PROBABILISTIC DYNAMIC EARTHQUAKE ANALYSIS OF NUCLEAR POWER PLANTS

by

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SYNOPSIS

To satisfy the requirements of probabilistic design, a modal frequency response analysis method incorporating stationary Gaussian random vibration theory is developed for the dynamic analysis of elastic structures subjected to three dimensional earthquake excitation. A direct random analysis approach to floor response spectrum problem is achieved such that, the need for analysis in the time domain is completely avoided. Due to the practical difficulties met in determining the site dependent acceleration power spectrum of the ground motion, for widening the applicability of the method, a direct conversion possibility between acceleration power spectrum and velocity response spectrum is established. The feasibility of using a generated average power spectrum is demonstrated on a sample structure.

INTRODUCTION

Because of the unpredictable nature of earthquakes, the safety of bold civil engineering structures which are to be built in seismic zones is of primary importance. Among these structures, careful attention must be paid to nuclear power plants with their very special problems. Many attempts have already been made to establish design methods and safety criteria for the seismic design of certain types of structures. Providing the dynamic characteristics of a structure are known, it is possible to determine the absolute amplitudes of the responses under a prescribed excitation by means of normal mode analysis methods. Because of the unpredictable nature of earthquake ground motions, the main difficulty in calculating the responses of a structure is to define the excitation itself. Therefore most designers have used an acceleration time history of a recorded past earthquake for response calculations, assuming that the earthquake excitation is a deterministic transient phenomenon. As already mentioned, however, earthquake ground motions are completely unpredictable and random in nature. A particular past earthquake can not be used as a basis for the seismic design of structures to withstand future earthquakes, since it will never exactly recur again. It is necessary that the phenomenon having random nature is described in terms of probability statements and statistical averages(ref.1).

RANDOM RESPONSE ANALYSIS

To satisfy the requirements of probabilistic design, a modal frequency response analysis method incorporating random vibration theory is developed and a series of computer programs has been prepared for the dynamic response analysis of elastic structures subjected to earthquake ground excitation.

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This method is developed and documented in Ref. 2. It is based on the well known relationship between the power spectral densities, $S_x(f)$ of the response, and $S_p(f)$ of the stationary random excitation source P(t):

$$S_x(f) = |H(if)|^2 \cdot S_p(f) \qquad \ldots(1)$$

where H(if) is the complex frequency response "transfer function" of any physical variable x(t). The practical application of this relationship to the response calculation of structures subjected to earthquake excitations requires the determination of the dynamic characteristics of the structure and of the acceleration power spectral density of the earthquake ground motion.

Having the power spectral density of the response using the above relationship for a given earthquake acceleration power spectral density, the other statistic and probabilistic values of the response can be obtained by using the following relationships:

Variance (mean square response)

$$\langle x^2(t) \rangle = \sigma_x^2 = \int_0^\infty S_x(f)df \qquad \ldots(2)$$

Standard deviation (RMS response)

$$\sigma_x = \left(\int_0^\infty S_x(f)df \right)^{1/2} \qquad \ldots(3)$$

Mean maximum response for the earthquake duration T

$$\bar{x}_{max} \ (T) = n \cdot \sigma_x \qquad \ldots(4)$$

where n, defined as the "magnification factor", is a function of earthquake duration and the shape of power spectral density function of the response.

RESPONSE TO THREE-DIMENSIONAL EARTHQUAKE EXCITATION

NEWMARK, et.al. (Ref.3) have indicated that the earthquake motions occur in all three directions simultaneously and with equal probability but without consistent relations among the motions in the various directions. That means it is possible to assume that the three dimensional earthquake ground motions are statistically independent. If excitation sources are statistically independent, the theory of random vibrations (ref.1) states that the power spectral density of the total response is equal to the sum of the power spectral densities of the responses due to individual sources. The application of this rule to three dimensional earthquake excitation is straightforward, such that, firstly the response power spectral densities for each orthogonal component of excitation are computed, then the total response power spectral density is obtained as the sum of those individual response power spectral densities. This procedure allows the application of different earthquake acceleration power spectral densities and different intensity ratios for each orthogonal components of ground excitation.

SEISMIC DESIGN OF EQUIPMENT

A very important requirement in the seismic design of nuclear power plant structures is to supply suitable information about the absolute floor accelerations for the seismic design of equipment. The presented random analysis method offers a direct approach to the seismic design of equipment. Replacing the "ground motion-structure"-system by the "floor motion-equipment"-system, the power spectral density of the equipment response can be determined and the probable mean maximum responses of the equipment items can be calculated. This procedure for the seismic design of equipment is consistent with the seismic design of the supporting structure, completely in frequency domain and eliminates the tedious time domain analysis. Moreover, if the equipment designer requires the floor response spectra, the presented random analysis method has also the capability of supplying that floor response spectra.

ACCELERATION POWER SPECTRAL DENSITY OF EARTHQUAKE

The presented method is based upon the assumption that a specific site dependent design acceleration power spectral density of earthquake can be obtained considering the seismic activity of the region and tectonic and geologic characteristics of the construction site. The method of analysis is arranged in such a way that the distribution of power of the ground acceleration, throughout the frequency range considered, can be represented by an average normalized acceleration power spectral density function, which reflects the dynamic characteristics of the foundation. This average normalized acceleration power spectral density can be multiplied by an intensity factor, which represents the seismic activity of the construction site.

Due to the practical difficulties met in supplying the site dependent acceleration power spectral density of the earthquake ground motion, in order to widen the applicability of the method presented, a direct conversion possibility between acceleration power spectral density and velocity response spectrum is established. Theoretical basis of this conversion can be stated considering the random response of a narrow-band oscillator excited by a source having a broad-band power spectral density (ref. 4). Starting from that basis, the first approximation to the exact earthquake acceleration power spectral density, $S_{\ddot{u}_g}(f_r)$, corresponding to a given earthquake velocity response spectrum, $R_V(\zeta, f_r)$, can be obtained as follows:

$$S_{\ddot{u}_g}(f_r) = 16\pi\zeta f_r \frac{R_V^2(\zeta, f_r)}{n^2} \qquad \ldots(5)$$

where, ζ is the damping ratio, f_r is the frequency and n is the magnification factor defined by equation (4). Using this power spectral density as input, the velocity response spectrum is calculated and compared with the given velocity response spectrum $R_V(\zeta, f_r)$. If the differences are greater than a prescribed tolerance, $S_{\ddot{u}_g}(f_r)$ values are adjusted automatically.

This iteration continues until the computed velocity response spectrum values at all of the test points coincide with the original velocity response spectrum within the prescribed tolerance. Usually, the first theoretical approximation is quite close to the exact $S_{\ddot{u}_g}(f_r)$, then, for example an accuracy within 1 % can be satisfied after a few cycles of iterations.

In order to demonstrate the application of this procedure and the damping dependancy of generated power spectrum, the horizontal and vertical design spectra recommended by US Nuclear Regulatory Commission (ref.5) scaled to 1 g ground acceleration have been chosen as the design spectra and for each damping ratio a corresponding normalized earthquake power spectral density is derived and plotted in Fig. 1 and 2. The probable mean-maximum ground accelerations obtained from those power spectra for an earthquake duration of 30 seconds are also shown in the same figures. As expected, normalized power spectral density curves do not coincide with each other. With the exception of very low damping, however the differences are not too great and it seems that it is feasible to obtain an average normalized power spectral density curve for a range of damping ratios which is of practical interest. The same observation can be made for the mean-maximum ground accelerations. Further, the generated earthquake acceleration power spectral densities (corresponding to 5 % damping design response spectra) have been chosen as the basic design earthquake power spectrum and the response spectrum curves for all other damping ratios are calculated and plotted in Fig. 3 and 4, in comparison with the corresponding original design response spectrum curves. The relative deviations from the target spectra are also given in the same figures for the frequency range 2.5 to 9 cps. Again, as it is shown that the reproduced response spectrum curves for an acceleration power spectral density based on a specific damping response spectrum deviate from the original response spectrum curves, but the differences are not significant for the damping ratios which are of interest. These results reinforce the feasibility of using an average power spectrum as a design earthquake acceleration power spectral density corresponding to a given design earthquake velocity response spectrum.

DYNAMIC EARTHQUAKE ANALYSIS OF A SAMPLE STRUCTURE

In order to demonstrate the application of the method presented, full dynamic earthquake analysis has been carried out on a sample structure (Fig.5). For the earthquake response calculations, NRC horizontal design velocity response spectrum (ref. 5) normalized to 0.15 g with 5 % damping and the generated earthquake acceleration power spectral density based on the same response spectrum are used as earthquake input. Three dimensional earthquake excitation acting simultaneously is considered and different intensity factors are applied to each orthogonal component.

As an example, absolute floor response acceleration power spectral densities of two selected grid points and corresponding floor response spectra derived for 5 % damping are given in

Figures 6, 7 and 8, respectively.

CONCLUSIONS

The results of the above mentioned analysis have demonstrated the capabilities and effectiveness of the probabilistic dynamic earthquake analysis technique presented herein and the method has been found well-suited the seismic design of all kinds of important structures, especially the nuclear power plants; due to being compatible with the random nature of the earthquake phenomenon and also having a direct approach to the equipment design that eliminates the costly time domain computations.

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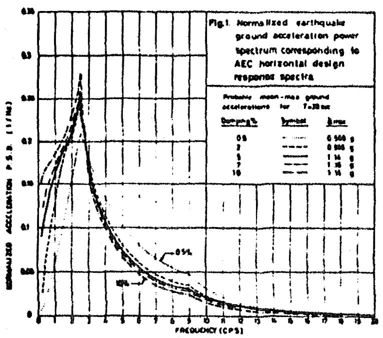

Fig.1 Normalized earthquake ground acceleration power spectrum corresponding to AEC horizontal design response spectra

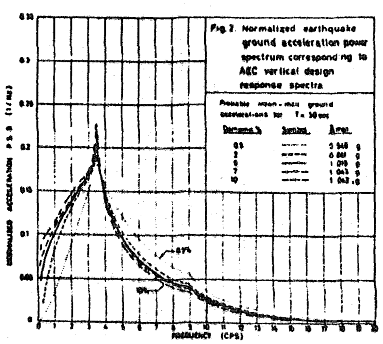

Fig.2 Normalized earthquake ground acceleration power spectrum corresponding to AEC vertical design response spectra

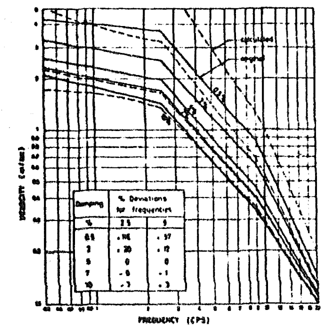

Fig. 3. Calculated horizontal response spectra based on generated earthquake acceleration power spectrum corresponding to 5% damping AEC horizontal design spectra

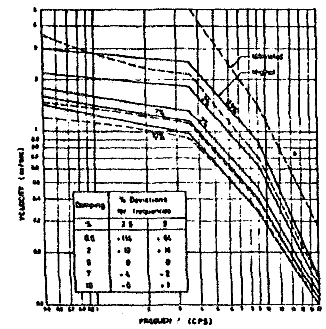

Fig. 4 Calculated vertical response spectra based on generated earthquake acceleration power spectrum corresponding to 5% damping AEC vertical design spectra

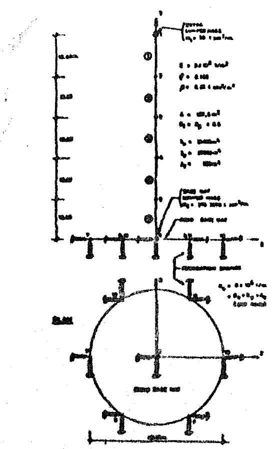

Fig. 5. Mathematical model of sample structure

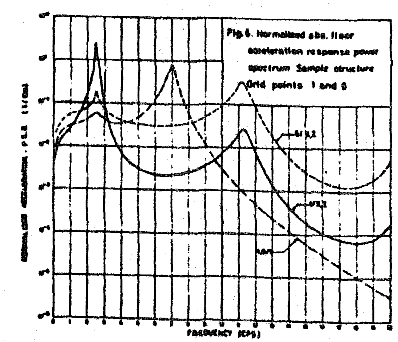

Fig.6. Normalized abs. floor acceleration response power spectrum Sample structure Grid points 1 and 6

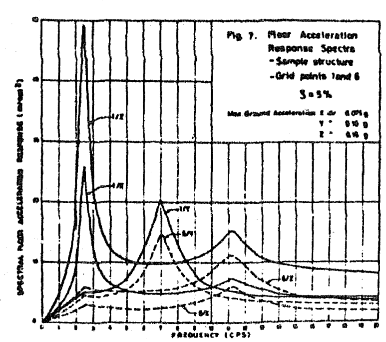

Fig. 7. Floor Acceleration Response Spectra - Sample structure - Grid points 1 and 6 $\xi = 5\%$

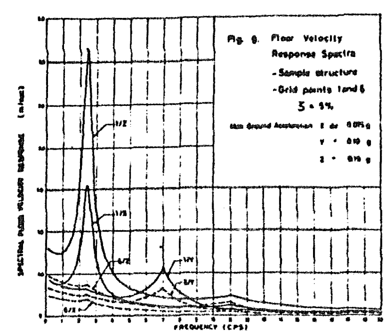

Fig. 8. Floor Velocity Response Spectra - Sample structure - Grid points 1 and 6 $\xi = 5\%$

DISCUSSION

Rudolf Grossmayer (Austria)

First of all, the discussor would like to ask the author, how the factor 'n' is to evaluate and which parameters have the greatest influence on it ? Secondly, it seems to me, not very reasonable to average response spectra that belong to different damping values. On the other hand, if you compute the power spectral density of the ground motion adjusted to a response spectrum with a specific damping value, and afterwards compute response spectra for other damping values, and it turns out, that they do not agree with the original ones for the same damping values, then it seems to me that the original response spectra are not consistent. Therefore it is preferable to define ground motions by means of power spectral densities, because they lead to consistent response spectra automatically. Finally, It would like to ask, if the author tested conversion procedure from response spectra to power spectral densities also by means of artifical time histories ?

E.H. Vanmarcke (U.S.A.)

The transient nature of earthquake ground motions limits the applicability of stationary random vibration analysis procedures in earthquake engineering. For lightly damped structures and equipment, for long period systems, nonstationary effects are significant if excessive conservatism is to be avoided. Relatively simple extensions to the stationary solutions which account for the transience of the input are available (see for example, Ref. 6). The author's formula (Eq. 5) to convert the velocity response spectrum to the earthquake acceleration power spectral density is valid neither at very low nor at very high frequencies, and it breaks down when the damping vanishes (6,7). Also, much information is available about the magnification factor "n" which itself depends on natural frequency, damping, and duration (6).

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R. Duarte (Portugal)

Assume you have computed the power spectral density $S_{\ddot{u}g}(f_r)$ from the velocity response spectrum of a given ground motion with maximum ground acceleration a, maximum ground velocity v and maximum ground displacement d.

From $S_{\ddot{u}g}(f_r)$ you can also compute the values of the expected maximum ground acceleration $\langle a_{max} \rangle$, expected maximum ground velocity $\langle v_{max} \rangle$, and expected maximum ground displacement d_{max} .

Are the $\langle a_{max} \rangle$, $\langle v_{max} \rangle$ and $\langle d_{max} \rangle$ very different from the a, v and d, respectively ?

Author's Closure

With regard to the question of Mr. Grossmayer, we wish to state that the factor "n" mainly depends on the earthquake and the frequency content of the response. Evaluation of this factor is quite a long procedure. Ref. 2 of the paper or Ref. 6 as mentioned by Vanmarcke should be consulted.

Feasibility of using an average response spectra that belong to different damping values has been tested and the results are due to be presented in SMIRT-4 Conference (Ref. 8), the results obtained seem to be reasonable.

The author completely agrees that the original design response spectra are not consistent and it is preferable to define ground motions by means of power spectral densities. Unfortunately this procedure has not yet been accepted by regulatory authorities, for this reason the author intends to find a way to use this method based on a well accepted earthquake input such as design response spectra.

The conversion procedure from response spectra to power spectral densities has already been tested by means of an artificial time history and the results are due to be presented in SMIRT-4 Conference (Ref. 8).

With regard to the question of Mr. Vanmarcke, we wish to state that the author realizes that the transient nature of earthquake ground motions limits the applicability of stationary random vibration analysis procedures in earthquake engineering, especially when lightly damped or long period systems are considered. However, the author desires to introduce a design method to be compatible with and as accurate as the well known response spectrum method and also not to loose the frequency content which is necessary to derive the floor response spectra. This method has shown to be accurate enough (Ref. 8) in case of nuclear power plant structures, that are short period and not very lightly damped systems. The author agrees that for a complete and pure random analysis the nonstationary effects may also be significant.

It is true that the formula (Eq.5) to convert the velocity response spectrum to the earthquake acceleration power spectral density is valid neither at very low nor at very high frequencies, and it breaks down when the damping vanishes. However, as mentioned above the nuclear power plant structures are neither very low nor very high frequency systems and the damping always exists. On the other hand, this formula is only used for the first approximation and the final conversion is achieved by means of a successive iteration procedure.

Availability of more information about the magnification factor "n" is appreciated and Mr. Grossmayer may find detailed answer to his first question in Ref. 6.

With regard to the question of Mr. Duarte, we wish to state that with the exception of very low damped response spectrum, the differences may not exceed 15%. It has to be realized that such differences are mainly spectrum dependent. For example, the average difference for USNRC Reg. Guide 1.60 horizontal response spectra is about 15%, while for vertical response spectra the average difference is found only about 3% (See Figs. 1 and 2).

Finally, the author would like to thank the three discussors for their kind interests and valuable comments which gave him the opportunity to clarify some more important points that cannot be included in detail, in the limited frame of the paper presented in the Conference.

Ref.8: Schneeberger B., Breulex R., Ermutlu H.E. - "Comparison between time-step-intergration and probabilistic methods in seismic analysis of a linear structures"-SMIRT-4 Conference, San Francisco, Aug. 1977.