Recent developments on the effect of asynchronous earthquake excitation on the dynamic response of soil-foundation-superstructure bridge systems

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Abstract

During strong ground motion, it is expected that bridge structures are subjected to excitation that is non-uniform along their longitudinal axis in terms of amplitude, frequency content and arrival time, a fact primarily attributed to the wave arrival delay, their loss of coherency and the effect of local site conditions. To this end, advanced analytical solutions and enhanced know-how is utilized in order to identify the relative importance of the aforementioned phenomena and investigate potential implications in engineering design. The scope of this paper therefore, is to review the recent developments on the problem by illustrating numerical examples of the dynamic response of characteristic bridge structures under asynchronous excitation scenarios of varying levels of complexity and refinement. It is concluded that the problem is multi-parametric and complex but certain situations can be identified where the assumption of identical excitation between support points strongly underestimates the imposed ductility demand both at the foundation and the superstructure.

INTRODUCTION

From all the parameters that define the non-linear dynamic response of complex structures such as bridges, the input motion has by far the highest level of uncertainty. The last three decades, different approaches, methodologies and tools have been utilized to deal with this uncertainty and put it in a framework that can be quantified and thus uniformly interpreted by the practicing engineers and the scientific community. The extensive use of refined response spectra and the utilization of real records from different soil and seismotechtonic conditions is a precious source of information regarding the potential excitation of bridges, which when combined with the increasingly enhanced capabilities for inelastic dynamic analysis provides a very good estimate of the expected response of bridge structures under earthquake loading. Nevertheless, the uniform application of the selected natural or even artificial motions along the supports is not necessarily valid for extended structures since, as recent research has shown, seismic motion can be not only significantly different at each pier support point but also induce forces and deformations that could not be predicted with the assumption of synchronous excitation.

The sources of spatial and temporal variations of seismic motion are well known (Der Kiureghian & Keshishian) and can be summarized as the effect of a) waves travelling at a finite velocity, so that their arrival at each support point is out of phase b) loss of coherency in terms of statistical dependence, that is, loss of signals 'similarity' due to multiple reflections, refractions and superpositioning of the incident seismic waves that occur during propagation and c) local soil conditions especially for cases that the soil profile through which motion propagates varies significantly. Due to the above, both peak ground acceleration and frequency content of the motion may be strongly varied among the foundations of the successive piers. Although often neglected, the potential filtering at the foundation level that results from the relative flexibility of the foundation-soil system components is an additional parameter that contributes to the extent of variability of the motions that are actually imposed at each separate pier.

Another simplification often made is that, bridge structures are commonly considered to be fully fixed at their pier base points. Nevertheless, it is well known that the bridge foundation is flexible, dissipates energy and interacts with the surrounding soil and the superstructure in such a way, that it filters seismic motion (kinematic interaction) while it is subjected to inertial forces generated by the vibration of the superstructure (inertial interaction) (Mylonakis et al., 1997, Gazetas & Mylonakis, 2002). This phenomenon is very complex and its beneficial or detrimental effect on the dynamic response of the bridge is dependent on a series of parameters such as the intensity of ground motion, the dominant wavelengths, the angle of incidence of the seismic waves, the stromatography, the stiffness and damping of soil, as well as the size, geometry, stiffness, slenderness and dynamic characteristics of the structure.

The final overall response of a bridge structure therefore, is a function of the above strongly coupled and frequency dependent phenomena that have to be considered together with the accurate representation of the structural dynamic characteristics, the latter inclusive of the effect of foundation compliance and energy dissipation at the pier-foundation interface.

It is also often considered that the importance of asynchronous excitation is only related (if it is accepted that it is related at all) to the dynamic performance of the superstructure. Consequently, the foundation is assumed to be completely unaffected by effect of Spatial Variability of Ground Motion (SVGM) while the potential coupling between SSI effects and the characteristics of earthquake ground motion are not accounted for. However, notwithstanding the complexity of the overall problem, there is strong evidence that the foundation is (Fig. 1):

a) kinematically affected by SVGM since the local soil conditions modify the frequency content of the incoming waves, and the (also frequency dependent) dynamic interaction of the soil-pile system is altered.

b) Inertially affected because the (actual). condition of asynchronous excitation of the bridge is very often related to the triggering of higher modes of vibration of the superstructure which in turn modifies the overall dynamic response of the superstructure. As a result, the inertial loads that are transmitted back to the foundation level may be different compared to the case where the hypothesis of synchronous excitation applies.

The scope of this paper is therefore, to assess the recent findings on the subject aiming at providing an insight to a number of issues that arise with respect to the effect of the (inevitable) spatial variation of seismic motion on the dynamic behavior of long bridges. In particular, an effort is made to:

a) evaluate the relative importance of the aforementioned sources of spatial variability on the structural and thus the foundation response while highlighting cases where their refined consideration is not necessary,

b) numerically compare approaches of different analysis complexity towards the identification of the optimal balance between accuracy and computational cost,

c) scrutinize the significance of accounting for local site conditions as a means to capture pier-dependent characteristics of incoming seismic motions compared to the implementation of coherency-based generation schemes,

d) investigate the effect of the interaction between soil, foundation and structure for cases where the incoming seismic wavefield arrives already significantly different in the frequency domain,

e) highlight potential advantages in using natural records as reference motion for generating the necessary artificial motions through a conditional simulation scheme,

f) investigate the sensitivity to the asynchrounous motion assumptions made of bridges with different dynamic properties, foundation configurations and geometrical characteristics the latter especially in terms of irregularity and curvature in plan.

Finally it is attempted to critically review the current seismic code provisions and recommendations for accounting for spatial variability of ground motion for the design of concrete bridges, particularly the recent provisions of Eurocode 8 - Part 2 that provide both a method of calculation as well as empirical formulae for quickly assessing the importance of asynchronous excitation.

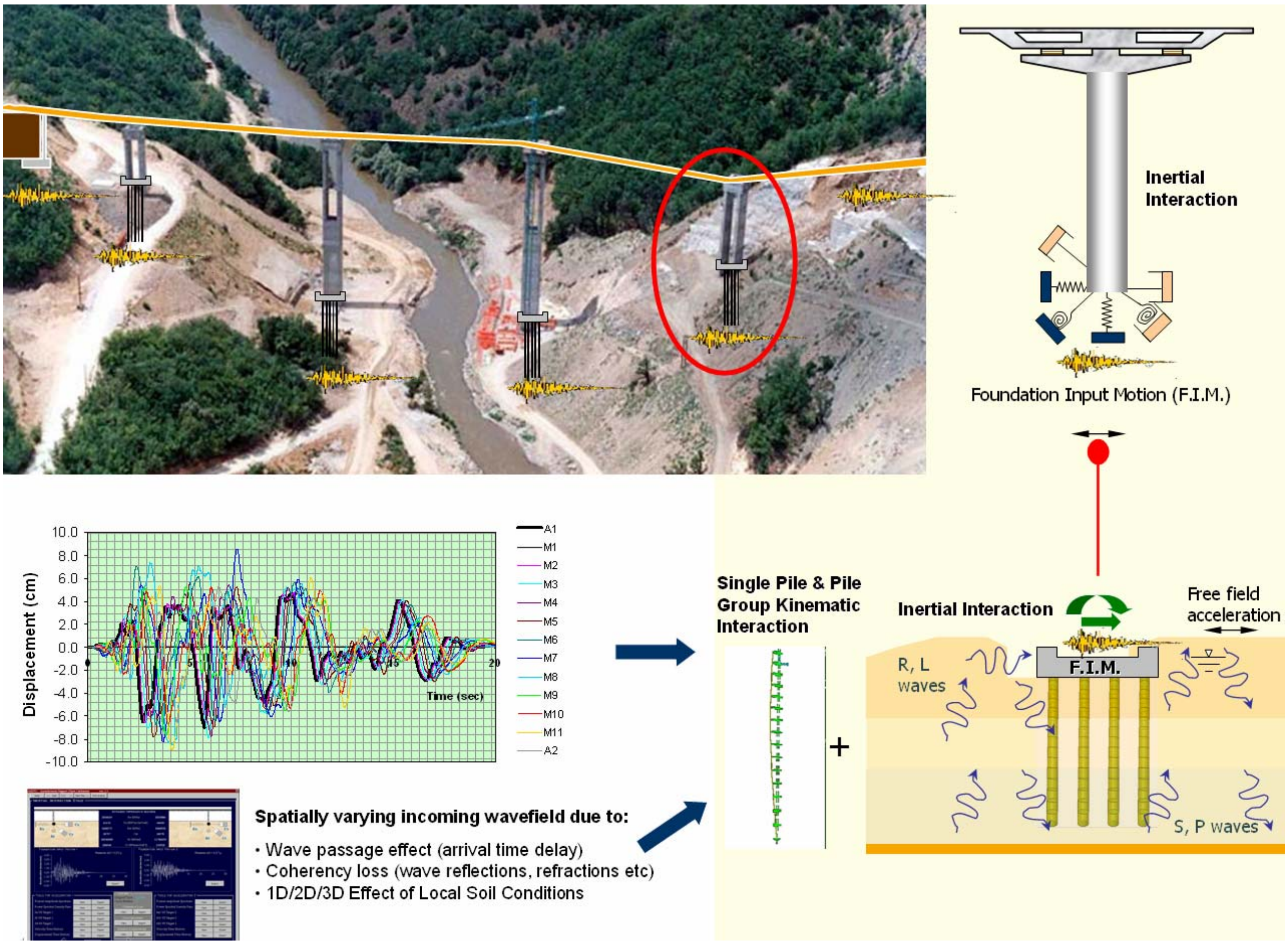


Fig 1: Coupling of ground motion variability with the dynamic performance of the soil-foundation-structure system.

RECENT STUDIES ON THE EFFECT OF ASYNCHRONOUS MOTION

The first pioneering studies on the effect of non-synchronism of the ground motion on bridge response date back to the '60s (Bogdanoff et al., 1965) though it is only since the '90s that this phenomenon has been seen from a more practical perspective. The effort was gradually extended to applications on simple structures, while analytically derived solutions for generating spatially variable seismic motions were developed (Hao, 1989, Harichandran & Wang, 1990, Zerva, 1990, and Deodatis, 1996).

More realistic bridge configurations were also studied by various researchers, either analytically and numerically (Monti et al, 1996, Simeonov et al., 1997, Shinozuka et al., 2000, Panza et al, 2001, Zanardo et al., 2002]) or experimentally (Pinto et al, 2002), implementing correspondingly refined analysis approaches and establishing the fundamental framework to consider the potential role played by multiple support excitation on the dynamic response of the structure itself. Recently, the effect of asynchronous motion on the *inelastic* dynamic behaviour of bridges has also been examined involving specific code-prescribed bridge configurations (Lou & Zerva, 2005), or a set of parametrically modified realistic bridge structures (Pitilakis et al., 2002 and Lupoi et al, 2005, Nutti & Vanzi, 2005). Extension of the proposed methodologies to account for the coupling effect of spatial variability, site effects and soil-structure interaction within a comprehensive framework has been performed by Sextos et al., 2003a and 2003b, while the importance of asynchronous excitation on curved bridges has also been studied (Ettouney et al., 2001, Allam & Datta, 2003, and Sextos et al, 2004). All the aforementioned efforts have a practice-oriented aim to provide a statistical basis for detecting systematic trends and quantifying the relative importance of the various phenomena involved in the seismic response of bridges.

Despite the discrepancy of the results therefore and the complexity of the problem, the potential detrimental effects of asynchronous motion and the subsequent need of research and code–oriented studies is widely recognised (Calvi, 2004).

Inevitably though, since current seismic design philosophy typically relies on energy dissipation through non-linear behaviour, the only available tool for a meaningful study of the problem is the generation of spatially distributed motions to be used in non-linear time-history analysis, a fact that is recognised in a forthcoming *fib* state-of-the-art document (2006).

A COMPREHENSIVE APPROACH FOR INELASTIC DYNAMIC ANALYSIS

A methodology has been proposed and validated against theoretical solutions, alternative computer codes and recorded data, where available. This comprehensive methodology incorporates and uncouples all important issues (asynchronous motion, site effects, soil-structure-interaction) within the context of a general scheme for the inelastic analysis of bridges in the time domain (Sextos et al., 2003b). The idea is to generate synthetic time histories which are distinct at each support point (piers and abutments), through a refined spatial variability model accounting for wave passage, loss of coherency and site effects, the latter being accounted for, primarily in terms of 1D site response analysis of multi-layer, damped soil profiles overlying an elastic bedrock but also in an envelope approach for the extreme case where lateral surface waves propagation is expected to further amplify ground motion (2D site effects).

A number of available coherency models can be used for the generation of the spatially variable seismic motions; the model of Luco and Wong (1986) was adopted herein among others available in the literature:

$$\gamma(\omega,\xi) = e^{-\left(\frac{\alpha\omega\xi}{V_s}\right)^2} \cdot e^{i\left(\frac{\alpha\omega\xi^L}{V_{app}}\right)} \qquad (1)$$

the first term being an exponential decay of coherency with separation distance ξ and frequency ω, which decreases as soil becomes stiffer, while the second term represents the wave passage effect which produces longer signal arrival delay as the projected horizontal inter-station distance ξ and the frequency ω increase and the apparent velocity V_{app} decreases. With the simplifying assumption made of a common power spectrum for all support points, the $n \times n$ cross power spectral density matrix can then be written as:

$$[S(\omega)] = \begin{bmatrix} 1 & \gamma_{12}(i\omega,\xi) & \dots & \gamma_{1n}(i\omega,\xi) \\ \gamma_{21}(i\omega,\xi) & 1 & \dots & \gamma_{2n}(i\omega,\xi) \\ \dots & \dots & 1 & \dots \\ \gamma_{n1}(i\omega,\xi) & \gamma_{n2}(i\omega,\xi) & \dots & 1 \end{bmatrix} \cdot S_0(\varpi) \qquad (2)$$

which is a Hermitian and positive definite matrix, that can be expressed as a product of a lower triangular matrix $\left[L\left(i\varpi\right)\right]$ and its Hermitian matrix $\left[L\left(i\varpi\right)\right]^H$:

$$\left[S\left(\omega\right)\right] = \left[L\left(i\varpi\right)\right]\left[L\left(i\varpi\right)\right]^H \cdot S_0\left(\varpi\right) \qquad (3)$$

where $\overline{\omega} = (2/3)\omega$ and $[L(i\overline{\omega})]$ is derived with the use of Choleski decomposition method as follows:

$$L(i\varpi) = \begin{bmatrix} l_{11}(\varpi) & 0 & \dots. & 0 \\ l_{21}(i\varpi) & l_{22}(\varpi) & \dots. & 0 \\ \dots. & \dots. & \dots. & \dots. \\ l_{n1}(i\varpi) & l_{n2}(i\varpi) & \dots. & l_{nn}(\varpi) \end{bmatrix} \qquad (4)$$

Consequently, the distinct acceleration time histories at all points that reflect the effect of time delay and loss of coherency only, can be expressed in the general form :

$$x_i(t) = 2\sum_{m=1}^{n}\sum_{l=1}^{N}\left|L_{jm}(\omega_{ml})\right|\sqrt{\Delta\omega} \cdot \cos[\omega_{ml}t + \theta_{jm}(\omega_{ml}) + \phi_{ml}] \qquad (5)$$

where φ_{ml} are independent random phase angles, uniformly distributed in the range $(0,2\pi)$, N represents the Nyquist frequency $\overline{\omega}_N$, $\Delta\omega$ is the frequency step and θ_{jm} is the phase which can be written as:

$$\theta_{jm}(\varpi_{ml}) = \tan^{-1}\frac{\mathrm{Im}[l_{jm}(\mathrm{i}\varpi_{ml})]}{\mathrm{Re}[l_{jm}(\mathrm{i}\varpi_{ml})]} \qquad (6)$$

Site Effects

The above uniform soil approach which accounts only for wave passage and loss of coherency but neglects the effect of local soil conditions is used when the anticipated ground motions are partially uncorrelated. For cases that the soil conditions along the bridge length are significantly different, an alternative approach is followed to account for the effect of soil conditions on the modification of ground motion.

As a first level of complexity, the method of proposed by Deodatis (1996) can be implemented, according to which different response spectra may be specified at each location, within a stationary stochastic vector scheme with prescribed spectral contents at each support. Alternatively, a target outcrop frequency content can be adopted for the generation of a sample motion compatible with the corresponding (outcrop) power spectrum, and deriving the bedrock Fourier spectrum through a deconvolution process. By applying the Inverse Fast Fourier Transform at the bedrock level, the corresponding power spectrum is computed and spatially variable accelerograms compatible with the above target spectrum and the (bedrock related) coherency function are generated at the bedrock level using the approach described above. Then, the distinct surface motions at each support point may be derived through multiple 1-D site response analyses. In particular, the assumption can be made that the equation of motion of an SH wave which vertically propagates at a velocity Vs through a Kelvin-Voigt soil of viscosity η :

$$\frac{\partial^2 u}{\partial t^2} = V_s^2 \left[\frac{\partial^2 u}{\partial t^2} + \eta \frac{\partial^2 \dot{u}}{\partial y^2} \right] \qquad (7)$$

has a solution that can be written in the form of an upward (first term) and a downward (second term) travelling wave, with amplitudes A and B which depend on boundary conditions:

$$u(z,t) = A \cdot e^{i(\omega t + k^* z)} + B \cdot e^{i(\omega t - k^* z)} \qquad (8)$$

where $k^* = \omega \sqrt{\rho / (G + i\omega\eta)}$ is the complex wave number. In such a case and for any given motion, the transfer functions between the surface points of multi-layer damped soil profiles which lay over elastic bedrock are derived using the 'reflectivity coefficient' algorithm (Kennet, 1983) in which all multiple reflections and conversions between wave types are retained in part of the soil structure.

For the general case that wave passage, coherency decay and local soil conditions are equally important, the aforementioned approaches are combined in a more refined hybrid spatial variability and site effects mode. In particular, the target bedrock motion can be defined first, whereas multiple independent site response analyses can be performed at each pier location in order to derive the corresponding target free-field response spectra. Apparently the 1-Dimensional site response analysis can be linear, equivalently linear, or purely non-linear, depending on the available tools, the first generally leading to higher (more conservative) amplification levels. The site-dependent spectra derived can then be used through the aforementioned approach proposed by Deodatis (1996) together with a prescribed coherency decay model, leading to spatially variable motions that reflect both the desired frequency content and coherency pattern. This modified approach is improved in the sense that its accuracy is enhanced when (independent) refined (1D/2D/3D) site response models are used for the determination of the site-dependent spectra.

Soil-Foundation-Structure Interaction

Having defined distinct seismic motions at the foundation level of each pier, further modification of motion takes place in the frequency domain, in order to account for kinematic interaction between the soil and the foundation piles. The derived motion can then be used as the asynchronous input motion to the bridge structure which is assumed to be supported on different Beam-on-Dynamic-Winkler-Spring systems, whose complex dynamic impedance matrices (i.e. stiffness and damping properties) are derived for all horizontal, rocking and coupled modes of vibration according to available solutions from the literature. Especially for the rotational stiffness, a non-linear moment-rotation relationship is proposed (Sextos et al., 2003a) which combines the rotational compliance of the foundation with a lumped plasticity model for the R/C section that accounts for the plastic rotations caused by yielding of the pier base.

With the complete set of linear and non-linear pier base springs and the distinct acceleration and displacement time histories at each support location it is feasible and relatively easy to perform dynamic inelastic analysis of the superstructure subjected to spatially varying motions and influenced by local site conditions and soil-structure interaction. The above scheme for considering spatial variability, site effects and soil-foundation-structure interaction of bridges in the time domain has been implemented into the computer code ASING (Asynchronous Support Input Generator, Sextos et al., 2003a) which was used through out this study.

OVERVIEW OF THE BRIDGES STUDIED

In the present study, a number of different bridges in terms of structural configuration (overall length, span length, curvature in plan and height, pier-deck connection) has been selected in order to be able to highlight particular correlation between the dynamic characteristics of the structure and its sensitivity to multiple support excitation. The characteristics of the bridges studied as well as excitation and analysis properties are summarized in Table 1.

As a first level of complexity, a short but curved in height bridge was selected (Bridge01) while a well studied (Der Kiureghian & Keshishian, 1997), 292m long structure was also examined (Bridge 02). A detailed description can be found in Kateris (2003).

Table 1: Configuration and analysis details of the bridges studied

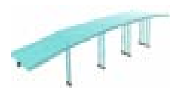
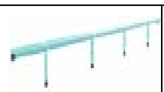
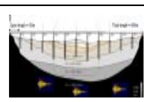
Structural Configuration and analysis							20 alternative bridges
Bridge ID #		Bridge01	Bridge02	Bridge03	Bridge04	Bridge05	Bridges 06-25
Bridge geometry	Number of spans	4	4	7	12	12	4, 6, 8
	Span length	30.5m	36.6-73.1m	62-67m		50m	50m, 100m, 150m
	Total length	152.4m	292.6m	459m	488m	600m	200m, 400m, 600m
	Pier-deck connection	Monolithic	Monolithic	Monolithic / Bearing type	Monolithic / Bearing type	Monolithic	Monolithic / Bearing type
	Curvature	In height	No	No	In plan	No	No
	Foundation	Fixed	Fixed	Footing	Pile Group	Pile Group	Pile Group
Spatial variability parameters	Loss of coherence	Yes	Yes	Yes	Yes	Yes	Yes
	Wave-passage	Yes	Yes	Yes	Yes	Yes	Yes
	Site response	No	No	No	Yes	Yes	Yes
	SSI	No	No	No	Kinemaric /Inertial	Kinemaric /Inertial	Kinemaric /Inertial
Total cases analysed		6	6	6	24	4	180
Type of analysis		LTHA RS	LTHA RS	LTHA RS	LTHA NLTHA	LTHA NLTHA	LTHA NLTHA
Direction of Seismic input		Transverse	Transverse	Transverse (one case longitudinal)	6 different angles of incidence	Transverse (one case longitudinal)	Transverse (one case longitudinal)
Reference Point Seismic Motion		EC8 artificial	EC8 artificial	EC8 artificial	EC8 artificial Kozani eq.	Athens eq.	Athens eq. Loma Prieta
References (structural configuration)		Kateris (2003)	Kateris (2003) Der Kiureghian & Keshishian, (1997)	Kateris (2003) Flesch et al., (2003)	Sextos et al., (2004)	Sextos et al., (2003b)	Sextos et al., (2003b)

Fig 2: Overview of Bridge01

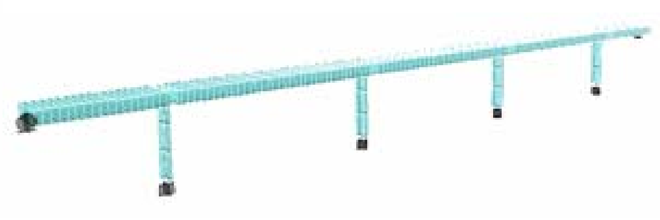
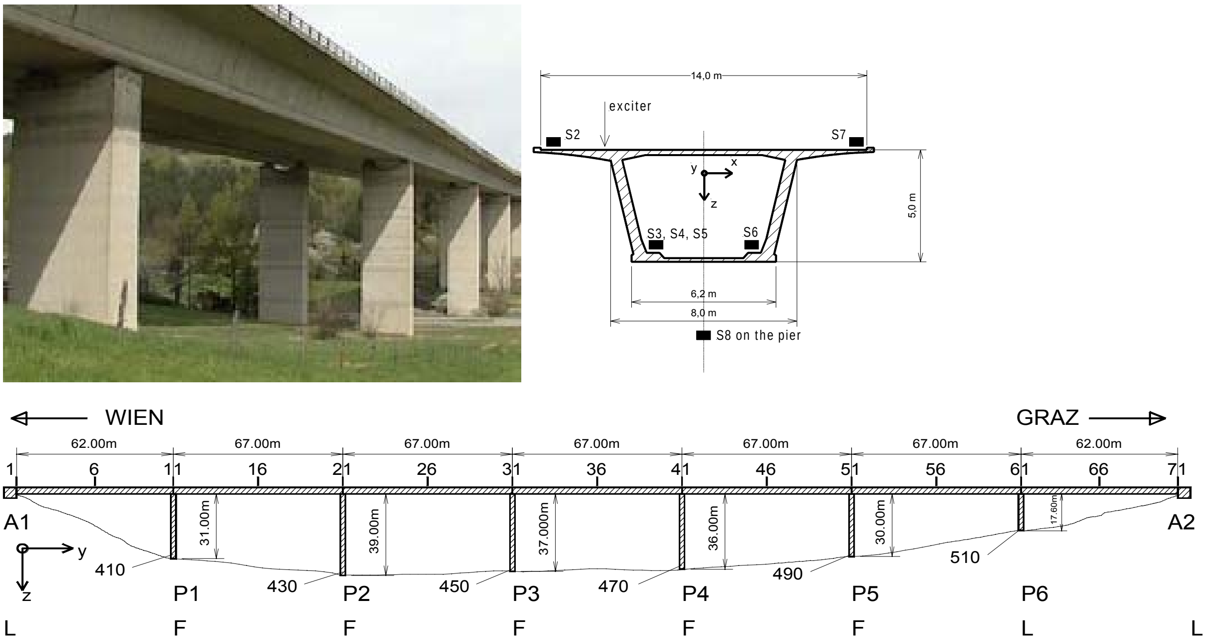


Fig 3: Overview of Bridge02



bearings:
L..........movable, longitudinal
F..........not movable

Fig 4: Overview of the Bridge03 (Talübergang Warth Bridge)

Fig 5: Overview of Bridge04 (Krystallopigi Bridge)

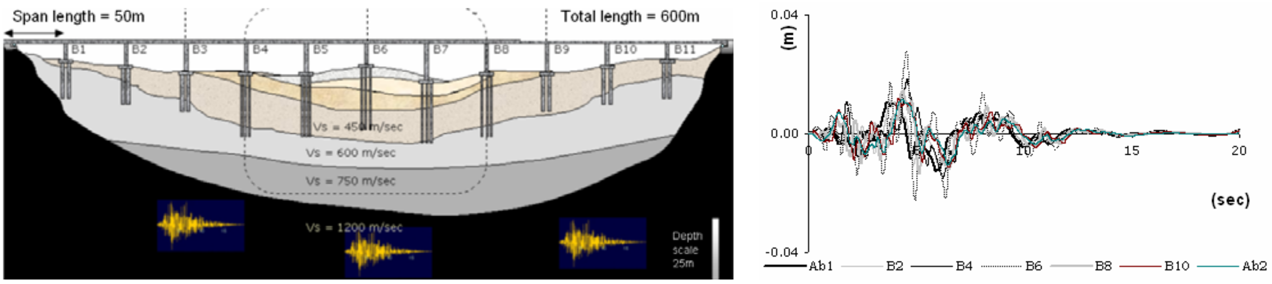


Fig. 6: Overview of Bridge05 (left) and spatially variable displacement time histories (right).

The Talübergang Warth Bridge located on motorway A2, 63 km south of Vienna. was also utilised for this study (denoted as Bridge03), not only on account of being an actual, already built and relatively long RC bridge but because its seismic behaviour has been extensively studied both analytically (Panza et al, 2001, Sokol and Flesch, 2004) and experimentally (Pinto et al., 2002, Flesch et al., 2003), inclusive of the effect of multiple support excitation.

Additionally, the Krystallopigi bridge (Bridge04) built along the 680km Egnatia Highway in Greece was also implemented. This is another real, twelve-span bridge having significant curvature in plan. Its overall 638m total length, curvature radius of 488m together with the non-uniform pier height which varies between 11 and 27m and the bearing type pier-to-deck connection contributes to the interest of the particular structure. The bridge has also been extensively studied in terms of its inelastic dynamic behaviour, including the effect of asynchronous motion (Kappos et al., 2005, Sextos et al., 2003c, Kappos et al, 2005).

The extreme case of a 600m bridge (Bridge05) founded on significantly different soil conditions was also examined (Sextos et al., 2004) in order to identify the relative effect of soil and site amplification in the light of soil-structure interaction. Finally, the experience gained by the inelastic analysis of 20 different bridge configurations (Sextos et al. 2003b) was also utilised to complement the observations derived by the study of Bridges01 to 05.

DEVELOPMENT OF ANALYSIS SCENARIOS

In order to determine the relative importance of the wave passage, incoherence effect and local site conditions to the dynamic structural response different method of analysis to the dynamic structural response as well, a number of scenarios were considered as they are summarized in Table 2. It is recalled that the details of the analysis assumptions as well as the target earthquake adopted at the reference point of each structure are illustrated in Table1 together with the structural configuration parameters.

At first, as a reference level, a uniform excitation analysis is performed for all structures (i.e. TH1 and SC1).Additionally to the cases of wave passage and/or incoherence, that were performed on the basis of equations (1) to (6) and the use of the computer code ASING, a conditional simulation was also performed for Bridges01, 02 and 03 with the computer code SIMQKE-II (Vanmarcke et al, 1997). Supplied with a target ground motion spectral density function, which may be evolutionary in nature, the latter program employs Covariance Matrix Decomposition in the frequency domain followed by Best Linear Unbiased estimation and an inverse Fast Fourier Transform to efficiently produce the nonstationary, spatially correlated, conditioned or unconditioned ground motions. The evolutionary nature of the motion is modeled by subdividing the motion into a sequence of time windows, within each of which a target spectral density function is specified. An averaging algorithm is also applied to smoothly connect the sequential windows. As a result, the spatial correlation is based on the phase aligned motions and is conveniently specified through simple parameters of a number of typical space-frequency correlation functions

Apart from the analyses in the time domain, it was also of particular interest to compare the member forces and deck displacements obtained by a 'standard' response spectrum analysis (RS1) with a similar analysis using an equivalent 'average' spectrum that considers the different (code-defined) soil categories along the bridge length. For Bridges01, 02 and 03 in particular, the RS2 and RS3 response spectrum cases stand for the assumption of a uniform spectrum in case of a profile which is of moderate variability and of significant variation with length respectively. For the determination of the 'average' response spectrum, the following equation proposed by the previous version of EC8 – Part 2 was adopted:

$$R_{average}\left(T,\beta\right)=\sum_{i=1}^{n}\frac{r_i}{\sum r_i}\cdot R_i(T,\beta) \qquad (9)$$

where r_i is the reaction force on the base of the pier i, when the deck is subjected to a unit displacement, and $R_i(T)$ is the site-dependent response spectrum appropriate to the soil conditions at the foundation of the pier i

Due to the significant curvature of Bridge04 a relatively different analysis strategy was followed. In particular, to investigate the effects of geometric incoherence three sets (scenarios) of artificial records were used. In the first two, two alternative target frequency spectra were used (i.e. Kozani earthquake and artificial Seismic Ground Motion compatible with the EC8 elastic response Spectrum both scaled at the target level of Peak Ground Acceleration equal to 0.24g). The motion along support points is initially considered as fully correlated and the arrival delay is defined by the angle between the direction of seismic wave propagation with respect to the bridge axis and the soil properties that determine the shear wave propagation velocity. For the third case, both phase and amplitude vary in space are accounted for (Table 2). Six different excitation angles (0°, 30°, 45°, 60°, 75°, 90°) with respect to the bridge axis were implemented and the sensitivity of the curved bridge to the rotation and spatially variable seismic motion vector was defined.

Bridge05 on the other hand, is again analysed using sets of artificial ground motions that initially match the target response spectrum of Athens earthquake (SC2, SC3) and lose their statistical dependence due to both the effect of local site conditions (SC4) and kinematic interaction between the foundation and the surrounding soil, given the incoming wave incoherence (SC5). The coupling between inertial interaction and the (significantly modified) motion between supports is also sough (SC6).

At all cases, the effect of considering a more refined scenario of earthquake ground motion is obtained on the basis of the ratio of the pier base member forces and deck displacements to the corresponding action effects of a ‘synchronous’ analysis using a single artificial or natural earthquake record.

RELATIVE EFFECT OF SOURCES OF SPATIAL VARIABILITY

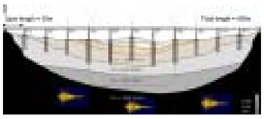
Through the above analysis scheme, the sensitivity of the different bridges to alternative approaches is investigated. From Fig. 7 it is depicted that for the short Bridge01, the effect of wave arrival delay or incoherence is indeed negligible. It is only the use of an ‘equivalent’ response spectrum, given a substantial variability in soil properties along the bridge length (RS3) that may lead to a minor increase in the resulting action effects. This fact was of course anticipated for a bridge 152m long, but it is interesting to notice that its curvature in height does not modify the overall trend that short bridges are either not sensitive at all or they are beneficially affected by the assumption of partially correlated travelling waves.

Bridge02 on the other hand which is of moderate length (292m) is found (Fig. 8) to be affected only by 10% both in terms of displacements and pier base member forces when multiply excited at its supports. This is another indication, among others in the literature, that for bridges of length less than 300m, the structural performance is not detrimentally affected by asynchronous motion provided that the local soil conditions are not varying significantly (Sextos et al, 2003b). It is also highlighted in Fig. 8 that the adoption of an ‘equivalent’ response spectrum for analysing the structure in the frequency domain is again strongly dependent on the assumptions made regarding the support-dependent spectra (i.e. the discrepancy between the two RS cases is almost 50%)

The most interesting feature though related to the dynamic performance of Bridge02, is that as the Fourier analysis reveals, the vibration of certain particles of the structure in the frequency domain is significantly different than that observed when the structure is excited using a single accelerogram. In particular, as seen in Fig. 9 despite the fact that the central pier top acceleration due to the fundamental mode vibration is decreased during asynchronous excitation, a higher (symmetric) mode is triggered and the pier also oscillates at the corresponding frequency of 085Hz (T=1.17sec). For the case of the actual and longer Bridge03 (Talübergang Warth) the above observation is even more pronounced. In fact, when the structure is analyzed with the artificially generated motions that consider the wave passage and loss of coherency effect (i.e. case TH4), the pier base member forces are increased up to 35% while the deck displacements are locally increased by 17%. Apparently, other piers exhibit lower distress or displacements under this asynchronous excitation scenario leading to an average overall increase of the order of 10%.

What is again of most interest though, is the fact that, as previously, the observed increase is the result of the excitation of two higher modes (one symmetric and one antisymmetric) that are excited due to the nature of the imposing seismic motion (Fig. 10).

Table 2: Description of analysis scheme

Bridge Studied	Analyses
	TH1: Synchronous Excitation TH2: Wave Passage only (Vapp=1000m/sec), Time History analysis using ASING TH3: Incoherence only (Vs/a=600m/sec), Time History analysis using ASING TH4:Incoherence (Vs/a=600m/sec) & Wave Passage (Vapp=1000m/sec),Time History using ASING TH5: Incoherence (Vs/a=600m/sec, Time History analysis using SIMQKE-II RS1: (Uniform) Response Spectrum Analysis RS2: (Equivalent) Response Spectrum Analysis RS3: (Equivalent) Response Spectrum - Significant soil variation
	TH1: Synchronous Excitation TH6:Incoherence & Wave Passage (Vs=400m/sec along the bridge chord axis),Time History using ASING TH7:Incoherence & Wave Passage (Vs=400m/sec at an angle of 30o),Time History using ASING TH8:Incoherence & Wave Passage (Vs=400m/sec at an angle of 45o),Time History using ASING TH9:Incoherence & Wave Passage (Vs=400m/sec at an angle of 60o),Time History using ASING TH10:Incoherence & Wave Passage (Vs=400m/sec at an angle of 75o),Time History using ASING TH11:Incoherence & Wave Passage (Vs=400m/sec at an angle of 90o),Time History using ASING
	SC1: Synchronous Excitation SC2: Wave Passage only (Vapp=1000m/sec), Time History analysis using ASING SC3: Incoherence (Vs/a=600m/sec) & Wave Passage (Vapp=1000m/sec),Time History using ASING SC4 : Incoherence, Wave Passage and Site Effects,Time History using ASING SC6 : Incoherence, Wave Passage, Site Effects and Kinematic SSI,Time History using ASING SC7 : Incoherence, Wave Passage, Site Effects, Kinematic and Inertial SSI, Time History using ASING

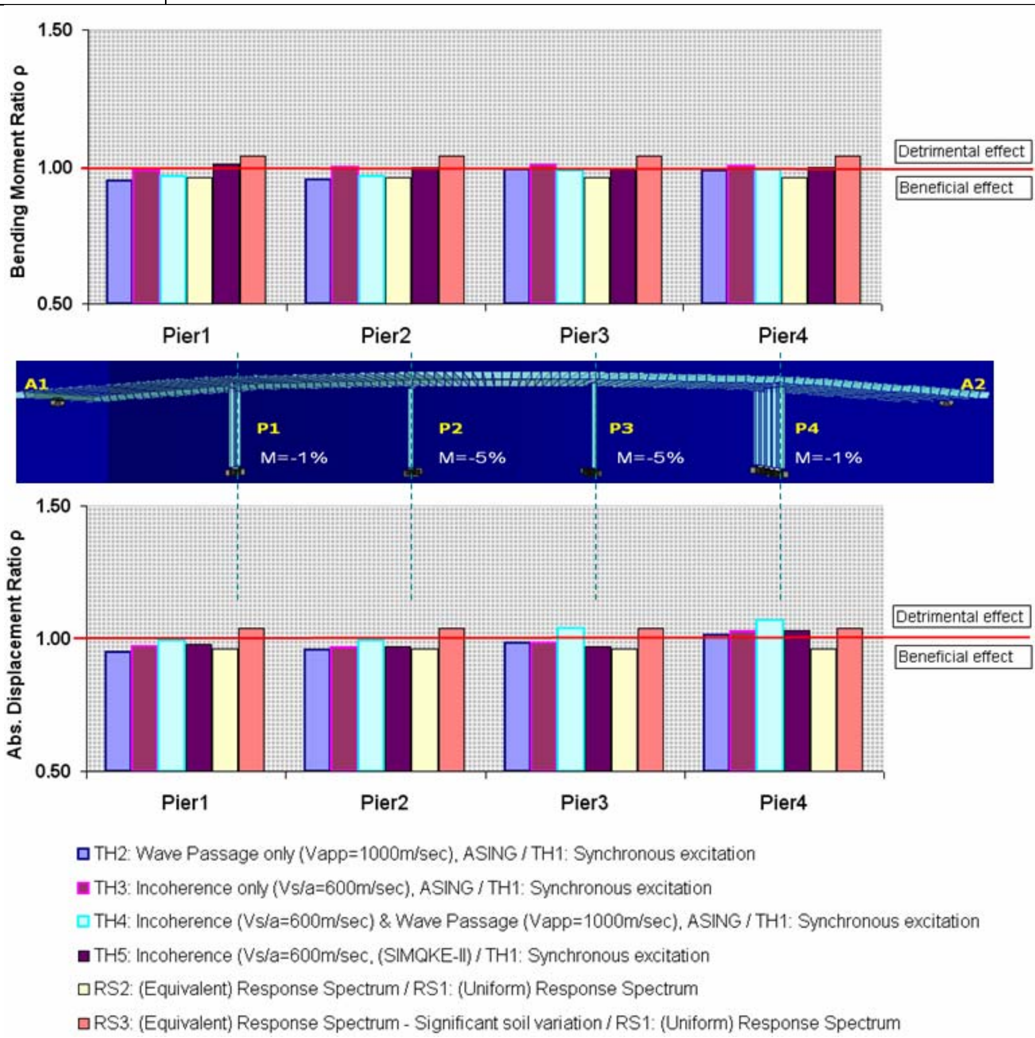


Fig. 7 : Effect on alternative asynchronous motion analyses on deck displacements and pier member forces for the case of Bridge01

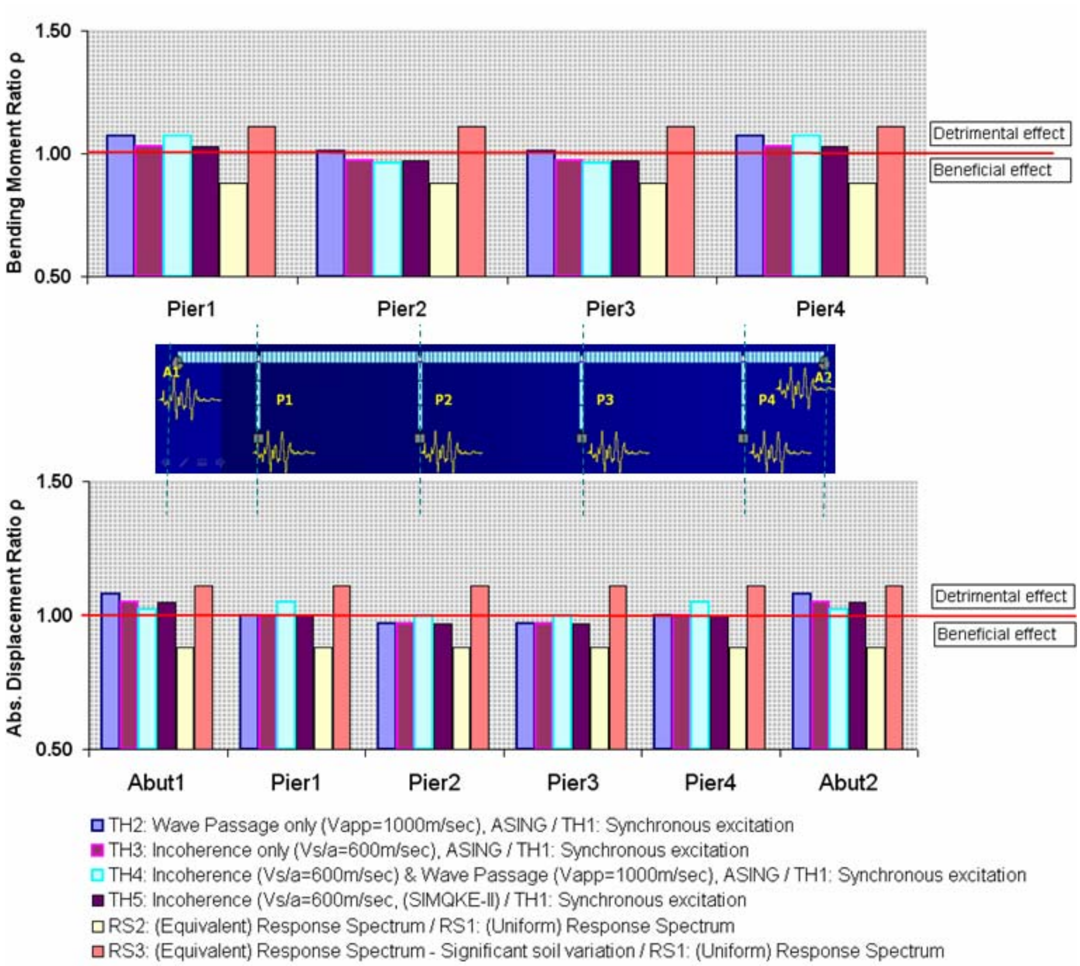


Fig. 8 : Effect on alternative asynchronous motion analyses on deck displacements and pier member forces for the case of Bridge02

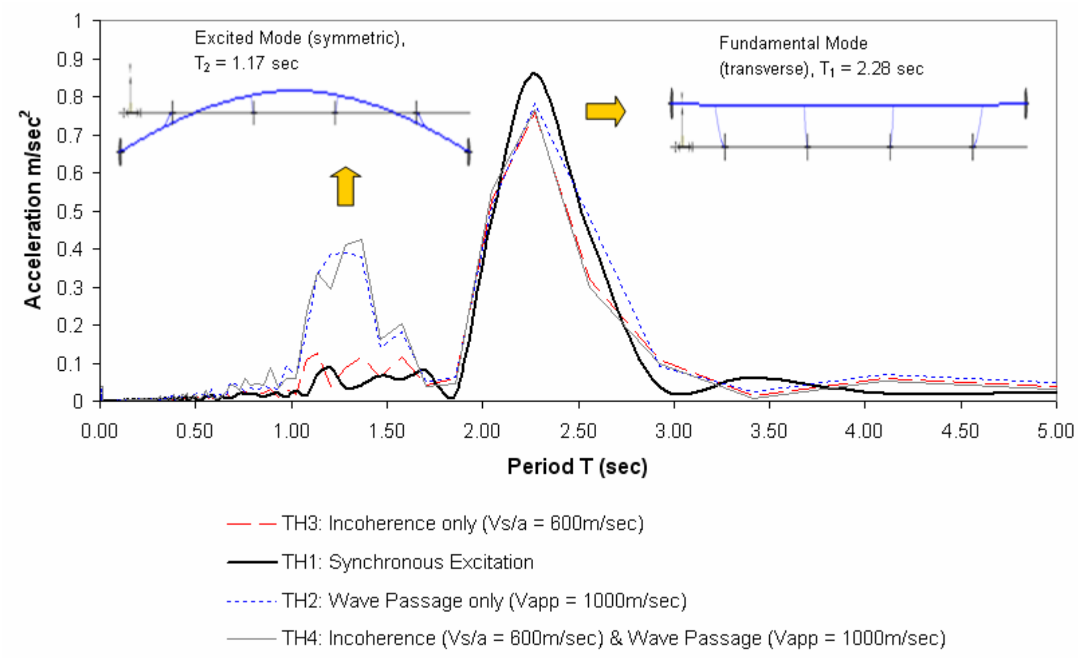


Fig. 9: Excitation of higher modes due to multiple-support excitation for the case of Bridge02

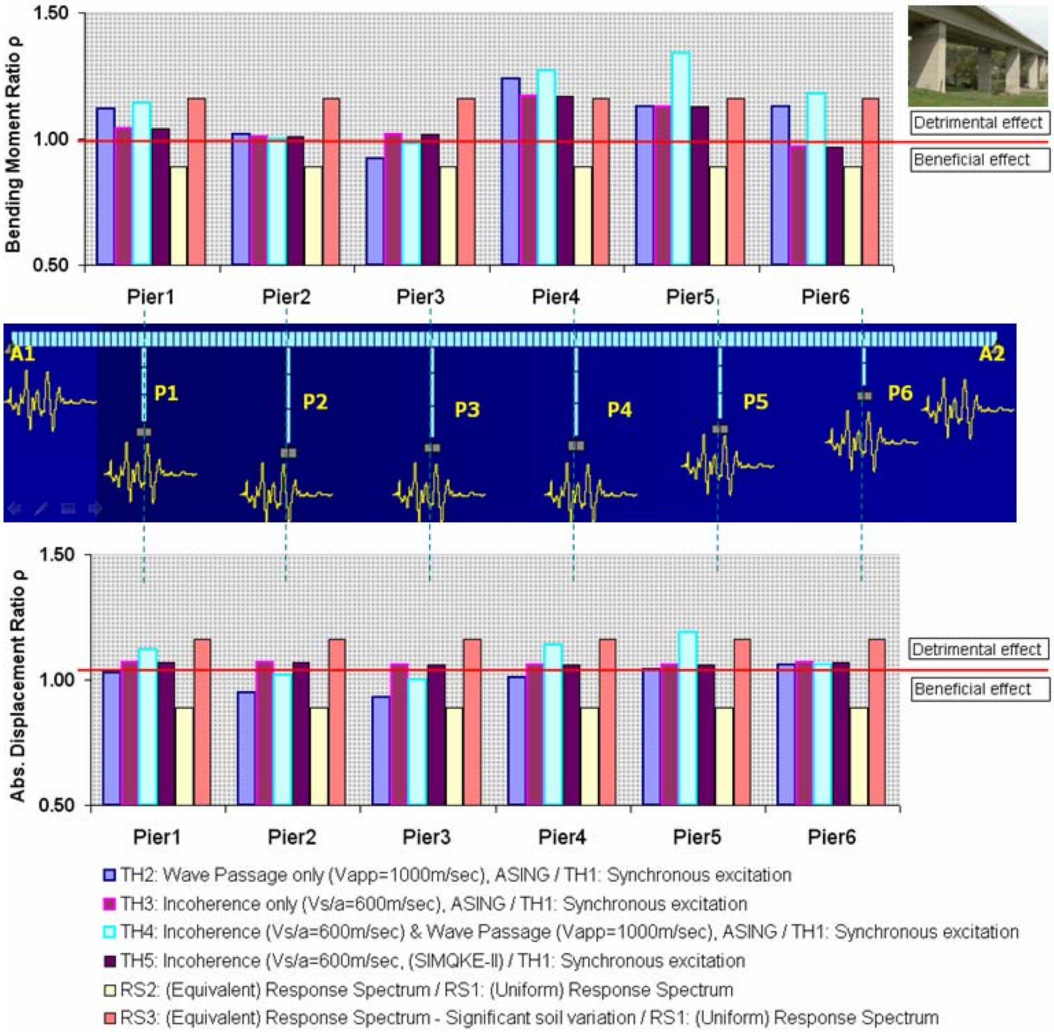


Fig. 10 : Effect on alternative asynchronous motion analyses on deck displacements and pier member forces for the case of Bridge03

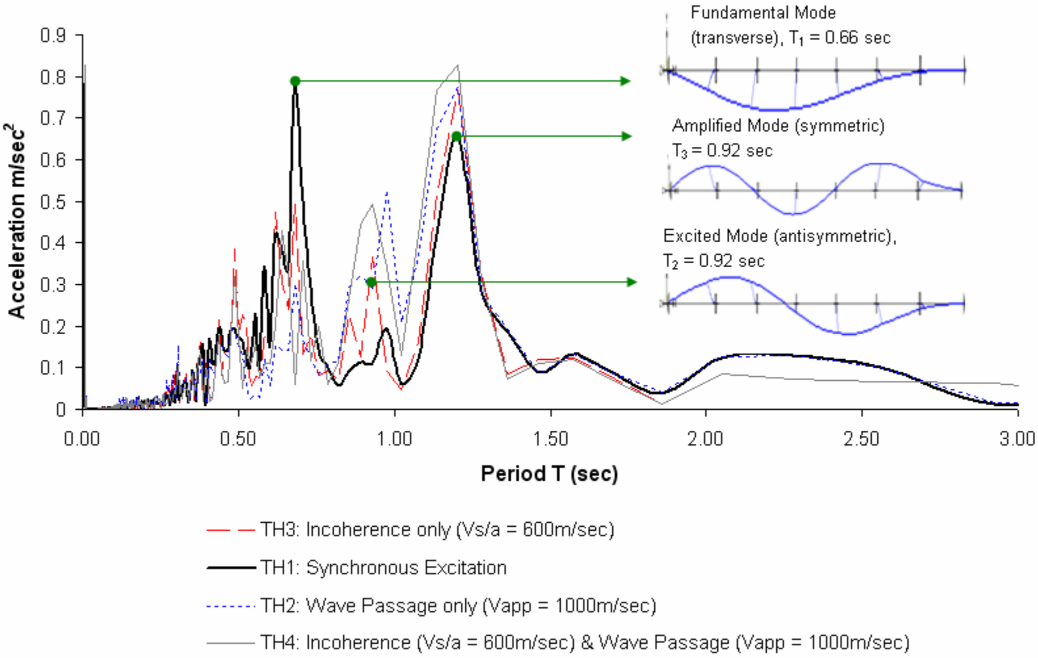


Fig. 11: Excitation of higher modes due to multiple-support excitation for the case of Bridge03

It is also seen that the piers that are more affected (i.e. Piers 1,4 and 5) are exactly the ones that a) their vibration was not dominated by the fundamental (transverse) mode in the first place while b) they were strongly related to the mode shapes triggered. As a result, when the structure is multiply excited, it can be stated that it is essentially 'obstructed' from vibrating at its fundamental mode, but as already denoted in the literature (Sextos et al., 2003b, Tzanetos et al., 2001, Price et al., 2002) the excitation of higher modes leads to a rather unpredictable modification of the dynamic (elastic or inelastic) response of the superstructure. It is therefore concluded that these oscilation characteristics cannot be easily assessed in advance, nor be captured with the use of a single, 'average' response spectrum in a frequency domain analysis scheme.

EFFECT OF SPATIAL VARIABILITY ON THE FOUNDATION DYNAMIC RESPONSE

Another significant implication of the above is that the dynamic response of the foundation is also affected. Indeed, even for cases that the local soil and site conditions are relatively uniform and the subsequent frequency content of the incoming waves does not vary substantially along the bridge length (hence the kinematic interaction is not affected by SVGM), the inertial loads that are transmitted to the foundation by the (significantly modified) oscillation of the superstructure may in turn also be significantly altered.

In order to demonstrate this effect, Bridge04 was subjected to spatially varying ground displacements applied at the foundation level. The assumption was made that each bridge pier was supported on a flexible foundation system, based on the concept of cast-in-drilled-hole (CIDH) pile shaft. It is noted that the actual foundation of the Krystallopigi bridge is footing-type due to the very good soil conditions (Sextos et al., 2004). A diameter of 1.50m was assumed for the CIDH foundation. It is clear that this foundation type is feasible provided that a) the soil conditions are good (which is valid in this case where the soil is classified as Soil Category A according to the Eurocode 8) and b) the system is capacity designed in order to avoid plastic hinge development along the pile (at a depth 1.5 to 2.5 pile diameters) where repair is problematic. Despite the fact that this is not a common foundation type in bridges built in Greece, it was adopted in the context of this case-study in order to avoid the implications arising by the complex frequency-dependent phenomenon of pile-to-pile interaction (Gazetas & Mylonakis, 1998) that is potentially coupled with the local site amplification issues and SVGM and could possibly hide particular aspects of the asynchronous excitation effect.

The CIDH pile was modeled as linear elastic 'Beam on Dynamic Winkler Spring' (BDWS) element and the appropriate coefficients along its depth were calculated according to Makris and Gazetas (1992). The commercial FE packages SAP2000 (CSI, 2001) and ANSYS (ANSYS Inc., 2001) were used for the 3D dynamic analysis. The non-linear static and dynamic performance of the Krystallopigi bridge has been examined thoroughly (Kappos et al., 2005) for various scenarios of excitation inclusive of asynchronous input motion (Sextos, 2004). By extending these studies herein to account for the presence of the CIDH foundation and monitoring the response of the foundation itself, its sensitivity to the assumption of realistic wave propagation was examined. It is very interesting to notice in Fig. 12 that even the slightest spatial variation of ground motion (i.e. fully correlated motions just traveling at a finite velocity within the soil media - loss of coherency and site effects are deliberately neglected in the particular run) the dynamic response of the pile head can be increased by almost 20% in the extreme case of Pier 1 and 11. Apparently, the effect of SVGM is not as significant at the central piers foundation while it is also reduced with depth. Nevertheless, since the variability of the selected motions is indeed minor, more uncorrelated motions are expected to trigger higher structural modes that can in turn lead to increased inertia forces transmitted to the foundation level and further foundation response modification. Clearly therefore, investigation is needed focusing on more complex pile group foundations and using the more refined ground motion scenarios that have already been developed for this bridge.

EFFECT OF BRIDGE IRREGULARITY

When the Kristallopigi Bridge04 is subjected to the aforementioned set of motions, the overall dynamic response of the structure is significantly affected in terms of pier base bending moments. It is seen in Fig. 13 that, especially, along the y-y (i.e perpendicular to the bridge chord) axis, the forces developed are substantially affected by the spatially variable character of motion (a reduction of up to 70% is observed together with an increase that exceeds 100%). As a result, it is clear that asynchronous motion cannot easily be replaced by an alternative 'reference' uniform motion (Zanardo et al., 2002). Nevertheless, this trend seems less dependent of the angle of incidence, verifying the observation made, that the particular curved bridge is far more sensitive to the spatially variable nature of ground motion than to the direction of wave propagation. Extreme cases that the angle of incidence plays a dominant role of the modification of the bridge dynamic response have also been highlighted (Sextos et al.,2004).

EFFECT OF TARGET GROUND MOTION

It was also deemed very important to investigate the relative response of the bridge for different target earthquakes. By subjecting Bridge04 to spatially varying motions that match the (target) spectrum of the Kozani earthquake record and the Eurocode 8 spectrum, it is seen (Sextos et al., 2004) that that the same trends (albeit to a relatively lower degree) presented above are observed for both motions. This observation was also made (Sextos et al., 2003b) by comparing the relative effect of SVGM using the Athens and Loma Prieta record. This by no means implies that the dynamic response of the structure in not sensitive to the frequency content of the input motion used – on the contrary, filtering of incoming waves, damping and soil-foundation-structure dynamic interplay set a strongly frequency-dependent problem. What can be stated though, is that the *relative effect* of accounting for wave passage and coherency loss as compared to the synchronous excitation using the same (unique) record may be more dependent on whether local soil amplification is considered, than on adopting different target earthquake characteristics.

COUPLING OF SSI AND SITE EFFECTS

Within the context of the above comparative analysis, it was considered interesting to investigate whether the conclusions drawn with respect to soil-structure interaction would remain valid if the effects of spatial variability and local soil conditions were neglected. Focusing on 20 different bridges (Sextos et al. 2003b) from which the most interesting is depicted in Fig. 14, the relative effect of SSI in the light of spatially variable (and local site affected) ground motions was studied. The significance is indeed impressive; the response parameters of the bridge accounting for both SSI and local site effects, were all consistently higher, i.e. ignoring the effect of local soil conditions underestimated significantly the results of the soil-structure interaction analysis by approximately 50% for pier top absolute displacements, 30% for relative displacements, 50% for vertical deck displacements, and 40% for bending moments at the pier base.

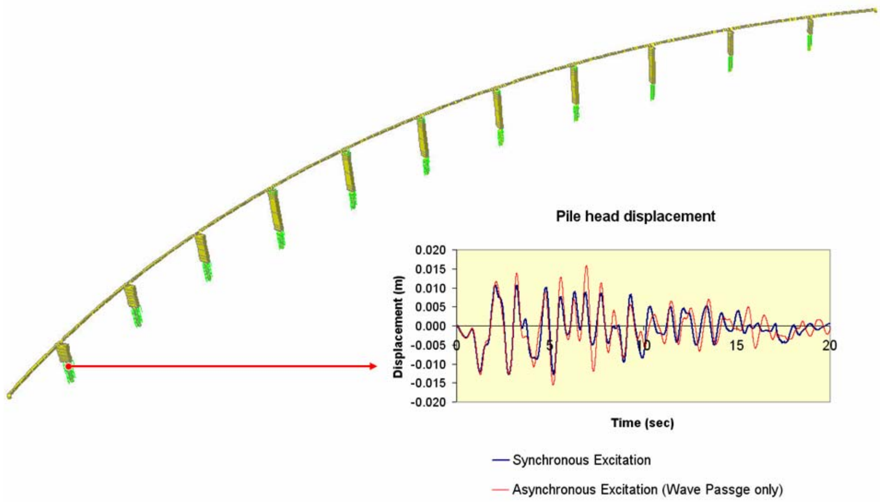
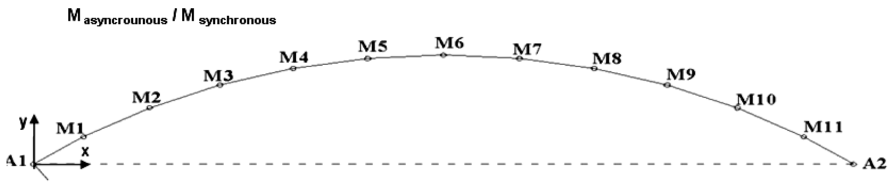


Fig. 12: Dynamic response at the pile head level of Krystallopigi bridge (Pier 1) for synchronous and asynchronous (wave passage only) excitation

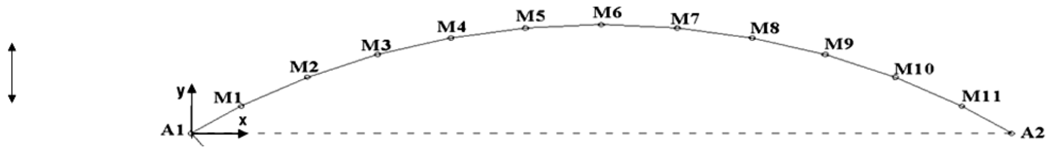
Wave Passage Effect

Response along the x-x axis



propagation direction	motion											
0° - parallel to chord	synchronous	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
0° - parallel to chord	θ+γ(ω)	0.92	0.91	1.18	0.76	0.77	0.77	0.75	0.77	1.00	0.92	0.79
30°	θ+γ(ω)	0.92	0.91	1.18	0.76	0.77	0.77	0.75	0.77	1.00	0.92	0.79
45°	θ+γ(ω)	0.93	0.91	1.16	0.53	0.55	0.57	0.55	0.55	1.00	0.92	0.71
60°	θ+γ(ω)	0.92	0.91	1.18	0.76	0.77	0.77	0.75	0.77	1.00	0.92	0.79
75°	θ+γ(ω)	0.92	0.92	1.17	0.92	0.92	0.91	0.91	0.93	0.99	0.92	0.82
90° -perpendicular to chord	θ+γ(ω)	0.92	0.92	1.17	0.75	0.76	0.76	0.75	0.76	0.99	0.93	0.60

Response along the y-y axis



propagation direction	motion											
0° - parallel to chord	synchronous	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
0° - parallel to chord	θ+γ(ω)	**1.32**	1.16	0.92	0.56	0.33	0.77	0.57	0.48	0.71	**1.67**	0.78
30°	θ+γ(ω)	**1.32**	1.16	0.92	0.56	0.33	0.77	0.57	0.48	0.71	**1.67**	0.78
45°	θ+γ(ω)	**1.27**	1.09	0.77	0.39	0.29	0.68	0.58	0.35	0.73	**1.64**	0.61
60°	θ+γ(ω)	**1.32**	1.16	0.92	0.56	0.33	0.77	0.57	0.48	0.71	**1.67**	0.78
75°	θ+γ(ω)	**1.38**	**1.21**	0.99	0.63	0.38	0.76	0.71	0.64	0.67	**1.62**	0.87
90° -perpendicular to chord	θ+γ(ω)	**1.25**	1.15	0.91	0.55	0.35	0.78	0.58	0.55	0.78	**2.24**	0.77

Fig. 13: Ratio of the asynchronously over synchronously excited bridge pier base bending moments for various angles of seismic wave incidence

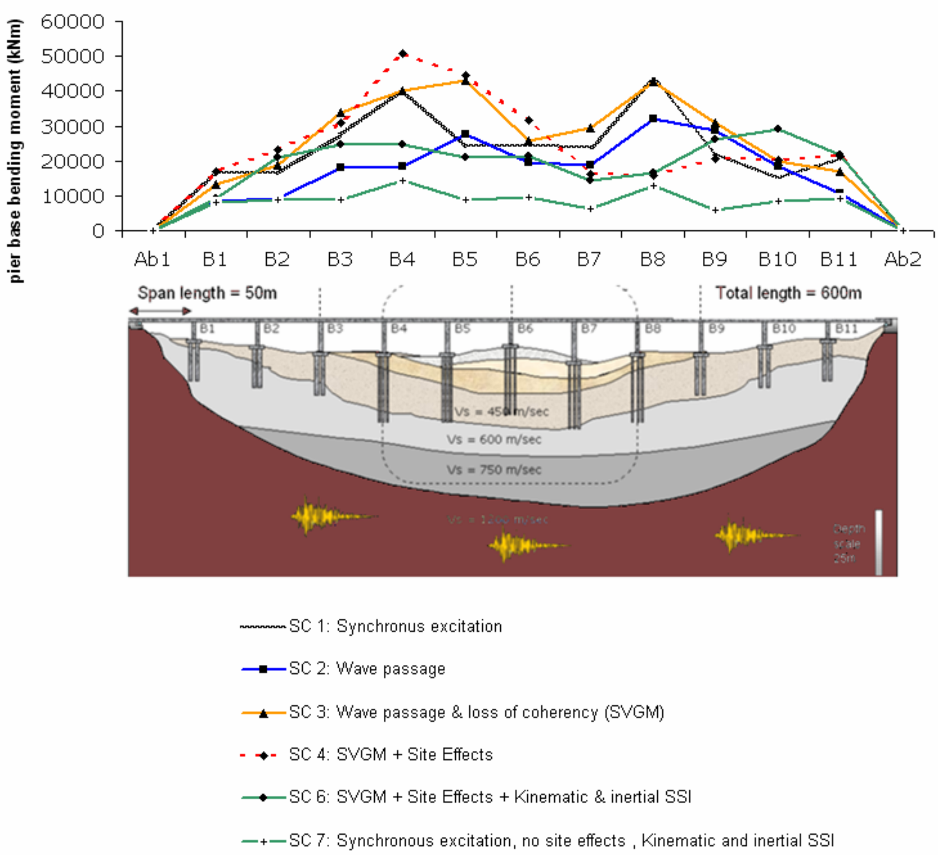


Fig. 14 : Maximum bending moments at the pier base of Bridge05 for various ground motion scenarios

The reason for such differences is that, as soil interacts with the foundation and the structural period elongates, the structure becomes more sensitive to long period pulses which have been amplified due to the presence of the soil (Pitilakis, 2003). Hence it is concluded that a realistic consideration of the multi-layer, damped soil structure is necessary prior to any effort to model the behaviour of the coupled soil-foundation-pier system.

RECENT PROVISIONS OF EUROCODE 8

Despite the major practical interest of generating such motions and the considerable research carried out over the last years, the multi-parametric character and the complexity of the problem has not yet led to the issue of detailed guidelines in modern codes. As a result, the potential effect of asynchronous excitation is only partially considered. In particular, most modern codes deal with the problem solely and rather indirectly, on the basis of seating length provisions. According to the AASHTO code, the required seismic design displacements are determined through any seismic analysis of the bridge provided that the analysis method is acceptable.

The Japanese Code, on the other hand, specifies the seat length S_E of a girder at the support as follows:

$$S_E \text{ (in cm)} = u_R + u_G \geq 70 + \frac{L}{2} \qquad (9)$$

where u_R is the differential displacement between the superstructure and substructure (in cm), u_G is the relative displacement of the ground occurring due to ground deformation between piers (in cm), and L is the clear span length (in m).

An effort to relate the expected relative displacements δ_a of a multiply excited bridge system to the overall length L, has been made through a statistically derived amplification factor R_D proposed by Sextos et al. (2003b):

$$\delta_a = R_D\, \delta_s = (0.8\, \mathit{ln}(L) - 2.8)\, \delta_s \qquad (10)$$

where δ_s are the relative displacements that would result from 'standard' synchronous motion analysis and L is the overall length (in m). Furthermore, a model to compute the differential displacements of points on the ground and on the top of a SDOF linear elastic system has also been proposed by Nuti & Vanzi (2005), while Kawashima and Sato (1996) suggested an alternative approach based on the use of a 'relative displacement spectrum'.

Along these lines, the latest version of Part 2 of Eurocode 8 seems to be the only seismic code worldwide that provides such a clear and detailed framework for considering the effect of spatial variability of ground motion in bridge design, through both a simplified and an analytical approach, the latter being included as an 'informative' annex. The corresponding code-prescibed Set A and Set B imposed static displacements are shown in Fig. 14 and 15 below. The accuracy and range of applicability of the new EC8 provisions, with emphasis on the simplified procedure proposed in its main body (the one expected to be used for practical design) is assessed in Sextos and Kappos (2005). An extension of the ASING program is also presented that calculates the EC8-prescribed displacements sets, facilitates their automatic import at the supports of finite element models developed in ANSYS, performs the subsequent static analyses for both displacement scenarios and returns the resulting additional action effect of the selected bridge members.

CONCLUSIONS

The scope of this paper is to review the recent developments on the sensitivity of bridges to the spatial variation of ground motion inclusive of site effects and soil-structure interaction. Numerical examples and analytical results are given and the relative effect of the

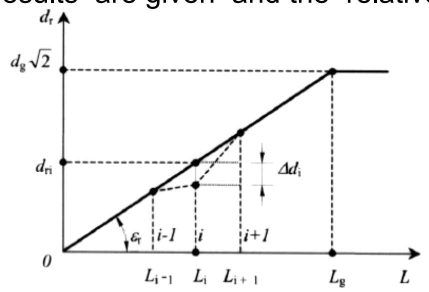


Fig. 14: Set A imposed displacements according to Eurocode 8 – Part 2 to account for SVGM

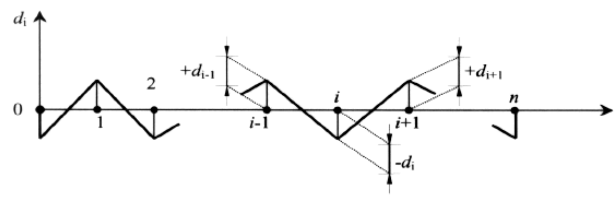


Fig. 15: Set B imposed displacements according to Eurocode 8 – Part 2 to account for SVGM

various phenomena involved as well as of the assumptions made to account for these phenomena are thoroughly examined. it is concluded that the problem is multi-parametric and complex but certain situations can be identified where the assumption of identical excitation between support points strongly underestimates the imposed ductility demand both at the foundation and the superstructure.

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