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Reliability-targeted behaviour factor evaluation for code conforming Italian RC bare and infilled buildings

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Abstract

Nowadays, many Codes develop a force-based design approach, which accounts for the inelastic capacity of structures by means of a reduction coefficient, i.e. behaviour factor. At the same time, in earthquake engineering there is an ever-increasing interest on the evaluation of structural safety, through the so-called reliability analysis. Consequently, some studies analysed failure probability of some code-conforming buildings and they pointed out that actual code's provisions lead to a non-uniform structural safety level. For this reason, the first main objective of this study is to investigate extensively the failure rates of Italian code-conforming frame RC buildings, in order to establish how the rate of failure varies along the Italian territory. For this purpose, buildings with different number of floors, i.e. 3-6-9, with both levels of ductility (DCM-DCH) and in bare or infilled configuration have been studied, carrying on parametrically both the design and non-linear dynamic analyses in order to define frames vulnerability. Next generation of Eurocodes will explicit define the target reliability level that must be ensured by code's provisions. For this reason, has been developed a framework to obtain a risk-targeted spectral acceleration for the design of uniform reliability frames, which is one of the main aims of a risk-oriented earthquake engineering. In particular, this acceleration has been related to the code's workflow by the formulation of an alternative behaviour factor, that depends not only on structural features but also on the target failure's rate. This task is achieved by means of a hands-on approach, in fact the proportional relationship between elastic and targeted spectral acceleration is exploited.

Key words: RC frames, seismic reliability, reliability-targeted, behaviour factor

1 Introduction

In earthquake engineering, scientific community is showing increased interest on the evaluation of structural safety, which is commonly estimated through the use of reliability analysis theory [1-4]. A structural reliability analysis is able to capture different sources of uncertainty on both demand and capacity sides and quantify as relevant output the safety margin of a structural system via the use of synthetic indicators like failure probability [5], which quantify structural performance referred to target performance levels or damage states. Modern building codes (e.g. [6-8]) are based on a design process philosophy oriented at ensuring tolerable safety margins, with the use of so-called semi-probabilistic methods and prescriptive requirements. In this way, engineers can design structural systems avoiding the development of complex fully-probabilistic analyses, like those required in a Performance-Based Seismic Design (PBSD) [9], which is based on either full probabilistic frameworks or simplified and less computationally demanding methodologies. In fact, the majority of modern building codes are currently based on a simplified force-based approach, which considers only in an implicit way the failure probability. In this scenario, a structural designer following qualitative and quantitative code requirements ensures the compliance of the designed building, i.e. he/she follows a classic design process where a design is "checked" at the element level looking at different parameters. This conceptual approach takes seismic actions provided by the codes for estimating seismic demand at the element level, and further compare it with seismic capacity offered by the sized element, with geometrical and mechanical characteristics higher than the code minimum prescriptive requirements. However, no information can be derived from the design process on the seismic performance of the sized building, i.e. the physical way in which the structural system performs when it experiences earthquake shaking [10].

Code compliance and seismic performance must be therefore strictly coupled, i.e. a designer following code prescriptions must implicitly satisfy target performance levels. In United States, the Federal Emergency Management Agency (FEMA) recently published the FEMA P-58-5 Guideline summarizing the results of an in-depth investigation aimed to quantify seismic reliability and risk categories for different code-conforming structural archetypes considering various consequences, e.g. causalities, loss of use or occupancy, repair and reconstruction costs [11]. In the Italian context, Iervolino et al. [12] quantified the seismic reliability of some case studies of building archetypes designed for three different sites according to the Italian Building Code, suggesting how this method fails in ensuring a uniform reliability indicator for code-conforming buildings.

In order to overcome complexity of the use of PBSD, some researchers recently proposed an alternative methodology able to maintain the commonly used force-based approach, but at the same time to guarantee the achievement of adequate performance levels via the formulation and use of a risk-targeted spectral acceleration [13]. In other words, such methodology consists in a redefinition of the behaviour factor q to be used

in the seismic design force evaluation. Such new paradigm in the seismic design panorama makes clearer the link between code compliance and expected performance.
In this context, the present study aims at filling this gap by means of an extensive computational analysis of the seismic reliability of different configurations of RC bare and masonry-infilled buildings compliant with the Italian Building Code requirements [8] to derive reliability-targeted behaviour factors q_{RT} to be used by designers in Italy. For this purpose, a prototype seismic design software was developed in order to automatically design frames with different geometric characteristics against increasing seismic actions consistent with the current range of spectral accelerations derived from the Italian seismic hazard maps. Hence, a second tool for the seismic reliability assessment was carried out to quickly obtain seismic failure rates. Based on these outcomes, the authors investigated the correlation between seismic hazard and other relevant features like required ductility levels or number of floors with the resulting seismic safety quantified in terms of failure rate values. As second relevant outcome, reliability-targeted behaviour factors were derived for the different archetypes, showing also how future seismic design codes may propose to designers some design q_{RT}-curves able to provide the value of the reliability-targeted behaviour factor q_{RT} as a function of the expected seismic performance represented by a pre-set target seismic reliability indicator.

2 Description of case-studies and application

The structural archetype herein considered is represented by a residential building with RC frame resisting scheme (Fig1). The archetype layout is characterized by a rectangular plan shape and it consists of five bays in longitudinal direction and three in the transversal one, where all spans are 5 meters long. Three different elevation configurations have been considered, namely 3-, 6-, 9- storeys with a constant inter-storey height of 3 meters. As regards loading actions, a 5.5 kN/m^2 dead load and a 0.5 kN/m^2 live load have been accounted for the roof, whereas a 6.5 kN/m^2 and 2 kN/m^2 have been respectively considered as dead and live loads for the remaining floors. A Concrete C25/30 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 have been identified as relevant mechanical properties required by the prototype seismic design software. The presence of the staircase has not been considered. As regards masonry infilled frames, traditional masonry infills have been selected with bricks of 20 x 25 x 19 cm uniformly distributed over the perimeter of the archetypes. Both high ductility class (DCH) and medium ductility class (DCM) have been analysed, thus investigating also the impact of this choice on the final results.

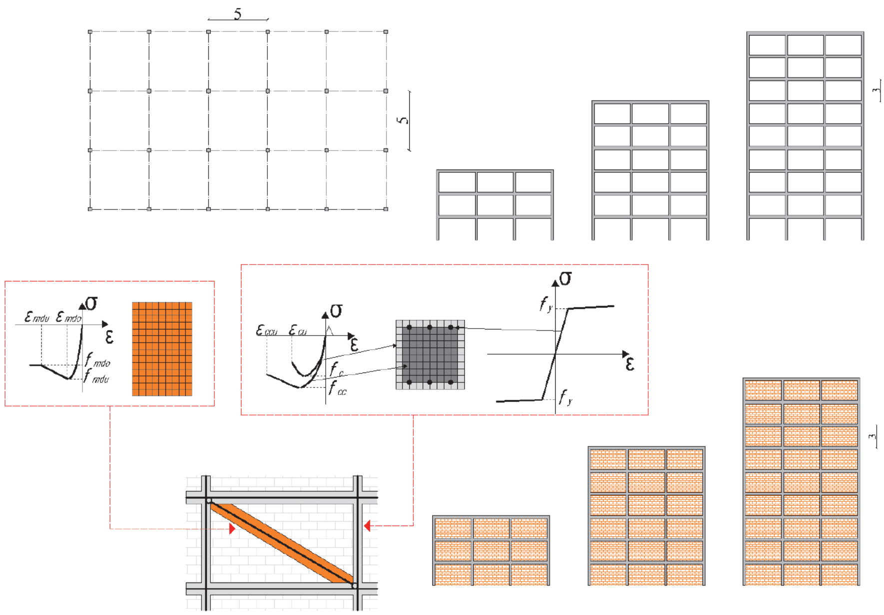

Figure 1. Bare and infilled RC frames and adopted constitutive laws for unconfined – confined concrete, steel rebars and masonry infills

Since the considered archetypes fulfil regularity conditions both in plan and in elevation, it is possible to ignore 3D effects and so designing frames in 2D on the smaller direction without loss of generality [14]. RSA method has been taken into account in the prototype seismic design software: to this aim, the fundamental period T_1 has been estimated with the simplified expression provided in the Italian Building Code [8]:

$$T_1 = 0.075H^{3/4} \tag{1}$$

where H is the building height in meters. Hence, the UHS elastic spectral acceleration $S_{ae}(T_1)$ based on T_1 can be derived considering UHS for the site of interest. In this work, 10 % exceedance probability in 50 years UHSs have been considered as reference input for the determination of the seismic actions as usually done in the majority of designs of buildings with residential use. For a proper parametrization of the design cases to be investigated, trials have been carried out starting from the Italian seismic hazard map to identify the range of UHS elastic spectral accelerations for each of the three different elevation layouts covering all the possible design scenarios that a structural engineer can face in Italy. In particular, the 3-storeys archetype is characterized by a range of $S_{ae}(T_1)$ between 0.1g and 1g, whereas the 6- and 9-storeys are enclosed in the intervals 0.1g – 0.75g and 0.1g – 0.5g, respectively.

Hence, the design-assessment framework has been launched for each combination of elevation layout, ductility class and location. The design process has been implemented so as the resultant sizing may fulfils code minimum requirements, but at the same time trying to optimize as much as possible elements' sections. All these data have been used as input for the seismic reliability assessment tool. In particular, geometrical and mechanical features are used to automatically define the nonlinear models for both bare and infilled frames, later tested with pushover analyses.

Pushover analyses have been subsequently carried out for the infilled RC frames. In this case, a significantly reduced ductile behaviour has been observed if compared with the RC bare configurations, with ultimate displacement values around a tenth of the RC bare ones, whereas the maximum base shear capacity is characterized by increments ranging between about 160 % and 220 %. These evidences can be directly attributed to the presence of the masonry infill panels enhancing the overall stiffness of the structural system and at the same time reducing the displacement capacity of the RC frame.

Starting from the trilinear backbone capacity curves, the hysteretic behaviour of the equivalent SDOF systems as well as relevant points useful for the damage state (DS) definition have been derived. In particular, three DSs have been fixed: the first one is *Slow Damage* (SD) which depicts the yielding point in the SDOF's behaviour curves, secondly there is *Near Collapse* (NC) that is placed at the beginning of the descending branch and lastly *Collapse* (C) when base shear is approximately equal to the 80 % of the maximum shear capacity. Hence, lumped models have been automatically created in OpenSees [15] and used for the execution of a huge number of NLTHAs. For this purpose, a dataset of 400 unscaled ground motion horizontal components recorded by 200 accelerometric stations has been built up, mainly considering records collected from the Italian Accelerometric Archive [16], and selected in order to obtain a widespread distribution of samples over the moment magnitude M_w vs epicentral distance R_{epi} plane. For each combination of elevation layout, ductility class and location, 400 NLTHAs have been subsequently executed deriving samples of the overall seismic response and storing the maximum displacement at the free end of the SDOF system due to the application of each ground motion record, as relevant *edp*. Fragility curves have been later derived with the use of Cloud Analysis method, first fitting with linear regression models the clouds of 400 *im-edp* data pairs, where PGA has been considered as relevant *im*, and then assuming as functional form a lognormal cumulative distribution function. A significant discrepancy between bare and infilled configurations was observed, in support of the considerations previously done when looking at the comparison between capacity curves. However, in this case, the higher is the number of floors and the lower is the probability to meet or exceed a damage state. It is worth to underline also how RC frames designed in sites characterized by low seismicity generally point out a worse performance than the ones sized in high seismic hazard locations: such difference seems to be more noticeable for the infilled frames rather than the bare ones.

3 Results

Seismic fragility curves have been finally convoluted with the fitted seismic hazard curves to obtain a seismic failure rate λ_f for all the investigated code-conforming RC archetype. Code-compliant Italian RC bare archetypes are characterized by seismic failure rates ranging from $4.11 \cdot 10^{-7}$–$2.75 \cdot 10^{-4}$ for NC damage state. This evidences how seismic failure rates increase as far as higher elastic spectral acceleration $S_{ae}(T_1)$ values are considered: such expected correlations are confirmed by results, showing also how a less marked increase can be observed for the higher part of the $S_{ae}(T_1)$ range. It is worth to recall how data points shown in Fig2 are strictly related to the selected location, i.e. in other terms considering a different site characterized by the same $S_{ae}(T1)$ value may lead to a different seismic failure rate. This issue has to be attributed to the specific characteristics of the slope of the hazard curve, that impacts the final result since it directly enters in the convolution integral. However, it can be noted how Italian regulations lead to design RC frames whose seismic reliability is directly proportional to the seismic hazard of the sites, thus failing to ensure a uniform level of seismic safety over the entire nation. In addition, Fig2 highlights how a significant reduction of the seismic safety (i.e. seismic higher failure rates) characterizes the RC infilled frames when compared to the bare ones, more proportional in low-to-medium seismicity locations.

4 Reliability-targeted behaviour factors evaluation

As aforementioned shown, code-conforming RC frames behave in a hazard-targeted way rather than in a reliability-targeted one. In order to overcome such underlying flaw, the design-assessment framework has been subsequently used for sizing and further assess seismic performance of RC frames designed with a wider range of fictitious inelastic spectral acceleration $S_{ad}^*(T_1)$ values (i.e. elastic spectral acceleration $Sae(T_1)$ divided by a unitary behaviour factor) in order to later quantify the reliability-targeted behaviour factor q_{RT} to be considered for each design action to get the expected target seismic reliability.

The $S_{ad}(T_1)$ values spans up to maxima that are automatically identified by the prototype design software by setting upper bound limits in terms of geometrical features for the designed main structural members (e.g. on beams' height): for higher values, in fact, it may be more convenient to change the horizontal resisting system into a stiffer RC shear-walls solution. The new wider set of fragilities is in such a way "site independent" since the link with the design phase is only made with the previously defined fictitious $S_{ad}^*(T_1)$. Each fragility can thus be convoluted with a fixed hazard curve of a location of interest to get its related seismic failure rate. Based on these outcomes, by fixing a priori a target performance seismic reliability with reference to a specific DS, it has been possible to identify among all the fragility curve that convolved with the hazard of the site of interest it is able to achieve the required performance, and thus identify the related

$S_{ad}^{*}(T_1)$ that it should be quantified in the classic design process. Hence, the associated reliability-targeted behaviour factor q_{RT} may be easily derived by diving the UHS elastic spectral acceleration $S_{ad}(T_1)$ with the sizing one $S_{ad}^{*}(T_1)$.

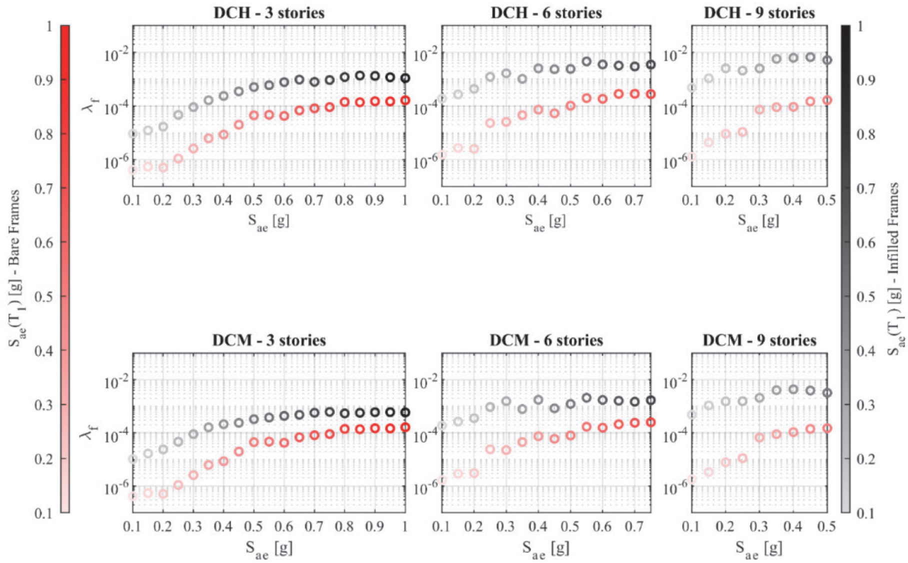

Figure 2. Comparison between NC seismic failure rates of bare and infilled RC frames

In this context, the selection of different target performance levels involves a consequent variation of the value of the reliability-targeted behaviour factor q_{RT} to be used in the design process. To clarify this relationship, a transformation of the failure rate in the Cornell's target seismic reliability index $\beta_{t,1}$ related to a 1-year reference time-window has been adopted in order to get a parameter with the same magnitude of q_{RT}, by means of the following equation:

$$\beta_{t,1} = -\Phi^{-1}(P_{t,1}) \tag{2}$$

$$P_{t,1} = 1-e^{-\lambda t} \tag{3}$$

where Φ^{-1} is the inverse of the standard normal distribution function, $P_{t,1}$ stands for the target failure probability in a yearly time-window, and λ_t is the target seismic failure rate.

Based on these considerations, it is possible to change the seismic design paradigm by starting from a desirable seismic safety (i.e. fixing a $\beta_{t,1}$ value) and thus identify the reliability-targeted behaviour factor q_{RT} for the derivation of a consequent reliability-targeted seismic action to be used in the classic force-based approach.

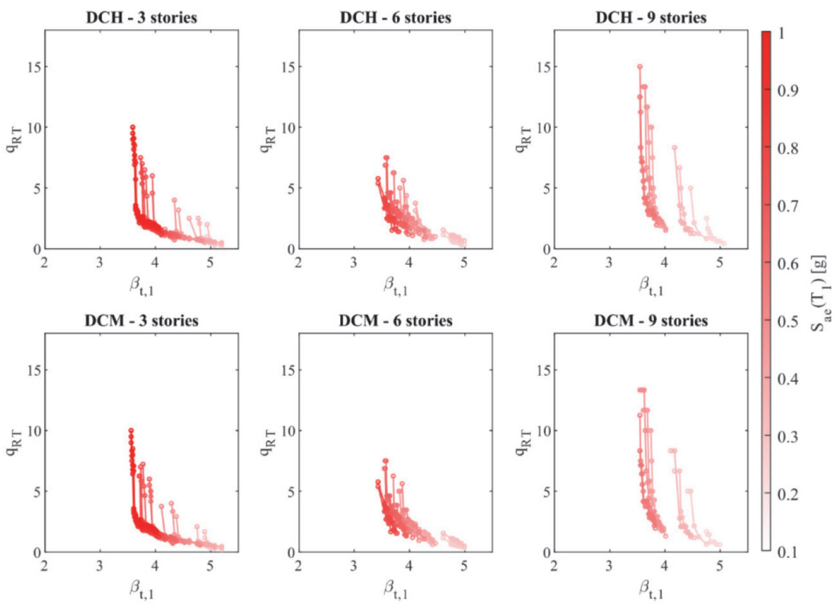

Figure 3. Adopted bridge FE model strategy

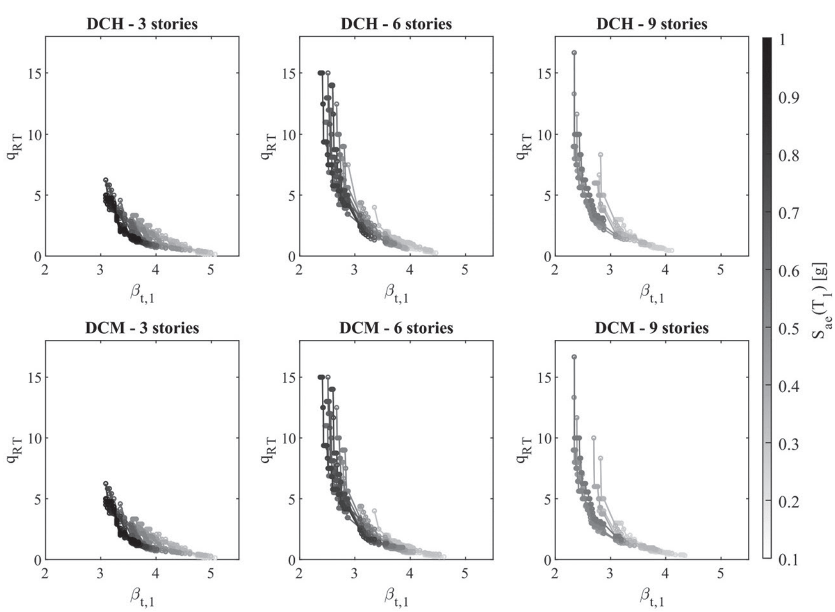

Figure 4. Adopted bridge FE model strategy

The q_{RT}-$\beta_{t,1}$ curves shown in Fig3 and Fig4 highlight an inverse proportional relationship between the reliability-targeted behaviour factor q_{RT} and the yearly target seismic reliability index $\beta_{t,1}$. It can be noted how low values of desirable seismic safety level correspond to high q_{RT} factors, given that it is sufficient to design with small spectral acceleration, and thus the UHS elastic spectral acceleration $S_{ae}(T_1)$ at the first period T_1 can be significantly reduced by q_{RT} to obtain the sizing $S_{ad}(T_1)$. On the contrary, high target seismic reliability indexes are related to low q_{RT} factors, since in those cases it may be necessary to increase the UHS elastic spectral acceleration $S_{ae}((T_1)$.
However, a limited range of desirable seismic reliability indexes must be considered for a specific site of interest, due to some constraints directly linked to code provisions and geometrical limitations. In particular, lower bound is governed by minimum provisions, given that below a certain spectral acceleration, seismic design is negligible if compared to the static one and so only minimum detailing provisions are the key elements that can contribute to the definition of the horizontal capacity of the structural system. On the opposite side, the upper bound is limited since elements sections cannot be increased indefinitely. Consequently, it is also not possible to satisfy higher values of $\beta_{t,1}$ than the one linked with the first sizing case where maxima in terms of geometrical dimensions of beams or columns are exceeded: for higher values.
The allowable $\beta_{t,1}$ values for RC bare frames placed in the locations shown in Fig3 are enclosed in the interval 3.5 to 5.2, whereas for the infilled configurations the reference range is between 2.3 and 5.0. It is worth to recall also how q_{RT}-$\beta_{t,1}$ curves seem to shift left-side as far as the seismicity of the location increases.
Lastly in the infilled frames a difference can be appreciated when looking at results for the 3-stories configurations with respect to the latter 6- and 9-stories ones. In particular, it seems that in the first case it is not possible to reach configurations whose their horizontal capacity is given by code minima detailing provisions, and such issue may be strictly linked to the enhanced impact of the infilled frames in the 3-stories configuration that already in itself is the most stiff among those analysed.

5 Conclusions

The present work illustrated the effects of current Italian Building Code provisions on the resulting performance of RC bare and infilled frames designed in different Italian locations. To this purpose, a general framework was built up and composed by an automatized prototype seismic design software coupled with a seismic reliability assessment tool in order to link seismic design actions to resulting failure rates. A wide numerical campaign based on the execution of NLTHAs was carried out investigating a large set of archetypes characterized by different design ductility classes and number of floors, as well as various locations over the Italian territory with the aim to consider different seismicity levels. Results highlighted how the seismic performance of code-compliant RC frames in Italy is strictly related to seismic hazard of construction's site,

whereas a negligible influence on the failure rates can be attributed to the elevation of the buildings as well as the design ductility class. The work demonstrated in such a way how current code provisions fail in ensuring a uniform seismic performance in Italy.
Moreover, to fill this serious shortcoming, the second part of the study was focused in developing q_{RT}-$\beta_{t,1}$ curves able to provide a new set of reliability-targeted behaviour factors q_{RT} for designers interested in achieving a desirable seismic safety a priori set by fixing a $\beta_{t,1}$ value of interest. Such new q_{RT} factors are in this way able to account for both the seismic hazard of the site as well as the target seismic safety acceptable for the designer. Results stressed the attention also on the role of masonry panels on the overall performance of the infilled RC frames, highlighting how even if current code provisions not explicitly indicate to account them as structural elements, they effectively interact with the RC frame providing a non-negligible stiffening contribution that reflects in a worsening of the seismic safety of such building archetypes. For this reason, it is suggested to consider as effective q_{RT} values those proposed from the results of the analyses carried out on the infilled configurations, so as to better capture the real seismic behaviour of the structural system.
This hands-on approach adopted in the present work could sound too hard-working because of the huge amount of analyses required to obtain such reliability-targeted behaviour factors. Future efforts will be therefore oriented in the development of analytical formulations that could immediately determine their values, and so facilitate the use also over the technical community of structural design engineers.

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