Preliminary study of the structural role played by masonry infills on RC building performances after the 2011 Lorca, Spain, earthquake

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SUMMARY:
On May 11, 2011 an earthquake of magnitude 5.1 (M_w) struck Murcia region causing 9 casualties and damage to buildings and infrastructures. Even if the main characteristics of the event would classify it as a moderate earthquake, the maximum PGA registered (equal to 0.37g) exceeded significantly the hazard at the site according to local code provisions. The latter is a result of directivity effects in the near source region. An overview of earthquake characteristics and a general study of the damage observed is provided. Finally, an approximate quantitative, large scale, explanation of the damage observed is carried out on the basis of recent literature findings. Notwithstanding the lack of proper structural design characterizing building stock in the area, most of the losses were caused by non-structural damage and, according to in field observations, it seems clear that masonry infills provided additional, "not designed", strength to reinforced concrete buildings.

Keywords: Lorca earthquake, RC building, Infill, Damage survey, Brittle failure

1. INTRODUCTION

Most of the experience regarding the early earthquake engineering was collected after disastrous earthquakes. Then earthquake engineering has progressed towards quantitative and probabilistic assessment frameworks for the control of seismic risk. Thanks to damage surveys after earthquake events in the last decade (e.g., Rossetto and Peiris 2009, Ricci et al. 2011a), it was observed that RC performance can be significantly increased or decreased thanks to the "structural" role played by "non structural" elements such as masonry infills. Recent studies have been aimed at the quantification of infill structural role (Dolsek and Fajfar 2001, 2004a, 2005, Ricci 2010). However it is hard to quantify their structural role unless a detailed macro-modelling approach is followed (Verderame et al. 2011).

In the following, data and general considerations, carried out after recent earthquake surveys (Ricci et al. 2011a) are employed to build up an approximate tool for the evaluation of infill structural contribution to the performances of RC buildings during the 11^{th} May, 2011 Lorca earthquake. Aimed at a comprehensive evaluation of the case study event considered, a general overview of earthquake seismological characteristics and main structural and non structural damage registered is provided. The approximate quantitative tool carried out, asks for a look into the evolution of Spanish design codes other than requiring basic information on building stock of the region.

2. CHARACTERISTICS OF THE EVENT

On 11^{th} May 2001 an earthquake of M_w=5.1 struck the Murcia region (IGN 2011); the epicentre was 3 km to NE from the seismic station of Lorca. It was preceded by a M_w=4.5 foreshock nearly in the same place (3.5 km to the seismic station), see Fig. 1. Focal mechanism was strike slip with low inverse influence (Fig. 1a). The proximity to the epicentre and the low hypocentre profundity (2-4 km for both events), caused very high macro seismic intensities in Lorca for such moderate event; VI and VII for the foreshock and mainshock, respectively (Cabañas et al. 2011).

The main earthquake was registered by 17 stations located from 3 to 185 km from the epicentre. Lorca station was the nearest one, placed on soil type B (Fig. 1). At this station a maximum PGA of 0.367g for the mainshock NS component was registered. This value is more than three times bigger than the design code PGA for housing in this type of soil (0.124g, see section 5.1) and has 0.01% probability of being exceeded in 50 years (RISMUR 2006). On the other hand, according to new probabilistic seismic hazard study, performed before the 11th May event, an increment from 0.12g to 0.19g for Lorca code basic acceleration was suggested (Mezcua et al. 2011). Furthermore, it is worth to observe that the event was characterized by a significant attenuation with the distance(Fig. 1b and 1c).

Ground motion characteristics in Lorca station show a significant difference between NS and EW components. The record registered at Lorca station was rotated according to parallel (FP) and normal (FN) directions of the fault of Alhama (strike = 230º). Such a representation (Fig. 2) allows the visual evaluation of T_C and T_D periods delimiting the constant velocity branch of the spectra (Lam et al. 2000). T_C and T_D periods are equal to 0.48s and 0.57s for FN component and equal to 0.24s and 0.88s for FP component. The velocity spectrum for the normal-fault component of the record shows a very short stretch of constant velocity value, typical of the impulsive motions (Chopra 2007). Indeed, the quantitative method by Baker (2007) confirmed that a velocity pulse occurs at period TP equal to 0.68s; Baker's pulse indicator for FN components is equal to 0.99 (Fig. 3).

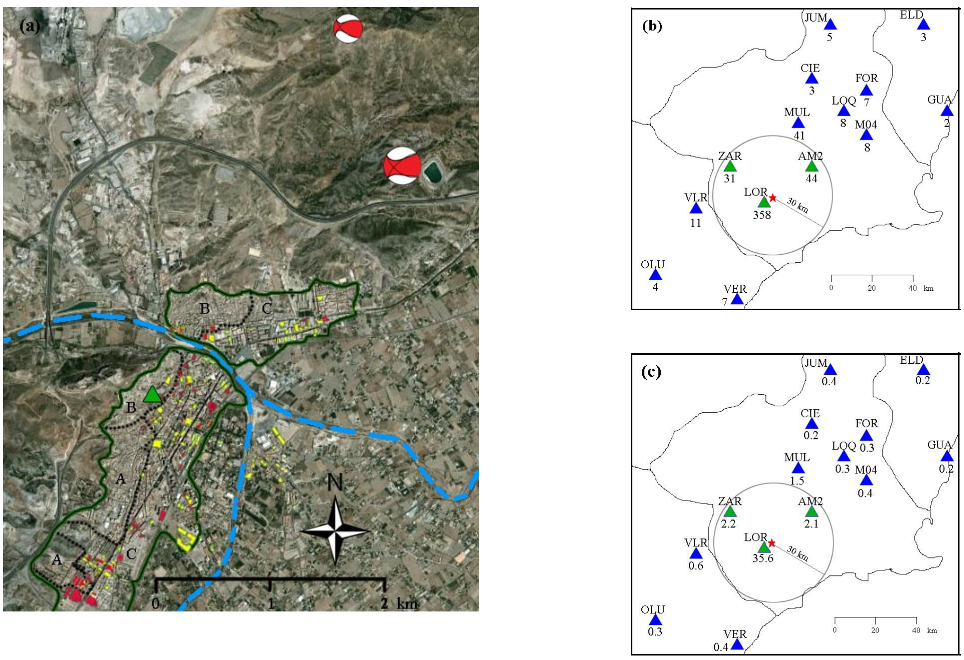

Figure 1. Lorca: river Guadalentín (discontinuous blue line), limit of the constructed area (green-black line), limits of different soil types (EC 8 classification, black dotted line), LOR seismic station (green triangle), mainshock and foreshock epicenter (big and small beach balls, respectively), high (red) and medium (yellow) damaged buildings (a); PGA in cm/s^2 (b) and PGV in cm/s (c) measured in the different stations.

3. COMMON DESIGN PRACTICE FOR RC BUILDINGS IN LORCA

Data from INE (2001) and Cabañas et al. (2011) were crossed, showing that 77% of the buildings are masonry structures and 23% are modern frames (mainly RC). Most of RC buildings have between 3 and 5 storeys (Fig. 4a), and had been designed according new Spanish seismic codes (Fig. 4b) NCSR-94 and NCSE-02, as few RC structures had been realized before 1985.

Since the first reference to seismic actions in the code, MV-101 (1962), four seismic codes have been released in Spain, classified in two groups: “old codes” PGS-1 (1968) and PDS-1 (1974) and “new codes” NCSR-94 (1994) and NCSE-02 (2002). Old codes measure the hazard by MSK intensity level, only provide simplified static analysis, and do not consider explicitly either behaviour factor (q) or capacity design criteria. However, new codes also have significant lacks if compared with Eurocode 8 (CEN 2004): there are not proper quantitative rules aimed at imposing strength hierarchy; capacity design is not required. Low-ductility structural design (q=2) is allowed with no restrictions for seismic zones characterized by higher hazard, and neither drift limitations, or q reduction because of structural and non-structural irregularities are provided. Nevertheless, the last RC code EHE-08 (2008) recommends the use of capacity design method and is very close to Eurocode 8, EC8 provisions (CEN 2004).

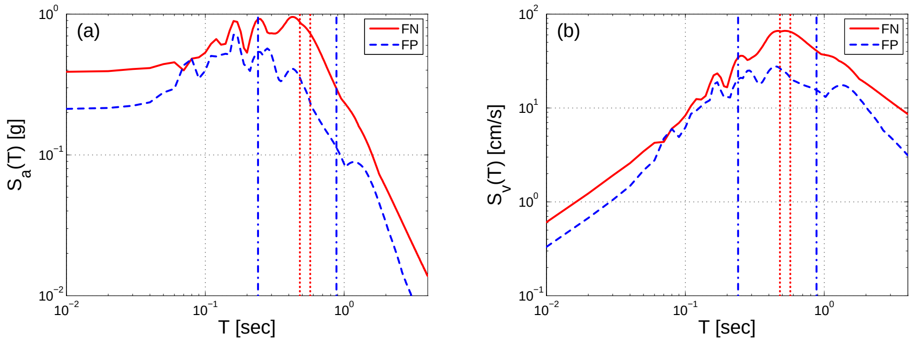

Figure 2. Acceleration (a) and velocity elastic spectra (b) for fault normal (FN), and fault parallel (FP) mainshock signals registered in Lorca (LOR) station with the evaluation of T_C and T_D.

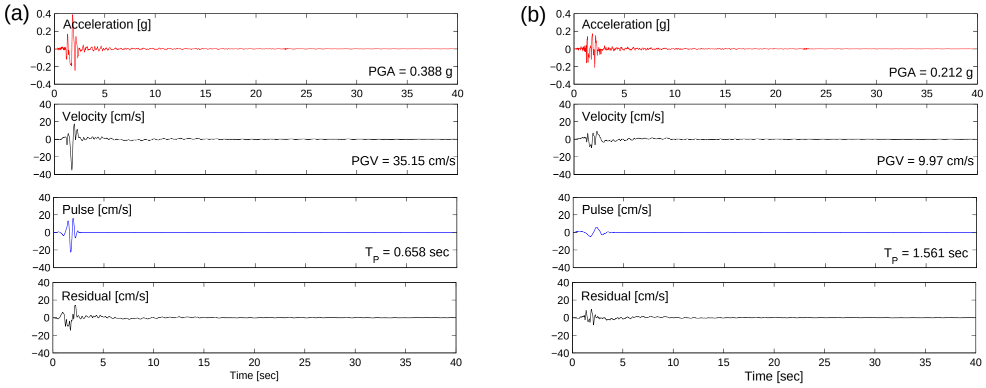

Figure 3. Baker (2007) quantitative classification of fault normal (FN) (a), and fault parallel (FP) (b) mainshock signals registered in Lorca (LOR) station, characterized by pulse indicators equal to 0.99 and 0.03, respectively.

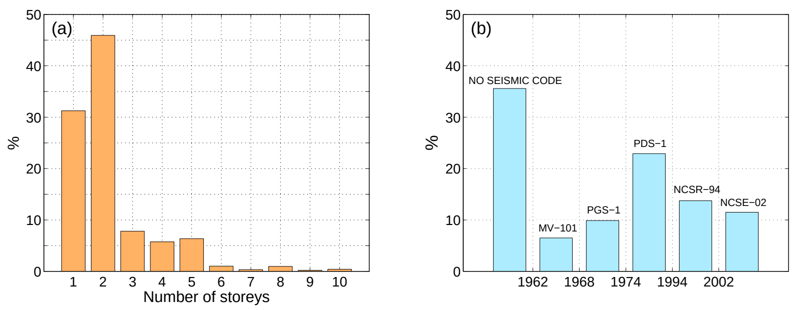

Figure 4. Frequency distribution in Lorca building stock for: number of storeys (a), applied code provision for the design of the building stock (b).

Conception of buildings in Spain does not follow the basic seismic provisions or construction rules normally adopted in seismically prone region. Low ductile slab arrangements are in all the country and in Murcia region. Moreover, given the severe deflection limitations provided by previous RC design codes, slabs usually got oversized, thus leading to the absence of capacity design,. RC shear walls are seldom employed, so masonry infilling panels play a significant role in the initial lateral stiffness and strength. Seismically questionable design solutions, likely causing soft-storey ground-floor behaviours are quite frequent, due to the lack of infill walls (*pilotis*) or much slender columns because of higher interstorey height; also squat or captive columns are quite frequent. The latter characteristics represent one of the main weaknesses of Spanish building stock.

4. DAMAGE OBSERVED

According to damage survey after the earthquake, buildings were classified as *yellow* or *red* (Cabañas et al. 2011). In the first case they were considered unsafe for normal use and characterized by low structural damage; in the second case they were characterized by significant structural damage (see Fig 1a). A summary of the results of rapid survey is provided; 13% of the buildings were damaged; red buildings were 8.1% while yellow buildings were 4.9% of the entire Lorca building stock. The damage was not homogeneous between similar and contiguous structures. It can be explained by directivity effects characterizing the event. Type of soil in Lorca has been classified (Gaspar-Escribano et al. 2008, RISMUR 2006), as shown in Fig. 1a; the east part of the city is characterized by C soil type (worse quality). The density of damage in this area is higher, and it is also the part in which most of the RC buildings are sited. Crossing the information available, a probable percentage of masonry buildings damaged of 11.7% was estimated, while for RC buildings is 16.9%. Seismic demand for RC buildings was higher than for masonry structures as a consequence of soil amplification. On the other hand, RC buildings' relative worse performances respect to masonry (16.9% versus 11.7%) suggest that typical construction practice of RC buildings in Spain can represents one of the principal cause of structural damage observed.

Figure 5. Brittle failures in RC columns; (b) and (c) (Calconsa XXI 2011).

Figure 6. Brittle failure of "squat" column (a) (Vidal et al. 2011), masonry staircase (b), beam-column joint (c).

Structural damage was not so frequent. Only one building collapsed. Most of the damage was caused by brittle failures in heads of columns at ground floor; the latter is the result of the lack of any capacity design rule. According to the construction practice ending up in very thick slab, no relevant damage was found in beams. Residual drifts recognized after the earthquake were not significant. The design approach did not allow the development of plastic deformation because of the occurrence of pre-emptive brittle failures or limited ductility failures (Sezen and Moehle 2004). Shear-axial failures in columns were frequent (Fig. 5a, 5b and 5c). Stirrups are characterized by low diameters, not proper spacing, and 90º hooks that do not confine the concrete core. Such shear design weaknesses accompanied by high longitudinal percentage ratios (see Fig. 5) increases significantly the possibility of brittle failures or limited ductility failures, ending up in typical shear diagonal cracking (Fig. 5d) or buckling of the longitudinal reinforcement and consequent axial load collapse. The other main cause of damage was the brittle failure of compressed diagonal concrete strut of "squat" (Fig. 6a) or "captive" columns (Fig. 5d), due to the presence of RC basement walls, masonry infilling panels or stairs (Fig. 6b). Beam-column joint failures were less frequent than expected (Fig. 6c), as flat beams favour more likely brittle failure in columns.

Figure 7. In-plane failures in masonry infill panels: shear diagonal cracking (a), horizontal sliding (b), corner crushing and induced brittle failure of RC column (c) (Calconsa XXI 2011)

Only in-plane failure of masonry infills of RC frames is analyzed. The three typical failures were found: diagonal cracking by tensile stress in the central zone (Fig. 7a); horizontal sliding (Fig. 7b); and corner crushing in the contact zone with the frame due to local compression efforts (Fig. 7c), leading sometimes to brittle failure into critical regions of RC columns because of the concentration of shear demand due to the interaction with the compressed diagonal of the panel (Verderame et al. 2011).

5. SPECTRAL CONSIDERATIONS BASED ON SIMPLIFIED CAPACITY ESTIMATION

Event and Lorca building stock's characteristics provide a clear scenario that justifies the main causes of the damage. On the other hand, considering directivity effects and design practice it can be stated that damage observed is relatively less spread than expected. A possible explanation can be given considering the structural contribution provided by masonry infills when pre-emptive brittle failures did not occurred. In the following a simplified approach is developed to quantify the performance increasing provided by infills, referring specifically to the design target of Lorca building stock.

Most of RC buildings in Lorca were designed following "new codes", and the vast majority of them have between 3 and 5 storeys considering q equal to 2 or giving implicitly the ductility correspondent to it in the case of old code approaches. Design spectral acceleration for the fundamental period T_1 is the key information to establish the minimum capacity of RC building according to different codes in the case of ductile behaviour (obviously discarding the occurrence of brittle failures that cannot be considered in such approximate framework). Equation 5.1 show the expressions of design spectra provided by old and new codes for a fundamental period higher than T_B (according to EC8 terminology); it is worth to note that old codes provide directly a design spectra while new codes employ q factor approach and provide elastic spectra. Approximate period formulations for old and new codes are provided in Equation 5.2, being H, L and n the height, length and number of storeys of the building. In the case of 3 to 5 storeys, T_1 is higher than T_B.

$$S_{a,old}\left(T\right)=C\left(T\right)\cdot g\cdot R\cdot\delta\cdot\beta\left(T\right);\qquad S_{a,new}\left(T\right)=a_c\cdot\alpha(T)\cdot\beta=\left(a_b\cdot\rho\cdot S\right)\cdot\alpha(T)\cdot\left(\nu/\mu\right)\qquad(5.1)$$

$$T_{1,old} = 0.09\,H/\sqrt{L}\,; \qquad T_{1,new} = 0.09n \tag{5.2}$$

In Fig. 8 code spectra are shown; the following values were used. For PDS-1, *C(T)* is the spectral amplification factor, dependent on T_1 and on the intensity MSK value (VIII for Lorca), and is equal to 0.15 in its maximum constant branch; *g* is the gravitational acceleration; *R* depends on the importance of the construction and on the MSK value, being equal to 0.9; δ is a "soil factor", dependent on the foundation (individual footings) and type of soil (B), it is equal to 1.1; and β is a "response factor", which depends on the infills contribution to damping (high for housing) and on T_1, being equal to 0.6. For NCSR-94 and NCSE-02, a_b is the basic PGA (0.12g for Lorca); ρ is the importance factor (1 for housing); *S* is the amplification factor for soil type B, equal to 1.00 for NCSE-94 and 1.04 for NCSE-02; $\alpha(T)$ is the spectral amplification factor, dependent on the T_1 and also on the soil for NCSE-94, whose maximum constant value is 2.20 for NCSE-94 and 2.50 for NCSE-02; v is the correction factor for damping, equal to 1.0 for 5% of the critical damping; and finally μ is the behaviour factor, assumed to be 2 in most of RC buildings (see section 3.2).

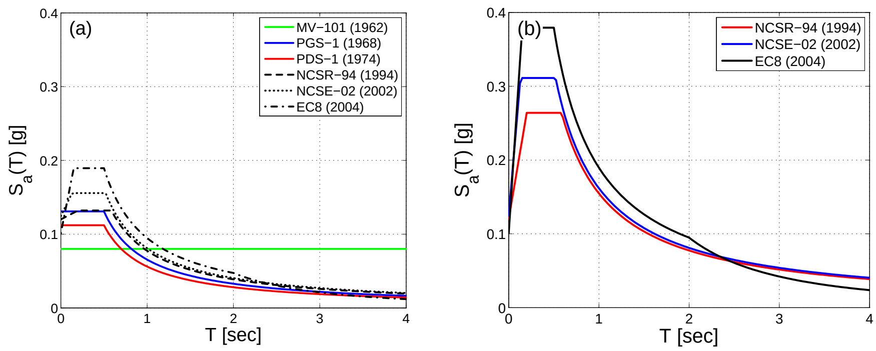

Figure 8. Design code spectra for Lorca in the case of implicit (old codes) or explicit (new codes and EC8) behavior factor q assumed equal to 2 (a); elastic code spectra for Lorca in the case of new codes and EC8 (b).

The minimum resistant capacity of representative RC buildings in Lorca, with no contribution of masonry infills, can be inferred as a function of the correspondent design spectral acceleration $S_a(T_1)$. The simplified inelastic acceleration capacity, C_s, can be evaluated according to Equation 5.3, being V_y the maximum base shear (corresponding to the formation of a plastic mechanism in the structure), V_d the design base shear, Γ_1 the first mode participation factor, m_1 the mass of the equivalent SDOF, *M* the mass employed for the evaluation of design base shear (in lateral force method it is generally an approximate evaluation of the participating mass to the first mode by means of a reducing coefficient). In Equation 5.3 the two terms R_α and R_ω refer to structural overstrength. R_α is the so called α_u/α_1 factor that accounts for structural redundancy and elements overstrength respect to design values, while R_ω is the materials' overstrength evaluated as the ratio between medium material strength and nominal material strength (Borzi and Elnashai 2000). Two conservative hypotheses have been made: ($M/(\Gamma_1 m_1)$)=1.00 and R_α=1.00. The latter is consistent with the attainment of a soft storey plastic mechanism of the structures. However, R_ω is assumed equal to 1.45, in analogy with the hypothesis in Borzi and Elnashai (2000) and similar to the ratio between medium and design yielding strength characterizing typical reinforcement steel (Galasso et al. 2010).

By the assumption of an adequate *R-μ-T* relationship (CEN 2004) and employing Equation 5.3, it is possible to define: *(i)* the inelastic displacement capacity C_d on the idealized elastic-plastic capacity curve (with a maximum base shear V_y); *(ii)* the corresponding IN2 curve (Dolsek and Fajfar 2004b), defining the capacity point ($S_a^c(T_1)$, C_d) in ADRS format. T_1 is evaluated according to the approximate formulation provided in (CEN 2004). The inelastic displacement capacity C_d and the elastic spectral acceleration capacity $S_a^c(T_1)$ are evaluated according to Equation 5.3.

$$C_s = \frac{V_y}{\Gamma_1 m_1} = \frac{V_d \cdot R_\alpha \cdot R_\omega}{\Gamma_1 m_1} = \frac{\left(S_a(T) \cdot M\right) \cdot R_\alpha \cdot R_\omega}{\Gamma_1 m_1}; \quad C_d = \mu \cdot C_s \cdot \left(\frac{T_1}{2\pi}\right)^2; \quad S_a^c(T_1) = C_s \cdot R_\mu \qquad (5.3)$$

Table 1. Simplified capacity and demand estimation for bare RC frames.

N. of storeys	3						5					
Heigth, H [m]	9						15					
Capacity	$S_a(T)$ [g]	C_s [g]	T_1 [s]	$C_{d,FN}$ [m]	$C_{d,FP}$ [m]	$S_a^c(T_1)$ [g]	$S_a(T_1)$ [g]	C_s [g]	T_1 [s]	$C_{d,FN}$ [m]	$C_{d,FP}$ [m]	$S_a^c(T_1)$ [g]
PDS-1	0.11	0.16	0.39	0.014	0.012	0.32	0.11	0.16	0.57	0.026	0.026	0.32
NCSE-02	0.16	0.23	0.39	0.019	0.017	0.45	0.16	0.23	0.57	0.037	0.037	0.45
Demand	$S^d_{a,FN}(T_1)$ [g]			$S^d_{a,FP}(T_1)$ [g]			$S^d_{a,FN}(T_1)$ [g]			$S^d_{a,FP}(T_1)$ [g]		
	0.90			0.39			0.73			0.24		

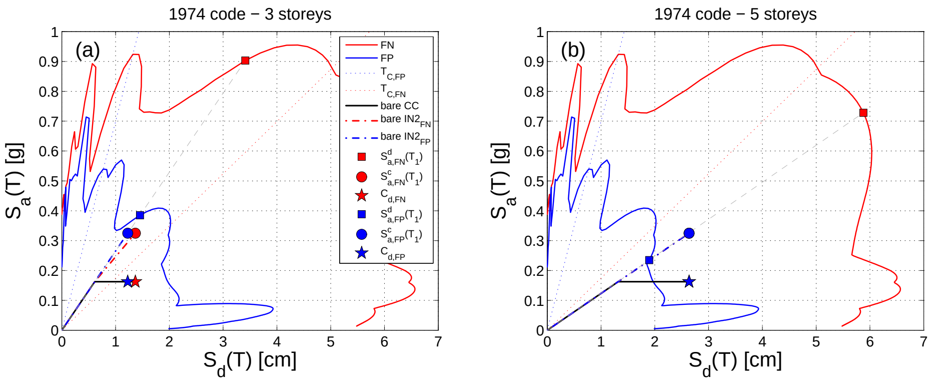

Figure 9. Simplified performance estimation of (a) 3 and (b) 5 storeys *bare* RC buildings designed according to PDS-1 (1974) code.

R_μ can be defined as q/R_α (Borzi and Elnashai 2000). In the applied example (Table 1), referred to 3 and 5 storey buildings, designed according to PDS-1 and NCSE-02, q=2, close to the value suggested in EC8 part 3 (CEN 2005); R_μ=2 in this case. R-μ-T relationship depends on T_C, so C_d value are computed for both FN and FP components considering the respective T_C values, resulting in a capacity dependence from the demand just in the case of $T_1<T_C$. After that, it is necessary to compute the seismic demand of the event; in Table 1 elastic spectral acceleration demand $S_a^d(T_1)$ are shown for FN and FP mainshock signals registered in Lorca station. In Fig. 9 is shown the performance estimation in the case of PDS-1 code for 3 and 5 storeys bare RC buildings.

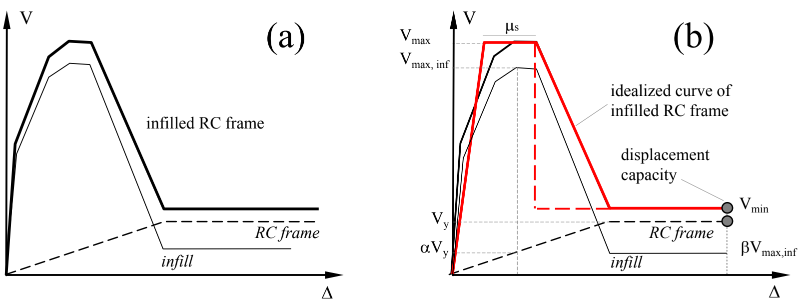

Figure 10. Example of infilled RC frame capacity curve (a), and its quadrilinear idealization (b).

Infills structural contribution on one hand increase the strength and the lateral stiffness (resulting in a decreasing of T_1); on the other hand infills lead to a strength degradation up to a minimum strength value, given the brittle nature of such non structural elements. The structural behaviour described above is mainly representative of substandard existing buildings (Dolsek and Fajfar 2001). Hence the simplified capacity curve of a fully infilled RC building (not including the case of *pilotis*) can be represented by quadrilinear backbone (Dolsek and Fajfar 2004a) characterized by an initial elastic plastic backbone (with the maximum base shear strength V_{max}) followed by a softening branch up to the minimum base shear strength (V_{min}). Herein the softening branch is characterized by a drop, the maximum and the minimum acceleration capacities, $C_{s,max}$ and $C_{s,min}$, respectively can be computed (Equation 5.4). Fig. 10 show a qualitative example of the approach followed.

$$C_{s,\max} = \frac{V_{\max}}{\Gamma_1 m_1} = \frac{V_{\max,\inf} + \alpha V_y}{\Gamma_1 m_1} = \frac{\tau_{\max} \cdot A_{\inf,x(y)} + \alpha V_y}{\Gamma_1 m_1} = \frac{\tau_{\max} \cdot \rho_{\inf,x(y)}}{n \cdot m \cdot \lambda} \cdot + \alpha C_s ; \quad C_{s,\min} = \frac{V_y + \beta V_{\max,\inf}}{\Gamma_1 m_1} \quad (5.4)$$

In Equation 5.4 $V_{max,inf}$ is the maximum base shear provided by the infills, τ_{max} is the maximum shear stress of the infills, according to Fardis (1997), in this case equal to 0.478MPa (CS. LL.PP. 2009); $A_{inf,x(y)}$ is the area in plan of the infills along the considered direction x(y), $\rho_{\inf,x(y)}$ is the ratio between the infill area and the building area A_b, assumed equal to 0.025 (Ricci 2010), n is the number of storeys, m is the medium storey mass normalized by the building area, equal to 0.8t/m^2 and λ is a coefficient for the evaluation of the first mode participant mass respect to the total mass of the MDOF (CEN 2004), in this example assumed equal to 1. Coefficient α and β, account, respectively, for the RC elements' contribution at the attainment of $V_{max,inf}$ (α=0.40) and for the residual strength contribution of the infills at the attainment of the plastic mechanism of the RC structure (β=0). Equation 5.4 are evaluated in the hypothesis of the attainment of a soft storey plastic mechanism (e.g., ground level) of the structures in analogy with the bare frame capacity evaluation; thus the inelastic displacement capacity of the infilled frame is equal to the corresponding value computed for the bare frame ($C_{d,infilled}$=$C_{d,bare}$=C_d). The ductility μ_s of the plastic branch at the attainment of $V_{max,inf}$ is equal to 1.5 (Ricci 2010).

Table 2. Simplified capacity and demand estimation for infilled RC frames.

N. of storeys	3					5				
Heigth, H [m]	9					15				
Capacity	$C_{s,\,max}$ [g]	$C_{s,\,min}$ [g]	T_1 [s]	$S^c_{a,\,FN}(T_1)$ [g]	$S^c_{a,\,FP}(T_1)$ [g]	$C_{s,\,max}$ [g]	$C_{s,\,min}$ [g]	T_1 [s]	$S^c_{a,\,FN}(T_1)$ [g]	$S^c_{a,\,FP}(T_1)$ [g]
PDS-1	0.58	0.16	0.12	0.71	0.93	0.38	0.16	0.21	0.68	1.19
NCSE-02	0.61	0.23	0.12	0.85	1.23	0.40	0.23	0.21	0.95	1.74
Demand	$S^d_{a,\,FN}(T_1)$ [g]			$S^d_{a,\,FP}(T_1)$ [g]		$S^d_{a,\,FN}(T_1)$ [g]			$S^d_{a,\,FP}(T_1)$ [g]	
	0.67			0.50		0.66			0.40	

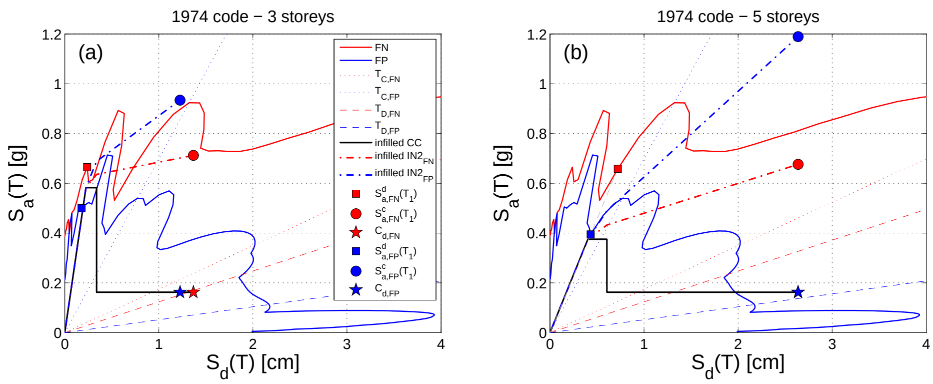

Figure 11. Simplified performance estimation of (a) 3 and (b) 5 storeys *infilled* RC buildings designed according to PDS-1 (1974) code.

R-μ-T relationship for infilled structures (Dolsek and Fajfar 2004a) and the simplified capacity curve defined above allow carrying out the IN2 curve (Dolsek and Fajfar 2004b), defining the capacity point ($S^c_a(T_1)$, C_d) in ADRS format for the infilled RC frame. T_1 is evaluated according to Ricci (2011b) for elastic period of infilled RC buildings amplified by a coefficient k_1, equal to 1.30, accounting for the decreasing of the lateral stiffness (period elongation) because of progressive cracking of infills along the height of the building (Ricci 2010), shear modulus G_w of infills was taken equal to 1500MPa (CS. LL.PP. 2009). In analogy with the bare case, in Table 2 are shown capacity and demand estimation carried out according to the methodology discussed above. In Fig. 11 is shown the performance estimation in the case of PDS-1 (1974) code for 3 and 5 storeys infilled RC buildings.

RC building stock has a bare seismic capacity much lower than the demand required by the 2011 Lorca event. In particular, approximate seismic capacity estimated for bare structure came out to be smaller up to a ratio of 2:1. However, it is worth noting that this capacity does not have into account the overstrength typical of these structures (minimum sizes of sections, longitudinal reinforcement percentage), and also that a low-ductile (q=2) mechanism was considered as no strength hierarchy prescriptions were provided by the codes, so according to this observation the bare capacity is a lower bound. On the other hand, such bare capacity can be assumed also to be an upper limit, as the adopted hypotheses do not contemplate the occurrence of brittle failures, which are not negligible since they appeared to be the most common type of damage.

When infill structural contribution is considered, buildings are able to resist to the demand of the mainshock event in most of the cases considered. Lateral strength increasing and period decreasing produced by infills lead to better performances of the buildings, notwithstanding the strength drop caused by brittle nature of the non structural elements. On the other hand, such a result represents a preliminary and approximate performance estimation. In fact, local interaction between infills and RC elements can lead to pre-emptive brittle failures, as observed during Lorca earthquake (see Fig. 7) and in other events in the Mediterranean area (e.g., Ricci et al. 2011a). More in general, evaluation of seismic performances of existing buildings cannot discard infill structural contribution; in fact in some cases (poor detailing or poor design) they can be the cause of a collapse mechanism (Verderame et al. 2011) and it is necessary to account for them for a proper assessment; in other cases they can increase the capacity as it was shown above and, as well, it can be important to compute their contribution.

6. CONCLUSIONS

An approximate explanation of the damage observed in RC structures during the 11th May 2011 Lorca earthquake was carried out, based on spectral, large scale, considerations, having in account the special characteristics of the ground motion and also the peculiarities of the building stock in the region as a consequence of seismic code provisions and common construction practice. Results lead to the following conclusions

- the event was much higher compared to typical code benchmarks, showing directivity effects in the normal direction to the fault, causing non-homogeneous effects in the near-source region;
- no proper seismic conception was present in most of Lorca RC buildings, because of common construction practice and because of Spanish seismic codes lacks;
- the lack of capacity design and interaction with masonry infill panels caused frequent brittle failures in columns.

However, despite the unfavourable conditions of both event and structural features, global collapse was rare; masonry infills have been generally proved to provide additional strength to RC structures.

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REFERENCES

Baker, J.W. (2007). Quantitative classification of near-fault ground motions using wavelet analysis. *Bulletin of the Seismological Society of America* **97**, 1486-1501.
Borzi B., Elnashai A. (2000). Refined force reduction factor for seismic design. *Engineering Structures* **22**, 1244-1260.
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. *IGN, UCM, UPM, IGME, AEIS*. http://www.ign.es/ign/resources/sismologia/Lorca.pdf

Calconsa XXI (2010, 2011). Photographic report of the building "Menorca", Avda. de las Fuerzas Armadas, 28-30-32, 30800 Lorca (Murcia), Spain. www.calconsa.com

CEN (2004) Eurocode 8: design of structures for earthquake resistance—Part 1: general rules, seismic actions and rules for buildings. European Standard EN 1998-1:2003- Comité Européen de Normaliation, Brussels.

CEN (2005) Eurocode 8: design of structures for earthquake resistance—Part 3: assessment and retrofitting of buildings, European Standard EN 1998-1:2005- Comité Européen de Normaliation, Brussels.

Chopra, A.K. (2007). Dynamics of the Structures, Prentice Hall, Upper Saddle River, New Yersey, 3rd edition.

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

Dolšek, M., Fajfar, P. (2001). Soft storey effects in uniformly infilled reinforced concrete frames. *Journal of Earthquake Engineering* **5**(1), 1-12.

Dolšek, M., Fajfar, P. (2004a). IN2 – A simple alternative for IDA. *Earthquake Engineering and Structural Dynamics* **33**,1395-1416.

Dolšek, M., Fajfar, P. (2004b). Inelastic spectra for infilled reinforced concrete frames. *Proceedings of the 13th World Conference on Earthquake Engineering*, August 1-6, Vancouver, B.C., Canada.

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

EHE-08 (2008). Instrucción de hormigón estructural, Comisión permanente del hormigón estructural, Ministerio de Fomento, Madrid.

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. Prenormative research in support of Eurocode 8.

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.

Gaspar-Escribano, J.M., Benito, B., García-Mayordomo, J. (2008). Hazard-consistent response spectra in the Region of Murcia (Southeast Spain): comparison to earthquake-resistant provisions. *Bulletin of earthquake engineering* **6**,179-196.

IGN (2011): Seismic catalog: database. Service of seismic information.

Instituto Nacional de Estadística, INE (2001). Service database.

Lam, N., Wilson, J., Adrian Chandler, A., Hutchinson, G., (2000). Response spectrum modelling for rock sites in low and moderate seismicity regions combining velocity, displacement and acceleration predictions *Earthquake engineering and structural dynamics* **29**,1491-1525.

Mezcua, J., Rueda, J., García, R.M. (2011). A new probabilistic seismic hazard study of Spain. *Natural Hazards* **59**,1087-1108

MV 101 (1963). Acciones en la edificación, BOE Nº 35, Madrid.

NCSE-02 (2002). Norma de construcción sismorresistente, BOE Nº 244, Madrid.

NCSE-94 (1995). Norma de construcción sismorresistente, BOE Nº 33, Madrid.

PDS-1 Parte A (1974). Norma sismorresistente, BOE Nº 279, Madrid.

PGS-1 Parte A (1968). Norma sismorresistente, BOE Nº 30, Madrid.

Regalado, F., Lloret, V. (2011). Análisis y reflexiones sobre los terremotos del 11 de mayo del 2011 acontecidos en Lorca (sugerencias para el futuro). FR&A.

Ricci, P. (2010). Seismic vulnerability of existing RC buildings. Ph.D. Thesis. University of Naples Federico II, Naples, Italy.

Ricci, P., De Luca, F., Verderame, G.M. (2011a). 6th April 2009 L'Aquila earthquake, Italy – Reinforced concrete building performance. *Bulletin of Earthquake Engineering* **9(1)**,285-305.

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

Rossetto, T., Peiris, N. (2009). Observations of damage due to the Kashmir earthquake of October 8, 2005 and study of current seismic provisions for buildings in Pakistan. *Bulletin of Earthquake Engineering*, **7**(3): 681–699.

Sezen, H., Moehle, J.P. (2004). Shear strength model for lightly reinforced concrete columns. ASCE Journal of Structural Engineering **130**(11), 1692-1703.

Verderame GM, De Luca F, Ricci P, Manfredi G. (2011). Preliminary analysis of a soft-storey mechanism after the 2009 L'Aquila earthquake. *Earthquake Engineering and Structural Dynamics*, **40**(8), 925-944.

Vidal, F., Alguacil, G., Feriche, M., Aranda, C., Morales, J., Stich, D., Pérez Muelas, J., Benito, J., López, J.M. (2011). El terremoto de Lorca: Mayo 2011. Causas del impacto y primeras medidas. Análisis preliminar. Informe IAGPDS, Universidad de Granada. http://www.ugr.es/~iag/lorca/Impacto_Lorca.pdf

VV.AA. (2006): *Riesgo Sísmico de la Comunidad Autónoma de la Región de Murcia (RISMUR)*. 6 Vols. Instituto Geográfico Nacional y Dirección General de Protección Civil CARM.