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DEVELOPMENT OF GENERIC FLOOR RESPONSE SPECTRA FOR EQUIPMENT QUALIFICATION FOR SEISMIC LOADS

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ABSTRACT

A generic floor response spectra has been developed for use in the qualification of electrical and mechanical equipment in operating nuclear power plants. Actual PWR and BWR - Mark I structural models were used as representative of a class of structures. For each model, the stiffness properties were varied, with the same mass, so as to extend the fundamental base structure natural frequency from 2 cps to 36 cps. This resulted in fundamental mode coupled natural frequencies as low as 0.86 cps and as high as 30 cps. The characteristics of 1000 floor response spectra were studied to determine the generic spectra. A procedure for its application to any operating plant has been established. The procedure uses as much or as little information that currently exists at the plant relating to the question of equipment qualification. A generic floor response spectra is proposed for the top level of a generic structure. Reduction factors are applied to the peak acceleration for equipment at lower levels.

INTRODUCTION

A generic floor response spectra has been developed for use in the qualification of electrical and mechanical equipment in operating nuclear power plants. The characteristics of 1000 floor response spectra were studied to determine the generic spectra. A procedure for its application to any operating plant has been established.

The background of a generic spectra starts with the concept that there is a degree of boundedness to the structural responses, characteristics and fragility limits of the various equipment in operating plants which arise from the similarities. The report follows this concept and shows that the response can be bounded within a useful range.

The general approach in the development of the generic floor response spectra was to study the effects on the dynamic characteristics of each of the elements in the chain of events that goes between the loads and the responses. This includes the loads, the soils and the structures. Actual structural models were used in the study. A free-field earthquake response spectra was used to generate horizontal earthquake time histories. In reporting the proposed generic response spectra, the peak values were normalized to a more realistic time history peak of 0.1 g. The excitation was applied through the soil and into the various structures to produce responses in equipment at each level. An entire range of soil conditions was used with each structure, from soft soil to solid rock, in seven steps. Actual PWR and BWR - Mark I structural models were used as representative of a class of structures. For each model, stiffness properties were varied, with the same mass, so as to extend the fundamental base structure natural frequency from 2 cps to 36 cps. This resulted in fundamental mode coupled natural frequencies as low as 0.86 cps and high as 30 cps. From all of these models of soils and structures, floor response spectra were generated at each floor level. The cases studied for this report as shown in Fig. 1.

The natural frequencies of the structures were varied to obtain maximum response conditions. The actual properties were first used to locate the natural frequencies. The stiffness properties were than varied, with the same mass, to extend the range of the fundamental base structure natural frequency to produce a coincidence with the frequency of an excitation spectrum.

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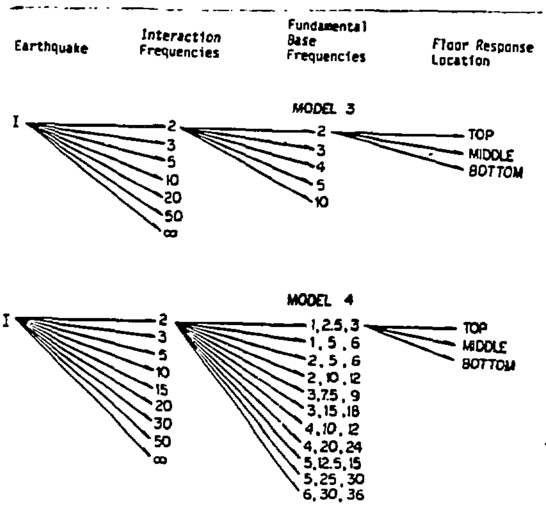


Fig. 1 Interaction and Model Frequencies Used

A horizontal generic floor response spectra is proposed. Different spectra are proposed for different soils over the full range of soil properties. These are shown to be valid for structures with different structural natural frequencies and therefore for a generic structure.

Only a horizontal spectra is discussed in this report. Vertical spectra were not studied because of time limitations. However, vertical generic spectra could be obtained in a similar manner.

The results show that the maximum floor response acceleration was obtained at the top level of a structure which was built upon solid rock. The peak was found to be 6.0 g's for the 0.1 g earthquake. This was increased by 20 percent to 7.2 g's because of certain limitations in the study. As the shear wave velocity of the soil decreases (softer soil), the maximum floor response acceleration decreases. The peak acceleration at the top level of a structure on soft soil was taken to be 5.0 g's. This is 30 percent less than the peak floor response acceleration of 7.2 g's at the same elevation for a solid rock soil.

Mechanical and electrical equipment must be designed to withstand the full range of normal and earthquake loads. For seismic, the original time history of the ground motion is generally not available. Seismic excitation is described in terms of an acceleration response spectra rather than as a time history of the acceleration. Seismic loads are frequently developed from a ground response spectrum. This is done in accordance with Regulatory Guide 1.60 which describes a procedure for defining response spectra for the seismic design of nuclear power plants. The general development of floor response spectra from the ground response spectra is shown in Fig. 2.

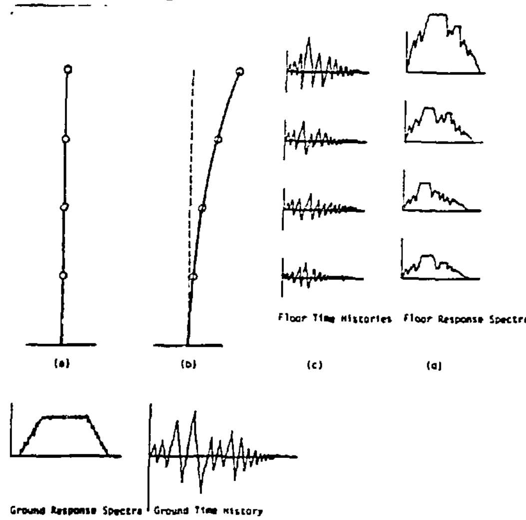


Fig. 2 General Development of Floor Response Spectra

From the ground response spectra, an admissible time history is deduced. The ground time history is used to obtain the floor response time histories. Each of these floor response time histories are used to determine the floor response spectra. The floor response spectra are the basis for the evaluation of the response of any equipment located on that floor. Basically, what is needed to use the floor response spectra is an understanding of the natural frequencies of the equipment.

GENERIC RESPONSES

There are many similarities between the various structures and equipment of operating nuclear power plants. Basically, almost all are producing power by two different methods: BWR or PWR. For each of these systems, certain equipment must be used to maintain and control the process. While there are, of course, many differences, there are also many structural features and system control characteristics that they have in common. The development of a generic spectra starts with the concept that there is a degree of boundedness to the structural responses and to the equipment characteristics and fragility limits of the various equipment in operating plants which arises from the similarities. Whether the responses can be bounded within a useful range, or whether the study results in a range that is entirely too wide to be of any practical use in a particular situation remains to be seen. This is the essence of this generic response study.

EARTHQUAKE SPECTRA-DEVELOPING THE TIME HISTORY

The procedure used to generate a ground acceleration time history to match the criteria response spectra (as specified in Reg. Guide 1.60) is essentially a trial and error procedure. The method begins by choosing an actual measured strong motion accelerogram which has a relatively long duration (say at least 20 seconds) and relatively wide banded frequency content.

Three separate measured time histories were chosen, to initiate the study (1940 Imperial Valley, 1952 Kern County, and a previously modified El Centro History). For each of these excitation pulses, a time history was developed which closely enveloped the required response spectra. The procedure was utilized to generate the final enveloping criteria motions as input to the structural models. The earthquake spectra used in this study is shown in Fig. 3.

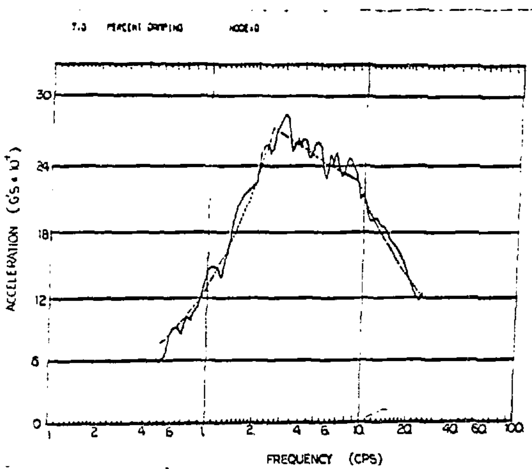


Fig. 3 Earthquake Spectra Used in This Study

DEVELOPING THE DYNAMIC MODEL

The free-field earthquake is applied to the base of the power plant structure. The dynamic model is developed to describe both the structure itself and the foundation interface of the structure with the free-field earthquake.

A typical soil-structure interaction model is shown in Fig. 4. The model shows all of the essential mass and elastic elements for the structure. Springs are shown at the base of the structure to represent the horizontal and rocking soil interation frequency characteristics. The mass nodes are identified by the numbers 1-9. Stiffness elements are shown joining the mass points. They represent the essential overall equivalent dynamic stiffness of the structure between the mass points. From level 1 on up, the stick model represents the structure as though it were built upon a rigid foundation. The free field-soil structure interface is modeled by springs, as shown below and on either side of node 1. The free-field earthquake is shown imposed on the opposite side of the soil elements.

The dynamic characteristics of the soil are described by soil parameters which include a vertical interaction frequency, a horizontal interaction frequency and a damping value associated with the interaction frequency. These are pictured in Fig. 4, as vertical springs and horizontal springs, both with parallel dampers. By changing the values of these parameters, all types of soils can be characterized. For solid rock, for example, the interaction frequency are taken as infinite. This produces the effect of imposing the earthquake directly on this base of the structure.

One excitation pulse was used with two different models. One model is of a BWR and the other is a PWR. Each was analyzed for their dynamic characteristics as per the description of the stick models with the values of mass and stiffness that was provided. The stiffness properties were then varied, with the same mass, to maximize the response. The modulus of elasticity was then adjusted, both up and down, to provide fundamental base natural frequencies that took on the values of between 2 and 35 cps. Each integer frequency was selected in the region where the pulse produced the most significant peaks.

SOIL-STRUCTURE INTERACTION

Structure In Media (SIM) Code determines the response of a structure embedded in a soil/rock media, to a specified dynamic disturbance in the media. The structure is modeled as a series of lumped mass, elastic beams which may be interconnected with elastic springs. The disturbance in the free field is specified in terms of an accelerogram, the scale of which may vary with depth. Soil/structure interaction in two parts. The first part determines the interaction forces developed at the base of the structure while the second evaluates the interaction forces developed along the side walls of the structure.

The output from SIM Code consists of displacement time histories and/or response spectra at specified nodes within the structure. The shock spectra may be obtained as: a listing, an on line plot; a Calcomp plot; or any combination of the three.

The analytical basis for SIM Code is presented in Reference (6). Detailed control and input requirements for using the code are also discussed in the reference.

SOIL-FOUNDATION INTERACTION PARAMETERS

The determination of the base interaction parameters is made by comparison with analytic solutions obtained for the steady state response of a uniform elastic half-space. The solutions were originally obtained by Bycroft(6) for circular foundations on the surface of the half-space. From these analyses, a series of studies were conducted from which equivalent base interaction springs and dampers were determined. In all cases, it was found that the base interaction constants were a function of the input steady state frequency. For a typical seismic input, however, the free field motion consists of energy over a band of frequencies. This implies that the interaction parameters should then be a function of frequency of the input motion. Fortunately, for most of the parameters, this frequency dependence is not severe so that constant spring and dashpot parameters can be used.

The specific parameters used in the analysis are based upon the relationship listed below.

Horizontal Spring Constant:

$$K_x = 0.9 \frac{32\ (1-\mu)}{(7-8\mu)} \rho V_s^2 a$$

Horizontal Dashpot Constant:

$$C_x = (0.57) \frac{32\ (1-\mu)}{(7-8\mu)} \rho V_s a^2$$

Rocking Spring Constant:

$$K_\theta = (0.8) \frac{8}{3(1-\mu)} \rho V_s^2 a^3$$

Rocking Dashpot Constant:

$$C_\theta = (0.15) \frac{8}{3(1-\mu)} \rho V_s a^4$$

In these equations, V_s is the shear wave velocity ($=\sqrt{G/\rho}$), G is the modulus of the soil, ρ is the soil mass density, μ is Poisson's ratio, and "a" is the base radius. The coefficients in parentheses in each equation are average values of the frequency dependent coefficients discussed above.

The equations for the soil spring constants show that the horizontal interaction frequency is approximately equal to the rocking interaction frequency. For this study both interaction frequencies will be taken as equal to each other. The lower value of the interaction frequency, which represents the dynamic characteristic of soft soil, will be taken as 2 cps. The entire range of soil frequency conditions will be examined, from 2 cps up to infinite frequency for solid rock.

The soil damping shows a range from 30 to 40 percent for horizontal motion and 10 to 20 percent for the rocking motion. For this study, the horizontal soil damping will be taken as 30 percent and the damping in the rocking direction will be taken as 10 percent.

SUMMARY OF DAMPING VALUES FOR GENERIC SPECTRA

There are four separate values of damping that must be specified to develop the floor response spectra. Two damping values must be assigned to the soil-structure interaction frequencies, a third value must be chosen for the structural response and a forth must be used in the one-degree-of-freedom systems which represent the equipment damping associated with the generation of the floor response spectra. The values that were used in this study are listed in Table 1.

Table 1
Damping Values Used in This Study

Frequency Item	Percent Damping Ratic
Horizontal Interaction Freq.	30
Rocking Interaction Freq.	10
Structure	5
Equipment	2

BWR MODEL AND RESULTS

The BWR modeled in this case as a single stick, as shown in Fig. 4 where it is labeled Model 3. This includes the containment structure of a BWR. The foundation is shown at an elevation of 26' below ground level. The height of the containment is at an elevation of 147'-25". A total of 9 mass points are used to describe the distribution of mass.

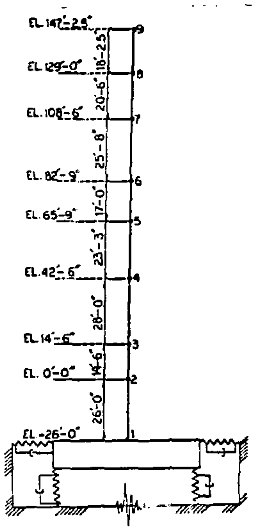


Fig. 4 Model 3

The actual fundamental natural frequencies were calculated to be 2.3 11.5 and 13.8 cps. There are the frequencies for the structure when it is embedded in solid rock. This is called the base structural natural frequency. For this case, soil structure effects are not involved. These frequencies were proportionally adjusted by changing the value of the elastic modulus so that the fundamental base frequency assumed the values of 2.0, 3.0, 4.0, 5.0 and 10.0 cps.

A computer analysis was made of the soil structure coupled natural frequencies for each of these five cases. Seven different soil interaction frequencies were selected to represent the entire range of soil conditions from soft soil (interaction frequency of 2 cps) to solid rock (interaction frequency of infinite). A total of 35 cases of soil-structure interaction were therefore studies, i.e., 5 different structural fundamental natural frequencies interacting with seven different kinds of soils.

The first three coupled natural frequencies for each of these cases were studied. The combinations produce combined fundamental natural frequencies that blanket the spectrum from 1.17 cps for the first mode, from 3.11 to 28.37 for the second mode and from 6.46 to 41.30 or the third mode. There were 31 natural frequencies between the ranges of 2 Hz to 4 Hz.

Floor response spectra were recorded at three levels. This includes node 1 at bottom, node 5 which is centrally located and at node 9, the top of the structure. The maximum accelerations were recorded for each of the 105 cases. The largest peak value of 60.3 g occurred at the top of the structure (node 9) with a structural natural frequency of 4 cps and infinite interaction frequency.

FACTORS AFFECTING RESPONSE

When a transient excitation is imposed upon a structure, several factors come into play to produce response peaks. The first is the nature of the pulse itself, its magnitudes, its frequency content and its duration. For an earthquake type of pulse, most of the energy is contained in the region between 2 to 10 cps. The highest amplitude components are located in the lower portion of this frequency band. A second factor is the proximity of a natural frequency to the frequency of the excitation peaks. The further the spread between the natural frequency and the peak, the less the response. The third is the damping value associated with the normal mode. The response is inversely related to the damping so that the greater the damping the less the response. The fourth is the modal participation factor. In an ideal situation, if a modal participation factor is zero, theoretically no response is possible even though the excitation is large and has a forcing frequency that is equal to the natural frequency. Finally, there is a fifth factor, the duration of the pulse, or the duration of the critical portions of the pulses which may affect the response. The development of a large response takes time even under the optimum excitation.

SOIL-STRUCTURE RESPONSE RESULTS - MODEL 3

The soil parameters of interaction frequency and damping greatly affect the nature of the response. The large differences between the structure damping and the soil damping create a system in which the modal damping can vary between the two values. Rigid body modes, which emphasize the soil properties, are highly damped and so the response are small. On the other hand, structure modes are associated with lighter damping and therefore can have larger responses.

These effects are seen in the response plots for node 9 as shown in Fig. 5. This figure shows the effect of soil-structure interaction on the response. This figure is for a structural natural frequency of 2 cps. Seven curves ar shown in the figure, one for each interaction frequency, including 2 cps, 3, 10, 20, 30, 50 and ∞ cps. Model damping numbers and corresponding natural frequencies are shown alongside some of the curves. The curve with interaction frequency for 3 cps (Case ADAA) has modal damping valves of p_1 = 10.50% and p_2 = 19.2%. The coupled natural frequencies are f_1 = 148 cps and f_2 4.09 cps. However, with interaction frequency of 30 cps (Case AEAA), the damping is p_1 = 5.08% and p_2 = 5.11% with frequencies f_1 = 2.00 cps and f_2 = 5.663 cps.

Large responses are produced in the case where the interaction frequency is high. Two peaks are produced for Case AEAA. The first peak is 42 g which occurs at 2.0 cps and the second is 51 g at 5.6 cps. At the lower interaction frequency, Case ADAA shows a peak of 18 g at 1.48 cps. and a second maximum of 16 g at about 4 cps. The responses in the two cases show the effect of modal damping in producing different peak amplitudes.

On the basis of the combined natural frequency alone, one would ordinarily expect that the largest response would occur at a frequency between 3 and 4 cps because the free field excitation has its largest values in this frequency range. However, the modal damping in this mode is primarily influenced by the soil, and so with p_1 = 19.2%, the peak response amplitude builds up to only 16 g.

On the other hand, Case AEAA has a second natural frequency of 5.6 cps with a modal damping of 5.1%. Even though the input excitation is reduced at this frequency, a response of 51 g builds up because of the reduced damping.

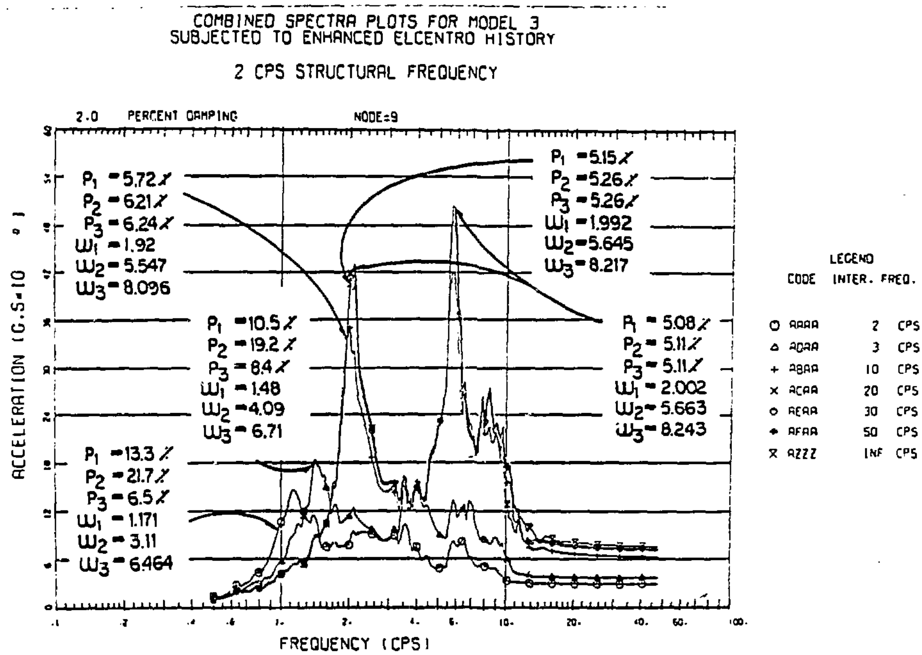


Fig. Combined Spectra Plot for Model 3 - 2 cps Structural Frequency

All of the other figures tell essentially the same story. In fact, with soft soil, interaction frequency of 2 cps, the maximum response is less than 13 g for all cases of structural frequency.

PWR MODEL

The PWR that was studied is shown in Fig. 6 and is labeled Model 4. The system is modeled as three separate structures on a common foundation. Three stick models are used to represent the shield structure, the steel containment and the interal structure. The shield structure rises to a height of 192' above the foundation, while the internal structure is only 39' high. Masses are shown lumped at the nodal points.

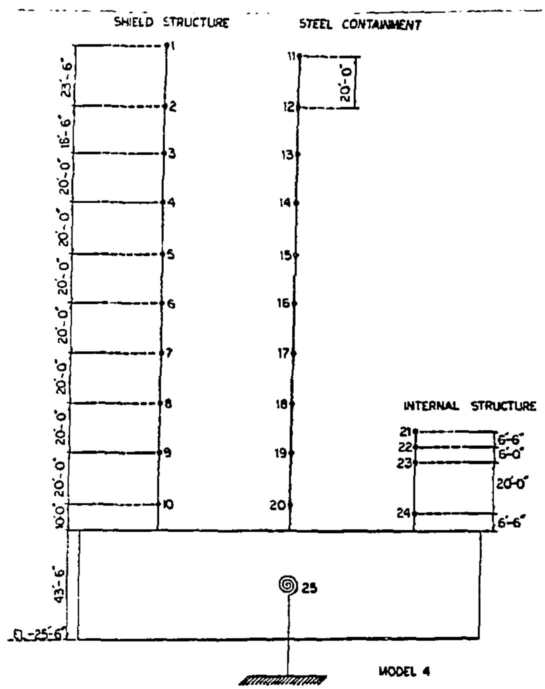


Fig. 6 Model 4

The same procedure was used in this to adjust the fundamental natural frequency of the system. Since there are three structures, the adjustment was made on the lowest natural frequency of the three. The "as is" natural frequencies of the separate structures on a rigid base were calculated to be in the ratio of 1:5:6, with the shield structure having the lowest value and the internals the highest.

The rigid base structural frequencies were again varied over the frequency range where the largest input excitation peaks were located. For this case, six integer frequencies were used for the shield structure, with the lowest value of 1 cps and the highest 6 cps. The frequency ratio between the three structures was maintained as 1:5:6. A second set of runs was also made where the ratio changes to 1:2.5:3.

Because of the greater complexity of the model, nine interaction frequencies were used to represent the entire range of soil conditions. This includes interaction frequencies of 2,3,5,10,15,20,30,50 and infinite cps.

The cases studied involved 11 sets of structures in combination with 9 interaction frequencies for a total of 99 different structural problems. Floor response spectra were obtained at 9 locations. In all, a total of 891 floor response spectra were studied for the PWR Model 4.

COMBINED SPECTRA - GENERIC SPECTRA

The information obtained from the study of the BWR and the PWR was combined to obtain a generic spectra.

The maximum acceleration of 60 g was produced in the stick Model 3 while the widest frequency spread was associated with the multiple stick Model 4. Therefore, Model 4 will be used to obtain the frequency spread characteristics of the peaks on the generic spectra. The frequency range will be widened to account for the anticipated effect of using other pulses. The peaks will then be scaled up to maximum peak acceleration value developed in Model 3. An additional 20 percent factor will be added to this in view of the methodology used in the study. The responses spectra will therefore have a peak of 72 g. These numbers were developed for a normalized 1 g peak excitation time history pulse. Applications to other cases of different time history acceleration peak will have to be appropriately scaled.

The magnitudes have been set in Fig. 7 as described. The frequency width at the peaks, and the slopes at the sides of the peaks, conservatively envelop all of the responses. The ZPA is bounded between 0.8 g and 0.4 g for the different soils. These are the proposed generic spectra for the topmost point on the models. They are based upon the use of local soil conditions. If no information is used for the soil condition, the largest spectra, the one for solid rock soil, should be used.

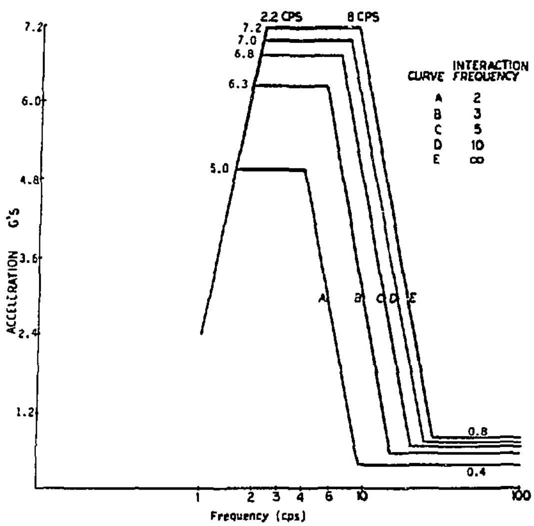


Fig. 7 Generic Floor Response Spectra (Normalized to 0.1 g peak time history)

Figure 7 is for the top floor. A similar procedure was used to determine the response at the middle and lower levels. Reduction factors were evaluated for equipment located at these lower levels. The factors can be deduced from Fig. 8.

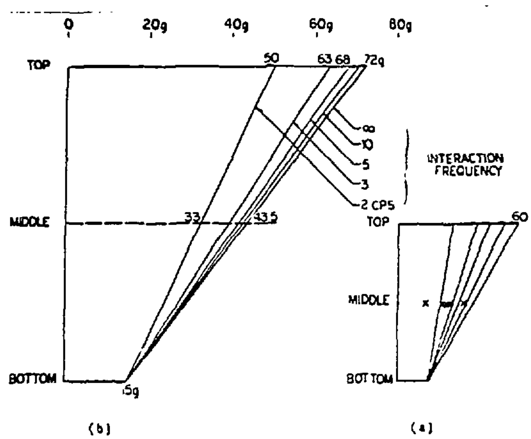


Fig. 8 Generic Peak Responses at Top, Middle and Bottom Levels. Peak Values for 1.0 g Time History Peak.

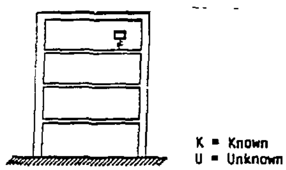
USE OF GENERIC SPECTRA - GENERAL PROCEDURE

Figure 9(a) shows the general problem of equipment qualification. A safety related equipment item is mounted at some level inside the nuclear power plant. It is required to establish the seismic qualification for the equipment.

For some operating nuclear power plants, all of the information regarding the soil, the structure and the equipment that is needed might be available. The qualification procedure is well defined. Generic spectra do not have to be used since the plant specific information can be applied directly. The generic spectra are intended to be used for those cases where only part of the information can be obtained.

Figure 9(b) shows the choices. It is assumed that the equipment fragility or malfunction limits or some knowledge about a limiting safe environment are known or can be deduced by similarity or by in-situ or laboratory tests. This is the last column in the figure. Six cases are listed along with whether the required qualification information is either known (K) or unknown (U) for the soil, the soil-structure response and the equipment characteristics.

Case A requires the greatest amount of engineering information, but the procedure for qualification is clearly defined by existing documents and do not need to refer to Generic Floor Response Spectra. In Case B, only the dynamic characteristics are unknown. As shown in the last column, the qualification procedure is as stated in Reference 8 and the frequency range of the Generic Floor Response Spectra do not have to be used. This study was undertaken for those cases where less than complete information is currently available, but yet, the equipment must be qualified. The Generic Floor Response Spectra that were developed in this report define a procedure for the qualification of electrical and mechanical equipment in operating nuclear power plants for cases C, D and E or where only equipment malfunction information is available, as in Case F.



(a)

Case	Soil Condition		Coupled Structure Characteristics		Equipment Characteristics		Procedure*
	K	U	K	U	K	U	
A	O		O		O		Direct- Standard Procedure
B	O		O			X	Plant Specific Section 5.3.3
C	O			X	O		Generic Floor Response Spectra Section 5.3.2
D	O			X		X	Select Site Spectrum and use Max. Accel.
E		X		X	O		Section 5.3.2, but use Generic Spectra Curve E
F		X		X		X	Use Maximum Acceleration of Generic Spectra Curve E

(b)

* (see Reference 8)

Fig. 9 General Procedure for Equipment Qualification in Operating Plants

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