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DESIGN ACTIONS FOR CONTINUOUS DECK BRIDGES CONSIDERING NON SYNCHRONOUS EARTHQUAKE MOTION

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ABSTRACT

This paper aims to develop and validate structural design criteria which account for the effects of earthquakes spatial variability. In past works (Nuti & Vanzi, 2004 & 2005; Carnevale et al., 2010) the two simplest forms of this problem were dealt with: differential displacements between two points belonging to the soil or to two single degree of freedom structures. Seismic action was defined according to Seismic Codes and structure was assumed as a linear elastic sdof oscillator.

Despite this problem may seem trivial, existing codes models appeared improvable on this aspect. For the differential displacements of two points on the ground, these results are now validated and generalized using response spectra of both EC8 and new Seismic Italian Code (Ministero Infrastrutture, 2008). The problem of statistically defining the differential displacement among any number of points on the ground (which is needed for continuous deck bridges) is approached. The model is used to compute the differential displacements of points on the ground, both for two and multiple points cases, and with different response spectra shapes. Preliminary results indicate that the design Codes can be strongly improved on this topic, both for the two points (e.g. simply supported decks) and the multiple points (e.g. continuous decks on multiple piers) cases.

The results, in terms of differential displacements, have further shown sensitivity to the spectral shape, an aspect which must be carefully investigated. So the earthquake spatial variability does appear to be a significant problem for failure modes governed by differential displacements, also for structures of minor importance like small bridges. Since its inclusion in the design phase brings about small or no extra cost for most situations, it is worth to stress the importance of a rapid Code update on this subject.

Keywords: Bridge design, Earthquake, Non synchronous motion, Support design, Random field, Probability

INTRODUCTION

Some different models defining the spatial variability of earthquakes have been developed in the last twenty years, departing from experimental observations of simultaneous recordings of earthquakes (Abrahamson et al., 1991; Oliveira et al., 1991).

From the classical work of Luco and Wong (Luco & Wong, 1986), different statistical descriptions have been proposed and fit to the experimental data (Vanmarcke & Fenton, 1991; Santa-Cruz et al. 2000), with varying degree of complexity and accuracy.

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The effects on structures have been also investigated, either in the linear field, with random vibration tools (Der Kiureghian & Neuenhofer, 1991 & 1992), or in the non linear one, via numerical simulations or equivalent linearization procedures (Monti et al., 1994 & 1996; Hao, 1998; Sextos et al., 2003).
The most important outcome of the studies could be appearing definitive and unambiguous: apart from a few cases, non synchronous action decreases the structural stresses with respect to the case with synchronous actions. There are however situations in which non-synchronism negatively influences structural behavior, e.g. deck unseating and some of the current design rules provided by the Codes appear improvable on this aspect. This topic was deeply discussed by the Authors above all considering Code provisions refinement in last years.
Departing from these observations on non-synchronism influence on structural response and considering the results of previous studies, this paper aims to validate structural design rules which account for the effects of earthquakes spatial variability: in particular two different Code provisions, according to Code changing in Italy, are considered and discussed.
In previous works (Nuti & Vanzi, 2004 & 2005) the two simplest forms of this problem were dealt with differential displacements between two points belonging to the soil or to two single degree of freedom structures. In these works seismic action was defined according to both EC8 [(Cen, 2002); at the time assumed in an original Italian Code (Presidenza del Consiglio dei Ministri, 2003)] and draft [at the time; now it was issued (Ministero Infrastrutture, 2008)] new Italian Code; the structures were assumed as linear elastic sdof oscillators.
In a recent paper (Carnevale et al., 2010) the previous results, in terms of differential displacements of two points, were validated and generalized using the newly developed response spectra contained in the new seismic Italian Code. Furthermore the problem of statistically defining the differential displacement among any number of points (which is needed for continuous deck bridges) is approached too and some preliminary results will be shown in this paper.
The results of these approaches are univocal and different at the same time: current Codes (both EC8 and Italian Code 2008) may be improved on this aspect yet the Italian Code is more efficient.

THEORETICAL APPROACH

For the sake of completeness, a short summary of the model is presented herein; obviously readers are however referred to the works of Nuti and Vanzi (Nuti & Vanzi, 2004; Nuti & Vanzi, 2005) for a more detailed presentation of mathematical aspects.
An earthquake acceleration recording at point P in space can be represented via its Fourier expansion as a sum of sinusoids (Vanmarcke & Fenton, 1991):

$$A_P(t) = \sum_k \left[B_{Pk} \cdot \cos(\omega_k \cdot t) + C_{Pk} \cdot \sin(\omega_k \cdot t)\right] \quad (1)$$

In equation (1), $A_P(t)$ is the measured acceleration in point P at time t, k is an index varying from 1 to the number of circular frequencies ω_k considered, B_{Pk} and C_{Pk} are the amplitudes of the k^{th} cosine and sine functions. Assuming that the acceleration $A_P(t)$ is produced by a wave, in the ground, moving with velocity V it is possible to define the acceleration in any point of the surrounding space.
Considering a different point in space, say Q, at distance X_{PQ} from P, in this point Q, at time t, the earthquake acceleration, depending on time delay τ_{PQ} of the signal, could be defined as:

$$A_Q(t) = \sum_k \left\{B_{Qk} \cdot \cos\left[\omega_k \cdot (t - \tau_{PQ})\right] + C_{Qk} \cdot \sin\left[\omega_k \cdot (t - \tau_{PQ})\right]\right\} \quad (2)$$

$$\tau_{PQ} = \frac{X_{PQ}}{V} = X_{PQ} \cdot \left(\frac{\cos(\psi)}{v_{app}} \right) \tag{3}$$

In equation (3) ψ is the angle between the vector of surface wave propagation and the vector that goes from P to Q and v_{app} is the surface wave velocity.
Equation (1) and equation (2) are equal and acceleration amplitude depends on coefficients of Fourier expansion of sinusoids sum, in particular the amplitudes B_{Qk} and C_{Qk} would be respectively equal to B_{Pk} and C_{Pk} if the medium through which the waves travel did not distort them.
But it isn't the case of a real medium; in this case B_{Pk} is correlated with B_{Qk} and C_{Pk} is correlated with C_{Qk} while the B's and C's are independent. I.e. the amplitudes B_{Pk} and C_{Qk} are statistically independent, for any points P and Q, and any circular frequency ω_k, with the only exception of B_{Pk} and B_{Qk} i.e. same circular frequency but different points in space. The same holds for C_{Pk} and C_{Qk}.
In order to simplify the approach, some hypothesis could be done: in particular the amplitudes are assumed normally distributed with zero mean and this assumption is experimentally verified. With this assumption, in order to quantify the acceleration time histories in different points in space, equations (1) ÷ (3), all it is needed is the definition of the correlation between amplitudes and of their dispersion, as measured by the variance or, equivalently, of the covariance matrix of the amplitudes.
The covariance matrix $\mathbf{\Sigma}$ of the amplitudes B and C is assembled via independent definition, at each circular frequency ω, of its diagonal terms (the variances in each space point and frequency) and of the correlation coefficients. The diagonal terms Σ_{PP} are quantified via a power spectrum; a traditional choice is the Kanai-Tajimi power spectrum, modified by Clough and Penzien (Clough & Penzien, 1975):

$$\Sigma_{PP} = G_{PP}(\omega) \cdot d\omega \tag{4}$$

$$G_{PP}(\omega) = G_0 \cdot \frac{\omega_f^4 + 4 \cdot \beta_f^2 \cdot \omega_f^4 \cdot \omega^2}{\left(\omega_f^2 - \omega^2\right)^2 + 4 \cdot \beta_f^2 \cdot \omega_f^4 \cdot \omega^2} \cdot \frac{\omega^4}{\left(\omega_g^2 - \omega^2\right)^2 + 4 \cdot \beta_g^2 \cdot \omega_g^4 \cdot \omega^2} \tag{5}$$

where its parameters are the scale factor G_0, the central frequencies of the filters, ω_f and ω_g, and their damping, β_f and β_g (see details in Nuti & Vanzi, 2004).
The Kanai-Tajimi power spectrum was adopted in the previous paper and the correlation coefficient between the amplitudes was expressed via the coherency function:

$$\rho = \exp\left(-\omega^2 \cdot X^2 \cdot \left(\frac{\alpha}{v} \right)^2 \right) \tag{6}$$

using the form originally proposed by Uscinski (Uscinski, 1977) on theoretical grounds and Luco (Luco & Mita, 1987). The correlation decreases with increasing distance X and circular frequency ω and increases with increasing soil mechanical and geometric properties as measured by the ratio v/α where α is the incoherence parameter, v the shear wave velocity.
The incoherence parameter α is the most difficult aspect in the coherency function assessment. For a more detailed discussion the reader is referred to previous paper (Nuti & Vanzi, 2004); however, values in a range as wide as 0.02÷0.50 are reported in past experimental studies.

Departing from the above earthquake spatial model, using random vibration concepts, it may be shown that the distribution of the maximum differential displacement can be found with the peak factor formulation (Vanmarcke et al., 1999), by setting:

$$Z^*_{s,p} = \sigma_{Z^*} \cdot r_{s,p} \tag{7}$$

where $Z^*_{s,p}$ is the displacement value which is not exceeded with probability p during an earthquake of duration s, and σ_{Z^*} is the standard deviation of Z^*. Typical values of the peak factor $r_{s,p}$ lie within 1.20÷3.50 range; $r_{s,p}$ is computed as set out in (Vanmarcke et al., 1999), in which proper account is taken for the non-stationarity of the response via the use of the equivalent damping.

DIFFERENTIAL DISPLACEMENTS BETWEEN TWO POINTS ON THE SOIL: CODE PROVISIONS VS. PREVIOUS AND CURRENT FINDINGS

In this chapter a comparison is made between some of the Code provisions and the findings of past and new analyses by the Authors. Only the case of differential displacement between two points on the ground is considered. In more detail, the Codes considered are:

- the European Seismic Code EC8 (Cen, 2002), partially adopted by the Italian Seismic Code of 2003 (Presidenza del Consiglio dei Ministri, 2003); this Code will be referred to with EC8/ICPC, meaning EuroCode 8 / Italian Civil Protection Code.
- the new Italian Seismic Code (Ministero Infrastrutture, 2008). This Code will be referred to as ICB, meaning Italian Code for Bridges. This Code, for non synchronism, has been drafted following also the results of Authors previous works (Nuti & Vanzi, 2005)

The analyses presented are:

- a summary of the results obtained by the Authors using EC8/ICPC response spectra with soils type A, B, D (respectively rock, stiff soil, loose soil)
- some results obtained using the ICB for soil types A and B (corresponding to EC8 soil types A & B).

For both Codes, reference is made to the ultimate limit state.
For this limit state, the Codes state the ground differential displacements be computed as:

$$\begin{cases} u^{I}_{PQ} = X_{PQ} \cdot pga \cdot \dfrac{T_C}{v_{app}} \cdot \left(\varepsilon \cdot \dfrac{1}{2 \cdot \pi} \right) \le u^{I\ MAX}_{PQ} \\ u^{I\ MAX}_{PQ} = 0.025 \cdot pga \cdot \sqrt{\left(\varepsilon_P \cdot T_{PC} \cdot T_{PD}\right)^2 + \left(\varepsilon_Q \cdot T_{QC} \cdot T_{QD}\right)^2} \end{cases} \qquad EC8/ICPC \tag{8}$$

$$\begin{cases} u^{II}_{PQ} = u^{II\ MIN}_{PQ} + \left(u^{II\ MAX}_{PQ} - u^{II\ MIN}_{PQ} \right) \cdot \left\{ 1 - \exp\left[-1.25 \cdot \left(X_{PQ}/v \right)^{0.70} \right] \right\} \\ u^{II\ MIN}_{PQ} = 1.25 \cdot 0.025 \cdot pga \cdot \left| \left(\varepsilon_P \cdot T_{PC} \cdot T_{PD}\right) - \left(\varepsilon_Q \cdot T_{QC} \cdot T_{QD}\right) \right| \\ u^{II\ MAX}_{PQ} = 1.25 \cdot 0.025 \cdot pga \cdot \sqrt{\left(\varepsilon_P \cdot T_{PC} \cdot T_{PD}\right)^2 + \left(\varepsilon_Q \cdot T_{QC} \cdot T_{QD}\right)^2} \end{cases} \qquad ICB \tag{9}$$

with X_{PQ} distance between points P and Q, ε_P and ε_Q soil coefficients in P and Q, pga peak ground acceleration, $\{T_{PC};T_{PD}\}$ and $\{T_{QC};T_{QD}\}$ periods defining the response spectra in P and Q, v_{app} surface and v shear wave velocities.

In all the analyses, the most severe condition for non synchronism, i.e. highest uncorrelation, has been studied; therefore the incoherence parameter, in equation (6), has been taken as $\alpha = 0.50$.
In Figure 1 the response spectra of EC8/ICPC and the ICB are shown. A few words, compatibly with the sake of brevity and space, about the ICB spectra are convenient. The spectra, obviously, are defined by nearly the same relationships as the EC8, with three important exceptions: the maximum spectral acceleration amplification is soil and site dependent, the periods defining each interval of the spectrum (T_B & T_C, lower and upper corner limits of constant spectral acceleration branch and T_D corner limit between constant velocity and constant displacement ranges) depend on the soil type and on the maximum site spectral velocity and finally topographic effects are explicitly accounted for.

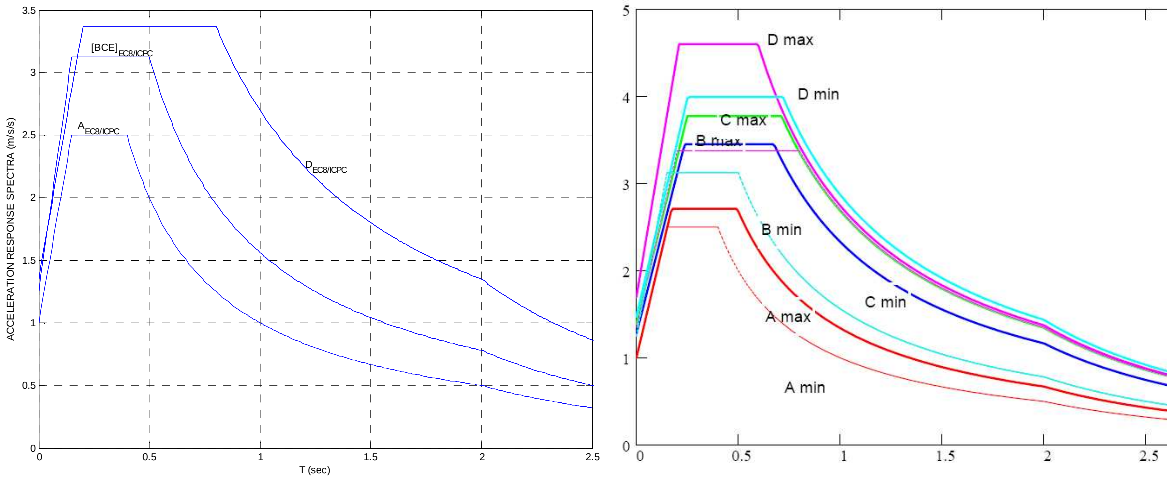

Figure 1. Acceleration response spectra of EC8 (left) and the ICB (right); $pga = 0.10g$. The $_{min}$ and $_{max}$ suffixes in the ICB spectra are relative to minimum and maximum topographic effects

In order to make a comparison between the model results obtained with the EC8/ICPC spectra, and those of the ICB, the above dependencies have been drastically simplified: the minimum value of the topographic effect (i.e. multiplicative parameter for topography = 1) has been adopted. Further, the maximum spectral velocity and maximum spectral acceleration amplification have been assumed constant and equal to the median values computed by Newmark and Hall (Newmark & Hall, 1982) for rock soil. These values are: PGV/PGA=0.91 [(m/sec)/g]; maximum spectral acceleration amplification equal to 2.12; maximum spectral velocity equal to 1.65 PGV. With these hypotheses, the ICB spectra (pictured in Figure 1, right) depend only on the ground type and the peak ground acceleration.
The first result shown is in Figure 2 (left). The figure shows the comparison between the soil differential displacements of EC8 versus those computed using the above discussed model, equation (7). Notice that the results coming from the analysis shortly described have been cast in the form expressed by equations (9) for inclusion in the ICB. Examining Figure 2 (left), one can see that the maxima differential displacements computed with EC8 and this model differ by about 1.25; further, the trend is very different. EC8/ ICPC increases linearly up to the maximum, the analyses results (and the ICB prescriptions too) grow in a parabolic fashion. In the range of distances where most civil engineering structures are, between 5 and 100 m, from buildings columns to long bridges piers, the differences are large: at 20 m distance, EC8/ICPC gives 2 mm or less while ICB forecasts differential displacements from 2 mm to about 40 mm, depending on the soil coupling. The relative displacements computed with the ICB spectra for soil B, and with the Newmark and Hall simplification described before (in terms of maximum spectral velocity and maximum spectral acceleration amplification), are next shown in Figure 2, right. From Figure 2, one can notice that the increase of differential displacement with the distance is the same (the abscissa of Figure 2, left, are in natural scale; that of Figure 2, right, in logarithmic scale).

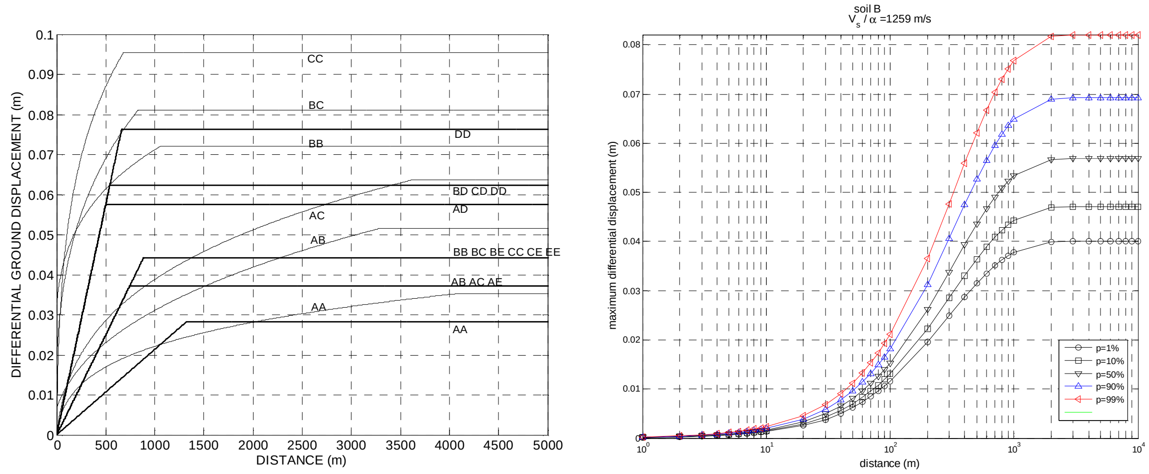

Figure 2. Left: soil differential displacements; thicker lines for EC8/ICPC; remaining lines for theoretical model. Right: differential displacement on soil type B for ICB spectra $pga = 0.10g$

The maximum values of the differential displacements appear to indicate dependence on the spectral shape: with the B soil type, the maximum (at high distance) differential displacement is equal to 72 mm, 58 mm and 83 mm respectively for EC8 and ICB.
These results indicate that there is indeed a dependence of the differential displacements on the spectral shape, although it must be investigated which part of the spectra this is due to.

DIFFERENTIAL DISPLACEMENTS BETWEEN ALIGNED POINTS ON THE SOIL: CODE PROVISIONS VS. PREVIOUS AND CURRENT FINDINGS

Bridges on multiple supports must be checked for spatial variability of seismic action. According to EC8 two different sets have to be considered; the first consists of relative displacements applied simultaneously with the same sign to all supports of the bridge in a considered horizontal direction.
The second considers the case of ground displacements occurring in opposite directions at adjacent piers; for this latter case the displacement set, occurring at the base of the piers, is pictured in Figure 3.

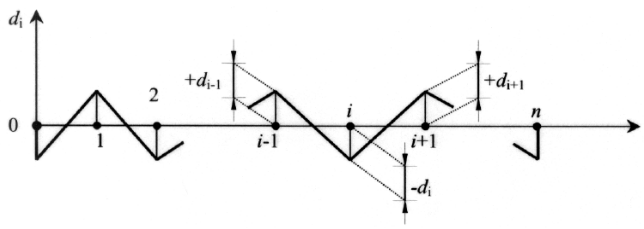

Figure 3. Displacement set for verification of multiple support bridges for displacements occurring in opposite directions (EC8)

The displacement set consists in opposite direction displacements of the same value $d_i = \pm \Delta d_i / 2$; the relative displacement between two adjacent piers equals the maximum differential displacement $u_{PQ}^{II\ MAX}$ (see equation (9)) times the ratio between the average piers distance $L_{i,av} = (L_{i,i+1} + L_{i,i-1})/2$ and the distance beyond which ground motion may be considered uncorrelated, L_g, ranging from 600 m (soil A) to 300 m (soil D).

$$\Delta d_i = \beta_r u_{PQ}^{II} {}^{MAX} \frac{L_{i,av}}{L_g} \tag{10}$$

In equation (10) β_r is a factor accounting for the magnitude of ground displacements occurring in opposite directions at adjacent supports; for different ground types this factor could be assumed as $\beta_r = 1$, this assumption is made in this paper for each type of ground.
For example, on soil type D, with average piers distance $L_{i,av} = 30$ m, $pga = 0.10g$, the maximum differential displacement is equal to 78 mm (see Figure 2 left), so the relative displacement can be calculated as $\Delta d_i = \beta_r u_{PQ}^{II} {}^{MAX} \frac{L_{i,av}}{L_g} = 1.00 \cdot 78 \cdot \frac{30}{300} = 7.80$ mm.
This rule appears unconservative on one side (i.e. 7.8 mm appears too small a value) and far too conservative on the other (the probability that all the piers are displaced in opposite directions by the same amount is zero, obviously from an engineering view – point).
Some preliminary analyses have then been carried out via Montecarlo sampling of the earthquakes generated with the model shortly described in the previous chapter. Three soil types, A, B and D, as defined by EC8, have been assumed; the peak ground acceleration has been taken $pga = 0.10g$ while different piers distances are considered. The results of this analysis are shown in Figure 4 for soil type D.

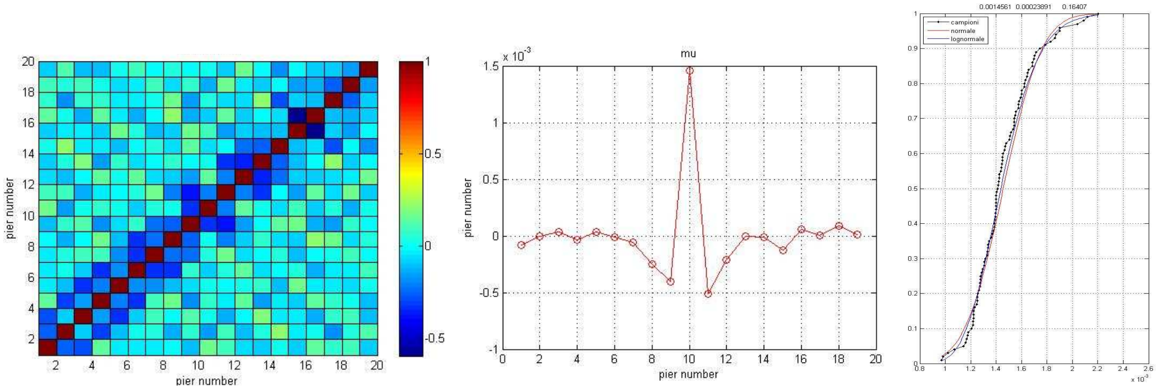

Figure 4. Statistics of soil curvatures and displacements for 21 piers at 20 m distance. 30 earthquake samples; soil D of EC8; $pga = 0.10g$. Left: correlation; middle: mean value; right: cumulative distribution function

The statistics of soil curvatures, sampled at the base of the piers, show negative correlation (equal or higher than -0.5) between adjacent piers and no significant correlation thereafter (Figure 4, left). The statistics of curvatures may be therefore easily computed since those for two adjacent piers suffice to define the entire curvature field. The mean value (across the earthquake samples) of the maxima of curvatures is shown as the middle figure in Figure 4. The maxima are equal to $1.5 \cdot 10^{-3}$ (soil D) while lower values (not shown here) are $1.5 \cdot 10^{-4}$ (soil A, ten times lower) and $3.0 \cdot 10^{-4}$ (soil B, five times lower).
The cumulative distribution function of the maxima of curvatures is finally shown as the right figure in Figure 4. Three curves are plotted: the sampled cdf and the normal (red) and lognormal (continuous blue one) interpolation. One can see that both approximation work rather well and that the three curves are rather undistinguishable.
Taking for simplicity the normal approximation as the reference one, the coefficient of variation of curvatures is approximately equal to 0.20 for all soil types; more precisely, it is equal to 0.20 (soil A), 0.22 (soil B), 0.16 (soil D). Hence, it appears reasonably simple to both define the mean values of the maxima of curvatures and the cdf of the maxima, for all soil types tested.

One may sum up the obtained results as follows:

- the statistics of soil curvatures, sampled at the base of the piers, show negative correlation (about -0.5) between adjacent piers and no significant correlation thereafter,
- the statistics of curvatures should therefore be easy to compute since those for two adjacent piers suffice to define the entire curvature field,
- design should be done with the following soil relative displacements:
  - in i : $d_i = u_{PQ}(X_{PQ})$ (see equation (9))
  - in $i-1$ and $i+1$: $d_{i-1} = d_{i+1} = u_{PQ}/2$
  - elsewhere: 0
- the above values are the mean, across the earthquake sample, of maxima. The distribution of maxima can be modelled as a normal random variable with 0.20 c.o.v..

As an example, in Figure 5 a comparison is made between the mean value of the design differential displacement (at the surface) of EC8 (top, $-d_i = d_{i+1} = 7.8$ mm from equation (10)) and herein proposed (bottom, $-d_i = 2 \cdot d_{i-1} = 2 \cdot d_{i+1} = 28$ mm). So the model provision in terms of design differential displacement (on ground surface) is 3.6 times higher than EC8 statement while the unconvincing hypothesis of a complete set of displacements in alternate opposite directions is over passed.
This result is a general one, for instance with a pier distance of 32 m, in soft soil, and $pga = 0.40g$, the estimation of soil differential displacements is 14 mm for EC8/ICPC while the model prediction is instead 112 mm (exactly 8 times greater, consistent with ICB (Nuti & Vanzi, 2005).

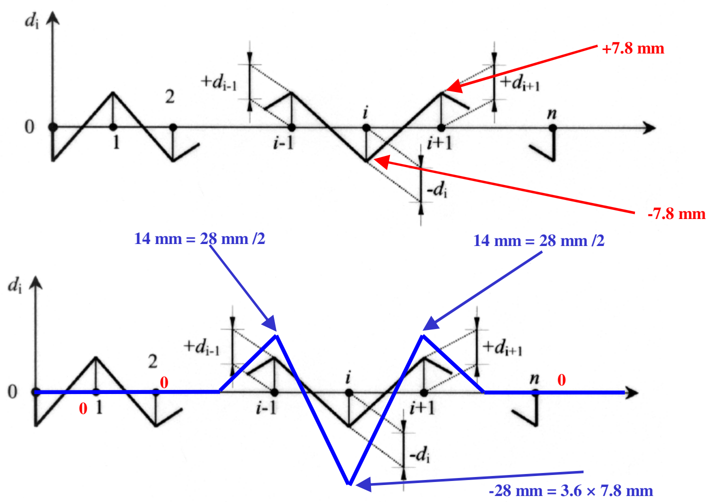

Figure 5. Comparison between mean value of the design differential displacements (at surface) of EC8 (top) and of proposed model (bottom). Soil type D, piers distance 30 m, $pga = 0.10g$

STRUCTURAL RESPONSE OF CONTINUOUS DECK BRIDGES: PRELIMINARY RESULTS

The analyses to assess the structural response of continuous deck bridges is currently in progress in the ambit of the National Research Program dealing with “Bridges under non synchronous earthquakes: modelling, analysis and synthesis of the results” that was funded by the Italian Instruction Ministry (Nuti, 2010). Some preliminary results of this activity are shown in a previous paper (Carnelvale et al., 2010) and will be summarized in this paper. The analyses documented in this paper are elastic ones, and the bridges considered have six identical piers.

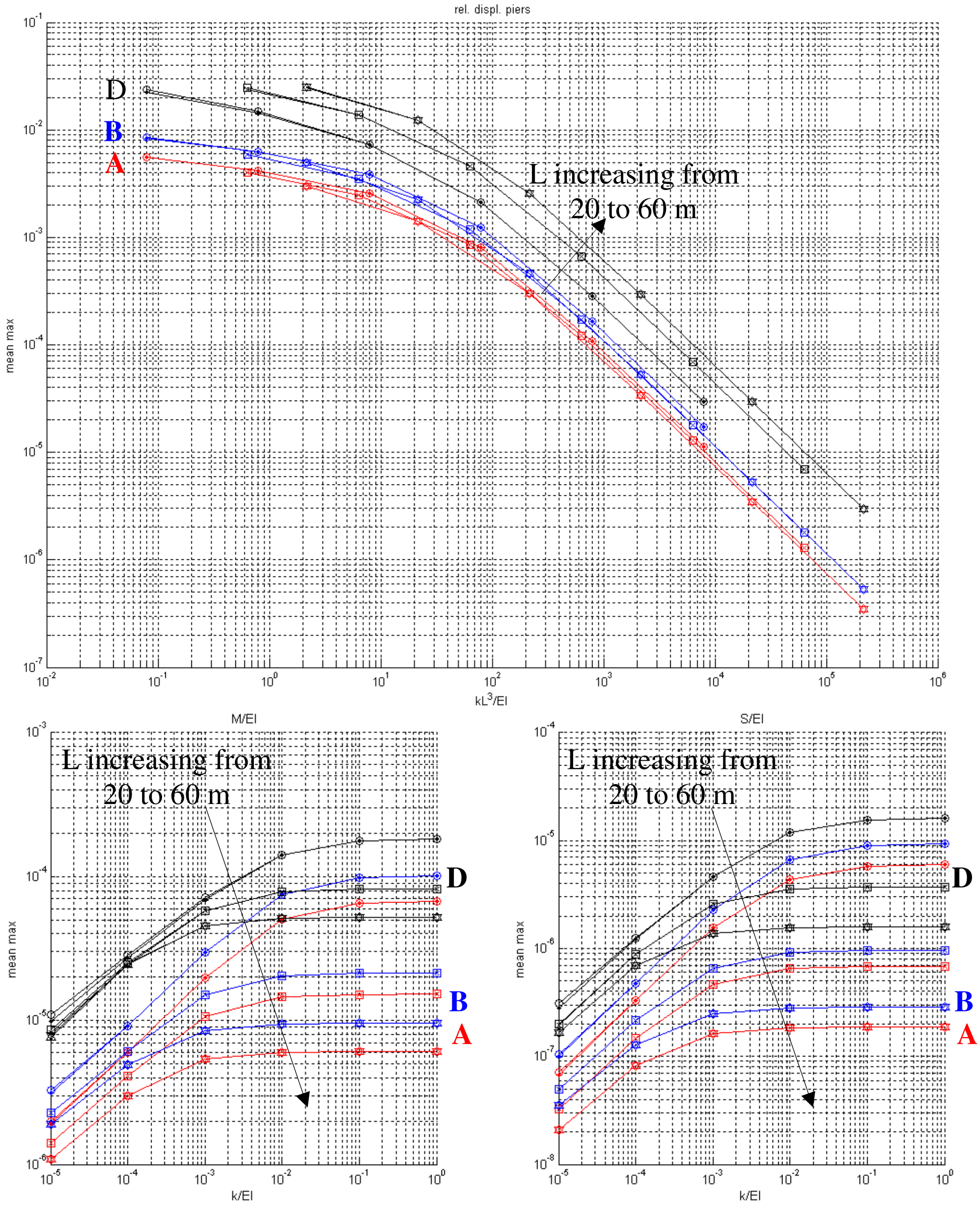

Figure 6. Top: pier differential displacements as a function of KL^3/EI. Bottom: bending moment and shear force in the deck as a function of K/EI. $pga = 0.10g$

The analyses were divided in two phases. Firstly we have done some static analyses of the bridges with the non synchronous signals, in order to assess the main controlling variables; after elastic dynamic analyses were carried out, in order to control the structural response

In particular rhese latter have the aim of assessing the correctness of designing [as is currently done, equation (11)] for non synchronism via summation of the effects of fixed base response spectrum analyses plus static superimposed displacements at the piers base.

$$Z_M(t) = Z_P(t) + Z_{MP}(t) \tag{11}$$

where the total displacement in M , $Z_M(t)$ is the sum of the ground displacement $Z_P(t)$ and of the s.d.o.f. system displacement with respect to the ground $Z_{MP}(t)$.
The response variables considered is one for the piers (the maximum top drift, denoted by δ) and two for the deck (the maximum bending moment and maximum shear, denoted by M and S). All the results discussed in what follows are the mean values of the response variables.
Notice that we have consistently found the coefficient of variation to be between 0.1 and 0.2.
Besides, we denote the horizontal pier stiffness, deck flexural stiffness and length between two piers respectively with K , EI , L .
First, the static analyses are discussed.
Theoretically, it can be shown that maximum top drift of a pier depends on both pier stiffness and ratio between deck flexural stiffness and length between two piers; i.e. $\delta = \delta\left(KL^3 / EI\right)$.
With 50 non synchronous earthquakes, sampled on soil types A, B, D, considering L varying between 20 and 60 m, K / EI varying between 10^{-5} and 10^{0} (in MKS units), we have assessed the influence on the mean responses of K / EI and L .
The results are shown in Figure 6 top and appear to prove the theoretical dependencies $\delta\left(KL^3 / EI\right)$ i.e. that the pier drift depends solely on this stiffness ratio.
The same dependency can be stated regarding to maximum bending moment of the deck $M / EI = (K / EI, L)$ and maximum shear of the deck $S / EI = (K / EI, L)$; these results are shown in Figure 6 bottom and seem to prove the dependency of deck curvature on K / EI and L .
Figure 6 diagrams give the correlations between the response and the input variables and they have permitted to calibrate the variables to investigate in the dynamic analyses.
The dynamic analyses are discussed in detail in (Carnevale et al. 2010).
In this case the same input variables as the static analyses are used with the exception of length L , taken equal to 60 m, and the soil type, assumed of type A.
The aim of those analyses was to check whether the dynamic bridge response (in terms of δ , M / EI , S / EI) to non synchronous earthquakes could be computed as the sum of a fixed base response spectrum analyses plus static superimposed displacements at the piers base.
For these ones the displacements of the previous analyses (Figure 5, bottom) was used while for structural analyses, the OPENSEES software has been chosen for its numerical robustness and diffusion in structural engineering community.
The preliminary results are discussed in (Carnevale et al., 2010), in particular in terms of the correlation between the assumed K / EI and the period of the first bridge mode. The maximum shear and bending moment (for the deck), and maximum pier drift are depicted as a function of the bridge first natural period. For sake of simplicity and in order to control design procedure, all quantities are adimensionalised to the target response, i.e. the one computed with the dynamic non synchronous analyses, and expressed in percentage.
For analysis result controlling it was considered the following values: *target* is the result of dynamic non synchronous analyses; *response spectrum* indicates the response spectra analyses; *superimposed displacements* the static analyses with the displacements of proposed model (see Figure 5, bottom) and *displacement sum* that is the sum of *response spectrum* and *superimposed displacements* [see for completeness (Carnevale et al. 2010)].
From the results it appears that, for the deck response variables, *response spectrum* values underestimate substantially the *target* results, while the *displacement sum* proves better.
On the contrary for the pier response variable, *response spectrum* analyses generally overestimate the *target* results, and so do the *displacement sum* results.
So, it generally appears that further research is necessary in order to define a simple and accurate design rule, above all regarding structural analysis; but, however, what is currently recommended in EC8 (see

Figure 5, upper part) with ground displacements occurring in opposite directions at adjacent piers is certainly improvable.
On this basis it is possible to state that the new Italian Seismic Code for Bridge (ICB, that was drafted following the present model) could be considered more efficient than EC8 provisions.

CONCLUSIONS

Based on well known expressions for spatial variability of seismic motion, a theoretical model founded on basic random vibration theory, has been developed in (Nuti & Vanzi, 2004; Nuti & Vanzi, 2005) and some preliminary results of structural analyses (static and dynamic9 based on this model are shown in a previous paper (Carnelvale et al., 2010).
The model is here used to compute the differential displacements of points on the grounds, both for two and multiple points cases, considering both different code provisions (EC8 and new Italian Seismic Code) and contiguous different soils.
Preliminary results indicate that the design codes can be strongly improved on this topic, both for the two points (e.g. simply supported decks) and the multiple points (e.g. continuous decks on multiple piers) cases. In fact, with the exception of the Italian Code for Bridges, the codes seem improvable on this aspect, both from the qualitative and quantitative viewpoint.
In particular for differential displacement for bridges, Eurocode 8 appears inaccurate and unconservative above all in the range of distances where most civil engineering structures are, below 100 m.
Considering structural influence of ground displacement spatial distribution; it appears that, for the deck response variables, response spectrum values underestimate substantially the target results, while the displacement sum proves better.
On the contrary for the pier response variable, response spectrum analyses generally overestimate the target results, and so do the displacement sum results.
This topic is, in the authors' opinion, at best difficult and questionable. Significant more research effort is needed on this point. However at the moment it is possible to state that the new Italian Seismic Code for Bridge (ICB, that was drafted following the present model) could be considered more efficient than EC8 provisions.
As a final remark, it is highlighted that earthquake spatial variability does appear to be a significant problem for failure modes governed by differential displacements, also for structures of minor importance like small bridges.
Since its inclusion in the design phase brings about small or no extra cost for most situations, it is worth to stress the importance of a rapid code update on this subject.

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