Towards a Reversal of Seismic Capacity Design. Part A : Analysis of Bridge Pier–Foundation System

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ABSTRACT: The paper illuminates a new seismic design philosophy, which takes advantage of soil "failure" to protect the superstructure. A reversal of conventional *"capacity design"* is introduced, through intentional under-designing of the foundation. A simple but realistic bridge is used as an illustrative example of the effectiveness of the new philosophy. Two alternatives are compared : one in compliance with conventional capacity design, with over-designed foundation so that the plastic "hinge" develops in the superstructure ; and one with under-designed foundation, so that the plastic "hinge" may occur in the soil. The seismic performance of the two alternatives is investigated through nonlinear dynamic time history analysis, using a variety of seismic excitations. It is shown that the performance of both alternatives is totally acceptable for moderate seismic motions. For large intensity seismic motions, the performance of the new scheme is proven advantageous, not only avoiding collapse but hardly suffering any inelastic *structural* deformation. The penalty to pay is a substantial foundation settlement.

1 INTRODUCTION

Seismic design of structures recognises that highly inelastic material response is un-avoidable under the strongest possible earthquake. "Ductility" levels of the order of 3 or more are usually allowed to develop under strong seismic shaking, implying that the strength of a number of critical bearing elements is fully mobilized. In the prevailing structural terminology, "plastic hinging" is allowed as long as the overall stability is maintained.

In marked contrast, a crucial goal of current practice in seismic "foundation" design, particularly as entrenched in the respective codes [e.g. EC8], is to avoid the mobilisation of "strength" in the foundation. In structural terminology : no "plastic hinging" is al-lowed in the foundation soil. In simple geotechnical terms, the designer has to ensure that the below-ground (difficult to inspect) support system will not even reach a number of "thresholds" that would statically imply failure. Thus, mobilisation of the "bearing-capacity" failure mechanism, foundation sliding and uplifting, or any relevant combina-tion is prohibited. To make sure that such mechanisms will not develop, "overstrength" factors plus (explicit and implicit) factors of safety larger than 1 are introduced against each of those "failure" modes. This way, the engineer is certain that foundation performance will be satisfactory and that there will be no need to inspect and/or repair after strong earthquake shaking.

However, a growing body of evidence suggests that soil–foundation plastic yielding under seismic excitation may be advantageous, and should be seriously considered in analysis and perhaps allowed in design [Pecker, 1998; Martin & Lam, 2000; FEMA-356, 2000; Kutter et al., 2003, Apostolou & Gazetas, 2005; Kawashima et al., 2007]. The urgent need to explicitly consider the possibility of the foundation system to go beyond "failure" thresholds, and the potential usefulness of doing so, emerge from :

(a) The large (often huge) effective ground acceleration, A, and velocity, V, levels recorded in several earthquakes in the last 20 years. Few examples : 1994 M_s 6.8 Northridge : A = 0.98 g, V = 140 cm/s ; 1995 M_{JMA} 7.2 Kobe : A = 0.85 g, V = 120 cm/s ; 1986 M_s 5.6 San Salvador : A = 0.75 g, V = 84 cm/s ; 2007 M_{JMA} 6.9 Niigata : A =1.20 g, V = 100cm/s. With the correspondingly large accelerations in the (above–ground) structure (spectral SA values well in excess of 1 g) from such ground motions, preventing "plastic hinging" in the foundation system is a formidable task. And in fact, it may not even be desirable: enormous ductility demands might be imposed to the structure if soil–foundation "yielding" would not take place to effectively limit the transmitted accelerations. Several present-day critically–important structures on relatively loose soil could not have been de-signed against severe ground motions if "plastic hinging" of some sort could not be allowed to take place in the "foundation". [Example : the 90 m in diameter, 80 m tall foot-ings of the Rion-Antirrion bridge piers in Greece: allowing sliding under certain seismic conditions was a beneficial necessity ; Pecker, 2003].

(b) In seismically retrofitting a structure, allowing for foundation–soil yielding is the only rational alternative. Because increasing the structural capacity of some elements, or introducing some stiff elements, would then imply that the forces transmitted onto the foundation will be increased, to the point that it might not be technically or economically feasible to undertake them "elastically". Thus, the new American retrofit design guidelines (FEMA 356) explicitly permit some forms of inelastic deformation in the foundation. A simple hypothetical example referring to an existing three–bay multi–story building frame which is to be retrofitted with a single–bay concrete "shear" wall was presented by Martin & Lam [2000]. Such a wall, being much stiffer than the col-umns of the frame, will carry most of the inertia-driven shear force and will thus trans-mit a disproportionately large horizontal force and overturning moment onto the foundation compared with its respective vertical force. If uplifting, sliding, and mobilisation of bearing capacity failure mechanisms in the foundation were spuriously ignored, or conversely correctly taken into account, would lead to dramatically different results. With "beyond–threshold" action in the foundation the shear wall "sheds" off some of the load onto the columns of the frame, which must then be properly reinforced ; the opposite is true when such action (beyond the thresholds) is disallowed. The Engineer therefore should be able to compute the consequences of "plastic hinging" in the foundation before deciding whether such "hinging" must be accepted, modified or avoided (through foundation changes).

(c) Compatibility with structural design is another reason for the soil–structure interaction analyst to compute the collapse lateral load of the foundation system, as well as the complete load–displacement or load–rotation response to progressively increasing load up to collapse. Indeed, in SOA structural engineering use is made of "pushover" analysis, which in order to be complete requires the development of such information from the foundation analyst. In addition to the above "theoretical" arguments, there is a growing need for estimating the "collapse motion": insurance coverage of major construction facilities is often based on estimated losses under the worst possible (as opposed to probable) earthquake scenario.

(d) The current trend in structural earthquake engineering calls for a philosophical change : from strength-based design (involving force considerations) to performance-based design (involving displacement considerations) [Pauley, 2002; Priestley, 2000]. Geotechnical earthquake engineering has also been slowly moving towards performance–based design : gravity retaining structures are indeed allowed to slide.

The time is therefore ripe for soil–foundation–structure interaction (SFSI) to also move from imposing "safe" limits on forces and moments acting on the foundation (aiming at avoiding pseudo-static "failure") to performance–based design in which all possible conventional "failure" mechanisms are allowed to develop, to the extent that maximum and permanent displacements and rotations are kept within acceptable limits. This paper introduces a new design philosophy, beyond conventional capacity design, in which superstructure plastic "hinging" is replaced by soil–foundation failure (see sketch of Figure 1) : i.e. soil failure is used as a "shield" for the superstructure (exactly the opposite of conventional capacity design). A simple but realistic typical highway bridge is used as an illustrative example of the effectiveness of the new seismic design philosophy.

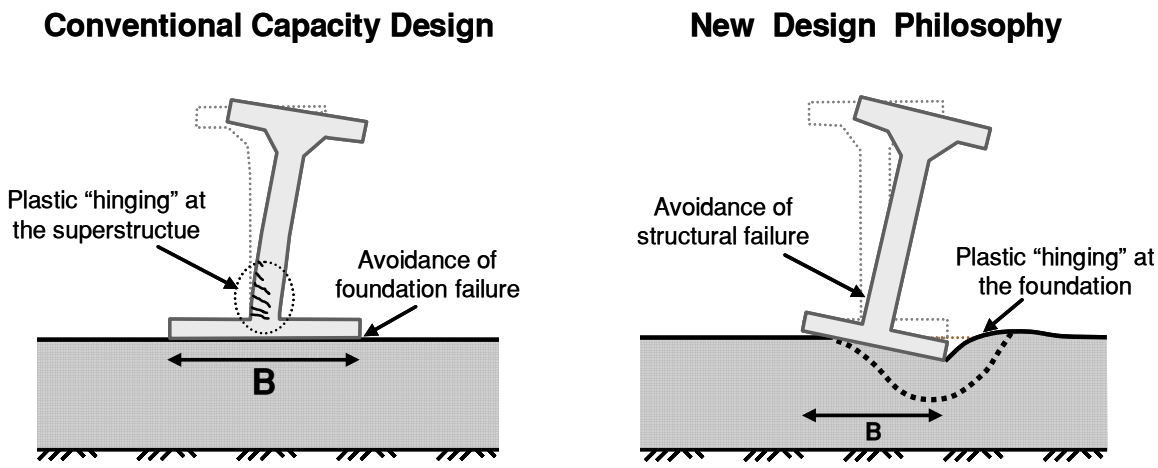

Figure 1. Conventional capacity design compared to the new design philosophy.

2 PROBLEM DEFINITION AND ANALYSIS METHODOLOGY

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 analysis is 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 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 sec and design spectral acceleration SA = 0.3 g, to undertake the resulting design bending moment $M_D \approx 43$ MNm, a longitudinal reinforcement of 100 d_{bL} = 32 mm bars (100Φ32) is required, combined with d_{bw} =13 mm hoops spaced at 8 cm.

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 ap-proaches. A larger foundation, B = 11 m, is designed in compliance with conventional capacity design, applying an overstrength factor γ_{Rd} = 1.4 to ensure that the plastic "hinge" will develop in the superstructure (base of pier). Taking account of maximum allowable uplift (eccentricity $e = M / V \leq B/3$, where V is the vertical load), the resulting safety factors for static and seismic loading are FSV = 5.6 and FSE = 2.0, respec-tively. A smaller, under-designed, B = 7 m foundation is considered in the spirit of the new design philosophy. Its static safety factor FSV = 2.8, but is designed 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 ($FSE \approx 0.7$). In fact, as it will be shown below, the under-designed 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.

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 ─ representing with realism the tensionless and frictionless interface. The mass of the footing and of the pier are also taken into account.

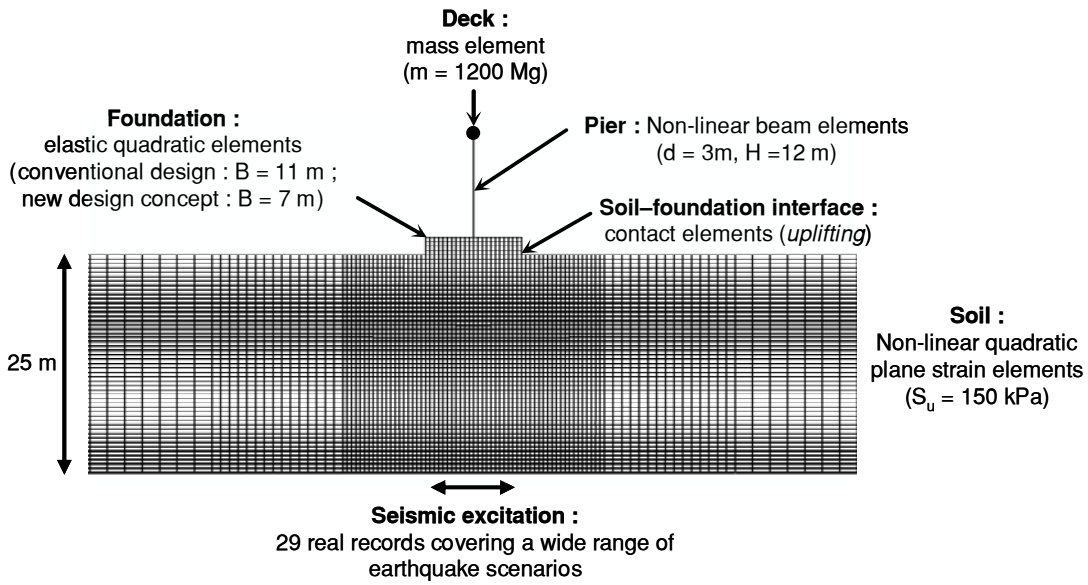

Figure 2. Methodology of finite element modelling.

2.1 *Constitutive Modeling of Soil*

A nonlinear constitutive model with Von Mises failure criterion, nonlinear kinematic hardening and associative plastic flow rule is employed. According to the Von Mises failure criterion, the evolution of stresses is described by the relation :

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

where : σ_o is the value of stress at zero plastic strain, assumed to remain constant. The parameter α is the backstress that defines the kinematic evolution of the yield surface in the stress space. The evolution of the kinematic component of the yield stress is de-scribed by the following law:

$$\dot{\alpha} = C\frac{1}{\sigma_0}(\sigma - \alpha)\dot{\bar{\varepsilon}}^{\mathrm{pl}} - \gamma\alpha\dot{\bar{\varepsilon}}^{\mathrm{pl}} \tag{2}$$

where : $\dot{\bar{\varepsilon}}^{\mathrm{pl}}$ is the plastic strain rate, C the initial kinematic hardening modulus ($C = \sigma_{\mathrm{y}}/\varepsilon_{\mathrm{y}} = E$) and γ a parameter that determines the rate at which the kinematic hardening decreases with increasing plastic deformation.

The maximum yield stress (at saturation) is :

$$\sigma_{\mathrm{y}} = \frac{C}{\gamma} + \sigma_0 \tag{3}$$

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

$$\sigma_{\mathrm{y}} = \sqrt{3}S_u \tag{4}$$

From equations (3) and (4) we have :

$$\gamma = \frac{\mathrm{C}}{\sqrt{3}S_u - \sigma_0} \tag{5}$$

Model parameters are calibrated to fit published G–γ curves of the literature, following the procedure described in Gerolymos et al. [2005]. Figure 3a illustrates the validation of the kine-

matic hardening model (through simple shear finite element analysis) against published G–γ curves by Ishibashi and Zhang [1993].

2.2 *Constitutive Modeling of Reinforced Concrete*

The same model is calibrated to match the response of the reinforced concrete pier 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., 1984] 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 :

$$M = 2\int_0^{\pi}\int_0^{d/2} \sigma r^2 \sin\theta \, dr d\theta \tag{6}$$

For the maximum yield stress σ_y this relationship gives :

$$M_y = 2\sigma_y \int_0^{\pi} \frac{r^3}{3} \sin\theta \Big|_0^{d/2} d\theta \tag{7}$$

which yields:

$$M_y = \frac{1}{6}\sigma_y d^3 \tag{8}$$

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

$$\sigma_y = \frac{6My}{d^3} \tag{9}$$

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 3b depicts the results of model calibration for the pier against moment–curvature relation of the reinforced concrete section calculated through section analysis utilizing the USC_RC software [Esmaeily-Gh & Xiao, 2002], which uses the Mander model [Mander et al., 1984] for confined concrete. As for soil, model parameters are calibrated using the aforementioned methodology of Gerolymos et al. [2005].

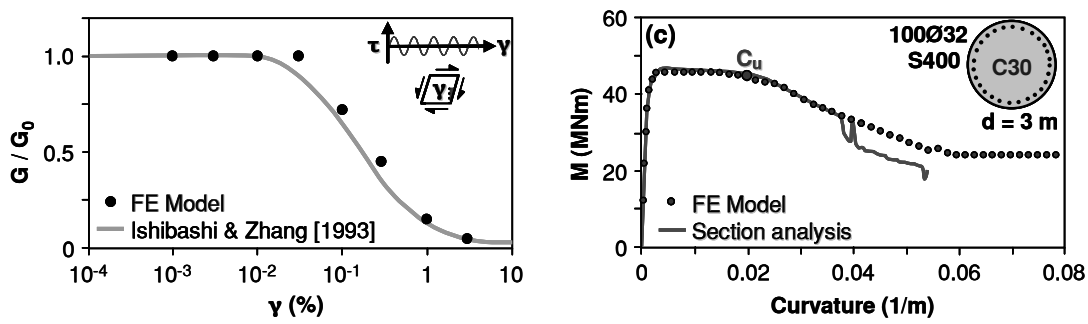

Figure 3. (a) Model calibration for soil (stiff clay, S_u = 150 kPa) against published G-γ curves by Ishibashi & Zhang [1993] ; (b) Model calibration for the reinforced concrete pier against moment-curvature response calculated using reinforced concrete section analysis (USC-RC).

3 PUSHOVER RESPONSE OF THE TWO ALTERNATIVES

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 4a 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] :

$$\sigma_{\mathrm{y}} = \frac{6My}{d^3} \tag{10}$$

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 :

$$L_p = 0.08L + 0.022 f_{ye} d_{bl} \geq 0.044 f_{ye} d_{bl} \tag{11}$$

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 4b 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 dis-placement ductility capacity μΔ is defined, based on foundation rotation :

$$\mu_\Delta = \frac{\Delta_{\mathrm{u}}}{\Delta_{\mathrm{y}}} = \frac{H\theta_{\mathrm{u}}}{H\theta_{\mathrm{y}}} = \frac{\theta_{\mathrm{u}}}{\theta_{\mathrm{y}}} \tag{12}$$

where : θ_u is the ultimate (critical for overturning) foundation rotation, and θ_y the "yield" rotation. This results in a displacement ductility capacity of the new concept (B = 7 m) $\mu_\Delta = 42.2$, which is almost an order of magnitude larger than the capacity of the conven-tionally designed system (B = 11 m).

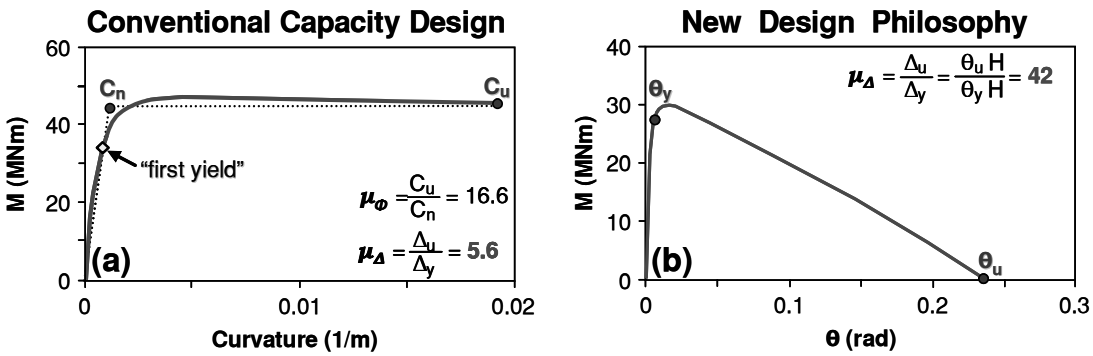

Figure 4. (a) "Pushover" analysis of the conventionally designed system : the curvature ductility capacity μ_Φ is equal to 16.6 (using a bilinear approximation for the moment-curvature relation of the pier), yielding displacement ductility capacity $\mu_\Delta = 5.6$; (b) "Pushover" analysis of the new 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).

4 TIME-HISTORY DYNAMIC ANALYSIS OF THE TWO ALTERNATIVES

We now investigate the seismic performance of the two alternatives through nonlinear dynamic time history analysis. An ensemble of 29 real accelerograms is used as seismic excitation. The latter is applied at bedrock level. As illustrated in Figure 5, 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.

In the following sections, we compare the response of the two alternatives for : (i) moderate intensity seismic motions not exceeding the design limits (at least not substantially), and (ii) large intensity seismic motions that substantially 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 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].

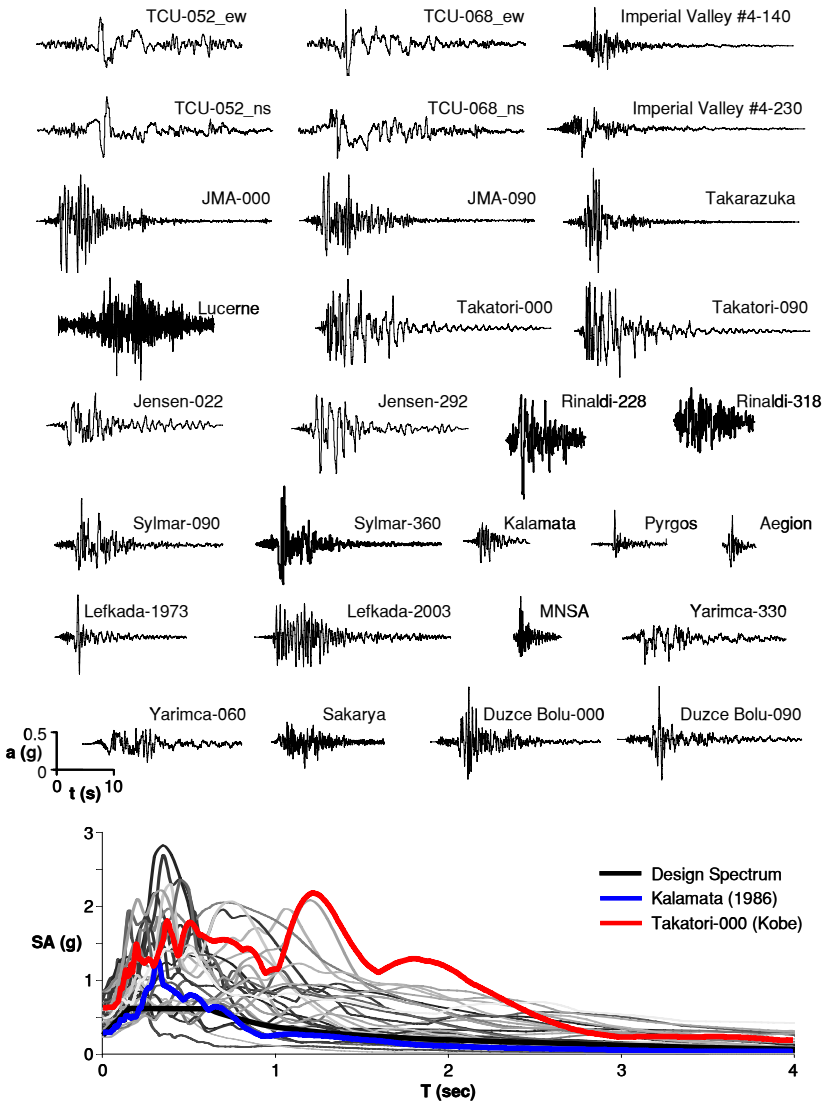

Figure 5. Real earthquake records used for analysis of the two alternatives, along with their elastic spectra and the design spectrum of the investigated bridge.

4.1 *Performance in Moderate Intensity Seismic Motions*

A comparison of the performance of the two design alternatives subjected to a moderate intensity earthquake is illustrated in Figures 6 and 7. 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 emphasized 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 (Figure 5), the record exceeds the design spectrum by a factor of almost 2 for periods T ranging from 0.2 to 0.6 sec ; for the longer periods that are of more relevance for inelastic systems, it is within the design SA.

In Figure 6a 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 (Figure 6a1), the under-designed foundation (new design philosophy) experiences some inelasticity (Figure 6a2). In Figure 6b the comparison is in terms of foundation settlement–rotation (w–θ). The conventionally designed system is subjected to limited settlement $w \approx 2$ cm (Figure 6b1). In marked contrast, the new concept (Figure 6b2) experiences larger but quite tolerable dynamic settlement : $w \approx 4$ cm.

Figure 7a 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.

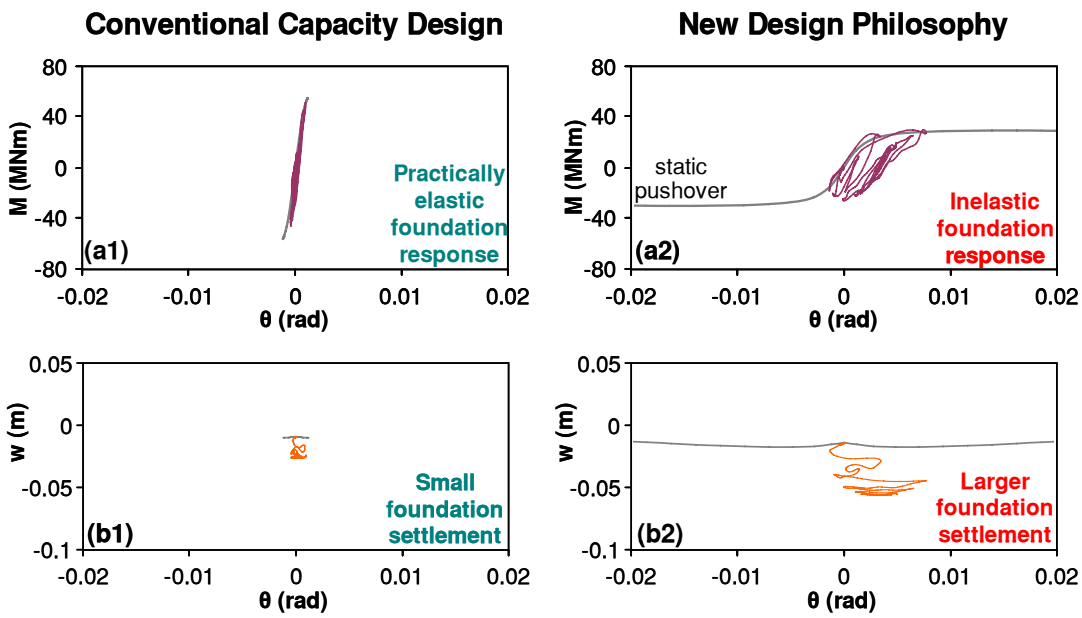

Figure 6. Comparison of the response of the two alternatives subjected to a moderate intensity seismic motion (Kalamata, 1986), within the design limits. (a1 and a2) Overturning moment versus rotation (M–θ) for the two foundations. While the conventional design entails practically elastic response of the foundation–soil system, the new design scheme experiences substantial inelastic action. (b1 and 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.

The time histories of deck horizontal displacement, i.e. the drift Δ, for the two alternatives are compared in Figure 7b. 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 ΔC (Figure 7b1), exactly the opposite is observed for the under-designed foundation of the new design philosophy : Δ is mainly due to foundation rotation Δr (Figure 7b2). Nevertheless, despite the differences in the mechanism leading to its development (pier distortion versus or foundation rotation), the total drift is quite similar : maximum and residual Δ is slightly larger for the new concept, but quite tolerable.

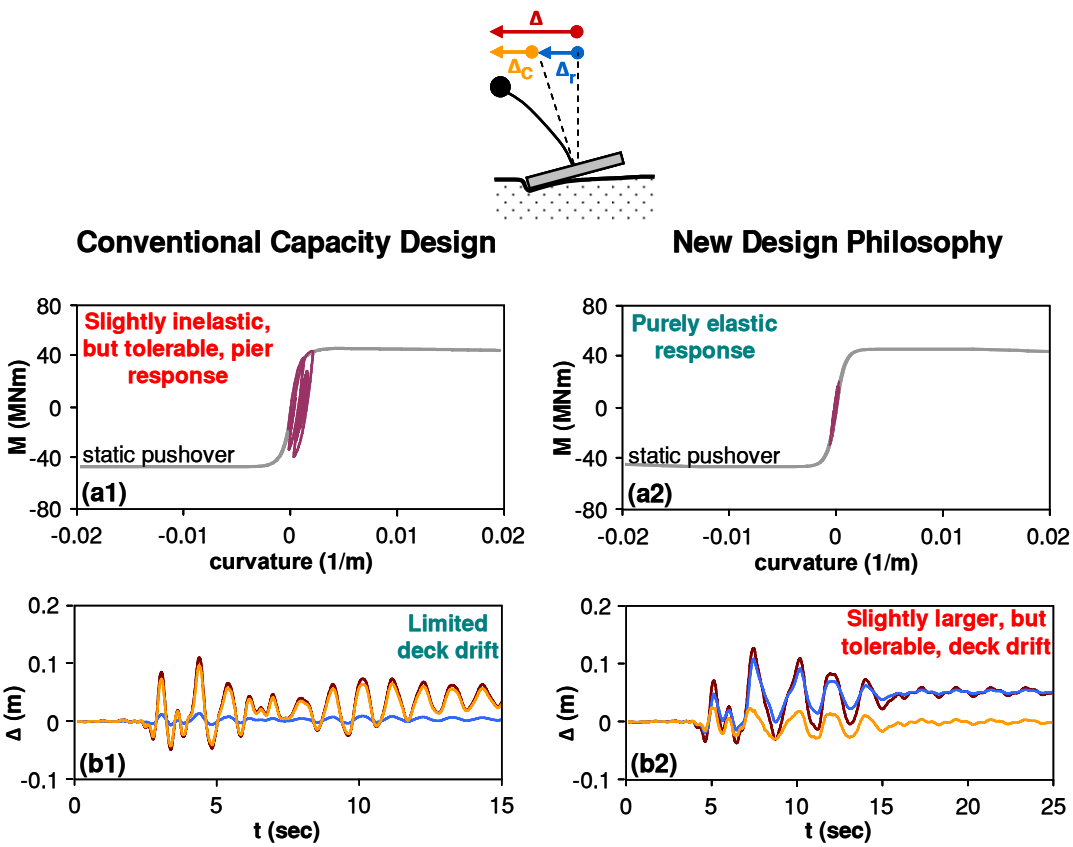

Figure 7. Comparison of the response of the two alternatives subjected to a moderate intensity seismic motion (Kalamata, 1986), within the design limits. (a1 and a2) Bending moment–curvature response at the base of the pier. In the conventionally designed system some inelasticity develops, but the ductility demand is totally tolerable. The response of the pier of the new concept is purely elastic. (b1 and b2) 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 Large Intensity Seismic Motions*

We now compare the response of the two alternatives for a large intensity seismic motion, substantially exceeding the design limits (Figures 8 and 9) : the Takatori accelerogram of the 1995 MJMA 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 Ms 8 Kanto earthquake, it is simply considered as one of the most dev-

astating 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 (Figure 5) to notice how much larger it is throughout the whole range of periods.

Figure 8a compares the response of the two alternatives, in terms of deformed mesh with superimposed plastic strain. In the conventionally designed system (Figure 8a1) 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 (Figure 8a2) 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 mechanism.

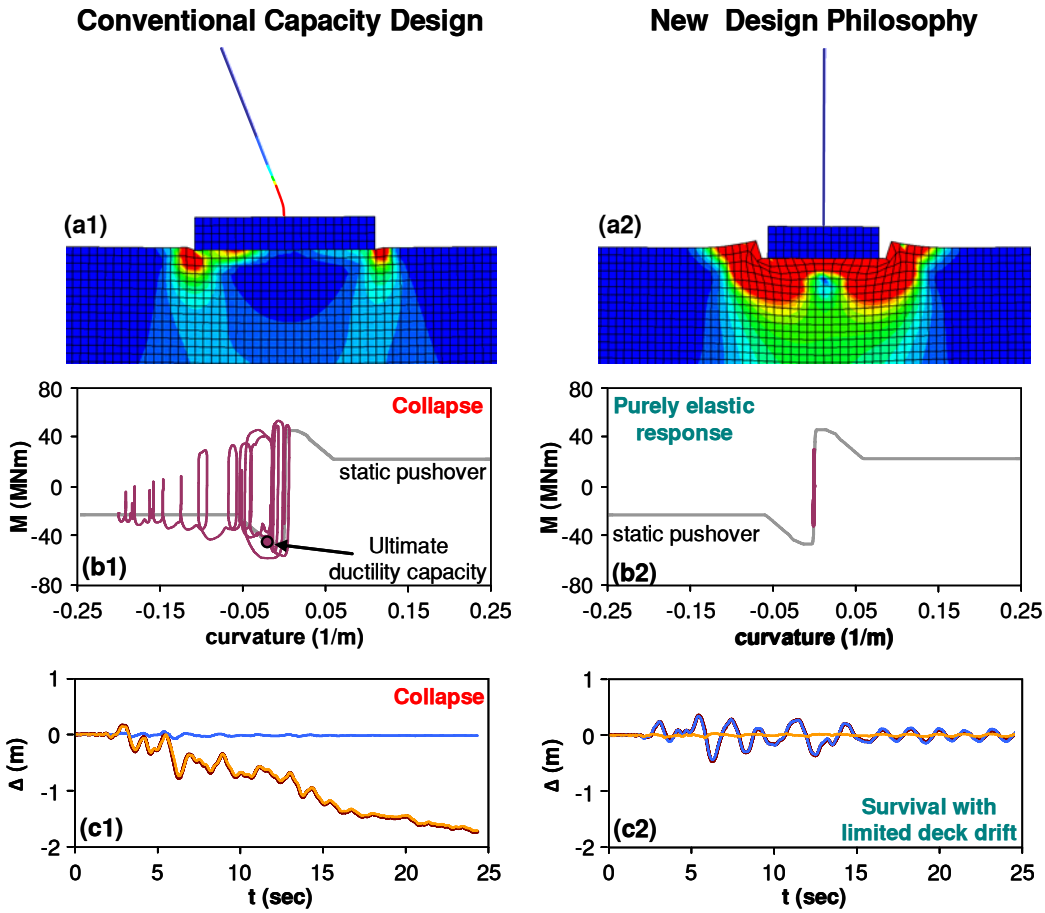

Figure 8. Comparison of the response of the two alternatives subjected to a large intensity seismic motion (Takatori, 1995), exceeding the design limits. (a1 and a2) Deformed mesh with superimposed plastic strain, showing the location of the "plastic hinge". (b1 and b2) Bending moment–curvature response at the pier base. Subjected to ductility demand far exceeding the design, the conventionally designed pier would collapse. With the new design philosophy, the pier remains elastic. (c1 and 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 concept is large (mainly due to foundation rotation), but it survives with insignificant residual drift.

As seen in Figure 8b, 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 (Figure 8c1). In marked contrast, the system designed according to the new philosophy easily survives (Figure 8c2). 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 Figure 9a. Respecting its design principles, the conventional B = 11 m foundation–soil system remains practically elastic (Figure 9b1) ; 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 (Figure 9b2) : mobilization 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 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 9b compares the settlement–rotation (w–θ) response for the two alternatives. While the practically elastic response of the conventional (over-designed) foundation leads to a minor 7 cm settlement (Fig. 9b1), the under-designed foundation of the new philosophy experiences an increased accumulated 24 cm settlement (Fig. 6b2). 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.

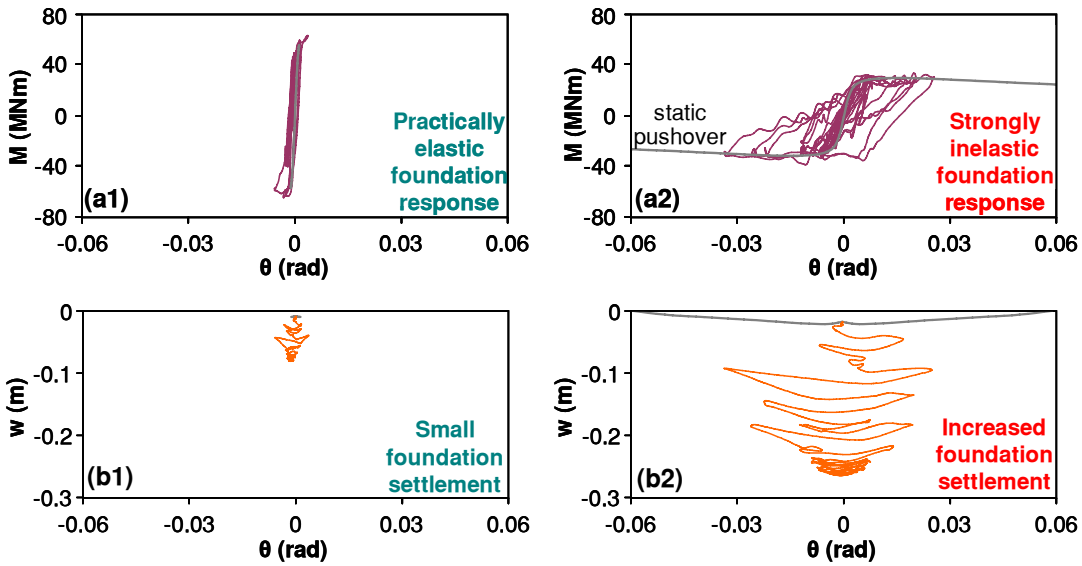

Figure 9. Comparison of the response of the two alternatives subjected to a large intensity seismic motion (Takatori, 1995), exceeding the design limits. (a1 and a2) Overturning moment–rotation (M–θ) response of the two foundations. While the response of the conventionally designed foundation remains practically elastic, the response of the new concept is strongly inelastic. (b1 and b2) 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 small price to pay to avoid collapse.

5 DISCUSSION AND CONCLUSIONS

The overall performance (for all 29 seismic excitations) of the two design alternatives is complied and synopsized in Figure 10. We present key performance indicators with respect to peak ground acceleration a_E of the seismic excitation (at bedrock).

Figure 10a compares the ratio of displacement ductility demand over ductility capacity $\mu_{demand} / \mu_{capacity}$, for the two alternatives. For the conventional design (Figure 10a1), 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 (Figure 10a2) : $\mu_{demand} / \mu_{capacity}$ 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. 10b), especially for large intensity earthquakes. The conventional design is superior in terms residual Δ only for small earthquakes, in which both superstructure and foundation remain completely elastic. Figure 10c 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 :

- For moderate intensity seismic motions not exceeding the design limits, the performance of both alternatives is totally acceptable : both of them would be utilizable 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.
- For large intensity seismic motions that clearly exceed the design limits, the performance of the system designed according to the new design 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, the new design philosophy seems to have a potential for significantly larger safety margins.

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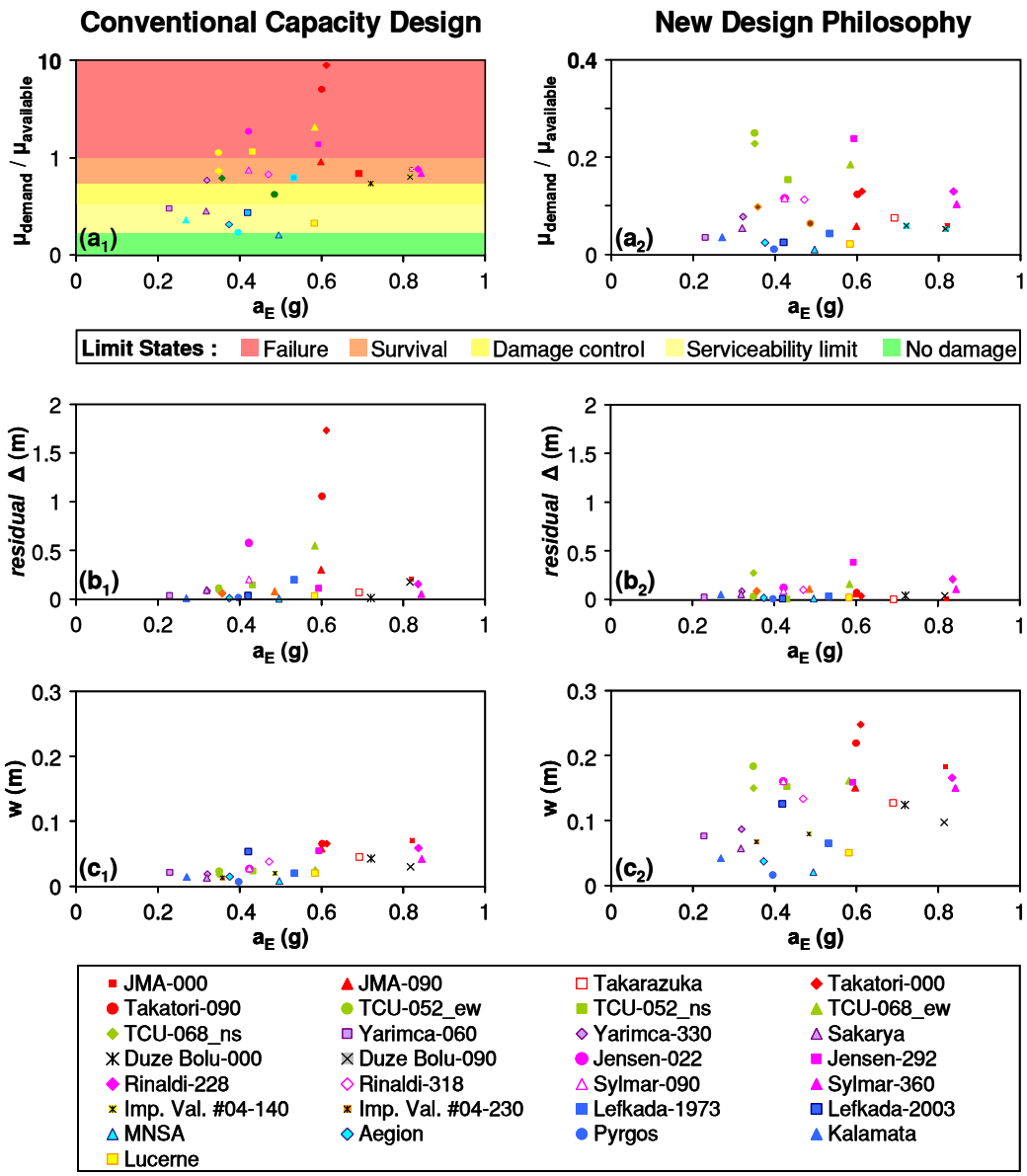

Figure 10. Synopsis of the response of the two alternatives with respect to peak ground acceleration a_E. (**a**$_1$ and **a**$_2$) Ratio of displacement ductility *demand* over ductility *capacity*. For the conventional design, we also indicate the damage level with reference to Response Limit States [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. (**b**$_1$ and **b**$_2$) 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. (**c**$_1$ and **c**$_2$) 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.

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