QUANTIFICATION OF DESIGN RULES BASED ON THE ASSESSMENT OF EARTHQUAKE RISKS

by

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Synopsis

Several aspects contributing to an improved rationalization of design codes are discussed. An overall treatment of structural safety problems is recommended, seismic loads being included in this general framework.

Alternative methods for the idealization of seismic loads are indicated. A computer program is used which allows the conversion of power spectral densities into response spectra.The statistical definition of seismic loads is exemplified.

Results on the statistical study of structural safety are indicated from which it is possible to derive practical design rules for combination of loads and for considering non-linear behaviour.

1 - INTRODUCTION

Practical design rules and, in particular, structural codes are benefiting but little from the very many advances in recent research (1). This is especially true in Earthquake Engineering.

Important progresses in the idealization of seismic vibrations, in their statistical quantification, in the definition of structural response, in the combination of different loading types and in the statistical definition of limit-states have been achieved. However even the most recent proposals for the unification of Earthquake Engineering design codes (2) maintain traditional concepts and innovate very little.

On the other hand there is a tendency to deal with earthquake safety problems separately from the general framework of structural design for other types of loads. From the user's standpoint, structural safety is comprehensively appraised without minding the causes of insecurity. The user is interested in the efficiency of structures so that his life and the lifes of his community fellows, his economy and the economy of his society are not affected by collapses and misfunctions. The task of fulfilling this aim devolves on code writers, designers and builders.

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The need to rationalize design leads to decision rules based e.g. on a limitation of the yearly rate of accidents. A convenient way to achieve this limitation consists in adopting lower limits for the probability of not reaching limit-states (probability of efficiency) within a standard interval of time (lifetime).

Consequently, unified concepts should be used for defining the different types of loadings.Furthermore it is important to quantify loads, structural response, and safety margins separately and in a compatible way.

2 - QUANTIFICATION OF SEISMIC VIBRATIONS

The statistical definition of seismic vibrations should be based on suitable idealizations including few parameters. Of these parameters some may be taken as deterministic, others as random variables.

A convenient idealization of seismic vibrations is obtained by using the basic concepts of the Theory of Random Vibrations. Although this has been often asserted in the last quarter of century, these concepts are not yet widely used, particularly in codes.

Assume each component of the soil vibration to be a sample of a stationary Gaussian process. Such a process is defined by its power spectral density of acceleration S(f), and its duration. The mean value of the acceleration being zero, the variance is given by

$$\bar{a}^2 = \int_0^{\infty} S(f)\, df \quad \ldots\ldots\ldots\ldots\ldots \quad 1)$$

f being the frequency. This integral indicates the overall power content of the vibration.

Earthquake duration influences but little the maxima of the response. Consequently duration can be taken as a deterministic variable, and a standard value — for instance 30 seconds — can be used.

The intensity of seismic vibrations can be measured in different ways: i) Housner's seismic intensity, ii) root-mean-square acceleration, $\bar{a}^2$, and iii) maximum acceleration, a_{max}. It is easy to show that all these measures are approximately equivalent and thus can be easily related (3). Maximum acceleration is recommended in practice as being the most intuitive. However owing to the inaccuracy of the direct determination of the maximum acceleration on an accelerogram, its value should be indirectly deduced from Housner's intensity or the power spectrum. It can be assumed with a sufficient accuracy that the mean value of the maximum acceleration is related to the spectral density of acceleration by

$$a_{max} \simeq 2.9 \sqrt{\overline{a^2}} = 2.9 \sqrt{\int_0^{\infty} S(f)\, df} \quad \ldots\ldots 2)$$

Strong earthquakes being rare events, the randomness of their occurrence can be defined by the distribution function of their maximum accelerations referred to a given lifetime (for instance 50 years) Thus, the distribution function $F(a_{max})$ indicates the probability of occurrence of earthquakes with intensities smaller than a_{max} during the lifetime.

Although a_{max} gives a good overall indication of the intensity, this quantity cannot adequately measure vibration effects on different types of structures. These effects depend not only on the area but also on the shape of the power spectral density of acceleration diagrams, S(f). Thus codes should specify this shape, which depends on several earthquake characteristics e.g. generating mechanism, magnitude, distance to the focus and propagation conditions. Yet, at a given location the shape of spectral diagrams can undergo wide changes. But considerable changes have only minor consequences for the response of the structure provided the total energy content remains constant (4). Consequently codes may indicate standard shapes corresponding to typical situations.

In order to make easier the practical use of power spectral diagrams these should be conveniently related to acceleration response spectra. A computer program available at the LNEC performs this conversion automatically (5).

Fig. 1 indicates the types of spectral density of acceleration diagrams included in the draft of the Portuguese code on loadings (6). The curved lines in Fig. 2 are the corresponding response spectra computed according to the mentioned program assuming a duration of 30 s.

In order to simplify the code, only three seismic zones and three soil conditions are considered. It would have been of interest to distinguish several soil conditions and to adopt different spectra in each zone, according to the possible distances to the epicenters corresponding to close and far earthquakes. In fact close earthquakes are much richer in high frequencies than far ones. However, the information available was deemed insufficient for specifying these differences.

The spectral densities of acceleration indicated in Fig. 1 refer to the zone of highest seismic intensity, zone A. In zone B spectral densities are one fourth those of zone A. In zone C seismic loads are considered in a very simplified way. The types of diagrams depend on soil conditions: type I, rocks and hard soils; type II, intermediate soils; type III, soft soils.

The straight lines in Fig. 2 give the simplified response spectra

to be used in design. The draft code separately indicates the ductility factors, which depend on the structural types.

3 - QUANTIFICATION OF THE SEISMIC RISK

Seismic history, seismological data and geological information should be combined to derive the statistical distribution of seismic intensities in the lifetime.

According to international recommendations ((7) and (8)) loadings should be defined by their characteristic values. The adoption of 95% fractiles in a 50 years' lifetime leads to a 1000 years' return period. Yet statistical data include all possible directions, and the loading intensity is to be defined in one direction only. By this reason it is justifiable to base design on values corresponding to return periods intermediate between 100 and 1000 years.

Information on the seismic risk in a given region can be expressed by isointensity lines (referred to given return periods). From the location an magnitude of the earthquakes felt in the region and by assuming a propagation law, it is possible to compute the intensities in a mesh of points. By fitting statistical distributions to these intensities, fractiles corresponding to the assumed return periods are obtained. Consequently isointensity lines can be plotted by interpolation. These results can be further improved by including information on subjective intensities and geological conditions.

This technique of computing the seismic risk is simple and has a good theoretical foundation (9).

Figs. 3 and 4 show the gross seismic risk maps of Portugal for 100 and 1000 years' return periods obtained as indicated above (10). Isointensity lines satisfactorily agree with the previous definition of seismic zones.

The spectral density of acceleration diagrams indicated in Section 2 were chosen so as to fit the results of seismic risk studies.

Table I indicates the maximum soil accelerations which correspond to the assumed spectral density of acceleration diagrams, computed according to Expression 2) and the 100 and 1000 years' return period accelerations derived from the seismic risk studies.

4 - PROBABILITY OF EFFICIENCY OF A STRUCTURE. COMBINATION OF LOADS

The efficiency of a structure or of its elements should be checked by imposing lower limits to the probability of efficiency during lifetime (or upper limits to the probability of failure). For computing these

probabilities one should distinguish loadings from limit-states.

In a space with generalized load-effects and generalized displacements as coordinates ((11) and (12)), densities of probability can be defined by indicating the probability $f_s(\underset{\sim}{X})$ of the loading falling in an elementary region of this space during lifetime. On the other hand, limit-state conditions in this space should also be established by indicating the probability of failure of the structure, $F_u(\underset{\sim}{X}_u)$ for $\underset{\sim}{X}$ ranging from zero to the considered point.

The probability of failure is

$$P_f = \int_{\Omega} f_s(\underset{\sim}{X})\, F_u(\underset{\sim}{X}_u)\, d\Omega \quad \ldots\ldots\ldots\ldots 3)$$

the integral being extended to the whole space Ω .

In the example presented in Fig. 5 the generalized load-effect vector has two components: a bending moment, M , and an axial force, N . Thus, two regions can be defined in plane (M , N): one within which the structure is efficient and another in which the structure is inefficient. The border between these two regions is defined by the distribution function $F_u(M_u, N_u)$.

During the reference lifetime, the structure is under the action of different loads due to different causes, e. g. permanent, superimposed, wind and earthquake loads. Combinations of these loads correspond to points in the plane (M, N). The probability of the loading vector falling within an element of the plane during lifetime is defined by the density of probability $f_s(M, N)$.

The probability of failure is given by the integral of expression 3). This integral takes significant values only in the regions where the edges of the loading curves intersect the border of the inefficiency region. Usually each edge corresponds to a type of loading. The directions along which the edges intersect contain a central point, $\underset{\sim}{\bar{X}}$, which corresponds to the sum of the means of the statistical distributions of the instantaneous load-effects due to the different types of loadings. The tails of the marginal distributions along each direction are affected mainly by the type of distribution and by the variance of the load-effect in the direction considered. Thus the two-variable problem can be transformed into several one-variable problems and simplified design rules can be obtained.

5 - NON-LINEAR BEHAVIOUR. DEFINITION OF RESISTANCE AND ULTIMATE DEFORMABILITY

In the usual cases of non-linear behaviour a formulation in terms of load-effects alone does not yield satisfactory solutions for structural

safety problems. In fact reaching the ultimate values of the load-effects may not indicate that limit-states have been attained. Consequently safety must be studied in a space having for coordinates not only generalized load-effects but also generalized-displacements, such as curvatures, rotations or displacements ((11) and (12)).

Two main lines can be followed for simplifying this problem: i) to check the safety directly in terms of generalized-displacements; or ii) to introduce the concept of ductility relating generalized load--effects and generalized displacements.

In any case it should be understood that the quantification of the limit-state of rupture implies an accurate definition of ultimate generalized-displacements. The information at present available for this definition is insufficient so that further research in this field is strongly recommended (11).

Fig. 4 exemplifies a moment-displacement diagram where the distribution function of the ultimate displacement $F_u(a_u)$ is indicated. The probability of failure is given by

$$P_f = \int_0^{\infty} f_s(a)\, F_u(a_u)\, da \qquad \ldots\ldots\ldots\ldots 4)$$

where $f_s(a)$ is the density of probability of the maximum displacements due to the loadings during the lifetime. If ultimate displacements are suitably transformed into ultimate load-effects and displacements due to the loads are also substituted by bending moments, the probability of failure can also be computed by performing the integration along the axis of the bending moments. In deterministic terms this simply corresponds to introducing the concept of ductility factor, $\mu = a_u/a_e$.

By putting together Figs. 5 and 6 it becomes clear how the problems of load combination, non-linear behaviour and definition of ultimate displacements can be combined. In this way practical design rules can be derived on a sound basis.

For the quantification of design rules it is particularly important accuratety to define ultimate deformabilities and resistances taking into account load repetition effects. Theoretical approaches for this purpose have used Manson-Coffin's type criteria (13). Yet the experimental evidence supporting these criteria is slender, particularly in the case of reinforced concrete structures. Anyhow it is deemed very important that codes explicitly indicate ultimate deformabilities, or the equivalent ductility factors, within the range of their real values. This provided seismic loads are also suitably quantified.

6 - CONCLUSIONS

The need to improve design codes by including results of recent research is emphasized.

A unified treatment of the different types of loadings including the seismic ones is suggested.

Theoretical results on the idealization and definition of seismic loads and on the generalized formulation of structural safety problems are presented. The practical application of these results is recommended.

Future progress strongly depends on further studies for defining the resistance and ultimate deformability of structures acted on by repeated loadings of the seismic type.

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TABLE I - Maximum soil accelerations, gal.

Seismic zone	Computed by Expression 2)			Derived from seismic risk studies	
	Soils Type I	Soils Type II	Soils Type III	100 years' return period	1000 years' return period
A	90	120	140	80	200
B	45	60	70	60	120

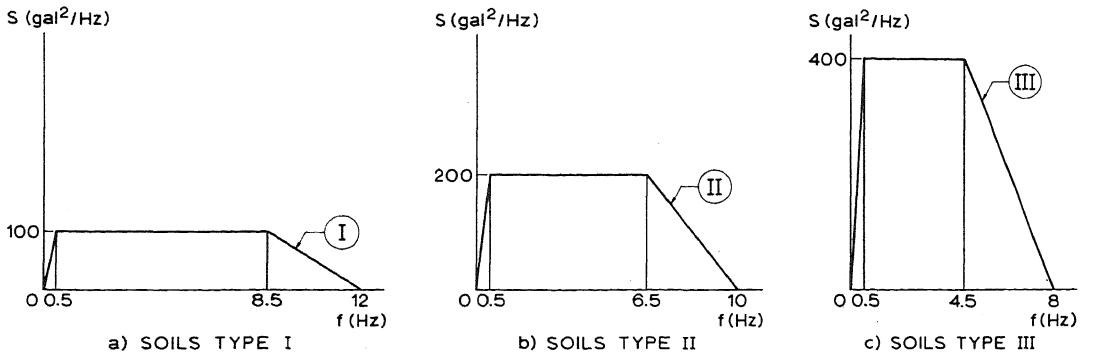

Fig. 1 - Power spectral density of acceleration diagrams of Zone A .

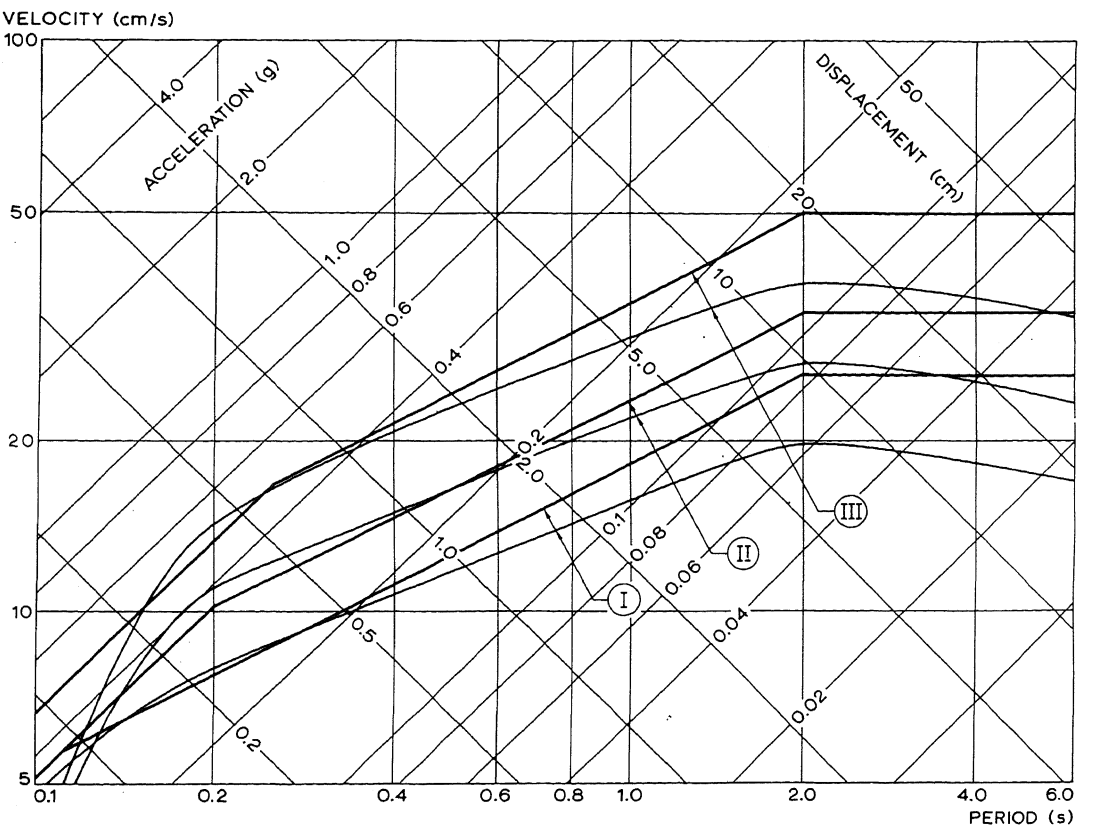

Fig. 2 - Response spectra of Zone A.

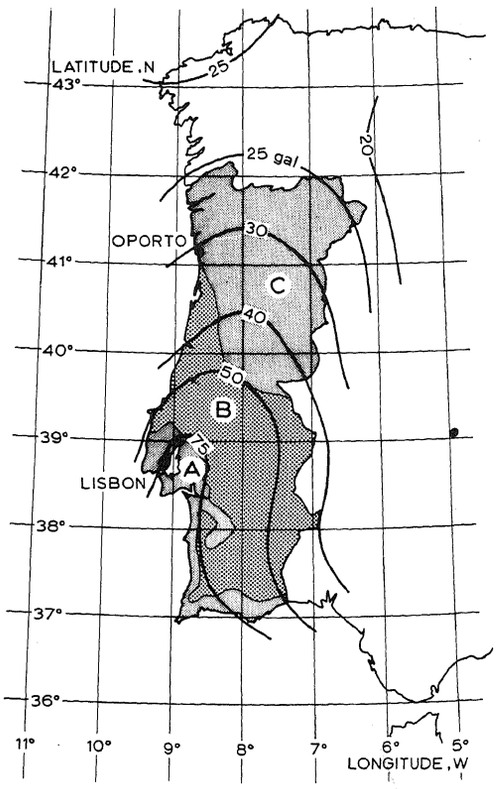

Fig. 3 - 100 years' return period accelerations.

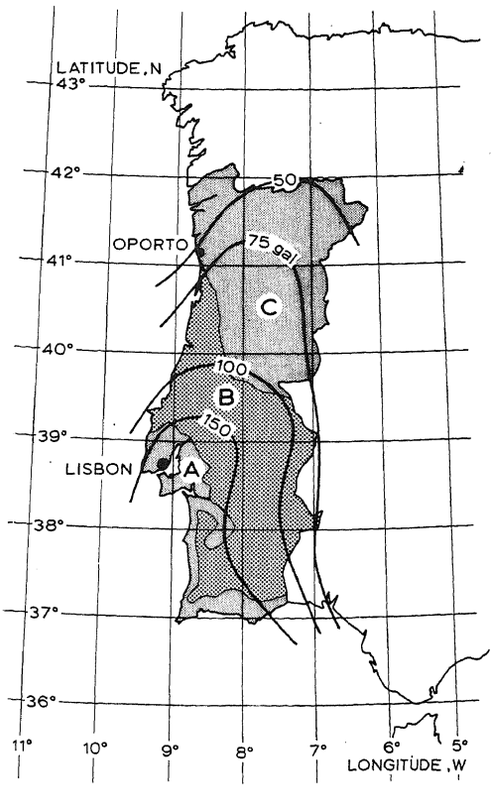

Fig. 4 - 1000 years' return period accelerations.

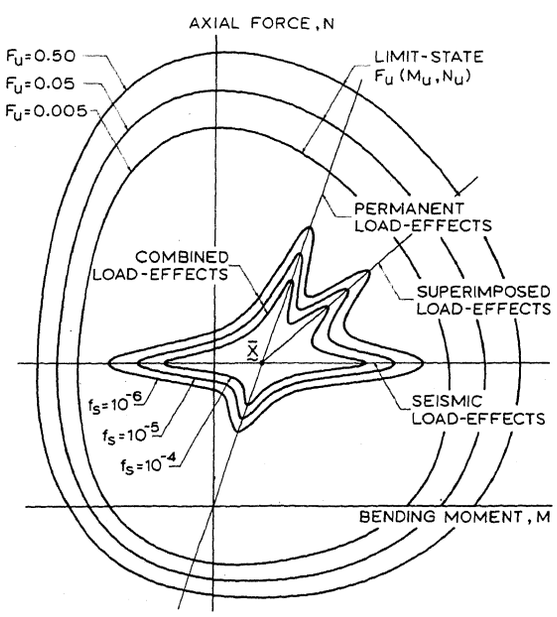

Fig. 5 - Statistical representation of the loadings and the limit--state in an axial force-bending moment diagram.

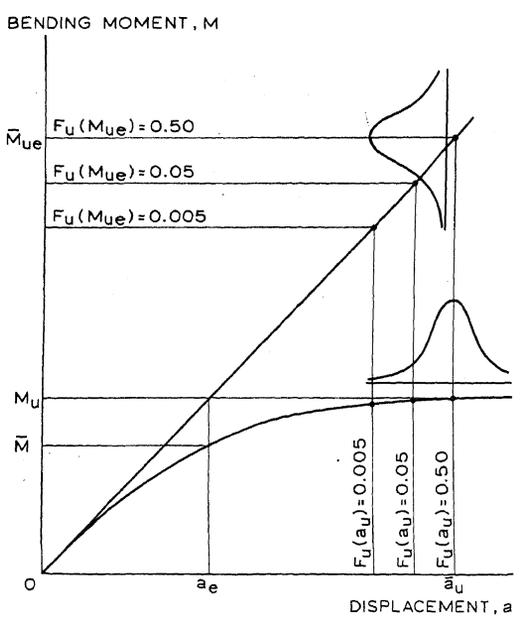

Fig. 6 - Statistical representation of the limit-state in a bending moment - displacement diagram.