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Issues and challenges towards full Seismic Risk Analysis

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Should we go ahead with the response spectrum?

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1 Introduction

Earthquake engineering started its development around the mid of the last century, and since the very beginning an issue was to identify a convenient interface between seismologists and engineers for the description of the seismic input motion. In order to capture the role of such an interface, we are first going to give a quick overview of the civil engineering and mechanical engineering practices in general, which were conceived and codified in order to deal with force-controlled loads. Then we shall expose why, in such a context, the response spectrum appeared naturally as the most convenient interface.

Drawbacks of the response spectrum appeared step by step with the sophistication of earthquake engineering and the needs of earthquake engineers, for instance the need of transferring the input motion towards the equipment in industrial facilities. In the light of these developments we shall discuss the pro and cons of the response spectrum. The current trend in building codes evolution is to promote the representation of the input motion in the Acceleration-Displacement Response Spectrum (ADRS) format. We shall briefly introduce the ADRS, with a focus on its inelastic version, which actually appears as a constant ductile-demand response spectrum.

Eventually we shall discuss the merits of an alternative description of the seismic motion in the form of a Power Spectral Density (PSD), which is a powerful tool in the general framework of structural dynamics based on the random vibration theory.

Attention of the reader is drawn on the fact that we shall develop considerations about civil engineering practices of conventional building industry and of nuclear industry, while considerations about mechanical engineering are implicitly in the nuclear industry frame only.

2 Outlines of engineering practices

For civil and mechanical engineers, the design procedure of a structure or component includes the following major steps:

a) To evaluate loads that the structure or component will have to sustain during its expected life duration. There are permanent loads and variable loads. For the latter the engineer should evaluate or should be informed of the expected maximum values.

b) To set up a model of the structure or component and, assuming that the response will be in the elastic regime, to calculate/compute it under a series of prescribed combinations of loads. In the vast majority of cases, the calculated outputs, the “demand”, are expressed in terms of stresses.

c) To verify that the structure or component passes the design criteria. It means to verify that theses calculated/computed stresses do not exceed the corresponding acceptable stresses, the “capacity”.

Some facets of this engineering practices need to be further developed:

Some loads, such as the weight, the wind pressure or the operating internal pressure of a pressure vessel, appear naturally in terms of (generalised) external forces and can be qualified as force-controlled loads. For some other loads, regarding them as of the force-controlled type is highly questionable; this is in particular the case of thermal loads, which should be regarded as strain-controlled. However, in such cases, the most frequent engineering practice, at least until very recently, has consisted in introducing an “equivalent” (generalized) applied force.

Categorisation of a given load as force-controlled or displacement-controlled is crucial because, in simple words, margins under force-controlled loads are linked to the hardening capacity of the structure or component, while margins under displacement-controlled load are linked to its ductile capacity. Consequently, as the ductile capacity is generally much larger than the hardening capacity, criteria associated to displacement-controlled loads can be laxer than those associated to force-controlled loads for the same security objective. For mechanical engineers this categorization is a routine, while this is not the case for the vast majority of civil engineers, who are not at ease with displacement-controlled or strain-controlled loads.

Some loads can be evaluated by a purely deterministic approach (weight, operating pressure), but for other ones a probabilistic approach is necessary. This is the case for those loads that represent effects of natural hazards. For instance, construction standards require that the effects of wind corresponding to a given return period (generally 100 years) are considered in the design process. For earthquakes the widely observed practice is that conventional buildings are designed against a 475-year return period input motion (See e.g. [3]). Engineers do not take care themselves of the evaluation of such events. Wind hazard maps, in terms of velocity or pressure, and seismic hazard maps, in terms of PGA, are provided by standards or by authorities.

Basically, three families of loads are considered in civil engineering: permanent, variable and accidental. Through a series of coefficients that play a role in assuring the security and the serviceability of the structure, they are combined into a series of conventional situations to be considered in the design/verification process. For instance, in the Eurocode [1], the following two load situations should be considered for security verification:

- Dead load multiplied by 1.35
- Dead load + Earthquake.

The first bullet corresponds to a permanent situation. The 1.35 coefficient reflects the associated required minimum margin. (It is not introduced to account for uncertainties in the structural mass; there is another mechanism for that in the standard). The second bullet is corresponding to an accidental situation; the coefficient 1.35 on permanent loads becomes 1.0 when associated with an accidental load (here the earthquake). This 1.35 coefficient explains the little role played by the vertical component of the seismic input motion in the design.

Acceptance criteria are based on Limit States; "Ultimate Limit States" are considered for security verification and "Serviceability Limit States" for serviceability verification. For instance, serviceability criteria limit deflection and cracking of concrete beams.

In the same spirit, mechanical engineering uses a different terminology, with regular and occasional situations, upset situations and accidental situations. Possible failure modes are identified and criteria are established in order to protect against these failure modes with margins. The most iconic criterion is provided by the ASME standard [10] for protection against the formation of plastic hinges and collapse. It reads:

$$B_1 \frac{PD}{2t} + B_2 \frac{M}{Z} \leq k\, S_h\,, \tag{1}$$

where the first term of the left side is the uniform axial stress in the pipe wall caused by the internal pressure and the second one is the maximum axial stress in the pipe wall caused by bending moments (dead load, earthquake …). In case of a non-permanent action, the maximum value over time should be considered (e.g. a water-hammer event has effects on both terms). On the right side, S_h is derived from the tensile curve of the considered steel and k is a coefficient that depends on the type of situation under consideration. Typically, k=1.8 for regular and occasional situations, 2.25 for upset and 3.0 for accidental situations. This criterion is applicable to primary stresses, which is a mechanical engineering terminology for stresses induced by force-controlled loads. Consequently, thermal stresses are not included in this criterion; they are considered in fatigue analysis.

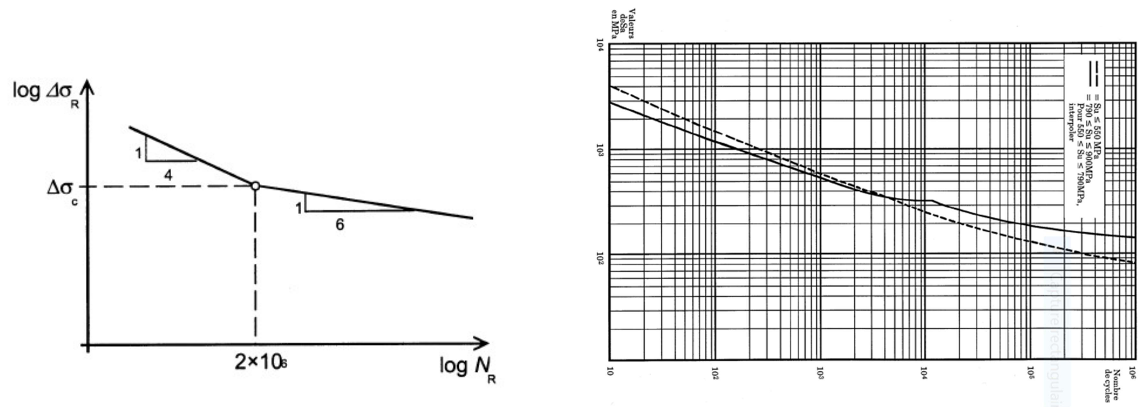

Fig. 1 Fatigue curves for cables used in bridge construction (EN 1993-1-11, [2]) on the left (Fig. 1-a) and for carbon steel (RCC-M 2016 Vol. Z, [11]) on the right (Fig. 1-b)

The effects of cyclic loads or fatigue are seldom considered in civil engineering, with the remarkable exception of bridges. For instance, a standard fatigue curve for cables is presented in Fig.1-a, excerpt from the Eurocode 3 [2].

In mechanical engineering the pattern is different. Cyclic loads result from the regular operation of a facility, especially of power plants. Fatigue analyses are carried out and codified in standards such as ASME in USA [10], RCC-M in France and China [11], PNAE G-7-002-87 in Russia [12] and JEAC4601 in Japan [13] (the later in Japanese, a summary of which is presented in [35]). A typical fatigue curve for pressure vessel carbon steels is presented in Fig. 1-b.

In addition, it is recognized in piping design standards that components can undergo beyond yield cyclic strains. A consequence is ratcheting, defined by the ASME [10] as "a progressive incremental inelastic deformation or strain which can occur in a component that is subjected to variations of mechanical stress, thermal stress, or both". In order to cope with this situation, the purpose of standard criteria is to ensure shakedown, defined by the ASME as follows: "Shakedown of a structure occurs if, after a few cycles of load application, ratcheting ceases. The subsequent structural response is elastic, or elastic–plastic, and progressive incremental inelastic deformation is absent. Elastic shakedown is the case in which the subsequent response is elastic." Although the post-elastic response is explicitly mentioned in the standard, the acceptance criteria are applicable to the outputs of an elastic analysis of the pipe response to thermal and other cyclic loads.

3 Handling seismic loads

When it appeared that earthquake effects should be taken into account in the design of structures and components, the focusing interest of engineers on the evaluation of the expected maximum load paved the way to the response spectrum. We can even wonder whether the engineers' interest for the maximum expected load triggered the idea of representing the seismic input motion in the form of a response spectrum. This was clearly in Biot's mind when he created the concept: *"If the response spectrum of a given earthquake is known, an upper limit for the stresses produced by that earthquake in any structure may be readily evaluated if we know the natural periods and modes of oscillation of this structure."* (Biot, [17], in his conclusions).

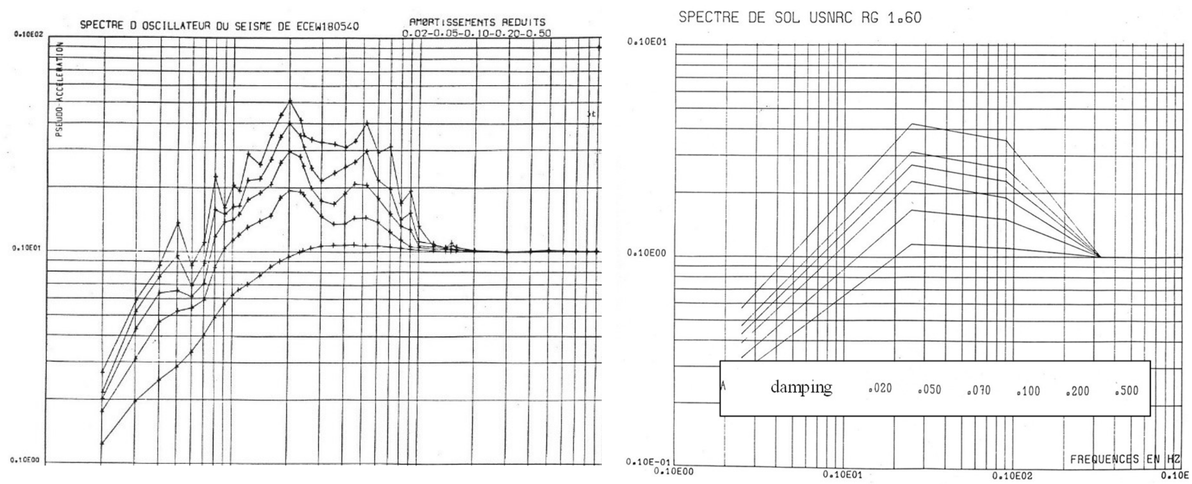

Fig. 2 Response spectrum of a seismic motion (Fig. 2-a, left) and Response spectrum as load case for seismic design, the NRC ground spectrum (Fig2-b, right)

When introducing the response spectrum, a first comment is that it can be plotted from a recorded accelerogram (Fig.2-a); in which case it is actually the response spectrum of a seismic input motion. It can also take the form of a standard spectrum (Fig. 2-b), in which case it should not be regarded anymore as the response spectrum of a seismic input motion, but only as a load case to be used by engineers for the purpose of seismic design. In this regard, although they are unrealistic, flat response spectra are very convenient engineering tools to handle uncertainties in the input motion and in the structural model in a concurrent manner.

In the wake of Biot's pioneering work, the widely spread design engineering practice for conventional buildings consist of

a) running elastic analyses. However, as it is concurrently assumed that structures should respond in the post-elastic regime, a reduced stiffness is considered for concrete structures, accounting for cracking effects.
b) Regarding the seismic action as a force-controlled load (force-based approach).

In this framework, the applied forces are derived from the response spectrum in pseudo-absolute acceleration, while, in addition to the stiffness degradation, the non-linear response effect is taken into account through the so-called behaviour factor (Eurocode 8 terminology[3], designated by q) or the equivalent response modification factor (ASCE 7 terminology [6], designated by R_p) or the ductility index (Japanese terminology according to [28]) The common idea is to reduce the elastically calculated stresses by a factor larger than 1 so as to get "effective" stresses, which are used to verify that the ultimate limit state criteria are met. The larger the ductile capacity of the structure, the larger the value of this reducing factor, which is of the order of 4-5 (depending on the applied standard) for high ductility frame structures. Ductile capacity of structures is categorized on the basis of compliance with global (e.g. structural regularity) and detailed (e.g. number and diameter of stirrups) provisions described in the standards.

A paradox of this approach is that, for relatively flexible structures, the standards assume that the seismically induced displacements in plastic regime are the same as those that are calculated assuming an elastic response. In this respect it means that the seismic load is regarded as of the displacement-controlled type. The current trend in design standard evolution is to promote the use of displacement-based in lieu of force-based approaches. Displacement-based approaches have been developed and implemented in standards for the analysis and verification of existing structures, such as ATC 40 [8], FEMA 273 [9] or Eurocode 8 Part 3 [4].

In the nuclear industry the seismic input motion is generally described in response spectrum form at the design stage, with the exception of the Japanese practice, which uses accelerograms and develops time-response analyses. A comparison of practices is presented in an IAEA Technical Document [16]. The above-mentioned reduction factors are disregarded at the design stage and the structural stiffness is generally calculated without accounting for cracking effects. This practice (which implies that the structural response is actually in the elastic regime for any input motion that does not exceed the design) is questioned by the IAEA in the same Technical Document, the IAEA promoting use of linearization techniques similar to those routinely implemented by geotechnical engineers (see hereunder). This approach is now in evolution; the ASCE 43-05 [7] for the seismic design of structures systems and components of nuclear facilities considers four possible limit states, from essentially elastic to short of collapse, authorizes reduction factors and provides guidance for implementation of displacement-based approach. Regarding the seismic assessment of existing nuclear installations, the IAEA authorizes prudent reduction factors, with values significantly lower than those applicable to conventional buildings [15].

The situation is similar in mechanical engineering. The seismic input motion is described by its response spectrum, which is not a ground response spectrum but a floor response spectrum (FRS). FRS is an output of ground input motion filtering by the supporting structure of the considered mechanical components. For a given installation, multiple FRS are generated corresponding to the multiple floors where equipment is installed. Two examples of FRS are presented in Fig. 3; they are "raw" outputs derived from a time-response of the supporting structure. As opposed to usual ground response spectra, FRS are narrow banded. In practice they are enlarged so as to cover modelling uncertainties in an elastic analysis framework. The seismic response of piping systems is calculated in elastic regime and the maximum values of seismically induced moments are derived from the applicable FRS. The maximum seismically induced moments are incorporated into M in the above equation (1), which means that seismically induced stresses are regarded as primary (force-controlled load). The design seismic load is regarded as accidental, resulting in k=3 in the application of equation (1) for this situation.

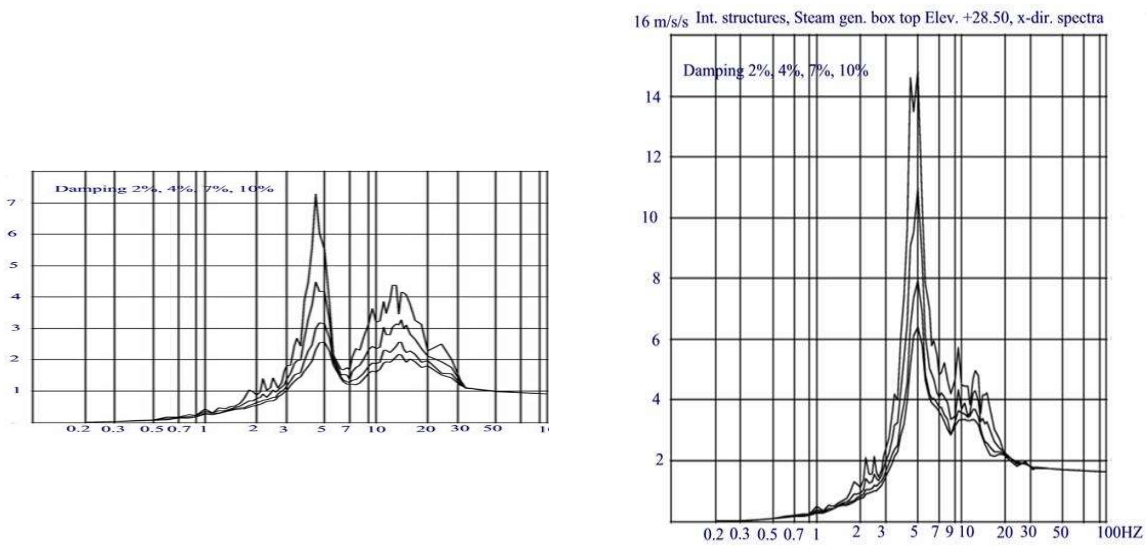

Fig. 3 Floor response spectra at two floors of the same building, 5 m above ground (left) and 28.5 m above ground (right), plotted at the same scale

A feature of earthquake engineering for equipment is that strong non-linear behaviours are encountered, such as shocks resulting from gaps in the supporting system of primary loops and in fuel assemblies, sliding of bridge cranes on their rails … In the current practice, dealing with such situations implies that FRS compatible accelerograms are generated, which may be an issue.

Contribution of earthquake to fatigue is not considered in civil engineering. In mechanical engineering, it is inclusively taken into account by incorporating in the fatigue analysis the contribution of a certain number of cycles (usually 20) with an amplitude equal to the calculated maximum of the seismically induced stresses. Ratcheting effects are ignored.

At this stage it is worth mentioning some geotechnical engineering practices. To account for non-linear effects in soils, which appear for very small strains (of the order of 10^{-5}), geotechnical engineers implement as a routine the linearization technique proposed by Seed and Idriss [21]. The method is based on "degradation curves" that account for the degradation of the soil shear modulus due to seismically induced shear strains and the corresponding increase of the damping ratio. The soil profile model is updated in a rapidly converging iterative process. As the model remains linear there is no issue in using a response spectrum as representation of the input motion. The method is applicable for seismically induced strains up to the order of 10^{-3}, the validity domain being larger for a larger soil plasticity index. This approach provides a reasonable evaluation of the effects of the non-linear soil response on the horizontal seismic input motion in a soil profile. However, as the model remains linear, it cannot address non-reversible effects such as seismically induced settlement or pore pressure built-up (liquefaction) of sandy soils.

Strong motion duration plays an important role in the liquefaction phenomenon, which is very sensitive to the number of strain cycles experienced by the soil. Criteria for liquefaction are empirical, based on SPT or CPT or pressuremeter outputs of *in situ* tests for the capacity and calculation of seismically induced shear strains in the considered soil profile for the demand. Duration of the input motion is indirectly taken into account by the fact that liquefaction criteria are Magnitude dependent

4 Pro and cons of the response spectrum

From the engineer viewpoint, as it is practically impossible to predict the seismic motion that would occur on a site, the response spectrum is an effective option to describe a load case to be used for seismic design. Calculating the average of a series of possible input motion would not make sense, while calculating an average response spectrum does. This has to do with the fact that response spectrum derivation is not a linear application. Another candidate for representing the effects of earthquakes should offer similar possibility.

In addition, a great plus of the response spectrum for engineers is that it keeps them in their professional comfort zone. Even qualified as “pseudo”, an acceleration times a mass is a force, and designing a structure or a component against an applied force is a very familiar situation, which engineers feel comfortable with.

Said that, it should be discussed whether an evaluation of the maximum of the seismic response pertains in terms of seismically induced damage. The answer is clearly yes for concrete shear walls. It is established that the maximum drift value experienced by a shear wall is a reliable indicator of its degradation [30]. If an equivalent stiffness should be calculated for a shear wall, it would be the secant stiffness corresponding to the maximum response.

On the other hand, the answer is clearly no for piping systems and more generally for steel structures. Outputs of experiments carried out in the 80’-90’ in the USA [31], in the 2000’ in Japan [32], and still ongoing in India [33] conclude that the failure mode of pressurized piping systems under strong seismic input motion is fatigue-ratcheting. The failure mode aimed by the ASME and other similar standards (Equation (1) above) is never observed on pressurized pipes. This experimental result questiones the conventional categorisation of seismic load as of the force-controlled type [36] and raises a renewed interest on the subject. The Japanese industry has made a proposal for a dramatic change in the standards: to ignore seismically induced moments in Equation (1) and to better take them into account in fatigue analysis [34]. Such a change, not yet accepted by the Japanese regulator, would focus on the duration of the strong motion and on its effective amplitude over this duration instead of focussing on its maximum.

To large extent the case of concrete frames is similar to steel structures, although the plastic models applicable to steel structures should be amended when dealing with reinforced concrete. The conclusion would also be that the maximum of the response, such as derived from the response spectrum, is not representative of the seismically induced damage.

When dealing with industrial facilities an additional difficulty appears about the calculation of the floor response spectra. Features of response spectra are so that there is no mathematical approach that enables a simple transfer from the ground response spectrum to the floor response spectra. Several attempts were made to create engineering tools that could fix this issue, but there is no satisfactory method at the moment. A frequent practice is to select or create a series of accelerograms that fit the ground response spectrum, to calculate the time-response of the structure and to convert the floor responses into floor response spectra. On the one hand this is a heavy methodology with questions about the selection/generation of ground input motions, but on the other hand it offers the option of integrating possible non-linear behaviour of the supporting structure.

Response of piping systems in the plastic regime is relatively weak non-linearity that could be addressed by linearization techniques and keep the response spectrum as representation of the input motion. To a certain extent sliding of bridge cranes on rails could also be treated in a similar manner because Coulomb's friction is the rheological model of plasticity. For other components it is not the case because they exhibit strong non-linearities. For instance, the supporting system of the primary loop include gaps, large enough for free thermal expansions but resulting in strong shocks between the primary loop and the neighbouring civil structures in seismic situations. Some engineering recipes have been proposed to address the situation with a seismic load expressed in the form of a response spectrum, but they are not actually satisfactory. Although they raise numerous questions, it seems that time-response analyses are unavoidable in such situations.

An issue frequently presented about response spectra is the difficulty to generate time series that fit a given response spectrum. It is true that it is a difficult exercise and that there is no fully satisfactory method to resolve the problem. However, although there is some mathematical concern with the response spectrum concept, it should be recognized that practical issues are, to a large extent, due to the unrealistic shape adopted for response spectra regarded as load cases for the purpose of seismic design. Generating accelerograms compatible with a response spectrum such as presented in Fig. 2-b, or a similar standard spectrum, does not make sense because the spectrum is deemed to be the mean plus some standard deviation of motions with a given PGA and because of its corner frequencies. The author opinion is that the engineering practice should evolve so that generating such accelerograms is precluded.

5 Another type of response spectrum

In the conventional building industry, using displacement-based approaches is a more and more popular practice. Main features of these approaches are

- a pushover curve that expressed the relationship between the top displacement of the building and the shear force at the base, converted into an acceleration
- a representation of the input motion in the Acceleration-Displacement Response Spectrum (ADRS) format, in which the structure frequencies or periods are represented by radial lines.

Once the pushover curve and the ADRS are plotted in consistent in a consistent manner, their intersection provides an estimate of the inelastic acceleration (strength) and displacement demand in the structure under the considered ADRS.

In a first approach, initially developed by Freeman [26], the ADRS consists of a series of curves corresponding to different damping values (exactly as for a conventional response spectrum). The core of the method requires that an effective damping is associated to the ductile demand, so that the retained intersection is corresponding to a common damping value (Fig. 4-a, [8]).

In a second approach, implemented in FEMA 273 [9] and further developed by Fajfar [27], the ADRS consist of a series of curves representative of an inelastic response spectrum, so that the retained intersection is corresponding to a common ductile demand. To a large extent this method is similar to the Newmark et Hall approach [25].

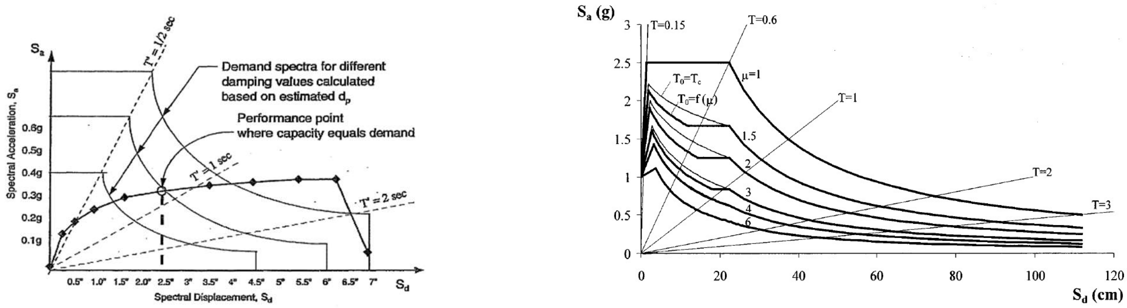

Fig. 4 Response spectrum in ADRS format for different damping ratios (Fig.4-a, left [8]), and for different ductile demands (Fig. 4-b, right, [27])

At this stage we introduce the inelastic response spectrum. Let's consider a given accelerogram g(t) and oscillators of a given damping ratio (say 5%) and of variable frequencies. We should answer the following question: Assuming that these oscillators have no ductile capacity, what should be their minimum elastic limit so that they do not collapse under g(t)? Of course, the answer is the conventional 5% response spectrum (either in conventional or in ADRS format). Now we should answer a second question: Assuming that these oscillators have a given ductile capacity (say μ=2), what should be their minimum elastic limit so that they do not collapse under g(t)? The output is obviously a series of values lower than in the previous case, which values all together are the inelastic response spectrum for μ=2. The procedure is repeated for different ductile capacities so as to plot the inelastic response spectrum of g(t).

Actually, we should not speak of "the" inelastic response spectrum of g(t) because we should indicate whether the oscillators have a hardening capacity (bilinear or other form) and more importantly elicit the constitutive relationship: is-it a plasticity model or a damage model or a mix of them? Every choice results in a different inelastic response spectrum. Some authors have proposed methods to derive inelastic response spectra from elastic ones, starting by Newmark et Hall [25] for elastic-perfectly plastic oscillators. It is thinkable that similar methods are developed in the future to account for more sophisticated or more realistic constitutive relationships and that the response spectrum get a renewed interest in its inelastic format.

So far, the inelastic response spectrum has not found its way in mechanical engineering although the actual behaviour of piping system is much better represented by plasticity models than the concrete structures behaviour is.

6 An alternative representation of the seismic input motion

Before the Eurocodes uniformize engineering practices across Europe, the Portuguese seismic design code presented the design seismic input motion in the form of a Power Spectral Density (PSD) [14]. Different PSD shapes were considered corresponding to different seismic scenarios (Type 1 for moderate magnitude earthquakes and Type 2 for large magnitude distant earthquakes) and different types of site. This Portuguese approach is illustrated in Fig. 5, where a table describing PSDs for different types of site in Lisbon area (for Type 1 earthquakes) is presented on the left and the PSD for rock sites on the right-hand side. By calculating its integral and considering a peak factor of 3 [23], we conclude that the presented PSD is corresponding to a 0.18g PGA.

Densidades espectrais de potência de aceleração das componentes horizontais para a zona A, S(f)

(f : Hz   S(f) : $(cm/s^2)^2/Hz$)

Acção sísmica tipo 1

Terreno tipo I		Terreno tipo II		Terreno tipo III	
f	S (f)	f	S (f)	f	S (f)
0,04	0	0,03	0	0,02	0
1,05	250	0,9	220	0,75	190
2,1	360	1,8	300	1,5	240
4,2	360	3,6	300	3,0	240
8,4	160	7,2	130	6,0	100
16,8	50	14,4	40	12,0	35
20,0	20	20,0	16	20,0	12

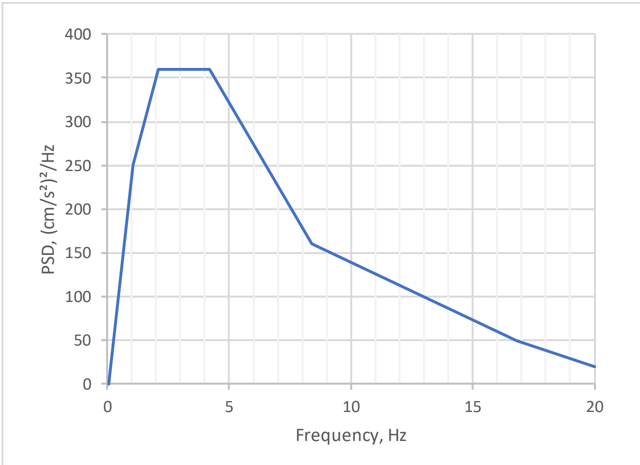

Fig. 5 Table of seismic input motion PSD values for different types of sites in Lisbon area and for Type 1 earthquakes [14], and the plotted rock site PSD

As of the late 50', Kanai [18] and Tajimi [19] proposed to represent seismic ground motion in the form of the absolute acceleration of a filtered white noise, with a filter frequency related to the ground stiffness (from rock to soft soil) and a filter damping ratio of the order of 0.5-0.6. It appears that this model has a too large low frequency content and provides time series that are far to meet natural values of the non-dimensional number PGAxPGD/PGV² such as reported for instance by Betbeder [29]. An additional low frequency filter was added by Clough and Penzien [24], resulting in a model widely used at the moment.

As opposed to response spectra, generating time series associated to a given PSD is not a mathematical issue. However, these time series should be multiplied by an envelope function, which incidentally carries the duration of the input motion. The resulting signal is weakly stationary. An additional step towards realistic representations of seismic motions could be to model non-stationary input motions, as proposed by Priestley as of 1965 [20].

The major advantage of representing the input motion in the PSD format is that we can easily calculate the PSD of any output quantity, just by multiplying the input PSD by the transfer function and its conjugate. For instance, we can derive PSDs of bending moments in all beams and columns. The transfer function between the input and a given bending moment is a combination of modal transfer functions, very similar to the combinations of modal responses in the current engineering practice.

In particular we can directly derive floor PSDs and, after calculation of transfer functions in a given mechanical component, calculate the PSD of any quantity of interest such as bending or torsion moments in a pipe.

Because of its flexibility and effectiveness, the approach seems very attractive. However, there is some price to pay for that:

a) First the structural behaviour should be linear. It is not necessary that the considered structure behaves actually in a linear manner, but at least a reliable linearization technique should be available. This is the case in geotechnical engineering, but not in civil or mechanical engineering at the moment. Actually, methods such as the capacity method can be suitable for the design or verification of the supporting structure but they are not suitable for calculating the transferred input motion.
b) Second, the outputs cannot be directly used by engineers in a design or verification procedure. For the civil engineer the bending moment PSD is useless when deciding how much reinforcement should be implemented in a beam; he or she would like to know the (expected) maximum bending moment. A simple way could be to calculate the bending moment standard deviation by integrating its PSD, and to multiply it by the appropriate peak factor.

Regarding b) a more sophisticated approach would be, from its PSD, to derive the distribution or repartition function of the bending moment maximum value according to Vanmarke [23]. Then, the design bending moment could be defined by an accepted probability of exceedance. This latest step is likely the most difficult to address: How to calibrate an accepted probability of exceedance of a number that is an output of linear analysis? It is clear that for a given security objective, the larger is the ductile capacity of the structure or component the larger should be the accepted probability of exceedance.

Anyway, this last step would be necessary only in a framework where the expected maximum value controls the design/verification of structures and/or components. We may think of engineering procedures in which this maximum value is not necessary and the design/verification is based, for instance, on the standard deviation of the bending moment and on the strong motion duration. It would obviously make sense for dealing with fatigue ratcheting. Even in case an evaluation of the maximum is required, it could be based on the standard deviation and an inclusive peak factor value provided by a standard.

7) Conclusion

There is likely no representation of the seismic input motion that should be regarded as the best for engineers in an absolute manner. We should first consider the type of damage generated by a strong motion in a structure or a component and identify the mechanical parameters that control this damage: “effective” (generalized) stress or strain, maximum stress or strain (ductile demand), number of cycles, effective/peak displacement, velocity or acceleration, shocks… And then we should discuss which seismic input representation is the most convenient with the objective of evaluating those parameters.

For years the response spectrum has ben regarded by engineers as the best representation because it enables them to include the seismic load in the design process as a force-controlled load and to estimate the induced expected maximum demand. Drawbacks of the response spectrum appeared step by step with the sophistication of earthquake engineering and the needs of earthquake engineers.

For conventional buildings, an example of PSD representation is available in the past Portuguese standard. At the moment, developments of the response spectrum towards the concept of inelastic response spectrum, which appeared some decades ago, seem to attract a renewed interest in conjunction with the development of the displacement-based approach.

In the context of industrial facilities, and particularly for the nuclear industry, the possibility of representing the seismic input motion in the form of a PSD associated to a strong motion duration should be investigated with the perspective of setting up a renewed engineering practice, which could be used as a routine for designing structures and components as well as for transferring the ground input motion towards the equipment.

When input motions in the form of time series are necessary in the design/verification process, generating them would be easier and less controversial than it is when starting from response spectra, at ground level as well as at floor levels. In case a more realistic representation of the input motion is necessary, it could be possible to use mathematical models such as evolutionary PSD, or to search for appropriate input motions in strong motion databases.

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