Some presumptions on the nature of base excitation may erroneously affect the response of strongly inelastic systems

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SUMMARY:
Three presumptions on how the design base ground motion is defined are entrenched in earthquake engineering codes of practice: (a) elastic Design Response Spectra (consisting of a horizontal constant-acceleration branch from very low to medium periods and a descending branch at higher periods) adequately describe the seismic threat at a site; hence they must be closely respected from the selected accelerograms–excitations, even for *highly inelastic systems*; (b) for relatively soft and medium soil categories (as broadly defined in the codes) the shape of the acceleration design spectra, S_a /A, is flatter than for the stiffer soil categories, with its horizontal plateau extending to higher periods — "*the softer, the flatter*"; and (c) the vertical component of ground shaking can be very important in all cases and, for geotechnical systems, its effect is best accounted for by *vectorially combining* the vertical and horizontal effective ground accelerations. Severe limitations of the above concepts (which most often lead to unsafe results) are shown in the presentation, along with alternatives that largely avoid some of the detrimental consequences.

Keywords: Design spectrum, elasto-plastic response, rigid-plastic behaviour, sliding systems, soil categories, soil amplification, near-fault motions, long-period pulses, vertical acceleration, fallacy

1. ESTABLISHMENT OF DESIGN EARTHQUAKE EXCITATION

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 of earthquake engineering. Over the years, the shape and size of design spectra have evolved to account for the nature of *seismicity and soil* 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*:

$$R_y = 1 \qquad \text{for} \qquad T_{elastic} < T_a \tag{1a}$$

$$R_y = (2\mu - 1)^{1/2} \qquad \text{for} \quad T_b < T_{elastic} < T_c \tag{1b}$$

$$R_y = \mu \qquad \text{for} \qquad T_{elastic} > T_d \tag{1c}$$

in which $T_a < T_b < T_c < T_d$ depend on soil and seismicity characteristics of the particular site. Linear interpolation is assumed between T_a and T_b as well as between T_c and T_d (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). For critical facilities this approximation has to a large extent been replaced today 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).

2. EVIDENCE TO REFUTE 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 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 sine 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 Duzce-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 1 and 2: the "equivalent" motions (from the view point of the sliding block) in Fig. 1, and their respective elastic response spectra in Fig. 2. 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.

Conclusion: the excitations that induce damage to inelastic and to elastic systems are of fundamentally different nature. An *EDRS* specifies the damaging potential of the compatible earthquake motions *only* to elastic (or perhaps nearly-elastic) systems.

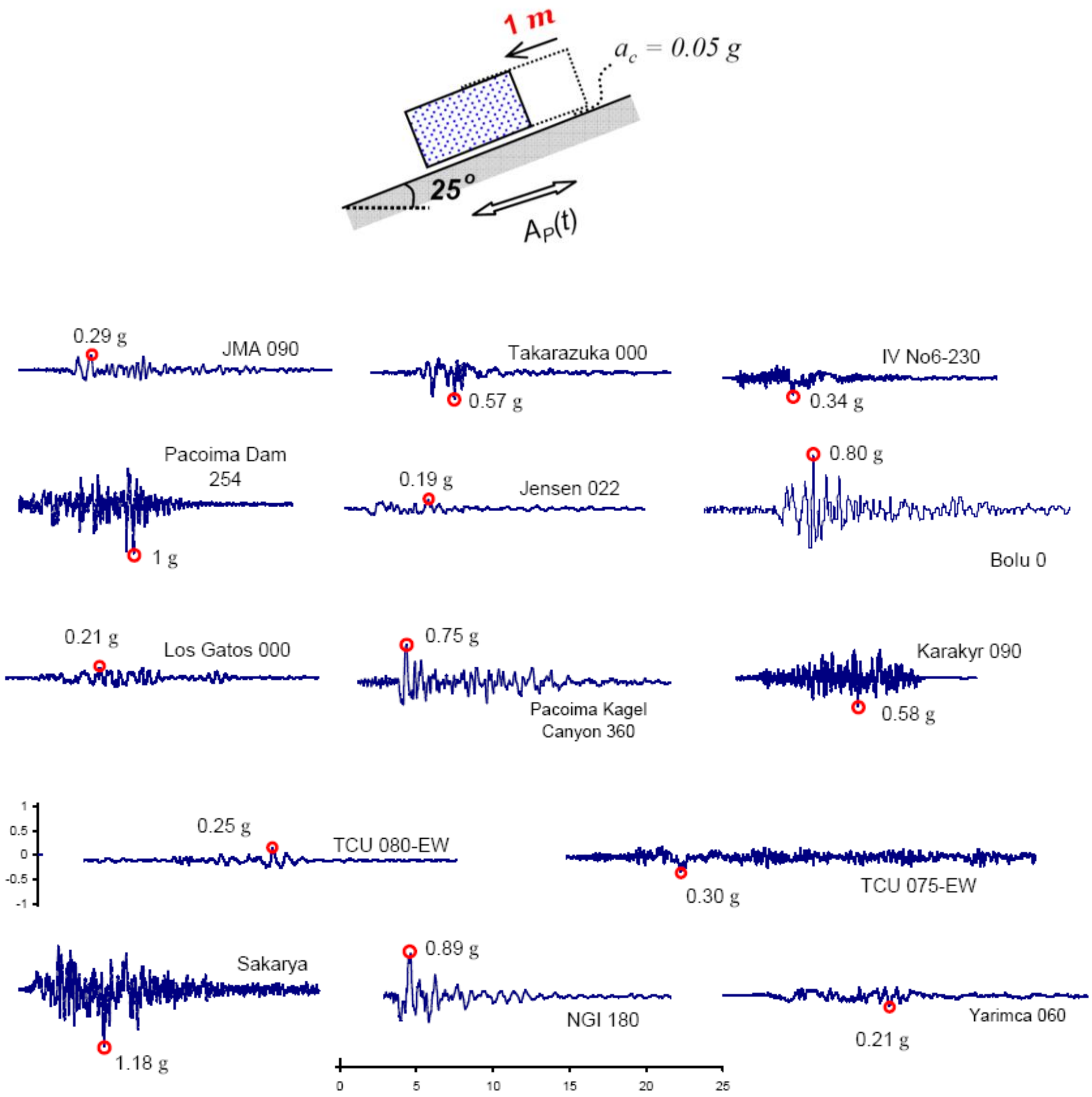

Figure 1. The asymmetric sliding block model and the14 *equivalent* acceleration histories, that is all of which induce the same 1 m slippage to the block when they excite its base.

3. DESIGN ACCELERATION RESPONSE SPECTRA FOR SOFT SOILS: *"THE SOFTER, THE FLATTER"* PRESUMPTION

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, S_a/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.

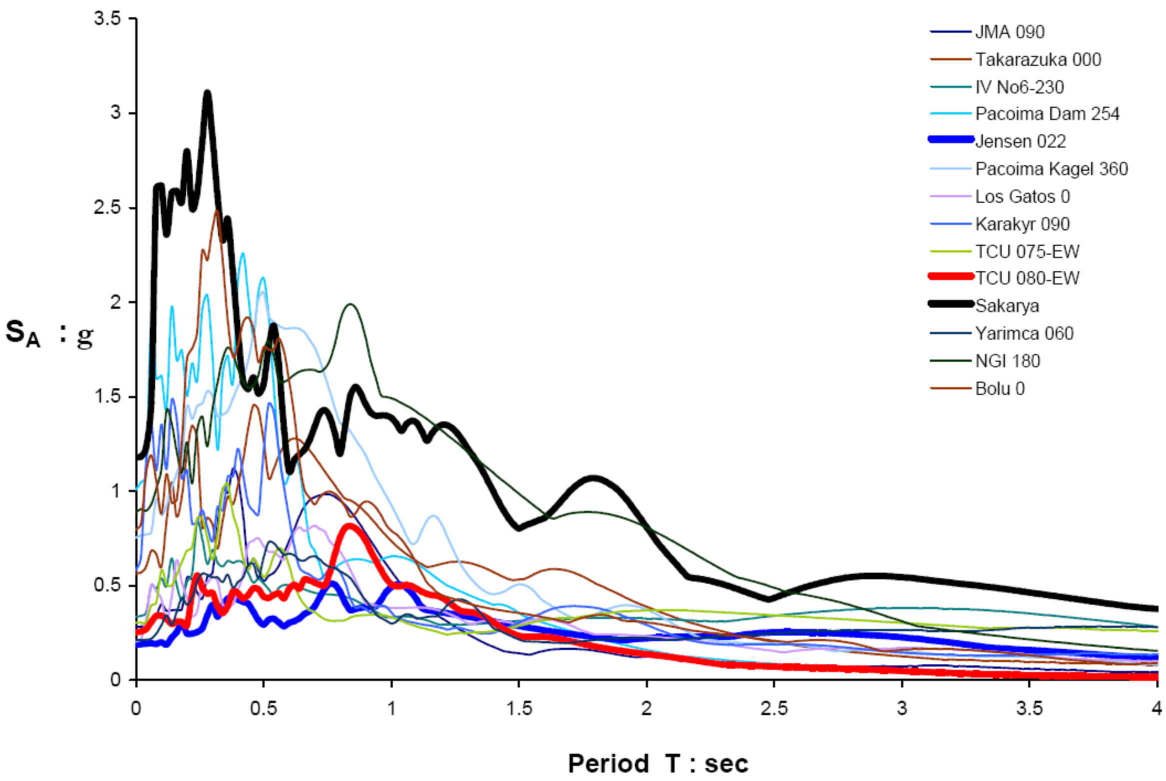

Figure 2. The response spectra of the 14 *equivalent* motions shown in Fig.1 which all cause the same 1 m sliding downhill displacement.

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, $S_a = S_a(T)$, 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, $S_a(T)/A$. 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 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. 3(a), 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. 3(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.

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* Spectrum was introduced by Xu & Xie (2004). 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 T_p itself, which is a function of all these factors.

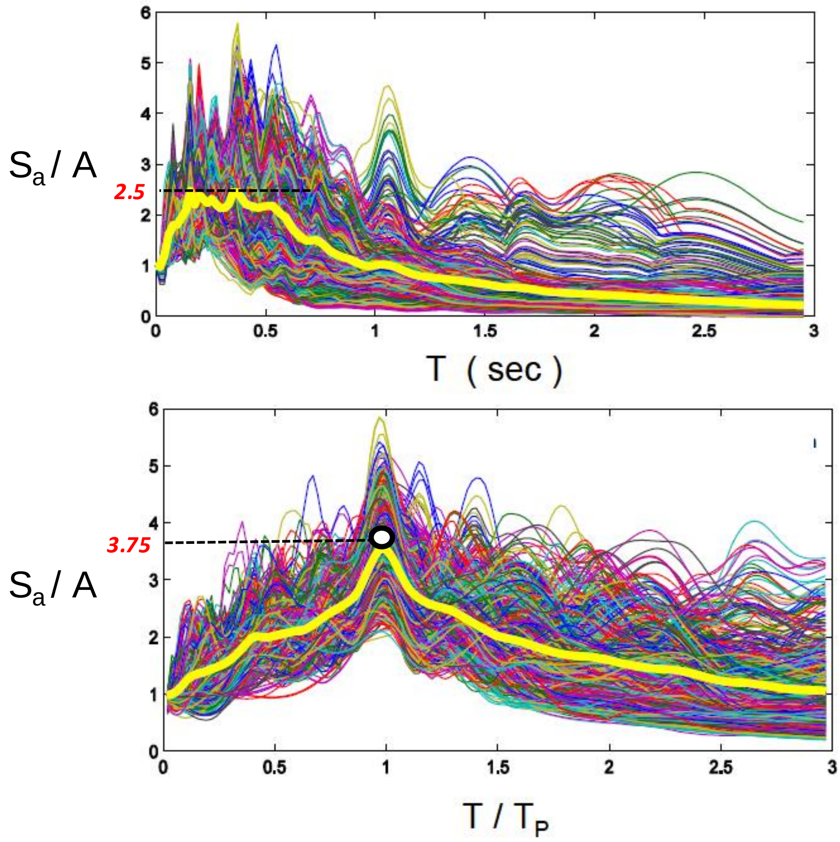

Figure 3. Spectral shapes, S_a/A, from averaging the elastic response spectra of 1020 soil amplified motions: (a) versus period, T *[Normalized Spectrum]*, and (b) versus the ratio T/T_p, where T_p is the predominant period of each motion *[Bi-Normalized Spectrum]*. The generic soil profiles examined belong to soil category C of EC8.

5. VERTICAL COMPONENT OF ACCELERATION: A GEOTECHNICAL FALLACY

In geotechnical systems designed pseudo-statically against failure (such as retaining systems, slopes, and footings), it has been a persistent misconception that vertical and horizontal amplitudes of acceleration should be applied simultaneously. In times past, when the design levels of acceleration were merely of about 0.05 g to 0.10 g for the horizontal component, and a fraction (1/2 or 2/3) of that for the vertical one, the error was insignificant. These days when ground records reveal an order of magnitude larger accelerations, often with similar values for the two components, the simultaneous pseudo-static application of the peak (or even "effective" values) of the two components is a gross violation of the physics of the problem.

It turns out that when proper time integration analysis is performed with simultaneous application of the two complete time histories (A_x and A_z), rather than solely of their amplitudes, the result are hardly different from those when only A_x is imposed ! This may sound paradoxical, but it stems from the nature of the two components whose frequency content and phasing details differ substantially, and hence they do not essential "combine" their effects. And in fact, since usually the vertical component comprises much higher frequencies, even when its peak value is higher than that of the horizontal, it is barely felt by, e.g., a sliding system with a Coulomb interface.

In support of the above arguments we refer to Gazetas et al (2009), Sarma & Scorer (2009), and Garini al (2011) which demonstrate that even the strongest simultaneous vertical excitation has no practically-discernible effect on either the maximum or the residual slip of symmetric or asymmetric friction-controlled systems, such as those mentioned above. Similar was the conclusion for the rocking of foundations at large angles of rotation: permanent rotation and toppling of the supported structure are hardy affected by a simultaneous vertical excitation (Gazetas et al 2012).

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