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# EFFECT OF ANALYSIS COMPLEXITY ON THE CALCULATED DUCTILITY DEMAND OF R/C BRIDGE PIERS

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## ABSTRACT
A detailed parametric analysis of different bridge structures is carried out, to highlight the potential effect of analysis and modelling assumptions on the calculated ductility demand of reinforced concrete (R/C) bridge piers. Through a comprehensive approach for dealing with spatial variability, site effects and soil-structure-interaction phenomena within the context of inelastic dynamic analysis of bridge structures, 20 alternative bridge structures that vary in terms of structural type (fundamental period, symmetry, regularity, abutment conditions, pier-to-deck connections), dimensions (span and overall length), as well as ground motion characteristics (earthquake frequency content and direction of excitation) are examined. It is concluded that the ductility demand calculated when ignoring the coupling of the above phenomena can, under certain circumstances, underestimate the actual conditions.

*Keywords:* Bridges: Spatial variability: site effects: soil-structure interaction: Inelastic dynamic analysis; Reinforced concrete members.

## INTRODUCTION
Capacity design is a well-established approach that aims at a controllable and hierarchically developed damage control. Its importance has been highlighted during recent earthquakes (Loma Prieta, 1989; Northridge, 1994; Kobe, 1995; Taiwan, 1999; Turkey, 1999) where it was shown that the design seismic forces may well be exceeded, and strength alone cannot ensure good seismic performance. Extensive research has been carried out, therefore, towards the identification of the optimal strength hierarchy within the structure and the subsequent selection, design and detailing of plastic hinges in pre-defined locations that would allow damage control and collapse prevention. The key parameters in this approach are the reliable estimation of the ductility demand and supply whose relative magnitude is crucial for the evaluation of the force redistribution and has to be assessed in advance. Nevertheless, a number of uncertainties related to the characteristics of the incident earthquake motion and the response of the foundation-soil system exist, directly affecting the calculated ductility demand, thus raising questions regarding the numerical accuracy of the capacity design procedure.

Along these lines, it was considered of particular interest to attempt to quantify the sensitivity of the inelastic dynamic response of bridges to three key simplifying assumptions made for the analysis. At first, the assumption that earthquake excitation is considered synchronous along the bridge length is examined. It is indeed very common to totally ignore spatial variability of ground motion and excite the structure uniformly although, both strong earthquakes and research results [1,2] provide nowadays convincing evidence that earthquake ground motion may significantly differ among the support points, especially for the case of long bridges. This spatial and temporal variation can be primarily attributed to the fact that the waves travel at a finite velocity, thus arriving at each support point with delay, while at the same time they are subjected to multiple reflections, refractions and superposition that leads to the loss of their coherency in terms of statistical dependence. Moreover, local soil conditions at each support point affect the frequency content and amplitude of motion in a different way, which depends on the degree of soil properties variation along the bridge length. 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 by the incoming waves. As a result, seismic motion at each foundation location may be significantly different in terms of amplitude, frequency content and arrival time, thus inducing, under certain circumstances, significant forces and deformations that would not develop if the assumption of synchronous excitation was adopted [3,4].

Another simplifying assumption often made during seismic design of bridges is that local site conditions are accounted for through the shape of a code-defined design spectrum, being therefore dependent on the reliability of the site categorization process. Although this is clearly a practical and necessary design approach, inevitably it does not consider the fact that soil columns are actually multi-layered and damped, hence they may potentially amplify both the spectral and the peak ground acceleration especially in cases of abrupt stiffness change or of complex subsoil conditions [5,6].

Whereas the above simplifications are anticipated on the basis of the difficulty to describe the stochastic nature of earthquake motion and soil properties within a deterministic scheme, there is an additional assumption that results from the reluctance to make recourse to the existing knowledge regarding soil-structure interaction (SSI) and utilise the numerous advanced modelling techniques for the representation of soil flexibility and damping under dynamic loading. As a result, bridge structures are commonly considered to be fully fixed at their pier base points, whereas it is well known that the bridge foundation is flexible, dissipates energy and interacts with the surrounding soil and the superstructure in such a way, that it filters seismic motion (kinematic interaction) while it is subjected to inertial forces generated by the vibration of the superstructure (inertial interaction) [7,8,9]. This phenomenon is very complex and its beneficial or detrimental effect on the dynamic response of the bridge is dependent on a series of parameters such as the intensity of ground motion, the dominant wavelengths, the angle of incidence of the seismic waves, the stromatography, the stiffness and damping of soil, as well as the size, geometry, stiffness, slenderness and dynamic characteristics of the structure.

The rationale behind such assumptions is the perception that they lead to conservative design, and also that attempting to incorporate more complex models leads to a multiparametric procedure which induces other uncertainties and that is often uneconomic and/or numerically sensitive. Furthermore, there is also lack of theoretical results and experimental data towards the combined effect of spatial variability, site effects and soil-structure interaction, not to mention that the few existing ones are often contradictory. Therefore, it was considered of particular interest to attempt to develop a comprehensive methodology and analysis tool that would allow a more refined inelastic dynamic analysis procedure which would be a reasonably simple, but still accurate way to account for the above phenomena towards a more reliable estimation of the expected ductility demand and subsequently, a higher level of confidence in the capacity design process.

## A COMPREHENSIVE APPROACH FOR INELASTIC DYNAMIC ANALYSIS

In view of the remarks made in the previous section, a methodology has been proposed and validated against theoretical solutions, alternative computer codes and recorded data, where available. This comprehensive methodology incorporates and uncouples all important issues (asynchronous motion, site effects, soil-structure-interaction) within the context of a general scheme for the inelastic analysis of bridges in the time domain [10]. The idea is to generate synthetic time histories which are 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, the latter being accounted for, primarily in terms of 1D site response analysis of multi-layer, damped soil profiles overlying an elastic bedrock but also in an envelope function approach for the extreme case where lateral surface waves propagation is expected to further amplify ground motion (2D site effects). Having defined distinct seismic motions at the foundation level of each pier, further modification of motion takes place in the frequency domain, in order to account for kinematic interaction between the soil and the foundation piles.

The derived motion can then be used as the asynchronous input motion to the bridge structure which is assumed to be supported on different Beam-on-Dynamic-Winkler-Spring systems, whose complex dynamic impedance matrices (i.e. stiffness and damping properties) are derived for all horizontal, rocking and coupled modes of vibration according to available solutions from the literature. Especially for the rotational stiffness, a non-linear moment-rotation relationship is proposed [10,11] which combines the rotational compliance of the foundation with a lumped plasticity model for the R/C section that accounts for the plastic rotations caused by yielding of the pier base. It is shown that in such a case, this specific spring element is characterized by a first branch stiffness equal to $\Im_{\Theta}$ that represents the rotational stiffness of the soil-foundation system and a second branch stiffness $\Im'_{\theta}$ equal to (Figure 1):

$$\Im'_{\theta} = \frac{1}{\dfrac{1}{\Im_{\theta}} + \dfrac{(0.08L + 0.022 f_{yl} \cdot d_{bl}) \cdot (\phi_u - \phi_y)}{M_u - M_y}} \tag{1}$$

where $M_y$ and $M_u$ are the yield and ultimate moment respectively, L is the distance from the critical section to the point of contraflexure, $f_{yl}$ is the yield strength of the longitudinal bars and $d_{bl}$ represents the diameter of the longitudinal reinforcement. With the complete set of linear and non-linear pier base springs and the distinct acceleration and displacement time histories at each support location it is feasible and relatively easy to perform dynamic inelastic analysis of the superstructure subjected to spatially varying motions and influenced by local site conditions and soil-structure interaction. The displacement ductility demand in the cantilever pier (typical case for excitation in the transverse direction) in the presence of the foundation flexibility and damping may then be evaluated through the well-established approach of Priestley at al. [12], as also presented in Figure 1. The rotational ductility demand can be calculated accordingly. In order to verify the feasibility of applying the above

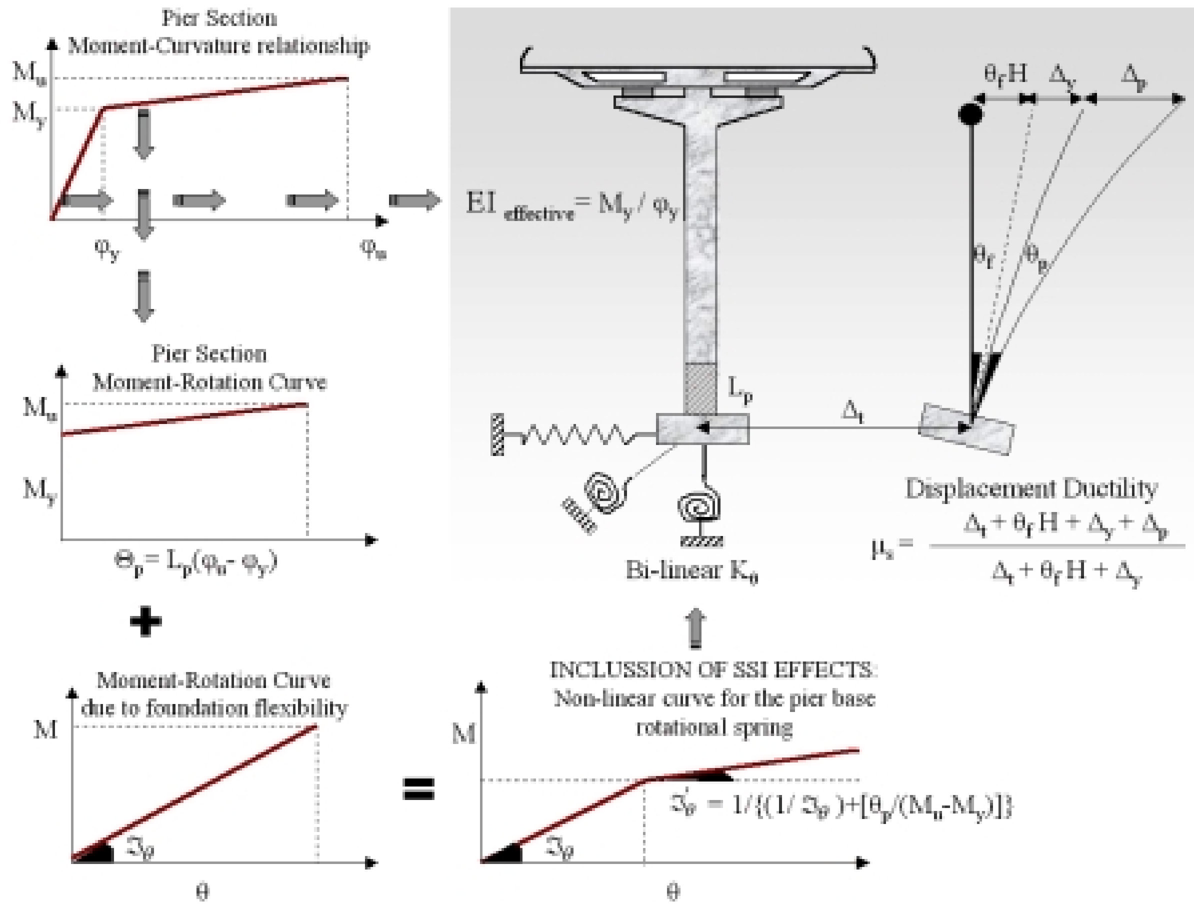


**Figure 1.** Non-linear rotational spring that accounts for soil flexibility and pier base yielding

procedure but also for evaluating the bridge sensitivity to spatial variability, site effects and soil-foundation-structure interaction through parametric analysis, the computer code ASING (Asynchronous Support Input Generator) was developed [10] providing with ready-to-use linear/non-linear springs and dashpots and distinct synthetic ground motions at each support point of the bridge while accounting for the above phenomena.

## OVERVIEW OF THE PARAMETRIC ANALYSIS SCHEME

In order to meet the above requirements, a well-studied bridge structure [13] was selected as the reference case of the parametric analysis and was denoted as Model A. It is a straight, asymmetric, four-span bridge of 200m total length, supported on hollow section piers of height that varies from 7 to 21m. The concrete deck consists of a hollow box cross-section, which was taken as uniform along the length of the bridge. Section geometrical characteristics and bridge overview are presented in Figure 2. The piers are assumed monolithically connected to the superstructure and the abutment bearings are pinned in the transverse while being free in the longitudinal direction. Material characteristics are in accordance with Eurocode 2 provisions; the concrete is class C30/35 and the modulus of elasticity is reduced by 50% to roughly account for R/C section 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 $\varepsilon_{cu}$ and the ultimate steel strain $\varepsilon_{su}$ were taken equal to 0.008 and 0.1 respectively. The R/C sections' moment-curvature (M-φ) relationship and the corresponding ultimate and yield values ($M_y$=54465 kNm, $M_u$=65064 kNm, $\varphi_y$=0,000781 $m^{-1}$, $\varphi_u$=0,00443 $m^{-1}$) were calculated through the fiber-analysis code RCCOLA-90 [14] while the required M-θ relationships for each one of the three piers were derived by estimating the plastic hinge length for each pier [12].

In order to investigate the effect of local soil conditions on the modification of earthquake motion, a hypothetical subsoil structure was assumed. Its geometry, stiffness, density and damping (in terms of quality factor Q) properties are also presented in Figure 2. Moreover, a pile group foundation was designed for all piers according to the Eurocode 7 and Eurocode 8 provisions, resulting in a 3×4 pile group system of 1m diameter, 45m long piles, arranged at an axial spacing ratio S/D=3, and connected through a 8.0×11.0×2.0m pile cap. The foundation mass was hence taken as 18% of the overall pier-deck mass. This relatively stiff pile group foundation may be primarily attributed to the soft upper soil layers but has also been selected for capacity design purposes, to ensure that the inelastic response would not take place at the piles and that no excessive rocking would occur at the soil-pier interface. The assumption was also made that the bridge is located within a seismic zone characterised by a peak ground acceleration of 0.24g. The Kallithea (ATH-03) signal was used, recorded by the permanent instrumentation array of ITSAK (Earthquake Engineering Institute, Thessaloniki) during the 1999, $M_w$=5.9, Athens earthquake (0.3g maximum acceleration at 0.2 seconds period), scaled to the level of the desired peak ground acceleration and deconvoluted at the bedrock level. All ground motion simulations as well as the linear/nonlinear springs and dashpots that are required at the three pier support points for the inelastic SSI analysis were calculated with the use of the aforementioned code ASING. Using these key input data calculated from the program, the structure was discretized in finite elements (3D beams) and analysed in the time domain using the commercial FE package SAP2000. Results are reported here for the reference structure excited in the transverse direction only.

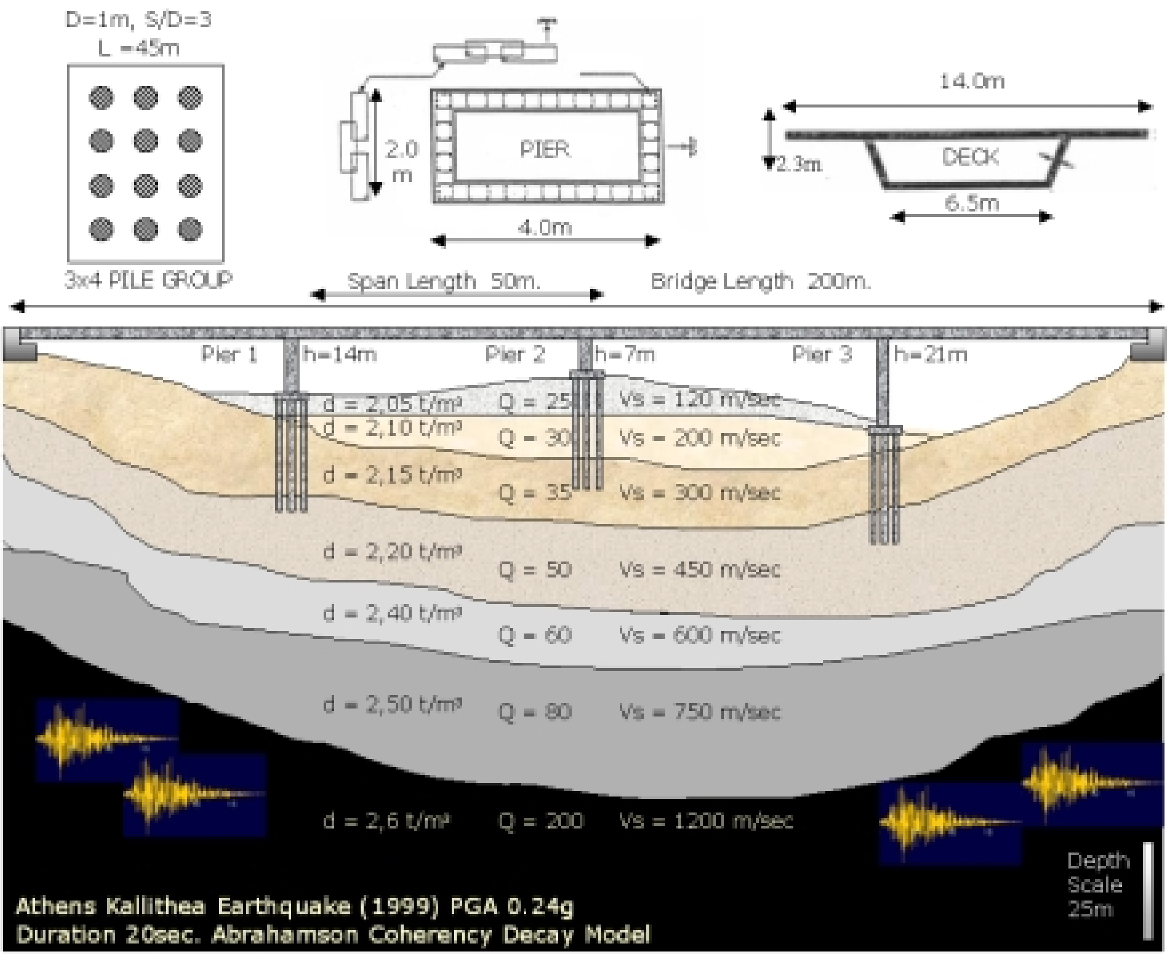


**Figure 2.** Overview of the Reference Bridge (Model A)

Apart from the study of the Reference Model A described above, it was deemed necessary to study the dynamic response of alternative bridge structures, in order to investigate the stability of the observed trends when selected parameters are modified, one at a time. At first, four bridge configurations were selected retaining the geometrical characteristics of Model A but representing different levels of stiffness and subsequent changes in the fundamental period (Models B1-B4). Bridge symmetry and regularity was also examined by appropriately modifying the height of the piers (Models C1-C3). By applying a uniform modification of the pier height at all piers, the dynamic characteristics of the reference bridge were then modified and the effect of fundamental period on the relative importance of spatial variability, site effects and soil-structure interaction was investigated (Models D1-D2). It was also deemed crucial to study the influence of different approaches regarding the deck-to-abutment connection, a key issue for both modelling and design. At first (Model E1), full fixity conditions were assumed for the left abutment. 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 accounted for (Model E2) and modelled through appropriate springs in the longitudinal and transverse direction, which were defined by applying a soil-abutment interaction methodology [15]. The recently popular construction approach of abutments that are free transversely (up to a limit deflection) 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 at all obvious whether the observations of the beneficial or detrimental effect of spatial variability on the transverse dynamic response of the bridge can be extended for cases that the structure is excited along its longitudinal axis (Model F1). Moreover, in order to examine the most critical situation, the deck was assumed in this case to consist of segments simply supported on the piers (a configuration that may lead to unseating of deck segments). The frequency content of the target motion was also modified in order to capture potential bridge sensitivity to the assumed earthquake input. Therefore, the case of a structure identical to the reference model but excited by the Loma Prieta, Gilroy bedrock motion instead of the Kallithea record, was examined in Model F2. A set of five additional bridge models was also considered in order to study the effect of total bridge and span length on the extent of the three phenomena studied (Models G1-G5). A number of calibrating assumptions was made, and presented elsewhere [10] in order to ensure that the 20 Models would refer to common conditions in terms of soil properties, deck design (for bridges of different span length) and stochastic properties of earthquake motion. The differences in each of the 20 alternative bridge structures are summarised in Table 1, where the variety of structural configurations as well as the wide range of their dynamic characteristics (in terms of fundamental frequencies achieved) are illustrated.

## COMPARATIVE INELASTIC DYNAMIC ANALYSES

The inelastic dynamic analysis of the alternative 20 bridge structures is performed by incorporating the non-linear rotational spring whose first branch represents the soil flexibility while the second branch additionally accounts for the plastic rotations that arise from the exceedance of the yield moment $M_y$ and the subsequent pier yielding, as described in the previous section. This special rotational spring is used together with the other (linear) springs ($K_x$, $K_{xr}$) and dashpots ($C_x$, $C_r$, $C_{xr}$) that were defined during the soil-structure interaction stage for each vibration mode (horizontal, rocking and coupled). It is pointed out that the synthetic seismic motions have been generated on the basis of a peak ground acceleration equal to 0.72g, in order to further highlight the features of the structure's inelastic behaviour.

In order to examine the sensitivity of the inelastic dynamic bridge response to the assumptions made in terms of spatial variability, site effects and soil-structure interaction and the analysis complexity adopted, two sets of analyses are employed: (a) a 'classic' approach which corresponds to the most common ('standard') case encountered in practice, that is a fully fixed, elastic bridge structure which is uniformly excited with the reference ground motion while no account is taken for the effect of the multi-layer soil formations, and (b) a 'comprehensive' approach which investigates the dynamic response of a flexibly supported bridge, subjected to spatially variable earthquake input for which wave passage, loss of coherency and site effects have been considered in the analysis.

The ratio of the rotational ductility demand calculated from the 'classic' to that from the 'comprehensive' approach is shown in Figure 3, while the actual M-θ relationships for the G5 bridge piers are illustrated in Figure 4. It is observed that the final rotational ductility demand is strongly affected by the assumption of pier base fixity, the neglect of site effects and the synchronous excitation approach. This significant scatter observed can be primarily attributed to the fact that through the proposed comprehensive methodology, effects of the propagation of seismic waves and the dynamic behaviour of the soil profiles and the soil-foundation systems for each pier often cancel each other in terms of both displacements and forces (and the subsequent inelastic response of the piers). Moreover, their coupling is also dependent on the dynamic characteristics of the structure studied each time, which are in turn affected by the excitation of higher modes due to wave passage effect [4,10] and loss of coherency, as well as by the soil-foundation system compliance. This modification of the structural

TABLE 1

ALTERNATIVE BRIDGE MODELS

| Model | Difference with respect to the reference bridge | $T_1$ (sec) | $T_{1\ transv.}$ (sec) |
|---|---|---|---|
| A | None (reference bridge) | 0.60 | 0.58 |
| B1 | $EI_{eff} / EI_{gross}$ = 100% | 0.42 | 0.40 |
| B2 | $EI_{eff} / EI_{gross}$ = 75% | 0.50 | 0.48 |
| B3 | $EI_{eff} / EI_{gross}$ = 40% | 0.67 | 0.64 |
| B4 | $EI_{eff} / EI_{gross}$ = 30% | 0.74 | 0.70 |
| C1 | $H_1$=14m, $H_2$=7m, $H_3$=14m | 0.61 | 0.46 |
| C2 | $H_1$=14m, $H_2$=14m, $H_3$=14m | 0.80 | 0.61 |
| C3 | $H_1$=14m, $H_2$=21m, $H_3$=14m | 0.92 | 0.77 |
| D1 | $H_1$=11m, $H_2$=4m, $H_3$=18m | 0.59 | 0.51 |
| D2 | $H_1$=17m, $H_2$=10m, $H_3$=24m | 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 400m. Span length 50m. | 0,70 | 0.70 |
| G2 | Overall length 400m. Span length 100m | 2.17 | 0.83 |
| G3 | Overall length 600m. Span length 50m | 0,69 | 0.69 |
| G4 | Overall length 600m. Span length 100m | 1.67 | 0,77 |
| G5 | Overall length 600m. Span length 150m | 3.05 | 1.13 |

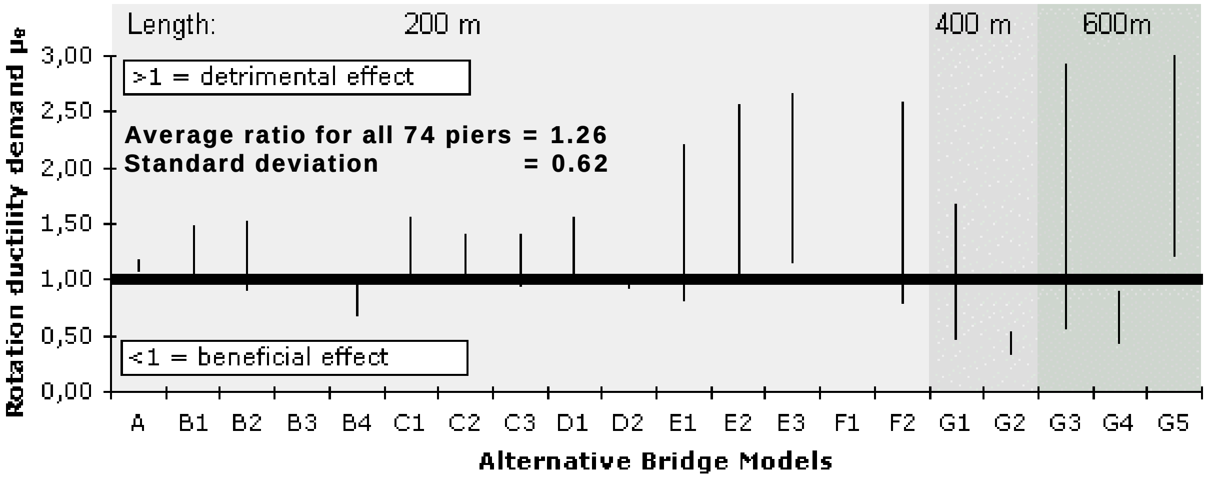


**Figure 3.** Ratio of the rotation ductility demand $\mu_\theta$ derived by the 'classic' and the 'comprehensive' approach

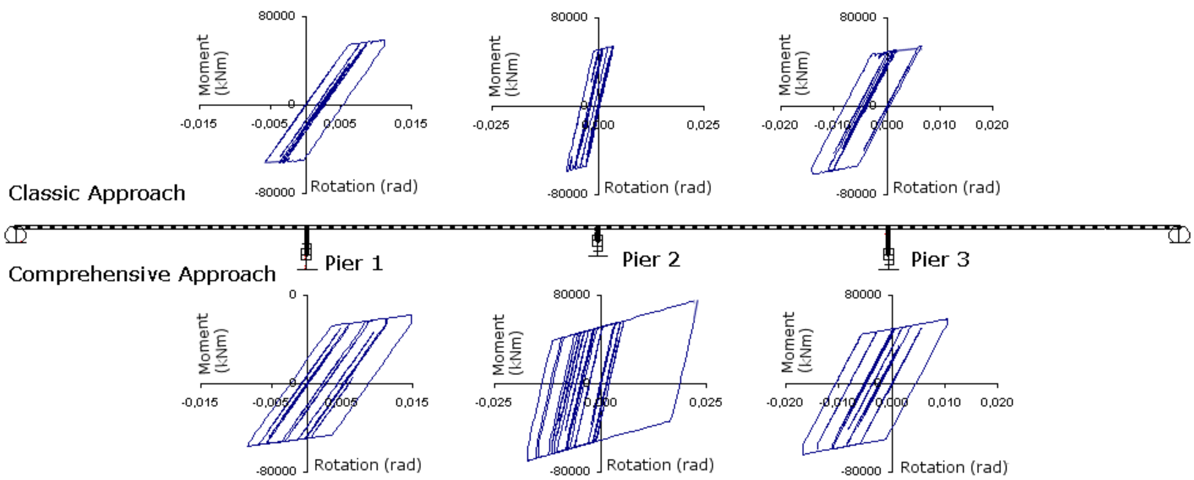


**Figure 4.** Calculated Moment-Rotation relationships at the base of Model G5 Piers according to the 'classic' and the 'comprehensive' approach

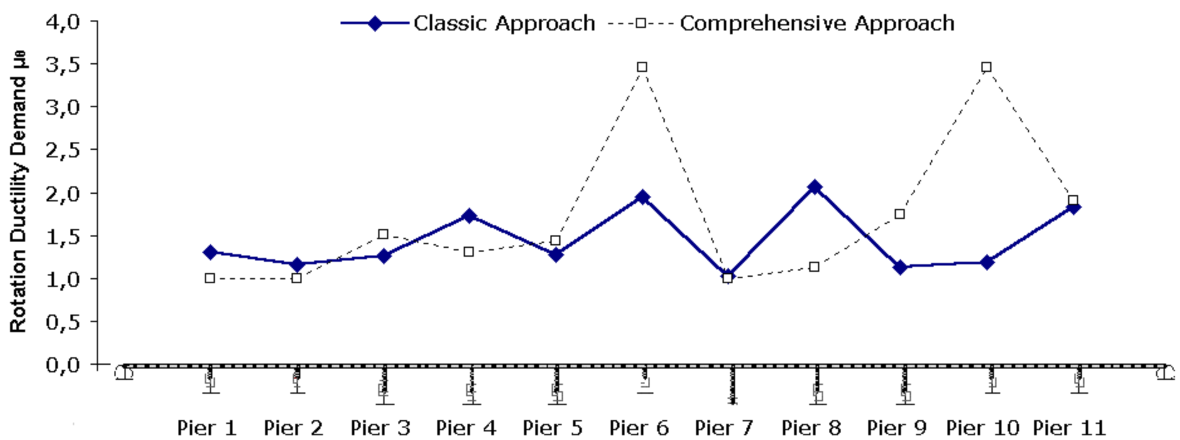


**Figure 5.** Calculated rotational ductility demand $\mu_\theta$ at the base of Model G3 piers according to the 'classic' and the 'comprehensive' approach

vibration frequencies is of paramount importance since it may be the key parameter to decide whether the amplification of particular seismic motion frequencies that occurs due to the complexity of the soil profiles would affect the structure. In fact, the worst case scenario refers to the case that the modified dynamic modes of vibration that result due to spatial variability and the foundation flexibility, practically lies within the range of the dominant period of motion as it depends on the incident earthquake motion characteristics and soil profile geometry and dynamic properties. In Figure 3 it is observed that such a detrimental combination of the above phenomena can lead to an average 25% increase in the ductility demand with respect to the assumption of a fixed base, uniformly excited bridge structure for which site effects are totally ignored, while a substantial increase by a factor of 3 is observed in the extreme case (Bridges G3 and G5). Such detrimental effect of SSI in the presence of earthquake motions rich in low frequencies has also been observed in the literature [16].

On the other hand, it is also possible that, under certain circumstances, spatial variability has a beneficial effect on the developed bending moments at the pier base. Moreover, it may occur that the amplification of motion due to site effects is not directly related to the periods of vibration of the bridge as they are modified due to soil flexibility, hence its influence on the dynamic response of the structure can be rather limited. In addition, the damping introduced at the foundation level and the filtering of particular seismic motion frequencies attributed to the kinematic interaction of the foundation with the (uniform) soil around it, are all source of inertial forces reduction, thus leading to ductility demand decrease with respect to the 'classic' approach. It is critical to note though, that even in cases that the calculated ductility demand is lower for particular piers when the 'comprehensive' approach is followed instead of the 'classic' one, it may occur (as seen in Figure 5 for bridge G3) that the overall demand distribution that may result by the 'comprehensive' approach may be less uniform, a detrimental effect which is of high significance.

It can be stated therefore, that the ductility demand of R/C bridge piers is indeed sensitive on the assumptions made during the dynamic analysis and may be either overestimated or underestimated when the seismic design is performed without appropriate consideration of spatial variability, site effects and soil-structure interaction phenomena. As a general trend observed, the latter is more likely to occur in cases that the earthquake motion is rich in low frequencies (higher probability that the increased, due to SSI effects, fundamental period of the structure lies closely to the dominant frequency range of the incident earthquake motion), the soil profile is characterised by abrupt stiffness changes (higher soil amplification), the overall topography and morphology of the subsoil structure lead to 2D or 3D site response phenomena (higher soil amplification) and the bridge is longer (higher possibility of detrimental bending moment increase due to wave passage and loss of coherency effects).

## CONCLUSIONS

A comprehensive methodology has been developed for accounting for asynchronous motion, effect of local soil conditions and soil-foundation-structure interaction phenomena within the context of inelastic dynamic analysis of R/C bridges. Through a parametric analysis scheme that involved 20 alternative bridge structures the sensitivity of the calculated ductility demand to a set of simplifying assumptions that are commonly made in practice was investigated. The results indicate that significant coupling exists between spatial variability, effects of local soil conditions and SSI effects while the degree of their coupling cannot be easily assessed in advance. Moreover, under certain circumstances the assumptions of pier base fixity, synchronous excitation and uniform soil profile may lead to a calculated ductility demand that is 25% lower on average and may underestimate the actual ductility demand by a factor of 3 in the extreme case. As a result, it is strongly recommended that for bridges of high importance, extensive parametric analyses be performed to estimate an envelope of the potential inelastic bridge response under various earthquake, soil and foundation conditions and establish a higher level of confidence with respect to the calculated ductility demands.

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