# Assessment of Seismic Behavior of R.C. Bridges under Asynchronous Motion and Comparison with Simplified Approaches

**L. Carnevale, S. Imperatore, D. Lavorato, C. Nuti**
*Department of Structures "Roma Tre"University, Rome*

**G. Leoni**
*University of Camerino - Camerino – Italy*

**G. Tropeano**
*University Federico 2° - Naples - Italy*

**SUMMARY:**
The seismic motion is characterized by temporal and spatial variability, that become significant in structures with long longitudinal development, such bridges and viaducts. In those cases, the differential displacements of the supports may have adverse effects, either on the piers responses either on the deck deformations.
In this paper, asynchronous and synchronous dynamic analyses on continuous-span bridges are carried out using the finite element method. To assess when considering the synchronous action becomes disadvantageous, a parametric analysis, varying the geometry of the bridge and the soil foundation characteristics, is conducted. The bridges are subjected to sets of artificial accelerograms generated on the base of previous study of the authors (Nuti and Vanzi, 2005). The study is part of a research project, funded by Italian Ministry for Research, that deals the response of bridges subjected to asynchronous motion, including local site amplification.

*Keywords: Continuous deck bridge, Asynchronous motion, Seismic assessment, FEM Analysis.*

## 1. INTRODUCTION

The spatial variability of earthquake ground motion has been recorded in various arrays (SMART-1 and LSST-Lotung in Taiwan, Chiba in Tokyo, USGS-Parkfield and Imperial Valley in California). The phenomenon depends on the waves velocity, coherency loss and the local site effects (Der Kiureghian and Neuenhofer, 1992).

The first studies on the effect of asynchronism on simple bridge were conducted in the '90. More realistic configurations were studied subsequently, both numerically (Sextos et al., 2003; Lupoi et al., 2005; Nuti and Vanzi, 2005; Sextos and Kappos, 2009) both experimentally (Pinto et al., 2002; Norman et al., 2006). In most of these studies, the potentially detrimental consequence of asynchronous motion emerges. As result, the need of research on this issue is widely pointed out in the recent state-of-the-art documents (fib 2007).

The main question, obviously, is understanding when the asynchronous response of the structure would be detrimental compared to the prediction made assuming an uniformly excited structure.
To fill this gap, a Research PRIN 2008 project was founded by the Italian Ministry on the evaluation of the response of bridges subjected to asynchronous motion, including local site amplification. The goal of this work is to assess when the asynchronous response becomes disadvantageous. This evaluation is made, preliminary, on continuous-span bridges with piers of equal spacing and geometry. A parametric analysis, varying the geometry of the bridge (therefore the period of the bridge) and the soil foundation characteristics, is conducted. The artificial accelerograms imposed on the bridges, are determined with the procedure defined by some of the authors (Nuti and Vanzi, 2005).

The bridge response is evaluated by means of linear and non-linear analyses. The linear analysis has permitted a comparison either to the code provisions either to the alternative formulations proposed by some of the authors (Nuti and Vanzi, 2009). The non-linear analyses, instead, has been conducted to evaluate when the effects of the asynchronous actions become significant, also in terms of failure mechanisms, forces redistribution and plastic hinge development.

## 2. SPATIALLY VARYING GROUND MOTION MODEL

The model of generation of non-synchronous accelerograms is based on random dynamic theory. Then, the seismic acceleration measured at a point P in space may be represented by its Fourier spectrum:

$$A_P(t) = \sum_k \left[ B_{Pk} \cos(\omega_k t) + C_{Pk} \sin(\omega_k t) \right] \tag{1.1}$$

Where $A_P(t)$ is the value at the point P at time t, k is an index ranging from 1 to $\omega_k$ (the circular frequency), $B_{PK}$ and $C_{PK}$ are the amplitudes of the k-th cosine and sine functions.
Assuming that the wave travels with velocity V through a medium that transmits the signal without distortion, the acceleration at a point Q at a distance $X_{PQ}$ from P is:

$$A_Q(t) = \sum_k \left[ B_{Qk} \cos(\omega_k (t - \tau_{PQ})) + C_{Qk} \sin(\omega_k (t - \tau_{PQ})) \right] \tag{1.2}$$

With:

$$\tau_{PQ} = \frac{X_{PQ}}{V} = \left( \frac{X_{PQ}}{v_{app} \cdot \sin(\psi)} \right) \tag{1.3}$$

where $\psi$ is the wave propagation angle, $\tau_{PQ}$ is the signal delay and $v_{App}$ is the wave velocity. The accelerograms in different points of the space, defined in equations (1.1) and (1.2), are computed by means the relationships between the amplitudes; their dispersion is defined through the covariance matrix, assembled by means of the coherence function $\rho$ (Uscinski, 1977):

$$\rho = \exp\left( \omega^2 \cdot X_{PQ}^2 \cdot \left( \frac{\alpha}{v} \right)^2 \right) \tag{1.4}$$

The correlation depends by the soil characteristics, by means the $v/\alpha$ ratio (where $v$ is the shear wave velocity and $\alpha$ is a coherence parameter, which vary in the range 0.02-0.5).

## 3. PARAMETRIC INVESTIGATION OF CONTINUOUS DECK BRIDGES

The effects of the asynchronous motion are evaluated by mean of a parametric investigation on a bridge characterized by a continuous deck. The bridges are subjected to linear and non-linear dynamic analyses. The non-linear FEM models are fully compatible with linear ones.

**Table 1:** Geometrical characteristics of bridges: Sec bridge deck equivalent section width x height (rectangular); L span length; H pier height, D pier diameter; $I_{deck}$ inertia moment deck, $I_{pier}$ inertia moment pier section.

| **Bridge** | **Deck** | | **Piers** | | **Ratio $I_{pier}/EI_{deck}$** | **Period** |
|---|---|---|---|---|---|---|
| tag | L [m] | Sec [m] | H [m] | D [m] | $\beta$ [1/m$^3$] | T [s] |
| 1 | 40 | 3x0.4 | 6m | 2m | 1.88E-02 | 0.197 |
| 2 | 40 | 10x0.5 | 15m | 5m | 1.07E-03 | 0.274 |
| 3 | 40 | 20x0.6 | 23m | 7m | 1.13E-04 | 0.410 |
| 4 | 40 | 10x0.5 | 26m | 3m | 1.07E -04 | 0.739 |
| 5 | 40 | 20x0.6 | 24m | 4m | 1.06E-05 | 1.098 |
| 6 | 40 | 10x0.5 | 20m | 2m | 1.09E-05 | 2.140 |

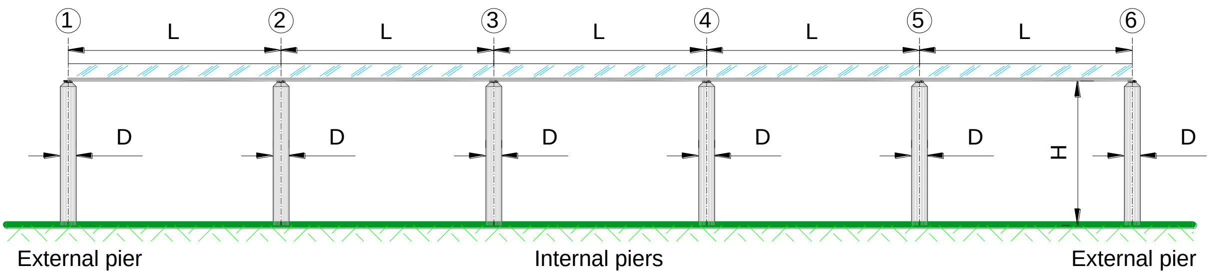



**Figure 1.** R.C. bridges analyzed in the parametric investigation

The analyses are carried out on six r.c bridges (table 1), each characterized by six circular piers spacing 40 m and a different ratio between the pier and the deck stiffness (β), that varies between $10^{-5}$ and 1. The bridge piers have slenderness in the range of 2-15; the bridge decks, modelled with a continuous rectangular equivalent section, are representative of one, three or six lanes.

The reinforcement assigned to the piers is shown in table 2. Since the deck does not yield during the analysis, it is assumed elastic without reinforcement during the non-linear analyses.

**Table 2:** Reinforcement assigned to the piers for the non-linear analyses. $A_{s,long}$ longitudinal reinforcement, D pier diameter, r geometric reinforcement ratio, $A_{s,tras}$ transversal reinforcement.

| | | | **External pier** | | | **Internal pier** | | |
|---|---|---|---|---|---|---|---|---|
| **Soil** | **Bridge** | **D** [m] | $A_{s,\,long}$ | ρ [%] | $A_{s,\,trans}$ | $A_{s,\,long}$ | ρ [%] | $A_{s,\,trans}$ |
| **A, B** | 1 | 2 | 62 ø 26/10' | 1.0 | ø 10/10’ | 62 ø 26/10' | 1.0 | ø 10/10’ |
| | 2 | 5 | 306 ø 30/5' | 1.0 | ø 10/10’ | 306 ø 30/5' | 1.0 | ø 10/10’ |
| | 3 | 7 | 218 ø 24/10' | 1.0 | ø 10/10’ | 218 ø 24/10' | 1.0 | ø 10/10’ |
| | 4 | 3 | 92 ø 32/10' | 1.0 | ø 10/10’ | 92 ø 32/10' | 1.0 | ø 10/10’ |
| | 5 | 4 | 244 ø 26/5' | 1.0 | ø 10/10’ | 244 ø 26/5' | 1.0 | ø 10/10’ |
| | 6 | 2 | 62 ø 26/10' | 1.0 | ø 10/10’ | 62 ø 26/10' | 1.0 | ø 10/10’ |
| **D** | 1 | 2 | 62 ø 26/10' | 1.0 | ø 10/10’ | 62 ø 26/10' | 1.0 | ø 10/10’ |
| | 2 | 5 | 306 ø 30/5' | 1.0 | ø 10/10’ | 306 ø 30/5' | 1.0 | ø 10/10’ |
| | 3 | 7 | 218 ø 24/10' | 1.0 | ø 10/10’ | 218 ø 24/10' | 1.0 | ø 10/10’ |
| | 4 | 3 | 180 ø 28/5' | 1.5 | ø 10/10’ | 180 ø 28/5' | 1.5 | ø 10/10’ |
| | 5 | 4 | 302 ø 30/3' | 2.0 | ø 10/10’ | 302 ø 30/3' | 2.0 | ø 10/10’ |
| | 6 | 2 | 118 ø 28/5' | 3.0 | ø 10/10’ | 118 ø 32/5' | 3.0 | ø 10/10’ |

### 3.1. Asynchronous seismic Input

Each pier was subjected to different asynchronous displacement histories, evaluated starting by the EC8 response spectrum for soil A-B-D (fig. 2): starting from the theoretical approach shortly illustrated into the previous section, from a single sample, chosen utilizing the conventional selection techniques (Der Kiureghian, Neuenhofer, 1992), the other displacement histories was calculated by means the signals correlations. The sample depends on the soil morphology and the piers interspaces.

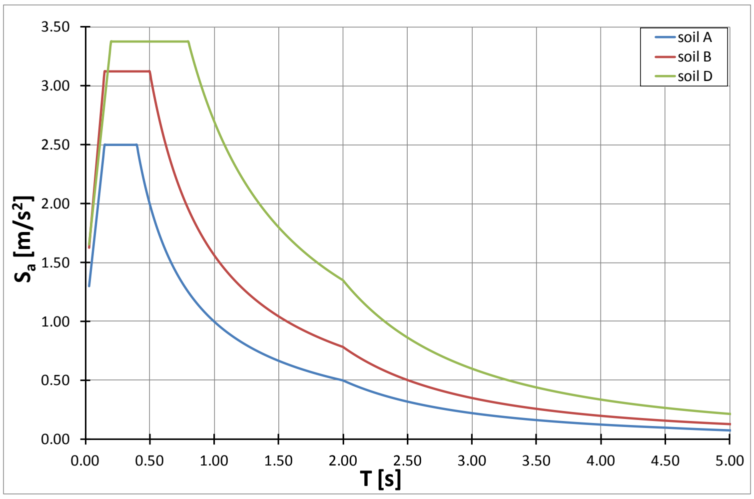



**Figure 2.** EC8 response spectrum for soil A-B-D used to define the asynchronous displacement histories

On each bridge different arrays of displacements histories (one for each pier), defined for the three soil types chosen, are imposed. In particular, the linear analyses were performed considering 50 different arrays of signals for each ground type. Given the high computational effort, the non-linear analysis were carried out using 10 events for the soil A, 5 for the soil B and 3 for the soil D. An example of array of signals for the soil D is shown in figure 3.

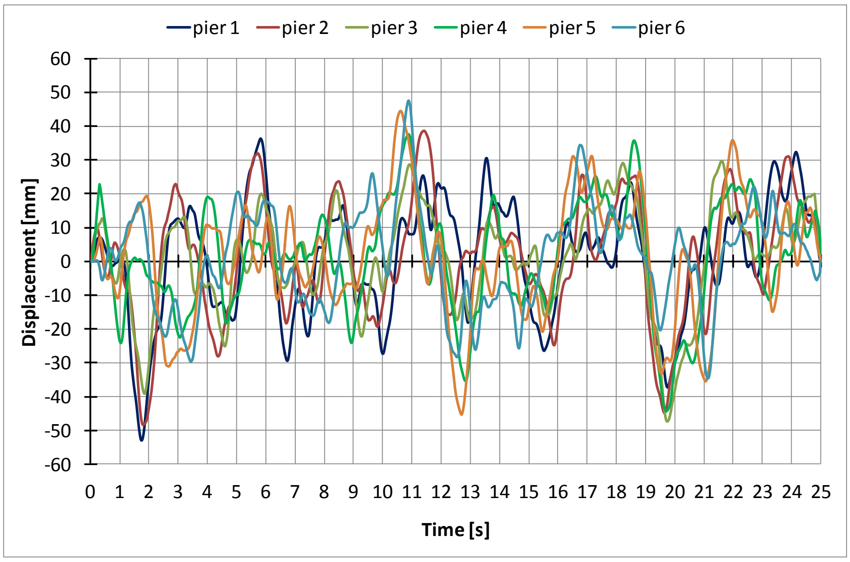



**Figure 3.** Array of asynchronous displacement histories for soil D

### 3.2. Materials

The materials associated to the bridges elements are a concrete C32/40 for the deck and C28/35 for the piers. In the linear analyses to those materials a reduced Young Modulus is associated to account the cracking. In particular, a 50% reduction is associated to the deck and a 20% reduction to the piers. In the non-linear analyses, a Kent & Park model is considered for the confined concrete by transverse reinforcement. Since in all the considered cases the cracks develop at the first analysis steps, the initial elastic modulus is considered un-cracked. Menegotto & Pinto non-linear constitutive model is used for the steel B450C class (Italian classification according to NTC 2008) with 1.15 hardening ratio.

## 4. STRUCTURAL RESPONSE OF CONTINUOUS DECK BRIDGES

The bridges described above are subjected to FEM linear and non-linear analyses. The structural response of the linear analyses is compared to (synchronous) response spectrum analyses, the EC8 code provisions and an alternative formulations proposed by authors (Nuti and Vanzi, 2009).
The non-linear analyses are conducted to assess when it is more appropriate consider the asynchronous motion, the results are compared with synchronous non-linear analyses. Moreover, the effects of the asynchronous motion on the failure mechanisms and the forces redistribution are evaluated.

### 4.1. Eurocode 8-2 provisions and Simplified Nuti and Vanzi Method

The Eurocode 8-2 introduce a simplified model to account the effects of non-synchronism. The approach superimposes the inertial forces (using a single response spectrum or equivalent accelerogram sets, corresponding to the most severe ground type underneath the bridge supports) to the pseudo-static effects of two displacement sets, imposed at the supports of the bridge deck, that consist in a linear distribution of displacements and a set of phase shifts. In this case, the relative displacement between two adjacent piers is defined as:

$$d_i = \pm \frac{1}{2} \cdot \beta_r \cdot \frac{d_g \sqrt{2}}{L_g} \cdot L_{av,i} \tag{1.5}$$

where $\beta_r$ takes into account the magnitude of ground displacements in opposite direction at adjacent supports, $d_g$ is the design ground displacement at support i, $L_g$ is the distance beyond which the ground motion may be considered completely uncorrelated ($L_g$ vary from 600 m for soil A to 300 m for soil D), $L_{av,i}$ is the average distances between the support i and the adjacent i-1 and i+1.

In each horizontal direction the most severe effects resulting from the pseudo static analyses of have to be combined with the relevant effects of the inertia response, by using the SSRS. The result of this combination constitutes the effects of the analysis in the direction considered.

Since the EC8 simplified approach seems at the same time undersize the imposed displacements and overestimate the bridge response (the probability that all the piers are stressed in opposite directions by the same amount is equal to zero), recently the authors (Nuti and Vanzi, 2009) have developed an alternative formulation that add the spectral response to the following soil relative displacements:
o in i : $d_{ij}$
o in i-1 and i+1 : $d_{ij}$ / 2
o elsewhere : 0

The differential displacement between two points on the ground associated to the Nuti and Vanzi method is that reported in the new Italian seismic code (Min. Infr., 2008):

$$d_{ij}(x) = 1.25\left|d_{gi} - d_{gj}\right| + 1.25\left(\sqrt{d_{gi}^{\,2} + d_{gj}^{\,2}} - \left|d_{gi} - d_{gj}\right|\right)\left[1 - e^{-1.25(x/v_e)^{0.7}}\right] \qquad (1.6)$$

where $v_s$ is the velocity of shear waves, $d_{gi}$ and $d_{gj}$ are the maximum design ground displacement at support i and j respectively.

As an example, in fig. 4 a comparison is made between the mean value of the design differential displacement (at the surface) of EC8 (top) and the Nuti and Vanzi method (bottom).

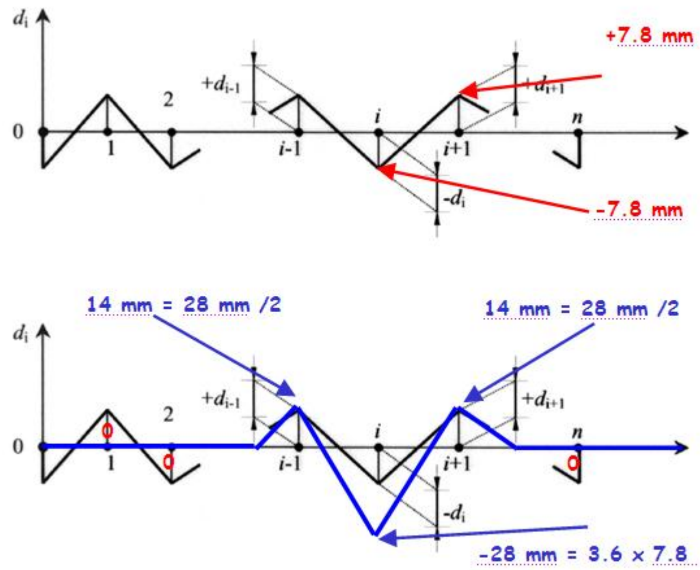
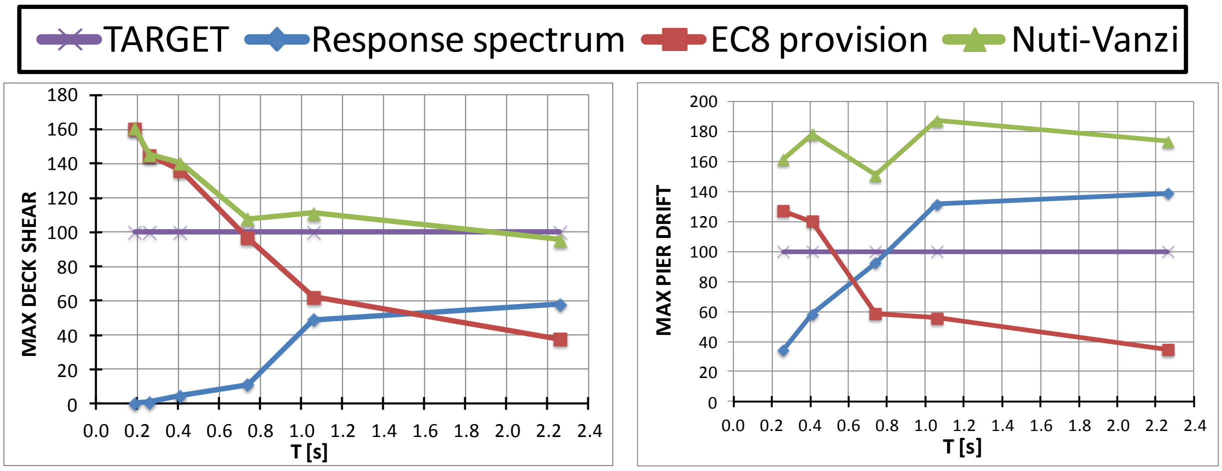



**Figure 4.** Comparison between the mean value of the design differential displacement (at the surface) of EC8 (top) and herein proposed (bottom). Soil type D, piers distance equal to 30 m, PGA of 0.1g

### 4.2. Comparison between linear analyses and code provisions

The FEM linear analyses are developed to verify if the bridge asynchronous seismic response can be calculated as the sum of a response spectrum analysis and a pseudo-static component. Due to this issue, the asynchronous dynamic analyses are considered representative of the effective behavior of the structure (label "Target"). The pseudo-static bridge response is defined either by the code provision (label "EC8 provisions") either by the Simplified Nuti and Vanzi Method (label "Nuti - Vanzi").

For sake of brevity, the response variables here reported are the maximum shear for the deck and maximum piers drift (the difference between the pier top displacement and the ground displacement) for piers in function of the natural period of each bridge on soil D (Figure 5).




a) b)

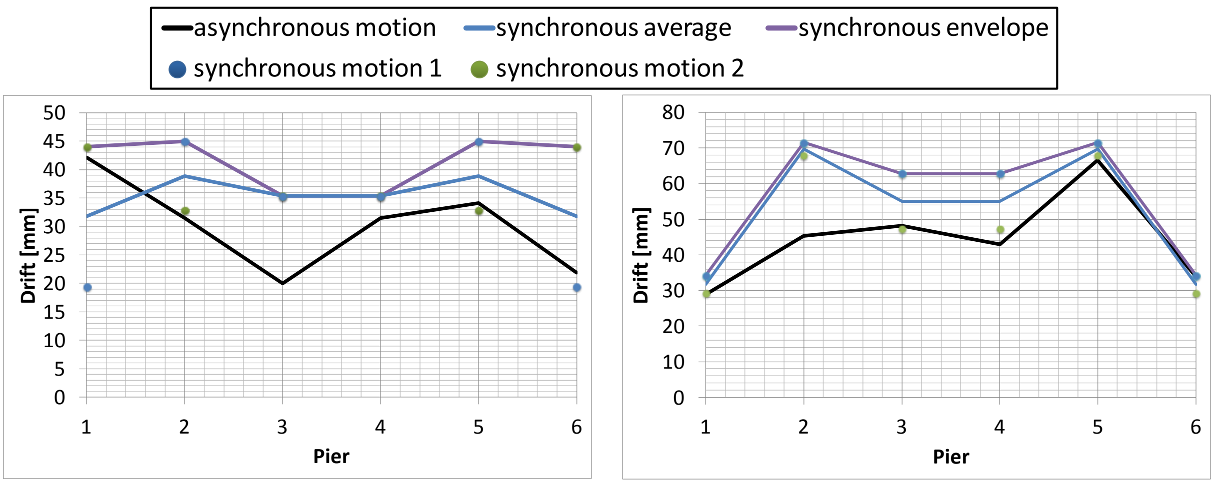
**Figure 5.** Results obtained for soil D: a) maximum shear deck; b) drift of single pier. (Carnevale et al., 2010)

The graphs show that the response spectrum analysis is even un-conservative. The Simplified Nuti and Vanzi Method give a good approximation of the real behavior of the structure when the period is greater than 0.4 seconds. In the same range, the EC8 simplified approach is rather to the detriment of safety. The differences become very marked if the drift of the single pier is considered (Figure 5b).

### 4.3. Non-Linear analyses

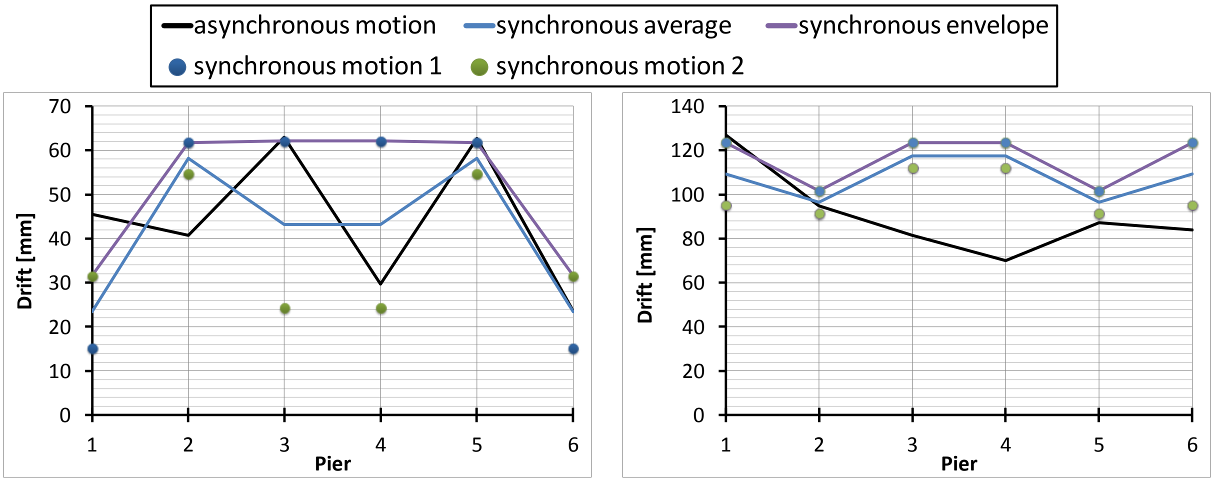
The non-linear bridge asynchronous dynamic behaviour due to different spatially varying ground motion sets has been carried out in terms of differential displacement between the top of each pier and its foundation. The analyses shortly described here are the starting point to define a non-linear simplified methodology to evaluate the response of bridges subjected to asynchronous motion.

For sake of brevity, only the more significant results are presented below. Generally, the analyses carried out show that, increasing the natural period and considering a soil category with worse characteristics, the seismic response of the bridge is considerably influenced by the effects of the asynchronous motion. In the figures 6-8 a comparison between the synchronous and the asynchronous response is also presented for a single displacement history set. The shown cases indicate that consider only the synchronous motion may be unsafe, especially for the external piers.



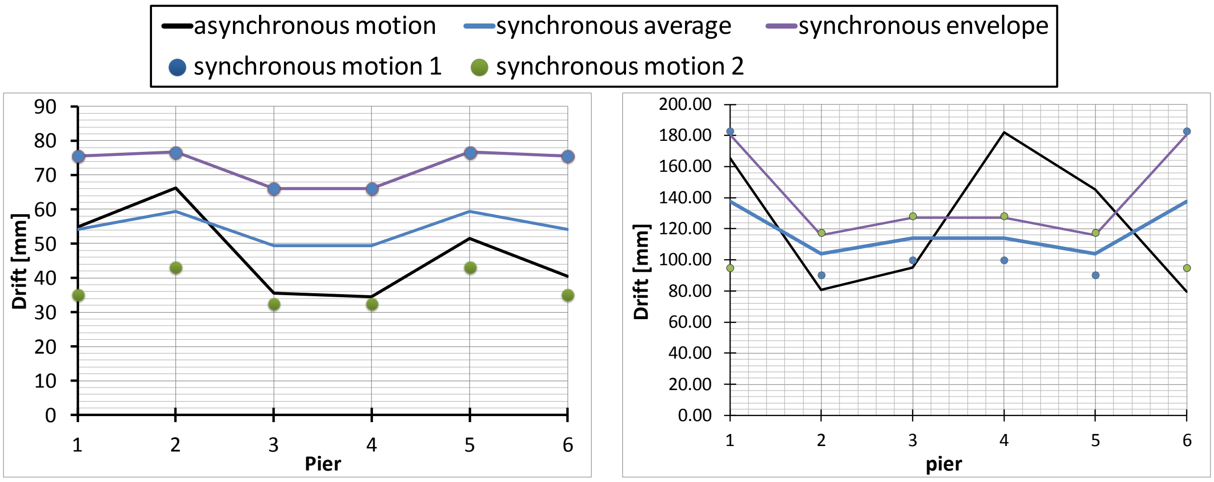

a) b)

**Figure 6.** Results obtained for one displacement history for soil A in terms of piers response: a) max drift for bridge 2; b) max drift for bridge 6.



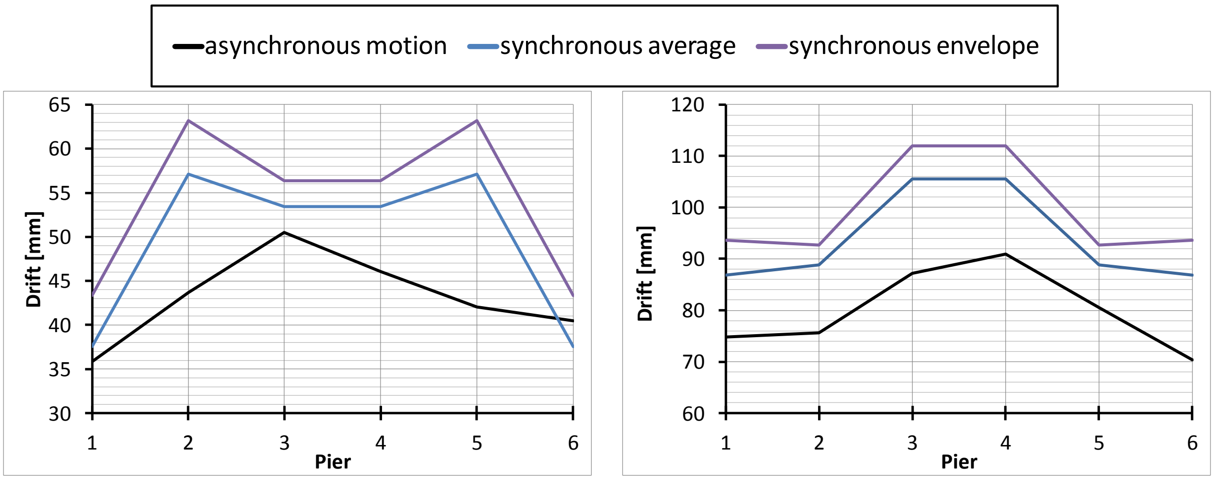

a) b)

**Figure 7.** Results obtained for one displacement history for soil B in terms of piers response: a) max drift for bridge 2; b) max drift for bridge 6.




a) b)

**Figure 8.** Results obtained for one displacement history for soil D in terms of piers response: a) max drift for bridge 2; b) max drift for bridge 6.




a) b)

**Figure 9.** Average results obtained for soil B in terms of piers response: a) max drift for bridge 2; b) max drift for bridge 6.

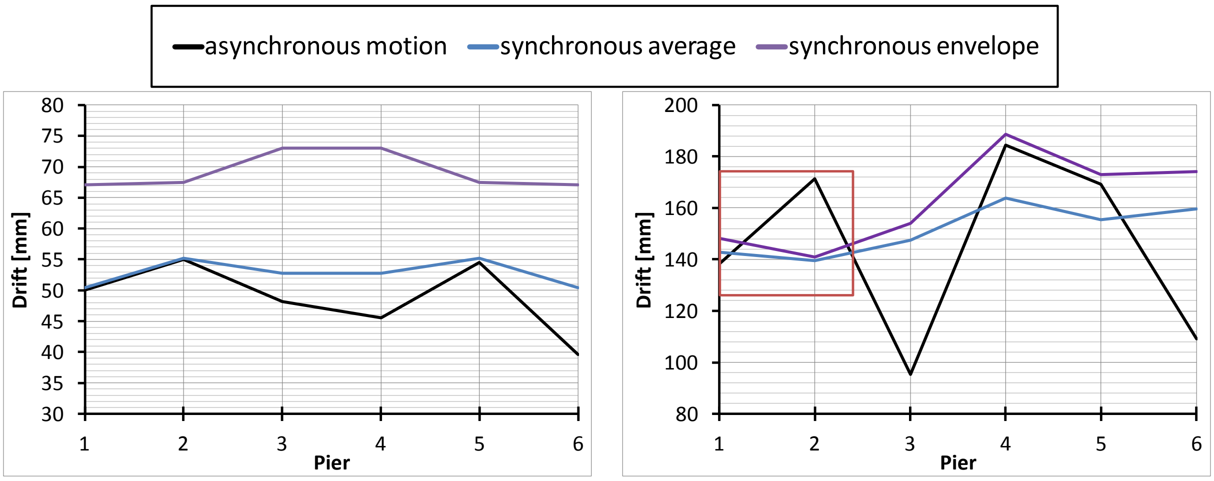



**Figure 10.** Average results obtained for soil D in terms of piers response:
a) max drift for bridge 2; b) max drift for bridge 6.

The difference between the two analysis methodologies becomes less pronounced if a scenario analysis is effected and the synchronous analyses from all the displacement histories imposed to consider the asynchronous motion are carried out. A comparison is reported below for the soils B and D (figg. 9-10). The asynchronous motion, in fact, is not so relevant for the case of the soil A.

In those cases, the envelope synchronous analyses of the (violet curves in the figures), rather than the average of responses (blue curves in the figures), provides more precautionary results compared to the case in which one considers a single displacement history. Also, the effects of the asynchronous motion on the piers behaviour may be neglected, until it becomes particularly significant (as in the case of the bridge 6, soil D, fig 10b).

Finally, the study include a comparison between synchronous and asynchronous analysis, in order to evaluate either when may be safe consider the only the synchronous motion either the effects of the asynchronous motion on the failure mechanisms or the forces redistribution.

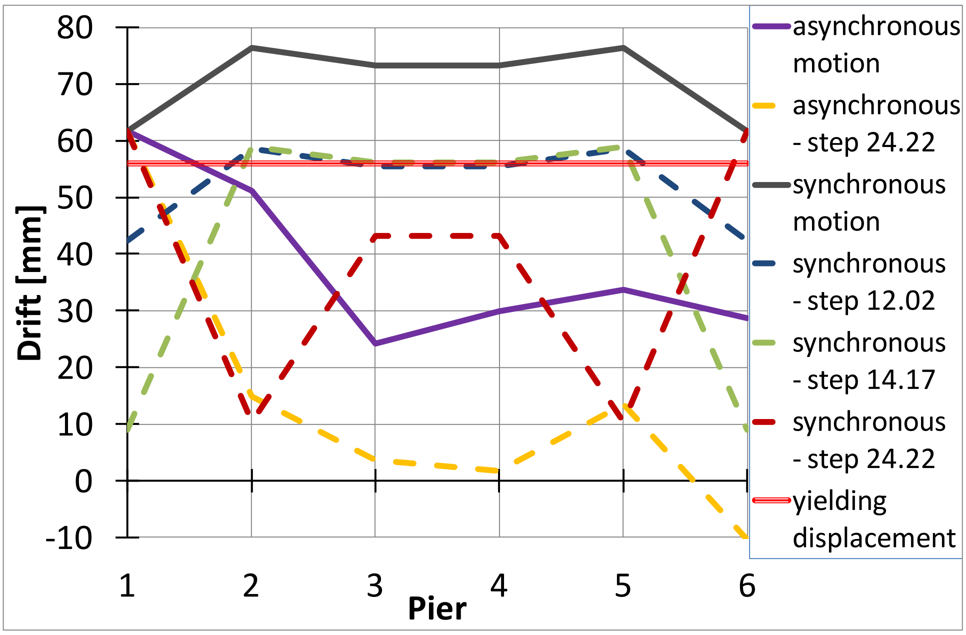



**Figure 11.** Effects of the asynchronous motion on the failure mechanisms: bridge 2- soil D - displacement history 1, with the same plastic hinge development in synchronous and asynchronous analyses. Into the asynchronous analysis yielding occur on the pier 1. Into the synchronous analysis all the piers are plasticized.

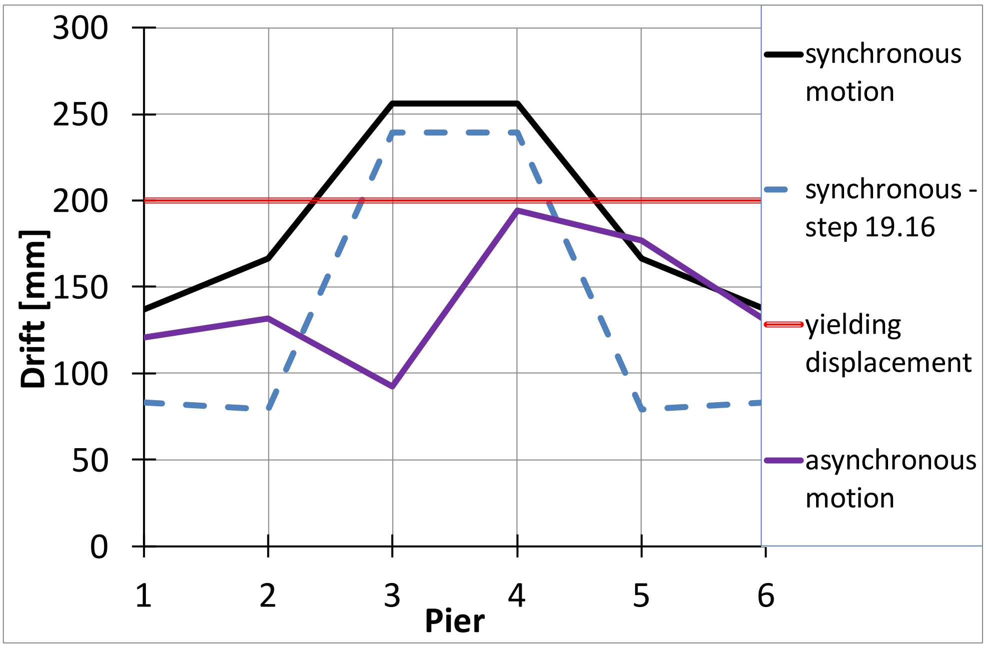



**Figure 12.** Effects of the asynchronous motion on the failure mechanisms: bridge 6- soil D - displacement history 1, with different plastic hinge development in synchronous and asynchronous analyses. Into the asynchronous analysis no yielding occur. Into the synchronous analysis the piers 3 and 4 are plasticized.

The effects of the asynchronous motion on the failure mechanisms appear, instead, less clear: in some cases (figg. 11-12) the plastic hinge development is the same either in the synchronous either in the asynchronous analyses, in other cases appears completely uncorrelated.

## 6. CONCLUSIONS

The study shows that the asynchronous input motions have a significant effect on the response of the bridges that may be more severe than those from synchronous inputs. This behaviour becomes more significant when the bridge natural period increase and the soil characteristics worsen.

To clarify this aspect, linear and non-linear parametric analysis on bridges of different geometry and founded on various soils type (code category) have been carried out. The geometry variability is obtained varying the ratio between the pier and the deck stiffness, and then the bridge natural period. The bridge piers have slenderness in the range of 2-15; the bridge decks, modelled with a continuous rectangular equivalent section, are representative of one, three or six lanes.

In particular, the linear analyses highlight that the asynchronous response may be caught superimposing the inertial forces to the pseudo-static effects. This latter, if evaluated with the EC8 provision, appear un-conservative when the asynchronous effects become significant. For this reason an alternative formulation proposed by the authors has been tested.

The bridge non-linear asynchronous response is also been investigated. The results of synchronous and asynchronous analyses are compared; the effects of the asynchronous motion on the failure mechanisms are evaluated. The analyses show that, increasing of the natural period and worsening the soil characteristics, the bridge response is influenced by the asynchronous motion that is not so relevant for the case of the soil A. The best accord between synchronous and asynchronous analysis is obtained by means an envelope of the synchronous displacement histories. However there are cases where the synchronous action is unfavourable. The effects of the asynchronous motion on the failure mechanisms appear, instead, less clear.

This work has focused the attention especially on the piers behaviour. Anyway, is well know that the asynchronous motion is more important than the synchronous one on the deck responses. This aspect must be further investigated, because it can cause the collapse of the deck supports.


**ACKNOWLEDGMENTS**

The present work is developed under Italian Ministry of Research PRIN 2008 project prot. 20083FFYWP and the RELUIS Convention - Research line 1.1.2.


**REFERENCES**


Carnevale L., Lavorato D., Nuti C., Vanzi I. (2010). Response of continuous deck bridges to non synchronous seismic motion. Rome, Sustainable Development Strategies for Constructions in Europe and China, Roma 19-20 Aprile 2010.

CEN [Comité Européen de Normalisation] (2005) Eurocode 8: Design provisions of structures for earthquake resistance—Part 2: Bridges (prEN1998-2, Final Draft). CEN, Brussels

Der Kiureghian A, Neuenhofer A (1992) Response spectrum method for multiple support excitations. Earthq Eng Struct Dyn 21:713–740. doi:10.1002/eqe.4290210805

fib [fédération international du béton] (2007) Structural solutions for bridge seismic design and retrofit—a state of the art. fib Bull. 39, May 2007. Analysis. Earthq Eng Struct Dyn 32:629–652. doi:10.1002/eqe.242

Lupoi A, Franchin P, Pinto PE, Monti G (2005) Seismic design of bridges accounting for spatial variability of ground motion. Earthq Eng Struct Dyn 34(4–5):327–348. doi:10.1002/eqe.444

Ministero Infrastrutture, Norme Tecniche per le Costruzioni, DM 14 gennaio 2008, Gazzetta Ufficiale n. 29 del 4 febbraio 2008 - Suppl. Ordinario n. 30

Norman JA, Virden DW, Crewe AJ,Wagg DJ (2006) Physical Modelling of bridges subject to multiple support excitation. In: 8th National conference on earthquake engineering, San Francisco, California, U.S, Paper No. 373

Nuti C, Vanzi I (2005) Influence of earthquake spatial variability on differential soil displacements and SDF system response. Earthq Eng Struct Dyn 34(11):1353–1374. doi:10.1002/eqe.483

Nuti C, Vanzi I (2009) Effects of Non Synchronous Seismic Motion on Continuous Deck Bridges Response. In: XIII convegno l'ingegneria sismica in Italia (ANIDIS 2009). Bologna, 28 giugno – 2 luglio 2009

Pinto A, Pegon P, Magonette, Tsionis G (2002) Pseudo-dynamic testing of bridges using non-linear substructuring. Earthq Eng Struct Dyn 33(11):1125–1146. doi:10.1002/eqe.393

Sextos A, Kappos A, Pitilakis K (2003) Inelastic dynamic analysis of RC bridges accounting for spatial variability of ground motion, site effects and soil-structure interaction phenomena. Part 2: Parametric

Sextos A, Kappos A (2009) Evaluation of seismic response of bridges under asynchronous excitation and comparisons with Eurocode 8-2 provisions. Bull Earthquake Eng 7:519–545. DOI 10.1007/s10518-008-9090-5

Sextos A, Pitilakis K, Kappos A (2003) Inelastic dynamic analysis of RC bridges accounting for spatial variability of ground motion, site effects and soil-structure interaction phenomena. Part 1: Methodology and Analytical tools. Earthq Eng Struct Dyn 32:607–629. doi:10.1002/eqe.241