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SEISMIC RISK FOR ITALIAN TYPE R.C. BUILDINGS

Iunio IERVOLINO[1], Gaetano MANFREDI, Maria POLESE[1], Gerardo Mario VERDERAME[1] and Giovanni FABBROCINO[2]

SUMMARY

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. 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 consider seismic buildings designed with old codes not accounting for capacity design concepts.

1. INTRODUCTION

When specific structure is concerned, to assess the seismic risk, the engineer will seek the frequency of one or more events leading to conventional structural failure [Cornell, 2004]. 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* (P_f); its complement $P_s=(1 - P_f)$, 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 $P[C \le D]$ in close or even approximate form and a comprehensive review of these methods is given in Pinto et al. [Pinto et al, 2004] and references thereof. 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):

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

[1] Dipartimento di Analisi e Progettazione Strutturale, Università di Napoli Federico II, Via Claudio 21, 80125 Naples, Italy
Email : iunio.iervolino@unina.it; gaetano.manfredi@unina.it; mapolese@unina.it ; verderam@unina.it
[2] Dipartimento SAVA, Università del Molise, Via De Sanctis, 86100 Campobasso, Italy
Email : giovanni.fabbrocino@unimol.it;

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) [McGuire, 1995];$P[C \leq D|IM]$, the *fragility*, is the failure probability for a given IM value.

Equation (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. It is required to probabilistically characterize the seismic capacity and demand for the class and an analytical evaluation of these terms seems the appropriate approach to the risk assessment. In the case of class/regional scale seismic assessment, in fact, vulnerability data are often represented by means of statistical analysis of post earthquake damage surveys [Rossetto et al., 2003]. The empirical approach is adopted worldwide, in Italy for example, by the procedures issued by Gruppo Nazionale Difesa dai Terremoti – GNDT [Di Pasquale et al., 2005], [CNR-GNDT, 1994]. The accuracy of empirical methods may be affected by the unavailability of a sufficient database of damage observations, which usually consists of heterogeneous structural data. Moreover, those empirical studies often formulate damage probability as function of macro-seismic intensity scales [Dolce et al, 2006] and, therefore, do not fit the seismic risk computation including PSHA as in Eq. (2) because the two terms are not consistently expressed by the same variable (even though some conversion is possible it would introduce further uncertainty in the process). Consequently, much research has been attempted to obtain vulnerability curves by alternative approaches less dependent on post-event surveying. An advance in this direction is represented by HAZUS methodology [HAZUS, 1999], which provides vulnerability functions for categories of structures depending, for example, on the design code enforced at time of construction: a bilinear *capacity curve*, associated to each category, is compared to inelastic spectral demand to get the *performance point* so that class-scale lognormal fragility curves can be obtained. 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. [Crowley et al, 2004]. A further attempt to estimate the vulnerability of the class by some representative mechanical models is given by the “semi-quantitative” methods [Calvi, 1999] which need a limited number of input data in compliance with the amount of information generally available. In conclusion, even though some interesting effort exists [Fischer et al., 2002], the degree of knowledge about structural models required by structure-specific methods and the computational exertion to perform non-linear dynamic analysis to compute Eq. (2), are not of easy application to a class-scale analysis. On the other hand the traditional observational vulnerability methods seem to be inadequate to probabilistic seismic risk assessment.

Herein, the formulation of an analytical method for class-scale risk analysis is discussed. A set of cases representative of the buildings within the class is reproduced by means of a 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) analysis 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), derived from inventory. 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 the explicit account of several uncertainties related to both seismic response and structural damage, avoiding the shortcomings of empirical vulnerability analysis, moreover the spectral seismic inelastic demand analysis ensures a computational effort appropriate to the scale of the problem. As for explanatory application the method is used to compute seismic risk of Italian existing rectangular plan view R.C. buildings seismically and pre-code designed.

2. FORMULATION AND METHODOLOGY

The estimation of the fraction of structures in the class expected to not 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 a probabilistic characterization of the *class-capacity* and of the *class-demand* are needed. These are functions relating to any building belonging to the class its seismic performance. 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 structure-to-structure variability of structural system and details. For example, if the class of interest is the one of Italian pre-code rectangular R.C. frames with a given number of storeys, the class is represented by a generic building as in Figure 1.

A particular structure belonging to the class is defined by a realization x of a vector of random variables, $X=\{X_1,X_2,\ldots,X_n\}$, which components may also include plan dimensions, bay lengths and inter-storey height. Then the limit state function may be expressed as in Eq. (3):

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

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 of the $\overline{X}$ vector components are available. This paper will discuss the analysis of *C* and *D* and will not deal with the even relevant issue of estimating the distributions they depend on, which may be carried out for example, by sampling the population which is under analysis; details on the topic may be found in [Ozceb et al., 2004], [Steimen et al., 2004].

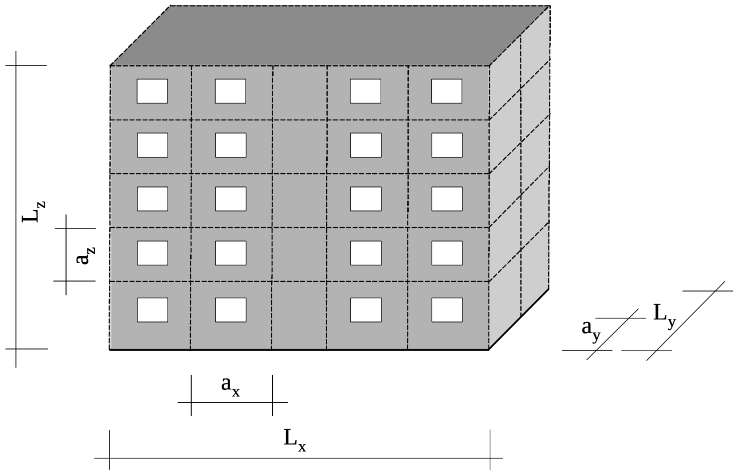

Figure 1: Generic element representing the class and geometric random variables

2.1 Class scale capacity

The class-capacity may be defined as a function mapping the capacity curve to the vector $\overline{X}$; so that for any realization $\overline{x}$, which identifies a particular structure, the function returns the appropriate set of effective period, ultimate displacement and strength able to define 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 of the $\overline{X}$ vector within the class are assumed to be known and s-independent. The approach consists of a preliminary planning of a number of structural analyses defined according to the distributions of the parameters. Then one may consider, for example, a 3^n factorial plan as for the Design of Experiments (DoE) in the Response Surface Method (RSM) [Iervolino et al., 2004], [Cosenza et al., 2005]. The levels of factors $\overline{X}$ are selected to capture their variability in the class; for instance, if a relevant factor is a narrowly normally distributed around its mean in the population of structures to be investigated, the three levels for experiments may be mean (μ_X) plus or minus one standard deviation (σ_X). From this standpoint, a series of meaningful levels of the vector $\overline{X}$, hence a series of particular structures, are defined and analysed to observe the capacity; finally these results are interpolated, by a multi-parametric regression, for further predictions. In other words, considering three different values for each X_i variable, 3^n structures belonging to the class are defined. If a SPO evaluation is performed at each point of the DoE, 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 (i.e. linear) which provides capacity for any structure of the class (e.g. $\overline{x}$ realization) not specifically investigated (Eq. (4)):

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

Since for the 3^n structures a capacity curve is available by SPO, similarly may be defined approximated functions for the strength (C_s) and the effective period (T) as in Eq. (5) and Eq. (6):

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

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

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 the points of the DoE are relatively far from each other, the linear or even quadratic regressions may not be appropriate for capturing the actual variation of capacity within the class. In other words, 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 may be to compute capacity for many cases defined by scanning the ranges of the X variables. This kind of dense experimental plan does not require fitting of 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.
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 $\overline{X}$ components; the density of the DoE has also to be calibrated accounting for the available computational capabilities.

2.2 Seismic demand

Demand *(D(*$\overline{X}$*))* refers to inelastic spectral analysis, in the sense of the Capacity Spectrum Method (CSM) [Fajfar, 1999]. 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 each spectral ordinate having 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 probabilistic distributions of pseudo-acceleration spectral ordinates $S_{a,e}(T)$; therefore the corresponding elastic displacement's PDF can be obtained by a deterministic transformation as in Eq. (7), where $\omega = 2\pi T^{-1}$:

$$S_{d,e}\left(T\right) = \frac{S_{a,e}\left(T\right)}{\omega^2\left(T\right)} \quad (7)$$

In order to evaluate the inelastic demand, the elastic spectral ordinates should be adequately modified by spectral amplification factors, $C_R(R,T)$ [Miranda et al., 2002]:

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

where $(S_{d,i}(T))$ is the demand for the structure; R is the strength reduction factor defined, for an elastic-perfectly-plastic oscillator, as the ratio of $S_{a,e}(T)$ times the mass m over its strength C_S:

$$R = \frac{S_{a,e}\left(T\right) m}{C_S} \quad (9)$$

In order to account for all uncertainties in computation of Equation (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. (10):

$$C_R = \hat{C}_R \, \varepsilon_{C_R} \quad (10)$$

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 Equation (11) depending on the vector $\overline{X}$ of structural random numbers (e.g. materials, members size, plan view geometry, etc.):

$$Z\left(\overline{X}\right) = C_d - S_{d,e}\left(T\right) C_R\left(R,T\right) \qquad (11)$$

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. (11).
In any single run, indicated by the ordinal *j,* the $\overline{X}$ vector is sampled (1) according to the marginal distributions of its components and a realization $\overline{x}_j = \left\{x_{1,j}, x_{2,j}, \ldots, x_{n,j}\right\}$ is obtained; (2) the capacity of the building defined by $\overline{x}_j$ may be retrieved by option A (the $\overline{X}$'s realization is plugged into the global capacity regression functions) or by Option B (just a local regression is made among points adjacent to the $\overline{X}$ realization and already analyzed by the dense DoE) and a particular $\left\{C_{d,j}, C_{s,j}, T_j\right\}$ set, e.g. a capacity curve, is retrieved; (3) the $S_{a,e}\left(T_j\right)$ distribution is sampled; (4) the $S_{d,e}\left(T_j\right)$ and R_j are retrieved as in Eq. (7); (5) the median $\hat{C}_R\left(T_j, R_j\right)$ factor is calculated; (6) the conditioned distribution of the residual $\left(\varepsilon_{C_R}\right)$ of C_R is sampled and the actual $C_{R,j}$ and the inelastic demand are obtained; (7) the capacity and the demand are compared to see if the collapse occurs in the *j-th* run. At the end of all runs the seismic risk of the class is estimated by the ratio of counted failures over total number of runs.

3. CAPACITY ANALYSIS FOR A CLASS OF R.C. BUILDINGS

At the urban level of analysis collecting buildings in a homogenous 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 [ATC, 1996]. 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 the nonlinear analysis of a series of buildings in the Design of Experiments plan. Then a specific structure has to be associated to any realization of the $\overline{X}$ vector of the DoE; to this aim a specific re-design procedure, based on a simulated design, has been developed. In this step a crucial aspect of knowledge of the structural characteristics of a building is the period of construction. Using all information obtained through reference to the codes, the design methods and the typical current practice at the time of construction, a group of structural characteristics typical of the buildings of a certain period and of a certain region can be determined. Specifically, through an examination of the codes enforced, specifications on the prescribed values of loads and material strengths, the minimum values of the dimensions of structural elements and of reinforcement amounts can be drawn up. More difficulties occur for the evaluation of the location of reinforcement and of the detailing solutions. For this reason, reference has to be made to the handbooks commonly adopted in the period and to the technical documentation of real buildings found in the archives of public administrations, building firms and professional offices. From the handbooks more accurate indications can be obtained regarding the design methods and the arrangement of reinforcement in the structural elements. Finally, the technical documentation of real buildings enables verification of the reliability of the data obtained from codes and handbooks, as it shows the design and construction rules actually adopted in practice. All this process assures that the set of buildings is representative of the class.
The steps of building re-design process may be summarized in three steps: (1) definition of geometric/structural model; (2) elements design; (3) definition of the mechanical model.
The procedure herein discussed refers to rectangular plan shape; nevertheless, it could be extended to other morphologic configurations. Non-linear analysis allows the retrieval of the seismic capacity of the building in terms of $\left\{C_d, C_S, T\right\}$. Among non-linear tools, SPO is an attractive solution for the trade-off of investigating the building seismic behaviour performing a comparatively simple, yet accurate, analysis. In particular, lumped plasticity models seem to be sufficiently accurate to assess seismic performance including different sources of deformability [Cosenza et al., 2002].

3.1 Re-design process

Adopting a three-dimensional mesh with variable module's linear dimensions a_x, a_y, a_z it is possible to reproduce a geometric model that is consistent with the global building dimensions L_x, L_y and L_z, as shown in Figure 2. Geometrical mesh discontinuities are explicitly considered; e.g. the number of stairs (n_s) and the length of the stair module a_s, and inter-story height of the first floor (a_{1z}). The latter may differ from a_z for structural (foundation level) and/or architectural reasons. The Modules' lengths (a_x, a_y, a_z) also depend both on architectonical and structural issues. 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* direction, n_{px} and n_{py} respectively (see Figure 2). In gravity load design (pre-code building) according to experience from field observations [Verderame et al., 2002] it is assumed that only the lateral plane frames exist in the short direction ($n_{py} = 2$). Conversely, when seismically designed buildings are concerned [Pecce et al., 2004] the number of plane frames in 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 column and weak column orientation.

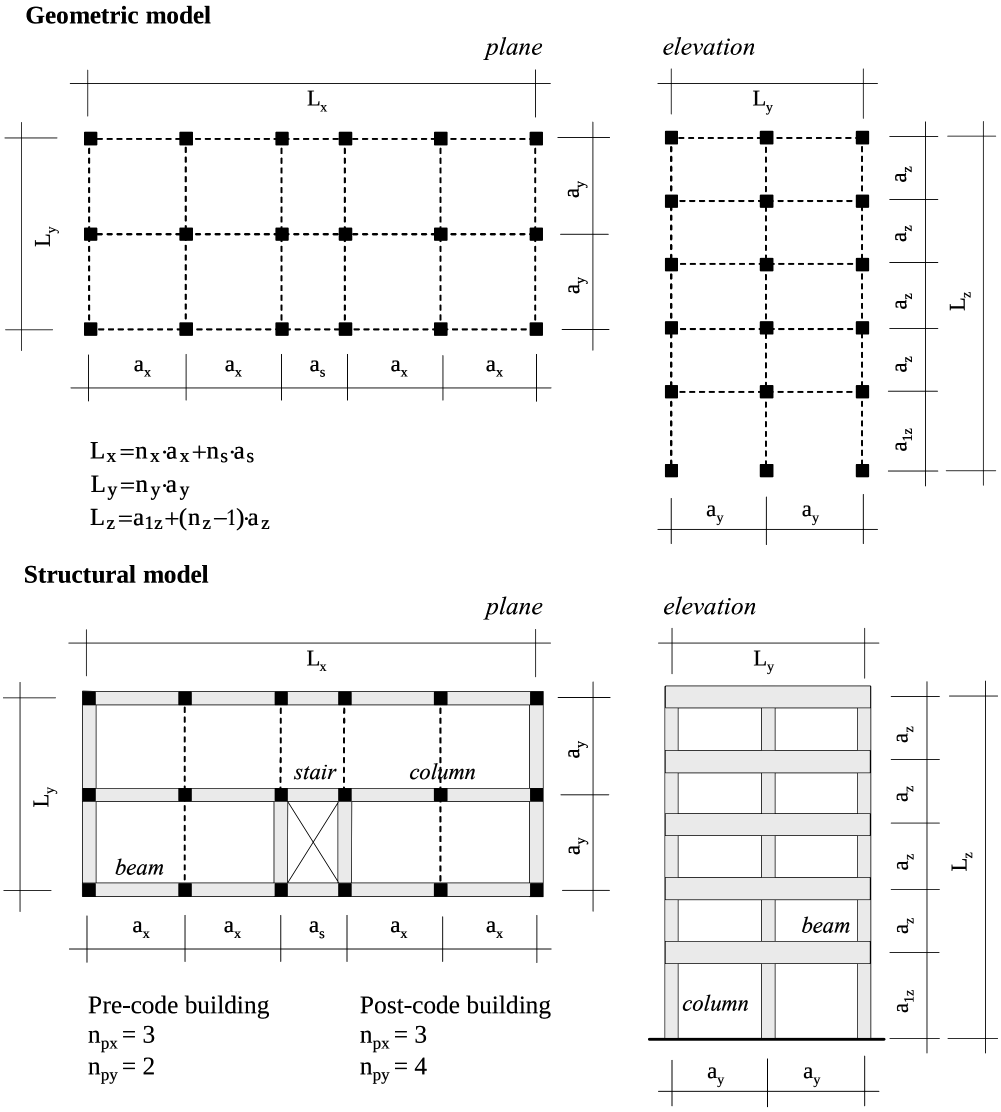

Figure 2: Building model: geometric mesh and structural model

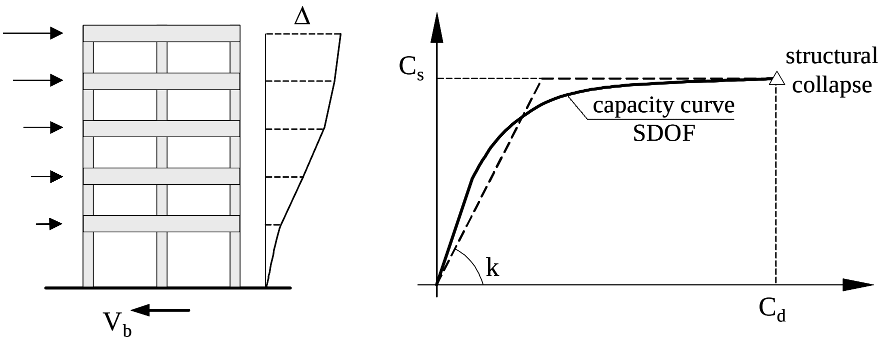

Figure 3: Capacity parameters

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, a number of rules affecting the entire design process have to be established: Definition of external loads; Definition of analysis model; and Material design characteristics. External loads are assigned depending on whether seismic or non seismic design is concerned. Definition of analysis models is strongly influenced by common practice, rules and manuals adopted at the age of construction. In gravity load design (pre-code building) an element level analysis model was generally adopted (e.g. axial load for the columns, simple bending for the beams).
Regarding seismically designed buildings, it was a 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 Non-linear 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 drawn. Existing mechanical and/or experimental capacity models [Fib, 2003] allow defining yielding and ultimate rotational capacity for the elements; in particular, the yielding (θ_y) and ultimate (θ_u) rotations were computed as proposed by Panagiotakos and Fardis [Panagiotakos and Fardis, 2001]. The influence of shear action was modelled based on a reduction of the shear strength depending on the local ductility, expressed in terms of rotation varying with a linear trend [Priestley et al., 1994]. 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 non-linear strength $C_s\left(\overline{X}\right)$; the displacement capacity $C_d\left(\overline{X}\right)$; and effective period $T\left(\overline{X}\right)$, as shown in Figure 3, where $\overline{X}=\left\{L_x, L_y, ..., f_{sy}\right\}$ is the vector of parameters the limit state function depends on, as listed in Table 1.

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*		

4. APPLICATION

The procedure presented in the previous sections has been applied to compute total risk for R.C. buildings 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 recently classified as seismic, according to last Italian hazard map [OPCM 3431, 2005]. On the other hand, many seismically designed constructions reflect old codes, [R.D.L. 2105, 1937], [R.D.L. 640, 1935],[Legge 1684, 1962], which do not account for *capacity design* rules.

Hazard curves, computed by PSHA, used in this application reflect the seismicity of the site where the classes are supposed to be located. In Figure 4 selected curves for several T values class are given referring to the exceeding probability in fifty years.

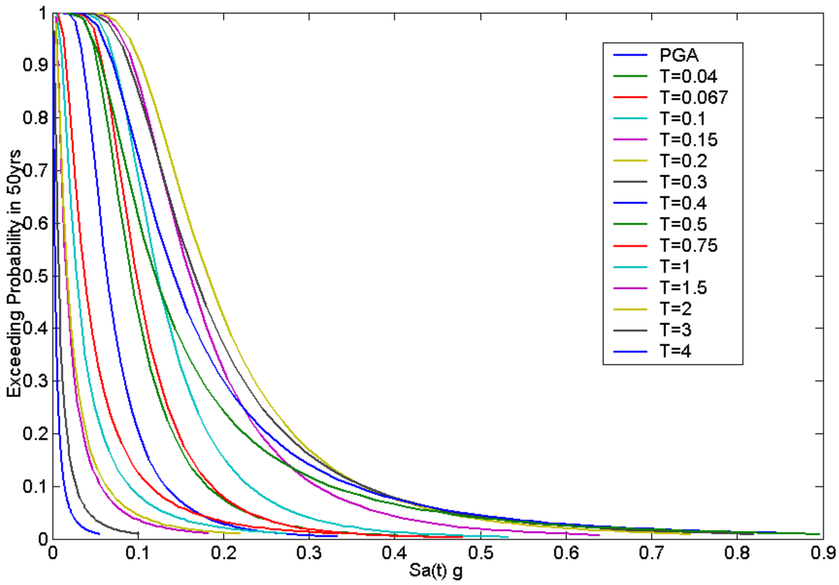

Figure 4: Seismic hazard of this study

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 25.0m (10.0m) and a Coefficient of Variation (CoV) of 12% (6%). These values are chosen based on the results of field surveys carried out in Campanian region [Verderame et al., 2002], [Pecce et al., 2004] and expert judgements.

The height is determined by the number of storeys and assuming a_z =3.00 m and a_{1z} =4.50 m for all the classes. Hence for 3, 4 and 5 storey classes building height L_z is 10.5m, 13.5m, 16.5m 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. Accomplishing the design practices 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 25N/mm^2 and 400N/mm^2 and CoV 25% and 15% respectively [Verderame et al., 2001a], [Verderame et al., 2001b].

Starting from these assumptions 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.

Table 2. Design of Experiments details

Variable	Range	Scanning step
L_x	[15-32]	1.0m
L_y	[8-12]	1.0m
f_c	[5-45]	10MPa
f_{sy}	[200-600]	50MPa
a_x, a_y	[3-5]	See compatibility equations of Figure 2

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}

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 points ($C_d\left(\overline{X}\right), C_s\left(\overline{X}\right), T\left(\overline{X}\right)$) 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 Table 3.

The given results show that, even 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 in 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 only be taken as an example of the application 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 the approach of structure-specific reliability methods. The mechanical evaluation of the seismic capacity terms allows accounting for uncertainties related to seismic response, avoiding 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, even though the distributions of the parameters do not reflect a real case, is useful to discuss the procedure. The expected fraction of failures refers to three to five storey R.C. buildings with pre or poor seismic design. The hazard analysis refers to a moderate seismicity site in southern Italy. Results show that the seismic code design, even not considering the capacity design philosophy, lowers the risk of an order of magnitude in respect to pre-code buildings.

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