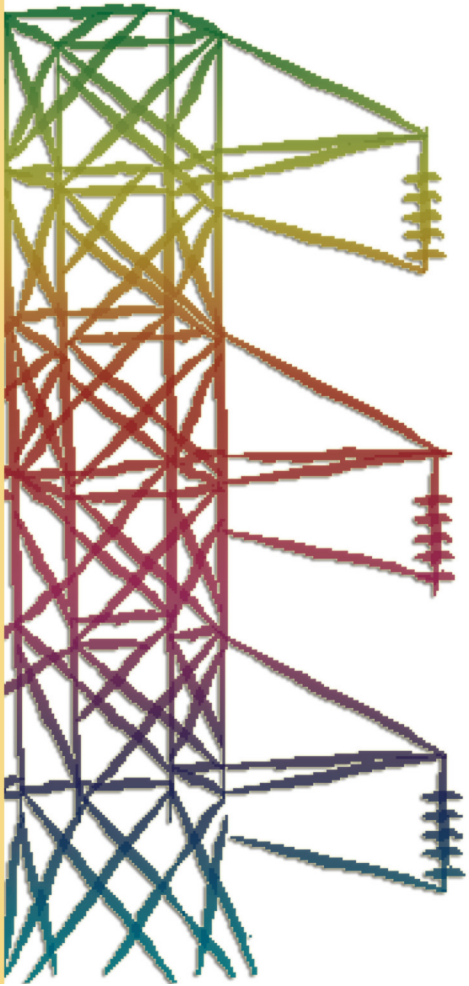
Ente per le Nuove tecnologie,
l'Energia e l'Ambiente

Ministero dello Sviluppo Economico

RICERCA SISTEMA ELETTRICO

Calcolo delle probabilità di rottura (fragility curves) e sviluppo di una metodologia innovativa e sua applicazione all'edificio del reattore IRIS

S. De Grandis, M. Domaneschi, F. Perotti



Report RSE/2009/76

Ente per le Nuove tecnologie,
l'Energia e l'Ambiente

Ministero dello Sviluppo Economico

RICERCA SISTEMA ELETTRICO

Calcolo delle probabilità di rottura (fragility curves) e sviluppo di una metodologia innovativa e sua applicazione all'edificio del reattore IRIS.

S. De Grandis, M. Domaneschi, F. Perotti

Report RSE/2009/76

CALCOLO DI PROBABILITA' DI ROTTURA (FRAGILITY CURVES) E SVILUPPO DI UNA METODOLOGIA INNOVATIVA E SUA APPLICAZIONE ALL'EDIFICIO DEL REATTORE IRIS

S. De Grandis, M. Domaneschi, F. Perotti (CIRTEN)

Dicembre 2008

Report Ricerca Sistema Elettrico
Accordo di Programma Ministero dello Sviluppo Economico – ENEA
Area: Produzione e fonti energetiche
Tema: Nuovo Nucleare da Fissione
Responsabile Tema: Stefano Monti, ENEA

CIRTEN

CONSORZIO INTERUNIVERSITARIO
PER LA RICERCA TECNOLOGICA NUCLEARE

POLITECNICO DI MILANO
DIPARTIMENTO DI INGEGNERIA STRUTTURALE
DIPARTIMENTO DI ENERGIA

TITOLO

Calcolo delle probabilità di rottura al sisma (fragility curves) e sviluppo di una metodologia innovativa e sua applicazione all'edificio del reattore IRIS

AUTORI

Silvia De Grandis

Marco Domaneschi

Federico Perotti

CIRTEN - Polimi RL 1126/2008

Milano 2008

Lavoro svolto in esecuzione della linea progettuale LP2 punto R dell'AdP ENEA MSE del 21/06/07,

Tema 5.2.5.8 – "Nuovo Nucleare da Fissione"

1. Introduction

The continuously increasing level of safety of Nuclear Power Plants (NPPs) against internally initiated events has recently forced the nuclear engineering community to concentrate a significant research effort on the evaluation and mitigation of risks associated to external phenomena such as natural hazards. Among these, as demonstrated by a large number of Probabilistic Risk Assessment (PRA) studies performed worldwide, earthquakes play a very important role. This is because of their unique ability to initiate a fault event and, at the same time, to cause failure of components needed to mitigate the accident itself; in addition, large uncertainties are associated to the earthquake occurrence and to the seismic performance evaluation of components and systems.

This situation requires, on one hand, a higher level of attention on seismic design issues; on the other hand, unnecessary conservatism must be removed, as much as possible, from seismic risk analysis. This is the convolution of structural and equipment fragility with seismic hazard; the first implies the probabilistic evaluation of structural performance conditioned to the ground shaking intensity, while the second usually amounts to estimating the frequency of occurrence for each intensity level. Even though seismic hazard plays a fundamental role in the estimation of the overall risk, both randomness and uncertainty significantly affect the evaluation of structural behaviour under extreme loads and thus of seismic reliability. 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.

This is the aim of the activity described in the present report, where attention is particularly devoted to the probabilistic evaluation of the seismic performance of a NPP reactor building for the computation of equipment fragility. It is worth noting, in this respect, that the computation of seismic fragility of NPP components has been performed, in the last decades, by means of a simplified but well consolidated approach, whose engineering significance and practical validity for analyzing normal cases is not questionable; the topic addressed here is how to develop a more sophisticated, and obviously more costly, procedure able to eliminate part of the uncertainties and thus of the conservatism inherent in the traditional analysis. This is intended to be applied to a limited number of components of utmost importance or when the screening performed via the traditional procedure has shown a critical situation.

Within this context, the overall framework of seismic risk assessment will be reviewed, in the next section, with particular reference to the peculiarities of the NPP case. Starting from this, the analytical and computational aspects related to the assessment of reliability for a NPP component subject to random dynamic excitation, as it typical of the seismic case, will be treated in Section 3, while in the Section 4 an example of application will be shown, regarding an initial design solution for the IRIS reactor building and in the last section some conclusive considerations will be drawn.

2. Fragilities and seismic risk: the case of NPP components

Following the PEER (Pacific Earthquake Engineering Research) approach [l], the computation of failure probability for a mechanical component under seismic loading can be cast into the following integral evaluation, based on repeated application of the total probability theorem:

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

According to the revised formulation by Der Kiureghian [1], here adopted, the random variables appearing in (1) are defined as follows.

DM is a Damage Measure, associated to the assumed limit state, with dm_f denoting 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).

IM is an Intensity Measure (peak ground acceleration, spectral acceleration,...) characterizing the severity of the earthquake motion at the reactor site. The statistics of IM is usually defined in term of annual extreme values; in this case equation (1) delivers a risk estimate in terms of annual probability of failure of the component.

Application of (1) is particularly appealing within the complex framework of NPP design, since the main tasks, which can be performed by different groups, are well defined and differentiated. More precisely:

- the determination of $p_{IM}(im)$, i.e. of the probability density function (PDF) of the intensity measure at the site, which is the object of Seismic Hazard Analysis;
- the determination of the conditional PDF $p_{EDP}(edp|IM = im)$, denoted as Fragility Analysis in [1], which is the output of the reliability analysis of the reactor building under a seismic input of intensity equal to *im*;
- the determination of the conditional PDF $p_{DM}(dm|EDP = edp)$, denoted as Damage Analysis in [1], which is the output of the reliability analysis of the component subject to a seismic demand equal to *edp*.

In addition, if the component failure must be considered within an overall "Plant Fragility" estimation the total conditional probability $P\left\{DM > dm_f \middle| IM = im\right\}$ must supplied to the

system reliability analyst; to this purpose we can split the integration (1) into two subsequent steps, as:

$$P_f = \int\left[\int P\left\{DM > dm_f \middle| EDP = edp\right\} p_{EDP}(edp|IM = im)\, d(edp)\right] p_{IM}(im)\, d(im) = \\ = \int P\left\{DM > dm_f \middle| IM = im\right\} p_{IM}(im)\, d(im) \tag{2}$$

Expression (2) implicitly defines the required component fragility (including Damage Analysis) as:

$$P\left\{DM > dm_f \middle| IM = im\right\} = \int P\left\{DM > dm_f \middle| EDP = edp\right\} p_{EDP}(edp|IM = im)\, d(edp) \tag{3}$$

The above framework applies to the analysis of complex and/or critical components; for a "simple" component 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:

$$P_f = \int P\left\{EDP > edp_f \middle| IM = im\right\} p_{IM}(im)\, d(im) \tag{4}$$

3. Computation of fragility via the Response Surface Method

We shall denote in the following the fragility *F(edp,im)* as:

$$F(edp, im) = P\left\{EDP > edp \middle| IM = im\right\} = 1 - P_{EDP}(edp / IM = im) \tag{5}$$

where $P_{EDP}(edp / IM = im)$ is the conditional Cumulative Density Function of the *edp* random variable.

The use of the expressions derived in section 2 implies the availability of either the fragility function *F(edp,im)*, in equation (4), or its derivative, appearing in (1) and (3). In fact we get, from (5):

$$p_{EDP}(edp|IM = im) = -\frac{dF(edp, im)}{d(edp)} \tag{6}$$

With reference to the case of equipment components located inside the reactor building of a NPP, we shall consider as *EDP* the extreme value, denoted as *A*, of the absolute acceleration at the component supports; the peak acceleration A_g of the most severe component of horizontal ground motion will be taken as *IM*.

In general, we shall assume that the damage analysis can be performed by studying the response of the equipment component (or subsystem such as the reactor vessel) to the

dynamic excitation defined by suitable time-histories of support acceleration components, having an overall peak value equal to A. The support motion can be in turn computed by studying the building response via a structural model in which the component is represented in a simple way (rigid mass or simplified mechanical model).

This implies that no significant structure-equipment dynamic interaction occurs, so that structure and equipment analyses can be performed in decoupled form. Note that the performance of decoupled dynamic analysis and the use of expressions (1) or (3) is particularly attractive when, as it usually the case in NPP design, elastic behaviour can be assumed for the reactor building, so that non-linearity can be confined within the component damage analysis.

The fragility function can therefore be interpreted as the probability of exceeding a given structural dynamic amplification, that is:

$$F(edp, im) = P\left\{A > a \middle| A_g = a_g\right\} = P_{exc}(a, a_g) = P_{exc}(a / a_g) \tag{7}$$

where the last rhs holds for a linearized structural model.

3.1 Deterministic dynamic excitation

We shall first address a linear model under deterministic seismic input; in this case the exceedance probability (7) is associated to a limit-state function which can be written in the following "capacity minus demand" format:

$$g(\mathbf{X}, a, a_g) = C - D(\mathbf{X}, a_g) = a - r(\mathbf{X})\, a_g = 0 \tag{8}$$

in which $g(\mathbf{X}, a, a_g)$ is the performance function, $\mathbf{X}$ is the vector listing the random variables and $r(\mathbf{X})$ denotes the structural response for a unit *pga*.

To fully exploit response linearity the preceding expression can be rewritten in the non-dimensional form:

$$\tilde{g}(\mathbf{X}, a / a_g) = \frac{a}{a_g} - r(\mathbf{X}) = 0 \tag{9}$$

in which the peak amplification factor a / a_g is explicitly stated as *EDP*.

Once the limit-state function is stated the exceedance probability (7) can be expressed by the multidimensional integral

$$P_{exc}(a/a_g) = \iiint_{\tilde{g}<0} p_{\mathbf{X}}(\mathbf{x})d\mathbf{x} \tag{10}$$

which can be evaluated, in principle, via Monte Carlo Simulation (MCS); in fact the response $r(\mathbf{X})$ is algorithmically known, i.e. can be deterministically computed by structural dynamic analysis, for every realization of the random variables $\mathbf{X}$. It must be considered, however, that a huge computing time and cost would be required for running a complex finite element model, as it is the case for a NPP reactor building, for the number of evaluations which are necessary for MCS, especially for the estimation of small probabilities.

For the above consideration, according to the well-established Response Surface Methodology (RSM, see [2,3]), the "true" response function is replaced by a simple analytical representation by assuming:

$$r(X) \cong y(X) = \sum_{i=1}^{p} a_i z_i(X) + \varepsilon = \mu_y(X) + \varepsilon \tag{11}$$

where the a_i's are coefficients to be estimated, the z_i's are the "explanatory" functions (usually polynomials), μ_y is the mean value of the response and ε is a zero mean random deviation or "error" term. The latter accounts for the variability of y around its actual mean and for the lack of fit of the adopted model, i.e. for the inadequate analytical form of the RS and for missing variables (i.e. not comprised in (11) though influencing the response). To estimate the coefficients and the properties of ε a suitable number ($n \geq p$) of numerical experiments must be run, according to the adopted strategy ("experimental design").

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 term is normal with constant variance; under these hypotheses an unbiased estimate of the coefficients a_i can be obtained by the Ordinary Least Square (OLS) method, independently of the variance of ε. An unbiased estimate s_ε^2 of the latter can be obtained, once the mean model is defined, in terms of the so called residuals r_j: these are defined, for each (j-th) experiment, as the difference between the observed value and the value predicted by the mean model. The following expression can be derived:

$$s_\varepsilon^2 = \frac{1}{n-p}\sum_{j=1}^{n} r_j^2 \tag{12}$$

Once the "metamodel" (11) is set the failure probability can be easily derived from MCS, given the very low computational cost associated to each evaluation of the limit state function.

3.2 Random dynamic excitation

In the preceding section the excitation as been assumed deterministic; this means that, once the intensity and frequency content are fixed, even though controlled through some random variable listed in $\boldsymbol{X}$, a unique realization of the external dynamic forces is possible. This does not hold true in the seismic case since, even though overall properties (e.g. spectral parameters) are given, an extremely large number of variables affect the seismic source, the source-to-site transmission path and the local ground response, so that the time history of ground motion can be described only in a probabilistic sense.

In such situation the computation of structural response becomes, once fixed the seismic input spectral parameters and the structural properties, the result of a random vibration analysis; we shall denote, in this light, $R(\mathbf{X})$ as a new random variable whose realization $r(\boldsymbol{X})$ is the extreme value of support acceleration conditioned to the values of the random variables in $\boldsymbol{X}$. The limit state functions (8 or 9) take the form

$$g(\mathbf{X}, R, a, a_g) = a - R(\mathbf{X})\, a_g = 0 \tag{12}$$

$$\tilde{g}(\mathbf{X}, R, a, a_g) = \frac{a}{a_g} - R(\mathbf{X}) = 0 \tag{13}$$

in which the dynamic response R is the only random variable explicitly appearing, even though strongly correlated with the other variables listed in $\boldsymbol{X}$. This is because the result of a random vibration problem is obviously dependent of the mechanical properties of the system and the spectral properties of the excitation. Assuming that the distribution of R can be described by its mean value μ_R and its standard deviation σ_R, both parameters are function of the component of $\boldsymbol{X}$, i.e.:

$$\mu_R = \mu_R(\mathbf{X}) \quad ; \quad \sigma_R = \sigma_R(\mathbf{X}) \tag{14}$$

The preceding considerations support the adoption of the so called "dual response surface" approach for solving the reliability problem under stochastic input; in fact, the functions appearing in (14) are known, such as $r(\boldsymbol{X})$ in (9), in algorithmic sense. Thus the method

summarized in the previous section can be applied to both of them; if the same model is used for the mean and the standard deviation the following expressions hold:

$$\mu_R(\mathbf{X}) = \sum_{i=1}^{m} a_i z_i(\mathbf{X}) + \varepsilon_\mu$$
$$\sigma_R(\mathbf{X}) = \sum_{i=1}^{m} b_i z_i(\mathbf{X}) + \varepsilon_\sigma \qquad (15a,b)$$

Again, to compute the coefficients in (15a,b) a number of experiments must be run; 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 $\boldsymbol{X}$. For each realization the extreme value of R is computed (e.g. via FE modelling and step-by-step analysis); the mean and variance of R are then estimated. The procedure is repeated for all experimental points, leading to n observed values for the parameters in (14); applying the OLS method the coefficients in (15a,b) can be computed. The variances of ε_μ and ε_σ can be finally estimated via residual analysis and formula (12).

Once models (15a,b) are established, MCS can be carried on. Note that to compute the fragility curve (7) the integral (10) must be evaluated for a number of amplification values a/ag, even though the Response Surface estimates (11) or (15) remain the same.

3.3 Design of Experiments

Second-order models are the most widely used in the application of the RSM to structural problems. A full model (i.e. encompassing all quadratic terms) requires, for m random variables, the estimation of $p = 1 + m + m(m+1)/2$ coefficients. In this situation the most suitable experimental strategy is the "Central Composite Design" (CCD); once fixed a "center point", CCD is the combination of a classical "two-level factorial design", in which all the combinations of two levels (high/low) of the rv's are considered, with a "star design". In the latter $2m$ points are considered in which one variable takes an intermediate value $\pm\alpha$ and the others are at the central value. Including the central point, a total number of experiments equal to $n = 2^m + 2m + 1$ is reached; if $m=3$, for example, we have $p=10$ and $n=15$.

Reasoning in terms of non-dimensional zero-mean random variables $\eta_i = \left(x_i - \mu_{x_i}\right)/\sigma_{x_i}$, high and low levels are usually fixed in the range $\eta_i = \pm(1-3)$, while for preserving the "rotatability" of the design (see [DS]) the star points must be placed at $\alpha = \pm\sqrt[4]{2^m}$.

3.4 Refinement of the Response Surface

Significant research effort has been devoted, in the last two decades, to the problem of refining, or updating, the RS (see, for example [5]); in reliability problems the refinement aims to an improved fitting in the region of the failure domain where $f(\mathbf{X})$ is still relatively large, giving the largest contribution to the failure probability. An obvious choice, in this light, is to assume as design center the design or minimum norm point; this is defined by transforming the rv's into the space of standard normal variables Y_i. Subsequently the limit-state function is expressed in the same space and its point which is closest to the origin is found; this can be also interpreted as the "most likely failure point". Obviously, the design point is not known when the analysis is started, so that an iterative refinement procedure is required in principle.

When the application of the above criterion is here sought, a problem arises since the probability of exceedance (10) must be computed for a number of values of the amplification ratio; this implies that, at each point, a different limit-state function (13) is introduced, having a different design point. This would lead, under the above considerations, to a different updating procedure for each amplification value; to avoid this task a different reasoning has been applied by considering that the aim of the reliability evaluation is here the computation of the integrals (3) or (4). If we consider the latter, along with expressions (5) and (7), we can write the probability of failure, for a given reference "capacity" a_f as:

$$\begin{aligned} P_f &= \int F(edp, im) p_{IM}(im)\, d(im) = \\ &= \int P_{exc}(a_f / a_g) p_{A_g}(a_g)\, d(a_g) \end{aligned} \tag{18}$$

It can be noticed that, once a_f is fixed and a first evaluation of the P_{exc} function is available, the integrand function in (18) can be analyzed and the *PGA* range giving the maximum contribution to the total probability can be detected; for the a_f value under consideration this delivers a range of amplification values, and thus of design points in (13), which can be considered for refining the Response Surfaces (15a,b).

For a practical implementation of the above sketched procedure, the FORM technique, which has no significant cost once the design point is found, is an obvious choice for the evaluation of $P_{exc}(a_f / a_g)$; the Rosenblatt Transformation has been here applied to deal with the correlation between the random response R and the other rv's, while the error term has been disregarded. Accordingly the functions expressing the transformation into the standard space are the following:

$$
\begin{aligned}
&y_1 = \Phi^{-1} P_1(x_1) \\
&y_2 = \Phi^{-1} P_{2|1}(x_2 | x_1) \\
&\dots \\
&y_{m+1} = \Phi^{-1} P_{R|1\dots m}(r | x_1, \dots x_m)
\end{aligned}
\tag{19}
$$

where Φ is the standard Gaussian CDF, $P_1(x_1)$ is the marginal CDF of the first rv and $P_{i|1\dots i-1}(x_i | x_1, \dots x_{i-1})$ is the CDF of X_i conditioned to the values of the variables $X_1, \dots X_{i-1}$. The last of (19), expressing the CDF of dynamic response conditioned to the structural and spectral variables is the result of the random vibration analysis. Here a type I extreme-value distribution has been assumed, so that the CDF of R takes the form:

$$
P_{R|1,\dots,m}(r | x_1, \dots, x_m) = \exp\left[-\exp\left(-\alpha(x_1, \dots, x_m)\left(r - u(x_1, \dots, x_m)\right)\right)\right] \tag{20}
$$

being α and u the parameters depending on the estimated mean and variance of R.

The procedure for fragility evaluation can be thus subdivided into the following steps.

1. Performance of a first set of experiments by centering the CCD at the average values of the rv's and by assuming high/low levels at $\eta_i = \pm 3$.
2. Estimation of the coefficients of the RS's expressing the mean and variance of the response.
3. Performance of FORM analysis for evaluating, for each amplification value in the range of interest, the design point position and estimating P_{exc}.
4. Computation of the integrand function $P_{exc}(a / a_g) p_{A_g}(a_g)$ and selection of the PGA range in which refinement must be pursued.
5. Performance of a new sets of experiments centered on the median of the integrand function, with high/low levels smaller than in (1) but covering the selected range.
6. Back to point 2 for iterating the procedure.

7. Once the refinement procedure is ended the final value of the fragility $F(edp, im) = P_{exc}(a / a_g)$ is computer via MCS. Importance sampling is applied, centred on the final design point; error terms are obviously included in the final version of the Response Surfaces (15).

have been collected; each component has been subsequently corrected, by iteratively modifying its Fourier amplitude spectrum, to match in a satisfactory way the EC8 spectra.

4 Example of application: the IRIS reactor building

The above described procedure has been applied to the analysis of a preliminary early design of the auxiliary 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 21 partners from 10 countries, led by Westinghouse Electric Company (see [4]). Installation in a site characterized by a low-to-average seismicity level has been here assumed.

4.1 Structural and seismic input modelling

Details on the criteria adopted for setting dynamic models for the seismic analysis of the building can be found in [6]; for performing repeated analysis, as required for fragility estimation, a "simplified" FE model has been set, encompassing about 5x105 degrees of freedom. The model is based on simplified approaches for representing soil-structure interaction effects and sloshing effect in RWST pools; shell finite element are introduced for modelling all walls and slabs, including the foundation mat (see Figure 1a).

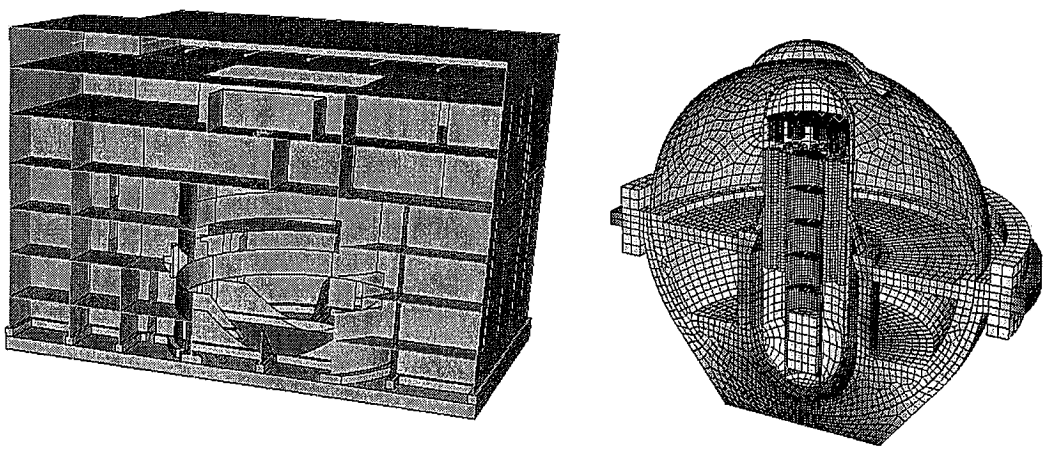
Based on the results obtained by a more refined model (see Figure 1b) the lowest part of the containment structure is considered as a rigid body, while the upper part (steel liner) is replaced by an equivalent two-degree-of-freedom system.

A simplified model is introduced as well for the vessel, based on the observation that, in the refined model, most of the deformation is concentrated in the supporting plate and in the surrounding portion of the shell structure. Accordingly, the model is composed of a FE shell discretization (central part) and of two rigid bars; the upper end of the top bar, located at level of the reactor coolant pumps, will be here taken as reference point for computing the extreme value of acceleration A, assumed as EDP.

The natural frequencies of the first vibrations modes, encompassing significant soil deformation, range between 2 and 5.2 Hz. The vessel modes have frequencies in between 13 and 30 Hz (see also Table 1).

Table 1: Natural frequencies of the lowest vibration modes of the simplified model

Mode	Natural frequency [Hz]	Mode description
5	2.084	global z translation
6	2.161	global rocking about x axis
7	2.241	global rocking about y axis
8	3.368	global rotation about z
10	4.679	foundation rocking about x axis + y translation – "cantilever" building deformation
11	5.155	foundation rocking about y axis + x translation – "cantilever" building deformation.
18	12.128	2° "cantilever" building mode in y direction
48	18.669	2° "cantilever" building mode in x direction
19	13.177	vessel rotation about y axis
22	13.188	vessel rotation about x axis
42	18.669	vessel x translation
43	18.813	vessel y translation
94	30.237	vessel z translation



a) b)

Figure 1: Reactor building (a), containment and vessel (b)

The response spectrum prescribed by Eurocode 8 (EC8) for a type I earthquake and for local soil conditions type C was adopted as seismic input. The spectral parameters where treated as deterministic, so that a single set of ten input motions, each described by three components, has been generated and used at all experimental points. Generation was performed starting from real accelerograms and iteratively correcting their Fourier Amplitude Spectra in order to match the EC8 curve.

4.2 Random variables, RS model and experimental design

For a preliminary test of the procedure only three random variables have been here selected to represent the main sources of randomness for the computation of the response of an equipment located inside the vessel:

- a random variable (lognormal distribution) describing the soil shear modulus G_s, with mean value of 200 MPa and c.o.v equal to 0.2;
- a random variable (lognormal) for the vessel damping factor, ν_v; the mean value of vv has been chosen equal to 0.03, and a coefficient of variation of 0.2 has been considered;
- a random variable (lognormal) to describe the viscous soil damping; more in detail, the ratio between the actual value and the nominal value of each damping factor associated to foundation modes is considered, named δ, with a mean value of 1 and a c.o.v. of 0.2.

It must be noted, with respect to the last two RVs, that damping has been here treated in a simplified way. This was due to the difficulty to deal with composite damping, by means of the software package at hand, within modal superposition analysis. In the case here shown, modal damping factor were directly stated and given in input by recognizing, with some engineering judgement, modes dominated by foundation or by vessel movements. As a result, nominal damping factors imposed to the foundation motion components were equal to 20, 7 and 10% for vertical, mixed translation-rotation and torsional modes respectively. Damping of other modes was fixed at 5%.

The model chosen for the mean and variance (15) of the dynamic response is a complete second order polynomial; a cubic mixed term (proportional to $x1\ x2\ x3$) has been also added, leading to a total number coefficients to be estimated equal to eleven for each RS.

Table 2: Initial experiments

Experiment	G_s	ν_v	δ	μ_R	σ_R
1	0.4	0.4	0.4	4.170900	0.564442
2	0.4	0.4	1.6	2.946300	0.374085
3	0.4	1.6	0.4	4.200900	0.559332
4	0.4	1.6	1.6	3.044000	0.385625
5	1.6	0.4	0.4	3.236000	0.480297
6	1.6	0.4	1.6	2.261700	0.480300
7	1.6	1.6	0.4	3.249900	0.483655
8	1.6	1.6	1.6	2.439500	0.320718
9	1.3364	1	1	2.597982	0.419865
10	0.6636	1	1	2.929579	0.533907
11	1	1.3364	1	3.194488	0.552034
12	1	0.6636	1	3.200842	0.537174
13	1	1	1.3364	2.988532	0.533657
14	1	1	0.6636	3.482527	0.548909
15	1	1	1	3.274700	0.495196

Table 3: First iteration experiments

Experiment	G_s	ν_v	δ	μ_R	σ_R
1	1.3364	1	1	2.597982	0.419865
2	0.6636	1	1	2.929579	0.533907
3	1	1	1.3364	2.988532	0.533657
4	1	1	0.6636	3.482527	0.548909
5	1.2687	1	1	2.611578	0.433613
6	0.7589	1	1	2.922170	0.548878
7	1	1	1.2687	2.647432	0.491708
8	1	1	0.7589	3.026624	0.526790
9	1	1	1	3.274700	0.495196

In the initial phase, considering $k=3$ random variables, the CCD is composed of 15 experiments: the 8 points of the $2k$ factorial design, located at $\eta_i=\pm 3$, the central point and 6 star points, with α chosen equal to 1.6868 for rotatability (see section 2.3).

4.3 Initial results

In Table 2 the first cycle of experiments is summarized; non dimensional values (wrt the mean) of the random values are given, along with results in terms of mean and variance of dynamic response amplification. Based on this results a first evaluation of the response functions was performed and an initial fragility function was obtained by FORM analysis; the result is shown in Figure 2 (dotted line).

To the purpose of refining the Response Surfaces, a risk estimation was performed for the site described, in terms of seismic hazard, by the pga-return period curve shown in Figure 3; from this the PDF of the annual pga extreme was derived. A reference value a_f=25 m/s2 was chosen as a possible collapse value for the support acceleration of a safety component inside the vessel.

In Figure 4 the integrand function in the risk estimation (18) is depicted, allowing for a visual appreciation of the pga range (6 to 13 m/s^2) mostly contributing to the failure probability.

From this and given the a_f value the most significant range of amplification factors have been determined; for each amplification the rv values at the design point found in the FORM determination of $P_{exc}(a_f/a_g)$ have been detected. According to this investigation and given the very low sensitivity of the probability of failure to the vessel damping the following criteria were adopted for refining of the Response Surfaces

- the random variable describing the vessel damping has been eliminated from the analysis;
- the initial experimental points of the factorial design were replaced by the star points values corresponding to k = 2 (points 5 to 8 in Table 3), thus focusing the parametric analysis on a smaller range of the rv's, still centered on average values.

From the results of Table 3 updated Response Surfaces were obtained and the new fragility curve (Figure 2) was computed. Table 4 reports the variance of the error for the predicted responses at the initial experimental design and at the first iteration, pointing out an improvement in the fitting.

Conclusions

A numerical procedure has been proposed for evaluating the seismic fragility of NPP components; though innovative, the procedure is based on consolidated numerical techniques such as Finite Element modelling, step-by-step dynamic integration, Response Surface Methodology, Monte Carlo Simulation. A risk-based criterion for updating the Response Function has also been proposed. The example of application here shown, though considering a limited number of variables, demonstrates the applicability of the procedure to a real-life case.

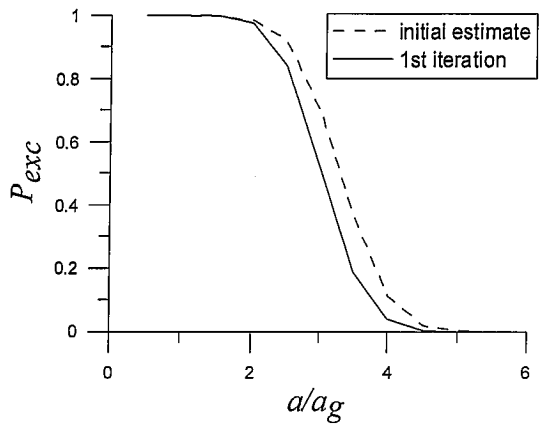


Figure 2: Fragility functions

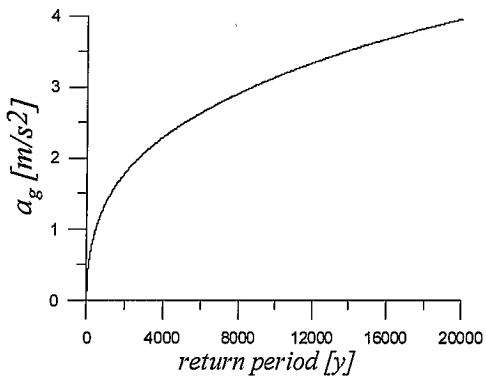


Figure 3: *pga* vs return period

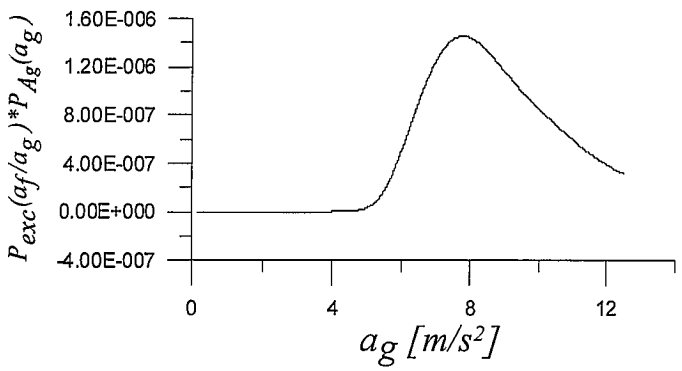


Figure 4: Fragility times hazard vs *pga*

Table 4: Error variance

	μ_R	σ_R
Initial	0.0883	0.0044
1st iteration	0.0706	0.00027

References

[1] Der Kiureghian, A., Non-ergodicity and PEER's framework formula, *Earthquake Engineering and Strucural Dynamics*, Vol. 34, p. 1643-1652, 2005.

[2] Faravelli, L.,Response surface approach for reliability analysis, *J. of Engineering Mechanics*. Vol. 115, p. 2763-2781, 1989.

[3] Casciati, F., Cimellaro, G.P., Domaneschi, M., "Seismic reliability of a cable-stayed bridge retrofitted with hysteretic devices", *Computers & Structures*, In Press, Available online 21 March 2008. DOI: 10.1016/j.compstruc.2008.01.012.

[4] Carelli, M.D. *et al*, The design and safety features of the IRIS reactor, *Nuclear Engineering and Design*, Vol. 230, p. 151-167, 2004.

[5] Rajashekhar M.R., Ellingwood, B.R., A new look at the response surface approach for reliability analysis, *Structural Safety*, Vol. 12, p. 205-220, 1993.

[6] Bianchi, G., Corradi, L, De Grandis, S., Mantegazza, D.C., Perotti, F., "Metodo semplificato per il calcolo delle probabilità di rottura al sisma (fragility curves) ed applicazione all'edificio del reattore IRIS", *relazione **CIRTEN** - Polimi RL 1121/2008.*