

A PROPOSAL FOR AN ALTERNATIVE TO SEISMIC ISOLATION PRACTICE: IMPLEMENTATION IN A SEISMIC ISOLATED RAILWAY BRIDGE

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Abstract. *The present study proposes an alternative to seismic isolation practice, which aims at the reduction in the bridge seismic actions mainly for the longitudinal design earthquake, which is more critical than the transverse one. The aforementioned purpose is achieved through the improved seismic participation of the bridge abutments. The deck slab is extended onto the embankments and is rigidly connected either with transversely directed RC walls or with transversely directed rows of hollow section steel tubes. This system is constructed into a concrete box-shaped substructure which replaces the conventional wing-walls. As a result the approach embankment does not affect the response of the proposed system. The efficiency of the proposed system was assessed by utilizing a conventional seismic isolated railway bridge. The analytical investigation was performed by means of non-linear dynamic time history analysis implemented with the FE commercial code SAP 2000. The bridge systems were subjected to artificial earthquake motion that is compatible to Eurocode 8 elastic spectra. The assessment of the efficiency was mainly carried out by comparing the response of the resulting modified bridge system with the response of the initial bridge system. The proposed system has the ability to accommodate the in-service movements of the deck, by means of the flexibility of the restraining members. The seismic response of the bridge is also enhanced. The displacements of the deck in the longitudinal direction of the bridge were found to be significantly reduced. The restraining effect also reflects on the seismic actions of the piers (shear actions and bending moments) leading to cost-effective bridge design.*

1 INTRODUCTION

Many countries around the world suffer from earthquakes and people expect that structural engineers will design structures so that they can survive the effects of these earthquakes. The basic requirement that the engineer work to meet during the design process is the ensuring that the capacity of the structure is greater than the demand [1,2]. The earthquake causes inertial forces proportional to the structure's mass and the earthquake ground accelerations. As the ground accelerations increase, the capacity of the structure must be increased to avoid structural damage. However it is not practical to continue to increase the strength of the structure indefinitely as this also increase the structural cost. So the Codes [3] allow engineers to use ductility to achieve the capacity. Ductility is a concept of allowing the structural elements to deform beyond the elastic limit in a controlled manner. Beyond this limit, the displacements increase with only a small increase in force.

Seismic isolation represents an opposite, to the aforementioned, approach, as it attempts to reduce the demand rather than increase the capacity of a structure [4-6]. In this framework, the effort on protection of bridges against earthquakes is mainly focused on minimizing the forces to be carried by the piers, in particular the shears. Isolation systems are basically typified into bearings and energy dissipation devices. Rubber bearings with high lateral flexibility are meant to shift the vibrational periods of the structures so as to avoid resonance with the excitations [7]. They are usually combined with high damping material and fluid dampers to increase the in-structure damping and to reduce the lateral movements of the bridge due to the seismic loading. Sliding bearings are introduced to filter out the imparting earthquake forces through the frictional interfaces. The support of the deck to the piers and abutments through bearings also accommodates the in-service induced movements of the deck due to shrinkage creep and thermal effects [8]. However, although seismic isolation is considered one of the most promising practices worldwide, the high purchase cost of the devices, which increases the total construction cost of the bridge constitutes a major disadvantage of this practice.

In this regard, concerning the reduction on the demand rather than the increase on the capacity of a structure, the seismic performance of bridges can also be enhanced by the abutment and backfill soil participation. Current research [9-11] focuses on the known problem of embankment-bridge interaction which can contribute to the enhancement of the seismic response of bridge structures. Specifically, recent studies have investigated the problem and came up to the conclusion that the dynamic participation of these elements results in significant displacement reductions [12]. Furthermore the seismic participation of the abutment was found to lead to cost-effective bridge design according to Nutt [13].

In the present study the seismic performance of bridges is enhanced through an external restraining system consisting of steel tubes which are arranged in rows behind the abutment's web. The efficiency of the proposed restraining system is examined by analyzing the seismic response of a railway bridge in Greece which is considered to be the "reference" bridge.

2 DESCRIPTION OF THE PROPOSED SYSTEM

The aim of the present study is the investigation on the efficiency of an external restrainer which has the ability to reduce the bridge seismic movements. The proposed restrainer is located behind the abutment's web and is consisted of transversely directed members which are either pairs of concrete walls or rows of pairs of steel tubes. The efficiency of the system consisting of pairs of concrete walls was extensively investigated in previous works [14]. In this study the investigation is extended in the field of the composite steel and concrete structures. The proposed system is given in Figure 1. Each pair of steel tubes is encased in the pile- cap

while their heads are connected in the longitudinal and transverse direction through steel plates which are welded around their perimeter, see Detail 1 in Figure 1. The head of the pairs of the steel tubes is properly connected with the extension of the deck slab. As a result the in-service as well as the seismic loading causes the activation of the system. This system is constructed into a concrete box-shaped substructure which replaces the conventional wing-walls. Therefore the approach embankment does not affect the response of the proposed system.

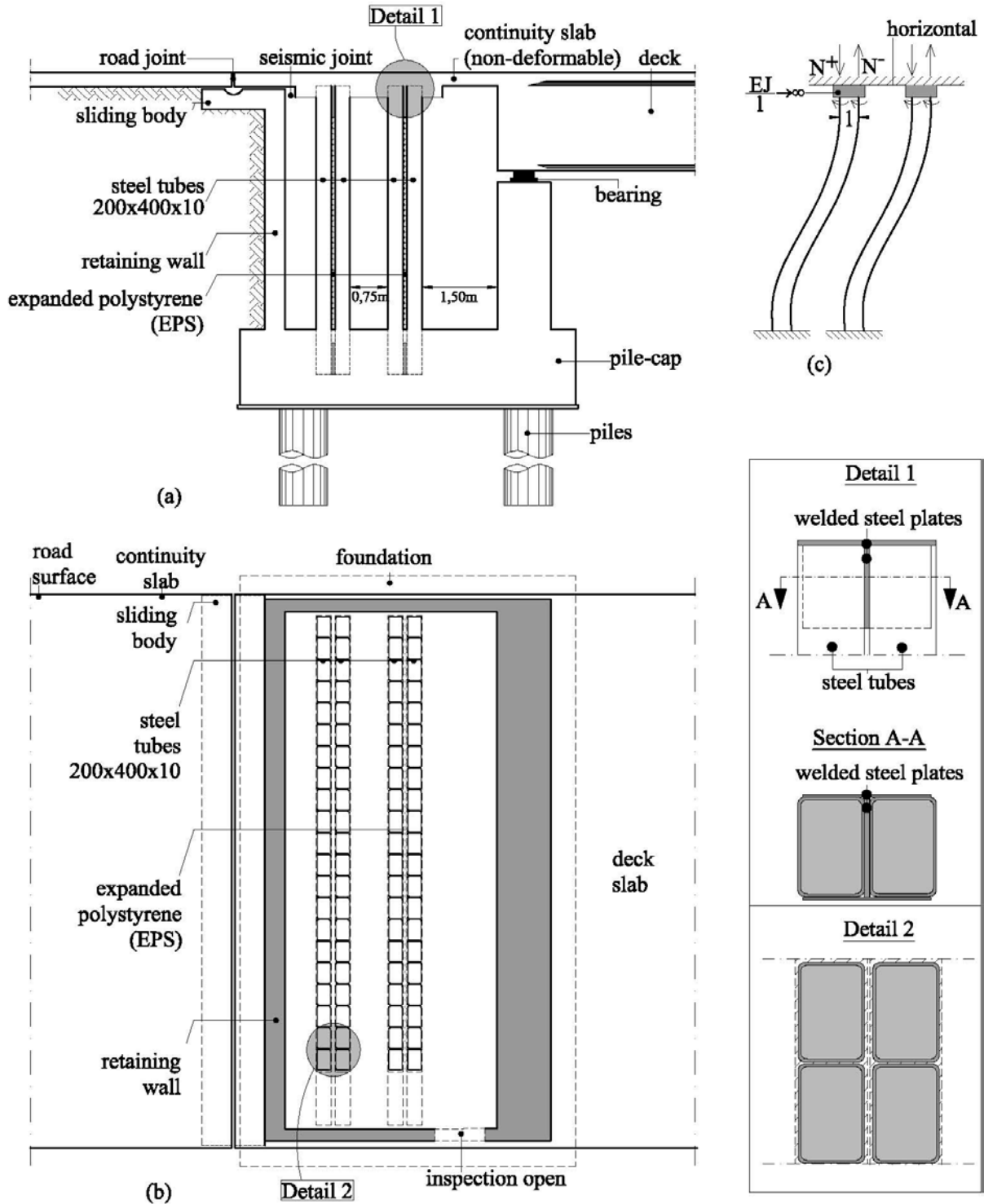


Figure 1:(a) Longitudinal section of the proposed restraining system, (b) Plan view of the proposed restraining system and (c) Restraining members' model.

Any relevant movement between the tube's head and base causes (a) linear variant bending moments along the height of the tube (the zero point is at the middle of the height), (b) constant shear forces along the height of the tube which resist to the movement of the extension of the deck's slab, (c) tension and compression axial forces corresponding to the direction of the head's movement (every pair has one tensile and one compressive tube), (d) second-order effects due to the aforementioned axial loading. The additional bending moment due to these effects increases the initial bending moment at the base of the tensile member and decreases the bending moment at the base of the compressive one. It is also notable that the second-order effects affect the performance of the tube's pair and induce the synchronous yielding at the critical region of the tubes' bases. Otherwise the absence of the second-order effects causes the yielding at the base of the member subjected to tensile axial loading before the corresponding one at the compressive member. Although the aforementioned assumption cannot reasonably be made in the case of the members' heads, the inversion of the loading direction due to the cycling event leads to the acceptance of the asynchronous yielding at the members' heads. The aforementioned analysis regarding the behavior of the proposed restraining system rationalize the consideration of the restraining tubes as fixed members in the computational framework.

The steel tubes used in this system can either be rectangular or square cross-section [15]. The choice of the tube's cross-section determines the number of the steel tubes which can be used as the distance between the wing-walls is given. The response of the proposed system is strongly dependant on the dimensions of the aforementioned members which also affect the constructability of the system. The use of steel tubes with big dimensions complicates the constructability but increases the efficiency of the system. In this study has been chosen rectangular steel tubes whose dimensions are 200x400mm. Each row consists of 25 steel tubes whose weak axis is oriented parallel to the longitudinal direction of the bridge.

The proposed restraining system is rationally combined with sliding bearings which permit the free movement of the deck in the longitudinal direction of the bridge. The support of the deck to the piers through rubber bearings in bridge systems which develop the aforementioned system is in slight contrary to this proposal as on one hand the period of the system is reduced and on the other hand is attempted its increase through the flexibility of the system due to the presence of the bearings. However a solution with rubber bearings is quite interesting even for reasons concerning its economic efficiency and is presented in this study.

Finally, based on the aforementioned assumption concerning the static behavior of the restraining members, it is necessary to be mentioned an significant characteristic of the proposed system. The investigation on the efficiency of the system showed that two pairs of rows of steel tubes (one pair per abutment / 25 steel tubes per row) with a RHS cross-section 200x400x10mm and a height $H_t=4.50\text{m}$, have the same seismic resistance as five concrete piers monolithically connected to the deck. These piers have circular cross-section with a diameter $d_c=1.5\text{m}$ and a height $H_c=13.0\text{m}$. The aforementioned conclusion derives from the equation of the resistances of the above members. The seismic resistance of a steel tube is calculated according to Equation (1). Equation (2) gives the seismic resistance of a pier. The resistance of two pairs of rows of steel tubes is 214 071 kN/m and is about five times the resistance of a pier with the above properties:

$$R_t = \frac{12E_s \cdot I}{H_t^3} \quad (1)$$

$$R_c = \frac{12E_c \cdot \pi \cdot d_c^4}{64H_c^3} \quad (2)$$

Respectively, 20 piers with the aforementioned properties, which are connected to the deck through bearings and their longitudinal movements are restrained through seismic links have the same resistance as two pairs of rows of steel tubes.

3 OVERVIEW OF THE BRIDGE STUDIED

In order to investigate the efficiency of the proposed practice a seismic isolated railway bridge located at Northern Greece was chosen as the case study. This bridge is straight, has four-spans and a total length equal to 168.0m, Figure 2. The end spans have a length equal to 39.0m, while the two intermediate spans are 45.0m long. The prestressed deck, Figure 3(a), has a hollow T-beam-like section, is 13.60m wide and 3.60m high, is supported through LRB bearings placed on the two abutments and the three middle piers and is separated from the backwall through an expansion joint. Seismic forces are also resisted by the activation of stoppers (in the transverse direction) which are constructed at the seating of the abutments. Each abutment is equipped with two fluid dampers. The piers have a 3.00 x 5.50m hollow cross-section, 0.45m wall thickness and heights equal to 20.35m, 23.80m and 14.35m, Figure 3(b). The pier foundation consists of a 4x4 pile group of 15.0m long piles, connected with a 2.0x11.0x11.0m pile cap, while the abutments are supported on a 3x4 pile group of 12.0m long piles connected with a 2.0x7.5x11.0m pile cap. The axial spacing between all piles is 3.0m.

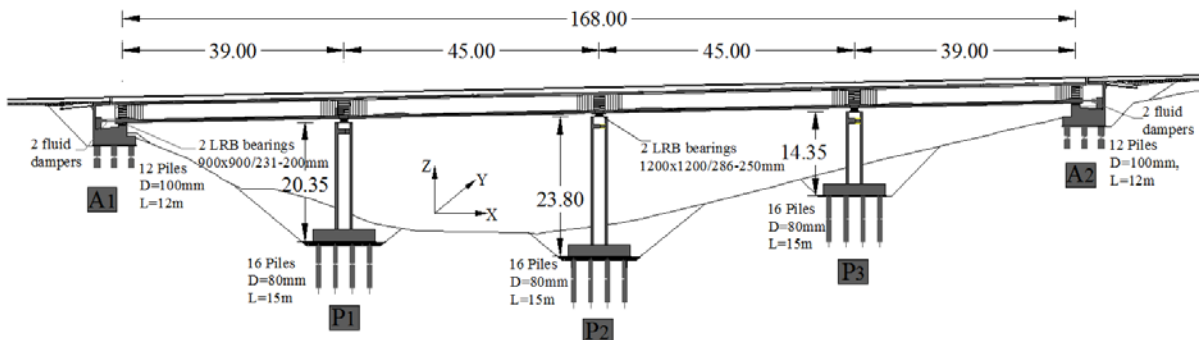


Figure 2: Longitudinal section of the seismic isolated bridge used for the purposes of the present study.

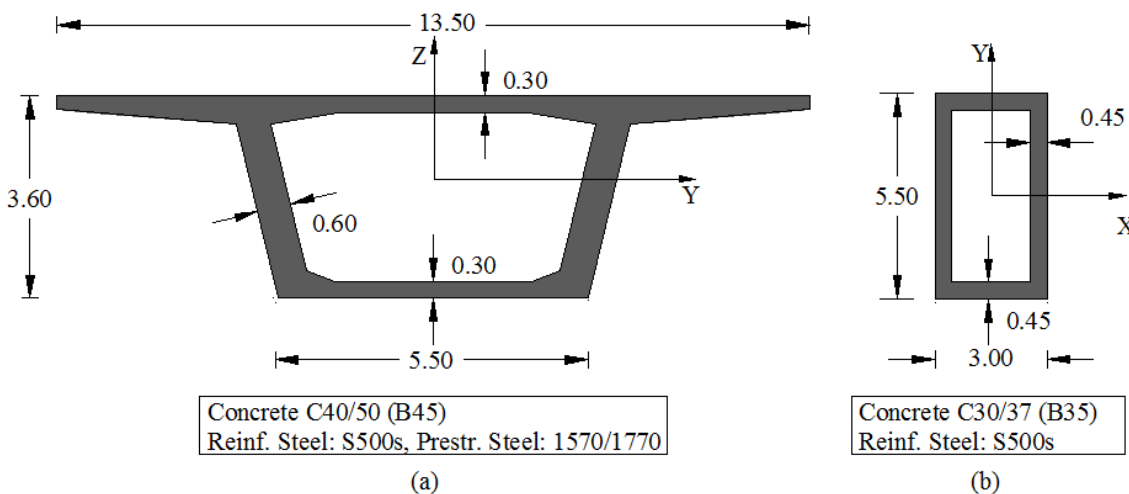


Figure 3: (a) The cross-section of the deck in the middle and (b) the cross-section of the pier.

The bridge was designed for normal loads according to the German Norms [16] while the seismic design was carried out according to the Greek Seismic Code [17] and the relevant Greek standards [18] for the seismic design of bridges. The bridge is founded on a ground type B according to the Greek Seismic Code [17]. The corner periods of the spectrum used are 0.15s and 0.60s for ground type B. The site of the bridge belongs to Seismic Zone II according to the Greek Seismic Code, which is characterized by a Peak Ground Acceleration of 0.24g. The behavior factors of the system adopted for design according to the relevant Greek standards were $q_x=q_y=q_z=1.0$ for the response in the three principal directions, respectively.

4 COMPUTATIONAL FRAMEWORK

The proposed restraining system is implemented in the bridge described in the previous section. The modified bridge doesn't have hydraulic dampers and the LRB bearings are replaced by low-damping rubber bearings whose dimensions are calculated according to the Codes provisions [7]. The steel tubes are rectangular hollow sections 200x400mm with a wall thickness 10mm. Four rows of 25 steel tubes are considered per abutment as shows Figure 1. These tubes have a height equal to 4.5m and their weak axis is oriented parallel to the longitudinal direction of the bridge.

The numerical simulations of the two bridge systems (initial and modified) studied in the present investigation were carried out with the FE- Code SAP 2000 [19]. The aforementioned bridge systems were subjected to corresponding artificial Earthquake motions that are compatible to Eurocode 8 elastic spectra [3] and two different peak ground accelerations $a_g=0.24g$, $a_g=0.36g$ were considered. In Figure 4 are given the response spectra of the artificial earthquake motions used in the present investigation. Dynamic non-linear time history analysis was implemented and the direct integration, known as β -Newmark method, was used [20] as this method is the most robust to be used for the step-by-step dynamic analysis. The mass and stiffness proportional damping was chosen and critical damping ratios equal to 5% were considered for the first and the second period of the analyzed bridge systems [21].

Figure 5(a) gives the stick model of the initial bridge system. The deck of the bridge was modeled by frame elements, which have the section properties of the deck, Figure 3(a). The piers were also modeled by frame elements. The flexibility of their foundations was taken into account by assigning six spring elements -three translational and three rotational- whose stiffness values are given in Figure 5(a). These spring values were obtained by the in-situ geotechnical tests conducted for the design of the as-built bridge given in Figure 2. The LRB bearings which have a non-linear force-displacement relationship in the two horizontal directions were modeled by rubber isolator link elements. Their stiffness values were calculated according to Naeim and Kelly model [5]. Damper elements which are available in SAP 2000 were used for the modeling of the four fluid dampers of the bridge. These elements have a non-linear force-velocity relationship and their damping properties are based on the Maxwell model of viscoelasticity [22].

Figure 5(b) gives the stick model of the modified bridge system. The modeling of this bridge system developed the modeling of the initial bridge. The deck and the piers have the same cross-sections in the two bridge systems. As mentioned above the modified bridge doesn't have hydraulic dampers and the LRB bearings were replaced by low-damping rubber bearings. These bearings were modeled by link elements and their stiffness values were calculated according to Naeim and Kelly model [5]. The restraining system consists of two parts: (a) the extension of the deck slab, namely the continuity slab, which was modeled by frame elements. This slab has a width equal to 13,50m and a thickness equal to 0,40m and (b) the group of the steel tubes. The tubes were modeled by frame elements, which have an RHS

cross-section 200x400x10mm. The steel tubes have a length equal to 4.5m and were considered to be fixed at the pile-cap of the foundation.

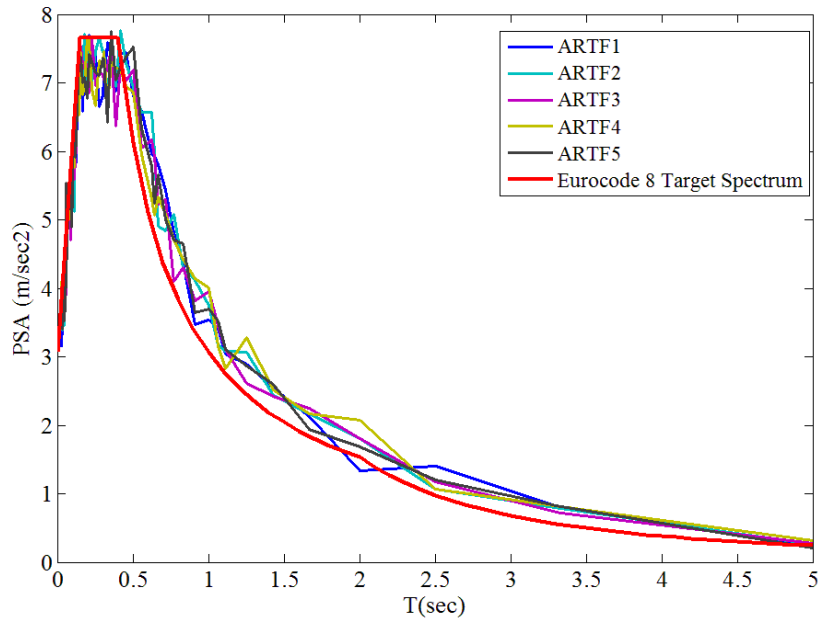


Figure 4: Response spectra of the artificial earthquake motions used in the present study and Eurocode 8 target spectrum (Soil Class B).

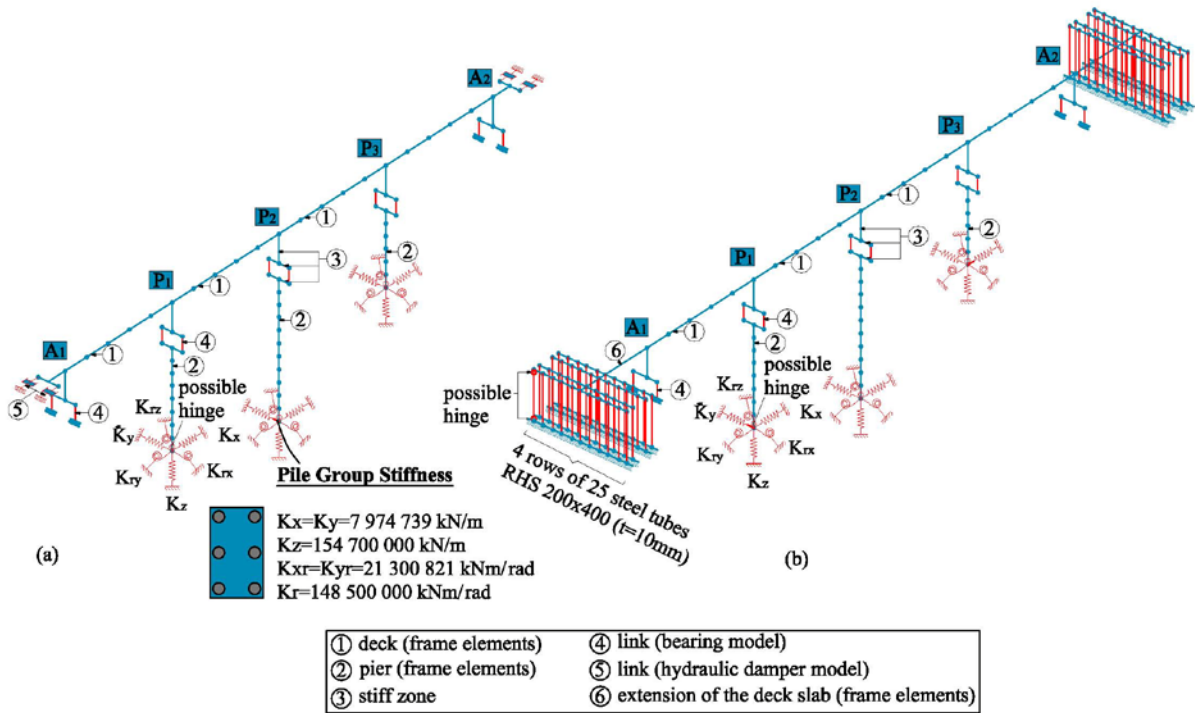


Figure 5: (a) Overview of the finite element models developed: (a) seismic isolated bridge system and (b) modified, according to the proposal of the present study, bridge system.

The possible plastic hinges at the piers' feet as well as at the restraining members' feet and heads were also modeled. The moment-curvature ($M-\phi$) curves were calculated by means of RCCOLA-90 [23]. The model involves beam elements with inelastic springs located at their

ends. Based on the members' geometry and the moment-curvature curves derived from the analysis of the cross-sections, the constants for the rotational springs are calculated. These springs are activated only whenever the developed bending moment exceeds the yield moment of the section and follows a non-linear force-rotation law that is a function of the cross-section properties.

5 EVALUATION ON THE EFFICIENCY OF THE PROPOSED SYSTEM

For the above two finite element models, non-linear dynamic analysis was performed using the 5 artificial earthquake motions. All analyses were conducted in the inelastic range and the excitation was performed in the longitudinal direction. As mentioned in the previous sections the proposed system is expected to dissipate part of the induced seismic energy through the flexural response of the restraining members. Figure 6(a) illustrates the hysteresis loop of a restraining member for a design ground acceleration $a_g=0.24g$. This figure shows that the restraining members respond in an inelastic manner. Consequently, the steel tubes are not only resisting with their stiffness, but also dissipate energy through hysteretic behavior. Figure 6(b) depicts the total input energy in the bridge system as well as the energy absorbed through the hysteretic behavior of the steel tubes for the same design ground acceleration. It can be deduced that a part of the input energy, about 35%, is absorbed through the hysteretic behavior of the restraining members.

The restrain of the bridge by the proposed system leads to an increase in the overall stiffness of the bridge. Specifically, the period of the first longitudinal mode shape is up to 73% reduced in the modified bridge system, in comparison to the period of the initial one. The reduction in the modal period of the modified bridge was found to lead in increases in its seismic loads, due to the fact that the modal periods are closer to the dominant periods of the elastic response spectrum. Despite this fact, the investigation showed that the movements of the deck of the modified bridge system are effectively reduced, in comparison to the corresponding ones of the initial bridge. Figure 7 shows the time histories of the longitudinal seismic displacements of the joint of the deck over the pier P_2 for the two bridge systems. From this figure it can be derived that the modified bridge system responds with smaller displacements in the longitudinal direction. The time histories also show that the overall resisting system of the modified bridge becomes stiffer. Both time histories given in Figure 7 correspond to an artificial accelerogram, which is compatible to Eurocode's 8 [3] elastic spectrum for a ground acceleration equal to 0.24g.

The efficiency of the proposed practice was mainly determined by calculating the percentage reductions in the longitudinal movements and seismic actions of the initial bridge system, Figure 5(a), compared with the corresponding ones of the modified bridge system, Figure 5(b). The ratio of this percentage reduction (P.R.) is given in Equation (3). In this equation $P.R.$ is the ratio of the percentage reduction of the movements of the deck or the seismic actions, $A_{,E1}$ is the seismic movement or the seismic action of the initial bridge system and $A_{,E2}$ is the seismic movement or the seismic action of the modified bridge system.

$$P.R. = \left[1 - \frac{A_{,E2}}{A_{,E1}} \right] \cdot 100 \quad (3)$$

In Figure 8 the percentage reductions in the longitudinal movements of the deck are illustrated for the two ground accelerations 0.24g and 0.36g. It is deduced that the proposed restraining system has the ability to reduce by up to 27% the longitudinal movements of the deck when the ground acceleration is 0.24g, while its efficiency is greater in higher ground

acceleration ($a_g=0.36g$) as the aforementioned reduction is 33%. This can be attributed to the energy dissipation through the hysteretic behavior of the restraining members.

In Figure 9 the percentage reduction in the bending moments M_{yy} at the base of the piers due to the longitudinal seismic action is illustrated for two different design ground accelerations 0.24g and 0.36g. The analysis showed that the M_{yy} moments were reduced up to 28% when the ground acceleration is 0.24g. The corresponding percentage reduction is greater when the bridge system is subjected to greater ground accelerations. Finally it seems that the restraining effect of the proposed restraining system is limited in the central pier of the bridge which is the tallest one.

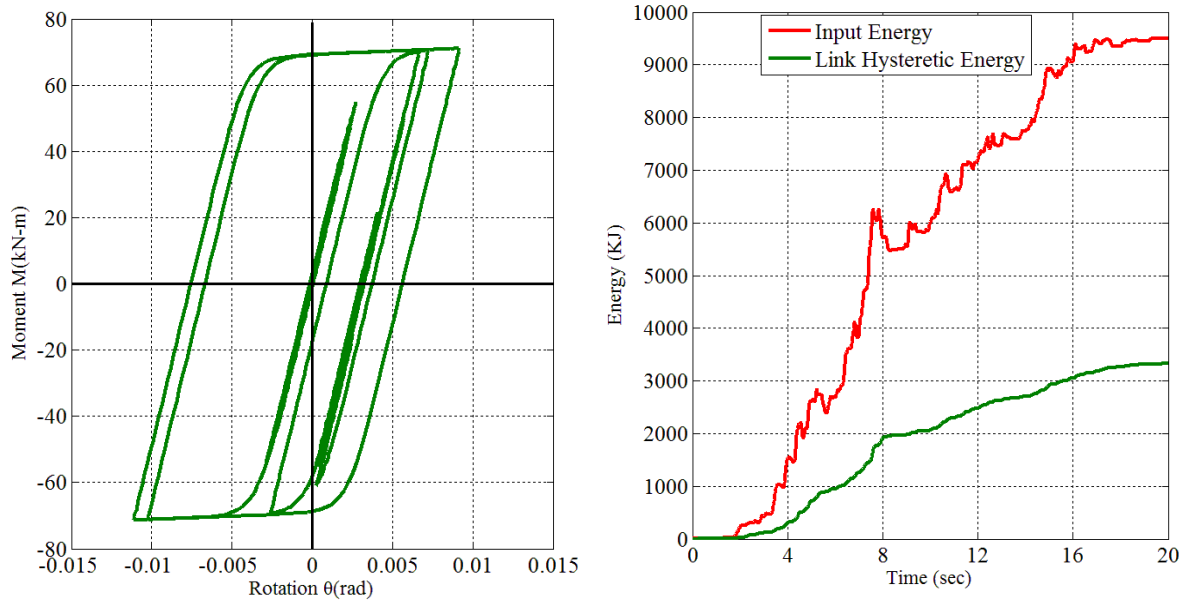


Figure 6: (a) The hysteresis loop of the steel tubes and (b) The input energy and the energy absorbed through the hysteretic flexural behavior of the steel tubes (Link Hysteretic Energy) for design ground acceleration $a_g=0.24g$.

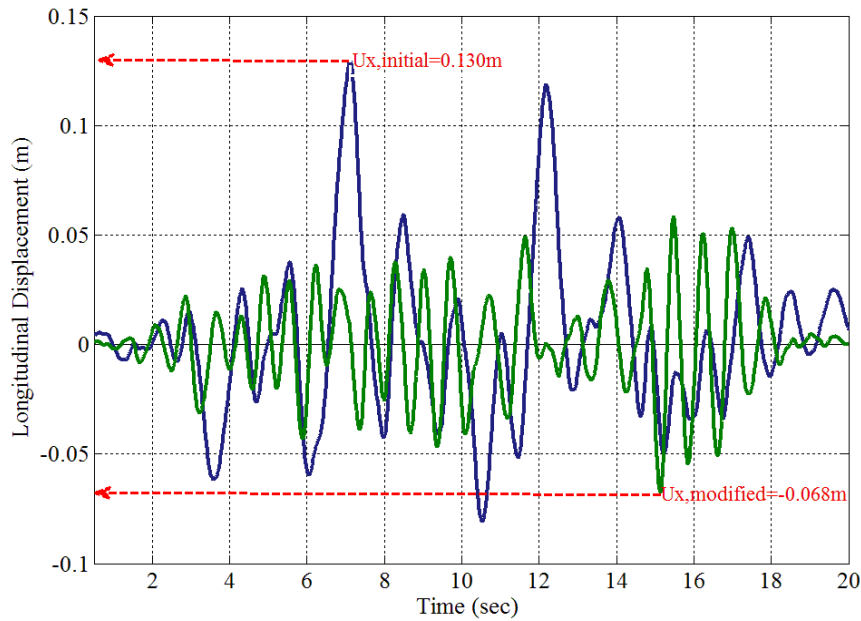


Figure 7: Longitudinal displacement time history (Joint of the deck over pier P2) for the initial and modified bridge system (soil class B, ground acceleration: 0.24g).

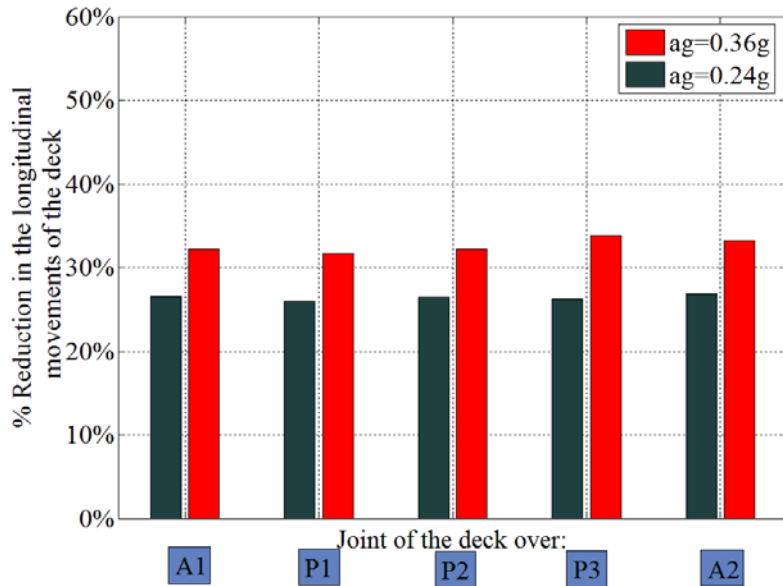


Figure 8: The percentage reductions in the longitudinal movements of the deck for two ground accelerations 0.24g and 0.36g.

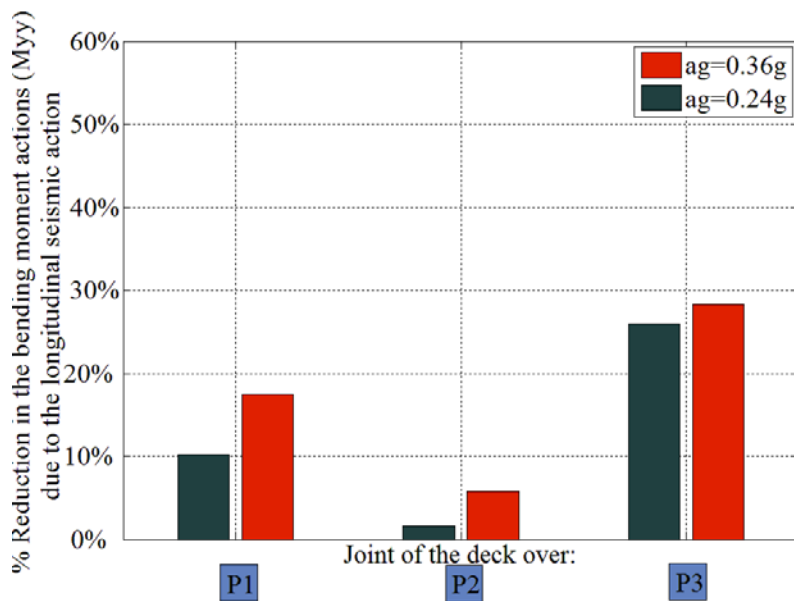


Figure 9: The percentage reductions in the bending moment actions M_{yy} at the base of the piers for two ground accelerations 0.24g and 0.36g.

6 CONCLUSIONS

In this study the seismic efficiency of an alternative to seismic isolation practice is investigated. The aforementioned purpose is achieved through the improved seismic participation of the bridge abutments. The deck slab is extended onto the embankments and is rigidly connected with transversely directed rows of hollow section steel tubes. Based on the results of this study, the following conclusions are drawn:

- The system is significantly efficient in seismic isolated bridge systems as a great part of the induced seismic energy is dissipated through the hysteretic behavior of the restraining members.
- The proposed seismic restrainer effectively reduces the movements of the deck and by extension the seismic actions of the piers, their foundations and the actions of the bearings. The aforementioned reduction is of the order of 27% for the longitudinal design earthquake.
- The ground acceleration influences the efficiency of the proposed restraining system as the restraining members dissipate greater part of the induced seismic energy through their hysteretic behavior.
- The restraining effect also reflects on the seismic actions of the piers (shear actions and bending moments) as well as on the bearings type and dimensions leading to cost-effective bridge design.
- The restraining effect of the proposed system is significant even in stiffer bridge resisting systems. The resulting bridge system is jointless, and has explicit advantages concerning durability and driving convenience.

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