5 and β=0.0; they weight the contribution of RC and infills, at the maximum and residual part of the curve respectively. R_ω(=1.45) and R_α(=1.00), the overstrength factors of material and structural redundancy, respectively [6]. Finally, m_r, the average storey superficial mass, assumed to be similar in every storey; and τ_{max}, maximum infills shear.

2) Code parameters: $S_a(T)$, spectral design acceleration; m_d, average design storey superficial mass assumed to be similar in every storey; λ_d, design ratio of first mode participating mass respect to the total one for the MDOF; and γ_d, design seismic combination factor.

3) Dynamic parameters: (m^*/M), ratio of first mode participating mass respect to the total one for the SDOF; Γ, first mode participating factor; and (K_g/A_b), global elastic stiffness of the structure, normalised by the area of the building.

4) Building parameters: n, number of storeys; $\rho_{w,1}$, ground floor ratio between the effective area of infills in each direction respect to the building area, A_b.

$$C_{s,\max} = \alpha \cdot C_{s,RC} + C_{s,w}; \qquad C_{s,\min} = C_{s,RC} + \beta \cdot C_{s,w}; \qquad C_{s,\max} \geq C_{s,\min} \tag{1}$$

$$C_{s,RC} = \frac{V_y}{\Gamma m^*} = S_a(T) \frac{m_d}{m_r} \frac{\lambda_d \gamma_d}{\Gamma\left(\frac{m^*}{M}\right)} R_\omega R_\alpha; \qquad C_{s,w} = \frac{V_{w,\max}}{\Gamma m^*} = \frac{\tau_{\max}}{m_r} \frac{\rho_{w,1}}{\Gamma\left(\frac{m^*}{M}\right)} \frac{1}{n} \tag{2}$$

$$T_{eff,\inf} = \kappa \cdot T_{el,\inf} = \kappa \cdot 2\pi \sqrt{\frac{m^*}{K^*}} = \kappa \cdot 2\pi \sqrt{\left(\frac{m^*}{M}\right)\frac{M}{K_g}} = \kappa \cdot 2\pi \sqrt{\left(\frac{m^*}{M}\right)\frac{m_r}{\left(K_g/A_b\right)}} \sqrt{n} \tag{3}$$

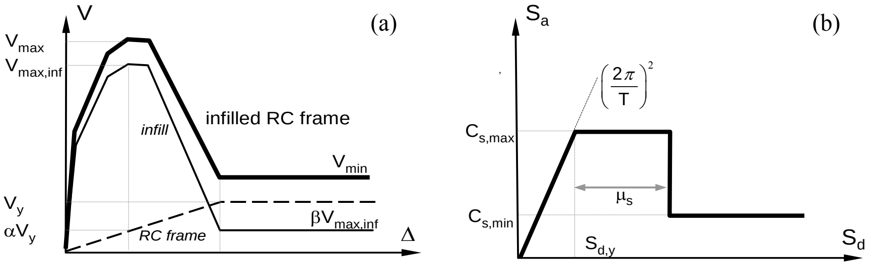

Figure 1: Infilled RC MRF pushover curve (a) and ADRS idealisation (b)

2.1.2 DS interpretation

The SDOF spectral displacement for each DS, $S_{d|DSj}$, is obtained considering a pure shear-type frame with no deformation of the horizontal elements (eqn (4)). In order to obtain an approximated resolution of the dynamic problem, based on Rayleigh method, the algorithm needs the definition of an *a priori* lateral load pattern of the seismic forces that results in elastic displacements proportional to them, considering equal masses in every floor. This load pattern increases in homothetic way to obtain the different DS. The top displacement, $d_{n|DSj}$, is calculated as the addition of the first floor displacement, that attains the DS, and the smaller contribution of the rest of the storeys –owing to their different elastic stiffnesses, shear force and stiffness degradation. Three reduction factors account for these differences (eqn (5)) being *i* the ordinal of the storey:

- χ represents the ratio between the elastic stiffness at ground floor with respect to the average one of the rest of the storeys, due to the typical decrease in the amount of infills at ground storey. Upper stiffnesses are equal between each other, as the method requires upper floors to have the same amount of infills.
- ζ represents the ratio between the average shear force in the upper floors and the base shear; it is independent of the DS_j, as the shear force pattern increases in homothetic way.
- γ represents the ratio between the secant stiffness post-cracking in the ground floor when DS2 is attained and the average equivalent secant stiffness in the upper floors; this progressive degradation of the elastic stiffness is represented by a factor Ω_i that varies between 1.00 in the storeys without cracking and an approximate value of Ω_{sec}=0.25 [5] for the maximum degradation at the attainment of the secant stiffness.

The interstorey drift thresholds (IDR_{DSj}) assumes values of 0.03%, 0.2% and 1.2% for DS1, DS2 and DS3, respectively [7]; h_1 and h_s are the interstorey heights of the ground and upper floors, respectively

$$S_{d|DS1} = \frac{d_{n|DSj}}{\Gamma}\ ; \qquad d_{n|DSj} = \begin{cases} d_{n|DS1} = IDR_{DS1} \cdot h_1[1+\chi\cdot\zeta\cdot(n-1)] & ,\ j=1 \\ d_{n|DS2} = IDR_{DS2} \cdot h_1[1+\chi\cdot\zeta\cdot\gamma\cdot(n-1)] & ,\ j=2 \\ d_{n|DS3} = d_{n|DS2} + h_1(IDR_{DS3} - IDR_{DS2}) & ,\ j=3 \end{cases} \tag{4}$$

$$\chi = \frac{K_1}{\overline{K_{1<i\le n}}} = \frac{K_1}{K_s}\ ; \quad \zeta = \frac{\overline{V_{1<i\le n}}}{V_1}\ ; \quad \gamma = \frac{\Omega_1}{\overline{\Omega_{1<i\le n}}} = \frac{\Omega_{sec}}{\overline{\Omega_{1<i\le n}}}\ ; \qquad 0<\chi,\ \zeta,\ \gamma\ \le 1 \tag{5a,b,c}$$

2.2 Generalisation of the method: non-regularly infilled RC MRF

Infilled RC-MRF with a substantial reduction or even inexistence of infills at ground floor are usually called pilotis frames. In FAST approach, this situation is represented by a lower value of χ if compared with the value of 1.0 for the uniformly infilled frame. It is worth to note that all the intermediate situations from uniformly infilled to pilotis are susceptible to be considered since χ can take any value from 1 to 0. In this section, expressions for required parameters χ, ζ, γ, (m^*/M), Γ, (K_g/A_b) are developed for both cases.

2.2.1 Definition of the irregularity of stiffness

The factor χ depends on the stiffness of ground and upper floors, both normalized to the building area A_b. The stiffness of a storey *i* is approximately the sum of the contribution of the infills' shear stiffness and the RC columns' flexural stiffness. The first one can be expressed (eqn (6a)) as function of the infills' area ratio ($\rho_{w,i}$), the shear modulus (G_w) and the interstorey height; while the second one (eqn (6b)), assuming a square section for the columns, depends on the Young modulus (E_c), the dimension of the columns section (b_c), the tributary area of loads for each column (A_{trib}) and the interstorey height.

$$K_{w,i}/A_b = \frac{G_w A_{w,i}}{h_1 A_b} = \frac{G_w \rho_{w,i}}{h_1}\ ; \qquad K_{RC,i}/A_b = n_c \frac{12 E_c I_{c,i}}{h_i^3} = \frac{A_b}{A_{trib}} \frac{12 E_c b_{c,i}^4}{12 A_b h_i^3} = \frac{E_c b_{c,i}^4}{A_{trib} h_i^3} \qquad \text{(6a,b)}$$

$$\chi = \frac{K_1}{K_s} = \frac{K_{w,1} + K_{RC,1}}{K_{w,s} + K_{RC,s}} \approx \frac{K_{w,1} + K_{RC,1}}{K_{w,s}} = \left(1 + c_\chi\right) \frac{\rho_{w,1}}{\rho_{w,s}} \frac{h_s}{h_1}\ ; \qquad c_\chi = \frac{K_{RC,1}/A_b}{K_{w,1}/A_b} \qquad \text{(7a,b)}$$

Therefore, χ is expressed (eqn (7a)) as function of the ratios of interstorey heights and infills areas between ground and upper floors, multiplied by a factor c_χ (eqn (7b)) which accounts for the contribution of the RC only in the ground floor, as for upper floors it may be neglected (see section 3.2.2).

2.2.2 Lateral load pattern

The value of χ determines the classification of a frame into uniformly infilled or pilotis. In order to have a better approximation of the two situations, two different load patterns are considered: linear and constant, respectively (Figure 2, eqn (8a)). H_i is the height to the floor i, and a and a' are free positive increasing parameters. It has been considered χ=0.5 as the corner value for distinguishing both cases. This value corresponds approximately to that for which the error respect to the exact results - in terms of first mode participating mass - obtained using both load patterns, become comparable.

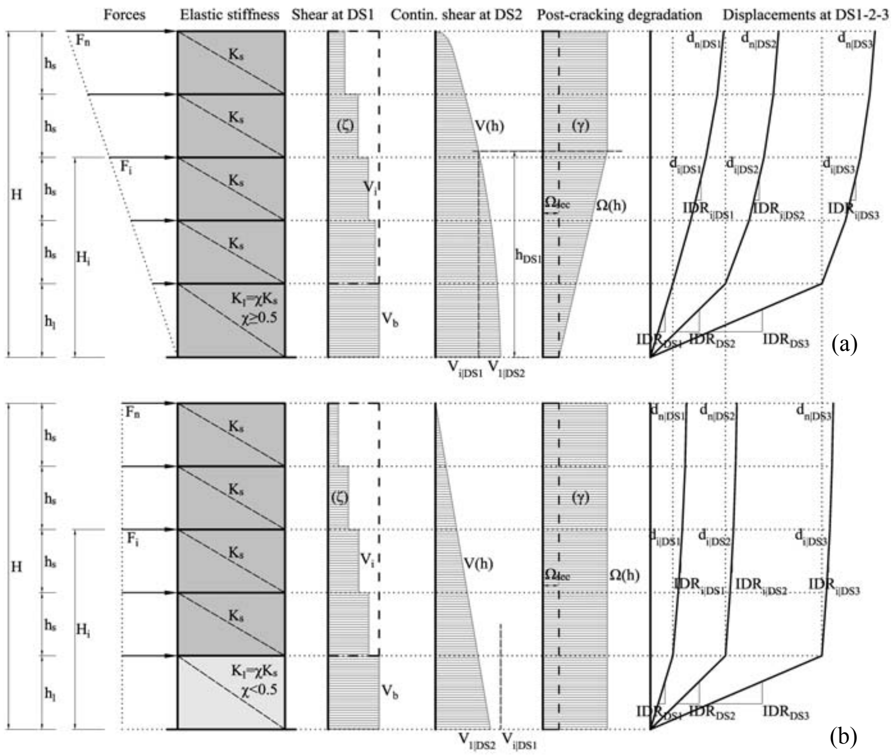

Figure 2: Models adopted for uniformly infilled (a) and pilotis (b) frames

$$F_i = \begin{Bmatrix} aH_i, \chi \geq 0.5 \\ a' \quad , \chi < 0.5 \end{Bmatrix} \Rightarrow V_i = \sum_i^n F_i = \begin{Bmatrix} a\sum_i^n H_i \quad , \chi \geq 0.5 \\ a'(n+1-i), \chi < 0.5 \end{Bmatrix} \Rightarrow V_{(i=1)} = V_1 = \begin{Bmatrix} a\sum_1^n H_i, \chi \geq 0.5 \\ na' \quad , \chi < 0.5 \end{Bmatrix} \tag{8}$$

$$\zeta = \frac{\sum_2^n V_i h_s}{V_1(H_n - H_1)} = \frac{\sum_2^n V_i}{V_1(n-1)} = \begin{Bmatrix} \frac{\sum_2^n \sum_i^n H_i}{(n-1)\sum_1^n H_i} \approx \frac{2}{3} \ , \chi \geq 0.5 \\ \frac{\sum_1^{n-1} i}{n(n-1)} = \frac{1}{2} \quad , \chi < 0.5 \end{Bmatrix} \tag{9}$$

Consequently, shear forces in each floor V_i and base shear V_1 are obtained for each model (eqn (8b)). Shear force shapes are parabolic and linear, respectively for uniformly infilled and pilotis frames. The corresponding factors ζ are obtained in eqn (9) by developing eqn (5b) and substituting shear forces expressions from eqn (8). For the uniformly infilled frame it assumes a value of 2/3 –the area of the parabola respect to the circumscribed rectangle— for $h_1=h_s$, being some hundredths down for $h_1>h_s$. For the pilotis frame, it takes an independent value of 1/2 (the area of the triangle respect to the circumscribed rectangle). In all the subsequent, for the uniformly infilled frame, if $h_1=h_s$, H_i may be substituted by i in the formulations.

2.2.3 Elastic dynamic properties

From geometry, shear forces and relative stiffnesses, interstorey displacements (Δd_i) and top displacements (d_n, eqn (10)) can be obtained, the latter evaluated as the sum of the ground and upper contributions, expressing the ground stiffness and the upper shear forces by using χ and ζ factors, respectively. Furthermore, as masses are similar for every floor, (m^*/M) can be expressed (eqn (11a)) as function only of storey displacements. The final expression ((eqn (11b)) is developed by replacing the values for shear forces and top displacement. In a similar way, Γ is obtained (eqn (12a and b)). Different values of (m^*/M) and Γ for equal interstorey heights are shown in Table 1, being consistent with [8].

Table 1: (m^*/M) and Γ for 4-8-storey uniformly infilled and pilotis frames

Uniformly infilled frame					Pilotis frame				
χ	n	m^*/M	Γ	$\Gamma\, m^*/M$	χ	n	m^*/M	Γ	$\Gamma\, m^*/M$
1.0	4	0.71	1.25	0.89	0.3	4	0.86	1.13	0.98
	8	0.67	1.28	0.85		8	0.80	1.19	0.95

$$d_n = \Delta d_1 + \sum_2^n \Delta d_i = \frac{V_1}{K_1} + \sum_2^n \frac{V_i}{K_s} = \frac{V_1}{K_s}\left[\frac{1}{\chi} + \zeta(n-1)\right] \tag{10}$$

$$\frac{m^*}{M} = \frac{\sum_1^n d_i}{nd_n} = \frac{\Delta d_i + \sum_2^n \Delta d_i(n+1-i)}{nd_n} = \frac{\frac{nV_1}{\chi} + \sum_2^n V_i(n+1-i)}{nd_n K_s} \tag{11a}$$

$$\frac{m^*}{M}=\begin{cases}\dfrac{\dfrac{n}{\chi}\sum_1^n H_i+\sum_2^n\left[(n+1-i)\sum_i^n H_i\right]}{n\left[\dfrac{1}{\chi}+\zeta(n-1)\right]\sum_1^n H_i}, & \chi\geq 0.5\\ \dfrac{\dfrac{n^2}{\chi}+\sum_1^{n-1} i^2}{n^2\left[\dfrac{1}{\chi}+\dfrac{1}{2}(n-1)\right]}, & \chi<0.5\end{cases}\tag{11b}$$

$$\Gamma=d_n\frac{\sum_1^n d_i}{\sum_1^n d_i^2}=\frac{V_1\left[\dfrac{1}{\chi}+\zeta(n-1)\right]\left[\dfrac{nV_1}{\chi}+\sum_2^n V_i(n+1-i)\right]}{\dfrac{V_1^2}{\chi^2}+\sum_2^n\left(\dfrac{V_1}{\chi}+\sum_2^i V_i\right)^2}\tag{12a}$$

$$\Gamma=\begin{cases}\dfrac{\sum_1^n H_i\left[\dfrac{1}{\chi}+\zeta(n-1)\right]\left[\dfrac{n\sum_1^n H_i}{\chi}+\sum_2^n\left[(n+1-i)\sum_i^n H_i\right]\right]}{\dfrac{\left(\sum_1^n H_i\right)^2}{\chi^2}+\sum_2^n\left[\dfrac{\sum_1^n H_i}{\chi}+(i-1)\sum_1^n H_i-\sum_2^i\sum_1^{j-1} H_j\right]^2}, & \chi\geq 0.5\\ \dfrac{n\left[\dfrac{1}{\chi}+\dfrac{1}{2}(n-1)\right]\left[\dfrac{n^2}{\chi}+\sum_2^n(n+1-i)^2\right]}{\dfrac{n^2}{\chi^2}+\sum_2^n\left[\dfrac{n}{\chi}+(i-1)(n+1)-\sum_2^i i\right]^2}, & \chi<0.5\end{cases}\tag{12b}$$

$$K_g/A_b=\frac{V_1}{A_b d_n}=\frac{K_s/A_b}{\dfrac{1}{\chi}+\zeta(n-1)}=\frac{G_w\rho_{w,s}}{h_s\left[\dfrac{1}{\chi}+\zeta(n-1)\right]}\tag{13}$$

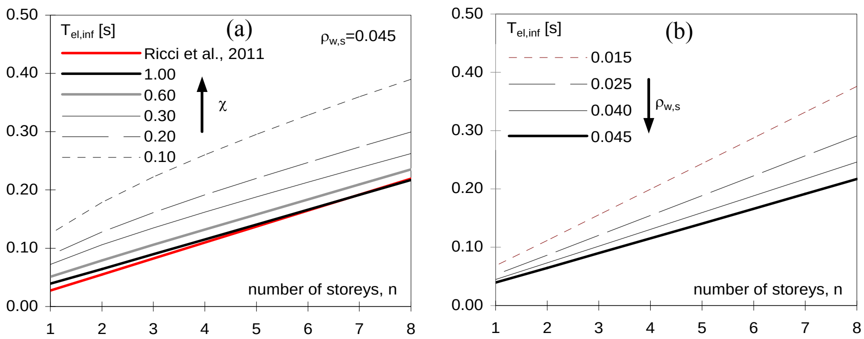

Figure 3: Elastic fundamental period depending on χ (a) and $\rho_{w,s}$ (b)

2.2.4 Elastic fundamental period

In eqn (3), the elastic fundamental period is expressed as function of n, (m^*/M) and (K_g/A_b). The last parameter (normalized global stiffness of the frame) can be obtained (eqn (13)) as the ratio between base shear force and top displacement, normalized by the building area. Accordingly, the so calculated elastic fundamental period (Figure 3) increases its value with the decreasing of χ, showing a quasi-linear shape when expressed as function of the number of storeys. For χ=1 (uniformly infilled frame), the values of $T_{el,inf}$ for different values of infill ratio are similar to those obtained in [5].

2.2.5 Post-cracking stiffness degradation

Finally, in order to calculate the factor γ (see subsection 2.1.2), it is necessary to find out how many floors are beyond cracking at the attainment of the DS2 in the ground floor. The specific values of shear that causes the DS2 in the ground floor and the DS1 in any upper floor are calculated in eqn (14). Thus, it is possible to determine the corresponding shear distribution whose base shear is $V_b=V_{1|DS2}$ and to find out the height corresponding to the shear $V(h)=V_{i|DS1}$ (Figure 2). By using the discrete expression of storey shear in eqn (8), it becomes impossible to clear up the height as dependent on the shear. Hence, a continuum expression for shear forces is used for both models (eqn (15)), expressing the relative height h/H as dependent on the shear function $V(h)$ and the base shear V_b (eqn (16)). Then, by replacing values (eqn (17)), the "cracked" height is obtained.

A negative value of h_{DS1}/H would mean that a greater shear force than $V_b=V_{1|DS2}$ is necessary to make the upper floors overcoming the DS1. For this reason, h_{DS1}/H must be limited to be positive. Common values for this relative height are 0.65 for the uniformly infilled frame (2/3 of the height cracked) and 0.00 for the pilotis frame (upper floors elastic).

The function $\Omega(h)$ (see point 2.1.2) (eqn (18a)) has a linear part, from Ω_{sec} in the base to 1.0 at h_{DS1}, remaining constant in the upper elastic part. Therefore, the factor γ is developed from eqn (5c) to be expressed as in eqn (18b). Typical values are 0.35 and 0.25 for uniformly infilled and pilotis frame, respectively.

$$IDR_i=\frac{\Delta d_i}{h_i}=\frac{V_i}{K_i h_i}\Rightarrow\begin{Bmatrix}V_{i|DS1}=IDR_{DS1}K_s h_s\\ V_{1|DS2}=IDR_{DS2}\Omega_{sec}K_1 h_1\end{Bmatrix} \tag{14}$$

$$V(h)=\begin{Bmatrix}\frac{a}{2}\left(H^2-h^2\right),\ \chi\geq 0.5\\ a'\left(H-h\right)\ \ ,\ \chi<0.5\end{Bmatrix}\Rightarrow V(0)=V_b=\begin{Bmatrix}\frac{aH^2}{2},\ \chi\geq 0.5\\ a'H\ ,\ \chi<0.5\end{Bmatrix} \tag{15}$$

$$\frac{V(h)}{V_b}=\begin{Bmatrix}1-\left(\frac{h}{H}\right)^2,\ \chi\geq 0.5\\ 1-\frac{h}{H}\ \ ,\ \chi<0.5\end{Bmatrix}\Rightarrow\frac{h}{H}=\begin{Bmatrix}\sqrt{1-\frac{V(h)}{V_b}}\ ,\ \chi\geq 0.5\\ 1-\frac{V(h)}{V_b}\ \ ,\ \chi<0.5\end{Bmatrix} \tag{16}$$

$$\frac{h_{DS1}}{H}=\left\{\begin{array}{ll}\sqrt{1-\frac{V_{i|DS1}}{V_{1|DS2}}}=\sqrt{1-\frac{1}{\chi\Omega_{\text{sec}}}\frac{IDR_{DS1}}{IDR_{DS2}}\frac{h_s}{h_1}} & ,\ \chi\geq 0.5\\ 1-\frac{V_{i|DS1}}{V_{1|DS2}}=1-\frac{1}{\chi\Omega_{\text{sec}}}\frac{IDR_{DS1}}{IDR_{DS2}}\frac{h_s}{h_1} & ,\ \chi<0.5\end{array}\right\}\geq 0 \tag{17}$$

$$\Omega(h)=\left\{\begin{array}{ll}\Omega_{\text{sec}}+h\frac{1-\Omega_{\text{sec}}}{h_{DS1}}, & h<h_{DS1}\\ 1 & ,\ h\geq h_{DS1}\end{array}\right\};\qquad \gamma=\frac{H\Omega_{\text{sec}}}{\int_0^H\Omega(h)dh}=\frac{2\Omega_{\text{sec}}}{\frac{h_{DS1}}{H}\left(\Omega_{\text{sec}}-1\right)+2}\geq\Omega_{\text{sec}} \tag{18}$$

3 Application to the 2011 Lorca, Spain, earthquake

The Mw=5.1 11th May 2011 Lorca (Murcia, Spain) earthquake, whose special characteristics have been described in details in [1,9], is used as a benchmark for testing the generalized FAST. In this section, specific values for the parameters depending on the local construction practice are discussed, and results of predicted damages are presented are interpreted.

3.1 Real damage scenario

Collected damage information [10], following the EMS-98 classification and its subsequent disaggregation for the only RC structures (Figure 4), shows that severe structural damage (DS4 and DS5) is limited (8.5%) and non-structural damage increases with the number of storeys, being the median included in the middle range of DS2.

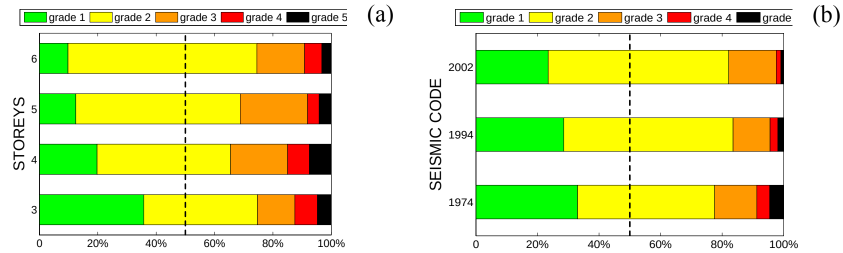

Figure 4: DS for RC buildings by number of storeys (a) and seismic code (b)

3.2 Common design practice

Lorca RC building stock is approximately characterized by buildings in the range of 3 and 6 floors, being 3.50m and 3.00m the respective interstorey heights for ground and upper floors. RC building stock is constructed mainly after 1974 [1,9,10] and so following the Spanish seismic codes PDS-1 (1974), NCSR-94 (1994) and NCSE-02 (2002). As these codes do not provide any quantitative capacity design, drift limitations or resistance increase because of irregularity in

elevation, the usual structural solution is: wide beams, slender columns, no shear walls and typical reduction of infills' percentage at ground floor.
Aimed at defining the representative factor of irregularity (χ, eqn 7a) for Lorca, it is necessary to establish the typical values of $\rho_{w,i}$ and $\rho_{w,s}$ for infills and b_c and A_{trib} for RC columns (see point 2.2.1). Besides, m_r is estimated in 0.8t/m^2, G_w in 1350MPa [11] and τ_{max} is assumed to be 1.3·0.35MPa [11,12].

3.2.1 Infills

Data available in literature [13] for the typical amount of masonry infills in the residential buildings of Mediterranean area suggest a constant-in-elevation $\rho_{w,i}$ equal to 2.5%. These data are integrated by an in-field observational analysis aimed at distinguishing between ground floor and upper floor. Two typical residential buildings with commercials in ground floor of Lorca (Figure 5) are analysed, distinguishing the infills in three types: external (ex), which are facades or walls thicker than 15cm, inserted into a structural frame; internal aligned (al), thin walls of usually 10cm inserted into a structural frame; and internal not aligned (in), the rest of them. Openings are taken into account.

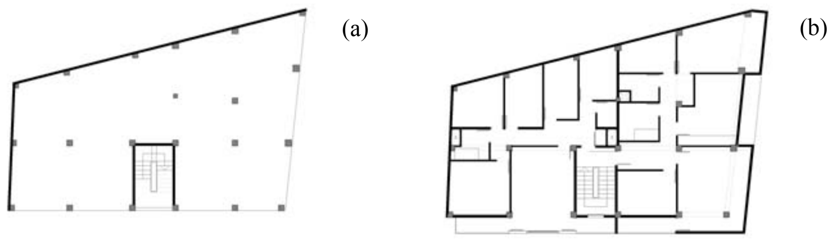

Figure 5: Ground (a) and upper floor (b) of building A

Results (Table 2) show that average value for the consideration of both external and aligned infills is 2.6%; neglecting the internal aligned walls furnishes rather low values. The walls which are not within a frame are usually neglected, as they are considered not to be able to develop a post-cracking diagonal-strut behaviour. However, they may influence the value of the elastic period and, which is more important, they may remain elastic in a pilotis frame (see point 2.2.5). If considering all the type of walls, the amount of upper walls increases to 4.2% resulting in a ratio of 0.36 between ground and upper storeys.

Table 2: Infills ratios for buildings A and B

Building	Direction	$\rho_{w,1}$ [%]			$\rho_{w,s}$ [%]		
		ex	ex+al	ex+al+in	ex	ex+al	ex+al+in
A	x	1.3	1.5	1.5	2.1	2.8	4.5
	y	0.8	1.4	1.4	2.1	3.1	4.6
B	x	0.8	1.0	1.3	1.6	2.1	3.9
	y	1.1	1.4	1.8	1.9	2.3	3.9
	average	1.1	1.3	1.5	1.9	2.6	4.2
				$\rho_{w,1}/\rho_{w,s}$	0.52	0.51	0.36

3.2.2 RC structure

A lower-bound estimation of RC columns dimensions have been evaluated as the maximum of the gravitational and seismic estimations (eqns 19 and 20, respectively) proposed by the Spanish code DA-EHE, that does not provide any capacity design. A_{trib} has been considered to be $15m^2$ (based on in-field observations), and typical values of design loads, material properties and code provisions have been considered, being $N_{d,grav}$, $N_{d,seis}$ and $M_{d,seis}$ the design demands, $A_{s,tot}$ and A_{s1} are the reinforcement areas and f_{cd} and f_{yd} the design resistances. Hence, values of c_χ (see point 2.1.2) oscillate between a minimum of 4% and 27%, considering the values of infills ratio exposed in the previous point. Furthermore, it has been proved that in upper floors, even considering only the external infills, c_χ<5%. Thus the RC influence can be neglected in this case.

$$N_{d,grav}\left(1+2.5\frac{\max\left(b_{c,i}/20,20mm\right)}{b_{c,i}}\right)=b_{c,i}^2 f_{cd}+A_{s,tot}f_{yd} \tag{19}$$

$$M_{d,seis}-\frac{N_{d,seis}}{2}\left(b_{c,i}-\frac{N_{d,seis}}{b_{c,i}f_{cd}}\right)=A_{s1}f_{yd}\left(b_{c,i}-2d'\right) \tag{20}$$

3.3 Results: predicted damages

Predicted DS for the PGA of Lorca earthquake have been obtained with the generalized FAST (Figure 6) for Lorca infilled uniformly infilled and pilotis RC-MRF from 3 to 6 storeys, designed according the 1974, 1994 and 2002 Spanish seismic codes, and considering the three different ways of accounting for the infills discussed in the point 3.2.1, in order to estimate upper and lower limits of the average estimated damage aimed at a comparison with the observed damage.

Results show that the expected lower limit of damage is DS1 (uniformly infilled frame considering all the walls) and upper limit is DS3 (pilotis frames considering only the external walls). Expected damage is inversely proportional to the amount of infills considered, and low influence of the number of storeys is observed, because in this range of periods the spectral demands do not vary uniformly. As explained in point 3.2.1, the expected damages for both cases (uniformly infilled and pilotis) may be those intermediate between the results accounting and not accounting for the internal not aligned walls. For uniformly infilled frames, not accounting for them may be a reasonable consideration; otherwise the unreal increment of base shear would distort the results. However, for pilotis frames, as all the upper floors remain elastic and usually there are not many interior not-aligned infills in the ground floor, the best way may be to consider all the walls. Thus, average DS2 can be expected for the RC building stock of Lorca, prediction in accordance with the observed damage (Figure 4).

4 Conclusion

A general form of FAST vulnerability method for infilled RC-MRF buildings with a certain reduction of the amount of infills in the ground floor, covering all intermediate situation between the uniformly infilled and pilotis case, is presented. The theoretical development is supported by assumptions based on the existing literature and the expressions for some required variables, as the elastic fundamental period of regular and irregular infilled frames, are consistent with other works. Finally, FAST approach is quite satisfactorily tested with an observed earthquake damage scenario.

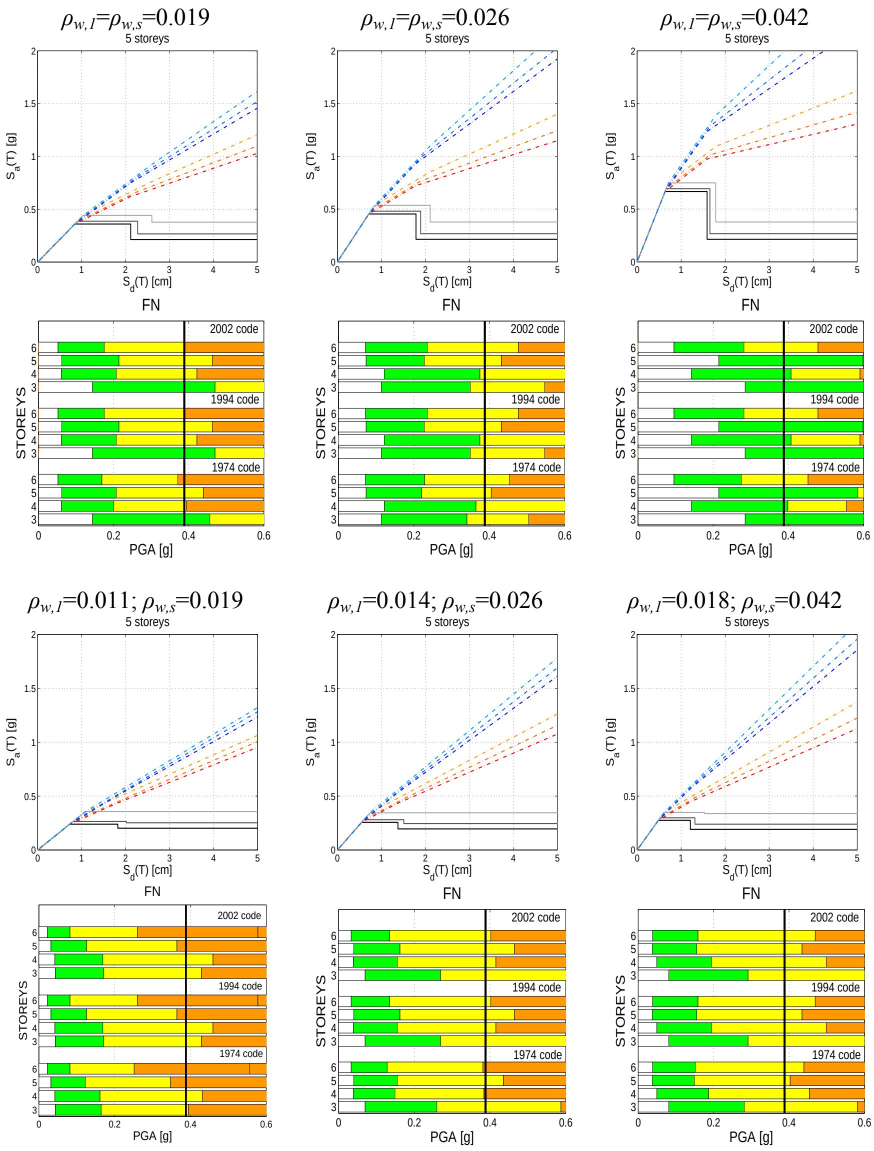

Figure 6: Predicted IN2+CC curves and DS for all the cases

References

[1] De Luca F., Verderame G.M., Gómez-Martínez F., Pérez-García A. The structural role played by masonry infills on RC building performances after the 2011 Lorca, Spain, earthquake. Bulletin of Earthquake Engineering (2013) (under review).

[2] Manfredi G., Verderame G.M., Prota A., Ricci P., De Luca F., (2012a). 2012 Emilia earthquake, Italy: Reinforced Concrete buildings response. Bulletin of Earthquake Engineering (under review).

[3] Dolšek M., Fajfar P. (2005). Simplified non-linear seismic analysis of infilled reinforced concrete frames. Earthquake engineering and Structural Dynamics 34,49-66

[4] Manfredi G., Ricci P., Verderame G.M. (2012). Influence of Infill Panels and Their Distribution on Seismic Behavior of Existing Reinforced Concrete Buildings. The Open Construction and Building Technology Journal, 2012, 6, (Suppl 1-M1).

[5] Ricci P., Verderame G.M., Manfredi G. (2011). Analytical investigation of elastic period of infilled RC MRF buildings. Engineering Structures 33(2), 308-319

[6] Galasso C., Cosenza E., Maddaloni G. (2011). Statistical analysis of reinforcing steel properties for seismic design of RC structures. Proceedings of the 14th European Conference on Earthquake Engineering, August 30-September 3, Ohrid, Republic of Macedonia

[7] Colangelo F. (2012). A simple model to include fuzziness in the seismic fragility curve and relevant effect compared with randomness. Earthquake Engineering and Structural Dynamics, 41, 969-986

[8] American Society of Civil Engineers (ASCE), Seismic Rehabilitation of Existing Buildings, ASCE/SEI 41-06, Reston, Virginia, 2007

[9] Cabañas L., Carreño E., Izquierdo A., Martínez J.M., Capote R., Martínez J., Benito B., Gaspar J., Rivas A., García J., Pérez R., Rodríguez M.A., Murphy P. (2011). Informe del sismo de Lorca del 11 de mayo de 2011.

[10] Feriche M., Vidal F., Alguacil G., Navarro M., Aranda C. Vulnerabilidad y daño en el terremoto de Lorca de 2011 (2012). 7ª Asamblea hispano-portuguesa de Geodesia y Geofísica. San Sebastián, 25-29 de junio de 2012

[11] CS.LL.PP.; 2009: Istruzioni per l'applicazione delle norme tecniche delle costruzioni. Gazzetta Ufficiale della Repubblica Italiana, 47 (in Italian).

[12] Fardis M.N., 1997. Experimental and numerical investigations on the seismic response of RC infilled frames and recommendations for code provisions. Report ECOEST-PREC8 No 6.

[13] Crowley H., Pinho R., (2010). Revisiting Eurocode 8 formulae for periods of vibration and their employment in linear seismic analysis. Earthquake Engineering and Structural Dynamics, 39(2), 223-235