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SEISMIC PERFORMANCE COMPARISON OF TWO ROCKING ISOLATION ALTERNATIVES FOR AN OVERPASS BRIDGE

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ABSTRACT

Rocking isolation of structures is evolving as an alternative design concept in earthquake engineering. The present paper investigates the seismic performance of an actual overpass bridge of the Attiki Odos motorway (Athens, Greece), employing two different concepts of rocking isolation: (a) rocking of the piers on the foundation (rocking piers); and (b) rocking of the pier and foundation assembly (rocking footings) on the soil. The examined bridge is an asymmetric 5-span system having a continuous deck and founded on surface foundations on a deep clay layer. The seismic performance of the two rocking isolated bridges is compared to that of the existing bridge, which is conventionally designed according to current seismic design codes. To that end, 3D numerical models of the bridge–foundation–abutment–soil system are developed, and both static pushover and nonlinear dynamic time history analyses are performed. For the latter, an ensemble of 20 records (10 ground motions of 2 perpendicular components each) that exceed the design level are selected. The conventional system collapses in 5/10 of the (intentionally severe) examined seismic excitations. The rocking piers design alternative survives in 8/10 of the cases examined, with negligible residual deformations. The safety margins of the rocking footings design concept are even larger, as it survives in all examined cases. Both rocking isolation concepts are proven to offer increased levels of seismic resilience, reducing the probability of collapse and the degree of structural damage. Nevertheless, in the rocking piers design alternative high stress concentrations at the rotation pole (pier base) are developed, indicating the need for a special design of the pier ends. This is not the case for the rocking footings concept, which however is subject to increased residual settlements but no residual rotations.

Keywords: Rocking Bridges; Soil Structure Interaction; Uplifting Structures; Bridge Engineering

1. INTRODUCTION

Motivated by the exceptional behavior of tall, slender and seemingly unstable structures (such as water-tanks and tombstones) during the 1960 Chilean Earthquake, Housner published his seminal paper (Housner 1963) where he explained the dynamic stability of rocking structures. In the same earthquake, other seemingly more stable structures, such as buildings and bridges, collapsed. Since then, the rocking motion has been extensively studied using conceptual models (Yim et al. 1980; Zhang & Markis 2001, Dimitrakopoulos & DeJong 2012 Bachmann et al. 2017 among others) to conclude that large slender structures present remarkable stability against earthquakes.

This superior performance of rocking structures has led researchers to propose rocking as an earthquake hazard mitigation technique. According to the concept of “rocking isolation”, instead of trying to fix the structure firmly to the ground, uplifting and rocking is allowed. Such uplifting acts as a mechanical fuse, limiting the forces transmitted to the structure. Rocking isolation aims to increase

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the safety of a structure against collapse, while simultaneously decreasing the residual displacements. Therefore, rocking isolation enables resilient structures. Additionally, rocking isolation often emerges as a cost-effective approach for the seismic upgrading of existing bridges that do not meet current seismic design code provisions resulting in poor seismic performance, or when the decks of existing bridges need to be widened leading to an increase of the corresponding inertia forces.

2. CONCEPTS FOR ROCKING ISOLATION OF BRIDGES

The scope of the present study is to comparatively assess the performance of two rocking isolation concepts for a typical motorway bridge, comparing them to each other, but also to a bridge conventionally designed according to current seismic design codes (Figure 1a). In the field of rocking isolation of bridges, two approaches can be found in the relatively recent literature. The first one refers to rocking of the piers on the foundation (rocking piers, Figure 1b). The piers are not monolithically connected to the foundation; instead, they are designed to uplift and rock under seismic motion, analogous to the response of a rigid block rocking on a rigid base. The second concept promotes rocking of the pier and foundation assembly on the underlying soil (rocking footings, Figure 1c). This behavior is achieved by intentional under-sizing of the foundation to promote uplifting. The main objective of the present work is to identify the advantages and disadvantages of each design concept (rocking piers, rocking footings and conventional design). To that end, rigorous 3D numerical models of the entire bridge–foundation–abutment–soil system are developed within the ABAQUS finite element (FE) analysis environment (ABAQUS 2012), and subjected to static pushover and dynamic nonlinear time history analyses with biaxial excitation.

2.1 Rocking Piers concept

Within the rocking piers concept, two approaches have been suggested. A restraining tendon can be used (Mander & Cheng 1997; Palermo et al. 2004, Chen et al. 2006; Mariott et al. 2009, Makris & Vassiliou 2014b, Dimitrakopoulos and Paraskeva 2015, Vassiliou & Makris 2015, Giouvanidis & Dimitrakopoulos 2017a), or the piers can be allowed to rock without a tendon (Ackigoz & DeJong 2012, Makris & Vassiliou 2013; 2014a, Dimitrakopoulos & Giouvanidis 2015, Vassiliou et al. 2014, 2015, 2016, Vassiliou 2017). In this paper only the systems without restraining tendons are studied.

2.2 Rocking Footings concept

The second concept refers to the introduction of a “safety-valve” under the foundation blocks of the piers, by under-sizing them to promote full mobilization of their moment capacity during seismic shaking (Figure 1c). In this way, the soil experiences inelastic behavior and the footings are allowed to uplift under seismic excitation (e.g., Pecker 2003, Mergos et al. 2005, Gajan et al. 2008, Anastasopoulos et al. 2010, Gelagoti et al. 2012, Antonellis et al. 2015). Depending on the safety factor ($FS_v$) against static (vertical) loading, the mode of intentional foundation failure is either
uplifting (for large $FS_v$) or soil yielding (for small $FS_v$) (Anastasopoulos et al. 2012). In the latter case, the improved seismic performance comes at the cost of increased settlements, which need to be accounted for in design.

3. DESCRIPTION OF THE BRIDGE MODEL

An actual overpass bridge (A01-TE23) of the Attiki Odos motorway (Athens, Greece) is used as case study, forming the basis for the developed numerical models. The models are based on the actual bridge, bearing the necessary modifications depending on the analyzed design alternative (conventional, rocking-piers, or rocking-footings). The original bridge is an asymmetric 5-span system, having a total length of 115.6 m (Figure 2a). More details on the original bridge can be found in Agalianos et al. (2017). The bridge is designed according to the provisions of the Greek Seismic (EAK 2000) and Reinforced Concrete (EKOS 2000) Codes. The moment-curvature relationships of the piers are shown in Figure 2b. In the existing bridge, piers P1 and P2 are connected to the deck using a single sliding bearing on top of each pier, allowing for relative pier-deck displacement only in the longitudinal direction. Piers P3 and P4 are monolithically connected to the deck. All piers are founded on square (7x7m for P1 and P2, 8x8m for P3 and P4), shallow footings. At each abutment, the deck is sitting on 4 elastomeric bearings.

3.1 Conventional system

A slightly modified version of the existing A01-TE23 bridge is studied. The relevant numerical model (Figure 2c) is based on the existing bridge and forms the basis for comparison with the rocking-isolated design alternatives. The dimensions of the model are those of the actual bridge, with the exception of pier height, which is simplified to 9 m for all piers. The deck and the piers are modelled with elastic and inelastic beam elements, respectively. The inelastic pier response is simulated using a nonlinear model, according to the results of pier cross section moment-curvature analysis. Appropriate gap elements are introduced to model the sliding bearings at the abutments, having a vertical clearance $\delta_c = 0$ and a friction coefficient $\mu = 0.05$. The footings and the abutments are modeled with elastic brick elements, assuming reinforced concrete material with $E = 30$ GPa. Geometric nonlinearities (large displacements) are also taken into account in the analysis.

A 20 m deep homogeneous clay layer of undrained shear strength $S_u = 150$ kPa is considered, also modeled with brick elements. Nonlinear (inelastic) soil behavior is modelled with a thoroughly validated kinematic hardening model, with a Von Mises failure criterion and associated flow rule (Anastasopoulos et al. 2011). The evolution law of the model consists of a nonlinear kinematic hardening component, which describes the translation of the yield surface in the stress space, and an isotropic hardening component, which defines the size of the yield surface as a function of plastic deformation (Gerolymos & Gazetas 2005). Tensionless interfaces with an appropriate friction coefficient $\mu = 0.7$ are introduced between the subsoil and the footings to model uplifting and sliding, and also between the retaining wall and the embankment to model possible separation of the embankment from the wall. More details on the development of the rigorous model can be found in Anastasopoulos et al. (2011; 2015) and Agalianos et al. (2017).

3.2 Rocking Piers model

The Rocking Piers model (Figure 2d,f) is based on the model of the conventional system, with the necessary modifications to the piers and the pier–deck and abutment–deck connections in order to ensure purely rocking motion of the piers on top of the corresponding footings. No sliding is allowed for this purpose. As shown in Figure 2d, the piers are modeled with solid elements in order to simulate the rocking interface, where plane sections are not expected to remain plan. The pier ends of the actual structure would be protected with steel jackets and no concrete spalling would be expected as it was observed in experimental testing (Thonstad et al. 2016, Mashal & Palermo 2017). Therefore, elastic concrete material is used to model the piers. Their diameter is increased from 1.8 to 2.0 m to increase the uplifting acceleration and the safety margins against toppling collapse.
Figure 2. Conventionally designed A01-TE23 overpass bridge of the Attiki Odos motorway: (a) key dimensions; (b) main attributes of the FE model of the structural system; and (c) rigorous 3D FE model of the entire bridge–foundation–abutment–soil system.
The sliding bearings at the piers and the abutments are also simulated with gap elements, which allow uplifting of the deck and relative pier-deck rotation. Thus, the piers exhibit a purely rocking response with the weight of the deck and the pier being the only restoring force. More information on the modelling of the bearings is given in Agalianos et al. (2017).

3.3 Rocking Footings model

The Rocking Footings model (Figure 2e,g) is based on the Rocking Piers model with appropriate modifications. First of all, the piers are monolithically connected to the footings. Moreover, in order to introduce rocking isolation by foundation rocking, the dimensions of the footings are determined so that their moment capacity is smaller than that of the corresponding piers. Thus, full mobilization of the soil bearing capacity and uplifting of the footings will occur before failure of the piers. To that end, an “understrength” factor is applied to the footings and their moment capacity is reduced to \( 1/\gamma_{rd} = 1/1.4 \approx 0.7 (FS_E) \) of the moment capacity of the relevant piers. Thus, the apparent safety factor against seismic loading (\( FS_E \)) is lower than 1. Nevertheless, the safety factor against vertical loads (\( FS_v \)) remains within the range 2.5 – 3.0. More details on rocking-isolation of foundations can be found in Anastasopoulos et al. (2010).

4. NONLINEAR STATIC PUSHOVER ANALYSIS

Figure 3 plots the results of displacement-controlled nonlinear static analysis (pushover analysis) of each model in the longitudinal and the transverse directions. The displacement is applied at the deck level. The vertical axis plots the base shear (of the total structure) and the horizontal one the displacement of the top of Pier P4 relative to the bottom of its footing.

![Figure 3. Comparison between the examined systems in terms of static pushover total base shear–drift (F–δ) response of pier P4 in: (a) the longitudinal; and (b) the transverse direction.](image)

5. NONLINEAR STATIC AND NONLINEAR DYNAMIC TIME-HISTORY ANALYSIS

To further investigate the seismic performance of the three design alternatives, a series of nonlinear dynamic time history analyses is performed. An ensemble of 20 historic earthquake records (10 ground motions of 2 perpendicular components each) is used as seismic excitation. The latter covers a range of seismic excitations from strong (e.g. Lefkada-2003) to very strong intensity, characterized by forward-rupture directivity effects, large number of significant cycles, and/or fling step effects (e.g. Takatori, JMA, TCU052). Figure 4 shows the selected records along with their elastic spectra and the design spectrum of the A01-TE23 bridge. It can be seen that all of the selected records exceed the conventional bridge design spectrum, in many cases quite significantly.

First, the response of the three systems is compared using the devastating Takatori record from the 1995 Kobe earthquake as an illustrative example. The selected record constitutes one of the most demanding seismic motions ever recorded, with \( PGA = 0.70g \), \( PGV = 169 \text{ cm/s} \) bearing the mark of forward rupture directivity. The pulse duration is roughly 0.7s and the peak ground acceleration of the
pulse is 0.6g. The Takatori ground motion records are rotated such that the Takatori-000 component lies in the longitudinal direction of the bridge. This comparison aims at assessing the resilience of the three bridge design alternatives subjected to a very severe seismic excitation, by far exceeding the conventional bridge design spectrum.

Figure 4. Elastic spectra of the real earthquake records used for the analysis of the three examined bridge systems, along with the design spectrum of the investigated bridge.

5.1 Response of the Conventional Design bridge model

Figure 5 plots the time history response of the pier drifts for the Takatori ground motion. It analyzes the drift to its two components: the flexural and the one originating from rigid body rotation. The maximum displacement at the top of P4 in the longitudinal direction is 69 cm, while the corresponding displacement capacity (as given by the Pushover curve) is only 28 cm. In the transverse direction, the maximum displacement reaches 46 cm, also substantially exceeding the capacity (31 cm). Evidently, such seismic excitation applies such displacements that the columns might lose their vertical bearing capacity. The damage and possible collapse is due to the concentration of all the displacement to the “flexural” mode and the failure to mobilize any rigid body mode, as Figure 5 (left column) indicates.

Figure 6 plots the time histories of the bending moments in the two directions at the base of pier P4. The left column refers to the conventionally-designed system. The bending strength is reached in both directions. The lack of interaction of the bending strength along the two directions is clearly seen, as the vector sum of the maximum moment reaches $\sqrt{2}M_{ul}$ This is a limitation of the model. However, since it improves the performance of the conventional system, it further strengthens the key conclusion of the paper regarding the advantageous performance of the rocking systems.

Figure 7 plots the foundation settlement-rotation ($w-\theta$) curves for Pier P4. The curves are more useful for the Rocking Footings model described in a later section, but in the case of the conventional system one can observe that there is a residual vertical deformation of 1 cm and no residual rotation.

5.2 Response of the Rocking Piers bridge model

In contrast to the conventionally-design bridge, the rocking piers design alternative survives the Takatori ground motion, avoiding collapse without any structural damage. Figure 5 (middle column) shows that the maximum total (i.e. vector sum) horizontal drift of the piers reaches about 1 m. The latter is significantly smaller than the overturning horizontal drift capacity of the piers (1.87 m): the rocking columns bridge has a large safety margin against toppling collapse. It is also worth noting that even though the piers are not connected to the foundation, the maximum displacement for the Takatori case is only 23% larger than that of the conventional system. This observation confirms the conclusion of Makris & Vassiliou (2014b) and Vassiliou & Makris (2015) that the presence of reinforcement or restraining tendons has marginal effect on the maximum deformations of large rocking structures. Moreover, it contradicts the widely-established belief that increased displacements is always a price that needs to be paid for decreased design loads.
Figure 5 shows that there is essentially zero flexural deformation in the piers, and that all of the displacement is taken by rigid body rotation of the piers. The total drift plot (center column, bottom) also shows that there is only one impact (at \( t = 15.5 \) s when \( \delta_{xy} = 0 \)). From the same plot, one can deduce that the size of the gap between the column and the foundation does fluctuate, but generally it does not become zero, indicating that the column sustains a wobbling motion (i.e., rolling on the perimeter of its base). The change of pivot point is sometimes fast, but it is only scarcely instantaneous (Vassiliou et al. 2017).

Figure 6 shows that 3D wobbling motion avoids the very large spikes in the moment time history that planar rocking structures experience (Aciakgoz & DeJong 2012, Vassiliou et al. 2015, Truniger et al. 2015, Giouvanidis & Dimitrakopoulos 2017b). Spikes do occur, at every fast (but not instantaneous) change of pivot point, but their magnitude is clearly smaller than the ones observed in planar motion and are always smaller than the moment strength of the pier (even when they are vectorially added), as shown in Figure 6. The oscillations after the spike are attributed to the oscillation of the pier itself due to the impact. In any case, in the rocking piers model, the bending moment results should only be perceived as an indication and cannot be used for flexural design, because in the contact area between the piers and the footing, plane sections of the piers do not remain plain, there is a discontinuity of the structure and ordinary RC cross section analysis (which is based on an equivalent continuous beam and only implicitly takes cracking into account) is not expected to yield trustworthy results.

**5.3 Response of the Rocking Footings bridge model**

Similar to the rocking piers design alternative, collapse is easily avoided in the rocking footings
bridge, with the maximum column drift reaching 0.77 m (Figure. 5), i.e. roughly 4 times smaller than the pier toppling capacity (3 m). It can be seen that the major component of the drift is the rocking one, albeit there is some minor contribution from flexure. Unlike the rocking piers model, the total displacement, $\delta_{xy}$, does become zero: there is limited wobbling (i.e., limited rolling on the perimeter of the foundation). However, the soil deformability limits the harshness of the rocking impacts and the bending moment time history (Figure 6, right) does not have the spikes observed in the rocking pier model. The total bending moments at the base of pier P4 ($M_{tot}$) never exceed its corresponding moment capacity ($M_{ult}$).

Figure 7 (right column) reveals that the inelastic behavior of soil results in accumulation of settlements. It is observed that during the first strong motion cycles of the excitation, the footings are subjected to significant rotations, which are subsequently reduced, while the settlement increases. The residual rotation is negligible in all cases, but the settlement reaches 15 cm.

Figure 6. Comparison of the three design alternatives in terms of normalized moment at each direction and total moment to moment capacity time histories $M/M_{ult}$, indicatively for pier P4 for the Takatori record.

### 5.4 Bridge model response comparison for 10 ground motions.

Figure 8 collectively plots the results for all 10 ground motions. The maximum displacement $\delta_{xy}$ (along any direction) is plotted versus the PGV of the ground motion. For the conventional and the rocking piers system, it is expected that failure along any direction would occur at a displacement roughly equal to the ultimate longitudinal and transverse direction displacement, because the pier displacement failure surface is circular. Therefore, the ultimate displacement (i.e., the one that corresponds to pier failure) is taken equal to 0.28 m and 1.87 m respectively, in any direction. The rocking footings model ultimate displacement capacity is conservatively taken equal to 3 m in any direction.
Figure 7. Comparison of the three design alternatives in terms of settlement–rotation ($w$–$\theta$) response at each direction, indicatively for pier P4 of the bridge for the Takatori record.

Figure 8 shows that in 5/10 cases (of intentionally selected extremely severe seismic excitations), the conventional bridge collapses. In stark contrast, both rocking design alternatives exhibit remarkable stability. The rocking piers system overturns in only 2/10 ground motions and the rocking footings survives all of them with a maximum drift not exceeding 1/3 of its ultimate capacity. Moreover, Figure 8 shows that the rocking footings system experiences only slightly larger maximum drifts (the median increase is 27%) than the conventional system (event though the former has a much lower strength). The rocking piers system experiences larger displacements than the conventional one (the median increase is 68%). For the calculation of these median values only the analyses for which the conventional system did not fail are considered (5/10).

The price to pay for the superior stability and reduced displacements of the rocking footings system is the increased settlement of the footings (Figure 8, bottom row), which is clearly larger than the negligible settlement of the conventional and the rocking piers system.

6. DISCUSSION AND CONCLUSIONS

The present paper used an actual motorway overpass bridge to comparatively assess the seismic performance of three different design alternatives: one conventional and two based on rocking. For this purpose, 3D numerical models of the entire bridge–foundation–abutment–soil system were developed, and both static pushover and biaxial excitation nonlinear dynamic time-history analyses were performed. For the latter, 10 very strong earthquake records were used to examine the performance of the three systems subjected to excitations that significantly exceed the design limits. The conventional, rocking piers, and rocking footings system have maximum displacement capacities on the order of 0.30, 1.90 and 3m.

For the selected pier and foundation dimensions, the conventional system collapses in half of the examined seismic excitations. Even for design-level earthquakes, for which failure should be avoided, the plastic design concepts on which the conventional design is based lead to non-negligible residual displacements, possibly rendering the bridge non-repairable (note that such behavior is acceptable according to the current seismic design codes, based on the plastic design philosophy).
Figure 8. Summary of the response of the three examined systems in terms of maximum drift $\delta_d$, displacement demand over displacement capacity $\delta_d/\delta_u$, and maximum settlement $w_d$ with respect to the peak ground velocity $PGV$ for the selected ensemble of 20 records (10 ground motions of two perpendicular components each).

The rocking pier design alternative avoids toppling collapse in 80% of the (intentionally very severe) seismic excitations examined, even without prestressing tendons or extra damping. A design with wider piers or wider pier ends would survive all ten ground motions. The residual deformations are negligible, and, therefore, the structure is resilient and ready-to-use even after severe earthquakes. However, the maximum displacements are larger than the ones of the conventional system (the median increase is 68%). The rocking pier system requires special protection the pier ends (e.g. steel jackets) and a restrain to prevent the piers from rolling out of their position. Such jackets have already been used and tested on pier models with prestressing tendons (for which the contact stresses are expected to be larger). Furthermore, a special design of the bearings at the pier-deck connection is necessary, so that the rotation of the piers is not constrained.

The rocking footings solution avoids toppling collapse in all cases examined, clearly offering the largest margins of safety. The residual rotations are practically equal to zero and no residual tilting of the structure is observed in all cases examined. The median increase of maximum displacements (compared to the conventional system) is only 27%. However, the mobilization of foundation bearing capacity unavoidably leads to increased residual settlement, which is the price to pay for such advantageous performance. Compared to the rocking piers solution, it does not require any special construction detailing, and hence it is more straight-forward for contractors to construct. Compared to the rocking piers solution, where stress concentrations at the pier-footing contact areas are expected to develop and where potential impacts are more severe, the soil acts as a pillow offering additional damping and limiting the effects of these detrimental phenomena. The rocking footings approach can also be used for the seismic retrofit of existing bridges that were built according to obsolete seismic codes. In such cases, the retrofit could focus on the superstructure, keeping the existing foundations and avoiding expensive pilings. By relaxing the allowable uplift criterion for the foundation, the rocking footing design can be introduced in the retrofit, offering the extra benefit of acting as a safety valve, limiting the forces transmitted to the superstructure. Both rocking systems result in significantly smaller foundation design moment that could justify avoiding costly pile foundations.
7. REFERENCES


Giouvanidis, A. I., & Dimitrakopoulos, E. G. (2017b). Nonsmooth dynamic analysis of sticking impacts in


Pecker, A. (2003). Aseismic foundation design process, lessons learned from two major projects: the Vasco de Gama and the Rion Antirion bridges. *ACI Int. Conf. Seismic Bridge Design and Retrofit*, University of California at San Diego, La Jolla, USA.


