# Ishihara Lecture
# Soil-Foundation-Structure Systems beyond Conventional Seismic Failure Thresholds

## Conférence Ishihara
## Les systèmes sol-fondation-structure qui dépassent les limites de la rupture parasismique conventionnelle

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ABSTRACT: A new paradigm has now emerged in performance–based seismic design of soil−foundation−structure systems. Instead of imposing strict safety limits on forces and moments transmitted from the foundation onto the soil (aiming at avoiding pseudo-static failure), the new dynamic approach "invites" the creation of two simultaneous "failure" mechanisms: substantial foundation uplifting and ultimate-bearing-capacity slippage, while ensuring that peak and residual deformations are acceptable. The paper shows that allowing the foundation to work at such extreme conditions not only may not lead to system collapse, but it would help protect (save) the structure from seismic damage. A potential price to pay: residual settlement and rotation, which could be abated with a number of foundation and soil improvements. Numerical studies and experiments demonstrate that the consequences of such daring foundation design would likely be quite beneficial to bridge piers and building frames. It is shown that system collapse could be avoided even under seismic shaking far beyond the design ground motion.

RÉSUMÉ : Un nouveau paradigme a émergé dans la conception sismique de la performance des systèmes sol – fondation – structure. Au lieu d'imposer des coefficients de sûreté sur les forces et les moments transmis par la fondation sur le sol (pour éviter la rupture pseudo-statique), la nouvelle approche dynamique permet la création de deux modes de rupture simultanés : le soulèvement important de la fondation et le dépassement de la capacité portante ultime, tout en assurant que les déformations maximales et résiduelles sont acceptables. L'article montre que, quand on permet à la fondation de travailler dans ces conditions extrêmes, l'effondrement du système peut être évité et de plus la structure peut être protégée du dommage sismique. Un prix potentiel à payer : le déplacement et la rotation résiduels, qui peuvent être contrôlés avec différentes méthodes d'amélioration de la fondation et des sols. Des études numériques et expérimentales montrent que les conséquences d'une telle conception audacieuse de la fondation seraient certainement très bénéfiques pour les ponts et les bâtiments. On montre que l'effondrement du système pourrait être évité, même pendant des secousses sismiques qui dépassent le mouvement de calcul.

KEYWORDS: seismic analysis, performance-based design, foundation rocking, bearing capacity failure, nonlinear vibrations

## 1 CURRENT STATE OF PRACTICE: THE CONVENTIONAL "WISDOM"

Seismic design of structures recognises that highly inelastic material response is unavoidable under the strongest possible shaking of the particular location and for the specific soil where the structure is founded. "Ductility" levels of the order of 3 or more are usually allowed to develop under seismic loading, implying that the strength of a number of critical bearing elements is fully mobilized. In the prevailing structural terminology "plastic hinging" is allowed to develop as long as the overall stability is maintained.

By contrast, a crucial goal of current practice in seismic "foundation" design, particularly as entrenched in the respective codes is to *avoid* the mobilisation of "strength" in the foundation. In the words of EC8 (Part 2, § 5.8) :

> *"…foundations shall not be used as sources of hysteretic energy dissipation, and therefore shall be designed to remain elastic under the design seismic action."*

In *structural* terminology : *no* "plastic hinging" is allowed in the foundation. In simple *geotechnical* terms, the designer must ensure that the below-ground (and hence un-inspectable) support system will not even reach a number of "thresholds" that would conventionally imply failure. Specifically, the following states are prohibited :

- plastic structural "hinging" in piles, pile-caps, foundation beams, rafts, and so on
- mobilisation of the so-called *bearing-capacity failure* mechanisms under cyclically−uplifting shallow foundations
- sliding at the soil–footing interface or excessive uplifting of a shallow foundation
- passive failure along the normal compressing sides of an embedded foundation
- a combination of two or more of the above "failure" modes.

In this conventional approach to foundation design, "overstrength" factors plus (explicit and implicit) factors of safety larger than 1 (e.g. in the form of "material" factors) are introduced against each of the above "failure" modes, in a way qualitatively similar to the factors of safety of the traditional static design. Thus, the engineer is certain that foundation performance will be satisfactory and there will be no need to inspect and repair after strong earthquake shaking — a task practically considered next to impossible.

Some of the above thresholds stem not just from an understandable engineering conservatism, but also from a purely (pseudo) static thinking. It will be shown that such an approach may lead not only to unnecessarily expensive foundation solutions but also, in many situations, to less safe structures.

## 2 SOME COMPELLING REASONS TO GO BEYOND CONVENTIONAL THRESHOLDS

A growing body of evidence suggests that soil–foundation plastic yielding under seismic excitation is unavoidable, and at times even desirable; hence, it must be considered in analysis and perhaps allowed in design. *[See for an early recognition : Pecker 1998, Faccioli & Paolucci 1999, Martin & Lam 2000, FEMA-356 2000, Kutter et al 2001, Gazetas & Apostolou 2003.]* The urgent need to explicitly consider the possibility of the foundation system to go beyond "failure" thresholds, and the potential usefulness of doing so, have emerged from :

**(a)** The large (often huge) effective ground acceleration, A, and velocity, V, levels recorded in several earthquakes in the last 25 years. A f*ew examples :*

- *1994 $M_s \approx 6.8$ Northridge : A = 0.98 g, V = 140 cm/s ;*
- *1995 $M_J \approx$ 7.2 Kobe : A = 0.85 g, V = 120 cm/s ;*
- *1986 $M_s \approx 5.6$ San Salvador : A = 0.75 g, V = 84 cm/s ;*
- *2003 $M_s$ = 6.4 Lefkada : A $\approx$ 0.55 g, V = 50 cm/s ;*
- *2007 $M_J \approx 6.9$ Niigata : A =1.20 g, V = 100cm/s .*

With the correspondingly large accelerations in the (above–ground) structure from such ground motions (spectral $S_a$ values well in excess of 1 g), 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 survived severe ground shaking if "plastic hinging" of some sort had not taken place in the "foundation" — usually unintentionally.

**(b)** In seismically retrofitting a building or a bridge, allowing for soil and foundation yielding is often the most rational alternative. Because increasing the structural capacity of some elements, or introducing some new stiff elements, would then imply that the forces transmitted onto their foundation will be increased, to the point that it might not be technically or economically feasible to undertake them "elastically". The new American retrofit design guidelines (FEMA 356) explicitly permit some forms of inelastic deformations 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 had been introduced by Martin & Lam 2000. Such a wall, being much stiffer than the columns of the frame, would carry most of the inertia-driven shear force and would thus transmit a disproportionately large horizontal force and overturning moment onto the foundation compared with its respective small vertical force. If uplifting, sliding, and mobilisation of bearing capacity failure mechanisms in the foundation had been all spuriously ignored, or had been conversely correctly taken into account, would have led to dramatically different results. With "beyond–threshold" action in the foundation the shear wall would "shed" off some of the load onto the columns of the frame, which must then be properly reinforced ; the opposite would be 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)** Many slender historical monuments (e.g. ancient columns, towers, sculptures) may have survived strong seismic shaking during their life (often of thousands of years). While under static conditions such "structures" would have easily toppled, it appears that sliding at, and especially uplifting from, their base during oscillatory seismic motion was a key to their survival (Makris & Roussos 2000, Papantonopoulos 2000). These nonlinear interface phenomena cannot therefore be ignored, even if their geometrically–nonlinear nature presents computational difficulties.

In fact, it is worthy of note that the lack of recognition of the fundamental difference between pseudo-static and seismic overturning threshold accelerations has led humanity to a gross under-estimation of the largest ground accelerations that must have taken place in historic destructive earthquakes. Because, by observing in numerous earthquakes that very slender blocks (of width *b* and height *h, with h >> b*) or monuments in precarious equilibrium that had not overturned, engineers had invariably attributed the fact to very small peak accelerations, less than *(b/h)g*, as would be necessary if accelerations were applied pseudostatically in one direction. Today we know that sometimes even five times as large peak ground acceleration of a high-frequency motion may not be enough to overturn a slender block (Koh et al 1986, Makris & Roussos 2000, Gazetas 2001). Simply stated: even severe uplifting (conventional "failure") may not lead to overturning (true "collapse") under dynamic seismic base excitation.

**(d)** Compatibility with structural design is another reason for the soil–structure interaction analyst to compute the lateral load needed for collapse of the foundation system, as well as (in more detail) the complete load–displacement or moment–rotation response to progressively increasing loading up to collapse. Indeed, in State of the Art (SOA) structural engineering use is made of the so-called "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 sometimes based on estimated losses under the worst possible (as opposed to probable) earthquake scenario.

**(e)** Several persuasive arguments could be advanced on the need *not to* disallow structural plastic "hinging" of piles:

- Yielding and cracking of piles (at various critical depths) is unavoidable with strong seismic shaking in soft soils, as the Kobe 1995 earthquake has amply revealed.
- Refuting the contrary universal belief, post-earthquake inspection of piles is often feasible (with internally placed inclinometers, borehole cameras, integrity shock testing, under-excavation with visual inspection ), although certainly not a trivial operation. Again, Kobe offered numerous examples to this effect.
- The lateral confinement provided by the soil plays a very significant role in pile response, by retarding the development of high levels of localised plastic rotation, thereby providing an increase in ductility capacity. Sufficient displacement ductility may be achieved in a pile shaft with transverse reinforcement ratio as low as 0.003 (Butek et al 2004).
- The presence of soil confinement leads to increased plastic hinge lengths, thus preventing high localised curvatures (Tassios 1998). Therefore, the piles retain much of their axial load carrying capacity after yielding.

Thus, a broadly distributed plastic deformation on the pile may reduce the concentrated plastification on the structural column — so detrimental to safety.

Furthermore, when subjected to strong cyclic overturning moment, end-bearing piles in tension will easily reach their full frictional uplifting capacity. It has been shown analytically and experimentally that this does not imply failure. The same argument applies to deeply embedded (caisson) foundations.

**(f)** 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 et al 2000, 2003, Calvi 2007]. *Geotechnical* earthquake engineering has also been slowly moving towards performance–based seismic design**:** gravity retaining structures are indeed allowed to slide during the design earthquake. 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.

## 3 THE CONCEPT OF "ROCKING ISOLATION" IN FOUNDATION DESIGN

The paper addresses the case of structure-foundation systems oscillating mainly in a rotational mode (rocking).

Subjected to strong seismic shaking, structures tend to experience large inertial forces. For tall-slender structures these forces will lead to overturning moments onto the foundation that may be disproportionally large compared to the vertical load. As a result, a shallow foundation may experience detachment (uplifting) of one edge from the supporting soil. This in turn will lead to increased normal stresses under the opposite edge of the foundation. Development of a bearing capacity failure mechanism is quite possible if such a concentration leads to sufficiently large stresses. But, in contrast to a static situation, even *then* failure may not occur. Thanks to the *cyclic* and *kinematic* nature of earthquake induced vibrations : (i) the inertial forces do not act *"forever"* in the same direction to cause failure (as would be the case with static load), but being cyclic, very soon reverse and thereby relieve the distressed soil; and (ii) the developing inertial forces are not externally applied predetermined loads, but are themselves reduced once the soil-foundation system reaches its (limited) ultimate resistance — the foundation system acts like a fuse. As a result, the system experiences nonlinear-inelastic rocking oscillations, which may or may not result in excessive settlement and rotation. But failure is almost unlikely.

In the last 10 years a number of research efforts have explored the consequences of substantial foundation rocking on the response of the supported structure, theoretically and experimentally : Kutter et al 2003, Gajan et al 2005, Harden et al 2006, Kawashima et al 2007, Apostolou et al 2007, Paolucci et al 2008, Chatzigogos & Pecker 2010, Deng et al 2012. The results of these studies confirmed the idea that strongly-nonlinear rocking oscillations under seismic excitation can be of benefit to the structure.

Taking the whole idea one small step farther, it is proposed that the design of a shallow foundation should actively "invite" the creation of two simultaneous "failure" mechanisms: substantial foundation uplifting and ultimate bearing-capacity sliding. This would be accomplished by substantially under-designing the foundation — e.g., by reducing its width and length to, say, one-half of the values required with current design criteria. This can be thought of as a reversal of the "capacity" design: "plastic hinging" will take place in the foundation-soil system and not at the column(s) of the structure. Fig. 1 elucidates the main idea of Rocking Isolation. The benefits of designing the foundation to work at and beyond its conventional limits will become evident in the sequel. To this end, three examples will elucidate the dynamics of "Rocking Isolation" in comparison with the dynamics of the conventional design :

(a) a bridge pier, free to rotate at its top
(b) a two-storey two-bay asymmetric frame (MRF)
(c) a three-storey retrofitted frame–shearwall structure.

In each case, the two alternatives ( the conventional and the rocking-isolated system) are subjected to numerous acceleration time histories the overall intensity of which is either within or well beyond the design earthquake levels.

## 4 ROTATIONAL MONOTONIC RESPONSE OF SHALLOW FOUNDATIONS

Much of the research in earlier years on dynamic rocking of foundations and dynamic soil–structure interaction had focused on linear response. Elastic stiffness and damping as functions of frequency have been developed and utilised to describe the dynamic action of the foundation system. The various US seismic codes in the last $30^+$ years have promulgated linear approximations to deal with seismic soil–structure interaction.

The behavior of "Rocking Foundations" significantly deviates from linear visco-elasticity: uplifting introduces strong geometric nonlinearity and even damping due to impact ; soil yielding and plastic deformation generate hysteresis, implying significant frequency-independent damping, while when bearing-capacity slippage mechanisms develop a limiting plateau restricts the passage of high accelerations from the ground into the superstructure.

In monotonic loading, a most crucial parameter controlling the moment–rotation, M–θ, relation of a specific foundation is the factor of safety against vertical static bearing capacity failure :

$$F_s = N_{uo}/N \tag{1}$$

where $N_{uo}$ is the ultimate load under purely vertical loading and N the acting vertical load. Fig. 2 offers typical results for a homogeneous ($G$ and $s_u$ ) soil for three $F_s$ values : a very high one (20), a low one (2), and an extremely low one (1.25). M is normalized by $N_{uo}$ B, where B is the width of the footing in the direction of loading. This leads to curves which, for the homogeneous profile considered, depend solely on the so-called "rigidity index", $G/ s_u$ , and the shape of the footing.

Also shown in Fig. 2 are the snapshots of the deformed soil and the contours of plastic strain as they develop when the maximum moment is reached — apparently at different angles of rotation. The following are worthy of note in the figure:

- The foundation with $F_s = 20$ (which can be interpreted either as a very-lightly loaded foundation or as a "normally"-loaded foundation on very stiff soil) despite its largest initial elastic rocking stiffness fails at the smallest value of applied moment:

$$M_u \approx 0.025\, N_{uo}\, B \tag{2a}$$

  Indeed if $F_s \rightarrow \infty$ , i.e. there is no vertical load onto the foundation, $M_u$ would vanish, due to the tensionless nature of the soil–footing interface.

- As expected from the literature (Meyerhof 1963, Georgiadis and Butterfield 1988, Salençon and Pecker 1995, Allotey and Naggar 2003, Apostolou and Gazetas 2005, Gajan and Kutter 2008, Chatzigogos et al. 2009, Gouvernec 2009, Gajan and Kutter 2008) the largest maximum moment is attained by the $F_s = 2$ footing :

$$M_u \approx 0.13\, N_{uo}\, B \tag{2b}$$

  but its elastic initial rocking stiffness is smaller than for the $F_s = 20$ foundation. Evidently, the extensive plastic deformations upon the application of the vertical (heavy) load soften the soil so that a small applied moment meets less resistance — hence lower stiffness. However, $F_s = 2$ achieves the largest ultimate $M_u$ as it leads to an optimum combination of uplifting and bearing-capacity mobilization.

- A more severely loaded foundation, however, with the (rather unrealistic) $F_s = 1.25$ will only enjoy an even smaller initial stiffness and a smaller ultimate moment than the $F_s = 2$ foundation. Notice that in this case no uplifting accompanies the plasticification of the soil.

The failure envelope (also called interaction diagram) in N-M space is given in Fig. 3 for the specific example. It was obtained with the same numerical (FE) analysis as the curves and snapshots of Fig. 2, and can be expressed analytically as a function of the static factor of safety ($F_S$) as

$$M_u = \frac{1}{2F_S}\left(1-\frac{1}{F_S}\right)N_{uo}B \tag{3}$$

The specific plot is in terms of $N/N_{uo}$ which is $1/F_s$ which ranges between 0 and 1. Notice that heavily and lightly loaded foundations with $1/F_s$ symmetrically located about the $1/F_S = 0.5$ value where the $M_u$ is the largest, have the same moment capacity : yet their behavior especially in cyclic loading is quite different as will be shown subsequently.

## 5 MONOTONIC RESPONSE ACCOUNTING FOR P–δ EFFECTS

An increasingly popular concept in structural earthquake engineering is the so-called *"pushover"* analysis. It refers to the nonlinear lateral force-displacement relationship of a particular structure subjected to monotonically increasing loading up to failure. The development (theoretical or experimental) of such pushover relationships has served as a key in simplified dynamic response analyses that estimate seismic deformation demands and their ultimate capacity. We apply the pushover idea to a shallow foundation supporting an elevated mass, which represents a tall slender structure with $h/B = 2$ (or "slenderness" ratio $h/b = 4$, *where* $b = B/2$). This mass is subjected to a progressively increasing horizontal displacement until failure by overturning. Since our interest at this stage is only in the behavior of the foundation, the structural column is considered absolutely rigid. The results are shown in Fig:4(a) and (b) for two $F_s$ values : 5 and 2.

The difference in the $M$-$\theta$ response curves from those of Fig. 2 stems from the so-called P-δ effect. As the induced lateral displacement of the mass becomes substantial its weight induces an additional aggravating moment, $mgu = mg\theta h$, where $\theta$ is the angle of foundation rotation. Whereas before the ultimate moment $M_u$ is reached the angles of rotation are small and this aggravation is negligible, its role becomes increasingly significant at larger rotation and eventually becomes crucial in driving the system to collapse. Thus, the (rotation controlled) $M$-$\theta$ curve decreases with $\theta$ until the system topples at an angle $\theta_c$ . This critical angle for a rigid structure on a rigid base ($F_S = \infty$) is simply :

$$\theta_{c,\infty} = arctan\ \frac{b}{h} \tag{4}$$

where b = the foundation halfwidth. For very slender systems the approximation

$$\theta_{c,\infty} \approx \frac{b}{h} \tag{4a}$$

is worth remembering.

As the static vertical safety factor ($F_S$) diminishes, the rotation angle ($\theta_C$) at the state of imminent collapse ("critical" overturning rotation) also slowly decreases. Indeed, for rocking on compliant soil, $\theta_c$ is always lower than it is on a rigid base (given with **Eq. 4**). For stiff elastic soil (or with a very large static vertical safety factor) $\theta_c$ is imperceptibly smaller than that given by Eq. 4, because the soil deforms slightly, only below the (right) edge of the footing, and hence only insignificantly alters the geometry of the system at the point of overturning. As the soil becomes softer, soil inelasticity starts playing a role in further reducing $\theta_C$. However, such a reduction is small as long as the factor of safety ($F_S$) remains high (say, in excess of 3). Such behaviour changes drastically with a very small $F_S$: then the soil responds in strongly inelastic fashion, a symmetric bearing-capacity failure mechanism under the vertical load N is almost fully developed, replacing uplifting as the prevailing mechanism leading to collapse $\theta_c$ tends to zero.

The following relationship has been developed from FE results by Kourkoulis et al, 2012, for the overturning angle $\theta_c = \theta_c(F_s)$ :

$$\frac{\theta_c}{\theta_{c,\infty}} \approx \left(1-\frac{1}{F_S}\right) + \frac{1}{3}\left[1 - log\frac{h}{B}\right]\frac{1}{\sqrt{F_S}} \tag{5}$$

## 6 CYCLIC RESPONSE ACCOUNTING FOR P–δ EFFECTS

Slow cyclic analytical results are shown for the two aforementioned systems having static factors of safety ($F_S$ = 5 and 2). The displacement imposed on the mass center increased gradually; the last cycle persisted until about 4 or 5 times the angle $\theta_u$ of the maximum resisting moment. As can be seen in the moment–rotation diagrams, the loops of the cyclic analyses for the safety factor $F_S = 5$ are well enveloped by the monotonic pushover curves in Figure 7(a). In fact, the monotonic and maximum cyclic curves are indistinguishable. This can be explained by the fact that the plastic deformations that take place under the edges of the foundation during the deformation-controlled cyclic loading are too small to affect to any appreciable degree of response of the system when the deformation alters direction. As a consequence, the residual rotation almost vanishes after a complete set of cycles — an important (and desirable) characteristic. The system largely rebounds, helped by the restoring role of the weight. A key factor of such behaviour is the rather small extent of soil plastification, thanks to the light vertical load on the foundation.

The cyclic response for the $F_S$ = 2 system is also essentially enveloped by the monotonic pushover curves. However, there appears to be a slight overstrength of the cyclic "envelope" above the monotonic curve. For an explanation see Panagiotidou et al, 2012.

But the largest difference between monotonic and cyclic, on one hand, and $F_S = 2$ and 5, on the other, is in the developing settlement. Indeed, monotonic loading leads to monotonically-upward movement ("heave") of the center of the $F_S = 5$ foundation, and slight monotonically-downward movement ("settlement") of the $F_S = 2$ foundation. Cyclic loading with $F_S$ = 5 produces vertical movement of the footing which follows closely its monotonic upheaval.

But the $F_S = 5$ foundation experiences a progressively accumulating settlement — much larger that its monotonic settlement would have hinted at. The hysteresis loops are now wider. Residual rotation may appear upon a full cycle of loading, as inelastic deformations in the soil are now substantial.

The above behavior is qualitatively similar to the results of centrifuge experiments conducted at the University of California at Davis on sand and clay (e.g., Kutter et al. 2003, Gajan et al. 2005) large-scale tests conducted at the European Joint Research Centre, (Negro et al. 2000, Faccioli et al. 1998), and 1-g Shaking Table tests in our laboratory at the National Technical University of Athens on sand (Anastasopoulos et al 2011, 2013, Drosos et al 2012).

In conclusion, the cyclic moment–rotation behavior of foundations on clay and sand exhibits to varying degrees three important characteristics with increasing number of cycles :

- no "strength" degradation (experimentally verified).
- sufficient energy dissipation — large for small $F_S$ values, smaller but still appreciable for large ones. (Loss of energy due to impact will further enhance damping in the latter category, when dynamic response comes into play.)
- relatively low residual drift especially for large FS values — implying a *re-centering* capability of the rocking foundation.

These positive attributes not only help in explaining the favorable behavior of "Rocking Foundation", but also enhance the reliability of the geotechnical design.

## 7 SEISMIC RESPONSE OF BRIDGE PIER ON SHALLOW FOUNDATION

The concept of "Rocking Isolation" is illustrated in Fig. 5 by comparing the response of a 12 m tall bridge pier carrying a deck of four lanes of traffic for a span of about 35 m — typical of elevated highways around the world.

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

The pier is founded through a square foundation of width $B$ on an idealized homogeneous 25 m deep stiff clay layer, of undrained shear strength $s_u$ = 150 kPa (representative soil conditions for which a surface foundation would be a realistic solution). Two different foundation widths are considered to represent the two alternative design approaches. A large square foundation, $B$ = 11 m, is designed in compliance with conventional capacity design, applying an overstrength factor $\gamma_{Rd}$ = 1.4 to ensure that the plastic "hinge" will develop in the superstructure (base of pier). Taking account of maximum allowable uplift (eccentricity $e = M / N < B/3$, where $N$ is the vertical load), the resulting safety factors for static and seismic loading are $F_S$ = 5.6 and $F_E$ = 2.0, respectively. A smaller, under-designed, $B$ = 7 m foundation is considered in the spirit of the new design philosophy. Its static safety factor $F_S$= 2.8, but it is designed applying an "*understrength*" factor $1/1.4 \approx 0.7$ for seismic loading. Thus, the resulting safety factor for seismic loading is lower than 1.0 ($F_E \approx$ 0.7).

The seismic performance of the two alternatives is investigated through nonlinear FE dynamic time history analysis. An ensemble of 29 real accelerograms is used as seismic excitation of the soil–foundation–structure system. In all cases, the seismic excitation is applied at the bedrock level. Details about the numerical models and the requisite constitutive relations can be seen in Anastasopoulos et al, 2010, 2011.

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

Fig. 5 compares the response of the two alternatives, in terms of deformed mesh at the end of shaking with superimposed the plastic strains. In the conventionally designed system there is very little inelastic action in the soil; the red regions of large plastic deformation are seen only under the severely "battered" edges of the rocking foundation — but without extending below the foundation. "Plastic hinging" forms at the base of the pier, leading to a rather intense accumulation of curvature (deformation scale factor = 2).The P–δ effect of the mass will further aggravate the plastic deformation of the column, leading to collapse.

In stark contrast, with the new design scheme the "plastic hinge" takes the form of mobilization of the bearing capacity failure mechanisms in the underlying soil, leaving the superstructure totally intact. Notice that the red regions of large plastic shearing are of great extent, covering both half-widths of the foundation and indicating alternating mobilization of the bearing capacity failure mechanisms, left and right.

The above observations are further confirmed by the time history of deck drift shown in Fig. 5(c). The two components of drift, are shown, one due to footing rotation in blue and one due to structural distortion in green. Their sum is shown in red. Evidently, the conventional design experiences essentially only structural distortion which leads to uncontrollable drifting — collapse. In marked contrast, the system designed according to the new philosophy easily survives. It experiences substantial *maximum* deck drift (about 40 cm), almost exclusively due to foundation rotation. Nevertheless, the *residual* foundation rotation leads to a tolerable 7 cm deck horizontal displacement at the end of shaking.

Fig. 5(d) further elucidates the action of the foundation-soil system. The M-θ relationship shows for the $11 m^2$ foundation a nearly linear viscoelastic response, well below its ultimate capacity and apparently with no uplifting. On the contrary, the $7 m^2$ (under-designed) foundation responds well past its ultimate moment capacity, reaching a maximum $\theta \approx 30$ *mrad*, generating hysteretic energy dissipation, but returning almost to its original position, i.e. with a negligible residual rotation.

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

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

## 8 SEISMIC RESPONSE OF TWO–STOREY TWO BAY ASYMMETRIC FRAME

The frame of Fig. 6 was structural designed according to EC8 for an effective ground acceleration A = 0.36 g and ductility-dependent "behavior" factor q = 3.9. The soil remains the stiff clay of the previous example. Two alternative foundation schemes are shown in the figure .

The conventionally *over-designed* footings can mobilize a maximum moment resistance $M_u$ from the underlying soil, larger than the bending moment capacity of the corresponding column $M_{COL}$ .. For static vertical loads, a factor of safety $F_S \geq 3$ is required against bearing capacity failure. For seismic load combinations, a factor of safety $F_E$ = 1 is acceptable. In the latter case, a maximum allowable eccentricity criterion is also enforced: $e = M/N \leq B/3$. For the investigated soil–structure system this eccentricity criterion was found to be the controlling one, leading to minimum required footing widths $B$ = 2.7 m, 2.5 m and 2.4 m for the left, middle, and right footing, respectively. Bearing capacities and safety factors are computed according to the provisions of EC8, which are basically similar to those typically used in foundation design practice around the world.

The *under-sized* footings of the rocking isolation scheme, are "weaker" than the superstructure, guiding the plastic hinge to or below the soil–footing interface, instead of at the base of the columns. The small width of the footings promotes full mobilization of foundation moment capacity with substantial uplifting. The eccentricity criterion is completely relaxed, while $F_E < 1$ is allowed. The static $F_S \geq 3$ remains a requirement as a measure against uncertainties regarding soil strength. Moreover, it turns out that $F_S \geq 4$ might be desirable in order to promote uplifting–dominated response, and thereby limit seismic settlements [Kutter et al. 2003, Faccioli et al. 2001,Pecker &

Pender 2000, Kawashima et al. 2007, Chatzigogos et al. 2009; Panagiotidou et al. 2012]. Applying the methodology which has been outlined in Gelagoti et al. 2012, the footings were designed to be adequately small to promote uplifting, but large enough to limit the settlements. Aiming to minimize differential settlements stemming from asymmetry, the three footings were dimensioned in such a manner so as to have the same $F_S$. Based on the above criteria, the resulting footing widths for the rocking–isolated design alternative are $B$ = 1.1 m, 1.8 m, and 1.3 m, for the left, middle, and right footing, respectively: indeed, substantially smaller than those of the code-based design. Footing dimensions and static factors of safety against vertical loading of the two designs are summarized in Table 1.

Table 1. Footing dimensions and corresponding factors of safety (computed following the provisions of EC8) against vertical loading for the seismic load combination ($G$ + 0.3$Q$) for the two design alternatives of Fig. 6.

| *Conventional Design* | | | *Rocking Isolation* | | |
|---|---|---|---|---|---|
| Footing | B (m) | $F_S$ | Footing | B (m) | $F_S$ |
| Left | 2.7 | 32.6 | Left | 1.1 | 5.4 |
| Middle | 2.5 | 10.6 | Middle | 1.8 | 5.4 |
| Right | 2.4 | 18.1 | Right | 1.3 | 5.4 |

The performance of the two design alternatives is compared in Fig. 6. The deformed mesh with superimposed plastic strain contours of the two alternatives is portrayed on top **(Fig. 6a).** With the relentless seismic shaking of the Takatori motion, the conventionally designed frame collapses under its gravity load (due to excessive drift of the structure, the moments produced by $P–\delta$ effects cannot be sustained by the columns, leading to loss of stability and total collapse). As expected, plastic hinges firstly develop in the beams and subsequently at the base of the three columns, while soil under the footings remains practically elastic. The collapse is also evidenced by the substantial exceedance of the available curvature ductility of the columns **(**Fig. 6b). Conversely, the rocking–isolated frame withstands the shaking, with plastic hinging taking place only in the beams, leaving the columns almost unscathed (moment-curvature response: elastic). Instead, plastic hinging now develops within the underlying soil in the form of extended soil plastification (indicated by the red regions under the foundation. The time histories of inter-storey drift further elucidate the aforementioned behavior of the two design alternatives **(**Fig. 6d).

Thanks to the larger bending moment capacity of the column than of the footing, damage is guided “below ground” and at the soil–foundation interface in the form of detachment and uplifting — evidenced in **Fig. 6d** by the zero residual rotation, unveiling the re-centering capability of the under-designed foundation scheme.

The price to pay: large accumulated settlements. Moreover, despite the fact that the three footings have been dimensioned to have the same static factor of safety $F_S$ (in an attempt to minimize differential settlements exacerbated from asymmetry), the central footing settles more than the two side footings, leading to a differential settlement of the order of 3 cm. The difference in the settlement stems of course from their differences in width. As previously discussed, the central footing was made larger ($B$ = 1.8 m, compared to 1.1 m and 1.3 m of the two side footings) in order to maintain the same $F_S$. Since the latter is common for the three footings, if the loading is more-or-less the same, their response should be similar. However, such equivalence refers to dimensionless quantities, not absolute values [see Kourkoulis et al., 2012b]. In other words, while the three footings sustain almost the same dimensionless settlement $w/B$, which is roughly equal to 0.025 (≈ 3 cm/1.2 m) for the two side footings and 0.033 (≈ 6 cm/1.8 m) for the central one, the latter is substantially larger in width and hence its settlement is larger in absolute terms. Naturally, the three footings are not subjected to exactly the same loading, something which further complicates the response. Such differential settlements may inflict additional distress in the superstructure, and are therefore worthy of further investigation.

## 9 THREE–STOREY FRAME RETROFITTED WITH SHEAR–WALL

The results presented now are not from numerical analysis as the previous one, but from Shaking Table experiments. They refer to a 3-storey two-bay frame which was designed according to the pre-1970 seismic regulations, for a base shear coefficient of 0.06. Because of the small value of this coefficient and the otherwise inadequate design, the frame has columns of cross-section 25 x 25 cm$^2$ and beams 25 x 50 cm$^2$ resulting in a strong beam–weak column system. Naturally, it fails by first “soft-story” type of collapse when excited by motions corresponding to today’s codes with effective ground accelerations of the order of 0.30g and more. To upgrade the frame, a strong and stiff Shear Wall 1.5 m x 0.3 m in cross-section is constructed replacing the middle column, as shown in Fig. 7.

The 1:10–scale model is supported on dense fine–grained $D_r$ ≈ 80% sand. The original footings of all three columns were 1.5 m square. For the retrofitted frame the two columns retained their original 1.5 x 1.5m$^2$ footings. The foundation of the Shear Wall (SW) is of special geotechnical interest : due to its disproportionately large lateral stiffness the SW tends to attract most of the seismically induced shear force and hence to transmit onto the foundation a large overturning moment. By contrast, its vertical load is relatively small. To meet the eccentricity limit e = M/N < B/3, a large foundation 6.0m x 0.80 m is thus necessary. Hence, the conventional solution of Fig. 8. Of course the resulting vertical bearing-capacity factor of safety is unavoidably large, $F_S \cong 10$, and the seismic apparent factor of safety against moment bearing-capacity is also far more than adequate : $F_E = 2$.

The decision to reduce the footing width to merely B = 3.5 m is not only economically favorable, but in the harsh reality of old buildings it may often be the only feasible decision in view of the usual space limitations due to pipes, small basements, walls, etc, present in the base. We will see if it is also favorable technically in resisting a strong seismic shaking.

To be practical, in the above sense, no change is made to the column footings. (1.5 m square).

We subject all three structures [ i.e., “a” the original frame, “b” the retrofitted with a SW founded on conventionally-conservative footing, and “c” the retrofitted with the underdesigned SW footing] to a number of strong ground excitations. Frame “a” easily fails as sketched in **Fig. 8,** where the physical collapse was artificially prevented by an external protective barrier in the Shaking Table experiment. The conventionally retrofitted SW-frame “b” could withstand most excitations. But with some of the strongest motions it developed substantial plastification at its base and led to residual top drift of an unacceptable 8%.

The unconventionally–founded system “c” behaved much better with residual top drift of merely 2%.

Figure 8 sketches the deformation pattern of the three systems while Fig. 7 plots the time histories of structural–distortion and foundation–rotation induced top drift ratio. It is seen that not only is the total drift of the Rocking-Isolated system only 2% but at least half of it is solely due to foundation rotation, rather than damage to the SW.

The penalty to pay is the increased settlement (1.5 cm rather 0.8 cm) which nevertheless in this particular case would be acceptable for most applications.

## 10 CONCLUSIONS

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

(b) On the contrary, seriously under-designed foundations limit the transmitted accelerations to levels proportional to their (small) ultimate moment capacity. This leads to much safer superstructures. In earthquake engineering terminology the plastic "hinging" moves from the columns to the foundation-soil system, preventing dangerous structural damage.

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

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

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

The results and conclusions of this paper are in harmony with the numerous experimental and theoretical findings of Professor Bruce Kutter and his coworkers at U.C. Davis, and of Professors Alain Pecker and Roberto Paolucci and their coworkers in Paris and Milano.

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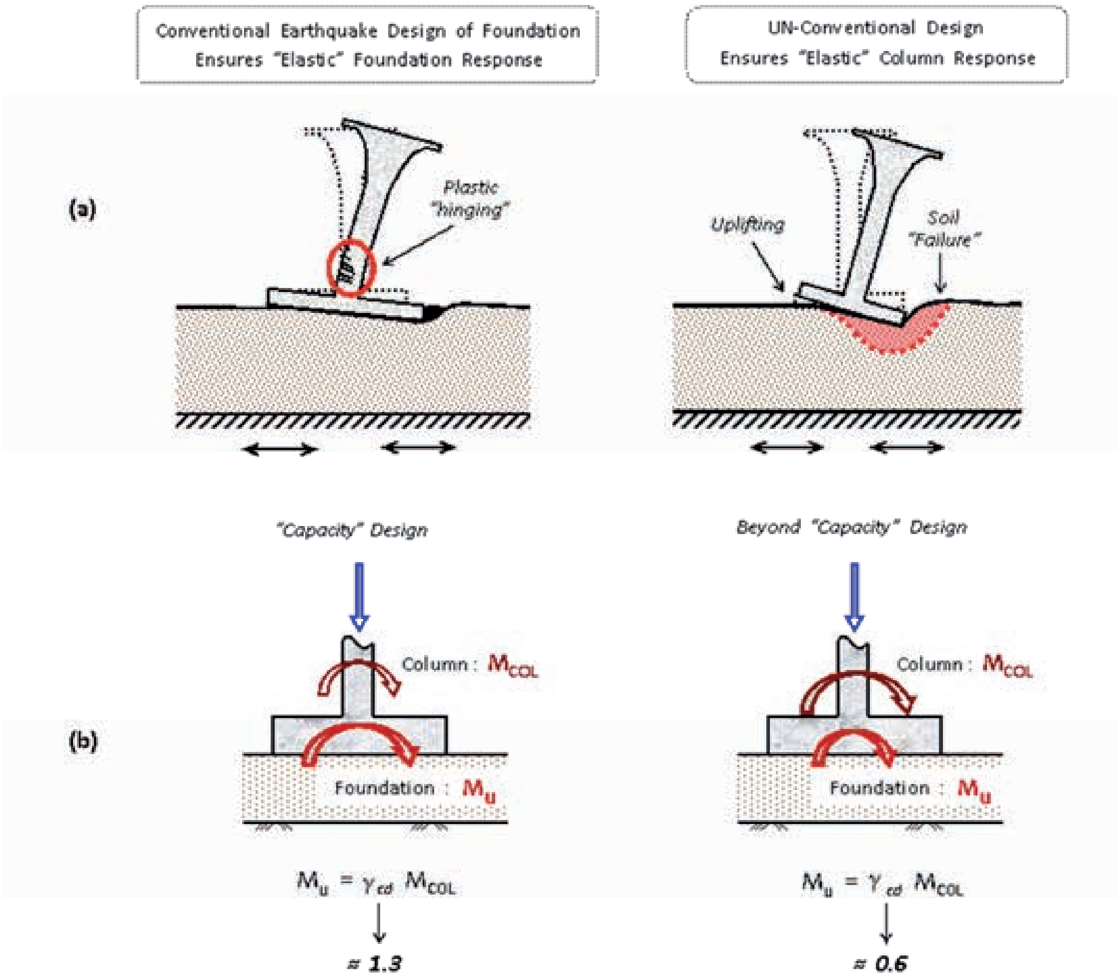


Figure 1. Conceptual illustration of *(a)* the response of a conventional and a "rocking-isolation" design of a bridge-pier foundation; and *(b)* the "capacity" design principle as conventionally applied to foundations, and its reversal in "rocking isolation".

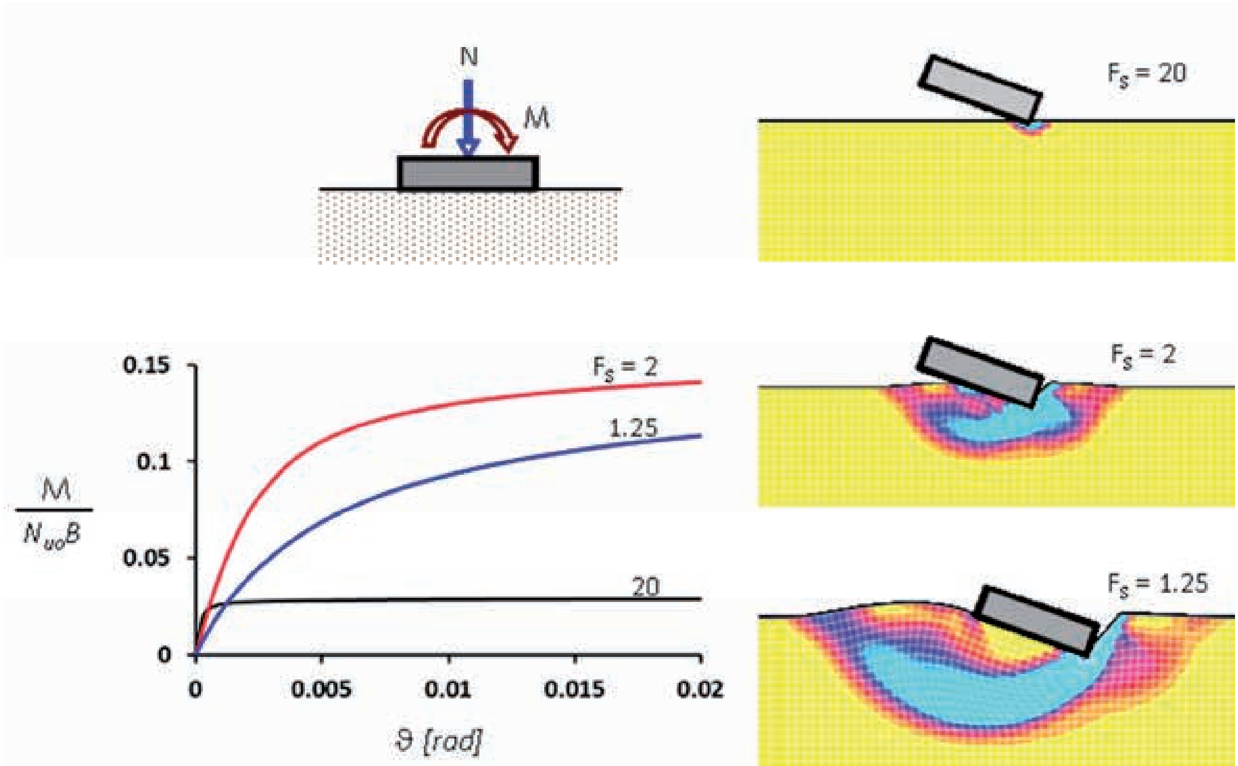


Figure 2. Typical moment–rotation relations of three foundations and corresponding snapshots of their ultimate response with the contours of plastic deformation. The only difference between foundations : their static factor of safety.

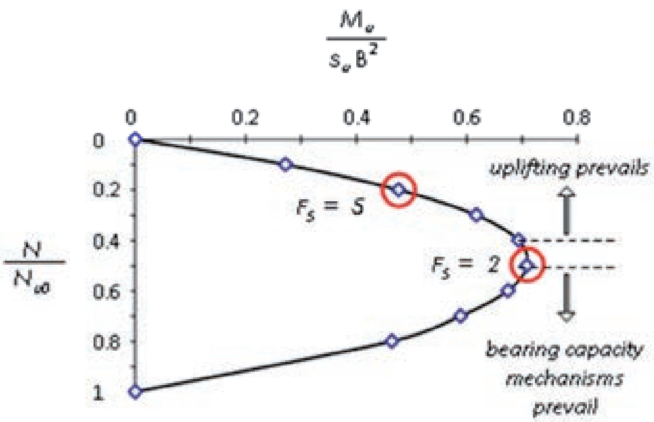


Figure 3. Dimensionless $N_u - M_u$ failure envelope for strip foundation

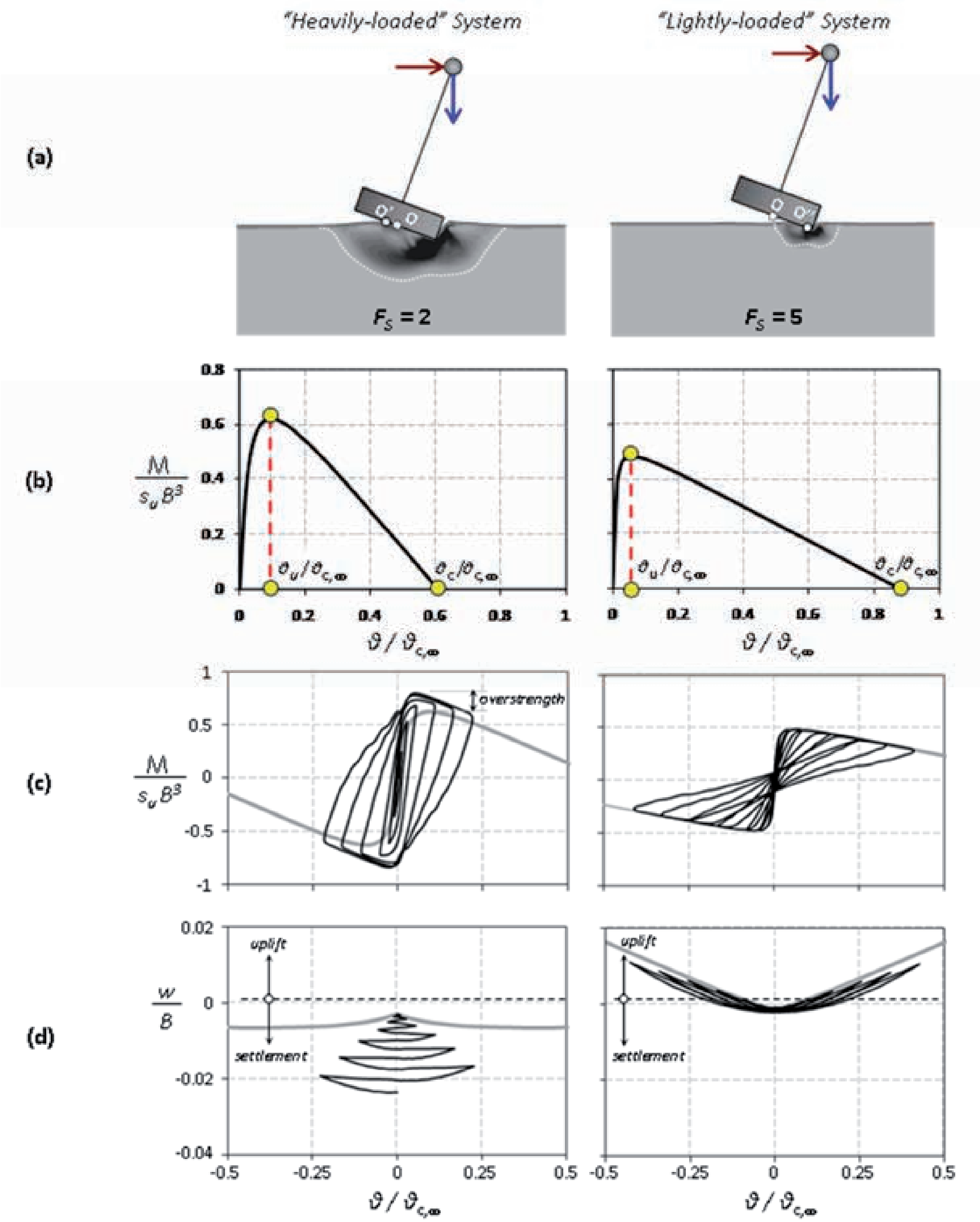


Figure 4. Comparison of two slender systems (differing only in $F_S$) subjected to monotonic and cyclic loading: *(a)* deformed mesh with plastic strain contours at ultimate state; *(b)* dimensionless monotonic moment–rotation response; *(c)* cyclic moment–rotation response; and *(d)* cyclic settlement–rotation response (the grey line corresponds to the monotonic backbone curves).

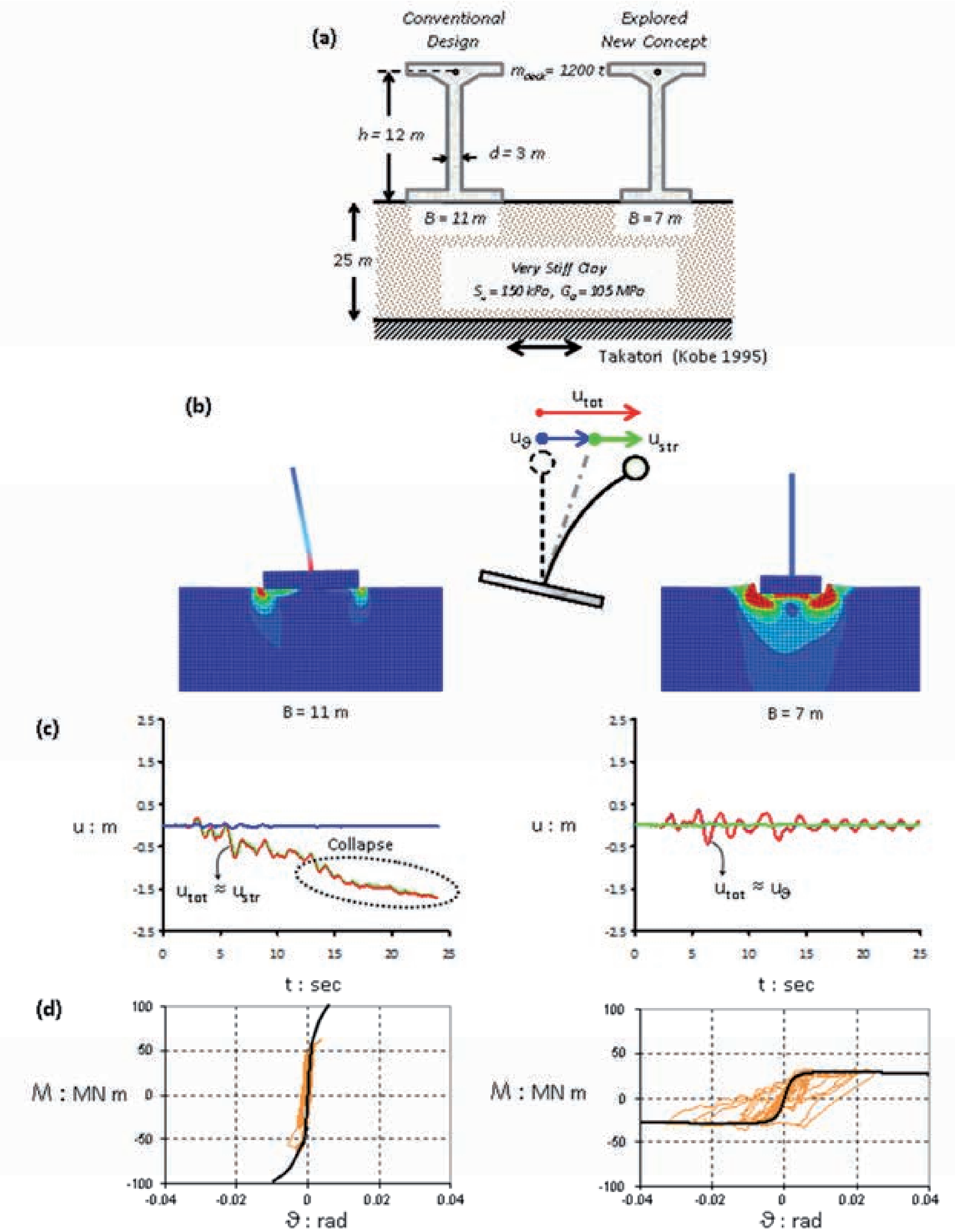


Figure 5. *(a)* Two bridge piers on two alternative foundations subjected to a large intensity shaking, exceeding the design limits; *(b)* deformed mesh with superimposed plastic strain, showing the location of "plastic hinging" at ultimate state; *(c)* time histories of deck drift; *(d)* overturning moment–rotation (M–θ) response of the two foundations.

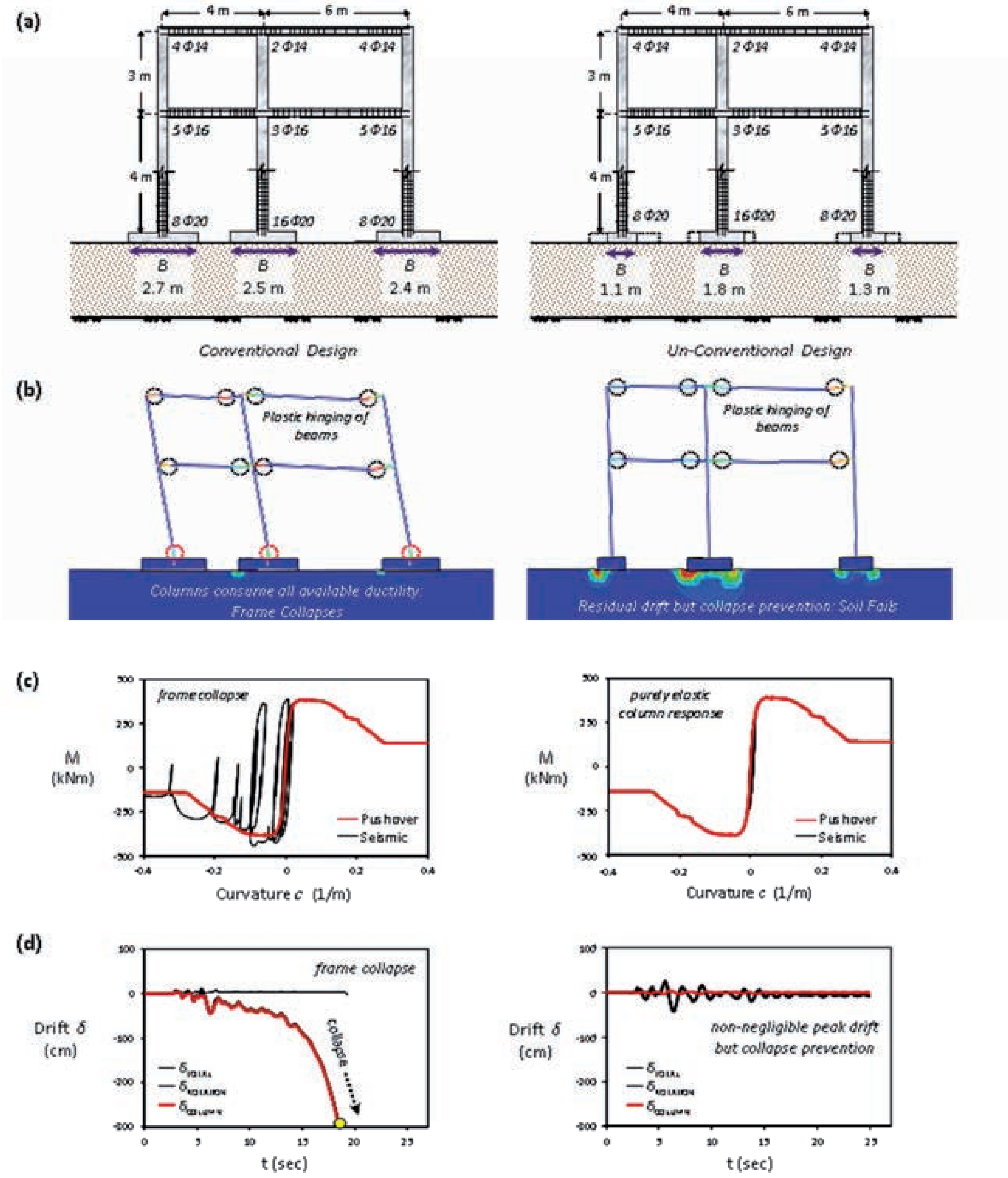


Figure 6. *(a)* Two building frames on two alternative foundation subjected to a large intensity earthquake, exceeding the design limits; *(b)* deformed mesh with superimposed plastic strain, showing the location of "plastic hinging" at ultimate state; *(c)* bending moment–curvature response of the central columns; *(d)* overturning moment–rotation (M–θ) response of the two central foundations.

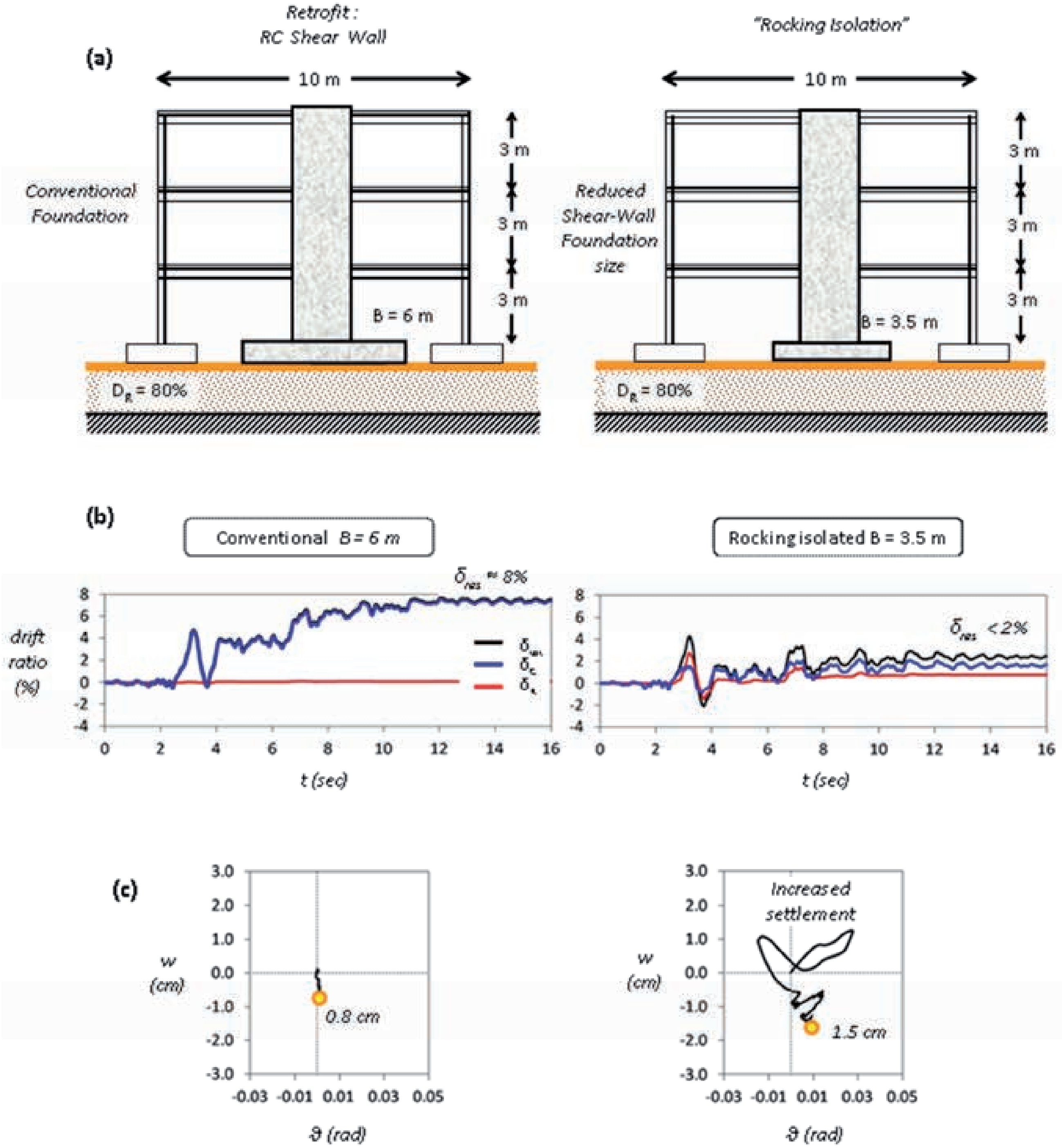


Figure 7. *(a)* Old frame retrofitted with stiff Shear Wall on two different foundations — conventional B = 6 m and unconventional B = 3.5 m; *(b)* time histories on top floor drift ratio; *(c)* settlement–rotation curves of the Shear Wall footings.

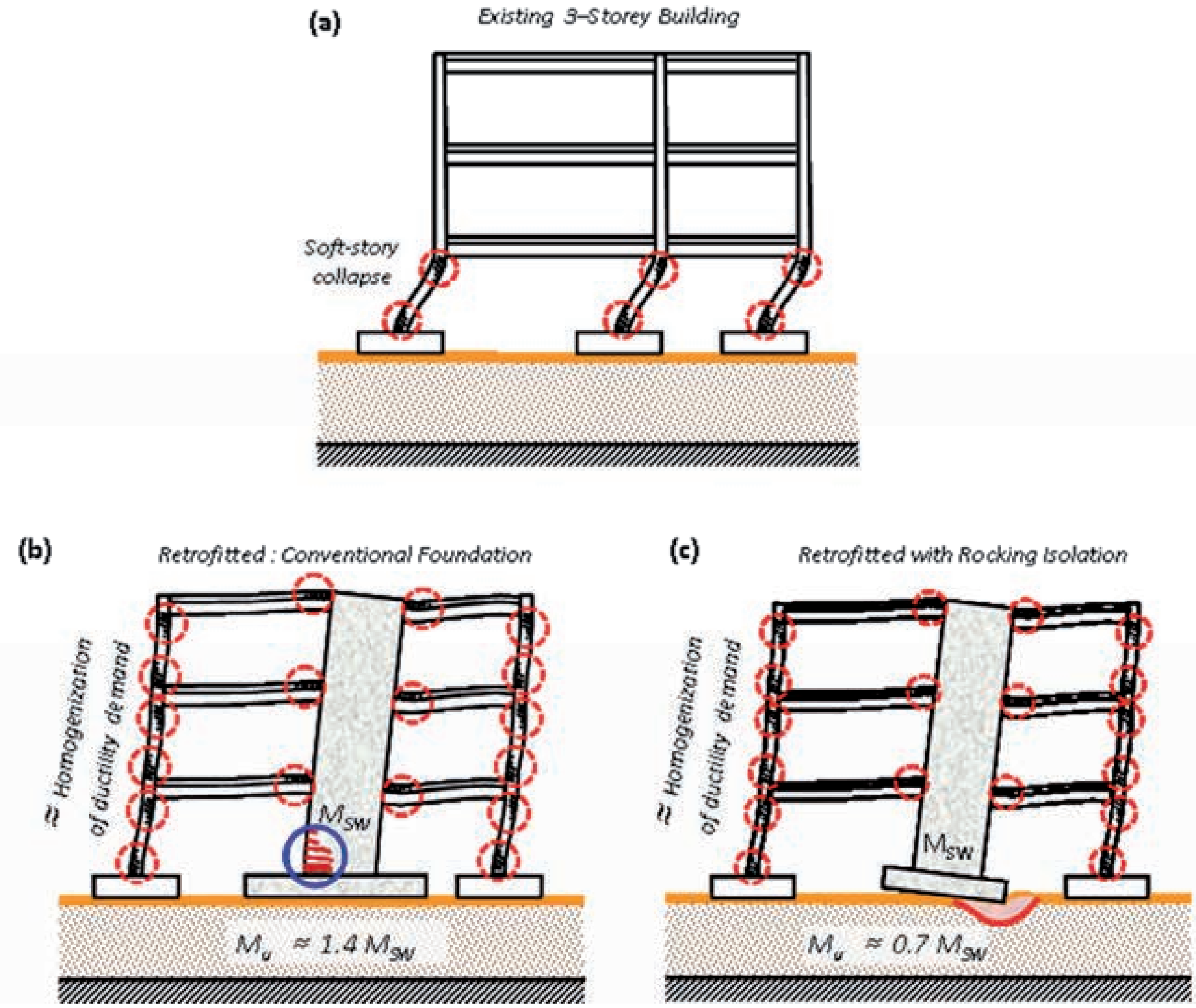


Figure 8. Sketches of damaged states of the three structures.