

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION
BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

JAN 26 P131

In the Matter of

SOUTH CAROLINA ELECTRIC & GAS
COMPANY (Virgil C. Summer Nuclear
Station, Unit 1)

Docket No. 50/395 OL

APPLICANTS' ADDITIONAL SEISMIC TESTIMONY

December 18, 1981



SOUTH CAROLINA ELECTRIC & GAS COMPANY

SOUTH CAROLINA PUBLIC SERVICE AUTHORITY

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NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of:)
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SOUTH CAROLINA ELECTRIC)
& GAS COMPANY, et al.) Docket No. 50-395 OL
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(Virgil C. Summer Nuclear)
Station, Unit 1))

APPLICANTS' MEMORANDUM OF TRANSMITTAL OF
SUPPLEMENTAL FILED TESTIMONY
AND DESIGNATION OF EXHIBITS ON
SEISMIC QUESTIONS WITH ATTACHMENTS

Pursuant to the Board's "Notice of Rescheduling of Hearing," dated December 8, 1981, and the schedule approved in our conference call of November 18, 1981:

(1) Applicants herewith file prepared direct testimony, including professional qualifications (unless previously filed), of the following witnesses with reference to the Board questions on seismic issues:

John A. Blume (URS/John A. Blume & Assoc., Engineers)

(with qualifications attached)

1. Testimony of John A. Blume, Ph.D.
2. Appendices^{1/}
 - a. B-2, On the Relationship of Peak Ground Acceleration, Intensity, and Local Magnitude.
 - b. B-3, Application of the "SAM" equations.
 - c. B-4, Regarding USGS Open File Report 81-365.

^{1/} There is no Appendix B-1, B-6, B-8 or B-13 (the data were included in the listed appendices or in general testimony).

- d. B-5, Amplification of Motion at Oroville Dam.
- e. B-7, The Strong Motion Record of October 16, 1979, at Monticello Reservoir and Structural Response.
- f. B-9, On Directional Components of Ground Motion.
- g. B-10, Material Strength 1) Attachment B-10A, Concrete Properties of the Major Structures at the Virgil C. Summer Plant. 2) Attachment B-10B, Reinforcement Steel Strength Data for Major Structures at the Virgil C. Summer Plant. 3) Attachment B-10C, Structural Steel Strength Data for Major Structures at the Virgil C. Summer Plant.
- h. B-11, Some Lessons From RIS at Hsinfengkiang Reservoir.
- i. B-12, Generic LL Documents for Diablo Canyon and the El Centro Steam Plant.
- j. B-13, The Effect of Variations in Peak Ground Velocity and Displacement on the Virgil C. Summer Structures and Equipment.
- k. B-14, Ground Acceleration Versus Damage, Project Rulison.
- l. B-15, Unrecognized Margins.

Otto W. Nuttli (St. Louis University) (with qualifications attached)

- 1. Testimony of Otto W. Nuttli, Ph.D.
- 2. Appendix A, Epicentral Intensity, Magnitude and Depth of the July 26, 1905, Upper Michigan Earthquake.
- 3. Appendix B, on the Question of Magnitude Versus Depth of Very Shallow Earthquakes in the Eastern United States.

Robin K. McGuire (ERTEC Rocky Mountain, Inc.)

- 1. Summary - Effects of Reservoir-Induced Seismicity on the Virgil C. Summer Nuclear Station.
- 2. RM-1, Response Spectra Shapes for Reservoir-Induced Seismicity at Virgil C. Summer Nuclear Station.

3. RM-3, Mathematical Model Used to Estimate Peak Acceleration at Virgil C. Summer Nuclear Station.^{2/}
4. RM-4, Probability Estimates of Seismicity and Ground Motion at Virgil C. Summer Nuclear Station.
5. RM-5, Processing and Analysis of Accelerograms From Aftershocks of the 1975 Oroville, California Earthquake.
6. RM-6, Estimates of Peak Acceleration Using Brune Seismic Source Model.
7. Applicant Evaluation of Joyner-Fletcher Report on Virgil C. Summer Nuclear Station Seismicity Studies.
8. Applicant Evaluation of Luco Report on Virgil C. Summer Nuclear Station Seismicity Studies.

Malcolm R. Sommerville (URS/John A. Blume & Assoc., Engineers)

(with qualifications attached)

1. Comparison of Free-Field (Saprolite) and Foundation (Bedrock) Motions Recorded In Two Explosion Tests at the Virgil C. Summer Nuclear Station.
2. Applicant Evaluation of Trifunac Report on Virgil C. Summer Nuclear Station Seismicity Studies.

Geoffery R. Martin (ERTEC Western, Inc.) (with qualifications attached)

1. Testimony of Geoffery R. Martin, Ph.D. (re: Soil Amplification Studies)

Shelton S. Alexander (Penn. State University)

1. Some Characteristic Seismic Source Parameters for Eastern United States Earthquakes and Induced Earthquakes at Monticello Reservoir.

^{2/} There is no report RM-2. Jenkinsville accelerograph data has been incorporated in Applicants' response to FSAR Question 361.23.

James McWhorter (Dames & Moore).

1. Comparison of Global Reservoir Induced Seismicity (RIS) to Piedmont RIS Experience.
2. Charleston 1886 Earthquake: Summary of Materials Presented in Connection with NRC Review of V.C. Summer Nuclear Station.

Chang Chen (Gilbert/Commonwealth Companies).

1. Testimony of Chang Chen, Ph.D.

(2) In addition, Applicants will make available for questioning, as panel members, the following witnesses whose qualifications are filed herewith (unless previously filed):

Pradeep Talwani (University of South Carolina).

Dilip Jhaveri (URS/John A. Blume & Associates, Engineers)

(with qualifications)

(3) It should be noted that Applicants will, in accordance with the usual practice, present some or all its witnesses in panels, and that one individual may appear on more than one panel.

(4) Applicants, may of course, call any of the witnesses named herein for redirect or rebuttal, or call additional witnesses for rebuttal, to respond to the Board as required, or as developments may warrant.

(5) Applicants hereby designate the following exhibits as part of their direct case. Copies (including attachments or enclosures to letters) are enclosed as volume two of this submittal enclosed:

Exhibits

(a) Applicants' Exhibit 41:

Description of Seismometric Data Recorded at Mammoth Lakes, California.

(Submitted under cover of letter from Nichols to Denton dated November 19, 1981, in response to NRC Staff request for additional seismic information)

(b) Applicants' Exhibit 42:

Virgil C. Summer Nuclear Station Active Field Experiments Report.

(Submitted under cover of letter from Nichols to Denton dated November 19, 1981, in response to FSAR Question 361.24)

(c) Applicants' Exhibit 43:

Accelerograph Deployment Information and Records Obtained at Monticello Reservoir, South Carolina.

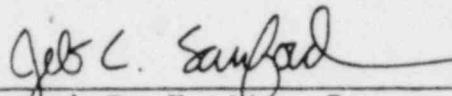
(Submitted under cover of letter from Nichols to Denton dated December 2, 1981, in response to FSAR Question 361.24)

(d) Applicants' Exhibit 44:

Mammoth Lakes Earthquake Response Spectra for a Nearby
Magnitude 4.5 Event.

(Being submitted to the NRC Staff in response to
FSAR Question 361.25)

Respectfully Submitted,



Joseph B. Knots, Jr.
Jeb C. Sanford

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 - h. B11 - Some Lessons from RIS at Hsinfenkiang Reservoir

^{1/} There is no appendix B1, B6, B8, or B13. The material contained in those reports have been incorporated into other appendices or the general testimony.

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 6. Applicant Evaluation of Joyner-Fletcher Report on Virgil C. Summer Nuclear Station Seismicity Studies

^{2/} See II, A. above for "Summary" by Dr. McGuire

^{3/} The Report RM-2 on Jenkinsville Accelerograph records has been incorporated in Applicants' Response to FSAR Question 361.23

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III. Designation of Exhibits

- A. Applicant's Exhibit 41 - Description of Seismometric Data Recorded at Mammoth Lakes, California (submitted under cover of letter from Nichols to Denton dated November 19, 1981 in response to NRC Staff request for additional information)
- B. Applicants' Exhibit 42 - Virgil C. Summer Nuclear Station Active Field Experiments Report (submitted under cover of letter from Nichols to Denton dated November 19, 1981 in response to FSAR Question 361.24)

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- C. Applicants' Exhibit 43 - Accelerograph Deployment Information and Records Obtained at Monticello Reservoir, South Carolina (submitted under cover of letter from Nichols to Denton dated December 2, 1981 in response to FSAR Question 361.23)

- D. Applicants' Exhibit 44 - Mammoth Lakes Earthquake Response Spectra For a Nearby Magnitude 4.5 Event Recorded at Site Conditions Most Similar to the Summer Site (to be submitted to the NRC Staff in response to FSAR Question 361.25).

SUMMARY -- EFFECTS OF
RESERVOIR-INDUCED SEISMICITY
ON THE
VIRGIL C. SUMMER NUCLEAR STATION

by Robin K. McGuire

INTRODUCTION

Much has been said and written in this proceeding about induced seismicity at Monticello Reservoir, and the potential effects of that seismicity on the Virgil C. Summer Nuclear Station. This document is intended to clarify the bases for the position of the Applicants that these small, low energy events of short duration pose no seismic hazard to this facility. Also documented herein, where relevant to the understanding of certain issues, are facts related to the history of the seismic evaluation and its review by the various parties involved.

EARTHQUAKES AND DAMAGE

It is important to view the current issues of concern in the context of world-wide experience regarding earthquakes and the damage they cause. Nowhere, to the Applicant's knowledge, is there evidence of earthquakes with magnitudes less than five causing damage to engineered structures or equipment whether or not any seismic design was involved. This is documented by testimony submitted by Dr. J.A. Blume. As almost all parties agree that reservoir-induced seismicity (RIS) at Monticello Reservoir will be limited to magnitudes less than five, there should

there should be no concern for the seismic safety of the facility. This is particularly the case because nuclear facilities are very carefully designed to resist earthquake motions, and RIS is not a new phenomenon (although the physical causative mechanisms have only been studied in recent years). If earthquakes of these magnitudes, whether reservoir induced or not, were capable of inducing damage, empirical evidence of such damage would be available, and there is simply no such empirical evidence from anywhere in the world. Additional insight can be obtained from evidence at Monticello Reservoir itself. The earthquakes which have occurred have caused no damage to either the Virgil C. Summer Nuclear Station or the Fairfield Pumped Storage Facility (located close to the SMA on the dam abutment). This is documented in the Applicants' response to FSAR question 361.23 dated December 2, 1981, and is provided as Exhibit 43. The observation of no damage to the Fairfield Pumped Storage Facility is particularly relevant because this facility was not designed or constructed to the same high standards as the nuclear plant with regard to seismic motion, and the RIS earthquakes have occurred very close to it. (The pumped storage facility lies, in location, between the strong motion instrument and the epicenters of earthquakes which have produced the largest acceleration records.)

One of the reasons for the lack of damage during earthquakes of magnitude less than five is the built-in safety margins of engineered structures. The margins

existing in typical structures are discussed in testimony presented by Dr. Blume. The fact that such margins are inherent in the Virgil C. Summer Nuclear Station is documented in testimony submitted by Dr. C. Chen and in Blume Appendix B10, B15. These margins have led to the frequent observation that real structures subjected to earthquake motions do not fail or sustain damage of the levels which we would predict from theoretical studies but rather have substantially more strength and incur significantly less damage. This was the case, for example, at the El Centro Steam Plant during the 1979 Imperial Valley earthquake. This example and others are documented in testimony submitted by Dr. Blume.

A second reason for this lack of damage, and the major cause of debate at Monticello Reservoir, is that levels of damage are not closely related to peak instrumental acceleration or elastic response spectrum representations of earthquake ground motion. Such representations provide a convenient and conservative means of seismic design for structures and equipment when the earthquakes of concern are of magnitude six or seven, involving ten or twenty seconds of strong ground motion. Peak instrumental acceleration and elastic response spectra are inordinately conservative and overestimate levels of damage to a great extent when the ground motions of issue are high frequency oscillations which last on the order of one second or less, such as those RIS events occurring at Monticello Reservoir. Because the

Nuclear Regulatory Commission (NRC) guidelines suggest the use of peak acceleration and elastic response spectra to represent earthquake ground motions, the debate at Monticello Reservoir has centered on what might be appropriate levels of peak acceleration and elastic response to represent ground motions from RIS, not what might be the damage associated with those ground motions. There is a substantial body of evidence concerning earthquake damage (testimony of Dr. Blume) which indicates the conservatism inherent in peak instrumental acceleration representations of RIS ground motion, and this should be factored into any decision regarding seismic safety.

It is convenient to divide the discussion of specific issues related to RIS at Monticello Reservoir into two categories. The first category is shallow-depth earthquakes of magnitude less than four, which the Applicant has concluded are the largest which can be induced by Monticello Reservoir. The evidence for this position is based on geological and geophysical observations at Monticello which are documented in testimony by Prof. S. Alexander (submitted in June 1981) and in the Supplemental Seismologic Investigation report submitted by the Applicant. These events are of concern only if they are very shallow (in the top several kilometers of the earth) and close to the facility. They are discussed in the next section, entitled "Shallow Earthquakes."

The second category consists of larger earthquakes (up to magnitude of about 5) which have been suggested by various parties as upper limits to RIS at Monticello Reservoir. These events must necessarily occur at deeper, more normal depths for crustal earthquakes. Observations of earthquakes within the central and eastern U.S. support this conclusion (testimony of Prof. O. Nuttli). These normal-depth earthquakes are discussed in the section entitled "Normal-Depth Earthquakes."

SHALLOW EARTHQUAKES

The RIS at Monticello Reservoir has all been of shallow depths, i.e., 98 percent of the events less than 2 km deep (Supplemental Seismologic Investigation report). Geological and geophysical observations indicate that the magnitudes of such events will be limited to less than four (testimony of Prof. Alexander submitted in June 1981 and Supplemental Seismologic Investigation report). Moreover, probability studies using both site-specific data and regional data indicate that a magnitude 4 at Monticello in the vicinity of the facility would be a very rare event, i.e., it is unlikely (testimony labelled RM-6 of Dr. R.K. McGuire). Experience from earthquakes throughout the entire central and eastern U.S. suggests that magnitude 4 is the upper limit for such shallow earthquakes (testimony of Prof. Nuttli). Furthermore, earthquakes in the range 3-1/2 to 4

must occur at a depth of 1-1/2 km or more, based on observations of earthquake depths and magnitudes in the central and eastern U.S. (Prof. Nuttli, Appendix B).

A general discussion regarding earthquakes of this size and distance is warranted. First, historical evidence indicates that such events cause, at maximum, Modified Mercalli Intensity VII effects, which involve nothing more severe than superficial damage to engineered structures (the two exceptions involve earthquake sites affected by mining activities). These observations are discussed in testimony prepared by Prof. Nuttli. The reason for this lack of damage is the high frequency, short duration characteristics of the ground motion during these earthquakes.

A second set of observations relating damage to ground motion of this type is available from such studies during underground nuclear blast testing. These studies indicate that very little damage is induced in structures (including non-engineered structures) even during relatively high levels of ground shaking (Dr. Blume Appendix B14). These high frequency, short duration ground motions may produce large peak accelerations and even high spectral amplitudes at high frequencies, but they have a fraction of the energy associated with an earthquake of the same peak acceleration lasting ten or more seconds. Thus the short duration motions are not observed to induce significant damage to structures. A distinction between the two types of motion can be made using concepts of "effective acceleration",

rather than "instrumental acceleration" (testimony of Dr. Blume).

In addition to these general observations regarding the level of damage associated with these small earthquakes, the Applicant has, at the request of NRC, made quantitative estimates of ground motion amplitudes for such events. This has been accomplished using all accurate strong motion data available from the dam abutment instrument site at Monticello Reservoir. It should be evident that the most reasonable interpretation of ground motion amplitudes can be made only by using all data available, rather than by analyzing only one or several records. The Applicant has identified six strong motion records from the USGS SMA-1 instrument which are of sufficient quality to be used for analysis (FSAR Question 361.23). A dynamic amplification of the strong motion instrument site was discussed previously by the Applicant (in the "Applicant Evaluation, Luco Report," dated October, 1981), but in the early stages of this seismic evaluation was not accounted for quantitatively. This resulted in excessive conservatism of ground motion estimates made for rock sites (with no amplification). Recently the Applicant has demonstrated this amplification through an active field experiment (FSAR Question 361.24) and testimony of Dr. M. Somerville) and through computer studies of site response (Testimony of Dr. Martin). Thus more accurate estimates of earthquake ground motions are made by modifying observations at the dam abutment site

by a factor of approximately 0.5 to estimate observations at an equivalent bedrock site.

An analysis of the six strong motion records indicates that a seismic source model (testimony labelled RM-3 of Dr. McGuire) with a stress drop of 25 bars (FSAR Question 361.23) is appropriate and conservative. Estimates made in this way may be biased upward (on the conservative side) because the available observations may involve anomalously large stress drops (smaller stress drops do not trigger the instrument to produce a usable record) and the value of 25 bars probably reflects extreme radiation and path effects as well as general source effects. Uncertainties in radiation and path effects are properly taken into account by using a mean-plus-one-standard-deviation ($M + \sigma$) response spectrum, which the Applicant has done. Therefore, the use of a 25 bar stress drop, which also includes these effects, introduces added conservatism. It is worth noting that neither Dr. Joyner (deposition dated 11/17/81, p. 88 and 90) nor Prof. Luco (deposition dated 11/19/81, p. 53) disagree with the Applicant's position that 25 bars is an appropriate stress drop for the 8/27/78 earthquake (the only data analyzed at that time) when no account is taken of soil amplification and an upper cut-off frequency of 40 or 50 hz is used (i.e., the record is digitized at an adequate sampling rate). Thus all strong motion data obtained at Monticello Reservoir to date support the ground motion model and its parameters used by the Applicant. Estimates

of ground motion using this model indicate peak accelerations on the order of 0.15g (the SSE acceleration for the facility) for an $M_L = 4$ RIS earthquake; estimates up to approximately 0.22g are obtained for extreme choices of source parameters and source-to-site distance (testimony labelled RM-6 of Dr. McGuire). The Applicant has analyzed the effects of such ground motions (0.22g peak acceleration and magnitudes of 4 to 5) and has shown that they present no safety hazard to the facility (testimony of Dr. Chen).

In conclusion, the Applicant has examined both site-specific and regional data, and has presented evidence that magnitudes no greater than 4 will occur at shallow depths in the vicinity of Monticello Reservoir. The ground motion from such earthquakes does not present a safety problem for the facility: this is demonstrated by lack of damage from such earthquakes in the past, lack of damage from similar ground motion induced by man-made blasts, and by quantitative estimation of elastic response spectra of such motion and its effect on the facility.

NORMAL-DEPTH EARTHQUAKES

The NRC staff has concluded that a magnitude 4.5 earthquake is possible as a result of the influence of the reservoir, and the ACRS sub-committee has recommended that the facility be analyzed assuming a magnitude 5 earthquake could be induced by the reservoir. While such an event is considered impossible by the Applicant (all seismicity to date has been shallow, there has been no migration

of seismicity to deeper depths during the almost four years of seismic monitoring, and events greater than magnitude 4 have not been observed anywhere in the central and eastern U.S. at these shallow depths), the Applicant has agreed to analyze the effects of these postulated earthquakes on the facility.

The most direct estimates available of ground motion in the magnitude 4 to 5 range is from events of similar magnitude in other parts of the country which have been well-recorded. An example is the sequence of earthquakes which occurred at Mammoth Lakes, California, in mid-1980. The Applicant has analyzed a series of strong-motion records from a single site at Mammoth Lakes (the one available which most closely represents a bedrock site condition) which recorded motions greater than those obtained at nearby alluvium sites through which the ground motion was attenuated. These analyzed motions, which show some site amplification at certain frequencies because of wave guide and/ or site effects, indicate that the $M + \sigma$ response spectrum for magnitudes in the range four to five matches the mean + σ RIS spectrum estimated by the Applicants for magnitude 4.5 (FSAR question 361.25). The Applicants' RIS spectrum has been shown to cause no safety concern for the facility (testimony of Dr. Chen).

Another set of data is available from the sequence of aftershocks of the 1975 Oroville, California, earthquake. A statistical analysis of motions recorded during magnitude

4 to 5 earthquakes on rock sites for this sequence indicates, again, that the mean $\pm \sigma$ spectrum developed by the Applicant for a magnitude 4 to 5 earthquake envelopes the mean $\pm \sigma$ spectrum of recorded events in this magnitude range (testimony of Dr. McGuire).

A further set of observations is available from Hsin-fengkiang Reservoir in China. Peak accelerations recorded during reservoir-induced earthquakes at this location indicate that similar ground motions at Monticello Reservoir would cause no safety concern for the facility (Dr. Blume Appendix B11).

Another indication of instrumental ground motions to be expected during magnitude 4 to 5 earthquakes can be obtained from studies of a large number of earthquakes in California. Representative is the study published by Drs. Joyner and Boore of the U.S. Geological Survey. They find that the expected maximum acceleration (larger of two components) during a magnitude five earthquake would be approximately 0.22 g for a site located near the source (at zero epicentral distance). Analysis of a magnitude five, mean $\pm \sigma$ spectrum scaled to this peak acceleration indicates that the seismic design of the facility is adequate to ensure safe shutdown during such an earthquake (testimony of Dr. Chen).

A final set of comparisons is available in the form of peak accelerations estimated for normal-depth earthquakes in the eastern U.S. (testimony of Prof. Nuttli). Conclusions from several investigators are consistent. The estimates

developed by Prof. Nuttli indicate peak horizontal acceleration in the near-source region of magnitude 5 earthquakes to be approximately .11g, which is less than the SSE for the facility.

In summary, there are several lines of evidence available to investigate the seismic safety of the facility during magnitude 4 to 5 earthquakes. Recorded response spectra for earthquakes of these magnitudes in California and generic studies of a wide range of earthquakes in California, and in the eastern U.S. all indicate that such events near the Virgil C. Summer Nuclear Station pose no hazard to the public health and safety.

TESTIMONY OF
JOHN A. BLUME, PH.D.
ON BEHALF OF
SOUTH CAROLINA ELECTRIC & GAS COMPANY
BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

My name is John A. Blume. I am Senior Consultant and Chairman of the consulting firm URS/Blume with its main office in San Francisco. A statement of my qualifications and relevant experience is attached hereto. My experience pertinent to earthquake matters for the Virgil C. Summer facility includes a 50-year span of pioneering and continued research, study, code development, design, consultation, writing, lecturing, reports, and testimony in earthquake engineering and structural dynamics. I have been consultant to the Nuclear Regulatory Commission and to industry in licensing matters and was the principal earthquake engineering consultant for the applicant on the Diablo Canyon Nuclear Power Plant, Units 1 and 2. Our firm has variously developed seismic criteria, done dynamic analyses of soil, rock, structures, equipment and piping, and structural design for about 80 nuclear plants and other major facilities in many parts of the world, including South Carolina. The work of our firm was used extensively in NRC Regulatory Guides 1.60 and 1.61. We have investigated and/or studied the effects of major earthquakes throughout the world, including reservoir-induced or suspected reservoir-induced earthquakes at Koyna Dam, Oroville Dam, Hsinfengkiang

Dam, and recently, Monticello Reservoir. I have been a panel consultant on earthquake criteria and problems to the Division of Water Resources, State of California, for about 14 years and in that capacity have been consulted on the Oroville Dam, the proposed Bureau of Reclamation Auburn Dam, and various types of hydraulic structures. These two dams constitute the largest of their types in the world.

My testimony is about the earthquake shaking criteria for the Virgil C. Summer Nuclear Station and the ability of the plant to resist that shaking. The attached appendices designated B2 through B5, B7, B9 through B12, B14 and B15 and the tables and figures therein, as well as the tables and figures in this testimony, unless otherwise noted, were prepared by me and constitute part of my testimony.

Some Basic Terms and Definitions

Before proceeding with my testimony on specific points, it may be desirable to discuss some of the basic terms that will be used repeatedly in these proceedings. Although many of these may be familiar to all interested parties, there should not be differences in definitions or interpretations that could lead to misunderstanding.

When the ground moves resulting from an earthquake, there is acceleration, velocity, and displacement, as for the movement of anything else such as an automobile. An accelerating automobile is increasing its velocity (miles per hour) and also moving a distance (miles), which in

dynamics we call displacement. Earthquake motion, unlike the automobile motion, reverses back and forth many times during the time duration of a strong earthquake. Thus the acceleration, velocity, and displacement not only vary with time but have opposite directions. Records are made of ground motion using instruments and recording systems that delineate the actual motion of the ground and how it varies with time. These records are called time histories. Acceleration is often measured. The maximum or peak acceleration, whether moving in one direction or the other, during the entire record of strong motion is called the absolute peak instrumental acceleration, or often simply instrumental acceleration, or peak ground acceleration (PGA).

It has become more or less traditional for earth scientists and many engineers to consider the peak ground acceleration (PGA) of an earthquake at a given location. There is often confusion, however, between the peak acceleration as measured by the instrument, "instrumental acceleration," and the acceleration value that might be used in developing the criteria for the analysis or design of a plant which hereinafter is designated as "effective acceleration." The peak instrumental acceleration, which usually represents an extremely short duration spike or pulse on a time history, need not be used directly for design purposes. The reasons for this are many and will be discussed subsequently. Another point of confusion is the direction of the peak acceleration--the two horizontal components

are assigned equal values for design, whereas peaks are in fact not equal nor concurrent as generally recorded.

The effective acceleration used as the basis for the evaluation of the Diablo Canyon plant for the hypothetical 7.5M earthquake on the nearby Hosgri fault was 0.75g. However, the peak instrumental acceleration from which that value was derived was 1.15g. The ratio of the two values is 0.65. (ALAB-644, June 16, 1981)

The term natural period or natural period of vibration will frequently be used. The natural period is the time required for an oscillating body to move from any given point away from and back again to that same starting point. A pendulum, for example, may swing from the highest point on the right side to the highest point on the left side. The time required to do this is its natural period, usually given in seconds. Structures, equipment, piping systems, etc., have natural periods of vibration, including not only a fundamental or basic mode, but various other modes. These periods are considered constant in the elastic state for all small amplitudes of motion. The natural frequency of vibration is simply the reciprocal of the period, and is given in cycles per second, now termed hertz.

Damping is related to the energy change during vibration and it varies for different materials and structures. Energy is never lost, but it changes form. The kinetic energy of motion of a vibrating body or system is reduced by energy converted to heat through friction and the internal stressing of materials, and by other means. The rate or

degree of this loss of kinetic energy is called damping. If there were no damping at all an oscillating system would never stop. If the system were critically damped it would not oscillate and, upon being displaced, it would simply return to its static position. Although damping is a very complex subject and has many forms, in earthquake analyses viscous damping is generally assumed and it is given as a ratio to or percentage of critical damping, which in turn is that damping value which could just prevent oscillation.

The recording instruments have natural periods and damping values and these in turn determine what the instrument measures.

An earthquake, whether a naturally occurring tectonic event or a reservoir-induced event, involves a shear dislocation on a fault. One measure of the size of the earthquake is the seismic moment that is related to the permanent slip or dislocation at the source, the rupture surface area, and the rigidity modulus of the rocks bearing the stress. The seismic moment of an earthquake can be estimated from geologic data (if the rupture length and displacement can be measured), and/or from seismographic data (from the long period spectral displacement amplitude of the shear wave), or from the extent of aftershock locations. The static stress drop is defined in terms of the ratio of average static displacement to rupture dimension (with the proportionality constant depending on rupture geometry

and rigidity modulus). Generally, surface ruptures are observed only for large (magnitude above about 6) shallow focus earthquakes. For smaller earthquakes the static stress drop cannot be determined readily. Surface ruptures are not known in the eastern United States with the possible exception of the New Madrid earthquakes of 1811 and 1812.

The earthquake rupture is accompanied by the radiation of elastic energy in various propagation modes (compressional wave, shear waves, and Love and Rayleigh surface waves). An earthquake ground motion record consists of a generally complex train of directly arriving, reflected and refracted body waves (i.e., compressional and shear waves), surface waves, and other guided waves.

In the far field the shear wave displacement spectrum can be represented, on a plot of log spectral displacement amplitude versus log frequency, by two asymptotic lines. The low-frequency spectral asymptote (from which seismic moment is measured) has a slope of zero, and the high frequency asymptote has a negative slope of 2 (or more). The frequency where the spectral asymptotes intersect is known as the corner frequency, and its inverse, the corner period.

According to the Brune model of the earthquake source, the corner period is determined by and is proportional to the rupture length. One of the common methods over the last decade to estimate the rupture length is the

Brune model, so one can solve for the fault displacement (given the seismic moment) and thereby estimate the stress drop. The stress drop determined from dynamic observations (as opposed to static or geologic observations) is sometimes called the dynamic stress drop.

The spectral corner period of earthquakes is generally found to increase monotonically with seismic moment. This phenomenon accounts in large part for the lack of correspondence between magnitude scales (systems of classification of earthquakes by size). Magnitude is determined from peak seismogram trace amplitude in a particular frequency band (depending on the scale), and with reference to a specific wave mode or phase. Magnitudes determined from scales of different frequencies generally differ because the proportion of high-frequency to low-frequency energy varies (decreases with increasing seismic moment).

For structural response considerations, the most directly applicable magnitude scales are those where magnitude is determined for a frequency (or frequency band) comparable with the frequency band of structural interest (generally above 1 Hz for low-rise structures). For earthquakes in the California-Western Nevada region, Richter local magnitude (M_L) is determined from the maximum trace amplitudes (regardless of wave mode) for two horizontal components registered by standard Wood-Anderson displacement seismographs (natural period .8 sec) in the distance range up to 600 kilometers. In the eastern United States (east

of the Rocky Mountains), the most commonly used scale is the m_b (Lg) scale developed at St. Louis University, based on the maximum trace amplitude registered by a standard short-period (1 sec) vertical electromagnetic (velocity) seismograph for a guided-wave group named Lg. M_L and m_b (Lg) are examples of scales that are relevant in structural response considerations, as opposed, for example, to the Richter surface wave magnitude scale M , which is based on teleseismic observations of the amplitudes of 20-second period surface waves.

Moment magnitude scales are based on seismic moment, which is determined from the shear wave spectral displacement amplitude for frequencies up to the spectral corner frequency: this scale differs from the other magnitude scales in that the frequency band wherein moment magnitude is evaluated changes with earthquake size (or moment), because the corner frequency varies with moment. Because moment magnitude is a measure of long period motion, it is not particularly appropriate to structural response consideration.

As materials are loaded they deform. For example, a steel bar anchored at one end and subjected to an applied pull, or tension, at the other end, lengthens. The applied tension or force creates stress or force per unit area of cross-section of the bar. The lengthening or deformation creates strain, or deformation per unit of length.

The elastic state of stress is that in which the strain or deformation is or may be considered as directly proportional to the stress or the loading. A common misconception is that failure occurs if the design stress is exceeded, or the yield point of ductile materials is exceeded. Such is not the case--there is generally a wide remaining margin to the point of failure. Moreover, local failure in a redundant system does not signify general failure.

The inelastic state or the ductile range is that range of stress or loading beyond the elastic state or beyond the yield point, wherein strain or deformation increases more rapidly than stress or loading. The properties in the inelastic state may range from brittle to extremely ductile, depending upon the materials and how they are used.

The response spectrum is an extremely important concept in the analysis and design of nuclear power plants for earthquake motion and will be referred to repeatedly. If a complete time history of motion is used as the disturbance input, it is possible to calculate the maximum response of a simple one-degree-of-freedom elastic, damped oscillator when subjected to the entire time history of motion. Such a simple oscillation might be represented by a single rigid mass on a vertical stick having stiffness but no weight, or a "lollipop" shape. The results of such a calculation would produce only one point for a response

spectrum curve and that point would be for the natural period of vibration of this particular oscillator with its particular damping ratio. If a whole series of oscillators of the same damping are subjected one at a time to the same ground motion record, and if each oscillator has a different natural period, there would be a whole series of points for a plot of spectral acceleration versus period such as shown in Figure 1. Connecting these points would provide a "response spectrum" for the particular ground motion record and for the particular damping of the oscillators. If the same procedure were repeated using oscillators with other damping values, a whole family of spectral curves would be obtained for the particular strong motion record. Figure 2 represents a set of such spectral curves for the 1940 Imperial Valley earthquake recorded at El Centro, California. Of course, these extensive calculations are done in computers.

Most acceleration response spectra made from an earthquake record are rather jagged with many peaks and valleys. It is customary to obtain smooth curves for use in analysis and design in order to avoid sensitivity in response caused by minor variations in natural period. There are various ways this "smoothing" can be done. One simple way is to draw the smooth curve through the jagged one either by averaging the peaks and valleys or, as is more often done, to almost envelope the peaks. A better way is to not rely upon one ground motion time history but to use several

appropriate records representing as near as possible the conditions under consideration. This results in a whole series of response spectra for each damping value, which can then be treated statistically by various methods to obtain an average curve for all the records used as well as other curves representing any statistical deviation from the average that may be desired. This procedure has the advantage of providing probabilistic distributions at any period value or statistical confidence level of interest.

Response spectra can also be constructed artificially, or they can be obtained by the use of standards like NRC Regulatory Guide 1.60, or from ratios of spectral values to either ground acceleration, velocity, or displacement, depending upon the period or frequency under consideration. A most convenient procedure is to consider the dynamic amplification factor, DAF, as the ratio of the spectral response at any given period, damping, and statistical confidence level to the effective acceleration. It so happens that the effective acceleration used to construct spectral curves is the same as spectral response at any damping value at zero period or infinite frequency. Effective acceleration is therefore sometimes referred to as zero period acceleration or anchor point acceleration. Using the DAF factor for any desired confidence level one can readily adjust spectral curves to any specified effective acceleration. This is sometimes referred to as scaling the acceleration value.

Response spectra may be in units of acceleration, velocity, or displacement, each of which may be plotted against period or frequency and on linear or log scales. In addition, a useful device is a 4-way log paper on which one can read spectral acceleration, velocity, and displacement plotted against period or frequency on one diagram. An example is Figure 3.

It is often convenient in analysis to use a time history instead of a response spectrum. However, as discussed previously, time histories produce spectra with peaks and valleys. To overcome this problem, a time history is selected to best represent the conditions of the problem and it is then artificially altered, usually with additions of pulses of proper sizes and at strategic locations in the time domain to cause the spectrum made from the modified time history to closely match the prescribed spectral diagram. This work has to be carefully done and, of course, with computer aid.

The motion of the ground beneath a foundation causes forces to be developed in the structure above; the ground motion per se is not force. According to one of Newton's classic laws a body at rest tends to remain at rest and one in motion tends to remain in motion. Thus a structure at rest tends not to move when the ground moves and thus inertial forces are created that cause the structure to move, not only as a unit but in some deformed shape depending upon the ground motion characteristics and the dis-

tribution of mass and stiffness throughout the structure. The deformations create stresses, which stresses may remain below the yield and/or cracking point (with no damage), or go beyond yield or cracking (with stretching or damage, respectively). The continuation or duration of the strong motion increases the probability of damage and, should damage occur, it also tends to increase the extent of the damage. This is one reason among several why short bursts of energy or spikes on a time history record are less significant structurally than motion of long duration and many strong cycles as for major, local earthquakes of a tectonic nature.

In order to analyze the response of structures and/or systems to ground motion, models are made that represent the dynamic properties of mass, stiffness, and damping. One commonly used model is the "stick" model, or "lumped mass" model, in which discrete masses are used to represent weight divided by the acceleration of gravity in appropriate units. These masses are connected by assumed weightless members or springs that represent the stiffness of the connecting elements. All items are, of course, properly located geometrically. The more masses employed, the closer the model represents the prototype. However, where masses are properly located, as few as one mass per story of a typical tall building generally provides good representation of actual conditions. The model is used to compute the natural periods of vibration, the mode shapes,

and the relative importance of the modes in response to a given motion.

The model may be subjected to a complete time history of ground motion and the responses of all the masses or node points obtained, for any particular time, or perhaps just at the times of reaching maximum values. Alternatively, the characteristics of the model such as natural periods, mode shapes, participation factors (the relative importance of modes), and damping may be used to obtain responses from spectral diagrams (Blume, Bull. Seis. Soc. Am., Feb., 1970). Responses are obtained not only for the directional components, but for the various modes in each direction. The maximum responses of direction and mode do not occur simultaneously, so various procedures of a probabilistic nature are used to get combinations for analysis and design. A common method is the square root of the sum of the squares (SRSS).

It seems desirable to note at this point that peak ground acceleration alone--even effective acceleration--is not the sole criterion for analysis or design. There are many other parameters that are equally or more important as, for example, spectral response acceleration, damping, allowable stresses, ductility, etc. In fact, it is possible to omit peak ground acceleration and go directly to spectral response. However, the inclusion of peak acceleration has become a traditional approach and one that especially appeals to earth scientists and others who record ground

motion with instruments. I emphasize that the structure "feels" spectral response acceleration and not peak ground acceleration.

Instrumental versus Effective Acceleration

The decision of the Atomic Safety and Licensing Appeal board in the matter of the Diablo Canyon Nuclear Power Plant (ALAB-644, June 16, 1981) covered the use of effective acceleration in considerable detail (pp. 67-72). On page 72 it was decided:

"in the circumstances, for the reasons discussed, we find that the use of an anchor point or effective acceleration is a physically valid and acceptable procedure for structures in the near field."

The Appeal Board's decision, which for Diablo Canyon was concerned with an instrumental acceleration of 1.15g and an effective acceleration of 0.75g, or 65% of the instrumental, was based upon the testimony of many experts, including this witness. The decision noted testimony of this witness regarding spikes of short duration, "clipping" analyses, and empirical evidence as to the survival of structures despite peak acceleration measurements that should have caused damage "were those spikes truly indicative of effective acceleration" (ALAB-644, p. 70, 71).

It is important, in the matter of the Virgil C. Summer nuclear station, to consider the differences between peaks or spikes on a record and for what level of acceleration the plant should have capacity to resist, especially for near-field reservoir-induced earthquakes that produce records

very short duration with high-frequency spikes. An extreme way to visualize the nature of the situation is to imagine a strong man hitting a massive concrete wall a hard blow with a hammer. There would no doubt be an acceleration of several gravities (g's) at the point of impact, and perhaps some local spalling, but one would not design the whole wall, or a building, for several g's. The energy in the hammer blow is limited. The energy in a spike in a time history of acceleration is also limited.

It gradually became clear as reliable strong motion records were obtained that peak instrumental ground acceleration, even for moderate earthquakes, was considerably greater than the base shear coefficient values of buildings that had survived even much stronger earthquakes. The difference was so great that it could not be reconciled with typical safety factors or the elastic dynamics of the problem. The definition of this problem and its extension to response spectra, together with the first attempt to reconcile recorded motions with building performance, was by Blume (1958). It was shown that (a) earthquakes were stronger than they had been given credit for, but (b) most buildings were also much stronger than conventional analyses would indicate. A procedure was proposed to reconcile the kinetic energy of the earthquake demand with the stored energy and work capacity of real, complex buildings. (Blume, 1958a, 1960, 1961)

It is essential to clarify at this point that "effective" acceleration is not the same as the base shear design coefficient except for a completely rigid mass, which is rare if indeed one ever exists except as a theoretical model. Most structures have many degrees of freedom, or modes of vibration, and they, and the ground under them, have some compliance. The result is that peak ground acceleration, instrumental or effective, should not be used directly in design. Effective ground acceleration can be used to construct response spectra or to proportion time histories of motion for use in analysis. However, for general purposes of discussion only, peak ground acceleration can be compared herein to base shear coefficients of assumed rigid structures. Real structures have base shears that depend upon the dynamic characteristics of the structure as well as the ground motion.

There are many reasons, some well known today and some not yet generally recognized, why effective acceleration would be less than instrumental peaks. (D.C. Report, D-LL: Blume, 1979) The reasons can generally be explained by one or more of four approaches, although the data for these may in some cases be sparse. The approaches, which are not mutually independent, are:

- o Observation of what has happened and what has not happened in earthquakes
- o Theory and analysis
- o Testing and experiments
- o Engineering judgment

For examples of observation, the damage, or lack of same, from three major close-in earthquakes will be considered herein. These three cases have in common the fact that under the rules and procedures being followed for the design of nuclear plants, none would qualify: in fact, they wouldn't come anywhere near qualifying and, on paper, would be total losses.

The first case is the great San Francisco earthquake of 1906 ($M_S = 8.25$) with the moving San Andreas fault about 10 miles from downtown San Francisco. Of the 52 major buildings (none specifically designed for earthquake forces), all but 7 were repaired and put back into service. Most are still in use today. Of those that did not go back into service, 4 were destroyed by fire and at least one was very poorly constructed. The tallest building, of 19 stories, is still in service today. A few of the surviving buildings still in use include the Central Tower, the Fairmont Hotel, the old part of the St. Francis Hotel, the Post Office Building at Seventh and Mission, the Ferry Building, the Monadnock Building, the Emporium, and the Flood Building. The old Palace Hotel had rather minor earthquake damage but its floors were completely burned out subsequently. Fort Point, only a few miles from the moving fault, had only minor damage.

The second case is the ESSO refinery complex at Managua, Nicaragua, which was subjected to the 1972, 6.25M earthquake. The accelerations were recorded on a modern

instrument right at the ESSO refinery--the peaks were 0.39g EW, 0.34g NS, and 0.33g vertical. The plant structures and vessels had various design levels ranging up to a maximum of 0.20 base shear coefficient and averaging about 0.10 to 0.13 under old Uniform Building Code criteria. For some of the more rigid structures these coefficients could roughly be compared with "effective" acceleration. For other structures, comparisons should be made with the much greater spectral response accelerations, properly adjusted. There were all sorts of vertical vessels, pumps, heat exchangers, pipes, buildings, tanks, foundations, and instruments. The ESSO plant had only minor damage. It was shut down for inspection and then started up again in less than 24 hours.

The third case is the Huachipato Steel Plant, near Concepcion, Chile, which was subjected to $7.5M_s$ earthquake on May 21, 1960 that caused about 0.4% damage but no collapses. The plant was shut down for 6 days and was then back on normal operations. The epicenter was about 80 km south of the plant. Because a larger earthquake occurred to the south of the May 21 epicenter on the following day, and because the steel plant is to the north of the first epicenter, there is reason to believe the plant on May 21 was directly opposite the moving fault.

The plant design was on a static rather than dynamic basis for coefficients estimated to have been in the range of 0.10 to 0.30. However, not only important dynamic

phenomena but buckling phenomena were not fully considered in the design and much of the damage is attributable to those factors. A rather generous equivalent design coefficient would be 0.12 at 1-1/3 times normal stresses. There is no record of the instrumental peak acceleration. However, an extensive study of the plant by me (Blume, 1963) led to the development of the most likely spectral response acceleration diagram. The probable spectral acceleration value at the period and damping of the most critical structures is 1.2g and the probable effective acceleration was 1/2 to 1/3 of this.

There are many cases of very weak structures surviving earthquake motion. The caretaker's house at Pacoima Dam (1971 San Fernando Earthquake) is a classic example. Peak acceleration nearby was over 1g and yet the brick chimney was undamaged. Small, local earthquakes of short duration may alarm people who are close to the epicenter, but they are not the cause of damage and loss of life. Magnitudes of 3, 4, and 5 occur rather frequently in seismic areas such as California, with no damage except to perhaps some precarious object already on the verge of falling. There may be sharp accelerations at or very close to the epicentral region, but these are without damage potential. They are generally at some depth. In this respect, small shallow events such as shallow RIS, or explosions in boreholes, may cause significant local acceleration but do not have the energy and duration required to cause damage to an

engineered structure. Obviously, acceleration is not the sole criterion for damage.

The major new structures in San Francisco, Oakland, Los Angeles, and in other earthquake regions of severe earthquakes are generally designed for base shear coefficients in the range of 0.05 to 0.10. Special structures like schools and new hospitals have about double that value, all at 1/3 increase in allowable stresses. Allowing for the further stress increases to yield values, and for the differences between accelerations and base shear coefficients, modern buildings would have effective acceleration values based on conventional methods in the range of 0.15g to 0.25g. Yet, these western cities are subject to instrumental accelerations in the range of 0.5g to 1.0g plus dynamic amplification in the buildings.

There are various procedures in getting from a given instrumental acceleration to an effective acceleration. The number of cycles of peak motion are sometimes considered and several of the highest "spikes" on the record are discounted as having no structural significance. Observations and judgment must enter this process because the measured data are sparse in some areas. We have made studies that show that extensive clipping of peaks from time history records has only a minor effect on peaks of response spectra, which are the real indicators of structural performance (D.C. Report D-LL30). Likewise, time history peaks can be augmented with similar results. If an

acceleration peak or spike has very short duration, the energy involved is small. This can be visualized in view of the fact that the time integral of acceleration is velocity, and the kinetic energy of motion is velocity dependent. Random spikes that lack periodicity and have short duration are apparently not effective in dynamic amplification nor therefore in structural response.

Figures 4 and 5 are from the "clipping" study (D.C. Report D-LL30) conducted with 20 earthquake records. For the figures, two California earthquake records were used as noted. Large decreases in peak ground acceleration caused only minor decrease in spectral acceleration, S_a . It is the latter that is meaningful in response.

A similar clipping and augmentation study was conducted for the Monticello dam record of October 16, 1979, 180 degree component. The work is described in Appendix B7. Figure 6 (Figure B7-14 taken from that report) shows the results for three arbitrarily selected periods. Obviously, the spikes on the record are not effective for response as measured by S_a . In fact, at $T = 0.15$ sec, they have no effect at all.

From all considerations--observations, theory and analysis, testing and experiments, and engineering judgment based upon all of these and the record of performance in actual earthquakes--it is my opinion that there is no need to design to peak instrumental acceleration values, especially for close-in events; in fact, to do so would

be extremely wasteful. The amount of reduction, or the clipping factor C, depends upon many considerations. A reduction factor of about 0.6 seems reasonable for reservoir-induced seismicity in the Monticello Reservoir area. However, records taken at the top of a dam or on soil should not be the basis for the instrumental acceleration to be adjusted.

Instrumental Acceleration ("a") Values

In the previous testimony I have explained why instrumental accelerations, especially for close-in RIS events, are not directly applicable in plant analysis or design. In this section, instrumental accelerations will be discussed, having in mind that they are subject to reduction for analysis and/or design purposes.

In the absence of reliable recorded strong motion records, intensity is often used as a basis for estimating acceleration. Intensity is subjective in that it is based upon the reporting of people, rather than instruments, and often these reporters are laymen. However, when thousands of reports are received and carefully studied, it is possible to develop meaningful isoseismal intensity maps based on what was seen, felt, damaged, undamaged, etc., on a broad statistical basis. It is my opinion that eastern intensity reporting may be somewhat greater than that for western, for a common earthquake situation, for three reasons: (1) the people reporting have had less

experience with earthquakes and are more shocked by them; (2) the buildings and structures in the east are generally not designed for earthquakes and therefore are subject to damage at lower levels of shaking. and (3) the west has had more prior shocks that tend to remove the least resistant buildings and lead to improvements in the design and construction of new buildings.

A great many seismologists, mathematicians, and engineers have worked on the relationship of intensity and acceleration. Richter, for example, in his book Elementary Seismology (1958, p. 140) suggests:

$$\log a = I/3 - 1/2$$

wherein a = peak acceleration, cm/sec²

I = Intensity MM (Modified Mercalli)

with this equation, we get the following:

<u>I (or MM)</u>	<u>Peak Acceleration</u>
V	0.015g
VI	0.032g
VII	0.069g

At intensity VI there should be some cracking of plaster or of very poor quality masonry if there are houses in the area (Note: if there are not houses and people, it is often difficult to assign the intensity). At VII, the damage will be negligible in structures of good design and construction but will occur in ordinary structures, i.e., some walls should be cracked, some weak chimneys are broken at the roof line, there may be some local slides along banks, concrete ditches damaged, etc. I know of none

of these things happening from RIS at Monticello Reservoir. One could deduce that the RIS intensities there were IV or less.

Murphy and O'Brien (BSSA, vol. 67, no. 3, June 1977) had the advantage over Richter of a much greater population or set of data. They also had the advantage of the products of many other workers. Their equation using Richter's terms is:

$$\log a = I/4 + 1/4$$

which produces:

	mean	mean + 1 σ
IV	0.018g	0.041g
V	0.032g	0.073g
VI	0.057g	0.13g
VII	0.10g	0.23g

The mean values are slightly greater than Richter's. Absence of reports indicating intensities of V or VI for the Monticello RIS events would indicate low accelerations by these procedures, less than indicated above for MM = V. The recorded acceleration values on the crest of the hill next to the dam are incompatible with the intensity ratings. What the instrument no doubt felt was the amplified motion of the soil at the top of the hill.

The data reported in Appendix B11 on the Hsinfengkiang Dam RIS are also pertinent. Twenty-eight RIS events produced no free-field motion near the dam in excess of 0.094g. The average was much lower at .042g. This reservoir, in contrast to Monticello, is underlain by active faults of some length.

This witness feels strongly that the measured data on soil near the top of the dam are not applicable to the Summer plant because they do not represent the free field.

I know of no reservoir-induced earthquake of magnitude greater than about 4.6 that was not in a known or probable active faulting condition prior to the impoundment of water. With some 13,000 reservoirs in the world and hundreds of thousands of reservoir-years exposure, this is a remarkable situation. With the use of assumed magnitudes, accelerations can be predicted by a variety of methods providing adjustments are made for epicentral or normal fault distance, and depth to the center of energy release. One method that has been used in this case is the Brune model involving stress drop and moment. Joyner and Fletcher used moment related to the August 27, 1978 earthquake record. Without getting into discussion of the assumptions and values used by them, I reiterate that the 1978 or the 1979 record is inapplicable as input motion for the Summer Plant for the reasons already cited. Thus the results obtained by Joyner and Fletcher are not applicable to design conditions for that reason and also because they do not allow for the matter of effective vs. instrumental acceleration. They do, however, conclude with:

"Whether the motions represent a problem for the facility or not is an engineering judgment which is not ours to make."

The open file report 81-365 by Joyner et. al. provides an equation relating acceleration, magnitude, and distance.

I used 3 km for d, the closest distance to rupture (Appendix B4), and obtained the following with their equation:

	<u>Instrumental Acceleration</u>		
M	Median	Plus 1 σ	
4.0	0.094g	0.17g	
4.5	0.13g	0.24g	
5.0	0.18g	0.33g	

If factored by C = 0.65 as for Diablo Canyon (ALAB-644) all of the above values would be below 0.25g (the Joyner et al, data is largely for soil), and all but one would be 0.16g or less. In other words this OFR 81-365 equation is not incompatible with the design criteria for Summer even with an excessive magnitude of 5.0. A factor of 0.60 is considered even more appropriate.

The China report based on RIS at the Hsinfengkiang Dam reservoir (Appendix B11) includes an equation based on 22 RIS earthquakes:

$$a_g = 35 \times 10^{(0.15M_s - 0.5 \log D)}$$

wherein a_g = peak instrumental acceleration, cm/sec²

D = epicentral distance, km

This equation produces the following values, g units:

M_s	D = 1	D = 3	D = 5
4.0	0.14	0.082	0.063
4.5	0.17	0.098	0.075
5.0	0.20	0.12	0.090

These instrumental acceleration values are compatible with the Summer design criteria.

The Blume "SAM" equations covered in more detail in Appendix B3 and based on western earthquakes produce

following peak instrumental accelerations with R the hypo-central distance:

<u>M</u>	R (km)	<u>PGA, g units</u>	
		<u>Median</u>	<u>Plus 1σ</u>
4.0	4	0.02	0.051
4.5	4.5	0.032	0.081
5.0	5	0.052	0.13

All of the above (except the Joyner-Fletcher use of the Monticello record, which I consider inapplicable) produce accelerations that are compatible with the Virgil C. Summer criteria for magnitudes up to 5 when proper adjustments are made for confidence level and effective acceleration. Inasmuch as magnitude 5 is incompatible with the world history for non-faulted sites, the design values of 0.15g and 0.25g for rock and soil, respectively, cover quite adequately future RIS events.

Structural Capacities

There are many factors that affect the capacity of a structure or a system to resist the shaking caused by an earthquake without damage or collapse. Some of these are strength, ductility, redundancy, damping, natural periods of vibration, mode shapes, symmetry, mass distribution, the allowable design stress as compared with the actual strength of the materials, soil-structure-interaction, foundation conditions, and how the directions of motion and the model responses are combined in analysis. Another factor is the way the response spectra and the floor response

spectra are developed and used. This is a very complex subject. All that can be done herein, however, is to outline some of the important considerations and to indicate where the compounding of certain conservative assumptions in nuclear plant design often leads to safety margins far above those generally assumed.

Unrecognized safety factors or safety margins are very important in reconciling recorded instrumental ground motion with damage, or lack of same, and in reaching engineering opinions. Many of these unrecognized items are unrecognized in the sense that they are only just beginning to be understood, and many in the sense that they are not allowed under current design procedures or standards, including NRC standards. It is with the latter context that this section of testimony is basically concerned. The subject is relevant to the matter of how the plant resists tectonic earthquakes and RIS because it is, no doubt, qualified at greater values than 0.15g and 0.25g because these safety margins are generally ignored. I shall only list some of the most pertinent unrecognized "bonus" values in the system. This is not intended to be of any reflection on the NRC review process or reviewers, but on the state-of-the-art and traditional practices.

(a) In establishing design values for materials, the conventional practice is to make tests, to plot test values on a graph, then to draw a line or curve that represents the lowest value of these test points, and finally

to establish safety factors based on that line or curve. Figure 7 is an example, taken at random, for some concrete tests, the exact nature of which is immaterial to this discussion. The point is that the equation to be used is based on a line that sub-envelopes all test points; i.e., the real average value is greater than recognized, say in the range of 15 to 30 percent (D.C. Report D-LL18C)

(b) Material strength is specified in such a manner that very few test values for the material supplied can fall below that value without rejection of the whole lot. Thus the suppliers provide extra margin to avoid this severe penalty. Real test values could well be used for walls and redundant systems. The actual test values for the Summer plant are shown in Appendix B10 and are well above the specified values. Actual test values were allowed for the Diablo Canyon plant (ALAB-644, June 16, 1981, pp. 161-163).

(c) When the horizontal components of ground motion are used in analysis it is customary to assume that both components are equal to the peak prescribed ground acceleration, and are thus also equal to each other. The facts are that in measurements of actual ground motion the minor component orthogonal to the major component is invariably less than the major component, generally much less. In other words, they are not equal. Figure 8 is the ratio of the maximum to the average peak acceleration, normalized to $M = 6$ for plotting convenience, plotted against hypo-

central distance for all recorded California and Nevada earthquakes in the period 1954-1970. At short distances, such as 10 km, the ratio R is 1.13. This is equivalent to the small components being only 77 percent of the large components m. Yet in analysis it is taken as 100% of m! This is another conservatism, and it could provide excess strength in the order of 10 to 30 percent. (D.C. Report D-LL18A)

(d) Analysis procedures assume constant natural periods or frequencies of vibration for structural and mechanical systems. In reality, there are small variations in period even at non-damaging stress levels. This is due to the nature of materials, especially concrete, and to other factors. These small variations are quite effective in preventing resonance and in decreasing dynamic amplification. To demonstrate this principle, Figure 9 is a plot of part of a resonance curve for a 7% damped oscillator responding to a steady state harmonic forcing function. At perfect tuning, the ratio of the forcing frequency and the natural frequency is 1.0 and the response is maximum or 100%. However, if the natural frequency varies only slightly, say 5%, the response is about 80%, or 20% less. Thus, the assumption of constant natural periods is conservative and could lead to overdesign in the order of 10 to 30 percent. (D.C. Report D-LL18B)

(e) Floor response spectra are used for upper levels. This, in the first place, is generally considered conserva-

tive compared with coupled system analysis. Moreover, the floor response spectra are computed for constant periods. Nevertheless, the spectral peaks have been widened to allow for possible differences in natural periods from those computed, but without any reduction of the peak response value. Thus, there is greater area under the response curve and thus more energy introduced into the disturbance than would be expected from the earthquake. The amount can be estimated only on a case by case basis, but can be considerable.

(f) Smooth response spectra are used in analysis, whereas for any one earthquake (or even for several earthquakes) the actual spectra are jagged with peaks and valleys. Because the smooth curves tend to envelope the peaks, this introduces another conservatism or safety margin into the system. The earthquake peak response may fall where a valley should be: in fact, this is quite likely. This conservatism may lead to 10 to 20 percent overdesign in many cases.

(g) Ductility and work potential (which absorbs energy in the inelastic range) are not generally allowed; i.e., the response must be completely elastic. There is thus a great reserve capacity in the inelastic range to absorb energy with even very slight damage which has thus not been tapped. This is very conservative. Every tall building in a major earthquake has to enter the inelastic

range to survive, even under the most modern building code requirements! And yet the nuclear plant structures are to remain in the elastic range under more severe earthquake criteria. There could be reserve capacity for this item estimated at 30 to 100 percent.

(h) Seismic stress in most members and elements is only a part of the total stress picture. For example, a pipe has internal pressure, a concrete wall supports loads from above. The only exception is bracing designed solely for lateral forces of wind or earthquake. Members or elements designed for other than seismic stress alone have much more reserve strength for seismic loading than they are given credit for. The reason for this involves the allowable stresses under each type of loading and the fact that more material is provided than would be needed for seismic purposes only. This item can vary from no extra value (for braces) to several hundred percent. (D.C. Report D-LL21)

In view of the above conservatisms in analysis procedures, as well as others, it is clear that there are unrecognized safety margins (if properly considered on a joint probabilistic basis using mean values and deviations from the mean values) such that when a plant is qualified for its specified effective acceleration, its most likely capacity is greater by as much as several hundred percent.

Damping values are important parameters in dynamic analysis. The values in NRC Regulatory Guide 1.61 have

been in use for several years for many plants. However, new data were obtained and studied, old data were reviewed, and reports on damping values were prepared by URS/Blume Engineers for the Diablo Canyon project.

Two facts regarding this complex subject are particularly important. One is that elements with friction between parts, such as bolted steel joints or concrete with minor cracks, have considerably greater damping at the same strain levels than where friction is not possible, as for example in welded joints or in uncracked concrete. The second point is that damping increases with strain or deformation. These two factors are not necessarily mutually exclusive. Another important consideration is that a structure not only receives energy from the moving ground but returns some of it to the ground: this is often termed radiation damping.

Another point that is often misunderstood is that it is not necessary to develop high strain levels throughout an entire structure to develop high damping levels. Local high strain levels can be quite effective in absorbing the kinetic energy of motion, as shown by tests.

Various tests and measurements of damping have been shown in the Diablo Canyon report D-LL 9. Summary data will be shown here. Table I shows damping results, from nine test series, for two levels of strain--at micro levels and at or about the yield point. At the yield levels, all test results are at or above 7% of critical damping.

TABLE I - SUMMARY OF DAMPING VALUES

	<u>At micro levels of stress and strain</u>	<u>In the yield range of stress and strain</u>
CVTR Reactor	6 to 9%	----
EGCR Reactor	1.5% to 5%	----
22 Concrete Buildings	mean 5.6% 1.25 x mean 7.0%	----
Bridge Piers	3.4% to 16.6% (average 8%)	----
Models of of Bridge Piers	----	7%
Models of Coupled Shear Walls	----	10% at 1.1 x yield
Models of Coupled Shear Walls	----	8% at 0.9 x yield
Models of Shear Walls	2% to 4%	7% to 10%
Scaled Buildings Models	2% to 3%	up to 9%

The test of the shear wall models (Figures 10 and 11) are particularly interesting for two reasons--they are of reinforced concrete shear walls, as is much of the Summer plant structures; and the base, and also the support of the base, of the wall test specimens was such as to essentially eliminate all radiation damping to the soil. The latter point is significant for those who may contend that radiation damping is present in all damping tests (plant structures), as compared with these models, will have the physical benefits of any radiation damping even though it is not credited or taken in the analysis).

Figure 12 shows the wall test results. The line "average curve" was drawn by the test authors, and the 7% line was drawn by us for comparison purposes. The 7% damping value occurs at a strain level in the reinforcing steel of about .16%, well below the yield value.

It is my opinion, based on valid data, that 7% of critical damping, as in Regulatory Guide 1.61, is conservative for concrete walls and structures approaching hypothetical yield point levels as in SSE design. The value could be 8 to 10% at yield point levels.

Each of the items noted above under structural capacities indicate that the plant is stronger (as designed to the original criteria) than it is given credit for. When these items are compounded, the real safety margins increase considerably above the margins on paper. Because items (a), (b), (c), (d), (e), (f), (g), and (h) all apply to the Summer facilities, the plant structures could no doubt withstand twice, or more, of what they were designed for without impact on the public health or safety from ground shaking. (Blume, 1977: 1979)

Conclusion

I have read the pertinent testimony and reports regarding RIS at Monticello Reservoir, the plant design criteria, and have considered the data and experience available on a worldwide, as well as a local basis, in earthquake engineering and structural dynamics for both

natural tectonic events and RIS events whether or not associated with active faults. Based on this and the testimony submitted herewith and in the appendices to this document, it is my considered opinion that:

- o The plant could be considered to be subjected to hypothetical RIS events of up to 4.5 magnitude but with slant distances to the foci (in kilometers) at least as great as the magnitude values.
- o The ground motion from such events could be withstood based upon the letter of the current plant design criteria.
- o The plant has much more capacity due to generally unrecognized safety margins and conservatism than current criteria would indicate.
- o The SMA records taken at the hill top (near Monticello Reservoir dams B and C) of the RIS events are not rock or true free field motion and are inapplicable to the plant conditions.

FIGURES

<u>FIGURE</u>	<u>TITLE</u>
1	CONSTRUCTION OF A RESPONSE SPECTRUM....
2	RESPONSE SPECTRA, DAMPING = 0, 2, 5, 10, and 20% 1940 EL CENTRO EARTHQUAKE, SOOE.....
3	4-WAY LOG PLOT OF RESPONSE SPECTRA, 1940 EL CENTRO EARTHQUAKE, SOOE.....
4	EFFECT OF CLIPPING ON SPECTRAL AREAS FOR TAFT 1952, S69E RECORD.....
5	EFFECT OF CLIPPING ON SPECTRAL PEAKS, EL CENTRO 1940, NS RECORD.....
6	EFFECT OF PGA ON RESPONSE, 2% DAMPING, BASED ON MONTICELLO RESERVOIR EARTH- QUAKE OF AUGUST 27, 1978.....
7	DIAGONAL CRACKING IN THOSE REGIONS OF BEAMS PREVIOUSLY CRACKED IN FLEXURE....
B	RATIO OF MAXIMUM TO AVERAGE HORIZONTAL PEAK GROUND ACCELERATION.....
9	RELATIVE RESPONSE OF A 7% DAMPED OSCILLATOR IN THE STEADY STATE UNDER A HARMONIC FORCING FUNCTION.....
10	NOMINAL DIMENSIONS OF TEST SPECIMAN WITH RECTANGULAR CROSS-SECTION.....
11	NOMINAL CROSS-SECTIONAL DIMENSIONS OF TEST SPECIMANS.....
12	DAMPING VS. STRAIN LEVEL IN THE PCA SPECIMANS.....

REFERENCES

Note: For Diablo Canyon D-LL Report Reference, see Appendix B-12.

ALAB-644, Atomic Safety and Licensing Appeal Board Decision on Diablo Canyon Units 1 and 2, June 16, 1981.

Blume, John A. (1958), "Structural Dynamics in Earthquake Resistant Design," Journal of the Division, ASCE, July 1958, and discussions ending September 1959; also published in Transactions, ASCE, 125:1088-1139, 1960.

Blume, John A. (1960), "A Reserve Energy Technique for the Earthquake Design and Rating of Structures in the Inelastic Range," Proceedings, Second World Conference on Earthquake Engineering, Tokyo.

Blume, John A., N.M. Newmark, and Leo H. Corning, (1961), "Design of Multistory Reinforced Concrete Buildings for Earthquake Motions," Portland Cement Association, Skokie, Illinois.

Blume, John A. (1963), "A Structural-Dynamic Analysis of Steel Plant Structures Subjected to the May 1960 Chilean Earthquakes," Bulletin of the Seismological Society of America, 53:439-480.

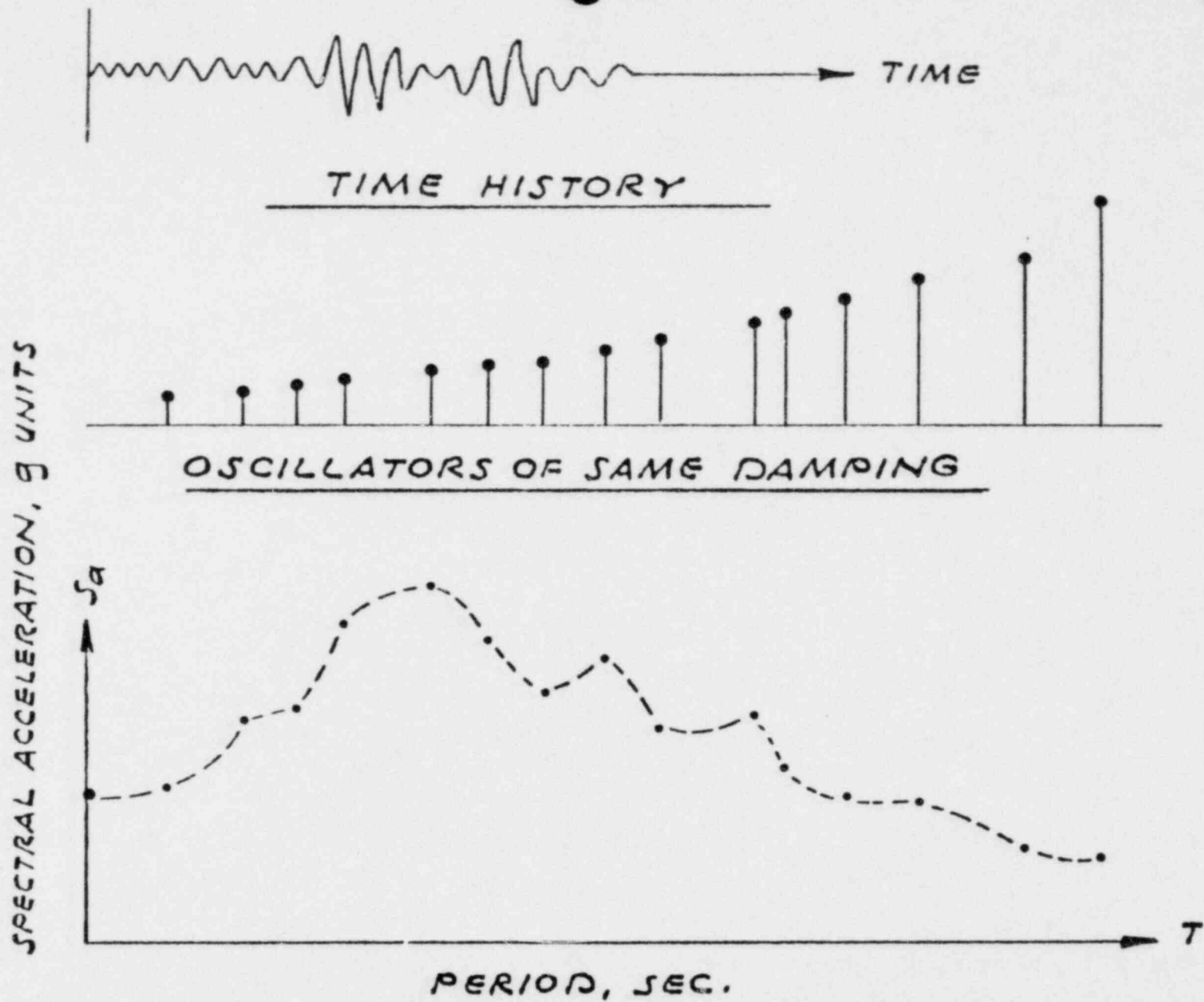
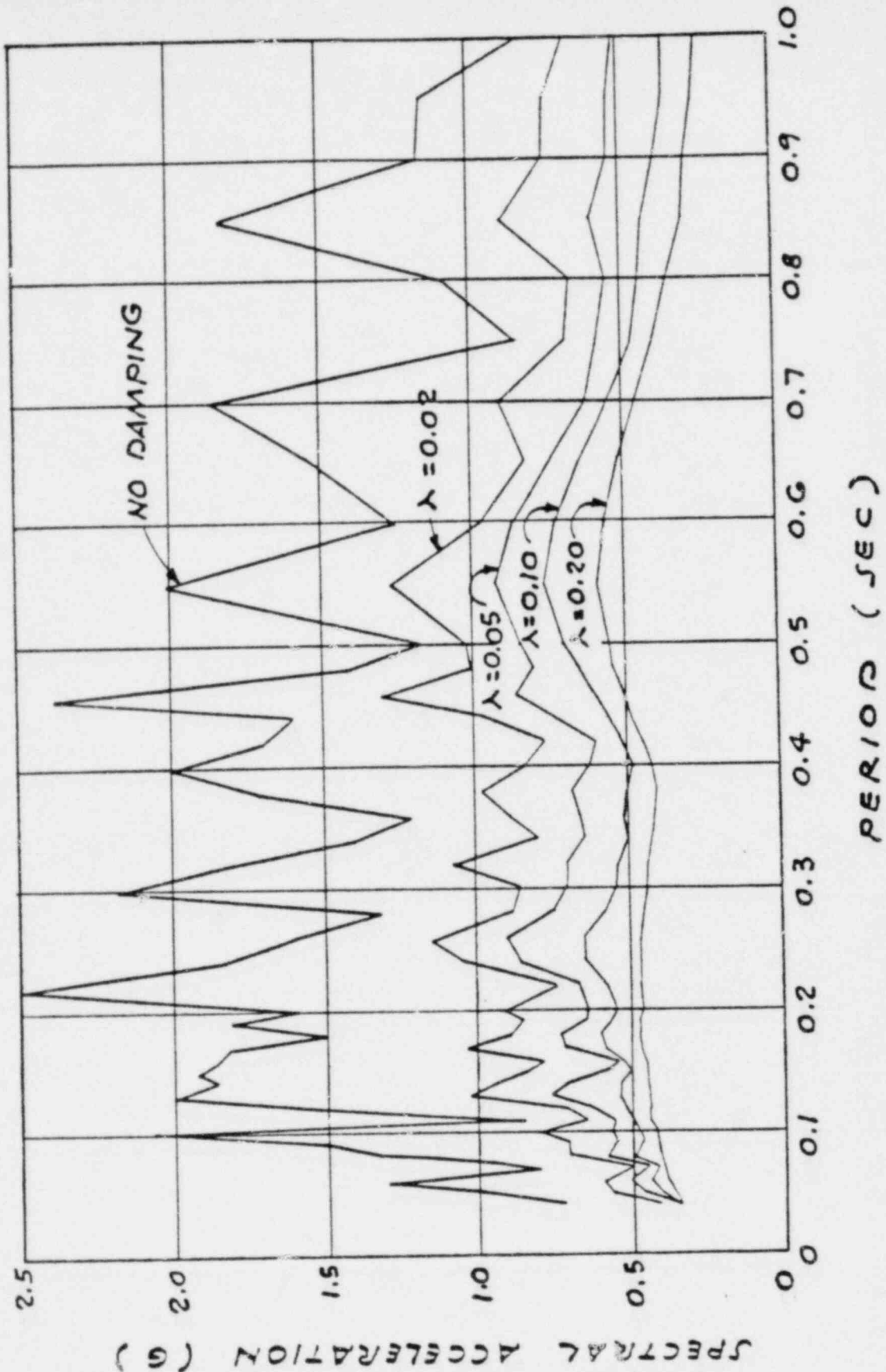


FIG.1 - CONSTRUCTION OF A RESPONSE SPECTRUM

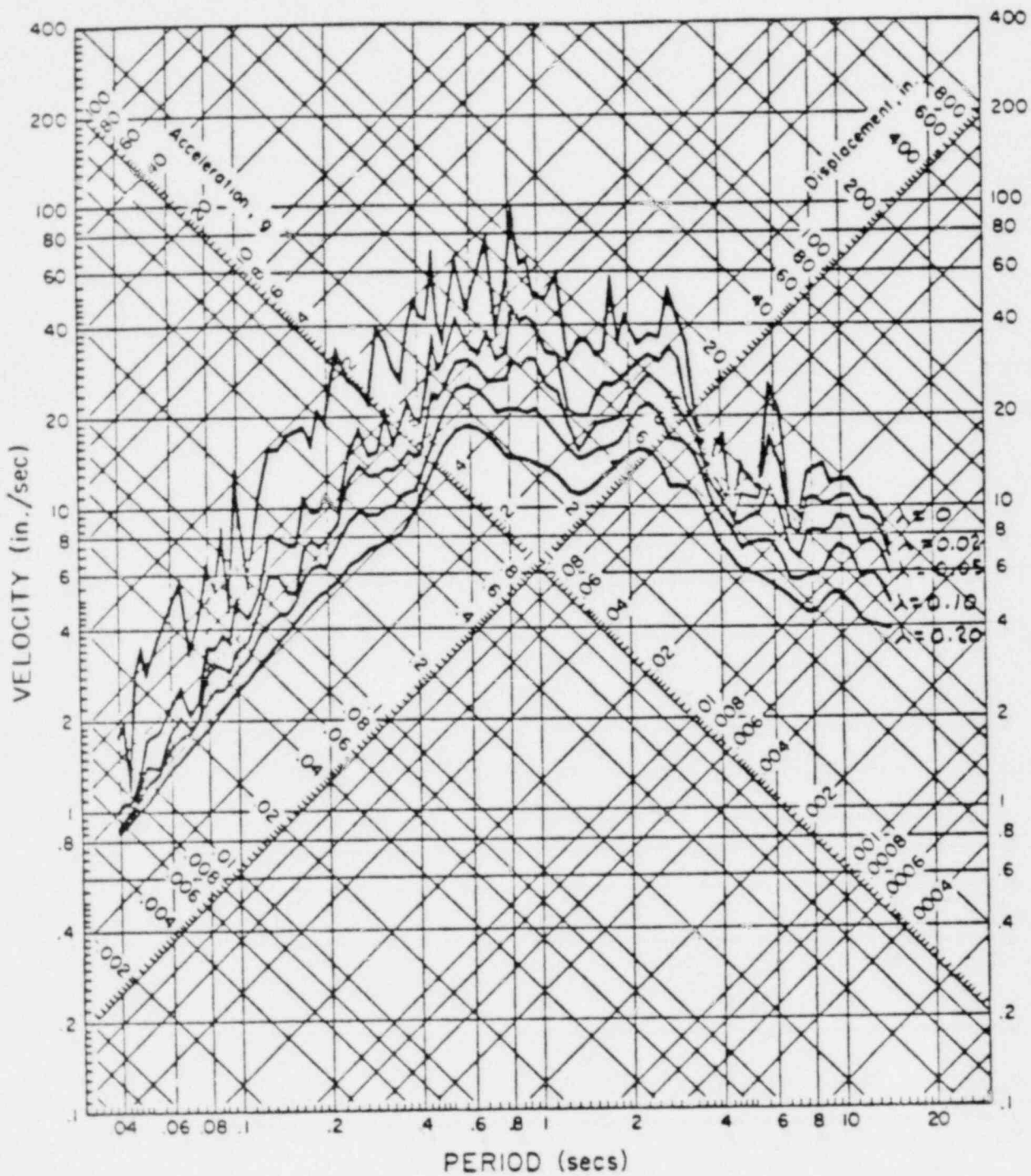


RESPONSE SPECTRA

DAMPING = 0, 2, 5, 10 & 20 %

1940 EL CENTRO EARTHQUAKE, 500E

FIGURE 2



A- WAY LOG PLOT OF RESPONSE SPECTRA
1940 EL CENTRO EARTHQUAKE, 5005

FIGURE 3

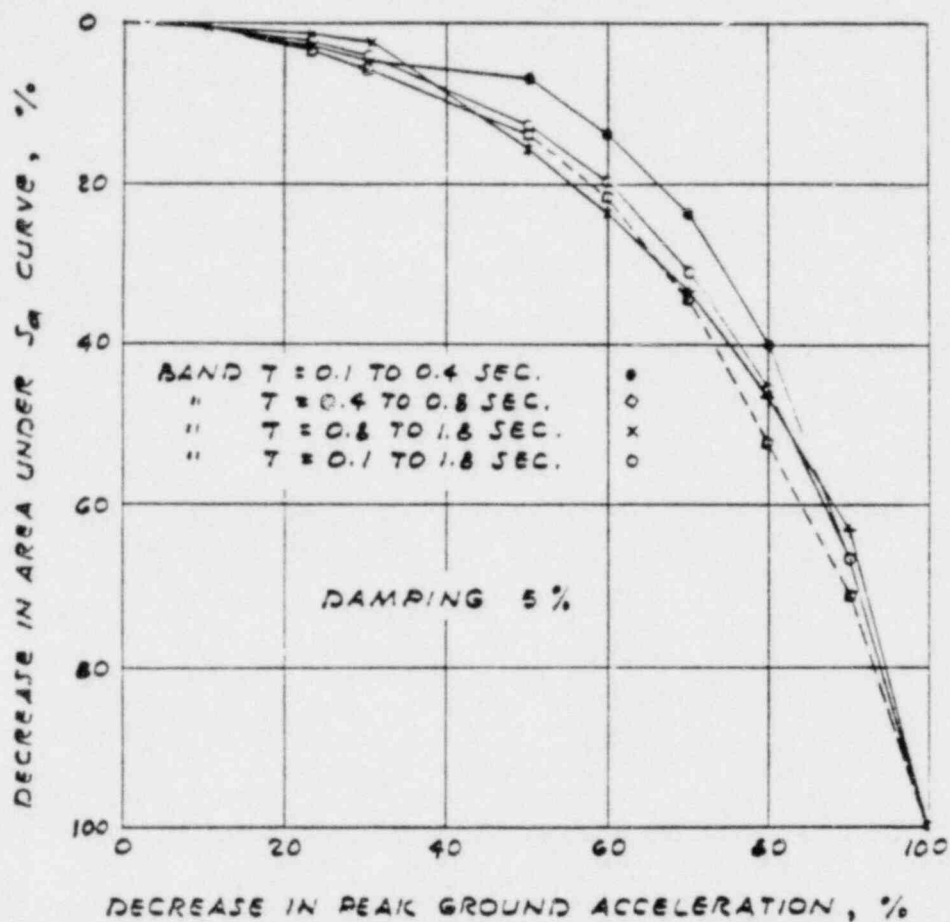


FIG. 4 - EFFECT OF CLIPPING ON SPECTRAL AREAS FOR TAFT 1952, 569E RECORD

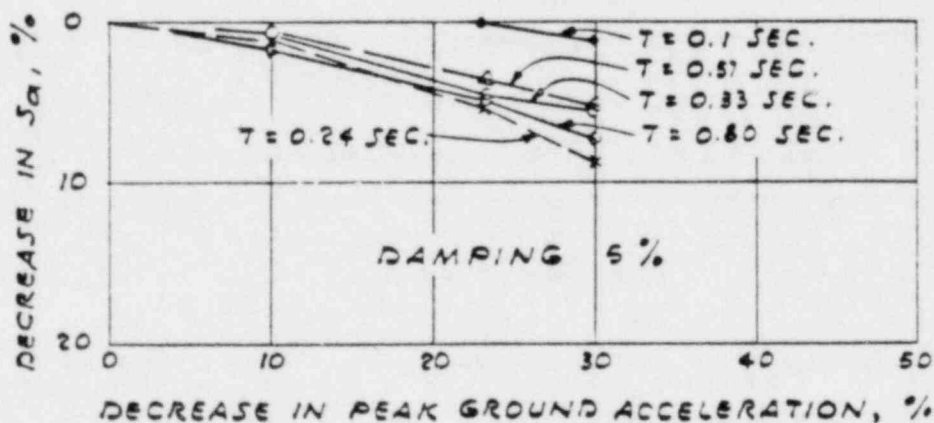


FIG. 5 - EFFECT OF CLIPPING ON SPECTRAL PEAKS, EL CENTRO 1940, NS RECORD

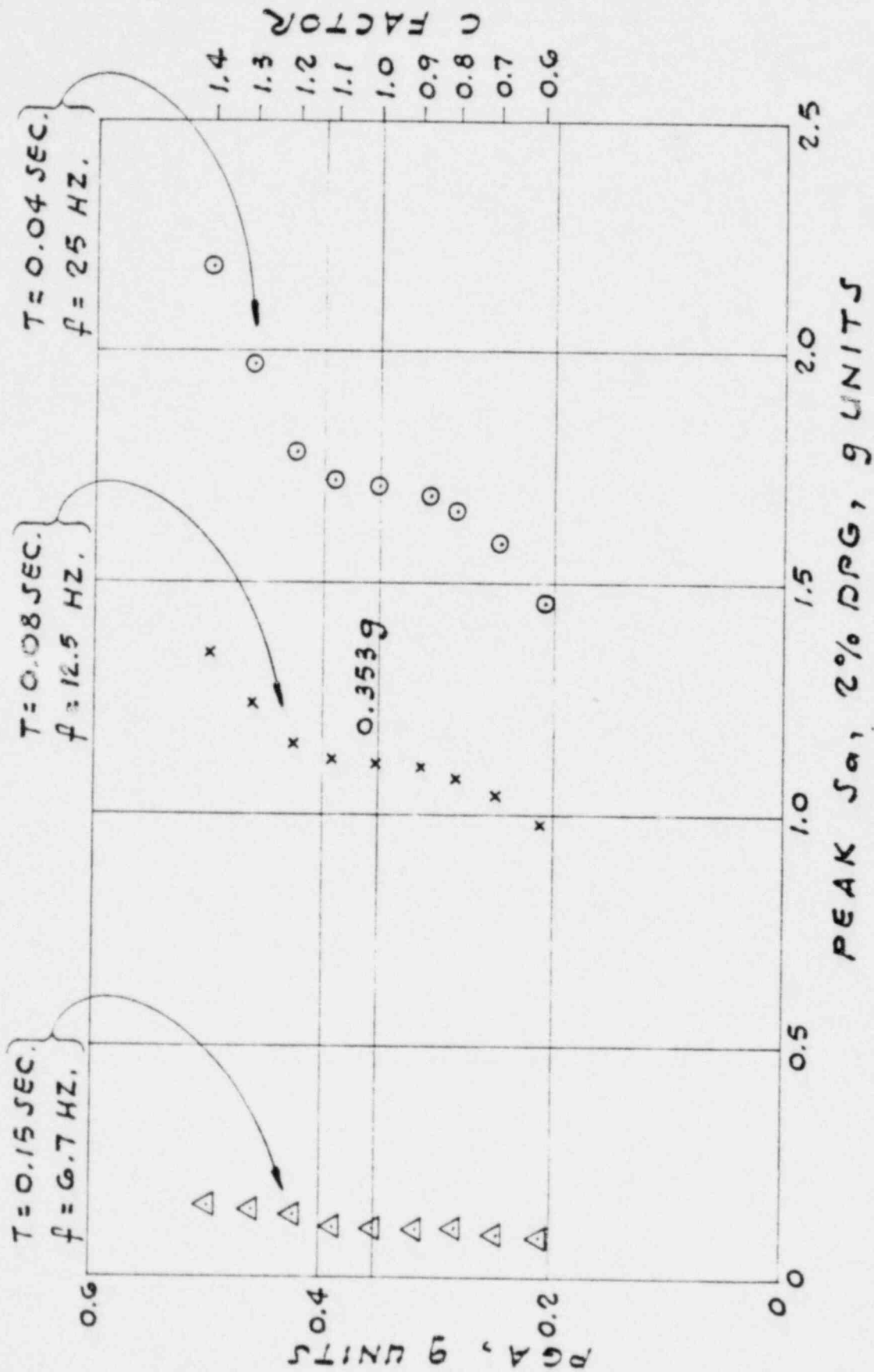


FIG. 6 -- EFFECT OF PGA ON RESPONSE, 2% DAMPING, BASED ON MONTICELLO RESERVOIR EQ. OF OCTOBER 16, 1979, MONTICELLO DAM CREST, 180° COMPONENT

(Fig. 18C-2 in D-LL18C)

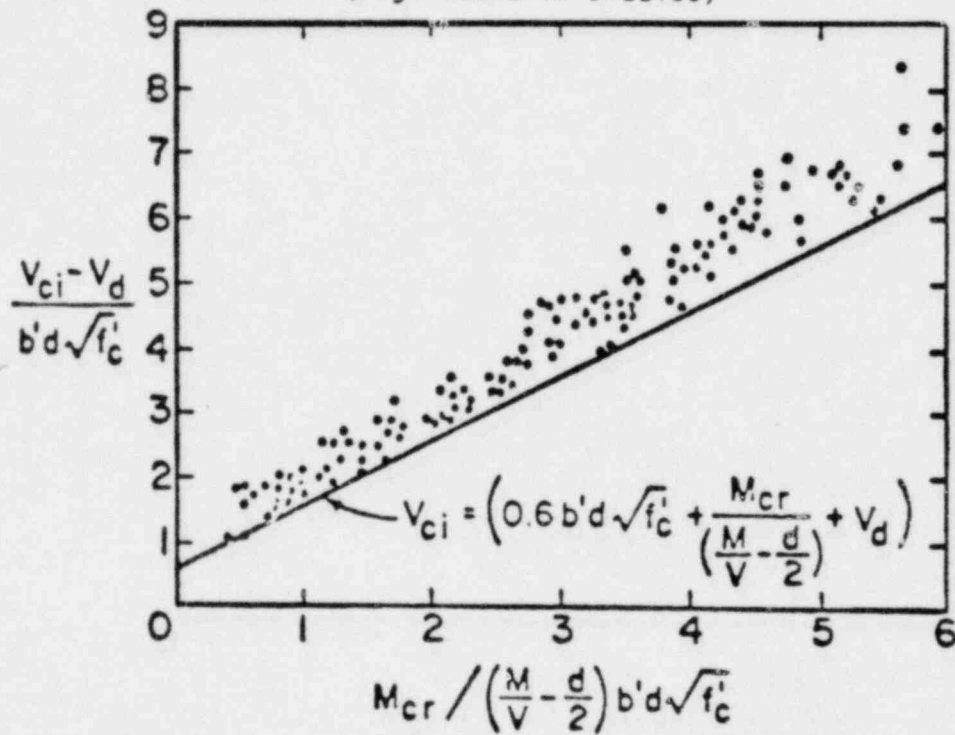
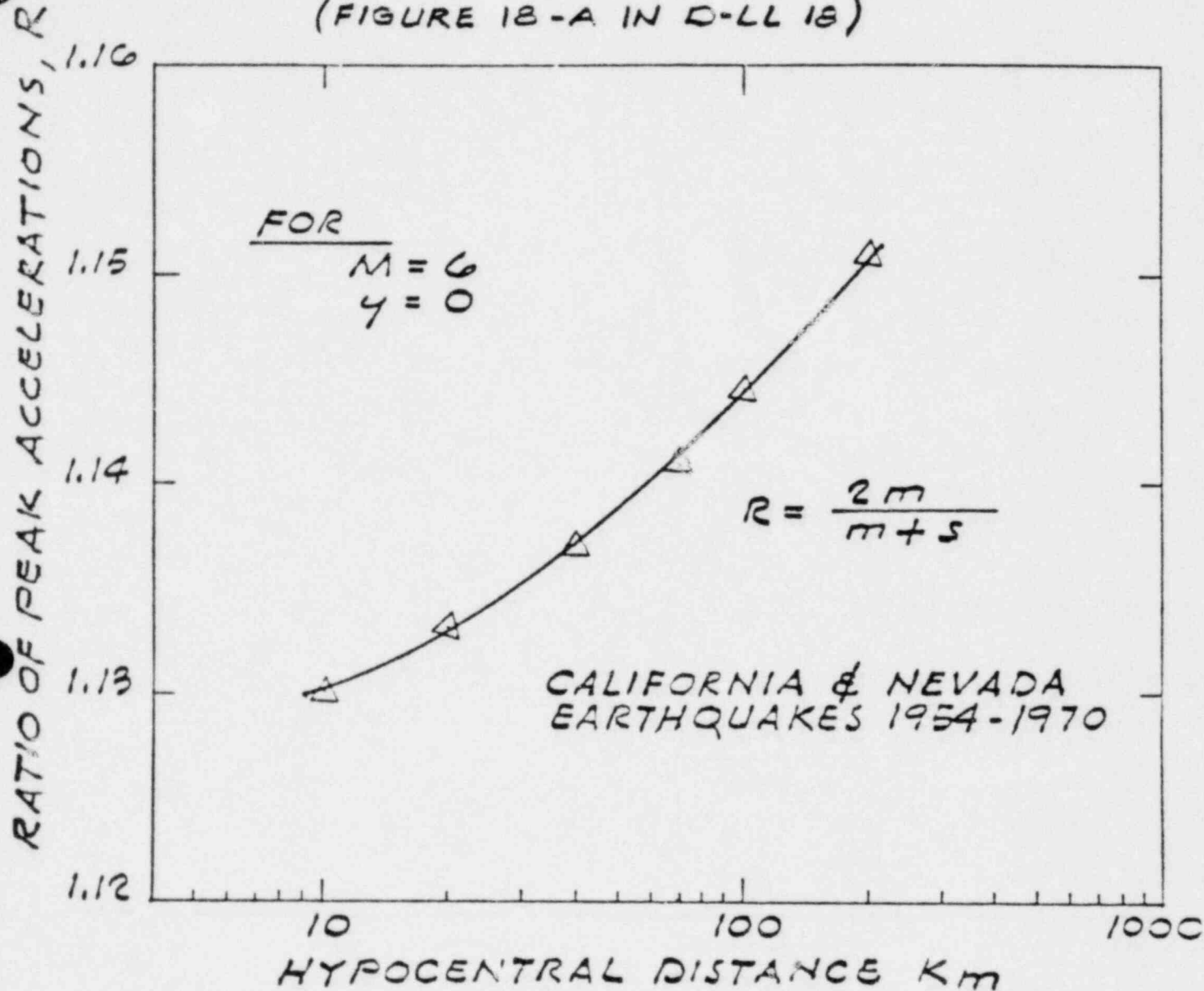


Fig. 7: Diagonal cracking in those regions of beams previously cracked in flexure

FIGURE 7

BLUME

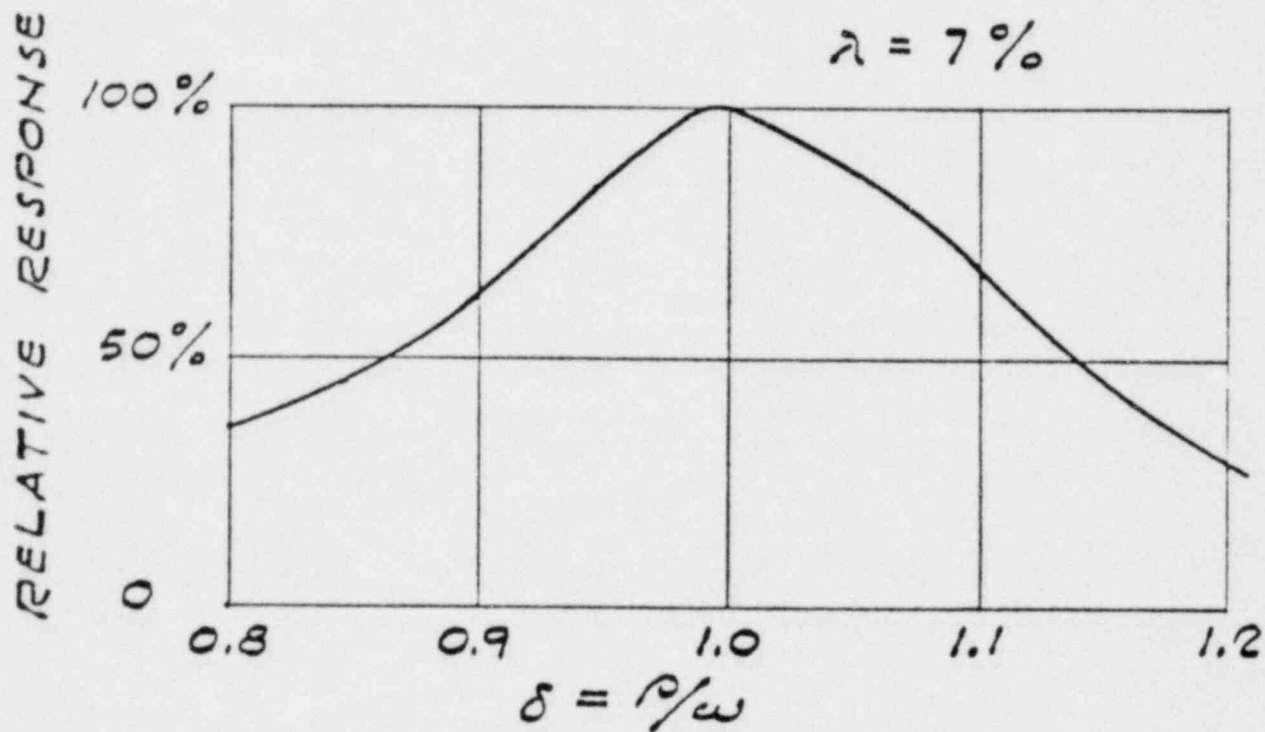
(FIGURE 18-A IN D-LL 18)



RATIO OF MAXIMUM TO AVERAGE
HORIZONTAL PEAK GROUND
ACCELERATION

FIGURE 8

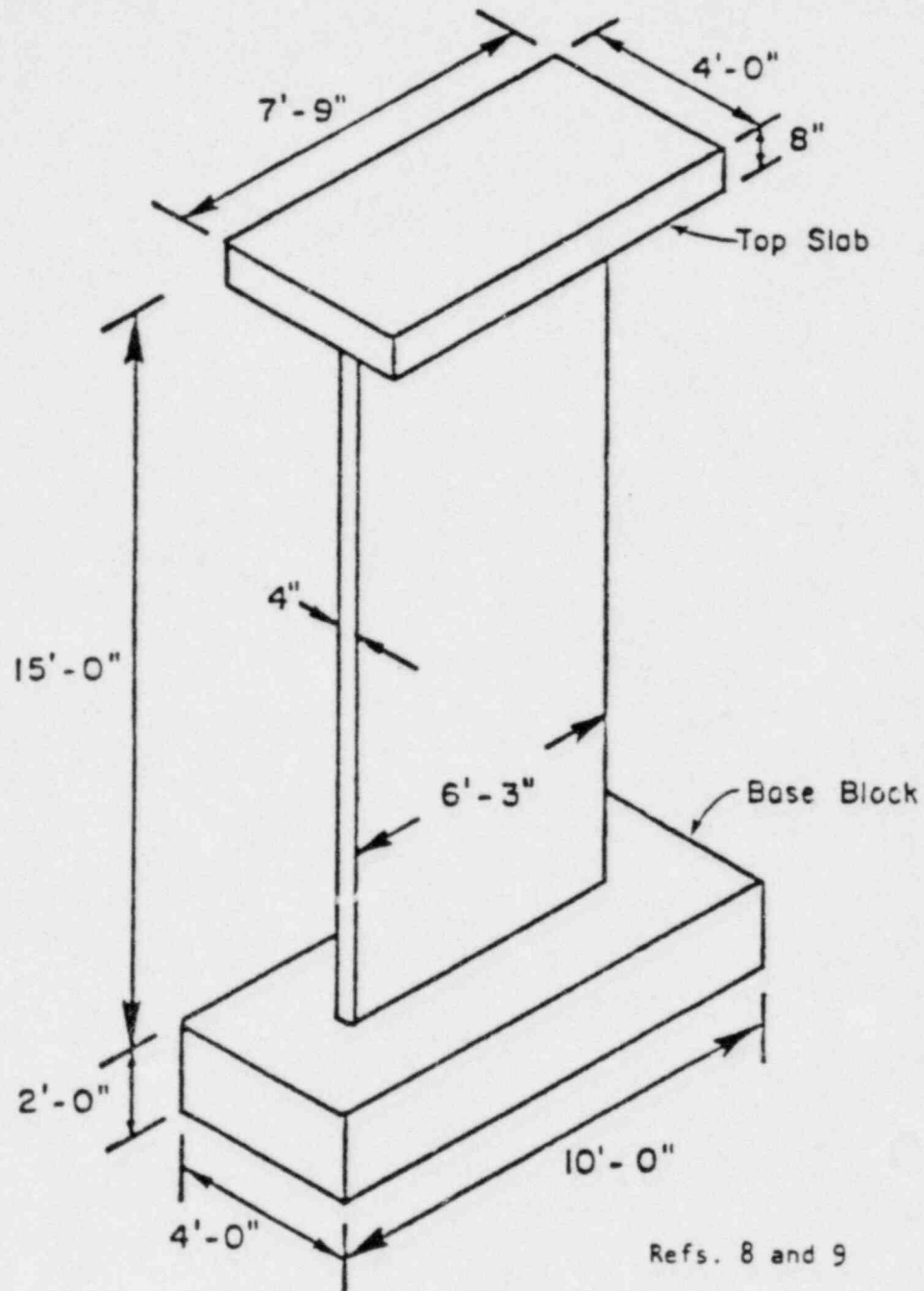
(FIGURE 18-B IN D-LL18)



RELATIVE RESPONSE OF A
7% DAMPED OSCILLATOR
IN THE STEADY STATE UNDER
A HARMONIC FORCING
FUNCTION

FIGURE 9

BLUME

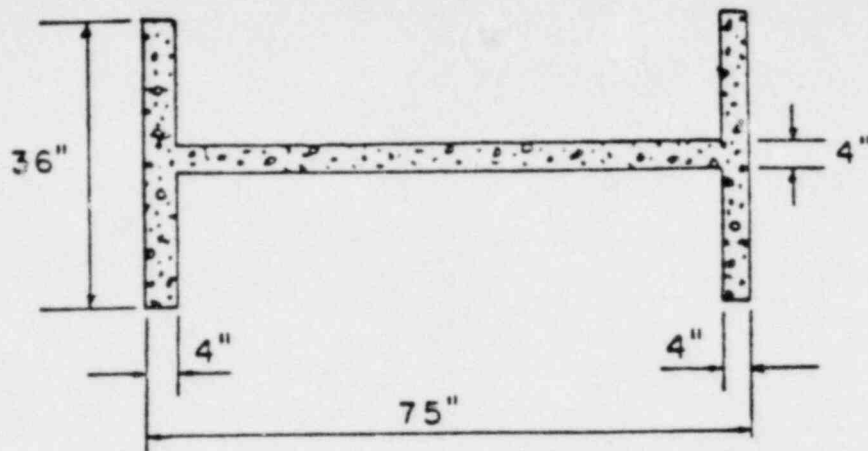


Nominal Dimensions of Test Specimen
With Rectangular Cross Section

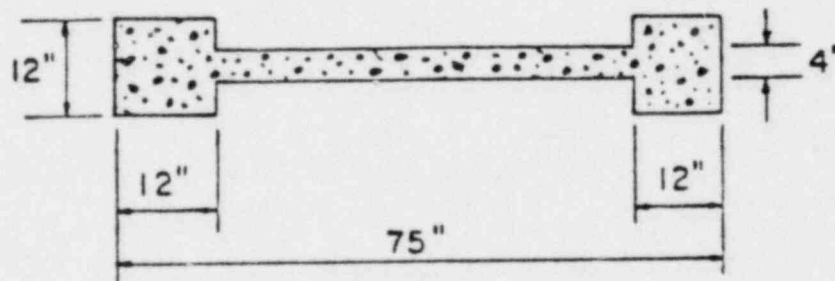
(Figure 9-H, in D-LL9)

FIGURE 10

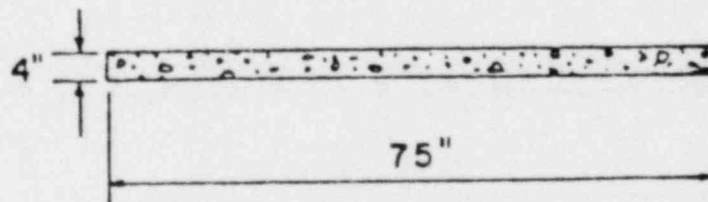
BLUME



(a) Flanged Section



(b) Borbell Section



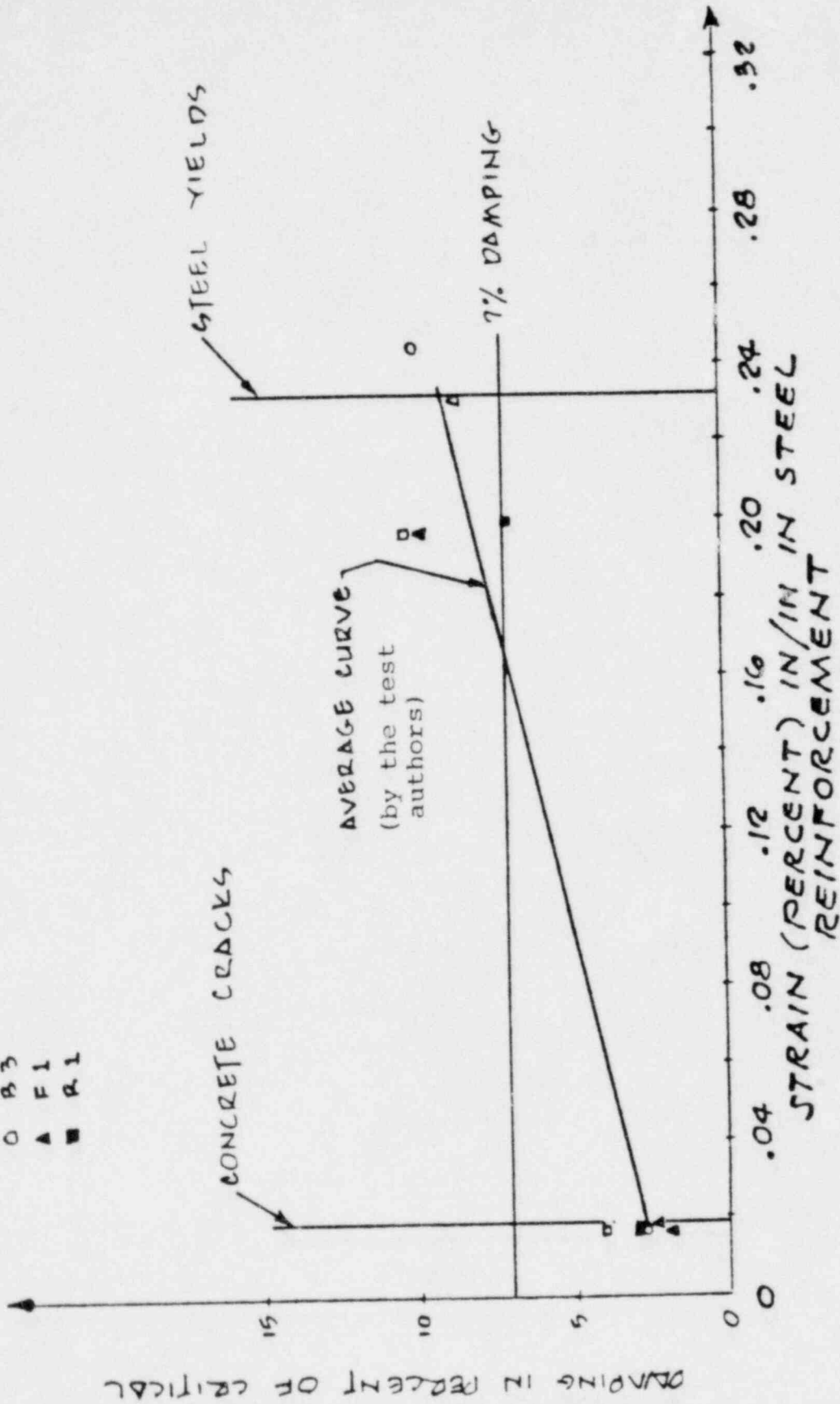
(c) Rectangular Section --- (See Fig. 10)

Nominal Cross-Sectional Dimensions
of Test Specimens

(Figure 9-1 in D-LL9)

FIGURE 11

- Δ B 1
- B 2
- B 3
- ▲ F 1
- R 1



(FIGURE 9-J IN D-LL9)
DAMPING IN PERCENT OF CRITICAL

DAMPING VS. STRAIN LEVEL IN THE
PCA SPECIMENS

FIGURE 12

APPENDICES

<u>NUMBER</u>	<u>TITLE</u>
B2	ON THE RELATIONSHIPS OF PEAK GROUND ACCELERATION, INTENSITY AND LOCAL MAGNITUDE
B3	APPLICATION OF THE "SAM" EQUATIONS
B4	REGARDING USGS OPEN FILE REPORT 81-365
B5	AMPLIFICATION OF MOTION AT OROVILLE DAM
B7	THE STRONG MOTION RECORD OF OCTOBER 16, 1979 AT MONTICELLO RESERVOIR AND STRUCTURAL RESPONSE
B9	ON DIRECTIONAL COMPONENTS OF GROUND MOTION
B10	MATERIAL STRENGTH
B11	SOME LESSONS FROM RIS AT HSINFENGLIANG RESERVOIR
B12	GENERIC LL DOCUMENTS FOR DIABLO CANYON AND THE EL CENTRO STEAM PLANT
B14	GROUND ACCELERATION VERSUS DAMAGE, PROJECT RULISON
B15	UNRECOGNIZED MARGINS

NOTE: THERE IS NO APPENDIX B1, B6, B8, or B13 (THE DATA WERE INCLUDED IN THE ABOVE APPENDICES OR IN THE WRITTEN TESTIMONY).

APPENDIX B2
ON THE RELATIONSHIPS OF PEAK GROUND
ACCELERATION, INTENSITY AND LOCAL MAGNITUDE

By John A. Blume

For purposes of comparison and not necessarily as endorsement, other procedures than the Brune model were used for computations of hypothetical conditions. This appendix is for one such alternative procedure.

J. R. Murphy and L. J. O'Brien wrote a comprehensive paper (BSSA, Vol. 67, No. 3, June 1977) on the relationships of peak ground acceleration, intensity and local magnitude. They used a worldwide data sample with nearly 1500 strong-motion accelerograms. They compare their results to those of prior workers in the same field. Their data as shown in their Figure 4 is quite sparse at short epicentral distances. However, it will be used as another approach to the matter of Summer plant and RIS.

For a subset of data in which $a_H \geq 10 \text{ cm/sec}^2$ and consisting of nearly 900 observations they obtain an equation:

$$\log a_H = 0.14 I_{MM} + 0.24M_L - 0.68 \log R + \beta_k \quad (1)$$

wherein a_H = peak ground horizontal acceleration, cm/sec^2

I_{MM} = Modified Mercalli Intensity

M_L = local magnitude

R = epicentral distance, km

β_k = constant for region k (with values of 0.60,

0.69, and 0.88 for western USA, Japan, and southern Europe, respectively)

The antilog of the standard error for equation 1 is given as 2.00

Following are example data from the above equation for two intensity levels and arbitrary magnitudes and distances:

$$\underline{I_{MM} = VI}$$

$\underline{M_L}$	\underline{R} (km)	Mean Peak Instrumental Acceleration for $\beta_k =$		
		<u>0.60</u>	<u>0.69</u>	<u>0.88</u>
4	5	0.086g	0.11g	0.16g
4	10	0.054g	0.067g	0.10g
4.5	5	0.11g	0.14g	0.21g
4.5	10	0.071g	0.087g	0.13g
5	5	0.15g	0.18g	0.28g
5	10	0.093g	0.11g	0.18g

$$\underline{I_{MM} = VII}$$

4	5	0.12g	0.15g	0.23g
4	10	0.074g	0.091g	0.14g
4.5	5	0.16g	0.19g	0.30g
4.5	10	0.098g	0.12g	0.19g
5	5	0.21g	0.25g	0.39g
5	10	0.13g	0.16g	0.24g

The mean accelerations would double at the mean plus one sigma level, per Murphy and O'Brien.

The above are hypothetical instrumental accelerations based upon the Murphy and O'Brien formula. The β_k value for the east coast is not given. Because there is no evidence of damage from the RIS events to date, and since larger magnitudes, if any, would tend to be deeper than the shallow events of record, it can be assumed that intensities to date have been much less than the VI shown in the table, probably IV or less, and that intensities of V or VI would be the maximum credible RIS events at this site. Taking 4.5 for M_L at 5 km distance as a maximum credible situation for analysis, the average mean peak instrumental acceleration between the β_k bounds would be $(0.11g + 0.21g)/2$ or 0.16g. This value should be multiplied by a factor of $\frac{1}{2}$ or less to allow for the difference of response between the SMA site at the dam abutment and the plant foundations on rock. Whether or not the mean or a value above the mean is taken as a spectral anchor point depends upon the confidence level of the spectral shape to be used.

It would appear that the design values for the plant are reasonable and proper for RIS events based upon the Murphy and O'Brien paper.

APPENDIX B3

APPLICATION OF THE "SAM" EQUATIONS

by John A. Blume

For purposes of comparison and not necessarily as endorsement, other procedures than the Brune model were used for computations of hypothetical conditions. This appendix is for one such alternative procedure.

The first attenuation relationship was developed by Gutenberg and Richter (BSSA 46:2, 1956). The second was by Blume (Proc. 3WCEE, 1965) which was termed "SAM" for Site-Acceleration-Magnitude. The original SAM equation has long since been superseded by new Blume equations as more data became available. In spite of frequent literature reference to the current equations, termed SAM IV and SAM V, replacing all previous versions, several have continued to use the older equations and even have done so erroneously by not providing for proper site impedance factors as called for by the method. The current SAM IV and V (Proc. 6WCEE, 1977) are based upon 795 records from California and other western states and include all strong motion data from "U.S. Earthquakes" through 1970. In addition, data from 2713 accurate records of motion from underground nuclear explosions are utilized to develop the effect of local site conditions on the motion.

A copy of a brief paper on SAM IV and V from the proceedings of the Sixth World Conference on Earthquake Engineering (New Delhi,

1977) is attached.

The current Blume SAM equations are:

SAM IV (for $M \leq 6\frac{1}{2}$):

$$a_y = 0.318 \exp^{1.03M} (29)^{1.14\bar{B}} (R + 25)^{-1.14\bar{B}} (2.53)^y \quad (1)$$

SAM V (for $M > 6\frac{1}{2}$):

$$a_y = 26.0 \exp^{0.432M} (29)^{1.22\bar{B}} (R + 25)^{-1.22\bar{B}} (1.81)^y \quad (2)$$

$$\text{In both cases, } \bar{B} = 1/2 \log_{10} (\rho V_s) \quad (3)$$

wherein:

a_y = peak horizontal ground acceleration associated
with probability level y , gal

\bar{B} = the Blume site factor

M = magnitude as given in U.S. Earthquakes

R = hypocentral distance, km

V_s = site shear velocity, ft/sec

ρ = site specific density, dimensionless

y = standard normal variable with zero mean and unit
standard deviation

Several hypothetical situations have been computed using equations 1 and 3, with ρV_s taken arbitrarily at 12,000 ft/sec.* From equation 3,

$$\bar{B} = 1/2 \log (12,000) = 2.04$$

For values of magnitude below 5.5 there is not a great difference

*At close-in distances, the results are insensitive to ρV_s .

between M_S and M_L . M values of 4 to 5.5 were used with R values taken as equal to M, in km. This is arbitrary, but it is a conservative assumption since by geometry, larger M's would tend to be farther and deeper from a given point than smaller M's.* The value of R could no doubt better be taken as 1.2M or 1.4M.

The results are shown in the following table.

"SAM" Accelerations for $\rho V_S = 12,000$ fps

<u>M</u>	<u>R</u> (km)	<u>a_y, g units</u>		<u>Zero period accel-</u> <u>eration using 0.60 a_y</u>	
		<u>y = 0</u>	<u>y = 1</u>	<u>y = 0</u>	<u>y = 1</u>
4	4	0.020	0.051	0.012	0.031
4.5	4.5	0.032	0.081	0.019	0.049
5	5	0.052	0.13	0.031	0.078
5.5	5.5	0.083	0.21	0.050	0.13

The factor 0.60 in the last two columns is a reduction factor to convert peak (instrumental) ground acceleration to effective or zero-period acceleration for design response spectra.**

The value of ρV_S is not sensitive at close distances; in fact, as can be seen by the equations, when $R = 4$, the value used has no effect on the results.

*See also appendix B-11

**ALAB-644 Decision, p.72; Note also that the explosives test at the plant site indicate a reduction factor of 0.5 or less.

Based on the above table, the Virgil C. Summer plant with 0.15g for rock and 0.25g for soil would be able to withstand any of the above hypothetical events at $y = 0$ (median values). If one standard deviation above the mean ($y = 1$) needs to be considered, a reduction factor should also be applied, and the plant zero period criteria would be adequate for all events.

Enclosure, "SAM" paper

THE SAM PROCEDURE FOR SITE-ACCELERATION-MAGNITUDE RELATIONSHIPS

by

John A. Blume^I

SYNOPSIS

Early work on the relationships of site characteristics, horizontal peak accelerations, magnitude, attenuation with distance, and probabilistic variations is extended with more data, refined, and simplified. New estimation procedures called SAM IV and SAM V are provided to supersede previous SAM versions. The data used include all California and western Nevada strong motion records from 1933 through 1970 and, for studies of rock and alluvium motion, statistics from 2,713 records of ground motion induced by underground nuclear explosions. Comparisons to studies and estimation procedures by others are provided.

GLOSSARY OF TERMS

- a = peak ground acceleration, gal
- a_y = peak ground acceleration associated with probability level y, gal
- b_1, b_2, b_3 = constants determined from the data
- \bar{b} = the Blume site factor per Equation (4)
- G = the standard geometric deviation
- ln = natural logarithm, base e
- M = Richter magnitude, as given in United States Earthquakes
- R = hypocentral distance, km
- SAM = acronym for Site-Acceleration-Magnitude
- V_s = site shear velocity, ft/sec
- ρ = site specific density, dimensionless
- y = standard normal variable with zero mean and unit standard deviation

INTRODUCTION

In an earlier paper (1), I outlined a procedure for estimating the relationships of site materials, horizontal peak acceleration, magnitude, and epicentral distance, which came to be known as SAM, for Site-Acceleration-Magnitude. Subsequently, the procedure was improved to include its probabilistic aspects on a more formal basis; this became SAM II. Another version, which included more data, became SAM III. Neither SAM II or III were published except in report form. In recent years others have published papers comparing the results of different studies and data sets. Some of these comparisons have been based on soil characteristics improper for SAM comparisons, and in one case the SAM equation was reprinted incorrectly. In view of this and the availability of more recorded ground motion data from both earthquakes and underground nuclear explosions, new

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procedures called SAM IV and SAM V, have been developed, superseding the earlier SAM, SAM II, and SAM III. SAM IV applies to earthquake magnitudes of $6\frac{1}{2}$ or less and SAM V to magnitudes greater than $6\frac{1}{2}$. The SAM procedures continue to be unique in that they consider, in simple form, magnitude, epicentral distance, focal depth, site characteristics, peak acceleration, and probabilistic variations.

THE DATA

The natural earthquake data used are from United States Earthquakes (2), which provides the official record of corrected peak acceleration, magnitude, and epicentral distance. All data from 1933 through 1970 for California and western Nevada were used. This excludes data from the 1971 San Fernando earthquake, which were not published at the time of the study. Furthermore, it seemed desirable not to include the bias that might result from so much data from one earthquake, especially from a somewhat non-typical thrust fault motion. The focal depth is not provided in United States Earthquakes, and yet it was desired to use hypocentral rather than epicentral distance. SAM IV and V involve the considered assumption that the average focal depth is 8 km.

A statistical study such as this should include all appropriate data. Using only the greater of the two horizontal motions does not seem logical or representative of design conditions and tends to bias the data. In this study, both horizontal values were used when available in United States Earthquakes. Some investigators have used all data available at the time of their study; some have arbitrarily cut off at some level such as 1 gal, 5 gal, or more; and some have used a combination of cutoff levels. Of course, the instrument used to record the motion is in itself a cutoff. The effects of cutoff were studied.

Altogether, 795 horizontal strong motion record-components from natural earthquakes were used. In addition, for consideration of the relative motion on rock and alluvium at various distances, results from the statistical analysis of 2,713 ground motion records from nuclear seismology were included (3).

TREATMENT OF THE DATA WITHOUT SITE ADJUSTMENT

After considerable study, the form of equation selected (4) was

$$a = b_1 e^{b_2 M} (R + 25)^{-b_3} \quad (1)$$

Taking the natural logarithm of Equation (1) results in Equation (2):

$$\ln a = \ln b_1 + b_2 M - b_3 \ln(R + 25) \quad (2)$$

There are two independent variables, M and $\ln(R + 25)$, and one dependent variable, $\ln a$. Multiple linear regression analysis (5) was performed with various data sets to obtain the mean value of the three variables, the three constants, and the standard deviation ($\ln G$). At this stage no distinction was made for site conditions. Table I provides data for several runs. The expression for the acceleration at any probability level, y, in lognormal form is

$$a_y = e^{\ln b_1 + b_2 M} (R + 25)^{-b_3} G^y \quad (3)$$

The results for runs 51 and 52 are almost the same, indicating very few data points below 1 gal. The cutoff levels for acceleration in runs 52, 53, and 54 affect $\ln G$ significantly. It was decided to use all data (run 51) for developing SAM IV for M values not exceeding 6.5. It can be seen by comparing runs 51 and 71 that there is little difference whether all data (run 51) are used or only data equal to or less than 6.5M (run 71) are used. There is less dispersion in the major earthquake data. It was decided after study that the data set of run 56 provided the optimum combination of M cutoff level and the number of sample points for major earthquakes, and it was used as the basis of SAM V for $M > 6.5$.

SAM IV AND SAM V

The original SAM procedure included adjustments for site characteristics based on data from the work of Gutenberg and Richter (6). New data show that the relative peak motion on rock versus alluvium is strongly dependent on epicentral distance. Although there is still a sparsity of data for rock stations under earthquake motion, there is considerable information from both rock and alluvium stations under motion from underground nuclear explosions. Figure 1 shows the ratio of peak acceleration in alluvium to that on rock plotted against hypocentral distance. This plot is based on a statistical study of 1,911 records on alluvium and 802 on rock (3). Most of the records were taken in Nevada as part of the seismic effects monitoring program associated with underground nuclear detonations at the Nevada Test Site. The findings are consistent with the more limited data from natural earthquake records. It was assumed that rock motion and alluvial motion are equal at 4 km hypocentral distance (see Figure 1), and also that the site impedance, taken as the product ρV_S , as in the original SAM procedure (1), is the best single measure of site conditions. Consideration of station conditions where earthquake strong motion has been recorded in California and western Nevada led to the assumption that the average ρV_S for the 1933-1970 data analyzed in Table I can be taken as 2,000 fps. Adjustments are to be made for other site conditions.

The original Blume site factor, \bar{b} , was determined from plots (1). It has since been found that Equation (4) gives equally useful results.

$$\bar{b} = \frac{1}{2} \log_{10} (\rho V_S) \quad (4)$$

With Equation (4), \bar{b} for 2,000 fps is 1.65, which is applicable to the data in Table I. Exponent b_3 in Equation (3) is replaced by $x\bar{b}$, where $x = b_3/\bar{b}$. Moreover, Equation (3) will be normalized so that, at $R = 4$ km, the peak acceleration will be the same for all values of \bar{b} and motion will be as provided by the data from run 51 for SAM IV and run 56 for SAM V, all at constant values of y . The relative attenuation of $\rho V_S = 2,000$ material to $\rho V_S = 12,000$ material will be generally in accordance with Figure 1. Thus

SAM IV (for $M < 6\frac{1}{2}$):

$$a_y = 0.318e^{1.03M} (29)^{1.14\bar{b}} (R + 25)^{-1.14\bar{b}} (2.53)^y \quad (5)$$

SAM V (for $M > 6\frac{1}{2}$):

$$a_y = 26.0e^{0.432M} (29)^{1.22\bar{b}} (R + 25)^{-1.22\bar{b}} (1.81)^y \quad (6)$$

The value of y selected is to be associated with the corresponding probability of exceedance from standard tables. When $y = 0$ the median acceleration is obtained. The value of a_y is the estimated peak acceleration that would be recorded by an instrument. It is not intended to be used directly in design or as a seismic coefficient without adjustment for other considerations (7). The equations are for California and western Nevada data, which may not be a good model for other locations.

COMPARISONS

Comparisons of attenuation curves and relationships of site characteristics, acceleration, and magnitude are difficult because of the complexity of the problem and the fact that investigators have used different data sets, parameters, cutoff levels, assumptions, and analyses. Comparisons will be made here by superimposing SAM curves on three plots by others.

Figure 2 is a plot by Donovan (8), in which the original SAM data (1) were plotted erroneously and/or hard rock ρV_S values were used erroneously in comparison to data generally for soft materials. The bottom curve should be replaced by the heavy curve, which is SAM (1) for $\rho V_S = 2,000$ fps, a better basis for comparison. A SAM IV curve, not shown, would be better.

Figure 3 is a set of curves by Trifunac and Brady (9). SAM IV curves for $M = 6.5$ and $\rho V_S = 3,000$ fps are superimposed for $y = 0, 1, \text{ and } 2$. The $y = 0$ curve coincides with that shown for Esteva (4). If ρV_S were a smaller value, such as 2,000 fps, the accelerations would be greater at long distances.

Figure 4 from Page et al (10) shows acceleration points for three levels of magnitude. Curves are superimposed for $M = 7$ by the SAM V equation with $\rho V_S = 2,000$ fps. Magnitude 7 is an average value for the data points from 6.0 to 7.9. Disregarding the 5.0 to 5.9 points, there is good correlation of the $M = 7$ curves and the 6.0 to 7.9 data points.

Trifunac (17) plotted curves (not shown) for peak acceleration for three magnitudes, three site classifications, and 0.9 confidence level. SAM V was used to plot comparison curves for his 8.5M and 5.5M earthquakes with the same confidence levels, using $\rho V_S = 2,000$ fps for soft soil and 12,000 fps for hard soil. There was good general correlation for 8.5M except that SAM V provided somewhat lower values at short distances and somewhat greater values for soft soil at long distances. SAM V also provided more variation between soft and hard soil at long distances and less at short distances. SAM IV was used for 5.5M with generally good comparisons beyond 20 or 30 kilometers and lower values at shorter distances.

REFERENCES

1. Blume, J. A., "Earthquake Ground Motion and Engineering Procedures for Important Installations Near Active Faults," Proceedings, 3WCEE, 1965.
2. United States Earthquakes, U. S. Department of Commerce, Washington, D. C., 1933-1970.
3. Environmental Research Corporation, Prediction of Ground Motion Characteristics of Underground Nuclear Detonations, report NVO-1163-239 to NVO, U.S. ERDA, March 1974. Available from NTIS, Springfield, Virginia.
4. Esteva, L., "Seismic Risk and Seismic Design Input for Nuclear Power Plants," Seismic Design for Nuclear Power Plants, The MIT Press, 1970.
5. Hald, A., Statistical Theory with Engineering Applications, John Wiley & Sons, New York.

6. Gutenberg, B., and Richter, C., "Earthquake Magnitude, Intensity, Energy, and Acceleration (second paper)," Bull. Seis. Soc. Am., Vol. 46:2, 1956.
7. Blume, J. A., "Allowable Stresses and Earthquake Performance," 6WCEE.
8. Donovan, N. C., "A Statistical Evaluation of Strong Motion Data including the February 9, 1971, San Fernando Earthquake," Proceedings, 5WCEE, 1973.
9. Trifunac, M. D., and Brady, A. G., "On the Correlation of Peak Acceleration of Strong Motion with Earthquake Magnitude, Epicentral Distance and Site Conditions," U.S. National Conf. on Earthquake Engineering, 1975.
10. Page, R. A., Blume, J. A., and Joyner, W. B., "Earthquake Shaking and Damage to Buildings," Science, Vol. 189, August 1975.
11. Housner, G. W., "Intensity of Earthquake Ground Shaking Near the Causative Fault," Proceedings, 3WCEE, 1965.
12. Kanai, K., "Improved Empirical Formula for Characteristics of Stray Earthquake Motions," Proceedings, Japan Earthquake Symposium, 1966.
13. Milne, W. B., and Davenport, A. G., "Distribution of Earthquake Risk in Canada," Bull. Seis. Soc. Am., Vol. 59:2, 1969.
14. Cloud, W. K., and Perez, V., "Unusual Accelerograms Recorded at Lima, Peru," Bull. Seis. Soc. Am., Vol. 61:3, 1971.
15. Schnabel, P. B., and Seed, H. B., "Accelerations in Rock for Earthquakes in the Western United States," Bull. Seis. Soc. Am., Vol. 63:2, 1973.
16. Nuttli, O. W., "The Mississippi Valley Earthquakes of 1811 and 1812: Intensities, Ground Motion, and Magnitudes," Bull. Seis. Soc., 63:1, 1973.
17. Trifunac, M. D., "Preliminary Analysis of the Peaks of Strong Earthquake Ground Motion," Bull. Seis. Soc. Am., Vol. 66:1, 1976.

TABLE I - DATA FROM MULTIPLE REGRESSION ANALYSES*

Run	M	Mean value of M	Mean of $\ln a$ (gal)	Mean of $\ln(R+25)$ (km)	$\ln b_1$	b_2	b_3	$\ln G$
51	All	5.324	2.330	4.435	5.195	1.030	1.883	0.930
52	All	5.326	2.339	4.434	5.211	1.020	1.873	0.923
53	All	5.311	2.897	4.291	5.044	0.805	1.497	0.770
54	All	5.324	3.330	4.182	5.265	0.691	1.343	0.727
70	$\geq 5\frac{1}{2}$	6.189	2.292	4.926	5.913	1.045	2.049	0.801
55	≥ 6	6.548	2.269	5.149	7.464	0.900	2.154	0.768
56	$\geq 6\frac{1}{2}$	6.871	2.655	5.142	10.026	0.432	2.010	0.592
57	$\geq 6\frac{3}{4}$	7.363	2.494	5.334	9.883	0.516	2.097	0.372
66	≥ 6	6.531	3.176	4.742	7.934	0.815	2.125	0.742
67	$\geq 6\frac{1}{2}$	6.843	3.553	4.718	9.519	0.412	1.862	0.677
68	$\geq 6\frac{3}{4}$	7.231	3.300	4.928	10.408	0.469	2.130	0.275
71	$< 6\frac{1}{2}$	5.127	2.288	4.344	5.123	1.034	1.873	0.959
72	$> 6\frac{1}{2}$	7.200	2.877	5.086	8.940	0.659	2.125	0.470

*All United States Earthquakes data (1933 through 1970) were used except in runs 66, 67 and 68, for which distances > 150 km were deleted; in runs 52, 53, and 54 accelerations were cut off at 1, 5, and 10 gal, respectively.

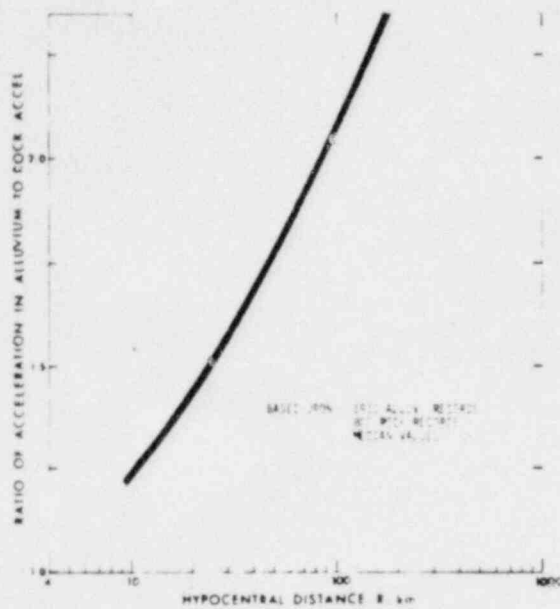


FIG. 1--ACCELERATION RATIOS FOR NUCLEAR EXPLOSIONS

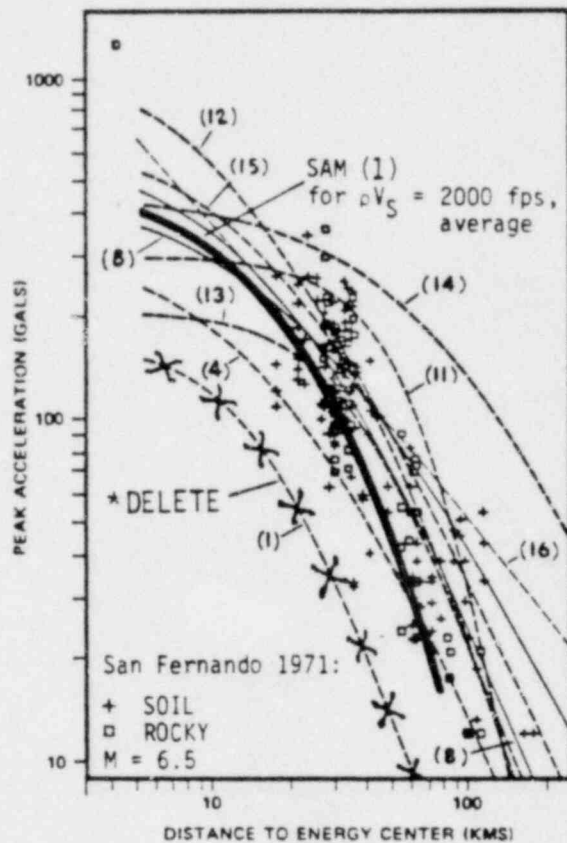


FIG. 2--COMPARISON OF ORIGINAL SAM TO REFERENCE 8 DATA

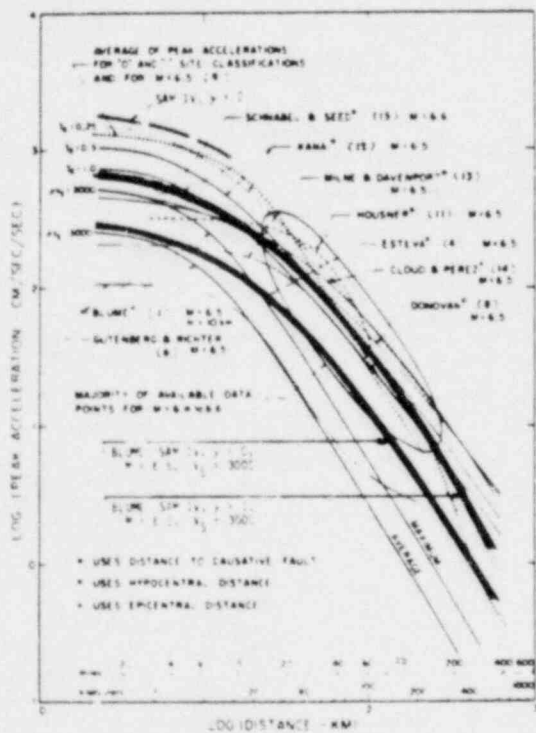


FIG. 3--COMPARISON OF SAM IV TO REFERENCE 9 DATA

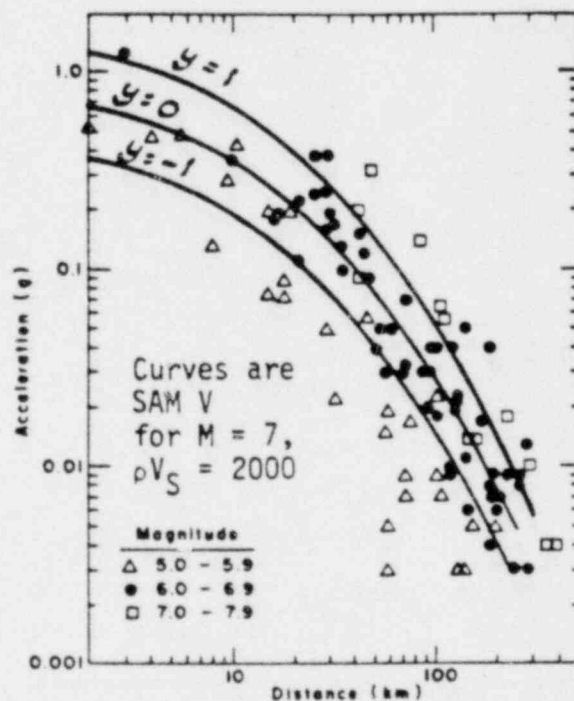


FIG. 4--COMPARISON OF SAM V TO REFERENCE 10 DATA

*The word "delete" refers here to an erroneous plot by others in prior papers; i.e., the curve shown is not comparable to the others.

APPENDIX B4

REGARDING USGS OPEN FILE REPORT 81-365

by John A. Blume

For purposes of comparison and not necessarily as endorsement, other procedures than the Brune model were used for computations of hypothetical conditions. This appendix is for one such alternative procedure.

USGS Open File Report 81-365 (Joyner, et al) gives (for western data):

$$\log A = -1.23 + 0.280M - \log r - 0.00255r + 0.27P$$

wherein

A = peak horizontal ground acceleration, g units

$$r = (d^2 + 7.3^2)^{1/2}$$

P = 0 for median; = 1 for plus one standard deviation

d = closest distance to rupture, km.

Using d = 3 km, r = 7.89. For this value the equation produces the following accelerations:

<u>M</u>	<u>Median</u>	Median plus One σ	Factored by 0.60 for effective value	
			<u>Median</u>	<u>Median + One σ</u>
4.0	0.094g	0.17g	0.056g	0.10g
4.5	0.13g	0.24g	0.078g	0.14g
5.0	0.18g	0.33g	0.11g	0.20g
5.5	0.25g	0.46g	0.15g	0.28g

The Summer plant has design target values of 0.15g and 0.25g rock and soil, respectively. The actual plant values are no doubt

considerably greater as discussed elsewhere. The Joyner et al OFR values, based mostly on soil data, compare very well with the plant criteria as shown above.

APPENDIX B5

AMPLIFICATION OF MOTION AT OROVILLE DAM

by John A. Blume

Large earthen structures amplify earthquake motion over that of the ground per se. Earth-fill dams, even natural peaks or hills, constitute "structures" in this sense. Motion recorded on the crest of an earth-fill dam or a hill by no means represents the motion of the "free field" or the motion that would shake a building or a plant at foundation level.

It is my opinion, based upon what I have learned to date, that the essentially crest-level motion recorded August 27, 1978 and October 16, 1979 was amplified due to dynamic response of the soil and/or the topography.

As an example of amplification phenomenon I refer to the Oroville Dam which is a very large earth-fill dam in California. On August 1, 1975, that dam was shaken by a 5.7 M_L earthquake with its epicenter about 12 km away. There was no damage to the dam but the measured motion at the crest level was several times that measured at the ground level. Tables 1, 2 and 3 show data from a report by the USGS dated September 9, 1975, entitled "Preliminary Strong Motion Spectral Analysis of the Oroville Earthquake of August 1, 1975". Brady and Perez were the apparent authors. The ratios shown are mine, from their data.

Table 1 - Absolute Peak Accelerations, cm/sec²

At the crest:	<u>N46E</u> (Transv)	<u>N44W</u> (Long)	<u>Vertical</u>
	115.	85.4	129.
At ground level:	<u>N37E</u>	<u>N53W</u>	<u>Vertical</u>
	35.8	32.1	46.1
Ratio: <u>crest</u> ground	3.21	2.66	2.80

Table 2 - Absolute Peak Velocities, cm/sec
(same directions)

At the crest:	12.9	6.1	6.4
At ground level:	1.8	2.0	2.1
Ratio: <u>crest</u> ground	7.17	3.05	3.05

Table 3 - Absolute Peak Displacements, cm
(same directions)

At the crest:	1.6	2.1	1.2
At ground level	0.5	0.6	1.1
Ratio: <u>crest</u> ground	3.20	3.5	1.09

The horizontal motion at the crest was roughly 3 to 7 times greater than at ground level. Obviously, this dam was responding to the motion in its natural modes. The Fourier spectra bear this out nicely with very sharp peaks at the crest at about 1.2 Hertz and no such peaks at the base where the Fourier peaks are at about 5 Hertz.

The record taken of the earthquakes near the dam of Monticello Reservoir should not be considered as representative of the motion

that would drive the plant -- it would no doubt be, instead, amplified motion. The amount of amplification would, of course, be much less than shown above. But even a fraction of the above ratios would destroy the applicability of the record to Summer.

APPENDIX B7

THE STRONG MOTION RECORD OF OCTOBER 16, 1979 AT MONTICELLO RESERVOIR AND STRUCTURAL RESPONSE

by John A. Blume

The purpose of the study reported herein was to obtain the relationship of peak acceleration measured October 16, 1979 near the crest of the dam and on a natural hillock at the Monticello Reservoir and the peak response spectrum acceleration. It was found that the measured peak acceleration values could be arbitrarily decreased with but minor resulting changes in the response spectrum which is the true indicator of response. This is one indication of why effective accelerations are indicated for design purposes rather than peak instrumental accelerations.

The strong motion record of October 16, 1979 at Monticello Reservoir was taken on top of a (former) natural hill against which were constructed two dams, B and C. In my opinion -- and this subject will be addressed elsewhere -- the record is not that of the "free field" but it shows amplified motion due to response of the hill, of the dams and/or of the soil at the instrument site.

180° Component

The record at 180°, shown in Figure B7-1, is very short, with a significant motion duration of less than 0.40 seconds. As shown in Figure B7-2, the spectral response peaks at very high

frequencies.

The time history (record) should not be applied or scaled for the plant analysis because pronounced spikes or peaks on acceleration time histories have little if any effect on structures. This has been demonstrated for many natural earthquake records (Blume, 1977) and is the subject of this paper which for now neglects the non-applicability of the record to the Summer Plant and analyzes the record per se without prejudice.

There is good scientific reason why sharp acceleration peaks at high frequency have little structural significance. The integration of acceleration with respect to time yields velocity which in turn is a measure of energy. It is energy that shakes structures. The time interval within the peaks is so small that the product or integration of time and acceleration is likewise too small to produce significant energy. If the time scale of Figure B7-1 were compressed as in most records, the peaks would appear essentially as vertical lines.

To demonstrate the above statement, the digitized record as corrected by USGS has been "clipped" or shaved in progressive amounts, and for each such clipped record a response spectrum has been made for arbitrarily selected 2% and 7% damping.

Figures B7-3, 4, 5 and 6 show the result of the clipping where the clipping factor, c , has been respectively 0.9, 0.8, 0.7 and

0.6; i.e., the records have had 10%, 20%, 30% and 40% of their absolute peak values deleted. Note that in Figure B7-5, 3 peaks have been clipped, two above and one below. The "number of points clipped" as shown in the figures does not refer to the number of peaks but to the number of digitized time intervals taken 0.002 seconds apart for plotting control. In Figure B7-6, 5 peaks have been clipped.

Figure B7-2 shows the 7% damped response spectra for these clipped time histories. The response spectrum is indicative of structural response, contrary to peak acceleration which is not. Note that there is very little difference in the spectra whether the peaks are clipped or not. At some periods there is essentially no difference in response. Figure B7-7 shows 2% damped spectra; there is little difference between the curves.

The reverse of clipping, or augmentation, was also tried with $c = 1.1, 1.2, 1.3,$ and 1.4 . Figure B7-8 shows the resulting 7% response spectra -- again, with very little difference, and Figure B7-9, the 2% damping spectra. Figures B7-10, 11, 12 and 13 show the augmented time histories for $c = 1.1, 1.2, 1.3,$ and $1.4,$ respectively.

Figure B7-14 shows overall 180° component results for three frequencies, 25 Hz, 12.5 Hz, and 6.7 Hz. The c factors are shown on the right hand side and the peak accelerations, PGA, on the left. The important spectral acceleration is the horizontal scale. At 6.7 Hz there is no difference in response re-

ardless of peak acceleration. At 12.5 Hz there is little difference and at 25 Hz there is little difference for clipping. All of which demonstrates that sharp peaks on acceleration records are not meaningful or sensitive in structural response. This is the same result as obtained for many earthquakes (Blume, 1977).

90° Component

The 90° component has about the same peak acceleration as the 180° for the October 1979 event. Figure B7-15 shows the time history without clipping or augmentation and Figures B7-16, 17, 18, 19, 20, 21, 22 and 23 show the time histories for $c = 1.4, 1.3, 1.2, 1.1, 0.9, 0.8, 0.7$ and 0.6 , respectively.

Figure B7-24 is the 90° component 7% damped spectra, clipped; and B7-25, the 7% damped spectra, augmented. Figure B7-26 and 27 are for the 90° component clipped and augmented, respectively. Figure B7-28 shows the 90° overall results at 3 frequencies. The 90° and 180° results are generally similar.

General

Clipping can be done with little or no change in spectral values at most frequencies. This indicates that peak ground accelerations are not good indicators of response.

APPENDIX B7-A

Table B7-1 provides numerical data for the 180° component and B7-2 for the 90° component of the October 16, 1979 earthquake.

Table B7-3 provides the same type of data for the August 27, 1978 record, 180° component.

Figure B7-29 shows the plotted overall results for three frequencies for the 180° component of the August 27, 1978 event recorded at the same (abutment) station.

Table B7-1

South Carolina Eq. 10/16/1979, Monticello Crest 180°

Factor (C)	PGA (G)	2% Sa At			7% Sa At		
		6.7 Hz	12.5 Hz	25 Hz	6.7 Hz	12.5 Hz	25 Hz
1.4	0.4948	0.1573	1.3461	2.1953	1.4155	0.9907	1.3783
1.3	0.4594	0.1464	1.2415	1.9718	1.3223	0.8849	1.2168
1.2	0.4241	0.1338	1.1555	1.7805	1.2140	0.7981	1.0760
1.1	0.3887	0.1157	1.1243	1.7266	1.0505	0.7732	1.0492
1.0	0.3534	0.1067	1.1103	1.7078	1.0155	0.7612	1.0391
0.9	0.3181	0.1067	1.1008	1.6951	1.0155	0.7530	1.0323
0.8	0.2827	0.1043	1.0825	1.6643	0.9930	0.7378	1.0197
0.7	0.2474	0.0996	1.0461	1.5959	0.9182	0.7062	0.9858
0.6	0.2120	0.0977	0.9797	1.4561	0.8232	0.6415	0.8948

Table B7-2

South Carolina Eq. 10/16/1979, Monticello Crest 90°

Factor (C)	PGA (G)	2% Sa At			7% Sa At		
		6.7 Hz	12.5 Hz	25 Hz	6.7 Hz	12.5 Hz	25 Hz
1.4	0.4989	0.1913	1.1069	2.2984	0.1683	0.7266	1.6181
1.3	0.4633	0.1584	1.0571	2.1418	0.1397	0.6742	1.4981
1.2	0.4276	0.1465	1.0281	1.9865	0.1299	0.6385	1.3782
1.1	0.3920	0.1286	1.0011	1.9010	0.1146	0.6127	1.3191
1.0	0.3564	0.1218	0.9880	1.8509	0.1084	0.5999	1.2857
0.9	0.3207	0.1170	0.9789	1.8182	0.1041	0.5911	1.2640
0.8	0.2851	0.1078	0.9613	1.7568	0.0958	0.5742	1.2230
0.7	0.2495	0.0999	0.9376	1.6663	0.0891	0.5503	1.1577
0.6	0.2136	0.1135	0.9060	1.5422	0.0961	0.5175	1.0654

Table B7-3

August 27, 1978, 180° Component

Factor (C)	PGA (G)	2% Sa At			7% Sa At		
		6.7 Hz	12.5 Hz	25 Hz	6.7 Hz	12.5 Hz	25 Hz
1.4	0.372	0.109	0.532	1.12	0.100	0.392	0.901
1.3	0.346	0.114	0.506	1.01	0.093	0.367	0.807
1.2	0.319	0.108	0.479	0.982	0.075	0.334	0.767
1.1	0.293	0.104	0.459	0.961	0.072	0.311	0.734
1.0	0.266	0.102	0.450	0.951	0.073	0.300	0.724
0.9	0.240	0.101	0.443	0.942	0.073	0.290	0.711
0.8	0.213	0.099	0.428	0.926	0.073	0.273	0.686
0.7	0.186	0.101	0.409	0.906	0.074	0.261	0.656
0.6	0.160	0.104	0.387	0.879	0.076	0.248	0.614

References

Blume, John A., "The Effect of Arbitrary Variation in Peak Ground Acceleration on Spectral Response", Section LL-30 of Final Safety Analysis Report, Units 1 and 2, Diablo Canyon Site, Amendment No. 50, "Seismic Evaluation for Postulated 7.5M Hosgri Earthquake," Pacific Gas & Electric Company, San Francisco, Calif., 1977.

ALAB Decision 644, June 16, 1981, p. 70

SOUTH CAROLINA EO 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 130 DEG

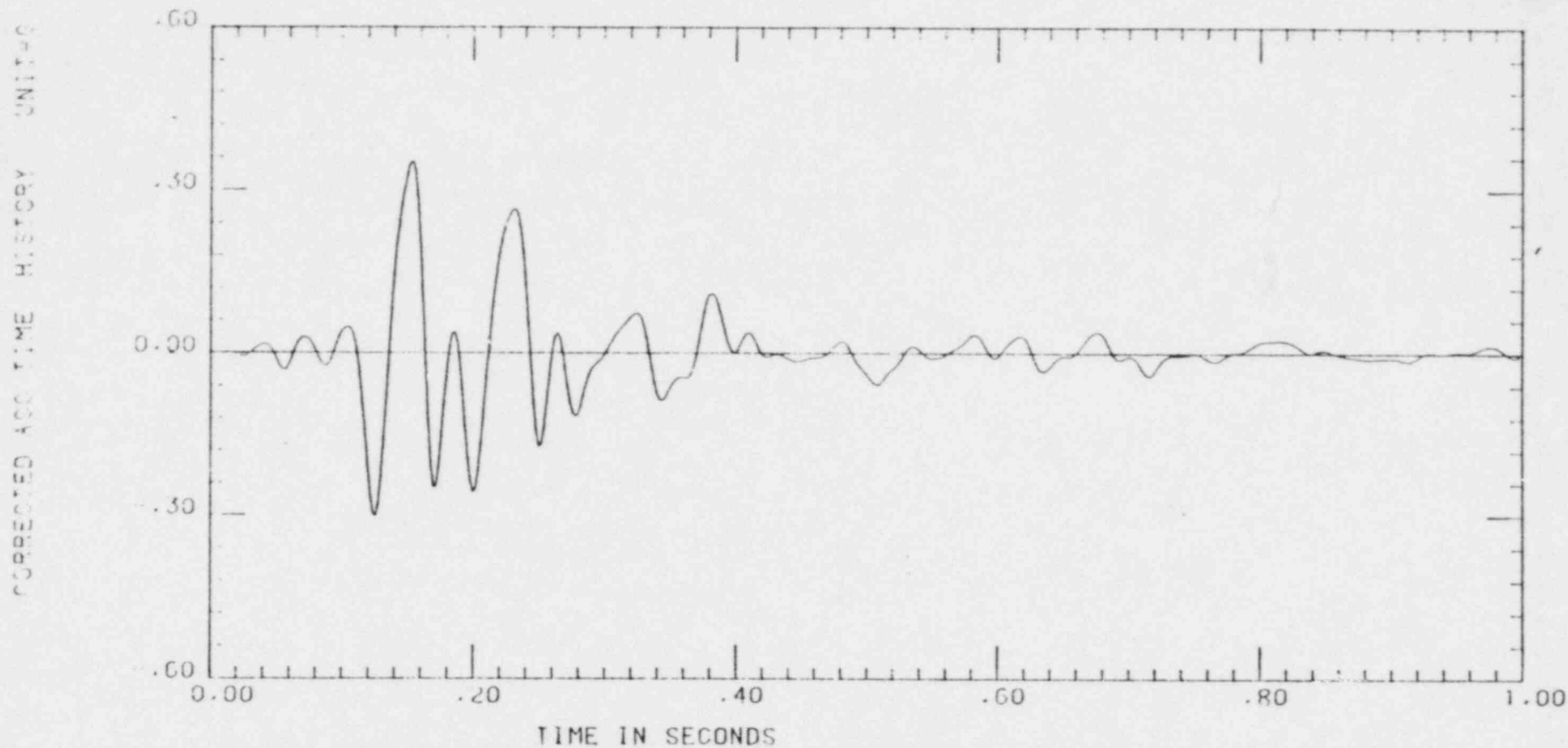
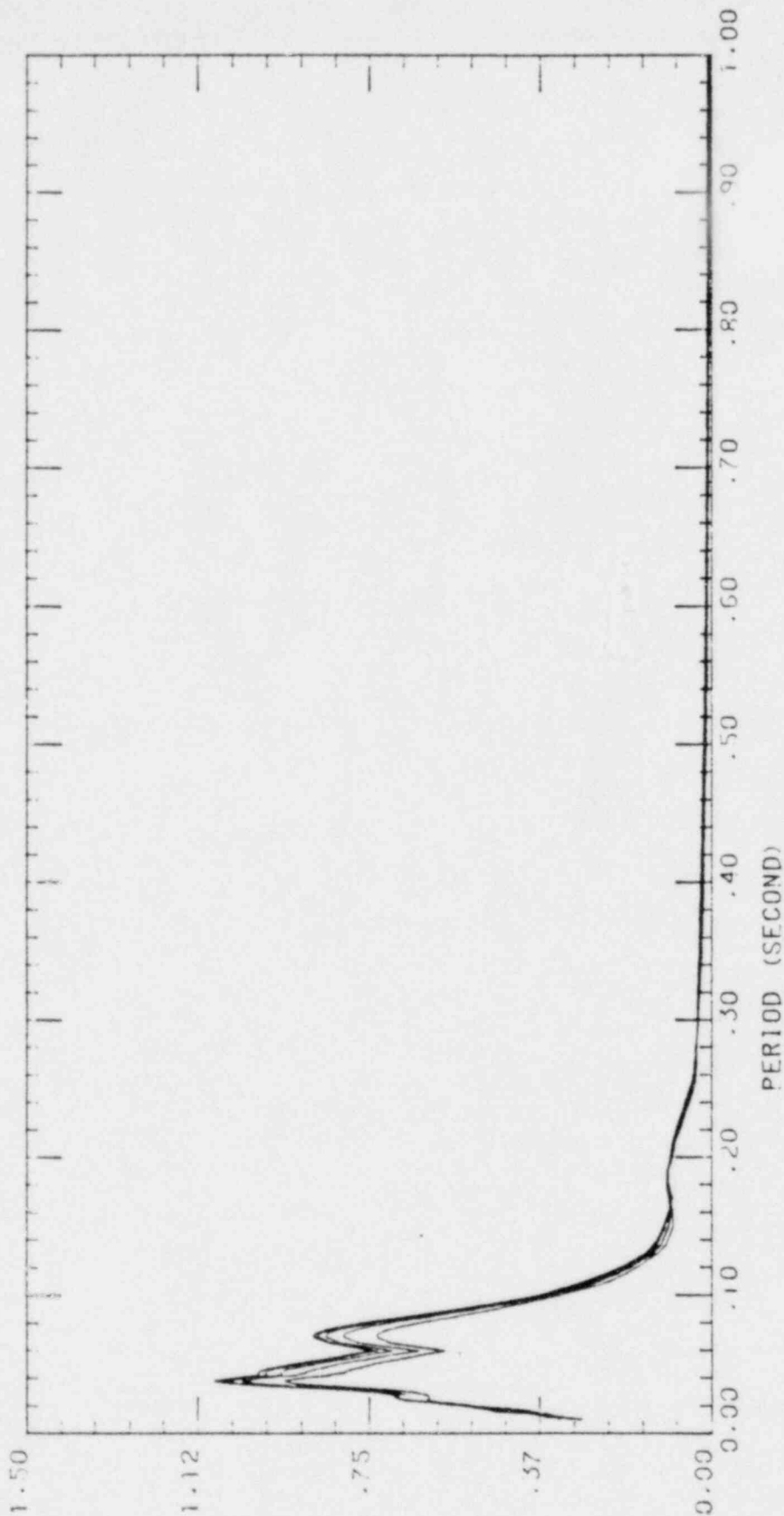


FIG. B7-1 -- TIME HISTORY OF CORRECTED 180° RECORD WITHOUT CLIPPING

SOUTH CAROLINA EO 160CT79 CORR ACC T-H
MONTICELLO CEN CREST 180 DEG



PERIOD (SECOND)

SA FOR C=1.0, 0.9, 0.8, 0.7, 0.6
7 PCT DAMPING

ACC. RESPONSE SPECTRA (UNIT=G)

FIG. B7-2 -- RESPONSE SPECTRA, 7% DAMPING, VARIOUS CLIPPING FACTORS

SOUTH CAROLINA EQ 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 180 DEG

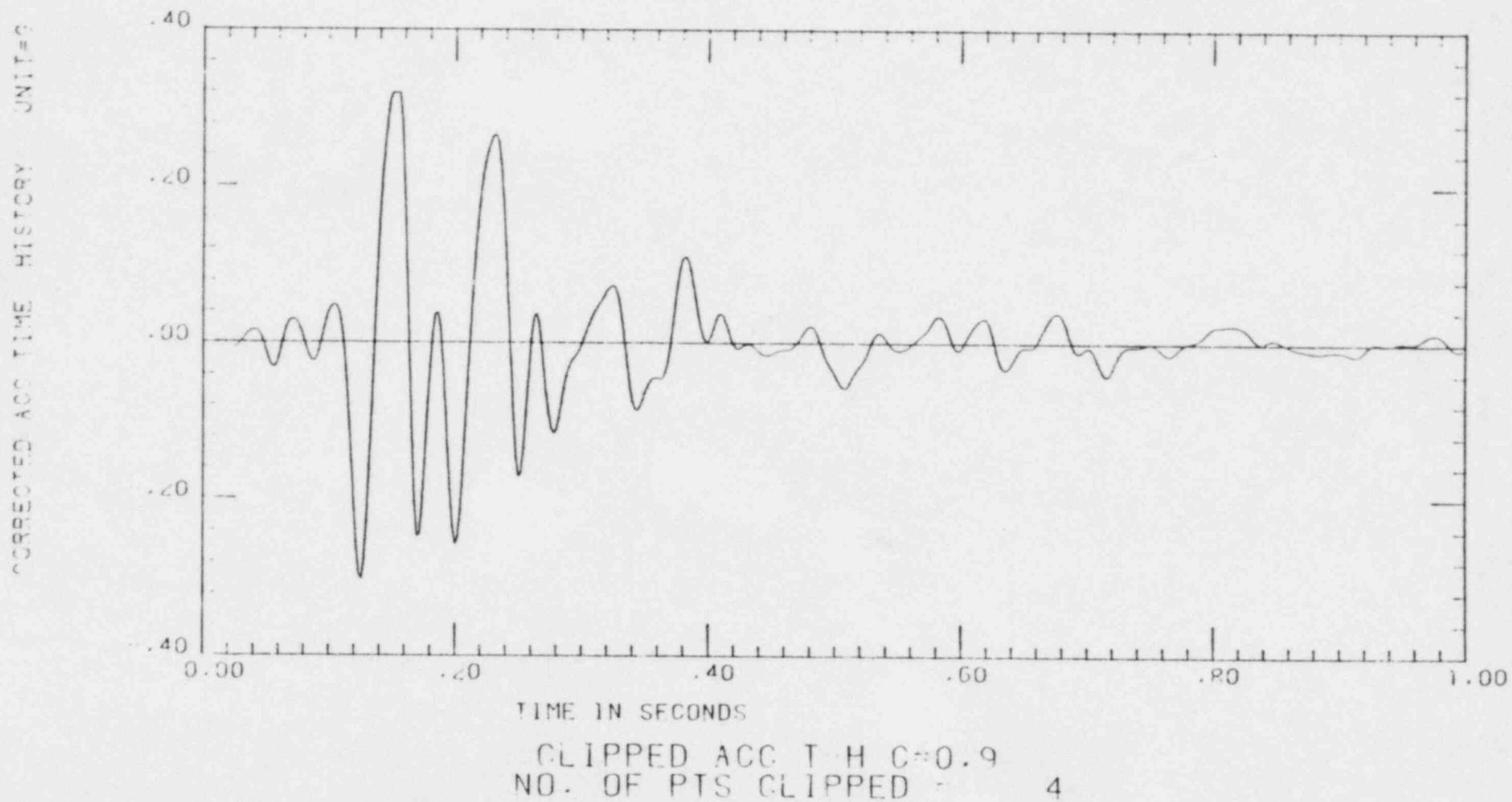


FIG. B7-3 -- 180° TIME HISTORY WITH C = 0.90

SOUTH CAROLINA EQ 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 180 DEG

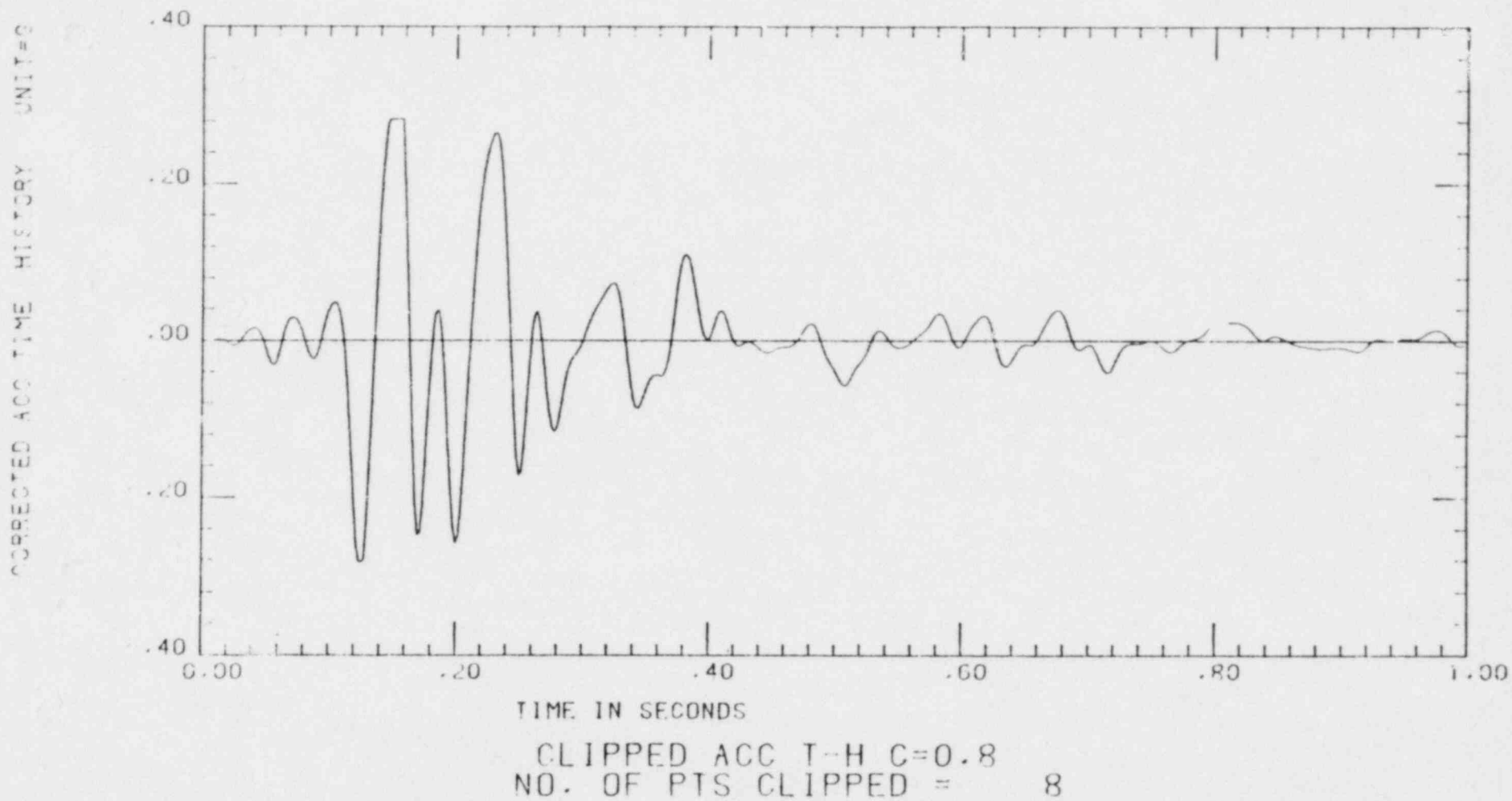


FIG.B7-4 -- 180° TIME HISTORY WITH C = 0.80

SOUTH CAROLINA EQ 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 180 DEG

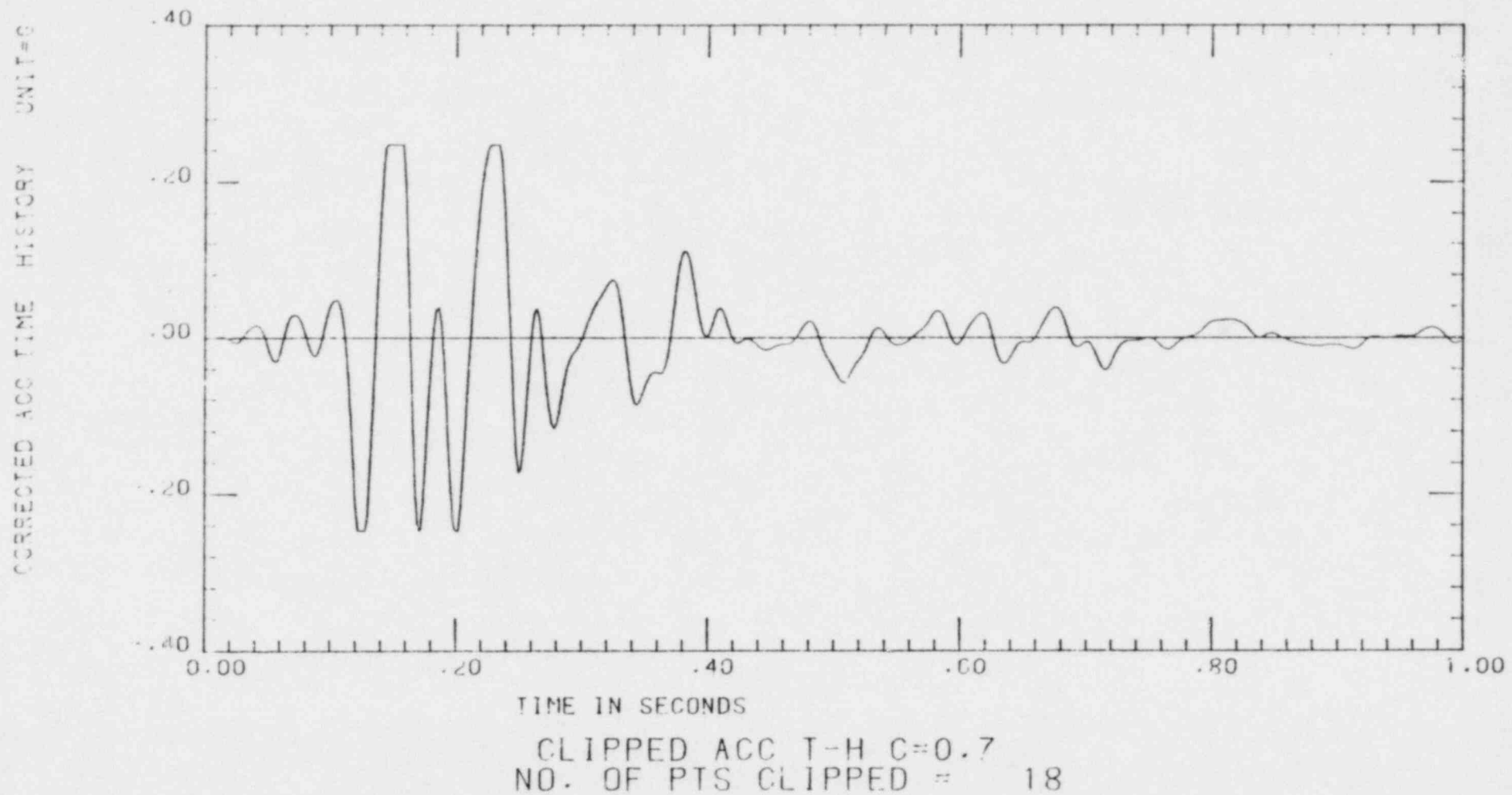


FIG. B7-5 -- 180° TIME HISTORY WITH C = 0.70

SOUTH CAROLINA EO 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 180 DEG

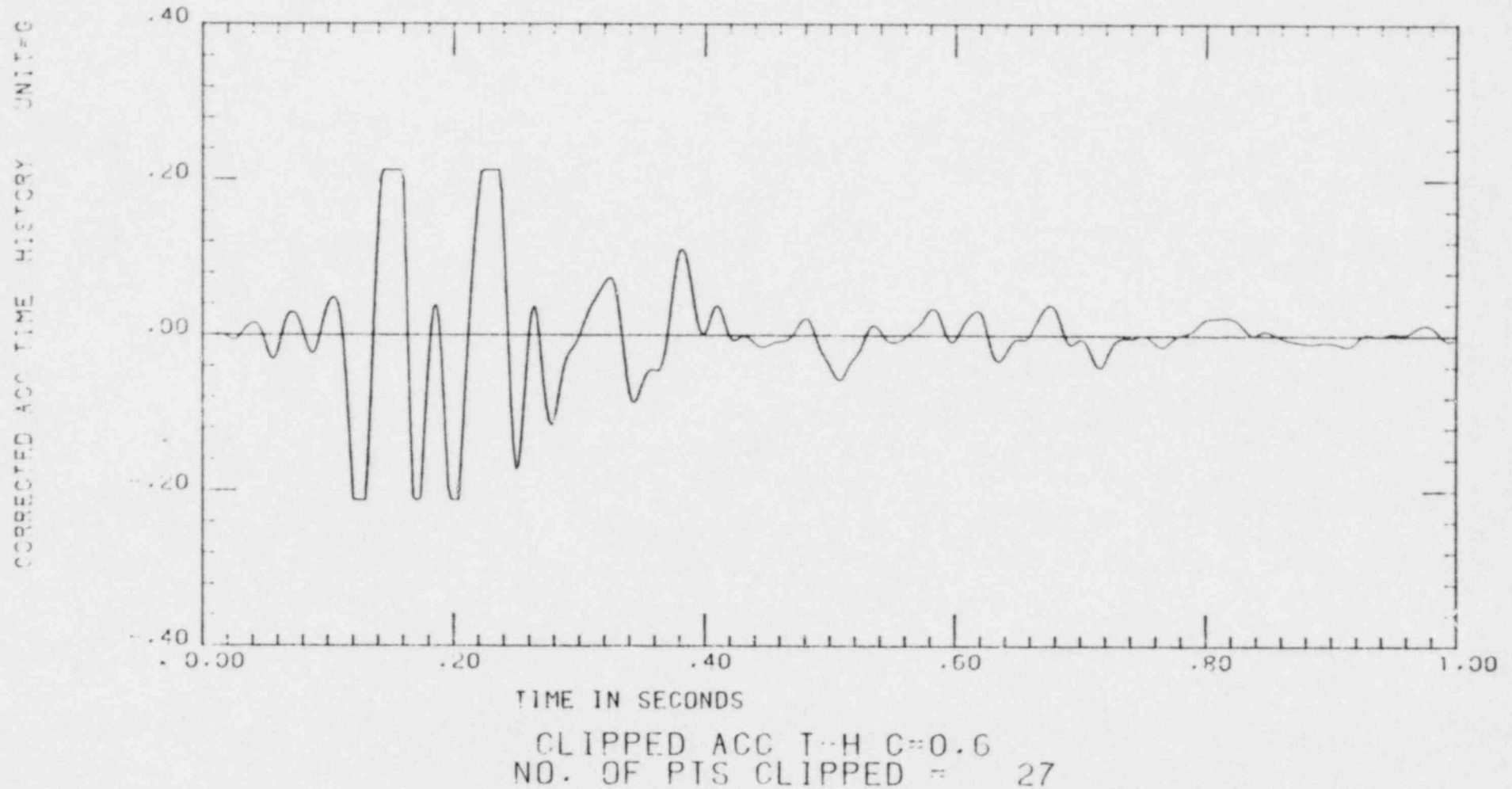
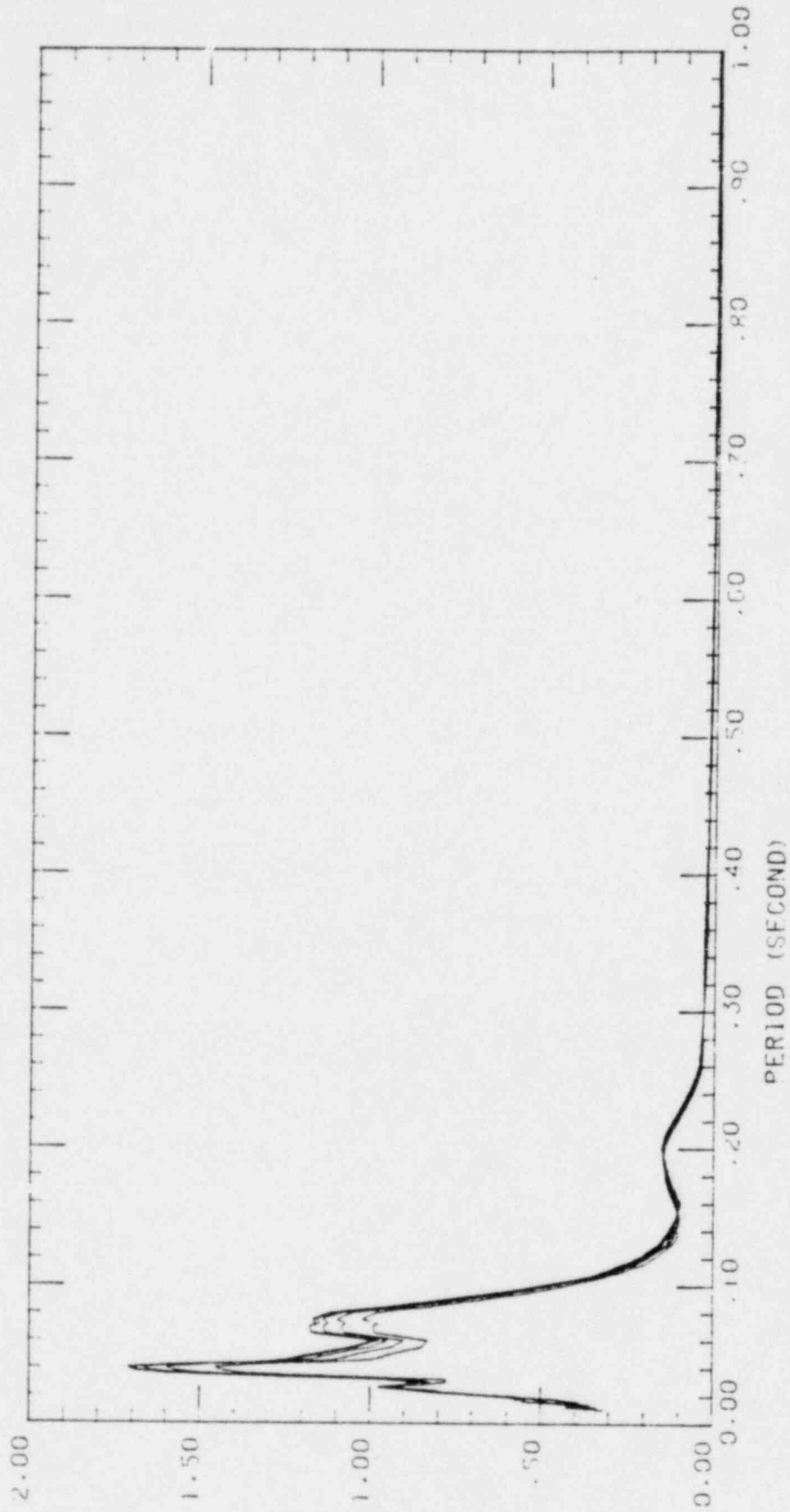


FIG. B7-6 -- 180° TIME HISTORY WITH C = 0.60

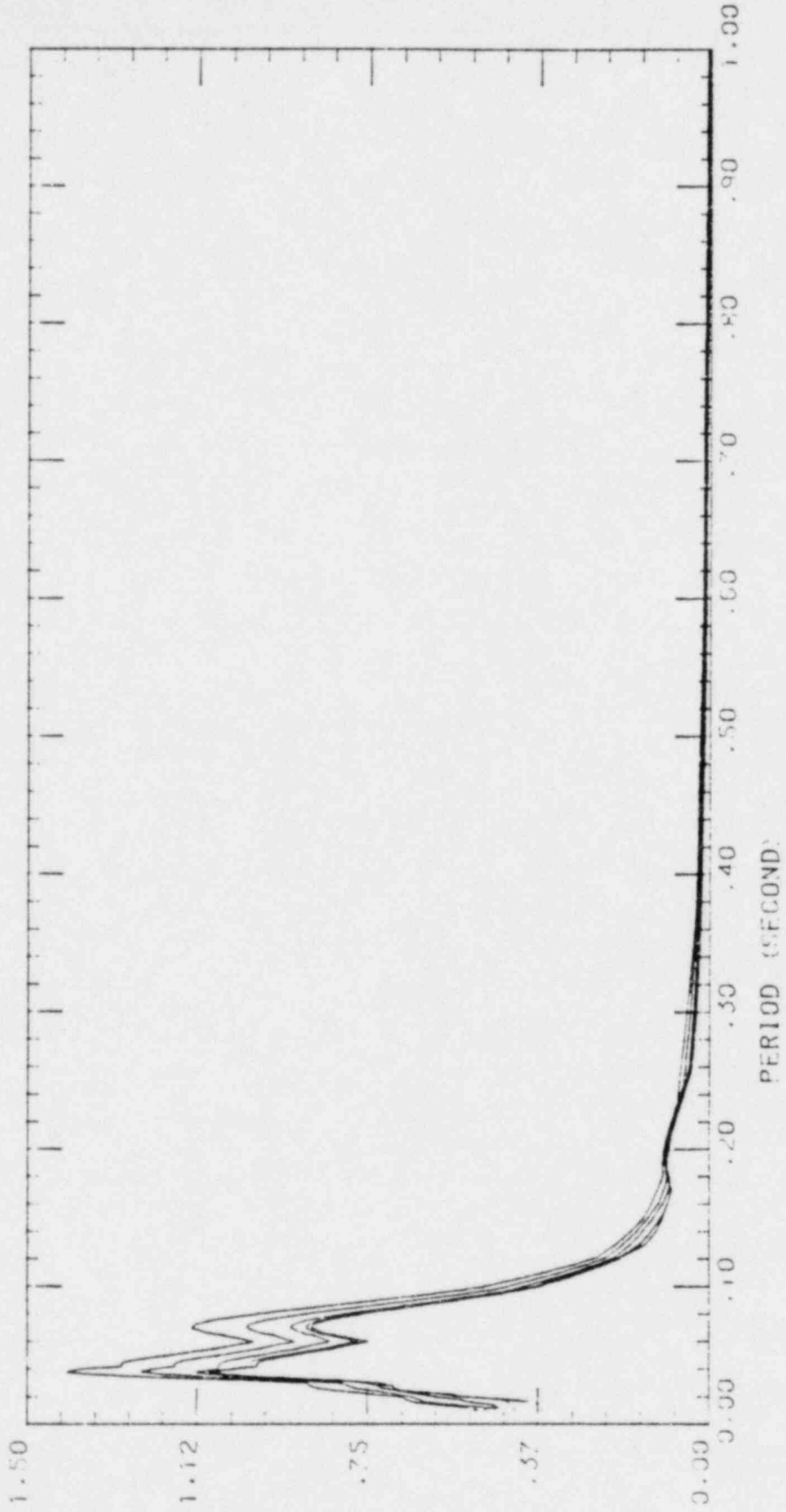
SOUTH CAROLINA EO 160CT79 CORR ACC T--H
MONTICELLO CFN CREST 180 DEG



SA FOR C=1.0; 0.9, 0.8, 0.7, 0.6
2 PCT DAMPING

FIG. B7-7 -- 180° RESPONSE SPECTRA, 2% DAMPING, VARIOUS CLIPPING FACTORS

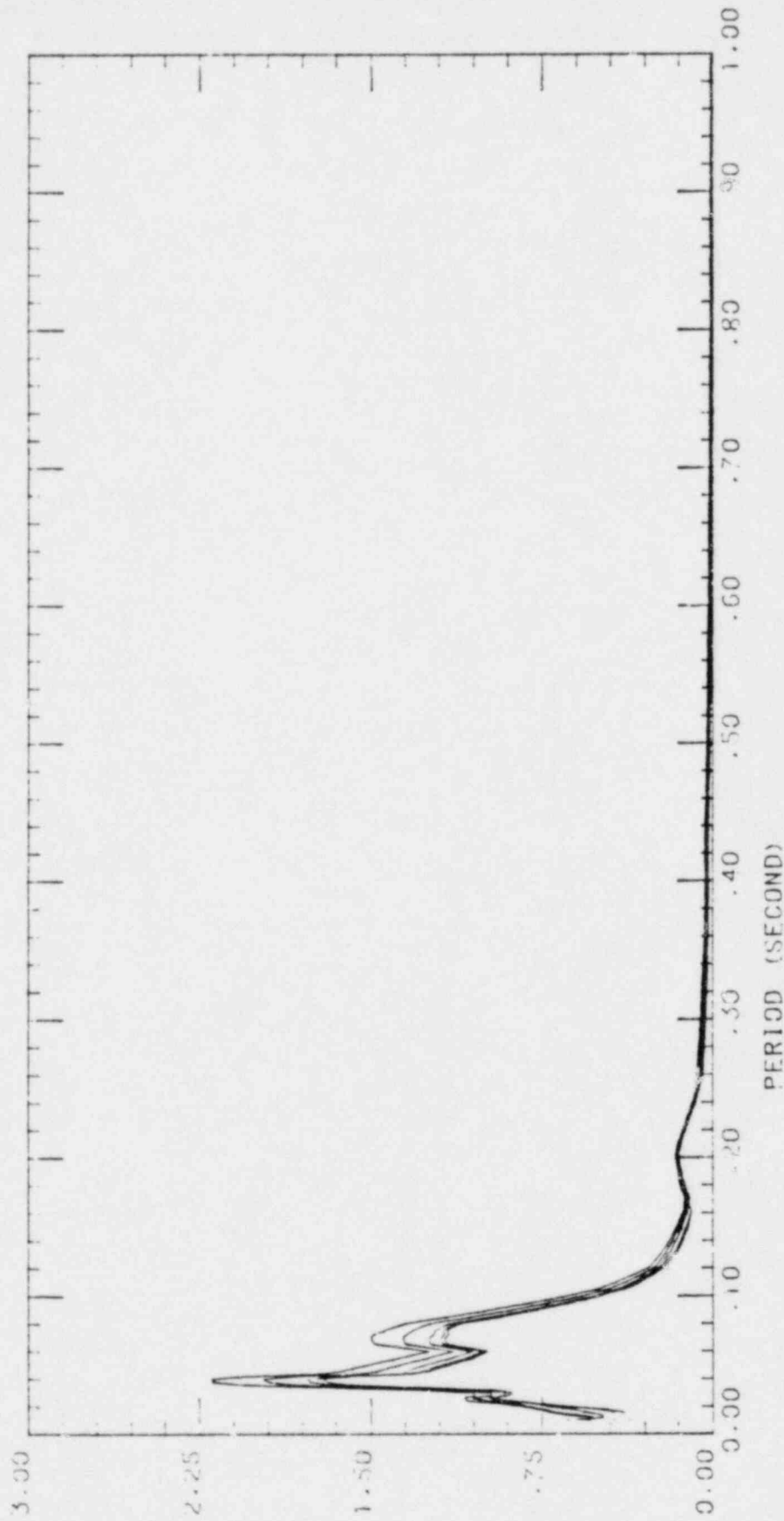
SOUTH CAROLINA EO 160CT79 CORR ACC T-H
MONTICELLO GEN CREST 180 DEG



SA FOR C=1.0, 1.1, 1.2, 1.3, 1.4
7 PCT DAMPING

FIG. B7-8 -- 180° RESPONSE SPECTRA, 7% DAMPING, VARIOUS AUGMENTATION FACTORS

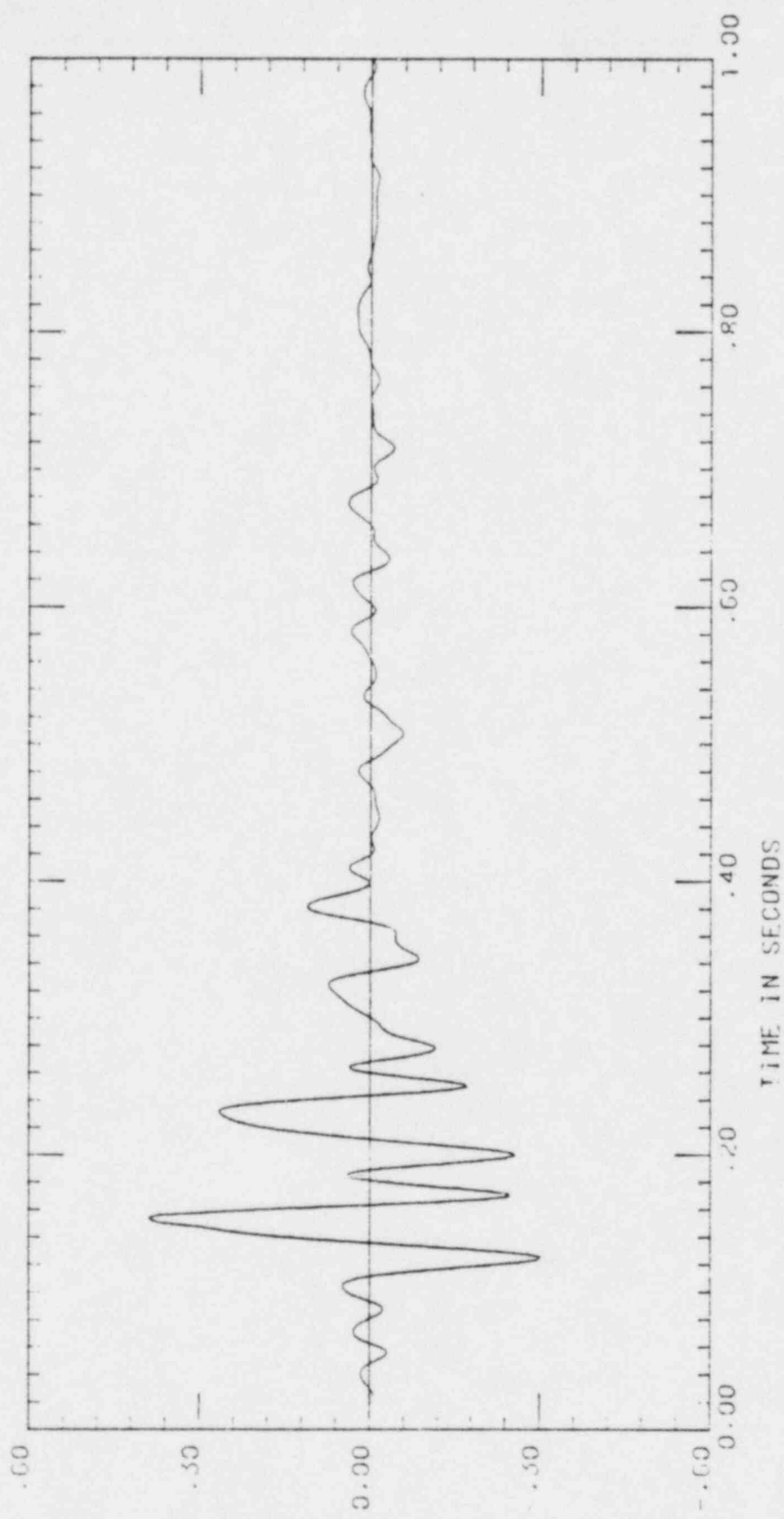
SOUTH CAROLINA ED 1600179 CORR ACC T-H
MONTICELLO CEN CREST 180 DEG



SA FOR C=1.0; 1.1, 1.2, 1.3, 1.4
2 PCT DAMPING

FIG. B7-9 -- 180° RESPONSE SPECTRA, 2% DAMPING, VARIOUS AUGMENTATION FACTORS

SOUTH CAROLINA EQ 16OCT79 CORR ACC TH
MONTICELLO CEN CREST 180 DEG



AUGMENTED ACC TH IAUG=1 C=1.1
NO. OF PTS AUGMENTED= 4

FIG. B7-10 -- 180° TIME HISTORY WITH C = 1.1

SOUTH CAROLINA EQ 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 180 DEG

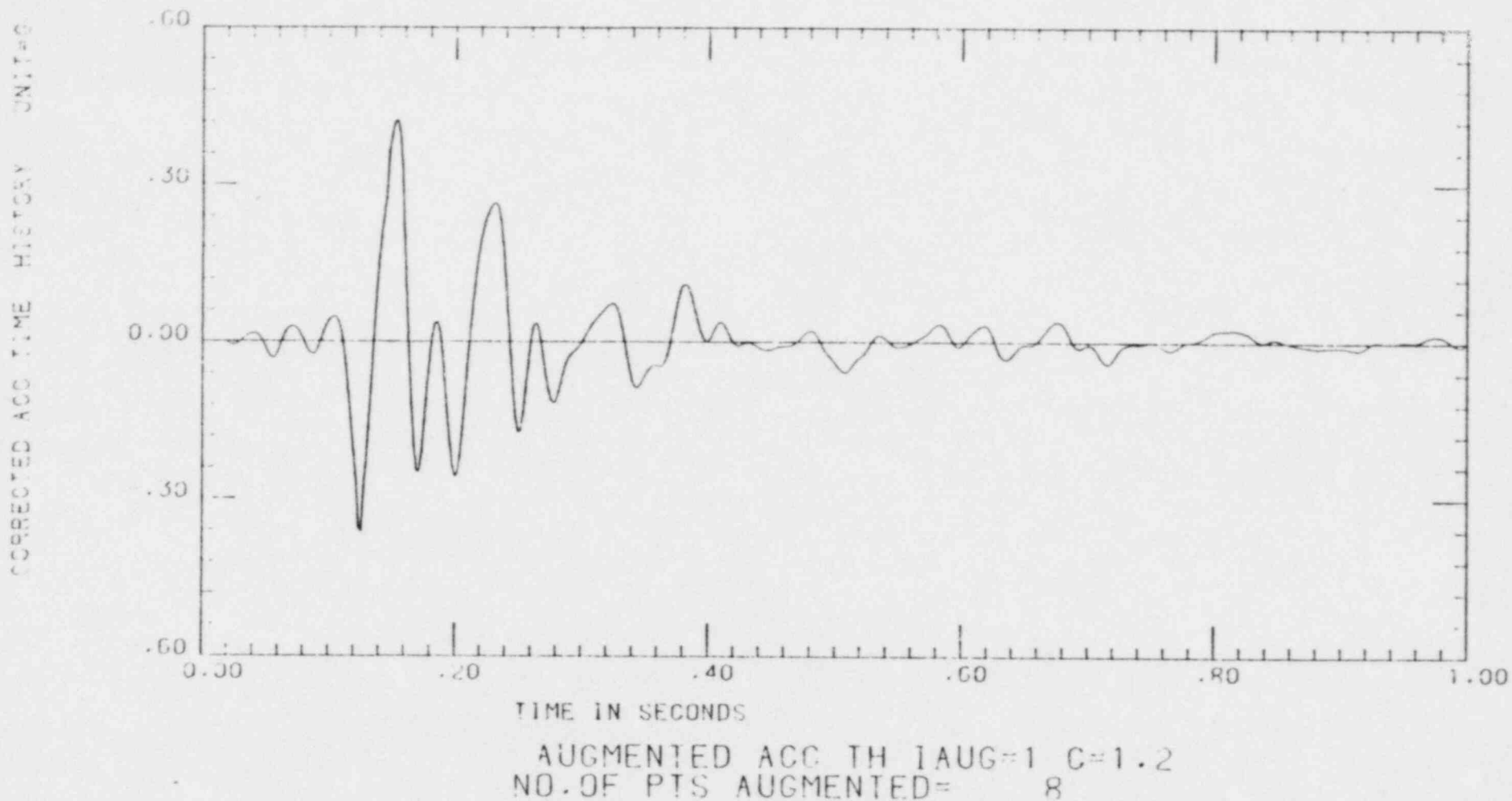
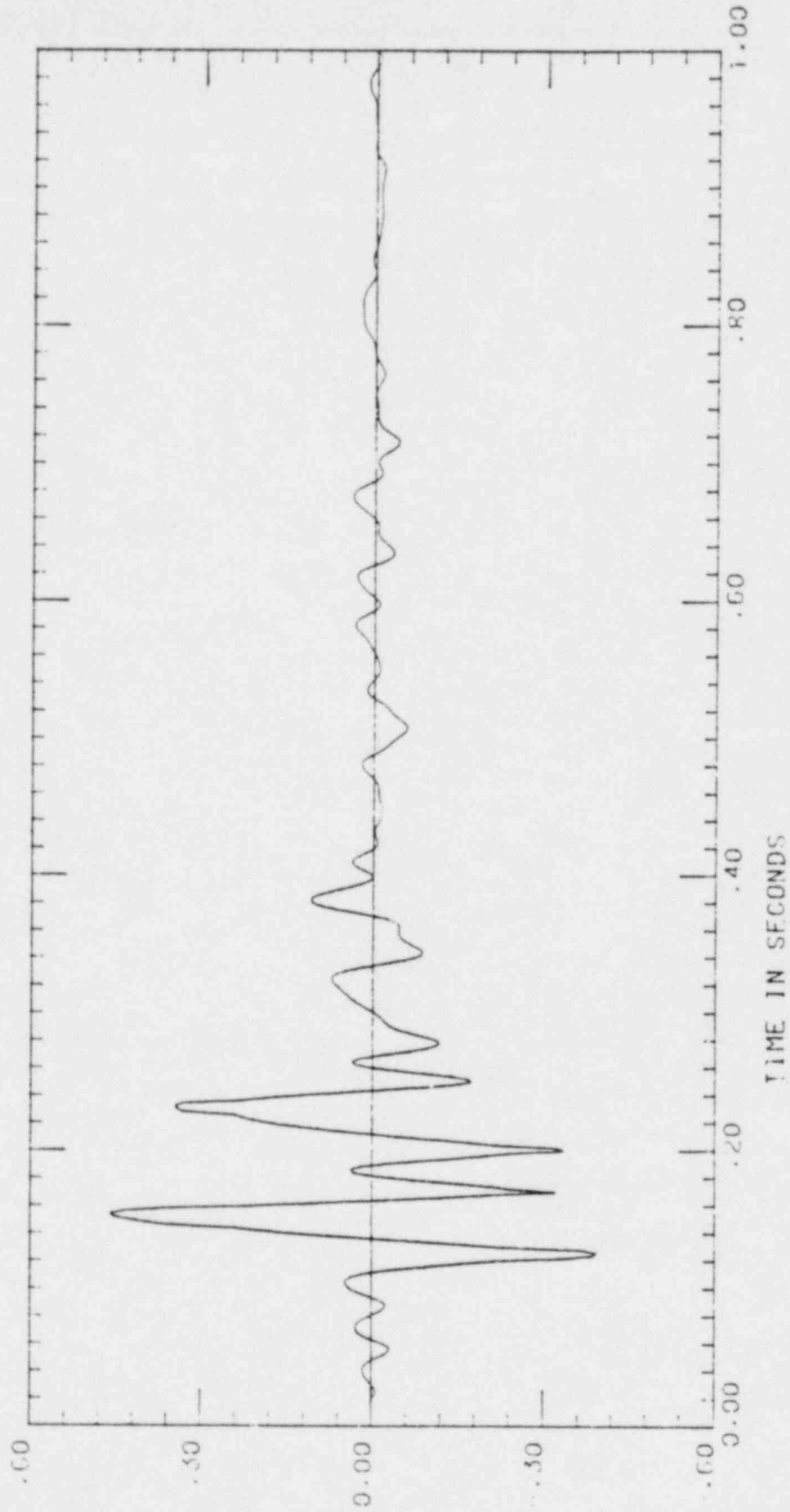


FIG. B7-11 -- 180° TIME HISTORY WITH C = 1.2

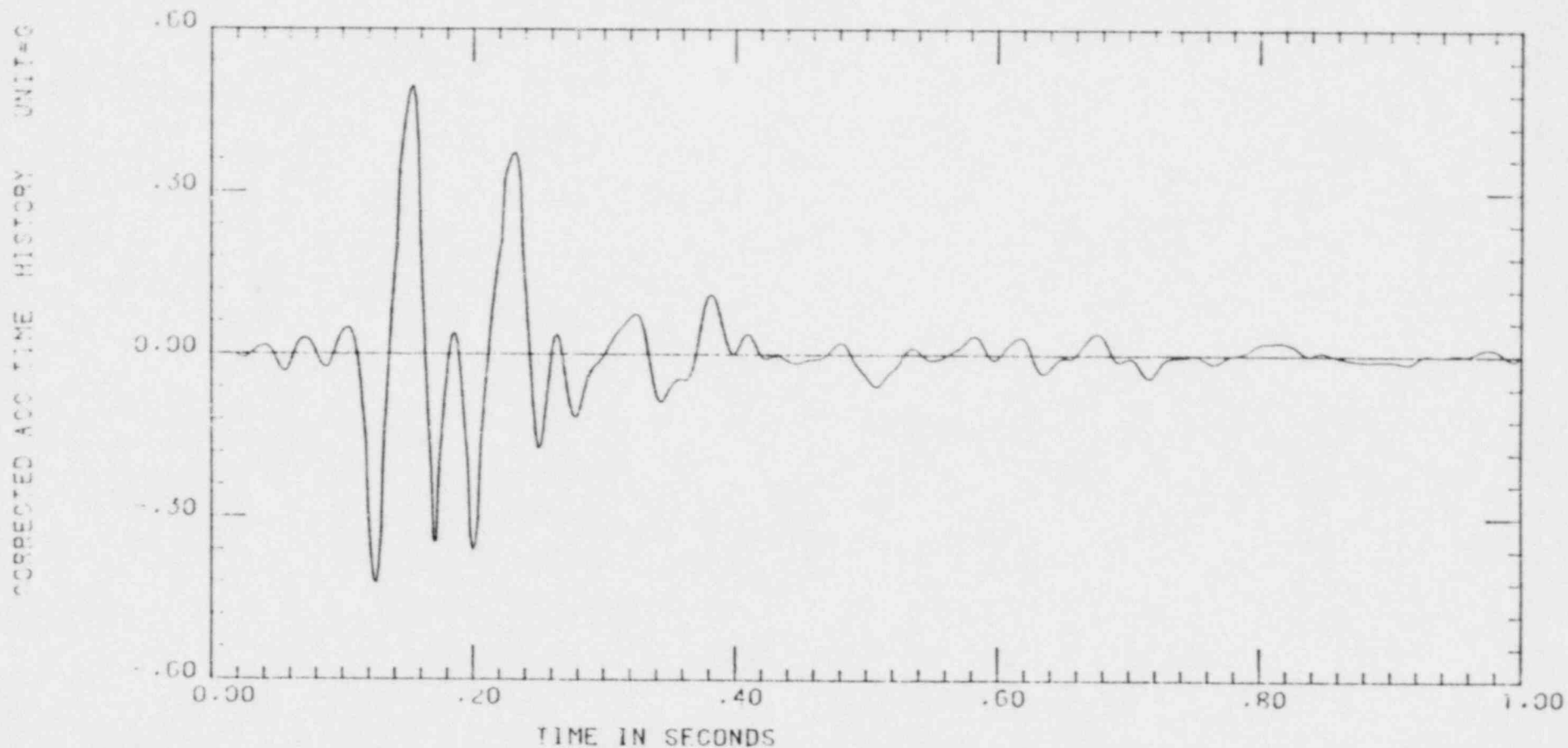
SOUTH CAROLINA EO 160CT79 CORR ACC T-H
MONTICELLO GEN CREST 180 DEG



AUGMENTED ACC TH I AUG = 1 C = 1.3
NO. OF PTS AUGMENTED = 18

FIG. B7-12 -- 180° TIME HISTORY WITH C = 1.3

SOUTH CAROLINA EQ 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 180 DEG



AUGMENTED ACC TH 1AUG=1 C=1.4
NO. OF PTS AUGMENTED= 27

FIG. B7-13 -- 180° TIME HISTORY WITH C = 1.4

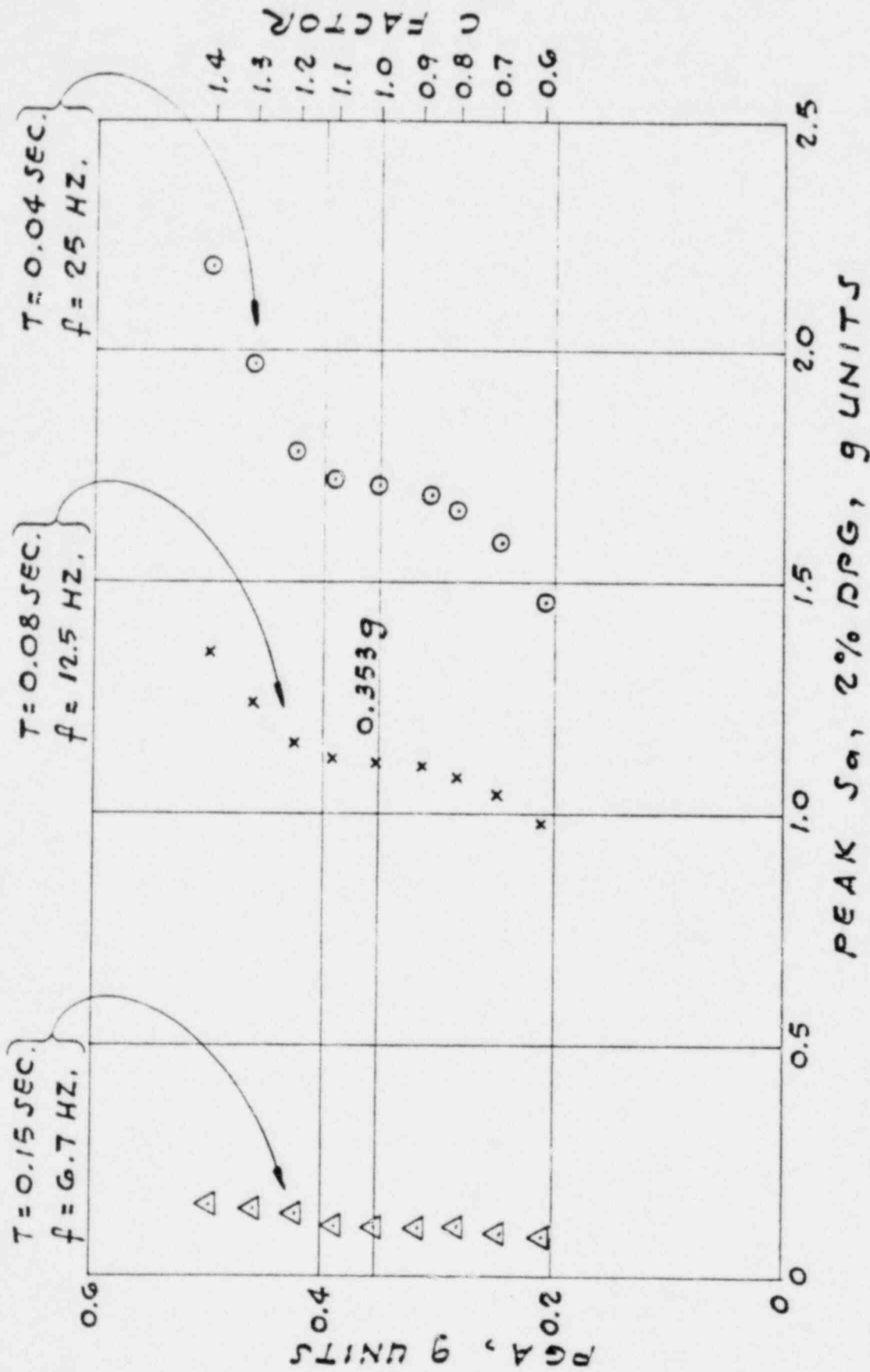
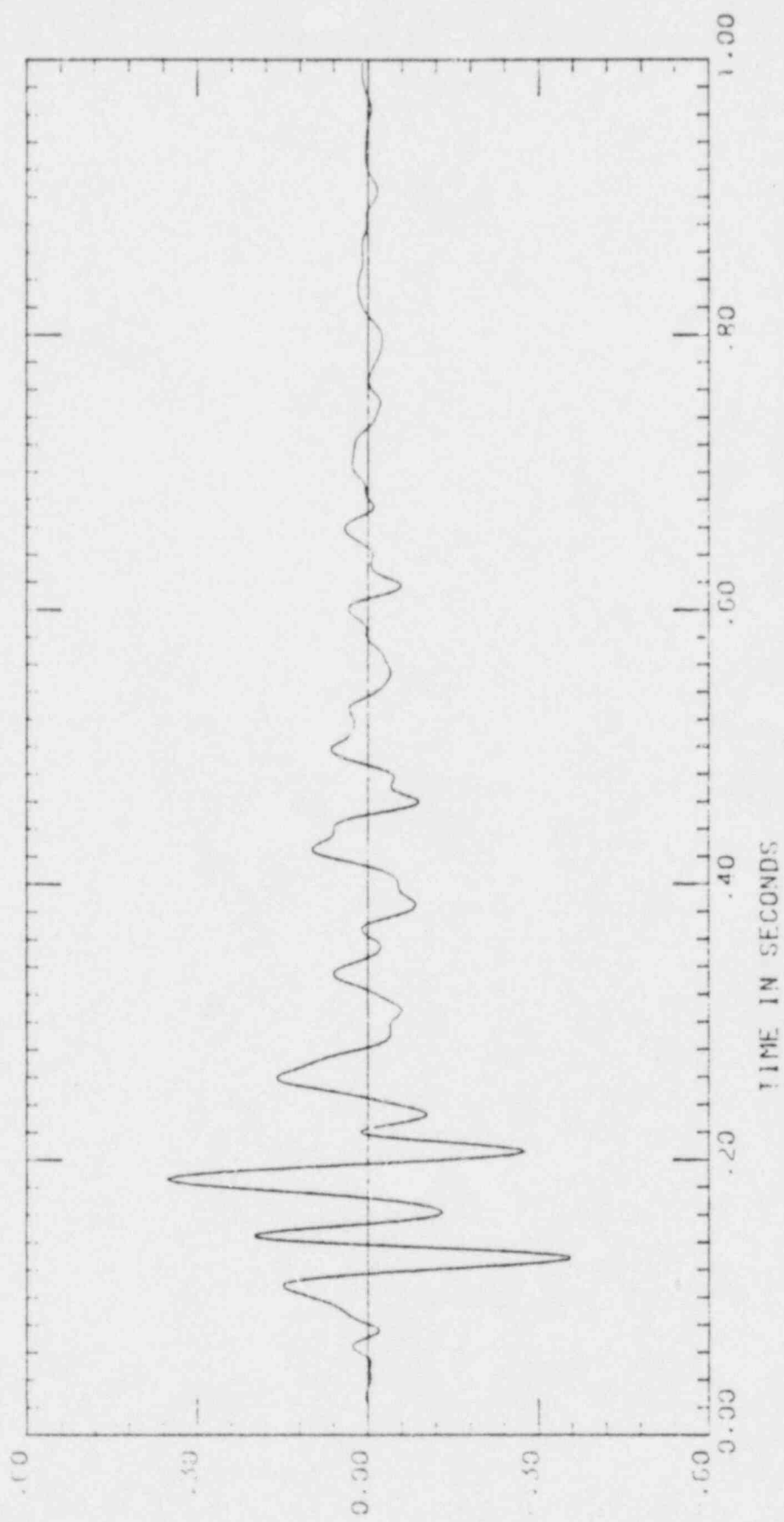


FIG. B7-14: SOUTH CAROLINA EQ. 10/16/79, MONTICELLO CREST, 180° COMPONENT

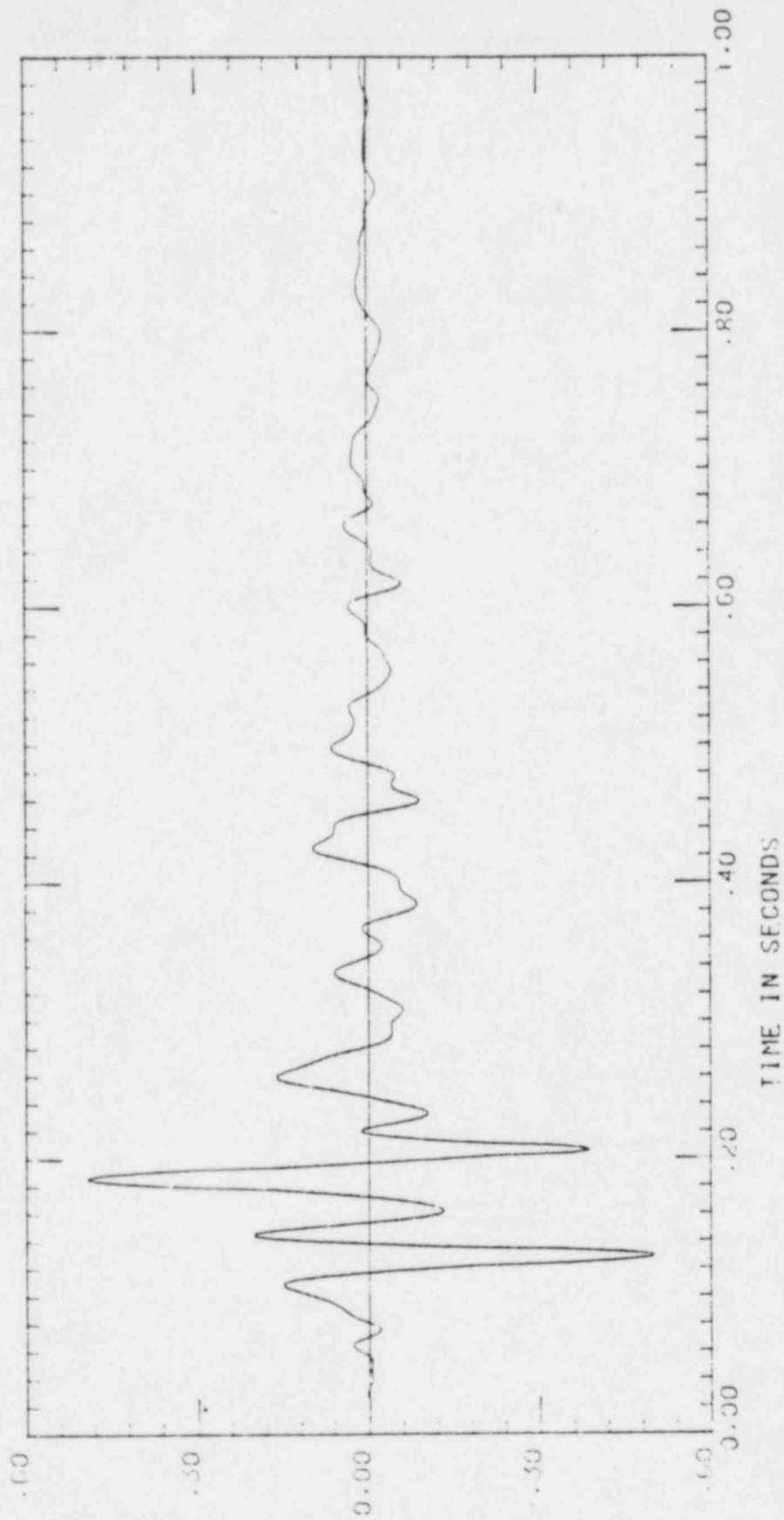
SOUTH CAROLINA EQ 16OCT79 CORR ACC T-H
MONTICELLO GEN CREST 90 DEG



NON-AUGMENTED ACC T-H C=1.0

FIG. B7-15 -- 90° TIME HISTORY (WITH C = 1.0)

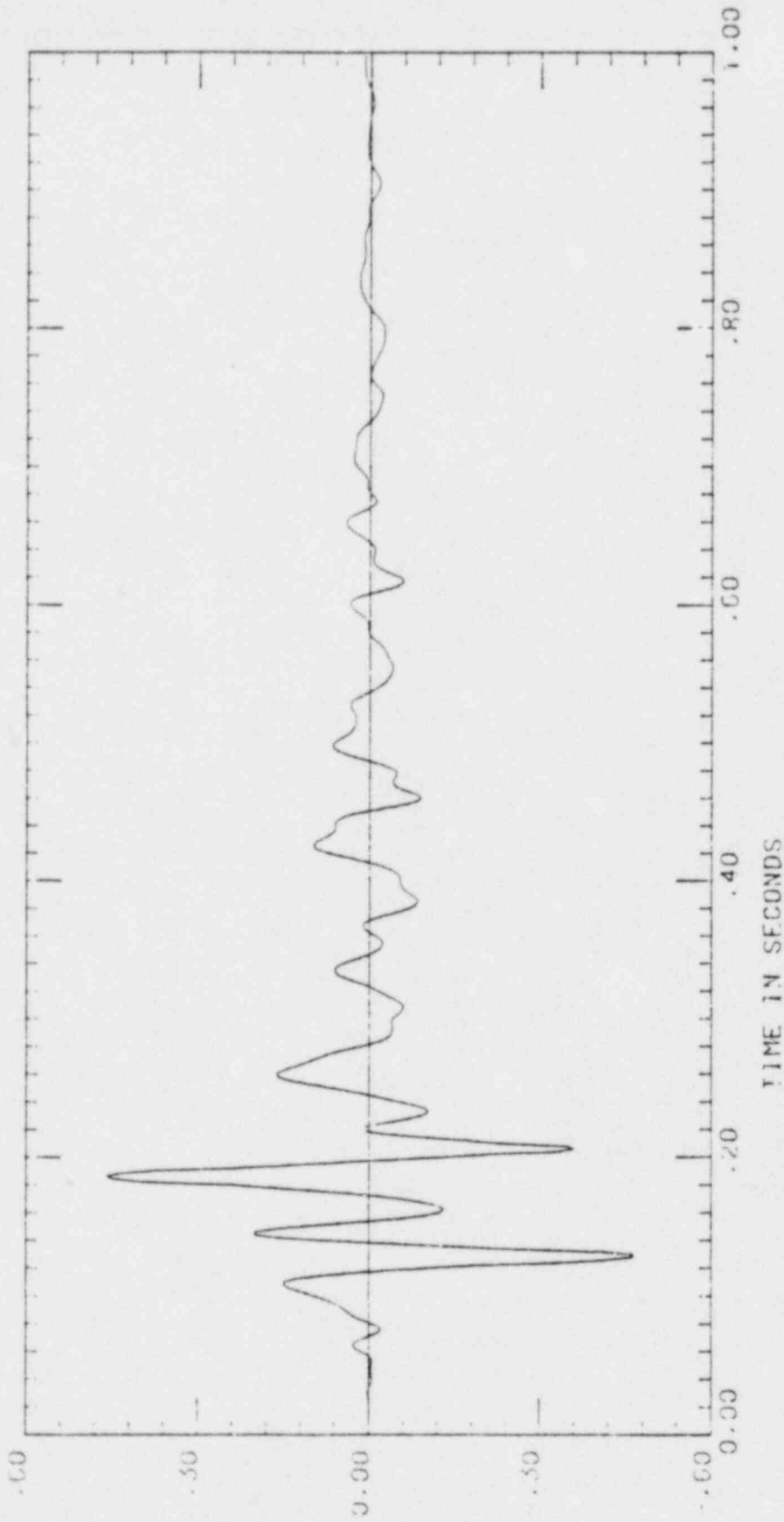
SOUTH CAROLINA EO 160CT79 CORR ACC TH
MONTICELLO CEN CREST 90 DEG



AUGMENTED ACC TH I AUG=1 C=1.4
NO. OF PTS AUGMENTED= 17

FIG. B7-16 -- 90° TIME HISTORY WITH C = 1.4

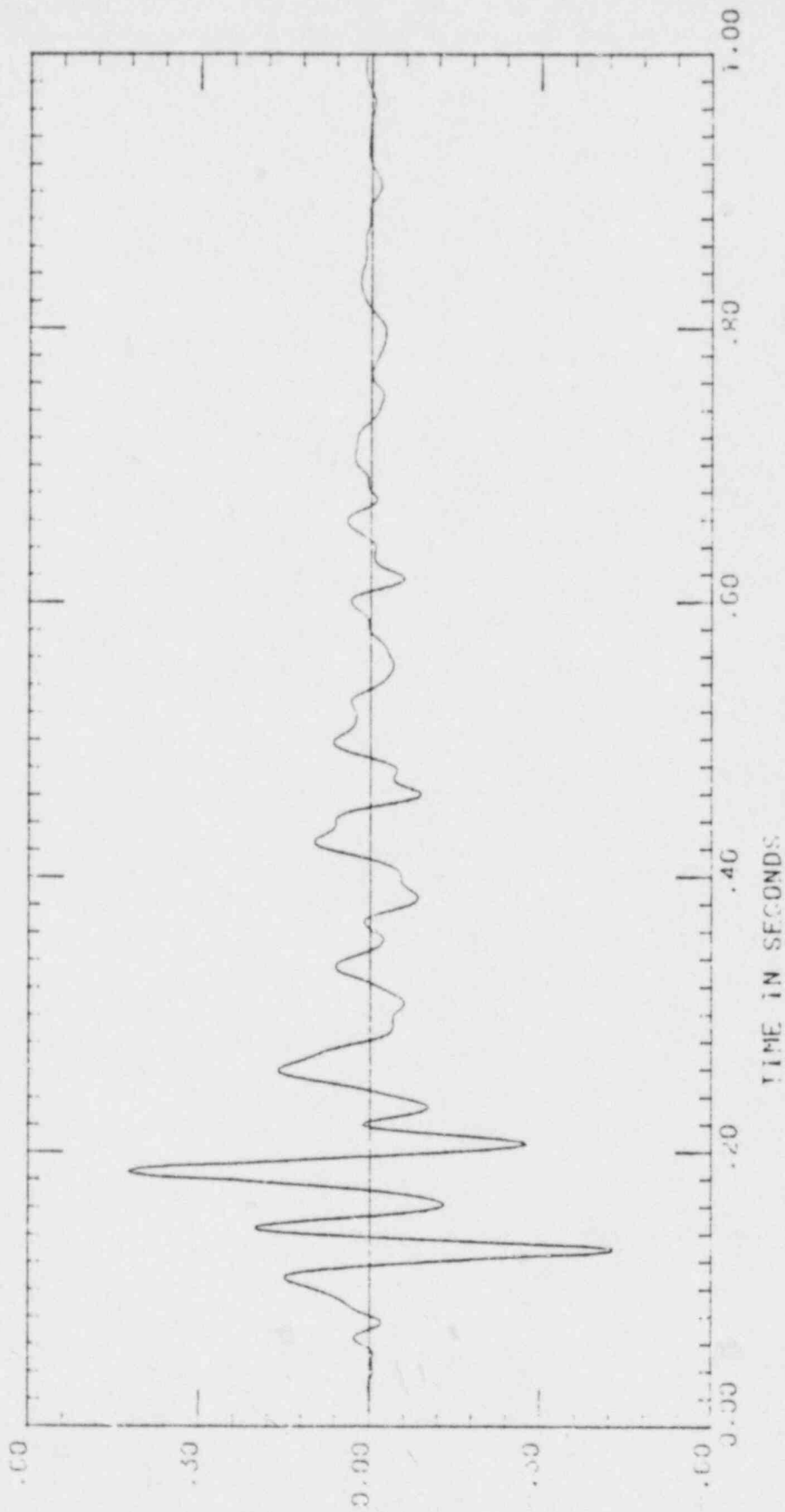
SOUTH CAROLINA EQ 16OCT79 CORR ACC T-H
MONTICELLO GEN CREST 90 DEG



AUGMENTED ACC TH IAUG=1 C=1.3
NO. OF PTS AUGMENTED= 13

FIG. B7-17 -- 90° TIME HISTORY WITH C = 1.3

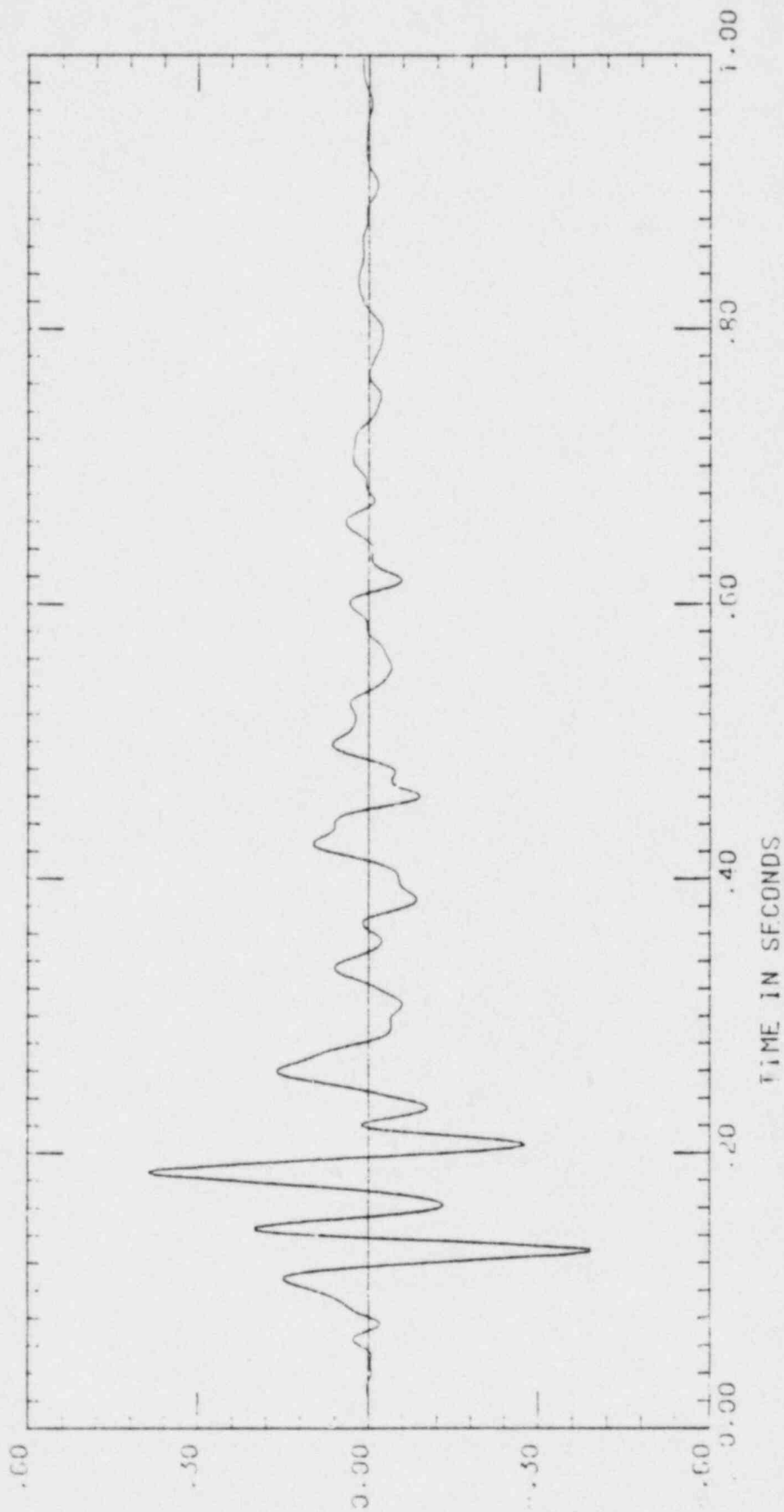
SOUTH CAROLINA EQ 1600179 CORR ACC T-H
MONTICELLO CEN CREST 90 DEG



AUGMENTED ACC TH 1AUG=1 C=1.2
NO. OF PTS AUGMENTED= 9

FIG. B7-18 -- 90° TIME HISTORY WITH C = 1.2

SOUTH CAROLINA EQ 16OCT79 CORR ACC T--H
MONTICELLO CEN CREST 90 DEG



AUGMENTED ACC TH I AUG = 1 C = 1.1
NO. OF PTS AUGMENTED = 6

FIG. B7-19 -- 90° TIME HISTORY WITH C = 1.1

SOUTH CAROLINA EQ 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 90 DEG

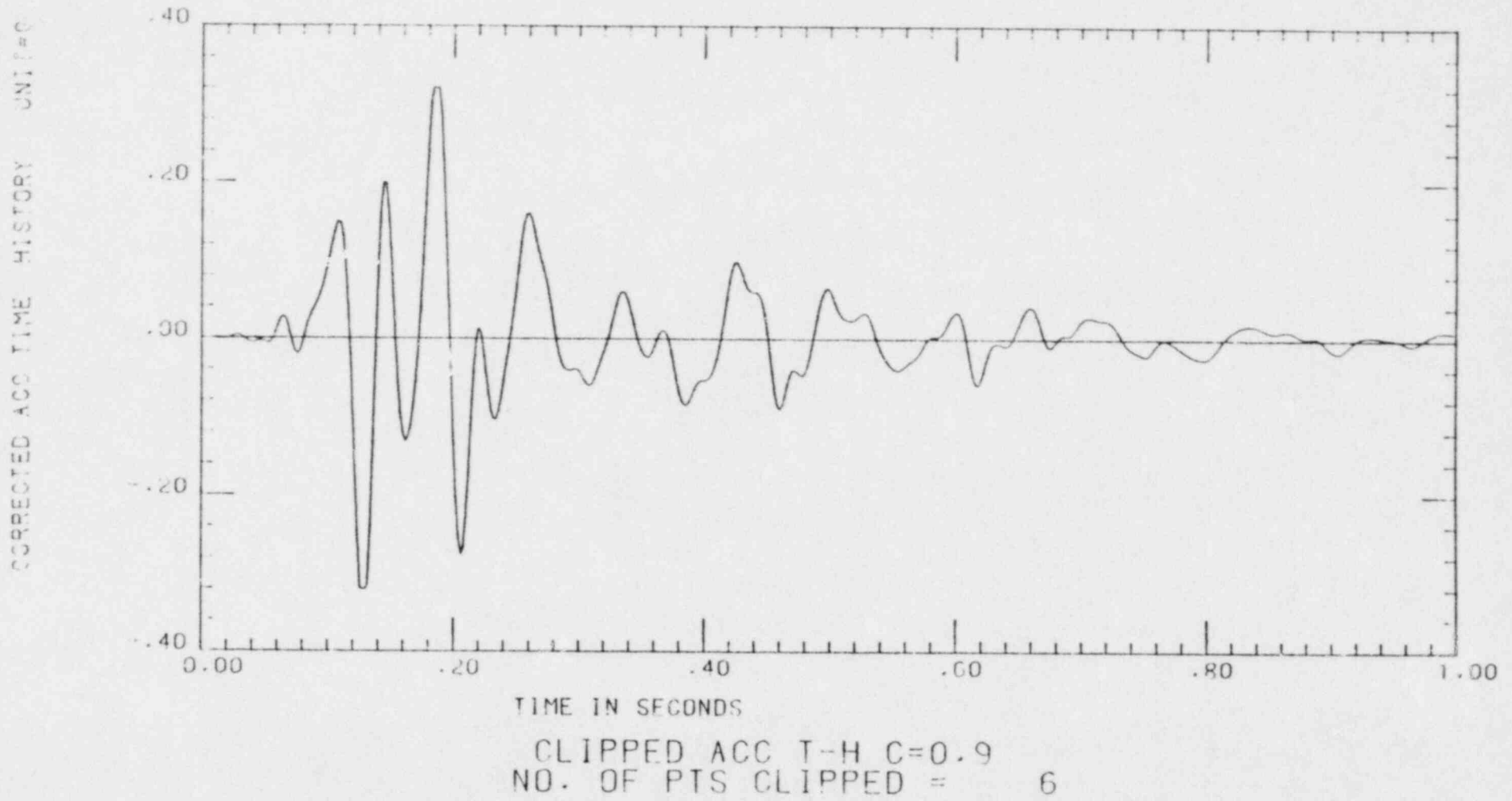


FIG. B7-20 -- 90° TIME HISTORY FOR C = 0.90

SOUTH CAROLINA EO 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 90 DEG

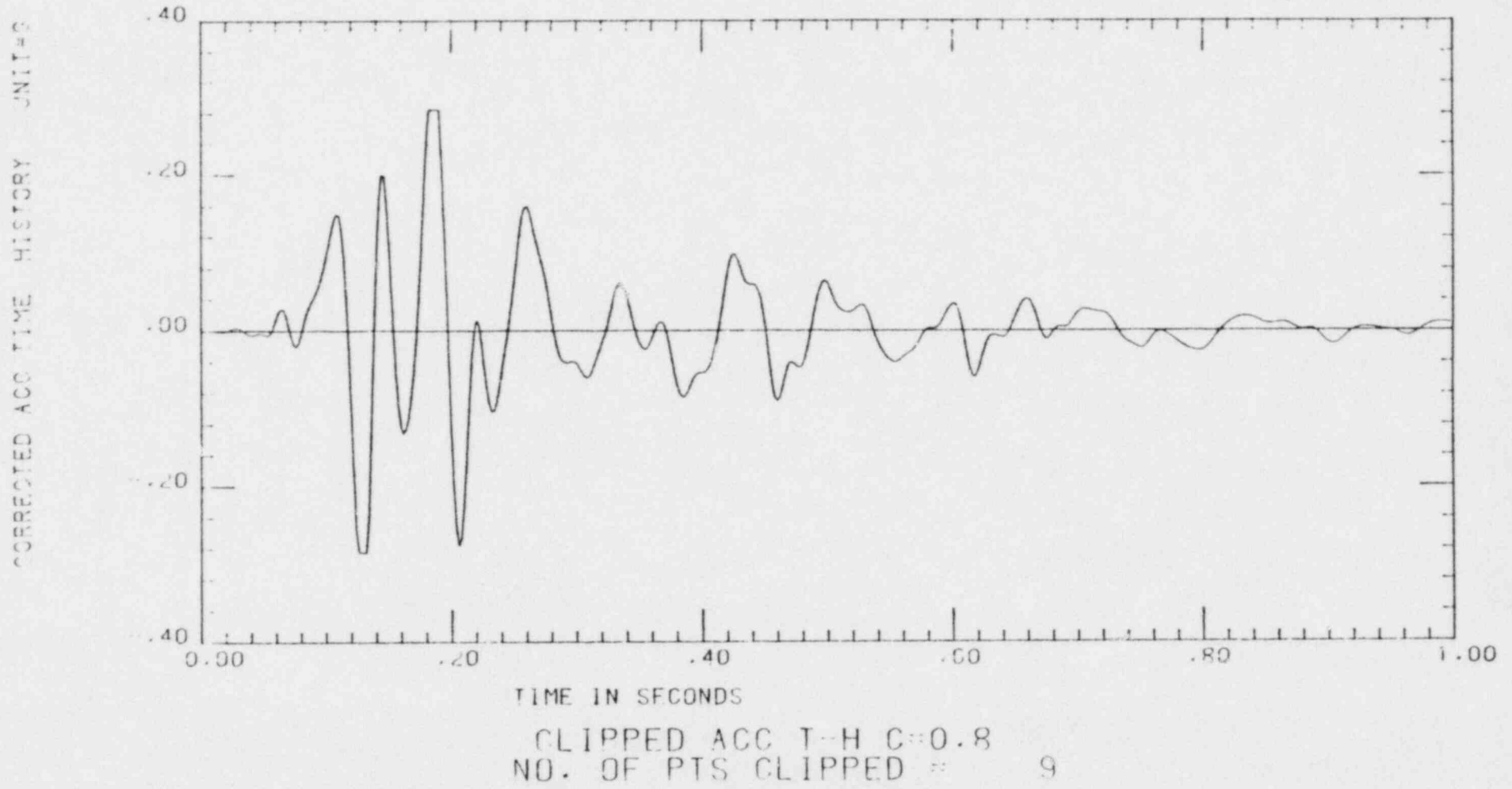


FIG. B7-21 -- 90° TIME HISTORY FOR C = 0.80

SOUTH CAROLINA EQ 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 90 DEG

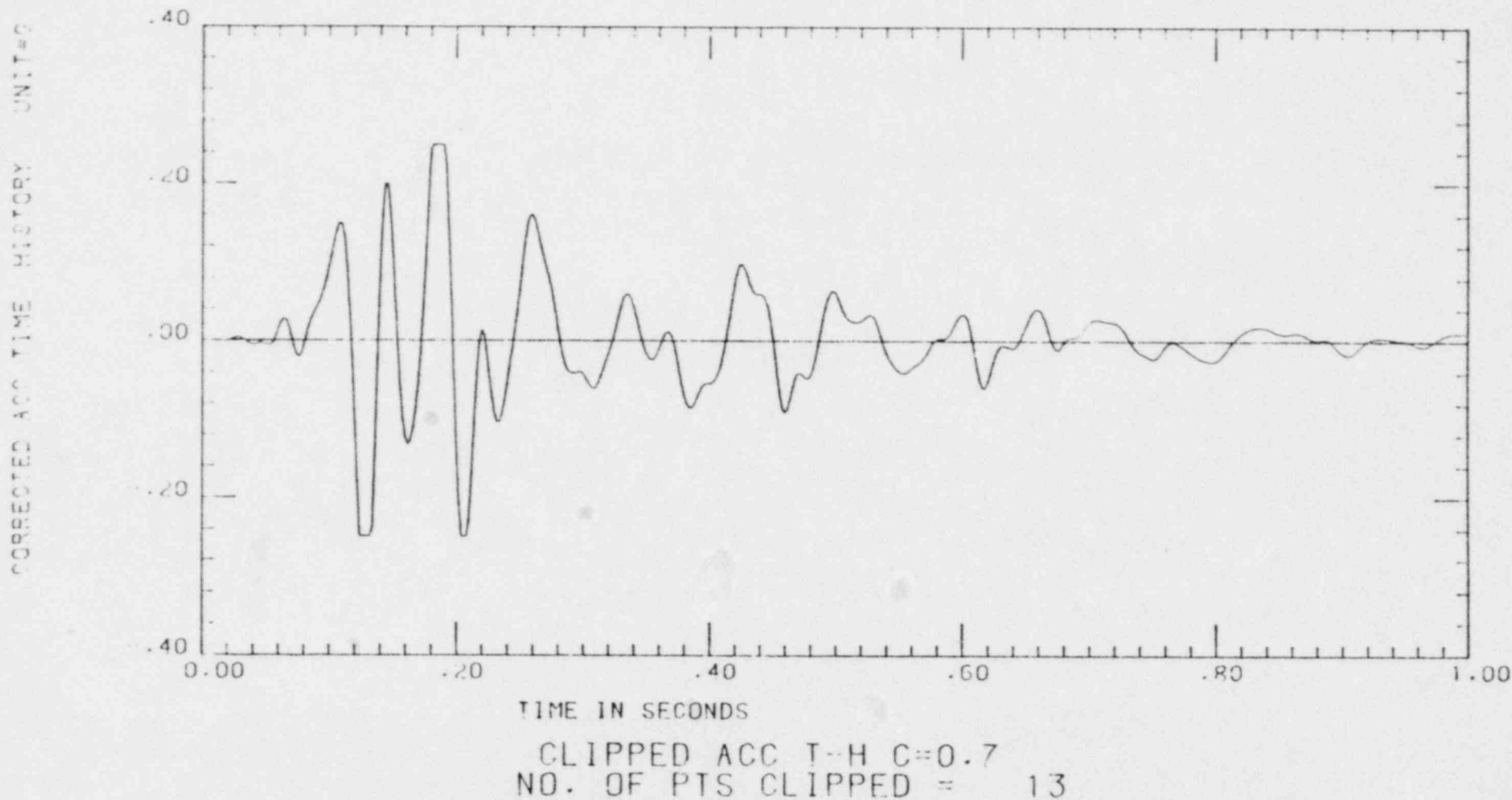


FIG. B7-22 -- 90° TIME HISTORY FOR C = 0.70

SOUTH CAROLINA EQ 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 90 DEG

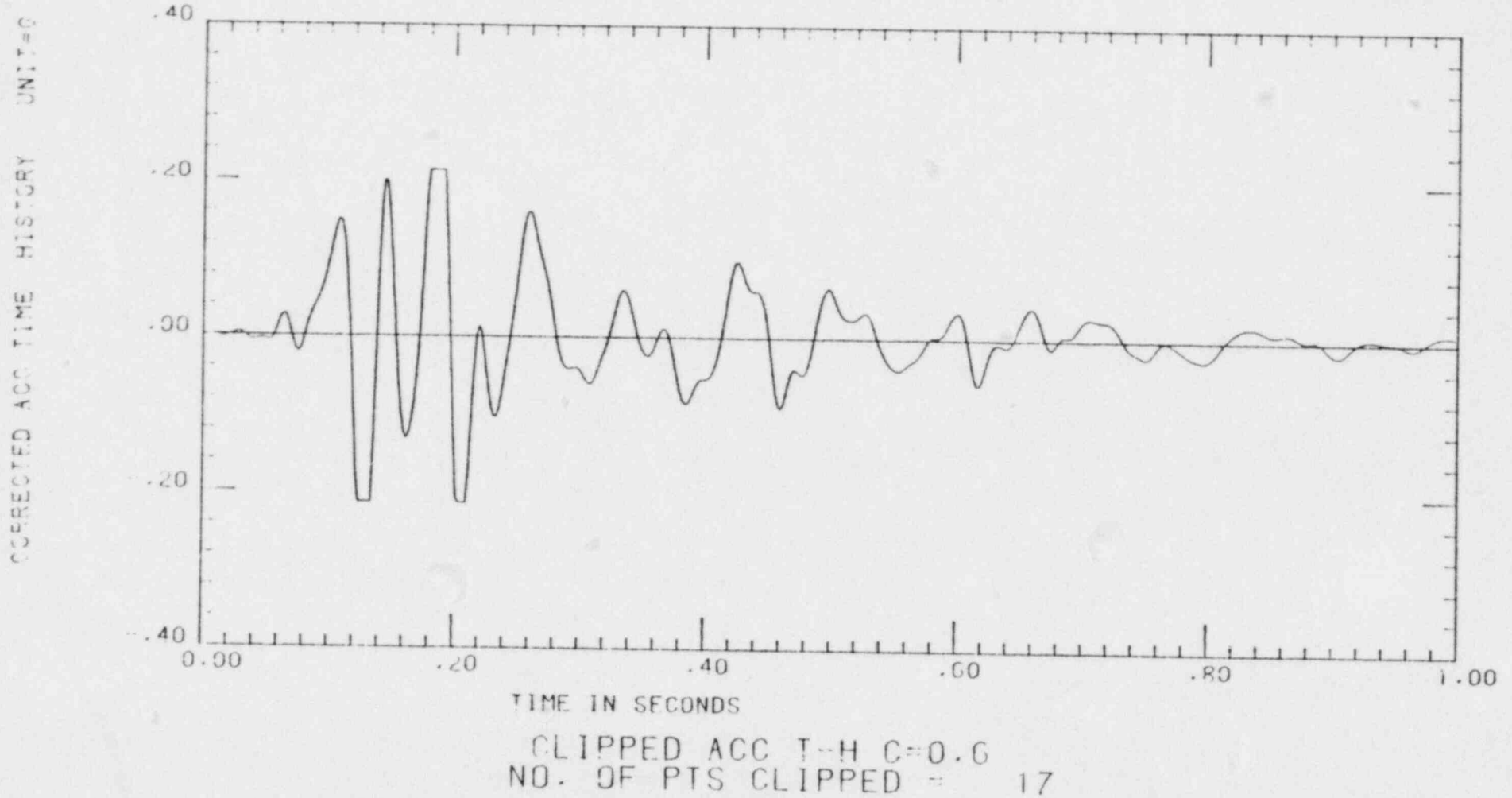


FIG. B7-23 -- 90° TIME HISTORY FOR C = 0.60

SOUTH CAROLINA EQ 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 90 DEG

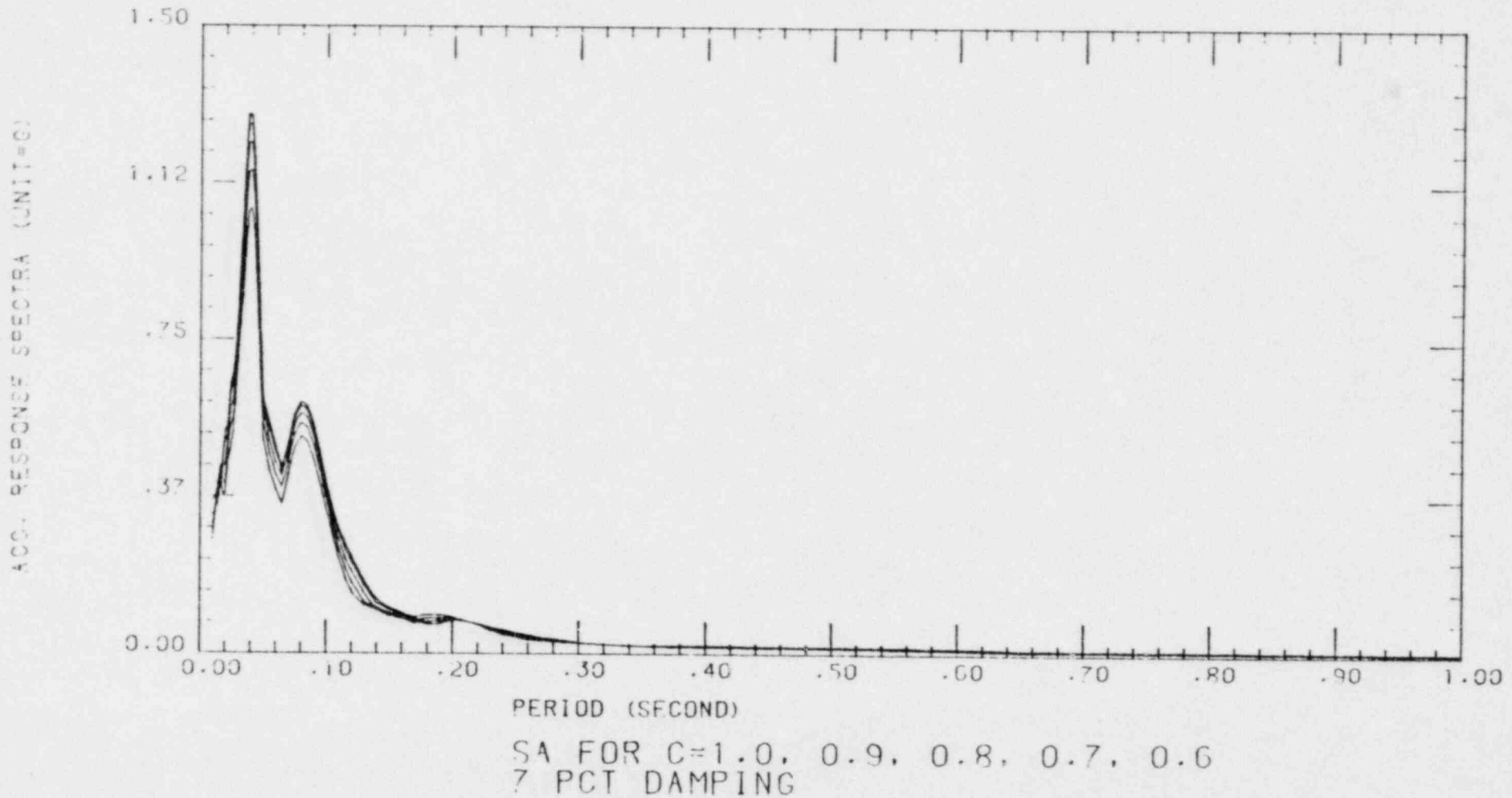
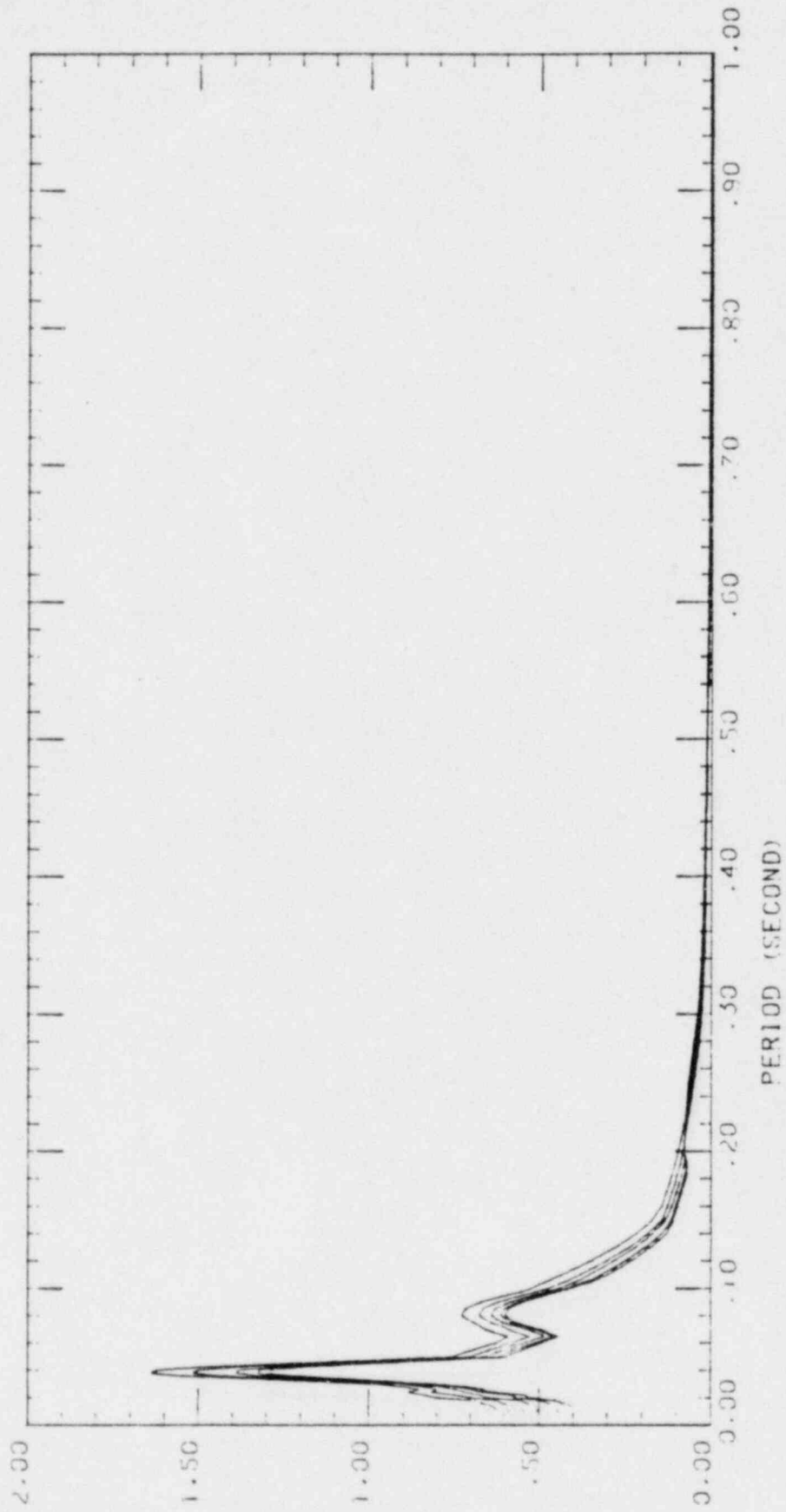


FIG. B7-24 -- 90° SPECTRA, CLIPPED, 7% DAMPING

SOUTH CAROLINA EO 160CT79 CORR ACC T--H
MONTICELLO CEN CREST 90 DEG



SA FOR C=1.0; 1.1, 1.2, 1.3, 1.4
7 PCT DAMPING

FIG. B7-25 -- 90° SPECTRA, AUGMENTED, 7% DAMPING

SOUTH CAROLINA EQ 16OCT79 CORR ACC T-H
MONTICELLO CEN CREST 90 DEG

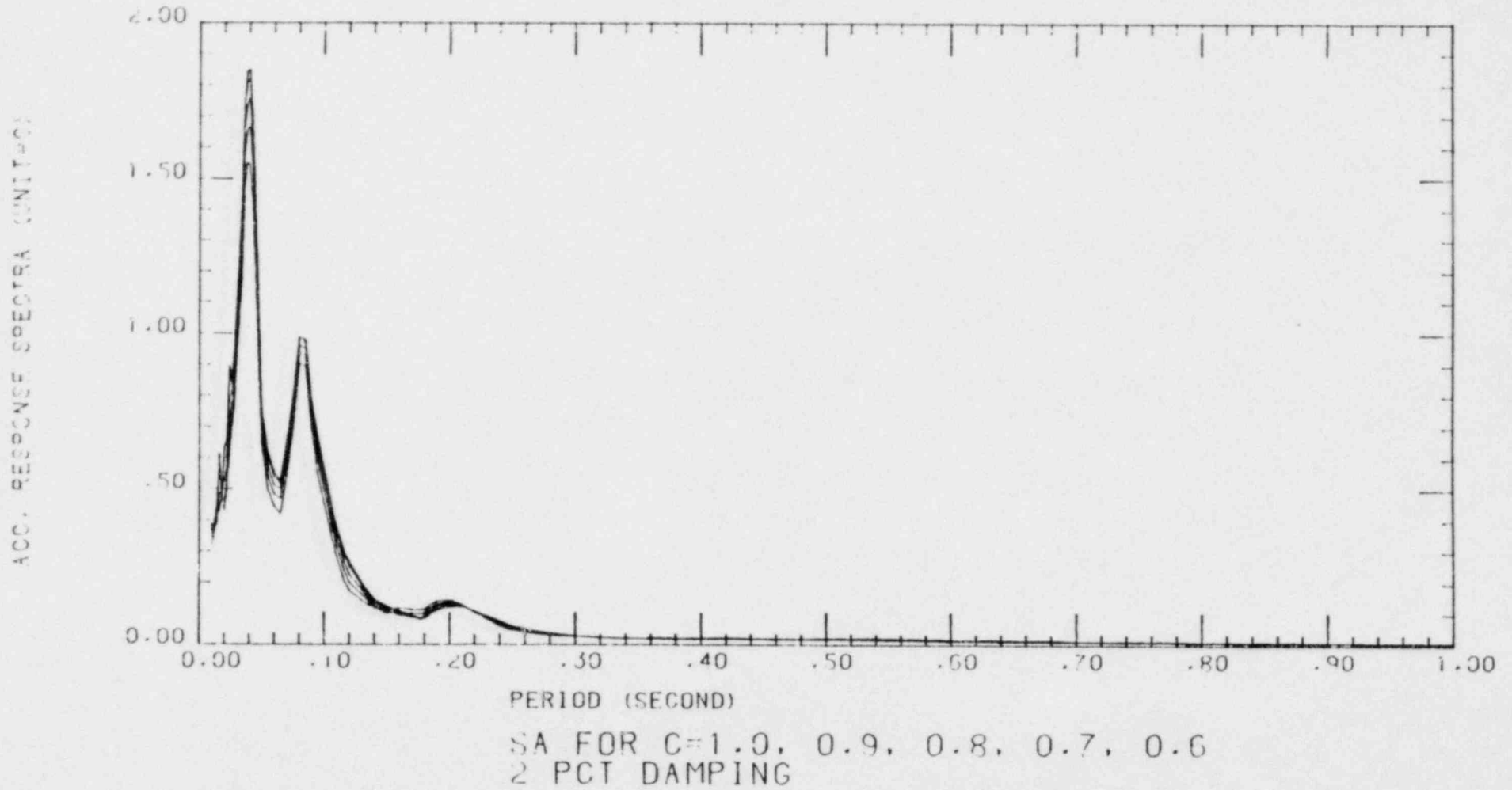
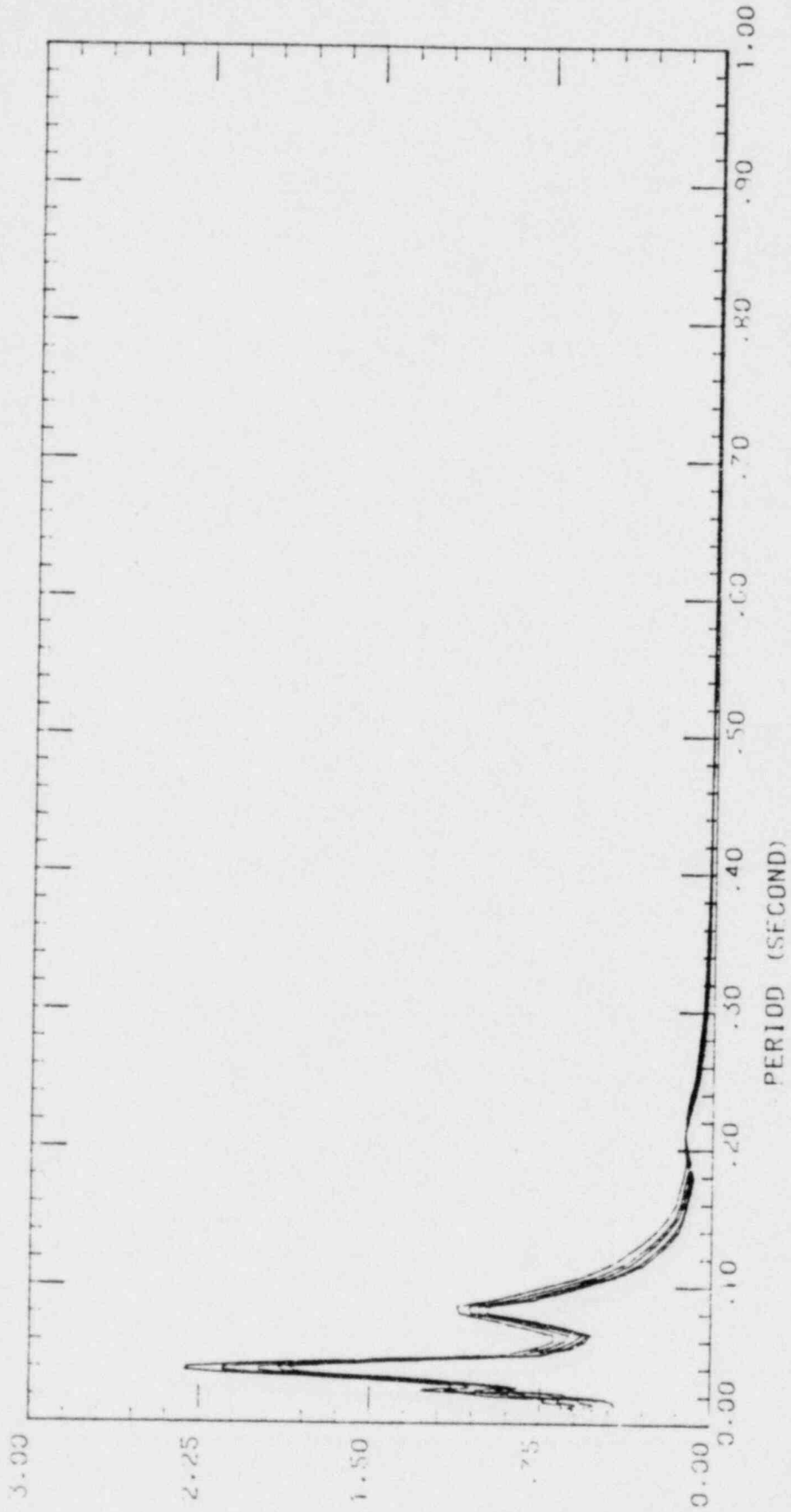


FIG. B7-26 -- 90° SPECTRA, CLIPPED, 2% DAMPING

SOUTH CAROLINA E.O. 1600T79 CORR ACC T-H
MONTICELLO GEN CREST 90 DEG



SA FOR C=1.0, 1.1, 1.2, 1.3, 1.4
2 PCT DAMPING

FIG. B7-27 -- 90% SPECTRA, AUGMENTED, 2% DAMPING

400. 9809024P 98018A (UNIT 9)

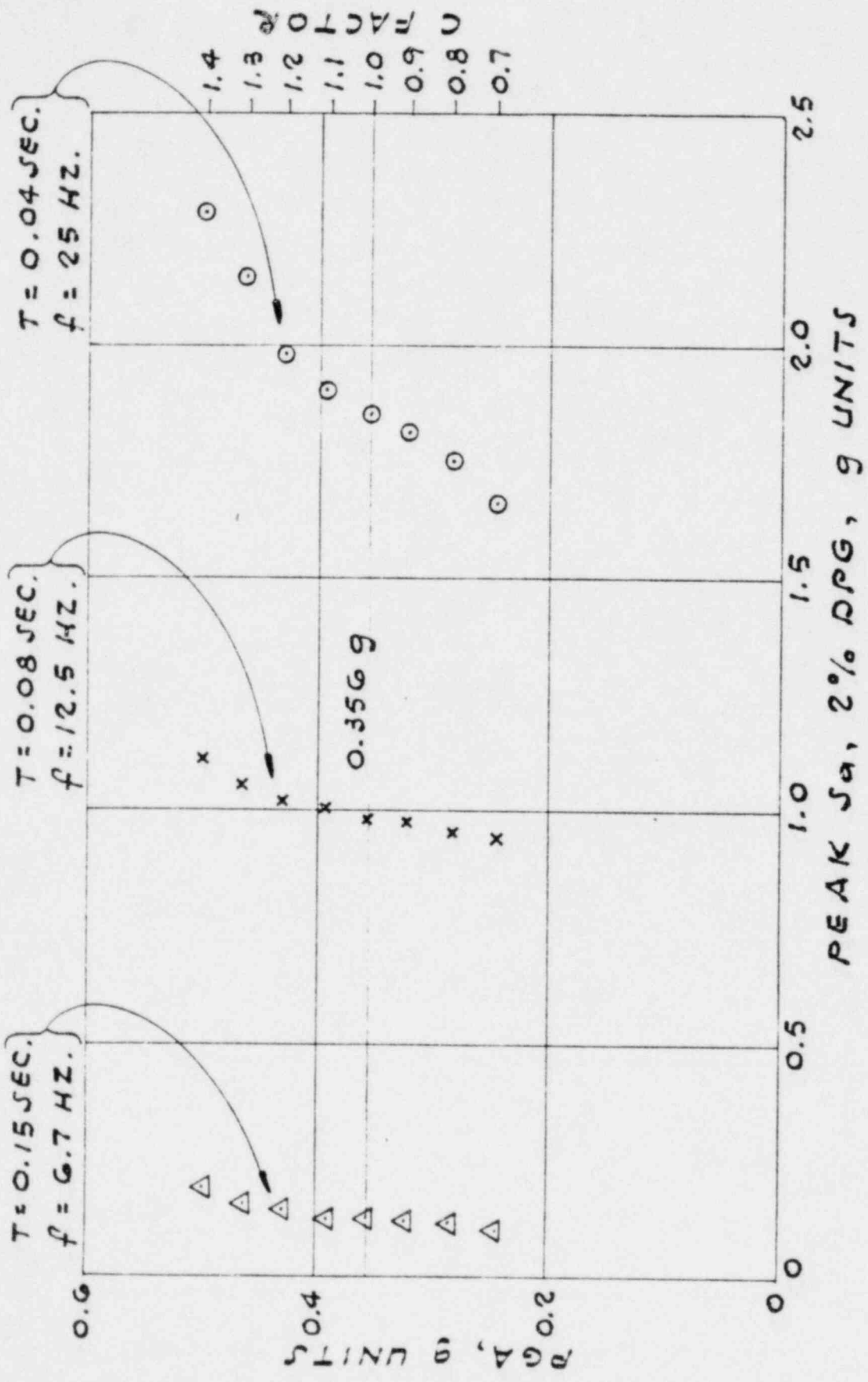


FIG. B7-28 -- SOUTH CAROLINA EQ. 10/16/79, MONTICELLO CREST, 90° COMPONENT

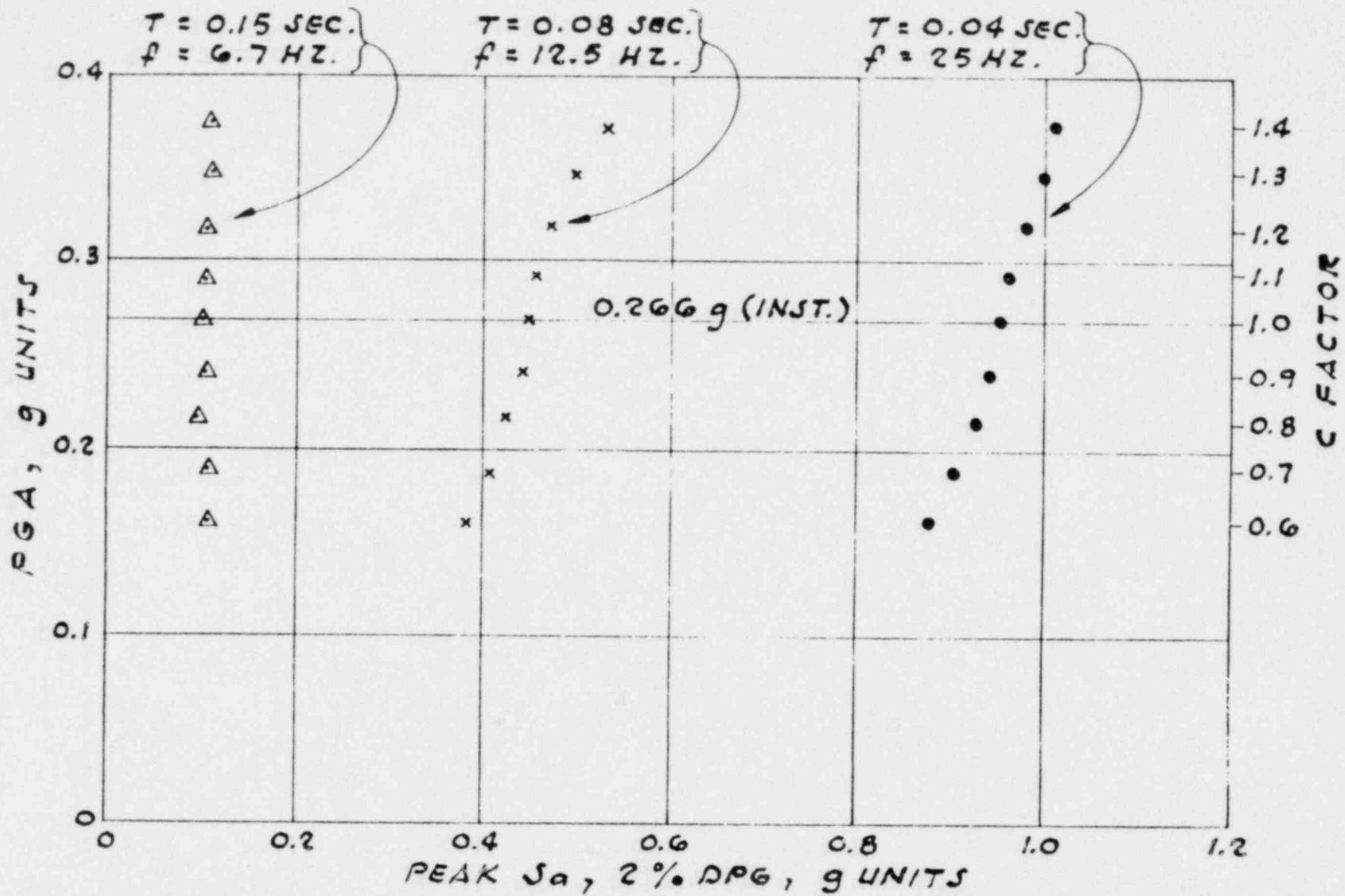


FIG. B7-29 -- EFFECT OF PGA ON RESPONSE, 2% DAMPING, BASED ON

MONTICELLO RESERVOIR EQ. OF AUGUST 27, 1978, 180° COMPONENT

APPENDIX B9
ON DIRECTIONAL COMPONENTS OF GROUND MOTION

by John A. Blume

There are many conservative assumptions in the design of nuclear plants for earthquake resistance and these are usually compounded to create margins of safety much greater than is generally recognized. One of these assumptions is that the directions of ground motion X, Y, and/or Z might obtain their peak design values simultaneously in both the time domain and in a directional sense (plus or minus). This brief report is about a recent study (URS/Blume, in process) of how the peaks really occur relative to each other on an absolute value basis.

In dynamic design there are two basic problems related to combining effects. These are (1) how to combine responses from the three components of ground motion, X, Y, and Z, and (2) how to combine responses from all effective modes of vibration. Many methods have been used, the most popular of which is the square root of the sum of the squares (SRSS) method which makes some allowance for the fact that all responses from directions and/or modes do not occur at the same time nor in the same direction. There is general agreement that the absolute sum method -- which is the maximum possible result -- would be ridiculously conservative with essentially a zero probability of occurrence.

There is growing evidence that SRSS may be conservative in many cases. For example, with regard to X, Y, and Z earthquake components, we are currently engaged in a study using 276 sets of 3-component real earthquake records. As part of that work we have considered the number of cycles between peaks in the different components. If the peaks in X, Y, and/or Z occur at different times and are sufficiently separated, there would be no reason to combine their effects. Such is not the case, but the probabilities of significant additive results seem to be very small. For example, we obtained the following probabilities of more than 5 cycles between the component peaks.

<u>Response frequency (Hz)</u>	<u>Probability (r, Absolute basis (ignoring sign))</u>	<u>> 5 cycles Between Peak Responses Using 1/2 of the residual tail as an allowance for sign (.5 + .5P)**</u>
5	0.64	0.820
10	0.75	0.875
15	0.84	0.920
20	0.85	0.925
25	0.87	0.935
30	0.88	0.940

*Note: In some component stress situations the sign would not be important; in most cases it would be; this is quite complex.

The above, based on 276 earthquakes, indicates that there is a strong probability at high frequency of there being more than 5 cycles between the peak components from the directions X, Y, and/or Z. This probability is at least one standard deviation above the median and possibly closer to 2 standard deviations. Translated to structural response, more than 5 cycles means that the effect of the peak motion under typical damping values has largely decayed; i.e., the peaks should not be combined in any significant amount as in SRSS. Another way of stating this is that the peaks tend to be statistically independent and time-separated insofar as response is concerned.

This is one of the many conservatisms in the design of nuclear plants and is different from and independent of that one which assumes for design that the two horizontal peaks are equal.

APPENDIX B10
MATERIAL STRENGTH
by John A. Blume

Introduction

The subject of this report is the strength of concrete and steel. In the body of the report, generic in nature, it is noted that the average strength obtained on well controlled projects is considerably in excess of the specified strength used by the designer. This is especially so for concrete, a basic material in nuclear plants. Appendix B10-A provides specific data for the concrete in the Virgil C. Summer station structures; Appendix B10-B for the reinforcing steel; and Appendix B10-C for the structural steel. The strengths are well above specified levels and the quality control was excellent as shown by the test results and particularly by the low coefficients of variation. The test data were assembled for us by Mr. R. C. Lindler of South Carolina Electric & Gas Company.

Discussion

The average strength of material actually provided in an engineered project is typically considerably greater than the specified strength used in design. In addition, the dispersion of strength from the mean value is generally small; i.e., the coefficient of variation, defined as the standard deviation divided by the mean, is small. Therefore, under conventional procedures there is much more resistance to seismic forces, and resulting safety factors or margins are much greater than is generally recognized. If the analysis were done on a probabilistic rather than a deterministic basis, the true values and safety factors would be identified.¹

Concrete is provided in greater average strength than needed (to insure that few test samples fall below the specified level). It continues to gain strength, at a decreasing rate, with age. For concrete of a

specified 28 day cylinder strength, f'_c , a mix is designed such that a very large percentage, often 90%, of the test cylinders at age 28 days will (be expected to) fall above the specified minimum value of f'_c . The average concrete strength will be much greater than the specified value of f'_c , as shown in the following table which is obtained under the assumption of normal distribution and the 90% figure noted above.

<u>Assumed coefficient of variation</u>	<u>Theoretical mean cylinder strength at 28 days</u>
0.10	$1.15 f'_c$
0.15	$1.24 f'_c$
0.20	$1.34 f'_c$

In addition, it is frequently found that other conservative factors increase the average concrete test values well above the theoretical values. For example, the 5000 psi f'_c specified value for the Virgil C. Summer containment, with 549 tests, had a ratio of average test value to specified strength of 1.32, a coefficient of variation of 0.081. This average test value would be 1.32 times 5000 psi, or 6600 psi, at the age of the tests. At one standard deviation below the mean, the test value would be approximately 6060 psi, and at two standard deviations below the mean it would be approximately 5520 psi. The low coefficient of variation, 0.081, indicates excellent job control of concrete quality.

There is no valid reason why the average test strength should not be utilized in assessing the seismic resistance of structures or structural assemblies as a whole, such as buildings, walls, or redundant elements.

The same general situation prevails for structural steel and for reinforcing bars. They are controlled by mill tests rather than as custom designed mixes like concrete.

Aging

In addition to the above, the increased value of concrete with age provides even more resistance that is normally ignored.

The type of cement, the type of aggregate, the additives, the mix and curing all affect the strength-age relationship. Figure B10-1 shows the strength variation up to 5 years.^{2,3} Beyond 5 years the increase is small but the increase in strength from 28 days to a few years of age is most significant and provides more seismic resistance than is normally recognized. For example, with Type II cement, the compressive strength at 28 days in Figure B10-1 is about 4200 psi whereas at 5 years it is about 6400 psi, a gain of 52% from the value that would be used in design, which, in turn, is low by perhaps 20% or more because of the mix conservatism first noted above. Thus, the true average value of this hypothetical concrete sample at age 5 years would be $(1.20)(1.52) f'_c = 1.82 f'_c$, or 82% greater than normally recognized.

Attachment B10A provides concrete test data for the Virgil C. Summer plant, Attachment B10B for reinforcing steel, and Attachment B10C for structural steel.

Conclusion

The plant structures are much stronger than indicated by calculations based on the minimum specified (design) values of structural materials. Although specified design values are applicable for use in advance of construction, there is valid reason why actual, known mean test values should be used for evaluating the strength of walls, redundant elements, or whole structures. For concrete, it may also be appropriate to allow for increased strength with age.

B10
REFERENCES

- (1) Blume, John A., "Allowable Stresses and Earthquake Performance," Proceedings, Sixth World Conference on Earthquake Engineering, India, 1977.
- (2) Troxell, G. A., and H. E. Davis, "Composition and Properties of Concrete," McGraw-Hill, New York, 1956.
- (3) U.S. Bureau of Reclamation, Concrete Manual, 1963.

STRENGTH OF CONCRETE

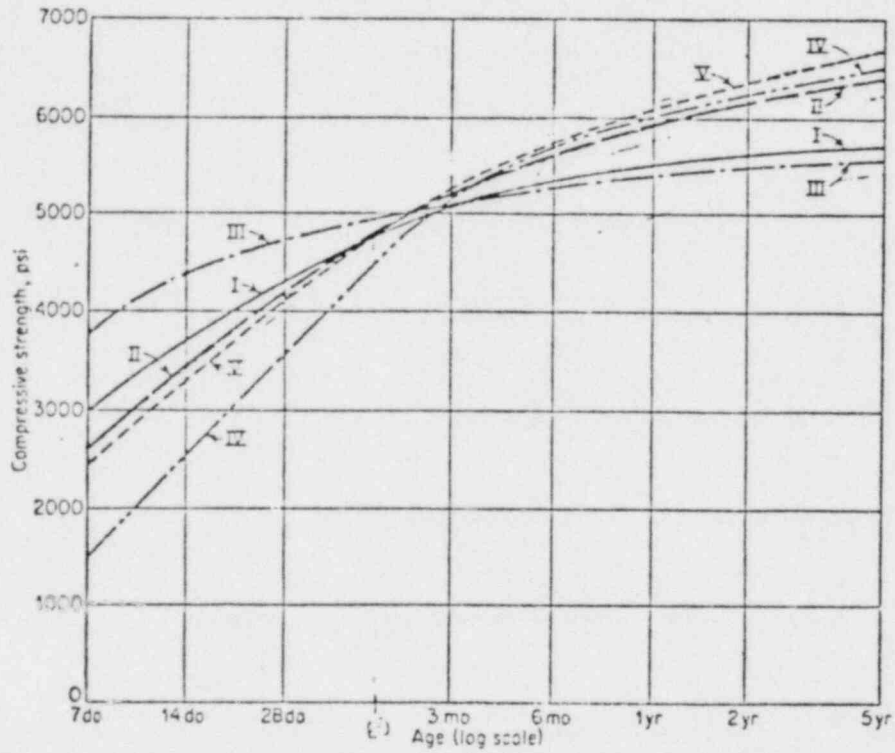


FIG. 10.5. Effect of type of cement on compressive strength of concrete. Aggregate 0 to 1½ in.; 6 sacks per cu yd; 6 by 12-in. cylinders; standard curing.

Type of cement	Compound composition, %							
	C ₁ S	C ₂ S	C ₃ A	C ₄ AF	CaSO ₄	Free CaO	MgO	Igni. loss
I Normal.....	49	25	12	8	2.9	0.8	2.4	1.2
II Modified.....	46	29	6	12	2.8	0.6	3.0	1.0
III High-early-strength.....	56	15	12	8	3.9	1.3	2.6	1.9
IV Low-heat.....	30	46	5	13	2.9	0.3	2.7	1.0
V Sulfate-resisting.....	43	36	4	12	2.7	0.4	1.6	1.0

(From U.S. Bureau Reclamation [106].)

ATTACHMENT B10A

CONCRETE PROPERTIES OF THE MAJOR STRUCTURES
AT THE VIRGIL C. SUMMER PLANT

B10A.1

Concrete Strength Data

Structural capacities under severe earthquake conditions depend upon the actual properties of the materials in a completed structure. The design structural capacities are based on allowable stresses which in turn are based on specified minimum material properties. It is well known that the material properties vary over a specified range of values. In addition, the various structural elements are designed for stresses which are below the allowable. Thus, when considering an entire completed structure, a whole spectrum of material strength values are available. In addition, on the basis of the strength tests which are required during construction, actual strength values can be used to evaluate the capacities of the structures and their elements. It is therefore appropriate to utilize average actual material properties in analysis for severe seismic events rather than highly conservative and inappropriate design values intended for individual members; alternatively the problem can be treated probabilistically.

Actual average strengths, as determined by tests, will be used in this analysis. Three different strengths of concrete were used for the various major structures at the Virgil C. Summer plant. The $f'_c = 5000$ psi mixes were used on the containment structure. The $f'_c = 3000$ psi mixes were used on the remaining major structures (e.g., Auxiliary building, Turbine building, Control building, Fuel Handling building, Intermediate building, Diesel Generator building, etc.). The $f'_c = 1500$ psi was used predominately as fill concrete for building foundations.

A variety of concrete mixes was used in the construction of the Virgil C. Summer major structures; many mixes utilized Type II, low alkali, moderate heat, Portland cement. Numerous basic mixes constitute most of the concrete, and a total of 2545 tests were performed on 6 x 12 concrete cylinders for all of the major structures. Tables 1 and 2 summarize the average strength and related statistical data.

In general, the average strength of the two different strengths ($f'_c = 5000$ and $f'_c = 3000$) of concrete in the existing structures exceeds the specified design strength between 32% and 55%. Note also the low coefficients of variation, indicating excellent mix control and also that mean values can be used with confidence.

There exists an additional margin of safety in that concrete gains considerable strength with age.^{1,2,3,4} For Type II cement the gain in strength with age for a two-year period ranges from 35- to 50-percent above the 28 (or 90)-day test values. These gains in strengths exceed the standard deviations indicated in Tables 1 and 2.

Table 1

SPECIFIED & AVERAGE CONCRETE TEST STRENGTHS FOR THE V.C. SUMMER PLANT

Concrete Mix	Minimum Specified f'_c (psi) @ No. of days	N	M(psi)	R(%)	σ (psi)	V
(A) B2-W/C-E7P(R) II	5000 @ 90	166	6,759	35.2	582	.086
(B) B2-W/C-E7P(R2) II	5000 @ 90	159	6,118	22.4	524	.085
(C) B2-W/C-E7	5000 @ 90	119	6,914	38.3	487	.070
(D) B2-W/C-E7P	5000 @ 90	55	6,916	38.3	449	.065
(E) B1-W/C-G2P II	5000 @ 90	23	5,962	19.2	722	.121
(F) B1-W/C-F4	5000 @ 90	21	7,304	46.1	582	.080
(G) B2-W/C-E7R	5000 @ 90	4	6,473	29.5	502	.077
(H) B2-W/C-E4	5000 @ 28	2	6,032	20.6	110	.018
(I) B2-W/C-E2P (R6.0) II	3000 @ 28	477	4,662	55.4	523	.112
(J) B2-W/C-E2	3000 @ 28	442	4,653	55.1	520	.112
(K) B2-W/C-E2 (R6.0) II	3000 @ 28	136	4,595	53.2	397	.086
(L) B2-W/C-E2P	3000 @ 28	107	4,661	55.4	469	.100
(M) B2-W/C-E2P(R6.0) I	3000 @ 28	40	4,565	52.2	595	.130
(N) B1-W/C-BR5 I	3000 @ 28	28	4,715	57.2	718	.152
(O) B1-W/C-G3P	3000 @ 28	22	4,685	56.2	605	.129
(P) B1-W/C-A (R)	3000 @ 28	21	4,030	34.3	346	.086
(Q) B1-W/C-B	3000 @ 28	16	4,803	60.1	353	.073
(R) B2-W/C-E2 (R6.0) I	3000 @ 28	12	4,391	46.4	465	.106
(S) B2-W/C-E2P (R6.5) II	3000 @ 28	11	4,219	40.6	419	.099
(T) B1-W/C-BR(2)	3000 @ 28	8	5,157	71.9	602	.117
(U) B1-W/C-B(R4) I	3000 @ 28	4	5,513	83.8	366	.066
(V) B2-W/C-A	3000 @ 28	1	5,245	74.8	135	.026
(W) B2-W/C-E1	1500 @ 28	653	2,984	98.9	355	.119
(X) B2-W/C-F1	1500 @ 28	12	2,382	58.8	431	.181
(Y) B2-W/C-II	1500 @ 28	6	2,450	63.3	260	.106

Table 2

GROUPED CONCRETE STRENGTH STATISTICS FOR THE V.C. SUMMER PLANT

Group Description	¹ N _{total}	² M̄ (psi)	³ R̄ (%)	⁴ V̄	⁵ σ̄ (psi)	⁶ N _σ
All f' _c = 5,000 mixes	549	6,605	32.1	.081	535	2.98
All f' _c = 3,000 mixes	1,325	4,642	54.7	.109	506	3.25
All f' _c = 1,500 mixes	671	2,968	97.9	.120	356	4.12

Nomenclature

N = number of samples per test sequence

M = mean or average strength

R = average strength increase above specified

V = coefficient of variation (eg. σ ÷ M)

σ = standard deviation from mean

N_σ = the number of σ's above the specified corresponding to the strength

$$^1 N_{total} = \sum_i N_i$$

$$^2 \bar{M} = (\sum_i M_i N_i) \div N_{total}$$

$$^3 \bar{R} (\%) = 100 \times \left[\frac{\bar{M}}{f'_c} - 1 \right]$$

$$^4 \bar{V} = (\sum_i V_i N_i) \div N_{total}$$

$$^5 \bar{\sigma} = \bar{V} \times \bar{M}$$

$$^6 N_{\sigma} = \frac{\bar{M} - f'_c}{\bar{\sigma}}$$

B10A

REFERENCES

- (1) Troxell, G. A., and H. E. Davis, "Composition and Properties of Concrete," McGraw-Hill, New York, 1956.
- (2) U.S. Bureau of Reclamation, Concrete Manual, 1963.
- (3) Kee, C. F., "Relation Between Strength and Maturity of Concrete," Title No. 68-21, ACI Journal, March 1971.
- (4) Washa, G. W., and K. F. Wendt, "Fifty Year Properties of Concrete," Title No. 71-4, ACI Journal, January 1975.

B10A.6

ATTACHMENT B10B

REINFORCEMENT STEEL STRENGTH DATA FOR MAJOR
STRUCTURES AT THE VIRGIL C. SUMMER PLANT

Reinforcing Steel Strength Data

Table 1 summarizes the steel strength data for the reinforcing steel for all of the major structures. All reinforcing steel used at the Virgil C. Summer plant was Grade 60, in which the minimum specified yield strength (f_y) is 60,000 psi and the minimum specified tensile strength is 90,000 psi. The test results listed in Table 1 are summarized by building, which are then combined to yield the weighted average results shown in Table 2 for the overall plant.

In summary, the average yield strength of reinforcing steel for the plant is more than 13% higher than the specified, whereas the ultimate (tensile) is 70% or more higher than the grade specified yield strength.* The low values of the average coefficients of variation for the plant indicate that there was excellent mill and job control for the quality of the reinforcement steel.

* The 70% shown is the important value because design is controlled by yield rather than ultimate values; the actual average tensile strength is 13% above the specified tensile strength.

Table 1

REINFORCING STEEL STRENGTH DATA FOR THE V.C. SUMMER PLANT

Structure	Yield Strength				Tensile Strength			
	N	M	σ	V	N	M	σ	V
Diesel Generator Bldg.	41	68,651	4,974	.072	41	103,884	6,329	.061
Intermediate Bldg.	177	67,884	4,202	.062	177	102,152	5,754	.056
Control Bldg.	121	67,507	4,469	.066	121	102,839	5,547	.054
Fuel Handling Bldg.	101	66,921	3,658	.055	101	100,457	6,821	.068
Containment Bldg.	497	68,258	4,940	.072	497	101,173	6,312	.062
Service Water Intake	74	67,175	4,238	.063	74	102,609	5,785	.056
Auxiliary Bldg.	491	68,350	4,730	.069	491	101,923	5,802	.057
Circulating Water Str.	105	68,059	4,035	.059	105	103,241	5,719	.055
Overall Plant	1,607 ¹	68,051 ²	-	.0672 ³	1,607 ¹	101,860 ²	-	.0588 ³

Table 2

OVERALL PLANT STATISTICS

Item	Value	
	Yield	Tensile
(A) $R = (\bar{M} \div D)$	1.134	1.131
(B) $\bar{\sigma} = (\bar{V} \times \bar{M})$	4,573 psi	5,989 psi
(C) $N_{\sigma} = \frac{\bar{M} - D}{\bar{\sigma}}$	1.76	1.98

Nomenclature

- N = number of samples
- M = mean or average value (psi)
- σ = standard deviation (psi)
- V = coefficient of variation (eg. $\sigma \div M$)
- D = minimum specified material strength value
(eg. 60 ksi for yield, & 90 ksi for tensile)

Notes: All reinforcing steel is GRADE 60 in which the minimum specified yield strength (f_y) is 60 ksi and the minimum tensile (or ultimate) strength is 90 ksi

$${}^1N_{total} = \sum_i N_i$$

$${}^2\bar{M} = (\sum_i M_i N_i) \div N_{total}$$

$${}^3\bar{V} = (\sum_i V_i N_i) \div N_{total}$$

ATTACHMENT B10C

STRUCTURAL STEEL STRENGTH DATA FOR MAJOR
STRUCTURES AT THE VIRGIL C. PLANT

B10C.1

Structural Steel Strength Data

Table 1 summarizes the steel strength data for the A36 structural steel for all of the major structures. For A36 steel, the minimum specified yield strength (f_y) is 36,000 psi and the minimum specified tensile strength is 58,000 psi. The test results listed in Table 1 are summarized by building, which are then combined to yield the weighted average results shown in Table 2 for the overall plant.

In summary, the average yield strength of the A36 structural steel for the plant is more than 21% higher than the specified, whereas the ultimate (tensile) is 89% or more higher than the grade specified yield strength. The low values of the average coefficients of variation for the plant indicate that there was excellent mill and job control for the quality of the A36 structural steel.

Table 1

STRUCTURAL STEEL STRENGTH DATA FOR THE V.C. SUMMER PLANT

Structure	Yield Strength				Tensile Strength			
	N	M	σ	V	N	M	σ	V
Containment Bldg.	260	44,052	4,410	.100	259	67,939	3,926	.058
Auxiliary Bldg.	66	44,079	3,696	.084	66	69,048	4,195	.061
Fuel Handling Bldg.	159	43,811	3,882	.089	159	68,466	3,710	.054
Service Water Pumphouse	51	42,963	3,578	.083	51	67,260	3,849	.057
Intermediate Bldg.	16	42,854	4,019	.094	16	67,166	3,561	.053
Control Bldg.	33	43,298	3,387	.078	33	66,496	3,627	.055
Overall Plant	585 ¹	43,819 ²	-	.0923 ³	584 ¹	68,045 ²	-	.057 ³

Table 2

OVERALL PLANT STATISTICS

Item	Value	
	Yield	Tensile
(A) $R = (\bar{M} \div D)$	1.217	1.173
(B) $\bar{\sigma} = \bar{V} \times \bar{M}$	4,044	3,878
(C) $N_{\sigma} = \frac{\bar{M} - D}{\bar{\sigma}}$	1.933	2.590

Nomenclature

N = number of samples

M = mean or average value (psi)

σ = standard deviation

V = coefficient of variation (eg. $\sigma \div M$)

D = minimum specified material strength
(eg. 36 ksi for yield, 58 ksi for tensile)

Notes: All structural steel results are for A36 steel where the minimum specified yield strength (f_y) is 36 ksi and the minimum tensile strength is 58 ksi

$${}^1N_{\text{total}} = \sum_i N_i$$

$${}^2\bar{M} = (\sum_i M_i N_i) \div N_{\text{total}}$$

$${}^3\bar{V} = (\sum_i V_i N_i) \div N_{\text{total}}$$

APPENDIX B11
SOME LESSONS FROM RIS AT HSINFENGIANG RESERVOIR
By John A. Blume

Introduction

The reservoir induced seismicity near Hsinfengkiang Dam in the People's Republic of China, about 160 km northeast of Canton, has been studied in depth by PRC engineers and scientists. There is much to be learned from this work that is of interest in connection with the Virgil C. Summer Station. Although, I was not able to visit the dam when I was in China in 1980, I saw dam models, talked with and lectured to many engineers and scientists, and obtained authoritative information about the dam and its RIS.

The Dam

The dam is situated on a rock site composed of Jurassic-Cretaceous granite. It is a diamond-head buttress dam of massive blocks of concrete and it has been reinforced twice for increased earthquake resistance. There are active local faults but the original design was only for MM VI. The dam, as modified, is generally similar to what we would term a gravity dam. The maximum height is 105 m, the overall crest length 440 m and the approximate length of the base 130 m. Its base thickness normal to the crest is about 130 m. The crest and the dam axis are straight. The reservoir stores 11,500 million m³ and its surface area is 390 km².

Impounding of water began in October 1959 and first reached its maximum level on September 23, 1961. On March 19, 1962 there was a main shock of magnitude 6.1, 1.1 km northwest of the dam. The focal depth was 5 km and the intensity at the epicenter was rated VIII. There were some foreshocks and a great many aftershocks. An instrumentation network, with stations on the dam and also away from the dam, has produced a great volume of valuable data. The material to follow is about certain features of interest in connection with the Summer plant.

Crest Motion versus Base Motion

Hsu Tsung-ho et al (1976), provided a tabulation of data from 28 local earthquakes as shown in Table 1. In this material the M_S magnitudes range from 1.1 to 4.5, the focal depths from 2.9 km to 9.9 km, and the epicentral distances from 0.8 km to 4.9 km.

There is a trend of increasing magnitude with depth, but with considerable scatter. There is also a similar trend with epicentral distance, also with scatter. The more meaningful slant distance to the focus, R , was therefore computed from the data and plotted against M_S in Figure B11-1.

It is obvious in Table 1 that the top of the dam moves much more than the foundation of the dam. This is structural response or amplification which is provided for in the design of all nuclear plants. The average amplification for the dam crest is about 4.4 times the foundation motion.

Ratio of Nearby Ground Surface Acceleration to Foundation Acceleration

The motion at the foundation of the dam is generally less than that of the nearby ground surface. The ratios of motion were computed and have the range of 0.254 to 0.987 with an average of 0.611 for all the data points provided. The ground instrument was located on bedrock about 100 m downstream of the dam axis.

This is a strong indication that the dam foundation has an attenuation effect on the free field ground waves, no doubt especially so at the higher frequencies. This effect, which has been termed the "Tau effect" for large foundations, is similar to that causing a large ship in a rough sea to "iron out" or average certain waves and thus ride with much less rolling, pitching and yawing than a small boat in the same sea.

To quote from a special report by the Chinese engineers and scientists (Sheng Chung-kang, et al., 1973):

For the same earthquake, the acceleration at the dam foundation a_g is less than that at the bedrock about 100 m away from the dam a_g' . The ratio of the former to the latter is about 2/3 (Fig 2-4). The peak points on the response spectra and Fourier spectra at the two places coincide approximately with each other, and both the horizontal and vertical components show such similar tendency (Figs 2-5 and 2-6). The decrease of the ground acceleration at the dam foundation may probably reflect the interaction of the foundation and the dam, for which further investigation is still being carried on.

In other words, here is a classical example of the effect of a large structure in reducing free field peak amplitudes, even in bed rock. The quoted 2/3 value is quite similar to what was used at Diablo Canyon in analysis and accepted by the ALAB-644 decision for that plant.

Peak Horizontal Accelerations

The greatest free field motion (Table 1) is 92.4 gal (cm/sec^2). This is only 0.094g! For the 4.5 M_s event, the ground motion was only 74 gal or 0.075g. There are no strong motion data available for the main shock of March 1962.

Sheng Chung-kang et al. (1973), produced an "approximate" formula based on 22 events with M_s from 1.1 to 4.5:

$$a_g = 35 \times 10^{0.15 M_s - 0.5 \log D}$$

wherein

a_g is peak ground acceleration, gal

D is epicentral distance, km

This gives the following accelerations, cm/sec²:

M_s	Equivalent M_l for Eastern USA**	D = 1	D = 3	D = 5	D = 10
4.0	4.8	139*	80	62	44
4.5	5.0	166*	96*	74	52
5.0	5.3	197*	114*	88*	62

Although the events shown above with an asterisk are not considered possible at the Virgil C. Summer Station, an effective design acceleration of 0.15g would accommodate all of them, even with increases for confidence levels. A formula based on slant distance to the focus would be more meaningful for close in events.

Stress Drop and Fault Dimensions

The main shock at Hsinfengkiang was $M_s = 6.1$. The report by Sheng Chung-kang et al. (1973), which was very carefully developed with 8 author specialists, provides the following data on the focus parameters:

Strike-slip faulting (mainly)

Length of fault - 14 km (strike N62°E, dip NW 80°)

Dislocation - 9.5 cm

Seismic Moment - 1.1×10^{25} dyne-cm

Stress Drop - 7.5 bars

Propagation velocity - 2.2 km/sec

** From O. Nuttli, November 16, 1981

Discussions of 50 to 100 or more bars for stress drop at Summer plant seem wholly incompatible with 7.5 bars for a 6.1 M_s earthquake.

Summary

Based on all I have heard, seen or read about the Hsinfengkiang Dam RIS, as well as other RIS, the design conditions at the Virgil C. Summer Station appear both adequate and conservative.

References

Sheng Chung-hang, et al., "Earthquakes Induced by Reservoir Impounding and Their Effect on the Hsinfengkiang Dam," report, Peking, China, April 1973.

Hsu Tsung-ho, et al., "Strong-Motion Observation of Water-Induced Earthquakes at Hsinfengkiang Reservoir in China," Engineering Geology, 10(1976): 315-330, printed in The Netherlands.

HSINFENGIANG DAM
DIAMOND HEAD BUTTRESS

TABLE I

Earthquake no.	Date*	Time* hr. min. sec.	Magnitude M_s	Focal depth (km)	Epicentral distance (km) (1)	A_{max} (goll)		
						Nearby ground surface	Foundation of the dam (2)	Top of the dam
1	1966.5.19	10-14-24.5	2.4	4.5	1.8 455	-	8.0 -	62.1
2	1966.5.28	10-08-18	3.2	7.2	2.8 775	36.7	9.8 267	81.8
3	1966.9.19	00-52-29.1	3.3	4.5	2.6 520	41.0	10.4 254	105.7
4	1967.7.29	19-07-31.2	3.6	5.3	3.3 634	64.2	22.0 343	248.0
5	1968.3.7	16-54-15.6	3.5	4.2	2.9 510	32.5	22.0 677	184.0
6	1968.3.19	02-28-29.9	3.5	6.5	1.6 669	7.7	4.7 610	54.9
7	1968.8.2	21-27-30.9	2.8	3.2	1.2 342	29.3	27.6 942	199.9
8	1968.8.4	04-56-40	2.6	3.4	0.8 349	31.4	24.0 324	124.0
9	1968.8.23	12-45-13.9	3.7	6.4	2.0 671	70.2	41.5 571	329.3
10	1970.2.19	04-47-7.3	1.7	9.0	4.8 1020	-	20.2 -	116.8
11	1970.4.19	21-23-56.5	3.5	4.0	1.0 422	56.8	55.7 951	606.5
12	1970.4.19	21-23-59.8	1.1	4.0	1.0 422	23.7	23.4 957	66.4
13	1970.4.19	21-24-05	2.1	4.0	1.0 422	47.1	30.3 453	177.9
14	1970.5.9	00-01-32	2.8	3.7	2.5 447	47.5	40.5 853	516.0
15	1970.10.3	23-38-10.5	3.5	2.9	2.0 352	92.4	58.1 629	493.5
16	1970.10.3	23-36-23.6	1.3	2.9	2.0 352	34.7	10.2 292	215.0
17	1971.1.2	07-41-54.5	3.0	3.4	1.8 355	62.8	39.4 627	255.0
18	1971.1.2	08-23-59.6	3.1	3.3	2.1 391	58.1	33.6 579	159.5
19	1971.2.25	13-09-50	3.5	3.0	1.9 355	-	68.6 -	480.0
20	1971.10.22	17-57-08	3.2	6.5	1.2 641	69.9	36.9 522	491.0
21	1971.11.15	07-45-45.8	2.3	5.0	0.8 526	29.0	16.5 560	116.5
22	1972.12.18	17-05-20	4.5	9.9	4.9 1105	74.0	56.0 757	597.0
23	1973.3.11	20-47-31	3.0	7.2	1.8 742	20.1	11.9 592	152.0
24	1973.6.2	20-42-1.1	2.6	6.2	1.0 625	10.3	7.9 267	56.3
25	1974.1.24	05-45-16	3.0	4.8	4.0 675	30.9	19.7 632	229.0
26	1974.3.1	06-14-46	3.1	6.9	2.5 734	15.4	6.6 420	32.0
27	1974.6.22	16-23-37.7	2.2	5.3	2.0 566	12.9	4.3 333	56.4
28	1974.8.16	07-33-58	3.0	6.8	2.5 724	-	4.3 -	56.9

* Peking local time.

(After Hsu Tsung-ho et al 1976)

(1) Slant distance, R

(2) Ratio of acceleration of the ground surface to the acceleration at the foundation of the dam

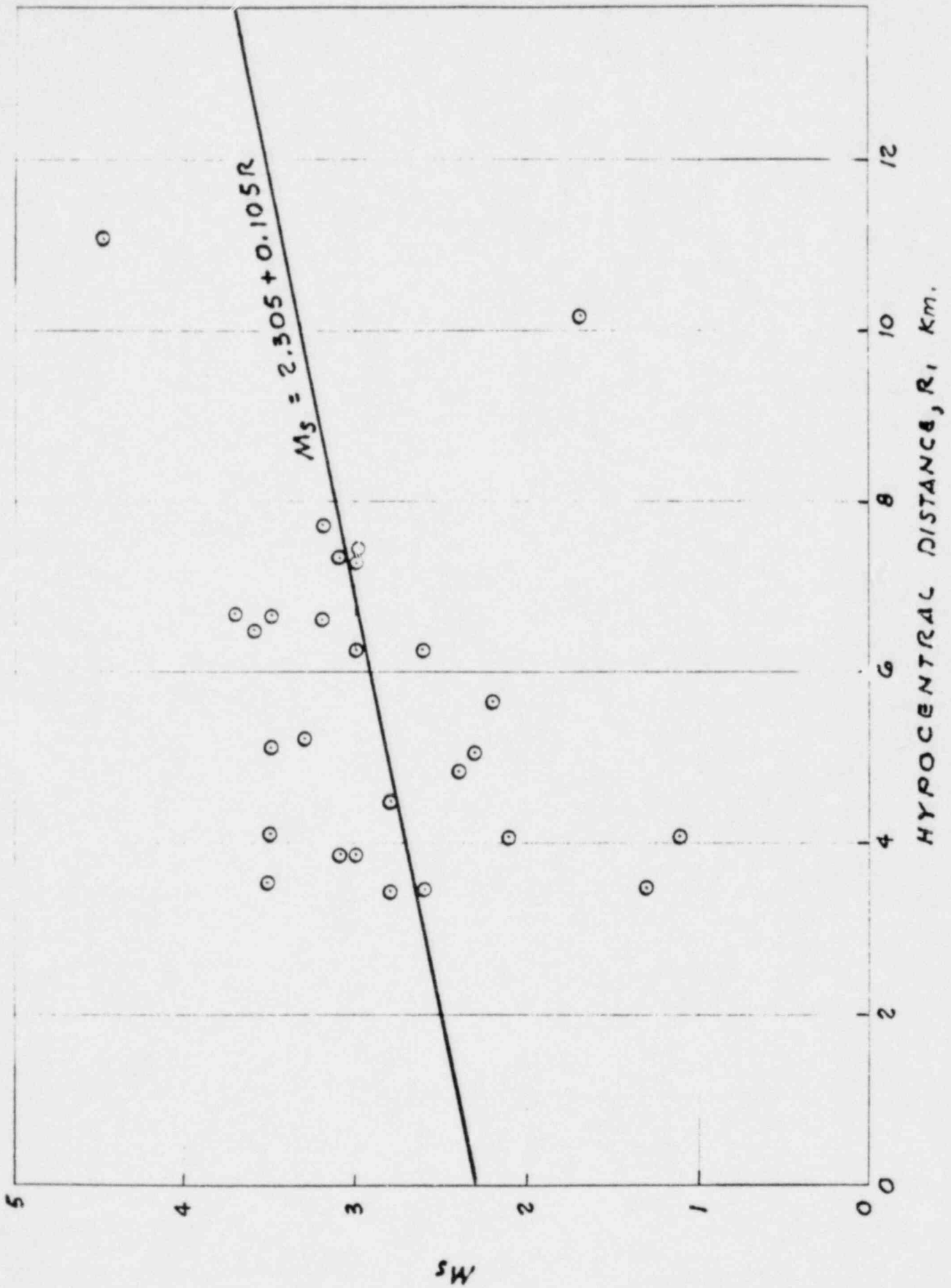


FIG. B11-1

APPENDIX B12
GENERIC LL DOCUMENTS FOR DIABLO CANYON
AND THE EL CENTRO STEAM PLANT

by John A. Blume

For the various hearings in the matter of the Diablo Canyon Nuclear Power Plant, Units 1 and 2, many special papers or reports were developed for the Pacific Gas & Electric Company, principally by me. Many of these were applicable only to that plant, such as comprehensive probabilistic studies for acceleration and spectral shape. Others were generic in nature and would apply to most, if not all, nuclear power plants. Abstracts from several of the latter type have been included in my written testimony in the main document and/or in its appendices.

The designation of these various reports was "D-LL"* with the D referring to Appendix D (Amendments 50, 53, and subsequent amendments) of the Final Safety Analysis Report, Units 1 and 2, Diablo Canyon Site, "Seismic Evaluation for Postulated 7.5M Hosgri Earthquake," Pacific Gas & Electric Company, 1977, U.S. Nuclear Regulatory Commission Docket Nos. 50-275 and 50-323.

The following outlines some of the generic reports pertinent to the Virgil C. Summer Plant. An asterisk indicates those referred to or abstracted for my testimony. The numbers in parentheses are the applicable amendment numbers.

*Derived from "Laundry List"

	<u>LL No.</u>		
*	5	"On the Adjustment of Response Spectra", Blume	(53)
*	9	"Data on Damping Ratios", Blume and Kabir	(50,53)
*	18A	"On the Major Component of Horizontal Ground Motion versus the Other Component", Blume	(50)
*	18B	"Effect of Natural Period Variations", Blume	(50,53)
*	18C	"On the Transition from Test Data to Design Equations", Blume	(50)
*	21	"Seismic Stress versus Total Stress", Blume	(53)
*	26	"Instrumental versus Effective Acceleration", Blume	(50)
*	30	"The Effect of Ground Acceleration on Spectral Response", Blume	(50)
*	35	"Performance of Industrial and Power Facilities in Major Earthquakes", Blume	(50)
	39	"On the Attenuation of Ground Motion by Large Foundations", Blume	(50)

The El Centro Steam Plant

In addition to the LL report series, special studies were made of the October 15, 1979, Imperial Valley earthquake for the appeal board hearing. One of these had to do with the performance of the El Centro Steam Plant, Unit 4, which unit was analyzed in detail after the 1979 earthquake for comparison of the damage sustained to the analysis results. The plant actually suffered minor structural damage. Unit 4 is the newest and largest at the plant; it is an 80-MW oil or gas facility completed in 1968 with

its steel frame designed for a lateral force coefficient of 0.20.** However, the spectral acceleration based on a nearby recording of the ground motion approached 1.2g at 7% damping.

We proceeded to analyze the building by the same methods used for the Diablo Canyon power plant analysis but with the aid of actual nearby recorded ground motion. The results of the analysis predicted much greater structural damage than actually occurred. On paper the steam plant sustained forces sufficiently high to cause buckling of many bracing members in the boiler structure, cracking of the operating floor diaphragm at the end of the turbine pedestal opening, and yielding of isolated columns in the turbine building and boiler structure. However, actual observed structural damage was confined to four buckled bracing members in the boiler structure. Although there are explanations for a portion of this result, most of it has to be attributed to conservatism in normal methods and procedures, and also to design criteria and capacity assumptions.

Reference is made to the ALAB-644 Decision of June 16, 1981, pages 151-159 for more information on the El Centro Steam Plant.

**Concrete walls were added to 3 sides of the building.

APPENDIX B14

GROUND ACCELERATION VERSUS DAMAGE, PROJECT RULISON

by John A. Blume

An underground nuclear explosion, termed Project Rulison, was detonated in Colorado on September 10, 1969. The resulting ground motion was measured at several stations ranging from 5 km to 300 km from ground zero. In addition, all damage, of any type, was investigated and the cost of repair or replacement was paid. The buildings in these Colorado towns were not designed for earthquake resistance, they had been subjected to minor, if any, prior natural earthquakes, and had natural periods in the general range of the peak values recorded. This brief report compares measured ground motion with actual damage which was estimated in great detail.

Table B14-1 is a summary of pertinent data. The damage shown was paid as of March 1, 1970 and is either the final (total) amount paid, or very close to that value. The nature of the damage included mostly brick chimneys and interior plaster cracks, but also glass, settlement, T.V. sets, cisterns, wells, utilities, etc. There were 557 credible claims from 300 towns and 257 rural areas.

TABLE B14-1 RULISON GROUND MOTION VERSUS DAMAGE

<u>Town</u>	<u>Distance from Ground Zero, km</u>	<u>Peak Horiz. Acceleration</u>	<u>Peak 5% Damped Horiz. Spectral Acceleration</u>	<u>Frequency of Spectral Peaks (Hz)</u>	<u>No. of Buildings in Town</u>	<u>Total Damage</u>
Grand Valley	10	0.36g	1.0g	7 to 15	146	\$15,044
Rifle*	18 to 20	0.14g 0.08g 0.06g	0.50g 0.33g 0.21g	7 to 10	759 plus industrial	\$18,995
Collbran	19	-----	0.13g	3 to 14	127	\$ 1,864
DeBeque**	23	0.05g 0.13g	0.22g 0.43g	5 to 10	102	\$ 1,320
Silt	30	0.034g	— 0.15g	3 to 10	194	\$ 235

*3 recording stations

**2 recording stations

The average dollar damage per exposed building is not available. If the fact that all damage is included, including that to cisterns, wells, T.V. sets, etc., is ignored, an upper bound damage value per building is obtained as follows:

<u>Town</u>	(1) <u>No. of Bldgs.</u>	(2) <u>Total Damage</u>	<u>(2)/(1)</u>	<u>Peak Horiz. Ground Acceleration</u>
Grand Valley	146	\$15,044	\$103	0.36g
Rifle	759 plus industrial	\$18,995	\$ 25	0.09g av.
Collbran	127	\$ 1,864	\$ 15	-----
DeBeque	102	\$ 1,320	\$ 13	0.09g av.
Silt	194	\$ 235	\$ 1	0.034g

The above shows that for Grand Valley with 0.36g peak horizontal ground motion (instrumental), the average damage per non-seismic-engineered building was less than \$103! In other words, peak acceleration is not too meaningful in response, and buildings have more seismic value than they are given credit for.

At Rifle, with 3 instruments peaking horizontally at 0.14g, 0.08g, and 0.06g for an average peak of 0.09g, the damage cost was below \$25 per non-seismic building.

Based upon the above, nuclear plants, well engineered to severe and meticulous seismic requirements, should have no damage at peak ground motions well above their postulated peak ground accelerations.

Reference

"Structural Response Studies for Project Rulison," Report JAB-99-78, February 1971, by John A. Blume & Associates Research Division, prepared for the Nevada Operations Office, USAEC, under Contract AT (26-1)-99.

"Observed Seismic Data, Rulison Event," Report NVO-1163-197, November 1969, by Environmental Research Corporation, prepared for the Nevada Operations Office, USAEC, under Contract AT (29-2)-1163.

"Announced United States Nuclear Tests, July 1945 through December 1979," Report NVO-209, January 1980, by Office of Public Affairs, U.S. Department of Energy, Nevada Operations Office.

APPENDIX B15
UNRECOGNIZED MARGINS

by John A. Blume

In my written testimony I listed some of the conservatisms in general practice and in NRC requirements that tend to reduce "on-paper" safety factors and margins to values much less than exist physically. In this appendix the effect of the items listed are combined on the basis that they are independent random variables. The values used for each item are my best judgment for the specific case of the Virgil C. Summer plant. The factors shown below represent the ratios of the estimated actual values to the allowable values.

	<u>Item</u>	<u>Reference</u>	<u>Estimated Ratios</u>	
			<u>Lower Bound</u>	<u>Upper Bound</u>
(a)	Test reduction	p. 30, 31	1.15	1.3
(b)	Material strength	p. 31	1.13	1.6
(c)	Equal horizontal comp.	p. 31, 32	1.1	1.3
(d)	Constant periods	p. 32	1.1	1.2
(e)	Floor spectra	p. 33	1.1	1.3
(f)	Smooth spectra	p. 33	1.1	1.2
(g)	Ductility	p. 33, 34	1.0	2.0
(h)	Seismic vs total stress	p. 34	1.0	3.0
-	Directional component peaks	App. B9	1.1	1.3
	Product of all ratios shown		2.09	39.5
	Product of all ratios but (g) and (h)		2.09	6.58

Additional items to the above list could be several, including the attenuating effect of large, rigid foundations, embedment below the surface, and others. However, even without them, the real values are estimated at a minimum of over twice the values credited, and from 7 to 40 times as an upper bound, depending upon whether items (g) and (h) exist and are allowed. Thus the real margins are great, say from 2 to 10 times the allowable values for structures, plant equipment, piping, etc.

PUBLICATIONS (Continued)

Design of Multistory Reinforced Concrete Buildings for Earthquake Motions, with N. M. Newmark and L. H. Corning; Portland Cement Association, Chicago, Illinois (1961)
(book used worldwide)

"Structural Dynamics in Earthquake Resistant Design," Journal of the Structural Division, ASCE, July 1958 and discussions ending September 1959; also published in Transactions, ASCE, Vol. 125 (1960)

"On Instrumental Versus Effective Acceleration, and Design Coefficients," 2nd U.S.A. National Earthquake Conference (August 1979)

PUBLICATIONS (Continued)

- "Damage Prediction for Low-Rise Buildings," with R. E. Scholl, *Proceedings*, Fifth World Conference on Earthquake Engineering, Rome, Italy (1973)
- "Inelastic Earthquake Analysis by a Code-Form Reserve Energy Technique," AAAS-CONACYT Latin-American Congress, Science and Man in the Americas, Mexico City, Mexico (1973)
- "Survival or Failure of Buildings in Major Earthquakes," presented at the Joint Session, SEAONC and ASCE, ASCE National Structural Engineering Meeting, San Francisco, California (April 1973)
- Recommendations for Shape of Earthquake Response Spectra*, with R. L. Sharpe and J. S. Dalal, Report to the Directorate of Licensing, U.S. Atomic Energy Commission, WASH-1254 (February 1973) (Basis for much of NRC Reg. Guide 1.60)
- "Probability of Earthquakes and Resultant Ground Motion," Chapter 15 of "Tailing Disposal Today," *Proceedings*, World Mining International Tailing Symposium, Miller Freeman Publications, Inc., San Francisco, California (1973)
- "Structural Response to Seismic Motion Generated by Underground Nuclear Explosions," with R. E. Skjei, *The Military Engineer* (January-February 1973)
- "Analysis of Dynamic Earthquake Response," State-of-Art Report No. 3, Technical Committee 6, Earthquake Loading and Response, *Proceedings*, ASCE-IABSE International Conference on the Planning and Design of Tall Buildings (1972)
- "Structural Dynamic Theory," *Proceedings of the Structural Response Seminar, August 25, 1971*, URS/John A. Blume & Associates, Engineers, NVO-118, prepared for the U.S. Atomic Energy Commission, Nevada Operations Office (August 1972)
- "High-Rise Building Characteristics and Responses Determined from Nuclear Seismology," *Bulletin of the Seismological Society of America*, Vol. 62 (April 1972)
- "Civil Structures and Earthquake Safety," *Earthquake Risk*, Conference Proceedings, Joint Committee on Seismic Safety, California Legislature (September 1971)
- The Spectral Matrix Method of Predicting Damage from Ground Motion*, with R. E. Monroe, John A. Blume & Associates, Engineers, JAB-99-81, prepared for the U.S. Atomic Energy Commission, Nevada Operations Office (September 1971)
- "Civil Structures and Public Safety: Safety in Design and Construction of Civil Structures," *Public Safety, A Growing Factor in Modern Design*, National Academy of Engineering, Washington, D.C. (1970)
- "An Engineering Intensity Scale for Earthquakes and Other Ground Motion," *Bulletin of the Seismological Society of America*, Vol. 60 (February 1970)

PUBLICATIONS (Partial List)

"Allowable Stresses and Earthquake Performance," *Proceedings*, Sixth World Conference on Earthquake Engineering, New Delhi, India, 1977

"Engineering Intensity Scale Data for the San Fernando Earthquake of February 9, 1971," *Proceedings*, Sixth World Conference on Earthquake Engineering, New Delhi, India, 1977

"The SAM Procedure for Site-Acceleration-Magnitude Relationships," *Proceedings*, Sixth World Conference on Earthquake Engineering, New Delhi, India, 1977

The following reports and papers, among others, were prepared for Pacific Gas and Electric Company and are published in the *Final Safety Analysis Report*, Units 1 and 2, Diablo Canyon Site, Amendment No. 50, "Seismic Evaluation for Postulated 7.5M Hosgri Earthquake," 1977, U.S. Nuclear Regulatory Commission Docket Nos. 50-275 and 50-323.

"Probabilities of Peak Site Accelerations and Spectral Response Accelerations from Assumed Magnitudes up to and Including 7.5 in All Local Fault Zones

"Instrumental versus Effective Acceleration"

"The Effect of Arbitrary Variation in Peak Ground Acceleration on Spectral Response"

"Probabilities of Peak Site Accelerations Based on the Geologic Record of Fault Dislocation"

"Earthquake Shaking and Damage to Buildings," with R. A. Page and W. B. Joyner, *Science* (22 August 1975)

"Predicting Damage Probabilistically for Buildings Subjected to Ground Motion," with J. W. Reed, ASCE Conference on Probabilistic Methods in Engineering, Stanford University, Stanford, California (June 1974)

"Concrete and Steel Structures," *Proceedings*, Earthquake Engineering for Water Projects, Department of Water Resources, State of California, Sacramento (January 1974)

Testing as an Aid to Analysis, Final Report and Proceedings of a Workshop on Simulation of Earthquake Effects on Structures, San Francisco, September 1973, National Academy of Engineering (1974)

"Seismic Design Spectra for Nuclear Power Plants," with N. M. Newmark and K. K. Kapur, *Journal of the Power Division*, ASCE (November 1973)

"Structural Response to Man-Induced Ground Motion," with S. A. Freeman, *Proceedings*, Fifth World Conference on Earthquake Engineering, Rome, Italy (1973)

Forces of Earthquake and Wind" (jointly authored) in 1953.

I received this same award in 1961 for my paper, "Structural Dynamics in Earthquake Resistant Design," and also in 1969 for my paper, "Dynamic Characteristics of Multistory Buildings."

I received the Building Industry Achievement "Man of the Year" Award in 1961 from the Building Industry Conference Board in recognition of outstanding contributions to the industry and for service to the community.

In 1962, I received the Ernest E. Howard annual award for research in the earthquake resistance of structures and in structural dynamics from the American Society of Civil Engineers.

In 1969, I received the Concrete Award from the Rock, Sand, and Gravel Producers Association of Northern California and Northern California Ready-Mixed Concrete and Materials Association in recognition of major continuing research and design leading to improved uses of reinforced concrete and providing for greater earthquake resistance.

California since 1967. This Board provides guidance in all the major earthquake-related problems of the state's water resources, including the \$2.8 billion State Water Project and the safety of dams (including the proposed Auburn and the Oroville dams) and other hydraulic structures and systems.

I am an Honorary Member of the International Association of Earthquake Engineering and was elected to the U.S.A. National Academy of Engineering in 1969. I am an Honorary Life Member of the New York Academy of Sciences. I am an Honorary Member of the American Society of Civil Engineers and am Past President of the San Francisco Section. I am an Honorary Member of the 1000 member Earthquake Engineering Research Institute, as well as a founder and immediate Past President. I am a Fellow of the American Consulting Engineers Council and the American Association for the Advancement of Science. I am Past President of the Consulting Engineers Association of California and the Structural Engineers Association of California. I am an Honorary Member and Past President of the Structural Engineers Association of Northern California. I am an Honorary Member of the American Concrete Institute. I am a Fellow of the Society of American Military Engineers. I am a Consulting Professor of Civil Engineering at Stanford University.

I have been registered in California as a Structural Engineer since 1940 and as a Civil Engineer since 1939.

I was the recipient of the American Society of Civil Engineers' Leon S. Moisseiff annual award for paper, "Lateral

I participate in and direct special projects and research operations for the firm. I am a recognized authority on earthquake engineering and structural dynamics in which I have pioneered the development and application of the original and many new concepts of a basic nature in dynamics.

I have served for fourteen years as principal consultant for studies of structural response to underground nuclear explosions for the U.S. Department of Energy Nevada Operations Office. I am also an active consultant on the earthquake aspects of nuclear power plant licensing, seismic criteria, and design and have been engaged by both the Federal government and private industry. I am an advisor to the National Science Foundation on research policy and earthquake engineering.

I was Chairman of the Management Committee for the design and construction of all structures and site work for the \$114-million Stanford Linear Accelerator Center near the San Andreas fault. I was honored by Stanford University naming their Earthquake Engineering Center for me.

I have served as the principal earthquake engineering consultant during the entire term of the Diablo Canyon Nuclear Power Plant NRC licensing reviews and hearings, and provided expert testimony on the seismic design criteria for the given site conditions and the earthquake design aspects, including the dynamic response of the plant's structures, piping, and equipment.

I have been a member of the Consulting Board for Earthquake Analysis, Department of Water Resources, State of

PROFESSIONAL QUALIFICATIONS

JOHN A. BLUME

My name is John A. Blume. I am Chairman of the Board and Senior Technical Consultant of URS/John A. Blume & Associates, Engineers of San Francisco, California.

In 1933 I received a B.A. in Civil Engineering from Stanford University. In 1935 I received an "Engineer" degree in Structural Engineering and in 1967 I received a Ph.D. in Structural/Earthquake Engineering from Stanford.

From 1933-1934, I was a Research Assistant in earthquake dynamics at Stanford.

I was employed as a Research Engineer with the U.S. Coast and Geodetic Survey, Seismological Division, San Francisco, California, from 1934-1935.

From 1935-1936, I was employed as an Engineer by the Division of Highways of the State of California.

I was employed as an Engineer with Standard Oil Company of California, San Francisco from 1936-1940.

I was Engineer in Charge of Design with H. J. Brunner, San Francisco, California from 1940-1945.

In 1945, I started my own consulting practice, John A. Blume, Structural Engineer. In 1957, my organization became John A. Blume & Associates, Engineers, a corporation, and I was President from 1957-1981. In 1971, we merged with URS Corporation for which company I serve as Director and Senior Engineer/Scientist.

TESTIMONY OF

OTTO W. NUTTLI, PH.D.

SOUTH CAROLINA ELECTRIC & GAS COMPANY

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD.

My name is Otto W. Nuttli. I am employed by St. Louis University as a Professor of Geophysics in the Department of Earth and Atmospheric Sciences. A statement of my professional qualifications and relevant experience is attached hereto. I have been retained as a consultant by the South Carolina Electric & Gas Company since August 1981, for the purpose of providing information about the magnitudes and depths of shallow earthquakes in the eastern United States. In my testimony I shall address this point and describe its relevance to seismic studies for the Virgil C. Summer Nuclear Station in South Carolina. The tables and figures included and the Appendices attached were prepared by me and constitute a part of my testimony.

Magnitudes and Depths of Eastern United States Earthquakes

Since the 1960s, the distribution of seismograph stations in the eastern United States (defined as the area east of the Rocky Mountains) has been adequate for seismologists accurately to calculate magnitudes and focal depths of many of the earthquakes that were large enough to be felt by humans. By so doing, seismologists also were able to quantify the relations expressing the attenuation of earthquake wave energy. From these studies they found that most of the eastern United States earthquakes characteristically had areas of perceptibility by humans much larger than

western United States earthquakes of the same magnitude or the same epicentral intensity. These "normal" or "tectonic" earthquakes had focal depths of 3 to 25 km. In addition, there were a few earthquakes that occurred in southwestern Illinois in August 1965 that had small magnitudes, relatively high epicentral intensities, and small areas of perceptibility (when compared to "normal" earthquakes of the same epicentral intensity). For example, an August 14, 1965 earthquake of 1.5 km focal depth had an epicentral intensity of VII and a radius of perceptibility of less than 25 km. (The earthquake was not felt in the city of Cape Girardeau, Missouri, only 25 km away.) The November 9, 1968 Illinois earthquake of 22 km depth and the July 28, 1980 Kentucky earthquake of 12 km depth both also had epicentral intensities of VII, but their radii of perceptibility were approximately 500 km each. The M_L values of the three earthquakes were 3.8, 5.5, and 5.3, respectively.^{1/} Theoretical studies show that these results can be explained by the fact that very shallow earthquakes (less than 3 km in depth) strongly excite fundamental-mode high-frequency surface waves that are quickly attenuated with distance (Herrmann and Nuttli, 1975a, b). For larger focal depths these particular waves are not strongly excited. Thus we

^{1/} Herrmann and Nuttli (1982) have shown in a recent study that the m_b of eastern United States earthquakes, which commonly is used for that region, is numerically equal to the M_L of western United States earthquakes. Therefore I shall use M_L in what follows, although in some seismological literature m_b usually is employed to give the magnitude of eastern United States earthquakes.

have a means of going back to the earthquake catalogs and sorting out the very shallow earthquakes from the "normal" earthquakes, by considering both their epicentral intensity and area of perceptibility.

A study of the catalogs contained in Earthquake History of the United States, published by the U.S. Geological Survey, shows that there were only five events in the eastern United States that had epicentral intensities of VII or greater and small areas of perceptibility. These are the 1891 east Texas event of $I_0 = VII$, the 1905 and 1906 Upper Michigan events of $I_0 = VIII$, the 1954 Pennsylvania event of $I_0 = VII$, and the previously mentioned 1965 Illinois event of $I_0 = VII$. For the three earthquakes of $I_0 = VII$, the radius of perceptibility was no greater than 25 km. The 1906 Michigan event had maximum intensity effects at the Atlantic Mine, and probably was related to the mining activity. The 1905 Michigan earthquake is discussed in Appendix A, in which it is shown that the intensity VIII effects were anomalous. The point to be made from these observations is that in the history of the eastern United States there has been no "very shallow" earthquake of epicentral intensity much greater than VII intensity VIII effects were anomalous. The point to be made from these observations is that in the history of the eastern United States there has been no "very shallow" earthquake of epicentral intensity much greater than VII (the $I = VIII$ effects of the 1906 Michigan event were damage within the

mine itself, right at the focus of this likely mining-related event, and of the 1905 Michigan event were at points directly above the mine workings). From seismographic data we know the M_L of the 1965 Illinois earthquake of $I_0 = VII$ was only 3.8. Thus, based on the long historic record of the eastern United States, all very shallow earthquakes can be expected to have an M_L no greater than 4. If the Monticello Reservoir could induce an earthquake of M_L greater than 4, the depth would be greater than 3 km, which would make it a "normal" depth earthquake.

Dr. Trifunac, in his comments to the Atomic Safety and Licensing Board (Docket No. 50-395-OL, September 15, 1981) concluded that ". . . the occurrence of $MMI = VII$ ($I_0 = VII$) at the site would be equivalent to an earthquake of magnitude 5 to 5.5 with epicenter at the site." These are exactly the kinds of epicentral intensity - magnitude pairs which were observed for the 1968 Illinois earthquake and the 1980 Kentucky earthquake, both of "normal" focal depth. Later in the report Dr. Trifunac noted ". . . as long as the largest reservoir-induced earthquake is less than intensity VII or even VIII the background seismicity still plays the dominant role in contributing to the URS amplitudes." (Trifunac Comments, p. 13.) I agree with these statements, and note from the arguments given above that any "very shallow" induced earthquake will not exceed the intensity value of VII in the eastern United States. If induced earthquakes can occur at Monticello Reservoir of M_L greater than 4, they will be of normal focal depth.

Another argument in support of the statement that eastern United States earthquakes of M_L greater than 4 do not occur at very shallow depths is the observation that no eastern United States earthquakes, except possibly the great 1811-1812 New Madrid earthquakes, have produced surface rupture. The arguments for surface rupture by the 1811-1812 earthquakes are indirect, based on contemporary reports that the Mississippi River temporarily reversed its flow, which could occur if the downstream side were up-faulted and formed a dam across the river.

As discussed above, no earthquake of depth less than 3 km has had an M_L greater than 4.0. As shown in Appendix B, these data on magnitudes and depths of very shallow eastern U.S. earthquakes can be used to obtain a relation between M_L and minimum focal depth.

In summary of this section, the historic and seismographic evidence indicates that all eastern United States earthquakes of M_L greater than 4 have occurred at depths exceeding 3 km. If it is possible to have a reservoir-induced earthquake near the V.C. Summer Nuclear Station in South Carolina of M_L greater than 4, then that earthquake will have a "normal" focal depth and its ground motion can be estimated in the same manner as is done for tectonic earthquakes. It is not permissible to scale up from the ground motion values of very shallow events, such as those

of August 27, 1978 or October 16, 1979 near the Virgil C. Summer Nuclear Station, to obtain acceleration values for an earthquake of M_L greater than 3 because such earthquakes will occur at deeper depths than those which have been observed.

Ground Acceleration Versus Magnitude Relations for Normal Depth Eastern United States Earthquakes

Attenuation curves for ground motion acceleration in the eastern United States differ in some respects from similar curves for the western United States because of differences in anelastic attenuation, in stress drop, and because the normal depth eastern United States earthquakes do not rupture the earth's surface. Figure 1 shows three of the most recent eastern United States acceleration attenuation curves, for $M_L = 5.0$ and 6.5 , as proposed by Campbell (1981), Nuttli (1979) and Nuttli and Herrmann (1981). The ordinate is the arithmetic average of the peaks on the two horizontal components. To obtain the larger of the two peak values of the horizontal components, the ordinate values should be multiplied by 1.14 (obtained from empirical studies). Note the small amount of difference between the three curves at distances beyond 15 km. Nuttli's (1979) curves were obtained by scaling up or down from the far-field data of the 1971 San Fernando, California earthquake, assuming a surface point source of waves and correcting for differences in anelastic attenuation. This assumption becomes invalid at the shorter

epicentral distances, such as approximately 20 km and less for $M_L = 5.0$ and 35 km and less for $M_L = 6.5$. Campbell's (1981) curves also are derived from a western United States data base, and show to a lesser extent the same problem at near distances, particularly for $M_L = 6.5$. The Nuttli and Herrmann (1981) curves attempted to take account of the "normal" focal depth of eastern United States earthquakes and the observed lack of surface rupture, and represent the most current research on the subject.

Figure 2 shows the Nuttli and Herrmann (1981) curves, along with all the presently available eastern United States strong-motion acceleration values for "normal" depth earthquakes. Note in particular that the data point for the $M_L = 4.3$ earthquake recorded at a distance of 9 km is consistent with the idea of the curves flattening out at the shorter distances. The one anomalously large acceleration value at 100 km distance for an $M_L = 5.0$ earthquake was obtained at the crest of a dam.

In summary of this section, the preferred acceleration attenuation curves for the eastern United States are those given in Figure 2. They indicate that an $M_L = 5.0$ earthquake at the site would produce a horizontal acceleration (arithmetic average of the two peak values) of about 0.11g, and an $M_L = 5.5$ earthquake at the site a horizontal acceleration of about 0.17g. If maximum sustained (exceeded by only two values) accelerations are desired, the numbers

above should be divided by 1.22, thus yielding effective values that would be more appropriate for design purposes. Actually the factor of 1.22 applies for distant recordings: for near-field motion a larger factor would be appropriate.

CONCLUSION

In summation it can be concluded that very shallow eastern United States earthquakes (those having depths less than 3 km) do not exceed magnitude 4. If a reservoir-induced event of magnitude greater than 4 were possible near the Virgil C. Summer Nuclear Station it would have a "normal" focal depth. Because of the relationship between magnitude and minimum focal depth (shown in Appendix B), very shallow events recorded at Monticello Reservoir such as the August 27, 1978 event or the October 16, 1979 event should not be scaled upward to obtain higher magnitude earthquakes, without accounting for the greater depths at which those earthquakes would occur. Higher magnitude, normal depth earthquakes in the eastern United States would follow the acceleration attenuation curves shown in Figure 2.

REFERENCES

Campbell, K.W., "A Ground Motion Model for the Central United States Based on Near-Source Acceleration Data," Earthquakes and Earthquake Engineering - Eastern United States, J.E. Beavers, editor, vol. 1, pp. 213-224, 1981, Ann Arbor Science Publishers, Inc.

Herrmann, R.B., and O.W. Nuttli, "Ground-Motion Modelling at Regional Distances for Earthquakes in a Continental Interior, I. Theory and Observations," International Journal of Earthquake Engineering and Structural Dynamics, vol. 4, pp. 49-58, 1975a.

_____, "Ground-Motion Modelling at Regional Distances for Earthquakes in a Continental Interior, II. Effect of Focal Depth, Azimuth and Attenuation," International Journal of Earthquake Engineering and Structural Dynamics, vol. 4, pp. 59-72, 1975b.

_____, "Magnitude: The Relation of M_L to m_{bLg} ," accepted for publication in the Bulletin of the Seismological Society of America, vol. 72, 1982.

Nuttli, O.W., "The Relation of Sustained Maximum Ground Acceleration and Velocity to Earthquake Intensity and Magnitude," State-of-the-Art for Assessing Earthquake Hazards in the United States, Miscellaneous

REFERENCES (Continued)

Paper S-73-1, Report 16, Waterways Experiment Station,
U.S. Army Engineers, Vicksburg, Mississippi, 1979,
100 pp.

Nuttli, O.W., and R.B. Herrmann, "Consequences of Earth-
quakes in the Mississippi Valley," preprint 81-519,
American Society of Civil Engineers Meeting, 14 pp.,
October, 1981.

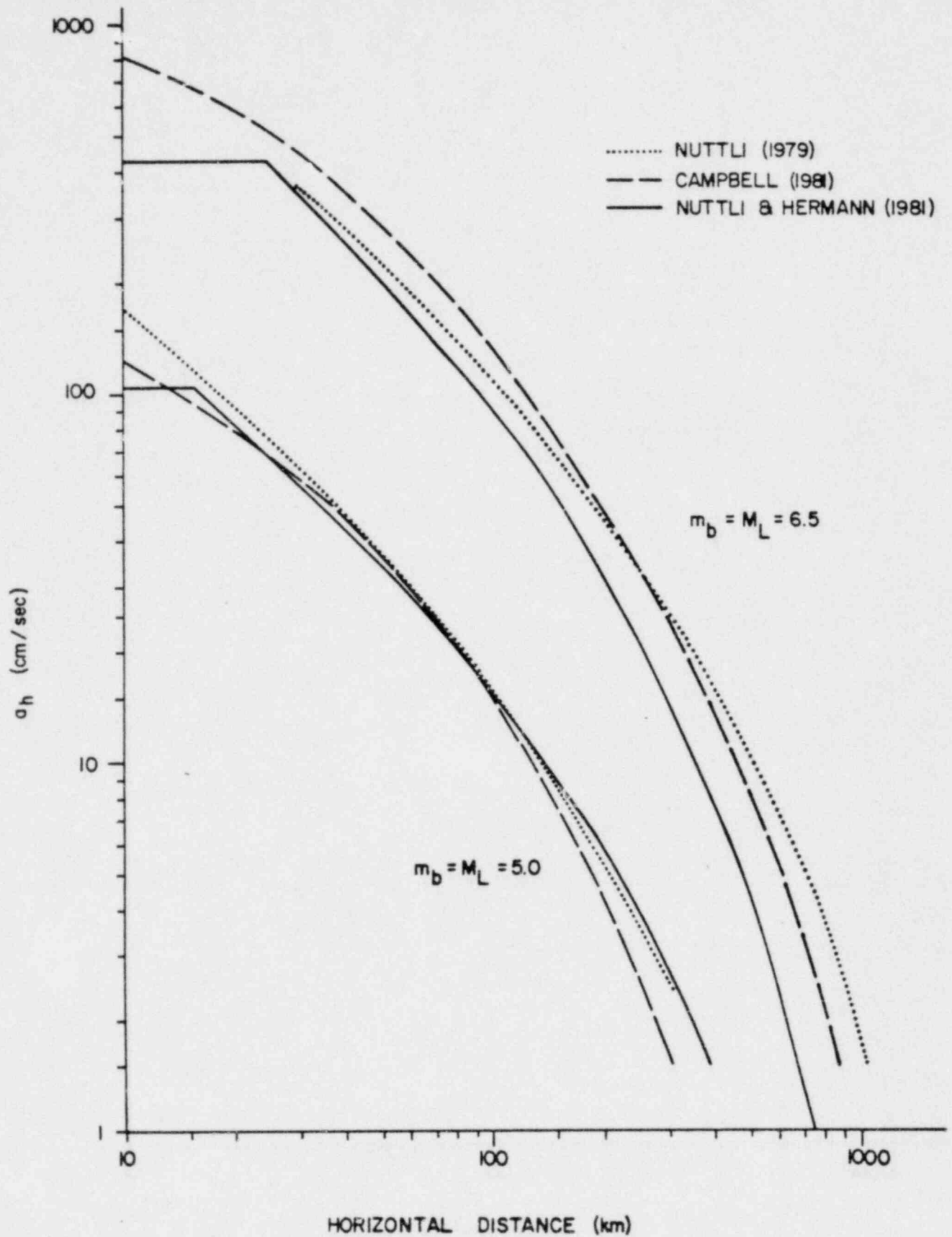


FIGURE 1 - ARITHMETIC AVERAGE OF PEAK ACCELERATIONS
ON TWO HORIZONTAL COMPONENTS
EASTERN UNITED STATES

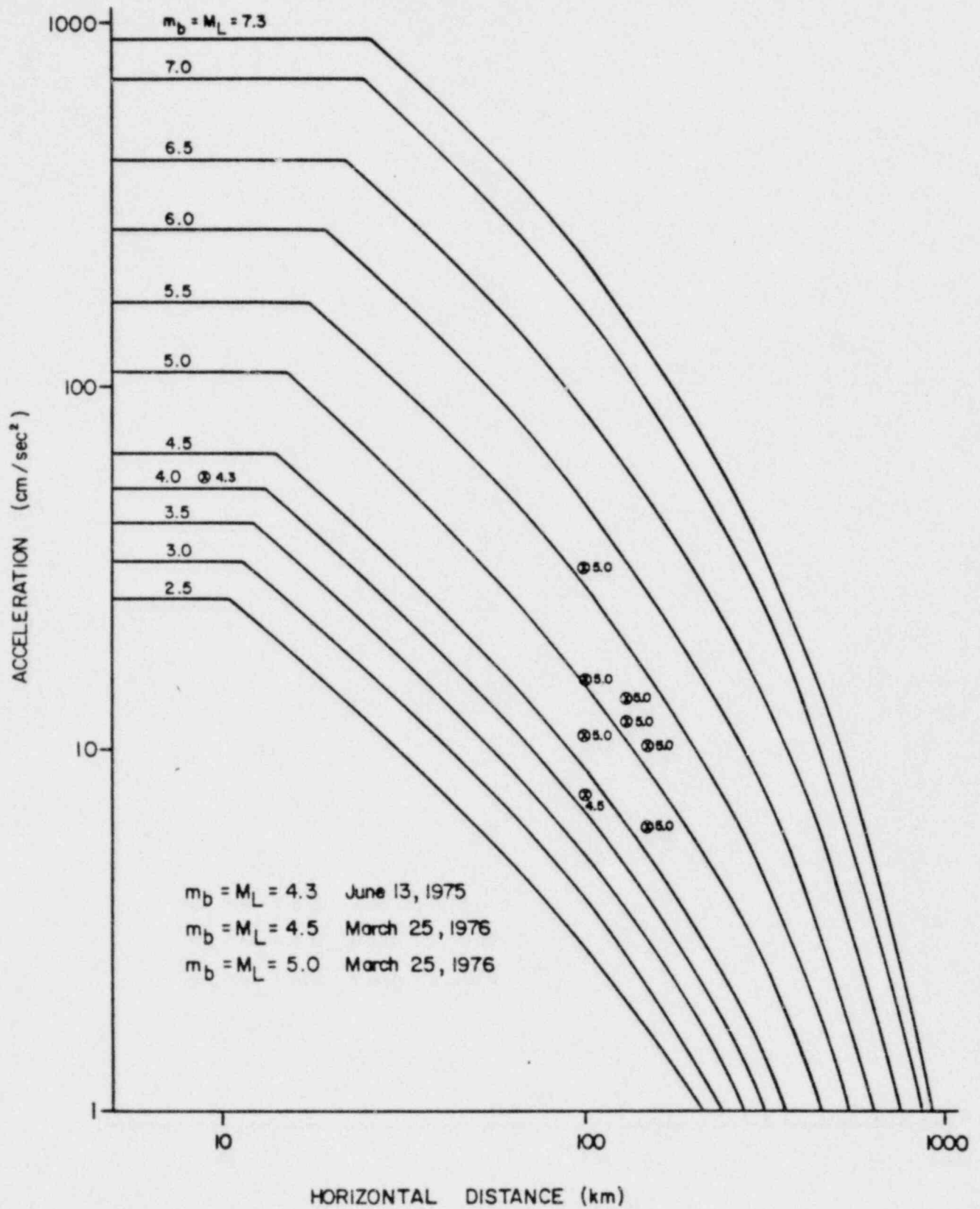


FIGURE 2 - ARITHMETIC AVERAGE OF PEAK ACCELERATIONS
 ON TWO HORIZONTAL COMPONENTS
 FOR NORMAL FOCAL DEPTH IN EASTERN UNITED STATES

APPENDIX A

EPICENTRAL INTENSITY, MAGNITUDE AND DEPTH OF THE
JULY 26, 1905 UPPER MICHIGAN EARTHQUAKE

The Upper Michigan earthquake had a reported epicentral intensity of VIII and a relatively small felt area of 17,000 km² (Frantti and Bacon, 1977, figs. 8 and 9). Thus the question may be asked whether it could have been a very shallow depth earthquake, one of the largest of that kind in the eastern United States. Also there is the question of the magnitude of the earthquake.

Figure A-1 shows a plot of the M.M. intensity as a function of epicentral distance, in the manner developed by Nuttli et al. (1979) to estimate body-wave magnitude m_b , from an isoseismal map. The range of distances, for a given intensity, corresponds to the interval between the semi-minor and semi-major axes of the isoseismal curves. Also shown in the figure is an attenuation curve, with somewhat extra weight given to the V, VI and VII isoseisms, the best determined ones. Beyond 4 km the ordinate varies as $r^{-5/6}$ (r is epicentral distance), a result expected on theoretical grounds for a distance range for which anelastic attenuation is not important.

For a reference earthquake of $m_b = 5.5$, the $(A/T)_z$ value at a distance of 10 km is 0.03 cm/sec (Nuttli et al. 1979). As seen from Fig. A-1, the $(A/T)_z$ value at a distance of $r = 10$ km for the 1905 earthquake was 0.0036 cm/sec. The quantity $\Delta\mu$, which is $\log_{10} (0.0036/0.03)$,

equals -0.92 . Thus $\mu = 5.5 - 0.92 = 4.58$. From Nuttli et al. (1979):

$$m_b = 1.44\mu - 2.63.$$

For the 1905 earthquake

$$m_b = (1.44 \times 4.58) - 2.63 = 4.0.$$

From Fig. A-1, the intensity attenuation curve is seen to depart from linearity at a distance of approximately 4 km. This bending of the curve at that distance results from a non-zero focal depth of about 2 to 3 km. If the earthquake were of very shallow focal depth, say 1 km, the curve would be expected to be linear back to almost 1 km epicentral distance.

The flattening of the attenuation curve in Fig. A-1 for distances less than 4 km, with an intensity value of VII at distances of 1 to 2 km, indicates that the appropriate epicentral intensity was VII. Frantti and Bacon (1977) noted that all the intensity VIII observations came from sites above underground mine workings, which likely tended to accentuate the ground shaking.

In summary, the July 26, 1905 earthquake had an m_b of 4.0, an epicentral intensity of VII (if the anomalous VIII values above the underground mine workings are disallowed) and a focal depth of about 2 to 3 km. Thus it should be classified as a shallow event, but of depth greater than 1 km, and on the basis of its magnitude and depth would not be considered atypical for the eastern United States.

REFERENCES

Frantti, G. E. and L. O. Bacon (1977). Seismicity and Tectonic Relationships for Upper Great Lakes Precambrian Shield Province, Quarterly Progress Report MTU/NRC 77-78-01, 1 May - 31 July 1977, Michigan Technological University.

Nuttli, O. W., G. A. Bollinger and D. W. Griffiths (1979). On the Relation between Modified Mercalli Intensity and Body-Wave Magnitude, Bulletin of the Seismological Society of America, vol. 59, 893-909.

JULY 26, 1905 MICHIGAN EARTHQUAKE

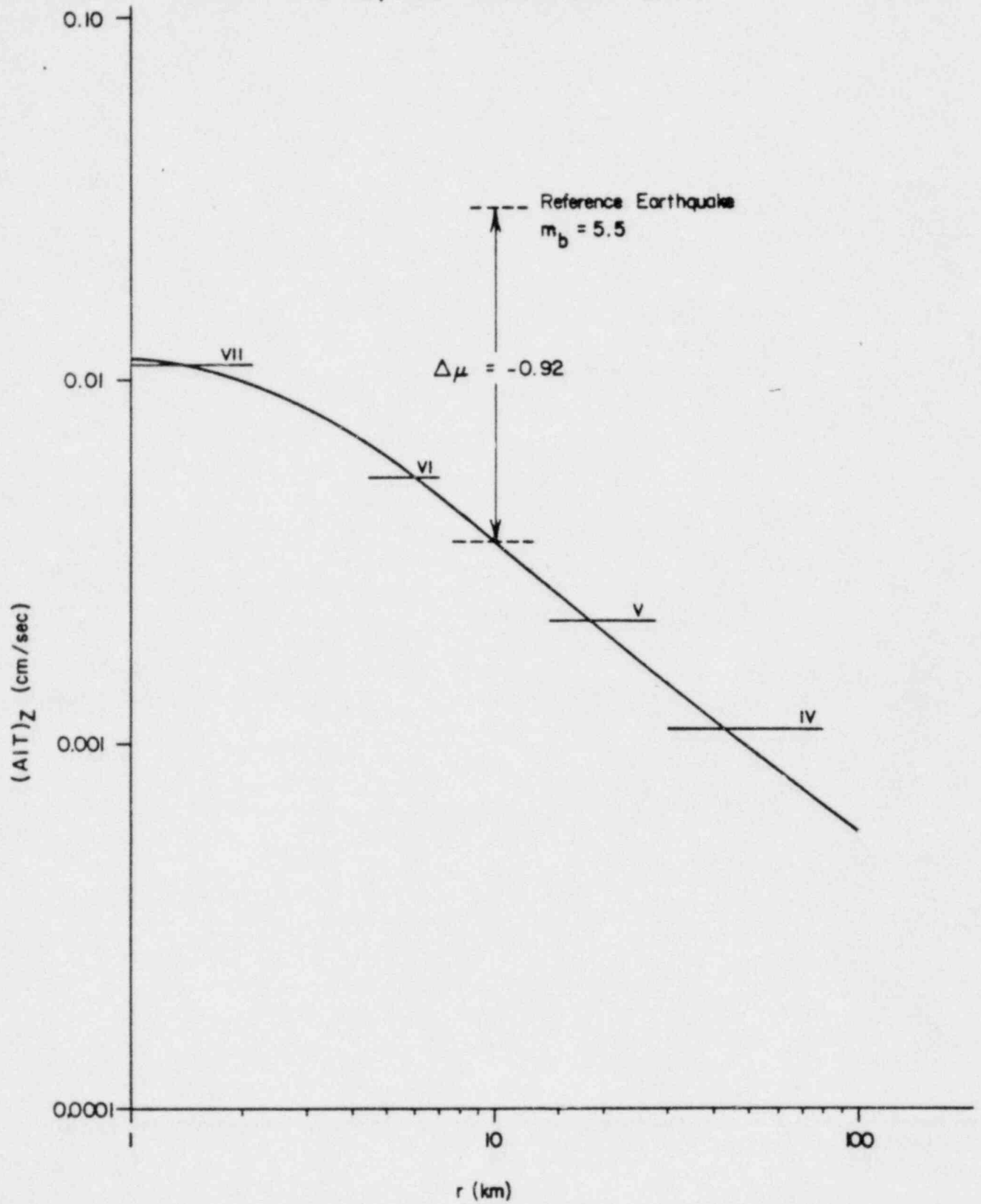


FIGURE A-1 m_b DETERMINATION FROM INTENSITY DISTRIBUTION

APPENDIX B

ON THE QUESTION OF MAGNITUDE VERSUS
DEPTH OF VERY SHALLOW EARTHQUAKES
IN THE EASTERN UNITED STATES

It has been pointed out that very shallow depth earthquakes can occur in the eastern United States, and that they have caused epicentral M.M. intensities as large as VII. They are characterized by relatively small felt areas and large epicentral intensities, when compared to normal depth earthquakes of the same magnitude. The best documented example is the 1965 southwestern Illinois earthquake ($M_L = 3.8$ and depth = 1.5 km), which had an epicentral intensity of VII. The Monticello, South Carolina accelerograms for the nearby earthquakes of August 27, 1978 ($M_L = 2.8$ and focal depth = 0.10 km) and of October 16, 1979 ($M_L = 2.8$ and focal depth = 0.07 km) showed peak accelerations of 0.26g and 0.35g, respectively. Inasmuch as the latter two earthquakes did not cause any damage, their epicentral intensity can be assumed to have been no greater than V.

Previously it was shown that the magnitudes of very shallow (depth less than 3 km) eastern United States earthquakes do not exceed 4. A logical question is: Is there a relation between minimum focal depth and magnitude for these very shallow events, e.g. can an $M_L = 4$ earthquake occur at a depth of 100 m?. The discussion that follows will attempt to provide an answer to this question.

The available data on the relation of minimum focal depth to magnitude are very limited. The best are given

above. In addition, we can use the deduction that the 1905 Michigan earthquake had an M_L of 4.0 and a focal depth of 2 to 3 km, and the observation that the 1979 Lake Jocassee, S.C. earthquake of $M_L = 3.7$ had a depth of 2 ± 1 km. All these data are plotted in Figure B-1.

On a semilogarithmic plot the data points can be fitted by a straight-line curve. This curve gives the minimum focal depth for very shallow earthquakes as a function of magnitude. For example, an earthquake of $M = 3.5$ would occur a depth no less than 0.6 km.

On the basis of the data presented in Figure B-1, we can conclude that the very shallow earthquakes of the eastern United States (those of depth less than 3 km) exhibit a relation between magnitude and minimum focal depth in the depth range of 70 meters to about 3 km. On a log-log plot this relation is described by a straight-line curve, with an M_L value of 2.8 at a depth of 70 meters and an M_L value of 4.0 at a depth of 2.3 km. This relation makes use of all existing data on very shallow eastern United States earthquakes since historic time, and thus is considered to be the best presently available.

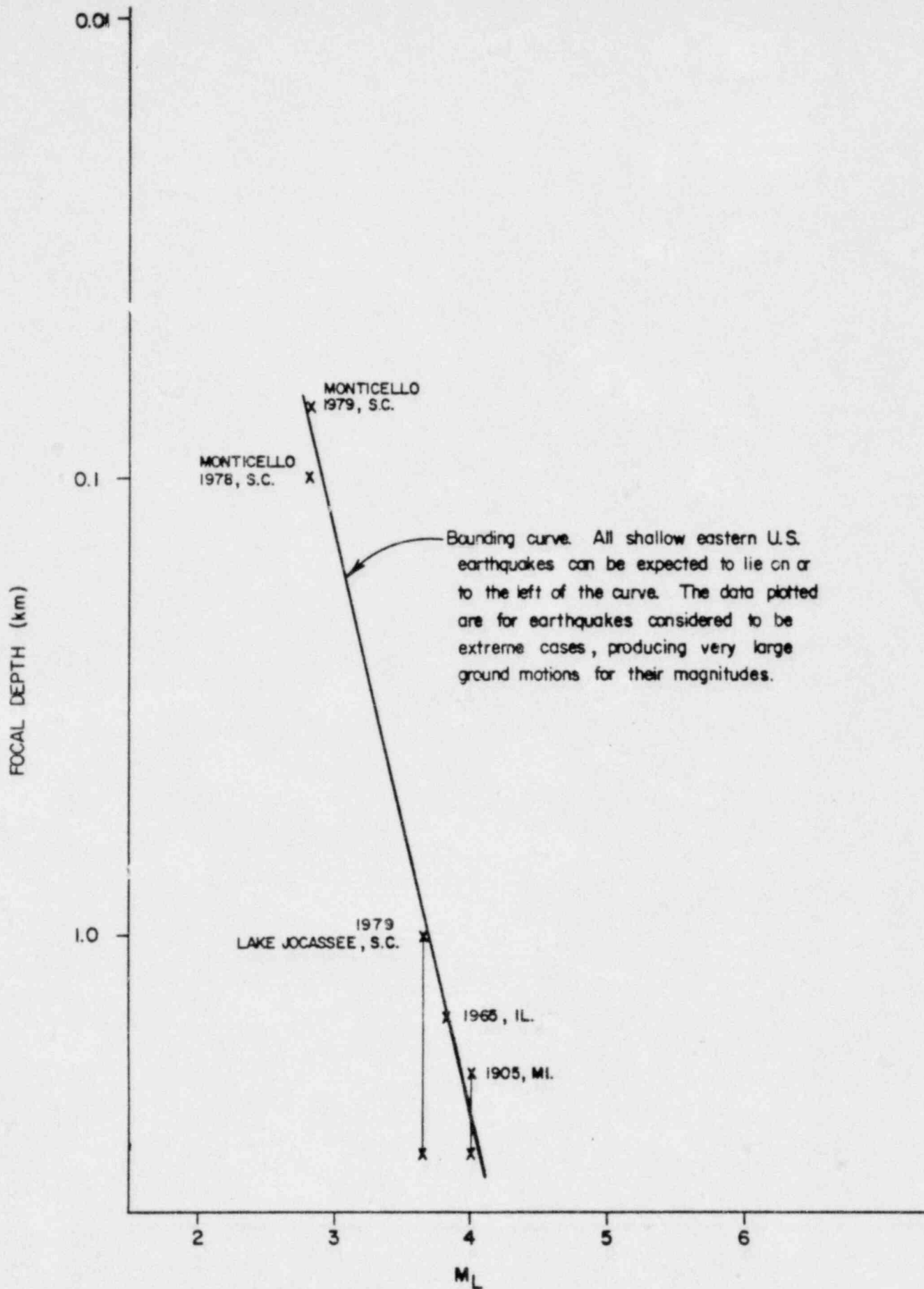


FIGURE B-1 MAGNITUDE VERSUS MINIMUM FOCAL DEPTH FOR FOR VERY SHALLOW EASTERN EARTHQUAKES

PROFESSIONAL QUALIFICATIONS

OTTO W. NUTTLI

My name is Otto W. Nuttli. I am employed by Saint Louis University as a Professor of Geophysics in the Department of Earth and Atmospheric Sciences, which position I have held since 1962. My first appointment as a faculty member of the University was in 1952. In my capacity as a Professor of Geophysics, I teach courses in seismology, classical geophysics and exploration geophysics, conduct and supervise research in seismology and consult in engineering seismology.

My university degrees were all earned at Saint Louis University: the B.S. in Petroleum Geophysics in 1948, the M.S. in Geophysics in 1950, and the Ph.D. with a major in Geophysics and minors in Mathematics and Engineering in 1953. Further educational experience was acquired at the University of Michigan at Ann Arbor, where I was a visiting research scientist in the summer of 1962, and at the University of California at Berkeley, where I was a visiting research seismologist during the summers of 1964 and 1967.

One of my principal research interests has been the quantification of earthquakes and of the ground motion resulting from them. The accomplishments have been published in the Bulletin of the Seismological Society of America, the Journal of Geophysical Research, other geophysical and engineering journals, and in a series of State-of-the-Art reports by the Waterways Experiment Station of

the U.S. Army Corps of Engineers. The accomplishments included the development of methods to determine magnitudes of eastern United States earthquakes, both from seismographic and non-instrumental data. Related research included the determination of attenuation of high-frequency earthquake waves, for the purpose of estimating strong ground motion, and estimation of maximum-magnitude earthquakes. Recent research includes the inter-relations between the various earthquake magnitude scales, and the estimation of fault rupture area and stress drop for various magnitude earthquakes. Another of my major research interests concerns the seismicity of the eastern United States, which recently culminated in the publication of a catalog of earthquakes for the central United States.

My membership in professional societies includes the Seismological Society of America, the American Geophysical Union (elected fellow), the Earthquake Engineering Research Institute, the Royal Astronomical Society of London, and the Society of Exploration Geophysicists. I was vice-president of the Seismological Society of America in 1975-1976, president in 1976-1977 and a member of the Board of Directors from 1974-1980. Since 1976, I have been a member of the Committee on Seismic Risk in the United States of the Earthquake Engineering Research Institute. I was editor of the Bulletin of the Seismological Society of America from 1971-1975 and an associate editor of the Journal of Geophysical Research from 1978-1980.

From 1975-1979, I was a member of the U.S. National Committee on Geology of the National Research Council - National Academy of Sciences. In 1976, I served on the Newmark-Stever Advisory Group on Earthquake Prediction and Hazard Mitigation of the National Science Foundation, established by President Gerald Ford to develop a national research effort on earthquake prediction and hazard mitigation. From 1976-1979, I was a member of the Committee on Seismology of the National Research Council - National Academy of the Sciences. Since 1978 I have been a seismology collaborator for the National Oceanic and Atmospheric Administration and the U.S. Geological Survey. I have also been chairman of the Technical Advisory Committee of the Earthquake Hazard Mitigation Panel of the State of Missouri since 1980.

In 1979, I was appointed to a panel of ten "experts" to provide estimates of seismicity and attenuation in the eastern United States for the research project "Seismic Hazard Analysis: Site Specific Response Spectra" by TERA Corporation and Lawrence Livermore National Laboratory for the U.S. Nuclear Regulatory Commission. Later that year I was the sole member of the first panel to be appointed to a panel of six to review the first study. In 1980 I was one of a panel of approximately twelve brought together by TERA Corporation and the Lawrence Livermore National Laboratory for the Nuclear Regulatory Commission to evaluate the

current state-of-the-art on the excitation and attenuation of strong ground motion produced by eastern United States earthquakes.

Most of my consulting work has been related to problems of engineering seismology, principally with estimating ground motion at sites of existing or planned nuclear power plants, dams, tall buildings and other critical structures. The organizations and companies for which I am or have been a consultant include Waterways Experiment Station of the U.S. Army Corps of Engineers, various district offices of the Corps of Engineers, Federal Emergency Management Agency, Federal Energy Regulatory Commission, National Science Foundation, Lawrence Livermore National Laboratory of the University of California, Bechtel, Dames & Moore, Ebasco, Ertec, Gillum & Colaco, Nuclear Fuel Services, TERA, Washington Public Power Supply System, Weston Geophysical and Woodward-Clyde.

RECENT PUBLICATIONS OF OTTO W. NUTTLI (since 1973)

- Magnitude and Ground Motion of some 19th Century Mississippi Valley Earthquakes, Earthquake Research in NOAA 1971-1972, edited by J. C. Stepp, Tech. Report ERL 256-ES L26, U. S. Dept. of Commerce, Boulder, CO, 77, 1973.
- Research in Seismology: Final Report (with C. Kisslinger), AFCRL-TR-73-0057, Air Force Cambridge Research Laboratories, Bedford, MA, 36 pp, 1973.
- Design Earthquakes for the Central United States, Misc. Report S-73-1, Report 1 of State-of-the-Art for Assessing Earthquake Hazards in the United States, U. S. Army Engineer Waterways Experiment Station, Vicksburg, MS, 55pp, 1973.
- Seismic Wave Attenuation and Magnitude Relations for Eastern North America, Jour. of Geophysical Research, 78, 876-885, 1973.
- The Mississippi Valley Earthquakes of 1811 and 1812: Intensities, Ground Motion and Magnitudes, Bull. of the Seismological Society of America, 63, 227-248, 1973.
- Seismic Hazard East of the Rocky Mountains, Meeting Preprint 2195, ASCE National Structural Engineering Meeting, Cincinnati, April 22-26, 22 pp, 1974.
- Earth Science in 1973: Seismology, Geotimes, 32-33, 1974.
- The Relation between Felt Area and Magnitude for Central United States Earthquakes, (with J. E. Zollweg), Bull. of the Seismological Society of America, 64, 73-85, 1974.
- Structure of the Shear Wave Low-Velocity Channel in the Western United States, (with T. Yasar), Geophysical Journal of the Royal Astronomical Society, 37, 353-364, 1974.
- Earthquake Magnitude Scales, (with S. J. Duda), Geophysical Surveys, 1, 429-458, 1974.
- Some Ground Motion and Intensity Relations for the Central United States (with A. Necioglu), International Journal of Earthquake Engineering and Structural Dynamics, 3, 111-119, 1975.
- Earthquake Mechanics in Central United States, (with R. L. Street and R. B. Herrmann), Science, 184, 1285-1287, 1974.
- Magnitude Recurrence Relations for Central Mississippi Valley Earthquakes, Bull. of the Seismological Society of America, 64, 1189-1207, 1974.
- The Mississippi Valley Earthquakes of 1811 and 1812, Earthquake Information Bulletin, U. S. Geological Survey, 6, 8-13, 1974.
- Research in Seismology: Earthquake Magnitudes, (with S. G. Kim), Scientific Report No. 1, AFCRL-TR-74-0326, Air Force Cambridge Research Laboratories, Hanscom AFB, MA, 28 pp, 1974.
- Ground-Motion Modelling at Regional Distances for Earthquakes in a Continental Interior, I. Theory and Observations, (with R. B. Herrmann), International Journal of Earthquake Engineering and Structural Dynamics, 4, 49-58, 1975.
- Ground-Motion Modelling at Regional Distances for Earthquakes in a Continental Interior, II. Effect of Focal Depth, Azimuth and Attenuation, (with R. B. Herrmann), International Journal of Earthquake Engineering and Structural Dynamics, 4, 59-72, 1975.
- Surface Wave Magnitudes of Eurasian Earthquakes and Explosions, (with S. G. Kim), Bull. of the Seismological Society of America, 65, 693-709, 1975.
- Spectral Characteristics of the Lg Wave Generated by Central United States Earthquakes, (with R. L. Street and R. B. Herrmann), Geophysical Journal of the Royal Astronomical Society, 41, 51-63, 1975.

- Research in Seismology: Earthquake Magnitudes, (with S. G. Kim and H. Y. Wen), Scientific Report No. 3, AFCRL-TR-75-0433, Air Force Cambridge Research Laboratories, Hanscom AFB, MA, 47 pp, 1975.
- Comments on "Seismic Intensities, Size of Earthquakes and Related Parameters" by Jack F. Evernden, Bull. of the Seismological Society of America, 66, 331-338, 1976.
- Seismicity of the Mississippi Embayment, Proceedings of the Advisory Committee on Reactor Safeguards, Seismic Activity Subcommittee Meeting in Los Angeles, March 22-23, 1976.
- Spatial Attenuation of Intensities for Central U. S. Earthquakes, (with I. N. Gupta), Bull. of the Seismological Society of America, 66, 743-751, 1976.
- On the Specification of a Design Earthquake, Proceedings of the International Symposium on Earthquake Structural Engineering, 1, St. Louis, MO, 1-19, 1976.
- Spectral and Magnitude Characteristics of Anomalous Eurasian Earthquakes, (with S. G. Kim), Bull. of the Seismological Society of America, 67, 463-478, 1977.
- A Time-Domain Study of the Attenuation of 10-Hz Waves in the New Madrid Seismic Zone, Bull. of the Seismological Society of America, 68, 343-355, 1978.
- Nomenclature and Terminology in Seismology, Editor's Handbook 3, Association of Earth Science Editors, 10 pp, (with B. A. Bolt, E. A. Flinn and J. C. Savage), 1978.
- Attenuation of High-Frequency Seismic Waves in the Central Mississippi Valley, Report 10, State-of-the-Art for Assessing Earthquake Hazards in the United States, Miscellaneous Paper S-73-1, U. S. Army Engineer Waterways Experiment Station, Vicksburg, MS, 75 pp, (with J. J. Dwyer), 1978.
- Credible Earthquakes for the Central United States, (with R. B. Herrmann), Report 12, State-of-the-Art for Assessing Earthquake Hazards in the United States, Miscellaneous Paper S-73-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS, 100 pp, 1978.
- Research in Seismology, Final Report, 1 March 1973-30 September 1978, (with B. J. Mitchell), Defense Advanced Research Projects Agency, Washington, D.C., 62 pp, 1978.
- Archeoseismology Applied to the New Madrid Earthquakes of 1811-1812, (with R. B. Herrmann and S. H. Cheng), Bull. Seismological Society of America, 68, 1751-1759, 1978.
- Research in Seismology, Semi-Annual Technical Report No. 1, 1 October 1978-31 March 1979, (with B. J. Mitchell), Defense Advanced Research Projects Agency, Washington, D.C., 82 pp, 1979.
- On the Relation between Modified Mercalli Intensity and Body-Wave Magnitude (with G. A. Bollinger and D. W. Griffiths), Bull. Seismological Society of America, 69, 893-909, 1979.
- Seismicity of the Central United States, (in) Hatheway, A. W., and C. R. McClure, Jr., (eds.), Geology in the Siting of Nuclear Power Plants, Reviews in Engineering Geology, 4, Geological Society of America, 67-93, 1979.
- A Statistical Method for Determining Attenuation of High-Frequency Seismic Waves (with J. J. Dwyer), Final Report, U. S. Geological Survey, Golden, CO, 115 pp, 1979.
- The Body-Wave Magnitude of the Great 1857 California Earthquake, Bull. of the Seismological Society of America, 69, 2117-2119, 1979.

- Earthquake Risk and Damage Functions: An Integrated Preparedness and Planning Model Applied to New Madrid, (with B. C. Liu, C. T. Hsieh, R. Gustafson, R. Gentile and W. Park), Final Report to National Science Foundation, Washington, D.C., 320 pp, 1979. (Also published as a hardcover book by Fairfield Press, Boulder, CO, 1981).
- The Relation of Sustained Maximum Ground Acceleration and Velocity to Earthquake Intensity and Magnitude, Report 16, State-of-the-Art for Assessing Earthquake Hazards in the United States, Miscellaneous Paper S-73-1, U. S. Army Engineer Waterways Experiment Station, Vicksburg, MS, 100 pp, 1979.
- Designing for Earthquakes, Chapter 2 of Nature of Earthquakes, American Institute of Architects Research Corporation, New York, 23-38, 1979.
- Research in Seismology, Semi-Annual Technical Report No. 2, 1 April 1979-30 September 1979, Defense Advanced Research Projects Agency, (with B. J. Mitchell), ARPA Order No. 3291-21, 96 pp, 1979.
- Strong-Motion Studies in the Central United States, (with R. B. Herrmann, C. Y. Wang, M. Goertz and E. J. Haug), Lifeline Earthquake Engineering - Buried Pipelines, Seismic Risk, and Instrumentation, American Society of Mechanical Engineers, PVP-34, 279-285, 1979.
- Research in Seismology, Semi-Annual Technical Report No. 3, 1 October 1979-31 March 1980, (with B. J. Mitchell), Defense Advanced Research Projects Agency, ARPA Order No. 3291-21, 88 pp, 1980.
- The Excitation and Attenuation of Seismic Crustal Phases in Iran, Bull. of the Seismological Society of America, 70, 469-485, 1980.
- Strong Motion Investigations in the Central United States, (with R. B. Herrmann), Proceedings of the Seventh World Conference on Earthquake Engineering, Geoscience Aspects, Part II, Istanbul, Turkey, 533-536, September 1980.
- Research in Seismology, Semi-Annual Technical Report No. 4, (with B. J. Mitchell), 1 April 1980-30 September 1980, Defense Advanced Research Projects Agency, ARPA Order No. 3291-21, 95 pp, 1980.
- Seismic Hazard Associated with the New Madrid Faulted Zone, Technical Papers, Fiftieth Annual International Meeting of the Society of Exploration Geophysicists, Houston, TX, 6, 3469-3484, 1980.
- Evaluation of Past Studies and Identification of Needed Studies of the Effects of Major Earthquakes Occurring in the New Madrid Fault Zone, Report Submitted to Federal Emergency Management Agency, Region VII, Kansas City, MO, January 21, 1981, 62 pp.
- New Madrid Earthquakes and St. Louis, Discovery/81, 1, no. 3, 2-3, 1981.
- Numerical Study of Attenuation of High Frequency Lg-Waves in the New Madrid Seismic Region, (with J. J. Dwyer and R. B. Herrmann), U. S. Geological Survey Open-File Report 81-112, 131 pp, 1981.
- On the Attenuation of Lg Waves in Western and Central Asia and their Use as a Discriminant between Earthquakes and Explosions, Bull. Seismological Society of America, 71, 249-261, 1981.
- The Earthquake Problem in the Eastern United States, Preprint 81-050, American Society of Civil Engineers, New York City Meeting, 16 pp, May 1981.
- Earthquake, McGraw-Hill Yearbook of Science and Technology 1981, McGraw-Hill Book Co., New York, 157-159, 1981.

On the Problem of the Maximum Magnitude of Earthquakes, Proceedings of Conference XIII, Evaluation of Regional Seismic Hazard and Risk, U.S. Geological Survey Open-File Report 81-437, 111-123, 1981.

An Approach to Seismic Zonation for Siting Nuclear Electric Power Generating Facilities in the Eastern United States, (with N. L. Barstow, K. G. Brill, Jr., and P. W. Pomeroy), U.S. Nuclear Regulatory Commission NUREG/CR-1577, Washington, D.C., 315 pp, 1981.

TESTIMONY OF

ROBIN K. MCGUIRE, PH.D.

SOUTH CAROLINA ELECTRIC & GAS COMPANY

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

My name is Robin K. McGuire. I am Director of Decision Analysis at Ertec Rocky Mountain, Inc. A statement of my qualifications and relevant experience was submitted to the Board at the previous evidentiary hearing on June 23, 1981. My testimony consists of eight reports: 1) "Summary - Effect of Reservoir-Induced Seismicity on Virgil C. Summer Nuclear Station," 2) RM-1, "Response Spectra Shapes for Reservoir-Induced Seismicity at Virgil C. Summer Nuclear Station," 3) RM-3*, "Mathematical Model Used to Estimate Peak Acceleration at Virgil C. Summer Nuclear Station," 4) RM-4, "Probability Estimates of Seismicity and Ground Motion at Virgil C. Summer Nuclear Station," 5) RM-5, "Processing and Analysis of Accelerograms from Aftershocks of the 1975 Oroville, California Earthquake," 6) RM-6, "Estimates of Peak Acceleration Using Brune Seismic Source Model," 7) "Applicant Evaluation of Joyner-Fletcher Report on Virgil C. Summer Nuclear Station Seismicity Studies," and 8) "Applicant Evaluation of Luco Report on Virgil C. Summer Nuclear Station Seismicity Studies." All figures and tables and appendices included in these reports were prepared by me and constitute part of my testimony.

*Report RM-2 on Jenkinsville accelerograph data has been incorporated in Applicants' response to FSAR Question 361.23.

RESPONSE SPECTRA SHAPES
FOR RESERVOIR-INDUCED SEISMICITY
AT VIRGIL C. SUMMER NUCLEAR STATION

Robin K. McGuire

The purpose of this report is to clarify and document the method and justification for shapes of response spectra used by the Applicant to represent vibratory ground motion from reservoir-induced earthquakes at Monticello Reservoir. Although parts of this documentation have been presented elsewhere (e.g. in the Applicant's response to Luco's comments), they are repeated here in order to provide a convenient reference in one document for the methodology used to derive response spectrum shapes. These spectra were derived following requirements indicated in Reg. Guide 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants." Specifically, Reg. Guide 1.60 states that the standard design response spectrum procedure "...does not apply to sites which (1) are relatively close to the epicenter of an expected earthquake or (2) which have physical characteristics that could significantly affect the spectral combination of input motion. The Design Response Spectra for such sites should be developed on a case-by-case basis." The Virgil C. Summer Nuclear Station would be close to the epicenter of any reservoir-induced seismicity of concern; hence site-specific response spectra were developed to represent ground motion for these events.

This procedure consisted of using response spectrum shapes for earthquake ground motions recorded at magnitudes, distances, and site conditions representative of reservoir-induced earthquakes at the Virgil C. Summer facility. These response spectrum shapes, for magnitudes in the range of interest, were then compared to other available data to ensure their applicability.

The shapes for these spectra were taken from the publication of Johnson and Traubenik (1978). These spectral shapes represent ground motions based on records obtained on rock sites for earthquakes with magnitudes (M_L) between 4.7 and 6.5, with source-to-site distances of less than 20 kilometers. The derived spectra for 5 percent damping for $M_L = 4.0, 4.5,$ and 5.3 events scaled to 0.15 g peak acceleration are labeled "RIS" in Figure 1. These are mean + σ spectra, based on the amplification factors reported by Johnson and Traubenik (1978). Use of the mean + σ spectrum is consistent with the procedure defined as acceptable for standard design response spectra in Reg. Guide 1.60.

Also shown in Figure 1 are the Reg. Guide 1.60 spectrum for 5 percent damping, and the Virgil C. Summer Nuclear Station SSE spectrum for 5 percent damping, both scaled to 0.15 g acceleration (the SSE acceleration at the facility) to permit meaningful comparison of spectral shapes. It is apparent that the derived RIS spectra generally match both the Virgil C. Summer spectrum and the RG 1.60 spectrum

at the highest frequencies, but deviate at intermediate and low frequencies, the extent depending on both the earthquake magnitude and the frequency of interest. The reason for this deviation is that broad-banded design spectra typically represent ground motions for earthquakes of magnitude around 6.5 (they are derived from recorded ground motions during seismic events with an average magnitude of 6.5). The RIS spectra, on the other hand, logically reflect the relative lack of intermediate and low frequency energy which will be generated during magnitude 4.0 to 5.3 earthquakes with small source to site distances.

Two steps are required to generate site-specific spectra of the type shown in Figure 1, and in comparing these spectra to other results available it is convenient to break the comparison into these two steps. The first step is the estimation of a peak velocity and a peak displacement which are consistent with the peak acceleration of the earthquake of interest. In the present application, the peak velocity-to-acceleration ratio is most critical because it determines the upper corner frequency of the spectrum. The peak displacement is not important in the present application because the Virgil C. Summer SSE spectrum greatly exceeds the RIS spectra at low frequencies (in the displacement - controlled region of the spectrum).

To evaluate the peak velocities derived by Johnson and Traubenik (1978), we compare them to results derived from

other studies. Table 1 summarizes values of peak velocity appropriate for 1g peak acceleration as obtained for RIS spectra and as derived from results reported by other investigators. Values for rock sites indicate that a peak velocity of about 50 cm/sec for magnitudes around 5.0 is appropriate, as used to characterize RIS spectra. Values for soil sites are generally higher, as shown in the lower half of Table 1; these are included here for completeness and to explain results which might be extracted from the literature. To be consistent with the work of Newmark (1973) and Blume et al (1973), on which RG 1.60 is based, the values shown for peak velocity are mean values, rather than mean + σ or some other values.

The second step in estimating response spectra is to determine amplification factors for the various frequency ranges. These are ratios of spectral response to ground motion parameters. For example, in the high frequency range, one is interested in the ratio of spectral acceleration to peak ground acceleration; at intermediate frequencies one is interested in the ratio of spectral velocity to peak ground velocity.

Table 2 compares spectral amplification factors for the acceleration and velocity ranges as recommended in RG 1.60, as derived for the RIS spectra shown in Figure 1, and as recommended by Newmark and Hall (1969). (As discussed above, the displacement - controlled frequency range is not

of particular concern for reservoir-induced earthquakes because of the large degree of conservatism inherent in the design spectrum at lower frequencies.) The RG 1.60 spectra in Table 2 are mean + σ results; the RIS results are also mean + σ amplifications. The velocity amplifications shown in Table 2 for RG 1.60 were calculated from acceleration and displacement amplifications at the indicated frequencies, using an assumed peak velocity of 48 inches per second for 1 g acceleration based on the work of Newmark (1973) and Blume et al (1973) on which RG 1.60 is based.

The RIS spectral amplifications shown in Table 2 indicate that the representation of reservoir-induced earthquake ground motions is appropriate. These amplifications generally agree with those of RG 1.60, particularly for the higher dampings (5 percent to 10 percent) characterizing structures critical for safe shutdown of the facility, and particularly for the higher frequencies in each range. The Johnson and Traubenik (1978) results on which the RIS amplifications are based were derived for events recorded specifically on rock sites at small source-to-site distances, whereas the RG 1.60 results were obtained from a variety of sites and source-to-site distances. Thus, it would be logical if the uncertainty in spectral amplification would be less for the RIS spectra than for the RG 1.60 spectra; this would result in lower mean + σ spectral amplification for the RIS spectra. The results for 0.5 percent damping undoubtedly reflect this difference in the

data sets. This difference in no way detracts from the RIS spectra results: the site and distance conditions of the records on which these results are based more closely reflect the conditions expected during reservoir-induced earthquakes at Monticello Reservoir than do generic broad-banded spectra.

In summary, the spectra developed to represent reservoir-induced earthquake ground motions are based on records obtained at close distances on rock sites. These spectra are consistent with published work of other investigators; they represent mean + σ spectra and have been developed to meet the requirements of Regulatory Guide 1.60.

TABLE 1
Peak Velocity for 1 g Peak Acceleration

Site Conditions	Author	Magnitude	Distance	Peak Velocity, cm/sec			
	This Study, RIS Spectra	4.0	near-field	13			
		4.5	near-field	26			
		5.3	near-field	51			
ROCK	Joynes et al (1981)	5.0	R ≈ 0	50			
	USGS OF 81-365						
	Trifunac and Brady (1976)	5.0-5.9	All	57			
	Joynes et al (1981) USGS OF 81-365	5.0	R ≈ 0	73			
					Boore et al (1978) USGS Circ. 795	5 km	90 (small struc.) 95 (all struc.)
						10 km	80 (small struc.) 70 (all struc.)
SOIL	Trifunac and Prady (1976)	5.0-5.9	All	92			
	Page et al (1972) USGS Circ. 672	5.5	3-5 km	110			

TABLE 2

Comparison of Spectral Amplification Factors

	Damping	Regulatory Guide 1.60	Newmark & Hall (1969)	PIS Spectra
Acceleration Amplification	0.5%	4.96 at 9 hz 5.95 at 2.5 hz	5.8	4.5
	5%	2.61 at 9 hz 3.13 at 2.5 hz	2.6	2.6
	10%	1.90 at 9 hz 2.28 at 2.5 hz	1.5	2.0
Velocity Amplification	0.5%	3.05 at 2.5 hz 3.77 at 0.25 hz	3.6	2.3
	5%	1.60 at 2.5 hz 2.42 at 0.25 hz	1.9	1.6
	10%	1.17 at 2.5 hz 2.00 at 0.25 hz	1.3	1.3

REFERENCES

- Boore, D.M., W.D. Joyner, A.A. Oliver, and R.A. Page (1978), "Estimation of Ground Motion Parameters", U.S.G.S. Circular 795, 43 pp.
- Johnson, J.A., and M.L. Traubenik (1978), "Magnitude - Dependent Near Source Ground Motion Spectra", Proc., A.S.C.E. Conf. on Earthquake Eng. and Soil Dynamics, Pasadena, pp. 530-539.
- Joyner, W.B., D.M. Boore, and R.L. Porcella (1981), "Peak Horizontal Acceleration and Velocity from Strong-Motion Records Including Records from the 1979 Imperial Valley, California, Earthquake", U.S.G.S. Open-File Rept. 81-365, March, 46 pp.
- Newmark, N.M., Consulting Engineering Services (1973), "A Study of Vertical and Horizontal Earthquake Spectra", Urbana, Illinois, USAEC Contract No. AT (49-5)-2667, WASH-1255, April.
- Newmark, N.M., and W.J. Hall, (1969). "Seismic Design Criteria for Nuclear Reactor Facilities," Proc., 4th World Conf. on Earthquake Eng., Santiago.
- Page, R.A., D.M. Boore, W.B. Joyner, and H.W. Coulter (1972), "Ground Motion Values for Use in the Seismic Design of the Trans-Alaska Pipeline System", U.S.G.S. Circular G72, 23 pp.

REFERENCES (Continued)

Trifunac, M.D., and A.G. Brady (1976), "Correlation of Peak Acceleration, Velocity, and Displacement with Earthquake Magnitude, Distance, and Site Conditions", Earthquake Eng. and Struc. Dyn., Vol. 4, pp. 455-471.

Blume, J.A., R.L. Sharpe, and J.S. Dalal (1973), "Recommendations for Shape of Earthquake Response Spectra," J.A. Blume & Assoc., San Francisco, USAEC Contract No. AT(49-5)-3011, February.

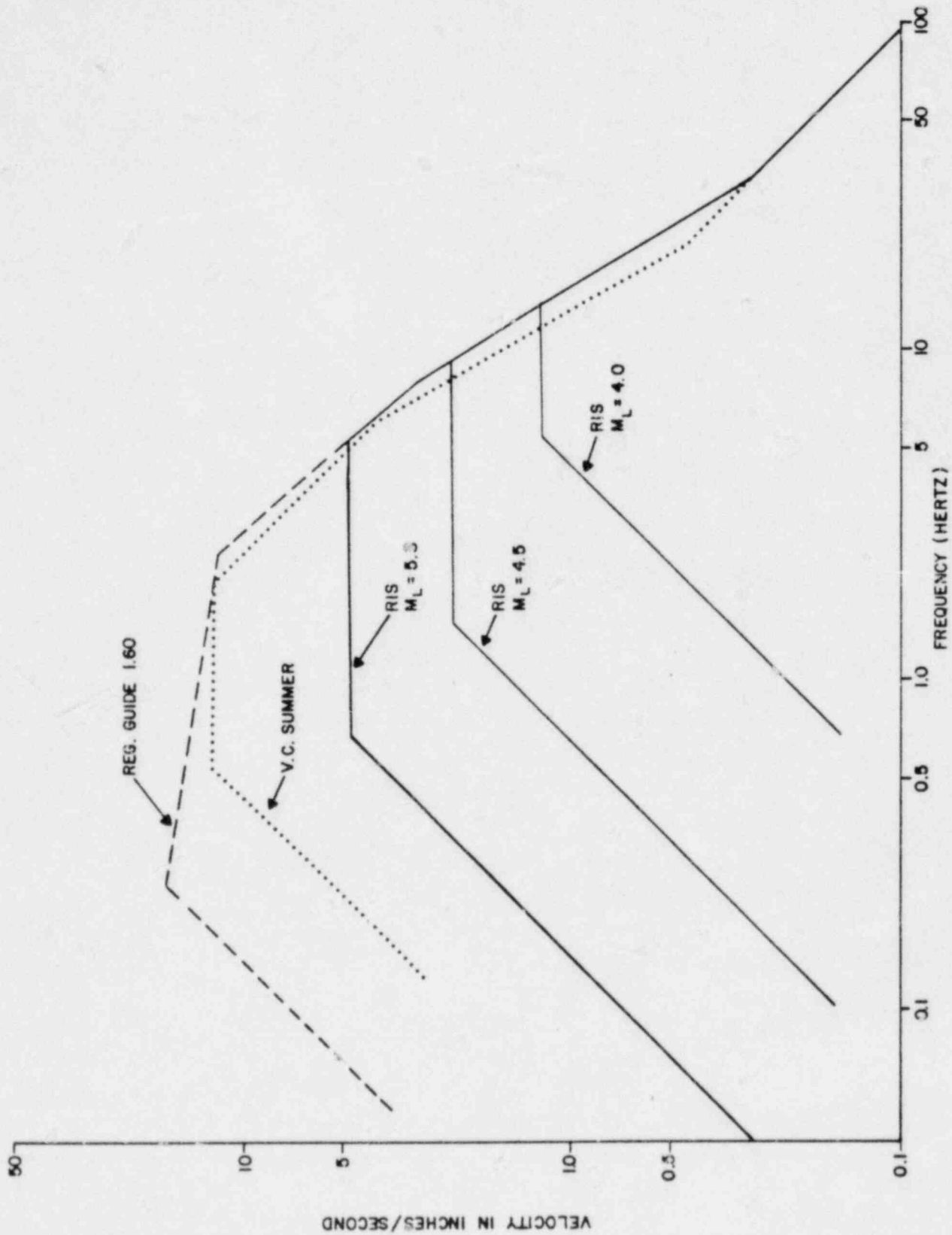


FIGURE I V.C. SUMMER, REG. GUIDE 1.60, and RIS SPECTRA: 5% DAMPING

MATHEMATICAL MODEL USED TO ESTIMATE
PEAK ACCELERATION AT
VIRGIL C. SUMMER NUCLEAR STATION

Robin K. McGuire

The Applicant has used a mathematical methodology calibrated with available data to estimate ground motions to be expected during reservoir-induced earthquakes at Monticello Reservoir. The purpose of the present document is to summarize the method used, for the convenience of interested parties, and to elaborate certain details regarding the justification of that method. Contained herein are comparisons with only one strong motion record from Monticello (that from the August 27, 1978 event) without accounting for any site amplification effects, because the purpose of this document is to elaborate the methodology used. Comparisons and data from a larger suite of Monticello earthquake records is presented in other documentation.

The major problem in estimating ground motions for such earthquakes is the lack of numerous empirical data on which to base estimates. In this sense, the fact that Monticello Reservoir has generated no damaging earthquakes and very few which have even been felt, ironically leads to questions and uncertainties in interpretation and prediction of seismic ground motion.

The strong motion data which are available at Monticello, in California, and elsewhere in the world, must be interpreted carefully to ascertain their potential applicability to the site. In this context it is useful to review the seismic events which might occur at Monticello. These may be divided into two categories: the first consists of very shallow events (depth <2 km). Observed seismicity at Monticello has been of this type. The second category consists of deeper events which have not been observed but which have been hypothesized.

For the magnitude range of interest ($M_L = 4$ to 5), there are a substantial number of strong motion records available from California and elsewhere. The majority of the California strong motion records which are readily available for analysis were obtained during events with 5 to 10 km depth. The Oroville earthquake aftershock sequence is a good example. Earthquakes at these depths can be characterized by stress drops on the order of 100 bars (Hanks and McGuire, 1981) for the purposes of estimating strong ground motion. At shallower depths, lower stress drops characterize the energy release; this has been documented for the Oroville aftershocks by Fletcher (1980) using digital seismographs and stress drops estimated in the frequency domain, rather than strong motion records. Thus the available data from California might be considered appropriate for estimating events in the second category

described above (they have an appropriate depth and, perhaps, an appropriate stress drop), but they are not for the first.

To estimate ground motions for very shallow, low stress drop events, several methods are possible. The available data for such methods in 1980, the time this methodology was first applied, consisted of one strong motion record for which the causative event has been identified, and several records which were questionably associated with other events (the earthquakes were of such a nature that the records were not suspected or discovered until routine maintenance of the instrument after a period of several weeks).

The first method might be to scale ground motions (specifically, peak accelerations) using the one unambiguous record available in 1980 (for the August 27, 1978, $M_L = 2.8$ earthquake). Scaling relations are available (for example from California data) to adjust the recorded peak accelerations for different magnitudes and for distances. This method is inadvisable because (1) it is based on a single earthquake record, (2) it gives no insight on how expected ground motions will vary with changes in source properties (seismic moment, stress drop), instrument characteristics, and record processing techniques (filtering and digitization), and (3) it allows no evaluation to be made of the distances within which ground motions will "saturate" because of geometric effects.

An alternative would be to develop a completely theoretical model of earthquake ground motions and use this for estimation in the magnitude and distance range of interest. This would relieve the second two objections described above, but would remain completely theoretical; it would have no tie to the few observations which are available for shallow events at close distances.

A third methodology, the best available and the one used by the Applicant, is to adopt a theoretical model which has been shown to appropriately estimate ground motions in other areas, and to select parameter values as input to that model using the observations available at Monticello Reservoir. All the observations can be used by backfiguring the parameter values required to explain these observations, then selecting parameter values for estimation of future earthquake ground motions by taking into account any ambiguity in the observations. The effect of changes in source parameters on estimated ground motions can easily be determined, as can changes in instrument characteristics and record processing procedures. Thus the assumptions used in the analysis are apparent and can be evaluated and argued individually on their merits.

The Applicant has employed a model which estimates seismic shear wave characteristics using the work of Brune (1970, 1971). This methodology yields predictions of root-mean-square acceleration; a simple result from random

vibration theory allows the peak acceleration to be calculated. The parameters necessary for application of the model are the seismic stress drop, the earthquake size (its seismic moment or, equivalently, the dimension of the rupture surface), the density, shear wave velocity, and Q (specific attenuation) of the medium, and the limiting frequencies which are of interest. Of these, the only real debate thus far has surrounded the appropriate value of stress drop.

In the usual application of Brune's seismic source model, the stress drop represents an effective stress, that is, an average stress applied over the entire fault surface before rupture, minus a frictional stress which acts to stop rupture. In reality, the stress field over a rupture surface is quite heterogeneous, consisting of areas in which the stress drop is high and areas in which it is low.

There are two methods to determine the stress drop for recorded events using the Brune model. The usual one which has been used is to rely on the Fourier spectrum of ground displacement. According to the Brune model, the theoretical spectrum of shear waves is as shown in Figure 1. To estimate stress drop, the long period (low frequency) spectral level ω_0 and the corner frequency f_0 are determined. These lead (respectively) to estimates of seismic moment M_0 and source radius r :

$$M_0 = 4 \pi \rho \beta^3 R \omega_0 / R_{\theta\phi} \quad (1)$$

$$r = 2.34 \beta / 2 \pi f_0 \quad (2)$$

where ρ is density, β is shear wave velocity, R source-to-site distance, and $R_{\theta\phi}$ a term which accounts for radiation pattern effects. Stress drop $\Delta\sigma$ is then estimated using the relation:

$$\Delta\sigma = 7 M_0 / 16r^3 \quad (3)$$

In practice there is substantial uncertainty in applying these ideas to Fourier spectra of real earthquake records. Figures 2 and 3 illustrate the application for the two horizontal components of the earthquake recorded at Jenkinsville on August 27, 1978 (taken from Fletcher, 1981). The arrows on the spectra indicate the corner frequency and long period level chosen by Fletcher. For the same earthquake, the two horizontal components indicate corner frequencies which differ by a factor of 2.5 and long period levels which differ by a factor of 0.36. Note, from equations (2) and (3), that stress drop is proportional to the cube of corner frequency. Thus stress drop estimates for this earthquake differ by a factor of 5.6 (the product of 2.5^3 and 0.36). The purpose of this is not to dispute the choices of corner frequency and long period spectral level made by Fletcher, but to illustrate how unstable these estimates are when made in this manner using Fourier spectra.

A more satisfactory method of estimating stress drop has been developed and demonstrated recently. Its application is documented for California earthquakes in McGuire and Hanks (1980), and Hanks and McGuire (1981). It is based on

analyzing earthquake records in the time domain, rather than in the frequency domain as above.

As shown in the Appendix to this report, the root-mean-square acceleration can be estimated using the following equations:

$$a_{rms} = (0.85) \frac{(2\pi)^2}{106} \frac{\Delta\sigma}{\rho} R^{-3/2} \sqrt{\frac{2Qr}{2.34}} a_u \quad (4)$$

$$a_u = \left[\exp \frac{(-2\pi f_o R)}{Q\beta} - \exp \frac{-2\pi f_u R}{Q\beta} \right]^{1/2} \quad (5)$$

where f_u is the upper frequency passed by the recording and digitizing system, and other variables are as previously defined. The time domain procedure consists of observing values of a_{rms} on strong motion records, and finding values of $\Delta\sigma$ that, when used with equation (4), give predicted values which match the observed values. The observed value of a_{rms} on each record is determined for a time period starting with the direct shear wave arrival and lasting for the estimated duration of the earthquake, which can be approximated as $1/f_o$. Thus to apply the procedure, a corner frequency must first be estimated in order to determine a duration over which to observe a_{rms} from the earthquake records. Fortunately in typical earthquake records the observed values of a_{rms} are not sensitive to the duration (or f_o value) initially selected, so the procedure is not sensitive to this initial selection.

Table 1 shows stress drops in units of bars (one million dynes per square cm) estimated from the two components recorded during the 1978 event, as derived by the

procedure described above. It is evident that, for the same horizontal components analyzed by Fletcher in the frequency domain, this time-domain procedure gives more stable estimates of stress drop. This is the case because the time-domain technique yields an average stress drop over the entire rupture surface, whereas the frequency-domain technique is affected by peak stress drops which may occur over a very small portion of the faulting surface and which may affect the two horizontal components of motion differently. Thus, in addition to uncertainties in picking the corner frequency and long-period level of the Fourier spectra, the frequency-domain technique has the added uncertainty of sometimes reflecting high stress drops over small areas. These should not be extrapolated and assumed to be representative of the entire rupture surface.

Another point to be made is that, because equation (4) is to be used to predict ground motion characteristics for hypothesized events, it is most logical to analyze and interpret past events using that same equation. It would make little sense to estimate stress drops for recorded earthquakes via one technique, and use those estimates to characterize ground motion with a different technique.

There is no disagreement on record that a stress drop of about 25 bars is an appropriate characterization of the events recorded in 1978 at Jenkinsville when the time-domain procedure is used. Dr. Murphy used Fletcher's frequency

domain estimates in arriving at 100 bar stress drops. Fletcher (letter to Jackson dated February 24, 1981) calculates 30 and 24 bars for the two components of the August 27, 1978 earthquake. (The small difference between these numbers and those cited in Table 1 is due to slightly different assumptions on parameter values.) Although Joyner's calculations are somewhat larger (about 30 bars), this difference is not important for estimation of strong ground motion, therefore, a stress drop of about 25 bars is appropriate to characterize future earthquakes at Monticello.

This methodology is appropriate and conservative for estimating stress drops at Monticello Reservoir during future events for two reasons:

1. The thrust mechanism at Monticello implies a stress regime with relatively large stress differences (and potential stress drops) very close to the surface, and relatively smaller stress differences at depths of 1 to 2 km. The available borehole stress measurements support this conclusion. Ground motion records were obtained during earthquakes (depths ~ 0.2 km) where relatively large stress differences are expected. Therefore it is conservative to assume that a larger faulting surface, necessarily propagating to lower depths, will be characterized by the same average stress drop over its entire surface.

2. The majority of earthquakes at Monticello, with many in the range $2.0 \leq M_L \leq 2.8$, have not triggered the strong motion instrument, or have produced low levels of ground motion. This implies that the well-recorded events thus far have had anomalously large stress drops and associated ground accelerations; the average event at Monticello may well be characterized by a 5 or 10 bar time-domain stress drop.

Estimates of stress drops for six well-recorded Monticello earthquakes, accounting for soil amplification of these records, are documented in the Applicants' response to FSAR Question 361.23, "Accelerograph Deployment Information and Records obtained at Monticello Reservoir, South Carolina."

TABLE 1

Stress drops estimated from August 27, 1978
Monticello Earthquake using time-domain procedure

Component	f_u	Observed $a_{rms}, cm/sec^2$	Estimated $\Delta\sigma, bars$
90°	40	104	22
180°	40	112	24

REFERENCES

- Brune, J. N. (1970), "Tectonic Stress and the Spectra of Seismic Shear Waves from Earthquakes", Journal of Geophy. Res., Vol. 75, No. 26, September 10, pp. 4997-5009.
- Brune, J. N. (1971), "Correction", Journal of Geophy. Res., Vol. 76, No. 20, July 10, p. 5002.
- Fletcher, J.B. (1980), "Spectra from High-Dynamic Range Digital Recordings of Oroville, California, Aftershocks and Their Source Parameters", Bull. Seis. Soc. Am., Vol. 70, No. 3, June, pp. 735-755.
- _____ (1981), "A Comparison between the Tectonic Stress Measured In Situ and Stress Parameters from Seismic Waves at Monticello, South Carolina: A Site of Induced Seismicity", submitted to Bull. Seis. Soc. Am.
- Hanks, T. C., and R. K. McGuire (1981), "The Character of High Frequency Strong Ground Motion", Bull. Seis. Soc. Am., Vol. 71, No. 6, December (in press).
- McGuire, R. K., and T. C. Hanks (1980), "RMS Accelerations and Spectral Amplitudes of Strong Ground Motion During the San Fernando, California, Earthquake", Bull. Seis Soc. Am., Vol. 70, No. 5, October, pp 1907-1919.

APPENDIX

Derivation of a_{rms} for case where lower bound is finite:

$$\hat{a}(f) = \begin{cases} (.85) \frac{\Delta\sigma r}{\rho R_B} \exp\left(-\frac{\pi f R}{Q_B}\right) \left(\frac{f}{f_c}\right)^2 & f < f_0 \\ (.85) \frac{\Delta\sigma r}{\rho R_B} \exp\left(-\frac{\pi f R}{Q_B}\right) & f \geq f_0 \end{cases}$$

where symbols are as defined in Section 361 of the FSAR.

$$\begin{aligned} a_{rms}^2 &= \frac{1}{T_d} \int_c^{T_d} |a|^2 dt = \frac{1}{\pi T_d} \int_0^{2\pi f_u} |\hat{a}(\omega)|^2 d\omega \\ &= \frac{c^2}{\pi T_d} \left\{ \int_0^{2\pi f_0} \exp\left(-\frac{2\pi f R}{Q_B}\right) \left(\frac{f}{f_0}\right)^4 d\omega + \int_{2\pi f_0}^{2\pi f_u} \exp\left(-\frac{2\pi f R}{Q_B}\right) d\omega \right\} \end{aligned}$$

where $c = (.85) \frac{\Delta\sigma r}{\rho R_B}$ and $2\pi f = \omega$

Neglecting, conservatively, the first integral,

$$\begin{aligned} a_{rms}^2 &= \frac{c^2}{\pi T_d} \left[-\frac{Q_B}{R} \exp\left(\frac{-\omega R}{Q_B}\right) \right]_{2\pi f_0}^{2\pi f_u} \\ &= \frac{c^2}{\pi T_d} \frac{Q_B}{R} \left[\exp\left(\frac{-2\pi f_0 R}{Q_B}\right) - \exp\left(\frac{-2\pi f_u R}{Q_B}\right) \right] \end{aligned}$$

so that

$$a_{rms} = (.85)(.37) \frac{\Delta\sigma}{\rho R^{1.5}} \sqrt{\frac{2Qr}{2.34}} \left[\exp\left(\frac{-2\pi f_0 R}{Q_B}\right) - \exp\left(\frac{-2\pi f_u R}{Q_B}\right) \right]^{1/2}$$

For f_0 small and f_u large, the above is the same as equation (9) in McGuire and Hanks (1980). For f_0 non-negligible and f_u non-infinite, and for typical values of R , Q , and β :

$$\frac{2\pi f_u R}{Q\beta} < 0.1$$

so:

$$a_{\text{rms}} = (.85)(.37) \frac{\Delta\sigma}{\rho R^{1.5}} \sqrt{\frac{2Qr}{2.34}} \left[\frac{2\pi R}{Q\beta} (f_u - f_0) \right]^{1/2}$$

To determine the effect of including or excluding f_0 on estimates of $\Delta\sigma$, the above equation can be inverted to give:

$$\Delta\sigma = \frac{\rho R^{1.5} a_{\text{rms}}}{(.85)(.37)} \left[\frac{4\pi Rr}{2.34\beta} (f_u - f_0) \right]^{-1/2}$$

From the last equation it is evident that, for strong motion records digitized at 500 points-per-second, when f_u is 40 or 50 hz, neglecting f_0 (i.e. assuming $f_0 = 0$) is an appropriate approximation for $M_L = 2.8$ earthquakes. For these events, $f_0 = 8$ or 10 hz; neglecting or including this value amounts to only about a 10 percent difference in estimated values of stress drop.

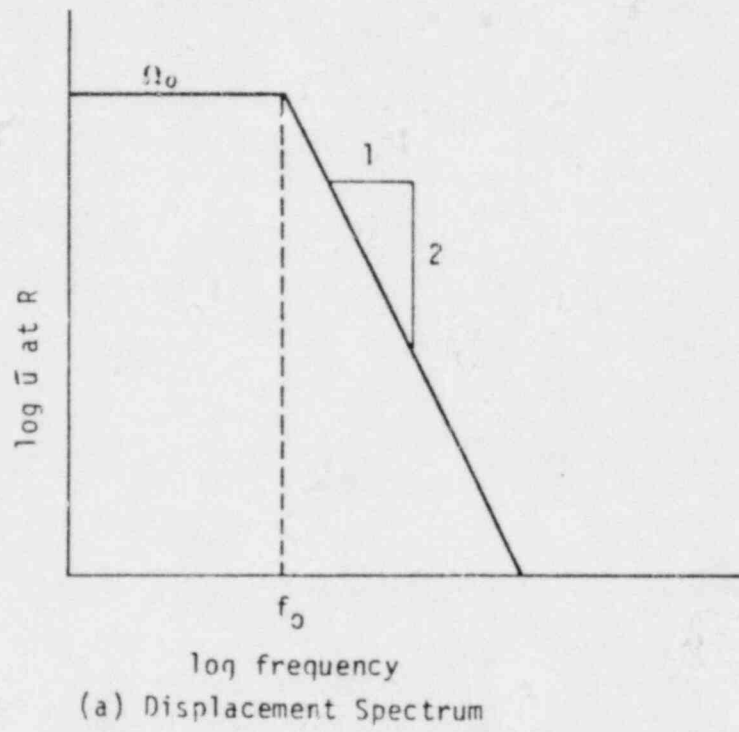


FIGURE 1

THEORETICAL FOURIER SPECTRUM OF DISPLACEMENT

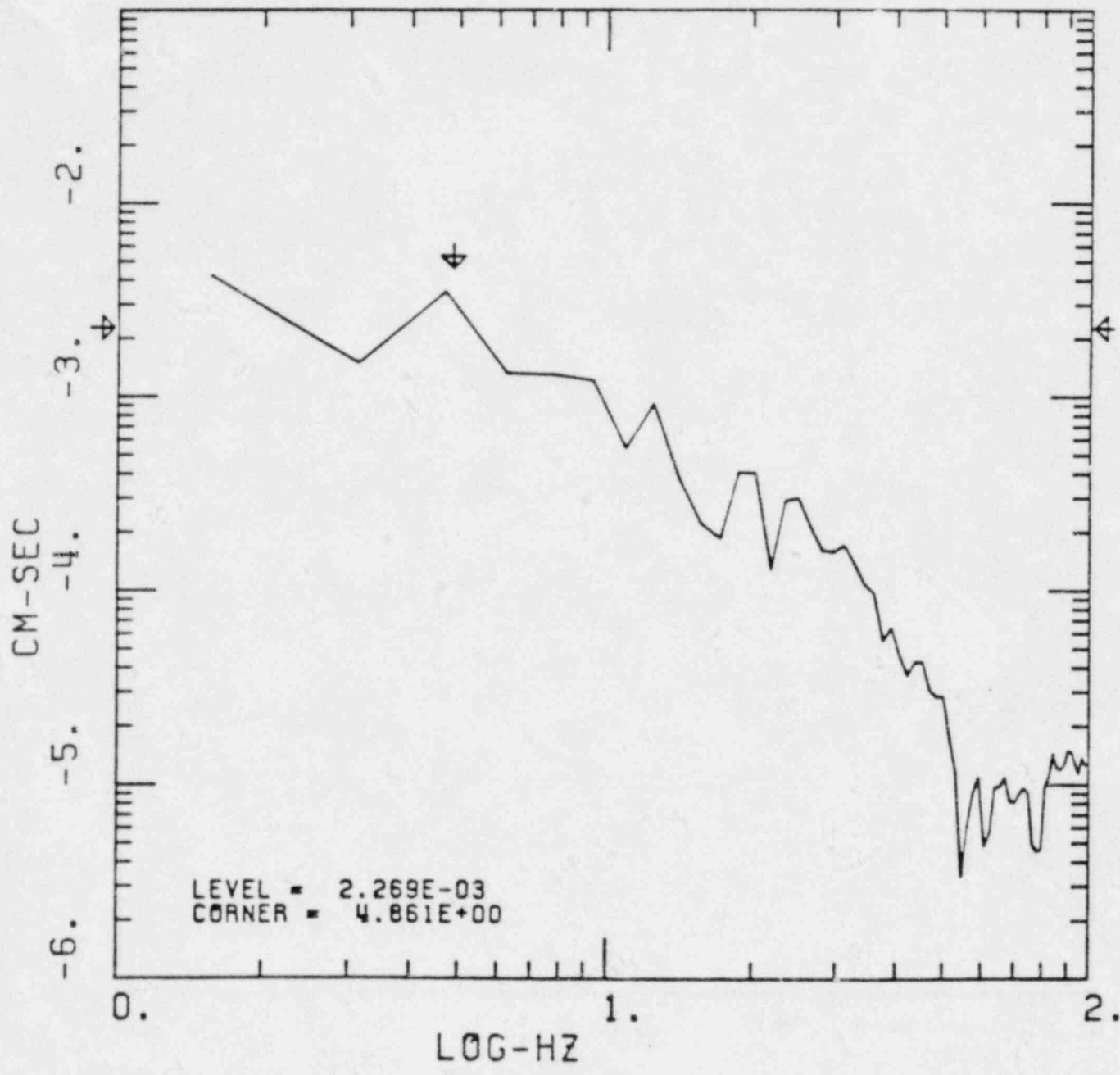


FIGURE 2

DISPLACEMENT SPECTRUM OF 27 AUGUST 1978 MONTICELLO EARTHQUAKE, 90° COMPONENT, AND CHOICES OF CORNER FREQUENCY AND LONG PERIOD LEVEL (AFTER FLETCHER, 1981)

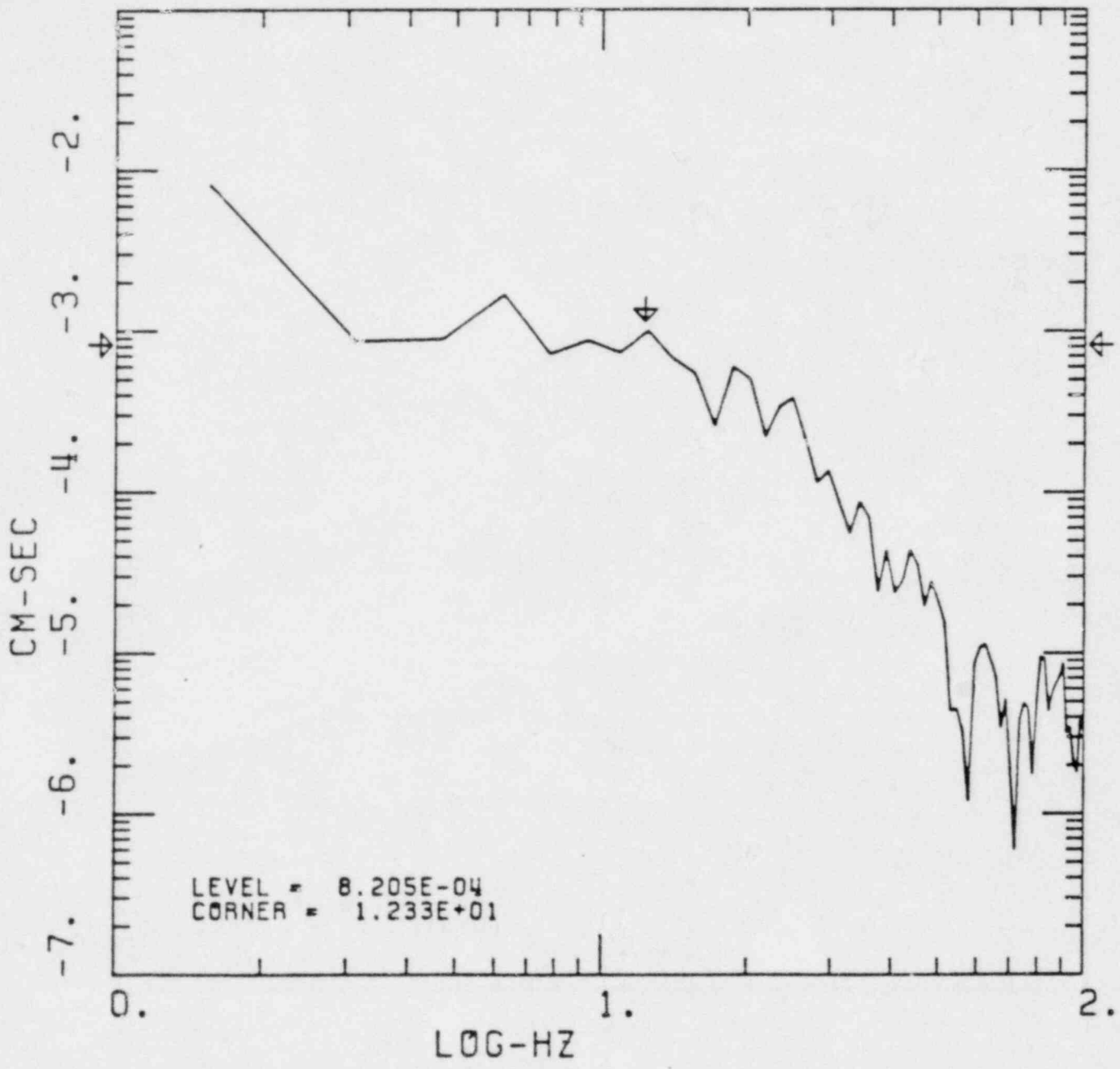


FIGURE 3

DISPLACEMENT SPECTRUM OF 27 AUGUST 1978 MONTICELLO EARTHQUAKE, 180° COMPONENT, AND CHOICES OF CORNER FREQUENCY AND LONG PERIOD LEVEL (AFTER FLETCHER, 1981)

PROBABILITY ESTIMATES OF SEISMICITY AND GROUND MOTION
AT VIRGIL C. SUMMER NUCLEAR STATION

Robin K. McGuire

The Applicant has conducted probability studies of seismicity and ground motion in order to give some perspective to the seismic issues which have been considered for the V. C. Summer Nuclear Station. These studies have been in the areas of reservoir-induced earthquakes and tectonic earthquakes.

Reservoir-Induced Earthquakes

There are several sets of data which allow calculation of probabilities of occurrence of earthquakes with certain magnitudes, as a result of reservoir-induced seismicity (RIS).

Site-Specific Data. The first data set is shown in Figure 1, and represents the cumulative number of earthquakes at Monticello Reservoir plotted versus magnitude for the period June 1978 through September 1981. This excludes the first six months of 1978 during which seismicity was high because of initial filling of Monticello Reservoir. The data shown in Figure 1 represent RIS which has occurred during a time when the reservoir level has been relatively stable. Therefore these are the proper data to use to derive probabilities of occurrence of larger events, if it is assumed that no change in the earthquake triggering mechanism will occur. Normal fluctuation of the

reservoir from the operation of Fairfield Pumped Storage Facility is 4 1/2 feet, from elevation 420.5' to elevation 425'. Emergency draw down past elevation 418.0' is not feasible because of potential wave damage to the Monticello Reservoir Dams (Testimony Mr. W. E. Moore, TR. pg. 1302)

In order to calculate probabilities of occurrence, we note that the data for local magnitude M_L greater than 2.0 have a slope of -1.45, i.e., the b-value in the familiar relationship:

$$\log N_c (M_L) = a - bM_L$$

is 1.45, where N_c is the cumulative number of earthquakes with magnitude greater than M_L . The total number of earthquakes with $M_L \geq 2$ is 147 during the 3.25 year interval, or 45 events per year. This implies that the current rate of occurrence of earthquakes as a function of M_L can be written as:

$$v (M_L) = (45) 10^{-1.45 (M_L - 2)} \quad 2 < M_L < 2.8$$

To calculate a probability of occurrence of larger earthquakes, we must account for the observation that RIS generally decreases and ceases after a number of years. In the manner of Trifunac (1981), we might estimate that RIS will continue for ten years and wish to calculate an average annual probability over 50 years (a typical expected life for a facility such as this). Thus current rates of occurrence must be multiplied by the factor 0.2 to obtain average annual rates of occurrence over a 50 year life.

A further consideration is that large earthquakes must be close to the facility to have any potential for concern. We assume that the reservoir's radius of influence is equal to its largest dimension (about 9 km). This is a common assumption for reservoir-induced seismicity (e.g., Packer et al, 1979). Future reservoir-induced seismicity is assumed to be equally likely anywhere within this radius of influence. The area of influence encompassed by such a radius of influence is larger than the area which has exhibited seismicity to date, but this must be the case to be consistent with the assumption that larger events will occur at all. For these events to occur, the effects of the reservoir must extend to new areas where stress stabilization has not already begun. In those areas of Monticello Reservoir which have exhibited seismicity, earthquakes with magnitudes greater than 2.8 are unlikely; if they were possible they would already have occurred. Therefore, if magnitudes in the range 3 to 4 are at all possible, they will probably occur in areas which have not yet exhibited seismicity.

Taking a horizontal distance of 1 km as the distance of concern (large earthquakes would also have to occur at depths greater than those which have been recorded), the ratio of the area encompassed by a circle of radius 1 km to the area of reservoir influence is $(1/9)^2 = 0.012$. Magnitude 4 has been suggested as the largest earthquake which can occur at shallow depths (see testimony by Prof. Nuttli). From the above assumptions we can calculate the

average annual probability that an earthquake of $M_L \geq 4$ will occur at an epicentral distance less than 1 km as:

$$(45) 10^{-1.45 (M_L - 2)} (0.2) (0.012) = 1.4 \times 10^{-4}$$

This implies a return period of about 7400 years. A similar calculation for a distance of 2 km gives:

$$(45) 10^{-1.45 (M_L - 2)} (0.2) (0.049) = 5.6 \times 10^{-4}$$

or a return period of about 1800 years. Thus we conclude from available site-specific data that an RIS event with $M_L \geq 4$ in the vicinity of the facility (within one or two km) is an unlikely event.

Regional Data. Another useful set of data is that from the history of RIS in the Piedmont region of the eastern U.S. (see testimony by McWhorter). These data provide 422 years of RIS experience, with a maximum magnitude of 3.8 (excluding the event at the Clark Hill Reservoir, as discussed by McWhorter). Most of these reservoirs are larger and/or deeper than Monticello. Using this larger data base, and assuming an average of 422 reservoir-years per occurrence of an $M_L = 4$ RIS earthquake, the probability of occurrence of such an earthquake within 1 km of the Virgil C. Summer Nuclear Station is:

$$(1/422) (1/9)^2 = 2.9 \times 10^{-5}$$

or a return period of 34,000 years. The equivalent probability for 2 km distance is:

$$(1/422) (2/9)^2 = 1.2 \times 10^{-4}$$

which corresponds to a return period of 8500 years. We conclude from these regional RIS data that the probability of occurrence of an $M_L \geq 4$ earthquake in the vicinity of the facility is extremely remote.

Tectonic Earthquakes

The Applicant has conducted several probabilistic seismic hazard analyses for tectonic earthquakes in order to give the reservoir-induced earthquake issues a broader perspective. Various hypotheses on tectonic provinces were assumed during these analyses to present as complete a picture as possible.

The general assumptions used in these analyses are as follows:

1. Zones of potential future earthquakes are delineated by seismicity and tectonic evidence; the average predicted rates of occurrence in these zones are accurately estimated by historical occurrences in these zones.
2. The relative frequency of earthquake sizes as measured by epicentral MM intensity I_e in seismogenic zones can be represented by a truncated exponential distribution.
3. The MM intensity at the site of interest, I_s , can be represented as a function of epicentral intensity and the distance between the site and the source of energy release.

Given these assumptions, the probabilistic hazard analysis consists of mathematically integrating over all possible epicentral intensities and locations and calculating for each epicentral intensity and location the distribution of site intensity to evaluate the probability that various levels of site intensity will be exceeded annually. A standard computer program (McGuire, 1976) was used for calculations.

For all probability calculations, the following attenuation function was used to describe the variation of I_s with I_e and epicentral distance Δ :

$$\begin{aligned} I_s &= I_e & \Delta &\leq 10 \text{ km} \\ I_s &= 3.08 + I_e - 1.34 \ln \Delta & \Delta &> 10 \text{ km} \end{aligned}$$

This equation is based on the observed attenuation of intensities during the 1886 Charleston earthquake. Uncertainty in the predicted intensity was described by a normal distribution with a standard deviation of 1.19 intensity units, which is typical of observed scatter in ground motion estimates.

Results of these analyses are presented in Table 1 in terms of return period associated with various values of I_s for different seismogenic zones. The first set of seismogenic zones considered are those used in the Final Safety Analysis Report for the Virgil C. Summer Nuclear Station, which are shown in Figure 2. Two cases were considered: that where the maximum possible MM intensity in

each zone is one unit greater than the maximum historical intensity, and that where they are equal. The second set of zones considered are those used by Algermissen and Perkins (1976), which are reproduced in Figure 3. The third and fourth sets of zones correspond to two hypotheses which have been proposed to explain the Charleston earthquake (the "Mesozoic Rifting" hypothesis and the "Decollement" hypothesis). These have been explained in detail in the "Supplemental Seismological Investigation" report submitted by the Applicant. Figures 4 and 5, reproduced from the above document, delineate these zones.

Table 1 indicates that, for a site MM intensity of VII, which is the Safe Shutdown Earthquake, the calculated return period varies between 1,700 and 10,000 years depending upon seismogenic zones and maximum intensities ascribed to those zones. This is a longer return period than indicated by similar studies at other nuclear plants (e.g. TERA Corp., 1980). The Decollement hypothesis is an unlikely explanation for the Charleston earthquake, for reasons explained by the Applicant in the "Supplemental Seismological Report." Of the remaining hypotheses, the FSAR zones with maximum intensity one unit greater than the maximum historical intensity, and the Mesozoic Rifting hypothesis both make the conservative assumption, for the purposes of seismic probability calculations, that events up to MM intensity VIII-IX can occur in the seismic zone encompassing the site. Similarly, the Algermissen-Perkins zones are conservatively

assumed to allow an MM intensity X-XI to occur in the South Carolina-Georgia seismic zone. These three sets of zones indicate nearly the same return period (3,100 to 4,500 years) for MM intensity VII at the site.

One convenient comparison is available in the form of seismic risk results reported for the U.S. by Algermissen and Perkins (1976). At the site location, these authors indicate that a peak horizontal acceleration of about 0.10g has a return period of 500 years. We can calculate an approximate return period for 0.15g (the SSE acceleration) by using the rule-of-thumb that return period varies as the cube of acceleration (e.g. doubling the acceleration implies increasing the return period by a factor of eight). It follows that a peak acceleration of 0.15g has a return period of $500 (1.5)^3 = 1700$ years. This is not inconsistent with the values reported in Table 1, given the generic nature of the Algermissen-Perkins study and the differences associated with estimating MM intensities in one study and peak accelerations in another.

These results indicate that an MM intensity VII is an appropriate earthquake for the SSE, compared with probability results calculated at other facilities. Further, the choice of seismogenic zones from Table 1 used to represent earthquake occurrences in the southeastern United States is not critical for evaluating seismic hazard at the Virgil C. Summer Nuclear Station.

REFERENCES

- Algermissen, S.T., and D.M. Perkins (1976), "A Probabilistic Estimate of Maximum Acceleration in Rock in the Contiguous United States," U.S.G.S. Open File Report 76-416, 45 pp.
- McGuire, R.K. (1976), "FORTRAN Computer Program for Seismic Risk Analysis," U.S.G.S. Open File Report 76-67, 92 pp.
- Packer, D. L., L. S. Cluff, P. L. Knuepfer and R. J. Withers (1979), "Study of Reservoir Induced Seismicity", Final Technical Report to U.S.G.S., Woodward-Clyde Consultants, August.
- TERA Corp., (1980), "Seismic Hazard Analysis," NUREG/CR-1582, U.S.N.R.C., August, 59 pp.
- Trifunac, M.D. (1981), "Comments in the Matter of South Carolina Electric & Gas Company (Virgil C. Summer Nuclear Station, Unit 1)," Report to ASLB, September.

TABLE 1

Return Periods (in years) for Various Seismogenic Zones

MMI	V	VI	VII	VIII
*FSAR zones with max. intensity = largest hist. intensity + 1	200	830	4,500	32,000
*FSAR zones with max. intensity = largest hist. intensity	270	1,400	10,000	67,000
*Algermissen-Perkins zones with max. intensity = largest hist. intensity	180	714	3,100	15,000
** Mesozoic Rifting	140	590	3,300	30,000
**Decollement	110	400	1,700	8,300

* FSAR Table 361.19-1, Amendment 21

** "Supplemental Seismologic Investigation, Virgil C. Summer Nuclear Station", section entitled "Charleston Earthquake"

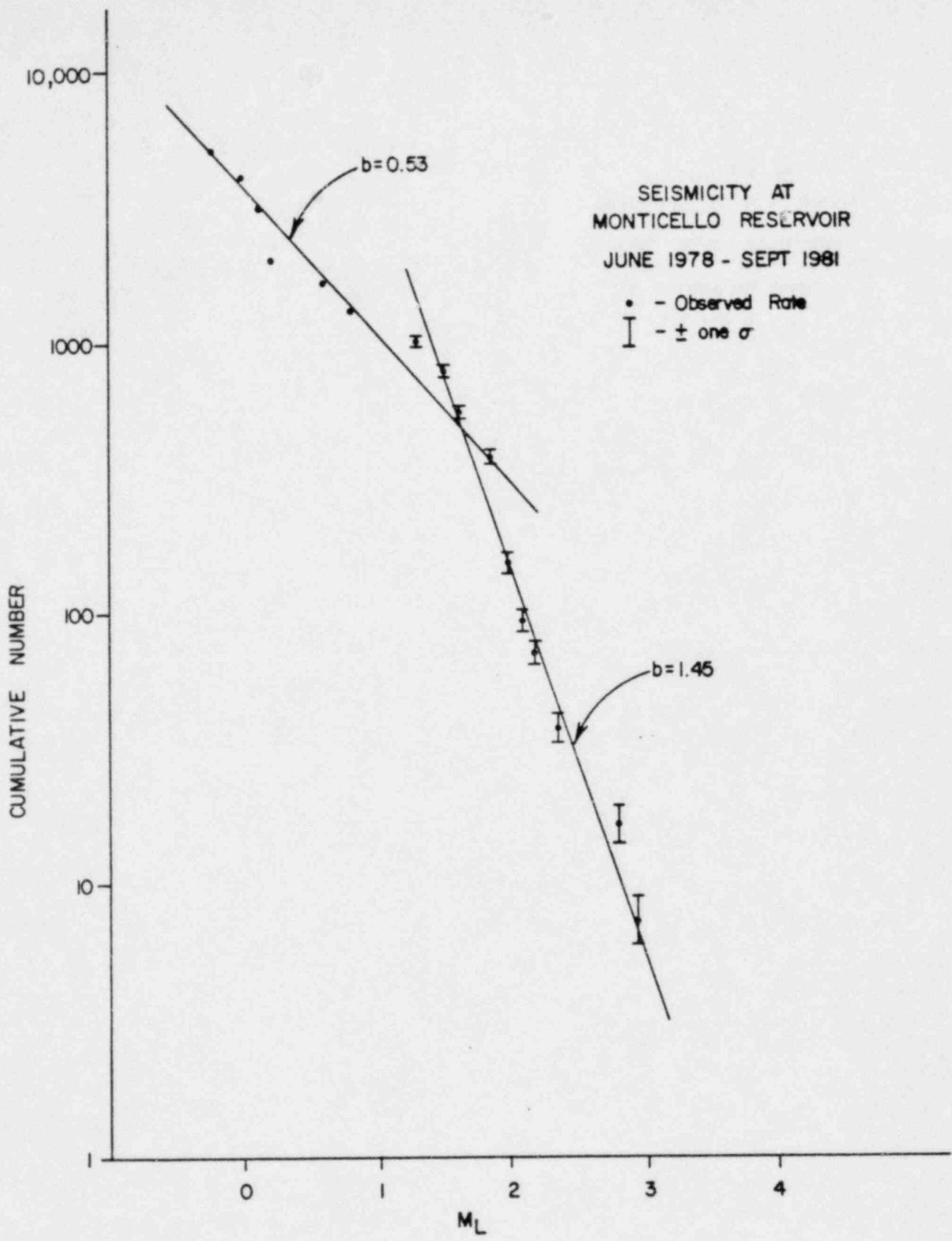


FIGURE 1 - CUMULATIVE NUMBER OF EVENTS VERSUS M_L

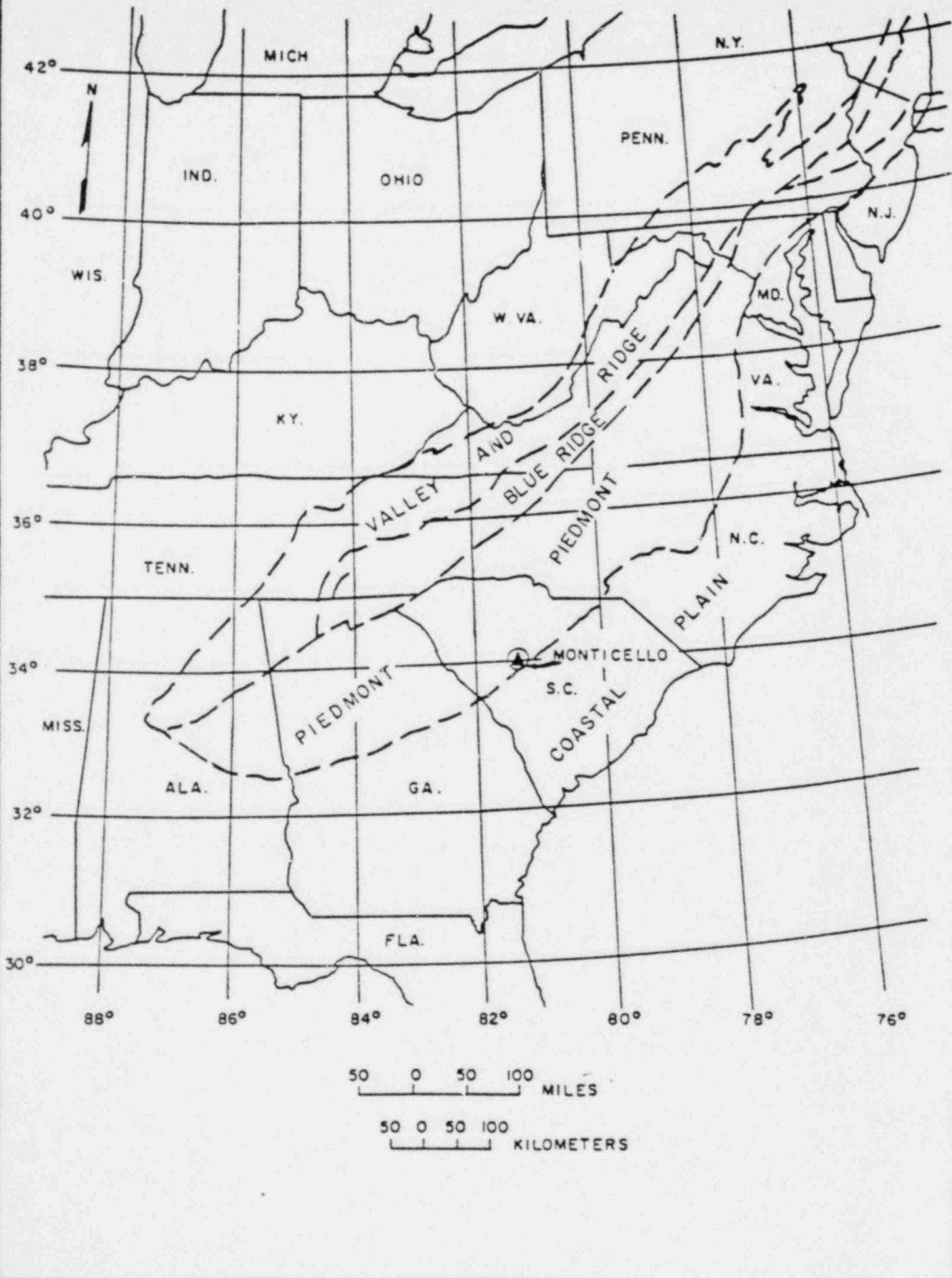


Figure 2
SEISMOGENIC ZONES USED IN FSA

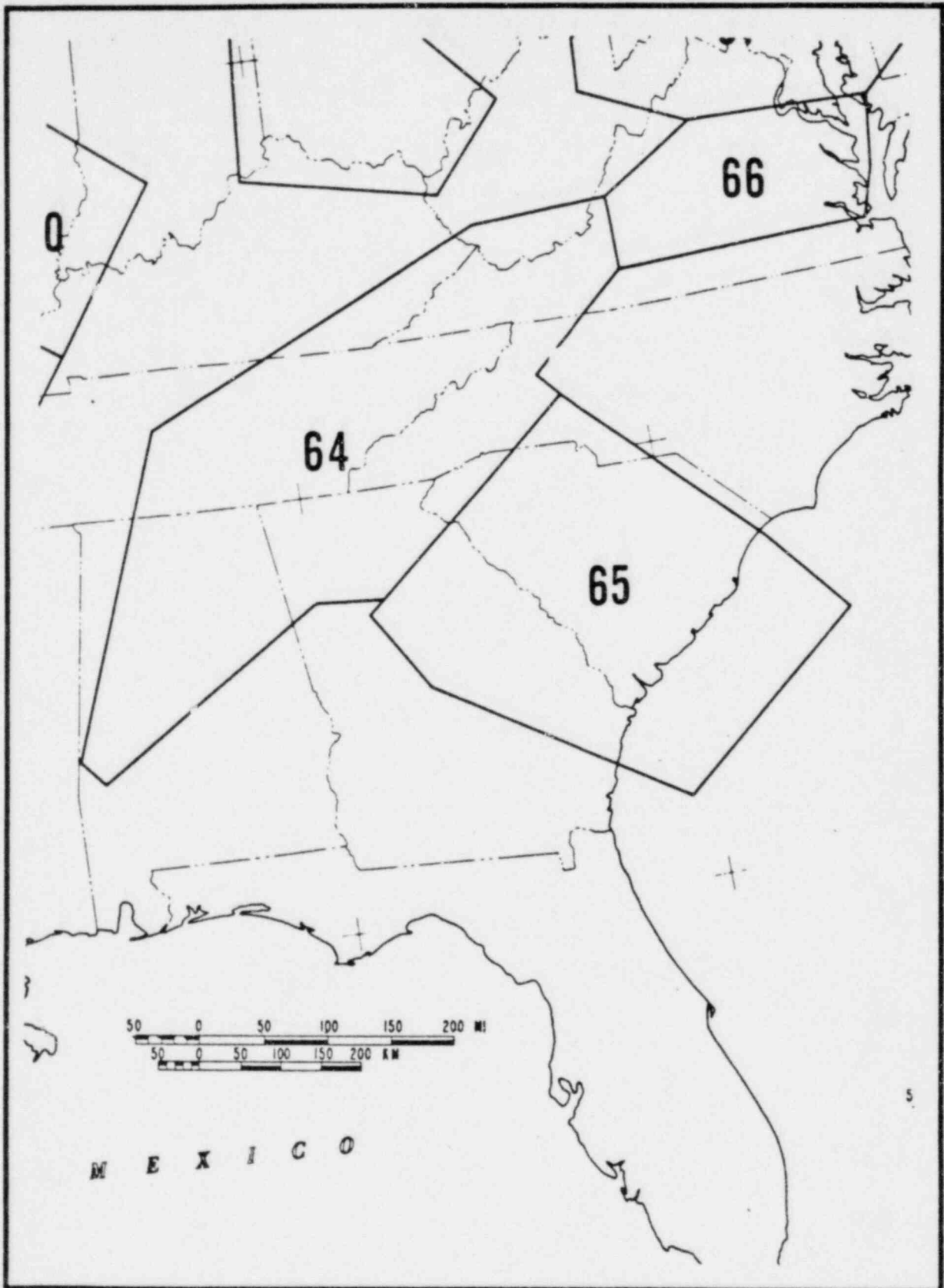


Figure 3

SEISMOGENIC ZONES IN SOUTHEAST U.S.
USED BY ALGERMISSEN AND PERKINS (1976)

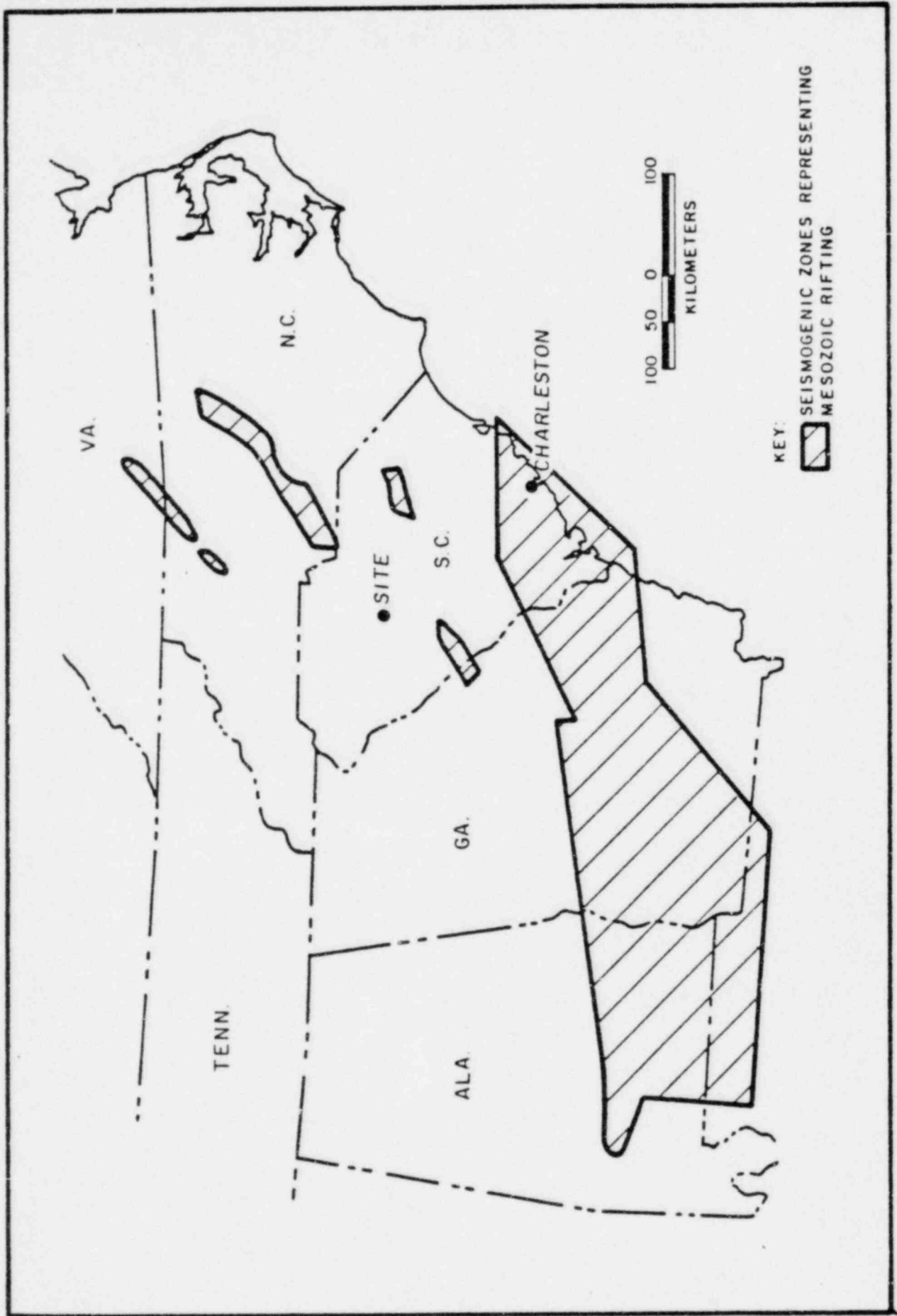


FIGURE 4
SEISMOGENIC ZONES REPRESENTING MESOZOIC RIFTING

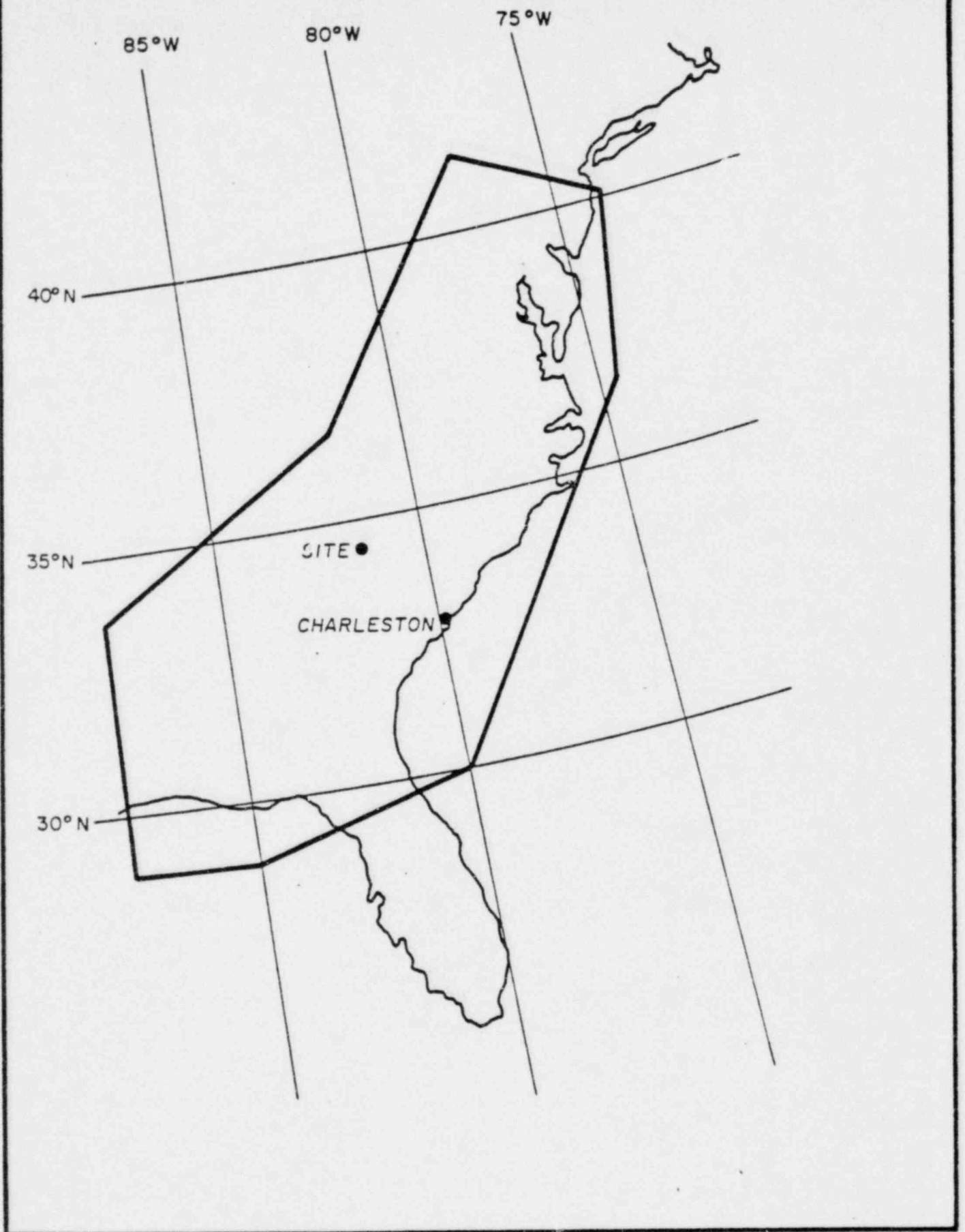


FIGURE 5
SEISMOGENIC ZONE REPRESENTING DECOLLEMENT HYPOTHESIS

PROCESSING AND ANALYSIS OF
ACCELEROGRAMS FROM AFTERSHOCKS OF
THE 1975 OROVILLE, CALIFORNIA, EARTHQUAKE

Robin K. McGuire

INTRODUCTION

This report documents the processing and statistical analyses of the horizontal components of accelerograms recorded during the 1975 Oroville aftershocks sequence. This work was performed as part of the seismic investigations for the Virgil C. Summer Nuclear Station located near Jenkinsville, South Carolina. Accelerograms recorded at the following rock sites (Seekins and Hanks, 1978) during the 1975 Oroville aftershock sequence were selected for analysis:

- i) Oroville Medical Center (OMC)
- ii) EBH Station
- iii) CDMG Station 6
- iv) CDMG Station 7
- v) CDMG Station 8

A total of 44 horizontal components were obtained from these sites and were extracted from a magnetic tape containing the Volume I uncorrected data obtained from NOAA. The analyses consisted of Volume II processing (instrument response correction and band pass filtering), calculation of response spectra and statistical analyses of the calculated response spectra.

VOLUME I DATA

Table 1 summarizes the 44 horizontal components from the 1975 Oroville aftershocks that were recorded at the five

sites. Local site geology for each of the five sites is summarized in Table 2. Although the data in Table 2 indicate that only two sites, CDMG stations 6 and 8, are crystalline rock, Seekins and Hanks (1978) considered all these sites as bedrock sites because the surficial material overlying the crystalline rock at the sites was hard cemented deposits of shallow depth (less than 50 m) (Hanks, personal communication, 1981).

Earthquake magnitudes of the 44 components vary between 4.0 and 5.2 with an average magnitude of 4.4. Focal depths of the 44 components vary between 6.3 and 12 km with an average of 9.4 km. Hypocentral distances are less than about 15 km (Seekins and Hanks, 1978).

VOLUME II PROCESSING

The basic procedures used to correct the accelerograms at the Volume II stage were those developed by the Earthquake Engineering Research Laboratory at Caltech which are described by Trifunac and Lee (1973). Initial processing was performed with decimation of the data to equally spaced time intervals of .02 seconds and band-pass filtered between .40 and 23 Hz. This was found to be inadequate because clipping of the high frequency pulses occurred. The data were reprocessed with decimation to equally spaced time intervals of .005 seconds prior to instrument correction and with the following filter parameters:

BAND1 = .65 Hz
BAND2 = 46 Hz

DF1 = .5 Hz
DF2 = 4.0 Hz

BAND1 and BAND2 represent the low and high cutoff frequencies, and DF1 and DF2 represent the width of the roll off ramps. Comparison of the Volume II data computed with these parameters to the original Volume I data revealed no clipping as opposed to the comparison when the initial processing parameters were used. The effect of the different processing parameters can be seen in Figure 1 for the N24W accelerogram recorded at OMC on 75/08/6. In this figure the 5 percent damped pseudo-relative velocity spectra that were computed from the Volume II data processed with different parameters are plotted. Using the initial parameters resulted in underestimation of the high frequency (low period) spectral ordinates.

The high frequency cutoff of 46 Hz used for the Volume II processing is adequate. No earthquake signal present at the Volume I stage is lost during the Volume II processing.

The values of corrected peak acceleration, velocity and displacement for each component are summarized in Table 3.

RESPONSE SPECTRA AND STATISTICAL ANALYSES

Pseudo-relative velocity (PSRV) response spectra were calculated for each of the 44 components. Three values of damping were used; 2, 5 and 10 percent of critical. A composite plot of 5 percent PSRV spectra for the 12 components recorded at the crystalline rock sites (CDMG stations 6 and 8) is shown in Figure 2. In Figure 3 a composite plot of 5 percent PSRV spectra for all 44 components is shown. Data are plotted over a spectral period range of from .02 to 2.0 seconds.

For the purposes of performing statistical analyses the spectral data were divided into two groups. The first group consisted of the 12 components recorded at the two crystalline rock sites (CDMG Stations 6 and 8) and the second group consisted of all 44 components listed in Table 1.

In Figures 4, 5 and 6 arithmetic mean and mean plus one standard deviation PSRV spectra calculated from the 12 component data group, for 2, 5 and 10 percent damping respectively, are plotted. In Figures 7, 8 and 9 the arithmetic mean and mean plus one standard deviation PSRV spectra calculated from all 44 components, for 2, 5 and 10 percent damping respectively, are plotted. Also shown on Figures 4 through 9 are RIS spectra estimated by the Applicant for $M_L = 4.5$, which are anchored at 0.22g. It is evident that, in all cases, the Applicant's RIS spectra match the Oroville mean plus one standard deviation spectra, for both sets of data.

The conclusion from this study is that the RIS spectra for $M_L = 4.5$ developed by the Applicant, and anchored to 0.22g, are conservative spectra for those magnitude earthquakes. The RIS spectra match the mean plus one standard deviation spectra from rock and rock-like sites at Oroville. Because these Oroville data were not used in the derivation of the Applicant's RIS spectra, this constitutes an independent check on, and verification of, the validity of both spectral amplitudes and spectral shapes developed by the Applicant.

REFERENCES

Seekins, L. C. and Hanks, T. C., 1978, "Strong-Motion Accelerograms of the Oroville Aftershocks and Peak Acceleration Data," Bull. Seism. Soc. Am., Vol. 68, No. 3, pp. 677-689.

Trifunac, M. D. and Lee, V. W., 1973, "Routine Computer Processing of Strong-motion Accelerograms," California Institute of Technology, EERL Report No. 73-03, 360 p.

TABLE 1

ACCELEROGRAMS RECORDED DURING
THE 1975 OROVILLE AFTERSHOCKS

<u>Date Time</u>	<u>Station (Record No.)</u>	<u>Local Magnitude</u>	<u>Focal Depth (km)</u>	<u>Uncorrected Peak Horiz. Accel. (g)</u>
75/08/06 03:50G	OMC	4.7	9.3	.39, .17
75/08/06 03:50G	EBH Station	4.7	9.3	-.19, -.21
75/08/02 20:59G	OMC	5.2	-	-.05, .08
75/08/08 07:00G	CDMG Station 6	4.9	9.2	.08, .11
75/08/08 07:00G	CDMG Station 7	4.9	9.2	-.12, -.09
75/08/08 07:00G	EBH Station	4.9	9.2	.12, .16
75/09/27 22:34G	CDMG Station 8	4.6	12.	-.16, -.08
75/09/27 22:34G	OMC	4.6	12.0	.12, -.13
75/09/27 22:34G	EBH Station	4.6	12.0	-.15, .13
75/08/03 01:03G	OMC	4.6	8.3	-.17, .18
75/08/03 01:03G	EBH Station	4.6	8.3	-.15, 0.18
75/08/03 02:47G	OMC	4.1	6.3	.09, .15
75/08/03 02:47G	EBH Station	4.1	6.3	-.15, -.09
75/08/16 05:48G	CDMG Station 6	4.0	9.4	-.11, .10

TABLE 1 (Continued)

ACCELEROGRAMS RECORDED DURING
THE 1975 OROVILLE AFTERSHOCKS

<u>Date Time</u>	<u>Station (Record No.)</u>	<u>Local Magnitude</u>	<u>Focal Depth (km)</u>	<u>Uncorrected Peak Horiz. Accel. (g)</u>
75/08/16 05:48G	CDMG Station 7	4.0	9.4	-.07, -.06
75/08/16 05:48G	CDMG Station 8	4.0	9.4	.04, 0.07
75/08/16 05:48G	OMC	4.0	9.4	.10, 0.03
75/08/15 05:48G	EBH Station	4.0	9.4	-.12, -.07
75/09/26 02:31G	CDMG Station 6	4.0	9.7	.10, .07
75/09/26 02:31G	CDMG Station 8	4.0	9.7	-.07, .04
75/09/26 02:31G	OMC	4.0	9.7	-.08, .09
75/09/26 02:31G	EBH Station	4.0	9.7	-.06, -.09

Notes:

- 1) Reference for Date, Station, Magnitude, and Focal Depth is:

Seekins, Linda C. and Hanks, Thomas C., 1978, "Strong-Motion Accelerograms of the Oroville Aftershocks and Peak Acceleration Data," Bull. Seism. Soc. Am., Vol. 68, No. 3, pp. 677-689.

- 2) Uncorrected peak ground accelerations were obtained from magnetic tape provided by NOAA.

TABLE 2
LOCAL SITE GEOLOGY

<u>Site</u>	<u>Local Geology</u>
Oroville Medical Center	Pleistocene gravels and alluvium. Near recent alluvial deposits and Tertiary gravels and conglomerates.
EBH Station	Recent alluvium
CDMG Station 6	Pre-Tertiary crystalline rocks
CDMG Station 7	Tertiary gravels and conglomerates
CDMG Station 8	Pre-Tertiary crystalline rocks

Notes:

- 1) Reference for local site geology is Seekins and Hanks (1978).

TABLE 3

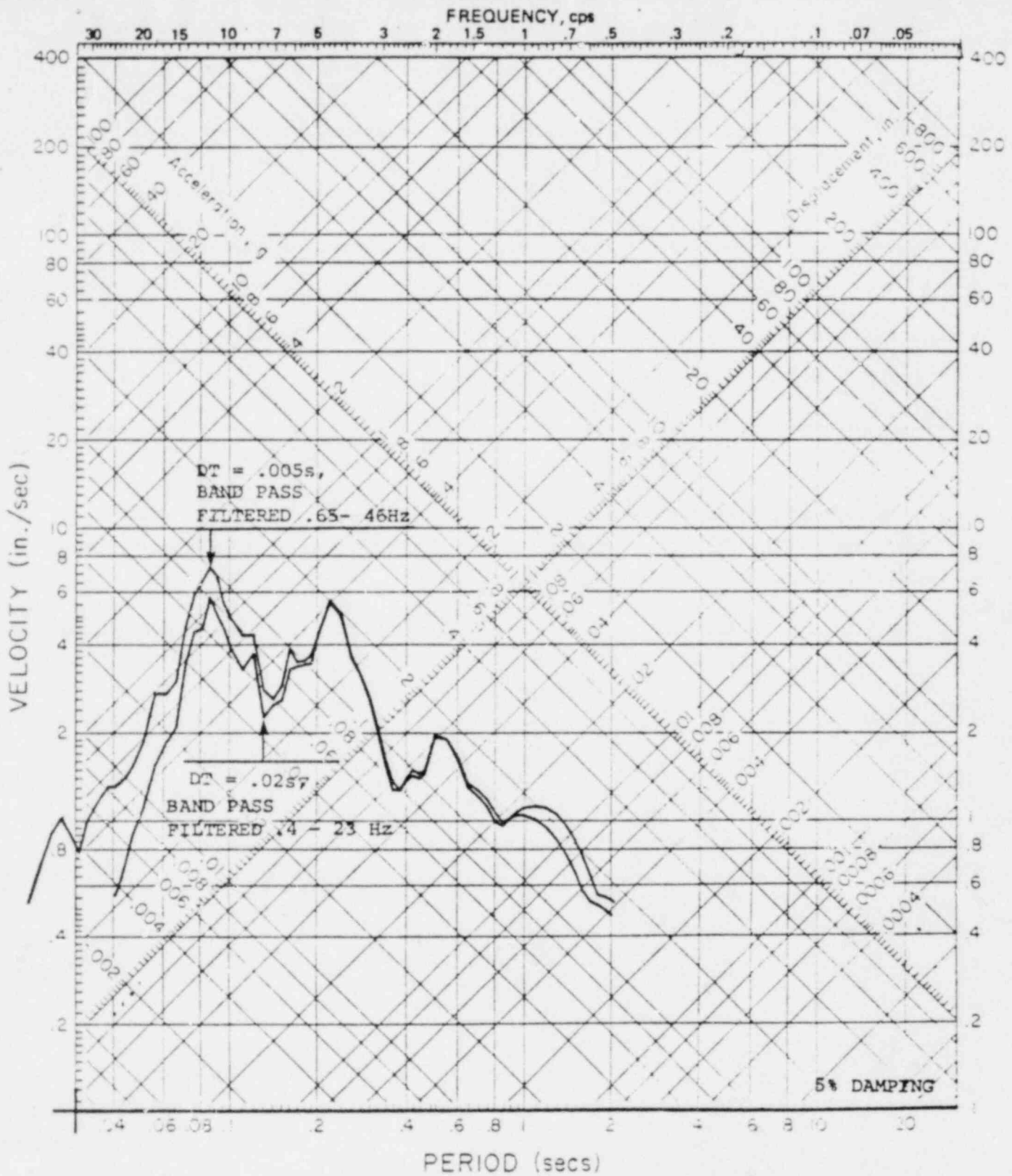
CORRECTED PEAK MOTIONS OF 1975 OROVILLE AFTERSHOCKS

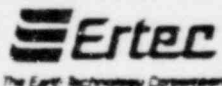
Date	Station	Component	Local Magnitude	Corrected Peak Motion		
				a (cm/s ²)	v (cm/s)	d (cm)
75/08/06 03:50G	OMC	N24W	4.7	334.2	4.59	.239
		S66W	"	161.4	4.52	.272
75/08/06 03:50G	EBH Station	N90E	4.7	178.6	6.34	.376
		N00E	"	204.6	7.71	.542
75/08/02 20:59G	OMC	N24W	5.2	51.3	2.24	.142
		S66W	"	76.3	3.31	.338
75/08/08 07:00G	CDMG Station 6	S55E	4.9	73.5	1.33	.068
		N35E	"	102.7	2.49	.085
75/08/08 07:00G	CDMG Station 7	N90W	4.9	121.0	1.26	.073
		S00W	"	82.9	1.42	.068
75/08/08 07:00G	EBH Station	N90E	4.9	114.6	2.68	.197
		N00E	"	149.9	3.00	.165
75/09/27 22:34G	CDMG Station 8	N90W	4.6	155.7	7.24	.445
		S00E	"	70.9	2.79	.148
75/09/27 22:34G	OMC	N24W	4.6	116.1	2.57	.127
		S66W	"	112.7	2.71	.135
75/09/27 22:34G	EBH Station	N90E	4.6	142.8	3.48	.178
		N00E	"	130.2	5.24	.290
75/08/03 01:03G	OMC	N24W	4.6	158.8	4.45	.272
		S66W	"	173.1	5.27	.396
75/08/03 01:03G	EBH Station	N90E	4.6	147.5	7.66	.486
		N00E	"	177.5	6.37	.454
75/08/03 02:47G	OMC	N24W	4.1	80.3	2.06	.112
		S66W	"	144.7	4.71	.279
75/08/03 02:47G	EBH Station	N90E	4.1	148.9	4.56	.283
		N00E	"	89.0	2.97	.227
75/08/16 05:48G	CDMG Station 6	S55E	4.0	100.2	.923	.033
		N35E	"	96.8	.959	.027

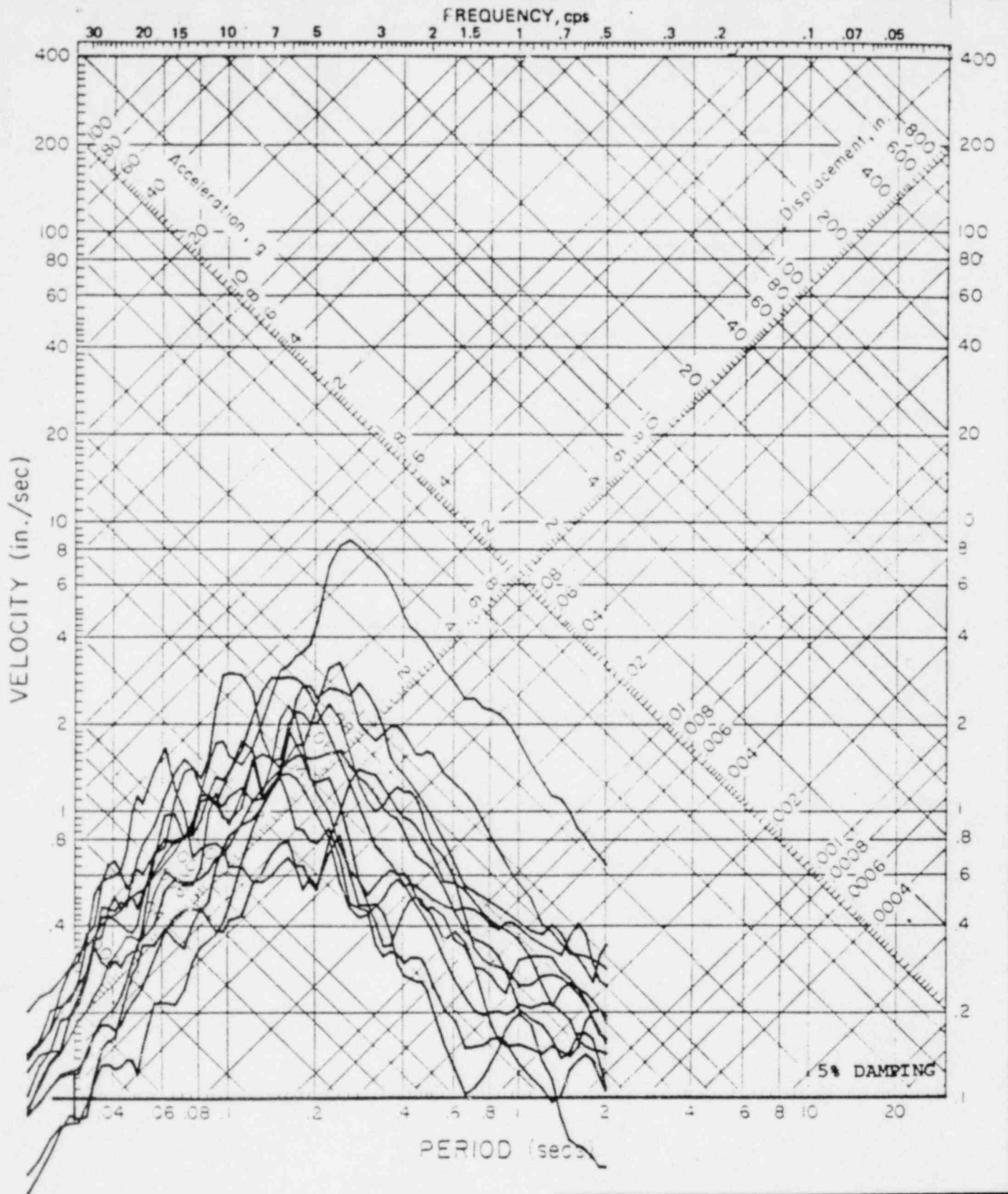
TABLE 3 (Continued)

CORRECTED PEAK MOTIONS OF 1975 OROVILLE AFTERSHOCKS

Date	Station	Component	Local Magnitude	Corrected Peak Motion		
				a (cm/s ²)	v (cm/s)	d (cm)
75/08/16 05:48G	CDMG Station 7	N90W	4.0	62.9	1.02	.050
		S00W	"	58.4	1.20	.067
75/08/16 05:48G	CDMG Station 8	N90W	4.0	36.2	.994	.077
		S00E	"	64.9	2.11	.084
75/08/16 05:48G	OMC	N24W	4.0	91.5	1.80	.100
		S66W	"	28.7	.686	.040
75/08/16 05:48G	EBH Station	N70E	4.0	120.7	2.65	.139
		N00E	"	65.7	1.71	.096
75/09/26 02:31G	CDMG Station 6	S55E	4.0	90.6	2.48	.088
		N35E	"	60.9	1.15	.040
75/09/26 02:31G	CDMG Station 8	N90W	4.0	65.0	2.09	.137
		S00E	"	41.7	1.36	.082
75/09/26 02:31G	OMC	N24W	4.0	75.3	1.54	.063
		S66W	"	80.4	1.80	2.99
75/09/26 02:31G	EBH Station	N90E	4.0	55.0	1.28	.062
		N00E	"	86.1	2.03	.108



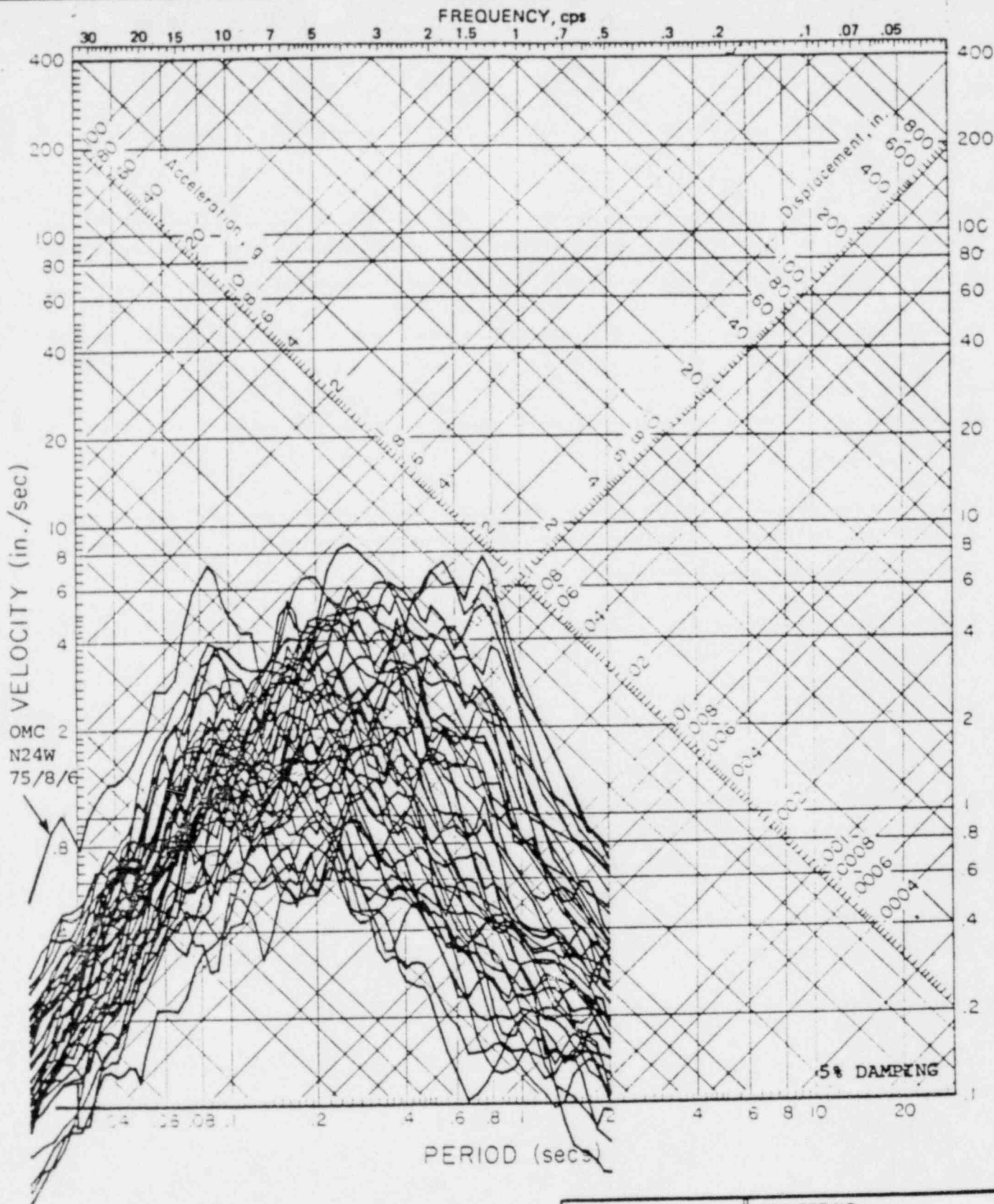
 <p style="font-size: small;">The Earth Technology Corporation</p>	PROJECT NO.:
FIGURE 1. COMPARISON OF 5% PSRV OF 75/8/6 OMC ACCELEROGRAM (COMPONENT N24W) PROCESSED WITH DIFFERENT PARAMETERS AT VOLUME II STAGE.	



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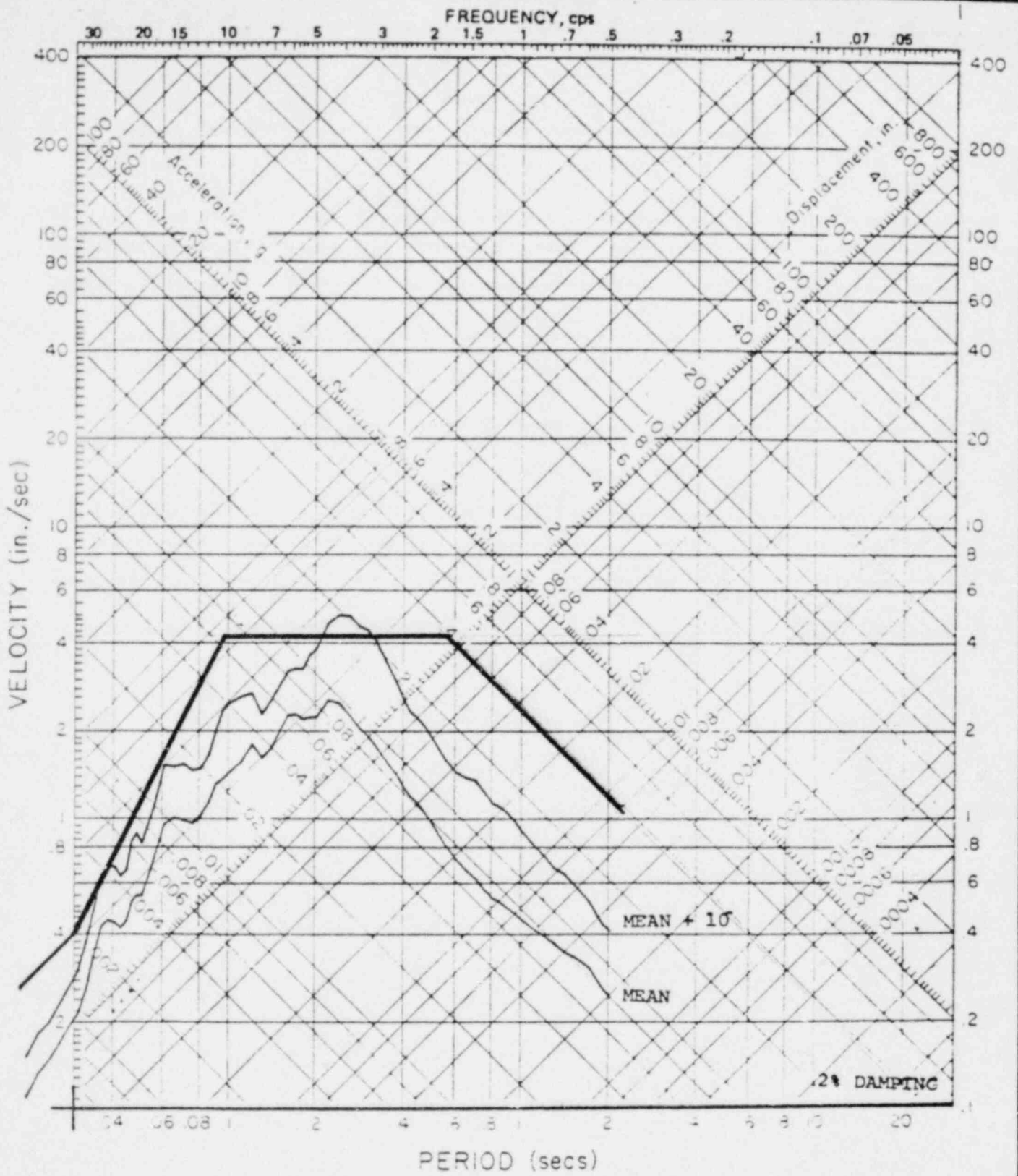
FIGURE 2. COMPOSITE PLOT OF 5% PSRV SPECTRA RECORDED AT CDMG STATIONS 6 AND B (12 COMPONENTS).



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The Earth Technology Corporation

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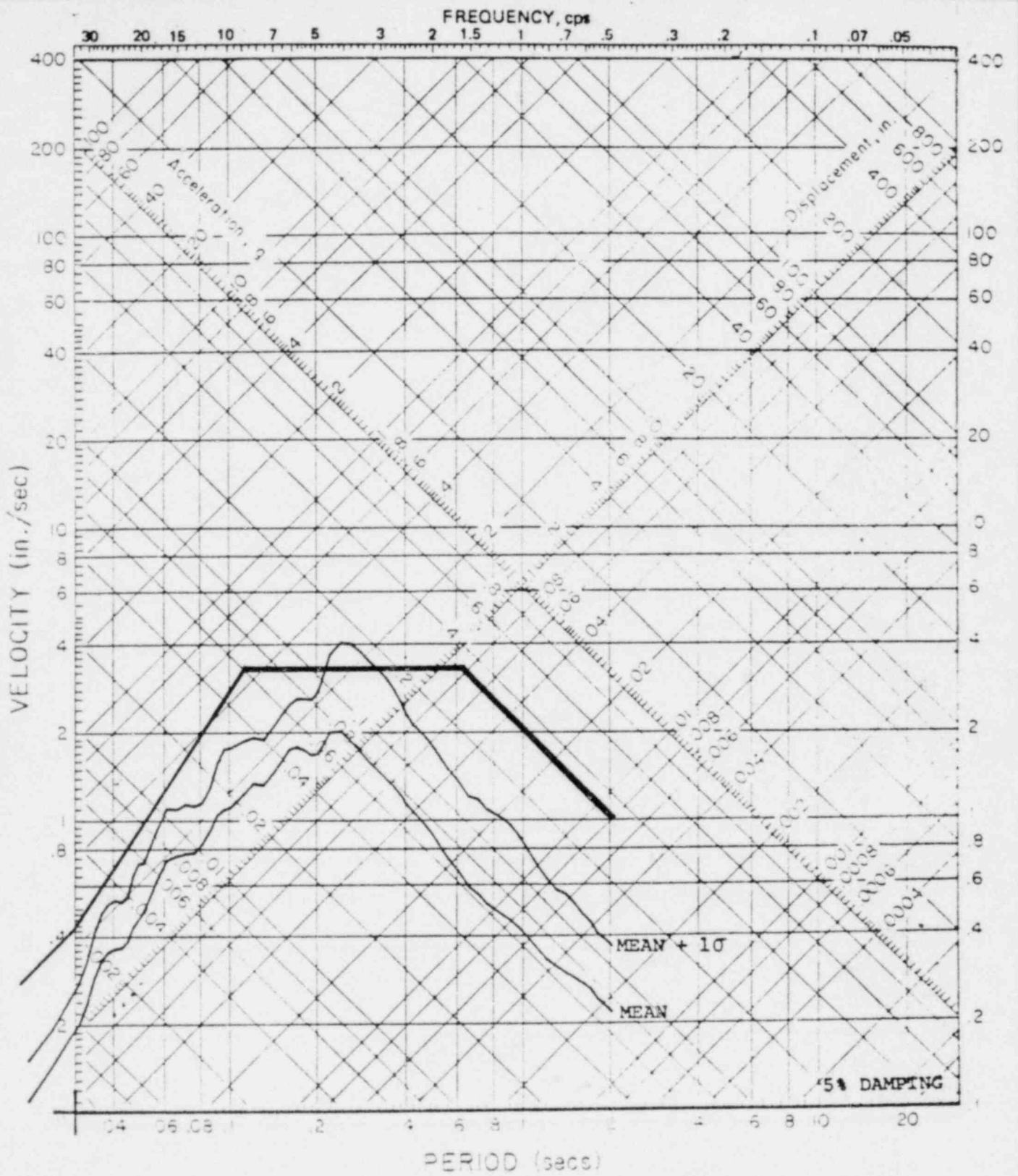
FIGURE 3. COMPOSITE PLOT OF 5% PSR SPECTRA RECORDED AT ALL SITES (AA COMPONENTS).



Ertac
The Earth Technology Corporation

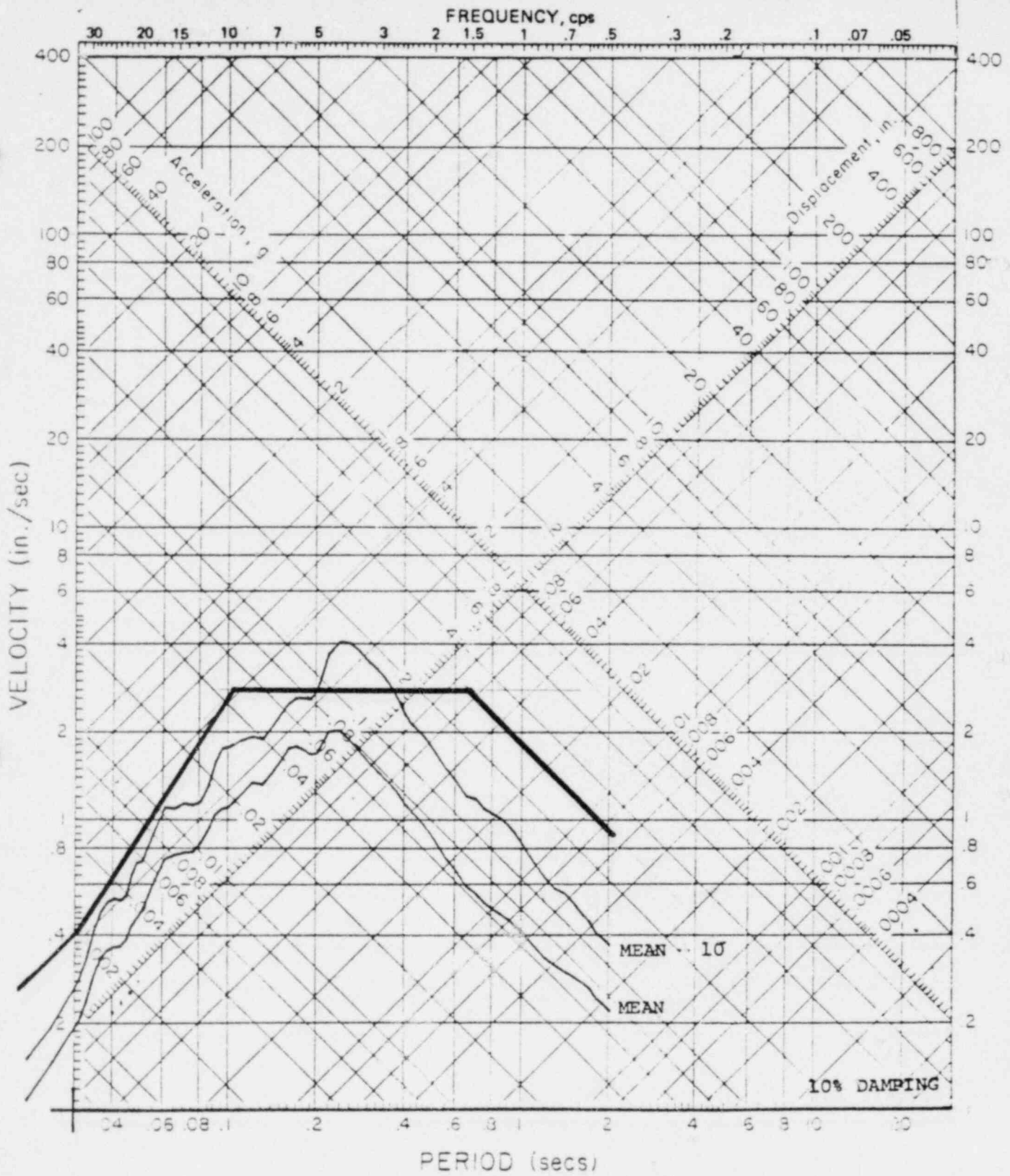
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FIGURE 4. ARITHMETIC MEAN AND MEAN PLUS 10% PSRV SPECTRA CALCULATED FROM 12 ACCELEROGRAMS RECORDED AT CDMG STATIONS 6 AND 8.



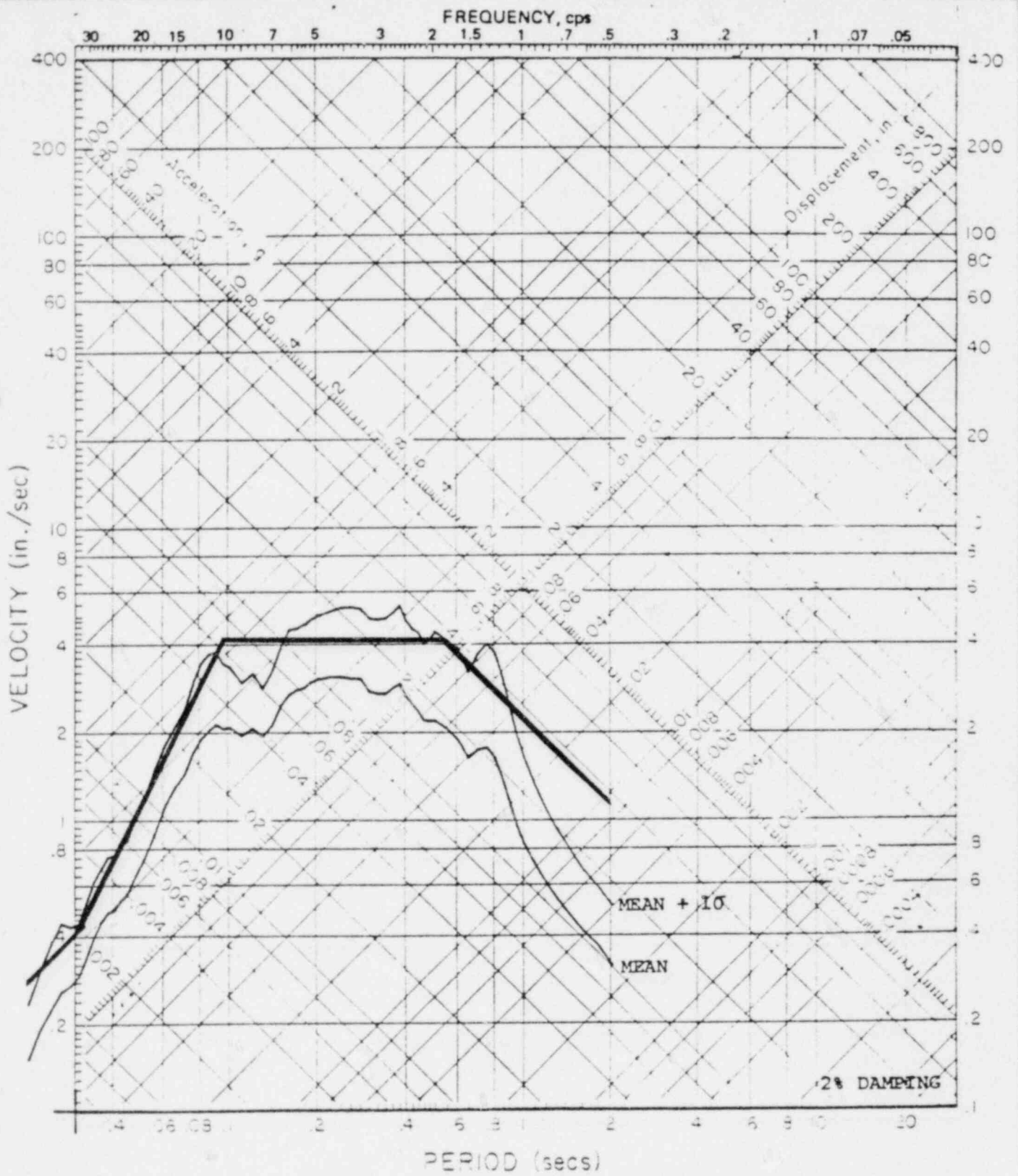
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FIGURE 5. ARITHMETIC MEAN AND MEAN PLUS 10% 5% PSRV SPECTRA CALCULATED FROM 12 ACCELEROGRAMS RECORDED AT CDMG STATIONS 6 AND 8.

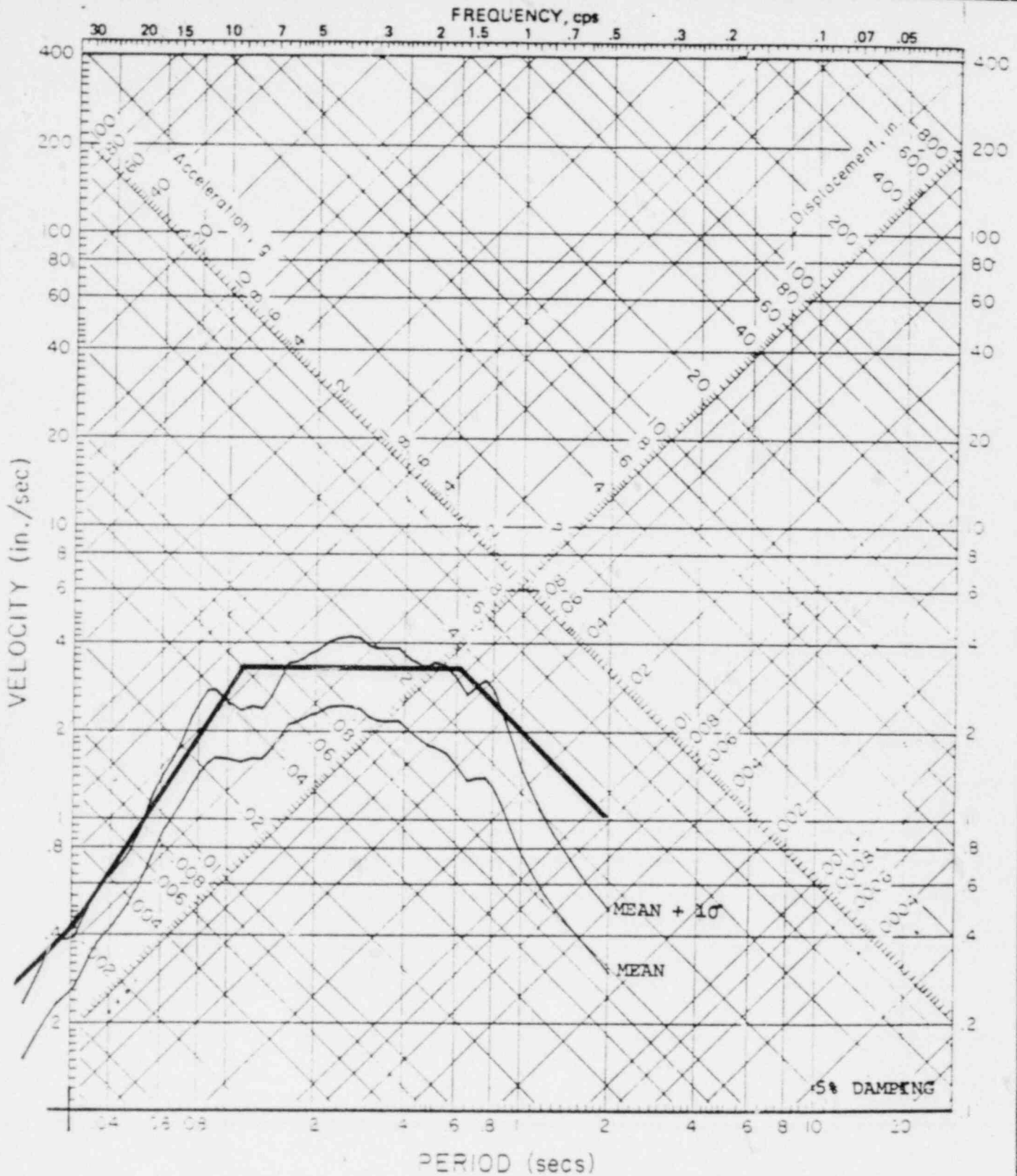



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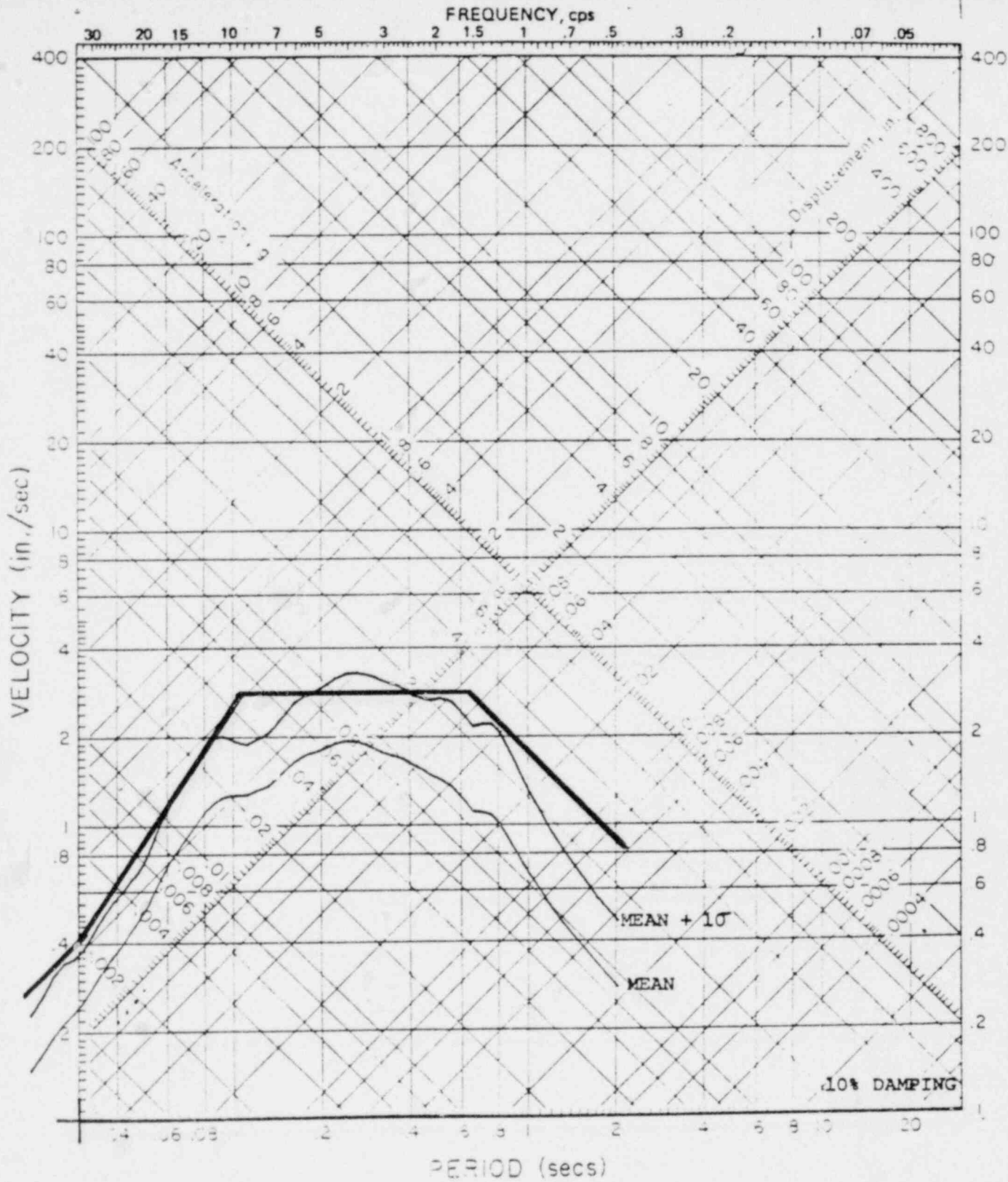
FIGURE 6. ARITHMETIC MEAN AND MEAN PLUS 1σ 10% PSRV SPECTRA CALCULATED FROM 12 ACCELEROGRAMS RECORDED AT CDMG STATIONS 6 AND 8.



 The Earth Technology Corporation	PROJECT NO.:
FIGURE 7. ARITHMETIC MEAN AND MEAN PLUS 1σ 2% PSRV SPECTRA CALCULATED FROM 44 COMPONENTS OF OROVILLE AFTER-SHOCKS.	



 The Earth Technology Corporation	PROJECT NO.:
FIGURE 8. ARITHMETIC MEAN AND MEAN PLUS 1σ 5% PSRV SPECTRA CALCULATED FROM 44 COMPONENTS OF OROVILLE AFTERSHOCKS.	



PROJECT NO.:

FIGURE 9. ARITHMETIC MEAN AND MEAN PLUS 1σ 10% PSRV SPECTRA CALCULATED FROM 44 COMPONENTS OF OROVILLE AFTER-SHOCKS.

ESTIMATES OF PEAK ACCELERATION USING
BRUNE SEISMIC SOURCE MODEL

Robin K. McGuire

In accompanying testimony entitled "Mathematical Model Used to Estimate Peak Acceleration at Virgil C. Summer Nuclear Station," a methodology is described with which peak accelerations of ground motion can be estimated for reservoir-induced seismicity (RIS). These peak accelerations can be used to scale earthquake response spectra for $M_L = 4$ events, as described in "Response Spectra Shapes for Reservoir-Induced Seismicity at Virgil C. Summer Nuclear Station."

Table A1 shows root-mean-square and peak accelerations for an earthquake of $M_L = 4.0$. This is the largest earthquake thought possible by the Applicant, and is certainly the largest which can occur at depths less than 3 km (see testimony by Nuttli). Moreover, the shallowest depth at which an $M_L = 4$ event can occur is at 2.3 km (see testimony by Nuttli). Thus, for the earthquake to occur at this distance from the facility, it must occur directly under it. If such earthquakes occur at all, the probability that they occur within 2 km horizontal distance is extremely low, less than 1/1000 per year (see accompanying testimony entitled, "Probability Estimates of Seismicity and Ground Motion at Virgil C. Summer Nuclear Station"). This conclusion is based on an analysis of both site-specific and regional RIS data.

A depth of 2.3 km and a horizontal distance of 2 km implies a hypocentral distance of 3 km. Table A1 shows estimates of peak acceleration for both 2.3 km and 3 km, in order to give a range of values; the first corresponds to an extreme upper-bound estimate, and the second to a low probability estimate corresponding to an unlikely event.

The stress drop used to estimate peak acceleration deserves special mention. The extreme value obtained from strong motion observations is 25 bars, as documented in FSAR Question 361.23. As discussed in that document, this is a conservative estimate for the following reasons: 1) it corresponds to the largest value deduced from the available records, and probably represents extremes in radiation pattern and attenuation as well as in source parameters; and 2) all available records indicate an average stress drop on the order of 15 bars, and even this value is probably biased on the high side (low stress drop earthquakes have not triggered the instrument to produce a usable record).

Although a stress drop of 25 bars is conservatively high, Table A1 gives estimates of peak acceleration for stress drops of 50 bars. This is done to give a range of results and to indicate that even such a large stress drop does not represent a hazard to the facility. Several values of cut-off frequency are shown in the Table, again to give a range of results. The most appropriate value to use for scaling response spectra is 20 or 25 hz, because that is the frequency at which standard earthquake ground motions are filtered and from which spectral shapes are derived.

Previous work by the Applicant, reported in the Supplemental Seismologic Investigation and in testimony by Chen, indicates that accelerations up to 0.22 g (216 cm/sec^2) for an $M_L = 4.0$ to 5.0 earthquake represents no seismic safety hazard. All of the peak accelerations listed in Table A1 are below or very close to this acceleration. All values listed for a distance of 3 km (a more reasonable representation of an unlikely event than the extreme of 2.3 km) indicate peak accelerations less than 0.22g, even under extreme and unrealistic choices of values for cut-off frequency and stress drop.

The conclusion from these estimates, and from the studies of ground motion effects referenced above, is that RIS of magnitudes up to $M_L = 4.0$ represent no safety concern to the facility. This is the case over a wide range of choices of parameter values which influence ground acceleration estimates.

TABLE A1
 RMS AND PEAK ACCELERATION
 ESTIMATE FOR $M_L = 4.0$

R, km	f_u , hz	$\Delta\sigma$, bars	a_{rms} , cm/sec ²	a_{peak} , cm/sec ²
2.3	20	25	49	119
2.3	25	25	55	139
2.3	30	25	60	157
2.3	20	50	86	200
2.3	25	50	97	235
2.3	30	50	107	266
3.0	20	25	37	90
3.0	25	25	42	105
3.0	30	25	46	119
3.0	20	50	65	152
3.0	25	50	74	178
3.0	30	50	81	202

Note: Other parameter values used in analysis are:
 $Q = 1000$, $\beta = 3.2$ km/sec, $\rho = 2.7$ g/cm³

APPLICANT EVALUATION OF JOYNER AND FLETCHER REPORT ON
VIRGIL C. SUMMER NUCLEAR STATION
SEISMICITY STUDIES

Robin K. McGuire

Joyner and Fletcher have reviewed the "Supplemental Seismologic Investigation" Report, the Safety Evaluation Report, and Section 361 of the Final Safety Analysis Report for the Virgil C. Summer Nuclear Station. Their response is contained in a memorandum to Morris dated September 9, 1981. Joyner and Fletcher apparently have not read transcripts of ACRS subcommittee meetings or of ASLB hearings to date. The issues raised by Joyner and Fletcher are caused by misinformation or misinterpretation (indeed, Joyner and Fletcher state that, "... we did not have sufficient time for a thorough review ..."), and deserve a direct response by the Applicant to clarify the record. The form of this response follows the issues raised by Joyner and Fletcher, in order.

MAXIMUM MAGNITUDE OF THE RESERVOIR-INDUCED EARTHQUAKES

Joyner and Fletcher give values ranging from 30 to 44 bars for the August 27, 1978 earthquake. Joyner and Fletcher give three methods by which they have calculated these values: rms accelerations, numerical integration of the squared spectrum, and a "straightforward application of the Brune model," but no formulas or parameter values are given. Although it is not clear from Joyner and Fletcher's report, the major difference between their estimates of stress drop for the 1978 earthquake and those of the Applicant is the assumption of the highest frequency that can be recorded and documented in the digitization process (Fletcher, personal communication 1981). Since stress drop is an important parameter, and one which has been the subject of some debate, this point deserves further elaboration.

The peak accelerations recorded on an accelerometer during an earthquake are a function of the highest frequency which the instrument and record processing procedure can transmit, among other factors. For records obtained very close to sources of high frequency energy (e.g., rock bursts), accelerations can be almost arbitrarily high if the instrument and processing procedures are adequate to transmit the high

frequencies of motion at which high accelerations occur. McGarr et al. (1981) documented accelerations as high as 12g during mine tremors in South Africa, where the magnitudes were less than 1.5 and source-to-site distances were several hundred meters. These peak accelerations occurred at frequencies of several hundred hz, and the instruments were specially designed to record ground motion at these high frequencies.

Typical strong motion instruments, including the one installed at Jenkinsville, have a natural oscillation frequency of 25 hz, meaning that the instrument itself tends to damp out motion at higher frequencies. Joyner and Fletcher have taken 25 hz as the upper limit of motion that can be recorded. However, accelerographs can easily record frequencies higher than their natural frequency. The upper solid curve in Figure 1 shows the response of an accelerograph with natural frequency of 25 hz and damping 0.6 of critical (the characteristics of the SMA-1 accelerograph at Jenkinsville, according to Brady et al., 1981) plotted as a function of frequency. Not only can the accelerograph itself record frequencies higher than 25 hz, but standard record processing procedures (including those used by Brady in the above reference) "correct" for the instrument response, effectively by dividing the recorded ground motion at each frequency by the ordinate on Figure 1. This effect can be significant: the peak acceleration of the "2nd aftershock" record, 90° component, documented by Brady et al. (1981), increases 35 percent due to instrument correction procedures.

Furthermore, the Jenkinsville data indicate that frequencies higher than 25 hz have been recorded. Brady et al (1981) find that, "... these (Jenkinsville) records have frequencies as high as 25 and 30 hz." A perusal of the Brady et al. (1981) document shows that the August 27, 1978 record, 90° component, has a peak acceleration with a 33 hz frequency, and the "2nd aftershock" record, 90° component, has a peak acceleration with a 40 hz frequency.

That there is substantial energy^{1/} in the ground motion recorded at Jenkinsville can also be inferred from the plots of response spectra 1/ i.e., Above 25 Hz.

provided by Brady et al. (1981), one of which (August 27, 1978 earthquake 90° component) is reproduced here as Figure 2. Although spectra are only plotted down to a period of 0.04 seconds (up to a frequency of 25 hz), it is evident that there is no decrease of energy near 25 hz, and it is safe to assume that the spectra, if plotted at higher frequencies, would continue horizontally to frequencies as high as 35 or 40 hz, and this would indicate ground motions at those frequencies.

The Applicant has used an upper frequency of 40 hz to accurately characterize these records, making it clear that it is the record corrected for instrument response and digitized at 500 points per second to which this upper bound applies^{1/}. The choice of upper bound f_u affects estimates of stress drop $\Delta\sigma$ in the following way:

$$\Delta\sigma = C \frac{a_{rms}}{(f_u - f_o)^{1/2}} \quad (1)$$

where a_{rms} is the root-mean-square acceleration from the record and f_o is the corner frequency (see the Appendix for a derivation of this).

Both the Applicant and Joyner and Fletcher have used a lower bound frequency f_o of about 10 hz (the issue of corner frequency is addressed in detail below). Therefore, for the same observation of a_{rms} , the choice of $f_u = 25$ hz leads Joyner and Fletcher to an estimate of $\Delta\sigma$ which is high relative to $f_u = 40$ hz, by the factor:

$$\frac{\Delta\sigma \text{ (J \& F)}}{\Delta\sigma \text{ (Applicant)}} = \frac{(40-10)^{1/2}}{(25-10)^{1/2}} = 1.4 \quad (2)$$

This explains why Joyner and Fletcher obtain $\Delta\sigma = 35$ bars for the August 27, 1978 earthquake, and the Applicant obtains $\Delta\sigma = 25$ bars.

^{1/} We note that Dr. Luco suggests an upper bound frequency of 50 Hz, which yields even lower values of stress drop than those of the Applicant.

Joyner and Fletcher have used an upper-bound frequency equal to the nominal frequency of the instrument; the Applicant has accounted for the higher frequencies evident in the strong motion record.

As a separate issue, Joyner and Fletcher assert that the Applicant did not correctly account for the corner frequency in making estimates of $\Delta\sigma$. While this is implied by the equations in section 361 of the FSAR, which Joyner and Fletcher reviewed, the effect of corner frequency was examined and found not critical by the Applicant. The Appendix to this report derives the theory with which the effect of corner frequency can be included in estimating $\Delta\sigma$; estimates using this theory were presented to the ACRS seismic subcommittee on February 26, 1981. Table 1 reproduces the data presented at that meeting, which is a matter of public record. Using the appropriate corner frequency f_0 , the stress drops derived for the August 27, 1978 earthquake are still on the order of 20 bars. Thus it is the Applicant's position that 25 bars is an appropriate and conservative stress drop to use for characterizing earthquakes at Monticello Reservoir for the purposes of estimating strong ground motion characteristics.

Joyner and Fletcher have reviewed the Applicant's arguments on stress barriers, stress heterogeneities, and material properties defining maximum rupture dimensions, and find these arguments "... unconvincing." It is not clear what alternative physical explanation Joyner and Fletcher have for the observations that have been made, nor why they do not accept the Applicant's explanations. In any case, Joyner and Fletcher base their estimate of the maximum rupture dimension and of the associated magnitude on the spatial extent of observed seismicity, without consideration of whether the seismicity "lines up" or indicates any through-going structure (in fact it does not). Such an analysis is unsupported by observations anywhere in the world, to the Applicant's knowledge, i.e., there is no location where swarm-like seismicity has indicated the size of a later, larger earthquake. Frequently in seismology the locations of after-shocks are used to infer the dimensions of a main shock (even this has been suggested as giving a conservatively

large estimate of the main shock area). This is a far different procedure from using the location of diffuse seismicity to infer a main shock area. What has frequently been done by investigators is to use the length of an identified fault to estimate a maximum magnitude, and here only one-half of the entire fault length is presumed to rupture. Thus Joyner and Fletcher's procedure is without validity in terms of world-wide empirical observations, does not constitute an accepted method, and has not had the benefit of peer review.

In calculating the magnitude associated with source radii of 1 and 1.4 km, Joyner and Fletcher have used a stress drop of 40 bars. Since magnitude is proportional to the logarithm of stress drop in this calculation, this leads to Joyner and Fletcher magnitude estimates that are only marginally higher (~0.1 magnitude units) than those supplied by the Applicant at the request of NRC.

The experience of induced earthquakes at Denver is entirely irrelevant to the issues at Monticello. The Denver earthquakes were caused by cyclical fluid injection in deep wells; the correlation of earthquakes with injection is a point made by the reference cited by Joyner and Fletcher (Healy et al., 1968). Thus at Denver the causative mechanism was cyclical. At Monticello there has been a one time change in water elevation^{1/} during operations, lake fluctuations will not exceed about 2 meters total range. Thus the causative mechanisms^{2/} of the two phenomena are fundamentally different, and to suggest that the experience at one site would or should guide us at the other is inapposite.

GROUND MOTION ESTIMATES

The first difference (concerning digitization rate) mentioned by Joyner and Fletcher between their and the Applicant's ground motion analysis is not a difference at all. In 1980 the Applicant used the records digitized at 100 points per second to estimate stress drop during the August 27, 1978 event, because at that time (when the relevant parts of Section 361 of the FSAR were prepared), these were the only data available. In February 1981 the digitizations at 500 points per second

1/ i.e., Initial filling of Monticello Reservoir

2/ In the sense of the scale of the changes. Also very important are the differences in the hydrologic and tectonic regimes.

were made available by USGS (Brady et al., 1981) and the Applicant confirmed that its analysis was appropriate for the higher digitization rate. Table 1 reproduces data presented at the February 26, 1981 ACRS subcommittee meeting which shows ground motion estimates made by the Applicant which are in agreement with Monticello earthquake records digitized at 500 points per second. Thus the Applicant can and has explained the factor-of-two difference in peak accelerations due to digitization rate.

Where the Applicant's procedure does differ from that of Joyner and Fletcher is in the implied digitization rate associated with the peak acceleration used to characterize ground motion for seismic analysis of the facility. To determine the appropriate digitization rate, one must consider how the peak acceleration is to be used to generate response spectra for structural analysis. Thus the structural engineering considerations cannot "... be kept separate from the seismological analysis," as Joyner and Fletcher wish.

The manner in which response spectra are derived for the seismic design and analysis of nuclear facilities is straightforward: (1) an expected peak acceleration is selected corresponding to the largest ground motion anticipated, (2) an effective acceleration is calculated from the peak acceleration, and (3) a response spectrum is scaled to that effective acceleration. For the Virgil C. Summer facility, step (2) has conservatively been ignored, i.e., peak acceleration has been assumed to equal effective acceleration.^{1/} For tectonic earthquakes, a broad-banded spectrum is used to represent the wide frequency content of the motion. For reservoir-induced earthquakes at Monticello, the important events will occur close to the facility; in this case, appropriate high frequency spectra have been developed as suggested by Regulatory Guide 1.60. This development is documented in Section 361 of the FSAR.

For the high frequencies of interest, it is the high frequency components of the structure which are of concern. These frequencies lie

^{1/} The effect of this conservatism is addressed in separate testimony by Dr. Blume.

in what is often termed the "acceleration-amplification" portion of the spectrum, that is, amplitudes of response are most sensitive to the peak acceleration of the input motion, rather than by the peak velocity or peak displacement.

The mathematical representation of this two-step procedure to calculate high frequency structural response is as follows:

$$a_{res} = a_p \times \frac{a_{res}}{a_p} \quad (3)$$

where a_{res} is the structural response in terms of maximum response acceleration, and a_p is peak ground acceleration (step (1) above). The ratio on the right-hand-side is step (3) above, the "acceleration amplification factor" used to determine both standard spectral shapes (e.g., Regulatory Guide 1.60) and the spectral shapes used on this project to represent reservoir-induced earthquakes.

It should be evident that the peak acceleration estimated for the earthquakes of concern (the first " a_p " on the right-hand-side of equation (3)) should be determined in a consistent manner with the value of a_p used to calculate the acceleration amplification factor. This implies, among other things, that records processed in the same manner should be used to calculate a_p and the ratio a_{res}/a_p . In determining the appropriate ratio of a_{res}/a_p for near-source, hard rock sites, records digitized at 50 points per second (Johnson, personal communication, 1981) were used. It follows that peak accelerations for reservoir-induced earthquakes should be estimated for a digitized record at 50 points per second, not for some other digitization rate.

The Applicant has estimated values of a_p in an appropriate and consistent way. The effect of digitization at 50 points per second was accounted for by using an upper frequency f_u of 20 hz for the estimates of peak acceleration. For comparison, $f_u = 40$ hz is appropriate to estimate peak acceleration from a 500 points-per-second record. This is illustrated in Table 1, as described above.

Joyner and Fletcher's procedure only uses the peak accelerations of the 500 points-per-second digitized record, and makes no attempt to account for other digitizing rates used in scaling response spectra. Under this procedure, if the instruments of McGarr et al. (1981) had recorded the August 27, 1978 earthquake with frequencies up to several hundred hz, and a peak acceleration of several g had been obtained, this high acceleration would be scaled up to estimate peak acceleration during a $M_L = 4.5$ earthquake. Such an extreme hypothetical example illustrates why, in addition to other considerations such as effective peak acceleration, instrument characteristics, record processing and correction procedures, and response spectrum scaling methods must be incorporated into the estimates of peak acceleration, as the Applicant has done.

In summary, the theory to estimate peak accelerations used by the Applicant is consistent with instrumental observations at Jenkinsville, with digitized versions of those observations made by USGS, and with the way in which response spectra should be scaled. Further, this methodology for calculating reservoir-induced earthquake response spectra is consistent with the methodology recommended for tectonic earthquakes (Regulatory Guide 1.60). The implications by Joyner and Fletcher that (a) the Applicant has not accounted for strong-motion records at Monticello digitized at 500 points per second, and (b) the peak accelerations from these records are the only data on which seismic evaluations should be made, are erroneous, and do not account for the way peak accelerations are used to evaluate structures.

The second difference mentioned by Joyner and Fletcher is in the area of saturation of ground motion with distance. Joyner and Fletcher imply that the Applicant has changed its position on this issue, but this is decidedly not the case, and Joyner and Fletcher's confusion apparently comes from misreading the record. The Applicant's position is illustrated in Figure 3. At a distance $R < 4r$, the use of a point-source model "... is not strictly applicable; these values (calculated at these distances) are therefore conservative." This is stated in Applicant's Table 361.17.4-2. This is shown in Figure 3 as point A, where the solid line deviates from the dotted line. At closer distances, "... extrapolation of the far-field model to a source-to-site distance of one source

diameter ($R=2r$) gives a reasonable approximation to the saturation level." This is stated in Appendix XI of the Supplemental Seismological Investigation Report. This statement is illustrated in Figure 3 as point B, where the dotted line and dashed line cross. Whether or not Joyner and Fletcher agree with these statements, they are consistent, and the Applicant has not, "introduce(d) distance saturation in a slightly different way in Appendix XI ...," as Joyner and Fletcher state.

The Applicant agrees with Joyner and Fletcher's statement that, "... the assumption that the saturation level corresponds to the value computed at any fixed multiple of the source radius leads to the unpalatable (sic) conclusion that the saturation level decreases with magnitude." In fact the Applicant noted this effect in Appendix XI of the Supplemental Seismologic Investigation: "... earthquakes of $M_L = 5.0$ and 5.5 would have faulting diameters of 3.6 and 6.3 km, respectively. A blind application of the distance limits discussed above ($R=2r$) yield peak accelerations of $0.17g$ and $0.13g$, respectively. This does not imply that saturated peak accelerations decrease with magnitude; rather, other factors are important." Among these is the observation that smaller magnitude ($M_L \lesssim 5$) earthquakes are not generally known to rupture the earth's surface, particularly in the Eastern U.S. Thus it is unlikely that a site on the earth's surface would ever be in the near-field, at $R=2r$, from such an event. Use of the $R=2r$ distance saturation limit is thus conservative for such earthquakes.

The Applicant notes that Joyner and Fletcher do not propose any alternative to choosing saturation distance by scaling by source size. Further, Joyner and Fletcher's mention of $R=r$ as the saturation distance appears to be motivated more by where ground motions are anticipated to decrease from any saturation level (point C on Figure 3) than what distance is appropriate to extrapolate point source models.

The peak acceleration values listed in Joyner and Fletcher's Table 1 are calculated by the following equation:

$$a_p(M) = a_p(2.8) 10^{.25(M-2.8)} \quad (4)$$

where $a_p(M)$ is the predicted peak acceleration for magnitude M and $a_p(2.8)$ is the larger of the two horizontal peak accelerations recorded during the August 27, 1978 earthquake (0.26g). Implicit in equation (4) is the use of a source-to-site distance of 0.7 km for all earthquakes. It is appropriate to make several comments on this methodology.

1. The Applicant knows of no other major facility where the proposed peak accelerations for seismic analysis are based on a single component of one ground motion record, and use such a simple scaling relation as equation (4). The physical parameters which are associated with reservoir-induced earthquakes at Monticello are not addressed adequately.
2. The values from Joyner and Fletcher are derived from an instrumental frequency peak acceleration not appropriate for scaling response spectra.
3. Joyner and Fletcher's Table 1 is critically dependent on the distance between the August 27, 1978 event and the Jenkinsville accelerometer, which was a random occurrence. Suppose this distance had been twice as far, and had caused 0.13g at the accelerometer; would they recommend values half as large as those in Table 1? In effect Joyner and Fletcher have established ground motion saturation levels and distances on the basis of a single chance occurrence.
4. Joyner and Fletcher present no observed data in the magnitude and distance range of Table 1 to support their estimates.
5. There is no method suggested by Joyner and Fletcher to limit the magnitudes for which peak accelerations can be calculated by equation (4).

The Joyner and Fletcher method of scaling peak ground acceleration (a_p) and velocity (v_p) with magnitude (M) can also be written:

$$\log_{10} a_p = -1.285 + 0.25 M \quad (5)$$

$$\log_{10} v_p = -1.038 + 0.50 M \quad (6)$$

where a_p in equation (5) is in units of gravity and v_p is in cm/sec.

It is instructive to compare these results, by extrapolation, with those given by Joyner and Boore (1981). This is an appropriate comparison because the magnitude coefficients 0.25 and 0.50 in equations (5) and (6) were taken by Joyner and Fletcher from Joyner and Boore (1981). For the case where the distance to the surface projection of the fault rupture is zero, Joyner and Boore (1981) obtain

$$\log_{10} a_p = -1.902 + 0.249 M \quad (7)$$

$$\log_{10} v_p = -1.282 + 0.489 M \quad (8)$$

Equations (7) and (8) are supported by near-field data for earthquakes in the magnitude range 5.0 to 6.5.

Equations (5) through (8) are evaluated in Table 2 for various magnitudes. Results of extrapolation are indicated by asterisks. The results of Joyner and Fletcher are not similar to those of Joyner and Boore (1981). For magnitude 6.5, equations (5) and (6) yield peak ground acceleration and velocity greater than have ever been measured for naturally-occurring or reservoir-induced earthquakes. For all magnitudes, the results of Joyner and Fletcher greatly exceed those of Joyner and Boore (1981).

There are several reasons for this difference. The Joyner and Fletcher equations are based only on a single horizontal component of one earthquake record. The peak acceleration and velocity of this horizontal component occurred during a very high frequency pulse (and should not be used to scale response spectra, as discussed above). Further, the motion recorded at Monticello Dam is undoubtedly amplified over free-field conditions due to the topographic effects (the instrument sits on an earth dam abutment). The Joyner and Boore (1981) equations are based on a large number of earthquake records from California, including near-field records, and reflect free-field conditions. Thus they are more appropriate to estimate peak accelerations and velocities for important facilities such as nuclear power plants.

SUMMARY

Joyner and Fletcher's review of Virgil C. Summer Nuclear Station seismicity studies is based, in part on a misinterpretation of certain documents and, perhaps in part, on not having had access to complete transcripts of ACRS subcommittee meetings and ASLB hearings. Two concerns of Joyner and Fletcher, the effect of corner frequency on the stress drop estimate for the August 27, 1978 earthquake, and the digitization of the record from that event at 500 points per second, are not issues at all. The Applicant has analyzed both in detail, and its recommendations incorporate those analyses. The estimates of maximum magnitude made by Joyner and Fletcher are based on the area^{1/}of observed seismicity; such a method is not valid in the seismic design of important facilities. The third area of Joyner and Fletcher's concern, ground motion saturation, involves significant interpretation and judgment, and the Applicant has acknowledged this. Joyner and Fletcher offer no alternative methods to determine the distance within which ground motion amplitudes are saturated, except to use the distance between the source and recording site for the August 27, 1978 event, a chance occurrence. Further, Joyner and Fletcher use a single component peak acceleration from that event's record to scale peak acceleration and make recommendations. Such a procedure is without precedent. It takes no account of important parameters such as earthquake stress drop, distance to larger events, instrument and record processing procedures, and scaling of response spectra from the predicted peak accelerations. Joyner and Fletcher state that the methods of Newmark and Hall (1969) can be used to compute response spectra given its estimates of peak acceleration (and velocity), but the broad-band amplification factors of Newmark and Hall (1969) would be wholly inappropriate for what Joyner and Fletcher admit would be high frequency motions. This illustrates a position which the Applicant has taken since the beginning: the estimates of peak acceleration must be made in light of the overall design problem and local conditions at the facility.

1/ i.e., Spatial extent.

REFERENCES ^{1/}

- Brady, A.G., P.N. Mork, and J.P. Fletcher (1981), "Processed Accelerograms from Monticello Dam Jenkinsville, South Carolina 27 August 1978, and from Later Shocks", USGS Open-File Report 81-448, March, 35 pp.
- Healy, J.H., W.W. Rubey, D.T. Griggs, and C.B. Raleigh (1968), "The Denver Earthquakes," Science, vol. 161, pp. 1301-1310.
- Hudson, D.E. (1979), "Reading and Interpreting Strong Motion Accelerograms", Earthquake Eng. Res. Inst. Monograph, 112 pp.
- Joyner, W.B., and D.M. Boore (1981), "Peak Horizontal Acceleration and Velocity from Strong Motion Records including Records from the 1979 Imperial Valley Earthquake," Bull. Seis. Soc. Am., vol. 71, Dec. (in press).
- McGarr, A.R., W.E. Green and S.M. Spottiswoode (1981), "Strong Ground Motion of Mine tremors: Some implications for near-source ground motion parameters", Bull. Seis. Soc. Am., vol. 71, pp. 295-319.
- McGuire, R.K. and T.C. Hanks (1980), "RMS Accelerations and Spectral Amplitudes of Strong Ground Motion During the San Fernando Earthquake", Bull. Seis. So. Am., vol 70, pp. 1907-1919.
- Newmark, N.M., and W.J. Hall (1969), "Seismic Design Criteria for Nuclear Reactor Facilities", Proceedings, 4th World Conference on Earthquake Engineering, Santiago, Chile.

^{1/} Additional Reference: Blume, J.A., "On Instrumental Versus Effective Acceleration, and Design Coefficients", Proceedings, 2nd USA National Earthquake Conference, August, 1979.

APPENDIX

Derivation of a_{rms} for case where lower bound is finite:

$$\tilde{a}(f) = \begin{cases} (.85) \frac{\Delta\sigma r}{\rho R B} \exp\left(-\frac{\pi f R}{Q_B}\right) \left(\frac{f}{f_0}\right)^2 & f < f_0 \\ (.85) \frac{\Delta\sigma r}{\rho R B} \exp\left(-\frac{\pi f R}{Q_B}\right) & f \geq f_0 \end{cases}$$

where symbols are as defined in Section 361 of the FSAR.

$$a_{rms}^2 = \frac{1}{T_d} \int_0^{T_d} |a|^2 dt = \frac{1}{\pi T_d} \int_0^{2\pi f_u} |\tilde{a}(\omega)|^2 d\omega$$

$$= \frac{c^2}{\pi T_d} \left\{ \int_0^{2\pi f_0} \exp\left(-\frac{2\pi f R}{Q_B}\right) \left(\frac{f}{f_0}\right)^4 d\omega + \int_{2\pi f_0}^{2\pi f_u} \exp\left(-\frac{2\pi f R}{Q_B}\right) d\omega \right\} \quad \underline{1/}$$

where $c = (.85) \frac{\Delta\sigma r}{\rho R b}$ and $2\pi f = \omega$

Neglecting, conservatively, the first integral,

$$a_{rms}^2 = \frac{c^2}{\pi T_d} \left[-\frac{Q_B}{R} \exp\left(\frac{-\omega R}{Q_B}\right) \right]_{2\pi f_0}^{2\pi f_u}$$

$$= \frac{c^2}{\pi T_d} \frac{Q_B}{R} \left[\exp\left(\frac{-2\pi f_0 R}{Q_B}\right) - \exp\left(\frac{-2\pi f_u R}{Q_B}\right) \right]$$

so that

$$a_{rms} = (.85)(.37) \frac{\Delta\sigma}{\rho R^{1.5}} \sqrt{\frac{2Qr}{2.34}} \left[\exp\left(\frac{-2\pi f_0 R}{Q_B}\right) - \exp\left(\frac{-2\pi f_u R}{Q_B}\right) \right]^{1/2}$$

1/ Numerical correction to formula.

For f_o small and f_u large, the above is the same as equation (9) in McGuire and Hanks (1980). For f_o non-negligible and f_u non-infinite, and for typical values of R, Q, and β :

$$\frac{2\pi f_u R}{Q\beta} < 0.1$$

so:

$$a_{rms} = (.85)(.37) \frac{\Delta\sigma}{\rho R^{1.5}} \sqrt{\frac{2Qr}{2.34}} \left[\frac{2\pi R}{Q\beta} (f_u - f_o) \right]^{1/2}$$

If $\Delta\sigma$ is being estimated from recorded a_{rms} , the above equation can be inverted to give:

$$\Delta\sigma = \frac{\rho R^{1.5} a_{rms}}{(.85)(.37)} \left[\frac{4\pi Rr}{2.34 \beta} (f_u - f_o) \right]^{-1/2}$$

TABLE 1

DATA AND ESTIMATES ON MONTICELLO EARTHQUAKES
PRESENTED TO ACRS SUBCOMMITTEE ON FEBRUARY 26, 1981

EVENT	M_L	Δ , KM	DEPTH, KM	R, KM	F_U , HZ	$\Delta\sigma$, BARS	A_{RMS} , CM/SEC ²	A_{PEAK} , CM/SEC ²
AUGUST 27, 1978 1023 UTC	2.8	0.66	0.1	0.67	40	22	104	221
	OBSERVATIONS:						108	225
AUGUST 27, 1978 1023 UTC	2.8	0.66	0.1	0.67	20	17	53	96
	OBSERVATIONS:						--	93
OCTOBER 27, 1978 072E UTC (?)	2.7	1.03	0.2	1.05	40	65	106	182
	OBSERVATIONS:						100	185
OCTOBER 27, 1978 1627 UTC (?)	2.8	0.15	0.5	0.52	40	11	77	173
	OBSERVATIONS:						83	169

TABLE 2

Comparison between Joyner and Fletcher
Memorandum and Joyner and Boore (1981)

Moment Magnitude (M)	Joyner and Fletcher		Joyner and Boore	
	Eq. (1) PGA (g)	Eq. (2) PGV (cm/sec)	Eq. (3) PGA (g)	Eq. (4) PGV (cm/sec)
2.8	0.26	2.3	.06*	1.2*
4.6	0.73*	18.3*	.17*	9.3*
5.0	0.92*	29.0*	.22	14.5
5.5	1.23*	51.5*	.29	25.5
6.0	1.64*	91.6*	.39	44.8
6.5	2.19*	162.8*	.52	78.7
7.0	2.91*	289.6*	.69*	138.3*
7.5	3.89*	514.9*	.92*	242.8*

* Extrapolated

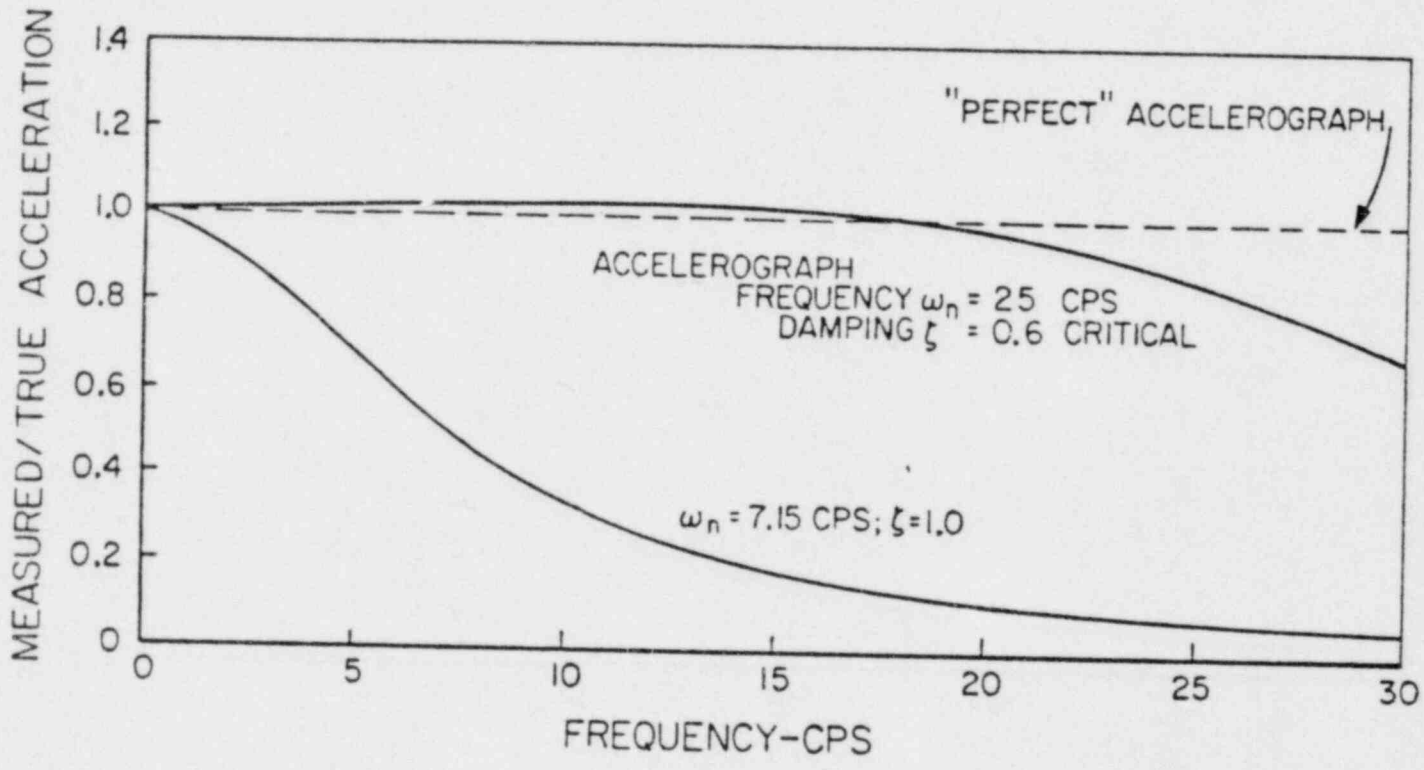
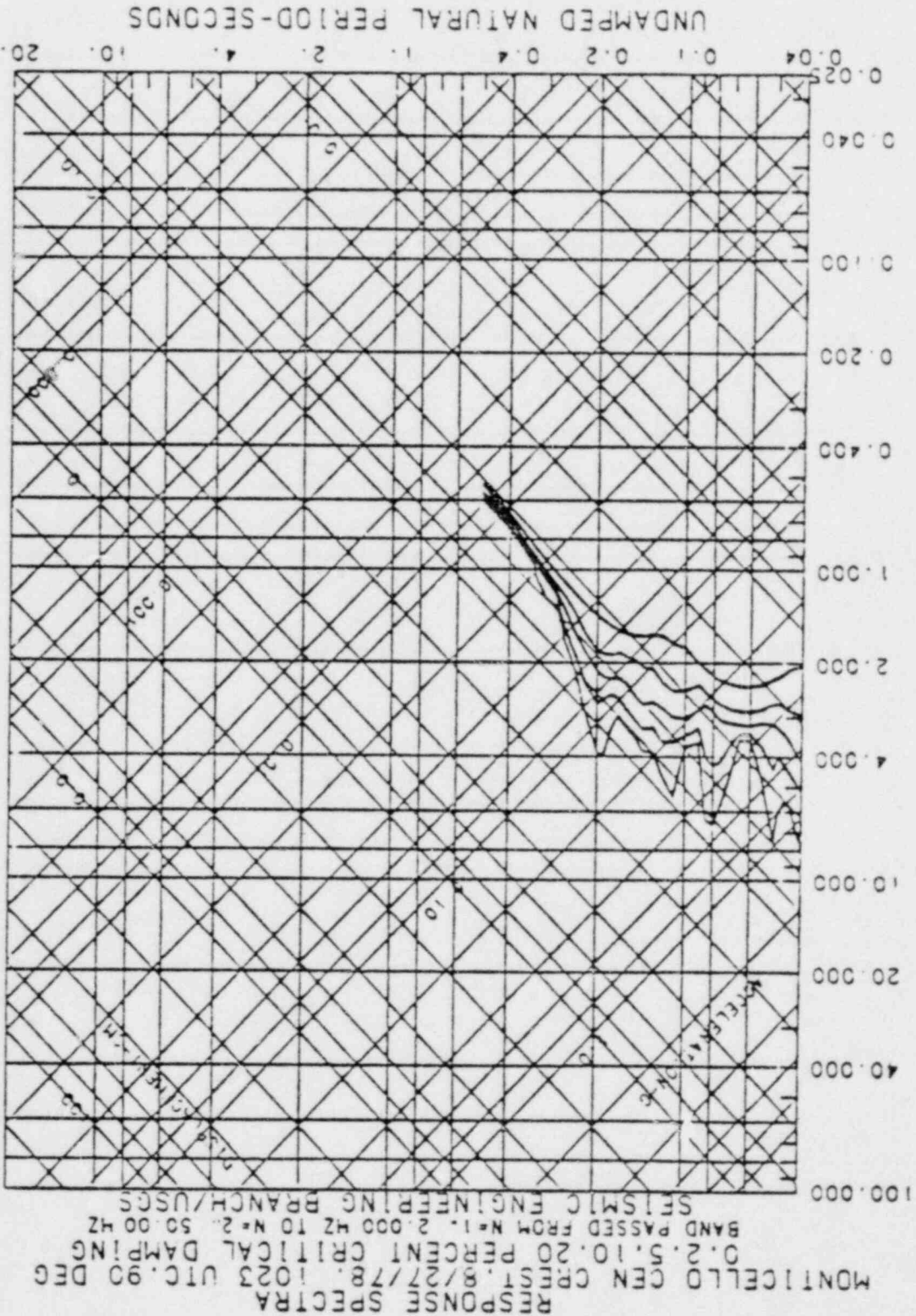


FIGURE 1

TYPICAL ACCELEROGRAPH RESPONSE AS A FUNCTION OF FREQUENCY
(AFTER HUDSON, 1979)

FIGURE 2



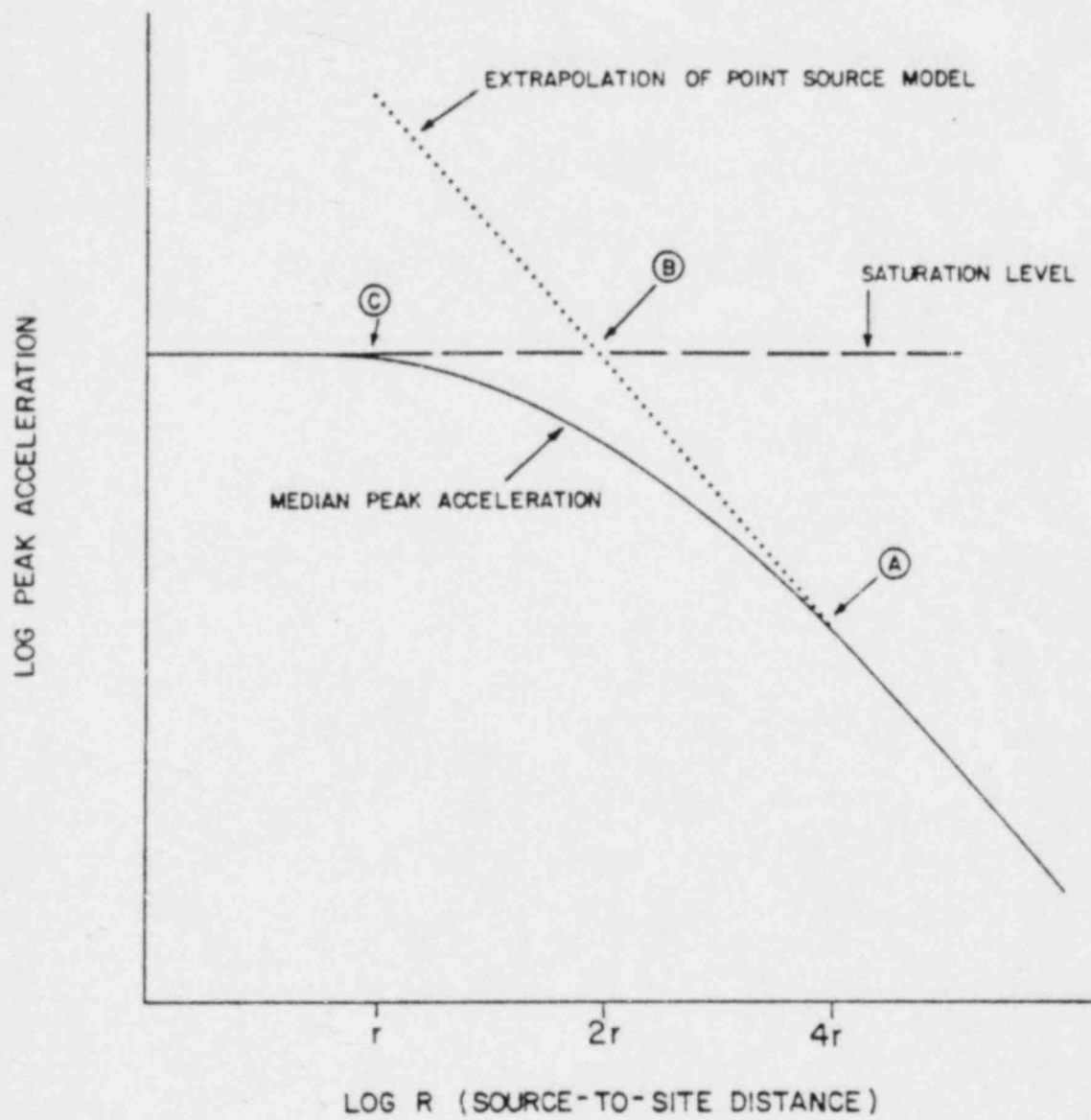


FIGURE 3

CONCEPTUAL REPRESENTATION OF
 MEDIAN PEAK ACCELERATION VERSUS DISTANCE

APPLICANT EVALUATION OF LUCO REPORT ON
VIRGIL C. SUMMER NUCLEAR STATION
SEISMICITY STUDIES
Robin K. McGuire

Prof. J. Enrique Luco has reviewed the "Supplemental Seismologic Investigation", including Appendix XI, portions of the FSAR (361.13, 361.17.4, 361.21) and portions of the Safety Evaluation Report. His response is contained in a report entitled, "Comments on Estimates of Strong Ground Motion for the V. C. Summer Nuclear Station, Unit I", dated September 23, 1981. The issues raised by Luco result from misinterpretations of the studies which have been performed by the Applicant or from the use of incorrect parameter values in his analyses. This deserves a direct response; the form of the response follows the issues raised by Luco, in order.

ON THE HANKS - MCGUIRE METHOD TO ESTIMATE PEAK ACCELERATION

Luco is correct in pointing out that in the usual characterization of earthquakes via the Brune model (which is done through observations of spectral amplitudes in the frequency domain), stress drops vary greatly and corner frequency and spectral decay at high frequencies are the subject of current discussions. However, the Applicant is not using the Brune model in this usual, frequency-domain application but in the method proposed by Hanks and McGuire (1981). This method (which uses observations of ground accelerations in the time domain) provides remarkably stable estimates of stress drops for past earthquakes (in fact, this is one of the major points of Hanks and McGuire, 1981). That this is the case is recognized later in Luco's report when he states, "The stress drop parameter appearing in the estimate of peak acceleration obtained by Hanks and McGuire has no relation with the stress drop determined by standard seismologic methods. In particular, Hanks and McGuire found that the peak accelerations for events in California could be approximated by a constant stress drop of 100 bars, independent of the stress drops calculated for these events by standard seismological methods." Hanks and McGuire also point out that, regardless of the accuracy or

inaccuracy of this methodology in characterizing corner frequency and high frequency spectral decay, the model works in predicting both root-mean-square (rms) and peak acceleration. Thus Luco's concerns about uncertainties in stress drop, corner frequency, and spectral decay are applicable to frequency domain methods, not to the Applicants' time domain method. Applicants' method does not lead to estimates of peak acceleration which are highly uncertain, as Luco implies, but rather leads to peak acceleration estimates with confidence as high as available by using other state-of-the-art methods.

ESTIMATES OF STRESS DROP

Luco implies that stress drops estimated (in the frequency domain) by standard seismological methods and presented by the Applicant are irrelevant. This is not the case. The Applicant has presented such data in Appendix VII of the Supplemental Seismologic Investigation to give as complete a picture as possible about the data which have been gathered at the site. In the context in which Luco views these data (that of the experience of Hanks and McGuire with California data), the standard stress drop data presented in Appendix VII are entirely consistent. In California, standard stress drop values range from 6 to 140 bars, and the values appropriate for rms acceleration estimates is 100 bars; at Monticello, standard stress drop values range from 1 to 5 bars and the value appropriate for rms acceleration is 25 bars.

The derivation of rms acceleration a_{rms} by Luco is slightly different from that of the Applicant because Luco explicitly includes the term $(1 + (f_0/f)^2)^{-1}$ in the integral, whereas the Applicant does not. The Applicant's derivation, which was used to calculate values of a_{rms} and peak acceleration presented to the ACRS Seismic Subcommittee on February 26, 1981, is given in the attached Appendix. To be sure, the term $(1+(f_0/f)^2)^{-1}$ appears in equations in the Applicants' FSAR section 361.17.4, but the question of whether more accuracy is gained by discarding the term and integrating from $f=f_0$, or including the term and integrating from $f=0$, is moot: the available spectra from Monticello can be fit either way with equal accuracy. The more important point is that it makes little difference

to conclusions gained by comparison of estimates to data at high digitization rates (e.g. 500 points per second, which are presumably most accurate) where $f_u=40$ or 50 hz is appropriate. This is shown in the attached Appendix (a typical effect on stress drop estimates for these records is 15%). At lower digitization rates there is more effect which accounts in part for the results Luco obtains in comparison to data presented by the Applicant in FSAR Table 361.17.4-1. In any event, conclusions from FSAR Table 361.17.4-1 are obsolete: relocation of the August 27, 1978 earthquake now indicates a source-to-site distance of 0.67 km rather than 0.8 km, and digitization at 500 points-per-second have become available. A more enlightened conclusion is obtained by comparison of predictions with records digitized at 500 points per second. Table 1 shows the appropriate parameters, observations, and predictions made by the Applicant that were presented at the ACKS Subcommittee meeting February 26, 1981. It also shows estimates made by Luco's equation (2) which indicate that a stress drop of 26 bars explains the observations (using Luco's preferred value of $f_u=50$ hz). Thus the analysis and equations developed by Luco fully support the stress drop value of 25 bars used by the Applicant for the most recent accurate data available on the August 27, 1978 earthquake. The value of 100 bars obtained by Luco in his report is based on an erroneous source-to-site distance (calculated from preliminary depth estimate) of 1.6 km; all investigators familiar with the data including USGS (Fletcher, personal communication, 1981) now agree that a source-to-site distance of about 0.67 km (as used in Table 1) is accurate.

Luco's calculation of stress drop for earthquakes at Hsinfengkiang Reservoir (his Table 2) is incorrect on two counts, thus rendering his conclusions invalid. First, Luco uses an upper frequency of 20 hz, whereas the Chinese strong motion instruments provide linear response up to 35 hz (the Applicant states this in its Appendix XI). Thus $f_u=35$ hz or greater would be more appropriate. Second, Luco used surface-wave magnitude M_s in place of local magnitude M_L . For the Chinese data ($M_L = 3$ to 5), M_s is less than M_L by about one unit. As a result, smaller source sizes and larger stress drops are obtained than is correct

for these data. In any case, the important result from the Chinese data is that stress drops determined from peak acceleration do not increase with magnitude; the Applicant made this point in Appendix XI, and Luco apparently agrees with this conclusion. One further point is that the seismological stress drop calculated for the $M_s=6.1$ main shock of Hsinfengkiang was 7.5 bars (Sheng et al., 1973), a value which is not inconsistent (given the above discussion) with the rms acceleration stress drops reported by the Applicant in Appendix XI.

Luco concludes that a stress drop of 150 bars is appropriate. Not only is this view unsupported by data, it is contradicted by data at Monticello, at Hsinfengkiang, and in California. For the first two locations, stress drops less than 25 bars are indicated; California data are irrelevant to the issue of very shallow induced seismicity and, in any case, indicate a stress drop for rms accelerations of 100 bars. Luco has presented no data which indicate that a stress drop of 150 bars is appropriate to use with the Hanks and McGuire method.

ESTIMATES OF PEAK GROUND ACCELERATION

The peak acceleration values shown by Luco in his Table 3 are invalid for the Virgil C. Summer Nuclear Station because they are based on a 100 bar stress drop. It is not surprising that Luco's Table 3 values agrees with the equations of Joyner et al. (1981) at $R=7.3$ km (zero epicentral distance)* because these equations are based on California data and Hanks and McGuire have shown that 100 bars is appropriate for California earthquakes.

Data shown by Luco in his Table 4 and his Figure 1 are misleading. He states, that "This sample may be biased towards the largest peak accelerations," (emphasis added), but in fact the sample is biased. For the Oroville data which Luco finds of particular interest, the average of the larger peak accelerations on each record for $4.0 \leq M_L < 5$ is 382 cm/sec^2 , whereas, the mean of the Oroville aftershock peak accelerations on bedrock sites for the same magnitude range is 164 cm/sec^2 (Seekins and Hanks, 1978). Thus the data presented by Luco are very much

* It appears that Luco has misinterpreted the meaning of the parameter $R=7.3$ km used by Joyner et al. (1981); this is not a depth estimate, so that R is not hypocentral distance as Luco states. Joyner et al. (1981) simply use constant R as a parameter to fit their data.

biased toward higher accelerations and should not be used to determine peak acceleration levels. Further, the Oroville aftershocks are characterized by an rms-acceleration determined stress drop of 100 bars which the Applicant has shown is inappropriate for Monticello Reservoir earthquakes.

VERTICAL PEAK ACCELERATIONS

The Applicant agrees with Luco's observation that vertical peak accelerations are generally less than horizontal peak accelerations during earthquakes of magnitude less than 6. Data supporting this have been presented by the Applicant in Section 361.17.4 of the FSAR.

ROCK VERSUS SOIL SITES

The SMA instrument is located on the abutment between Monticello Dams B and C. An examination of the topography of this region indicates that the instrument site is located almost at the top of a hillock that is partly man-made and partly natural. The SMA recording, in all likelihood, represents amplification of the motion of the hillock relative to motion that would be observed in the free-field at either a soil or rock site. Nonetheless, the Applicant has conservatively assumed that no such amplification has occurred in its use of the SMA recording of the August 27, 1978, earthquake to evaluate seismic source parameters. The Applicant maintains, however, that the accelerograph records are not strictly representative of free-field motion, a distinction that Luco fails to draw.

In the free-field, and for short epicentral distances, peak ground accelerations are comparable for rock and soil sites (Campbell, 1981; Joyner and Boore, 1981).

RESPONSE SPECTRUM AT FOUNDATION LEVEL

Luco states that it is not appropriate to compare the 5% and 7% SSE spectra with the 2% $M_L=4.5$ spectrum to study the effects on equipment at the lower levels of the plant. This statement (which points to the lack of effect of structural damping for foundation equipment) would be true if a fixed base model were used in the analysis. Since foundation compliance was taken into account in the soil structure interaction analysis and the base mat response was amplified 10% relative to the input motion, it is appropriate to compare the 5% and 7% SSE spectra with the 2% $M_L=4.5$ spectrum for the effects on equipment at the lower levels of the plant.

The conclusions reached by Luco regarding the level of conservatism of response spectra are incorrect. The velocity amplification factor used by Luco (a value of 1.9) is in fact a mean-plus-one standard-deviation ($\text{mean} + \sigma$) amplification factor, not a mean factor. This is apparent from comparisons with Regulatory Guide 1.60 spectral amplification factors and with the ($\text{mean} + \sigma$) amplification factor developed by the Applicant for the velocity range (see Table 3). Thus Luco's pseudo-velocity spectral amplitude for 5% damping of 0.29 ft./sec. is a ($\text{mean} + \sigma$) amplitude, not a mean amplitude.

Further documentation of the Applicant's methodology is provided as follows. The response spectra developed to represent vibratory ground motion from reservoir-induced earthquakes at Monticello Reservoir were derived following requirements indicated in Reg. Guide 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants." Specifically, Reg. Guide 1.60 states that the standard design response spectrum procedure "...does not apply to sites which (1) are relatively close to the epicenter of an expected earthquake or (2) which have physical characteristics that could significantly affect the spectral combination of input motion. The Design Response Spectra for such sites should be developed on a case-by-case basis." The Virgil C. Summer Nuclear Station would be close to the epicenter of any reservoir-induced seismicity of concern; hence site-specific response spectra were developed to represent ground motion for these events.

This procedure consisted of using response spectrum shapes for earthquake ground motions recorded at magnitudes, distances, and site conditions representative of reservoir-induced earthquakes at the Virgil C. Summer facility. These response spectrum shapes, for magnitudes in the range of interest, were then compared to other available data to ensure their applicability.

The shapes for these spectra were taken from the publication of Johnson and Traubenik (1978). These spectral shapes represent ground motions based on records obtained on rock sites for earthquakes with magnitudes (M_L) between 4.7 and 6.5, with source-to-site distances of less than 20 kilometers. The derived spectra for 5 percent damping for $M_L = 4.0, 4.5, \text{ and } 5.3$ events scaled to 0.15 g peak acceleration are

labeled "RIS" in Figure 1. These are mean + σ spectra, based on the amplification factors reported by Johnson and Traubenik (1978). Use of the mean + σ spectrum is consistent with the procedure defined as acceptable for standard design response spectra in Reg. Guide 1.60.

Also shown in Figure 1 are the Reg. Guide 1.60 spectrum for 5 percent damping, and the Virgil C. Summer Nuclear Station SSE spectrum for 5 percent damping, both scaled to 0.15 g acceleration (the SSE acceleration at the facility). It is apparent that the derived RIS spectra generally match both the Virgil C. Summer spectrum and the RG 1.60 spectrum at the highest frequencies, but deviate at intermediate and low frequencies, the extent depending on both the earthquake magnitude and the frequency of interest. The reason for this deviation is that broad-banded design spectra typically represent ground motions for earthquakes of magnitude around 6-1/2 (they are derived from recorded ground motions during seismic events with an average magnitude of 6-1/2). The RIS spectra, on the other hand, logically reflect the lack of intermediate and low frequency energy which will be generated during magnitude 4.0 to 5.3 earthquakes with small source to site distances.

Two steps are required to generate site-specific spectra of the type shown in Figure 1, and in comparing these spectra to other results available it is convenient to break the comparison into these two steps. The first step is the estimation of a peak velocity and a peak displacement which are consistent with the peak acceleration of the earthquake of interest. In the present application, the peak velocity-to-acceleration ratio is most critical because it determines the upper corner frequency of the spectrum. The peak displacement is not important in the present application because the Virgil C. Summer SSE spectrum greatly exceeds the RIS spectra in the displacement-controlled region (at lower frequencies).

To evaluate the peak velocities derived by Johnson and Traubenik (1978), we compare them to results derived from other studies. Table 2

summarizes values of peak velocity appropriate for 1 g peak acceleration as obtained for RIS spectra and as derived from results reported by other investigators. Values for rock sites indicate that a peak velocity of about 50 cm/sec for magnitudes around 5.0 is appropriate, as used to characterize RIS spectra. Values for soil sites are generally higher, as shown in the lower half of Table 2; these are not appropriate for the Virgil C. Summer Nuclear Station and are only included here for completeness and to explain results which might be extracted from the literature. To be consistent with the work of Newmark (1973), on which RG 1.60 is based in part, the values shown for peak velocity are mean values, rather than mean + σ or some other values.

The second step in estimating response spectra is to determine amplification factors for the various frequency ranges. These are ratios of spectral response to ground motion parameters. For example, in the high frequency range, one is interested in the ratio of spectral acceleration to peak ground acceleration; at intermediate frequencies one is interested in the ratio of spectral velocity to peak ground velocity.

Table 3 compares spectral amplification factors for the acceleration and velocity ranges as recommended in RG 1.60, as derived for the RIS spectra shown in Figure 1, and as recommended by Newmark and Hall (1969). (As discussed above, the displacement - controlled frequency range is not of particular concern for reservoir-induced earthquakes because of the large degree of conservatism inherent in the design spectrum at lower frequencies.) The RG 1.60 spectra in Table 3 are mean + σ results; the RIS results are also mean + σ amplifications. The velocity amplifications shown in Table 3 for RG 1.60 were calculated from acceleration and displacement amplifications at the indicated frequencies, using an assumed peak velocity of 48 inches per second for 1 g acceleration based on the work of Newmark (1973) on which RG 1.60 is based in part.

The RIS spectral amplifications shown in Table 3 indicate that the representation of reservoir-induced earthquake ground motions is appropriate. These amplifications generally agree with those of RG 1.60, particularly for the higher dampings (5 percent to 10 percent) characterizing structures critical for safe shutdown of the facility, and particularly for the higher frequencies in each range. The Johnson and Traubenik (1978) results on which the RIS amplifications are based were derived for events recorded specifically on rock sites at small source-to-site distances, whereas the RG 1.60 results were obtained from a variety of sites and source-to-site distances. Thus, it would be logical if the uncertainty in spectral amplification would be less for the RIS spectra than for the RG 1.60 spectra; this would result in lower mean + σ spectral amplification for the RIS spectra. The results for 0.5 percent damping undoubtedly reflect this difference in the data sets. This difference in no way detracts from the RIS spectra results: the site and distance conditions of the records on which these results are based more closely reflect the conditions expected during reservoir-induced earthquakes at Monticello Reservoir than do generic broad-banded spectra.

In summary, the spectra developed to represent reservoir-induced earthquake ground motions are based on records obtained at close distances on rock sites. These spectra are consistent with published work of other investigators; they represent mean + σ spectra and have been developed to meet the requirements of RG 1.60.

FLOOR RESPONSE SPECTRA

Luco states that detailed review of structural elements and equipment with high fundamental frequencies is needed. To the extent he means that this review should take account of the high ground motion input levels which he suggests, the Applicant has shown that these levels are inappropriate. To the extent he means that a more detailed review should be undertaken than that described in Appendix X of the "Supplemental Seismologic Investigation", confirmatory studies are ongoing by the Applicant as required in the final recommendations made by ACRS.

SUMMARY

The questions raised by Luco regarding seismicity studies for the Virgil C. Summer Nuclear Station result from misinterpretations of

analyses performed by the Applicant or incorrect data used in Luco's independent analyses. The equations derived by Luco differ little from those used by the Applicant, and lead to an estimate of a 26 bar stress drop for the August 27, 1978 earthquake when the correct source-to-site distance is used (0.67 km). Luco's result of a 100 bar stress drop is obtained because he uses incorrect distances.

Similarly, Luco's characterization of earthquakes at Hsinfenkiang Reservoir with a 100 bar stress drop is incorrect. He has erroneously used surface-wave magnitude for local magnitude, and uses an upper frequency of 20 hz whereas the instruments are linear up to 35 hz. Both errors result in an erroneously large estimated stress drop for the recorded events. The results provided by the Applicant in Appendix XI to the "Supplemental Seismologic Investigation" (stress drops less than 25 bars) are correct. Thus Luco has presented no new analyses or data to indicate that a stress drop greater than 25 bars should be used at Monticello Reservoir.

The predictions of peak acceleration made by Luco are invalid because they assume a stress drop of 100 bars, which is unsupported by any analysis. The peak acceleration data from Oroville aftershocks presented by Luco are biased by a factor of more than two and therefore cannot be used to choose peak accelerations for seismic evaluations. Further, most of the data presented by Luco are from California earthquakes where stress drops of 100 bars are common. It follows immediately that these data are inappropriate to characterize the very shallow, low stress drop ^{1/}events at Monticello.

The response spectra developed by the Applicant conform to the requirements of Regulatory Guide 1.60. They are consistent with mean $\pm\sigma$ spectra and were developed to reflect the near-field, rock site conditions of reservoir-induced earthquakes affecting the Virgil C. Summer Nuclear Station.

^{1/} i.e., About 25 bars

REFERENCES

- Boore, D.M., W.D. Joyner, A.A. Oliver, and R.A. Page (1978), "Estimation of Ground Motion Parameters", U.S.G.S. Circular 795, 43 pp.
- Campbell, K.W. (1981), "Near-Source Attenuation of Peak Horizontal Acceleration", submitted to Bull. Seis. Soc. Am.
- Hanks, T.C., and R.K. McGuire (1981), "The Character of High Frequency Strong Ground Motion," Bull. Seis. Soc. Am., Vol. 71, December (in press).
- Johnson, J.A., and M.L. Traubenik (1978), "Magnitude - Dependent Near Source Ground Motion Spectra", PNC, A.S.C.E. Conf. on Earthquake Eng. and Soil Dynamics, Pasadena, pp. 530-539.
- Joyner, W.B., and D.M. Boore (1981), "Peak Horizontal Acceleration and Velocity from Strong Motion Records, including Records from the 1979 Imperial Valley, California Earthquake", submitted to Bull. Seis. Soc. Am.
- Joyner, W.B., D.M. Boore, and R.L. Porcella (1981), "Peak Horizontal Acceleration and Velocity from Strong-Motion Records including Records from the 1979 Imperial Valley, California, Earthquake", U.S.G.S. Open-File Rept. 81-365, March, 46 pp.
- Newmark, N.M., Consulting Engineering Services (1973), "A Study of Vertical and Horizontal Earthquake Spectra", Urbana, Illinois, USAEC Contract No. AT (49-5)-2667, WASH-1255, April.
- Newmark, N.M., and W.J. Hall, (1969). "Seismic Design Criteria for Nuclear Reactor Facilities," Proc, 4th World Conf. on Earthquake Eng., Santiago.
- Page, R.A., D.M. Boore, W.B. Joyner, and H.W. Coulter (1972), "Ground Motion Values for Use in the Seismic Design of the Trans-Alaska Pipeline System", U.S.G.S. Circular G72, 23. pp.
- Seekins, L.C., and T.C. Hanks (1978), "Strong Motion Accelerograms of the Oroville Aftershocks and Peak Acceleration Data," Bull. Seis. Soc. Am., Vol. 68, pp. 667-689.
- Sheng, C.K., et al., (1973), "Earthquakes Induced by Reservoir Impounding and their Effect on Hsinfengkiang Dam," Scientia Sinica, Peking, April.
- Trifunac, M.D., and A.G. Brady (1976), "Correlation of Peak Acceleration, Velocity, and Displacement with Earthquake Magnitude, Distance, and Site Conditions", Earthquake Eng. and Struc. Dyn., Vol. 4, pp. 455-471.

APPENDIX

Derivation of a_{rms} for case where lower bound is finite:

$$\tilde{a}(f) = \begin{cases} (.85) \frac{\Delta\sigma r}{\rho R b} \exp\left(-\frac{\pi f R}{Q\beta}\right) \left(\frac{f}{f_0}\right)^2 & f < f_0 \\ (.85) \frac{\Delta\sigma r}{\rho R b} \exp\left(-\frac{\pi f R}{Q\beta}\right) & f \geq f_0 \end{cases} \quad \underline{1/}$$

where symbols are as defined in Section 361 of the FSAR.

$$\begin{aligned} a_{rms}^2 &= \frac{1}{T_d} \int_0^{T_d} |a|^2 dt = \frac{1}{\pi T_d} \int_0^{2\pi f_u} |\tilde{a}(\omega)|^2 d\omega \\ &= \frac{c^2}{\pi T_d} \left\{ \int_0^{2\pi f_0} \exp\left(-\frac{2\pi f R}{Q\beta}\right) \left(\frac{f}{f_0}\right)^4 d\omega + \int_{2\pi f_0}^{2\pi f_u} \exp\left(-\frac{2\pi f R}{Q\beta}\right) d\omega \right\} \quad \underline{1/} \end{aligned}$$

where $c = (.85) \frac{\Delta\sigma r}{\rho R b}$ and $2\pi f = \omega$

Neglecting, conservatively, the first integral,

$$\begin{aligned} a_{rms}^2 &= \frac{c^2}{\pi T_d} \left[-\frac{Q\beta}{R} \exp\left(\frac{-\omega R}{Q\beta}\right) \right]_{2\pi f_0}^{2\pi f_u} \\ &= \frac{c^2}{\pi T_d} \frac{Q\beta}{R} \left[\exp\left(\frac{-2\pi f_0 R}{Q\beta}\right) - \exp\left(\frac{-2\pi f_u R}{Q\beta}\right) \right] \end{aligned}$$

so that

$$a_{rms} = (.85)(.37) \frac{\Delta\sigma}{\rho R^{1.5}} \sqrt{\frac{2Qr}{2.34}} \left[\exp\left(\frac{-2\pi f_0 R}{Q\beta}\right) - \exp\left(\frac{-2\pi f_u R}{Q\beta}\right) \right]^{1/2}$$

1/ Numerical correction to formulas.

For f_o small and f_u large, the above is the same as equation (9) in McGuire and Hanks (1980). For f_o non-negligible and f_u non-infinite, and for typical values of R, Q, and β :

$$\frac{2\pi f_u R}{Q\beta} < 0.1$$

so:

$$a_{rms} = (.85)(.37) \frac{\Delta\sigma}{\rho R^{1.5}} \sqrt{\frac{2Qr}{2.34}} \left[\frac{2\pi R}{Q\beta} (f_u - f_o) \right]^{1/2}$$

If $\Delta\sigma$ is being estimated from recorded a_{rms} , the above equation can be inverted to give:

$$\Delta\sigma = \frac{\rho R^{1.5} a_{rms}}{(.85)(.37)} \left[\frac{4\pi Rr}{2.34 \beta} (f_u - f_o) \right]^{-1/2}$$

Changing variables leads to

$$\Delta\sigma = C a_{\text{rms}} \sqrt{\frac{f_o}{2f_u}} \left(1 - \frac{f_o}{f_u}\right)^{-1/2}$$

where C is a constant. Equation (2) of Luco can be put in the same form:

$$\Delta\sigma (\text{Luco}) = C a_{\text{rms}} \sqrt{\frac{f_o}{2f_u}} I (f_u/f_o)^{-1}$$

where $I (f_u/f_o)$ is given in Luco's equation (3).

Note that both of the last two equations must be solved by recursion because f_o is function of $\Delta\sigma$.

The difference in calculated $\Delta\sigma$ between the two methods for any given a_{rms} is a function of f_u/f_o :

f_u/f_o	$\Delta\sigma (\text{Luco})/\Delta\sigma (\text{Applicant})$
2	1.36
4	1.19
5	1.15

From the above values it is evident that the difference is only important when f_u is close to f_o . In other cases, e.g. when f_o is 10 hz and f_u is 40 or 50 hz, which is the case for the most accurate records digitized at 500 points per second, there is only some 15% to 19% difference between the method used by the Applicant and that derived by Luco.

TABLE 1

Observations and estimates of rms and peak accelerations for August 27, 1978 earthquake. (Observations and Applicant's estimates presented at February 26, 1981 ACRS Subcommittee meeting.)

Parameters	Applicant's Estimate	Estimate using Luco Equation (2)
M	2.8	2.7
M_0 , dyne-cm	1.12×10^{20}	1.12×10^{20}
$\Delta\sigma$, bars	22	26
f_c , hz	40	50
R, km	0.67	0.67
a_p/a_{rms}	2.14	2.16
a_{rms} , cm/sec. ²	104	111
a_p , cm/sec. ²	221	240
Observations:		
average a_{rms} , cm/sec. ²	108	108
average a_p , cm/sec. ²	225*	241**

* Filtered/Windowed Record, as presented to ACRS.

** Volume II Record (CIT Procedures)

TABLE 2

Peak Velocity for 1 g Peak Acceleration

Site Conditions	Author	Magnitude	Distance	Peak Velocity, cm/sec
	This Study, RIS Spectra	4.0	near-field	13
		4.5	near-field	26
		5.3	near-field	51
ROCK	Joyner et al (1981) USGS OF 81-365	5.0	R = 0	50
	Trifunac and Brady (1976)	5.0-5.9	All	57
	Joyner et al (1981) USGS OF 81-365	5.0	R = 0	73
SOIL	Boore et al (1978) USGS Circ. 795	5.3-5.7	5 km	90 (small struc.) 95 (all struc.)
			10 km	80 (small struc.) 70 (all struc.)
	Trifunac and Brady (1976)	5.0-5.9	All	92
	Page et al (1972) USGS Circ. 672	5.5	3-5 km	110

TABLE 3

Comparison of Spectral Amplification Factors

	Damping	Regulatory Guide 1.60	Newmark & Hall (1969)	RIS Spectra
Acceleration Amplification	0.5%	4.96 at 9 hz 5.95 at 2.5 hz	5.8	4.5
	5%	2.61 at 9 hz 3.13 at 2.5 hz	2.6	2.6
	10%	1.90 at 9 hz 2.28 at 2.5 hz	1.5	2.0
Velocity Amplification	0.5%	3.05 at 2.5 hz 3.77 at 0.25 hz	3.6	2.3
	5%	1.60 at 2.5 hz 2.42 at 0.25 hz	1.9	1.6
	10%	1.17 at 2.5 hz 2.00 at 0.25 hz	1.3	1.3

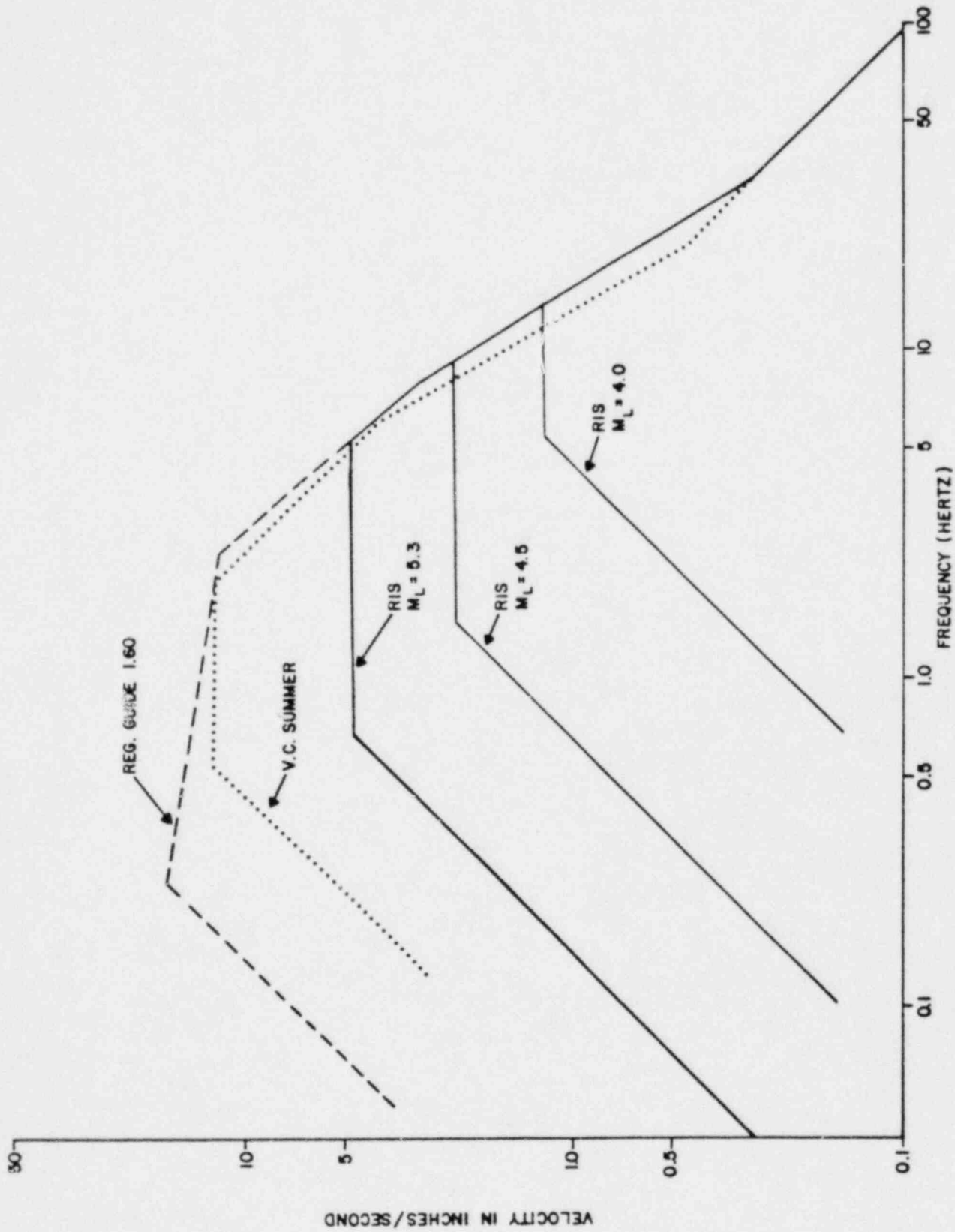


FIGURE I V.C. SUMMER, REG. GUIDE 1.60, and RIS SPECTRA, 5% DAMPING

TESTIMONY OF
MALCOLM R. SOMERVILLE, PH.D.
SOUTH CAROLINA ELECTRIC & GAS COMPANY
BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

My name is Malcolm R. Somerville. I am a seismologist employed by URS/John A. Blume & Associates, Engineers. A statement of my professional qualifications and relevant experience is attached. My testimony consists of two reports. The first, "Comparison of Free-Field (Saprolite) and Foundation (Bedrock) Motions Recorded in Two Explosion Tests" documents the differences between free-field motion on saprolite and massive foundation motion on bedrock based on active field experiments conducted at the site. The second report is the Applicant's evaluation of the Trifunac report which has been submitted to the Board previously. The figures and tables included in each report, unless otherwise noted, were prepared by me and constitute part of my testimony.

COMPARISON OF
FREE-FIELD (SAPROLITE) AND FOUNDATION
(BEDROCK) MOTIONS RECORDED IN TWO EXPLOSION TESTS
AT THE
VIRGIL C. SUMMER NUCLEAR STATION

INTRODUCTION

In October 1981, South Carolina Electric and Gas Company conducted two explosion tests at the site of the Virgil C. Summer Nuclear Station. The purpose of the experiments was to acquire data for a comparative study of ground motion at two sites currently occupied by USGS accelerographs (USGS SMA-1 #603 and #267) and at additional sites in the free-field and in building foundations. This report documents the differences between free-field motions on saprolite and the motions of massive structure foundations on bedrock. In the frequency band from 5 to 50 Hz, amplitudes in the free-field on saprolite are found to be twice those recorded in massive structure foundations on bedrock. Accelerograms recorded on the dam abutment for RIS events are not representative of foundation motions on bedrock, and must be modified accordingly in assessing the effects of RIS on the massive embedded structures of the Nuclear Station founded on bedrock.

EXPERIMENT CONFIGURATIONS

The configuration, instrumentation, conduct, and results of the experiments are described in a previous report ("Active Field Experiments", response to FSAR question 361.24, November 19, 1981).

Figures 1 and 2 show locations of the shotpoints, and recording sites in the free-field and in the foundations of the auxiliary building and hydroplant.

Shotpoint #1 was located at nearly equal distance (~15,000 ft) from four sites occupied by seismographs. These sites are the auxiliary building basement (60 ft below grade on bedrock), and three free-field sites: the dam abutment, the tailrace, and the meteorological tower. Shotpoint #2 was located at nearly equal distance (~1,100 ft) from three sites occupied by SMA accelerographs. These sites are the dam abutment, the tailrace, and the hydroplant foundation (70 ft below grade on bedrock).

Because the comparisons discussed below are limited to stations recording either shot at nearly equal distances, attenuation of motion with distance is not an issue in the analysis. Ground motion is taken to be radially uniform about the shotpoints.

TEST #1

Seismographs recording Test #1 were calibrated to better than 10%, and had flat velocity response in the band 2-50 Hz, with the exception of the vertical component at the auxiliary building. The latter exhibited anomalous response characteristics, and is not included in the analysis. Fourier spectra were calculated for record lengths of 5.12 sec and sample rates of 200 per sec. Unsmoothed Fourier magnitude spectra of signals recorded at the dam abutment, tailrace, meteorological tower, and auxiliary building basement are shown in Figures 3-6. The seismograms were

not corrected for seismometer response, and so the Fourier spectra are not reliable for frequencies less than 2 Hz.

Figure 7 shows spectra of ambient noise for 5 seconds immediately preceding the shot, for the horizontal components at the dam abutment, tailrace, and auxiliary building. Comparing signal with noise spectra, it can be seen that signal-to-noise ratios at the free-field stations generally exceed 50 in the band 2-20 Hz, and usually exceed 10 in the band 20-50 Hz. In the auxiliary building, noise levels are relatively high, with a strong peak at 30 Hz, the frequency of rotation of electric-motor driven equipment in the building. Signal-to-noise ratios in the auxiliary building are in the range 5-10 in the band 2-50 Hz.

Figures 8-10 show spectral ratios for the auxiliary building relative to the dam abutment, tailrace and meteorological tower. These spectral ratios show substantially lower motion of the auxiliary building foundation relative to the field stations for nearly all frequencies between 5 and 50 Hz. As a generalization, the spectral ratios are one half in the band from 5 to 50 Hz.

The most significant comparison is that between the auxiliary building foundation and the dam abutment, where strong motion accelerograms have been recorded for numerous nearby RIS events. In the band from 5 to 30 Hz, the spectral ratios are generally less than 0.4. In order to summarize the motion in the auxiliary building, relative to that on the dam abutment, integrals of power spectral density in several passbands were computed using an ave-

rage of N-S and E-W components. The ratios of these values for the two sites were computed for each passband; the square roots of these ratios were also computed, and represent the ratio of root mean square (RMS) particle velocity in each passband. These data are summarized in Table 1. The data of Table 1 show that RMS velocities in the auxiliary building are from 10 percent to 30 percent of those recorded on the dam abutment. In the passband 20 to 33 Hz, the ratio is 0.305, showing greater than 3-to-1 reduction of RMS velocities in the auxiliary building.

Test #2

Test 2 was recorded by nearly equidistant accelerographs at the dam abutment, the tailrace, and on the foundation of the hydroplant. It was not possible to reliably digitize the accelerographs recorded in the hydroplant foundation, due to the presence of harmonic equipment motion with amplitude not much lower than that of the explosion signal.

Peak accelerations scaled directly from enlarged copies of the accelerographs are given in Table 2. Peak horizontal accelerations in the hydroplant (founded on bedrock) were nearly 60% of those on the dam abutment (on saprolite), comparing components oriented approximately radially and transversely with respect to the shot-point. This result is similar to the spectral ratios between the auxiliary building foundation and the dam abutment observed in Test #1.

EVALUATION OF ACCELEROGRAMS RECORDED AT SHOTPOINT #2

As shown in Table 2, peak accelerations recorded at the shotpoint were an order of magnitude larger than recorded elsewhere. Peak accelerations on the horizontal components were approximately 40% g. These data are relevant to the question as to the response of the saprolite at strain levels substantially higher than recorded elsewhere in Tests 1 and 2. If the response of the saprolite near shotpoint #2 were strongly nonlinear, it would be expected that due to absorption effects, the spectrum of motions recorded at the shotpoint would differ in shape from that recorded elsewhere at much lower strain levels. However, this is not found to be the case. Figure 11 compares 2% damped response spectra for the longitudinal components of motion recorded at the shotpoint and at the dam abutment. For ease of comparison, the shotpoint spectral amplitudes have been divided by a factor of 10. The similarity of the spectral shapes suggests that the response of the saprolite is not strongly nonlinear for the given range of motion amplitudes. This observation indicates that the difference between foundation motion on bedrock and free-field motion on saprolite as documented in Tests 1 and 2, would hold for strain levels considerably higher than achieved at the relevant recording sites during either test. Note that the horizontal accelerations recorded at shotpoint #2 exceed those recorded by the USGS accelerograph at the dam abutment during RIS events.

SUMMARY AND CONCLUSIONS

Two independent field experiments have demonstrated a substantial difference in motions recorded in massive building foundations on bedrock as compared with free-field motions on saprolite. This result is significant in assessing the effect of RIS on the Nuclear Station because all strong motion data acquired for RIS events to date has been recorded in the free-field on saprolite.

In Test #1, Fourier spectral ratios between the auxiliary building foundation on bedrock and free-field sites on saprolite are approximately 0.5 in the frequency band from 5 to 50 Hz. Comparing the auxiliary building foundation with the dam abutment, the spectral ratios are generally less than 0.4 in the band from 5 to 30 Hz. In Test #2, peak acceleration ratios between the hydroplant foundation on bedrock and the dam abutment were nearly 0.6 for radial and transverse components. Peak accelerations were registered at frequencies of approximately 20 Hz.

The observed differences between foundation and free-field motion are attributable to several effects which can not be resolved uniquely from the data. Because of the absence of rock outcrops in the site vicinity, it was not possible to obtain free-field records on rock and thereby isolate the effects due to the saprolite layer. It is likely that the acoustic impedance contrast between rock and saprolite accounts in large part for the observed differences in motion amplitude between foundation and free-field sites. Another important effect is that due to the presence of massive structures on large, deeply embedded foundations. In general it is expected

that foundation motion will differ from free-field motion due to the effects of elastic wave incoherence (on the scale of the foundation dimension), elastic wave scattering by the foundation, inertial resonance of the building mass, and energy transmission between the ground and the structure. These effects can not be resolved from the field test data.

The same physical phenomena as caused the observed differences between foundation and free-field motion in the field tests would also be in play in the case of an earthquake source. A comparison of shotpoint and dam abutment spectral shapes for Test #2 indicates that the saprolite exhibits essentially elastic response up to motion levels higher than have been recorded on the dam abutment for RIS events. Thus it is not appropriate to assess the effect of RIS on deeply embedded structures of the Nuclear Station on the basis of accelerograms recorded for RIS events at the dam abutment on saprolite: these accelerograms are not representative of the motions of large foundations on bedrock.

To illustrate the difference between foundation (bedrock) and free-field (saprolite) motion, an accelerogram of the October 16, 1979 RIS event (Figure 12) was filtered by the empirical transfer function shown in Figure 13. The filtered accelerogram is shown in Figure 14. This record represents the expected motion of a large deeply embedded foundation on bedrock corresponding to the free-field accelerogram recorded by the USGS accelerograph on the dam abutment on saprolite.

TABLE 1

RATIOS OF ENERGY FLUX AND ROOT MEAN
SQUARE PARTICLE VELOCITY FOR AUXILIARY
BUILDING/DAM ABUTMENT⁽¹⁾

<u>Frequency Band (hz)</u>	<u>Energy</u>	<u>Ratio</u>	<u>RMS Velocity</u>
2.93 - 9.96	0.0109		0.104
9.96 - 19.92	0.0241		0.155
19.92 - 33.00	0.0930		0.305
5.08 - 33.00	0.0283		0.168
9.96 - 33.00	0.0396		0.199

(1) Computed from integrals of power spectral density (PSD) of average horizontal motion for the two pairs of seismograms (N-S and E-W components).

TABLE 2

PEAK ACCELERATIONS RECORDED BY
SMA-1 ACCELEROGRAPHS FOR SHOT 2

Site	Instrument Serial No.	Peak Accelerations (g)*		
		L	Z**	T
Shotpoint	4722	0.42	1.02	0.39?
Dam Abutment	603	0.021	0.018	0.033
Hydro Plant	4673	0.019	0.021	0.013
Tailrace	267	0.022	0.040	0.052

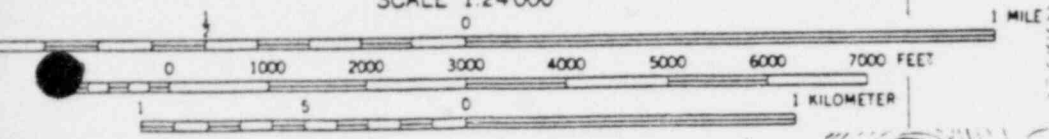
* Data scaled directly from enlarged copies of accelerograms, measuring from the trace edges.

** The ratio of vertical to horizontal motion from blasts is not the same as from earthquake motions.

JENKINSVILLE QUADRANGLE
SOUTH CAROLINA
7.5 MINUTE SERIES (TOPOGRAPHIC)

SCALE 1:24,000

2" = 36 MILS
0"11" = 3 MILS



UTM GRID AND 1969 MAGNETIC NORTH DECLINATION AT CENTER OF SHEET

CONTOUR INTERVAL 10 FEET
DATUM IS MEAN SEA LEVEL

SHOT NO. 1
GROUND ZERO

MONTICELLO

RESERVOIR

TOP OF DAM
ELEV. 434

TOP OF DAM
ELEV. 434

N480,000

FAIRFIELD
POWERHOUSE

NUCLEAR
EXCLUSION
ZONE

V. C. SUMNER
NUCLEAR STA.
N34°17' -56"
W81°18' -50"

TOP OF DAM
ELEV. 434

SHOT NO. 2
GROUND ZERO

DISCHARGE CHANNEL
BOTTOM ELEV. 404

BORR
AREA
E

BORR
AREA
A



FIGURE 2

13

12

11

N 473,000

AUXILIARY BLD. BASEMENT STATION

V.C. SUMMER NUCLEAR STATION

MET. TOWER STATION

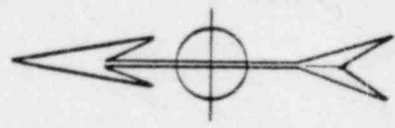
PROJECT BOUNDARY LINE IS 430 CONTOUR BETWEEN HEAVY BLACK ARROWS. (SEE EXHIBIT A-2)

METEOROLOGICAL TOWER

E 1,903,000

DAM "D" SEE EXHIBIT "D"

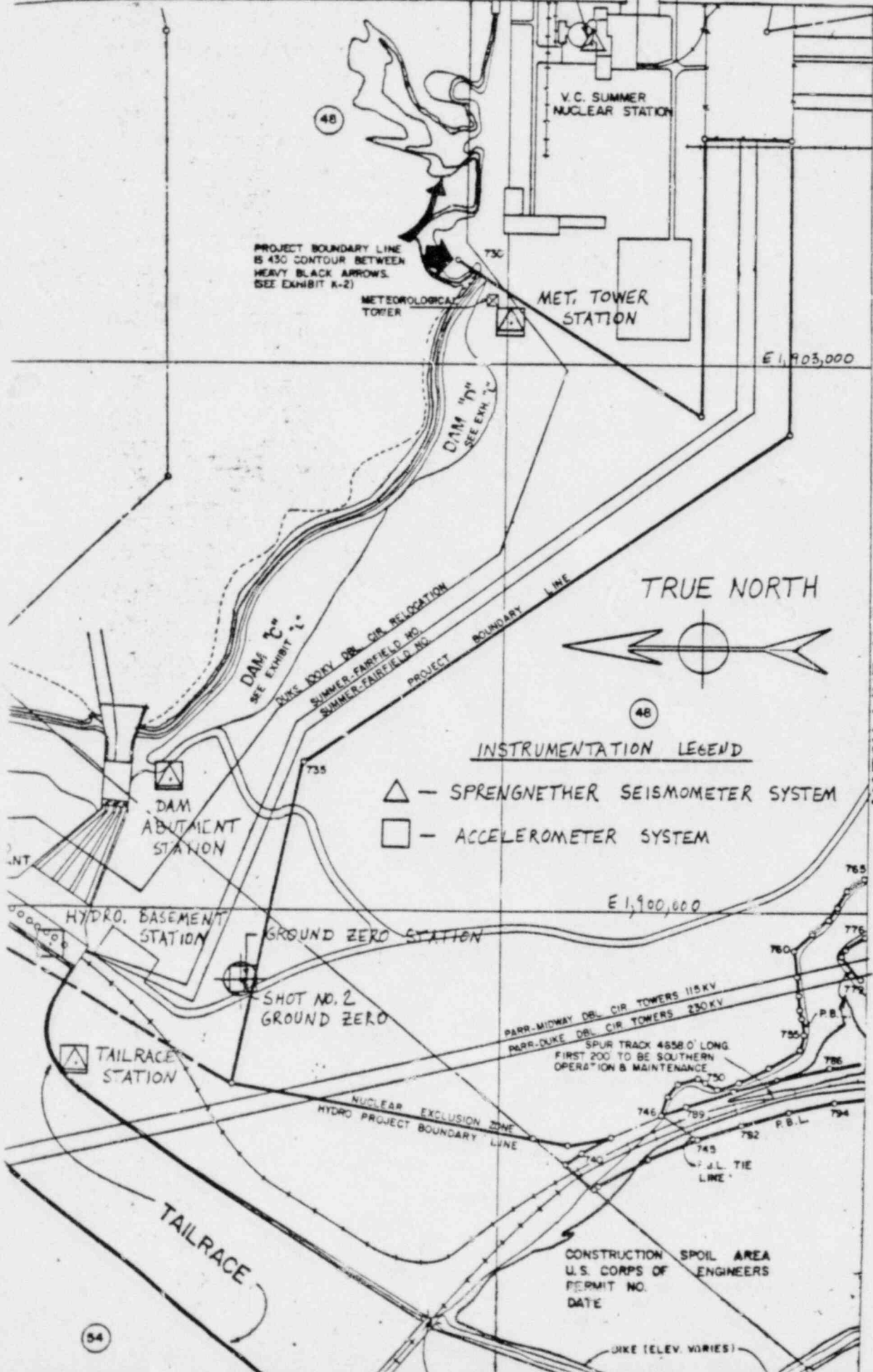
TRUE NORTH



INSTRUMENTATION LEGEND

- △ - SPRENGNETHER SEISMOMETER SYSTEM
- - ACCELEROMETER SYSTEM

MATCH LINE K-5



54

FIGURE 3A. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRUM
DAM ABUTMENT
VERT. COMP. (BLUME)

46 6210
FOURIER MAGNITUDE (CM)
K·E. ELECTRONIC FILM & LENSER CO. MADE IN U.S.A.
50 MILLIGRAMS/FILM 5 CYCLES X 70 DIVISIONS

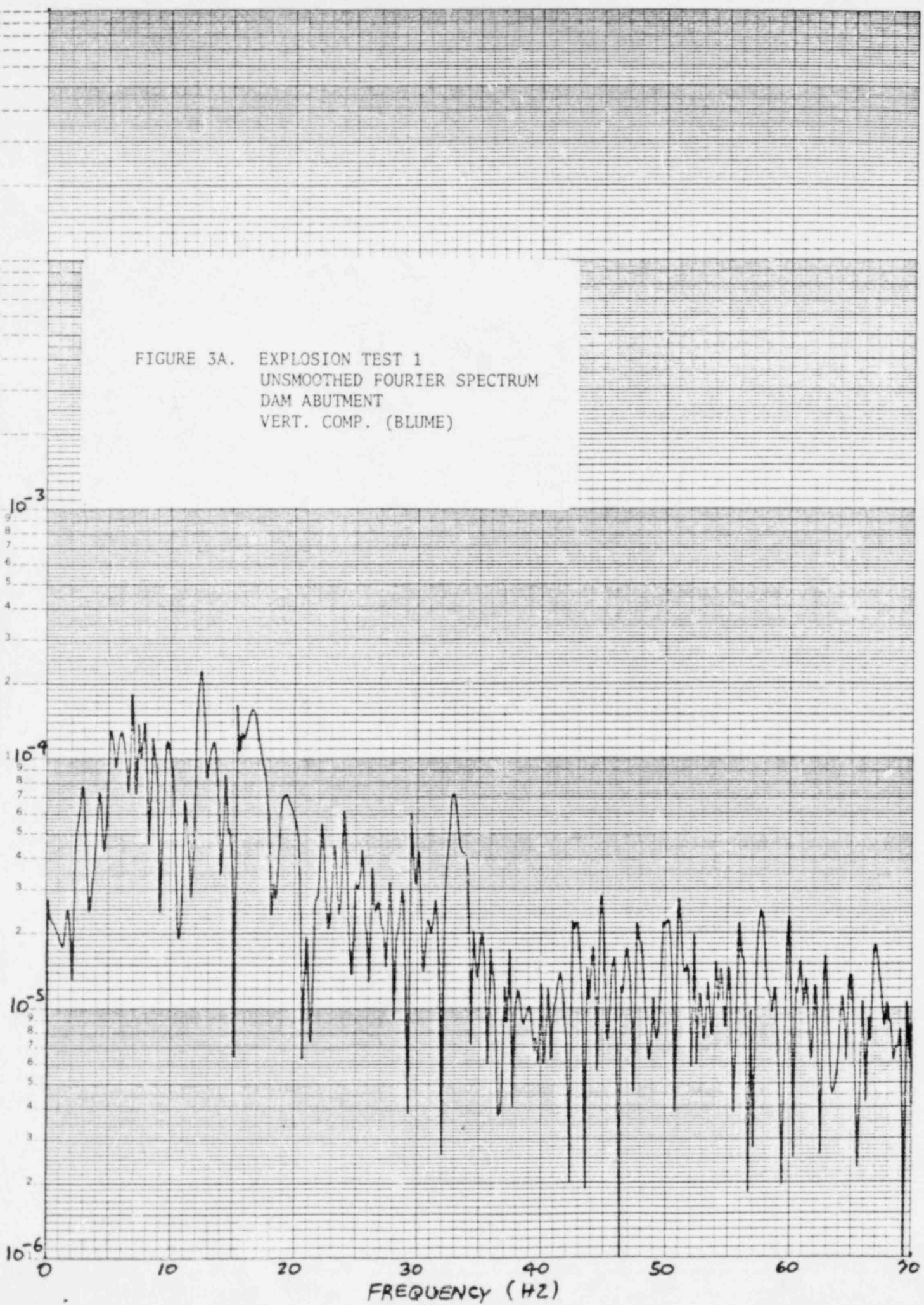


FIGURE 3B. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRUM
DAM ABUTMENT
N-S COMP. (BLUME)

46 6210
K·E SEMI-LOGARITHMIC 5 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

FOURIER MAGNITUDE (CM)

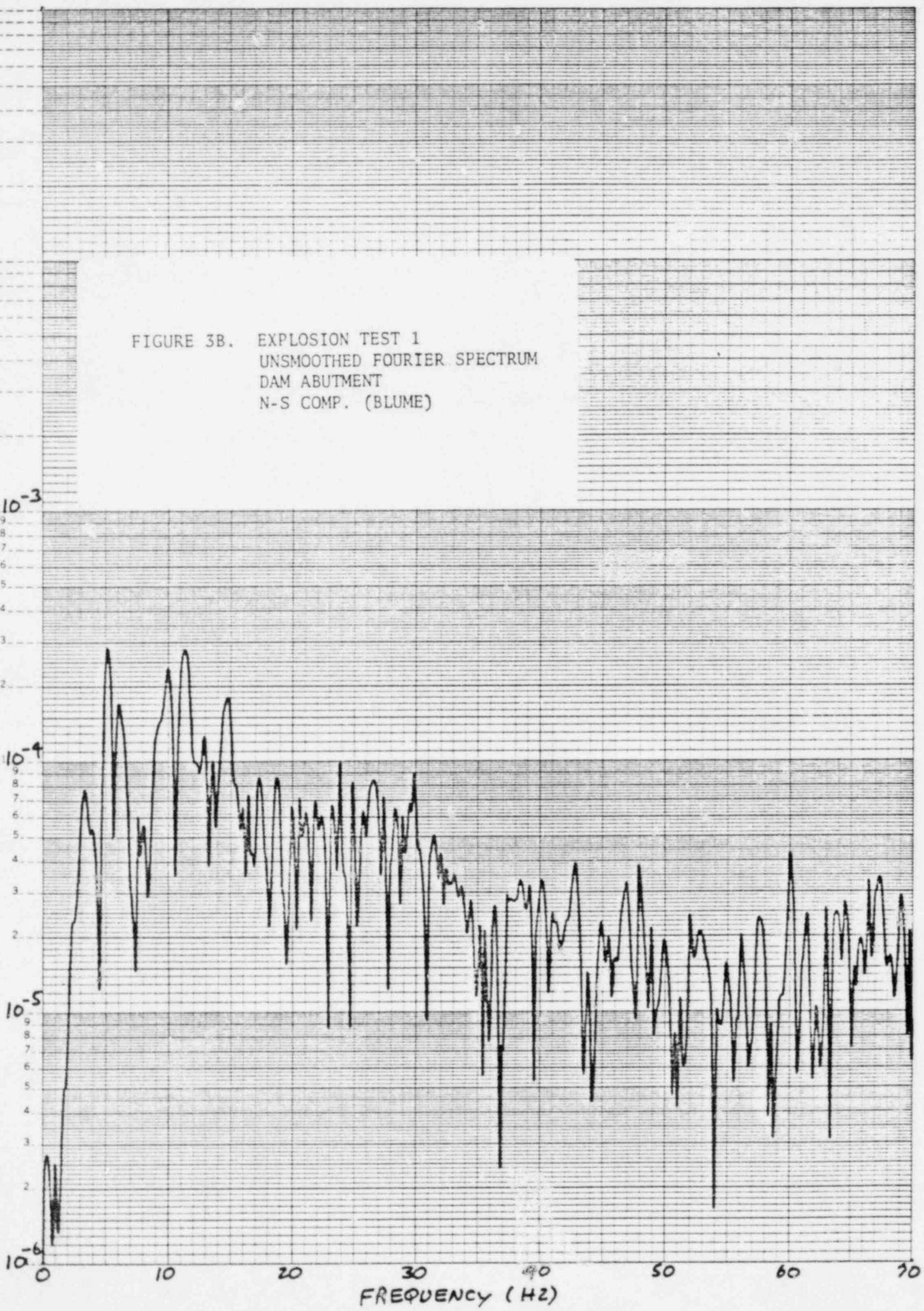


FIGURE 3C. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRUM
DAM ABUTMENT
E-W COMP. (BLUME)

46 6210

FOURIER MAGNITUDE (CM)

K·E SEMI-LOGARITHMIC 5 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

10^{-3}
9
8
7
6
5
4
3
2
1
 10^{-4}
9
8
7
6
5
4
3
2
1
 10^{-5}
9
8
7
6
5
4
3
2
1
 10^{-6}

0 10 20 30 40 50 60 70
FREQUENCY (HZ)

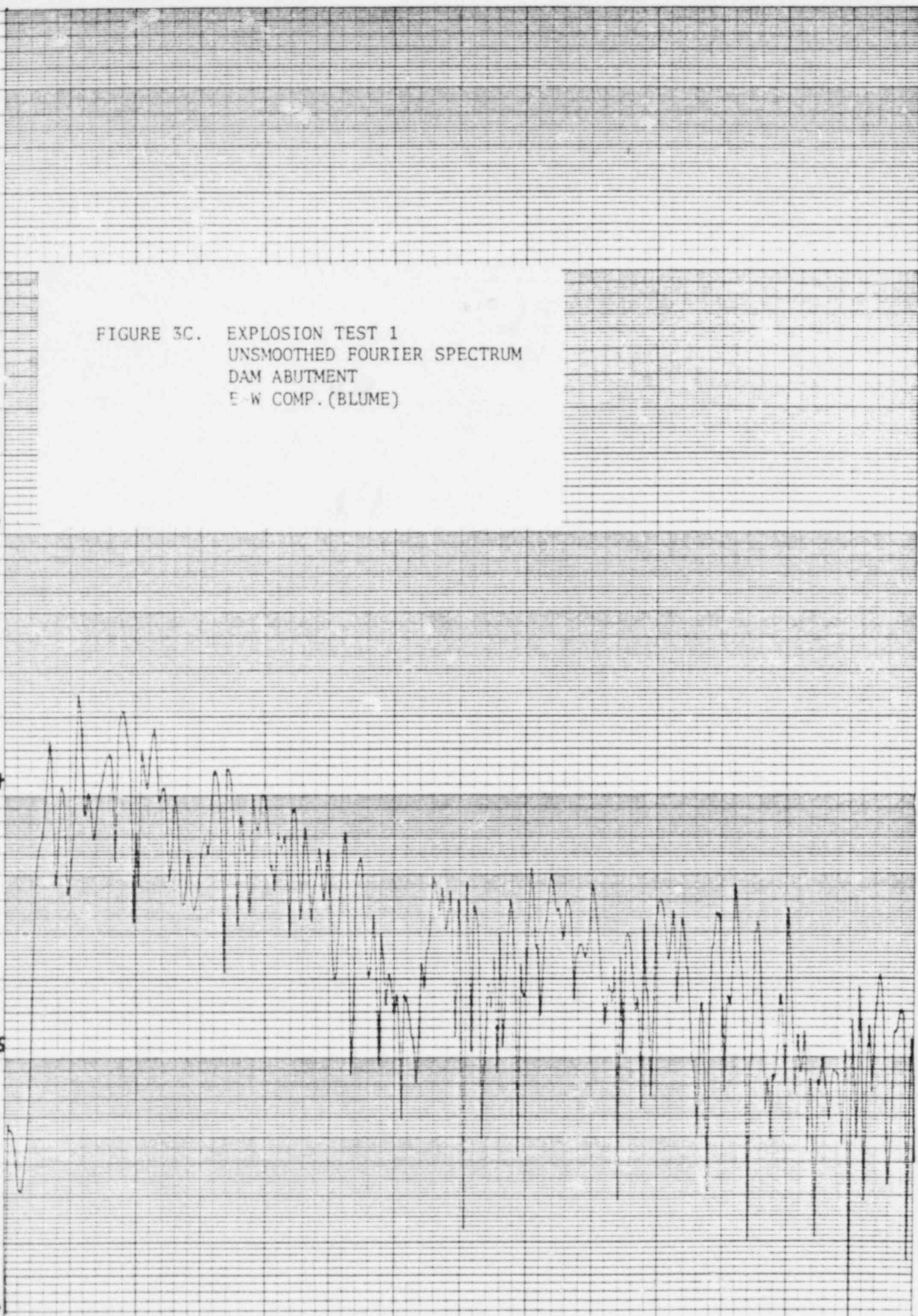


FIGURE 4A. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRUM
TAILRACE
VERT. COMP (BLUME)

46 6210

FOURIER MAGNITUDE (CM)

SEMI-LOGARITHMIC 5 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

K-E

10^{-3}

10^{-4}

10^{-5}

10^{-6}

FREQUENCY (HZ)

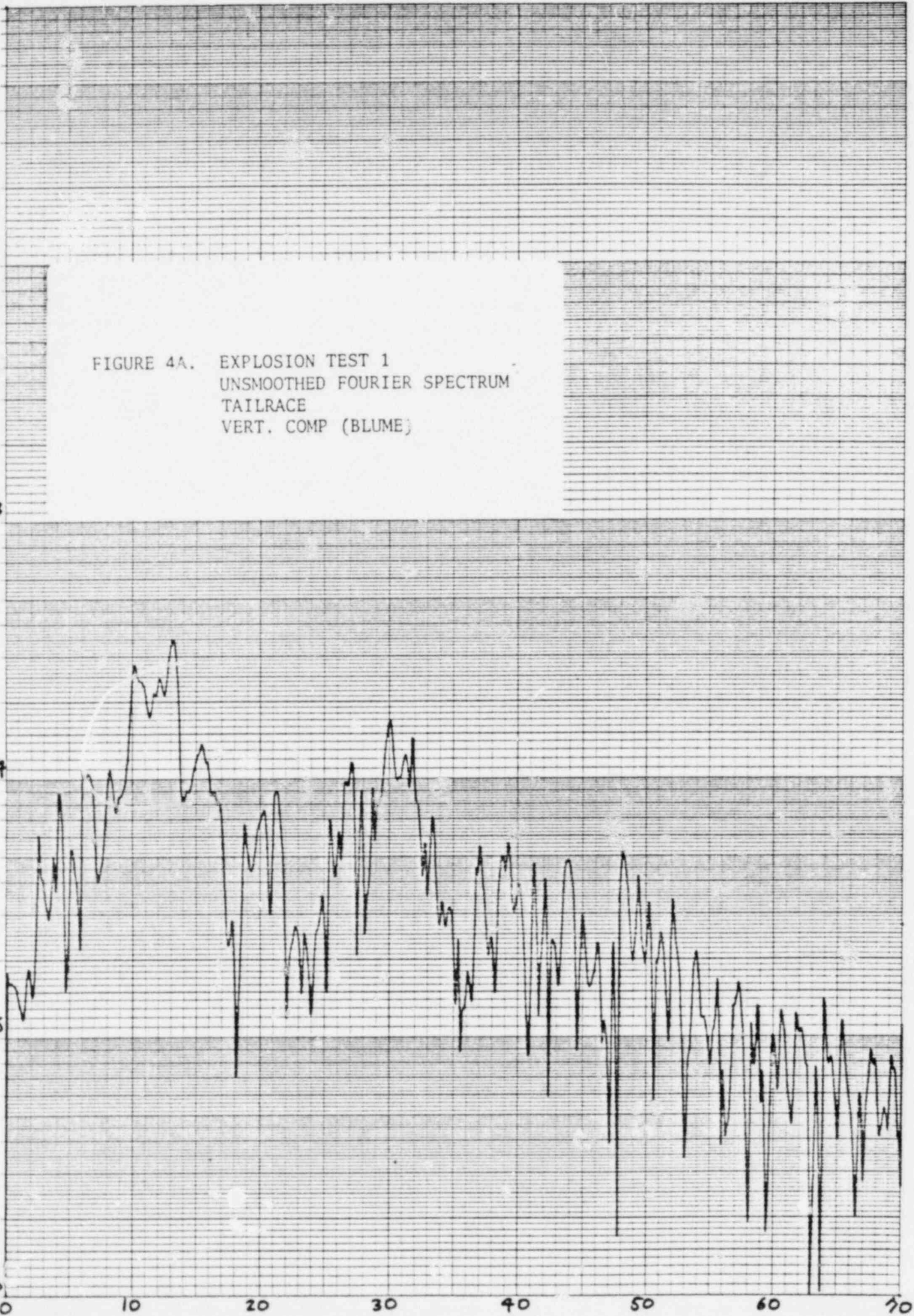


FIGURE 4B. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRUM
TAILRACE
N-S COMP. (BLUME)

46 6210

K·E SEMI-LOGARITHMIC 5 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

FOURIER MAGNITUDE (CM)

10^{-3}

10^{-4}

10^{-5}

10^{-6}

FREQUENCY (HZ)

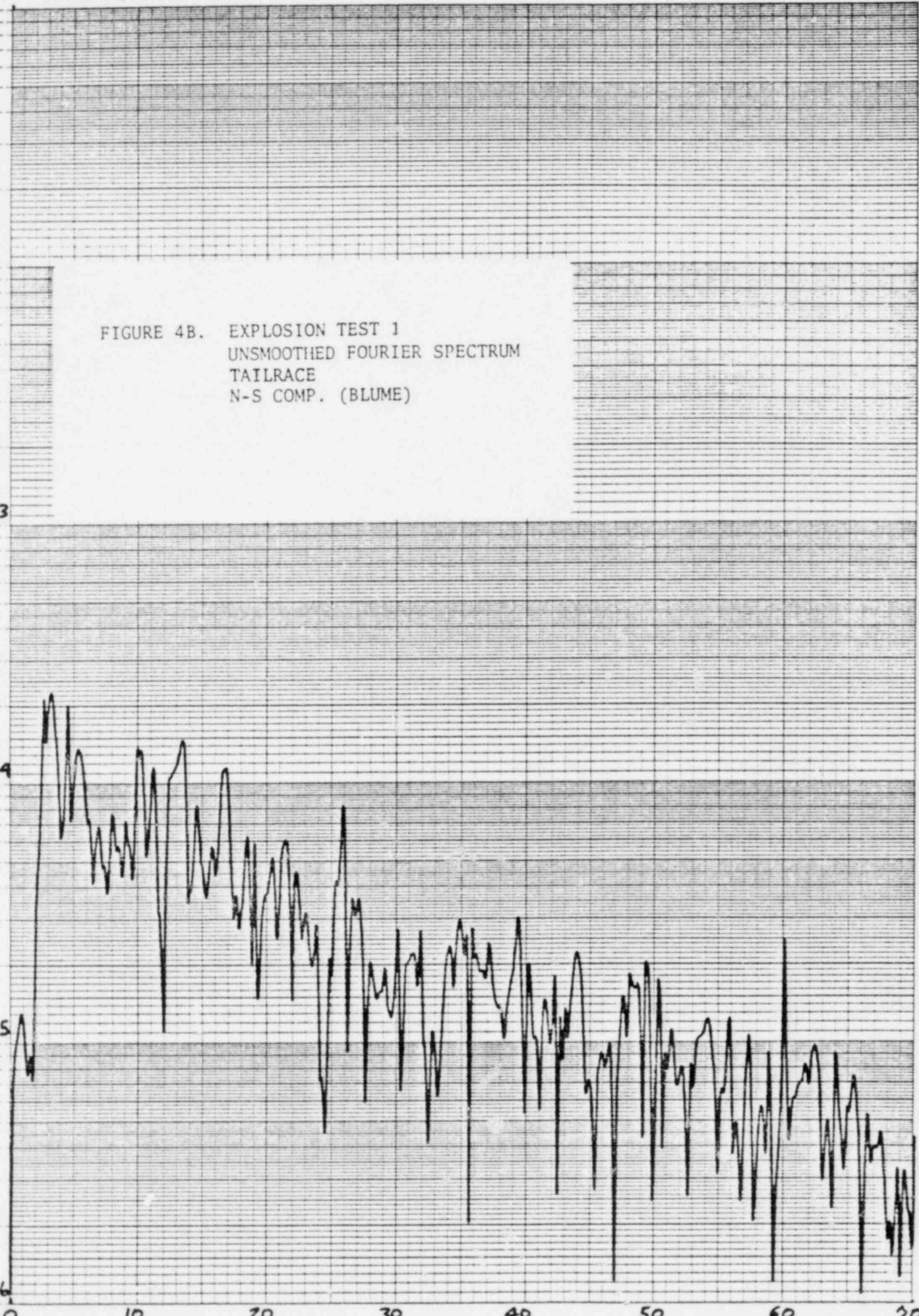
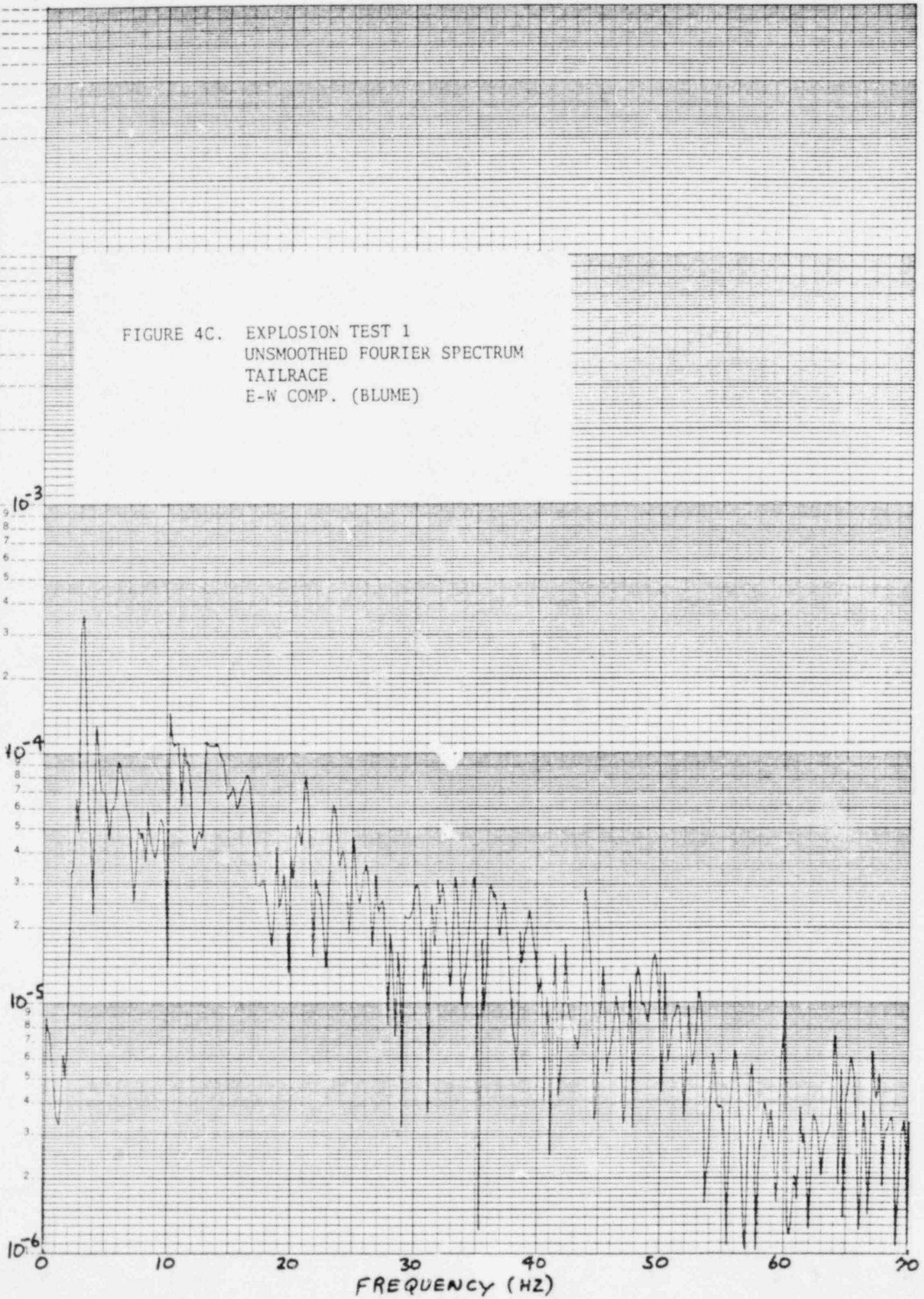


FIGURE 4C. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRUM
TAILTRACE
E-W COMP. (BLUME)

46 6210
FOURIER MAGNITUDE (CM)
K&E SEMI-LOGARITHMIC 5 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

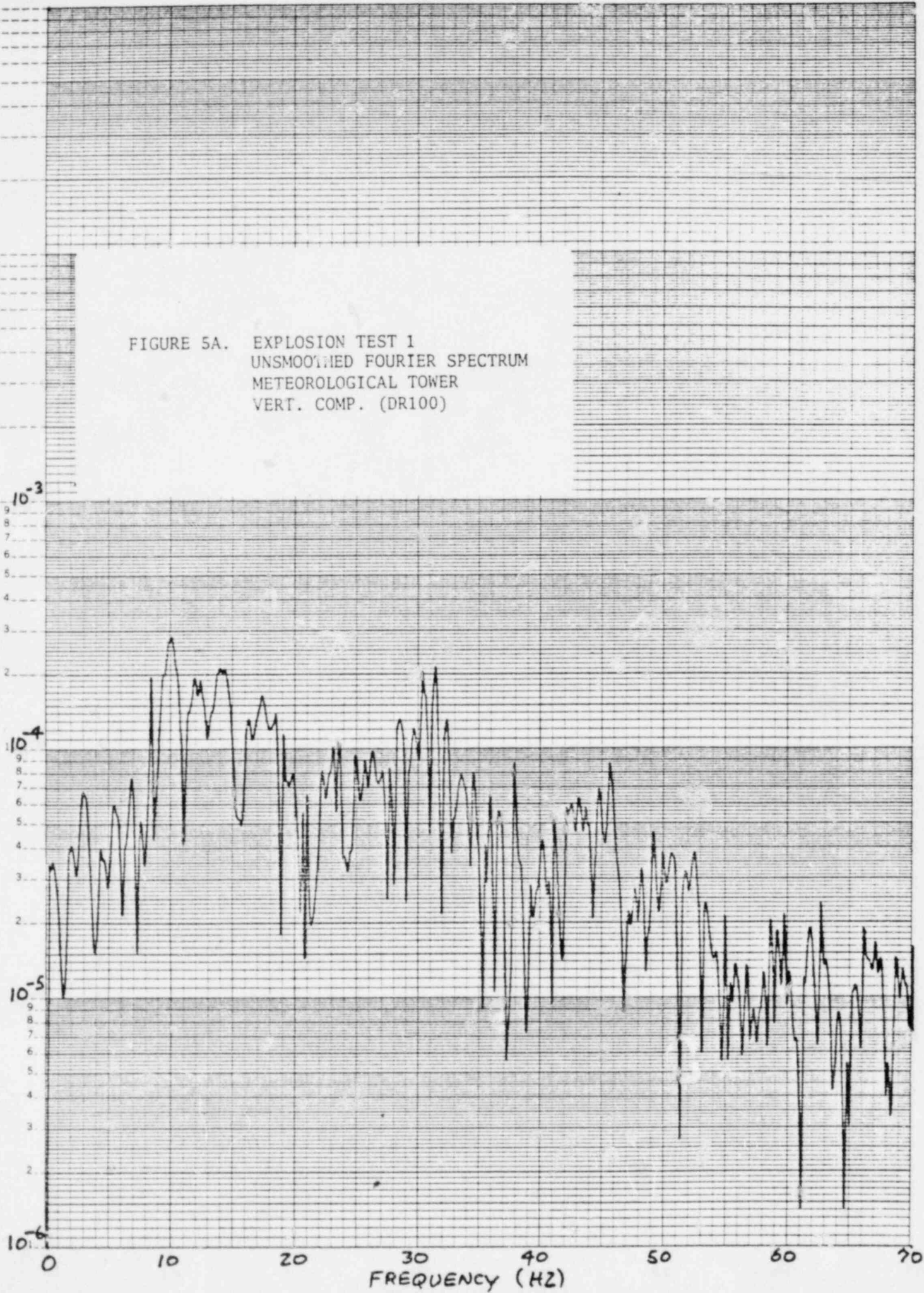


46 6210

K&E SEMI-LOG-ARITHMIC 5 CYCLES X 70 DIVISIONS
KEUFFEL & ESSNER CO. MADE IN U.S.A.

FOURIER MAGNITUDE (CM)

FIGURE 5A. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRUM
METEOROLOGICAL TOWER
VERT. COMP. (DR100)



K-E SEMI-LOGARITHMIC 5 CYCLES X 70 DIVISIONS KEUFFEL & ESSER CO. MADE IN U.S.A. 46 6210

FOURIER MAGNITUDE (CM)

FIGURE 5B. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRUM
METEOROLOGICAL TOWER
E-W COMP. (DR100)

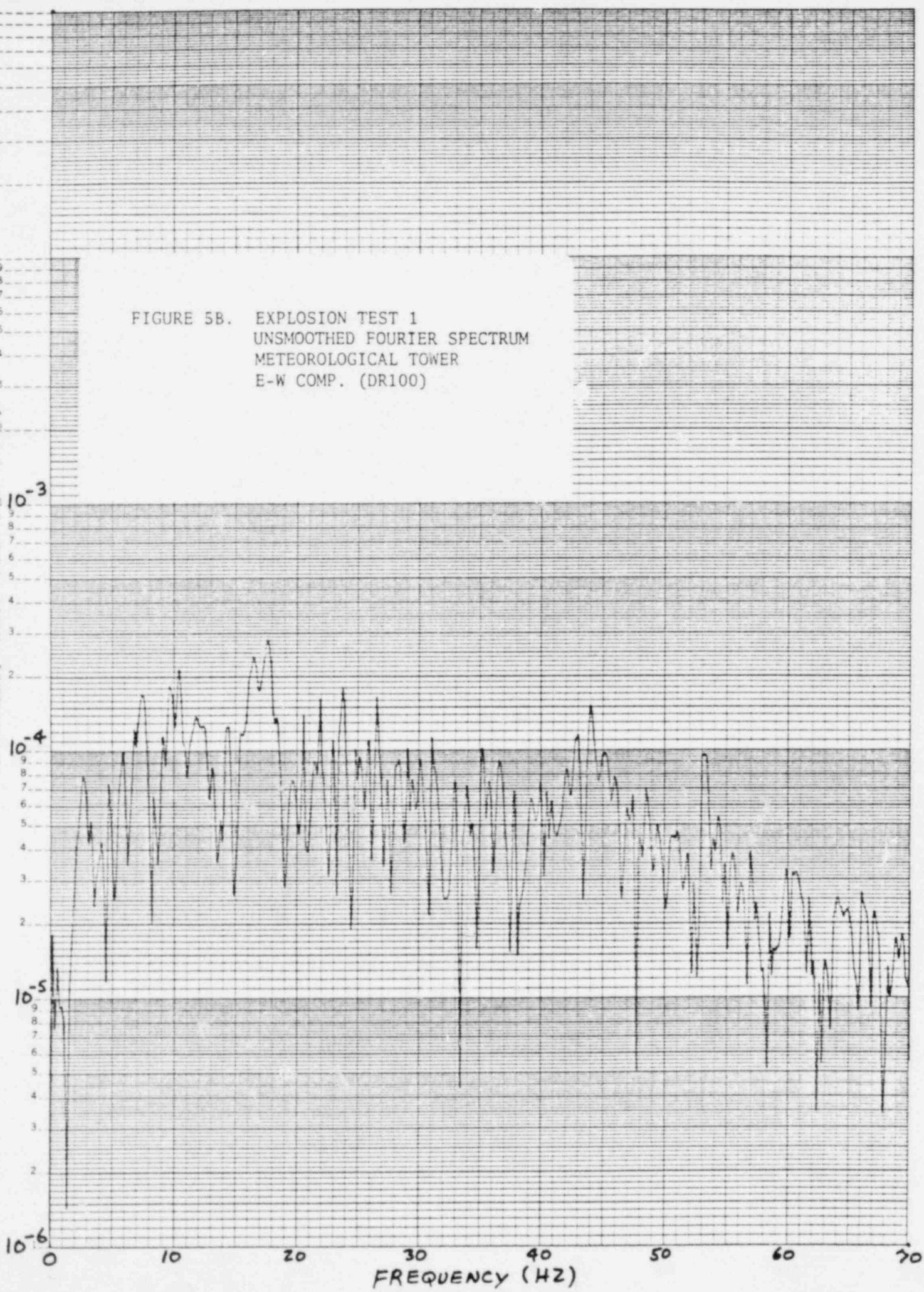
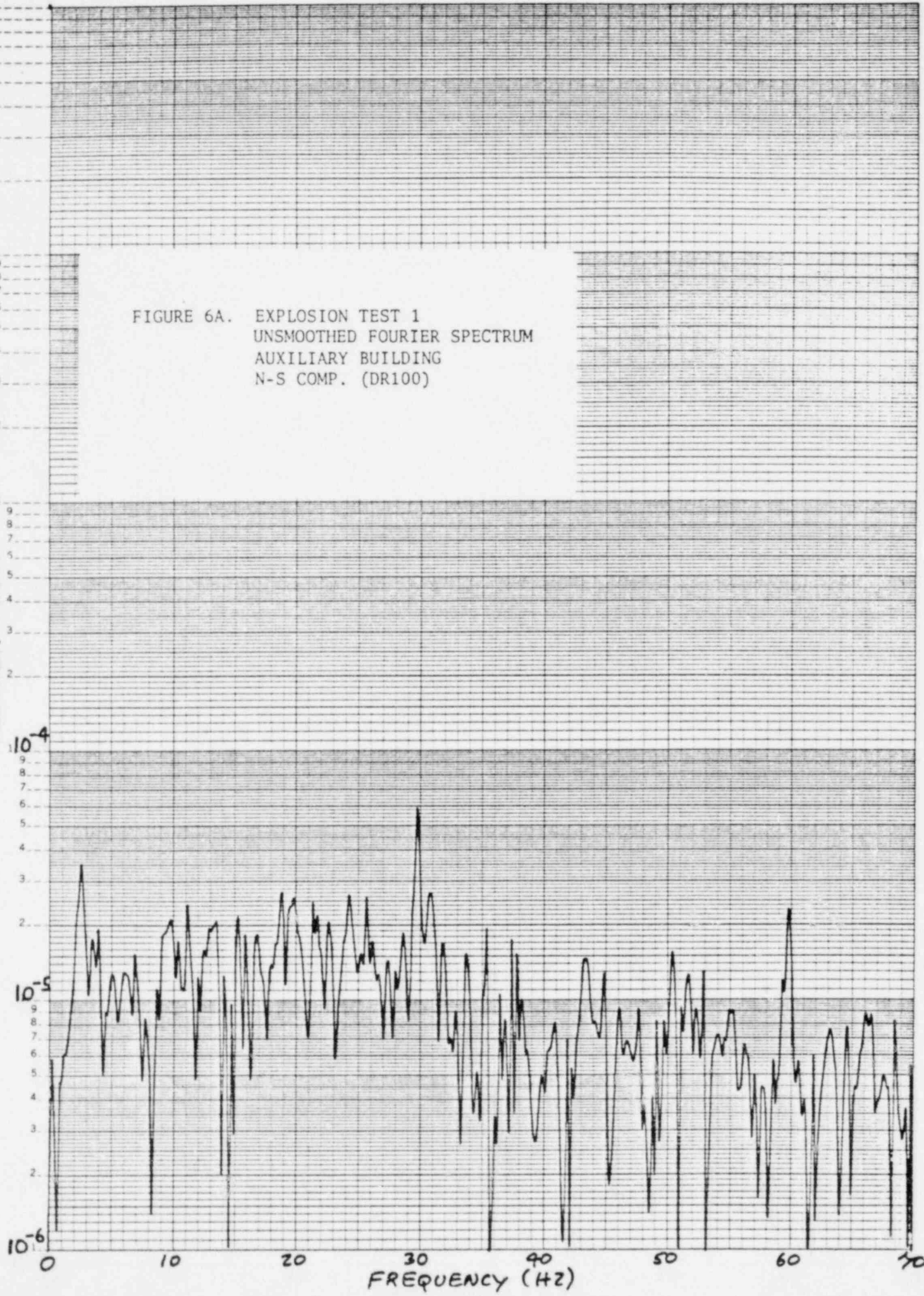


FIGURE 6A. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRUM
AUXILIARY BUILDING
N-S COMP. (DR100)

46 6210
FOURIER MAGNITUDE
(CM)

K&E SEMI-LOGARITHMIC 5 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.



46 6210

K-E SEMI-LOGARITHMIC 5 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

FOURIER MAGNITUDE (CM)

FIGURE 6B. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRUM
AUXILIARY BUILDING
E-W COMP. (DR100)

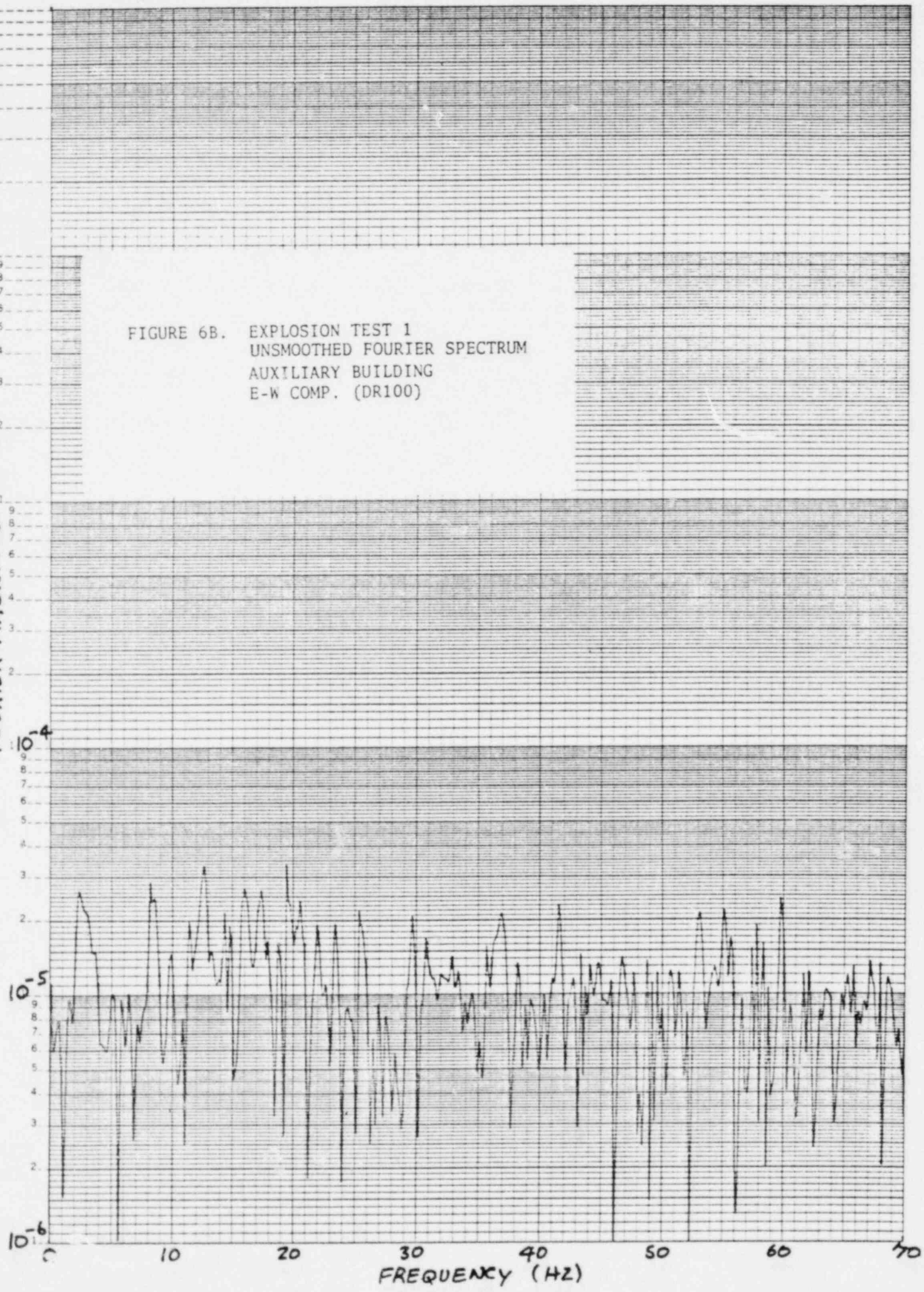


FIGURE 7. EXPLOSION TEST 1
SMOOTHED FOURIER SPECTRA
AMBIENT NOISE AT THREE SITES

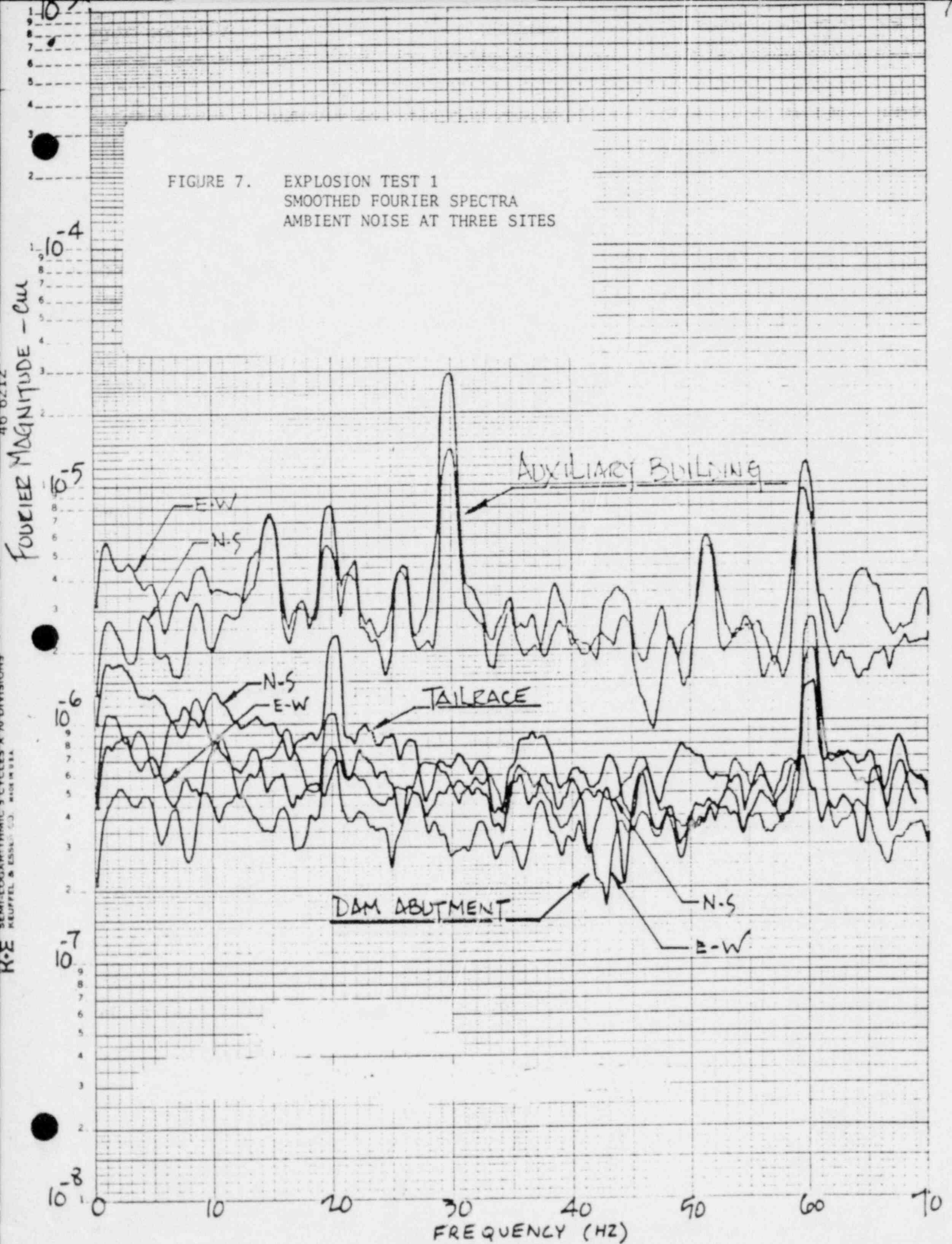


FIGURE 8A. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRAL
RATIO
AUXILIARY BUILDING/DAM ABUTMENT
N-S COMP. (DR100/BLUME)

46 6210

K·E SEMI-LOGARITHMIC 5 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

SPECTRAL RATIO

10

1.0

0.1

0.01

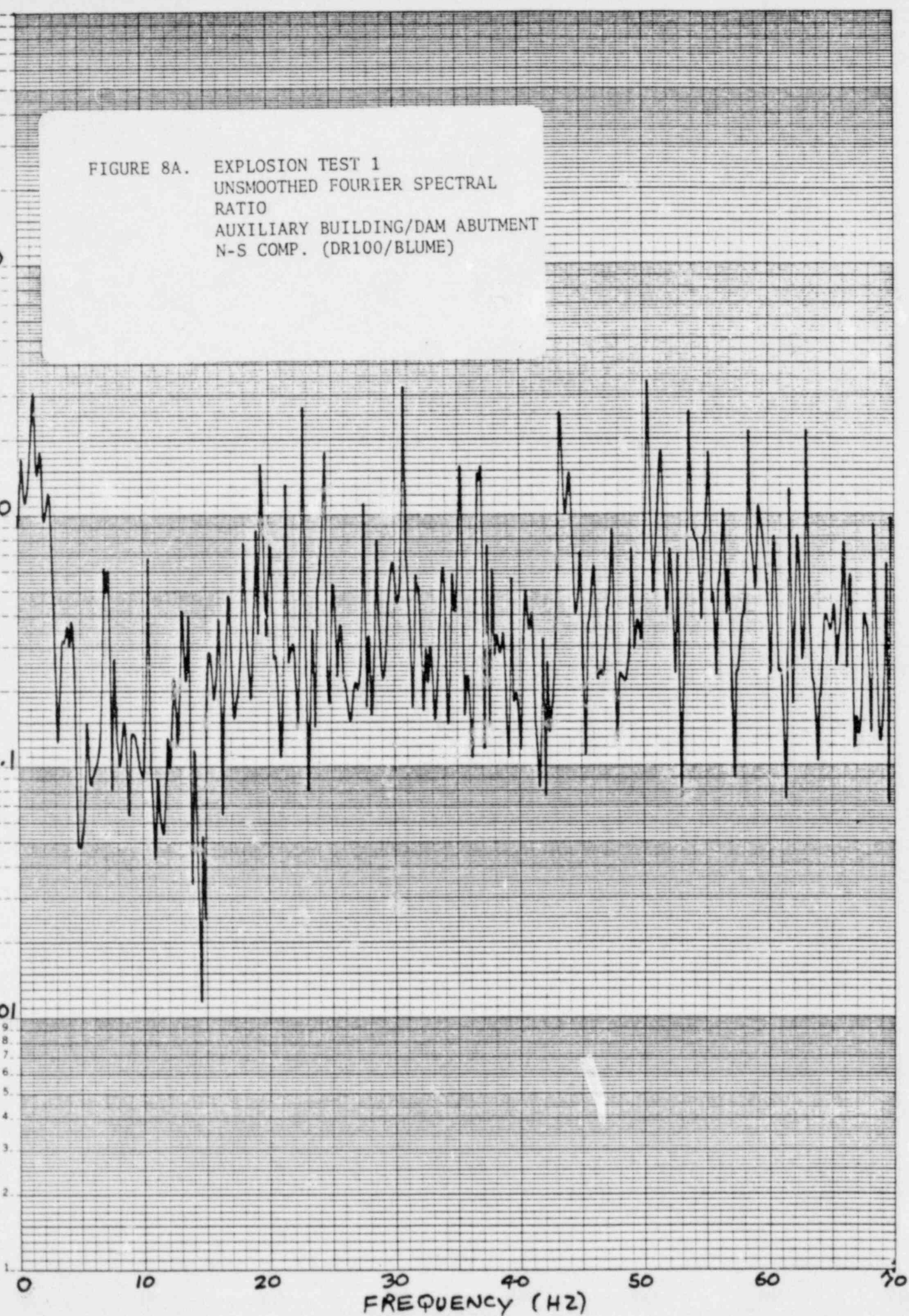


FIGURE 8B. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRAL
RATIO
AUXILIARY BUILDING/DAM ABUTMENT
E-W COMP. (DR100/BLUME)

46 6210

SEMI-LOGARITHMIC 5 CYCLES X 7 1/2 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

K-E

SPECTRAL RATIO

10

1.0

0.1

0.01

0 10 20 30 40 50 60 70
FREQUENCY (HZ)

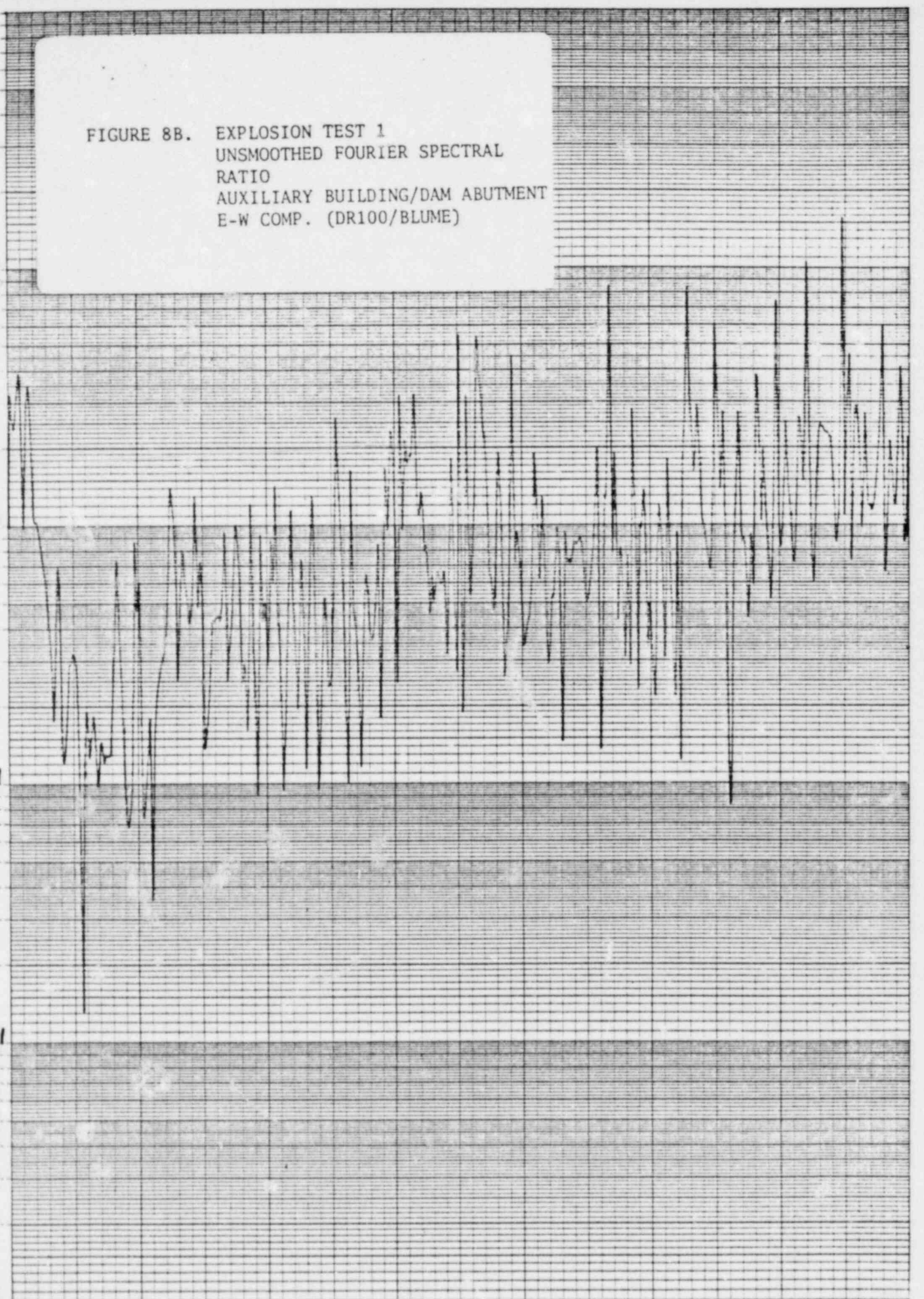


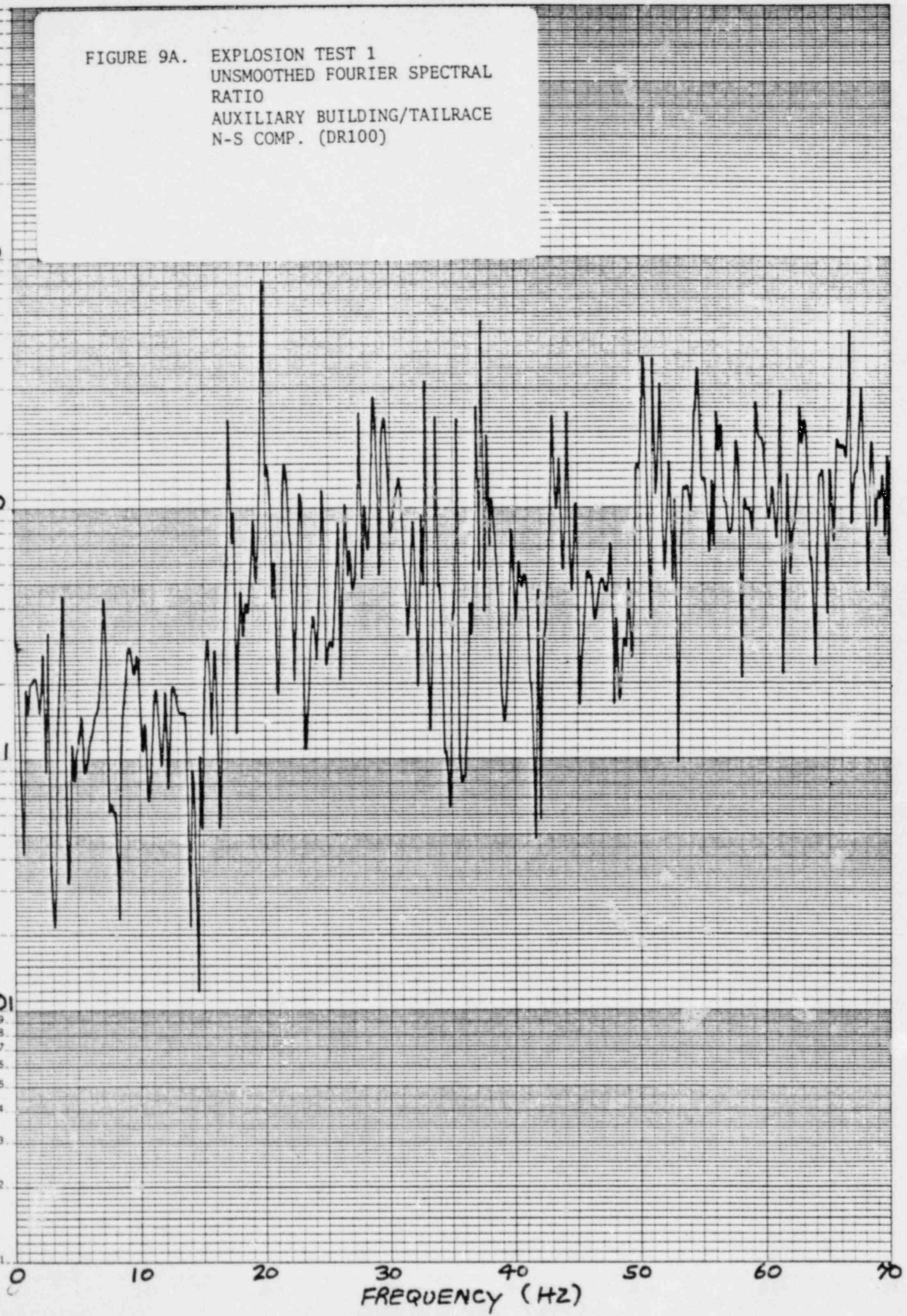
FIGURE 9A. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRAL
RATIO
AUXILIARY BUILDING/TAILRACE
N-S COMP. (DR100)

46 6210

K·E SEMI-LOGARITHMIC 5 CYCLES X 20 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

SPECTRAL RATIO

10
1
9
8
7
6
5
4
3
2
1
0.1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.01
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1



FREQUENCY (HZ)

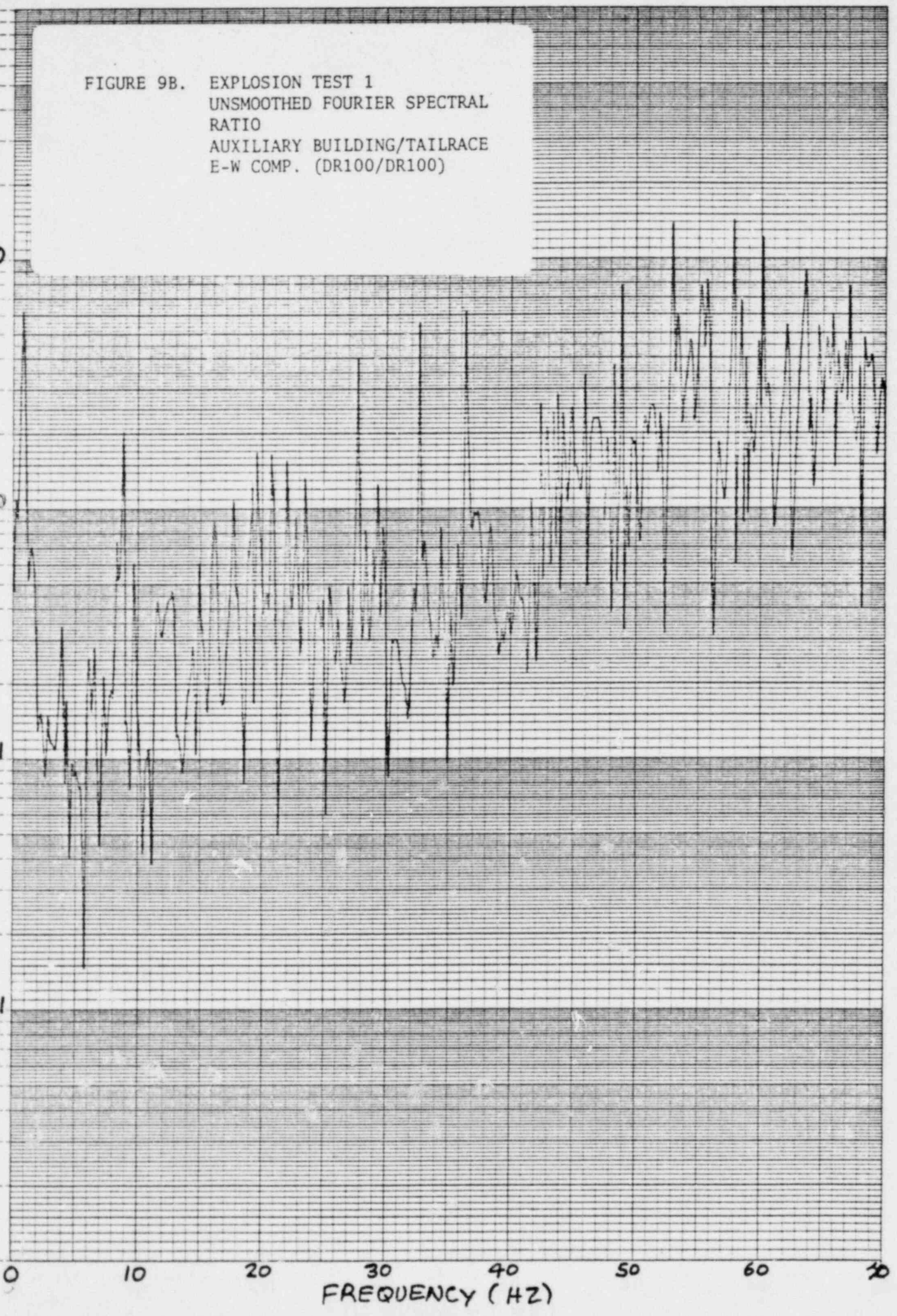
FIGURE 9B. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRAL
RATIO
AUXILIARY BUILDING/TAILRACE
E-W COMP. (DR100/DR100)

46 6210

SPECTRAL RATIO

K-E SEMI-LOGARITHMIC 5 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

10
10
0.1
0.01



FREQUENCY (HZ)

FIGURE 10. EXPLOSION TEST 1
UNSMOOTHED FOURIER SPECTRAL
RATIO
AUXILIARY BUILDING/
METEOROLOGICAL TOWER
E-W COMP. (DR100/DR100)

46.6210
SPECTRAL RATIO

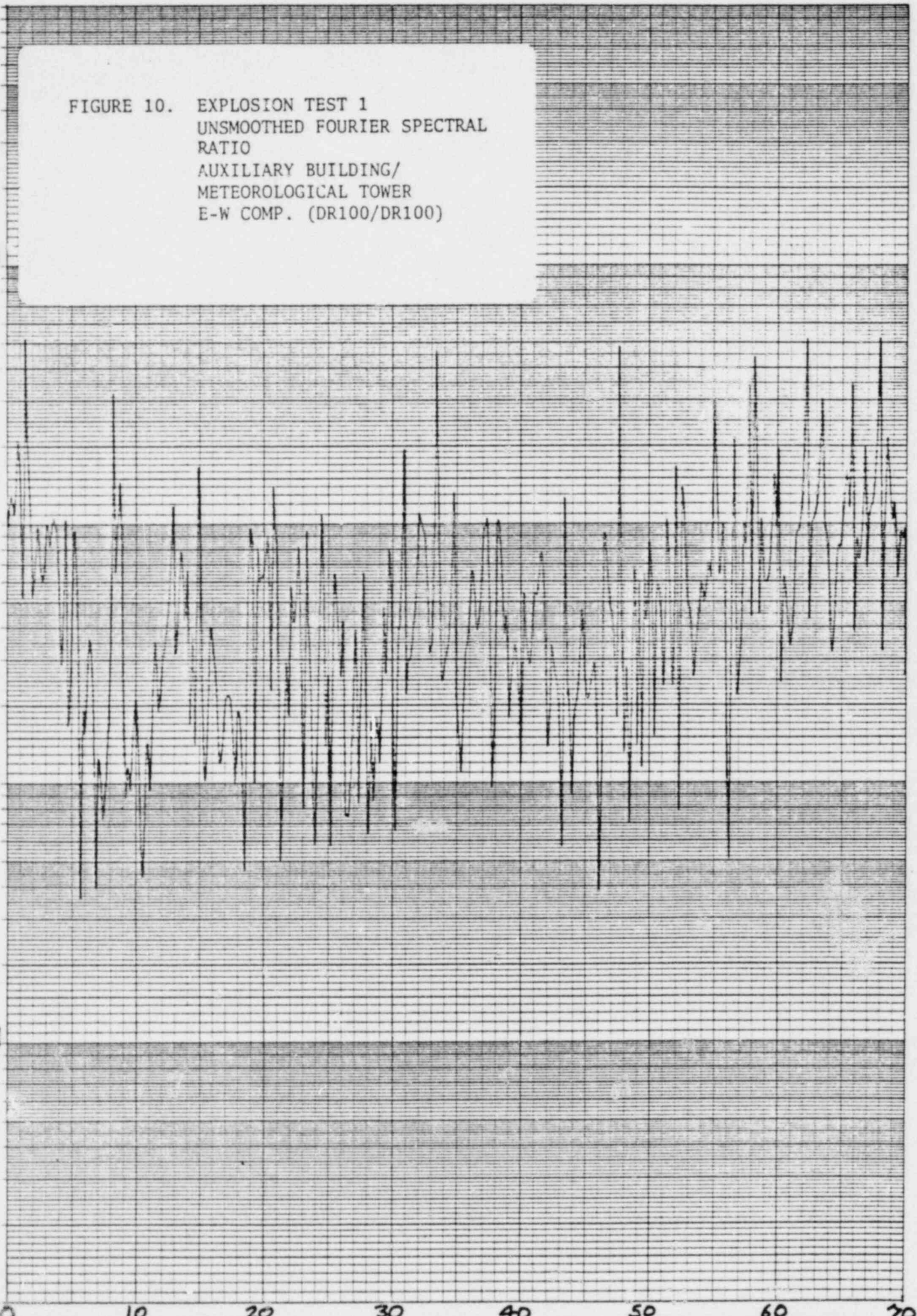
K-E SEMI-LOGARITHMIC 5 CYCLES X 70 DIVISIONS
KEUFFEL & ESSER CO. MADE IN U.S.A.

10

0.1

0.01

FREQUENCY (HZ)



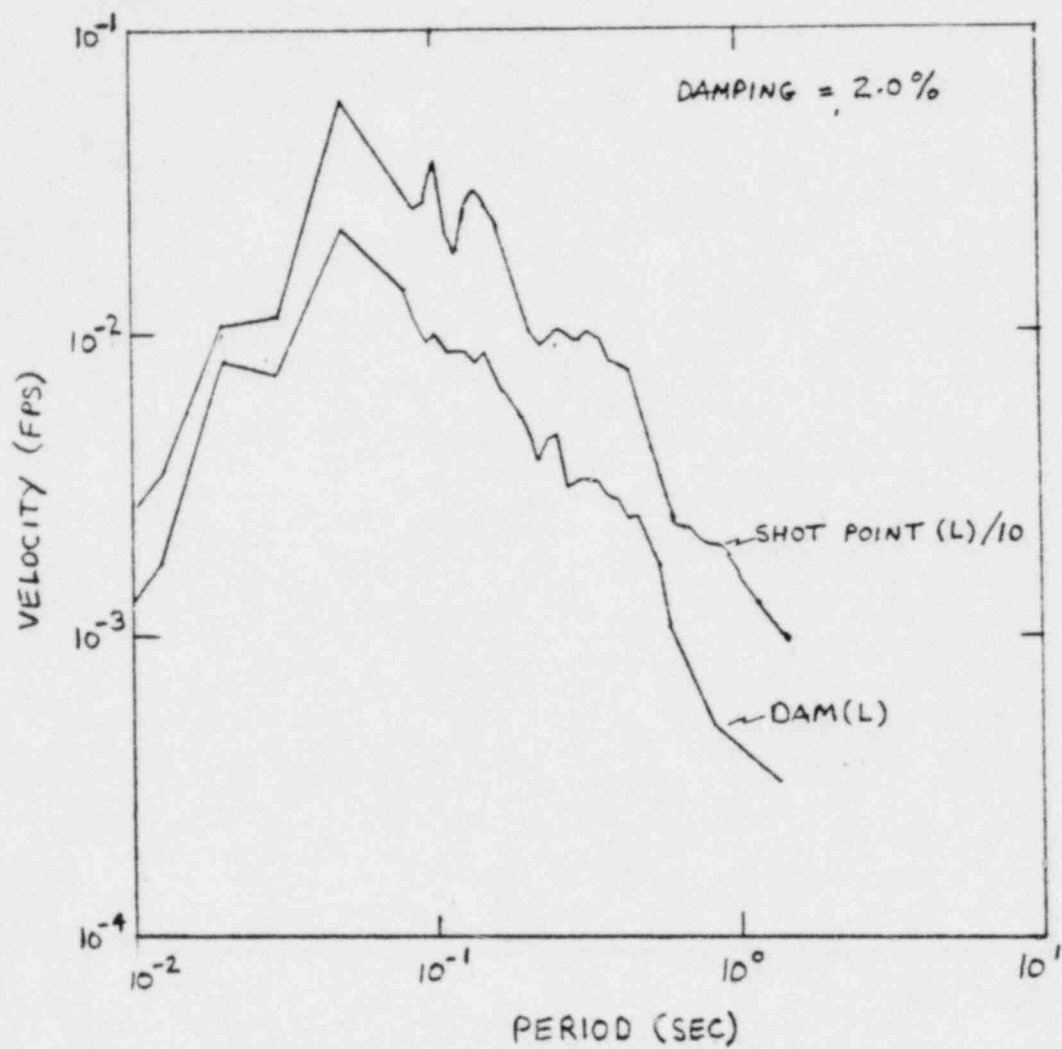
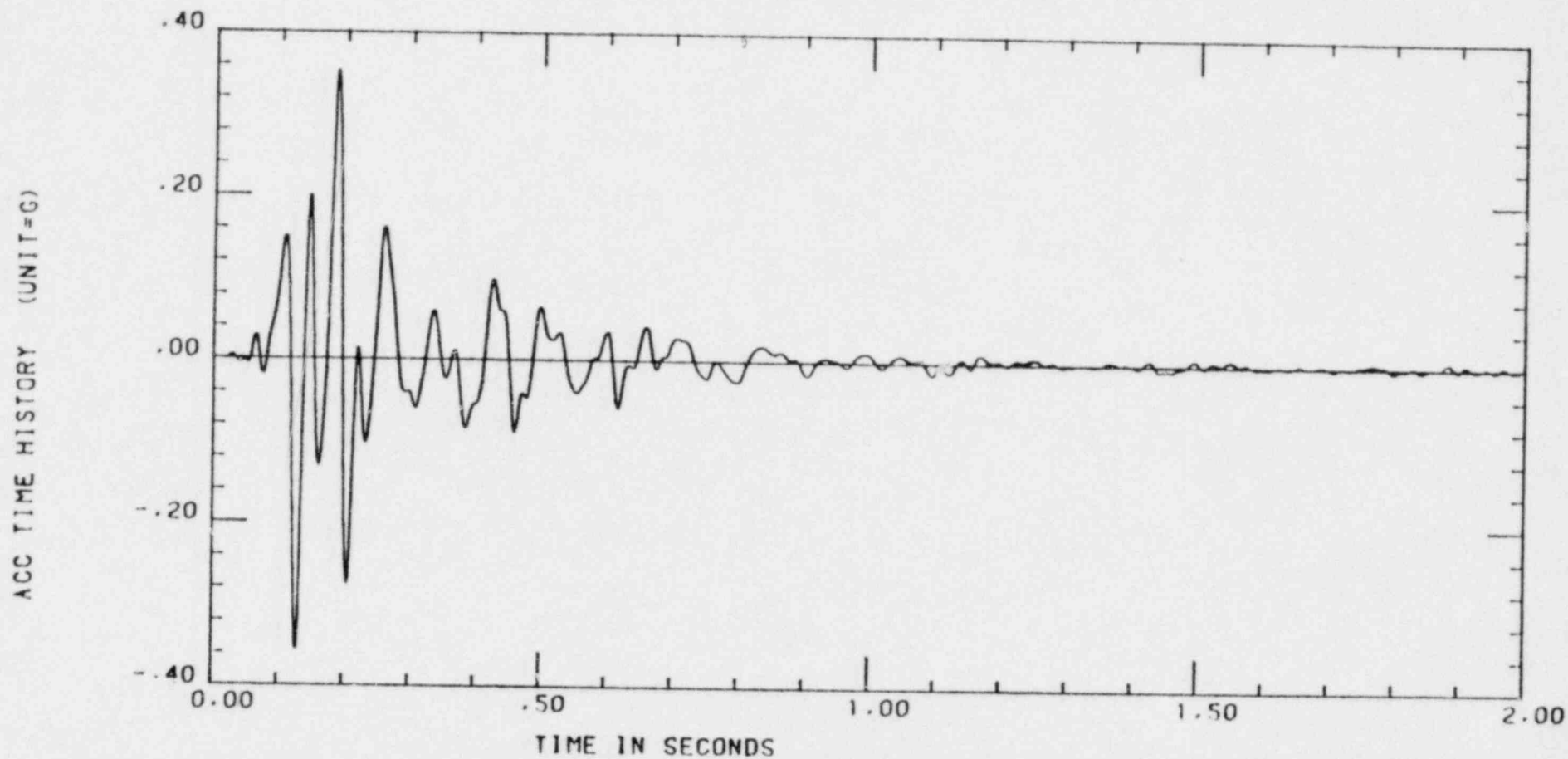


FIGURE 11. EXPLOSION TEST 2
 COMPARISON OF RESPONSE SPECTRA
 AT SHOTPOINT 2 AND (IN DAM
 ABUTMENT

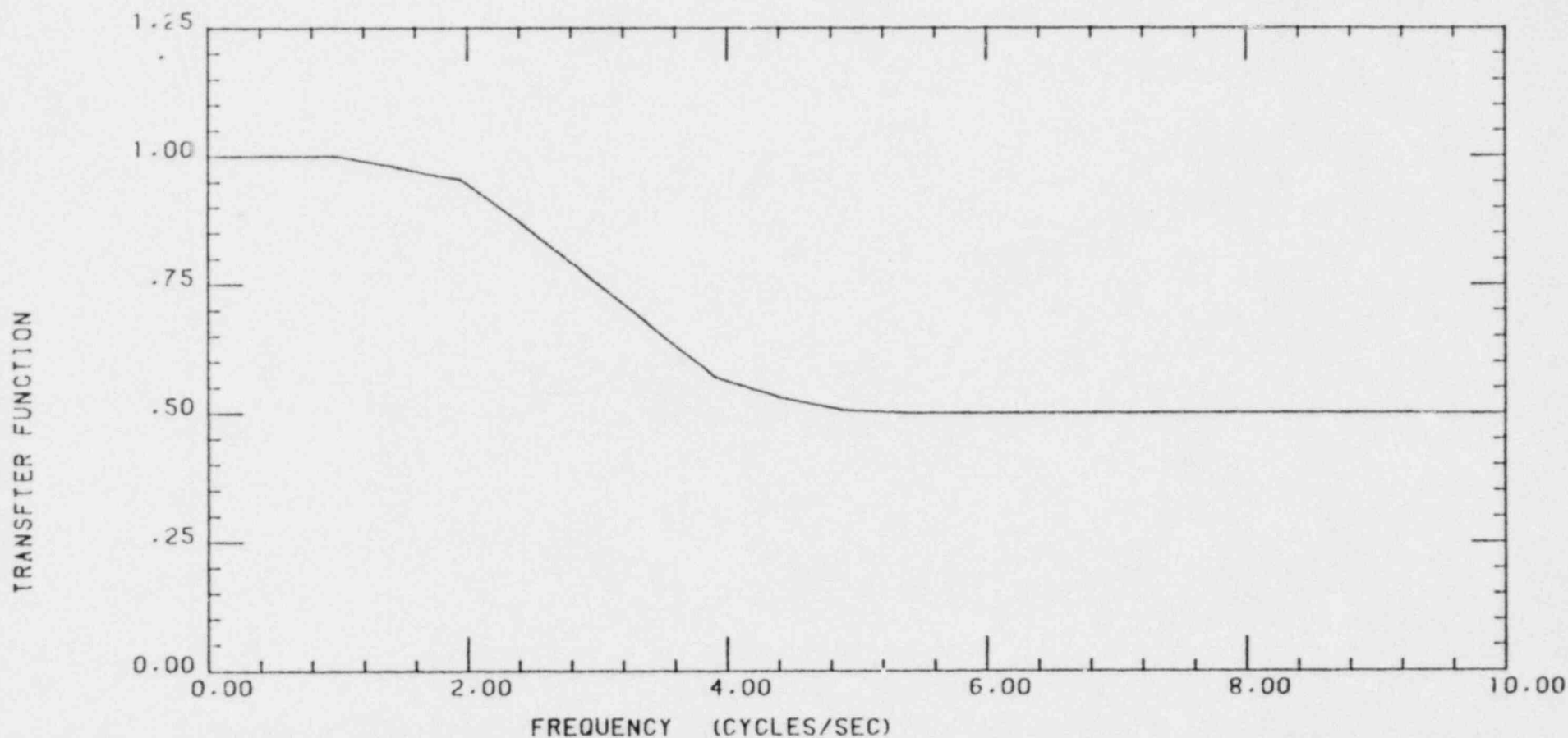
MONTICELLO DAM DATE OF RECORD=10/16/79
CEN CREST COMP 90DEG



PGA=.356G NPTS=1000 DT=.002

FIGURE 12. USGS- CORRECTED ACCELEROGRAM
EARTHQUAKE OF 10/16/79
RECORDED AT MONTICELLO DAM

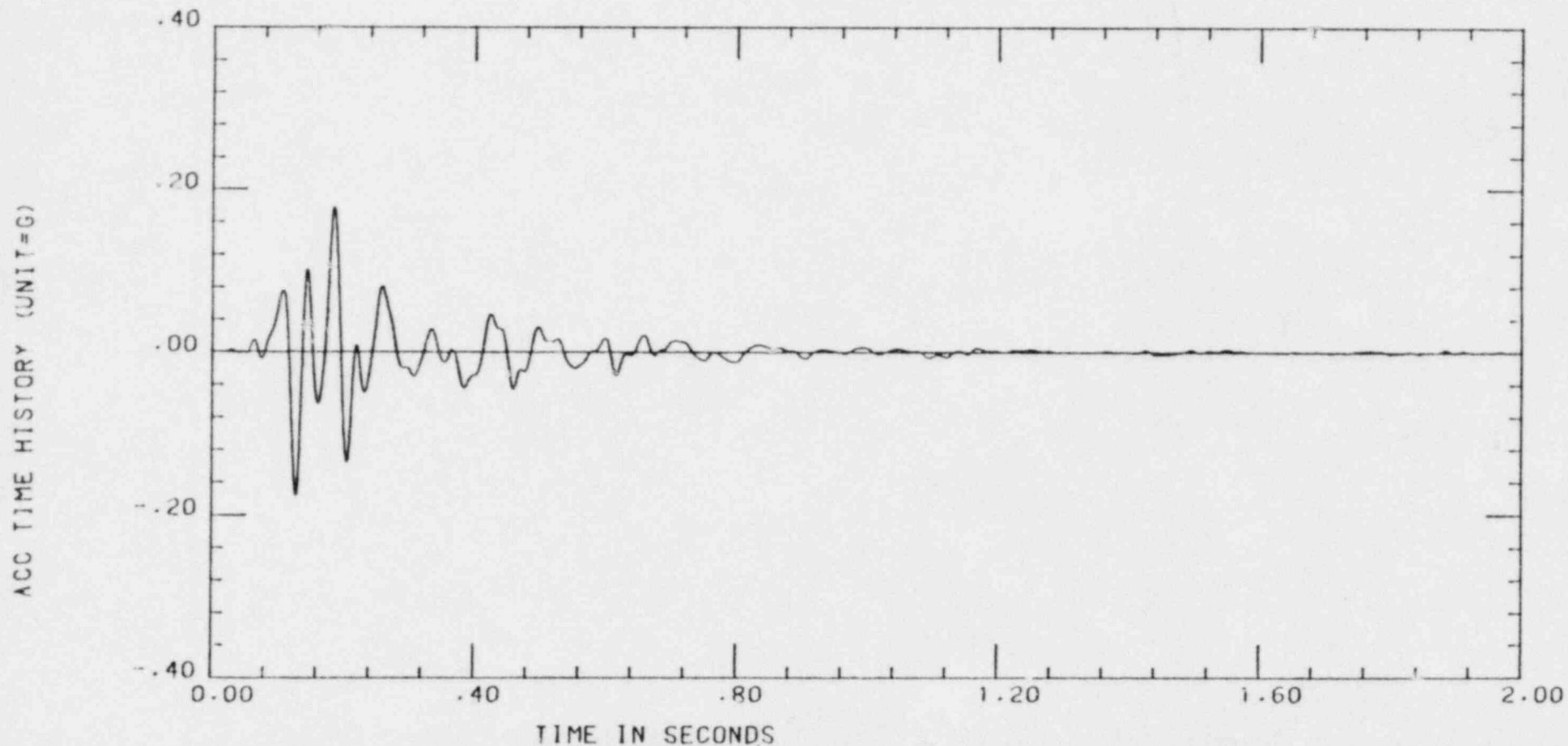
MULTIPLICATION FACTORS TO BOTH REAL AND
IMAGINARY COEFF. OF FFT



MONTICELLO DAM 10/16/79 90DEG

FIGURE 13. EMPIRICAL TRANSFER FUNCTION
FOUNDATION (BEDROCK)/FREE FIELD (SAPROLITE)

MODIFIED ACC T-H AFTER MULTIPLIED BY
THE TRANFER FUNCTION



MONTICELLO DAM 10/16/79 90 DEG
PGA = .1781G

FIGURE 14. FILTERED ACCELEROGRAM
EARTHQUAKE OF 10/16/79
RECORDED AT MONTICELLO DAM

APPLICANT EVALUATION OF TRIFUNAC REPORT ON
VIRGIL C. SUMMER NUCLEAR STATION
SEISMICITY STUDIES

Malcolm R. Somerville

INTRODUCTION

Professor M.D. Trifunac has written comments on seismic studies performed in connection with the Virgil C. Summer Nuclear Station, dated September, 1981. Trifunac's comments are based, in large part, on studies that are not applicable to the site, or on a misunderstanding of the bases for the seismic studies submitted by the Applicant. In order to clarify these issues, the Applicant addresses the points that Trifunac raises.

INSTRUMENTAL VERSUS DESIGN ACCELERATION

Trifunac's comments imply that design or effective acceleration (the acceleration at zero period in a spectral response diagram; i.e., the acceleration used to "anchor" the design spectrum) can be compared directly with the peak instrumental acceleration. For example, in his Figure 1, which is referred to repeatedly, the OBE and SSE vertical lines at 0.10g and 0.15g, respectively, are compared, or mingled with, a_0 , the peak "instrumental" acceleration. Because strong motion accelerographs may record high frequency acceleration pulses that have no effect on structures, particularly for ground motions close to the causative fault, the two are not equivalent. The Diablo Canyon plant, as a recent example, uses 1.15g instrumental acceleration and 0.75g effective or design acceleration, which value was upheld after years of hearings. The Diablo Canyon ratio is 0.75/1.15 or 0.65. Trifunac generally has agreed with such concepts as can be inferred from statements in his scientific papers, e.g.: "... serious damage to structures comes predominantly from long shaking and not from one or two high-frequency, high-acceleration pulses which, because of their short duration, may represent only small, impulsive excitation." (from Trifunac, 1972). Also: "Finally, it should be pointed out here that from the practical earthquake engineering point of view, high acceleration amplitudes should not

necessarily be associated with a proportionally higher destructive potential. An extended duration of strong ground motion and high acceleration amplitudes characterize destructive earthquake shaking, while one or several high-frequency high-accleration peaks may, in fact, constitute only minor excitation because of the short duration involved and may lead to only moderate or small impulses when applied to structural system." (from Trifunac, 1976). ^{1/}

ESTIMATES OF PEAK ACCELERATION

Trifunac's commentary rests very largely on regression analyses performed by himself, either individually or with associates. Work by other authors on regression analysis of strong motion data is ignored. On the second page of the section entitled "General Considerations," Trifunac begins the first full paragraph with the following statement: "The body of the strong motion data which is now available is not adequate to find the form of the distribution functions of the amplitudes of peak recorded ground accelerations." Concerning this statement, there are two pertinent comments. First, in making this statement, Trifunac renders his commentary unexaminable. Second, the statement is not correct. For example, the distribution of peak ground accelerations, for given levels of MM intensity, was studied by Murphy and O'Brien (1977). Attached is Figure 1 from Murphy and O'Brien, showing distributions of a set of 67 pairs (two horizontal components) of peak ground acclerations corresponding to MM intensity VI. These data are from a study by Trifunac and Brady (1975). Two distributions are shown: one about the arithmetic mean (82.46 cm/sec^2) and the other about the geometric mean (51.98 cm/sec^2). The distribution about the geometric mean matches the normal distribution quite well, i.e., peak ground accelerations are approximately lognormally distributed. This refutes Trifunac's claim that the data now available are not adequate to find the form of the distribution function.

Note that the arithmetic mean exceeds the geometric mean by a substantial margin. This largely accounts for the difference between the Trifunac and Brady (1975) and Murphy and O'Brien (1977) intensity-acceleration correlations. Trifunac and Brady computed arithmetic means

^{1/} i.e., Trifunac (1976 b).

of peak accelerations for each intensity level (their Table 3) and then fitted these means with a linear equation relating the logarithm of peak acceleration with intensity (their equation 1). Because Trifunac and Brady assumed normal rather than lognormal distribution of peak accelerations, the results of their regression analysis are seriously biased, as shown by Murphy and O'Brien (1977).

In subsequent work, both with intensity and magnitude data, Trifunac (1976a, b) performed regressions using the logarithms of peak ground accelerations, but adopted an unorthodox regression scheme, the statistical meaning of which cannot be ascertained. In his work on intensity, Trifunac (1976a) used the same data set as did Trifunac and Brady (1975). Comparison of the results of these studies shows that Trifunac (1976a) obtained practically the same mean values as Trifunac and Brady (1975), indicating that he again used arithmetic rather than geometric averaging. The results are not directly comparable because Trifunac (1976a) includes site geology as a regression parameter. In doing so, he reduces the population of his data cells considerably.

Tables 1, 2, and 3 compare various estimates of peak horizontal ground acceleration for Modified Mercalli intensities VI, VII, and VIII. Estimates are given according to Murphy and O'Brien (1977; equation 9), Trifunac and Brady (1975; equation 1 and Table 3), and Trifunac (1976a; Table III). The means given in Figure 1 of the Trifunac report correspond to Trifunac (1976a) for $s = 2$ (rock sites). These exceed the expectations given by Murphy and O'Brien (1977) by factors of about 2. The acceleration given by Trifunac for intensity VII (177.8 cm/sec^2) equals the expectation of Murphy and O'Brien (1977) for intensity VIII.

The results of regression analysis similarly performed by Trifunac (1976b) using magnitude data are likewise marred by erroneous statistical treatment.^{1/} Such work should not be used in appraising peak ground accelerations for the Virgil C. Summer Nuclear Station.

As noted above, a further difficulty in applying Figure 1 of Trifunac in assessing design accelerations is that the difference between ^{1/} i.e., Inapplicable distribution function.

peak instrumental acceleration and design acceleration is overlooked. If adjusted for statistical error and for the difference between design and free-field instrument acceleration, Trifunac's Figure 1 would indicate that the SSE design acceleration is appropriate for ground shaking of MM intensity VII, or ground motion due to an earthquake of magnitude 5 to 5.5 occurring in the immediate vicinity of the site.

In summary, the methodology used by Trifunac in estimating peak accelerations for given intensities and magnitudes leads to over-estimation of acceleration. Thus conclusions regarding the inadequacy of the SSE are inappropriate.

VERTICAL ACCELERATIONS

Trifunac suggests that the ratio of peak vertical accelerations to peak horizontal accelerations should be close to 1, and cites a "number of recent recordings" to substantiate this view. These recordings are apparently from magnitudes greater than 6. For smaller magnitudes in the range 4 to 6, the vertical-to-horizontal acceleration ratio is closer to 0.5.^{1/} This was documented by the Applicant in section 361 of the FSAR, Figures 361.17.4-20 through 361.17.4-23. The data from the Monticello accelerograph support this: The ratio for the 27 August 1978 earthquake, computed as the vertical peak divided by the average of the two horizontal peaks, is 0.34.

SOIL AMPLIFICATION

Trifunac questions the Applicant's and NRC staff's conclusion that the August 27, 1978, earthquake recording on a soil site represents an amplification of wave motion through the soil. To support his argument he cites Trifunac and Brady (1975) to assert that "average of peak accelerations recorded on rock is higher than the average of acceleration recorded on soil and alluvium."^{2/} More recent studies, by Campbell (1981) and Joyner and Boore (1981), which include consideration of near-field records, conclude that level of accelerations recorded on soil and rock are similar.^{3/} The accelerations discussed above refer to free-field accelerations. The potential that the SMA recording represents an amplified response of a natural hill-like structure is discussed below.

^{1/} We note that Dr. Luco reports observations consistent with those of Applicant on this matter.

^{2/} i.e., For a given intensity level.

^{3/} i.e., For the same magnitude and distance.

The SMA instrument is located on the abutment between Monticello Dams B and C. An examination of the topography of this region indicates that the instrument site is located almost at the top of a hillock that is partly man-made and partly natural. The surrounding region slopes down rapidly around the area formed by the dam crests and the abutment area with the surface elevation of 300' in the region of epicenter. Thus, the SMA recording, in all likelihood, does represent an amplification of the hillock responding to the free field acceleration. Nonetheless, the Applicant has conservatively assumed that no such amplification has occurred in its use of the SMA recording of the August 27, 1978, earthquake to evaluate earthquake source parameters.

EARTHQUAKE STRESS DROPS

Trifunac states that stress drop estimates in California are highly variable; this is certainly true when these estimates are made in the frequency domain from long period level and corner frequency observations. However, when stress drop estimates are made from time domain data (specifically, observations of a_{rms}), they are quite stable and invariant for California earthquakes (Hanks and McGuire, 1981). The latter is the methodology used in deriving an appropriate stress drop value to characterize reservoir-induced earthquakes at Monticello.

The comparisons of peak-acceleration-to-stress-drop ratio by Trifunac is invalid. The stress drops used by the Applicant are derived from a_{rms} ; those cited in Trifunac's references are determined by spectral methods, which are often one-tenth the value determined by a_{rms} for the same earthquake (Hanks and McGuire, 1981). Thus the discrepancy found by Trifunac is easily explained by the factor of ten difference in stress drop estimates by various methods, and does not imply that the Applicant's peak acceleration estimates are low.

PROBABILITY STUDIES

Trifunac finds, in his Tables 1 and 2, return periods for the SSE that are substantially different from those presented by the Applicant in Tables 361.19-1 and 361.19-2. The Applicant's analysis was based

on several sets of seismogenic zones: the zones used for the FSAR are available in that document and are shown in Figure 2, and the zones proposed by Algermissen and Perkins (1976) are reproduced in Figure 3. Both allow tectonic events to occur at the site.

There are several reasons why Trifunac finds larger probabilities than those of the Applicant. First, he uses the recurrence curve of Chinnery (1979) for the southeastern United States. This is a combination of Bollinger's (1973) South Carolina-Georgia seismic zone and Southern Appalachian seismic zone. Since the latter has more historical seismicity than the former (see Figure 4), combining the two increases the perceived hazard for any site within the former zone (such as the Virgil C. Summer Nuclear Plant). No investigator, to the Applicant's knowledge, has proposed combining these zones for the purposes of determining seismic hazard; Chinnery's (1979) investigation had the purpose of comparing general seismicity characteristics in different parts of the eastern United States, not calculating seismic hazard at sites.

The second difference is in the attenuation curves that are used to estimate ground motion characteristics. The Applicant has used, for Modified Mercalli (MM) intensity, an equation based on MM intensity observed during the 1886 Charleston earthquake, which is the most extensive data base available for the southeastern United States. For acceleration an equation developed by Nuttli for the central United States was used. These attenuation functions are described in section 361.19-4 of the FSAR, and are the most site-specific, least interpretive attenuation equations available. Those used by Trifunac are described in NUREG/CR-689 and estimate spectral velocities as a function of earthquake intensity and distance. While this is a novel approach,^{1/} there are no eastern U.S. earthquake data with which to judge its appropriateness, nor has this methodology received substantial peer review. Thus the use of this equation to make probability calculations and statements results in highly tenuous conclusions that should be viewed with caution.

STRUCTURAL DAMPING

The primary reason for using 7 percent, instead of 2 percent, damping is due to the fact that 7 percent is more realistic than 2

^{1/} Applicant chose the more conventional and widely accepted approach, similar to that method used by Algermissen and Perkins.

percent during a 0.22g near-field earthquake for structures originally designed for a 0.15g far-field earthquake, and not solely because it is permitted by the Regulatory Guide. The 7 percent damping was verified by test data that were discussed extensively in the Diablo Canyon ALAB hearings. The decision of the same ALAB hearings acknowledged that 7 percent damping is appropriate.

The effect of structural damping used in the analysis is to control the amplified motion from the input to the top of the building such that the amplification factor matches the recorded data in general. In the reevaluation of the Virgil C. Summer Station design, the resulting amplification factor based on 7 percent damping was 3.0, which is generally higher than recorded amplifications. In the original design with 2 percent damping, an amplification of 4.75 was obtained. This large amplification factor is totally unrealistic. The value of 7 percent was used to provide calculation of realistic, but still conservative, structural response.^{1/}

EFFECTIVE ACCELERATION^{2/}

Trifunac disputes the SER statement that "the finite size of large structures would attenuate high frequencies" claiming that it has not been demonstrated so far, and that it does not reduce the high frequency input motions significantly and systematically to warrant its use in design calculations. The reference cited for this claim (Feng, et al., 1982) is unavailable to the Applicant. However, in a recent study, Campbell (1981) reports comparisons between small building/free-field recordings (115 components) at ground level, and recordings obtained in the lowest basement of large buildings (40 components). Campbell found that peak acceleration recorded in the basement of large buildings was on the 24^{3/} percent lower than that recorded at ground level.^{4/} This result was found to be significant at the 90 percent confidence level.

1/ The damping values of 5% and 7% conform to R.G. 1.61 and are further discussed in testimony by Dr. Blume.

2/ A more accurate title for this section is: ATTENUATION EFFECTS OF LARGE FOUNDATIONS (The subject matter of this section and related phenomena are discussed in testimony by Dr. Blume).

3/ i.e., The average 24 percent.

4/ i.e., In small buildings or in the free-field.

REFERENCES

- Algermissen, S.T., and D.M. Perkins (1976), "A Probabilistic Estimate of Maximum Acceleration in Rock in the Contiguous United States," USGS Open-File Report 76-416, 45 pp.
- Bollinger, G.A. (1973), "Seismicity and Crustal Uplift in the South-eastern United States," American Journal of Science, Cooper vol. 273-A, pp. 390-408.
- Campbell, Kenneth W. (1981), "Near-Source Attenuation of Peak Horizontal Acceleration," submitted to the Bulletin of the Seismological Society of America.
- _____ (1981), "Near-source Attenuation of Peak Horizontal Acceleration, submitted to the Bulletin of the Seismological Society of America.
- Chinnery, M.A. (1979), "A Comparison of Seismicity of Three Regions of the Eastern United States," Bulletin of the Seismological Society of America, vol. 69, pp. 757-772.
- Hanks, T.C., and R.K. McGuire (1981), "The Character of High Frequency Strong Ground Motion," Bulletin of the Seismological Society of America, vol. 71, Dec. (in press).
- Joyner, William B., and David M. Boore (1981), "Peak Horizontal Acceleration and Velocity from Strong Motion Records, including Records from the 1979 Imperial Valley, California, Earthquake," submitted to the Bulletin of the Seismological Society of America.
- Murphy, J.R., and O'Brien, L.J. (1977), "The Correlation of Peak Ground Acceleration with Seismic Intensity and Other Physical Parameters," Bulletin of the Seismological Society of America, vol. 67, pp. 877-915.
- Trifunac, M.D. (1972), "Stress Estimates for the San Fernando, California, Earthquake of February 9, 1971: Main Event and Thirteen Aftershocks," Bulletin of the Seismological Society of America, vol. 62, pp. 721-750.
- _____ (1976a), "A Note on the Range of Peak Amplitudes of Recorded Accelerations, Velocities and Displacements with respect to the Modified Mercalli Intensity, Earthquake Notes, vol. 47, no. 2, pp. 9-24.
- _____ (1976b), "Preliminary Analysis of the Peaks of Strong Earthquake Ground Motion - Dependence of Peaks on Earthquake Magnitude, Epicentral Distance and the Recording Site Conditions," Bulletin of the Seismological Society of America, vol. 66, pp. 189-219.
- Trifunac, M.D., and A.G. Brady (1975), "On the Correlation of Seismic Intensity Scales with the Peaks of Recorded Strong Ground Motion," Bulletin of the Seismological Society of America, v. 65, pp. 139-162.

TABLE 1

PEAK HORIZONTAL GROUND ACCELERATION
ESTIMATES FOR MM INTENSITY VI ^{1/}

Author	Assumed distribution function	Expected acceleration (cm/sec ²)	Accelerations for -/+ 1 standard deviation (cm/sec ²)
Murphy & O'Brien (1977): Eq. (9)	Lognormal	56.23	24.55/128.78
Trifunac & Brady (1975): Eq. (1) Table 3	Normal	65.16 82.46	4.79/160.13
Trifunac (1976a)			
S=0 (alluvium)	?	46.77	
S=1 (intermediate)		66.07	
S=2 (rock)		91.02	

^{1/} Although the calculations give results to 2 decimal places as reflected here, accuracy is not implied beyond the decimal point.

TABLE 2

PEAK HORIZONTAL GROUND ACCELERATION
ESTIMATES FOR MM INTENSITY VII ^{1/}

Author	Assumed distribution function	Expected acceleration (cm/sec ²)	Accelerations for +/- 1 standard deviation (cm/sec ²)
Murphy & O'Brien (1977): Eq. (9)	Lognormal	100.00	43.67/229.09
Trifunac & Brady (1975): Eq. (1) Table 3	Normal	130.02 131.29	69.99/192.59
Trifunac (1976a)			
S=0 (alluvium)	?	93.33	
S=1 (intermediate)		128.82	
S=2 (rock)		177.83	

^{1/} Although the calculations give results to 2 decimal places as reflected here, accuracy is not implied beyond the decimal point.

TABLE 3

PEAK HORIZONTAL GROUND ACCELERATION
ESTIMATES FOR MM INTENSITY VIII ^{1/}

Author	Assumed distribution function	Expected acceleration (cm/sec ²)	Accelerations for -/+ 1 standard deviation (cm/sec ²)
Murphy & O'Brien (1977): Eq. (9)	Lognormal	177.83	77.65/407.23
Trifunac & Brady (1975): Eq. (1) Table 3	Normal	259.42 166.67	82.61/250.73
Trifunac (1976a)			
S=0 (alluvium)	?	181.97	
S=1 (intermediate)		251.19	
S=2 (rock)		346.74	

^{1/} Although the calculations give results to 2 decimal places as reflected here, accuracy is not implied beyond the decimal point.

FIGURE 1

(Taken from Murphy and O'Brien, 1977)

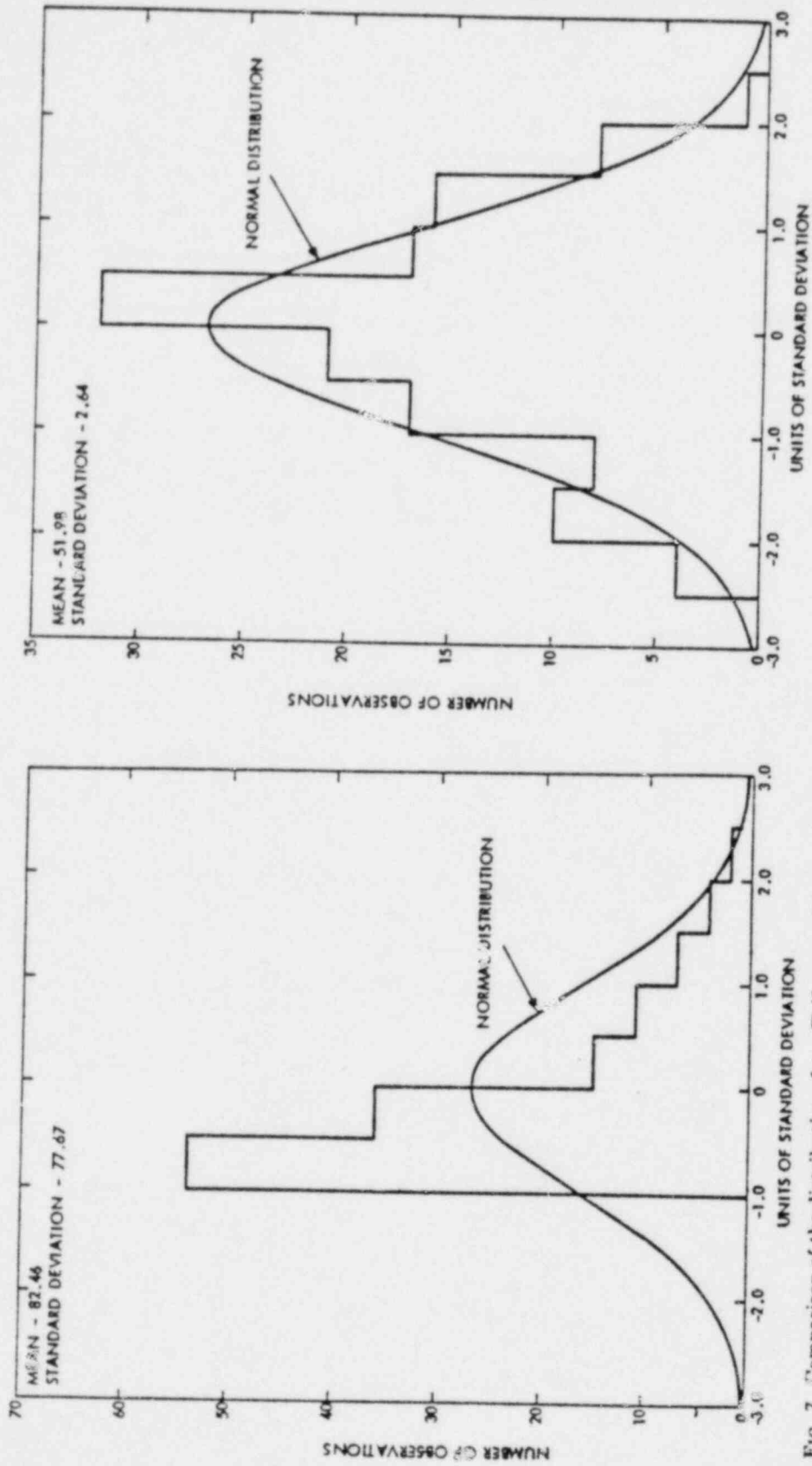


Fig. 7. Comparison of the distribution of the Trifunac and Brady (1975) sample with respect to the arithmetic (left) and logarithmic (right) means, Modified Mercalli Intensity VI.

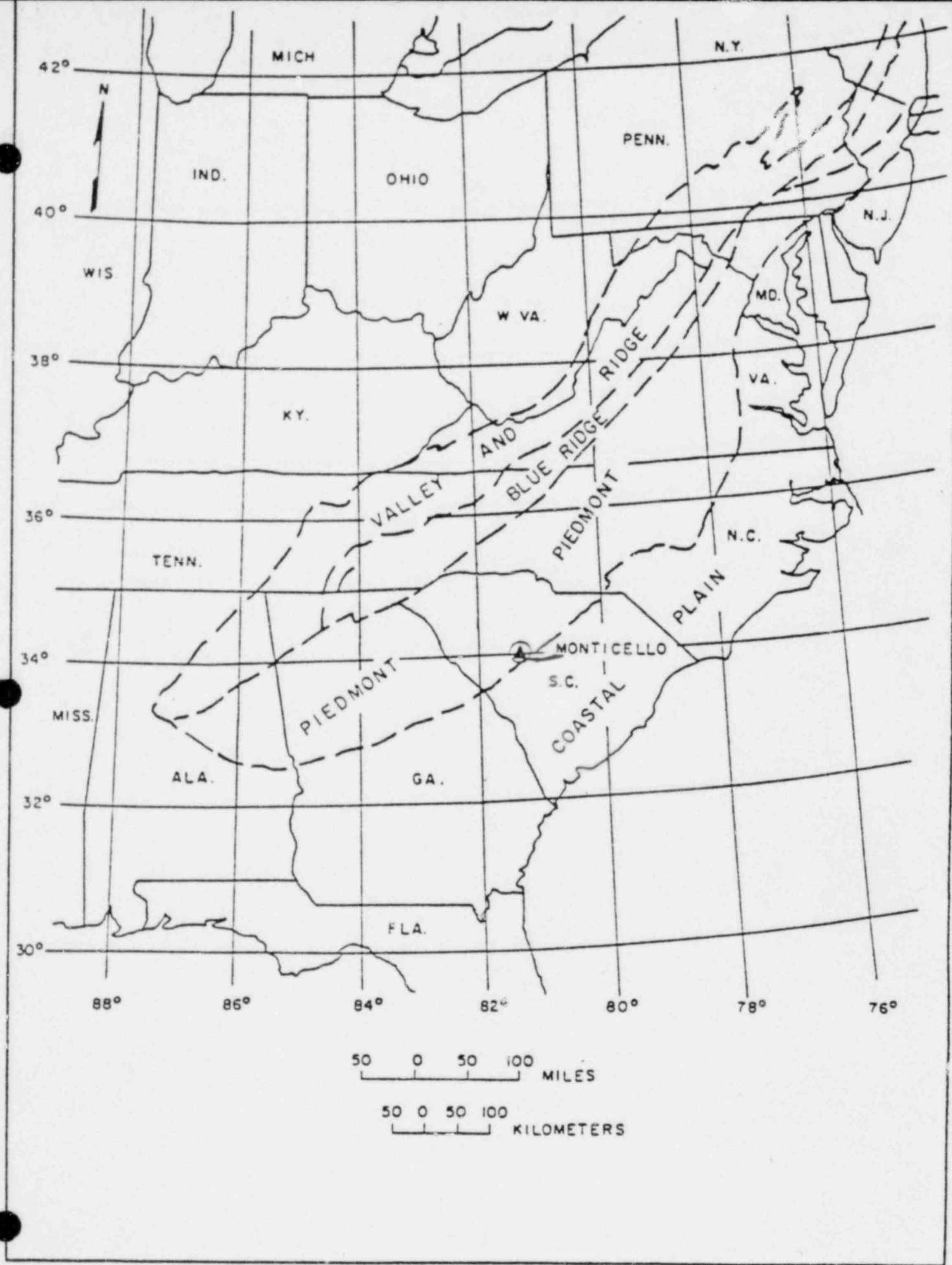


Figure B/ 2
SEISMOGENIC ZONES USED IN FSAR

Correction

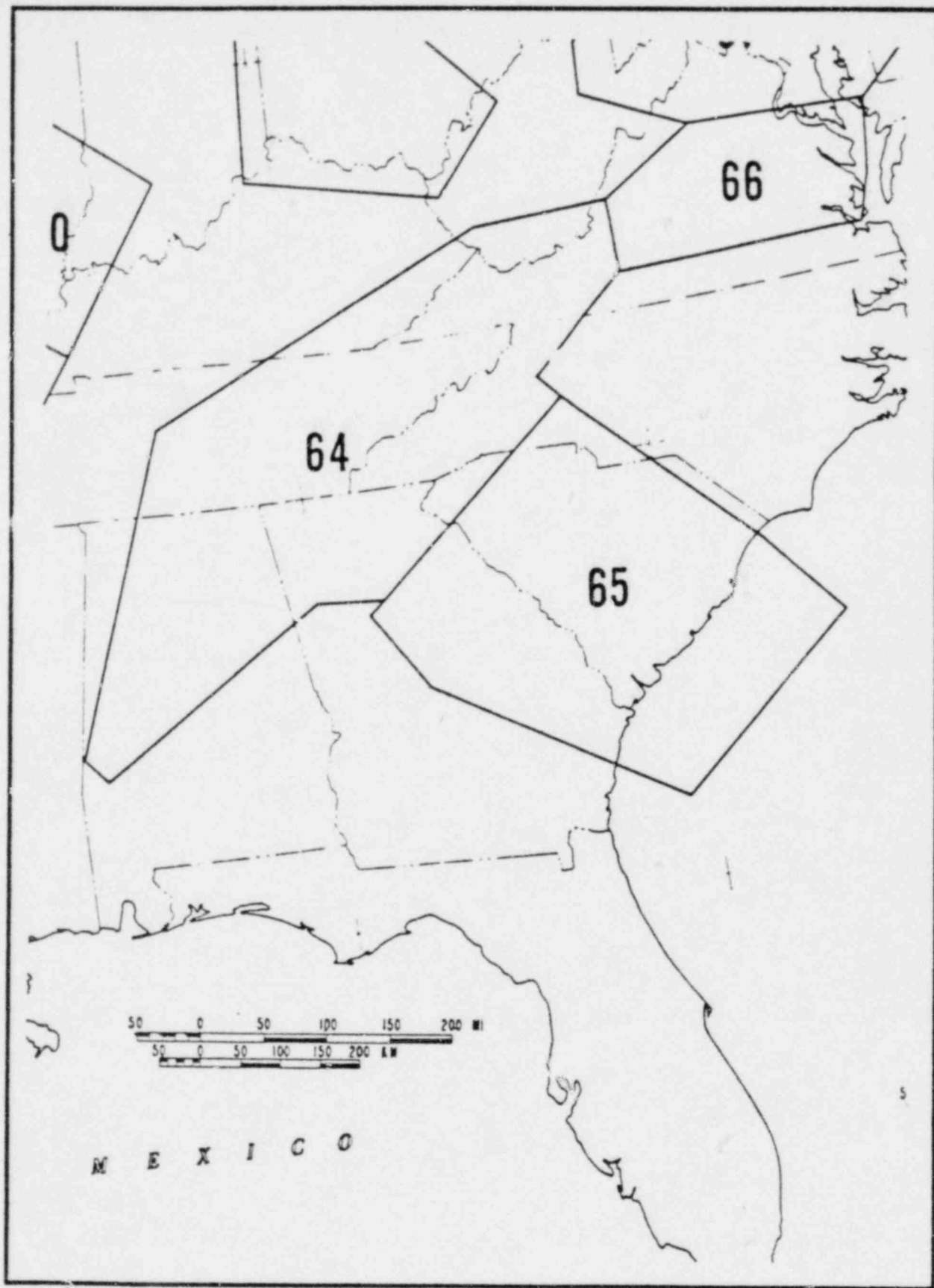


Figure 4³

SEISMOGENIC ZONES IN SOUTHEAST U.S.
USED BY ALGERMISSSEN AND PERKINS (1976)

1/ Correction of Figure No.

FIGURE 2 4
(Taken from Bollinger, 1973)

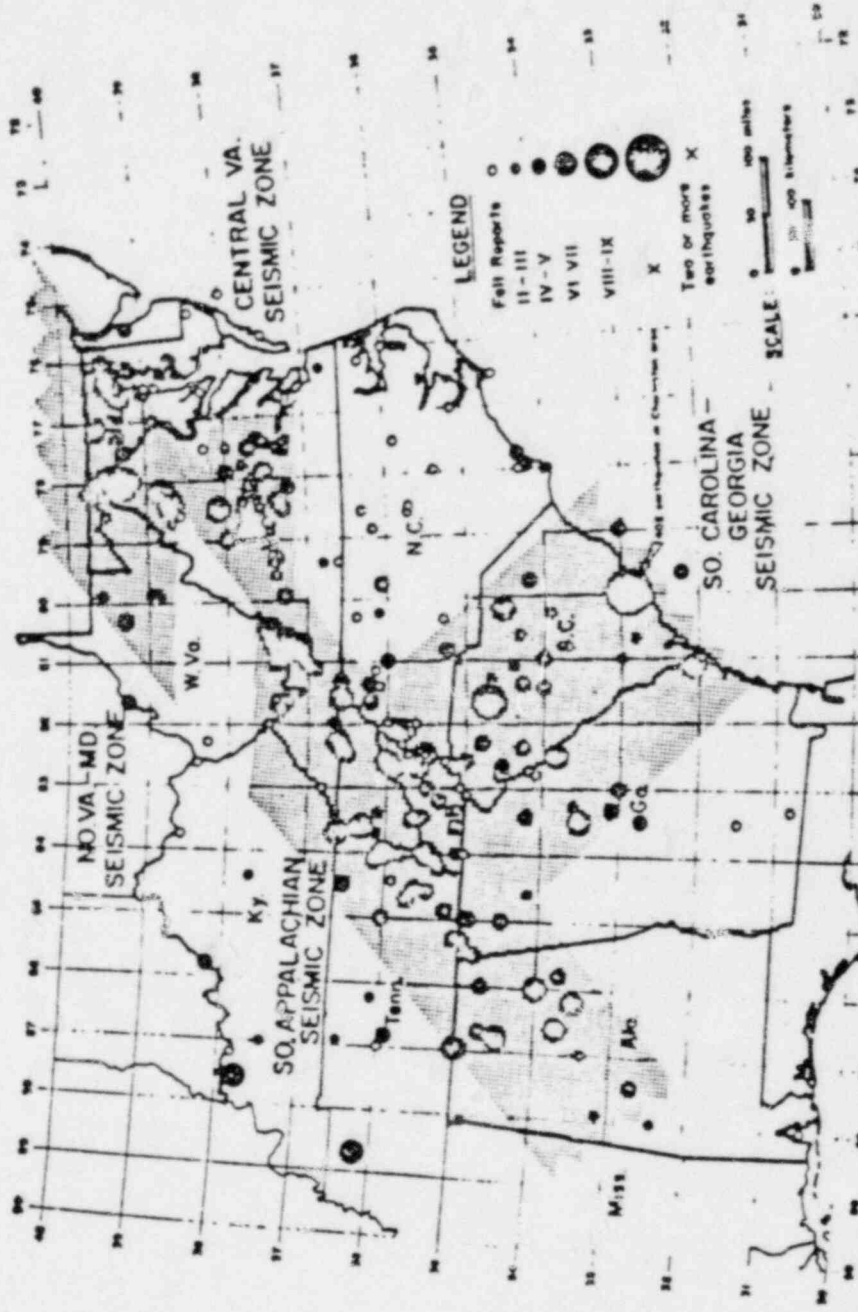


Fig. 3. Seismic zones in the Southeastern United States.

PROFESSIONAL QUALIFICATIONS

MALCOLM R. SOMERVILLE

My name is Malcolm R. Somerville. I am employed by URS/John A. Blume & Associates, Engineers of San Francisco, California, as Project Seismologist.

I received a B.Sc. Degree in Physics in 1966 from the University of New England, Australia. In 1973 I received a M.A. Degree in Geophysics from the University of California, Berkeley, and in 1977 received my Ph.D. Degree in Geophysics from the University of California.

I was employed as a Teaching and Research Assistant at the University of California in their Geology and Geophysics Department from 1968-1974.

From 1974-1977, I was employed as an Associate Seismologist with URS/John A. Blume & Associates, Engineers.

From 1978-1979, I was a Postdoctoral Research Fellow in the Seismological Laboratory of the California Institute of Technology.

From 1979-1980, I was an Assistant Professor of Geophysics/Research Seismologist with the University of Nevada at the Mackay School of Mines.

I have held my present job with URS/John A. Blume & Associates since 1980.

I have expertise in seismology, seismotectonics, and tectonophysics. My studies have included investigations on the seismicity and seismotectonics of California and Nevada

and experimental and theoretical work on the equation of state of solids.

I have been responsible for the development and review of ground motion criteria for numerous critical facilities, including the seismic criteria review of the Fast Flux Test Facility at Hanford, Washington. I have established ground motion criteria for proposed sites of nuclear facilities in the United States and Iran. I was responsible for the seismic review of the Diablo Canyon Nuclear Power Plant near San Luis Obispo, California, of the Pantex Plant in Texas, and of buried atomic waste tanks at Hanford, Washington, and at the Savannah River Plant in South Carolina.

My experience includes acquisition of seismic data in the field in both active and passive seismic investigations.

I am a member of the Seismological Society of America and the American Geophysical Union. I was the recipient of the Fulbright Travel Grant in 1967 and the University of California Chancellor's Patent Fund Award in 1972.

PUBLICATIONS

"A High Temperature Equation of State and the Resolution of Lower Mantle Mineralogy," with H. C. Helgeson, *Journal of Geophysical Research*, in press

"Shock Compression of $KFeS_2$ and the Question of Potassium in the Core," with T. J. Ahrens, *Journal of Geophysical Research*, Vol. 85, No. B12 (1980)

"An Earthquake Sequence in the Sierra Nevada-Great Basin Boundary Zone: Diamond Valley," with W. A. Peppin and J. D. VanWormer, *Bulletin of the Seismological Society of America*, Vol. 70, No. 5 (1980)

"Seismotectonic Regionalization of the Great Basin, and Comparison of Moment Rates Computed from Holocene Strain and Historic Seismicity," with R. W. Greensfelder and F. C. Kintzer, *Geological Society of America Bulletin*, Part II, Vol. 91, No. 9 (1980)

"Recent Seismicity Patterns near Mammoth Lakes, California," with W. A. Peppin, *Earthquake Notes*, Vol. 50, No. 4 (1980)

"Elasticity, Constitution, and Temperature of Earth's Lower Mantle," Ph.D. Thesis, University of California, Berkeley (1977)

"Diffracted ScS and the Shear Velocity at the Core Boundary," with B. A. Bolt and M. Niazi, *Geophysical Journal of the Royal Astronomical Society*, Vol. 19, No. 2 (1970)

"Earthquakes and the Registration of Earthquakes from January 1, 1968, to June 30, 1968," with M. Niazi and L. Dengler, *Bulletin of the Seismograph Stations*, Vol. 38, No. 1, University of California, Berkeley (1969)

TESTIMONY OF
GEOFFREY R. MARTIN, PhD
SOUTH CAROLINA ELECTRIC AND GAS COMPANY
BEFORE ATOMIC SAFETY LICENSING BOARD

My name is Dr. Geoffrey Martin. I am Vice President of Engineering at Ertec Western, Inc. located in Long Beach, California. A statement of my professional qualifications, credentials and work experience is submitted with this testimony.

The purpose of this testimony is to report the results of investigations on the potential for ground motion amplification at the USGS Strong Motion Accelerograph (SMA) location. This instrument is located on the ground surface near the north abutment of Dam C within the overall Frees Creek Dam System. Over the past three years, the USGS instrument has recorded several relatively intense, but very short duration earthquakes. Peak accelerations from some of the earthquakes exceeded 0.2 g. The durations of significant shaking for the earthquakes were less than 1.0 seconds; peak accelerations were associated with frequencies in excess of 20 Hz. The primary concern was whether or not these surface motions had amplified as they propagated from the underlying bedrock. If amplification occurred, peak accelerations within bedrock would be less than recorded at the instrument location.

To evaluate this concern, seismic response analyses were performed using a simplified one-dimensional model of the site. The model was defined to have stiffness characteristics similar to soil and rock conditions occurring at the

instrument location. Earthquake motions with high frequency, short duration characteristics were used as input at the base of the soil model. The ratio of input acceleration to output acceleration at the ground surface was used to quantify the potential for amplification of earthquake motions.

The results of this study clearly show that the site amplifies a rock motion. The amount of amplification varied from 1.4 to 2.9 depending on the characteristics of the input motion and the properties of the soils at the site. The results also show that site soils are capable of transmitting high frequency (20 to 50 Hz) earthquake motions. This capability provides a quantitative explanation for the intense, short duration earthquake records obtained by the USGS instrument.

Details about this study are provided in the following sections. These details include a summary of geotechnical conditions at the site, a description of the analytical method used to perform the study, a summary of input data for the analytical study, a presentation of results and, finally, a number of concluding remarks based on the results.

SITE CONDITIONS

The USGS Strong Motion Accelerograph (SMA-1, #603) is located on the north abutment of Dam C, next to the intake structure for the Fairfield Pump Storage Facility (Figure 1 and 2). The instrument is situated on a 4-foot by 4-foot by 1.0-foot concrete pad which rests on soil.

A soil boring was made at the site in February of 1980 (Law, 1980). The results of this boring program indicate that the soil profile comprises the following sequence:

- o 0 to 8 feet: A very stiff, red brown slightly clayey to medium sandy silt, described locally as residual soil,
- o 8 to 41 feet: a micaceous fine to medium sandy silt grading to a fine to coarse silty sand, described locally as a saprolite,
- o 41 to 56 feet: partially weathered rock which breaks-down during sampling to silty fine to coarse sand, and
- o Beyond 56 feet: relative intact rock (Adamellite) with slight to severe weathering.

Figure 3 presents an idealization of this soil profile. Other borings performed in conjunction with the Frees Creek dam investigation confirm this general sequence of soil types.

Engineering data for the site are limited to blowcounts obtained by performing Standard Penetration Tests (SPTs). The SPTs were conducted using a 140 pound hammer falling 30 inches; a 1.4 inch diameter sampler was employed. Ranges in blowcounts recorded during the SPTs are summarized below:

- o Residual Soils (0-8 feet): N = 20 to 24
- o Saprolites (8-40 feet): N = 10 to 40
- o Decomposed Rock (40 to 56 feet): N = 80 +

The underlying rock had rock quality designations (RQD) of 29 and 53. No water table was encountered at the site.

Although no other geotechnical data exist for the SMA site, appreciable data are available at the site of the V. C. Summer nuclear station. This station is located approximately 4200 feet southeast of the dam abutment. Soils at the nuclear station comprise the same general sequence of residual soils at the ground surface, underlain by saprolites and decomposed rock. By making adjustments for depth and soil stratum thickness changes between the abutment and power plant site, it was possible to extrapolate much of the geotechnical data from the nuclear station site to the abutment location. The extrapolated data included the results of crosshole seismic surveys, static strength tests, dynamic modulus characteristics and soil classification information (water contents, unit weights, etc).

DESCRIPTION OF SITE RESPONSE ANALYSES

The potential influence of surficial soils upon ground motions at the SMA site was examined by performing non-linear, one dimensional dynamic response analyses using an explicit finite difference computer program called DETRAN (Lam et al, 1978). The DETRAN program is a modified version of the program NONLI3 (Joyner, 1977). The main features of the program include:

1. The use of a composite elasto-plastic element commonly known as the Iwan model (Iwan, 1967) to simulate the non-linear and hysteretic behavior of the soil

2. A degradation model to approximate the progressive softening of certain soil types when subjected to cycles of high-amplitude loading, and
3. A transmitting boundary at the base of the soil model (Figure 4) to simulate the energy dissipation phenomenon that occurs where a rigid boundary does not exist.

As noted, the DETRAN program uses an Iwan model to represent the soil system. This model incorporates an array of elasto-plastic elements to simulate a given non-linear stress-strain curve. Each element of the array consists of a linear spring and Coulomb slider for which the stiffness and friction values can be adjusted to model virtually any mathematical or experimentally derived stress-strain relationship. For complicated arbitrary cyclic loading conditions, such as those of earthquakes, the Iwan model memorizes all relevant portions of the past loading history to achieve stable, non-drifting hysteresis loops under non-uniform cyclic loadings. In addition, a yielding or failure criterion is incorporated to simulate the combined non-linear and failure behavior of certain soil types. In this study the Iwan model was fit to available cyclic stiffness and static strength data from the V.C. Summer nuclear station site.

A transmitting boundary is incorporated in the DETRAN program to account for the existense of rock with finite stiffness located at the base of the soil column. With the

transmitting boundary, energy is allowed to propagate up and down as a function of the relative rigidity of the material above and below the input point. The transmitting boundary formulation satisfies the boundary conditions at the interface for a wave vertically incident from below.

A degradation model which can simulate the progressive reduction in stiffness of soft soils under earthquake loading is also included in DETRAN. However, for soils such as those at the SMA site the soil will not degrade in any significant amount under the low amplitude, short duration shaking.

In the DETRAN analysis, an acceleration time history is applied to the transmitting boundary interface. Acceleration modifications occur both at the interface and within the overlying soil profile. Parameters, such as the shaking intensity, soil stiffness, degradation characteristics and the ratio between the half space stiffness and that of the soil above the base will influence the amount of modification. The soil layer motions are generated using an explicit finite difference technique with step-by-step integration in the time domain. The surface motion then reflects the modifications of the input motion by the soil profile and the transmitting boundary.

INPUT DATA TO GROUND RESPONSE STUDIES

To perform the DETRAN analyses, it was necessary to model the stiffness characteristics of the soil profile. The analyses also required specification of earthquake

records which could be used as the input motions for the analyses. Procedures used to model soil conditions and select earthquake records are described in the following two subsections.

Earthquake Records

Artificial time histories were developed to simulate the earthquake ground motion expected from a small magnitude ($M < 4$) earthquake located at a hypocentral distance of about 3 km. Earthquakes this small and this close will generate ground motions of short duration (generally a few seconds or less) and relatively high frequency content (up to 50 Hz). Such motions were recorded on the north abutment of Dam C near the V.C. Summer nuclear station site during recent earthquakes (Brady and others, 1981; Mork and Brady, 1981).

To simulate high frequency, short duration motions, the time scales of existing time histories, both real and artificial, were compressed by dividing the time value for each point in the time history by an appropriate scale factor. These time histories with the time-axis scaling factor are listed below:

<u>Record</u>	<u>Time Scale Factor</u>
Station 8 from Oroville aftershock on 9/27/75 @ 22:34 GMT. Component N90W.	2
Caltech Artificial Accelerogram C1 (Jennings and others, 1968)	5
Caltech Artificial Accelerogram D1 (Jennings and others, 1968)	5

Scaling in this manner shifts the frequency content to higher frequencies such that the Fourier Amplitude Spectrum (FAS) of a scaled record is the FAS of the original record with the frequency axis multiplied by, and amplitude axis divided by, the scale factor.

After the time scaling was performed, the amplitudes (accelerations) of each compressed time history were scaled to 0.15g, the peak ground acceleration to which the rock-site SSE design spectra were normalized. The resulting motions were used as the rock outcrop motions in the one-dimensional DETRAN analyses.

Soil Properties

The soil properties required for the site response analyses included stress-strain curves for cyclic loading, unit weights of soil layers and relative stiffness, or shear modulus, values. These data were derived using data available in the FSAR for the V.C. Summer Nuclear Station and data from the soil boring at the SMA site.

The first step in the assignment of soil properties involved discretizing the idealized, one dimensional soil profile, shown in Figure 3, into a number of sub-layers. These sublayers were defined according to soil type and material properties such as unit weight and undrained shear strength. The layer thicknesses in the model were chosen considering the earthquake frequency content so that artificial filtering of input motions would not occur and numerical stability would be maintained.

Once the sublayers were established, a generalized stress-strain curve was assigned to each layer. These stress-strain curves were obtained by defining a generalized strain-dependent modulus ratio (G/G_{\max}) curve. A single modulus ratio curve was used to represent the residual soils, saprolites and decomposed rock. This G/G_{\max} curve was obtained from the results of resonant column and cyclic triaxial tests presented in the PSAR. The curve is similar to that presented in the literature by Seed and Idriss (1971). The difference in strain-dependent modulus ratios was insufficient to justify use of different modulus ratio curves for each general soil type (residual, saprolites and decomposed rock). The modulus ratio curve was subsequently converted to a strain dependent shear stress-maximum shear modulus curve (τ/G_{\max}) by multiplying the G/G_{\max} value by the shear strain (γ). The resulting curve is shown in Figure 5.

The stress-strain curve for each layer was obtained by multiplying the τ/G_{\max} curve by the G_{\max} value for the layer. Three G_{\max} profiles were defined for this computation: an upper bound G_{\max} profile, a lower bound profile and a best-estimate profile. These profiles are shown in Figure 4. The profiles were generated from laboratory resonant column data for the V.C. Summer Nuclear Station, shear wave velocity measurements obtained at the V.C. Summer Nuclear Station using crosshole testing methods and empirical relationships based on blowcounts from the SPT

data at the SMA site. These profiles were confirmed using two empirical relationships. For clays it was assumed that the best estimate profile was defined by

$$G_{\max} = 1000K_2 \bar{\sigma}_0^{0.5} \quad (1)$$

where $\bar{\sigma}_0$ is the mean effective pressure in psf and K_2 is a constant based on the soil type. The best-estimate K_2 values from the FSAR are 70 and 250 for the saprolite and decomposed rock, respectively.

The resulting stress-strain curves for each layer were checked to assure that the yield stress at large strains was compatible with a rate adjusted shear strength for the layer. The static shear strengths were obtained using frictional angles of 36° and 38° for the saprolite and decomposed rock, respectively. Rate adjustment factors were consistent with published data.

A shear modulus was also assigned to the underlying rock layer to characterize the stiffness of the halfspace. This modulus was determined from crosshole measurements performed in the competent rock at the V.C. Summer Nuclear station site.

RESULTS OF SITE AMPLIFICATION STUDY

Different combinations of soil stiffness and input rock motions were utilized in the site response analyses. For all cases the rock outcrop acceleration assigned at the base of the soil column (56 feet) was scaled to have peak acceleration of 0.15g. Response from each computer analyses was

obtained at the ground surface. A summary of the results from all the case studies is presented in Table 1. Both key input and output (surface motion) parameters are given in this table. A discussion of the results is presented in the following paragraphs.

Site Amplification

On the basis of the above results, it can be concluded that local soil conditions near the SMA site will amplify low to medium levels of rock motion. The extent of amplification is primarily dependent upon two factors: the stiffness of the soil at the site and the characteristics of the rock motion. In this study, amplification is defined as the ratio between the Maximum Surface Acceleration and the Maximum Acceleration of the Input Rock Motion. As shown in Table 1, all cases have amplification ratios greater than 1.0. Values of the amplification ratio range from 1.4 to 2.9.

Despite the limited amount of soil property information at the SMA site, results of this study strongly indicate that the site soils will amplify the maximum acceleration values of the input rock motion. It is noted that data available in the literature also suggest the same amplification for shallow soil sites subjected to low to medium levels of shaking. (Seed et. al 1975).

Effects of Soil Stiffness on Surface Motion

In order to further study the motion amplification characteristics at the SMA site, Fourier spectra of the

input and output motions were examined and plotted. These spectra are shown in Appendix A. By decomposing the irregular earthquake motion into a series of harmonic motions with different frequencies, the distribution of the amplitudes at various frequencies is shown. In addition, by comparing the Fourier spectra of the surface and input motions, the motion transmissibility of the site soils with respect to different frequencies is also obtained.

Values of transmissibility are plotted in Appendix A as transfer functions. Since the most amplification will occur near the natural frequencies of the site soils, these plots of transfer function indicate the natural frequencies of the site. As shown in Appendix A, the first mode natural frequency of the site ranges from 3 to 5 Hz depending upon the assigned modulus profile. The second mode natural frequency of the site is between 10 to 15 Hz. It is also worthwhile noting that although there is no peak in the transfer function, above 20 Hz, the site can still transmit considerable motion in the high frequency region. This result confirms that a surface motion recorded at the SMA site can have a significant amount of high frequency contribution.

Effect of Input Motion on Surface Response

In addition to site soil stiffness, one of the most important factors affecting the surface motion is the characteristics of the rock motion. Since there is no recording available from the rock outcrop near the site,

artificial motions were generated to represent the anticipated rock motion at the site. A detailed description of the approach utilized in selecting these records, as well as motion selected, was presented previously. As shown in Table 1 and Appendix A, the maximum surface acceleration and frequency content of the surface motion can be somewhat different depending on the characteristics of the artificial motion.

CONCLUDING REMARKS

On the basis of the results presented in the preceding section, the following concluding remarks can be made.

1. Rock motions at the SMA site will be amplified as they propagate through the soil column. The surface motion at the first and second fundamental modes of vibration for the site will be from 1.4 to 2.9 times the earthquake motion at the rock soil interface.
2. The variation in amplification can be attributed to changes in the stiffness of the soils and changes in the characteristics of the input motion. The smallest amplification ratio (1.4) occurs when the lower bound soil conditions are used in the analyses. This lower bound modulus profile is roughly 70 percent of the best-estimate profile.
3. Soil conditions at the site are such that high frequency components of motion can be carried to the ground surface. Whereas the amplification

ratio at high frequencies is approximately 1.0, the composite effects of these frequencies can produce single high amplitude acceleration values characterized by a high frequency.

REFERENCES

- Brady, A.G., Mork, P.N., and Fletcher, J.P., 1981, "Processed accelerograms from Monticello Dam, Jenkinsville, South Carolina, 27 August 1978, and two later shocks," USGS Open File Report 81-0448, March.
- Iwan, W.D., "On a Class of Models for the Yielding Behavior of Continuous and Composite Systems," Journal of Applied Mechanics, Vol. 34, No. 3, Sept., pp. 612-617.
- Jennings, P.C., Housner, G.W., and Tsai, N.C., 1968, "Simulated earthquake motions," Earthquake Engineering Research Laboratory, Caltech.
- Joyner, W.B. (1977) "NONLI3 -- a Fortran Program for Calculating Nonlinear Seismic Ground Response", U.S. Geological Survey Open File Report, 77-671.
- Lam, I., Tsai, C.F., and Martin, G.R. (1978), "Determination of Site Dependent Spectra Using Nonlinear Analysis," Second International Conference on Microzonation, San Francisco, Calif., Nov.
- Law Engineering Testing Company (1980), letter report on "core boring next to seismograph, intake structure area, Fairfield pumped storage facility, Jenkinsville, South Carolina, LETCo job No. CO-509", submitted to South Carolina Electric and Gas Co., February.
- Mork, P.N. and Brady, A.G., 1981, "Processed accelerogram data from Monticello Dam, Jenkinsville, South Carolina, 16 October 1979, 0706 UTC," USGS Open File Report 82-1214, October.
- Seed, H.B., and Idriss, I.M. (1970) "Soil Moduli and damping factors for Dynamic Response Analysis", EERC Report 70-10, University of California, Berkeley.
- Seed, H.B., Murarka, R., Lysmer, J. and Idriss, I.M. (1975) "Relationships between Maximum Acceleration, Maximum Velocity, Distance from Source, Local Site Conditions for Moderately Strong Earthquakes", EERC report 75-17, University of California, Berkeley, Calif.

TABLE 1

SUMMARY OF DETRAN SITE RESPONSE ANALYSES

<u>Earthquake Used as Input</u>	<u>Maximum Acceleration Input (g)</u>	<u>Soil* Stiffness Profile (G_{max})</u>	<u>Maximum Surface Acceleration (g)</u>	<u>Amplification Ratio ($\frac{a_{\text{max Surface}}}{a_{\text{max Input}}}$)</u>
Oroville	0.15	Upper bound	0.30	2.0
aftershock	0.15	Best estimate	0.23	1.5
Station 8 N90W	0.15	Lower bound	0.26	1.7
Caltech C1	0.15	Upper bound	0.28	1.9
(Jennings	0.15	Best estimate	0.34	2.3
et al, 1981)	0.15	Lower bound	0.29	1.9
Caltech D1	0.15	Upper bound	0.20	1.4
(Jennings	0.15	Best estimate	0.20	1.4
et al. (1968)	0.15	Lower bound	0.43	2.9

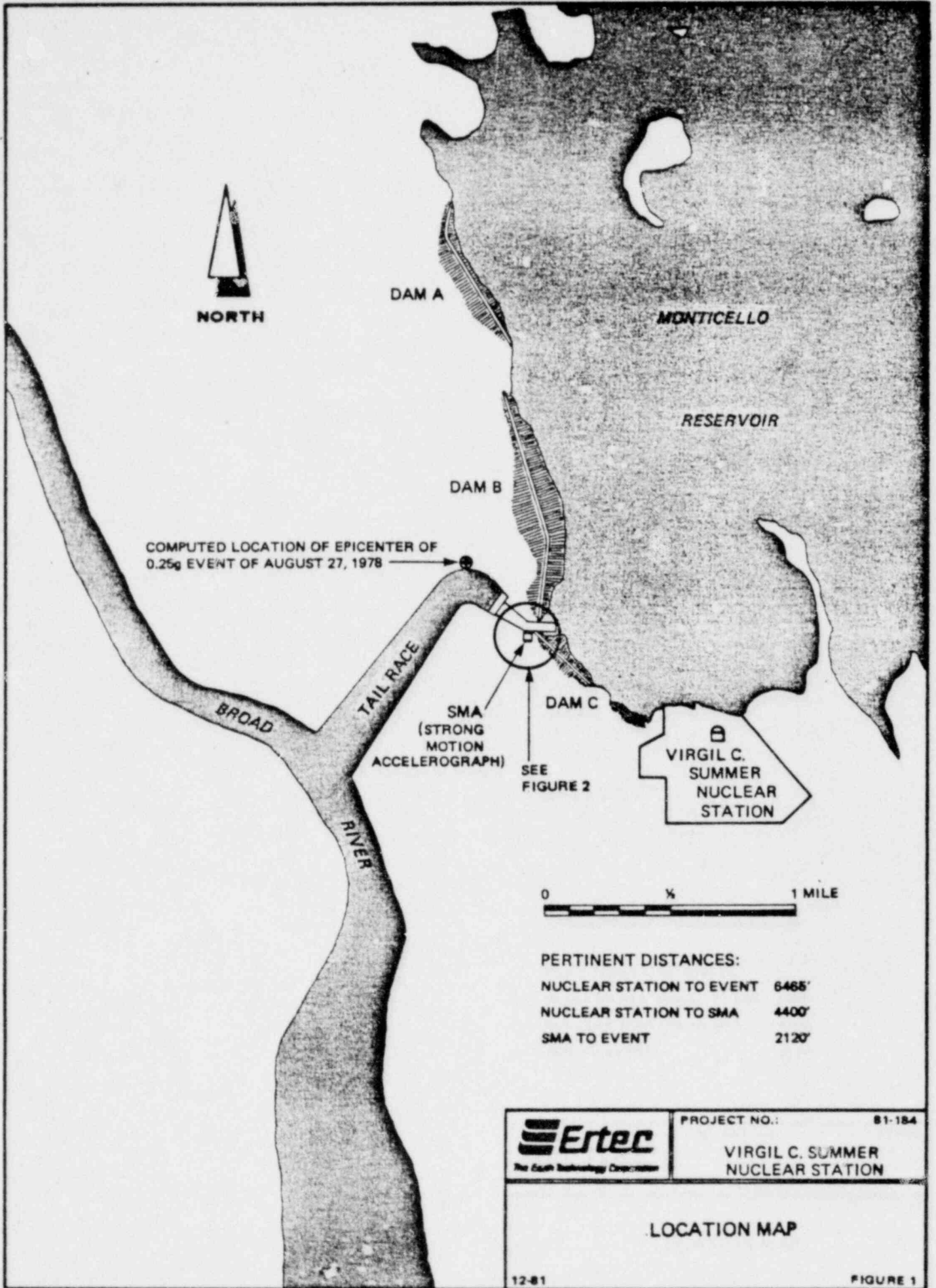
*Values are shown in Figure 4

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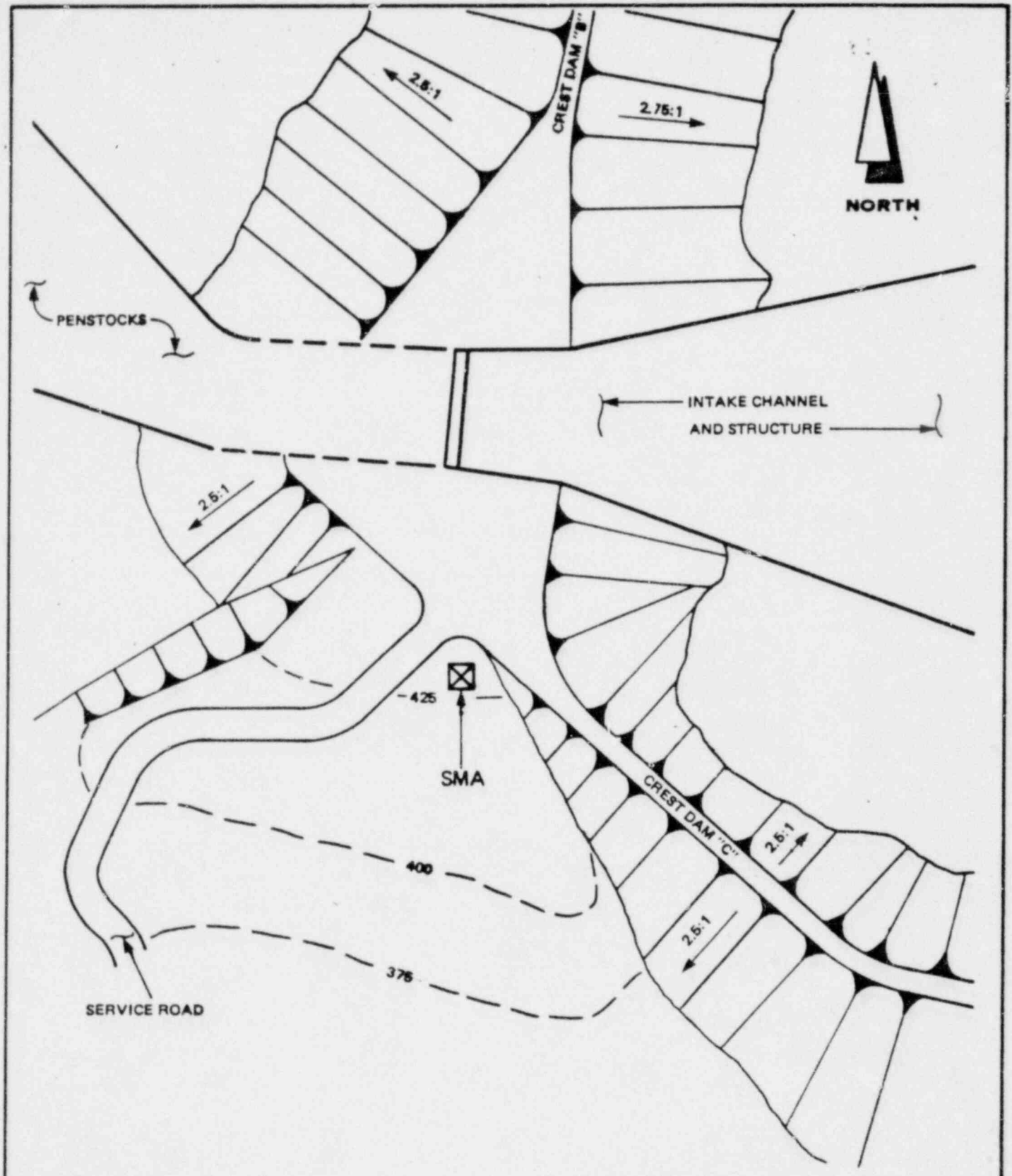
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LOCATION MAP	
12-81	FIGURE 1

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INSET FIGURE 1-
SMA LOCATION

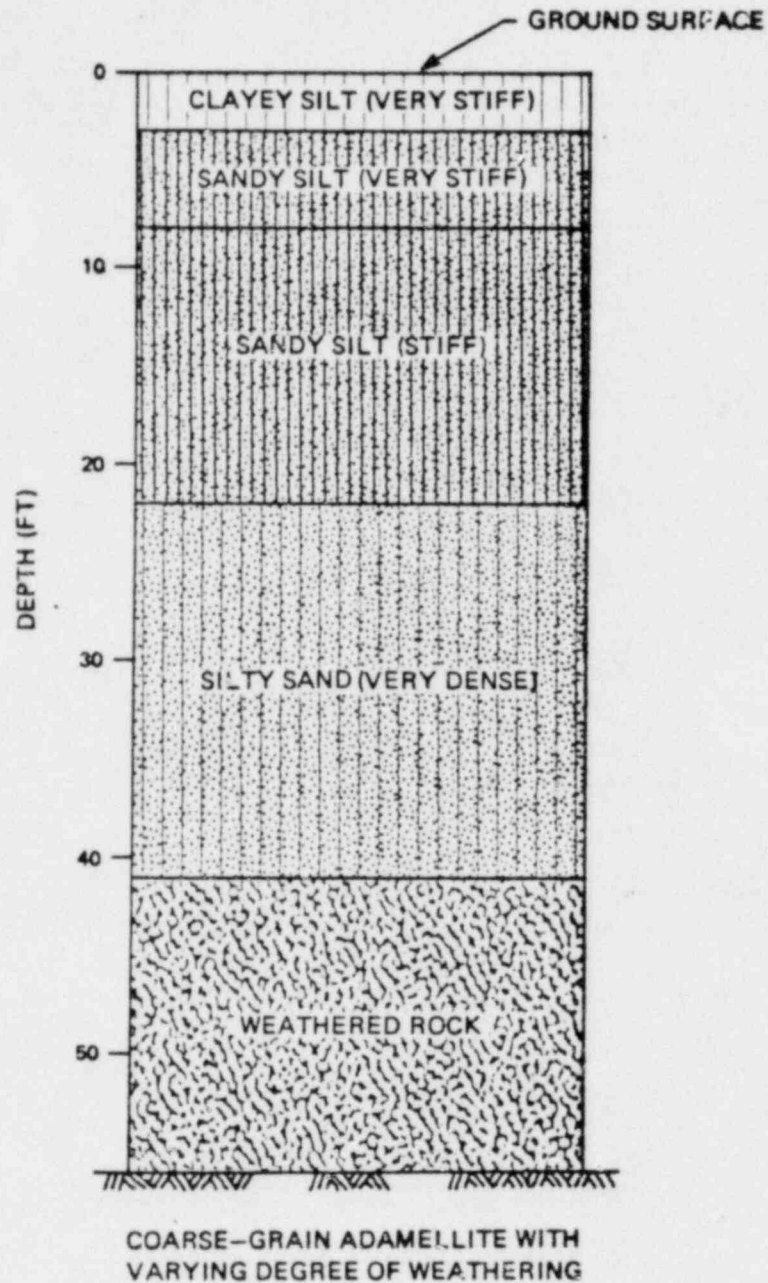
12-81 FIGURE 2

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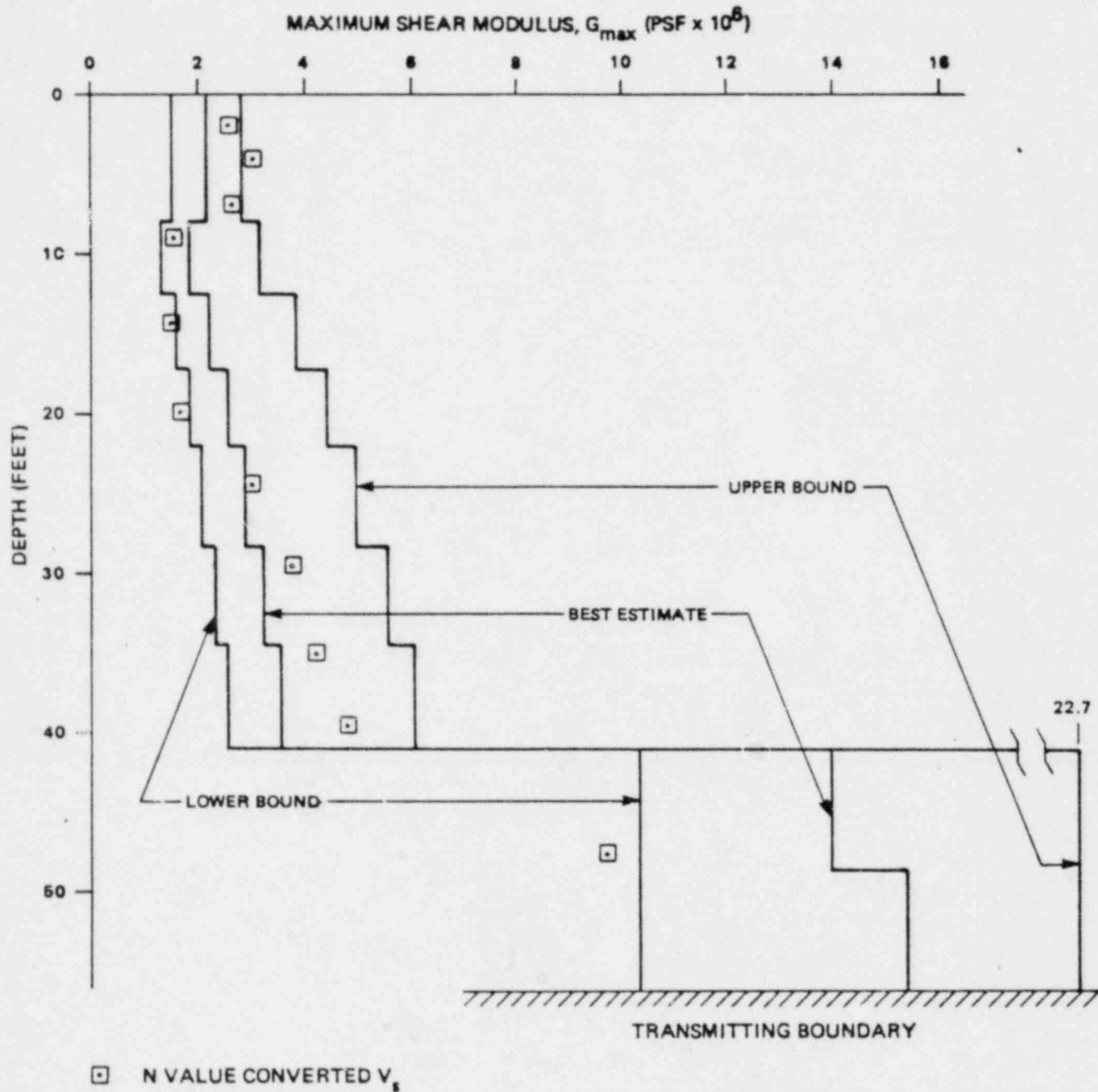
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IDEALIZED SOIL PROFILE USED IN THE DETRAN ANALYSES	
12-81	FIGURE 3

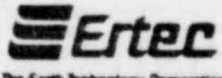
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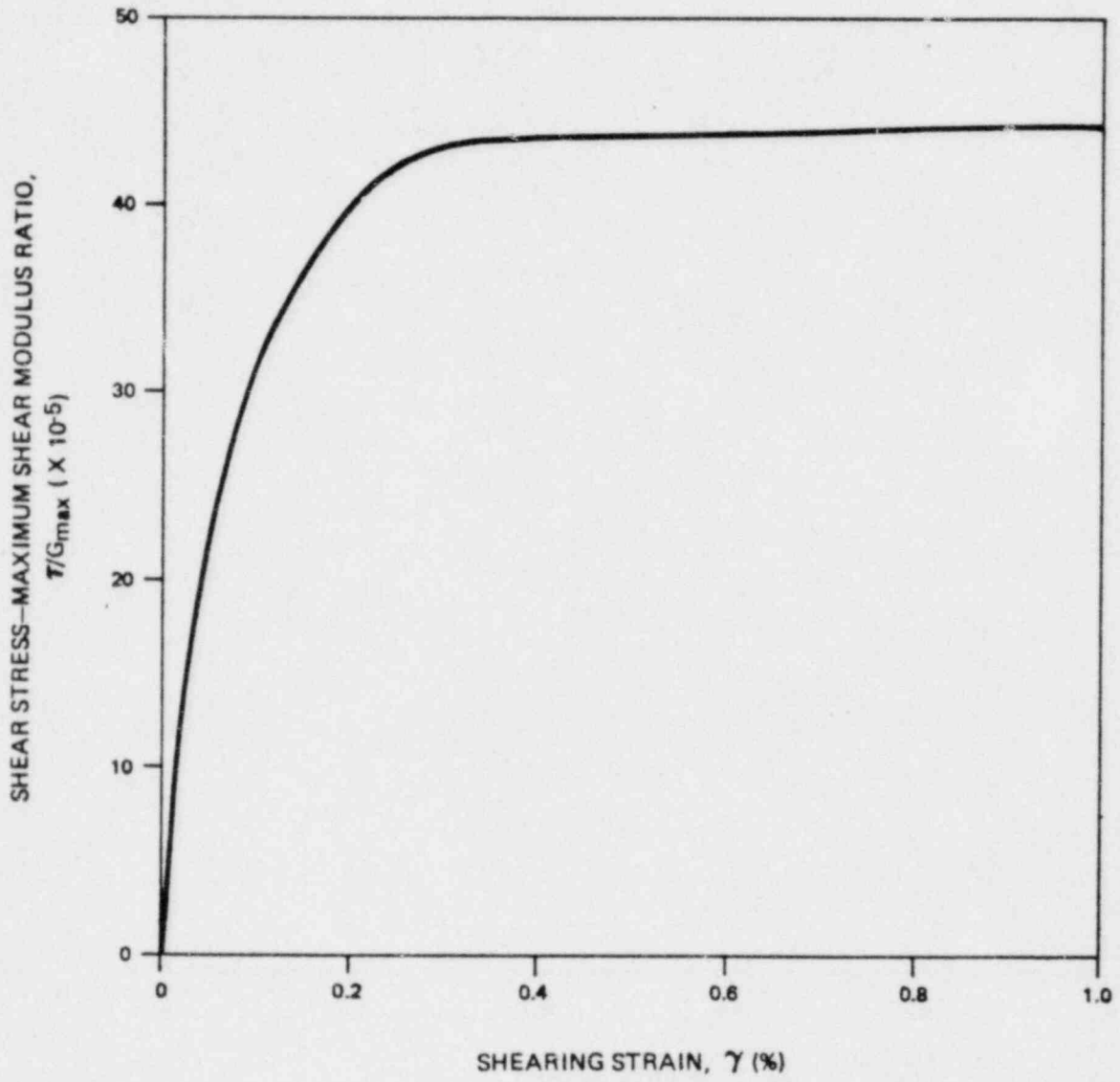
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SHEAR MODULE PROFILES USED IN THE DETRAN ANALYSES	
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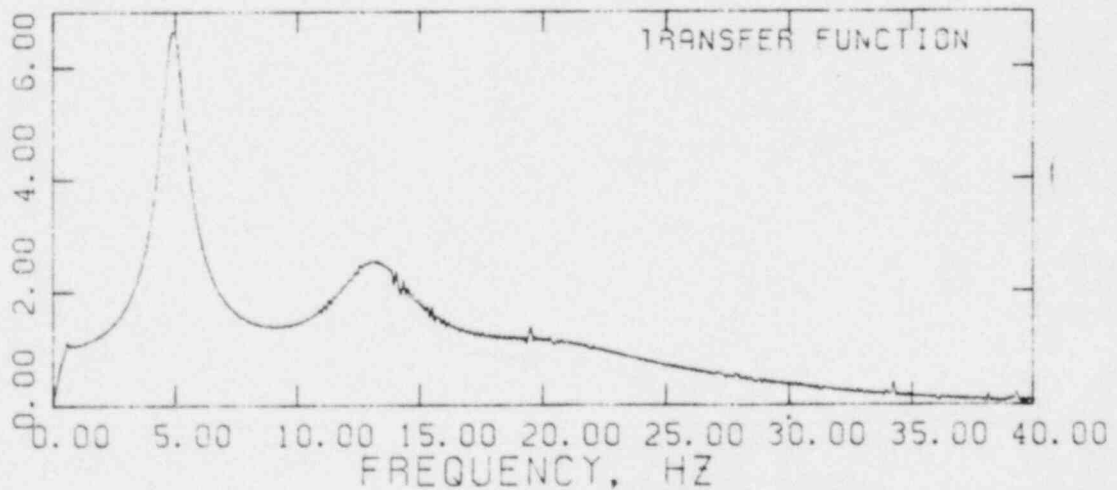
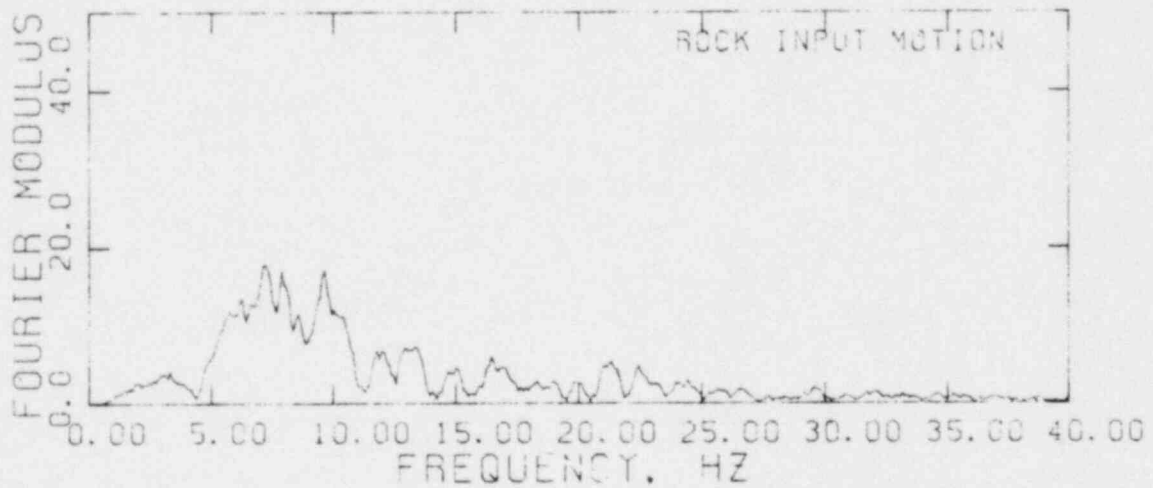
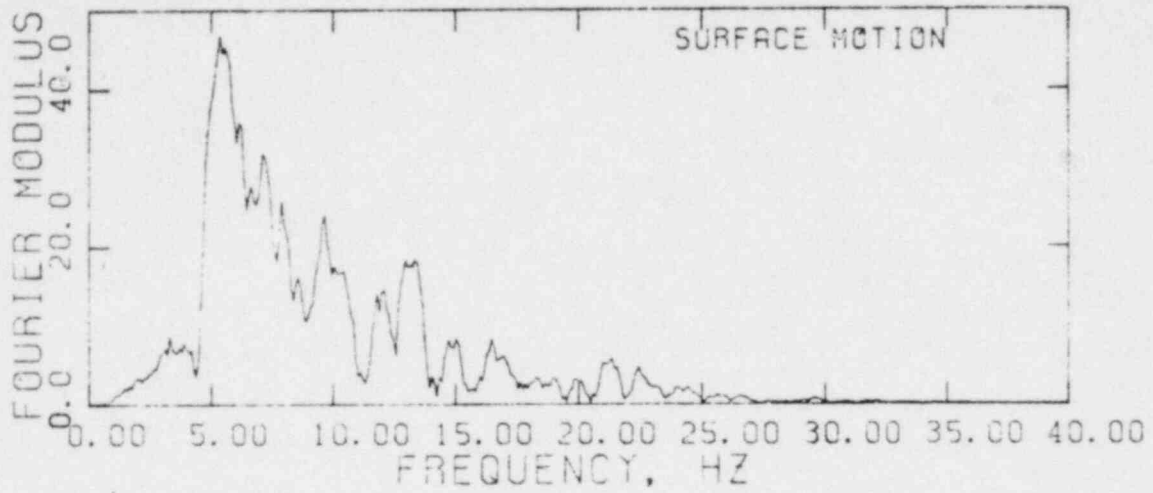
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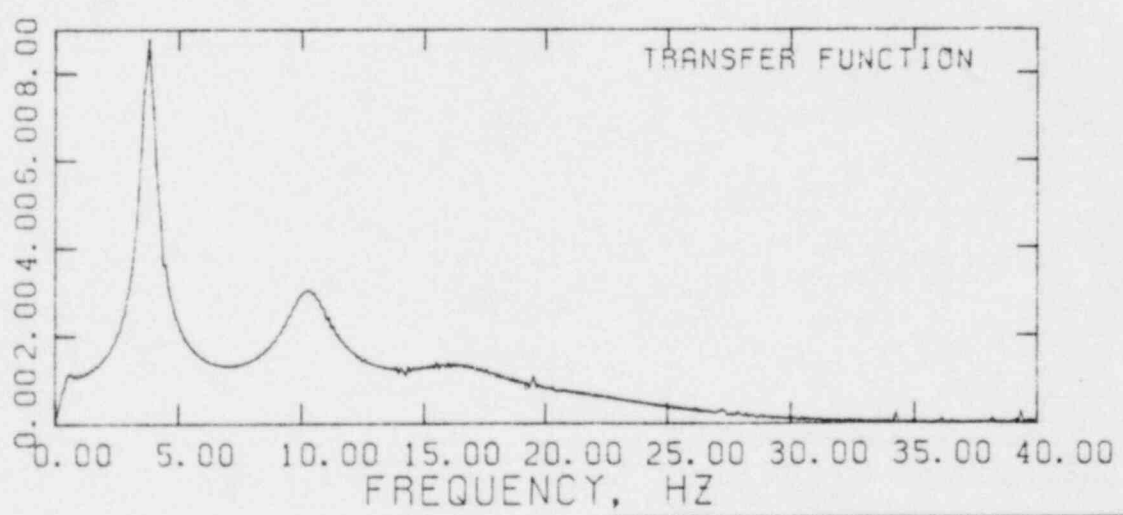
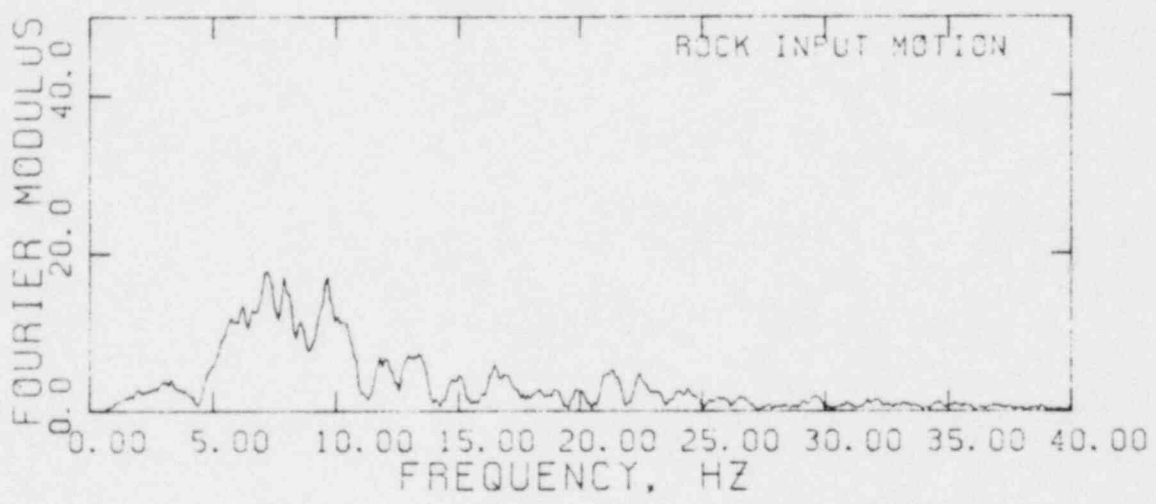
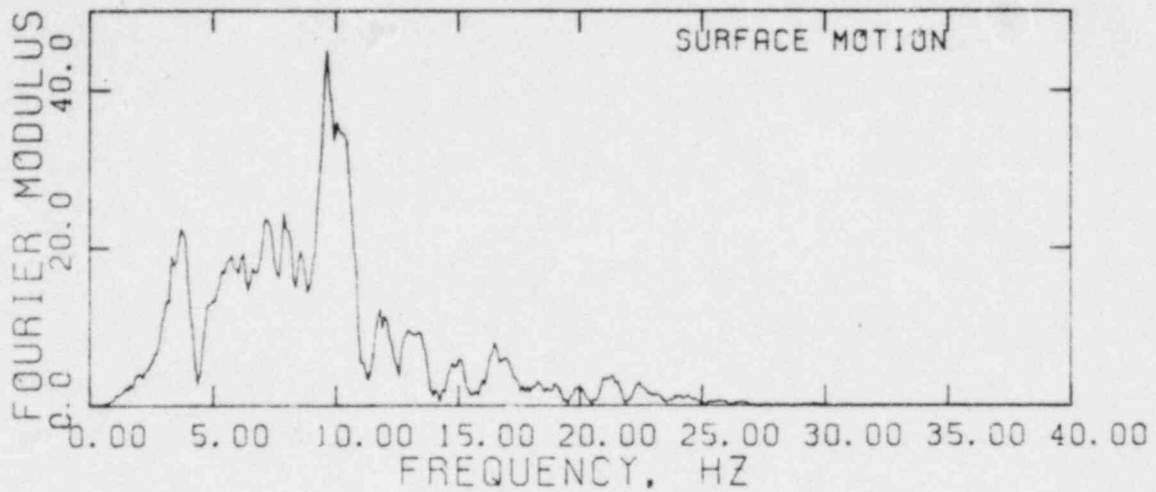
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GENERALIZED SOIL
STRESS-STRAIN CURVE

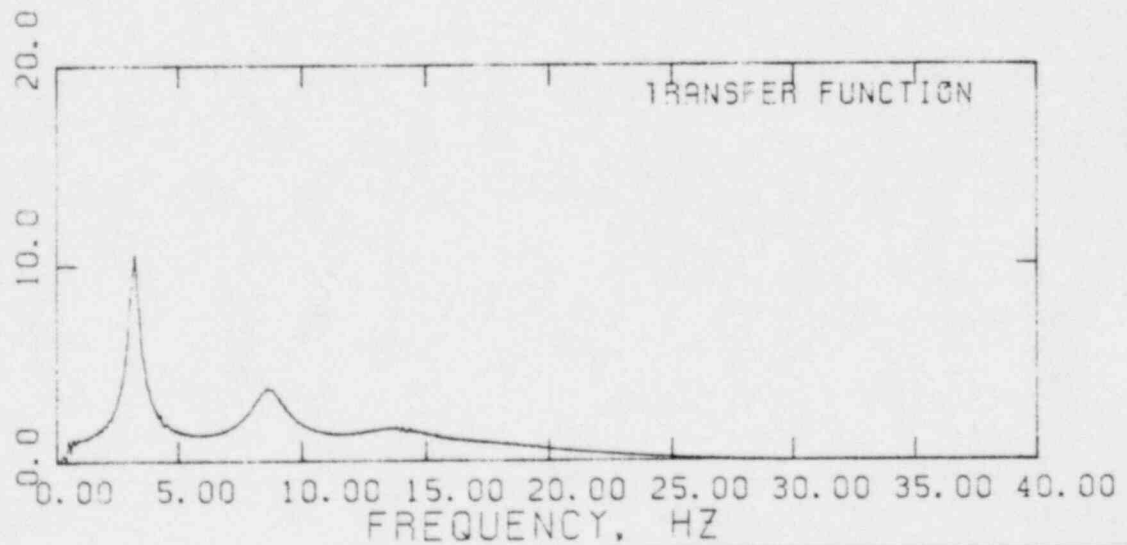
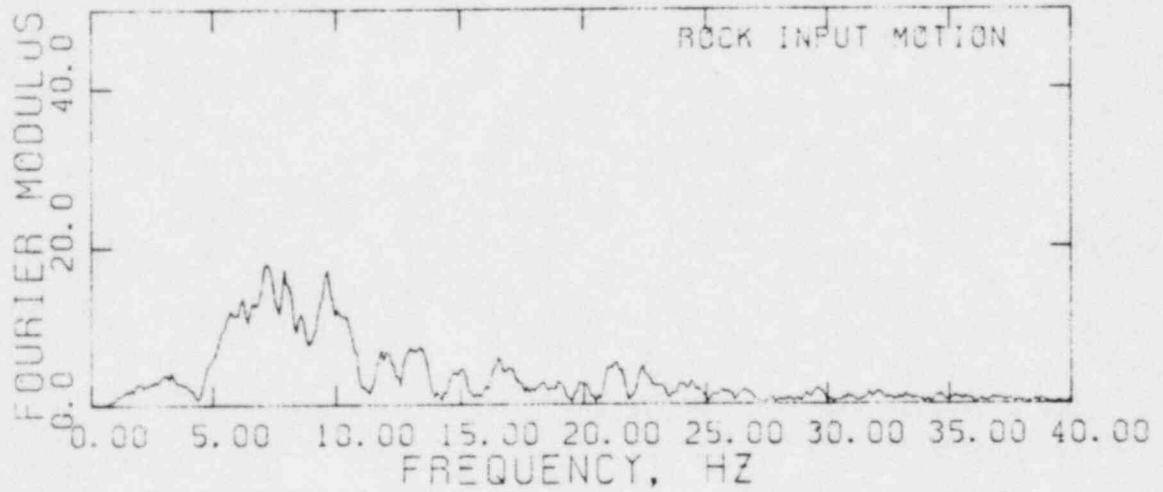
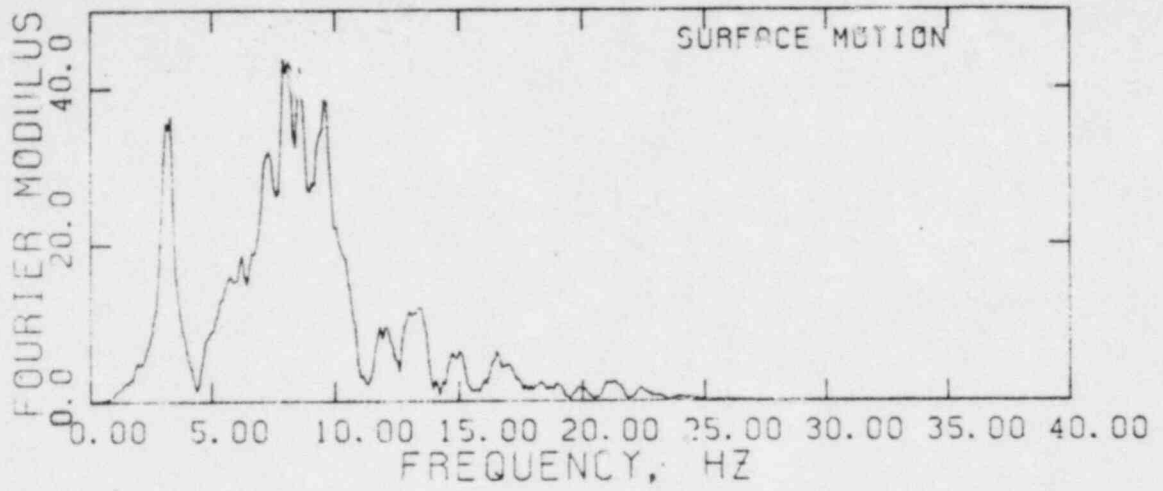
APPENDIX A
FOURIER SPECTRA AND
TRANSFER FUNCTIONS



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FOURIER SPECTRA AND TRANSFER FUNCTION PLOT EARTHQUAKE OROVILLE NO. 8 SOIL STIFFNESS UPPER BOUND	
12-81	FIGURE A-1



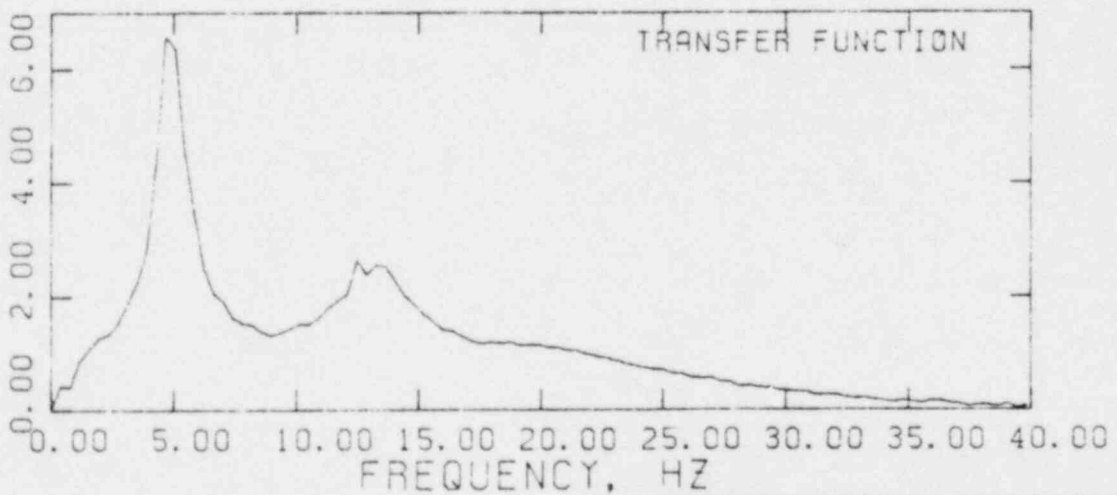
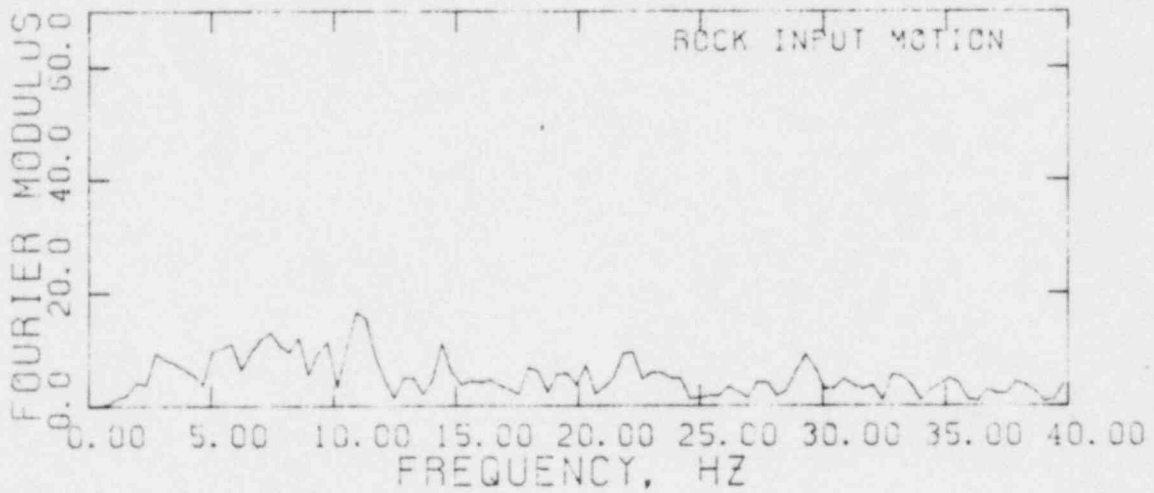
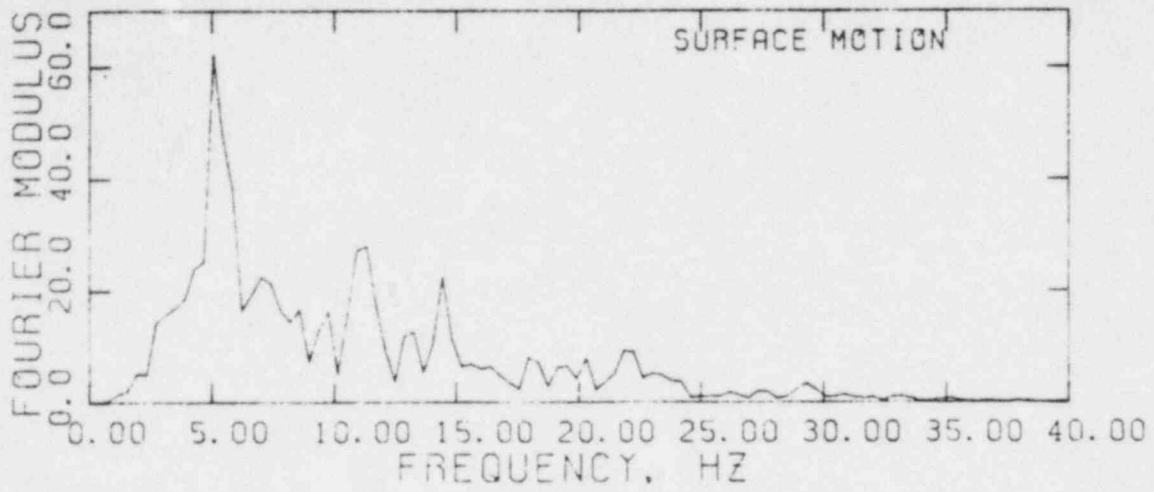
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	VIRGIL C. SUMMER NUCLEAR STATION
FOURIER SPECTRA AND TRANSFER FUNCTION PLOT EARTHQUAKE OROVILLE NO. 8 SOIL STIFFNESS BEST ESTIMATE	
17-81	FIGURE A-2



PROJECT NO.: 81-184

VIRGIL C. SUMMER
NUCLEAR STATION

FOURIER SPECTRA AND
TRANSFER FUNCTION PLOT
EARTHQUAKE OROVILLE NO. 8
SOIL STIFFNESS LOWER BOUND



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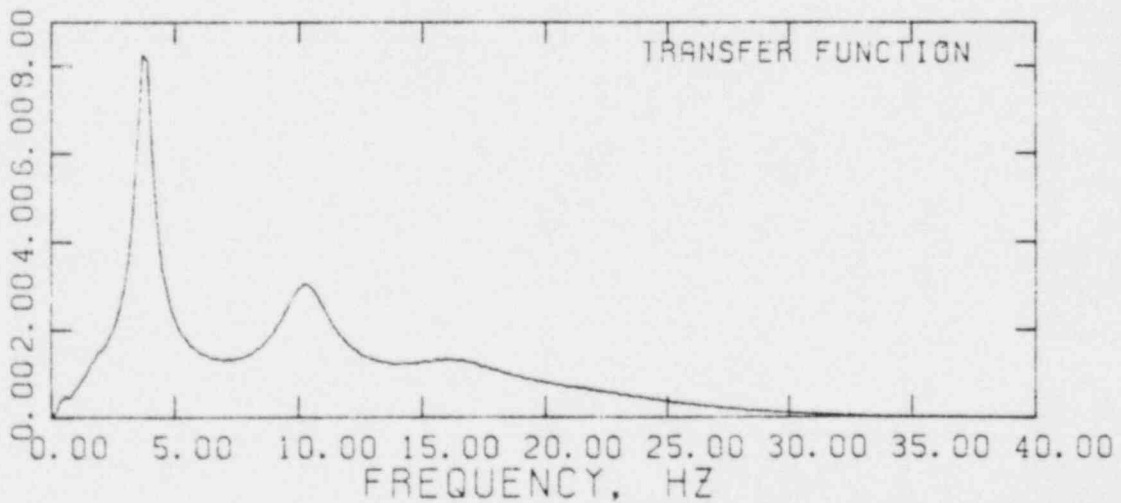
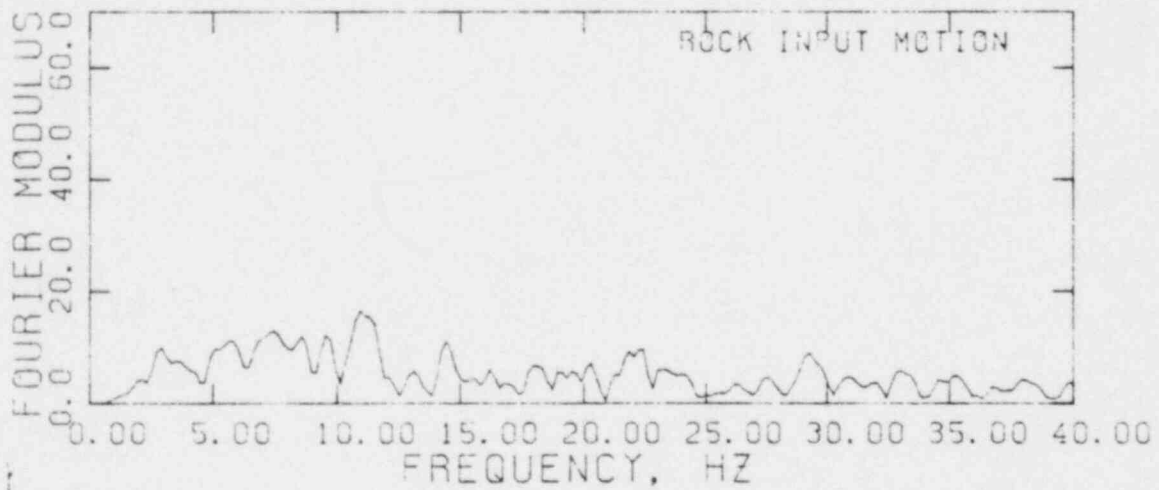
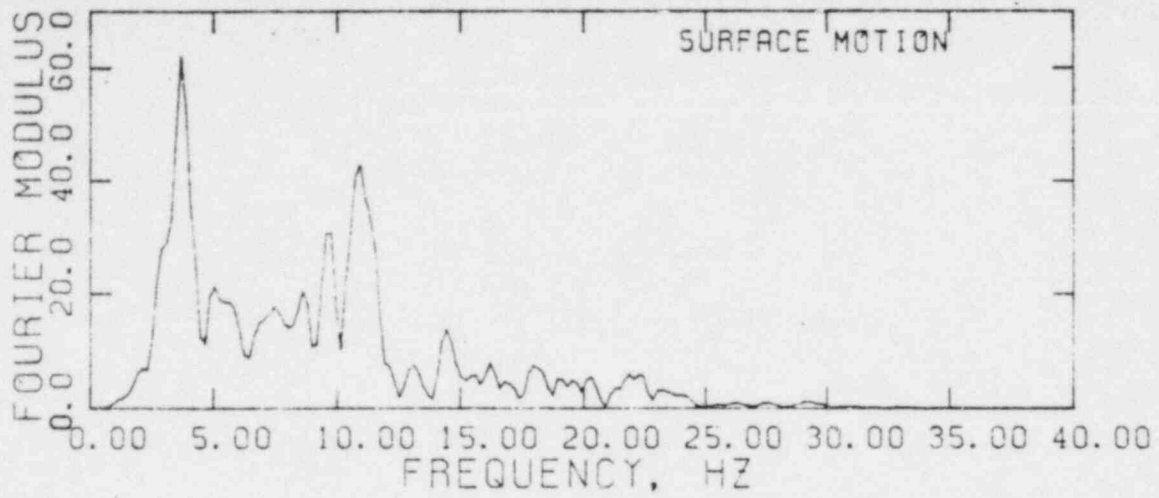
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NUCLEAR STATION

FOURIER SPECTRA AND
TRANSFER FUNCTION PLOT
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SOIL STIFFNESS UPPER BOUND

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FIGURE A-4

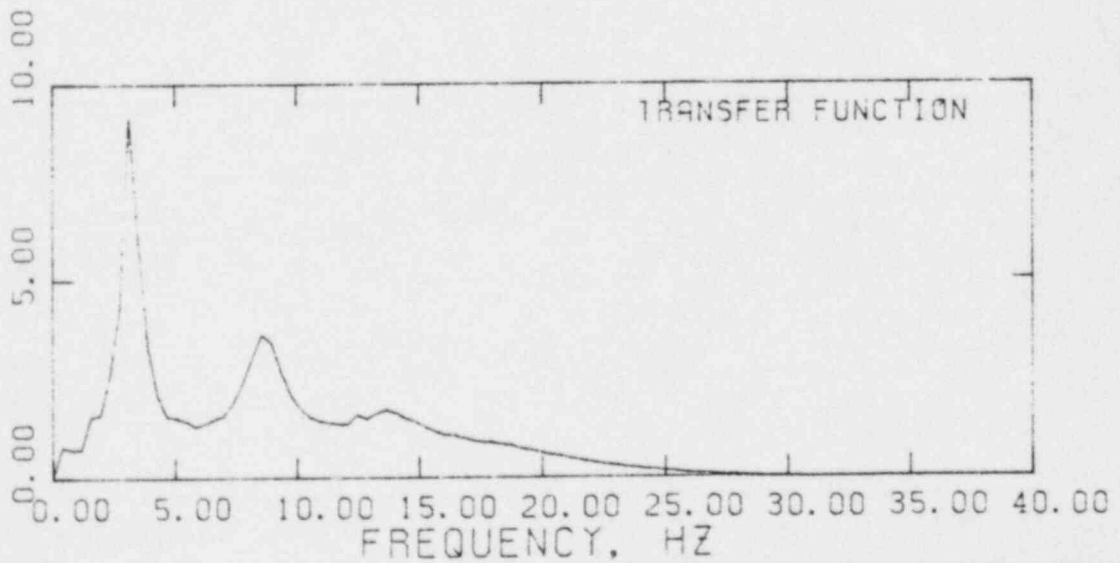
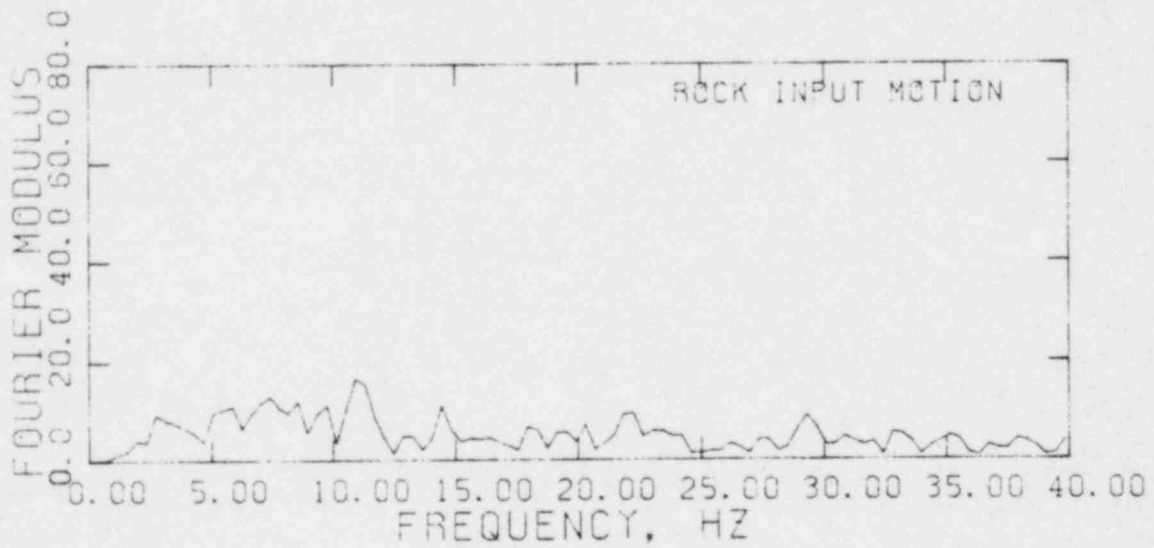
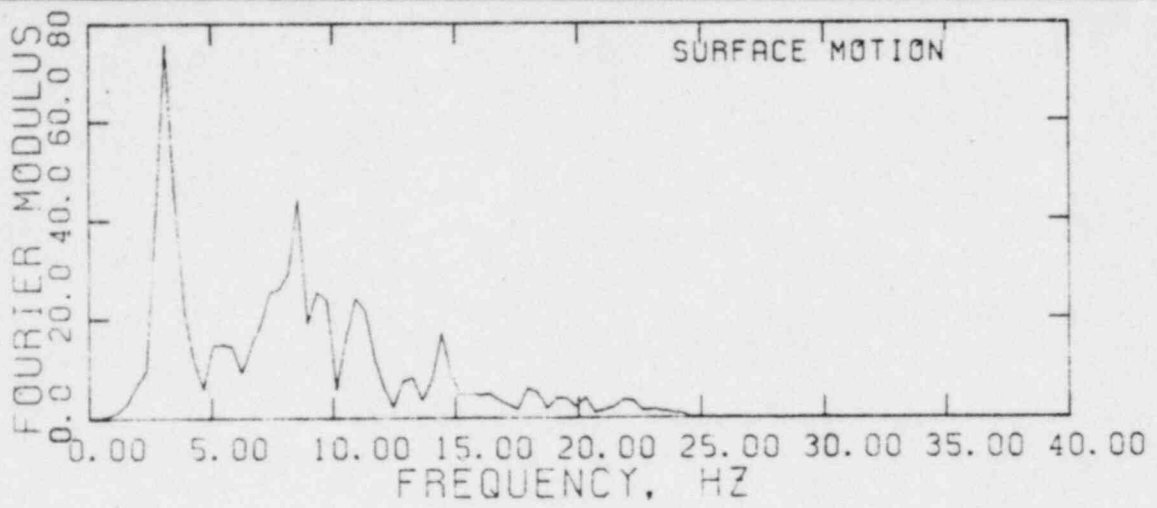


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FOURIER SPECTRA AND
TRANSFER FUNCTION PLOT
EARTHQUAKE CAL TECH C1
SOIL STIFFNESS BEST ESTIMATE

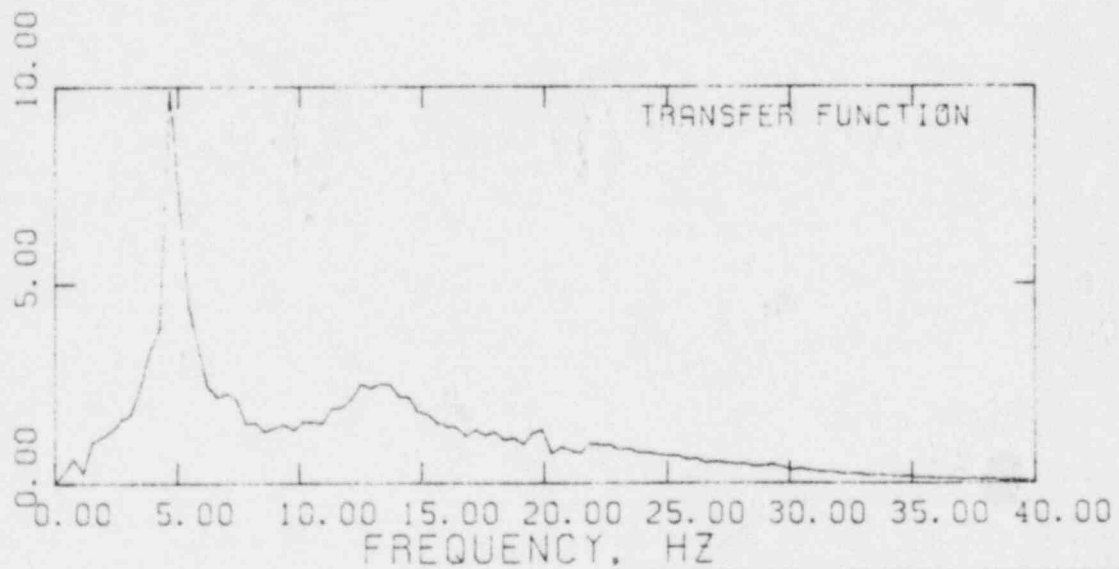
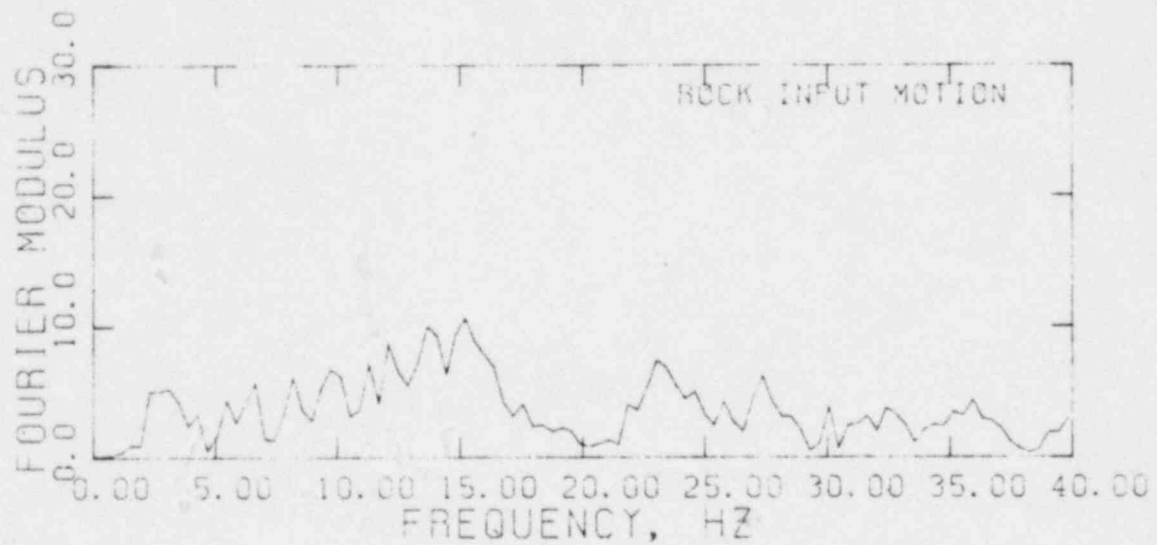
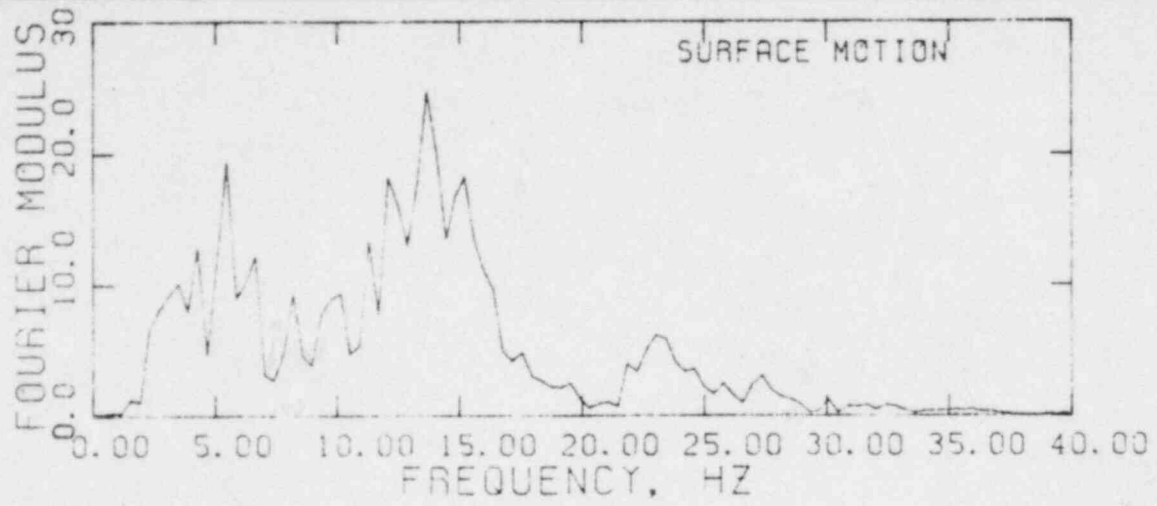


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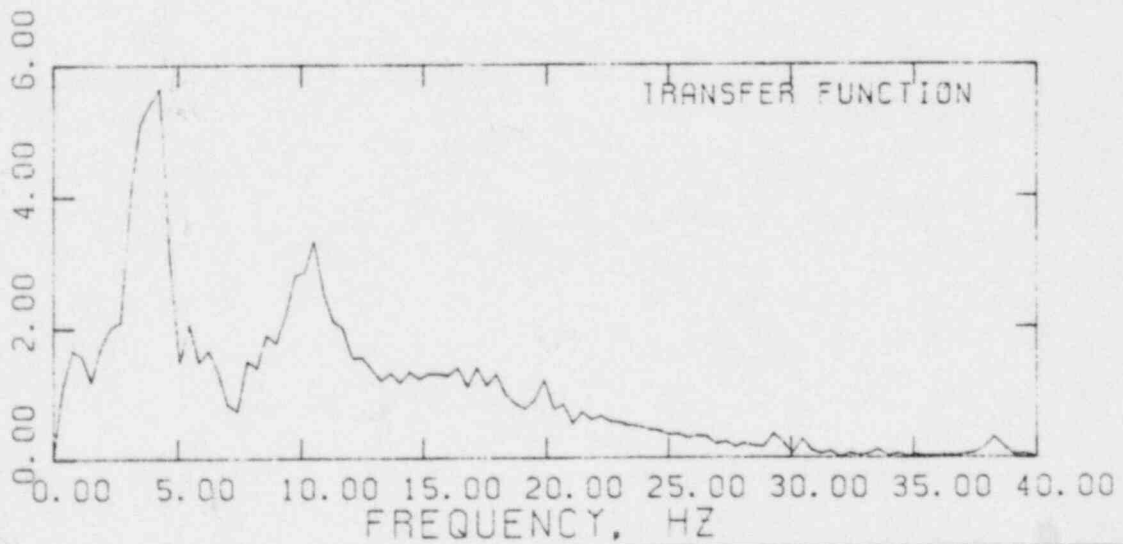
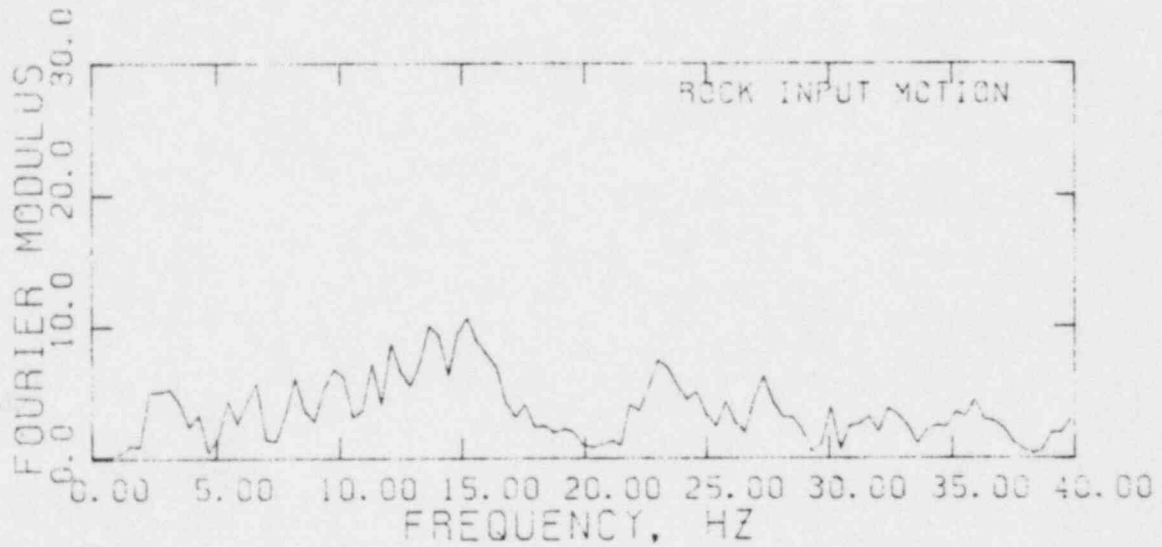
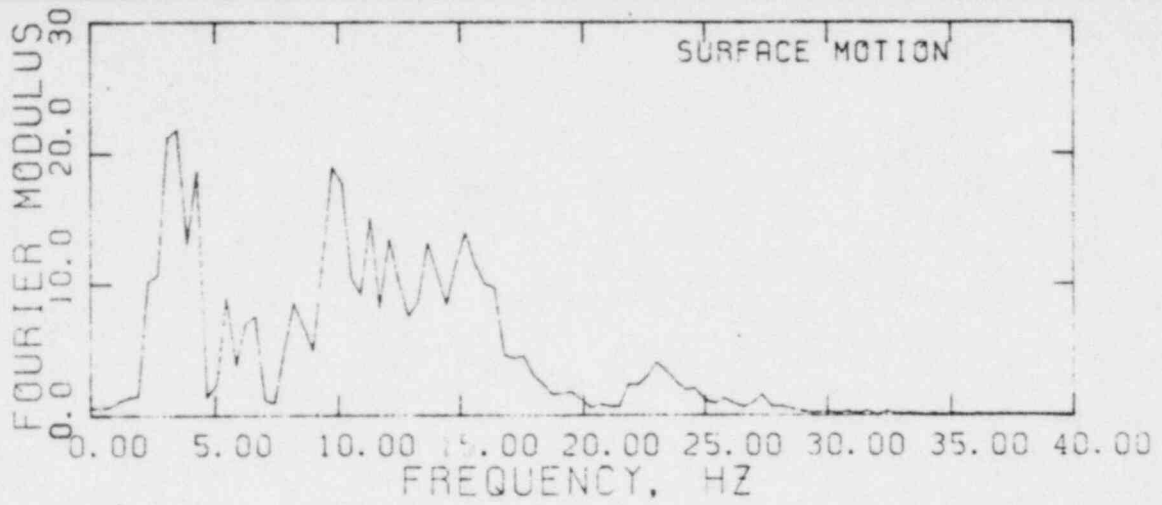
FOURIER SPECTRA AND
TRANSFER FUNCTION PLOT
EARTHQUAKE CAL TECH C1
SOIL STIFFNESS LOWER BOUND



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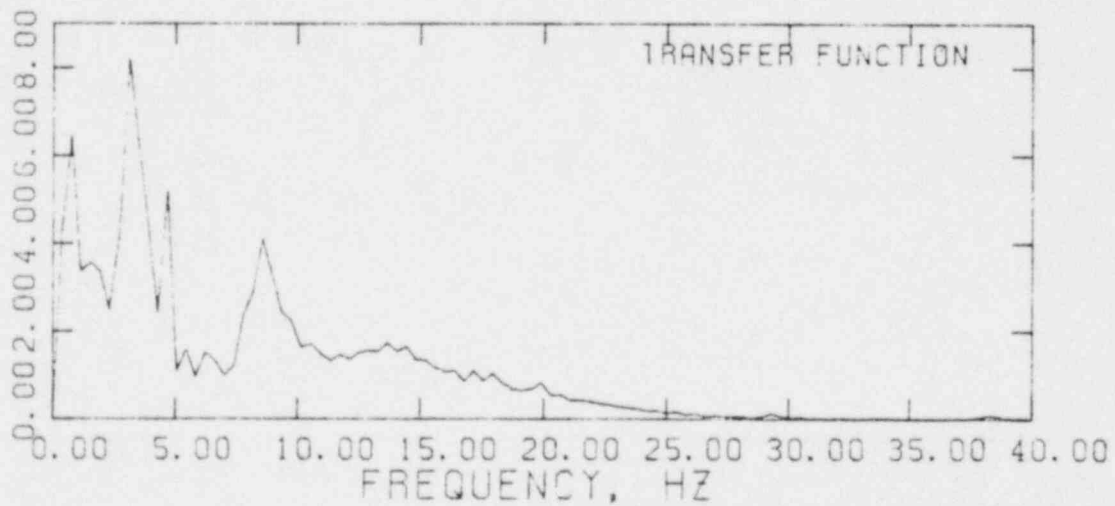
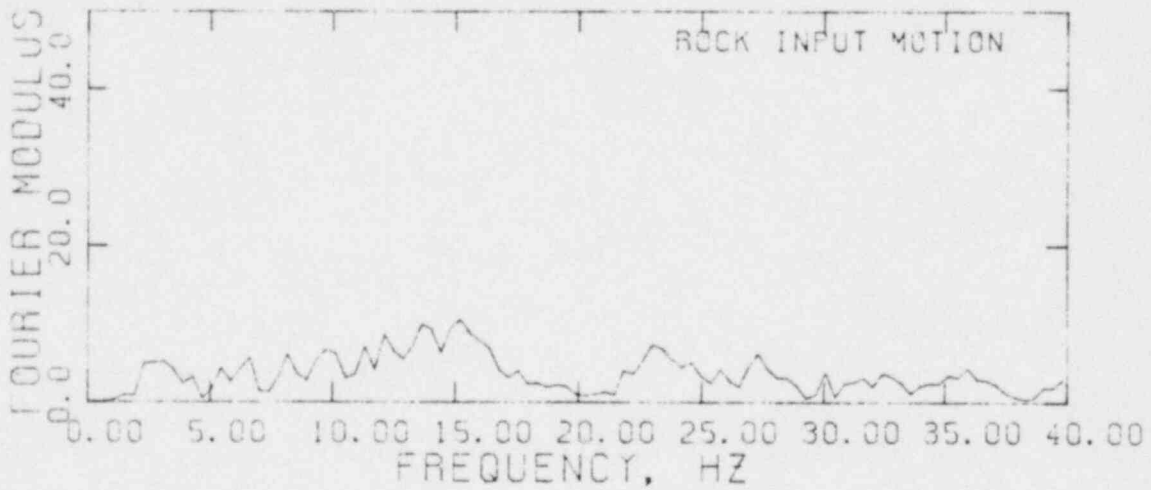
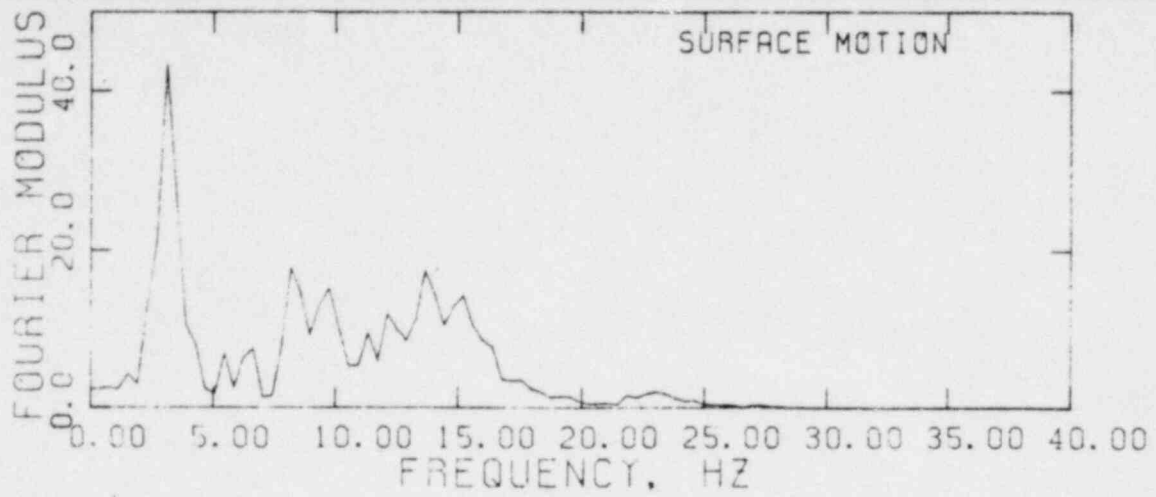
VIRGIL C. SUMMER
NUCLEAR STATION

FOURIER SPECTRA AND
TRANSFER FUNCTION PLOT
EARTHQUAKE CAL TECH D1
SOIL STIFFNESS UPPER BOUND



PROJECT NO.: 81-184
 VIRGIL C. SUMMER
 NUCLEAR STATION

FOURIER SPECTRA AND
 TRANSFER FUNCTION PLOT
 EARTHQUAKE CAL TECH D1
 SOIL STIFFNESS BEST ESTIMATE



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VIRGIL C. SUMMER
NUCLEAR STATION

FOURIER SPECTRA AND
TRANSFER FUNCTION PLOT
EARTHQUAKE CAL TECH D1
SOIL STIFFNESS LOWER BOUND

PROFESSIONAL QUALIFICATIONS

GEOFFREY R. MARTIN

My name is Geoffrey Martin. I am Vice President of Engineering of Ertec Western, Inc., Long Beach, California.

In 1961, I received a Bachelor of Engineering degree from the University of Auckland, New Zealand. In 1963, I received a Masters of Engineering degree in Geotechnical Engineering from the University of Auckland, and in 1965, I received a Ph.D. in Geotechnical Engineering from the University of California, Berkeley. My graduate studies at the University of California dealt with the earthquake response of embankments.

From 1965 to 1976 I was Associate Professor of Civil Engineering Department for the University of Auckland. My teaching activities at the University included courses in earthquake response of soils and structures. I was instrumental in the development of analytical procedure for predicting ground response during earthquake loading. In 1977, I became Manager of Earthquake Engineering and Soil Dynamics of Ertec Western, Inc. In 1980, I became Director of Engineering and in December of 1980 I became Vice President of Engineering. My duties at Ertec have

included supervision of ground response studies for two nuclear power plants. I have also directed the inhouse development and validation of nonlinear ground response programs.

GEOFFREY R. MARTIN
VICE PRESIDENT
FOR
ENGINEERING

Education

1961 B.E. (Hons.), University of Auckland, New Zealand
1963 M.E., University of Auckland, New Zealand
1965 Ph.D., University of California, Berkeley

Experience

1977-
present Ertec Western. Vice President for Engineering. Oversees all engineering discipline activities including geotechnical services, construction engineering, geomechanics, earthquake engineering, in-situ testing, and laboratory testing. Responsible for technical review of major project activities, development of new engineering capabilities, and technical training of staff. Provides technical guidance to major company projects in areas related to earth dam investigation and design, slope stability, offshore foundations, pavement design, rock mechanics, soil dynamics, and earthquake engineering.

Previously Manager of Earthquake Engineering and Soil Dynamics (1977-80) and Director of Engineering (1980).

1965-
1976 University of Auckland, New Zealand. Associate Professor, Civil Engineering Department. Responsibilities included teaching in the field of geotechnical engineering, research work in the areas of earthquake engineering, dynamic properties of soils, slope stability, foundation design, earth dams, pavement design, rock mechanics, and specialist consulting services to private and public companies and agencies in New Zealand.

Professional Societies, Activities and Awards

Member of New Zealand Institution of Engineers
Member of New Zealand Geomechanics Society
Member of New Zealand Society for Earthquake Engineering
Member of American Society of Civil Engineers

Professional Societies, Activities and Awards (Cont.)

Fulbright Travelling Scholarship, 1962
Harry H. Hilp Memorial Scholarship (Berkeley), 1962
Post Doctoral Research Fellowship (University of Auckland),
1965

Registered Professional Engineer, New Zealand, 1968

Selected Publications and Papers

Research Reports

"Analysis of the Fourth Avenue and "L" Street Slide Areas,"
Report on the Preliminary Studies for the Turnagian Buttress,
Shannon and Wilson, Inc. for U.S. Army Corps of Engineers,
February 1965, coauthor with H. B. Seed.

"Dynamic Response Analyses for Earth Dams," Report No. TE-65-1
to State of California Department of Water Resources, Soil
Mechanics and Bituminous Materials Laboratory, University of
California, Berkeley, October 1966, coauthor with H. B. Seed.

"An Investigation of the Dynamic Response Characteristics of
the Bon Tempe Dam, California," Report, Soil Mechanics and
Bituminous Materials Laboratory, University of California,
Berkeley, October 1966, coauthor with H. B. Seed.

"Rock Slope Stability," Report to the New Zealand National
Roads Board, University of Auckland, School of Engineering
Report No. 81, June 1972, coauthor with L. R. Richards and
P. J. Miller.

"Analysis of the Effect of Multi-Directional Shaking on the
Liquefaction Characteristics of Sands," Report No. EERC 75-41,
University of California, Berkeley, December 1975, coauthor
with H. B. Seed and R. Pyke.

"Methods for the Investigation and Design of Cut Slopes in
Fractured Rock," Report to the New Zealand National Roads
Board, University of Auckland, School of Engineering Report
No. 158, August 1977, coauthor with I. R. Brown.

Selected Publications and Papers (Cont.)

Technical Papers

"The Seismic Coefficient in Earth Dam Design," Journal of the Soil Mechanics and Foundation Division, ASCE, vol. 92, p. 25-58, coauthor with H. B. Seed, 1966.

"The Dynamic Response of Cohesive Earth Dams to Earthquakes," Proceedings Fifth Australia-New Zealand Conference on Soil Mechanics and Foundation Engineering, Auckland, p. 121-131, 1967.

"Earthquake Resistant Design of Cohesive Earth Slopes," Bull. N.Z. Soc. for Earthquake Engineering, vol. 4, no. 1, p. 51-72, 1971, coauthor with P. W. Taylor.

"Effects of Anisotropy and Sample Disturbance on the $\phi = 0$ Stability Analysis," 1st Australia-N.Z. Conference on Geomechanics, Melbourne, p. 349-354, 1971, coauthor with T. J. Kayes.

"Joint Strength Characteristics of a Weathered Rock," Proceedings 3rd Congress International Society of Rock Mechanics, Denver, vol. 2A, p. 263-270, 1974, coauthor with P. J. Miller.

"Stability of Slopes in Weathered and Jointed Rock," Proceedings of a Symposium on the Stability of Slopes in Natural Ground, N.Z. Geomechanics Society, p. 7.1-7.14, 1974, coauthor with P. J. Miller.

"Fundamentals of Liquefaction under Cyclic Loading," Journal of the Geotechnical Engineering Division, ASCE, vol. 101, no. GT5, p. 423-438, 1975, coauthor with W. D. L. Finn and H. B. Seed.

"Dynamic Triaxial Testing of Basecourse Aggregate," Conference on Repeated Loading of Soils with Particular Reference to Road Pavements, University of New South Wales, 1975.

"The Effects of Saturation on Basecourse Deformation Characteristics under Dynamic Loading," N.Z. Roading Symposium, Wellington, 1975, coauthor with D. V. Toan.

"Stress Strain Relations for Sand in Simple Shear," Session 58, Seismic Problems in Geotechnical Engineering, ASCE Annual Conference, Denver, November 1975, coauthor with W. D. L. Finn and K. W. Lee.

Selected Publications and Papers (Cont.)

Technical Papers (Cont.)

"Seismic Response and Liquefaction of Sands," Journal of the Geotechnical Engineering Division, ASCE, vol. 102, no. GT8, August 1976, p. 841-856, coauthor with W. D. L. Finn and P. M. Byrne.

"Constitutive Laws for Sand in Dynamic Shear," Proceedings of the 2nd International Conference on Numerical Methods in Geomechanics, Blacksburg, Virginia, June 1976, vol. 1, p. 270-281, coauthor with W. D. L. Finn and K. W. Lee.

"An Effective Stress Model for Liquefaction," Journal of the Geotechnical Engineering Division, ASCE, vol. 103, no. GT6, June 1977, p. 517-532, coauthor with W. D. L. Finn and K. W. Lee.

"Dynamic Effective Stress Analysis of Sands," Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, 1977, vol. 2, p. 231-236, coauthor with W. D. L. Finn and K. W. Lee.

"Effect of Multidirectional Shaking on Pore Pressure Development in Sands," Journal of the Geotechnical Engineering Division, ASCE, vol. 104, no. GT1, January 1978, p. 27-44, coauthor with H. B. Seed and R. M. Pyke.

"Response of Saturated Sands to Earthquake and Wave Induced Forces," in Numerical Methods in Offshore Engineering, Ed. Zienkiewicz Lewis and Stagg, John Wiley, 1978, p. 515-554, coauthor with W. D. Finn, K. W. Lee and P. M. Byrne.

"Effects of System Compliance on Liquefaction Tests," Journal of the Geotechnical Engineering Division, ASCE, vol. 104, no. GT4, p. 463-480, April 1978, coauthor with W. D. Liam Finn and H. Bolton Seed.

"Application of Effective Stress Methods for Offshore Seismic Design in Cohesionless Seafloor Soils" Proceedings 10th Annual Offshore Technology Conference, vol. 1, p. 521-528, coauthor with W. D. Liam Finn and Michael K. W. Lee.

"Comparison of Dynamic Analyses for Saturated Sands," Proceedings of the ASCE Specialty Conference of Earthquake Engineering and Soil Dynamics, vol. 1, p. 472-491, Pasadena, June 1978, coauthor with W. D. Liam Finn and Michael K. W. Lee.

Selected Publications and Papers (Cont.)

Technical Papers (Cont.)

"Determination of Site Dependent Spectra Using Non-Linear Analysis," Proceedings of the Second International Conference on Microzonation for Safer Construction - Research and Application, San Francisco, 1978, coauthor with C. F. Tsai and I. Lam.

"Seismic Design Considerations for Bridge Foundations and Site Liquefaction Potential," Proceedings of a Workshop on Earthquake Resistance of Highway Bridges, Applied Technology Council, January 1979.

"Factors Involved in the Seismic Design of Bridge Abutments," Proceedings of a Workshop on Earthquake Resistance of Highway Bridges, Applied Technology Council, January 1979, coauthor with David A. Elms.

"Seismic Design of Piled Offshore Platforms in Sand, Specialty Session on "Soil Dynamics in the Marine Environment," Preprint 3604, ASCE Convention, Boston, April 1979, coauthor with W. D. Liam Finn.

"Seismic Response of Cohesive Marine Soils," Specialty Session on "Soil Dynamics in the Marine Environment," Preprint 3604, ASCE Convention, Boston, April 1979, coauthor with Chan-Feng Tsai and Ignatius Lam.

"Dissipation of Pore Pressures During Offshore Cyclic Loading," Specialty Session on "Soil Dynamics in the Marine Environment," Preprint 3604, ASCE Convention, Boston, April 1979, coauthor with Ignatius Lam and Chan-Feng Tsai.

GEOFFREY R. MARTIN
ADDENDUM TO RESUME

EARTH DAM EXPERIENCE

- 1964- The Response of Earth Dams to Earthquakes -
1966 This Ph.D. research project entailed both a theoretical and experimental study of dynamic response characteristics of earth dams, and initiated the now commonly used finite element dynamic analysis methods. Experimental studies included forced vibration tests on existing dams and dynamic laboratory tests.
- 1969- Computer Analysis of Earth Dam Stability -
1970 Supervision of a research project reviewing the state of the art for earth dam stability analyses and leading to the development of a comprehensive computer program for earth dam stability analyses.
- 1971- Design of the Compacted Earth Fill Manqatangi Dam for
1976 the Auckland Regional Authority (New Zealand). Responsibilities in this major design project for a 250 foot high dam, undertaken as a consultant to Tonkin and Taylor, Geotechnical Engineers, included borrow investigations, compaction test strip evaluations, laboratory test programs, embankment zoning and seepage control details, stability analyses, earthquake resistant design considerations, instrumentation, and review and analysis of dam performance during construction.
- 1975- Seismic Stability Analysis of Cossey's Dam for
1976 the Auckland Regional Authority (New Zealand). A field and theoretical investigation to assess the earthquake stability of a 100 foot high earth fill dam.

depend on the complete time-space dynamical history of stress release at the source, together with significant distortions introduced by propagation to the receiver. We rarely if ever have enough information in real geologic settings to adequately characterize this detailed behavior of earthquake sources. Therefore, averages of observations at several stations typically are needed to obtain stable, systematic estimates that can be interpreted in terms of our current understanding of the mechanics of faulting.

Seismological Parameters Describing Earthquake Sources

In 1935 C. F. Richter derived his magnitude scale which has become the most commonly used and best-known single-parameter estimate of the strength of an earthquake from seismographic recordings. Patterned after the methods used in astronomy for measuring the brightness of stars, the Richter scale is logarithmic, and the unit is magnitude (M_L), specifically defined as

$$M_L = \log A(R) - \log A_0(R)$$

where $A(R)$ is the maximum trace amplitude (mm) observed at distance R on a standard Wood-Anderson seismograph. $A_0(R)$ is the trace amplitude that defines a magnitude zero event. Richter choose this arbitrarily to be a trace amplitude of 1 (mm) at 100 km distance. Observation at other distances are corrected to give the equivalent amplitude at 100 km. From the above definition, one unit of magnitude corresponds to a factor of 10 in the amplitude of ground motion.

Subsequent work by many investigators, including Richter, has led to a number of (approximately equivalent) magnitude scales, all keyed to this initial Richter scale. At close distances, M_L commonly is calculated from duration of ground motion rather than the maximum trace amplitude. Except for M_L from duration, all of the magnitude scales were developed to make use of more distant observations of different types of ground motion generated by earthquakes and entail both empirical distance and frequency corrections. They are of the general form:

$$M = a \log (A(R,T)/T) + B(R,T)$$

where $A(R,T)$ is the observed ground displacement at distance R and period T , $B(R,T)$ is an empirical correction to adjust the observed ground motion at period T for effects of propagation to distance R , and a is a constant. Thus, m_b represents magnitude estimates based on the first arriving (P-wave) signals with periods around 1 sec observed at large distances. M_s represents estimates based on long-period (T=20 sec) surface waves observed at regional or teleseismic distances. M_{bLg} represents estimates based on short period (T=1 sec) surface waves (Lg) observed at local, regional, or teleseismic distances. A review of the relationships among these scales for different geographical regions is given by Chung and Bernreuter (1981) and Nuttli (1982). For small ($M_L < 4.5$) earthquake in the eastern United States, $M_L = m_b = m_{bLg} = M_s + 1.2$ (Street and Turcotte, 1977; Nuttli, 1982). We know that these one-parameter descriptors (each

based on only a small portion of the seismic signature) cannot adequately describe various source characteristics that are of interest. Kanamori and Anderson (1975) discuss the difficulties in relating magnitude estimates to key physical characteristics of the source, such as seismic energy (E_s), moment (M_o), fault dimension (radius r , length L , width W , area S), fault displacement (D) and stress drop ($\Delta\sigma$).

Their relationships include:

$$M_s \sim \log L^2 \sim 2/3 \log M_o \sim 2/3 \log E_s$$

(most moderate to large earthquake)

$$M_s \sim \log L^3 \sim \log M_o \sim \log E_s \text{ (small events)}$$

$$\log M_o = 1.5 \log S + \log \Delta\sigma + \log C$$

where

$$\begin{aligned} C &= 16/7 \pi^{3/2} \text{ (circular faults)} \\ &= (\pi/2) (W/L)^{1/2} \text{ (strike slip)} \\ &= \frac{\pi}{4} \frac{W}{L}^{1/2} \frac{V^2}{(V_p^2 - V_s^2)} \text{ (dip slip)} \end{aligned}$$

where V_p , V_s are P and S-wave velocities at the source.

Nuttli (1982) presents an excellent summary of several of these empirical relationships for interplate and intraplate regions, specifically the eastern United States. It should also be noted that the empirical relationship given by Street and Turcotte (1977) $m_{blg} = 1.13 \text{ Log}_{10} (\text{area within the Mercalli intensity IV isoseismal}) - .32$ gives an appropriate, relatively stable means of relating magnitude and intensity contour observations in the eastern United States.

As an example relevant to the extensive earlier discussions of maximum magnitude and stress drop at Monticello we can combine these relations as follows:

$$\log M_o = c M_L + d \quad c = \text{constant} = 1.5 \quad d = \text{constant}$$
$$M_o = \frac{16}{7} \Delta\sigma r^3 \quad \text{for a circular fault of radius } r \quad \text{(Brune model)}$$

Thus

$$1.5 M_L + d = \log \left(\frac{16}{7} \Delta\sigma r^3 \right)$$
$$= \log (16/7) + \log \Delta\sigma + 3 \log r$$
$$1.5 M_L + d = \log \left(\frac{16}{7} \right) + \log \Delta\sigma + 3 \log r$$

From this relation the effect of a change in stress drop from 25 to 100 bars (a factor of 4) is to increase the $\log \Delta\sigma$ term by .6 and M_L by $.6/1.5 = .4$ magnitude units. By contrast, increasing the fault radius r by a factor 3 changes M_L by $(3 \times .48)/1.5 = 1$ magnitude unit (i.e., changing the assumed fault diameter ($2r$) from 1 km to 3 km is a factor as 3 in r and implies about a one unit increase in magnitude from approximately $M_L = 4$ to $M_L = 5$). Thus most of the difference in Dr. Murphy's estimate of a maximum magnitude of 5.3 for Monticello compared to the 4.0 estimated maximum by the applicant results from his postulating a maximum fault dimension approximately 3 times as great as the Applicant's estimate, which estimate is based on a large body of site-specific information (Supplementary Seismological Investigation, 1980).

In fact, a large body of published estimates of fault length vs. magnitude (e.g., Wyss, (1979); Kanamori and

Anderson, (1975); Thatcher and Hanks, (1972), 1973); and O'Neill and Healy (1973) indicate that, in general, an increase of 1 magnitude unit corresponds to an increase in fault length by a factor of 4 to 5. If this empirical relationship holds at Monticello, then the magnitude Dr. Murphy attributes to a postulated 3 km fault should be smaller by approximately .3 magnitude units. However, it should be emphasized that there is no evidence of any throughgoing faults at Monticello that would be potential candidates for such an event, regardless of which scaling relation is used.

Stress Drops (Physical Considerations)

It is important to consider certain physical factors that control the stress drop that can be achieved when an earthquake occurs. Clearly one of the fundamental controls is the deviatoric stress (departure from a lithostatic pressure) at the hypocenter. A rupture cannot be initiated or propagate in a zero deviatoric stress environment. At most, a fault rupture can reduce deviatoric stress to zero everywhere on and in the vicinity of the fault surface. This can rarely occur in nature because dynamic friction will stop the rupture before a zero deviatoric stress condition is achieved. Because the confining (lithostatic) pressure increases with depth the dynamic friction that would resist movement on any fault surface increases. Instances in which large fractions of the available deviatoric stress would be relieved would include very shallow

regimes where the confining pressure (thus dynamic frictional resistance during fault movement) is reduced or where there are localized stress concentrations in the neighborhood of which the deviatoric stress is very small. An example of the latter would be a setting where creep processes (slow movements) relieve the stress on a fault surface except at local asperities that are locked across the fault; the rupture of these asperities could result in large stress drops (i.e., a large fraction of the available deviatoric stress is relieved when the asperity fails). The increased pore pressure from reservoir impoundment may also contribute to lower frictional resistance at shallow depths beneath the reservoir.

Empirical seismic observations reveal that stress drops typically are only a small fraction (less than 0.1) of the deviatoric of ambient stress that causes the earthquake. This evidence is summarized in the papers of Hanks (1977), Kanamori and Anderson (1975) and McGarr et al. (1979) for example. Thus, while reported stress drops for a wide range of magnitudes and source regions fall in the 1 to 100 bar range, deviatoric stresses at depth in the lithosphere may lie in the range of hundreds of bars to kilobars. If there are joints and fractures at shallow depths it is unlikely that large deviatoric stresses can be built up except for very local stress concentrations. The borehole stress data and microseismicity patterns indicate such a setting at Monticello. Moreover, if the existing stresses are residual

stresses associated with the boundaries of the plutons, as argued previously by the applicant, then the effect of the microearthquakes that have occurred is simply to relieve these residual stresses. The steadily declining levels of seismicity suggest that such a relaxation process is occurring at Monticello.

The reason for fractional stress drop generally is that the dynamic frictional strength remains at a high percentage of the static friction. A considerable body of laboratory experience in rock mechanics bears on this (e.g., Byerlee, 1978). Typically, sliding friction is found to be roughly 0.9 times the static value in room temperature experiments. Further confirmation of fractional stress drop is given by the mechanical model of King (1975) where stress drops are on the order of 10% of the deviatoric stress and by the observational data for mines by McGarr et al (1981). At the high confining pressures within the lithosphere, it is highly unlikely that dynamic friction could approach zero, thereby allowing complete stress drop. Thus it can be concluded that deviatoric stresses are usually only partly relieved in typical fault movements associated with earthquakes.

Another factor that controls the average stress drop observed from the radiated seismic waves is the mechanism of faulting (i.e., the manner in which the final distribution of fault displacements is achieved once a rupture is initiated). There is much debate about the best mathematical

model to represent an earthquake source; this in part reflects a lack of detailed understanding of fault mechanics under in-situ conditions, especially where pre-existing fractures are present. Three-dimensional computer codes that are capable of representing realistic geologic situations are just beginning to become available and commonly only limited data are available on the three-dimensional distribution of materials properties and stress conditions. A recent report by Day (1979) presents numerical modeling calculations that illustrate some of the effects that heterogeneous stress conditions can have on the rupture process and extent of faulting. His results support our earlier arguments that stress barriers will stop a propagating rupture and thereby impose a limit on the extent of any single fault movement at Monticello.

There also is a basic difference in the physical basis for spectral stress drop estimates that use the Brune (1970) model and those obtained from time-domain calculations using strong motion accelerograph recordings. The spectral estimate theoretically represents the static stress drop and it depends only on the low frequency displacement spectral level and the corner frequency where the displacement spectrum begins to fall off towards higher frequencies. The dynamic stress drop obtained from the rms acceleration in the time domain depends mainly on the high frequency excitation beyond this corner frequency where details of the rupture process over the fault surface are more important.

Hanks (1977) gives a good discussion of this. Further elaboration on the method using rms accelerations and results for Monticello events are given in other testimony by McGuire. Preliminary results for Monticello events indicate reasonable agreement between the two approaches, although the highest (and lowest) estimates have come from the spectral method; it may be inherently more unstable than the time-domain method as argued by Boatwright (1982) for example.

Stress Drop (Observations)

Available estimates of stress drops for earthquakes in intra-plate regions and eastern North America in particular show that stress drop generally increases with earthquake magnitude. Below M_L of about 4.5 the dependence is weaker or less clear. These results have been shown by Nuttli (1982), Street et al. (1975) and Street and Turcotte (1977). It is significant that their data show that for earthquakes with magnitude less than about 5.0, average stress drops are less than approximately 30 bars and below magnitude 4 average stress drops are less than 10 bars. Stress drop determinations by Marion and Long (1980) for southeastern United States microearthquakes are all small (less than 5 bars with most less than 1 bar).

Stress drop estimates by Talwani and co-workers (1981) for Monticello reservoir earthquakes ($M_L < 2.8$) show considerable scatter but typically are less than 20 bars; and

most are considerably less than 10 bars. Additional preliminary results obtained by Talwani (1981) for a suite of events at Monticello whose hypocenters are accurately determined are given in the Appendix (Table A1, Figures A1, A2). Other estimates for which accurate depths are not yet available also show a large scatter in stress drop for a given magnitude, especially at shallow depths. Stress drop estimates given by Fletcher (1981) also exhibit comparable scatter for a similar range of moment (magnitude). The data in the Appendix are all from the same cluster (IV) so that the variations seen are associated principally with source behavior. Stress drop estimates by Boatwright (1982) using the time-domain approach for several Monticello events give comparable estimates as a function of seismic moment. All of these observations taken together, and the data in the Appendix specifically, suggest that the average stress drop increases with magnitude for those small events, consistent with the general experience for the eastern United States discussed earlier. However, data in the Appendix and that given by Boatwright (1982) suggest that the scatter decreases with increasing focal depth and that the envelope of maximum values decreases with depth. This is the type of behavior that would be expected in this heterogeneous setting where the deviatoric stress is not increasing significantly with depth (from Zoback's borehole observations), but the confining (lithostatic) pressure (hence

dynamic friction) is increasing implying that large fractional stress drops are less likely at greater depths.

Given all the heterogeneities known to exist beneath Monticello it is unlikely that the mean stress drop on a large fault surface could exceed the maximum observed for smaller events in the same source region. Thus the maximum values observed of about 25 bars in this shallow regime at Monticello would appear to be a reasonable limit for larger magnitude events. Fletcher (1981) argues that the apparent increase in average stress drop with magnitude implies a limiting magnitude in this environment. We have reached the same conclusion from other arguments given previously (Supplementary Seismological Investigation, 1980) and in new testimony by Nuttli concerning magnitude vs. minimum focal depth. This additional information provides further support for our conclusion that $M_L = 4.0$ is the appropriate maximum magnitude event that can be induced at Monticello.

REFERENCES

- Boatwright, J., 1982, "A Dynamic Model for Far-Field Accelerations, (preprint manuscript).
- Byerlee, J., 1978, "Friction of Rocks," Pageoph, 116, 615-626.
- Chung, D. H. and D. L. Bernreuter, 1981, "Regional Relationships Among Earthquake Magnitude Scales," Rev. of Geophysics and Space Physics, v. 19, No. 4, 649-663.
- Day, S.M., 1979, "Three-Dimensional Finite Difference Simulation of Fault Dynamics," Final Report, NASA Contract No. NAS2-10459, Systems, Science, and Software, La Jolla, California, 72 pp.
- Fletcher, J. B. 1981, "A Comparison Between the Tectonic Stress Measured In-Situ and Stress Parameters from Induced Seismicity at Monticello Reservoir, South Carolina, (manuscript appended to deposition).
- Hanks, T., 1977, "Earthquake Stress Drops," Pageoph, 115, 441-458.
- Hanks, T. C., 1979, "b-values and $W^{-\gamma}$ Seismic Source Models: Implications for Tectonic Stress Variations Along Active Crustal Fault Zones and the Estimation of High-Frequency Strong Ground Motion," JGR, 84, No. 85, 2235-2242.

Kanamori, H., and D. Anderson, 1975, "Theoretical Basis for Some Empirical Observations in Seismology," BSSA, 65, 1073-1096.

King, C.Y., 1975, "Model Seismicity and Faulting Parameters," BSSA, 65, 245-259.

Marion, G.E. and L. T. Long, 1980, "Microearthquake Spectra in the Southeastern United States," BSSA, v. 70, No. 4, 1037-1054.

McGarr, A., S. M. Spottiswoode, and N.C. Gay, 1979, "Observations Relevant to Seismic Driving Stress, Stress Drop, and Efficiency," JGR, 2251-2261.

McGarr, A., R.W.E. Green, and S.M. Spottiswoode (1981), "Strong Ground Motion of Mine Tremors: Some Implications for Near-Source Ground Motion Parameters," Bull. Seis. Soc. Am., 71, 295-319.

Nuttli, O., 1982, "Empirical Magnitude and Spectral Scaling Relations for Mid-Plate and Plate-Margin Earthquakes," Tectonophysics (in press).

O'Neill, M., and J. Healy, 1973, "Determination of Source Parameters of Small Earthquakes from P-Wave Rise Time.," BSSA, 63, 599-614.

Richter, C. F., 1935, "An Instrumental Earthquake Magnitude Scale," BSSA, 25, 1-21.

- Street, R. L., R. B. Herrman, and O.W. Nuttli, 1975, "Spectral Characteristics of the Lg wave Generated by Central United States Earthquakes," Geophysical Journal of the Royal Astronomical Society, 41, 51-63.
- Street, R. L, and F. T. Turcotte, 1977, "Study of Northeastern North American Spectral Moments Magnitudes and Intensities."
- Talwani, P., 1961, (Previous testimony and personal communication).
- Thatcher, W., and T. Hanks, .973, "Source Parameters of Southern California Earthquakes," JGR, 78, 8547-8576.
- Thatcher, W., 1972, "Regional Variations of Seismic Source Parameters in the Northern Baha California Area, JGR, 77, 1549-1565.
- Wyss, M., 1979, "Estimating Maximum Expectable Magnitude of Earthquakes from Fault Dimensions," Geology, 7, 336-340.
- Wyss, M., and J. Brune, 1968, "Seismic Moment, Stress, and Source Dimensions for Earthquakes in the California-Nevada Region," JGR, 73, 4681-4694.
- Brune, J. N., 1970, "Tectonic Stress and the Spectra of Seismic Shear Waves from Earthquakes," JGR, 75, 4977-5009.

TABLE A1
STRESS DROP VALUES AT MONTICELLO
(FROM OCT. 1979 SWARM)

	Origin	Magnitude	Location		Stress Drop				Mean
			Latitude	Longitude	1	6A	7	9A	
			34°	81°					
	05 40 06	1.09	18.04	20.16			1.24		1.24 (1)
	06 38 07	0.99	18.09	19.96	1.18		1.49		1.34 (2)
	07 26 33	0.57	18.33	20.34	2.77	0.83		2.45	2.02 (3)
	12 55 35	1.44	18.03	20.03		2.81			2.81 (1)
791014	06 24 26	1.12	18.43	20.41	5.06	5.36			5.21 (2)
	09 59 54	0.82	18.43	20.11	2.64	1.89	1.82	2.23	2.15 (4)
	12 06 41	2.33	18.43	19.91				5.95	5.95 (1)
	12 16 45	1.02	18.32	81.20	2.09		1.08	1.56	1.58 (3)
	13 20 30	0.99	18.61	19.14	2.4	1.47			1.94 (2)
	19 18 28	0.78	18.39	19.91			0.98	1.01	0.99 (2)
791015	04 44 53	1.56	18.28	19.87		2.36	3.01		2.69 (2)
	06 02 17	1.18				2.41	1.50		1.96 (2)
	06 02 45	1.80	18.42	19.89		2.52	3.8	1.33	2.55 (3)
	06 11 09	1.54	18.39	19.77		2.36	2.24		2.3 (2)
	06 19 54	1.66	18.46	19.93			4.23	6.09	5.26 (2)
	06 20 39	1.32			3.28		1.06		2.17 (3)
	06 30 45	-							
	06 31 29	0.99						2.7	2.7 (1)
	06 58 22	1.09			4.58	3.66		2.13	3.46 (3)
	07 23 48	1.87	18.61	20.18		3.42		0.735	-
	23 57 32	1.02					1.70	2.83	2.27 (2)
	23 59 54	0.82						2.86	2.86 (1)
791016	00 20 16	1.32	18.58	20.09		2.40		4.49	3.45 (2)
	06 33 16	1.44	18.57	20.23	4.87	2.80			3.84 (2)
	07 09 15	1.18	18.46	20.05			1.32	2.50	1.91 (2)
	07 09 55	1.02	18.56	20.27	2.51			1.29	1.90 (2)
	07 27 59	0.82	18.45	20.17	3.5	0.662		2.64	2.27
	22 55 10	1.02	18.57	19.78	2.51			1.75	2.13 (2)
791017	02 32 42	1.37	18.14	20.11			1.88		1.88 (1)
	09 45 48	1.32	18.45	19.85	5.77				5.77 (1)
	09 47 09	0.57	18.41	19.81		0.617			0.617 (1)
791018	15 46 36	1.02	18.02	19.70		1.93	0.81		1.37 (2)
791019	04 53 53	1.18	18.16	20.07	3.06				3.06 (1)
791020	00 97 08	1.37	17.94	19.09		1.19	1.96	1.74	1.63 (3)

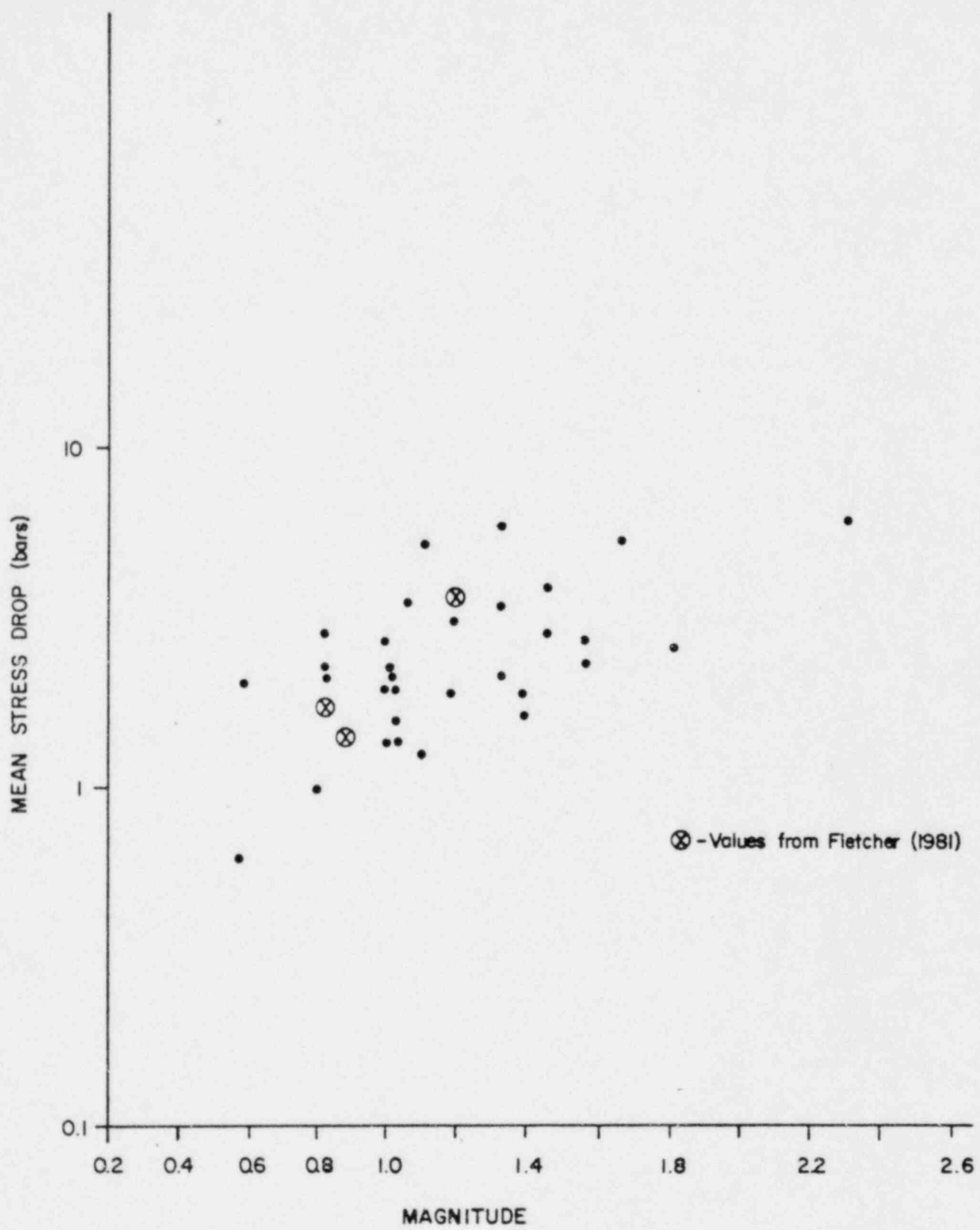


FIGURE A-1 MEAN STRESS DROP VERSUS MAGNITUDE

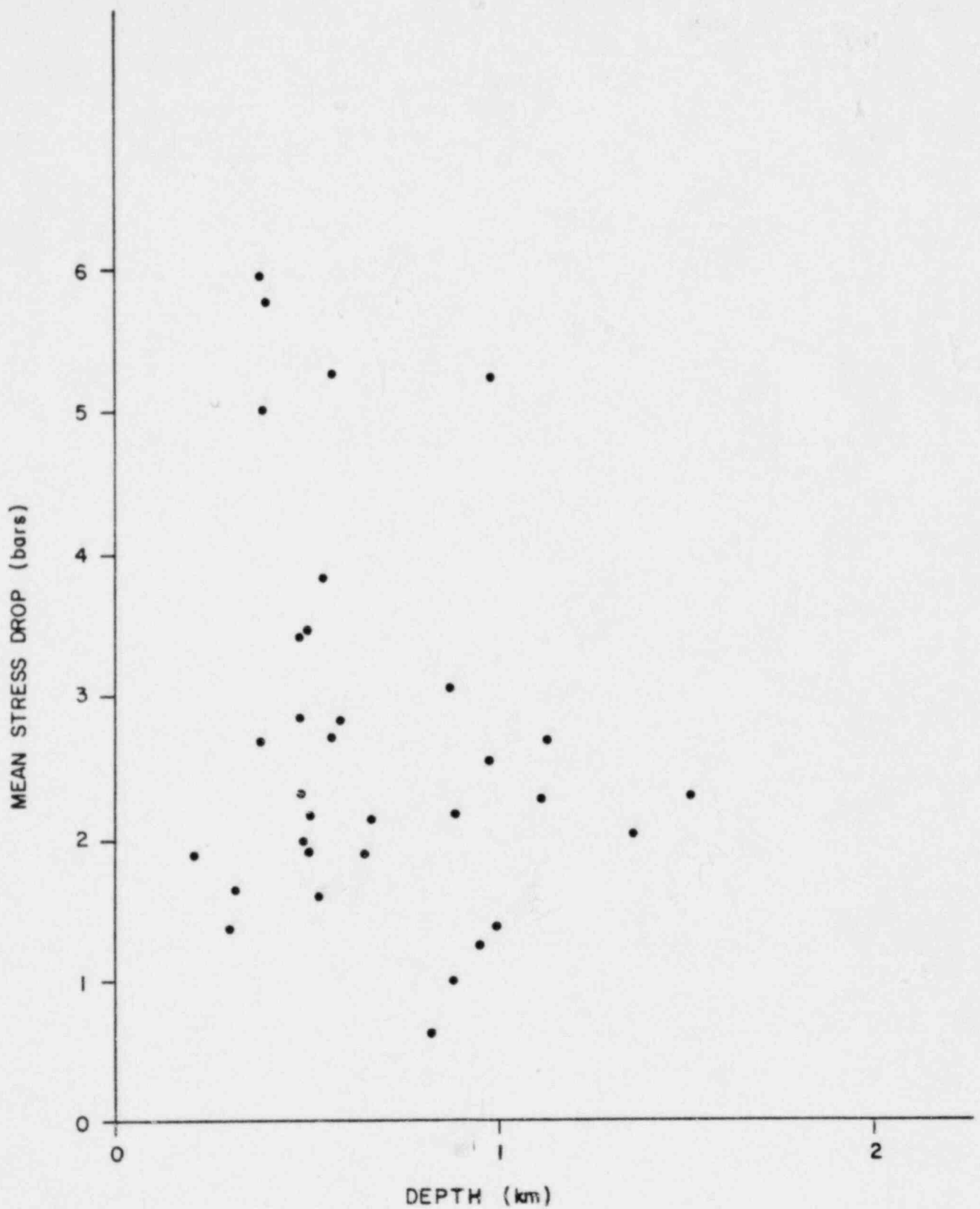


FIGURE A-2 MEAN STRESS DROP (S wave)
VERSUS
DEPTH (all tape location)

TESTIMONY OF
JAMES McWHORTER
SOUTH CAROLINA ELECTRIC & GAS COMPANY
BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

My name is James McWhorter. I am employed by the consulting firm of Dames & Moore, Cranford, New Jersey. A statement of my professional qualifications and relevant experience has been filed previously with the Board. My testimony consists of two reports prepared by me for the Applicants with respect to the Virgii C. Summer Nuclear Station. The first, entitled "comparison of Global Reservoir Induced Seismicity (RIS) to Piedmont RIS Experience" is a comparison of aspects and causative factors of worldwide reservoir-induced seismicity to known or questionable cases of RIS in the Piedmont Province of the eastern United States. The second report, entitled "Charleston 1886 Earthquake - Summary of Licensing V.C. Summer Nuclear Station" is a brief summary of the treatment of the 1886 Charleston earthquake as a design basis for the Virgil C. Summer Nuclear Station.

COMPARISON OF GLOBAL RESERVOIR
INDUCED SEISMICITY (RIS) TO
PIEDMONT RIS EXPERIENCE

James McWhorter

A study, funded by the U.S. Geological Survey on the comparative aspects and causative factors of the phenomenon of Reservoir Induced Seismicity was utilized as a comparison to known or questionable cases of RIS in the Piedmont of the eastern United States.

Figure 1 shows a comparison of reported cases of RIS and very large and/or very deep reservoirs (by depth and volume). Also plotted on this figure are known and questionable cases of Piedmont (USA) RIS. Eleven thousand (11,000) reservoirs without RIS are not plotted within the boxed-out area in the lower left portion of Figure 1. Based on the exhaustive study by Woodward-Clyde (1979), 11 reservoirs having induced earthquakes larger than magnitude 5.0 (out of a total of 64 confirmed cases of RIS) were examined for active faults, nine out of the 11 were found to have active faulting, and the other two were found to be probably associated with active faults, although the data were not unequivocal. These data are tabulated in Table 1.

As reported at the ACRS meeting on March 11, 1981, the cumulative experience of reservoirs in the Piedmont similar to or larger than Monticello Reservoir is a total of 59 reservoirs with 2193 reservoir years of operation. For all

of this experience, the maximum magnitude which has been potentially induced is $M = 4.3$ (Clark Hill), and that occurred several decades after filling the reservoir. There is a question as to whether the August 2, 1974 earthquake was induced by Clark Hill Reservoir, as there was previous seismicity in the region prior to construction and filling of the reservoir. If that event is excluded, there are 12 reservoirs (three with confirmed and nine with questionable RIS) which represent 422 reservoir years of operation and a maximum magnitude of $M = 3.8$ (Keowee, S.C.). These data are plotted on Figure 1 for comparison to the global data, and tabulated in Table 2. A recent study by Dewey (in press) has shown that the events possibly associated with Keowee and Lake Murray were as much as 20 km distant from their respective reservoir associations, which, if true, would reduce the Piedmont RIS experience by two.

Based on the global and Piedmont experience, for reservoirs such as Monticello Reservoir in an intraplate tectonic setting away from active tectonic elements, a maximum magnitude of about 4.0 appears to be appropriate.

REFERENCES

Advisory Committee on Reactor Safeguards, Virgil C. Summer Nuclear Station, transcript of testimony, p. 351, March 11, 1981.

Dewey, James W., in press. Relocation of Intrumentally Recorded Pre-1974 Earthquakes From the South Carolina Region; U.S. Geological Survey Professional Paper.

Woodward Clyde Consultants, 1979, Study of Reservoir Induced Seismicity, Final Technical Report, Contract No. 14-08-0001-16809 (USGS).

TABLE 1
RESERVOIR INDUCED SEISMIC EVENTS
WITH MAXIMUM MAGNITUDE OF 5 OR GREATER

	<u>Packer & Others, 1977</u>		<u>Woodward Clyde, 1979</u>	
	<u>Magnitude^a</u>	<u>Active Faulting^b</u>	<u>Magnitude</u>	<u>Active Faulting</u>
Benmore	5.0	probable	5.0	yes ^c
Eucumbene	5.0	unknown	5.0	yes ^c
Hoover (Lake Mead)	5.0	probable	5.0	yes ^c
Xinfengjiang	6.1	yes	6.0	yes
Kariba	5.8	probable	6.25	probable
Koyna	6.5	probable	6.5	yes ^c
Marathon	> 5	probable	5.75	probable
Oroville	5.7	yes	5.7	yes
Kresmasta	6.3	probable	6.3	yes ^c
Kastraki	6.3	probable	4.6	yes ^c
San Luis	5.0	unknown	not RIS	—
Coyote Valley (Lake Mendocino)	(reservoir not included)		5.3	yes

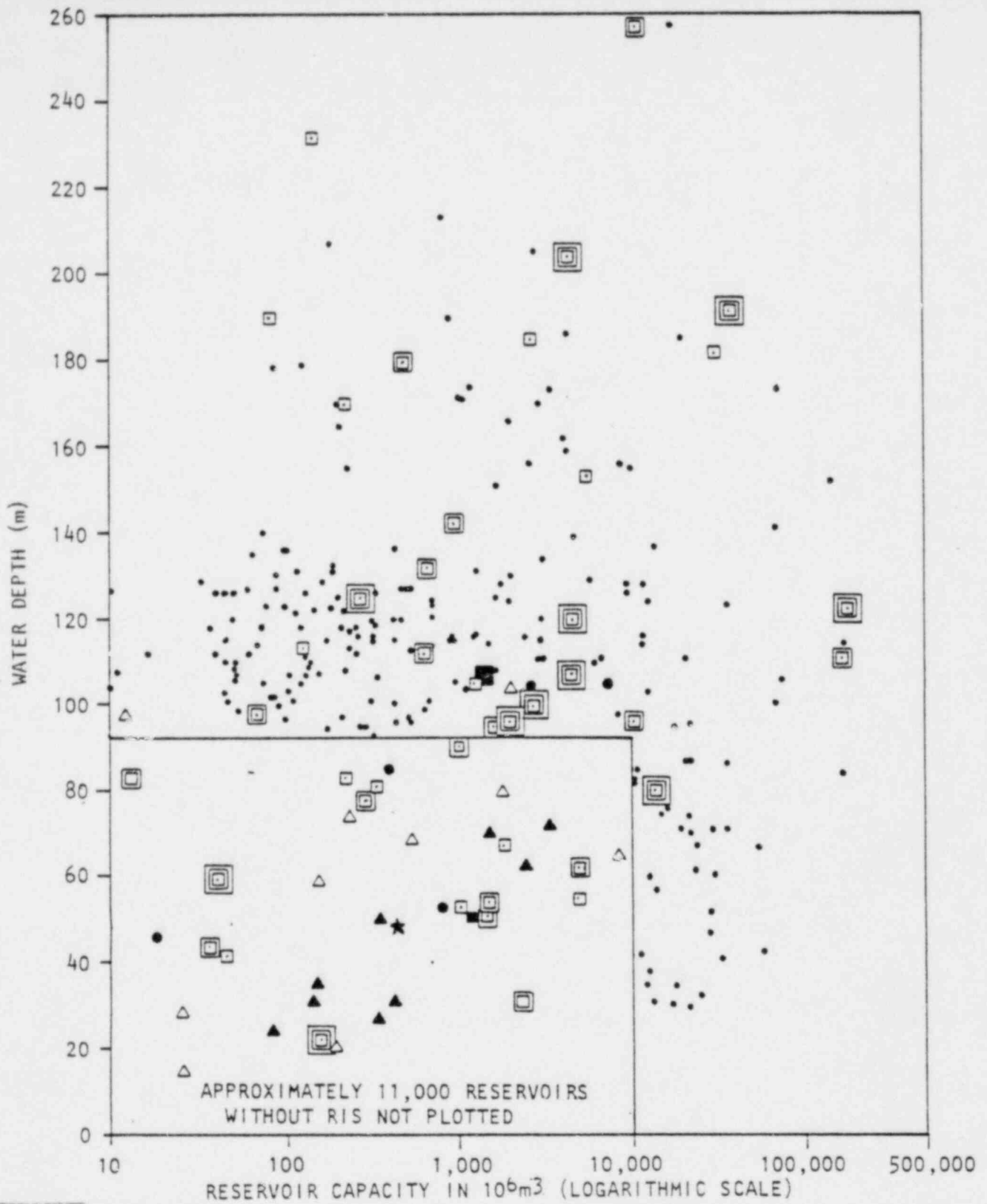
^a Accepted and questionable cases of RIS

^b Faults having displacement in the present tectonic stress regime

^c Field reconnaissance studies

TABLE 2

Reservoir/Dam Name	Location	Dam Height (M)	Reservoir Capacity (10^6 M^3)	Bedrock Geology	Date Comp.	Date 1st Eq.	Max. Eq. Remarks
1. Keowee	Seneca, S.C.	53	1,179	Hornblende and granitic gneiss	1969	12/13/81	$M_{LG} = 3.5$
2. Jocassee	Seneca, S.C.	133	1,431	Hornblende and granitic gneiss	1/1974	11/25/73	$M = 3.2$
3. Oxford	Hickory, N.C.	35.4	151	Mica schist and gneiss	1928	9/9/70	$MMI = V (?)$
4. Saluda/Lake Murray	Irmo, S.C.	63	2,600	Charlotte Belt Gneiss, Slate Belt Volcanics	1930	7/26/45	$MMI = VI (?)$
5. Buzzard's Roost	Greenwood, S.C.	25	334	Charlotte Belt - granitic gneiss	1940	12/73	Felt reports in Greenwood Co. - seismic
6. Clark Hill	Augusta GA/S.C.	67	3,096	Slate Belt Metavolcanics	1952	1960(?)	$M_L = 4.3 \text{ 8/2/74 } (?)$
7. Fairfield	Jenkinsville, S.C.	48	492	Charlotte Belt Gneiss - plutonic rocks	2/78	1/78	$M_L = 2.8$
8. Sinclair	Milledgeville, GA	32	407	Charlotte Belt Gneiss	1953	3/12/64	$MMI = V (?)$
9. Hartwell	Hartwell GA/S.C.	73	3,145	Inner Piedmont Gneiss	1961	10/20/68	$MMI = V (?)$
10. Smith Mountain	Altavista, VA	69	1,357	Precambrian Gneiss and Lynchburg Formation	1963	—	$MMI = V (?)$
11. Lloyd Shoals	Jackson, GA	30	132	Charlotte Belt Gneiss	1910	3/5/14	$MMI = VI (?)$
12. Bridgewater	Marion, N.C.	50	370	Precambrian Mica Gneiss	1919	1919	$MMI = VI (?)$
13. Rhodhiss	Granite Falls, N.C.	22	83	Mica gneiss and schist	1925	7/8/26	$MMI = VI (?)$



EXPLANATION:

- DEEP AND/OR VERY LARGE RESERVOIR
- ◻ ACCEPTED CASE OF RIS, MAXIMUM MAGNITUDE ≥ 5
- ◻ ACCEPTED CASE OF RIS, MAXIMUM MAGNITUDE 3-5
- ◻ ACCEPTED CASE OF RIS, MAXIMUM MAGNITUDE ≤ 3
- △ QUESTIONABLE CASE OF RIS
- NOT RIS
- ▲ QUESTIONABLE CASE OF RIS - PIEDMONT, U.S.A.
- ACCEPTED CASE OF RIS, MAXIMUM MAGNITUDE ≤ 3 (PIEDMONT, U.S.A.)
- ACCEPTED CASE OF RIS, MAXIMUM MAGNITUDE 3-5 (PIEDMONT, U.S.A.)
- ★ MONTICELLO

REFERENCE:
WOODWARD CLYDE CONS. STUDY OF RESERVOIR
INDUCED SEISMICITY FINAL TECHNICAL REPORT
AUGUST 1979

NOTES:
THE FOLLOWING RESERVOIRS WERE NOT PLOTTED BECAUSE
OF INSUFFICIENT DATA: KINARSANI, SHARAWATHI.

COMPARISON OF GLOBAL RIS TO PIEDMONT RIS

FIGURE 1

DAMES & MOORE

CHARLESTON 1886 EARTHQUAKE
SUMMARY OF MATERIALS PRESENTED
IN CONNECTION WITH NRC REVIEW OF
V.C. SUMMER NUCLEAR STATION

James McWhorter

INTRODUCTION

This short topical note is presented to provide a summary of the treatment of the 1886 Charleston earthquake as a design basis for the V. C. Summer Nuclear Station throughout the licensing process. Also discussed is the use of Appendix A to 10 CFR 100 criteria, as it applies to the tectonic province approach to selecting a Safe Shutdown Earthquake.

PSAR INVESTIGATION AND CONSTRUCTION PERMIT APPLICATION

At the time of the preparation of the PSAR (1971) neither Appendix A to 10 CFR 100 nor the accompanying Regulatory Guides and Standard Review Plans were in existence. The applicant proposed an MM Intensity VII (similar to the event at Union County, S.C. in 1913) earthquake at the site as the Design Basis Earthquake (DBE), producing 0.12g on rock in the free field. The Operating Basis Earthquake (OBE) was proposed as a recurrence of the 1886 Charleston Earthquake in Charleston, resulting in a maximum intensity of MM VI-VII at the site (based on isoseismal reports from the site vicinity after the 1886 event) and less than 0.10g acceleration on rock.

During the review by the AEC (now NRC), the applicant agreed to 0.15g on rock and 0.25g on soil for the DBE and 0.10 on rock and 0.15g on soil for the OBE (Amendment 18, PSAR, December 1972).

In the SER dated August 29, 1972, the AEC staff found that (p. 20) "The Charleston Earthquake is related to the structural geology beneath the southeastern Coastal Plain and is believed to be associated with a specific structural anomaly that is confined to the area in the vicinity of Charleston. Evidence indicated that the numerous earthquakes that have occurred in the vicinity of Charleston are localized along the deepest part of the northwest-trending Savannah (SE Georgia) Basin." Additionally, in the Supplement No. 1 to the SER, dated January 12, 1973, the AEC (in response to comments made by ACRS member David Okrent in the ACRS letter to the U.S. AEC dated November 15, 1972) listed several factors which they felt contributed to the rationale of considering the occurrence of a "Charleston type" earthquake not migrating outside the environs of Charleston, South Carolina:

1. The frequency per unit area of historical earthquakes is much higher than elsewhere in the eastern United States. Over 400 earthquakes have been located in the vicinity of Charleston, South Carolina. This represents a frequency per unit area far in excess of that in any other area of the southeastern United States.

2. The seismic event distribution within the Charleston zone of high frequency per unit area does not reflect trends in any direction or predominant patterns which would suggest lateral migration of activity. Conversely, it appears to represent a very localized phenomenon.
3. The microseismic flux in the Charleston area is higher than that measured elsewhere in the eastern United States.
4. Seismic refraction and aeromagnetic data suggest a typical basement structures in the Charleston area.

Although none of these factors is in itself definitive, the cumulative weight supports restriction of the Charleston seismic zone.

In the judgment of the USGS, NOAA, and the Regulatory staff, the present approach for determining seismic design values for the Coastal Plain and Piedmont Provinces provides adequate margins of safety."

Thus, the applicant proposed, and the AEC supported, the rationale that the Charleston 1886 Earthquake was associated with a specific, yet unknown, seismogenic structure and that the cumulative data base did not support migration of that event to a point closer to the site. Interestingly enough, the rate of occurrence of earthquakes per unit area in the Charleston Region appeared to be the most compelling justification in support of localizing "Charleston-Type" events to Charleston.

FSAR PREPARATION, SUPPLEMENTAL SEISMOLOGIC INVESTIGATION AND OPERATING LICENSE APPLICATION

The FSAR was submitted in December 1976 and the NRC staff completed its acceptance review of the FSAR in January

1977. The position regarding the Charleston Earthquake was unchanged from that in the PSAR. Because of the acquisition of new information from USGS research funded by the NRC regarding the geological, geophysical and seismological aspects of the Charleston, S.C. area, in 1980 the NRC requested the applicant to reassess the impact of the Charleston seismicity on the site.

This reassessment was presented in the applicant's report, "Supplemental Seismologic Investigation, Virgil C. Summer Nuclear Station, Unit 1, December 1980." An important aspect of the reassessment of the possible causes of the 1886 Charleston Earthquake was that no interpretation, whether it be decollement reactivation, association with high angle basement faults or stress amplification at the margins of mafic plutons, could unequivocally explain why the Charleston Earthquake occurred where it did. However, based on the weight of evidence available, it would appear that an association of the 1886 event with a high angle basement fault in the Charleston-Summerville area would be the leading candidate at this time. (The Cooke and Helena Banks faults; Behrendt, et al., 1980; and the Woodstock Fault; Talwani, 1981.)

Field evidence to support decollement reactivation during the Charleston 1886 Earthquake is almost non-existent, whether it be slip toward or away from the continent. Likewise, stress amplification around the margins of mafic

plutons in the Charleston area as a causal mechanism for the 1886 event is equally as speculative as the decollement reactivation mechanism.

In summary, the conclusions, both deterministic and probabilistic, of the applicants' investigation revealed that:

1. Several explanations for the cause of the Charleston Earthquake of 1886 are possible (i.e., reactivation of steep basement faults, stress amplification at the margins of mafic plutons or decollement reactivation);
2. Until answers to such generic questions regarding rates of strain accumulation (past and present) in the crust at Charleston; the source of strain energy stored in the crust near Charleston; the specific nature of geologic structure in the basement rocks near Charleston; are obtained, a more accurate assessment of the association of a specific geologic structure to the 1886 earthquake will not be forthcoming;
3. The dense historical seismic activity in the Charleston-Summerville area may be in the most diagnostic constraint in assessing the distribution of similar sized earthquakes in the immediate future;
4. There is no justification based on the cumulative weight of evidence for dismissing the commonly accepted conclusion of restricting the location of a recurrent event of similar intensity as the 1886 Charleston earthquake to the Charleston-Summerville, S.C. zone.

The NRC, in their SER of 1981, reached very similar conclusions regarding the Charleston 1886 earthquake and they were supported by their advisors, USGS and LASL (letter from J. Devine to R. Jackson, dated December 30, 1980, and

letter from C. Newton to R. Jackson, dated December 24, 1980). Principally, their conclusion was (p. 2-39 of the SER);

- "3. We agree with the applicant that the 1886 Charleston earthquake is not the Safe Shutdown Earthquake design event because the weight of the seismic and geologic evidence supports localization of seismicity with structure near Charleston. However, because a clear association between structure and seismicity has not been demonstrated, geological and seismological research should be continued in the Charleston area."

Appendix A to 10 CFR requires at IV (a) (6) that

"correlation of epicenters or locations of highest intensity of historically reported earthquakes, where possible, with tectonic structures any part of which is located within 200 miles of the site. Epicenters or locations of highest intensity which cannot be reasonably (emphasis added) correlated with tectonic structures shall be identified with tectonic provinces any part of which is located within 200 miles of the site;"

It is clear from the licensing record of the V. C. Summer Station from the beginning that the applicant and the regulatory agencies and their advisors have considered the continually emerging weight of geologic and seismologic evidence as a reasonable basis for associating the recurrent dense seismicity in the Charleston area with tectonic structure specific to that locale. Consequently, the Charleston 1886 earthquake or a recurrence of a similar event has not been associated with a tectonic province and migrated to a point closer to the site for purposes of establishing a Safe Shutdown Earthquake.

REFERENCES

South Carolina Electric and Gas Co., 1971, Preliminary Safety Analysis Report, Virgil C. Summer Nuclear Station, Unit 1.

_____, 1976, Final Safety Analysis Report, Virgil C. Summer Nuclear Station, Unit 1.

_____, 1980 Supplemental Seismologic Investigation, Virgil C. Summer Nuclear Station, Unit 1, December 12, 1980.

U. S. Atomic Energy Commission, 1972, Safety Evaluation Report In the Matter of Virgil C. Summer Nuclear Station, Unit 1, 29 August 1972.

_____, 1973, Safety Evaluation Report, Supplement No. 1, In the Matter of Virgil C. Summer Nuclear Station, Unit 1, January 12, 1973.

_____, 1973, Appendix A to 10 CRF 100, December 1973.

U. S. Nuclear Regulatory Commission, 1981, Safety Evaluation Report In the Matter of Virgil C. Summer Nuclear Station, Unit 1, March 1981.

REFERENCES (Continued)

Behrendt, J. C. Hamilton, R. M. Ackerman, H. D. Henry, J. V., and Bayer, K. C., 1980; Seismic Reflection Evidence for Cenozoic Reactivation of Older Faults on Land and Offshore in the Area of the Charleston, South Carolina 1886 Earthquake; Geol. Soc. Amer., Vol. 12, No. 7 Abst., 93rd Ann. Mtg., Atlanta, Ga.

Talwani, P., 1981; The Woodstock Fault is Alive and Ticking Near Charleston, S. C. Abst. presented at Amer. Geophys. Union Ann. Mtg., Fall 1981, San Francisco, Calif.

TESTIMONY OF
CHANG CHEN, PH.D.

SOUTH CAROLINA ELECTRIC & GAS COMPANY
BEFORE ATOMIC SAFETY LICENSING BOARD

My name is Chang Chen. I am the Section Manager of Specialty Structures, Power Division, Gilbert/Commonwealth Companies (G/C). A statement of my professional qualifications, credentials, and work experience was submitted to the Board during the previous evidentiary hearing on June 23, 1981. The purpose of this testimony is to discuss the seismic safety of structures and equipment at the Virgil C. Summer Nuclear Station. First, the overall procedure in performing the seismic resistant design of nuclear power plants is explained. Next, the effects of reservoir-induced seismicity (RIS) on the design of the Virgil C. Summer Nuclear Station will be discussed.

Seismic-Resistant Design of Nuclear Power Plants

The first step in seismic-resistant design is estimation of the ground motion acceleration value for the Safe Shutdown Earthquake (SSE). For sites in the eastern and central United States, seismicity data are typically used to determine acceleration values where no capable faults can be determined in the same seismotectonic region. By comparison, for sites on the Pacific Coast, where capable faults can be located in the same seismotectonic region, the

maximum earthquake potential (generally speaking, in terms of magnitude) of capable faults is estimated first based on characteristics such as fault length, depth, etc.

Next, an attenuation curve (as a function of distance) is used to estimate the acceleration value at the site. This procedure works well where estimates of ground motion are being made for sites located some distance from the source of energy release (several tens of km or greater). For this distance range, sufficient data are available, at least for seismically active areas, to allow empirical estimation of seismic ground motion. For near-source ground motion estimates, fewer empirical data are available and records must be interpreted carefully to account for effects of topography, soil conditions, and frequency content.

After determination of SSE acceleration values, distant earthquakes recorded in the free field or in the basement are often scaled up or down to the SSE values; and then are applied to the mathematical models of nuclear power plants as input motions. The response of a building to earthquakes is a function of the building's natural frequencies. For the same earthquake input, one building may experience a very high (peak) response; yet another building with slightly higher or lower natural frequencies may experience very low (valley) response. An experienced engineer can determine that this is not a satisfactory approach.

For the building with a valley response, a future earthquake with the same maximum acceleration value might produce a peak response because of the different frequency content of the future earthquake. For the building with peak response, the response is overestimated if a linear elastic analysis is being performed; yet, because the concrete and supporting soil have nonlinear characteristics, the result will be a lower actual response. To avoid these problems, many recorded strong motions are used as input and the mean value (or mean value plus one standard deviation) of the maximum responses is used as design value.

To facilitate design and analysis, it is desirable to have a unified approach for all sites. Thus, the mean values plus one standard deviation of maximum responses of single degree of freedom oscillators at different damping values were calculated by Drs. Newmark and Hall using about 28 records, and independently by Dr. Blume using another set of data with 33 records as input. These calculations are the basis of NRC Regulatory Guide 1.60, which describes the method of defining design response spectra based on amplification factors in the acceleration and displacement region (with one g acceleration corresponding to 36-inch displacement). Footnote No. 2 on page 2 of the Regulatory Guide 1.60 states that the guide is not applicable to near-field events. The value on the high-frequency side of the spectrum is called the zero period acceleration (ZPA) or anchor point acceleration of the spectra and is the same as the SSE

value or the effective acceleration of a near-field earthquake. This concept will be discussed further in the next section.

Like recorded strong motions, design response spectra can be used as input to the mathematical model of nuclear power plants. The calculations that use design response spectra as input are much simpler than those that use recorded strong motions. For equipment design, amplified "floor response spectra" are derived from the building characteristics and the ground design response spectra. The calculated raw floor response spectra have sharp peaks. They are broadened and enveloped to produce floor response spectra for design purposes.

Various methods are used to generate floor response spectra. One method is to generate floor response spectra directly from the ground design response spectra without using any time history (of simulated ground motion). The current practice is to generate an artificial time history (of simulated ground motion) in such a way that the calculated response spectra envelopes the design response spectra of Regulatory Guide 1.60. The conservatism built into this enveloping and broadening process is discussed later.

The seismic responses of buildings and equipment are added to the responses of other loads in the same loading combination. The combined stresses are compared with the code allowable stresses. Whenever there is an

overstress, a larger sized member or additional reinforcement is used in the design.

The Effect of Monticello RIS on V.C. Summer Nuclear Station Design

Based on the "Supplemental Seismologic Investigation Report of Virgil C. Summer Nuclear Station," Docket No. 50/395, dated December 1980, and world-wide RIS experience, the maximum RIS event expected at the site is of magnitude $M_L = 4.0$. The effective acceleration of $M_L = 4.0$ is 0.14g which is less than the SSE value of 0.15g (SSIR, App. X). Thus, there is no effect on the V.C. Summer design due to RIS at Monticello.

The NRC staff took a more conservative position by estimating $M_L = 4.5$ as the maximum RIS event. The Applicant has represented this earthquake with an effective acceleration of 0.22g which is higher than the V.C. Summer SSE value of 0.15g on rock and less than the SSE value of 0.25g on soil. Thus, the structures supported on rock were reevaluated for the $M_L = 4.5$ event. As indicated earlier, Regulatory Guide 1.60 is not applicable to near field events. Thus, near-source mean plus one standard deviation design response spectra with an effective acceleration 0.22g as the anchor point were generated. A conservative 2 percent structural damping value was used in the original Virgil C. Summer SSE analysis. The reevaluation used a more realistic 7 percent damping value. The comparison of the Virgil C. Summer 2 percent SSE spectrum and the 0.22g near

field 7 percent spectrum in the Supplemental Seismologic Investigation Report indicated that the 2 percent SSE spectrum was exceeded only in the frequency region greater than 9 (Hz). Among all the Seismic Category I structures, only the Interior Concrete Structures of the reactor building have dominant frequencies higher than 9 (Hz). Hence, the reevaluation was performed for the Interior Concrete Structures of the reactor building only.

The Virgil C. Summer SSE analysis used an artificial time history with high frequency content as input. The high frequency content as shown in FSAR Figure 3.7-5, proved to be very helpful in requalifying the Interior Concrete Structures and the equipment therein. As explained earlier, the artificial time history with calculated response spectrum enveloping the SSE design response spectrum has built-in conservatism. In order to quantify the conservatism, the same statistical concept behind the generation of Regulatory Guide 1.60 was used in the reevaluation. In the statistical analysis, 36 components of time histories were derived to match the near field spectrum in the mean in the frequency region higher than 9 Hz. Each of the 36 components of time histories was applied to the Interior Concrete Structural Model. The 36 components of time histories are modified and scaled up 1975 Oroville aftershocks having M_L between 4 and 5, which are representative of the duration of motion expected for these magnitudes. The mean value of the 36 sets of floor response spectra

with equipment damping at 2% was calculated and compared with the 2% damping SSE floor response spectrum in Appendix X of the Supplemental Seismologic Investigation Report. The comparison showed the SSE floor response spectra exceeded the near field floor response spectra by a large margin except in a narrow frequency region near 25 Hz.

The equipment seismic qualification can be classified in two categories: functionability of active components and structural integrity of non-active components. Active electrical and I & C components are qualified by tests with random wave, sine beat, or sine dwell as input. The basic input criterion is that the calculated response spectrum of input motion envelopes the broadband floor response spectrum. Due to this envelope criterion, the resulting Z.P.A. value of input motion is always much higher than the Z.P.A. value of the broadened floor response spectrum, resulting in considerable margin in the entire frequency region and especially at high frequencies. Non-active components are qualified by calculation.

Rigid systems with fundamental modes in the region of 20 to 30 (Hz) have large moments of inertia and sections in order to reach high frequencies. These systems have ample design margin.

To demonstrate the additional margins available in system design, the seismic stress, design stress, and allowable stress are shown in Table 1 for the emergency feedwater and residual heat removal systems. As shown in

the Table, ample margins are available in the original design to accomodate ground motion from reservoir-induced seismicity.

The ACRS Sub-committee recommended that the facility be analyzed for an $M_L = 5$ earthquake. Such an earthquake would be deeper and farther away than those previously discussed, and the Applicant estimates that the attenuated effective acceleration at the site would be 0.22g. The near field design response spectra of $M_L = 5.0$ events have similar amplification factors as the $M_L = 4.5$ events, but with some higher velocity. The conservatisms built into the Virgil C. Summer design were quantified in the same manner as discussed above for the $M_L = 4.5$ earthquake, and the conclusion reached in the reevaluation of $M_L = 4.5$ events was found to apply to $M_L = 5.0$ events.

Conservatisms built into the structure and its seismic analysis, some of which were quantified in the reanalysis and others of which were not are described in the following paragraphs. All of these contribute to seismic safety at the facility.

Large Foundation Effect

The near field design response spectrum used in the reevaluation is applicable to buildings with small foundation mats. For buildings with large foundation mats, such as nuclear power plants, the design response spectrum can be reduced on the high frequency side. This is analogous to the phenomenon where by large ships "iron out"

wave motions more than do small ships. Although not taken into account in the reevaluation, this effect was acknowledged in the ALAB decision on the Diablo Canyon Plant on 6/16/81 (hereinafter referred to as ALAB 6/16/81 decision).

Ductility Effect Not Accounted for in the Building Design

History shows that buildings designed to one-directional static acceleration values much lower than the effective acceleration value have survived earthquakes. This can be explained by the ductility effect and energy absorption in the plastic deformation. This effect was also acknowledged in the ALAB 6/16/81 decision. The Virgil C. Summer Station design is based on 3 directional earthquakes without using the ductility effect.

Only Design Strength of Concrete at 28 Days or 90 Days and ASTM Specified Minimum Yield Steel Strength Used in Design

The cylinder test strength of concrete at 28 days or 90 days is always higher than the design strength of concrete. Furthermore, as concrete ages, it becomes even stronger. The coupon test strength of steel is also always higher than the ASTM specified minimum strength. This higher in-situ strength of concrete and steel is not used in Virgil C. Summer design or reevaluation.

Actual Member Size Larger Than Required

Prefabricated steel members and reinforcements of certain sizes are used in design and construction. It is very seldom that the required size matches the specified

size exactly. Thus, the members with sizes larger than required are always used.

Radiational Damping Not Accounted For

Radiational damping is the term used to account for the energy lost into the supporting soil or rock. This effect was not accounted for in the reevaluation or the original design.

Broadening and Enveloping Process of Floor Response Spectra

The purpose of broadening the floor response spectra is to account for possible frequency variation. However, when piping systems have more than one dominant frequency that falls into the broadened frequency band of the peak floor response spectrum, the responses are overestimated.

Mass Ratio Effect of Heavy Equipment Not Accounted For

The responses of heavy equipment like the primary coolant loops and polar crane can be reduced when they are in resonance with the supporting structures and the mass ratio effect is taken into account.

Equipment Qualification by Test

Quite often, equipment is qualified by shake table tests using artificial time history as input. The artificial time history is generated in such a way that the calculated response spectrum envelops the broadened floor response spectra which is already the envelope of the raw data. Not only is the conservatism in the envelope on top

of the envelope process, but also this process dictates an artificial time history with a maximum acceleration much higher than the anchor point of the floor response spectrum.

Strain Hardening Not Accounted For and Static Allowables Used For Dynamic Load

In equipment design, material is assumed to behave linearly up to yield point, then deform continuously to collapse when the external load is maintained. In reality, all material used in equipment design exhibits characteristics of strain hardening. This means that resistance to deformation increases after the deformation exceeds the yield point. Furthermore, even if we assume no strain hardening, the material can resist dynamic loads having peak values higher than the yield strength by energy absorption in the plastic region.

Observation of the El Centro Steam Plant Response to the 1979 Imperial Valley Earthquake

The El Centro Steam Plant was designed to 0.2g static lateral load. The recorded peak horizontal motion at the site was 0.5g. The station tripped when station power was lost. One unit was restored to service in 15 minutes and another one in 2 hours. According to calculations performed by Lawrence Livermore Laboratories the load experienced by the plant was 2 to 9 times higher than the design value. Nonetheless, the plant suffered essentially no damage. This exemplifies that a well-engineered structure can resist seismic loads many times higher than its

design value due to the combined conservatisms built into the design.

CONCLUSIONS

I conclude from these studies that reservoir-induced earthquakes at Monticello Reservoir present no safety concern for the Virgil C. Summer Nuclear Station. Inherent conservatisms used in the original SSE analysis, plus additional conservatisms existing in the structure which have not been quantified, imply substantial safety margins for the facility in the event of a reservoir-induced earthquake.

TABLE 1
 VIRGIL C. SUMMER NUCLEAR STATION
 SEISMIC BUILT-IN DESIGN MARGINS

Component	Calculated Combined Stresses or Calculated Input G Values	Seismic Stress	Allowable Stresses or Qualification G Values	Frequency
Emergency Feedwater Piping	21,800 PSI	15,900 PSI	27,000 PSI	4.5 - 44.7 hz
RHR Piping (Class 2)	23,300 PSI	20,600 PSI	29,520 PSI	4.5 - 33.6 hz
Turbine Driven EFW Turbine	.36G/.36G/.21G Test		.5G/.5G/.4G	Larger than 33 hz
Turbine Driven EFW Pump Appurtenances	.36G/.36G/.21G Test		.48G/.48G/0.4G	Larger than 46 hz
RHR Pump & Motor	.21G/.31G/.17G Analysis		2.0G/1.5G/1.5G	Larger than 40 hz
Safety Injection Charging Pump	.29G/.24G/.19G Analysis		3.0G/3.0G/2.0G	Larger than 35 hz

PROFESSIONAL QUALIFICATIONS

DILIP P. JHAVERI

My name is Dilip P. Jhaveri. I am Vice President of URS/John A. Blume & Associates, Engineers of San Francisco, California.

I received a B.E. Degree in Civil Engineering from Gujarat University, India, in 1961. In 1963, I received a M.S.E. Degree in Civil Engineering from the University of Michigan, and in 1967, I received my Ph.D. in Civil Engineering from the University of Michigan.

I was employed as an Assistant Design Engineer by N.N. Purandare, Consulting Engineer, Bombay, India from 1961-1962.

From 1963-1966, I was a Research Associate with the Department of Civil Engineering at the University of Michigan.

In 1967, I began working for URS, John A. Blume & Associates as a Structural Dynamics Engineer. I held this position until 1969, when I became a Project Engineer. In 1977, I became Vice President.

I have extensive experience in structural analyses, and soil-structure dynamics studies. Projects have included seismic analyses of high-rise buildings, piping and equipment, buried structures, and other special structures. I have worked on several projects involving both linear and nonlinear dynamic analysis under earthquake loading.

I have also been responsible in various capacities for seismic analysis of structures for nuclear power plants, including Kewaunee in Wisconsin, Indian Point Units 1 and 2 in New York, and Diablo Canyon in California. I have provided technical consultation on all analyses involving very high g-level seismic inputs for Diablo Canyon. I also have been in charge of a study to improve the seismic resistance capability of General Electric's BWR/6 Mark 888 standard plant and a generic review of the seismic design adequacy of the High-Temperature Gas-Cooled Reactor core and internals performed for the U.S. Nuclear Regulatory Commission.

I have been in charge of soil-structure interaction research conducted for the Electric Power Research Institute and for the Pacific Gas and Electric Company's Diablo Canyon Nuclear Power Plant. I have performed statistical analysis of ground motion records to observe the influence of local soil conditions and have been responsible for the development of analytical techniques and computer programs for two- and three-dimensional static and dynamic analysis of high-rise structures.

I have directed linear and nonlinear seismic analyses of buried radioactive waste tanks and other waste-processing facilities at the Savannah River Plant in South Carolina and the Hanford Atomic Reservation in Washington. I have also been in charge of analysis and seismic design of several high-rise buildings such as the One Embarcadero Center

office building and the Hyatt Regency Hotel in San Francisco
and the Bonaventure Hotel in Los Angeles.

"Evaluation of Residual Response in Truncated Modal Analysis Procedure," presented at the Civil Engineering and Nuclear Power ASCE Specialty Conference, Knoxville, Tennessee (September 1980)

"Seismic Reevaluation and Retrofitting of the Diablo Canyon Nuclear Power Plant Structures," with R. P. Gallagher and D. A. Lang, presented at the Civil Engineering and Nuclear Power ASCE Specialty Conference, Knoxville, Tennessee (September 1980)

"Effective Seismic Input Through Rigid Foundation Filtering," with D. Ray, Transactions of the 4th International Conference on Structural Mechanics in Reactor Technology, San Francisco, California (August 1977)

"Seismic Analysis and Design of Nuclear Power Plant Structures," with P. I. Yanev, CENTO Seminar on Recent Advances in Earthquake Hazard Minimization, Tehran, Iran (November 1976)

"Computer-Aided Structural Analysis and Design of the 37-Story Bonaventure Hotel," with J. P. Nicoletti and L. Z. Emkin, ASCE National Structural Engineering Conference, Madison Wisconsin (August 1976)

"Response of the Bunker Hill Towers Building," with J. P. Nicoletti and W. H. Chaw, Proceedings, National Conference on Earthquake Engineering, Investigations of the San Fernando Earthquake, Los Angeles, California (February 1972)

"Spectral Simulation and Earthquake Site Properties," with S. C. Liu, Journal of the Engineering Mechanics Division, Vol. 95, No. EM5, American Society of Civil Engineers (October 1969)

"Spectral and Correlation Analysis of Ground Motion Accelerograms," with S. C. Liu, Bulletin of the Seismological Society of America, Vol. 59, No. 4 (August 1969)

"Time-History Response of Buildings with Unusual Configurations," with J. A. Blume, Proceedings, Fourth World Conference on Earthquake Engineering, Santiago, Chile (January 1969)

"Earthquake Forces in Tall Buildings in the Setbacks," Ph.D. Thesis, University of Michigan, Ann Arbor (1967)