ON THE SEISMIC RELIABILITY OF CODE-COMPLIANT REINFORCED CONCRETE BUILDINGS IN ITALY

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Introduction. Current building codes require structural engineers to design new earthquake-resistant structures able to offer an adequate horizontal capacity with respect to a set of predefined performance levels. However, the sized code-compliant buildings, once assessed, may not show a controlled probability of failure, even if the design ground-shaking intensities are probabilistically defined, as in many codes currently in force worldwide (Bradley 2011; Faleschini et al. 2019). In other terms, the use of semi-probabilistic approaches for the seismic design of new buildings is not able to explicitly control the resulting seismic reliability. Code compliance and seismic performance are in fact strictly coupled by a strong underlying relationship, and a modern code must be able to indicate simple and effective prescriptions that can implicitly be reflected in the fulfillment of target performance levels defined a priori. For these reasons, this paper focuses on a more in-depth seismic reliability assessment of code-compliant RC bare and masonry-infilled archetypes to analyze the underlying relationship between seismic design accelerations and resulting performance in terms of seismic failure rates, and to compute the Italian seismic reliability maps of such building types. Different configurations are considered in terms of the number of stories (i.e. 3-, 6- and 9- stories) as well as assumed design ductility classes (e.g. high (DCH) and medium (DCM) ductility class). Buildings are automatically designed and later assessed with the use of a prototype software described in detail in Zanini and Feltrin (2021).

Prototype seismic design and assessment software. This section presents a brief description of the prototype seismic design-assessment software used for sizing and subsequently assess the archetype structures.

First, the user has to specify materials to be considered, main geometrical features of the archetype RC frame, class of use to derive design accidental loads, and desirable ductility class (i.e. select the behavior factor q). The software then designs the beams' sections and reinforcements with an iterative loop, then sizes the columns to fulfill a correct capacity design (i.e. ensuring strong column-weak beam criterion). Once designed, the code-compliant frame is assessed with respect to relevant performance levels to quantify related fragility curves. To do this, the capacity curve is first derived by means of a pushover analysis on a non-linear model that uses a lumped plasticity modeling technique with plastic hinges calibrated via a fiber-cross section discretization modeling strategy with the joint adoption of suitable non-linear stress-strain material laws. Masonry infills are accounted only for the infilled configurations to capture the increment in the stiffness on the overall seismic response. Given the large computational burden, frames are idealized as Single Degree of Freedom Systems (SDOFs), with hysteretic behavior calibrated based on the idealized tri-linear capacity curve. NLTHAs are later performed to obtain samples of the non-linear seismic behavior to be post-processed with the Cloud Analysis method (Cornell et al. 2002) to obtain fragility curves.

In detail, SDOF systems are subject to a limited set of n unscaled ground motion records and the fragility curve takes origin from the sample of n ground motion intensities and the corresponding sample of structural responses quantified by a proper engineering demand parameter edp (i.e., a metric that can be used to estimate the structural damage), with the following expression:

$$P[f|im] = P\left[EDP > \overline{edp}\middle|im\right] = 1 - P\left[EDP \leq \overline{edp}\middle|im\right] = 1 - \Phi\left[\frac{\ln(\overline{edp}) - \ln(edp)}{\sigma\beta}\right] \quad (1)$$

where $\overline{edp}$ represents the specific undesired threshold level of the edp, and σ is the demand standard deviation. Hence, under the hypothesis that the occurrence of earthquakes at the construction site is a Homogenous Poisson Process (HPP), the seismic failure rate λ_f is computed as:

$$\lambda_f = \int_{im} P[f|im] \cdot |d\lambda_{im}| \quad (2)$$

where λ_{im} is the seismic hazard curve representative of the seismicity at the site of interest commonly computed via a Probabilistic Seismic Hazard Analysis (PSHA, Cornell 1968 and McGuire 1995), and im is a relevant intensity measure.

Case studies. This paper investigates the seismic reliability of code-compliant residential buildings with an RC frame-resisting scheme. Fig. 1 illustrates the main features of the different configurations analyzed, which fulfill plan and elevation regularity criteria, and are characterized by three increasing elevations, i.e. 3-, 6- and 9-stories archetypes all with a constant inter-story height equal to 3 m. All configurations have a rectangular plan with 5 x 3 bays of 5m span each.

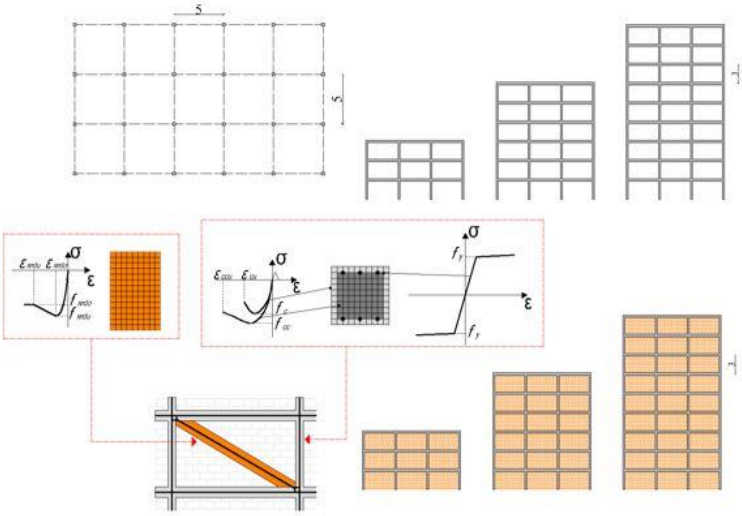

Fig. 1 - *Main geometrical and material characteristics of the analyzed structural archetypes*.

Beams and columns were designed considering a C25/30 according to [NTC] with characteristic compressive strength f_{ck} equal to 25 MPa, and a reinforcing steel B450C with characteristic yielding tensile strength f_{yk} equal to 450 MPa. To account for the non-linear material behavior, suitable models are adopted: in particular, Mander et al. (1998) model *Concrete04* and Menegotto and Pinto (1973) model *Steel02* materials for core/cover concrete and reinforcement rebars are used, whereas single-strut truss elements with a non-linear behavior characterized by Di Trapani et al. (2018) model are adopted to capture the stiffening effect caused by masonry infills. Masonry compressive strength f_m and the elastic modulus E_m along the two orthogonal directions are assumed equal to 2.4 MPa and 4408 MPa for the direction, and about to 7.28 MPa and 7400 MPa for the other one. Masonry infills are characterized by a thickness of 25 cm and distributed over the entire external perimeter of the buildings, whereas the contribution of the staircase to the stiffness of the building was neglected. Regarding the loading actions, 5.5 kN/m^2 and 0.5 kN/m^2 are considered as the dead and live loads for the roof, and a 6.5 kN/m^2 dead load and a 2 kN/m^2 live load are considered for the remaining floors. Both high ductility class (DCH) and medium ductility class (DCM) are considered, thus leading to a total of 12 different archetypes resulting from the combination of the different number of stories, ductility class, and presence/absence of masonry infills.

Results. The seismic hazard curves computed for each Italian municipality with reference to its main soil class are coupled with the appropriate fragility curves representative of the code-compliant archetype that a designer may have sized in that location to get the seismic failure rates associated with a code-compliant design, and thus obtain the respective seismic reliability maps for bare and infilled code-compliant RC frames.

The following relevant damage state (DS) are defined:

- Low Damage (ds_1), corresponding to the achievement of the yielding point in the SDOF's behavior curve;
- Near Collapse (ds_2), placed at the beginning of the backbone's descending branch;
- Collapse (ds_3), identified when base shear is approximately equal to the 80% of the maximum shear capacity.

The results show how infilled configurations are generally characterized by significantly higher seismic failure rates than bare frames, and such difference is magnified in low-seismic hazard regions. The choice of DCM or DCH design ductility class leads to similar results in terms of seismic safety: however, DCM seems to imply the design of slightly safer code-compliant buildings, although a designer is led to think otherwise, i.e. that DCH may allow safer designs than DCM. Results show also how 6- and 9-stories archetypes are characterized by similar λ_f intervals, with values higher than the 3-stories configurations.

As sake of example, Fig. 2 shows results for ds_2. Here for bare RC frames, 6-stories layouts display the worst performance, with the worst seismic failure rates equal to $4.41 \cdot 10^{-4}$ for DCH designs.

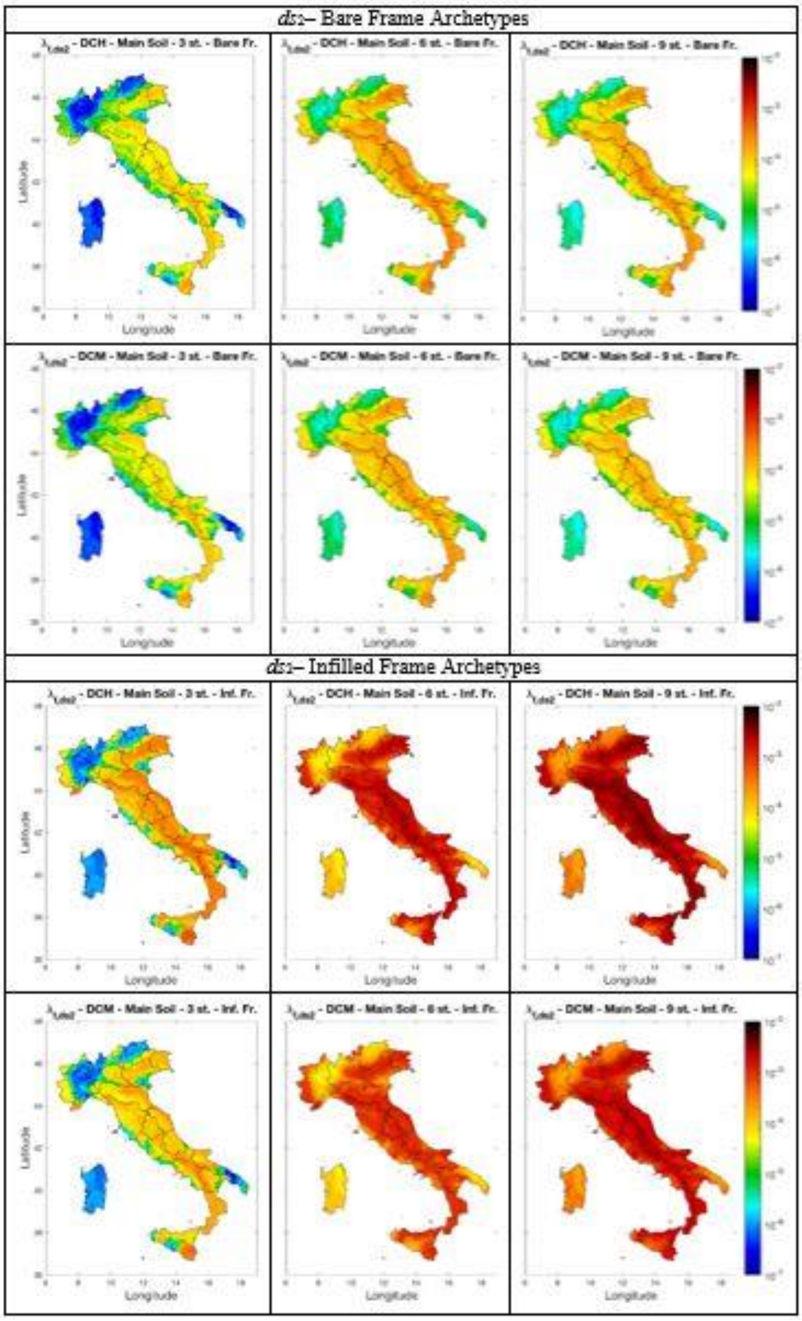

Fig. 2 - *Ds_2 seismic reliability maps for bare and infilled code-compliant RC frames.*

Overall, it is observed how λ_f values are between $1.13{\cdot}10^{-7}$ and $4.41{\cdot}10^{-4}$ for the entire subset of bare frame archetypes. As regards the companion infilled configurations, more severe performances are observed, with worst effects for the 9-stories layouts, whereas in low seismicity regions the presence of infills appears to be beneficial for 3-stories archetypes, with λ_f reaching minimum values around 5.68·10-8. In summary, all the analyzed infilled configurations have as overall λ_f interval values between $5.68{\cdot}10^{-8}$ and $4.24{\cdot}10^{-3}$.
Lastly, Fig. 3 illustrates a graphical comparison between λ_f intervals derived for the abovementioned design layouts.

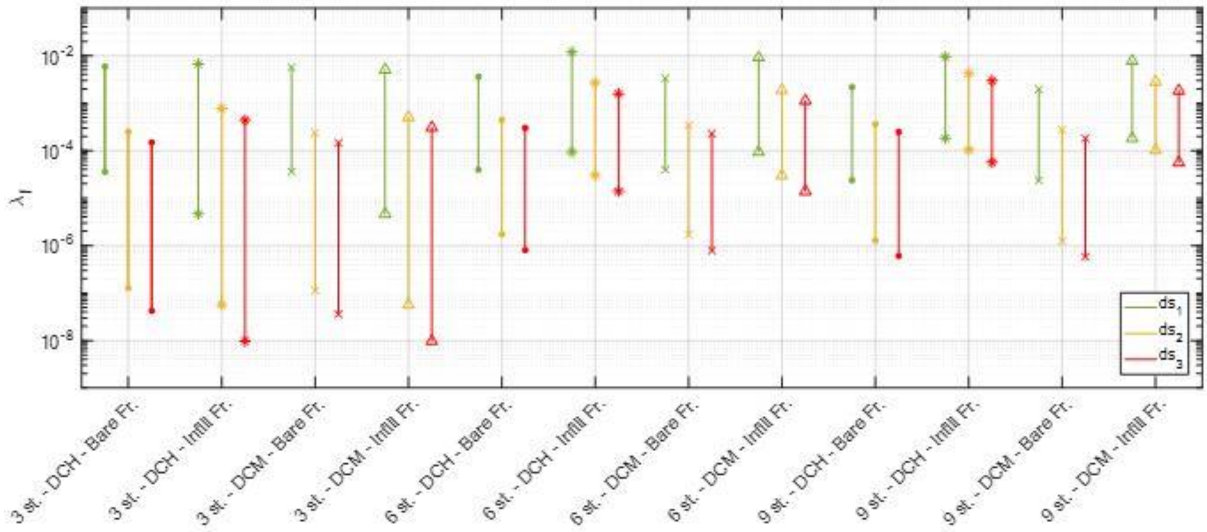

Fig. 3 - *Seismic failure rate intervals for bare and infilled code-compliant RC frames*.

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