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Docket Nos. 50-275
and 50-323

Dr. Dade Moeller
Chairman, Advisory Committee
on Reactor Safeguards
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dear Dr. Moeller:

Sixteen copies of a draft report by Dr. Nathan M. Newmark dated June 15, 1976 and presentations prepared by Dr. K. Kapur and Mr. I. Sihweil are enclosed.

These documents are intended to form a basis for discussions concerning the seismic design reevaluation of the Diablo Canyon Nuclear Power Station at the Diablo Canyon subcommittee meeting on June 25 and June 26, 1976. We have not yet received the applicant's proposed design response spectra and the justifications for them. Accordingly, the information in Section V and Figures 16 and 17 of Dr. Newmark's draft report concerning design response spectra should be considered as tentative or typical pending completion of a review of the applicant's proposals by Dr. Newmark and the staff. Dr. Newmark's final report and the staff's conclusions will be provided in a future supplement to the Safety Evaluation Report.

Sincerely,

/S/ D. Vassallo

for R. C. DeYoung, Assistant Director
for Light Water Reactors
Division of Project Management

Enclosures:
As stated

cc: See page 2

| OFFICE | LWR 1 | LWR 1 | LWR | | | |
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| DATE | 6/ /76 | 6/ /76 | 6/ /76 | | | |

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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

15 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 do 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 underlying 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, at a distance of the order of about half a mile away, did not suffer major damage from the earthquake itself.

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 g 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 frequently 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 reported 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 transit 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 transit 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 transit 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 spectra shown in Fig. 16 are for the plant complex and for the reactor building but not for the free field, which would correspond to higher acceleration bounds than are shown in Fig. 16.

The reduction factors for these response spectra are 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, a reduction factor of 0.67 is used to obtain a 0.5 g ground acceleration design value. For the reactor building, the diameter of 160 ft. gives a transit time of 0.04 sec. and a 0.6 g design value. Small separate structures not close to the main complex should be designed for a higher spectrum, however, corresponding to the free field value of 0.75 g.

Finally, Fig. 17 shows the spectra 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 effect 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. | Reactor Bldg. | Plant Complex | Vel, in/sec Both | Displ, in Both |
| Ground | 0.75 | 0.6 | 0.5 | 24 | 8 |
| Spect. Amplif. 7% Damping | 2.4 | 2.4 | 2.4 | 2.1 | 1.9 |
| Spect. Bounds | 1.8 | 1.4 | 1.2 | 50 | 15 |

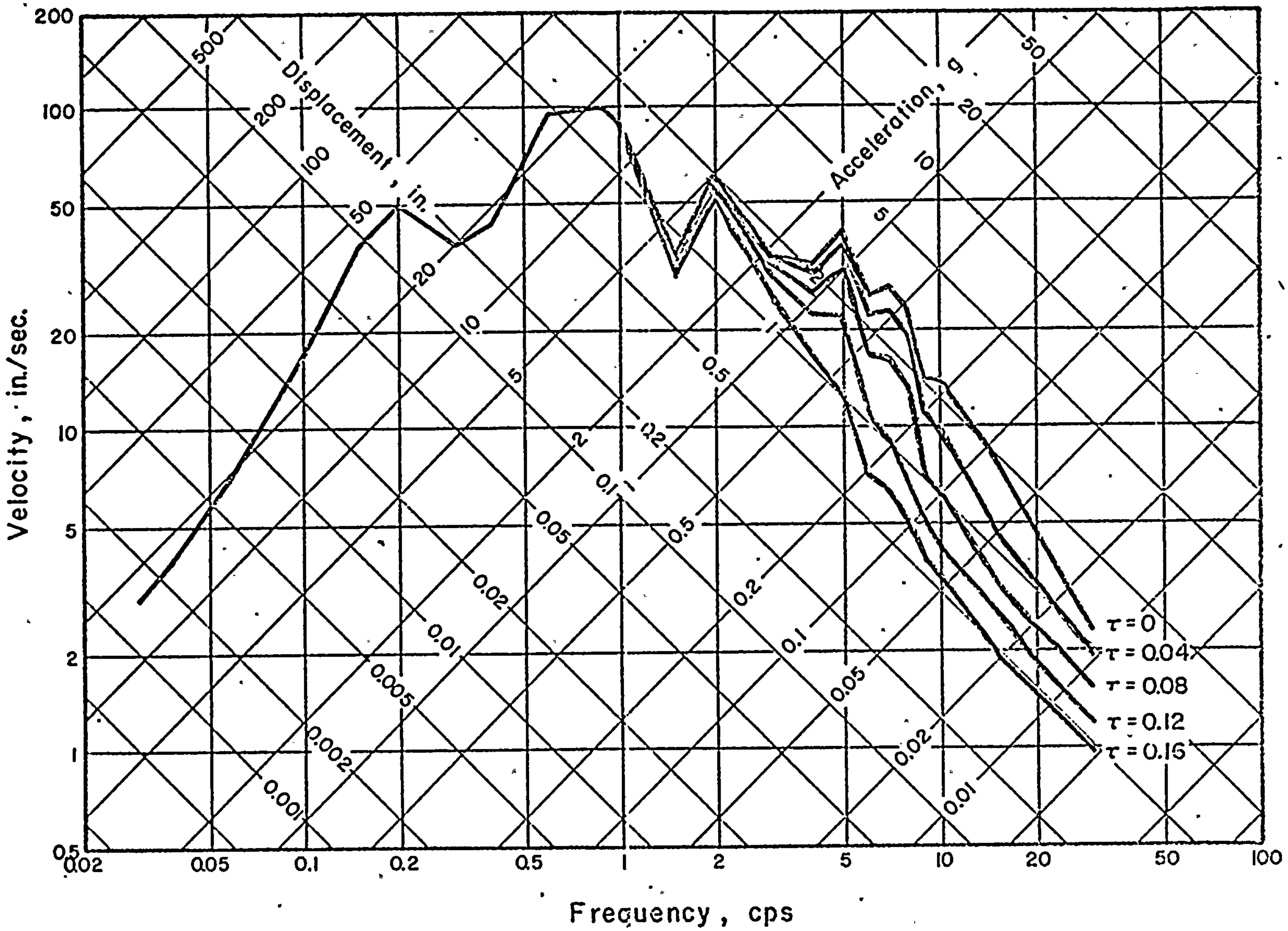



FIG. 1 PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, SIGE, 2 PERCENT DAMPING, $\tau$ = 0, 0.04, 0.08, 0.12, 0.16 sec.

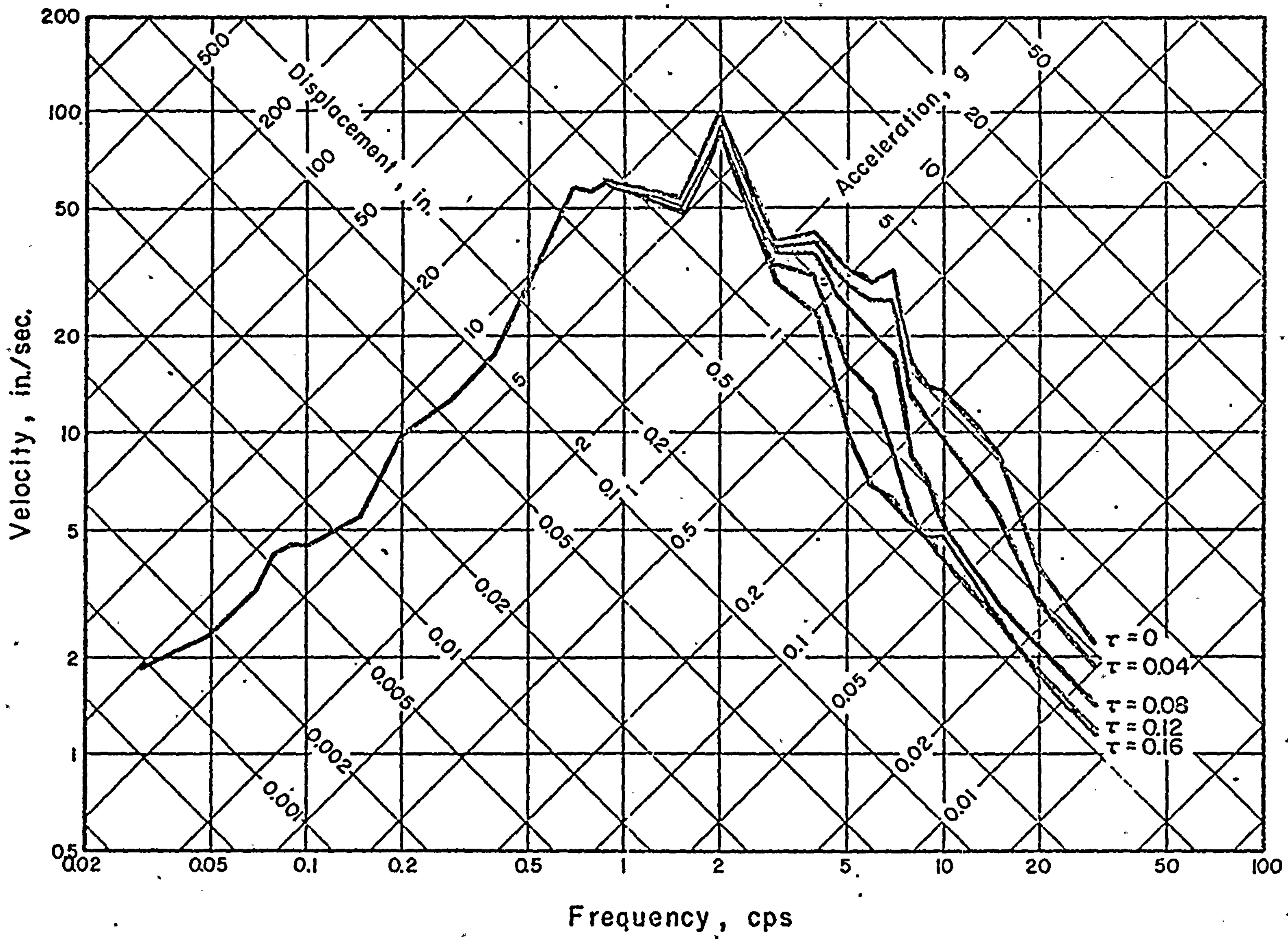



FIG. 2 PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S74W, 2 PERCENT DAMPING, $\tau$ = 0, 0.04, 0.08, 0.12, 0.16 sec.

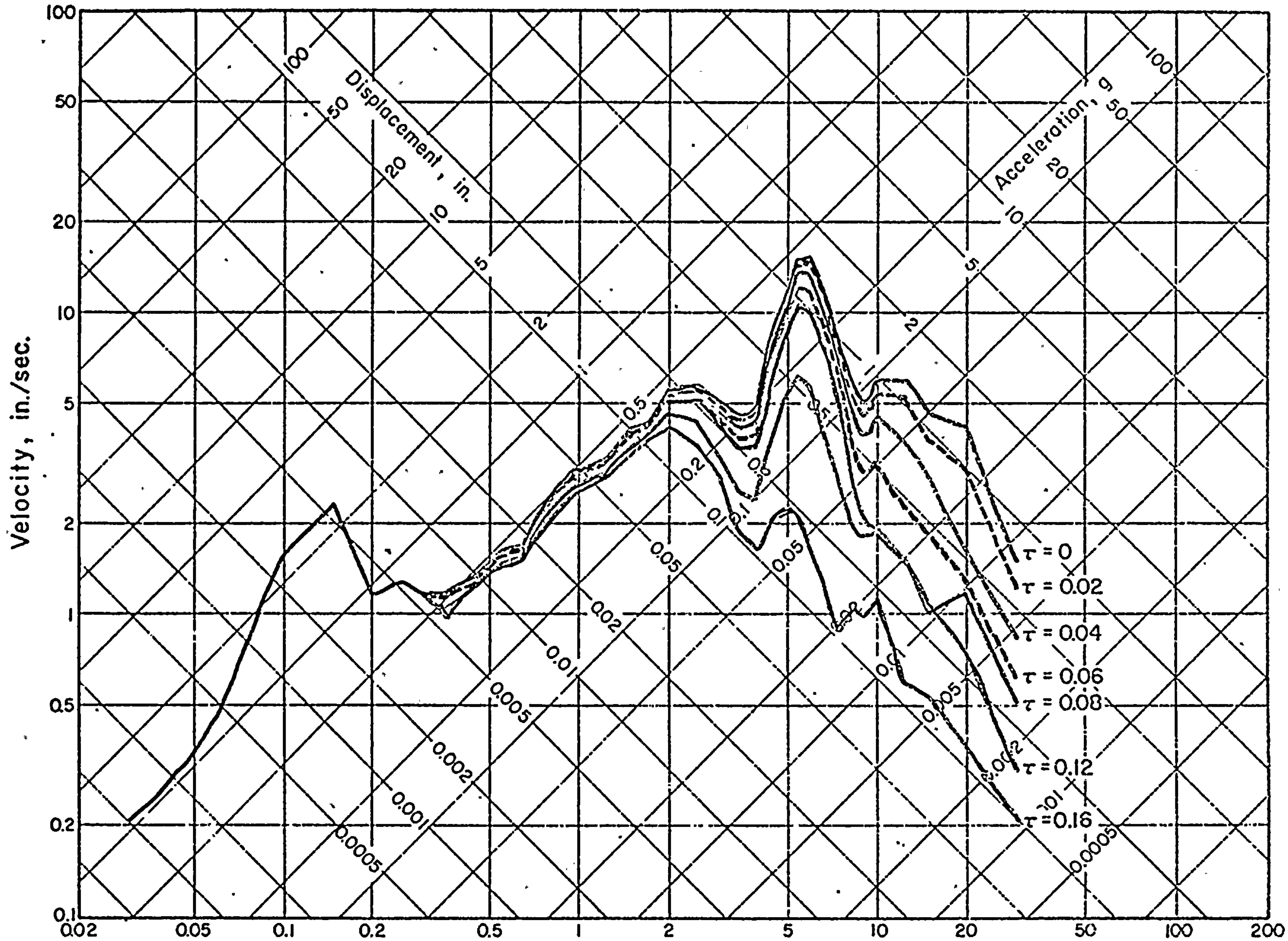



FIG. 3 ANCONA, ROCCA 6-14-72 GMT-NORTH $\tau = 0, 0.02, 0.04, 0.06, 0.08, 0.12, 0.16$
SPECTRUM COMPUTED USING 5.0 PERCENT CRITICAL DAMPING

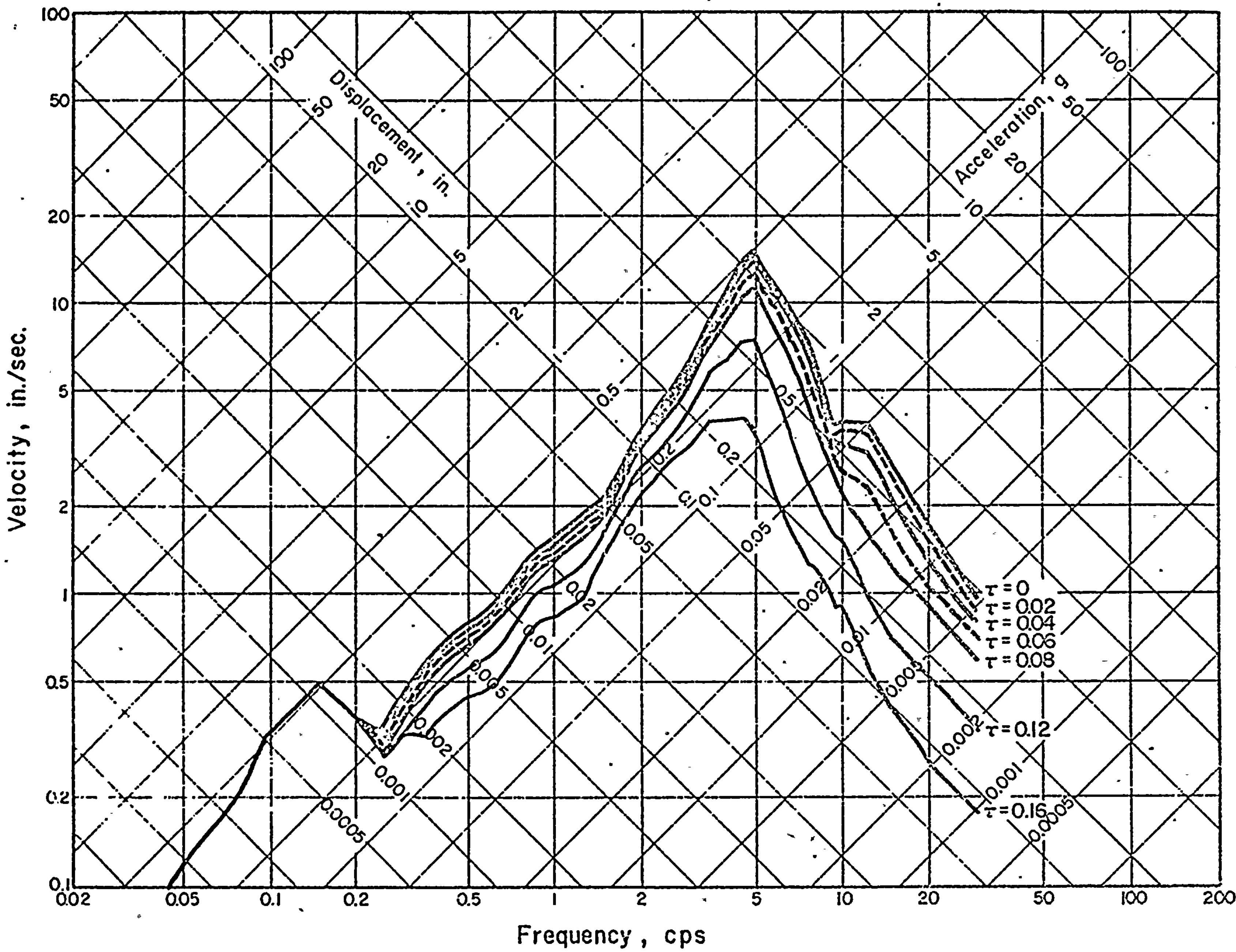



FIG.4 ANCONA, PALOMBINA 6-21-72 GMT-NS $\tau$=0,0.02,0.04,0.06,0.08,0.12,0.16
SPECTRUM COMPUTED USING 5.0 PERCENT CRITICAL DAMPING

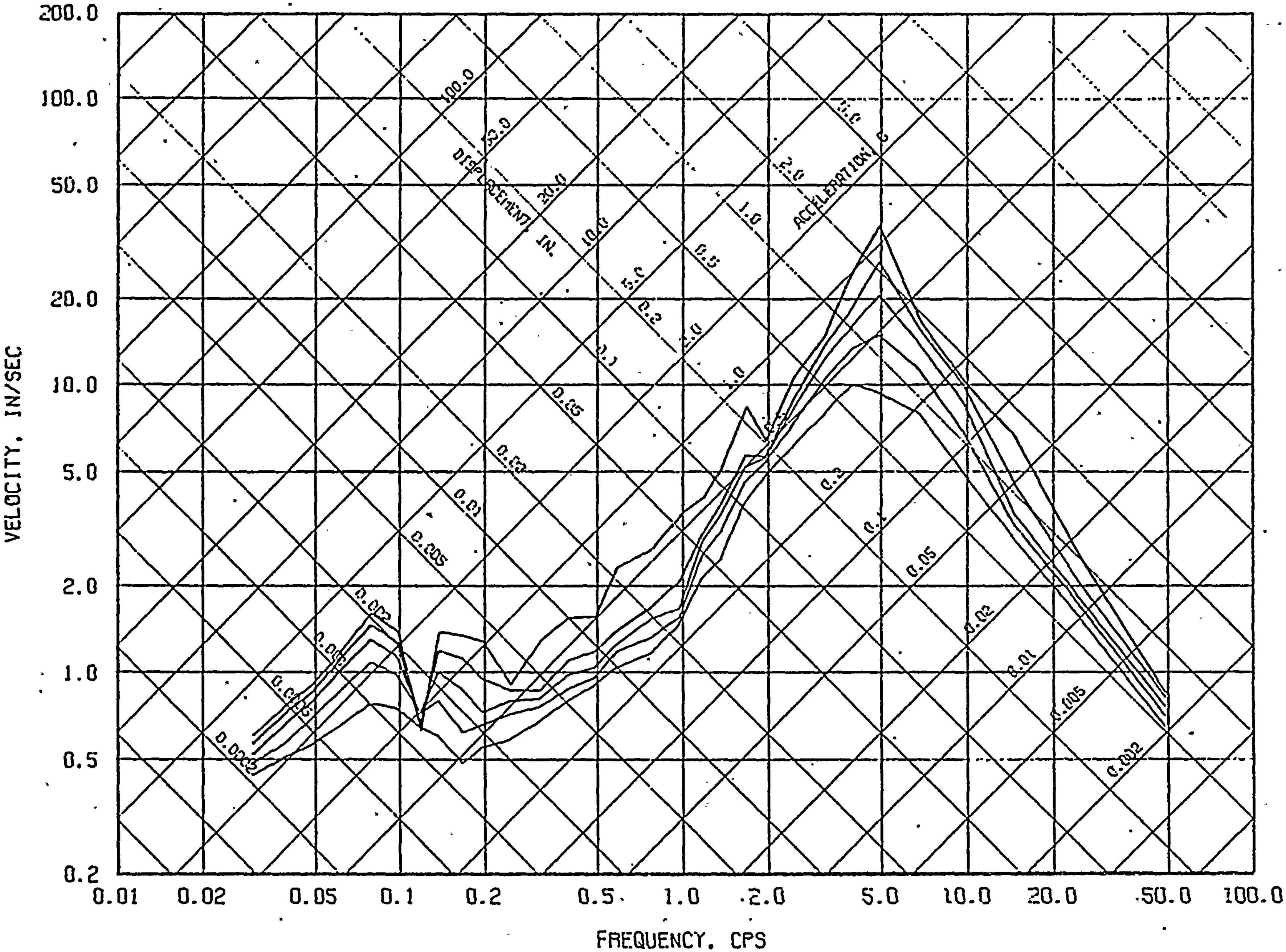



FIG. 5 RESPONSE SPECTRA FOR MELENDY RANCH BARN, 9/4/72 – N61E COMPONENT
0%, 2%, 5%, 10% AND 20% DAMPING (PARABOLIC BASE LINE ADJ. AND FINAL VEL. SET=0)

22

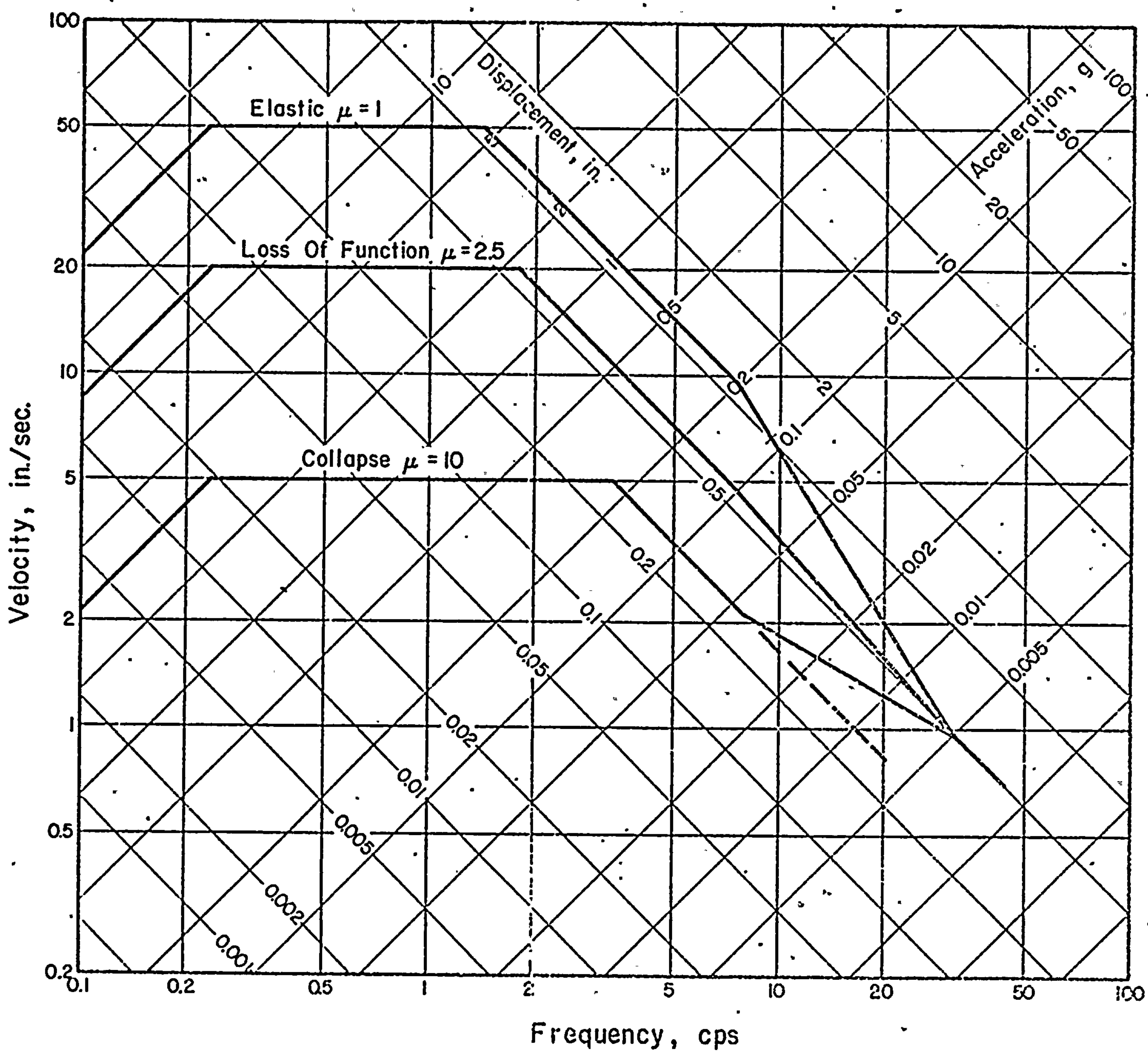



FIG. 6 INELASTIC DESIGN SPECTRAL REQUIREMENT FOR PEAK GROUND ACCELERATION OF 0.5 G, 7 % DAMPING

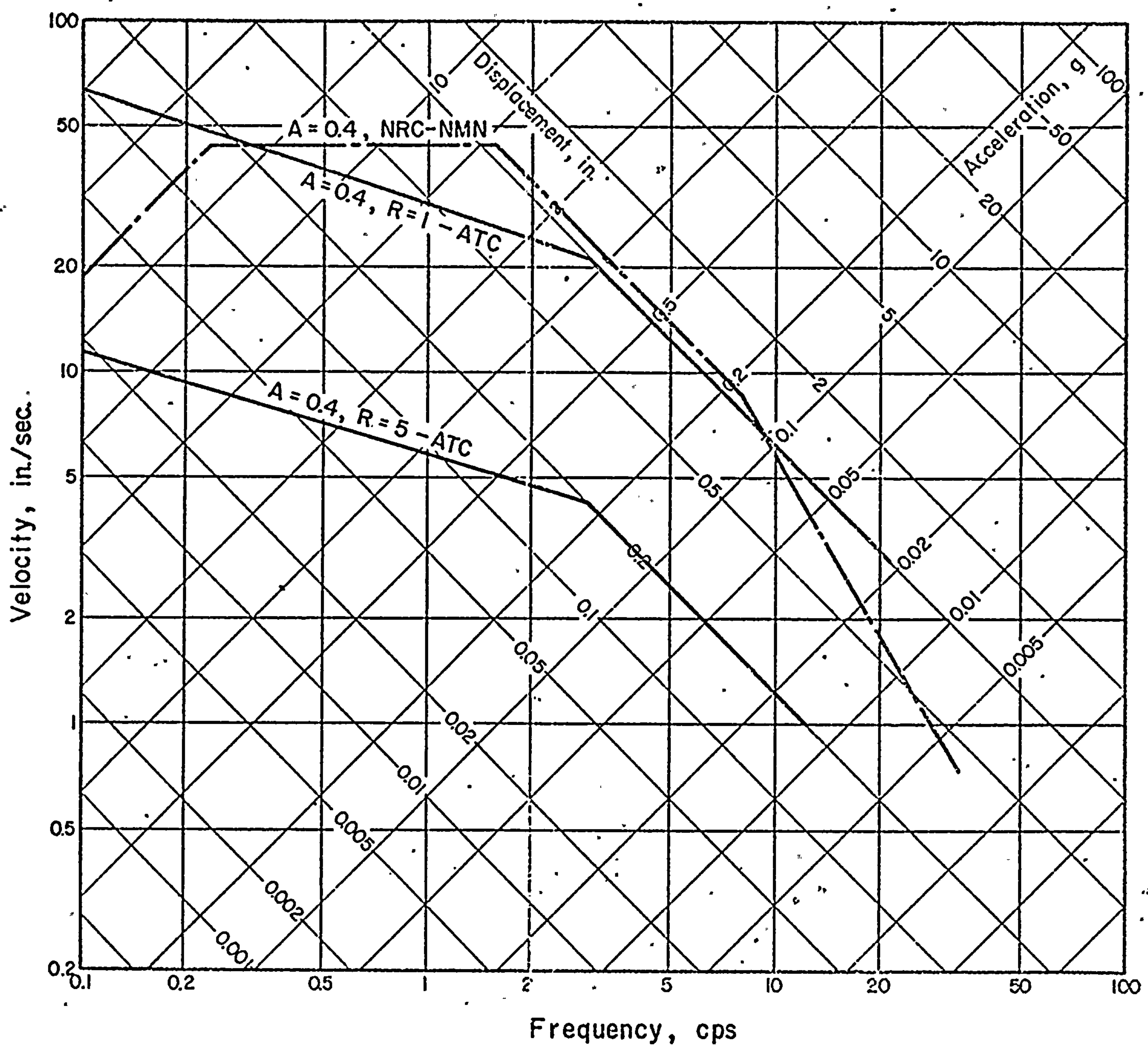



FIG. 7 COMPARISON OF ATC AND NRC DESIGN COEFFICIENTS, 5% DAMPING

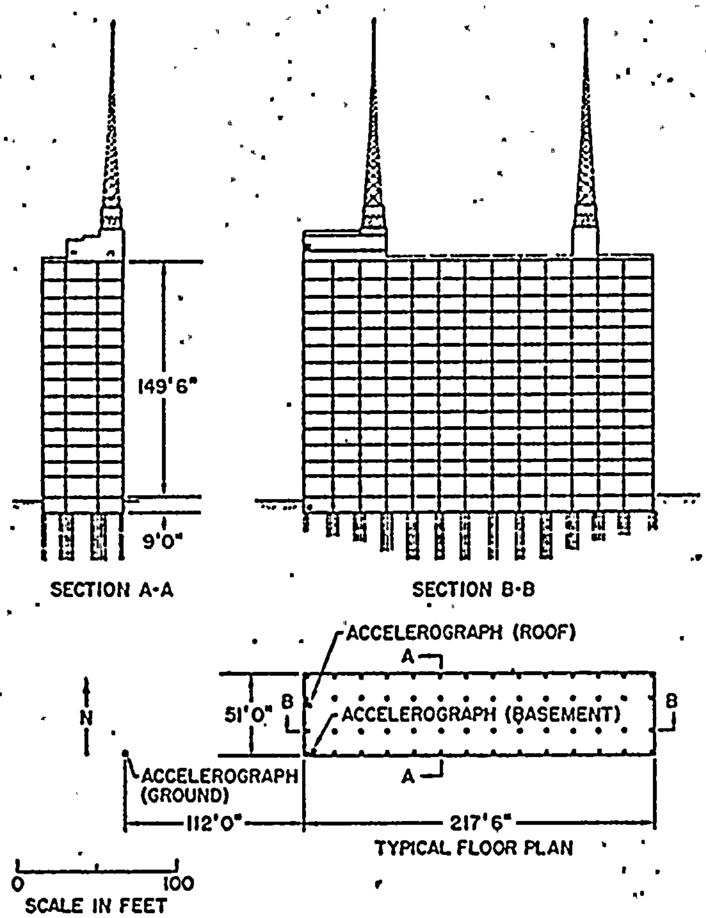



FIG. 8. PLAN AND ELEVATION OF BUILDING

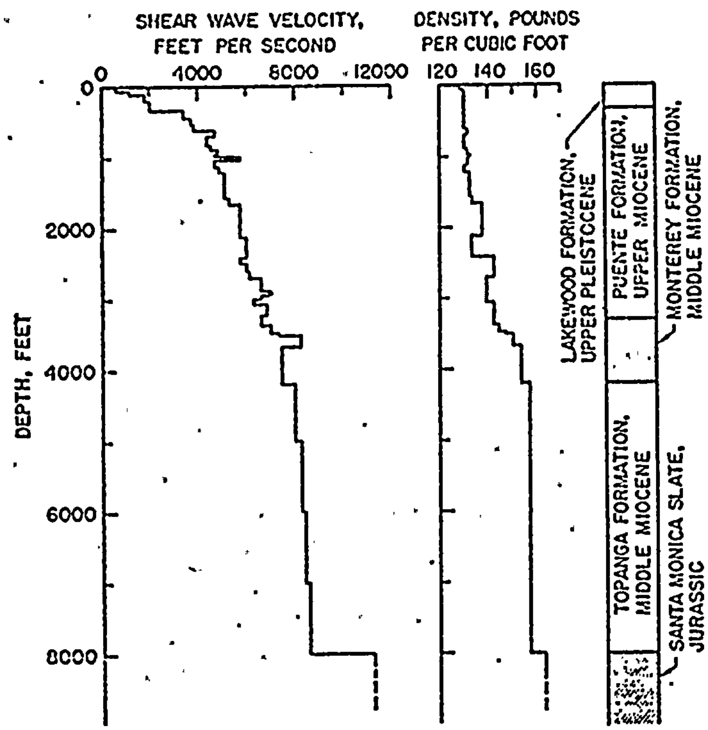



Fig. 9 Subsurface model.

TABLE 2

Natural Frequencies of Building from Vibration Test*

| Mode of Vibration | Frequency (cps) North-south | Frequency (cps) 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.

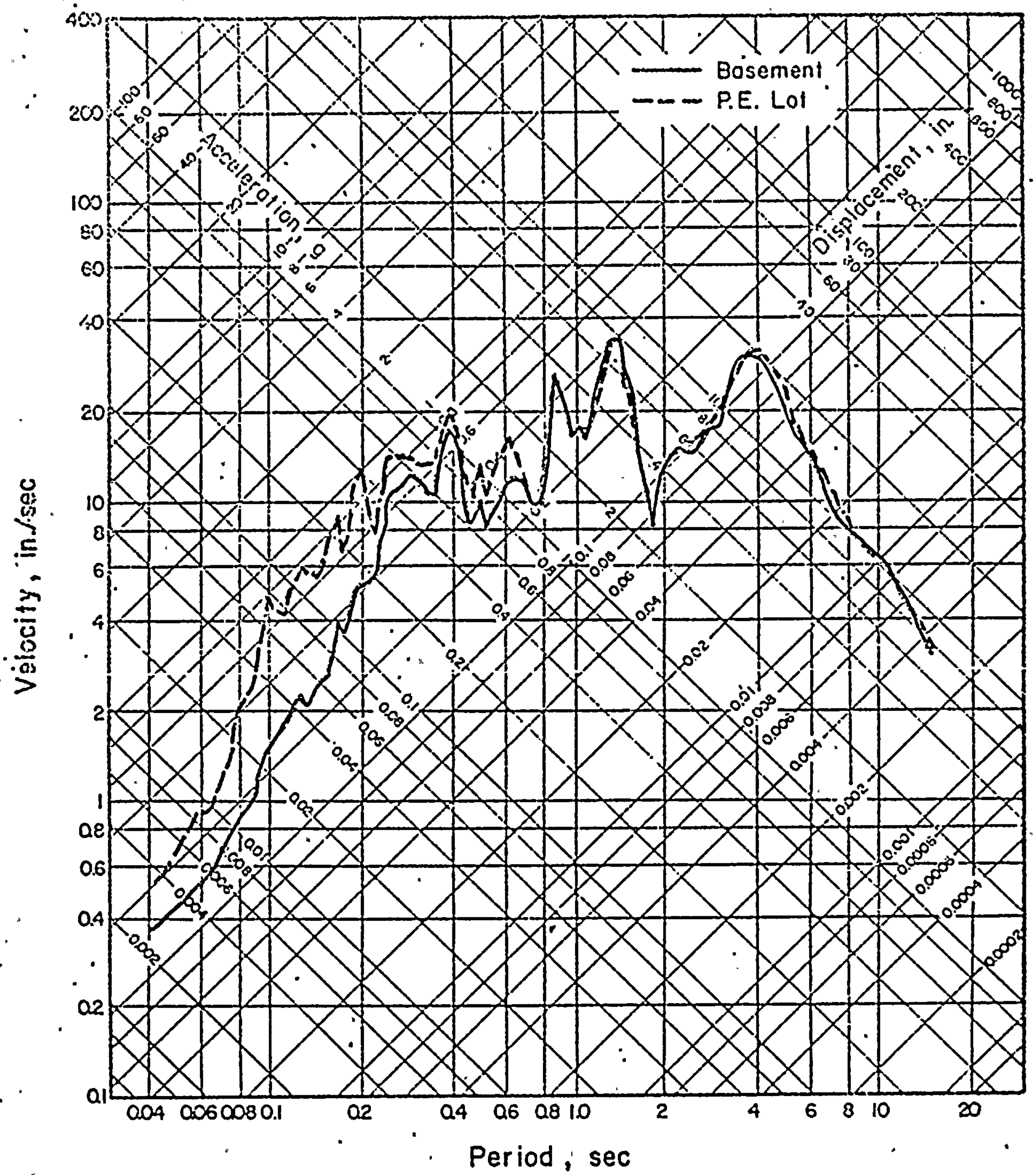



FIG. 10 SAN FERNANDO EARTHQUAKE, FEB. 9, 1971 - 0600 PST HOLLYWOOD STORAGE BASEMENT AND P.E. LOT, COMPONENT EAST, DAMPING VALUE 2% OF CRITICAL

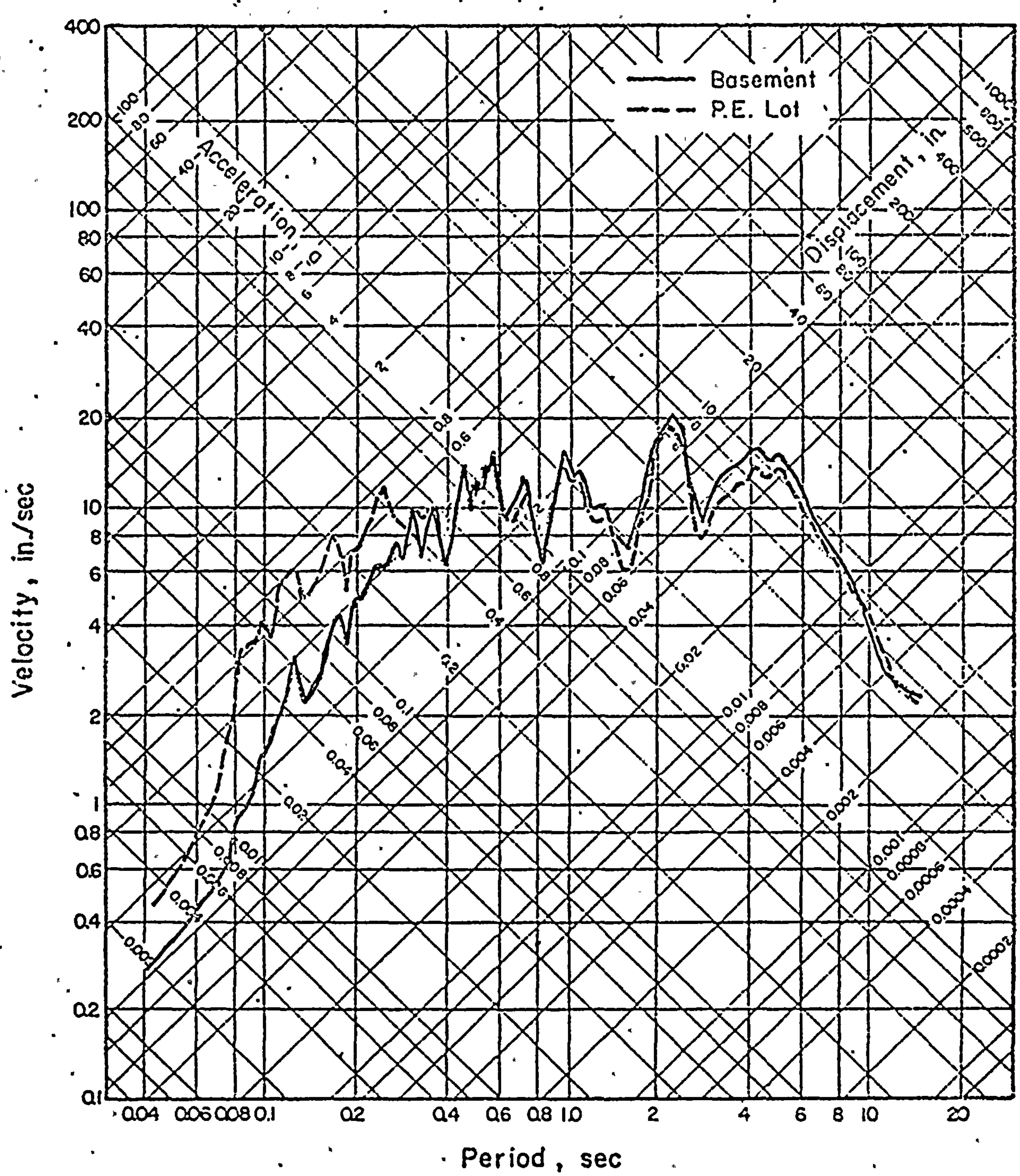



FIG.11 SAN FERNANDO EARTHQUAKE, FEB. 9, 1971 - 0600 PST
HOLLYWOOD STORAGE BASEMENT AND P.E. LOT; COMPONENT
SOUTH, DAMPING VALUE 2% OF CRITICAL

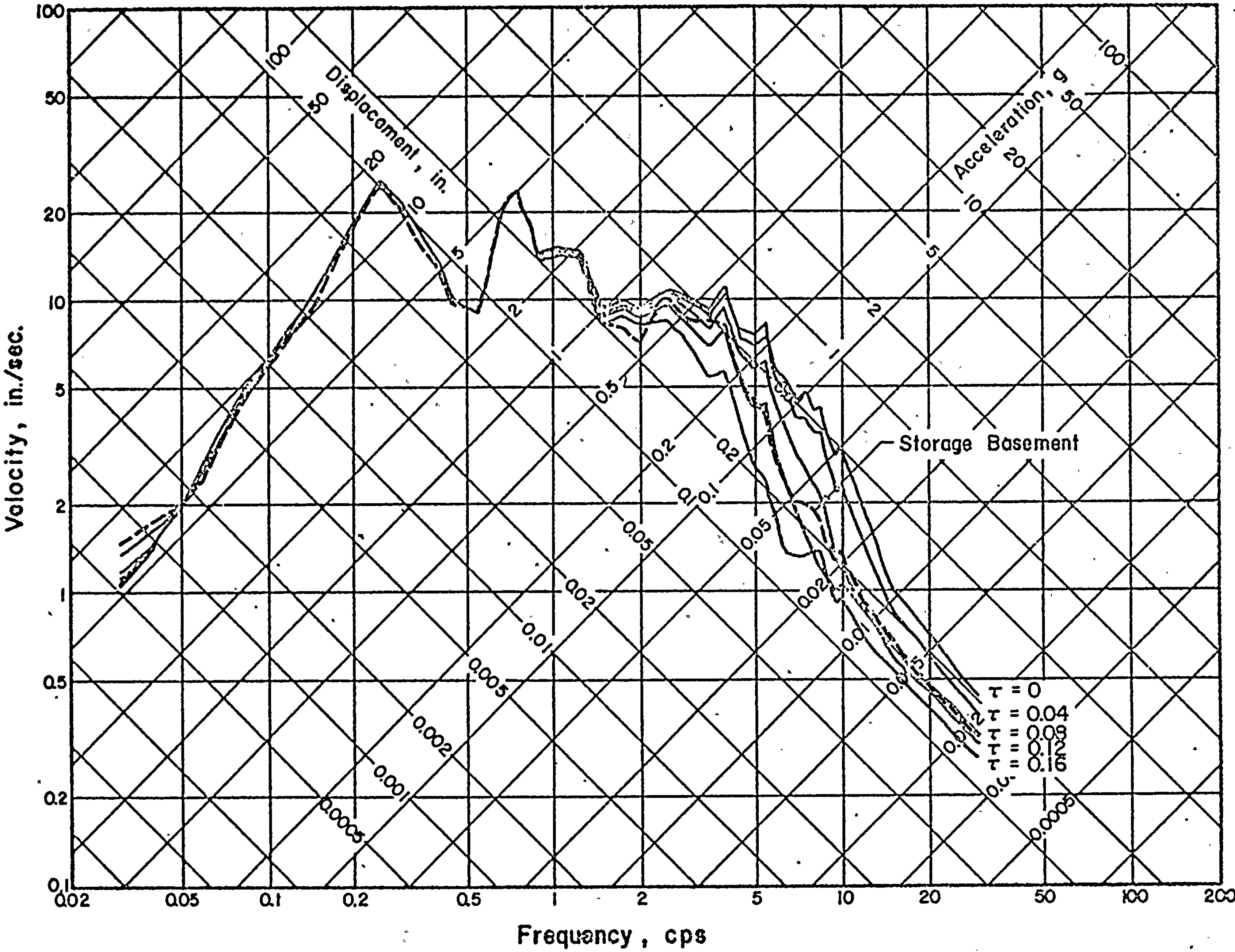



FIG. 12 HOLLYWOOD STORAGE P.E. LOT, SAN FERNANDO EARTHQUAKE FEB. 9, 1971, COMPONENT EAST, DAMPING 5 % OF CRITICAL, $\tau = 0, 0.04, 0.08, 0.12,$ AND 0.16 SEC.

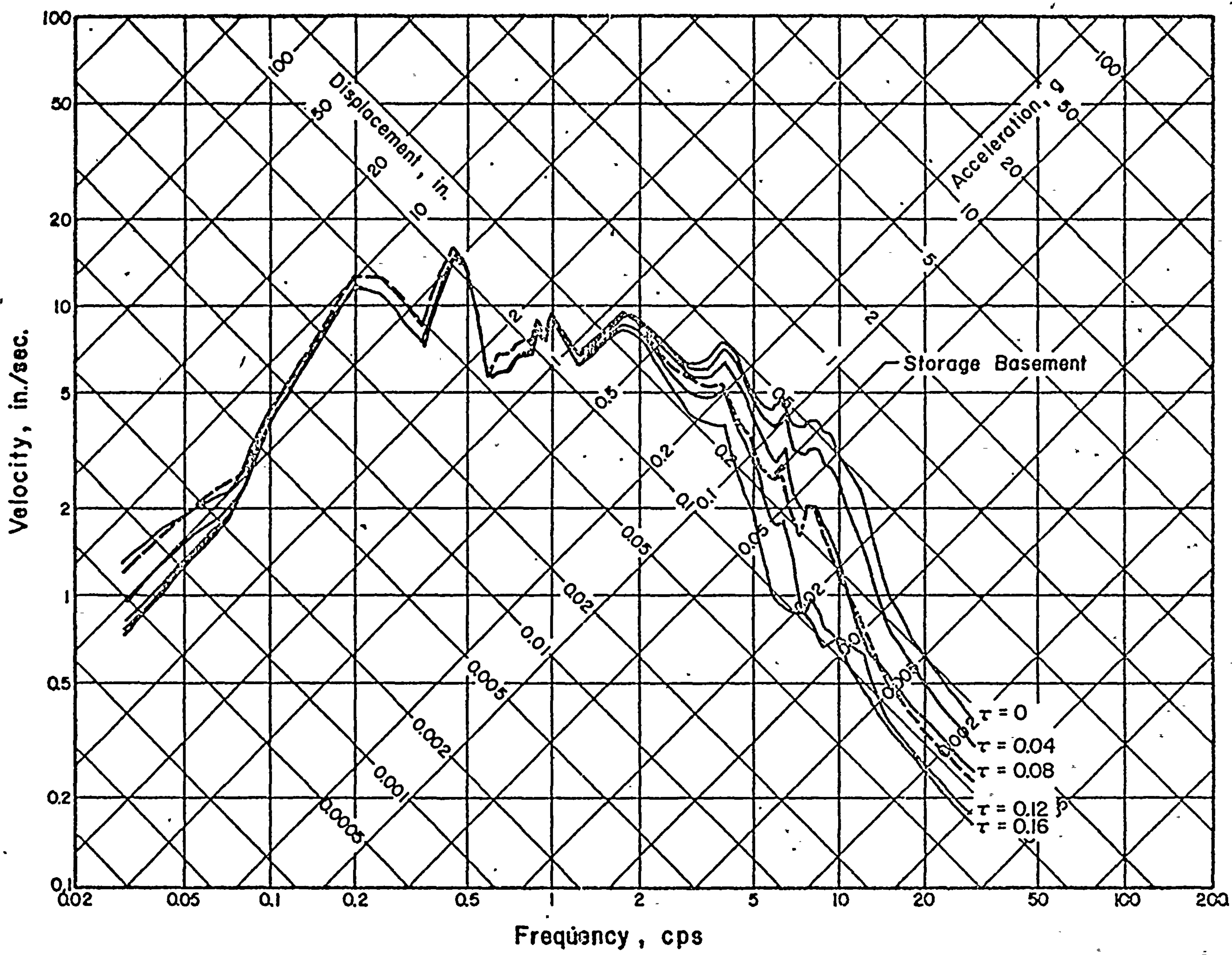



FIG.13 HOLLYWOOD STORAGE P.E. LOT, SAN FERNANDO EARTHQUAKE FEB. 9, 1971, COMPONENT SOUTH, DAMPING 5 % OF CRITICAL, $\tau$ = 0, 0.04, 0.08, 0.12, AND 0.16 SEC.

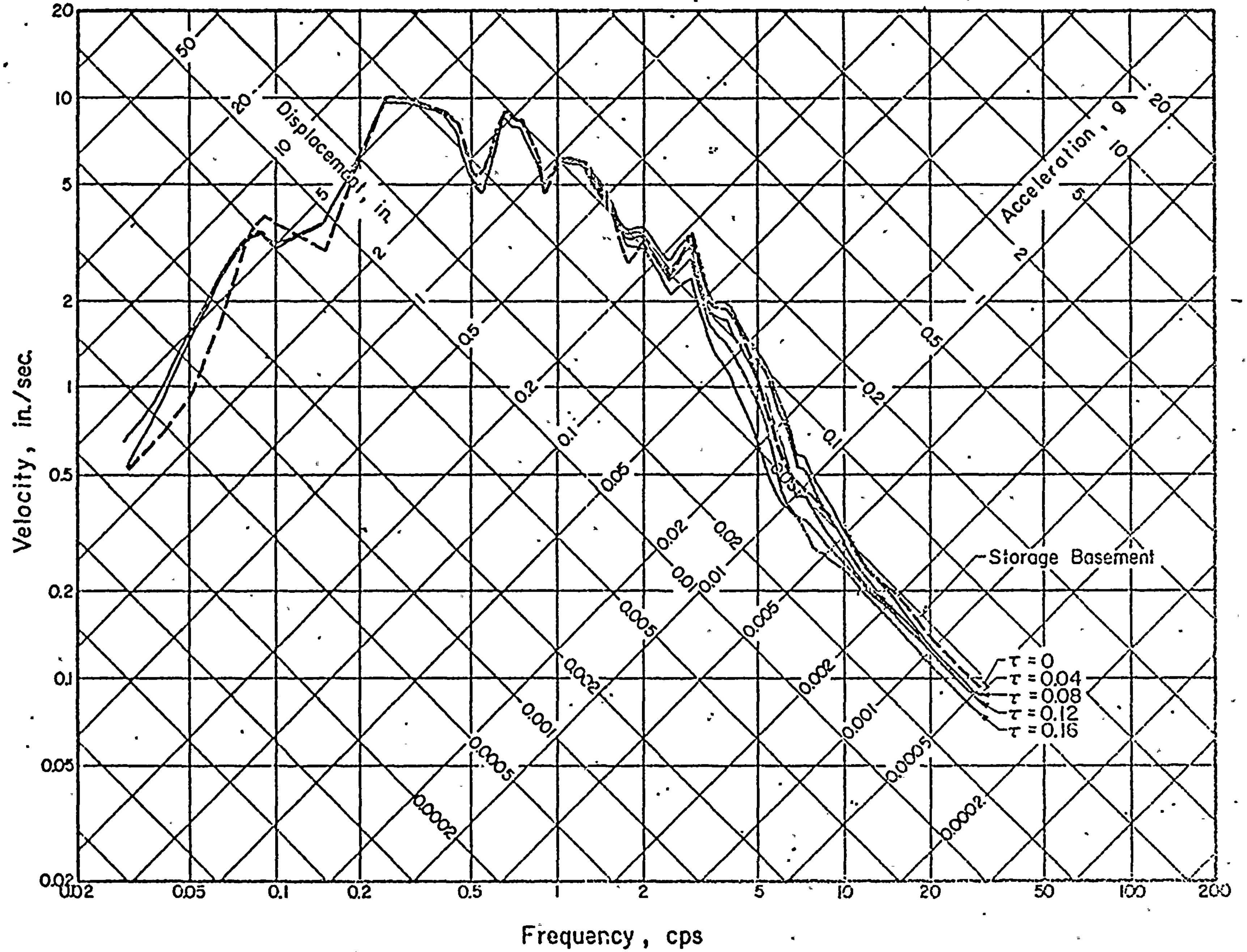



FIG. 14 HOLLYWOOD STORAGE P.E. LOT, KERN COUNTY EARTHQUAKE JULY 21, 1952, COMPONENT EAST, DAMPING 5 % OF CRITICAL, $\tau$ = 0, 0.04, 0.08, 0.12, AND 0.16 SEC.

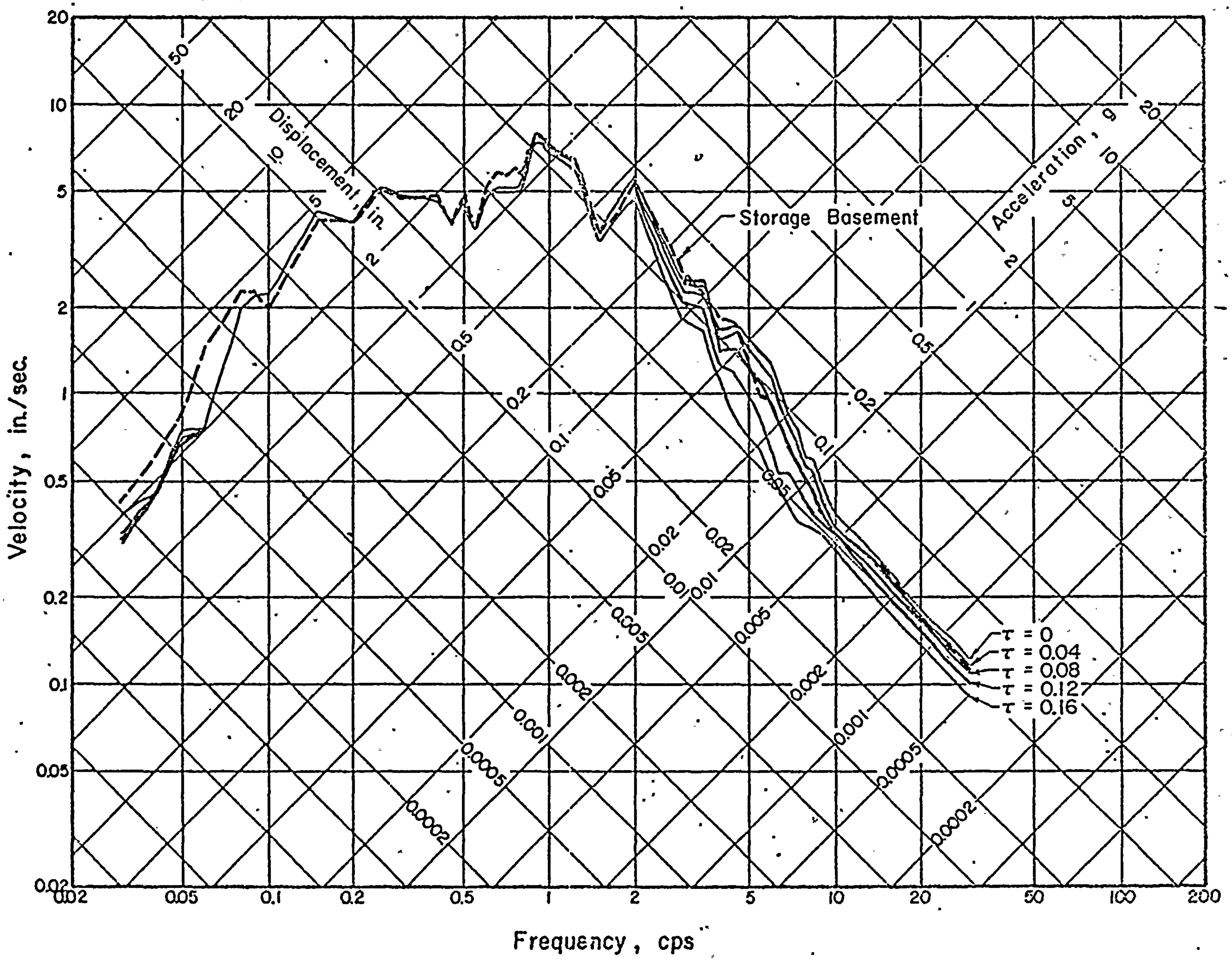



FIG. 15 HOLLYWOOD STORAGE P.E. LOT, KERN COUNTY EARTHQUAKE JULY 21, 1952, COMPONENT SOUTH, DAMPING 5% OF CRITICAL, $\tau$ = 0, 0.04, 0.08, 0.12, AND 0.16 SEC.

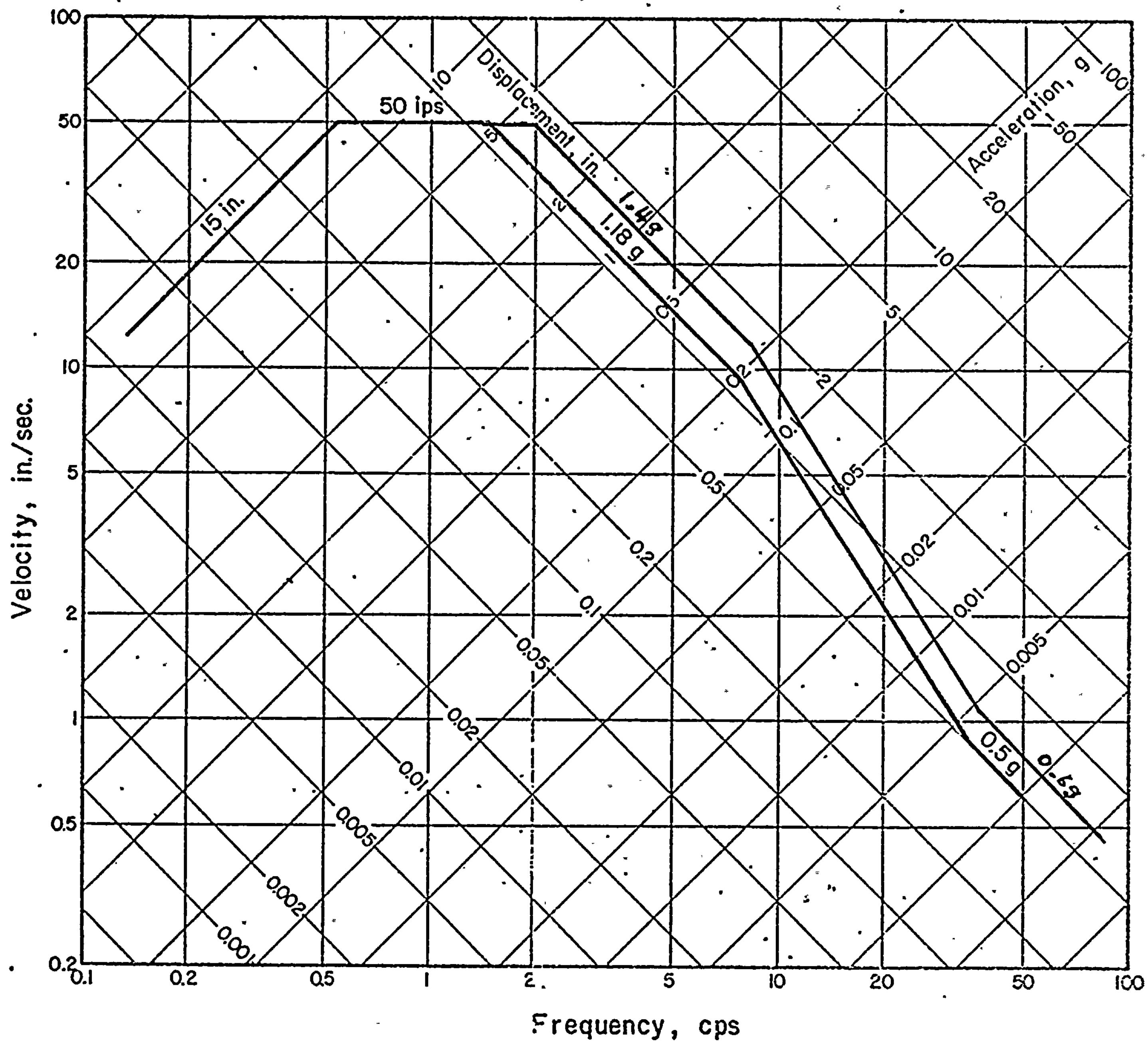



FIG. 16 RECOMMENDED "DESIGN" SPECTRUM FOR PLANT

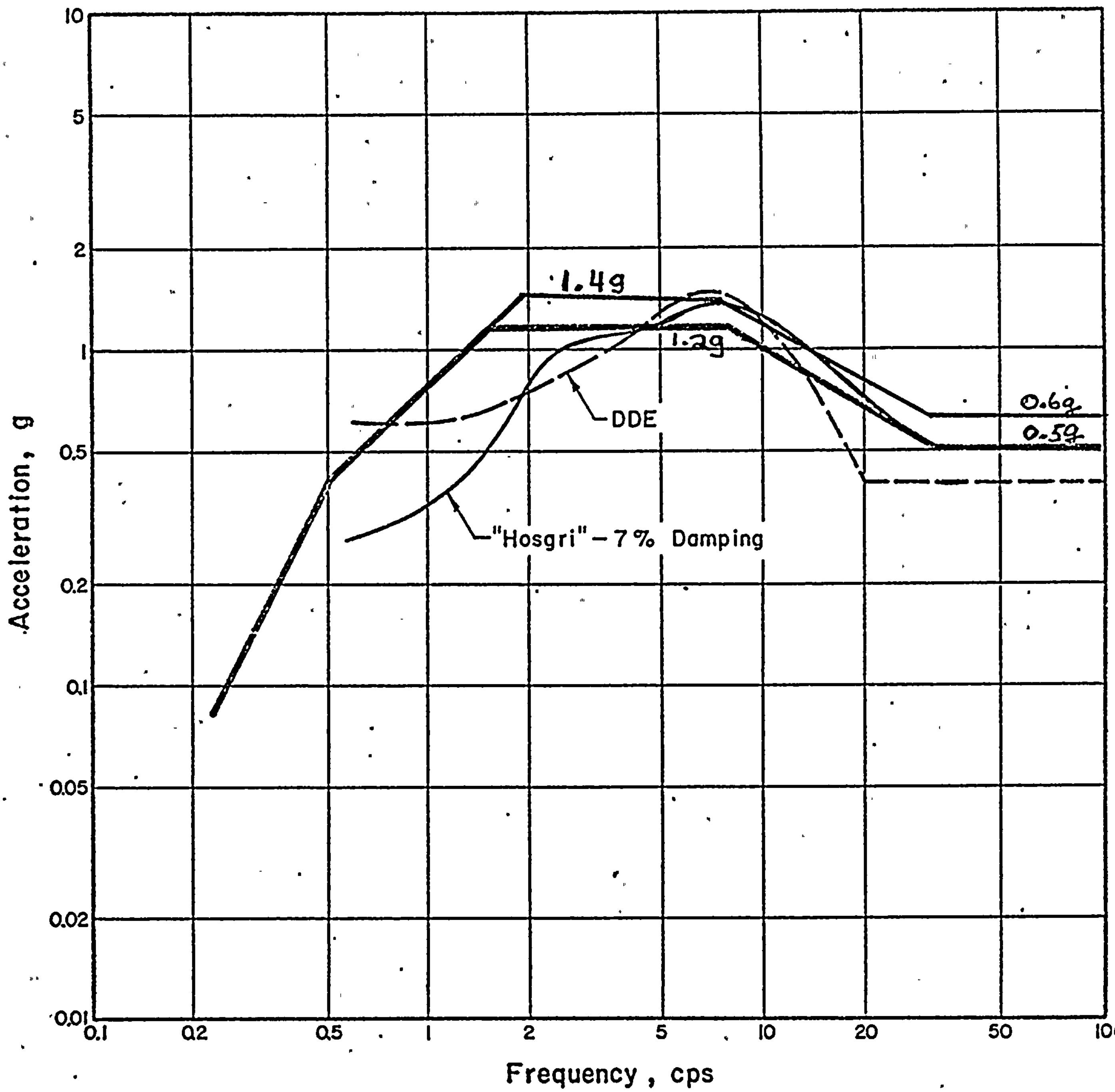



FIG. 17 RECOMMENDED "DESIGN" SPECTRA, 7% DAMPING, COMPARED WITH "HOSGRI" AND DDE SPECTRA

PRESENTATION BY K. KAPUR

KAPUR

PROPOSED BASIS FOR SEISMIC DESIGN
RE-EVALUATION FOR DIABLO CANYON

At the present time, the method of analysis commonly used for estimating earthquake response of structures is based on the assumption that the ground just beneath the foundation vibrates in phase and with the same amplitude everywhere, implying as if the structure were resting on a shake table. Under such an assumption, all points beneath a large foundaton will, apart from soil-structure interaction effects, attain their peaks in acceleration at the same moment. The staff recognizes that this simplified representation of seismic input is an analytical covenience suitable for computing seismic forces, stresses and displacements. The actual earthquake, in fact, will consist of waves propagating in all directions, and it is prudent to note that points widely separated beneath a structural foundation will not achieve the same acceleration at the same moment.

Theoretical work, including studies underway by Dr. Nathan M. Newmark, and a recent paper by Scanlan presented in the Third International Conference on Structural Mechanics in Reactor Technology in Berlin, indicates that large structure foundations do not respond to high frequency or even intermediate frequency earthquake motions with the same intensity as smaller foundations do, particularly foundations associated with the support of instruments. Further verification of this phenomenon is indicated by the response measured in the Hollywood Storage Building compared with the response computed from records in the free field about 150 feet away (see Fig. 1). Here the high frequency components are attenuated by a factor of 2 to 3, in the range of frequencies higher than about 1.5 hertz, for earthquakes even 22 miles away.

Yamahara in Japan made similar observations during the Tokachioki earthquake of 1968. It was found by Yamahara that the maximum

amplitude of a building foundation is always smaller than that of the adjacent ground and that the difference becomes larger as the wave length becomes shorter. In other words, if the input vibration frequency is relatively high, the effective input power to a building is greatly decreased, because there is a large phase difference among the movements of different points of building foundations.
This is why ground motion having high frequency content does not usually cause severe response of a building, as it is shown by the current methods of calculation even if the acceleration of the ground motion is fairly large.

Yamahara developed an analytical method for numerically estimating the input loss. He applied this method to Tokachioki earthquake record on ground surface and showed excellent correlation with the observed earthquake record at the ground floor of a building. Reduction in response spectra using his approach is shown in Fig. 2 for Tokachioki earthquake. A similar approach was used by Newmark on Pacoima Dam earthquake and was recommended by him and the staff for use on Diablo Canyon. Typical results for Pacomia Dam earthquake are shown in Fig. 3. Here T is the effective length of the foundation slab divided by the shear wave velocity. This approach, as recommended by Newmark introduces an average response spectrum dependent on the area of the foundation, provided some account is taken of additional tilting and torsion which may result as a consequence of the nonsynchronized earthquake motions. In effect, both Dr. Newmark and the staff are recognizing that response spectra, strictly speaking, are applicable at a point only. When structures are built over large areas some modification of these response spectra is justified.

Another refinement of current seismic criteria is the use of ductility factor. The ductility factor is the ratio of the maximum useful (or design) displacement of a structure to the "effective" elastic limit displacement, the later being determined not from the actual resistance-displacement curve but from an equivalent elasto-plastic function (See Fig. 4). This equivalence requires that the energy

absorbed in the structure (or area under the resistance-displacement curve) at the effective elastic limit and at the maximum useful displacement must be the same for the effective curve as for the actual relationship at these two displacements. Ductility levels for use in design may be as large as 2 to more than 5. In Diablo Canyon, at the recommendation of Dr. Newmark, we have permitted the applicant to use a very low ductility ratio of 1.2 which we consider to be quite conservative. Typical Response Spectra for elasto plastic systems are shown in Fig. 5 for the El Centro earthquake. Figure 6 shows a procedure for generating inelastic response spectra from the elastic response spectra. In Figure 6, D, V and A refer to the bounds of the elastic spectrum while the symbols D', V' and A' to the bounds of elasto-plastic spectrum for acceleration. In general, the response spectrum is decreased by a factor of $\mu$ for acceleration up to a frequency of 2 hertz and by the factor of square root of $2\mu-1$ between 2 and 8 hertz. There is no reduction above 33 hertz.

Some judgment has to be used in selecting proper ductility factor for use in Diablo Canyon reevaluation. Observation of the performance of structures in earthquakes, interpretation of Laboratory tests, including those on earthquake simulations and shake tables, observations of damage to structures and structural models in nuclear tests, including damage from both air blast and ground shock, all are pertinent factors in arriving at a judgment as to the appropriate ductility factor to be used in design. We were guided by Dr. Newmark in selecting a ductility factor of 1.2 for Diablo Canyon nuclear plant.

FIG. 1 SAN FERNANDO EARTHQUAKE, FEB. 9, 1971 - 0600 PST
HOLLYWOOD STORAGE BASEMENT AND P.E. LOT,
COMPONENT EAST, DAMPING VALUE 2% OF CRITICAL.

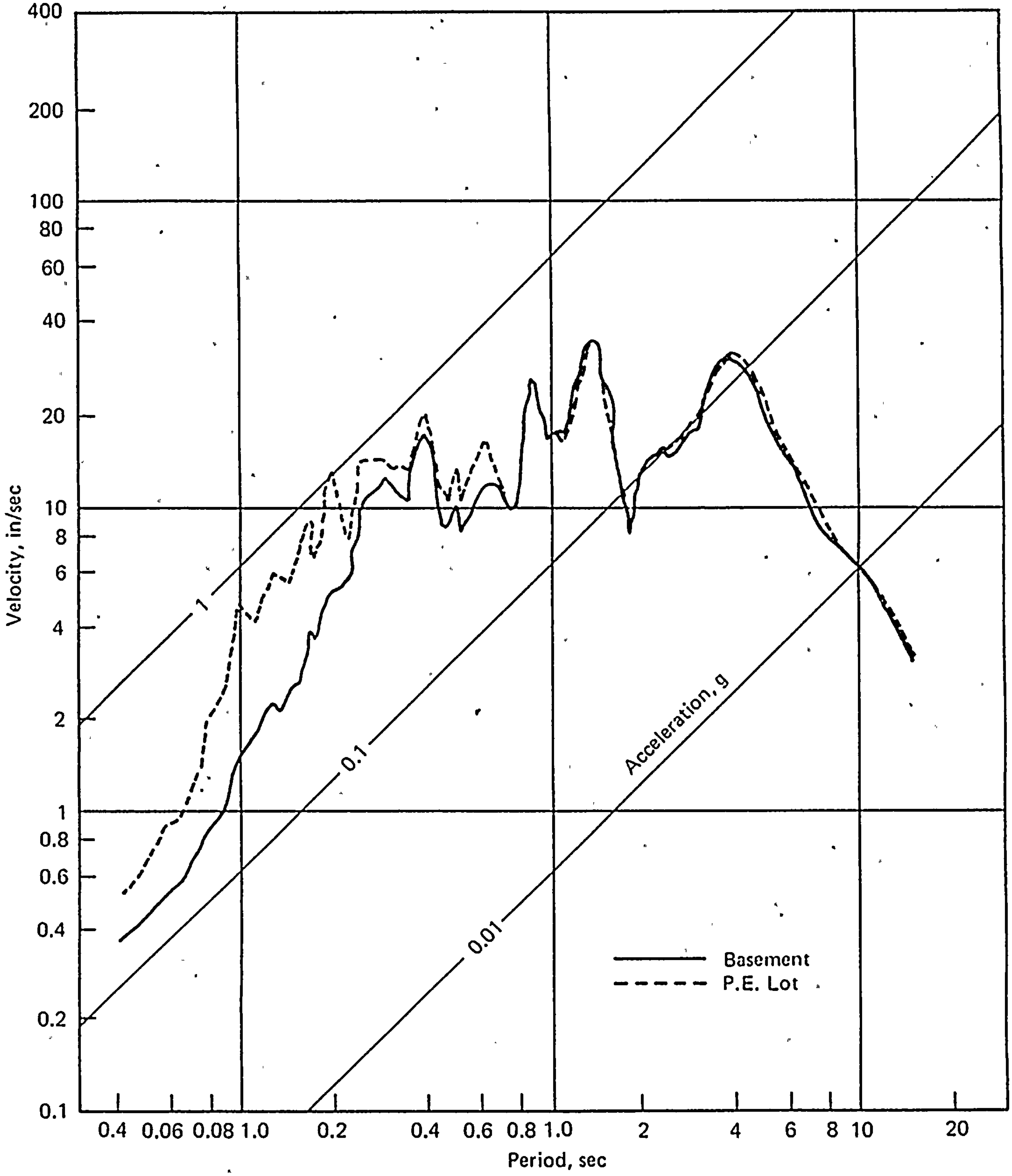


FIG.2 CHANGE IN ACCELERATION RESPONSE SPECTRA OF THE EFFECTIVE INPUT MOTIONS

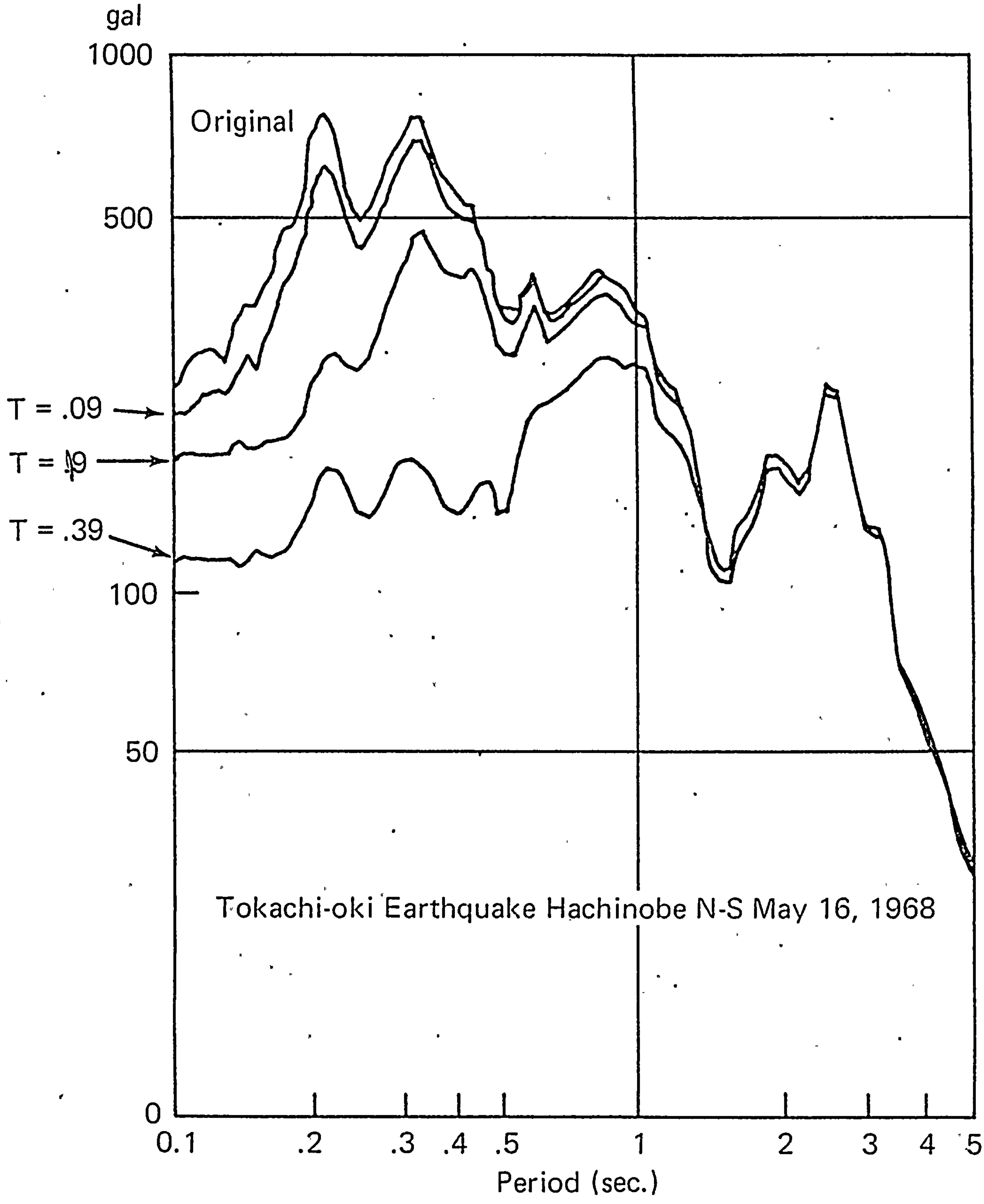


FIG. 3 PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S16E, 2 PERCENT DAMPING, $\tau$ = 0, 0.04, 0.08, 0.12, 0.16 SEC.

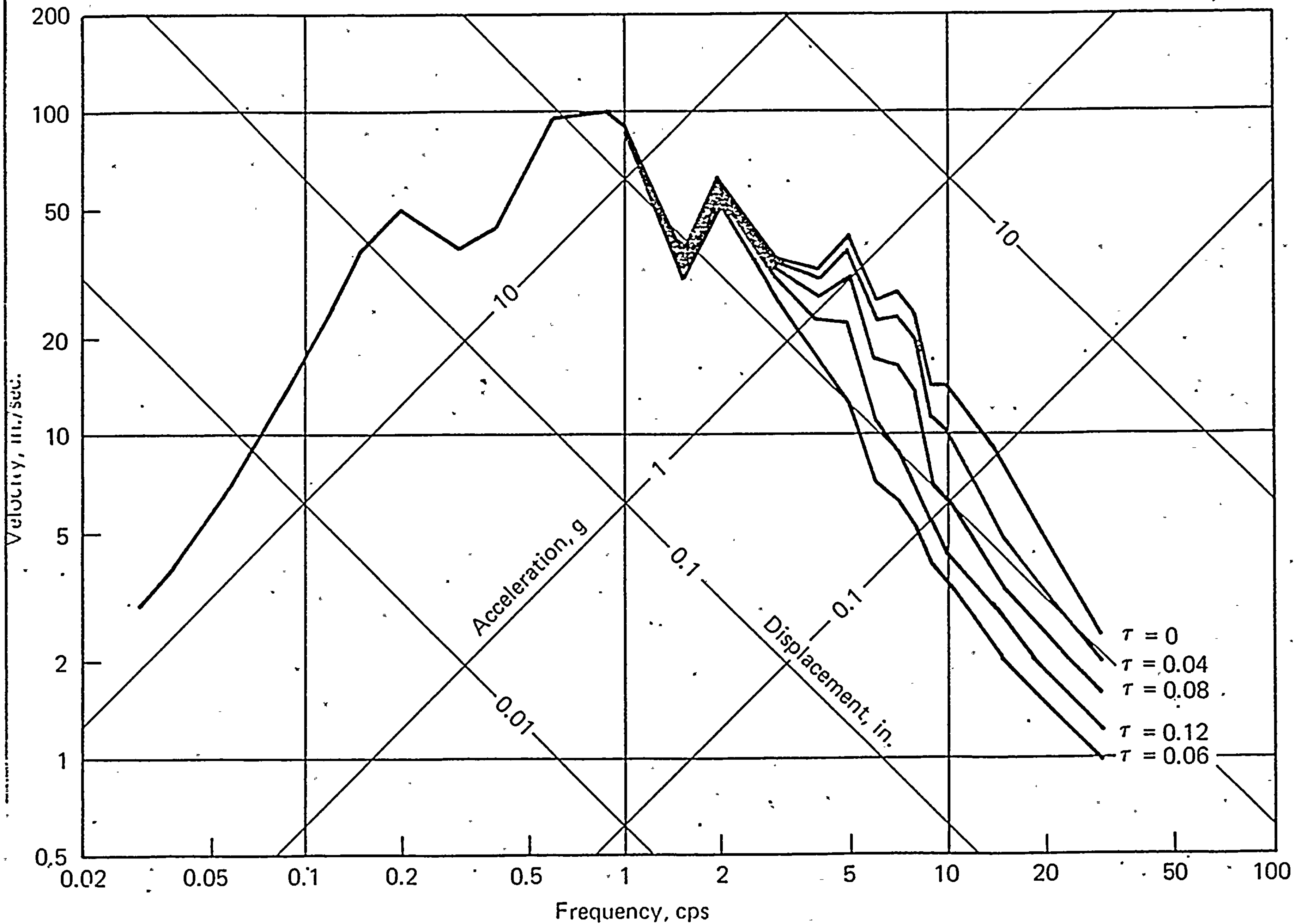


FIG. 4 RESISTANCE-DISPLACEMENT RELATIONSHIP

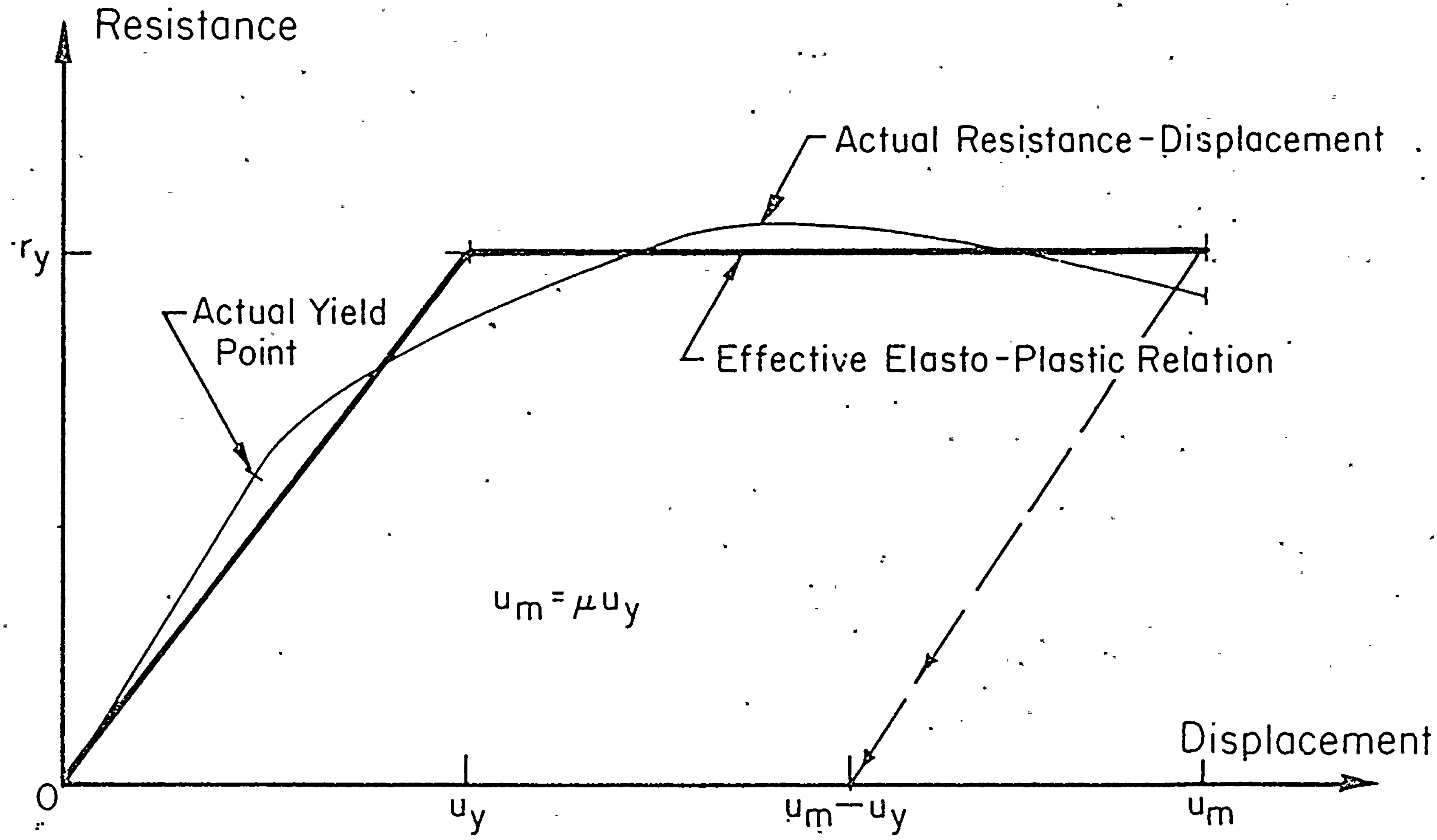


FIG.5 DEFORMATION SPECTRA FOR ELASTO-PLASTIC SYSTEMS WITH 2% CRITICAL DAMPING SUBJECTED TO THE EL CENTRO EARTHQUAKE.

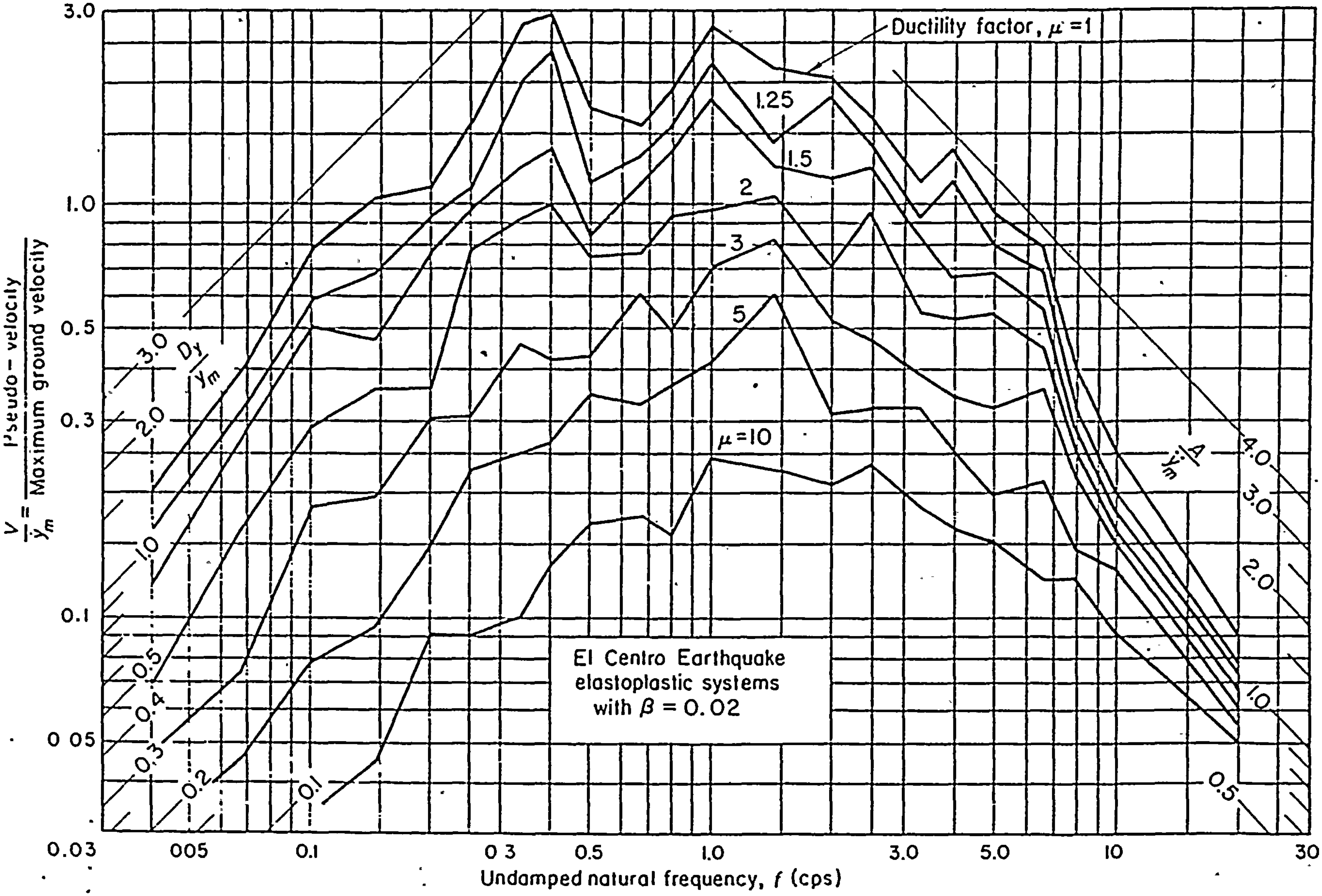
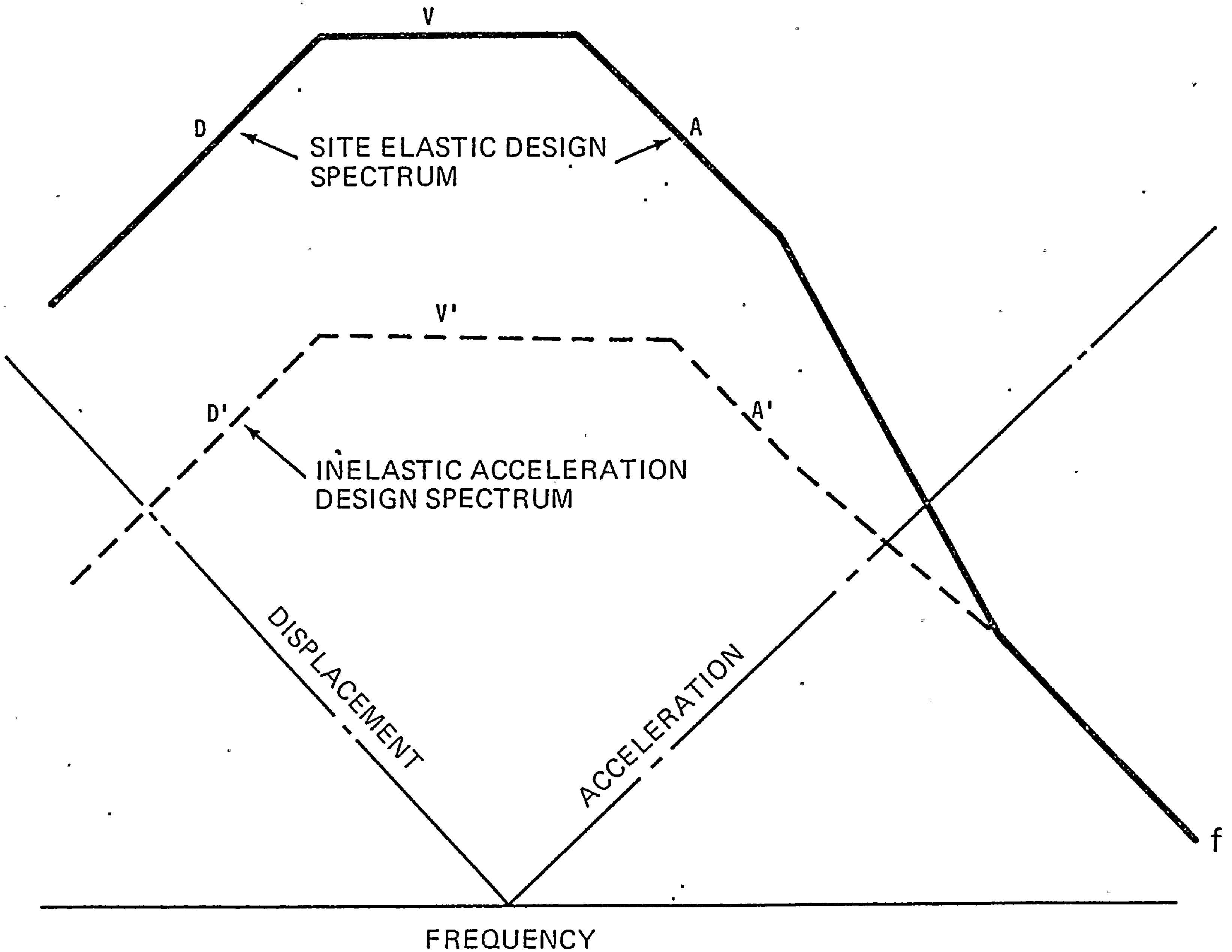


FIG. 6 INELASTIC DESIGN SPECTRA GENERATION



PRESENTATION BY I. SIHWEIL

CONSERVATISMS IN SEISMIC DESIGN

Safety margins were incorporated in the seismic design procedures used for Diablo Canyon primarily because of the inherent conservatisms that exist in the various steps of the process. The staff cannot at this time quantify each of the parameters that contribute to the overall safety margin. Nevertheless, I will present to you today a qualitative assessment of these margins.

To facilitate your understanding of the various aspects of seismic analysis and design, I would first like to explain briefly how the process is accomplished. The whole process can be subdivided into three major steps. The first step is the selection of the earthquake event and a subsequent definition of the associated ground motion that is to be used in the analysis. The ground motion is usually characterized by a response spectrum which essentially defines the maximum response of the structures to the ground motion as a function of the frequency of the structure. The second step in the process is the mathematical modeling and analysis of the various plant structures and components. It should be noted at this point that before the plant structures and equipment are mathematically analyzed for seismic loads they are first sized up and physically arranged within the plant for reasons other than seismic. The analysis is then performed using highly sophisticated analytical

techniques that have been developed and already qualified and verified by comparing the analytical results with measured results. For electrical and mechnical components that are not amenable to analysis such as instruments and control panels, the seismic qualification is usually accomplished by experimental shake table testing. Finally in the third step of the process, the forces, moments, deflections, shears, etc., obtained from the second step are used to check, verify and/or modify the size of the structural members that have been previously selected such as shear walls, beams, equipment supports, pipe restraints, anchor bolts, etc.

SLIDE (1)

In each of these three major steps there are several conservatisms inherent in the various substeps that have been developed over the past several years. For example, in the first step, there are inherent conservatisms in the selection of the design event. SLIDE (2).

I.A. Selection of a Low Probability Extreme Event

The intensity of the earthquake selected is based on conservative assumptions and a very low probability event. Although conventional structures in California are designed for earthquake loads, they are not designed to such extreme events.

I.B. Wide Band Ground Response Spectra

The ground response spectra used for the definition of seismic input are usually smoothed wide band spectra which conservatively eliminate the irregularity of response spectra of actual earthquakes. A wide

band response spectrum is essentially equivalent to an earthquake motion that is very rich in frequency content.

I.C. Conservative Amplification Factors

The amplification factors that define the design response spectra are conservatively chosen based on statistical studies of several past earthquake records. They are based on the mean recorded amplification factors plus one standard deviation.

I.D. Enveloping Synthetic Time Histories

For the analysis of systems and components, the time history method of analysis is usually used and the time history motion is so developed that its response spectra will envelope the smoothed wide band design response spectra. This obviously provides an additional conservatism in the design of systems and components. The second major step of the process includes the following conservatisms: SLIDE (3)

II.A.1 Elastic Dynamic Analysis

Despite the well known fact that most of the structural materials possess a considerable strength reserve in the inelastic range, the seismic analysis of structures, systems, and components is performed on the basis of elastic material behavior. This approach results in an overestimation of the response and thus a conservative design.

II.A.2 Damping Values

The damping values used in the seismic design process of structures,

systems and components are usually lower than those obtained from actual experimental results. Higher actual damping values will result in a lower response than that determined by analysis using low damping values.

## II.A.3 Load Combinations

Seismic loads are combined with other normal and/or extreme loads that may or may not be present during the earthquake. For example, we require some combinations that are considered very improbable such as the combination of pipe rupture loads and earthquake loads. High energy pipes are designed for these extreme seismic loads and are not expected to rupture during earthquakes. Nevertheless, to achieve a higher margin of safety, such a combination of transient and dynamic loads is required.

## II.A.4 Structural Period Variations

The natural period or frequency of a structure which determines the maximum response may not be constant as assumed for the idealized system in the response spectrum analysis. A slight variation in this period will tend to decrease the build-up of resonance and dynamic amplification factors and thus lower the maximum response.

## II.A.5 Exclusion of Non-Structural Elements

Non-structural elements are usually not included in the mathematical models for analysis. These elements tend to increase the resistance capacity of the structures and result in a higher calculated response.

II.B.1 Peak Widening of Floor Response Spectra

For the design of systems and components, floor or in-structure response spectra are generated at various locations in the structure. The peaks of these floor response spectra are widened to account for any adverse variations in the material properties or approximations in the modeling process that were used in the analysis.

II.B.2 Use of Envelop Response Spectra for Multiple-Supported Systems

When a system such as the steam pipe running from the top of the steam generator to the containment has multiple supports, the envelop spectra are used to generate seismic loads. Additional conservatisms inherent in the third major step of the process include the following: SLIDE (4).

A. Allowable Stresses

The determination of allowable stresses in building codes usually involves empirical test data and conservative judgment. Tests are conducted to failure to determine buckling capacity of columns, moment resistance of beams, shear resistance of concrete, optimum joint details, bolt loads, ... etc., and allowable values are selected from the data for use in design formulas. The selected values are not the mean failure values but values at or near the level where few, if any, failures occur.

B. Material Specifications

Material specifications call for minimum test values such as the yield strength in structural steel or reinforcing bars and the 28-day

compressive strength of concrete cylinders. The penalty for not meeting the tests and subsequent rejection can be severe, particularly if the material is already incorporated in the structure. The result is overconservatism in specifying material strengths to minimize the potential of failing the test and subsequent rejection. Essentially all rebars exhibit test results that are better than called for; the same is true for structural steel and concrete. In designing a concrete mix for a 3000 psi specified value at 28 days, the ingredients will be selected so that a large percentage of the test cylinders would fall above 3000 psi. The mean strength of the concrete may be 15 to 25% above the specified (design) value, at 28 days; it will be even more as the concrete gets much older and drier.

C. Designer's Habits

In many cases the materials actually provided exceed the required amount indicated by the design calculations. There is overall economy in duplicating member sizes rather than have too many variations in sizes or shapes, in using identical wall thicknesses or column sizes even though not required, etc. Furthermore, for standard shapes and sizes, usually the next higher size is selected.

D. Static Strength Vs. Dynamic Strength

The strength of structural materials tends to be greater under dynamic loading conditions such as those encountered during earthquakes than that under static loading conditions. This potential

increase in strength is, however, neglected in the seismic analysis and design.

E. Ductility to Failure

The greatest contribution to structural capacity in many buildings is that of the inelastic range beyond yield where ductility and capacity to absorb energy mean the difference between little or no damage and collapse.

F. Seismic Stress Not Always Important

In many members, elements, and joints, the seismic stress is usually a small part of the total stress that controls the design. This is generally the case in beams, girders and columns, and sometimes in bearing walls, but is not necessarily the case for seismic braces and shear walls. Thus, even an appreciable increase in seismic stress may only have a normal effect on the member element or joint.

G. Redundancy of Structural Elements

Redundancy of structural elements can greatly increase capacity by transmitting a local overstress along to other elements which in turn can redistribute their overstress along to others. This effect does work and absorbs energy. It also provides a reserve capacity that would often justify greater allowable stresses. In the process, the natural period may increase and thus further limit dynamic amplification. Damping may also increase.

CONCLUSION - SLIDE (5)

PROBABILISTIC CONSIDERATIONS

Assessment of the overall probability of significant failure in a nuclear power plant as may be caused by an earthquake, involves the quantification of the probability of occurrence of a chain of several events.

There are a number of ways in which such several events in this chain can be categorized. One simplified way of expressing the overall probability of failure $P_f$ is:

SLIDE (6)

The events in this chain were considered in formulating the several steps involved in setting earthquake analysis methods and design criteria for nuclear plants. Items 1 and 2 on the right side of the equation essentially deal with the probability of occurrence of a certain magnitude earthquake and items 4 and 5 deal with the probability of significant failure in the plant. Item 3 has mixed aspects of both the earthquake occurrence and structural behavior or response to the earthquake.

There are uncertainties involved with each element of this chain, and in general conservatisms are introduced at each step to cover these uncertainties as I have discussed earlier.

For a complete assessment of the overall probability of failure, it is necessary to define the probability of all of these individual uncer-

tainties, including the degree of dependency or correlation between the various events considered in the chain.

It should also be understood that the conditional events shown in the Equation are themselves made up of a series of uncertainties. A complete and reliable analysis of this type is not yet available. The NRC staff, however, recognizes the need for such an analysis and we are at present in the process of formulating a long-term research program to achieve this objective.

The NRC staff has in the past identified as a desirable safety objective for a large population of reactors that the probability of an accident with radioactive releases that would significantly exceed the 10 CFR Part 100 guidelines from one accident source should be of the order of $10^{-7}$ per reactor-year or less. This objective was primarily set for application in postulated accidents where the staff was of the opinion that it is possible to quantify or at least bound the probabilities (e.g., in the ATWS case and in considering potential aircraft crashes), but it is emphasized that this number was not intended for use in evaluating seismic design and related risk.

In the case of seismic risk assessment, the staff believes that a realistic quantitative definition of various probabilistic parameters is still beyond the reach of the current state-of-the-art. Therefore, the use of a deterministic and conservative approach to ensure seismic design adequacy of safety related structures and systems is believed

to be more appropriate at this time.

We do recognize that such data addressing failure probabilities of nuclear facilities subject to earthquake loads is available in the literature. However, it should be noted that because of the many broad assumptions and engineering judgments that were inherently involved in the development of such probabilistic approaches, a direct use of the conclusions reached as a basis for licensing decisions pertaining to Diablo Canyon or to any other plant's seismic design adequacy is not acceptable to the NRC staff. The conclusions obtained by these probabilistic studies, however, do provide an independent means for at least assessing the adequacy of the current seismic design criteria.

## AREAS OF CONSERVATISMS

I. CONSERVATISM IN THE SELECTION OF THE DESIGN EVENT

II. CONSERVATISMS IN THE SEISMIC ANALYSIS PROCESS

A. Conservatisms for Structures, Systems and Equipment

B. Additional Conservatisms for Systems and Equipment.

III. CONSERVATISMS IN THE STRUCTURAL AND MECHANICAL DESIGN (RESISTANCE)

(2).

I. CONSERVATISM IN THE SELECTION OF THE DESIGN EVENT

A. Selection of a Low Probability Extreme Event

B. Wide Band Ground Response Spectra

C. Conservative Amplification Factors

D. Enveloping Synthetic Time Histories

(3)

## II. CONSERVATISMS IN THE SEISMIC ANALYSIS PROCESS

### A. Conservatisms for Structures, Systems and Equipment

1. Elastic Dynamic Analysis
2. Damping Values
3. Load Combinations
4. Structural Period Variations
5. Exclusion of Non-Structural Elements

### B. Conservatisms for Systems and Equipment

1. Peak Widening of Floor Response Spectra
2. Use of Envelop Response Spectra for System with Multiple Supports
3. System Redundancy

(4)

## III. CONSERVATISMS IN THE STRUCTURAL AND MECHANICAL DESIGN (RESISTANCE)

A. Allowable Stresses

B. Materials Specifications

C. Designer's Habits

D. Static Strength Vs. Dynamic Strength

E. Ductility to Failure

F. Seismic Stress not Always Important

G. Redundancy of Structural Elements

## CONSERVATISMS IN OVERESTIMATION OF THE SEISMIC LOAD

(1) Selection of a Low Probability Extreme Event
(2) Wide Band Ground Response Spectra
(3) Conservative Amplification Factors
(4) Enveloping Synthetic Time Histories
(5) Elastic Dynamic Analysis
(6) Damping Values
(7) Load Combinations
(8) Structural Period Variations
(9) Exclusion of Non-Structural Elements
(10) Peak Widening of Floor Response Spectra
(11) Use of Envelop Response Spectra for System with Multiple Supports
(12) System Redundancy

## CONSERVATISMS IN UNDERESTIMATION OF THE SEISMIC RESISTANCE

(13) Allowable Stresses
(14) Materials Specifications
(15) Designer's Habits
(16) Static Strength Vs. Dynamic Strength
(17) Ductility to Failure
(18) Seismic Stress not Always Important
(19) Redundancy of Structural Elements

Assuming an arbitrary average contribution of 3-5% to the margin of safety for all of these conservatisms, one would obtain approximately 60-100% additional resistance.

CONCLUSION: A plant designed for .40g may be good for .65g to 1g acceleration.

(6)

FAILURE = Radioactive release in excess of 10 CFR 100 limits

OVERALL PROBABILITY OF "FAILURE" --- $P_f$

THEN:

$$P_f = P_{oe} \times P_{me} \times P_{sa} \times P_{os} \times P_{sf}$$

where:

$P_{oe}$: probability of Occurrence of Earthquake

$P_{me}$: probability of the Earthquake having a certain Magnitude at the site

$P_{sa}$: probability of achieving a certain level of Spectral Acceleration

$P_{os}$: probability of there being an Over Stress given the Spectral Acceleration

$P_{sf}$: probability of a significant System Failure given an Over Stress [illegible] [illegible]