Seismic risk analysis of R.C. building classes

Iunio Iervolino, Giovanni Fabbrocino, Gaetano Manfredi, Maria Polese, Gerardo Verderame
Department of Structural Analysis and Design, University of Naples Federico II, Naples, Italy
Via Claudio 21, 80125, Naples – Italy. Tel. +397683488 – fax +397683491 email: iunio.iervolino@unina.it

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ABSTRACT: Seismic risk assessment at urban scale may be defined as estimation of number of buildings expected to reach a given limit state in the region and time period of interest. It is related to the failure probability of a homogeneous class of buildings and its characterization. This definition introduces new issues in respect of reliability analysis for single structures which, under some conditions, has been already addressed in close form. Mechanical approach may be suitable in respect of the requirements of such territorial evaluation, thus appropriate limit state functions have to be developed to get the failure probability of classes belonging to the building stock. Non-linear seismic capacity, in terms of base shear and/or displacement, has to be expressed as function of those structural parameters which identify single structures within the class. The latter may be carried out by multiple regression analysis. Distributions of random variables affecting capacity are estimated by sampling the territorial population. In a force-based approach inelastic seismic demand may refer to probabilistic seismic hazard analysis at the site and strength reduction factors. All these issues are discussed in the paper; moreover, an explanatory application to R.C. structure classes is presented.

1 INTRODUCTION

Probabilistic assessment of structural seismic risk consists of integration over the product of estimated seismic hazard and fragility curves; the latter have to be expressed as a function of the same ground motion measure the hazard analysis refers to (Jalayer and Cornell, 2003). Several options are available to engineers to carry out this job in numerical form (Cornell, 2002).

Currently, in the case of regional/urban scale, risk assessment often refers to vulnerability data retrieved by post earthquake damage surveys. Such empirical approach is largely adopted worldwide; in Italy, for example, 1st and 2nd level methods issued by National Group for the Defense against Earthquakes – GNDT are utilized (Benedetti e Petrini, 1984; CNR-GNDT, 1994; Di Pasquale et al., 2001). However, accuracy of empirical methods may be affected by unavailability of comprehensive database of damage observations, which basically consists of collecting heterogeneous structural data. This circumstance can reduce helpfulness in risk-based decision making; for example vulnerability data contain also information about soil-structure interaction which may be hard to disaggregate. Finally some empirical vulnerability studies are not developed specifically for seismic risk computation including PSHA since damage probability is dependent on macro-seismic intensity scales.

An alternative approach to seismic risk analysis is represented by HAZUS methodology (HAZUS, 1999). It provides vulnerability functions for categories of structures depending, for example, on the design code and the age of construction. A bi-linear capacity curve, associated to each category of the building stock, 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 appears to be optimized for *scenario analysis* (e.g. for a given ground shaking level) rather than for *risk* evaluations including seismic hazard at the site.

As a single structure is concerned, SAC FEMA method (Cornell et al. 2000) represents the present state of the art in terms of *total risk* assessment. It is based on the numerical evaluation of the probabilistic seismic demand by means of Incremental Dynamic Analysis (Carballo 2000; Vamvakistos and Cornell, 2002). Level of input information required by this procedure fits structure-specific problems, but is hardly applicable to a urban scale analysis.

Lack of information about classes of structures may be related to the development of the so called "semi-quantitative" methods (Calvi, 1999); they refer to simplified mechanical models which need a limited number of input data, in compliance with territorial scale computation requirements.

Herein, the formulation of a method for seismic risk analysis intermediate to those discussed above is proposed; it is based on the definition of mechanical capacity functions for classes of buildings and considers inelastic spectral analysis for demand estimation. This approach allows to explicitly accounting for several uncertainties related to both seismic response and structural damage phenomena, avoiding the shortcomings of empirical vulnerability analysis. On the other hand, spectral methods for seismic inelastic demand analysis ensure a computational effort appropriate to the scale of the problem and to the level of information generally available. In a force-based approach inelastic seismic demand is derived from PSHA in terms of elastic spectral acceleration reduced by an appropriate force reduction factor

Capacity analysis is carried out by a multiple regression approach (Iervolino et. al, 2004a; Cosenza et al. 2005) to express capacity as a function of the parameters of interest. Such approach extends the mechanical assessment methods for single structures to a class of buildings representing, therefore, an intermediate level between structure-specific reliability analysis and urban scale semi-empirical seismic risk.

Seismic capacity and demand are estimated accounting their variability; virtually any source of randomness or epistemic uncertainty may be included.

An explanatory application to R.C. building class based on a strength limit state function is presented; it gives an overview of the possible results of such an evaluation. In fact, any other kind of safety assessment (i.e. capacity spectrum or displacement-based) can be used for risk evaluation and easily implemented.

2 FORMULATION

Urban scale seismic risk may be defined as the expected fraction of structures exceeding a given limit state during an observation period. Then, it is worth to explore a formulation of the problem for numerical evaluation of the probability of the capacity being exceeded by seismic demand for a class of buildings (Iervolino et al., 2004b).

In this case uncertainty related to the capacity is larger than in the case of single structures; therefore, capacity analysis may be particularly relevant. Demand refers to an inelastic spectral analysis rather than estimated by non-linear dynamic as in the method proposed by Cornell and his co-workers.

Elastic spectral accelerations ($S_{a,e}$) in the period-range of the class are computed by PSHA. The latter basically allows getting a hazard spectrum.

In order to evaluate inelastic demand, PSHA related spectrum for the region of interest must be adequately reduced by spectral modification factors (R) depending also on ductility (μ).

$$D = \frac{S_{a,e}(T)}{R(T,\mu)} \tag{1}$$

In Equation 1 D is the demand for the structure featuring equivalent period of oscillation T and ductility μ. Therefore D depends on the two random variables $S_{a,e}$ and R; their probabilistic distributions have T and μ as parameters.
If the strength capacity of the structure is indicated as C_s the limit state function can be written as in Equation 2

$$Z = C_s - \frac{S_{a,e}(T)}{R(T,\mu)} \tag{2}$$

where capacity depends on the vector ($\overline{X}$) of structural random numbers (e.g. materials, members size, plan view geometry, etc.). Period and ductility may depend on $\overline{X}$ as well or, possibly, on its sub-vectors.

Alternatively, if C_d is displacement capacity rather than strength Z is written as in Equation (3) where $S_{d,i}$ is the inelastic displacement demand.

$$Z = C_d - S_{d,i}(T,\mu) \tag{3}$$

In such cases $C(\overline{X})$ has to be a function able to provide capacity for any value of $\overline{X}$ in the domain of interest describing, therefore, a *class of buildings*. Details about such functions calculation (e.g. by multiple regression) are given in the following sections.

2.1 *Computation of risk*

In the region of interest probability of Z being non-positive (total risk) may be interpreted as the expected number of structures to not survive the time period the PSHA refers to. Then, risk is given by the integral over the failure domain (Ω) where Z is non-positive of the joint probability distribution function of Z (f_z):

$$P_f = \int_{\Omega} f_Z(C, S_{a,e}, R)\, d\Omega \tag{4}$$

If components of $\overline{X}$ are stochastically independent and their probabilistic distributions are retrieved by sampling surveys on the building stock, then the joint PDF of Z is given by the product of the marginal distributions of random variables. Simulation methods (i.e. Montecarlo analysis) may be helpful in the matter and $P[Z \leq 0]$ may be estimated as

$$P[Z \leq 0] = \int_{\Re^k} I(Z) f_Z \, dv \approx \frac{1}{N} \sum_{i=1}^{N} I(z_i) \qquad (5)$$

where, k is the dimension of the space of the random variables; N is the total number of samples from the distributions of random variables Z depends on and $I(Z)$ is an indicator function equal to 1 if $Z \leq 0$ and equal to 0 otherwise.

It is worth noting that Z depends on the probabilistic distribution of the spectral acceleration, thus the equivalent oscillation period of the structure in the class plays a relevant role. In fact, assuming that all the building in the class have the same fundamental oscillation period the probabilistic distribution of the spectral acceleration in Z is the hazard corresponding to that period. Otherwise, since period of the structures in the class varies, the PDF distribution of $S_{a,e}$ changes with the period. Therefore, in the *i-th* run of simulation the total probability theorem allows to write:

$$P[S_a > a] = \sum_j P[S_{a,e} > a \mid T = T_j] P[T = T_j] \qquad (6)$$

Where j indicates all the possible values of T. Then, in each run of the Montecarlo, T is an indicator variable being 1 if $T = T_i$ and 0 otherwise. Therefore one is allowed to sample alternatively form hazard corresponding to T_i ($P[S_{a,e} > a \mid T = T_j]$) in that run without introducing any further complication. The same applies to the reduction factor.

3 CLASS-SCALE CAPACITY OF BUILDINGS

Variability of morphologic, geometric and structural configurations within the building stock is large when analysis of building vulnerability at a territorial scale is concerned, so it has to be carefully taken into consideration in the assessment procedure. In this framework, the collection of buildings in homogenous classes helps to reduce epistemic uncertainty of some variables and to guide the analysis towards more useful results.

Several criteria to define building classes may be adopted depending on the scope of such categorization. A common prerequisite is the simplicity of attribution of a building to a class, thus the classification should be based on global parameters available at a large scale.

In this paper, the definition of building classes is based on considerations that are typically related to vulnerability assessment methods. In particular, a typological criterion that strictly links a building class to the relative seismic capacity is chosen.

First of all, in order to reduce epistemic uncertainty on capacity, the entire building stock is divided in morphologic macro-classes. This preliminary classification is based on parameters that are easily available at the territorial scale and that are contemporarily relevant for the seismic capacity.

Here only rectangular morphologic shape and 3D frame structural type are addressed; this choice however is not related to intrinsic limitation of the method that can be extended to other structural configurations.

The analyzed building class is defined depending on construction age and number of storeys. The former enables to relate constructions of a certain area to building codes adopted at the time of construction, and consequently to design actions, rules and practice (e.g. pre and post-seismic code). The latter has been selected due to its direct influence on seismic capacity. Building class capacity is determined in terms of base shear coefficient C_s and displacement at the roof level Δ.

3.1 *Building class generation*

Mechanical approach to seismic capacity analysis leads firstly to the selection of an effective model for the building class analysis. It is dependent upon the type and amount of input data available for such models and its level of sophistication may increase, affecting the accuracy of the expected results.

Usually the scale of the problem and the limited financial resources do not allow collecting as many data as necessary required by structure-specific analysis. In response to this issue, some relevant approaches have been proposed that allow a first estimate of building class capacity depending on poor parameters (Calvi, 1999; FEMA, 1999; Whitman et al., 1997).

On the other hand, the seismic behavior of buildings may be significantly affected by a number of factors that cannot be accounted for in an over-simplified model. Referring to R.C. frame structures, for example, brittle failure of short columns or beams, shear cracking of beam column joint (panel node zone), structure stiffening due to stairs, are all factors that can influence the overall response to horizontal actions, and that cannot be managed using very simplified methods. At the same time, the improved modeling features for structural components and systems, accompanied by the enhanced power of new generation of computers, make the use of detailed analyses more attractive.

Pushover analysis is probably the best compromise between the need to investigate building's non-linear behavior and to perform a relatively simple, yet accurate, static analysis. In this framework lumped plasticity models seem to be enough accurate to assess global system behavior and allow to include different sources of deformability (Cosenza et al. 2002). For a complete model characterization, then, elements dimensions and reinforcement, as well as material properties (steel and concrete strength and strain capacity), are required.

Due to the classification of buildings according to their age, structural system and morphology, however, certain homogeneity can be recognized among structures belonging to the same class, being designed with the same codes and practice rules.

Based on such assumption, a specific generation procedure for a building class has been implemented; it allows to (re)design buildings of a class and developing the relative model starting by few geometric/structural and mechanical parameters.

The generic building is generated and (re)designed adopting the following procedure: firstly the *geometric* model is defined by a modular grid in the main directions, then the *structural* system fitting such mesh is located and the *elements* (beams and columns for a moment resisting frame system) are *designed*. In order to complete modeling process it is often necessary to rely on engineering judgment, and on the knowledge of design codes adopted at the time of construction.

Following the process which is at the base of building design it is possible to trace its main steps and to select the model input variables that have a discriminative role. The steps are listed below (Cosenza et al., 2004; Cosenza et al., 2005):

- Definition of geometric/structural model;
- Elements design;
- Mechanical model.

3.2 *Geometric/structural model definition*

Given rectangular building morphology a three-dimensional mesh in the three main directions x y z is identified, and elements that play a role in the horizontal bearing system (columns and beams) are located. In particular, structural model identification depends on the choice of a number of parameters that allow the clear definition of dimensions and structural mesh-grid of the building. Adopting a 3D mesh with variable module's linear dimensions a_x, a_y, a_z it is possible to reproduce a geometric model that is globally compatible with building dimensions L_x, L_y and L_z (see Figure 1); at the same time the replication of x, y and z module, defines a structural mesh of a number of n_x, n_y and n_z modules.

Generally, for each geometric model having global dimensions L_x, L_y and L_z it is possible to consider a number of structural models depending on the combinations a_i times n_i. In the geometric/structural model adopted herein, beam number and position are determined referring to the number of plane frames in x and y direction n_{px} and n_{py}. Finally, another significant parameter for the definition of the structural model is the main column orientation. In order to evaluate main orientation effect, two limit schemes are adopted considering for each direction x and y the extreme situations ($OR = 1$) strong column ($OR = 0$) weak column orientation. Main column orientation effect may be more significant for tall buildings, where transversal sections of the bottom floor columns may be very deep.

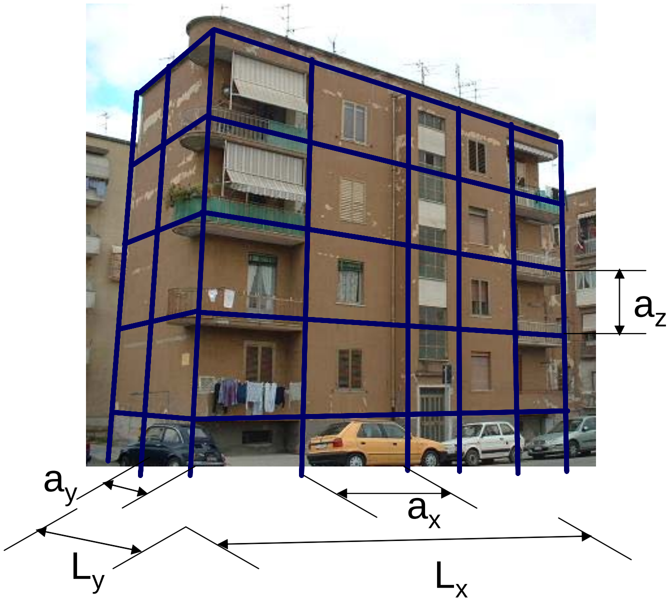

Figure 1. Typical R.C. building generated.

3.3 *Members design*

The structural elements (columns and beams for a moment resisting R.C. frame) that represent the horizontal bearing system and have been identified in the previous step are designed (element cross section and reinforcement) according to prescribed code rules (seismic or non seismic code) and design practices related with the construction age, together with common detailing criteria at the territorial scale. Material properties selected for design derive from prescribed codes and considering steel and concrete types commonly used at the age of construction. Moreover, code principles and manual's rules help in the establishment of minima transverse section dimensions and prescribed reinforcement percentage.

3.4 *Mechanical model*

Lumped plasticity model for the elements is considered. Allowable strength and deformation for the structural elements are established depending on material properties (concrete compressive strength f_c and steel yielding strength f_{sy}) and on member model. The global seismic capacity, in terms of lateral strength and deformation, is determined with non-linear pushover analysis.

The described procedure enables to generate a building model with the aid of few selected parameters, $\overline{X} = \{L_x, L_y, ..., f_y\}$, as listed in Table 1.

The proposed procedure exploits potentialities of non-linear static analyses, thus accuracy of results depends basically on quality and quantity of inventory data; eventually multi-level analyses can be carried out depending on amount of data and on availability of funds for large scale relief. Similarly, defeats, deterioration or even unsatisfactory design can be accounted for.

Table 1 Building model parameters.

MODEL PARAMETERS		
Geometric	Plan dimensions	L_x , L_y
	Elevation data	
	height	L_z
	number of storeys	n_z
Structural	Bay length in x, y direction	a_x, a_y
	Number of x and y plane frames	n_{px}, n_{py}
	Column orientation	*OR*
Mechanical	Material properties	
	concrete	f_c
	steel	f_{sy}

3.5 *Capacity evaluation*

The generation process and the evaluation of seismic capacity described in the previous sections allows to reproduce a generic single building model and to determine its seismic supply in terms of lateral strength (intended as base shear coefficient C_s) and displacement at the roof level, Δ. In particular, for each building model pushover analysis is performed. Next, the MDOF-SDOF equivalence and the transformation of the capacity curve in a bilinear form (Fajfar, 1999), allows to estimate system's fundamental period T and ductility μ, as illustrated in Figure 2.

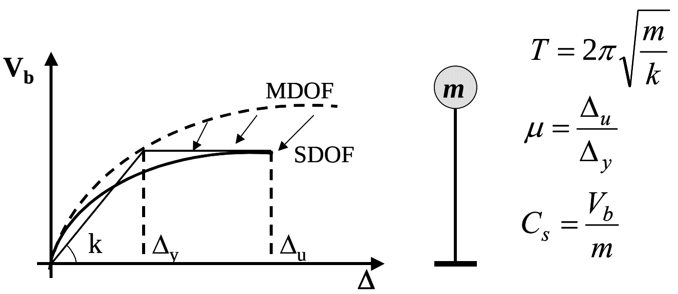

Figure 2. MDOF-SDOF equivalence.

Obviously, when the vulnerability analysis is performed at a territorial scale, a number of building models reproducing the building stock have to be generated and their response investigated.

Referring to a building class it is possible to determine the variation (distribution) of the relative model parameters in the investigated territorial area. With the established parameters distribution, a number of building models are generated for the class, which constitutes the base for capacity analysis. The sample building stock artificially reproduced is analysed and seismic capacity is determined.

The relation of the model generation parameters with building class capacity, next, is evaluated by multiple regression analysis. In particular, for either C_s, T and μ (dependent variable Y_i with $\{i = 1,2,3\}$, analytical representation in the form:

$$Y_i = b_{o,i} + \sum b_{j,i} \cdot X_j \tag{7}$$

is built as a function of the independent input variables X_j. Other possible regression functions could be adopted, such as higher orders polynomials, exponentials etc. (Vitali, 2000)

4 SEISMIC DEMAND

Limit state in Equation (2) refers to base shear capacity (C_s) compared to an inelastic force demand represented by elastic spectral acceleration divided by reduction factor. Acceleration corresponds to the spectral ordinate at the period of the structure which is determined by a realization of vector $\overline{X}$. Therefore probabilistic characterization of $S_{a,e}$ is given by the well known Probabilistic Seismic Hazard Analysis (McGuire, 1995). A large number of hazard curves have to be available to perform a study as the one herein presented. Virtually, curves corresponding to all possible oscillation periods in the class have to be computed. Hazard curves used in the application described in the following section reflect PSHA (Convertito V., 2003) for a real, *moderate-seismicity,* site in the southern Italy region where the building classes are supposed to be located. In Figure 3 selected curves for several values of T are given referring to the exceeding probability in fifty years. In Figure 4 Uniform Hazard Spectra are given for 10% and 2% exceeding probability in 50 years by acceleration ordinates at $T = \{0s, 0.5s, 1s, 3s, 4s\}$.

The force reduction factor (R) or *response modification factor* allows estimating the demand force of a structure. Several expressions are given in literature about mean R-factors.

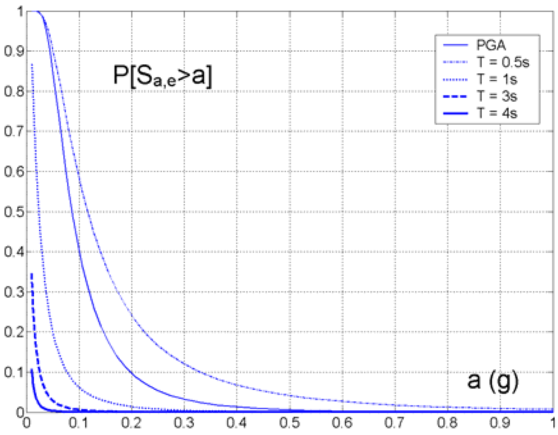

Figure 3. Selected hazard curves used in this study.

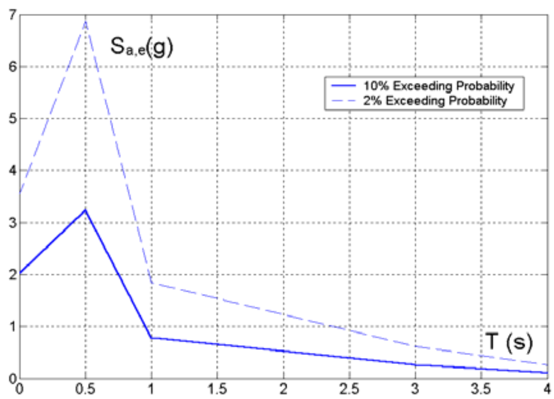

Figure 4. Uniform hazard spectra.

However, in order to account for all uncertainties in computation of failure probability by Equation 2 the variability of the force reduction factors depending on ground motions have to be included. Since the scattering is so large, only the mean values of R are not sufficient. Therefore, in the present study R suggested by Watanabe and Kawashima is used. Those authors provide expression for mean-R and standard deviation (σ_R) as a function of ductility. Statistic of scattering of R around its mean at given μ is considered to be 0-mean Gaussian with σ_R^2 variance. In particular R and its variation, σ_R are determined by the Equations 8 – 10 where coefficients a and b depend on soil conditions.

$$R = (\mu - 1)\Psi(T) + 1 \tag{8}$$

$$\Psi(T) = \frac{T - a}{ae^{bT}} + 1 \tag{9}$$

$$\sigma_R = -0.3 + 0.4\mu \tag{10}$$

5 APPLICATION

The procedure presented in the previous sections has been applied to compute total risk for R.C. buildings classes located in a medium to high seismic zone in the Campania Region, southern Italy. In particular, the example refers to gravity load designed buildings, which represent the majority of the building stock in many areas that have been recently classified as seismic, according to last Italian Seismic Code (OPCM 3274, 2003). Hence, only *pre-seismic code* R.C. frames are considered; in particular, 3, 4, 5 and 6 storey classes have been studied. It is worth noting that data used in the present application are obtained from limited building inventory data, so that they cannot be representative of Italian or even Regional R.C. building stock. This circumstance does not reduce the significance of the present section, since basically operational information is of interest.

Model parameters distributions are assumed to be the same for the 4 building classes. Mean base plant dimensions L_x and L_y are 33 m and 11.6 m while the relative C.O.V. are 27% and 18%, respectively. Regarding mean bay length a_x and a_y they are assumed to vary between 3.00 and 5.00 m. Main column orientation OR is an indicator variable $\{0,1\}$ (0 representing weak column orientation, and 1 strong). Finally, concrete and steel strengths are normally distributed with mean 25 and 370 N/mm^2 and C.O.V. 30% and 11% respectively.

Because buildings are gravity load designed and considering the plant shape morphology it is assumed that plane frames in short direction exist only along structure's perimeter (n_{py} = 2). Finally, constant inter-storey height is adopted (a_z = 3.00 m).

Based on parameters distributions a number of building models are generated and seismic capacity in terms of C_s, T and μ for the equivalent SDOF (see Figure 2) are determined. Starting from this artificially reproduced 'capacity' sample, regression analysis is performed, and formulation of $C_s(\overline{X})$, $T(\overline{X})$ and $\mu(\overline{X})$ depending on $\overline{X}$ is found (Equation 7). For each realization $\overline{x}$ the capacity terms $C_s(\overline{x})$, $T(\overline{x})$ and $\mu(\overline{x})$ are evaluated trough Equation 7; this allows to determine elastic spectral demand S_{ae} and reduction factor R as a function of $T(\overline{x})$ and $\mu(\overline{x})$. Failure probability, next, may be evaluated through Equation (2), (5), by Montecarlo simulation.

In Figure 5 to Figure 7 building class 'capacity curves' are shown for the 3 output parameters C_s, T and μ. They represent the percentage of buildings, within a class, whose capacity is under fixed levels. Referring to Figure 5 it can be noted that non linear strength C_s for building class decreases with the increase of the number of storeys (i.e. for different classes). This is due to a global rising of the bared mass, that is not compensated by relevant system strengthening.

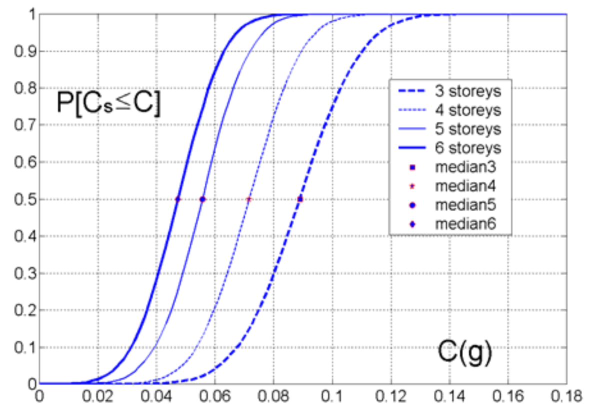

Figure 5. C_s capacity curve for 3, 4, 5 and 6 storey classes.

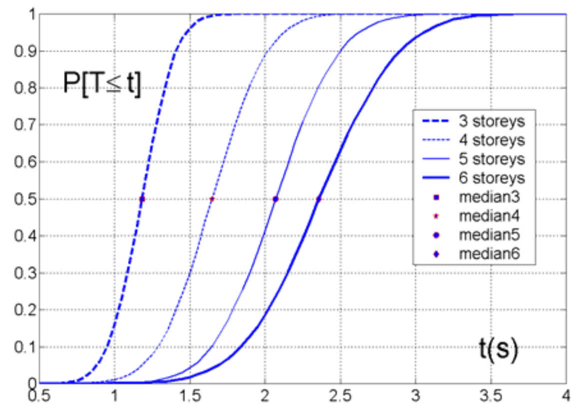

Figure 6. T curve for 3, 4, 5 and 6 storey classes.

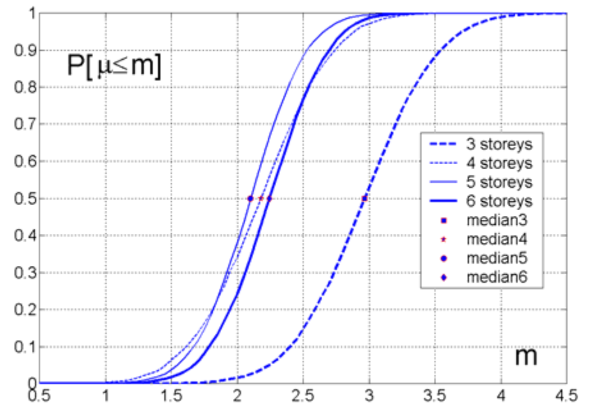

Figure 7. μ capacity curves for 3, 4, 5 and 6 storey classes.

Inverse trend is observed for system equivalent period T, which dispersion tends also to increase with the storey number. Mean 'equivalent' period for each class is obviously higher than elastic period as suggested in current codes (period-height relationships). In fact, 'equivalent' stiffness k (see Figure 2) is smaller than the elastic one.

Regarding displacement ductility it can be noted a fair homogeneity among 4, 5 and 6 storey classes. The difference observed for 3 storey class can be related to the relevance of code minima for member cross sections and amount of reinforcement that have to be taken into consideration in the case of low-rise buildings.

Therefore, it can be confirmed that the storey number is a good parameter for the class definition: first of all for its easy definition; secondly for its direct influence on capacity parameters T and C_s. Moreover, the median values of building capacities in terms of C_s, T and μ, for the analyzed building classes, as well as their variation ranges, fully comply with typical values for under-designed buildings (Cosenza et al., 2002, Verderame et al. 2000, Crowley and Pinho, 2004).

Failure probabilities P_f are given in Table 2; the risk is comparable for all classes even though the storey number strongly affects capacity. This is due to the joint effect of probabilistic seismic hazard analysis for the site and of distribution of C_s, T, μ parameters.

Table 2. Classes risk.

Storey number	Class Failure Probability in 50 years
3	0.0040
4	0.0100
5	0.0112
6	0.0096

6 CONCLUSIONS

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

Probabilistic characterization of the demand is given by PSHA and scattering of force reduction factor due to ground motion variability.

Evaluation of capacity at the territorial level can be carried out with reference to entire building stocks that can be (re)designed and analysed using a specifically developed computer program based on inventory data, e.g. geometric, structural and mechanical parameters. Such mechanical evaluation of the seismic capacity terms allows accounting for uncertainties related to seismic response, avoiding some limitations of empirical vulnerability analysis.

Reduction of epistemic uncertainties about structural configuration is achieved by the definition of homogeneous building classes depending on factors directly affecting the seismic capacity (e.g. morphology, construction age (pre and post-seismic code) and/or the number of storeys).

Multiple regression of capacity on vector $\overline{X}$ components can be used whenever the regression scatter is negligible compared to $\overline{X}$ uncertainty. However, different options are available for risk computation if such an assumption does not fit the data set, e.g. 'exact' capacity computation within each Montecarlo run.

Limit state function is written in terms of base shear capacity and demand; however, this assumption is not limiting and others assessment methods (i.e. Capacity Spectrum Method) may be easily implemented in the procedure.

The simple application, even though not real (uncertainties are assumed and not retrieved by population sampling), is useful to understand and discuss the procedure operatively.

Although this limitation, it worth to notice that the building generation and analysis procedure gives capacity results which are expected for those buildings types which are investigated. The computed risks does not refer to a real case; therefore, failure probabilities are consistent with the probabilistic seismic hazard analysis, which has been specifically developed for a site in the Campania region, and with the assumed distributions of inventory parameters. Due to sensitiveness of the spectral hazard curves to period *T*, it is important a careful evaluation of this structural parameter.

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