

Shake table testing and finite element modeling of a modular bridge-like model with controlled rocking piers

Antonios A. Katsamakas

Assistant Professor, Polytechnique Montreal

Michalis F. Vassiliou

Assistant Professor, National Technical University of Athens

ABSTRACT

This study presents the results of triaxial shake table testing and three-dimensional (3D) finite element (FE) modeling of a modular bridge-like specimen. The specimen comprised four prefabricated reinforced concrete (RC) cylindrical columns capped with a slab. The structural connections were non-monolithic, allowing controlled relative motion of the members (rocking and sliding). The columns were connected to the slab with stiff tendons that provided positive post-uplift stiffness. The specimen was subjected to 184 triaxial shake table tests. Subsequently, a detailed 3D FE model of the specimen was developed and validated statistically. The objectives of the study were to: (i) investigate the triaxial shake table response of a modular bridge with positive post-uplift stiffness, (ii) develop an FE model of the proposed system, and (iii) investigate the influence of geometrical imperfections on rocking structures. After 184 shake table tests, the specimen showed zero damage, moderate displacements and tendon forces (TFs), low slab torsion, and zero residual displacements. The proposed FE model accurately captured the statistics of the experimental response. Results showed that negative stiffness systems are sensitive to geometrical imperfections, whereas positive stiffness systems are not.

Keywords: rocking bridge, prefabrication, resilient design, shake table testing

INTRODUCTION

Prefabrication reduces construction time and improves quality control in bridges. The current practice in seismic design of precast bridges is to replicate monolithic behavior using prefabricated elements with stiff, strong connections, which increases construction time and cost. Over the past decades, the dogma of monolithic structures has been revisited. Non-monolithic (modular) structures that allow relative motion (rocking) of their members can have superior seismic performance. Rocking motion acts as a fuse, limiting the forces transmitted to the superstructure and foundation. In parallel, rocking systems are resilient since they recenter with minimal (if any) damage (Christopoulos et al, 2002). Rocking isolation can be applied in both bridges (Agalianos et al., 2017; Dimitrakopoulos and Giouvanidis, 2015; Makris and Vassiliou, 2015; Salehi et al., 2021; Katsamakas and Vassiliou, 2022, 2023; Reggiani-Manzo and Vassiliou, 2022) and buildings (Bantilas et al., 2021; Anastasopoulos et al., 2015; Bachmann et al., 2017). Rocking structures inherently have negative post-uplift stiffness (Giouvanidis et al., 2022; Vlachakis et al., 2021; Elmorsy and Vassiliou, 2023; Elmorsy et al., 2024; Liu et al., 2023), and their behavior cannot be related to any equivalent elastic system (Makris and

Konstantinidis, 2003). The use of stiff tendons can increase their post-uplift stiffness (controlled rocking).

Rocking isolation with negative stiffness is a design alternative inspired by ancient Greco-Roman and Chinese temples (Drosos and Anastasopoulos, 2014; Sarhosis et al., 2016; Konstantinidis and Makris, 2005). Recently, Reggiani-Manzo et al. (2022) performed 184 shake table tests of a negative stiffness bridge-like specimen. The system proved resilient in terms of column damage, but collapsed due to tendon failure in two tests. The tests also revealed significant deck torsion, likely caused by geometrical imperfections: inspection of the pier top surfaces showed inclinations of 1.5%. Such torsional response cannot be captured by simplified numerical models (Vassiliou et al., 2017).

This paper presents shake table tests of a bridge-like controlled rocking specimen with a restraining system stiff enough to provide positive post-uplift stiffness to the system. Then, it develops a detailed 3D finite element (FE) model that can capture the statistics of rocking motion and account for imperfections of the piers.

MECHANICS OF CONTROLLED ROCKING FRAMES

The linearized lateral force-displacement relation of a controlled rocking frame with unbonded tendons (anchored at the bottom end of the piers) appears in Equation (1) (Reggiani-Manzo and Vassiliou, 2022).

$$F = [(0.5 + \gamma) N m_c g a \operatorname{sgn}(u)] + [0.5 N k_{res} b a - (0.5 + \gamma) N m_c g] \frac{u}{2h} \quad (1)$$

In Equation (1) and Figure 1, m_c and m_b are the mass of each pier and the deck, respectively, and N is the total number of piers. The width, height, semi-diagonal, slenderness and tilt angle of the pier are $2b$, $2h$, R , $a = 2b/2h$, and θ , respectively. The gravitational acceleration is g . Under lateral force F , the lateral displacement of the frame is u . The axial stiffness of the restraining system is k_{res} . The deck-to-pier mass ratio is defined as $\gamma = m_b/(N \times m_c)$. Figure 1 (C) plots the pushover response of the rocking frame for increasing values of k_{res} . The critical stiffness ($k_{crit} = \frac{[(1+2\gamma)m_c g]}{\alpha b}$) is the one that leads to zero post-uplift stiffness ($k_{res} = k_{crit}$). The post-uplift stiffness is positive when $k_{res} > k_{crit}$ and negative when $k_{res} < k_{crit}$. For positive post-uplift stiffness, the post-uplift eigenperiod ($T_{post-uplift}$) is:

$$T_{post-uplift} = 2\pi \sqrt{\frac{m_b + (0.5 N m_c)}{K}} \quad (2)$$

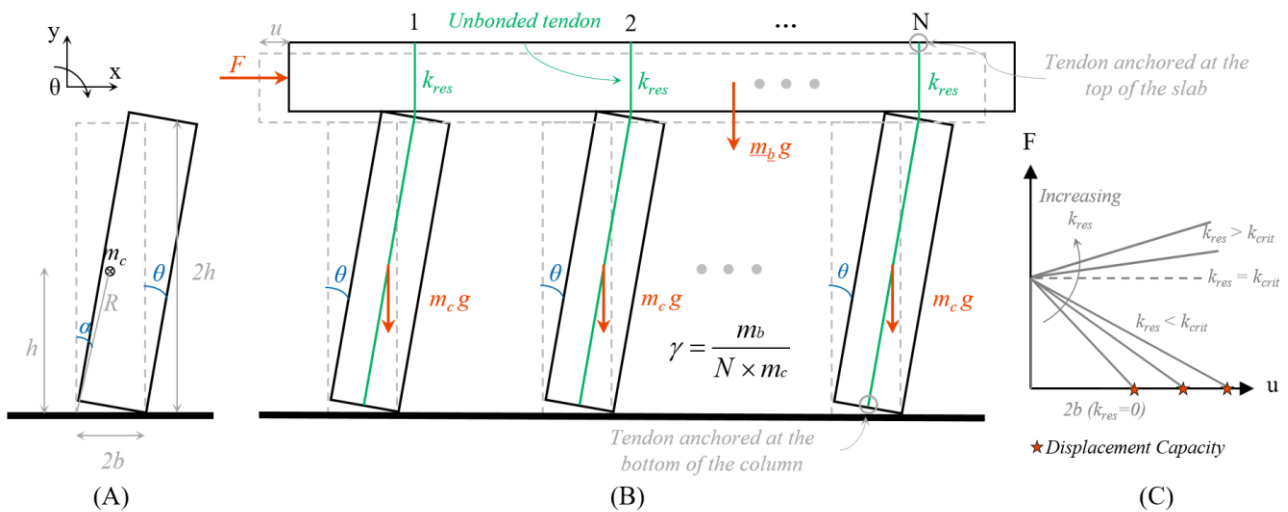


Figure 1. (A) Freestanding rocking block; (B) Controlled rocking frame; (C) Lateral force-displacement response of the frame for varying k_{res} .

METHODOLOGY

The tested specimen comprised four reinforced concrete (RC) piers and an RC deck at 1:5 scale ($S_L = 1/5$) (Figure 2). The scaling laws for reduced-scale physical modeling appear in Katsamakas et al. (2022). An unbonded steel tendon passes through each pier. It is anchored at the bottom of the pier and above the deck. Detailed construction drawings of the specimen appear in (Katsamakas et al., 2024). In both the tests of Reggiani-Manzo et al. (2022) and in this work, the ground motions were scaled in frequency by $\sqrt{5}$. Compared to the specimens of Reggiani-Manzo et al. (2022), the tendons used in this study had higher strength and stiffness, resulting in a positive post-uplift eigenperiod. The tendon used was a 19-wire strand with a diameter of 16 mm. The average strength and stiffness of the tendons were 160.3 kN and 7200 kN/m, respectively. According to Equation (2), this gives a post-uplift period of 2.2 sec (model scale). Since rocking bridges are designed to remain undamaged under design-level seismic loads, they can be subjected to many ground motions without accumulating damage. A total of 184 ground motions spanning all three categories of FEMA P695 were used in the shake table tests. The ground motions were identical to those of Reggiani-Manzo et al. (2022) to allow direct comparison between the two systems. Further details on the ground motions are provided in Katsamakas et al. (2024).

The shake table tests were performed at the Laboratory for Earthquake Engineering (LEE) of the National Technical University of Athens (NTUA). The shake table comprises a 4×4 m, six-degrees-of-freedom platform. The displacement, velocity, horizontal acceleration, and vertical acceleration capacity of the shake table is ± 100 mm, 1000 mm/sec, 2g and 4g, respectively. The RC deck and the shake table were instrumented with accelerometers and displacement sensors. All tendons were equipped with load cells at their top end (Figure 2 (B)). The sampling frequency ranged between 225 Hz and 450 Hz, depending on the duration of the ground motion. More details on the instrumentation setup appear in Katsamakas et al. (2024). The main response quantities were the displacement at the center of the deck (U_{center}), the maximum displacement at the corner of the deck (U_{corner}), the deck torsion around its vertical axis (z-axis), the tendon forces (TF), and the acceleration at the center of the deck (A_{center}). This study used a one-by-one (deterministic) and a statistical validation procedure (Vassiliou et al., 2021; Malomo et al., 2021; Zhong and Christopoulos, 2021, 2023; Zaki et al., 2025; Zhang et al., 2025; Del Giudice et al., 2022, 2024; Bachmann et al., 2018; Katsamakas and Vassiliou, 2023; Zhu et al, 2026). The one-by-one comparison uses scatter plots and the Pearson correlation coefficient (R). If the two datasets are identical, all points of the scatter plot should fall on a diagonal line of the scatter plot, and $R=1$.

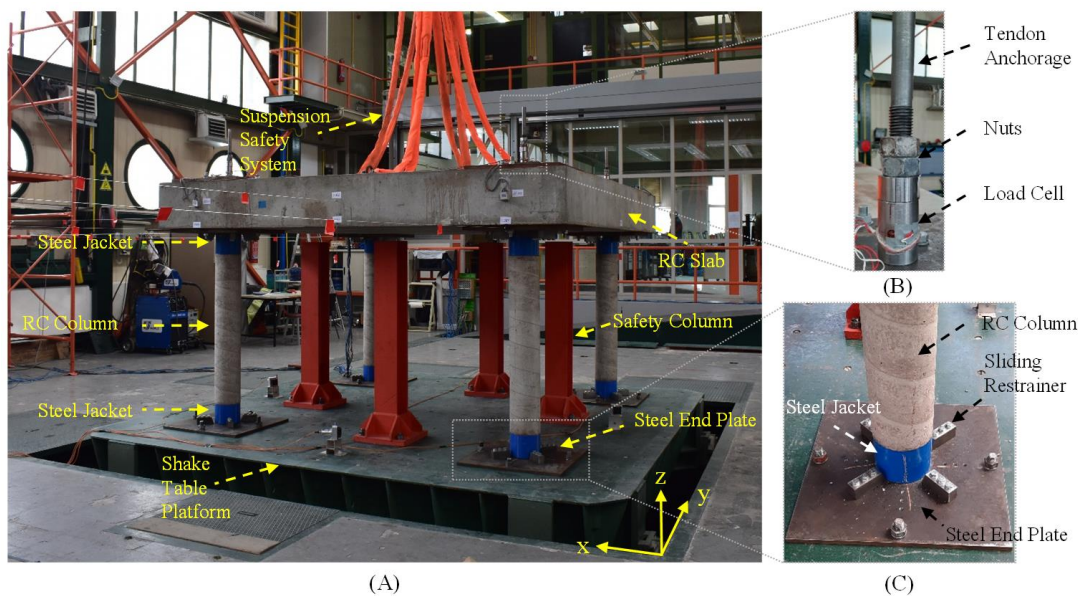


Figure 2. (A) View of the tested model; (B) Detail of the top anchorage of the tendon; (C) Detail of the base of the free-standing piers and the sliding restrainers.

The statistical validation procedure compares the cumulative distribution function (CDF) of two datasets and is quantified with the Kolmogorov-Smirnov (K-S) p -value test (Kolmogorov, 1933; Smirnov, 1939). When $p > 0.1$ the two datasets are statistically identical. Moreover, the validation procedure uses the relative error (e) at the median (e_{50}) and at the 90th (e_{90}) percentile of the distribution and the Kolmogorov-Smirnov distance (K-S Dist.) (i.e., the maximum vertical distance between the two CDFs) to compare the datasets.

EXPERIMENTAL RESULTS

Figure 3 offers insight into the structural response under the 1999 Chi-Chi Taiwan (FF) ground motion (CHY101 station), which induced the highest displacements and tendon forces. Figure 3(A) shows the time history of the deck torsion. The torsion amplifies the displacements at the corners of the deck and increases the tendon forces. Figure 3(B) shows the acceleration-displacement plot at the center of the deck along the x- and y-axis. Figure 3(C) shows the total displacement and the total acceleration at the center of the deck. The structure has finite stiffness in the pre-uplift phase, contrary to what rigid-body models assume. For each direction (x or y), the model uplifts at approximately 0.09–0.10g of acceleration. When the two directions are combined (Figure 3(C)), the uplift acceleration is approximately 0.13g, very close to the slenderness of the piers ($\alpha = 0.135$).

The acceleration-displacement loops of Figure 3(B) enclose a relatively small area. However, this does not necessarily mean that there is no damping. Energy is dissipated during impacts at the rocking interfaces and between the piers and the sliding restrainers. This form of energy dissipation is instantaneous and does not appear as an enclosed area in force-deformation loops. Energy is also dissipated due to friction at the pier-base and pier-deck interfaces from sliding and twisting of the piers. Figure 3(D) plots the time history of the displacement at the center of the deck (U_{center}) together with the displacements at the four corners ($U_{corner,1}$, $U_{corner,2}$, $U_{corner,3}$, $U_{corner,4}$). The displacements at the corners are slightly larger than the displacements at the center due to the torsion of the deck. The variability of the tendon forces was marginal and lower than the variability of the corner displacements, as shown in Figure 3(E).

The bridge system sustained no damage after 184 ground motions. The concrete members, steel jackets, and steel restrainers remained intact, and the tendons remained elastic throughout all tests with no thread slippage.

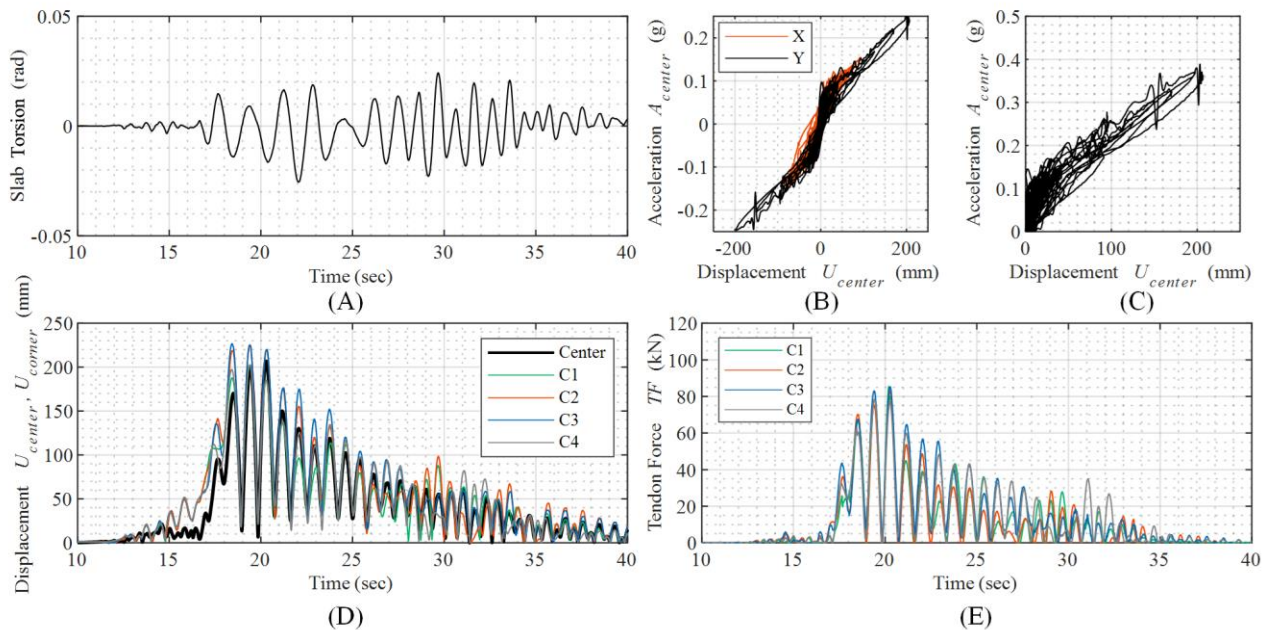


Figure 3. Insight into the structural response under the 1999 Chi-chi ground motion.

Figure 4 compares the results of the present study to the results of Reggiani-Manzo et al. (2022). Apart from the tendon stiffness, the structures and the ground motions are identical in both studies allowing for a direct comparison of their performance. The results show that the positive stiffness system has significantly lower displacements (U_{center} and U_{corner}) and substantially lower slab torsion. This comes at the expense of slightly higher tendon forces that are, however, below 50% of the tensile strength of the tendons. Most importantly, the negative stiffness system collapsed in two tests, but the positive stiffness system did not, proving its superior performance.

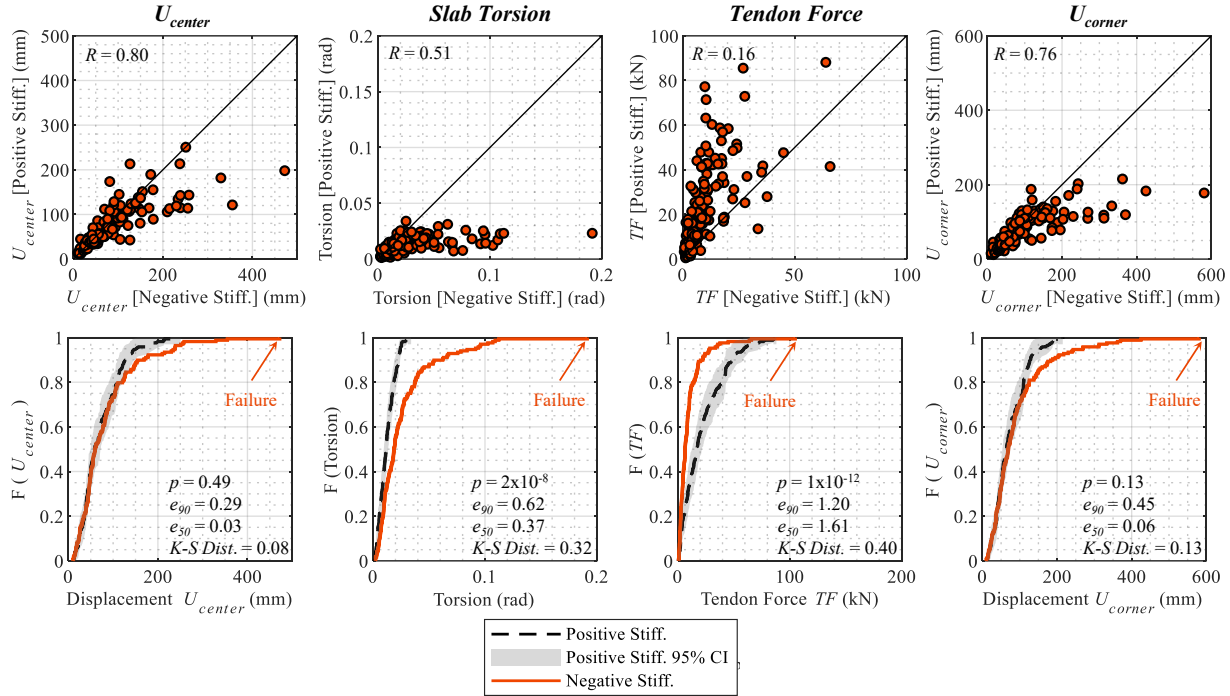


FIGURE 4. Comparison of the shake table response of the system with positive (present study) to the negative (Reggiani-Manzo et al. (2022) post-uplift stiffness). Apart from the tendon stiffness, the model and the ground motions were identical in both studies.

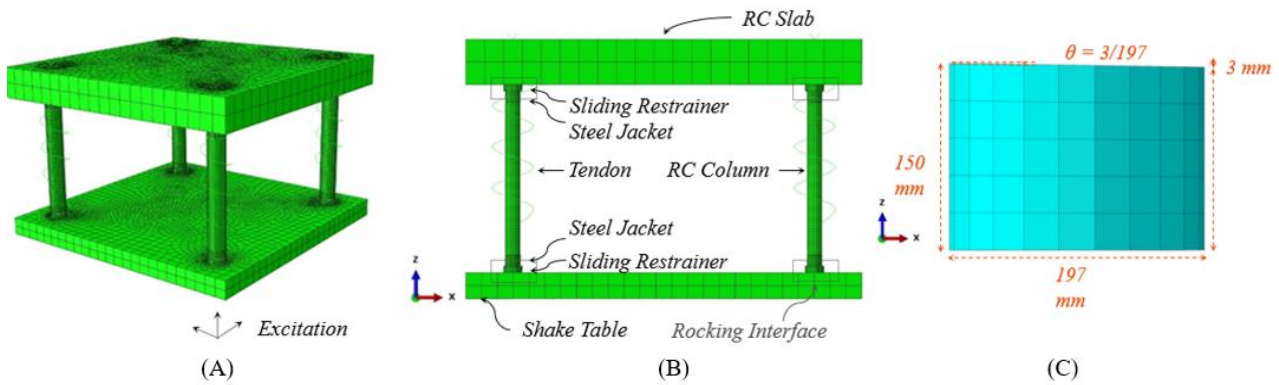


Figure 5. Three-dimensional finite element model.

FINITE ELEMENT MODELING (FEM)

A three-dimensional finite element model was developed in ABAQUS (ABAQUS, 2022) to statistically capture the shake table response of controlled rocking systems and to investigate the influence of geometrical imperfections on their seismic response. Structural elements were modeled using 8-node hexahedral (brick) elements (C3D8 in ABAQUS notation) (Katsamakos et al., 2021, 2022), as shown in Figure 5. Ground motions were applied as base accelerations at

the shake table level (Figure 5A). All analyses accounted for geometrical nonlinearity. Since no yielding was observed, all parts were modeled as linear elastic with material properties for concrete being $E = 33 \text{ GPa}$, $\nu = 0.2$, $\rho = 2500 \text{ kg/m}^3$ and for steel $E = 200 \text{ GPa}$, $\nu = 0.3$, and $\rho = 7850 \text{ kg/m}^3$.

The mesh was refined at contact areas (element size of 30 mm) and coarsened elsewhere for the piers, deck, shake table, and restrainers. Parts belonging to the same structural element but with different mesh sizes were connected using tie constraints. Further details of the FE model are provided in Katsamakas et al. (2024). The tendons were modeled as linear elastic spring elements with zero energy dissipation and an axial stiffness of 7200 kN/m; four spring elements were used, each representing one tendon. Time integration was performed using the implicit Hilber-Hughes-Taylor (HHT) method (Hilber et al., 1977) with a variable time step. Katsamakas and Vassiliou (2024) showed that a time step of $dt = 10^{-3} \text{ s}$ and a damping parameter of $\alpha_{HHT} = -0.2$ yield sufficiently accurate results.

Surface-to-surface contact elements were used to define the interaction between members. Contact behavior is characterized by its normal and tangential components. The tangential response follows the classical Coulomb friction model, with a friction coefficient of $\mu = 0.3$ and zero cohesion. Hard contact (no surface penetration) was used for the normal behavior. The model explicitly accounts for energy dissipation through friction at the sliding interfaces (pier-to-base, pier-to-deck, and pier-to-restrainer), and neglects impact damping. Geometrical imperfections observed by Reggiani-Manzo et al. (2022) were also explicitly included in the model (Figure 5C) to investigate their influence on the seismic response of rocking systems. Inspections of the columns of Reggiani-Manzo et al. (2022) showed that the top surface of the columns was inclined by 3 mm over the 197 mm diameter (net inclination of 3/197). This was explicitly included in the FE model (Figure 5C).

FEM RESULTS

Figure 6 validates the FE model of the positive post-uplift stiffness system. Two versions of the numerical model are explored: with and without imperfections. Both the one-by-one and the statistical comparison of U_{center} and deck torsion show high correlation between the experimental and FE results. The geometrical imperfections have negligible influence on U_{center} . The statistical comparison shows that CDFs of the FE model are always inside the 95% boundaries of the experimental results. These observations are confirmed by the corresponding indices (R, p, e90, e50, and K-S Dist.). The one-by-one validation between the experimental and FE results shows moderate-to-low correlation for TF and U_{corner} . Tendon forces are directly influenced by U_{corner} , and both quantities are highly sensitive to geometric imperfections, as the rotational stiffness of the structure is considerably lower than its translational stiffness. While the addition of stiff tendons increases rotational stiffness, this increase is substantially smaller than the corresponding gain in translational stiffness. However, the statistical comparison shows that the CDFs of both FE models are inside the 95% confidence interval (CI) of the experimental CDF, with p-values above 0.1. All error indices (e50, e90) are below 0.2. Therefore, both FE models, with and without imperfections, predict the experimental statistical distribution of TF and U_{corner} with reasonable accuracy.

When the system has positive stiffness, its sensitivity to geometrical imperfections is smaller, since its restoring force mainly originates from the elongation of the restraining tendons, not the gravity load that each column bears, which is highly sensitive to imperfections in statically indeterminate systems. Therefore, it can be concluded that modeling systems with stiffer tendons is easier, as the geometrical imperfections do not significantly influence the results and do not have to be modeled explicitly.

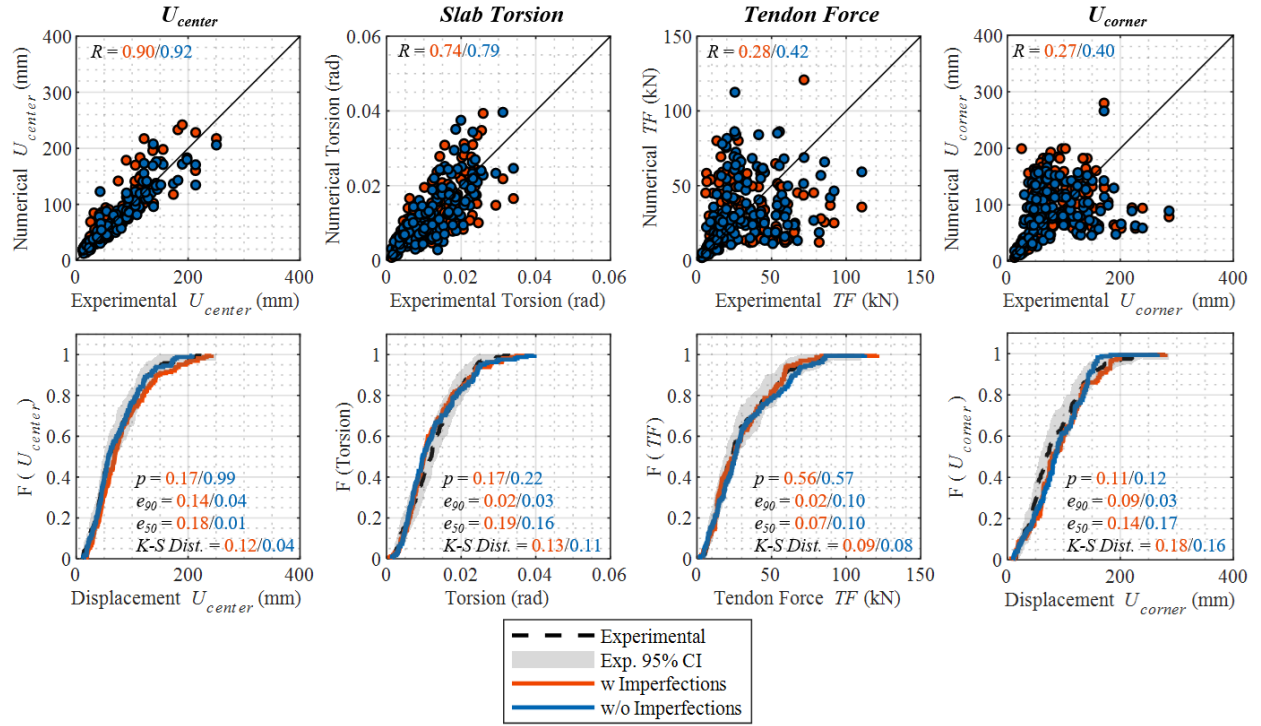


Figure 6. Comparison of the experimental results of the present study (positive post-uplift stiffness) to the FE model, with or without the geometrical imperfections.

CONCLUSIONS

The present study investigated the shake table response and finite element modeling of a modular bridge-like specimen with rocking piers. The specimen comprised four cylindrical RC piers capped with an RC deck. All members were prefabricated. Stiff steel tendons connected the base of the piers to the top of the deck, leading to a controlled rocking response with positive post-uplift stiffness. The specimen was subjected to 184 triaxial shake table tests. Subsequently, a detailed three-dimensional FE model was developed to capture the experimental response. The main outcomes are summarized as follows:

- The bridge specimen showed zero damage and perfect recentering after being subjected to 184 triaxial ground motions.
- The structural response is characterized by moderate displacements (below 200 mm in model scale) and low deck torsion.
- The use of stiff tendons leads to positive post-uplift stiffness of the system. This reduces displacements and rotations while leading to a substantial isolation effect (reduction of accelerations and zero damage).
- The acceleration-displacement response is practically bilinear. The uplift force and post-uplift stiffness are very close to the analytical predictions.
- Energy dissipation is moderate and is due to sliding and twisting of the piers and impacts with the displacement restrainers.
- Geometrical imperfections have negligible effect on the response of systems with positive post-uplift stiffness. This is because the restoring force of these systems mainly originates from the steel tendons, not gravity.
- The developed FE model accurately captures the experimental response, including the displacement at the center (U_{center}) and at the corner (U_{corner}), the deck torsion, and the tendon force (TF).

ACKNOWLEDGEMENTS

Funding to the first and second author was provided by the European Research Council (ERC) under Starting Grant 803908.

REFERENCES

- ABAQUS Standard User's Manual. Providence. RI: Dassault Systèmes Simulia Corp; 2022
- Agalianos, A, Psychari, A, Vassiliou, MF, Stojadinovic, B, and Anastasopoulos, I. (2017). Comparative Assessment of Two Rocking Isolation Techniques for a Motorway Overpass Bridge. *Front. Built Environ.*
- Anastasopoulos, I., Drosos, V., and Antonaki, N. (2015), Three-storey building retrofit: rocking isolation versus conventional design. *Earthquake Engng Struct. Dyn.*, 44, 1235–1254.
- Bachmann JA, Strand M, Vassiliou MF, Broccardo M, and Stojadinović B. (2018). Is rocking motion predictable? *Earthq Eng Struct Dyn.* 47(2):535-552.
- Bachmann JA, Vassiliou MF, and Stojadinović B. (2017) Dynamics of rocking podium structures. *Earthq Eng Struct Dyn.* 46:2499.
- Bantilas KE, Kavvadias IE, Vasiliadis LK, and Elenas A. (2021). Seismic fragility and intensity measure investigation for rocking podium structures under synthetic pulse-like excitations. *Earthq Eng Struct Dyn.* 50(13):3441-3459.
- Christopoulos, C., Filiatrault, A., & Folz, B. (2002). Seismic response of self-centring hysteretic SDOF systems. *Earthquake engineering & structural dynamics*, 31(5), 1131-1150.
- Del Giudice, L, Wrobel, R, Katsamakas, AA, Leinenbach, C, and Vassiliou, MF. (2022) Physical modelling of reinforced concrete at a 1:40 scale using additively manufactured reinforcement cages. *Earthquake Engng Struct Dyn.* 51: 537–551.
- Del Giudice, L., Katsamakas, A., Liu, B., Sarhosis, V., & Vassiliou, M. F. (2024). Physical modelling of unreinforced masonry walls using a sand-based 3D printer. *Engineering Structures*, 305, 117665.
- Dimitrakopoulos EG. and Giouvanidis AI. (2015). Seismic response analysis of the planar rocking frame. *J Eng Mech.* 141(7):04015003. DOI: 10.1061/(ASCE)EM.1943-7889.0000939
- Drosos, V. and Anastasopoulos, I. (2014). Shaking table testing of multidrum columns and portals, *Earthquake Engineering & Structural Dynamics*, 43, pages 1703–1723, doi: 10.1002/eqe.2418
- Elmorsy, M. and Vassiliou, MF. (2023). Effect of ground motion processing and filtering on the response of rocking structures. *Earthquake Engng Struct Dyn.* 52: 1704–1721.
- Elmorsy, M., Katsamakas, A., Jones, L., Brunschweiler, E., Anastasopoulos, I., & Vassiliou, M. F. (2024). Preliminary Centrifuge Tests on Unreinforced Masonry Buildings Built Using a Sand-Based 3D Printer. *Proceedings of the 4th ACPMG 2024*.
- FEMA P695. (2009) Quantification of Building Seismic Performance Factors [Rep. FEMA P695]. Washington, D.C: Federal Emergency Management Agency.
- Giouvanidis, A. I., Dimitrakopoulos, E. G., and Lourenço, P. B. (2022). Chattering: an overlooked peculiarity of rocking motion. *Nonlinear Dynamics*, 109(2), 459-477.
- Hilber HM, Hughes TJ and Taylor RL.(1977) Improved numerical dissipation for time integration algorithms in structural dynamics. *Earthq Eng Struct Dyn.*;5(3):283-292
- Katsamakas AA, Papanikolaou VK, and Thermou GE. (2021). A FEM-based model to study the behavior of SRG-strengthened R/C beams. *Composite Structures.* 266, 113796, <https://doi.org/10.1016/j.compstruct.2021.113796>
- Katsamakas AA, Papanikolaou VK, Thermou GE, and Katakalos K. (2022) Experimental and numerical investigation of the uniaxial compression behavior of SRG-Jacketed R/C columns. *Structures.* 2022; 44, 603-617. <https://doi.org/10.1016/j.istruc.2022.07.089>

- Katsamakas, A. A., Vassiliou, M. F., & Mouzakis, C. (2024). Shake table testing and finite element modeling of a modular prefabricated concrete bridge-like specimen accounting for geometry imperfections and additional damping. *Earthquake Engineering & Structural Dynamics*, 53(11), 3607-3629. <https://doi.org/10.1002/eqe.4189>
- Katsamakas, A., Vassiliou, M. F., & Mouzakis, C. (2025). Triaxial shake table testing and finite element modeling of a modular prefabricated bridge-like specimen with restrained rocking piers. In 19th World Conference on Seismic Isolation (19WCSI).
- Katsamakas, A.A., and Vassiliou, M. F. (2024). Finite element modeling of the seismic response of a bridge-like model with restrained rocking piers. In 18th World Conference on Earthquake Engineering (WCEE2024).
- Katsamakas, AA and Vassiliou, MF (2023). Finite element modeling of the shake-table response of a bridge-like model comprising rocking columns. 9th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering (COMPdyn 2023).
- Katsamakas, AA and Vassiliou, MF. (2022). Finite element analysis of the dynamic response of a free-standing cylindrical column: A statistical approach. 3rd International Conference on Natural Hazards & Infrastructure (ICONHIC2022).
- Katsamakas, A. A., & Vassiliou, M. F. (2022). Finite element modeling of free-standing cylindrical columns under seismic excitation. *Earthquake Engineering & Structural Dynamics*, 51(9), 2016-2035.
- Katsamakas, AA and Vassiliou, MF. (2023). Experimental parametric study and phenomenological modeling of a deformable rolling seismic isolator. *Journal of Earthquake Engineering*. DOI: 10.1080/13632469.2023.2189978
- Katsamakas, AA., Belser, G., Vassiliou, MF., Blondet, M. (2022). Experimental investigation of a spherical rubber isolator for use in low income countries. *Engineering Structures*, 250, 113522. <https://doi.org/10.1016/j.engstruct.2021.113522>
- Kolmogorov A. (1933). Sulla determinazione empirica di una legge di distribuzione. *Inst Ital Attuari Giorn*.4:83-91.
- Konstantinidis D, Makris N. (2005). Seismic response analysis of multidrum classical columns. *Earthq Eng Struct Dyn*. 34:1243-1270.
- Liu P, Zhang Y-M, Chen H-T, and Yang W-G. (2023). Experimental study on rocking blocks subjected to bidirectional ground and floor motions via shaking table tests. *Earthquake Engng Struct Dyn*. 52: 3171–3192. <https://doi.org/10.1002/eqe.3918>
- Makris N, and Vassiliou MF. (2015). Dynamics of the rocking frame with vertical restrainers. *J Struct Eng.*; 141:04014245. (ASCE)ST.1943-541X.0001231
- Makris, N. and Konstantinidis, D. (2003), The rocking spectrum and the limitations of practical design methodologies. *Earthquake Engng. Struct. Dyn.*, 32: 265-289.
- Malomo, D, Mehrotra, A, and DeJong, MJ. (2021). Distinct element modeling of the dynamic response of a rocking podium tested on a shake table. *Earthquake Engng Struct Dyn*. 50: 1469–1475.
- Reggiani Manzo, N, and Vassiliou, MF. (2022). Cyclic tests of a precast restrained rocking system for sustainable and resilient seismic design of bridges. *Engineering Structures*. 252: 113620.
- Reggiani Manzo, N, Vassiliou, MF, Mouzakis, H, Badogiannis, E. (2022). Shaking table tests of a resilient bridge system with precast reinforced concrete columns equipped with springs. *Earthquake Engng Struct Dyn*. 51: 213–239. <https://doi.org/10.1002/eqe.3563>
- Salehi, M, Sideris, P, and Liel, AB. (2021). Experimental testing of hybrid sliding-rocking bridge columns under torsional and biaxial lateral loading. *Earthquake Engng Struct Dyn*. 50: 2817–2837. <https://doi.org/10.1002/eqe.3474>
- Sarhosis V, Asteris P, Wang T, Hu W, and Han Y. (2016). On the stability of colonnade structural systems under static and dynamic loading conditions. *Bull Earthq Eng*. 14(4):1131-1152.

- Sideris, P, Aref, AJ, and Filiatrault, A. (2014). Large-scale seismic testing of a hybrid sliding-rocking posttensioned segmental bridge system. *Journal of Structural Engineering*.140(6), 04014025. DOI: 10.1061/(ASCE)ST.1943-541X.0000961
- Smirnoff N. (1939) Sur les écarts de la courbe de distribution empirique. *Matematicheskii Sbornik*. 48(1):3-26.
- Vassiliou, M. F., Burger, S., Egger, M., Bachmann, J. A., Broccardo, M., and Stojadinovic, B. (2017) The three-dimensional behavior of inverted pendulum cylindrical structures during earthquakes. *Earthquake Engng Struct. Dyn.*, 46: 2261–2280.
- Vassiliou, MF, Broccardo, M, Cengiz, C, et al. (2021). Shake table testing of a rocking podium: Results of a blind prediction contest. *Earthquake Engng Struct Dyn*. 50: 1043–1062.
- Vlachakis, G., Giouvanidis, A. I., Mehrotra, A., and Lourenço, P. B. (2021). Numerical block-based simulation of rocking structures using a novel universal viscous damping model. *Journal of Engineering Mechanics*, 147(11), 04021089. doi.org/10.1061/(ASCE)EM.1943-7889.000198
- Zaki, M., Zhong, C. and Christopoulos, C. (2025), Analytical Study on the Predictability of Controlled Rocking Frame and Podium Systems. *Earthquake Engng Struct Dyn.*, 54: 1558-1578.
- Zhang, Z., Lachanas, C. G., Vamvatsikos, D., Zhou, Y., and Vassiliou, M. F. (2025). A comprehensive investigation into the effect of vertical component of ground motion on response of sliding unanchored blocks. *Engineering Structures*, 323, 119251.
- Zhu, H., Aloisio, A., Bezabeh, M.A. (2026). Development, validation, and application of reduced-order numerical models for post-tensioned cross-laminated timber rocking walls. *Structures*, 86, 111397
- Zhong, C and Christopoulos, C. (2021) Finite element analysis of the seismic shake-table response of a rocking podium structure. *Earthquake Engng Struct Dyn*. 50: 1223–1230.
- Zhong, C and Christopoulos, C. (2023) Seismic response of slender MDOF structures with self-centering base shear and moment mechanisms. *Earthquake Engng Struct Dyn*. 52: 1658–1677.