NRC PUBLIC DOCUMENT ROOM

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of)
)
PACIFIC GAS AND ELECTRIC COMPANY) Docket Nos. ~~50-275 O.L.~~
) 50-323 O.L.
(Diablo Canyon Nuclear Power Plant,)
Units Nos. 1 and 2))

CERTIFICATE OF SERVICE

I hereby certify that copies of "TESTIMONY OF DR. CARL J. STEPP, DR. NATHAN M. NEWMARK, DR. WILLIAM J. HALL, FAUST ROSA, RENNER B. HOFFMAN, AND JAMES P. KNIGHT," dated November 15, 1978, in the above-captioned proceeding, have been served on the following by deposit in the United States mail, first class, this 15th day of November, 1978:

* Elizabeth S. Bowers, Esq., Chairman
Atomic Safety and Licensing Board Panel
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

* Mr. Glenn O. Bright
Atomic Safety and Licensing Board Panel
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dr. William E. Martin
Senior Ecologist
Battelle Memorial Institute
Columbus, Ohio 43201

Philip A. Crane, Jr., Esq.
Pacific Gas and Electric Company
77 Beale Street, Room 3127
San Francisco, California 94106

Mrs. Elizabeth Apfelberg
1415 Cozadero
San Luis Obispo, California 93401

Mrs. Raye Fleming
1920 Mattie Road
Shell Beach, California 93449

Mr. Frederick Eissler
Scenic Shoreline Preservation Conference, Inc.
4623 More Mesa Drive
Santa Barbara, California 93105

Mrs. Sandra A. Silver
1792 Conejo Avenue
San Luis Obispo, California 93401

Mr. Gordon Silver
1792 Conejo Avenue
San Luis Obispo, California 93401

Richard B. Hubbard
MHB Technical Associates
366 California Avenue
Palo Alto, California 94306

7812040321

Paul C. Valentine, Esq.
321 Lytton Avenue
Palo Alto, California 94302

Yale I. Jones, Esq.
100 Van Ness Avenue
19th Floor
San Francisco, California 94102

John R. Phillips, Esq.
Simon Klevansky, Esq.
Margaret Blodgett, Esq.
Center for Law in the
Public Interest
10203 Santa Monica Drive
Los Angeles, California 90067

David F. Fleischaker, Esq.
1025 15th Street, N. W.
5th Floor
Washington, D. C. 20005

Arthur C. Gehr, Esq.
Snell & Wilmer
3100 Valley Center
Phoenix, Arizona 85073

Janice E. Kerr, Esq.
Lawrence Q. Garcia, Esq.
350 McAllister Street
San Francisco, California 94102

Mr. James O. Schuyler
Nuclear Projects Engineer
Pacific Gas & Electric Company
77 Beale Street
San Francisco, California 94106

John Marrs
Managing Editor
San Luis Obispo County
Telegram-Tribune
1321 Johnson Avenue
P. O. Box 112
San Luis Obispo, California 93406

Bruce Norton, Esq.
3216 North 3rd Street
Suite 202
Phoenix, Arizona 85012

* Atomic Safety and Licensing
Board Panel
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

* Atomic Safety and Licensing
Appeal Panel
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

* Docketing and Service Section
Office of the Secretary
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

James R. Tourtellotte

James R. Tourtellotte
Assistant Chief Hearing Counsel

TESTIMONY OF FAUST ROSA

7812040352

Diablo Canyon Testimony
of John Knox and Faust Rosa
Seismic Qualification of
Class 1E Equipment

Introduction

A detailed description of our evaluation of the seismic qualification of Class 1E equipment for Diablo Canyon is contained in Section 3.10 of the Safety Evaluation Report and its Supplements 7 and 8. This description includes identification of the Class 1E equipment and the applicable seismic criteria, and a discussion of how these criteria were applied in evaluating the seismic qualification that was performed. This testimony will augment this description with emphasis on the electrical aspects of the seismic evaluation, particularly the areas identified in the Intervenor's Response to Applicant's Interrogatories Dated September 27, 1978. A summary status of the seismic evaluation of Class 1E equipment as of December 1, 1978 is also included.

General

As stated in Section 3.10.2 of SER Supplement 7, the majority of the safety-related electrical instrumentation and control equipment was qualified by testing. The balance was qualified by analysis, or a combination of test and analysis. This equipment was previously qualified in accordance with IEEE Standard 344-1971, "IEEE Guide for Seismic Qualification of Class I Electrical Equipment for Nuclear Power Generating Stations," to the level of the double design earthquake approved for the construction permit or higher. Where the original

qualification level does not envelope the required seismic inputs to equipment for the Hosgri event, we have required the applicant to requalify the equipment for the Hosgri required response spectra. This has been done, principally by retesting using the required response spectra.

In the requalification process the applicant employed seismic qualification methods that conform to our current criteria (Regulatory Guide 1.100, Revision 1, "Seismic Qualification of Electrical Equipment for Nuclear Power Plants," August 1977, and IEEE Standard 344-1975, "IEEE Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations").

This updating to current criteria applies to the seismic qualification methods including shake testing methods and the type and severity of shaking employed. It did not, however, include the sequential aging requirements and other general environmental requalification recommendations that are reflected in our current positions for new plants and are referenced in Regulatory Guide 1.100. That is, the sequential aging requirement prior to seismic testing is not included in the qualification criteria for plants of the Diablo Canyon vintage. Our current criteria for environmental qualification for new plants are described in Regulatory Guide 1.89, "Qualification of Class 1E Equipment for Nuclear Power Plants," November 1974, and IEEE Standard 323-1974, "IEEE Standard for Qualifying Class 1E Equipment for Nuclear Power Generating Stations," February 1974.

Seismic Evaluation Summary

For Diablo Canyon, the seismic qualification of Class IE equipment must (1) demonstrate that the equipment can withstand the effects of five Operating Basis Earthquakes, and following this, (2) demonstrate the equipment's ability to perform its required function during and after the time it is subjected to the forces resulting from one Safe Shutdown Earthquake.

Our evaluation includes review of test data and other supporting analyses and documentation to ascertain the adequacy of the required demonstration of seismic capability. More specifically, since qualification of most equipment is based on seismic (shake) testing, our review has or will establish that the equipment performance monitoring performed during testing provides a valid demonstration of functionalability during and following a seismic (Hosgri) event.

The following tabulation provides a summary of the seismic qualification including: (1) a list of the Class 1E equipment, (2) the location of the corresponding seismic documentation, and (3) the basis for acceptability and present status of our evaluation. A detailed description of our evaluation for specific equipments is contained in Supplements 7 and 8 of the Safety Evaluation Report.

Basis and Status Category of Seismic Qualification of Class 1E Equipment

A. Original qualification per IEEE Std 344-1971 enveloped the Hosgri event and is acceptable.

B. Requalification was required to envelope Hosgri event. This was performed per Regulatory Guide 1.100, Rev. 1 and IEEE Std 344-1975 (except aging), and was found acceptable.

C. Requalification to envelope Hosgri was required and performed. This was found acceptable subject to submission of additional confirmatory justification or test results.

D. Requalification to envelope Hosgri was required and performed. Additional testing required to confirm electrical functionability will be performed. Found acceptable subject to successful confirmatory testing.

E. Further seismic evaluation is required; if evaluation of the qualification performed is not acceptable, then additional testing, additional justification, design modifications, or replacement of equipment will be required.

CLASS 1E EQUIPMENT	FSAR AMENDMENT 50 SECTION NO.	SEISMIC QUALIFICATION BASIS AND STATUS
Nuclear Steam Supply System Equipment		
1. Auxiliary Safeguards Cabinet	10.3.2	A
2. Static Inverter	10.3.10	A
3. Nuclear Instrumentation System	10.3.16	A
4. Pressure and Differential Pressure Transmitters	10.3.17	E
5. Process Control and Protection Equipment	10.3.19	A
6. Reactor Trip Switchgear	10.3.20	A
7. Solid State Protection System	10.3.22	A
8. Resistance Temperature Detectors	10.3.27	E
9. Safeguards Test Cabinet	10.3.28	A
10. Fan Cooler Motor	3.10.2.16 (FSAR)	A
Balance of Plant Equipment		
1. Battery Charger	10.3.3	D
2. Station Battery	10.3.4	C
3. DC 125/250 VDC Motor Control Center	10.3.5.1	D
4. 125 VDC Distribution Panel	10.3.5.2	D
5. Diesel Generator Excitation Cubicle	10.3.6	D

CLASS 1E EQUIPMENT	FSAR AMENDMENT 50 SECTION NO.	SEISMIC QUALIFICATION BASIS AND STATUS
Balance of Plant Equipment		
6. Diesel Generator Control Cabinet and Subpanel	10.3.6	D
7. Fire Pump Controller	10.3.7	D
8. Emergency Light Battery Pack	Later	B
9. Hot shutdown panel	10.3.9	
(a) Indicating Meters		C
(b) Switches		B
(c) Fisher Controller		B
10. Instrumentation Power AC Panelboards	10.3.11	E
11. Instrument Panels PIA, PIB and PIC	10.3.12	A
12. Local Instrument Panels	10.3.13	A
13. Local starters	10.3.14	E
14. Main Control Board		
(a) Indicating Meters		C
(b) Switches		B
15. Pressure and Differential Pressure transmitters	10.3.18	A
16. Safeguards Relay Board	10.3.21	C
17. Ventilating Control Logic Cabinet	10.3.23	E
18. Ventilating Control Relay Cabinet	10.3.24	E
19. Vital Load Centers	10.3.25	E
20. Vital Load Center Auxiliary Relay Panel	10.3.25A	E
21. Fan Cooler Motor Controller	10.3.25B	E
22. 4160-volt Switchgear	10.3.26	D
23. Limotorque Valve Operator with Gear and Stem Mounted Limit Switches	10.3.30	C
24. Diesel Generators	10.3.6	A
25. Cable Trays	10.3.29	A
26. Penetrations	10.3.7	A

The seismic qualification of the equipment in status categories A and B has been found acceptable on the basis indicated. The equipment in categories C and D are also considered to be acceptably qualified; however, additional justification or testing is required to resolve any questionable monitoring of functionalability during prior testing. We will review the additional justification and confirmatory testing to verify the adequacy of qualification in this regard. Our evaluation of the qualification of the equipment in status Category E is incomplete; we will require that this equipment be acceptably qualified by the methods indicated prior to completion of our review.

Non Inclusion of Aging in Seismic Qualification

As stated above the acceptance criteria for the qualification of Class 1E equipment for plants of the Diablo Canyon vintage did not include the aging consideration specified in IEEE-Standard 323-1974 and Regulatory Guide 1.89 (which endorses IEEE-323-1974).

In 1974, during the deliberations of the NRC's Regulatory Requirements Review Committee on the implementation of Regulatory Guide 1.89, consideration was given to the incremental improvements to safety it afforded in comparison of the then current staff review practice. The Committee recommended that the guide be applied only to future CP applications; i.e., it should not be backfitted. The decision was based on the Staff's judgment that the incremental improvements were not significant to safety and that full implementation of IEEE-323-1974 required the further development of other ancillary standards to provide guidance on specific safety-related equipment and components.

Subsequent public comments and review by the ACRS did not alter the recommendation concerning implementation of Regulatory Guide 1.89.

We recognize that additional guidance is needed in the area of accelerated aging techniques used to establish a qualified life for electrical equipment and assemblies. Our Category A technical activity on equipment qualification (Task Action Plan A-24) and an NRC extensive research program being carried out at Sandia Laboratires are intended to provide additional guidance for the development of test methods and licensing review procedures on aging. These programs will also allow us to make informal judgements regarding the effects of aging. In addition, as part of the Staff's Systematic Evaluation Program (SEPO, the Staff is assessing the surveillance and maintenance records of the eleven SEP plants for equipment inside and outside of containment. Since this equipment has been effectively "aged", the assessment of these records should provide additional information on the effects of aging.

Following completion of these ongoing activities -- the Task Action Plan A-24, the NRC research program, and the SEP effort -- we will reconsider our position on the need for backfitting the aging requirements. At that time, should we deem it necessary, we will take appropriate steps to ensure that aging effects are considered in assessing the adequacy of Class 1E equipment used in the Diablo Canyon plant. It is our judgment that the natural aging that the Class 1E equipment will undergo in the period prior to this reassessment will have little effect on its seismic capability.

Installation of Seismically Tested Equipment

Some of the Class 1E equipment which has been shake tested to seismically qualify it for the Hosgri event will be installed in the plant. In all such cases only one of a redundant set of equipment will have been tested. The tested equipment will have demonstrated electrical functionability during and following the testing; and it will be carefully inspected after testing to assure that its structural and electrical integrity has not been impaired, and that it remains fully capable of withstanding a Hosgri event. Therefore, we conclude that the installation of tested equipment is acceptable.

FAUST ROSA

PROFESSIONAL QUALIFICATIONS

POWER SYSTEMS BRANCH

DIVISION OF SYSTEMS SAFETY

I have been employed by the Nuclear Regulatory Commission since January 1971. From January 1977 to the present time, I have been Chief, Power Systems Branch, Division of Systems Safety. Prior to my present assignment I served as a Section Chief in the Electrical, Instrumentation and Control Systems Branch, Division of Systems Safety, and in the Plant Systems Branch, Division of Operating Reactors. I have participated in the review of instrumentation, control and electrical systems of numerous nuclear power stations and in the formulation of related standards and Regulatory Guides.

The Power Systems Branch performs an in-depth technical review of the design fabrication, qualification and operation of nuclear power plant electrical power systems important to safety and the related instrumentation and controls. The area of branch review responsibility also includes that portion of the steam system downstream of the main steam isolation valves. This review includes a comprehensive assessment of these systems for all power reactors for adherence to appropriate codes and standards and encompasses complete evaluation of applicant's safety analysis reports, generic reports, and other related system design information. Further, the Branch develops the bases for Regulatory acceptance criteria for electrical power systems designs; evaluates experience obtained during the construction and operation

of nuclear power plants and relates this information to future evaluations and acceptance criteria; and participates in the development of Regulatory Guides and regulations pertaining to electrical power systems and other systems in the branch area of responsibility.

I hold a Bachelor of Electrical Engineering degree from the University of Pittsburgh, Pittsburgh, Pennsylvania. In addition, I have taken courses in Mathematics, Theoretical Physics, Nuclear Physics and Engineering, and Radiation Shielding at the University of Pittsburgh and at the Reactor School of the Bettis Atomic Power Laboratory, Westinghouse Electric Corporation.

My nuclear engineering experience background derives from my employment at the Bettis Atomic Power Laboratory of Westinghouse Electric Corporation, West Mifflin, Pennsylvania, from May 1955 to September 1962; and from my employment at the Bechtel Corporation, Vernon, California, from September 1969 to January 1971. At Bettis Laboratory I was a lead engineer in the nuclear submarine power plant group with technical responsibility for nuclear instrumentation, rod control, and reactor protection systems. Work involved component and system design, installation, testing, modification and documentation. I also served as Bettis representative during full-scale tests conducted by the Navy. At Bechtel I conducted engineering studies and prepared reports and specifications relating to the design and construction of the Rancho Seco Nuclear Power Station. This work was primarily in the areas of safety-related electrical power, instrumentation and control systems.

My non-nuclear engineering background derives primarily from my employment in the Construction Engineering Department of the National Tube Company, United States Steel Corporation, Lorain, Ohio, from June 1947 to April 1955; and from my employment at the Rocketdyne Division of North American Rockwell Corporation, Canoga Park, California, from October 1962 to March 1968. At National Tube I served as a Senior Engineer engaged in design and development of electrical power and control systems for new pipe mills from conceptual design through detail design, procurement, installation, and initial operation. This work extended through completion of two major pipe mill construction projects. At Rocketdyne I was a Research Specialist engaged in design and development of controls and instrumentation for a dual turbo-pump liquid hydrogen feed system for a nuclear rocket engine. My primary responsibility was for control system integration extending from conceptual design through procurement, installation, and completion of the test program.

I am a member of the Institute of Electrical and Electronic Engineers and have participated in the nuclear standards development work of this organization since 1972.

TESTIMONY OF DR. NATHAN M. NEWMARK

TESTIMONY

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	
	)	
PACIFIC GAS AND ELECTRIC COMPANY	)	Docket Nos. 50-275 O.L.
	)	50-323 O.L.
(Diablo Canyon Nuclear Power Plant	)	
Units Nos. 1 and 2)	)	

My name is Nathan M. Newmark. I am a consultant to the United States Nuclear Regulatory Commission, and have been a consultant to the NRC and the Atomic Energy Commission for more than 15 years.

My biographical sketch, including my earned and honorary degrees, my awards and medals, and describing in brief my extensive experience and responsibilities, is attached. A more detailed description of my experience in earthquake resistant design is attached as Appendix I. I am the author of nearly 240 technical papers and several books, one of which is "Fundamentals of Earthquake Engineering," by me and Dr. Emilio Rosenblueth, published in 1971 by Prentice-Hall, Inc., now in its 6th printing.

This testimony is in partial response to several Intervenor's contentions, including numbers 3, 5, 4, 6, and 7. My contact with the operating license review of the Diablo Canyon facilities began in late 1975. Since then, with the assistance of my principal associate, Dr. Willaim J. Hall, I have attended a number of conferences and meetings with

NRC staff personnel; I have participated in many telephone discussions with the staff; I have testified at hearings before the Advisory Committee on Reactor Safeguards, both subcommittee and full committee hearings; I have reviewed drafts of the Safety Evaluation Report and its supplements; I have reviewed answers by the applicant and its consultants of questions raised by the staff, some of which were suggested by me; and I have reviewed documents from various sources related to seismicity, structural and equipment response to seismic excitation, and other aspects to the re-evaluation of the Diablo Canyon facility.

I have prepared a number of reports and documents for the NRC staff describing my recommendations and conclusions, the most important of which is "A Rationale for Development of Design Spectra for Diablo Canyon Reactor Facility," dated 3 September 1976, attached hereto, and designated hereafter as Reference A.

My views and recommendations are presented quite adequately in an informal discussion attached and is referred to hereinafter as Reference B.

Since many of the Intervenor's contentions seem to be based on the implicit assumption that a great deal of reliance can be placed on the results of complex analyses, I should like to point out that analysis is always based on assumptions, many of which are not directly applicable to the real world of soil and rock, steel and concrete. The assumption of

elastic behavior generally overestimates dynamic response. Analysis has a use as a guide to judgment and experience; it cannot replace these as the sole source of design criteria. Observations of earthquake damage, or more importantly, lack of damage, in actual earthquakes; observations of dynamic response in tests of structures and equipment subjected to blast and ground shock movement; both have demonstrated the fact that elastic analyses generally overestimate the required design levels. These points are discussed more fully in Ref. B.

With regard to specific contentions, my comments follow.

Contention 3 - Adequacy of 0.75 g Safe Shutdown Acceleration
My views on this topic are presented in Ref. A. In that reference I pointed out that the Pacoima Dam record showed a peak acceleration of 1.20 g, but a response spectrum based on my procedures, very much the same as for NRC Reg. Guide 1.60, drawn for an anchor (or SSE) value of 0.75 g, would envelop the Pacoima Dam record spectrum.

It was pointed out that, close to the source, the peak ground acceleration is not sensitive to magnitude. Earthquakes of magnitudes 4 or 4.5 up to 8 or 8.5 may have the same or very nearly the same peak ground acceleration. The values of peak acceleration are a function of magnitude at larger distances, however.

The point is further amplified in the first eleven pages of Ref. B.

The so-called τ-effect, or reduction in high frequency (above 2 hertz) parts of the response spectrum was first described by Yamahara in 1970, and was further reported on by Ambraseys in 1975, and Scanlan in 1976. (See Refs. 14, 15, 16, of Ref. A). I derived a slightly different approach to the problem, and then hedged it with considerable conservatism by limiting the τ-effect consideration only to very close earthquakes and putting a floor on the reduction value. Another basis for reduction in response of a large structure is due to spatial differences in simultaneous values of acceleration over large areas, arising from inhomogeneities of the soil-rock medium, and discussed briefly on p. 113 of Ref. B.

However, in all cases the torsional effect was specified to be taken into account in the design, in Ref. A. The combined effect of decrease in response due to seismic wave propagation, and the increase due to torsion, is generally overestimated by analysis.

The assumptions which cause the results of calculations to be excessively conservative are the following:

(1) Only systematic motions over the base are taken into account. The true motions are in large part random, and therefore cause much lower torsional responses, but about the same translational reduction.

(2) The major torsional effect comes from horizontally propagated vertical wave fronts of motion. Since only part of the motions in an earthquake is of this type, the torsional responses are again exaggerated, but the translational reductions are not greatly affected.

(3) The assumption that the torsional and translational frequencies are identical causes an overestimate of the torisonal response compared with the translational response. In general these frequencies are different for real structures.

In view of these facts it is my recommendation that currently acceptable building code procedures be used to define the effects of torsion, combined with the procedures developed in Ref. A for reduction of the translational effects.

Contention 5 - Adequacy of the Dynamic Analysis

The design spectra that I recommended in Ref. A are generally more conservative than those proposed by the applicant, and were intended to be applied without allowance for inelastic structural response except where proper justification could be made. Hence the design criteria were intended to and do cover aftershocks, since no or little permanent deformation would result from the main shock.

In my review of the structural design, I felt that my intentions were achieved by the applicant.

In my opinion the design criteria for the structures are conservative, and the retrofit proposed by the applicant and agreed to by the staff will assure the safety of the system. However, I was not directly involved in the mechanical or electrical equipment responses, although I had contact with the staff on some aspects of these.

Contentions 4, 6 - Adequacy of OBE Seismic Acceleration and OBE Design. The OBE was originally proposed some 10 to 15 years ago as one of two levels of earthquake motion, to be designed for at "working stress" limits, in contrast to the higher level to be designed for at or just slightly beyond yield levels. At that time, when it was proposed by me, no load factors or other conservatisms were intended, and the selection of an OBE of one-half the SSE was consistent with the values of the spectral responses and with the allowable stresses for the two earthquakes.

There has been a feeling among many engineers that with the present concepts and factors a proper value for the OBE is from one-forth to one-third the SSE level. This is my opinion also.

For example, I do not believe that the low damping levels specified for the OBE are consistent with my recommendation that the damping be dependent on the stress levels or response level, nor do I believe that accident loads should be included in the OBE.

In my opinion an OBE level of 0.2 g will not impair the capability of the structure or equipment to resist the SSE earthquake.

Contention 7 - Stress or Strain Levels in Excess of Yield. In so far as structures are concerned, seismic responses in excess of yield strains involve deformations or displacements that increase in the same

ratio as the strain in all cases where the ductility factor or ratio of total strain to elastic component of strain is less than 1.3, and only at a slightly increased ratio for high frequency components up to a ductility factor of 1.5.

In my opinion, all structures can meet these criteria without danger of damage that would impair safety.

General - In my opinion, the design spectra I recommended in Ref. A are adequately conservative to assure safety of the Diablo Canyon facility. Any higher value of SSE acceleration would not really increase safety. Using the design criteria I proposed, the most important remaining aspects of safety are those concerned with construction details and quality control of construction.

Facility design in accordance with my recommendations, which I believe have been followed by the applicant, will have a sufficient margin of safety for ground motion in earthquakes that have been experienced, or that can reasonably be expected, at the site or at similar sites on the West Coast of the United States.

APPENDIX I

The procedures and principles that I have developed have been used as the bases for seismic design for nuclear reactor facilities in the United States, and in a number of foreign countries as well. In addition, I have had the major responsibility for the development of seismic design criteria for the Trans-Alaska Oil Pipeline, a proposed gas pipeline of the Canadian Arctic Gas Pipeline study in Canada, and the Bay Area Transit System, among others. The procedures that I have developed for seismic design and review of earth and rock-fill dams and embankments, which were presented in detail in my Rankine Lecture in 1965 at the Institution of Civil Engineers in Great Britain, are used throughout the world for seismic design of dams.

I have been engaged as a consultant on a number of projects for which I have had the primary responsibility for selecting the design criteria for seismic resistance. These include the following:

(1) Several major dams, including Kremasta Dam in Greece; Portage Mountain Dam of the Peace River in Canada; Mirpur Dike, a part of the Mangla Dam project in Pakistan; and several dams for the U.S. Army Corps of Engineers, including the proposed Richard B. Russell Dam in Georgia, the proposed Mentone Dam near Riverside, California, and the strengthening of Prado Dam in California. The Mentone Dam is located very close to the San Andreas Fault, and a seismic design ground acceleration of 0.6 g was selected as the design criterion for that dam because of the hazardous location and the serious consequences of failure.

(2) The development of the seismic design criteria for the Bay Area Rapid Transit System in California including the tube under San Francisco Bay. A basic maximum ground acceleration of 0.6 g, with an allowance for reduction in response due to inelastic behavior was used in the development of these criteria.

(3) The development of design criteria for a proposed pipeline to carry gas from Prudhoe Bay and the Canadian gas fields through northern Alaska and Canada, to southern Canada, including pump stations and other facilities.

(4) The Trans-Alaska Oil Pipeline for Alyeska Pipeline Service Company, covering detailed design of the pipeline itself; terminal facilities, including tanks and loading docks; river crossings; pump stations; communication towers and facilities, etc.; and especially fault motions of as much as 20 ft. in crossings of two major fault systems along the pipeline. The range of the equivalent of SSE ground motions was from 0.1 g for the seismically quieter portions of the pipeline in the north slope, upward to 0.6 g in the regions near the Denali Fault, which is considered to have the same potential of movement and maximum earthquake intensity as the San Andreas Fault, and 0.6 g for ground motion at the terminal near Valdez where major earthquakes have been experienced before. In all cases, there was a reduction in design values for structures consistent with inelastic response.

(5) The Proton-Electron-Positron extension of the Stanford Linear Accelerator, within one or two miles of the San Andreas Fault in California, where a maximum ground acceleration of 0.6 g was used in design with a considerable measure of reduction in response due to inelastic behavior.

(6) A number of important buildings, including the Latino Americana Tower in Mexico City, the Chateau Champlain Hotel in Montreal, and a major building project in Vancouver, where provision was made for major earthquake motions consistent with regional seismicity. It is of particular interest to note that the Latino Americana Tower, which was the first major building to be designed in accordance with the most recent knowledge and research results, in the period 1949-1951, was subjected to its design earthquake in 1957; and instruments placed in the building recorded motions at several levels in the structure within 10 percent of those predicted by the methods of analysis developed by me.

(7) Buildings in general in the United States. My efforts have been involved as the principal technical director of a project to develop a report, "Tentative Provisions for the Development of Seismic Regulations for Buildings," prepared by the Applied Technology Council, and published in June 1978 by that Council, the National Science Foundation, and the National Bureau of Standards. These provisions are intended to be used throughout the United States for buildings of all kinds in all parts of the country, with seismic motions ranging from maximum ground accelerations of 0.05 g up to 0.4 g, which was considered in the provisions to be the maximum "effective" ground acceleration needed in the design of apartment buildings, schools, and other structures housing large numbers of people. Substantial reductions in design levels are permitted for inelastic action in these provisions.

REFERENCE "A"

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. 1a 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. 1a 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. P rhaps a better measure of the reduction in effectiveness of an thquake 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 o about 0.08 sec in the north-south direction, and for the east-west directio 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 seismi 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

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

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.

REFERENCES

1. N. C. Donovan, A Statistical Evaluation of Strong Motion Data Including the February 9, 1971 San Fernando Earthquake, Proceedings Fifth World Conference on Earthquake Engineering (Rome), Vol. 1, 1974, pp. 1252-1261.

2. R. A. Page, D. M. Boore, W. B. Joyner, and H. W. Coulter, Ground Motion Values for Use in the Seismic Design of the Trans-Alaska Pipeline System, U.S. Geological Survey Circular 672, 1972.

3. H. B. Seed, R. Murarka, J. Lysmer, and I. M. Idriss, "Relationships between Maximum Acceleration, Maximum Velocity, Distance from Source, and Local Site Conditions for Moderately Strong Earthquakes", Earthquake Engineering Research Center, University of California, Berkeley, EERC 75-17, July 1975.

4. N. M. Newmark, W. J. Hall, B. Mohraz, "A Study of Vertical and Horizontal Earthquake Spectra", Directorate of Licensing, U.S. Atomic Energy Commission, Report WASH-1255, April 1973.

5. B. Mohraz, A Study of Earthquake Response Spectra for Different Geological Conditions, Institute of Technology, Southern Methodist University, Dallas, Texas, 1975.

6. G. W. Housner, Earthquake Research Needs for Nuclear Power Plants, Journal Power Division, Proceedings ASCE, Vol. 97, 1971, pp. 77-91.

7. W. K. Cloud, Intensity Map and Structural Damage, Parkfield, California, Earthquake of June 27, 1966, Bull. Seism. Soc. of America, Vol. 57, No. 6, 1967, pp. 1161-1178.

8. J. F. Lander, editor, Seismological Notes, January-February 1972, Bull. Seism. Soc. of America, Vol. 62, No. 5, 1972, pp. 1360-1362. Lander, J. F., editor, Seismological Notes, September-October 1973.

Bull. Seism. Soc. cf America, Vol. 63, No. 3, 1973, pp. 1177-1178.

9. R. Console, F. Peronaci, A. Sonaglia, Relazione Sui Fenomeni Sismica Dell'Anconitano (1972), Annali di Geofisica, Vol. 26, Supplement 1973, Rome.

10. W. J. Hall, B. Mohraz, and N. M. Newmark, Statistical Studies of Vertical and Horizontal Earthquake Spectra, U.S. Nuclear Regulatory Commission, Contract AT(49-5)-2667, Report NUREG-0003, January 1976.

11. N. M. Newmark, Earthquake Resistant Design of Nuclear Power Plants, Article for UNESCO Intergovernmental Conference on Assessment and Mitigation of Earthquake Risk, Paris, February 1976.

12. N. M. Newmark and W. J. Hall, Procedures and Criteria for Earthquake Resistant Design, Building Practices for Disaster Mitigation, National Bureau of Standards, Building Science Series 46, Vol. 1, February 1973, pp. 209-236.

13. N. M. Newmark, A Response Spectrum Approach for Inelastic Seismic Design of Nuclear Reactor Facilities, Transactions, Third International Conference on Structural Mechanics and Reactor Technology, London, 1975, Paper K 5/1.

14. H. Yamahara, Ground Motions during Earthquakes and the Input Loss of Earthquake Power to an Excitation of Buildings, Soils and Foundations, Vol. 10, No. 2, 1970, pp. 145-161, Tokyo.

15. N. Ambraseys, Characteristics of Strong Ground Motion in the Near Field of Small Magnitude Earthquakes, Invited Lecture, Fifth Conference European Committee for Earthquake Engineering, Istanbul, September 1975.

16. R. H. Scanlan, Seismic Wave Effects on Soil-Structure Interaction, Earthquake Engineering and Structural Dynamics, Vol. 4, 1976, pp. 379-388.

17. C. M. Duke, J. E. Luco, A. R. Carriveau, P. J. Hradilek, R. Lastrico, and D. Ostrom, Strong Earthquake Motion and Site Conditions: Hollywood, Bull. Seism. Soc. of America, Vol. 60, No. 4, 1970, pp. 1271-1289.

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

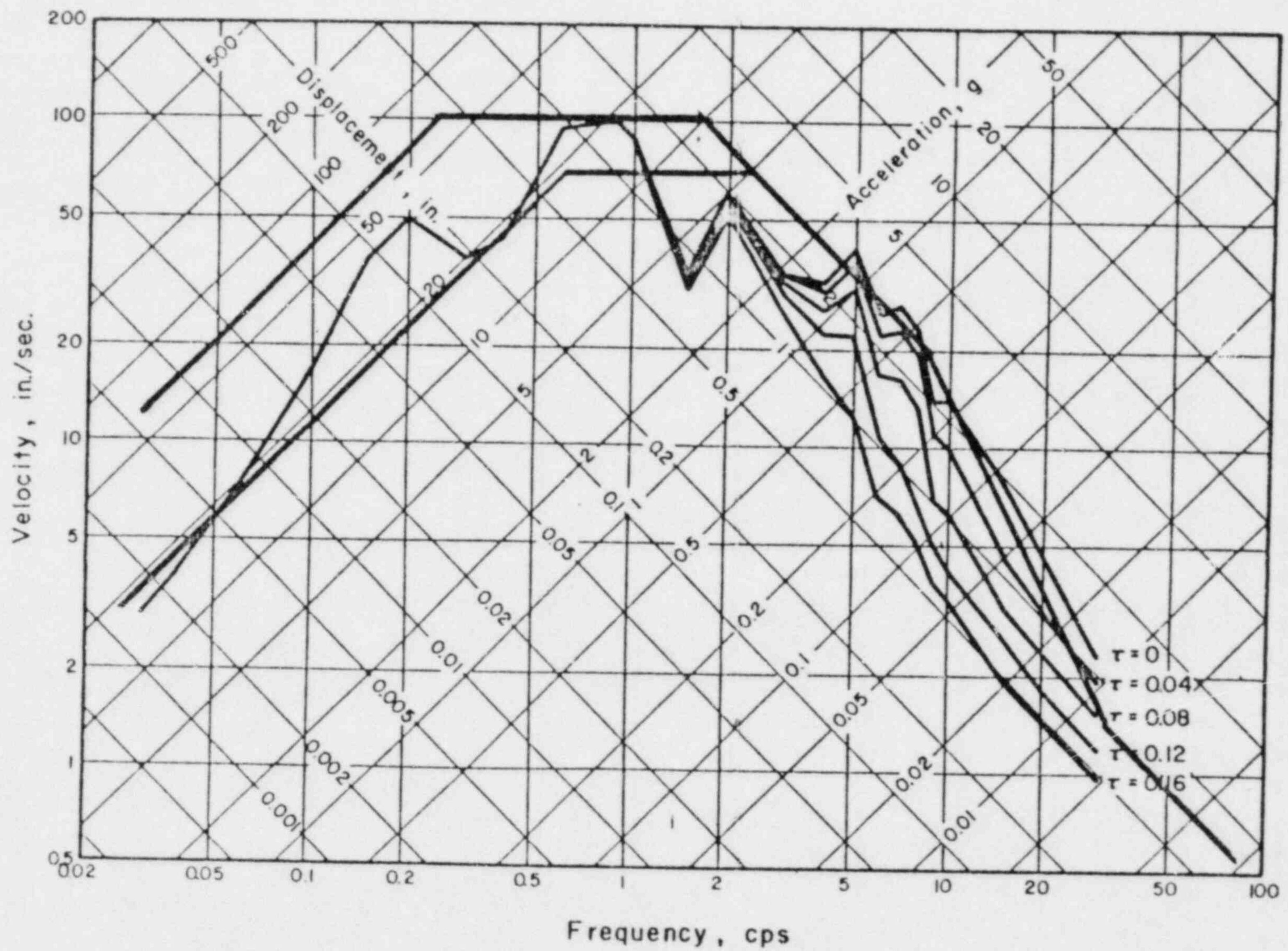


FIG. 1a PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S16E, 2 PERCENT DAMPING, τ = 0, 0.04, 0.08, 0.12, 0.16 sec. COMPARED WITH DESIGN SPECTRA

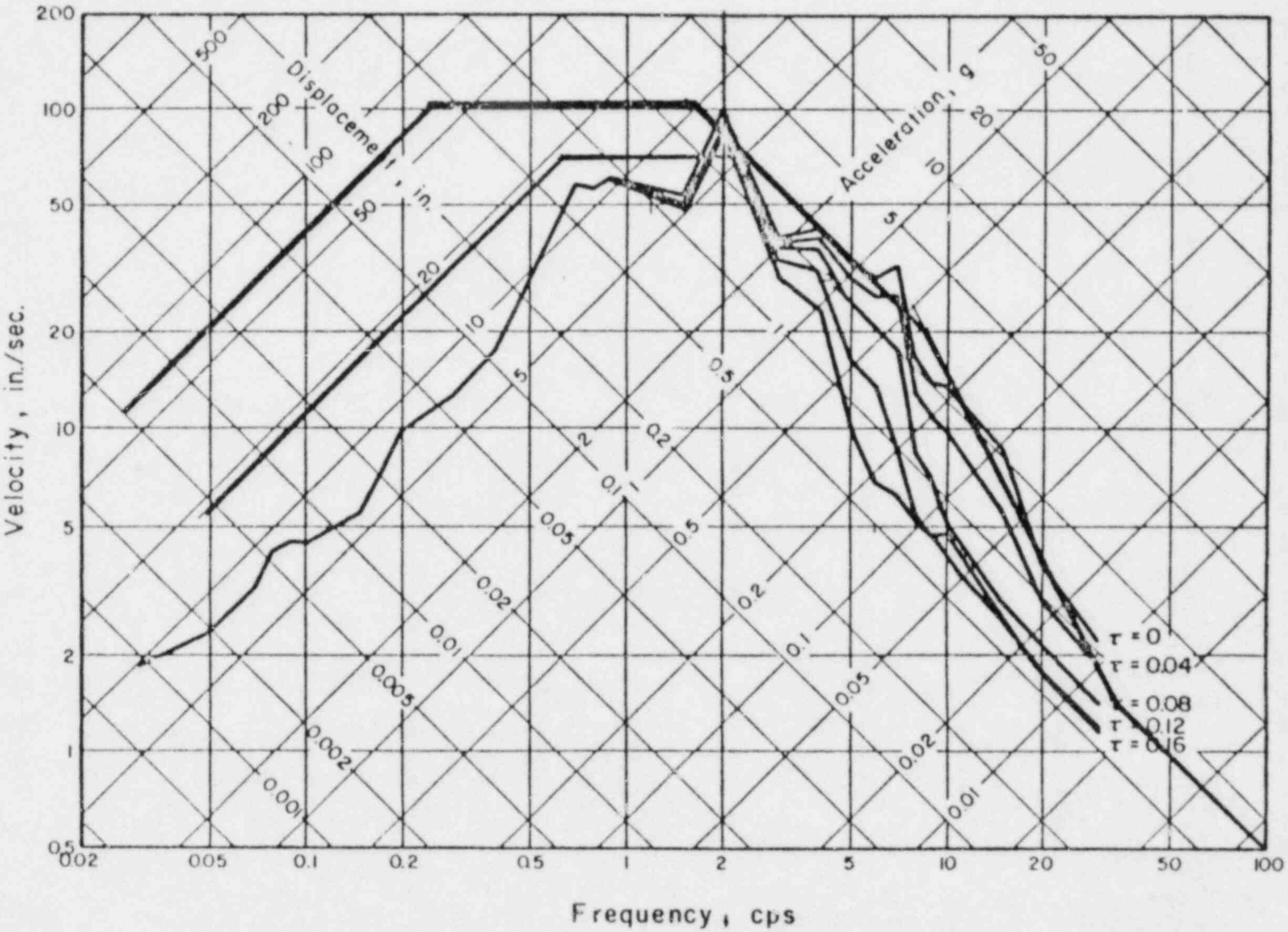
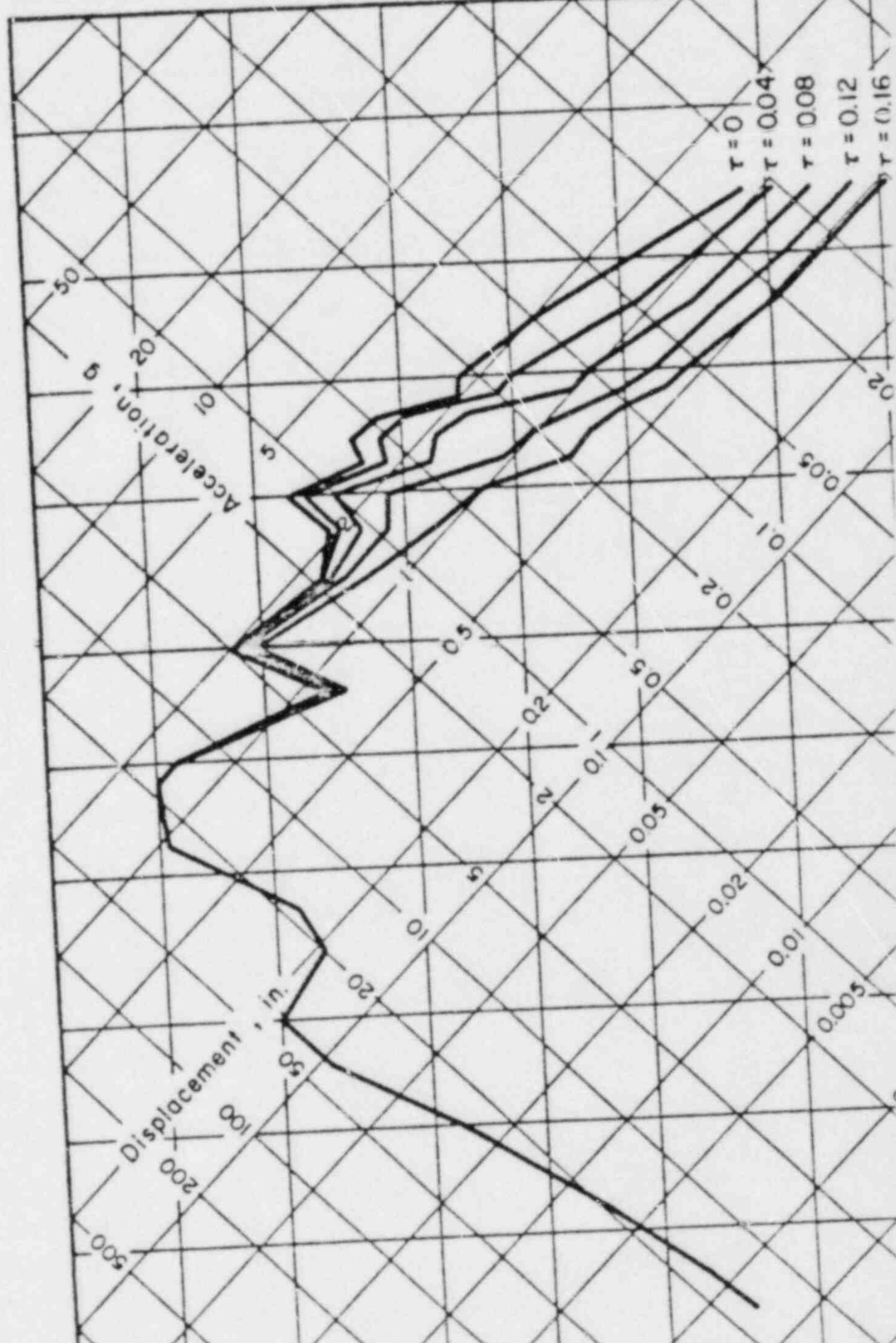


FIG. 1b PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S74W, 2 PERCENT DAMPING, τ = 0, 0.04, 0.08, 0.12, 0.16 sec. COMPARED WITH DESIGN SPECTRA


Velocity , in./sec.
Displacement , in.
Acceleration, g
τ = 0
τ = 0.04
τ = 0.08
τ = 0.12
τ = 0.16

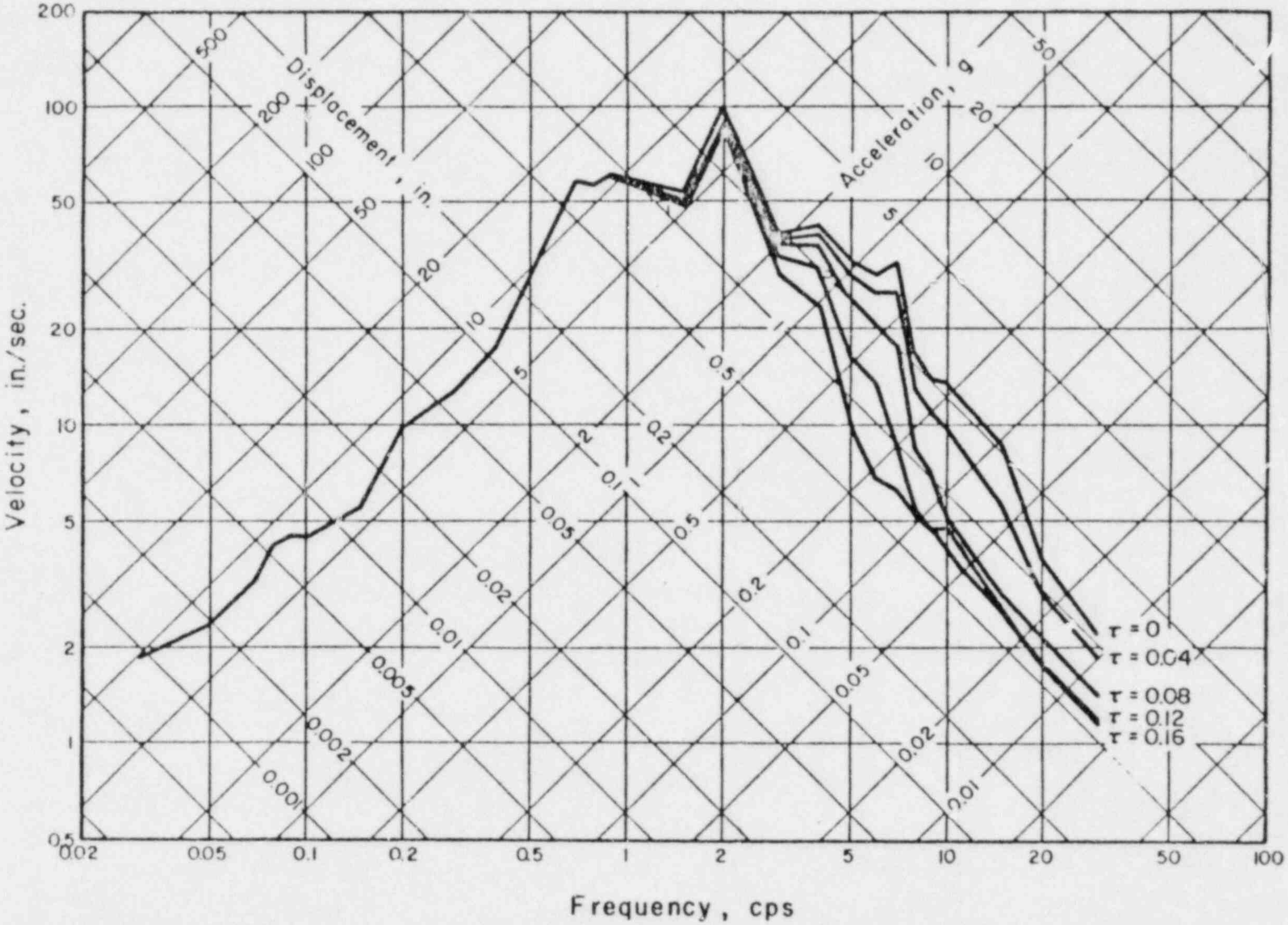


FIG. 2b PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S74W, 2 PERCENT DAMPING, τ = 0, 0.04, 0.08, 0.12, 0.16 sec.

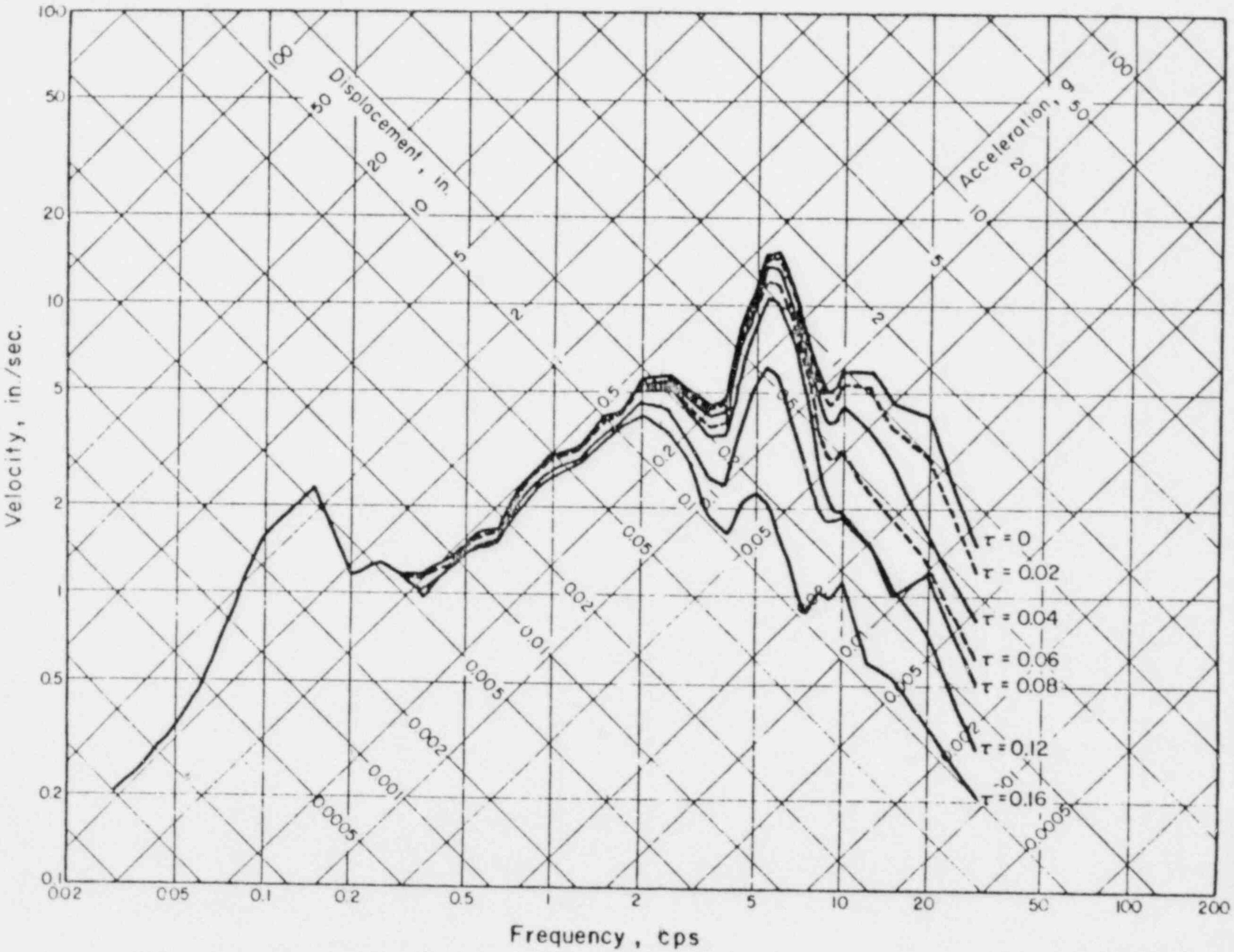


FIG. 3 ANCONA, ROCCA 6-14-72 GMT-NORTH $\tau = 0, 0.02, 0.04, 0.06, 0.08, 0.12, 0.16$
SPECTRUM COMPUTED USING 5.0 PERCENT CRITICAL DAMPING

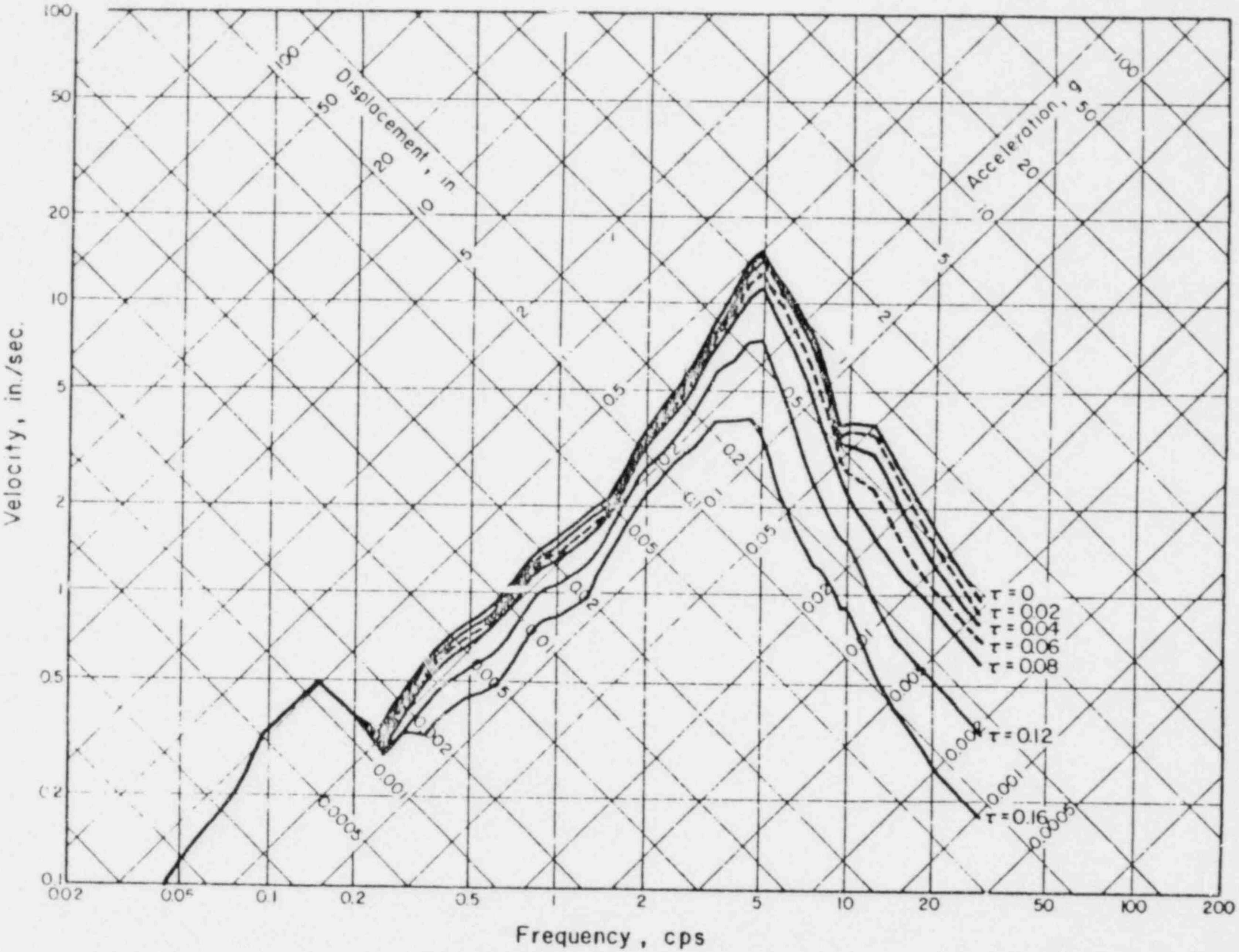


FIG. 4 ANCONA, PALOMBINA 6-21-72 GMT-NS $\tau = 0, 0.02, 0.04, 0.06, 0.08, 0.12, 0.16$
SPECTRUM COMPUTED USING 5.0 PERCENT CRITICAL DAMPING

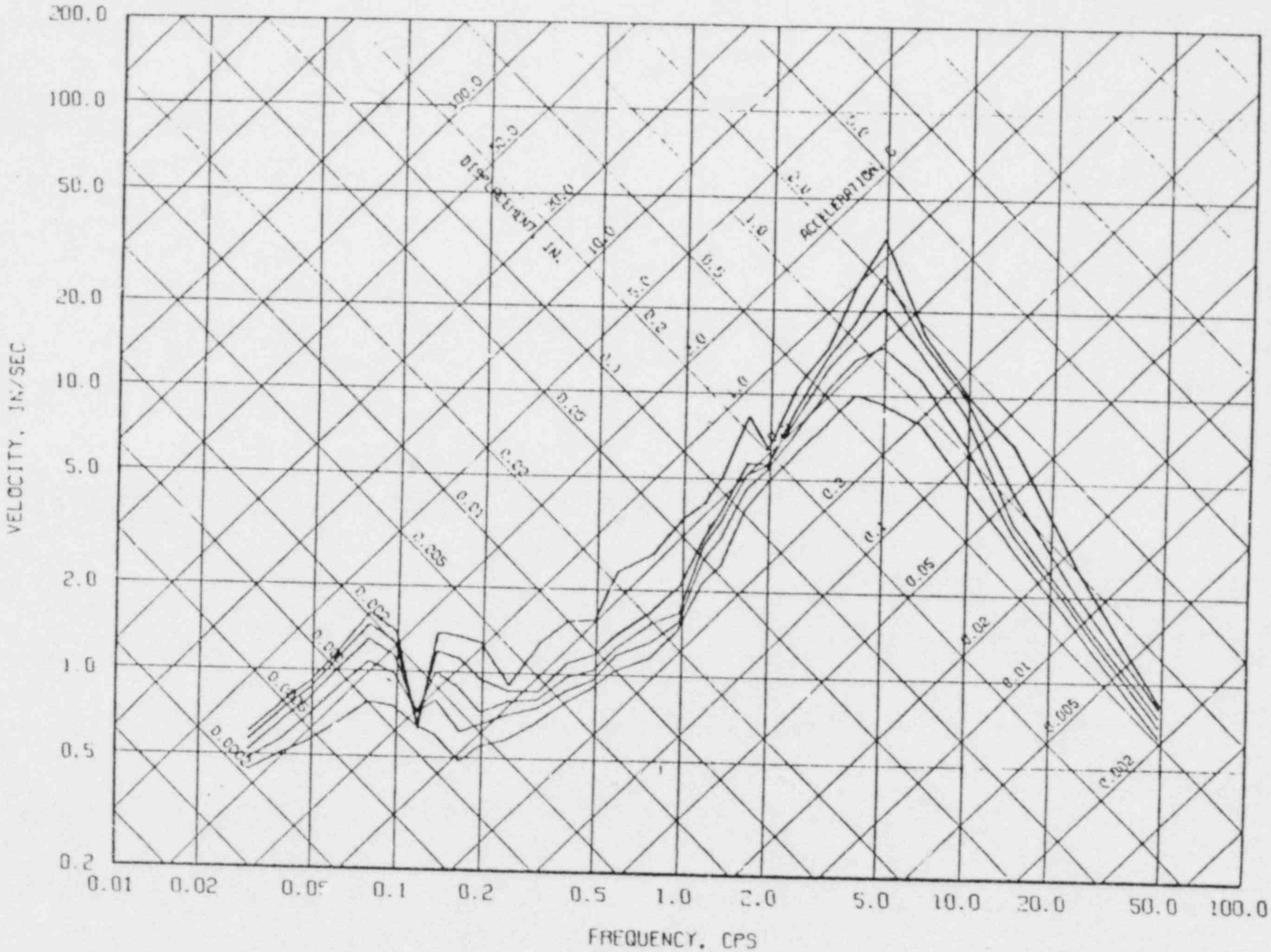


FIG. 5 RESPONSE SPECTRA FOR MELENDY RANCH BARN 9/4/72 N61E COMPONENT

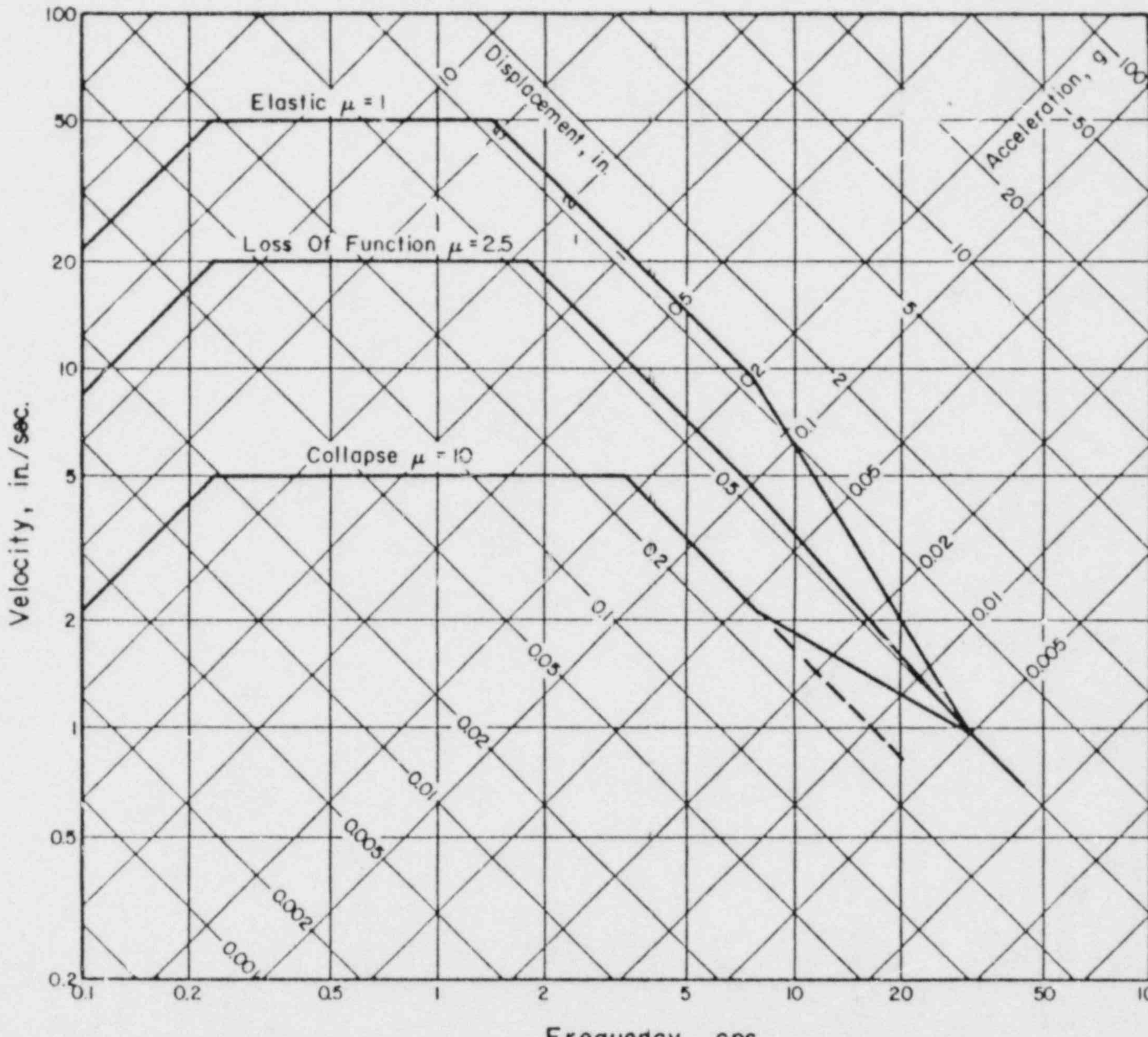


FIG. 6 INELASTIC DESIGN SPECTRAL REQUIREMENT FOR PEAK GROUND ACCELERATION OF 0.5 G, 7% DAMPING

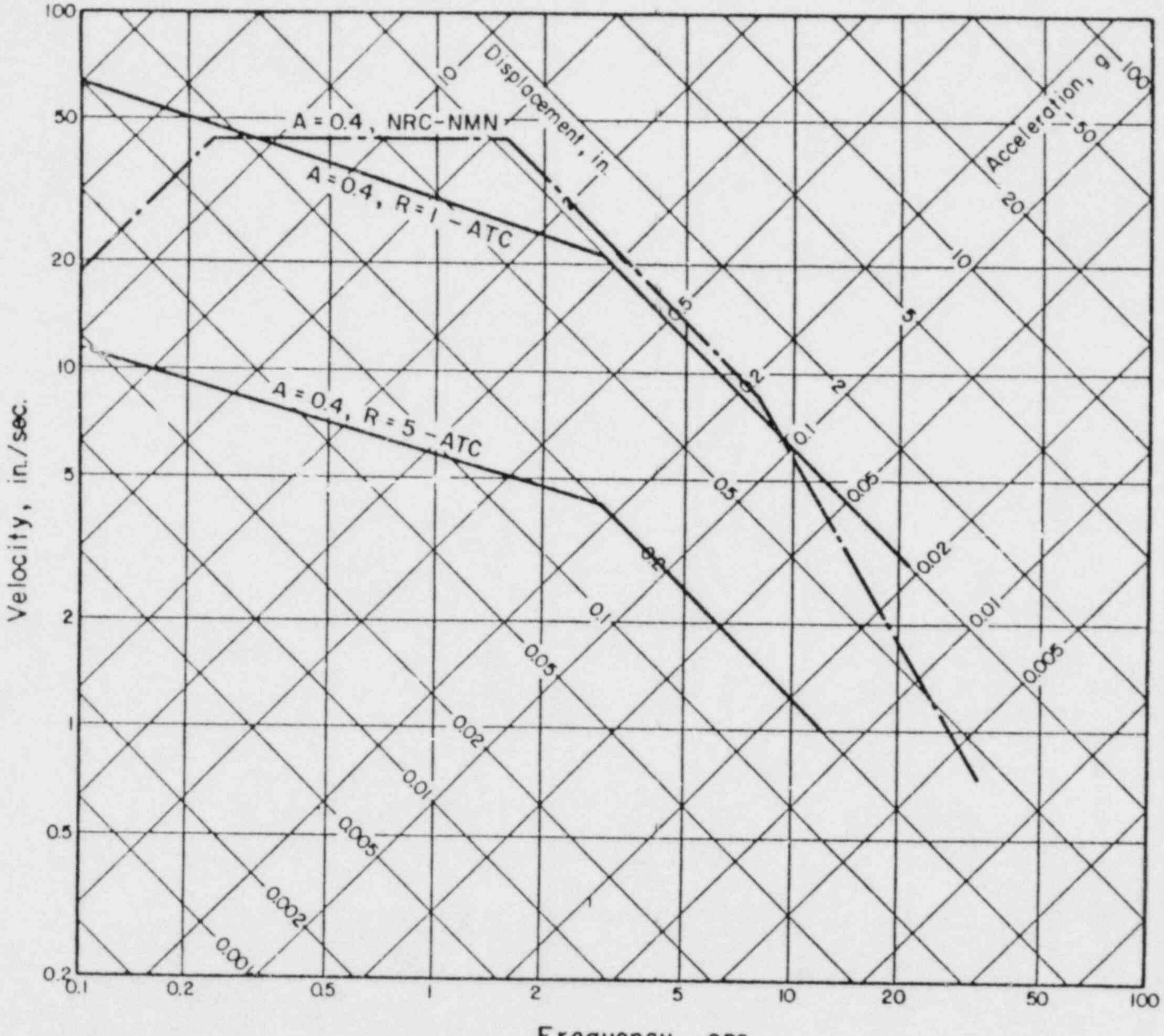


FIG. 7 COMPARISON OF ATC AND NRC DESIGN COEFFICIENTS, 5% DAMPING

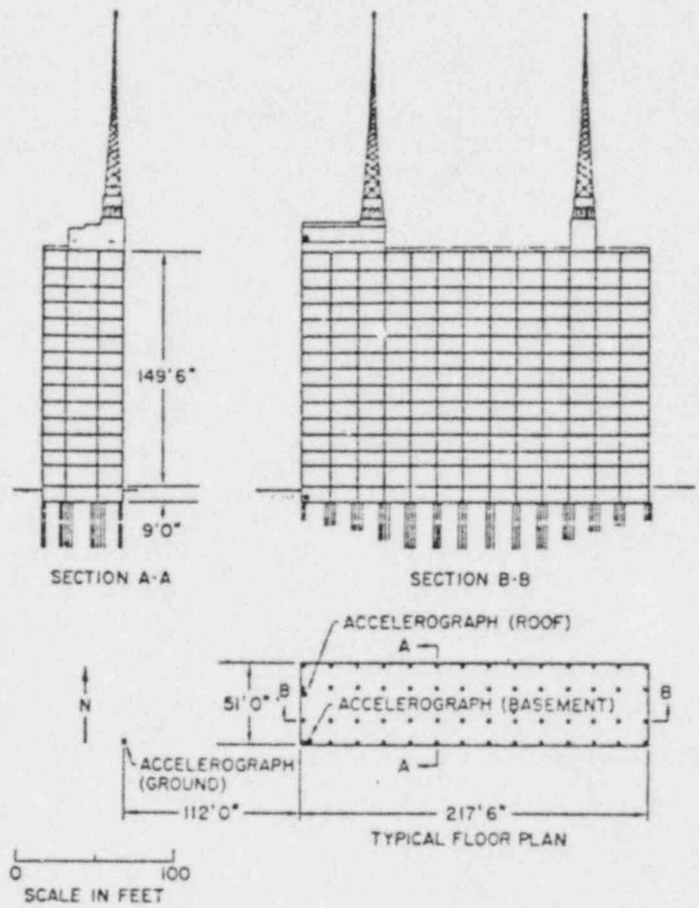


FIG. 8 PLAN AND ELEVATION OF BUILDING

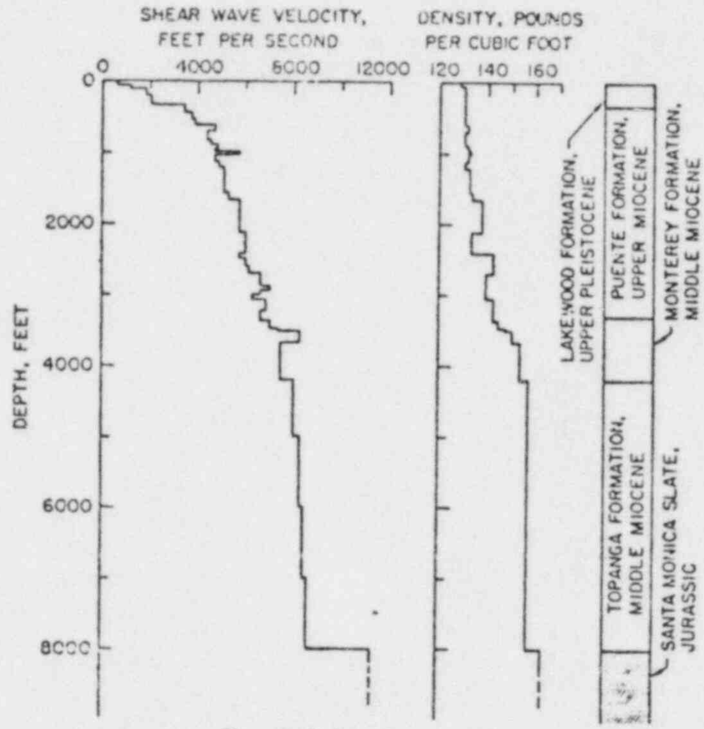


FIG. 9 Subsurface model.

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.

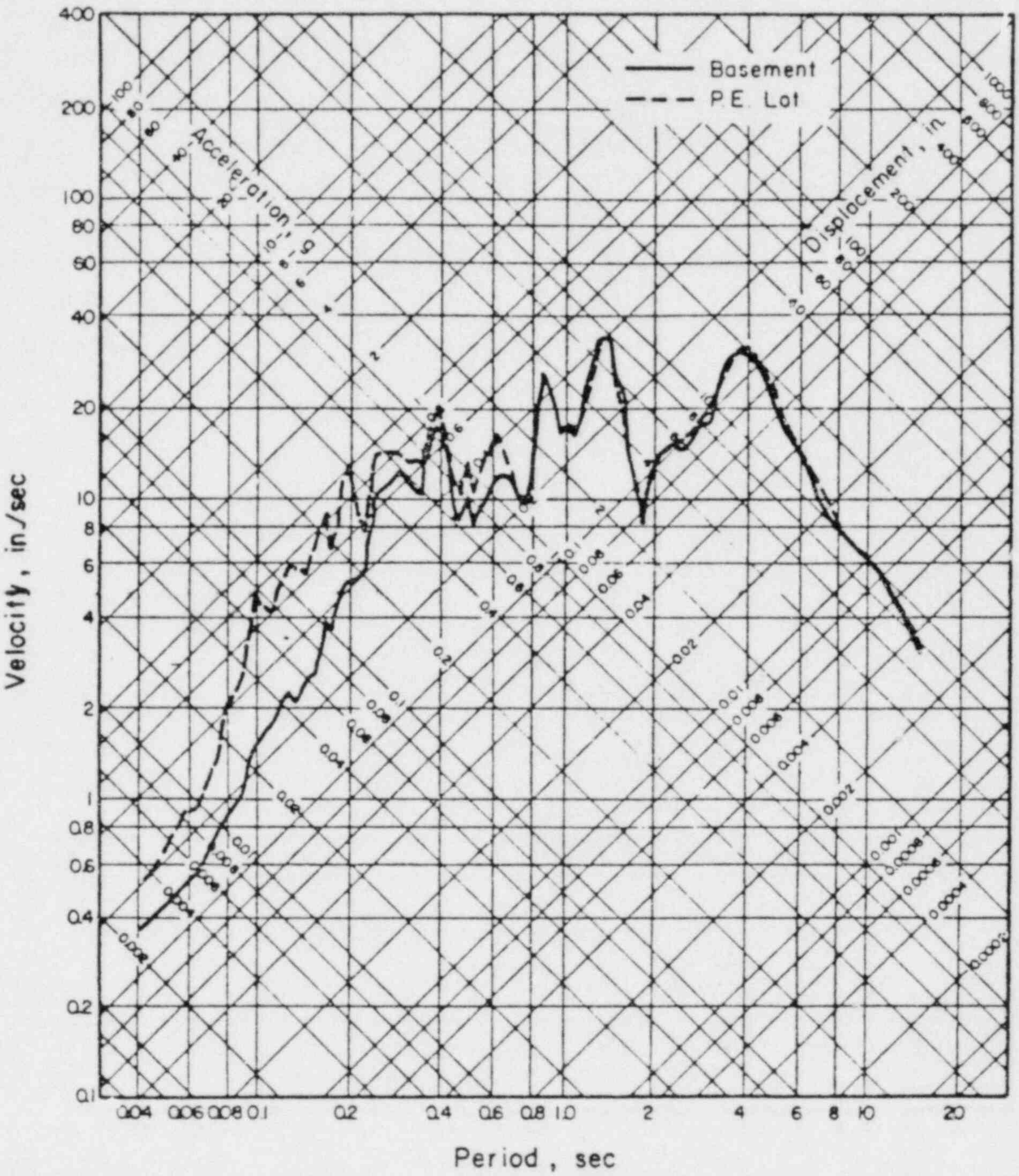


FIG. 10 SAN FERNANDO EARTHQUAKE, FEB. 9, 1971 - 0600 PST
HOLLYWOOD STORAGE BASEMENT AND P.E. LOT, COMPONENT
EAST, DAMPING VALUE 2% OF CRITICAL

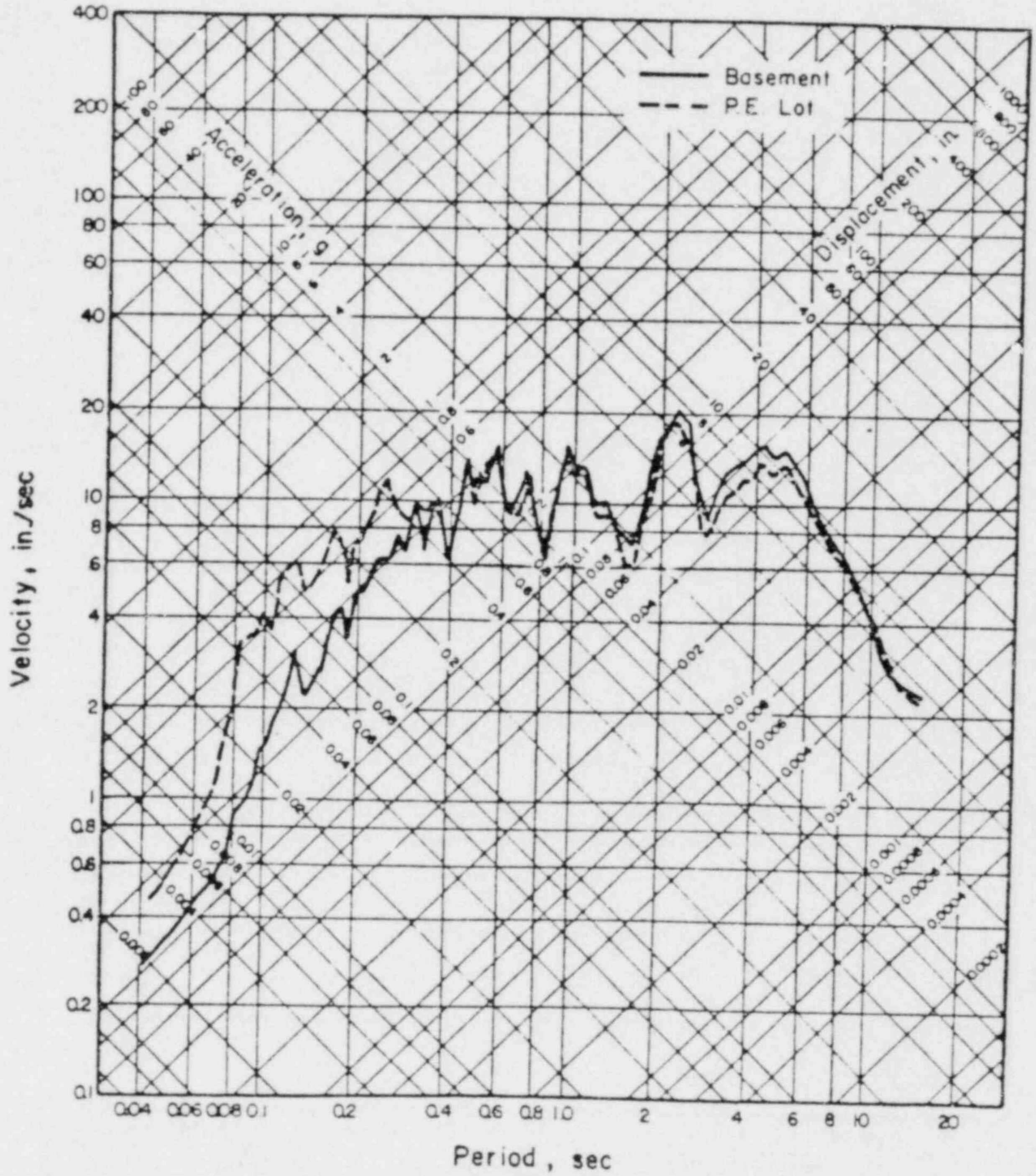


FIG. II SAN FERNANDO EARTHQUAKE, FEB. 9, 1971 - 0600 PST HOLLYWOOD STORAGE BASEMENT AND P.E. LOT, COMPONENT SOUTH, DAMPING VALUE 2% OF CRITICAL

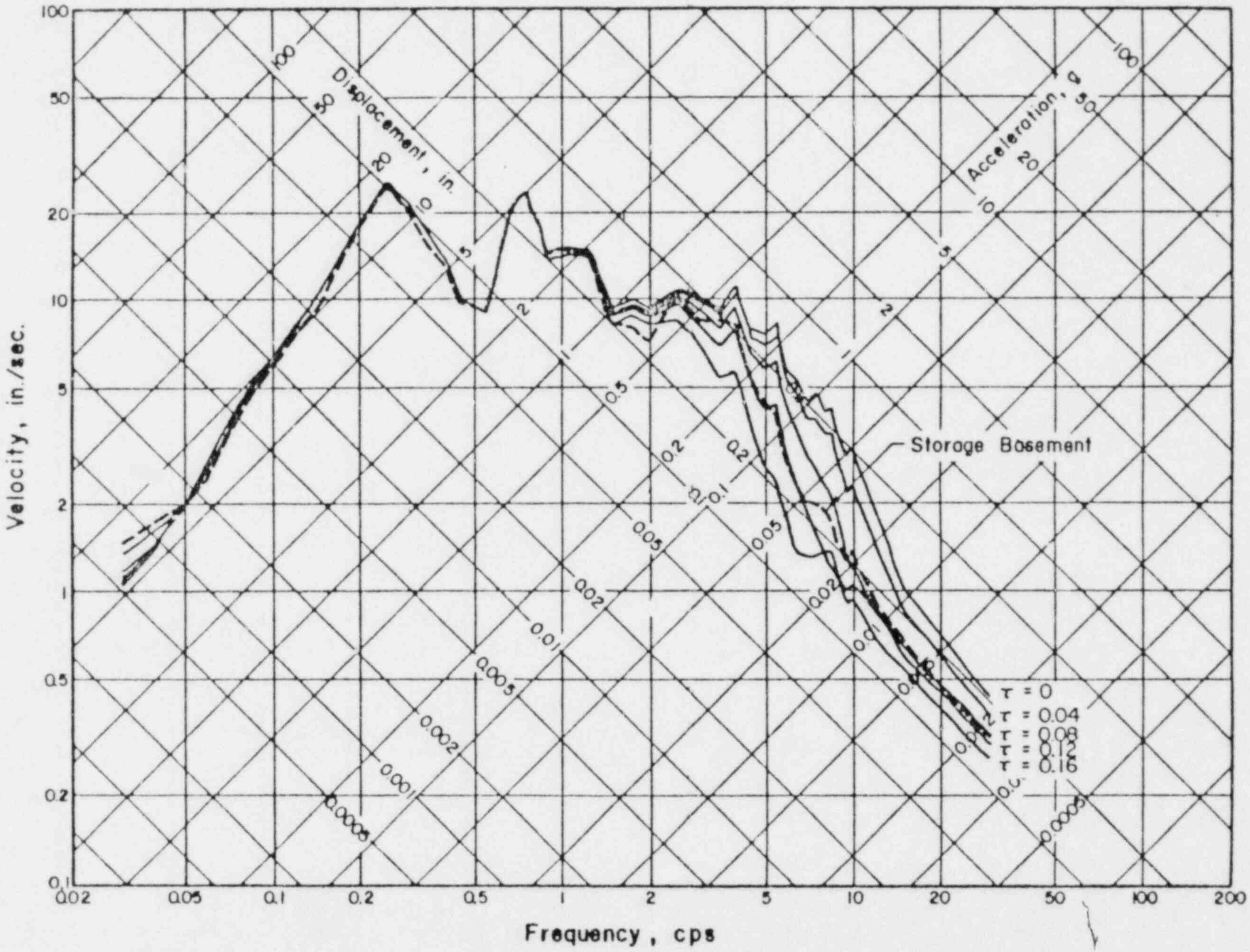


FIG. 12 HOLLYWOOD STORAGE P.E. LOT, SAN FERNANDO EARTHQUAKE FEB. 9, 1971, COMPONENT EAST, DAMPING 5 % OF CRITICAL, τ = 0, 0.04, 0.08, 0.12, AND 0.16 SEC.

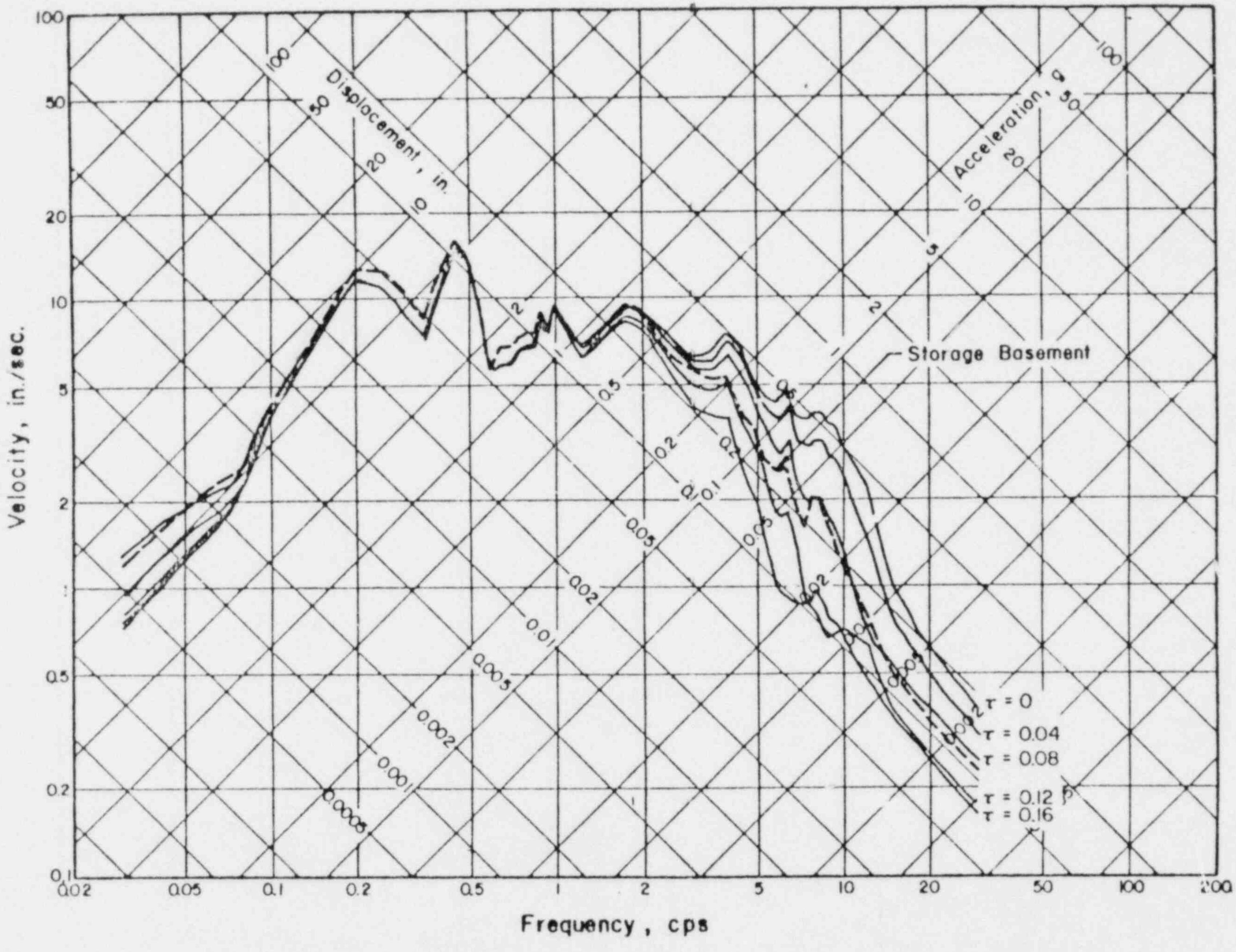


FIG.13 HOLLYWOOD STORAGE P.E. LOT, SAN FERNANDO EARTHQUAKE FEB. 9, 1971, COMPONENT SOUTH, DAMPING 5 % OF CRITICAL, τ = 0, 0.04, 0.08, 0.12, AND 0.16 SEC.

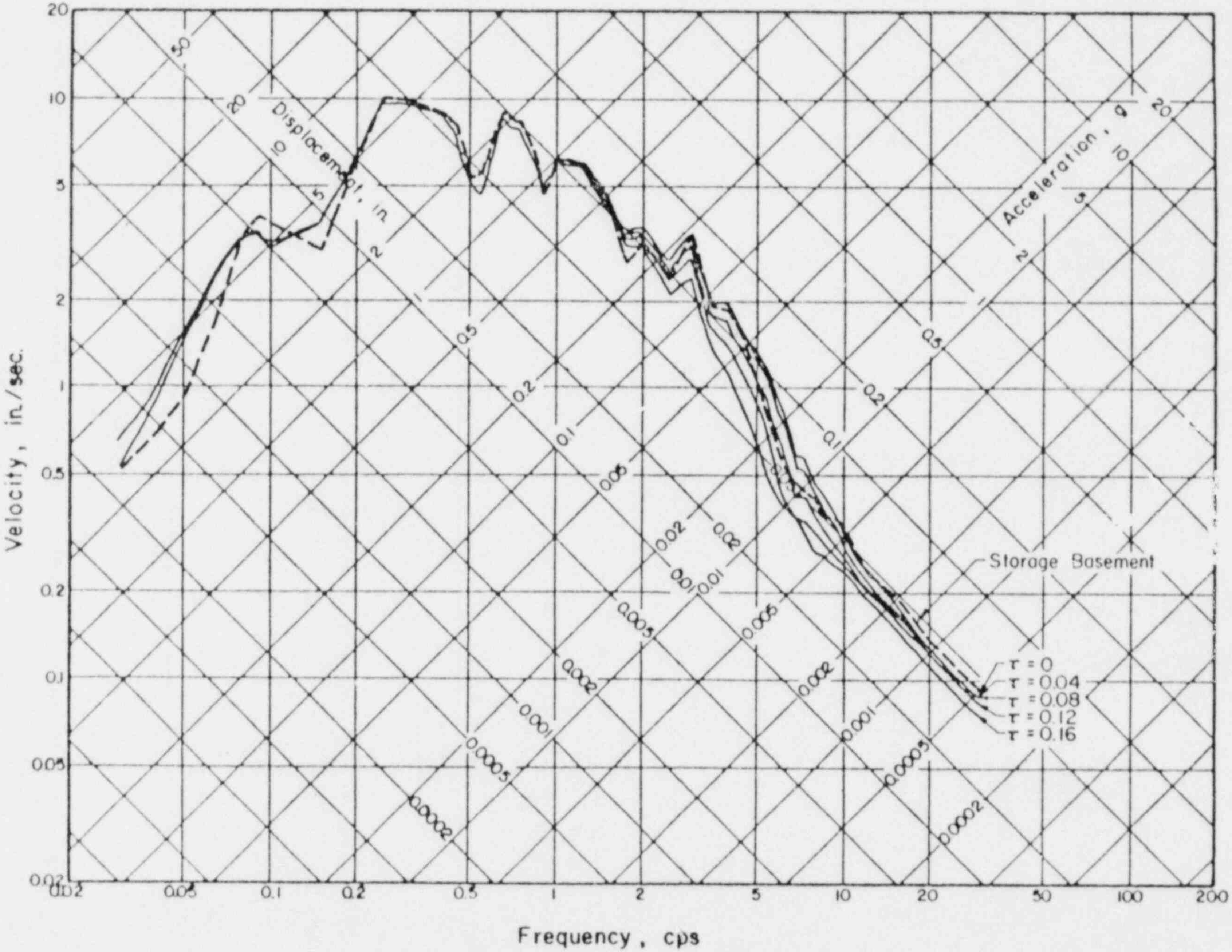


FIG. 14 HOLLYWOOD STORAGE P.E. LOT, KERN COUNTY EARTHQUAKE JULY 21, 1952, COMPONENT EAST, DAMPING 5 % OF CRITICAL, τ = 0, 0.04, 0.08, 0.12, AND 0.16 SEC.

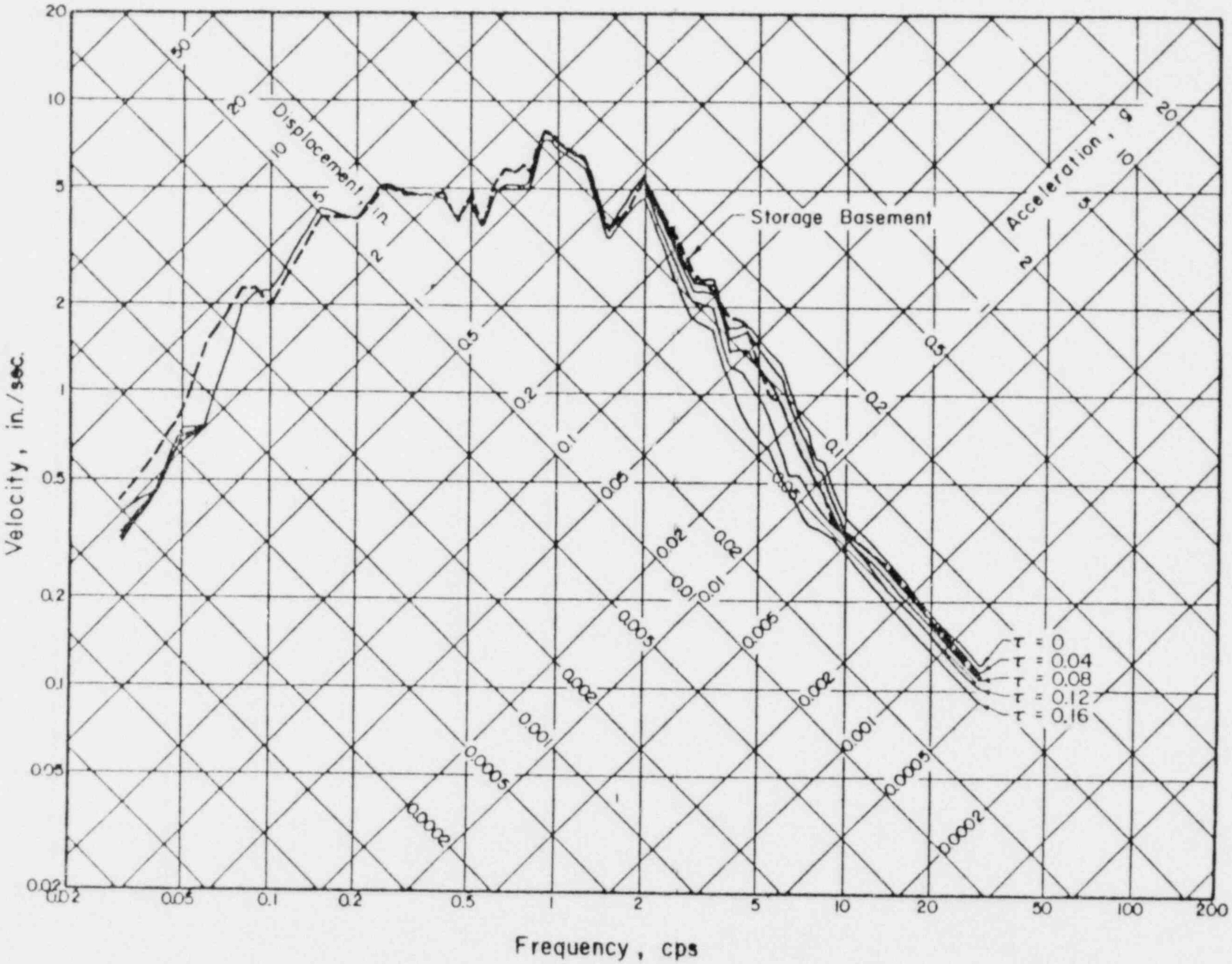


FIG. 15 HOLLYWOOD STORAGE P.E. LOT, KERN COUNTY EARTHQUAKE JULY 21, 1952, COMPONENT SOUTH. DAMPING 5 % OF CRITICAL, τ = 0, 0.04, 0.08, 0.12

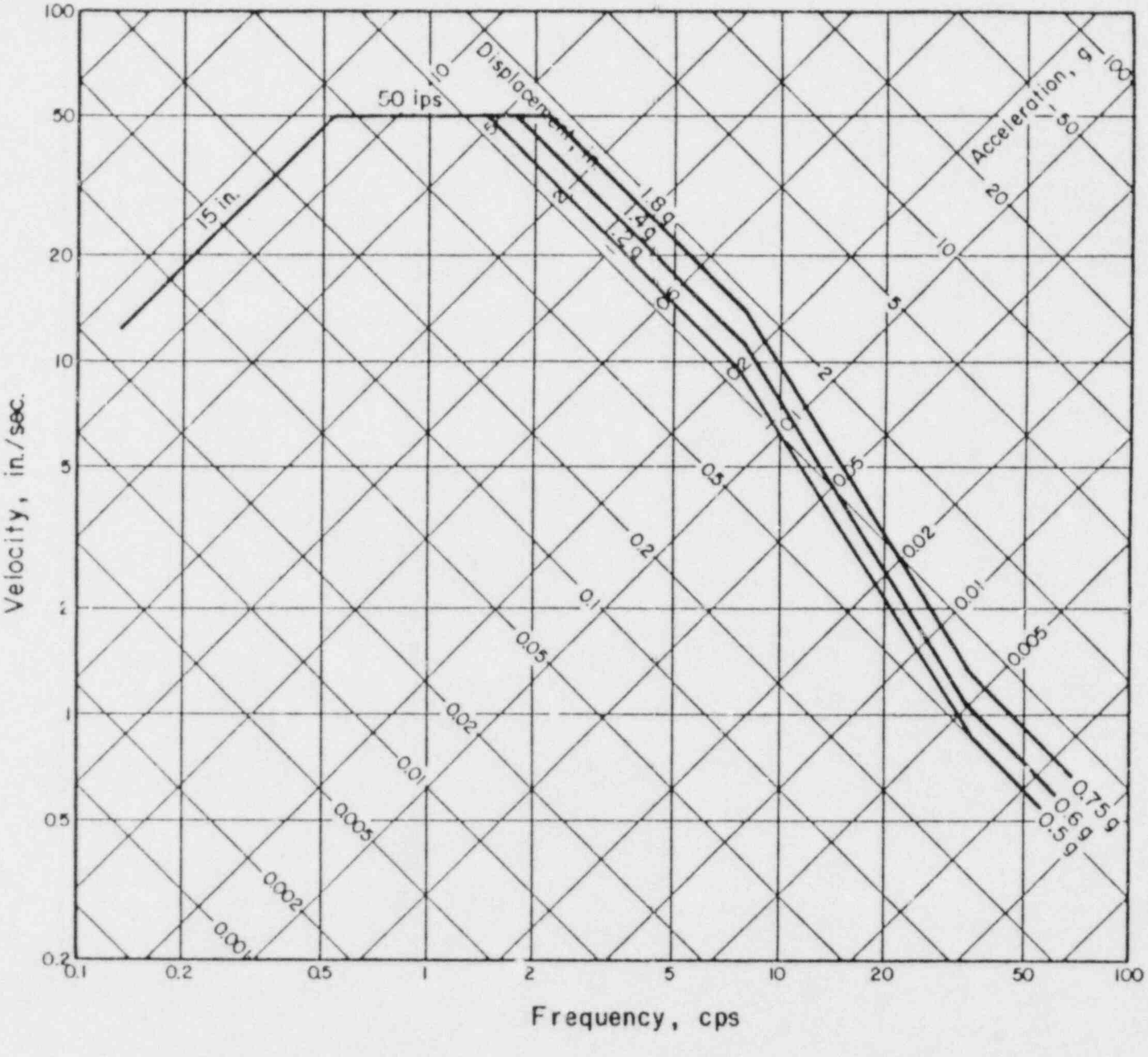


FIG. 16 RECOMMENDED "DESIGN" SPECTRA FOR PLANT

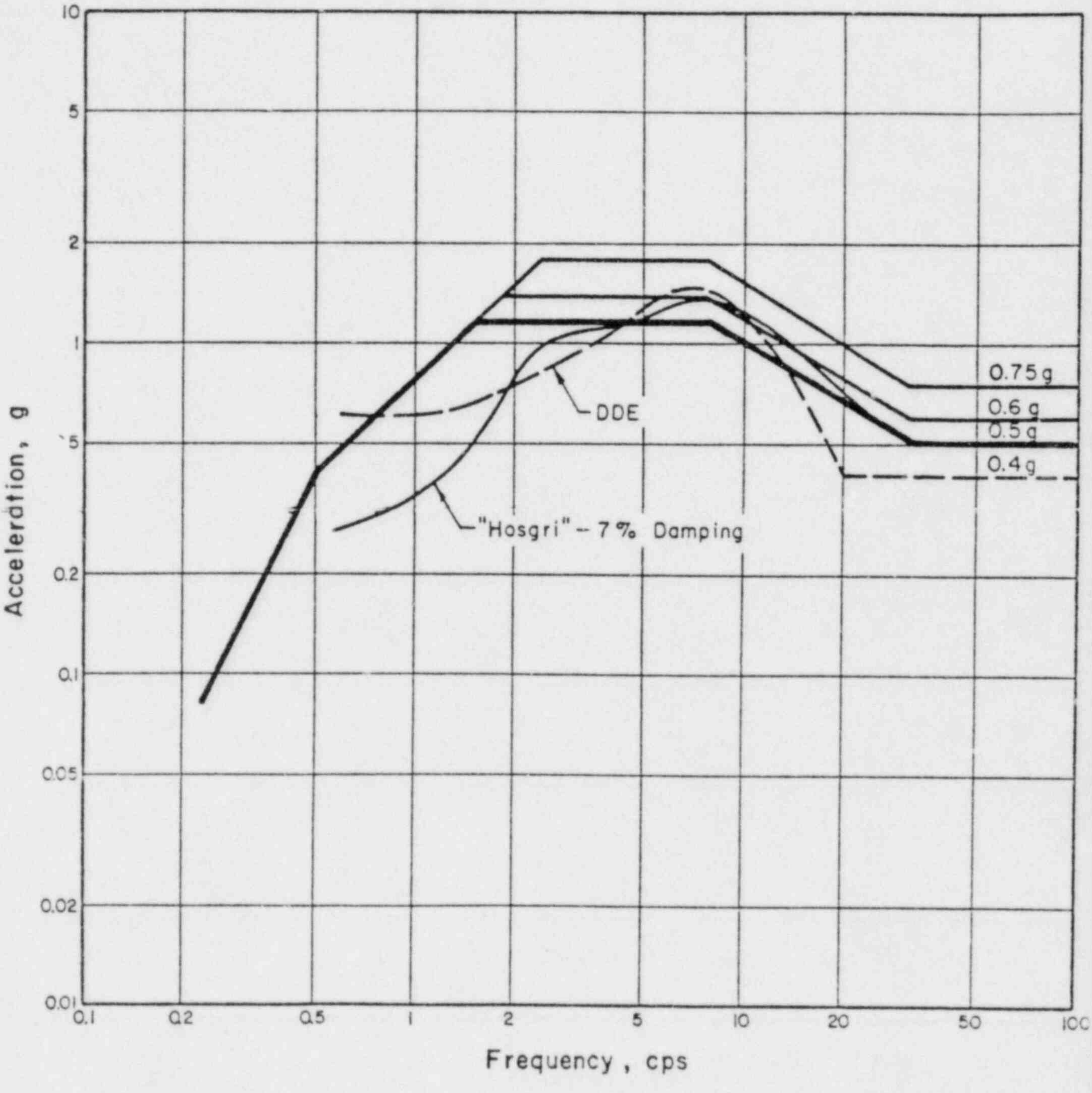


FIG. 17 RECOMMENDED "DESIGN" SPECTRA, 7% DAMPING, COMPARED WITH "HOSGRI" AND DDE SPECTRA

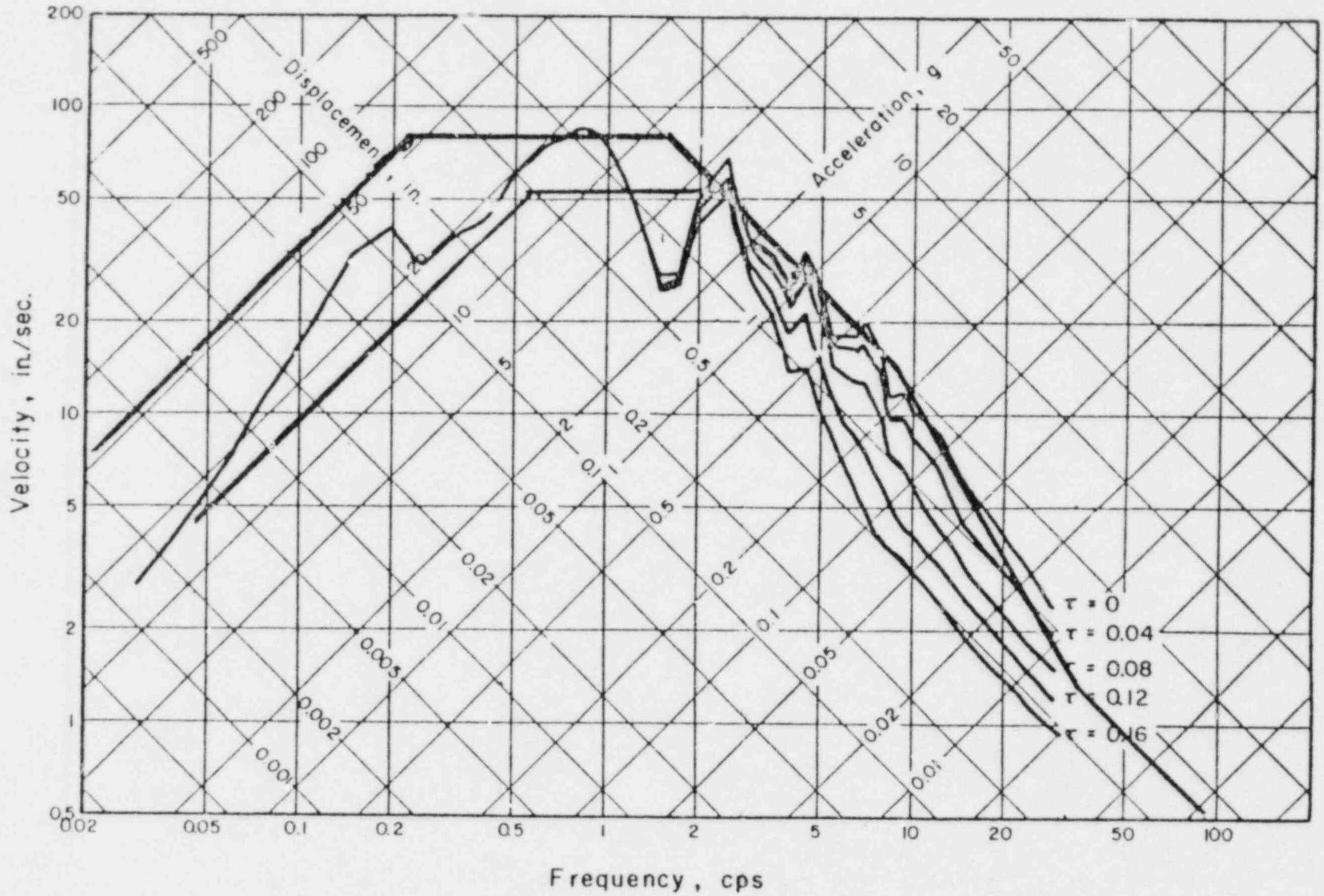


FIG. 18 PACOIMA DAM RESPONSE SPECTRUM 9 FEB 1971, S16E, DAMPING 5% OF CRITICAL, τ = 0, 0.04, 0.08, 0.12, 0.16 SEC. COMPARED WITH DESIGN SPECTRA

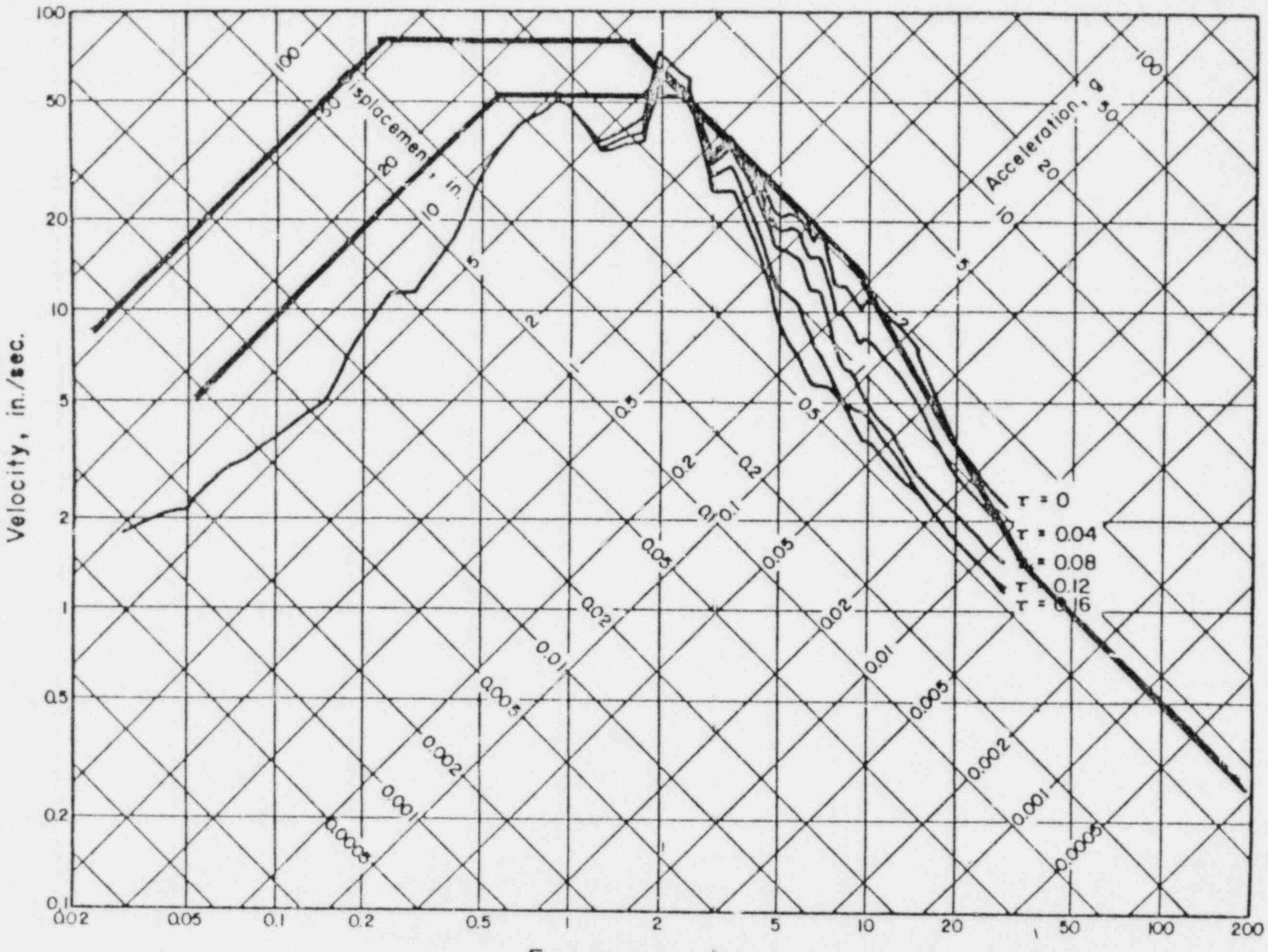


FIG. 19 PACOIMA DAM, SAN FERNANDO EARTHQUAKE, 9 FEB 1971, COMPONENT S74W, DAMPING 5% OF CRITICAL, τ = 0, 0.04, 0.08, 0.12, AND 0.16 SEC. COMPARED WITH DESIGN SPECTRA

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

REFERENCE "B"

REFERENCE B

The purpose of the studies that I have made for the Staff have been to consider the adequacy of the Diablo Canyon plant to resist the earthquake expected at the site.

I do not intend to discuss the design criteria for new plants or the general criteria or the standard review plan and things of this sort. The Diablo Canyon plant exists. It's completed. The question is whether or not it will operate safely. I will address that question only.

I would like to discuss some basic principles of earthquake resistant design and to point out that the earthquake engineer must take into account information of various kinds from various sources, not only the results of analyses but also observations, laboratory tests, and experience with similar kinds of phenomena, wherever information is available that might be pertinent.

Engineering design is an art, and part of the art is involved in deciding what is important, how important it is, what needs to be taken into account specifically, what assumptions can be made to design around uncertainties and, of course, to consider what can be done economically, whether one should construct anything at all under certain circumstances, whether over-design really provides safety or causes difficulties in other ways one does not expect. And in part this is rleated to our experience with the design and performance of ordinary structures in earthquakes.

Let me point out that hospitals, schools, major apartment complexes, structures in general that house a large number of people, or essential facilities that have to be designed to resist extensive loss of life, are designed by current criteria that are for the same earthquake exposure in terms of acceleration of gravity from 8 to 20 times less conservative than those applied to a nuclear power plant. As a matter of fact, it has only been reduced to 8 to 20 times less conservative in the past couple of years. Before that there was even a larger difference. I have been spending a major part of my time for the past 15 years trying to increase the design levels for ordinary structures. I think I have this to a point now where with the new ATC 3 Code which should be issued within a couple of months, we have increased the factors of safety pertaining to these essential structures that I am talking about, by factors of 1-1/2 to 2, over past procedures.

If I were convinced that earthquake effective accelerations for which a spectra can be drawn reach levels of 1.2 G or more I would go out and get as many people as I could to protest the horrible problems that would exist in every community in California that is near any fault or elsewhere in the country that is near a previous earthquake epicenter, to make a major change and revision in those structures. The calamities would be terrible, if an earthquake of that intensity, that really had that kind of spectrum, were to occur. So I think we have to look at what experience shows us in connection with earthquakes and what earthquake records mean and how they can be used. It is not only the theoretical calculations that

must be taken into account but their effects on structures that must be considered.

Expereince with effect of blast and shock on structures, experience with structures designed to resist the certain levels of ground motion from blast, experience in connection with structures designed to survive as well as those designed to fail have led to certain conclusions that have been taken into account in general by those engineers who have the responsibility for defining earthquake-resistant design criteria; experience in actual earthquakes has been taken into account.

Some of these things can't be done analytically, but certainly in a qualitative fashion. For example, we have a number of plants of various kinds that have been subjected to large earthquakes, some of them where there have been actual records obtained for the earthquake motions.

The Esso refinery in Managua is a good example. The pump stations in the Exxon pipeline in Italy, subjected to the Frioli earthquakes is another example. These were structures that were designed by ordinary codes, with perhaps the seismic design coefficient of the order of .05 to .08 G, as compared with a 1.1 or 1.2 that we get when we apply the spectrum to the values of ground motion that we get in the Diablo Canyon design.

The earthquakes that occurred had accelerations that were measured of the order of .35 G in Managua and perhaps more than that in Frioli. The Esso

refinery was able to continue operating with no damage to any of the equipment. The pump stations on the Exxon pipeline were able to continue operating without damage to the equipment. There have been other examples of similar nature. As a matter of fact, some earthquake engineers feel that proper constructions and attention to detail are more important than making elaborate analyses based on the kinds of assumptions that have to be made about earthquake motion. There is still a great deal of difference of opinion on what really happens in an earthquake. The theoretical calculations that are made are based on assumptions. They may depend on elaborate computer programs but that does not make them more accurate. The assumptions define the accuracy. And we don't know enough yet about source mechanisms and propagation mechanisms to be able to speak with any assurance of the validity of analyses made on strictly analytical grounds using elastic behavior concepts.

They are certainly not the case in the real world. They overestimate by far what actually happens. Even the response of instruments is suspect. Some years ago -- I can't remember exactly when, about six or seven -- I wrote a short paper on an explanation of why instruments recorded vertical motions in excess of those that the ground actually suffered. A very recent paper, I think published in February of this year in the Bulletin of the Seismological Society of America by Bruce Bolt and one of his students, explored this concept further and agreed with my conclusion that in many cases the instruments recorded perhaps 50 percent more than the actual vertical motion. Whether there is a similar response amplification

horizontally in the instrument, I am not prepared to say at this time.

But the earthquake in Russia, the Gazli earthquake, had a measured major vertical ground acceleration that might well have corresponded in accordance with Bruce Bolt's calculations, to an actual ground accleration considerably less than was measured and possibly even less than the measured 8/10 G. The Melindy Ranch instrument proably was another case of severe overrecording because of the mounting of that instrument on what Bolt calls a manure pile.

But in any event, one has to consider the fact that instruments themselves are responding elements, and they respond not only in terms of the response of the instrument itself but the mounting and the connection to the mounting to the soil. So one has to look at the data and interpret it properly and take it into account with big question marks and not believe only in the analytical results in making a design.

Another point that is of extremely great importance that has been raised by several of the ACRS consultants is the statement that if the magnitude of 7-1/2 really occurred, we would get 50 percent more acceleration than for a 6-1/2 earthquake corresponding to the Pacoima Dam record which I used for my estimate of the motions at Diablo Canyon. But the data indicate in general from Ancona, Melendy Ranch as we accept it, a few other cases, that the maximum acceleration is practically independent of the magnitude, very close to the source. It depends only on the nature of the breaking

strength of the rock and its ambient stress before, and the magnitude itself has little influence. When you go out some distance then magnitude has an important effect because it defines how much energy is focused to the point where we are concerned with placing a structure or responding element.

In many cases we are interested in building structures that are some distance away from the potential source of motion. For those cases it is quite appropriate to use the inferred maximum ground acceleration and a spectrum based on that in making a design. Close in, one must be somewhat careful about extrapolating on the basis of magnitude when there are no actual records and inferring motions that might be considerably greater than those that have actually been determined. There is only one record that I think actually is recorded, and is dependable, that shows acceleration greater than 1 G. That is the Pacoima Dam record. There is evidence, however, that that is the response of the little pinnacle of rock in that vicinity where the instrument was located, as is shown by the very good correlation of the frequencies of the motions and the peak amplitudes at particular frequencies that correspond -- that are correlated in the three directions of response. This is something that has not occurred in any other earthquake series of records that I know.

In any event, the response spectrum for the Pacoima Dam record, it is generally enveloped by and bounded by a Reg Guide 160 spectrum, based on .75g ground acceleration, except for frequencies that are of the order of more than 30 hertz.

There is no way of making an estimate of what the maximum acceleration might be really. It could be several times 1g very close to the source. I can even imagine it being almost infinite, right at the crack in a brittle material. But the velocities that occur and are generated by the motions at the source are limited, and they are limited by the fracture strain in the material through which the earthquake waves are propagated. Based on this concept, one arrives at maximum ground velocities of the order of 5, or 5 slightly plus, maybe up to 6 feet per second, that can be propagated.

Dr. Ambraseys of the Imperial College, London, arrived at numbers slightly lower, based on slightly different concepts. And in general, the velocity is a better measure of what happens to the response spectrum, because of the fact that it takes account of the energies that are available, rather than the peak acceleration, which might occur at a very high frequency. The actual effective peak occurs at a lower frequency. And we can infer the effective peak acceleration from that value which is consistent with the velocity response determined from the record more accurately than by any other means.

So, let me then close this part of the discussion by pointing out that the data that have been presented in various ways for design of major and

important structures and the way those data have been used by design engineers show the following trends: In general, the responses of structures as inferred by observed damage, damage to equipment as well as to structural elements themselves, damage from blast or other kinds of ground motion, all indicate that the response is indeed governed by something less than the peak instrumental recorded acceleration, when one is close to the source of motions. The values given in the circular 672, which has been referred to of the U.S. Geological Survey for the peak values are estimated values, extrapolated from observations.

Other values of similar nature are also extrapolated from observations. The methods of extrapolation are not generally agreed on. There are a large number of people, experienced seismologists included in them, who feel that the values that really affect the design are considerably less, and this is only a qualitative basis. Nevertheless, it is the basis for the use of what one calls an effective acceleration, rather than a peak acceleration. But one should not use an effective acceleration different from the peak unless one is very close to the source.

Here, the uncertainties are greater, the observations of damage, however, are consistent with lesser effects and the accelerations do not in fact, increase at such a tremendous rate compared with magnitude.

Let me go back in the history a little. In the design of the Bay Area Rapid Transit System, which is in San Francisco, as you know, and Oakland, surrounded by fault systems, the San Andreas on one side, the Hayward Fault on the other, actually one of the tunnels crosses the Hayward Fault, Dr. George Housner and I were members of the board of consultants who set the design criteria. We selected at that time .6g as the value to anchor the spectrum to and the value as being consistent with, and much more conservative than, the responses inferred from the damage in the San Francisco earthquake of 1906, (neglecting, of course, the effect of the fire, which was not a part of the earthquake).

Housner at that time had stated, in writings, that the maximum acceleration would not exceed .5g for earthquakes in California.

I disagreed with him.

We compromised on .6g as the number to use. We have reviewed this value for other cases of design near major faults in the San Francisco Bay area sewer outfall, which crosses the San Andreas Fault under water. Again we have used .6g. In the design of an addition to the Stanford linear accelerator, when they added a proton electron accelerator as part of the system, the same value was used. That structure is very close to the San Andreas Fault. Knowing full well that the ground accelerations might exceed the .6g, we recognized this was all that was necessary to design the structures for in order to have complete assurance that they could survive without loss of life or major damage.

The ATC code uses .4g as the maximum value in California. Of course, they make a major reduction for ductility, but with that, the factors that I mentioned before of a ratio of 8 to 20 for nuclear reactor design compared with other structures designed in California, such as school buildings, hospitals, fire stations and the like, would apply even if the design SSE value for the reactor were .4g. Generally, nuclear reactors use higher values of acceleration, so that the ratios increase even beyond 8 to 20.

These comments are based not only on my judgment and experience, but on the judgment and experience of other competent engineers, seismologists such as Clarence Allen, Bruce Bolt, and others with whom I have served on consulting boards for major structures (such as dams) in the California area.

I think it is not appropriate to raise this issue of whether the Diablo Canyon Reactor should be designed for 1.2 or 1.5g. I think the .75g is more than enough. It is amply conservative.

Now I would like to discuss the tau effect. Let's call the variation in acceleration over an area the tau effect. This does not necessarily have to do only with wave propagation effects. It has to do with the soil, the ground the earth, is inhomogeneous and scattering takes place. If you had an array of instruments mounted over an area, they would not simultaneously record the same maximums at all points. There would be differences in phasing, slight differences perhaps, but you would get differential values over the area. No matter whether the wave approaches from the bottom, from the side, from the middle or how. And the tau effect is only a way of trying to account for this in some systematic and reasonable fashion. One can do other things as well. It was developed essentially in relationship to the earthquake damage explanation by Yamahara, in 1970, in trying to explain the relative lack of damage in a number of structures in the Tokachi-Oki, earthquake in Japan, in 1968. There have been other rediscoveries of the same principle since then.

In part, I used that concept in developing somewhat earlier the paper that I wrote on torsion in earthquakes that was published in 1969 considering wave propagation effects in making an estimate of the accidental torsion in symmetrical buildings. The assumption of wave motion with time of travel, giving a transit time tau, does in fact give a variation over an area, and may be considered as an approximation to identify the net effect of the variations, but it should only be used for the purposes of an approximation.

The validity of the method depends upon comparisons with observations. Yamahara's comparisons in his paper and the data concerning the Hollywood storage building parking lot which I presented in one of my earlier reports are the only two examples that I know of to date. We need more experiments. By that I do not mean to say I would like to see more major earthquakes, especially in inhabited areas, but we need to have that kind of information and steps are being taken to develop arrays of instruments in various parts of the world where earthquakes may be expected, to give this basic information.

For the record, it should be noted that at the Hollywood storage building, the record was recorded at a corner of the building and therefore recorded not only the translational effect but the compounding of that with the torsional effect.

Now, so much for that tau effect. There is always a torsional effect, no matter where the earthquake occurs and there is a rotational motion of the ground. There has to be, unless the whole world moves at once horizontally and those waves are propagated vertically upward. Even if there is that sort of situation, with a variation in the time or phasing of the peaks of motion, there will still be a torsional effect.

There may be even a torsional effect due to ground torsion, other than that corresponding to the wave effect, because theoretically there must be six components of motion at a point, three translations and three rotations. There are no basic data on variations from point to point, especially at high frequencies. The wave propagation leads only to a proper measurement of variation in movement, but it gives a reasonable result and reasonable comparisons with observations for low frequencies. The systematic variation at the high frequencies, especially when one bases the variance on the wave propagation effects, overestimates the torsional response because it involves this very systematic variation which is always affected by the spatial variation even with that. Moreover, to make the assumption that the earthquake wave motion always comes in the worst way to give the maximum torsional effect, thereby

giving an upper boundary is also somewhat unreasonable. Even under these circumstances, there is an energy loss at the interface between a structure and the ground or rock or soil on which it is based. This energy loss has not been taken into account in any soil structure interaction analyses.

There is a well-accepted basis by engineers over the world that some allowance must be made for torsional effect in structures. This allowance in building codes, in the United States, in Canada, in New Zealand and in Mexico generally states that 5 percent of the greatest dimension of the building should be used as an eccentricity to make a static correction for the torsional effect by considering the translational force applied with that eccentricity in design. The analyses that I made some years ago indicated that 5 percent might be increased to 10, 15, or even 20 percent in some circumstances. But those circumstances corresponded to conditions which I regarded as unreasonably simplistic that did not represent reality and were not consistent with observations.

The damage from torsional effect generally has been confined to structures in which there is either a large amount of real eccentricities, like with unsymmetrical shear walls, or where all of the torsional resistance is concentrated in a central core where there is very little or insufficient torsional resistance even for 5 percent.

Structures in a reactor facility, generally with only one exception that I know of, and I won't dwell on that -- it's not part of the Diablo Canyon reactor -- are structures in which the major part of the resistance comes in the external walls and, therefore, the effect is not as important as it would be for a building in which all of the torsional resistances are concentrated around an elevator shaft at the center. There are ways by which one can make apparently more precise calculations. However, I do not believe that these are any more believable, because the assumptions on which those calculations are made are not defensible. I think they are reasonable approximations. These calculations should be taken into account qualitatively, but I would not be willing to use them quantitatively in any assessment of design.

I feel it is more important to have a proper allowance made in such a way that one can depend on being there and that the static torsional, additional effect taken into account, both in terms of stress in the structure and in terms of response at the outer edges where equipment is mounted, can be adequately defined in this way.

Diablo Canyon structures have been reviewed for torsion and in general some preliminary analyses in a study just underway by one of Dr. Hall's and my graduate student, Mr. James Morgan, indicates the general nature of the results that one might expect and confirms those indications that come from the earlier studies.

In these studies, we took several earthquake records and considered the time history of these. We then averaged the torsion and translational responses by actually computing the best torsional translational response by a method of the least squares, and finding the translational response as affected by the tau effect considering a vertical wave propagating horizontally, computing the torsional effect alone, and computing the combined effect at one side of the structure, taking the time history that gives simultaneously the combined effect and then comparing this with what we get when we either add or take the square root of the sums of the squares of the two effects.

And we find that there is a good agreement between the various combinations. The maximum effect in time agrees very well with the square root of the sums of the squares of the combinations, combined -- or the separate effects. And to be sure, when one adds back the torsional effects, taking into account the fact that there is a reduction due to the translation, the direction of the translational effect due to the wave propagation factor, tau, over some relatively small range in frequencies, there is sometimes a slight additional effect over that response which is calculated without any translational effect.

Under those cases, it is assumed, however, that the torsional frequencies of the structures are the same as the translational frequency. That overestimates the factors.

As shown in my paper "Torsion in Symmetrical Buildings" published in the Proceedings of the Fourth World Conference on Earthquake Engineering, Santiago, Chile, 1969. In general the torsional frequency is different from the translational frequency, and when one takes this difference into account the factor reduces.

I would not be satisfied with the Diablo Canyon analyses that only took the tau-effect and neglected the torsion. Neither would I be satisfied with the design of a reactor 50 miles from a fault that didn't take accidental torsion into account. It has not been until recently that the staff has required accidental torsional calculations to be submitted. I think that they will probably do so in the future for all cases. I think it's a desirable and necessary thing to do.

I feel very strongly about the fact that the design criteria for the Diablo Canyon reactor, based on the original concept for design and the retrofit proposed when reviewed in the way that these have been reviewed, and looked at very carefully by a number of people on the staff and in various consulting firms employed to make this review, I think that these designs are more adequate than that of most of the other reactors that have not undergone this intensive audit. I feel that we have made major conservative assumptions and that the state of the art of nuclear reactor design as reflected in current practices, not those of 20 years ago, gives an adequately conservative design.

I started back in 1964, to try to increase the then existent response spectra used in reviewing the design of nuclear reactors. Shortly after I started as a consultant for NRC, these inquiries were made in two steps. One was a change in the use of the so-called Housner spectrum, by a factor increase of one and a half to two. That took place about 1965 or '66. Then a later increase that results from a paper that I presented in 1967 at the IAEA, International Atomic Energy Agency meeting in Tokyo, and these were changed only slightly with the studies that were made by Dr. Blume and my group, and presented essentially as Reg Guide 160 and 161.

I have since revised my way of doing things, to take account more specifically of site-related spectra. The so-called Newmark spectra differs from the Reg Guide spectra only in using lines parallel to the coordinate lines of the tripartite logarithmic graph. It's more conservative in some areas, slightly less in others that are not as important, but it does have the feature that where there is information available for the maximum acceleration, maximum velocity and maximum displacement, one can take this into account in developing a site-related spectrum that may be less conservative in some cases or more conservative in others, than the so-called Reg Guide spectrum.