

IBSBI 2014, October 16-18, 2014, Athens, Greece

## **SOME NEW TRENDS IN SEISMIC DESIGN OF BRIDGE-PIER FOUNDATIONS**

**George Gazetas**

*National Technical University of Athens, Greece*

**ABSTRACT:** Numerical and experimental research is outlined on the idea of rocking-isolation of bridge pier through a non-conservative design of the foundations. Shallow foundations are studied in detail, but piled foundations and embedded caissons are also discussed.

### **1. INTRODUCTION**

Inspired by post-seismic field observations and engineering concerns in actual bridge projects, in the last 10-15 years significant theoretical and experimental research aimed at challenging one of the cornerstones in earthquake engineering: “capacity design”. One of the motives of such design was to control seismic damage by strategically directing inelastic deformation to structural components that could be inspected and repaired after a damaging event. For bridge piers this meant ensuring that “plastic hinging” will occur in the column rather than the foundation. Hence, current design leads to strong unyielding foundation-soil systems. Overstrength factors are invoked to increase the maximum moment that can be (nominally) transmitted by the pier column, and foundation design is performed with this increased moment, and (in addition) conservatively. As a result, in regions of high seismicity piled foundations or rigid caissons or oversized footings are often required even in excellent ground conditions. Evidence from numerous earthquakes shows that such unyielding foundations (“nailing” the structure to the ground) have not prevented heavy damage and even catastrophic collapse in case of a strong seismic excitation. In response, a number of studies have explored the possibilities and constraints of an alternative design concept: allowing the development of plastic hinging in the soil or at the soil–foundation interface, so as to reduce the possibility of damage to the structure.

Focusing on surface foundations, where nonlinearity manifests itself through uplifting and/or soil yielding, a “reversal” of the current capacity design principle is proposed: the foundation is intentionally underdesigned compared with the supported pier/column to promote rocking response and accumulation of plastic deformation at the soil–foundation interface. Supporting evidence for this new approach has been provided as follows:

- Several theoretical and numerical studies on the rocking response of rigid blocks and elastic single-degree-of-freedom (SDOF) oscillators provide compelling evidence that uplifting drastically reduces the inertial load transmitted into the oscillating structure.
- Because of the transient and kinematic nature of seismic loading, rocking response does not lead to overturning even in the case of very slender structures except in rather extreme cases of little practical concern.
- Referred to as rocking isolation, allowing for foundation uplift has been proposed, and in a few exceptional cases employed in practice, as a means of seismic isolation (Rion-Antirion bridge, Greece; Rangitikei Railway Bridge, N. Zealand).
- Even in the case of relatively heavily loaded footings or footings on soft soils, when rocking is accompanied with yielding of the supporting soil (and possibly momentary mobilizing bearing capacity failure mechanisms), substantial energy is dissipated in the foundation providing increased safety margins against overturning owing to the inherently self-centering characteristics and the ductile nature of rocking on compliant soil.
- Most importantly, a number of studies have recently investigated the scheme of rocking isolation, with emphasis on its effects on structures, which consistently point to a beneficial role of nonlinear foundation behavior for the overall system performance.
- A variety of modern numerical tools have been developed enabling comprehensive modeling of non-linear rocking response alleviating to some degree the skepticism regarding the uncertainties traditionally associated with prediction of the performance of rocking foundations for use in design.

## 2. SEISMIC RESPONSE OF BRIDGE-PIER ON SHALLOW FOUNDATION: THE “ROCKING-ISOLATION” CONCEPT

### 2.1 Theoretical Studies

The concept of “Rocking Isolation” is illustrated in Fig. 1 by examining the response of a 12 m tall bridge pier carrying a deck of four lanes of traffic for a span of about 35 m — typical of elevated highways around the world.

The bridge chosen for analysis is similar to the Hanshin Expressway Fukae bridge, which collapsed spectacularly in the Kobe 1995 earthquake. The example bridge is designed in accordance to EC8-2000 for an effective acceleration  $A = 0.30$  g, considering a (ductility-based) behavior factor  $q = 2$ . With an elastic (fixed-base) vibration period  $T = 0.48$  sec the resulting design bending moment  $M_{COL} \approx 45$  MN m.

The pier is founded through a square foundation of width  $B$  on an idealized homogeneous 25 m deep stiff clay layer, of undrained shear strength  $s_u = 150$  kPa (representative soil conditions for which a surface foundation would be a realistic solution). Two different foundation widths are considered to represent

the two alternative design approaches. A large square foundation,  $B = 11$  m, is designed in compliance with conventional capacity design, applying an overstrength factor  $\psi_{Rd} = 1.4$  to ensure that the plastic “hinge” will develop in the superstructure (base of pier). Taking account of maximum allowable uplift (eccentricity  $e = M / N < B/3$ , where  $N$  is the vertical load), the resulting safety factors for static and seismic loading are  $F_S = 5.6$  and  $F_E = 2.0$ , respectively. A smaller, under-designed,  $B = 7$  m foundation is considered in the spirit of the new design philosophy. Its static safety factor  $F_S = 2.8$ , but it is designed applying an “*under-strength*” factor equal to  $1/1.4 \approx 0.7$  for seismic loading. Thus, the resulting safety factor for seismic loading is lower than 1.0 ( $F_E \approx 0.7$ ).

The seismic performance of the two alternatives is investigated through nonlinear FE dynamic time history analysis. An ensemble of 29 real accelerograms is used as seismic excitation of the soil–foundation–structure system. In all cases, the seismic excitation is applied at the bedrock level. Details about the numerical models and the requisite constitutive relations can be seen in Anastasopoulos et al [4, 5].

Results are shown here only for a severe seismic shaking, exceeding the design limits: the Takatori accelerogram of the 1995  $M_{JMA} 7.2$  Kobe earthquake. With a direct economic loss of more than \$100 billion, the Kobe earthquake needs no introduction. Constituting the greatest earthquake disaster in Japan since the 1923  $M_s = 8$  Kanto earthquake, it is simply considered as one of the most devastating earthquakes of modern times. Of special interest is the damage inflicted to the bridges of Hanshin Expressway, which ranged from collapse to severe damage. The aforementioned bridge chosen for our analysis is very similar to the Fukae section of Hanshin Expressway, 630 m of which collapsed during the earthquake of 1995. It is therefore logical to consider this as a reasonably realistic example of an “*above the limits*” earthquake. In particular, the Takatori record constitutes one of the worst seismic motions ever recorded :  $PGA = 0.70$  g,  $PGV = 169$  cm/s, bearing the “mark” of both forward rupture directivity *and* soil amplification.

Fig. 1 compares the response of the two alternatives, in terms of deformed mesh at the end of shaking with superimposed the plastic strains. In the conventionally designed system there is very little inelastic action in the soil; the red regions of large plastic deformation are seen only under the severely “battered” edges of the rocking foundation — but without extending below the foundation. “Plastic hinging” forms at the base of the pier, leading to a rather intense accumulation of curvature (deformation scale factor = 2). The  $P-\delta$  effect of the mass will further aggravate the plastic deformation of the column, leading to collapse.

In stark contrast, with the new design scheme the “plastic hinge” takes the form of mobilization of the bearing capacity failure mechanisms in the underlying soil, leaving the superstructure totally intact. Notice that the red

regions of large plastic shearing are of great extent, covering both half-widths of the foundation and indicating alternating mobilization of the bearing capacity failure mechanisms, left and right.

The above observations are further confirmed by the time history of deck drift shown in Fig. 1(c). The two components of drift, are shown, one due to footing rotation in blue and one due to structural distortion in green. Their sum is shown in red. Evidently, the conventional design experiences essentially only structural distortion which leads to uncontrollable drifting — collapse. In marked contrast, the system designed according to the new philosophy easily survives. It experiences substantial *maximum* deck drift (about 40 cm), almost exclusively due to foundation rotation. Nevertheless, the *residual* foundation rotation leads to a tolerable 7 cm deck horizontal displacement at the end of shaking.

Fig. 1(d) further elucidates the action of the foundation-soil system. The  $M-\theta$  relationship shows for the 11 x 11 m<sup>2</sup> foundation a nearly linear viscoelastic response, well below its ultimate capacity and apparently with no uplifting. On the contrary, the 7 x 7 m<sup>2</sup> (under-designed) foundation responds well past its ultimate moment capacity, reaching a maximum  $\theta \approx 30$  mrad, generating hysteretic energy dissipation, but returning almost to its original position, i.e. with a negligible residual rotation.

However, energy dissipation is attained at a cost: increased foundation settlement. While the practically elastic response of the conventional (*over-designed*) foundation leads to a minor 4 cm settlement, the *under-designed* foundation experiences an increased accumulated 15 cm settlement. Although such settlement is certainly not negligible, it can be considered as a small price to pay to avoid collapse under such a severe ground shaking.

Perhaps not entirely fortuitously, the residual rotation in this particular case turned out to be insignificant. The recentering capability of the design certainly played some role in it, as will be discussed in the sequel.

## 2.2 Experimental Studies

Numerous experimental investigations have been conducted by Kutter and coworkers in the large centrifuge of the University of California Davis. Here we summarize two other studies, one conducted in small-scale 1-g shaking tests in the Laboratory of NTUA, and one conducted in the centrifuge of the University of Dundee. The small-scale (1:20) tests refer to the system of Fig. 2

The Gilroy record from the 1989Ms 7.1 Loma Prieta earthquake is utilized as an example of relatively strong seismic shaking, slightly exceeding the design. As in the previous case, the response of the two design alternatives is comparatively assessed in Figs. 3-5 in terms of deck acceleration time histories, foundation  $M-\theta$  and  $w-\theta$  response, and time histories of deck drift. Time histories of deck acceleration of the two systems are compared in Fig. 3. The

increase of seismic demand has a marked effect on the response of both systems, which now clearly mobilize their ultimate moment capacity as evidenced by the acceleration cut-off at 0.40 g for the conventionally designed foundation (Fig. 3a), and at 0.19 g for the rocking-isolation alternative (Fig. 3b). Both values are in very good agreement with the previously discussed  $\alpha$  estimates (on the basis of monotonic and cyclic pushover tests).

These observations are confirmed by the  $M-\theta$  loops of Fig. 4. The larger conventionally designed foundation reaches its ultimate moment capacity, but without exhibiting substantial nonlinearity (Fig. 4a). In stark contrast, the smaller foundation of the rocking-isolated system experiences strongly nonlinear response, as evidenced by its oval-shaped  $M-\theta$  loops. As a result (and as it would be expected), the conventional system experiences substantially lower rotation compared to the rocking-isolated system. As evidenced by the  $w-\theta$  curves, the larger foundation demonstrates uplifting-dominated response (observe the very steep edges of the corresponding loops) resulting in minor residual settlement of merely 1.1 cm.

On the contrary, the smaller foundation of the rocking-isolated system moves downwards upon each cycle of rotation, accumulating about three times larger settlement (3.2 cm). However, the superior performance of the larger foundation (with respect to permanent displacements) is unavoidably associated with the development of larger inertia forces. While for the rocking-isolated system the bending moment that develops at the base of the pier is bounded by the inferior moment capacity of the footing ( $M_u \approx 30$  MNm), in the case of the conventionally designed foundation a moment of roughly 60 MNm is allowed to develop, substantially exceeding the capacity of the RC pier ( $M_{up} \approx 46$  MNm). In reality, this would be associated with flexural cracking at the base of the pier, and its survival (or collapse) would be a function of the ratio of ductility demand to ductility capacity. The larger rotation of the smaller foundation is also reflected in the time histories of deck drift (Fig. 5).

The rocking-isolated system experiences substantially larger maximum deck drift  $\delta \approx 10$  cm, as opposed to roughly 6 cm of the conventional system. Interestingly, thanks to the inherent self-centering mechanism of rocking, the residual deck drift is limited to 2.4 cm (instead of 1.9 cm of the conventional system) — a value that can easily be considered tolerable. In reality, however, the system on conventionally designed foundation would be subjected to bending failure, unavoidably experiencing additional permanent drift due to plastic flexural distortion. Although the extent of such additional deformation cannot be quantified, on the basis of numerical analysis results it is almost

certain that the comparison would be largely in favor of the rocking-isolated alternative had the inelastic response of the RC pier been taken into account.

### **3. SEISMIC RESPONSE OF BRIDGE-PIER ON UNDER-DESIGNED DEEP FOUNDATIONS.**

#### **3.1 Unconnected Piles**

Along similar lines, the idea of unconnected piles has been explored in recent years and has been applied in a number of actual projects. In some cases the piles are treated as simply inclusions, aimed at improving the bearing characteristics of the soil. Example: The foundations of the four piers of the Rion-Antirion Bridge.

But the idea of piles not connected to the base of their cap can be extended to earthquake design: as sketched in Fig. 6 using such piles with an interposed stiff sand-and-gravel layer offers a number of advantages for structural performance (e.g. reduced accelerations) while saving the pile heads from large shear forces that result from the inertial loading of the bridge deck.

On the other hand, it was found that the disconnection of the piles from the cap does not lead to any appreciable reduction of the vertical bearing capacity of the foundation, provided that the interposed soil layer (between the raft and pile heads) is adequately stiff.

#### **3.2 Under-designed Embedded Caissons**

In several recent studies it was shown that embedded caissons are currently designed very conservatively, and that despite this fact they do not improve the seismic safety of the whole bridge pier. Ignoring or even reversing the “capacity” design and the conservative rules regarding uplifting of the foundation, leads to reduced size of the caisson and mobilises nonlinearities in the interfaces between caisson and soil and inelastic action in the soil. Increased capacity for energy dissipation and perhaps limited level of transmitted acceleration contribute to better seismic performance.

### **4. CONCLUSIONS**

**(a)** Current seismic design practice leads most often to very conservative foundation solutions. Not only are such foundations un-economical but are sometimes difficult to implement. Most significantly: they are agents of transmitting relatively large accelerations up to the superstructure. The ensuing large inertial forces send back in “return” large overturning moments (and shear forces) onto the foundation — a vicious circle.

**(b)** On the contrary, seriously under-designed foundation dimensions limit the transmitted accelerations to levels proportional to their (small) ultimate moment capacity. This is one of the reasons of achieving much safer superstructures. In

earthquake engineering terminology the plastic “hinging” moves from the columns to the foundation-soil system, preventing dangerous structural damage.

(c) For tall-slender systems that respond seismically mainly in rocking, under-designing the footings “invites” strong uplifting and mobilization of bearing capacity failure mechanisms. It turns out that the statically determined ultimate overturning moment capacity is retained without degradation during cyclic loading, at least for the few numbers of cycles of most events — hence the geotechnical reliability in such a design. Moreover, the cyclic response of such foundations reveals that the amount of damping (due to soil inelasticity and uplifting–retouching impacts) is appreciable, if not large, while the system has a fair re-centering capability. These are some of the secrets of their excellent performance.

(d) The key variable in controlling the magnitude of uplifting versus the extent of bearing–capacity yielding is the static factor of safety  $F_S$  against vertical bearing–capacity failure. The designer may for example, choose to intervene in the subsoil to increase  $F_S$  and hence enhance uplifting over soil inelasticity. Such intervention need only be of small vertical extent, thanks to the shallow dynamic “pressure bulb” of a rocking foundation.

(e) In classical geotechnical engineering, avoiding bearing capacity failure at any cost is an unquestionably prudent goal. Seismic “loading” is different — it is not even loading, but an imposed displacement. Sliding mechanisms develop under the footing only momentarily and hence alternately, and may at worst lead to (increased) settlement. It would be the task of the engineer to “accommodate” such settlements with proper design.

#### **ACKNOWLEDGMENTS**

The financial support for this paper has been provided by the research project “FORENSEIS” (Investigating Seismic Case Histories and Failures of Geotechnical Systems), implemented under the “ARISTEIA” Action of the “Operational Programme Education and Lifelong Learning ” and is co-funded by the European Social Fund (ESF) and national resources.

The help of Marianna Loli in preparing the paper is kindly acknowledged.

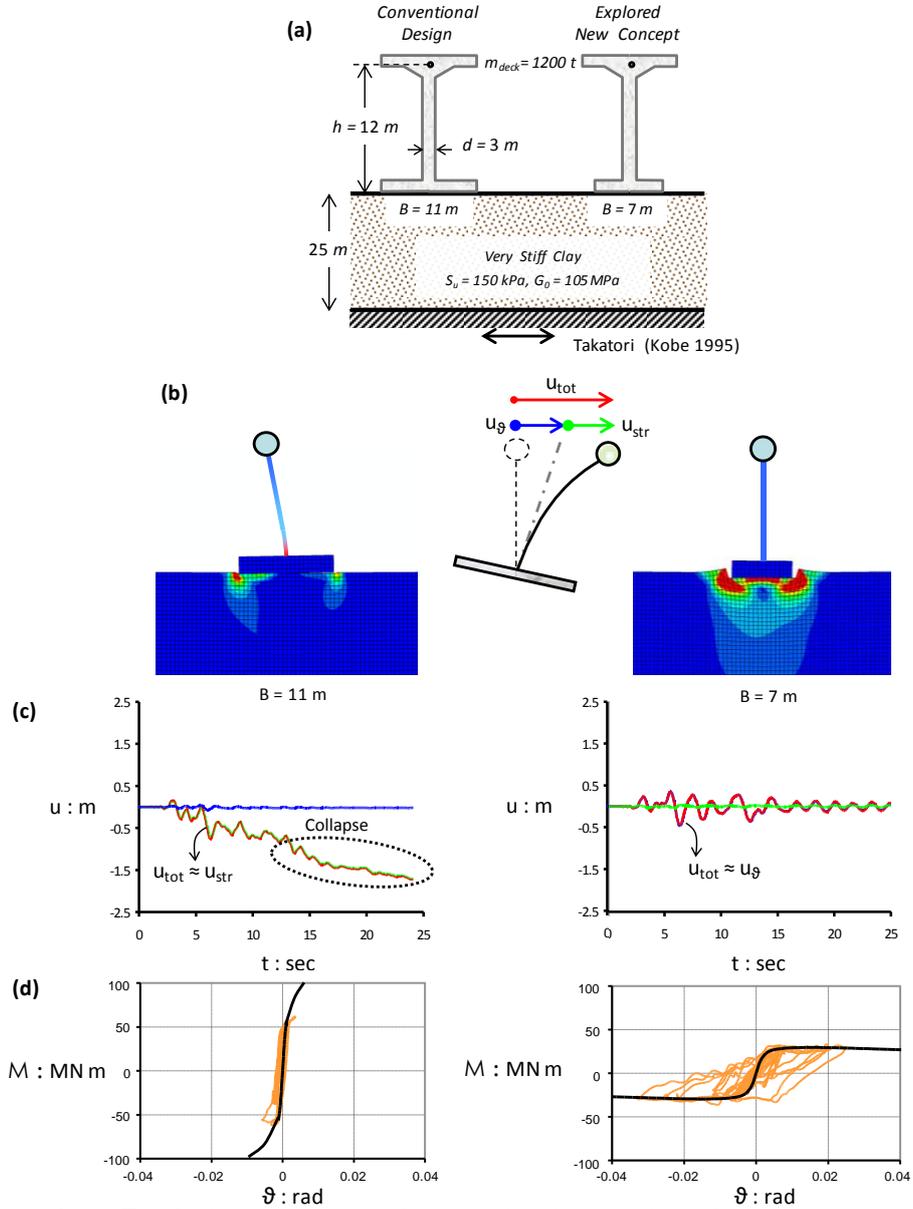


Figure 1. (a) Two bridge piers on two alternative foundations subjected to a large intensity shaking, exceeding the design limits; (b) deformed mesh with superimposed plastic strain, showing the location of "plastic hinging" at ultimate state; (c) time histories of deck drift; (d) overturning moment–rotation ( $M$ – $\theta$ ) response of the two foundations.

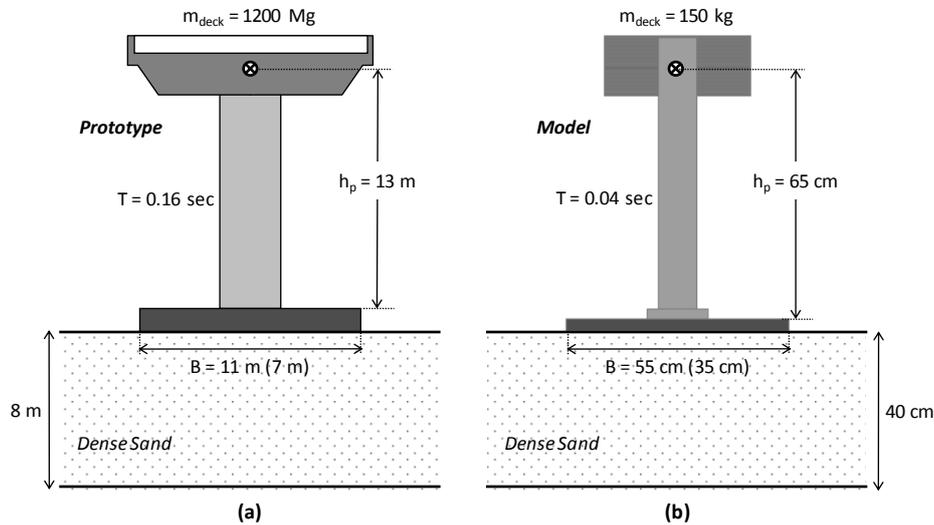
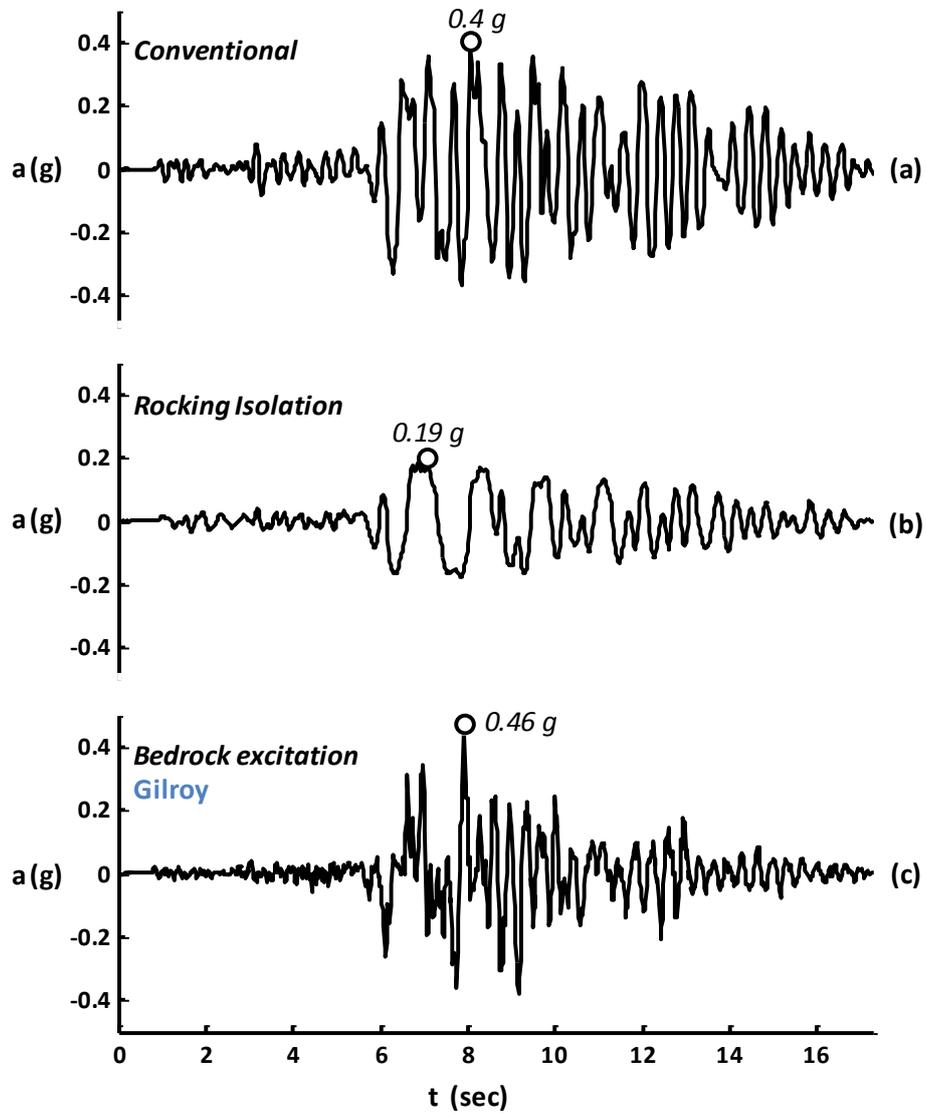


Figure 2. The 1-g experiment : (a) prototype; (b) model (scale = 1:20).

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*Figure 3* Deck acceleration time histories for strong seismic shaking (Gilroy): (a) conventional system with over-designed  $B = 11$  m foundation, compared to (b) rocking isolated alternative with under-designed (to promote uplifting)  $B = 7$  m foundation ; (c) bedrock excitation

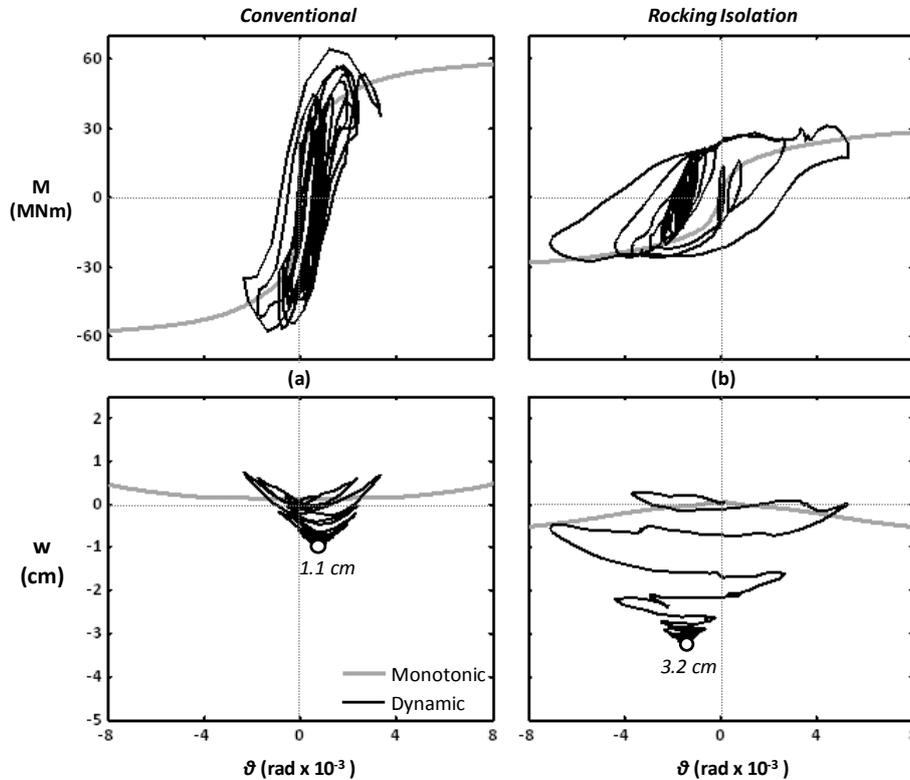


Figure 4. Foundation performance for *strong* seismic shaking (Gilroy). Moment–rotation ( $M-\theta$ ) and settlement–rotation ( $w-\theta$ ) response for : (a) conventional system with over-designed  $B = 11$  m foundation, compared to (b) rocking isolated alternative with under-designed  $B = 7$  m foundation.

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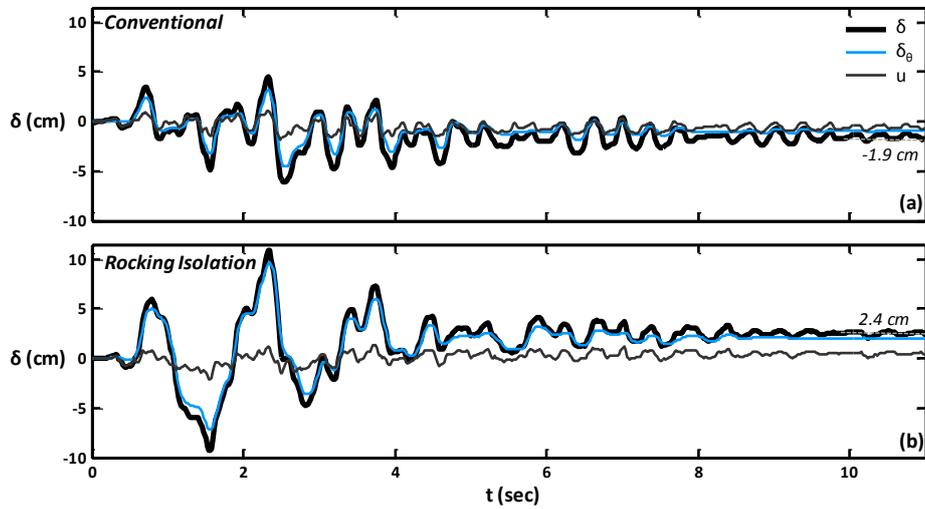


Figure 5. Time histories of deck drift  $\delta$ , due to foundation rotation  $\delta_\theta$  and swaying displacement  $u$ , for strong seismic shaking (Gilroy): (a) conventional system with over-designed  $B = 11$  m foundation, compared to (b) rocking isolated alternative with under-designed  $B = 7$  m foundation

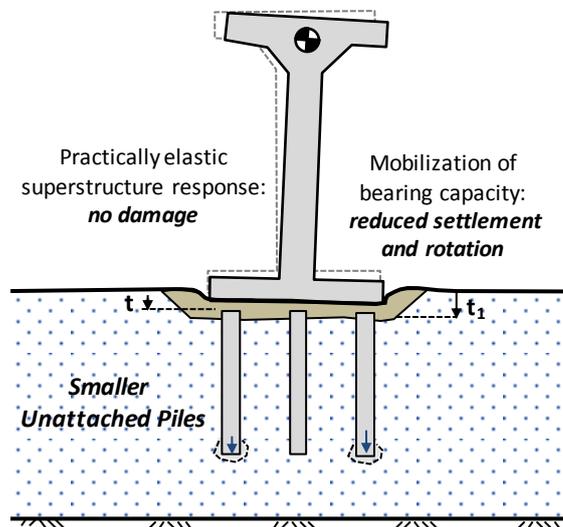


Figure 6. Bridge Pier on unconnected piles with interposed stiff soil layer

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