Paper No. 2112. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

EUROPEAN PERSPECTIVES ON SEISMIC DESIGN OF BRIDGES

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ABSTRACT

The situation in Europe is reviewed under the triple perspective of the availability of a unified code for the design of new bridges, of the amount of the research, completed and under way, to improve the quality of the code, and of the risk to which bridges built in the post-war years are exposed. The content of the Eurocode 8/2: Bridges is briefly outlined, the main emphasis being in its conceptual layout and in a few aspects of detail which may differentiate it from other comparable codes: soil-structure interaction, variability of soil motion, control of compatibility of the displacements, design of the bearings and of the connections. A sample of the relatively large amount of research presently devoted to bridges is then presented: the selected topicis the effect of the variability of soil motion on the inelastic demand on bridges designed disregarding this phenomenon, and the effect of the same phenomenon on the behavior or seismically isolated bridges. Instructive, though partial, conclusions are drawn in both cases. Finally it is mentioned that the activity necessary to face the retrofit needs, which are grave, is progressing slowly, both in practice and in the research field where, however, the first large programs are being initiated.

KEYWORDS

Asynchronouos motion, Bridges, Isolation, Prenormative Research, Seismic Codes.

INTRODUCTION

Seismic design of bridges in Europe has progressed with considerable delay with respect to that of buildings. As far back as 1980, under the auspices of the CEB, a committee formed by specialists from Europe, New Zealand, U.S.A. and Japan drafted a model code for the seismic design of buildings (CEB, 1980) already embodying the philosophy and the approaches which are now commonly accepted. This document provided the basis for the development of the Eurocode (CEN, 1994) for the seismic design of buildings, which started a few years later and appeared in its first draft form in 1988.

Not until 1994 a document of similar quality: the EC8 Part 2 (Seismic Design of Bridges) has been available for bridges; and in the meantime the existing national codes were (and are) for the largest part grossly inadequate.

The consequences of this delay vary depending on the amount of bridge construction that has taken place, mainly after the war, in the european countries of larger seismic activity: Greece, Italy and Portugal. The situation is more critical in Italy, where highway construction started in the late fifties and ended in the mid seventies, with a total of more than 6.000 Km, a large part of which crossing mountainous areas of significant seismicity. The design of bridges was made according to simplistic seismic regulations based on elastic design, low seismic forces and no ductility provisons at all, i.e., in a way not substancially different from that followed in those years in other parts of the world, as for example in California, and the consequences are obviously of similar nature.

The effects of the seismic events occurred starting from 1989 in the U.S.A. and in Japan have been strong incentives to the development of a modern unified european code.

Drafting of the code has had in turn two parallel beneficial effects: the birth and a continuing growth of a line of research in support of its provisions, and an increasing awareness by the public of the state of risk of most bridges as they are now, and of the need of drastic inteventions to reduce it to acceptable levels.

The state of advancement along these fronts, however, is unequal: the code is finished, the research has concentrated up to now on problems related to new designs, while procedures and techniques for assessment and redesign have received insufficient attention. And actual interventions for retrofitting have been implemented until now only occasionally i.e. when a bridge was in need of repair works.

On the positive side, a program for the systematic assessment of the seismic risk of the highway bridges is prsently being developed in Italy, and cooperative european research on retrofit techniques will also start soon: the next few years will hopefully see Europe keeping up with the ongoing progress in these areas as well.

OUTLINE OF EUROCODE 8 PART 2 : BRIDGES

Scope, Requirements and Format

The code covers in full the design of road/railway bridges of the most common types and geometry: continuous or simply supported beam-like superstructures, supported by vertical pier systems of different types: simple or multiple bents, walls, trusses, etc. The material can be reinforced or prestressed concrete, steel or composite. For other bridge types the code is meant to provide basic criteria, but has to be supplemented by ad hoc provisions.

The requirement of EC8 Part 2 with regard to the design of bridges is that "*..communications shall be maintained, with appropriate reliability, after the design seismic event*", followed by a classification of bridges into three categories of decreasing importance, for which the design seismic event is progressively less severe. In the Appendix, it is indicated that a probability of exceedance of 10% in 50 years for the design action could be assumed for bridges of average importance.

Next, two limit states (LS) are identified, to serve as the basis for the design: the Ultimate: ULS, and the Serviceability: SLS.

The ULS is defined as a state still relatively far from the actual collapse of the bridge, in which considerable damage may have occurred to the parts expected to contribute to energy dissipation, but without much degradation, so as to allow for easy repair, and such as to preserve the capability of the bridge of carrying emergency traffic.To comply with the above behavior it is required that the deck should remain essentially elastic, while the dissipation of energy should take place in well defined regions of the piers.

The SLS is more commonly defined as a state of light damage, restricted to those structural parts which are meant to dissipate energy only, and such as not require reduction of traffic or immediate repairs.

Seismic Action

The model of the seismic motion is the same for all Parts of EC8: Buildings, Bridges, Towers, Tanks, ... It is brieflly recalled here that the basic description of the soil motion is in terms of an *elastic response spectrum*. The shape of the spectrum comprises four portions, (i.e.: increasing acceleration, followed by constant acceleration, velocity and displacement, respectively) separated by the values of three periods.
The spectral shapes are assumed not to depend on the magnitude of the scaling factor, which is the peak ground acceleration (PGA).
The elastic response spectrum is to be considered as a uniform risk functional, whose ordinates have a probability of exceedance (given the value of the PGA) of 50%.

Three types of soil profiles are defined: A, B, C, ordered with decreasing overall stiffness; from profile A to C there is a shift of the corner periods towards higher values, while the maximum amplification is essentially unaffected, with only a 10% reduction foreseen for subsoil class C.

As in EC8 Part 1, representations of the seismic motion equivalent to the *elastic response spectrum* are allowed, in the form of a stationary gaussian random process characterized by a power density spectrum, or in the form of a number of artificially generated, spectrum-compatible, time histories.

Characterization of the spatial variability. Consideration of the spatial variability is mandatory:
- for bridges whose length exceeds 600 m, even if the soil properties are uniform.
- when the soil beneath the bridge presents marked geological variations (ex. alluvial soil in one part and rock in another), or marked topographical features (ex. deep valleys).

Acceptable methods to account for spatial variability are described in the Appendix. Briefly, they may be classified into the following three types.

I) The motions at the various points are components of a random field, homogeneous in space (i.e. the differences in the motion depend from the relative distance between any two points only, not on their absolute positions), and stationary in time.
The random field is completely defined by the covariance matrix, whose diagonal terms contain the power spectral densities at the various stations, and the i,j terms the cross power spectral densities between stations i and j. This model is suited for linear random vibration analyses or, if correlated samples of the field are generated at every station by numerical simulation, for linear or non-linear step-by-step response analyses.

II) A simplified random field model, where independent, band-limited motions are generated and allocated to the nodes of square meshes, one square mesh of different size for each frequency band. The motion at a generic point is a weighted average of the motions defined at the four nodes of the squares.

III) A purely kinematic model, consisting on a set of static relative displacements, whose magnitude depends on the distance of the generic support point from a reference one.
Calling ΔX this distance, the expression for the relative displacement is:

$$d = \Delta X \; vg/c_p \leq \sqrt{2} \; dg$$

where vg and dg are the peak ground velocity and displacement, and cp is the velocity of the body waves.

Analysis

Modeling of the structure and of the soil. In the case of straight bridges, it is permitted to use two separate models, for the analysis in the longitudinal and transverse direction, respectively.
When using linear methods of analysis, the actual stiffness of the various elements should be evaluated as realistically as possible. For this purpose: for non ductile bridges and for r.c. or prestressed components not undergoing yielding, the uncracked stiffness is to be used, while for ductile piers an effective moment of inertia, which takes into account of the distribution of the curvatures along the height, is in the Appendix.

Soil deformability, and hence soil-structure interaction (SSI) may be disregarded where the contribution of soil flexibility to the total displacement (measured at the centre of mass of the deck), is less or equal to 30%. When SSI is considered, and the evaluation of soil properties is subject to uncertainty, upper and lower bounds have to be estimated, and the analysis repeated using the two sets of values.

Behavior factors

Values of the behavior factor are given below as an example for reinforced concrete ductile bridges only. They are:
- vertical piers: $q = 3{,}5$ (H/L ratio $\geq$ 3,5)
- squat vertical piers $q = 1{,}0$ (H/L $\leq$ 1)
- inclined struts in bending $q = 2{,}0$
- abutments $q = 1{,}0$

linear interpolation of q for intermediate H/L ratios

The maximum value of q is kept intentionally low, in comparison with the values found in other codes and also with respect to the actual amount of ductility that bridges designed according to EC8 Part 2 are able to develop. The main reason for this choice is to allow for concentrations of ductility demand on some of the piers due to the "irregular" geometry of the bridge, which cannot adequately accounted for by an elastic analysis.
Non-linear step-by-step analyses, using time-histories which are compatible with the elastic spectrum (q = 1) appropriate for the site, are allowed. These analyses, however, can only be used *in addition* to the standard response spectrum analyses, to provide insight in the post elastic response and to compare required with available ductilities. In no case the results from the non linear analyses can be used t relax the requirements resulting from the response spectrum analyses.

Verifications

The design strengths of the elements/mechanisms are calculated as for non seismic conditions, using the relevant material code with the same partial safety factors, with the exception of the shear strength in the plastic hinge regions, which is appopriately reduced to account for the interaction with flexure.

Capacity design (CD) effects are to be used for the design of:
- flexural and shear strength of the deck
- shear strength of the piers
- connections between deck and piers/abutement
- foundations of the piers/abutments

The values of the moments which are used to calculate the CD effects are obtained by the expression:

$$M_o = \gamma_o \cdot M_{Rd}$$

where M_{Rd} is the resisting moment calculated with the actual amount of reinforcement, and the overstrength factor γ_o is expresed as:

$$\gamma_o = \{1 + 2(\eta_k - 0,1)^2\} \cdot (0,7 + 0,2\ q)$$

in which $\eta_k = N_d/A.f_{CK}$ is the reduced axial force. It is noted that γ_o increases, logically, with the amount of normal force and with the value of q.

Specific detailing rules

The final chapter contains important dimensioning criteria and rules for:
a) ensuring a ductile behavior of the piers (and of the piles, when required);
b) designing bearings and links;
c) designing the abutments.

Omitting the relatively standard provisions for ductile detailing, a mention will be made to points b) and c).

b) The design of *connections* between deck and piers/abutment is one essential element to fullfil the ULS requirements. The criterion adopted is to require that:
   - *fixed bearings* be either designed for CD effects, or for the effects deriving from the design seismic condition, but in the latter case be combined with seismic links acting as a second line of defense.
   - movable bearings be designed to accomodate without damage the sum of the displacements due to the seismic action, the long-term displacements due to permanent actions, and the displacements due to thermal effects

   In addition to the above, either adequate seating lengths or seismic links need to provided, as specified in the following:
   - *elastomeric bearings*
   - when they are used without any other accompanying type of bearing or link to resist the seismic action, they are seen as isolating devices, and their design, as well as that of the whole bridge, is subject to the provisions of the relevant section of the code.
   - when they are used in combination with fixed bearings or seismic links which resist the design seismic action, they must be designed to accomodate without damage the maximum shear deformation
   - *seismic links*

   In those cases where seismic links are required, they must be designed to resist:
   - capacity design effects, when the links are combined with elastomeric bearings
   - when links combine with movable bearings, and the seating length is not adequate, in the absence of a rational analysis which accounts for the dynamic interaction between piers and desk, a force equal to: $\alpha \cdot Q$, where $\alpha = A_g/g$ with A_g the design ground acceleration, and Q the weight of the section of the deck which is linked or, when two deck sections are linked together, the least of the two weights.

   The links must be provided with a slack so as to remain inactive under the design seismic action.
   Minimum overlap lengths. When no links are provided at supports where relative displacements are expected to occur, the minimum overlap between supported and supporting elements must be larger or equal to the sum of: the minimum support length fr non-seismic situations ($\geq$ 40 cm), the relative displacement due to the differential motion of the soil, the relative displacement due to the design seismic action.

c) The design criterion regarding abutments is that they should resist the design seismic action with an almost elastic response. A distinction is made between flexible or rigid connections with the deck.

In the case of *flexible connection* (movable or elastomeric bearings) the capacity criterion requires the abutments to remain stable under the earth pressure including seismic effects, the inertia forces due to its own mass, and the actions from the bearings determined as capacity design effects, (30% increase of the friction coefficient, 30% increase of the stiffness of the elastomeric bearings, for the shear deformation due to the design seismic action).
If the earth pressure is determined on the basis of an acceptable displacement of the abutment, it has to be proved that this displacement does not in fact lead to the functional failure of the abutment.

In the case of *rigid connection*, the model adopted for the seismic analysis has to account, in an appropriate way, of the interaction between soil and abutment, using high and low estimates of the soil parameters. This analysis has to be made with q-factor equal to 1, and the resulting forces are used both for the design of the connection and the verification of the abutment.

SELECTED RESEARCH TOPICS

Of the several subjects on which code-oriented research has been active recently in Europe, a summary presentation is possible here of two of them: the effects of the spatial variability of the motion on the inelastic response of bridges designed according to the customary assumption of rigid base motion, and the influence of the same phenomenon on the design of seismically isolated bridges. A detailed presentation of the results can be found in (Monti *et al.*, 1995).

Spatial variability of the motion

Although the spatial variability of the soil is a well recognized phenomenon, and for certain types of extended-in-plan structures, as for ex. pipelines, a differential soil motion is at the base of their seismic design, bridges, even long ones, are normally designed either ignoring altogether this reality or using extremely idealized models. EC8 Part 2 is at present perhaps the only code where consideration of spatial variability is made mandatory in certain situations and guidance is given on how to account for it, as mentioned in a previous paragraph.

The publication of EC8/2 is too recent for having bridges already designed by taking into account the clauses for the variability of the motion, and hence for seeing how design is affected.
Actually, experience with the matter is really scarse internationally. On one hand the data necessary to describe quantitatively the phenomenon in real cases is missing and on the other designers have yet to acquire sensitivity on the likely effects induced by the phenomenon on the response.
A program of systematic investigations on simple bridge geometries has been started recently, (Monti *et al.* 1995), with the purpose of assessing the influence of non-synchronuous ground motion on the inelastic response of bridges conventionally designed with rigid base input and the q-factor approach. A brief account of these studies and of the interesting indications already obtained is given in the following.

The selected geometry of the bridge is shown in Fig. 1: it is a 6-span continuous bridge with 5 piers of the same height H and a diameter of 2,5 m. Three piers heights have been considered: H = 7,50 m, 10,0 m, 15,0 m, so as to have widely different bridge stiffnesses. The corresponding fundamental periods are T = 0,43 sec, 0,60 sec and 1,20 sec. The bridges have been designed elastically for the transverse direction only, for a PGA of 0,42 g and the spectral shapes given in EC8/2 for the case of firm (F) and intermediate (M) soil conditions, respectively. In the design, q-values of 2,4 and 6 have been adopted.
The model used for the spatial variability is worth being described in some detail, since a good understanding of the input makes the interpretation of the results much easier.
It is a random field having the same power spectral density (PSD): $S(\omega)$ at all points, consistent with the response spectrum used for the design.

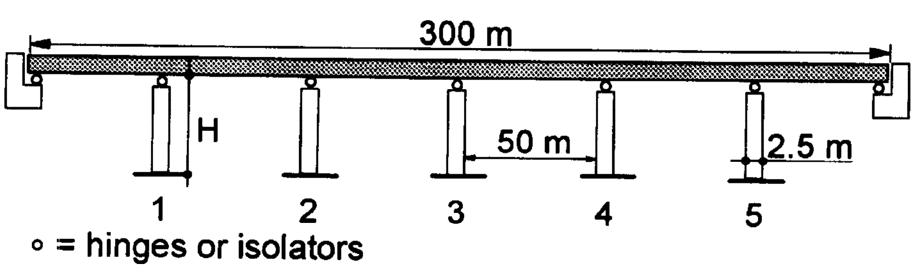

Fig. 1 - Schematic geometry of the analyzed bridge

The cross power spectral density: $S_{ij}(x, \omega)$, which expresses the correlation between the generic points i and j separated by a distance x, is defined through the expression:

$$\gamma(x,\omega) = \frac{S_{ij}(x,\omega)}{S(\omega)}$$

in which $\gamma(x, \omega)$ is called the coherency function. The following form (Luco-Wong, 1986), has been selected for $\gamma(x, \omega)$:

$$\gamma(x,\omega) = \exp\left[-\left(\frac{\alpha\omega x}{V_s}\right)\right]^2 \exp\left[\frac{i\,\omega x^L}{V_{app}}\right]$$

The function $\gamma(\cdot)$ is in the form of the product of two functions.
The first one decays exponentially with the square of the frequency and of the distance; V_s is the shear wave velocity and α a parameter. The second one depends on the projected horizontal distance x^L along the direction of wave propagation, and is a measure of the delay in the arrival times of the waves due to the finite value of the apparent velocity V_{app}. Samples of the field generated from the first term only (as if $V_{app} = \infty$ in the second one) are trains of waves travelling with the same speed and differing from one another by random phase angles only, whereas the second term would generate at each station identical processes, but shifted in time.
The second term is therefore deterministic in nature, while the first one is essentially stochastic. This last remark implies that in the case of large uncorrelation (= large values of V_s/α) the motions at any two points become statistically independent, so that at given instant in time the two motions might hypotetically be the same but opposite in sign.

In the study performed the coherency function $\gamma(x, \omega)$ has been considered as function of two parameters only: the ratio V_s/α and the apparent velocity V_{app}. The range of values adopted for each of the parameters is:

$V_s/\alpha \equiv 300,\ 600,\ \infty$

$V_{app} = 600,\ 1200,\ \infty$

The case of $V_s/\alpha = V_{app} = \infty$ coincides with a rigid body motion. For the non-linear analyses, samples of the random field described above have been numerically generated, each sample consisting of 7 time-histories, 20 secs long, one for each of the piers and abutments.

Selected results are presented for the case of H = 7,5 m, on soil types F and M.
Fig. 2 shows the required maximum ductilities at the piers bases, obtained as the average of the maxima from 10 analyses, each one with a different set of ground motion histories.

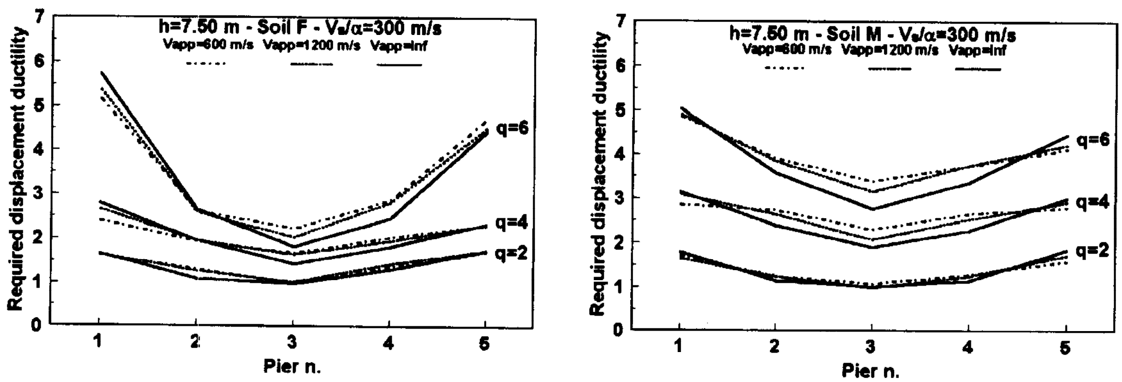

Fig. 2a) - Conventional bridge H=7.5 m on soil F and M. Required ductility for $v_s/\alpha = 300$ m/s.

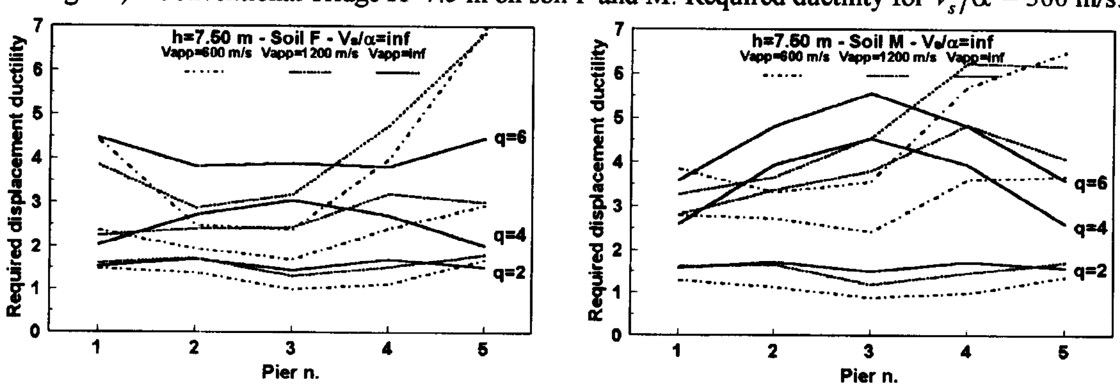

Fig. 2b) - Conventional bridges H = 7,5 m on soil F and M. Required ductility for $v_s/\alpha = \infty$ m/s.

Fig. 2a) contains the results for the largest uncorrelation of the geometric type: $V_s/\alpha = 300$ m/sec coupled with all the 3 values of the second source of uncorrelation (V_{app}). The lack of correlation due to the first term is such as reduce drastically the net dynamic effect with respect to the case of synchronous excitation, so that the response is mainly due to the imposed pseudo-static differential displacements at the piers bases (second term).
The result is that the ductility demands are larger for the lateral piers than for the central ones. This is not surprising, since from the rigid base design the central piers are made much stronger than the lateral ones while the distortions imposed by the differential soil motion are almost the same between all piers, and consequently the weakest ones are called to greater ductility demands.

Fig. 2b) shows the results for the case of full correlation of the first component, again coupled with the three values of the second source. Here the dynamic effects are almost the same as for the rigid-base motion, to which the (static) effects of wave propagation are added. The response in terms of required ductilities for the rigid-motion case is given in Fig. 2b) in the full line.

Seismic Isolation

Within Europe, seismic isolation of bridge structures has been employed almost exclusively in Italy, with a number of applications exceeding 150, both for new designs and for retrofitting existing structures.
For what concerns the codes, the very recent EC8/2 contains a section on the subject, which will be developed more fully when it will become a final European Norm while progress is being achieved in the standardization of the isolating devices as european industrial products, covered by an ad hoc norm (CEN, 1993)
To the extent that technological problems related to the mechanical devices, including production control, stability of the characteristic, maintenance, etc., can be considered as solved, the design of isolated structures is a rather straightforward task which has the advantage, among other things, of not requiring the difficult art of ductile detailing.

One aspect which is practically ignored by the codes on seismic isolation of bridges is the spatial variability of the motion: this is surprising, given the role the displacements play on the success of the design.

Within the same framework of the study previously reported on the effects of soil motion variability, the investigation has been extended to the case of bridges with bi-linear isolating devices.
The results of this study, which provide a first, order of magnitude assessment of the importance of the phenomenon, are presented next.
The bridge model is the same as for the non-isolated case, and is shown in Fig. 1.
For the design of the isolation different criteria have been considered: only one of them will be discussed here, consisting in distributing the total horizontal force transmitted from the deck to the sub-structure in equal parts among all piers and abutments. This criterion involves having the same isolators all through the bridge and, consequently, also the same amount of reinforcement in all piers. The total horizontal force mentioned in the above is the sum of the shear forces at all bearings, obtained by means of a modal analysis of the non-isolated bridge using the EC8/2 response spectrum for a PGA = 0,42 g divided by the so-called "protection factor": μ.
The values adopted for μ have been: 4,2, 5,3 and 7, which correspond to an elastic response of the isolators for PGA values of 0,10 g, 0,08 g and 0,06 g, respectively. The elastic stiffness of the isolators has been set pragmatically at: $k_i = 150\ F_{yi}$, which implies having the same yield displacement irrespectivly of the yield force, and equal to: 1/150 = 0,0067 m.
Finally, for what concerns the design of the isolators, the hardening ratio has been attributed two values: $b = 0$ and $b = \bar{b}$, the latter value calculated so as to have a total hardening ratio of 0,10 for the series system composed by the isolator and the pier.
The piers of the bridge have been designed to satisfy the condition:

$$M_y = \gamma_o\left(F_{yi} + F_{P,max} + c \cdot N\right) \cdot H + N \cdot \Delta_{max}$$

where $F_{P,max}$ is the base shear due to the mass of the pier, c is the friction coefficient of the isolator, N the gravity load on its top, Δ_{max} is the maximum displacement at the piers top, and γ_o a factor to cover the uncertainties on the values of F_{yi}, c, and of the hardening ratio of the isolator.

The model used for the spatial variability is identical to the one already described, as are the values of the parameters used in the non-linear dynamic verifications. Selected results are presented for the bridge with H = 10 m: those with H = 7,5 and 10 m show similar behaviour.

The first fact to note is that the design criterion adopted for the piers proved to be adequate in protecting them from yielding in all cases of synchronous and non-synchronous motion.
Fig. 3 shows the maximum displacements of the isolators: the values represent the averages of the maxima over ten analyses performed with different sets of ground motion histories. The figures on the left and those

on the right contain the results for the hardening b = 0 and b = 0,1, respectively. It can be seen that the hardening has a considerable effect in reducing the displacements, and that the reduction is roughly proportionally the same in all cases: the comments can therefore be addressed indifferently to either of the two cases: the one with b = 0 will be considered.

The most important remark is that even in the extreme cases the variability of the motion does not alter the orders of magnitude of the displacements.

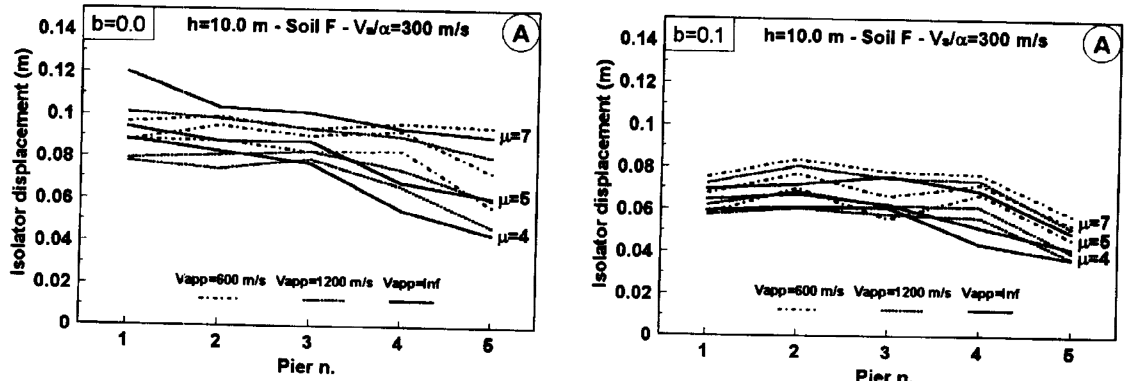

Fig. 3a) - Bridge H=10.0 m. Total hardening = 0.0 and 0.1. Isolators displacement for $v_s/\alpha = 300$ m/s.

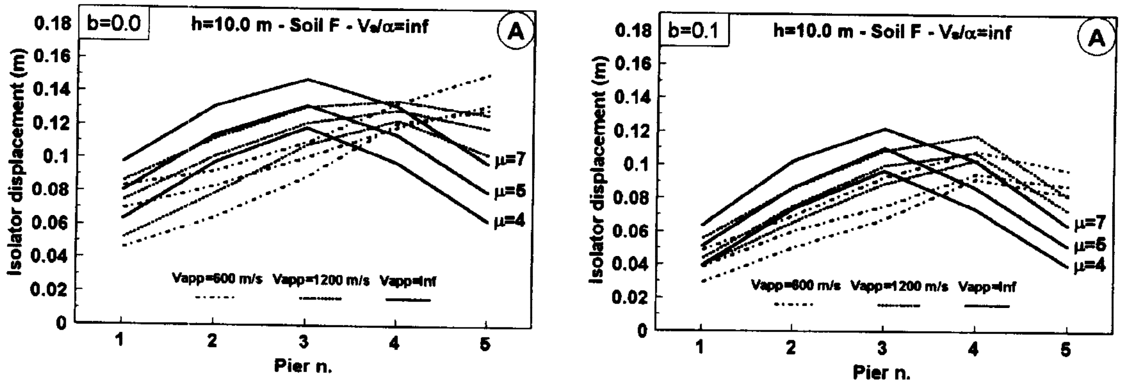

Fig. 3b) - Bridges H = 10,0 m. Total hardening = 0,0 and 0,1. Isolators displacement for $v_s/\alpha = \infty$ m/s.

The reference values (synchronous motion) are given by the solid lines in Fig. 3b, for the 3 values of μ. Taking for ex. $\mu = 5$, the displacements vary in this case from 8 cm over the lateral piers to 13 cm at mid-length (the synchronous motion excites the first mode of the deck).

Considering now Fig. 3a, which represents the case of max geometric uncorrelation (i.e. reduced net dynamic excitation) one has 9,5 cm as the maximum at the extremities and 9 cm at the centre (the curves for $V_{app} = 600$ and 1200 m/sec are not symmetrical with respect to midspan: this occurs due to the directionality of the term representing wave propagation.

A symmetrical envelope should be considered however, corresponding to the envelope of the cases for $\pm V_{app}$).

Globally therefore, one might conclude that, similarly to what happens for the ductilities in the non-isolated bridges, geometric uncorrelation (≅ statistical independence of the motion) leads to a reduced response.

If now Fig. 3b is examined, in which the only source of uncorrelation is wave propagation, the results for the lowest velocity $V_{app} = 600$ m/sec gives 13 cm over the lateral piers and 10 cm at the centre.

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