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JUN 3 0 1976

Docket Nos. 50-275 and 50-323

Applicant: Pacific Gas and Electric Company (PG&E)

Facility: Diablo Canyon Nuclear Power Station, Units 1 and 2 (Diablo Canyon)

SUMMARY OF MEETING HELD ON June 10, 1976 TO DISCUSS SEISMIC DESIGN REEVALUATION OF DIABLO CANYON

We met with the applicant on June 10, 1976 to discuss the seismic design reevaluation of Diablo Canyon. A list of attendees is provided in Enclosure No. 1.

We had previously adopted the assessment of the U.S. Geological Survey (USGS) that a magnitude 7.5 earthquake could occur in the future at any point on the Hosgri fault. We had also adopted the recommendation of Dr. Newmark that a horizontal ground acceleration of 0.75 g be used in the development of design response spectra for use in evaluating the plant's capability to withstand such an earthquake, including an adjustment to the spectra according to methods published by Scanlan and Yamahara. We had also adopted a ductility ratio of 1.2 to be used in the reevaluation. The details leading to the exact ground response spectra to be used had not yet been worked out.

Dr. Newmark provided a draft report to the staff which included a discussion of the bases for the horizontal ground acceleration, a discussion of the adjustments to the spectra and a tentative design response spectrum. A copy of the draft report is provided as Enclosure No. 2. We discussed this draft report with Dr. Newmark and PG&E.

PG&E's consultant, Dr. J. Blume, had been working on developing design response spectra. Although he was not yet ready to provide a specific recommendation on the design response spectra, he described the calculations and other work he had done which would provide the bases for his recommendation.

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JUN 3 0 1976

A number of topics concerning the design response spectrum were discussed. They include the following:

(1) The tentative spectrum in Enclosure No. 2 would be raised to about 0.6g at the high frequency asymptote if the actual dimension of the containment base slab were used in the calculation instead of the equivalent dimension.

(2) We discussed the possibility of using a value of the mean plus one half of a standard deviation (sigma) instead of the usual mean plus one sigma in Dr. Blume's statistical calculations concerning design response spectra.

(3) We discussed the concept of making an appropriate reduction in the design response spectra to take credit for the ductility factor of 1.2 at the beginning of the evaluation process.

(4) We discussed the idea that, with regard to the ductility factors, calculation of a floor response spectrum assuming ductile behavior could yield low values for the purpose of system design and equipment qualification since the structure may not actually yield. In the case of ductile components such as piping systems this may not matter since the components' responses could then take credit for the ductility. For other components the same rationale might not hold true.

(5) We discussed the thought that an article by Hwang and Trifunac in 1974 may be equivalent to the approaches published separately by Scanlan, Yamahara, and Ambraseys when the authors' different approaches or assumptions regarding stiffness, mass, and other parameters are considered.

We did not reach definite conclusions concerning the points discussed above. Dr. Newmark and the staff agreed to transmit Dr. Newmark's draft report to the ACRS to form a basis for discussion at the ACRS subcommittee meeting on June 25 and June 26, 1976. Before transmitting the report some modifications would be made by Dr. Newmark. The report would remain in draft form. In particular, the exact form of the design response spectra would be considered tentative pending further discussion and a review of PG&E's proposals (which had not yet been received).

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JUN 30 1976

PG&E intended to submit specific proposals concerning the design response spectra as soon as possible. These would be based on Dr. Blume's work and recommendations. It was hoped that this information could be submitted prior to the ACRS subcommittee meeting on June 25 and June 26, 1976.

Original Signed By
Dennis P. Allison

D. P. Allison, Project Manager
Light Water Reactors Branch No. 1
Division of Project Management

Enclosures:
As stated

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ENCLOSURE NO. 1

LIST OF ATTENDEES

PG&E

R. V. Bettinger
W. J. Lindblad
V. J. Ghio
J. B. Hoch

PG&E Consultants (John A. Blume & Associates)

J. Blume
R. Gallagher

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UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

JUN 3 0 1976

Docket Nos. 50-275 and 50-323

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Facility: Diablo Canyon Nuclear Power Station, Units 1 and 2 (Diablo Canyon)

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DP Allison

D. P. Allison, Project Manager
Light Water Reactors Branch No. 1
Division of Project Management

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ENCLOSURE NO. 2

Preliminary Draft

DESIGN SPECTRA FOR DIABLO CANYON REACTOR FACILITY

by

Nathan M. Newmark

A Report to the U.S. Nuclear Regulatory Commission

Nathan M. Newmark Consulting Engineering Services
1211 Civil Engineering Building
Urbana, Illinois 61801

8 June 1976

DESIGN SPECTRA FOR DIABLO CANYON REACTOR FACILITY

by

Nathan M. Newmark

I. INTRODUCTION AND SUMMARY

This report summarizes recommendations for the design spectra to be considered in the possible re-design and retrofit of Diablo Canyon Unit No. 1 Nuclear Reactor Facility, taking into account the earthquake motions attributable to a possible earthquake on the recently discovered Hosgri fault offshore from the plant. The recommendations are consistent with the statement by the U.S. Geological Survey that an earthquake with a magnitude of about 7.5 could occur in the future anywhere along the Hosgri fault, and the near field ground motions attributable to such an earthquake should be considered in addition to other earthquakes previously considered in the design of the plant.

In the assessment of the potential motions and design criteria for such an earthquake, the closeness to the site, the site conditions, and the general nature of response to near field motions were taken into account. The design spectrum is drawn for a value of "effective" ground acceleration of 0.75 g, although it is recognized that occasional peaks of higher acceleration might be experienced. In addition, consideration is given to the maximum ground velocities and displacements consistent with the site geology, and consideration is also given to the attenuation of high frequency motion input in the major parts of the facility caused by the large size and close spacing of these parts of the facility.

. The recommended design spectrum exceeds in certain ranges of frequencies the original design spectrum used for the plant. However, many of the items of structure and equipment were designed with sufficient margin that the recommended design spectra does not generally exceed the original design spectrum except in some ranges where further studies are needed to review the resistance provided.

II. DESIGN INTENSITY OF SITE MOTIONS

Relations were given by Donovan (Ref. 1) for the attenuation of maximum ground acceleration as a function of magnitude and hyperfocal distance from the source. With this relationship, involving an exponent for decay of acceleration with distance of -1.32 and a geometric standard deviation of 2.0, the maximum ground acceleration for 1 standard deviation from the median is approximately 0.75 g, for a horizontal distance of 7 km and a focal depth of 12 km from the earthquake source. This value is not inconsistent with the values in USGS Circular 672 (Ref. 2) for near field strong motions, considering a repeated acceleration peak of several times, rather than one isolated peak.

Although, for more distant sources, response spectrum calculations indicate that the peak acceleration value is a reasonable basis from which to draw the design spectrum, for near field earthquakes this does not appear to be the case, judging from the spectra for the several near field earthquakes for which records are available, and from the lack of damage consistent with the near field peak measurements in those near field earthquakes, such as the Pacoima Dam record, the Parkfield record, the Ancona records, and the Melendy Ranch record.

The foundation conditions at the Diablo Canyon site are very good. The material on which the major facilities are founded is a competent rock, with somewhat less competent material near the surface. However, the depth of the less competent material is quite limited. The seismic shear wave velocity of the more competent material underying the plant foundation structure is slightly higher than 5000 ft/sec at low stress levels. One would expect that the velocity for higher stress levels, accompanying a major earthquake, might be considerably reduced, of the order of 4000 ft/sec.

In making estimates of the response or design spectra, one must make estimates also of the maximum ground velocity and maximum ground displacement. Although values have been given by Seed for maximum ground velocity in rock corresponding to something of the order of 24 to 26 in/sec for a 1 g maximum acceleration (Ref. 3), it is believed that a somewhat higher velocity is more appropriate to use. However, it does appear that the velocity might be less in rock than in alluvium, where one expects a value of the order of 48 to 50 in/sec (Ref. 4). Values are also given by Mohraz (Ref. 5), of the same order of magnitude given by Seed in Ref. 3. For the purpose of this study, a value of 32 in/sec for 1 g maximum ground acceleration is used. This is believed to be conservative. Consequently, for 0.75 g the maximum ground velocity is considered to be 24 in/sec.

In making an estimate of maximum ground displacement in vibratory motion, a value of the product of acceleration times displacement divided by the square of velocity is used as a basis. This parameter has a mean value of about 6 for a large number of earthquakes (Ref. 4). However, for close-in earthquakes the value appears to be somewhat less, and for this study the value is taken as 4. With this value, the maximum ground

displacement is computed as approximately 8 in. These values are summarized in Table 1.

III. RESPONSE TO NEAR EARTHQUAKES

Several earthquake records have been obtained close to the source. These include the Parkfield earthquake of 27 June 1966, for which the maximum recorded acceleration is 0.5 g; the Melendy Ranch earthquake of 4 September 1972 with a maximum acceleration of 0.7 g; the Ancona earthquakes of June 1972, for which the record at Rocca (on rock) had a maximum acceleration of about 0.6 g and at Palombina (on sediment) where a maximum acceleration of 0.4 g was experienced; and the Pacoima Dam earthquake record of 9 February 1971 with a maximum acceleration of about 1.2 g. In all of these earthquakes the damage suffered by the buildings near the source was considerably less than would have been expected from the acceleration levels or from the response spectra corresponding to the near field records. This is in contrast to the fact that for more distant earthquakes, at distances over about 40 km, the damage levels appear to be consistent with response spectra when inelastic behavior of the structure is taken into account.

Both Housner and Cloud (Refs. 6 and 7) refer to the small damage occurring in the Parkfield earthquake. Lander (Ref. 8) indicates the relatively light damage in the Melendy Ranch earthquake. Observations by Italian seismologists and engineers (Ref. 9) indicate the relatively small damage in the Ancona earthquakes, and the fact that buildings designed with a seismic coefficient of 0.07 g, in accordance with the then recently adopted Italian earthquake code, suffered no damage. Near Pacoima Dam, the caretaker's cottage, of the order of about half a mile away, did not have its chimney damaged and suffered practically no damage otherwise.

Response spectra for these several earthquakes are given herein. Figures 1 and 2 show the Pacoima Dam response spectra, in two directions, for 2% damping. Figures 3 and 4 show the spectra for the two Ancona earthquakes for 5% critical damping. In these figures, the curve for $\tau = 0$ is the response spectrum from the actual record. In Fig. 5 there is shown the response spectrum for the Melendy Ranch barn record, for various amounts of damping. The record for the Melendy Ranch and Ancona earthquakes are surprisingly similar, with a relatively sharp spike at about 5 to 6 hertz frequency. The Pacoima Dam response spectrum has peak responses at several frequencies including the higher frequencies just cited and several lower frequencies.

In order better to understand the relationship between response spectra and actual response of a nonlinear or inelastic structure, one may observe Fig. 6. This figure is drawn for average conditions, using the procedures described in Refs. 4 and 10. The design spectrum marked "elastic" in Fig. 6 is drawn, as are the other spectra, for a peak ground acceleration of 0.5 g, with 7% damping. The spectral amplification factors used for ground acceleration, velocity, and displacement, are given in the second line of Table 1. These values are taken from Refs. 4, 10, or 11. The response spectrum bounds are approximately 1.2 g for amplified acceleration, 50 in/sec for amplified velocity, and about 33 in for displacement response.

Modifications of the elastic response spectrum are made in accordance with procedures described in Refs. 11, 12 and 13, and are shown in Fig. 6 for two values of ductility factor. The value corresponding to "loss of function" is drawn for a ductility factor of 2.5, and that for "collapse" for a ductility factor of 10. It is noted that these are overall

ductility factors, and the local factors in structural members might be somewhat higher. However, these would correspond also to the ductility factors in items supported on floors or walls or on the ground foundation structure.

All of these are drawn for a peak ground acceleration of 0.5 g. For larger values of ground acceleration, the required values would be higher, in proportion to the "effective" ground acceleration value. The latter is defined as that value which corresponds to the acceleration level which is used as a basis for drawing the spectrum.

These various levels can be compared in terms of the seismic coefficient in the frequency range corresponding to the amplified acceleration level, since the spectra are generally proportional to these values in the range of important frequencies for structural or equipment design in nuclear reactor facilities, although the values are more nearly proportional to the ductility factor levels or the amplified velocity portion of the diagram for longer period or lower frequency structures.

The significance of these diagrams may be considered as follows: Low buildings, school buildings, and other structures of one or two stories, would have been designed in the past for a seismic coefficient of 0.1 g. This, at amplified working stresses, corresponds to a strength of about 0.15 g. It can be seen that a structure designed in this way would lie below the collapse level in general, and would fail in an earthquake having a maximum ground acceleration of 0.5 g. However, it could survive a maximum ground acceleration of 0.28 g or less, in general. A structure designed in accordance with the recent modification of the SEAOC Code would have 50% greater resisting capacity, and could survive an earthquake with about 0.42 g

maximum ground acceleration without collapse. Damage would occur at lower levels of maximum ground acceleration, but not collapse.

A hospital designed in accordance with the latest hospital design code might have a seismic coefficient of 0.25 g, which corresponds to about 0.38 g at yield levels. This would certainly lose function in a 0.5 g maximum ground acceleration earthquake, and probably would not be able to continue to function in earthquakes stronger than about 0.32 maximum ground acceleration (the El Centro earthquake, for example).

A further estimate of the significance of the design requirements is indicated by Fig. 7, which gives a comparison of the latest recommended earthquake design specifications in the ATC design recommendations, in comparison with those developed for the Nuclear Regulatory Commission. This figure compares the ATC design spectrum for a spectral reduction factor of 1, corresponding to elastic behavior, for the maximum effective peak ground acceleration value of 0.4 considered in the ATC code. This is compared with the response spectrum or the design spectrum for elastic behavior corresponding to the methods in Refs. 4 and 11, marked NRC-NMN in the figure. It is seen that these are very similar and closely related. However, the design seismic coefficients used in that code generally carry, for well-designed structures, values of spectral reduction factors of the order of 5. This is shown by the lower curve, where there is essentially a ratio of a factor of 5 corresponding to the design level, with a maximum seismic coefficient of 0.2 g. This cannot be directly compared with Fig. 6 unless one adjusts Fig. 6 to correspond to an earthquake of 0.4 g rather than 0.5 g peak acceleration. It will be seen, when this is done, that collapse will generally be avoided by the ATC design code for ordinary structures, unless the earthquake does exceed a level of the order of 0.4

to 0.5 g effective ground acceleration, or possibly somewhat higher than this value.

The importance of this discussion lies in the fact that an effective peak ground acceleration of 1 g would cause loss of function and collapse of practically all structures of any sort in an area, even those designed in accordance with the best current codes. This has never been observed. The only structures that have failed have been those that have been either grossly deficient in design or designed to levels considerably below those which are appropriate for the region, Hence it is felt that a value of 0.75 g for the construction of the design spectrum for the Diablo Canyon site is a value consistent with experience and observation, and designs need not be made for a response spectrum anchored to the maximum peak ground acceleration that might be recorded on an instrument for near field earthquakes.

IV. EFFECT OF SIZE OF FOUNDATION ON DESIGN SPECTRUM

The observation has frequency been made that structures on large foundations appear to respond with less intensity to earthquakes than do smaller structures, and more specifically, than does free-field instrumentation. The first paper that attempted to give a rational explanation for this behavior was apparently that by Yamahara in 1970 (Ref. 14). The same procedure appears to have been independently rediscovered by Ambraseys (Ref. 14) and by Scanlon (Ref. 16). These references give in general a relationship between the average acceleration over the width of the foundation as a function of the relative wave length of the acceleration pulse to which the foundation is subjected, compared with the width of the

foundation. Perhaps a better measure of the reduction in effectiveness of an earthquake on a large building is given by use of the average acceleration taken from the record itself. A number of examples of this kind of calculation are given herein. This has the virtue of not requiring an assessment of the particular frequencies of acceleration included in the earthquake motion, but rests entirely on the basis of a time average over a passage time of the acceleration record, and then a calculation of the response spectrum from that averaged acceleration record.

There are only a limited number of examples of responses measured in a building foundation and in the free field near the building. The most complete and useful records are those obtained in two earthquakes for the Hollywood Storage Building and the Hollywood Parking Lot. The building itself is shown in elevation and in plan in Fig. 8. The free-field acceleration record, in the Hollywood Parking Lot, was measured at 112 ft away from the nearest corner of the building, which is 51 ft in the north-south direction and 217.5 ft in the east-west direction. The building is 150 ft high and is supported on piles. The basement accelerograph is located in the southwest corner of the building. Figure 9 shows the subsurface model of the building, with Figs. 8 and 9 being taken from a study by Duke et al (Ref. 17).

The shear wave velocity in the upper strata near the building is approximately 2000 fps, and this can be considered as possibly the wave propagation velocity.

Response spectra have been resported for this building in both the San Fernando earthquake and in the Kern County earthquake. Typical of the results are those shown in Figs. 10 and 11, which give the response

spectrum for the storage basement and for the parking lot, in both the east and the south directions, for a damping value of 2% critical, as a function of period. It can be seen that for periods less than about 0.4 sec there is a significant decrease in the response spectrum for the building compared with that for the parking lot, whereas for longer periods the response spectra are practically identical. This shows the filtering effect, discussed above. It is of interest to note, however, that the reduction is of the order of a factor of 2 to 2.5. Similar effects are observed for 5% damping spectra as well.

On the other hand, no attenuation was observed for the Kern County earthquake in the same building, which was considerably further away, both the San Fernando earthquake source and the Kern County earthquake source being approximately north of the structure. The natural frequencies of the building, from a vibration test, are given in Table 2, taken also from Ref. 17. The fundamental period of the building in the east-west direction is 0.5 sec and in the north-south direction about 1.2 sec. This is in the range where practically no change in the response spectrum is observed. It appears that there is practically no soil-structure interaction as such under this building, but the major effect is one of smoothing out the acceleration input from the earthquake motions. Figures 12 and 13 show a series of spectra for the San Fernando earthquake for 5% damping for travel times across the width of the building in the east-west and the north-south direction of 0, 0.04, 0.08, 0.12, and 0.16 sec. The curve for a transit time of 0 sec is the spectrum for the parking lot unmodified, and the others are spectra for the parking lot record smoothed by averaging values over times corresponding to the transit time listed in the figure. The response spectrum for the

structure is shown by the dashed line in the figures, which is very nearly identical with the computed value for the parking lot for a transmit time of about 0.08 sec in the north-south direction; and for the east-west direction the agreement is almost exact for a transit time of 0.12 sec, which corresponds almost identically with a width of 217 ft divided by the seismic velocity of 2000 ft/sec. It appears that either the longest dimension of the building or the mean or geometric mean of the dimensions controls the effective transmit time insofar as the reduction in response is concerned.

Similar results are shown for the Kern County earthquake in Figs. 14 and 15, where again the transit time of 0.08 appears to be the best value. However, there is very little attenuation, which is indicative of the fact that at the very large distance of the Kern County earthquake the major influences reaching the building are surface waves with a much longer wave length than those for the closer San Fernando earthquake.

Now, referring again to Figs. 1 and 2 we may observe how the responses of the structure to the Pacoima Dam record would be affected by transmit time. There is apparently a substantial reduction as the transit time increases from 0 to 0.12 sec, but only a slight reduction beyond that to 0.16 sec. However, this reduction affects only the high frequency range, above about 2 hertz. Similarly, Figs. 3 and 4 show a large reduction for the Ancona earthquakes as a function of transmit time. The much simpler, more sharply defined input motion produces a larger reduction in effect on structures, and is consistent with the very low level of observed damage of buildings designed to resist even moderate earthquakes in the Ancona region.

V. DIABLO CANYON DESIGN SPECTRA

Referring again to Table 1, one finds spectrum bounds defined by the ground motions discussed earlier and the spectrum amplification factors given in Table 1, as shown on the last line of Table 1. These values are plotted in Fig. 16 in terms of the usual type of design spectrum considered earlier in this report. The spectrum shown in Fig. 16 is for the plant itself and not for the free field, which would correspond to a higher acceleration bound than is shown in Fig. 16, with approximately a 50% greater acceleration level.

The reduction factor for this response spectrum is based on the results in Figs. 1 and 2, where, taking into account the dimensions of the plant complex, one obtains an effective width (the square root of the area of the plant structures) of 480 ft, corresponding to a transit time of 0.12 sec, using the seismic velocity of 4000 ft/sec discussed earlier. With this value, the reduction factor of the order of 0.67, used to obtain a 0.5 g design value, is not inappropriate and is justified by the data shown in Figs. 1 and 2. Small separate structures not close to the main complex should be designed for the higher spectrum, however.

Finally, Fig. 17 shows the spectrum in Fig. 16 plotted in another way, in terms of acceleration values as a function of frequency, and compared with previously used design spectra for the plant. These previously used values are defined as the DDE or the double design earthquake spectrum originally used of 0.4 g maximum ground acceleration, and the so-called "Hosgri" spectrum which has been developed by Dr. John A. Blume for PG&E. It appears that the latter is relatively close to the recommended design spectrum developed herein for frequencies higher than about 2 or 3 hertz, but may be somewhat low for lower frequency elements.

Consistent with the concept of a wave motion of earthquake deformation, there are torsions and tiltings of a building foundation. Both effects are less on rock than on soil. The torsional effects are taken account of in current codes by assuming an eccentricity of horizontal seismic force of 5 percent of the width of the structure. This effort is less, however, for a very large structure, and the tilting effect is even smaller. Account should be taken of these effects in design.

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TABLE 1. MAXIMUM GROUND MOTIONS
AND SPECTRAL BOUNDS

	Maximum Values			
	Accel, g Small Structs.	Plant	Vel, in/sec Both	Displ, in Both
Ground	0.75	0.5	24	8
Spect. Amplif. 7% Damping	2.4	2.4	2.1	1.9
Spect. Bounds	1.8	1.2	50	15

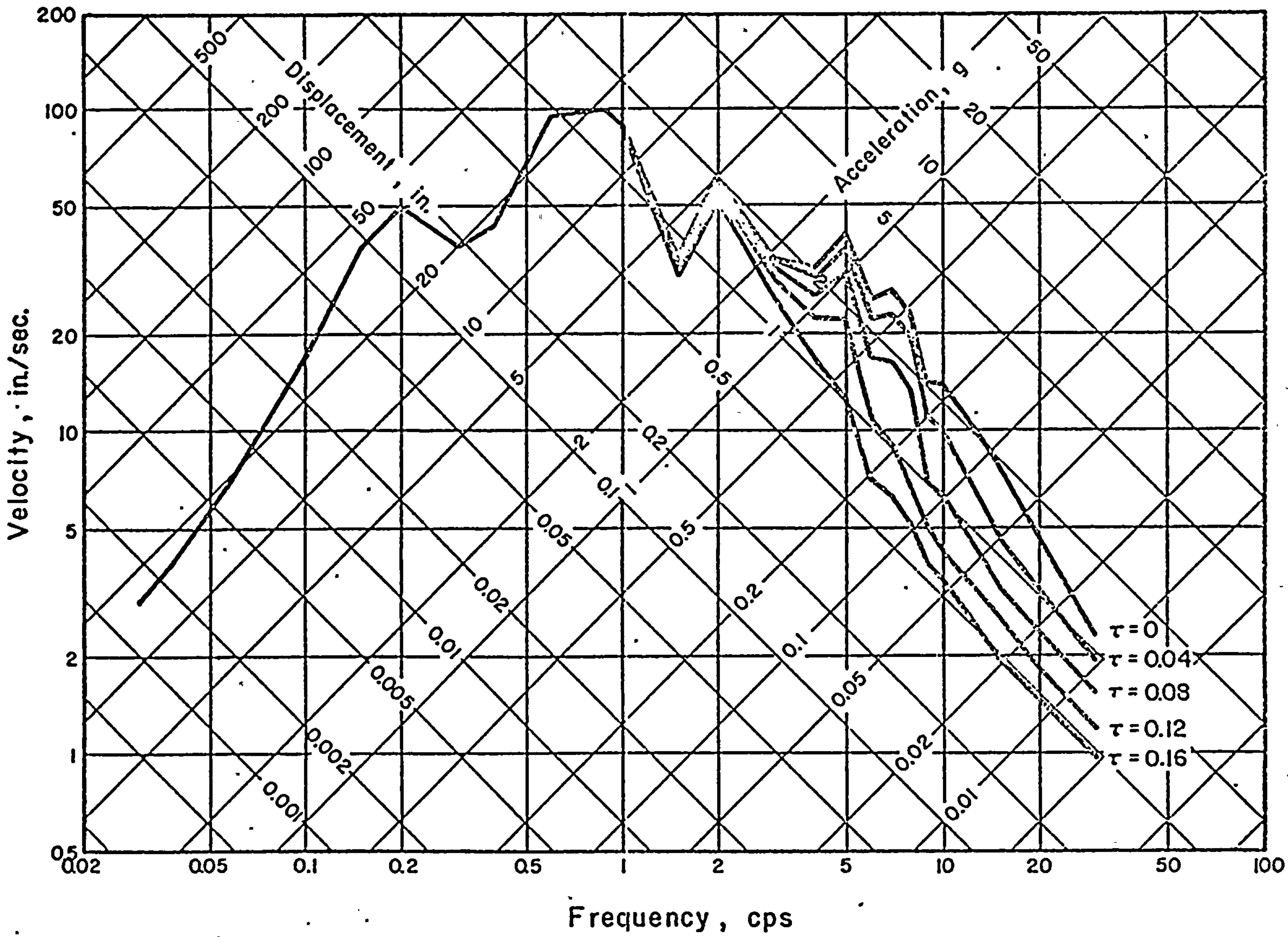

FIG. I PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S16E, 2 PERCENT DAMPING, τ = 0, 0.04, 0.08, 0.12, 0.16 sec.

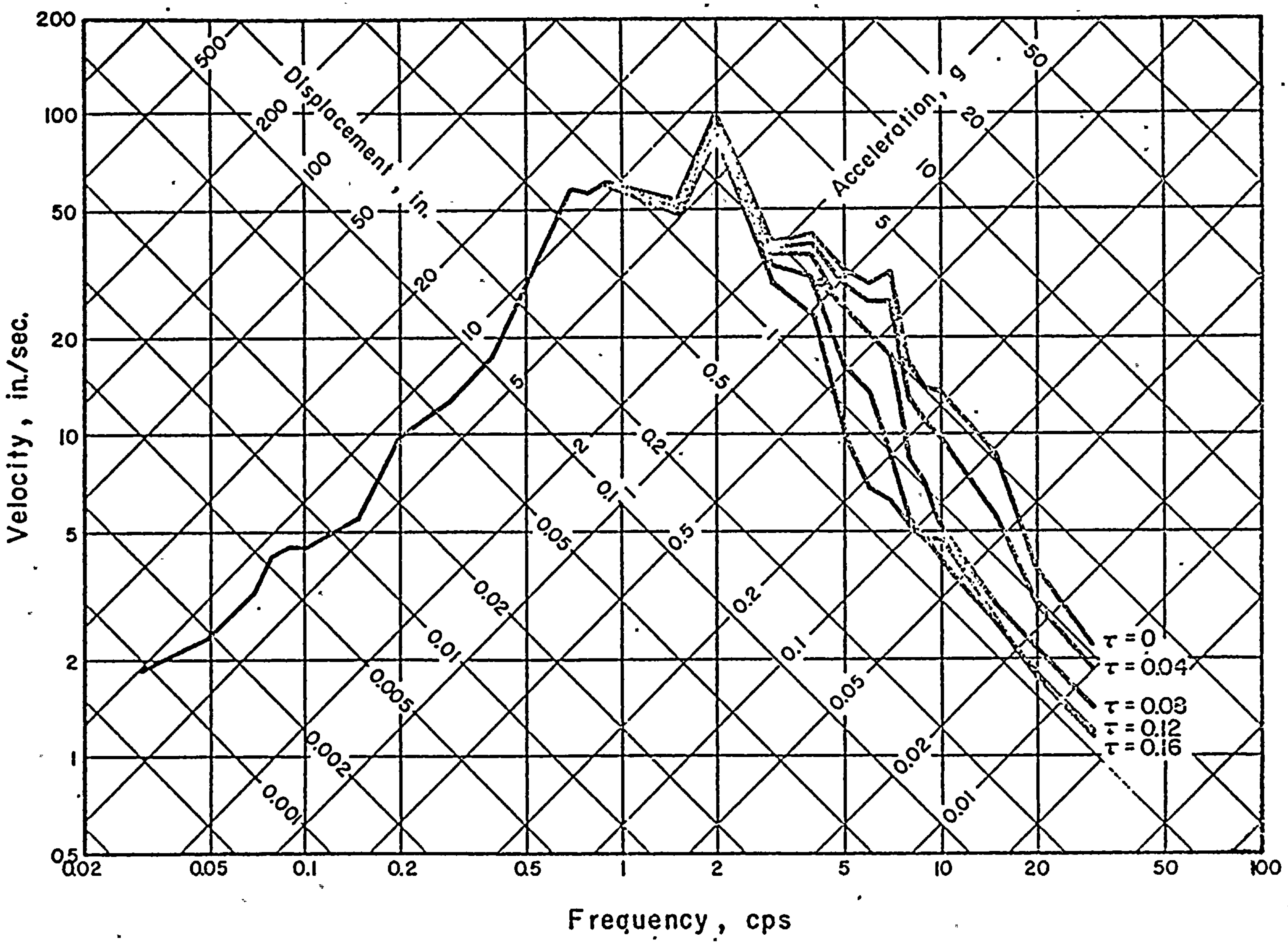

FIG. 2 PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S74W, 2 PERCENT DAMPING, τ = 0, 0.04, 0.08, 0.12, 0.16 sec.

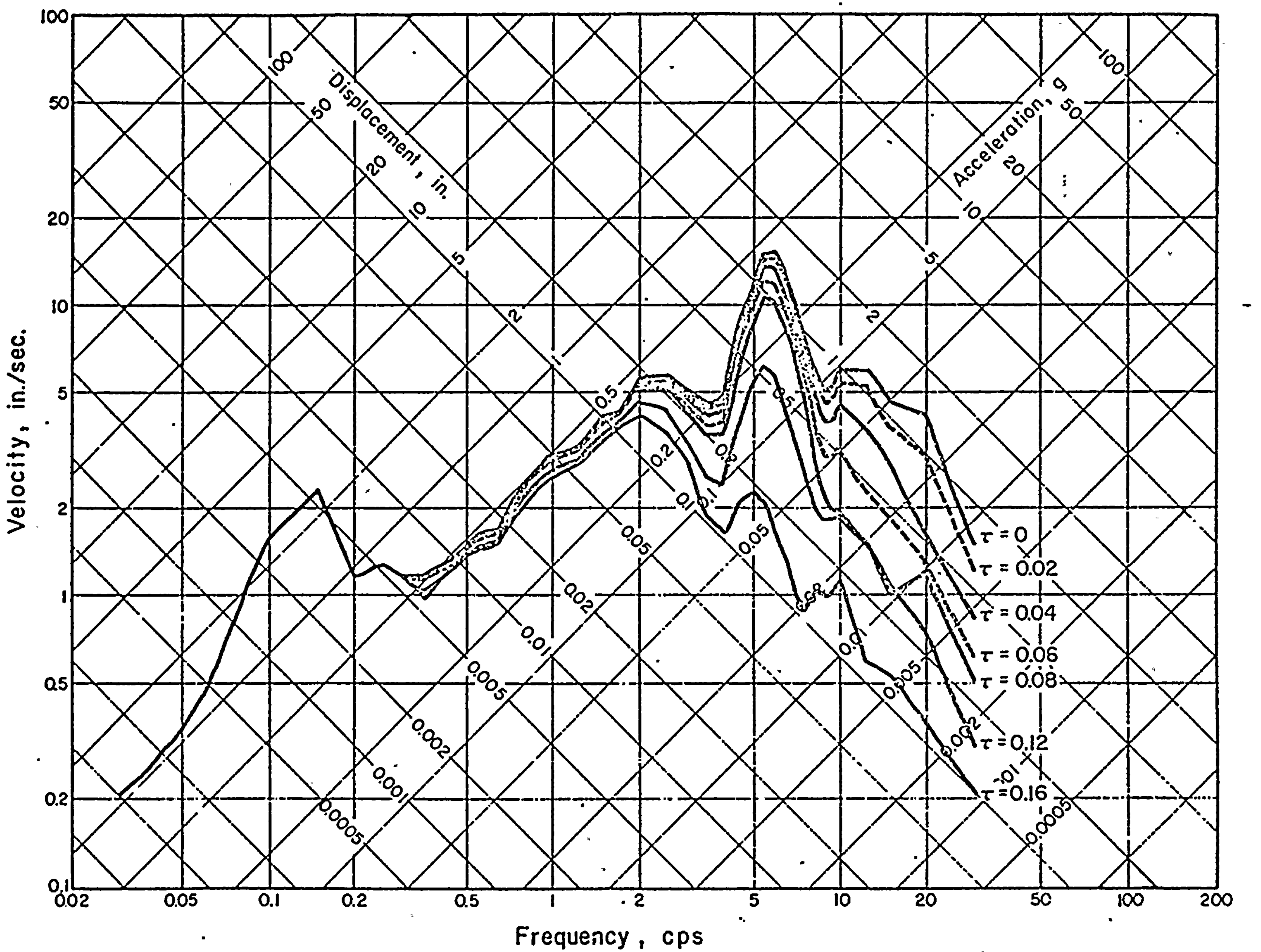

FIG. 3 ANCONA, ROCCA 6-14-72 GMT-NORTH $\tau = 0, 0.02, 0.04, 0.06, 0.08, 0.12, 0.16$
SPECTRUM COMPUTED USING 5.0 PERCENT CRITICAL DAMPING

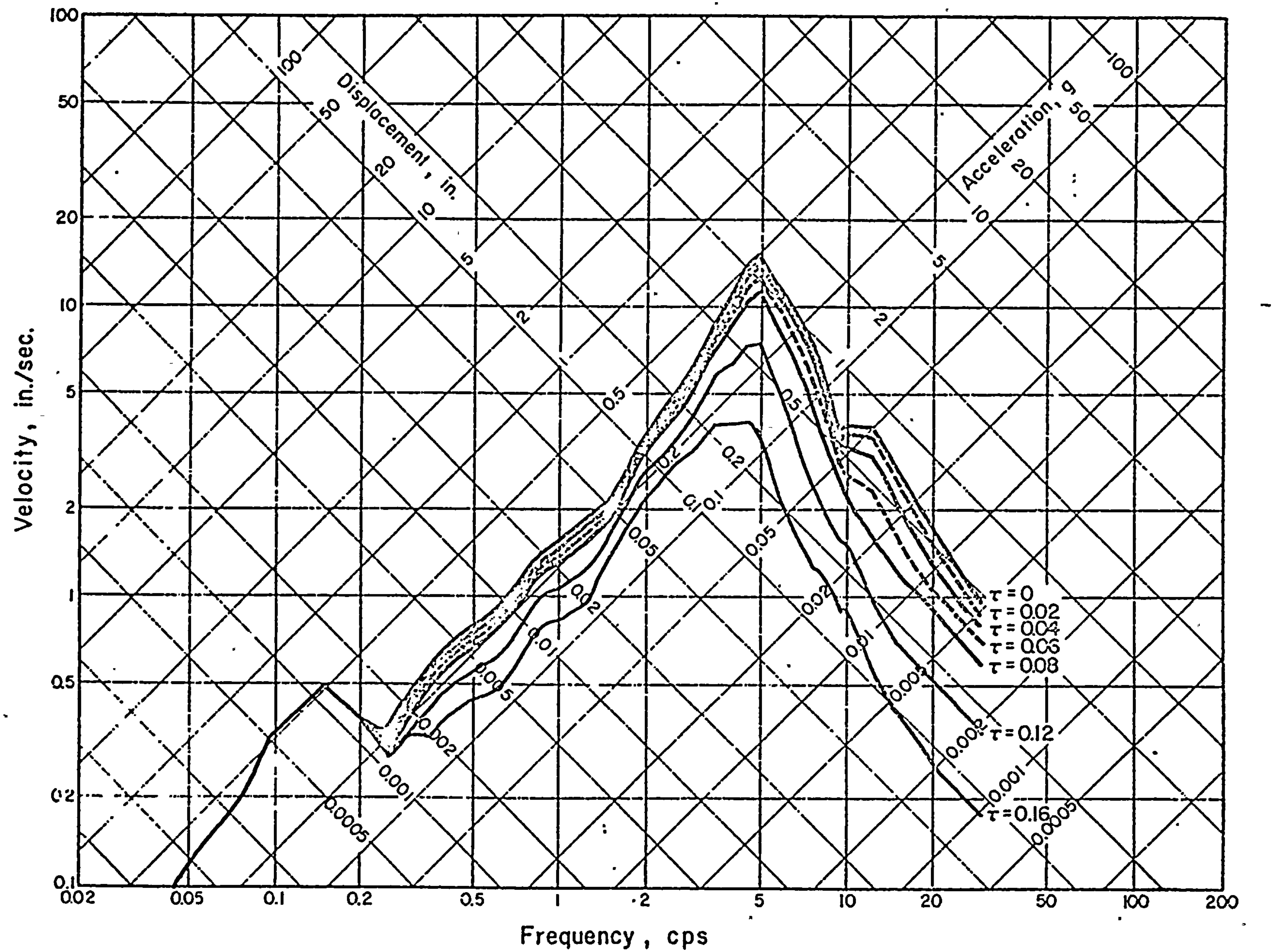

FIG.4 ANCONA, PALOMBINA 6-21-72 GMT-NS $\tau = 0, 0.02, 0.04, 0.06, 0.08, 0.12, 0.16$
SPECTRUM COMPUTED USING 5.0 PERCENT CRITICAL DAMPING

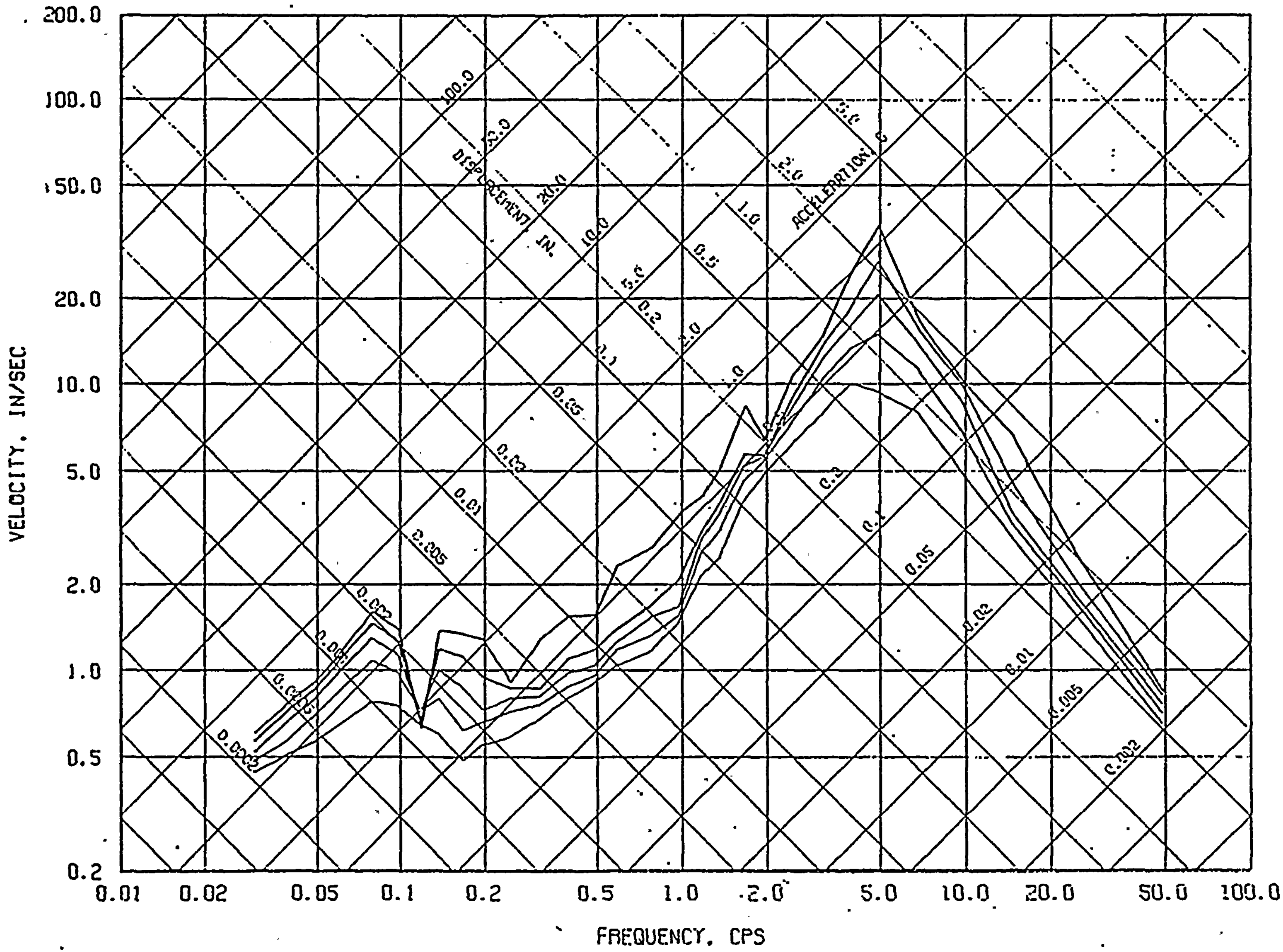

FIG. 5 RESPONSE SPECTRA FOR MELENDY RANCH BARN, 9/4/72 - N61E COMPONENT

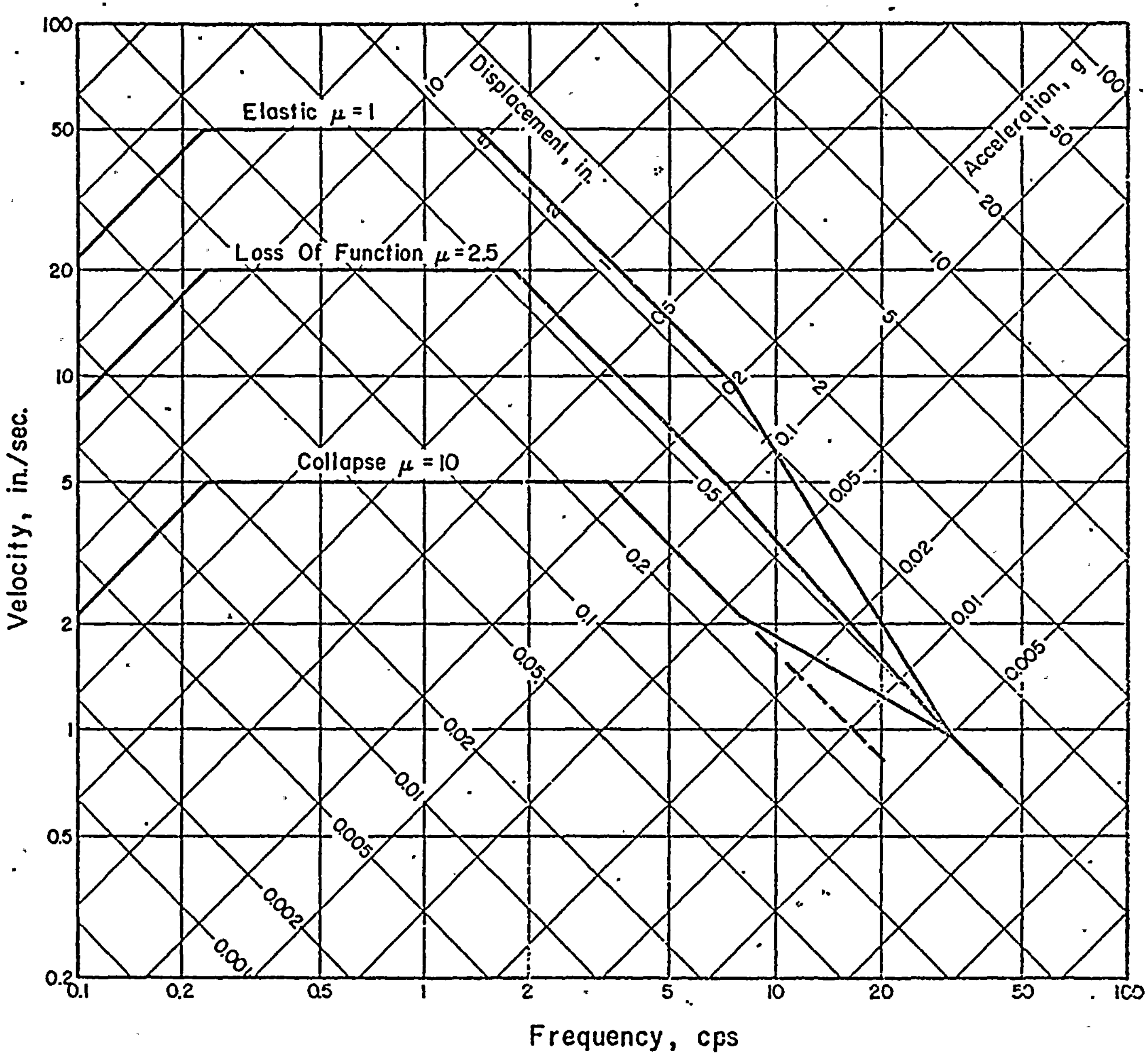

FIG. 6 INELASTIC DESIGN SPECTRAL REQUIREMENT FOR PEAK GROUND ACCELERATION OF 0.5 G, 7 % DAMPING

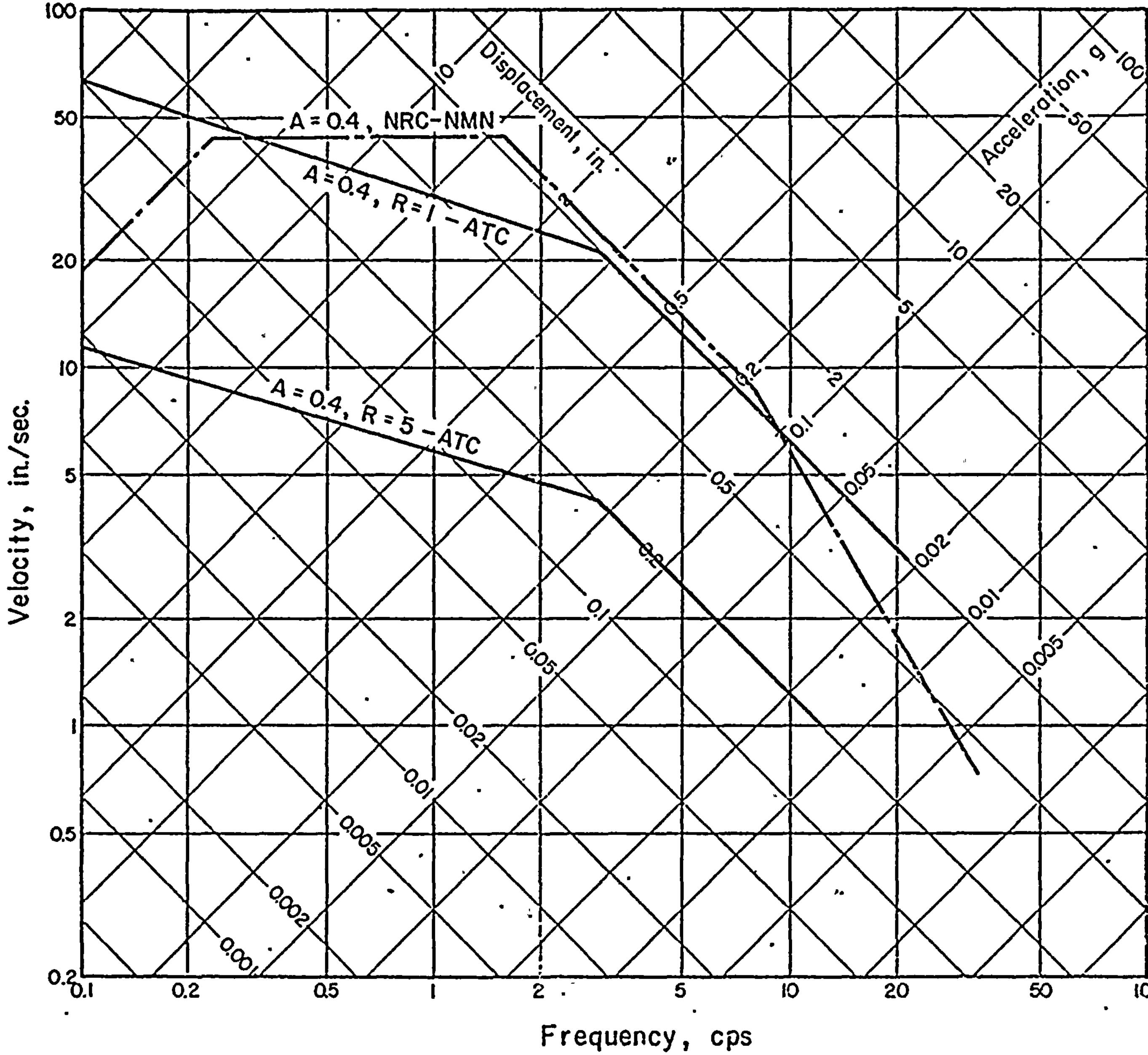

FIG. 7 COMPARISON OF ATC AND NRC DESIGN COEFFICIENTS, 5% DAMPING

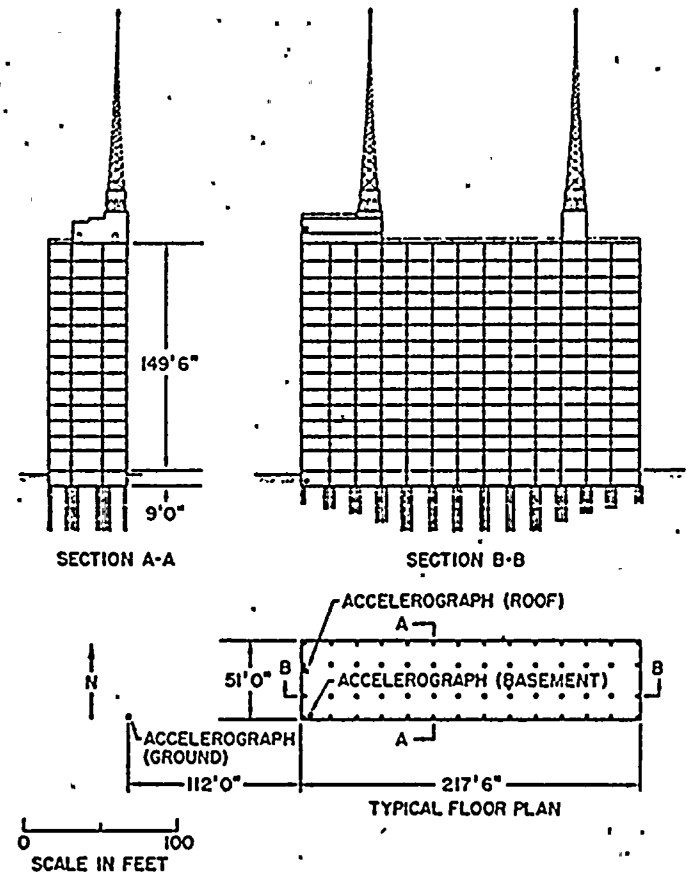

FIG. 8 PLAN AND ELEVATION OF BUILDING

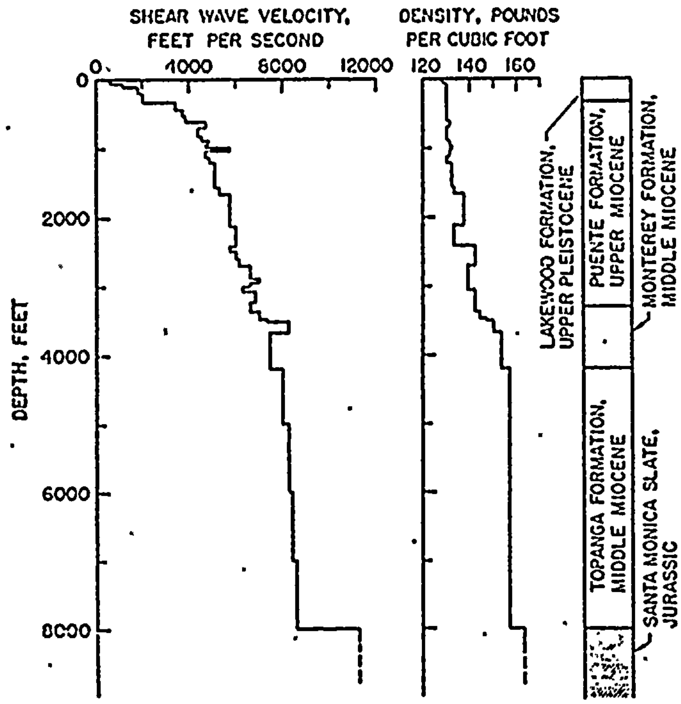

FIG. 9 Subsurface model.

TABLE 2

NATURAL FREQUENCIES OF BUILDING FROM VIBRATION TEST*

Mode of Vibration	Frequency (cps)	
	North-south	East-west
Fundamental translation	0.83	2.0
Second translational	2.7	
Third translational	4.5	
Fundamental torsional	1.57–1.67	
Second torsional	5.9	
Third torsional	9.1	
Others	1.0, 5.0	

* Source: Carder, 1964.

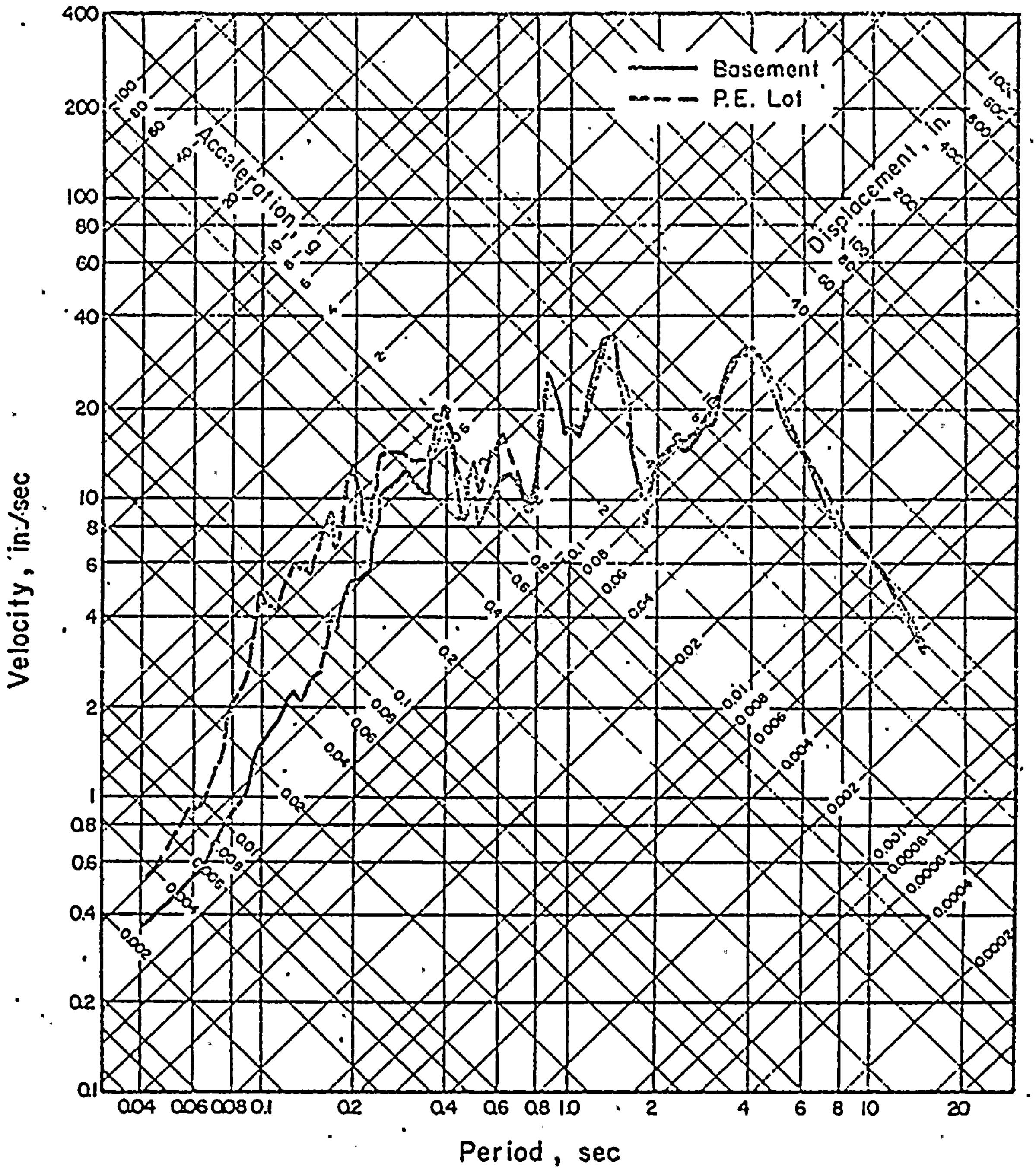

FIG. 10 SAN FERNANDO EARTHQUAKE, FEB. 9, 1971 - 0600 PST
HOLLYWOOD STORAGE BASEMENT AND P.E. LOT, COMPONENT
EAST, DAMPING VALUE 2% OF CRITICAL

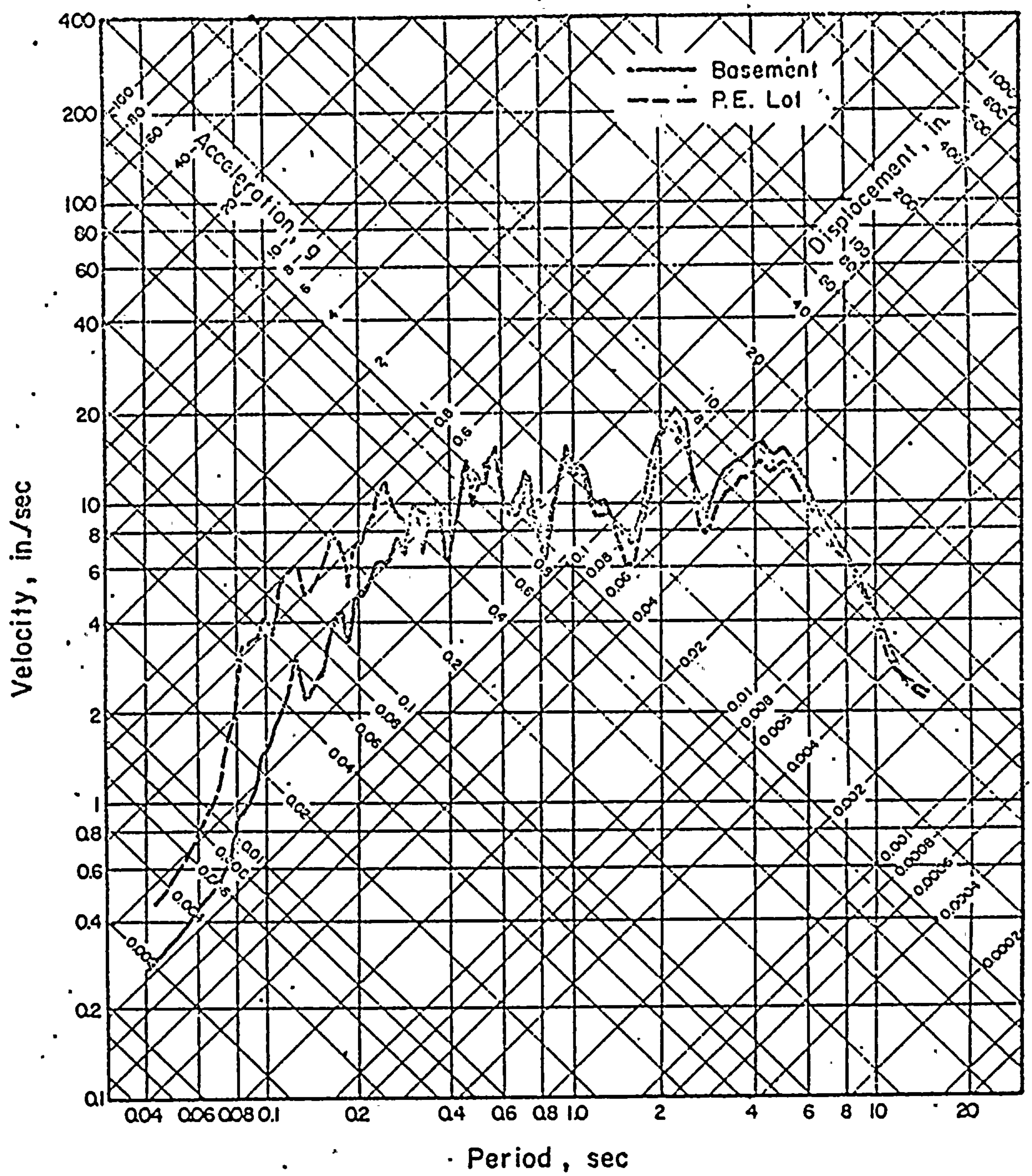

FIG.11 SAN FERNANDO EARTHQUAKE, FEB. 9, 1971 - 0600 PST HOLLYWOOD STORAGE BASEMENT AND P.E. LOT; COMPONENT SOUTH, DAMPING VALUE 2% OF CRITICAL

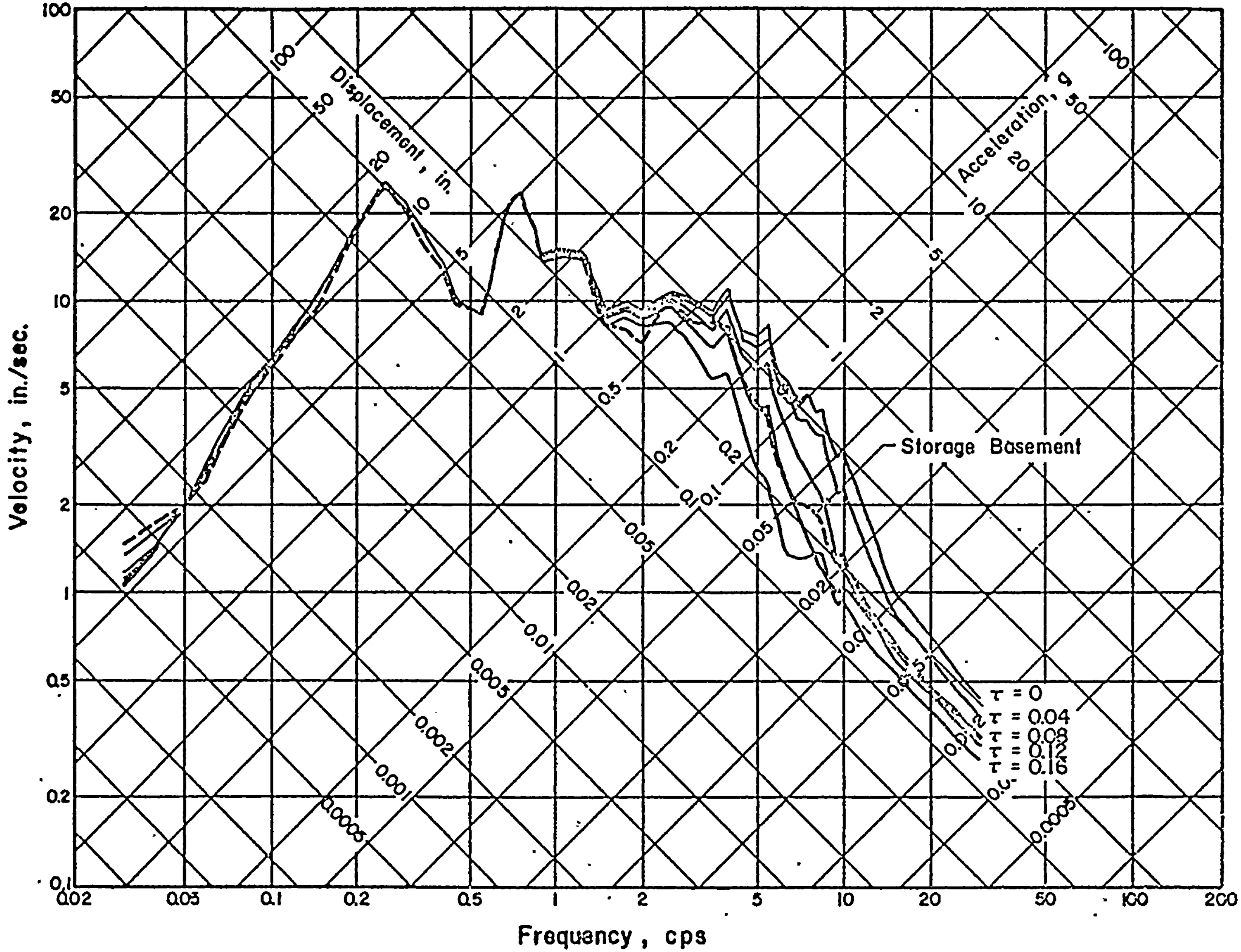

FIG. 12 HOLLYWOOD STORAGE P.E. LOT, SAN FERNANDO EARTHQUAKE FEB. 9, 1971, COMPONENT EAST, DAMPING 5 % OF CRITICAL, τ = 0, 0.04, 0.08, 0.12, AND 0.16 SEC.

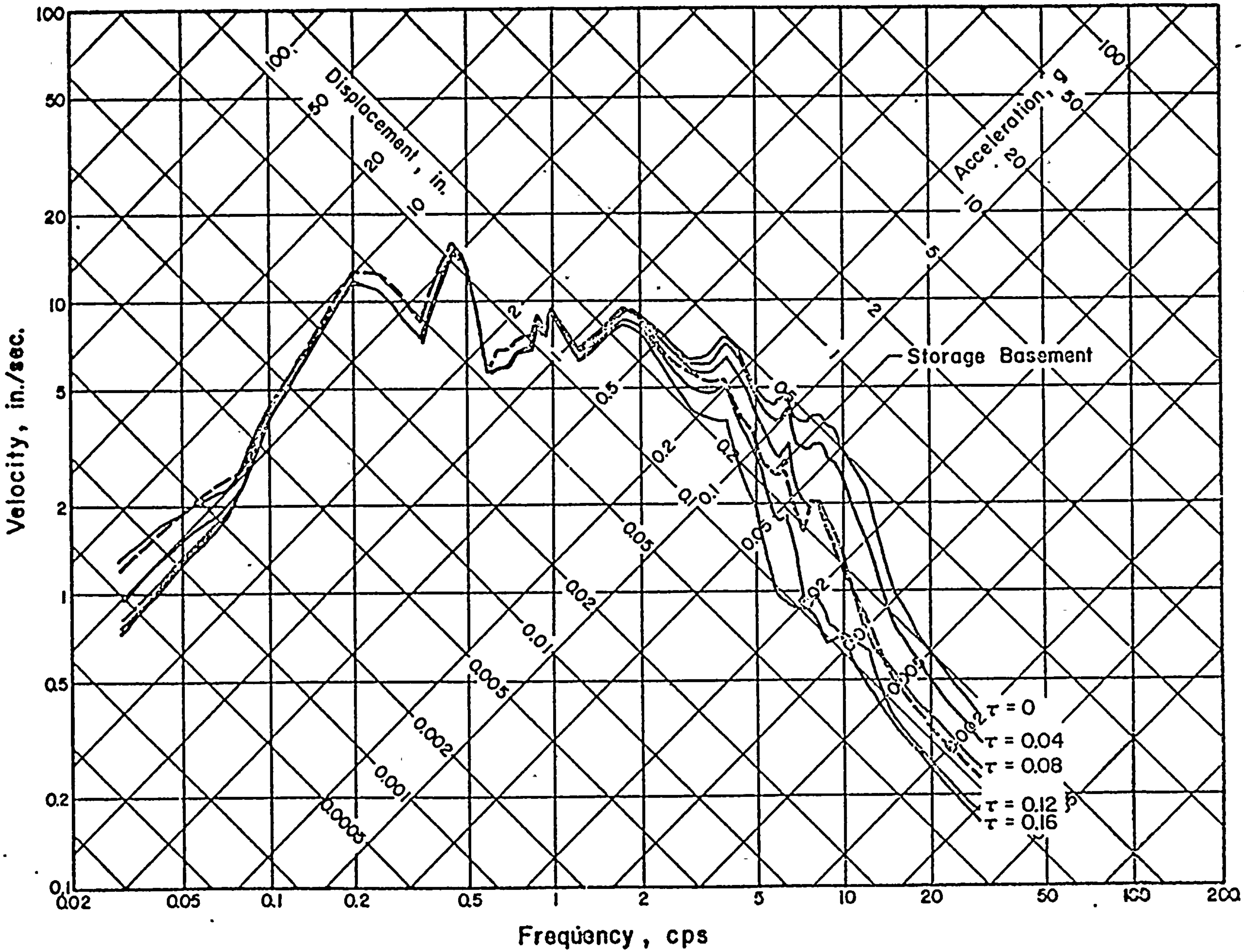

FIG. 13 HOLLYWOOD STORAGE P.E. LOT, SAN FERNANDO EARTHQUAKE FEB. 9, 1971, COMPONENT SOUTH, DAMPING 5 % OF CRITICAL, $\tau = 0, 0.04, 0.08, 0.12,$

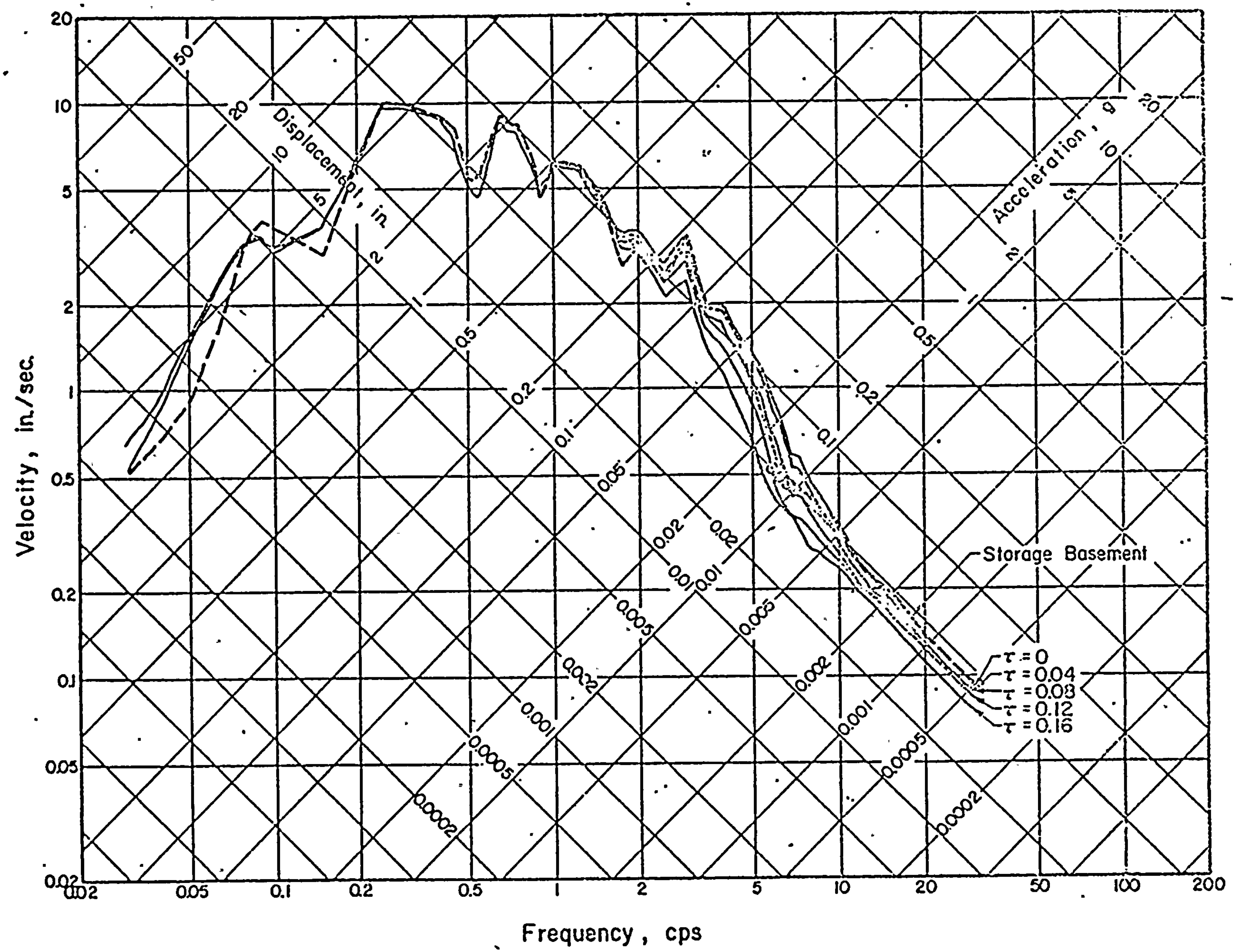

FIG. 14 HOLLYWOOD STORAGE P.E. LOT, KERN COUNTY EARTHQUAKE JULY 21, 1952, COMPONENT EAST, DAMPING 5 % OF CRITICAL, τ = 0, 0.04, 0.08, 0.12,

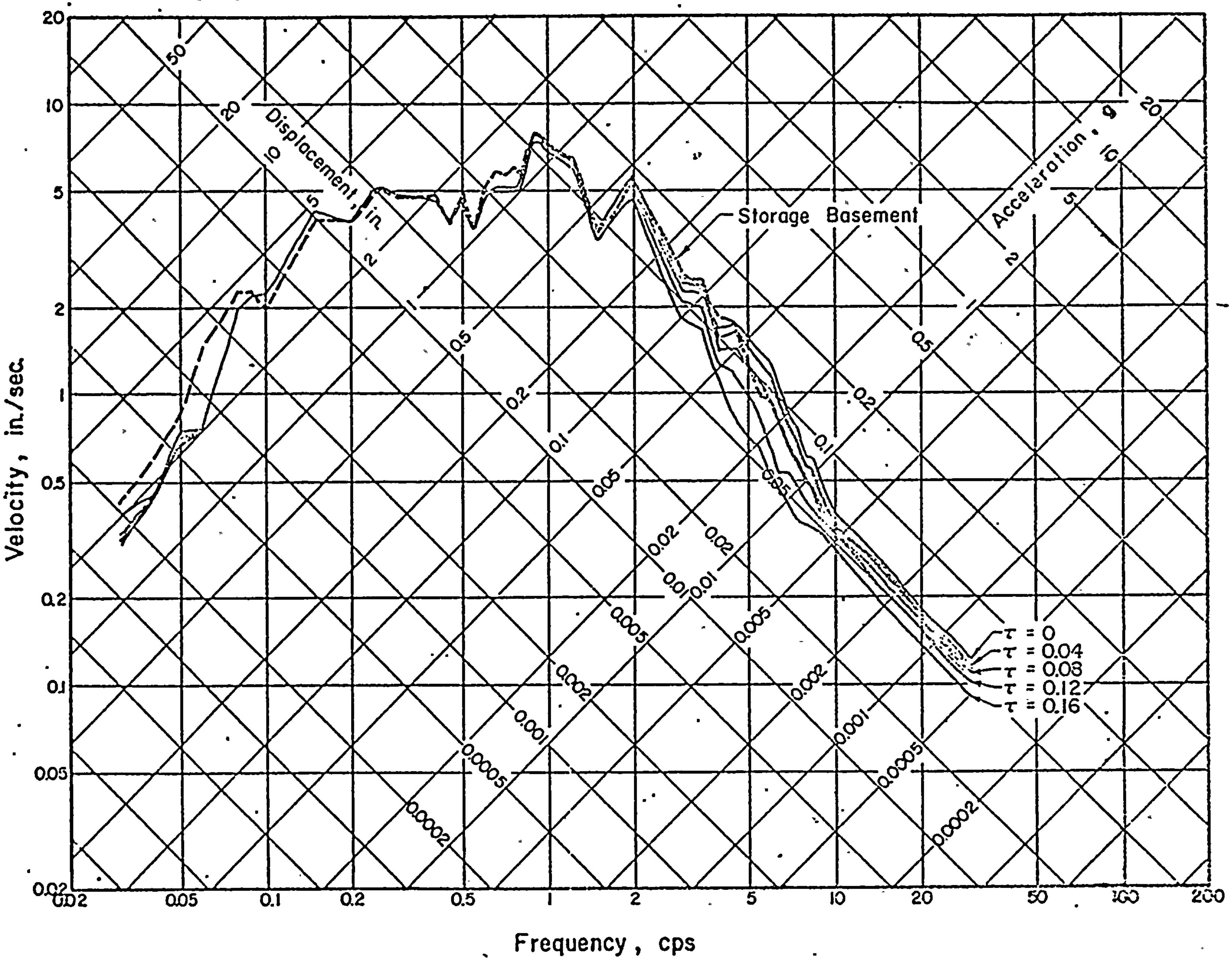

FIG. 15 HOLLYWOOD STORAGE P.E. LOT, KERN COUNTY EARTHQUAKE JULY 21, 1952, COMPONENT SOUTH, DAMPING 5 % OF CRITICAL, τ = 0, 0.04, 0.08, 0.12,

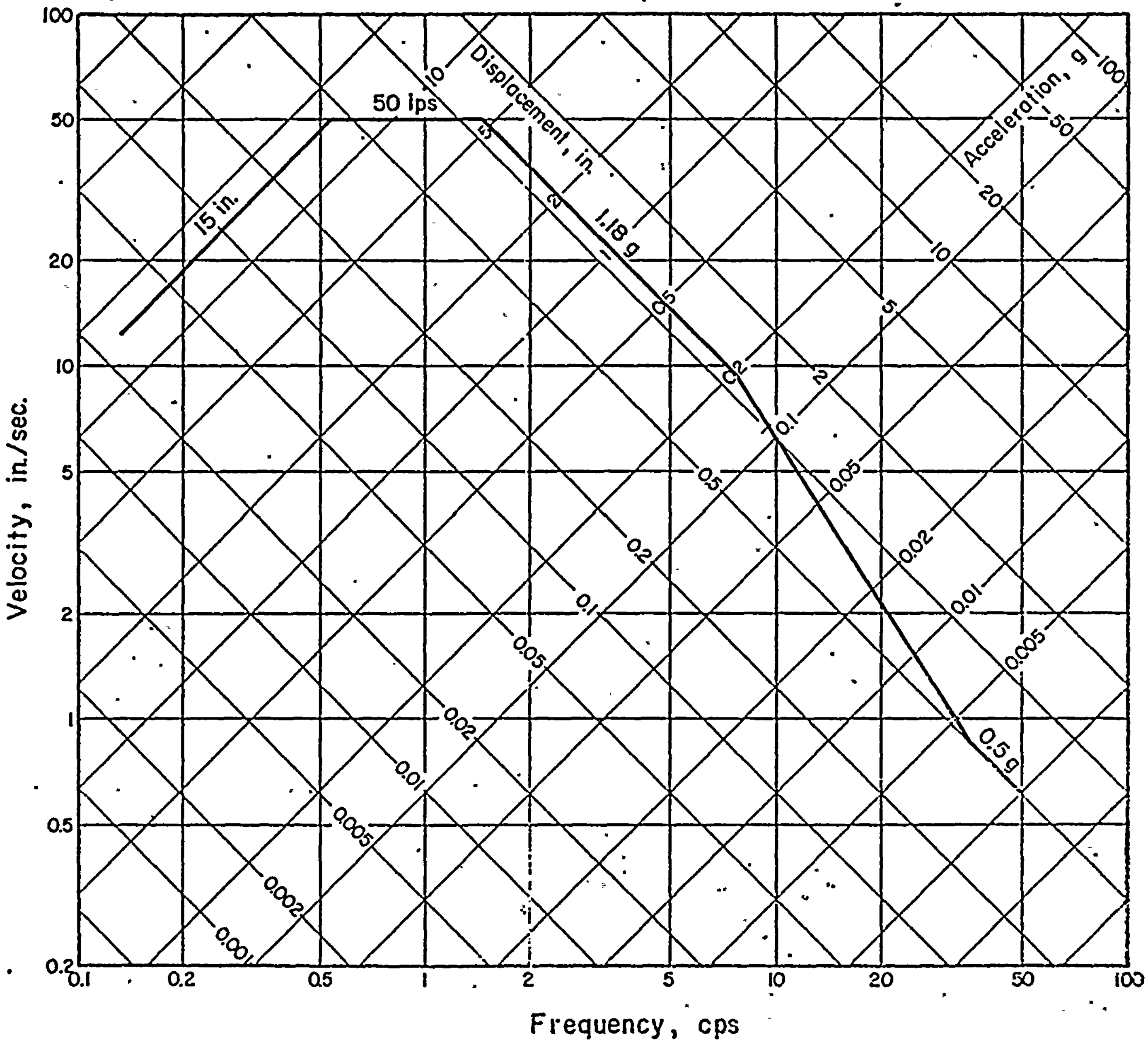

FIG. 16 RECOMMENDED "DESIGN" SPECTRUM FOR PLANT

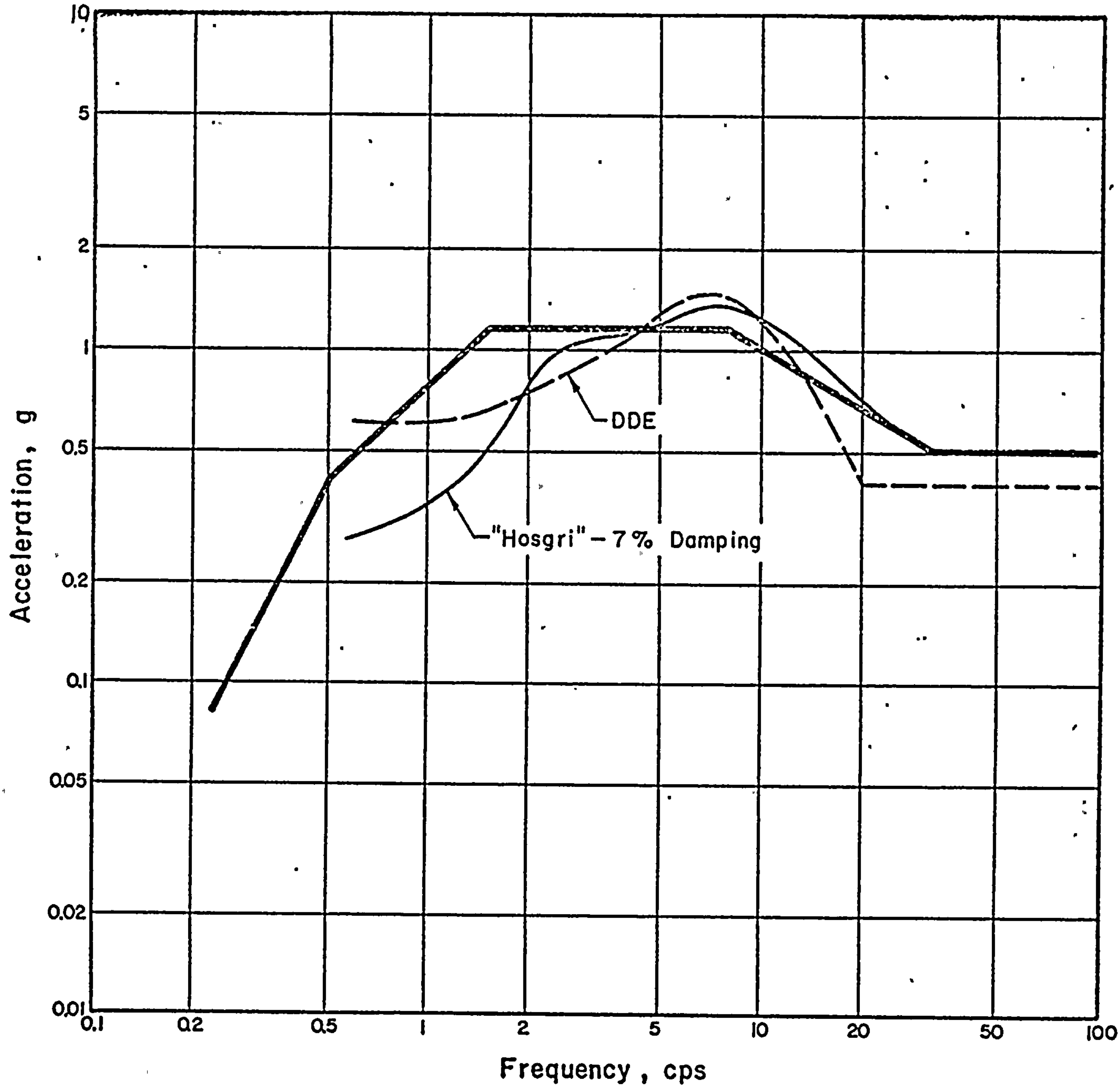

FIG. 17 RECOMMENDED "DESIGN" SPECTRA, 7% DAMPING, COMPARED WITH "HOSGRI" AND DDE SPECTRA