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RELIABILITY METHODS FOR TERRITORIAL SEISMIC RISK ASSESSMENT

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SUMMARY

The goal of territorial seismic risk assessment is a probabilistic estimate of portion of construction stock reaching a selected limit state or even collapse in the region of interest during a reference time window. This result can be obtained crossing structural vulnerability and seismic hazard. Often the first issue is analysed on the base of empirical fragilities retrieved by structural damages observed during post-earthquake damage recognitions. An alternative approach is based on mechanical modelling of constructions similarly to single structure oriented numerical methods for seismic risk assessment. These methods generally require detailed information about the structure and development of non-linear dynamic analyses resulting inappropriate at the territorial scale. Therefore, development of simplified models for probabilistic capacity and demand analysis for classes of buildings is certainly of interest in the matter. Demand may be estimated by Probabilistic Seismic Hazard Analysis and probabilistic spectra reduction factors, while response surface based methods can be useful for the seismic capacity analysis of structural classes. The objective of present paper is the definition of a limit-state function for the seismic risk assessment developed specifically for the territorial case, and completely described probabilistically for quantitative computation. The feasibility of such method is discussed in terms of accuracy and uncertainty propagation.

INTRODUCTION

Seismic risk analysis at urban scale is commonly carried out utilizing hazard estimation, generally based on Probabilistic Seismic Hazard Analysis (PSHA) [1, 2] and vulnerability data coming from empirical approaches. This kind of methods, even if largely adopted worldwide (i.e. in Italy 1st and 2nd level methods issued by National Group for Seismic Prevention [3, 4, 5]) may be questionable in terms of both efficiency and accuracy in case of use for seismic risk estimation.
In fact, vulnerability estimation reflects availability of extended database filled by field post-earthquake damage observations.
Consequently, heterogeneous structural information is generally collected, so that structural significance and predictability are not satisfactory. They depend strictly on the type and nature of the analysed building stock, so they cannot be exported to other geographical and hazard areas. In addition, this vulnerability inferred data already contain information about interaction of structures with ground shaking

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and soil structure interaction which are hard to separate later. Finally, empirical vulnerability studies are not developed specifically for crossing data with PSHA outcomes; in fact, they are often referred to macro-seismic intensity or converted to peak ground acceleration by formulae carrying in more uncertainty; while field campaigns, that are the basis of such studies, are useful to infer territorial distribution information about structural features.

An advancement is represented by HAZUS methodology [6], developed by Federal Emergency Management Agency (FEMA), which gives the vulnerability for categories of structures classified according to design code, construction type and age. A bi-linear "capacity curve" is associated to each category taking into account some uncertainties and engineering judgement. These curves are crossed with inelastic spectral demand to get the "performance point" providing lognormal fragility curves for the class of buildings.

The HAZUS loss assessment procedures is not however developed to carry out *total risk* but *scenario analysis*; in other words, ground shaking level is usually given and the losses are computed crossing fragility with territorial information by Geographic Information System (GIS) data.

At a lower scale, SAC FEMA method [7, 8, 9] represents the state of the art in terms of seismic risk assessment of single structure. It is a purely numerical approach based on demand assessment through non-linear dynamic analysis. Detail level required by this method is fairly high and hardly applicable to a higher scale analysis where the critical factor is using as low of information level as possible.

This is why recently so called "semi-quantitative" methods have been proposed [10, 11]; they make use of simplified mechanical models that need a limited number of input data, in compliance with territorial scale computation requirements.

In the present paper, the concept of class is given as the group of structures which can be described by the approximated function of the same vector of variables measured by field survey; details about models dealing with this definition for reinforced concrete structures may be found in [12].

Herein, the formulation of a method for seismic risk analysis intermediate to those above discussed is proposed; it is based on the availability of mechanical capacity functions for classes of buildings and refers to inelastic spectral analysis.

The main advancement is represented by numerical demand and capacity analysis, the latter allow to explicitly account for several uncertainties related to both seismic response and structural damage phenomena, avoiding the limitation given by empirical vulnerability analysis; on the other hand simplified numerical models for seismic inelastic demand analysis ensure a computational effort appropriate to the scale of the problem and to the level of information generally available.

Capacity analysis may be carried out by a response surface approach to express capacity as function of parameters of interest, and then simulation (or any other equivalent reliability method) is performed to get the probability distribution of capacity. Seismic demand is given by PSHA in terms of elastic spectra (force or displacement) modified by factors carrying on their own uncertainty given by regression of SDOF inelastic analysis by real records.

MODELING OF SEISMIC RISK

Seismic risk quantitative analysis

Seismic risk is defined as probability measure, which it is the consistent mathematical quantification of the knowledge level and predictability of a certain phenomenon, of the loss amount of a catastrophic event such as earthquake. This definition leads to subjective probability meaning that it is variable depending on the observer uncertainty in the comprehension and control of the experiment/event.

When structures are concerned seismic risk may be defined as the failure probability in a given place during a given time interval. This definition is the seismic engineering transfer of industrial reliability concept which consists of the probability that a given engineered system accomplishes its mission, say surviving at the end of the time interval elapsed.

Requisite of the probabilistic approach is given by significant uncertainty associated to events and ground motion levels. Commonly, seismic signal randomness, and then structural demand, is considered as predominant upon capacity variability. This is particularly applicable when a single structure is

concerned: in such case it is possible to demonstrate that the order of magnitude of risk analysis results are driven by hazard [7].
At territorial scale this result may no longer apply; in fact, uncertainty on the capacity increases since an entire class of structures is considered at one time; in this case probabilistic capacity analysis may be of some significance.
For building classes, conversely, non-linear probabilistic demand analysis cannot be performed by non-linear dynamic analysis and/or Incremental Dynamic Analysis [13]; the latter may be computationally unaffordable if the structures are not standardized, unfit available information for each structure in the considered class or lead to poor representation of the whole buildings class using a single function.
Therefore, use of spectral demand analysis reducing elastic spectra by inelastic reduction factors seems of interest in the matter. Critical points of such an approach are related to the class definition, to the development of structural analysis methods and assessment of uncertainty associated to all the terms of the class wise limit state function.

Time-invariant formulation
Structural reliability, for seismic risk analysis purposes, defines a *limit state function* for any limit state of interest; it describes the state of the system related to the variability of the parameters that come into it; in particular if it is positive the system "is safe" otherwise it is collapsed. The range of variables for which the limit state function is positive is so called *safe domain*, its complement in the space of the random numbers in the limit state function is the *failure domain* (Eq. 1).

$$Z : \begin{cases} > 0 \Rightarrow Safe \\ = 0 \Rightarrow Limit \\ < 0 \Rightarrow Collapse \end{cases} \tag{1}$$

Limit state function typically refers to a vector of random variables which are material or structure related but may be also related to age of construction and/or codes. This definition that is given for a single structure can be extended to a class of buildings keeping the same meaning of *Z*.
A very common representation of the limit state function for structures is given in Eq. 2 in terms of difference between actions and resistances; in the seismic case, resistance means seismic capacity *C* while the action is represented by seismic demand *D*.

$$Z\left(\overline{X}_C, \overline{X}_D, t\right) = C\left(\overline{X}_C, t\right) - D\left(\overline{X}_D, t\right) \tag{2}$$

$\left(\overline{X}_C, \overline{X}_D\right)$ are the variables governing Capacity and Demand respectively, while t is a continuous parameter.
Computation of the probability that the limit state function is negative means to evaluate the seismic risk of the structural system. If random variables in Eq. 2 are time dependent they should be described as random processes; otherwise the problem may be considered as time-invariant.
Formally evaluation of the seismic risk requires availability of joint probability density function for vector of random variables vector and knowledge of failure domain as in Eq. 3.

$$P_f = \int_{\Omega} f_Z\left(\overline{X}\right) d\Omega \tag{3}$$

Eq. 3 is a n-dimensional integral where n is the dimension of the vector and Ω is the failure domain in the space of $\overline{X}_C, \overline{X}_D$ where the limit state function is not positive.
Exact computation of (3) is often computationally demanding; in fact, joint Probabilistic Distribution Function (PDF) is not always available in close form $f_Z\left(\overline{X}\right)$ and, moreover, the failure domain definition

and computation of the above integral may be not trivial.

HAZUS loss assessment methodology

HAZUS methodology developed by U.S. Federal Emergency Management Agency (FEMA) [8] is the best current practice in terms of scenario analysis at urban scale. Method's goal is computing fragility curves for a class of building for different damage levels *ds* (*light, moderate, extended or total*). Damage levels definitions are qualitative function of losses that damage can provoke in economic and social terms. Fragility functions (or damage functions) are assumed as lognormal conditioned to a spectral displacement.

$$P[ds \mid S_d] = \Phi\left[\frac{1}{\beta_{ds}} \ln\left[\frac{S_d}{\overline{S}_{d,ds}}\right]\right] \tag{4}$$

Function in Eq. (4) expresses probability of reaching a *ds* damage state due to the spectral displacement S_d, while β_{ds} is the dispersion of the lognormal random variable ε_{ds} (see Eq. 5) relative to the spectral displacement threshold which trigger *ds* damage state.

$$S_d = \overline{S}_{d,ds}\varepsilon_{ds} \tag{5}$$

Median $\overline{S}_{d,ds}$ follows structural considerations, in particular:

$$\overline{S}_{d,ds} = \delta_{R,Sds}\, \alpha_2\, h \tag{6}$$

where $\delta_{R,Sds}$ is drift determining damage level considered; $\alpha_2 h$ è is the height of the building where the push over analysis displacement occurs for the considered building class. Variability of damage functions derives from uncertainties on capacity, demand and damage threshold, all of these sources of uncertainty are assumed as lognormally distributed.

$$\beta_{Sds} = \sqrt{\left(CONV\left[\beta_C, \beta_D, \overline{S}_{D,Sds}\right]\right)^2 + \beta_{M(Sds)}^2} \tag{7}$$

In Eq. 7: β_C is capacity curve dispersion; β_D demand spectrum variability and $\beta_{M(Sds)}$ is the uncertainty related to the estimation of median of threshold triggering the damage state, assumed as independent from capacity and demand.

Getting the spectral intensity value that defines the threshold for the considered *ds* means to cross the capacity of a certain buildings class with inelastic demand spectrum, which is site dependent.

Building classes of Eq. 4 are defined by means of structural type (materials, structural system, etc); in particular, 36 typological categories are defined and subdivided due to height and quality of seismic codes presumably used for design. There are 4 code classes (*High-Code, Moderate-Code, Low-Code, Pre-Code*) referring to required seismic performance.

This classification implicitly includes construction age distinction. For hospitals and other strategic constructions specific damage functions are derived assuming that those structures have been presumably designed with performance standards higher than for other structures.

To each geographical area within U.S. a certain design level is assigned. For higher seismicity zones: higher standards are applied for design of recent structures (i.e. Californian structures after 1973 are considered as *High Code*), going back in time design quality decreases (i.e. Californian structures built during period 1940-1973 are *Moderate Code*). Structures built previously than 1940 are assumed as *Pre-Code* meaning that no specific seismic standard has been used.

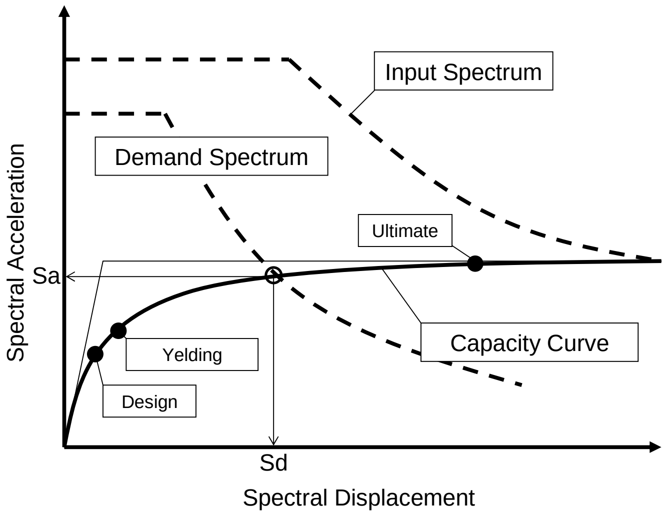

Figure 1. Seismic capacity vs. demand in the HAZUS methodology.

Fragility assessment is based on class capacity that is described by bi-linear push-over curves. Three fundamental points are: design capacity, yielding capacity and ultimate capacity. Capacity curves are defined by design variables such as fundamental oscillation period, over-strength and ductility. Some of those can be obtained by codes, once the design level has been defined; others are considered code-independent and are assumed a priory according to the structural type. Capacity curves fundamental values are listed for all classes. Lognormal random variable is associated to uncertainty and depends on design code level. Performance point is determined intersecting demand spectrum and capacity curve as described in Figure 1.

SAC FEMA 2000 method

This method [7] is a state of the art procedure for numerical seismic risk analysis of when single structure is concerned. It approaches seismic risk assessment by a time-invariant formulation. Saturation of time parameter is possible by non-linear dynamic analyses and regressions relating seismic intensity measure (the same used expressing probabilistic hazard) to structural demand (Figure 2).

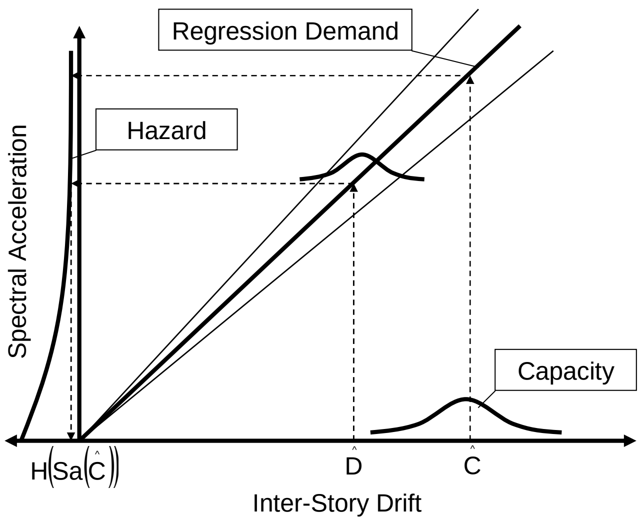

Figure 2. SAC – FEMA 2000 method.

The method makes use of total probability theorem to separate hazard analysis from structural analysis through the seismic intensity measure. This concept may be summarized as follows

$$P_f = P[Z] = P[C - D \le 0] = P[C \le D \mid D = d]P[D = d(IM) \mid IM = im]P[IM = im] \tag{8}$$

Hypothesis behind Eq. (8) is capacity being independent to seismic intensity measure.

$$P[C \le D \mid D = d \mid IM = im] = P[C \le D \mid D = d] \tag{9}$$

In the continue domain seismic risk may be formulated in integral form (10). Assuming seismic demand and capacity as lognormal random variables seismic risk (failure probability) is given in close form (Eq. 10).

$$P_f = \int_0^{+\infty} [1 - F_D(a)] f_C(a) da = H\left(IM\left(\hat{C}\right)\right) e^{\frac{1}{2}k^2\left(\beta_D^2 + \beta_C^2\right)} \tag{10}$$

Where *H(IM)* is hazard at the intensity measure value corresponding to median of the capacity; β's are dispersions of demand and capacity random variables; *k* is a constant depending on the regression of demand as function of intensity measure by non-linear dynamic analysis, i.e. Incremental Dynamic Analysis. This formulation especially fits for those structures which the collapse is driven by a single spectral parameter, i.e. displacement.

SAC-FEMA method, since it has been developed for single structure case, assumes some mechanical variables as deterministic in order to get a numerical model to dynamic analysis use. This feature may be a limitation at territorial level of analysis.

Application to a urban scale seismic risk analysis may be not easy due to the greater uncertainty on the structural configuration, but also due to the larger number of failure modes. Moreover, for those structures that have the seismic demand driven by a non-unique or hard to predict in terms of Probabilistic Hazard Analysis seismic intensity measure, procedure extension may be difficult.

RISK FORMULATION AT TERRITORIAL SCALE

Limit state function for the territorial scale

Intrinsic limits of SAC-FEMA due to development purposes are complementary to difficulties related to the extension of any numerical/mechanical method for the vulnerability analysis of structures, also due to the required computational effort.

At territorial scale, seismic risk, or analogously failure probability, has to be considered as the expected fraction of structures not surviving during the observation period. Then, it is worth to explore a formulation of the problem based on the evaluation of the probability of the capacity being exceeded by seismic demand.

From this standpoint, analysis of capacity becomes more relevant since its uncertainty related to the number of buildings collected in the same class may not be overwhelmed by seismic hazard uncertainty.

On the demand side, Probabilistic Seismic Demand Analysis (PSDA) [14] is simplified to an inelastic spectral analysis.

Reference spectrum derived from PSHA is a spectrum which takes into account seismic signal variability and associates a probabilistic distribution to each spectral ordinate. Selection of a certain percentile in this distribution provides the Uniform Hazard Spectrum corresponding to that given percentile. In order to evaluate inelastic demand, PSHA related spectrum for the region of interest must be adequately reduced by spectral reduction factors.

Working in terms of strength, limit state function Z for the building class can be defined as shown in Eq. 11 where C is the capacity of a building in the class, D is a measure of inelastic spectral demand and R is the corresponding strength elastic spectrum reduction factor; $\overline{X}$ is the vector of variables that associates to each structure in the class the corresponding capacity; μ is the ductility capacity of the structure and T is the first period of oscillation.
If it is possible to extract from $\overline{X}$ vector two others vectors $\{\overline{Y},\overline{W}\}$ made of those variables determining ductility and period, it is then possible to write the following:

$$Z = C(\overline{X}) - \frac{Sa(T(\overline{W}))\varepsilon_{Sa}}{R(\mu(\overline{Y}),T(\overline{W}))\varepsilon_R} \tag{11}$$

where terms ε*'s* conventionally represent unit median lognormal random variability of each factor around its median value given by the above functional.
In particular ε_{Sa} is the uncertainty related to spectral ordinate related to PSHA, while ε_R is the uncertainty associated to elastic spectrum reduction factors which typically are obtained by regression of non-linear dynamic analysis of SDOF systems.
If all the building in the class have the same number of floors and, as consequence, it can be supposed that they have the same fundamental oscillating period, to better understand the concept herein expressed it is possible to re-arrange Eq. 11 in the Eq. 12 form, where both spectral demand and capacity are expressed in terms of the same seismic intensity measure in the same form as Eq. 2.

$$Z = C\,R\,\varepsilon_R - Sa\,\varepsilon_{Sa} = \hat{IM}_C\,\varepsilon_{IM_C} - \hat{IM}_D\,\varepsilon_{IM_D} \tag{12}$$

In equation (12) relation between structural local and global parameters is expressed, while elastic spectral intensity measure is dependent upon fundamental period of oscillation; inelastic reduction factor depends on the same fundamental period of oscillation and available ductility (see Fig. 3).

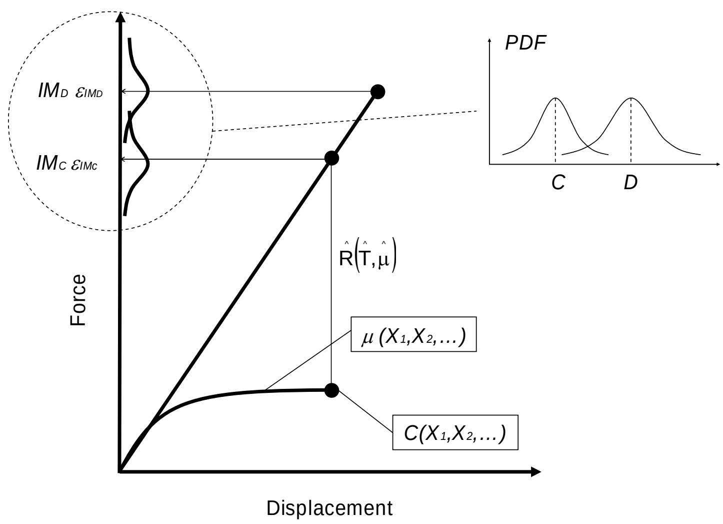

Figure 3. Limit state function representation.

Correlation between residuals ε terms may play an important role in the a feasibility of such computation at territorial scale; in particular ε_R and ε_{IM} may be negatively correlated, but, at least for inelastic ratios no dependence on magnitude and distance is proved in the far field and beyond the short period range [15]; moreover the correlation may be estimated and correctly taken into account in the computation or

neglected and results corrected by subtracting introduced bias.
Analogous limit-state function formulation holds working in terms of displacement.
In a fully probabilistic approach to seismic risk problem, each term of Z is a random variable; once needed probability distributions are obtained, the problem remains as time invariant with a considerable reduction of complication in the computation of risk.
It's necessary to define relation among base random variable that influences the risk. Same structural factors that govern the capacity drive fundamental period of oscillation and ductility. In order to make the approach effective, it is worth to reduce as much as possible the number of joint density probability functions needed for the computation of the integral (3).
It is assumed that is possible to get distributions of elements of $\overline{X}$ which are independent. So far, if distributions of parameters are available, seismic risk is expressed by Eq. (13); the n-dimensional time-invariant integral where n is dimension of $\overline{X}$ vector.

$$P_f = \int_{\Omega} f_{X_1}(x_1)\, f_{X_2}(x_2) \cdots f_{\varepsilon_R}(\varepsilon_R)\, f_{\varepsilon_{IM}}(\varepsilon_{IM})\, d\Omega \tag{13}$$

Known all terms of Eq. (14) seismic risk, that is urban class failure probability, may be approximated using methods as simulation or limit state function first or second order approximation.
If simulation is used, integral (14) becomes integral (15) where $I(z)$ is the indicator function which is equal to 1 in the failure domain and zero outside in each simulation run looking at the sign of the realization of Z.

$$Seismic\,Risk = P_f = \int_{x_1}\int_{x_2}\cdots\int_{x_n}\int_{\varepsilon_R}\int_{\varepsilon_{IM}} I(Z) f_{X_1}(x_1)\, f_{X_2}(x_2)\cdots f_{\varepsilon_R}(\varepsilon_R)\, f_{\varepsilon_{IM}}(\varepsilon_{IM}) dx_1 dx_2 \cdots dx_n\, d\varepsilon_R\, d\varepsilon_{IM} \tag{14}$$

Then, it is clear that the formulation (13-14) directly follows basic concepts of structural reliability. Eq. 14 accomplishes the goal of providing seismic risk at urban level in a full numerical form, however this may not be useful to get the safety margin of the area's building stock since it works at limit state.
Applicability and feasibility is the critical success factor of the method; basically they are related to the availability of distributions of factors driving capacity and demand and also to de determination of relation between those parameters and capacity or ductility and period for the class of buildings considered.
So far it seems worth to define a building class taking into account computability of integral (15), i.e. considering the building class as the one characterised by the same period (e.g. same height or floor number) avoiding the introduction of the spectral shape in the limit state function assuming the structure as first mode dominated.
At urban scale, distributions of vectors governing capacity have to be measured in the field of the reference area. Probabilistic distributions plugged into the integral (14) should reflect also geographical distribution of factors in the building stock. Nevertheless capacity should be valid not for the single structure anymore but for a group extended as much as justified by the effort required by a so demanding vulnerability analysis.

Class capacity analysis

In the territorial case class vulnerability may be defined as a functional that associates a seismic fragility function to each building, e.g. each realization of the vector $\overline{X}$
In the single structure case it is easy to imagine as the probabilistic characterization of capacity is related to uncertainty of those parameters at cross section level or element level that are not easy to evaluate a priori as material properties of reinforcing steel amount.
If class capacity is defined as a functional also considering global parameters, as geometric dimensions or structural configuration, it is possible to transfer vulnerability concept not limiting it to the single structure.

Due to these definition two distinct buildings belongs to the same class if and only if their capacity is described by the same functional in the same definition domain differing only for the different realization of input variables vector $\overline{X}$.

Capacity issue formulated this way seems to be best resolved by Response Surface Method [16]. Goal is representing capacity approximated function trough regression as input for a FORM or simulation analysis to obtain probabilistic characterization.

Response surface method is used to replace structural relationship parameters measured in the area with an explicit function derived by regression of a certain number of observations determined by non linear push-over analysis. Structural models should be able to describe main behavioural phenomena of existing structures on examined region.

Minimum number of experiments for estimating RS parameters is equal to their number then is depending on the order of polynomial chosen as function approximating the response. To this aim the capacity is approximated by a model such as the one in Eq. 16.

$$C \cong \beta_0 + \beta_1 X_1 + \beta_2 X_2 + ... + \beta_{11} X_1^2 + ... \qquad (16)$$

This model is linear in the β_0, $\beta_{i,1}$ β_{ij},.. unknowns. For the βs estimation least squares method is frequently applied; it corresponds to find those values of βs minimizing sum of squares of differences between observed values with the experiments and values predicted by polynomial approximated function.

Critical step for the purposes given above is the definition of a suitable experimental plan, then in the analyses to run to get coefficients of relation (16).

The same concept may be applied to get the ductility of the building class. The input vector $\overline{X}$ can be partitioned such that $\overline{X} = \{Y, W\}$ (see Eq. 13) to identify those structural parameters that influence the ductility (and/or the period) the function expressing how ductility vary within the class is estimated by a regression as in Eq. (17).

$$\mu \cong \gamma_0 + \gamma_1 W_1 + \gamma_2 W_2 + ... + \gamma_{11} W_1^2 + ... \qquad (17)$$

Approximated function of ductility can be then plugged into integral (15) as capacity. Although design of experiments issues have been analysed in the case of reinforced concrete structures or industrial structures [17, 18, 19] problems facing no experimental error such as pushover analysis is rising up in statistics and it's quite new for structural engineering.

Such problem refers to the category of so called Computer Experiments [20] where no experimental error can be associated to the response observations, therefore traditional methods for best fit and interpolation of response surface are not proven to be optimal. It is worth to point out that since no experimental error can be associated to the regression no standard error can be measured in the estimation of seismic risk.

Probabilistic inelastic spectral demand analysis

If the class of building is assumed to be first oscillation mode dominated, demand can be obtained by elastic spectra appropriately scaled due to predicted inelastic behaviour of each building varying in the class.

In this case, demand (spectral ordinate) uncertainty is the one coming from Probabilistic Seismic Hazard Analysis while the demand in the spectrum abscissa is eventually due to a variation of oscillating fundamental period in the class (Fig. 4).

To this regard, it is worth noting that uncertainty related to period makes the computation of integral (15) more complex, since the shape of the spectrum cannot be neglected. For sake of simplicity, period can be in some cases assumed as representative of a single class, if the relative distribution, if the relative definition is well established and optimised.

Defined the median shape of the spectrum the problem of uncertainty is reduced to characterization of error variable ε. This residual distribution directly comes from Probabilistic Seismic Hazard Analysis.

Reduction factor used to get inelastic demand carry on their own uncertainty as well of the spectra. Each of them available in literature or codes has a median functional form determined by regression of thousands of real record non-linear time-history analysis of single degree of freedom systems. They may also depend on further factors such as soil type. If territorial ductility and vibrating period distributions are obtained, reduction factors will be randomly variable as well because combination of others random variables, its dispersion will also depend on residuals distribution of regression.

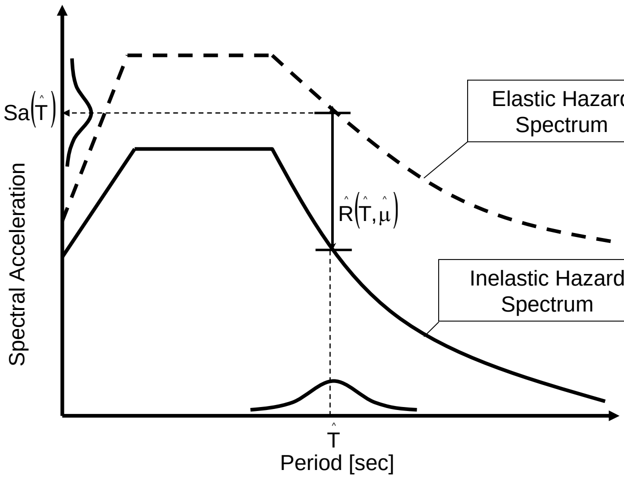

Figure 4. Inelastic spectral demand.

CONCLUSIONS

Seismic Risk Assessment at territorial scale for structures is characterised by an increasing interest in the earthquake engineering due to its mitigation and planning resource aspects. From the scientific point of view advances in Probabilistic Seismic Hazard Assessment and single structure risk analysis have not been followed by the same level of development for vulnerability analysis at territorial scale.
This situation results in a wide use of probability matrices (i.e. Italy) to assess vulnerability of building stock; but also in some heuristic aspects embedded in HAZUS loss estimation methodology.
Proposed formulation gives an intermediate approach between HAZUS and SAC-FEMA single structure oriented method. The procedure is quantitative and fully probabilistic, using simplified models to estimate both capacity and seismic demand, fitting purposes of territorial scale and its computing effort.
Demand is simplified in respect of non-linear dynamic procedures and counts on well known and code adopted tools as spectra and elastic demand reducing factors.
Regression methods are proposed for class capacity analysis relating factors seismically defining of the class to the capacity trough push-over analyses. However feasibility of such approach should take into account the Computer Experiments issues in design of experiments.
Several uncertainty sources may be included in the analysis if they can be measured. The field estimation of probabilistic distribution of structural factors is crucial; it should be done by observational campaigns.
For any of the building classes considered, the evaluation of some joint probability density functions may be required due to the correlation of some factors , i.e. period and ductility.
The computation of seismic risk as the fraction of the structures of the class of interest that collapses during the observation period can be thus attained.

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