Statistical Uncertainty of Response Characteristic of Building-Appendage System for Spectrum-Compatible Artificial Earthquake Motion

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SUMMARY

Spectrum-compatible artificial time histories of ground motions are frequently used for the eismic design of nuclear power plant structures and components. However, statistical uncertainty of the responses of building structures and mechanical components mounted on the building (building-appendage systems) are anticipated, since an artificial time history is no more than one sample from a population of such time histories that match a specified design response spectrum. This uncertainty may spoil the reliability of the seismic design and therefore the extent of the uncertainty of the response characteristic is a matter of great concern.

In this paper, above-mentioned uncertainty of the dynamic response characteristics of the building-appendage system to the spectrum-compatible artificial earthquake is investigated.

For this purpose, 35 sample time histories each of which is consistent with a specified design response spectrum are synthesized by the following procedure.

(1) Generation of initial time history using sinusoidal superposition with an envelop function.

(2) Computation of response spectrum.

(3) Manipulation of the amplitude of the Fourier transform representation of this time history at each frequency to obtain better compatibility of the computed response spectrum with the design response spectrum.

(4) Generation of a new time history using Fourier inverse transform.

(5) Repeat of above iterative steps (2) through (4).

Responses of the building-appendage system are computed with the two-degree-of-freedom system in which the appended mass is very small in comparison with the supporting mass. The statistical uncertainty is evaluated with respect to the response acceleration, the relative displacement and the cumulative fatigue damage.

Through this study, the significant uncertainty is made obvious. It is also pointed out that the equivalent number of stress cycles of fatigue analysis for the seismic structural response should be more than generally expected. Finally a practical method for estimating floor response spectra is proposed to cope with the uncertainty employing the statistical approach and the adequacy of the method is proved through its application to an example problem.

1. Introduction

Artificial earthquake motions compatible with smoothed design response spectra are frequently used for the seismic design of nuclear power plants and a lot of techniques have been proposed for generating time histories.

On the other hand, the statistical uncertainty of the response characteristics to those artificial time histories were pointed out by some investigators [1], [2]. Further, the effect of applying smoothed response spectra neglecting peaks and troughs which are generally observed in the response spectra of actual earthquake motions cannot be said clear.

In this paper, the uncertainty is studied with respect to the response characteristics of the building-appendage system and a practical method is proposed for estimating floor response spectra.

2. Generation of artificial earthquake accelerograms

The standard response spectrum presented by Osaki and others [3] for a free lock surface is used as an example of target design spectra. The velocity response spectrum was selected for 5% of damping ratio, 10 km of epicentral distance and magnitude M7.

The time duration of accelerograms and the duration of the stationary part are 30s and 7s respectively, which are determined based on the equations proposed by Watabe [4].

The following procedure was applied for the generation of time histories.

(1) Generation of a initial time history through sinusoidal superposition with the envelop function.

(2) Computation of a response spectrum

(3) Manipulation of the amplitude of the Fourier transform representation of the time history at each frequency to obtain better compatibility of the computed response spectrum with the target design spectrum.

(4) Generation of a new time history using Fourier inverse transform.

(5) Repeat of above iterative steps (2) through (4) until a reasonable convergence of the computed response spectrum to the design spectrum is obtained.

Using the above procedure, 35 different accelerograms were generated to match the aforesaid design spectrum by means of giving the different sets of uniform random phase angle. Each was developed after 6 repetitions of the iterative steps.

Fig. 1 and Fig. 2 respectively show one example of the computed accelerograms and all of the 35 response spectra. The computed response spectra are well consistent with the design spectrum for the natural periods longer than 0.1s where the coefficients of variation are less than 8%, while those are gradualy increasing from 13% (T=0.833s) to 31% (T=0.05s) with the shorter periods.

3. Uncertainty of response characteristics of the appended system.

3.1 Analytical model and its parameters

The coupled model with two-degree-of-freedom is considered, in which either an appended system or an supporting system is represented by a single-degree-of-freedom lumped mass system. The response characteristics of this kind of model was fully examined by Sato [5] and others with actual earthquakes.

In the present paper, the damping ratios of 1% for the appendage and 5% for the supporting system are chosen having regard to those which are generally selected for mechanical component including pipings and ferroconcrete structures respectively. Mass of the system corresponding to the appendage is assumed negligible in comparison with that of the supporting structure throughout this paper. The natural periods of the appendage and the supporting system represented hereinafter by Ta and Ts respectively.

3.2 Response acceleration and response relative displacement

The response acceleration and the relative displacement was computed for the 35 artificial earthquake motions introduced in the previous chapter.

Fig. 3 shows the maxima, the averages and the minima of the response acceleration of the appendage, and Fig. 4 shows those of the response displacement of the appendage relative to that of the supporting mass. The figures indicate that the fluctuation of the response is not small and it is outstanding at the resonant periods. The ratios of the maximum to the minimum are observed to exceed 5 even at the periods longer than 0.1s where the fluctuation of the response spectral values is very small. The coefficients of variation (C.O.V.) are summarized in Table 1.

Table 2 shows the coefficient of determination (C.O.D.) which were obtained by the multiple regression analysis employing the maximum acceleration for the dependent variable and response spectral values at Ta and Ts for the independent variables. The coefficient of determination are large for Ta=0.05s, 0.08s reflecting the wide fluctuation of the response spectral values at these periods, whereas the coefficients are very small for the periods longer than 0.1s. In this region with the longer periods, therefore, the fluctuation may be mostly caused by the randomness of the phase angle given to the artificial time histories.

Those analytical results indicate that the significant uncertainty could be anticipated for the responses of the appendage in spite of the good consistency of an artificial earthquake accelerogram with a design response spectrum. Further, it can be mentioned that the broadenning technique for a floor response spectrum is not sufficient to ensure the safety since the uncertainty of the response is especially significant at resonant periods.

3.3 Fatigue damage

The fatigue damage was examined with a view to evaluating the uncertainty of the cumulative cyclic effect of the seismic response. For this purpose, the stress occures at the appendage was assumed to be proportional to the response displacement relative to the supporting mass. Using the deisgn fatigue curve shown in ASME Code Sec. III for austenitic steels, the cumulative damage was estimated with the hysteresis loop count method under the assumption of Miner's concept.

At first, in spite of the fluctuation of the maximum response, each of the maximum stresses at the appendage was made identical to the stress yield 100 allowable cycles on the fatigue curve to obtain the cumulative damage, U_1. Next, in order to take the effect of the fluctuation of the maximum response into account, each of the stress time history was revised so that the maximum stress amplitude of the time history which has the largest amplitude at each period of Ts become identical to the aforesaid stress, and then the cummulative damage, U_2, was calculated.

Table 3 shows the statistical characteristics of U_1 and U_2 at the (almost) resonant periods. The ratios of the maximum to the minimum are revealed remarkable for U_2. Fig. 5 is the equivalent number of cycles, Ne, based on U_1 for the artificial earthquakes and indicates large fluctuation. Fig. 6 is that calculated for the actual earthquakes, the 32 horizontal components of 15 earthquakes in the U.S.A. The consistency of the results shown in Fig. 5 with those in Fig. 6 probes the application of the artificial accelerograms for the analysis to be reasonable.

According to the figures, the averages of Ne are from 5 to 20 cycles, however, it can be expected that Ne frequently exceed 40 at resonant periods and the larger number of Ne should be suggested considering the remarkable fluctuation of U_2.

4. Alternative method for estimating the floor response spectra

The significant uncertainty was made obvious in the above chapter with respect to the response of the appendage on the floor of a supporting structure. This requires the approach alternative to the convensional time history response method for estimating the floor response spectra. Hence, this chapter propose a practical method immune to the uncertainty hereunder.

4.1 Floor response amplification function

The following function is introduced as the floor response amplification function.

$$C_f = \frac{|\ddot{x}_a|_{max} \cdot |\ddot{x}_g|_{max}}{S(T_a,\ h_s) \cdot S(T_s,\ h_s)} \tag{1}$$

where $\ddot{x}_a$ max and $\ddot{x}_g$ max are the maximum response acceleration of the appendage and the maximum ground motion respectively and S(T,h) is the response spectrum of the ground motion.

Fig. 7 is the plot of C_f for the above artificial earthquake acclerograms (white marks) and El Centro NS component of May 18, 1940 (black marks). Table 4 shows C.O.V. of C_f.

According to those, C_f is characterized as follows.

(1) C_f depends both on the response characteristics of a structure and on a input ground motion.

(2) Since the variation of C_f with respect to Ts is practically small, it is possible to represent C_f for Ta/Ts as shown in Fig. 7.

(3) The extent of statistical fluctuations is almost identical to that of the response acceleration of the appendage.

4.2 Basic equations for estimating floor response spectra

The response acceleration of a multiple-degree-of-freedom supporting structure with n significant modes is given for i-th mass as

$$\ddot{x}_i(t) = \sum_{j=1}^{n} \phi_{ij}\psi_j \frac{d^2}{dt^2}\int_o^t I_j(t-\tau)\ddot{x}_g(\tau)d\tau \tag{2}$$

where ϕ_{ij}, ψ_j and $I_j(t)$ are normarized modal displacement of i-th mass, perticipation factor and impulse respone function respectively for j-th mode and $\ddot{x}_g(t)$ is an earthquake ground motion.

The j-th modal response of the supporting structure excites a single-degree-of-freedom system with ω_k and h_k appended on the i-th mass and the equation of motion is written as

$$\ddot{Z}_j + 2h_k\omega_k\dot{Z}_j + \omega_k{}^2Z_j = -\phi_{ij}\psi_j\frac{d^2}{dt^2}\int_o^t I_j(t-\tau)\ddot{x}_g(\tau)d\tau \tag{3}$$

Since $|\ddot{Z}_j|max/\phi_{ij}\psi_j$ corresponds to the response acceleration of the appendage of the coupled model with two-degree-of-freedom, the following equation can be introduced for the floor response spectrum, S_f, by employing SRSS of the maximum modal responses obtained through eq.(1) with the floor response amplification function and the design response spectrum, Sg, of the earthquake ground motion.

$$S_f(\omega_k,h_a,h_s) = \sqrt{\sum_{j=1}^{n} \{\phi_{ij}\psi_j C_f S_g(\omega_j,h_s)\cdot S_g(\omega_k,h_s)/|\ddot{x}_g|_{max}\}^2} \tag{4}$$

where h_a and h_s are the replacements of h_k and h_j respectively.

4.3 Application of the analysis method

The floor response spectra was calculated both by the proposed method and the conventional method.

For the dynamic characteristics of an example supporting structure, the first five modal results was selected based on the free vibration analysis of a certain multiple-degree-of-freedom structural model. The floor response amplification function, C_f,was roughly estimated through smoothing the average of C_f calculated for three accelerograms arbitrarily from those generated in chapter 2. The envelope of the response spectra for the three accelerograms was adopted as the design response spectrum.

The floor response spectra are shown in Fig. 8. The reasonable agreement between the spectrum estimated by the proposed method and the spectra computed by the conventional time history method is observed throughout the whole periods, and the spectrum by the proposed method conservatively envelops the peaks of those by the time history method, while the tendency of underestimation exist at troughs in the spectrum.

5. Conclusions

(1) The statistical uncertainty was studied with respect to the response characteristics of the building-appendage system for spectrum-compatible artificial earthquake motions and the principal conclusions are: (i) the significant statistical fluctuations can be pointed out for the cumulative cyclic effect as well as the peak response characteristics. (ii) Those are especially outstanding at resonant periods. (iii) The equivalent number of cycles for an earthquake should be more than generally expected.

(2) The practical method for estimating floor response spectra was newly proposed to cope with the statistical uncertainty.

6. Acknowledgement

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Table 1 Coefficients of variation for response acceleration of appended system

Ta \ Ts	0.0833	0.1	0.125	0.2
0.05	0.23	0.20	0.11	0.07
0.08	0.52	0.32	0.23	0.12
0.1	0.22	0.32	0.17	0.12
0.125	0.21	0.22	0.33	0.15
0.2	0.20	0.19	0.20	0.31
0.5	0.20	0.19	0.18	0.17

Table 2 Coefficients of determination for response acceleration of appended system

Ta \ Ts	0.0833	0.1	0.125	0.2
0.05	0.75	0.83	0.59	0.72
0.08	0.73	0.73	0.59	0.64
0.1	0.14	0.11	0.21	0.33
0.125	0.06	0.05	0.05	0.07
0.2	0.14	0.12	0.12	0.07
0.5	0.12	0.08	0.10	0.18

Table 3 Statistical characteristics of cumulative fatigue damage of appended system

Ts	Ta	U_1			U_2		
		average	C.O.V.	max/min	average	C.O.V.	max/min
0.0833	0.08	0.0072	0.41	9.08	0.0012	1.72	24115
0.1	0.1	0.0076	0.36	4.24	0.0028	1.00	159
0.125	0.125	0.0069	0.35	4.42	0.0024	1.05	916
0.2	0.2	0.0058	0.36	4.31	0.0014	1.17	157

Table 4 Coefficients of variation for the floor response amplification function

Ta \ Ts	0.0833	0.1	0.125	0.2
0.05	0.19	0.17	0.20	0.22
0.08	0.36	0.18	0.15	0.13
0.1	0.22	0.31	0.16	0.11
0.125	0.23	0.22	0.32	0.16
0.2	0.25	0.20	0.20	0.32
0.5	0.24	0.20	0.19	0.15

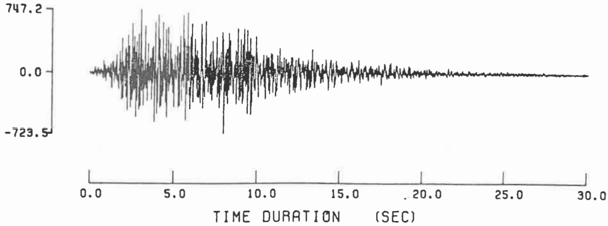

Fig. 1 An example of artificial earthquake accelerograms

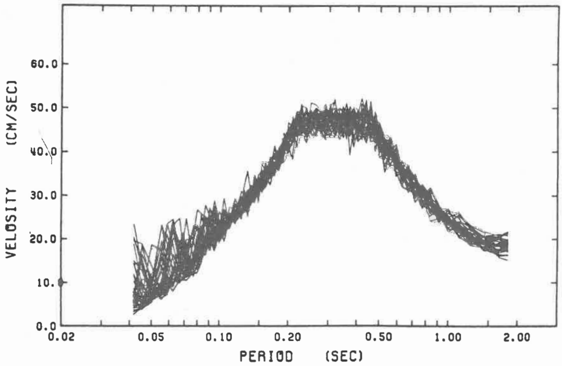

Fig. 2 Computed response spectra of artificial earthquake accelerograms

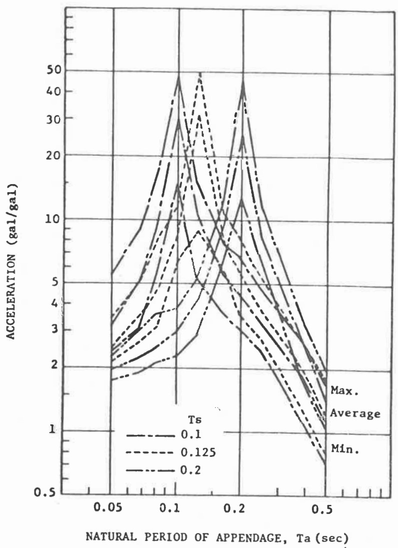

Fig. 3 Maximum response acceleration of appended system

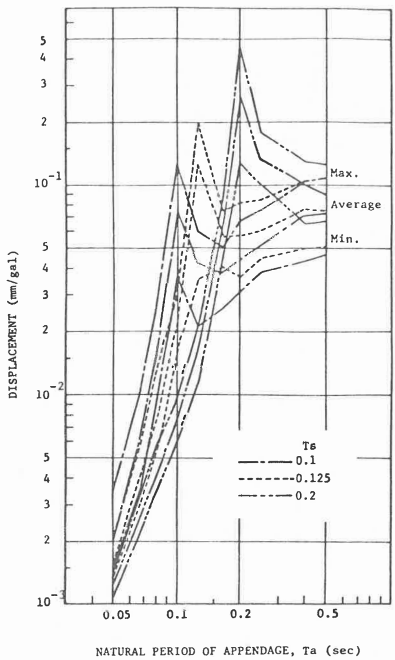

Fig. 4 Maximum response displacement of appended system

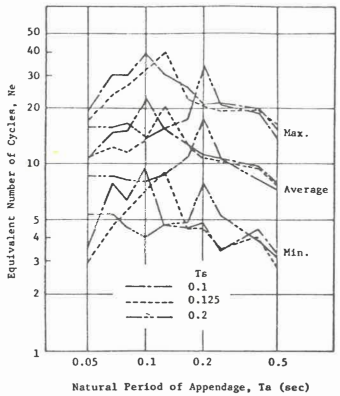

Fig. 5 Number of equivalent cycles for artificial earthquakes

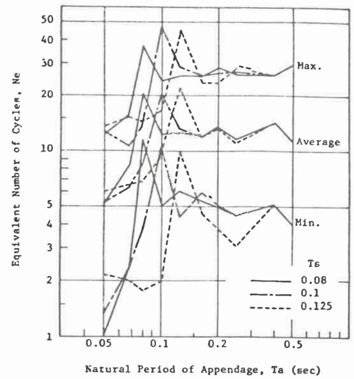

Fig. 6 Number of equivalent cycles for actual earthquakes

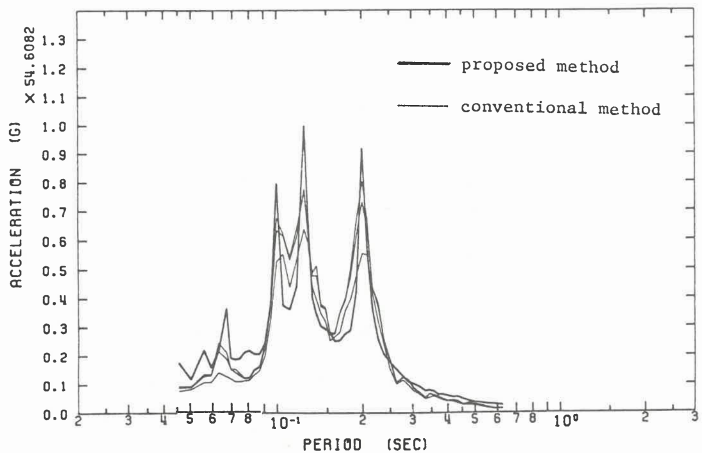

Fig. 8 Floor response spectra

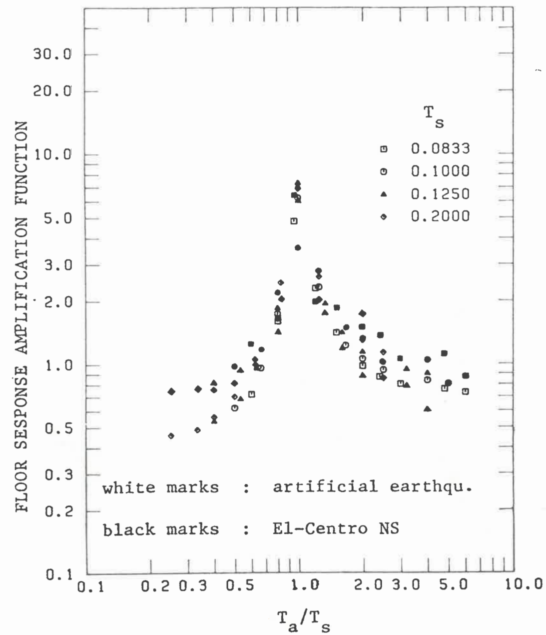

Fig. 7 Floor response amplification function