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Quantification of behaviour coefficients for RC bridges

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ABSTRACT: In this paper the influence of the nonlinear behaviour of reinforced concrete bridge structures on their performance requirements is studied considering three-component earthquake motions. Bridge structures are idealized by a spatial model with 6 degrees of freedom per node assuming that nonlinear behaviour will occurr only in the piers. Nonlinear behaviour is supposed to occurr in bending and so a fibre model is used to take it into account. Three different bridges were selected and designed according to the portuguese code provisions quantified for the Lisbon seismicity which are very similar to those of EC8. The reliability of those bridges is assessed through the probability of the ultimate limit state being exceeded.

1 DESCRIPTION OF THE BRIDGES

1.1 Bridge A

This bridge has length of 420 m and is constituted by 9 spans of 50 m except the first one and the last one which have only 35 m. The piers have an hollow rectangular section (exterior dimensions: 3.60×1.20 m^2; thickness: 0.30 m) and their height increases in a linear way from the extremities to the centre where bridge height is 48 m; therefore the piers are 15, 24, 34 and 43 m high. The piers are assumed to be built-in on the deck and on the foundations. The deck is simply supported at the abutments where rotations around its longitudinal axis and transversal displacements are restrained; the displacements in the longitudinal direction are free.

1.2 Bridge B

This bridge has length of 260 m and is constituted by 2 extreme spans 40 m long and 3 intermediate spans 60 m long. The piers have an hollow rectangular section (exterior dimensions: 4.4×1.60 m^2; thickness: 0.40 m) with major inertia along the transversal direction and their height his 12, 22, 24 and 18 m from left to right abutment. The piers are assumed to be built-in on the deck and on the foundations. The deck is simply supported in the abutments but the displacements are free in both the transversal and longitudinal directions.

1.3 Bridge C

Bridge C is 200 m long with four 50 m spans; the centrar pier is 30 m high and the lateral ones are 15 m high. The piers have approximately an I cross section with 6.00 m web and 2.40 m flange. The deck is simply supported at the abutments and over the piers, save for rotations around its longitudinal axis; for those rotations the deck may be considered built-in on the abutments and piers. The deck is fixed in the longitudinal direction to one abutment by a mechanical system which gives a controlled amount of longitudinal flexibility. The deck is supported on the piers and abutments by rubber bearings.

2 DESIGN

The three bridge structures were designed according to the present provisions of Eurocode 8. Reinforcement in the piers was determined according to the Portuguese Code for Reinforced and Prestressed Concrete Structures (1983) which is similar to Eurocode 2.

For Bridge A a base acceleration of 150 cm/s^2, a safety coefficient $\gamma_E = 1.5$ and behaviour coefficients q = 1.5 and q = 3 were considered. Bridge B was designed for similar conditions with behaviour coefficients q = 2, q = 3 and q = 4. It should be referred that for values q = 3 in Bridge A and q = 4 in Bridge B the reinforcement in a significant number of piers is controlled by the minimum reinforcement prescribed in the code and hence larger

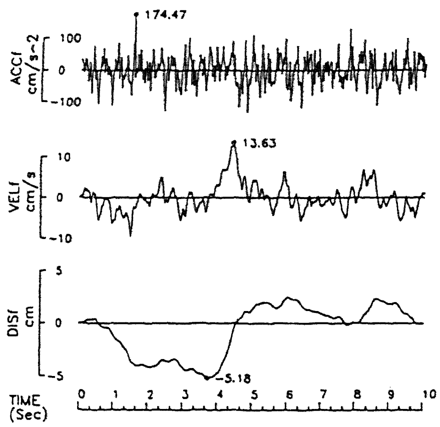


Figure 1. Accelerations, velocities and displacements of a realization of earthquake action type 1.

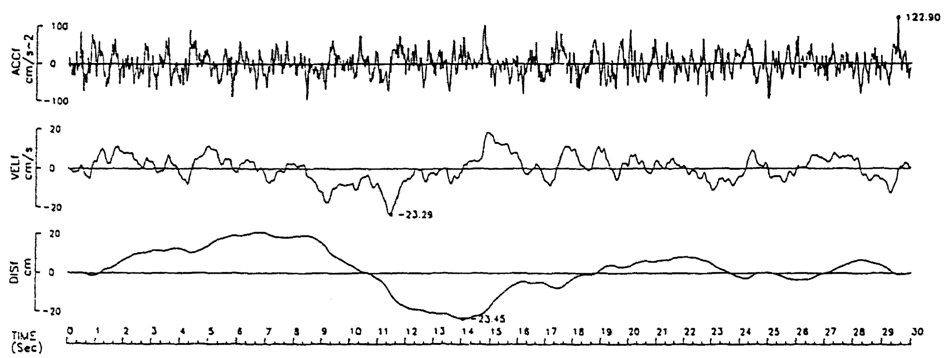
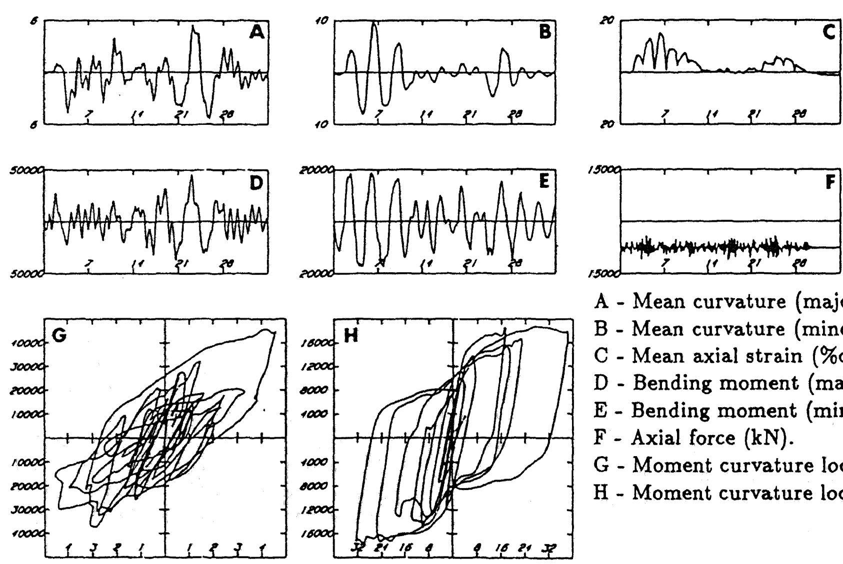


Figure 2. Accelerations, velocities and displacements of a realization of earthquake action type 2.



A - Mean curvature (major inertia) (10^{-3}/m).
B - Mean curvature (minor inertia) (10^{-3}/m).
C - Mean axial strain (%o).
D - Bending moment (major inertia) (kNm).
E - Bending moment (minor inertia) kNm).
F - Axial force (kN).
G - Moment curvature loops (major inertia).
H - Moment curvature loops (minor inertia).

Figure 3. Typical outputs of the numerical model

values of the behaviour coefficient would not correspond to smaller stell quantities. Bridge C was designed for 2 values, 0.25 g and 0.40 g, of base acceleration. An importance factor of 1.3 and a behaviour coefficient q = 3.5 were adopted. Because the design of Bridge C was controlled by requirements other than seismic resistance the reinforcement in the piers is identical for the 2 values of base acceleration considered, the one and only difference being the reinforcement at the base of the central pier where an increase from 0.52% to 0.68% in the longitudinal steel content was achieved.

The design earthquake effects were obtained by linear stochastic dynamic analyses using structural models similar to the ones used in the nonlinear analyses. In the linear analyses the earthquake action was represented by power spectra. The fundamental frequencies obtained by the linear analyses are those ones presented in Table 1.

Table 1. Fundamental frequencies (Hz)

Direction	Bridge		
	A	B	C
Longitudinal	0.39	1.09	0.77
Transversal	0.32	0.99	1.02

3 EARTHQUAKE ACTION MODEL AND HAZARD DEFINITION

3.1 Representation of the actions

The earthquake actions considered are the two actions prescribed in the Portuguese Code for Safety and Actions (1983), which are idealized by stochastic models representative of a moderate magnitude nearby earthquake with a duration of 10 s (type 1 action) and of a larger magnitude more distant earthquake with a duration of 30 s (type 2 action). The main reason for the choice of those actions lies in the need of having long duration actions because one of the bridges considered has a long period (about 3 s) and hence is almost invulnerable to short duration actions (as will be shown below).

For each analysis 3 time series are used, corresponding to 2 horizontal components with the same intensity and a vertical component scaled to 2/3 of the horizontal components intensity; those three time series constitute a realization of the stochastic process representative of the action. Orthogonal components are uncorrelated and rotational components are disregarded.

3.2 Hazard

Earthquake is represented by the probability distributions of the maximum value of the peak ground acceleration. Those distributions were calibrated on the basis of results presented by Oliveira and Campos-Costa (1984) for Lisbon. The following extreme type I distributions were obtained (considering a 50 years reference period and the peak ground acceleration a expressed in g):

- Type 1 action

$$p = \exp\left(-\exp\left(-15.92\left(a_1 - 0.1733\right)\right)\right) \qquad (1)$$

- Type 2 action

$$p = \exp\left(-\exp\left(-22.4\left(a_2 - 0.0203\right)\right)\right) \qquad (2)$$

The hazard represented by these distributions may be deemed to correspond to zones with a medium-high seismicity as can be seen from the results in Table 2.

Table 2. Results from expressions (1) and (2)

Retur period	Peak ground acceleration	
(years)	a_1(cm/s^2)	a_2(cm/s^2)
100	212	50
1000	354	151
1000	496	252

In Figures 1 and 2 the time histories of accelerations, velocities and displacements for a realization of one component of both action type 1 and action type 2 are ilustrated.

4 NUMERICAL MODEL

The bridge structures were idealized by spatial models with 6 degrees of freedom per node. It was assumed that the energy dissipation mechanism is constituted by hysteretic hinges at the base of the piers. Those hinges are represented by nonlinear beam elements with a length equal to the equivalent plastic hinge length. The equivalent plastic hinge length was estimated on the basis of the results presented by Priestley and Park (1984); values of 1.0 m for Bridges A and C and of 1.5 m for Bridge B were achieved.

The nonlinear behaviour at the plastic hinges is quantified by moment-curvature relationship determined by a fibre model. That model involves the discretization of the sections in each nonlinear element in a large number of concrete "filaments" with uniaxial behaviour; steel bars are considered one by one. The force-deformation loops for steel are based on the model proposed by Giufré and Pinto (1970) and the force-deformation loops for concrete

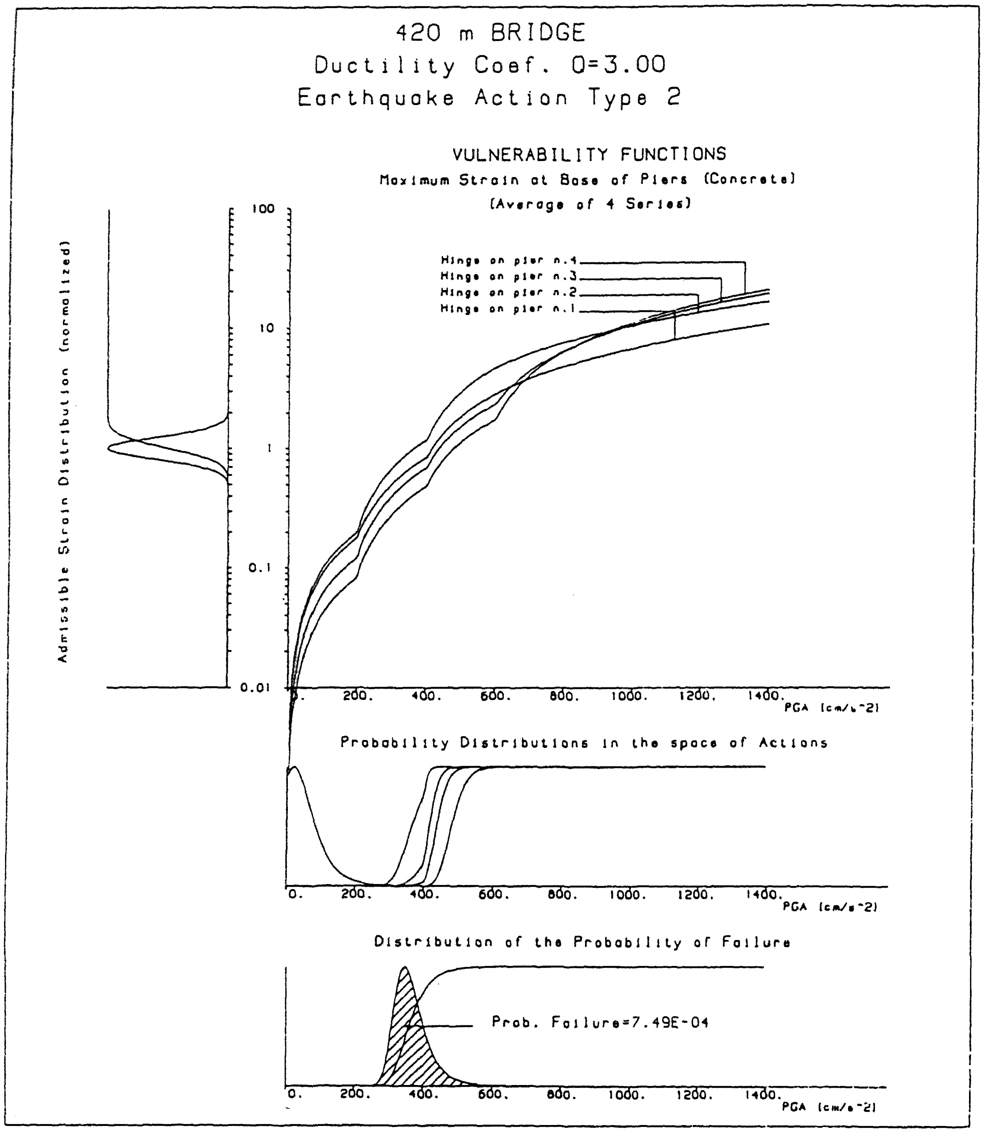


Figure 4. Probability distributions used in the computation of the probabilities of failure of Bridge A.

are based on the Kent-Park model modified as proposed by Park et al (1982).

The response is obtained through a step-by-step integration of the equations of motion by the Newmark method. Damping was assumed to be of Rayleigh type and giving about 2% damping in the fundamental modes of vibration. A detailed description of the numerical model is given by Vaz (1992). In figure 3 some of the possible outputs are ilustrated.

5 CONTROL VARIABLES AND LIMIT STATES

The control variables considered are the maximum values of the compressive strain in the concrete and of the tensile strain in the steel bars, for each hysteretic hinge.

According to Eurocode 8, the limit state to be considered is such that the bridge retains its structural integrity and residual resistance capacity although considerable damage may occur. This limit state is considered to be attained is one of the following conditions is fulfilled:

- The maximum compressibe strain in the concrete in at least one hysteretic hinge is greater than the allowable concrete strain;

- The maximum tensile strain in at least one steel bar in all hysteretic hinges is greater than the allowable steel strain.

The allowable concrete and steel strains ε are quantified by lognormal distributions defined by

$$p = \frac{1}{\sqrt{2\pi\delta\epsilon}}\ exp\left(-ln^2\ \frac{\epsilon/\beta}{2\delta^2}\right) \quad (3)$$

The values of parameters β and δ adopted for the analyses of the bridges under consideration are listed in Table 3.

Table 3. Parameters of Equation (3)

Bridge	Concrete		Steel	
	β	δ	β	δ
A	0.026	0.2107	0.1471	0.198
B	0.0147	0.198	0.0998	0.0499
C	-	-	0.05	0.20

6 RELIABILITY

6.1 Vulnerability function

One of the purposes of the analyses is the identification of the vulnerability function $\mathbf{V}$, defined as the function relating the value of the parameters h describing the severity of the load with the values of the control variables c i.e. $\mathbf{c} = \mathbf{V}(\mathbf{h})$. A rigorous definition of the vulnerability function was presented by Duarte (1990).

The usefulness of the vulnerability function derives from its role in the computation of the probability of failure p_f. In effect, the probability of failure may be obtained (in the one variable case) by

$$p_f = \int_o^\infty f_h(h) F_c(V(h)) dh \quad (4)$$

where f_h is the probability density for the action and F_c is the cumulative probability function for the resistance. The importance of the evaluation of the probability of failure in the case of structures with nonlinear behaviour derives from the fact nonlinear analysis may be performed only when all the yielding values are defined and, in consequence, may not used to find the same yielding values.

The vulnerability function to be considered in equation (4) is obtained from the mean value of control variables calculated for several realizations of the stochastic process idealizing the earthquake action. The determination of the vulnerability function for a sufficient number of values of the parameter h involves high computational effort and hence Corrêa and Duarte (1992) develloped a probabilistic approach for the calculation of the vulnerability function based on the Bayes theorem, allowing the optimization of computational effort.

6.2 Probabilities of failure

The reliability of the bridges is assessed through the values of the probabilities of failure. The probabilities of failure were computed from equation (4) considering 4 realizations of the stochastic process representative of the earthquake action for the determination of the vulnerability functions. The results are summarized in Tables 4, 5 and 6. The probability distributions used in the computation of the probability of failure of Bridge A for action type 2 are presented in Figure 4.

Table 4. Probabilities of failure for Bridge A

Behaviour Coefficient	Probabilities of failure	
	Action type 1	Action type 2
1.5	9.4×10^{-7}	1.9×10^{-4}
3	2.5×10^{-6}	7.6×10^{-4}

Table 5. Probabilities of failure for Bridge B

Behaviour Coefficient	Probabilities of failure	
	Action type 1	Action type 2
2	6.9×10^{-9}	5.2×10^{-7}
3	3.2×10^{-9}	8.4×10^{-7}
4	3.8×10^{-9}	1.3×10^{-6}

Table 6. Probabilities of failure for Bridge C

Design PGA	Probabilities of failure	
	Action type 1	Action type 2
0.25g	1.1×10^{-9}	-
0.40g	6.8×10^{-11}	-

Concerning Bridges A and B it can be seen that although action type 1 corresponds to higher peak ground accelerations the probabilities of failure for the action type 1 (duration 10 s) are much smaller than for action type 2 (duration 30 s) indicating the very important influence of the duration.

On the other hand some reasons may explain the low values obtained for Bridge C. Firstly, the bridge is overdesigned in regard to earthquake actions due to the fulfillement of other requirements. Secondly, the low transversal frequency and relatively small duration of the earthquake action is responsible for a lower level of the ductility demand than would be the case with an higher frequency or longer duration earthquakes. Besides, as can be seen in Table 2, peak ground accelerations much higher than the design values are relatively improbable in terms of the hazard considered. In this case, due to the very high resistance of the structure, it should be necessary to assess the possibility of having failures at the bearings.

Comparing the results for Bridge A and Bridge B it seems that the probabilities of failure of low frequency bridges are much more sensitive to the variation of the behaviour coefficient than the ones of stiffer bridges . Although these results can not be faced as a trend because of the little number of cases studied this subject deserves further investigation.

7 FINAL REMARKS

The evaluation of the earthquake behaviour of 3 different bridges was presented, as well as the correspondence between values of the behaviour coefficient or the design peak acceleration and the probabilities of failure. Those probabilities of failure were computed for a given definition of the limit states, envolving maximum strains in concrete and steel. It seems that the more critical uncertainties for the interpretation of studies such as the one carried out in this paper are connected with the accurate definition of limit states. This definition needs both a clarification of the concept of limit state in phenomonological terms and its translation into the variables that quantify the numerical model.

On the other hand the results have shown that the bridge structures analysed are more vulnerable to long duration earthquakes, thus suggesting that Eurocode 8 provisions concerning hazard definition should be reformulated to account for this fact.

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