SIMSG ISSMGE

Paper No. xxxx

SECOND INTERNATIONAL CONFERENCE ON PERFORMANCE-BASED
DESIGN IN EARTHQUAKE GEOTECHNICAL ENGINEERING

May 28-30, 2012 - TAORMINA (ITALY)

NONLINEAR SOIL–FOUNDATION INTERACTION: AN EXPERIMENTAL STUDY ON SAND

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ABSTRACT

Recent studies have highlighted the beneficial role of foundation uplifting and the potential effectiveness of guiding the "plastic hinge" into the foundation soil by allowing full mobilization of bearing capacity during strong seismic shaking. With the inertia loading transmitted onto the superstructure being limited by the capacity of the foundation, such concept may provide an alternative method of "in-ground" seismic isolation: the so called rocking isolation. Attempting to unravel the effectiveness of such alternative design method, this paper investigates experimentally the nonlinear response of a surface foundation on sand and its effect on the seismic performance of an idealized slender 1-dof structure. Using a bridge pier as an illustrative prototype, three foundation design alternatives are considered, representing three levels of design conservatism. Their performance is investigated through static (monotonic and slow-cyclic "pushover") loading, and reduced-scale shaking table testing. It is shown that foundation rocking isolation may provide a valid alternative for the seismic protection of structures and encouraging evidence is presented in favor of the innovative idea of moving foundation design towards a less conservative, even unconventional, treatment.

Keywords: Shallow foundations; nonlinear behavior; bridge pier; shaking table testing

INTRODUCTION

Seismic design of structures recognizes that highly inelastic material response is unavoidable under strong seismic shaking (performance based design). Ductility levels of the order of 3 or more are usually allowed to develop at bearing structural elements and “plastic hinging” is directed appropriately so as the overall stability is maintained (capacity design). By contrast, as reflected in the respective seismic codes, current seismic design practice demands a very conservative treatment of the foundation. Hence, increased safety factors and overstrength design ratios are adopted, lest "failure" be transferred below the ground level. However, this conservative treatment of the foundation, which is designed to retain "elastic" behavior even for extreme loading, conflicts modern research findings indicating that nonlinear foundation response : (i) may be highly probable even for seismic events of moderate intensity, (ii) may be favorable for the overall

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system performance, and (iii) may result in permanent deformations which could be restrained within acceptable limits thanks to the transient nature of seismic loading.

In the case of shallow foundations, nonlinearity manifests itself through alternating uplifting of the foundation (geometric nonlinearity), sliding at the soil–foundation interface (interface inelasticity), and/or mobilization of bearing capacity failure mechanisms in the supporting soil (soil inelasticity). When slender structures are considered, rocking motion prevails and the geometric component of nonlinearity dominates. Earlier studies on rocking structures [Housner, 1963; Meek, 1975; Psycharis & Jennings, 1983; Chopra & Yim, 1985] have indicated the beneficial role of foundation uplifting on the performance of the supported structure, particularly during severe seismic shaking. Furthermore, allowing for foundation rocking has been proposed by several researchers as an effective method of seismic isolation [e.g. Beck & Skinner, 1974; Huckelbridge & Ferencz, 1981; Priestley et al., 1996; Mergos & Kawashima, 2005; Chen et al., 2006; Sakellaraki & Kawashima, 2007] and has been applied in the design of modern bridges (e.g. the Rion Antirion Bridge : Pecker, 2005). However, in the last decade the research community has ventured one significant step further acknowledging that in a way similar to pure uplifting, concurrent inelastic soil response may also help to protect the superstructure against increased seismic demands [e.g. Martin & Lam, 2000; Pecker & Pender, 2000; Faccioli et al., 2001; Gajan et al., 2005; Harden et al., 2006; Gazetas et al., 2007; Paolucci et al., 2007; Anastasopoulos et al., 2010a].

This paper investigates experimentally the role of nonlinear foundation response on the seismic performance of a slender 1-dof structure. The configuration of the conceptual prototype problem is portrayed in Figure 1. It involves a bridge pier of moderate height founded upon a layer of dense sand through a square shallow foundation of varying width B. Three different foundation sizes were considered, designated as "large", "medium", and "small", representing a conservatively designed foundation, a less conservative one, and a seriously under-designed foundation, respectively. The performance of the three systems under static (monotonic and cyclic) and earthquake loading was thoroughly investigated through a series of 1-g physical tests and evaluated with respect to the effectiveness of their design concept regarding the prohibition or permission of foundation nonlinearity.

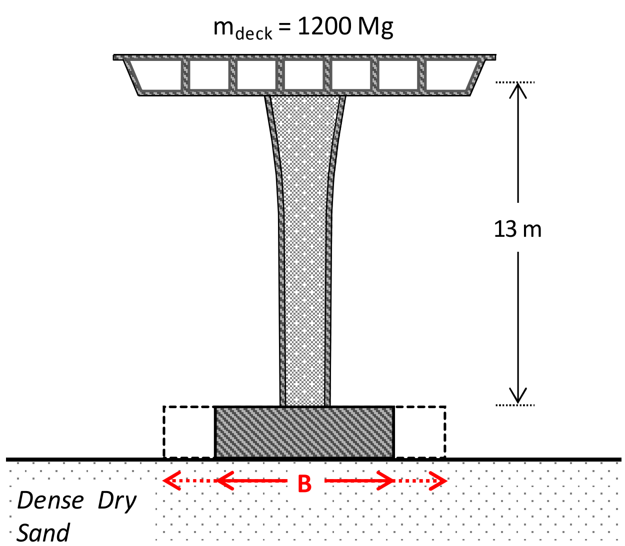

Figure 1. Definition of the studied problem concerning the foundation design (footing size, factor of safety) of a typical bridge pier.

METHODOLOGY

A number of simplified bridge pier physical models with shallow foundations of various sizes (i.e. *FS* values) were built at the Laboratory of Soil Mechanics of NTUA and tested against (vertical and horizontal) static and dynamic (shaking table) loading. A linear geometric modeling scale of 1:20 was selected with regard to the shaking table capacity and the physical models dimensions and properties were appropriately scaled down according to the relevant scaling laws [Muir Wood, 2004].

Soil Sample

Dry Longstone sand [see, Anastasopoulos et al., 2010b] was used in the experiments. Nine identical soil specimens were constructed within a rigid container of dimensions160 x 90 x 75 cm (at model scale), upon which each one of the pier models was tested separately. The sand was placed into the container through an electronically-controlled sand raining system designed to produce soil samples of controllable relative density D_r , ensuring repeatability. In the present study the initial soil sample was chosen to be of high density, $D_r \approx 85\%$ for all tests, to minimize soil densification during shaking. The effective soil friction angle was estimated as $\varphi' \approx 44°$ through a series of vertical pushover tests on three different foundation models making use of traditional bearing capacity equations [Meyerhof, 1951].

Pier–Foundation Model

Figure 2 gives an overview of the pier model geometry. This is comprised of three parts: the deck (an assembly of steel plates with total weight of 150 kg), the column, and the foundation.

With the exception of the foundation size, the three tested pier models were identical. It may be readily observed that the very elongated shape of the foundation model is essentially different from the square shape considered in the conceptual prototype pier. The reason for this intentional discrepancy lies in the treatment of small scale effects originating from the pressure dependence of soil behavior. As the magnitude of the applied confining stress presumably depends on the geometric scale, reduced scale modeling unavoidably leads to misreproduction of the stress field in the soil model in comparison to the prototype and hence to misreproduction of the soil strength in terms of both magnitude and distribution. As a result, geometric scaling of the foundation size would result to incorrect scaling (actually overestimation) of its capacity.

Aiming to compensate for this limitation of small-scale modeling and achieve similitude between model and prototype foundation it is essential to satisfy the following three conditions:

(i) for similarity in the vertical direction to be preserved, the ratio of the total vertical load carried by the foundation to its vertical capacity (N/N_u) must be the same in model and prototype;

(ii) in the same way, the ratio of the lateral load to the lateral capacity (Q/Q_u, and M/M_u) should be instantaneously preserved;

(iii) lastly, as rocking response is controlled by the slenderness ratio (here equal to the height of the centre of mass h divided by the foundation length in the direction of shaking L), it is essential that this parameter remains unchanged.

With regard to foundation design practice, the first two conditions basically reduce to preserving the Factors of Safety (*FS*) for vertical and combined-seismic loading (FS_V and FS_E respectively) in the model the same as in the prototype. Given the overestimation of the soil strength in the model, this may only be achieved by reducing the foundation area. However, doing so in both directions would violate the requirement for preservation of the slenderness ratio h/L. Therefore the foundation area was reduced by decreasing only the out-of-plane foundation dimension.

For stability in the out-of-plane direction the deck-mass was supported through a Π shaped column-foundation system with the two footings of breadth *B* being in adequately large distance to prevent any interaction effects. *B* was calculated with respect to the intended *FS* values for each one of the two systems making use of common practice bearing capacity formulas for pure vertical loading [Meyerhof, 1951] and combined *N–Q–M* loading [Butterfield & Gottardi, 1994] for an average pre-estimated effective soil friction angle of $\varphi' \approx 44°$. Table 1 summarizes the geometry, elastic properties of column sections, and design characteristics of the three pier-foundation systems.

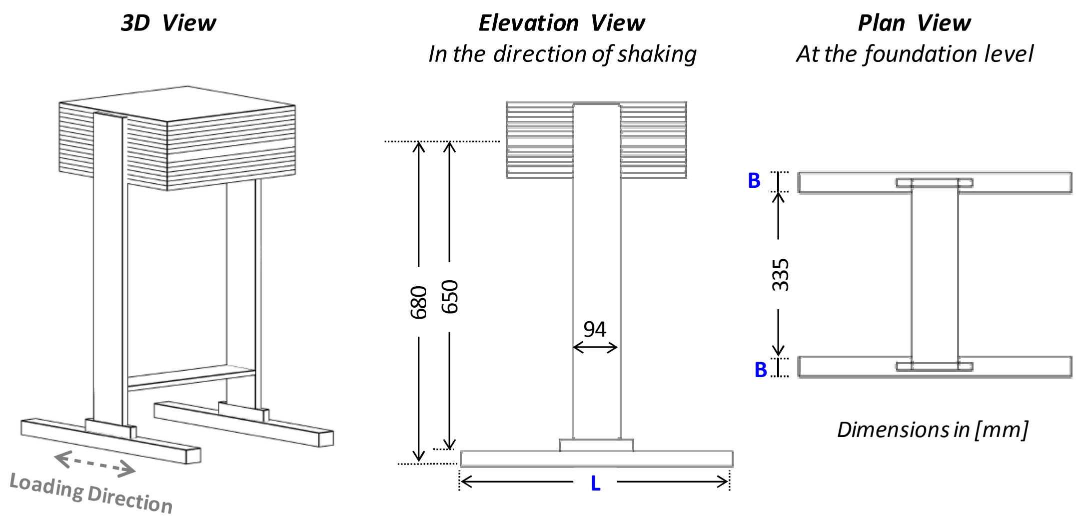

Figure 2. Geometry of the pier model.

Table 1. Summary of the pier models geometry and design characteristics (in prototype scale).

PIER						
Deck Mass	*Pier Height*	*Total Height*	*Column Section*			*Fix. base Period*
M : Mg	**h_p : m**	**h : m**	**A : m^2**	**I_x : m^4**	**E : kPa**	**T_0 : sec**
1200	13	13.6	1.06	0.32	40 x 10^8	0.16

FOUNDATION						
size	*Length*	*Width*	*Slenderness*	*Total Weight*	*Design Safety Factors*	
	L : m	**B : m**	**h/L**	**N : kN**	**FS_V**	**FS_E**
large	11	1.70	1.2	14362	7.49 > 3	1.07 > 1
medium	7	1.40	1.9	13593	3.41 > 3	0.55 < 1
small	7	1.14	1.9	13436	2.29 < 3	0.43 < 1

Set-up and Instrumentation

The experimental series involved three types of tests, namely: (i) vertical-push tests; (ii) monotonic and cyclic lateral pushover tests; and (iii) shaking table testing.

During monotonic and slow-cyclic push tests, load was applied in the horizontal or vertical direction through a servo-electric actuator, and measured by a load cell connected at its edge. Wire and laser displacement transducers measured vertical and horizontal displacements of the pier model. In the dynamic (shaking table) tests, the motion of characteristic points within the soil and on the structure were recorded by vertical and horizontal accelerometers. Strain gauges installed at the base of the column measured section bending strains and verified the results derived by the acceleration measurement of the deck-mass. Figure 3 displays the set-up and instrumentation for the three test types.

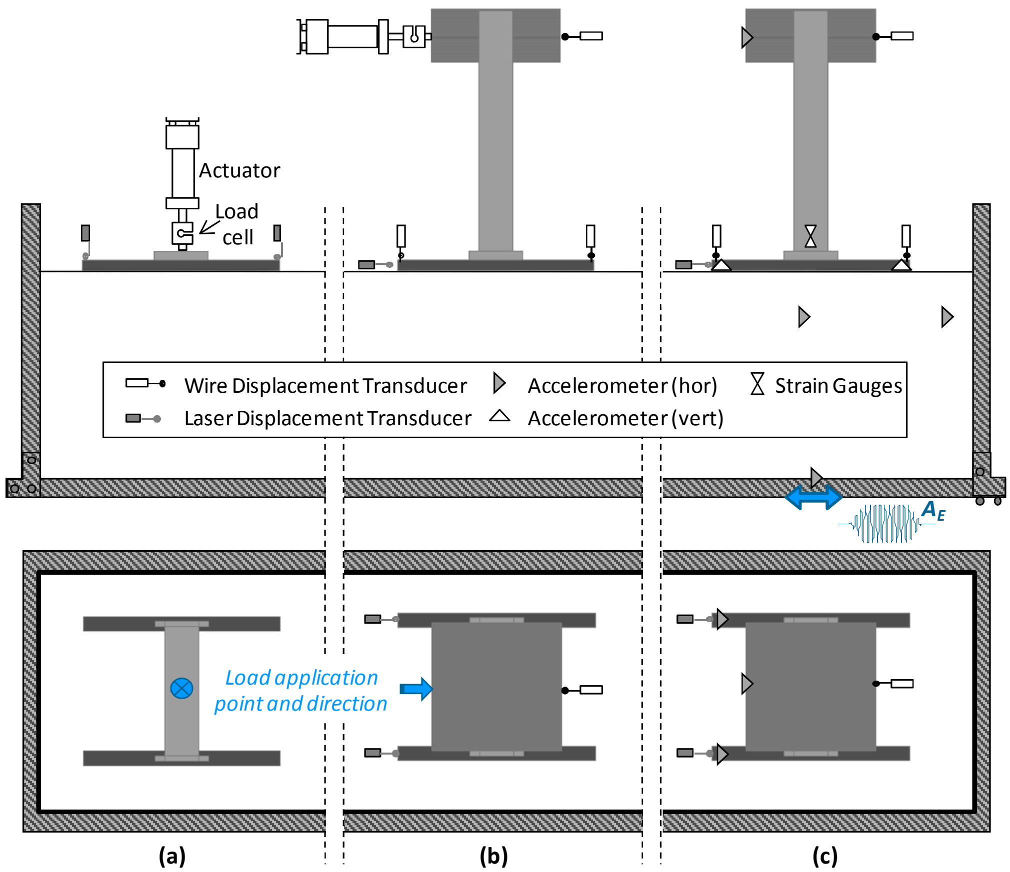

Figure 3. Experimental set-up and instrumentation for : (a) vertical pushover, (b) horizontal pushover, and (c) shaking table tests.

PRESENTATION OF CHARACTERISTIC RESULTS

Vertical Push

Slow vertical push was applied by an actuator, which was placed precisely at the centre of the foundation area, as shown in Figure 3a. Successive loading–unloading cycles produced the load–settlement curves shown in Figure 4 for the three different foundations. The ultimate bearing capacity N_{ult} is given in each case. The large footing carries an ultimate load of about 99 MN (corresponding to $FS_V \approx 6.9$), which is well above the current code requirements and exceeds the ultimate capacity of the medium and small foundation by a factor of 2.2 and 3.6, respectively. It should be noted that the measured foundation capacities are slightly lower than initially estimated (see design FS_V values in Table 1). This is due to the postulation of a constant secant friction angle ($\varphi' = 44°$) made in the analysis — an unavoidable simplification of a more complex reality where φ' varies with the applied stress.

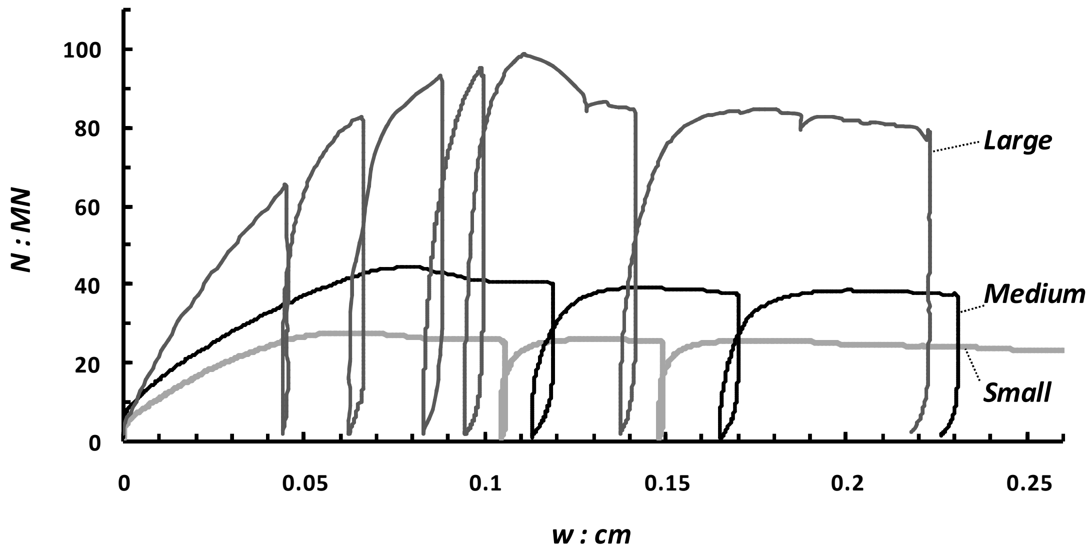

Figure 4. Vertical load–settlement curves for the three studied foundation systems.

Lateral Pushover

Figure 5 summarizes the moment−rotation (M−θ) and settlement−rotation (w−θ) response of the three foundations during both monotonic and cyclic lateral pushover tests.

Foundation moment capacity primarily depends on foundation size, and hence it comes as no surprise that the large foundation transmits the greatest moment. In particular, when loaded monotonically it transmits approximately 2 and 2.4 times larger moment than the medium and small foundations, respectively, verifying their design (see Table 1).

Switching into cyclic mode has an important effect on the behavior as it leads to apparent overstrength especially for the small foundation. Comparison of cyclic M–θ response with the corresponding monotonic shows that whereas for the two larger footings the monotonic curves almost envelope the cyclic loops, with M_{ult} being quite the same under monotonic and cyclic loading, the cyclic loops of the smaller foundation surpass appreciably the monotonic curve. As a result, when loaded cyclically, the small foundation bears significantly higher lateral loads than estimated in its design, thus appearing to transmit approximately the same peak moment as the medium-size foundation. It should be noted that the two smaller foundation systems have exactly the same slenderness ratio h/L, which appears to be the most decisive parameter for the ultimate lateral capacity of rocking systems, perhaps overshadowing the effect of FS_V.

Yet, FS_V presumably plays a dominant role when foundation displacements are considered, this being elucidated by the settlement–rotation loops of Figure 5, where the cyclic movement of the foundation midpoint is depicted as a function of footing rotation. As expected, settlement increases consistently with reducing FS_V. Hence, although there is only a minor difference in terms of peak transmitted moments, the small foundation settles almost twice as much as the medium-size foundation.

Furthermore, FS_V controls the interplay between uplifting and bearing capacity failure mechanisms. The gradient of w–θ curves indicates whether the foundation midpoint loses contact with the supporting soil as the foundation rotates, giving evidence on the amount of uplift that takes place during the test. Evidently, the large foundation experiences significant uplifting, indicated by the ascending slope of w–θ in Figure 5a.

Observe that in monotonic loading the large foundation midpoint moves upwards almost from the beginning of loading, revealing that more than half of the foundation detaches from the supporting soil. Yet, as FS_V reduces, soil nonlinearity becomes prevalent, resulting in greater rates of settlement per cycle, and reducing the extent of foundation uplift. Figures 5b and 5c clearly show downwards movement of the foundation midpoint with rotation, for both the medium-size and the small foundation respectively. Yet, the significant difference in the inclination of the respective cyclic curves indicates some limited uplifting of the medium-size foundation in contrast to the pure sinking response of the small foundation. The increased structural weight relative to the small foundation capacity makes uplifting much more energy-consuming than soil yielding, and hence the latter takes place for smaller foundation rotation. The supporting soil complies as the foundation rotates, and the foundation midpoint settles in every half-cycle of loading increasing dramatically the amount of settlement per cycle.

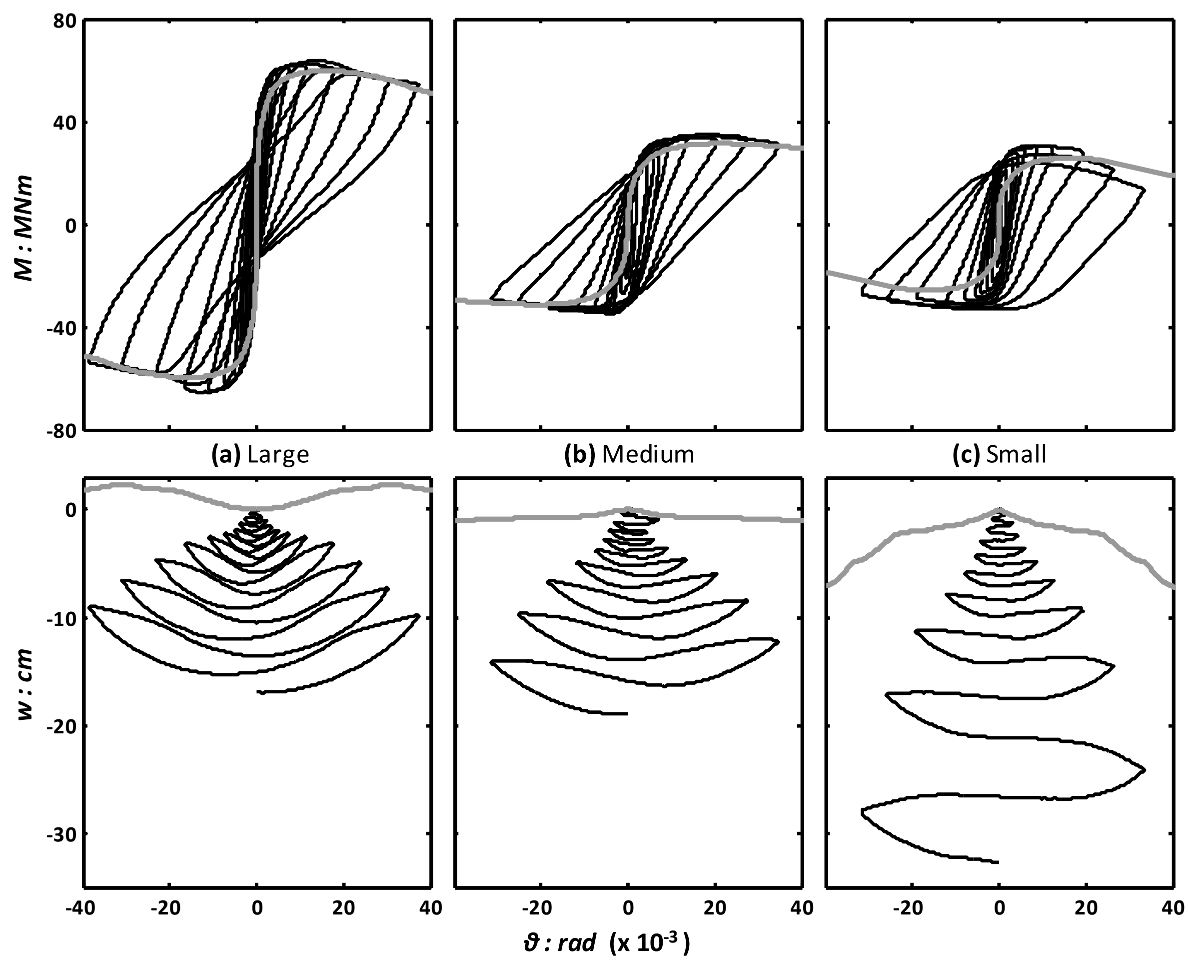

Figure 5. Monotonic (grey line) and slow-cyclic (black line) lateral pushover test results in terms of moment–rotation and settlement–rotation foundation response for : (a) the large (FS_V = 7.3) ; (b) the medium foundation (FS_V = 3.5) ; and (c) the small foundation (FS_V = 2.3).

Seismic Table Testing

Figure 6 presents the set of seismic motions used as excitations in the shaking table tests. Being selected so as to represent motions of various characteristics and intensities, this ensemble of acceleration histories involves both real earthquake records and artificial pulses of varying intensities and dominant periods.

For the sake of brevity and for the purpose of focusing on the potentially favorable role of foundation nonlinearity under strong earthquake motion, the herein presented shaking table results will be limited to one only excitation case — that in which the model was excited by a 2 Hz 12-cycle sine pulse with acceleration amplitude $A_E = 0.5$ g.

Under such excitation all three foundations respond well within the nonlinear regime as indicated by the respective M–θ and w–θ loops of Figure 7. It is important to observe that the large foundation experiences a rotational motion of similar or larger amplitude than the two smaller foundations, possibly because its advantage of having larger moment resistance and rocking stiffness is counterbalanced by the two times greater inertial loading that it suffers. Hence, its design conservatism succeeds only in the limitation of the resulting settlement, which is indeed significantly reduced for the large foundation in comparison to the two smaller ones. On the other hand, being the product of foundation rotation, lateral pier displacements may not be directly correlated to foundation design safety factors.

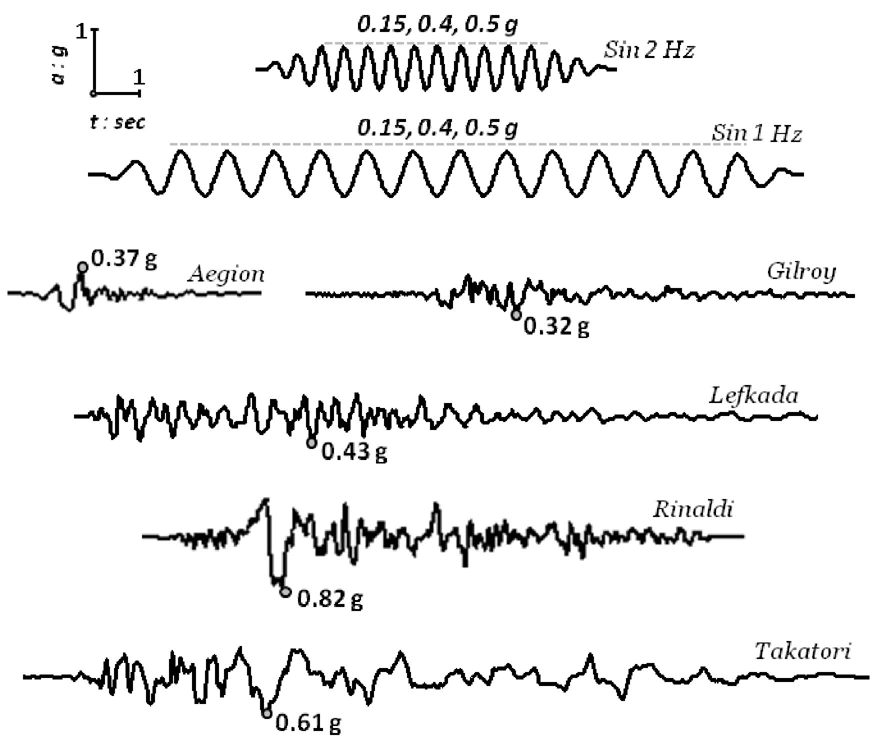

Figure 6. Real records and artificial accelerograms used as excitation in the shaking table tests.

Furthermore, it is worth noting that the acceleration time histories recorded at the deck of the pier (shown in Figure 8) are strictly cut-off at a particular critical value (α_c) for each one of the three systems, this value being controlled by the foundation capacity in such way that the maximum transmitted inertial load may not exceed the lateral capacity at any instance. With reference to the ultimate moment capacity determined by pushover tests (Figure 5), the large foundation system may sustain $\alpha_c \approx 0.36$ g. Having about half the moment capacity of the large foundation, the two smaller foundations bound the seismic motion transmitted to the superstructure to a much lower level : $\alpha_c \approx 0.18$ g and 0.16 g, for the medium and the small foundation, respectively.

Acceleration time histories of Figure 8 confirm that the dynamic motion developed at the pier deck mass is bounded by the above calculated limiting values and verify this "rocking isolation" mechanism, which is presumably associated with full mobilization of foundation–soil moment capacity (expressed as uplifting and soil yielding) and hence forms the cornerstone of the new idea for allowing, and taking advantage of, nonlinear foundation response. The two under-designed foundation systems provide a drastic reduction of the seismic acceleration transmitted to the pier to only one third of the input peak acceleration A_E. Some limited isolation effect is observed even in the case of the large foundation system ($\alpha_{max} / A_E = 0.72$). Yet, having a significantly larger capacity M_u compared to the other two systems, the conservatively designed pier suffers much more intense shaking.

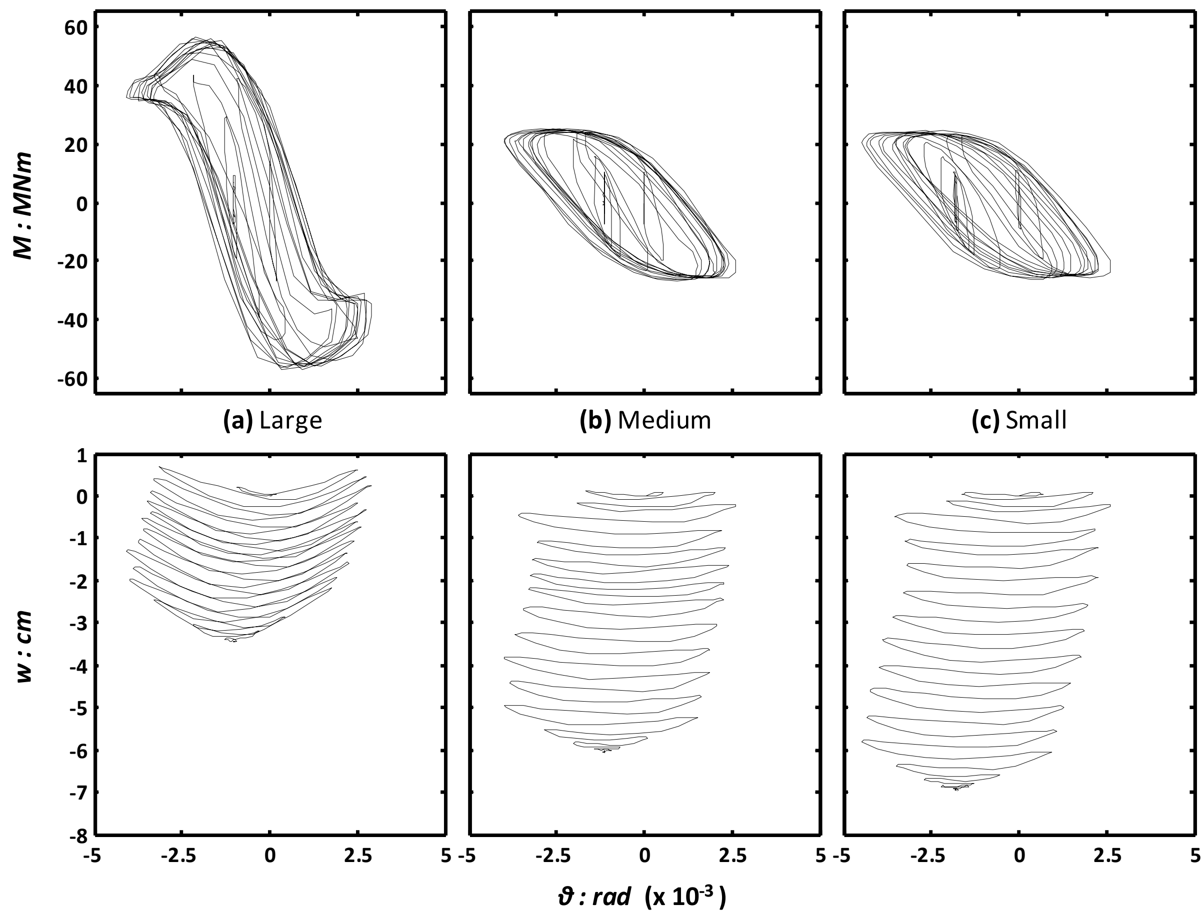

Figure 7. Foundation response to excitation with a 12-cycle 2 Hz sine pulse with 0.50 g acceleration amplitude (Sin2-0.50 g). Moment−rotation and settlement−rotation response for : (a) the large (FS_V = 7.3) ; (b) the medium (FS_V = 3.5) ; and (c) the small foundation (FS_V = 2.3).

CONCLUSIONS

The most significant outcome of this study is the experimental verification of the potential effectiveness of rocking isolation as a means of seismic protection of a bridge pier. Acting as a safety "fuse", full mobilization of foundation capacity (in the form of uplifting and soil yielding) constrains the acceleration transmitted onto the superstructure to a value below a critical acceleration α_c, which is directly associated with foundation capacity M_{ult} and, hence, decreases with reducing foundation size. The effectiveness of rocking isolation in terms of inertial loading for the entire set of studied earthquake excitations is summarized in Figure 9. Evidently, the two under-designed foundations (medium and small) drastically reduce the maximum acceleration α_{max} transmitted to the deck for all the studied seismic excitations.

Despite having quite different *FS* values, the medium and small foundations sustain practically the same moment loading and consequently permit similar levels of inertial loading to be transmitted onto the superstructure. This similarity in their capacity can be attributed to two observations : (i) lateral load capacity is principally controlled by the slenderness ratio *h/L* being much less sensitive to changes in the foundation out-of-plane dimension; and (ii) during cyclic loading, an overstrength mechanism was observed to take place and affect mainly the capacity of small foundations.

FS_V affects the development and accumulation of permanent displacements. In the case of symmetric seismic motions such as the sine pulse presented herein, the increase of settlement appears to be the only significant argument against the rocking isolation concept (i.e. under-designing the foundation for the sake of structural safety). Real – asymmetric excitations may also bring about some considerable permanent foundation rotation, which will unavoidably result permanent deck drift. Yet, the problem reduces to defining the acceptable displacements of the superstructure in relation to performance requirements.

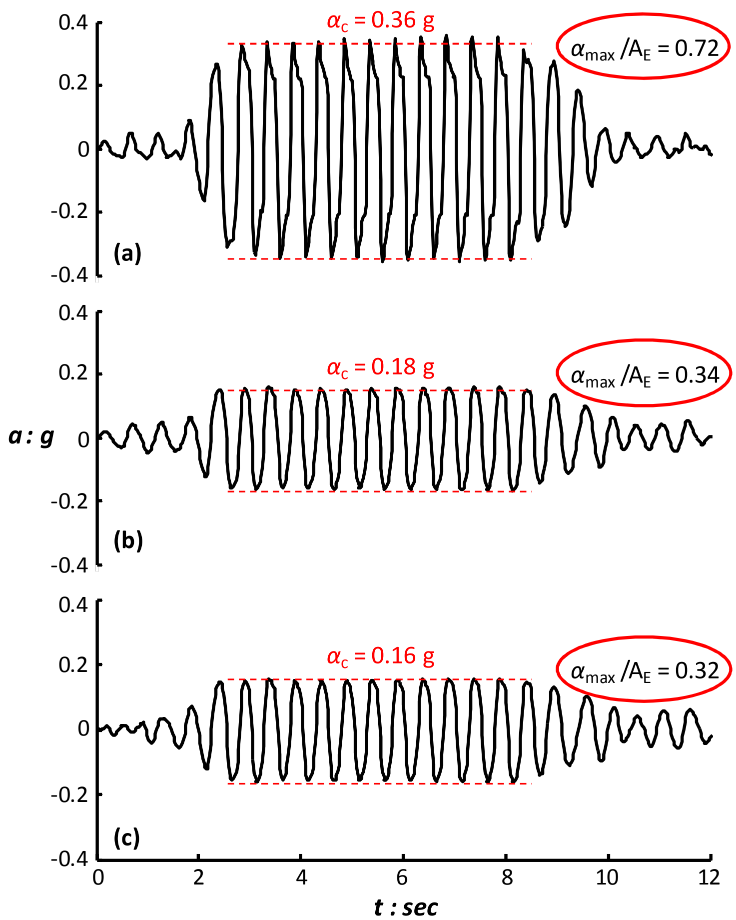

Figure 8. Acceleration time histories recorded at the level of the deck (center of mass) for base excitation with a 12-cycle 2 Hz sine of 0.15 g acceleration amplitude (Sin2-0.15 g) for : (a) the large (FS_V = 7.3) ; (b) the medium (FS_V = 3.5) ; and (c) the small foundation (FS_V = 2.3).

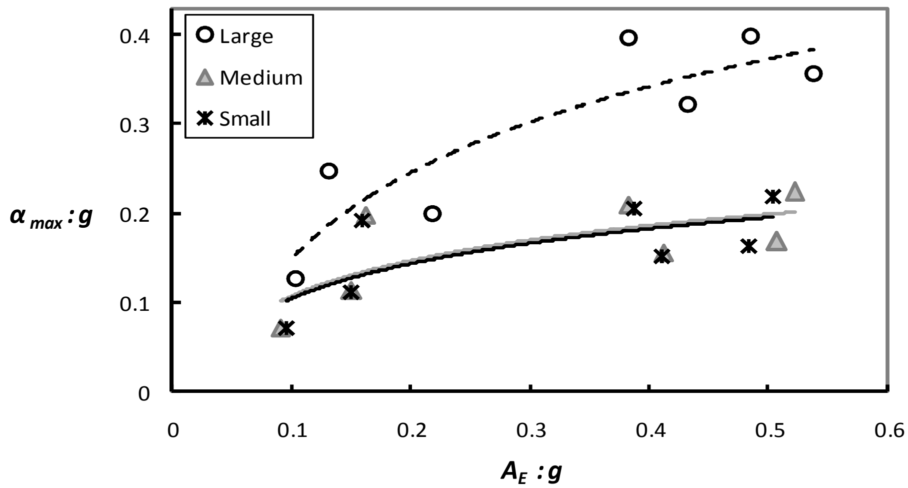

Figure 9. Rocking isolation effectiveness for the three pier–foundation systems : maximum deck acceleration α_{max} with versus the acceleration amplitude A_E of the base excitation.

ACKNOWLEDGEMENT

The research presented in this paper was funded by the European Research Council (ERC) Program "IDEAS, Support for Frontier Research – Advanced Grant", under Contract number ERC–2008–AdG 228254–DARE.

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