Soil Dynamics and Earthquake Engineering 68 (2015) 23–39

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn

4th Ishihara lecture: Soil–foundation–structure systems beyond conventional seismic failure thresholds

George Gazetas

Professor of Civil Engineering, National Technical University of Athens, Greece

ARTICLE INFO

Available online 18 October 2014

Keywords:
Performance-based design
Foundation rocking
Bearing capacity failure
Footing uplifting
Nonlinear vibrations
Capacity design
Rocking isolation

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 may not only 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, building frames, and simple frames retrofitted with a shear wall. It is shown that system collapse could be avoided even under seismic shaking far beyond the design ground motion. Three key phenomena are identified as the prime sources of the success; they are illustrated for a bridge–pier: (i) the constraining of the transmitted accelerations by the reduced ultimate moment capacity of the foundation, to levels of about one-half of those developing in a conventional design; (ii) the beneficial action of the static vertical load of the structure which pushes down to "re-center" the leaning (due to uplifting and soil yielding) footing, instead of further distressing the plastic hinge of the column of the conventional design; and (iii) the substantial increase of the fundamental natural period of the system as uplifting takes place, which brings the structure beyond the significant period range of a ground motion, and hence leads to the abatement of its severe shaking.

1. Current state of practice: the conventional "Wisdom"

Seismic design of structures recognizes 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 mobilization of "strength" in the foundation. In the words of EC8 (Part 2, Section 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;
- mobilization 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; and
- a combination of two or more of the above "failure" modes.

In this conventional approach to foundation design, "over-strength" factors plus (explicit and implicit) factors of safety larger

http://dx.doi.org/10.1016/j.soildyn.2014.09.012
0267-7261/

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: Meek [69], Pecker [82], Paolucci [76], Martin and Lam [66], FEMA-356 [26], Kutter et al. [61], Apostolou et al. [8]). 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 *few examples* are

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

With the correspondingly large accelerations in the (above-ground) structure from such ground motions (spectral S_a values well in excess of 1g), 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) [26] 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 and Lam [66]. 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 mobilization 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 [56], it appears that sliding at, and especially uplifting from, their base during oscillatory seismic motion was a key to their survival [50,63,52,42,91,58,99,65]. 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 larger than the static peak ground acceleration of a high-frequency motion may not be enough to overturn a slender block [58,66,7,42]. Simply stated: even severe uplifting (conventional “failure”) may not lead to overturning (true “collapse”) under dynamic seismic base excitation.

(d) The favorable effects of the fast-cyclic and kinematic nature of seismic shaking has long been recognized in connection with strongly inelastic systems, especially those controlled by friction. Exceedance of the frictional capacity does not (in most cases) lead to failure and may in fact cause only a small slippage [32,33,38]. The design of slopes and retaining walls is presently based on the very idea of allowing slippage to occur, by reducing the capacity to values of one-half of those “demanded” from the earthquake.

(e) 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 [16,17,25,86,87].

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.

(f) 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 localized 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 [10].
- The presence of soil confinement leads to increased plastic hinge lengths, thus preventing high localized curvatures [25,80,94]. 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. The potentially beneficial role of pile structural yielding has been shown by Gerolymos et al. [44].

Furthermore, when subjected to strong cyclic overturning moment, end-bearing piles in tension may 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.

(g) 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) [79,80,86,87,70]. *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: [3–6,13,20,22,23,28–31,33,38–41,47,48,51,55,59–63,70,74,75,77,79,81–85,88,92,93,98]. 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); and
(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. A note on theoretical and experimental methods

The results and conclusions of this paper are based on

- A finite-element formulation using simplified plasticity models for soils (developed in Refs. [43,44,4,5]) and interface elements capable of modeling uplifting and sliding.
- Small-scale shaking-table as well as centrifuge base-shaking experiments.

Simplified equivalent-linear and Winkler-type models have also been developed [1,7,34,43], the calibration–corroboration of which was based on the results of the more rigorous analyses.

It is also worthy of note that in the literature several other methods have developed by a number of researchers:

- finite-element formulations using a variety of soil constitutive relations [11,67,68,78];
- rigorous, plasticity-based macro-elements reproducing the soil reaction to an arbitrary footing motion [14,15,18,19,49];
- combined nonlinear springs and dashpots to represent the soil–foundation interface [9,30,43,7,84];
- large-scale 1*g* and centrifuge experiments [23,24,28,29,31,57,61,62,72,92,93]; and
- equivalent-linear approximations of foundation rocking stiffness and damping [27,79,84,85].

No further discussion on methods of analysis is presented in the paper.

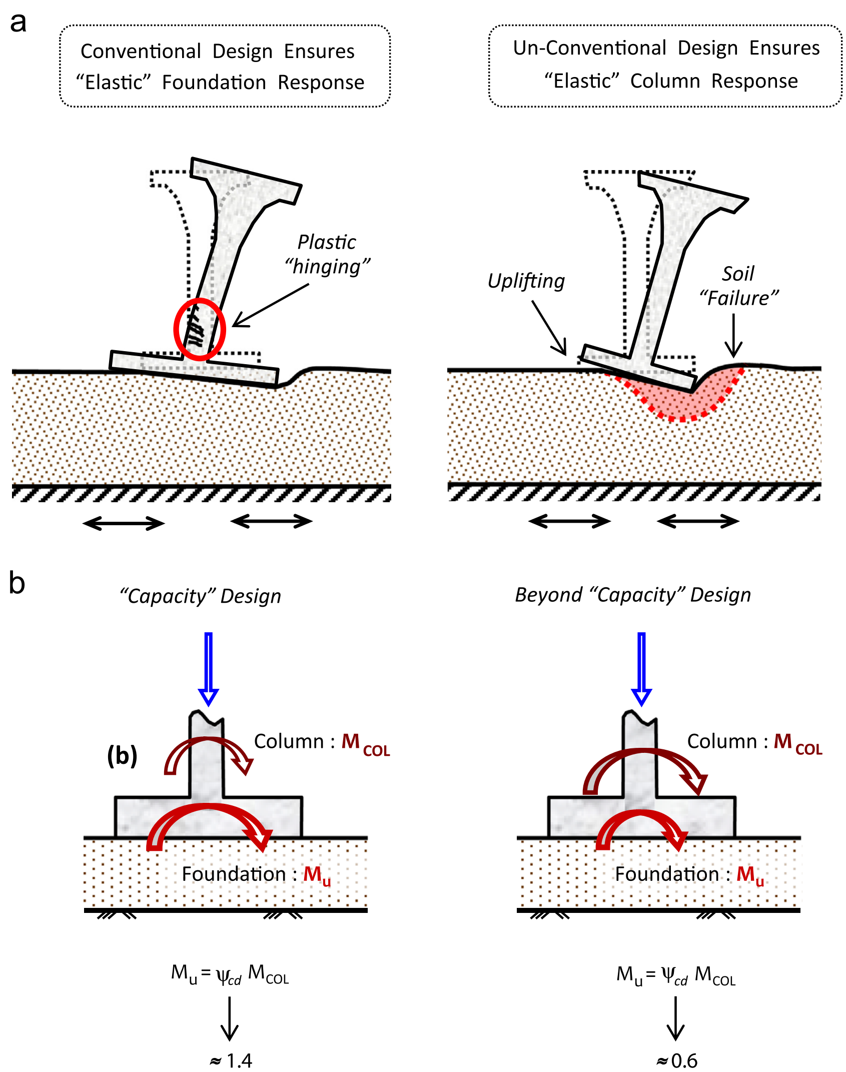


Fig. 1. Conceptual illustration of (a) the response of a conventional and a "rocking-isolation" design of a bridge–pier foundation and (b) the "capacity" design principle as conventionally applied to foundations, and its reversal in "rocking isolation".

5. 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 utilized to describe the dynamic action of the foundation system [21,36,37,53,64,84,89,96,97,35]. The various US seismic codes in the last 30+ years have promulgated an approximate method to deal with seismic soil–structure interaction [95].

The behavior of "rocking foundations" significantly deviates from linear visco-elasticity: uplifting introduces strong geometric non-linearity 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 force under purely vertical loading and N is the acting vertical force. Fig. 2 offers typical results for a homogeneous (G, s_u) soil for three F_s values: a very high one ($F_s=20$), a low one ($F_s=2$), and an extremely low one ($F_s=1.25$). M is the 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 "typically"-loaded foundation but 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 $F_s \to \infty$, i.e. if there is no vertical load on the foundation, M_u would vanish due to the tensionless nature of the soil–footing interface.
- As expected from the soil mechanics literature [71,41,2,45,46,90] the largest maximum moment is attained by the $F_s \approx 2$ footing:

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

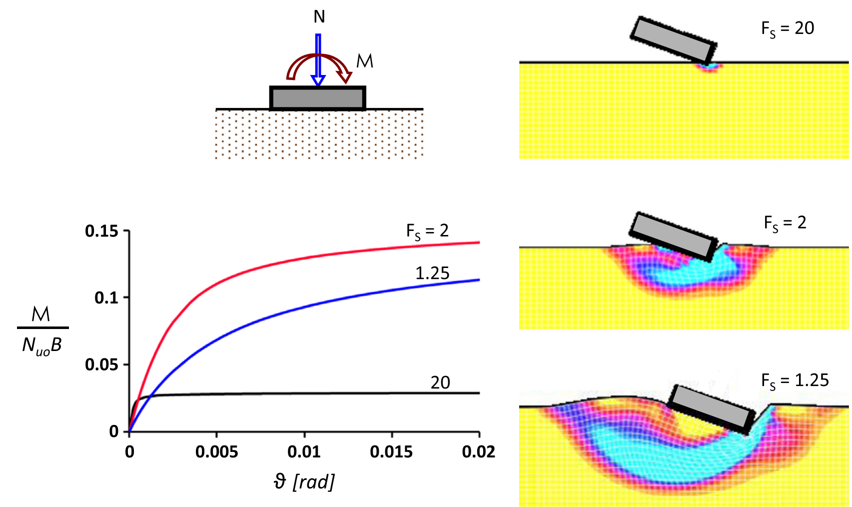


Fig. 2. Typical moment–rotation relations of three foundations and corresponding snapshots of their ultimate response with the contours of plastic deformation. The only difference between foundations : their static factor of safety.

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

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

It is interesting to portray the footing–soil failure under combined loading conditions in the currently popular form of failure envelopes (also called "interaction diagrams") [12,73,49,54]. Such a diagram in N–M space is given in Fig. 3 for the specific example. It was obtained for a variety of foundation shapes with the same numerical (finite-element) analysis as the curves and snapshots of Fig. 2, and can be approximated analytically (and conservatively) as a function of the static factor of safety (F_S) as

$$M_u \approx \frac{0.55}{F_S}\left(1 - \frac{1}{F_S}\right) \; N_{uo}B \tag{3}$$

This expression is very similar with the classical Meyerhof [71] bearing capacity equation under eccentric loading, and agrees with the numerical results of Gourvenec [46]. The specific plot is in terms of N/N_{uo} that 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 $1/F_S \approx 0.5$ 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. The largest value of M_u for $F_s \approx 2$ is

$$M_u \approx \; 0.138 \;\; N_{uo}B \tag{4a}$$

which is slightly different from the classical

$$M_u = 0.125 \; N_{uo}B \tag{4b}$$

of Meyerhof [71].

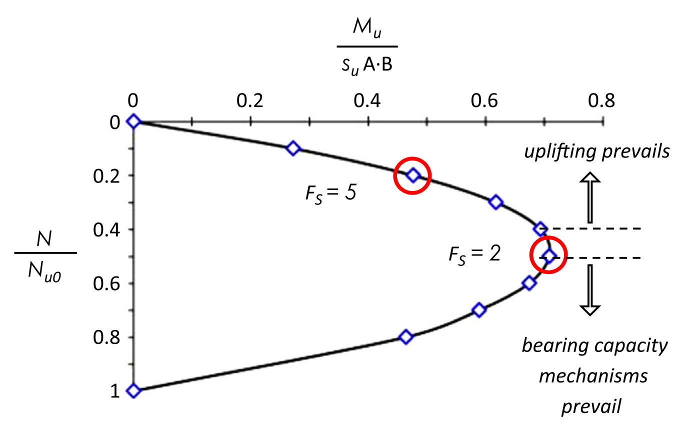


Fig. 3. Dimensionless N–M_u failure envelope for strip foundation (A: the area of the footing).

6. 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. Fig. 4(a) illustrates the forces and displacements at an ultimate pushover stage, and explains the meaning of the P–δ effect. The development (theoretical or experimental) of such pushover relationships has served as a key in simplified dynamic response analyses to estimate seismic deformation demands and 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 static 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. As an example, for a square footing having width $B = 3.3$ m, carrying a vertical force $N = 1500$ kN at a height $H = 6.6$ m, and founded on saturated clay with undrained shear strength $s_u = 110$ kPa, Fig. 4(b) shows the "FH versus ϑ" pushover curve along with the distribution of soil reactions developing at four instances of loading: (1) at initial static conditions; (2) at the limit of vanishing normal stress below the unloaded edge of the footing; (3) at the point (M_u, θ_u) of the

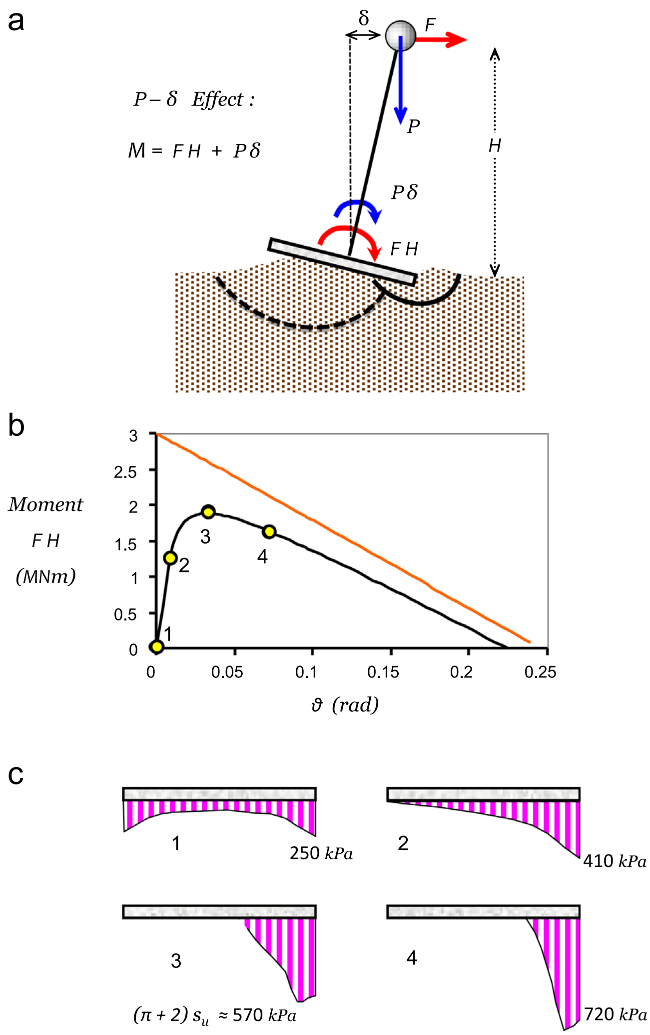


Fig. 4. (a) The effect of P–δ, (b) imposed external overturning moment versus angle of rotation of the footing and (c) distribution of soil reactions at four stages of loading.

maximum moment of the externally applied horizontal force; and (4) at an angle of rotation two times larger the angle θ_u. Unsurprisingly, the maximum soil reaction, σ_u, is about equal to $(\pi+2)s_u$ at the time of the maximum moment. Thereafter, as the footing continues to uplift and its contact surface to recede, the maximum soil reaction increases to 720 MPa at point "4" thanks to 3-D "confining" stress state under the loaded edge of the footing.

Further dimensionless results are shown in Fig. 5(a) and (b) for the same slender system as above but with two F_s values: 5 and 2.

The difference in the M–θ 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 θ 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–θ curve decreases with θ until the system topples at an angle θ_c. This critical angle for a rigid structure on a rigid base ($F_S = \infty$) is simply

$$\theta_{c,\infty} = \arctan\frac{b}{h} \tag{5a}$$

in which b is the foundation half-width. For very slender systems, the approximation

$$\theta_{c,\infty} \approx \frac{b}{h} \tag{5b}$$

is sufficient and worth remembering.

As the static vertical safety factor (F_S) diminishes, the rotation angle (θ_c) at the state of imminent collapse ("critical" overturning rotation) also slowly decreases. Indeed, for rocking on compliant soil, θ_c is always lower than it is on a rigid base (given with Eq. (5a)). For stiff elastic soil (or with a very large static vertical safety factor) θ_c is imperceptibly smaller than that given by Eq. (5a), 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 θ_c. However, such a reduction is small as long as the factor of safety (F_S) remains high (say, in excess of 3). Such behavior changes drastically with a very small F_S: then the soil responds in a strongly inelastic fashion, a symmetric bearing-capacity failure mechanism under the vertical load N is almost fully developed, replacing uplifting as the prevailing mechanism, and leading to collapse as θ_c decreases rapidly, tending to zero.

7. 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 θ_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 Fig. 5(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 behavior 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. [74].

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

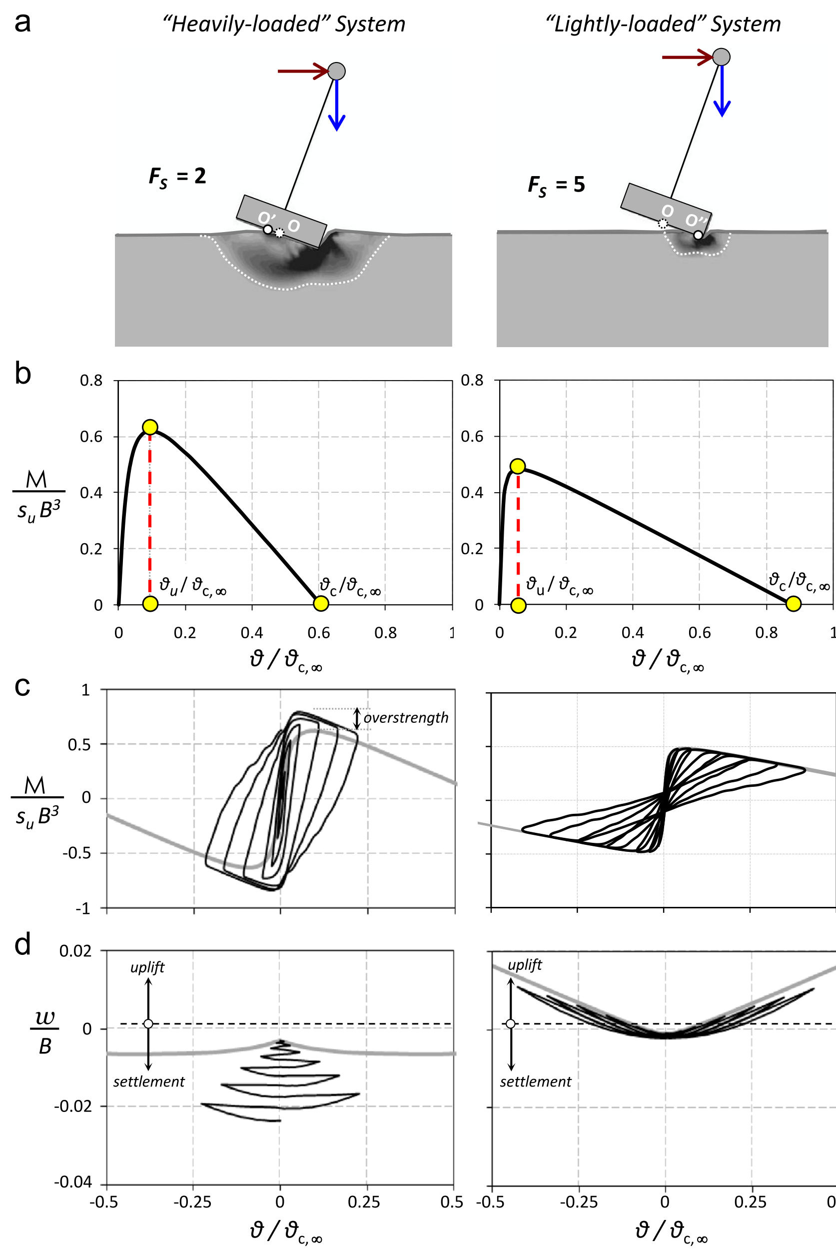


Fig. 5. Comparison of two slender systems (differing only in F_S) subjected to monotonic and cyclic loading: (a) deformed mesh with plastic strain contours at ultimate state; (b) dimensionless monotonic moment–rotation response; (c) cyclic moment–rotation response; and (d) cyclic settlement–rotation response (the gray line corresponds to the monotonic backbone curves).

Davis on sand and clay [62,28] large-scale tests conducted at the European Joint Research Center [24,72], and 1g shaking table tests in our laboratory at the National Technical University of Athens on sand [5,22].

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).
- Sufficiently large energy dissipation – quite 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 F_S values (i.e. with lightly loaded foundations or very stiff soils) – 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.

8. Seismic response of bridge–pier on shallow footing

The concept of "rocking isolation" is illustrated in Fig. 6 by examining the response of a 12 m tall bridge–pier carrying a deck of four lanes of traffic for a span of about 35 m – typical of elevated highways around the world.

The bridge chosen for analysis is similar to the Hanshin Expressway Fukae bridge, which collapsed spectacularly in the Kobe 1995 earthquake. The example bridge is designed in accordance to EC8-2000 for an effective acceleration $A=0.30g$, considering a (ductility-based) behavior factor $q=2$. With an elastic

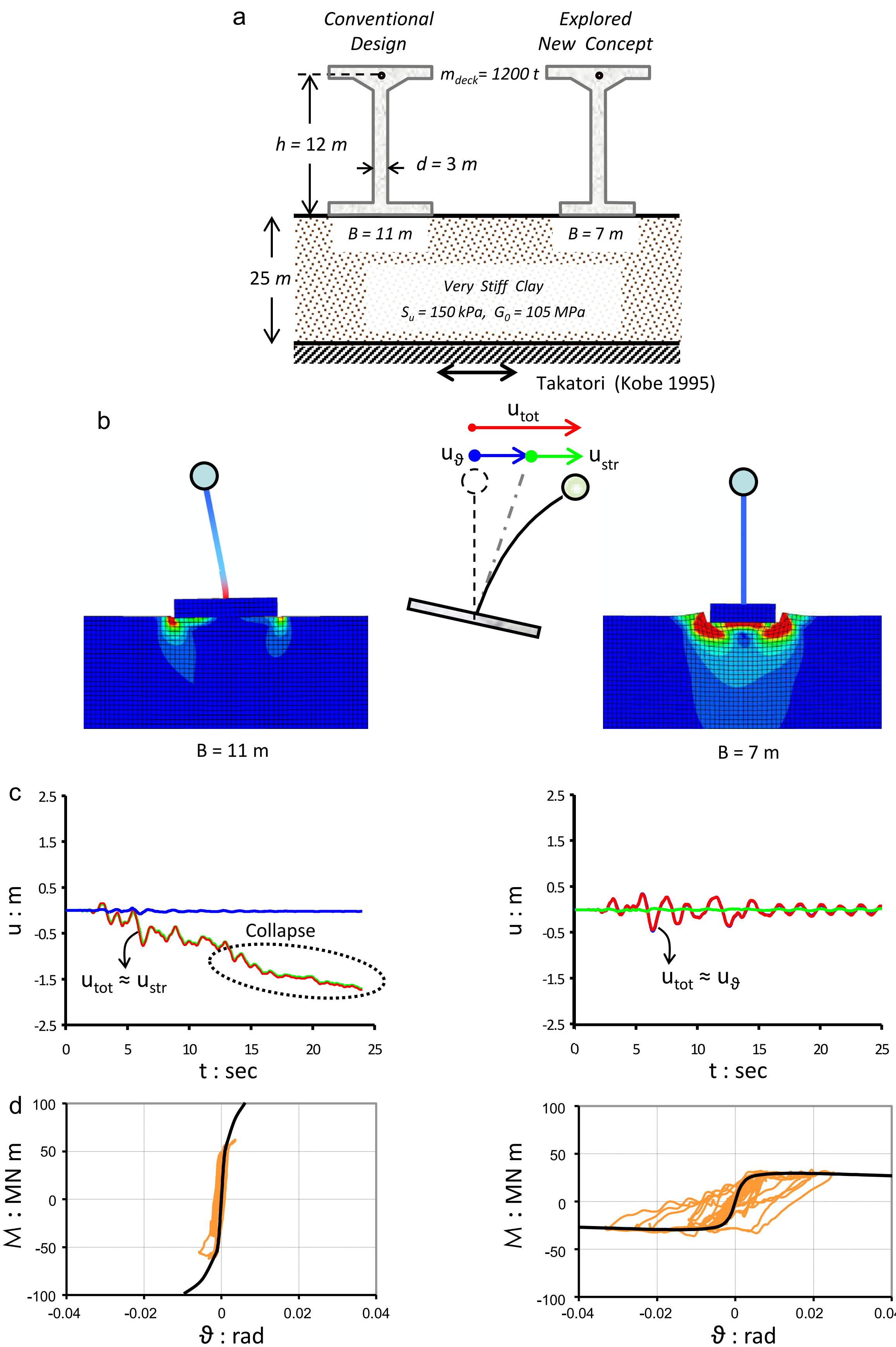


Fig. 6. (a) Two bridge piers on two alternative foundations subjected to a large intensity shaking, exceeding the design limits, (b) deformed mesh with superimposed plastic strain, showing the location of “plastic hinging” at ultimate state, (c) time histories of deck drift and (d) overturning moment–rotation (M–θ) response of the two foundations. (For interpretation of the references to color in this figure, the reader is referred to the web version of this article.)

(fixed-base) vibration period $T = 0.48$ s the resulting design bending moment $M_{COL} \approx 45$ MN m.

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

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

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

Fig. 6 compares the response of the two alternatives, in terms of deformed mesh at the end of shaking with superimposed the plastic strains. In the conventionally designed system there is very little inelastic action in the soil; the red regions of large plastic deformation are seen only under the severely "battered" edges of the rocking foundation – but without extending below the foundation. "Plastic hinging" forms at the base of the pier, leading to a rather intense accumulation of curvature (deformation scale factor = 2).The $P-\delta$ effect of the mass will further aggravate the plastic deformation of the column, leading to collapse.

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

The above observations are further confirmed by the time history of deck drift shown in Fig. 6(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. 6(d) further elucidates the action of the foundation–soil system. The M–θ relationship shows for the 11×11 m^2 foundation a nearly linear viscoelastic response, well below its ultimate capacity and apparently with no uplifting. On the contrary, the 7×7 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, as will be discussed in the sequel.

9. 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 $A = 0.36g$ 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 Fig. 7.

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. Applying the methodology which has been outlined in Gelagoti et al. [39,40], 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

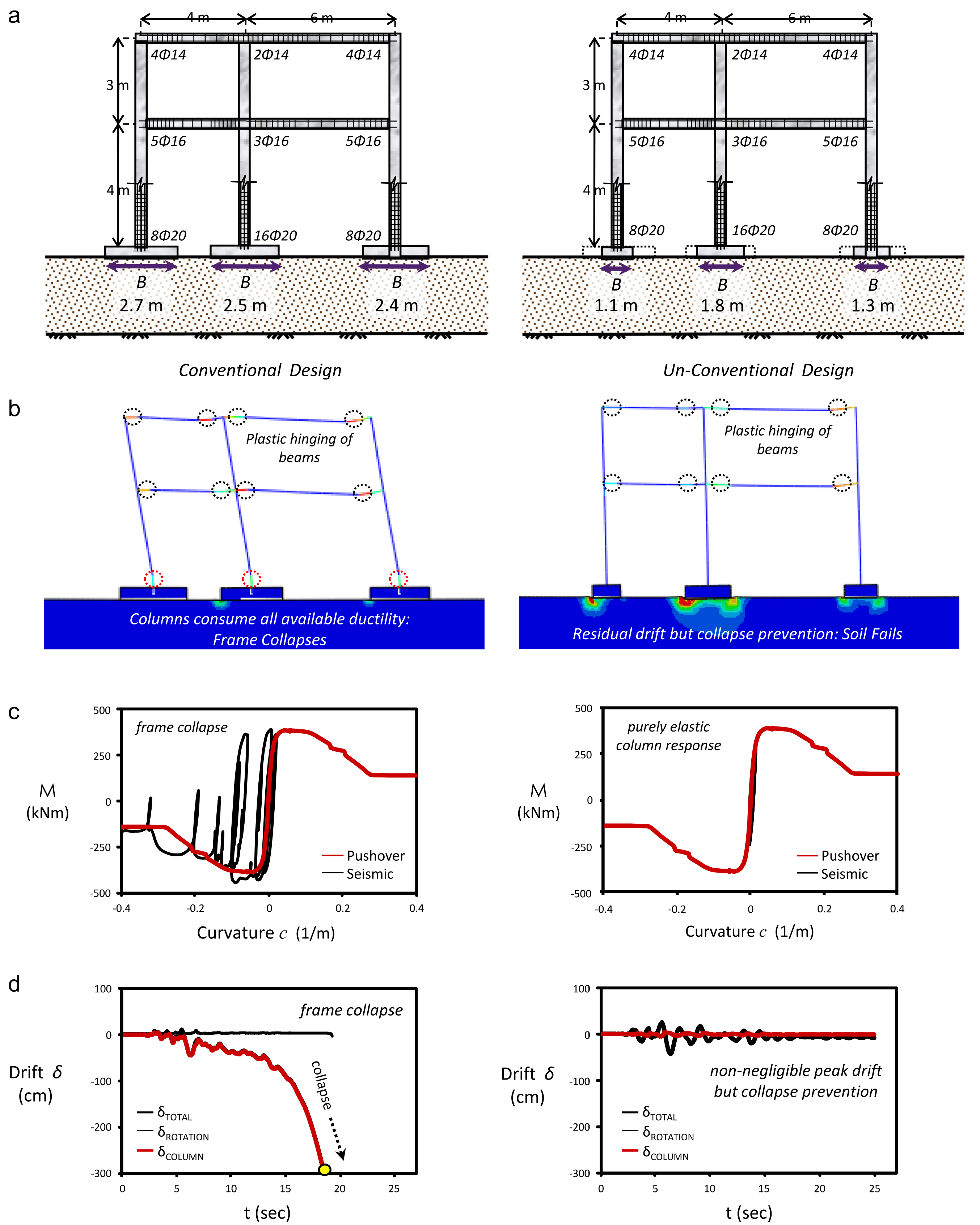


Fig. 7. (a) Two building frames on two alternative foundation subjected to a large intensity earthquake, exceeding the design limits, (b) deformed mesh with superimposed plastic strain, showing the location of "plastic hinging" at ultimate state, (c) bending moment–curvature response of the central columns and (d) overturning moment–rotation (M–θ) response of the two central foundations. (For interpretation of the references to color in this figure, the reader is referred to the web version of this article.)

Table 1
Footing dimensions and corresponding factors of safety (computed following the provisions of EC8) against vertical loading for the seismic load combination ($G+0.3Q$) for the two design alternatives of Fig. 7.

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

design. Footing dimensions and static factors of safety against vertical loading of the two designs are summarized in Table 1.

The performance of the two design alternatives is compared in Fig. 7. The deformed mesh with superimposed plastic strain contours of the two alternatives is portrayed on the figure. 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–δ 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. 7(b)). 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. 7(d)).

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. 7(d) 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 [60]. In other words, while the three footings sustain almost the same dimensionless settlement w/B, which is roughly equal to 0.025 ($\approx$ 3 cm/1.2 m) for the two side footings and 0.033 ($\approx$ 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.

10. 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 25×25 cm^2 and beams 25×50 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 $\times$ 0.3 m in cross-section is constructed replacing the middle column, as shown in Fig. 8.

The 1:10-scale model is supported on dense fine-grained $D_r\approx80\%$ sand. The original footings of all three columns were 1.5 m^2. For the retrofitted frame the two columns retained their original 1.5×1.5 m^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 6.0 m $\times$ 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, $F_S\cong10$, 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^2).

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%.

Fig. 9 sketches the deformation pattern of the three systems while Fig. 8 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.

11. Why can we trust the performance of rocking foundations?

In the preceding sections we have mentioned or implied some of the deeper causes behind the observed superiority of the structure–foundation systems designed to experience "plastic hinging" below ground. We summarize herein four interrelated phenomena that explain the successful performance produced by an under-designed footing, and may convince on its reliability.

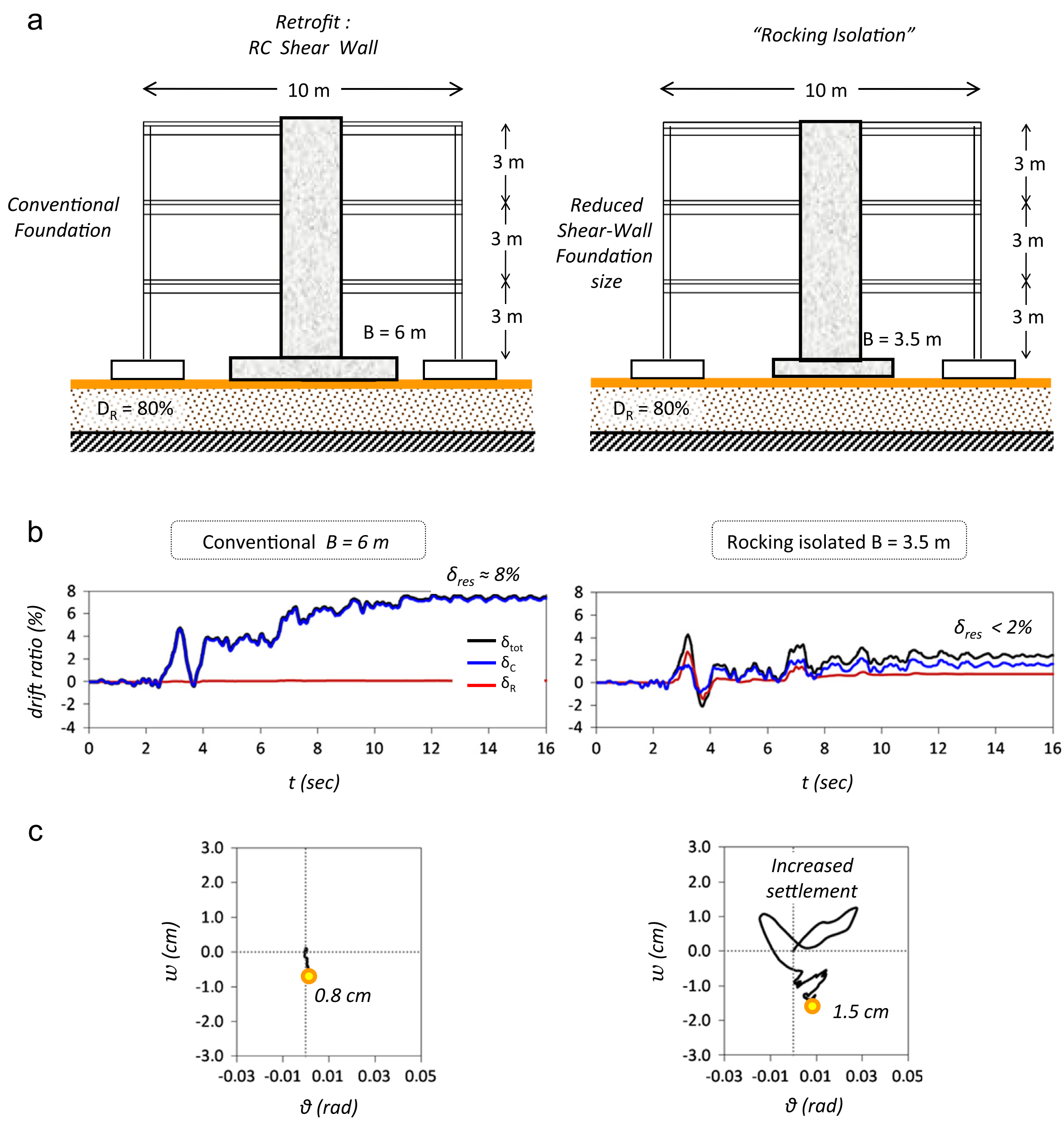


Fig. 8. (a) Old frame retrofitted with stiff shear wall on two different foundations – conventional $B=6$ m and unconventional $B=3.5$ m, (b) time histories of top floor drift ratio and (c) settlement–rotation curves of the shear wall footings.

11.1. Reduction of the levels of acceleration transmitted on the structure

This stems from the kinematic nature of seismic shaking: accelerations and associated inertial forces are not predetermined constants, but are variables limited by the "strength" of the supports; thus, by limiting their "strength" we reduce the accelerations. And this is the case with both the conventional "capacity" design and the "daring" foundation design studied in this paper. But there is a difference, explained with the sketches of Fig. 10.

For a bridge–pier foundation designed conventionally the largest acceleration, $A_{1,max}$, that can be transmitted on the top mass (computed with a few mildly-simplifying assumptions) is

$$A_{1,\,max} \approx \frac{M_{col}}{mh} \tag{6}$$

where M_{col} is the moment capacity at the base of the column (pier) and m is the superstructure mass. With an unconventional foundation design M_{col} is replaced by the foundation capacity $M_{found} \approx 0.6M_{col}$ and hence

$$A_{2,\,max} \approx \frac{(0.6)M_{col}}{m(h+d)} \approx 0.5A_{1,\,max} \tag{7}$$

if we take $h+d \approx 1.2h$. This reduction by a factor of about 2 is a key ingredient of the success of the new design concept.

Fig. 11 demonstrates the (approximate) validity of Eq. (7) with the compiled results of many analyses. Notice that when the peak base excitation $A_{R,max}$ is a very weak motion, say with peak ground acceleration of about 0.1g or less, both $A_{1,max}$ and $A_{2,max}$ increase linearly with $A_{R,max}$. The two designs experience rather similar peak accelerations at the top, as their response is almost linear and ultimate resistance (in the column or in the ground, respectively) is far from being mobilized. Their small differences arise from the different compliance of the foundation: the smaller dynamic stiffnesses of the unconventional (small) foundation make that system more flexible; its rotational oscillations and the ensuing increase in fundamental period and damping lead to slightly smaller top acceleration on the average (for the particular system examined, not in all possible cases).

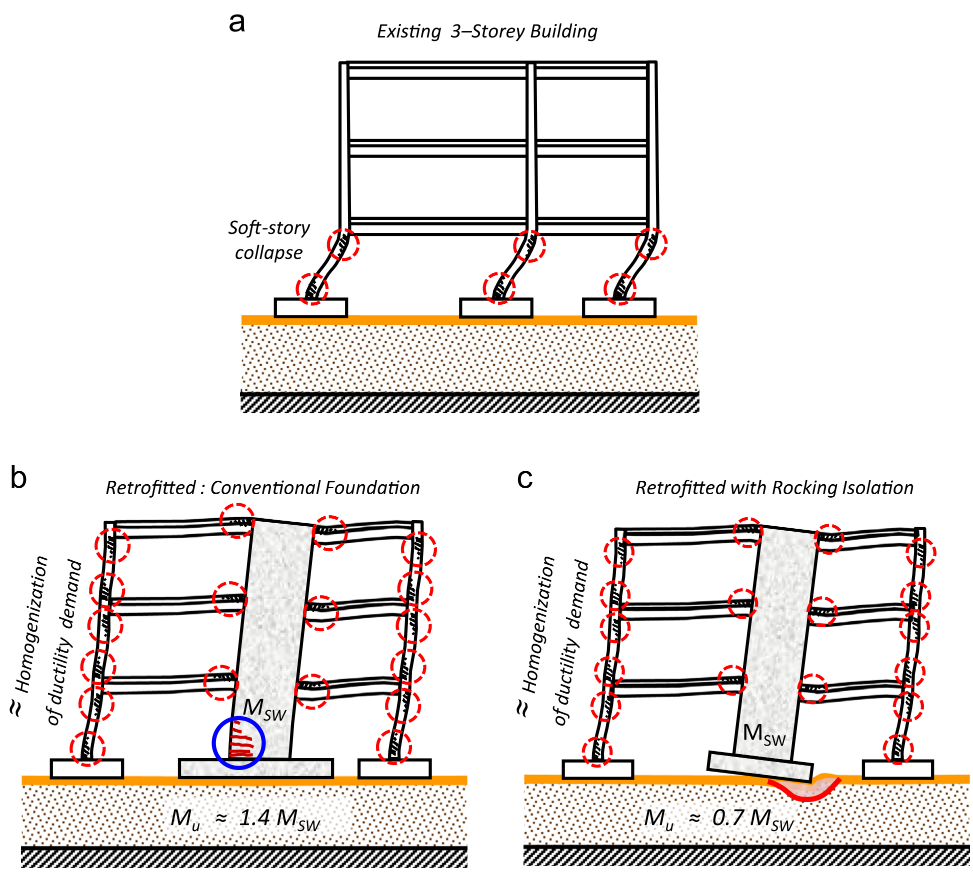


Fig. 9. Sketches of damaged states of the three structures, revealing a rather unexpected beneficial role of a rocking shear wall.

But as the excitation becomes stronger the ultimate moment capacity at the base of the column or at the soil–footing interface is being fully mobilized. The accelerations $A_{1,max}$ and $A_{2,max}$ stop growing linearly with the excitation intensity $A_{R,max}$. In fact $A_{2,max}$ barely changes when $A_{R,max}$ increases from 0.40g to 1.20g: $A_{2,max} \approx 0.20g$. Overall, in this phase of large excitation, the simplified relation

$$A_{2,\ max} \approx \frac{1}{2} A_{1,\ max} \tag{8}$$

is a good approximation indeed.

11.2. *The role of P–δ effects at the end of shaking acts as a recentering mechanism*

Indeed, for an uplifted foundation, Fig. 12(a) illustrates the beneficial effect of the weight which tends to bring the system closer to the vertical position; hence $\vartheta_{residual}$ is unusually small. By contrast, as sketched in Fig. 12(b), the conventional design with its unyielding foundation will tend to suffer a detrimental bending moment, $\Delta M = P\delta$, that may lead to exceedance of the ductility capacity of the column's "plastic hinge"; hence collapse is a possibility with large foundations.

11.3. *The effective natural period T_n of the unconventionally founded structure grows significantly with the amplitude of rotational uplifting*

An extreme example is the rocking of a rigid block on a rigid oscillating base whose fundamental period increases from 0 (at small base accelerations not exceeding the critical acceleration $A_c = [b/h]g$, where b, h = semi-width and semi-height of the block) to ∞ (at the largest possible angle of rotation before toppling: $\vartheta_c = \tan^{-1}[b/h]$). Fig. 13 presents a more realistic $T_n = T_n(\vartheta)$ parametric plot, pertaining to the shown system of a rigid superstructure with $h/b = 4$ founded on stiff soil. Notice that regardless of the initial static factor of safety F_s, even angles of rotation of about 0.02 rad may lead to doubling of T_n compared to the elastic one, $T_n(0)$.

More significant is the fact that when "*things for the system get tough*" and ϑ increases to values of about 1 rad, T_n grows to values of at least 3–4 s, which would very-very rarely "attract" any additional seismic excitation.

The combined effect of all the above, i.e. the reduced acceleration levels, the recentering role of the axial force, and the additional likely decrease in acceleration due to enlargement of the fundamental period, is the origin of the excellent performance of the "daring" foundation system.

11.4. *The reliability of the new design method has been demonstrated in numerous experiments*

(shaking tables in 1g and Ng centrifuge testing). Moreover, with no exception, all tests on sand have shown that the foundation moment–rotation loops M:ϑ are quite stable at least for a few number of cycles appropriate for seismic shaking. The plots of Fig. 5(c) were obtained numerically, but as the reader can check in the cited literature, the laboratory experiments are invariably quite similar. This should offer a substantial peace of mind with the method when adopted in practice.

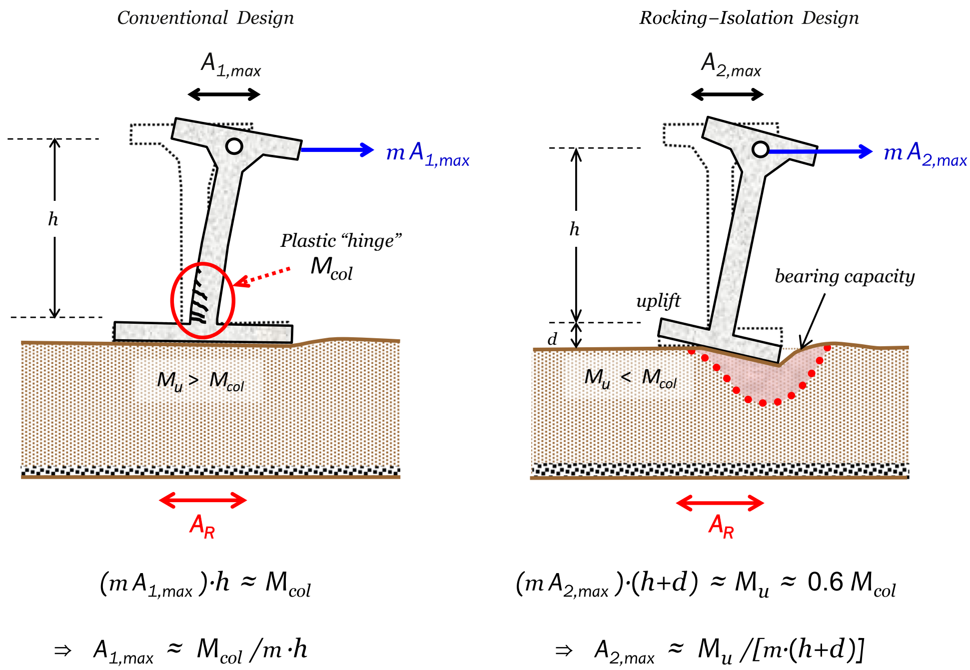


Fig. 10. The peak accelerations on the top of the bridge–pier are essentially independent of maximum A_R and depend only on the lowest moment capacity at the base, i.e. the column ultimate moment (left) or the foundation capacity (right), as long as the intensity of motion is enough for yielding to be reached.

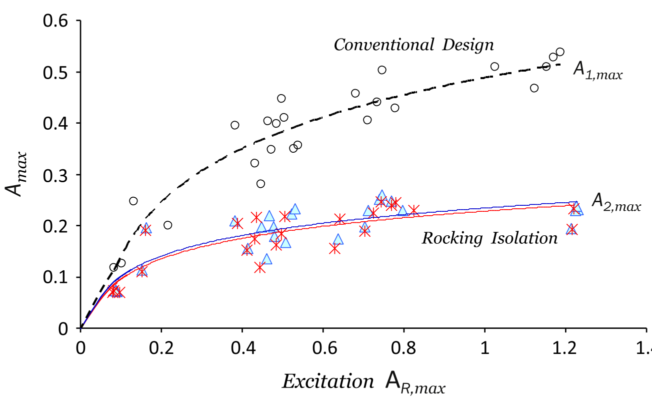


Fig. 11. Compiled numerical results show that the rocking isolation design leads to reduction of peak top accelerations to about one-half of those developing in a "capacity" based design, for $A_{R,max} > 0.3g$.

11.5. Drawbacks

The method is certainly not a panacea. It is not appropriate for all structures and all soils. The major limitation: the maximum and residual settlement of the rocking foundation may be excessive for a particular system. To ensure that such settlements are small, one may choose to have foundations, with F_s values > 5. Techniques to ameliorate the consequences of settlement or to minimize them are quite possible. Some have been introduced in the recent article by Anastasopoulos et al. [3].

12. Conclusions

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

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

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

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

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

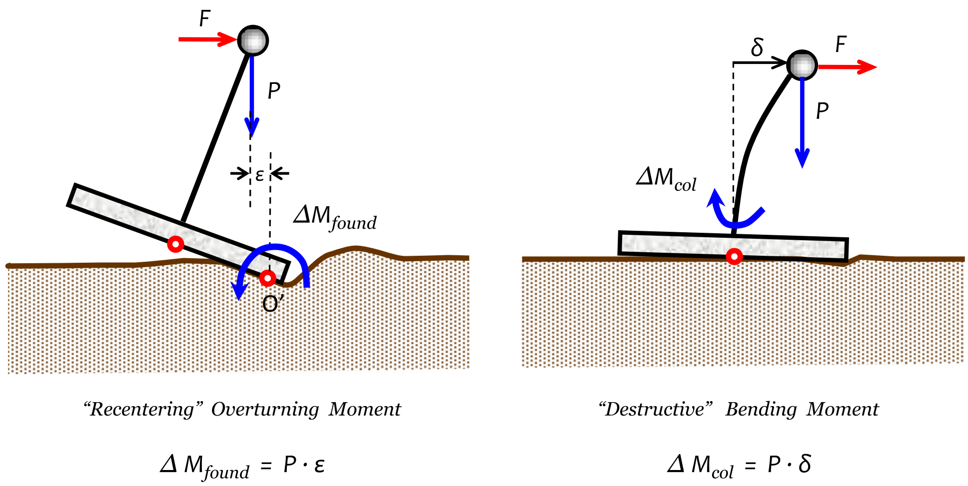
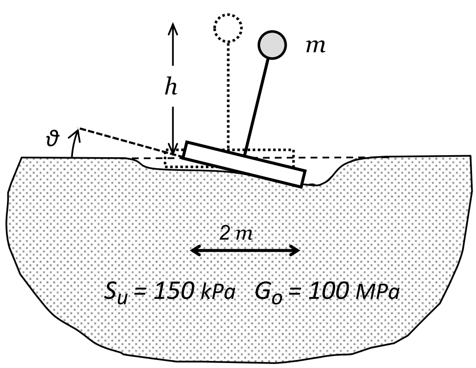


Fig. 12. The different effect of the vertical static force: in recentering the rocking isolated structure (left) and further distressing the column plastic hinge of the capacity designed structure (right).



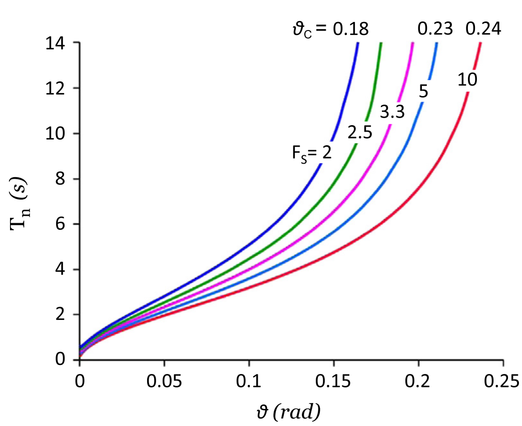


Fig. 13. Increase of the fundamental period of a simple rocking oscillator as the angle of rotation increases due to uplifting and soil yielding.

The results and conclusions of this paper are in harmony with numerous experimental and theoretical findings of Professor Bruce Kutter and his coworkers at U.C. Davis, as well as the theoretical work of Alain Pecker, Roberto Paolucci, and several others cited in the paper.

Acknowledgments

The author would like to pay homage to Professor Kenji Ishihara, whose pioneering research and publications in earthquake geotechnical engineering during the last 40 years have been a source of inspiration for engineers and researchers in soil dynamics.

Financial support for the work outlined in this paper has been provided through the research project "DARE", funded by the European Research Council (ERC), "IDEAS" Programme in Support of Frontier Research. Contract/number ERC–2–9–AdG228254–DARE.

The research outlined in the paper has been a collective work with my co-workers: I. Anastasopoulos, N. Gerolymos, V. Drosos, F. Gelagoti, E. Garini, R. Kourkoulis, M. Loli, P. Tasiopoulou, A. Tsatsis, A. Zafeirakos, as well as with several of my former diploma students. I am indebted to all of them.

References

[1] Adamidis O, Gazetas G, Anastasopoulos I. Equivalent-linear stiffness and damping in rocking of circular and strip foundations. Bull Earthq Eng 2014;12:1177–200. http://dx.doi.org/10.1007/s10518-013-9554-0.
[2] Allotey N, El Naggar MH. Analytical moment–rotation curves for rigid foundations based on a Winkler model. Soil Dyn Earthq Eng 2003;23:367–81.
[3] Anastasopoulos I, Gelagoti F, Spyridaki A, Sideri TZ, Gazetas G. Seismic rocking isolation of asymmetric frame on spread footings. J Geotech Geoenviron Eng 2014;140:133–51.
[4] Anastasopoulos I, Gazetas G, Loli M, Apostolou M, Gerolymos N. Soil failure can be used for seismic protection of structures. Bull Earthq Eng 2010;8:309–26.
[5] Anastasopoulos I, Gelagoti F, Kourkoulis R, Gazetas G. Simplified constitutive model for simulation of cyclic response of shallow foundations: validation against laboratory tests. J Geotech Geoenviron Eng 2011;137:1154–68.
[6] Anastasopoulos I. Beyond conventional capacity design: towards a new design philosophy. In: Orense RP, Chouw N, Pender MJ, editors. Soil–foundation–structure interaction. New York: CRC Press, Taylor & Francis Group; 2010. p. 213–20.
[7] Anastasopoulos I, Kontoroupi TH. Simplified approximate method for analysis of rocking systems accounting for soil Inelasticity and foundation uplifting. Soil Dyn Earthq Eng 2014;56:28–43.
[8] Apostolou M, Gazetas G, Garini E. Seismic response of slender rigid structures with foundation uplifting. Soil Dyn Earthq Eng 2007;27:642–54.
[9] Bartlett PE. Foundation rocking on a clay soil. (ME thesis, report no. 154). New Zealand: School of Engineering, University of Auckland; 1976.
[10] Budek AM, Priestley MJN, Benzoni G. Inelastic seismic response of bridge drilled-shaft RC pile/columns. J Geotech Geoenviron Eng 2000;126:510–7.
[11] Borja RI, Wu WH, Amies AP, Smith HA. Nonlinear lateral, rocking, and torsional vibration of rigid foundations. J Geotech Eng ASCE 1994;120(3):491–513.
[12] Butterfield R, Gottardi G. A complete three-dimensional failure envelope for shallow footings on sand. Géotechnique 1994;44(1):181–4.
[13] Chang BJ, Raychowdhury P, Hutchinson T, Thomas J, Gajan S, Kutter BL. Centrifuge testing of combined frame–wall–foundation structural systems. In: Proceedings of the 8th US national conference on earthquake engineering; April 18–22 2006: paper no. 998.
[14] Chatzigogos CT, Pecker A, Salençon J. Macroelement modeling of shallow foundations. Soil Dyn Earthq Eng 2009;29(5):765–81.

[15] Chatzigogos CT, Figini R, Pecker A, Salencon J. A macroelement formulation for shallow foundations on cohesive and frictional soils. Int J Numer Anal Methods Geomech 2010;35(8):902–31.
[16] Chen XC, Lai YM. Seismic response of bridge piers on elastic-plastic Winkler foundation allowed to uplift. J Sound Vib 2003;266:957–65.
[17] Chopra AK, Yim CS. Earthquake response of structures with partial uplift on Winkler foundation. Earthq Eng Struct Dyn 1984;12:263–81.
[18] Crémer C, Pecker A, Davenne L. Cyclic macro-element for soil–structure interaction: material and geometrical nonlinearities. Int J Numer Anal Methods Geomech 2001;25(12):1257–84.
[19] Cremer C, Pecker A, Davenne L. Modeling of nonlinear dynamic behaviour of a shallow strip foundation with macro-element. J Earthq Eng 2002;6:175–211.
[20] Deng L, Kutter BL. Characterization of rocking shallow foundations using centrifuge model tests. Earthq Eng Struct Dyn 2012;41(5):1043–60.
[21] Dobry R, Gazetas G. Dynamic response of arbitrarily-shaped foundations. J Geotech Eng 1986;113:109–35.
[22] Drosos V, Georgarakos P, Loli M, Zarzouras O, Anastasopoulos I, Gazetas G. Soil–foundation–structure interaction with mobilization of bearing capacity: an experimental study of sand. J Geotech Geoenviron Eng 2012;138:1369–86.
[23] Faccioli E, Paolucci R, Vanini M. 3D site effects and soil–foundation interaction in earthquake and vibration risk evaluation. Final report of the European research project TRISEE. Brussels: European Commission; 1998.
[24] Faccioli E, Paolucci R, Vivero G. Investigation of seismic soil–footing interaction by large scale cyclic tests and analytical models. In: Proceedings of the 4th international conference on recent advances in geotechnical earthquake engineering and soil dynamics. CD-ROM, S. Prakash Foundation Publisher, University of Missouri-Rolla; 2001.
[25] Advances in performance-based earthquake engineering. In: Fardis MN, editor. Greece: Springer, University of Patras; 2010. p. 485.
[26] FEMA-356. Prestandard and commentary for the seismic rehabilitation of buildings. Washington, DC: Federal Emergency Management Agency; 2000.
[27] Figini R. Nonlinear dynamic soil–structure interaction: application to seismic analysis of structures on shallow foundations. (Ph.D. thesis). Italy: Politecnico di Milano; 2010.
[28] Gajan S, Phalen JD, Kutter BL, Hatchinson TC, Martin G. Centrifuge modeling of load–deformation behavior of rocking shallow foundations. Soil Dyn Earthq Eng 2005;25:773–83.
[29] Gajan S, Kutter BL. Capacity, settlement and energy dissipation of shallow footings subjected to rocking. J Geotech Geoenviron Eng 2008;134:1129–41.
[30] Gajan S, Kutter BL. Contact interface model for shallow foundations subjected to combined loading. J Geotech Geoenviron Eng 2009;135:407–19.
[31] Gajan S, Kutter BL. Effects of moment-to-shear ratio on combined cyclic load–displacement behavior of shallow foundations from centrifuge experiments. J Geotech Geoenviron Eng 2009;135:1044–55.
[32] Garini E, Gazetas G, Anastasopoulos I. Asymmetric 'Newmark' sliding caused by motions containing severe 'Directivity' and 'Fling' pulses. Géotechnique 2011;61:753–6.
[33] Gazetas G, Anastasopoulos I, Garini E. Geotechnical design with apparent seismic safety factors well-bellow 1. Soil Dyn Earthq Eng 2014;57:37–45.
[34] Gazetas G, Anastasopoulos I, Adamidis O, Kontoroupi Th. Nonlinear rocking stiffness of foundations. Soil Dyn Earthq Eng 2013;47:83–91.
[35] Gazetas G, Mylonakis G. Seismic soil–structure interaction: new evidence and emerging issues. In: Soil dynamics III, ASCE, specialty geotechnical conference, vol. 2. Seattle; 1998. p. 1119–74.
[36] Gazetas G. Simple physical methods for foundation impedances. Chapter 2. In: Benerjee PK, Butterfield R, editors. Dynamics of foundations and buried structures. Barking Essex, UK: Elsevier Applied Science; 1987. p. 44–90.
[37] Gazetas G. Formulas and charts for impedances of surface and embedded foundation. J Geotech Eng 1991;117:1363–81.
[38] Gazetas G, Garini E, Anastasopoulos I, Georgarakos P. Effects of near-fault ground shaking on sliding systems. J Geotech Geoenviron Eng 2009;135:1906–21.
[39] Gelagoti F, Kourkoulis R, Anastasopoulos I, Gazetas G. Rocking-isolated frame structures: margins of safety against toppling collapse and simplified design approach. Soil Dyn Earthq Eng 2012;32(1):87–102.
[40] Gelagoti F, Kourkoulis R, Anastasopoulos I, Gazetas G. Rocking isolation of low-rise frame structures founded on isolated footings. Earthq Eng Struct Dyn 2012;41:1177–97.
[41] Georgiadis M, Butterfield R. Displacements of footings on sands under eccentric and inclined loading. Can Geotech J 1988;25:199–212.
[42] Gerolymos N, Apostolou M, Gazetas G. Neural network analysis of overturning response under near-fault type excitation. Earthq Eng Eng Vib 2005;4:213–28.
[43] Gerolymos N, Gazetas G. Development of Winkler model for static and dynamic response of Caisson foundations with soil and interface nonlinearities. Soil Dyn Earthq Eng 2006;26:363–76.
[44] Gerolymos N, Drosos V, Gazetas G. Seismic response of single-column bent on pile:evidence of beneficial role of pile and soil inelasticity. Bull Earthq Eng 2009;7:547–73.
[45] Gottardi G, Houlsby GT, Butterfield R. The displacement of a model rigid surface footing on dense sand under general planar loading. Soils Found 1995;35:71–82.
[46] Gourvenec S. Shape effects on the capacity of rectangular footings under general loading. Géotechnique 2007;57:637–46.
[47] Harden C, Hutchinson TC. Investigation into the Effects of Foundation Uplift on Simplified Seismic Design Procedures. Earthq Spectra 2006;22:663–92.
[48] Harden CW, Hutchinson TC. Beam on nonlinear Winkler foundation modeling of shallow rocking-dominated footings. Earthq Spectra 2009;25:277–300.
[49] Houlsby GT, Cassidy MJ, Einav I. A generalized Winkler model for the behavior of shallow foundation. Geotechnique 2005;55:449–60.
[50] Housner GW. The behavior of inverted pendulum structures during earthquakes. Bull Seismol Soc Am 1963;53:403–17.
[51] Huckelbridge AA, Clough R. Seismic response of uplifting building frame. J Struct Eng 1978;104:1211–29.
[52] Ishiyama Y. Motions of rigid bodies and criteria for overturning by earthquake excitations. Earthq Eng Struct Dyn 1982;10:635–50.
[53] Kausel E, Roesset JM. Dynamic stiffness of circular foundations. J Eng Mech Div 1975;101:771–85.
[54] Karapiperis K, Gerolymos N. Combined loading of Caisson foundation in cohesive: finite element versus Winkler modeling. Comput Geotech 2014;56:100–20.
[55] Kawashima K, Nagai T, Sakellaraki D. Rocking seismic isolation of bridges supported by spread foundations. In: Proceedings of the 2nd Japan–Greece workshop on seismic design, observation, and retrofit of foundations. Tokyo: Japanese Society of Civil Engineers; 2007. p. 254–65.
[56] Kirkpatrick P. Seismic measurements by the overthrow of columns. Bull Seismol Soc Am 1927;17:95–109.
[57] Knappett JA, Haigh SK, Madabhushi SPG. Mechanisms of failure for shallow foundations under earthquake loading. Soil Dyn Earthq Eng 2006;26:91–102.
[58] Koh AS, Spanos P, Roesset JM. Harmonic rocking of rigid block on flexible foundation. J Eng Mech 1986;112:1165–80.
[59] Kourkoulis R, Gelagoti F, Anastasopoulos I. Rocking isolation of frames on isolated footings: design insights and limitations. J Earthq Eng 2012;16 (3):374–400.
[60] Kourkoulis R, Anastasopoulos I, Gelagoti F, Kokkali P. Dimensional analysis of SDOF systems rocking on inelastic soil. J Earthq Eng 2012;16(7):995–1022.
[61] Kutter BL, Martin G, Hutchinson TC, Harden C, Gajan S, Phalen JD. Status report on study of modeling of nonlinear cyclic load–deformation behavior of shallow foundations. In: PEER workshop. Davis: University of California; 2003.
[62] Kutter BL, Wilson DL. Physical modeling of dynamic behavior of soil–foundation–superstructure systems. Int J Phys Model Geotech 2006;6(1):1–12.
[63] Kutter BL, Martin G, Hutchinson TC, Harden C, Gajan S, Phalen JD. Workshop on modeling of nonlinear cyclic load–deformation behavior of shallow foundations. PEER report 2005/14. Berkeley, CA: Pacific Earthquake Engineering Research Center, University of California; 2006.
[64] Luco JE, Westman RA. Dynamic response of circular footings. J Eng Mech Div 1971;97:1381–95.
[65] Makris N, Roussos Y. Rocking response of rigid blocks under near source ground motions. Géotechnique 2000;50:243–62.
[66] Martin GR, Lam IP. Earthquake resistant design of foundations: retrofit of existing foundations. In: Proceedings of the geoengineering 2000 conference (GeoEng2000). Melbourne, Australia; 19–24 November 2000.
[67] Massimino MR, Maugeri M. Physical modelling of shaking table tests on dynamic soil–foundation interaction and numerical and analytical simulation. Soil Dyn Earthq Eng 2013;49:1–18.
[68] Maugeri M, Musumeci G, Novità D, Taylor CA. Shaking table test of failure of a shallow foundation subjected to an eccentric load. Soil Dyn Earthq Eng 2000;20:435–44.
[69] Meek J. Effect of foundation tipping on dynamic response. J Struct Div 1975;101:1297–311.
[70] Mergos PE, Kawashima K. Rocking isolation of a typical bridge pier on spread foundation. J Earthq Eng 2005;9:395–414.
[71] Meyerhof GG. Some recent research on the bearing capacity of foundations. Can Geotech J 1963;1:6–26.
[72] Negro P, Paolucci R, Pedrett S, Faccioli E. Large-scale soil–structure interaction experiments on sand under cyclic loading. In: Proceedings of the 12th world conference on earthquake engineering. Auckland, New Zealan; 2000: paper no. 1191.
[73] Nova R, Montrasio L. Settlement of shallow foundations on sand. Géotechnique 1991;41:243–56.
[74] Panagiotidou AI, Gazetas G, Gerolymos N. Pushover and seismic response of foundations on stiff clay: analysis with P–δ effects. Earthq Spectra 2012;28:1589–618.
[75] Panagiotidou AI. 2D and 3D inelastic seismic response analysis of foundation with uplifting and P–δ effects. (Diploma thesis). Athens, Greece: National Technical University; 2010.
[76] Paolucci R. Simplified evaluation of earthquake induced permanent displacements of shallow foundations. J Earthq Eng 1997;1:563–79.
[77] Paolucci R, Shirato M, Yilmaz MT. Seismic behavior of shallow foundations: shaking table experiments vs. numerical modeling. Earthq Eng Struct Dyn 2008;37:577–95.
[78] Paolucci R, Pecker A. Seismic bearing capacity of shallow strip foundations on dry soils. Soils Found 1997;37:95–105.
[79] Paolucci R, Figini R, Petrini L. Introducing dynamic nonlinear soil–foundation–structure interaction effects in displacement based seismic design. Earthqu Spectra 2013;29:475–96.
[80] Paulay T, Priestley MJN. Seismic design of reinforced concrete and masonry buildings. New York: John Wiley & Sons; 1992.
[81] Pecker A. A seismic foundation design process, lessons learned from two major projects: the Vasco de Gama and the Rion Antirion bridges. In: Proceedings of the ACI international conference on seismic bridge design and retrofit. University of California at San Diego; 2003.

[82] Pecker A. Capacity design principles for shallow foundations in seismic areas. Keynote lecture. In: Bisch P, Labbe P, Pecker A, editors. 11th European conference earthquake engineering. Rotterdam, Netherlands: A.A. Balkema; 1998. p. 303–15.
[83] Pender MJ. Seismic design and performance of surface foundations. Chapter 10. In: Pitilakis K, editor. Earthquake geotechnical engineering. Springer Editor: Atilla Ansal; 2007. p. 216–25.
[84] Pender MJ. Seismic assessment and improvement of building foundations. Supplement to "assessment and improvement of structural performance of buildings in earthquake". New Zealand: University of Auckland; 2010.
[85] Pender MJ. Inferred beneficial effects of SFSI for a M-story building with shallow foundation on gravel. In: Proceedings of the ICEGE – Ishihara Conference, Istanbul; 2013.
[86] Priestley MJN. Myths and fallacies in earthquake engineering – conflicts between design and reality. Bull N Z Soc Earthq Eng 1993;26:329–41.
[87] Priestley MJN. Myths and fallacies in earthquake engineering, revisited. In: Ninth Mallet–Milne lecture. Pavia, Italy: Rose School, IUSS Press, Instituto Universitario di Studi Superiori; 2003.
[88] Raychowdhury P, Hutchinson T. Performance evaluation of a nonlinear Winkler-based shallow foundation model using centrifuge test results. Earthq Eng Struct Dyn 2009;38:679–98.
[89] Roesset JM. Stiffness and damping coefficients of foundations. Dynamic response of foundations: analytical aspects. In: O'Neil MW, Dobry R, editors. Reston, VA: American Society of Civil Engineers; 1980. p. 1–30.
[90] Salençon J, Pecker A. Ultimate bearing capacity of shallow foundations under inclined and eccentric loads. Part II: purely cohesive soil without tensile strength. Eur J Mech A: Solids 1995;14:377–96.
[91] Shi B, Anooshehpoor A, Zeng Y, Brune J. Rocking and overturning of precariously balanced rocks by earthquake. Bull Seismol Soc Am 1996;86:1364–71.
[92] Shirato M, Kouno T, Nakatani S, Paolucci R. Large-scale model tests of shallow foundations subjected to earthquake loads. In: Proceedings of the 2nd Japan–Greece workshop on seismic design, observation, and retrofit of foundations. Tokyo: Japanese Society of Civil Engineers; 2007. p. 275–99.
[93] Shirato M, Kuono T, Asai R, Fukui J, Paolucci R. Large scale experiments on nonlinear behavior of shallow foundations subjected to strong earthquakes. Soils Found 2008;48:673–92.
[94] Tassios TP. Assessment of concrete structures and design procedures for upgrading. CEB Bull 1983;162.
[95] Veletsos AS, Nair VV. Seismic interaction of structures on hysteretic foundations. J Struct Eng ASCE 1975;101:109–29.
[96] Veletsos AS, Wei YT. Lateral and rocking vibration of footings. J Soil Mech Found Div 1971;97:1227–48.
[97] Wolf JP. Soil–structure interaction analysis in time-domain. Englewood Cliffs, NJ: Prentice-Hall; 1988.
[98] Wotherspoon LM, Pender JJ. Effect of shallow foundation modeling on seismic response of moment frame structures. In: Orense RP, Chouw N, Pender MJ, editors. Soil–foundation–structure interaction. New York: CRC Press, Taylor & Francis Group; 2010. p. 117–24.
[99] Zhang J, Makris N. Rocking response of free-standing blocks under cycloidal pulses. J Eng Mech 2001;127(5):473–83.