The Inefficacy of Seismic Isolation in Bridges with Tall Piers

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SUMMARY:
Seismic isolation is widely used in contemporary bridge engineering. Typically, the bearings and dampers isolate and hence protect the bridge piers, by either reducing the seismic actions or through the increase in the damping of the structure. However, there are bridge design cases in which the seismic loading of piers is not reduced due to the use of seismic isolation. The last effect was observed in bridges with tall piers, in which the conventional design of the bearings over the piers are stiffer than the combined pier-foundation subsystem. In this case, seismic isolation is considered to be ineffective and superfluous and should be avoided in order to reduce the structural cost of the bridge. This can be achieved by connecting the tall piers with the deck through rotation free connections, such as fixed bearings. A simple rational, which identifies this effect, is proposed and validated with a real bridge.

Keywords: bridge; seismic; isolation; ineffectiveness; tall piers

1. INTRODUCTION

Seismic isolation has numerous applications the last three decades, especially in important structures such as bridges, which should maintain the emergency communications, with appropriate reliability, after the design seismic event (Eurocode 8-Part 2; 2005). Over the last 20 years, a lot of research has been conducted in order to raise the safety level, while keeping construction costs reasonable (Kunde et al., 2003).

One of the major goals of the seismic isolation is to shift the fundamental frequency of a structure away from the dominant frequencies of earthquake ground motion and fundamental frequency of the fixed base superstructure. The other purpose of isolation is to provide an additional means of energy dissipation, thereby reducing the transmitted acceleration of the deck that should be designed to avoid damage. On the other hand, bearings with relatively low shear stiffness are utilised to protect the end piers of bridges, which are typically the shortest and most vulnerable ones, against serviceability induced movements (Mitoulis[a] et al., 2010). Finally, the use of seismic isolation devices is inevitable and say one-way practice when the most common bridge construction methods are used that are either: (a) the construction of decks with precast and prestressed I-beams and (b) the construction of incrementally launched decks (Chen et al., 1999). These two construction methods result in bridge flexible earthquake resisting systems (ERS) with isolation as the rigid connection of the deck with piers has not established in common practice. Hence, isolation bearings are utilized for the seating of the deck regardless of the seismic loading of the piers.

Seismic isolation was found to have high impact not only on the initial but also on the final cost of bridges (Seidl et al., 2005). The bridge piers of isolated bridge have to remain essentially elastic (Eurocode 8-Part 2; 2005) during the design earthquake, as the hysteretic behaviour of the piers is not a possible mechanism that will dissipate part of the induced seismic energy. The seismic energy is required to be received and dissipated in the seismic isolation devices. Hence, the isolating system is required to provide increased reliability under severe earthquakes that raises significantly the cost of the isolation system. On the other hand, bearings, viscous dampers and expansion joints are considered to be expendable, which means that these devices have to be replaced after some years of bridge service (Australian Standard, 1999).

The selection of the seismic isolation system has a significant impact on the design of the piers. Design typically takes into account safety, serviceability and also esthetics and constructability (Mitoulis[b] et al. 2010). Additionally, the design of tall piers has to take into account possible second order effects and have to be designed against buckling (Eurocode 2 Part 2 section 4.3.5.6, 2004). Safety and serviceability usually influence the required capacity of the pier, in terms of cross sectional area, stiffness and reinforcements of the pier, while esthetics and local conditions (i.e. bridges crossing rivers) influence strongly the shape of the cross section, as the selection of piers' size must also satisfy architectural, and perhaps other requirements, which may govern design, also stated by AASHTO (Appendix B3, 2007). A major design selection that is strongly related to esthetic issues is the construction of all the piers with the same cross section geometry along the bridge, regardless of piers loading, while design should also maintain a proportion of dimensions between the deck profile and the piers. This means that, on the one hand, the piers' cross sections are at least preliminarily affected by the deck's depth, and on the other hand, piers may have a significant variability in their stiffnesses taking into account that they usually have different heights.

The last, say, esthetic restraint may lead to piers that attract most of the induced seismic loading (i.e. the squat columns), while the taller piers of the bridge may not require seismic protection, i.e. seismic isolation. In many bridge cases the mid-piers' requirements for flexural (i.e. longitudinal) reinforcement is typically lower than the minimum requirement of the codes (Tegos et. al, 2010), that is 1% (Eurocode 2 Part 1, 2004). This overdesign of the piers may also lead to an increase in the foundation size and cost (AASHTO, 2007). Hence, the mid-piers of isolated bridges with large vertical clearances (i.e. with tall piers) are developing only part of their elastic flexural capacity (at yield) and as a result seismic isolation seems to be superfluous, while their foundations are quite expensive. On the other hand, the isolation bearings, which are typically designed against seismic movements (Eurocode 8-Part 2; 2005), are overdesigned. This is due to the fact that the resulting seismic movements are quite large, due to the flexibility of the earthquake resisting system (ERS) of the bridge, despite the fact that the bearings over the mid-piers respond with relatively small movements, as the flexible mid-piers "follow" the movements of the deck.

However, the excessive displacements of the tall mid-piers and the required bearing sections could be effectively reduced in case the piers are connected with hinged connections to the deck. These connections can be longitudinal stoppers, seismic links (i.e. hinged steel rods) or fixed bearings (Eurocode 8-Part 2 section 6.6.2.1; 2005) and should allow the relative rotations of adjacent spans. Despite the fact that the connection of piers with the deck seems to be promising for future cost-effective design of bridges, no procedure for the selection of these piers is given in international literature. This paper proposes a simple procedure, based on a benchmark bridge of Egnatia Highway, which identifies the piers which do not require to be isolated from the moving deck. The procedure, which can be easily extended to include other bridge and pier types, can be useful for the cost-effective design of bridges with isolation devices.

2. THE BENCHMARK SEISMICALLY ISOLATED BRIDGE

The study utilised the bridge of Aliakmon-Kostarazi, which belongs to Egnatia Odos, as benchmark. The bridge, illustrated in Fig. 1 and 2, has a total length equal to 148.9 m. The two end spans have a length equal to 29.45 m, while the three intermediate spans are 30.0 m long. The deck is supported on the abutments and on the piers through 5 and 10 low damping rubber bearings correspondingly. The dimension of the bearings are Ø400x126(66) for the support of the deck on piers P_1,P_2, P_3, while bearings with dimensions Ø450x186(110) where used over the abutments and over pier P_4. The deck of the bridge consists of five prestressed and precast I-beams, precast slabs and cast-in-situ part of the slab. The deck has a total width equal to 13.45 m. The piers are circular with a diameter equal to 2.50 m and have heights H_{P1}=16m, H_{P2}=31m, H_{P3}=28m, H_{P4}=13m. The bridge piers are founded on ground type A (Eurocode 8 - Part 1, 2005), through 3 by 3 pile groups. The design ground acceleration was equal to 0.16 g. The importance factor adopted was equal to γ_I=1.0, while the behavior factors were equal to 1.0 for the longitudinal, the transverse and the vertical direction of the bridge. It is noted, that all the piers were found to have longitudinal and transverse reinforcement ratios equal to the minimum code requirement (Eurocode 2 Part 1, 2004), i.e. the longitudinal reinforcement is ρ_{min}=1%.

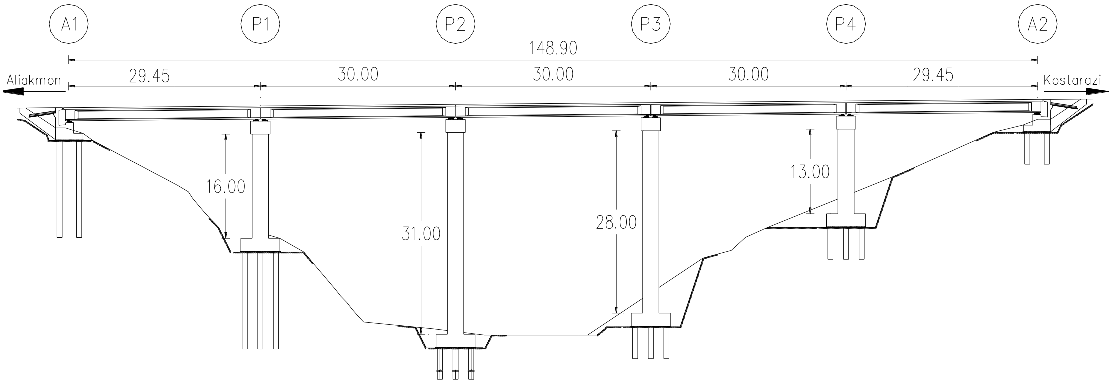

Figure 1. Longitudinal section of the benchmark bridge (Aliakmon-Kostarazi bridge of Egnatia Motorway)

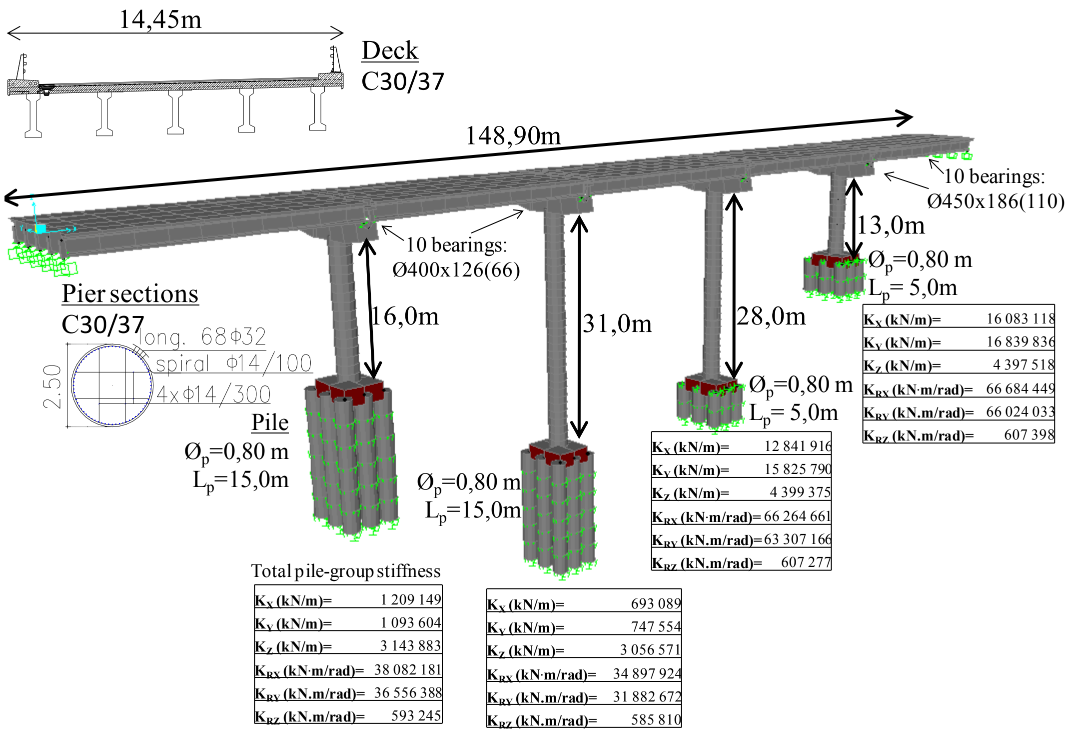

Figure 2. Three dimensional (3-D) model of the Aliakmon-Kostarazi bridge

3. IDENTIFICATION OF THE PROBLEM THROUGH A PARAMETRIC STUDY

The benchmark bridge was analysed for seven different pier heights (5, 10, 15, 20, 25, 30, 35 meters). Hence, a wide range of the ERS stiffness was investigated corresponding to either stiff piers, (i.e. H_P=5m) or stiff ones (i.e. H_P=35m). The response of the different bridge systems was analysed utilizing the modal response spectrum method and the elastic spectra of Eurocode 8 (Eurocode 8- Part 1, 2005). The study compared, for all different pier heights: (a) the eigenperiods and the modal shapes of the bridge systems, (b) the seismic loading of the piers (shear forces and bending moments at piers' base), (c) the displacements at the piers' top and the shear strains (movements) of the bearings. Then, (d) the elastic flexural capacity of the piers (yield moment), was estimated (Kappos, 2002) and was compared to the seismic bending moments at the piers' base. The objective of the above parametric analysis was, for the given isolation system, on the one hand to identify the development of piers' capacity in the elastic range and on the other hand to point out a specific pier stiffness, which is related to the pier's height for the given cross section, that develops effectively its elastic flexural capacity. The results and a simple procedure that indicates the piers that do not need to be isolated are given in the next section of the paper.

The above analyses were repeated for the benchmark bridge keeping the heights of the piers equal to the ones of the benchmark bridge. The benchmark bridge was analysed utilizing three (3) different bridge design concepts: (i) the initial isolated bridge concept, (ii) the use of isolated end-piers and the use of hinged pier-deck connections at bridge mid-supports and (iii) the use of hinged pier-deck connections at all bridge supports, namely at all deck-to-pier supports. The objective of this parametric study was to verify the successful prediction of the proposed procedure that is the identification of the piers which are not required to be isolated. It is pointed out that the procedure illustrates a cost-effective bridge design alternative with either isolated piers or hinged piers to deck connections, while keeping the piers loading smaller than their elastic flexural capacity. This design requirement is consistent with the initial design of the as-built isolated bridge that was for the bridge piers to remain essentially elastic after the design earthquake. Additionally, the piers had the minimum reinforcement ratio, i.e. ρ_{min}=1%. This means that an extra cost reduction would be possible if either the piers utilized higher reinforcement ratios (i.e. $\rho_{min} \leq \rho \leq \rho_{max}$=4%) that would increase their elastic flexural capacity (yield moment) or if the piers where designed to dissipate part of the induced seismic energy through hysteretic behavior, namely if the piers were allowed to exhibit inelastic behavior during an earthquake (Tegou et al., 2010).

4. RESULTS

4.1 Identification of piers that do not require to be isolated

Figure 3 illustrates the three important modal periods of the bridges with different pier heights ranging from 5 to 35 m. The figure shows that the fundamental period (1st mode) of the isolated bridge is up to 3 times the period of the bridge with the hinged pier-deck connections, when the piers are relatively short (i.e. 5 to 10 meters). However, this discrepancy between the fundamental periods seems to be effectively reduced for bridges with taller piers (i.e. when $H_p \geq 25$m). It follows that the influence of the seismic isolation is effectively reduced in bridges with tall piers. More specifically, the periods of bridges with isolated decks are 10%, 5% and 3% greater than the corresponding periods of bridges having the deck connected to all the piers with hinged connections, when the piers' heights are 25, 30 and 35 meters correspondingly. Hence, seismic isolation does not seem to govern the dynamic system of bridges with tall piers.

Figure 4 shows, for different pier heights, the ratios of the bending moments at the piers' base, which are developed during the design earthquake, to the corresponding flexural capacities, i.e. the bending moments at yield,. The discrepancies between the piers with the same heights are due to the different stiffnesses of their foundations, which are given in Fig. 2. Figure 4 shows that none of the piers utilizes its flexural capacity. Hence, the piers remain elastic, regardless of their height, as the most critical pier P_3 exhibits an overstrength up to 23% for all different bridge design cases, namely for different pier heights. It was also observed that the bridge piers with heights equal to or greater than 25 meters ($H_p \geq 25$m) utilize part -i.e. 77%- of their elastic capacity. Hence, seismic isolation of taller piers seems to be ineffective, namely the seismic loading of the piers is not influenced strongly by the selection of the seismic isolation system. This outcome also agrees with the findings of Fig. 3.

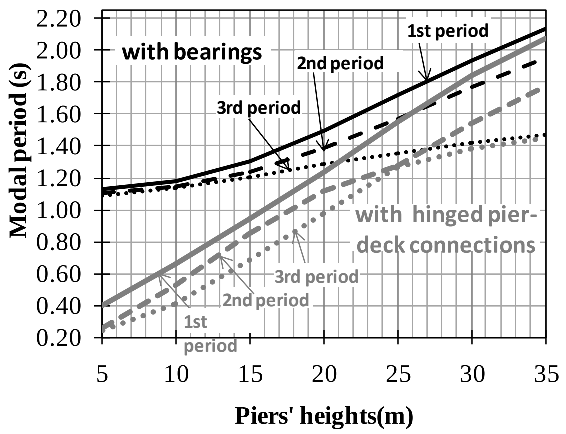

Figure 3. The three first eigenperiods of the isolated bridges and the bridges with hinged pier-deck connections for different pier heights

On the other hand the shear strains ε_s (Eurocode 8-Part 2; 2005) of the bearings are effectively reduced when the height of the piers is increased. Figure 5 shows that the bearings contribute from 30% to 50% to the flexibility of the bridge system with tall piers (i.e. $H_p \geq 25$m) as the seismic movement of the deck was found to be received mostly by the flexible piers. Therefore, the bearings' shear strains (movements) correspond to a small portion of the deck's movements despite the fact that the seismic movements of the deck were found to be increased when the heights of the piers were increased. This was found to be attributed to the fact that the tall, i.e. flexible, piers on the one hand "follow" the movements of the deck, while on the other hand the piers exhibit movements due to their inertial seismic loading (Mitoulis[a] et al., 2010).

The above finding is considered to be contradictable to the concept of seismic isolation of bridges. Seismic isolation is required to protect the piers, i.e. to reduce their seismic loading, which was not found to be significantly influenced by the seismic isolation system. On the other hand, the design of bearings, namely the selection of their movement capacity, is affected by the flexibility and the self-oscillation of the piers that are related to the elastic movements and the movements that are produced due to the inertial loading of the piers. Besides, the tall mid-piers were not found to require seismic isolation by the deck, as it was found that only a small portion of their capacity is developed during earthquake (see Fig. 4). Thus, the next step of the study was to connect the piers with the deck, through hinged connections, while preserving the final bridge design concept, which was to provide a bridge that responds essentially elastic during the design earthquake, i.e. a q-factor equal to 1 was used during the analysis. Hence, the piers' seismic bending loading was required to be smaller than their elastic flexural capacity (yield moment).

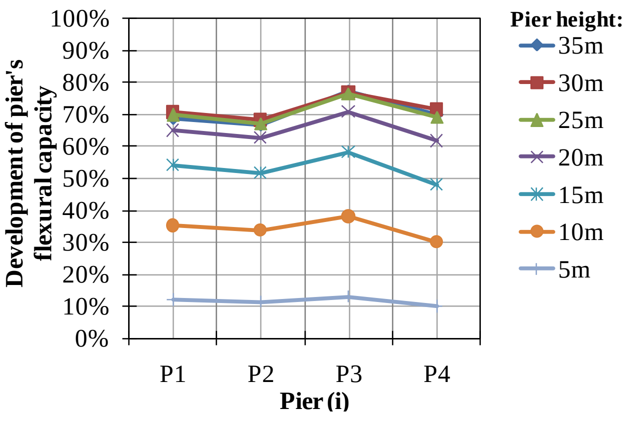

Figure 4. The development of the piers' flexural capacity for different pier heights (isolated bridges)

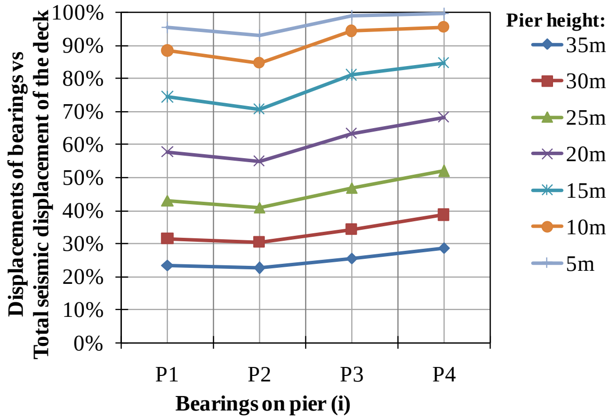

Figure 5. The displacements of the isolation system (isolated bridges)

4.2 Re-design bridges considering hinged pier-deck connections

The piers, which can be connected with the deck by hinged connections (i.e. seismic links or fixed bearings) as prescribed by Eurocode 8, cannot be selected at a preliminary design stage as the dynamic system of the bridge is altered due to this connections. In order to formulate a decision process, all bridge systems with different pier heights were re-analysed considering that all piers were connected to the deck with hinged connections. In Fig. 6 the bending moments at piers' base, which are developed during the design earthquake, are compared to the corresponding elastic flexural capacities (yield moments). It is observed that the piers with heights $H_p \geq 25$m remain elastic, i.e. their seismic loading in terms of bending moments is smaller than their elastic flexural capacity. It was also found that the proposed hinged pier to deck connections can be developed in the bridge with squat columns i.e. with pier heights H_P=5m. However, in this case the serviceability movements of the bridge deck would impose relatively large constraint movements and hence serviceability loading to the piers. The hinged pier-deck connections are not suggestive for bridges with squat columns. Finally, piers P_1 and P_2 did not develop their elastic flexural capacities in any case. This was found to be attributed to the fact that piers P_3 and P_4 have quite stiff foundations (see Fig. 2) and as a result these piers attracted most of the seismic loading of the bridge. Hence, the selection of the piers, which can be connected to the deck, was selected to be related to the stiffness of the foundation, the pier and the isolators on the top of the pier.

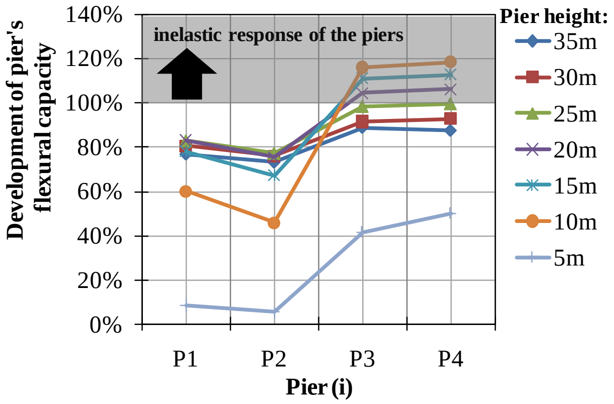

Figure 6. The development of the piers' flexural capacity for different pier heights (bridges with pier-deck connections)

5. A SIMPLE APPROACH TO IDENTIFY THE INEFFICACY OF SEISMIC ISOLATION IN ISOLATED BRIDGES

The above outcomes showed that there are isolated bridges whose piers do not require to be isolated due to their flexibility and relatively high inertial loading. In this section the development of a simple and sound rational is attempted. This rational aims at indicating the piers of an isolated bridge that do not require to be protected by the deck's seismic movements. The objective of this rational was: (a) to easily recognize the piers that can be connected to the deck with hinged connections and hence reduce the initial and final cost of the bridge, (b) to maintain the main design objective of the benchmark bridge, i.e. to ensure that the piers will respond in an elastic manner during the design earthquake and (c) to introduce a simple equation, that includes the three major flexibility parameters of the single-pier sub-system, which are the effective flexibility of all the bearings on the pier, the effective flexibility of the pier (Eurocode 8-Part 2; 2005) and the flexibility of the foundation, i.e. the translational and rotational flexibility of the pile-groups.

The procedure is based on the use of low damping rubber bearings (LDRBs), but is relatively easy to be extended and include bridges with other isolator types such as high damping, lead rubber or sliding bearings. Additionally, the rational is based on the calculations that were performed using the modal response spectrum method, which is a simplified method. The results will be validated with more rigorous non-linear dynamic time history analyses in the near future. The implementation of the procedure also requires that the bridge's seismic isolation system had been selected at least preliminarily according to a techno-economical methodology, which is described in detail by Manos et al. (2011).

Figure 7 illustrated the simplified foundation-pier-bearings response model that was used for the estimation of the combined single-pier stiffness ($K_{f\text{-}p\text{-}b}$). This stiffness is given by the following expression, which is an extension of the Eurocode's 8 (Part 2 section 7.5.4(3)) equation. Equation 1 includes an additional term, which corresponds to the translational stiffness of the foundation:

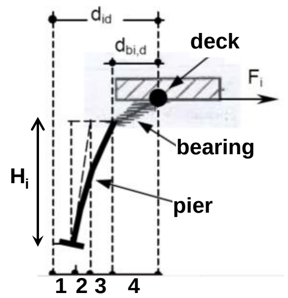

Figure 7. The total displacement of the deck according to Eurocode 8 Part 2:
The contribution of the foundation's (1) translation and (2) rotation,
(3) the elastic deformation of the pier and (4) the shear deformation of the bearings

$$\frac{1}{K_{f-p-b,i}} = \frac{1}{K_{ft,i}} + \frac{1}{K_{fr,i}} + \frac{1}{K_{p,i}} + \frac{1}{\sum_{1}^{n} K_{b,i}} \tag{1}$$

In Fig. 7 the total seismic displacement of the deck is distributed as follows: (1) is the translational displacement of the deck that equals $F_i/K_{ft,i}$, (2) is the displacement of the deck due to the rotation of the foundation that equals $F_i \cdot H_i{}^2{}_i/K_{fr,i}$, (3) is the displacement due to the elastic deformation of the pier ($F_i/K_{p,i}$) and (4) is the shear deformation of the bearings that equals $F_i/\Sigma K_{b,i}$.

The total stiffness of the foundation of pier i ($K_{f,i}$) that includes both its translational ($K_{ft,i}$) and the rotational ($K_{fr,i}$) flexibility is expressed by the following equation :

$$\frac{1}{K_{f,i}} = \frac{1}{K_{ft,i}} + \frac{1}{K_{fr,i}} \Leftrightarrow K_{f,i} = \frac{K_{ft,i} \cdot K_{fr,i}}{K_{ft,i} + K_{fr,i}} \tag{2}$$

The total foundation-pier stiffness ($K_{p\text{-}f,i}$) is given as follows:

$$\frac{1}{K_{f-p,i}} = \frac{1}{K_{f,i}} + \frac{1}{K_{p,i}} = \frac{1}{K_{ft,i}} + \frac{1}{K_{fr,i}} + \frac{1}{K_{p,i}} \Leftrightarrow K_{f-p,i} = \frac{\dfrac{K_{ft,i} \cdot K_{fr,i}}{K_{ft,i} + K_{fr,i}} \cdot K_{p,i}}{\dfrac{K_{ft,i} \cdot K_{fr,i}}{K_{ft,i} + K_{fr,i}} + K_{p,i}} \tag{3}$$

If the stiffness of one bearing on pier i is $K_{b,i}$ then the total stiffness of the **n** bearings on pier i is expressed by the following equation:

$$\sum_{1}^{n} K_{b,i} = n \cdot K_{b,i} \tag{4}$$

Figure 6 shows that pier P_4, which has a height equal to 25m develops almost 100% of its elastic flexural capacity during the design earthquake. In this case the stiffnesses of the foundation (translational and rotational) the pier and the bearings were respectively: $K_{ft,4} = 16{,}1 \cdot 10^6$kN/m, $K_{fr,4} = 66{,}0 \cdot 10^6$ kNm/rad, $K_{p,4} =$ 11780,97 kN/m and $\sum_{1}^{10} K_{b,4} =$ 26659,0 kN/m. Hence, $K_{f-p-b,4} =$ 7580,2 kN/m. It yields that the ratio of the combined foundation-pier stiffness to the total stiffness of the bearings is:

$$\frac{K_{f-p,4}}{\sum_{1}^{10} K_{b,4}} = 0{,}40 \tag{5}$$

Hence, the analysis showed that if the stiffness ratio S.R.:

$$\text{S.R.} = \frac{K_{f-p,i}}{\sum_{1}^{n} K_{b,i}} \le 0{,}40 \tag{6}$$

then it is more preferable to connect through a hinged connection (stoppers-seismic links, steel rods or fixed bearings) the piers with the deck, while if:

$$\text{S.R.} = \frac{K_{f-p,i}}{\sum_{1}^{n} K_{b,i}} > 0{,}40 \tag{7}$$

then seismic isolation should be utilized to protect the piers, which are designed to remain elastic and are reinforced with the minimum longitudinal reinforcement ratio. It is noted that the above procedure, which is quite simple, gives a clue in which piers the Bridge Engineer should pay more attention during design in order to avoid the use of expensive bearings on all the piers. The final design of the bridge should be performed after deciding on the piers that are connected to deck with hinged connections (i.e. fixed bearings) and the piers which are seismically isolated. In order to validate the above rational the benchmark bridge was analyzed for three different design cases, as described in the next section. It is finally noted that the upper and lower limits of S.R., which are expressed by Eq. 6 and 7, are different for different pier sections, i.e. other than circular sections.

6. IMPLEMENTATION OF THE SIMPLE APPROACH TO A REAL BRIDGE

The as-built bridge, illustrated in Fig. 1, has a seismic isolation system that consists of low damping rubber bearings (LDRBs). The seismic check of the piers showed that the piers' elastic flexural capacity is developed by 52% (pier P_2) to 77% (pier P_1), as shown in Fig. 8. The check of the stiffness ratios for all the piers $S.R._i$ yielded $S.R._1=1{,}25>0{,}40$, $S.R._2=0{,}19<0{,}40$, $S.R._3=0{,}28<0{,}40$ and $S.R._4=3{,}39>0{,}40$. Hence, piers P_2 and P_3 are proposed to be connected to the deck through hinged connections according to the rational described in section 5.

It was found that the connection of the two mid-piers (P_2 and P_3) with the deck through hinged connections did not alter the eigenperiods of the bridge significantly. The alterations were 0,5%, 3% and 0,5% for the 1st, 2nd and 3rd eigenperiod of the bridge, while the modal shapes remained almost the same. Hence, no additional seismic loading of the bridge was induced to the bridge when P_2 and P_3 were connected to the deck. The seismic check of the piers showed that the piers' elastic flexural capacity is developed by 61% (pier P_2) up to 85% (pier P_3), as shown in Fig. 8. The distribution of seismic actions seems to be almost uniform to the piers, while the bridge has a total of 20 bearings less (cost-effective design) than the initial isolated bridge.

The third bridge design case included the connection of all the piers with the deck through hinged connection. In this case the alterations were 17%, 29% and 17% for the 1st, 2nd and 3rd eigenperiod of the bridge. The seismic check of the piers showed that pier's P_4 loading is greater than the elastic flexural capacity by 62%, while pier P_2 developed its total elastic flexural capacity (see Fig. 8). Hence, the rational described above seems to predict successfully the piers that can be connected to the deck.

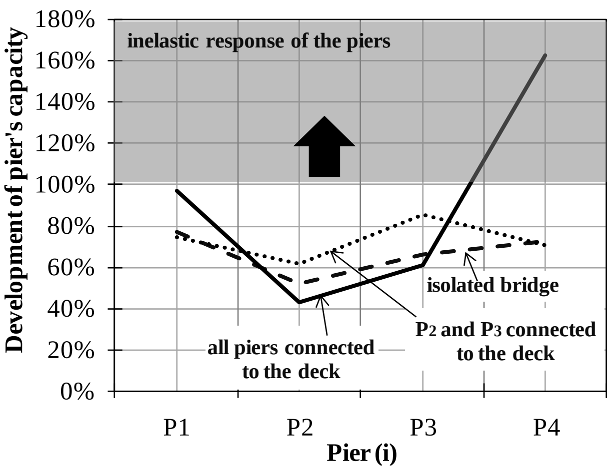

Figure 8. The development of the piers' flexural capacity for the different design cases of benchmark bridge

7. CONCLUSIVE REMARKS

A benchmark bridge with precast I-beams was used to illustrate the inefficacy of seismic isolation in tall bridge piers through a parametric study. Dynamic response spectrum analysis of stiff to flexible bridge earthquake resisting systems was performed for variable pier heights ranging from 5 to 35 meters. The seismic actions of bridge piers were compared to their corresponding elastic flexural capacities (yield moments) to determine which piers can receive additional seismic loading, while remaining elastic. A simple approach was developed and validated. This approach, which indicates the piers that may be connected to the deck through hinged connections, is expressed through a simple inequality. Finally, the approach and the inequality were verified through the analyses of three different bridge design cases based on the final design of the as-built bridge. The study demonstrated that:

In isolated bridges it is possible to have tall piers that do not require to be protected by the moving deck. The use of isolators over these piers was found to be ineffective and superfluous and should be avoided in order to reduce the initial and final cost of the bridge that is strongly influenced by the foundations and the isolation scheme. The last effect was found to be valid in bridges with tall piers, in which the current state of practice for the design of the seismic isolation results in bearings' total stiffnesses (ΣK_{bi}) greater than the one of the combined foundation-pier subsystem ($K_{f\text{-}p}$).

It is proposed that bridges, in which the use of seismic isolation is inevitable, i.e. in bridges with precast and prestressed I-beams or bridges that are being constructed using the incrementally launching method, should be checked for the above possibility. In that case, the tall piers should be connected to the deck with hinged connections (longitudinal stoppers, seismic links or fixed bearings) that are proposed by Eurocode 8 (Eurocode 8-Part 2; 2005).

A simple rational, that introduces a stiffness ratio S.R., which is the ratio of the combined foundation-stiffness of the pier to the total stiffness of the bearings over the pier, can be used in order to decide which piers should be connected to the deck in order to design a cost-effective bridge system. The simple rational for circular piers, being designed with the minimum longitudinal reinforcement ratio (1%), yielded S.R.$\leq$0,40, while the same rational can be utilized for other pier cross sections.

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