EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS
Earthquake Engng Struct. Dyn. 2003; **32**:607–627 (DOI: 10.1002/eqe.241)

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

Anastasios G. Sextos[1,†], Kyriazis D. Pitilakis[1,*,‡] and Andreas J. Kappos[2]

[1] *Laboratory of Soil Mechanics and Foundation Engineering, Department of Civil Engineering, Aristotle University of Thessaloniki, Thessaloniki, GR-54006, Greece*
[2] *Laboratory of Reinforced Concrete, Department of Civil Engineering, Aristotle University of Thessaloniki, Thessaloniki, GR-54006, Greece*

SUMMARY

During strong ground motion it is expected that extended structures (such as bridges) are subjected to excitation that varies along their longitudinal axis in terms of arrival time, amplitude and frequency content, a fact primarily attributed to the wave passage effect, the loss of coherency and the role of local site conditions. Furthermore, the foundation interacts with the soil and the superstructure, thus significantly affecting the dynamic response of the bridge. A general methodology is therefore set up and implemented into a computer code for deriving sets of appropriately modified time histories and spring–dashpot coefficients at each support of a bridge with account for spatial variability, local site conditions and soil–foundation–superstructure interaction, for the purposes of inelastic dynamic analysis of RC bridges. In order to validate the methodology and code developed, each stage of the proposed procedure is verified using recorded data, finite-element analyses, alternative computer programs, previous research studies, and closed-form solutions wherever available. The results establish an adequate degree of confidence in the use of the proposed methodology and code in further parametric analyses and seismic design. Copyright © 2003 John Wiley & Sons, Ltd.

KEY WORDS: bridges; spatial variability; site effects; soil–structure interaction; seismic design

INTRODUCTION

Seismic design of important bridges is increasingly performed using dynamic analysis in the time domain, wherein the response of the structure to appropriately selected and scaled time histories is strongly dependent on three key simplifying assumptions that are often made: (i)

* Correspondence to: Kyriazis D. Pitilakis, Laboratory of Soil Mechanics and Foundation Engineering, Department of Civil Engineering, Aristotle University of Thessaloniki, Thessaloniki, GR-54006, Greece.
† E-mail: asextos@geo.civil.auth.gr
‡ E-mail: pitilakis@evripos.civil.auth.gr

Received 10 December 2001
Revised 20 June 2002
Accepted 30 July 2002

the seismic motion that is transmitted to the structure through its supports is synchronous and identical for all piers and abutments, (ii) the local site conditions are accounted for in terms of site categorization, and (iii) the superstructure is fully fixed at the pier base points.

The rationale behind such assumptions (of which (i) and (ii) are very common in current design practice) is not the certainty that they lead to conservative design, but rather that attempting to incorporate more complex models leads to a multi-parametric procedure, often uneconomic and sometimes numerically sensitive. On the other hand, observations from strong earthquakes and results of research relevant to the above assumptions provide ample evidence that:

(i) Earthquake ground motion may significantly differ among the support points, especially for long bridges, in terms of amplitude, frequency content and arrival time, thus inducing under certain circumstances significant forces and deformations [1, 2]. These spatial and temporal variations of seismic motion can be primarily attributed to the *travelling of the waves* at a finite velocity, *Loss of their coherency* in terms of statistical dependence, i.e. due to multiple reflections, refractions and superpositioning of the incident seismic waves propagation, *Effect of local soil conditions* and *Attenuation of motion* due to geometrical spreading of the wave front and the loss of kinematic energy [3]. Additionally to the above, seismic motion is further modified by the foundation, depending on its relative flexibility with respect to the soil, since the foundation is not always able to vibrate according to the displacement field that is imposed to it by the incoming waves.
(ii) Local site conditions have a much more complex effect than the spectral modification prescribed by the code design spectra. For multi-layer damped soil columns, both peak ground acceleration and frequency content at the surface motion are strongly dependent on soil and site conditions and the velocity contrast between the bedrock and the overlaying layers [4, 5].
(iii) Piers are clearly not fixed at their base as 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) leading to a very complex and case-dependent dynamic response [6, 7].

Notwithstanding the extensive research [8] carried out over the last 20 years in all the aforementioned fields, limited number of studies exist involving a comprehensive approach for the coupling of soil–structure–interaction (SSI), spatial variability and site effects even in pairs, i.e. spatial variability and site effects [3, 9] spatial variability and SSI [10] spatial variability and inelastic response of bridges [11, 12] and SSI with inelastic analysis [13], while the study with the broadest scope known to the authors is that by Simeonov *et al.* [14].

Moreover, the results of research on each of the above phenomena, although extensive, are only partially reflected in modern seismic codes. As far as spatial variability is concerned, with the exception of Eurocode 8, Part 2, for bridges [15] that provides an expression for the relative displacement of adjacent piers and an informative annex for spatial variability analysis, modern codes either treat the problem on the basis of seating length provisions, such as the U.S. Standard Specifications for Highways and Transportation Bridges [16] and ATC-32 [17] or do not address the problem at all, like the Japanese Design Specifications for Highway Bridges [18].

A major discussion is also still open with respect to the feasibility of a more refined soil categorization and its implication for the corresponding amplification factors in the design spectra [4, 19] resulting only recently into the incorporation of a more refined site categorization in UBC and EC8. SSI on the other hand, is often treated as a beneficial phenomenon by ATC-3 [20] and FHMA provisions [21] on the basis of the anticipated period elongation of the structure, as well as the energy dissipation at the foundation level caused by wave radiation and hysteretic damping. These two factors are assumed to always lead to reduced design acceleration and base shear values even though, under certain circumstances, it has been showed that this is not the case [13]. It is only in Eurocode 8 that foundation flexibility coefficients are provided, but only as an informative annex.

The reason for such a lack of detailed guidelines with respect to the above three issues is their significant complexity as well as the fact that the results derived even by the study of a particular phenomenon, are case dependent and often contradictory. The above, together with the few cases of comprehensive studies on the coupling of spatial variability, SSI and site effects and the limited number of available recordings on the bridge response under seismic loading, call for a design on a case-by-case basis, hence requiring extensive validation in order to ensure accuracy and stability.

OVERVIEW OF THE ADOPTED COMPREHENSIVE METHODOLOGY

The methodology adopted herein and the analytical tool developed for its implementation aim to introduce a comprehensive global approach to the seismic design and/or assessment of bridges that would allow for the application of a parametric analysis scheme, hence for the systematic study of the parameters involved. This is achieved by incorporating and uncoupling all important issues (asynchronous motion, site effects, SSI) within the context of a feasible general scheme for the inelastic analysis of bridges in the time domain.

The idea is to first generate synthetic time histories, distinct at each support point (piers and abutments), through a refined spatial variability model which accounts for wave passage, loss of coherency and site effects. Next, further modification of motion in the frequency domain allows for the consideration of kinematic interaction between soil and the foundation piles. The derived motion can then be used as the asynchronous input to the bridge structure which is assumed to be supported on different beam-on-dynamic-Winkler-spring systems (BDWS), whose dynamic impedance matrices are derived for all horizontal, rocking and coupled modes of vibration. For the rotational stiffness in particular, a non-linear moment–rotation relationship is proposed in order to combine the rotational compliance of the foundation with a lumped plasticity model for the RC section that accounts for the plastic rotations caused by yielding at the pier base.

Having obtained different time histories and the linear/non-linear spring–dashpot systems for all support points, dynamic analysis of the superstructure can then be performed with the use of any commercial finite-element (FE) code, without the requirement of complex (and often prone to errors) FE modelling of wave propagation, site response and SSI, and without the requirement for advanced concrete model features. Consequently, the study of the bridge sensitivity to the above phenomena may be performed though inelastic dynamic analysis and, if necessary, a Monte-Carlo or directional simulation scheme.

STRUCTURE OF THE PROPOSED PROCEDURE

Step 1: Spatial variability

Two main approaches can be adopted for spatially correlated time histories depending on the available soil data and the accepted degree of complexity. In the simplest case that may be called 'Approach A,' and which is the most widely used in the literature [1, 11] it is assumed that the subsoil along the support points is uniform, thus being described by a common power spectral density function $S_0(\bar{\omega})$, either in the form of a Kanai–Tajimi power spectrum passed through a white noise soil filter or in accordance with the frequency content of a selected earthquake, in both cases compatible with the desired peak ground acceleration. The required coherency function $|\gamma(\omega, \xi)$, i.e. the smoothed cross-spectrum of motions between two points normalized to the corresponding power spectra, should then be selected among the coherency loss models available in the literature (i.e. References [22–24]).

The possibility of using more than one loss of coherency curves may be considered, since the aforementioned experimentally derived relationships refer to different site, soil and earthquake characteristics, hence it is not obvious that they can be used elsewhere without affecting the dynamic response of the bridge or the level of the induced relative displacements.

Although numerous methods exist for assessing the frequency-dependent apparent propagation velocity (that affects the rate of decay of most coherency functions) from seismic data, the complexity of the problem is such that it is rather difficult to collect all the soil stratification and wave propagation information required for such an analysis (i.e. source, ray and path related data) within the framework of the design process. As a result, the apparent velocity may be generally taken to be frequency independent, i.e. expressed as the (average) shear wave velocity between two locations divided by the angle of vertical incidence.

The uniform soil approach that accounts only for wave passage and loss of coherency and neglects the effect of local soil conditions has been widely used in the literature because it permits a quick sensitivity analysis or can be performed whenever the available soil parameters are limited. Nevertheless, it might be inadequate especially in cases of bridges crossing rivers where the abutments are often located on relatively stiff soil while the middle piers are founded on softer deposits. Therefore, an alternative procedure could be followed as proposed by Deodatis [25], 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 ('Approach B'). In particular, by using the spectral representation-based algorithm and writing the simulated stochastic process in a form that takes advantage of the fast Fourier transform, it is feasible to obtain different acceleration time histories at each point that are compatible not only with the prescribed loss of coherency patterns but also with the site-dependent response spectra.

Independently of the method used, and having computed the stochastic vector the desired shape and duration, i.e. the non-stationarity, of the acceleration is obtained by an appropriate modulating function which could be even code-defined [15].

At this point, it is also important to select the desired duration of the signal, especially when inelastic dynamic analysis of the bridge is sought. Duration may be either directly selected or related to the expected magnitude, hence for a given epicentral distance and attenuation relationship to the target acceleration. Zero final velocity and displacement is also achieved by applying baseline correction. It is also suggested to perform an iterative optimization procedure by updating the target power spectral density function as proposed by Gasparini and VanMarcke [26] for one-dimensional, uni-variate stochastic vector process but in a sequence that adjusts the spectra prior to modulation [12]. Having obtained relatively stable mean values of the ground response characteristics by appropriately repeating the above procedure, the achieved coherency, compared to the target one, may be derived using an 11-point Hamming window for the frequency smoothing, an assumption which is considered reasonable [23] for time windows of less than 2000 samples and structural damping approximately equal to 5%. The corresponding velocity and displacement time histories can then be derived through single and double integrations, respectively.

Another important aspect is the investigation of the applicability of the above procedure for motions applied along different axes of the bridge. This is of particular importance since the bridge's structural system is usually different in the transverse and longitudinal direction. As a first simplification, the relative orientation of the wave propagation direction with respect to the orientation of the bridge can be accounted for by modifying the phase angle of the motion. Moreover, the spectral density function may be taken identical for both directions, based on the assumption that soil homogeneity and isotropy produce directionally independent site effects at least for vertically propagating S-waves, as typically assumed for engineering analysis purposes.

The loss of coherency pattern is also assumed common in the two directions, not only because this simplifying assumption is in line with the vast majority of existing proposals but also because it has been verified through experimental observations [1], although later studies [2] consider coherency as path dependent, hence direction dependent. The procedure adopted herein was to assume, that the use of the above procedures in the two directions is valid, but carry out separate simulations, for the two bridge axes in order to ensure that the corresponding motions will be fully uncorrelated.

Step 2: Site effects

The site-dependent spectra 'Approach B' is obviously superior to the uniform power spectral density matrix assumption ('Approach A') even if the site-specific spectra used are code based (e.g. Reference [27]). In the case of code spectra, though, the accuracy of the approach is strongly related to soil profile complexity of the actual problem. Indeed, it is widely recognized that for multi-layer damped soil columns, both peak ground acceleration and frequency content at the soil surface motion are strongly dependent on soil conditions in a way that is not satisfactorily described in most codes.

Along these lines, within the context of the adopted methodology, 'Approach B' is extended to explicitly include the presence of different soil deposits at each location by allowing for multiple soil layers with varying stiffness, damping characteristics and boundaries that reflect and transmit elastic wave energy.

At a first level of complexity ('Approach C'), this goal is achieved by selecting the target outcrop frequency content, generating a sample motion compatible with the corresponding (outcrop) power spectrum, and deriving the bedrock Fourier spectrum through a deconvolution process. For simplicity, the deconvolution procedure may be approximated by assuming a factor of 2 between the surface and the bedrock Fourier spectral amplitude. This is a rather simple way to account for the 'free surface effect,' but is also a reasonable assumption for a non-weathered rock formation and for frequencies higher than 2 Hz [28]. In case that the dominant frequency of the motion does not lie within the above range, a complete deconvolution analysis has to be applied. 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 A.' Then, the distinct surface motions at each support point may be derived through multiple one-dimensional (1D) site response analyses. 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 [29], in which all multiple reflections and conversions between wave types are retained in part of the soil structure.

The procedure described as 'Approach C' can be considered as the state of the art in current research [8]. However, it is unclear to which extent numerical methods that start with the motion defined at the bedrock of a basin and propagate vertically, can reproduce realistic two-dimensional (2D) incoherence. Therefore, 'Approach C' is recommended for bridges of short length with piers located on significantly varying soil conditions where the local motion amplification is expected to be a more important spatial variability parameter than the distance-dependent coherency loss.

For the general case that wave passage, coherency decay and local soil conditions are equally important, 'Approaches B' and 'C' may be combined in a more refined hybrid spatial variability and site effects procedure ('Approach D'). The target bedrock motion can be defined first, whereas multiple independent site response analyses can be performed at each pier location to derive the corresponding target free-field response spectra. Apparently the 1D 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 together with a prescribed coherency decay model in a pure 'Approach B' procedure, leading to spatially variable motions that reflect both the desired frequency content and coherency pattern. Moreover, by uncoupling the problem and isolating the effect of local soil conditions, it is possible, for bridges that cross irregular topography, to perform a 2D site response analysis which captures potential spectral amplification of motion due to lateral wave propagation.

Independently of the complexity of the site response analysis, approaches 'C' and 'D' are expected to be more reliable than 'A' and 'B' since they account for the presence of multi-layered, damped soil profiles lying over an elastic bedrock and represent to the highest possible degree the parameters that affect the motion amplification/deamplification. Nevertheless, they should be adopted only whenever the required geometric and material properties of the soil structure can be accurately measured or reliably estimated; otherwise, a disproportional degree of uncertainty may be introduced. In such a case, 'Approach B' is recommended.

Step 3: Soil–structure interaction stage

The interaction of the foundation with the surrounding soil and the superstructure is another important aspect of the proposed procedure that can be efficiently dealt with by uncoupling kinematic and inertial interaction. The surface seismic motion derived at all support points in the previous steps using any of the suggested four approaches, can subsequently be further modified to account for the scattering of the incoming waves by the foundation (kinematic interaction between soil and foundation).

The assumption of an equivalent uniform soil around the pile length is obviously a simplification but is considered to be a reasonable approximation for deep soil deposits that far exceed the length of the piles.

The relative flexibility of soil and piles is a crucial parameter with regard to the extent of kinematic ground motion modification. Along these lines, it is proposed within the context of the proposed methodology that at least approximately, the inelastic response of both the RC piles and the soil should be accounted for. In particular, the stiffness of the pile as well as that of the soil may be reduced by up to 50 and 65%, respectively, both depending on the expected strain level. Notwithstanding the limitations of such a crude assumption, this is deemed necessary, not only because it has been widely shown that at strains of the level of 10^{-2} the soil shear modulus G may be reduced even to 0.2 of its initial ($G_{\max}$) value, but also because the uncertainty related to the accurate definition of the above modification is less significant than the uncertainty induced by completely ignoring the effect. Moreover, such a stiffness adjustment is valuable within the context of a parametric analysis since pile section and soil stiffness reduction are expected to have opposite effect on the extent of kinematic interaction. Along these lines it is proposed to relate the soil stiffness $V_s/V_{s\,\max}$ and damping ζ (%) values, to the selected peak ground acceleration level a (where $a<0.3$ g) by curve fitting to the relevant values proposed in Eurocode 8:

$$V_s/V_{s\,\max} = \sqrt{41.6a^3 - 17.5a^2 - 0.66a + 1} \quad \text{and} \quad \zeta = 0.0319e^{4.082a} \tag{1}$$

while the effective stiffness $\mathrm{EI_{eff}}$ of the cracked RC pile section for a peak ground acceleration level $\alpha > 0.1$ g may also be reduced according to the empirical relationship:

$$\mathrm{EI_{eff}/EI} = 1 - 0.72\sqrt{a - 0.1} \tag{2}$$

The kinematic *pile group* effect may be considered insignificant at least with respect to the modification of the motion, hence it is a reasonable assumption to be taken approximately the same as the kinematic effect on a single pile [30].

Having uncoupled kinematic and inertial interaction, the different pile head displacement time histories at each location that have resulted from spatial variability, site effects and kinematic interaction are applied as *foundation input motion* to the n piers which are then assumed to be supported by springs and dashpots which reflect the flexibility and damping of the soil–foundation system under dynamic loading. For the case of single piles, the required static stiffness matrix is first derived, on the basis of the relevant flexibility coefficients for coupled horizontal and rocking modes of vibration which can be calculated through closed-form equations, among the many available in the literature [31, 32].

For the case that the foundation consists of an $n \times m$ pile group, the above equations are modified in order to account for the waves that are emitted from the piles and propagate towards the other members of the group. For this purpose, the complex dynamic interaction factors $\alpha_{ij}^{\mathrm{dyn}}$ are calculated for all modes of vibration, incorporating the most widely used expressions in the literature [33, 34].

Both kinematic and inertial soil–pile and pile-to-pile interaction are strongly frequency dependent. Nevertheless, it is assumed that the complex dynamic impedance matrix is calculated based on the predominant frequency of the input motion whenever the FE code used does not support frequency dependent elements. This assumption is rather common in the literature but it may not be accurate under certain conditions [30]. Hence, it is suggested to supplement it with a sensitivity analysis using alternative dominant frequencies, at least in cases that the input motions are rich in a wide range of frequencies. In such a way an envelope of the overall bridge response may be derived.

The above uncoupling of kinematic and inertial interaction may also be applied in the case of shallow bridge foundations, utilizing spring and dashpot expressions that account for the soil-footing system flexibility and damping [6, 35].

Step 4: Inelastic dynamic analysis

Having defined the seismic input and the foundation dynamic impedance matrix for each support point, and depending on the potential of plastic hinge development at the base of the pier, a lumped plasticity model is adopted for the pier involving an elastic beam with inelastic springs located at its ends. According to this well-established approach, the model consists of inelastic rotational springs that are connected to an elastic beam in a series system. As a result, the total rotation of the beam end is equal to the plastic rotation of the springs in addition to the rotation due to bending of the elastic beam. The effective stiffness (rigidity) of the beam, is taken as $\mathrm{EI}_{\mathrm{eff}} = M_{\mathrm{y}}/\varphi_{\mathrm{y}}$, where M_{y} is the yield moment and φ_{y} is the corresponding curvature derived from fibre analysis of the pier section. Based on the pier geometry and the selected stress–strain relationship of the confined section, the constants for the rotational springs are calculated. These springs are activated only whenever the developed bending moment exceeds the yield moment of the section and follow a non-linear force-displacement law that is a function of the RC section geometric and material properties (Figure 1). The critical point is the definition of the rotational capacity θ_{pu}, that can be estimated by multiplying the plastic curvature by an equivalent plastic hinge length [36].

By combining the flexibility of the non-linear pier-base inelastic spring and the linear rotational foundation spring that was calculated in the inertial SSI stage, the final rotational spring (Figure 1) is derived, being characterized by a first branch (uncoupled rotational) stiffness equal to $\Im_{\Theta}$ and a second branch stiffness $\Im'_{\theta}$ equal to

$$\Im'_{\theta} = \frac{1}{\frac{1}{\Im_{\theta}} + \dfrac{\theta_{\mathrm{p}}}{M_{\mathrm{u}} - M_{\mathrm{y}}}}$$

$$= \frac{1}{\mathrm{Re}\left[\dfrac{K_{HH}^{\mathrm{dyn}} - K_{HM}^{\mathrm{dyn}}/e}{K_{HH}^{\mathrm{dyn}} K_{MM}^{\mathrm{dyn}} - K_{HM}^{\mathrm{dyn}\,2} + K_{\theta V}^{\mathrm{dyn}} K_{HH}^{\mathrm{dyn}}}\right] + \dfrac{(0.08L + 0.022 f_{yl} d_{bl})(\phi_{\mathrm{u}} - \phi_{\mathrm{y}})}{M_{\mathrm{u}} - M_{\mathrm{y}}}} \tag{3}$$

where θ_{p}, M_{u}, M_{y} are the plastic rotation, the ultimate and the yield moment of the pier base RC section, respectively, $K_{HH}^{\mathrm{dyn}}, K_{MM}^{\mathrm{dyn}}, K_{HM}^{\mathrm{dyn}}$ are the horizontal, rocking and coupled modes of vibration terms of the dynamic stiffness matrix, which for the case of pile groups are functions of the damping coefficients $\zeta_{HH}, \zeta_{H}, \zeta_{MM}$ and the dynamic interaction factors $\alpha_{ij}^{\mathrm{dyn}}, K_{\theta}^{\mathrm{dyn}}$ is the (static) rotational stiffness component attributed to the antisymmetric vertical loading of the piles [31], $e = H/M$ is the foundation eccentricity, L is the distance from the critical pier

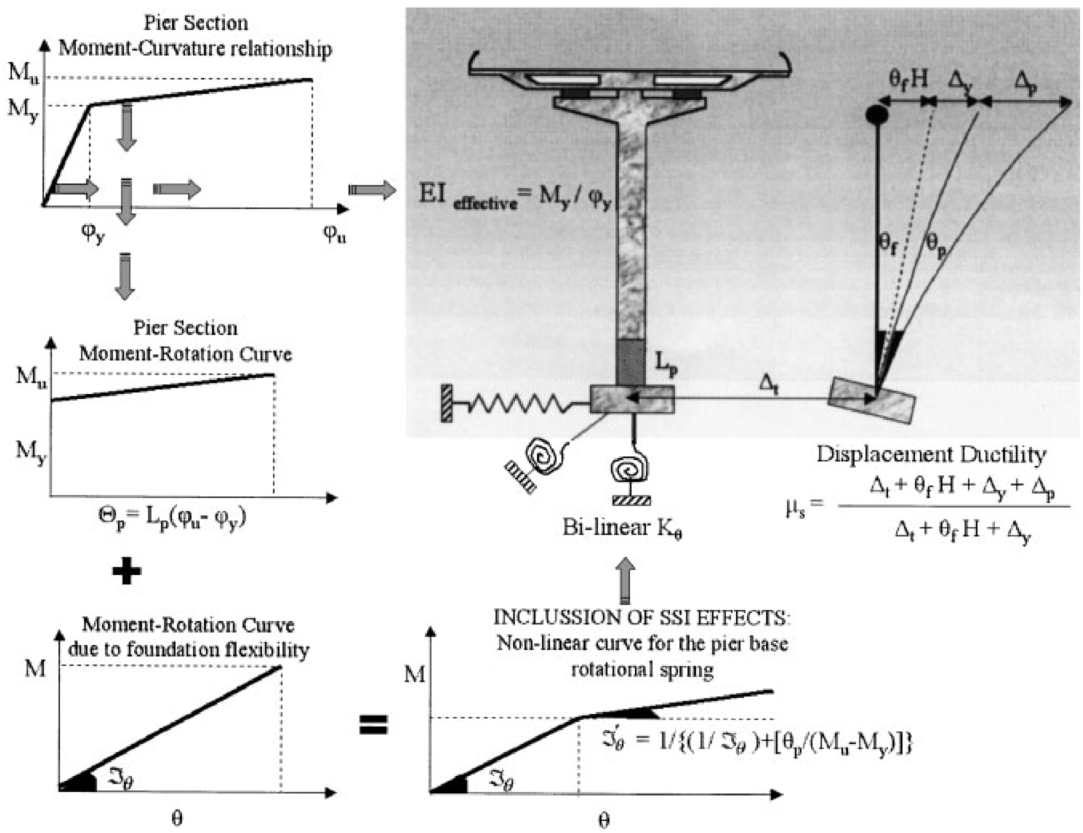

Figure 1. Combination of SSI and post-yield RC pier response.

section to the point of contraflexure, f_{yl} is the yield strength of the longitudinal bars and d_{bl} is the diameter of the longitudinal reinforcement.

With the complete set of linear and non-linear pier base springs and the distinct acceleration or 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 SSI. The ductility demand in the cantilever pier (typically for excitation in the transverse direction) in the presence of the foundation flexibility and damping may then be evaluated through a well-established approach [36] as shown in Figure 1. For the case that the non-linear behavior is expected to be concentrated at the pier top, the proposed procedure can still be applied by retaining the non-linear rotational spring at the pier's base while extending the linear rotational springs which are activated when $M > M_y$ along the pier length, in order to capture the plastic hinge wherever this is developed. Such an approach of non-prescribed locations of inelastic behavior concentration has successfully been followed and tested [37] not only for the pier but also for the foundation piles.

IMPLEMENTATION INTO A COMPUTER CODE

In order to verify the feasibility of applying the above procedure but also for evaluating, through parametric analysis, the bridge sensitivity to spatial variability, site effects and SSI, the computer code ASING (Asynchronous Support Input Generator) was developed. The algorithm, whose structure is presented in Figure 2, follows exactly the four-step methodology presented above, allowing for 'Approaches A' and 'B' of the spatial variability step and 'Approaches C' and 'D' of the site effects step, while employing the coupled SSI and non-linear RC section features.

The general data required are related to ground motion (PGA, duration, angle of incidence, direction of wave propagation with respect to the bridge axis) and its frequency content (Kanai–Tajimi spectrum, power spectrum or response spectrum, definition of free-field, bedrock or outcrop location of target spectrum, potential 2D site effects filter), while the pier-specific information refers to soil conditions (thickness, density, damping, shear wave velocity, loss of coherency model), foundation properties (single pile/pile group, material damping, Poison ratio, concrete category, diameter, pile length, group geometry, s/D ratio) and pier RC section moment–rotation relationship (ultimate/yield moment and rotation).

The output, as described above, is spatially variable time histories and spring/dashpot coefficients at all support points. The methodology and the code developed for its implementation are deemed to offer the following advantages:

(a) It is a comprehensive but feasible procedure that combines spatial variability, site effects and SSI for the inelastic analysis.
(b) The site-dependent spectra approach [25] is extended by accounting for multi-layered, damped soil profiles over an elastic bedrock, while the foundation dynamic flexibility matrix approach [30, 34] is extended by considering pier base yielding (in the efficient form described by Equation (3)).
(c) Spatially modified time histories and the spring–dashpot coefficients are provided at each support point, hence allowing inelastic dynamic analysis of RC bridges to be performed accounting for the above phenomena with any commercial FE code and without requiring enhanced soil, wave and concrete features, and/or refined modelling.
(d) By offering various combinations of analysis complexity (i.e. readily shifting between approaches, formulae and parameter values) the overall comprehensive methodology and code permit carrying out extensive parametric analyses that target to envelope the dynamic response of a bridge, a fact which is of particular importance for such complex, coupled and multi-parametric phenomena.

VALIDATION OF THE PROPOSED PROCEDURE

In order to check the reliability and the limitations of the proposed procedure and the ensuing program ASING, extensive tests were performed for all stages of the methodology, leading to a series of partial models and individual tests using recorded data, FE analyses, alternative (non-FE) computer codes, results from previous research studies and closed-form solutions in all available combinations.

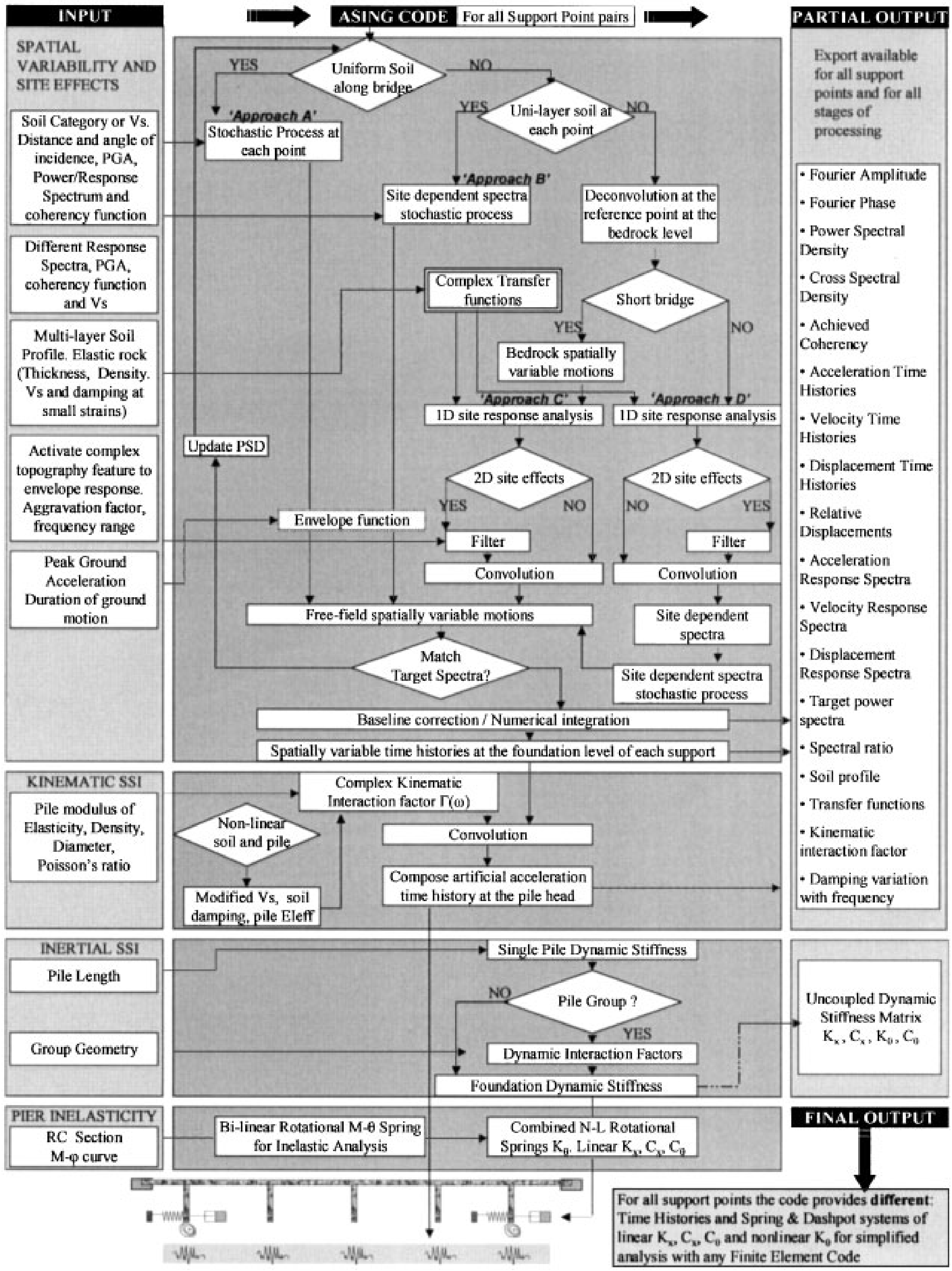

Figure 2. General methodology for the dynamic analysis of bridges under the combined effect of local site conditions, spatial variability and SSI.

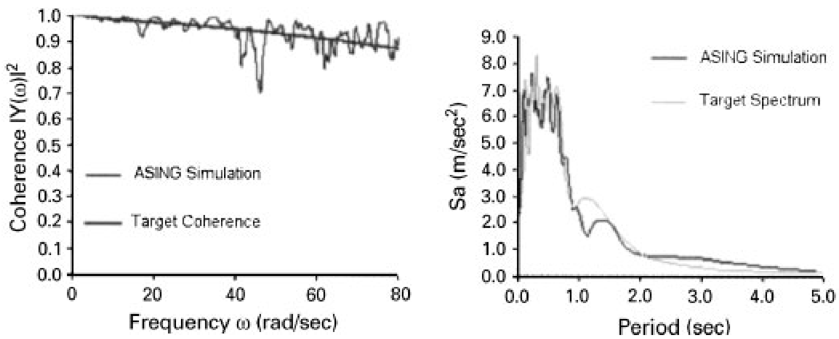

Figure 3. Simulated versus target absolute values of coherency for the Loh and Yeh model and the Kalamata response spectrum.

Spatial variability

With respect to the spatial variability model, it was deemed necessary to first verify that the generated synthetic accelerograms are compatible with the prescribed target response spectra and a selected coherency function, within the context of an 'Approach A' type analysis. For the case of two points at a separation distance equal to 70 m and a uniform soil profile, the target frequency content of the Kalamata, Greece, 1986 earthquake and the Loh and Yeh [24] coherency model, a very good agreement is achieved in terms of the target response spectra. Figure 3 illustrates that even for the first cycle of analysis the matching is acceptable, while it can be further improved by the iterative optimization procedure described previously. The matching with the target variation of coherency with circular frequency (also Figure 3) was satisfactory as well, notably though smoothing dependent.

The comparison was extended to investigate whether the effect of arrival delay and loss of coherency on the ground displacements could be acceptably described using 'Approach A.' In particular, within the framework of a research project in support of Eurocode 8 [38], a bridge structure was subjected to non-synchronous earthquake input. The bridge, presented in Figure 4, is a six span continuous deck with total length of 300 m, which is supported on five circular piers, assumed hinged to the deck in the transverse direction and it is excited in this direction by asynchronous artificial accelerograms. According to the initial brief [38], the generated motions are compatible with a prescribed Kanai–Tajimi power spectrum corresponding to a soil profile of medium soil stiffness, a PGA equal to 0.42 g and the Luco and Wong [22] coherency model. For the sake of simplicity, critical comparison with the motions derived by the ASING code, was performed for values of apparent velocity equal to 2500 m/s (infinite V_{app} case) and 600 m/s. The ratio $V_s/a = 600$ (a being the model coefficient for controlling the decay of the Luco and Wong coherency) that was employed in the initial brief, was considered to lead to relatively rapid loss of coherency compared to the decay resulted by other values proposed for the parameter a [2], but it was finally left unchanged for comparison purposes.

What is presented in Figures 5(a) and 5(b) is the maximum value of the mean of the displacements attained at all support points during ground motion (normalized to that of the

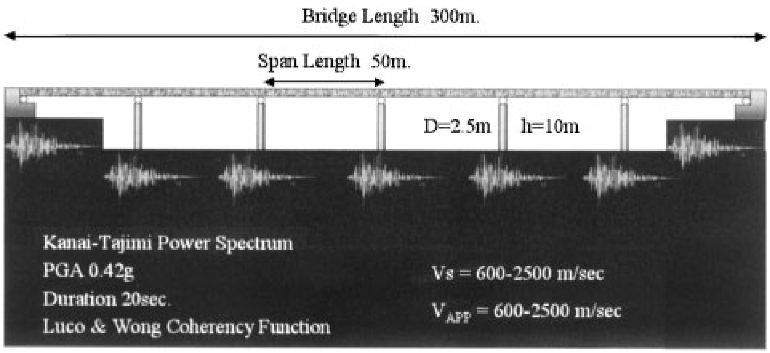

Figure 4. Overview of the bridge for the comparative study.

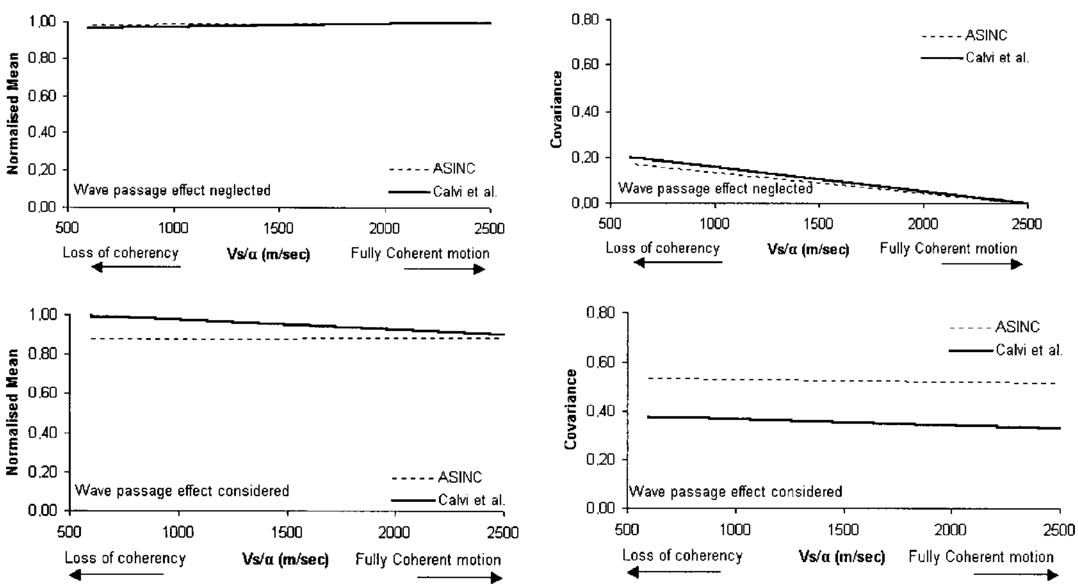

Figure 5. Effect of coherency loss on the mean and standard deviation of the ground displacements (a) when wave passage is not considered and (b) in the presence of wave passage effect.

synchronous case) and the corresponding covariance for both the Calvi and Pinto [38] and the ASING simulation. This a good measure of the expected wave passage and loss of coherency effect on the response of the superstructure, since the mean ground displacements at each time step impose the dynamic component of the bridge (i.e. the rigid body motion), whereas the covariance represents the degree to which pseudostatic distortion is imposed to the structure. An excellent agreement is observed at first, between the reference study and the ASING generated motions for the case where wave arrival delay is totally neglected ($V_{app} = 2500\,\mathrm{m/s}$

infinite velocity case) as seen in Figure 5(a). Clearly, when V_s/a is also equal to 2500, the motion can be considered as fully coherent, hence the ratio of mean displacement to the synchronous case is equal to 1 and the standard deviation of motions equal to 0 (motions are identical). As V_s/a and the subsequent coherency is reduced, both models produce an increase in the observed mean ground displacements, whereas the spatially variable motions result, in both models, in a standard deviation that is approximately 20% of the corresponding mean displacements between support points. In other words, both analyses conclude that although loss of coherency alone has limited effect on the dynamic response of the structure, it tends to produce relative displacements that are of a non-negligible magnitude. This observation is expected to be even more valid in the case of soft soil wherein the shear wave velocity is relatively lower.

When wave passage effect is taken into account together with loss of coherency (Figure 5(b)), the two models conclude that the dynamic component is slightly reduced, while the normalized mean displacements are not significantly affected by the coherency loss alone. They also agree in indicating that the pseudodynamic component as expressed by the covariance is substantially increased, an observation which is in agreement with previous studies [39]. It has to be noted, though, that shear and apparent wave velocity cannot be modified independently as is the case in the present example; nevertheless, it was a practical way used in the reference study to highlight the effect of loss of coherency alone without considering the arrival delay of the waves and as such, it was kept in the present study.

Site effects

Within the validation scheme, it was also deemed necessary to compare the four alternative approaches ('A', 'B', 'C' and 'D') of the proposed procedure (i.e. uniform soil, site-dependent spectra, multiple 1D site-effect analyses, hybrid site effects and site-specific spectra approach) with recorded data available at Euroseis-test (http://euroseis.civil.auth.gr), a densely instrumented and geophysically well-investigated valley located at Volvi near Thessaloniki, Greece [40]. Indeed, for the data recorded during the Arnea earthquake (05/03/1995, $M = 5.8$, $R = 32$ km) and previously used in other studies [5], synthetic acceleration time histories were constructed. Approaches 'A' and 'B' would obviously produce motions that would match the selected spectra, hence any comparison with the spectra of the recorded motions would be meaningless. Therefore, the comparison primarily involves Approaches 'C' and 'D'.

By comparing the normalized acceleration spectra of the recorded motions with the normalized average spectra of five synthetic accelerograms obtained at four stations (GRB, TST, FRM, STC, see Figure 6), it is clear that a very good agreement is achieved at periods up to 0.3–0.4 s. The differences for higher periods have been anticipated since it has been observed in the Euroseis-test site [40] that complex topography induces 2D site effects, which in turn significantly amplify ground motion in the long period range. As a result, even though in cases of complex geological structures the ASING simulation does not precisely capture surface ground motion at low frequencies, the satisfactory matching with recorded motions at frequency ranges where 2D effects are negligible, provides confidence that the approach proposed and the code developed can predict soil surface motion accurately for cases that the topography is regular.

Bearing in mind the fact that response spectra do not fully reflect the modification of earthquake motion in terms of phase, it can be concluded that the combined spatial variability

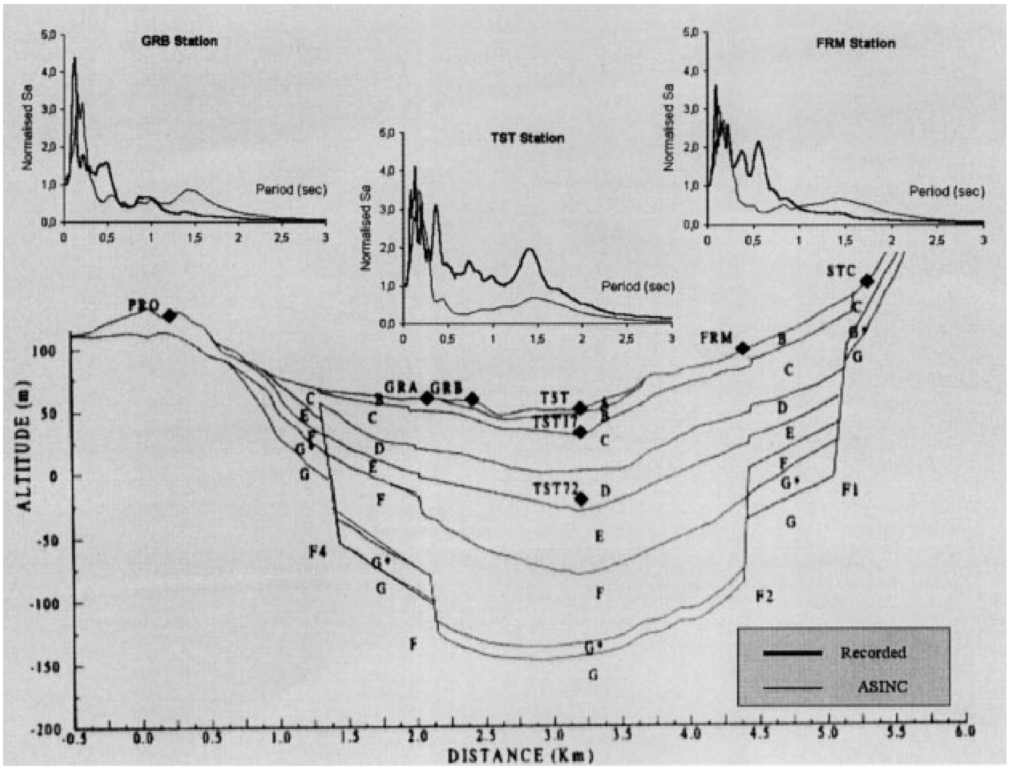

Figure 6. Validation of ASING against recorded data. Cross-section after Raptakis *et al.* [5].

and 1D site effects Approaches 'C' and 'D' provide a more accurate estimate of the motion frequency content compared to the assumption of a set of code-defined spectra at each location ('Approach B'), and even more so versus the adoption of a uniform acceleration response spectrum or power spectrum ('Approach A'). At the same time Approaches 'C' and 'D' remain relatively easy since the data required for their implementation (i.e. soil profile, shear wave velocity, density and damping) are typically available at the design stage.

The foregoing validation against recorded data triggered an effort to incorporate 2D site effects in the proposed procedure, at least on a worst-case scenario basis. Towards this direction, and admitting that the observations regarding complex site effects at the Euroseistest cannot be easily generalized, since they are strongly dependent on the fundamental frequency of the site and the geometry of the valley [40] it is proposed (and implemented in the code) to envelope the ground response by performing a separate analysis where a specific Fourier frequency range is amplified by a factor, which at least regarding response spectra is estimated between 1 and 3 [4]. This is of particular importance bearing in mind that the particular range is close to the fundamental period of the vast majority of bridge structures while it can also be incorporated when there is evidence that 1D vertical wave propagation analysis over a sharply dipping bedrock would underestimate the magnitude of shaking.

Table I. Calculated single pile head displacement resulting from different approaches.

Model	Pile head displacement for horizontal load of 2300 kN (mm)			
	Analytical solution	Pile	SAP 2000	MSC/ NASTRAN
Linear curve—constant stiffness with depth	22.2	21.9	24.9	25.4
Linear curve—linear stiffness with depth	—	20.1	26.1	25.8
Nonlinear curve—constant stiffness with depth	—	32.9	29.7	29.5
Nonlinear curve—linear stiffness with depth	—	27.2	31.4	31.4

Soil–structure interaction

With respect to the interaction between soil, foundation and the structure, the complexity of the problem and the uncertainties involved are such, that an extensive investigation scheme was devised in order to (a) decide the approach to be adopted within the proposed methodology and (b) validate the selected procedure. As a result, it was first attempted to obtain a minimum agreement between the simplest static case: a linear elastic static 'BDWS' model, and analytical solutions. This test was performed for a free head single pile of 1.5 m diameter founded in a soil with constant stiffness. Next, the comparison was extended for cases of stiffness varying with depth as well as for non-linear soil response. For the latter, a bilinear curve was adopted that was found to yield similar results to those derived using a more detailed lateral soil resistance–deflection relationship [37]. FE analyses were also performed for comparison purposes, using two commercial codes (SAP2000, MSC/NASTRAN) as well as a geotechnical program (PILE). The results are presented in Table I and a good agreement between alternative methods is seen. Minor differences do exist though, even in this simplified case, and may be attributed to the fact that analytical solutions are based on the assumption of an infinite-length pile in contrast to the finite length of the FE model.

Next, the comparison was extended in order to investigate the feasibility and potential advantages of representing the soil in a full two-dimensional discretization, still within the context of static analysis. A set of calibration assumptions was required for establishing a correspondence between the Winkler and the plane-strain FE approach especially with respect to the correlation of the modulus of subgrade reaction k_h used for the evaluation of the spring constants and the modulus of elasticity E_s assumed for the 2D soil elements. Again varying stiffness with depth and non-linear soil response were examined. The results, presented elsewhere [37], show very good agreement in the elastic range while, in the inelastic range, indicate the complex character of a two-dimensional plane strain FE representation with respect to a BDWS model.

The above difficulties and uncertainties arising from 2D soil or pile FE modelling even within the context of static analysis, as well as the fact that the foundation action effects are beyond the scope of the proposed procedure, led to the decision to uncouple kinematic and inertial interaction. The effect of kinematic interaction as well as the derivation of the coupled complex dynamic impedance matrix were checked through a detailed sensitivity analysis scheme and, the bridge response has been compared to all available theoretical solutions wherever possible. As an example, the satisfactory agreement between the absolute value of

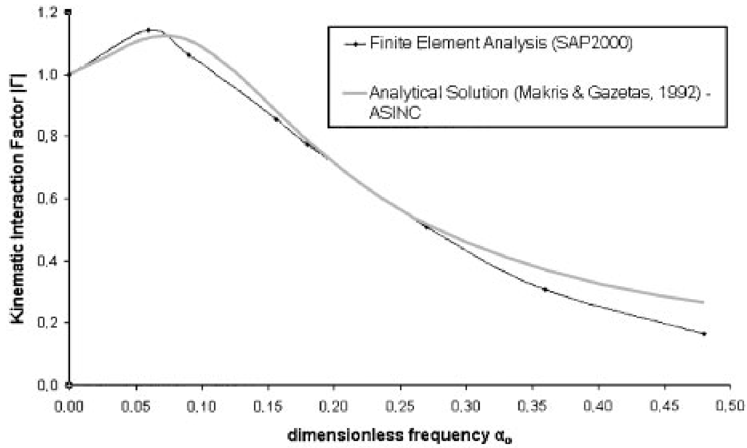

Figure 7. $|\Gamma(\omega)|$ factor derived from ASING and FE analysis.

the complex kinematic interaction factor, derived for a 2×2 pile group foundation with the analytical approach [30] and with FE analysis performed with the computer code SAP2000 is presented in Figure 7.

It was also interesting to verify whether the equivalent non-linear approach proposed for the soil surrounding the foundation piles can be used within the context of the proposed comprehensive methodology. For this purpose, the spring and dashpot coefficients for horizontal, rocking and coupled modes of vibration that where calculated for a single pile of 1 m diameter, drilled in a 200 m/s shear wave velocity uniform soil layer, using the equivalent shear modulus reduction and the subsequent damping increase proposed in Equation (1), were compared with a rigorous solution by Michaelides [41]. The corresponding values are presented in Figures 8(a) and 8(b) showing an excellent agreement between the two approaches in the elastic range but also indicating that at least for medium to low frequency earthquake motions, the proposed simplification does not introduce an error higher that the one that would result by not accounting for non-linear soil response at all.

Inelastic dynamic analysis and finite element modelling

Having validated the first three steps of the methodology and code, it was deemed necessary to check the lumped-plasticity model described in the previous sections on its own, before adopting it within the comprehensive approach. At first, the required moment—curvature curve was evaluated using refined fibre model analysis. Next, a complete pushover analysis was utilized, using the corresponding moment–rotation curve as the key input for appropriate non-linear link elements. Having established a level of confidence with respect to the (static) plastic rotations of the pier base, the FE modelling of the inelastic response of the pier was extended for the case of dynamic analysis. Additional FE codes (MSC/NASTRAN, SAP2000) were used to verify that the above non-linear rotational spring would be accurately activated in the presence of the springs and dashpots being also connected at the base of the pier while the bridge is subjected to multiple support excitation. The results were found to be

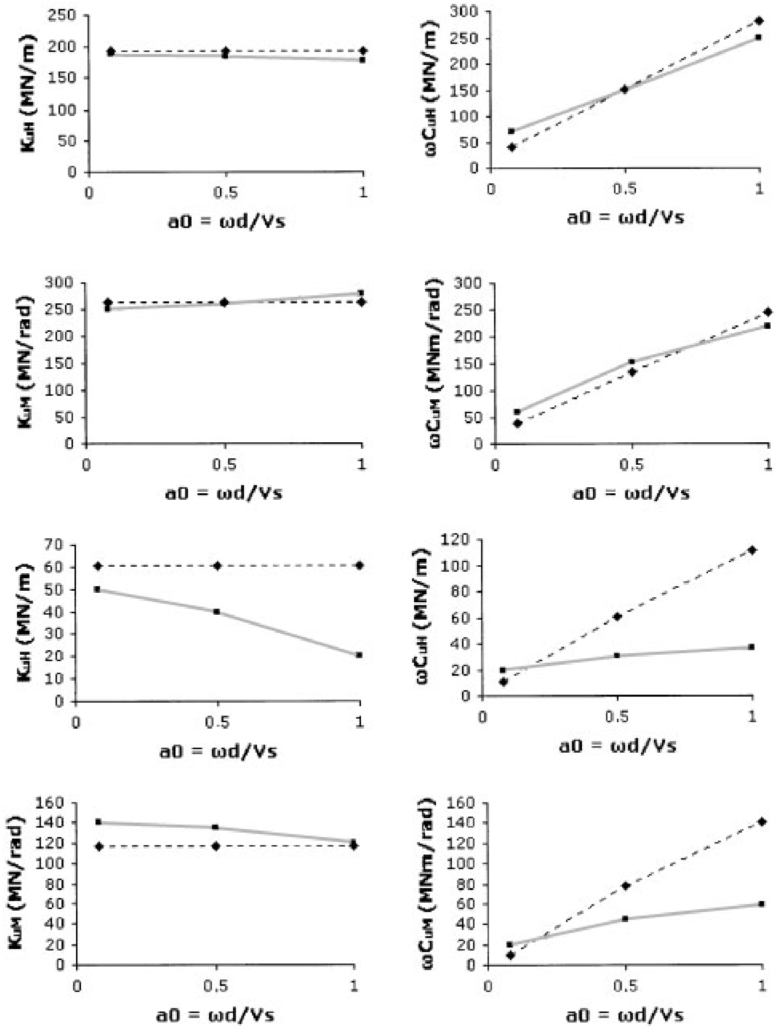

Figure 8. Comparison of single pile dynamic impedance (a) for the linear elastic soil case (b) for the non-linear soil case.

in very close agreement provided that a, different in each FE code, combination of element constraints and restraints was applied. As an example, the comparison of the spring rotation time history of an RC section as obtained using the equivalent non-linear rotational spring element in SAP2000, with the yielding code derived by NONLIN [43], shows that the lumped plasticity model is indeed activated properly during dynamic analysis (Figure 9). Details of the foregoing, as well as of other verification tests, can be found elsewhere [43].

Finally, since the expected bridge pier drifts during strong ground motion are large, it was considered essential to ensure that the pier top displacements are not significantly affected by

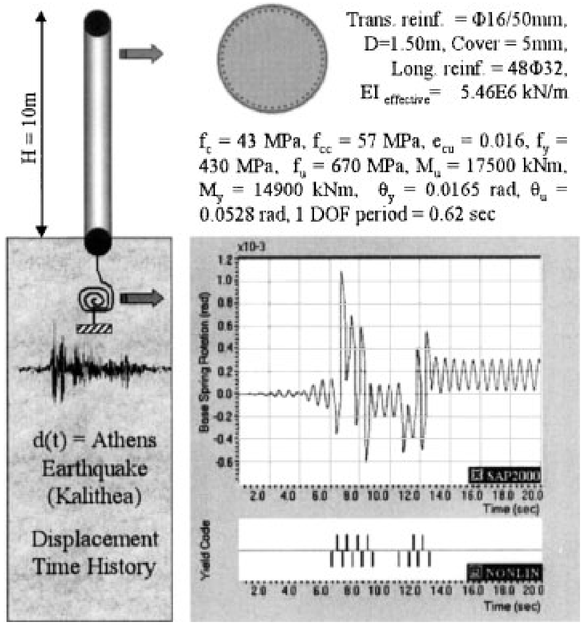

Figure 9. Validation of RC inelastic model.

$P - \Delta$ effects. Focusing on the case of non-linear static analysis and accounting for second-order effects, it was shown [37] that the effect of such a geometric non-linearity is minor in the presence of section yielding and consequently this feature was not considered necessary to be implemented in the proposed procedure, nor the ASING code.

CONCLUSIONS

A comprehensive methodology for the global inelastic dynamic analysis of RC bridges under spatial variability, local site conditions, and soil–foundation–superstructure interaction is set up and implemented into a specifically developed, fully parameterized computer code, that combines state-of-the-art knowledge in geotechnical earthquake engineering, and earthquake structural engineering. The aim is to provide the designer with a set of appropriately modified time histories and spring–dashpot coefficients for all supports that can be used in any standard FE software without requiring special dynamic SSI or inelastic analysis features for the soil, the foundation and the substructure.

An extensive validation was carried out for all stages of the proposed procedure. The spatial variability and site effects stage of the model were verified against both the results of a previous study and recorded data and were found to be in good agreement. Soil–structure interaction and the non-linear RC pier response were also coupled and verified through FE analyses, alternative computer codes, previous research studies, and closed-form solutions, wherever available. The results establish an adequate degree of confidence in the use of the methodology and code for further parametric analyses and seismic design or assessment. Having developed and validated the above comprehensive procedure, the dynamic response of 20 different bridge structures can then be examined and compared for various cases of analysis complexity as presented in a companion paper.

REFERENCES

1. Hao H. Effects of spatial variation of ground motions on large multiply-supported structures. *UBC/EERC-89/06*. EERC, University of California: Berkeley, 1989.
2. Zerva A. Spatial variability of seismic motions recorded over extended ground surface areas. In *Wave Motion in Earthquake Engineering*, Volume in the Series Advances in Earthquake Engineering, Kausel E, Manolis GD (eds). WIT Press, Southampton, UK, 1999.
3. Der Kiureghian A, Keshishian P. Effects of incoherence, wave passage and spatially varying site conditions on bridge response. *Proceedings of the FHWA/NCEER Workshop on the National Representation of Seismic Motion*, *Technical Report* 97-0010. National Center for Earthquake Engineering Research: Buffalo, New York, 1997; 393–407.
4. Pitilakis K, Makra K, Raptakis D. 2D Versus 1D site effects with potential applications to seismic norms: the cases of EUROSEISTEST and Thessaloniki. *Proceedings of the 15th International Conference on Soil Mechanics & Geotechnical Engineering*, *TC4 Satellite Conference on* '*Lessons Learned from Recent strong earthquakes*', Istanbul, Turkey, 2001; 123–133.
5. Raptakis D, Chavez-Garcia FJ, Makra, Pitilakis K. Site effects at EURO-SEISTEST. 2D model, observations and comparison with 1D analysis. *Soil Dynamics and Earthquake Engineering* 2000; **19**(1):1–22.
6. Wolf JP. *Foundation Vibration Analysis Using Simple Physical Models*. Prentice Hall: Englewood Cliffs, NJ, 1994.
7. Gazetas G, Mylonakis G. Seismic soil-structure interaction: new evidence and emerging issues, *Geotechnical Special Publication 75*, *Geotechnical Earthquake Engineering and Soil Dynamics III*, vol. 2. American Society of Civil Engineers: Reston, Virginia, 1998; 1119–1174.
8. Tseng W-S, Penzien J. Soil—foundation—structure interaction. In *Bridge Engineering Handbook*, Chen WF, Duan L. (eds). CRC Press: Boca Raton, FL.
9. Kahan M, Gibert R, Bard P-Y. Influence of spatial wave variability on bridges: a sensitivity analysis. *Earthquake Engineering and Structural Dynamics* 1998; **25**(8):795–814.
10. Keshishian P, Der Kiureghian A. Effect of soil-bridge interaction on response to spatially varying ground motion. *Structural Engineering World Wide* 1998. Elsevier Science Ltd.: Oxford, England, CD-ROM Volume, Paper No. T149-3.
11. Monti G, Nuti C, Pinto P, Vanzi I. Effects of non-synchronous seismic input on the inelastic response of bridges. *Proceedings of the 2nd International Workshop on Seismic Design of Bridges*, vol. 1, Queenstown, New Zealand, 1994; 90–107.
12. Shinozuka M, Saxena V, Deodatis, G. Effect of spatial variation of ground motion on highway structures, *MCEER-00-0013*. Multidisciplinary Center for Earthquake Engineering Research: Buffalo, NY, 2000.
13. Mylonakis G, Gazetas G. Seismic soil-structure interaction: beneficial or detrimental? *Journal of Earthquake Engineering* 2000; **4**(3):277–301.
14. Simeonov V, Mylonakis G, Reinhorn A, Buckle I. Implications of spatial variation of ground motion on the seismic response of bridges: case study. *Proceedings of the FHWA/NCEER Workshop on the National Representation of Seismic Motion*, *Technical Report*, *97-0010*. National Center for Earthquake Engineering Research: Buffalo, NY, 1997; 359–392.
15. CEN. Eurocode 8—Design provisions for earthquake resistance of structures. Part 2: *Bridges*, ENV 1998-2, CEN, Brussels, 1994.
16. American Association of State Highway and Transportation Officials (AASHTO). *Standard Specification for Highway Bridges* (16th edn), 1996.
17. Applied Technology Council (ATC). *Improved Seismic Design Criteria for California Bridges*: *Provisional Recommendations*: *ATC-32*. National Bureau of Standards: Washington, DC, 1996.
18. Public Works Research Institute (PWRI). *Design Specifications of Highway Bridges*, *Part V*, Seismic design Technical Memorandum of EED, No. 9801, Japan, 1998.
19. Ambraseys NN, Simpson KA, Bommer JJ. Prediction of horizontal response spectra in Europe. *Earthquake Engineering and Structural Dynamics* 1996; **25**:371–400.
20. Applied Technology Council (ATC). *Tentative provisions for the development of seismic regulations of buildings*: *a cooperative effort with the design profession*, *building code interests and the research community*, *ATC-3*. National Bureau of Standards: Washington, DC, 1978.

21. FEMA. *NEHRP*: *recommended provisions for seismic regulations for new buildings and other structures*, *Parts 1 and 2*. Building Seismic Safety Council, Washington, DC, 1997.
22. Luco JE, Wong HL. Response of a rigid foundation to a spatially random ground motion. *Earthquake Engineering and Structural Dynamics* 1986; **14**:891–908.
23. Abrahamson NA, Schneider JF, Stepp JC. Empirical spatial coherency functions for soil-structure interaction analyses. *Earthquake Spectra* 1991; **7**(1):1–27.
24. Loh CH, Yeh Y. Spatial variation and stochastic modeling of seismic differential ground movement. *Earthquake Engineering and Structural Dynamics* 1988; **16**:583–596.
25. Deodatis G. Simulation of ergodic multi-variate stochastic processes. *Journal of Engineering Mechanics*, (ASCE) 1996; **122**(8):778–787.
26. Gasparini D, VanMarcke EH. Simulated earthquake motions compatible with prescribed response spectra, *Technical Report*. Department of Civil Engineering, Massachusetts Institute of Technology, Publication No. R76-4, 1976.
27. Shinozuka M, Deodatis G. Effect of spatial variation of earthquake ground motion on seismic response of bridges. *Proceedings of the FHWA/NCEER Workshop on the National Representation of Seismic Motion*, *Technical Report*, *97-0010*. National Center for Earthquake Engineering Research: Buffalo, NY, 1997; 343–358.
28. Steidl H, Tumarkin A, Archuleta R. What is a reference site? *Bulletin of the Seismological Society of America* 1996; **86**(6):1733–1748.
29. Kennett BLN. *Seismic Wave Propagation in Stratified Media*, Cambridge University Press, Cambridge, MA, 1983.
30. Makris N, Gazetas G. Dynamic pile-soil-pile interaction. Part II: lateral and seismic response. *Earthquake Engineering and Structural Dynamics* 1992; **21**(2):145–162.
31. Pender MJ. Aseismic pile foundation design analysis. *Bulletin of the New Zealand National Society on Earthquake Engineering* 1993; **26**(1):49–161.
32. Kavadas M, Gazetas G. Kinematic seismic response and bending of free-head piles in layered soil. *Geotechnique* 1993; **43**:207–222.
33. Carter JP, Kulhawy FH Analysis of laterally loaded shafts in rock. *Journal of Geotechnical Engineering*, (ASCE) 1995; **118**(6):839–855.
34. Gazetas G, Fan K, Kaynia A, Kausel E. Dynamic interaction factors for floating pile groups, vol. 1, *NCEER-90-0021*. National Center for Earthquake Engineering Research: Buffalo, NY, 1990.
35. Veletsos AS. Dynamics of structure-foundation systems. In *Structural and Geotechnical Mechanics*, Hall WJ (ed). Prentice Hall: Englewood Cliffs, NJ, 1977; 333–361.
36. Priestley MJN, Seible F, Calvi GM. *Seismic Design and Retrofit of Bridges*. Wiley Interscience: New York, 1996.
37. Kappos AJ, Sextos AG. Effect of foundation compliance on the lateral load response of R/C bridges. *Journal of Bridge Engineering*, (ASCE) 2001; **6**(2):120–130.
38. Calvi M, Pinto P. Experimental and numerical investigations on the seismic response of bridges and recommendations for code provisions. *Prenormative Research in Support of Eurocode 8*, *Report 4*, European Consortium of Earthquake Shaking Tables, 1996.
39. Zerva A. Response of multi-span beams to spatially incoherent seismic ground motions. *Earthquake Engineering and Structural Dynamics* 1990; **19**(6):819–832.
40. Pitilakis K, Raptakis D, Lontzetidis K, Tika-Vassilikou Th, Jongmans D. Geotechnical and geophysical description of EUROSEIS-TEST: using free field, laboratory tests and moderate strong-motion recordings. *Journal of Earthquake Engineering* 1999; **3**(3):381–409.
41. Michaelides O. Non-linear dynamic analysis of piles. *Ph.D. Thesis*, National Technical University of Athens, 1997.
42. Advanced Structural Concepts, Inc. *Nonlinear Dynamic Time History Analysis of Single Degree of Freedom Systems* (*NONLIN*), Golden, Colorado, 1998.
43. Sextos A. Effect of spatial variability, local soil conditions and soil-structure interaction on the inelastic dynamic analysis of RC bridges. *Ph.D. Thesis*, Aristotle University of Thessaloniki, Greece, 2001.

