Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1269

IMPACT OF MODELLING ASSUMPTIONS FOR ASSESSING THE SEISMIC RESPONSE OF TWIN BRIDGES CONSIDERING SOIL-STRCUTURE-INTERACTION IN 3D SPACE

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ABSTRACT

The scope of this paper is to investigate the importance of various modelling assumptions on the overall seismic response of twin bridges (i.e. bridges constructed with two, almost identical branches) considering the coupling and dynamic interaction of the deck-abutment-backfill-embankment and superstructure-foundation-subsoil system, as well as different earthquake input scenarios, in terms of selected ground motion records. For this purpose, a real bridge already constructed in Greece is studied and a parametric analyses scheme is developed for different finite element models of increasing analysis complexity. It is shown that at least for the particular bridge studied, the most critical links in the chain of analysis reliability is the definition of a 'reasonable' earthquake scenario and the appropriate modelling of the abutment-embankment dynamic stiffness.

Introduction

It has been already shown through scientific research worldwide that a technical project should not be designed without considering the effect of soil-structure interaction especially in the case of structures of major significance, with specific dynamic characteristics or resting on soft and/or varying soil profiles. After years of research, the most efficient ways nowadays to account for this phenomenon in the time domain is (a) by de-coupling the problem to a kinematic and an inertial substructure (Mylonakis and Gazetas, 2000), that is, to separate the filtering of seismic waves due to the presence (and stiffness) of the foundation from the waves that are radiated back to the soil due to the vibration of the superstructure and (b) by numerically modeling the performance of soil, structure and foundation as a whole (i.e. Wolf, 1988). Despite the fact that significant advances have been made lately in terms of understanding and simulating the above frequency-dependent phenomenon, still, the highest level of uncertainty is associated with the definition of a 'reasonable' incoming wavefield that can be used for the dynamic excitation of a bridge structure and that is certainly not known in advance in a deterministic way. The scope of this paper is to investigate the relative importance of a number of analysis assumptions made with respect to earthquake loading and the finite element approach adopted on the final performance

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of R/C bridges. In particular an effort is made to:

(a) quantify the uncertainty in structural response that is related to the selection of an 'appropriate' set of earthquake records, according to the current seismic code provisions, for the dynamic analysis of ordinary highway R/C bridges,

(b) quantify the potential influence of the embankment-abutment stiffness, damping, and local site amplification as well as the impact of other decisions related to the dynamic stiffness of large caissons and the modeling of bridge bearings, stoppers and gaps, and,

(c) investigate the actual degree of coupling (if any) between nearby twin bridge branches (i.e. bridges constructed closely to service traffic in both directions but designed as single structures ignoring the wavefield radiated back to the soil by their vibration the nearby branch).

The latter bridge-to-bridge inertial interaction phenomenon has not attracted scientific attention, primarily due to the large computational effort required to discretize the large subsoil domain involved, but has been found significant in cases of similarly massive structures, such as, tanks or nearby silos as well as in the case of large buildings responding in a coupling manner at a city scale (Tsogka and Wirgin, 2003; Ghergu and Ionescu, 2009; Wirgin and Bard, 1996, Padron et al., 2009). Along these lines, the 2nd Kavala Bypass Ravine Bridge, already constructed in northern Greece, was chosen and modeled in 3D space, considering the soil-pier-superstructure and the embankment-abutment-deck interaction, as well as the actual bridge configuration.

Overview of the bridge studied and the earthquake scenarios developed

The 2nd Kavala Bypass Ravine Bridge (Ntotsios et al., 2008) is a newly built bridge located in Section 13.7 of Egnatia, a major 670km highway constructed on the traces of the ancient Roman path, crossing northern Greece from its western to its eastern border. Its overall length is 170m and comprises of two statically independent branches, with four identical simply supported spans of 42.5m. Each span is built with four precast post-tensioned I-beams of 2.80m height, that support a continuous, along the overall length, deck of 26cm thickness and 13m width. The I-beams are supported through laminated elastomeric bearings placed on the two abutments and the three middle piers (M1, M2 and M3). The latter have a 4×4m hollow cross-section, 40cm wall thickness and heights equal to 30m (M1, M3) and 50m (M2), all supported with large caissons on relatively stiff soil (corresponding to soil class "A" according to both the Greek Seismic Code and the Eurocode 8 soil classification). The four spans of the deck are interconnected through a 2-m long 20-cm thick continuity slab over the piers. The site of the bridge belongs to Seismic Zone I according to the Greek Seismic Code, which is characterized by a Peak Ground Acceleration of 0.16g. The particular bridge is permanently monitored by Egnatia Highway S.A.

Selection of earthquake ground motions

In order to assess the effect of modelling assumptions on the overall seismic response of the coupled bridge-abutment-embankment-subsoil system, a number of sequential parametric dynamic analyses were performed using finite element models of increasing complexity.

Figure 1. Typical examples of bridges built as twin branches to serve traffic in two directions: (a) 2nd Kavala Bypass Ravine Bridge studied herein (b) Bridge G4 and (c) Polymylos-Veria Bridge (courtesy of Egnatia Highway S.A., Greece).

In this context, the required earthquake record sets were defined based on the prescriptions of Eurocode 8-Part 2 for bridge design (CEN, 2005) according to which seven pairs of earthquake records were selected accounting for the site-specific seismotechtonic conditions as related to the earthquake magnitude and source distance. Given the target spectrum provided in Eurocode 8 - Part 1 (CEN 2003), seven pairs of horizontal ground motions were selected and the mean structural response was obtained. It is noted herein that Eurocode 8 also permits the formation of sets using three records, provided that the maximum structural response is considered. The selection of the seven pairs of records was based on four additional steps as denoted in § 3.2.2.4 of EC8-Part 2:

- SRSS spectra were first calculated for all pairs of horizontal ground motions,
- the average ensemble spectrum of the set was derived from the individual SRSS spectra,
- the ensemble spectrum was then scaled to match the (EC8-Part1) 5% damped elastic response spectrum, multiplied by a factor of 1.3, to account for multi-directional excitation. Matching was imposed in the period range ($0.2T_1$ - $1.5T_1$), where T_1 is the fundamental period of the bridge.
- the scaling factor required for the above matching was applied uniformly to all individual seismic motion components.

It has to be noted herein the aforementioned earthquake record selection process constitutes a significant source of uncertainty of its own, since it may lead to large structural response discrepancies despite the final averaging of the response obtained (Iervolino et al. 2008; 2009, Sextos et al., 2009, Katsanos et al., 2009). Along these lines, in order to investigate the importance of selecting a particular set of seven pairs of earthquake records, compared to the selection of another set (also complying with the EC8 spectral matching requirements), two different Sets of earthquake records (A and B) were formed using accelerograms from the European Strong-Motion Database (ESD, Ambrasseys , 2000), and reflecting the overall seismotechtonic environment of the South-eastern Mediterranean (Tables 1 and 2). The resulting uniform scaling factors of the two sets were found equal to 2.36 and 2.77 respectively. All records were applied at the support level of the fixed-based structures or were appropriately deconvoluted to the bedrock level for the case of finite element models where the soil volume was modelled in 3D space, thus, considering the effect of local soil conditions in terms of different amplification among the abutments and the pier supports. The vertical component of seismic actions (§ 3.2.2.4, § 4.1.7 of EC8), near source effects (§ 7.4.1.3) and explicit (i.e. additional to ground motion variability attributed to local site effects) asynchronous excitation (§ 3.3, Annex D of EC8) were not considered; the latter decision was based on the observations of previous studies for the particular bridge (Sextos et al., 2003a; 2003b, Sextos and Kappos, 2008) where it was shown that, primarily due to the short overall length of the structure, the importance of wave incoherency and passage effects was minor compared to that of local soil conditions.

Table 1. Selected records for the set A (ESD).

Seismic event - Country	Date	Station Name	M_S	Soil	code
Friuli – Italy	15.09.1976	Kobarid - Osn.Skola	5.98	alluvium	000138
Biga – Turkey	05.07.1983	Goven - Meteoroloji	6.02	stiff	000352
Campano Lucano – Italy	23.11.1980	Calitri	6.87	stiff	000288
Lazio Abruzzo – Italy	07.05.1984	San Agapito	5.79	stiff	000366
Manjil – Iran	20.06.1990	Qazvin	7.32	alluvium	000476
Montenegro – Montenegro	15.04.1979	Petrovac – H. Oliva	7.04	stiff	000196
Umbro Marchigano – Italy	26.09.1997	Matelica	5.9	stiff	000602

Table 2. Selected records for the set B (ESD).

Seismic event - Country	Date	Station Name	M_S	Soil	code
Montenegro – Montenegro	24.05.1979	Bar – Skupstina	6.34	stiff	000228
Umbro Marchigano – Italy	14.10.1997	Norcia	5.6	stiff	000640
Caldiran – Turkey	24.11.1976	Maku	7.34	stiff	000153
Friuli – Italy	11.09.1976	Forgaria - Cormio	5.52	stiff	000123
Heraklio – Greece	01.03.1984	Heraklio - Prefecture	3.9	stiff	000355
Ionian – Greece	23.03.1984	Argostoli - OTE	6.16	stiff	002015
Kars – Turkey	30.10.1983	Horasan - Meteoroloji	6.74	stiff	000354

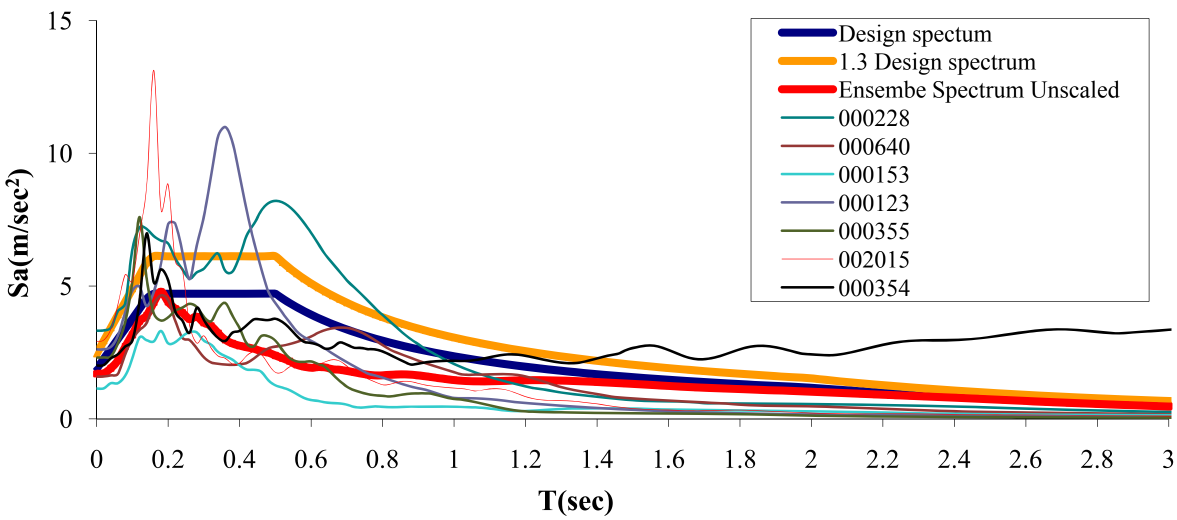

Figure 2. SRSS response spectra of the Set A selected ground motions, their ensemble spectrum (without scaling) and the EC8 target spectrum for PGA equal to 0.16g.

Numerical analysis framework

In order to assess the relative impact of the modelling assumptions made, a series of finite element models were developed at increasing levels of analysis complexity. The numerical simulations were carried out with the FE-Code ABAQUS 6.8, starting from a simple fixed base frame superstructure ('1D-Fixed'), a spring supported frame bridge ('1D-Springs') for which the foundation dynamic impedance matrix was derived according to Gazetas (1991) analytical expressions and a 3D fixed-base superstructure ('3D-Fixed') where bearings, I-beams and stoppers were modeled in maximum possible detail in 3D space (Figure 3). Having established a level of confidence between the 1D and 3D finite element models, the exact geometry of the abutment-backfill-embankment system and the middle piers-caisson-soil substructure system was also considered using 73170 elements (model '3D-3DSoil' illustrated in Figure 4). The alternative approach of a monolithic abutment-deck connection was also investigated in model '3D Int-3Dsoil" as an upper bound of the abutment contribution for resisting seismic forces. The most refined model developed ('3D Twin-3Dsoil') consisting of 243580 3D elements, involves both branches of the twin bridge, their abutments and caissons as well as the complete soil volume in the vicinity of the structures (Figure 5). Due to the size of the models and the subsequent computational cost, all analysis were linear elastic using cracked section properties (i.e 2/3 of the gross stiffness according to the Greek Seismic code) for the piers and appropriately reduced soil stiffness based on the computed levels of strain. A uniform Rayleigh damping of 6% was adopted for the system under study while absorbing lateral boundaries were also implemented to eliminate wave reflections. More details with regard to the alternative finite models developed can be found in Faraonis (2009).

Table 3. Summary of alternative FE models developed

Name (*Deck-Soil*)	Super-structure	Subsoil	Abutments	Embank-ments	Twin branches	Abutment-Deck
1D – Fixed	1D	Fixed	No	No	No	Joint
3D – Fixed	3D	Fixed	No	No	No	Joint
1D – Springs	1D	Springs	No	No	No	Joint
3D – 3DSoil	3D	3D	Yes	Yes	No	Joint
3DInt. – 3DSoil	3D	3D	Yes	Yes	No	Monolithic
3DTwin – 3DSoil	3D	3D	Yes	Yes	Yes	Joint

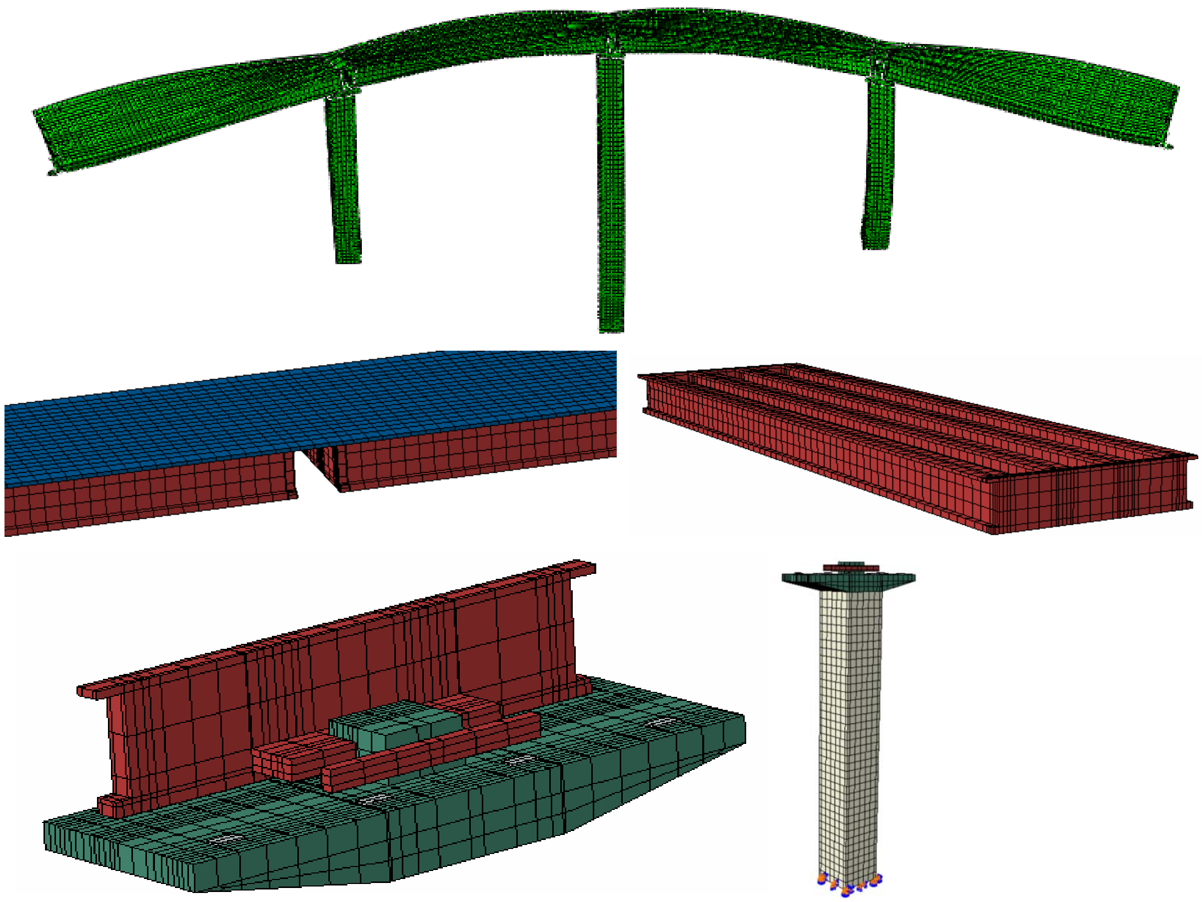

Figure 3. Overview and fundamental mode in the transverse direction of the fixed-base, 3D superstructure finite element model ('3D – Fixed', T=1.369sec) as well as modeling details of the deck, stoppers and bearings.

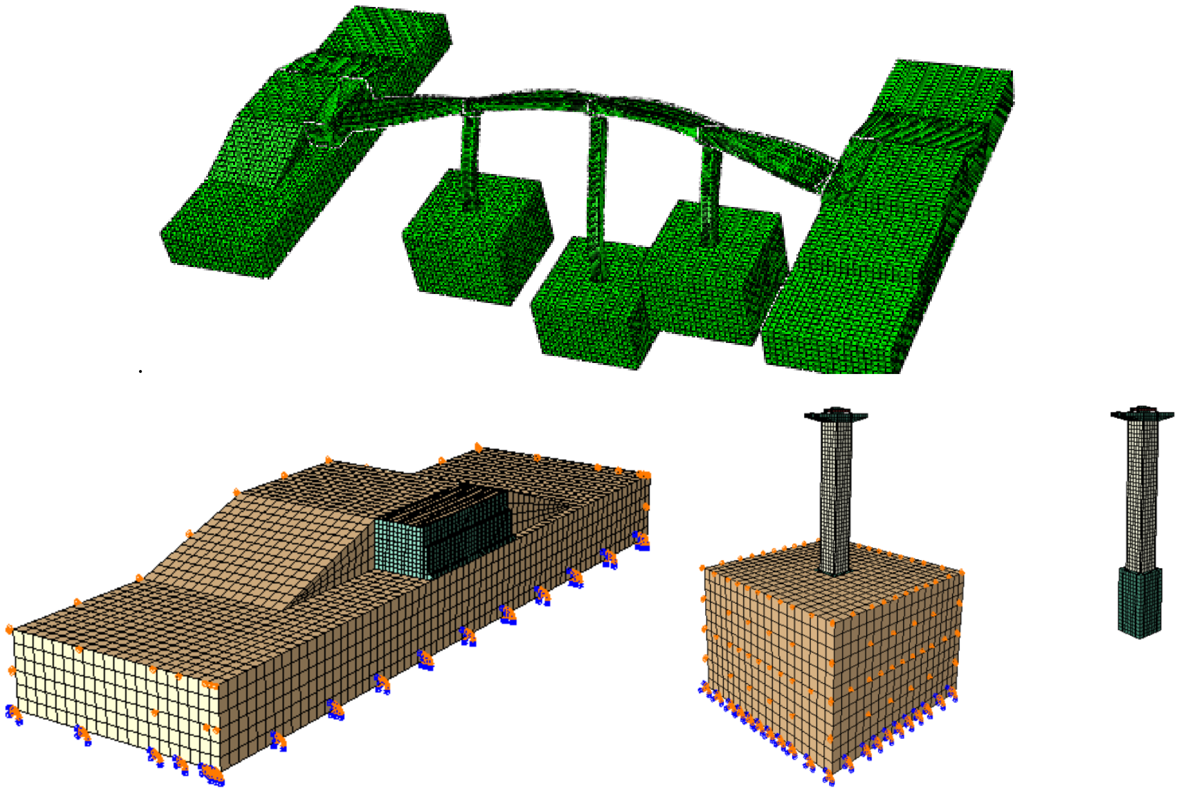

Figure 4. Overview and fundamental mode in the transverse direction of the flexibly supported model of the bridge ('3D – 3DSoil', T=1.414sec) as well as modeling details of the abutment-embankment system and the pier foundation subsoil.

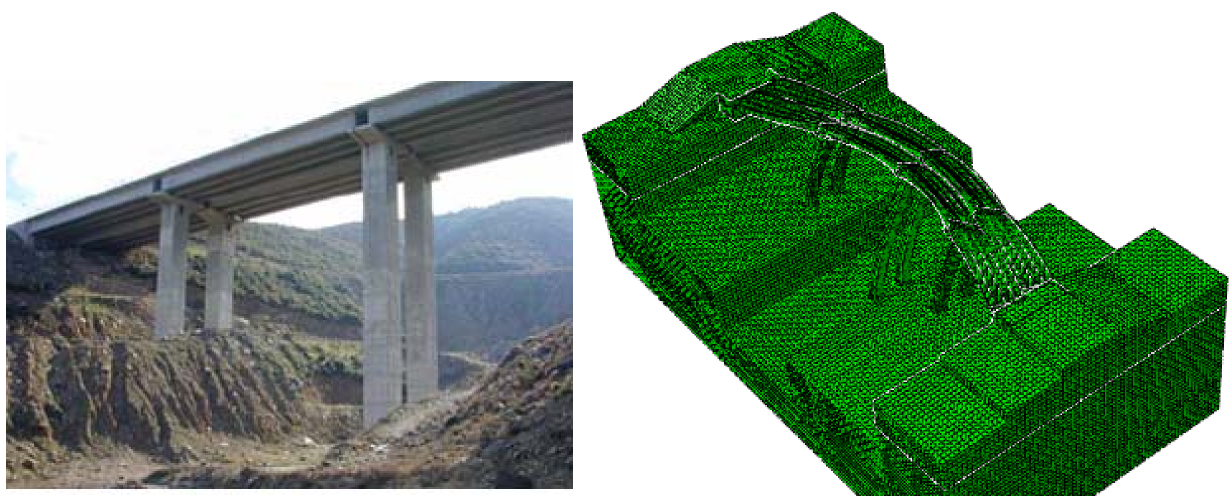

Figure 5. Overview and fundamental mode in the transverse direction of the flexibly supported model of the twin bridge ('3DTwin – 3DSoil', T=1.429sec).

Comparative evaluation of the results and conclusive remarks

For the above alternative finite element models, (7 records) x (2 Sets)=14 transient dynamic analyses were performed using the 14 pairs of the selected records simultaneously along the two principal directions (x-x and y-y). It is only the complex '3DTwin – 3DSoil' model that was subjected to a single pair of records due to the high computational cost. Figure 6 illustrates the coefficient of variation of the pier top displacement demand of all piers (M1, M2 and M3), for the two sets of earthquake records selected (A and B) and for each specific direction. It is seen that in general, the dispersion of the displacement demand increases with the complexity of the model for all piers and directions. Nevertheless, as an average, it can be claimed that the selection of a particular set of earthquake records (A or B) leads to a value of C.O.V. that is approximately equal to 0.25. In other words, although earthquake record Sets A and B were deemed equally 'legitimate' according to Eurocode 8, they lead to a non-negligible dispersion of structural response (at least in terms of pier top displacements), independently of the finite element model used. This observation highlights the limitations of the EC8-based earthquake record selection process to achieve a stable mean of structural response for the case of bridges, following a similar observation made for multi-storey R/C buildings (Sextos et al., 2009, Katsanos et al. 2009). Figure 7 on the other hand, attempts to isolate the relative effect of various modelling assumptions made by dividing the displacement demand of each pier for specific finite element models and all earthquake records used in order to compute the corresponding coefficient of variation. Although such a comparison is rather indirect and would normally require a comprehensive Monte Carlo analysis scheme, it is deemed that it provides a rough estimate of the relative impact of specific modelling decisions such as: (a) the 1D frame superstructure approach compared to the 3D modelling of the deck (1D-Fixed over 3D-Fixed), (b) the representation of foundation stiffness through springs or 3D soil elements (1D-Springs over 3DSoil), (c) the incorporation of the abutment-embankment stiffness and site amplification (3D-Fixed over 3D-3DSoil), (d) the activation of abutment stiffness due to the closure of the abutment-deck joint or due to the integral abutment-deck connection (3Dint-3DSoil over 3D-3DSoil) and (e) the aforementioned effect of selecting Set A or Set B pairs of seven earthquake records. It can be seen that in the longitudinal (x-x) direction, the largest discrepancy results from the activation (and appropriate modelling) of the abutment-embankment stiffness independently of whether the joint remains open or closes (i.e. it is recalled that the deck is connected with abutments through bearings hence forces are in any case transmitted to the abutment-embankment system). It is also observed that the uncertainty related to the selection of earthquake ground motion is one of the most important analysis parameters, thus highlighting the necessity for more reliable seismic code provisions for the selection of ground motions appropriate for transient dynamic analysis.

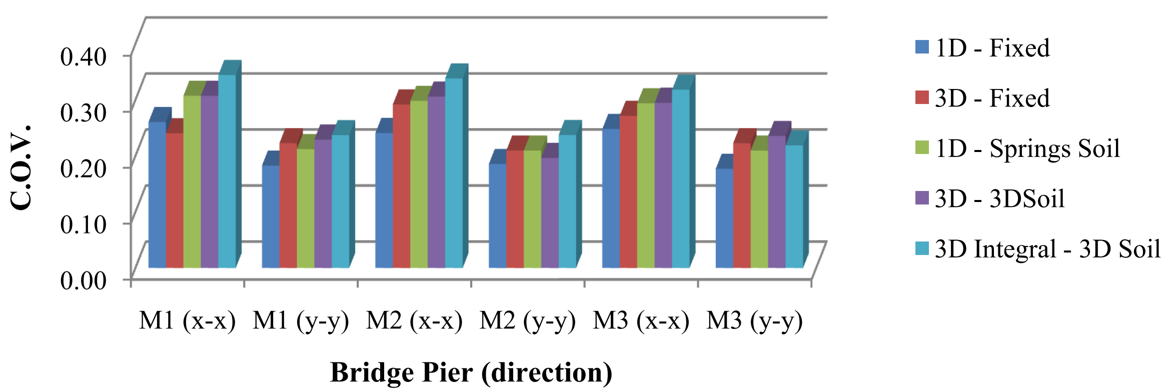

Figure 6. Longitudinal (x-x) and transverse (y-y) direction pier top displacement discrepancy attributed to the selection of earthquake record Set A or Set B for five different levels of modeling complexity.

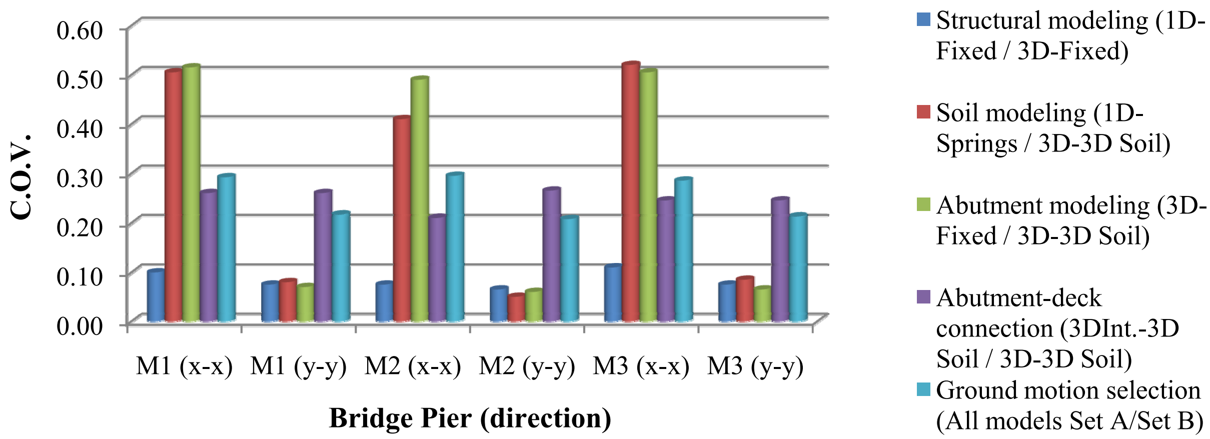

Figure 7. Longitudinal (x-x) and transverse (y-y) direction pier top displacement discrepancy attributed to various modeling and analysis assumptions.

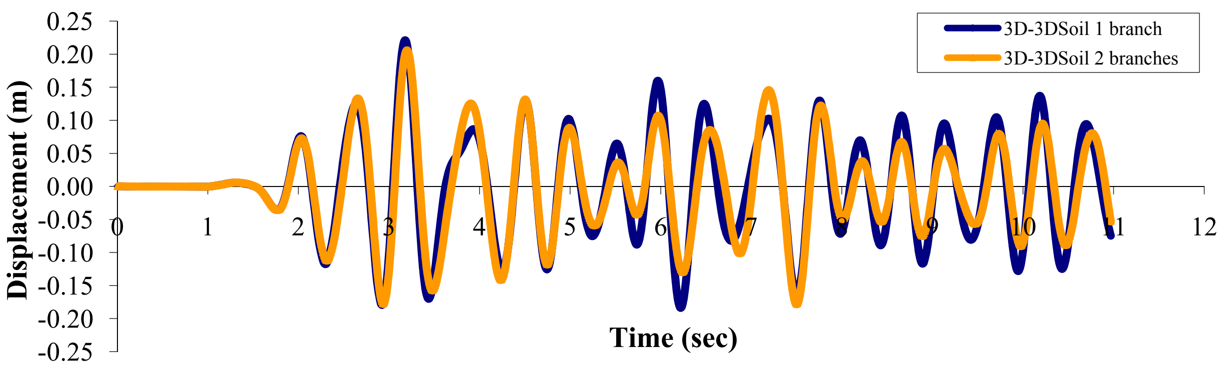

Figure 8. Longitudinal pier M2 top displacement time history for the single and the twin bridge.

Figure 8 illustrates the inertial interaction between the two bridge branches. It is seen that when the presence of the southern branch is considered, the displacement demand of the northern branch for a specific earthquake event (Friuli – Italy) is reduced up to 40% at specific points in time. This visible difference might be attributed to the fact that the longitudinal vibration of each deck leads to the torsion of the (common for the two branches) abutment, due to the eccentricity of the longitudinal seismic forces transmitted from each branch. However, no actual effect is observed in terms of maximum displacement. It is also notable that in the transverse direction the two displacement response histories (with and without coupling) are also found almost identical. As a result, it can be concluded that at least for the particular case and the (unique) earthquake case studied, the impact of the two branches coupling is minimal compared to the structural response dispersion attributed to the other modeling assumptions. In any case, the particular problem requires further investigation especially for the case of stronger ground motions (where the radiated wavefield due to the vibration of the superstructure is expected to be of higher amplitude), or potential resonance between the incoming wave frequency content, the soil profile fundamental mode and the structural dynamic characteristics.

Acknowledgements

The authors wish to thank Dr P. Panetsos (EGNATIA S.A., Thessaloniki, Greece), for his valuable assistance regarding the study of Kavala Bridge as well as Prof. A. Kappos (Aristotle University Thessaloniki), scientific responsible of the Research Project 'Seismic Protection of Bridges' funded by the Greek Secreterial of Research and Technology, within the framework of which the preliminary analyses regarding the Kavala bridge were conducted.

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