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Chapter 10
L'Aquila Earthquake: A Wake-Up Call for European Research and Codes

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Abstract From the L'Aquila 2009 earthquake three issues, among others, strongly emerged to be addressed for the engineered structures, at least in Europe. They are related to near-source effects, non-structural damage, and reparability. Although they are well known since quite long time, still regulations seem giving little, if any, practice-ready tools to account for them. In the chapter, evidences from the event and scientific needs are briefly reviewed and discussed. The modest aim of the paper is to stimulate debate and research in the light of next generation of seismic codes.

Keywords Near-source • Directivity • Pulse-like records • Seismic hazard • Inelastic displacement ratio • Response spectrum • Non-structural elements • Infills • Reinforced concrete • Seismic assessment • Seismic design • Capacity • Codes • Damage • Collapse • Reparability • Substandard structures • Shear failure • Soft-storey • Residual drift • Non-linear structural analysis

10.1 Introduction

The April 6, 2009 L'Aquila earthquake (M_W 6.2) caused about three hundreds of fatalities, more than a thousand injuries, and extensive and severe damage to buildings and other structures. About 66,000 residents were temporarily evacuated, and more than 25,000 were medium-term homeless.

The area of interest is known to be seismically active since a long time. Several events comparable in magnitude to this last earthquake, are reported by the

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national seismic catalogue. The main documented events, considering an estimated magnitude larger than 6.5, date to 1315, 1349, 1461, 1703 and 1915. In fact, the first modern-era seismic classification of L'Aquila refers to 1915, after the catastrophic Avezzano earthquake. The subsequent estimations of seismic hazard lead to the current value of expected peak ground acceleration, or PGA, (for a return period of 475 years) equal to about 0.26 g on rock. In fact, it is one of the largest seismic hazard sites according to the national hazard map (http://esse1.mi.ingv.it/).

Analysis of buildings stock in the city of L'Aquila shows a percentage of reinforced concrete buildings equal to 24 %, while the masonry buildings are 68 %; a percentage of 8 % refers to buildings of different typology. For what concerns the age of construction, 55 % of buildings was built after 1945. Therefore, the most of existing buildings was built using some seismic provisions; nevertheless with non-modern seismic standards. Moreover, seismic demand during the 2009 earthquake was, locally, much larger than the design one. This seems also due to near-source directivity effects. Generally, the seismic performance of buildings stock in L'Aquila was considered unsatisfactory. In fact, in the 6 months after the mainshock, the national department of civil protection organized a global survey of all the buildings in the area affected by earthquake. About 80,000 field surveys were performed by specialized teams. The results show that about 20,000 building suffered of large structural damage, while about 10,000 buildings suffered of light structural damage and/or non-structural damage. Lacks and deficiencies of seismic design are believed to be responsible for such a bad performance, which is going to have a large reconstruction cost for the country.

Starting from L'Aquila experiences, some remarks on possible improvements of next generation of codes are discussed in the following. Of the many facets of seismic risk, which a number of researchers studied after the earthquake, by far the best documented event in Italy, three are those briefly discussed in this chapter: (i) directivity-related near-source effects of engineering interest and their predictability; (ii) seismic behaviour and structural dynamics' influence of infills in reinforced concrete structures; (iii) reparability and the possibility to explicitly include this limit-state in design.

The reader may argue these are well known earthquake engineering topics since quite some time, yet European codes, at least, are somewhat lacking in their respect, while level of scientific knowledge is such they may be considered in the next generation of seismic regulations.

10.2 Near-Source Pulse-Like Engineering Issues

Near-source (NS), or directivity, effects in ground motion depend on the relative position of the site with respect to the fault rupture, and typically appear by means of large velocity pulses concentrating energy in the starting phase of the fault-normal, or FN (i.e., normal to the rupture's strike), component. This results in waveforms different from *ordinary* ground motion recorded in the far field, or in geometrical

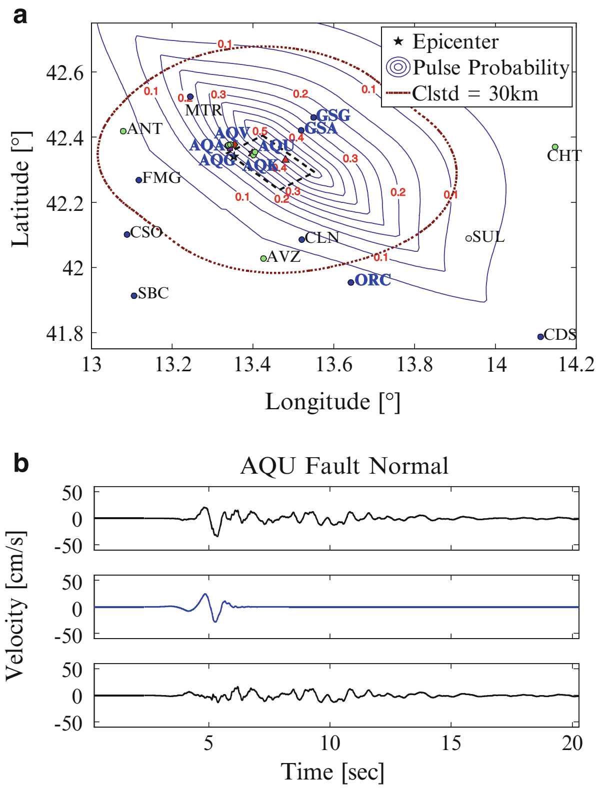

Fig. 10.1 Near-source stations in L'Aquila 2009 earthquake and pulse occurrence probability contours (according to the model of Iervolino and Cornell 2008) together with actual identified (*in bold*) velocity pulses (**a**); impulsive signal of AQU station in the same earthquake; from *top to bottom*: velocity time history, extracted pulse and residual velocity (**b**)

conditions not favorable with respect to directivity. Near-source pulse-like traces were found in the records of L'Aquila earthquake (Chioccarelli and Iervolino 2010).

In Fig. 10.1a the seismic stations, which have recorded the event close to the source, are shown and superimposed to a probability model for occurrence of pulse-like records (Iervolino and Cornell 2008). In bold there are the stations where the records, in which pulses supposed to be originated by directivity, were found; consistency may be observed. In Fig. 10.1b, for one accelerometric station (AQU), the original velocity signal is shown together with the extracted pulse and the residual ground motion once the pulse is removed (according to the algorithm in Baker 2007).

In Chioccarelli and Iervolino (2010), analyzing the NGA database (http://peer.berkeley.edu/nga/), it was found that the three main characteristics of pulse-like records of earthquake engineering interest are:

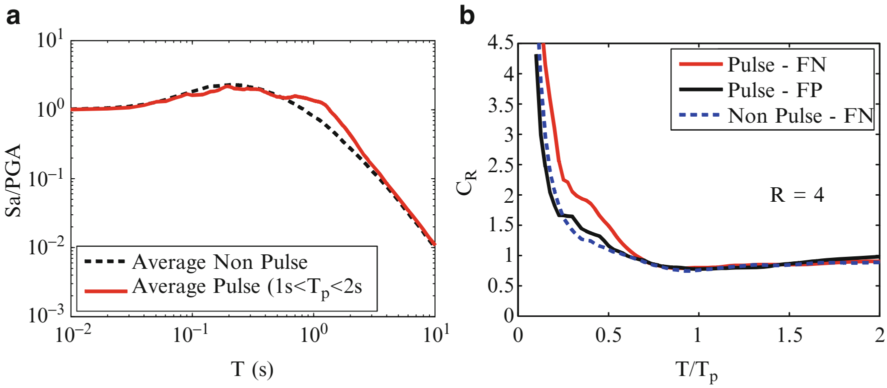

Fig. 10.2 Elastic 5 % damped spectra for FN pulse-like with 1 s < T_P < 2 s and ordinary records (**a**); empirical C_R for FN pulse-like records, for their fault-parallel (FP) components, and for ordinary records, for a strength reduction factor R = 4 (**b**) (Adapted from Iervolino et al. 2012)

1. pulse-like signals are characterized by fault normal records generally stronger than both fault parallel components and non-pulse-like ground motions;
2. fault-normal pulse-like records are characterized by a non-standard spectral shape with an increment of spectral ordinates in a range around the pulse period (Tp), Fig. 10.2a;
3. inelastic-to-elastic seismic spectral displacement ratio (C_R) for pulse-like records can be 20–70 % higher than that of ordinary motions depending on the non-linearity level; such increments are concentrated in a range of period between 30 and 50 % of pulse period of each record, Fig. 10.2b.

These points show that near-source directivity is of interest to earthquake resistant design, which has to be adjusted for near-source because: (i) traditional hazard assessment may be unable to predict (1) and (2), that is, the elastic peculiar features of pulse-like records; (ii) point (3) shows that current static design, based on *equal displacement rule*, may fail when pulse-like records are concerned; (iii) it is not implicit in current design practice that safety margins are consistent between ordinary and pulse-like records.

Regarding (i): in near-source conditions the probabilistic seismic hazard analysis (NS-PSHA), expressed in terms of mean annual frequency of exceeding a spectral acceleration (Sa) level, or λ_{Sa}, is a linear combination of two terms, which account for the occurrence or the absence of the pulse, $\lambda_{Sa,NoPulse}$ and $\lambda_{Sa,Pulse}$, weighted by the pulse occurrence probability, Eq. (10.1). Moreover, NS-PSHA requires dealing with two more tasks, which are not faced in traditional hazard analysis: pulse period prediction; and pulse amplitude prediction.

$$\lambda_{Sa}(x) = \lambda_{Sa,NoPulse}(x) + \lambda_{Sa,Pulse}(x) \qquad (10.1)$$

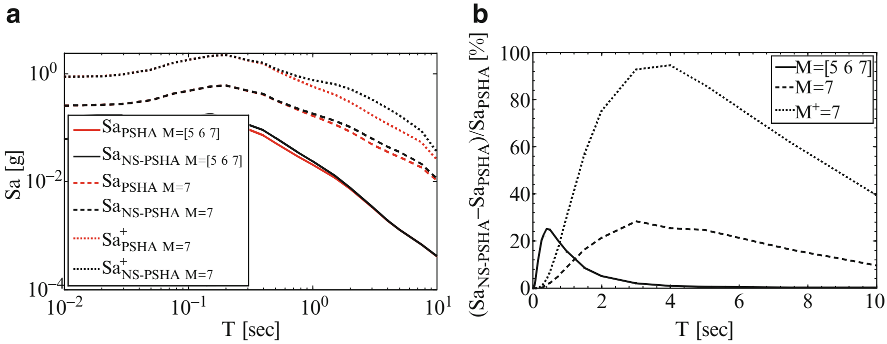

Fig. 10.3 475 years UHSs with modified and classical PSHA (**a**); increments due to directivity effects (**b**) (Adapted from Chioccarelli and Iervolino 2013)

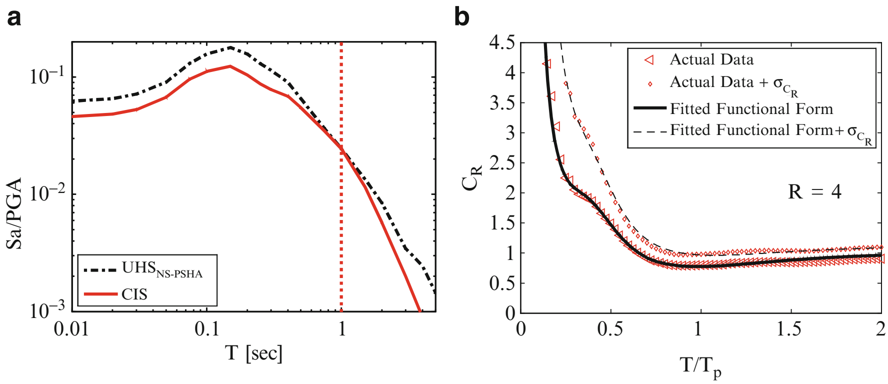

Fig. 10.4 (**a**) Near-source possible design scenarios (Chioccarelli and Iervolino 2013); (**b**) inelastic displacement ratio functional form for pulse-like records fitted to actual data (Iervolino et al. 2013)

As an example, in Fig. 10.3a near-source uniform hazard spectra (UHS, 10 % in 50 years), are compared to UHS from traditional PSHA for different cases of magnitude distribution and source-to-site-geometry for a strike-slip fault (Chioccarelli and Iervolino 2013). In Fig. 10.3b increments of NS-PSHA with respect to PSHA are given. It is to observe that these may be significant. It also appears the UHS is only able to capture the NS amplitude, not the peculiar spectral shape of Fig. 10.2a. Therefore, it seems necessary to find alternate solutions to get design seismic actions.

One possible approach is that proposed in Chioccarelli and Iervolino (2013) and named *close-impulsive spectrum* (CIS), conceptually consistent with the conditional mean spectrum Baker (2011). CIS is derived from disaggregation of NS-PSHA in terms of magnitude, distance from the source, and pulse period. In Fig. 10.4a, starting from the hazard for a 0.5 s Sa, the shape of CIS is reported for a case in which the design scenario is 0.7 s, 5, and 10 km, in terms of Tp, magnitude, and source-to-site distance, respectively. The figure also shows the UHS for the same example, to appreciate differences and how CIS reproduces a ‘bump’ around the design pulse period.

Regarding point (ii), it was found in Iervolino et al. (2012) that the inelastic displacement ratio of near-source pulse like records, requires a specific functional form to fit observed data as a function of T/Tp, which is substantially different from ordinary records. In fact, the functional form in Eq. (10.2), which consists of adding two opposite bumps in two different spectral regions to the traditional hyperbolic format of C_R in codes (e.g., FEMA 2005), may be suitable. In Fig 10.4b this functional form is plotted (also plus one standard deviation) against data of Fig 10.1b once the θ coefficients are fitted.

$$C_R = 1 + \theta_1 \cdot \left(T_p/T\right)^2 \cdot (R-1) + \theta_2 \cdot \left(T_p/T\right) \cdot \exp\left\{\theta_3 \cdot \left[\ln\left(T/T_p - 0.08\right)\right]^2\right\} + \theta_4 \cdot \left(T_p/T\right) \cdot \exp\left\{\theta_5 \cdot \left[\ln\left(T/T_p + 0.5 + 0.02 \cdot R\right)\right]^2\right\} \tag{10.2}$$

10.3 Non-structural Elements and Their Influence on Structural Assessment/Design

Infill walls are usually employed in reinforced concrete (RC) buildings for partition use and for thermal/acoustic insulation. Hence, they are considered as non-structural elements; nevertheless, post-earthquake damage observation, experimental and numerical research, showed that their influence on seismic behaviour of RC buildings can be not negligible at all.

Modern seismic codes (e.g., FEMA 2000; CEN 2004; DM 2008) prescribe to account for the possible influence of infills on seismic behaviour of RC frames, both at local and global level. In particular, according to Eurocode 8 (CEN 2004), if walls take at least 50 % of the base shear from a linear analysis, the interaction of the structure with the masonry infills may be neglected. This may be taken to imply that it is allowed then to disregard the infills in the structural model. However, this is not always a safe assumption. An asymmetric layout of the infills in plan may cause torsional response to the translational horizontal components of the seismic action; so, according to Eurocode 8 part 1, infills with strongly asymmetric or irregular layout in plan should be included in a 3D structural model and a sensitivity analysis of the effect of the stiffness and position of the infills should carried out. Even though the awareness about this issue in earthquake engineering is not very recent, it is likely to state that practically no existing RC building was designed accounting for the presence of these elements.

This latter observation becomes a key issue in countries, such as Italy, in which most of the building stock was realized before 1990s. It has to be noted that not even limit state approach was provided in the Italian regulations until 1996,

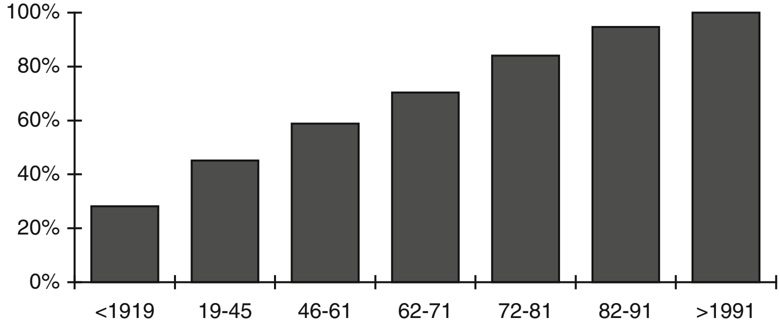

Fig. 10.5 2001 census ISTAT data for L'Aquila. The construction age intervals on the abscissa identify the percentage of buildings to add to the preceding bar to get the bar they correspond to, that is, to obtain the cumulative distribution (Ricci et al. 2011a)

Fig. 10.6 (**a**) Shear failure of column adjacent to partial infilling panels; (**b**) diagonal cracking failure in concrete joint panel (Ricci et al. 2011a)

see Ricci et al. (2011a) for details. Figure 10.5 shows the empirical cumulative distribution of the age of construction of L'Aquila buildings that can be approximately considered representative of most of the Italian region in terms of urban development.

Observation of damage on RC structures after L'Aquila event emphasized the importance of taking into account the contribution of non-structural elements such as infills. Most of the observed damage was localized in infill panels, and their stiffness and strength contribution in some cases preserved the RC buildings from structural damage. On the other hand, it is to note that infill irregular distribution in plan and elevation can be addressed as one of the main causes of the structural collapses observed during in-field campaigns, together with brittle failure mode of columns and beam-column joints (see Fig. 10.6).

The lack of capacity design and shear-flexure hierarchy can lead to brittle failures in primary elements (De Luca and Verderame 2013), and interaction with masonry infills can favor the occurrence of them. Effect of infills on the whole structural behavior depends on different factors that are briefly recalled in the following.

First, the evaluated fundamental period of the infilled structure (Ricci et al. 2011b) changes significantly if compared to the period of the bare frame model, since the structure is significantly stiffer. Furthermore, infill presence can change regularity characteristics of the structure in plan and in elevation and consequently can affect the mode of vibration of the building. Second, infills are characterized by a brittle behavior and the high contribution in strength they provide to a RC building suddenly decreases for low values of drift. On the other hand, it is worth to note that infill impact on the structural performances of a building becomes a critical issue when RC primary elements are designed according to obsolete criteria and the structure is characterized by an insufficient global and local ductility. The latter can be observed when seismic performances of contemporary and existing infilled frame structures are compared (Dolsek and Fajfar 2005). If infill distribution is irregular in plan or elevation, their contribution introduces a source of irregularity (e.g. *Pilotis effect*) and the possibility to register a soft-storey mechanism is dramatically increased, especially when no capacity design criterion has been employed in the design of the bare frame.

Mechanical properties of the infills represent another critical factor that can vary the effects on the performances of the whole structure, since they are considered non-structural elements and their properties, not systematically checked, can vary significantly because of the local building practice. Regarding this latter issue, it is important to stress the relative weight that infills have with respect to the mechanical properties of the bare frame. Because of all the variables considered (seismic intensity level, old or contemporary design approach, distribution and mechanical properties of the infills) it is tough to say if structural contribution of these “non-structural” elements increases or decreases the overall seismic capacity of the building (Ricci et al. 2012).

In (Verderame et al. 2011) one of the few collapsed buildings after L’Aquila earthquake is assumed as case study; the building collapsed as a result of a soft storey mechanism at the first level (see Fig. 10.7). Observed damage points to collapse as a result of a brittle failure mechanism. Given the likely scenario collapse inferred by observed damage, an analytical model of the building was built taking into account nonlinear behavior of the infills; local interaction with columns was also considered by means of a three strut macro-model. Two parametric hypotheses based on Italian code prescriptions were assumed for infill mechanical properties. Time history analyses were carried out assuming as seismic input the three components of the real registered signals during the mainshock in the vicinity of the case study structure. Given the brittle failure highlighted by damage, beside capacity models suggested by codes, other shear failure mechanisms (*sliding shear failure*) not typical for columns were considered.

Numerical results seem to confirm the collapse scenario inferred by damage observation; the lack of proper structural and executive details was addressed as

Fig. 10.7 (**a**) Pre-event view of case study building, placed in Pettino (L'Aquila) (from Virtual Earth); (**b**). Collapsed building (Verderame et al. 2011)

the main cause that made a critical issue the local interaction between columns and infills, other than the strong vertical component registered.

The case-study structure, even if characterized by structural peculiarities, represents in itself a lesson for future code provisions and, above all, it emphasizes an effect typically disregarded in conventional assessment procedures.

10.4 Is a Reparability Limit State Needed and Feasible?

One of the most controversial problems in the aftermath of damaging earthquakes is the lack of agreed and transparent policies for acceptable levels of safety, as well as of advanced technical standards for repair and/or strengthening of damaged buildings. In fact, the technical difficulties for the assessment of the safety loss of damaged buildings and for the choice of the most appropriate method for repair and/or strengthening of damaged elements, avoided the development of sound and agreed re-occupancy criteria meeting engineering consensus on the different aspects to be considered.

The criteria for reconstruction funds assignments after L'Aquila earthquake distinguish between buildings classified as B/C (generally having slight damages) by the damage survey forms used by the Italian Civil Protection (Baggio et al. 2007), with respect to those classified as E (heavy structural damage). However, if the usual tagging procedures, based on an expert assessment of damage level and extent by a team of experienced practitioners, are deemed acceptable in an emergency phase in order to establish building usability, they cannot be considered as the only tool for choosing intervention categories and assigning grantable funds or insurance refunds for damaged buildings. A proper approach should rely on a performance based policy framework, as suggested in FEMA 308 (FEMA 1998b), where building's performance index and performance loss due to earthquake damage are the parameters considered to drive decisions among the alternatives of simply accepting the building as it is (insignificant damages), repairing or upgrading it.

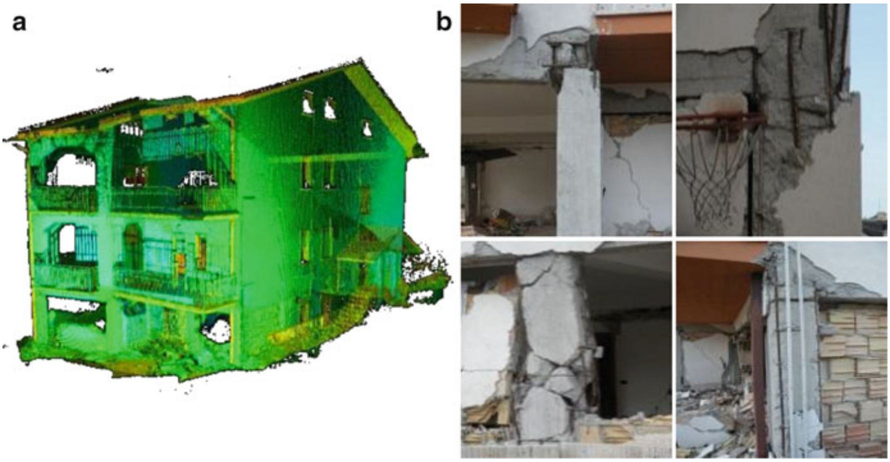

Fig. 10.8 (**a**) Dense digital elevation model of the building in Pianola (AQ) obtained with laser-scanner 3D. (**b**) Examples of damages on the elements at the first storey (Polese et al. 2011)

Only partly in line with this approach, post L'Aquila regulations implicitly associate E buildings with medium-high performance loss, and the repairing and upgrading of buildings becomes mandatory when the performance index (ratio of the PGA corresponding to building collapse versus the design PGA in the nominal building life), evaluated for the intact building, is lower than 60 %. Moreover, with the purpose of avoiding length and costly investigations and analyses for heavily damaged buildings, an evaluation policy was introduced considering the amount of residual drifts as discriminative parameter for reparability. In particular, the code regulation OPCM 3881 (2010) establishes that it is possible to avoid demonstration of the economic convenience of demolishing and re-building if there are permanent drifts $\geq$1.5 % for at least 50 % of the columns at the same storey. Such value is somewhat greater than current literature values; for example, in ATC 58-1 (2011) for a residual drift of 1 % it is hypothesized that major structural realignment is required to restore safety margin for lateral stability, and the required realignment and repair of the structure may not be economically and practically feasible (i.e., the structure might be at total economic loss). However, considering the typical shear sliding mechanism at the beam-column interface of existing under-designed Italian buildings, due to masonry infill interactions with columns (Verderame et al. 2011), residual drift is often related to a local damage of some (typically external) columns, and larger permanent drifts may be expected, as reported in Polese et al. (2011). In fact, maximum residual drifts measured for two buildings in L'Aquila (Fig. 10.8 and Table 10.1), that were severely damaged at the first storey, is well beyond 1.5 % in both cases. However, the real measures for these two buildings show that it is very difficult to comply with the requirements of OPCM 3881 (2010) (at least 50 % of columns in a storey with residual drifts > 1.5 %) even for very severely damaged buildings.

Table 10.1 Permanent drifts (θ_A and θ_B on orthogonal sides of columns) evaluated at the first storey for two damaged buildings in Pettino (AQ) and in Pianola (AQ)

Pettino (AQ)			Pianola (AQ)		
Col. ID	θ_B (%)	θ_A (%)	Col. ID	θ_B (%)	θ_A (%)
01	+0.74	–	01	+1.04	−0.56
02	+0.13	–	05	+0.25	+0.64
04	+0.96	–	13	−1.79	+0.60
06	+0.51	–	19	+0.77	+0.35
07	+2.52	+0.40	17	+1.89	−0.54
32	−0.93	+0.19	14	+0.29	0.00

Only perimeter columns, where it was possible to measure residual deformations, are indicated (Polese et al. 2011)

$$REC_{Sa} = C_b \cdot \mu_{cap} \quad \text{for } T_{eq} \geq T_c$$

$$REC_{Sa} = C_b \cdot (\mu_{cap} - 1) \cdot \frac{T_{eq}}{T_c} + 1 \quad \text{for } T_{eq} < T_c$$

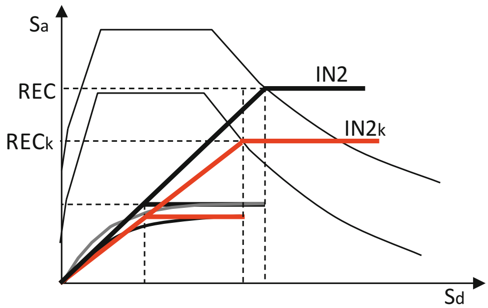

Fig. 10.9 Application of IN2 method for determining REsidual Capacity in the intact (REC) and damaged state (REC_k) (Polese et al. 2013)

Obviously, there is a need to further investigate on the relationship between residual drifts, damages and performance loss in a performance-based policy framework. If varied vulnerability is to be considered in a consistent quantitative assessment framework, analytical modeling of building performance loss, accounting for building damage and residual drifts, is preferable. Building performance in the pre-event, damaged and eventually restored/upgraded state may be investigated with nonlinear static analysis considering proper variation of element's force-deformation relationships, as outlined in FEMA (1998a).

The seismic behavior of damaged buildings, and the relative seismic safety, may be adequately represented by their seismic capacity modified due to damage, the so called REsidual Capacity (REC). In the framework of a mechanical based assessment of seismic vulnerability, REC may be evaluated based on pushover curves obtained for the structure in different (initial) damage state configurations and accounting for possible residual drifts. In particular, residual capacity REC_{Sa} is defined, for each global damage state D_i, as the minimum spectral acceleration (at the period T_{eq} of the equivalent single degree of freedom system) corresponding to building collapse, and can be determined, among other approaches, with the IN2 method (Dolsek and Fajfar 2004), see Fig. 10.9.

A preliminary application Polese et al. (2013) shows that the adoption of nonlinear static analyses for damaged building, with explicit consideration of the damage and residual drifts in the main resisting elements, allows a consistent assessment of the building safety factor and performance loss with respect to intact structure. In this application, suitable modification factors for moment rotation plastic hinges of the columns have been calibrated based on a number of cyclic tests performed by the authors on non-conforming columns. However, models to predict the modification factors based on a wider number of experimental tests available in the literature (on columns representative of existing RC buildings of European Mediterranean regions) are currently under investigation (Di Ludovico et al. 2012).

Referring to the case study, an interesting conclusion is that while for structures that have been slightly or moderately damaged the ultimate deformation capacity does not significantly change with respect to the undamaged structure, at the same time these structures are more deformable (having higher T_{eq}) and have a lower ductility capacity, μ_{cap}, with a consequent lower residual capacity. This circumstance is reflected in the increasing performance loss, that reaches values nearly of 10 % for slight damage, and up to 20 % for a structure that has reached a moderate damage state due to an hypothetical mainshock.

A key issue for a proper assessment of performance loss, to be further developed, is the experiment-based characterization of typical existing RC columns performance in the intact and damaged states, distinguishing their possible behavior modes (e.g. flexural, flexure/shear, sliding shear etc.) and significant damage levels. Moreover, in order to verify the effect of alternative retrofit solutions, also the influence of local retrofit interventions on such elements should be evaluated.

10.5 Conclusion

A few research opportunities and codes' needs were highlighted starting from the evidence of L'Aquila 2009 earthquake. In fact, analysis of records and damages to the built environment shows that the earthquake engineering research appears close to be capable of a practice-ready implementation on some issues, while other still require investigations, experimental tests, and competency building.

Three topics were found especially significant and briefly reviewed. In particular, near-source pulse-like effects, infills of reinforced concrete structures, and reparability, were analysed with respect to design and assessment.

On the near-source seismic demand side, it appears that there is a need for accounting for the peculiar pulse-like features in the elastic (i.e., seismic hazard) and inelastic range; state-of-the-art tools are available, while current provisions may be unable to capture those.

The influence of infills on the structural behaviour of engineering structures has to be considered in the seismic design for two main reasons: (1) infills can dramatically change the seismic behaviour determining unpredictable collapse modes with respect to the bare structure; (2) in a consequence-based framework, a reduction of infills damage can increase the immediate occupancy conditions and reduce the economic losses due to non-structural damage.

Finally, the definition of a reparability limit state determines potential improvements in seismic response; e.g., the definition of a clear condition of repair/upgrade convenience in the post-event situation, and new criteria in the choice of alternative retrofit solutions in a multi-criteria decision making method approach.

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