A Method for Generating Floor Response Spectra Through Power Spectra/Response Spectra Relationship

B.J. Sullivan

Franklin Research Center, 20th & Race Streets, Philadelphia, Pennsylvania 19103, U.S.A.

ABSTRACT

In this work a method is presented for deriving response spectra of points within a three-dimensional structure directly from mutally orthogonal base response spectra. In buildings, response spectra of floors are used to design mechanical equipment and secondary systems such as piping, which are supported on building floors. In seismic qualification of structures and components in nuclear power plants, required response spectra may be generated analytically to be used as criteria in the dynamic testing of the devices.

The traditional method of deriving such response spectra curves has been deterministic in its approach. Generally, artificial accelerograms are generated such that the response spectra of the simulated base excitations envelop the given base response spectra. The base excitations are then applied to the model of the structure and the time history of the resulting absolute acceleration of points of interest are used to generate response spectra. This process, while straight forward in its approach, may nevertheless be tedious in the handling of the time history data, and costly as well from a computational point of view.

In this work a method is proposed for deriving floor response spectra using probabilistic techniques. By modelling an earthquake as a stationary random process, a relationship may be derived between its power spectral density function (PSDF) and the response spectrum. Thus, given a set of base response spectra, a set of consistent PSDF's can be generated for the base of the structure. Then, making use of standard random vibration theory, PSDF's for points of interest in the structure can be obtained by appropriate multiplication of complex frequency response (transfer) functions with the derived base PSDF's. Finally, response spectra for the points of interest are obtained using the inverse form of the relationship between a PSDF and a response spectrum.

To date, the approach outlined above has been used to generate response spectra of points in some actual three-dimensional structures, and comparisons with response spectra for the same points generated by the time history method have been quite favorable. The limited number of cases performed have demonstrated that the method provides close correspondence of results throughout the frequency domain. While more work is needed to completely qualify this approach, initial results have been very promising. If the approach can be completely verified and found acceptable to the appropriate regulatory bodies, considerable savings in the computation of floor response spectra would result.

1. Introduction

In this paper a method is presented for deriving response spectra of points within a three-dimensional structure directly from mutually orthogonal base response spectra. In buildings, response spectra of floors are used to design mechanical equipment and secondary systems such as piping, which are supported on building floors. In seismic qualification of structures and components in nuclear power plants, required response spectra may be generated analytically to be used as criteria in the dynamic testing of devices.

The traditional method of deriving response spectra curves has been deterministic in its approach. Typically, artificial accelerograms are generated such that the response spectra of the simulated base excitations envelop the given base response spectra. The base excitations are then applied to the model of the structure. The time history of the resulting absolute acceleration of points of interest are used to generate response spectra. This process, while straight forward in its approach, may nevertheless be tedious in the handling of the time history data, as well as costly from a computational point of view. Apart from the solution of the eigenproblem (i.e. the calculation of the natural frequencies and mode shapes of the structure) computational expenses incurred in using this technique included the cost of performing the time history analysis of response, and the cost of generating the base excitations as well. It should be noted that the cost of generating these spectrum consistent base accelerograms is a significant aspect. Depending upon the modal characteristics of the structure, obtaining the time history of response may actually require less computation than generating the synthetic accelerograms. Note also that a common criticism of the time history method of generating seismic floor spectra is that two independent time histories may give quite different floor spectra even though they both encompass the base response spectrum used to generate them. To say the least, such uncertainties associated with the use of spectrum-compatible time histories is a disturbing aspect of this approach.

2. Review of Alternative Techniques

In recent years various attempts have been made to circumvent the drawbacks of generating seismic floor spectra by the time history method. Initial efforts by Biggs [1] and by Kapur and Shao [2] proposed generating floor spectra using magnification curves obtained on the basis of the observed behavior of oscillators subjected to a set of earthquake records. Results obtained using this approach were generally conservative, however confidence in this method was lacking due to its semi-empirical nature.

Subsequent research in this area has centered on the use of probabilistic techniques. The first notable effort along these lines was that of Singh [3]. In this work the earthquake motion was characterized as a zero mean stationary Gaussian random process. Using standard random vibration theory, the power spectral density function (PSDF) of the floor was generated by appropriately combining complex frequency response or transfer functions (which reflect the structure's modal characteristics) with the PSDF characterizing the base excitation. In this work Singh used the simple relationship, that the response spectrum value at a frequency ω_o was equal to a constant multiplied by the standard deviation of the response at the floor of interest, where this standard deviation is directly obtainable from the floor PSDF. Comparisons of results using Singh's method and the time history method were provided in NUREG/CR-1123, reference [4]. It was concluded in this work [4] that the two methods gave comparable results; however, for most of the period axis, the time history method gave values which exceeded those of Singh's random vibration method. At least some of the discrepancies were the result of using the smooth Regulatory Guide base spectrum in generating the floor response spectra. More uniform results might have been obtained using Singh's method if the spectrum corresponding to the actual artificial accelerogram had been used.

Scanlan and Sachs [5] used a similar relationship involving the product of the standard deviation of the acceleration with a factor to obtain the desired floor response spectra. In this work the absolute acceleration response of an oscillator mounted to the floor is expressed as a function of time, in terms of a series of sinusoids whose coefficients are dependent on the structure's transfer function and the earthquake acceleration amplitude at the corresponding frequency. The previously mentioned factor is a function of the structure's natural frequencies and is calculated using an iterative scheme in which its form is obtained in the process of computing the amplitudes of the series representing the time varying nature of the earthquake acceleration, such that a target base response spectrum is matched.

This method has been shown to compare well with time history results, however it does not eliminate the need to work in the time domain. While time history analysis need not be performed, artificial accelerograms consistent with base response spectra must still be generated. This requirement, as well as the need to work with time domain data, diminishes the attractiveness of this approach.

More recent efforts have focused on establishing relations between power and response spectra. In the work of Chen and Chen [6], this relationship was established by the use of the extreme value theorem of random processes. Results cited in this work and in a summary paper [7] by the same authors, have shown very favorable comparisons with time history solutions.

A similar approach involving a relationship between the power and response spectra is reported here. The work of Kaul [8], who developed an approximate and an exact relationship between the power and response spectra, is employed. The results derived in this work have used the approximate expression. As shown by both Kaul, and by Unruh and Kana [9], this approximate relationship can be used to derive a PSDF consistent with a given response spectrum by means of an iterative process. Given a set of base response spectra; a set of consistent PSDF's can be generated for the base of the three dimensional structure. Then, by making use of standard random vibration theory, PSDF's for points of interest in the structure can be obtained by appropriate multiplication of the transfer functions with the derived base PSDF's. Finally, response spectra for the points of interest are obtained using the inverse form of Kaul's relationship between a PSDF and a response spectrum.

The background theory for this approach will now be derived.

3. Theory of the Method

The response spectrum of a floor or, more generally, the response spectrum corresponding to a particular degree of freedom, is a plot of the maximum response versus the natural frequency of an oscillator subjected to the time history of response of the degree of freedom. If v is the displacement of the oscillator relative to the point of interest, the equation representing its motion may be written as

$$\ddot{v} + 2\xi_o\omega_o\dot{v} + \omega_o^2 v = -\ddot{u}(t) \tag{1}$$

where ξ_o and ω_o are the damping factor and natural frequency of the oscillator, and the absolute acceleration of the degree of freedom of interest is represented by $\ddot{u}(t)$. By solving eq. (1) for a range of values of ω_o, the response spectrum corresponding to $\ddot{u}$ is obtained. To obtain $\ddot{u}$, it is necessary to solve the equations of motion for the structure subjected to ground acceleration

$$[M]\{\ddot{y}\} + [C]\{\dot{y}\} + [K]\{y\} = -[M]\{\ddot{x}\} \tag{2}$$

where [M], [C] and [K] are mass, damping and stiffness matrices, respectively, {y} is the relative displacement vector, and the elements of $\{\ddot{x}\}$ are the base accelerations in the same global directions as the corresponding element of {y}. By solving the eigenproblem, eq. (2) may be transformed to the uncoupled form, one equation of which is

$$\ddot{Y}_m + 2\xi_m\omega_m\dot{Y}_m + \omega_m^2 Y_m = Q_m \tag{3}$$

In eq. (3), ξ_m and ω_m represent the modal damping and natural frequency, respectively, of mode m, and Q_m is an element of the vector

$$\{Q\} = -[\Omega]^{-1}[\Phi]^T[M]\{\ddot{x}\} \tag{4}$$

In eq. (4), $[\Omega]^{-1}$ represents the inverse of

$$[\Omega] = [\Phi]^T[M][\Phi]$$

and $[\Phi]^T$ is the transpose of the matrix of eigenvectors. The absolute acceleration for the k^{th} degree of freedom is given by

$$\ddot{u}_k = \ddot{x}_k + \ddot{y}_k = \ddot{x}_k + \sum_m \phi_{km}\ddot{Y}_m \tag{5}$$

where ϕ_{km} is an element of the matrix of eigenvectors. Using eq. (3) this becomes

$$\ddot{u}_k = \ddot{x}_k - \sum_m \phi_{km}Q_m - \sum_m \phi_{km}\omega_m^2 Y_m - 2\sum_m \phi_{km}\xi_m\omega_m\dot{Y}_m \tag{6}$$

and provided a sufficient number of modes is used, eq. (6) may be written in the form

$$\ddot{u}_k(t) = -\sum_m \phi_{km}\omega_m^2 Y_m(t) - 2\sum_m \phi_{km}\xi_m\omega_m\dot{Y}_m(t) \tag{7}$$

At this point, the theory of random processes is used by first assuming that the earthquake motion $\{\ddot{x}\}$ is a zero mean stationary Gaussian random process. While this is not strictly correct for actual earthquakes, it is a commonly made assumption and is considered to be conservatively acceptable. Random vibration theory states that for linear time invariant systems, a Gaussian excitation will result in a Gaussian response. To statistically characterize such a process, the mean and autocorrelation are required. We have postulated a zero mean process; the autocorrelation function will now be derived and used.

The autocorrelation of the absolute acceleration response for the k^{th} degree of freedom is

$$R_{\ddot{u}_k}(\tau) = E\left\{\ddot{u}_k(t)\ddot{u}_k(t+\tau)\right\} = E\left\{\sum_{mn}\sum\phi_{km}\phi_{kn}\,\omega_m^2\omega_n^2Y_m(t)Y_n(t+\tau) + 2\omega_m^2\xi_n\omega_n\dot{Y}_n(t+\tau)Y_m(t)\right.$$

$$\left. + 2\xi_m\omega_m\omega_n^2\dot{Y}_m(t)Y_n(t+\tau) + 4\xi_m\omega_m\xi_n\omega_n\dot{Y}_m(t)Y_n(t+\tau)\right\} \tag{8}$$

where E { } represents the expected value of the term within the brackets. Using the convolution integral form of the response relation, we may write

$$Y_n(t) = \int_{-\infty}^{t} Q_n(\tau)h_n(t-\tau)d\tau = \int_{-\infty}^{\infty} Q_n(t-\theta)h_n(\theta)d\theta \tag{9}$$

where $h_n(t)$ represents the unit impulse function. Consequently, the first term in (8) may be written as

$$E\left[\sum_{mn}\sum\int_{-\infty}^{\infty}\int_{-\infty}^{\infty}\phi_{km}\phi_{kn}\omega_m^2\omega_n^2Q_m(t-\theta_1)Q_n(t-\theta_2)h_m(\theta_1)h_n(\theta_2)d\theta_1d\theta_2\right]$$

$$= \sum_{mn}\sum\int_{-\infty}^{\infty}\int_{-\infty}^{\infty}\phi_{km}\phi_{kn}\omega_m^2\omega_n^2R_{Q_mQ_n}(\tau+\theta_1-\theta_2)h_m(\theta_1)h_n(\theta_2)d\theta_1d\theta_2 \tag{10}$$

The power spectral density function (PSDF) for the acceleration $\ddot{u}_k(t)$ is obtained by taking the Fourier transform of the autocorrelation function:

$$S_{\ddot{u}_k}(\omega) = \frac{1}{2\pi}\int_{-\infty}^{\infty} R_{\ddot{u}_k}(\tau)e^{-i\omega\tau}d\tau \tag{11}$$

Substituting (10) into (11) we get for the first term of $S_{\ddot{u}_k}(\omega)$

$$\frac{1}{2\pi}\int_{-\infty}^{\infty}\left\{\sum_{mn}\sum\int_{-\infty}^{\infty}\int_{-\infty}^{\infty}\phi_{km}\phi_{kn}\omega_m^2\omega_n^2R_{Q_mQ_n}(\tau+\theta_1-\theta_2)h_m(\theta_1)h_n(\theta_2)d\theta_1d\theta_2\right\}e^{-i\omega\tau}\,d\tau$$

$$= \frac{1}{2\pi}\sum_{mn}\sum\phi_{km}\phi_{kn}\omega_m^2\omega_n^2\left[\int_{-\infty}^{\infty}h_m(\theta_1)d\theta_1\cdot\int_{-\infty}^{\infty}h_n(\theta_2)d\theta_2\cdot\int_{-\infty}^{\infty}R_{Q_mQ_n}(\tau+\theta_1-\theta_2)e^{-i\omega\tau}d\tau\right]$$

$$= \sum_{mn}\sum\phi_{km}\phi_{kn}\omega_m^2\omega_n^2H_m^*(\omega)H_n(\omega)S_{Q_mQ_n}(\omega) \tag{12}$$

In obtaining eq. (12) the following equations for the complex frequency response functions were used:

$$H_m^*(\omega) = \int_{-\infty}^{\infty}h_m(t)e^{i\omega t}dt = \frac{1}{(\omega_m^2-\omega^2) - i(2\xi_m\omega\omega_m)} \tag{13}$$

$$H_n(\omega) = \int_{-\infty}^{\infty}h_n(t)e^{-i\omega t}dt = \frac{1}{(\omega_n^2-\omega^2) + i(2\xi_n\omega\omega_n)} \tag{14}$$

By a similar development, as that used to get the first term, the remaining terms of $S_{\ddot{u}_k}(\omega)$ can be obtained, giving the result

$$S_{\ddot{u}_k}(\omega) = \sum_m\sum_n \phi_{km}\phi_{kn}\left[\omega_m^2\omega_n^2 + 2i\omega\omega_m\omega_n(\omega_n\xi_m - \omega_m\xi_n) + 4\xi_m\omega_m\xi_n\omega_n\omega^2\right] H_m^*(\omega)H_n(\omega)S_{Q_mQ_n}(\omega) \quad (15)$$

Consider now the PSDF $S_{Q_mQ_n}(\omega)$. Recall that the generalized excitation vector is given by

$$\{Q(t)\} = -[\Omega]^{-1}[\Phi]^T[M]\{\ddot{x}\} \quad (16)$$

This may be rewritten in a more convenient form

$$\{Q(t)\}_{rx1} = -[P]_{rx3}\{a_g(t)\}_{3x1} = -[P]\begin{Bmatrix} a_x \\ a_y \\ a_z \end{Bmatrix} \quad (17)$$

where $\{a_g(t)\}$ is the vector of size 3 containing the base accelerations in the equilibrium coordinate system; r is the total number of normal modes included in the analysis, and [P] is the rectangular matrix of modal participation factors, computed from

$$[P]_{rx3} = [\Omega]^{-1}_{rxr}[\Phi]^T_{rxp}[M]_{pxp}[T]_{px3} \quad (18)$$

In eq. (18), p is the total number of degrees of freedom in the structure and [T] is a transformation matrix populated with zeros and ones. Thus,

$$\lfloor Q(t+\tau)\rfloor_{1xr} = \lfloor a_g(t+\tau)\rfloor_{1x3}[P]^T_{3xr} \quad (19)$$

and so

$$\{Q(t)\}\lfloor Q(t+\tau)\rfloor = [P]_{rx3}[A_g]_{3x3}[P]^T_{3xr} \quad (20)$$

where

$$[A_g] = \{a_g(t)\}\lfloor a_g(t+\tau)\rfloor \quad (21)$$

Taking the expected value of eq. (20), multiplying by $e^{-i\omega\tau}/2\pi$, and integrating from $-\infty$ to $+\infty$ gives the excitation PSDF matrix

$$[S_{QQ}(\omega)]_{rxr} = [P]_{rx3}[S_{a_g}(\omega)]_{3x3}[P]^T_{3xr} \quad (22)$$

where the base acceleration PSDF matrix is

$$\left[S_{a_g}(\omega)\right] = \begin{bmatrix} S_{a_xa_x}(\omega) & S_{a_xa_y}(\omega) & S_{a_xa_z}(\omega) \\ S_{a_ya_x}(\omega) & S_{a_ya_y}(\omega) & S_{a_ya_z}(\omega) \\ S_{a_za_x}(\omega) & S_{a_za_y}(\omega) & S_{a_za_z}(\omega) \end{bmatrix} \quad (23)$$

Since the excitations in orthogonal directions are required to be statistically independent [10], the off-diagonal terms in eq. (23) are zero, so that

$$\left[S_{a_g}(\omega)\right] = \begin{bmatrix} S_{a_x}(\omega) & 0 & 0 \\ 0 & S_{a_y}(\omega) & 0 \\ 0 & 0 & S_{a_z}(\omega) \end{bmatrix} \quad (24)$$

By substituting eq. (24) withing eq. (22) and using the appropriate elements of $[S_{QQ}(\omega)]$ within eq. (15), the PSDF of the degree of freedom of interest is obtained.

We now focus on the oscillator positioned at degree of freedom k. Random process theory states that the PSDF of the oscillator response $S_o(\omega)$ is related to its base input PSDF $S_{\ddot{u}_k}(\omega)$ via the oscillator transfer function as

$$S_o(\omega) = H_o(\omega)H_o^*(\omega)S_{\ddot{u}_k}(\omega) \tag{25}$$

where $H_o(\omega)$ is obtained from eq. (1) via a Fourier transform as

$$H_o(\omega) = V(\omega)/U(\omega) = \frac{\omega_o^2 + 2i\omega_o\xi_o\omega}{(\omega_o^2 - \omega^2) + 2i\omega_o\xi_o} \tag{26}$$

and $H_o^*(\omega)$ denotes the complex conjugate transfer function. The standard deviation, or RMS value of the oscillator response is obtained from

$$\sigma^2(\omega_o)= \int_{-\infty}^{\infty} S_o(\omega)\,d\omega = \int_{-\infty}^{\infty} \left[\frac{\omega_o^4 + 4\omega_o^2\xi_o\omega^2}{(\omega_o^2 - \omega^2) + 4\omega_o^2\xi_o^2\omega^2}\right] S_{\ddot{u}_k}(\omega)\,d\omega \tag{27}$$

At this point we make the supposition that the response spectrum is related to the standard deviation of the oscillator response through the expression

$$R(\omega_o) = F(\omega_o)\sigma(\omega_o) \tag{28}$$

where F_o is an amplitude factor by which the standard deviation must be multiplied to account for the expected peak response. Provided that the response spectrum value for a PSDF is the response level which would be exceeded with only a small probability, F_o may be expressed as [9]

$$F_o(\omega_o) = [-2\ \ln\{-(\pi/T)(\sigma/\dot{\sigma})\ln(1-p)\}]^{1/2} \tag{29}$$

where T is the earthquake effective time duration, p is the probability of exceedence, and $\dot{\sigma}$ is the standard deviation of the time derivative of the response, given by

$$\dot{\sigma}^2(\omega_o) = \int_{-\infty}^{\infty} \omega^2 S_o(\omega)\,d\omega \tag{30}$$

The calculation of $R(\omega_o)$ using eqs. (28) through (30) is largely precise; it will give approximate though nearly exact results only when the expected number of local extremum of $\ddot{v}(t)$ is small in time duration T, or when the quantity ε, given by

$$\varepsilon = [1-\dot{\sigma}^2/(\sigma\ddot{\sigma})]^{1/2} \tag{31}$$

is very close to 1 (see reference [8] for details).

Completing this development, the base excitation PSDF's $S_{a_x}(\omega)$, $S_{a_y}(\omega)$ and $S_{a_z}(\omega)$ must be obtained, using the given base response spectra. While the determination of the response spectrum of a Gaussian acceleration process from its power spectrum is easily accomplished, the solution of the inverse problem, calculating a PSDF from a response spectrum, is not as straight forward. Kaul [8] develops an exact solution in which the power spectrum is assumed to be a finite linear combination of appropriately selected functions. The functions coefficients are solved in an iterative fashion until the response spectrum, obtained using the calculated PSDF, is consistent with the given response spectrum. Also in this paper, an approximate solution is provided in the form

$$S_a(\omega_o) = \frac{2\xi_o}{\pi\omega_o} R_a^2(\omega_o) \cdot \left\{-2 \ln\left[-\left(\frac{\pi}{\omega_o T}\right) \ln(1-p)\right]\right\}^{-1} \tag{32}$$

where the subscript a, denotes correspondence of the quantity with base acceleration. Kaul demonstrated through numerical examples that the use of eq. (32) yielded reasonably accurate results for frequencies between 0.25 and 6 hz., and conservative results outside this range.

In this effort an alternate iterative scheme proposed by Unruh and Kana [9], was used to obtain a compatible mapping between the response spectrum and power spectrum of the base of the structure. This procedure can be summarized as follows:

Step 1. Compute S_a from R_a using eq. (32).

Step 2. Compute the approximate response spectrum $\hat{R}_a$ using S_a and eq. (28).

Step 3. Compare $\hat{R}_a$ with R_a.

Step 4. If $\hat{R}_a \cong R_a$ stop the process and use the last S_a. If $\hat{R}_a \neq R_a$ adjust the power spectrum using

$$S_a(\omega_o)_{i+1} = S_a(\omega_o)_i \left[\frac{R_a(\omega_o)}{\hat{R}_a(\omega_o)_i}\right] \tag{33}$$

and return to Step 2.

In summary, power spectra for the base of the structure are computed using the iterative scheme described above, using eqs. (32) and (33). Substituting the computed power spectra into eq. (24), using eq. (24) within eq. (22), and using the appropriate terms of eq. (22) within (15), the PSDF of degree of freedom k is obtained. Then the response spectrum corresponding to $\ddot{u}_k(t)$ is computed from eq. (28).

4. Numerical Results

The mathematical model of the three dimensional structure shown in Figure 1 was used in generating response spectra of various points, using both the time history approach and the method proposed here. In performing the seismic analysis, the response of the structure was calculated using the simultaneous action of three mutually orthogonal components of base motion. In the time history approach, the artificial accelerograms generated using the base response spectra had a maximum correlation of 5.3%, corresponding to the base motion components in the X1 and X3 directions. The random vibration method proposed here assumes complete statistical independence of the base motion components, as shown in eq. (24). The base response spectrum curve in the X1 direction is shown in Figure 2. The dashed line represents the given smoothed spectrum and the solid line corresponds to the response spectrum of the generated X1 accelerogram. In this work the base response spectra for the X2 and X3 directions were specified as identical to that of X1.

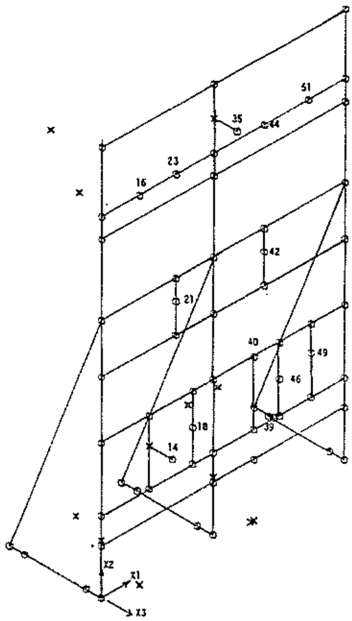
Figures 3 and 4 show the response spectra calculated by both methods for the model points 18 and 46 respectively. In these two figures, the dashed line represents the spectrum computed by the time history method, and the solid line provides the response spectrum calculated using the denoted random vibration method. These plots establish the close correspondence between the results of the two approaches. It is important to note that the random vibration method accurately predicted the amplified response occurring at the structure's natural frequencies, all of which were beyond 10 hz.

In generating the response spectra by the random vibration method, the smoothed base response spectra were used. In Figure 2, note that the response spectrum of the artificial accelerogram lies above that of the given base spectrum for frequencies beyond 10 hz. Since Figures 3 and 4 show good agreement between the curves of the two different methods beyond 10 hz, it is reasonable to assume that if the spectra of the generated accelerograms had been used in the random vibration method, conservative results for frequencies beyond 10 hz would have been obtained.

5. Conclusions

In this paper, a method has been described for obtaining the response spectra of specific degrees of freedom in a three dimensional linear elastic structure. The method is based on the theory of random processes and calculates the spectra completely within the frequency domain. Generating spectrum-compatible synthetic base motions and performing time history analysis are thus eliminated with this approach, thereby offering considerable savings in computational effort. These savings would be reflected in lower costs for plants whose design is not complete. More importantly, the probabilistic approach proposed here would provide a more economical method for evaluating the seismic margins of existing nuclear power plant structures.

Due to approximations inherent in the approach for obtaining a PSDF from the given base response spectra, conservative results in the computed response spectra may be obtained for frequencies beyond some cut-off value. Using the exact iterative procedure for obtaining S_a from R_a as reported in Kaul [8], should be considered rather than the approximate approach suggested by Unruh and Kana [9] which was used here. In addition the applicability of other relationships between S_a and R_a reported in the literature [6], [11] should be investigated.



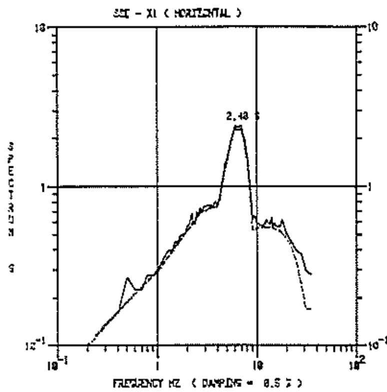
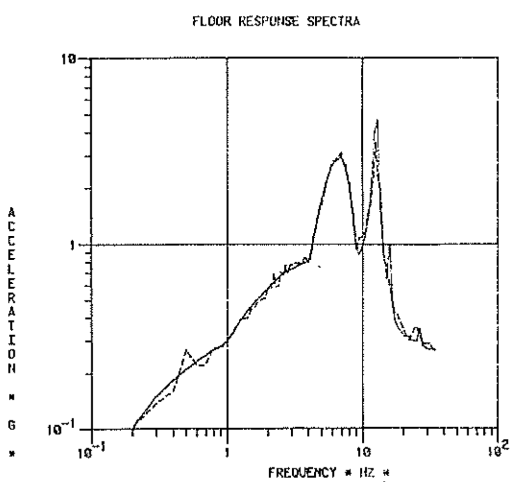


Figure 1 - Isometric Plot showing Mathematical Model of Linear Elastic Structure used in Seismic Analysis

Figure 2 - Comparison between Base Response Spectrum and Response Spectrum Curve from Artificial Accelerogram in X1 Direction



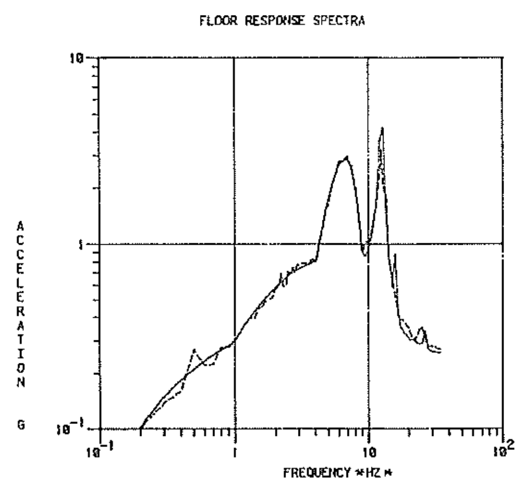


Figure 3 - Comparison between Response Spectrum Curves Calculated using Time History and Random Vibration Methods for Model Point 18 in the X1 Direction

Figure 4 - Comparison between Response Spectrum Curves Calculated using Time History and Random Vibration Methods for Model Point 46 in the X1 Direction

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