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Inelastic dynamic analysis of RC bridges accounting for spatial variability of ground motion, site effects and soil–structure interaction phenomena. Part 2: Parametric study

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SUMMARY

The methodology for dealing with spatial variability of ground motion, site effects and soil–structure interaction phenomena in the context of inelastic dynamic analysis of bridge structures, and the associated analytical tools established and validated in a companion paper are used herein for a detailed parametric analysis, aiming to evaluate the importance of the above effects in seismic design. For a total of 20 bridge structures differing in terms of structural type (fundamental period, symmetry, regularity, abutment conditions, pier-to-deck connections), dimensions (span and overall length), and ground motion characteristics (earthquake frequency content and direction of excitation), the dynamic response corresponding to nine levels of increasing analysis complexity was calculated and compared with the 'standard' case of a fixed base, uniformly excited, elastic structure for which site effects were totally ignored. It is concluded that the dynamic response of RC bridges is indeed strongly affected by the coupling of the above phenomena that may adversely affect displacements and/or action effects under certain circumstances. Evidence is also presented that some bridge types are relatively more sensitive to the above phenomena, hence a more refined analysis approach should be considered in their case.

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

INTRODUCTION

Several past studies involving bridge structures have focused on the effects of spatial variability of earthquake ground motion, the presence of the subsoil structure, its interaction with the

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foundation and the superstructure, as well as the inelastic behaviour of the supporting piers, providing theoretical, analytical and empirical evidence and solutions, and shedding some light on the sensitivity of the bridge's response to the earthquake, soil, foundation and structural properties.

However, research carried out so far is rather limited in the context of a comprehensive treatment of the problem, in the sense that the relative importance and the interdependency of the aforementioned effects are still not clearly established.

Existing studies attempting to couple the above phenomena, are often in disagreement or even contradict each other, a fact primarily attributed to the different assumptions used, hence their conclusions cannot be easily generalized and, as a result, code provisions, whenever considering the above phenomena, treat them separately and sometimes empirically.

A methodology and a computer code [1] for tackling all the above issues in a unified and sequential way is used herein for setting up appropriate scenarios that couple the above phenomena in different combinations, in order to investigate the sensitivity of the calculated response of the bridge to the adopted approach. A parametric analysis scheme is therefore developed aiming to:

(a) Evaluate the inelastic dynamic response of a well-studied bridge structure when spatial variability, site effects and soil–structure interaction (SSI) are either included or ignored during seismic analysis, in various combinations.
(b) Examine the validity of the above observations when minor modifications (i.e. altering one parameter at a time) take place in terms of the bridge's structural properties (fundamental period, symmetry, regularity, abutment conditions, pier-to-deck connections) and dimensions (span and overall length).
(c) Investigate whether the analysis approach leads to the same beneficial or detrimental effect for all response parameters (i.e. horizontal/vertical and absolute/relative displacements or forces).
(d) Study the relative importance of the aforementioned phenomena with regard to the inelastic dynamic response of different bridge structures.
(e) Investigate the feasibility of drawing some general conclusions on the sensitivity of the calculated response to the analysis complexity and identify the circumstances, if any, under which neglecting some of the above earthquake phenomena, consistently leads towards conservative seismic design, and eventually seek to establish threshold values for earthquake, soil, foundation and structural parameters that can be used to identify particular cases where refined analysis might not be necessary.

OVERVIEW OF THE STRUCTURES STUDIED AND THE PARAMETRIC ANALYSIS SCHEME

Reference bridge structure

A previously studied bridge structure [2] was selected as the reference case of the parametric analysis (Model A). It is a four equal span bridge of 200 m total length, supported on rectangular hollow piers of unequal height that varies from 7 to 21 m, arranged in an irregular configuration with the shortest pier in the middle (Figure 1). The concrete deck has a box

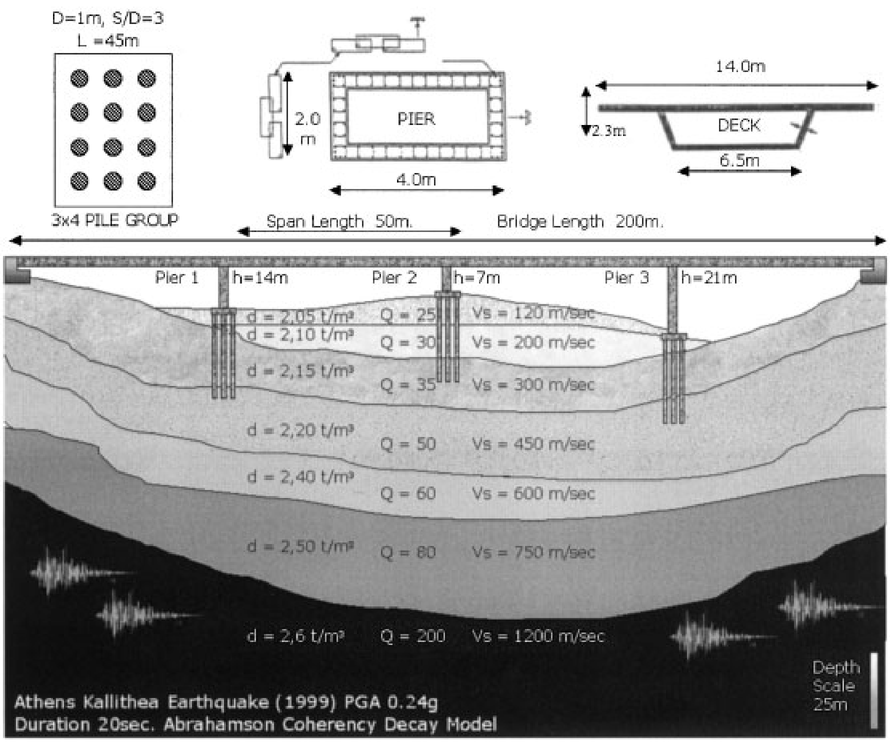

Figure 1. Model A (reference bridge) overview.

girder section, which (in this as well as in the previous [2] study) was assumed, for simplicity, uniform along the length of the bridge. The columns and the superstructure are assumed for this reference case as monolithically connected, while the abutment bearings are pinned in the transverse direction and free to slide in the longitudinal direction. The effective rigidity of the pier section (EI_{eff}) was taken as 50% the gross value (EI_g), to account for cracking. Effects of creep, shrinkage and thermal expansion are neglected, assuming they have been accounted for in the initial (non-seismic) design. The ultimate concrete strain ε_{cu} and the ultimate steel strain ε_{su} were taken equal to 0.008 and 0.10, respectively [2]. The RC sections' moment–curvature (M–φ) relationship and the corresponding yield and ultimate values were calculated using fibre analysis. The M–θ relationships for each of the three piers required within the frame of the proposed methodology [1] were derived using the, varying in each pier, plastic hinge length.

The reference bridge A was assumed to be located on a hypothetical subsoil structure whose geometry, stiffness, density and damping properties (quality factor $Q=\frac{1}{2}\xi$) are also presented in Figure 1. Moreover, a 3×4 pile group foundation was designed for all piers according to the Eurocodes 7 and 8 provisions, for a PGA of 0.24 g. The piles have 1 m diameter and 45 m length, whereas their axial spacing ratio $S/D=3$. A $8.0\times11.0\times2.0$ m pile cap was also assumed, resulting in a foundation mass approximately equal to 18% of the overall pier-to-deck mass. The foundation design ensures that the inelastic response will not be concentrated at the piles and no excessive rocking will occur at the pier base.

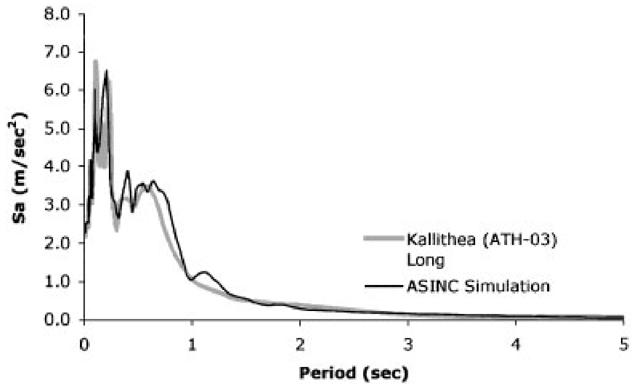

Figure 2. Comparison between target and simulated motion acceleration response spectrum.

The Kallithea (ATH-03) ground motion was used, recorded during the $M_w = 5.9$, 1999 Athens earthquake (PGA = 0.3g, and maximum spectral acceleration appearing at 0.2s period); in most analyses it was scaled to the level of the design peak ground acceleration (0.24g). The comparison between target and simulated motion acceleration response spectra is presented in Figure 2. The generated ground motions, as well as the linear/non-linear springs and dashpots required at the three pier support points for the inelastic SSI analysis were calculated using the code ASING [1]. The motions were used for the excitation in the transverse direction of the bridge, and the springs/dashpots for modelling the supports of a finite-element (FE) model set-up using the commercial FE package SAP2000 [3].

Other bridge structures considered

In addition to the aforementioned reference structure, it was also deemed necessary to study the dynamic response of alternative bridge configurations, to investigate the validity of the conclusions drawn when selected parameters are modified one at a time. At first, four bridge configurations were selected retaining the geometric characteristics of reference Model A but representing different levels of stiffness reduction and subsequent variation in the fundamental period (Models B1–B4); i.e. gross stiffness ratio (EI_{eff}/EI_{gross}) equal to 100, 75, 40 and 30%, respectively, resulting in fundamental periods that ranged from 0.7 to 1.4 times the natural period of the reference Model A.

Bridge symmetry and regularity were also examined by appropriately modifying the height of the piers. An effort was made to avoid significantly altering the dynamic characteristics of the structures, in order to focus on the role of regularity and symmetry. Model C1 (see Table I) therefore corresponds to a symmetric but irregular bridge (unequal pier heights) of natural period very close to that of Model A, while the case of a bridge having uniform pier heights was also examined in Model C2. Model C3 on the other hand is a more flexible (longer end piers) but still symmetric and regular structure.

By applying a uniform modification of all pier heights, the dynamic characteristics of the reference bridge were then modified and the effect of fundamental period with regard to the relative importance of spatial variability, site effects and soil–structure interaction was

Table I. Alternative bridge configurations.

Model	Difference with respect to the reference bridge	T_1 (s)	$T_{1\mathrm{transv}}$ (s)
A	None (reference bridge)	0.60	0.58
B1	$\mathrm{EI_{eff}/EI_{gross}} = 100\%$	0.42	0.40
B2	$\mathrm{EI_{eff}/EI_{gross}} = 75\%$	0.50	0.48
B3	$\mathrm{EI_{eff}/EI_{gross}} = 40\%$	0.67	0.64
B4	$\mathrm{EI_{eff}/EI_{gross}} = 30\%$	0.74	0.70
C1	$H_1 = 14\,\mathrm{m},\ H_2 = 7\,\mathrm{m},\ H_3 = 14\,\mathrm{m}$	0.61	0.46
C2	$H_1 = 14\,\mathrm{m},\ H_2 = 14\,\mathrm{m},\ H_3 = 14\,\mathrm{m}$	0.80	0.61
C3	$H_1 = 14\,\mathrm{m},\ H_2 = 21\,\mathrm{m},\ H_3 = 14\,\mathrm{m}$	0.92	0.77
D1	$H_1 = 11\,\mathrm{m},\ H_2 = 4\,\mathrm{m},\ H_3 = 18\,\mathrm{m}$	0.59	0.51
D2	$H_1 = 17\,\mathrm{m},\ H_2 = 10\,\mathrm{m},\ H_3 = 24\,\mathrm{m}$	0.85	0.68
E1	Monolithic abutment–deck connection	0.60	0.58
E2	Abutment–backfill interaction	0.61	0.59
E3	Transversely free abutment–deck connection	1.60	1.60
F1	Excitation in the longitudinal direction	1.98	0.86
F2	Excitation with alternative 'target' frequency content	0.60	0.58
G1	Overall length 400 m. Span length 50 m	0.70	0.70
G2	Overall length 400 m. Span length 100 m	2.17	0.83
G3	Overall length 600 m. Span length 50 m	0.69	0.69
G4	Overall length 600 m. Span length 100 m	1.67	0.77
G5	Overall length 600 m. Span length 150 m	3.05	1.13

investigated. Both a more flexible structure (D1) with piers taller by 3 m and a relatively stiffer bridge (Model D2) with 3 m shorter piers are considered.

It was also deemed important to study the influence of different approaches regarding the deck-to-abutment connection, a key issue for both modelling and design. At first, full fixity conditions were assumed for the left abutment (Model E1). This is often the case for relatively small abutments that are monolithically connected to the deck. The case of a non-rigid backfill was also considered (Model E2) and modelled through appropriate springs in the longitudinal and transverse direction. The abutments were modelled by defining appropriate spring constants derived following a recently proposed soil–abutment interaction methodology [4]. The currently common construction approach of abutments that are free to move transversely was also studied in Model E3.

The direction of excitation is another important aspect that was studied within the context of the parametric analysis since it is not clear whether the observations of the beneficial or detrimental effect of spatial variability on the transverse dynamic response of the bridge can be extended to cases that the structure is excited along its longitudinal axis. Bearing in mind the limitations of assuming that the loss of coherency and wave propagation patterns remain the same for motions in the longitudinal direction, the dynamic response of Model F1 when subjected to earthquake input along its longitudinal axis was studied. Moreover, in order to examine the most critical situation, the deck was assumed in this case to be hinged on the top of one pier and simply supported on the top of the next pier.

The frequency content of the target motion was also modified in order to capture potential bridge sensitivity to the assumption of input earthquake. Therefore, the case of a structure

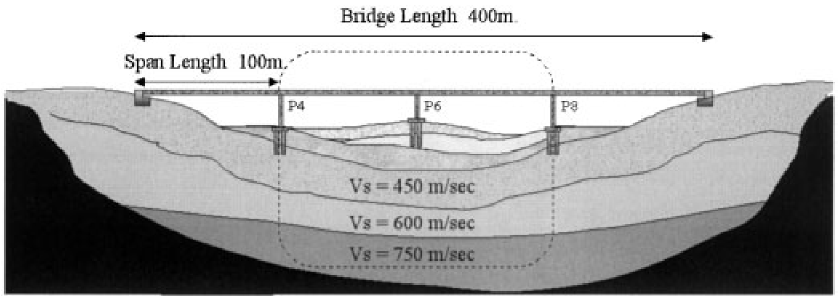

Figure 3. Model G2 overview.

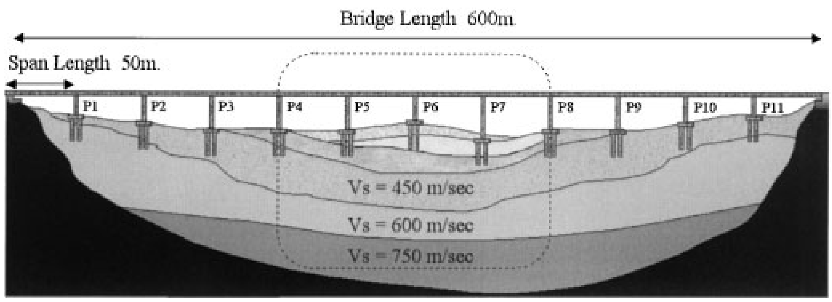

Figure 4. Model G3 overview.

identical to the reference model but excited by the 1989 Loma Prieta, Gilroy bedrock motion instead of the Kallithea record, was examined in Model F2. It has to be noted that the above two records do not represent all possible motions that might possibly be critical for the bridge studied. On the contrary, input motion and the resulting dynamic response of a bridge, are expected to be strongly dependent on different input frequency content, loss of coherency models, soil conditions and foundation type, not to mention additional topography or two-dimensional site effects. Nevertheless, since the mechanics of the problem are significantly complicated, it was considered necessary to examine the trends in the relative effect of spatial variability, site effects and SSI when *minor* modifications are applied at first, with respect to a reference scenario. In other words to examine whether for a given set of nine different scenarios the observations would be similar for all 20 models.

Along these lines, a set of five additional bridge models was included in the analysis in order to study the importance of total length (L) and span length (ℓ) with respect to the three phenomena studied. An eight equal span bridge of total length 400 m (Model G1) was studied and compared with a second 400 m overall length bridge consisting of four 100 m spans (Model G2), shown in Figure 3. Three 600 m total length bridges (Models G3, G4 and G5) were also studied, varying only in terms of span length, which was assumed to be 50, 100 and 150 m respectively; Model G4 is shown in Figure 4.

It has to be noted, that it would be unrealistic to assume that the deck geometry would remain unaffected when a larger span was considered. As a result, for the five G-models having span length different from that of the reference case, for which the initial deck design was performed, an appropriate deck stiffness modification was applied. As a simple rule, it was decided to keep constant the ratio of the deck to pier normalized stiffness:

$$\left[\frac{EI_{\text{deck}}/\ell}{EI_{\text{pier}}/H}\right]_{\text{Model A}} \Bigg/ \left[\frac{EI_{\text{deck}}/\ell}{EI_{\text{pier}/H}}\right]_{\text{Model G}} \tag{1}$$

Moreover, in order to be able to compare the dynamic response of bridges of unequal span and overall length, the assumption was made that all 400 and 600 m models had equivalent structural properties (pier heights) and subsoil conditions (soil profiles) with that of the Reference Model A. In order to satisfy this requirement, the 200m central part of G1, G2, G3, G4, G5 Models was kept essentially the same as that of Model A. Figures 1, 3 and 4 illustrate the aforementioned correspondence. Table I summarizes the 20 alternative bridge configurations.

Levels of analysis complexity

To investigate the way that the dynamic response of a bridge is affected by analysis assumptions regarding spatial variability, site effects and SSI phenomena, nine distinct scenarios were constructed in order of increasing complexity for each bridge. All scenarios can be considered as feasible approaches that could be followed during modelling and design. Scenario 1 (SC1), which is the basis for all comparisons, corresponds to the crudest assumption made in practice, that is, a fully fixed, elastic bridge structure which is uniformly excited with an appropriate ground motion, while no account is taken for the effect of the multi-layer soil formations. In this case, the supports are excited by synchronous displacement time histories generated to correspond to the target power spectrum of the Athens–Kallithea (ATH-03) record.

Each of the comparative scenarios applied to all bridges, considers one additional (to the standard case SC1) feature at a time, aiming at a more realistic representation and a more refined analysis approach. Scenario 2 (SC2) accounts for the wave passage effect; the bridge supports are excited by waves that arrive with a certain delay, while the motions still match the above target spectrum. Scenario 3 (SC3) represents the case where ground motions, while retaining the frequency content and the delay pattern of SC2, are assumed to have lost their coherency; the Luco and Wong [5] spatial coherency decay model was used as the target coherency loss relationship.

Scenario 4 (SC4) considers the case that the bridge is no longer resting on the (uniform) stiff soil to rock of SC1, SC2 and SC3 but on the relatively softer, deeper and varying between supports subsoil structure of Figure 1, i.e. on a damped, multi-layered soil profile, overlying an elastic bedrock of 1200 m/s shear wave velocity. Since the Kallithea motion used was recorded on very stiff-to-hard silty-to-sandy, marly clay and clayey marl with a depth of 12–28 m, overlying the Athenian schist which could be considered as the bedrock [6], the motion was first deconvoluted at the bedrock level. Then, spatially variable motions were generated at multiple bedrock points and they were amplified/deamplified through multiple 1D site response analyses, according to approach *C* of the proposed methodology [1]. As anticipated, the motion is primarily amplified in the range of the fundamental frequency of an equivalent uniform soil profile beneath Pier 2, which is characterized by approximately 100 m depth and average shear wave velocity of 500 m/s.

Further modification of ground motion takes place within the context of Scenario 5 (SC5) due to the scattering of the wave field caused by the relative stiffness of piles and soil (kinematic interaction). This set of time histories generated at the support level is the most refined (spatially variable, local soil affected, kinematically modified) input motion and is used as the *foundation input motion* [7] at the bridge structure of Scenario 6 (SC6), which is supported on more realistic foundation conditions in terms of stiffness and damping. The required frequency-dependent springs and dashpots which are derived to correspond to the fundamental frequency of the motion are eventually varying between support points.

Additionally to the above, Scenario 7 (SC7) investigates the dynamic response of the previously described flexibly supported bridge, subjected to the initial synchronous ground motion of SC1. The aim is to separate spatial variability and site effects from SSI effects (considered together in SC6), and focus on the importance of SSI phenomena. The final approach of Scenario 8 (SC8) allows for potential formation of a plastic hinge at the pier base representing the final and most comprehensive analysis that accounts for spatial variability, effect of local site conditions, SSI interaction and non-linear behaviour of the RC sections. The required displacement ductility demand is obtained by accounting for the foundation flexibility induced displacement:

$$\mu_{\mathrm{S}} = \frac{\Delta_{\mathrm{f}} + \theta_{\mathrm{f}} H + \Delta_{\mathrm{y}} + \Delta_{\mathrm{p}}}{\Delta_{\mathrm{f}} + \theta_{\mathrm{f}} H + \Delta_{\mathrm{y}}} \tag{2}$$

where Δ_{f} is the foundation translation, θ_{f} the foundation rotation, Δ_{y} the displacement at yield caused by bending, Δ_{p} the post-yield displacement and H the pier height. For assessment purposes, and for SC8 only, a target acceleration of 0.72g (corresponding to 3 times the design earthquake) has been considered. Moreover, since hysteretic energy dissipation is modelled directly in SC8, the assumption was made that the viscous damping ratio of the system is 2%, in contrast to the elastic scenarios (SC1–SC7) where 5% critical damping was considered.

Apart from performing the 'complete' inelastic dynamic analysis accounting for spatial variability, site effects and SSI (SC8) it was considered necessary to compare the derived ductility demands of the piers with those that would have resulted if a 'standard' non-linear fixed base, no site effects, synchronous motion approach was employed; the latter was considered as Scenario 9 (SC9). Table II summarizes the differences between the aforementioned nine analysis scenarios.

COMPARATIVE DYNAMIC RESPONSE OF ALTERNATIVE BRIDGE MODELS

An important issue related to all the scenarios and bridge models studied, is the generation of ground motions appropriate for all bridges, especially those differing in terms of length. The ideal approach would clearly be performing a complete Monte-Carlo simulation to ensure that the stochastic nature of the synthetic accelerograms would not affect the mean response values for the various models. Nevertheless, due to the very large number of analyses required (7 support points on average $\times$ 5 scenarios of different input $\times$ 20 bridges $\times$ 50 simulations = 35.000 input accelerograms), it was decided that the complete set of the 13 time histories generated for the longest (600 m) Model G3, should be used for all models throughout the parametric analysis, provided that, with an appropriate use of the motion generation code (ASING), the stochastic part of the motions would be retained common for all nine scenarios and for all 20 bridges. In such a way, even if a random error (albeit significantly reduced by the optimization procedure) with respect to the target frequency content and the loss of coherency

Table II. Nine analysis approaches, in various combinations of complexity.

Scenarios in ascending order of complexity	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6	Scenario 7	Scenario 8	Scenario 9
Uniform excitation	√						√		√
Fixed base supports	√	√	√	√	√				√
Elastic response of R/C piers	√	√	√	√	√	√	√		
Wave passage effect		√	√	√	√	√		√	
Loss of coherency			√	√	√	√		√	
Site effects				√	√	√		√	
Kinematic interaction					√	√	√	√	
Inertial interaction						√	√	√	
Inelastic response of R/C piers								√	√

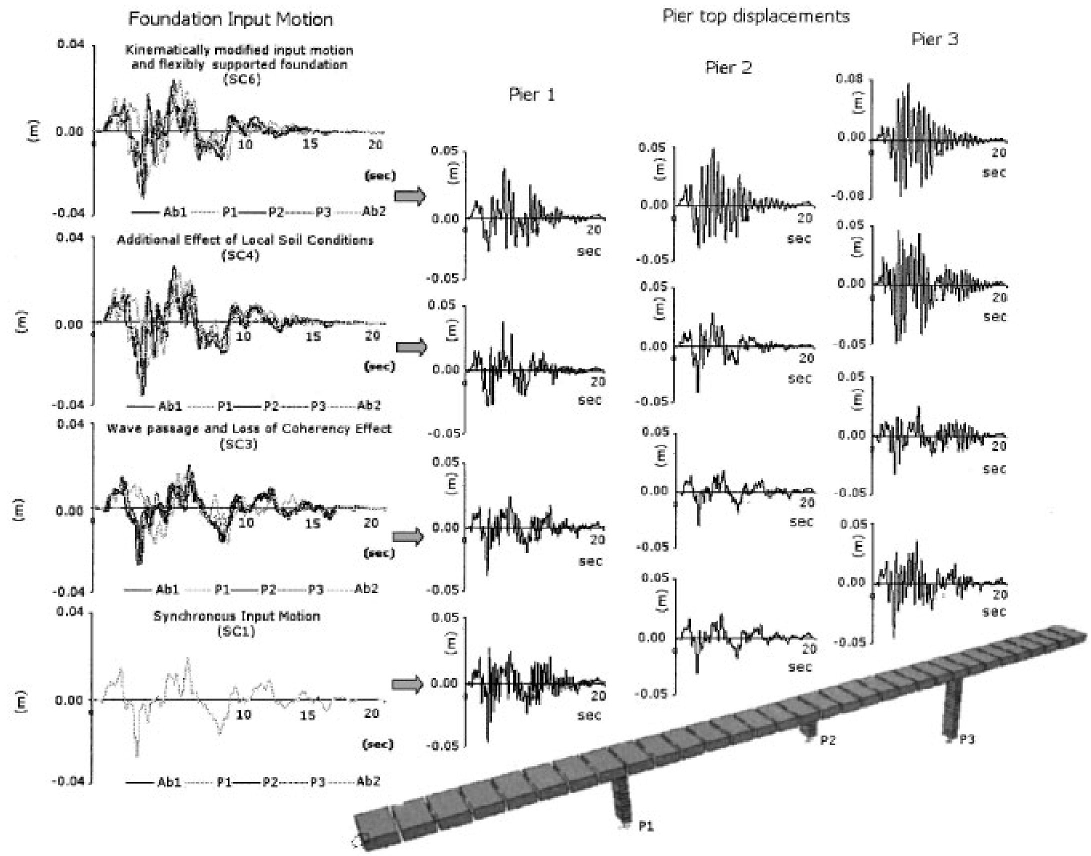

Figure 5. Pier top transverse response under various excitation scenarios.

pattern does exist in absolute terms, the relative difference of including wave passage and loss of coherency effects in a synchronous motion analysis can still be studied for the 20 bridges, while it is not crucial for site effects and SSI analyses. Based on this rationale, a set of 65 accelerograms (13 points $\times 5$ scenarios of different input) was generated and used in 180 (20 bridges $\times 9$ scenarios) FE analyses, together with the calculation of 142 (74 supports $\times 2$ SSI Scenarios) fundamental frequency-dependent linear/non-linear spring and dashpot sets.

As an example, the spatially variable synthetic displacement time histories corresponding to the four major scenarios of different input are shown in Figure 5, and it is seen that input motion can indeed vary significantly between support points, even for a relatively short bridge, especially when local soil conditions are considered, as is the case for SC4. There is also a common approach followed with respect to the key parameters studied; the absolute pier top displacements, the relative displacements between piers, the deck's vertical displacements (also a key parameter for deck design), as well as the bending moments developed at the pier bases that resulted for the nine scenarios, are comparatively evaluated in the following paragraphs.

Wave passage effect (SC2 vs SC1)

In order to highlight the relative importance of including the wave passage effect (SC2) with respect to the current common ('standard') practice (SC1), the action effects are presented in normalized form in Figure 6 (SC2/SC1). A value of the response ratio SC2/SC1 > 1.0 represents the unfavourable case of displacement or bending moment increase, while the beneficial effect of including wave passage is represented by SC2/SC1 < 1.0. It is also recalled that all response parameters refer to the transverse direction wherein the excitation is assumed.

It can be observed that the assumption of waves that travel at a constant velocity resulted in generally lower response. With the exception of bridge model C3, the pier top absolute displacements of the 200 m bridges (Models A–F2) were all found to be reduced by 20% on average, while the bending moments developed were also reduced by approximately 30%.

In fact, Model C3 is an interesting case since it exhibits the two most common features observed between all models: At first, the structure is somehow prevented from vibrating in its fundamental mode as seen by the comparison of the corresponding pier top Fourier amplitude spectra for Pier 2 (Figure 7). This fact is observed, to a varying extent, for all 200 m bridges and can be considered as the primary cause of the displacement reduction in all these models. The displacement reduction observed is also in line with similar studies involving short-span bridges [2] while it has also been found to be proportional to the phase difference [8].

A second dynamic response modification due to the wave passage effect, that arises from the evaluation of the response Fourier spectra of Figure 7, is the excitation of higher, antisymmetric in particular, modes. This fact that has also been verified in previous studies [9–11] and implies that the response of the bridge, especially of a symmetric one, is no longer symmetric and cannot be described by any modal combination of the synchronous motion excitation case. The above feature is quite frequent (observed in six out of the 15 short span models) and has indeed modified the overall dynamic response. Nevertheless, it does not result in an increase in the pier top displacements of the 200 m models.

As the bridge length increases (i.e. for $L \geqslant 400\,\mathrm{m}$), both pier top displacements and bending moments are increased by up to 40%, leading to the conclusion that accounting for input motion arrival delay may have a detrimental effect on longer structures, a fact which has also been verified by other researchers [9–11].

Relative displacements, on the other hand, reflect another detrimental effect that results from the wave passage consideration; they increase by approximately 25% on average for the short length models and substantially increase (up to 350%) for the 600m models. It has to be noted, though, that as different piers oscillate at different stages of the acceleration or displacement time histories, the relative displacements are inevitably strongly (and randomly) dependent on the input motion, hence an extension of the parametric analysis using alternative simulated motions would be necessary. Within the scope of the present study, Model F2 which was subjected to a different earthquake motion (Loma Prieta–Gilroy) was examined. The dynamic response of the above model was found to exhibit similar increase in relative displacements to that of the reference Model A, confirming the observation that the relative displacements are indeed very sensitive to the input motion variability, but show a tendency to increase.

Another interesting point is related to the fact that asynchronous transverse excitation significantly affects vertical displacements as well, which are reduced by more than 50% on the average for the short span bridges, independently of boundary conditions and geometric characteristics. Moreover, a linear dependency appears to exist between the extent of decrease

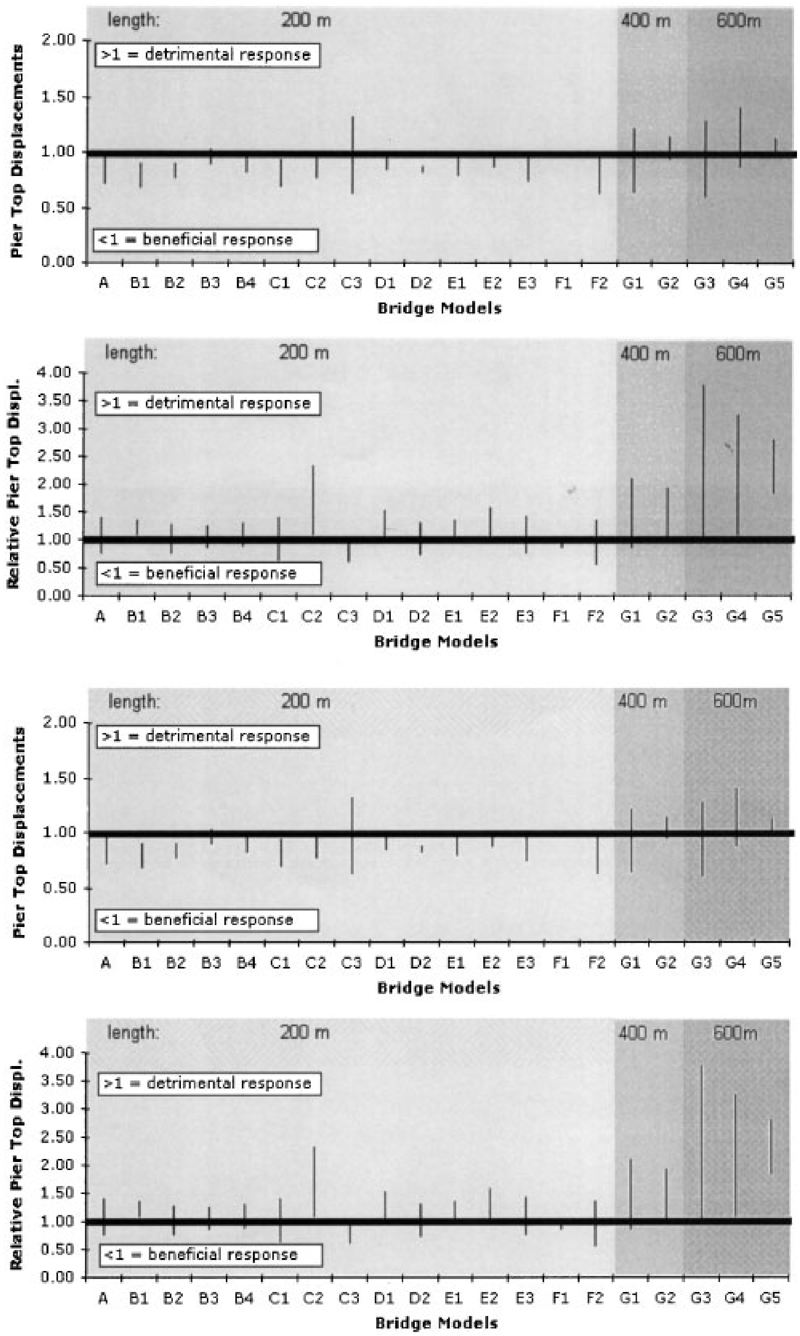

Figure 6. Wave passage effect (SC2) response parameters normalized to 'standard' approach (SC1).

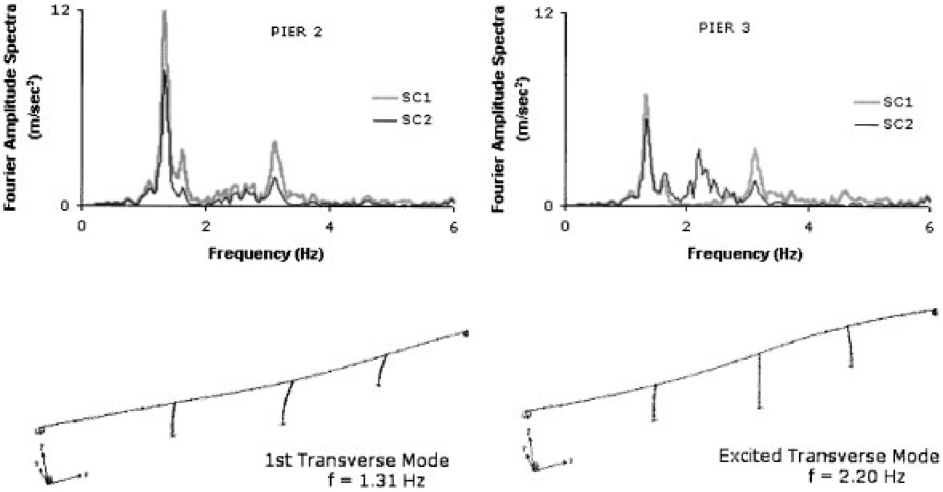

Figure 7. Fourier amplitude spectra of Pier 2 (model C3) for wave passage effect (SC2) and 'standard' approach (SC1).

and the span length; in particular, as the span length increases from 50 to 100 m (Models G1, G2) and from 50 to 150 m (Models G3, G4, G5), the beneficial vertical displacement reduction becomes less apparent. This implies that for span lengths longer than 150 m, the vertical displacements could even increase.

Loss of coherency effect (SC3 vs SC1)

In the case that wave passage effect is combined with a loss of coherency model (SC3) the dynamic response of the 20 bridges follows in general the trends observed for the arrival delay case. Asymmetric modes that where not present during synchronous motion were also triggered in this case while, in absolute terms, the dynamic bridge response could be considered as generally beneficial at least for the 200 m bridges. The latter is an observation again in agreement with other studies [12, 13].

Nevertheless, bending moments and absolute pier top transverse and vertical displacements, which again are presented (Figure 8) normalized to the 'standard' case (SC3/SC1), are similarly increasing with the overall length; this may be primarily attributed to the fact that coherency decay is also distance dependent, hence longer bridges are expected to have their supports excited by motions that are characterized by higher standard deviation at a given time. As previously, 400 m could be roughly considered as a threshold length value for asynchronous motion effects, which when exceeded may be followed by up to 60% higher bending moments, up to 40% increase in absolute displacements and up to 350% higher relative displacements. Along these lines, it is worth noting that Eurocode 8 prescribes the length of 600 m as the limit for starting considering spatial variability, an assumption that may lead to unconservative design under certain circumstances. On the other hand, what seems to be a reasonable Eurocode 8 assumption is the fact that the limit length value is set on an overall length, not a span length basis. Indeed, from the G1, G2 and G3, G4 and G5 comparative

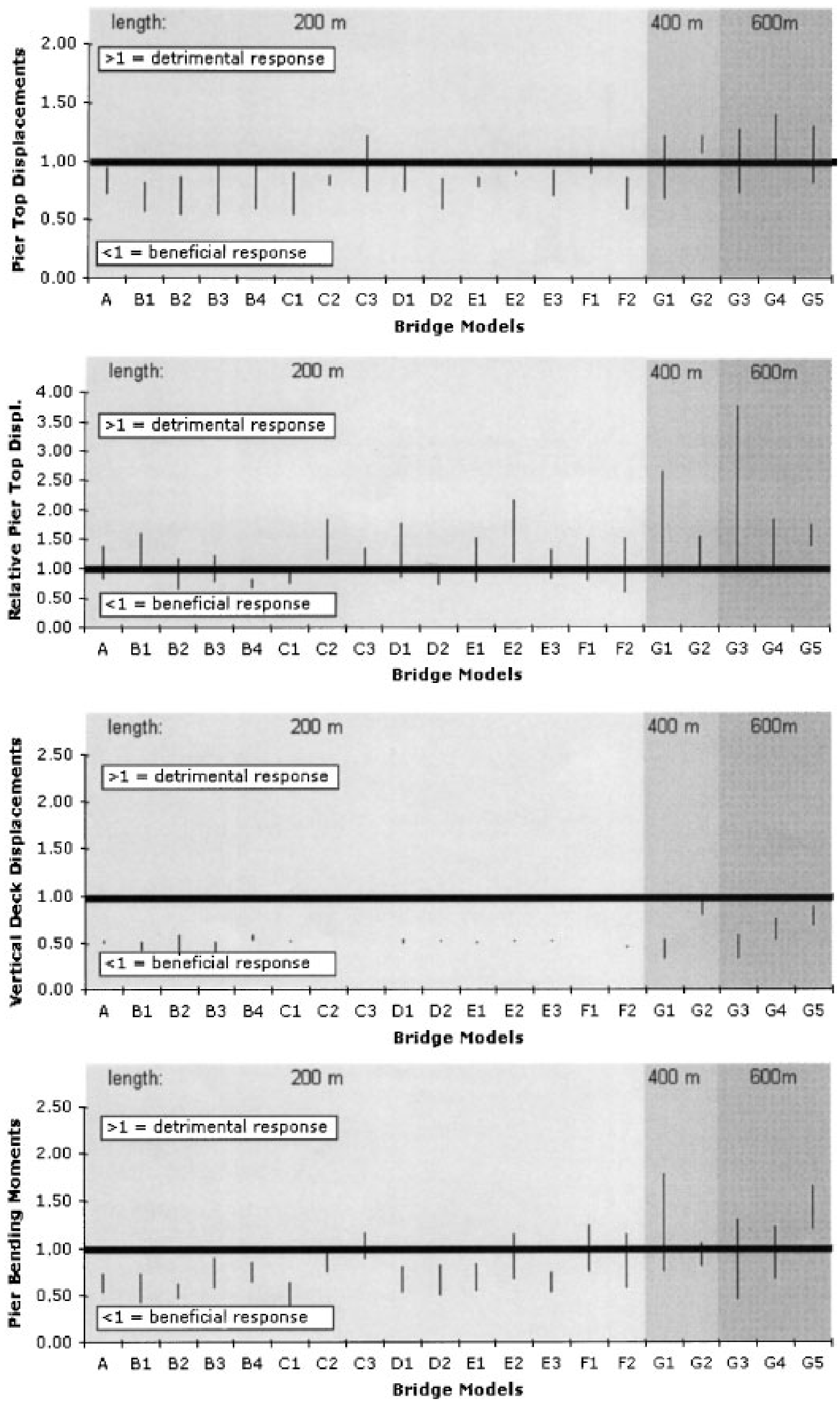

Figure 8. Loss of coherency (SC3) response parameters normalized to 'standard' approach (SC1).

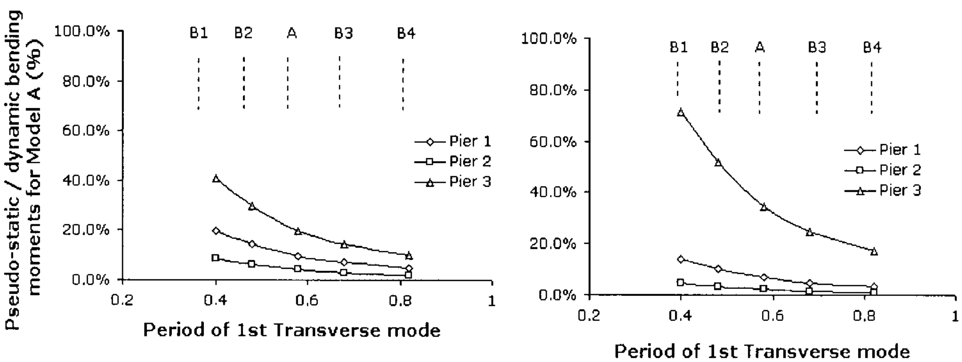

Figure 9. Pseudostatic force increase with structural stiffness for excitation that accounts (a) for the wave passage effect (b) for wave passage effect and loss of coherency.

study, there is no clear trend relating transverse displacements and bending moments to the length of the span. On the contrary, it is observed that bridges of shorter span length (i.e. G1) may develop higher forces than structures of longer spans (i.e. G2) under certain circumstances.

It is also notable that although the increase in the variability of the input leads to an increase in pier top response variability (i.e. the range of pier maxima is wider), the latter can by no means be considered proportional to the first. In other words, significantly less coherent motions do not necessarily imply proportionally varying pier response, rendering the absolute displacement modification rather unpredictable.

By plotting the pseudodynamic bending moments developed at the pier bases of models B1, B2, B3, B4 and A, in ascending order of the first transverse mode period, it is verified that the wave passage effect generated higher pseudostatic forces as structural stiffness increases (Figure 9(a)), an observation which is even more valid in the case that loss of coherency is additionally considered (Figure 9(b)). It is clear therefore that the dynamic behaviour of bridges subjected to spatially variable support motion is a very complex issue, being dependent not only on the modes excited, but also on the interplay between the dynamic and the pseudostatic component which is triggered in each case.

Relative displacements on the other hand, are also significantly increasing with the overall length. An effort was made to relate the average (μ) and the standard deviation (σ) of the relative displacements ratio SC3/SC1 calculated for all spans during scenarios SC3 and SC1 categorized for the three overall length categories (200, 400 and 600 m). It was found that both the average of the relative motion modification and its standard deviation show a trend to logarithmically increase with length, hence can be fitted to the empirical expression shown in Figure 10. As a result, it could be considered that in the worst-case scenario, calculated on a $\mu+\sigma$ basis, the relative displacements δ_a expected for a bridge with length $L \leqslant 600\,\text{m}$ could be calculated as the product of the displacements δ_s derived by the 'standard' synchronous approach and an amplification factor R_D:

$$\delta_a = R_D \delta_s = (0.8\ln(L) - 2.8)\delta_s \tag{3}$$

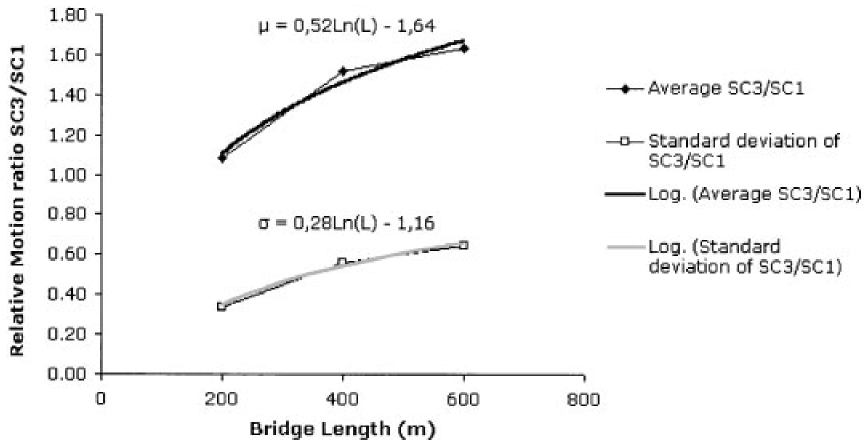

Figure 10. Relative motion dependency on overall bridge length.

For instance, for a 450 m bridge, the maximum relative displacements expected would be about twice the ones calculated through a uniform excitation analysis. Clearly, the above relationship cannot be directly generalized since it has not yet been checked against significantly different soil and input motion conditions. However, it may be used as a useful rule of thumb to roughly account for spatial variability in the design process.

Site effects (SC4 vs SC3)

When site response analysis is performed additionally to the inclusion of wave passage and loss of coherency effects (SC4) both the input seismic motion and the dynamic bridge response are strongly influenced. Again the response parameters are presented in a normalized form, this time being the ratio of the response calculated for the combined site effect step (SC4) to that for the loss of coherency and wave passage analysis of the previous step (SC3) in order to isolate the importance of the local soil conditions. The SC4/SC3 ratios are illustrated in Figure 11.

All response parameters are increased by approximately 50% on the average, while pier top transverse displacements and bending moments are more than doubled in certain cases. The importance of combining site effects with spatial variability models, which in turn resulted into increased displacement and action effects has been confirmed by other studies as well, for similar bridges of short to moderate spans [13].

This increase can be primarily attributed to the peak ground motion amplification that resulted from the site response analysis and to a lower extent to the spectral amplification of motion which is observed within the range of 0.6–0.8 s. The above PGA amplification (approximately 40% increase) is of particular importance since it is directly related to the dynamic response of an (elastic) bridge structure. Moreover, both from the validation analyses performed for the selection of the hypothetical soil structure, as well as from other research studies [14], it was found that the resulting surface peak ground acceleration is strongly related to the velocity contrast between the bedrock and the overlying layers. The observation that

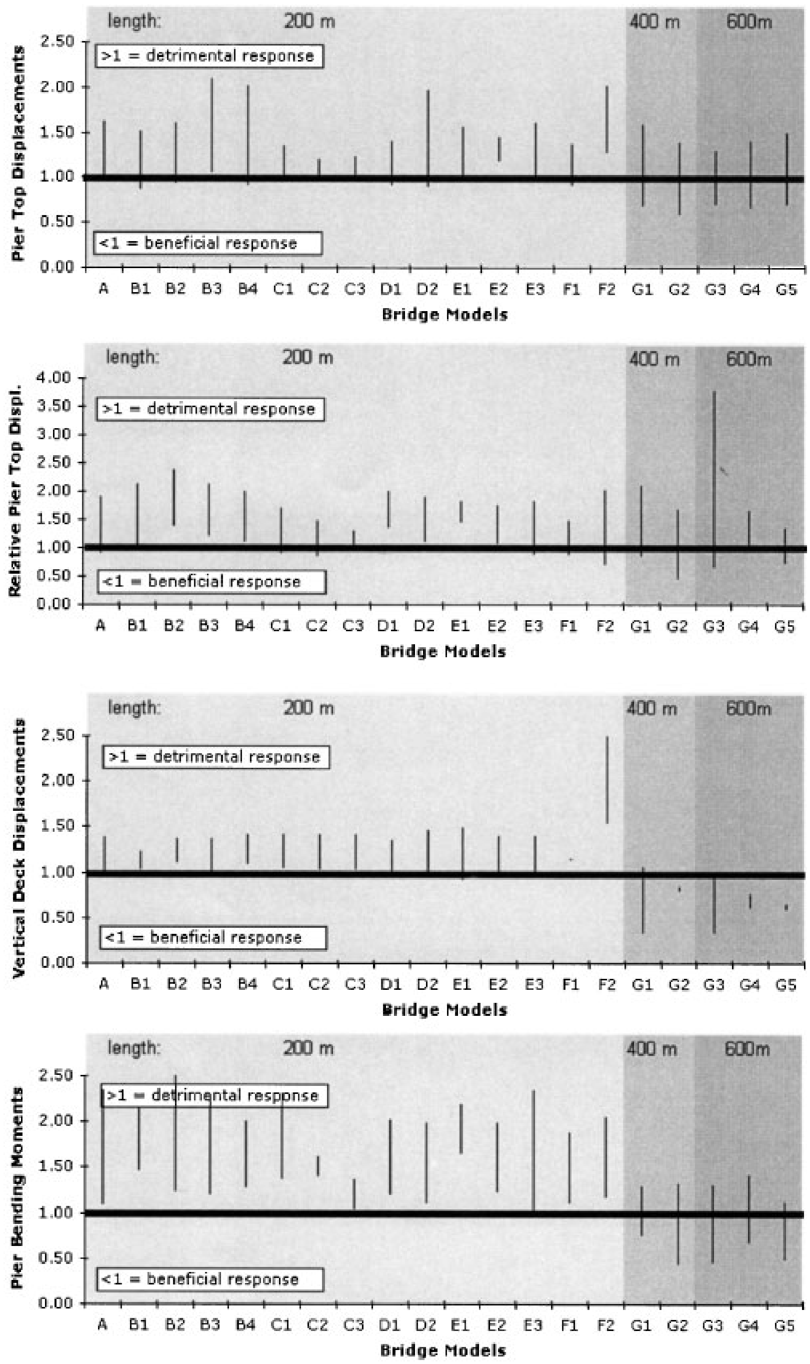

Figure 11. Site Effects (SC4) response parameters normalized to non-site effects case (SC3).

PGA is affected by soil properties and soil layers velocity contrast, is also in line with current UBC and NEHRP provisions which relate PGA to the soil characteristics. Unfortunately, this is not the case for the vast majority of modern seismic codes (including the prestandard version of Eurocode 8) hence leading to the potential underestimation of the ground motion amplification. Therefore, at least a 1D site response analysis appears to be crucial towards a more adequate representation of ground shaking.

An additional reason for including site effects in a comprehensive and refined analysis scheme is that, locally varying soil conditions and the subsequent different motion amplification is an additional source of input variability (as seen in Figure 5), which in turn further stresses the effects of asynchronous support excitation especially in terms of relative displacements (Figure 11).

Soil–foundation–structure interaction (SC5 vs SC4), (SC6 vs SC4), (SC7 vs SC1)

When kinematic interaction is included in the analysis (SC5) [1], filtering of the higher frequencies takes place. For this reason, the absolute and relative pier top displacements as well as the pier base bending moments, are uniformly decreased with respect to the site effects stage (SC4/SC3) by up to 10%, with the exception of the long period bridges G1–G5 for which kinematic interaction leads to limited amplification of motion. Notably though, kinematic interaction, like all previous phenomena, acts as a source of variability of seismic motion by itself leading to increased pseudostatic contribution, and 5% higher relative displacements. This interaction would be expected to be significantly more detrimental if the assumption of a non-uniform soil profile around the piles applied.

Inertial interaction (SC6) on the other hand, had, as anticipated, an important effect on the bridge response in terms of displacements. A general increase by approximately 30% on the average is observed in the absolute displacements. As previously, the displacements were normalized to the site effects case (SC6/SC4), hence, they can be attributed purely to SSI (Figure 12). This increase is the result of the introduced foundation flexibility and, as anticipated, the maximum observed modification refers to the middle pier of the bridges which is founded on relatively softer surface soil formations. Such detrimental role of the SSI is in agreement with recent observations [11].

Particular cases that exhibit reduced pier top movements do exist though (Models B4, C2, C3, D2 and specific piers of Models G1–G5, all structures of relatively long fundamental period). The reason is that the above structures, being flexibly supported, are subjected to earthquake actions which are generally decreased, due to the period elongation with respect to the particular spectral shape, the material and radiation damping introduced at the foundation–soil interface, but also, to a lower degree, due to the kinematic filtering of the higher motion frequencies. In general, therefore, it can be claimed that the non-uniform fluctuation of the pier top absolute and relative displacements observed in the present parametric analysis, is anticipated, but it is also a strong indication that the problem is complex and multi-parametric and that the overall dynamic response is an interplay between the modified dynamic characteristics of the structure and the excitation motion.

A general reduction, on the other hand, ranging from 10 to 50% with respect to the SC4 case, was observed in terms of pier base bending moments, a fact primarily attributed to the flexible foundation, the foundation damping and the reduced earthquake forces as previously. Nevertheless, an increase of up to 20% has been observed in a few piers of Models G1 and

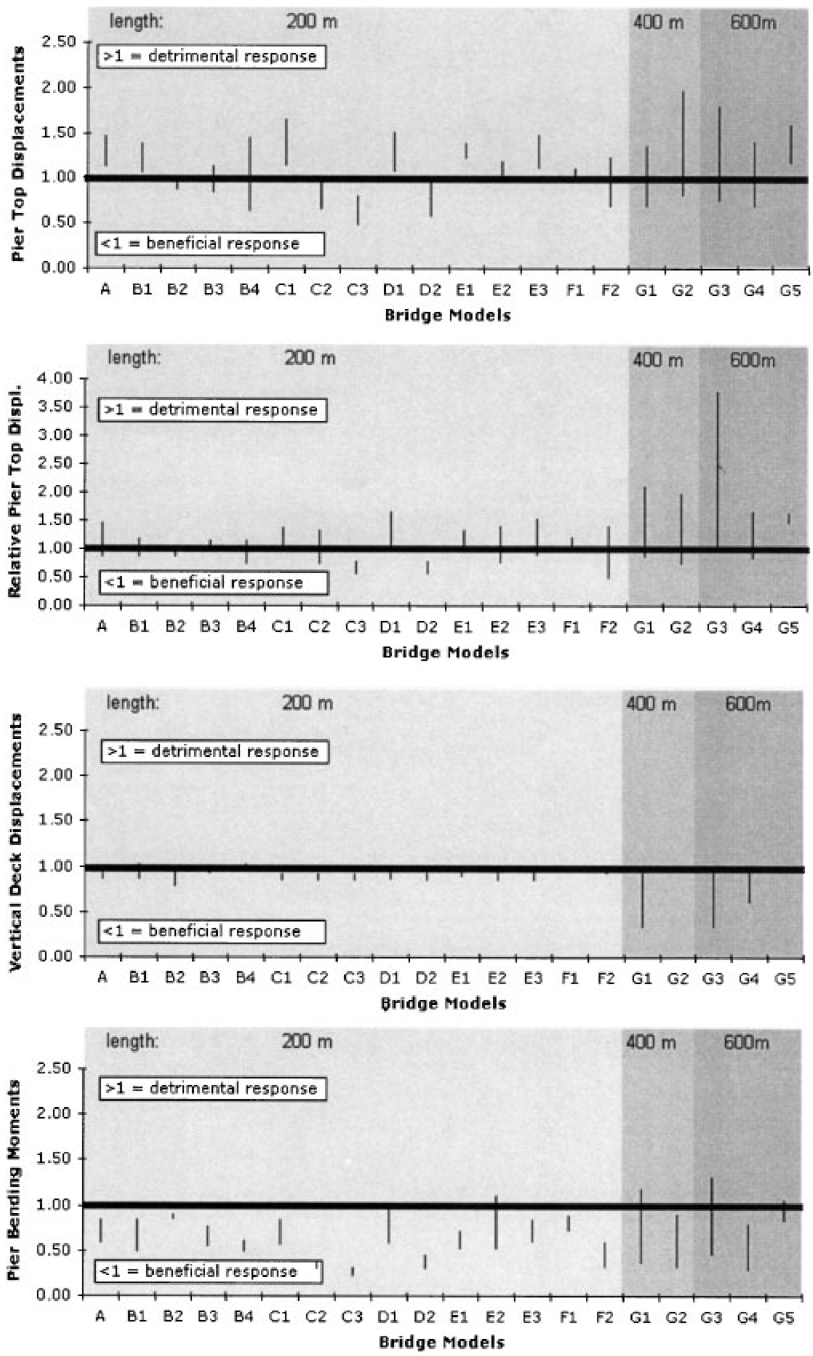

Figure 12. SSI (SC6) response parameters normalized to non-SSI case (SC4).

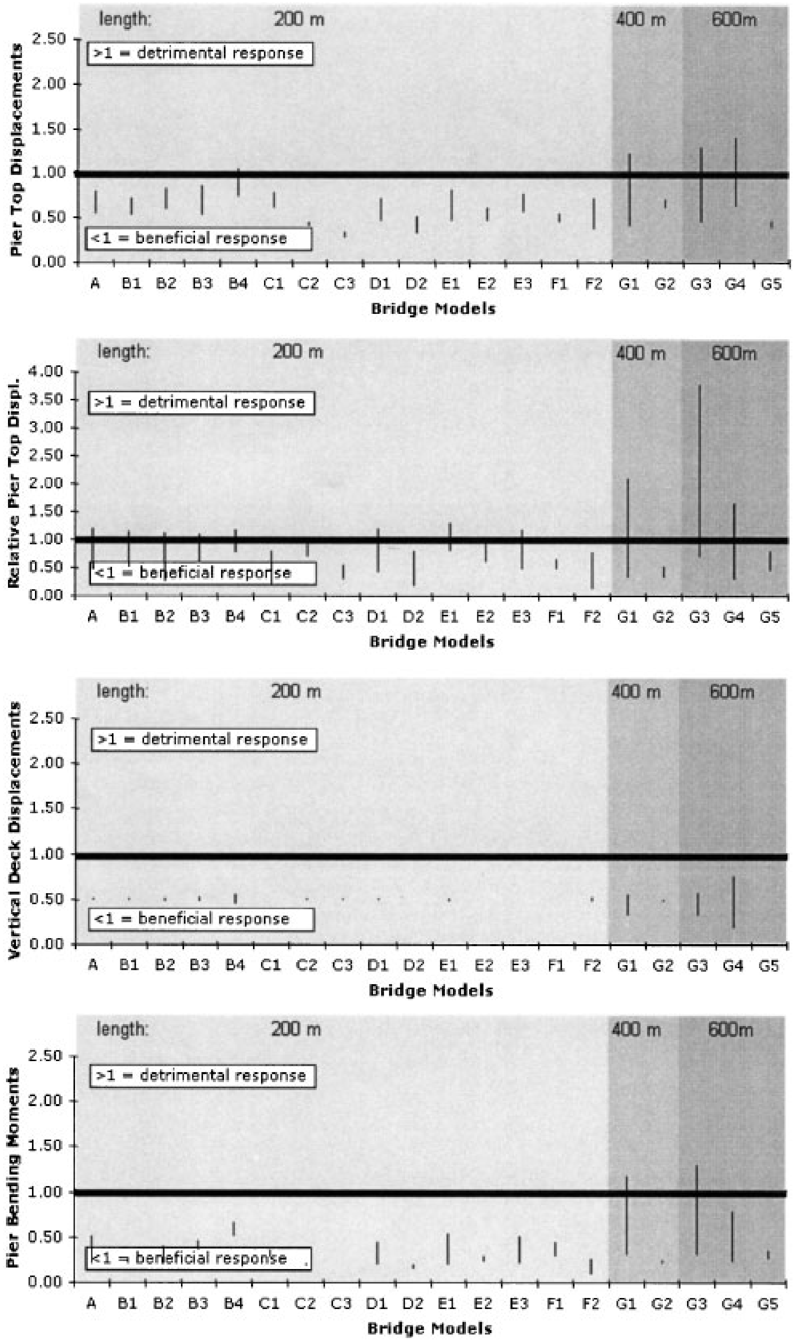

Figure 13. SSI, uniform excitation (SC7) response parameters normalized to 'standard' approach (SC1).

G3. This increase would indeed be significantly higher in case the spectral values of the input motion were not decaying towards periods longer than that of the fundamental period of the structures [15].

Based on the latter thought, it was considered interesting to investigate whether the conclusions drawn with respect to SSI would remain valid in case that the stages of spatial variability and site effects were skipped, that is, if the flexibly supported structure of SC6 was excited by the uniform motion of case SC1 (no site effects). Figure 13 presents the parameters examined, all normalized to the 'standard' case (SC7/SC1). The effect of considering SSI phenomena without accounting for asynchronous motion and site effects is indeed remarkable; unlike what happened in the previous case (SC6/SC4), response calculated accounting for SSI is typically lower, not only in terms of force but also of displacements. This suggests that ignoring the effect of local soil conditions underestimated significantly the results of the SSI analysis. The reason for such a difference, is that, as soil interacts with the foundation and the structural period elongates (an average increase of 38% resulted from the FE analyses), the structure becomes much more sensitive to long period pulses that have been amplified due to the presence of the soil.

Finally, it is also important to note that, as the soil varies among the supports, the inclusion of a (different) foundation flexibility and damping set for each pier is a source of input motion variability as well, especially in the case of different pile configurations or foundation types, thus leading to an increase of the importance of asynchronous motion and its ramifications.

Inelastic dynamic analysis (SC8)

At the last level of analysis complexity, the structure is allowed to enter the inelastic range (SC8), through an elastoplastic rotational spring [16], which combines the plastic rotation of the RC section with soil flexibility at the corresponding degree of freedom [1]. This is the most refined and realistic type of analysis, but also the most demanding in terms of FE model preparation and validation.

As previously, yielding of the concrete sections leads to an increase in both the energy dissipation and the natural period of the structures. The obtained moment–rotation curves for the base of the three piers of Model A during the SC8 case, are given in Figure 14. The

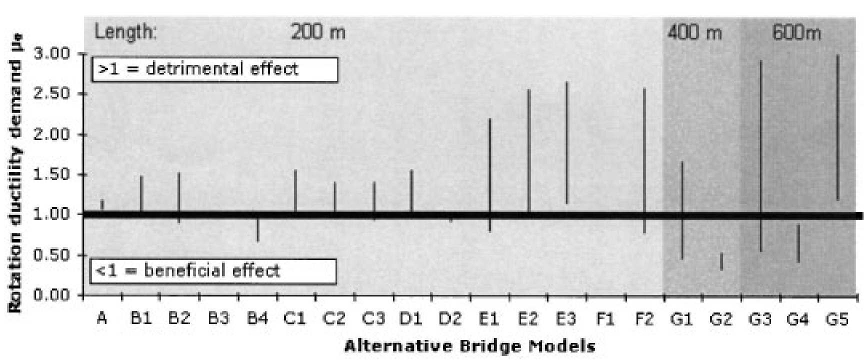

Figure 14. 'Complete' inelastic analysis (SC8) ductility demand normalized to 'standard' inelastic analysis approach (SC9).

calculated rotational ductility demand of the 'complete' analysis of SC8 is normalized to the ductility demand that would have resulted, had the 'standard' uniform excitation, fixed base, no site effects approach been employed. The results indicate that by ignoring the above phenomena the ductility demand could have been underestimated by approximately 25% on average compared to the comprehensive approach presented herein (Figure 14), which in the extreme case exceeds a factor of 3.

Another interesting observation is that non-linear pier response may also lead to increased relative displacements at the pier top, as a result of asynchronous yielding of the pier base.

CONCLUSIONS

An extensive parametric analysis was presented herein aiming to study the sensitivity of bridge response to spatial variability of ground motion, site effects and SSI phenomena. Based on the results of the comparative FE analyses, it may be concluded that the proposed methodology is a feasible and efficient way to generate more realistic earthquake motion scenarios than those commonly used and to account for the properties of the soil–foundation–pier system under seismic loading, within the context of a comprehensive approach. Moreover, the extensive application of the proposed methodology highlighted that:

(a) Significant coupling exists between spatial variability, effects of local soil conditions and SSI effects. Their relative importance cannot be easily assessed in advance. All these phenomena play an important role in the inelastic dynamic response of bridges and should be treated within the context of a comprehensive methodology.
(b) Bridges subjected to spatially variable input motions, are characterized by excitation of higher modes which are primarily antisymmetric. Symmetric structures no longer respond symmetrically and their dynamic behaviour cannot be adequately reproduced by any combination of synchronous motions.
(c) The effect of wave passage and loss of coherency in terms of absolute displacements and pier base bending moments is generally beneficial for short bridges, but also strongly related to the total length.
(d) Relative displacements increase even in short overall length bridges. Relative displacements also increase with the overall length, showing a tendency to follow a logarithmic increase at least for the range of lengths studied herein (up to 600 m).
(e) A tentative threshold value of 400 m may be considered for rendering consideration of spatial variability indispensable.
(f) Vertical displacements of the deck are affected by asynchronous excitation in the transverse direction. They show a clear trend to decrease, at least for span lengths smaller than 150 m.
(g) Site effects are an important part of the overall dynamic analysis process, both in terms of peak ground acceleration and spectral amplification. Site effects play also an important role in the reliable description of the SSI. Ignoring site response when studying SSI effects may introduce an error of the order of ±50% in terms of displacements.
(h) In general, SSI effects are beneficial in terms of forces developed, whereas an increase in the absolute and relative displacements should be expected. Nevertheless, the interaction of soil with the foundation and the structure is dependent on the modified

dynamic characteristics of the structure and the earthquake motion, hence cannot be assessed in advance. Such cases of detrimental effect of SSI on the dynamic response of bridges were also observed.

(i) Ground motion variation between support points is not only due to arrival delay and loss of coherency. Local soil amplification, kinematic interaction and asynchronous pier yielding are all sources of input motion variability.

(j) Ignoring the above interrelation, the ductility demand in bridge piers could be underestimated by 25% on average and up to a factor of 3 in an extreme case.

(k) Even for a constant set of earthquake motions, bridge structural properties (fundamental period, symmetry, regularity, abutment conditions, pier-to-deck connections), as well as its dimensions (span and overall length) modify the relative effect of the above phenomena.

(l) The assumption of advanced scenarios of earthquake motion did not alter all the response parameters beneficially, in any of the cases examined. Nevertheless, in a number of cases, it was only the relative displacements that were affected in an adverse way.

The research should be extended in order to verify the consistency of the above observations when different soil and earthquake properties are employed and to clearly define those cases where spatial variability, site effects and SSI are important for the expected action effects of bridges under earthquake loading. The latter is of particular significance but equally difficult to predict, bearing in mind the strong coupling of the above phenomena shown in this paper. Nevertheless, pending the studies required to draw general conclusions, it is tentatively proposed to follow a comprehensive inelastic dynamic analyses scheme such as the one presented herein, at least for the case of important bridges.

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