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Soil failure can be used for seismic protection of structures

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Abstract A new seismic design philosophy is illuminated, taking advantage of soil "failure" to protect the superstructure. Instead of *over-designing* the foundation to ensure that the loading stemming from the structural inertia can be "safely" transmitted onto the soil (as with conventional capacity design), and then reinforce the superstructure to avoid collapse, why not do exactly the opposite by intentionally *under-designing* the foundation to act as a "safety valve" ? The need for this *"reversal"* stems from the uncertainty in predicting the actual earthquake motion, and the necessity of developing new *more rational and economically efficient* earthquake protection solutions. A simple but realistic bridge structure is used as an example to illustrate the effectiveness of the new approach. Two alternatives are compared : one complying with conventional capacity design, with *over-designed* foundation so that plastic "hinging" develops in the superstructure; the other following the new design philosophy, with *under-designed* foundation, "inviting" the plastic "hinge" into the soil. Static "pushover" analyses reveal that the ductility capacity of the new design concept is an order of magnitude larger than of the conventional design: the advantage of "utilising" progressive soil failure. The seismic performance of the two alternatives is investigated through nonlinear dynamic time history analyses, using an ensemble of 29 real accelerograms. It is shown that the performance of both alternatives is totally acceptable for moderate intensity earthquakes, *not exceeding the design limits*. For large intensity earthquakes, *exceeding the design limits*, the performance of the new design scheme is proven advantageous, not only avoiding collapse but hardly suffering any inelastic structural deformation. It may however experience increased residual settlement and rotation: a price to pay that must be properly assessed in design.

Keywords Capacity design · Bearing capacity failure · Uplifting · Seismic performance · Dynamic analysis · Pushover · Constitutive modelling · Calibration through experimental data

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1 Introduction: the need for a new design philosophy

It has been more than 30 years since the realization of the earthquake engineering community that the increase of strength of a structural system does not necessarily enhance safety. This recognition has lead to the development of new design principles, aiming at rationally controlling seismic damage and rendering the structure "*fail-safe*". A *fail-safe* system can be defined as a system in which failure of some elements or subsystems, caused by unexpectedly extreme loading, does not lead to the collapse because an alternative load path is developed by the remaining elements or subsystems (Frangopol and Curley 1987). Accepting that failure of structural members cannot always be avoided, earthquake engineering research focused on ensuring: (1) that structural members can sustain dynamic loads that exceed their strength without collapsing—*ductility design*; (2) that failure is "*guided*" to members that are less important for the overall integrity of the structure (i.e. beams instead of columns); and (3) that failure is in the form of non-brittle mechanisms (bending instead of shear failure)—*capacity design* (Park and Paulay 1975).

However, while substantial research and regulatory efforts have been devoted to developing "*fail-safe*" (robust) structures, less attention has been given to the soil-structure system as a whole. Capacity design principles mainly refer to the superstructure, usually underestimating the effect of soil and foundation. 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, particularly as entrenched in seismic codes (e.g. EC8), attempts to avoid 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:

- mobilization of the "bearing-capacity" failure mechanisms under cyclically-uplifting shallow foundations;
- sliding at the soil-footing interface or excessive uplifting of a shallow foundation;
- passive and shear failure along the sides and base of an embedded foundation;

"Overstrength" factors plus (explicit and implicit) 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 (the inspection and rehabilitation of foundation damage after a strong earthquake is not easy), it may lead to nonconservative oversimplifications, especially in the case of strong geometric nonlinearities, such as foundation uplifting and sliding (e.g. Harden and Hutchinson 2006). Most importantly, neglecting such phenomena prohibits the exploitation of strongly non-linear energy dissipating mechanisms in defense of the superstructure in case of occurrence of ground motions larger than design. Today, 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, 2003; Martin and Lam 2000; FEMA 356 2000; Kutter et al. 2003; Faccioli et al. 2001; Gazetas et al. 2003; Gajan et al. 2005, 2008; Mergos and Kawashima 2005; Apostolou and Gazetas 2005; Paolucci et al. 2007; Kawashima et al. 2007; Gajan and Kutter 2008; Chatzigogos et al. 2009; Gerolymos et al. 2008, 2009).

This paper introduces a *new seismic design philosophy*, in which yielding of the soil-foundation system is "utilised" *to protect the superstructure*—exactly the opposite of

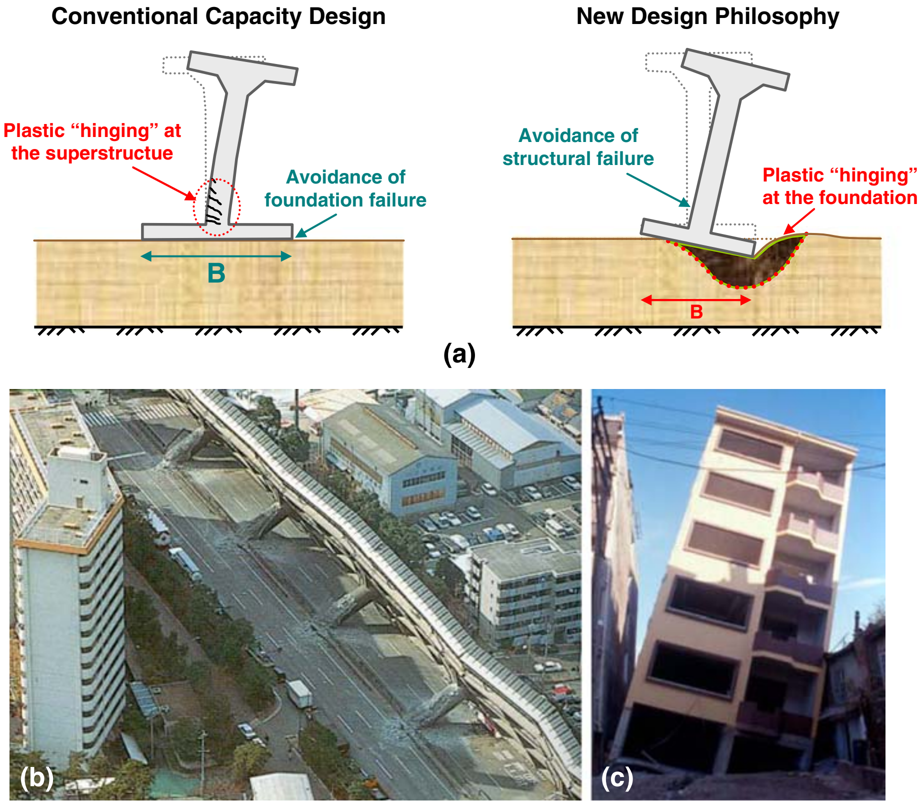

Fig. 1 **a** Conventional capacity design (plastic "hinging" in the superstructure) compared with the new design philosophy (plastic "hinging" below ground). **b** Real example of plastic "hinging" in the superstructure: collapse of 18 spans of the Fukae bridge (of Hanshin Expressway Route 3) during the Kobe 1995 earthquake. The bridge had been designed in the 60s, for much lower levels of acceleration than what it really experienced, and before modern seismic design concepts had been recognized. **c** Real example of unintended plastic "hinging" in the foundation: excessive tilting of a slender building founded on very soft soil in Adapazari after the Kocaeli (Turkey) 1999 earthquake. Due to foundation failure, the superstructure remained totally unscathed. The price to pay was heavy however: excessive rotation leading to collapse in many cases

conventional capacity design (in which plastic "hinging" is restricted to the superstructure). Figure 1a schematically illustrates the difference between conventional design and the new concept, and provides a real example of plastic "hinging" in the superstructure (Fig. 1b), and a real example of unintended plastic "hinging" in the foundation (Fig. 1c). The latter shows the excessive tilting of a slender building on very soft soil in Adapazari (Turkey, 1999), where soil failure can be seen to have (unintentionally) acted as a "shield" for the superstructure, which remained structurally unscathed. Naturally there is always a price to pay, which is none other than permanent rotation and settlement—in this particular case excessive, but not always so.

The need for this "*reversal*" of current seismic design stems from:

(a) *The inherent 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). Interestingly, there were several other records at similar distances from the fault where the PGAs were even an order of magnitude lower! Such observations lead to the conclusion that the probability of occurrence of such large near-fault PGAs can be substantial. On the other hand, the probability of capturing such records can be seen as a function of the density of accelerograph networks: i.e. it was the extreme density of instrumentation that allowed the recording of the aforementioned PGAs. Obviously, if the fault was not well-documented, and money had not been spent on instrumentation, these records would not exist. With such evidence, 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. Naturally, the magnitude of this risk will depend on the assumed earthquake hazard levels. On the other hand, evidence is accumulating that shows that PGA (alone) is not *the* crucial parameter. The frequency content, the pulse sequence, and the asymmetry of motion, may indeed be of more importance (e.g. Makris and Roussos 2000; Fardis et al. 2003). 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-repairable damage.

(b) *The necessity of developing economically efficient 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.

To unravel the effectiveness of the new design philosophy (compared to conventional capacity design), a simple but realistic bridge structure is used as an example. The results presented herein can be seen as a first demonstration 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*).

2 Design considerations and analysis methodology

As depicted in Fig. 2a, 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 analysis is similar to the Hanshin Expressway Fukae bridge (see also Fig. 1b), which collapsed spectacularly in the Kobe 1995 earthquake (Seible et al. 1995; Iwasaki et al. 1995; Park 1996). The bridge is designed in accordance to EC8 (2000) and the Greek Seismic Code (EAK 2000) for a design acceleration $A = 0.24$ g, considering a (ductility-based) behavior factor $q = 2$. With an elastic (fixed-base) vibration period $T = 0.48$ s and design spectral acceleration $\mathrm{SA} = 0.3$ g,

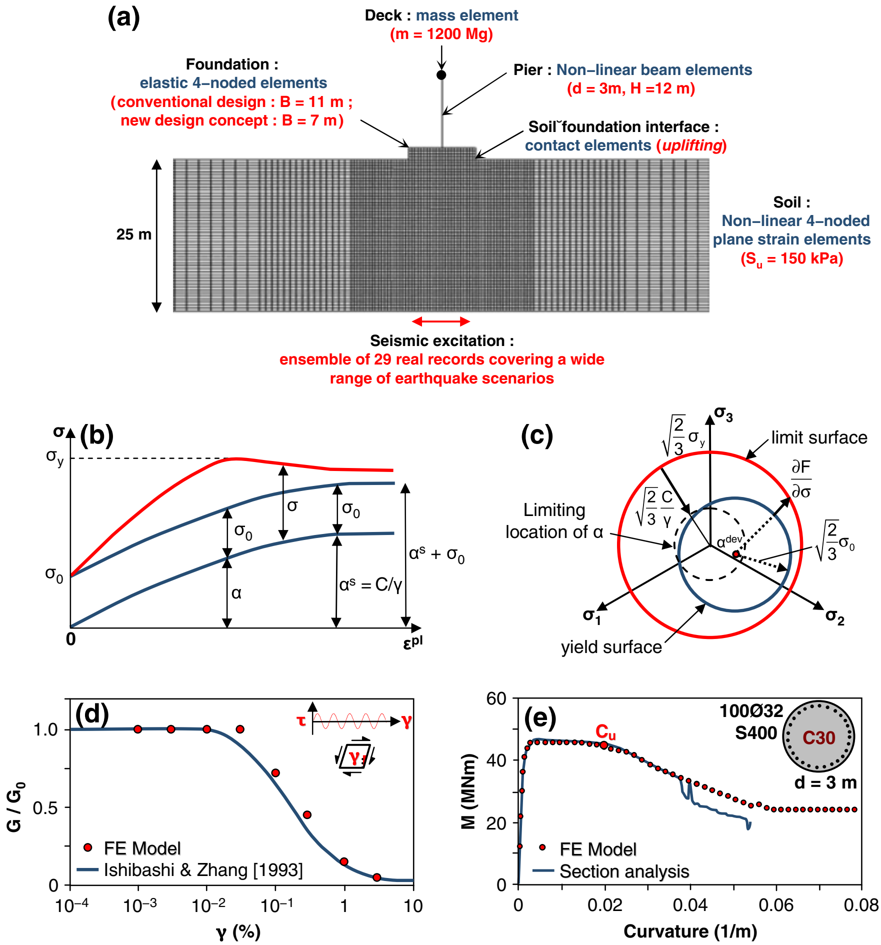

Fig. 2 **a** Overview of the finite element modeling: plane-strain conditions are assumed, taking account of material (soil and superstructure) inelasticity and geometric (uplifting and P–Δ effects) nonlinearities. **b** Simplified one-dimensional representation of the hardening. **c** Three-dimensional representation of the hardening in the nonlinear isotropic/kinematic model. **d** Calibration of kinematic hardening model for soil (stiff clay, $S_u = 150$ kPa) against published G–γ ($PI = 30$, $\sigma_v = 100$ kPa) curves (Ishibashi and Zhang 1993). **e** Model calibration for the superstructure against moment–curvature response calculated using reinforced concrete cross-section analysis (USC_RC)

to undertake the resulting design bending moment $M_{\mathrm{D}} \approx 43$ MNm, a longitudinal reinforcement of 100 $d_{\mathrm{bL}} = 32$ mm bars (100Φ32) is required, combined with $d_{\mathrm{bw}} = 13$ mm hoops spaced at 8 cm.

The pier is founded through a square foundation of width B on an idealised homogeneous 25 m deep stiff clay layer, of undrained shear strength $S_{\mathrm{u}} = 150$ 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 larger foundation, $B = 11$ m, is designed in compliance with conventional capacity design, applying an overstrength factor $\gamma_{\mathrm{Rd}} = 1.4$ to ensure that the plastic "hinge" will develop in the superstructure (base of pier). Taking account of maximum allowable uplift (eccentricity $e = M/V\ B/3$, where V is the vertical load), the resulting safety factors for static and seismic loading are $FS_V = 5.6$ and $FS_E = 2.0$, respectively. A smaller, under-designed, $B = 7$ m foundation is considered in the spirit of the new design philosophy. Its static safety factor $FS_V = 2.8$, but it is designed applying an "*understrength*" factor $1/\gamma_{\mathrm{Rd}} = 1/1.4 \approx 0.7$ for seismic loading. Thus, the resulting safety factor for seismic loading is lower than 1.0 ($FS_E \approx 0.7$). In fact, as it will be shown below, the *underdesigned* foundation will not allow the design seismic action to develop. Hence, FS_E does not really have a physical meaning in this case; it is just an *apparent* temporary factor of safety.

The analysis is conducted assuming plane-strain soil conditions, taking account of material (in the soil and the superstructure) and geometric (due to uplifting and P–Δ effects) nonlinearities. The pier is modeled with nonlinear beam elements, while the deck is represented by a mass element. Soil and foundation are modeled with quadrilateral continuum elements, nonlinear for the former and elastic for the latter. The foundation is connected to the soil with special contact elements, allowing for realistic simulation of possible detachment and sliding at the soil-foundation interface. The mass of the footing and of the pier are also taken into account.

2.1 Soil inelasticity

Soil behavior is modeled through a nonlinear constitutive model with Von Mises failure criterion, nonlinear kinematic hardening and associated plastic flow rule. According to the Von Mises failure criterion, the evolution of stresses is described by the relation:

$$\sigma = \sigma_0 + \alpha \tag{1}$$

where σ_0 is the value of stress at zero plastic strain, assumed to remain constant. The parameter α is the "backstress", which defines the kinematic evolution of the yield surface in the stress space. An associated plastic flow rule is assumed:

$$\dot{\epsilon}^{\mathrm{pl}} = \dot{\bar{\epsilon}}^{\mathrm{pl}} \frac{\partial F}{\partial \sigma} \tag{2}$$

where $\dot{\epsilon}^{\mathrm{pl}}$ is the plastic flow rate (obtained through the equivalent plastic work), $\dot{\bar{\epsilon}}^{\mathrm{pl}}$ the equivalent plastic strain rate, and F a function defining the pressure-independent yield surface:

$$F = f(\sigma - \alpha) - \sigma_0 \tag{3}$$

The evolution law of the model consists of two components: a nonlinear kinematic hardening component, which describes the translation of the yield surface in the stress space (defined through the backstress α), and an isotropic hardening component, which describes the change of the equivalent stress defining the size of the yield surface σ_0 as a function of plastic deformation. The kinematic hardening component is defined as an additive combination of a purely kinematic term (linear *Ziegler* hardening law) and a relaxation term (the *recall* term), which introduces the nonlinearity. The evolution of the kinematic component of the yield stress is described as follows:

$$\dot{\alpha} = C\frac{1}{\sigma_0}(\sigma - \alpha)\dot{\bar{\epsilon}}^{\mathrm{pl}} - \gamma\alpha\dot{\bar{\epsilon}}^{\mathrm{pl}} \tag{4}$$

where C the initial kinematic hardening modulus ($C = \sigma_{\mathrm{y}}/\epsilon_{\mathrm{y}} = E$) and γ a parameter that determines the rate at which the kinematic hardening decreases with increasing plastic deformation.

The evolution of the kinematic and the isotropic hardening components is illustrated in Fig. 2b and c for unidirectional and multiaxial loading, respectively. The evolution law for the kinematic hardening component implies that the backstress is contained within a cylinder of radius:

$$\sqrt{\frac{2}{3}\alpha^{s}} = \sqrt{\frac{2}{3}}\frac{C}{\gamma} \tag{5}$$

where α^{s} is the magnitude of α at saturation. Since the yield surface remains bounded, this implies that any stress point must lie within a cylinder of radius $\sqrt{2/3}\sigma_{\mathrm{y}}$. At large plastic strains, any stress point is contained within a cylinder of radius $\sqrt{2/3}\,(\alpha^{\mathrm{s}} + \sigma^{\mathrm{s}})$ where σ^{s} is the equivalent stress defining the size of the yield surface at large plastic strain.

The maximum yield stress (at saturation) is:

$$\sigma_{\mathrm{y}} = \frac{C}{\gamma} + \sigma_0 \tag{6}$$

According to the Von Mises yield criterion this ultimate stress is:

$$\sigma_{\mathrm{y}} = \sqrt{3}S_{\mathrm{u}} \tag{7}$$

From Eqs. 6 and 7 we have:

$$\gamma = \frac{C}{\sqrt{3}S_{\mathrm{u}} - \sigma_0} \tag{8}$$

Model parameters are calibrated to fit published G–γ curves of the literature, following the procedure described in Gerolymos et al. (2005). Figure 2d illustrates the validation of the kinematic hardening model (through simple shear finite element analysis) against published G–γ curves by Ishibashi and Zhang (1993).

2.2 Pier inelasticity

The same constitutive model is calibrated to match the pier response in the macroscopic moment–curvature level. The reinforcement of the pier circular section ($D = 3$ m) is calculated according to the provisions of the Greek Code for Reinforced Concrete (ΕΚΩΣ, 2000) for columns with large capacity demands in accordance with the capacity design provisions. The moment curvature relationship is derived from static concrete section analysis employing the USC_RC software, which uses the Mander model (Mander et al. 1988) to simulate the stress–strain relationship of confined concrete.

The bending moment of a circular section is by definition related to the normal stresses σ with the following expression:

$$M = 2\int_{0}^{\pi}\int_{0}^{d/2} \sigma r^2 \sin\theta \mathrm{d}r\mathrm{d}\theta \tag{9}$$

For the maximum yield stress σ_y this relationship gives:

$$M_y = 2\sigma_y \int_{0}^{\pi} \frac{r^3}{3} \sin\theta|_0^{d/2} \mathrm{d}\theta \tag{10}$$

which yields:

$$M_y = \frac{1}{6}\sigma_y d^3 \tag{11}$$

And so, the maximum yield stress can be expressed as:

$$\sigma_y = \frac{6My}{d^3} \tag{12}$$

The initial kinematic hardening modulus C is equal to the modulus of elasticity E.

To simulate the softening behavior of the reinforced concrete section after ultimate capacity is reached, a user subroutine is encoded in the ABAQUS finite element code. Figure 2e depicts the results of model calibration for the pier against moment–curvature relation of the reinforced concrete section calculated through section analysis utilising the USC_RC software (Esmaeily-Gh and Xiao 2002), which uses the Mander model (Mander et al. 1988) for confined concrete. As for soil, model parameters are calibrated using the aforementioned methodology of Gerolymos et al. (2005).

3 Static pushover analysis

Before proceeding with the dynamic time history analysis of the two alternatives, we investigate their response in terms of monotonic loading through simulation of the static "*pushover*" test. Displacement controlled horizontal loading is applied at the top of the pier (deck). Figure 3a illustrates the results of the static pushover analysis of the conventionally designed system, in terms of moment–curvature relation at the base of the pier. The curvature ductility capacity μ_ϕ of the reinforced concrete section is equal to 16.6 (applying a standard bilinear approximation), and the displacement ductility capacity of the pier is computed as follows (Priestley et al. 1996):

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} = \frac{M_u}{M_n} + 3(\mu_r - 1)\frac{L_p}{H}\left(1 - 0.5\frac{L_p}{H}\right) \tag{13}$$

where M_u the ultimate and M_n the "yield" bending moment of the reinforced concrete section (corresponding to c_n in the moment curvature diagram), H the height of the pier, and L_p the length of the plastic hinge:

$$L_p = 0.08L + 0.022 f_{ye} d_{bl} \geq 0.044 f_{ye} d_{bl} \tag{14}$$

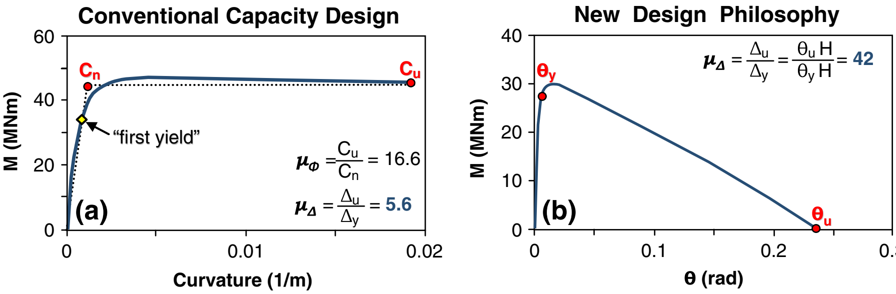

Fig. 3 **a** Static "pushover" analysis of the conventionally designed system: the curvature ductility capacity μ_Φ is equal to 16.6 (using a bilinear approximation for the moment–curvature relation of the pier), yielding displacement ductility capacity $\mu_\Delta = 5.6$. **b** Static "pushover" analysis of the new design concept. Since ductility is now associated with foundation rotation due to mobilization of the bearing capacity failure mechanism, a new definition of μ_Δ is introduced, based on foundation rotation θ; the estimated capacity $\mu_\Delta = 42$ is almost an order of magnitude larger (compared to conventional design)

where f_{ye} and d_{bl} the design yield strength (in MPa) and the diameter of the longitudinal reinforcement in the region of the plastic hinge. This results in a displacement ductility capacity of the conventionally designed system $\mu_\Delta = 5.6$.

Figure 3b depicts the monotonic response of the alternative design according to the new philosophy. Since the behavior of the pier is elastic, the ductility of the system is now associated with foundation rotation due to bearing capacity failure. This renders the conventional definition of curvature ductility not applicable. Thus, an equivalent displacement ductility capacity μ_Δ is defined, based on foundation rotation:

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} = \frac{H\theta_u}{H\theta_y} = \frac{\theta_u}{\theta_y} \tag{15}$$

where θ_u is the "ultimate" foundation rotation, and θ_y the "yield" rotation. The first, θ_u, is defined as the rotation critical for overturning, i.e. the rotation at which $M = 0$: if the foundation-structure system reaches this point, it will overturn. The latter, θ_y (which is practically equivalent to c_n of the conventional system) is defined as the rotation at which the foundation-structure system enters the nonlinear regime. This results in a displacement ductility capacity of the new concept ($B = 7\,\text{m}$) $\mu_\Delta = 42.2$, which is almost an order of magnitude larger than the capacity of the conventionally designed system ($B = 11\,\text{m}$).

4 Dynamic time–history analysis

The seismic performance of the two alternatives is investigated through nonlinear 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. As depicted in Fig. 4, the selected records cover a wide range of seismic motions, ranging from medium intensity (e.g. Kalamata, Pyrgos, Aegion) to relatively stronger (e.g. Lefkada-2003, Imperial Valley), and to very strong accelerograms characterized by forward-rupture directivity effects, or large number of significant cycles, or fling-step effects (e.g. Takatori, JMA, TCU). In terms of spectral accelerations (SA), many of the considered accelerograms exceed (*by far*, in many cases) the design spectrum of the bridge.

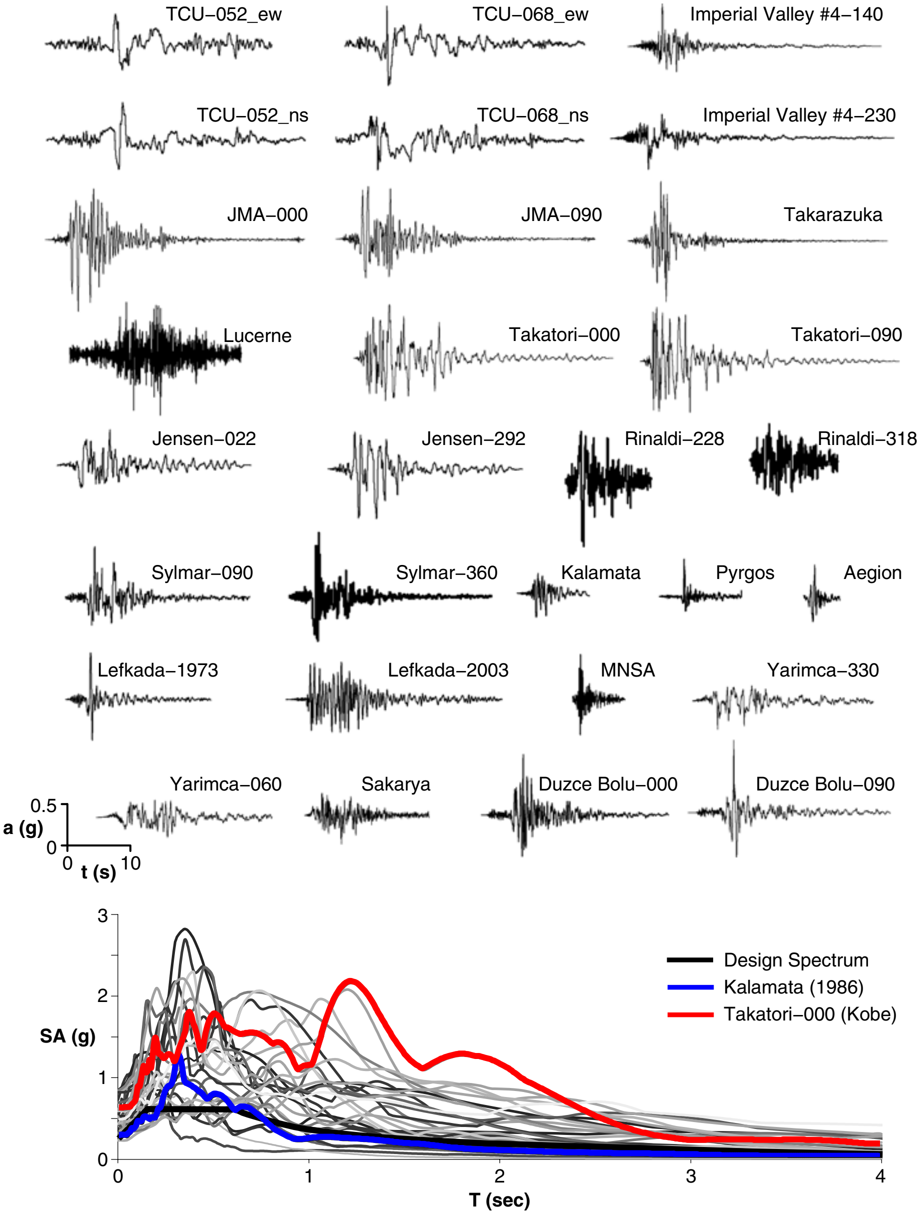

Fig. 4 Real earthquake records used for analysis of the two bridge systems, along with their elastic spectra and the design spectrum of the investigated bridge. The selected ensemble of 29 records covers a wide range of seismic excitations, ranging from medium intensity (e.g. Kalamata, Pyrgos, Aegion) to relatively stronger (e.g. Lefkada-2003, Imperial Valley), and to very strong accelerograms characterized by forward-rupture directivity effects, large number of significant cycles, and/or flingstep effects (e.g. Takatori, JMA, TCU-068)

In the following sections, we compare the response of the two alternatives for: (i) seismic motions that do *not* exceed the design limits (at least not substantially), and (ii) seismic motions that *seriously* exceed the design limits. In the first case, the objective is to determine the serviceability of the bridge after such a moderate intensity earthquake. In the latter case, the main objective is safety (i.e. avoidance of collapse in an almost "improbable" event). Bearing in mind that the spectral acceleration SA of a motion is not always the most crucial parameter of nonlinear response, the characterization of the seismic motions with respect to the exceedence of the design limits is conducted on the basis of spectral displacements SD, following the logic of displacement-based design (e.g. Bertero 1996; Tassios 1998; Priestley 2000; Faccioli et al. 2001).

4.1 Performance in earthquakes *not* exceeding the design limits

A comparison of the performance of the two design alternatives subjected to a moderate intensity earthquake is illustrated in Fig. 5 . The excitation accelerogram is from the 1986 M_s 6.0 Kalamata (Greece) earthquake. At a fault distance of 5 km from the city center, the earthquake caused substantial structural damage to a variety of building structures. With Modified Mercalli Intensity (MMI) levels reaching or exceeding VIII, almost 60% of the buildings had to be retrofitted after the earthquake (Gazetas et al. 1990). It is emphasised that the affected building stock had been designed and constructed according to older seismic codes, practically without any capacity design considerations. Evidently, the same degree of damage should not be expected for modern structures. In terms of SA (Fig. 4), the record exceeds the design spectrum by a factor of almost 2 for periods T ranging from 0.2 to 0.6 s; for the longer periods that are of more relevance for inelastic systems, it is within the design SA.

In Fig. 5a the comparison is portrayed in terms of the foundation experienced moment–rotation (M–θ). As expected, while the response of the conventionally designed foundation is practically elastic (Fig. 5a_1), the *under-designed* foundation (new design philosophy) experiences some inelasticity (Fig. 5a_2). In Fig. 5b the comparison is in terms of foundation settlement–rotation (w–θ). The conventionally designed system is subjected to limited settlement $w \approx 2$ cm (Fig. 5b_1). In marked contrast, the *new concept* (Fig. 5b_2) experiences larger but quite tolerable dynamic settlement: $w \approx 4$ cm.

Figure 5c illustrates the moment–curvature response at the base of the pier for the conventionally designed system. Some inelasticity takes place (i.e. minor structural damage), but the curvature ductility is tolerable: the demand is almost an order of magnitude lower than the capacity of the reinforced concrete section. In the case of the new design philosophy, thanks to foundation yielding the response of the pier (not shown herein) is purely elastic.

The time histories of deck horizontal displacement, i.e. the *drift* Δ, for the two alternatives are compared in Fig. 5d. As graphically illustrated in the adjacent 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 might have been expected, while for the conventional design (*over-designed foundation*) Δ is mainly due to pier distortion Δ_C (Fig. 5d_1), exactly the opposite is observed for the *under-designed* foundation of the new design philosophy: Δ is mainly due to foundation rotation Δ_r (Fig. 5d_2). Nevertheless, despite the differences in the mechanism leading to its development (pier distortion versus foundation rotation), the total drift is quite similar: maximum and residual Δ is slightly larger for the new concept, but quite tolerable.

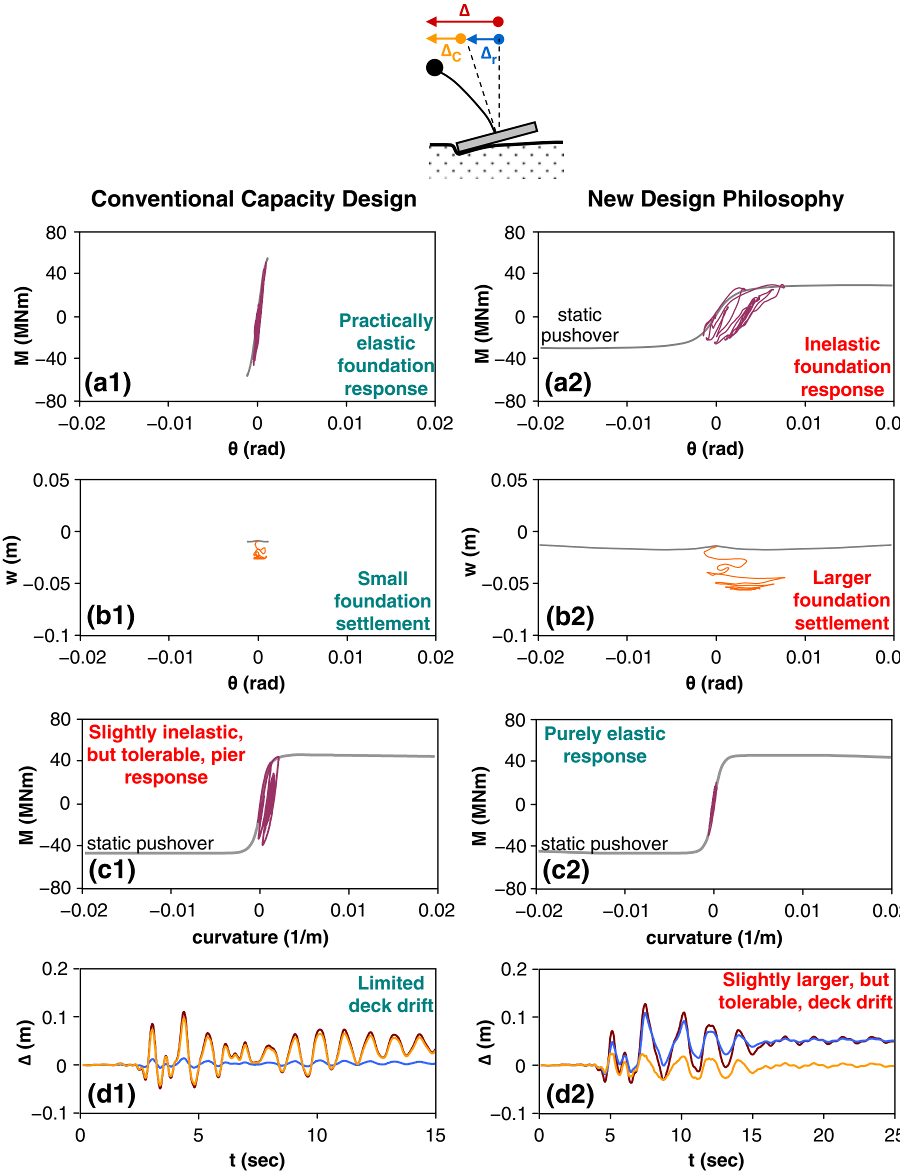

Fig. 5 Comparison of the response of the two alternatives subjected to a medium intensity earthquake (Kalamata 1986), within the design limits. **a1**, **a2** Overturning moment versus rotation (M–θ) for the two foundations. While the conventional design entails practically elastic response of the foundation-soil system, the new design scheme experiences substantial inelastic action. **b1**, **b2** Settlement–rotation (w–θ) response for the two foundations. Thanks to its large foundation and pier yielding, the conventionally designed system experiences limited settlement. In contrast, the smaller foundation (new concept) experiences larger cumulative settlement, which is still quite tolerable. **c1**, **c2** Bending moment–curvature response at the base of the pier. In the conventionally designed system some inelasticity develops, but the ductility demand is totally tolerable. The response of the pier of the new concept is purely elastic. **d1**, **d2** Time histories of deck drift Δ (horizontal displacement). While for the conventional design Δ is mainly due to flexural pier distortion Δ_C, for the new design concept the drift is mainly due to foundation rotation Δ_r. The residual drift is slightly larger in the new design scheme, but quite tolerable.

4.2 Performance in earthquakes *exceeding* the design limits

We now extend the comparison for a large intensity motion, exceeding the design limits (Fig. 6): the Takatori accelerogram of the 1995 M_{JMA} 7.2 Kobe earthquake. With a direct economic loss of more than $100 billion (EERI 1995), the Kobe earthquake needs no introduction. Constituting the greatest earthquake disaster in Japan since the 1823 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 (e.g. Seible et al. 1995). As aforementioned, the 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 (Iwasaki et al. 1995; Park 1996). It is therefore logical to consider this as a reasonably realistic example of an "*above the limits*" earthquake. In particular, the Takatori record (Fukushima et al. 2000) constitutes one of the worst seismic motions ever recorded: PGA = 0.70 g, PGV = 169 cm/s, bearing the "mark" of forward rupture directivity. Compare its response spectrum to the design SA (Fig. 4) to notice how much larger it is throughout the whole range of periods.

Figure 6a compares the response of the two alternatives, in terms of deformed mesh with superimposed plastic strain. In the conventionally designed system (Fig. 6a_1) 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). In stark contrast, with the new design scheme (Fig. 6a_2) the "plastic hinge" takes the form of mobilisation 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 mobilisation of the bearing capacity failure mechanism.

As seen in Fig. 6b, the pier of the conventional system suffers a curvature ductility exceeding the design limit by almost one order of magnitude—clearly a case of collapse. This is further confirmed by the time history of deck drift Δ (Fig. 6c_1). In marked contrast, the system designed according to the new philosophy easily survives (Fig. 6c_2). It experiences substantial maximum deck drift (about 40 cm), almost exclusively due to foundation rotation Δ_r. Nevertheless, the residual foundation rotation leads to a tolerable 7 cm deck horizontal displacement at the end of the earthquake.

The moment–rotation (M–θ) response of the two foundations is depicted in Fig. 6d. Respecting its design principles, the conventional $B = 11$ m foundation-soil system remains practically elastic (Fig. 6d_1); the causes are now evident: (i) the rocking stiffness of the foundation, being proportional to B^3, is large and leads to small stresses in the soil; and (ii) pier failure effectively limits the loading transmitted onto the foundation. Exactly the opposite is observed for the *under-designed* ($B = 7$ m) foundation, the response of which is strongly inelastic (Fig. 6d_2): mobilisation of bearing capacity failure acts as a *"safety valve"* or a *"fuse"* for the superstructure.

But despite such excessive soil plastification, not only the structure does not collapse, but the residual (permanent) rotation is rather limited (as already attested by the residual deck drift). Under static conditions, the development of this rotational mechanism on either side of the foundation would have lead to toppling of the structure. However, dynamically, each "side" of the rotational mechanism deforms plastically for a very short period of time ("momentarily"), producing limited inelastic rotation which is partially cancelled by the ensuing deformation on the opposite side. Obviously, exactly the same applies to structural

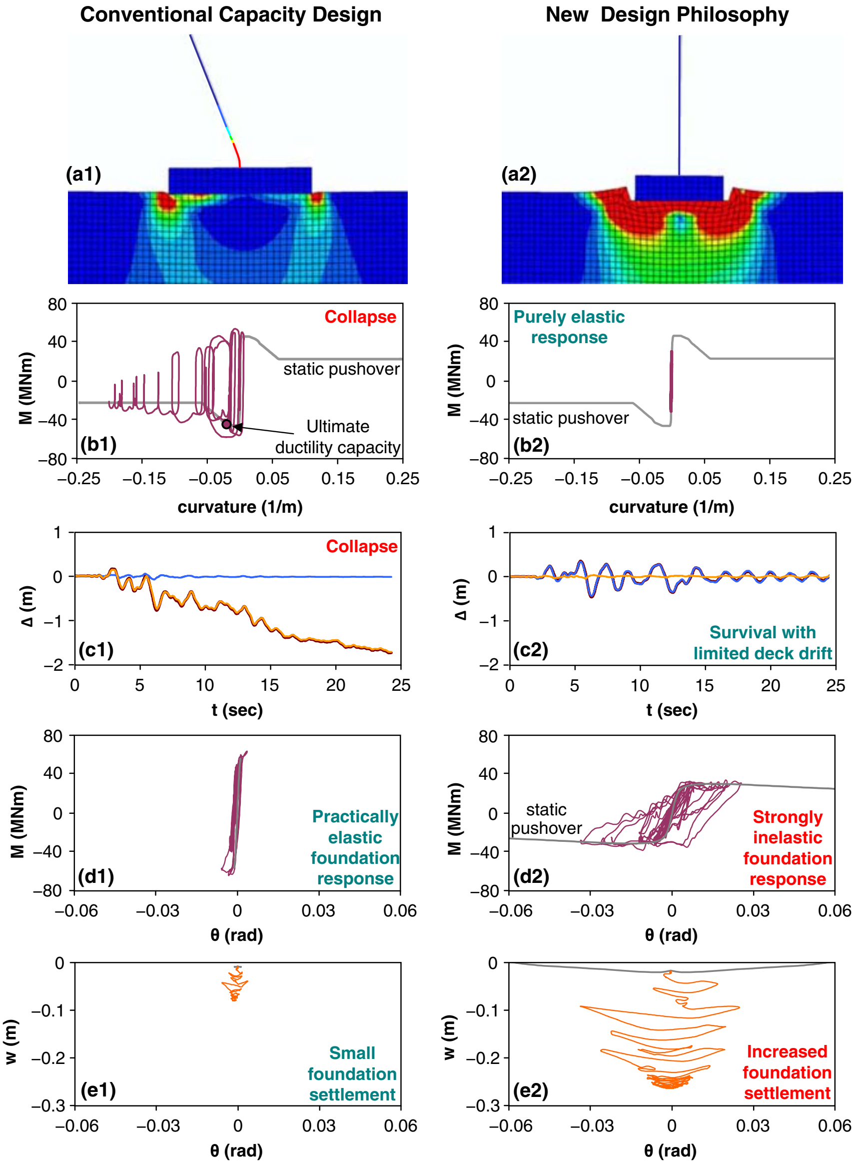

Fig. 6 Comparison of the response of the two alternatives subjected to a large intensity earthquake (Takatori, 1995), exceeding the design limits. **a1**, **a2** Deformed mesh with superimposed plastic strain, showing the location of "plastic hinging": in the base of the pier in the first case; in the foundation soil in the second. **b1**, **b2** Bending moment–curvature response at the pier base. Experiencing ductility demand far exceeding the design, the conventionally designed pier would collapse. With the new design concept, the pier remains elastic. **c1**, **c2** Time histories of deck drift Δ. With its response dominated by pier flexural failure, the conventionally designed system collapses. The maximum drift of the new design concept is large (mainly due to foundation rotation), but the system survives with insignificant residual drift. **d1**, **d2** Overturning moment–rotation (M–θ) response of the two foundations. While the response of the conventionally designed foundation remains practically elastic, the response of the new concept is strongly inelastic. **e1**, **e2** Foundation settlement–rotation (w–θ) response. Again, while the settlement of the conventional system is minor, the new design experiences a large (24 cm) settlement: a price to pay to avoid collapse

plastic “hinging” in conventional design. The main difference between the two alternatives lies in the mechanism of energy dissipation, and the related displacement ductility margins.

However, energy dissipation is not attainable at zero cost: in our case the cost is the increase of foundation settlement. Figure 6e compares the settlement–rotation ($w-\theta$) response for the two alternatives. While the practically elastic response of the conventional (*over-designed*) foundation leads to a minor 7 cm settlement (Fig. 6e_1), the *under-designed* foundation of the new philosophy experiences an increased accumulated 24 cm settlement (Fig. 6e_2). Although such settlement is certainly not negligible, it can be considered as a small price to pay to avoid collapse under such a tremendous ground shaking.

Perhaps fortuitously, the residual rotation in this particular case turned out to be insignificant.

5 Summary and conclusions

The overall performance (for all 29 seismic excitations) of the two design alternatives is compiled and synopsized in Fig. 7. We present key performance indicators with respect to peak ground acceleration a_E of the seismic excitation (at bedrock).

Figure 7a compares the ratio of displacement ductility demand over ductility capacity $\mu_{demand}/\mu_{capacity}$, for the two alternatives. For the conventional design (Fig. 7a_1), we also indicate the likely damage level according to Response Limit States of Priestley et al. (1996). In accordance with conventional design principles, the damage to the conventional system is within the *serviceability limits* only in moderate—*not exceeding the design limits*—earthquake motions (e.g. Kalamata, Aegion, MNSA). In stronger motions (e.g. Yarimca, TCU-068, Rinaldi-318), it falls within *damage control* or (barely) *survival.* Finally, for even stronger—*clearly exceeding the design limits* —earthquake shaking (e.g. Takatori-000, TCU-068, Jensen-022) *failure* is unavoidable. In fact, in some cases the ductility demand is an order of magnitude larger than capacity. In refreshing contrast, the “*unconservative*” system designed according to the new philosophy never comes close to its displacement ductility capacity (Fig. 7a_2): $\mu_{demand}/\mu_{capacity}$ is systematically lower than 0.25 for all seismic motions. Evidently, the new design concept appears to provide much larger safety margins.

The performance of the new design concept is also slightly superior in terms of residual deck drift Δ (Fig. 7b), especially for large intensity earthquakes. The conventional design is superior in terms of residual Δ only for small earthquakes, in which both superstructure and foundation remain completely elastic. Figure 7c compares the settlement w of the two alternatives after the end of the earthquake. Evidently, the new design scheme is subject to larger settlement for all seismic motions: w is roughly 3 times larger than for the conventionally designed system. However, even in the worst-case scenarios, w barely exceeds 0.2 m.

In conclusion :

(1) *For moderate intensity earthquakes not exceeding the design limits*, the performance of both alternatives is totally acceptable: both of them would be utilisable right after the earthquake, with only minor repair required. Sustaining limited structural damage (in the form of minor flexural cracking), the conventionally designed system would be easily repairable. On the other hand, the system designed according to the new philosophy would not sustain any structural damage, but would be subjected to slightly increased—but absolutely tolerable—deck drift and settlement. It should, however, be noticed that for more slender structures (i.e. bridges with taller piers), the increase of

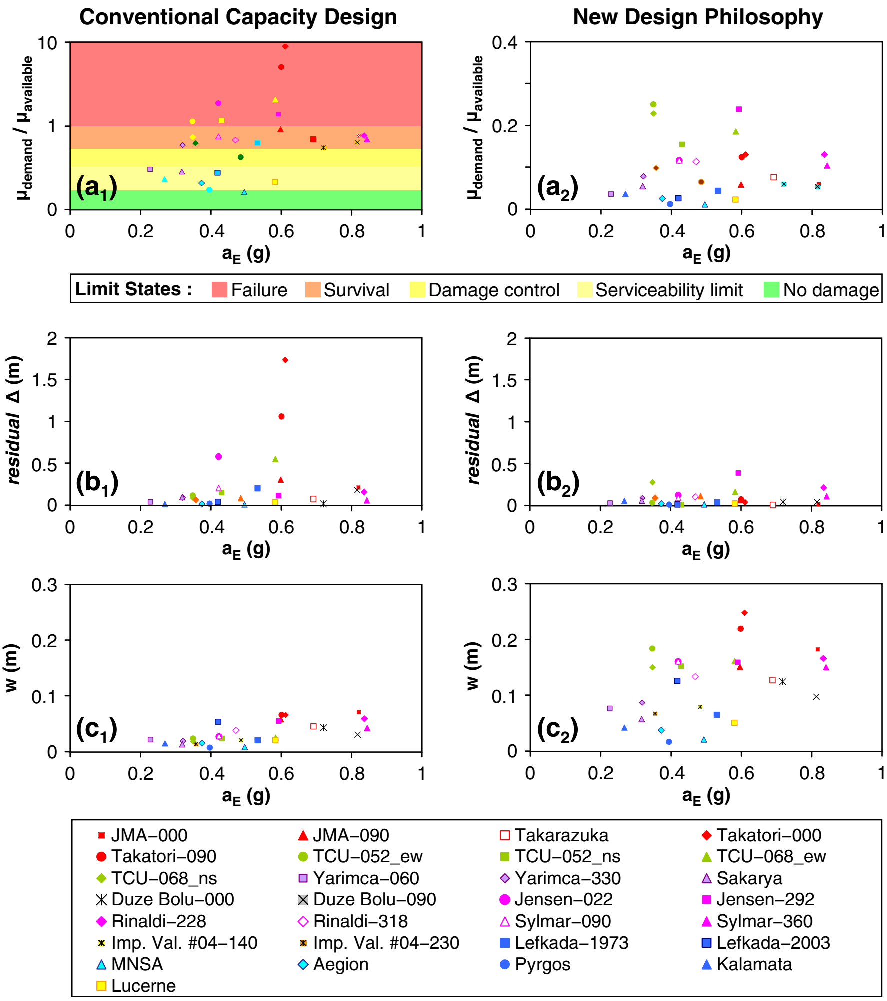

Fig. 7 Synopsis of the response of the two alternatives with respect to peak ground acceleration a_E. **a1**, **a2** Ratio of displacement ductility *demand* over ductility *capacity*. For the conventional design, we also indicate the damage level with reference to Response Limit States (Priestley et al. 1996): while for earthquakes not exceeding the design limits the bridge would survive with some damage (ranging from the "serviceability" to the "survival" limit state), it would probably collapse for several earthquakes that exceed the design. In some cases, the ductility demand is an order of magnitude larger than ductility capacity. **b1**, **b2** Residual deck drift Δ. For earthquakes not exceeding the design, the residual Δ of the two systems is comparable. The new concept is clearly advantageous for earthquakes that exceed the design limits. **c1**, **c2** Settlement w after the end of the earthquake. The new concept does suffer from larger settlement. However, only in the *very*-worst-case scenarios, does w barely exceed 0.2 m. Whether—and under which conditions—such a w can be tolerable will depend on the serviceability limits of the superstructure. In any case, the new design concept may provide larger safety limits, trading-off structural damage (or collapse) with increased settlement

rotation may become unacceptable. In such cases, the detrimental role of second order effects should be carefully evaluated.

(2) *For large intensity earthquakes that clearly exceed the design limits*, the performance of the system designed according to the new philosophy is quite advantageous: while the conventional system may collapse (as was the case with the Fukae bridge in Kobe), or

at least sustain severe (non-repairable) structural damage, the new design would survive with the damage being in the form of increased settlements. Whether the bridge would be repairable after such an earthquake depends on how *settlement tolerant* the design of its superstructure is. In any case, preservation of human life through avoidance of collapse is the main design objective against this type of extreme loading, and although it might be early to over-generalize (a variety of soil types and superstructure typologies should have to be thoroughly examined and evaluated), the new design philosophy seems to have a potential for significantly larger safety margins.

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References

Apostolou M, Gazetas G (2005) Rocking of foundations under strong shaking : mobilisation of bearing capacity and displacement demands. In: Proc. 1st Greece–Japan workshop, seismic design, observation, Retrofit, Athens, 11–12 October, pp 131–140

Bertero V (1996) State of the art report on: design criteria. In: Proceedings of 11th world conference on earthquake engineering, Acapulco, Mexico, Oxford, Pergamon

Bommer JJ, Abrahamson NA, Strasser FO, Pecker A, Bard P-Y, Bungum H, Cotton F, Faeh D, Sabetta F, Scherbaum F, Studer J (2004) The challenge of defining the upper limits on earthquake ground motions. Seismol Res Lett 70(1):82–95

Chatzigogos CT, Pecker A, Salencon J (2009) Macroelement modeling of shallow foundations. Soil Dyn Earthq Eng 29(6):765–781

Earthquake Engineering Research Institute (1995) The Hyogo-Ken Nanbu earthquake, January 17, 1995. Preliminary EERI Reconnaissance Report

EAK (2000) Greek seismic code. Organization of Seismic Planning and Protection, Athens (in Greek)

EC8 (2000) Design provisions for earthquake resistance of structures, part 5: foundations, retaining structures and geotechnical aspects, prEN, 1998–5 European Committee for Standardization, Brussels

ΕΚΩΣ (2000) Greek code for reinforced concrete. Organization of Seismic Planning and Protection, Athens (in Greek)

Esmaeily-Gh A, Xiao Y (2002) Seismic behavior of bridge columns subjected to various loading patterns. In: PEER Report 2002/15, Pacific Earthquake Engineering Research Center, College of Engineering, University of California, Berkeley

Faccioli E, Paolucci R, Vivero G (2001) Investigation of seismic soil-footing interaction by large scale cyclic tests and analytical models. In: Proc., 4th int. conf. recent advances in geotechnical earthquake engineering and soil dynamics

Fardis N, Georgarakos P, Gazetas G, Anastasopoulos I (2003) Sliding isolation of structures: effect of horizontal and vertical acceleration. In: Proceedings of fib international symposium on concrete structures in seismic regions, Athens, May 2003

FEMA 356 (2000) Prestandard and commentary for the seismic rehabilitation of buildings. Federal Emergency Management Agency, Washington DC

Frangopol DM, Curley JP (1987) Effects of damage and redundancy on structural reliability. J Struct Eng ASCE 113(7):1533–1549

Fukushima Y, Irikura K, Uetake T, Matsumoto H (2000) Characteristics of observed peak amplitude for strong ground motion from the 1995 Hyogoken Nanbu (Kobe) earthquake. Bull Seismol Soc Am 90:545–565

Gajan S, Phalen JD, Kutter BL, Hutchinson TC, Martin G (2005) Centrifuge modeling of load-deformation behavior of rocking shallow foundations. Soil Dyn Earthq Eng 25(7–10):773–783

Gajan S, Hutchinson TC, Kutter BL, Raychowdhury P, Ugalde JA, Stewart JP (2008) Numerical models for the analysis and performance-based design of shallow foundations subjected to seismic loading. In: Rep. to Pacific Earthquake Engineering Research Center (PEER), Univ. of California, Berkeley, Calif.

Gajan S, Kutter BL (2008) Capacity, settlement, and energy dissipation of shallow footings subjected to rocking. J Geotech Geoenviron Eng ASCE 134(8):1129–1141

Gazetas G, Dakoulas P, Papageorgiou AS (1990) Local soil and source mechanism effects in the 1986 Kalamata (Greece) earthquake. Earthq Eng Struct Dyn 19:431–456

Gazetas G, Apostolou M, Anastasopoulos I (2003) Seismic uplifting of foundations on soft soil, with examples from Adapazari (Izmit 1999, Earthquake). In: BGA int. conf. on found. innov., observations, design & practice, Univ. of Dundee, Scotland, September 25, pp 37–50
Gazetas G, Anastasopoulos I, Gerolymos N, Mylonakis G, Syngros C (2005) The collapse of the Hanshin expressway (Fukae) bridge, Kobe 1995: soil-foundation-structure interaction, reconstruction, seismic isolation, Entwicklungen in der Bodenmechanik, Bodendynamik und Geotechnik, Festschrift zum 60. Geburstag von Univ.-Professor Dr.-Ing.habil. Stavros A. Savidis (Honorary Volume for the 60th Birthday of Professor Savidis), Frabk Rackwitz, Springer, pp 93–120
Gerolymos N, Gazetas G, Tazoh T (2005) Seismic response of yielding pile in non-linear soil. In: Proc. 1st Greece–Japan workshop, seismic design, observation, and retrofit of foundations, Athens, 11–12 October, pp 25–36
Gerolymos N, Giannakou A, Anastasopoulos I, Gazetas G (2008) Evidence of beneficial role of inclined piles: observations and numerical results. Bull Earthq Eng 6(4):705–722 Special Issue: Integrated approach to fault rupture- and soil-foundation interaction
Gerolymos N, Drosos V, Gazetas G (2009) Seismic response of single-column bent on pile: evidence of beneficial role of pile and soil inelasticity. Bull Earthq Eng 7(2):547–573 Special Issue: Earthquake Protection of Bridges
Harden C, Hutchinson T (2006) Investigation into the effects of foundation uplift on simplified seismic design procedures. Earthq Spectra 22(3):663–692
Ishibashi I, Zhang X (1993) Unified dynamic shear moduli and damping ratios of sand and clay. Soil Found 33(1):12–191
Iwasaki T, chm, et al (1995) In: Report on highway bridge damage caused by the Hyogo-ken Nanbu earthquake of 1995, Committee on Highway Bridge Damage, Japan
Kawashima K, Nagai T, Sakellaraki D (2007) Rocking seismic isolation of bridges supported by spread foundations. In: Proc. 2nd Japan–Greece workshop on seismic design, observation, and retrofit of foundations, Tokyo, Japan, April 3–4, pp 254–265
Kutter BL, Martin G, Hutchinson TC, Harden C, Gajan S, Phalen JD (2003) Status report on study of modeling of nonlinear cyclic load-deformation behavior of shallow foundations. In: PEER workshop, University of California, Davis, March 2003
Makris N, Roussos Y (2000) Rocking response of rigid blocks under near source ground motions. Géotechnique 50(3):243–262
Mander JB, Priestley MJN, Park R (1988) Theoretical stress strain model for confined concrete. ASCE J Struct J 114(8):1804–1825
Martin GR, Lam IP (2000) Earthquake resistant design of foundations: retrofit of existing foundations. In: Proc. geoeng 2000 conference, Melbourne
Mergos PE, Kawashima K (2005) Rocking isolation of a typical bridge pier on spread foundation. J Earthq Eng 9(2):395–414
Paolucci R (1997) Simplified evaluation of earthquake induced permanent displacement of shallow foundations. J Earthq Eng 1(3):563–579
Paolucci R, Shirato M, Yilmaz MT (2007) Seismic behavior of shallow foundations: shaking table experiments vs numerical modeling. Earthq Eng Struct Dyn 37(4):577–595
Park R, Paulay T (1975) Reinforced concrete structures. Wiley, New York
Park R (1996) An analysis of the failure of the 630 m elevated expressway in Great Hanshin earthquake. Bull N.Z. Natl Soc Earthq Eng 29:2
Pecker A (1998) Capacity design principles for shallow foundations in seismic areas. In: Proc. 11th european conference on earthquake engineering, A.A. Balkema Publishing
Pecker A (2003) Aseismic foundation design process, lessons learned from two major projects: the Vasco de Gama and the Rion Antirion bridges. In: ACI international conference on seismic bridge design and retrofit, University of California at San Diego, La Jolla, USA
Priestley MJN (2000) Performance based seismic design. In: Proc. 12th world conference on earthquake engineering (12WCEE), Auckland, New Zealand, Paper No. 2831
Priestley MJN, Seible F, Calvi GM (1996) Seismic design and retrofit of bridges. Wiley, New York
Seible F, Priestley MJN, MacRae G (1995) The Kobe earthquake of January 17, 1995; initial impressions from a quick reconnaissance. In: Structural systems research report-95/03, University of California, San Diego, 1995
Shakal A, Haddadi H, Graizer V, Lin K, Huang M (2006) Some key features of the strong motion data from the M 6.0 Parkfield, California, earthquake of 28 September 2004. Bull Seismol Soc Am 96(4B):S90–S118
Tassios TP (1998) Seismic design: state of practice. In: Proceedings of 11th european conference on earthquake engineering, Rotterdam, AA Balkema, pp 255–267