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Seismic risk of R.C. building classes

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Abstract

Seismic risk assessment on a large scale may be defined as the prediction of the fraction of buildings expected to reach a conventional limit state in the region and time period of interest. This definition is the frequentistic interpretation of the failure probability for a homogeneous class of structures. Empirical post-event survey methods for vulnerability evaluation may not fit the purpose of seismic risk analysis at class level and a pure analytical approach may be required. To this aim this paper proposes the extension of structure-specific reliability procedures, but without assuming a single structure as representative of the class. The class-capacity function is approximated by regression of significant cases analyzed by Static Push-Over (SPO); the seismic demand is obtained by Probabilistic Seismic Hazard Analysis (PSHA). The seismic risk is computed by simulation of the former being exceeded by the latter via the Capacity Spectrum Method (CSM). Explanatory application refers to six classes of Italian rectangular R.C. buildings; three classes are of *pre-code* constructions, designed only for gravity loads, whereas the other three considered are of *seismic* buildings designed with old codes not accounting for capacity design rules.

Keywords: Seismic risk; Reinforced concrete; Class of buildings; Capacity spectrum; Montecarlo simulation

1. Introduction

When considering the seismic risk assessment of a specific structure, the engineer will seek the frequency of one or more events leading to conventional structural failure. It is possible to define, for those *limit states*, a function (Z) which is non-positive if the failure condition is reached or exceeded. If some of the parameters Z depends on are uncertain, then the assessment of whether the structure is safe or not can only be given in probabilistic terms. The probability that Z is non-positive is the *failure probability* $\left(P_f\right)$; its complement $\left(P_s = 1 - P_f\right)$, the probability of survival, is a measure of the *structural reliability*. In the seismic case the Z-function is expressed in terms of nonlinear capacity (C) versus demand (D), Eq. (1). The latter is the required performance for the structure at a specific site and the former is the supply of such performance.

$$P_f = P\left[Z \le 0\right] = P\left[C \le D\right]. \tag{1}$$

The rate of the capacity being exceeded by the demand, in a given time interval, may be interpreted as the seismic risk. Several methods are available to compute $P\left[C \le D\right]$ in close or even approximate form and a comprehensive review of these methods is given in Pinto et al. [1] and references therein. A possible strategy is to separate the estimation of the structural response from the probabilistic characterization of the seismic threat it is subjected to, Eq. (2), [2].

$$P\left[C \le D\right] = \sum_{\text{All } a} P\left[C \le D \mid IM = a\right]\ P\left[IM = a\right]. \tag{2}$$

The second term in the right hand side of Eq. (2) is the occurrence probability of a ground motion Intensity Measure (IM), typically reflecting some spectral properties, computed by means of Probabilistic Seismic Hazard Analysis (PSHA) [3, 4]; $P\left[C \le D \mid IM\right]$, the *fragility*, is the failure probability for a given IM value and summarises the vulnerability features of the structure.

Eq. (2) may also apply to a class of structures and the failure probability may be interpreted as the fraction of buildings within the class expected to collapse. To this aim it is required to

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probabilistically characterize the seismic capacity and demand for the class, and an analytical evaluation of these terms seems to be the appropriate approach. In the case of class/regional scale seismic risk assessment, in fact, vulnerability data are often represented by means of statistical analysis of post earthquake damage surveys [5]. The empirical approach is adopted worldwide, in Italy for example, by the procedures issued by Gruppo Nazionale Difesa dai Terremoti — GNDT [6, 7]. The accuracy of empirical methods may be affected by the unavailability of a sufficient database of damage observations, which usually consists of heterogeneous data. Moreover, those studies often formulate the damage probability as a function of macro-seismic intensity scales [8] and, therefore, do not fit the seismic risk computation including PSHA, as in Eq. (2), because the two terms are not consistently expressed in terms of the same variable (even though some conversion is possible it would introduce further uncertainty in the process). Consequently, much research attempted to obtain vulnerability curves via alternative approaches less dependent on post-event surveying. An advance in this direction is represented by HAZUS methodology [9], which provides fragility curves for categories of structures depending, for example, on the design code enforced at the time of construction. However, the HAZUS loss assessment procedure is optimized for scenario analysis (e.g. for a given ground shaking level and/or magnitude–distance pair) rather than for risk evaluations, as also pointed out by Crowley et al. [10]. A further attempt to estimate the vulnerability by class-representative mechanical models is given by the "semi-quantitative" methods [11], which need a limited number of input data in compliance with the amount of information generally available. The approach of Rossetto and Elnashai [12], which is a characterized by a state-of-the-art quantitative framework, still considers a specific structure as representative of the class. In conclusion, even though some interesting effort exists [13], the degree of knowledge about structural models required by structure-specific methods and the computational exertion to compute Eq. (2), are not easily applied to the class-scale analysis. On the other hand, the traditional (observational) vulnerability approach seems to be inadequate for probabilistic seismic risk assessment.

Herein, the formulation of an analytical method for class-scale risk analysis is discussed. A set of buildings, belonging to the class of interest, is reproduced by means of simulated design with reference to the codes, the available handbooks and the current practice at the time of construction. The capacity of the class is retrieved by performing a set of Static Push-Over (SPO) analyses for these significant cases. Then, multiple regression allows the expression of capacity as a function of the parameters of interest (materials, geometrical and structural features). The seismic demand is given by inelastic modification of the probabilistic elastic spectra resulting from PSHA. The expected number of failures within the class is estimated comparing C and D by a simple simulation method (i.e. Montecarlo). The approach allows to explicitly account for several uncertainties related to both seismic response and structural damage, avoiding the shortcomings of empirical vulnerability analysis. Moreover, the spectral demand analysis ensures a computational effort appropriate to the scale of the problem. As for illustrative application, the method is used to compute seismic risk of Italian existing rectangular plan view R.C. buildings, pre-code and designed for seismic action.

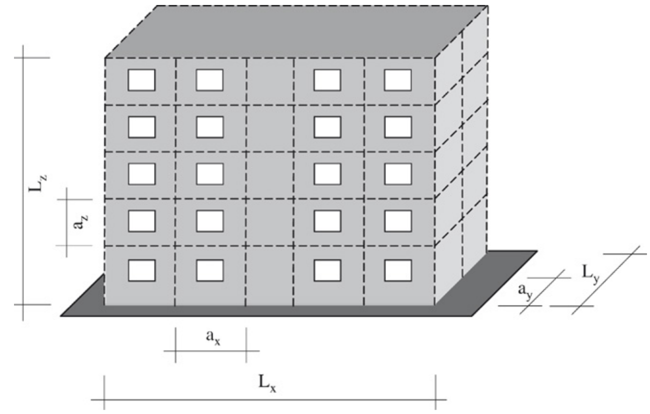

Fig. 1. Generic element representing the class and geometric random variables.

2. Formulation and methodology

The estimation of the fraction of structures in the class not expected to survive the period of observation may be formulated assuming that the class is the entity the failure probability has to be computed for, as seismic reliability methods compute P_f for specific structures. To this aim the probabilistic characterization of the *class-capacity* and of the *class-demand*, which are functions associating to any building belonging to the class of its seismic performances, is needed. In the case of structure-specific problems the uncertainties affecting C and D reflect the intra-structure variability of some global or local factors as material properties and the variability of response to ground motion. At the class-scale, in addition, uncertainty also includes the building-to-building variability of structural system and details. For example, if the class of interest is one of the Italian pre-code rectangular R.C. buildings with a given number of storeys, the class is represented by a generic building as in Fig. 1. A particular structure belonging to the class is defined by a realization $\overline{x}$ of a vector of random variables, $\overline{X} = \{X_1, X_2, \ldots, X_n\}$, which may also include plan dimensions, bay lengths and inter-storey height. Then the limit state function may be expressed as in Eq. (3).

$$P_f = P\left[Z(\overline{X}) \leq 0\right] = P\left[C(\overline{X}) \leq D(\overline{X})\right]. \tag{3}$$

Since for any $\overline{x}$, the $C(\overline{x})$ and $D(\overline{x})$ functions return the seismic capacity and demand respectively of the structure defined by $\overline{x}$, the risk assessment is possible only if statistics for the components of the $\overline{X}$ vector are available. This paper will discuss the analysis of C and D and will not deal with the issue of estimating the distributions they depend on, which may be carried out for example, by sampling the population under analysis.

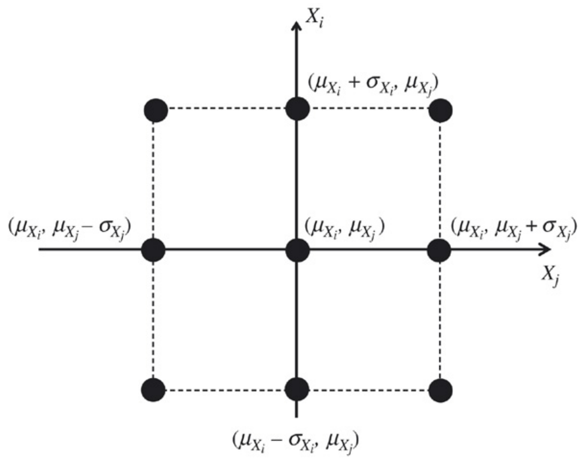

Fig. 2. Representation of $3n$ experimental plan for two generic variables affecting the capacity.

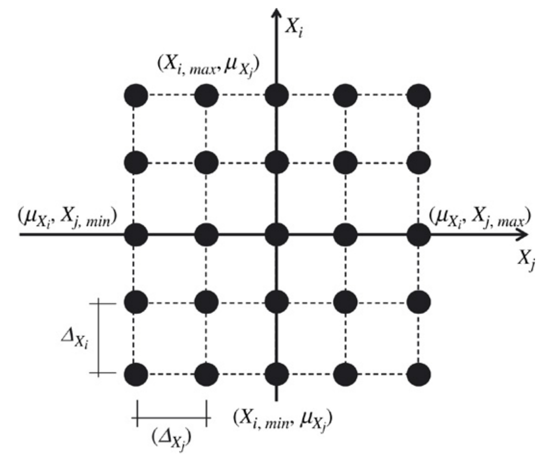

Fig. 3. Dense DoE for two variables. Each point require an SPO and the capacity function.

2.1. Class-scale capacity

The class-capacity may be defined as the function mapping the capacity curve to the vector $\overline{X}$. Then, for any realization $\overline{x}$, which identifies a particular structure, the function returns the appropriate set of effective period, ultimate displacement and strength defining its capacity curve (to follow). At least two options are available to get an approximate form of such capacity.

Option A: The marginal statistics of the n terms (variables) of the $\overline{X}$ vector, for each class, are assumed to be known and stochastically independent. The approach consists of preliminary planning of a number of structural analyses defined according to the distributions of the variables. Then one may consider, for example, a 3^n factorial plan as for the Design of Experiments (DoE) in the Response Surface Method (RSM) [14]. The levels of factors of $\overline{X}$ are selected to capture their variability in the class; for instance, if a relevant factor X_i is Gaussian and narrowly distributed around its mean in the population of structures to be investigated, the three levels for the experiments may be the mean $\left(\mu_{X_i}\right)$ plus or minus one standard deviation $\left(\sigma_{X_i}\right)$. In Fig. 2 a DoE plan for two generic variables $\left\{X_i, X_j\right\}$ is given. From this standpoint, a series of meaningful combinations of levels of the vector $\overline{X}$, hence a series of particular structures, are defined and analyzed to observe the capacity.

This approach has been developed by Iervolino et al. [15]; in that study the procedure associates each point of the DoE to a fragility curve via nonlinear dynamic analysis to obtain the seismic vulnerability, rather than the seismic risk, as a function of some structural parameters. Herein, a Static Push-Over (SPO) evaluation is performed at each point of the DoE, and the capacity depending on $\overline{X}$, for example in terms of ultimate displacement C_d, is obtained. Results from these analyses (i.e. ultimate roof displacement) are fitted by multi-parametric regression (e.g. linear), which provides capacity for any structure of the class (e.g. $\overline{x}$ realization) not specifically investigated. Such functional format is given in Eq. (4) where the a terms are the constants to be determined.

$$C_d(\overline{X}) \approx a_{C_d,0} + \sum_{i=1}^{n} a_{C_d,i} X_i. \tag{4}$$

Since for the 3^n structures analysed a capacity curve is available by SPO, similarly, approximated functions for the strength (C_s) and the effective period (T) may be defined as in (5) and (7).

$$C_s(\overline{X}) \approx a_{C_s,0} + \sum_{i=1}^{n} a_{C_s,i} X_i \tag{5}$$

$$T(\overline{X}) \approx a_{T,0} + \sum_{i=1}^{n} a_{T,i} X_i. \tag{6}$$

Option B: The procedure just discussed requires an intentionally limited number of structural analyses and it is able to provide an approximate form for the class-capacity with comparatively little effort. However, if the statistics of the $\overline{X}$ variables are disperse and the points of the DoE are relatively far from each other, the linear or even quadratic regressions may not be appropriate to capture the actual variation of capacity within the class. In fact, if the dispersion of the component of the $\overline{X}$ vector is large, the fitting by a regression may approximate the capacity too roughly. Therefore, another possible option, adopted herein, may be to compute capacity for many cases defined by scanning the range of the generic X_i variable. This kind of dense experimental plan (Fig. 3) does not require fitting a function covering a broad region of the $\overline{X}$ domain, but rather a series of local interpolating functions defined between adjacent points of the DoE (Fig. 4). The number of SPO required may be much larger than option A, but it has the benefit of reducing the approximation of the C function. The limits of this experimental plan have to be defined, as in option A, trying to capture as much as possible the variability of the components of the $\overline{X}$ vector; the density of the DoE has also to be calibrated accounting for the available computational capabilities.

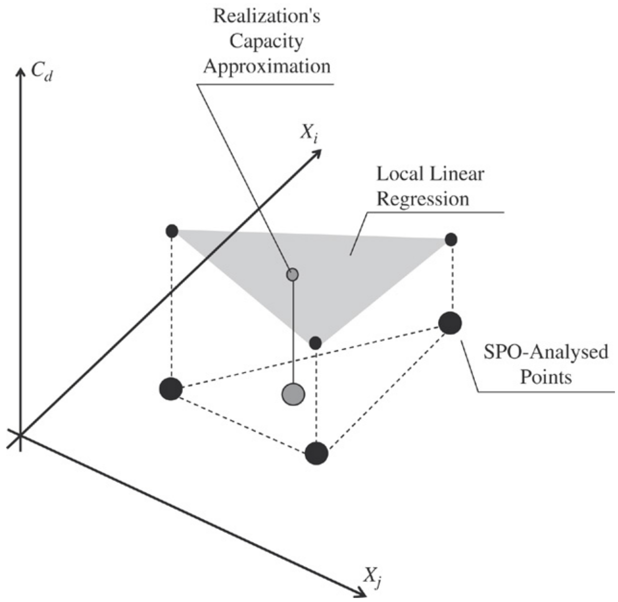

Fig. 4. In the case of Option B a local linear function among the pre-analyzed points is fitted.

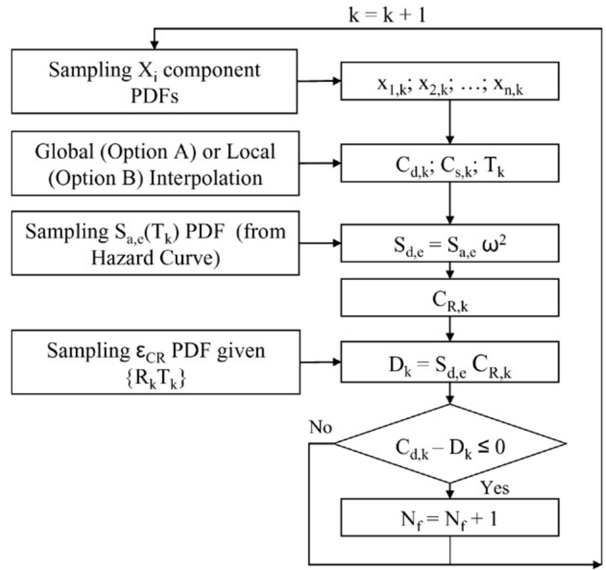

Fig. 5. Seismic risk computation flow chart.

2.2. Seismic demand

Demand, $D(\overline{X})$, refers to inelastic spectral analysis, in the sense of the Capacity Spectrum Method (CSM) in its modified format by Fajfar [16]. According to this approach it is necessary to get inelastic displacement demand for any possible period $T(\overline{X})$. In the light of seismic risk, this implies that each spectral ordinate has a corresponding probabilistic distribution reflecting the seismic hazard at the site. This task may be carried out referring to the common PSHA. In fact, Probabilistic Seismic Hazard Analysis provides distributions of pseudo-acceleration spectral ordinates, $S_{a,e}(T)$, at given period. Since the elastic displacement, $S_{d,e}(T)$, can be obtained by Eq. (7) where $\omega = 2\pi T^{-1}$, then the PDF of the latter is a simple transformation of the distribution of the former.

$$S_{d,e}(T) = \frac{S_{a,e}(T)}{\omega^2(T)}. \qquad (7)$$

In order to evaluate the inelastic displacement demand $S_{d,i}(T)$, see Eq. (8), the elastic displacement should be adequately modified by displacement modification factor, $C_R(R, T)$ [17]:

$$S_{d,i}(T) = S_{d,e}(T)\ C_R(R, T) \qquad (8)$$

where R is the strength reduction factor defined as the ratio of $S_{a,e}(T)$ times the mass m, over the strength C_S. Miranda has shown in [18] that this approach provides a better estimate of the maximum inelastic displacement than using a displacement ductility ratio.

In order to account for all uncertainties in computation of Eq. (1) the variability of C_R has to be included. The conditional distribution of C_R, given $\{T, R\}$, may be assumed to be lognormal (Miranda, personal communication, 2005) and therefore the random variable may be written as in Eq. (9),

$$C_R = \hat{C}_R\ \varepsilon_{C_R} \qquad (9)$$

where $\hat{C}_R$ is the median and the log of ε_{C_R} is normally distributed with zero mean and variance equal to the variance of C_R. Finally, if the displacement capacity of the structure is indicated as C_d, the limit state function can be written as in Eq. (10) depending on the vector $\overline{X}$ of structural random numbers (e.g. materials, members size, plan view geometry, etc.)

$$Z(\overline{X}) = C_d - S_{d,e}(T)\ C_R(R, T). \qquad (10)$$

2.3. Risk analysis

Given that the seismic class-capacity and class-demand may be computed, the CSM may be applied virtually to any structure, either specifically analyzed in the DoE or not, and the limit state function may be also checked for collapse or survival. Then, considering the marginal distributions of the components of $\overline{X}$, the risk of the class may be estimated by a conventional simulation method, as Montecarlo, applied to Eq. (10). Such simulation proceeds by the steps here listed and represented by the flow chart of Fig. 5. In any single run, indicated by the ordinal k, the $\overline{X}$ vector is sampled (i) according to the marginal distributions of its components and a realization $\overline{x_k} = \{x_{1,k}, x_{2,k}, \ldots, x_{n,k}\}$ is obtained; (ii) the capacity of the building defined by $\overline{x_k}$ may be retrieved by option A, or by option B, resulting in a $\{C_{d,k}, C_{s,k}, T_k\}$ set; (iii) the $S_{a,e}(T_k)$ distribution is sampled; (iv) $S_{d,e}(T_k)$ follows by Eq. (7); (v) the median $\hat{C}_R(T_k, R_k)$ factor is calculated; (vi) the conditioned distribution of the residual (ε_{C_R}) of C_R is sampled and the actual $C_{R,k}$ and the inelastic demand are obtained by Eq. (8); (vii) the capacity and the demand are compared to see if the limit state is exceeded in the kth run.

At the end of the simulation the seismic risk of the class is estimated by the ratio of counted failures over total number of runs (N_{tot}). Moreover, given that the structural failure is a Bernoullian random variable, then P_f (just computed by simulation) approximates its expected value, while the variance

(σ^2) may be estimated as $P_f(1-P_f)/N_{\text{tot}}$. Therefore, to also keep track of uncertainty in the risk assessment, a confidence interval may be computed, resulting (assuming a 0.05 confidence level) in $P_f \pm 1.96\sigma$.

The flow chart of Fig. 5 calls for an "option C" to the class-capacity analysis. In fact, once step Eq. (1) is completed and the $\overline{x}$ realization is obtained one may think of analyzing (by SPO) the corresponding structures in real time, skipping the approximation related to the global, or even local, regression. This approach, which may be referred as *direct Montecarlo*, is certainly the more attractive option in terms of accuracy, however it is extremely demanding in terms of computational effort and its benefits are almost negligible in respect of the option B if the latter has a sufficiently dense DoE.

3. Capacity analysis for a class of R.C. buildings

Collecting buildings in a homogeneous class may help to reduce epistemic uncertainty in the seismic capacity assessment. To this aim the definition of the class should be based on parameters which affect the seismic behaviour of the buildings, while they are available at a large scale [19]. The very simple features which may be directly related to the seismic assessment are: plan morphology, number of storeys and design code enforced at time of construction. Similar classification is adopted by HAZUS.

The procedure presented above requires that a specific structure then has to be associated to any realization of the $\overline{X}$ vector in the DoE; to this aim a specific re-design procedure, based on simulated design, has been developed in [20] and employed herein. Once the structures have been defined, nonlinear analysis allows the retrieval of the seismic capacity in terms of $\{C_d, C_S, T\}$. This job is generally carried out, on a class scale, by simplified methods based on the assumption of failure mechanisms [20,21]. On the other hand, the Static Push-Over analysis is an attractive solution for the trade-off in investigating the building seismic behaviour by performing a comparatively simple, yet accurate, assessment, including different sources of deformability [22]. For the purposes of this study, a lumped plasticity model was implemented to assess the class-capacity with a structure-specific accuracy.

3.1. Re-design process

Adopting a 3D mesh with variable module's linear dimensions $\{a_x, a_y, a_z\}$ it is possible to reproduce a set of geometrical models, which are consistent with the global building dimensions L_x, L_y and L_z, as shown in Fig. 6. Geometrical mesh discontinuities are explicitly considered; e.g. the number of stairs (n_s) and the length of the stair module a_s, and the inter-storey height of the first floor (a_{1z}). The latter may differ from a_z for structural (foundation level) and/or architectural reasons. Moreover, for each geometric model, it is possible to define a set of structural models depending on the number and location of the structural elements. Although the configuration of the columns is uniquely determined for a given geometric model, on the other hand, the beam number and position are determined by the number of plane frames in x and y directions, n_{px} and n_{py} respectively.

In gravity load design (pre-code buildings), according to experience from field observations [23], it is assumed that only the lateral plane frames exist in the short direction ($n_{py} = 2$). Conversely, when seismically designed buildings are concerned [24], the number of plane frames in the short direction is equal to the number of bays, $n_{py} = n_x$. Considering that column orientation (*OR*) complies with architectural rules, it is assumed that columns on the perimeter and those adjacent to the stair module are oriented so as to lay inside the infill thickness. For the remaining columns, two limit schemes are adopted, considering for each direction x and y the extreme situations of strong and weak column orientation. The columns and beams identified in the previous step are designed, in terms of cross-section and reinforcement, according to code and design practices related with the construction age. In particular, for the gravity load design an element level analysis model was generally adopted (e.g. axial load for the columns, simple bending for the beams). While, regarding seismically designed buildings, it was common practice to consider the horizontal slabs as deformable in their plane; the adopted analysis model assumes simple plane frames extracted from the 3D structure and do not consider the stair stiffness. Material properties selected for design derive from prescribed codes and consider steel and concrete types commonly used at the age of construction.

3.2. Nonlinear analysis

Seismic capacity is evaluated by means of SPO. As mentioned, the flexural behaviour of the beam/column element is characterized by a lumped plasticity model; a moment–rotation (M–θ) relationship, depending on geometric and mechanical features of the element end sections, has to be defined. The adopted M–θ elastic–plastic curve is defined by the yielding (θ_y) and ultimate (θ_u) rotations computed as proposed by Panagiotakos and Fardis [25]. The influence of shear action is modeled based on a reduction of the shear strength depending on the local ductility, expressed in terms of rotation varying with a linear trend [26]. These models depend mainly on compressive (f_c) and steel yielding (f_{sy}) strengths. Beam/column joint failure is not considered. The capacity curve, in terms of lateral strength V_b and displacement at the roof level Δ, is determined up to maximum lateral strength (*near collapse*), consistent with adopted mechanical models. Structural failure corresponds to the first attainment among element failure (ultimate rotation or ultimate shear strength of a structural member) and the near collapse condition of the structure. The MDOF–SDOF equivalence prescribed by CSM is performed referring to the structure's failure point. Furthermore, the transformation of the capacity curve in bilinear form allows estimating nonlinear strength $C_s(\overline{X})$; the displacement capacity $C_d(\overline{X})$; and effective period $T(\overline{X})$, as shown in Fig. 7, where $\overline{X} = \{L_x, L_y, \ldots, f_{sy}\}$ is the vector of parameters the limit state function depends on, as

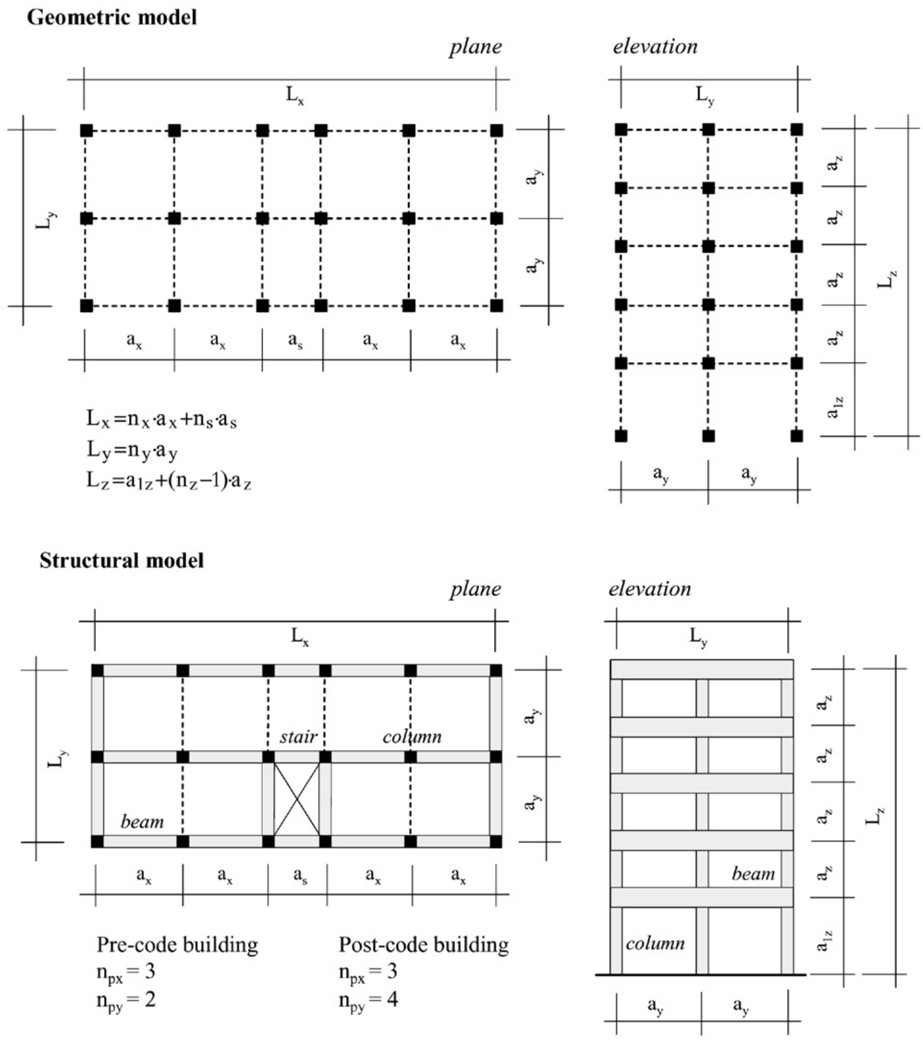

Fig. 6. Building model: Geometric mesh and structural model.

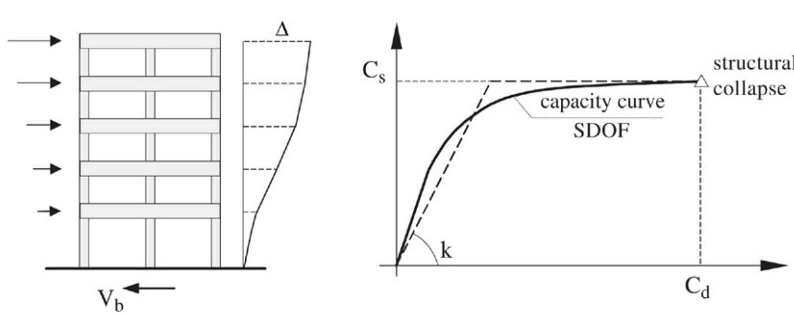

Fig. 7. Capacity parameters.

listed in Table 1. What has been discussed for the collapse may be similarly applied to any other limit state. In fact, the method allows evaluation of the capacity and demand parameters concerning any other limit state condition, which can be determined by the static push-over analysis and according to current seismic codes (i.e. first yielding and performance levels corresponding to values of inter-storey drift ratios).

4. Application

The procedure presented in the previous sections has been applied to compute total risk for R.C. building classes located in a moderate seismicity site in southern Italy. The application refers to 3, 4 and 5 storeys *pre-code* and *seismic* rectangular buildings. Pre-code, or gravity load designed, represent the majority of the building stock in many areas that have been

Table 1
Building model parameters

Geometric		Structural		Mechanical	
Plan dimensions	L_x, L_y	Bay length	a_x, a_y	Concrete	f_c
Height	L_z	Number of plane frames	n_{px}, n_{py}	Steel	f_{sy}
Number of storeys	n_z	Column orientation	OR		

Table 2
Design of experiments details

Variable	Range	Scanning step
L_x	[15–32]	1.0 m
L_y	[8–12]	1.0 m
f_c	[5–45]	10 MPa
f_{sy}	[200–600]	50 MPa
a_x, a_y	[3–5]	See compatibility equations of Fig. 6

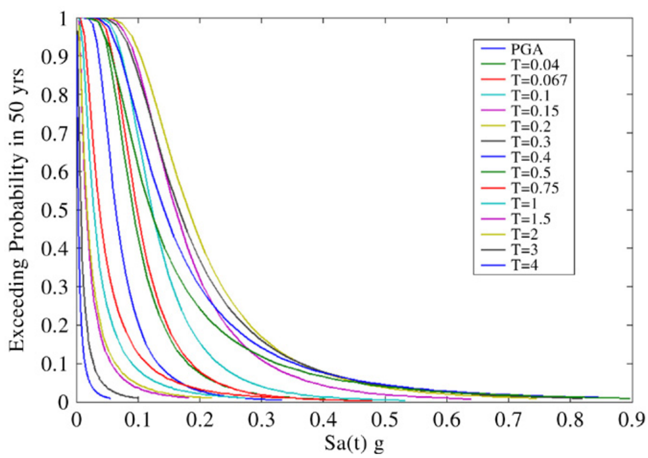

Fig. 8. Seismic hazard of this study.

recently classified as seismic, according to the last Italian hazard map [27]. On the other hand, many seismically designed constructions reflect old codes, [28–30], which do not account for *capacity design* rules. Hazard curves, computed by PSHA, used in this application define the seismicity of the site where the classes are supposed to be located. In Fig. 8 selected curves for several T value classes are given referring to a 50 years time span. In Fig. 9 the Uniform Hazard Spectrum (UHS), corresponding to a 10% exceeding probability in 50 years, is given.

Adopting option B, for determining the class-capacity and computing total risk, a large amount of SPO is performed considering all the possible cases defined by scanning the significant ranges of the input variables. These ranges are defined depending on $\overline{X}$ distributions. In particular, the base plan view dimensions $L_x(L_y)$ are assumed to be normally distributed with a mean of 25.0 m (10.0 m) and a Coefficient of Variation (CoV) of 12% (6%). These values are chosen based on the results of field surveys carried out in southern Italy and expert judgement [23,24]. The height is determined by the number of storeys and assuming $a_z = 3.00$ m and $a_{1z} = 4.50$ m for all classes, then height of storeys is not

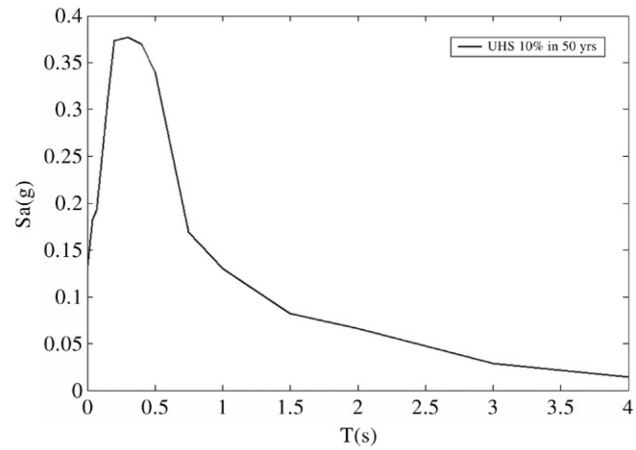

Fig. 9. Uniform hazard spectrum.

considered to be a random variable within a class. Hence for 3, 4 and 5 storey building classes height L_z is 10.5 m, 13.5 m, 16.5 m respectively. Compatibly with the assumed L_x dimension it is hypothesized that one single stair ($n_s = 1$) module accomplishes the building functionality. Stair module dimension $a_s = 3.00$ m is considered. According to the design practice and common architectural trends the a_x and a_y modules' linear dimensions are limited in the range of [3–5] meters. Only the strong column orientation is considered as a variable, *OR*. Finally, concrete and steel strengths f_c and f_{sy} are normally distributed with mean 25 N/mm^2 and 400 N/mm^2 and CoV 25% and 15% respectively [31,32]. Starting from these assumptions the input variables' ranges are defined and given in Table 2. The scanning step of such ranges was chosen in order to optimize the trade-off between having a dense DoE, as requested by option B of class-capacity analysis, and the computational effort. Hence, it is possible to compute total risk as outlined in Section 2.3. In particular, for each sampling of the distribution of the $\overline{X}$ variables, local interpolation of the capacity, $\{C_d(\overline{X}), C_s(\overline{X}), T(\overline{X})\}$, corresponding to the closest points in the analysis plan is performed. The so-determined inelastic displacement capacity C_d is, then, compared to the inelastic seismic demand D computed as in Eq. (8). The number of collapses ($C < D$) over the total number of trials is the expected fraction of failures in 50 years summarized in Table 3. The given results show that, even when not considering any capacity design principle, the seismic classes are characterized by a risk one order of magnitude lower than pre-code cases. The different structural system results in a shorter effective period of the former with respect to the latter which results in a lower displacement demand. However some input of the analysis are arbitrary, i.e. in the distribution of the $\overline{X}$ variables, and therefore the results in terms of total risk, should be only taken as an illustration of the proposed methodology.

5. Conclusions

The method presented in this paper deals quantitatively with the large number of factors involved in the *total risk* analysis for classes of buildings. Formulation explicitly takes into account uncertainties in inelastic capacity and demand extending

Table 3
Class failure probability P_f

Number of storeys of the class	Pre-code expected failures fraction	Seismic expected failures fraction
3	4.00×10^{-3}	8.00×10^{-5}
4	3.30×10^{-3}	2.40×10^{-4}
5	5.20×10^{-3}	5.80×10^{-4}

the approach of structure-specific reliability methods. The mechanical evaluation of the seismic capacity avoids some limitations of empirical vulnerability analysis. The limit state function is represented in terms of inelastic displacement and demand. Class-capacity is defined as a function mapping the bilinear force–displacement curve to the factors identifying a specific structure within the class. Two alternative options are given to get such function by interpolation of a number of push-over analyses. A specific computer code has been developed to re-design the buildings to be analyzed associating a specific structure to the poor information of the input variables. The probabilistic characterization of the limit state function is obtained considering the statistics of the capacity affecting variables (to be retrieved surveying the population under investigation). Seismic demand refers to PSHA modified by inelastic spectral amplification factors; uncertainty related to the latter is also included. The application investigates three to five storey R.C. buildings with pre-code or poor seismic design and a moderate seismicity site in southern Italy. Although the distributions of the parameters do not reflect a case study, results show that the seismic design, even when not considering the capacity design philosophy, lowers the risk of an order of magnitude in respect to pre-code classes.

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