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CONTRIBUTION TO THE IMPROVEMENT OF SEISMIC PERFORMANCE OF INTEGRAL BRIDGES

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ABSTRACT

Integral bridges are jointless bridge structures with continuous deck and monolithic pier-deck and/or deck-abutment connections. Mainly constructed in the U.S., but also built in Europe, integral bridges are structures of high redundancy that due to their increased stiffness exhibit lower seismic displacement demand while they do not present the disadvantages of the non-monolithic bridges such as the salt, dirt and eventually corrosion at the expansion joints. On the other hand, the inevitable penalty, are the unfavorable internal stresses attributed to the temperature variations. Integral bridges can be distinguished in three main categories mainly on the basis of the pier-deck and/or deck-abutment connections as well as on the flexibility of the abutment-backfill system: (a) integral bridges with monolithic deck connected to the abutment with expansion joints (b) integral bridges with movable abutments and (c) integral bridges with rigidly supported abutments. Each one of the above cases requires a different approach in terms of seismic design. What is examined in this paper, is the role played by the abutment when it is monolithically connected to the deck and its potential implications on the bridge integrity, regularity and complexity. For this purpose, a typical, conventional bridge belonging to the case (a) above (i.e. with joint-type deck-abutment connection) has been used as a 'reference' case while two alternative bridge systems (involving innovative abutments of appropriately modified rigidity and connectivity to the deck) have been used for comparison purposes. The target is the reduction of the uncertainties resulting from the currently applied practice. By comparing the three characteristic bridge types, it is concluded that the dynamic response of the structures in the critical longitudinal direction depends on the overall dynamic stiffness of the deck-abutment-foundation system. Moreover, drastic reduction of seismic demand is generally observed when the abutment stiffness is activated, provided that the

arising serviceability problems are appropriately tacked. As a result, the implementation of rigidly supported abutments on integral bridges can be seen as a possible and promising solution for the reduction of the seismic displacements of particular bridges.

INTRODUCTION

As it is known the design of bridges has to compromise both its functional and earthquake resistant performance, as they are conflictful components of the same problem and they impose opposite design requirements. The functional problem, which is mainly critical for the longitudinal direction of the bridge, requires the free contraction and expansion of the deck, due to annual thermal cycle, shrinkage and creep. On the other hand, the earthquake resistance of the bridge is enchanced by the use of monolithical systems.

During the last decade, the state-of-the-art and practice for R/C bridges worldwide is related to the construction of systems that are as monolithical as possible, as well as to the development of new technology bearings and damping devices that are interjected between the deck and the piers. The parallel progress in those two fields of modern, earthquake-resistant, Bridge Engineering, although may seem rather competitive, it in fact leads to designs where both monolithical connections and seismic isolation are used in a complementary way. An example of such a dual approach is the case of bridge systems whose deck is continuous and monolithically connected to the central piers while it is supported on the other end piers through high damping bearings offering both relative economy and efficiency.

Floating deck bridges, on the other hand, are systems whose deck is supported on piers through steel reinforced bearings, allowing the free movements of the deck in the longitudinal direction, without transmitting perceivable stress to the piers at the serviceability limit state. However, the reduction of the in-service distress of the piers in floating bridges is counteracted by the higher earthquake resistant requirements imposed. According to most seismic codes provisions, seismic design these systems has to be performed with the use of a behaviour factor equal to 1, which essentially implies that the bridge has to remain elastic for the design earthquake. As a result, the aforementioned requirements inevitably lead to economically burdensome solutions due to the resulting increase in the dimensions of the piers and the capacity of the bearings. Moreover, the latter have to be replaced regularly while the foundations required to safely transmit the high seismic forces to the ground are also more expensive.

Integral bridges have superior seismic performance compared to the aforementioned floating deck systems since they take advantage of the inherent ability of reinforced concrete to dissipate part of the induced seismic energy by hysteretic behaviour while their redundancy is high. Moreover, integral bridges do not require maintenance and replacement of expensive components, such as bearings and expansion joints. Nevertheless, limitations also exist; significant internal forces may develop due to the temperature variation, creep and shrinkage, especially at the (also critical from a seismic point of view) end piers which are usually short due to the geomorphology of the valley. As a result, integral bridges can be effective when the length is short to moderate.

It is questionable though, what the main role of the abutment is to the overall bridge integrity, since both the abutment and the backfill have their own stiffness and damping and their potential monolithical connection with the deck affects not only the structural stiffness of the bridge but also its dynamic characteristics. Along these lines, the present paper aims at investigating the efficiency of two different and innovative rigid abutment configurations in order to identify whether the activation of the abutment stiffeness can enchance the seismic performance of the structure by reducing the anticipated seismic demand.

THE ROLE OF THE ABUTMENT TO BRIDGE INTEGRITY

Nowadays, there is no common international agreement for the definition of monolithical (or integral) bridges. In Germany for instance, a bridge is characterised as integral when the deck is monolithically connected to the piers, while in the U.S., in addition to the above, it is required that the movable abutments are also monolithically connected to the deck.

Within this context, it is of particular interest to investigate the effect of the deck-abutment connectivity and the parametrs that control the modification of the system’s stiffness, since the particular location of the structure is in many aspects of major importance. In fact, in integral bridges the functional movements are maximized at the ends of the continuous deck, where the monolithical connection of the abutment with the deck is, while the central piers being usually higher and as such, more tolerant to deformations, experience lower functional displacements. Furthermore, these movements are essentially cyclic, hence the active and passive earthpressure of the backfill soil is an additional parameter that affects the overall abutment-integral system stiffness. As temperatures change daily and seasonally, the length of an integral bridge increases and decreases, pushing the abutment against the backfill and pulling it away. As a result, the bridge superstructure, the abutment, the backfill, the foundation piles (if any) and the foundation soil are all subjected to cyclic loading, while considerably interacting with each other.

Understanding this dynamic interaction is obviously of particular importance for the effective design and performance of integral bridges. Moreover, the cyclic service movement of the monolithical abutment’s top gradually leads to the alteration of the mechanical properties of both the backfill and the foundation soil. As a result, the ability of piles to accommodate lateral displacements is a significant factor in determining the maximum possible length of integral bridges. To this end, pile stresses should be kept as low as possible. Semi-integral short abutments, whose hinge detail reduces pile stress, in combination with flexible steel H-piles [1] give the ability to the piles and the abutments to withstand such cyclic loads.

Another important issue relative to the above, is the progressive development and the irreversible increase of the passive earthpressure of the approach embankment, to the integral abutment, which is known as ‘ratcheting’. This increase over time of the earthpressure results from the wedging of the soil during the annual winter contraction of the deck. The deck draws out the abutment to the centre of the bridge, leaving a subsidence behind, which is filled by soil. The wedged soil does not return to its original position due to its inherent nonlinear behaviour or flow and can thus occur with any type of soil, no matter how properly it was placed during original construction. For the solution of the ratcheting problem a combination of structural solutions of an EPS compressible inclusion, which separates the web of the abutment from the approach fill have been proposed. This inclussion is used together with an appropriately reinforced backfill soil [2].

Apart from the serviceability related problems, the seismic interaction between the embankment, the abutment and the bridge supperstructure is of particular importance. It has to be noted that such interaction is not only affecting stiffness and damping but it is also related to the modification of seismic energy that is input through the abutments (kinematic interaction), a problem that is both complex and multiparametric, while the determination of the dynamic soil properties at high strains (as expressed by the G-γ-d curves) is also necessary. Inertial soil - structure interaction is taken into account by the implementation of the impedance factors ($K + iC\omega$), which compose the dynamic resistance of the backfill soil to the relative displacement and velocity of the interacting parts. Recognizing that soil–structure interaction appreciably affects the earthquake response of conventional highway overcrossings, Zhang and Makris [3, 4] developed a validated procedure to compute the kinematic response functions and dynamic stiffnesses springs and dashpots of approach embankments. This work essentially extends the work of Wilson and Tan [5] by leading to closed-form solutions that incorporate a 'critical length', L_c, which is the ratio of the transverse static stiffness of an approach embankment to the transverse static stiffness of a unit-width wedge.

TWO PROPOSED ABUTMENT CONFIGURATIONS

In the present investigation, two types of rigid abutments are proposed, both monolithically connected with the deck of the bridge. The abutments are appropriately designed for the serviceablity and ultimate limit state requirements aiming at enhancing the bridge performance compared to the use of conventional configurations that are extensively used in practice nowadays. Figure 1 illustrates the two suggested non-conventional configurations of the bridge abutment. Figure 1(a) in particular presents the first configuration of the abutment (indicated as 1st abutment type). In this case, the abutment is monolithically connected to the deck of the bridge while being in contact with the approach embankment. The web of the abutment is not connected with the stiff wing-walls. The width of the joint, which separates the wing-walls and the monolithical abutment, does not allow their contact for the functional and seismic displacements. The web of the abutment has a relatively small thickness and enough height in order to receive the functional displacement of the continuous deck, with which it is connected, so that it remains in the elastic range. The foundation of this abutment is shallow. The stiffness of the abutment has to be reduced to effective values (i.e. K_{eff}) due to its functional cracking. The dimensions of the web of the abutment provide adequate buckling resistance.

Figure 1(b) on the other hand, presents the second configuration of the abutment (indicated as 2nd abutment type). In this structural solution the web of the abutment is, as in the first case, monolithically connected to the deck of the bridge (Part I). The second part of the hybrid abutment, which has common foundation with the web, is located between the embankment and the monolithical web and is supporting the backfill soil. The width of the expansion joint, which separates the two parts of the abutment, is determined, in the proposed technique, on the basis of the in-service requirements of the deck. The segment of the abutment, which supports the approach fill (Part II), has the shape of an open box as its shallow foundation, the wing-walls and the diaphragmatic beam, which connects the wing-walls' heads, form a volume in which the approach embankment enters. The role of the diaphragmatic beam is to transmit safely the pounding forces, which result from the pounding of the deck towards the abutment, to the wing-walls and to provide the integrity of the vertical segment, which supports the soil in the longitudinal direction. Within this context, it is feasible to define the failure hierarchy involving the stability of the abutment and the strength of the wing-walls, in order to ensure the abutment's integrity during the design earthquake. A value of the overstrength ratio of a_{CD}=1,40 is considered acceptable. It is also noticed that capacity design was also applied to the wing-walls in order to ensure that bending failure antecedes the shear failure.

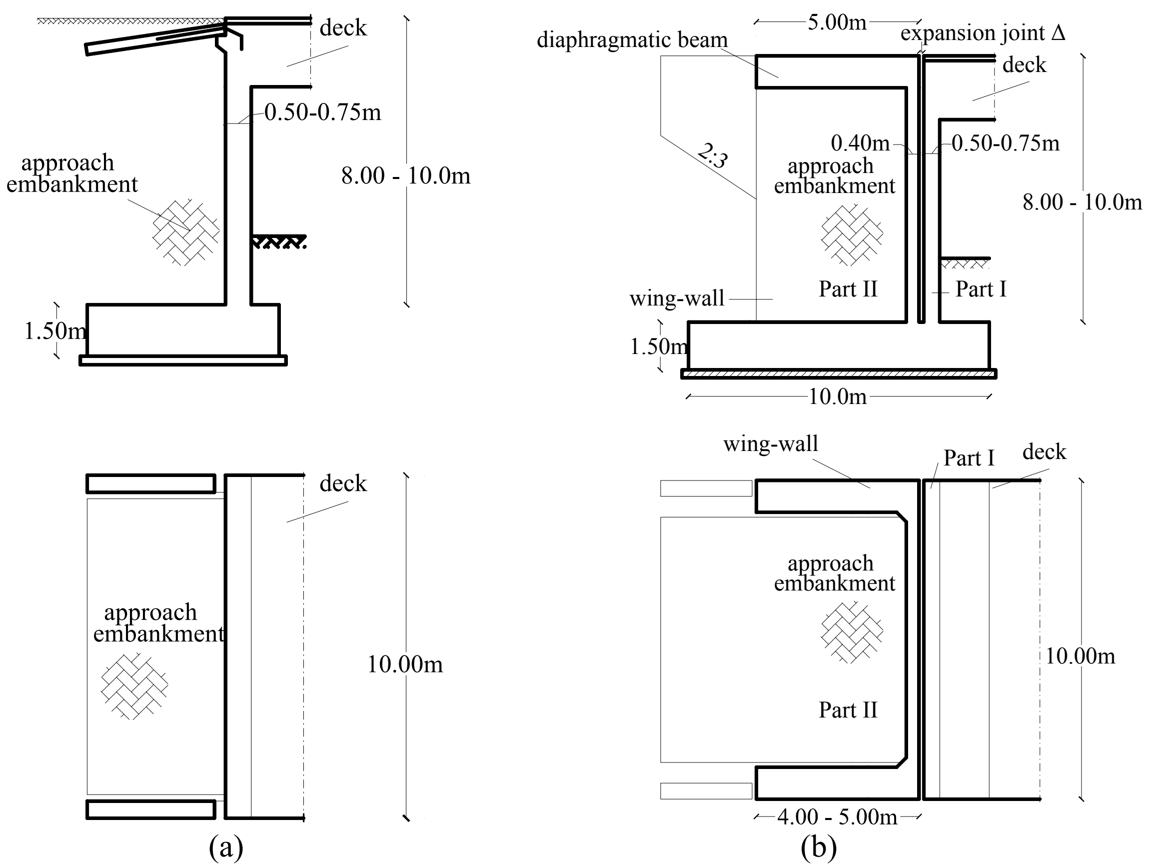

Figure 1: The proposed abutments:
(a) 1st configuration: Abutment monolithically connected to the deck while in contact with the approach embankment
(b) 2nd configuration: Hybrid abutment. Part I: Monolithical connection to the deck of the bridge. Part II: Supporting the backfill soil.

The aforementioned proposed abutment configurations have a number of explicit advantages compared to the currently implemented abutment design solutions. In particular, (a) the static and dynamic system of the bridge are more explicit than in the case where short semi-integral abutments with flexible piles are used,(b) the serviceability problems of the bridge are not bypassed by reducing the dimensions of the abutment but they are taken into account by checking the cracking of the abutments web and the compression of the deck, (c) the height of the proposed abutment develops, to the highest possible degree, the stiffness and the damping of the approach embankment. This stiffness and damping, which are activated during earthquake, as it is sholwn later, contribute to the reduction of the movement of the deck and as a result, to the reduction of the seismic inertial forces of the piers, whose head follows the movement of the deck [5, 6].

In the second solution, illustrated in Figure 1(b), the following advantages apply: (a) the part of the abutment, which is monolithically connected with the superstructure of the bridge (i.e. Part I) is separated of the backfill soil and, as a result, the factor of uncertainty, i.e. the response and the dynamic resistance of the embankment does not affect the seismic response of the bridge, (b) the passive earthpressure of the approach embankment towards the abutment is received from the second part of the hybrid abutment (Part II) which is separated from the deck of the bridge by a joint. Consequently the ratcheting effect described above, is eliminated as the supporting part of the abutment does not undergo annual displacements due to deck's displacements, (c) during an earthquake, when the movement of the superstructure causes the pounding of the deck towards the abutment, the second part of the abutment (Part II), which supports the backfill soil, is activated restraining in a way the free longitudinal movement of the deck. This periodic passive resistance of the abutment – embankment system increases the stiffness and the damping of the bridge, which results in the reduction of the seismic actions of the piers.

The two suggested configurations of the abutment reduce, as shown in the present investigation, the seismic distress of the piers. However, the monolithical connection of the abutment or a part of the hybrid abutment to the deck causes problems which concern the functional distress of the web of the abutment as well as the compression of the deck. In one of the following paragraphs this problem is presented and the check for the compression of the deck is given briefly.

Another parameter of the problem concerns the modified seismic energy induced in the aforementioned bridge systems through the abutments, a fact of particular importance in cases that the soil conditions change rapidly along the length of the bridge and the approach embankment is significantly modifying the motion [4].

Apart from the aforementioned aspectes related to the effect of the modified abutment-deck connectivity and rigidity to a number of issues related to the structural static performance and serviceability, the resulting modified stiffness and consequently, the shifted dynamic characteristics of the structure have a strong effect on the overall dynamic response of the structure. In the transverse direction the longitudinal movement of the deck is significantly restrained by the abutment (1st abutment type). On the other hand, in the case of the hybrid abutment (2nd abutment type), the contact of the deck with the part of the abutment which supports the approach embankment, creates a normal force which distresses the contact area. As a result, a transverse friction component exists as the product of this force and the coefficient of friction of the contact area. This friction is expected to modify the transverse response, that is, the period and the mode shape of the bridge, especially in the case of bridges of small length.

EVALUATION OF THE ALTERNATIVE BRIDGE SYSTEMS ON THE BASIS OF SERVICEABILITY REQUIREMENTS

The functional movements of the integral bridges under study, that is, the annual contraction and expansion, creep and shrinkage are increased with the length of the bridge. In the present investigation, the maximum length of the bridge, whose deck is continuous, is 200m. As mentioned earlier, these functional movements are maximized at the ends of the continuous deck. The contraction does not cause problems to the deck of the superstructure due to the low actions which are transmitted from the cracked webs of the abutment. However, expansion constitutes a critical check for the deck as the superstructure is enforced to high compression.

For the first solution described herein, the serviceability problem, caused by the expansion of the deck, is not critical given that the stiffness of the web is reduced due to functional cracking. For the initial three operational years of the bridge the approach embankment also resists to the expansion of the deck, while the passive earthpressure is not totally developed given that the ratcheting effect develops in the long run. After the first years of the operation of the bridge, creep and shrinkage relieve the compression of the deck as the aforementioned effects cause permanent contraction to the superstructure.

For the second abutment type, the serviceability problem, that is, the compression of the deck, is critical when the determination of the width of the expansion joint Δ, which is installed between the two parts of the hybrid abutment, does not take into account the expansion of the deck. In this case, and during the first years of the operation of the bridge, it is possible that the expanded deck of the bridge gets in contact to the part of the abutment which supports the approach embankment (i.e. Part II) which is very stiff due to the resistance of the wing-walls. The compression of the deck is increased due to the passive resistance of the approach fill, which, even though it hasn't developed the maximal earthpressure, resists to the tendency of the abutment to move towards its summer position. By checking the functional compression of the deck it was concluded that for a bridge of 200m with an expansion joint which is designed by taking into account the half (50%) of the expansion of the deck, the compressive force to the deck, is of the order of F_c=6500KN. It is noticed that a low value of the passive earthpressure coefficient K_p equal to 3 was used for the check of the functional problem. The force F_c compresses the upper part of the cross section of the superstructure and mainly influences the two terminal spans of the bridge, given that the compression zone is the upper bound of the spans. However, the serviceability problem is mitigated on account of the acceptable by the seismic codes, redistribution of the moments at the supports of the bridge as well as of other factors, that will be addressed in the following.

EVALUATION OF THE ALTERNATIVE BRIDGE SYSTEMS ON THE BASIS OF SEISMIC PERFORMANCE - A PARAMETRICAL STUDY

In the present paper the earthquake resistant efficiency of the two proposed abutment types was investigated through two different analysis schemes; (a) a general set of qualitative and relative parametric analyses aiming at identifying the trend observed in response when the abutment is utilised and (b) a more refined and realistic case that is implemented in order to obtain a quantitative insight on the actual anticipated bridge response related to the activation of the abutment.

Analysis Case 1: Qualitative identification of the response trend when the abutments are activated

At first, it was deemed necessary to focus on the effect of connecting the deck to the abutment for the case of four different bridges of different length, founded on Soil Category A, B and D according to the Eurocode 8 and subjected to the corresponding artificial earthquake motion that is compatible to the corresponding soil-dependent Eurocode 8 elastic spectrum. Peak ground acceleration was adopted equal to 0.24g while the artificial records were generated with the computer code ASING [8].

The length of the bridge mainly influences, through its mass *m*, the magnitude of the forces that arise from the interaction between the deck and the abutment as well as the value of the seismic actions of the piers. The length of the bridge also determines the functional distress of the bridge together with the initial width of the expansion joint at the beginning of the earthquake, which is of interest in the second type of the hybrid abutment. In order to determine the applicability range of the abutments proposed, four cases of bridges of varying total length were considered:
a) 35m, which corresponds to a bridge with one span,
b) 50m, which corresponds to, a bridge with two spans,
c) 100m, which corresponds to a bridge with three spans and
d) 200m, which corresponds to a bridge with six spans,.
The mass for the quasi-permanent loads was determined to be 20tn/m of length. For these cases the reduction of the inertial forces was investigated.

The structural configuration of these four reference bridge models in terms of pier height and cross section dimensions is summarized in Table 1. For all these four reference bridges, that were assumed to correspond to the conventional structure (i.e. where the deck was freely sliding in the longitudinal direction), two additional models were developed for the cases of the aforementioned two distinct abutment configurations. Consequently 4x3=12 bridge configurations were studied. The resulting three different configurations (including the reference conventional case) of deck-abutment connection are shown in Figure 2 for the longer (i.e. 200m) bridge.

End joints were assumed only for the case of the 2nd proposed abutment, where a joint is separating the two parts of the hybrid abutment. The abutments were considered to response in a viscoelastic manner which gives an upper bound of the participation of the abutment during the seismic event. The dynamic stiffness matrix of the abutment as well as the pier foundation spring-dashpot system properties were calculated approximately based on the simplified analytical expressions because the quantitative and relative character of the approach followed herein did not permit the 'accurate' derivation of such case-dependent properties. In other words, the target of the particular analysis case was to evaluate how a structure of different length supported on different soil and excited by different earthquakes is affected by the implementation of the proposed two abutment configurations and not the effect of a number of uncertain parameters on the dynamic response of the aforementioned systems. Such a refined calculation was performed in the following analysis case where the geometrical, material and seismic related data were made available.

In order to evaluate the effect of the abutment-deck connection, non-linear dynamic time history analysis is implemented with the FE commerical code SAP 2000 [9]. The non-linearity is purely geometrical and is related to the activation of the deck-abutment gaps. The interest of this analysis was to assess the degree of potential reduction of the integral bridge's deck displacements in the longitudinal direction, which are proporsional to the developed bending moments at the piers, when different abutment-deck configurations are adopted. This mitigation of the induced seismic energy was expressed by the ratio of the deck displacements of the bridge with the two distinct abutment types over the displacement of the conventional reference bridge of the same length that was supported on the same soil.

The variation of this effectiveness in terms of percentage reduction with bridge overall length is presented in Figure 3. It is observed that the displacement mitigation, and hence the piers' moment reduction, is higher for relatively shorter bridges for all soils and all abutment types. It is also noticed that the shorter the bridge, the more effective are the abutments involved. In particular, the reduction for the case of the 35m bridge is almost 75-85%, while it is approximately 40% for the longer structures of 200m. The observed decreased effectiveness of the abutment in the reduction of the seismic actions for longer bridges can be attributed to the fact that, as already explained, the suggested investigation determines the width of the end joints according to functional and not seismic requirements. As a result, the width of the expansion joints is higher in longer bridges and hence, the activation of the abutment is not immediate nor continuous during the during the earthquake.

Figure 3 also shows that the proposed abutment solution is more effective in softer ground types, especially for the case of shorter (and stiffer) bridges, where the relative flexibility has a more important role during the seismic event. In fact, a stiff bridge with fundamental period within the constant spectral acceleration branch of EC8 (which is wider when for soft soil types) has the advantage that it becomes significantly stiffer with the activation of the abutments without attracting additional seismic loads. On the contrary, a longer (and hence more flexible) bridge on stiff soils, has a larger probability to lie along the descending branch of the spectrum; consequently, the displacement reduction is counteracted by the increase of seismic forces that results by the fact the fundamental period is shifted leftwards. In any case, it has to be noted that this stiffness and dynamic load interplay has not altered the overall observation that the effect of the abutment activation was generally beneficial. Nevertheless, it is inevitably dependent on the shape of the spectrum used – as all soil-structure interaction problems essentially are. As a result, the trend shown is a possitive indication that the employment of the abutment stiffness may have beneficial effects on the displacements and forces developed in the case of integral bridges. Nevertheless, since the problem is complex, multi-parametric and frequency dependent, the above qualitative approach has to be validated through a more refined parametric study. This study is presented in the following analysis case.

<u>Analysis Case 2: Quantitative identification of the applicability of the proposed abutment configurations</u>

Having obtained the relative sensitivity of different bridge structures to the deck-abutment connection configuration, is was deemed necessary to repeat the analysis for a selected particular case, where this time an effort was made to calculate all the critical parameters of the problem with the highest possible degree of accuracy. The case selected was the 200m bridge (Figure 4) located on Soil Category B (an assumption of a stiffere formation A at the position of the abutments was made). The structure was initially excited with the earthquake record of Kozani (13/05/1995, Mw=6.5, PHA=0.20g) for the assessment of the bridge as obtained by the Internet-Site for European Strong-Motion Data [10]. Additionally, the earthquake records of two other major events in Greece (i.e. Athens, PHA=0.26g, 1999 and Kalamata, 0.23g, 1986) were also used.

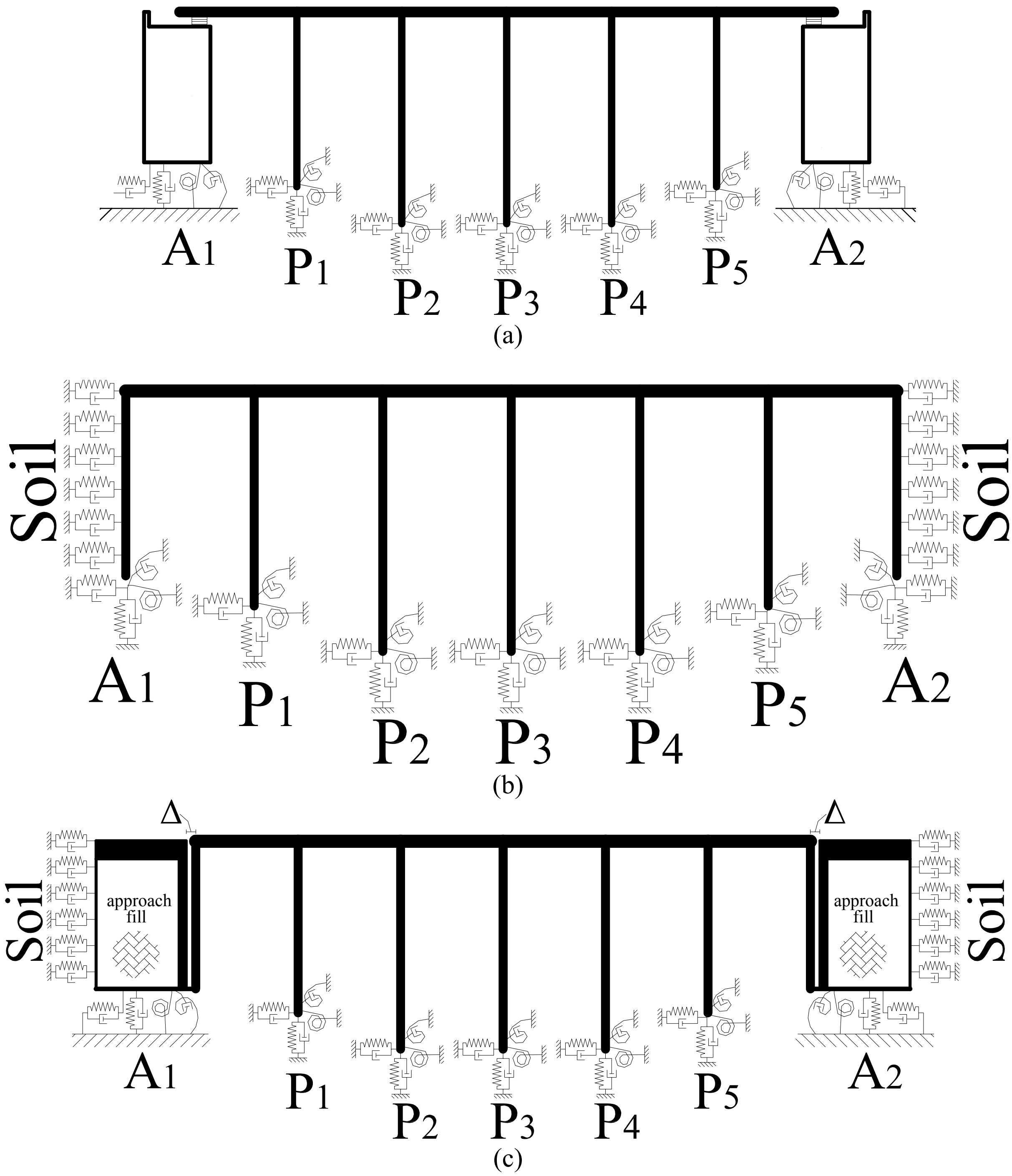

Figure 2: Alternative abutment configurations for the 200m bridge.

Table-1: Structural configuration of the three reference bridge models of 50, 100 and 200m.

Height of the piers (m)	Length of the bridge (m) - Spans				
		35m	50m	100m	200m
	Piers	1 span	2 spans	3 spans	4 spans
	A1	6.5	6.5	6.5	6.5
	P1	-	6.5	10	10
	P2	-	-	10	15
	P3	-	-	-	15
	P4	-	-	-	15
	P5	-	-	-	10
	A2	6.5	6.5	6.5	6.5

Cross Sections (m)			Length of the bridge (m)		
	Piers	35m	50m	100m	200m
		1 span	2 spans	3 spans	4 spans
	A1	Fig.1	Fig.1	Fig.1	Fig.1
	P1	-	1Ø1.00	2Ø1.20	2Ø1.20
	P2	-	-	2Ø1.20	2Ø1.20
	P3	-	-	-	2Ø1.20
	P4	-	-	-	2Ø1.20
	P5	-	-	-	2Ø1.20
	A2	Fig.1	Fig.1	Fig.1	Fig.1

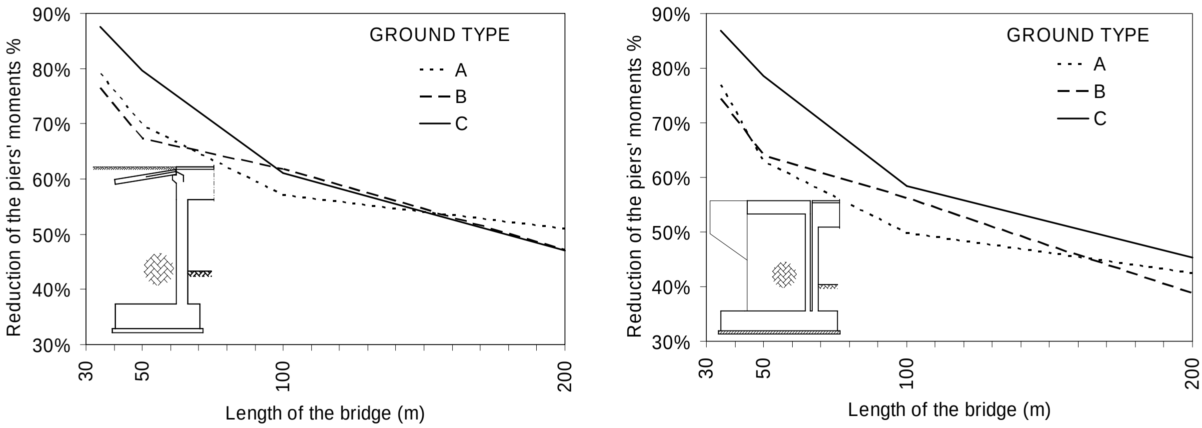

Figure 3: Variation of the percentage reduction of the displacement with bridge overall length for the two proposed abutment configurations.

The crucial issue of the abutment static and dynamic stiffness was dealt using available seismic code provisions as well as research results from the literature. In particular, the stiffness was derived according to CALTRANS [11] seismic code. As a result, for the linear elastic model an effective abutment stiffness, K_{eff} was adopted accounting for expansion gaps. Since the abutment backwall stiffness is nonlinear and is dependent upon on the material properties of the abutment backfill as well as on the seismically induced porewater pressures, the initial backwall stiffness was taken equal to K_i = 11.5kN/mm /m, a value also based on passive earth pressure tests at UC Davis,. The initial stiffness was therefore adjusted proportional to the backwall/diaphragm height:

$$\mathrm{K}_{abut} = K_i \cdot w \cdot \left(\frac{h}{1.7}\right) \tag{1}$$

where, w is the width of the backwall. Consequently, K_i = 11.5x0.5x6.0/1.7 = 20.3 kN/mm/m = 203 MN/m for each abutment (=33.8 MN/m for each spring).

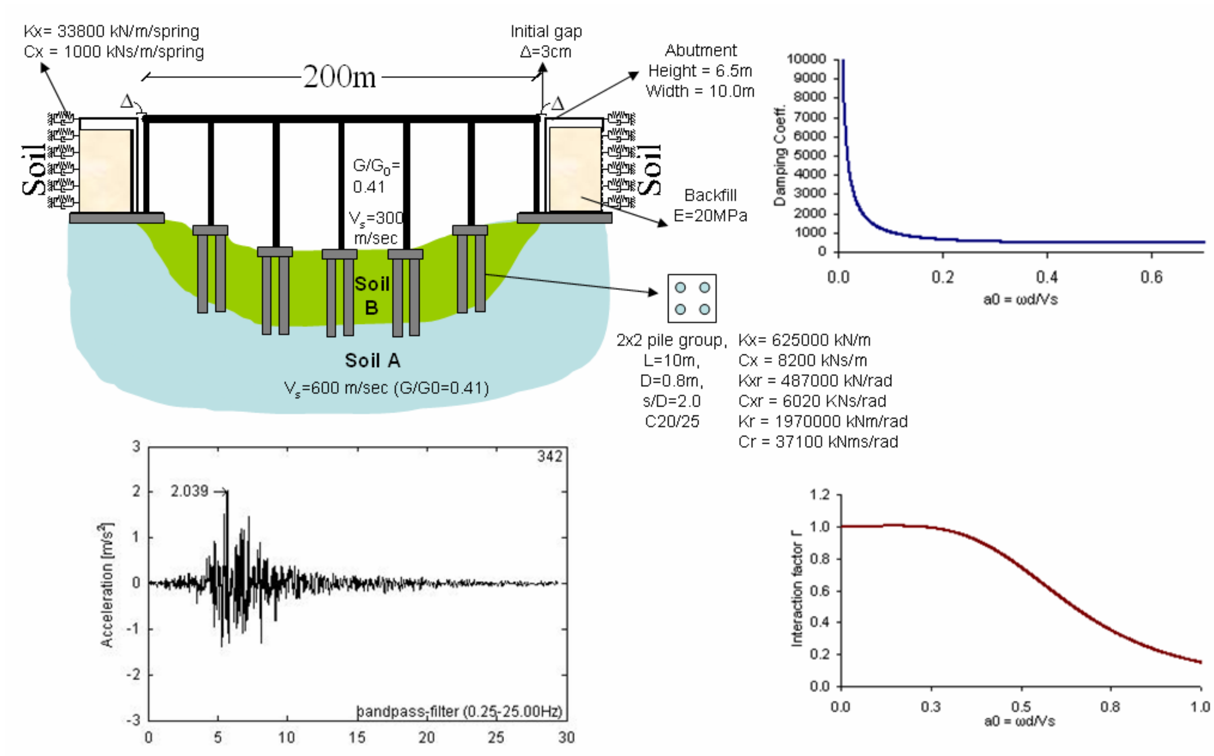

Figure 4: Refined analysis case study for the identifiaction of the range of applicability of the proposed abutment configurations. Kozani earthquake record (left) and single pile damping and kinematic interaction factor (right)

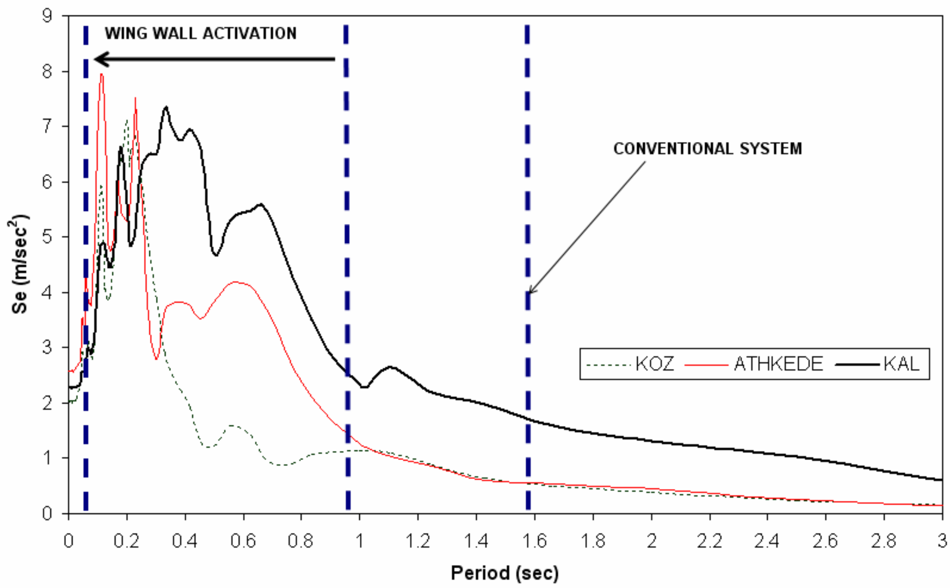

Figure 5: Response spectra of the earthquake excitations used compared to the dynamic characteristics of the structure

It is notable that the aforementioned CALTRANS provisions are based on the Wilson and Tan model [5]. Moreover, they yield similar stiffness with the refined approach proposed by Zhang and Makris [3,4]. The latter was used for the evaluation of the embankment-abutment-structure damping properties. The definition of damping is a critical issue since it is expected to be high in such cases, as it is also recognised in the aforemention seismic code [11]. A review of the above stiffness and damping values is also presented elsewhere [12, 13].

The dynamic stiffness matrix of the pile groups that were assumed for the foundation of the piers were derived using the computer code ASING [8]. The required static stiffness matrix was first derived, on the basis of the relevant flexibility coefficients for coupled horizontal and rocking modes of vibration which can be calculated through closed form equations, among the many available in the literature [14]. The corresponding single pile kinematic interaction factor and frequency dependent damping are presented in Figure 4. Then, the complex dynamic interaction factors were calculated for all modes of vibration, incorporating the most widely used expressions in the literature [8, 14].

It is important to note that although both kinematic and inertial soil-pile and pile-to-pile interaction are strongly frequency-dependent, it is assumed that the complex dynamic impedance matrix is calculated based on the predominant frequency of the input motion, an assumption that is common in the literature [3, 4]. Potential amplification of the crest motions of the approach embankments which may have an appreciable effect on the bridge response under certain circumstances was also ignored because is was considered out of the scope of the particular relative effectiveness comparison. Nevertheless, it is an important factor that may have a significant effect, especially when due to topography and stratigraphy the embankments essentially 'drive the motion' [3, 4]. The issues of spatial variation of ground motion were also not implemented in this study because the overall bridge length was relatively small. It has to be noted though that, in general, integral structures are more sensitive to asynchronous motion, primarily due to the development of relatively higher pseudo-static forces. However, comparative analysis of 20 different bridge configurations [15] inclusive of the cases of monolithic abutment-deck connection compared to the bearing-type connection, has revealed that in terms of *overall* dynamic response, an integral bridge of 200m is not necessarily more sensitive to the spatial variation of ground motion with the exception of the extreme case that the soil properties change rapidly and the local site effects are dominant.

An equivalently reduced value of V_s was also assumed for soil B according to the following EC8-based relationship [8]:

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

which relates the soil stiffness V_s/V_{smax} and damping ζ *(%)* values, to the selected peak ground acceleration level a (where a<0.3g). As a result, the values of V_s/V_{smax} = 0.63 and G/G_0 = 0.41 were used.

The response of the bridge studied in terms of deck displacements for the case of a deck free to move in the longitudinal direction and the implementation of the 2nd abutment solution, for the three earthquakes used is shown in Figure 6. It is observed that for the three excitation cases the displacements of the modified system (i.e. corresponding to the activation of the abutment) are indeed mitigated compared to the reference, conventional case. Nevertheless, for the case of the Kozani earthquake the effect of the proposed solution is not significant, primarily because the earthquake load at the period range that corresponds to the activation of the abutment is lower, as shown in Figure 5 where the comparative response spectra are presented. As a result, the overall displacement mitigation trends observed in the case of the qualitative identification analysis appear to be valid as well for the more detailed and refined quantitative case study. In any case though, the complexity and frequency dependence of the problem is such that extensive parametric analyses are required, inclusive of the effect of the abutment configurations on the response in the transverse direction, before concluding on the wide applicability of the deck-abutment solutions proposed.

CONCLUSIONS

The present paper, which aims at the effective reduction of the inertial seismic actions of the bridge through the restrain of the free longitudinal movement of the deck, investigates the earthquake resistant and mechanical behaviour of the two innovative and rigid abutments; the first is monolithically connected to the deck while in contact with the approach embankment and the second is a hybrid abutment, which partially connected to the deck with wing walls restraining the free longitudinal vibration of the deck. The earthquake resistant and mechanical behaviour of the suggested abutments was studied and the following conclusions were drawn:

1) The response of the bridges, implementing the two suggested abutment types, is generally satisfying as far as the functional and the earthquake resistant requirements are concerned.

2) In comparison with the existing technique for the construction of abutments that are monolithically connected to the deck, (i.e. involving flexible piles and hollow wall - pile cap), the suggested abutment system appreas superior on the basis of earthquake resistance, functional response and mechanical behaviour.

3) It is shown by the parametric investigation that the abutments are capable of reducing the seismic actions, and hence distress the bridge along the longitudinal direction, by up to 75% for the small bridge of 35m length. This efficiency is decreased for longer bridges (i.e. 200m), to 40%.

4) The suggested abutments are more efficient for bridges subjected to larger displacements when the abutment is relatively more active. The proposed configuration is also relatively more effective for short bridges founded on soft soil.

5) The validity of the present study is dependent on the resistance of the backfill and subsequently the accurate determination of its stiffness and damping is of paramount importance.

6) Further investigation, concerning real bridge systems, is necessary in order to study the complex interaction between the main parameters of the problem.

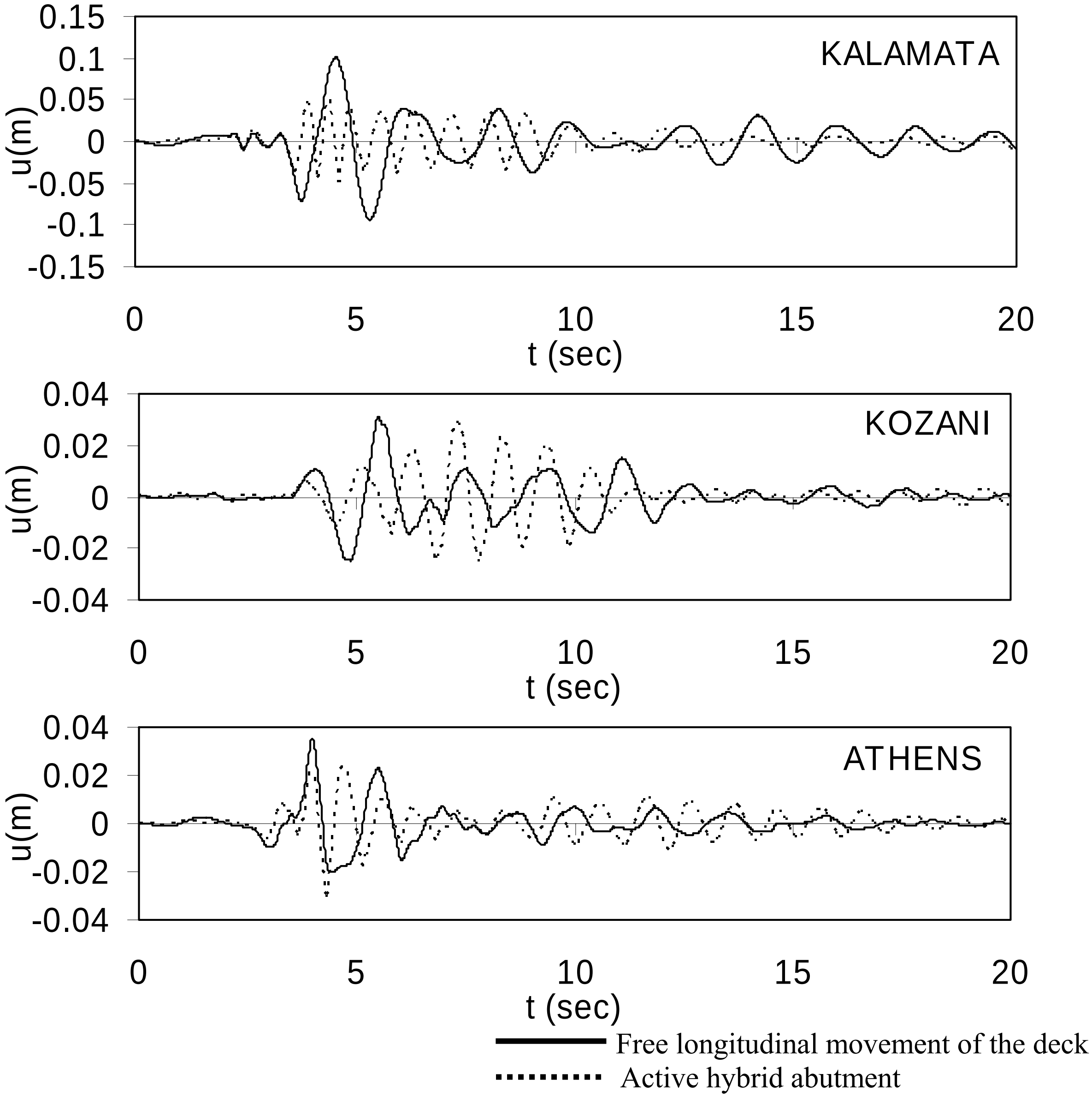

Figure 6: Response of the bridge studied in terms of deck displacements for the case of a deck free to move in the longitudinal direction and the implementation of the 2nd abutment configuration, for the three earthquakes used.

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