Towards a Reversal of Seismic Capacity Design : Part B. Shaking-Table Testing of Bridge Pier–Foundation System

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ABSTRACT : This paper investigates experimentally the effectiveness of a new seismic design philosophy, in which soil failure is "utilized" to protect the superstructure. A physical model of a simple bridge pier is used as an example. Two alternatives are considered: one in compliance with conventional capacity design, with over-designed foundation so that the plastic "hinge" will develop at the bridge pier ; and one following the new philosophy, with under-designed foundation, moving the plastic "hinging" will develop in the soil or the soil foundation interface. The seismic performance of the two alternatives is investigated through 1-G shaking table testing using real records and synthetic motions as base excitation. It is shown that the performance of the new design concept can be advantageous : in cases where the conventionally designed system collapses, the new concept can survive the seismic motion with the "damage" being limited to residual deck drift and increased settlement.

1 INTRODUCTION

It has been several decades since the realization that the increase of the strength of a structure does not always enhance safety. Accepting that failure of some structural members cannot always be avoided during strong seismic shaking, earthquake engineering research focused on ensuring : (a) that structural members can sustain dynamic loads that exceed their strength without collapsing—ductility design, (b) that failure is "guided" to members that are less important for the overall integrity of the structure (i.e. beams instead of columns), and (c) that failure is in the form of non-brittle mechanisms (bending instead of shear failure) – capacity design [Park & Paulay, 1976].

With capacity design principles mainly referring to the superstructure, the effect of soil and foundation is usually underestimated. In the words of Priestley [2000] *"the incorporation of foundation compliance effects into force-based design is generally carried out inadequately, if at all"*. Even when foundation compliance is taken into account, little care is given to the nonlinearity of soil and foundation. In fact, current practice in seismic "foundation" design (e.g. EC8), attempts to avoid "at all costs" the mobilization of "strength" in the foundation. In structural terminology : no "plastic hinging" is allowed in the foundation–soil system. In simple geotechnical terms, the designer must ensure that the foundation system will not even reach a number of "thresholds" that would conventionally imply failure. Thus, the following states are prohibited :

(a) mobilization of the "bearing-capacity" failure mechanisms under cyclically-uplifting shallow foundations ;

(b) sliding at the soil–footing interface or excessive uplifting of a shallow foundation ;

(c) passive and shear failure along the sides and base of an embedded foundation ;

"Overstrength" factors and factors of safety larger than 1 are introduced against each of the above "failure" modes, as in static design.

Although such a restriction may appear reasonable (since the inspection and rehabilitation of foundation damage after a strong earthquake is not easy), neglecting such phenomena prohibits the exploitation of strongly non-linear energy dissipating mechanisms in defence of the superstructure in case of occurrence of ground motions larger than design. A growing body of evidence suggests that soil–foundation plastic yielding under seismic excitation is not only unavoidable, but may even be beneficial [Paolucci, 1997; Pecker, 1998; Martin & Lam, 2000; Kutter et al. 2001; Faccioli et al., 2001; Gazetas et al., 2003; Gajan et al., 2005; Apostolou & Gazetas,2005; Kawashima et al., 2007; Gajan & Kutter, 2008; Chatzigogos et al., 2009].

The need for such a "reversal" of current seismic design stems from :

- The uncertainty of predicting the maximum credible earthquake and determining the characteristics of the corresponding seismic motion (PGA, PGV, frequency content, duration, details). For example, the notorious 1995 M_w7.2 Kobe earthquake was generated by an unknown fault, generating PGAs of up to 0.85 g, compared to 0.3 g of the design code [e.g. Gazetas et al., 2005]. In fact, in each new earthquake larger PGAs are recorded. A recent example is the "long-awaited" 2004 M_w6.0 Parkfield earthquake, where the maximum recorded PGA at close proximity to the seismogenic fault reached 1.8 g, accompanied by PGVs of the order of 100 cm/s [Shakal et al., 2006]. Such observations lead to the conclusion that the probability of occurrence of such large near-fault PGAs can be substantial. Thus, the challenge of defining upper bounds on earthquake ground motions [Bommer et al., 2004] can be seen from a different perspective. Therefore, it is considered logical to accept that the risk of occurrence of seismic ground motions larger than assumed in design will always be substantial. It is therefore important to develop new design methods that will allow structures to withstand earthquakes larger than assumed in design without collapsing or sustaining un-reparable damage.
- The necessity of developing economically efficient and environmentally-friendly earthquake protection solutions. The era of global economic crisis urgently calls for a drastic reappraisal of our way of thinking. Seismic safety and protection of human life is – and must remain – the first priority. However, a typical structure will have to withstand a strong earthquake only once or twice in its life. Hence, economy and respect to the environment should also play a role in the design process. So, instead of building larger and stronger (more expensive) foundations to make sure that strong seismic shaking will manage to get to the superstructure (i.e. conventional capacity design), and then reinforce the superstructure so that it may withstand the earthquake without collapsing (making it also more expensive and consuming more-and-more material resources), why not do exactly the opposite : intentionally under-design the foundations to act as "safety valves", limiting the acceleration transmitted onto the superstructure. This way, we may achieve economy in the foundation and the superstructure, without undermining safety. In fact, as it will be shown in the sequel, due to the substantially larger ductility capacity of soil failure mechanisms compared to structural yielding, the new design philosophy may provide increased safety margins.

This paper investigates experimentally the potential effectiveness of a new seismic design philosophy, in which yielding of the soil–foundation system is "utilized" to protect the superstructure — i.e. exactly the opposite of conventional capacity design. The difference between conventional design and the new philosophy is schematically illustrated in Figure 1. A simple but realistic physical model of a bridge was constructed and tested in the shaking table of the Laboratory of Soil Mechanics (of NTUA). The results presented herein can be seen as a first experimental proof of the potential advantages of the new concept. To become applicable in practice, the new design philosophy will have to be extensively verified analytically and experimentally (shaking table and centrifuge testing), something which is the scope of the EU-funded project "DARE" (Soil–Foundation–Structure Systems Beyond Conventional Seismic "Failure" Thresholds).

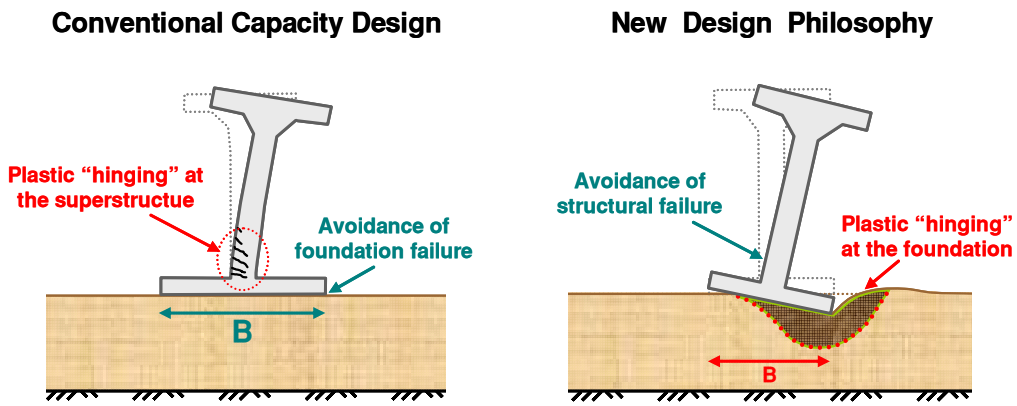


Figure 1. Schematic illustration of the difference between conventional capacity design (plastic "hinging" in the superstructure) with the new design philosophy (plastic "hinging" at the foundation soil).

2 ALTERNATIVE DESIGN PROTOTYPES

As depicted in Figure 2, we consider a typical highway bridge excited in the transverse direction. A deck of mass m = 1200 Mgr is monolithically connected to a reinforced concrete pier of diameter d = 3 m and height H = 12 m. The bridge chosen for shaking table testing is the same with the one analyzed with finite elements in Anastasopoulos et al. [2009], and is intentionally quite similar to the Hanshin Expressway Fukae bridge, which collapsed spectacularly in the Kobe 1995 earthquake [Seible et al., 1995; Iwasaki et al., 1995; Park, 1996]. The bridge prototype is designed in accordance to EC8 [2000] and the Greek Seismic Code [EAK 2000] for design acceleration A = 0.24 g, considering a (ductility-based) behavior factor q = 1.5. With an elastic fixed-base vibration period T = 0.48 sec, the design spectral acceleration is SA = 0.4 g.

The pier is founded through a square foundation of width B on an idealized 15 m deep dense sand layer, of relative density D_r = 85%. Two different foundation widths are considered to represent the two alternative design approaches : a larger foundation, B = 11 m, designed in compliance with conventional capacity design, applying overstrength factor γ_{Rd} = 1.4 to ensure that the plastic "hinge" will develop in the superstructure (base of pier) ; and a smaller, under-designed, B = 7 m foundation, in the spirit of the new design philosophy, applying an "understrength" factor $1/\gamma_{Rd} = 1/1.4 \approx 0.7$ for seismic loading. Thus, the resulting safety factor for seismic loading is lower than 1.0 (FSE ≈ 0.7). In fact, as it will be shown below, the underdesigned foundation will not allow the design seismic action to develop. Hence, FSE does not really have a physical meaning in this case ; it is just an apparent temporary factor of safety.

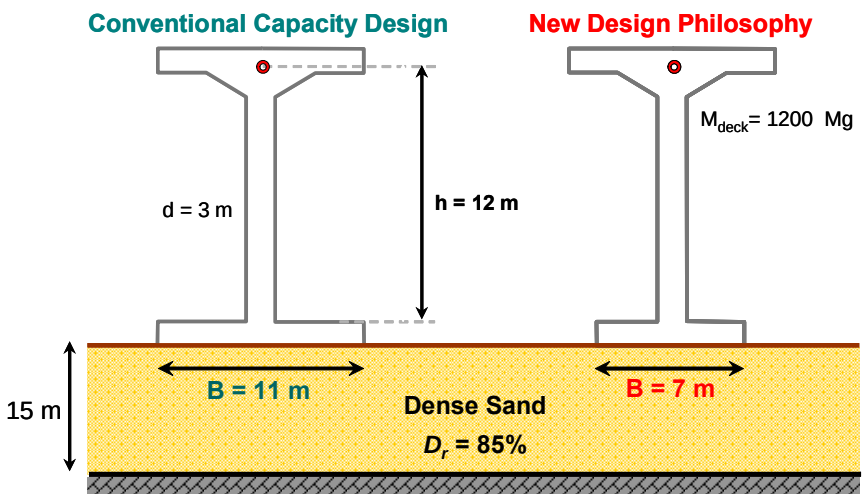


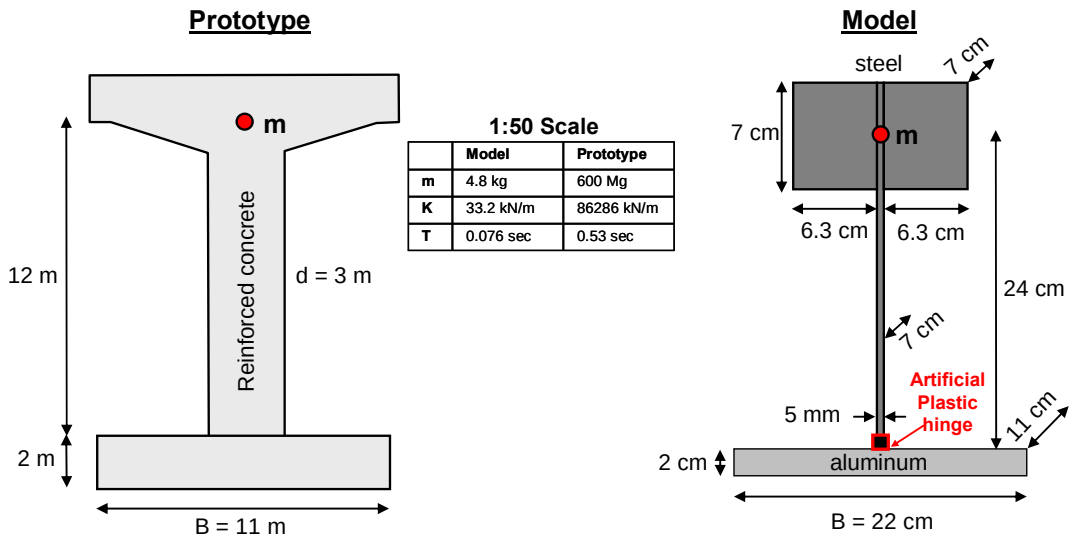
Figure 2. Considered prototype alternatives.

3 PHYSICAL MODEL CONFIGURATION

Physical models of the two bridge alternatives were constructed and tested at the Laboratory of Soil Mechanics of the National Technical University of Athens (NTUA), utilizing a recently installed ANCO R51 shaking table. The table, 1.3 m x 1.3 m in dimensions, is capable of shaking specimens of up to 2 tons at accelerations of up to 1.6 g. Synthetic accelerograms, as well as real earthquake records can be simulated. The actuator is equipped with a servo-valve, controlled by an analog inner-loop control system and a digital outer-loop controller ; it is capable of producing a stroke of ±75 mm.

At this point, it should be noted that the stress field in the backfill soil cannot be correctly reproduced in reduced–scale shaking table modeling, and this is the main advantage of centrifuge testing. However, a centrifuge is substantially more expensive both to acquire and to operate, and is not yet available in NTUA. Shaking table testing can be seen as a valid option, provided that the results are interpreted carefully, with due consideration of scale effects and the stress dependent behaviour of soil.

Taking account of the capacity of the shaking table, a $N = 50$ scale factor was selected for the experiments The selection of model materials was conducted taking account of scaling laws [Gibson, 1997], so that the simulation is as realistic as possible for the given prototype. The bridge piers were constructed using commercially available steel and aluminium plates, as schematically illustrated in Figure 3 (for the conventionally designed alternative).



	Model	Prototype
m	4.8 kg	600 Mg
K	33.2 kN/m	86286 kN/m
T	0.076 sec	0.53 sec

Figure 3. Basic dimensions and details of the physical model, compared to the corresponding prototype (conventionally designed alternative ; half bridge modelled).

At small scale, it is practically impossible to model stiffness correctly (in consistency with the scaling laws) and achieve the desired (scaled) ultimate bending moment capacity of the pier at the same time. Hence, an artificial plastic hinge was custom designed and constructed, and placed at the base of the pier of the conventionally designed alternative. As schematically illustrated in Figure 4a, the ultimate bending moment M_{ult} of the plastic hinge can be calibrated through adjustment of the torque applied at the nut-bolt assembly. To achieve repeatability, two the Teflon washers were added between the bolts and the central steel plate. The calibration of the assembly was performed through static pushover testing, using the experimental configuration of Figure 4b.

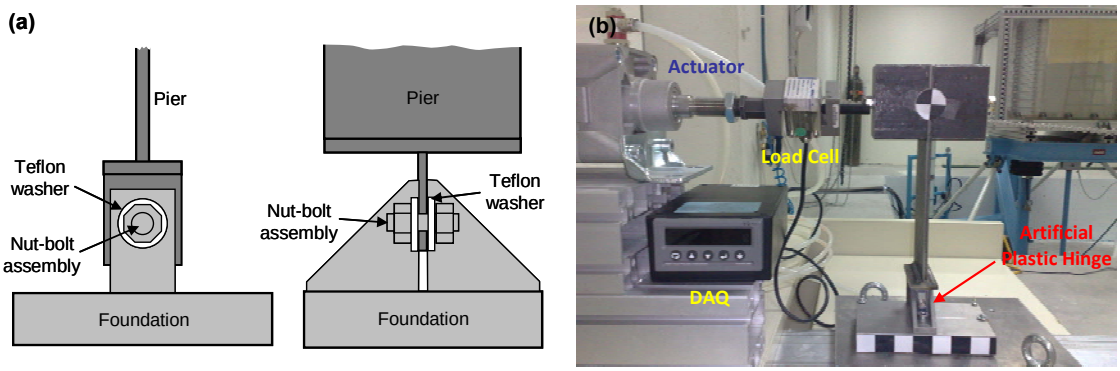


Figure 4. (a) Schematic illustration of the artificial plastic hinge, (b) experimental static pushover setup used for calibration of the ultimate bending moment M_{ult} of the plastic hinge.

The physical models were placed inside a transparent laminar box, custom designed and constructed in NTUA (Figure 5a). The foundation soil consisted of dry "Longstone" (M34) sand, a very fine industrially produced uniform quartz sand with D_{50} = 0.15 mm, uniformity coefficient D_{60}/D_{10} = 1.42, e_{max} = 0.995, e_{min} = 0.614, and G_s = 2.64. The shaking table models were prepared by raining the sand from a specific height with controllable mass flow rate (which controls the density of the sand), using a custom raining system, designed and constructed in NTUA (Figure 5b). For the maximum raining velocity and with the current configuration (width of sand container opening), for Longstone sand the system is capable of achieving relative densities D_r ranging from about 10% to 85%. The tests were conducted at the maximum density (i.e. D_r = 85%).

Tests were conducted with two different configurations : (i) both bridge physical models placed inside the laminar box (Figure 5a), and (ii) each model was placed and tested separately. The purpose of the first configuration was to demonstrate the differences between the two design alternatives under exactly the same conditions. The second comfiguration was used to measure horizontal and vertical displacements of the pier and its foundation. In that case, four accelerometers and four wire displacement transducers were installed, as illustrated in Figure 6. Two accelerometers were placed inside the sand specimen during construction at predetermined positions, to measure soil response at the free-field and under the foundation ; an additional accelerometer was installed at deck level to measure the response of the bridge. Two wire displacement transducers were installed in the horizontal direction to measure the drift of the structure, and two in the vertical direction to measure foundation rotation.

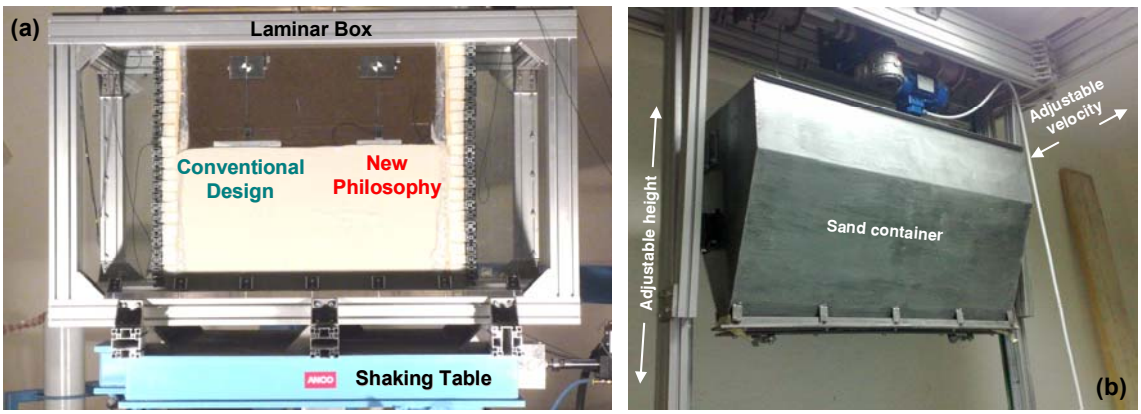


Figure 5. (a) Experimental configuration : the two physical models placed inside the transparent laminar box ; (b) the electronically controlled sand raining system used for preparation of the models.

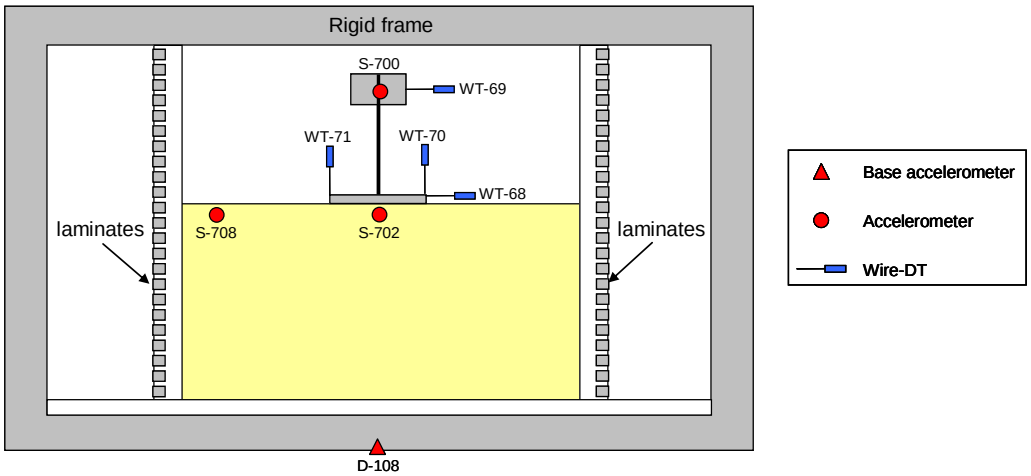


Figure 6. Sketch showing the instrumentation of the shaking table tests using the second configuration.

4 SEISMIC EXCITATION

As depicted in Figure 7, the shaking table tests were conducted using four real records and two artificial accelerograms as seismic excitations. More specifically, we selected the Aegion 1995 and Kalamata 1986 records as representative of moderate intensity earthquakes, Lefkada 2003 as representative of moderate intensity earthquakes but with a large number of strong-motion cycles, and Rinaldi (Northridge 1994) as representative of large magnitude earthquakes. The two artificial 30-cycle sinusoidal motions were used to investigate the performance of the two alternatives in extreme events. Fling–type pulses were also utilized as seismic excitation, but the results are not shown here due to space limitations.

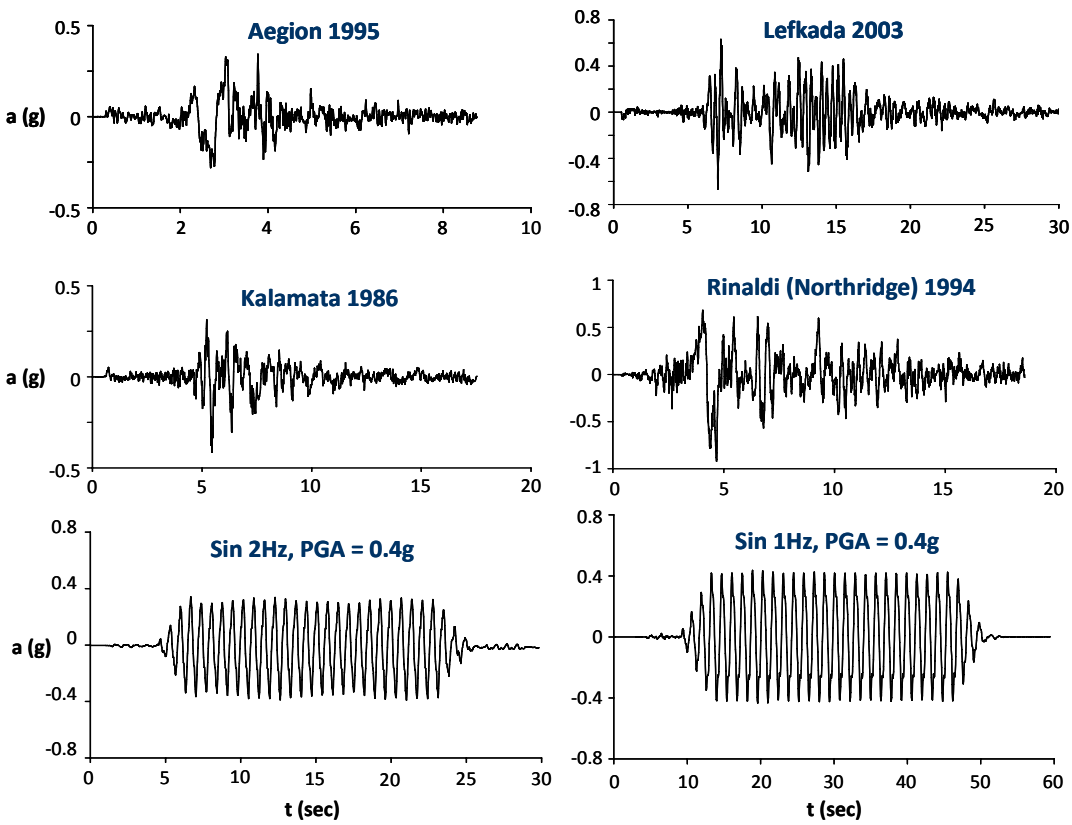


Figure 7. Real and artificial accelerograms utilized as seismic excitation in the experiments.

5 PERFORMANCE IN MODERATE INTENSITY SEISMIC SHAKING

We first compare the performance of the two alternatives subjected to a seismic excitation of moderate intensity. As such an example, we choose the record of the 2003 Ms 6.4 Lefkada (Greece) earthquake [Gazetas et al 2004, 2005, Karakostas et al., 2004].

The comparison is portrayed in Figure 8 in terms of deck acceleration a, deck drift Δ (i.e. horizontal displacement), and foundation settlement w. All results are shown in prototype scale. A first conclusion is that the conventionally designed system experiences larger deck acceleration : approximately 0.4 g instead of roughly 0.3 g. In the first case, the acceleration is limited by the ultimate bending moment of the plastic hinge ; in the latter case by the ultimate capacity of the under-designed foundation.

As schematically illustrated in the sketch notation, the drift has two components [see also Priestley et al., 1996] : (i) the "flexural drift" Δ_C , i.e. the structural displacement due to flexural distortion of the pier column, and (ii) the "rocking drift" $\Delta_r = \theta H$, i.e. the displacement due to rocking motion of the foundation. This way, the contribution of pier flexural distortion and foundation rotation to the final result of interest (i.e. the total drift Δ) can be inferred. As shown in the figure, while for the conventional design Δ is mainly due to pier distortion Δ_C , the opposite can be observed for the under-designed foundation : Δ is mainly due to foundation rotation Δ_r . Despite the differences in the mechanism leading to the development of Δ (pier distortion versus foundation rotation), the total drift is practically the same, and tolerable in both cases.

In terms of foundation settlement w, the conventionally designed system is subjected to limited settlement $w \approx 1.5$ cm ; the new concept experiences larger, but tolerable dynamic settlement : $w \approx 6$ cm.

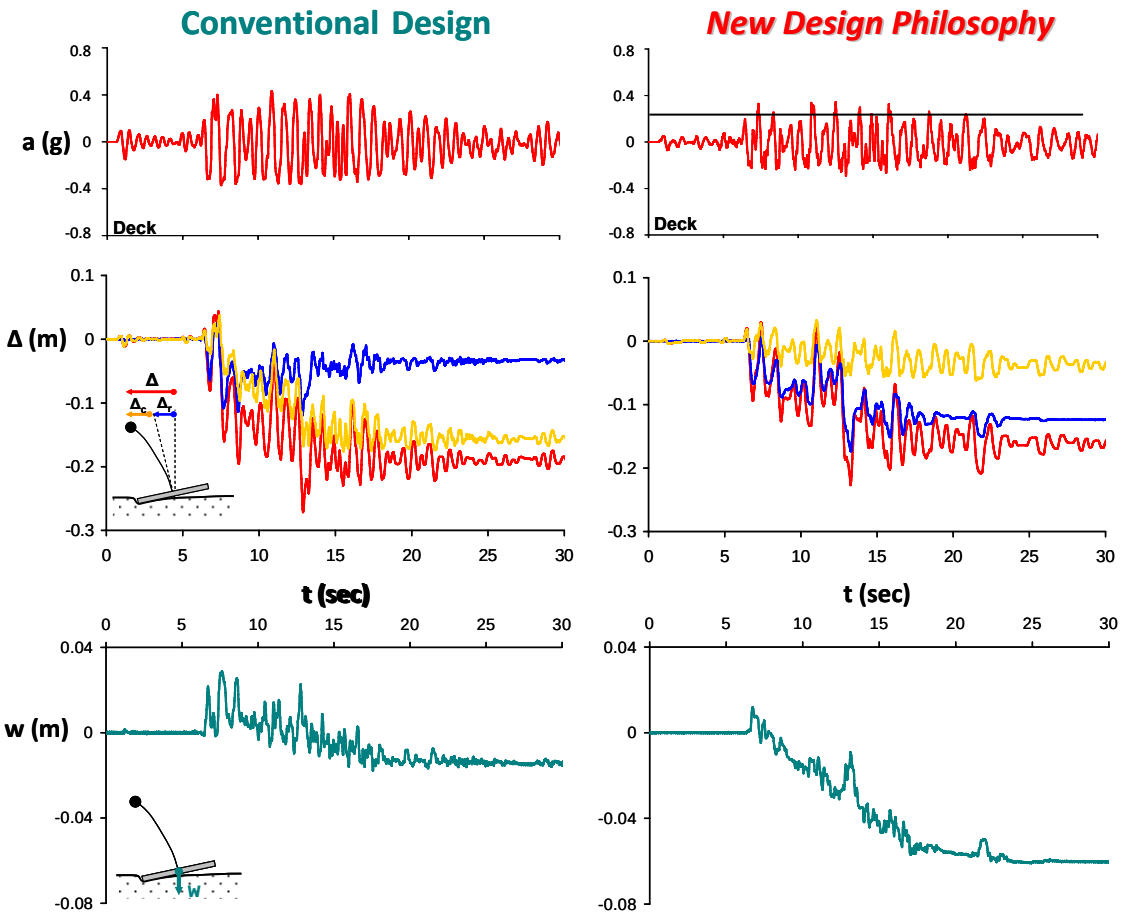


Figure 8. Comparison of the response of the two systems subjected to the Lefkada 2003 seismic excitation: acceleration time histories at deck level, deck drift time histories (total, flexural, and rotational), and foundation settlement time histories.

6 PERFORMANCE IN LARGE INTENSITY SEISMIC SHAKING

We now use the Rinaldi record of the 1994 Northridge M_s 6.8 earthquake [Trifunac et al., 1998] as an example of large intensity seismic shaking. With PGA = 0.79 g and PGV = 164 cm/s, and a very strong forward rupture directivity pulse, this record can be seen to constitute a very severe seismic motion.

Figure 9 synopsizes the comparison in terms of deck acceleration a, deck drift Δ, and foundation settlement w. As for the previous case, the conventionally designed system experiences larger deck acceleration compared to the new design concept.

In terms of deck drift, as for the Lefkada 2003 seismic excitation, Δ is almost purely related to flexural pier drift Δ_C for the conventional design. The opposite is observed for the new design concept : Δ is mainly due to foundation rotation Δ_r . In contrast to the previous case, the conventionally designed bridge is subjected to a maximum drift of about 65 cm (i.e. more than 5% of the pier height) and a residual drift of roughly 35 cm (i.e. about 3% of the pier height). In reality, such flexural distortion would probably imply collapse – or very serious damage – of the reinforced concrete pier. The reason why no collapse was observed in the experiment is none other than the ductility capacity of the artificial plastic hinge, which is substantially larger than the capacity of a reinforced concrete section.

Markedly different is the performance of the new design concept. The maximum drift is not exceeding 35 cm (i.e. roughly 3% of the pier height), and the residual drift is limited to less than 13 cm (i.e. about 1% of the pier height). Most importantly, this drift is not associated with pier structural damage but with inelasticity of the soil supporting the foundation. Naturally, the increase of settlement is the price to pay : $w \approx 3.5$ cm, instead of less than 2 cm of the conventionally designed system.

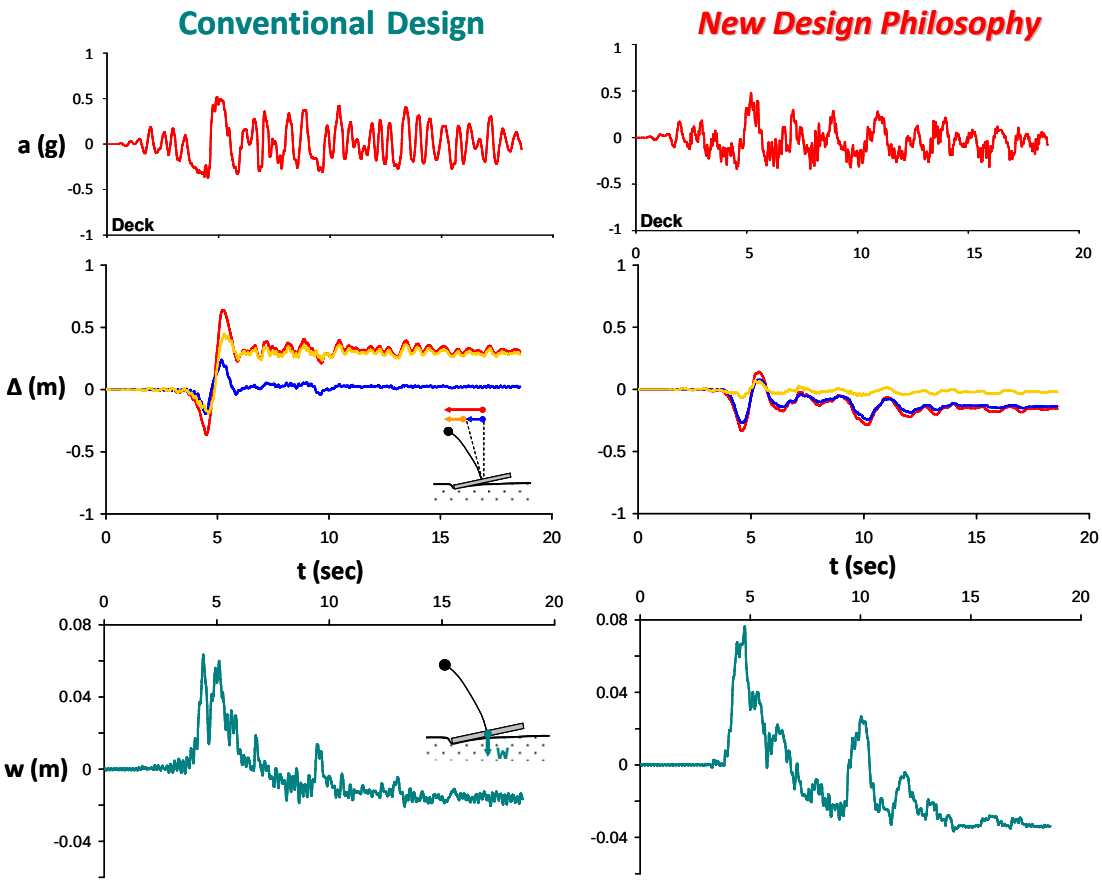


Figure 9. Comparison of the response of the two systems subjected to the Rinaldi (Northridge) seismic excitation : acceleration time histories at deck level, deck drift time histories (total, flexural, and rotational), and foundation settlement time histories.

7 PERFORMANCE UNDER EXTREME SHAKING

We now compare the response of the two systems subjected to the artificial 30-cycle sinusoidal motions, aiming to investigate their performance in extreme events.

7.1 *30-cycle sinus at 2 Hz*

Figure 10 compares the response of the two alternatives in terms of deck acceleration and drift, and foundation settlement.

The comparative performance of the two alternatives is qualitatively similar, with their main differences being more pronounced due to the large number of strong motion cycles. In particular, observe the accumulation of deck drift for both alternatives. In the case of the conventionally designed system, Δ is mostly due to flexural pier drift Δ_C , but some rotational drift Δ_r is also accumulated. As for the previous cases, exactly the opposite is observed for the new design concept. The conventionally designed bridge is subjected to a residual drift of about 90 cm (i.e. about 7.5% of the pier height). Obviously, in reality such flexural distortion would imply collapse of the bridge, and (as for Rinaldi) the survival in the experiment is only due to the unrealistically large ductility capacity of the artificial plastic hinge.

The accumulation of drift in the new design concept is much smaller : the residual drift does not exceed 20 cm (i.e. less than 2% of the pier height). Exactly the opposite conclusion can be drawn for the settlement : $w \approx 14$ cm for the new concept versus 8 cm of the conventionally designed system.

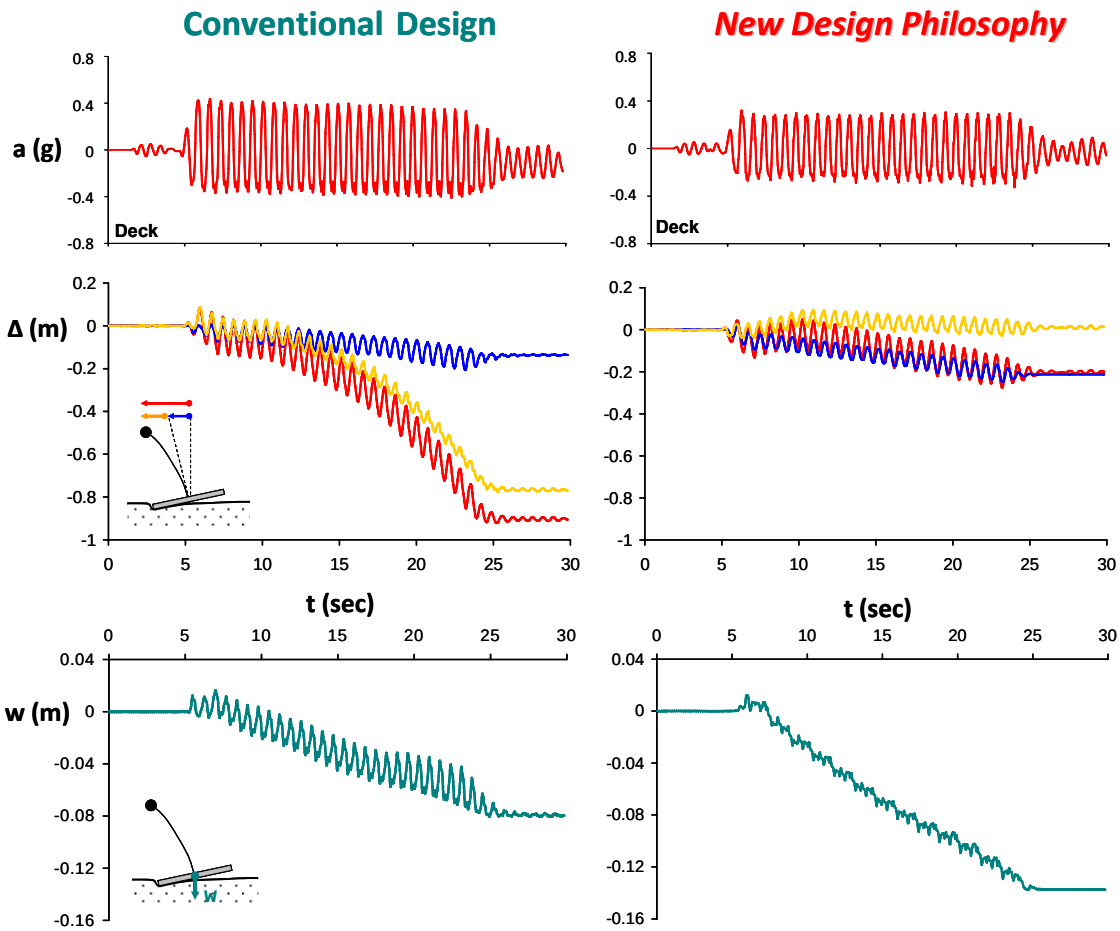


Figure 10. Comparison of the response of the two systems subjected to 30 cycle 2 Hz sinusoidal seismic excitation of PGA = 0.4 g : acceleration time histories at deck level, deck drift time histories (total, flexural, and rotational), and foundation settlement time histories.

7.2 *30-cycle sinus at 1 Hz*

We now compare the response of the two alternatives subjected to a small frequency (f = 1 Hz) seismic excitation. It is noted that such an excitation cannot be claimed to be representative of any real earthquake. However, it is useful to demonstrate the substantially larger safety margins of the system designed according to the new philosophy.

As shown in Figure 11, while the conventional system collapses at t = 23 sec (i.e. after 10 strong-motion cycles), the bridge with the under-designed foundation (new concept) survives this tremendous seismic motion at the price of residual deck drift $\Delta \approx 50$ cm and settlement $w \approx 18$ cm.

Before collapse, the performance of the two alternatives is qualitatively similar to the previous cases : lager deck acceleration and Δ predominantly associated with flexural pier drift Δ_C for the conventional system ; larger settlement w and Δ mostly due to rotational drift Δ_C for the new concept.

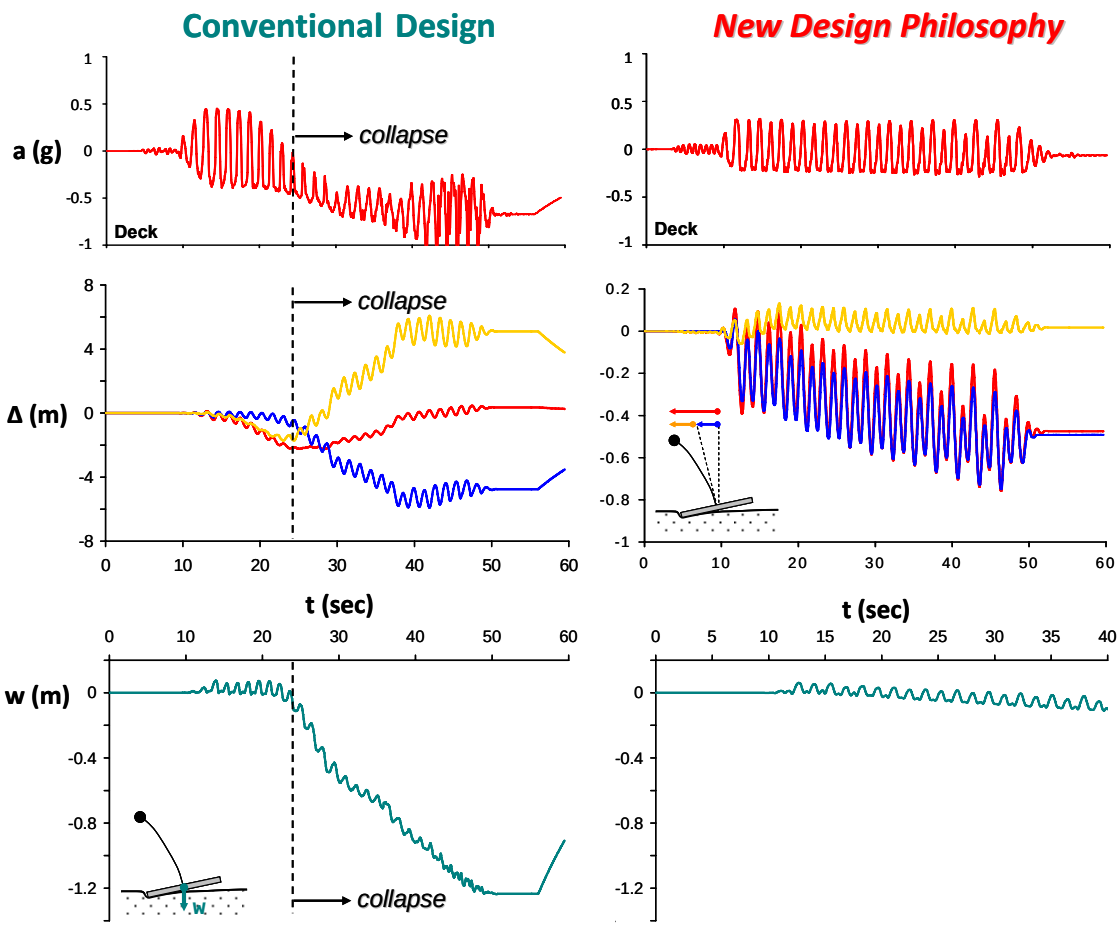


Figure 11. Comparison of the response of the two systems subjected to 30 cycle 1 Hz sinusoidal seismic excitation of PGA = 0.4 g : acceleration time histories at deck level, deck drift time histories (total, flexural, and rotational), and foundation settlement time histories.

8 CONCLUSIONS

The purpose of this paper was to investigate experimentally the effectiveness of a new seismic design philosophy, in which soil failure is "utilized" to protect the superstructure. The seismic performance of the new concept (involving an under-designed foundation) has been compared to a conventionally designed bridge (with over-designed foundation).

A simple but realistic physical model of a bridge was constructed and tested in the shaking table of the Laboratory of Soil Mechanics (of NTUA). The results presented herein can be seen as a first experimental proof of the potential advantages of this new concept. An artificial plastic hinge was custom designed and constructed, to model the nonlinear response of the bridge pier at small scale.

For moderate intensity seismic motions, the performance of both alternatives is totally acceptable, and both would be utilizable right after such an earthquake. The conventionally designed bridge would probably sustain limited structural damage (minor flexural cracking), and would be easily repairable. On the other hand, the new concept would be subjected to slightly increased – but tolerable – deck drift and settlement, but would remain structurally unscathed.

The advantage of the new seismic design philosophy becomes clear for large intensity seismic motions, clearly exceeding the limits of the design. In such cases, while the conventionally designed system is driven to collapse, the new concept may survive the seismic motion with the damage being "contained" in the form of deck drift and settlement.

The key conclusions of this work are in line with the results of the analytical work (referring to the same bridge, under different soil conditions) presented in Anastasopoulos et al. [2009].

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