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AN EXTERNAL RESTRAINING SYSTEM FOR THE SEISMIC RETROFIT OF EXISTING BRIDGES

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ABSTRACT

An unconventional retrofitting measure is proposed for existing bridges, which has the ability to reduce the seismic actions. This is mainly achieved by an external restraining system consisting of IPE-steel piles driven in the existing backfill soil and a restraining slab. The slab interacts with the deck slab of the existing multi-span simply supported (MSSS) bridge system and transmits part of its seismic actions to the piles and therefore to the backfill soil, which has the ability to dissipate part of the induced seismic energy. The proposed unconventional restraining system was implemented and analytically assessed in an existing MSSS bridge system. The study showed that the unconventional restraining system has the ability to reduce effectively the actions and to enhance the earthquake resistance of the existing bridge.

Introduction

In the recent two decades, the rapid deterioration of bridge structures has become a serious technical and economical problem in many countries, including highly developed ones. The problem is more intense in multi-span simply supported (MSSS) bridges, since most of these bridges were designed without consideration of seismic forces (DesRoches et al. 2003) at least until 1990 (Nielson and DesRoches 2006). In most bridge retrofitting schemes there are two alternative strategies that a designer may adopt. The first strategy is based on conventional strengthening techniques, which increase the capacity, namely the resistance (R_d), of the structure to meet the likely demand (Buckle et al. FHWA 2006). This can be characterized as the “direct retrofit”. The second strategy, which can be characterized as “indirect”, is based on reducing the demand namely the actions (S_d) on the structure such that its existing capacity is sufficient to withstand the given earthquake loading (Buckle et al. FHWA 2006). This latter approach usually involves the use of earthquake protective systems. New structural elements, which have the ability to control, in a passive manner, the response of the bridge or to increase the damping of the structure, are also introduced in many retrofitting schemes. In existing buildings the introduction of new shear walls is a common technique. These walls can be either internal i.e. infill walls, (Eurocode 8 Part 1-4 1996) or external walls. The use of new structural elements, which should

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respect the esthetics, has also been proposed in bridge structures, aiming at reducing the seismic actions. Link beams were introduced by Priestley (Priestley et al. 1996) in order to reduce the seismic actions, i.e. displacements or forces, of the multi-column bents. Also, the high superstructure moments can be limited in monolithic superstructure-column bridges, by use of longitudinal undeslung link beams. Unjoh (Unjoh et al. 2004) has also introduced the displacement restraint effect of the existing strengthened abutments in order to minimize the displacements of the bridge deck. These measures are consistent with the second alternative strategy described above.

In the present investigation, apart from the retrofit of deck slab of the bridge, which is made continuous, an external restraining system is proposed for the seismic upgrade of bridges. The retrofitting measures were parametrically investigated aiming at identifying, on the one hand, their earthquake efficiency and, on the other hand, their performance on an existing MSSS bridge system.

The existing bridge system

The present study utilized an existing multi-span simply supported (MSSS) bridge for the evaluation of the serviceability and earthquake resistance efficiency of the proposed retrofitting measures. The as-build bridge, given in Figure 1, which crosses Aliakmonas River in Northern Greece, was built in 1986, is straight, and has five spans and a total length equal to 151.0m. Each span has a length equal to 30.2m. The segments of the bridge are separated by narrow expansion joints, whose widths are of the order of 2.0cm. The deck of the bridge has a transverse width equal to 11.25m and consists of three simply supported precast and prestressed I-beams, precast deck slabs and a cast in-situ part of the slab. It is seated on both the abutments and piers through elastomeric bearings. The transverse movements of the deck are restrained by stoppers, which are installed on both the abutments and piers. The piers are I-wide flange cross sections, whose external dimensions are equal to 1.80m and 3.60m, in the longitudinal and transverse direction of the bridge correspondingly. The thickness of their web, which is oriented transversely, and flanges is equal to 0.60m. The piers are founded on spread foundations with dimensions 6.2m by 8.0m in the longitudinal and transverse direction of the bridge correspondingly. The bridge is founded on a stiff soil, which corresponds to ground type B, according to Eurocode 8 Part 1 (Eurocode 8 Part 1 2005). The seismic actions were taken into account, in the final stage of the as-built bridge design, by considering seismic coefficients 0.06 and 0.12 for the longitudinal and the transverse direction of the bridge correspondingly. These coefficients were found to correspond to ground accelerations 0.10g and 0.20g according to Eurocode 8 (Eurocode 8 Part 1, 2005).

The existing MSSS bridge described above was checked against the seismic actions set out in current codes. The checks of the critical members of the bridge were performed for the current site conditions and code requirements. It is noted that the design of a new bridge structure would require the use of a design ground acceleration equal to 0.16g and a ground type B according to Eurocode 8 (Eurocode 8 Part 1 2005). Hence, the seismic response of the bridge, in terms of displacements and actions, was determined for the seismic action described above.

The assessment of the existing piers' adequacy was mainly comprehended the check of their flexural and shear capacities against seismic actions. The piers' adequacy was assessed by comparing the yielding moment of the piers with the maximum action reached in each time history analysis. This means that the piers were considered to be sufficient if they remain elastic for the current seismic action. The analysis of the existing bridge showed that piers, P_2, P_3 and P_4 are inadequate to withstand the current seismic actions. More specifically, piers P_2, P_3 and P_4 found to be inadequate for the transverse seismic action. The shear capacities of the piers were estimated by Priestley (Priestley et al. 1994). It was found that the piers are adequate to receive the seismic shear actions in both the longitudinal and transverse direction. The existing spread foundations of the piers however were found to be inadequate to receive the resulting seismic actions. The critical check for the foundations was proved to be the one referring to their eccentricities, which are induced by the piers. The bearings of the existing bridge were also found to be inadequate. Finally, the stoppers were checked and found to be adequate for the seismic actions set out in current code design (Eurocode 8 Part 2 2005).

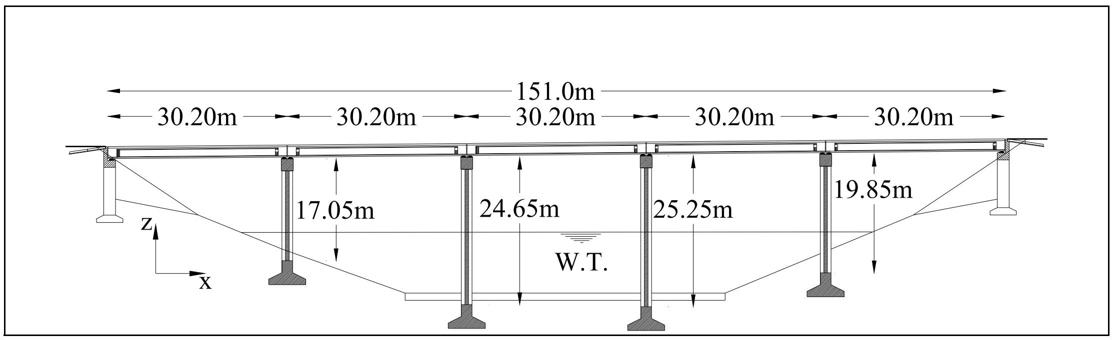

Figure 1. Longitudinal section of the existing as-built MSSS bridge system.

Description and optimization of the retrofitting scheme

The retrofitting scheme included conventional as well as unconventional structural measures, which were combined together. The selection and the design of each structural measure was based on an optimization procedure. Firstly, the deck slab was made continuous and only two expansion joints, one above each abutment, were kept open in order to accommodate the serviceability needs of the deck. The existing intermediate expansion joints were filled with high strength concrete, while cable restrainers were implemented in order to enhance the tension resistance of the connecting filling material. These expansion joints are separating the retrofitted continuous bridge deck from the restraining slab of the external system. It is noted that the existing wing-walls were separated from the backwall and the formulation of a knock-off detail at the top of the last was deemed to be necessary in order to avoid undesirable slumping and rotations of the existing abutment.

Secondly, the unconventional restraining system, given in Figure 2 was implemented. It consists of two parts: (a) the restraining slab and (b) the IPE-steel piles. The restraining slab is casted onto the existing approaching element. This slab plays the role of the pile-cap of the IPE-steel piles. Its thickness was selected to be equal to 0.50m over the pile groups, while this thickness is decreased over the abutment to 0.30m. The width of the restraining slab was selected to be equal to the distance between the existing wing-walls, i.e. equal to 11.25m. The second part of the external system is the IPE-steel pile groups. The piles were driven in the existing backfill soil in a depth equal to 7.0m. The ninety six (96) IPE 330 steel piles were set in two pile-groups of 48 piles. The distance between the sequential piles was selected to be equal to 1.0m in both directions-longitudinal and transverse.

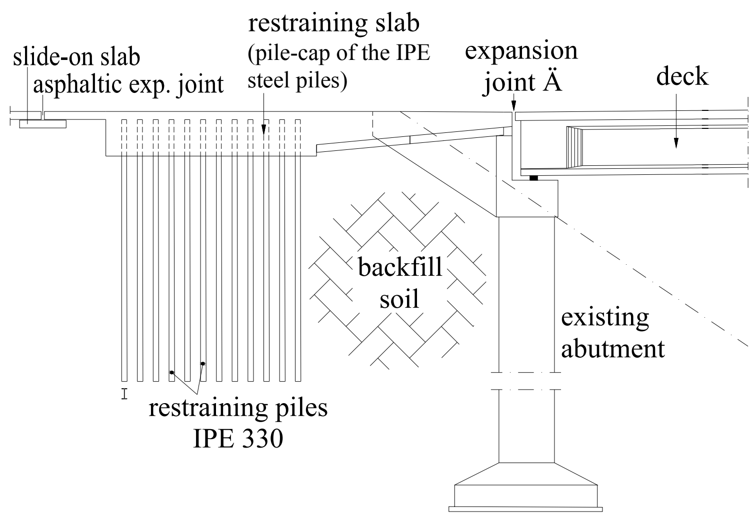

Figure 2. Longitudinal section of the proposed restraining system with the IPE-steel piles and the slab (pile-cap of the piles).

The design of the restraining pile groups was based on an optimization procedure, which is briefly discussed in the present paragraph. This optimization included the selection of their material, cross section, location and length. The steel piles have many advantages as far as their constructability and deformability is concerned (Arockiasamy et al. 2004). Among a large number of steel sections, the IPE-steel sections were found to be the optimum selection, as they provide one axis with a high value of the moment of inertia, while their weak-axis bending has a moment of inertia much smaller than the one in the stiff direction. This selection ensures, that the piles are not highly stressed during the bridge service, which is important for their durability according to Arsoy (Arsoy 2000). The piles were chosen to be oriented with their weak-axis bending longitudinally in order to minimize its stresses (Arockiasamy et al. 2004). The cross section of the IPE-piles was selected after checking different sections. The IPE 330 steel piles were found to be the optimum design selection as, on the one hand, they accommodate the serviceability needs of the expanding deck, while on the other hand maximize the desirable seismic participation of the backfill soil. It is noted that the critical design combination, which finally determined the optimum selection of the IPE-steel piles was the thermal expansion of the deck.

The width of the end expansion joints was also a critical design parameter during the optimization of the restraining system. It is reminded that the deck of the bridge was made continuous and preserved only two expansion joints, with a clearance equal to Δ, see Figure 2, at the abutments. The expansion joints were designed to absorb part of the expansion of the deck while, secondarily, the flexibility of the IPE piles accommodate the rest of the expansion of the deck when expansion joints Δ are closed. The in-service interaction of the deck with the restraining system could be avoided by the provision of wider expansion joints i.e. with a large Δ. However, wider expansion joints would reduce the earthquake efficiency of the restraining system, (Mitoulis 2005; Tegos et al. 2005) and would cause an increase in the magnitude of the collision forces according to Jonkowski (Jankowski et al. 2000). The selected restraining piles, i.e. IPE 330, were found to respond in an elastic manner for the maximum constraint longitudinal movement of their pile-cap up to 20mm. This means that the piles can absorb the total maximum expansion of the bridge deck. Therefore, the expansion joints were chosen to be closed (Δ=0) during the implementation of the bridge retrofitting.

Modeling of the analyzed bridge systems

The resulting model of the unconventional bridge system is given in Figure 3. The deck of the bridge was modeled by frame elements. The deck is supported on both the abutments and piers through low damping rubber bearings, see Detail 2 in Figure 3. The bearings were modeled by link elements, which correspond to its translational and rotational stiffness. These values were calculated according to Naeim and Kelly (Naeim and Kelly 1999) elastomeric bearing model. Stiff zones were used in order to take into account: (a) the distance of the center of gravity of the deck's cross section from the top (head) of the bearings, (b) the support of the deck on the bearings and (c) the transverse width of the pier's head.

The possible collisions which occur during earthquake were modeled by means of impact elements, see Detail 1 in Figure 3. The clearance of joint Δ, which corresponds to the separation

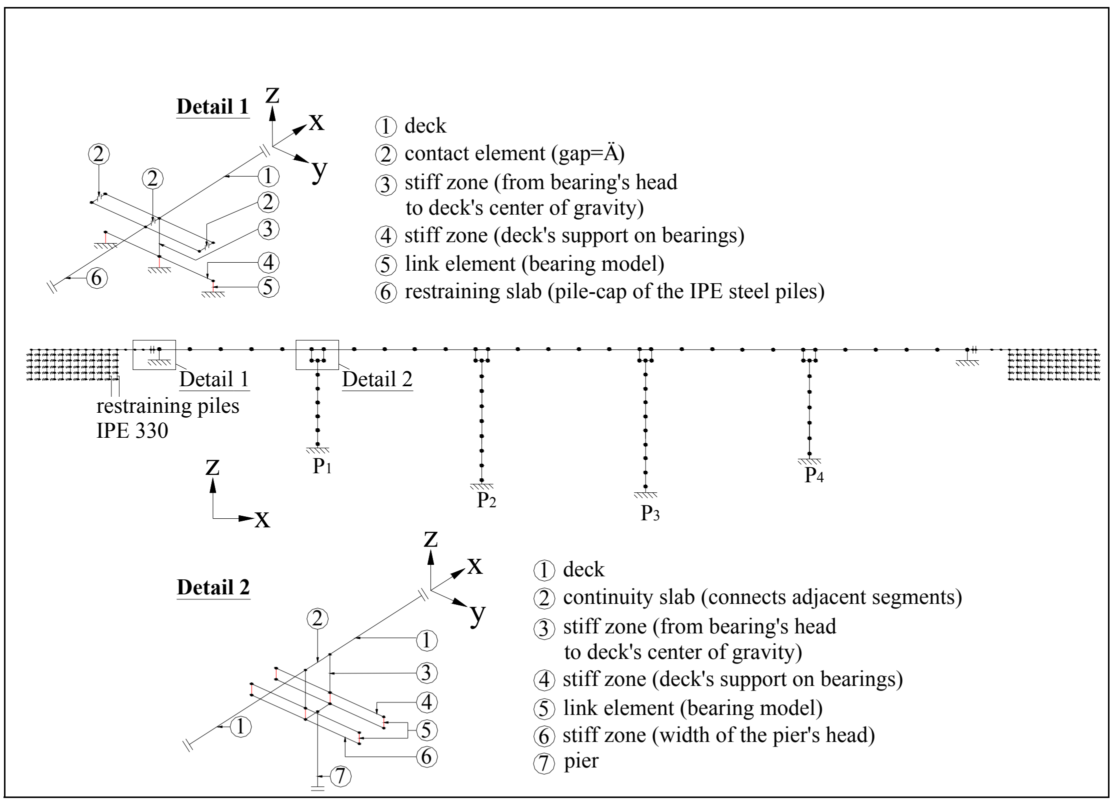

Figure 3. The analytical model of the retrofitted bridge with the external restraining system.

distance between the deck slab and the restraining slab, was modeled by gap elements offered by SAP 2000 (SAP 2000 Computers and Structures 2007). These elements are strictly geometrically non-linear and they are resisting only to compression, which is developed during the contact of the deck with the restraining slab. The compression stiffness of the impact elements used was determined by the average of the axial stiffness of the restraining slab and the axial stiffness of the deck slab according to Anagnostopoulos and Jankowski (Anagnostopoulos 2004; Jankowski et al. 2000). The Detail 1 given in Figure 3 shows the three multi-linear links used for the modeling of the possible impact interaction of the deck with the restraining slab at the end expansion joints. The width Δ of these joints at the beginning of the seismic event can vary between zero and a maximum width equal to 20mm, depending on the bridge temperature. All the analysis of the retrofitted bridge system were performed mainly by considering an average value of this joint, i.e. a Δ equal to 10mm. The damping, which occurs due to the inelastic collisions of the deck with the restraining slab, was modeled by an equivalent viscous damper according to Anagnostopoulos (Anagnostopoulos 2004). The three link elements are connected by transversely directed stiff zones, see Detail 1 in Figure 3, which correspond to the high value of the in-plane stiffness of the deck segments.

The resistance of the soil, surrounding the IPE-steel piles, was modeled by bilinear springs. The common bilinear P-y curve of API (API 1993) was adopted for the modeling of the soil resistance. The procedure followed the methodology described by Basu (Basu and Knickerbocker 2004). The ultimate loose resistance of the sand per unit length of the pile, (P_u), was determined according to Haliburton (Haliburton 1971). The effective soil weight was assumed to be equal to 18.0KN/m^3 and the diameter of the pile, which is necessary for the calculation of the P-y curves, was taken equal to 330mm, namely equal to the height of the IPE section. The soil was considered to be cohesionless and it was assumed to enter in its inelastic range at a deformation equal to 25mm, according to Sextos (Sextos et al. 2003). The group effects were ignored, namely unitary m-factors were assumed, as the distance between the piles (i.e. 1.0m in both horizontal directions) was chosen in order to ensure the reduction of these effects, see ATC-32 (ATC-32 1996). Moreover, the reported gapping between the pile and the soil (Rodriguez-Marek and Muhunthan 2005) was not taken into account in the analytical calculations, as the aforementioned effect would favor serviceability. Hence, the analytical study leads to the conservative design of the restraining piles, as far as their serviceability is concerned.

Parametric study-Results

The existing and the retrofitted bridge system were subjected to artificial accelerograms that were compatible to ground type A, B and C dependent Eurocode 8 elastic spectra, (Eurocode 8 Part1 2003). The non-linear dynamic time history analysis used five (5) artificial accelerograms which were generated with the computer code ASING (Sextos et al. 2003). Apart from the ground type also two different peak ground accelerations, i.e. 0.16g and 0.24g were considered. Dynamic non-linear time history analysis was implemented using the average Newmark acceleration method (Chopra 1995), as this method is the most robust to be used for the step-by-step dynamic analysis of large structural systems. The mass and stiffness proportional damping was chosen and critical damping ratios equal to 5% and 4% were considered for the first and the second modal period of the analysed bridge systems according to Aviram (Aviram et al. 2008).

The restraining effect of the proposed retrofitting measures was mainly assessed by calculating the percentage reductions in the longitudinal and transverse movements of the deck of the retrofitted bridge. The reductions were expressed by Equation 1, in which $u_{,E,EX.}$ and $u_{,E,RT.}$ are the seismic displacements of the deck of the existing and the retrofitted bridge respectively. These displacements are either longitudinal or transverse and they are measured on the joints of the deck above the abutments and the piers.

$$P.R. = \left(1 - \frac{u_{E,RT}}{u_{E,EX.}}\right) \cdot 100 \tag{1}$$

Figure 4 shows the variation of the P.R. factor in case the bridge is constructed on an area which belongs in Seismic Zone I, i.e. with a design ground acceleration equal to 0.16g. The horizontal axis corresponds to the deck joints above its sequential supports i.e. A_1, P_1, P_2, P_3, P_4 and A_2, where A_i is the support of the deck on **i**-abutment, while P_i is the support of the deck above **i**-pier. From Figure 4(a) it can be extracted that the proposed retrofitting measures reduce the longitudinal movements of the deck by 18% to 74%. The smaller reduction corresponds to a stiff ground type A while the greater value of P.R. factor refers to ground type C. Figure 4(b) illustrates the P.R. factors for the transverse movements of the deck. The figure shows that the restraining system also contributes in the transverse direction of the bridge, as the displacements of the deck are up to 46% reduced. In the figure no values are given above the abutments, as the transverse displacements of the deck are restrained by stoppers. Consequently, the transverse displacements of the deck over the abutments are negligible. The displacements of the deck are more effectively restrained over the central piers P_3 and P_4. The desired influence of the system in reduced over pier P_2 as the transverse displacement of the deck is up to 19% reduced. Finally, the displacements of the deck over pier P_1 seem to be increased by up to 16%, in case a ground type A or B was considered. However, the slight increase in the displacements of the deck does not influence pier P_1 or its bearings in a fashion that these elements need to be strengthened or replaced. Specifically, pier's P_1 loading was found to be increased by up to 12%. However the pier was proved to have capacity to withstand the slight increase in its actions. On the other hand, the bearings are protected by the existing stoppers of the bridge, which means that no significant shear deflection is induced to them in the transverse direction of the bridge.

Figure 5 shows the variation of the P.R. factor for the higher seismicity, which corresponds to ground acceleration 0.24g. The P.R. factors represent the alterations in the movements of the retrofitted bridge system in the longitudinal, Figure 5(a), and in the transverse, Figure 5(b), direction of the bridge. It is observed that the longitudinal movements of the bridge deck are reduced by 28 to 78%. The restraining effect of the external system seems to be increased in comparison to the corresponding one in the lower seismicity, i.e. 0.16g. This effect is attributed to the stronger participation of the restraining system, as the bridge deck responds with larger displacements when the seismicity is increased. The restraining system also influences the bridge response in the transverse direction. The seismic displacements of the bridge are up to 46% reduced in this direction.

The ground type, i.e. A, B or C, also seems to influence the seismic efficiency of the retrofitting scheme. The longitudinal displacements of the deck of the retrofitted bridge system seem to be more effectively restrained when the ground type is softer i.e. C instead of A. This means that the proposed retrofit is more effective in bridges which are founded on softer grounds. However, this finding does not seem to be in accordance with the corresponding response of the bridge in the transverse direction. In this direction, the results show that the ground type does not have a predictable influence on the efficiency of the proposed restraining system. Finally, the width of the end expansion joints, i.e. Δ, was investigated in order to identify its influence on the seismic efficiency of the retrofitting measures. The joints were considered to be either closed, (i.e. Δ=0) or open at the beginning of the seismic event. For the second case two different values were considered, i.e. 5mm and 10mm. The analytical study found that, in general, the wider the joint the lower the efficiency of the proposed restraining system. This finding applies to all the different types and ground accelerations considered in the parametric study.

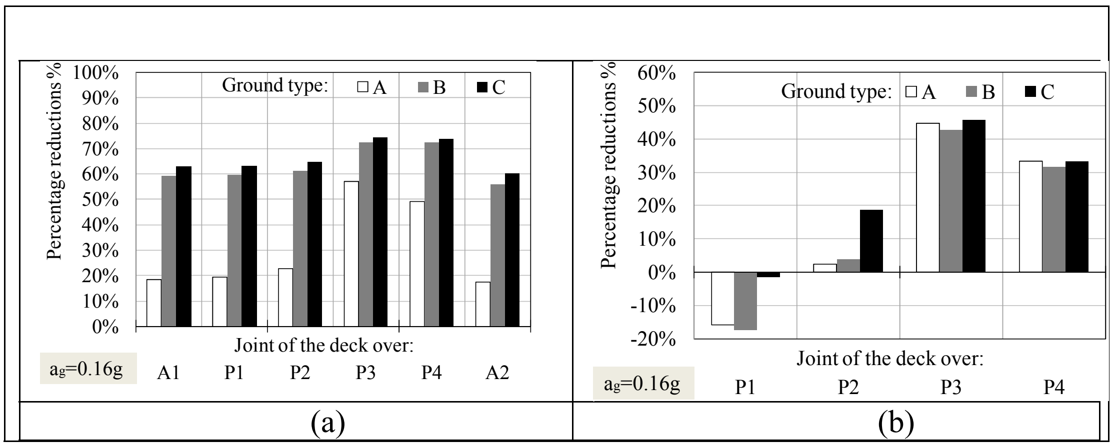

Figure 4. The percentage reductions (P.R. factors): (a) in the longitudinal and (b) in the transverse movements of the deck of the retrofitted bridge system for three different ground types, (ground acceleration: 0.16g).

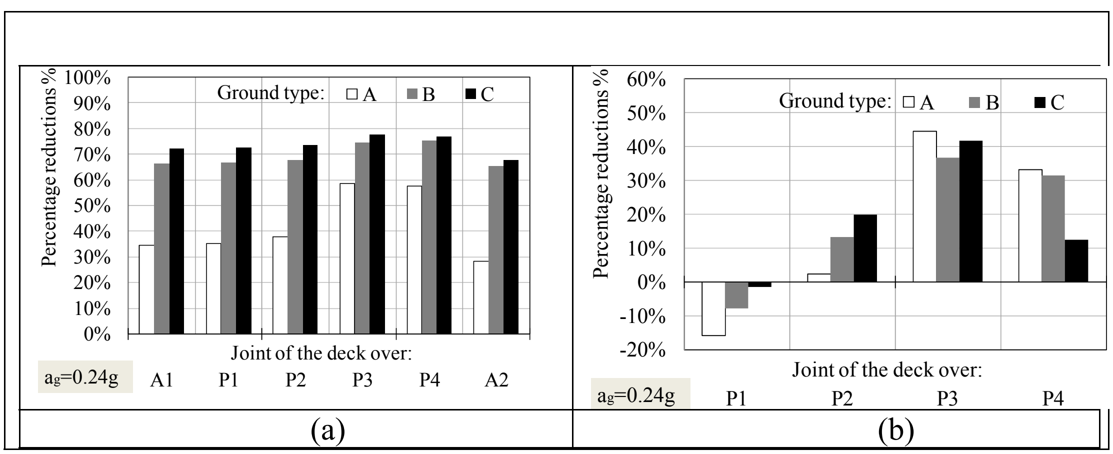

Figure 5. The percentage alterations (P.R. factors): (a) in the longitudinal and (b) in the transverse movements of the deck of the retrofitted bridge system for three different ground types, (ground acceleration: 0.24g).

Conclusions

An external restraining system in combination with the connection of the adjacent segments is proposed for the seismic retrofit of existing MSSS bridges. An analytical study was performed in order to identify the earthquake resistance efficiency of the proposed system. The restraining scheme adopted has a significant effect on the seismic displacements of the retrofitted bridge deck. The comparison of the longitudinal movements of the existing MSSS and the unconventionally retrofitted bridge system showed that the displacements of the last bridge are reduced by 18% to 78%. The restraining system also contributes in the transverse direction of the bridge, as the displacements of the deck are 46% reduced in this direction. The slight increase in the displacements of the deck over pier P_1 does not influence the pier or its bearings in a fashion that these elements should be strengthened or replaced. A parametric investigation conducted in order to identify the influence of the ground acceleration, the ground type and the width of the end expansion joints Δ on the seismic efficiency of the restraining system. It was found that the longitudinal displacements of the deck are more effectively restrained when the high seismicity or when the soft ground type is considered. These parameters do not have a uniform influence in the transverse direction of the bridge. As far as the influence of the width of the end expansion joints Δ is concerned, it was found that the wider the joint the lower the efficiency of the proposed restraining system.

The implementation and the efficiency of the restraining system is strongly case-dependent. The study needs a complement with analysis of more bridge structures, as the length of the bridge and the height of the piers were not included in the parametric study and they are considered to have a significant influence on the efficiency of the proposed restraining system.

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