Paper No. 1557. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

QUANTIFICATION OF BEHAVIOUR COEFFICIENTS FOR CURVED RC BRIDGES

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ABSTRACT

In this report a methodology for the assessment of behaviour coefficients to be adopted in the design of rc bridges is presented. This methodology is based on the computation of the probability of failure of the structure, which is judged against a target value of the probability of failure.

The results of the application of this methodology to a set of 36 idealised bridge structures, give some information about the relationships between ductility and reliability and evidence the major role played by the structural dynamic characteristics (fundamental frequencies) in the structural reliability. Bridge structures are idealised by a spatial model with 6 d.o.f. per node, assuming that nonlinear behaviour will occur only in the piers due to bending, and so a fibre type model is used.

KEYWORDS

Behaviour coefficients; bridge structures; seismic behaviour; ductility; nonlinear behaviour.

INTRODUCTION

The design philosophy for bridge structures to be built in medium to high seismicity zones is based on the general requirement that, although a considerable amount of damage may occur in some zones of the structure after the occurrence of the design seismic event, the structural integrity shall be retained and, in consequence, a minimum amount of resistance shall be available in order that emergency traffic is ensured and repair operations can be carried out easily.

The explicit acceptance of a considerable amount of damage promotes the consideration of the potential nonlinear hysteretic structural behaviour. Design codes generally allow that the structural earthquake response is performed assuming elastic linear structural behaviour and that the effects of nonlinear behaviour can be estimated by correcting the results of the linear analysis with behaviour coefficients whose upper values are prescribed by the codes. In general the values prescribed by the codes are calibrated for "regular structures", i.e., structures with straight axis and vertical piers with similar heights, thus having a simple dynamic behaviour. Some codes such as, for instance, the Portuguese Code for Safety and Actions (RSA, 1983) and Eurocode 8 (EC8, 1994) clearly state that the prescribed values of behaviour coefficients are not valid for special structural configurations, in which cases the behaviour coefficients adopted in the design shall be adequately justified. EC8 explicitly identifies a number of special structural configurations and states that for such structures "The assumption of a ductile behaviour ... is allowed if the selected behaviour factors can be justified by an appropriate nonlinear time history analysis...".

This paper presents a methodology based on nonlinear time history analyses, to evaluate the behaviour coefficients adopted in the design of rc bridges. This methodology is based on the concept of vulnerability function as a fundamental tool for the computation of the probability of failure and involves the consideration of predefined local failure conditions and its generalisation to the whole structure.

BRIDGES TO BE ANALYSED

The bridges contained in this set are regular bridge structures with 3 and 5 piers, having total length of 200 m and 300 m, respectively. These lengths correspond to 4 or 6 spans 50 m long. The piers have hollow circular section with diameters 2.0, 2.5 and 3.0 m and thickness 0.40 m; pier height varies from 14 m to 28 m. Piers were assumed to be built in on the deck and on the foundations. Longitudinal displacements are free in all cases; at the abutments it was assumed that transverse displacements to the bridge longitudinal axes are restrained. Curvature radii R=250 m, R=500 m and R=∞ have been considered. The general layout of the bridges included in this set are schematically shown in Figure 1. For the design of the structures, q values of 1.5, and 3.0 were selected, which gives a total number of 36 different structures to be designed (2 values of behaviour coefficients x 3 values of pier diameters x 2 structures x 3 curvatures).

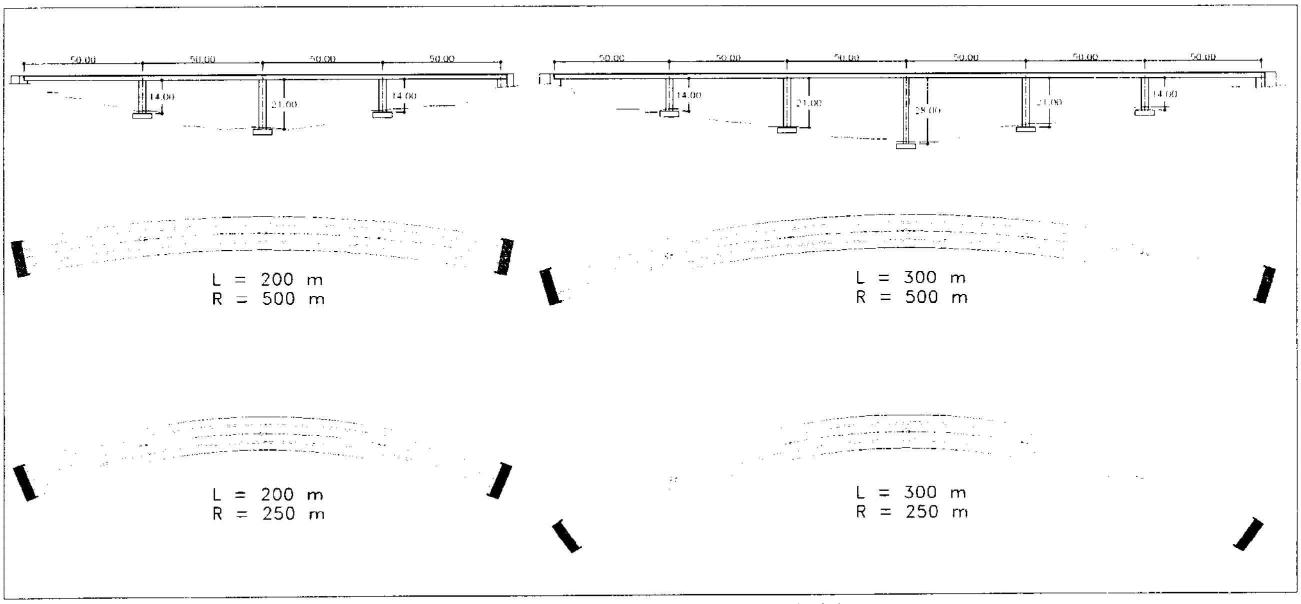

Figure 1 - General layout of the bridges.

Table 1 - Fundamental frequencies (Hz).

Bridge Length (m)	Pier Diameter (m)	Transverse	Longitudinal
	2.0	0.571	0.818
200	2.5	0.711	1.121
	3.0	0.887	1.384
	2.0	0.355	0.732
300	2.5	0.479	1.016
	3.0	0.623	1.274

The first step in the analysis consisted on the dynamic characterisation of the structures, i.e., on the computation of natural frequencies and corresponding mode shapes. Fundamental frequencies in the longitudinal and transverse directions are presented in Table 1. It should be noted that the values presented in this table concern just the case for which $R=\infty$, since for the curved configurations this separation is, in general, meaningless.

The seismic response of the straight axis configurations was computed considering the response spectrum for soil condition B, as prescribed at the EC8, assuming a peak ground acceleration of 150 cm/s^2. An uniform value for modal damping $\zeta=5\%$ was assumed. The longitudinal and transverse amounts of reinforcement in each critical section were obtained by combining the internal forces due to earthquake action with those resulting from the other actions (namely, dead loads and temperature), considering values of the behaviour coefficient $q=1.5$ and $q=3.0$ and a safety coefficient $\gamma_E=1.0$. For $q=3.0$ the reinforcement in a significant number of sections is governed by the minimum amount prescribed in the Portuguese Code for Reinforced and Prestressed Concrete Structures (REBAP, 1983) which is very similar to Eurocode 2 (EC2, 1992). For the curved configurations the critical sections were assumed to have the same arrangement of both the longitudinal and transverse reinforcement as the critical sections of the straight configurations with similar location.

NUMERICAL MODELS

The bridge structures were idealised by spatial models with beam elements with 6 degrees of freedom per node. The weight of the deck is distributed along the span length, resulting an axial force of about 10000 kN at the top of each pier. It should be noted that the structural model used in the linear and nonlinear analyses of each bridge is the same.

In what concerns the nonlinear analyses, the main assumption is that the deck will behave as linear elastic and that the critical zones are located at the piers' extremities. In consequence, it was assumed that the energy dissipation mechanism is constituted by hysteretic hinges at the bottom of the piers. These hinges are represented by nonlinear beam finite elements with a length equal to the equivalent plastic hinge length, which is estimated on the basis of the results presented by Priestley et al (1984); in this particular study lengths of 1.5 m have been estimated for the plastic hinges at the bottom of the piers. The nonlinear behaviour at the potential plastic hinges is quantified by moment versus curvature relationships determined by a fibre model. This model involves the discretization of the critical sections in a number of concrete "filaments" with uniaxial behaviour with the steel bars being considered one by one. The force versus deformation loops for steel are based on the model proposed by Giuffrè et al. (1970), whereas for the concrete the Kent et al (1971) has been adopted.

Earthquake input consists of sets of artificial accelerograms, constructed to match the design spectrum. Each set of accelerograms is constituted by 3 different accelerograms, corresponding to global X, Y and Z directions.

METHODOLOGY FOR THE ASSESSMENT OF BEHAVIOUR COEFFICIENTS

The methodology proposed for the assessment of behaviour coefficients is based on the computation of the probability of failure, which is considered as a measure of the structural reliability. The probability of failure computed for a given behaviour coefficient used in the design is compared against the target value of the probability of failure, enabling the judgement on the adequacy of the behaviour coefficient.

Computation of the probability of failure

The simplest problem of structural safety may be stated as follows (Borges et al, 1985). Consider a population of structures acted by forces S whose intensity is measured by a real variable x. Failure occurs when the variable x reaches the resistance R. Assuming that S and R are random variables it is possible to define f(S,R) as a density of probability function of S and R. The probability of failure is computed by:

$$P_f = \iint_{\Omega} f(S,R)\, dS\, dR \qquad (1)$$

If S and R are independent variables then

$$f(S,R) = f(S) \cdot f(R) = f_S(x) \cdot f_R(x) \qquad (2)$$

and

$$P_f = \int_0^{\infty} F_R(x)\, f_S(x)\, dx \qquad (3)$$

or, equivalently,

$$P_f = \int_0^{\infty} (1 - F_S(x))\, f_R(x)\, dx \qquad (4)$$

where $F_S(x)$ and $F_R(x)$ represent the cumulative functions of the actions and resistance, respectively.

In the present context function f(S,R) is an unknown function. However, the definition of failure of a structure can be based on the failure of its elements or on values of parameters defining its overall behaviour.

Vulnerability function

The methodology essentially consists in the estimation of a vulnerability function, as the fundamental tool to compute the probability of failure of the structure from the failure of its elements and consists on the following steps:

i) Computation of the control variables (variables describing the earthquake action effects on the structure) by linear analysis and considering the characteristic value of earthquake actions;

ii) Design of the structure using the results of the linear analysis corrected with the behaviour coefficient to be assessed;

iii) Estimate of the vulnerability function, defined as a nonlinear function relating the values of the parameters describing the severity of the loading (h) with the values of the control variables (c), i.e., c=V(h).

Step iii) is the critical one in this methodology, due to the high computational effort required by the direct estimation of the vulnerability function, involving the computation of nonlinear structural responses in order to obtain the values of the selected control variables. The computations are performed for several values of the peak ground acceleration, by adequately scaling the selected accelerograms. The method used to estimate the vulnerability function requires that for each value of the peak ground acceleration a few analyses are performed, considering different sets of accelerograms to be used as input. The value of the vulnerability function for each one of the peak ground acceleration values is estimated as the mean value of the maximum values of control variables obtained for the several sets of input accelerograms. On the other hand, the selection of the type and number of control variables is a consequence of the way how local failure (i.e. element failure) conditions are defined.

Control variables and element failure conditions

The structural model used to obtain the nonlinear response is an ordinary finite element model and therefore any quantities describing the response could be taken as control variables. Since, in this particular case, the nonlinear behaviour is quantified by a fibre type model it is possible to take advantage from this refined modelisation and select as control variables some quantities which are commonly dealt with at design level. Among the variety of the results coming from a fibre type model the selection of the maximum values of strains in each critical section appears very attractive due to the intuitive characteristics of those quantities.

Therefore, the only results retained from each nonlinear analyses are the maximum compressive strains at the concrete and the maximum tensile strains at the steel reinforcement in each critical section, meaning that 2 different materials (concrete and steel) are taken into account for the definition of local failure conditions. The problem arising at this point is how to transform the results of the nonlinear analyses in a single function representing the overall structural "resistance", so that the probability of failure can be computed by an expression similar to equation(4).

The first step to solve this problem is to assign a probability distribution to the maximum strains at concrete and steel. Usually, it is acceptable the adoption of log-normal distributions (Borges et al, 1985) :

$$p(\varepsilon) = \frac{1}{\sqrt{2\pi}\,\delta\varepsilon} \exp\left\{\frac{-\ln^2(\varepsilon/\beta)}{2\delta^2}\right\} \tag{5}$$

where β and δ are the parameters of the distribution. Note that, in general, β and δ have numerical values close to the mean value and coefficient of variation of the distribution, respectively.

Then, for a given value of the parameter h representing the severity of earthquake action (i.e. peak ground acceleration) it is possible to compute, for each critical section, the local probability of failure P_{ih} just by taking the ordinate of the cumulative function corresponding to the density function in equation(5). Assuming that the structure consists of a "brittle association" (Borges et al, 1985), the "global" probability of failure is equivalent to the probability of having at least one local failure, which is given by

$$P_{fh} = 1 - \prod_{i=1}^{n}(1 - P_{ih}) \tag{6}$$

where P_{fh} represents the global probability of failure for the actual value of parameter h and n is the number of critical sections. If the process is now inverted looking for the abscissa corresponding to the ordinate P_{fh} in the cumulative function the value of the "global" strain is then obtained. Repeating the procedure for all values of the parameter h a "global" vulnerability function is obtained. As it can be seen the result of this procedure consists on the transformation of the initial problem containing n variables into an "equivalent problem" with only one variable.

This procedure can be extended if more than one material is being taken into account. In that case the quantities P_{fh} shall be estimated for all the materials and a generalised probability of failure P_{gh} can be computed from the probabilities P_{fh} calling again the concept of brittle association. This probability P_{gh} takes into account the behaviour of the several critical sections and the several materials and is given by

$$P_{gh} = 1 - \prod_{m=1}^{mat}(1 - P_{fh})_m \tag{7}$$

Again, if the process is inverted, looking for the abscissa corresponding to the ordinate P_{fh} in the cumulative function, the value of the "generalised" strain (which takes into account, not only all critical sections, but also all the materials) is obtained. Repeating the procedure for all values of the parameter h, a "generalised" vulnerability function is obtained and the corresponding cumulative function F_c as well, whose meaning is similar to $F_R(x)$ in equation (4). The probability of failure is then computed by

$$P_f = \int_0^{\infty} f_h(h)\, F_c(V(h))\, dh \tag{8}$$

<u>Hazard definition</u>

Since the parameter most commonly used to describe the severity of earthquake actions is the peak ground acceleration, earthquake is represented by probability distributions of the peak ground acceleration. Those distributions have been calibrated on the basis of results presented by Oliveira et al (1984) for the Lisbon region. The following extreme type I distribution were obtained (considering a 50 years reference period and the peak ground acceleration a_g expressed in g):

$$p = \exp\left\{-\exp\left[-15.92\,(a_g - 0.1733)\right]\right\} \tag{9}$$

The hazard represented by this distribution may be deemed to correspond to zones with a medium-high seismicity, with maximum values of peak ground acceleration of 212, 354 and 496 cm/s^2 , respectively, for return periods of 100, 1000 and 10000 years.

RESULTS AND DISCUSSION

Strategy of analysis

The bridges have been designed considering the response spectrum prescribed at the EC8, soil condition B and a peak ground acceleration of 150 cm/s^2. Furthermore, 5 different sets of accelerograms, with a duration of 10 seconds, have been constructed to match this spectrum, each set containing 3 accelerograms corresponding to the global X, Y and Z directions, respectively. The accelerograms along the global Z direction represent the vertical component of earthquake action and, consequently, they have been scaled with a factor 2/3, relative to the intensity of the horizontal components. The duration of all accelerograms is 10 seconds.

The nonlinear analysis were performed for the following 15 values of peak ground acceleration (cm/s^2): 50, 100, 200, 300, 400, 450, 500, 550, 600, 650, 700, 750, 800, 900, and 1000.

For the computation of the probabilities of failure, the following values of the parameters of the lognormal distributions which describe the material characteristics (expressed in strains), were assumed:
i) Concrete: β=0.01 and δ=0.12;
ii) Steel: β=0.07 and δ=0.05.

Although a probability distribution of peak ground acceleration corresponding to EC8, soil condition B is not available some similarity was assumed to exist with action type 1 of the Portuguese code and therefore the probability distribution in equation (9) was adopted.

Presentation of results

The probabilities of failure computed are presented in Figure 2 for a 50 years period in a column chart form. These results suggest the following comments:
I) The effect of deck curvature is more remarkable for the bridges 300 m long, possibly indicating that, for a given deck curvature, vulnerability increases with bridge length. From Figure 2 this is more evident for the low frequency structures (pier diameter ϕ=2.0 and 2.5 m);
ii) Structures with low transverse frequencies are more vulnerable, which in accordance with older results obtained for similar bridges (Vaz, 1994);
iii) In general, the adoption of higher values of the behaviour coefficient corresponds, with a few exceptions, to higher values of the probabilities of failure;
iv) In some cases the deck curvature seems to have a slight influence in the probability of failure, as it can be observed for the bridges with pier diameter ϕ=2.0 m, designed for q=3.0. The maximum difference in the probabilities of failure of the structures analysed, considering the extreme values of the radius of curvature (R=250 m and R=∞), is about one order of magnitude.

The probabilities of failure are governed by the failure condition adopted for concrete, meaning that failure occurs due to concrete compression.

The probabilities of failure obtained for the straight longitudinal axis bridges have been plotted together with older results obtained for other bridges (Vaz, 1994) against the fundamental transverse frequency, as shown in Figure 3. Exponential regressions were performed to estimate the relationship between the behaviour coefficients and the dynamic characteristics of the structures.

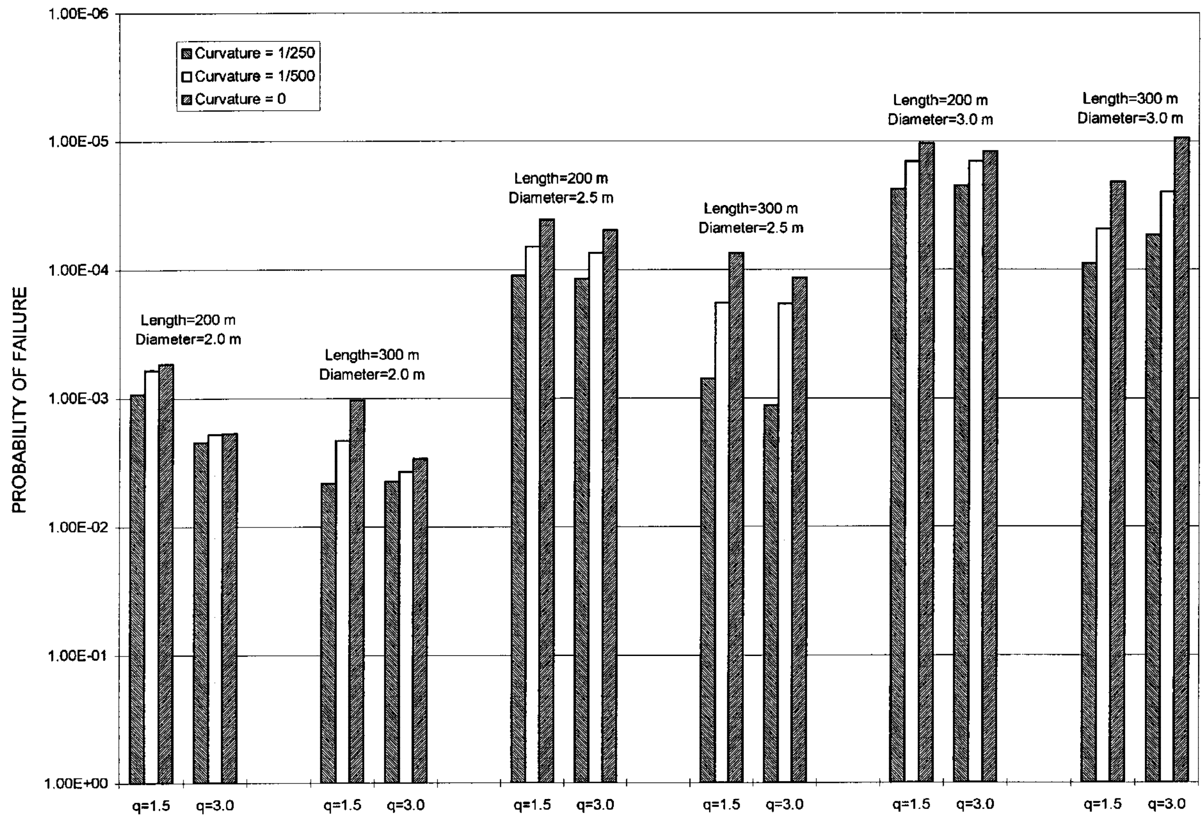

Figure 2 - Probabilities of failure.

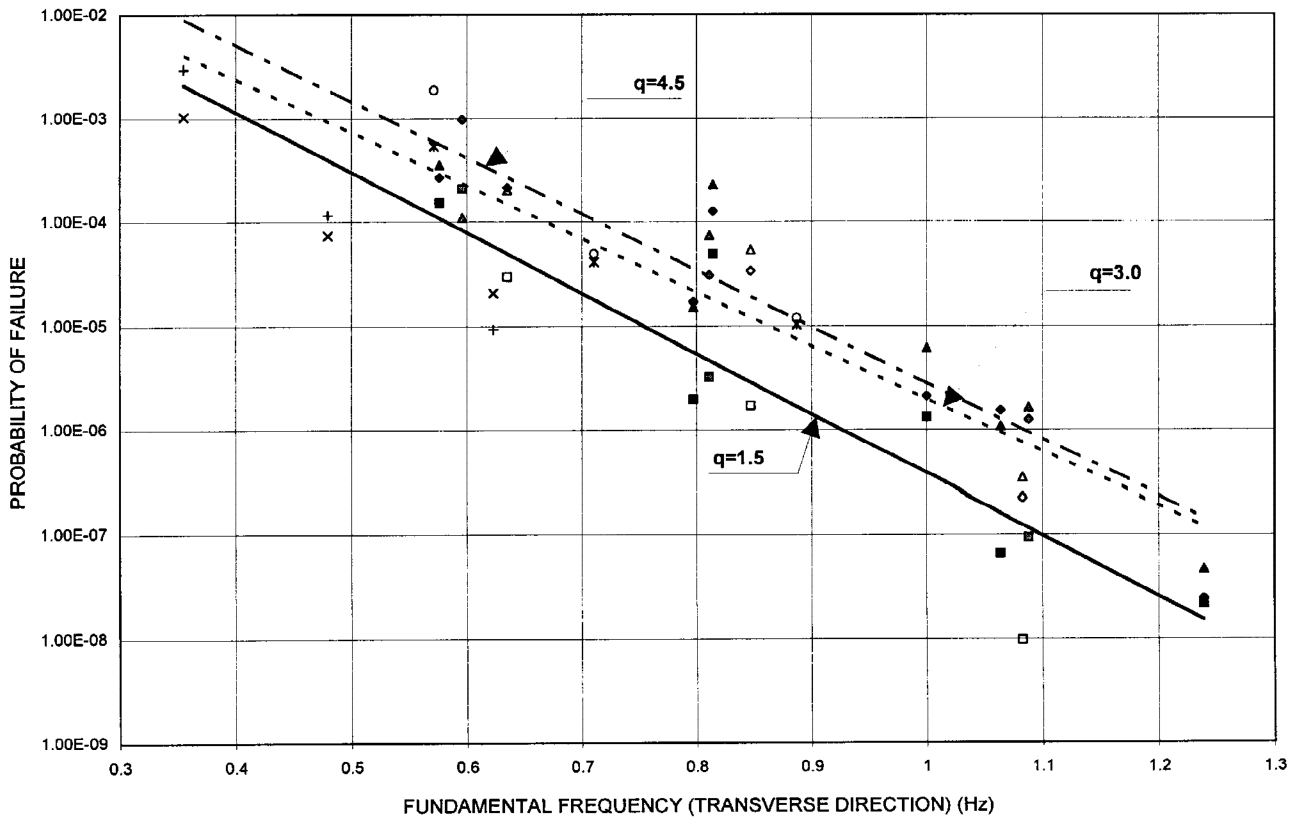

Figure 3 - Probabilities of failure of straight bridges.

CONCLUSIONS

The main result to be retained is presented in Figure 3, showing that the structural vulnerability increases when the natural frequencies in the transverse direction decrease. This result basically agrees with the CALTRANS prescriptions concerning force reduction factors. However, they are in deep disagreement with the results of recent research by several authors on SDOF nonlinear systems (Duarte et al, 1988; Miranda et al, 1994) and on building structures (Campos-Costa, 1994; Arede et al, 1995), which exhibit the opposite tendency. Further research, considering structural types other than the simple one considered in this report, seems, therefore, to be needed.

Also, the simplification inherent to the reduction of the structure to a brittle association for the definition of the global failure conditions, as previously exposed, may be unrealistic, at least in some cases, and improvements in this field seem to be very important.

One important issue concerns the numerical model itself and is related with its validity. The values of the probabilities of failure are probably underestimated, due to the assumption that deck behaviour remains elastic. The recent design of important bridges with prestressed decks in Portugal has shown that, for peak ground accelerations of about 500 to 600 cm/s^2, the traditional design procedures fail, since the design of the deck is governed by the seismic actions, due to the effect of the vertical component. For those intensities the design of bearing devices is also governed by the effects due to the vertical component of the seismic action. Consequently, these effects should not be disregarded in the computation of the vulnerability function, at least in the analyses considering peak ground accelerations higher than the above referred values. However, its consideration and the required improvement in the numerical model would increase computational costs to a level that would make this methodology virtually infeasible.

Finally, the analysis of the results has shown that all the earthquake components should be considered simultaneously in the nonlinear analyses. In fact, the piers placed in symmetric locations along the longitudinal axis exhibit significant differences in what concerns the axial stiffness, which together with the effects of the vertical component makes the symmetry of the numerical model to vanish and originates complex behaviour types.

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