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SEISMIC FRAGILITY ANALYSIS OF STRUCTURAL SYSTEMS

Giorgio LUPOI[1], Paolo FRANCHIN[2], Alessio LUPOI[3], Paolo E. PINTO[4]

SUMMARY

A method is presented for the evaluation of the seismic fragility function of realistic structural systems. The method is based on a preliminary, limited, simulation involving non linear dynamic analyses performed to establish the probabilistic characterization of the demands on the structure, followed by the solution of a system reliability problem with correlated demands and capacities. The results compare favorably well with the fragility obtained by plain Monte Carlo simulation, while the associated computational effort is orders of magnitude lower. The method is demonstrated with an application to a RC bridge structure subjected to both rigid and spatially varying excitation.

INTRODUCTION

Safety assessment of structures explicitly based on probabilistic approaches is gaining wider diffusion both for the calibration of deterministic design procedures (Cornell [1]), as well as for direct use in design. The required fragility functions can be obtained through a variety of methods, that range from expert judgment (ATC13[2]), to data analysis on observed damages (Singhal [3], Shinozuka [4]), to fully analytical approaches, as for ex. in Gardoni [5], Franchin [6], Au [7].

A feature common to most of the approaches of the latter category is the use of a reduced number of simulations to compare probabilistically the maximum structural responses with the corresponding capacities.

The difference among them lie essentially in their balance between cost and accuracy, i.e. in their ability to account economically for all the aspects entering the reliability problem. These latter include:

a. The possibility of the structure reaching collapse in more than one failure mode (system reliability problem)
b. The degree of dependence among the possible failure modes
c. The uncertainty in the capacity of the structure (due to the approximate nature of the models)
d. The influence on the dynamic response of the variability of the system parameters
e. The influence of the variability of the ground motion on the dynamic responses and on their correlation

In the paper a method is presented which is simple and, at the same time, able to account for all of the above mentioned aspects. The method takes profit of ideas and proposals that have appeared in different

[1] PhD candidate, European School for Advanced Studies on Seismic Risk, Pavia, Italy: glupoi@roseschool.it
[2] Research fellow, Dept. of Struct. & Geotech. Eng., University of Rome La Sapienza, Rome, Italy: paolo.franchin@uniroma1.it
[3] Research fellow, Dept. of Struct. & Geotech. Eng., University of Rome La Sapienza, Rome, Italy: alessio.lupoi@uniroma1.it
[4] Professor, Dept. of Struct. & Geotech. Eng., University of Rome La Sapienza, Rome, Italy: pinto@uniroma1.it

forms in the recent literature, though not formulated in a similarly coherent framework. It is presented here together with an application of realistic character that offers the possibility of exploring its features, among which effectiveness and practicality are believed to be the most attractive ones.

RELIABILITY METHOD

In the basic formulation of this method, as presented in Giannini [8], the variability of the response/demand is assumed to be due only to that of the ground motion, i.e. the structural response, given the input, is deterministic.

In case the performance of the structure can be characterised by a single failure mode, or when one mode is dominant over the others, denoting by D_k the maximum demand in this failure mode due to the k-th accelerogram and by C the corresponding capacity, completely defined by its cumulative distribution function $F_C(\cdot)$, the probability of failure conditional on the sample ground motion k is given by:

$$P_{f,k} = \Pr\{C \le D_k\} = F_C(D_k) \tag{1}$$

By repeating the analysis with a number of accelerograms, the probability of failure unconditional with respect to sample variability can be simply obtained as:

$$P_f = \frac{1}{n}\sum_{k=1}^{n} P_{f,k} \tag{2}$$

where the number of samples must be large enough to ensure stable estimates of P_f.

In general, failure may occur according to different modes of comparable importance (e.g. flexural failure, shear failure, joint failure, etc.) in different members of the structure. If the failure events can be considered as independent and arranged in series, the probability of failure of the system is easily evaluated as:

$$P_{f,k} = \Pr\left\{\bigcup_{i=1}^{m} C_i \le D_{ik}\right\} = 1 - \prod_{i=1}^{m}\left(1 - P_{f,ik}\right) = 1 - \prod_{i=1}^{m}\left[1 - F_{C_i}(D_{ik})\right] \tag{3}$$

which is the generalisation of Eq.(1) for the case of m independent modes.

In Eq.(2) the dependence of P_f on the intensity of the seismic action is omitted: the fragility function is obtained by calculating P_f for a convenient number of intensity values. The simplest version of the procedure is comprised in Eq.(2) and (3). Consideration of the correlation between failure modes, of the influence of the variability of the mechanical parameters on the demand and of possible non serial arrangements of the failure events are all areas where the basic procedure can be improved, as indicated in the following.

In a general formulation of the problem, both the demands and the capacities should be considered as functions of the basic variables $\mathbf{x}$, i.e. $C_i = C_i(\mathbf{x})$ and $D_i = D_i(\mathbf{x})$. In the basic procedure this dependence is ignored for what concerns the demand and only partially accounted for at the capacities level through their marginal distributions $F_{C_i}(\cdot)$. Actually, a significant degree of correlation normally exists among the capacities C_i's. This correlation can be evaluated based on that existing among the basic random variables $\mathbf{x}$ they have in common.

To illustrate this last point, assume as a simplification that the ductility capacity C_i at a section i can be expressed as a linear function of the ultimate deformation of concrete $\varepsilon_{cu,i}$ and of the yield strength of steel $f_{y,i}$ at the same section:

$$C_i = a_i \varepsilon_{cu,i} + b_i f_{y,i} \tag{4}$$

and that a correlation exists between the ultimate deformation and the yield strength at two different sections i and j. One has then:

$$\rho_{C_i C_j} = \frac{Cov(C_i, C_j)}{\sigma_{C_i} \sigma_{C_j}} \tag{5}$$

where, assuming independence between concrete and steel properties, the covariance between the ductility capacities at sections i and j can be written as a function of the known covariances between the basic variables as:

$$Cov(C_i, C_j) = a_i a_j Cov(\varepsilon_{cu,i}, \varepsilon_{cu,j}) + b_i b_j Cov(f_{y,i}, f_{y,j}) \tag{6}$$

In practice, most formulas for the capacity of failure modes of reinforced concrete members are built upon a relatively weak mechanical basis, to which elements of empirical origin are added. This formulas are presumed to be unbiased (i.e. to provide a correct prediction of the mean value), but they are accompanied by a significant scatter due to modelling error. Since the capacity is generally a positive quantity, the general format of these formula is additive when expressed in terms of some transformation of the capacity:

$$C_i = \mu_{C_i}(\mathbf{x}) + \varepsilon_{C_i} \tag{7}$$

or multiplicative as in:

$$C_i = \mu_{C_i}(\mathbf{x}) \cdot \varepsilon_{C_i} \tag{8}$$

In the former case ε_{C_i} is normally assumed to be a zero mean Gaussian random variable, while in the latter it can be assumed to be a unit mean Lognormal variable. It has to be observed that for distinct failure modes the corresponding random variables ε_{C_i} and ε_{C_j} are usually considered as independent and this reduces the correlation between the capacities C_i and C_j.

Coming now to the demands, rather than calculating the failure probability conditional on the k-th sample of ground motion, as in the basic procedure, the results D_{ik} from the entire set of non linear structural analyses can be used to compute the statistics of the D_i's, which include mean values μ_{D_i}, standard deviations σ_{D_i} and correlation coefficients $\rho_{D_i D_j}$. The i-th demand can then be expressed, similarly to the corresponding capacity, as:

$$D_i = \mu_{D_i}(\boldsymbol{\mu}_{\mathbf{x}}) \cdot \varepsilon_{D_i} \tag{9}$$

where the mean value μ_{D_i} of the demand is evaluated at the mean value of the basic variables $\boldsymbol{\mu}_{\mathbf{x}}$, ε_{D_i} can be assumed to be Lognormal[5] with unit mean, standard deviation equal to the *i*-th demand coefficient of variation $\delta_{D_i} = \sigma_{D_i} / \mu_{D_i}$ and correlation coefficient with ε_j equal to $\rho_{ij} = \rho_{D_i D_j}$.

Apart from the dependence of the demands on the basic variables $\mathbf{x}$, all the elements are in place to evaluate the probability of failure of a completely general system (not necessarily serial):

$$P_f = \Pr\left\{ \bigcup_{j=1}^{n_C} \bigcap_{i \in I_{\mathbf{C}j}} C_i(\mathbf{x}, \boldsymbol{\varepsilon}_C) \le D_i(\boldsymbol{\varepsilon}_D) \right\} \tag{10}$$

with $n_{\mathbf{C}}$ cut-sets $\mathbf{C}_j$ ($I_{\mathbf{C}_j}$ denoting the set of indexes of the failure modes belonging to the j-th cut set).

The system reliability problem in Eq.(10) can be evaluated either by FORM, first solving each component/failure mode and then using the multi-normal approximation for general systems, or by Monte Carlo simulation, which is simpler and in this case is comparatively inexpensive since it does not require any structural analysis.

As a final step, it remains to account for the dependence of the demands on $\mathbf{x}$. One simple, efficient, if apparently not particularly accurate way of doing this, is to consider this dependence as linear around the mean value of $\mathbf{x}$. This involves the first order partial derivatives of the demands with respect to $\mathbf{x}$ evaluated in the mean $\boldsymbol{\mu}_{\mathbf{x}}$ of the basic variables. These, often called *sensitivities* with respect to the system parameters, can be computed either numerically by a finite difference scheme, i.e. repeating the analysis for perturbed values of the parameters, or, more efficiently, by the Direct Differentiation Method (Franchin [9], Kleiber [10]). In practice, the sensitivities $\partial D_i / \partial x_j$ are computed as the mean values of the derivatives conditional on sample accelerogram:

$$\frac{\partial D_i}{\partial x_j} = \frac{\partial}{\partial x_j}\left(\frac{1}{n} \sum_{k=1}^{n} D_{ik} \right) = \frac{1}{n} \sum_{k=1}^{n} \frac{\partial D_{ik}}{\partial x_j} \tag{11}$$

where the derivatives $\partial D_{ik} / \partial x_j$ are calculated at the time instants where the corresponding maxima occur.

The demand in failure mode i can thus be rewritten accounting for its (linearized) dependence on $\mathbf{x}$ as:

$$D_i(\mathbf{x}) = \mu_{D_i}(\boldsymbol{\mu}_{\mathbf{x}}) \varepsilon_{D_i} + \sum_j \left. \frac{\partial D_i}{\partial x_j} \right|_{\boldsymbol{\mu}_{\mathbf{x}}} \left(x_j - \mu_{x_j} \right) \tag{12}$$

and the reliability problem can be written, similarly to Eq.(10), as:

$$P_f = \Pr\left\{ \bigcup_{j=1}^{n_C} \bigcap_{i \in I_{\mathbf{C}j}} C_i(\mathbf{x}) \le D_i(\mathbf{x}) \right\} \tag{13}$$

where now it is understood that the vector $\mathbf{x}$ includes, besides the basic variables, also the capacity error terms ε_C's and the demands variability terms ε_D's.

[5] Since the demand is defined in this context as the maximum value of the response over an interval of time, one might think of modelling its distribution as an Extreme one. While on one hand any choice of the distribution has to be validated with a goodness-of-fit test, on the other the use of the Lognormal one has gained wide acceptance for this type of problems.

APPLICATION TO A BRIDGE STRUCTURE

The method is demonstrated through an application to a bridge structure. The reason for choosing an extended in plan structure lies in the intent to show that the procedure works also if a dominant mode of vibration is absent. This occurs for example when a structure is subjected to a spatially distributed excitation such as that represented by a train of waves traveling with finite velocity in a random medium. The bridge is analyzed in two situations, i.e. when subjected to a "rigid" or uniform excitation and when instead, the input at each support differs due to waves scattering and refractions/reflections during the travel path.

Description of the bridge

The bridge shown in Figure 1 is composed of a pre-stressed concrete box-girder continuous over reinforced concrete cantilever piers with rectangular box section and heights of 14m, 21m and 14m, respectively. The total length of the structure is 200m, subdivided in four spans of 50m each. The piers are reinforced with $120\phi20$ bars subdivided in 67 and 53 bars on the outer and inner perimeter, respectively.

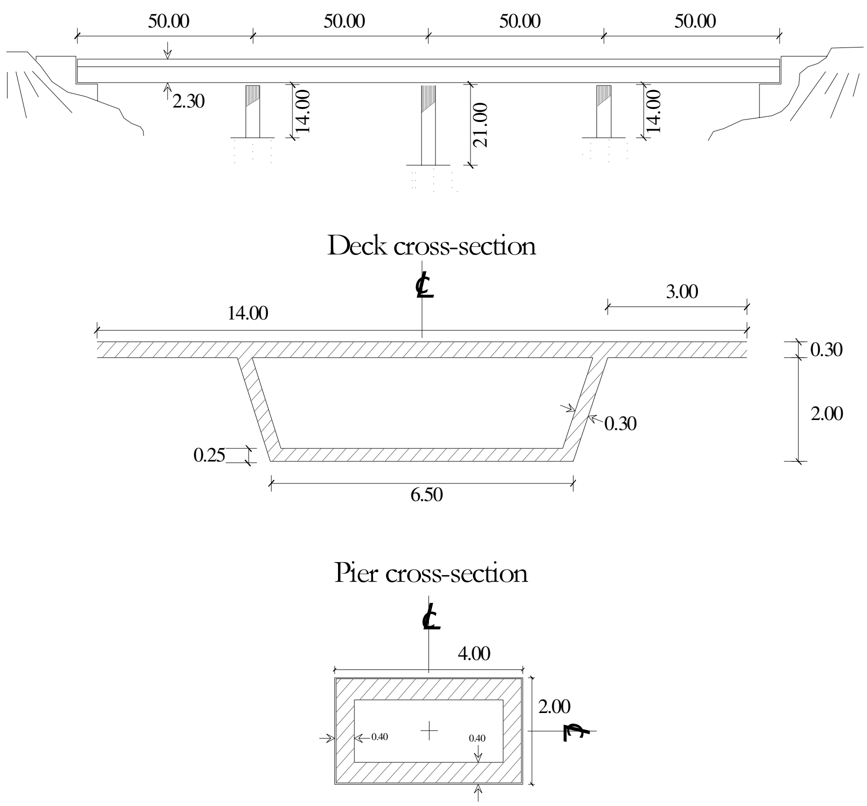

Figure 1 Bridge layout and cross-sections.

Failure modes and basic random variables

The seismic performance of the bridge is evaluated in this application with reference to six failure modes. These include the exhaustion of deformation capacity at the base of the piers measured in terms of curvature ϕ, and the exceedance of the shear strength of the piers. Given the topology of the bridge, failure is assumed to occur when any of the above modes reaches failure (series system).

The capacity formula used for the shear failure mode is the so-called "modified UCSD model", Kowalsky [11], *fib* [12]. To use this model, consisting of three additive contributions due to concrete V_c, transverse steel V_s and axial force V_p, respectively, an error term is used in the multiplicative form, i.e. the (random) shear force capacity of a pier is given by:

$$C_V = \left[V_c\left(f_c, \mu_d\right) + V_s\left(f_y\right) + V_p\right]\varepsilon_V \tag{14}$$

where μ_d is the maximum displacement ductility demand on the pier. The error term ε_V is Lognormal with unit mean and coefficient of variation equal to 0.13, which is the dispersion of the experimental results around the predictive formula according to *fib*[12].

The ultimate curvature ϕ_u, corresponding to the attainment of the ultimate concrete compressive strain ε_{cu}, is considered among the basic random variables.

The basic random variables of the problem are reported in Table 1. They are all assumed to be Lognormally distributed and statistically independent. The mean values of the ultimate curvatures correspond to a curvature ductility of about 20.

Table 1 Characterization of the basic random variables

Random variable	Mean value	Coefficient of Variation
f_c	30 [MPa]	0.35
f_y	430 [MPa]	0.20
ϕ_{u1}	0.0040 [m^{-1}]	0.30
ϕ_{u2}	0.0045 [m^{-1}]	0.30
ϕ_{u3}	0.0040 [m^{-1}]	0.30

Bridge under uniform excitation

Seismic action

For the purpose of the time-history analyses necessary for the characterization of the random demands *D*'s, both recorded and artificial accelerograms can be used. In this application, for the purpose of comparison with the results presented later for the bridge subjected to non-uniform excitation, artificial accelerograms are preferred. These latter are generated as samples of a random process having power spectral density compatible with the elastic response spectrum specified in Eurocode 8, CEN[13], for firm soil (denoted in the following by the letter F). The motions have been modulated in amplitude with an envelope having a total duration of $20\sec$ with initial parabolic and final exponential ramps of duration $2\sec$ and $3\sec$, respectively. The number of generated samples, 20, is larger than strictly necessary based on previous experience with similar structures, but again the figure is chosen for comparison purposes with the case of non uniform excitation.

Fragility analysis

In order to perform the system reliability analysis according to any of Eq.(3), (10) or (13), it is first necessary to collect response values from a limited number of non linear dynamic analyses of the bridge subjected to the generated motions. These latter are then used to establish the distribution of the demand random variables. The results of the analyses are shown in Figure 2 where, for all piers and failure modes, the maximum from each analysis together with the mean and mean plus or minus one standard deviation are given as a function of the peak ground acceleration which is here taken as the measure of seismic intensity.

Given that the choice of this measure is recognized to be consequential in reducing the variability of the demand due to that of the ground motion, it is noted here that since artificial spectrum-compatible accelerograms are used, the choice of the peak ground acceleration is equivalent to that of any other spectral ordinate.

Figure 2 shows that, as expected, the deformation demands increase almost linearly with the seismic intensity while the shear demands tend to flatten with increasing intensity due to the attainment of the flexural moment capacity at the base sections of the piers. The second slope of the shear demand curves is determined by the hardening ratio of steel (set to 3% in this example).

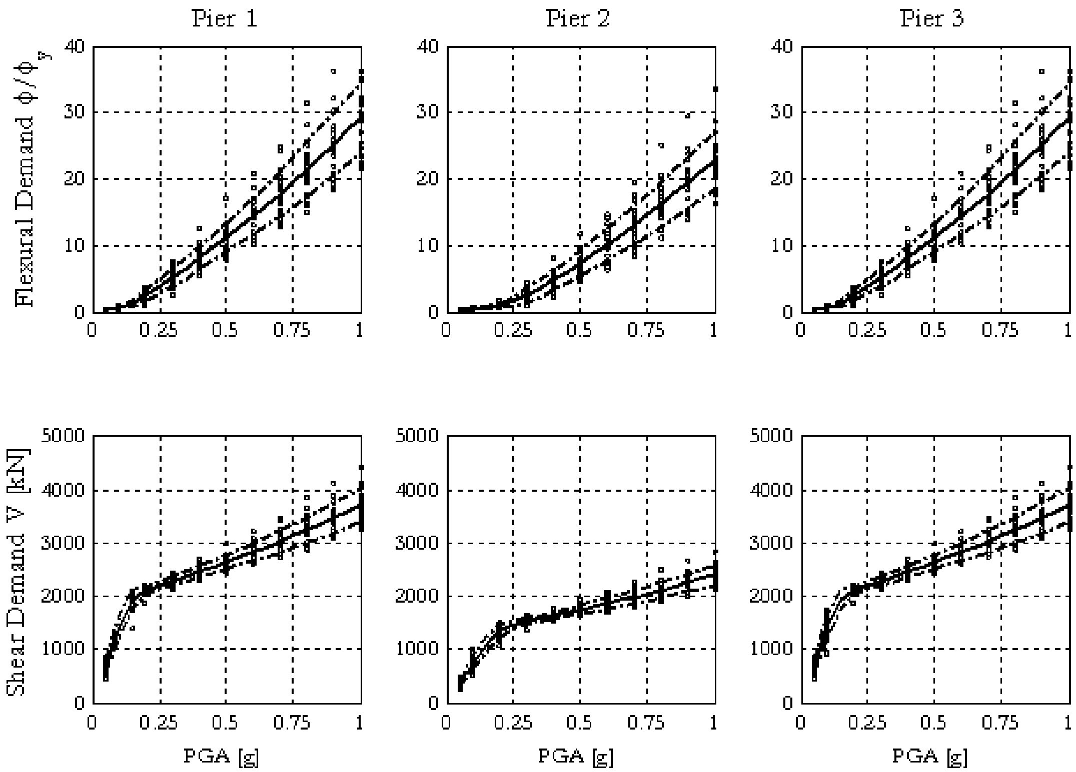

Figure 2 Results of the non linear analyses for the uniform excitation case. Mean demands and +/- one standard deviation curves for flexure and shear in the three piers. The flexural demand is expressed in terms of ductility.

Figure 3 shows the evolution of the mean and of the standard deviation of one of the demands (flexural, pier 1) with increasing number of records, i.e. non linear analyses, used. The plots are similar for the other demand variables. As anticipated 20 records are more than strictly needed for the mean and variances of the demands to stabilize, any number above 10 being enough for all practical purposes. This favorable result is due to the use of artificial accelerograms coming from the same power spectrum and cannot be generalized to the case in which recorded accelerograms are used. Experience shows, however, that also in the latter case the number of records required for stabilizing the demands is of the same order of magnitude.

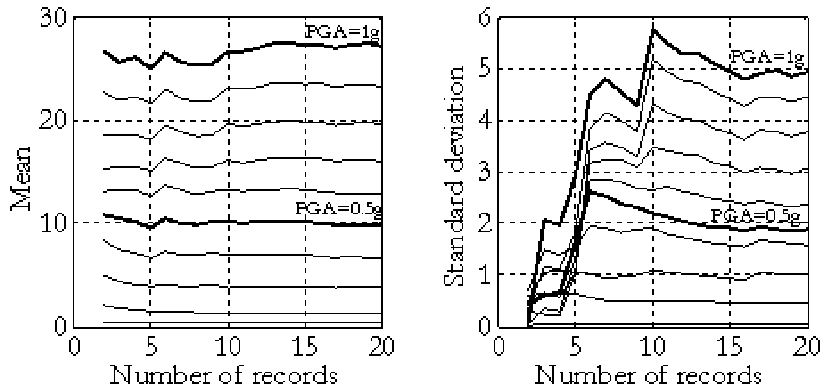

Figure 3 Mean and standard deviation of the flexural demand (in terms of curvature ductility) in pier 1 as a function of the number of records used. The curves depend on the particular ordering of the records.

The same results reported in Figure 2 can be used to numerically estimate the correlation coefficients between the demands. The total number of these is $6(6-1)/2=15$. Nine of these are shown in Figure 4 for increasing seismic intensity. Under a uniform excitation this symmetric "regular" bridge structure vibrates according to a symmetric first mode, hence the correlation coefficients between "symmetric" demands such as ϕ_1 and ϕ_3, or V_1 and V_3, are equal to one for all intensities. Small deviations from unity are observed for the correlation coefficients between the demands at the central and lateral piers around a PGA of $0.20g$, i.e. the value where some of the accelerograms induce yielding in the lateral but not in the central pier. When all piers are in the inelastic range, for higher seismic intensities, the correlation coefficients return close to one, with minor fluctuations due to the limited data used to estimate them.

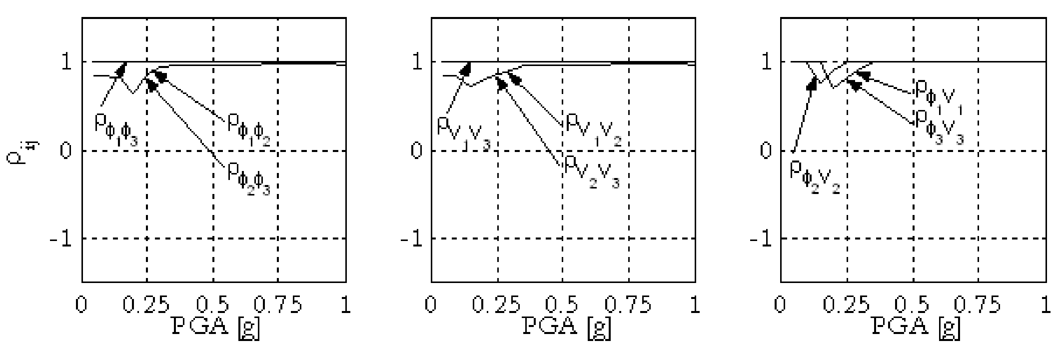

Figure 4 Selected correlation coefficients as a function of the seismic intensity: correlation between flexural demands at the three piers (Left); correlation between shear demands at the three piers (Centre); correlation between flexural and shear demands at each pier (Right)

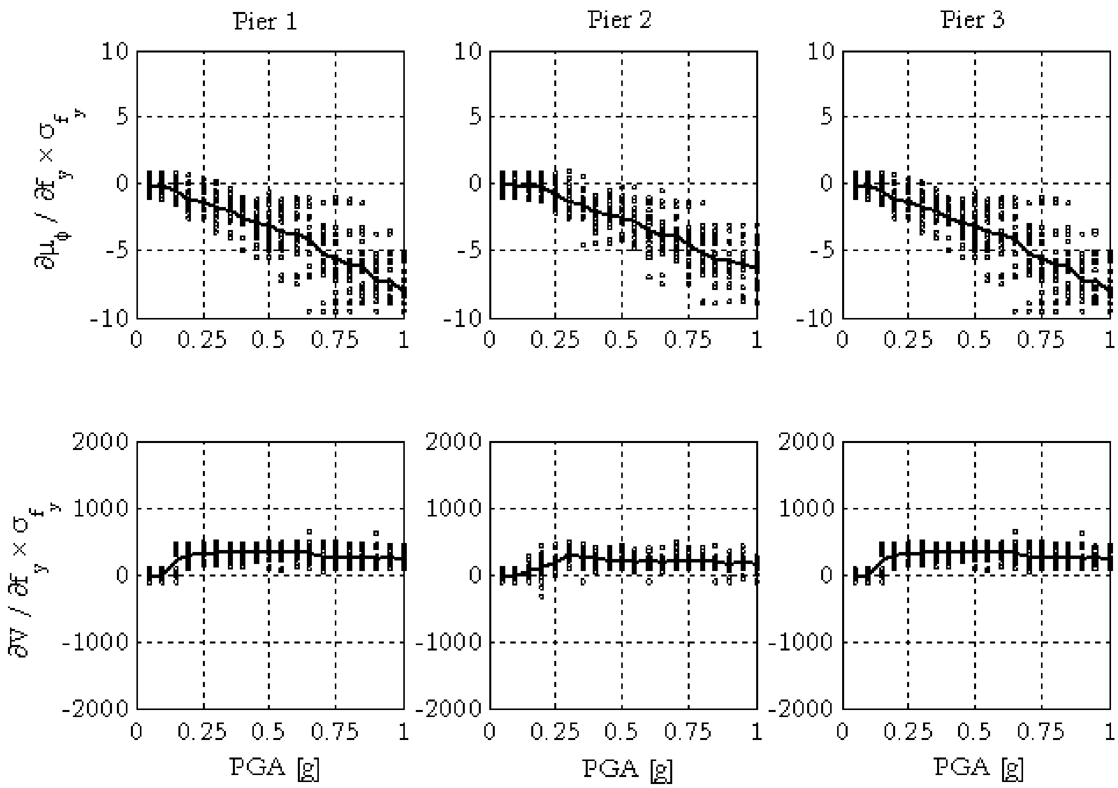

Figure 5 Sensitivities for the uniform excitation case. Mean curves for derivatives of flexural (curvature ductility) and shear demands in the three piers with respect to steel yield stress.

Figure 5 shows the sensitivities with respect to random variable f_y chosen among the five due to its higher influence on the response. As expected, the curvature ductility demands decrease with a positive variation of f_y, and the curves show that this dependence is almost linear in the intensity. For what concerns the shear demand, the results show that for a given positive variation of f_y the increase of the shear demand is independent of the PGA value. This is also expected since the variation in shear force due to an increase in the yield stress remains constant along the hardening branch. An increase of the yield strength by one standard deviation (CoV=20%) increases the shear force of about the same quantity at yield.

Before progressing further to the evaluation of the system fragility, it is of interest to check the approximation implied in the linearization in Eq.(12). This check is carried out by comparing the linearized demands from Eq.(12) with the actual demands obtained from the non linear analysis performed for the perturbed values of the basic random variables $\mathbf{x}$.

The results of this comparison are shown in Figure 6 with reference to the yield strength f_y and indicate that the linear approximation is quite good. The plots for the remaining components of $\mathbf{x}$ have similar trends.

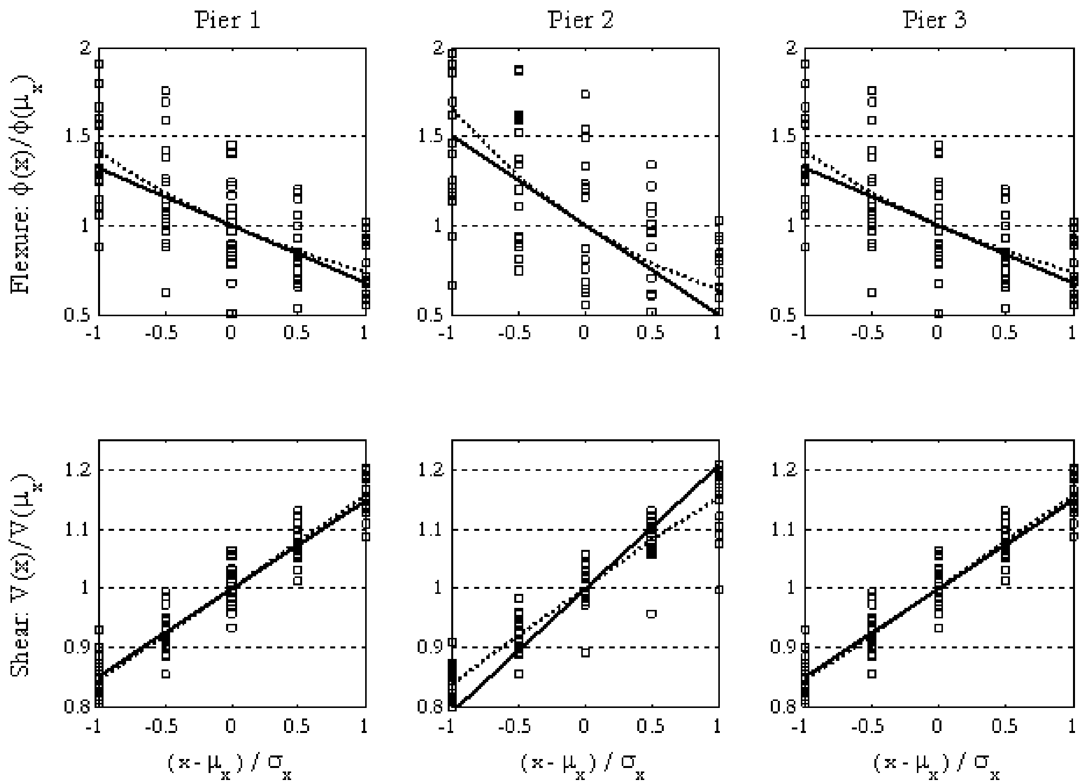

Figure 6 Linearised versus actual non linear dependence of the six demands on the random variables: the results refer to random variable f_y

The final results in terms of fragility curves are illustrated in Figure 7. The three curves in the left plot correspond to different degrees of completeness in the reliability computation. Curve (a) is obtained solving the full system reliability problem in Eq.(13) by Monte Carlo simulation. It therefore accounts for the dependence among the failure modes due to the shared basic variables $\mathbf{x}$, as well as for the dependence of the demands on $\mathbf{x}$ and on each other. This curve represents the most accurate result obtainable from this procedure and is compared in the right plot with the results of a "regular" Monte Carlo simulation. By this latter is meant the performance of non linear analyses for random samples of $\mathbf{x}$ and records from the same power spectrum used for collecting the demand values (The target coefficient of variation of the probability estimate is decreasing with increasing probability, ranging from 0.05 to 0.01, the total number of simulations for all points in the curve being above 60 000). The match between simulation results and those from the proposed procedure is remarkable.

The other curves in the left plot correspond to the following cases:

1. Curve (b), is the simplest estimate of the fragility provided by Eq.(3), i.e. considering independence of the failure modes. The distribution function of the shear capacity, necessary for the evaluation of Eq.(3), is determined starting from Eq.(14) and from the distribution of f_c, f_y and ε_V.
2. Curve (c), differs from curve (a) only in that the capacities error terms are not considered. This fragility curve allows to appreciate the importance of such terms.

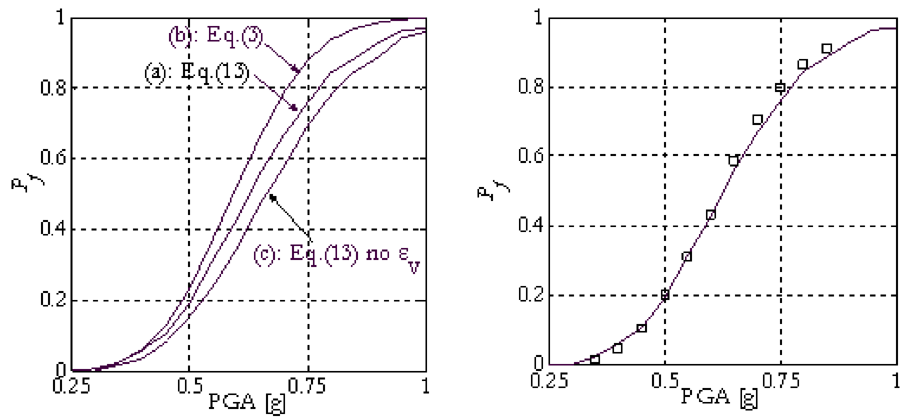

Figure 7 Fragility curves. (Left) (a) full system (b) independent components (c) full system without shear capacity error terms; (Right) Comparison of results obtained with the complete procedure (curve (a) in the Left plot) with Monte Carlo.

Importance analysis

The importance of the individual mechanisms in determining the fragility of the system can be evaluated by comparing of the fragility curves calculated for each failure mode separately with that for the whole bridge. These are all shown in Figure 8. It is apparent that the flexural mode of the lateral short pier dominates the fragility of the system, and that brittle shear failure is not a main concern for this particular structure.

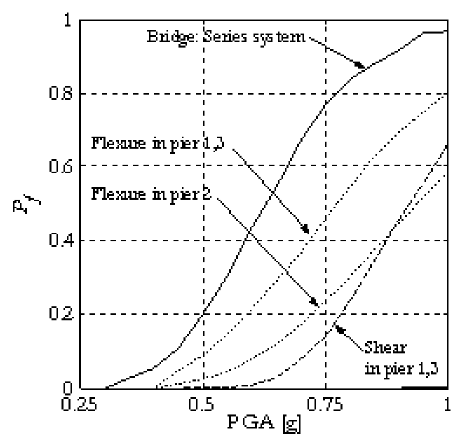

Figure 8 Fragility curves for the entire bridge and the individual components.

Bridge under spatially varying excitation

Seismic action

As anticipated, this bridge is a good example of a spatially extended structure for which a description of the excitation as a non rigid motion that varies between the supports is appropriate.

In this application this motion is described in terms of a vector of correlated random processes. The model used for describing the spatial variability of ground motion is that presented in Der Kiureghian [14] and already applied in Pinto [15] for the statistical study of its influence on the safety of a population of bridges. The model consists of amplitude modulated non stationary processes generated starting from a matrix $\mathbf{S}(\omega)$ of cross- and auto-power spectral densities. This latter is obtained from the power spectra at all support points $S_{ii}(\omega), i = 1,\ldots,n$, consistent with the local soil conditions, and from a model for the so-called coherency functions, i.e. the normalised cross-power densities between the supports:

$$\gamma_{ij}(\omega) = \frac{S_{ij}(\omega)}{\sqrt{S_{ii}(\omega)S_{jj}(\omega)}} \tag{15}$$

The diagonal of the 5×5 matrix of auto- and cross-power spectral densities used in this example contains in positions 1, 2, 4 and 5, corresponding to the abutments and the lateral piers, an identical power spectral density calibrated so as to be consistent with the elastic acceleration response spectrum for firm soil according to Eurocode 8 (the same used for the previous case of uniform excitation); in position 3 (central long pier) the input PSD has a different frequency content and in particular it corresponds to a medium (M) soil type, again according to EC8. For what concerns the coherency function in Eq.(15), the following expression is employed:

$$\gamma_{ij}(\omega) = \exp\left[-\left(\frac{\alpha\omega d_{ij}}{v_s}\right)^2\right]\exp\left[-i\frac{\omega d_{ij}^L}{v_{app}(\omega)}\right]\exp\left[i\theta_{ij}^S(\omega)\right] \tag{16}$$

where the velocity v_s/α and v_{app} are parameters of the spatial variability model that account for the so-called *loss of coherence* and *wave-passage* effects (Der Kiureghian [14]). In the application the following values of the parameters are used: $v_s/\alpha = 300 m/\sec$ and $v_{app} = 900 m/\sec$. The distance d_{ij} is the distance between support points *i* and *j* (50m in this example) and d_{ij}^L is the distance along the travel path of the waves. Finally, the last factor in Eq.(16) accounts for the difference in phase angles with frequency due to different soil conditions at the two supports *i* and *j*.
A set of 20 ground motion suites (each sample consisting of 5 ground acceleration, velocity and displacement histories) is generated so as to satisfy the local frequency content as well as the desired correlation structure, Shinozuka [16]. The motions have been amplitude modulated with the same envelope function used for the uniform excitation case.

Results and comparison with the uniform excitation case
The results in terms of maximum responses, their mean and standard deviation, or their sensitivity to the basic variables, such as those reported in Figures 2, 5 and 6 for the case of uniform excitation are qualitatively similar in this case and are not shown. The most important difference lies in the correlation coefficients between the demands, which are shown in Figure 9 for the same demands as in Figure 4.
One can see how the distance- and frequency-dependent loss of correlation between the input motions has a strong effect in reducing the correlation between the demands. All the correlation coefficients between demands at different piers are close to zero. The symmetry is lost and the response of piers 1 and 3 is not any more perfectly correlated. In addition, both for flexure and shear $\rho_{12} \neq \rho_{23}$: this is due to a directivity effect since the waves are assumed to travel from left to right. The only correlation coefficients that, apart from the small fluctuations around the "yield" value of the PGA, are still equal to unity are those between flexure and shear demands at the same pier, Figure 9 (Right).

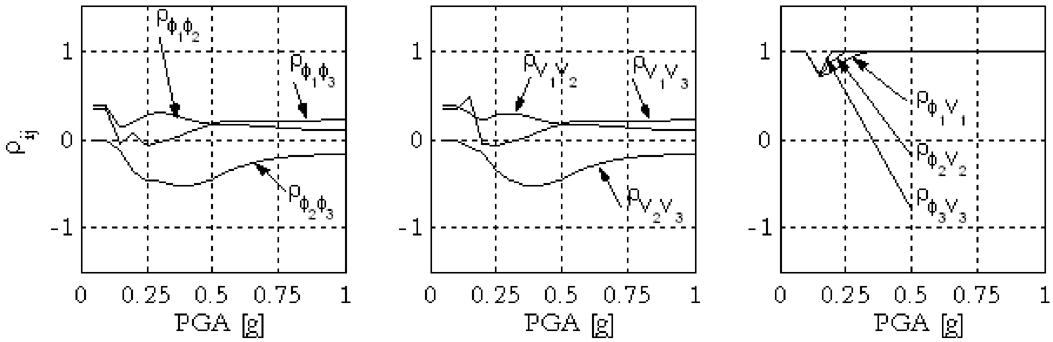

Figure 9 Selected correlation coefficients as a function of the seismic intensity: correlation between flexural demands at the three piers (Left); correlation between shear demands at the three piers (Centre); correlation between flexural and shear demands at each pier (Right).

Whether the described lack of correlation between the demands, due to the so-called "break-up of the modes" caused by the non uniform excitation, is detrimental or favorable for the overall safety of the bridge depends in general on the choice of the parameters of the model of spatial variability. This problem is investigated in more detail with reference to a population of bridges and of non-uniformity scenarios in Pinto [15]. Here the interest is only in showing that the procedure yields a fragility that compares to the "exact" one obtained by Monte Carlo with the same accuracy as in the uniform excitation case. This is shown in Figure 10 (Left), where the two fragilities obtained by solving the system problem in Eq.(13) are plotted together with the corresponding Monte Carlo results. Also shown in Figure 10 (Right) are the correlation coefficients for the shear demands from Figure 4 and 9.

The results shown indicate that, in this example, the case of uniform excitation represents a more severe condition for the structure than the non uniform one.

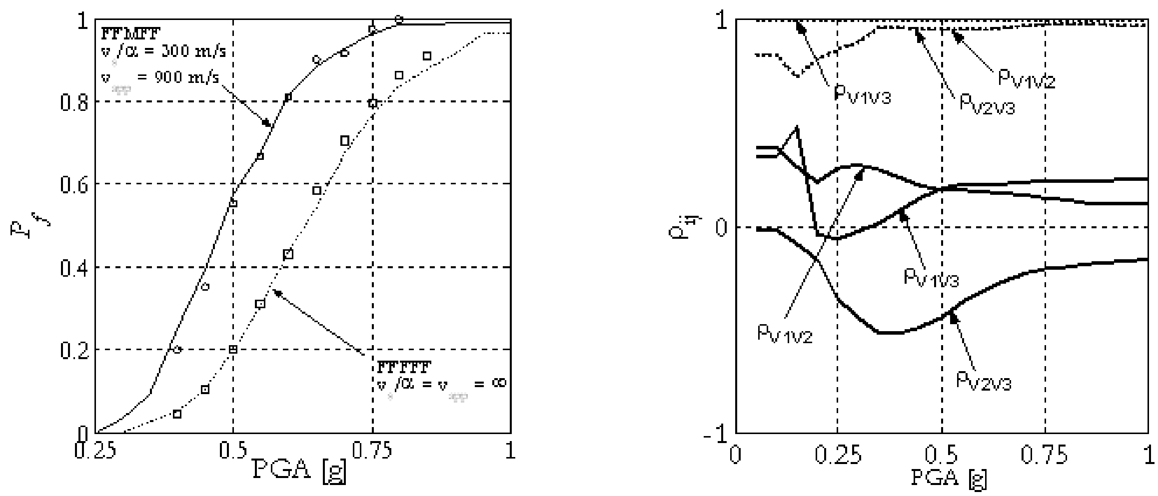

Figure 10 Comparison between results for the uniform and non uniform excitation case.

CONCLUSIONS

A method for the evaluation of the seismic fragility of general structures is presented. The method belongs to the category of analytical approaches to the fragility estimation and can account for all sources of variability, all possible modes of failure and their correlation. The method is demonstrated through an application which clarifies all of its aspects and confirms its effectiveness and accuracy.

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