IDENTIFICATION OF DESIGN SEISMIC ACTIONS CONSIDERING THE NONLINEAR BEHAVIOUR OF STRUCTURES

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ABSTRACT

This paper evaluates the different characteristics of earthquake ground motions in terms of their influence on the response of structures with the purpose of identifying representative vibration histories to be used in seismic design; conceptual aspects are emphasized. All the earthquake ground motions considered are artificial accelerograms; they are organized in 10 realizations samples of stationary and non-stationary models with different frequency content and time evolution characteristics deemed to represent a wide range of magnitudes and focal distances for firm soil sites; the influence of other soil conditions is not taken in account. Non-stationary models are considered to idealize real earthquake motions; stationary models are considered to be design vibration histories. Structural systems are modeled by an ensemble of buildings; different natural frequencies, post-elastic characteristics and available ductilities are considered. The equivalence between stationary and nonstationary vibrations is established in terms of probabilities of failure. The paper concludes by presenting some guidelines for the choice of the representation of the earthquake input for structural design purposes, derived from the results obtained.

INTRODUCTION

The design methods of structures for earthquake actions should always be associated, implicitly or explicitly, to some safety checking. When they are implicitly associated, it means that the operations of the design procedure are sufficiently exhaustive (as in general they are) to ensure by themselves a sufficient reliability. On the other hand, some design methods must be complemented by a safety assessment. This safety assessment usually involves setting up one or more hazard scenarios accompanied by the definition of appropriate limit states; this is what is known as a level 1 safety checking [20] which is no more than a very simplified procedure in regard to a full probabilistic evaluation of the structural reliability. The development of very powerful methods, both computational and experimental, to investigate the nonlinear behaviour of structures strained far beyond the elastic range by earthquake actions, calls for a corresponding increase in the improvement of criteria for the construction of hazard scenarios and for assessing the severity of earthquake effects in structures. One way of obtaining improved criteria is by establishing a more clear relation between the variables in the simplified procedure and the

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entities of the more sophisticated evaluation of reliability. For this purpose, the present paper studies the relation between seismic hazard and design earthquake actions and starts by inquiring into the relationship between simplified and non-simplified design and safety checking methods.

THE LEVELS OF SAFETY CHECKING AND STRUCTURAL DESIGN

Safety checking and structural design may be performed at different levels according to the sophistication of the algorithms utilized and to the more or less fundamental character of the entities employed; as an example: probabilistic model of seismic hazard - isoreliable spectra - design peak ground acceleration are a sucession of entities with decreasing fundamental character. Structural design is more natural at low levels since its object is to define some of the structure characteristics and, in consequence, it must operate with a structural model where those characteristics are not included; if models sophisticated enough to take into account all the characteristics of the structure are used, the structure must be completely defined before the analysis and thus only a safety checking may be performed. The relationships between design and safety checking, and between sophisticated and simplified procedures may be more easily discussed with help of the box diagrams presented in figure 1, which illustrate two methods of different levels for the safety checking or structural design of a reinforced concrete structure.

The left diagram refers to a sophisticated level. The data for the method is constituted by: i) the geometry of the structure, the mechanical characteristics of the concrete and the amount, distribution and mechanical characteristics of the steel; ii) the definition of the seismic hazard by a probabilistic model; and iii) the definition of the structure intended performance by a set of requirements (e.g. no collapse requirement, serviceability requirement...). The first step of the procedure is the evaluation of the vulnerability functions, by performing some nonlinear dynamic analysis; in these analyses the seismic hazard is discretized into a set of earthquake events, which are represented by models (stationary or non-stationary) of a sophistication consistent with the sophistication of the structural model. The vulnerability functions, which express the relation between the parameters describing the earthquake severity and the parameters describing the earthquake effects in the structure (control variables), must be of a type appropriate to the evaluation of the capacity of the structure to meet the requirements with an adequate reliability; this evaluation is carried out with a reliability model and its outcome is the ductility demand in the structural elements. The ductility demand is set against the detailing provisions through a ductile capacity model. If the detailing provisions enter into the ductile capacity model as data and the result is an assessment between the available and demanded ductility, it is a safety checking; if the detailing provisions are the outcome of the ductile capacity model, it is a design procedure.

The right diagram refers to a simplified level method. The data is constituted by: (i) the geometry of the structure and the elastic characteristics of the concrete; ii) the definition of the earthquake action by a design spectrum; and iii) the quantification of the structure performance through the definition of limit states (e.g. maximum strains in concrete and steel). The first step of the procedure is the computation of the action effects; in this computation three coefficients must be considered: the damping coefficient ξ, the partial safety coefficient γ^* and the behaviour

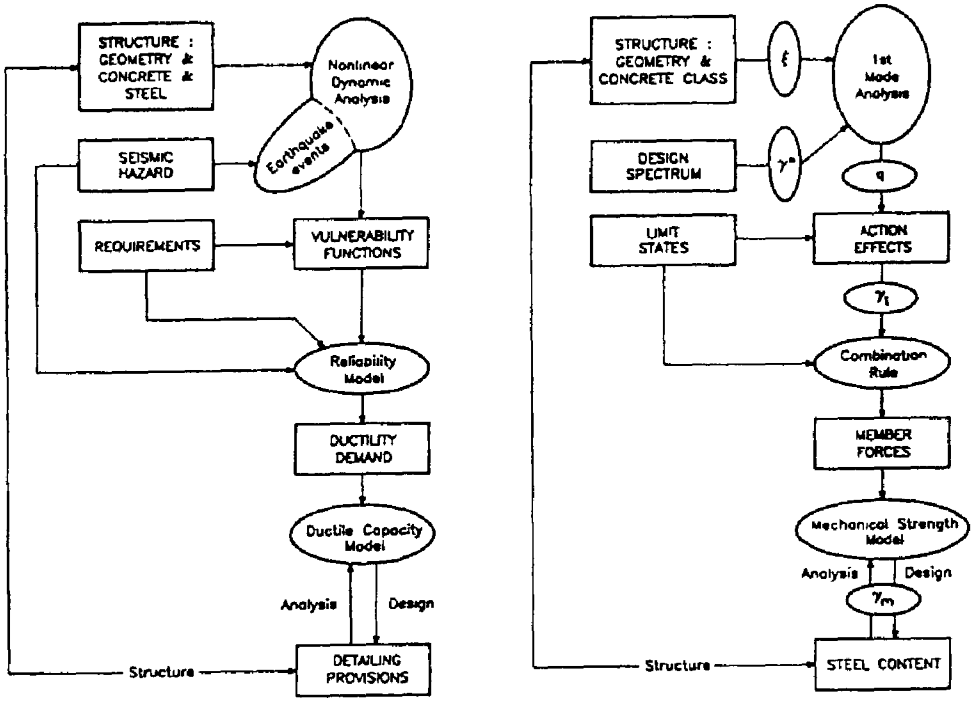

Figure 1: Sophisticated (left) and simplified (right) levels of safety checking or structural design

coefficient q; since the computation method, a 1st mode analysis, is linear, its results really depends on the value of $\gamma^* /q\sqrt{\xi}$ and not on the values of each coefficient. The action effects, which must be of the appropriate type for the comparison with the relevant limit states, are multiplied by an importance factor γI , which introduces reliability differentiation (to take into account consequences of failure) and are entered in the combination rule that gives the member forces to be compared with the member resistances through a mechanical strength model. This model gives the steel content, in the case of design, and assess the adequability of the steel content in the case of safety checking; the values of the material properties in this model are reduced by a material safety factor γ_m

The simillarity in organization of the two diagrams is noteworthy in face of the large differences in the models used in each method; in effect, this simillarity derives from the possibility of identifying some characteristic of the structure which is irrelevant for the quantification of the structural model and will be the object of the safety checking or the design decision.

It is now possible to conceive a large number of intermediate methods by considering, for each pair of corresponding boxes, models of intermediate complexity; for instance, between the 1st mode analysis and the nonlinear dynamic analysis, a multimodal analysis and stochastic linearization analysis may be placed. Within this conception the quantification of the coefficients that enter in the simplified method should be made in such a way that, if a simplified model is substituted by a more accurate one, the input for the sophisticated model ensures that its output is 'optimized' in terms of the information it provides; for instance if the structural model is upgraded from a linear to a nonlinear model, the value of the γ * coefficient (and/or of the design spectrum) must be such that the nonlinear analysis are performed for an earthquake intensity that gives the greatest amount of information in relation to the relevant limit states. It should also be noted that when going from low level methods to higher level methods the number of coefficients decreases, because when a model is substituted by a more sophisticated model the 'surrounding' coefficients may be 'absorbed' into the sophisticated model. It does not seem possible nor desirable, however, to dispense with the use of coefficients because of safety checking or design for other actions, furthermore, a set of values for those coefficients may be thought of as defining a class of structures, constituted by all structures which can be designed or checked, with acceptable errors, by the simplified method quantified by those values.

Those ideas suggest that the development of criteria to decide if a structure belong to a given class of structures, and thus can be designed with a given set of coefficients, and that the development of guidelines to judge at what level of methods a structure should be analysed are important problems. Important advances have already been made in this area [9].

SIMPLIFIED SAFETY CHECKING

A safety checking may be simplified in two senses: the checking procedure itself is simplified; the checking procedure is applied to simplified models of the structure and/or of the earthquake action. The last case is considered here and its main purpose is considered to be the elimination of irrelevant details in the description of the action and structural response.

Descriptive Functionals

The essential problem in the definition of a simplified model is the choice of its variables. The more difficult part of this choice is the passage from the time histories of the acting motion (a(t)) and of the structural response (r(t)) to the values or sets of values that may describe them in a simplified way, like the peak ground acceleration describe the intensity of an earthquake vibration and the maximum deformation describes the severity of the earthquake effects.

The mapping of time histories $x(t)$ values is called a functional in mathematics. Hence, the essential characteristics of the actions and responses may be expressed through the values of a number of functionals defined on the time histories $a(t)$ and $r(t)$, which will be referred to as descriptive functionals. If this number is finite, some information is necessarily lost; but if this number is infinite, all the information contained in the time histories may be preserved if the functionals are adequately selected. For illustration consider the functionals h_k defined by $h_k=(\int_0^{T_1} x(t)\, e^{-2\pi i k t/T_1} dt)/T_1$ $k = 0,1,-1,2,-2,\ldots$ Knowledge of the values of those functionals is equivalent to the knowledge of the time histories themselves since they are their Fourier coefficients.

There are a large number of possibilities for selecting the descriptive functionals. One possibility is to define a general functional, which is applied to every time history. For instance, the value of the supremum norm may be selected as the descriptive functional; the supremum norm of a time history $x(t)$ defined in a time interval T is $\| x \|^{\infty} = \max |x(t)|\ t \in T$. The number of descriptive functionals can be readily increased if the general functional is also applied to the time-derivatives or time-integrals of the histories.

Let **h** be the vector representing the values of the descriptive functionals **h** (a(t)) of the actions (intensity measures) and c the vector representing the values of the descriptive functionals c(r(t)) of the responses. Those last functionals will be called 'control variables' and it is worth mentioning that they can take the form of 'damage indexes' as for example [15]: $D = \delta_m / \delta_u + (\beta \; dE)/Q_y \delta_u$ where: D- damage index ($D \geq 1$ signifies collapse); δ_m - maximum deformation, δ_u ultimate deformation under monotonic loading; β- non-negative constant; Q_y yield strength; dE - incremental dissipated hysteretic energy.

Equations of Motion

The earthquake action will be considered to have several components and a finite duration T_1; hence, every possible earthquake action will be assumed to belong to the space A_s of vector valued functions Lebesgue integrable in absolute value. Conversely, every function belonging to A_s will be considered to be a generalized earthquake action.

The response of the structure is assumed to be described by a finite number r of variables (displacements, accelerations, internal forces ...); the time histories of the responses to all generalized actions constitute a 'space' R_r; the duration of interest T_2 of the response time histories will be assumed to be greater than T_1.

The relationship between an element $a(t)$ of A_s and the corresponding element **r** of R_r defines the structure operator:

$$\mathbf{r}(t_2) = \mathcal{E}(\mathbf{a}(t_1)) \quad t_1 \in [O, T_1] \quad t_2 \in [0, T_2] \tag{1}$$

The operator ε is assumed to be sufficiently continuous i.e. for two action histories $a_1(t_1)$ and $a_2(t_2)$ sufficiently close to one anoter, the corresponding response histories will also be close to one another. The equation (1), which is an equation of motion, should be understood in the sense that it establishes a correspondence between the complete time history of the action $a(t_1)$ and the complete time history of the response $r(t_2)$.

Vulnerability Function

The vulnerability function represents structural behaviour, not as a mapping of the time histories of the earthquake input into the time histories of the structural response, but as a much simpler relation between the intensity measures **h** and the control variables **c**. From the equation of motion it is possible to write:

$$\mathbf{c} = c(\mathcal{E}(a(t_1))) \tag{2}$$

In this new and simplified form, the equation of motion represents a mapping from the space of the actions into the Euclidean vector space C of the control variables. The last step needed for the vulnerability function to appear is to substitute $a(t)$ by $\mathbf{h}$. This substitution poses somes problems. In effect, it is possible to use the inverse image $F(\mathbf{h})$ of $\mathbf{h}$ defined by

$$\mathcal{F}(\mathbf{h}) = \{\mathbf{x}(t) : \mathbf{h}(\mathbf{x}) = \mathbf{h}, \mathbf{x} \in \mathcal{A}_s\} \tag{3}$$

In general, $F(\mathbf{h})$ will be constituted by a large number of time histories $x(t)$; for instance if the descriptive functional is the peak value of acceleration, $F(\mathbf{h})$ will be all acceleration time histories with peak value $\mathbf{h}$. Thus if the vulnerability function $V(\mathbf{h})$ is defined as $V(\mathbf{h}) = c(E(F(\mathbf{h})))$ for a given vector $\mathbf{h}$ a large number of possible values for the control variables would result. The effective way to side-step this indetermination is to define as the value of the vulnerability function an weighted average of all the possible c values. This can be done in a relatively straightforward manner when the earthquake action is idealized as a stochastic process [7]. A stochastic process is an ensemble of functions where a probability measure μ_α is defined. The advantage of introducing stochastic processes is that the expected values of the descriptive functionals can be considered to represent the whole space of actions or structural responses. Let $\mu_\alpha(i), i \in I$ be a family of probability measures such that $E(\mathbf{h}, \mu_\alpha(i)) \neq E(\mathbf{h}, \mu_\alpha(j))$ for $i \neq j$, where $E(\bullet,\bullet)$ means expected value in terms of the indicated probability measure. As an example, consider a power spectrum $S(\omega)$, $I \equiv (0,\infty)$, a family of probability measures are the measures corresponding to the stationary gaussian processes defined by the spectra $i^2 S(\omega)$. Let F be the inverse of the mapping $\bar{\mathbf{h}} = \int_{Ae} h(a(ti)) d\mu_\alpha(i)$. Then the vulnerability function is defined as

$$\bar{\mathbf{c}} = \mathbf{V}(\bar{\mathbf{h}}) = \int_{\mathcal{A}_e} c(\mathcal{E} a(t_1)\ d\mu_a(\mathcal{F}(\bar{\mathbf{h}})) \tag{4}$$

DESIGN METHODS AND SAFETY CHECKING

Safety checking can be formalized in the following way [6]:

Let $\mathbf{a}$ be a vector representing the actions on the structure; $\mathbf{a}$ will always be a probabilistic vector, but in some cases its values will correspond to forces (or imposed deformations) acting at given locations of the structure and in other cases to time histories of forces (or deformations).

Let $\mathbf{d}$ be a vector containing the 'variable' characteristics of the structure. A structure is considered to be defined in two phases: the type of the structural system is selected in the first phase; in the second phase the mechanical characteristics of the different structural elements are attributed; the values in $\mathbf{d}$ represent only the quantification carried out in the second phase. The vector $\mathbf{d}$ is a deterministic vector and its elements will be called 'decision variables' because they represent the decisions that are made during the design of the structure

Uncertainty about the future characteristics of the structure means that a probabilistic model should be considered. The 'deterministic part' of this model cames from the structural system and values of the decision variables, the 'probabilistic part' of the model is supposed to be quantified by a vector $\mathbf{p}$ which contains the constants of the probability distributions of the structural variables. Assuming the structural system to be a reinforced concrete frame, for ilustration purposes, the decision variables will be the geometric dimensions of the cross-sections of the beams and columns, the distribution of reinforcement and the classes of concrete and steel to be used; the vector **r** will then contain those values as may be needed to fully quantify a probabilistic model of the structure; in principle, the values in **p** will describe the variability of the mechanical properties of the elements as independent of their geometrical characteristics; hence **p** is not dependent on the actual value of **d.** But the uncertainties associated with the behaviour of the structure may depend on the vector of the decision variables; for instance, some elements of **d** may represent the intensity of inspection of the building process and the efficacy of the quality control of the materials.

Let **r** be a vector containing those responses of the structure which are relevant for the assessment of its safety (control variables). The elements of **r** usually would be internal forces, deformations, relative displacements between floors, or ductility demand in some or all structural elements. Since the elements in the actions vector **a** will be considered to be time-histories the elements in **r** will also be time-histories.

The behaviour of a structure in design terms can be formulated now by the following expression:

$$\mathbf{c} = \mathcal{E}(\mathbf{d},\mathbf{r},\mathbf{h}) \qquad (5)$$

A design method may be considered as an algorithm which permits the determination of the values of the decision variables. A relationship between design method and safety checking is discussed below for single variable structures, but others relationships could also be considered [8].

A single variable structure may be idealized as an oscillator characterized by the elastic stiffness $K_{O.}$, the yielding displacement d_y, ductile capacity μ and vulnerability function $V(d_y, \overline{\alpha})$, where $\overline{\alpha}$ is the mean value of the peak acceleration of the earthquake action modelled by a stochastic process. The ductile capacity is defined as $d^{u}_{0.005}/d_y$ where $d^{u}_{0.005}$ is the 0.005 fractile of the probability distribution of the ultimate displacement, i.e the maximum displacement the structure can withstand; it is assumed that this distribution is log-normal; the 0.005 fractile (which should be understood as resulting from the product of a characteristic value by a partial safety factor on material strength γ_m was selected as the representative value for the ductile capacity in order to make the value of the probability of failure almost independent from the characteristics of the ultimate displacement probability distribution; it is assumed that the value of the ductile capacity is fixed by the choice of a 'ductility class' for the structure. The maximum displacement is selected to be the control variable; hence the vulnerability function gives the average values of the maximum displacement imposed on the structure by the earthquake motions with an average peak acceleration $\overline{\alpha}$. both averages being referred to the probability distribution of the earthquake action stochastic process. The variability of the maximum displacement will be disregarded, because it is assumed small compared with the variability of the earthquake actions and of the ductile capacity. The vulnerability function $V(d_y,\overline{\alpha})$ is also a function of all the parameters that define the force-deformation relationship of the structure.

In the design method to be considered the yield force is the design variable and the control variable is the maximum displacement. In figure 2 are presented the different steps of this design method and the way the different coefficients are defined. Starting from the probability distribution $P(\bar{\alpha})$, the upper characteristic value $\bar{\alpha}_c$ is selected to be the representative value and the design value $\alpha_d=\gamma^*\bar{\alpha}_c$ is determined with the help of the safety factor γ^*. Provisorily assuming that the structures behaves linearly, the linear vulnerability function $V^l(\bar{\alpha})$, corresponding to a linear stiffness κ^l is used to find the action effects $S^d{}_E$, expressed in terms of maximum displacements, and $S^f{}_E=S^d{}_E/\kappa^l$, expressed in terms of maximum force. Dividing $S^f{}_E$ by the ductility coefficient η, the design yielding force f_{yd} is determined: $f_{yd} = S^f{}_E/\eta$. From this design force the average yielding force is obtained $f_y = f_{yd} / \gamma_f$, which defines the nonlinear force-displacement relationship $\kappa_n(d)$ and is used to quantify the nonlinear vulnerability function $V(\bar{d}_y,\bar{\alpha})$. This vulnerability function is used to transform the probability distribution $P(\bar{\alpha})$ the mean peak acceleration $P(\bar{\alpha})$, in the probability distribution $P(d^e)$ of the earthquake imposed displacements d^e. The convolution of $P(d^e)$ with the probability distribution of the ultimate displacements gives the probability of failure p^f

This design method is quantified by both the values of γ^* and η depends only on the product $\gamma^*\eta$; in consequence, if a given value is selected for the ductility coefficient η, the value of γ^* is determined (by trial and error) from the value of the probability of failure; on the other hand, if the value of the safety factor γ^* is selected, it is value of the ductility coefficient which must be determined in order to have a given failure probability.

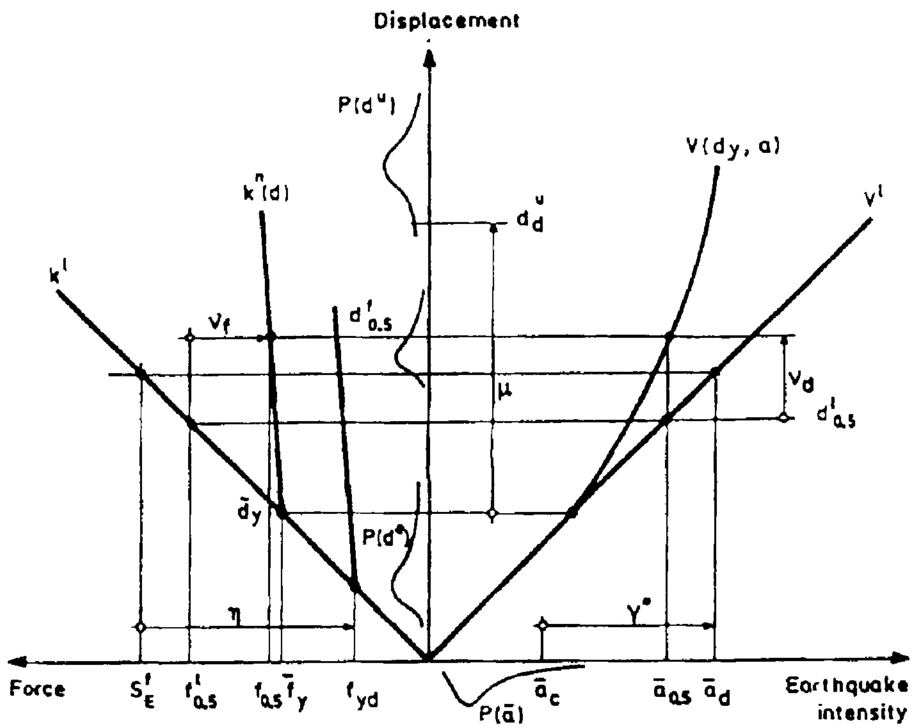

Figure 2: Correspondence between design process and safety checking

The constants of the corresponding safety checking procedure are quantified in the following way:

Choice of the 50% fractile $d_{0.5}$ of $P(p^f)$ as the checking point. To this displacement corresponds an intensity $\bar{\alpha}_{0.5}$. of the earthquake action;

Determination of the partial safety factor on the earthquake action $\gamma_{qE1}=\bar{\alpha}_{0.5}/\bar{a}c$;

Determination of the behaviour coefficient for displacements $\nu^d = V(\bar{d}_y, \bar{a}_{0.5})/V^l(\bar{a}_{0.5})$ and of the behaviour coefficient for forces

$$\nu^f = k^n(d_{0.5})/k^l d_{0.5};$$

Determination of the partial safety factors on the effects of the earthquake action:

$\gamma^d_{qER} = d_{0.005}/d_{0.5}$ when the effect considered is the maximum displacement;

$\gamma^f_{qER} = f_{yc}/k^n(d_{0.5})$ when the effect considered is the maximum force.

Since it is the maximum displacement which is the control variable the behaviour coefficient and partial safety factor to be considered are V_d and γ^d_{qER}. . Since the checking procedure is linear, it is possible to combine γ^d_{qER} into a single coefficient $\gamma_{qE} = \gamma_{qEI}\gamma^d_{qER}$ to be used on the action effects.

EARTHQUAKE GROUND MOTION

The concepts presented above will be illustrated by an application to Lisbon. Seismicity is idealized by a source-zones model with non-radial attenuation functions [13] and results are obtained through a computer algorithm based on McGuire's. Due to the particularities of the portuguese seismicity, where nearby moderate magnitude and long distance large magnitude earthquakes may be expected, these two types of earthquakes were considered separately [2]; if the focal distance is smaller or larger than 40 km earthquakes are considered to be near distance (type 1) or long distance (type 2). The hazard model adopted is constituted by the probability distribution of the maximum peak ground acceleration in a period of 50 years. This probability distribution is idealized as a Gumbel type I distribution·

$$f(\bar{a}) = \alpha(exp - \alpha(\bar{a} - u) - exp(-\alpha(\bar{a} - u))) \qquad (6)$$

The values of α and u were calibrated from numerical results obtained from the source-zones model and table the values $\alpha = 0.00225$ and $u = 87.36$ for the type 1 earthquakes and α 0.0031 and $u = 60.5$ for the type 2 earthquakes (with $\bar{\alpha}$ expressed in cm/s^2).

In order to work with earthquake motions representative of those for which failures are likely to happen, the hazard study was carried out for a return period of 10,000 years. The corresponding peak ground accelerations are 317 cm/s^2 for type 1 earthquakes and 233 cm/s^2 for type 2 earthquakes. The most likely earthquakes have a 7.1 magnitude and a 18 km focal distance and 8.3

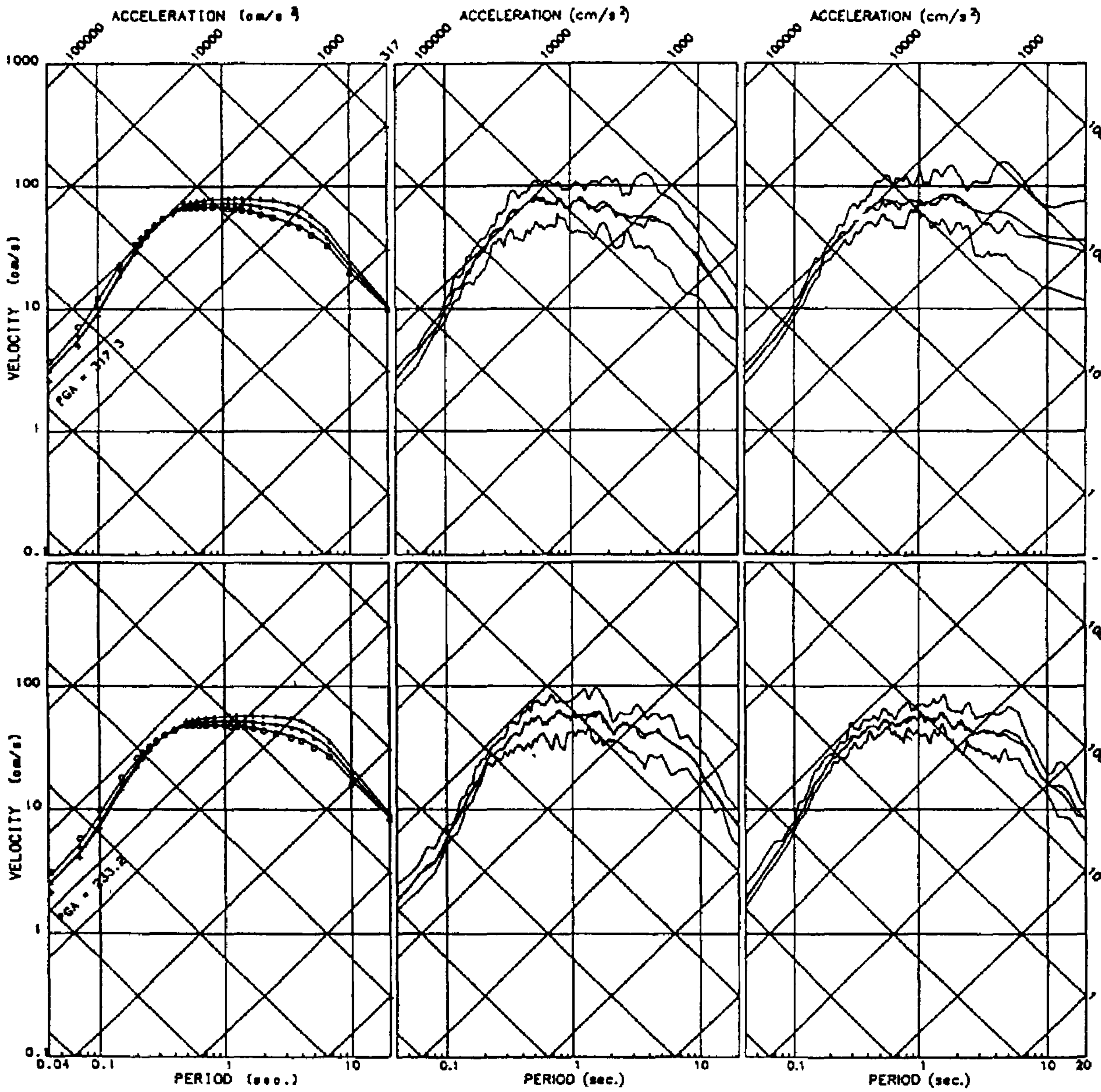

Figure 3: Reference response spectra (left column) for rock (o), soft (+) and intermediate soil (*) for type 1 (upper row) and type 2 earthquakes (lower row) and the corresponding sample spectra from the non-stationary (central column) and stationary models (left column); the mean and median values and 10% and 90% fractiles are plotted

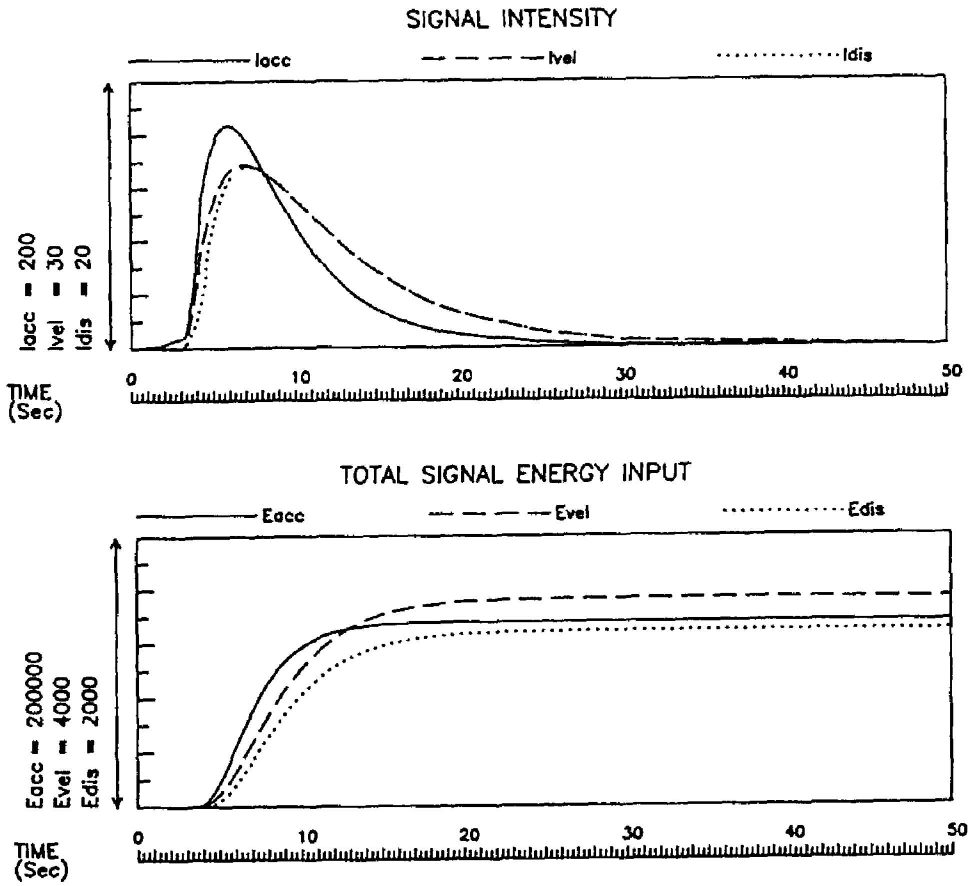

Figure 4: Time evolution of the expected signal intensity and total signal energy input of acceleration ( ), velocity ( ) and displacement ( ) of the type 1 earthquake non-stationary model (Units: cm, s).

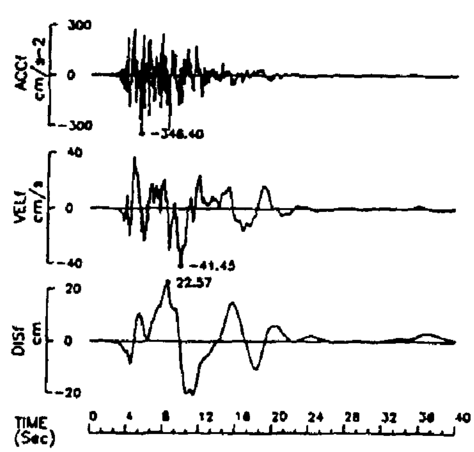

Figure 5: A realization of the type 1 non-stationary model.

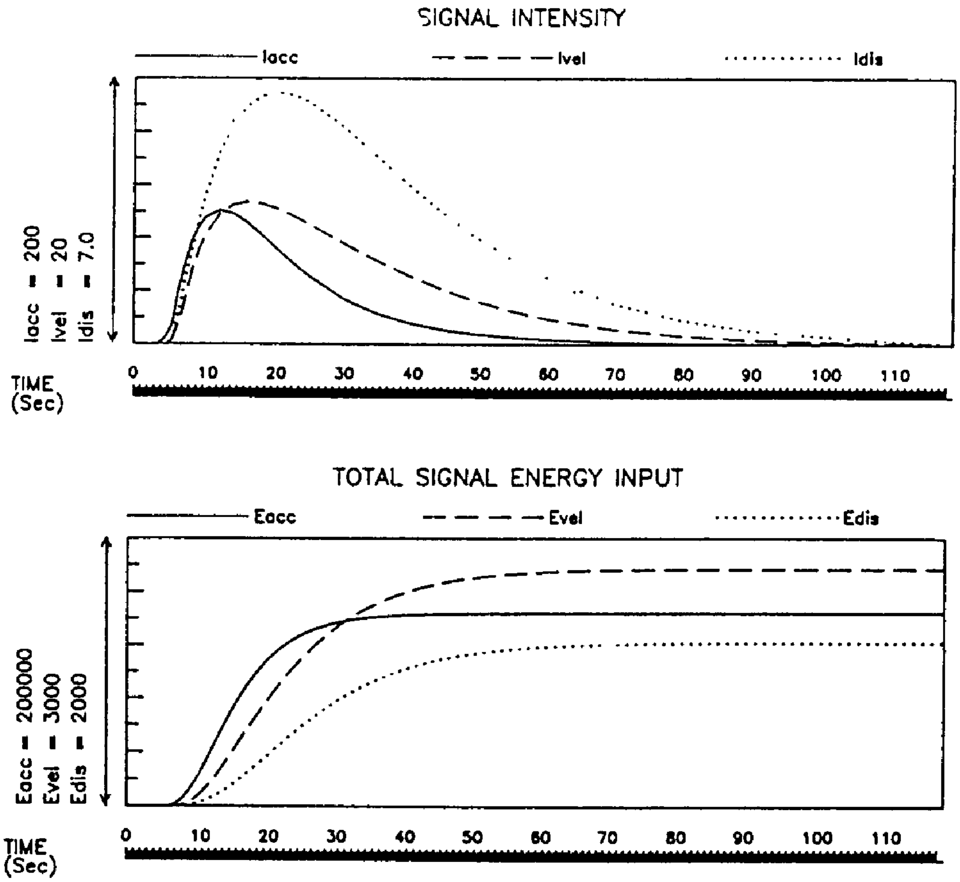

Figure 6: Time evolution of the expected signal intensity and total signal energy input of acceleration (——), velocity (- - -) and displacement (······) of the type 2 earthquake non-stationary model (Units: cm, s).

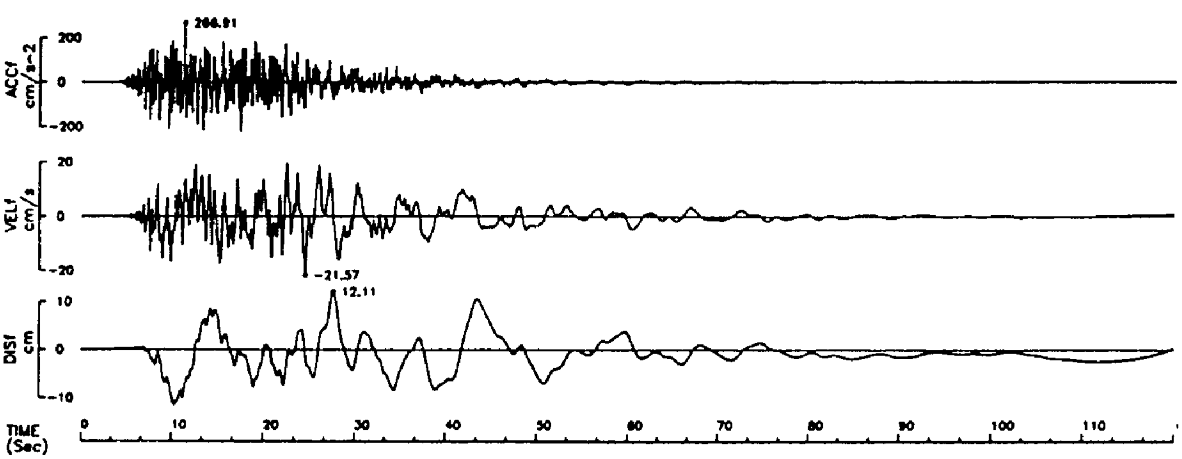

Figure 7: A realization of the type 2 non-stationary model.

magnitude and a 226 km focal distance [2], reference response spectra for those earthquakes were obtained through empirical models [19].

Two samples of 10 realizations were generated for each type. The first sample was obtained from a gaussian stochastic model with non-stationary amplitude and frequency [3] quantified by data obtained from recorded ground motion [12]; the second sample is constituted by time-segments of a gaussian stationary stochastic model, whose duration is identical with the Trifunac and Brady duration [18]; those durations are 7.2 s for type 1 earthquakes and 24 s for type 2 earthquakes. Both stochastic models were scaled to the corresponding reference response spectra. The comparison between those reference response spectra and the response spectra obtained from the samples are presented in figure 3 for a 5% damping. Some of the more descriptive time-dependent statistics of the nonstationary models are presented in figures 4 and 6. A realization of each non-stationary model is presented in figures 5 and 7. Non-stationary models are considered to idealize real earthquake vibrations while the stationary models are considered to be simplified models to be used in nonlinear design [16].

In terms of the results presented in figure 3 the differences in the response spectra for different soil conditions may be considered small, specially for the period range 0.2 - 1 s , which is the range of interest for most multistorey buildings; in consequence, this study was carried out only for intermediate soil conditions and the conclusions may be expected to apply to other types of soil unless there are very peculiar geological characteristics.

DESCRIPTION AND MODELING OF THE BUILDINGS CONSIDERED

Buildings of 2, 4 and 8 storeys with a reinforced concrete frame structure with and without masonry panels were considered. The structure of those buildings was based on the 8-storey building of a series of buildings that have been extensively used for earthquake studies [5]. The buildings were analysed for a single direction; the reinforced concrete frame was idealized by a simplified model with only one column and one beam per storey, having the possibility of hysteretic zones at the top and bottom of the column and at the beam end on the column side; the other beam end is only restrained in the vertical direction [2]. The stiffness due to the masonry was idealized by a shear beam. Natural frequencies of vibration are shown in Table 1.

Table 1 - Natural frequencies of the fundamental modes of vibration (Hz)

No. of Storeys	With masonry panels	Without masonry panels
2	6.1	2.6
4	3.5	1.5
8	1.9	0.9

The inelastic behaviour was represented by force deformation characteristics idealized by Takeda-type loops [4, 7] with stiffness degradation, strength degradation and pinching effect.

The buildings were designed for earthquake resistance (in terms of yielding values) through a dynamic linear analysis and the type 1 spectrum for a 10 000 years return period. In order to promote a good dissipation mechanism different ductility coefficients were used for beams ($\eta = 4$) and for columns ($\eta = 2$); it was also assumed that ductile capacity was also different for beams ($\mu = 15$) and for columns ($\mu = 7.5$) with the purpose of obtaining a favorable collapse mechanism.

A dissipation mechanism is a given distribution of hysteretic zones in the structure. A collapse mechanism is formed when in several hysteretic zones the ductility demand is so high that results in total strength degradation; the number of hysteretic zones with total strength degradation (hinges) must be greater than the degree of redundancy for a collapse mechanism to be formed; a global collapse mechanism is a mechanism which corresponds to a complete loss of resistance to horizontal loads [1].

According to current practice in Portugal, the building design was carried out only for the reinforced concrete structure i.e. this structure was designed to have strength enough to withstand the earthquake equivalent lateral forces, and the masonry panels were supposed to be not stressed by the earthquake action.

RELIABILITY ASSESSMENT

Principles

In terms of earthquake withstanding capacity, the concept of 'resistance' has two components: one related to the yielding values and the other related to the ductile capacity i.e. the structure may present the same reliability with a given yielding value and a given ductile capacity or with a greater yielding value and a smaller ductile capacity. In general the yielding values are defined by the design process and the ductile capacity is supposed to be defined by a given ductility class; a ductility class is defined by a set of rules about the detailing of the reinforcement and the quality of the concrete (e.g. confinement of concrete, % of longitudinal reinforcement); and a ductility class defines the probability distribution of the ductile capacity considered as a random variable. Its distribution may be taken as a log-normal distribution with a coefficient of variation $c_\mu = 0.3$ [14]. The log-normal distribution is given by the following density function:

$$f(\mu) = \frac{1}{\mu\delta\sqrt{2\pi}} \exp\left(-\frac{\ln^2(\mu/\beta)}{2\delta^2}\right) \tag{7}$$

where μ is the value of the ductile capacity and β and δ are the parameters that quantify the distribution. It is assumed that the ductile capacity is quantified by the 0.005 fractile $\mu_{0.005}$. Hence the values of β and δ are given by $\delta = \sqrt{\ln(1 + c_\mu^2)}$, $\beta = \mu_{0.005} \exp(2.576\delta)$. The choice of the 0.005 fractile to quantify the ductile capacity distribution makes the probability not very dependent on the actual value of the coefficient of variation c_μ [8], as have been already pointed out. The other component of the earthquake 'resistance', the yielding values, are defined by the design process and are taken as deterministic quantities, corresponding to the mean values of their probabilistic distributions.

The computation of the probability of failure is carried out through the convolution integral of the density probability $f\alpha(x)$ of the action with the cumulative probability $Fr(x)$ of the resistance [11]:

$$P^f = \int_o^\infty f_a(x) \; F_r(x) dx \tag{8}$$

The adaptation of this expression to the present analysis is performed by selecting as the domain of integration the values of the mean peak acceleration $\bar{\alpha}$ and consequently by expressing the 'resistance' of the structure in terms of that variable:

$$P^f = \int_o^\infty f(\bar{a}) \; F_\mu(\bar{a}) d\bar{a} \tag{9}$$

where $f(\bar{\alpha})$ is given by expression (6) and the 'resistance' of the structure is represented by the cumulative distribution $F\mu \; (\bar{\alpha})$ of its overall ductile capacity, expressed in terms of action intensity.

The overall ductile capacity is determined through the definition of limit states [1]: It will be considered that the collapse of the structure is deemed to occur when a collapse mechanism is formed. It should be remembered that one of the purposes of the design methods is to ensure that favorable dissipation and collapse mechanisms are achieved.

The relationship between action and actions effects (ductility demand) is expressed by the vulnerability function $V_i \; (\bar{\alpha})$ which gives the ductility demand in the i^{th} hinge as a function of the peak acceleration. Hence, the function $\bar{\alpha}=V_i^{-1}(\mu)$ gives the mean peak acceleration that corresponds to a given ductility demand. The function $V_i^{-1}(\mu)$ may be used to express the probability distribution of the ductile capacity in terms of mean peak acceleration $\bar{\alpha}$. Let $f^\mu_i \; (\mu)$ be the probability density of the ductile capacity of the i^{th} hinge which is given by expression (7) with parameters β_i and δ_i. In consequence: $f_i(\bar{a}) \equiv f_i^\mu(V_i^{-1}(\bar{a}))$. The corresponding cumulative distribution $F_i(\bar{\alpha})$ is given by

$$F_i(\bar{a}) = \int_o^{\bar{a}} f_i^\mu(V_i^{-1}(x)) dx \tag{10}$$

Under the assumption that the ductile capacities of the different hinges are stochastically independent, which is an acceptable assumption [10], the cumulative distribution $F_i(\bar{\alpha})$ of the ductile capacity of global colapse mechanism is obtained by $F_i(\bar{\alpha}) = \Pi_i F_i(\bar{\alpha})$. The probability P^g of occurring a collapse is computed by:

$$P^g = \int_0^\infty f(\bar{a})F(\bar{a})d\bar{a} \tag{11}$$

Numerical Results

The principal aspects of the reliability analysis performed on the ensemble of buildings considered in this study are presented in tables 2 and 3. In those tables columns (1) give the probability of failure. Columns (2) give the corrective factors that can be used to obtain a 10^{-5} probability of failure e.g. the 2 storeys building withou masonry panels should be designed with a ductility coefficient η=0.73 X2=1.46 for the colun.. and a ductility coefficient η= 0.73X4=2.92 for the beams. Columns (3) present the values $\bar{\alpha}_{0.5}$ of the peak acceleration corresponding to a 10^{-5} probability of failure and the 50% fractile of the probability distribution of the probability of failure (figure 2); those values were obtained by the approximate method of dividing the $\bar{\alpha}_{0.5}$ values fo the probability of failure indicated in the tables by the corresponding values of the columns (2), which is an acceptable method if the vulnerability function has only a small curvature.

Table 2. Risk from near distance earthquakes

No. of storeys	Masonry panels	Non-stationary model			Stationary model		
		(1)	(2)	(3)	(1)	(2)	(3)
	NO	360	0.73	640	100	0.82	625
2	YES	29	0.92	606	9.8	1.00	608
	NO	1.4	1.13	610	2.1	1.12	638
4	YES	1.8	1.28	616	0.26	1.26	630
	NO	0.37	1.26	660	1.2	1.15	605
8	YES	0.004	1.61	551	0.008	1.54	589

Table 3. Risk from long distance earthquakes

No. of storeys	Masonry panels	Non-stationary model			Stationary model		
		(1)	(2)	(3)	(1)	(2)	(3)
	NO	61	0 95	426	5 5	1 14	459
2	YES	0.3	1 26	451	0 7	1 19	428
	NO	0.03	1.46	452	0 002	1 63	420
4	YES	0.002	1.64	416	0 001	1 67	482
	NO	≈ 0	2.03	472	≈ 0	1 97	430
8	YES	≈ 0	2.53	425	≈ 0	2 52	359

Notes to tables 2 and 3
(1) probability of failure $\times 10^{-6}$
(2) corrective factores of the ductility coefficient for a 10^{-5} probability of failure
(3) checking intensity $\bar{a}_{0.5}$ of the earthquake action

Comments

Although the authors intend to present, in a paper to be published in a near future, a detailed analysis of the structural aspects involved, it seems worthwhile to briefly point out some of them:

i There is an acceptable equivalence between stationary and non-stationary models of the earthquake vibration in regard to the evaluation of the probability of failure;

ii The existence of masonry panels is associated to smaller probabilities of failure;

iii The fact that risk due to long distance earthquakes is clearly smaller than risk due to near distance earthquakes proves that in this case response spectra is a good measure of the earthquake severity;

iv The use of non-frequency dependent ductility coefficients may be responsible for important variations of the reliability with the number of storeys.

IDENTIFICATION OF DESIGN SEISMIC ACTIONS

It really does not seems possible to define design seismic actions without having a complete framework where design and safety checking may be given a rigorous meaning and where their relationship may be established and defined. This framework, however, can and should also be used for the quantification of all other quantities of interest, namely the ductility coefficients. The dependence of definition between the quantities involved is just a consequence of the fact that a truly nonlinear problem cannot be split into 'independent' parts which can be dealt with one by one. The methodology and concepts presented in this paper have been developped as a preparatory work for the quantification of the Eurocode 8 coefficients for the portuguese seismicity and 'optimal' levels of public safety [6].

It is notable that the $\bar{\alpha}_{0.5}$ are very similar for an ensemble of structures with significantly different dynamic properties and structural characteristics (presence or absence of masonry panels), but with the same probability of failure . This fact suggests that an average of those $\bar{\alpha}_{0.5}$ values can be considered as the intensity of the earthquake action to be used in nonlinear analysis. On the other hand the $\bar{\alpha}_{0.5}$ values are very high values which corresponds to very small hazard probabilities; thus, in the present state of uncertain knowledge about very low probability earthquake occurrences, it seems advisable to quantify the earthquake hazard in terms of the 10 000 years return period and use appropriate γ^* coefficients. The advantage in using higher intensity earthquakes and a smaller γ^* instead of smaller intensity earthquakes and an higher γ^* is due to the fact that the 'qualitative' aspects of the higher intensity earthquake (i.e. those aspects which are not taken into account by the hazard function) are more close to the 'qualitative' aspects of the earthquakes more likely to originate failure.

In principle, the vulnerability functions should be recalculated with earthquake actions corresponding to the average of the $\bar{\alpha}_{05}$ values; but since those actions have qualitative characteristics not significantly different from the ones used, and since the purpose of the present analysis is more to illustrate the procedure than to establish data for design, the recalculation of the vulnerability functions was thought unnecessary.

The representation of the earthquake actions by a stationary model gives almost the same results, in terms of probability of failure evaluation, as the non-stationary model. Hence, the stationary model may be used, specially when its shorter duration would correspond to a significantly smaller amount of computational work.

The numerical results obtained make clear that it is presently possible to evaluate the safety of structures taking into account its nonlinear behaviour, and the probability distributions of the earthquake action and the structural 'resistances'. In consequence it is important that codes include a much more complete definition of the seismic hazard or, at least, should quantify the design earthquake for smaller probabilities of occurrence. This evolution of codes seems specially important for buildings with unusual configurations and/or with structural systems.

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