THE SEISMIC FRAGILITY OF BASE-ISOLATED NPP BUILDINGS

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ABSTRACT

The research work here described is devoted to the development and testing of a numerical procedure for the computation of seismic fragilities for equipment and structural components in Nuclear Power Plants (NPP). The proposed procedure for fragility computation makes use of the Response Surface (RS) Methodology to model the influence of the random variables on the dynamic response. To account for stochastic loading the latter is computed by means of a simulation procedure. Given the RS, the Monte Carlo method is used to compute the failure probability. The procedure is here applied to the preliminary design of the NPP reactor building within the IRIS international project; a base isolation system based on the introduction of HDRB (High Damping Rubber Bearing) elements is considered, leading, on one hand, to a markedly non linear behavior. On the other hand, when supported by an isolation system, the building tends to behave as a rigid body, this allowing for a low-dimensional model for capturing the dynamic response under seismic excitation. The fragility analysis is performed on the basis of a tentative limit-state function relating horizontal and vertical loads applied to isolation devices.

INTRODUCTION

The introduction of isolation systems in the design of strategic buildings is likely to become, in the next future, a widespread seismic protection measure. In fact, the anticipated better performance, when compared to the one of traditional buildings, in terms of acceleration response to design seismic actions, makes isolation techniques a very attractive choice for buildings whose functionality after the event is of utmost importance.

This is the typical case of reactor buildings in future Nuclear Power Plants, where the adoption of isolation systems seems to be almost mandatory if the frequency of earthquake-induced accident scenarios must drop to values of the order of E-08/ry to make the seismic risk of the same order of magnitude if compared to risk related to internal events. In this respect, isolation systems based on High Damping Rubber Bearings (HDRB) represent a highly reliable design solution, given their diffusion and proven reliability; in [9] the application of this solution to the Nuclear Steam Supply System (NSSS) building designed within the IRIS international project [3] is described and some preliminary results are given. These results show how the isolation system is extremely effective in reducing the seismic acceleration transmitted to all structural and equipment components inside the building, this resulting in significant advantages for the design and standardization of the equipment. It must be observed, however, that in such conditions the isolation devices themselves are prone to become the critical components in terms of seismic fragility, since the response attenuation is obtained as the result of large relative displacements between the building and the foundation. Therefore, the need arises to assess the risk associated to the failure of the isolators, this being obviously related to “beyond design” loading conditions.

When the problem of risk evaluation is addressed, the fundamental role played by the hazard definition in the estimation of the seismic-induced CDF is immediately evident; both randomness and uncertainty, however, significantly affect also the evaluation of structural behaviour under extreme loads and thus of seismic reliability. Though randomness cannot be avoided, since is inherent to most of the input data of the analysis, uncertainties, being related to the lack of complete and accurate knowledge about models and methods, must be reduced as much as possible by refining analysis procedures.

In light of the above considerations, the research activity described in the present paper is devoted to the probabilistic evaluation of the seismic performance of a NPP reactor building, focusing on the case in which an isolation system is introduced. An innovative procedure for fragility estimation [6] will be summarized in the next section; an example of application will be subsequently shown with reference to the IRIS (International Reactor Innovative and Secure) system. This is a medium power (335 MWe) pressurized light water reactor under development by an international consortium which includes more than 20 partners from 10 countries (see [3,10])

FRAGILITY ANALYSIS OF NPP BUILDING COMPONENTS

Formulation

Single Following the PEER (Pacific Earthquake Engineering Research) approach (see [7] and references herein), the annual failure rate for a mechanical component under seismic loading can be obtained from the integral:

$$P_f = \iint P\left\{DM > dm_f \,\middle|\, EDP = edp\right\} p_{EDP}(edp|IM = im) p_{IM}(im) d(edp) d(im) \tag{1}$$

where *DM* is a Damage Measure, associated to the assumed limit state (dm_f denotes the damage level at failure), *EDP* is an Engineering Demand Parameter (support acceleration, relative displacement,…) expressing the level of the dynamic excitation imposed to the component due to the global seismic response of the structure (reactor building) and *IM* is an Intensity Measure (peak ground acceleration, spectral acceleration,…) characterizing the severity of the earthquake motion at the reactor site. As pointed out in [7] all statistics in (1) must be intended in term of annual extreme values, so that the equation delivers a risk estimate in terms of annual probability of failure of the component. For a "simple" equipment component, or for a preliminary evaluation, the limit state can be directly defined in terms of the *EDP* value at failure edp_f, thus avoiding the damage analysis step, i.e:

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where the integrand function can be written in terms of the fragility function *F(edp,im),* defined as:

$$F(edp, im) = P\left\{EDP > edp \,\middle|\, IM = im\right\} = 1 - P_{EDP}(edp / IM = im) \tag{3}$$

When a traditional non-isolated building is considered a typical choice for *edp* within the above context is represented by the peak acceleration at the support point of critical pieces of equipment. If a base-isolation system based on HDRB (High Damping Rubber Bearings) is introduced (see for example [11]) the acceleration values inside the building undergo a dramatic decrease. This is obtained at the price of significant relative displacements imposed to isolation devices, which are likely to become the "weakest link" in terms of seismic safety of the building; therefore the extreme value *u* of the relative displacement across the most strained isolator is a quite obvious first choice for the *edp*. The fragility function is therefore expressed as:

$$F(edp, im) = P\left\{U > u \,\middle|\, A_g = a_g\right\} = P_{exc}(u, a_g) \tag{4}$$

For a system under stochastic dynamic excitation, the associated limit state function can be expressed in the following "capacity minus demand" format:

$$g(\mathbf{X}, u, a_g) = C - D(\mathbf{X}, a_g) = u - U(\mathbf{X}, a_g) = 0 \tag{5}$$

where U is the random variable whose distribution delivers, for fixed **X**, the result of the random vibration analysis.

In [6] linearity of the building response was exploited, in the non-isolated case, for the probabilistic evaluation of the peak motion at the supports of an equipment component; differently, no linearization is here exploited, since the behaviour of HDRB is markedly non-linear, especially at the high level of deformations here anticipated. Once selected the probability distributions of X and U the probability of exceeding the limit state (6), i.e. the integral

$$P_{exc}(u, a_g) = \iiint_{g<0} p_U(u, \mathbf{x}) \, du \, d\mathbf{x} \tag{6}$$

could be computed, in principle, by direct application of the Monte Carlo Simulation (MCS) method; in fact the statistics of response U(X) is algorithmically known, i.e. can be deterministically computed by structural dynamic analysis for every realization of the random variables **X**. It must be considered, however, that a huge computing time and cost would be required for running a non-linear dynamic analysis, as it is the case for isolated systems of the type here considered, for the number of evaluations which are necessary for MCS, especially for the estimation of small probabilities.

Limit-state approximation via Response Surface Methodology

For the above consideration, according to the well-established Response Surface Methodology (RSM - [8], the "true" response function is replaced by a simple analytical representation. Here, assuming that the distribution of U can be described by its mean value μ_U and its standard deviation σ_U, the so called "dual response surface" approach [12] has been adopted for modelling their dependency on X. Assuming that the same model can be used for the mean and the standard deviation the following response functions have been introduced:

$$\mu_U(\mathbf{X}) = \sum_{i=1}^{m} a_i z_i(\mathbf{X}) + \varepsilon_\mu \qquad \sigma_U(\mathbf{X}) = \sum_{i=1}^{m} b_i z_i(\mathbf{X}) + \varepsilon_\sigma \tag{7}$$

where the a_i's and b_i's are coefficients to be estimated, the z_i's are usually polynomial functions and two "error" terms (ε_μ, ε_σ) are introduced as a zero mean random deviations. The latter account for the variability of estimated quantities and for the lack of fit of the adopted model, i.e. for the inadequate analytical form of the RS's and for missing variables (i.e. not comprised in (8) though influencing the response). To compute the coefficients in (8) a number of experiments must be run according to the chosen experimental design; at each of them the random vibration problem can be addressed via either an analytical or a simulation approach. In the second solution, here adopted, a sample of ground motion realizations must be generated, according to the spectral parameters appearing in X. For each realization, the extreme value of U is computed (e.g. via FE modelling and step-by-step analysis); the mean and variance of U are then estimated. The procedure is repeated for all experimental points, leading to n observed values for the statistical parameters of $U(\mathbf{X})$.

We shall assume in the following that the experiments are performed in homogeneous conditions (i.e. differing for the x_i values only), that their results are independent and that the error terms are normal with constant variance; under these hypotheses an unbiased estimate of the coefficients a_i,b_i can be obtained by the Least Square (OLS) method, independently of the variance of the error terms (ε_μ, ε_σ). An unbiased estimate of the latter terms can be subsequently obtained is defined, in terms of the residual values. Once models (8) are established, MCS can be carried on. In all applications here shown a Central Composite Design has been adopted for defining the experimental points.

Note that, differently from the linear case, analyzed in [6], the Response Surface evaluation must be repeated for every value of peak ground acceleration, this representing, potentially, a huge computational task. It can be considered, however, that in the isolated case the seismic behavior of the building can be captured, to the aim of evaluating the isolators' performance, by means of very simple mechanical models; the latter, in fact, can be based on the hypothesis of rigid-body motion of the building above the isolators.

As a second option the "capacity minus demand" function (10) can be associated to a limit state function [5] expressed in terms of horizontal and vertical loads acting on the most severely strained isolator; for typical HDRBs, made by alternate rubber and steel layers, the limit state here considered is the "first damage" condition related to the attainment of an admissible peak tensile stress at the steel-rubber interface. In this light and with reference to Fig. 1, demand is here defined, at each instant, as the distance between the points describing the static vertical loading on the isolator (point S) and the current loading (point C), while capacity is computed as the total distance, measured along SC, between the static loading condition and the limit state surface (point L). Both are made non-dimensional with respect to the capacity so that the EDP is represented by the ratio SC/SL, this being the inverse of the "instantaneous" safety factor, while the limit value edp_f takes a constant unit value. Under these assumption and criteria the limit-state function takes the form:

$$g(\mathbf{X}, a_g) = C - D(\mathbf{X}, a_g) = 1 - \frac{SC(\mathbf{X}, a_g)}{SL(\mathbf{X}, a_g)} = 0 \tag{8}$$

As an alternative, the actual distance CT can replace CL in the formulation of the safety factor. Note that considering first damage instead of actual failure in the formulation of the limit state surface appears to be reasonable choice until a complete experimental characterization of the behaviour of large HDRBs at collapse will be available.

Finally, in [6], a risk-based procedure for refining the Response Surfaces and the evaluation of the failure probability is proposed for the linear case; application to non-linear systems is at study.

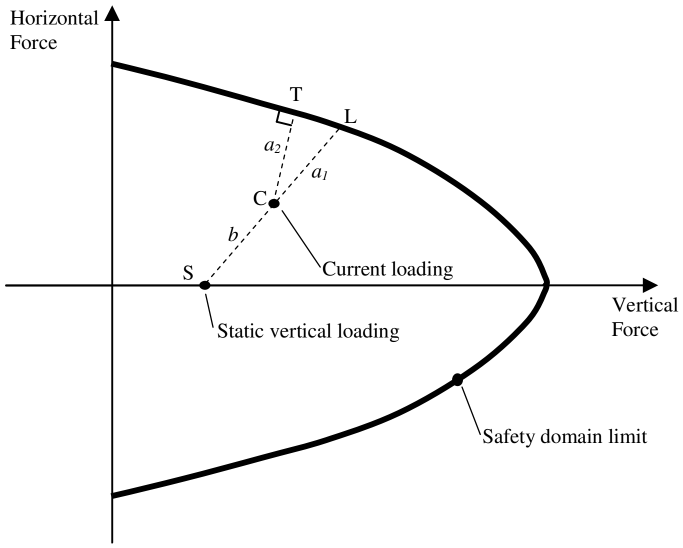


Fig.1: Limit state function for the isolator

EXAMPLE OF APPLICATION

The isolated IRIS NSSS building

The above described procedure has been applied to the analysis of a preliminary design of the reactor building of IRIS (International Reactor Innovative and Secure). This is a medium power (335 MWe) pressurized light water reactor under development by an international consortium which includes more than 20 partners from 10 countries (see [3,10]). Installation in a site characterized by a low-to-average seismicity level has been here assumed.

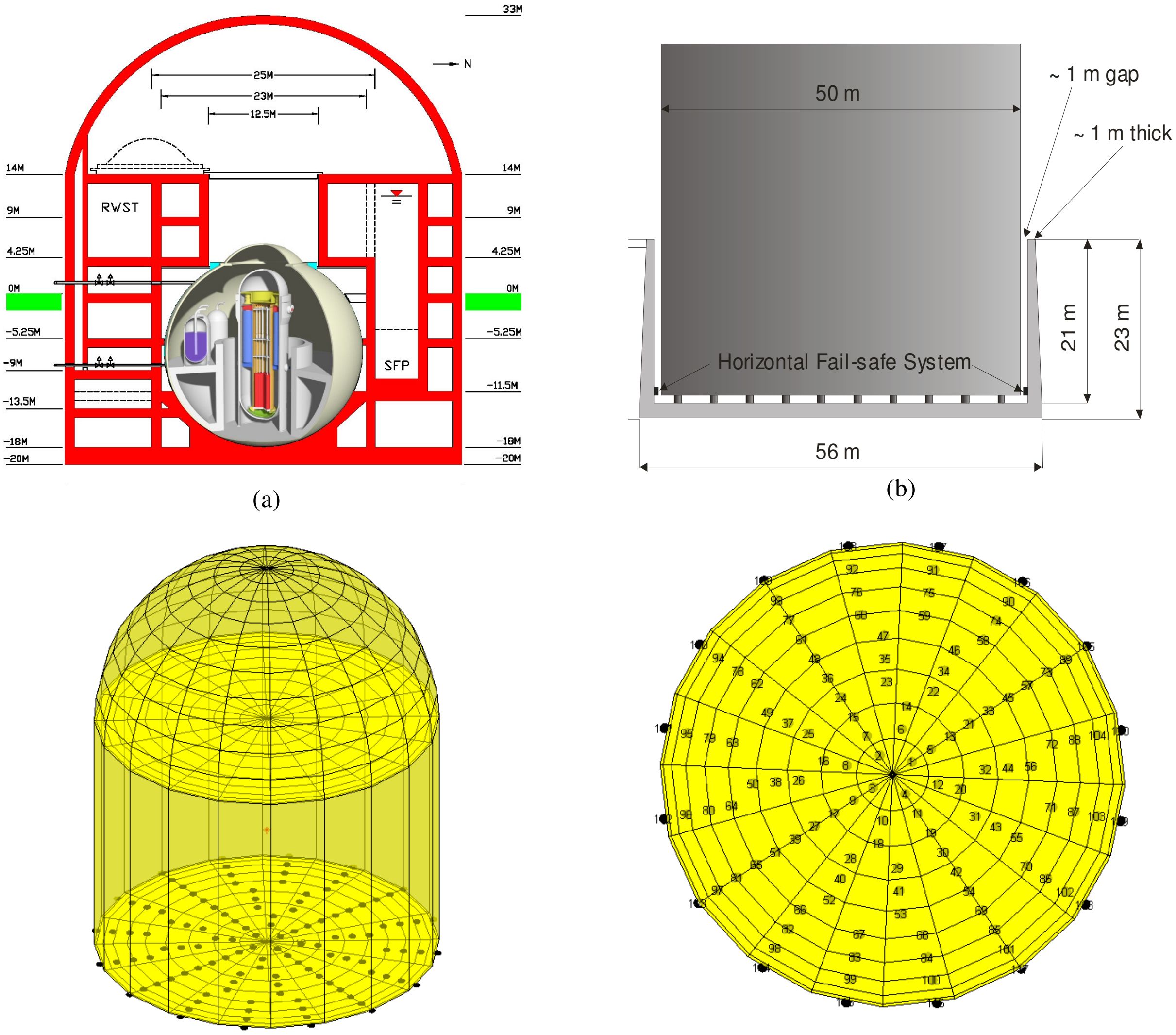
In a tentative design of the NSSS building (see Fig. 2a), the introduction of an isolation system was considered; the system is made by High Damping Rubber Bearings (HDRB) installed between the foundation slab and the base (Fig. 2b). The main scheme of the isolators layout, as considered in the preliminary design approach, is depicted in Figs 2c-d. The HDRB devices are made of alternated rubber layers and steel plates, bonded through vulcanization. Damping factor ranges from 10% to 20%, while shear modulus (G) lies in the 0.8-1.4 MPa range.

Steel plates give a high vertical stiffness to the isolator, though allowing large horizontal deformations. Therefore, the isolated building has low natural frequencies for motions lying in the horizontal plane, typically in the range 0.5 - 0.7 Hz, where the spectrum of ground motion has generally quite low energy. In such vibration modes the isolated building moves like a rigid body over the isolators, which are strained in shear (continuously carrying the dead load). The absolute acceleration of the building can be much smaller than the PGA, with no amplification at higher floors. This is obtained at the price of large relative displacements between the building and the adjacent ground; this can be a problem for the design of the expansion joints and the connections with non isolated buildings of all the pipelines and service networks. The design of the isolation system, therefore, must reach a reasonable compromise between limitation of absolute accelerations and relative displacements. For the case of the IRIS NSSS, having a fixed-base first natural frequency of 5.91 Hz (on firm ground) and natural frequencies around 9 Hz for the vessel local motion, this led to a 0.7 Hz isolation frequency, i.e. to a value which can be seen as an upper limit for the parameter. If some equipment component (e.g., some wide span pipeline) has a lower natural frequency a local specific measure (stiffening or energy dissipation device) must be adopted.

The choice of 0.7 Hz as isolation frequency limits the relative displacement between the isolated building and the ground to 10 cm at the SSE level which is advantageous both for the performance of the isolators in beyond design conditions and for the design of steam lines connecting the NSSS building with the turbine units.

Dynamic modeling and fragility analysis criteria

In the following, to compute the fragility of the IRIS isolation system a simplified approach will be followed with reference to the following aspects.



(a) (b) (c) (d)

Fig. 2: IRIS NSSS building (a); isolation system (b); isolators layout (c-d)

- A 6-DOF plane model has been adopted for the reactor building, under the hypothesis that the isolated superstructure behaves like a rigid body; soil-structure interaction has been neglected.
- The behavior of isolators under horizontal and vertical loading has been regarded as independent; it is assumed that isolators behave as linear elastic under vertical loading, showing the same stiffness in tension and compression. Their non linear behavior under horizontal loading has been modeled according to the two-dimensional model described in [1].
- To define the limit state function (Fig. 1), it was assumed, as a tentative criterion, that under the static vertical load a 400% shear deformation leads to the attainment of an admissible peak tensile stress at the steel-

rubber interface. This rather drastic assumption allows for the definition of the limit domain in the σ-τ plane by the straightforward numerical methodology proposed in [5]. Subsequently, the domain is expressed in terms of the horizontal and vertical forces acting on the most severely strained isolator. In parallel research activity the refinement of the limit state function is pursued.

- The seismic input has been introduced by means of twenty artificial time histories, whose spectra individually match the response spectrum prescribed by Regulatory Guide 1.60.

Four independent random variables have been considered in the fragility analysis; two of them account for the uncertainty of the limit state function (Fig.1). A parabolic shape has been assumed, governed by two coefficients which have been taken as lognormal with mean value, equal to the one obtained via the procedure [5], and c.o.v. equal to 0.22.

The other two random variables account for the randomness of the dynamic properties of the isolator, represented via the model in [1]; according to this the restoring force is the sum of three contributions, i.e. an elastic-plastic model (F_2 contribution) and two elastic non-linear springs, namely a non-linear elastic spring (F_1) and an hardening spring (F_3); the model allows to reproduce analytically some aspects of the experimental behavior of laminated rubber bearings. In light of these observations the resulting scheme for the Fujino model is represented in Fig. 3a [1].

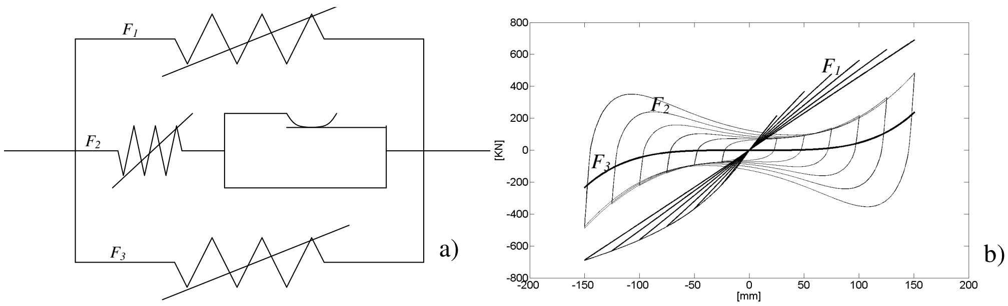


Fig. 3: Hysteretic model for the seismic isolators (a); F_1,F_2,F_3 components of the total isolator reaction force F under cyclic loading (b)

From an analytical point of view the force-displacement relation for the first non-linear spring consists of

$$F_1 = K_1\left\{\beta + (1-\beta)\exp(-\frac{U_{\max}}{\alpha})\right\}U + a\left[1-\exp(-b|U|)\right]\mathrm{sgn}(U) \tag{9}$$

Where U is the relative displacement and K_1, a e b parameters. In Equation (9), the first term reproduces the force linear evolution, while the second one the non-linear behavior. In Fig. 3b the F_1 contribution is depicted for a cyclic experimental test as force-displacement diagram up to displacement values equal to 300% of the rubber height and the stiffness degradation during the variable cycle amplitude is highlighted.

The hysteretic contribution F_2 is described by the differential equation

$$\dot{F}_2 = \frac{Y_t}{U_t}\left\{\dot{U} - |\dot{U}|\left|\frac{F_2}{Y_t}\right|^n \mathrm{sgn}(\frac{F_2}{Y_t})\right\} \tag{10}$$

The values of Y_t and U_t are defined as

$$Y_t = Y_0\left(1 + \left|\frac{U}{U_H}\right|^p\right) \quad ; \quad U_t = U_0\left(1 + \frac{U_{\max}}{U_S}\right) \tag{11}$$

where Y_0 is the initial yielding force, U_0 the initial yielding displacement, U_H the displacement where hardening starts, U_S a parameter for controlling the degradation of the elastic stiffness of the elasto-plastic spring, U_{max} the maximum displacement experienced during the loading history, p a parameter governing the shape of the hardening branch. Fig. 3b depicts the hysteretic components F_2 when the displacement is imposed according to loading cycles of increasing amplitude.

Finally a new non-linear spring is introduced in parallel for capturing the increment of the tangential stiffness experienced by the devices at very high strain levels. This results in the F_3 contribution (see also Fig. 3b), defined mathematically as

$$F_3 = K_2\left|\frac{U}{U_H}\right|^r U \tag{12}$$

where r is the parameter to prescribe the shape of the hardening curve, K_2 the proportional constant to describe the contribution of the hardening spring to the other springs.

To account for the randomness of the dynamic properties of the isolator, represented via the model in [1], a first random variable has been introduced to control the stiffness of the elastic parts by modifying the K_1, a, K_2 and U_0 parameters. The second random variables affects the energy dissipated by the hysteretic portion by modifying the yielding force Y_0. The two r.v.'s are lognormal; for both the mean value is one, which means adopting the values obtained by fitting experimental results, and the c.o.v. is equal to 0.22.

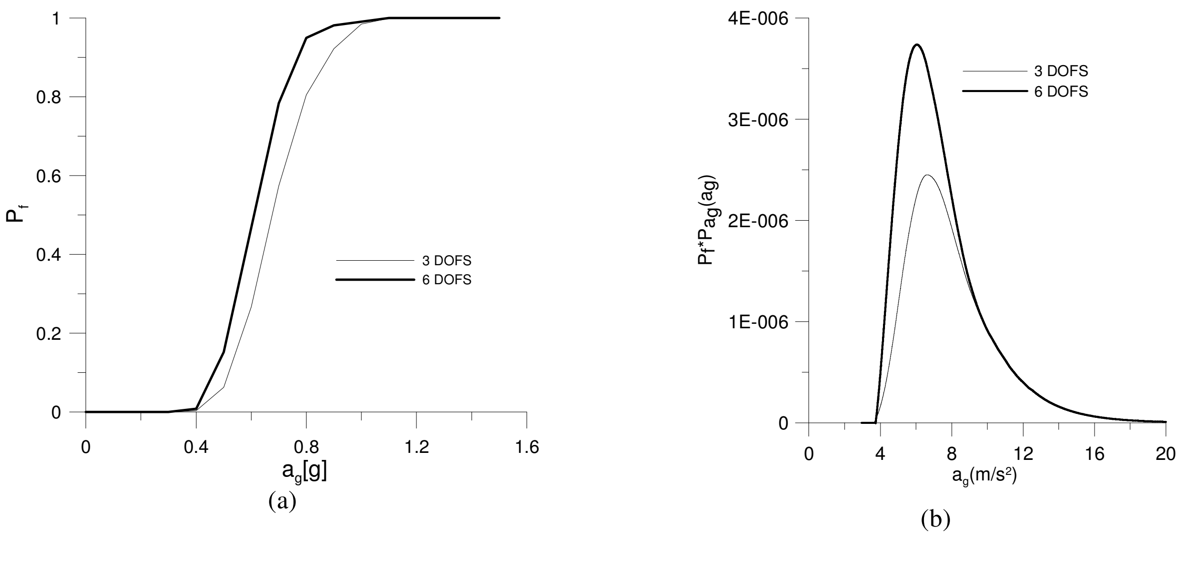


Fig. 4: (a) fragility function, (b) risk integrand function

In Fig. 4 the results of the analysis are presented; in 4a the fragility function relating the exceedance of the limit state function to the peak ground acceleration. In Fig. 4b the integrand function in the risk evaluation (eq. 2) is given; in the evaluation a prototype low-to-medium seismicity site has been considered, characterized by a return period of about 10000 years for a 0.32 *g* PGA. The results here obtained are compared to those given in [13], where a 3-DOF plane model has been used in the dynamic analyses.

The curves given in Fig. 4b clearly point out the PGA range mostly contributing to the risk related to the isolators failure; this is the range in which both mechanical models and fragility analysis tools (e.g. the response surfaces) have to be refined and calibrated. Note that, even though the risk analysis has been here performed according to some crude assumptions, the capabilities of the proposed procedure are highlighted.

As a further development of the procedure, more consistent and refined criteria for defining the limit state function of the isolators are pursued; this activity, which is presently based on extensive finite element testing, will benefit, in the future, by the performance of an experimental campaign.

CONCLUSIONS

A procedure for estimating the fragility of seismic isolation systems in NPP buildings is under development and testing; in this paper a simple example has been shown illustrating the adopted criteria for evaluating the seismic performance of the isolation devices and the associated risk. The procedure delivers the risk associated to the isolators failure and the PGA range mostly contributing to the risk itself, this being a fundamental information for the calibration and refinement of mechanical models and fragility analysis tools.

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