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UPLIFT OF ELASTOMERIC BEARINGS IN ISOLATED BRIDGES - A POSSIBLE MECHANISM: EFFECTS AND REMEDIATION

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ABSTRACT

Isolation devices are used to isolate bridge substructures from the forces associated with the inertia of the deck induced by earthquakes and from changes in the length of the deck due to temperature variation, creep and shrinkage. The safety of isolated bridges relies heavily on the response and the integrity of the supporting bearings. For "I" or "inverted T" section decks the bearings are normally set in one or two lines of support, which are parallel to the transverse axis of the pier cap. Connection details range from reliance on friction to resist lateral loads, various shear key details such as dowels or location in shallow recesses, to both ends anchored to the deck and pier cap using bolts. The latter detail is normally used for seismic isolation bridge bearings (see EN1337-3), but exposes the bearings to the possibility of tensile loading. Unseating prevention devices and restrainers are recommended in isolated bridges susceptible to such failure modes. During an earthquake, the longitudinal displacements of the deck induce rotations to the pier caps about a transverse axis, which in turn causes tensile and compressive displacements to the bearings. Although standards covering structural elastomeric bearings (EN1337-3, 2005) and anti-seismic devices (EN15129, 2009) require the structural engineer to predict or check the seismic loading on the bearings, the tensile displacements of the bearings, due to the pier cap rotations, have not been addressed in detail before in international literature.

An extended parametric study revealed that uplift of bearings may occur in isolated bridges, an effect that appears to be more pronounced for the bearings on shorter piers, probably because a major contribution to the longitudinal displacement is rotation of the piers on compliant foundations. Tensile displacements of bearings were found to be significantly increased when the isolators were eccentrically placed with respect to the axis of the pier, and when isolators having a low axial stiffness were used for the isolation of the bridge. Potential uplift effects should be taken into account during the design of the isolation system. The non-linear response and the correct design of the isolators against both the high horizontal and vertical displacements of current code designs must be assured to avoid bearing damage, rupture and correlated deck unseating mechanisms.

keywords: bridge; bearing; uplift; vertical; non-linear response; pier cap; rotation

INTRODUCTION

Elastomeric bearings, sliding bearings and a vast of isolation devices are extensively used in contemporary bridgeworks. A flexible interface, placed between the bridge superstructure and its supporting substructure, provides lengthening of the fundamental period of the bridge to reduce

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resonance effects, by providing a structural response that is away from the dominant frequencies of the motion. Certain devices offer supplementary damping to dissipate a useful amount of the damaging seismic energy and suppress build-up of resonant response. The need for the use of bearings became evident when serviceability loading of bridges due to thermal variations and concrete inherent creep and shrinkage deformations caused unexpected cracking to structural components. Bearings absorb these movements and protect the substructures against excessive serviceability loads and potential rotations of the deck (Aria, 2013). Thermal bearings used to have thick elastomer layers, whilst contemporary bearings have more layers of elastomer and the thickness of the layers is smaller. Obviously, the earthquake resisting system (ERS) of bridges with bearings relies heavily on the response of the bearings. As such, current code design provisions for isolated bridges (AASHTO 2010; Eurocode 8-2 2005, JRA 2007 and 2002) are placing emphasis on the reliability of the isolation system to ensure the integrity of the isolated bridge (NIST, 1996; Imbsen, 2007).

This paper addresses issues relating to the appropriate boundary conditions assumed for the bearings during design. Different choices introduced by the codes, illustrated in Figure 1, suggest that this issue deserves critical attention, and may be responsible for some aspects of damage that has been reported following earthquakes.

Amongst damaged bridges, bearing uplift and ruptures and the consequent span unseating as well as pounding effects were the most common failure modes after the 1971 San Fernando earthquake, the 1989 Loma Prieta earthquake (Priestly et al., 1996) and the 2011 Tohoku earthquake (Kawashima 2011; Kitahara et al., 2011; Buckle et al. 2011; EERI, 2011). Bridge bearings were not designed with seismic isolation in mind before the first two of these events. Subsequently, the option existed of designing them, as a key part of the seismic resistance strategy, to lengthen the period and increase the damping.

The span unseating and the bearing uplift concern are evident throughout most bridge design codes (CalTrans 1999; Eurocode 8-2 2005; JRA 2002). EN1337-3 section 8.2.1.2.7 (2005) and BS EN 15129 (2009) require that tensile stresses should not be greater than 2G, where G is the shear modulus measured at 100% strain, to avoid cavitation. AASHTO Methods A and B design of elastomeric bearing (AASHTO, 2013) require that no bearing uplift occurs due to the rotation of the bearing under seismic loads, despite the fact that the code seems not take into account the vertical response of the bearing, as shown in Figure 1. Similarly, Eurocode 8 (Eurocode 8-2 section 6.6.3.2, 2005) requires that no uplift of isolators is allowed under the design seismic combination. On the other side, Kelly et al. (2003a, 2007b) found that bearing uplift is not detrimental to bearing integrity as the interaction of shear-induced and tension-induced stresses is a factor that increases the capacity of the bearing, however the onset of cavitation in elastomers, volumetrically constrained by laminating with steel, occurs at low axial strains, of the order of 10^{-3}, which means that bearings show an abrupt drop in tensile stiffness at low strains. Indeed, Yang et al. (2010) found that elastomeric bearing response under tensile loading is expected to be non-linear. Hence, questions about the magnitude of bearings uplift displacements in bridges subjected to longitudinal seismic loadings arise, as these displacements may induce cavitation of the isolators under tensile loading that may affect the sound response and the integrity of the isolators and the bridge.

This paper studies an unrevealed uplift mechanism of elastomeric bearings. Uplift is originated by the seismic displacements of the deck in the longitudinal direction and the consequent rotation of the pier caps about its transverse axes. Simple calculations are first used to establish the credibility of such mechanisms, and the concern that inappropriate boundary conditions might have led to their occurrence. Next, the magnitude of bearing uplift displacements is estimated through an extended parametric study using 3D simplified numerical models. Emphasis is being placed on the non-linear response of elastomeric bearing in its vertical direction. Numerical data were used to validate the response of bearings under tensile stresses. The parametric analyses identified that bearings exhibit tensile displacements in most bridge models studied, while certain design parameters of a bridge earthquake resisting system amplify the uplift effect and hence increase the likelihood of bearing failures.

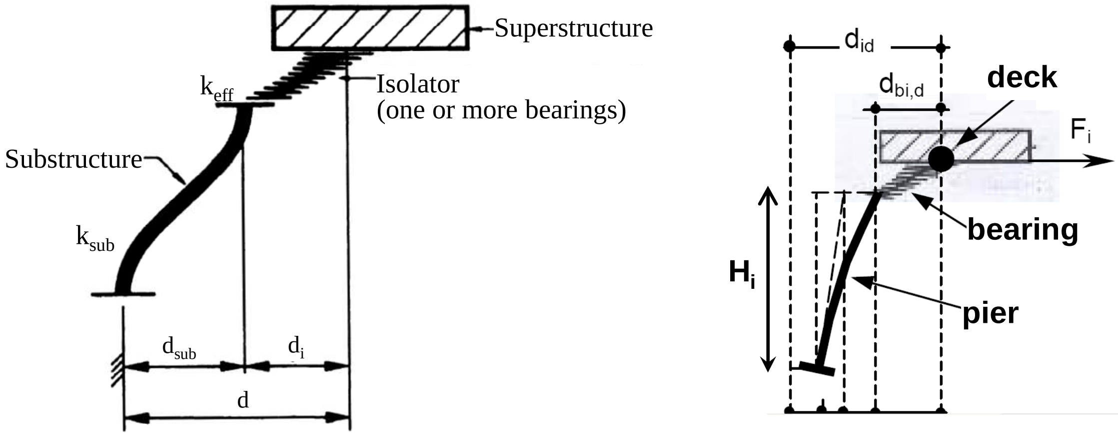

Figure 1. Models of an isolated bridge pier provided in two Standards (a) AASHTO (2010); (b)Eurocode 8 Part 2 (2005).

ESTIMATES OF BASIC MAGNITUDES GOVERNING BOUNDARY CONDITIONS AT THE INTERFACE BETWEEN PIER AND DECK

Take as an example the case of an RC pier, 2.5m in diameter, rising 16m above a strong foundation that resists rotation, as in Figure 1a. Ten laminated bearings, 400mm diameter and with 6 rubber layers, each 11mm thick, are placed on the pier cap and support the deck, which is assumed to be inflexible and thus constrains the top of the bearings to be horizontal, parallel to the pier foundation.

The laminated bearings provide an elastic joint between pier cap and deck, the shear stiffness of each one is given to a good approximation by:

$$K_s = \frac{G \cdot \pi \cdot a_b^{\ 2}}{n \cdot h} = 1.9\text{MN/m} \tag{1}$$

where G, the shear modulus of the rubber, has been taken as 1MPa, a_b = 0.2m is the radius of the rubber layers and h = 11mm is the thickness of each of the n layers.

The question arises as to through what angle the pier cap rotates when subjected to the horizontal force Q associated with a typical seismic deflection of the bearings. This force may be estimated, for the ten bearings together, as $Q = 10 \cdot G \cdot \gamma \cdot \pi \cdot a_b^2 \sim 1.6$MN where γ, the shear strain in the rubber, which has been taken as 1.28. The vertical load P on the pier is 10x750kN = 7.5MN.

The pier may be modelled using simple beam theory, with $E \approx 30$GPa, $I = \pi a_p^4/4 \approx 1.9\text{m}^4$. We shall consider first the implications of assuming the idealised boundary conditions depicted in Figure 1a. Assuming that the angular acceleration of the pier is negligible, a simple static argument shows that the couples applied at the ends of the pier are given by the following equation:

$$M = \frac{1}{2}(P \cdot d_p + Q \cdot L_p) = 12.9\text{MN} \cdot \text{m} \tag{2}$$

where $d_p = QL_p^3/(12EI) = 9.5$ mm is the lateral deflection of the pier under the load Q, and L_p = 16m is the pier length.

This couple must be transmitted from deck to pier cap by the bearings. Because the bearings are compliant, their deformation under the couple can be considered, and compared with the assumption of the boundary condition for the pier cap. The tilting stiffness of one bearing can be estimated as outlined by Gregory & Muhr (1995) as:

$$\frac{\pi \cdot a_b{}^2}{4 \cdot n \cdot h}\left(\frac{1}{2 \cdot G \cdot S^2} + \frac{3}{4 \cdot K}\right)^{-1} = 2.8\mathrm{MN} \cdot \mathrm{m/rad} \tag{3}$$

where $S = a_b/2h = 9.1$ is the "shape factor" and $K \sim 2000$MPa is the bulk modulus of the rubber. It follows that M would result, for a single line of ten bearings, in a tilt of 12.9MNm/(10x2.8MNm) = 0.46 radians. This is clearly inconsistent with the assumed boundary condition. On the other hand, if there are two lines of bearings 2.5m apart, M could be met by a reduction in load on one line, and an increase in load on the other, of magnitude 12.9MNm/2.5m = 5.16MN or 5.16MN/5 =1.05MN per bearing. The bearing vertical stiffness is estimated (Gregory & Muhr, 1995) as:

$$K_c = \frac{\pi \cdot a_b{}^2}{n \cdot h}\left(\frac{1}{6 \cdot G \cdot S^2} + \frac{4}{3 \cdot K}\right)^{-1} = 710\ \mathrm{MN/m} \tag{4}$$

so the deflection would be 1.05MN/0.71MN/mm = 1.48mm. This implies an angle of 1.48/1250 = $1.18\text{x}10^{-3}$ radians, again not consistent with the pier cap remaining horizontal, but it is not yet clear how near it might be to the other extreme of being free to rotate.

To check this last point, we may estimate the deflection of the pier with no constraint on its rotation at the pier cap as $d_p = QL_p{}^3/(3EI)$ = 38mm, and its angle of rotation as $QL_p{}^2/2EI \approx 3.58\text{x}10^{-3}$ rad. This rotation is three times higher than that estimated from the load fluctuations on the bearings, and suggests that the boundary conditions are nearer to Figure 1a, if we consider the deck and foundations to be rigid. The latter assumption, however, is not made in Eurocode 8, application of which led, in the numerical modelling described in the next section of this paper, to an assigned rotational stiffness of $38\text{x}10^6$ kNm/rad for the rotational stiffness of the pier/foundation interface. This would result in a rigid body rotation of our 16m long pier, subjected to a horizontal force of 1.6MN at its cap, of 6.74×10^{-4} radians. Note that such a rotation implies a horizontal movement of the pier cap by 16m x 6.74×10^{-4} = 10.8mm, of similar magnitude to its deflection as a clamped-clamped column.

The static compression of each bearing is estimated to be 750kN/(710kN/mm) = 1.06mm, so that in either simplified case for a rigid foundation discussed above the pier cap rotation is sufficient to relieve the load on one line of bearings, and create a tension if they are bolted rather than dowelled or located in recesses. If we allow for the rotational compliance of the foundation, this effect is increased.

Finally, a more accurate view of the deck of the bridge being discussed in this paper is that it consists of spans that are relatively rigid, each end resting on a line of 5 bearings, with a short concrete slab spanning the gap. It seems that this slab could bend, permitting differential vertical movement of the deck ends on the two lines of bearings. The slab is around 350mm thick, of width w = 13.45m and perhaps of length L_s = 1m. To estimate its deflection d_s under the lateral loading, 5.16MN, being the seismic vertical load change on one line of bearings, we may use the formula for a simple beam, this time of rectangular cross-section of width w and thickness t, with clamped-clamped ends: $I = wt^3/12 = 0.048\text{m}^4$; $d_s = 5.16\text{MN}.L_s{}^3/(12EI)$ = 0.3mm. Since often the slab joining deck spans will normally be less than 1m, it is concluded that the slab may be sufficiently stiff to permit one row of bearings to be substantially or completely unloaded.

In summary, if a sufficient horizontal force develops to shear the bearings by 128%, equivalent to a deflection of $1.28 \cdot n \cdot h$ = 84mm, then the pier cap movement will be at least 10.8mm due to rotational compliance of the pier foundation plus about 9.5mm due to flexibility of the pier (assuming clamped-clamped boundary conditions). If the bearings are in one line, the boundary condition of the pier-deck connection will resemble a pin joint and the pier deflection will be over three times larger, and the bearings will suffer a very high rotation. On the other hand, if the bearings are in two lines, around 2.5m apart, the pier will be nearer to a clamped condition, but each line of bearings will be subjected to uplift on alternate cycles. In these calculations the value for Young's modulus E of concrete was taken as 30GPa, appropriate to the uncracked condition. In the cracked condition, the value of E may be reduced to 10GPA, so the deflection of the pier could lie between about 28.5 and 114mm. Adding the displacement of 10.8mm of the pier cap due to rigid body rotation on the foundations, we see that the maximum seismic horizontal displacement of the pier cap could be comparable to the bearing deflection, taken as 84mm for these calculations.

ANALYTICAL RESPONSE OF A REAL BRIDGE ACCOUNTING FOR THE VERTICAL BEARING RESPONSE

Extensive parametric numerical modelling was conducted to quantify the tensile loading of the elastomeric bearings. The study utilized an isolated bridge shown in Figure 2 and 3a. A partially precast deck, with prestressed beams and cast in situ slab was employed for the construction of the bridge. The total length of the bridge is 148.9 meters. The deck is supported on the abutments and on the piers through five and ten low damping rubber bearings correspondingly. The piers are single-column circular sections and the diameter of the column is 2.50 meters. The dimension of the bearings B_1 at piers 1, 2, 3 are Ø400x126(66), where the dimensions are in mm and that in brackets indicates the total thickness of the rubber layers, while bearings B_2 of Ø450x186(110) were used for the support of the deck at the abutments and at pier 4. The width of the deck is 13.45 meters. The ground type is B according to Eurocode 8-1 (2005), and the foundation comprised of 3 by 3 pile groups. The translational stiffness of the foundation of piers 1, 2, 3 and 4 was $1.21 \cdot 10^6$, $0.69 \cdot 10^6$, $12.84 \cdot 10^6$ and $16.08 \cdot 10^6$kN/m respectively and the rotational stiffness of the foundation was $36.56 \cdot 10^6$, $31.88 \cdot 10^6$, $63.31 \cdot 10^6$ and $66.02 \cdot 10^6$ kN·m/rad correspondingly. The design ground acceleration was equal to 0.16g.

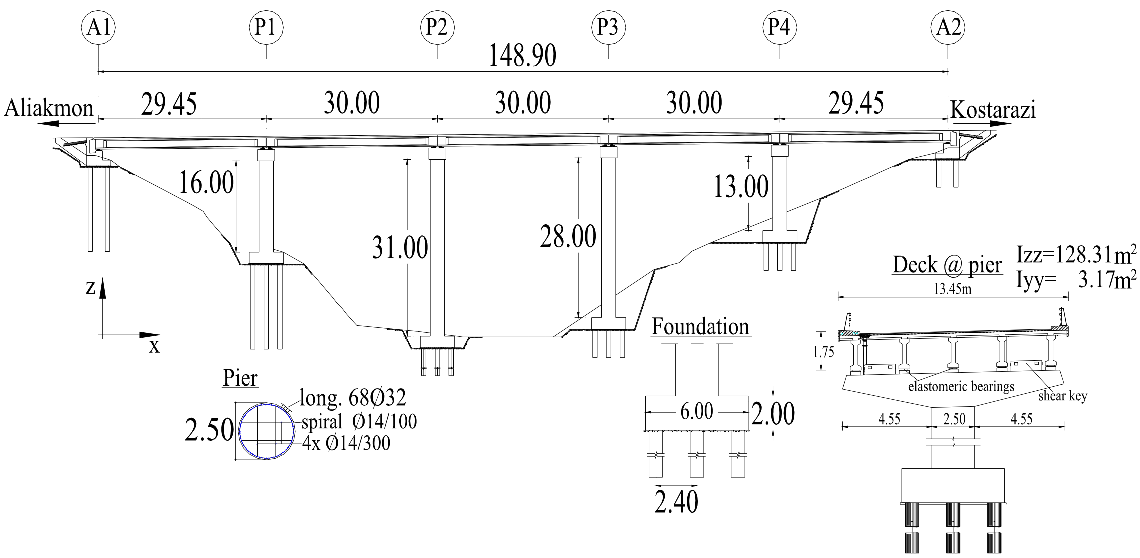

Figure 2. The geometry of the benchmark bridge and cross sections of the pier, the foundation and the deck.

Emphasis was placed on the modeling of the vertical response of the bearings, namely the tensile and compressive stiffness of the isolators and on the eccentricity e of the bearings with respect to the axis of the pier. The analysed bridge models were chosen to reflect potential alternative designs that a Bridge Engineer may adopt. Two simplified stick models of the bridge were analysed. The Typical Design model (TD) neglected the non-linear response of the bearings and their eccentricity with respect to the axis of the pier; thus treating the bearings as if placed in a single central line on the pier cap, and the piers as cantilevers. The Benchmark Bridge (BB) model accounted for boththe non-linear response of the bearings and their eccentricities.

Modeling of the bearing for the BB case took into account the shear and the rotational stiffnesses according to the Naeim and Kelly (1999) bearing model, as shown in Figures 3b and 4. The figure shows the elastic K_1, the post-elastic K_2 and the effective K_{eff} shear stiffness of the bearing model. The bearings of the benchmark bridge are low damping elastomeric bearings with a shear modulus 1 MPa. Damping was taken as 5% of the critical one, while post elastic shear stiffness was taken to be 50% of the elastic one. The effective shear stiffnesses of B_1 and B_2 were estimated to be 1713.60 kN/m and 1301.26 kN/m respectively. The rotational stiffnesses of the bearings were 998.4kN·m/rad and 1214.4kN·m/rad correspondingly. The vertical stiffnesses of the bearings under compression (K_c) were found to be 303470.37 kN/m for B_1 and 291660.51 kN/m for B_2 (Naeim et al, 1999). Modeling of the resistance of the bearings against tension was based on the research of Yang et al. (2010). The elastomeric bearings were considered to cavitate at a tensile stress equal to 2MPa and a tensile strain equal to 3%. The stiffness values K_{t2}, i.e. the axial tensile stiffness of the bearing after the onset of cavitation, were taken equal to 6% of the initial elastic ones (K_{t1}) according to the v parameter described by Yang et al. (2010). The bearings were set in two lines parallel to the transverse axis of the pier cap. Both bearing lines 1 and 2 have a longitudinal distance e with respect to the axis of the pier, as shown in Figure 3.

The TD model used a linear model for the vertical stiffness of the bearing, i.e. stiffness of the bearings under tension equal to the compressive one, and small eccentricities e, as provided by AASHTO (2010) and Eurocode 8 Part 2. The TD model used bearing shear and rotation stiffness values equal to the ones of the benchmark bridge BB bridge. Modeling of isolated bridges based on the TD case described above has been adopted for the analysis of bridges by many researchers (Kappos et al. 2012; Constantinou 2011; Mwafy et al. 2007; Hindi et al. 2006; Dicleli 2006).

All the analyses used simplified 3-D stick models, that followed the guidelines of Kappos et al., (2012, chapter 12). The post-elastic stiffness of the piers was assumed to be 2% of its initial elastic value. The flexibility of the foundation was taken into account by assigning linear and rotational springs in the two horizontal and in the vertical direction of the foundation. The soil spring values were obtained by the geotechnical in-situ tests of the as-built bridge. The bridges models were subjected to longitudinal seismic motion. Artificial accelerograms that were compatible to ground types B and C-dependent Eurocode 8-1 (2004) elastic spectra were used, while the accelerograms were scaled to three levels of seismic actions with peak ground accelerations (PGA): 0.25g, 0.5g and 0.75g. The non-linear response of the bridge models was analysed using the FEM code SAP 2000 ver. 14.2.0 (Computers and Structures, 2010). Dynamic non-linear time history analysis was implemented and the average acceleration method Newmark was chosen (Chopra, 1995).

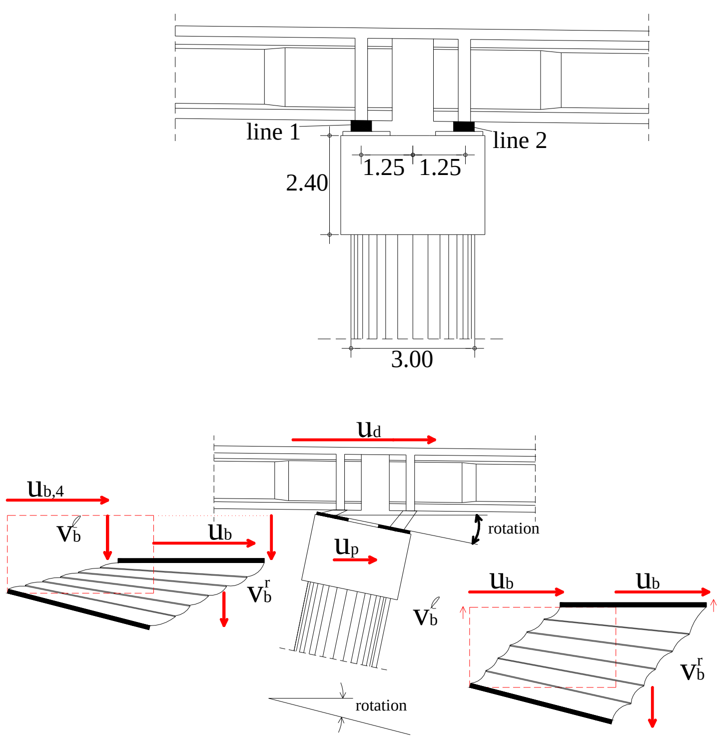

Figure 3. Top figure: the geometry of the pier head and the two lines of support of bearings and bottom figure: the bearings on the left line of support are under compression and the ones on the right are under tension.

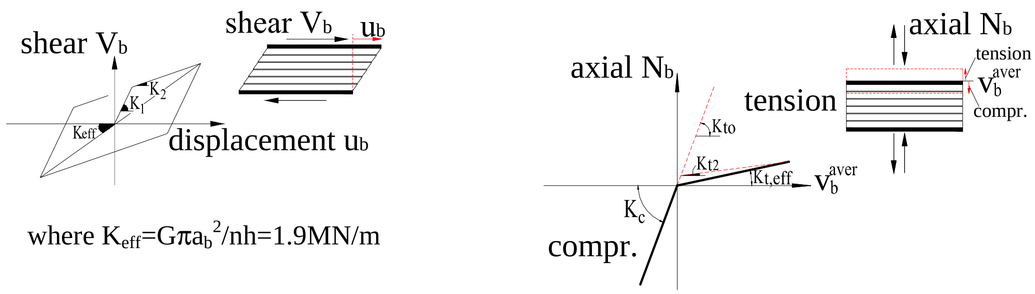

Figure 4. On the left figure: the response of the bearing under shear deflections; on the right: the response of the bearing under axial deflection.

DESCRIPTION OF THE ELASTOMERIC BEARING UPLIFT MECHANISM

The benchmark bridge BB was subjected to longitudinal seismic actions. Figure 5 shows the time histories of the longitudinal deck movements, when the bridge was subjected to a ground motion of a PGA equal to 0.50g. The maximum horizontal displacement of the deck along the +x axis is almost 460mm, while the one along the -x direction is 410mm. These displacements were found to induce rotations to the pier cap of pier 4 equal to $25\cdot10^{-3}$rads (clockwise) and $22\cdot10^{-3}$rad (counter-clockwise) respectively. The corresponding rotations of the deck were found to be almost three times smaller than the ones measured at the pier heads, indicating that the governing rotation for the tension of the bearings is the one induced by the piers, at least for the response of the bridge subjected to longitudinal seismic action.

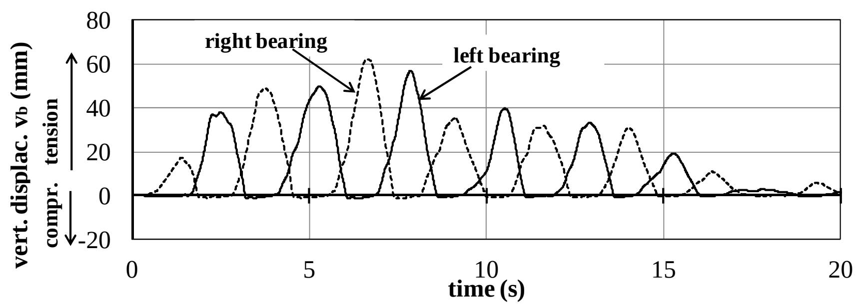

Figure 5. The response of the bearing under axial deflection.

Figure 5 illustrates the time history of the vertical deck displacements caused by the rotation of the pier 4 cap beam (dashed line). Deck movements due to the vertical component of the seismic action (continuous line) were also calculated. The clockwise rotation of the pier cap and the upward movement of the deck induce a tensile deformation of the bearings at the right hand line of support equal to 61mm. The opposite was found to be valid for the counter-clockwise rotation of the pier cap. When the deck exhibits its maximum negative displacement along -x axis, the bearings at line 1 were found to be subjected to a 54mm tensile displacement. The compressive deformation of the bearings due to quasi-permanent vertical loads was removed from all the results illustrated in the study, such that the initial vertical deformation of the bearings, which is negative (downwards), was set equal to zero, i.e. the bearing vertical displacements was measured from the deck dead load datum. Maximum tensile displacements of bearings at line 1 (61mm) and line 2 (54mm) correspond to axial tensile strains 56% and 49% that are greater than the tensile strain at the onset of cavitation, which is of the order of 3%. Figure 6 shows the shear displacements of the bearings when subjected to a longitudinal seismic action corresponding to a PGA equal to 0.50, whilst Figure 6b shows the vertical tensile displacements of the bearings. It is evident that the bearings above the abutments are subjected to large shear displacements, while the shear strains of the bearings above the piers are smaller due to the fact that the piers rotate and deform under the shear actions transmitted by the bearings and as such undertake part of the seismic displacement of the deck through its deflections. On the right figure, it is evident that the bearings on the shorter piers 2 and 4 are exhibiting larger vertical tensile movements.

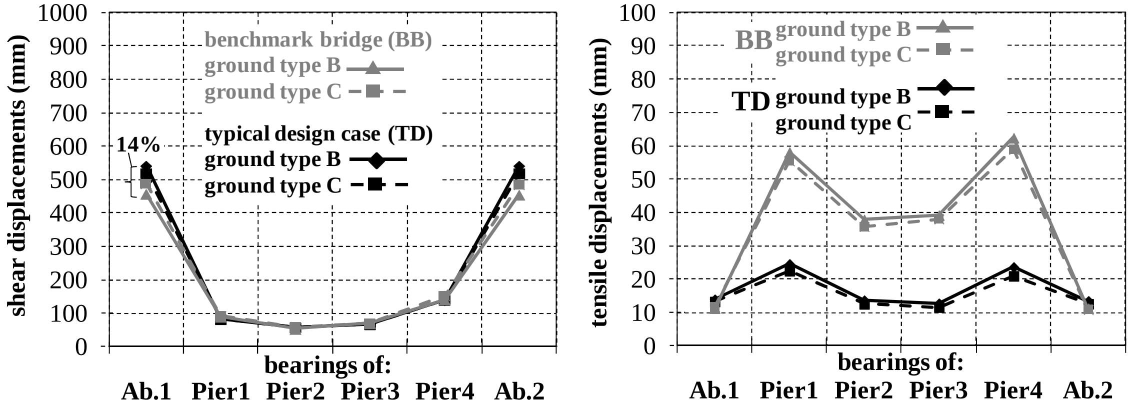

Figure 6. (a) The maximum shear displacements of the BB and the TD bearings at abutment 1, piers 1, 2, 3, 4, abutment 2 and (b) the maximum tensile displacements of the bearings (PGA 0.5g).

A THREE DIMENSIONAL REPRESENTATION OF THE PIER-BEARINGS-DECK SEISMIC RESPONSE

The maximum measured seismic displacement of the bearings shown at Figure 6 corresponds to a shear strain of ε_s=1.28 (i.e. 128%). To validate the effect of the bearing tension (right support) and compression (left support) and to measure the stresses developed at both support lines, i.e. left and right hand side supports, a three dimensional model of the pier-bearing and deck connection was developed in SAP 2000, as shown in Figure 7. The deck and the pier were modelled with beam elements, while a detailed modelling was used for the bearings with layers of steel and elastomer. Solid elements were used in that case. The thicknesses of the steel and the elastomer layers were 4mm and 11mm respectively this gives the right height only if there is only one end with an anchor plate: $1 \cdot 20+5 \cdot 4+6 \cdot 11$ = 126mm. The anchor plates of the bearings had a thickness equal to 20mm. The Ø400x126(66) and the first pier (height 16meters) were modelled in that case. The material properties were compatible to the materials used for the construction of the bearing: the modulus of elasticity E for the elastomer was taken 3Mpa and the corresponding G modulus 1Mpa, i.e. a Poisson ratio equal to 0.49 was considered. The steel plating has a modulus of elasticity equal to 200Gpa. Figure 8 shows how the bearings were connected to the deck and the piers, the cross sections of the piers and the deck and the attempted modelling of the circular bearings with solid elements.

The analysis of the sub-structure (pier-bearings-deck) described above was conducted for a horizontal longitudinal movement that caused shearing of the bearings of ε_s=1.28, i.e. a relative displacement of the bearing plates equal to 1.28 times the elastomer total thickness (66mm) i.e. a displacement equal to 84.5mm. At this stage linear elastic analysis of the effect was considered to be adequate, i.e. the materials were considered to respond in an elastic manner under the prescribed loads. The results of the analysis are quantified in Figure 8. The figure shows that indeed, the displacement of the bearings along the longitudinal direction caused a movement of the pier head (on the right), rotation of the pier head (clockwise) and shear deformation of the bearings. At the same time the bearings on the right-hand side are under tension while the bearings on the left line of support are under compressive loads. The analysis showed that if the bearings remained elastic the maximum tensile stress of the bearings (on the right) is 15Mpa, while the max tensile stress of 15Mpa is observed below the anchor plate of the bearing. At the middle of the height of the bearing three different stresses were measured i.e. 8.2, 10.5 and 13Mpa, showing that indeed the bearings plates not only are drawn away from each other (tensile deformation of the bearing), but there is also a relative rotation that is being developed between these plates. This rotation increases the right hand tensile stresses.

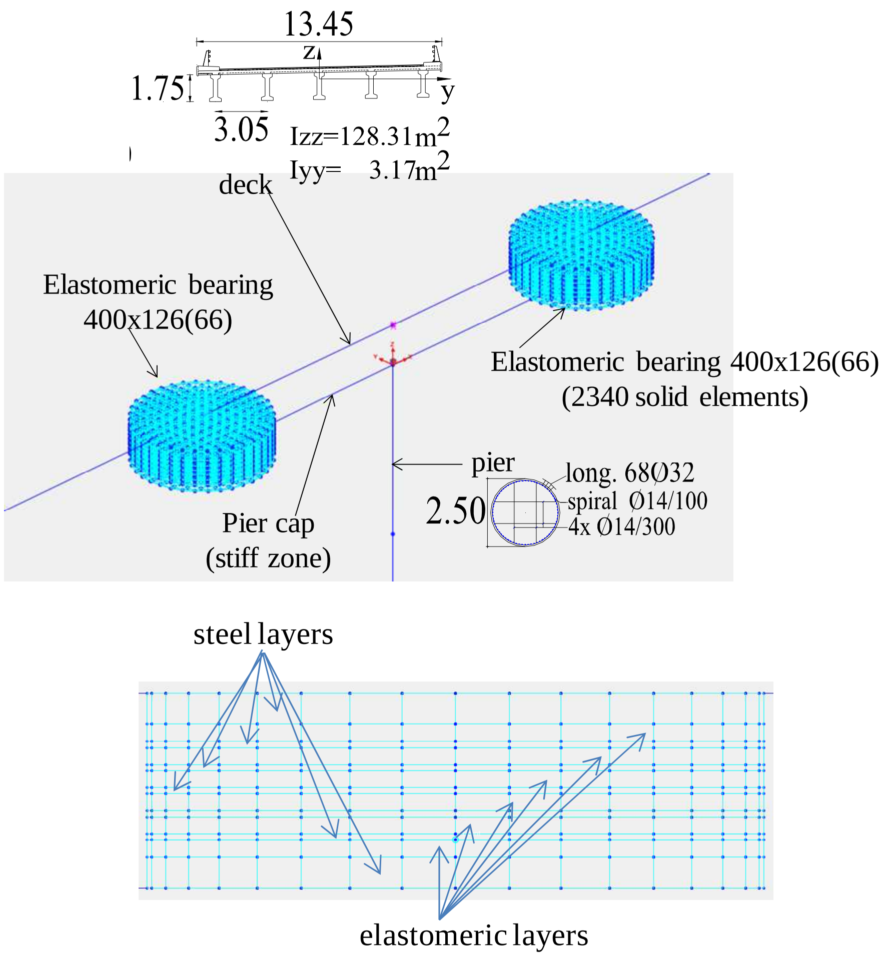

Figure 7. Top figure: A three dimensional representation of the pier - pier cap – bearings - deck connection and bottom figure a cross section of the 3D bearing model.

The opposite is valid for the bearings on the left hand line of support. More specifically, the bearings on the left hand support are under compressive loads varying between 7.8Mpa at the mid-height of their right edge up to 15.4Mpa below the anchor plate. Again, compressive load varies along a horizontal section of the bearing as the compressive stress is 13.8Mpa (value at the left edge of the isolator), 10.5Mpa (mid-point) and 7.8Mpa (right edge) along the bearing mid-height section, indicating that the anchor plates are approaching each other, whilst the plates are exhibiting a relative rotation.

The result of the described effect of the bearings under tension (right hand support) and compression (left hand support) creates bending moments to the piers. It was found that the piers respond as fixed-end beams with the following bending moments 9975kN·m (top) and 15600kN·m. Hence, the bearings axial stiffness provide a 63% fixity of the pier cap, despite the fact that in most cases the piers of seismically isolated bridges are perceived as cantilevers when subjected to seismic actions along the longitudinal direction of the bridge.

Finally, it is worth mentioning that the above analysis did not take into account potential overturning effects that can be developed due to the rigid body rotation of the pier due to rotations of the foundation or due to the development of a hinge at bridge piers. In that case, the tensile movements of the bearings are expected to be drastically increased.

REMEDIATION

Because of the concern regarding cavitation of the rubber and rupture of the rubber-metal bond, the first laminated bearings used dowelled or recess connection details. This was especially true for seismic isolation bearings (Muhr, 1994). However, as rubber bearings became more widely used, and starting with lead-plug laminated rubber isolators, there has been a trend towards bolted connection details. The probable driving force for this trend is the subordination of concerns about the rubber, from the pioneers who developed the technology, to the feeling of greater confidence commanded by bolted connections among the engineering community less familiar with rubber. The work highlighted in this paper suggests that this trend merits exposure to more critical discussion. It has, however, been shown earlier by one of the authors (Muhr,1994) that recessed connection details, in particular, are capable of significant uplift combined with shear without loss of the shear key provided by location in the recess. The consequence of this uplift is that the shear force falls below that calculated for a bearing with fixed boundaries, since the uplift involves rotation of the bearing, but this will improve isolation. Similarly, the vertical force remains compressive, albeit very small. Several isolated buildings have this connection detail, so in the fullness of time we may have field data showing how they perform in earthquakes. It is worth considering whether, in the meantime, future use of such connection details would be appropriate for bridges.

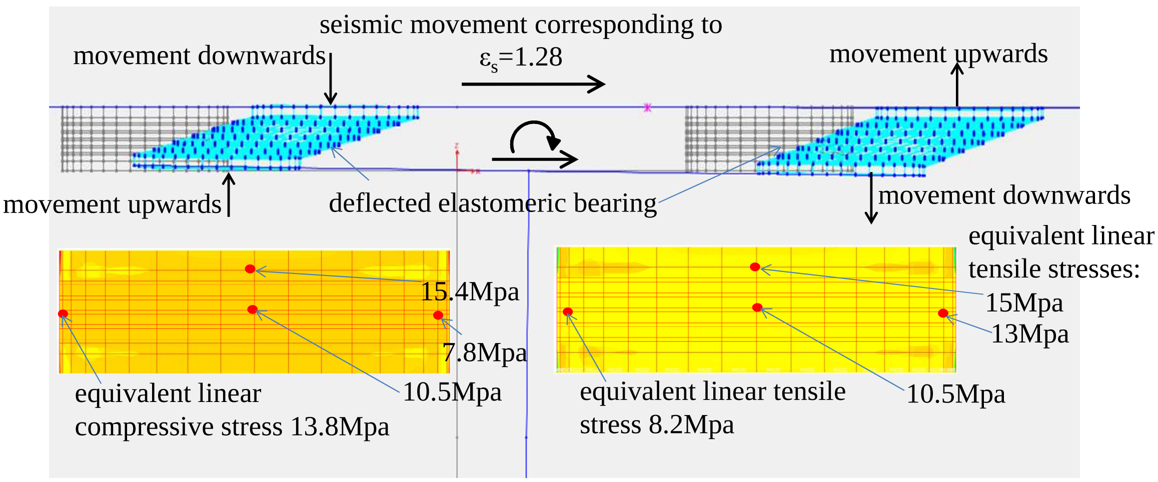

Figure 8. The tensile stress developed at the bearing due to ε_s=128%.

CONCLUSIONS

An uplift mechanism of anchored elastomeric bearings placed at two lines of support was described for the first time. The described detail of the pier-bearing-deck support is common in precast bridge construction methods, where seating of precast segments is provided for the precast segments on two lines of support. Estimates of basic magnitudes governing boundary conditions at the interface between pier and deck were given showing that the bearings might be under tensile stresses when the deck moves in the longitudinal direction of the bridge. Stick models of the bridge and a 3D modeling of the pier-bearings-deck connection was attempted considering typical bearing placement with two lines of support being parallel to the transverse direction of the pier cap. The bearings had eccentricity e with respect to the axis of the pier. The analysis showed that the isolators receive compressive and significant tensile (uplift) displacements, due to the rotations of the pier caps about a transverse axis. The magnitude of the bearing uplift displacements and its influence on the response of bridges was studied by comparing the response of a benchmark bridge (BB) to that of a bridge corresponding to the typical design (TD) case, which considered that the piers responded as cantilevers, i.e. the bearings are not restraining the rotations of the pier caps. The parametric study identified the influence of specific bridge design parameters on the predicted tensile displacements of the isolators. The study came to the following conclusions:

The longitudinal displacement of the deck induces displacements to the short piers (height 16 meters), primarily the consequence of rotation about their compliant foundations, which in-turn induce rotations of the pier caps. The pier cap rotations result in additional compressive and tensile strains imposed to the bearings placed at two lines of supports. For a seismic motion that causes a shear strain of the bearings equal to 1.28 induces a rotation of the pier cap equal to 1.18×10^{-3} radians up to 3.58×10^{-3} radians considering either a clamped-clamped ends pier or a cantilever correspondingly. These rotations of the pier cap induce a vertical tensile displacement of the bearings, despite the vertical load of the deck, which for the line of bearings that go into tension must be supported by the slab cast between deck segments..

The three dimensional model of the pier-bearing-deck connection considered a horizontal displacement of the deck that caused a shear deformation of the bearings equal to 1.28. Again, the bearings were found to receive tensile stresses. The equivalent elastic stresses of the elastomer layers, were found to be much larger than the tensile stress that can cause cavitation of the bearings, i.e. much larger than 2Mpa. Hence, non-linear response of the bearings that accounts for the non-linear and post-elastic behaviour of the bearings should be taken into account to calculate correctly the vertical displacements of the bearings and the potential of bearing cavitation and potential ruptures under earthquake excitations.

The simplified stick models of a typical bridge with precast I-beams were analysed considering two different models. The first model (TD) neglected the non-linear response of the bearings and the eccentricity of the bearings with respect the axis of the pier. The second model accounted for both the non-linear response of the bearings and their eccentricities. It was found that the bearings experience tensile movements that are strongly dependent upon the magnitude of the seismic motion and the magnitude of the eccentricity of the supports. For a ground acceleration 0.50g the tensile movements of the bearings, which are expected to cavitate after the tensile stress of 2Mpa, whilst being unable to restrain the pier rotation, can go up to 60 mm, that corresponds to a tensile strain equal to 90%. It is noted that the above tensile strain was calculated for a pier height equal to 16 meters and for a seismic action that caused a shear strain equal to 128% to the bearings. The effect of the bearings tensile stress seems to be more critical for the bearings placed on top of the shorter piers.

Further analysis is required to identify the influence of the stiffness of the foundation and the vertical stiffness and modeling of the bearings.

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