A new earthquake resistant abutment as means to reduce the seismic demand of a railway bridge

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SUMMARY:
Previous studies showed that seat type abutments, which are not considered to participate in the earthquake resisting system (ERS) of bridges, can reduce, with the assistance of the backfill soil, the seismic actions of bridges. In this framework, the seismic efficiency of a new seat-type abutment was studied. The new abutment decouples the in-service response of the bridge from the backfill soil by an expansion joint, while utilizing the resistance of the abutment's wing walls and the backfill soil during earthquake and reduces the seismic demand of the bridge. The investigation showed that the seismic participation of the abutment and the backfill soil reduce effectively the piers' seismic demand in terms of actions when utilized in railway bridge. The proposed unconventional bridge design scheme is considered to be useful for future design of intermediate to long-span bridges.

Keywords: bridge, unconventional seat-type abutment, pounding interaction, reduce, seismic demand

1. INTRODUCTION

The design of bridges has to compromise serviceability and earthquake resistance, which are conflicting components of the same problem while they impose opposite design requirements. Serviceability, 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 (Eurocode 1 Part 1-5, 2003), shrinkage and creep (Arockiasamy et al., 2005). Serviceability requires bearings and expansion joints, which uncouple the response of the deck from the abutment and from the embankment.

The design of expansion joints typically takes into account the in-service and part of the seismic displacements of the deck (Eurocode 8 Part 2, 2005). However the cost of joints to accommodate large seismic displacements is usually high and this is the reason why a compromise is usually adopted (Gloyd, 1996). In integral abutment bridges (Abs) the web of the abutment is separated from the backfill by an expanded polystyrene layer i.e. EPS geofoam (Horvath, 1998a; Horvath, 1998b; Pötzl et al., 2005). This EPS layer plays the role of the "expansion joint" for integral abutment bridges, which minimizes the in-service interaction of the deck with the approach embankments. Furthermore, stub-type abutments are usually implemented in the US, (Lock et. al., 2002) in order to minimize the backfill disturbances by the deck. The stub web of the abutment is usually founded on a row of steel piles oriented with weak axis in the longitudinal direction of the bridge; to minimize its in service stresses (Arockiasamy et. al., 2004). For long-span integral abutment bridges typically semi-integral abutments (Arsoy, 2000) are implemented as they offer low flexural loading of the abutment's piles.
In earthquake prone areas bridges are either seismically isolated (Kunde et al., 2003), by utilizing bearing and/or viscous dampers while controlling possible excessive displacements (Kawashima K., 2004), or develop he hysteretic behaviour of the piers that means that a q-factor greater than 1 is used during analysis (Eurocode 8 Part 2, 2005) or a R-factor (UBC, 1997). However, there are bridge's non-structural elements which are not considered to participate in the earthquake resisting system (ERS) which can be developed to reduce the seismic demand of bridges. The abutment and the

backfill soil can be utilized (Kotsoglou et al., 2007; Mitoulis et al., 2010) in order to reduce the movement of the deck (Mylonakis et al., 1999; Maragakis, 1985; Mikami et al., 2008) and, in turn, reduce the structural cost of the bridges (Nutt, 1999).

In this framework, a new seat type abutment is parametrically studied. The abutment accommodates both serviceability and earthquake resistance, as it participates strongly during earthquake and reduces the bridge's seismic demand. A benchmark railway bridge actually built along the longest highway in Northern Greece was used as the reference case on which different design scenarios were performed an analytically analyzed and compared. The study showed that the abutment can be utilized for future cost-effective seismic design of contemporary bridges.

2. BENCHMARK AND RE-DESIGNED BRIDGE SYSTEMS

2.1. Description of the benchmark bridge

A seismically isolated railway bridge that belongs to OSE and is located at Polykastro, Greece, Figure 1a, was used as benchmark for the study. This bridge was considered to be the benchmark bridge model BM_0. It is straight, has four spans and a total length of 168.0 m. The end span has a length of 39.0 m, while the two central spans are 45 m long. The box girder deck, Figure 1b, has a total width equal to w_{dect} = 13.40 m. The piers, Figure 1c, have a hollow rectangular cross section with longitudinal and transverse dimensions equal to 3.00 m and 5.50 m and a thickness 0.45 m. The piers are founded on 4x4 pile groups, which are connected to 11.0 by 11.0 m pile-caps, whose cross section height is equal to 1.50 m. The diameter of the piles is 0.80 m and their length is 15.0 m for all piers. The deck is seated on both the abutments and piers through lead rubber bearings, two on each pier and each abutment. The bearings dimensions in plan are equal to 900x900 mm and 1200x1200 mm on the piers and on the abutments respectively, the total thickness of the elastomeric rubber is 231 mm and 286 mm, while the diameter of the lead is 200 mm and 250 mm at the abutments and at all piers correspondingly.

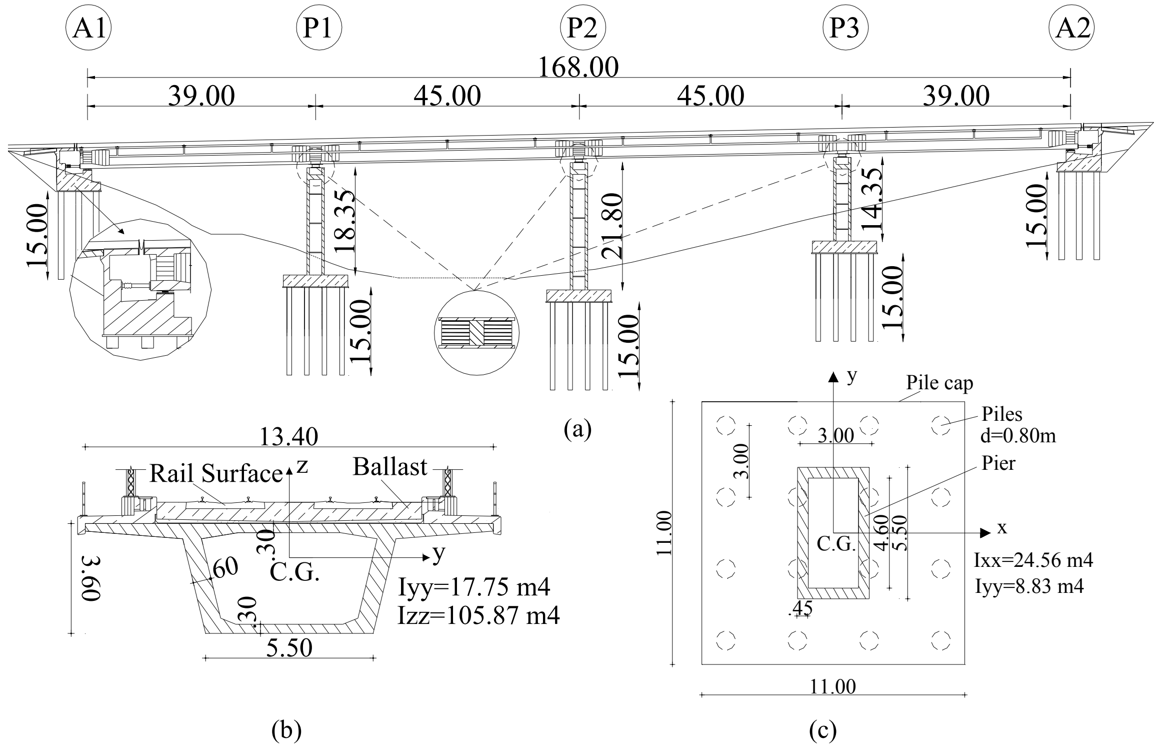


Figure 1. The benchmark bridge that belongs to OSE located at Polykastro, Greece, (a) Longitudinal section, (b) Cross section of the deck, (c) plan view of the pier's foundation.

Two hydraulic dampers are installed at each abutment with a capacity c=2350 kN s/m and a=0.15. The abutment, Figure 2, is seat-type and provides an expansion joint clearance (Eurocode 8 Part 2, 2005) between the deck slab and its back wall. Stoppers, which restrain the transverse movements of the deck, were installed on the piers. The bridge is founded on ground type B (Euocode 8 Part 2, 2005) and a design ground acceleration equal to 0.24g was used in the final design. The importance factor adopted was equal to γ_I=1.3, while the behavior factors were equal to 1.0 for the longitudinal, the transverse and the vertical direction of the bridge.

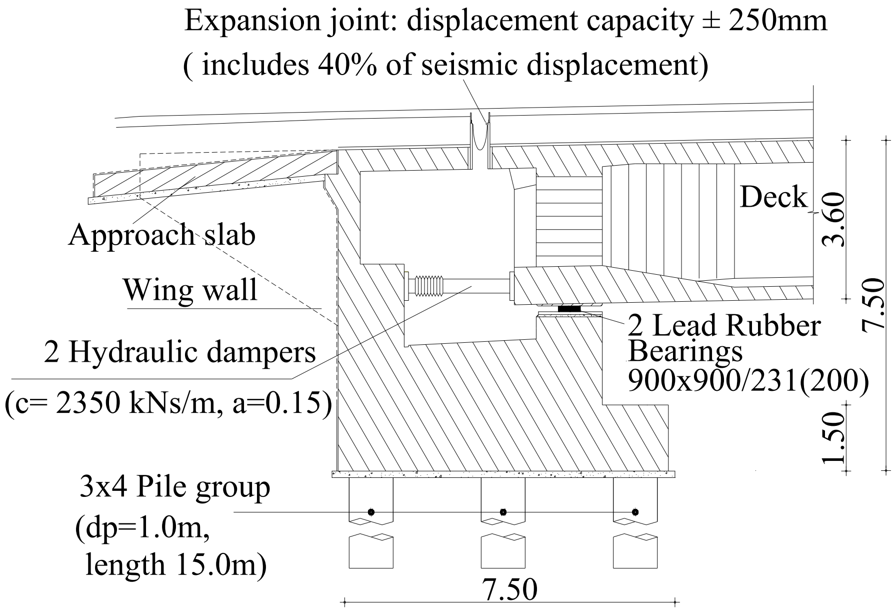


Figure 2. Longitudinal section of the as-built abutment

2.2. Description of the re-designed bridge

The re-designed bridge system has the same geometry as the benchmark bridge system, while the abutments were re-designed, as shown in Figure3. The primary feature of the new abutment, Figure 3, is the decoupling of the in-service response of the bridge from the backfill soil, which is ensured by the appropriate selection of a clearance at the expansion joints, and the utilization of the resistance of the abutment's wing walls, primarily, and the one of the backfill soil, secondarily. This re-design aimed at reducing the seismic demand of bridges. The in-service constraint movements of the deck were accommodated by small as possible clearances between the deck and the abutment, that do not allow the free longitudinal movement of the deck. The clearance accommodates only the thermal expansion of the deck. The abutment was also equipped with a seismic isolation system of low damping rubber bearings, two bearings per abutment.

Two more alternative bridge systems were examined aiming at identifying the impact of the design scheme on the structural cost of the bridge. The comparisons were performed on the basis of the initial isolated bridge that had both lead rubber bearings and viscous dampers. The first re-design scheme, shown in Figure 4, included: (a) abutment with robust wing wall and (b) low damping rubber bearings that are low damping bearings. The bearings dimensions in plan were equal to 900x900 mm and 1200x1200 mm at the abutments and at all piers correspondingly while all bearings had a total thickness of elastomer 100 mm. There are two bearings per pier or abutment. Stoppers, which restrain the transverse movement of the deck, were installed on the piers. This model was attached the code name BM_1.

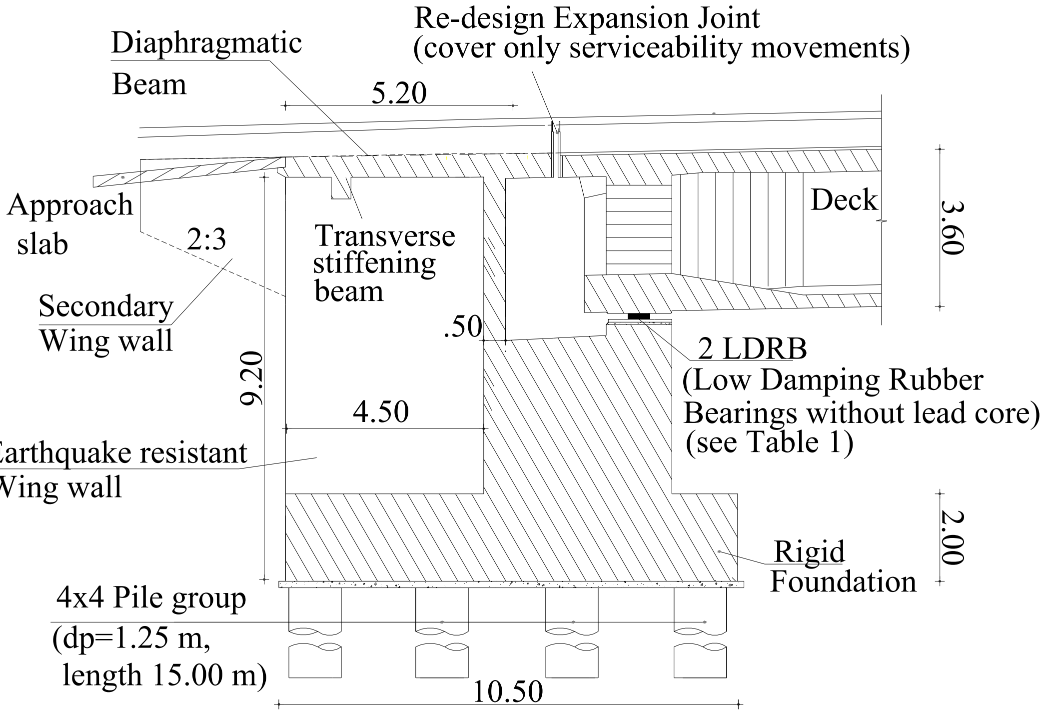


Figure 3. Longitudinal section of the proposed abutment with strong seismic participation

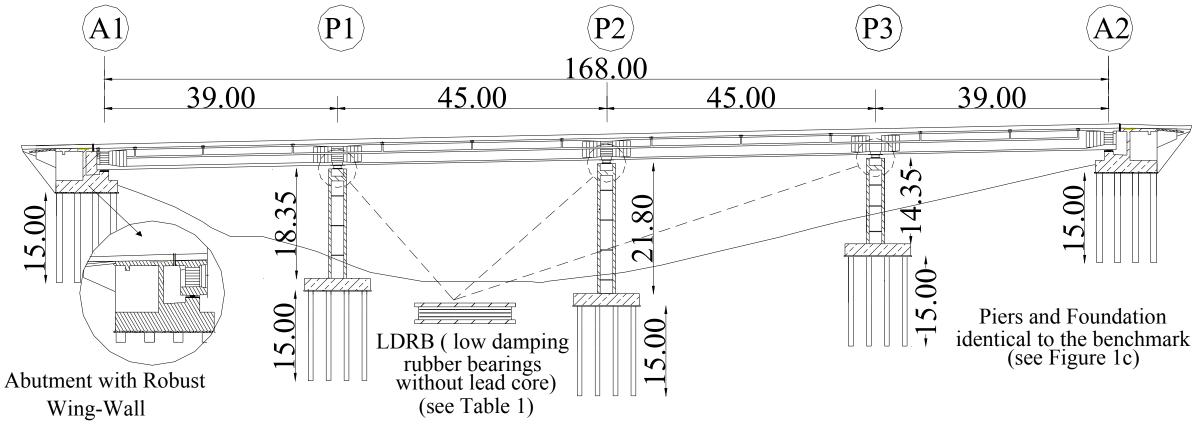


Figure 4. Longitudinal section of the first bridge design scheme

The second re-design scheme of the bridge, shown in Figure 5a, was also designed to have the abutments with robust wing walls. The piers were re-designed in order to provide flexibility to the bridge system as means to maximize the participation of the system abutment-backfill soil, that was expected to reduce the movements of the bridge deck and hence to effectively reduce the seismic actions and the seismically induced P-delta effects of the piers. This integral re-design approach on the earthquake resisting system led to the use of smaller pier sections and to an attempt to connect rigidly the mid-pier with the deck, as means to develop the hysteretic damping of the piers. This re-design scheme was deemed to be quite desirable as on the one hand the hollow piers of the benchmark bridge have disadvantages related to their construction as well as to their confinement and on the other hand the use of slight and flexible piers both improve construction convenience and seismic design of piers. The piers had circular section with a diameter equal to 2.0 m. The piers are founded on 3x3 pile groups, which are connected to 8.0 by 8.0 m pile-caps, whose cross section height is equal to 2.0 m, Figure 5b. The diameter of the piles is 0.8 m and their length is 15.0 m for all piers. The deck was rigidly connected to the mid-pier P_2 and was supported on the rest of the piers and on the abutments through low damping steel laminated rubber bearings, two per piers. The bearings dimensions in plan were equal to 900x900 mm and 1200x1200 mm at the abutments and at all piers correspondingly and the total thickness of the elastomeric rubber was 100 mm at all locations. Stoppers, which restrain the transverse movement of the deck, were installed on the piers. This model was attached the code name $BM_{2.}$

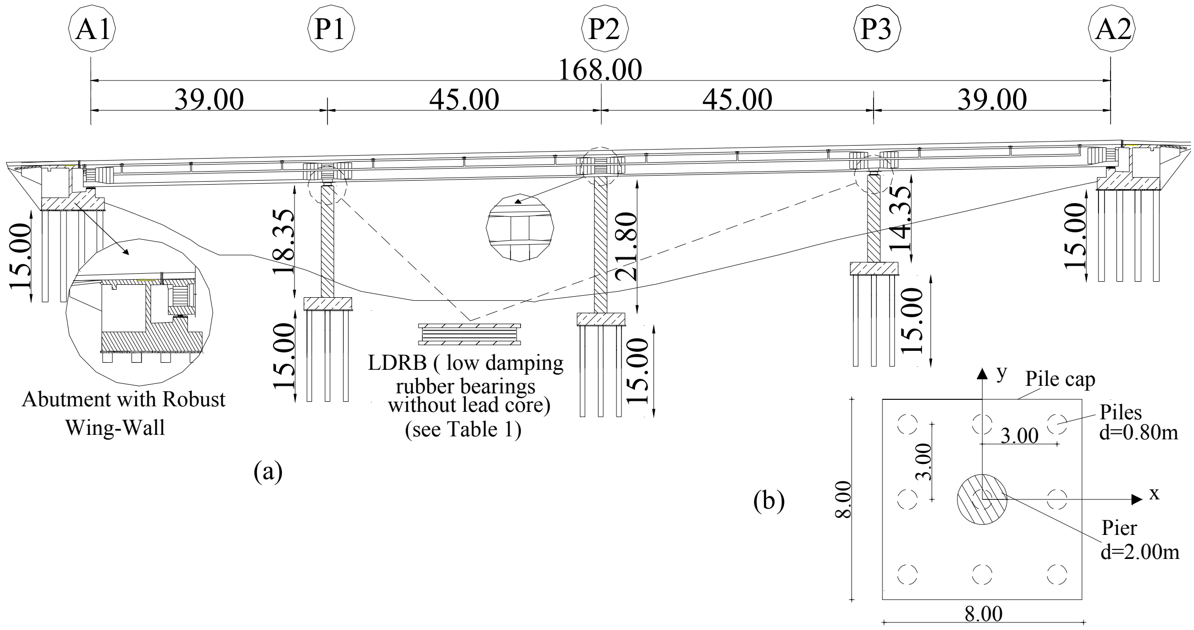


Figure 5. (a) Longitudinal section of the bridge BM_2, (b) Plan view of the pier's re-designed foundation.

Table 1. Benchmark and re-designed bridge systems

			Piers	**Abutment**		**Piers' foundation**	**Abutment's foundation**
Bridge	**Abutments**	**Piers**	**Bearings on piers**	**Bearings on abutments**	**Viscous Dampers**	**Pile-group Pile-cap Diameter**	**Pile-group Pile-cap Diameter**
Benchmark	Conventional Seat type	Hollow rectangular	Lead Rubber Bearings 1200x1200x286(250) *	Lead Rubber Bearings 900x900x231(200)	Yes C=2350 kNm/s a=0,15	4x4 piles 11x11m d_p=0.8m	3x4 piles 7,5x14 m d_p=1.0 m
1st Re-design scheme	With Robust wing walls	Hollow rectangular	Low Damping Rubber Bearings 1200x1200x100**	Low Damping Rubber Bearings 9000x900x100	No	4x4 piles 11x11m d_p=0.8 m	4x4 piles 10.5x14 m d_p=1.25 m
2nd Re-design scheme	With Robust wing walls	Circular	Low Damping Rubber Bearings 1200x1200x100**	Low Damping Rubber Bearings 900x900x100	No	3x3 piles 8x8 m d_p=0.8 m	4x4 piles 10.5x14 m d_p=1.25 m

* 1200x1200x286(250) Bearings dimension in plan 1200x1200, total thickness of elastomer 286mm, diameter of the lead core 250mm

** 1200x1200x100 Bearings dimension in plan 1200x1200 and total thickness of elastomeric rubber 100mm

3. MODELING

The seismically isolated bridge and the re-designed bridges were modeled and analyzed. The deck of the bridge was modeled by frame elements, which have the section properties of the deck, given in Figure 1b. The deck is supported on both the abutments and on the piers through bearings. The bearings were modeled by link elements, which model the corresponding translational and rotational stiffness of each bearing. The piers were also modeled by frame elements. The flexibility of their foundations was also taken into account by assigning six spring elements -three translational and three rotational, whose stiffness values are given in Table 2. These soil spring values were obtained by the geotechnical in-situ tests conducted for the design of the real bridge.

Table 2. Stiffness values of the spring elements used for the modeling of the foundations' flexibility

Piers	**K_x (kN/m)**	**K_y (kN/m)**	**K_z (kN/m)**	**K_{rx} (kNm/rad)**	**K_{ry} (kNm/rad)**	**K_{rz} (kNm/rad)**
P_1, P_2, P_3	$6.5x10^6$	$6.5x10^6$	$1.2x10^7$	$1.8x10^8$	$1.7x10^8$	$1.4x10^8$

The expansion joints are critical design parameters, which influence strongly the in service performance as well as the earthquake resistance of the re-designed bridges. For the re-designed bridges the expansion joints accommodate only the thermal movements of the deck. These movements are critical in the first years of the bridge service, and while creep and shrinkage have not been developed yet. The width of the expansion joints of the unconventional bridge deck by considering a conservative estimation of the existing joint at the occurrence of the seismic event. In order to minimize the expansion joints also the construction of the second part of the abutment is suggested to follow the completion of the deck of the bridge, which ensures that most of the total contraction of the deck due to creep and shrinkage has been developed.

The determination of the width of the expansion joint is influenced by two discrete design criteria: (a) the control of the maximum allowable compression of the deck by the stiff wing-walls and (b) the minimization of the width of the joint, which increases the desired seismic participation of the proposed abutment. Finally, it was assumed that joint ranges between 4.2 mm and 46 mm in case of the extreme expansion and contraction of the deck correspondingly (Mitoulis et al., 2010). On the other hand, a joint of the conventional bridge is designed to accommodate 40% of the nominal seismic longitudinal displacement and the width of it is 250 mm (Eurocode 9 Part 2, 2005). In Figure 6a and 6b, the stick models of the benchmark and the re-designed bridge systems are illustrated.

The backfill soil also participates during earthquake. Its seismic resistance was modeled by one linear spring element. The stiffness of the linear spring took into account the total unilateral passive resistance of the backfill soil according to CalTrans, equal to K_{imp}= 880 000 kNm (Caltrans, 1999; Elgmal and Saiidi, 2012).

The strongly non-linear response of the benchmark and the re-designed bridges was analyzed using the FEM code SAP 2000, ver. 14. Dynamic non-linear time history analysis was implemented and the direct integration, known as β-Newmark method, was used (Chopra AK., 1995). 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 period of the analyzed bridge systems correspondingly.

4. RESULTS AND DISCUSSION

The present study compared three different bridge alternative design schemes. The aim of this study was to determine the impact of the re-design attempts on the structural cost, Eurocode 8 Parts 1 and 2 were used for the re-design of bridges. The benchmark bridge was over designed in terms of joints and bearings.

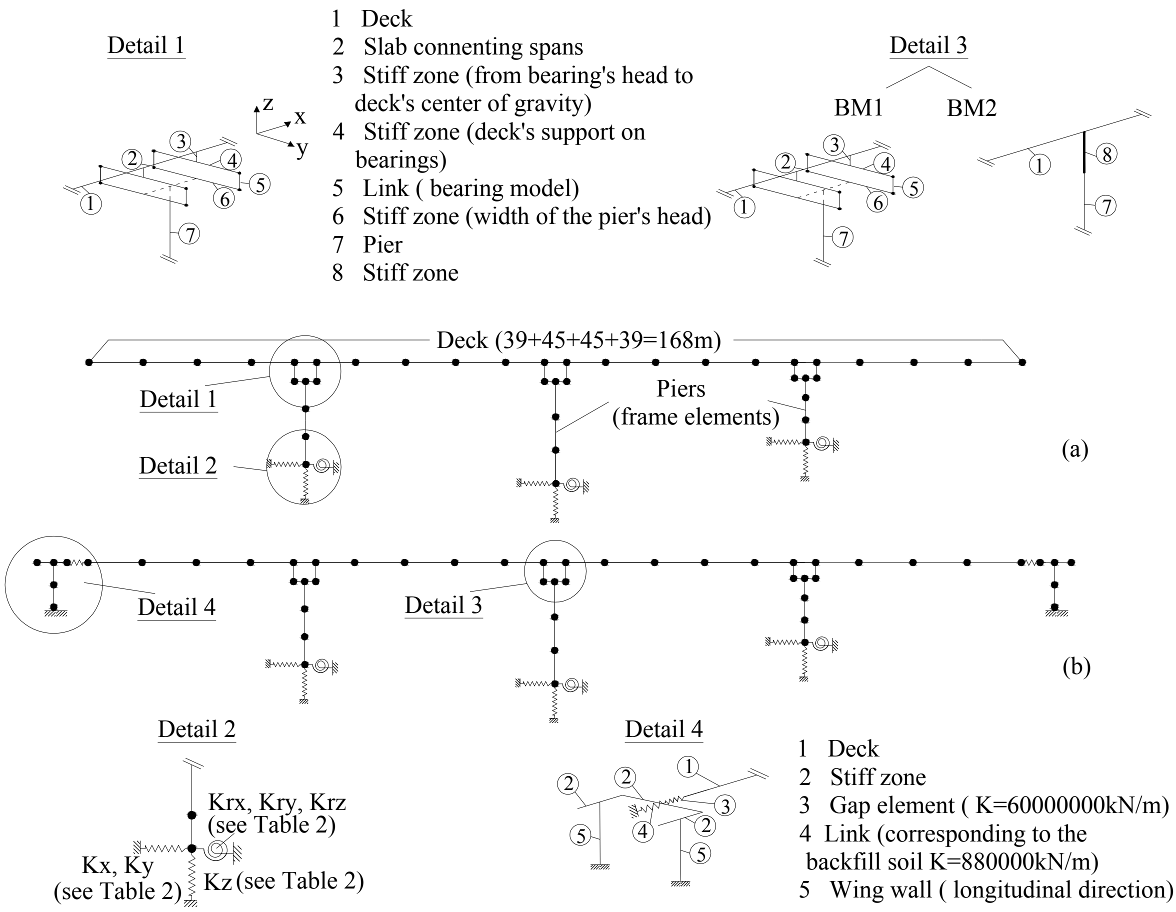


Figure 6. The model of: (a) Benchmark bridge, (b) Re-designed bridges

4.1. Seismic response of bridges

In Figure 7a the longitudinal response spectra of the deck for the BM_0, $BM_1$and BM_2 are illustrated. These spectra are the prospective response of the bridge deck in terms of deck pseudo-accelerations. The extrapolation of the spectra was performed using the deck's response acceleration time histories. All spectra were drawn for a mid-joint of the deck, using the SeismoSignal platform (SeismoSoft, 2010). Response spectra showed that the re-design attempts (BM_1 and BM_2), led to significant reductions in the longitudinal eigenperiod of the bridge. The longitudinal period of BM_0 was 1.10 s, while the period was reduced by 23 and 34% when the first (BM_1) or the second (BM_2) re-design schemes. The periods were found to be 0.85 and 0.73 s correspondingly. Consequently, the response accelerations of the bridge deck were increased from 6.5 m/s^2 (BM_0), to 25 m/s^2 (BM_1) and 24 m/s^2 (BM_2).

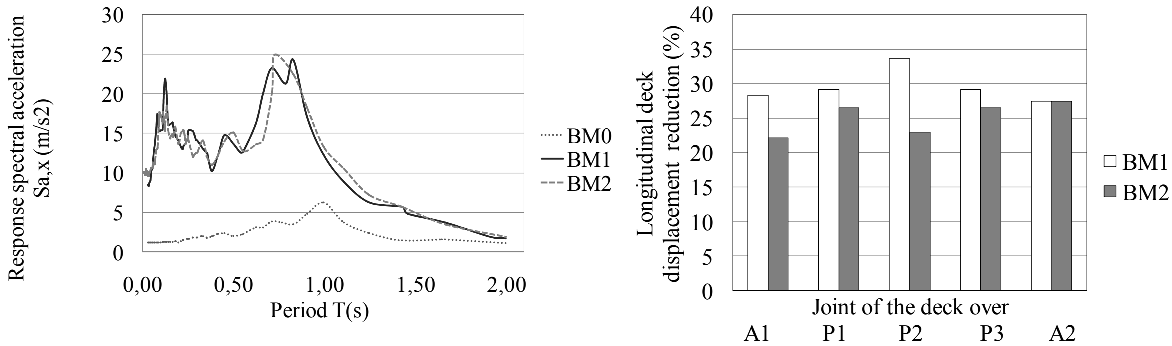
Despite the increase in the longitudinal acceleration of the bridge, the longitudinal displacements of the deck were found to be effectively reduced (see Figure 7b), in both bridge re-design attempts (BM_1 and BM_2). The percentage reductions in the deck's seismic displacements are expressed by Eqn. 4.1.

$$P.R. = \left(1 - \frac{u_{E,RE-D}}{u_{E.BENC}}\right) \cdot 100 \qquad \% \qquad (4.1)$$

In the above equation, P.R. is the percentage reduction in the displacements of the deck, $u_{E,RE-D}$. is the seismic displacement of the deck of the re-designed bridge and the $u_{E,BENC}$. is the seismic displacement of the deck of the benchmark bridge. It can be extracted that if P.R.>0 then the unconventional bridge system responds with smaller displacement. More specifically, the longitudinal displacements of the deck were found to be reduced up to 33 and 29% when BM_1 and BM_2 were studded correspondingly.

4.2. Economic considerations

The re-designed bridge systems were found to respond with smaller displacements and, further that, the seismic actions of the bearings were reduced. The re-design procedure followed the Eurocodes requirements. The estimation of the cost alterations in the re-designed bridge took into account the cost of the structural elements, whose design is strongly influenced by seismic loading. The deck of the bridge was not re-designed as it does not receive high seismic actions. The elements that were included in the estimation of the structural cost of the re-designed bridges were: (1) the piles, (2) the bearings, (3) the hydraulic dampers, (4) the expansion joints, (5) the abutment's reinforced concrete and (6) the piers reinforced concrete. The cost of the re-designed elements was the 21% of the total cost of the benchmark bridge and the cost of the other elements, which were not re-designed, remained constant. The percentage cost alterations cited in the paragraph below resulted by dividing the cost of each re-designed element by the known total structural cost of the as-built bridge, which was equal to 4608339 € according to the final design of the bridge.



(a) (b)

Figure 7. (a)The deck pseudo-accelerations longitudinal response spectra of benchmark bridge BM_0, 1st re-designed BM_1 and 2nd re-designed BM_2, (b) The percentage reduction in the longitudinal seismic displacements of the deck

In figure 8 the reduction of the total cost for the $BM_1$and BM_2 is illustrated. Specifically, the initial cost, namely the structural, was founded to be reduced up to 5% and 9% in the re-designed bridge system BM_1 and BM_2 correspondingly. Furthermore, the bearings and the expansion joints have to be replaced after some years of bridge service. It follows that not only the initial, namely the structural, but also the final cost of the seismic isolation system is higher in the conventional bridge. An estimation of the economic burden in case the bearings are re- placed every 20 years of bridge service leads to a reduction in the final cost of the bridge equal to 21% and 25%. The last estimation was performed by considering 120 years of bridge service life, which is a commonly used design assumption for bridges (Australian, 1999).

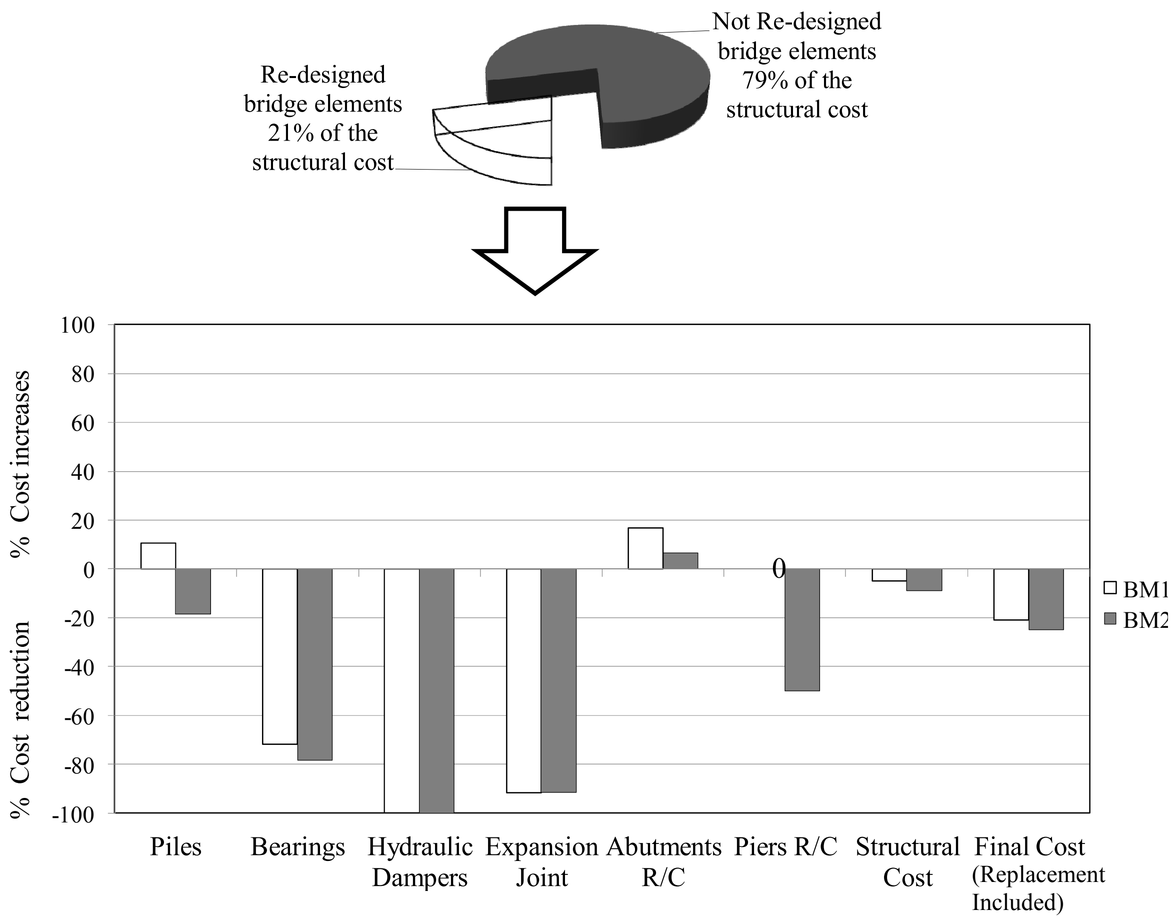


Figure 8. Cost sharing, the percentage cost alterations of the re-designed structural elements in the re-designed bridge systems and the percentage cost reduction of the structural and final cost of the 1st and 2nd re-designed bridge(BM_1 and BM_2)

5. CONCLUSIONS

Three different bridge design alternatives were investigated utilizing a benchmark railway bridge with a heavy isolation system. The study aimed at identifying on the one hand the possibility to avoid the use of expensive lead rubber bearings and dampers and on the other hand to estimate the impact of the attempted re-design on the structural and final cost of the bridge. The re-design of bridge-cases was attempted utilizing the current Eurocode provisions (Eurocode 8, 2005; Eurocode 2, 2004). The investigation reached the following conclusions:

1. The seismic performance of bridges was found to be strongly influenced by the dynamic contribution of the unconventional abutment and the backfill soil. The high stiffness of the abutment was found to be the predominant and most significant structural element of the earthquake resisting system of the bridge. The two re-designed bridges (BM_1 and BM_2), which employed the new abutment, were found to be quite stiffer in comparison to the benchmark bridge (BM_0). The reductions were 23 and 34% in the longitudinal modal periods correspondingly.

2. The seismic participation of the restraining system and the embankment leads to significant reductions in the seismic movements of the re-designed bridge systems. The reductions mainly refer to the longitudinal response of the bridge. More specifically, the maximum longitudinal seismic movements of the deck were found to be reduced by up to 33%. The proposed abutment was found to be more efficient in bridge structures which respond with large seismic displacements, due to the increase in the seismic participation of the abutment.

Hence, bridges which are founded in areas with high seismicity can efficiently develop the proposed design alternative.

3. The reduction of the structural (initial) cost was found to be reduced up to 5% and 9% in the re-designed bridge systems BM_1 and BM_2 correspondingly. The re-designed bridge systems were also found to have lower maintenance costs that correspond to the cost for the replacement of the expendable elements (expansion joints, bearings and dampers). It follows that not only the initial, namely the structural, but also the final cost of the proposed re-designed bridges is lower, hence the utilization of the proposed abutment is a cost-effective design alternative, as both bridge alternatives found to have lower final costs (21% and 25% for BM_1 and BM_2 respectively).

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