September 10, 1976

SUPPLEMENT NO. 5

TO THE

SAFETY EVALUATION REPORT

BY THE

OFFICE OF NUCLEAR REACTOR REGULATION

U. S. NUCLEAR REGULATORY COMMISSION

IN THE MATTER OF

PACIFIC GAS AND ELECTRIC COMPANY

DIABLO CANYON NUCLEAR POWER STATION, UNITS 1 AND 2

DOCKET NOS. 50-275 AND 50-323

8/22

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APPENDICES

1.0 INTRODUCTION

The Commission's Safety Evaluation Report in the matter of Pacific Gas and Electric Company's application for operating licenses for the Diablo Canyon Nuclear Power Station, Units 1 and 2, was issued on October 16, 1974. In the Safety Evaluation Report it was stated that supplemental reports would be issued to update the Safety Evaluation Report in those areas where the staff's evaluation had not been completed. Supplement Nos. 1, 2, 3 and 4 to the Safety Evaluation Report, issued on January 31, 1975, May 9, 1975, September 18, 1975 and May 11, 1976 respectively, documented the resolution of several outstanding items, and summarized the status of the remaining outstanding items.

The purpose of this supplement is to further update the Safety Evaluation Report by providing the staff's evaluation of certain matters which were not resolved when Supplement No. 4 was issued. Each of the following sections of this supplement is numbered the same as the corresponding section of the Safety Evaluation Report that is being updated. A summary of the remaining outstanding issues, which will be addressed in future supplements to the Safety Evaluation Report, is presented in Section 22.0 of this supplement.

Appendix A to this supplement is a continuation of the chronology of the Nuclear Regulatory Commission staff's principal actions with respect to radiological matters related to the processing of the application. Appendix B is a bibliography. Appendix C is a report by our consultant, Dr. Nathan M. Newmark, dated September 3, 1976.

2.0 SITE CHARACTERISTICS

2.4 Hydrology

In Sections 2.4.2, 2.4.3 and 2.4.5 of Supplement No. 1 to the Safety Evaluation Report, we stated that we had completed our evaluation of tsunamis due to distant generators. However, additional information was required from the applicant concerning the effects of nearshore generated tsunamis on safety related portions of the intake structure. Subsequent information and additional analyses concerning nearshore generated tsunamis were provided by the applicant in Amendments 27, 29, and 31 to the FSAR. Our evaluation of this information is described below.

Tsunami Analysis

There are two nearby offshore faults potentially capable of generating tsunamis that could result in appreciable runup at the intake structure. The Santa Lucia Bank fault is located 29 miles from the site and the Hosgri fault is about 3-1/2 miles offshore. The applicant analyzed tsunami runup from both faults using a two step approach. Tsunami height at the coast, outside the intake cove, was determined for various ground motion configurations at both faults, using a finite difference numerical model described in Reference 1 of Appendix B to this supplement. We consider the model used to be, at present, state of-the-art in the modeling of the generation and propagation of tsunamis. The resulting wave history outside the intake cove was used with a harbor response model to determine runup at the intake structure, which is flood protected to 30 feet, Mean Lower Low Water datum.

For purposes of analysis, vertical ground displacement was taken to be 6.6 feet for the Santa Lucia Bank fault and 7.3 feet for the Hosgri fault. Based on our review of the nature of these faults, we have concluded that both values are conservative.

The design water height is considered to be the maximum tsunami occurring coincident with high tide and short period, one-year storm waves. The greatest wave height at the intake structure resulted from a postulated large radius, circular area of ground displacement on the Hosgri fault. Including the effect of permanent ground displacement, the maximum wave height was calculated to be 9.2 feet. The tidal height (astronomical and meteorological) is 6.3 feet, Mean Lower Low Water datum. These values result in a total elevation of 15.5 feet, Mean Lower Low Water datum, not including the storm wave effects.

Storm wave height was taken as 18 feet outside the breakwater. The applicant calculated wave transmission by overtopping of the breakwater to estimate wave heights within the cove and calculated runup on the intake structure for waves with a period

of 10 seconds. The applicant determined the maximum runup at the intake structure to be 27.7 feet, Mean Lower Low Water datum.

We consider the method used by the applicant to be sufficiently conservative to more than compensate for the use of 10-second period waves rather than the more conservative 15-second period than can occur during the one year storm. As a confirmation of this, we calculated the maximum runup at 21 feet Mean Lower Low Water datum using Wiegel's model results presented in Appendix 2.4B to the FSAR and using the more conservative 15-second period design wave.

Effects of Breakwater Damage

The possibility of the breakwater being damaged or destroyed by the same seismic event that generates the tsunami was also investigated. At our request the applicant submitted, in Amendment 31 to the FSAR, additional information on the design and construction of the breakwater and the characteristics of the geologic material underlying and supporting the breakwater.

The assumption used in the calculation of runup is that the side slopes of the breakwater would slump to a 1 on 4 slope (vertical to horizontal) which corresponds to about 17 feet of slumping at the crest. Using conservative strength characteristics for the breakwater materials, a conservative method of analysis of the slope displacement and site motions for a magnitude 7.5 earthquake on the Hosgri fault, we determined that the crest of the breakwater would slump no more than 5 feet. Accordingly, we have concluded that the assumption used in the calculations is conservative.

The applicant, using a conservative method, calculated the maximum runup on the intake structure for waves with a 15-second period as 29.4 feet, Mean Lower Low Water datum with the breakwater slumped to a 1 on 4 slope. We independently calculated the maximum runup for this case and concluded that the applicant's estimate was conservative and acceptable.

Conclusions

Based on our review we conclude that: (1) a locally generated tsunami will not exceed the design basis flood level of 30 feet, Mean Lower Low Water datum at the intake structure; and (2) the breakwater, even if damaged by the initiating event, affords sufficient protection to the cove to prevent flooding of the intake structure in the event that the maximum tsunami occurred coincident with high tide and short period, one-year storm waves.

With regard to low water levels, any event large enough to generate a substantial tsunami will also lower the sea floor in the cove because the sense of displacement on the two nearshore faults is downward on the side closest to shore. This will have a compensating effect in relation to low water level. Therefore, we have concluded that the most adverse combination of tsunami drawdown, low tide and storm waves will

not result in water levels at or below the design basis low water level of minus 17.4 feet, Mean Lower Low Water datum.

Accordingly, based on our evaluations as described above and in Supplement No. 1 to the Safety Evaluation Report, we have concluded that the design features for protection against the effects of tsunamis are acceptable. We consider this matter resolved.

2.5 Geology, Seismology and Foundation Engineering

Effective Acceleration

In Supplement No. 4 to the Safety Evaluation Report we stated that our consultant, Dr. Nathan M. Newmark of N. M. Newmark Consulting Engineering Services, had recommended and we had accepted an effective horizontal ground acceleration of 0.75 g. This value was to be used in developing response spectra which define the effective ground motion at the plant site from a postulated magnitude 7.5 earthquake on the sector of the Hosgri fault nearest the site. These spectra would then be used in reevaluating the plant's seismic design capabilities. We also stated that we would provide further discussion of this matter and a report from Dr. Newmark in a future supplement to the Safety Evaluation Report.

Dr. Newmark's report, which is presented in Appendix C to this supplement, discusses the effective horizontal ground acceleration (0.75 g) and presents the rationale that it is based upon. As stated in Supplement No. 4 to the Safety Evaluation Report, we have accepted Dr. Newmark's recommendation. Further discussion concerning reevaluation of the facilities is presented in Section 3.7 of this supplement.

3.0 DESIGN CRITERIA - STRUCTURES, COMPONENTS, EQUIPMENT AND SYSTEMS

3.7 Seismic Design

In Supplement No. 4 to the Safety Evaluation Report we stated that we had requested that the applicant evaluate the plant's capability to withstand a magnitude 7.5 earthquake on the Hosgri fault with an effective horizontal ground acceleration of 0.75 g for the purpose of developing design response spectra. We also outlined the procedures that we believed would be appropriate for this evaluation.

Since then, the applicant has developed design response spectra based on the recommendations of the applicant's consultant, Dr. John A. Blume of John A. Blume & Associates, Engineers. These spectra were documented in Amendment 45 to the FSAR. At meetings on August 11, 1976, and September 7, 1976, the applicant provided additional information concerning these spectra.

In addition, our consultant, Dr. Nathan M. Newmark, has independently developed design response spectra. Dr. Newmark's recommended spectra and their bases are discussed in Appendix C to this supplement.

We have reviewed the applicant's proposals concerning design response spectra as well as our own consultant's recommendation. Based on our review and in accordance with the advice of our consultant, we informed the applicant, at a meeting on September 7, 1976, that response spectra conforming to the following criteria would be acceptable:

(1) The response spectra recommended by our consultant, as described in Appendix C to this supplement, are acceptable for use in reevaluating the plant's seismic design capabilities. As discussed in Appendix C to this supplement, the spectra would be modified to account for building size effects. In doing so, the response spectrum for each structure would be determined according to the following values for transit time parameter (Tau):

(a) Containment or Intake Structure: Tau = 0.04

(b) Auxiliary Building: Tau = 0.052

(c) Turbine Building: Tau = 0.08

(d) Smaller Structures: Tau = 0

Further reduction of these spectra to account for non-linear effects would not be made. However, if the applicant proposes to take credit for this in certain specific areas, then we will consider it on an individual case basis.

(2) Alternately, the response spectra proposed by the applicant are generally acceptable for use in reevaluating the plant's seismic design capabilities. They would be used in the following manner:

(a) As a starting point, for small structures where Tau = 0, the applicant's proposed response spectrum would be used. However, this spectrum would be raised as necessary in certain frequency ranges so that it does not fall below the spectrum for small structures recommended by our consultant in Appendix C to this supplement.

(b) For larger structures, the spectrum described in (a) above would be reduced to account for building size in accordance with the methods proposed by the applicant using the following values for Tau:

(i) Containment or Intake Structure: Tau = 0.04

(ii) Auxiliary Building: Tau = 0.052

(iii) Turbine Building: Tau = 0.08

(c) The spectra discussed above would be reduced to account for non-linear behavior using a ductility ratio up to 1.3.

(d) The calculated value for zero period acceleration would not be rounded to a lower value, but used as calculated.

(e) For each structure, the spectrum obtained by these means would be adjusted in certain frequency ranges as necessary so that it does not fall below the spectrum recommended by our consultant in Appendix C to this supplement.

The applicant is currently proceeding with the reevaluation on this basis. We will present our evaluation of the reanalysis in a future supplement to the Safety Evaluation Report.

6.0 ENGINEERED SAFETY FEATURES

6.3 Emergency Core Cooling System (ECCS)

In Supplement No. 4 to the Safety Evaluation Report we found the ECCS analyses to be acceptable, subject to satisfactory resolution of certain matters concerning single failures of motor operated valves and the time period before switching recirculation modes after a loss-of-coolant accident. These matters are now resolved and our evaluation is presented below.

Long-Term Boric Acid Concentration Buildup

In Supplement No. 4 to the Safety Evaluation Report we stated that we would require that the switchover time from cold leg to simultaneous hot leg and cold leg injection be changed to 19.5 hours (the applicant had proposed 24 hours). This switchover would effectively terminate the concentration of boric acid in the reactor vessel during the long term recirculation mode following a loss-of-coolant accident. The shorter time would allow a margin of four weight percent below the solubility limit for cold leg breaks to account for uncertainties in predicting the boric acid concentration, providing greater assurance that solid boric acid will not deposit in the reactor vessel. In Amendment 43 to the FSAR, the applicant committed to initiate hot leg recirculation about 19.5 hours after the beginning of recirculation, thus providing simultaneous hot and cold leg recirculation. Since this procedure would terminate the concentration of boric acid before the solubility limit is reached, we have concluded that this commitment is acceptable. We consider this matter resolved.

Single Failure Criterion

In Supplement No. 4 to the Safety Evaluation Report we stated that we would require modifications for certain motor operated valves in order to meet the single failure criterion and we described modifications that would be acceptable. In Amendment 43 to the FSAR the applicant incorporated acceptable modifications as described in Supplement No. 4 by disconnecting electrical power from the motor operators involved and, where required, providing the capability for restoring power from the control room. This ensures that single electrical failures will not result in spurious actuation of critical valves. Since these modifications assure that no single failure is worse than the single failures which were assumed in the ECCS analyses, we have concluded that they are acceptable.

In Supplement No. 4 to the Safety Evaluation Report we stated that we would review the detailed means of disconnecting power from motor operated ECCS valves and, where appropriate, the means of restoring power from the control room. In a letter dated

July 21, 1976 the applicant submitted detailed electrical drawings which describe these provisions. We have reviewed these provisions and determined that they are in conformance with our position which is documented in Branch Technical Position EICSB 18, "Application of the Single Failure Criterion to Manually-Controlled Electrically-Operated Valves," contained in Appendix 7A to our Standard Review Plan. Accordingly, we have concluded that the detailed means of removing power from and restoring power to motor operated ECCS valves are acceptable.

Conclusions

The items which were outstanding in our review of ECCS performance when Supplement No. 4 to the Safety Evaluation Report was issued have now been resolved, as discussed above. However, Westinghouse Electric Company recently informed us, in a telephone conversation on August 5, 1976 and a meeting on August 9, 1976, of two matters which will affect the results of our review. A brief discussion of these matters is presented below.

Measurements made in an operating plant and calculations have indicated that the temperature of the reactor coolant in the upper head of the reactor vessel may be higher than the temperature which was assumed in the ECCS analysis. Using the higher temperature in the ECCS analysis is expected to increase the calculated peak clad temperature which could change our conclusions concerning ECCS performance. Based on our experience in evaluating this effect for other Westinghouse reactors, we believe that the Diablo Canyon ECCS performance can be shown to be acceptable for full power operation, perhaps with reduced allowable peaking factors. However, we will present our evaluation of this matter in a future supplement to the Safety Evaluation Report.

Recent tests by Westinghouse have indicated that the effect of rod bow on departure from nucleate boiling (DNB) may be greater than previously reported in the Westinghouse Topical Reports WCAP-8176 (Proprietary) and WCAP-8323 (Non-Proprietary), "Effect of Bowed Rod on DNB." This could change our conclusions concerning rod bow as stated in Sections 4.4 and 6.3 of Supplement No. 4 to the Safety Evaluation Report. Based on our experience in evaluating this effect for other Westinghouse reactors we believe that the Diablo Canyon fuel design and ECCS performance can be shown to be acceptable for full power operation with appropriate restrictions on margin to DNB. However, we will present our evaluation of this matter in a future supplement to the Safety Evaluation Report.

22.0 CONCLUSIONS

In Section 22 of Supplement No. 4 to the Safety Evaluation Report, we stated that several items were still outstanding, and that favorable resolution of these items would be required before operating licenses for Diablo Canyon Units 1 and 2 could be issued. Items which currently remain outstanding are summarized below.

1. An evaluation of the plant's capability to withstand an earthquake of magnitude 7.5 on the Hosgri fault (Section 3.7 of Supplement No. 4).

2. An evaluation of the environmental and seismic qualification of Category I electrical, instrumentation and control equipment (Sections 3.10 and 7.8 of the Safety Evaluation Report).

3. An evaluation of the consequences of Anticipated Transients Without Scram (Section 7.2.5 of Supplement No. 4).

4. An evaluation of the ability of the liquid and gaseous radioactive waste management systems to meet Appendix I to 10 CFR Part 50 (Section 11.0 of Supplement No. 4).

5. An evaluation of the effects of postulated pipe breaks outside containment (Section 3.6 of the Safety Evaluation Report).

6. An evaluation of the plant's tornado missile protection (Section 3.5 of Supplement No. 3 to the Safety Evaluation Report).

7. An evaluation of the matters concerning fuel rod bowing and the temperature of the reactor coolant in the upper reactor head (Section 6.3 of this supplement).

Subject to favorable resolution of the outstanding matters described above, the conclusions as stated in Section 22 of the Safety Evaluation Report remain unchanged.

APPENDIX A

CONTINUATION OF THE CHRONOLOGY OF THE RADIOLOGICAL REVIEW

September 18, 1975	Supplement No. 3 to the Safety Evaluation Report issued.
March 26, 1976	Letter from applicant submitting velocity data related to the Western Geophysical Company offshore survey records submitted previously and requesting proprietary treatment for the velocity data.
April 21, 1976	Letter from applicant providing schedule for submitting information requested February 25, 1976 concerning Appendix I to 10 CFR Part 50.
May 6, 1976	Letter to applicant providing draft technical specifications for use in preparing proposed technical specifications with regard to Appendix I to 10 CFR Part 50.
May 11, 1976	Supplement No. 4 to the Safety Evaluation Report issued.
May 12, 1976	Submittal of Amendment 42 including (1) information on electrical, instrumentation and control equipment (equipment qualification) in partial response to request for information dated August 17, 1976, and (2) miscellaneous changes.
May 18, 1976	Letter to applicant requesting additional information regarding containment structural integrity test.
May 21, 1976	ACRS Subcommittee meeting in San Luis Obispo, California.
May 25, 1976	Letter from applicant requesting extension of Unit 1 construction permit.
June 4, 1976	Letter from applicant submitting document entitled "Information Required for Compliance with 10 CFR 50, Appendix I, for Diablo Canyon, Units 1 and 2."

June 10, 1976	Letter to applicant requesting a schedule for providing an evaluation of the adequacy of the reactor pressure vessel supports.
June 10, 1976	Meeting with applicant to discuss seismic design reevaluation.
June 11, 1976	Letter to applicant requesting schedule for submittal of information concerning outstanding items.
June 11, 1976	Letter to ACRS fowarding Dr. Newmark's draft report concerning Diablo Canyon seismic design bases.
June 21, 1976	Letter from applicant providing schedule requested June 11 for submitting information concerning outstanding items.
June 23, 1976	Meeting with applicant to discuss seismic design reevaluation.
June 24, 1976	Submittal of Amendment 43 including (1) information on electrical, instrumentation and control equipment (equipment qualification) in partial response to request for information dated August 17, 1975 and (2) miscellaneous changes.
June 25, 1976 and June 26, 1976	ACRS Subcommittee Meeting in San Luis Obispo, California.
July 2, 1976	Letter to applicant requesting schedule for submitting additional information concerning ATWS.
July 6, 1976	Letter to applicant stating that velocity data submitted March 26, 1976 would be withheld from public disclosure pursuant to 10 CFR 2.790.
July 6, 1976	Construction Permit for Unit 2 extended until January 1, 1977.
July 8, 1976	ACRS Full Committee Meeting in Washington, D. C.
July 19, 1976	Letter to applicant requesting additional information concerning the material submitted June 4, 1976 regarding Appendix I to 10 CFR Part 50.

July 20, 1976	Letter from applicant responding to a request for information concerning reactor vessel supports dated June 10, 1976.
July 21, 1976	Letter from applicant providing detailed information concerning design modifications to satisfy the single failure criterion for electrically operated ECCS valves.
July 29, 1976	Letter from applicant providing information requested on May 18, 1976 concerning the Unit 1 containment structural integrity test.
July 29, 1976	Submittal of Amendment 44 including information related to Appendix I to 10 CFR Part 50.
July 29, 1976	Submittal of Amendment 45 including information concerning reevaluation of seismic design capabilities.
July 30, 1976	Letter from applicant submitting document related to Appendix I to 10 CFR Part 50 entitled "Draft Model Technical Specifications for Pressurized Water Reactors, Docket Numbers 50-275, 50-323, Including Additions to Supply Requested Information."
August 11, 1976	Meeting with applicant concerning reevaluation of seismic design capabilities.
August 27, 1976	Meeting with applicant concerning reevaluation of seismic design capabilities.
September 7, 1976	Meeting with applicant concerning reevaluation of seismic design capabilities.

APPENDIX B

BIBLIOGRAPHY

(Documents referenced in or used to prepare Supplement No. 5 to the Safety Evaluation Report for the Diablo Canyon Nuclear Power Station, Units 1 and 2, are listed below. This list of documents is in addition to those previously listed in the bibliographies for the Safety Evaluation Report and Supplement Nos. 1 and 4 to the Safety Evaluation Report.)

Hydrology

1. Hwang, Li-San, H. L. Butler and D. J. Divoky, Tsunami Model: generation and open sea characteristics, Bull. Seism. Soc. Am., 62, 1972.

APPENDIX C

A RATIONALE FOR DEVELOPMENT OF 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

3 September 1976

A RATIONALE FOR DEVELOPMENT OF DESIGN SPECTRA
FOR DIABLO CANYON REACTOR FACILITY

by

Nathan M. Newmark

I. INTRODUCTION AND SUMMARY

This report describes a basis for development of 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 may be 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 most intense near field earthquake record available is that for Pacoima Dam in the San Fernando earthquake of 9 February 1971. For that record, there are shown in Figs. la and b comparisons of the response spectra computed from the two horizontal records, with the design spectra developed in WASH-1255 (Ref. 4) and NUREG-0003 (Ref. 10). The computed response spectra for 2 percent damping are the upper curves in Fig. 1, identified by the symbol $\tau = 0$.

The design spectra for 2 percent damping, based on an effective acceleration of 0.75g, with the appropriate amplification factors (summarized below for convenience) are shown by the upper solid lines in Fig. 1. The standard velocity and displacement values used for drawing these are $0.75 \times 48 = 36$ in/sec, and $0.75 \times 36 = 27$ in., respectively. With amplification factors of 3.66 for acceleration, 2.92 for velocity, and 2.42 for displacement, for 2 percent damping, and one standard deviation above the median, the amplified values plotted are 2.75g, 105 in/sec, and 65 in. The plotted values generally envelope (with a substantial margin, on the whole) the computed spectra.

This is the most direct indication that the "effective" peak acceleration for the Pacoima Dam record is not in fact the measured value of 1.20g, but actually does not exceed 0.75g. Therefore this is taken as the effective peak acceleration for design.

As a further comparison there are shown in Figs. la and b modified design spectra for 2 percent damping based on appropriately amplified values of the ground velocity and ground displacement expected in the near field in rock, of 24 in/sec and 8 in, respectively, as summarized in Table 1, and as explained below. These spectra are shown by the second highest polygonal solid lines in the figures. Similar spectra for 5 percent damping are shown in Figs. 18 and 19.

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 2a and 2b 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 Scanlan (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 or 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, for several acceleration levels. These values are plotted in Figs. 16 and 17 in terms of the usual type of design spectrum considered earlier in this report. The spectra shown in Fig. 16 are for the free field, for an acceleration level of 0.75g, the plant complex for an acceleration level of 0.6g, and for the reactor building for an acceleration level of 0.5g.

The acceleration levels for these response spectra are based on the results in Figs. 2a and 2b, taking into account the dimensions of the structure considered, and the wave transit time over the area of the structure.

The transit time parameter τ probably is more closely associated with the averaging of accelerations over the area of the structure than it is with an actual wave transit time. In other words, accelerations at any instant of time vary both in the direction considered and in the transverse direction. This is taken account of by use of a value of transit time determined by the "effective" width (the square root of the area, in general) of the foundation divided by the wave velocity, which is generally considerably less than the shear wave velocity of the foundation medium.

The reduction factor R used to obtain the ground acceleration design value for the foundation for the Diablo Canyon site conditions are based on the general level of reduction shown in Figs. 2a and 2b, or in Figs. 18 and 19 (for 5 percent damping), and are taken as follows:

$$A_f = A_o \cdot R \qquad (1)$$

where A_f = acceleration for foundation

A_o = acceleration for free field

and

$$R = 1 - 5\tau \quad (2)$$

but not less than 0.67.

The lower limit on R is kept purposely high for adequate conservatism in the application of this concept, in view of the small amount of data on which the concept is based.

With the use of this relationship, for the reactor building, with its diameter of 160 ft and a wave velocity of 4000 fps, one finds a transit time of 0.04 sec, for which R = 0.8 and A_f = 0.6g. If one considers the entire plant structure as effectively tied together through the foundation, the effective width of about 480 ft gives a transit time of 0.12 sec, and the lower limit of R = 0.67 is applicable, with A_f = 0.5g. Small separate structures not close to the main complex should be designed for a higher spectrum, however, corresponding to the free field value of 0.75g.

Figure 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 0.4g 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 for 0.5g developed herein for frequencies higher than about 2 or 3 hertz, but may be somewhat low for lower frequency elements.

For other damping levels, spectra may be drawn using the amplification values in Table 3, taken from Refs. 4 and 11.

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				
	Acceleration, g				
	Small Structs.	Reactor Bldg.	Plant Complex	Vel, in/sec All	Displ, in All
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

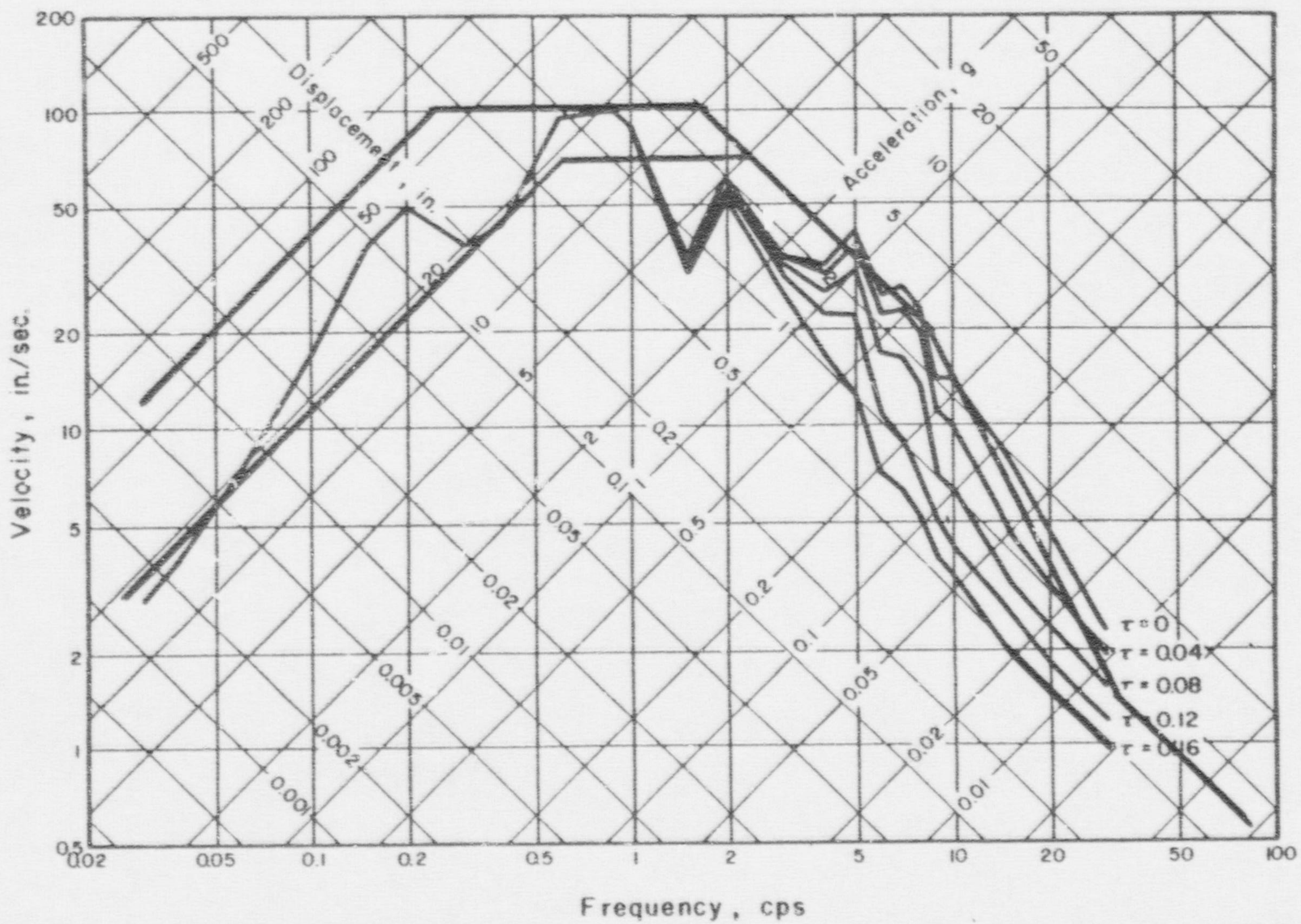

FIG. 1a PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S16E, 2 PERCENT DAMPING, τ = 0, 0.04, 0.08, 0.12, 0.16 sec. COMPARED WITH DESIGN SPECTRA

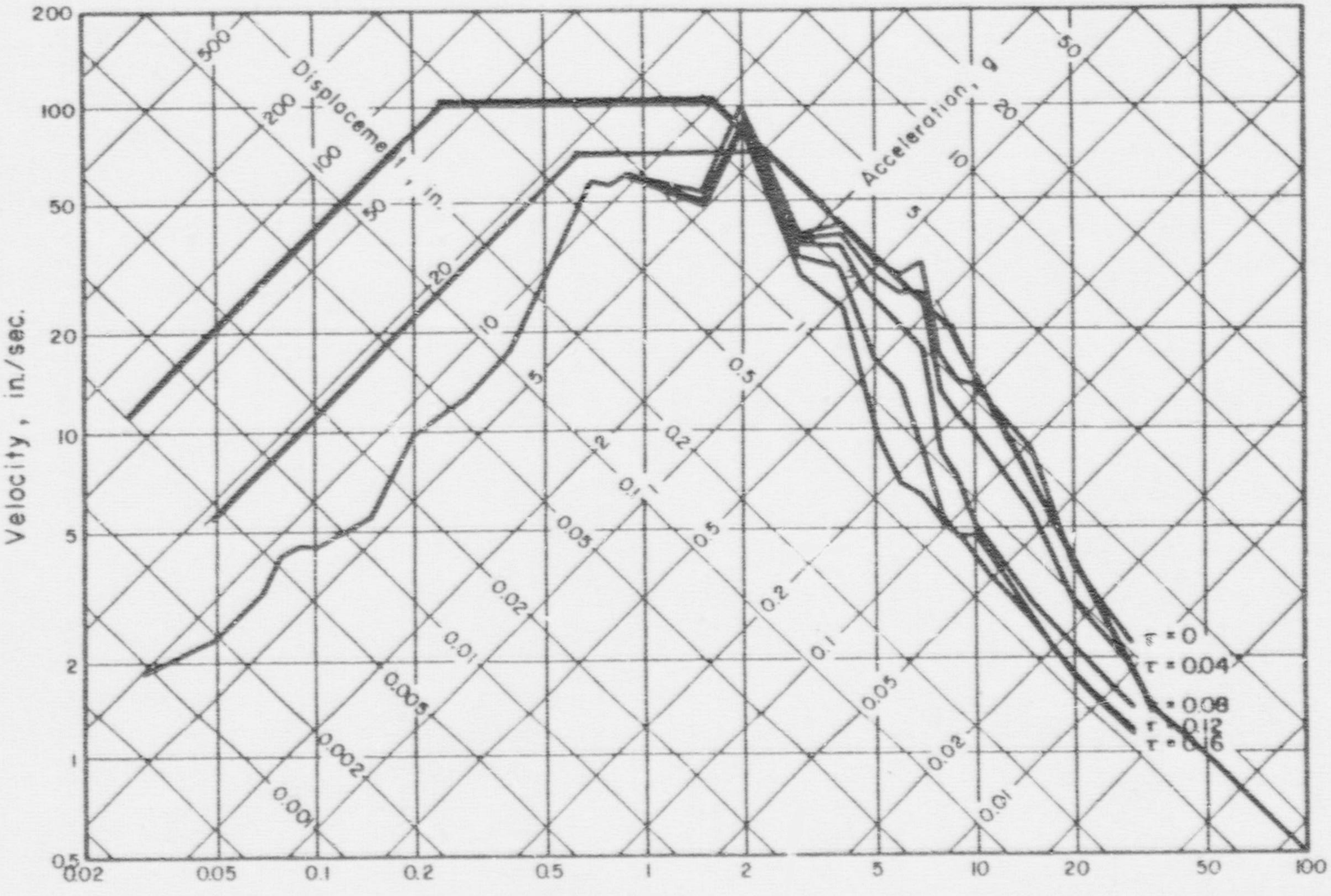

FIG. 1b PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S74W, 2 PERCENT DAMPING, τ = 0, 0.04, 0.08, 0.12, 0.16 sec. COMPARED WITH DESIGN SPECTRA

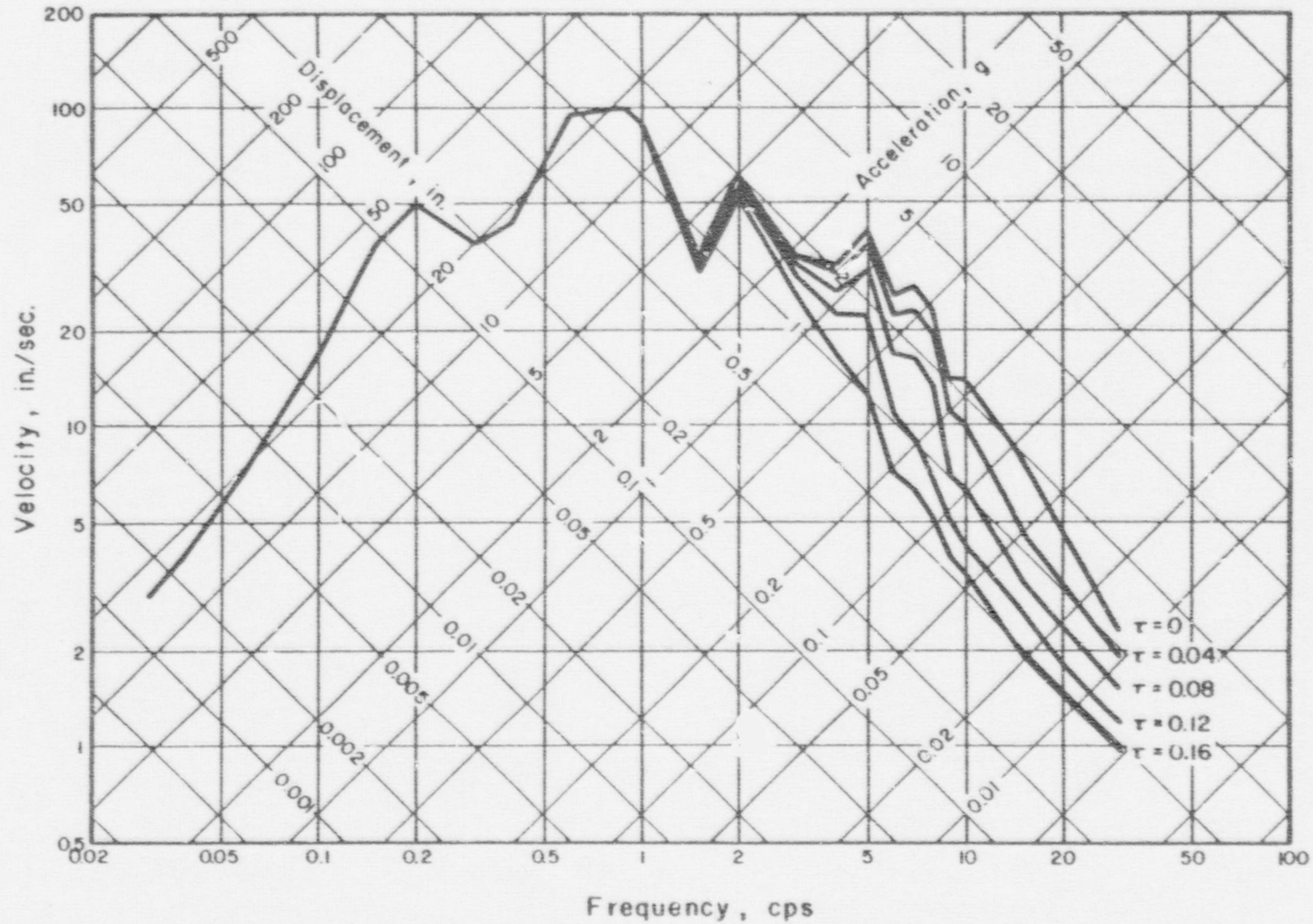

FIG. 2a PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S16E, 2 PERCENT DAMPING, τ = 0, 0.04, 0.08, 0.12, 0.16 sec.

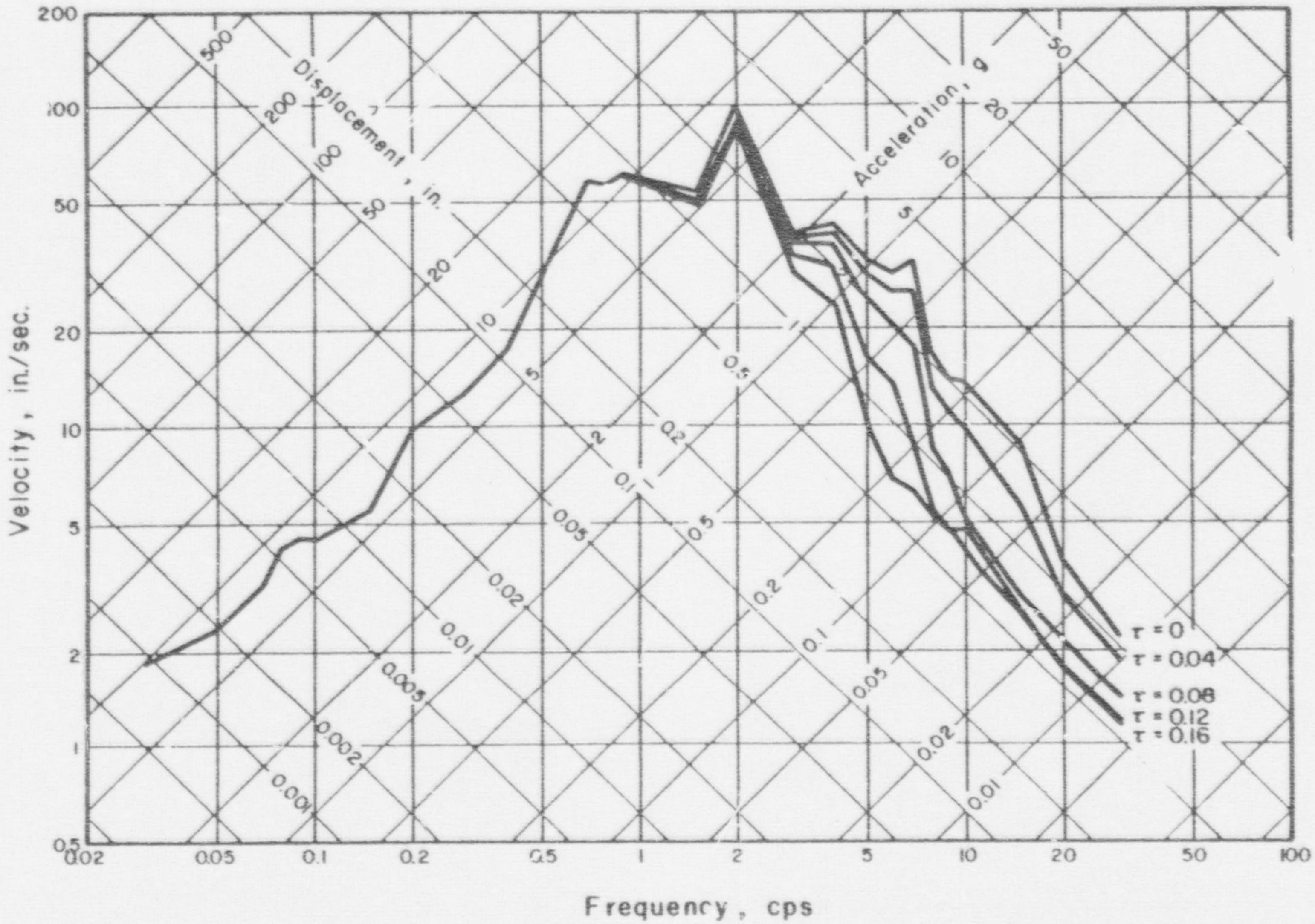

FIG. 2b PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S74W, 2 PERCENT DAMPING , τ = 0, 0.04 , 0.08 , 0.12 , 0.16 sec.

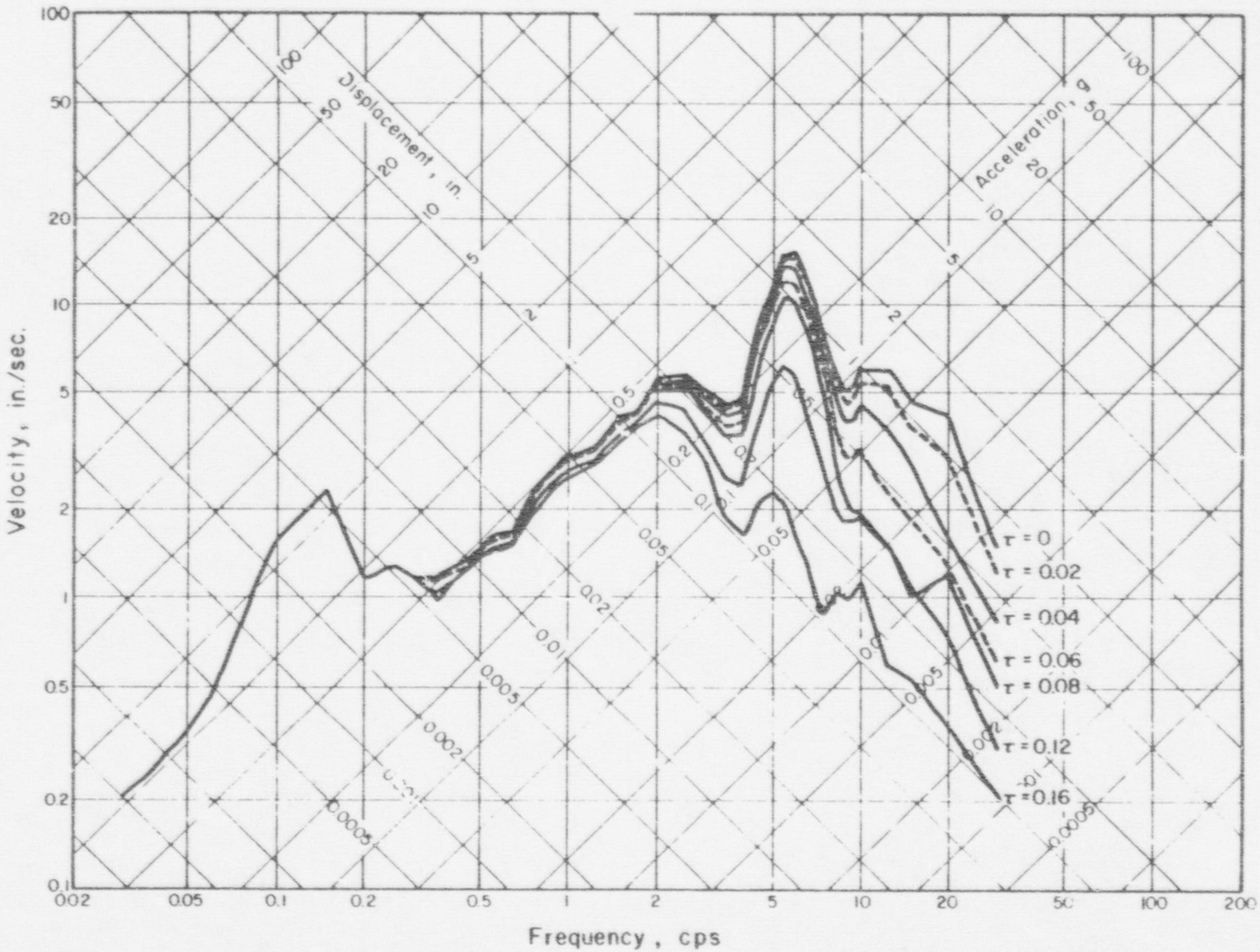

FIG. 3 ANCONA, ROCCA 6-14-72 GMT-NORTH $\tau = 0, 0.02, 0.04, 0.06, 0.08, 0.12, 0.16$
SPECTRUM COMPUTED USING 5.0 PERCENT CRITICAL DAMPING

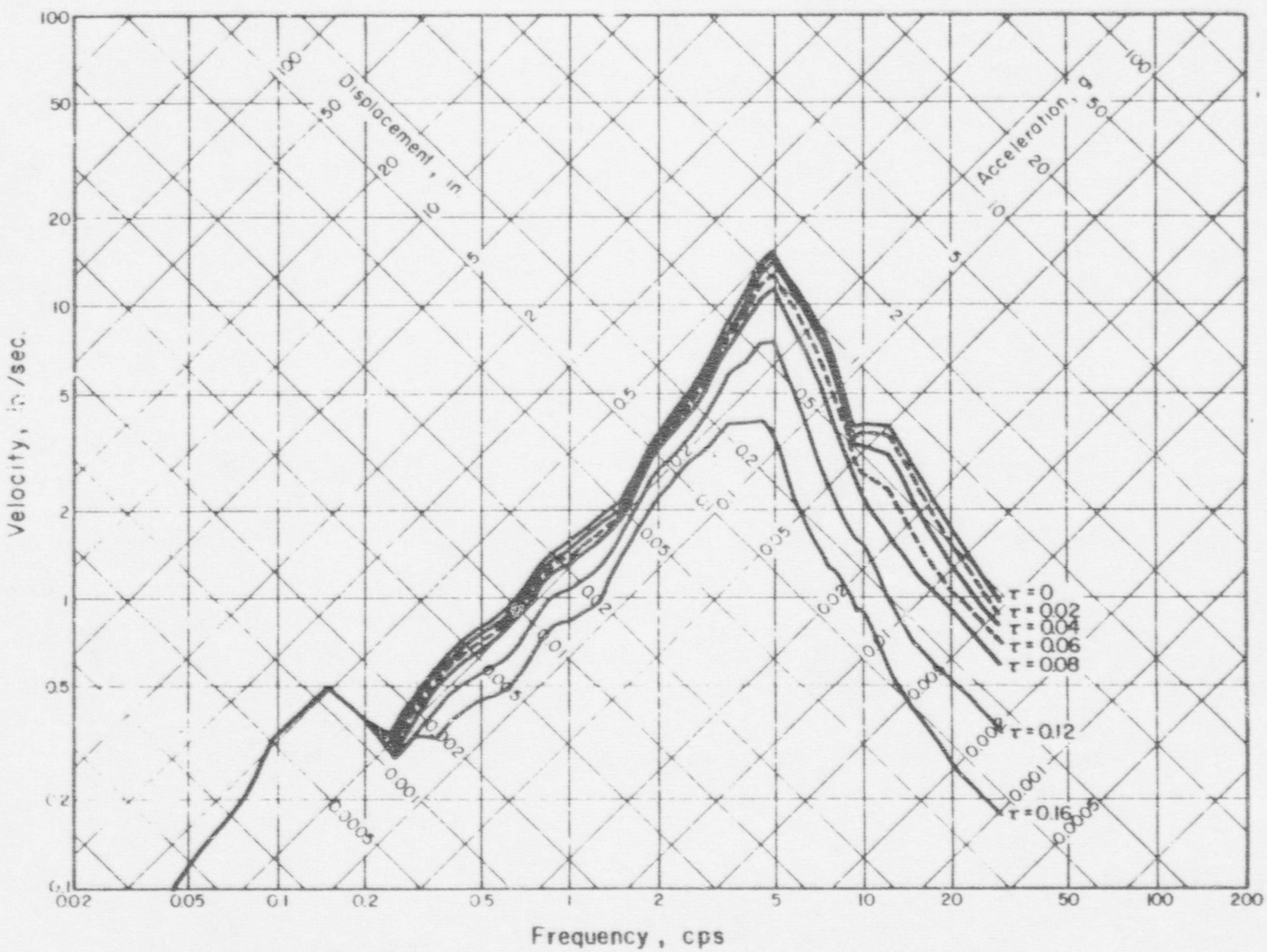

FIG.4 ANCONA, PALOMBINA 6-21-72 GMT-NS $\tau = 0, 0.02, 0.04, 0.06, 0.08, 0.12, 0.16$
SPECTRUM COMPUTED USING 5.0 PERCENT CRITICAL DAMPING

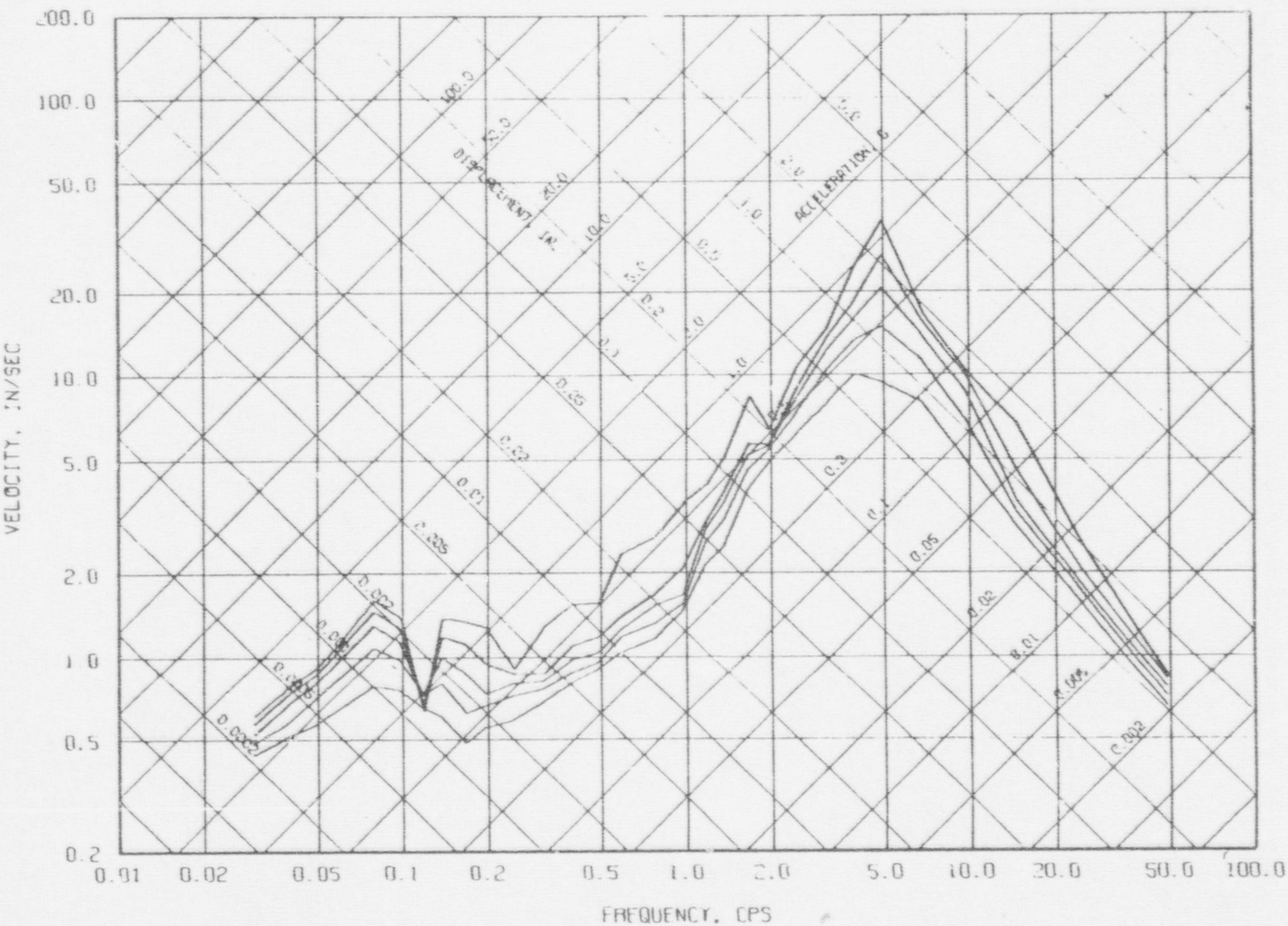

FIG. 5 RESPONSE SPECTRA FOR MELENDY RANCH BARN, 9/4/72 - N61E COMPONENT
0%, 2%, 5%, 10% AND 20% DAMPING (PARABOLIC BASE LINE ADJ. AND FINAL VEL. SET=0)

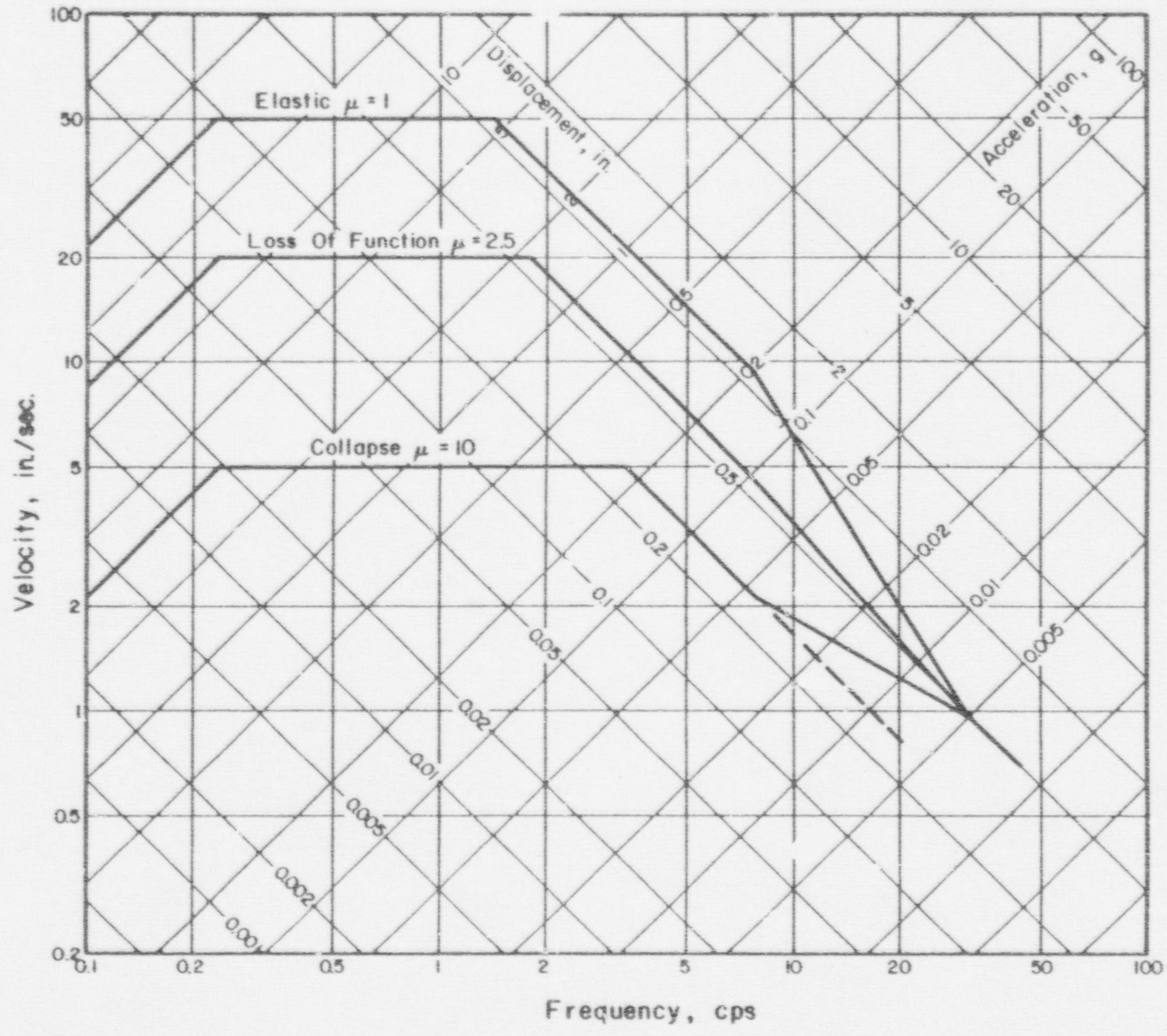

FIG. 6 INELASTIC DESIGN SPECTRAL REQUIREMENT FOR PEAK GROUND ACCELERATION OF 0.5 G, 7% DAMPING

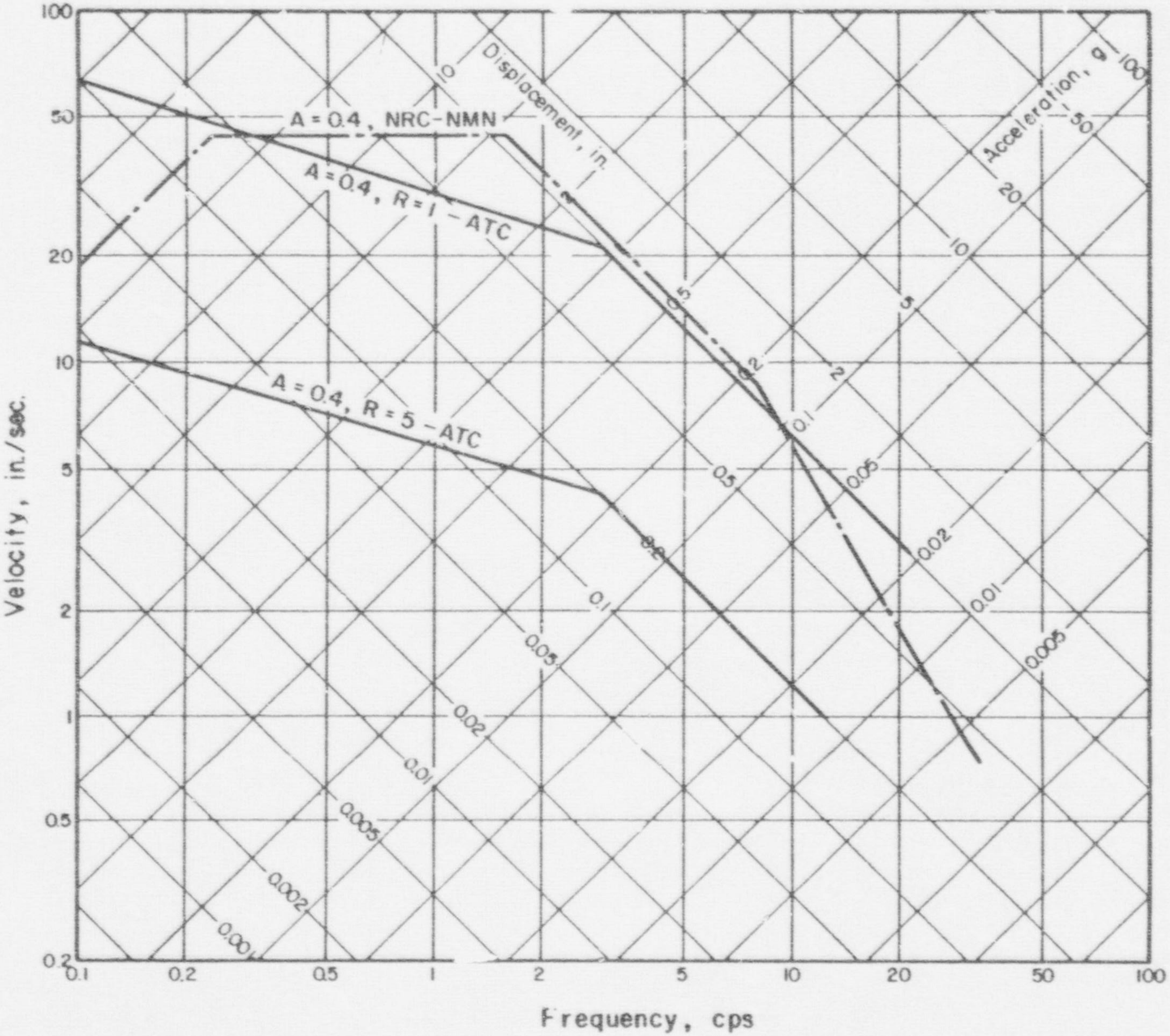

FIG. 7 COMPARISON OF ATC AND NRC DESIGN COEFFICIENTS, 5% DAMPING

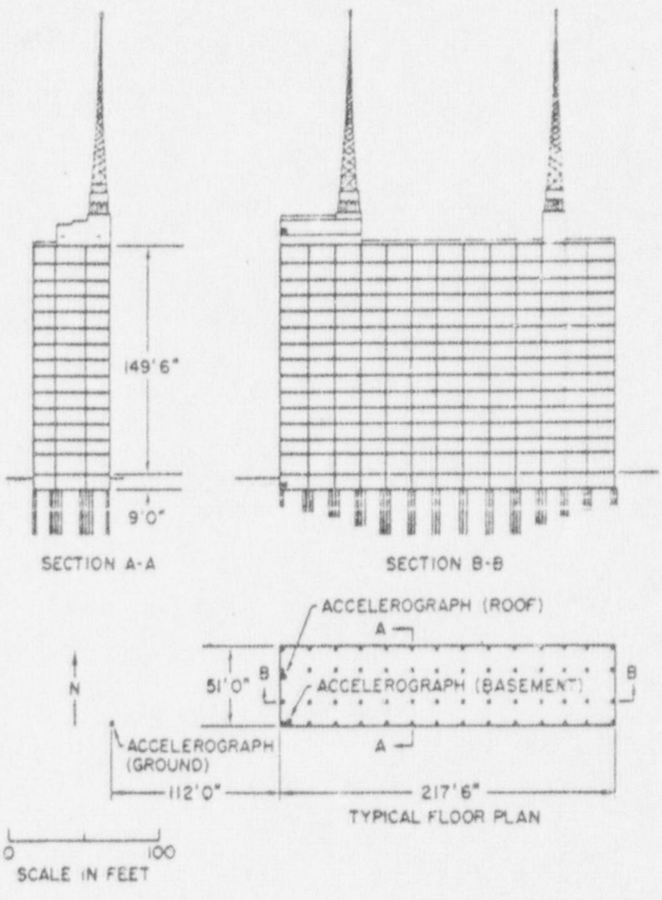

FIG. 8 PLAN AND ELEVATION OF BUILDING

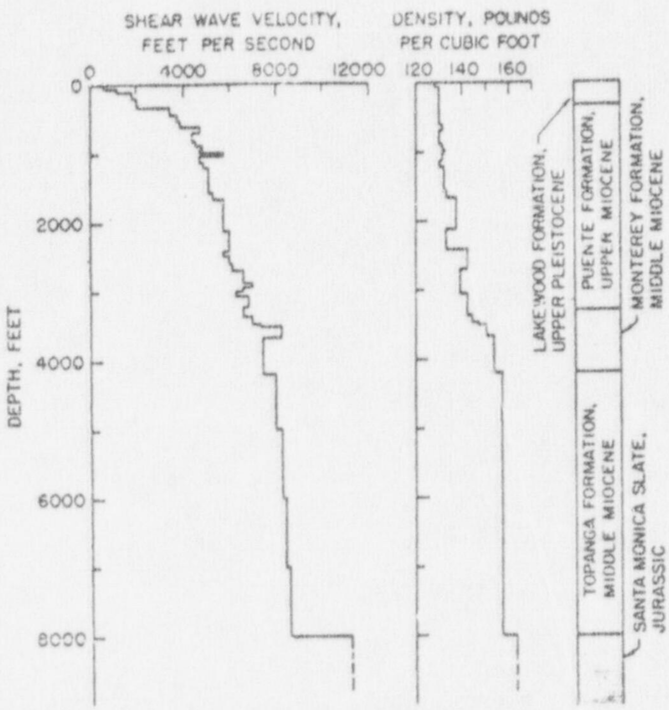

FIG. 9 Subsurface model.

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.

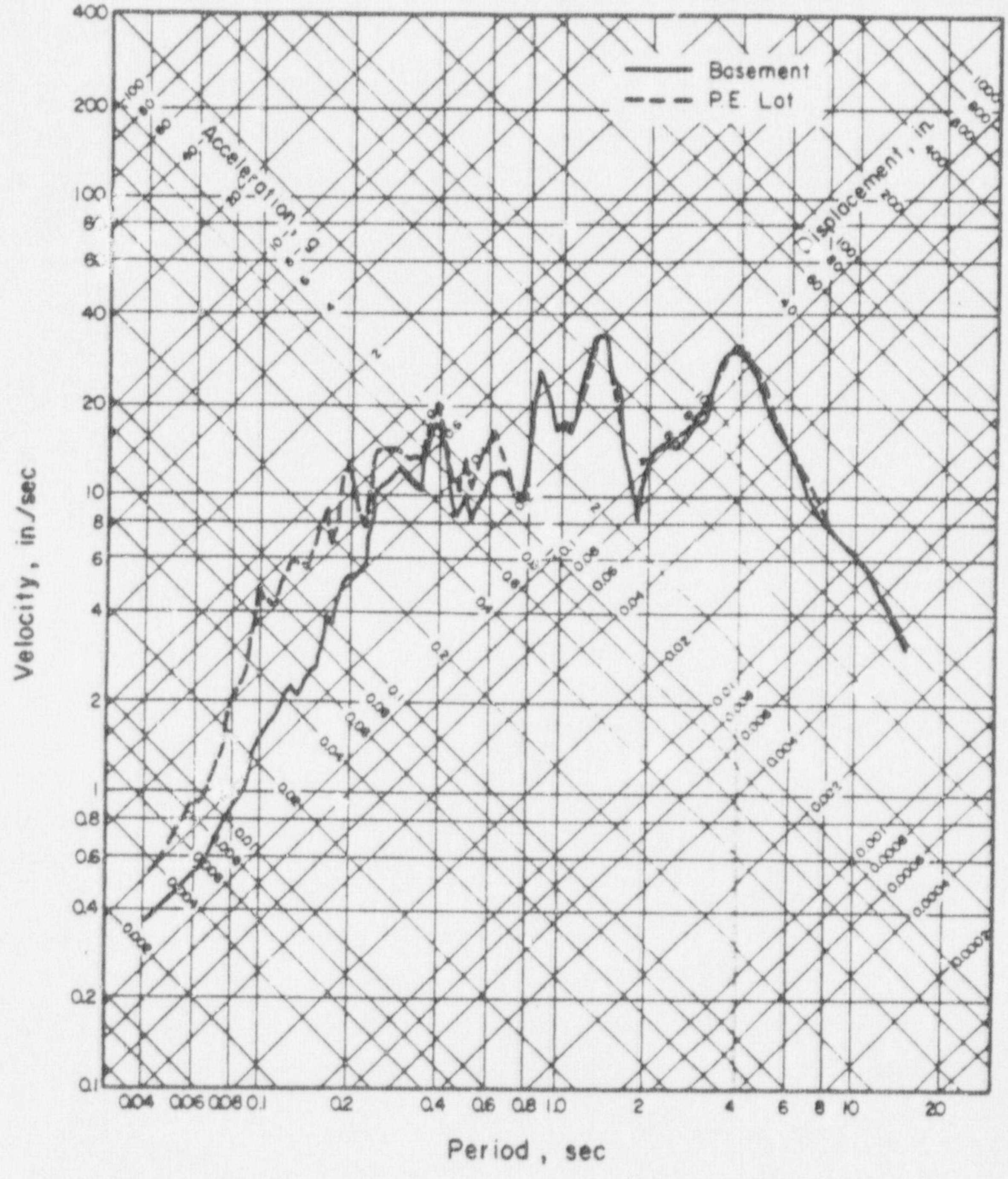

FIG. 10 SAN FERNANDO EARTHQUAKE, FEB. 9, 1971 - 0600 PST
HOLLYWOOD STORAGE BASEMENT AND P.E. LOT, COMPONENT
EAST, DAMPING VALUE 2% OF CRITICAL

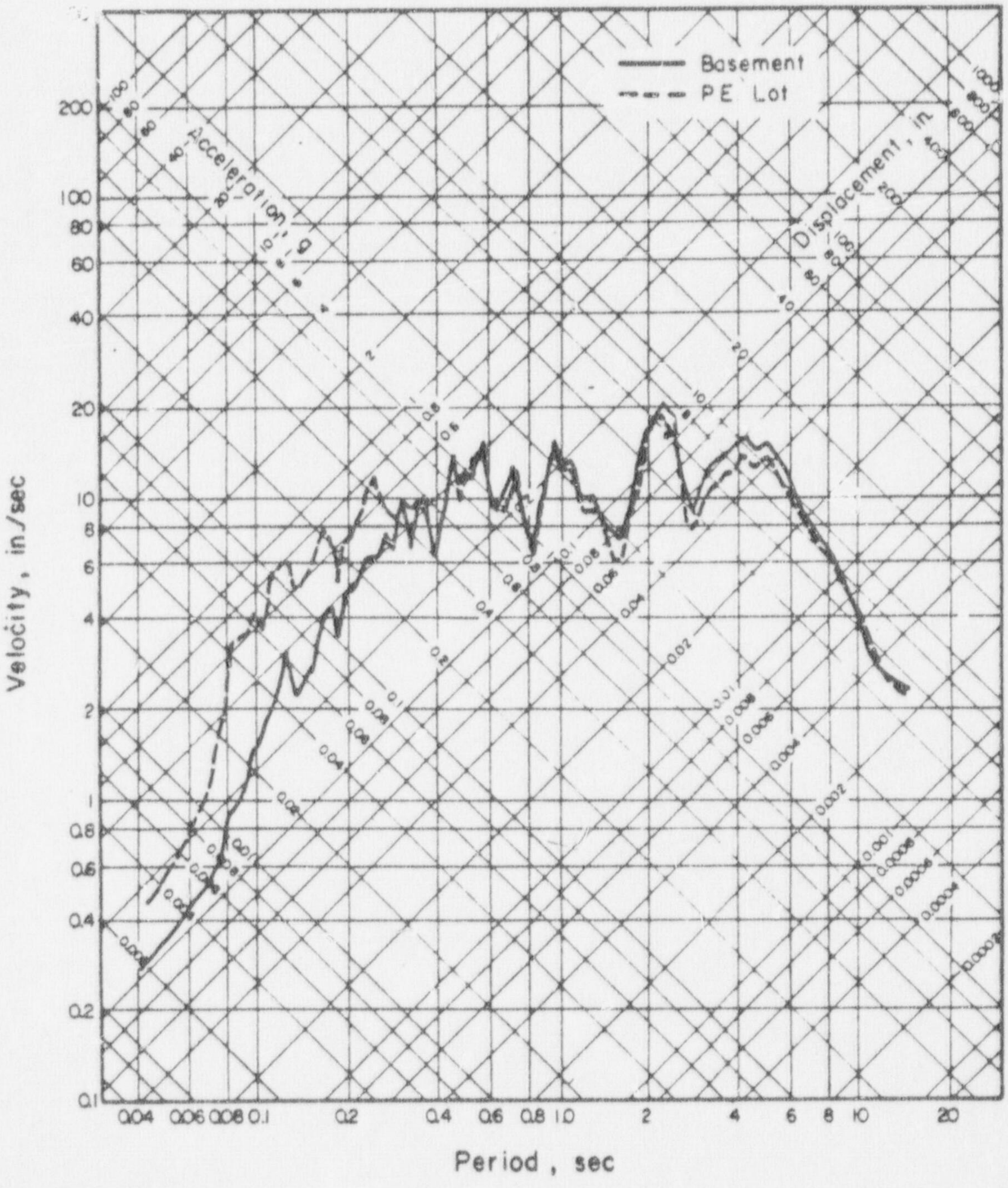

FIG.11 SAN FERNANDO EARTHQUAKE, FEB. 9, 1971 - 0600 PST
HOLLYWOOD STORAGE BASEMENT AND P.E. LOT, COMPONENT SOUTH, DAMPING VALUE 2% OF CRITICAL

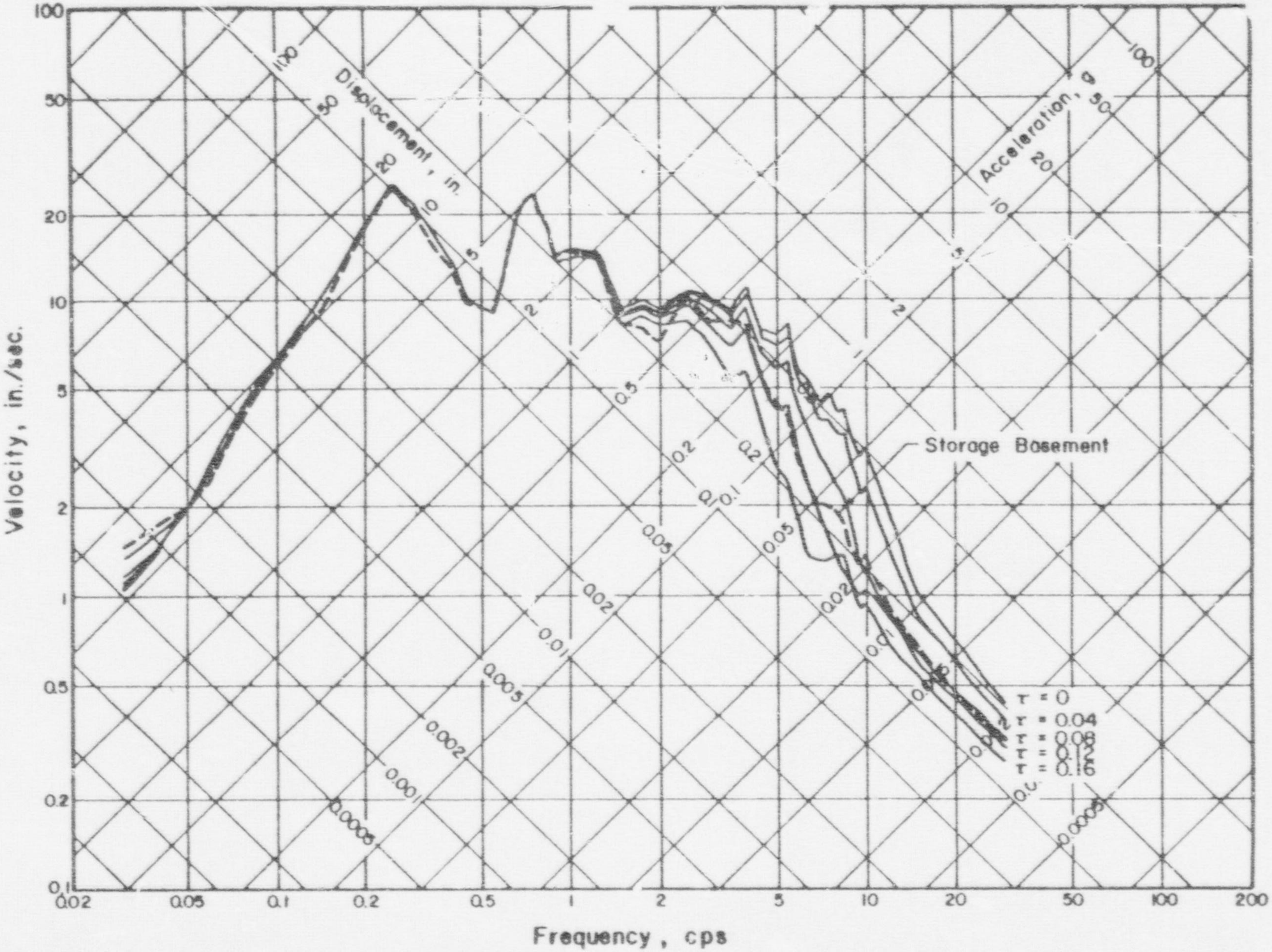

FIG. 12 HOLLYWOOD STORAGE P.E. LOT, SAN FERNANDO EARTHQUAKE FEB. 9, 1971, COMPONENT EAST, DAMPING 5 % OF CRITICAL, τ = 0, 0.04, 0.08, 0.12, AND 0.16 SEC.

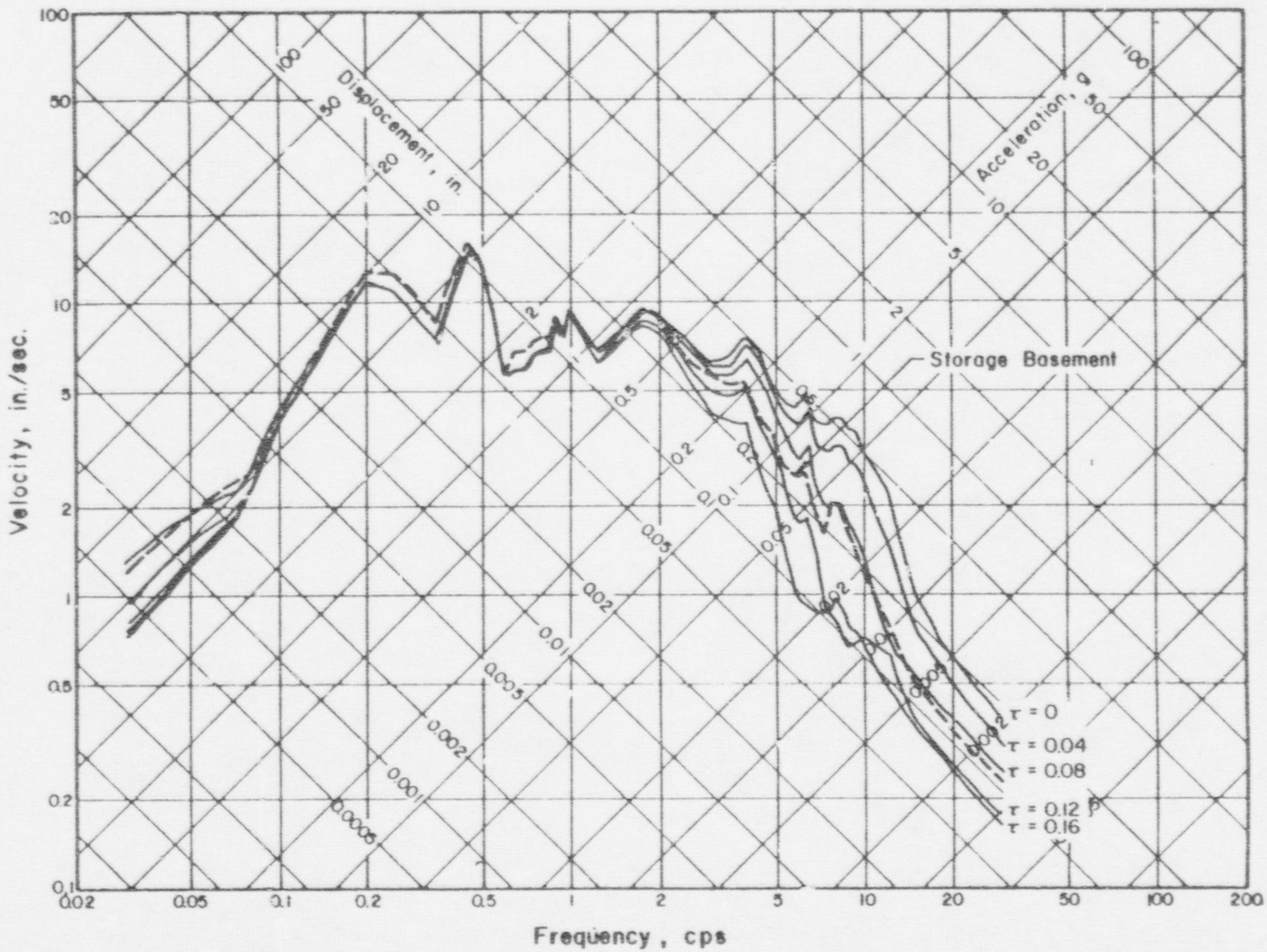

FIG.13 HOLLYWOOD STORAGE P.E. LOT, SAN FERNANDO EARTHQUAKE FEB. 9, 1971, COMPONENT SOUTH, DAMPING 5 % OF CRITICAL, τ = 0, 0.04, 0.08, 0.12, AND 0.16 SEC.

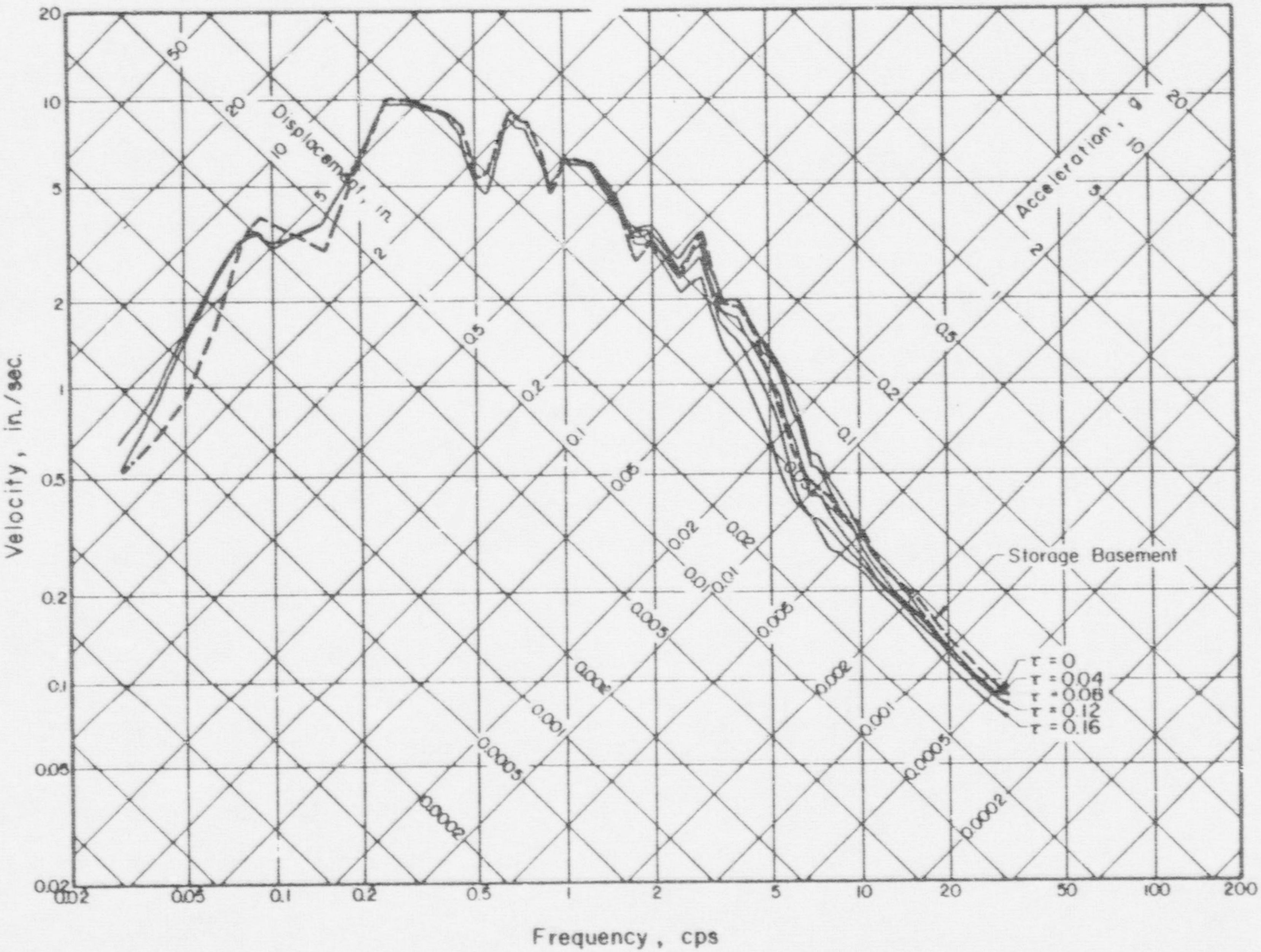

FIG. 14 HOLLYWOOD STORAGE P.E. LOT, KERN COUNTY EARTHQUAKE JULY 21, 1952, COMPONENT EAST, DAMPING 5 % OF CRITICAL, $\tau = 0, 0.04, 0.08, 0.12,$ AND 0.16 SEC.

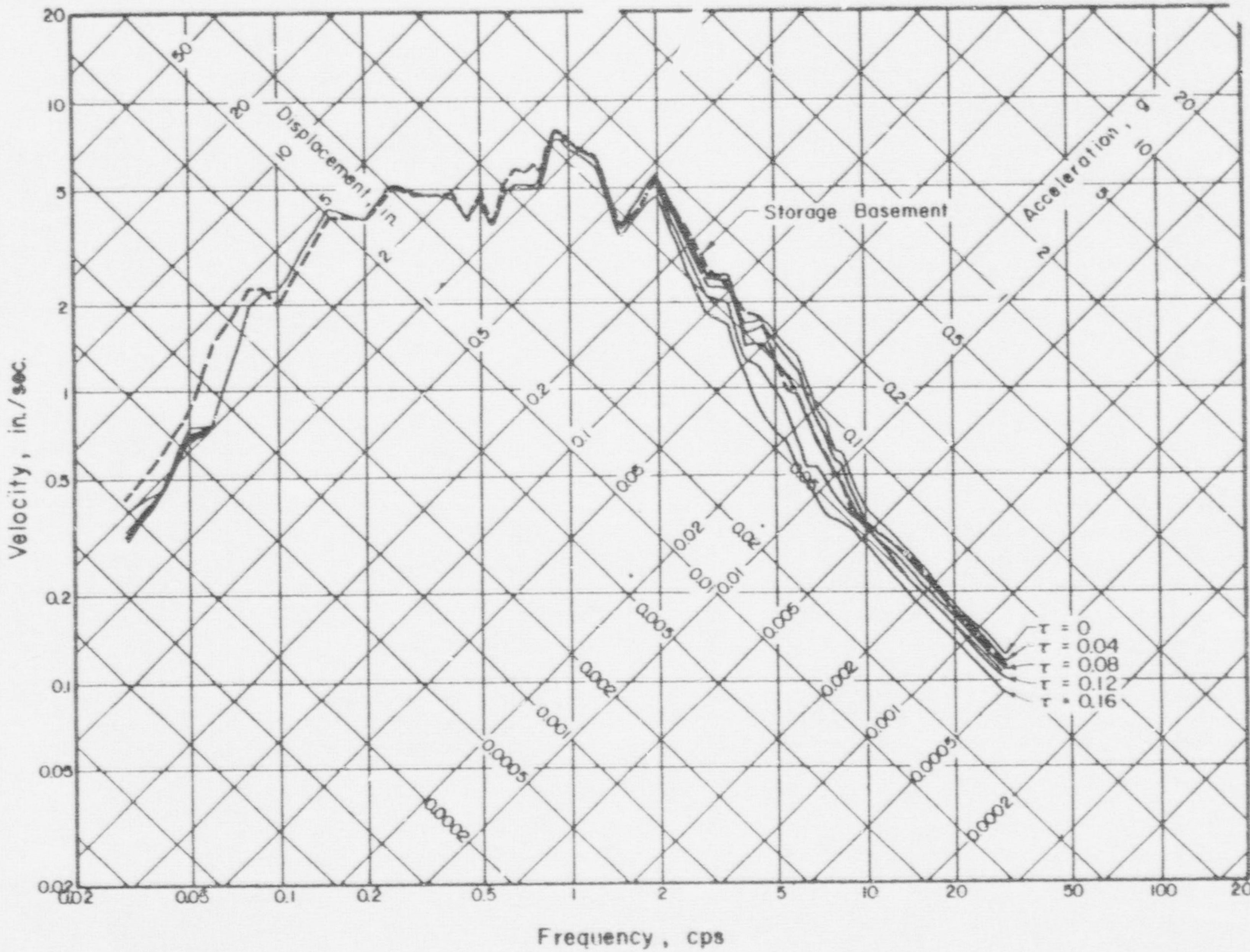

FIG.15 HOLLYWOOD STORAGE P.E. LOT, KERN COUNTY EARTHQUAKE JULY 21, 1952, COMPONENT SOUTH, DAMPING 5 % OF CRITICAL, τ = 0, 0.04, 0.08, 0.12, AND 0.16 SEC.

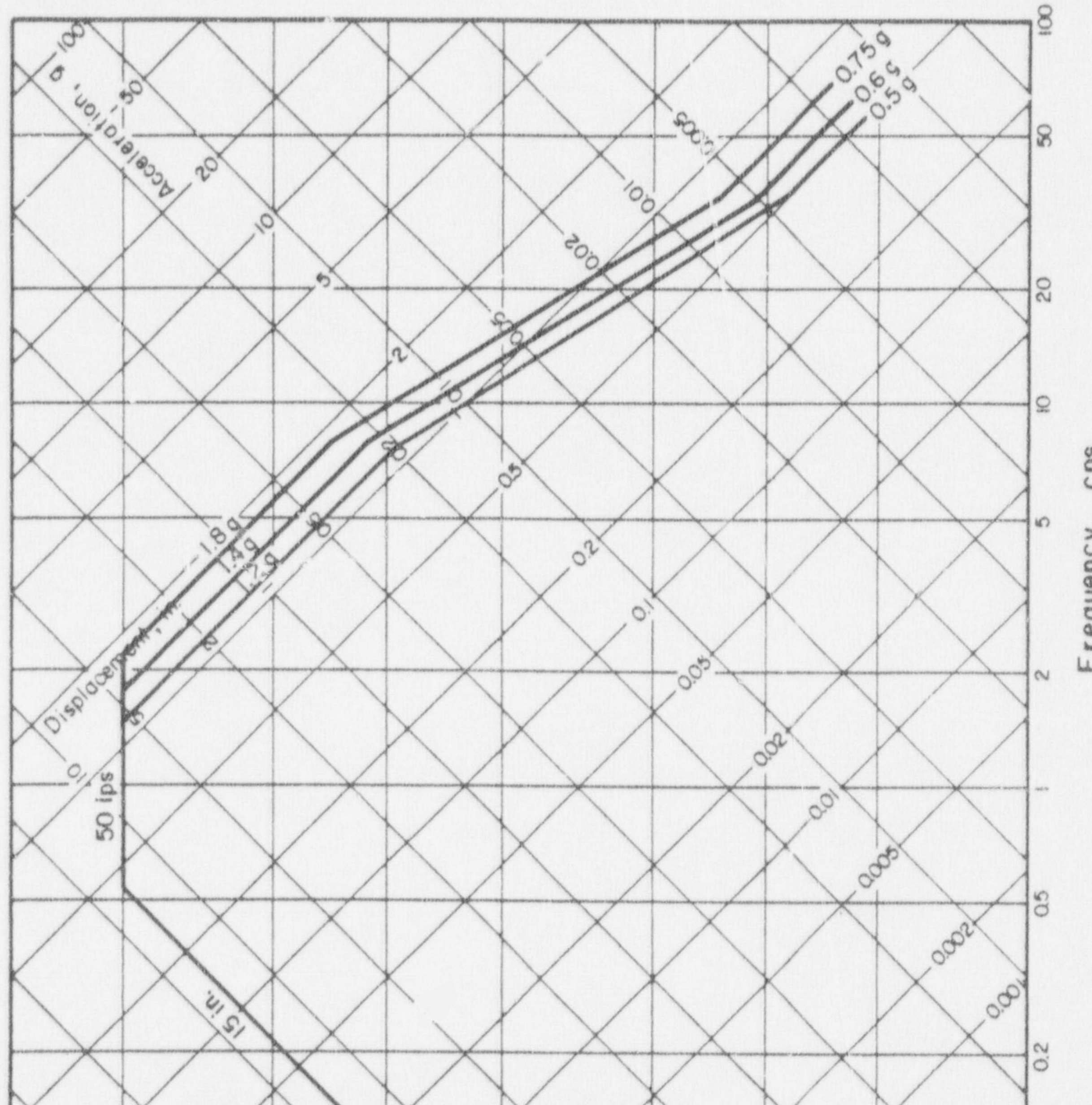

FIG. 16 RECOMMENDED "DESIGN" SPECTRA FOR PLANT

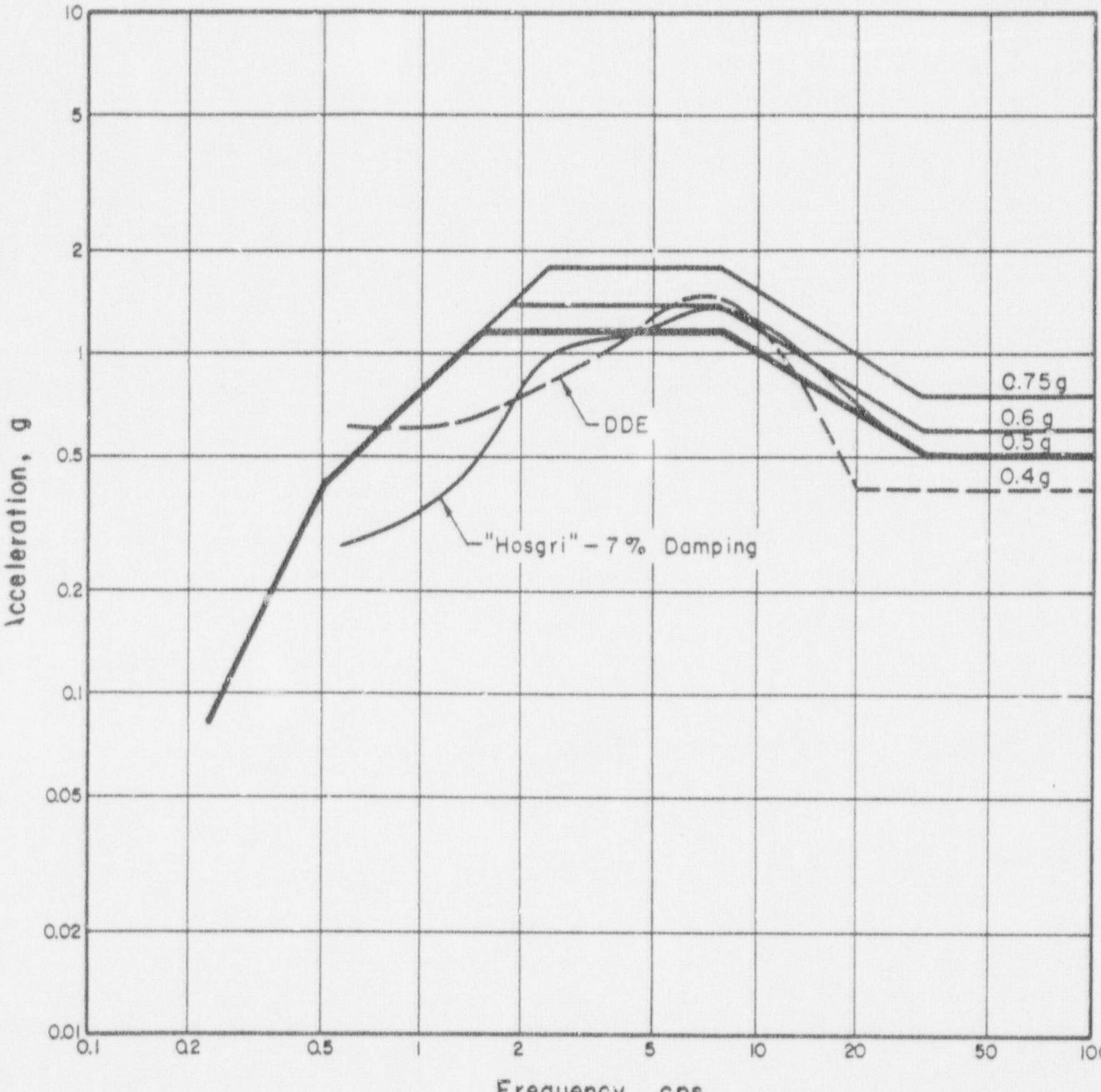

FIG. 17 RECOMMENDED "DESIGN" SPECTRA, 7% DAMPING, COMPARED WITH "HOSGRI" AND DDE SPECTRA

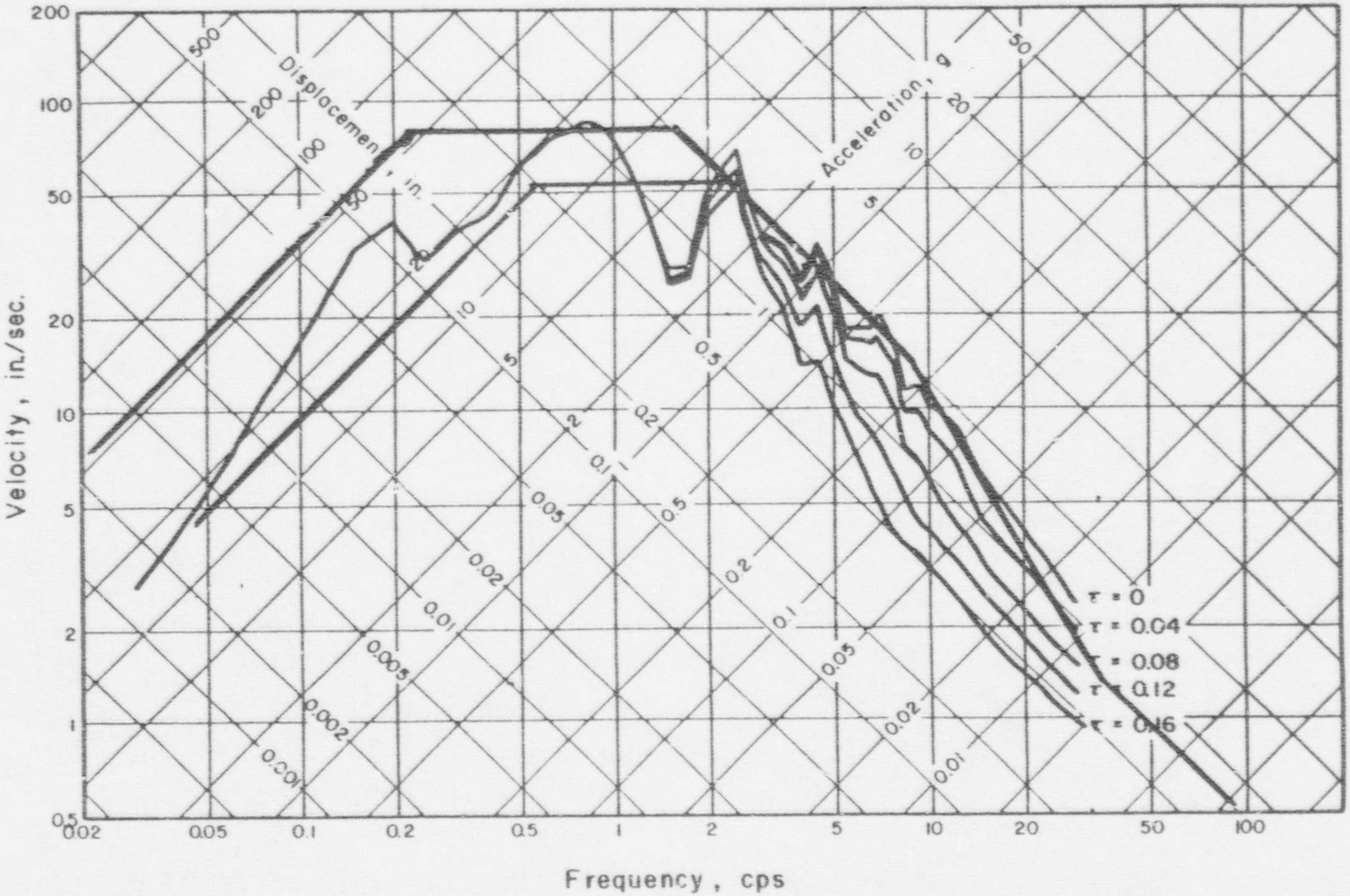

FIG. 18 PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S16E, DAMPING 5% OF CRITICAL, $\tau = 0, 0.04, 0.08, 0.12, 0.16$ SEC. COMPARED WITH DESIGN SPECTRA

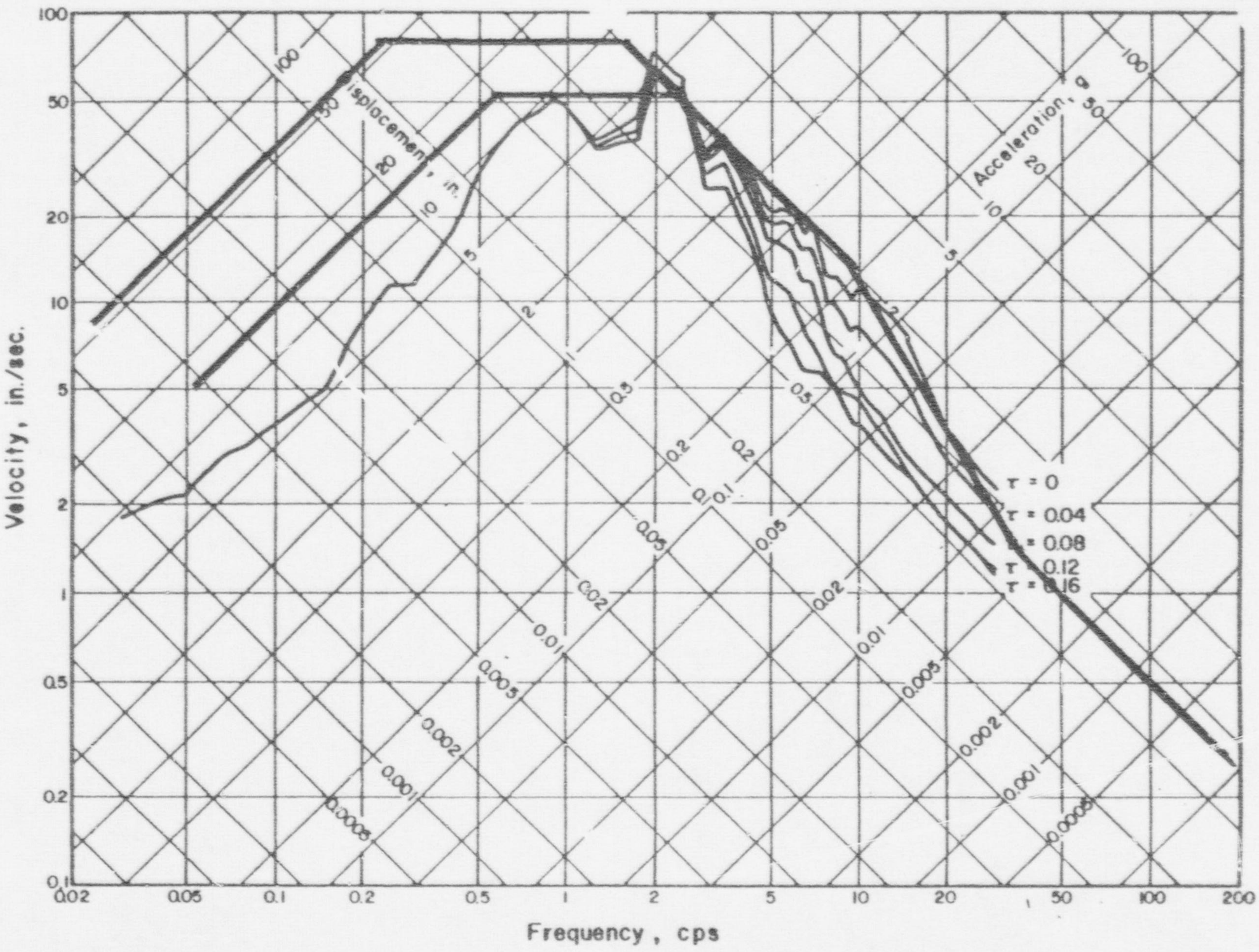

FIG. 19 PACOIMA DAM, SAN FERNANDO EARTHQUAKE, 9 FEB 1971, COMPONENT S74W, DAMPING 5% OF CRITICAL, τ = 0, 0.04, 0.08, 0.12, AND 0.16 SEC. COMPARED WITH DESIGN SPECTRA

TABLE 3. SPECTRUM AMPLIFICATION FACTORS FOR HORIZONTAL ELASTIC RESPONSE

Damping, % Critical	One Sigma (84.1%)			Median		
	A	V	D	A	V	D
0.5	5.10	3.84	3.04	3.68	2.59	2.01
1	4.38	3.38	2.73	3.21	2.31	1.82
2	3.66	2.92	2.42	2.74	2.03	1.63
3	3.24	2.64	2.24	2.46	1.86	1.52
5	2.71	2.30	2.01	2.12	1.65	1.39
7	2.36	2.08	1.85	1.89	1.51	1.29
10	1.99	1.84	1.69	1.64	1.37	1.20
20	1.26	1.37	1.38	1.17	1.08	1.01