VERTICAL DESIGN RESPONSE SPECTRA FOR ROCK SITES

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SUMMARY

The state of the art of nuclear plant seismic design involves the use of design response spectra together with modal analysis of a mathematical simulation of the actual structure. In an effort to describe and make available to the public methods and techniques acceptable to the USAEC Regulatory Staff for implementing specific parts of the USAEC regulations pertaining to design for natural phenomena, such as earthquakes, the USAEC published Regulatory Guide 1.60 entitled, "Design Response Spectra for Seismic Design of Nuclear Power Plants," in October 1973. It emphasized that Regulatory Guides are not substitutes for regulations and compliance with them is not required. Methods and solutions different from those set out in the guides are acceptable if they provide a basis for the findings requisite to issuance of a permit by the USAEC.

The USAEC prepared Regulatory Guide 1.60 on the basis of work performed by Newmark and Blume (WASH-1254, WASH-1255). Whereas the work by Blume in WASH-1254 is concerned primarily with horizontal motions, the work by Newmark in WASH-1255 treats both horizontal and vertical motions. Both researchers address the question of foundation compliances in a limited fashion, i.e., accelerograms recorded on rock versus accelerograms recorded on soil, but the corresponding effect is not reflected in the Regulatory Guide. This is understandable in that it is the objective of the Regulatory Guide to be useful for sites underlain by either rock or soil deposits.

The purpose of this paper is to present the results of additional research with respect to vertical response spectra for rock sites. Whereas Newmark (WASH-1255) treats vertical motion, only three of the fourteen vertical earthquake records considered were made on rock. However, both Blume (WASH-1254) and Newmark (WASH-1255) recognized that response spectra for records made on rock are less severe than response spectra for records made on soil deposits. Further, Newmark recommends that further studies be conducted to define more clearly the effect of site characteristics.

The research reported herein is based on analyses of thirty vertical recordings made on rock or rock-like sites. All analyses and statistical interpretations are very similar to the methods used by Newmark in WASH-1255. Each of the thirty strong-motion accelerograms was first baseline corrected following the methods of M.D. Trifunac by filtering the uncorrected accelerogram below 0.07 Hz. The baseline corrected accelerograms were then corrected for instrument response using the appropriate instrument natural periods and damping values. The corrected records were then used to compute damped response spectra at a total of 125 frequency points between 0.05 Hz and 50 Hz. Each of the individual response spectra was then normalized to peak ground motions of 1.0 g acceleration, 1.0 in/sec velocity and 1.0 inch displacement, respectively, thereby forming three normalized spectra for each record for each damping value considered. The resulting normalized spectra were then processed statistically to yield smooth design response spectra for each damping value following the methods of Newmark (WASH-1255).

The results of this research effort indicate that vertical response spectra for rock sites should be different from those sites on soil and somewhat less severe.

1.0 INTRODUCTION

Vertical design response spectra based upon a statistical study of 30 historical earthquakes obtained upon rock sites are reported herein. The recommended design response spectra for rock sites are somewhat less severe than previous recommendations,[1,2,3] specifically those recommended in USAEC Regulatory Guide 1.60. Results of the present study are summarized and a detailed description of the strong motion accelerogram processing and statistical analysis is presented.

2.0 SELECTION OF ACCELEROGRAMS

One of the issues encountered in the study of strong motion records for "hard" or rock sites is the classification of sites with respect to rock or soil. Several records can be easily distinguished as being recorded on rock. These are the Helene, Montana (October 31, 1935); Bear Valley, California (June 22, 1973); Blue Mountain Lake, New York (July 29, and August 3, 1973); Koyna Dam, India (December 11, 1967); Hsinfengkiang Dam, China (March 19, 1962); San Francisco, California, Golden Gate (March 22, 1957); and San Fernando, California (February 9, 1971) Pacoima Dam and California Institute of Technology Seismological Laboratory Records. For the remainder of the records used in the study, the distinction between a "hard" or soil site is not as obvious. One method, as suggested by Blume in Ref [4], for considering the site characteristics is to use a ground impedance parameter defined as the product of specific density and shear wave velocity. Table I shows the strong motion records used in the study with ground impedance following Blume.[4] Using Blume's[4] bounding value of 4000 ft/sec, it is apparent that the records used in this study truly reflect "hard" conditions.

It should be noted that the effect of including a strong motion accelerogram in the study which was in reality recorded at an alluvium site is to increase the amplification factors as shown by Newmark[3] and other investigators. Therefore, the result of classifying a recording made at a soil site as a recording at a rock site tends to bias the results on the conservative side by leading to larger amplification factors.

The earthquakes chosen were associated with fairly weak ground shaking. For the 30 earthquakes chosen the average vertical acceleration is approximately 0.076 g or well below usual design levels. Previous investigators[3,4] have indicated a negative correlation between peak ground acceleration and response amplifications. Hence, response amplifications derived from these 30 records are considered conservative when used for design.

3.0 DATA PROCESSING TECHNIQUES

Vertical strong-motion time history records were corrected using standard processing techniques for both long period baseline errors and high frequency instrument response errors. Baseline correction was performed by high pass filtering data using a digital filter with a cut-off frequency of 0.07 Hz. Instrument correction was performed by low pass filtering of data with a cutoff frequency of 25 Hz and using a single degree of freedom representation of the instrument to obtain ground motion. The cutoff of 25 Hz was chosen on the basis of the observation that above this frequency, errors resulting from low signal-to-noise ratios, and spurious instrument response can lead to significant errors.[6] The algorithms and theoretical basis for these corrections are discussed more fully in Refs [5] and [6], respectively. The resulting corrected time histories accurately represent absolute ground acceleration over a frequency band between 0.07 Hz and 25 Hz. This frequency band represents the practical limits over which reliable information may be extracted using present recording and digitation methods.

The work of Newmark[3] applied a "segmental parabolic" baseline correction to the strong-motion time histories. With this approach, the choice of a time interval over which a parabolic baseline is used is somewhat arbitrary. The method used herein has improved physical basis and does not treat earthquakes of different durations differently. The importance of baseline correction is reflected mainly in the integrated ground motion velocity and displacement. As peak ground velocities and displacements are used in this report to normalize response amplifications, the baseline correction would be considered a necessary part of the data processing. It is noted that the work of Blume[4] uses peak ground acceleration as the only parameter for normalizing response amplification and hence no baseline correction was considered necessary.

The instrument response correction mentioned above was not used in the previous work cited[3,4] but was used herein to obtain the maximum possible accuracy at the high frequency end of the spectra. Each corrected time history was then used as input to a single degree of freedom oscillator and the time history response calculated to determine the peak response of the oscillator having a given natural frequency. The frequency of the oscillator was then changed and the response determined again. In this manner, response spectra were calculated at a total of 125 points listed in Table II.

In addition to using the baseline and instrument corrected time history to determine response spectra associated with each strong motion record, the time histories were integrated to determine the velocity and displacement time histories as well as peak velocity and peak displacement. These peak velocities and displacements were then used to calculate ground motion parameters associated with each record.

From the response spectra determined from each strong motion accelerogram, three normalized response spectra were determined by normalizing to the following ground motion parameters suggested by Newmark:[3]

Acceleration - 1.0 g
Velocity - 10.0 in/sec
Displacement - 1.0 in

The normalizing process yielded three normalized sets of response spectra, i.e., acceleration, velocity and displacement, for each strong motion vertical accelerogram. Each of the normalized spectra were used to determine mean amplification factors at each of the 125 frequencies at which the response spectra were computed by averaging over all thirty strong motion records. In the same manner, the standard deviation of amplification factors was determined at each frequency point.

Having determined the mean and standard deviation of the amplification factors at each of the 125 frequencies, the means and standard deviations were averaged over selected frequency ranges following Newmark.[3] Acceleration amplification factors were averaged over the range of 3 to 10 Hz; the velocity amplification factors were averaged over the range of 0.3 to 3 Hz, and the displacement amplification factors were averaged over the range of 0.05 to 0.3 Hz. The results of this averaging process for the mean and standard deviation are shown in Table III.

After assessing the amplification factors, spectrum bounds were then determined by multiplying the amplification factor times the appropriate ground motion parameters confirmed by this study as follows:

Acceleration - 0.667 g
Velocity - 21.0 in/sec
Displacement - 18.0 in

These ground motion input parameters, which are conservative compared to Newmark's[3] values of 17 in/sec and 11 in for velocity and displacement, respectively, have been determined by a detailed review of the records considered in this study as well as those reported by Newmark.[3] First and foremost is the ratio of vertical acceleration to horizontal acceleration. A review of all the records used in this study as shown in Table I indicates a ratio of 0.667 is indeed conservative.

It should be noted that the input parameters for velocity and displacement are based on two ground motion parameters - (V/A) and (AD/V^2), where "A" is the peak acceleration, "V" is the peak velocity and "D" is the peak displacement. The values of (V/A) for the records used herein were averaged for the thirty strong motion records considered and an average value of 33.5 inches/sec/g was found. This corresponds reasonably well with the value of 28 inches/sec/g reported by Newmark.[3] When the largest and smallest values of (V/A) are removed and the average recomputed based on the twenty-eight remaining records, the value of 31 inches/sec/g is even closer to the value reported by Newmark,[3] and was adopted for the response spectrum bounds reported herein.

Since the peak ground displacement obtained by twice integrating the strong motion acceleration record is not considered to be a reliable measure of the displacement, the ground motion parameter (AD/V^2) coupled with the parameter (V/A) is used to estimate the peak ground displacement. The average value of (AD/V^2) from the rock records used herein is about 10.6 or slightly higher than the value of 10 suggested by Newmark[4] for vertical strong motion records. Also, it is noted that while (AD/V^2) is a function of the peak ground displacement, the results will only be influenced in the low frequency range below 0.3 Hz where ground displacement is constant. Frequencies in this range are well below structural frequencies typically encountered in nuclear power plants.

3.1 Faring Frequency

The faring frequency is defined as that frequency at which the amplified ground response spectra intersect the ground motion spectra at a constant peak ground acceleration. In terms of the amplification factors and normalized ground response spectra, the faring frequency is that frequency at which the amplification factors approach unity. In other words, it is the frequency above which the single degree of freedom oscillator responds as a rigid body. Inspection of the normalized response spectra calculation performed earlier in this study with no instrument correction indicates a faring frequency in the neighborhood of 50 Hz, as also suggested by Newmark.[3] It should be noted that the high frequency response, greater than 20 to 25 Hz, indicated on the response spectra is a conservative interpretation since the instruments used to record the strong motion accelerograms are not capable of resolving frequency components greater than 20 to 25 Hz.

3.2 Effects of Sample Size

As mentioned above, Table III presents the results in the form of the mean and standard deviation of the response spectra amplification factors corresponding to 0.5, 2.0, 5.0 and 10.0 percent critical damping. However, the ultimate objective of the present study is to draw conclusions regarding amplified response spectra for earthquakes in general, based on the limited data available in the form of the thirty vertical strong motion rock records considered. In this sense, the records considered in the present study represent a statistical sample possessing sample statistics from which it is desired to draw conclusions regarding earthquakes in general which may be considered to be the population.

3.2.1 Population Mean

As shown in Ref [7], the random variable

$$t = \frac{(\bar{x} - \mu) \sqrt{n}}{s} \qquad (1)$$

where $\bar{x}$ is the sample mean, s is the sample standard deviation, μ is the population mean and n is the sample size, has a t-distribution with n-1 degrees of freedom. Consequently, given the sample mean and standard deviation shown in Table III for the response spectra amplification factors, it is possible to derive confidence intervals for μ, the mean amplification factors of the population.

3.2.2 Population Standard Deviation

Analogous to the sample mean having a t-distribution with n-1 degrees of freedom about the population mean, it is also shown in Ref [7] that the random variable

$$\chi^2 = (n-1)\, s^2/\sigma^2 \qquad (2)$$

where s is the sample standard deviation, σ is the population standard deviation and n is the sample size has a chi-square (χ^2) distribution. Thus, from the sample amplification factor standard deviations shown in Table III it is possible to derive confidence intervals for the population standard deviation.

3.2.3 Sample Size

Both the t-random variable and the χ^2 random variable are based on the statistics of a sample of n independent random variables. Thus, a prerequisite for determining confidence intervals for the mean and standard deviation of the population is the statistical independence of the n samples from which the sample statistics were derived.

The sample sizes from which the sample mean and standard deviations shown in Table III were computed arise from two sources. First, the number of records considered affects the sample size. Since each historic strong motion event was a physically independent event or recording of the same event having differing geophysical paths from source to recording station, it follows that the thirty strong motion records considered represent thirty statistically independent events. This fact was also acknowledged by both Newmark[3] and Blume[4] and was inherent in their probability assessments based on normal and log normal distributions which require statistically independent events.

In addition to the sample statistics being based on thirty statistically independent strong motion records, each amplification factor statistic was averaged over a range of frequency points as shown in Table III. For two single degree of freedom oscillators having natural frequencies ω_1 and ω_2 where ω_1 and ω_2 are sufficiently far apart, the response of the oscillator with frequency ω_1 is statistically independent from the response of the oscillator with frequency ω_2. This, in fact, serves as the basis of the well known and accepted method of obtaining the response of a multiple degree of freedom system by taking the square root of the sum of the squares of individual modal responses, the SRSS method. Thus, for frequencies at which the response spectra were determined which are sufficiently far apart, the response spectrum ordinates arising from any one strong motion record may be considered to be statistically independent samples of the response spectra amplification factors.

With respect to the question regarding the required frequency spacing for response spectrum ordinates to be statistically independent, recourse is made to the accepted USAEC criteria for the combining of closely spaced modes as presented in USAEC Regulatory Guide 1.92. Modal responses are considered statistically independent if the modal frequencies differ by greater than 10% of the lower frequency. Within the 10% range, modal responses, and hence response spectrum ordinates, are considered to be statistically dependent to an undetermined degree.

Table II presents the grouping of frequency points between which the response spectrum ordinates may be considered statistically independent for each strong motion record. The sample sizes shown in Table IV corresponding to the three frequency ranges used in the averaging of the normalized response spectrum statistics were determined by multiplying the number of statistically independent frequency points by 30, the number of statistically independent strong motion records considered.

3.2.4 Confidence Intervals for Population Mean and Standard Deviations

Using the t and χ^2 distributions, it is possible to derive confidence interval estimates of the population mean and standard deviation which have the effect of removing the uncertainty involved in applying sample statistics to the entire population thereby considering the influence of sample size. Since the actual population statistics may be either above or below the sample statistics, the confidence intervals derived from the t and χ^2 distributions are two sided about the sample statistics.

By definition, a confidence interval is a concept of statistical decision theory that assesses the level of confidence of accepting a hypothesis regarding population statistics based on sample statistics. It is not a probability assessment per se. In removing the effect of sample size from the response spectra statistics, a range is sought in which the population statistic will fall when the sample statistic has a given value with a specified level of confidence. For the present study, 95% confidence intervals were considered adequate in removing the effect of sample size. In other words, given the response spectra amplification factor means and standard deviations presented in Table III, it can be stated that the population means and standard deviations will fall within the range given in Table V with 95% confidence. Table VI was thus prepared utilizing the upper bound confidence interval estimates of the population means and standard deviations, i.e., the mean plus one standard deviation.

4.0 RESULTS

It is noted that this same approach, i.e., the mean plus one standard deviation, was used in the preparation of USAEC Regulatory Guide 1.60 where only 14 vertical records [three (3) rock and eleven (11) soil sites] were considered as opposed to 30 rock records here. Also, use of the mean plus one standard deviation is justified on the basis of 95% confidence limits used in the present study; whereas, no such justification is provided with Regulatory Guide 1.60.

The overall results of this research study are shown on a series of figures included herewith. Amplification factors and spectral bounds derived from this study are presented in Table VII, Figures 1 to 4 corresponding to damping values of 10 percent, 5 percent, 2 percent and 0.5 percent, respectively, present vertical design response spectra for three cases. Spectrum No. 1 corresponds to the vertical design response suggested for all types of sites, both soil and rock, by Regulatory Guide 1.60. Spectrum No. 2 is the vertical design response spectrum (mean plus one standard deviation) resulting from this research study for "hard" or rock sites and Spectrum No. 3 is taken from Newmark's work[3] (mean plus one standard deviation). It is seen that the Regulatory Guide 1.60 is overly conservative for "hard" or rock sites. This latter consideration was more or less expected from the work of Newmark[3] by virtue of the fact that the objective of the Regulatory Guide was to envelop both soil and rock sites.

The response spectra recommended by this study may be conservative for the following reasons:

(1) The use of records with fairly weak ground shaking which generally yields higher amplifications than records of typical design level intensity.

(2) The ratio of maximum vertical to horizontal ground acceleration has been conservatively taken as 0.667.

(3) The faring frequency has been conservatively selected as 50 Hz when more accurate instrumentation may justify a lower value.

(4) The time history records used in the analysis probably included some "soft" rock sites.

5.0 CONCLUSIONS

Following the detailed approach of Newmark in Ref [3] and his suggestion that further study of site characteristics be conducted, a research study has been conducted with vertical design response spectra for "hard" or rock sites. Specifically, wherein Newmark[3] tested only three (3) motion records from "hard" sites, this study utilized a total of thirty (30) records and followed his normalization procedures and statistical analyses without significant deviation. It is concluded that this additional research work has shown that vertical design response spectra suggested in Regulatory Guide 1.60 for "hard" or rock sites are overly severe.

LIST OF REFERENCES

[1] "Design Response Spectra for Seismic Design of Nuclear Power Plants," Regulatory Guide 1.60, USAEC, October 1973 (Revised December 1973).

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[3] N.M. Newmark Consulting Engineering Services, "A Study of Vertical and Horizontal Earthquake Spectra," Urbana, Illinois, USAEC Contract No. AT(49-5)-2667, WASH-1255, April 1973.

[4] John A. Blume and Associates, "Recommendations for Shape of Earthquake Response Spectra," San Francisco, California, USAEC Contract No. AT(49-5)-3011, WASH-1254, February 1973.

[5] Trifunac, M.D., and V. Lee, "Routine Computer Processing of Strong-Motion Accelerograms," Earthquake Engineering Research Laboratory Report 73-03, Pasadena, California, October 1973.

[6] Trifunac, M.D., F.E. Unwadia, and A.G. Brady, "Analysis of Errors in Digitized Strong-Motion Accelerograms," Bulletin of the Seismological Society of America, Vol. 63, No. 1, pp. 157-187, February 1973.

[7] Bowker, A.H. and G.J. Lieberman, Engineering Statistics, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1959.

TABLE I

VERTICAL EARTHQUAKE ACCELEROGRAM RECORDS INCLUDED IN STUDY

Station Location	Date	Time *****	Duration (sec)	Soil Impedance (ft/sec X 10^3)	MAXIMUM VERTICAL GROUND MOTIONS** Acceleration-A (g's)	Velocity-V (in/sec)	Displacement-D (in)	$\frac{ad}{v^2}$	$\frac{v}{a}$	Epicentral Distance (km)	Richter Magnitude M***
Eureka Federal Building	12/21/54	1156	69.97	4.3	0.083	3.394	1.821	5.073	40.9	25	6.6
San Francisco Golden Gate Park	3/22/57	1144	39.72	5.4	0.038	0.475	0.265	17.195	12.5	13	5.3
Helena, Montana, Caroll College	10/31/35	1138	51.07	19.1	0.090	3.822	1.107	2.643	42.5	7	6.0
Eureka Federal Building	12/10/67	0406	30.09	4.3	0.008	0.585	0.520	4.838	73.1	47	5.8
Pacoima Dam, California	2/ 9/71	0600	41.77	Greater than 20*	0.710	22.999	7.677	3.976	32.4	9	6.6
Pacoima Dam, After-Shock at 52.6 Sec.	2/ 9/71	0600	12.64	Greater than 20	0.008	0.418	0.376	6.916	52.3	9	3.0 - 5.0
Pacoima Dam, After-Shock at 69.6 Sec.	2/ 9/71	0600	17.07	Greater than 20	0.021	0.414	0.250	11.636	19.7	9	3.0 - 5.0
Pacoima Dam, After-Shock at 104.6 Sec.	2/ 9/71	0600	30.90	Greater than 20	0.041	0.790	0.406	10.375	19.3	9	3.0 - 5.0
Pacoima Dam, After-Shock at 162.0 Sec.	2/ 9/71	0600	13.11	Greater than 20	0.015	0.227	0.257	28.373	15.1	9	3.0 - 5.0
Pacoima Dam, After-Shock at 230.1 Sec.	2/ 9/71	0600	19.52	Greater than 20	0.025	0.308	0.324	32.41	12.3	9	3.0 - 5.0
Pacoima Dam, After-Shock at 309.1 Sec.	2/ 9/71	0600	16.47	Greater than 20	0.008	0.449	0.616	8.973	56.1	9	3.0 - 5.0
Castaic Old Ridge Route, California	2/ 9/71	0600	61.84	4.3	0.156	2.539	1.38	12.917	16.3	30	6.6
James Ranch, California	6/22/73	0129	13.19	Greater than 20	0.018	0.595	0.475	9.070	33.1	11	3.9
Pinnacles National Monument, California	6/22/73	0129	13.65	Greater than 20	0.036	0.498	0.363	20.271	13.8	8	3.9
Bicmore Canyon, California	6/22/73	0129	14.16	Greater than 20	0.032	0.464	0.532	30.724	14.5	2	3.9
Koyna, India	12/11/67		10.35	Greater than 20	0.246	13.645	9.403	4.804	55.5	13	6.3
China, HsinfengKiang Dam, Station 5, Elev. 42M	****		4.96	Greater than 20	0.012	0.146	0.051	11.347	12.2	5	4.5
Blue Mountain Lake	7/29/73	2007	4.68	Greater than 20	0.005	0.182	0.090	5.675	36.4	1	2.0
Blue Mountain Lake	8/ 3/73	1809	5.54	Greater than 20	0.007	0.299	0.175	5.100	42.7	1	1.9
Cholame-Shandon, California, Array No. 5	6/27/66	2026	44.01	4.3	0.121	2.872	1.346	7.614	23.0	6	5.5
Cholame-Shandon, California, Array No. 2	6/27/66	2026	43.78	4.3	0.216	5.569	1.711	4.612	25.8	1	5.5
Temblor, California	6/27/66	2026	30.42	10.1	0.133	1.726	0.550	9.501	13.0	7	5.5
San Onofre, California	4/ 8/68	1830	45.22	Greater than 4	0.056	1.460	0.675	6.785	26.1	138	6.4
Allen Ranch, Cedar Springs, California	2/ 9/71	0600	14.76	Greater than 20	0.009	0.520	0.371	4.810	57.8	103	6.6
Cholame Shandon, California, Array No. 2	2/ 9/71	0600	29.48	4.3	0.006	0.713	0.826	3.741	118.8	265	6.6
Castaic Old Ridge Route, California	7/15/65	2346	7.19	4.3	0.027	0.228	0.071	14.179	8.4	18	4.3
Allen Ranch, Cedar Springs, California	9/12/70	0630	38.12	Greater than 20	0.060	1.018	0.456	10.278	17.0	19	5.4
Castaic Old Ridge Route, California	9/12/70	0630	16.03	4.3	0.009	0.347	0.243	7.125	38.6	109	5.4
Cal. Tech. Seismological Lab.	2/ 9/71	0600	101.21	Greater than 20	0.085	2.414	1.080	6.091	28.4	36	6.6
San Onofre, California	2/ 9/71	0600	52.48	Greater than 4	0.010	0.577	0.795	9.665	57.7	140	6.6

* Values greater than 20 based on geology reports of granitic or similar type rocks.

** Ground motion parameters based on instrument and baseline corrected time histories.

*** Reported magnitudes vary ± 0.3.

**** After shock following Magnitude 6.1 event on March 19, 1962.

***** Local time.

TABLE II

GROUPING OF FREQUENCY POINTS AT WHICH RESPONSE
SPECTRA WERE COMPUTED
(Hz)

1.	0.050		1.700	34.	6.800
2.	0.150	22.	1.800		7.000
3.	0.167		1.900		7.200
	0.183	23.	2.000		7.400
4.	0.200		2.100	35.	7.600
	0.217		2.200		7.800
5.	0.233	24.	2.300		8.000
	0.250		2.400	36.	8.500
6.	0.267		2.500		9.000
	0.283	25.	2.600	37.	9.500
7.	0.300		2.700		10.000
	0.317		2.800	38.	11.000
8.	0.333	26.	2.900		12.000
	0.350		3.000	39.	13.000
9.	0.367		3.100		14.000
	0.383	27.	3.200	40.	15.000
	0.400		3.300		16.000
10.	0.417		3.400	41.	17.000
	0.433		3.500		18.000
11.	0.450	28.	3.600	42.	19.000
	0.467		3.700		20.000
	0.483		3.800	43.	21.00
12.	0.500		3.900		22.000
	0.533	29.	4.000		23.000
13.	0.567		4.100	44.	24.000
	0.600		4.200		25.000
14.	0.633		4.300		26.000
	0.667		4.400	45.	27.000
15.	0.700	30.	4.500		28.500
	0.733		4.600		30.000
	0.767		4.700	46.	31.500
16.	0.800		4.800		33.000
	0.850	31.	4.900	47.	34.500
17.	0.900		5.000		36.000
	0.950		5.200	48.	38.000
18.	1.000	32.	5.400		40.000
	1.100		5.600	49.	42.000
19.	1.200		5.800		44.000
	1.300	33.	6.000		46.000
20.	1.400		6.200		48.000
	1.500		6.400		50.000
21.	1.600		6.600		

TABLE III

SAMPLE STATISTICS
AMPLIFICATION FACTORS
VERTICAL DESIGN RESPONSE SPECTRA

Statistic	Damping	Amplification Factor		
	%	Displacement (0.05-0.3 Hz)	Velocity (0.3-3.0 Hz)	Acceleration (3.0-10.0 Hz)
Sample Mean $\bar{x}$	0.5	2.12	1.64	3.39
	2.0	1.91	1.32	2.50
	5.0	1.66	1.06	1.91
	10.0	1.41	0.859	1.48
Sample Standard Deviation s	0.5	0.963	0.937	1.72
	2.0	0.869	0.695	1.11
	5.0	0.752	0.512	0.762
	10.0	0.619	0.380	0.536

TABLE IV

NUMBER OF STATISTICALLY INDEPENDENT SAMPLE POINTS USED IN COMPUTING AMPLIFICATIONS

Amplification Factor	Frequency Range	Number of Statistically Independent Frequency Points	Number of Time Histories	Number of Statistically Independent Sample Points
Displacement	0.05-0.3	7	30	210
Velocity	0.3 -3.0	20	30	600
Acceleration	3.0-10	12	30	360

TABLE V

95% TWO-SIDED CONFIDENCE INTERVALS AMPLIFICATION FACTORS VERTICAL DESIGN RESPONSE SPECTRA

Parameter	Damping %	Amplification Factor		
		Displacement	Velocity	Acceleration
Mean* μ	0.5	1.99 - 2.25	1.56 - 1.72	3.21 - 3.57
	2.0	1.79 - 2.03	1.26 - 1.38	2.38 - 2.62
	5.0	1.56 - 1.76	1.02 - 1.10	1.83 - 1.99
	10.0	1.33 - 1.49	0.829 - 0.889	1.42 - 1.54
Standard** Deviation σ	0.5	0.876 - 1.06	0.886 - 0.993	1.60 - 1.86
	2.0	0.791 - 0.961	0.657 - 0.737	1.03 - 1.20
	5.0	0.684 - 0.832	0.484 - 0.543	0.709 - 0.822
	10.0	0.563 - 0.685	0.359 - 0.403	0.499 - 0.579

* Two sided confidence interval based on "t" distribution.

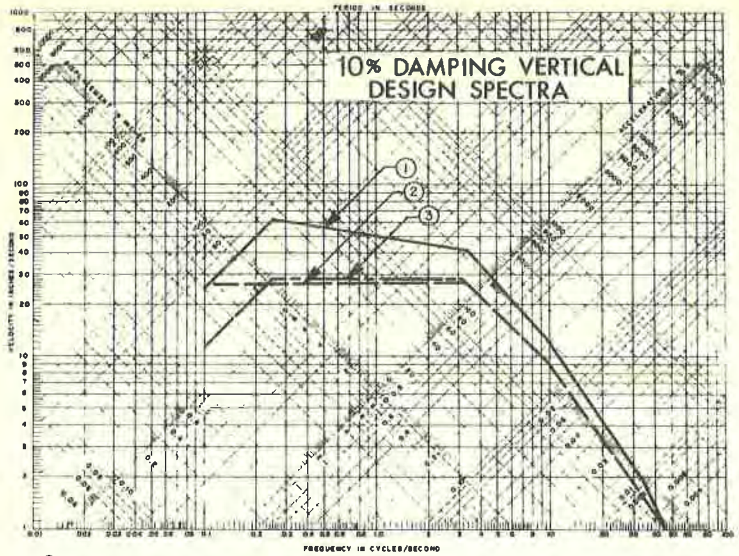
** Two sided confidence estimate based on chi-squared distribution.

TABLE VI

DESIGN AMPLIFICATION FACTORS AND SPECTRAL BOUNDS VERTICAL DESIGN RESPONSE SPECTRA

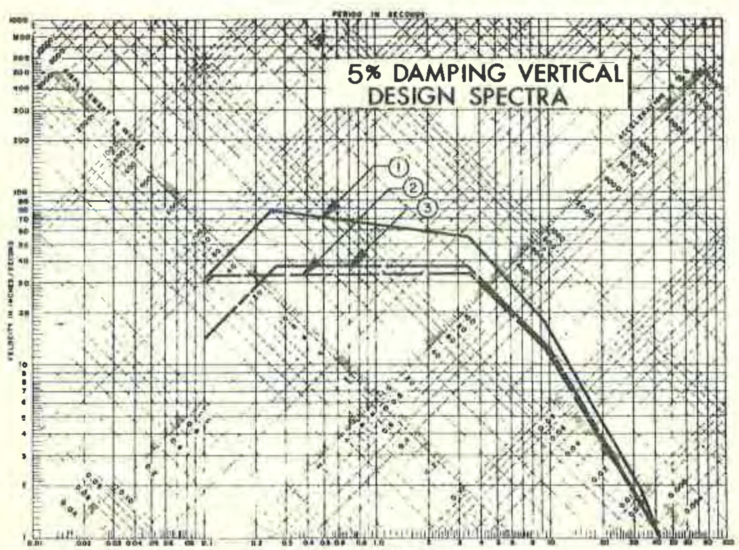
Damping %	Amplification Factor			Spectral Bounds*		
	Displacement	Velocity	Acceleration	Displacement (in)	Velocity in/sec	Acceleration g's
0.5	3.31	2.71	5.43	60	57	3.62
2.0	2.99	2.12	3.82	54	45	2.55
5.0	2.59	1.64	2.81	47	34	1.87
10.0	2.18	1.29	2.12	39	27	1.41

*Based on ground motion parameters 0.667 g, 21 in/sec and 18 in.



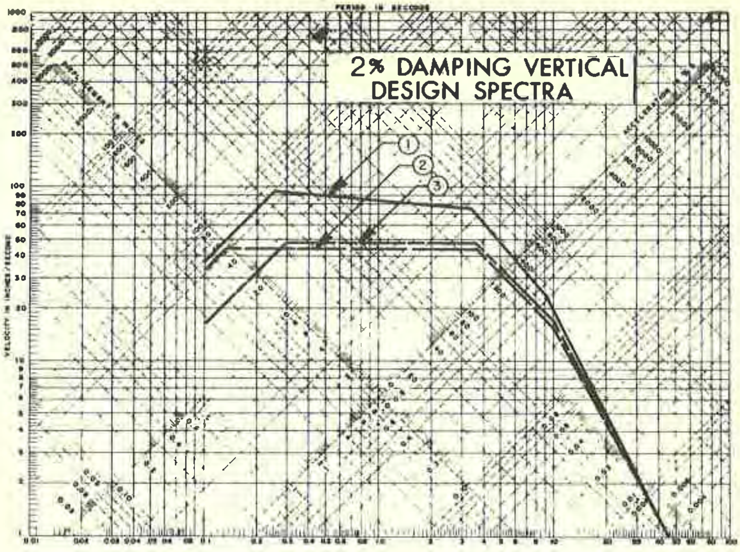
① USAEC REGULATORY GUIDE 1.60 VERTICAL SPECTRUM
② VERTICAL DESIGN RESPONSE SPECTRUM FOR ROCK SITES FROM THIS RESEARCH STUDY
③ NEWMARK VERTICAL RESPONSE SPECTRUM

FIGURE 1



① USAEC REGULATORY GUIDE 1.60 VERTICAL SPECTRUM
② VERTICAL DESIGN RESPONSE SPECTRUM FOR ROCK SITES FROM THIS RESEARCH STUDY
③ NEWMARK VERTICAL RESPONSE SPECTRUM

FIGURE 2



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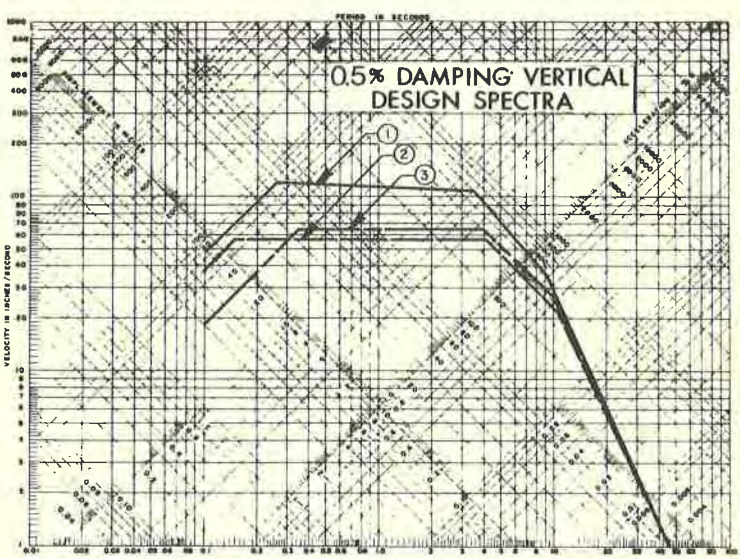
② VERTICAL DESIGN RESPONSE SPECTRUM FOR ROCK SITES FROM THIS RESEARCH STUDY
③ NEWMARK VERTICAL RESPONSE SPECTRUM

FIGURE 3



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② VERTICAL DESIGN RESPONSE SPECTRUM FOR ROCK SITES FROM THIS RESEARCH STUDY
③ NEWMARK VERTICAL RESPONSE SPECTRUM

FIGURE 4