RESEARCH ARTICLE

Shaking table tests of a resilient bridge system with precast reinforced concrete columns equipped with springs

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Abstract
This paper presents the shake table test results of a novel system for the design of precast reinforced concrete bridges. The specimen comprises a slab and four precast columns. The connections are dry and the columns are connected to the slab by an ungrouted tendon. One of the tendon ends is anchored above the slab, in series with a stack of washer springs, while the other end is anchored at the bottom of the column. The addition of such a flexible restraining system increases the stability of the system, while keeping it relatively flexible allowing it to experience negative post-uplift stiffness. It is a form of seismic isolation. Anchoring the tendon within the column, caps the design moment of the foundation, and reduces its size. One hundred and eighty-one shake table tests were performed. The first 180 caused negligible damage to the specimen, mainly abrasion at the perimeter of the column top ends. Hence, the system proved resilient. The 181st excitation caused collapse, because the tendons unexpectedly failed at a load less than 50% of their capacity (provided by the manufacturer), due to the failure of their end socket. This highlights the importance of properly designing the tendons. The tests were used to statistically validate a rigid body model. The model performed reasonably well never underestimating the median displacement response of the center of mass of the slab by more than 30%. However, the model cannot predict the torsion rotation of the slab that was observed in the tests and is due to imperfections.

KEYWORDS
disc springs, precast bridges, recentering systems, rocking, seismic isolation, shake table testing

1 | INTRODUCTION

Modern bridge design should fulfill a number of requirements. Modern bridges should often (a) be constructed quickly; (b) be resilient, that is, not only avoid collapse, but also suffer minimal or no damage under the design (or even a larger) earthquake.
Prefabrication can conform to the above requirements and offers a number of advantages, namely (a) reduced overall construction time, (b) reduced on-site construction time, and (c) better quality control and improved safety for the workers. In places where labor cost is high, prefabrication can be more competitive than on-site construction, as it requires less labor. On the other hand, prefabrication requires well-qualified labor, and the transport of large elements might require special vehicles.

Prefabrication has not been widely used in seismic regions because the state of the practice has been to try to emulate cast-in-place structures by connecting prefabricated elements with cast in situ concrete, in an effort to create monolithic connections—and this is neither always cost-effective nor presents optimal behavior. However, research and, recently, practice has proven that there are alternative design and construction methods that, apart from preventing collapse, they also provide re-centering. Engineers in New Zealand, the United States, and China have suggested connecting precast elements via ungrouted post-tensioned tendons, forming dry connections. Mild reinforcement can be added to provide extra energy dissipation. The seismic behavior of such systems is superior not only to precast elements with emulated connections, but also to cast-in-place systems. The reason is that the elongation of the reinforcement is distributed along all its length and therefore it does not yield. Hence the tendon offers a recentering mechanism and the structure does not exhibit damage. In fact, the widespread belief that the Achilles’ heel of precast structures is their seismic behavior is wrong: precast structures can be designed to be more resilient than cast in place.

The above concept is based on the early work of Priestley and Tao,1 Stanton et al.,2 and the PREcast Seismic Structural Systems (PRESSS) project.3,4 Different names have been used for similar concepts: damage avoidance design,5 controlled rocking,6–13 self-centering system,14–24 precast hybrid systems,25–29 hybrid sliding–rocking system,30–33 pre- or post-tensioned rocking,34–36 and it has recently found its way to practice in New Zealand37 and China.38

Soviet engineers have been using prefabricated elements dried connected as a seismic isolation method for buildings as referred elsewhere.39–43 The concept has a seismic behavior similar to the one of ancient Greco-Roman temples.44–49

All the above references focus on the superstructure. However, in seismic-prone regions, the foundation of conventional bridges can comprise up to 50% of the reinforced concrete used for the project. The design dogma that the superstructure should not be allowed to uplift often results in large design moments for the foundation that governs its size. This paper claims that this dogma is not necessary, hence it results in unnecessarily large pile foundations. It presents shake table tests of a slab supported on four precast columns. Ungrooved restraining tendons in series with washer springs (also known as Belleville washers or Belleville springs) connect the columns with the slab. The columns are freely supported on the shake table, that is, the tendon is anchored within the column, so that the design moment of the foundation is minimized (Figure 1A,B). The flexibility of the restraining system is governed by the flexibility of the springs, which make the system more deformable and reduce the design forces, at the expense of increasing displacements. To clarify the concept, Figure 2A,B shows such a column that was previously tested cyclically in the ETH Zurich.50

To the authors’ knowledge, this is the first paper that discusses shake table tests of a system that uses such a spring system for isolation. The proposed concept can be perceived as seismic isolation51–53 for precast structures. However, a detailed cost/performance comparison of conventional seismic isolation (i.e., spherical sliding or rubber bearings, which also reduce the design shear and moment of bridge piers) and the suggested approach lies beyond the scope of this paper.

Figure 1: Schematic drawing of specimen in (A) 3D View; and (B) Front View.
2 | MECHANICS OF THE ROCKING FRAME

2.1 Static planar behavior

Figure 3 presents two variations of a rocking frame comprising $N$ rigid columns of total mass $N \times m_c$ and a rigid beam of mass $m_b$. The columns are able to uplift but not slide at any end. They are restrained with a perfectly elastic restraining system. In both variations, the top end of the tendon is anchored above the beam, in series with a spring. The total stiffness of the restraining system (i.e., spring + tendon) is $k_{\text{res}}$. In Figure 3A, the bottom end of the tendon is anchored at the foundation, while in Figure 3B, it is anchored at the bottom end of the column.

Assuming a horizontal force $F$ applied at the beam, the linearized lateral force–deformation relation (“pushover curve”) when the frame is anchored at the foundation is:

$$F = \left(\frac{1}{2} + \gamma\right) N m_c g \alpha \operatorname{sgn}(u) + \left(\frac{2 N k_{\text{res}} b \alpha}{2} - \left(\frac{1}{2} + \gamma\right) N m_c g\right) \frac{u}{2h} \tag{1}$$

where $\gamma$ is defined in Figure 3.

When the tendon is anchored within the column (Figure 3B), the linearized pushover curve becomes:

$$F = \left(\frac{1}{2} + \gamma\right) N m_c g \alpha \operatorname{sgn}(u) + \left(\frac{N k_{\text{res}} b \alpha}{2} - \left(\frac{1}{2} + \gamma\right) N m_c g\right) \frac{u}{2h} \tag{2}$$
The stiffness originating from the restraining system is four times larger if the tendon is anchored in the foundation, rather than within the column. Figure 4A plots Equations (1) and (2) for different values of \( k_{res} \). For all values of \( k_{res} \), the system presents a bilinear elastic behavior. There is no hysteresis, and unloading follows the same branch.

When \( k_{res} = 0 \) (i.e., no tendon), the post-uplift stiffness of the system is negative. Collapse is reached not because of material failure, but when the restoring force becomes zero, that is, when the columns reach the point of neutral equilibrium. This defines the displacement capacity.

Adding a non-prestressed tendon algebraically increases the post-uplift stiffness. When the stiffness remains below \( k_{crit} \), where \( k_{crit} = \frac{(1+2\gamma)m_{cg}}{4b\alpha} \) or \( k_{crit} = \frac{(1+2\gamma)m_{cg}}{b\alpha} \) depending on whether the tendon is anchored within the foundation or at the column, the post-uplift stiffness of the system remains negative, the slope of the second branch is milder and the displacement capacity increases.\(^{54,55}\)

When the stiffness of the tendon becomes larger than \( k_{crit} \), the post-uplift stiffness becomes positive and the system never becomes instable, given the assumption of no sliding.

By choosing appropriate restraining systems, the pushover curves of the two variations of the rocking frame can be made identical. This is not the case for the base moment—top displacement curve: when the tendon is anchored in the foundation, the base moment increases with displacement (Figure 4B). However, if the tendon is anchored within the column, the base moment is capped and equal to the load carried by the column times its halfwidth (Figure 4C). As the goal is to reduce the base moment, this paper will focus only on the latter case.

The above analysis is highly idealized because:

a. Usually, a restraining system comprises solely a tendon that has a finite yield strain, and no spring is used. For a tendon strength of 1800 MPa, the yield strain is \( 9 \times 10^{-3} \). Assuming a bridge column of \( 9.6 \times 1.6 \) m, rigid body analysis shows that the tendon will yield at a drift ratio of 11%, if the tendon is anchored in the bottom of the column and 5.5%, if it is anchored in the foundation.

b. The column is not rigid, but will sustain flexural deformation along its length and local deformation at the column—foundation and column–cap beam contact zones. If a relatively stiff tendon is used and the column ends are not protected, the deformation will cause concrete spalling.

c. There might be sliding at the interfaces between the column and the cap beam or foundation.

To test the validity of the static analysis presented above, cyclic tests on a 1:5 scaled precast RC column (not of the same dimensions as the one used for the shake table tests) were performed in the laboratory of the Institute of Structural Engineering (IBK) of the ETH Zurich.\(^{50}\) To avoid damage to the column, its ends were protected with steel jackets. The restraining system comprised unbonded tendons in series with disc springs (Figure 2A,B). The column was subjected to a normalized axial load of 52.2 kN that would correspond to a normalized axial load \( (v = N/(A_t \times f_{cd})) \) of 5% and then laterally loaded in cycles up to 15% drift. No damage was observed and the force–deformation loops (Figure 2C) showed no deterioration. The energy dissipation implied from the loops is due to friction of the setup, not damage to the column. Note, that the columns were not identical to the ones tested on the shake table due to lab constraints.
2.2 | Dynamics of the planar rocking frame

Based on Makris and Vassiliou, and assuming that the columns are always in contact with the ground (i.e. they never fly), the equation of motion of the restrained rocking frame of Figure 3B is

\[ \ddot{\theta} = -\frac{1+2\gamma}{1+3\gamma} p^2 \left( \sin(\alpha \text{sgn}\theta - \theta) + \frac{u_g}{g} \cos(\alpha \text{sgn}\theta - \theta) \right) - \frac{1}{1+3\gamma} p^2 \sin\alpha \frac{k_{res}}{m_c g} \sin \theta \]

(3)

where \( p = \sqrt{\frac{3g}{4R}} \) and \( \theta \) is the tilt angle shown in Figure 3.

Linearizing Equation (3) gives

\[ \ddot{u} + \frac{1+2\gamma}{1+3\gamma} \frac{3g}{2} \alpha \text{sgn}u \left( \begin{array}{c} \frac{3}{2} \frac{1}{1+3\gamma} \frac{a k_{res}}{m_c} - \frac{3g}{2} \frac{1+2\gamma}{1+3\gamma} \end{array} \right) \frac{u}{2h} = -\frac{3}{2+1+3\gamma} \dot{u}_g \]

(4)

Unless extra yielding reinforcement is provided internally or externally, energy is only dissipated during impact, and it is usually taken into account via a coefficient of restitution defined as

\[ r = \frac{\dot{\theta}_{after}}{\dot{\theta}_{before}} \]

(5)

When the stiffness of the restraining system is positive, extra damping can be provided to the system in the form of extra yielding bars, and the force deformation loop takes a characteristic flag shape. Then the dynamic response of the system can be approximated by an equivalent elastic oscillator, and the design follows the standard elastic spectrum-based approach.

However, when the stiffness of the bilinear oscillator is negative, there is no equivalent elastic system and the elastic spectrum cannot be used. To avoid time history analysis, Reggiani Manzo and Vassiliou proposed simplified design methods.

2.3 | Extension in three dimensions

Figure 5 shows a 3D extension of the rocking frame model. The assumptions made for the planar frame (rigid bodies, no sliding or “flying” allowed, pointwise contact) are extended to include the following:
The columns are constrained not to roll-out of its initial position. The columns are always in contact with the support and the slab (i.e., they never fly). Therefore, the contact force is always compressive. No damping mechanism is included. The tendon is anchored above the spring and within the column.

Then the equations of motion become

\[ \ddot{\theta} = -\dot{p}^2 \left( \sin(\alpha - \theta) + \cos(\alpha - \theta) \left( \frac{\ddot{u}_g}{g} \cos \varphi + \frac{\dot{u}_k}{g} \sin \varphi \right) + \frac{k_{res} \cdot 2h}{m_\gamma g} \frac{1}{2 (2\gamma + 1)} \sin \alpha \tan \alpha \sin \theta \right) \]

\[ + \varphi^2 \left( \sin \theta \sin^2 \alpha \left( 4\gamma + \frac{3}{2} \right) \right) + \cos \alpha \sin \alpha \left( 4\gamma + 1 \right) \left( 1 + \cos \theta - 2\cos^2 \theta \right) \]

\[ \left( \frac{1}{4} \cos^2 \alpha - \frac{5}{4} \sin^2 \alpha + 4\gamma \left( \cos^2 \alpha - \sin^2 \alpha \right) \right) \sin \theta \]

\[ + \left( 3 + 8\gamma \right) \left( 1 - \cos \theta \right) \sin^2 \alpha + 2 \sin \alpha \cos \alpha \sin \theta \left( 1 + 4\gamma \right) \left( 1 - \cos \theta \right) \]

\[ \left( 2\gamma + 1 \right) \left( \sin \alpha + \sin(\theta - \alpha) \right) \left( \frac{\ddot{u}_k \sin \varphi - \ddot{u}_g \cos \varphi}{R} \right) \]

\[ \phi \cdot \dot{\theta} \]

\[ = (2\gamma + 1) \left( \sin \alpha + \sin(\theta - \alpha) \right) \frac{\ddot{u}_k \sin \varphi - \ddot{u}_g \cos \varphi}{R} \]

\[ \text{where} \]

\[ \dot{p}^2 = \frac{12 (2\gamma + 1)}{48\gamma + 15 + \cos^2 \alpha} \frac{g}{R} \]

\( \theta \) is the tilt angle. \( \varphi \) is the angle defining the contact point, \( T \), between the column and the support (Figure 5).

3 | SHAKE TABLE TESTING

A series of tests was performed at the Laboratory for Earthquake Engineering (LEE) of the National Technical University of Athens (NTUA). The objectives of the tests were two: (a) to evaluate the seismic performance of a precast system based on the concept of Section 2 and (b) to validate the numerical model presented in Section 2.3. The tested specimen consisted of four reinforced concrete columns supporting a reinforced concrete slab (Figure 1). The columns were connected to the slab via unbonded non-prestressed restraining tendons in series with washer springs, but were freely standing on the shake table. The end of each tendon was anchored to the bottom of its corresponding column using a threaded socket at the base.

3.1 | Column design and casting

The model was designed around the limitations of the shake table of the LEE and was not designed to represent a specific prototype bridge, as this was impossible at a scale larger than or equal to 1:5. However, at a 1:5 scale, the tested specimen will have a height of 7.25 m in the prototype scale that falls within the range of typical heights of columns of highway bridges, although on the lower end. The column aspect ratio (height to width) was slightly larger than typical highway bridges. The aspect ratio and the height chosen were controlled by the limitations of the shaking table. Moreover, the slab
had a mass that leads to a normalized load in the columns of $\nu = \frac{N}{(A_c \cdot f_{cd})} = 3.3\%$, where $N$ is the axial load and $A_c$ and $f_{cd}$ are the column cross-sectional area and the design compressive strength of the concrete, respectively. This value is larger than the value used by Mashal and Palermo$^{12}$ and Mashal$^{64}$ who tested a bridge bent of a typical highway bridge in New Zealand. It lies within the typical design range of overpass bridges, but still is relatively low. Such a low value was dictated by the capacity of the shake table. So, more tests with larger loads are needed.

Figure 6 shows an elevation of the columns and their critical sections. The columns were 1450-mm high and 197 mm in diameter. The ends of the columns were protected with steel jackets.

Longitudinal reinforcement was provided by 6 $\phi 8$ B500 rebars that were welded to the top and bottom jacket plates. This resulted in a reinforcement ratio of $\rho_l = 1.00\%$. Transverse reinforcement was provided by a spiral wire of 2.4-mm diameter and 400-MPa yield strength. This creates a distortion in the physical model as the steel is annealed and smooth, unlike standard rebars used in construction. However, the shear behavior is not expected to be critical and confinement to the critical sections is anyways provided by the steel jackets. The pitch of the spires was 52 mm providing a transverse reinforcement ratio of $\rho_s = 0.23\%$. S355 steel was used for the protection of both ends of the columns.

The columns were cast in the Laboratory of Reinforced Concrete of NTUA. The concrete was sampled in cubes of 100-mm edge, and the average compressive strength at the age of 28 days was measured at $44.5 \pm 0.74$ MPa. Therefore, the characteristic value of concrete’s compressive strength was estimated at $f_{ck} = 41.2$ MPa, classifying the concrete at the C30/37 strength class. The Young’s modulus of the specimen is considered to be 33 GPa, according to EC2.$^{65}$ The column was cast from the top and compacted with a vibrator that barely passed through the opening between the top steel jacket and the tendon duct. To avoid forming voids right below the end steel plate, the mixture was manually pushed towards the perimeter. Therefore, it is recommended that either the columns be casted from the lateral side,$^{34}$ or that the initial high workability of the mixtures be retained during the casting time, using the appropriate dosage of retarder.

The ends of columns were protected with a tube having 5-mm thickness, 140-mm height, and 197-mm external diameter (Figure 6). The steel tube was welded at its base to a steel plate of 10-mm thickness. No particular methodology was used for the design of the tube, as existing methodologies are based on the beam theory, which is not applicable in this case, because of the strong discontinuity caused by uplifting. To dimension the steel jacket, the jacket tested by Thonstad et al.$^{35}$ was approximately scaled down from 1:2 to 1:5 scale. The detailing of the steel plate is different in the bottom and top part of the column to allow for casting from the top. A threaded steel tube (Figure 6) was welded at the bottom steel plate.
3.2 End plates

In order to avoid damage to the slab or the column ends due to stress concentrations, S355 steel plates were placed at the interfaces of the column with the slab and the shake table. The steel plates were equipped with restrainers that limited sliding to 5 mm in each direction (Figure 7).

3.3 Restraining system

The model was designed to keep the post-uplift stiffness negative while providing a displacement capacity (i.e., displacement to reach the point of neutral equilibrium) of $u_{\text{cap}} = 4b = 394$ mm, which corresponds to doubling the displacement capacity of the equivalent unrestrained system. Using this value and Equation (2), the target stiffness calculated for the restraining system was $k_{\text{res}} = 1,808$ kN/m.

To obtain the target stiffness, the restraining system was composed of an unbonded tendon in series with washer springs. The tendon cross section was dictated by the market availability of materials. Tendons with threaded connections at their ends (Figure 8A) were only available as seven-wire strands with nominal cross-sectional area of 150 mm$^2$ and Young’s Modulus of 190 GPa. The tendons were 2134-mm long, resulting in an axial stiffness of 13,318 kN/m. One end was anchored at the bottom of the column using a threaded tube welded to the steel jacket and the other one was anchored at the top of the springs with a nut.
The spring system consisted of washer springs, also known as disc springs or “Belleville springs.” Washer springs are conical shells loaded along their axis. Similar to helical springs, they can be combined to achieve a desired total stiffness. However, unlike helical springs, the stacking direction of the springs affects the total stiffness. Stacking the washer springs in the same direction results in a stiffer system, while stacking them in alternate directions makes the system more flexible. An analytical expression describing the force–deformation relationship of a single spring was derived by Almen and Laszlo. The springs can exhibit hardening, softening, or linear behavior, depending on their geometry.

The washers of the spring system were not custom-made, because they are available in the market at specific dimensions and their manufacturer provides the force–deformation curve of each washer based on Almen and Laszlo. So, the design of the stack of the springs involved determining the target stiffness and displacement capacity and choosing the appropriate size and number of washers from the manufacturer’s catalog: ten washer springs of 125 × 61 × 6 mm (external diameter/internal diameter/thickness) were stacked in alternate directions (Figure 8B). Each washer could deform to a maximum of 3.6 mm, corresponding to a load of 64 kN. Figure 8C plots the force–deformation relation of the spring stack, as predicted by Almen and Laszlo. At the target displacement capacity of the specimen, the spring stack deforms by 21.5 mm and its secant stiffness is 1975 kN/m. Figure 8C also presents the force–deformation curve of the restraining system, assuming linear behavior of the tendon. At the displacement capacity of the specimen, the secant stiffness of the restraining system was 1720 kN/m, 5% smaller than the target stiffness of 1808 kN/m.

As shown in Figure 6, there was a gap of 8.7 mm between the tendon and tendon duct. The large internal diameter of the tendon duct (33 mm) was necessary for passing the 30-mm diameter threaded sockets through the duct. Although unavoidable, this detailing resulted in a different lateral force–displacement relation for the specimen than what Equation (2) predicts. Figure 9 presents the two curves: the one predicted by Equation (2), assuming that the tendon does not move from the centerline of the duct and one that considers that the tendon will move away from the duct’s centerline until it touches the duct’s wall (for the derivation, please refer to Appendix 2). The adjusted curve presents a displacement capacity of 333 mm, 11% smaller than the predicted displacement capacity of 374 mm. In the future, it is recommended that a spacer be fixed in the top of the end of the duct to guarantee that the tendon will stay in the duct’s centerline.

3.4 | Redundant mechanism

The specimen was designed with a redundant mechanism to prevent the toppling of the columns while protecting the tendons from exceeding their capacity. The redundant mechanism, “stoppers”, consisted of two M12 threaded rods of Grade 4.8 placed in parallel with each stack of springs (Figures 8B and 10A,B). The stoppers engage only if the slab experiences large displacements. In this case, the load in the restraining system increases. To protect the tendons, which could not be easily replaced without disassembling the specimen, the stoppers were designed as sacrificial elements, yielding at a load of 54.0 kN. In this experimental campaign, the stoppers engage after a 15-mm deformation of the stack of springs. Using the model that takes into account the gap between the tendon and the duct, this deformation of the spring system corresponds to a displacement of the slab of 280 mm. The force at the tendon when the stoppers engage is computed to be 31.2 kN. Therefore, when they yield, the tendon force is 85.2 kN, which is smaller than the yielding load of the tendons (237 kN).
4 TEST SETUP AND EXCITATIONS

4.1 Shaking table specifications

The tests were performed in the 4 × 4-m six degree-of-freedom shaking table of NTUA. The maximum stroke and velocity of the simulator in the three axes are ±100 mm and 1000 mm/s, respectively. Under 10 tons of payload, the maximum acceleration of the horizontal axes is 2g, while a 4g acceleration is permitted in the vertical direction.

4.2 Construction

Figure 11A–D shows key stages of the construction process. The steel jackets were fabricated in a workshop and the reinforcement was welded to them. The spiral was placed by hand, the tendon was screwed to the bottom threaded tube, and a steel duct was welded around the threaded tube. The columns were cast in the LRC of NTUA using carton formwork and
48 days after casting, when the concrete had developed an average compressive strength of $47.6 \pm 2.4$ MPa, the columns were placed on the shake table, and the slab was placed on top with a crane. Then, the spring stack was placed and the top nut was screwed at the top end of the tendon. Safety steel columns (in red in Figure 11C,D) were installed on the shake table to prevent crashing of the slab onto the shake table, in case of specimen collapse. A minimal prestress of $2$ kN was applied to the columns to ensure that they are not loose.

### 4.3 Excitation selection and scaling

Usually, shake table tests are performed under a handful of ground motions of increasing intensity because the tests are expected to cause damage to the specimen. However, the structure discussed in this paper is designed not to suffer from any major damage. Therefore, it can be tested under multiple ground motions, thus allowing for a better understanding of its behavior and for the evaluation of its resilience. Moreover, testing under sets of multiple ground motions allows for numerical models to be evaluated in the statistical sense, that is, be evaluated according to their ability to predict the cumulative distribution function (CDF) of the time maxima of the responses to sets of ground motions that characterize the seismic hazard.\textsuperscript{57–71}

Therefore, the specimen was subjected to all three sets of ground motions proposed by FEMA P695\textsuperscript{72} (near-field pulse-like (NFP), near-field no pulse-like (NF), and far-field (FF)). The sets are composed of $14$ NFP, $14$ NF, and $22$ FF ground motions. In addition to the ground motions proposed by FEMA, an extra $12$ NFP, $12$ NF, and $21$ FF ground motions were included. According to FEMA, the original FF set of ground motions includes Cape Mendocino, Rio Dell Overpass record. However, this was not available in the PEER ground motion database and, thus, was not included in this study.

In total, this experimental campaign consisted of $26$ NFP, $26$ NF, and $42$ FF ground motions. The specific records, along with the adopted numbering scheme, are provided in Table A1 of the appendix for reference.

All sets of ground motions were scaled to have the same PGV defined as

$$\text{PGV} = \sqrt{(PGV_x)^2 + (PGV_y)^2}$$ \hspace{1cm} (9)

where $PGV_x$ and $PGV_y$ are the peak ground velocities along the $x$ and $y$ directions. According to the capacity of the shake table, two different PGV levels were selected (in the model scale): (a) low PGV = 16.75 cm/s and (b) high PGV = 33.5 cm/s. Overall, there were three different ground motion sets (NFP, NF, and FF) scaled at two different PGVs resulting in 188 ground motions. The ground motions are provided in the Appendix, together with their PGA, after they were scaled to the high PGV. The longitudinal, lateral, and vertical components of each ground motion were simulated by the shake table.

The specimen was constructed assuming a 1:5 scale for the column height. Therefore, to preserve similitude of time, the excitations were scaled in time to $S_T = \sqrt{S_L}$, in which $S_L$ is the length scale equal to 1/5. Therefore, the PGV of the high- and low-intensity sets in the prototype scale is 75 and 37.5 cm/s, respectively. Figure 12A–L plots the linear 5% damped spectra for the excitations, as they were recorded on the shake table. It also includes the design spectrum for a site in Athens, Greece (in model scale), assuming soil type C and importance factor 1, according to EC8.\textsuperscript{73}

The order of the tests was defined based on the system’s response predicted using the analytical model proposed by Vassiliou,\textsuperscript{63} sorting them from low to high expected response. Some ground motions could not be simulated either because the structure failed before its execution or because the shake table could not reproduce them sufficiently accurately. The loading protocol is also given in the Appendix.

### 4.4 Instrumentation

Figure 13 presents the location of all instruments installed on the specimen. The horizontal displacements of the slab were measured using draw-wire sensors, the vertical displacements using cable-extension position transducers, and the accelerations using accelerometers. Load cells were also installed in series with the tendons of each column Figure 8B. The deformation of the spring system of column C2 was recorded using a string potentiometer. Furthermore, the sensors integrated into the actuators of the shaking table recorded the displacement and acceleration of the table platform.
FIGURE 12  FIGURE Linear spectra ($\zeta = 5\%$), as recorded on the shake table
5 | RESPONSE QUANTITIES

The horizontal displacement of the slab and the base shear of the system were the two quantities of interest in this experimental campaign.

5.1 | Horizontal displacement of the deck

Assuming that the slab is a rigid body and neglecting vertical displacements, the displacement and torsion at the center of mass of the slab were calculated using the elongation/shortening measured by the four-string potentiometers fixed to it. The horizontal projection of the displacement of the center of mass of the slab is defined as $U$. The specimen was symmetric and the slab was not supposed to sustain torsion. However, due to small imperfections, torsion could be observed during the tests. $U$ is not affected by the torsion of the slab. This slab torsion needs further experimental study, as a real bridge would most likely have eccentricity by design, so more tests need to be performed with an eccentric mass placed on top.

5.2 | Base shear of the system

The base shear of the system was estimated based on the inertial resistance of the slab

$$V_{base}(t) = m_{slab} \cdot \left( \sqrt{\ddot{u}^2_{x,slab}(t) + \ddot{u}^2_{y,slab}(t)} \right)$$

in which $m_{slab}$ is the mass of the slab, $\ddot{u}_{x,slab}$ and $\ddot{u}_{y,slab}$ are the total longitudinal and lateral acceleration of the slab at its center of mass.

The mass of slab was measured during assembling of the system and it weighed 9435 kg (including the concrete, the steel plates, the springs, and the mounting screws).

6 | INSIGHT ON THE BEHAVIOR

To gain insight in the response, this section presents the results of two tests that caused significant displacements: the 1940 El Centro Array #6 record (Exc. ID 1) and the 1999 Duzce, Turkey record (Exc. ID 14), both belonging to the high-intensity NFP set of ground motions (Figure 12). The former did not engage the stoppers, while the latter did.

Figure 14A shows the recorded base shear–drift ratio response. The drift ratio was defined herein as the ratio between the $U$ displacement of the slab and the height of the columns (1450 mm). The system has an initial branch with positive stiffness, which is followed by a second branch with negative stiffness. The finite stiffness of the initial branch sources from the system’s pre-uplift flexibility that is ignored by rigid body models. The system’s uplifting force can be estimated
FIGURE 14 (A) Base shear–drift response ($U$/column height); (B) time history of the loads recorded in the restraining system of each column; (C) time history of the slab displacement right above each column; (D) time history of twist rotation (torsion) of the slab; (E) the time history of the absolute horizontal displacement of the slab; (F) force–deformation relation of the stack of springs composing the restraining system of column C2
around 10 kN, which is 22% lower than the value predicted by the rigid body approximation (12.9 kN). Apparently, the pre-uplift flexibility influences the strength of the system. Figure 14A (right) shows that the base shear reaches a plateau at a drift ratio of 20%, which indicates that the stoppers of at least one of the columns engaged. Their engagement was visually confirmed after the test, even though the stoppers did not yield (in all but the 181st test, which cause collapse).

Figure 14B shows the time history of the loads recorded in the restraining system of each column. The tendons of the columns clearly exhibit different loads. The main reason for this is the torsion of slab that causes each column to displace differently. This is shown in Figure 14C that plots the time histories of the displacements of the vertical projections of the center of the column cross sections at the top surface of the slab. Figure 14D plots the twist rotation (torsion) time history. Figure 15 shows some snapshots of the horizontal position of the slab, during the 1940 El Centro Array #1 ground motion. Evidently, columns exhibit a different displacement because of the torsion of the slab.

Figure 14E presents the time history of the horizontal displacement of the center of the slab (U) for both tests. It also presents a numerical prediction of the time history, discussed in Section 8. It shows that because of torsion, U can be much larger than the maximum displacement of the columns. However, torsion on its own cannot justify such large differences in the tendon forces, because the load differences (Figure 14B) seem to be larger than the displacement differences (Figure 14C). Part of the discrepancy can be attributed to some tendons having slack after the top nut becoming loose and to the tolerances of the restraining system being larger than necessary. Therefore, in future tests, the tendon should be checked for becoming loose, and smaller construction tolerances should be applied.

Moreover, Figure 14B (right) shows that when the system was subjected to the 1999 Duzce, Turkey record, at around 5–6 s the forces in the restraining systems of columns C2 and C3 started becoming significantly larger than the forces of the tendons of the other columns, indicating that the stoppers of these two columns engaged. Not all columns engaged simultaneously, because of the torsion of the slab.

Figure 14F plots the force–deformation relation of the tendon of column C2 as recorded and as predicted by Almen and Laszlo’s equations. The plot in the right presents further evidence that the stoppers of column C2 engaged: at a deformation of around 15 mm, the system becomes much stiffer. As mentioned in Section 3.4, the stoppers were designed to provide redundancy to the system, to prevent collapse under extreme events, even at the cost of permanent deformation. In this particular test, they were successful, as they prevented collapse, even though there was a permanent displacement, not in the form of damage but in the form of the columns ending up tilted and being held by the stoppers. The specimen was recentered using the lab crane. The tendon was not damaged. In the other tests where the stoppers engaged, the system was still able to recenter—with the exception of the case where there was collapse (see Section 7.2).

The recorded force–deformation curves in both tests show that the stack of springs dissipates some minimal energy. Possible sources of energy dissipation are friction of the springs with the external guiders and friction of the spring with the end plates. A comparison of the experiment and analytical curve shows that the analytical curve predicts the behavior of the spring reasonably well, despite its inability to model the minimal damping that was experimentally observed.

7 EXPERIMENTAL RESULTS

This section collectively presents the results to all 181 excitations.
7.1 Measured displacement and base shear response

Figure 16 presents the maximum drift ratio of the slab (\(U/1450\) mm) for each excitation in ascending order. This was not the order by which the tests were performed. Different curves are shown for low and high-intensity sets of excitations. Some of the planned excitations were not run and the results for these excitations are not shown in the plots.

The system experienced drift ratios smaller than 5% when subjected to most of the low-intensity excitations. The stoppers engaged for the motions 14 and 16 of the high-intensity NFP set and for the motions 72 and 88 of the high-intensity FF set.

The maximum twisting (torsion) angle observed during the tests was 0.115 rad (6.59°) for the motion 72 of the high-intensity FF set.

7.2 Eventual collapse

The system eventually collapsed when it was excited by ground motion 1987, Superstition Hills-02 (Exc. ID 88) scaled to the high-intensity set. The collapse was caused by the unexpected failure of the tendon of column C1 that was followed by the failure of tendon of column C2. The collapse was unexpected because the tendons failed prematurely: C1 broke at 98.8 kN and C2 at 105.4 kN. Both tendons failed at their sockets, indicating some unexpected stress concentration during
7.3 Observed damage and performance of the slider restrainers

The specimen was visually inspected after each excitation and after an eventual collapse. No cracks and concrete spalling could be observed in the columns and steel jacket. The only observed damage was abrasion of the edges of the top (Figure 17A) and bottom (Figure 17B) steel jacket. Figure 17 shows that the abrasion of the top steel jacket is more pronounced than the abrasion of the bottom one—something expected as the top end of the column develops a larger bending moment. What looks like damage in the top steel jacket of C4 was made with the cutting wheel while cutting the duct that was sticking out of the column after casting. Interestingly, the abrasion was not caused by any impacts, but by the forces that were developed at large displacements, when the stoppers engage. Therefore, the steel jacket was enough to protect the specimens from the impacts, and what governs damage and design is the maximum moment applied at the top cross section, which occurs under maximum displacement.

The sliding restrainers managed to restrain sliding in all cases (but the one that the system collapsed). In several cases, the columns impacted on the restrainers, in some cases, they climbed up their inclined surface, but eventually, they slid back to the bottom steel plate.

8 Statistical validation of a rigid body model

Seismic response is inherently stochastic. Therefore, Almen and colleagues have claimed that, in earthquake engineering, structural model validation should follow a stochastic procedure. More specifically, the validation test of predicting the response to an individual ground motion is sufficient for a structural model, but it is not necessary. Models need to be able to predict the CDF of the time maxima of the responses to a set of ground motions that characterize the seismic hazard. This is a weaker but sufficient validation procedure. It is often the only possible one, as shake table tests are often not repeatable—especially for rocking structures. In such cases, trying to predict the response to an individual ground motion is meaningless.
### TABLE 1 Evidence classification p-value scale

| p-value | Evidence         |
|---------|------------------|
| <0.01   | Very strong against $H_0$ |
| 0.01–0.05 | Strong against $H_0$ |
| 0.05–0.10 | Medium–weak against $H_0$ |
| >0.10   | Small or none against $H_0$ |

### TABLE 2 Statistical comparison of the analytical model and the experimental data

| Set                     | p-value | $\epsilon_{50}$ |
|-------------------------|---------|------------------|
| Low-intensity (All)     | 0.75    | 0.04             |
| High-intensity (All)    | 0.14    | 0.24             |
| Low-intensity (NFP)     | 0.06    | 0.29             |
| High-intensity (NFP)    | 0.86    | 0.02             |
| Low-intensity (NF)      | 0.65    | 0.09             |
| High-intensity (NF)     | 0.59    | 0.20             |
| Low-intensity (FF)      | 0.26    | 0.09             |
| High-intensity (FF)     | 0.15    | 0.47             |

Abbreviations: FF, far field; NF, near field; NFP, near-field pulse-like.

This section attempts to statistically validate the model presented in Section 2.3 (without taking into account the nonlinearity of the spring or the presence of the duct) by comparing its predictions to the shake table tests results. The analytical model assumes that the columns do not slide or twist. In addition, the model also does not account for torsion of the slab, which was observed in this experimental campaign and is generated by any small imperfections in the specimen. Thus, it is a very simple model—to the point of being simplistic, as it grossly overestimated the response of a rocking system in the PEER 2019 blind prediction contest.\(^69,75,76\) With reference to Equations (7) and (8) and Figure 5, the model parameters that were used are: $2R = 1.463$ m, $\alpha = 0.1359$, $m_c g = 1.22$ kN, $m_s g = 94.35$ kN, and $k_{res} = 1720$ kN/m. Indicatively, Figure 14E (left) shows that the model underpredicts the response to this individual ground motion by roughly 35%.

Figure 18 presents the experimental and numerical CDFs of the displacement of the center of mass $U$. Figure 18A presents the data clustered in bins of low and high intensity, while Figure 18B-D presents bins of different excitation types (NFP, NF, FF). Figure 18 also plots 95% confidence intervals of the experimentally obtained $U$. The model performs reasonably well, as it generally lies within the 95% CI. To statistically validate the analytical model, a two-sample Kolmogorov–Smirnov test was conducted using the built-in Matlab routine “kstest2.”\(^77\) This test rejects or accepts the null hypothesis ($H_0$) that both data are from the same distribution. The null hypothesis $H_0$ is rejected when the p-value is lower than a given statistical significance value $\alpha_s$. The p-value is a measure of the evidence against $H_0$ and it does not represent the probability that $H_0$ is true.\(^78\) In this work, a fairly large value of statistical significance of 0.1 is used to allow for a nuanced qualification of null hypothesis validity using an evidence classification scale shown in Table 1. A detailed explanation of the two hypothesis testing procedures can be found in Wasserman.\(^78\) Table 2 provides the p-values for all eight sets shown in Figure 18. The numerical CDF is compared to the experimental CDF of $U$. In 7/8 cases there is small to none evidence against both data coming from the same distribution, while in 1/8 there is medium–weak. Hence, this statistical test shows that the analytical model is a good predictor of the response of the center of mass of the slab.

As p-values are not often used in earthquake engineering practice, Table 2 also presents the error of the median of $U$. In 7/8 cases, the error lies below 30%. In the high-intensity FF, the error is 47%—but the model is conservative. In no case did the model underestimate the median response by more than 30%. However, as the model disregards a number of physical mechanisms (e.g., energy dissipation, flexibility of the columns, geometric imperfections that lead to torsion), one cannot generalize, and more tests under more complicated and realistic geometries should be performed for its validation. Moreover, the model cannot predict the torsion of the slab and therefore it can only be used for the prediction of the center of the mass of the slab; not for the column drift ratios.

Figure 19 presents a scatter plot that gives a motion-by-motion comparison of the maximum $U$ of the columns obtained experimentally and by the analytical model. The correlation coefficients between the experimental and numerical $U$ are also shown in the same figure. The calculated correlation coefficients range from $\rho = 0.6249$ to 0.04 indicating that the
FIGURE 18  CDF plots obtained from the experimental data and CDF obtained using the analytical model with data clustered in (A) low- and high-intensity sets; (B) low- and high-intensity NFP sets; (C) low- and high-intensity NF sets; and (D) low- and high-intensity FF sets.
model cannot predict the response to individual ground motions. However, as a designer would design for sets of ground motion (not for individual excitations), this paper claims that the statistical validation approach is the appropriate one.

9 | CONCLUSIONS

A system for the seismic protection of precast bridges was suggested and tested on a shake table. Precast elements are connected with ungrouted tendons in series with washer springs. The bottom end of the tendon is anchored within the column, and the whole system is freely standing on its foundation. This kind of support minimizes the design moment of the foundation, thus reducing the need for piles, which can comprise up to 50% of the RC of the project.

The spring increases both the flexibility and the displacement capacity of the system. In that sense, it can be perceived as a form of seismic isolation using only steel and concrete. In the performed tests, the spring was designed to keep the post-uplift stiffness of the system negative. However, a negative stiffness is not a necessity—a stiffer spring can be used that would increase the post-uplift stiffness to positive values.

After 180 excitations, the system proved resilient: it presented minimal damage only at the steel jackets that were protecting the ends of the columns. It reached more than 20% drift without any concrete damage. The residual deformations were negligible and controlled by the sliding restrainers. Eventually, the system collapsed because of the unexpected failure of a tendon. This shows that the stability of the whole system depends on the tendons, which should be designed with large safety factors and they should be able to hold the structure, even if one of them fails.

The sliding restrainers that were used proved sufficient to restrain the structure from excessive sliding, even under strong vertical acceleration, while they allowed it to rock freely. The compressive force–deformation curve of the spring device (Figure 14F) was well-predicted by the Almen and Laszlo analytical model.66

The test results served as a model validation dataset for a simple model that is based on rigid body dynamics. It was proven that despite the general belief that 3D rocking motion is unpredictable, a simple model was able to predict the statistics of the displacement of the center of mass of the slab to sets of ground motions that characterize the seismic hazard, namely the CDFs of the time maxima to individual excitations.

More work is needed to characterize such systems: their behavior under larger column axial load should be explored, stiffer springs that could lead to positive post-uplift stiffness should be studied, solutions employing prestressing should be tested, a method to design the protective steel jackets should be developed, torsion should be better understood by testing specimens with eccentric masses, and more realistic geometries should be studied.

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**DATA AVAILABILITY STATEMENT**
The data that support the findings of this study are available from the corresponding author upon reasonable request.

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**REFERENCES**
1. Priestley MJN, Tao JR. Seismic response of precast prestressed concrete frames with partially debonded tendons. *PCI J*. 1993;38(1):58–69.
2. Stanton J, Stone WC, Cheok GS. A hybrid reinforced precast frame for seismic regions. *PCI J*. 1997;42(2):20–32.
3. Nakaki SD, Stanton JF, Sritharan S. Overview of the PRESSS five-story precast test building. *PCI J*. 1999;44(2):26–39.
4. Priestley MJN, Sritharan SS, Conley JR, Pampanin S. Preliminary results and conclusions from the PRESSS five-story precast concrete test building. *PCI J*. 1999;44(6):42–67.
5. Mander JB, Cheng CT. Seismic Resistance of Bridge Piers Based on Damage Avoidance Design [technical report NCEER-97-0014].
6. Palermo A, Pampanin S, Calvi GM. Use of ‘Controlled Rocking’ in the seismic design of Bridges. In: Proceedings of the 13th World Conference on Earthquake Engineering. 2004.
7. Palermo A, Pampanin S, Carr A. Efficiency of simplified alternative modelling approaches to predict the seismic response of precast concrete hybrid systems. In: Proceedings of fibSymposium “Keep Concrete Attractive”. 2005.
8. Palermo A, Pampanin S, Marriott D. Design, modeling, and experimental response of seismic resistant bridge piers with posttensioned dissipating connections. *J Struct Eng*. 2007;133(11):1648–1661.
9. Marriott D, Pampanin S, Palermo A. Quasi-static and pseudo-dynamic testing of unbonded post-tensioned rocking bridge piers with external replaceable dissipaters. *Earthq Eng Struct Dyn*. 2009;38:331–354.
10. White S, Palermo A. Quasi-static testing of posttensioned nonemulative column-footing connections for bridge piers. *J Bridg Eng*. 2016;21(6):04016025.
11. Liu R, Palermo A. Quasi-static testing of a 1/3 scale precast concrete bridge utilising a post-tensioned dissipative controlled rocking pier. In: Proceedings of the 16th World Conference on Earthquake Engineering. 2017.
12. Mashal M, Palermo A. Low-damage seismic design for accelerated bridge construction. *J Bridg Eng*. 2019;24(7):04019066.
13. Liu R, Palermo A. Multi-’hinge’ hierarchical activation to improve structural robustness of post-tensioned rocking piers. *ACI Symp Publ*. 2020;341:202–225.
14. Sakai J, Jeong H, Mahin SA. Reinforced concrete bridge columns that re-center following earthquakes. In: Proceedings of the 8th U.S. National Conference on Earthquake Engineering. 2006.
15. Restrepo JI, Rahman A. Seismic performance of self-centering structural walls incorporating energy dissipators. *J Struct Eng*. 2007;133(11):1560–1570.
16. Li C, Bi K, Hao H. Seismic performances of precast segmental column under bidirectional earthquake motions: shake table test and numerical evaluation. *Eng Struct*. 2019;178:314–328.
17. Christopoulos C, Tremblay R, Kim HJ, Lacerte M. Self-centering energy dissipative bracing system for the seismic resistance of structures: development and validation. *J Struct Eng*. 2008;134(1):96–107.
18. Cheng CT. Shaking table tests of a self-centering designed bridge substructure. *Eng Struct*. 2008;30(12):3426–3433.
19. Cohagen LS, Pang JBK, Stanton JF, MO E. A Precast Concrete Bridge Bent Designed to Re-Center after an Earthquake [Rep No. WA-RD 684.3/TNW 2008–09].
20. ElGawady MA, Sha’llan A. Seismic behavior of self-centering precast segmental bridge bents. *J Bridg Eng*. 2011;16(3):328–339.
21. Guerrini G, Restrepo JI, Massari M, Vervelidis A. Seismic behavior of posttioned self-centering precast concrete dual-shell steel columns. *J Struct Eng*. 2015;141(4):04014115.
22. Trono W, Jen G, Panagiotou M, Schoettler M, Ostertag CP. Seismic response of a damage-resistant recentering posttensioned-HYFRC bridge column. *J Bridg Eng*. 2015;20(7):04014096.
23. Kashani MM, Gonzalez-Buelga A, Thayalan RP, Thomas AR, Alexander NA. Experimental investigation of a novel class of self-centering spinal rocking column. *J Sound Vib*. 2018;437:308–324.
24. Wang J, Wang Z, Tang Y, Liu T, Zhang J. Cyclic loading test of self-centering precast segmental unbonded posttensioned UHPFRC bridge columns. *Bull Earthq Eng*. 2018;16(11):5227–5255.
25. Billington SL, Yoon JK. Cyclic response of unbonded posttensioned precast columns with ductile fiber-reinforced concrete. *J Bridg Eng*. 2004;9(4):353–363.
26. Ou YC, Tsai MS, Chang KC, Lee GC. Cyclic behavior of precast segmental concrete bridge columns with high performance or conventional steel reinforcing bars as energy dissipation bars. Earthq Eng Struct Dyn. 2010;39:1181–1198.

27. Motaref S, Saidi MS, Sanders D. Shake table studies of energy-dissipating segmental bridge columns. J Bridg Eng. 2014;19(2):186–199.

28. Panagiotou M, Trono W, Jen G, Kumar P, Ostertag CP. Experimental seismic response of hybrid fiber-reinforced concrete bridge columns with novel longitudinal reinforcement detailing. J Bridg Eng. 2015;20(7):04014090.

29. Bu ZY, Ou YC, Song JW, Lee GC. Hysteretic modeling of unbonded posttensioned precast segmental bridge columns with circular section based on cyclic loading test. J Bridg Eng. 2016;21(6):04016016.

30. Sideris P, Aref AJ, Filiatrault A. Large-scale seismic testing of a hybrid sliding-rocking posttensioned segmental bridge system. J Struct Eng. 2014;140(6):04014025.

31. Sideris P, Aref AJ, Filiatrault A. Experimental seismic performance of a hybrid sliding-rocking bridge for various specimen configurations and seismic loading conditions. J Bridg Eng. 2015;20(11):04015009.

32. Sideris P, Aref AJ, Filiatrault A. Quasi-static cyclic testing of a large-scale hybrid sliding-rocking segmental column with slip-dominant joints. J Bridg Eng. 2014;19(10):04014036.

33. Salehi M, Valigura J, Sideris P, Liel AB. Experimental assessment of second-generation hybrid sliding-rocking bridge columns under reversed lateral loading for free and fixed end rotation conditions. J Bridg Eng. 2021;26(10):04021071.

34. Thonstad T, Mantawy IM, Stanton JF, Eberhard MO, Sanders DH. Shaking table performance of a new bridge system with pretensioned rocking columns. J Bridg Eng. 2016;21(4):04015079.

35. Thonstad T, Kennedy BJ, Schaefer JA, Eberhard MO, Stanton JF. Cyclic tests of precast pretensioned rocking bridge-column subassemblies. J Struct Eng. 2017;143(9):04017094.

36. Yamashita R, Sanders DH. Seismic performance of precast unbonded prestressed concrete columns. ACI Struct J. 2009;106(6):821-830.

37. Routledge PJ, Cowan MJ, Palermo A. Low-damage detailing for bridges — a case study of Wigram-Magdala Bridge. In: Proceedings of the 2016 NZSEE Conference. 2016.

38. Qu H, Li T, Wang Z, Wei H, Shen J, Wang H. Investigation and verification on seismic behavior of precast concrete frame piers used in real bridge structures: experimental and numerical study. Eng Struct. 2018;154:1–9.

39. Bachmann JA, Vassiliou MF, Stojadinovic B. Dynamics of rocking podium structures. Earthq Eng Struct Dyn. 2017;46(14):2499–2517.

40. Bantilas KE, Kavvadias IE, Vasiliadis LK. Seismic response of elastic multidegree of freedom oscillators placed on the top of rocking storey. Earthq Eng Struct Dyn. 2021;50(5):1315–1333.

41. Bantilas KE, Kavvadias IE, Vasiliadis LK. Analytical investigation of the seismic response of elastic oscillators placed on the top of rocking storey. Bull Earthq Eng. 2021;19(2):1249–1270.

42. Cherepinskiy Y. Seismic isolation of buildings with application of the kinematics bases. In: Proceedings of the 13th World Conference on Earthquake Engineering. 2004.

43. Uzdin AM, Doronin FA, Davydova GV, Avidon GE, Karlina EA. Performance analysis of seismic-insulating elements with negative stiffness. Soil Mech Found Eng. 2009;46(3):15–21.

44. Makris N, Vassiliou MF. Planar rocking response and stability analysis of an array of free-standing columns capped with a freely supported rigid beam. Earthq Eng Struct Dyn. 2013;42(3):431–449.

45. Makris N, Vassiliou MF. Are some top-heavy structures more stable?. J Struct Eng. 2014;140(5):06014001.

46. Papaloizou L, Komodromos P. Planar investigation of the seismic response of ancient columns and colonnades with epistyles using a custom-made software. Soil Dyn Earthq Eng. 2009;29(11–12):1437–1454.

47. Mouzakis HP, Psycharis IN, Papastamatiou DY, Carydis PG, Papantonopoulos C, Zambas C. Experimental investigation of the earthquake response of a model of a marble classical column. Earthq Eng Struct Dyn. 2002;31(9):1681–1698.

48. Papantonopoulos C, Psycharis IN, Papastamatiou DY, Lemos JV, Mouzakis HP. Numerical prediction of the earthquake response of classical columns using the distinct element method. Earthq Eng Struct Dyn. 2002;31(9):1699–1717.

49. Konstantinidis D, Makris N. Seismic response analysis of multidrum classical columns. Earthq Eng Struct Dyn. 2005;34(10):1243–1270.

50. Reggiani Manzo N, Vassiliou MF. A negative stiffness reinforced concrete system for resilient seismic design. In: Proceedings of the fib Symposium 2021: Concrete Structures: New trends for Eco-Efficiency and Performance. ETH Zurich, Institute of Structural Engineering (IBK); 2021.

51. Tsopelas P, Constantinou MC, Kim YS, Okamoto S. Experimental study of FPS system in bridge seismic isolation. Earthq Eng Struct Dyn. 1996;25(1):65–78.

52. Buckle IG, Constantinou MC, Di Celi M, Ghasemi H. Seismic Isolation of Highway Bridges [No. MCEER-06-SP07]. 2006.

53. Konstantinidis D, Kelly JM, Makris N. Experimental Investigation on the Seismic Response of Bridge Bearings. Earthquake Engineering Research Center, University of California; 2008.

54. Vassiliou MF, Makris N. Dynamics of the vertically restrained rocking column. J Eng Mech. 2015;141(12):04015049.

55. Makris N, Vassiliou MF. Dynamics of the rocking frame with vertical restrainers. J Struct Eng. 2015;141(10):04014245.

56. Housner GW. The behavior of inverted pendulum structures during earthquakes. Bull Seismol Soc Am. 1963;53(2):403–417.

57. Christopoulos C. Frequency response of flag-shaped single degree-of-freedom hysteretic systems. J Eng Mech. 2004;130(8):894–903.

58. Giouvanidis AI, Dimitrakopoulos EG. Seismic performance of rocking frames with flag-shaped hysteretic behavior. J Eng Mech. 2017;143(5):04017008.

59. Makris N, Konstantinidis D. The rocking spectrum and the limitations of practical design methodologies. Earthq Eng Struct Dyn. 2003;32(2):265–289.
60. Reggiani Manzo N, Vassiliou MF. Displacement-based analysis and design of rocking structures. *Earthq Eng Struct Dyn*. 2019;48(14):1613–1629.

61. Reggiani Manzo N, Vassiliou MF. Simplified analysis of bilinear elastic systems exhibiting negative stiffness behavior. *Earthq Eng Struct Dyn*. 2021;50(2):580–600.

62. Vassiliou MF, Burger S, Egger M, Bachmann JA, Broccardo M, Stojadinovic B. The three-dimensional behavior of inverted pendulum cylindrical structures during earthquakes. *Earthq Eng Struct Dyn*. 2017;46(14):2261–2280.

63. Vassiliou MF. Seismic response of a wobbling 3D frame. *Earthq Eng Struct Dyn*. 2018;47(5):1212–1228.

64. Mashal M. Post-tensioned Earthquake Damage Resistant Technologies for Accelerated Bridge Construction [doctoral thesis]. University of Canterbury; 2015.

65. EN 1992-1-1. *Eurocode 2: Design of Concrete Structures—Part 1-1: General Rules and Rules for Buildings*. 2004.

66. Almen JO, Laszlo A. The uniform-section disk spring. *Trans Am Soc Mech Eng*. 1936;58(4):305–314.

67. Bachmann JA, Strand M, Vassiliou MF, Broccardo M, Stojadinovic B. Is rocking motion predictable?. *Earthq Eng Struct Dyn*. 2018;47(2):535–552.

68. Del Giudice L, Wrobel R, Leinenbach C, Vassiliou MF. Static testing of additively manufactured microreinforced concrete specimens for statistical structural model validation at a small scale. In: Proceedings of the 8th International Conference on Advances in Experimental Structural Engineering (SAESE); 2020.

69. Vassiliou MF, Broccardo M, Cengiz C, et al. Shake table testing of a rocking podium: results of a blind prediction contest. *Earthq Eng Struct Dyn*. 2021;50(4):1043–1062.

70. Vassiliou MF, Cengiz C, Dietz M, et al. Dataset from the shake table tests of a rocking podium structure. *Earthq Spectra*. 2021;37:8755293020988017.

71. Vassiliou MF, Cengiz C, Dietz M, Dihoru L, Broccardo M, Mylonakis G, Sextos A, Stojadinovic B. Data set from shake table tests of free-standing rocking bodies. *Earthquake Spectra*. 2021;37:87552930211020021. https://doi.org/10.1177/87552930211020021

72. FEMA P695. *Quantification of Building Seismic Performance Factors* [Rep. FEMA P695]. Washington, D.C: Federal Emergency Management Agency; 2009.

73. European Committee for Standardization (CEN). *Eurocode 8: Design of Structures for Earthquake Resistance, Part 1: General Rules, Seismic Actions and Rules for Buildings. EN 1998-1:2004*. Brussels; 2004.

74. Bachmann JA, Jost C, Studemann Q, Vassiliou MF, Stojadinovic B. An analytical model for the dynamic response of an elastic SDOF system fixed on top of a rocking single-story frame structure: experimental validation. In: Proceedings of the Eccomas Congress 2016: 7th European Congress on Computational Methods in Applied Sciences and Engineering; Heraklion; 2016:1–40.

75. Malomo D, Mehratra A, DeLong MJ. Distinct element modeling of the dynamic response of a rocking podium tested on a shake table. *Earthq Eng Struct Dyn*. 2021;50(5):1469–1475.

76. Zhong C, Christopoulos C. Finite element analysis of the seismic shake-table response of a rocking podium structure. *Earthq Eng Struct Dyn*. 2021;50(4):1223–1230.

77. MATLAB. *Version (R2019a)*. Natick, MA: The MathWorks Inc.; 2019.

78. Wasserman L. *All of Statistics: A Concise Course in Statistical Inference*. New York: Springer Science & Business Media; 2013.

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### TABLE A1 Excitations

| Exc. ID | M  | Year | Name                  | Recording station       | PEER RSN | PGA when PGV = 33.5 cm/s (in the model scale) |
|--------|----|------|-----------------------|-------------------------|----------|----------------------------------------------|
| Pulse-like records |
| 1      | 6.5| 1979 | Imperial Valley-06   | El Centro Array #6      | 181      | 0.38 g                                       |
| 2      | 6.5| 1979 | Imperial Valley-06   | El Centro Array #7      | 182      | 0.36 g                                       |
| 3      | 6.9| 1980 | Irpinia, Italy-01    | Storno                  | 292      | 0.36 g                                       |
| b4     | 6.5| 1987 | Superstition Hills-02| Parachute Test Site     | 723      | 0.31 g                                       |
| 5      | 6.9| 1989 | Loma Prieta           | Saratoga—Aloha          | 802      | 0.56 g                                       |
| 6      | 6.7| 1992 | Erzincan, Turkey      | Erzincan                | 821      | 0.36 g                                       |
| 7      | 7.0| 1992 | Cape Mendocino        | Petrolia                | 828      | 0.63 g                                       |
| 8      | 7.3| 1992 | Landers               | Lucerne                 | 879      | 0.53 g                                       |
| 9      | 6.7| 1994 | Northridge-01         | Rinaldi Receiving Sta   | 1063     | 0.44 g                                       |
| 10     | 6.7| 1994 | Northridge-01         | Sylmar—Olive View       | 1086     | 0.53 g                                       |
| 11     | 7.5| 1999 | Kocaeli, Turkey       | Izmit                   | 1165     | 0.47 g                                       |
| 12     | 7.6| 1999 | Chi-Chi, Taiwan       | TCU065                  | 1503     | 0.42 g                                       |
| b13    | 7.6| 1999 | Chi-Chi, Taiwan       | TCU0102                 | 1529     | 0.22 g                                       |
| 14     | 7.1| 1999 | Duzce, Turkey         | Duzce                   | 1605     | 0.45 g                                       |
| 15     | 6.6| 1971 | San Fernando          | Pacoima Dam (upper left abut) | 77  | 1.03 g                                        |
| 16     | 7.35| 1978| Tabas_Iran            | Tabas                   | 143      | 0.56 g                                       |
| 17     | 6.93| 1989| Loma Prieta           | Saratoga—W Valley Coll. | 803      | 0.38 g                                       |
| 18     | 6.9| 1995 | Kobe_Japan            | Takarazuka              | 1119     | 0.63 g                                       |
| 19     | 6.9| 1995 | Kobe_Japan            | Takatori                | 1120     | 0.38 g                                       |
| 20     | 6.93| 1989| Loma Prieta           | Los Gatos—Lexington Dam | 3548     | 0.32 g                                       |
| 21     | 7.01| 1992| Cape Mendocino        | Bunker Hill FAA         | 3744     | 0.26 g                                       |
| 22     | 6.6| 2003| Bam_Iran              | Bam                     | 4040     | 0.64 g                                       |
| 23     | 6.63| 2004| Niigata_Japan         | NIGH11                  | 4228     | 0.81 g                                       |
| 24     | 7.1| 1979| Montenegro_Yugoslavia | Bar-Skupstina Opstine   | 4451     | 0.53 g                                       |
| 25     | 7  | 2010| Darfield_New Zealand  | GDLC                    | 6906     | 0.44 g                                       |
| 26     | 7.2| 2010| El Mayor-Cucapah_Mexico| El Centro Array #12     | 8161     | 0.37 g                                       |
| No pulse-like records |
| 27     | 6.8|      | Gazli, USSR           | Karakyr                 | 126      | 0.86 g                                       |
| 28     | 6.5| 1979| Imperial Valley-06   | Bonds Corner            | 160      | 1.17 g                                       |
| 29     | 6.5| 1979| Imperial Valley-06   | Chihuahua               | 165      | 0.73 g                                       |
| 30     | 6.8| 1985| Nahanni, Canada       | Site 1                  | 495      | 1.88 g                                       |
| 31     | 6.8| 1985| Nahanni, Canada       | Site 2                  | 496      | 1.09 g                                       |
| 32     | 6.9| 1989| Loma Prieta           | BRAN                    | 741      | 0.87 g                                       |
| 33     | 6.9| 1989| Loma Prieta           | Corralitos              | 753      | 0.88 g                                       |
| 34     | 7.0| 1992| Cape Mendocino        | Cape Mendocino          | 825      | 1.20 g                                       |
| 35     | 6.7| 1994| Northridge-01         | LA—Sepulveda VA         | 1004     | 0.83 g                                       |
| 36     | 6.7| 1994| Northridge-01         | Northridge—Saticoy      | 1048     | 0.61 g                                       |
| 37     | 7.5| 1999| Kocaeli, Turkey       | Yarimca                 | 1176     | 0.27 g                                       |
| 38     | 7.6| 1999| Chi-Chi, Taiwan       | TCU067                  | 1504     | 0.42 g                                       |

(Continues)
| Exc. ID | M   | Year | Name                   | Recording station          | PEER RSN | PGA when PGV = 33.5 cm/s (in the model scale) |
|---------|-----|------|------------------------|----------------------------|----------|-----------------------------------------------|
| 39      | 7.6 | 1999 | Chi-Chi, Taiwan        | TCU084                     | 1517     | 0.50 g                                        |
| 40      | 7.9 | 2002 | Denali, Alaska         | TAPS Pump Sta. #10         | 2114     | 0.24 g                                        |
| 41      | 6.95| 1940 | Imperial Valley-02     | El Centro Array #9         | 6        | 0.54 g                                        |
| 42      | 7.35| 1978 | Tabas, Iran            | Dayhook                    | 139      | 0.83 g                                        |
| 43      | 6.54| 1987 | Superstition Hills-02  | Superstition Mtn Camera    | 727      | 1.59 g                                        |
| 44      | 6.9 | 1995 | Kobe, Japan            | Kobe University            | 1108     | 0.52 g                                        |
| 45      | 7.14| 1999 | Duzce, Turkey          | Lamont 375                 | 1617     | 1.84 g                                        |
| 46      | 6.61| 2000 | Tottori, Japan         | SMNH01                     | 3947     | 1.43 g                                        |
| 47      | 6.52| 2003 | San Simeon, CA         | Cambria—Hwy 1 Caltrans Bridge | 3979    | 0.79 g                                        |
| 48      | 6.52| 2003 | San Simeon, CA         | Templeton—1-story Hospital | 4031     | 1.07 g                                        |
| 49      | 6.63| 2004 | Niigata, Japan         | NIG017                     | 4207     | 0.85 g                                        |
| 50      | 6.63| 2004 | Niigata, Japan         | NIG019                     | 4209     | 0.88 g                                        |
| 51      | 7.1 | 1979 | Montenegro, Yugoslavia | Ulcinj—Hotel Albatros      | 4457     | 0.65 g                                        |
| 52      | 6.8 | 2007 | Chuetsu-oki, Japan     | Kashiwazaki NPP, Unit 1: ground surface | 4894 | 0.53 g |

**Far-field records**

| 53 | 6.7 | 1994 | Northridge | Beverly Hills—Mulhol | 953 | 0.55 g |
| 54 | 6.7 | 1994 | Northridge | Canyon Country-WLC    | 960 | 0.76 g |
| 55 | 7.1 | 1999 | Duzce, Turkey | Bolu                  | 1602 | 0.95 g |
| 56 | 7.1 | 1999 | Hector Mine   | Hector                | 1787 | 0.59 g |
| 57 | 6.5 | 1979 | Imperial Valley | Delta           | 169  | 0.71 g |
| 58 | 6.5 | 1979 | Imperial Valley | El Centro Array #11   | 174  | 0.68 g |
| 59 | 6.9 | 1995 | Kobe, Japan    | Nishi-Akashi          | 1111 | 0.92 g |
| 60 | 6.9 | 1995 | Kobe, Japan    | Shin-Osaka            | 1116 | 0.61 g |
| 61 | 7.5 | 1999 | Kocaeli, Turkey | Duzce                 | 1158 | 0.43 g |
| 62 | 7.5 | 1999 | Kocaeli, Turkey | Arcelik               | 1148 | 0.41 g |
| 63 | 7.3 | 1992 | Landers        | Yermo Fire Station    | 900  | 0.40 g |
| 64 | 7.3 | 1992 | Landers        | Collwater             | 848  | 0.74 g |
| 65 | 6.9 | 1989 | Loma Prieta    | Capitola              | 752  | 0.83 g |
| 66 | 6.9 | 1989 | Loma Prieta    | Gilroy Array #3       | 767  | 0.84 g |
| 67 | 7.4 | 1990 | Manjil, Iran   | Abbar                  | 1633 | 0.83 g |
| 68 | 6.5 | 1987 | Superstition Hills | El Centro Imp. Co. | 721  | 0.51 g |
| 69 | 6.5 | 1987 | Superstition Hills | Poe Road (temp)        | 725  | 0.79 g |
| 70 | 7.6 | 1999 | Chi-Chi, Taiwan | CHY101                 | 1244 | 0.33 g |
| 71 | 7.6 | 1999 | Chi-Chi, Taiwan | TCU045                 | 1485 | 0.78 g |
| 72 | 6.6 | 1971 | San Fernando   | LA—Hollywood Stor     | 68   | 0.80 g |
| 73 | 6.5 | 1976 | Friuli, Italy   | Tolmezzo               | 125  | 1.07 g |
| 74 | 6.19| 1966 | Parkfield      | Temblor pre-1969       | 33   | 1.27 g |
| 75 | 6.61| 1971 | San Fernando   | Lake Hughes #12        | 71   | 1.48 g |
| 76 | 6.53| 1979 | Imperial Valley | Calexico Fire Station | 162  | 0.83 g |
| 77 | 6.06| 1980 | Mammoth Lakes-01 | Long Valley Dam (Upr L Abut) | 231  | 1.51 g |
| 78 | 6.6 | 1981 | Corinth, Greece | Corinth               | 313  | 0.74 g |
| 79 | 6.36| 1983 | Coalinga-01    | Cantua Creek School    | 322  | 0.78 g |
| 80 | 6.36| 1983 | Coalinga-01    | Parkfield—Fault Zone 14 | 338  | 0.62 g |

(Continues)
**APPENDIX 2:**

**Derivation of the planar lateral load–deformation response of the system with shifted tendon**

The equations presented in Section 2 present two simplifications: (a) the restraining system is assumed to be linear and (b) the tendon always stays in the centerline of the column. The actual system tested (a) uses a slightly nonlinear restraining system and (b) includes a duct in the column for constructional reasons, which allows the tendon to slightly deviate from the centerline. To this end, the pushover curve of Figure 9 was computed based on a model that can take into account both the nonlinearity of the restraining system, and the existence of the gap between the tendon and the duct.

The planar lateral load–deformation response of the system composed of $N$ rigid columns of total mass $N m_c$, a rigid beam of mass $m_b$, and restrained by a perfectly elastic restraining system with stiffness $k_{res}$, when subjected to a lateral force $F$ applied at the beam can be derived via the principle of virtual work

$$ F \cdot \delta u - \left( \frac{m_b g N}{N} + \frac{N m_c}{2} \right) \cdot \delta v = \delta V_S $$  \hspace{1cm} (A.1)

where $V_S$ is the potential energy of the restraining system. Assuming that the columns are able to uplift, but not to slide, the virtual horizontal ($\delta u$) and vertical ($\delta v$) displacements are

$$ \delta u = \frac{du}{d\theta} \cdot \delta \theta = 2R \cdot \cos (\alpha - \theta) \cdot \delta \theta $$  \hspace{1cm} (A.2)

$$ \delta v = \frac{dv}{d\theta} \cdot \delta \theta = 2R \cdot \sin (\alpha - \theta) \cdot \delta \theta $$  \hspace{1cm} (A.3)
In which, \( R \) is the semi-diagonal length of the column; \( \alpha \) is the slenderness of the column, given by the angle between the semi-diagonal of the column and the vertical direction; and \( \theta \) is the tilt angle of the column.

Substituting Equations (A.2) and (A.3) into Equation (A.1) yields

\[
F \cdot 2R \cos (\alpha - \theta) = \left( \frac{m_b}{Nm_c} + \frac{1}{2} \right) \cdot N m_c g \cdot 2R \sin (\alpha - \theta) + N \cdot \frac{dV_S}{d\Delta l} \cdot \frac{d\Delta l}{d\theta} \tag{A.4}
\]

where \( \Delta l \) is the elongation of the restraining system. The force of the restraining system is \( F_{res} = \frac{dV_S}{d\Delta l} \), and Equation (A.4) becomes

\[
F = \left( \frac{m_b}{Nm_c} + \frac{1}{2} \right) \cdot N m_c g \tan (\alpha - \theta) + \frac{N \cdot F_{res}}{2R \cos (\alpha - \theta)} \cdot \frac{d\Delta l}{d\theta} \tag{A.5}
\]

When the gap between the tendon and tendon duct is taken into account, assuming that the tendon passes from (a) the center of the duct at the bottom end of the column and (b) the center of the duct at the top of the beam, the deformation of the restraining system is (Figure 20, left):

\[
\Delta l = \sqrt{\left(2R \cdot \cos (\alpha - \theta) + (\ell_{ten} - 2h) - b \cdot \sin \theta \right)^2 + \left[2R \cdot \sin (\alpha - \theta) - b \cdot (1 + \cos \theta) \right]^2} - \ell_{ten} \tag{A.6}
\]

in which, \( b \) and \( h \) are the half-width and half-height of the column, respectively; and \( \ell_{ten} \) is the initial length of the tendon. When the tendon touches the duct, the deformation of the restraining system is given by (Figure 20, right):

\[
\Delta l = \sqrt{\left( (b - c) \sin \theta \right)^2 + \left( (b - c)(1 - \cos \theta) + c \right)^2 + \sqrt{(2h)^2 + c^2} - 2h} \tag{A.7}
\]

where \( c = \frac{(\phi_{duct} - \phi_{ten})}{e_{ten}^2} \)

To produce Figure 9, the above equations were numerically solved in Matlab.