Should Elastic Response Spectra be the Basis of Seismic Design of Strongly Inelastic and Soft-Soil–Structure Systems ?

G. Gazetas
National Technical Univerity, Athens, Greece

ABSTRACT: Elastic Design Response Spectra (EDRS) have been adopted all over the world as the basic description of the seismic threat at a particular site, corresponding to its specific seismicity and soil conditions. Originally utilized directly in modal response analyses, today such EDRS are being used to derive compatible artificial time histories, i.e. whose response spectra closely match the EDRS. One of the main dogmas in earthquake engineering is that these EDRS and the artificial motions compatible with them completely and rationally describe the seismic excitation for all possible structures to be built on the particular site — *even for highly inelastic systems*. The presentation will demonstrate the fallacy of this presumption by showing that the critical motions which lead to maximum response of elastic and inelastic systems are fundamentally different. Highly inelastic-nonlinear structures in particular, such as sliding and rocking systems, are shown to be sensitive to motion characteristics that are not reflected at all in the elastic spectra. The concept of "equivalent" spectra is developed to illustrate this point.
Another presumption entrenched in earthquake engineering practice with severe consequences when assessing soil-structure interaction (SSI) effects refers to the shape of the EDRS for soft soil categories. In general, the design spectra have a constant maximum acceleration plateau, max $S_a \approx 2.5A$ to $3.0A$ where A = the effective ground acceleration, followed by a monotonically descending branch at higher periods. For soil categories the constant S_a plateau extends to higher periods. In other words : *the softer the soil, the flatter the spectrum*. Thus SSI effects are always beneficial. The presentation will illustrate the fallacy of this presumption and will propose the adoption of so-called "Bi-normalized" Spectra for (nearly) elastic design.

1 DESIGN EARTHQUAKE EXCITATION

To design structures against earthquakes the ground shaking that will be generated in a future critical event must be specified. This is a difficult task even if the magnitude and the distance from the source are known. Ground shaking (or ground motions, as it is usually called), recorded on accelerographs throughout the world during the last half century, have revealed :

- highly variable intensity and frequency characteristics
- a tremendous variability in the detailed sequence of cycles and the presence or not of distinct severe pulses

so that it did not seem practically feasible to anticipate time histories of a design event.

The invention of the concept of the Elastic Design Response Spectrum [EDRS] and its universal adoption in seismic codes of practice has been a stepping stone in earthquake engineering that seemed to have solved the problem. Over the years, the shape and size of design spectra have evolved to account for the nature of seismicity and soil conditions pertaining to a particular site. The basis for establishing such design spectra has been the statistical processing of the elastic spectra computed from available accelerograms recorded worldwide, followed by some unavoidable intelligent smoothening and modifications based on experience.

Certainly the introduction of EDRS in engineering practice has been a monumental step forward. One of the main presumptions that had emerged from the early years of the use of the design spectrum is that, although derived for linear (visco) elastic 1-dof oscillators, it can be the basis for design of inelastic systems. In other words, the EDRS completely and rationally (even if "generically") describes the seismic excitation for all possible structures to be built on the particular site.

In fact, Inelastic Design Response Spectra [IDRS] were derived for a specified constant ductility μ, to be used directly for elastic response analysis of inelastic multi-dof systems. For instance, for elastic–ideally-plastic force–displacement behavior of a system, the following expressions have been particularly popular for the reduction factor R_y which divides the EDRS to obtain IDRS:

$$R_y = 1, \qquad R_y = \left(2\mu - 1\right)^{1/2}, \qquad R_y = \mu \tag{1}$$

depending on the short, medium, and long-period range (e.g., Chopra 2000)
.

The idea seemed powerful at a time of limited access to reliable nonlinear analysis software; it has been implemented in seismic codes in a simplified format (e.g., in EC8, by using instead of the period-dependent R_y , a single period-independent factor, q). Today, for critical facilities at least this approximation has to a large extent been replaced by direct nonlinear time integration analysis. For such analyses, one needs to "devise" acceleration time histories, the response spectra of which are compatible with the EDRS, meaning that they fit the design spectrum almost at all periods. Consequently, in the words of EC8-1:

> "…the earthquake motion at a given point on the ground surface is represented by the elastic acceleration response spectrum…"

Stated differently, it is presumed, even if not said quite explicitly, that the motions which produce the largest response of elastic systems will also produce the largest response of (all possible) inelastic systems (within reasonable engineering accuracy, of course). On the basis of this, a huge amount of (unnecessary) research effort has been spent on developing methods of producing EDRS-compatible artificial time histories.

2 EVIDENCE AGAINST THIS PRESSUMPTION

Thirty-five years ago in his seminal work to explain the failures of the Olive View Hospital in the San Fernando 1971 earthquake, Professor Vitelmo Bertero seriously questioned the validity of the above presumption. He showed convincingly that, for a given site, the design seismic motion should not be unique, because

> "…the critical ground motion depends on the type of behavior that is expected to control the response of the building…" (Bertero, 1976)

meaning that the types of excitation that induce the maximum response in elastic and inelastic systems are, in his words, "fundamentally different".

To make a long story short, old and recent studies (e.g., Bertero et al 1978, Garini et al 2012) have persuasively shown that:

(a) Elastic systems suffer the most from excitations containing several cycles of nearly uniform amplitude and a nearly constant (dominant) period equal to the natural period of the system — "resonance"…Sinusoidal accelerograms in particular, such as those resulting from linear-soil amplification as was the case in Mexico City in 1985, are an extreme type of such motions for which the maximum dynamic magnification factor can reach $0.5/\xi$; for the typical damping ratio $\xi = 0.05$ this magnification is equal to 10 — a huge value indeed, rendering this type of motion critical. On the contrary, a single idealized severe pulse can at most induce a maximum dynamic magnification of barely 2 — hence ground motions containing long acceleration pulses, such as those prevalent in near-fault motions, could hardly be critical for linear systems.

(b) The opposite is true for strongly inelastic structures: even a single long acceleration pulse with amplitude exceeding the yield acceleration may lead to very large response. Periodic short acceleration cycles can only contribute to building the response of the system up to its yielding level — thereafter resonance is depressed and hysteretic action takes place. Thus, even a nearly–sinusoidal motion is unlikely to induce large inelastic displacement to be the critical motion for this system.

The above ideas are illustrated in Figs 1-3, modified from Bertero's (1976) original example.

The conclusion that emerges from the above can be restated as follows: given an EDRS with the intention of computing the response of a highly inelastic structure, we fit a ("compatible") multi-cycle periodic motion (almost a modulated sinusoid, of amplitude A — its peak ground acceleration). This motion may only induce a relatively minor plastic deformation to an inelastic system, even if the yield acceleration A_y of the system is much smaller than the peak acceleration A of that "compatible" sinusoidal motion (e.g., $A_y = A/3$) . Consider now a single half-cycle sinus or impulse motion of long duration (say, 2 seconds) but with peak acceleration a equaling only half the peak acceleration of the sinusoid: $a = A/2$. The elastic response spectrum of this pulse is likely to be a fraction only of the EDRS, which we recall the sinusoid fits quite well. Yet, the inelastic system will experience far greater inelastic displacement from this single pulse than from the periodic multi-cycle motion. This is contrary to what might be expected by comparing the respective elastic response spectra.

Arguments slightly different and from another perspective (but to the same effect and equally persuasive) have been advocated by Professor Nigel Priestley (1993, 2003). In these papers, a section under the title "The Fallacy of Design to Elastic Acceleration Spectra" showed that the "equal displacement" approximation [Eq. (1c), above] which is the fundamental way of translating elastic to inelastic response may lead to non-conservative results for a real (inelastic) structure.

Further evidence in support of the above arguments is provided here. We represent a highly inelastic restoring-force–displacement relationship with an idealized Coulomb friction mechanism, of constant coefficient of friction. Two systems are considered. In both of them, a rigid block rests (in simple frictional contact) on a rigid base which is shaken with a specific recorded accelerogram. The base is either horizontal or inclined. These are two conceptual models for, respectively,

- symmetrically–inelastic structures, such as frames, piers, foundations, and
- asymmetric, sliding–governed geotechnical systems, such as retaining walls and slopes.

Detailed studies of the seismic performance of these two models have been published by Garini et al (2011) and Gazetas et al (2009). Two of their conclusions pertaining to the problem at hand are worthy of summarizing here:

(1) Forward-directivity and fling-step affected near-fault motions, containing long acceleration pulses and/or large velocity steps, may have a profound detrimental effect on the induced slippage (symmetric or asymmetric), the magnitude of which can not possibly be predicted on the basis of their elastic response spectra.

(2) For asymmetric sliding systems, in particular, just reversing the polarity of a ground motion (implying no change of its elastic response spectrum) may have a most dramatic effect on the accumulating residual slippage — differences of up to 400 % between the magnitudes of slippages induced by applying the motion in the (+) and then in the (–) direction, despite the one single response spectrum.

An additional alternative way to convince that the elastic response spectrum of a motion is not a good indicator of its "destructiveness potential" is through the concept of the "Equivalent Motions for Sliding" (EMS). We define as EMS any number of recorded accelerograms that have been scaled up or down so as to induce exactly the same slippage to one of the aforementioned sliding systems.

For the inclined-base asymmetric sliding model of yield acceleration of 0.05 g pictured in Fig. 1, fourteen records (from San Fernando, Loma Prieta, Kobe, Chi-Chi, Kocaeli, Imperial Valley, San Salvador, and Düzce-Bolu) are selected as excitation. They are scaled up or down in small incremental steps until the resulting downward sliding equals 1.0 m. The outcome is shown in Figs 4 and 5: the *"equivalent"* motions (from the view point of the sliding block) in Fig. 4, and their respective elastic response spectra in Fig. 5. Evidently, there is no resemblance of peak values, frequency characteristics, or duration of these motions. Peak accelerations, for example, range from 0.19 g (for the Jensen-022–based motion) to the 1.18 g (for the Sakarya–based motion) — a factor of 6 ! The "equivalent" (in the above sense) elastic spectra reflect these differences between the motions: not only do the various spectra differ widely one from another, but in some cases there is absolutely no overlapping of spectral curves, throughout the period range examined (up to 4 s). For instance, the Sakarya–based spectrum exceeds the CHU080–based one by a factor of more than 2, everywhere. And so on.

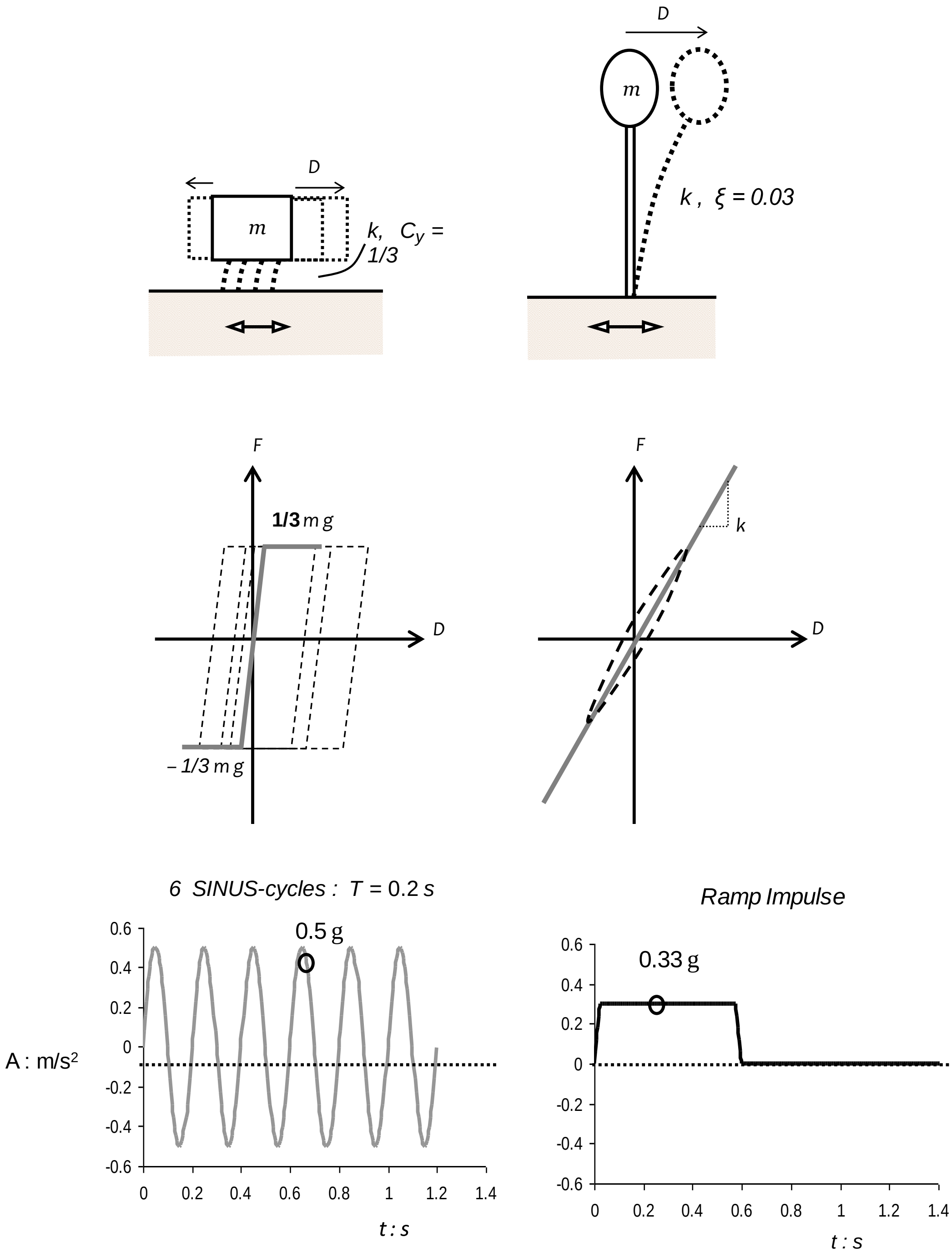

Figure 1. Two systems compared : an elastic–perfectly-plastic and a (visco) elastic oscillator. Their fundamental periods are the same, varying parametrically T_n = 0.1, 0.2, and 0.3 seconds. They will be excited by a sinusoidal (A = 0,5 g, T = 0,2s) and a “ramp impulse” (A = 0,33 g, duration = 0,65s) motions.

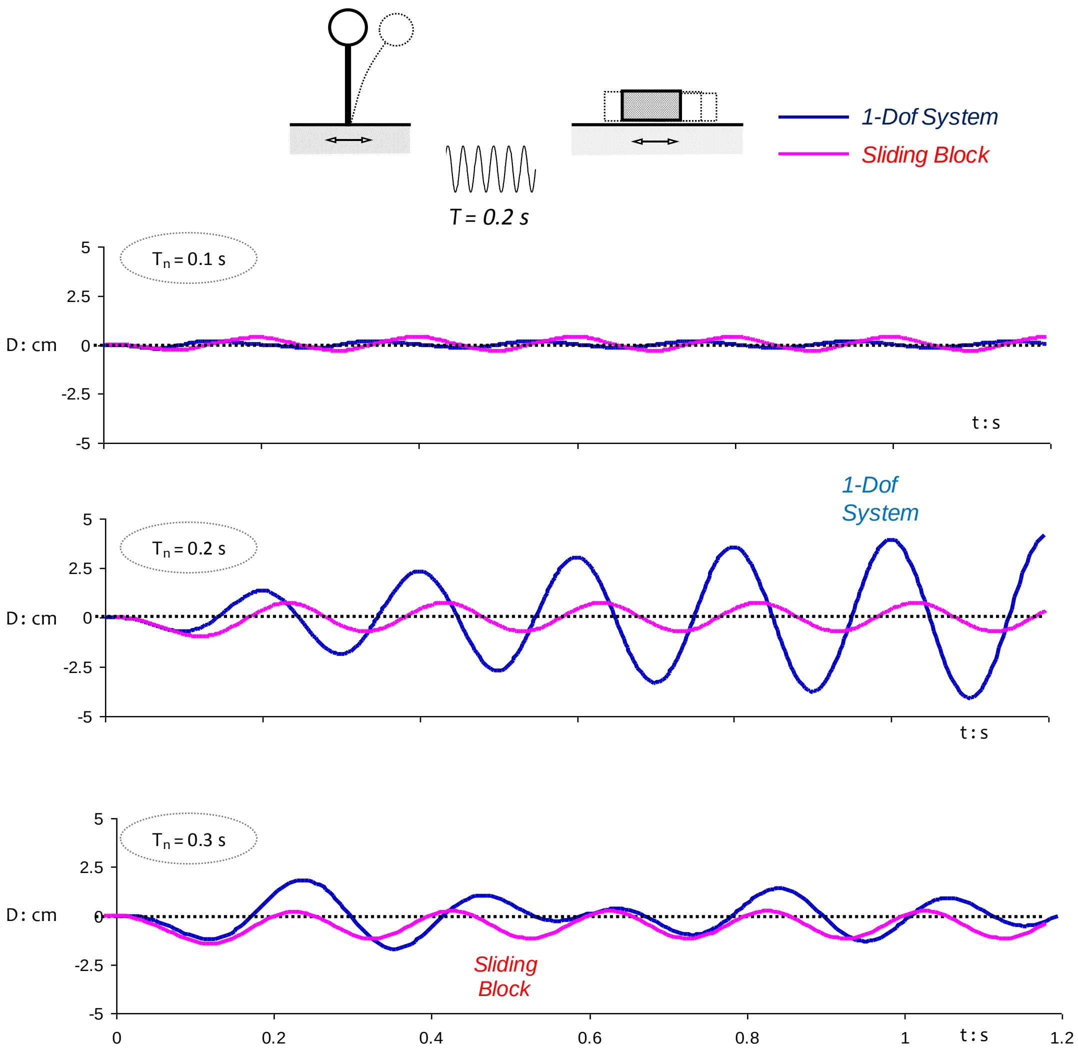

Figure 2. Comparison of the responses of the elastic and elastoplastic systems to the sinusoidal excitation to Figure 1.

Conclusion: the excitations that induce damage to inelastic and to elastic systems are of fundamentally different nature. An elastic design response spectrum (EDRS) specifies the damaging potential of the compatible earthquake motions *only* to elastic (or perhaps nearly-elastic) systems. If strong inelastic response is contemplated, time histories for excitation should be “sampled” directly from recorded motions in the required magnitude–distance–directivity–soil-variability space ; not artificially selected by matching a target (elastic) spectrum. It is interesting that a similar proposition has been made by Gail Atkinson (2012) in her keynote lecture in the 15 WCEE in Lisbon.

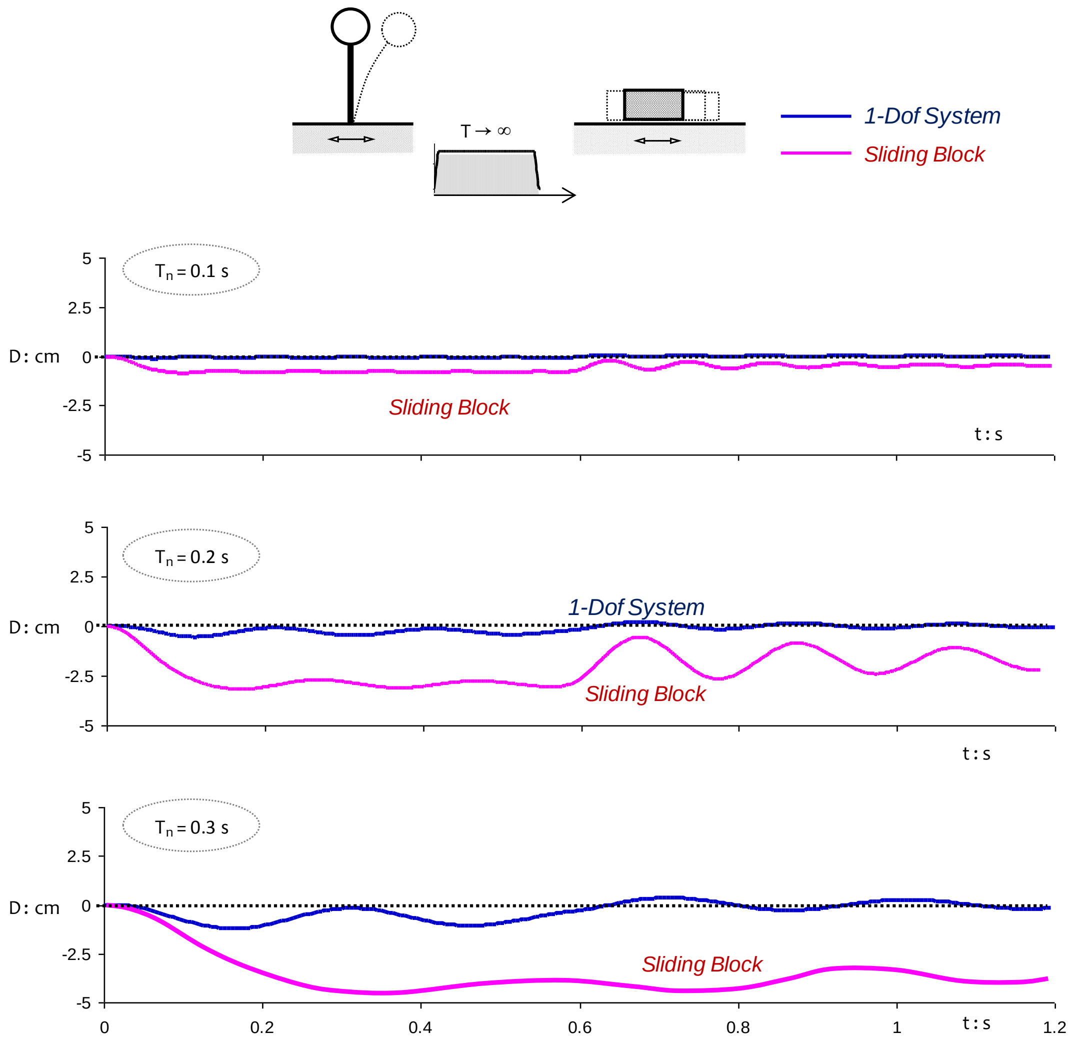

Figure 3. Comparison of the responses of the elastic and elastoplastic systems to the "ramp impulse" excitation of Figure 1.

3. DESIGN ACCELERATION RESPONSE SPECTRA FOR SOFT SOILS : *"THE SHOFTER, THE FLATTER"* PRESSUMPTION

Wave propagation through the near-surface soils may have a profound effect on the resulting ground motions. Equivalent-linear and truly nonlinear methods developed in the last forty years are presently used in state-of-the-art practice to predict these effects and come up with a realistic motion, for seismic design evaluation. On the other hand, seismic codes have, perhaps unavoidably, over-simplified the problem by: (i) classifying the soil deposits into a few very broad categories, and (ii) fixing the shape, Sa/A, of the EDRS for each soil category. All spectral shapes have a constant maximum value of 2.5–3.0 for the range from very low up to moderate periods, and subsequently decrease monotonically with period. For the "flexible" soil categories (e.g., categories C and D in EC8) the range of periods corresponding to the constant plateau expands towards higher periods. In other words: the more "flexible" a soil deposit (i.e., the smaller its stiffness and/or the larger its thickness) the flatter the design spectrum. And the maximum of the plateau rarely exceeds 2.5 for very soft soils.

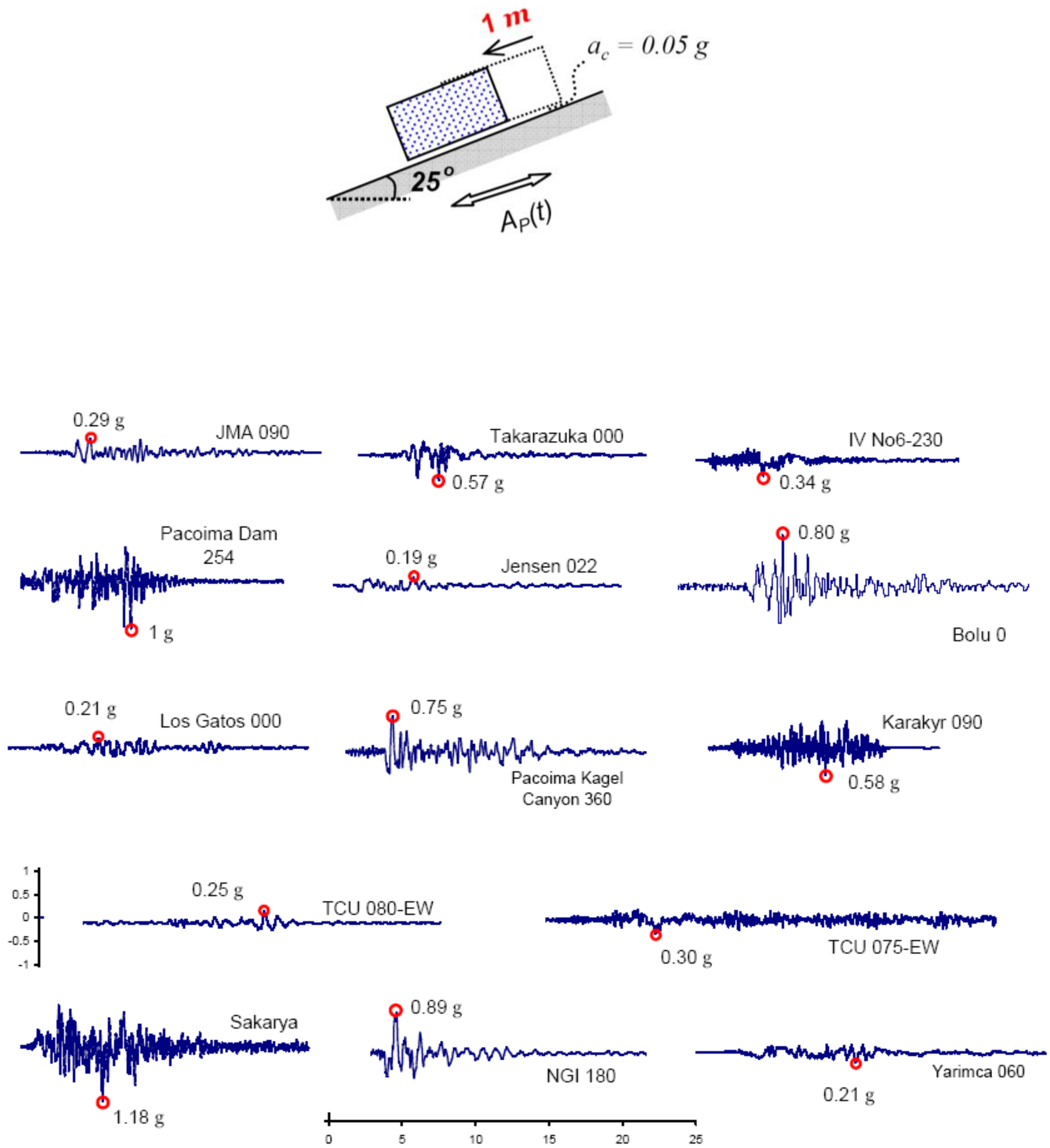

Figure 4. The asymmetric sliding block model and the14 *equivalent* acceleration histories, i.e., histories which all induce the same 1 m slippage to the block when they excite its base.

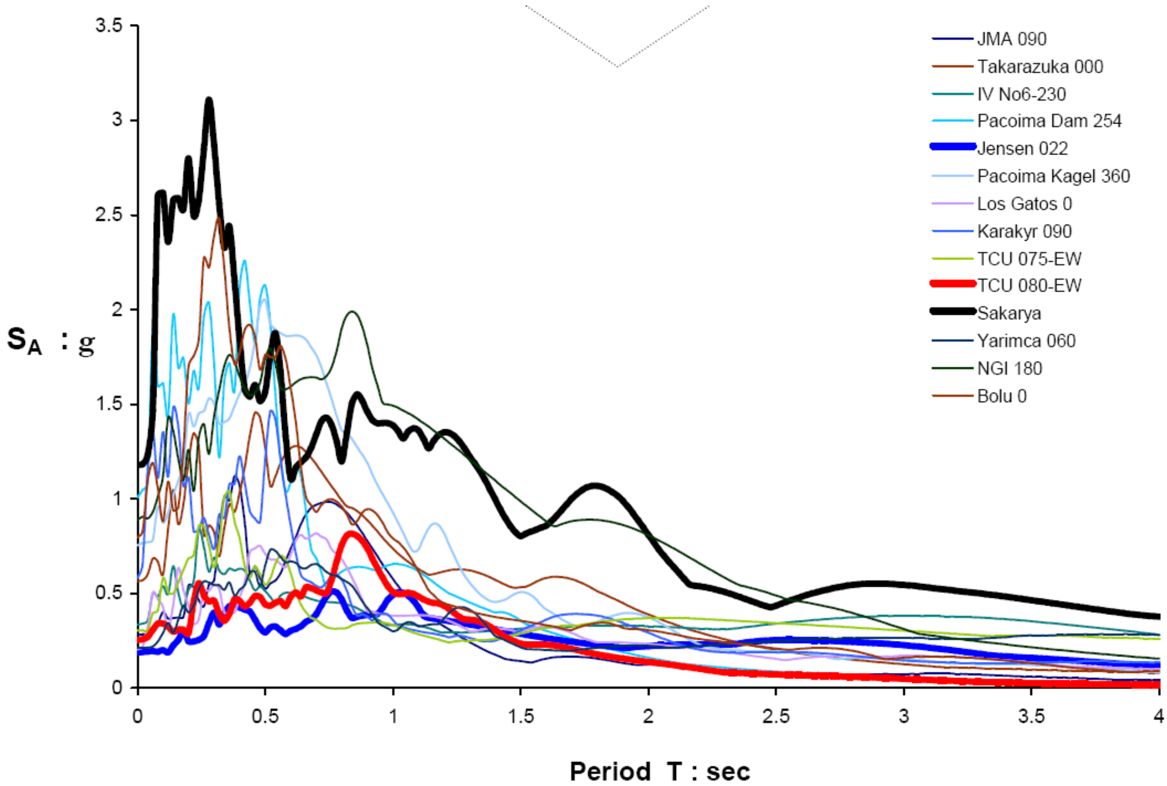

Figure 5. The equivalent (elastic) response spectra of the 14 motions that lead to the same downhill slippage (= inelastic response) of the block of Figure 4.

The questions to be answered are:

(a) what is the historic origin of this (flatter) shape for the softer soil deposits ?
(b) is it a rational or at least conservative simplification ?
(c) if not, how could it be changed ?

The shape of EDRS emerged from the early study of the late Professor Harry B. Seed in the aftermath of the San Fernando 1971 earthquake. Despite the fact that he and his coworkers had already developed SHAKE and could perform analyses to determine the effects of "soil amplification" on ground motions, some influential schools of earthquake-engineering thought opposed using the results of such analyses into the Code.

To overcome an increased skepticism, he turned into a purely empirical approach: the response spectra, Sa = Sa(T), were computed using nearly all the then available accelerograms recorded on top of soils that could be grouped into reasonably coherent categories. Statistically processing the corresponding spectra for each soil category, he came up with an average (at each period) normalized spectrum, Sa(T)/A. Today's EDRS have shapes that are close descendants of those normalized spectra (Seed et 1976). And basically what these shapes imply is that with increasing period (as when soft-Soil–Structure Interaction is taken into account) the structure will invariably develop reduced acceleration levels.

4 FLAW AND CORRECTION

What is wrong with this statistical approach and the current EDRS ?

The limitation stems mainly from the wide breadth in stiffness and thickness that characterize each one of the soil categories, especially the softer ones. This was unavoidable at the time: as the number of recordings were extremely (by today's standards) limited, being mostly from the San Fernando 1971 event, the categories had to be very broad so that a decent number of recordings could be found belonging to each one of them. Otherwise, there would have been no statistical significance in the procedure.

Therefore, a range of natural periods of the possible soil deposits belonging to one category, on the surface of which soil-modified records were available, could be in the ratio 1 : 4. It is thus quite likely that the response spectra of these actual motions had relatively sharp resonance peaks at well separated periods. Hence, at a particular period for which one spectrum had a peak, the spectra on sites with different periods (but still of the same soil category) were likely to have small or very-small values. Averaging these dissimilar values simply "smears" all the sharp peaks, resulting in a flat spectrum.

A simply stated conclusion: using the soft-soil–EDRS that are based on this averaging process we (erroneously) disregard the resonance between soil deposit and excitation — probably the most significant effect of soil on ground shaking. In fact, one after the other recorded motions on top of soft soils show indeed relatively sharp resonant peaks. Famous examples: the Mexico City SCT and CDAO 1985 records, the Treasure Island 1989 record, the Takatori 1995 record, and numerous other less well known motions (although perhaps with not quite so sharp spectral peaks).

To demonstrate the validity of the above arguments and approximately quantify the extent of error in the soft-soil–EDRS, we analyzed the response of a large number of generic soil profiles (with key variables: the distribution with depth of shear modulus, the thickness of the deposit, and the soil to rock impedance ratio) subjected to seven rock accelerograms (recorded in US, Turkey, Iran, Greece) after scaling them up and down to peak accelerations of 0.2 g, 0.4 g, and 0.6 g. Using both equivalent linear and nonlinear analyses (Gerolymos & Gazetas 2005, Drosos et al 2012) nearly 2000 motions were derived for the ground surface. They were processed in two different ways:

(a) In the conventional way of Seed et al (1976) which is still the basis of the EDRS in seismic codes: For each individual computed spectrum, at each period T, the normalized spectral value $S_a(T)/A$ is obtained. The average of all values of S_a/A, from all motions, for this particular period is one value of the desired spectrum: $S_a/A = f(T)$. Shown in Fig. 6a), this average spectrum of our study does indeed possess a more-or-less horizontal plateau with amplitude almost equal to 2.5.

(b) In a non-conventional way, in which both the spectral values and the period are normalized: for each individual computed spectrum, at each period T, the normalized spectral value S_a/A is obtained. The dominant period T_p of this spectrum is identified (admittedly not always without some ambiguity). Obviously, each motion has its own value of T_p. Then for each value of the period ratio T/T_p we obtain the average value of S_a/A, from all motions. Thus we arrive at the Bi–Normalized spectrum:

$$S_a/A = f(T/T_p) \qquad (2)$$

Plotted in Fig. 6(b), this spectrum has little resemblance with the conventional spectrum: there is no horizontal plateau but a dominant (rather sharp) peak at $T/T_p \approx 1$. The maximum value of the peak, $\max(S_a/A)$ reaches 3.75 — 50 % larger than the conventional 2.5.

It is clear that the (true or pseudo) resonances between soil and excitation are well preserved only in the Bi–Normalized Spectrum. The conventional spectrum instead does not respect the physics of the problem. It is un-conservative, especially for structures with $T \approx T_p$, and leads to erroneous and mostly unsafe results on the possible effects of soil–structure interaction.

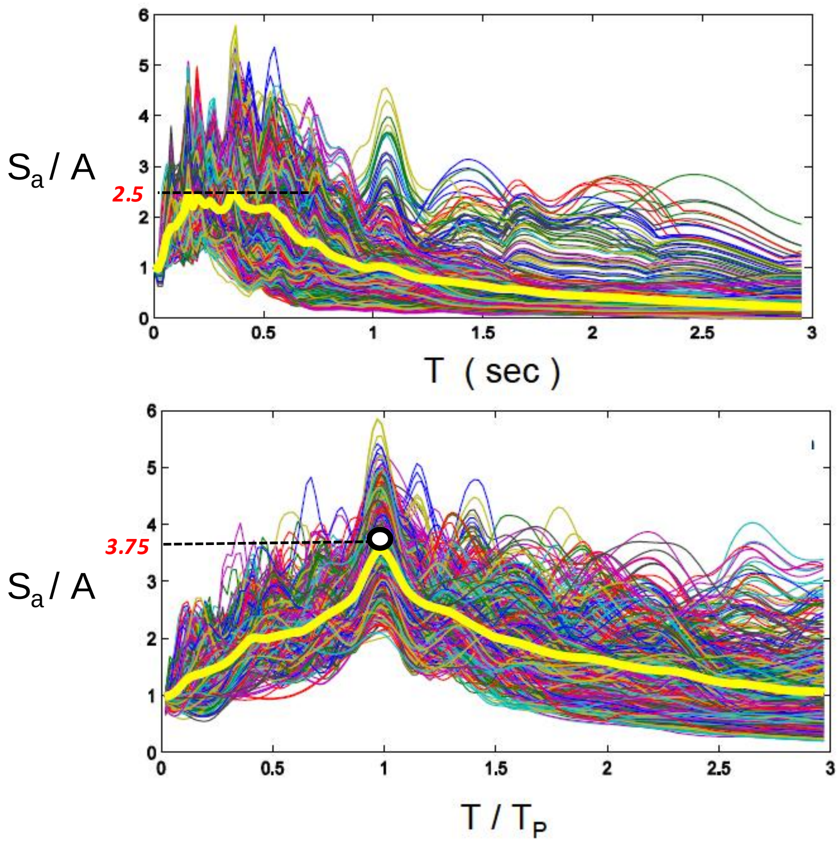

Figure 6. Spectral shapes, S_a/A, from averaging the elastic response spectra of 1020 soil amplified motions: (a) versus period, T [Normalized Spectrum], and (b) versus the ratio T/T_p, where T_p is the predominant period of each motion [Bi-Normalized Spectrum]. The generic soil profiles examined belong to soil category C of the seismic Eurocode EC8.

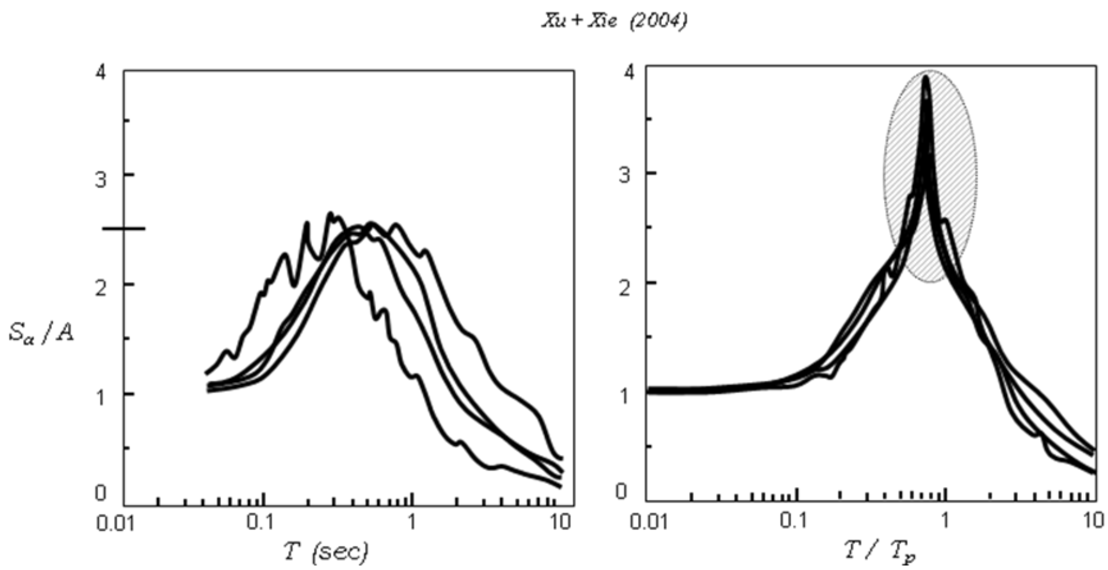

Figure 7. Normalized and "Bi-Normalized" response spectra of Chi-Chi earthquake ground motions for soil types B, C, D, E [from Xu and Xie, 2004]. The conclusion is similar to that of Fig. 6.

It is worthy of note that three detailed similar studies have come up with the same conclusion: Mylonakis & Gazetas (2000), Xu & Xie (2004), Ziotopoulou & Gazetas (2010). We refer to them for details. The term Bi–Normalized (BN) Spectrum was to my knowledge introduced by Xu & Xie (2004). Their BN spectrum from the records of the Chi-Chi earthquake is shown in Fig. 7 One of the fascinating findings of the last two of the above publications is that dominant peak of the Bi–Normalized Spectrum is independent of: soil category, intensity of shaking and hence degree of soil nonlinearity, and type of seismic excitation. Of course this particularly convenient outcome does not extent to the predominant period Tp itself, which is a function of all these factors.

AKCNOWLEDGEMENT

I am thankful for the support of the European Research Council (ERC) through its "Ideas" Programme: in Support of Frontier Research, under Contract number ERC-2008-AdG 228254-DARE

5 REFERENCES

Atkinson GM., 2012. Integrating advances in grounds-motion and seismic hazard analysis. *Proc. 15th World Conf. Earthq. Engng.*, Lisbon, Keynote/Invited Lecture.

Bertero VV., 1976. Establishment of design earthquakes: evaluation of present methods. *Proc. Int. Symp. on Earthq. Struct. Engng.*, Univ. of Missouri-Rolla, St. Louis, **1**, 551-580

Bertero VV, Mahin SA, Herrera RA., 1978. Aseiemic design implications of near-fault San Fernando earthquake records. *Earthq. Engng.& Struct. Dyn.*, **6** (1), 551-580

Chopra AK 2000. *Dynamics of Structures.* Prentice Hall.

Drosos V, Gerolymos N., Gazetas G., 2012. Constitutive model for soil amplification of ground shaking: parameter calibration, comparisons, validation. *Soil Dyn. Earhq. Engng*, 42, 255-274.

EC8 2009. Part 1 – General rules of the Seismic Eurocode.

Garini E, Gazetas G., Anastasopoulos I., 2011. Asymmetric Newmark siding caused by motions containing severe directivity and fling effects. *Géotechnique*, **61** (9), 773 – 756.

Gazetas G .,2006. Seismic design of foundations and soil-structure interaction. *Proc. Europ. Conf. on Earthq. Engng & Seismology*, Geneva, (cd Rom).

Gazetas G., Garini E., Anastasopoulos I., Georgarakos T., 2009. Effects of near-fault ground shaking on sliding systems. *Jnl. Geotech. & Geoenviron. Engng, ASCE*, **135** (12), 1906–1921.

Gazetas G., 2012. Some presumptions on the nature of base excitation may erroneously affect the response of strongly inelastic systems. *Proc. 15th World Conf. Earthq. Engng.*, Lisbon, Keynote/Invited Lecture.

Gerolymos N, Gazetas G., 2005. Constitutive model for 1-D cyclic soil behavior applied to seismic analysis of layered soils. *Soils & Foundations*, **45** (4), 147 – 159.

Mylonakis G., Gazetas G., 2000. Seismic soil-structure interaction: beneficial or detrimental? *Jnl. Earthq. Engng*, **4** (3), 277 -301.

Priestley MJN., 1993. Myths and fallacies in earthquake engineering – Conflicts between design and reality. *Bulletin NZ Soc. Earthq. Engng*, **26** (3), 329 – 341.

Priestley MJN., 2003. *Myths and fallacies in earthquake engineering, Revisited: The Mallet Milne Lecture 2003.* IUSS Press, Rose School, Pavia, Italy, pp 121.

Seed HB., Ugas C., Lysmer L., (1976). Site–dependent spectra for earthquake resistant design. *Bull. Seism. Soc. Am.*, **62,** 221-243.

Xu L, Xie L., 2004. Bi-normalized response spectral characteristics of the 1999 Chi-Chi Earthquake. *Earthq. Engng & Engng Vibrations*, **3** (2), 147 – 155.

Ziotopoulou A, Gazetas G.,2010. Are current design spectra sufficient for soil–structure systems on soft soils ?. *Advances in Performance-Based Earthquake Engineering*, MN Fardis, ed., Springer, 79 – 87.