MISCELLANEOUS PAPER S-71-17

EARTHQUAKE RESISTANCE OF EARTH AND ROCK-FILL DAMS

Report 2

ANALYSIS OF RESPONSE OF RIFLE GAP DAM TO PROJECT RULISON UNDERGROUND NUCLEAR DETONATION

by

J. E. Ahlberg, J. Fowler, L. W. Heller

June 1972

Sponsored by Office, Chief of Engineers, U. S. Army

Conducted by U. S. Army Engineer Waterways Experiment Station
Soils and Pavements Laboratory
Vicksburg, Mississippi

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List of Associated Reports

Previous reports prepared under Engineering Study 540 are:

"A Comparative Summary of Current Earth Dam Analysis Methods for Earthquake Response," issued by Office, Chief of Engineers, as Inclosure 1 to Engineer Technical Letter No. 1110-2-77, 9 December 1969.

"Earthquake Studies for Earth and Rock-fill Dams," issued by Office, Chief of Engineers, as Engineer Technical Letter No. 1110-2-79, 12 January 1970.

"Motion of Rifle Gap Dam, Rifle, Colorado; Project Rulison Underground Nuclear Detonation," published by the Waterways Experiment Station as Miscellaneous Paper S-70-1, January 1970.

"Earthquake Resistance of Earth and Rock-fill Dams; Report 1, Discussions by Professors H. B. Seed and R. V. Whitman," published by the Waterways Experiment Station as Miscellaneous Paper S-71-17, May 1971.

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FOREWORD

This report presents an analysis of the motion of Rifle Gap Dam during the underground nuclear explosion, Project RULISON. This analysis was made for the Office, Chief of Engineers (OCE), by the U. S. Army Engineer Waterways Experiment Station (WES) during fiscal year 1971 under Engineering Study 540, "Earthquake Resistance of Earth and Rockfill Dams."

Engineers of the Soils and Pavements Laboratory, WES, actively engaged in directing the work and report preparation were Messrs. S. J. Johnson, R. W. Cunny, J. Fowler, Dr. L. W. Heller, 1LT J. E. Ahlberg, and SP5 W. C. Moss. The work was under the general supervision of Mr. J. P. Sale, Chief, Soils and Pavements Laboratory. This report was prepared by 1LT Ahlberg with minor contributions by Mr. Fowler and Dr. Heller.

Director of WES during the analysis and the preparation of this report was COL Ernest D. Peixotto, CE, and Technical Director was Mr. F. R. Brown.

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CONTENTS

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CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
miles (U. S. statute)	1.609344	kilometers
pounds	0.45359237	kilograms
pounds per square inch	0.6894757	newtons per square centimeter
kips per square foot	47.8803	kilonewtons per square meter
pounds per cubic foot	16.0185	kilograms per cubic meter
inches per second	2.54	centimeters per second
feet per second	0.3048	meters per second

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SUMMARY

The motion of Rifle Gap Dam was measured in September 1969 during the Project RULISON underground nuclear explosion. The observed response was then compared with the response computed in a mathematical model. Observed and computed responses were similar. From this study it appears that the mathematical models used are applicable to the design and analysis of soil structures, at least for ground motion intensities comparable to those observed at Rifle Gap Dam.

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EARTHQUAKE RESISTANCE OF EARTH AND ROCK-FILL DAMS

ANALYSIS OF RESPONSE OF RIFLE GAP DAM TO PROJECT RULISON UNDERGROUND NUCLEAR DETONATION

PART I: INTRODUCTION

1. The U. S. Army Engineer Waterways Experiment Station (WES) was requested by the Office, Chief of Engineers (OCE), to measure and to analyze the response of Rifle Gap Dam to ground motions generated by the Project RULISON detonation because it was thought that these motions would be similar to those generated by earthquakes. The objective of this study was to determine the applicability of seismic design procedures in designing Corps of Engineers (CE) earth and rock-fill dams to withstand earthquake loadings.

2. Project RULISON, part of the PLOWSHARE program of the Atomic Energy Commission (AEC), was one of a series of detonations for investigating stimulation of the production of natural gas by the use of nuclear explosives. The Austral Oil Company conducted this experiment as a private commercial venture with the assistance of the AEC, which was responsible for safety and the detonation of the nuclear device.

3. The WES instrumented Rifle Gap Dam with the cooperation of the owner, the Bureau of Reclamation (Bu Rec). Other dams instrumented for ground motion measurements during Project RULISON were Harvey Gap Dam, instrumented by the National Ocean Survey and analyzed by the Environmental Research Corporation for the AEC, and Vega Dam, instrumented and analyzed by the Bu Rec. The analysis of Rifle Gap Dam includes an assessment of the geology and elastic properties of the site and the dam, calculations of the expected surface motions using available procedures, and a comparison of calculated responses of the dam with measured responses resulting from Project RULISON.

4. Ground zero (GZ) for Project RULISON was located southwest of

Rifle, Colorado, at a depth of 8442.5 ft.* The nuclear device was detonated at 3:00 p.m. MST, 10 September 1969, and had a design yield of 40 kt.

5. Rifle Gap Dam, an earth-fill structure, was completed in 1966 and is located north of Rifle, Colorado, 18.5 miles from RULISON GZ (see plate 1). Specifications for the dam are presented in reference 1. The dam has a crest length of 1500 ft, a maximum base width of 800 ft, and a maximum height of 120 ft. The dam consists of a mixture of clay, silt, sand, gravel, and cobbles. A cross section is shown in plate 2. The reservoir level was 41 ft below the crest during Project RULISON. Two Bu Rec borings (see boring logs in plate 3 and locations in plate 4) indicate that the foundation materials are alluvial soils consisting of interbedded clays, silts, sands, and gravels to depths greater than 100 ft. Bedrock was not reached in these borings. An assumed profile of the foundation soils is shown in plate 5. The method used to produce the assumed profile is discussed in paragraph 11.

* A table of factors for converting British units of measurement to metric units is presented on page ix.

PART II: FIELD OBSERVATIONS

6. The observed motions of Rifle Gap Dam during Project RULISON are reported in reference 2. The instruments used for the measurements consisted of particle velocity transducers (PVT) and particle acceleration transducers (PAT), and the measurements were recorded by oscillographs and tape recorders. The locations of this equipment are shown in plate 4, and a list of equipment is given in table 1 along with peak observed motions. Histories of the first 6 sec of observed motion and response spectra are presented in Appendix A for vertical and radial (normal to crest of dam) components of motion. Transverse (parallel to crest of dam) motions are reported by Fowler,[2] but are not analyzed herein.

7. There is excellent agreement between the velocity histories observed at location 6 (plate A11) and those calculated at location 5 (plates A9 and A10) by the integration of the acceleration histories. Comparison of the calculated velocity histories at location 1 (plates A1 and A2) in the radial and vertical directions with those observed at location 7 (plate A12) indicates remarkably good agreement, even though locations 1 and 7 were approximately 200 ft apart along the crest of the dam. However, radial components of locations 1 and 7 do indicate a considerable phase shift.

8. Location 4 was in the gate chamber, located in the left abutment of the dam. The motion observed at this location is assumed to be representative of bedrock motion in the valley, and motions for the first 6 sec were used for input in the analyses described later.

9. The Bu Rec took preshot and postshot survey readings from settlement markers on the dam and found that no permanent displacements occurred as a result of Project RULISON.

10. The shear wave velocities of the materials in the foundation and embankment were determined during a WES seismic field investigation (Appendix D). Plate 6 shows the shear wave velocity as a function of depth for the foundation profile obtained from the surface vibratory test data, and shear wave velocities obtained from the Rayleigh wave

dispersion data are shown in plate 7. Curro (Appendix D) suggested that the vibratory shear wave velocity data be used as a lower bound and the maximum Rayleigh wave velocity data be used as an upper bound for the material property description.

PART III: MATERIAL PROPERTIES

11. The foundation shear wave velocity profiles (Appendix D) and Bu Rec borings DH21 and DH22 (plate 3) were combined to produce the assumed foundation profile shown in plate 5. This was accomplished with some difficulty due to the heterogeneity of these soils, which is typical of alluvial deposits. Location 5 will be used as the location for which the observed and calculated responses of the foundation are compared. However, no borings were made at this location and borings DH21 and DH22 are over 500 ft and 400 ft away, respectively. Seismic measurements were made over a considerable area at the downstream toe of the dam (Appendix D).

12. The shear moduli were determined from the shear wave velocity data using the following equation:

$$G = v_s^2 \rho \qquad (1)$$

where

G = shear modulus

v_s = shear wave velocity

ρ = mass density

These moduli are compared in plate 8 with the shear moduli computed from Hardin's equation:[3]

$$G_{max} = \frac{1230(2.973 - e)^2}{1 + e} (OCR)^k (\sigma_o)^{1/2} \qquad (2)$$

where

G_{max} = shear modulus (at low strain amplitudes, i.e. $<0.25 \times 10^{-4}$ percent), psi

e = void ratio

OCR = overconsolidation ratio (1.0 assumed)

k = variable that is a function of the plasticity index

σ_o = mean principal effective stress, psi (horizontal normal stresses were assumed equal)

The shear moduli calculated using equation 2 compare favorably with those obtained using the vibratory technique except at the bedrock-alluvium interface. Thus, the properties in the foundation at location 5 calculated from the vibratory technique were modified according to equation 2 and were used to estimate shear modulus directly under the dam, where it was not measured. This modification is necessary because the weight of the dam will increase the confining pressure, thus increasing the modulus as compared with that measured in the foundation away from the dam. Confining pressures were obtained from a static finite element code (FESS 41) developed at WES. A plot of shear modulus versus depth of the foundation directly under the center line of the dam is shown in plate 9.

13. Test data from the Bu Rec showed that the embankment materials had an average dry unit weight of 119.6 pcf and an average moisture content of 12.6 percent. The shear wave velocity profile obtained using the vibratory technique is shown in plate 10. A Poisson's ratio of 0.4 was assumed for the dam and the foundation material.

14. The shear moduli and damping values used in the final response calculations were obtained by modification for the computed shear strain level.[4] The shear moduli and damping curves are shown in plates 11 and 12, respectively, for sand and in plates 13 and 14, respectively, for saturated clays. The shear moduli as determined from field data or equation 2 were assumed to be at a shear strain of 10^{-4} percent.

15. The depth to bedrock, measured using seismic techniques, ranged from 80 to 120 ft (Appendix D). The bedrock profile varied considerably, and different depths were used in the analyses. For the two-dimensional (2D) analyses, a horizontal bedrock profile was assumed for reasons of simplification and lack of specific information on depth of bedrock under the embankment.

16. The fundamental period of the structure was computed using Ambrasey's equation:[5]

$$T = \frac{2.61H}{V_s} \qquad (3)$$

where

T = fundamental period of the dam, sec

H = height of dam, ≈ 120 ft

V_s = shear wave velocity in dam, ≈ 950 ft/sec (plate 10)

This computation gives a fundamental period of 0.33 sec.

PART IV: CALCULATIONS

General

17. The calculations of the response of the alluvium 500 ft downstream from Rifle Gap Dam were made using three different methods of analysis; these were:

- a. One-dimensional (1D) lumped-mass analysis of foundation alluvium only[6]
- b. 1D Fourier analysis of foundation alluvium only[7]
- c. Two-dimensional (2D) finite element analysis, using modal superposition techniques, of foundation and embankment[6]

The same 2D finite element analysis was also used to calculate the response of the embankment.

18. The response of the foundation material downstream from the dam using the 1D lumped-mass analysis is evaluated in reference 6 as follows:

> Essentially a soil deposit is represented by a series of layers..., the mass of each layer is lumped at the top and bottom of each layer and the masses are connected by shear springs whose characteristics are determined by the stress-strain relationships of the soils in the various layers. Similarly, the damping characteristics of the system are determined by the soil properties.

Modal superposition techniques are used to evaluate the response of the deposit to the input base motion. The base is assumed to be rigid.

19. The dynamic Fourier analysis of layered systems allows consideration of energy radiation into the bedrock and, "uses a one-dimensional Fourier transform analysis to compute the response of linear, viscoelastic, nonuniform soil deposits, subjected to a base excitation."[8] Soil properties assumed were those obtained from the 1D lumped-mass analysis taken from the field vibratory tests and modified for the appropriate shear strain level. The base is assumed to be elastic in this analysis. The amount of energy radiated into the bedrock depends upon the relative stiffness of the soil and bedrock.

20. The finite element method of analysis consists of developing a finite element network, obtaining the stiffness of each element, assembling the elements into a structure, solving the equations of equilibrium using modal superposition techniques, and evaluating the response of the structure. The finite element mesh (plate 15) was generated to account for material zones, stress zones, and the phreatic surface. The element sizes were based on recommendations presented in reference 9. The locations for comparison of the observed and calculated motions are also in plate 15. The soil properties were modified for the shear strain levels obtained during excitation. A horizontal rigid base at the depth of bedrock was assumed.

One-Dimensional Analyses of Foundation

21. The cases investigated using the 1D analyses are listed in table 2. Cases 1-18 were analyzed using the lumped-mass analysis, and cases 19-21 were analyzed using the Fourier analysis. The shear modulus profile used was that obtained from surface vibratory data (plate 6) except in cases 14-18, in which the profile used was that from the Rayleigh wave dispersion data (plate 7). The shear moduli of the clay soil in cases 9-13 and 19 were adjusted by a factor of 1.875 to produce more comparable results. In all other cases, the modulus profile used was that observed from its respective field measurement. The damping value is that value used in the final response calculations after the soil properties have been modified for shear strain. The exact depth to bedrock was unknown; therefore, many depths were analyzed and the value listed in table 2 is that for each respective case. The material classification refers to the curves for modifying material properties for shear strain. S refers to sand and the modification curves in plates 11 and 12, C refers to clay and the curves in plates 13 and 14, and M refers to a layered mixture, as designated in the typical foundation profile (plate 5) in which the respective curve was used for each layer of material. Six seconds of horizontal input motion were used in most analyses. The effect of using 12 sec of motion was

determined in case 13. In case 12, the effect of using raw input data that had not been corrected for base-line shift was investigated. The calculated response spectra and maximum accelerations for the 1D analyses are given in Appendix B.

Two-Dimensional Analyses of Embankment and Foundation

22. Table 3 lists the cases investigated using the 2D analyses. For all cases, the shear moduli (G) were computed from the vibratory shear wave velocity profiles for the foundation and embankment. Because no field measurements were taken directly under the embankment, the shear moduli profile for that location was modified, as shown in plate 9, from the values computed using Hardin's equation.[3] Void ratios of the foundation material were computed from vibratory data. The soil properties of shear modulus and damping were modified for computed shear strains, and the damping value listed is that used in the final calculations. In some of the analyses, the moduli and damping values were changed as shown in table 3 to determine the effects on the computed response. For example, in case 27 a trial was made using a larger modulus than was measured. The modulus of the material in the embankment was multiplied by 2.5, while that of the material in the foundation was multiplied by 1.5. The damping value of the entire system was 4.6 percent. In case 26, the foundation depth was 80 ft; a depth of 100 ft was used in all other 2D analyses. Ninety modes of vibration were used in the 2D analyses. Six seconds of horizontal and vertical acceleration data were used as input motion. It was important to include the vertical component in these analyses because the energy source, an underground nuclear explosion, produced large vertical accelerations at Rifle Gap Dam. The calculated response spectra and maximum accelerations are presented in Appendix C. A damping ratio of 5 percent was used for all spectral calculations.

PART V: COMPARISONS OF OBSERVED AND CALCULATED RESPONSES

Method

23. A systematic method was needed to compare the amplitudes and frequency contents of observed motion records with those of computed motion records. The amplitudes can be compared by using maximum accelerations, while the frequency contents can be compared using response spectra. The number of peaks, periods at which peaks occurred, and relative magnitudes of peaks are used in the spectral comparisons. One-dimensional analysis results were compared with motions at location 5, on the surface of the alluvium. Two-dimensional analysis results were compared with motions at location 5 and at dam locations 1-3.

One-Dimensional Analysis

Observed

24. The maximum observed horizontal acceleration of the alluvium (location 5R) was 0.051 g. The acceleration response spectrum (plate B1) of the observed motion contained three peaks. The largest occurred at a period of 0.15 sec. A peak approximately two-thirds the size of the largest peak occurred at a period of 0.33 sec, and a relatively minor peak occurred at a period of 0.51 sec.

Cases 1-5 (effect of depth of alluvium)

25. The effect of the depth to bedrock, which varied from 80 ft in case 1 to 110 ft in case 5, was investigated in these cases. For cases 1-5, the lumped-mass analysis was used to analyze a sand profile with shear moduli obtained from the surface vibratory tests. The responses of cases 1-5 showed several marked similarities to the observed responses (plate B1). One similarity was the relative magnitudes of peaks, as the second peak was two-thirds the size of the maximum peak and the third peak (when present) was relatively minor. The periods at which peaks occurred were also similar. The maximum peak of the response spectra occurred in the period range of 0.15 to 0.19 sec. In all

cases, a subsequent peak occurred in the period range of 0.31 to 0.33 sec. A minor peak occurred in the period range of 0.49 to 0.51 sec for cases 4 and 5 but was not present for cases 1, 2, and 3. The maximum accelerations ranged from 0.035 to 0.046 g; the accelerations were less than those observed.

26. A comparison of the response spectra for cases 1 and 5 is shown in plate 16. The peaks of the shallower depth profile (80 ft, case 1) are shifted upwards and to the left, indicating more response at lower periods of higher frequencies. Plate 17 shows a comparison of the response spectra for the observed motion and that calculated in case 2, which had a depth to bedrock of 85 ft. The calculated response, case 2, compares favorably with the observed and indicates that response can be predicted using the 1D analysis method.

Cases 6 and 7 (effect of alluvial soil type)

27. The effect of using a clay profile was investigated in cases 6 and 7 (plate B2). For cases 6 and 7, the curves in plates 13 and 14 were used to modify soil properties for shear strain. On the response spectra for cases 6 and 7, the maximum peaks occurred at periods of 0.23 and 0.25 sec, and another peak, two-thirds the size of the maximum, occurred at periods of 0.14 and 0.15 sec. The maximum accelerations calculated were 0.033 and 0.044 g. The depth of soil deposit in case 6 was 90 ft and in case 7 was 110 ft. A comparison of case 6 (clay profile) with case 3 (sand profile) is shown in plate 18. The response curve for case 3 is shifted upwards and to the left of that for case 6, indicating more response at lower periods for case 3 (sand) than for case 6 (clay). The average shear strain used to modify the soil properties for the final response calculation in case 3 was 6.5×10^{-3} percent and in case 6 was 3.7×10^{-3} percent. These strains give modulus reductions of 15 and 32 percent in cases 3 and 6, respectively.

28. The shift in response spectra of case 3 (sand) versus case 6 (clay) (plate 18) is similar to the shift obtained in case 1 versus case 5 (plate 16), where the depth increased from 80 to 110 ft. The equation for the fundamental period $(T_1)i$ of horizontal soil layers, each

having uniform material properties (reference 10), is:

$$(T_1)i = \frac{4H_i}{G_i g/\gamma_9} \qquad (4)$$

where H_i is the thickness of the i^{th} layer, G_i is the shear modulus, g is acceleration due to gravity, and γ_i is the density. This equation shows that a deposit will have a similar change in fundamental period by increasing the depth and decreasing the modulus, or vice versa. This is illustrated by the similar response spectra shifts in plate 16 (case 1 versus case 5) for an increase in depth and in plate 18 (case 3 versus case 6) for a decrease in modulus.

29. A comparison of the observed motion with that calculated in case 6 (clay profile) is shown in plate 19. The agreement between measured and computed responses was better for case 2 (plate 17) than for case 6 (plate 19).

Cases 8-11 (effect of layering)

30. Plate B3 shows comparisons of observed and computed spectra for cases 8-11. The assumed foundation profile (plate 5) of interbedded clays, silts, sands, and gravels was used in case 8. The shear moduli were computed from the vibration data and modified for shear strain by the appropriate curves for sand or clay. Depth to bedrock was 90 ft for cases 8 and 9. The response of the motion in case 8 was similar to the response of case 6 for a 90-ft profile of entirely clay material. A comparison of computed spectra for cases 6 and 8 is shown in plate 20. Plate 21 shows moduli versus depth plots of the vibratory shear data, which have been modified for shear strain, for cases 6 (clay), 3 (sand), and 8 (layered mixture). The responses of cases 6 and 8 are similar, showing that the response at the surface (location 5R) is controlled by low-velocity or low-modulus layers in the profile, even though case 8 has some layers with higher modulus values than does case 6. A comparison of the response spectra of the motions calculated in case 8 with

those which were observed is shown in plate 22; these spectra are not similar.

31. Cases 9-11 used material types such as those shown in plate 5, but with the shear moduli for the cohesive material increased by 87.5 percent. This was done to produce a calculated response similar to the observed response. There were similarities between the computed responses for cases 9-11 and the observed responses. The maximum peak of the response spectra was at a period of 0.15 sec in case 8 and at a period of 0.16 in case 9. The magnitude of the second peak in all cases was approximately two-thirds as great as that of the maximum peak. Cases 10 and 11 had three peaks in the response spectra, while only two peaks were visible in case 8. The maximum accelerations for cases 9, 10, and 11 were 0.042, 0.039, and 0.040 g, respectively.

Cases 12 and 13
(effect of input motion)

32. The effect of base-line shift of the observed acceleration at location 4R (bedrock) was studied in case 12, which was exactly the same as case 11 except that the input data in case 12 were not corrected for base-line shift. As shown in plates B3 (case 11) and B4 (case 12), there was no appreciable difference in the response spectra. Both cases 12 and 13 had a maximum acceleration of 0.040 g.

33. The effect of using 12 sec of input motion was investigated in case 13. Case 13 was exactly the same as case 11 except that 6 sec of input motion was used in case 11. Because there was no appreciable difference between the response spectra for cases 13 and 11 (plate B4) or between the maximum accelerations (0.040 g in both cases), 6 sec of input motion was used in all other cases.

Cases 14-18 (effect of field moduli)

34. As previously stated, the soil properties for cases 14-18 were determined from the Rayleigh wave dispersion data, as shown in plate 7. The only similarity of the calculated response with the observed response was that case 15 had a maximum acceleration of 0.053 g and case 14 had a maximum acceleration of 0.044 g. Comparisons are shown in plate B5. Case 14 had a number of peaks, with the maximum peak

occurring at a period of 0.24 sec. Cases 15-17 each had two peaks, with the maximum occurring at a period of 0.31 to 0.35 sec and a lesser peak occurring at a period of 0.14 to 0.17 sec. The maximum accelerations ranged from 0.070 to 0.074 for cases 16 and 17. The soil properties in case 14 were modified for shear strain using plates 13 and 14; plates 10 and 11 were used to modify the soil properties for shear strain for cases 15-17. Plates 23 and 24 compare the observed responses with the calculated responses in cases 14 and 15, respectively. These plates do not show as good a comparison as do plates 17 and 19, which used the modulus determined from the surface vibratory data.

Cases 19-21 (effect of analysis method and damping)

35. Plate B6 shows comparisons of observed and computed responses for cases 19-21, which used the 1D Fourier analysis for computations of response. The effect of using this type analysis as opposed to the 1D lumped-mass analysis is given in plate 25. This plate shows a comparison of the response spectra for case 20 (Fourier analysis) with the response spectra for case 2 (lumped-mass analysis). Both cases had the same foundation material properties and input motion. The assumed shear wave velocity of the bedrock was approximately five times greater than the shear wave velocity of the overlying soil layer. Note that the response spectra are similar for cases 20 and 2. The response for case 20 is less than that for case 2. The maximum acceleration for case 20 is 0.033 g, whereas for case 2 it is 0.041 g. A like comparison can be made of cases 19 (plate B6) and 8 (plate B3).

36. The effect of reducing the internal soil damping in the Fourier analysis was studied in case 21, which was similar to case 20 except that the damping was reduced by 100 percent. The maximum acceleration for case 21 was increased to 0.035 g, as compared to 0.033 g for case 20. The response spectra were also very similar (see plate B6). Although it was expected that there would be a greater difference in the damped calculated responses, this was not true for the condition at Rifle Gap Dam.

Two-Dimensional Analysis

General

37. The observed radial and vertical motions at locations 1, 2, 3, and 5 were compared with the calculated motions. Transverse motions were measured, but could not be calculated using a 2D analysis method. Plate 4 shows the locations of the PAT's which measured the observed motions, and descriptions of the locations are given in table 1. The locations from which the calculated motions were taken are shown on the finite element network (plate 15). It was necessary for 2D analysis that all locations be in the same vertical plane. Although this is not the true field case, the lateral offsets between PAT and the vertical plane assumed for analysis were not considered large with respect to the horizontal distances in the finite element network. The detailed comparisons of the observed and calculated responses for the radial and vertical components at locations 1, 2, 3, and 5 are given in table 4. The response spectra for the observed and calculated motions for locations 1, 2, 3, and 5 are presented in plates C1–C8.

Case 22 (2D compared with 1D analysis)

38. Case 22 was a 2D finite element analysis of the 120-ft-high embankment and 100-ft-deep foundation as shown in plate 15. Although better comparisons could be made in the 1D analyses for shallower foundations at location 5 (alluvium), the seismic profiles (Appendix D) indicated that an average depth of 100 ft would be a more valid assumption. The shear moduli of the materials were computed from the vibratory shear wave velocities. The shear wave velocity profiles are shown in plate 10 for the embankment and in plate 6 for the foundation at location 5 (alluvium). The shear modulus profile of the foundation directly under the center line of the embankment, which was taken from the vibratory data and modified for confining pressures, is shown in plate 9. The material in the foundation was assumed to respond as a sand, and the curves in plates 11 and 12 were used to modify material properties for shear strain. The material in the embankment is a cohesive material, and the curves in plates 13 and 14 were used to modify material

properties for shear strain. The damping value of 5.3 percent was the average used during the final response calculation after the material properties had been modified for shear strain (see table 3).

39. Case 22 calculated motions were similar to the observed motions. The periods of the maximum calculated peaks were similar to the observed maximum peaks for the radial components at all locations. The maximum calculated accelerations at all radial and vertical locations were only slightly less than the observed accelerations except at locations 2R and 2V where the calculated accelerations were higher than the observed. The periods at which the peaks occurred in the vertical motion response spectra were different from those observed. The calculated response had a maximum peak at the period where the observed response had the second or third largest peak. In the same way, the maximum observed peak occurred in the same periods as the second or third largest peaks on the calculated response curves.

40. The effect of the addition of the vertical accelerations to the response of location 5R can be seen in plate 26, which shows the response spectra and maximum accelerations for cases 4 and 22. Note that the response spectra and maximum accelerations are similar. The foundation soil properties assumed in case 22 are the same as those assumed in case 4. Case 4 was a 1D analysis and only the horizontal input motion could be used. Case 22 was a 2D analysis and used both the horizontal and vertical bedrock acceleration histories as input.

Case 23 (effect of increased modulus)

41. Case 23 was computed to show the effect of increasing the shear modulus of the material in the embankment and foundation. Other assumptions were the same as in case 22 with a 120-ft clay embankment and a 100-ft sand foundation. The value of 4.7 percent damping was used in the final response calculation. Results are shown in plates 27-34. The calculated responses in case 23 of the vertical motions at locations 3V and 5V were similar to those observed. The peaks on the computed response spectra occurred at the same periods as did those observed, and the relative magnitudes were similar. The maximum accelerations of the vertical components were higher than those observed except at location 5V.

The computed horizontal maximum accelerations were usually lower except at location 2R, and the computed acceleration at location 5R was 0.050 g, as compared with an observed acceleration of 0.051 g. The response spectra for the observed and calculated components of the radial motion had peaks occurring at the same periods, but the maximum observed was often at a period corresponding to the second or third calculated maximum, and vice versa.

Case 24 (effect of reduced modulus)

42. The effect of a reduction of shear modulus was computed in case 24. The assumptions were similar to those in cases 22 and 23 except that in case 24 the shear moduli of the foundation and embankment were multiplied by the value 0.5. The calculated motion for case 24 had only a few similarities to the observed motion. The maximum accelerations at locations 2V and 5V were 0.042 and 0.092 g, respectively, and were similar to those observed. The horizontal maximum accelerations were usually lower than those observed except at location 2R. The computed vertical accelerations at locations 1V and 3V were lower and higher, respectively, than the observed accelerations. The period of the maximum, and usually only, peak on the response spectra occurred in the range of 0.30 to 0.34 sec, and the period of the second peak, present only in the vertical motion response spectra, was in the range of 0.14 to 0.18 sec.

Case 25 (effect of reduced damping)

43. Case 25 investigated the effect of reducing the damping value used in response calculations. The only difference between cases 22 and 25 was that the damping value used for case 25 was only two-thirds as great as that used for case 22. Consequently, the maximum accelerations for case 25 were greater than those for case 22, especially in the radial components of locations 1-3 on the dam. The computed response spectra for cases 22 and 25 had the same shape except that a few more minor spikes were present in the response spectra for case 25. This means that only the amplitude of the motion was changed and the frequency content remained essentially unaltered between cases 22 and 25.

Case 26 (effect of depth of alluvium)

44. The effect of an 80-ft-deep foundation was investigated in

case 26. This analysis was made because better comparisons could be made in the 1D analysis at location 5R, downstream from the dam, with the shallower profile. A 120-ft clay embankment, sand foundation, and moduli obtained from the surface vibratory data were used as input, as in case 22. The maximum accelerations computed at locations 1 and 3 were similar to those observed. Similar periods of the peaks for the vertical components at locations 1-3 in the dam were measured. Comparisons of the computed responses with the observed responses at locations 3R and 3V are shown in plates 35 and 36, respectively. The computed responses at locations 2R and 2V were greater than the observed, and at locations 5R and 5V were smaller than the observed. The peaks occurring in the range of 0.12 to 0.17 sec in the observed response spectra for locations 1R and 1V were only minor in the computed response for case 26. Note that not as good a comparison could be made at location 5R for case 26 (plate C7) as could be made with the 1D analysis in case 1 (plate B1). This may be due to the increased strain in the material from the inclusion of the vertical acceleration.

Case 27 (effect of stiff
foundation under stiffer dam)

45. In case 27, the effect of increasing the shear modulus of the material in the foundation by a factor different from that in the embankment was investigated. The shear modulus of the material in the foundation was increased by the factor 1.5, as in case 23. A factor of 2.5 was used to increase the shear modulus of the material in the embankment. As expected, the responses measured at the alluvium locations 5R and 5V were similar to those measured in case 23 (plates 37 and 38). The response in the embankment was similar to that observed. The maximum acceleration and response spectra computed at locations 2R and 2V were very similar to the observed (plates 39 and 40). At locations 3R and 3V, peaks of the calculated response spectra occurred at the same periods as those in the observed spectra, but the maximum acceleration at location 3V was much higher than the observed. The computed maximum acceleration at location 1V was similar to the observed acceleration, but at location 1R, the computed acceleration was much lower than the observed acceleration.

PART VI: DISCUSSION

One-Dimensional Analysis

46. The 1D analysis is used to determine the response of semi-infinite horizontal soil layers. For this reason, the 1D analysis could be used only to calculate response at location 5R on the alluvium downstream of Rifle Gap Dam. During the analysis, it became apparent that many factors were vital in accurately simulating response. These factors include the following:

- a. Shear moduli of the in situ medium
- b. Depth to bedrock
- c. Relationship of shear modulus and damping to strain amplitude for modification of soil properties

47. Use of the in situ shear moduli as measured by WES in the seismic field investigation using the surface vibratory technique gave computed results comparable to the field observations. The data determined by the Rayleigh wave dispersion technique did not give calculated results as good as those calculated using the vibratory technique.

48. The depth to bedrock in the foundation material varied considerably throughout the Rifle Gap Dam area. No borings were made at the site of location 5 on the alluvium downstream from the dam. Therefore, no accurate determination of soil depth could be used in the 1D analyses. The effect of depth to bedrock is shown in plate 16, which compares the response of an 80-ft-deep deposit with that of a 110-ft-deep deposit. Better agreement with the observed response was obtained using the shallower bedrock depths.

49. The type of material used, sand or clay, determined the relationship used to modify the material properties for shear strain level. The comparison of material types is made in plate 18 for cases 3 and 6. Both the maximum accelerations and response spectra were considerably different for the two cases investigated. For the foundation material at Rifle Gap Dam, better agreement with the observed data was obtained by considering the material as sand. The granular material was present in

both Bu Rec borings (plate 3), but cohesive material was also present and results comparable to those observed could be obtained only by multiplying the cohesive modulus by a factor of 1.875, which actually gave about the same modulus as that for the sand curves at that shear strain level. Because the exact soil profile is not known at location 5, it can only be concluded that the material in the profile at location 5 responded more closely to the sand curves used in modification of soil properties (plates 11 and 12).

50. For the motions measured at Rifle Gap Dam, the responses did not change in cases 11-13 (plate B4). Case 11 used 6 sec of input motion that had been corrected for base-line shift, whereas the input motion in case 12 was not corrected. Case 13 used 12 sec of input motion. The agreement of cases 11 and 12 showed that the actual acceleration data at location 4R did not have an appreciable base-line shift. The comparison of cases 11 and 13 showed that, when using an elastic analysis, the maximum response occurred before 6 sec and was not changed by the addition of 6 sec more of excitation.

51. The input motion for all analyses was that observed at location 4 on bedrock in the gate chamber (see plate 4). The motion at location 4 was assumed to occur in the bedrock underlying the foundation profile at location 5. Most of the analyses indicated that the response at location 5R could be predicted with the 1D analysis; thus, the assumption that the motion observed at location 4 was bedrock motion was apparently a valid assumption.

52. The 1D analysis gave results very similar to those observed for the alluvium (location 5R). With the addition of the vertical motion input in the 2D analysis, plate 26 shows that good agreement between the 1D and 2D analysis results is obtained. This means that the much simpler and cheaper 1D analysis can be used to predict the horizontal motion in a semi-infinite soil deposit even during three-dimensional dimensional excitation.

Two-Dimensional Analysis

53. Similar results were obtained at locations 5R and 5V for

cases 23 and 27, as shown in plates 37 and 38. The only difference in the two cases was the increase of modulus in the embankment for case 27 over that for case 23. This shows that the instrument located more than 520 ft downstream from the 120-ft-high dam was at a sufficient distance from the dam that its response was not affected by the structure (generally called free-field response).

54. The response of the foundation material was best predicted by case 23 (plates 33 and 34) in which the vibratory shear modulus was multiplied by 1.5. However, comparable results were obtained for case 22 in which the shear modulus was that computed from the field measurements. Thus, the shear wave velocities measured using the surface vibratory technique can be used in the 2D finite element models.

55. The responses computed at locations 2R and 2V were usually greater than those observed. Better agreement was obtained by increasing the modulus in the structure, and similar results were obtained for case 27, as shown in plates 39 and 40.

56. A change in the damping value for the structure, as in case 25, had only a slight effect on the magnitude of the response. The radial components of motion were more affected than the vertical components. Changing the damping value did not cause a noticeable change in the periods of the peaks.

57. The response at the crest of the dam was the most difficult to predict. Most cases produced similar maximum accelerations but had only one peak at a period of 0.30 to 0.33 sec, which is near the fundamental period of the structure, and did not have substantial response at lower periods, as was observed. Attempts to produce a greater response at the lower periods by increasing the modulus of the dam were successful, but the maximum accelerations were reduced considerably.

58. The change of modulus produces different modal frequencies of the structure. If these are different from the major frequencies of the input, the responses are low. This could have been the reason for the low response in case 27 at locations 1R and 1V, where a greater response was expected. Thus, it is important in design and analysis that a number of inputs be used to produce a smooth response spectrum so that the

structure (frequency of which is not exactly known) can be analyzed for a range of frequencies to find the maximum response.

PART VII: CONCLUSIONS

59. The following conclusions can be drawn from the 1D analyses:

<u>a</u>. The response of horizontal soil layers can be predicted using a 1D analysis.

<u>b</u>. The lumped-mass and Fourier analyses give similar results for a profile in which there is a definite change in shear wave velocities between the bedrock and soil. For the analysis of Rifle Gap Dam, the velocity of the bedrock material was approximately five times that of the soil.

<u>c</u>. It is important to have an accurate determination of the soil profile.

<u>d</u>. It is important to determine the exact depth to bedrock.

<u>e</u>. The soil property modification curves for shear strain level are applicable.

<u>f</u>. The shear wave velocities determined using the surface vibratory technique can be used in the 1D analyses.

60. The following conclusions can be drawn from the 2D analyses:

<u>a</u>. An instrument placed at a distance five times the height of the dam will give free-field response.

<u>b</u>. The shear wave velocities measured using the surface vibratory technique can be used in the 2D analyses, but due to the inclusion of the vertical input motion, a more similar response was calculated at Rifle Gap Dam when the modulus was increased by 50 percent.

<u>c</u>. Due to the complex geometry and material properties of the structure, use of the finite element analysis is necessary to predict the response of various locations in the structure.

<u>d</u>. It is important to use more than one input motion to analyze a structure. A variety of input frequencies is necessary to find the maximum response.

<u>e</u>. There was closer agreement between computed and observed maximum accelerations than between shapes of computed and observed response spectra.

61. For the analysis of Rifle Gap Dam, the following assumptions provided the best agreement between the observed and calculated motions:

<u>a</u>. One-dimensional analysis (location 5R on alluvium)

(1) Lumped-mass analysis method

(2) Shear modulus determined from vibratory test data

(3) Damping from relationships by Seed and Idriss[4] for shear strain

(4) Modulus modified from relationships by Seed and Idriss for shear strain

(5) Sand material

(6) 85-ft depth to bedrock

b. Two-dimensional analysis

(1) Finite element modal superposing analysis method

(2) Shear modulus determined from vibratory test data and increased 50 percent

(3) Damping from relationship by Seed and Idriss for shear strain

(4) Modulus modified from relationships by Seed and Idriss for shear strain

(5) Sand in foundation

(6) Clay in embankment

(7) Average 100-ft depth to bedrock

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Table 1

Summary of Equipment Used, Locations, and Field Measurements

No.	Transducer Location*	Orientation	Particle Acceleration Transducer	Particle Velocity Transducer	Peak Acceleration g's	Peak Velocity ips
1	Crest of dam	Vertical	x		0.062	
		Radial	x		0.094	
2	Downstream face of dam	Vertical	x		-0.048	
		Radial	x		0.038	
3	Near toe of dam	Vertical	x		-0.046	
		Radial	x		0.052	
4	Gate chamber	Vertical	x		-0.028	
		Radial	x		0.014	
		Transverse	x		0.011	
5	Alluvium	Vertical	x		0.088	
		Radial	x		0.051	
		Transverse	x		0.029	
6	Alluvium	Vertical		x		-0.88
		Radial		x		0.74
		Transverse		x		-0.58
7	Crest of dam	Vertical		x		-0.80
		Radial		x		-0.84
		Transverse		x		-1.00

Note: Initial arrival of motion was 6.9 sec after detonation.
* See plate 4.

Table 2

Cases for One-Dimensional Analyses, Location 5R

on Alluvium 500 Ft Downstream from Toe of Dam

Case No.	Type Analysis	Field Test from Which Material Properties Were Determined	Shear Modulus	Damping %	Depth to Bedrock ft	Material Classification*	Remarks
1	Lumped-mass	Vibratory	G	3.9	80	S	
2	Lumped-mass	Vibratory	G	3.8	85	S	
3	Lumped-mass	Vibratory	G	3.7	90	S	
4	Lumped-mass	Vibratory	G	3.5	100	S	
5	Lumped-mass	Vibratory	G	3.7	110	S	
6	Lumped-mass	Vibratory	G	3.2	90	C	
7	Lumped-mass	Vibratory	G	3.3	110	C	
8	Lumped-mass	Vibratory	G	3.3	90	M	
9	Lumped-mass	Vibratory	1.875G**	3.3	90	M	
10	Lumped-mass	Vibratory	1.875G**	3.2	100	M	
11	Lumped-mass	Vibratory	1.875G**	3.2	110	M	
12	Lumped-mass	Vibratory	1.875G**	3.2	110	M	Original acceleration data used for input
13	Lumped-mass	Vibratory	1.875G**	3.2	110	M	12 sec of input for calculating response
14	Lumped-mass	Rayleigh 2	G	4.0	110	C	
15	Lumped-mass	Rayleigh 2	G	4.7	110	S	
16	Lumped-mass	Rayleigh 1	G	4.7	125	S	
17	Lumped-mass	Rayleigh 1	G	5.9	80	S	
18	Lumped-mass	Rayleigh 1	G	3.2	80	C	
19	Fourier	Vibratory	1.875G**	3.4	80	M	
20	Fourier	Vibratory	G	3.8	85	S	
21	Fourier	Vibratory	G	1.9	85	S	1/2 damping of case 20

* S refers to sand and the modification curves in plates 11 and 12, C refers to clay and the curves in plates 13 and 14, and M refers to a mixture as designated in the typical foundation profile in plate 5.

** Only the shear moduli of the clay layers were changed by this factor.

Table 3

Cases for Two-Dimensional Analyses

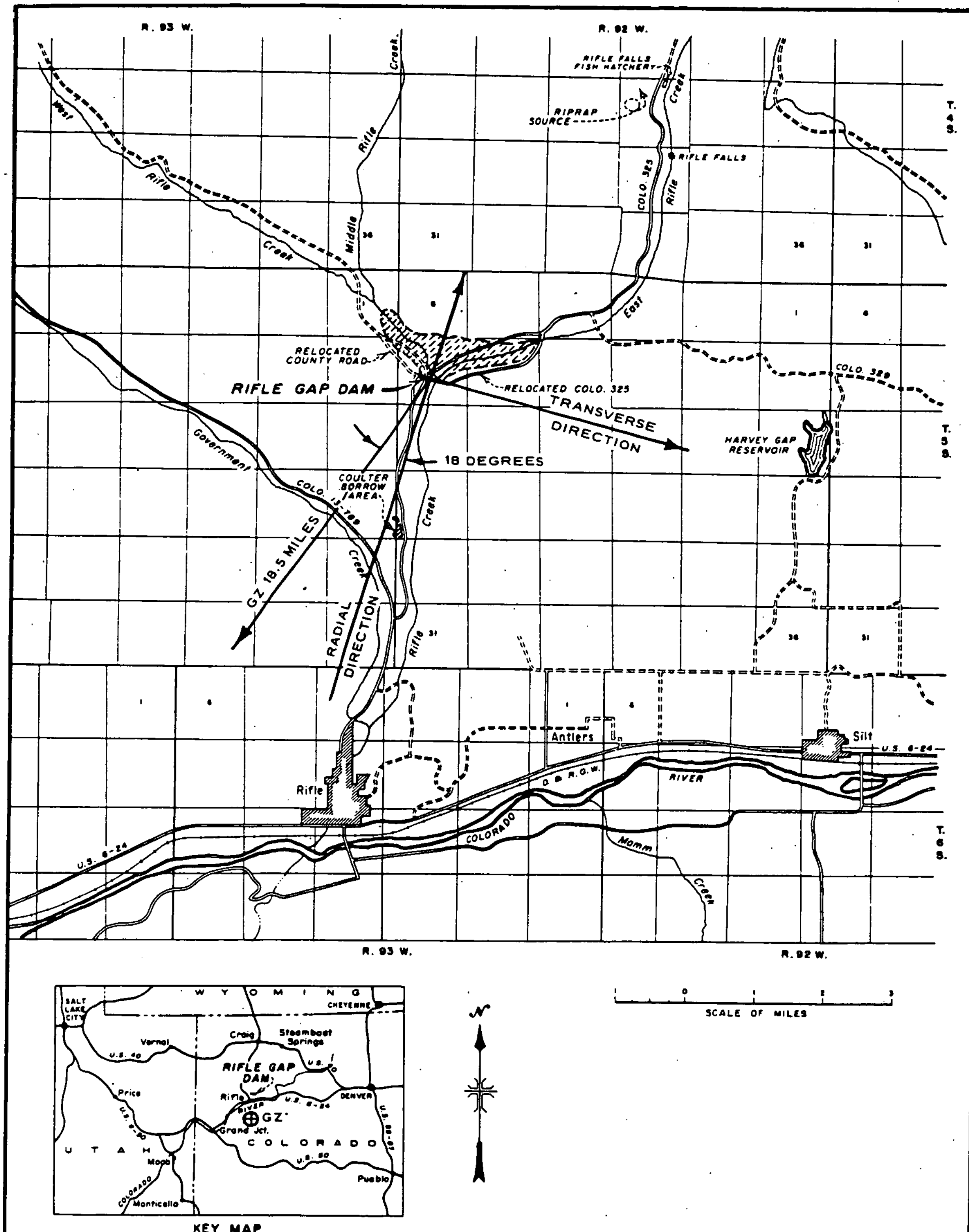
Case No.	Shear Modulus	Damping %	Foundation Depth ft	Remarks
22	G	5.3	100	
23	1.5G	4.7	100	
24	0.5G	5.6	100	
25	G	3.6	100	2/3 damping of case 22
26	G	5.3	80	
27	1.5G (foundation) 2.5G (dam)	4.6	100	

Note: Six seconds of horizontal and vertical acceleration data were used as input motion. The modal superposition analysis method was used. Ninety modes of vibration were considered.

Table 4

Comparison of Observed and Calculated Responses for 2D Analysis

			Period of Peak, sec			Amplitude Ratio of Peaks		
Case No.	Maximum Acceleration g's	Number of Peaks	Maximum Peak	Second Largest Peak	Third Largest Peak	Second to Maximum	Third to Maximum	Remarks
Location 1R, Radial Component, Crest of Dam								
Observed	0.094	3	0.32	0.12	0.17	3/4	3/4	
22	0.073	1	0.31					
23	0.052	3	0.27	0.32	0.14 to 0.18	4/5	2/3	
24	0.041	1	0.32					
25	0.119	2	0.31	0.19		1/3		
26	0.091	1	0.29					Minor spike at 0.17 sec
27	0.035	3	0.29	0.15 to 0.18	0.52	2/3	1/3	
Location 1V, Vertical Component, Crest of Dam								
Observed	0.062	3	0.13	0.31	0.21	2/3	1/3	
22	0.053	1	0.33					
23	0.073	2	0.30	0.14		1/2		
24	0.031	2	0.34	0.16		1/4		
25	0.059	2	0.32	0.19		1/3		
26	0.059	2	0.36	0.15 to 0.17		1/3		
27	0.059	2	0.30	0.13		1/2		
Location 2R, Radial Component, Halfway Down Face of Dam								
Observed	0.038	2	0.32	0.13		3/4		
22	0.076	2	0.29	0.16		1/3		
23	0.058	3	0.24	0.14	0.31	1/1	7/10	
24	0.050	1	0.33					
25	0.098	2	0.31	0.16		1/3		
26	0.087	2	0.30	0.16		2/5		
27	0.034	2	0.15	0.30		2/3		Minor peak at 0.56 sec
Location 2V, Vertical Component, Halfway Down Face of Dam								
Observed	0.048	2	0.08 to 0.13	0.30		4/5		
22	0.102	2	0.31	0.14		1/3		
23	0.060	2	0.29	0.14		5/8		
24	0.042	2	0.33	0.14		1/2		
25	0.123	2	0.30	0.14		1/3		
26	0.097	2	0.30	0.14		2/5		
27	0.059	2	0.29	0.14		1/1		Minor peak at 0.19 sec
Location 3R, Radial Component, Near Toe of Dam								
Observed	0.052	3	0.13	0.20	0.31	5/6	1/2	
22	0.039	3	0.27	0.24	0.34	1/1	2/3	Minor peak at 0.49 sec
23	0.037	2	0.18	0.29		2/3		
24	0.035	1	0.33					
25	0.054	2	0.27	0.24		1/1		Relatively minor peak at 0.51 sec
26	0.049	3	0.27 to 0.32	0.18	0.49	2/3	1/4	
27	0.047	3	0.24	0.14 to 0.19	0.32	1/1	3/4	
Location 3V, Vertical Component, Near Toe of Dam								
Observed	0.046	2	0.13	0.31		9/10		
22	0.075	3	0.30	0.13	0.21	3/5	3/5	
23	0.064	2	0.14 to 0.18	0.30		4/5		
24	0.077	2	0.31	0.14		3/8		
25	0.078	3	0.31	0.21	0.13	2/3	2/3	
26	0.066	3	0.30	0.14	0.21	4/5	3/4	
27	0.068	2	0.14	0.28		7/10		
Location 5R, Radial Component, Alluvium								
Observed	0.051	3	0.15	0.33	0.52	3/5	1/4	
22	0.032	2	0.19 to 0.24	0.50		1/2		Minor spikes present
23	0.050	2	0.15 to 0.18	0.36		9/10		
24	0.046	1	0.30					
25	0.039	4	0.19 to 0.23	0.28	0.32	2/3	1/2	Fourth peak at 0.49 sec
26	0.032	2	0.19	0.31		9/10		Two minor peaks at 0.24 and 0.47 sec
27	0.061	3	0.15	0.36	0.27	8/10	1/2	Minor peak
Location 5V, Vertical Component, Alluvium								
Observed	0.088	2	0.08 to 0.4	0.29		1/2		
22	0.061	3	0.29	0.13	0.20	9/10	8/10	
23	0.072	2	0.14 to 0.19	0.31		1/2		
24	0.092	2	0.32	0.14		3/10		
25	0.063	4	0.22	0.29	0.24	1/1	1/1	Fourth peak at 0.14 sec
26	0.050	3	0.19	0.14	0.28	3/4	2/3	
27	0.069	2	0.14 to 0.19	0.31		1/2		

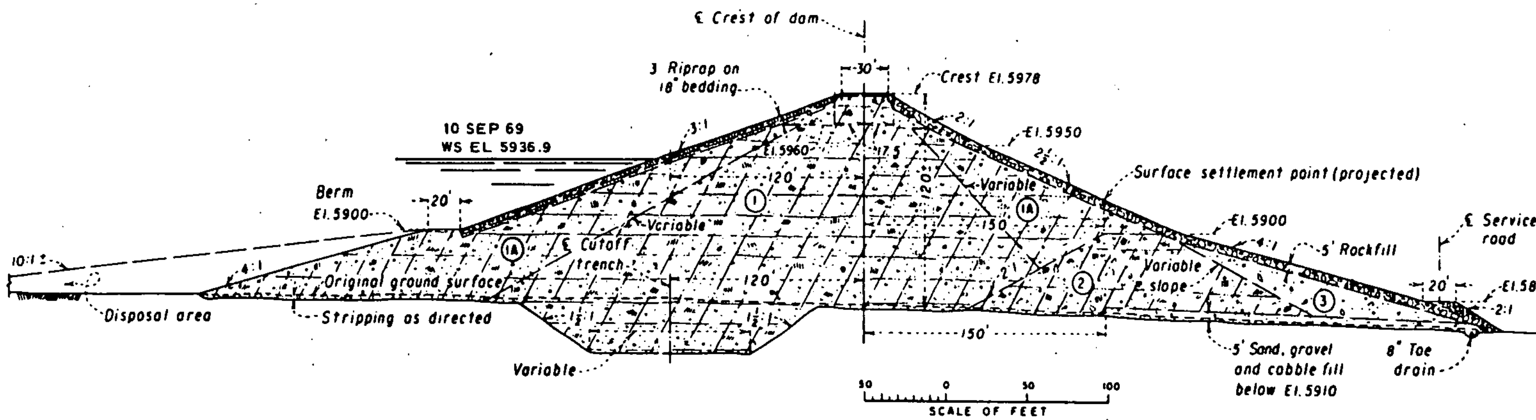


LOCATION MAP

RIFLE GAP DAM

PLATE I

PLATE 2



MAXIMUM SECTION

EMBANKMENT EXPLANATION

1. Clay, silt, sand and gravel compacted by tamping rollers to 6-inch layers.

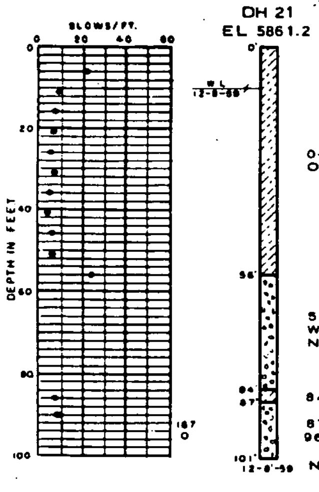
1A. Clay, silt, sand, gravel and cobbles compacted by tamping rollers to 12-inch layers.

2. Miscellaneous clay, silt, sand, gravel, and cobbles or rock fragments compacted by tamping rollers to 12-inch layers.

3. Selected sand, gravel and cobbles or rock fragments placed in 24-inch layers.

GENERAL PLAN AND SECTION

RIFLE GAP DAM



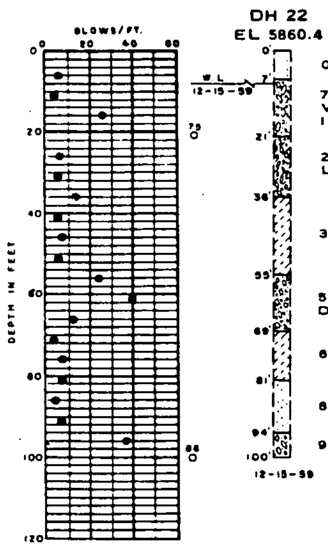
0-56' CLAY, SILTY, SOFT, WITH A FEW WELL PACKED HORIZONS, NO GRAVELS OR COBBLES

56'-84' GRAVEL, SANDY, WITH SOME CLAY, PEA GRAVEL SIZE; WHEN DRILL WATER WAS TURNED OFF THE GRAVEL WOULD FILL INTO HOLE AND WAS NOT ABLE TO TAKE DRIVE SAMPLE, NO LARGE FRAGMENTS WERE FOUND

84'-87' CLAY, SANDY, WITH ONLY A FEW SMALL ROCKS

87'-101' GRAVEL, CLAYEY, ABLE TO GET TWO DRIVE SAMPLES. BELOW 96', 2' BOULDER AND GRAVELS OF QUARTZITE AND LIMESTONE

NOTE: NX SIZE HOLE. NXCS 0-100'. DRIVE SAMPLES 0-55', AND 80'-95'. WASH SAMPLES 55'-80'.



0-7' SAND, FINE WITH LITTLE CLAY, LOOSELY PACKED, TAN

7'-21' BOULDERS, GRAVEL, AND SAND, WITH SOME SILT, DENSE TO VERY DENSE; COARSE MATERIALS CONSISTING OF A VARIETY OF IGNEOUS, SEDIMENTARY AND METAMORPHIC ROCKS.

21'-36' GRAVEL AND CLAY, SILTY, WITH LENSES OF SAND AND GRAVEL, LOOSE TO MEDIUM DENSITY

36'-55' CLAY, SILTY, GRAY, CONSISTENCY IS STIFF

55'-69' GRAVEL AND BOULDERS, WITH LENSES OF SILTY CLAY, MEDIUM DENSITY

69'-81' CLAY, SILTY, MEDIUM CONSISTENCY

81'-94' SAND, FINE WITH SOME CLAY; MEDIUM CONSISTENCY

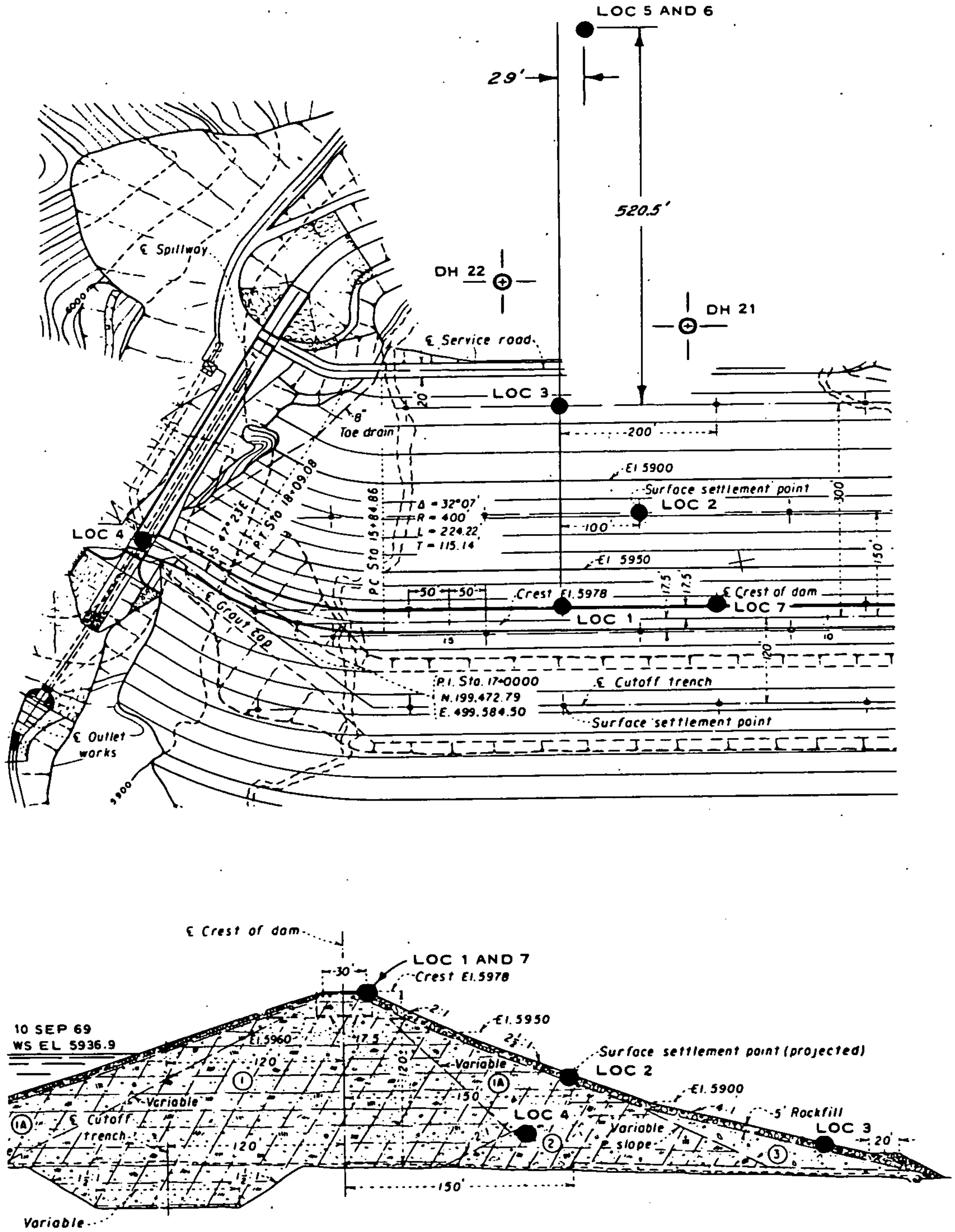
94'-100' GRAVEL, BOULDERS, AND SAND, DENSE TO VERY DENSE

NOTE: ALL MATERIAL FROM 0-100' ARE STREAM CHANNEL AND FLOOD PLAIN DEPOSITS. NXCS 0-100'. ALTERNATING WASH AND DRIVE SAMPLES.

NOTE: SEE PLATE 4 FOR BORING LOCATIONS.

BORING LOGS

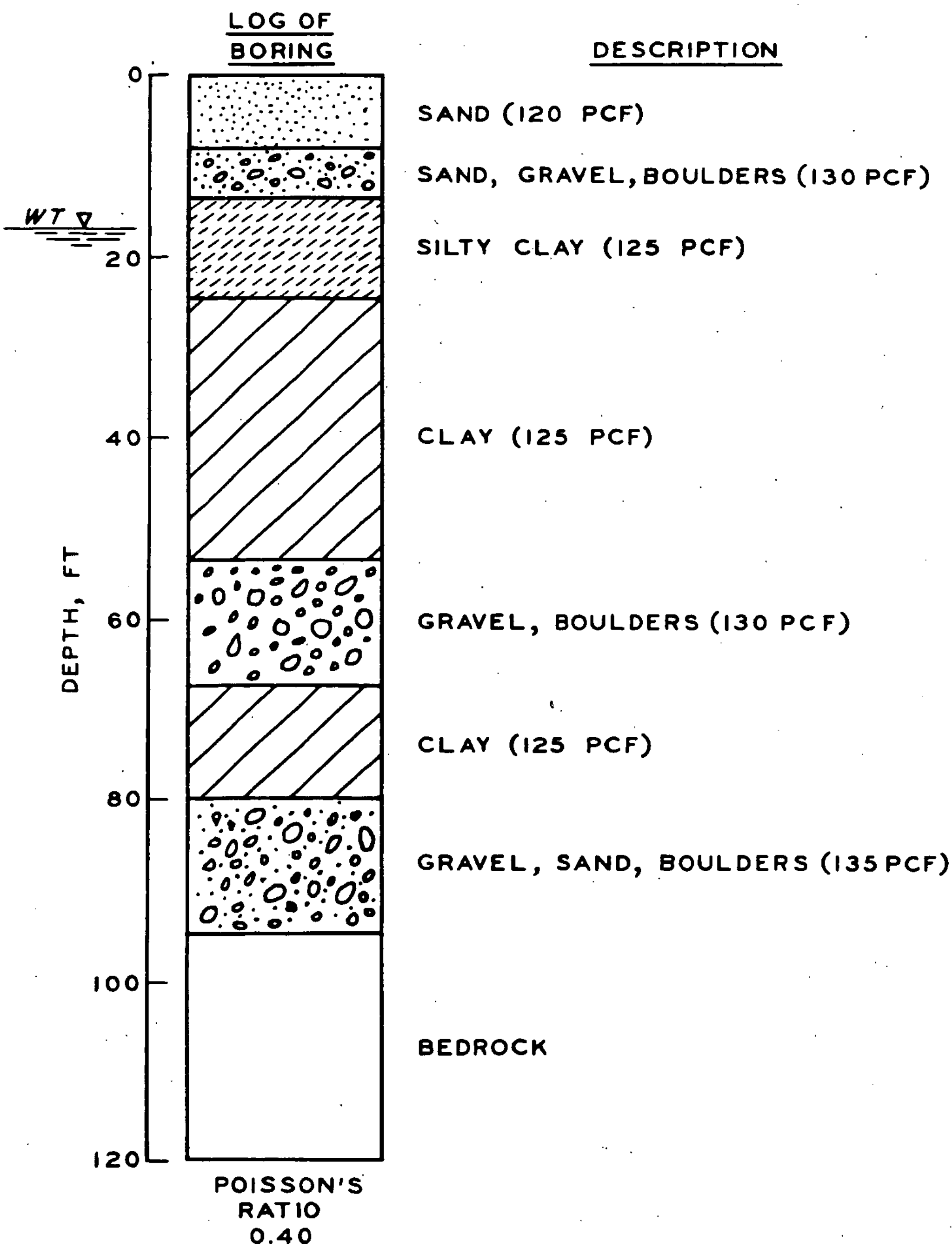
HOLES DH21 AND DH22



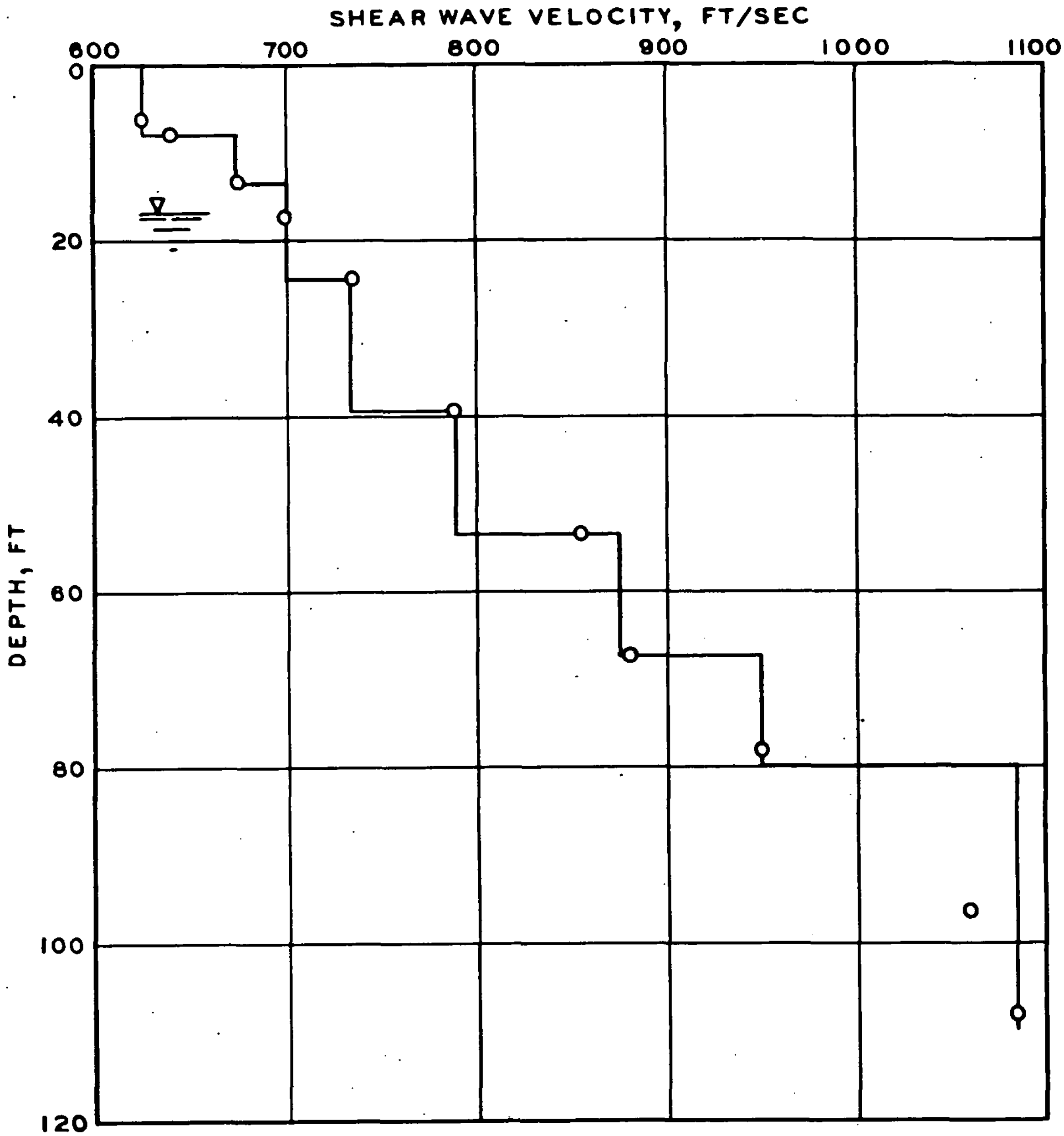
MAXIMUM SECTION

NOTE: LOCATION 4 IS OFFSET 560 FT EAST OF A LINE THROUGH LOC 1 AND LOC 3 AT EL 5878.81.

LOCATIONS OF MOTION MEASURING STATIONS

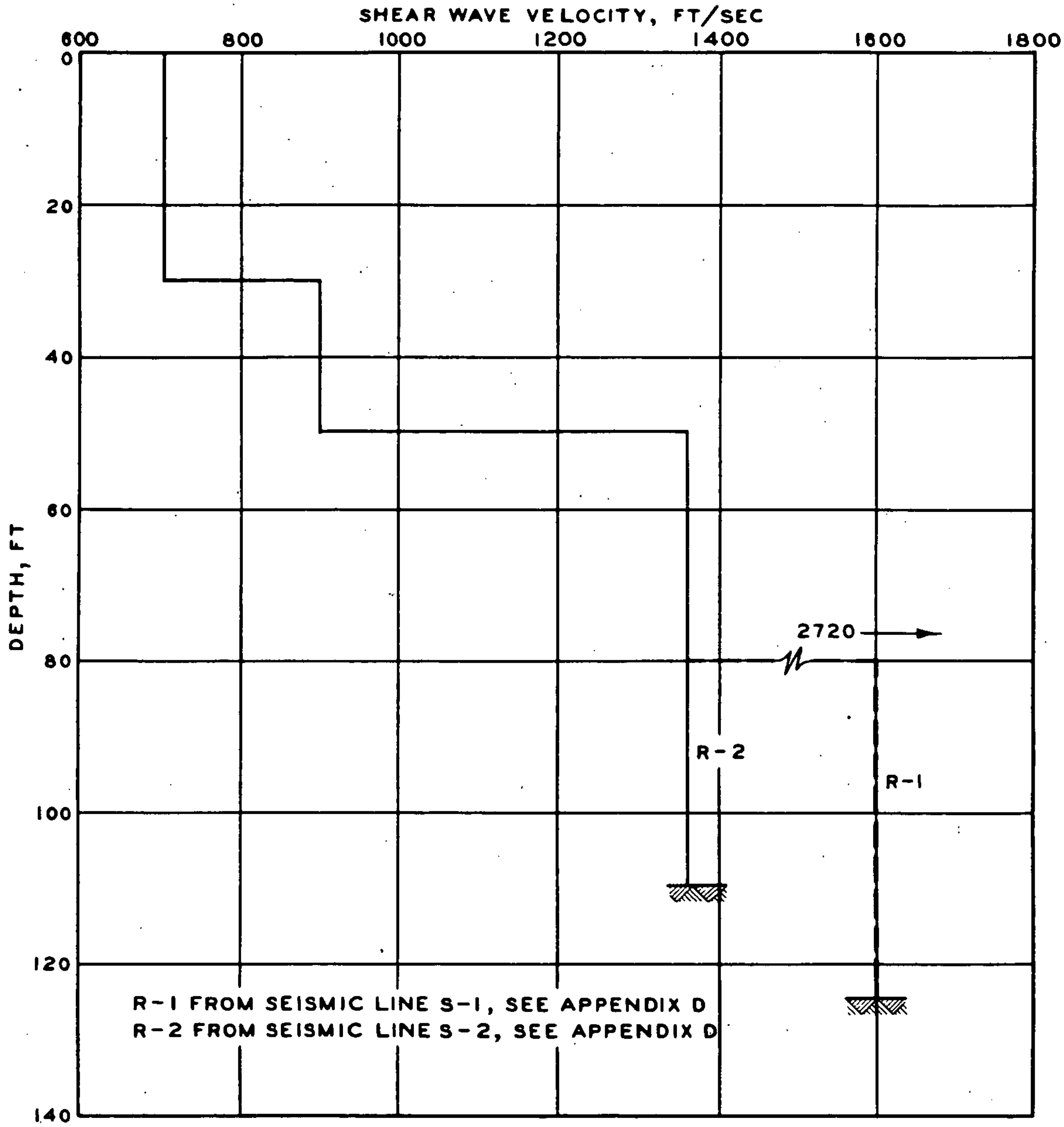


ASSUMED PROFILE OF FOUNDATION SOILS



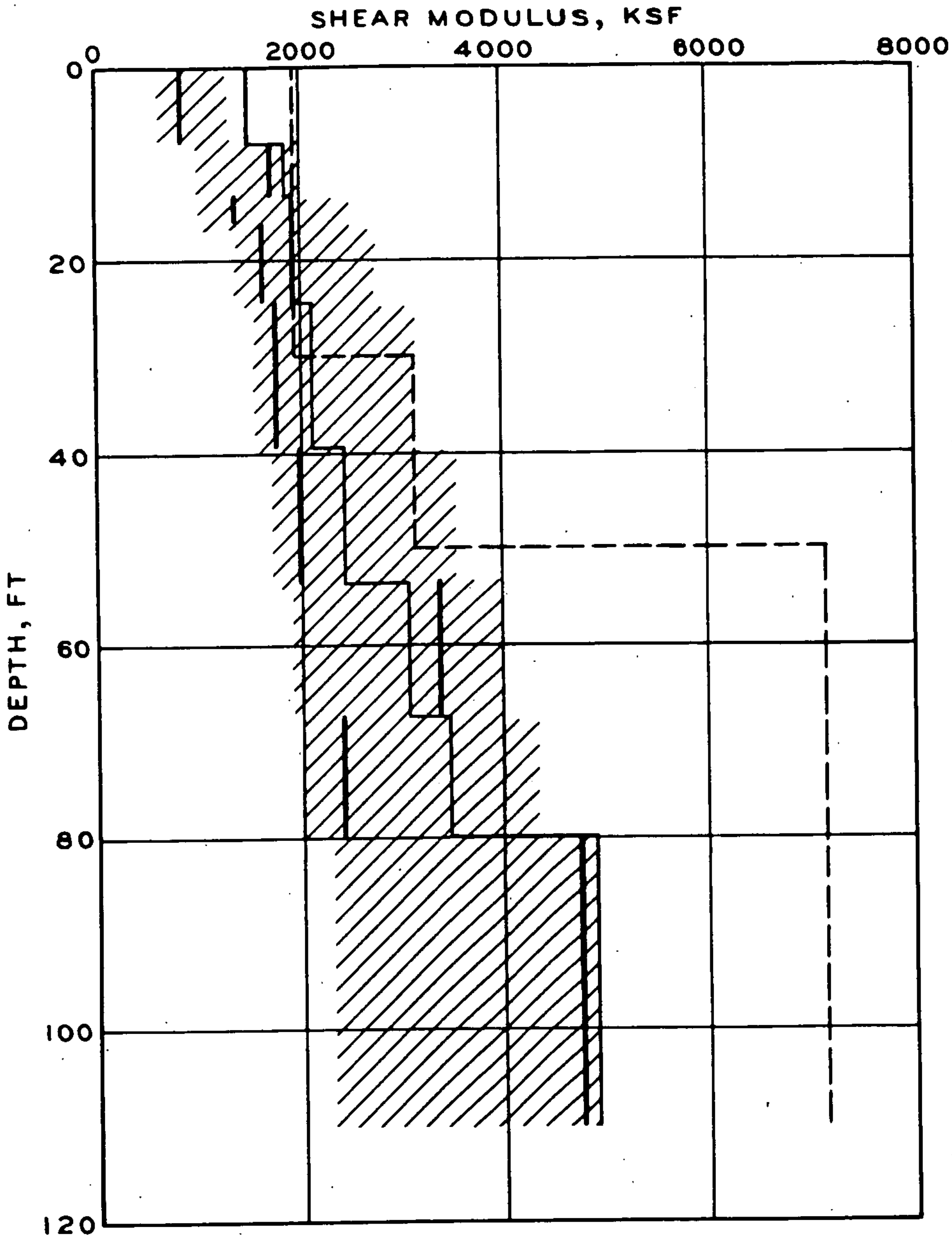
SHEAR WAVE VELOCITY
PROFILE FOR THE FOUNDATION
VIBRATORY TEST DATA

PLATE 6



SHEAR WAVE VELOCITY PROFILE FOR THE FOUNDATION

RAYLEIGH WAVE DISPERSION DATA



LEGEND

——— VIBRATORY DATA

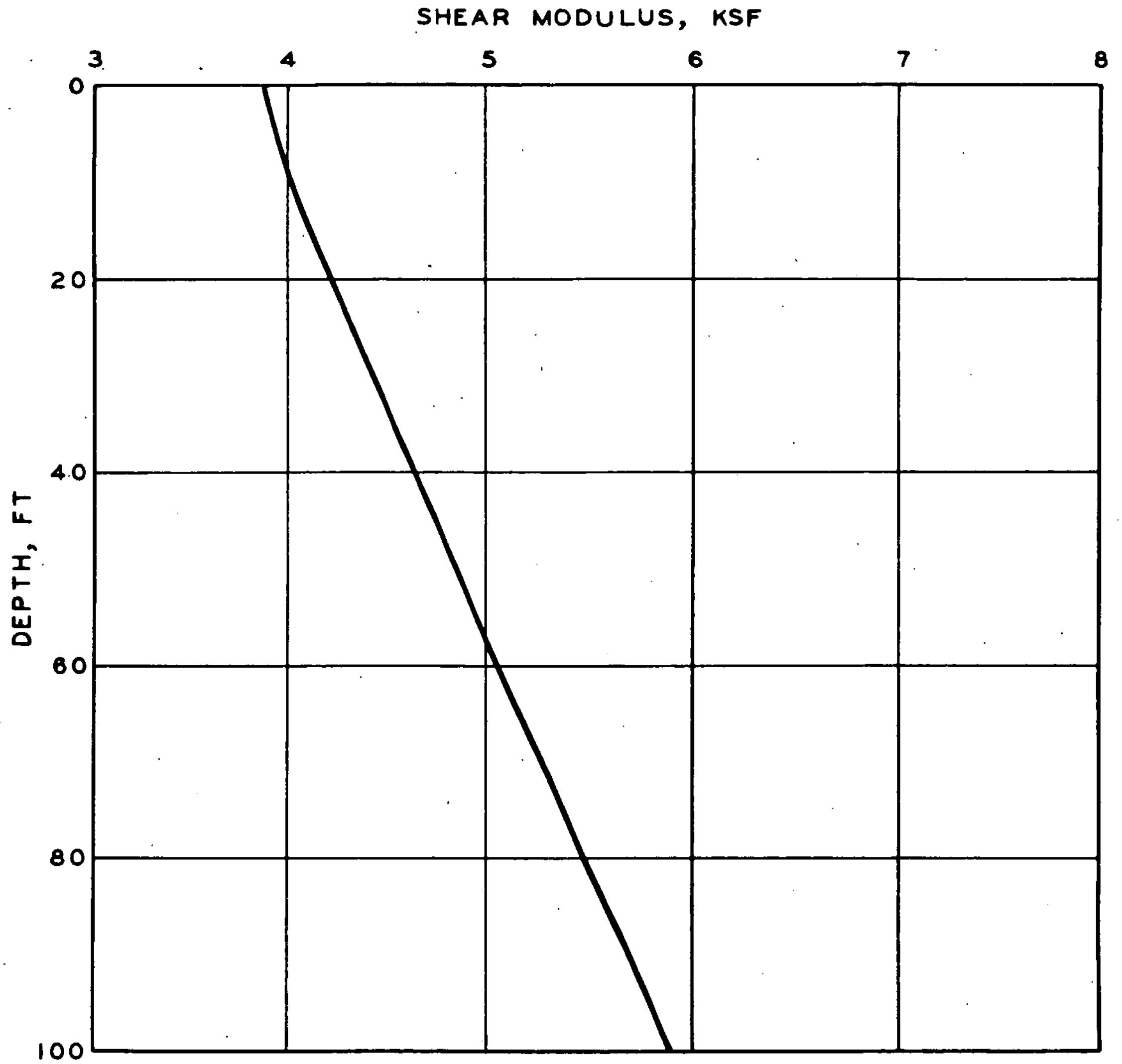
– – – RAYLEIGH DISPERSION DATA

////// RANGE FOR e = 0.9 AND e = 0.4, OCR^{K} = 1, USING MATERIAL PROPERTIES IN PLATE 5 — REF 3

——— (heavy line) RANGE FOR e = 0.9 AND e = 0.4, OCR^{K} = 1, USING MATERIAL PROPERTIES IN PLATE 5 — REF 3

FOUNDATION SHEAR MODULUS COMPARISONS

PLATE 8



SHEAR MODULUS VS
DEPTH OF FOUNDATION
AT CENTER LINE OF DAM

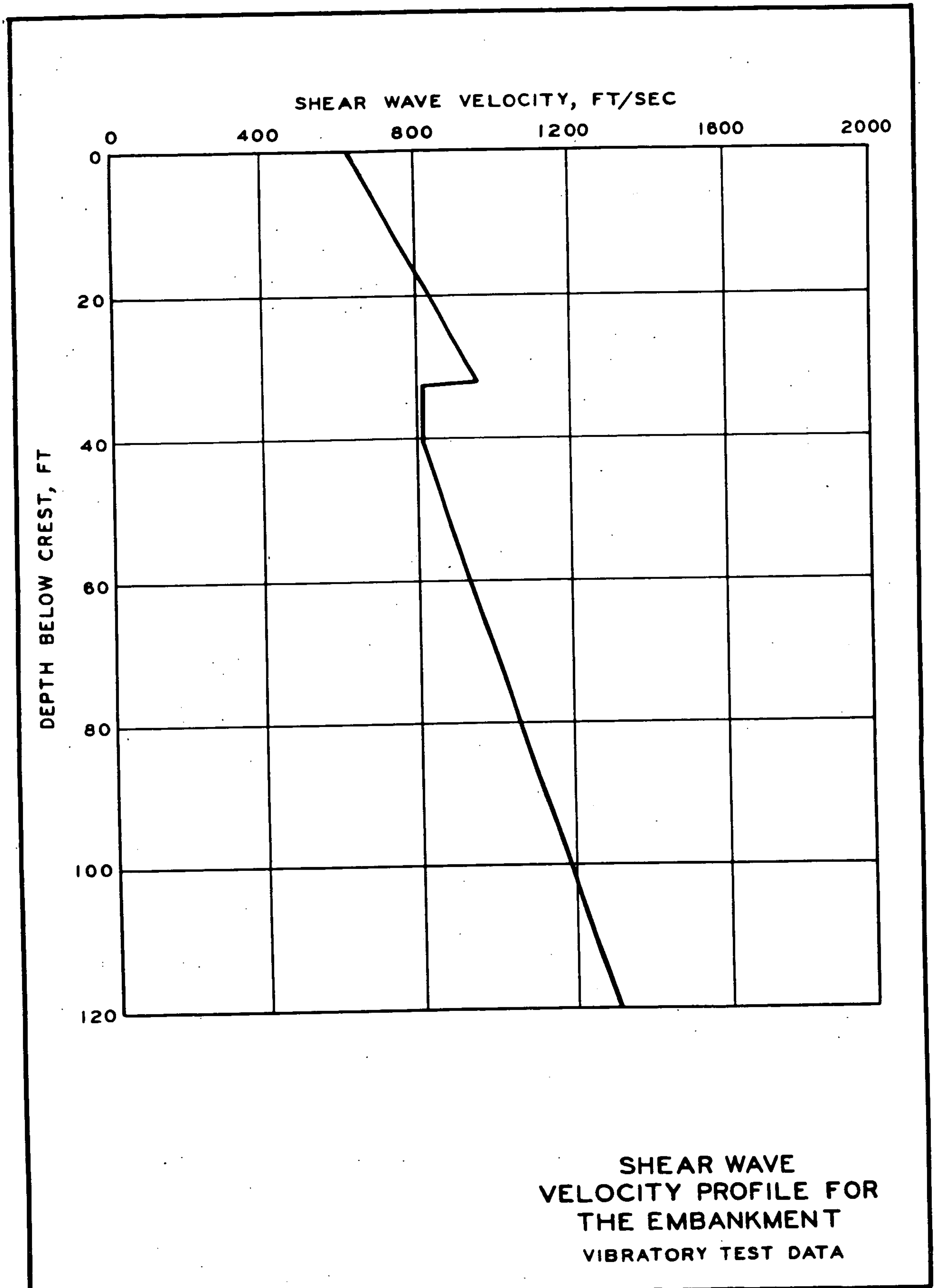
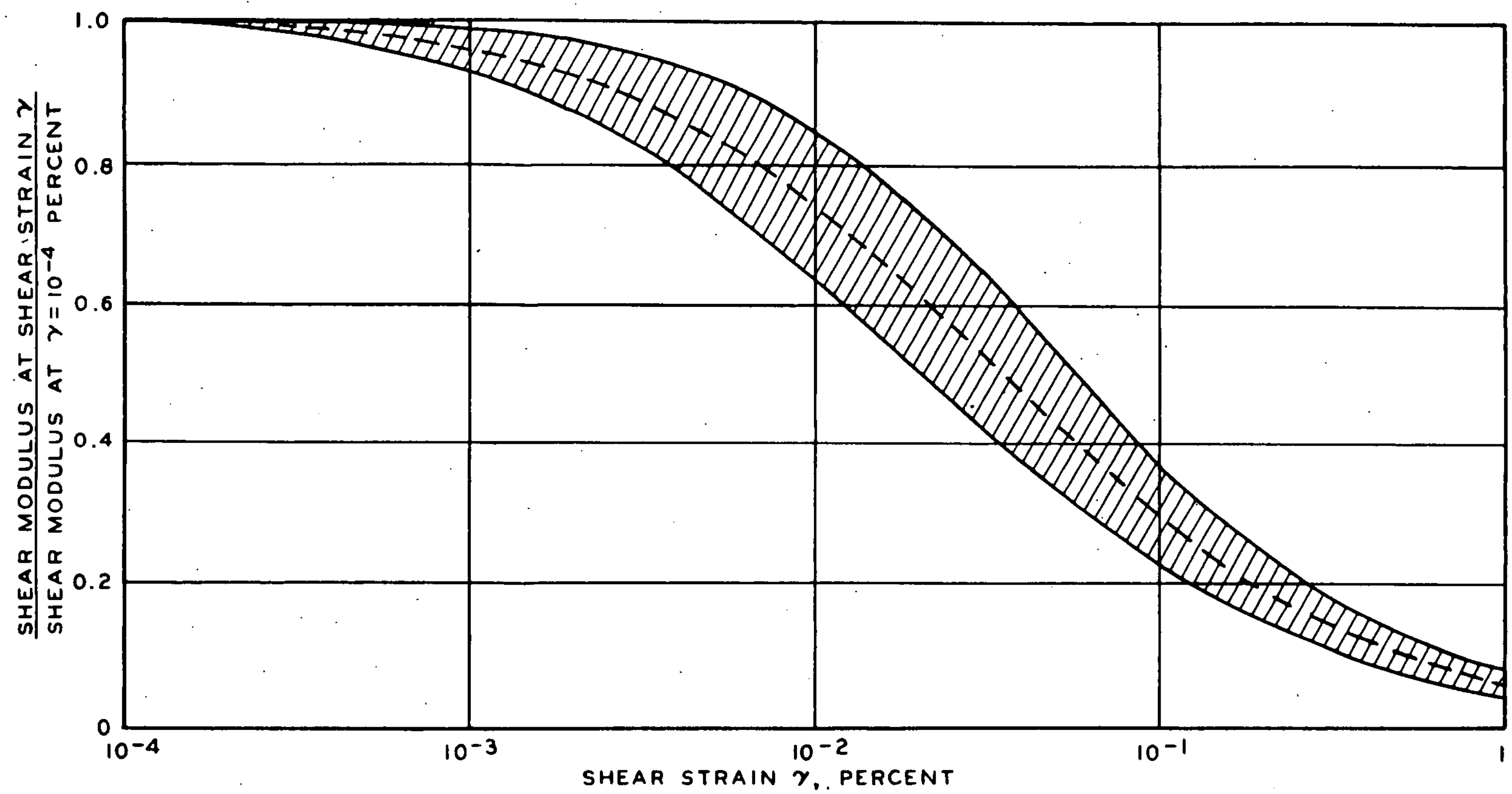
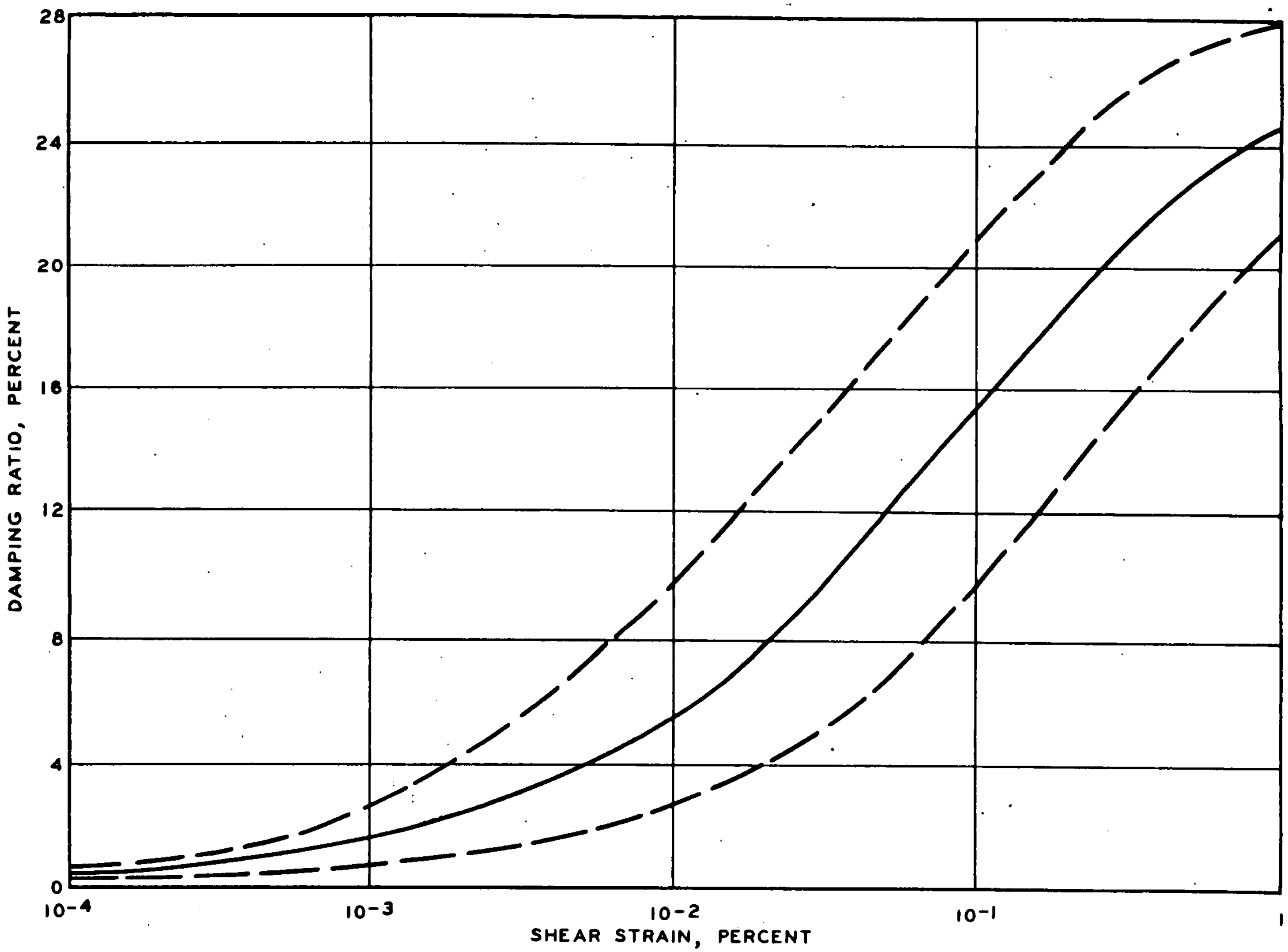

SHEAR WAVE VELOCITY, FT/SEC
0
400
800
1200
1600
2000
DEPTH BELOW CREST, FT
0
20
40
60
80
100
120
SHEAR WAVE
VELOCITY PROFILE FOR
THE EMBANKMENT
VIBRATORY TEST DATA

PLATE 10



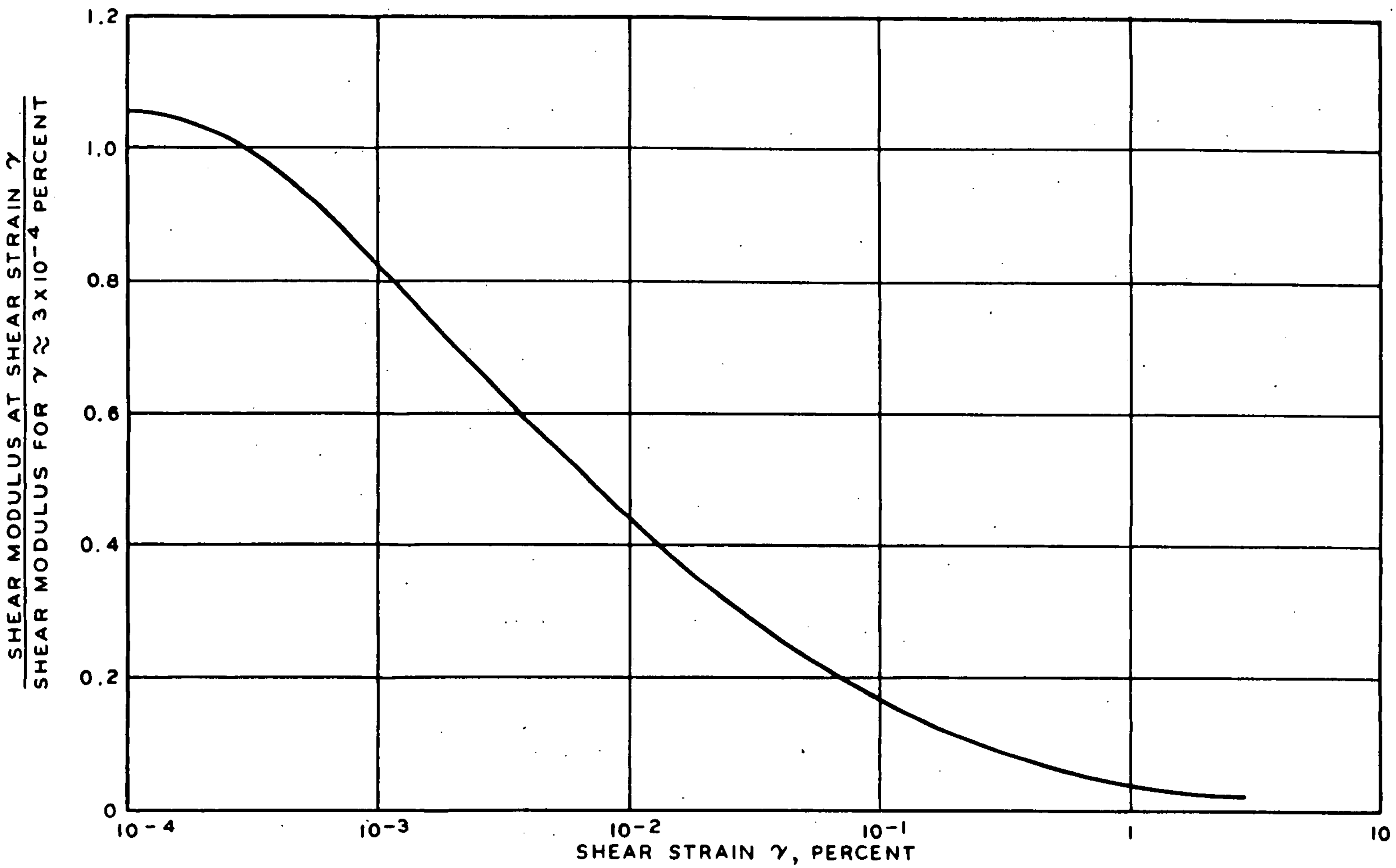
VARIATION OF SHEAR MODULUS
WITH SHEAR STRAIN FOR SANDS
AFTER REF 4

PLATE 11



DAMPING RATIOS FOR SANDS
AFTER REF 4

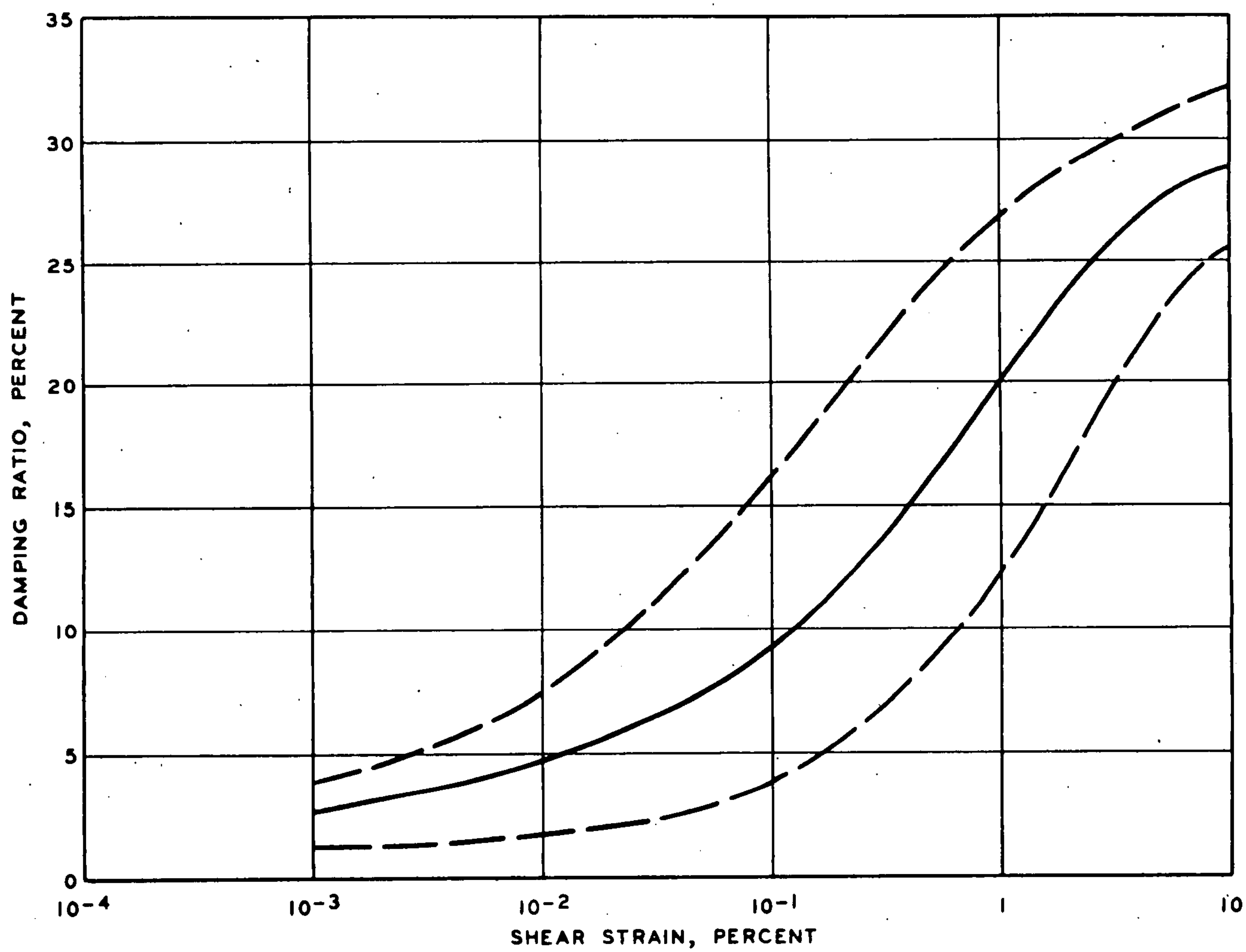
PLATE 12



TYPICAL REDUCTION OF SHEAR MODULUS WITH SHEAR STRAIN FOR SATURATED CLAYS

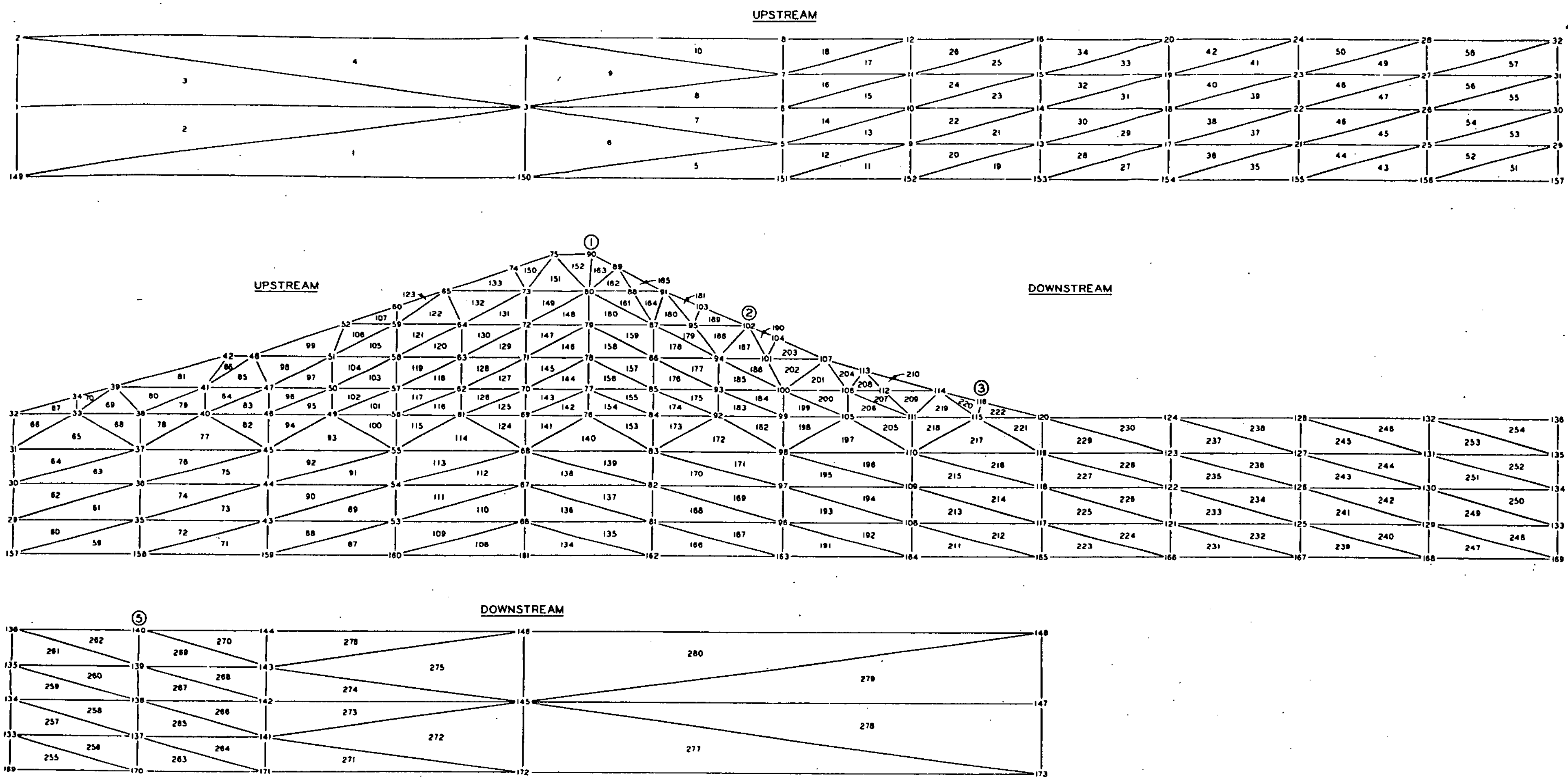
AFTER REF 4

PLATE 14

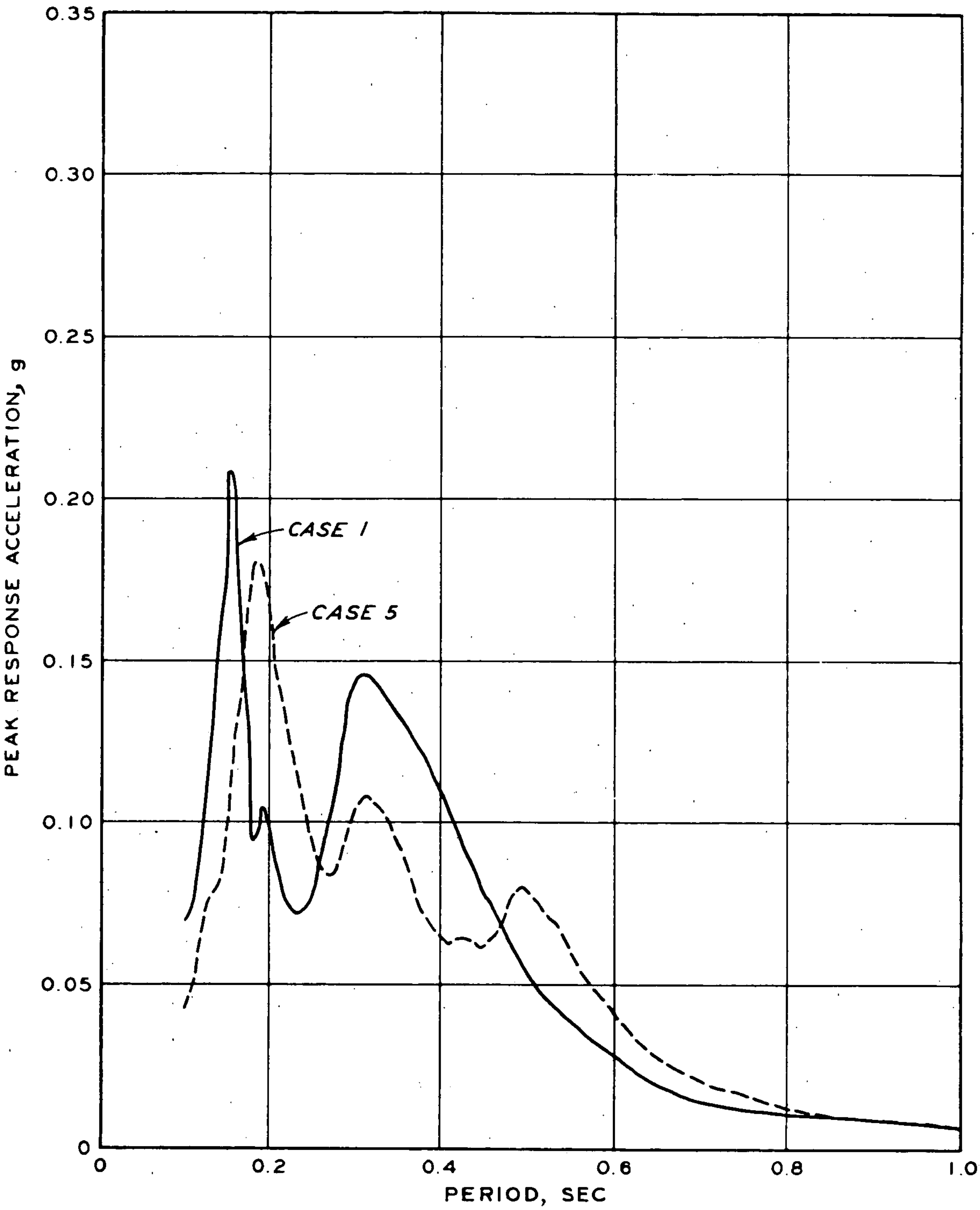


DAMPING RATIOS FOR SATURATED CLAYS

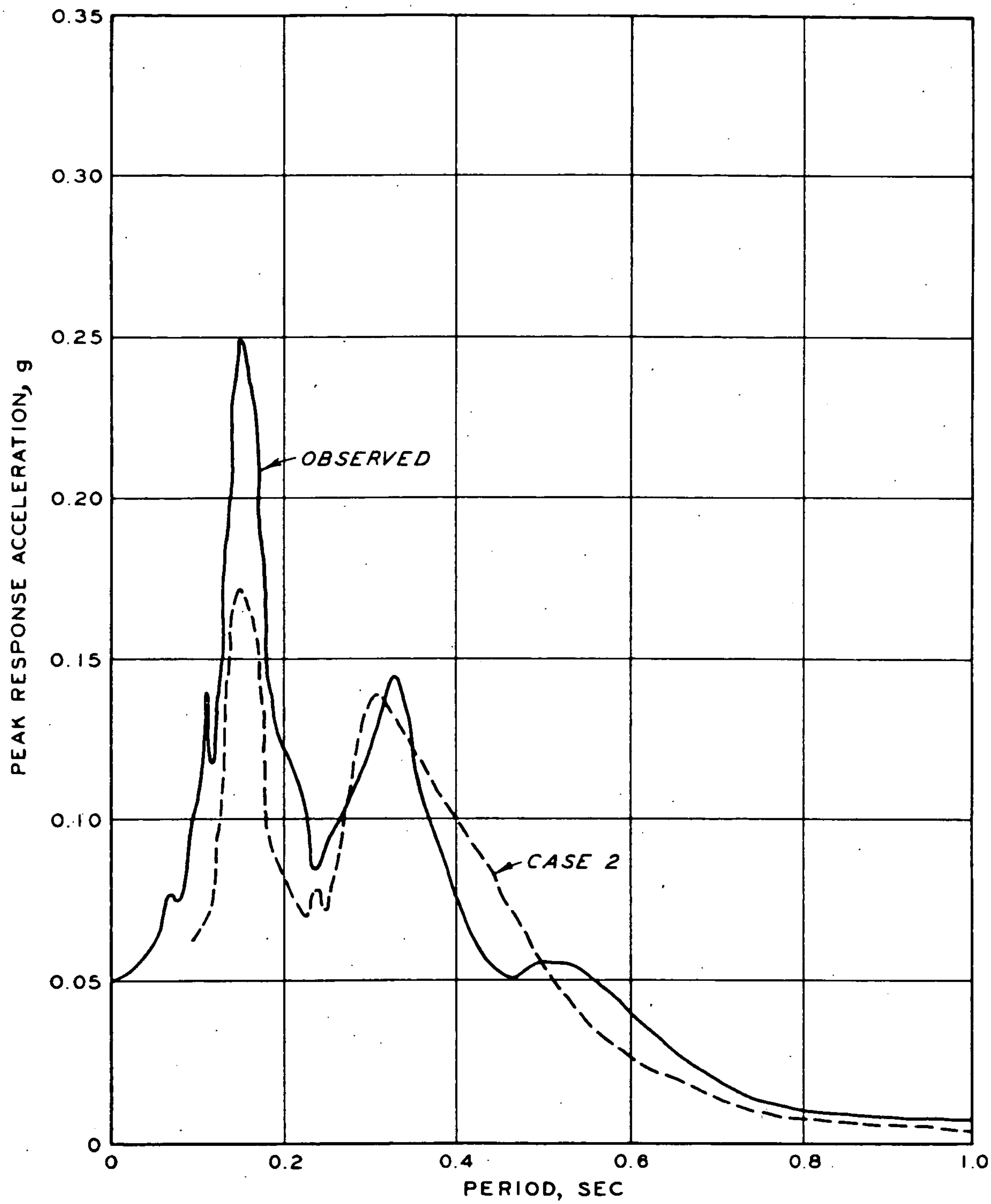
AFTER REF 4



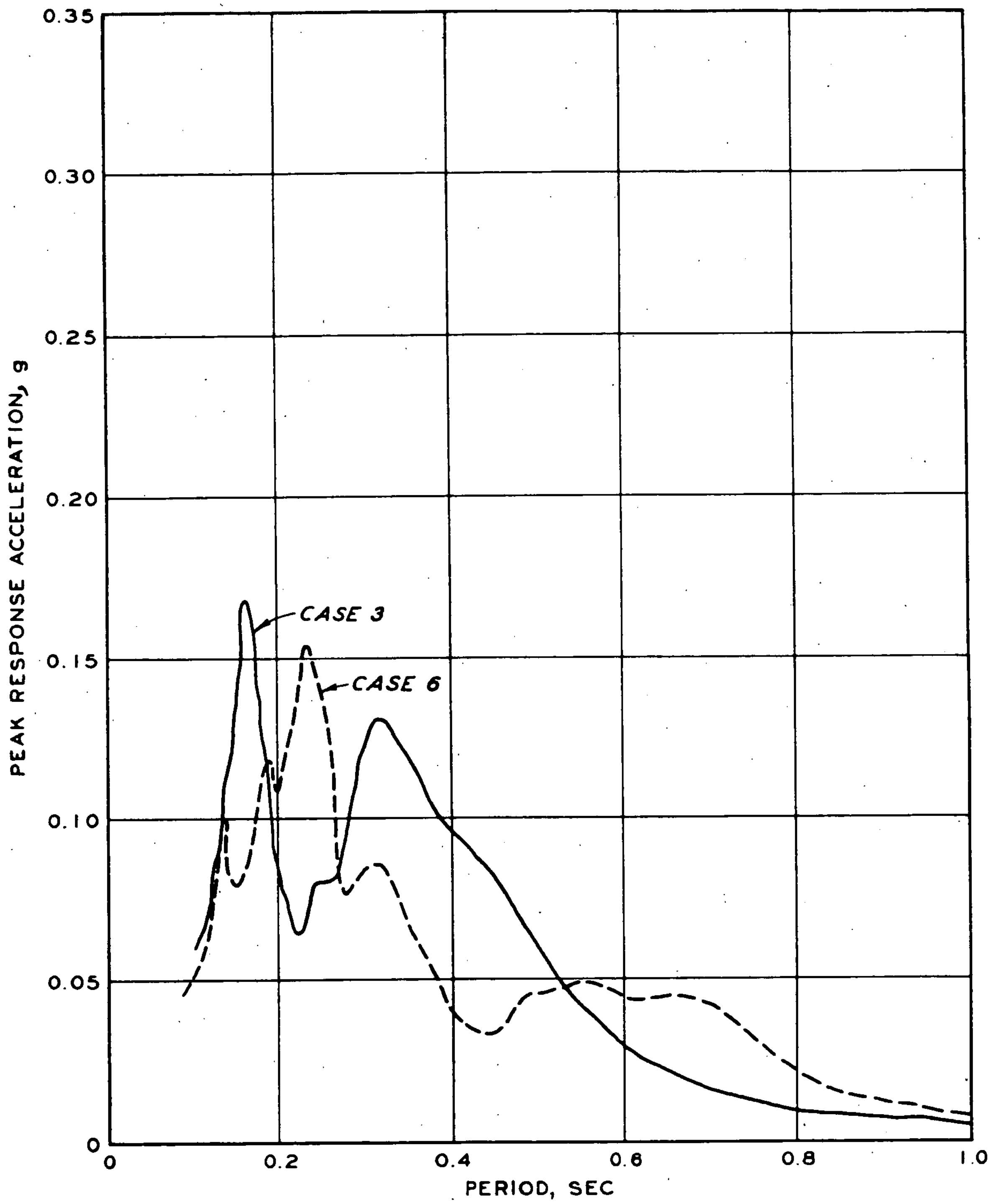
FINITE ELEMENT MESH FOR
2D ANALYSIS AND LOCATIONS OF
MEASUREMENT STATIONS



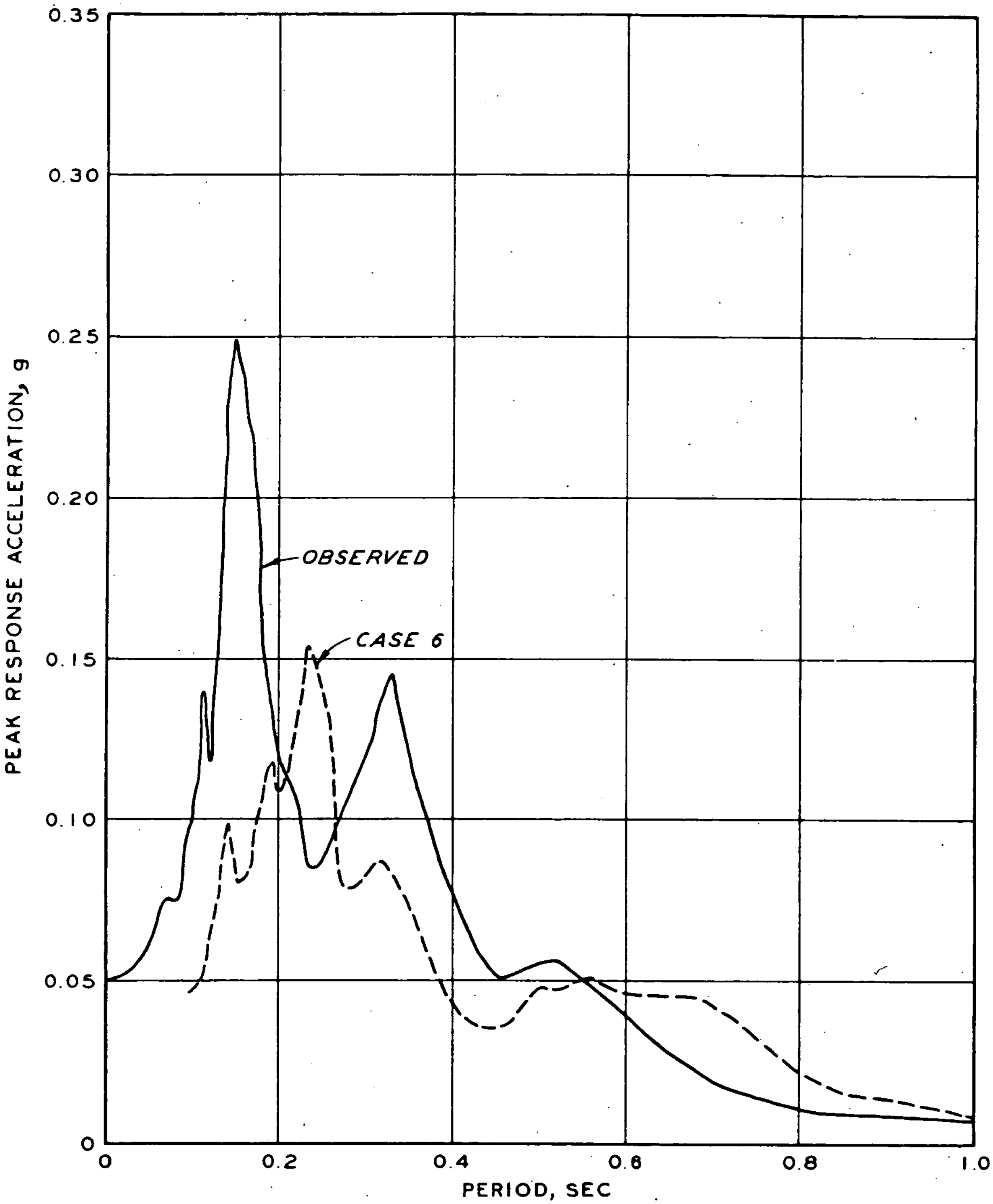
RESPONSE SPECTRA FOR
1D LUMPED-MASS ANALYSIS
CASES 1 AND 5



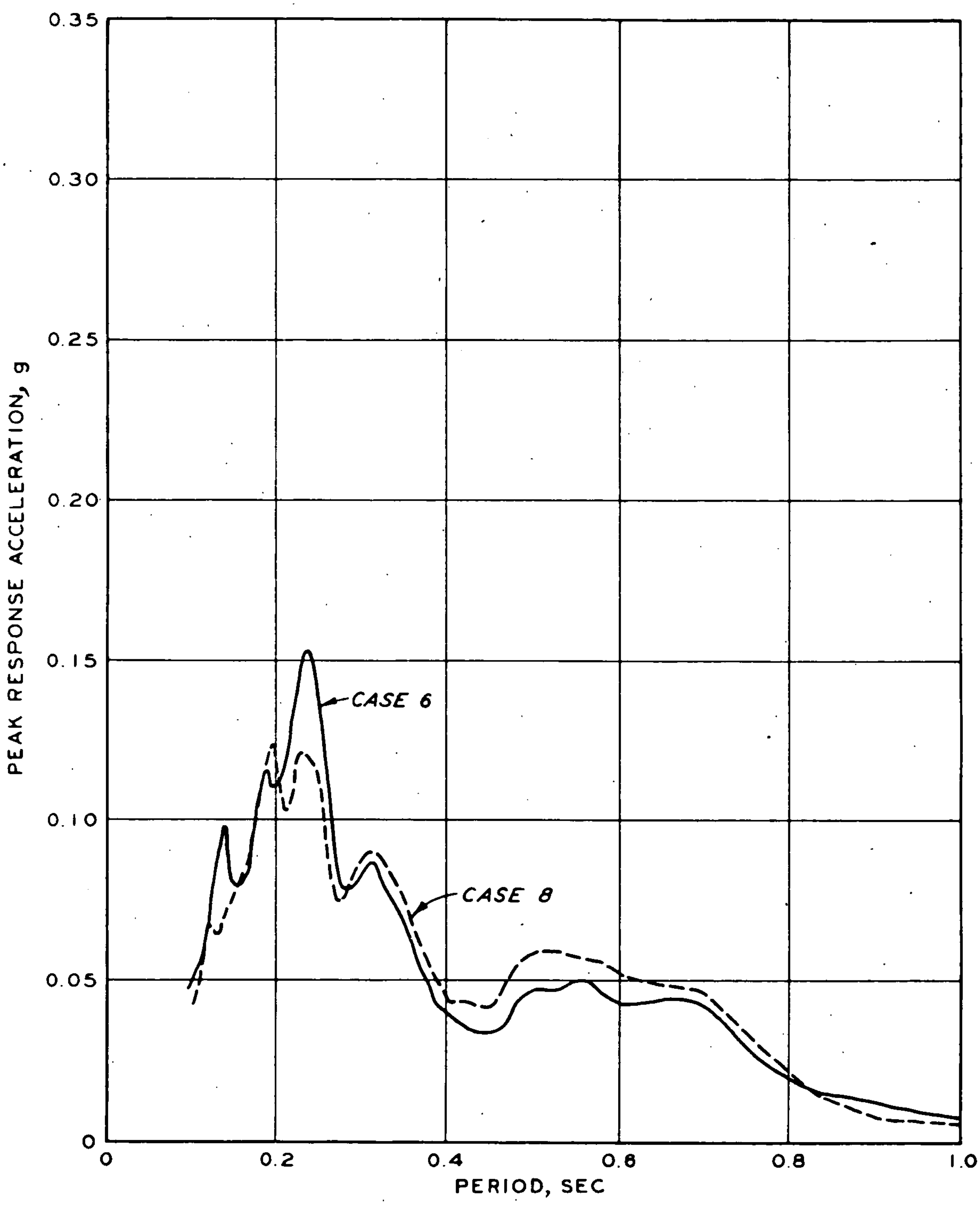
OBSERVED AND COMPUTED
I D RESPONSE SPECTRA
CASE 2



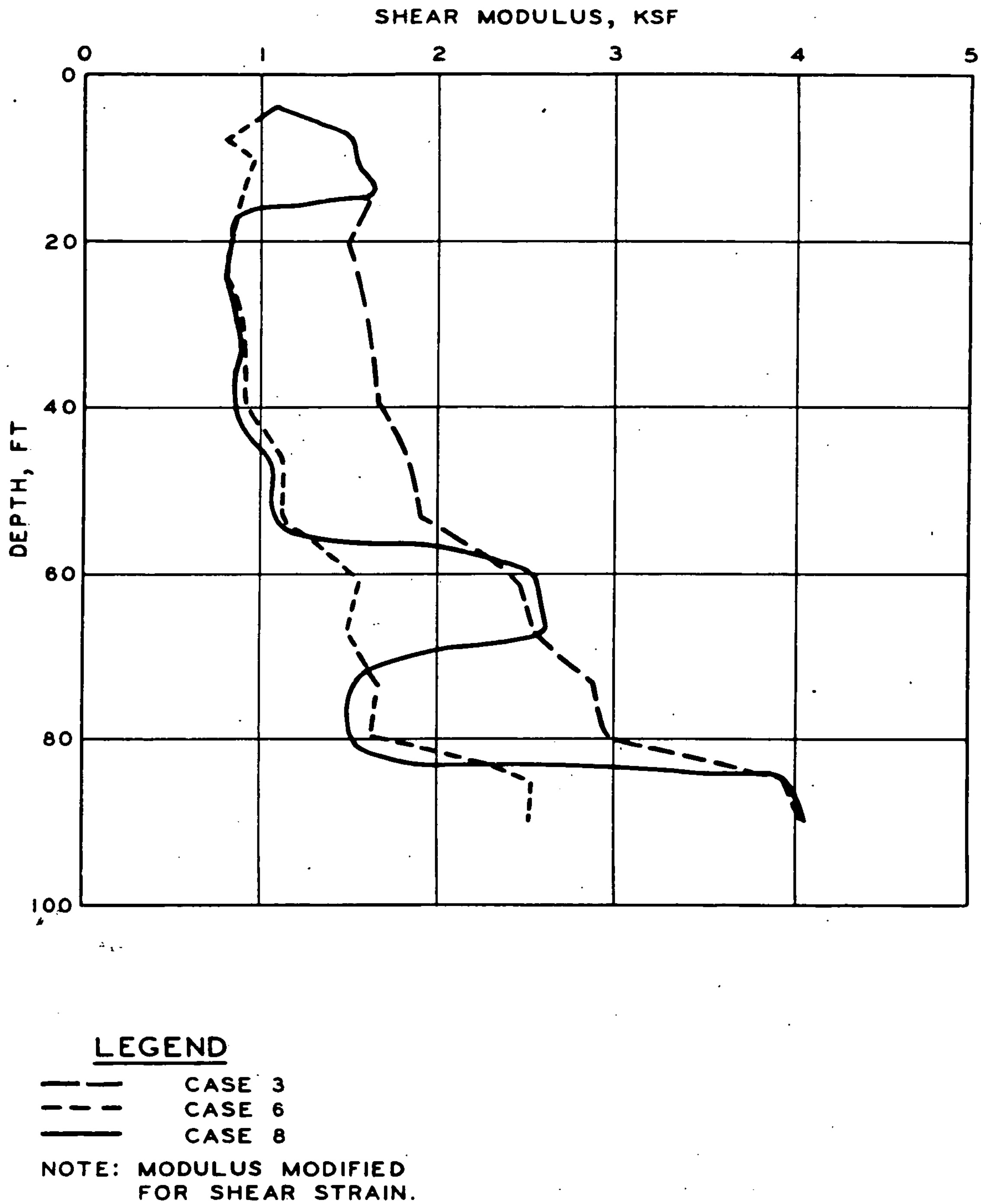
COMPUTED 1D
RESPONSE SPECTRA
CASES 3 AND 6



OBSERVED AND COMPUTED
1 D RESPONSE SPECTRA
CASE 6

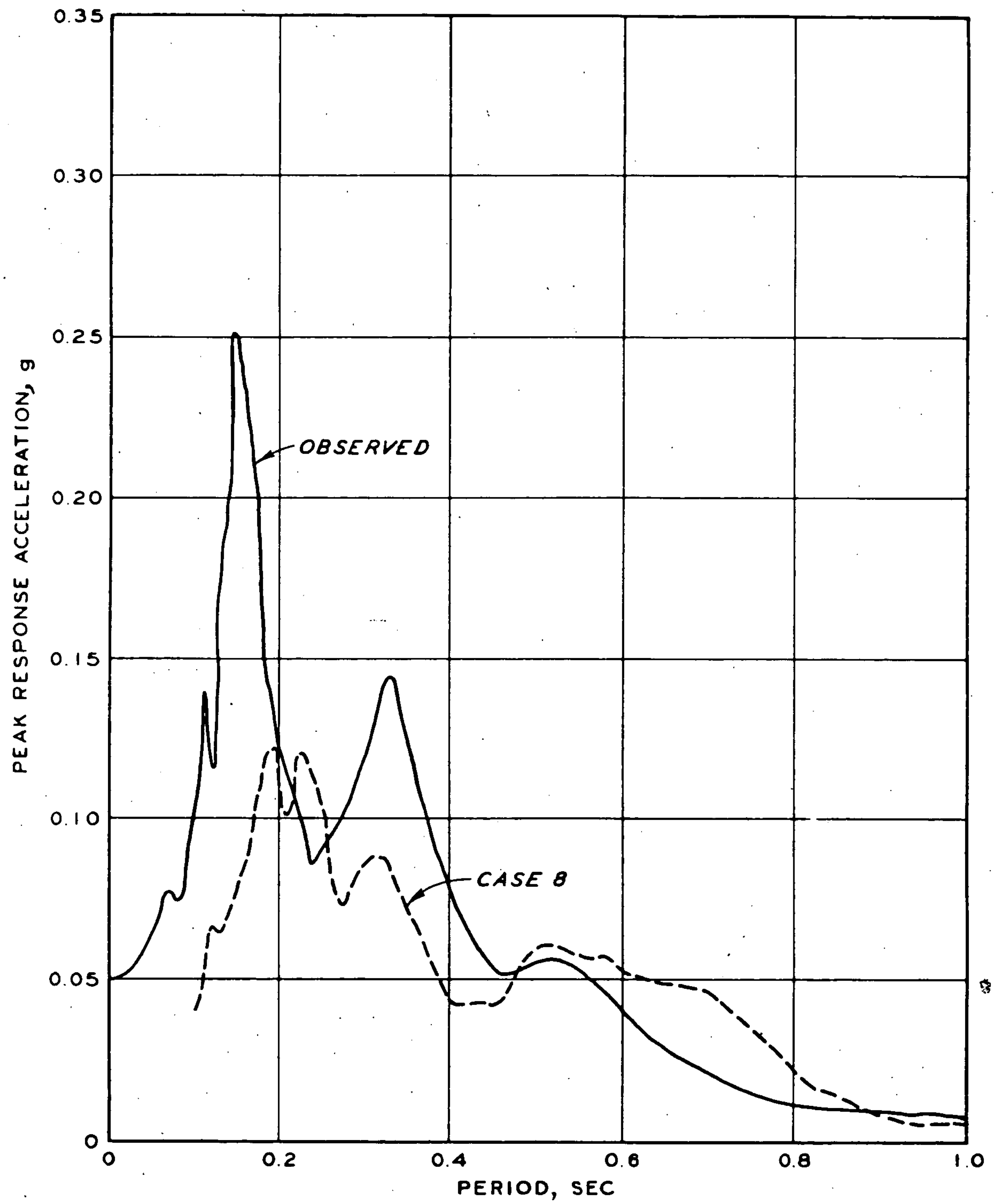


COMPUTED 1D
RESPONSE SPECTRA
CASES 6 AND 8

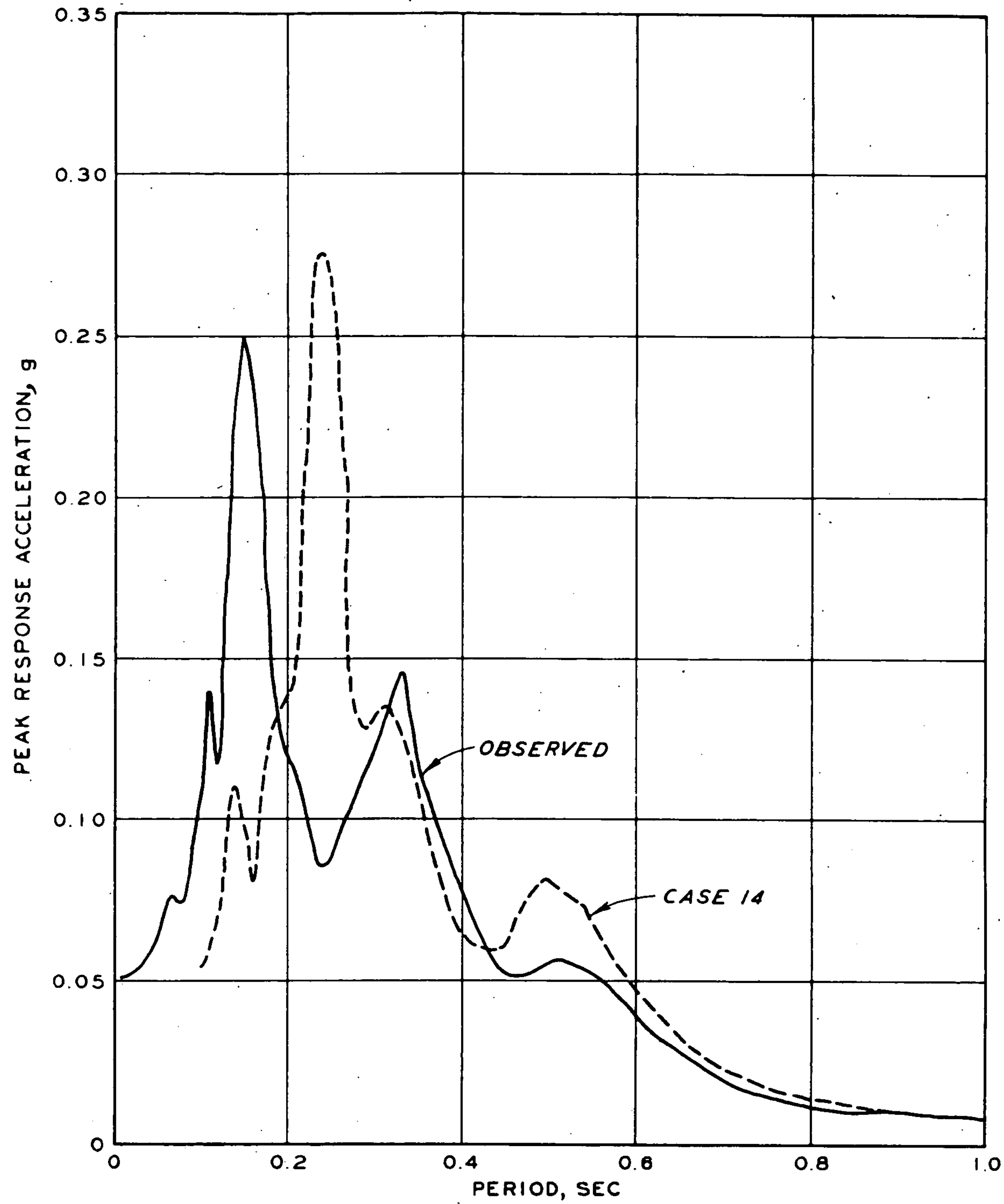


SHEAR MODULUS PROFILE
MODIFIED FOR SHEAR STRAIN

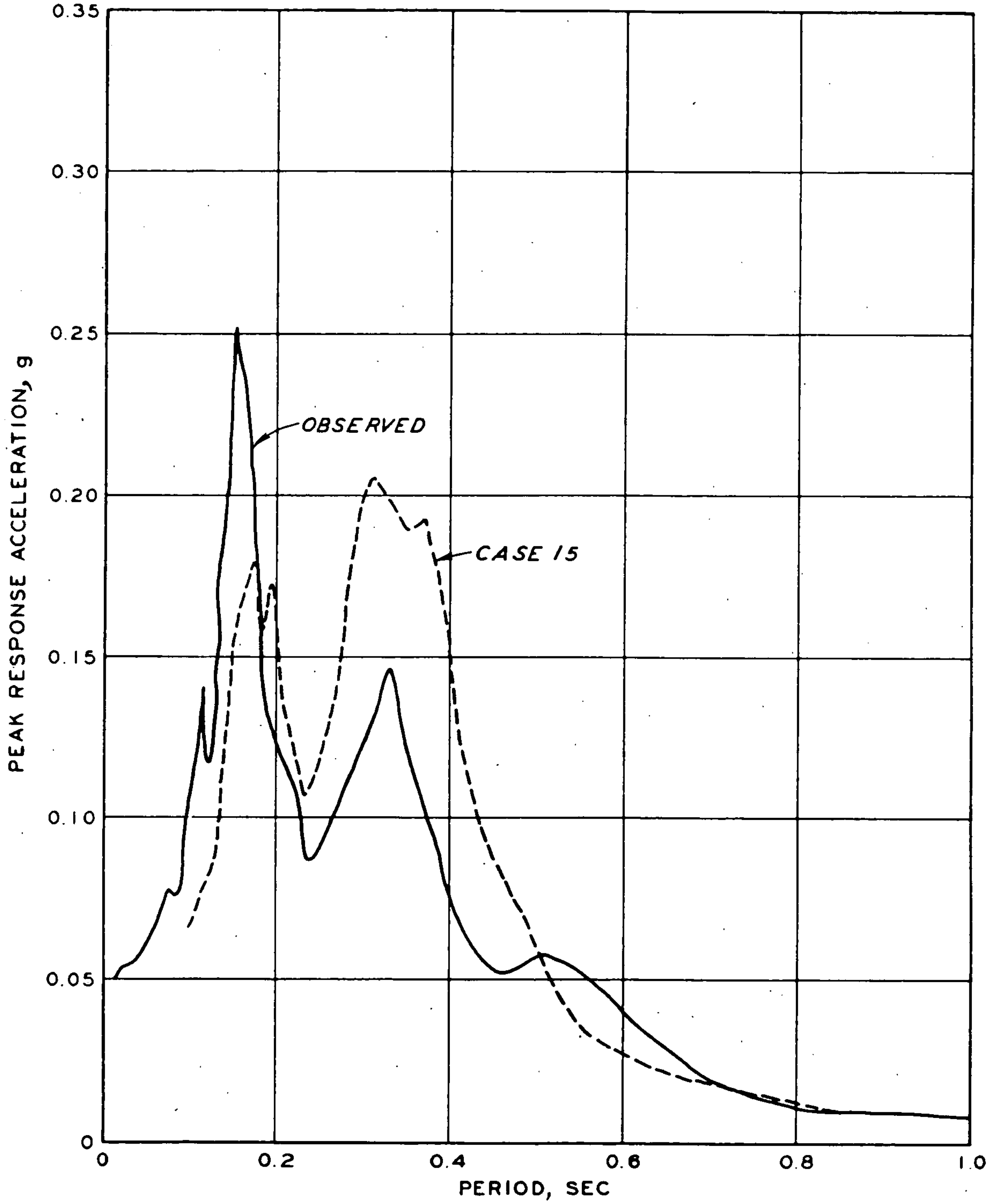
PLATE 21



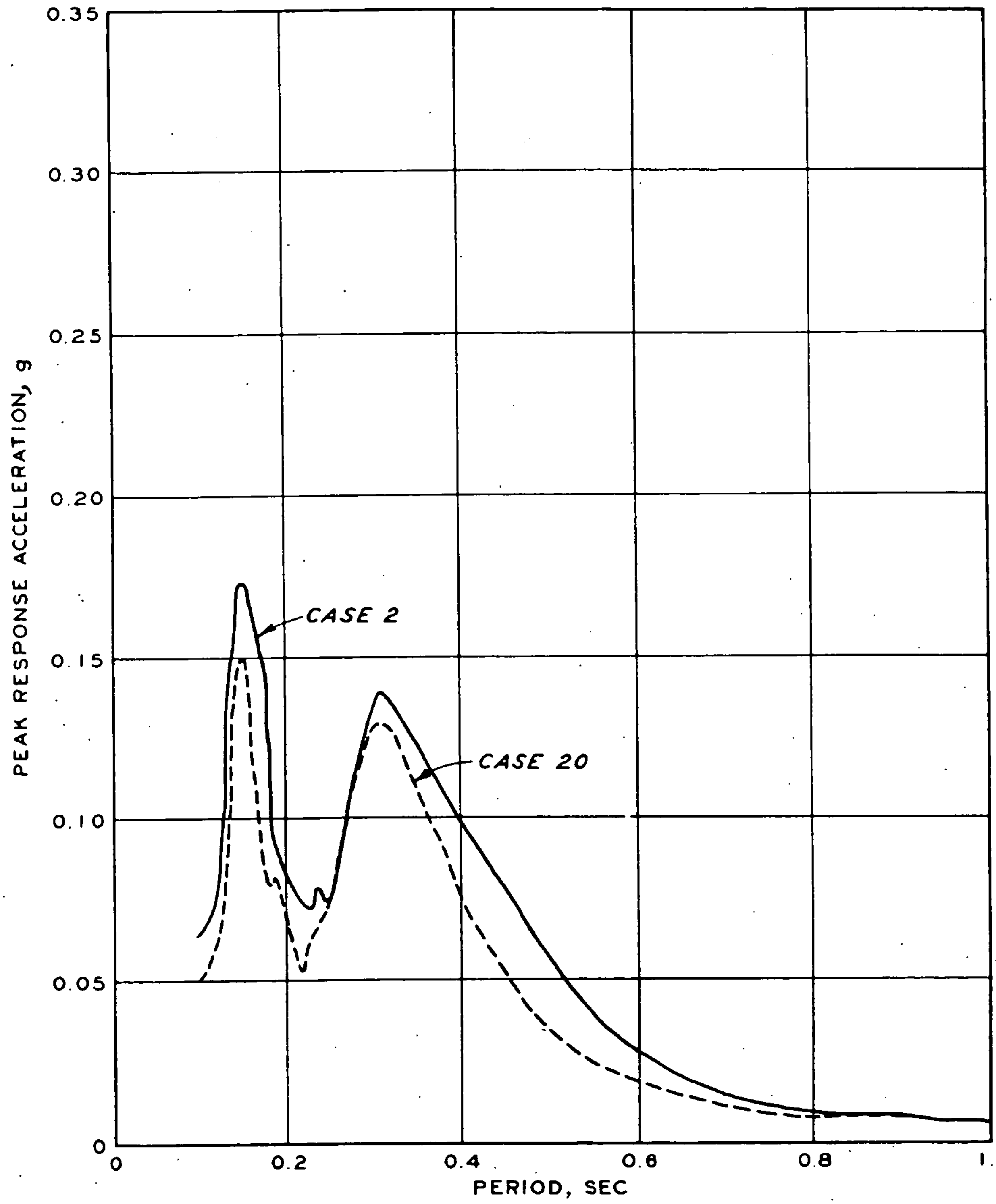
OBSERVED AND COMPUTED
1D RESPONSE SPECTRA
CASE 8



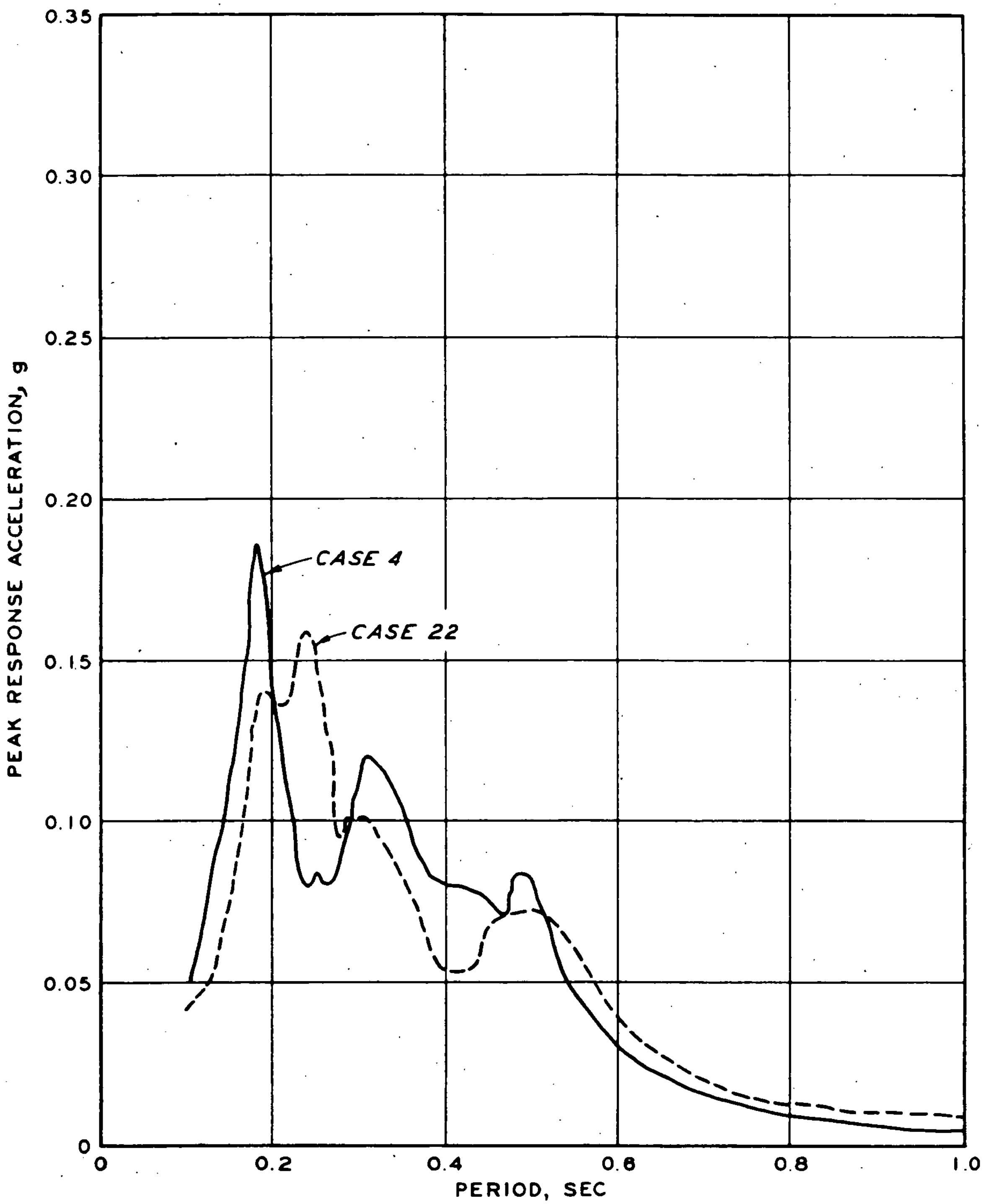
OBSERVED AND COMPUTED
1D RESPONSE SPECTRA
CASE 14



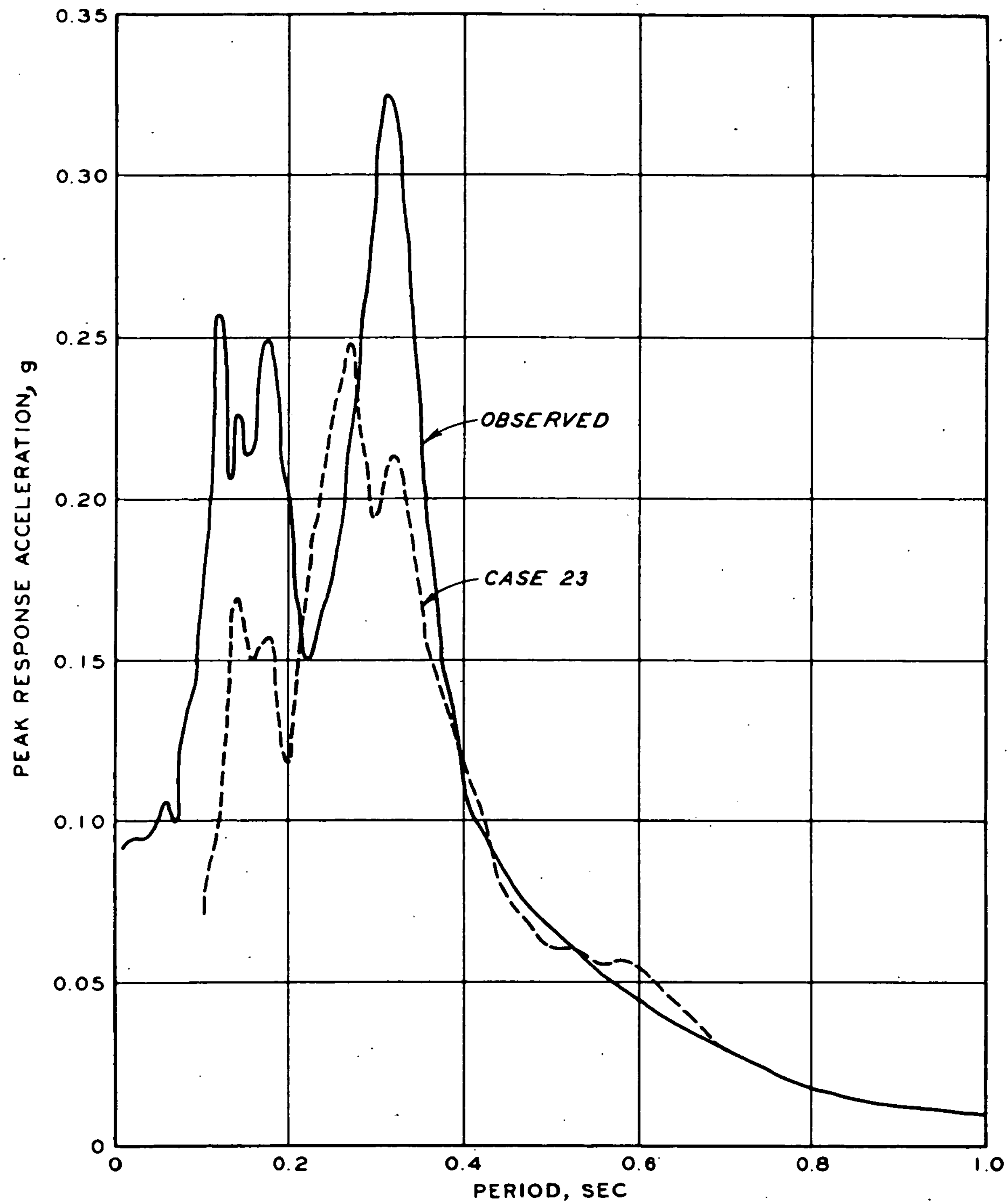
OBSERVED AND COMPUTED
1 D RESPONSE SPECTRA
CASE 15



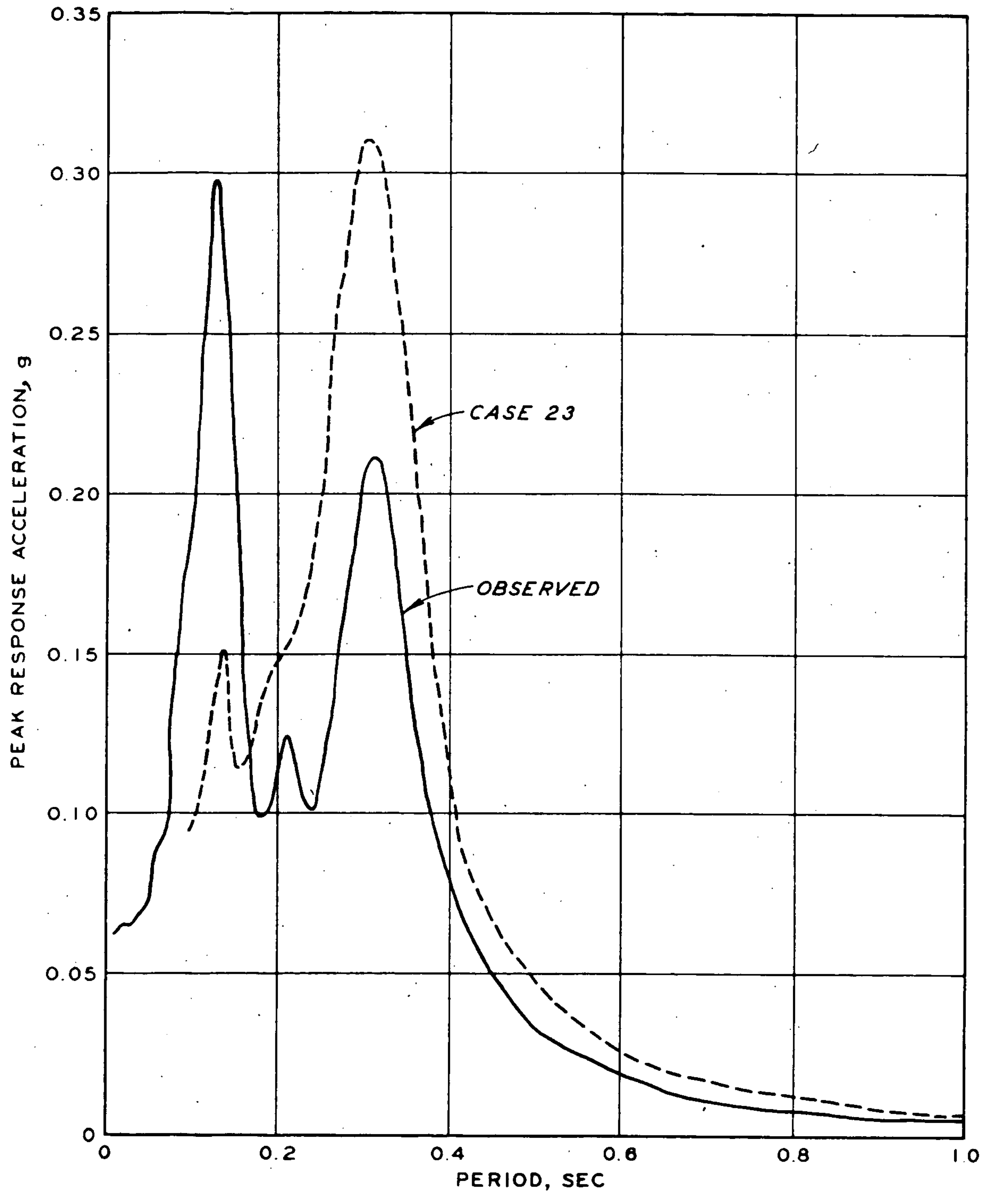
COMPUTED 1D
RESPONSE SPECTRA
CASES 2 AND 20



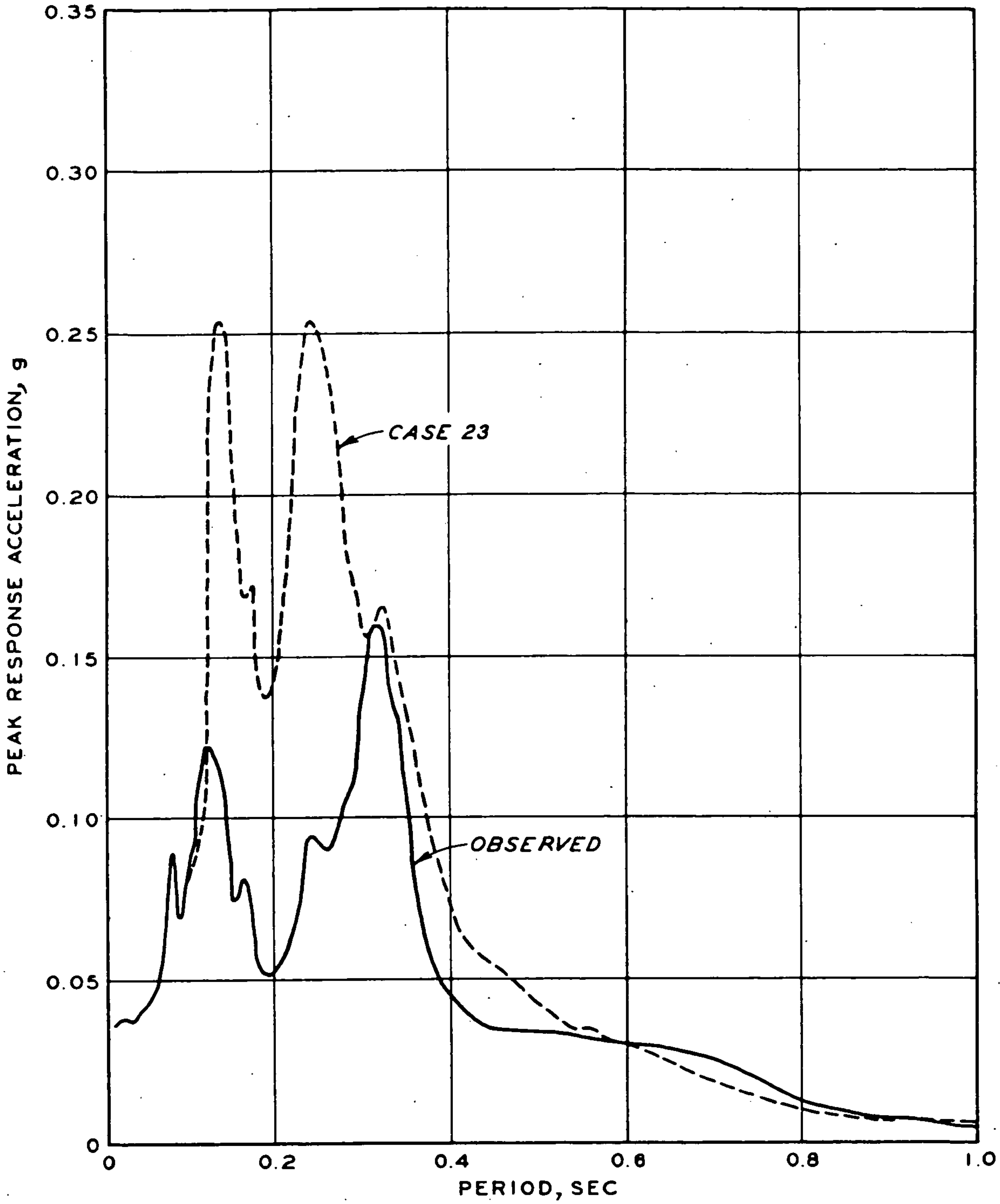
COMPARISON OF 1D AND
2 D RESPONSE SPECTRA
LOCATION 5
RADIAL COMPONENT
CASES 4 AND 22



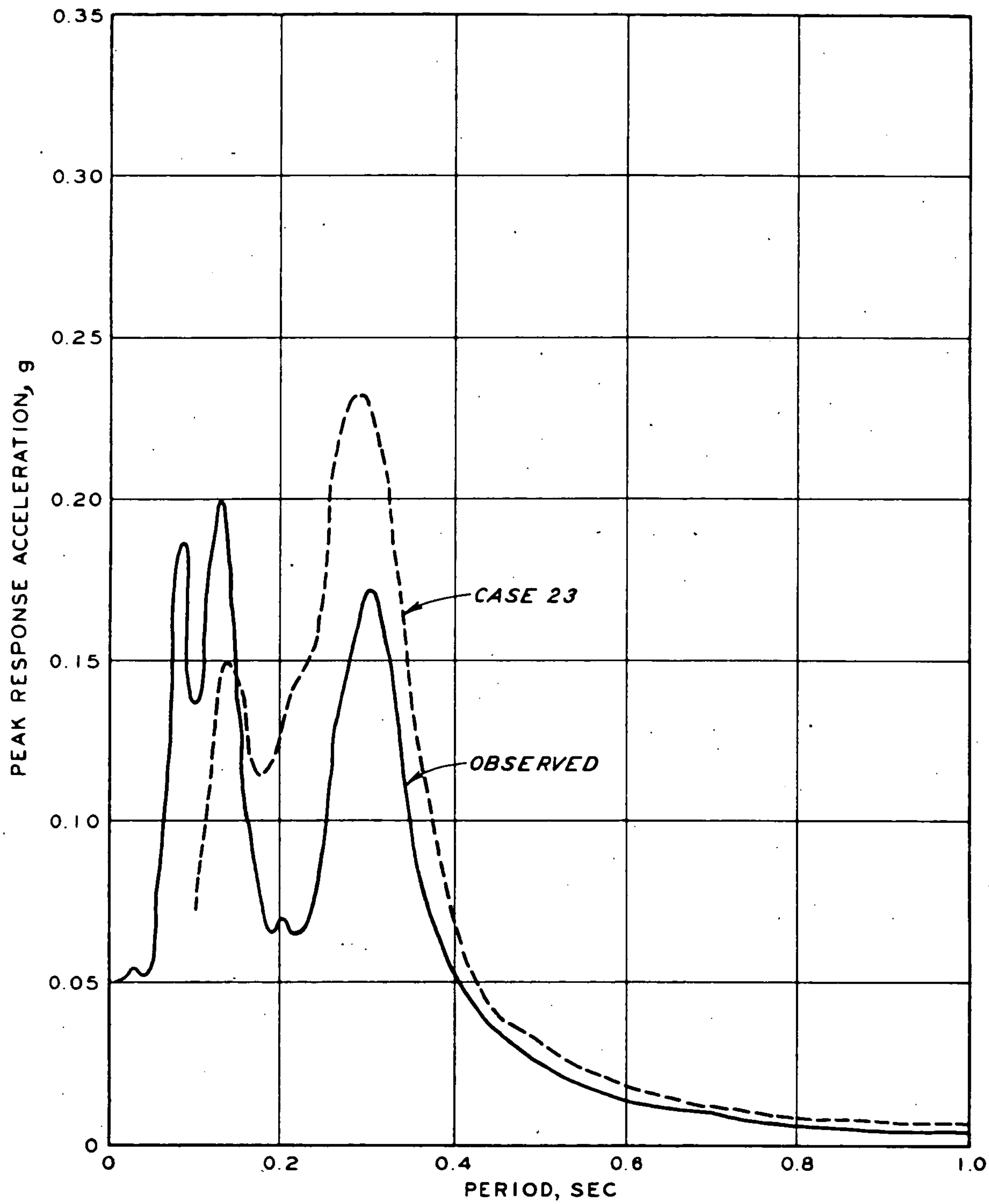
OBSERVED AND COMPUTED
2 D RESPONSE SPECTRA
LOCATION 1
RADIAL COMPONENT
CASE 23



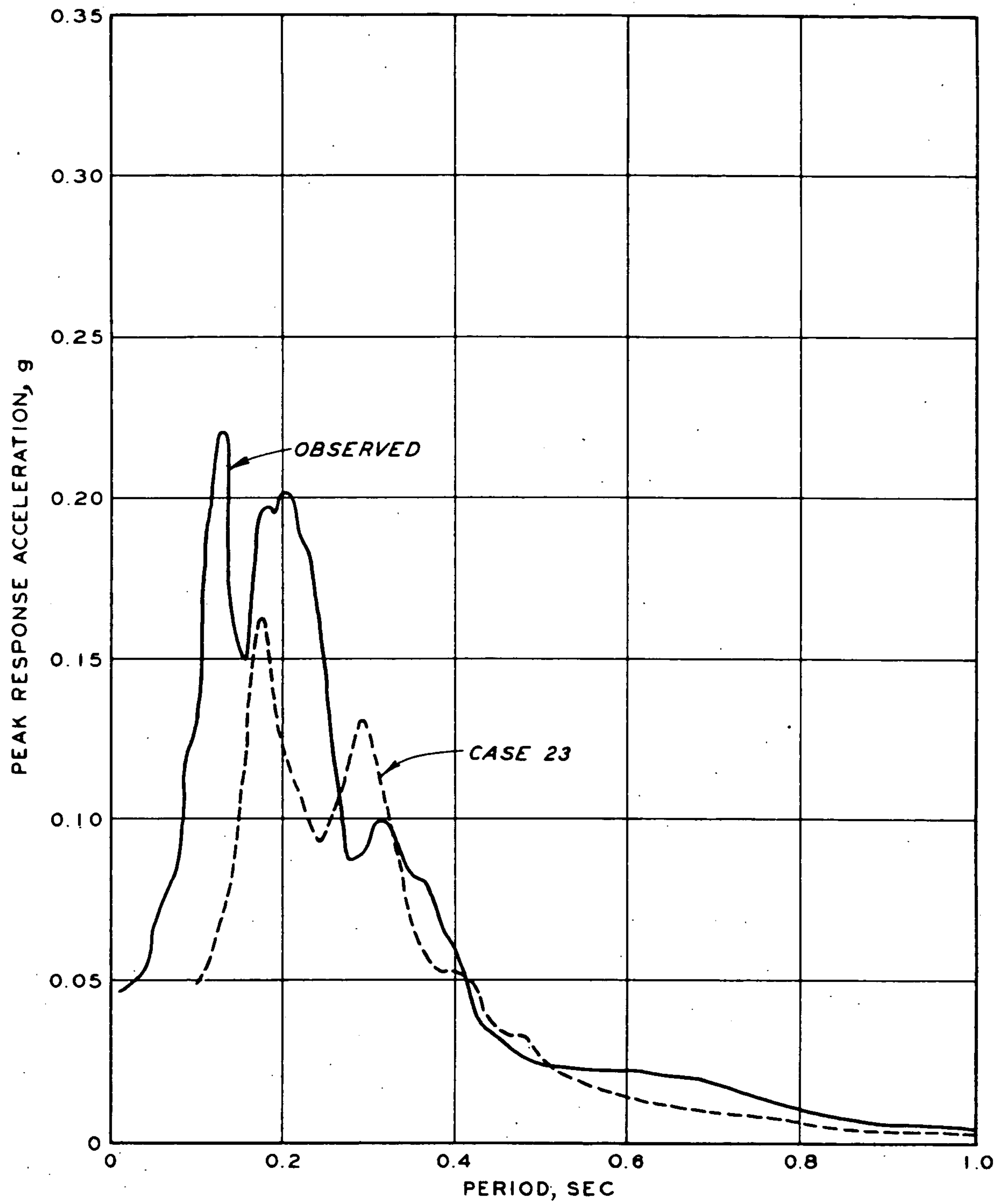
OBSERVED AND COMPUTED
2 D RESPONSE SPECTRA
LOCATION 1
VERTICAL COMPONENT
CASE 23



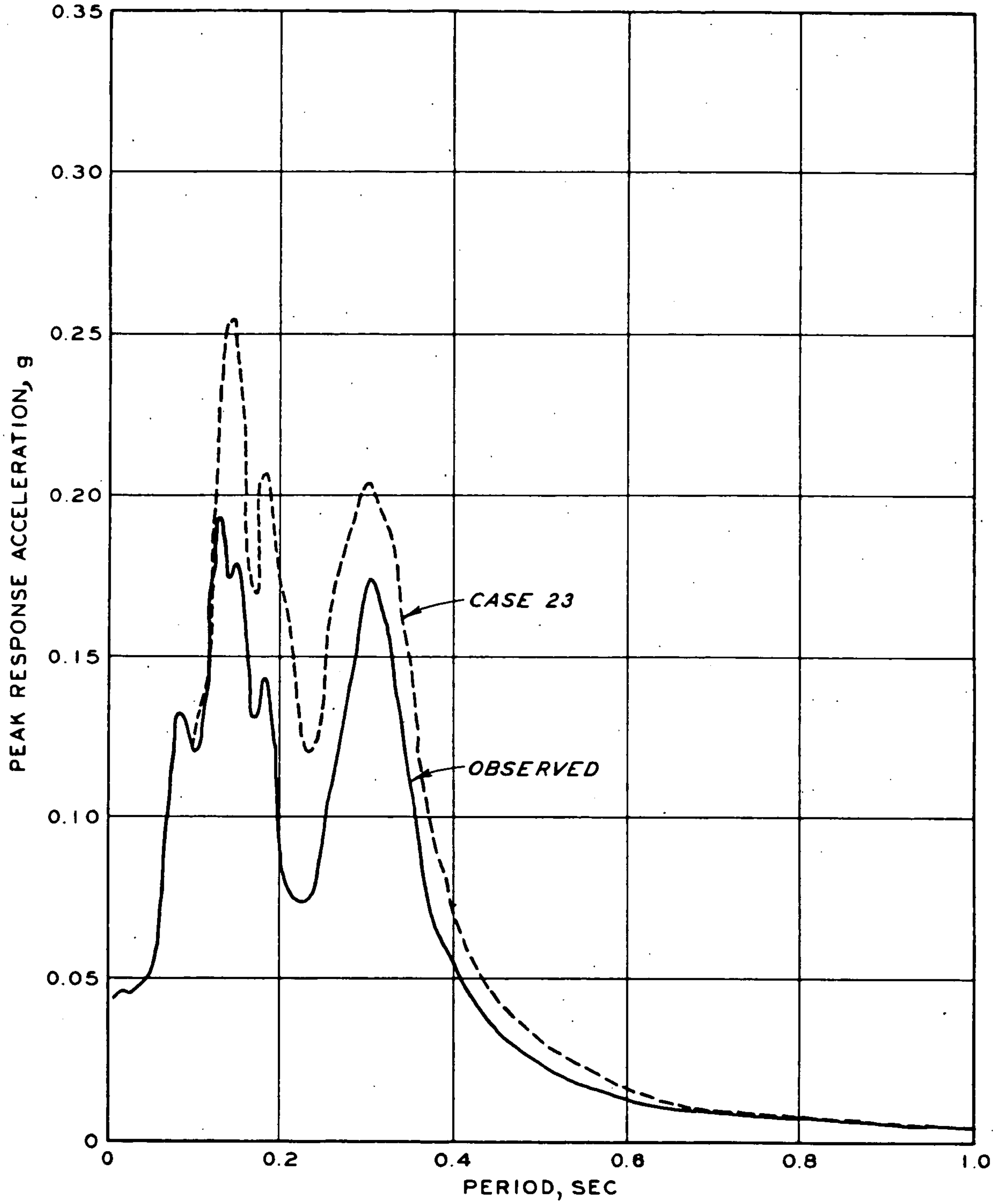
OBSERVED AND COMPUTED
2 D RESPONSE SPECTRA
LOCATION 2
RADIAL COMPONENT
CASE 23



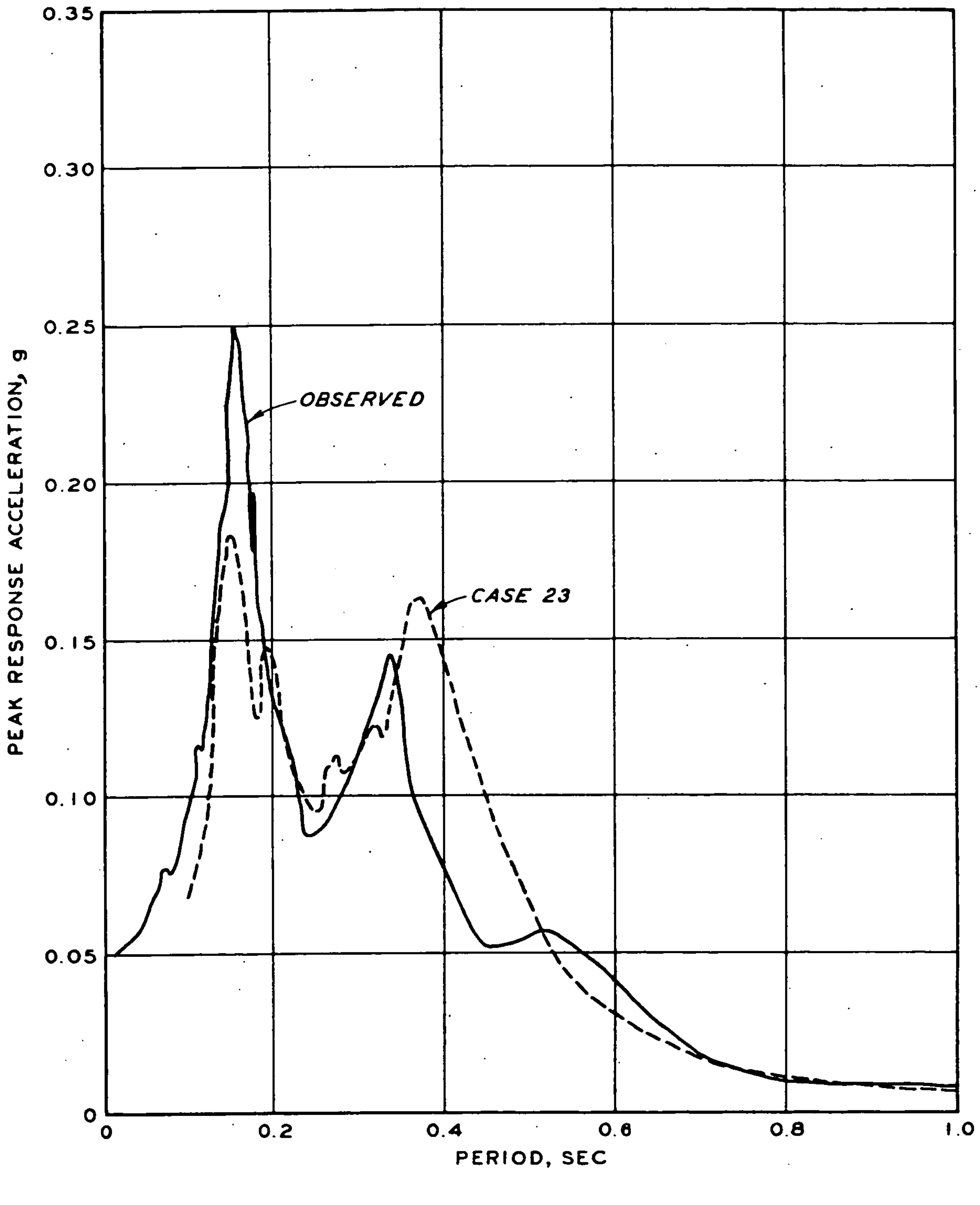
OBSERVED AND COMPUTED
2 D RESPONSE SPECTRA
LOCATION 2
VERTICAL COMPONENT
CASE 23



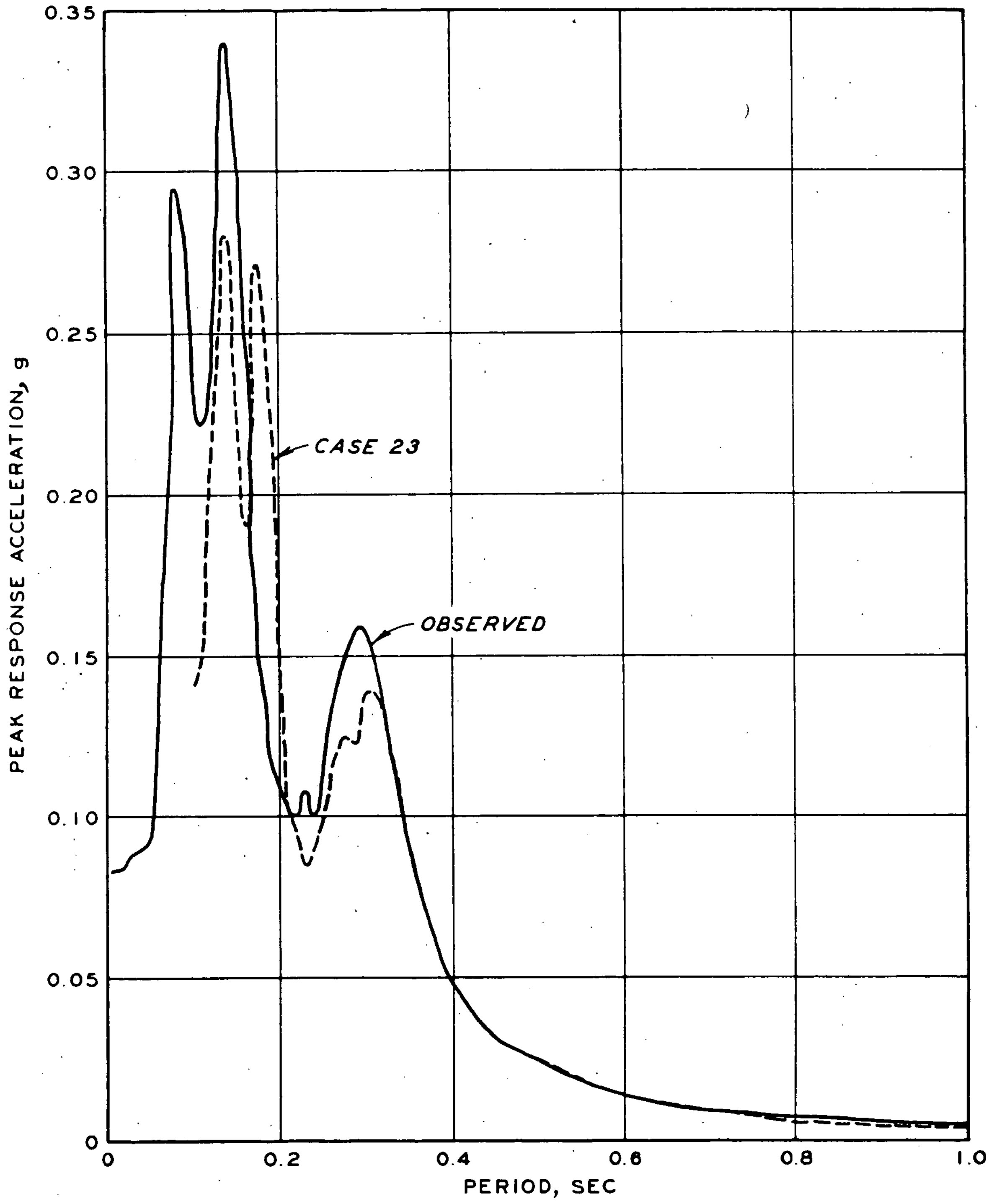
OBSERVED AND COMPUTED
2 D RESPONSE SPECTRA
LOCATION 3
RADIAL COMPONENT.
CASE 23



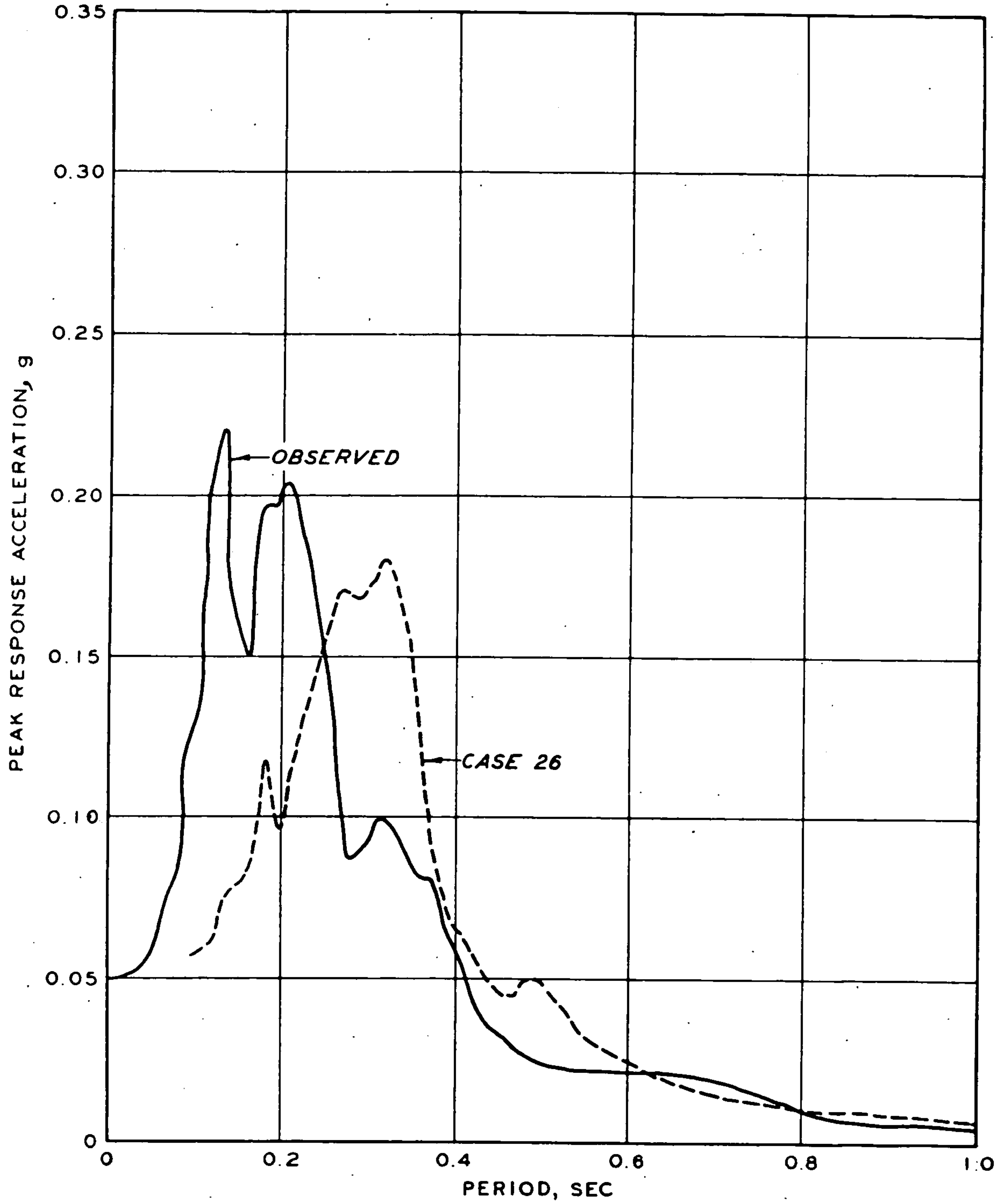
OBSERVED AND COMPUTED
2 D RESPONSE SPECTRA
LOCATION 3
VERTICAL COMPONENT
CASE 23



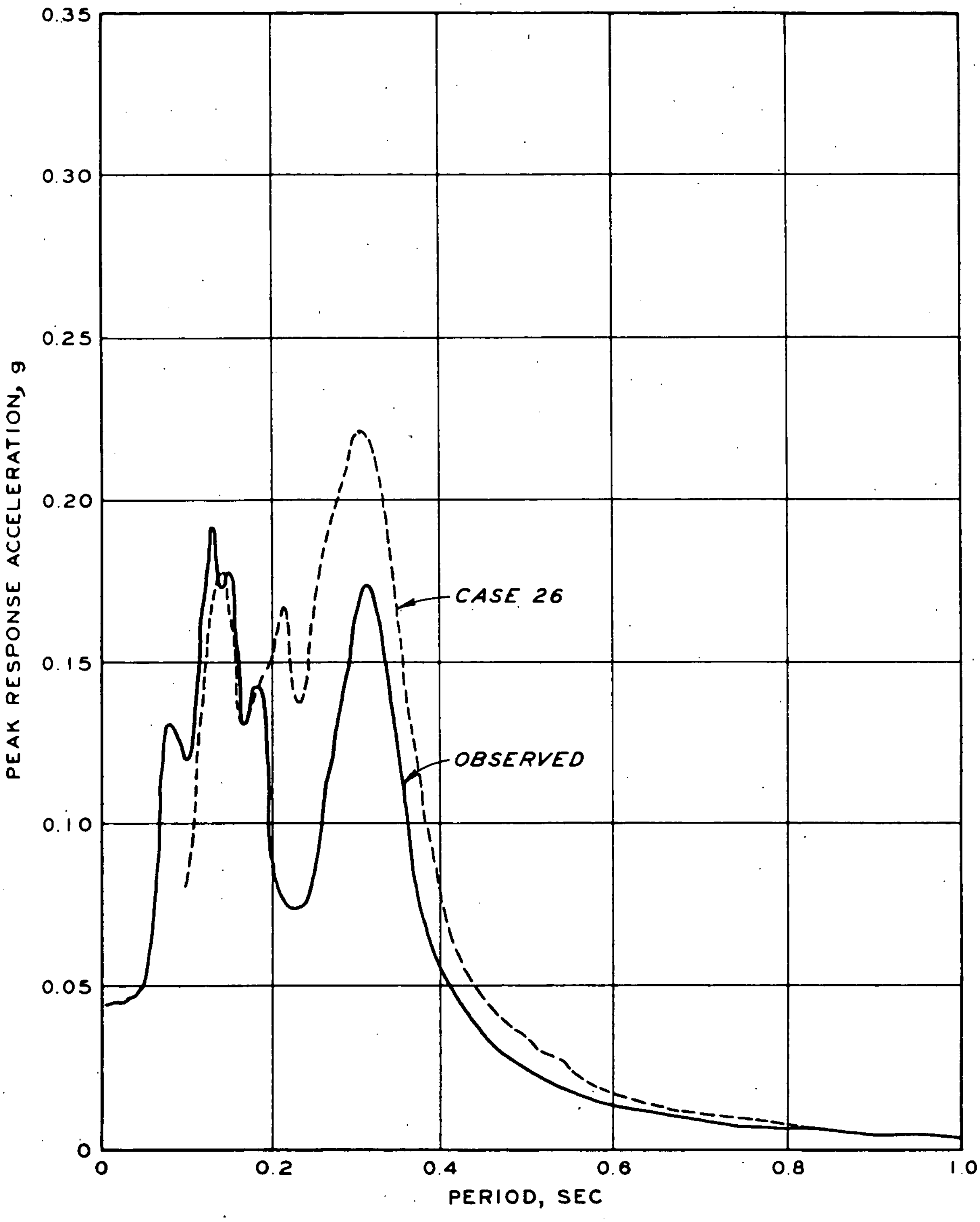
OBSERVED AND COMPUTED
2 D RESPONSE SPECTRA
LOCATION 5
RADIAL COMPONENT
CASE 23



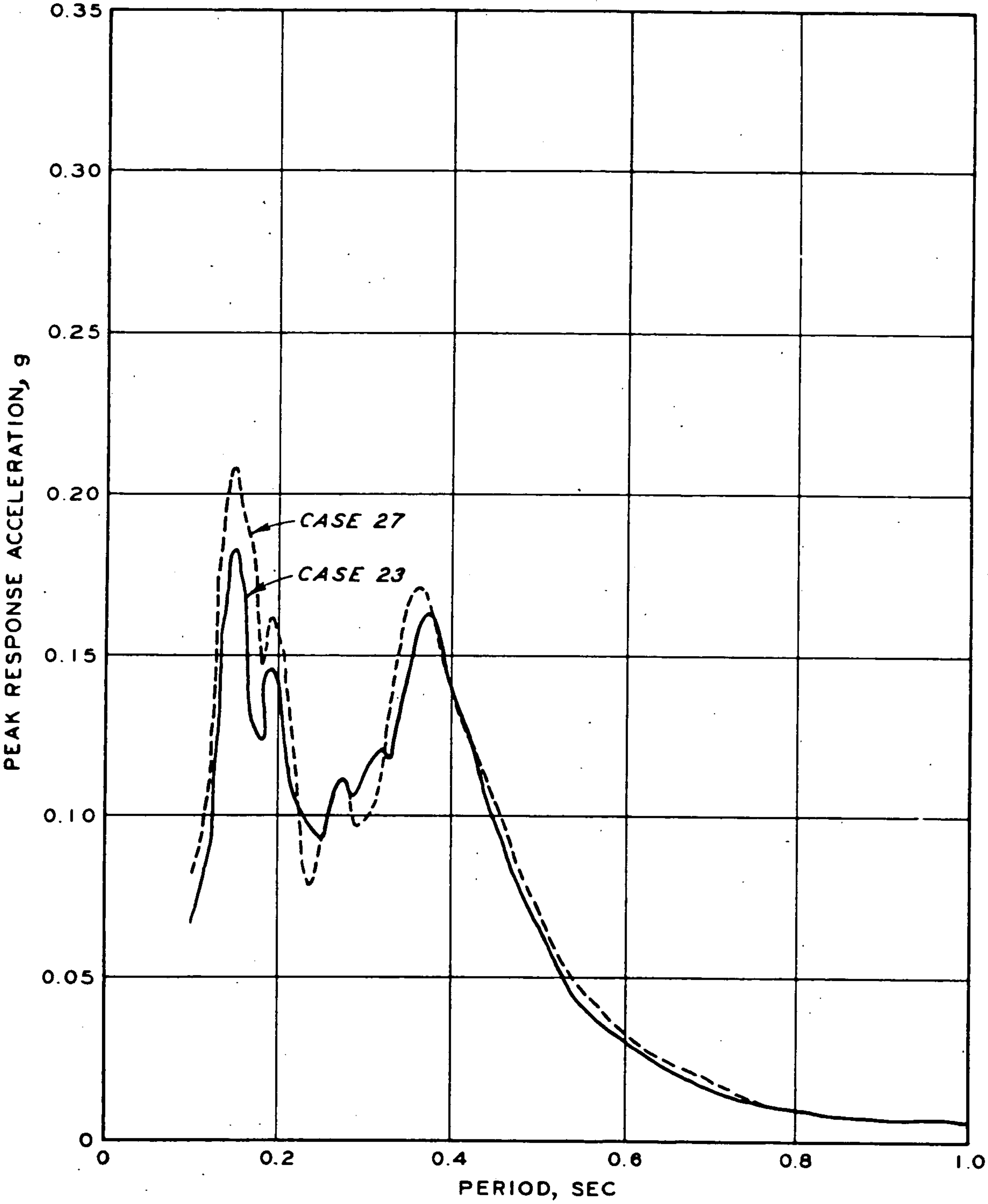
OBSERVED AND COMPUTED
2 D RESPONSE SPECTRA
LOCATION 5
VERTICAL COMPONENT
CASE 23



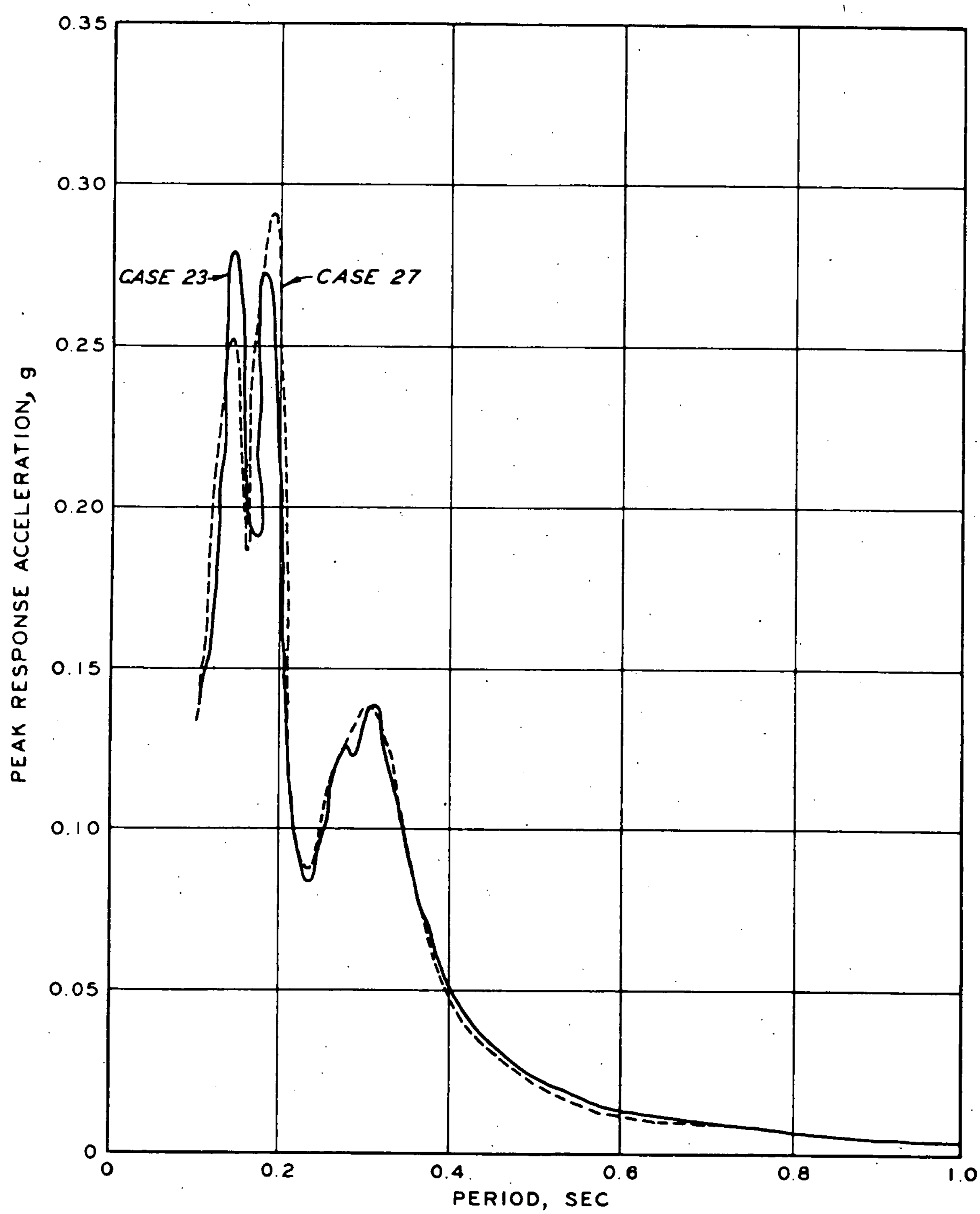
OBSERVED AND COMPUTED
2 D RESPONSE SPECTRA
LOCATION 3
RADIAL COMPONENT
CASE 26



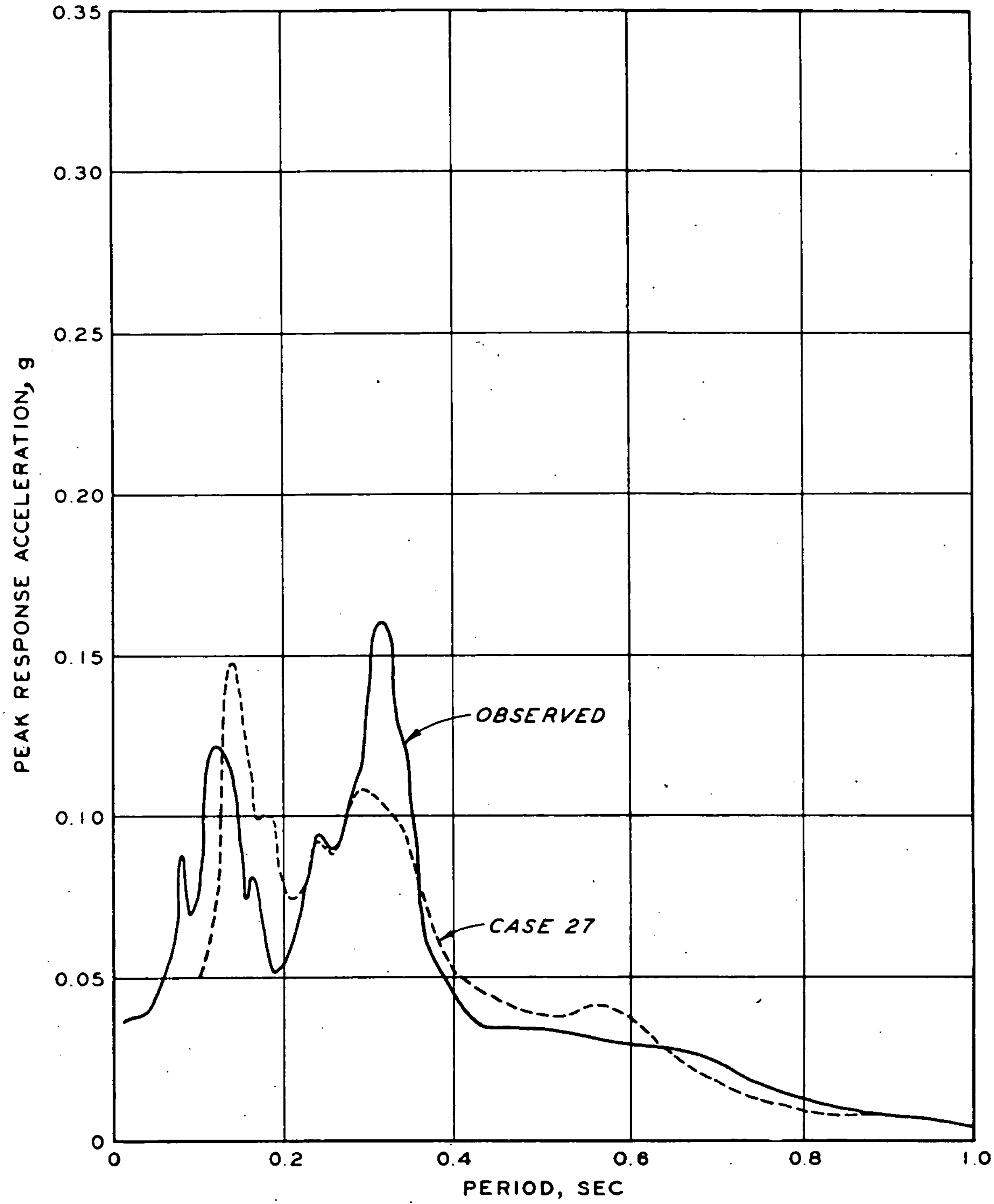
OBSERVED AND COMPUTED
2 D RESPONSE SPECTRA
LOCATION 3
VERTICAL COMPONENT
CASE 26



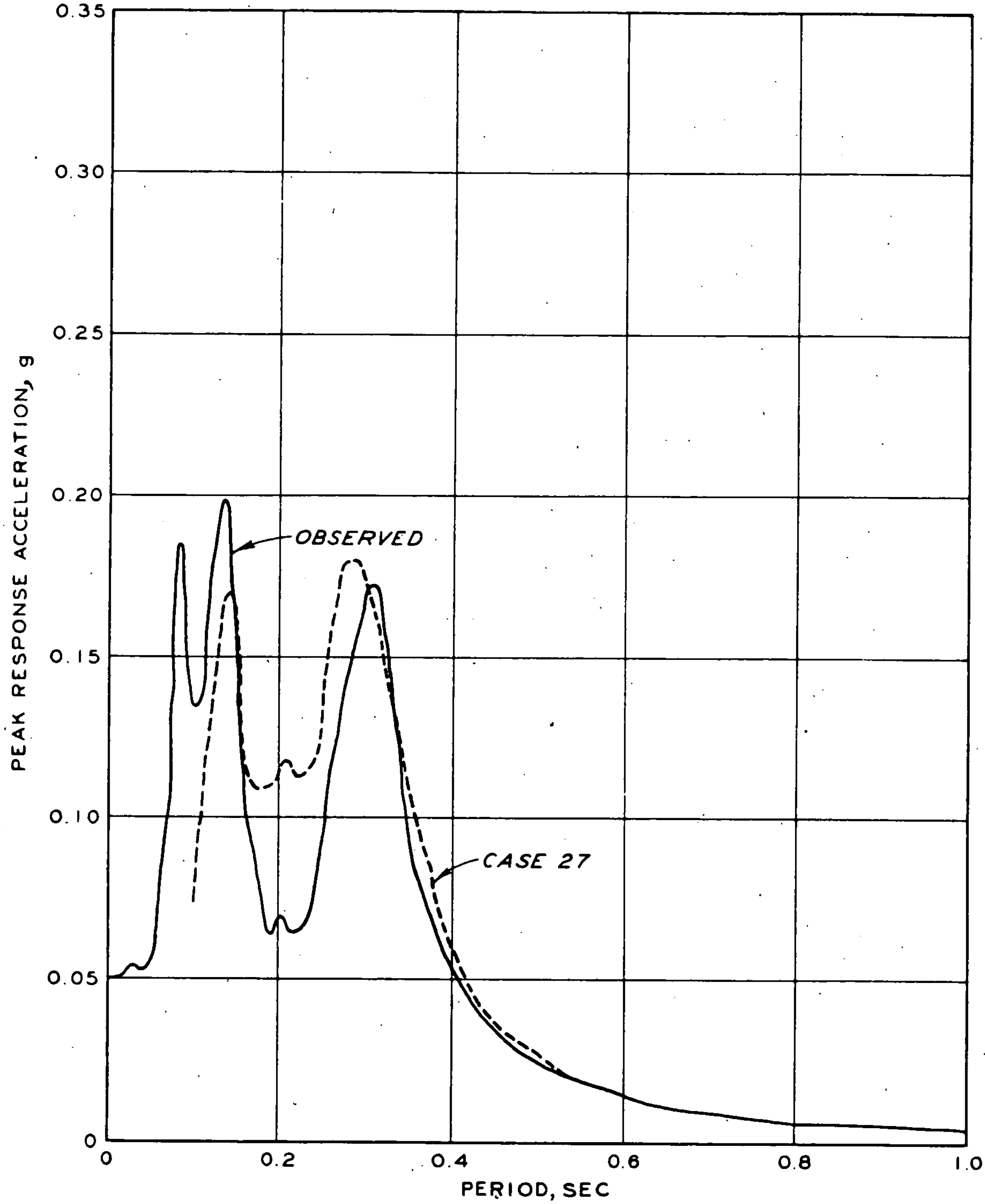
COMPUTED 2 D
RESPONSE SPECTRA
LOCATION 5
RADIAL COMPONENT
CASES 23 AND 27



COMPUTED 2 D
RESPONSE SPECTRA
LOCATION 5
VERTICAL COMPONENT
CASES 23 AND 27



OBSERVED AND COMPUTED
2D RESPONSE SPECTRA
LOCATION 2
RADIAL COMPONENT
CASE 27



OBSERVED AND COMPUTED
2 D RESPONSE SPECTRA
LOCATION 2
VERTICAL COMPONENT
CASE 27

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APPENDIX A: OBSERVED MOTIONS

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1. The first 6 sec of motion history for the radial and vertical components are shown in plates A1 through A10 for locations 1 through 5, respectively; acceleration response spectra are also shown in these plates. Plates A11 and A12 show the observed radial and vertical motion histories from PVT locations 6 and 7. The radial components were measured horizontally and perpendicular to the axis of the dam; transverse motion components were measured horizontally and parallel to the axis of the dam and are reported in reference 2. The acceleration histories have been modified for base-line shift with a parabolic correction. They have also been integrated to produce velocity and displacement histories.

2. A response spectrum is the maximum response of a single-degree-of-freedom system to an acceleration history, as illustrated in fig. A1. In this case, the spectra are plotted as curves of maximum acceleration versus natural period T for a value of damping D. By varying the values of T and D, a complete set of curves is developed. A response spectrum can be used as a vehicle to compare the frequency content of various motion histories. The relative velocity and relative displacement response spectra have been computed and are available from WES but are not presented herein.

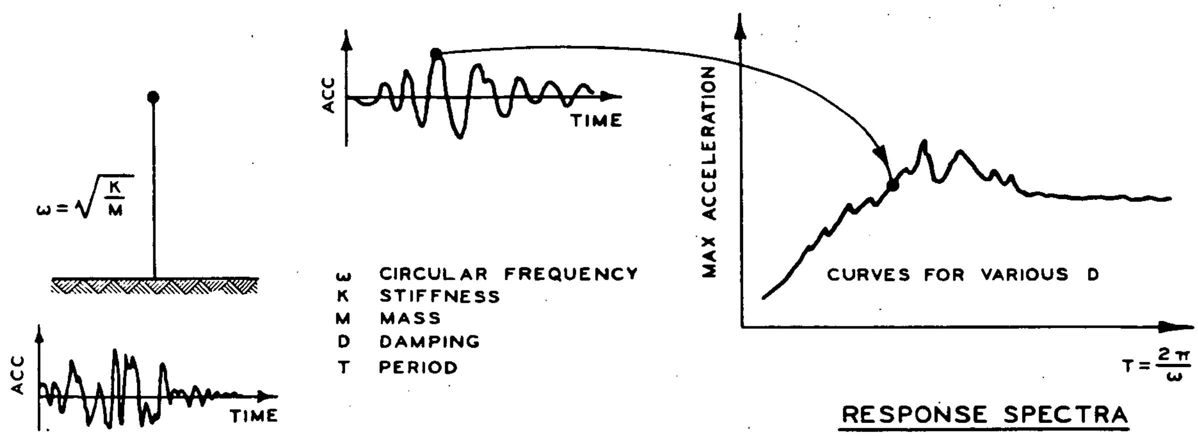
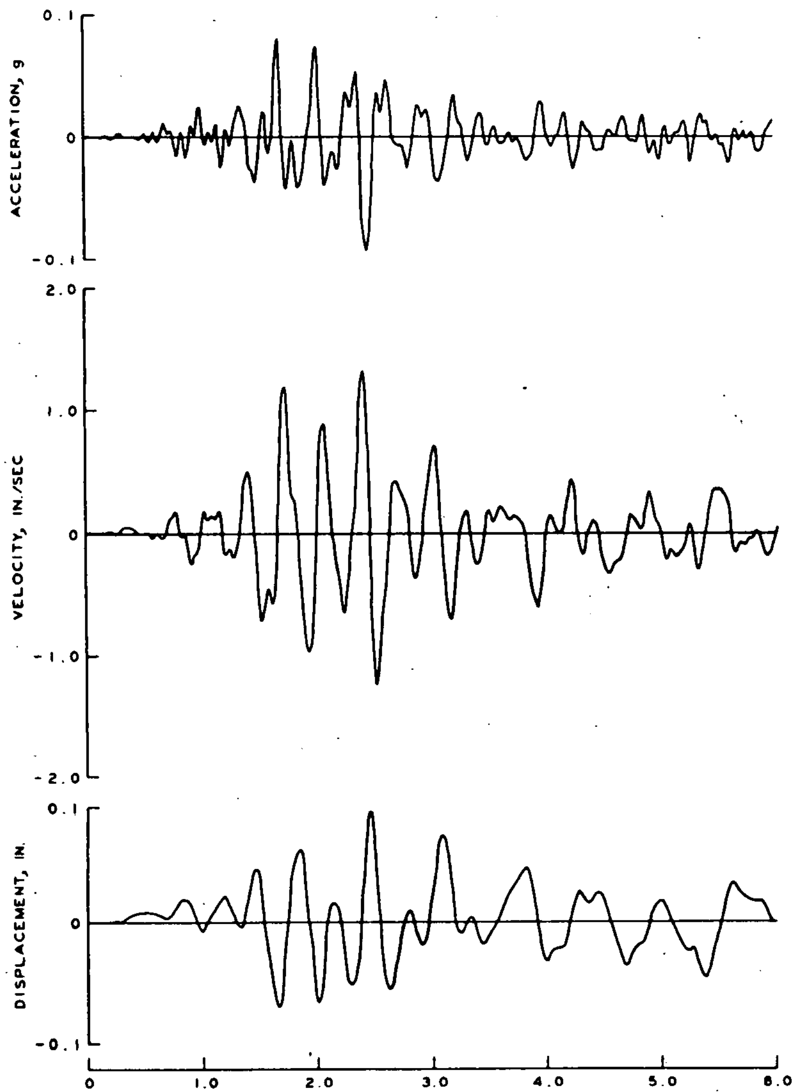
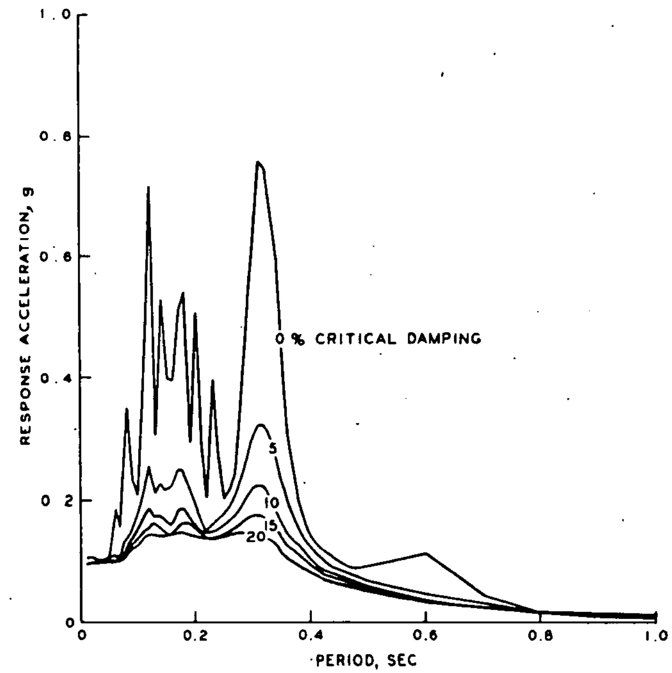


Fig. A1. Response spectra



a. MOTION HISTORY



b. ACCELERATION RESPONSE SPECTRUM

NOTE: LOCATION I WAS ON THE SURFACE OF THE DAM AT THE CREST.

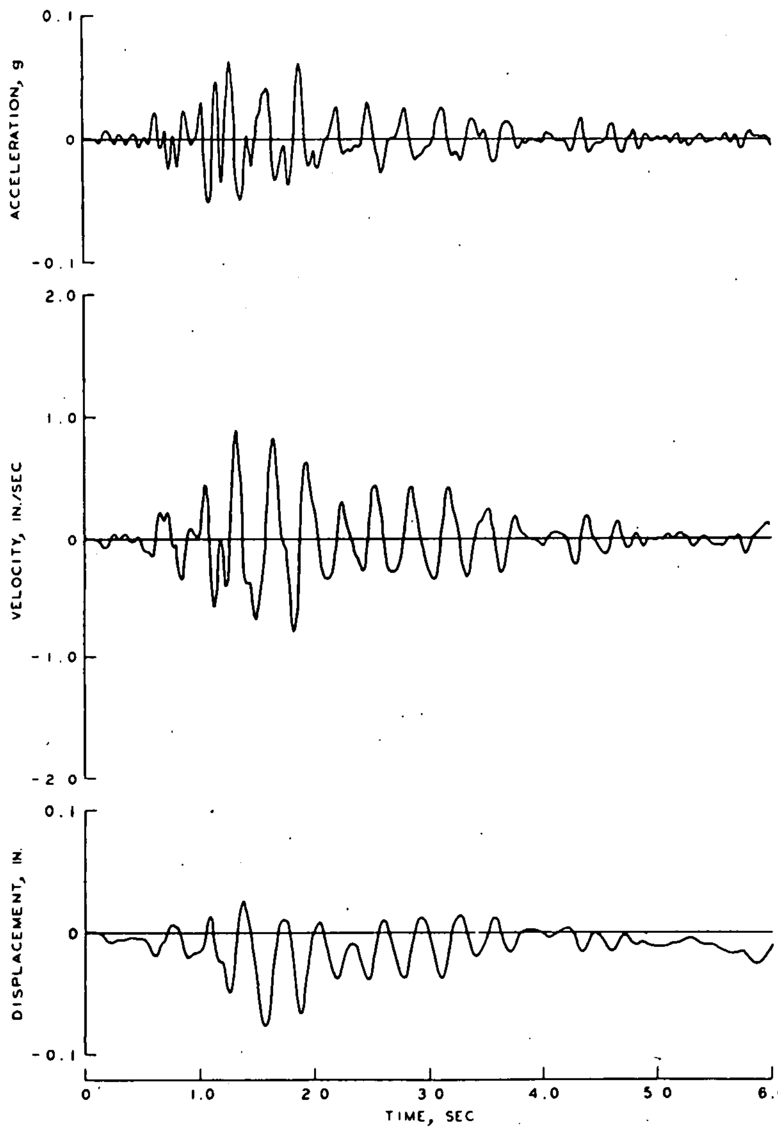
MOTION HISTORY AND ACCELERATION RESPONSE SPECTRUM

RADIAL COMPONENT

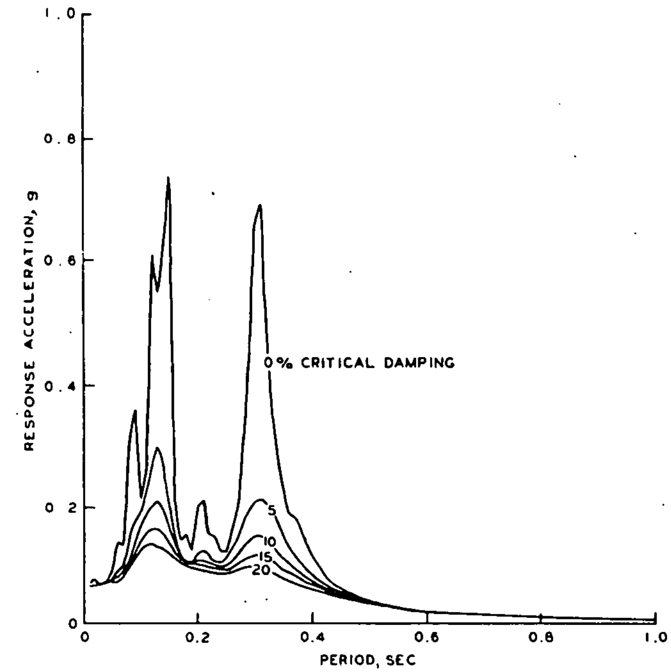
LOCATION I

PLATE A1

PLATE A2



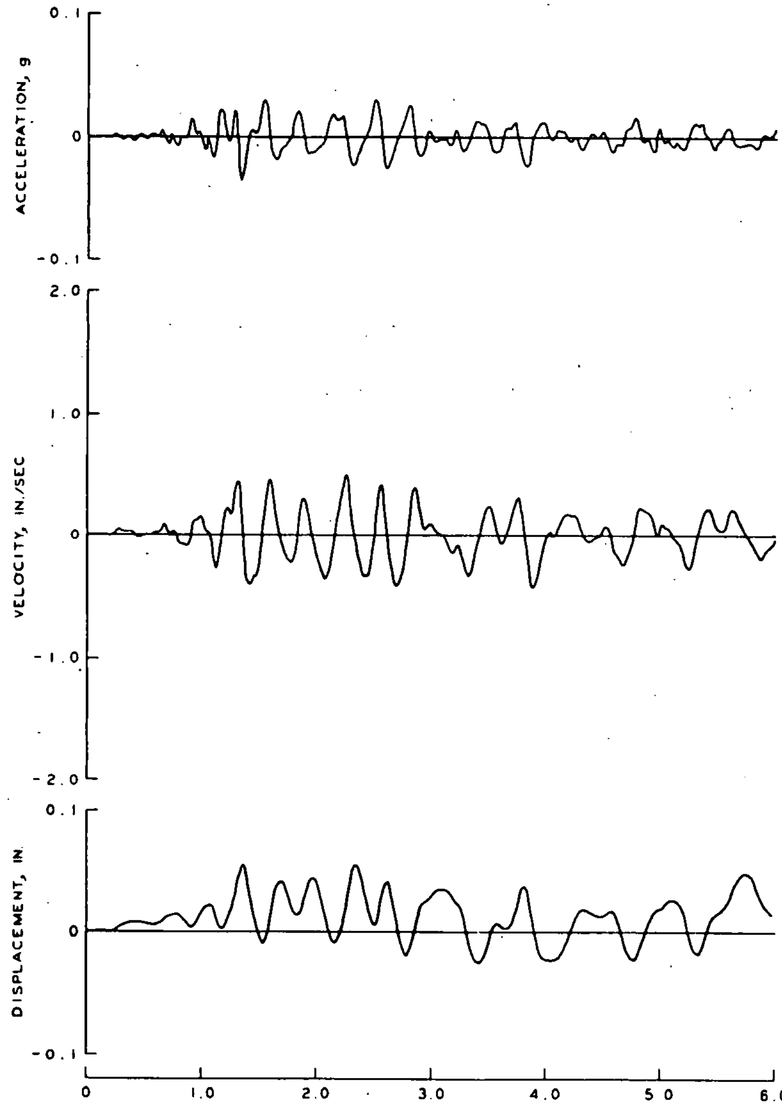
a. MOTION HISTORY



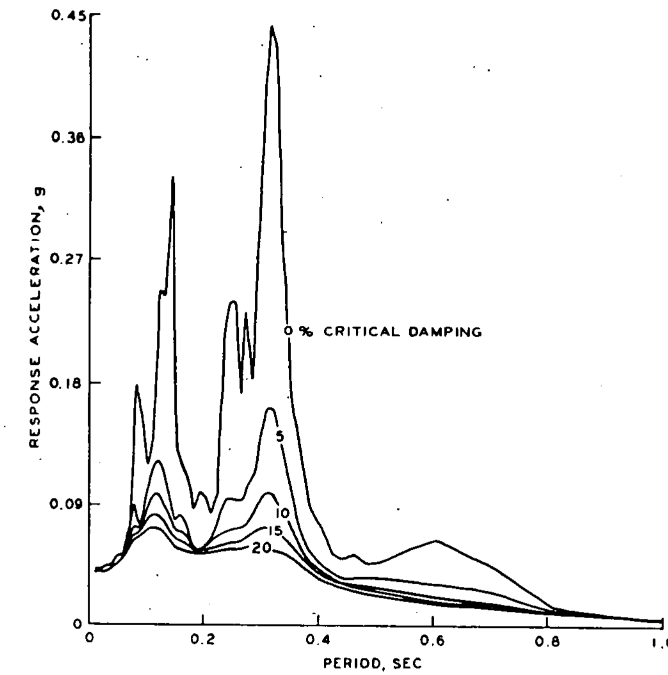
b. ACCELERATION RESPONSE SPECTRUM

NOTE: LOCATION I WAS ON THE SURFACE OF THE DAM AT THE CREST.

MOTION HISTORY
AND ACCELERATION
RESPONSE SPECTRUM
VERTICAL COMPONENT
LOCATION I



a. MOTION HISTORY

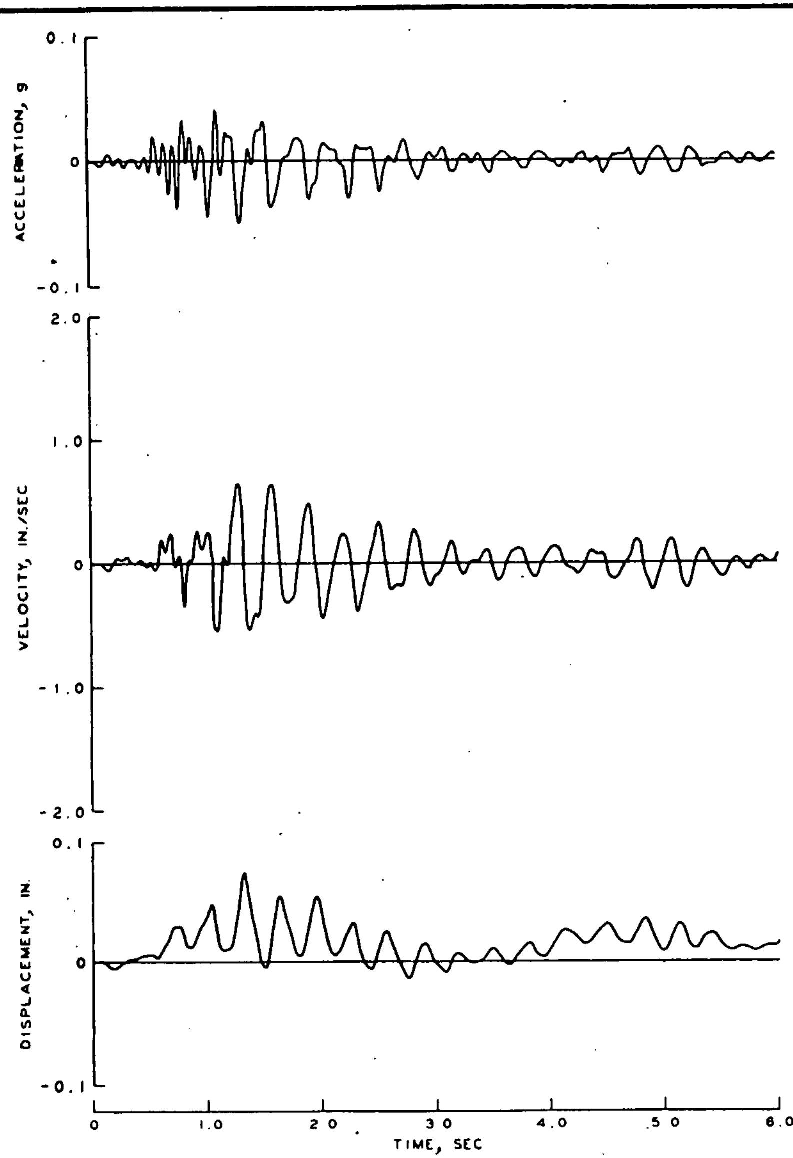


b. ACCELERATION RESPONSE SPECTRUM

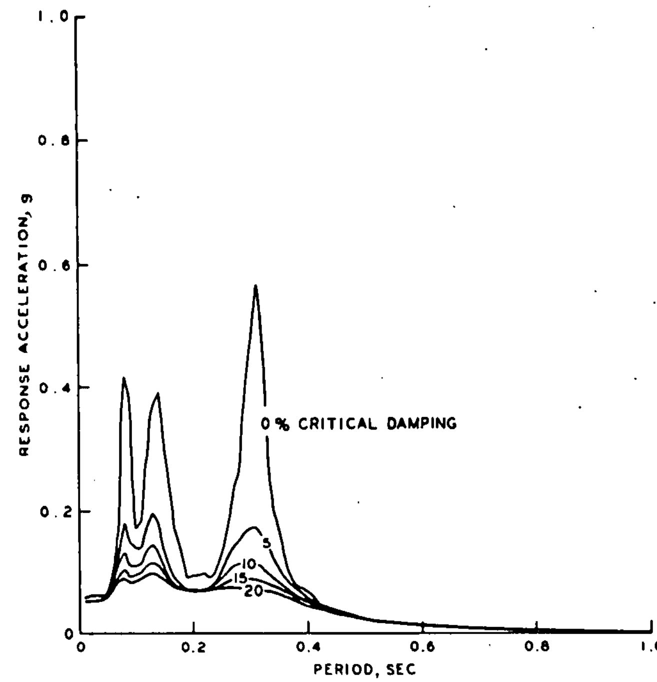
NOTE: LOCATION 2 WAS ON THE DOWNSTREAM FACE OF DAM.

MOTION HISTORY
AND ACCELERATION
RESPONSE SPECTRUM
RADIAL COMPONENT
LOCATION 2

PLATE A4



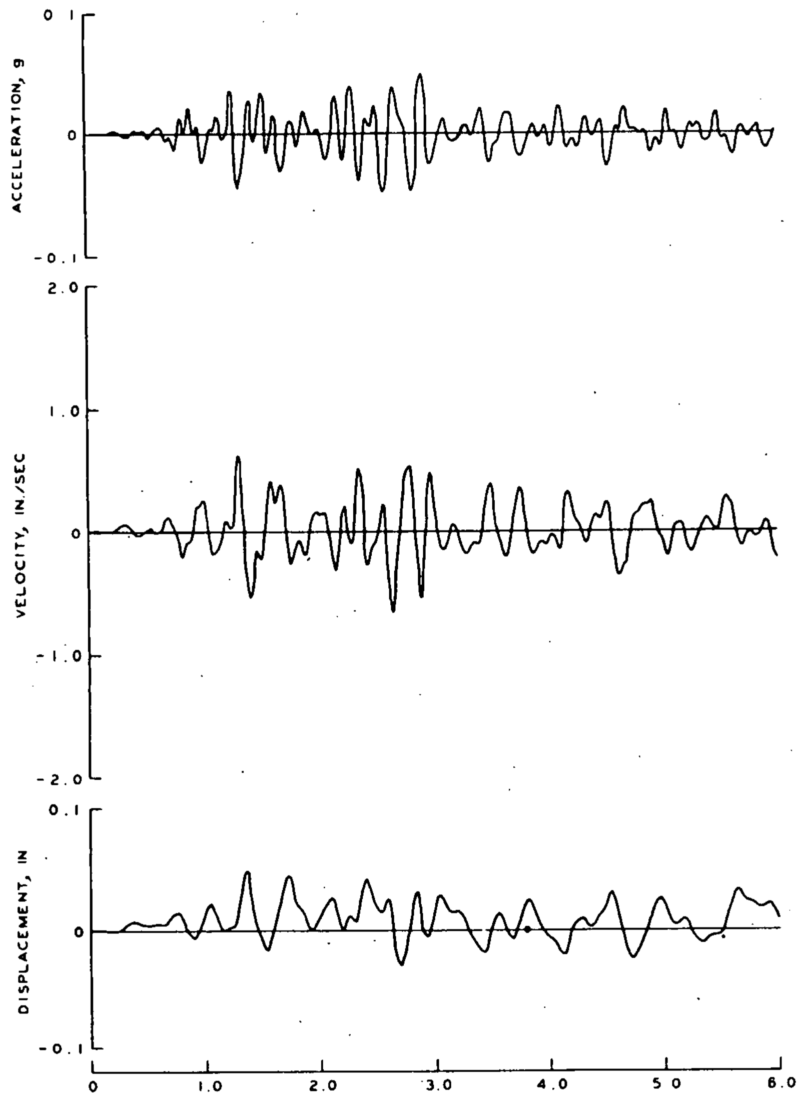
a. MOTION HISTORY



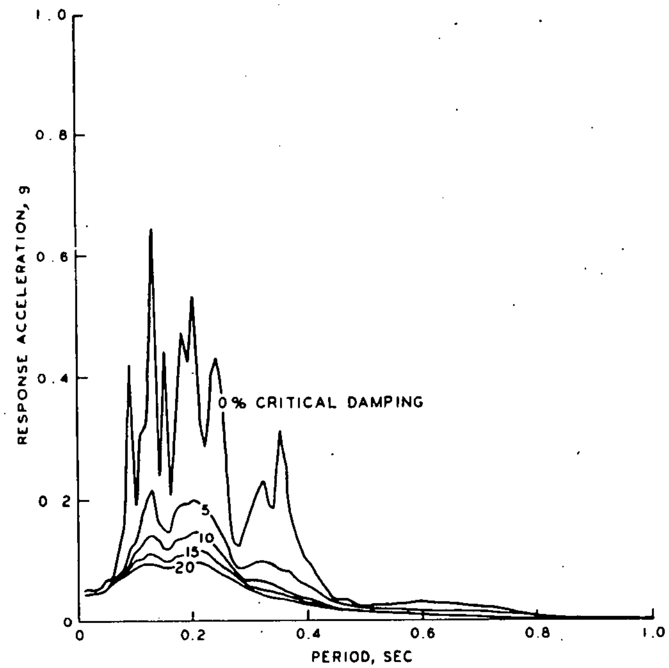
b. ACCELERATION RESPONSE SPECTRUM

NOTE: LOCATION 2 WAS ON THE DOWNSTREAM FACE OF DAM.

MOTION HISTORY
AND ACCELERATION
RESPONSE SPECTRUM
VERTICAL COMPONENT
LOCATION 2



a. MOTION HISTORY

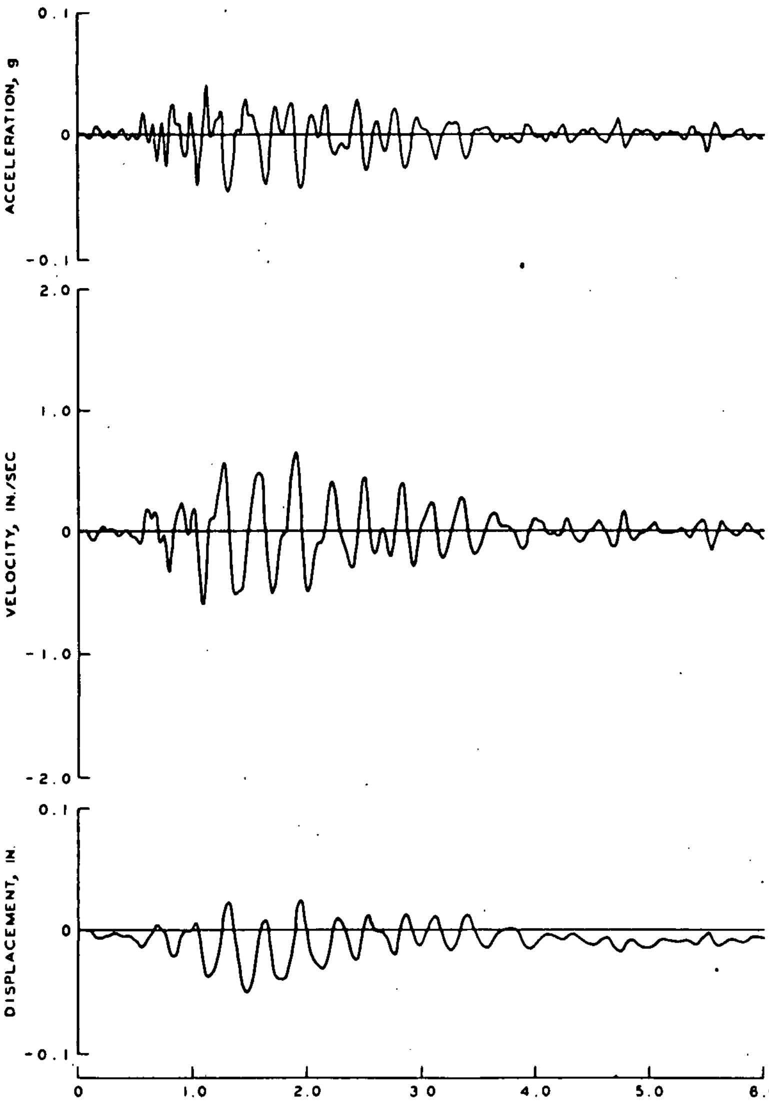


b. ACCELERATION RESPONSE SPECTRUM

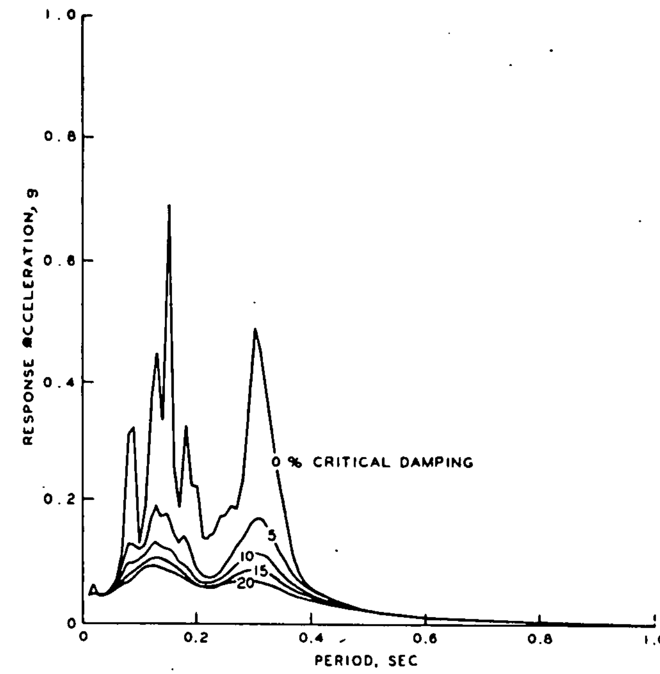
NOTE: LOCATION 3 WAS NEAR TOE OF DAM.

MOTION HISTORY
AND ACCELERATION
RESPONSE SPECTRUM
RADIAL COMPONENT
LOCATION 3

PLATE A6



a. MOTION HISTORY



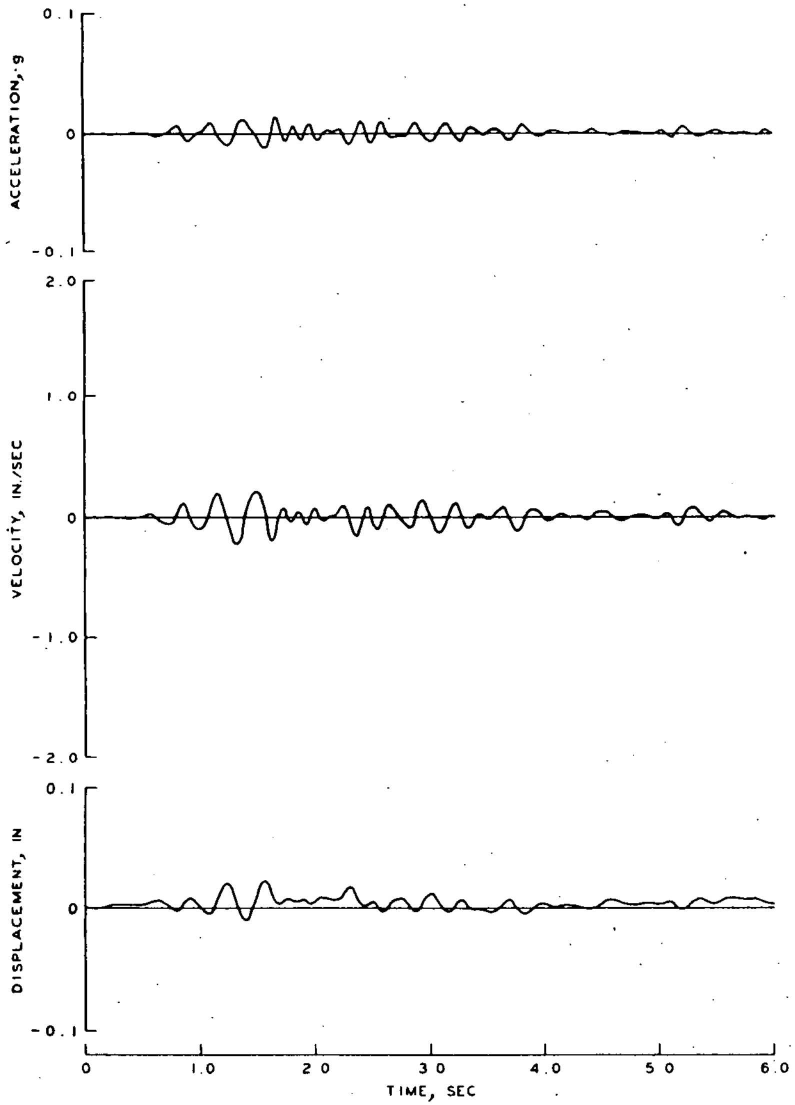
b. ACCELERATION RESPONSE SPECTRUM

NOTE: LOCATION 3 WAS NEAR TOE OF DAM.

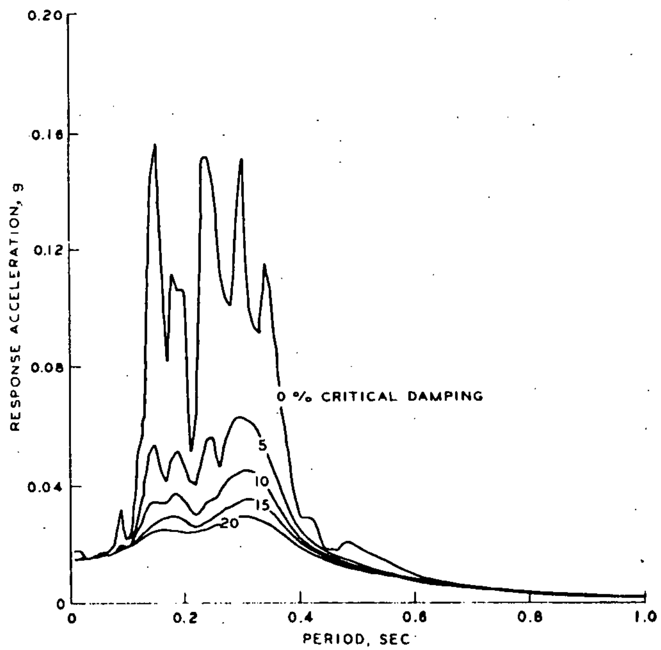
MOTION HISTORY AND ACCELERATION RESPONSE SPECTRUM

VERTICAL COMPONENT

LOCATION 3



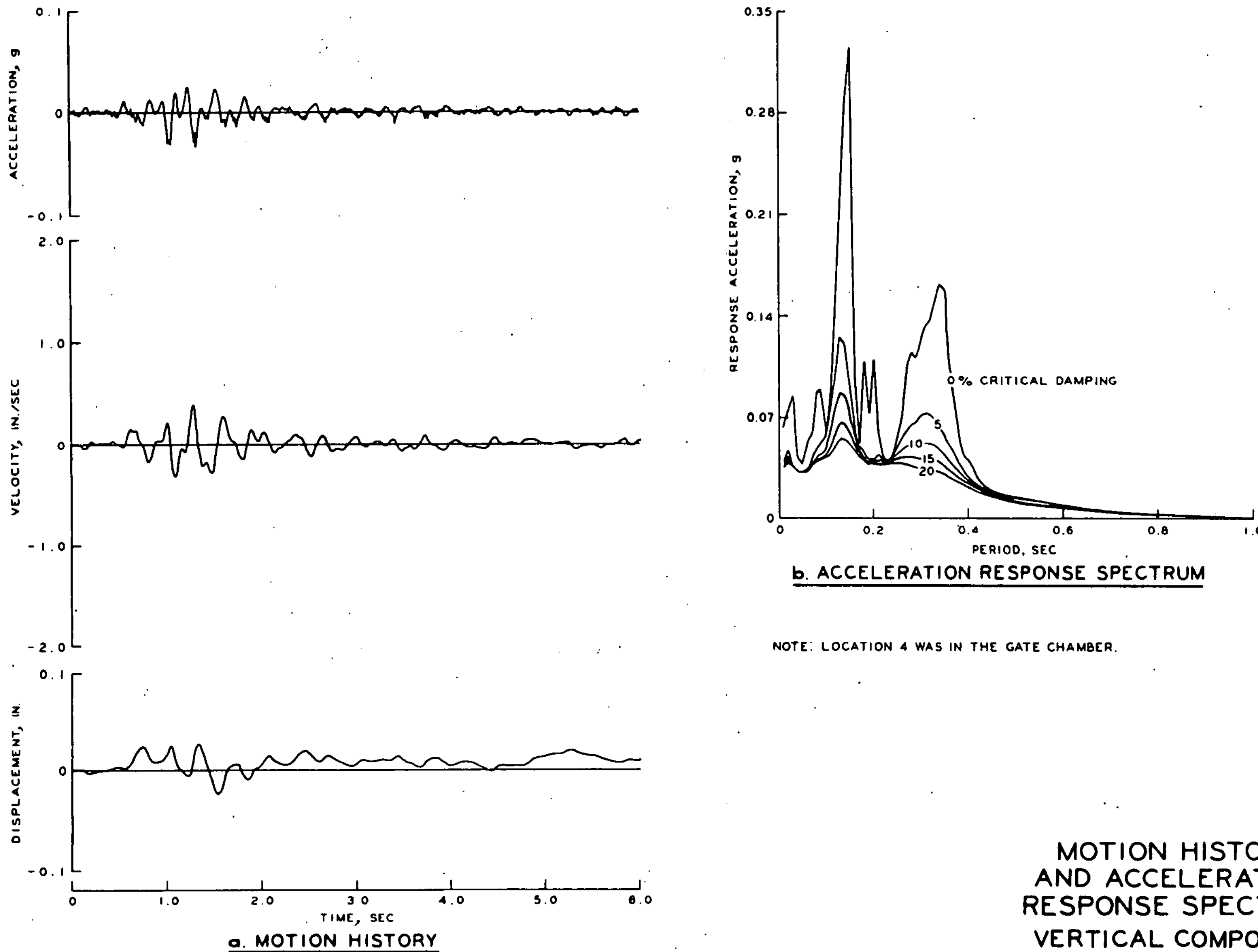
a. MOTION HISTORY



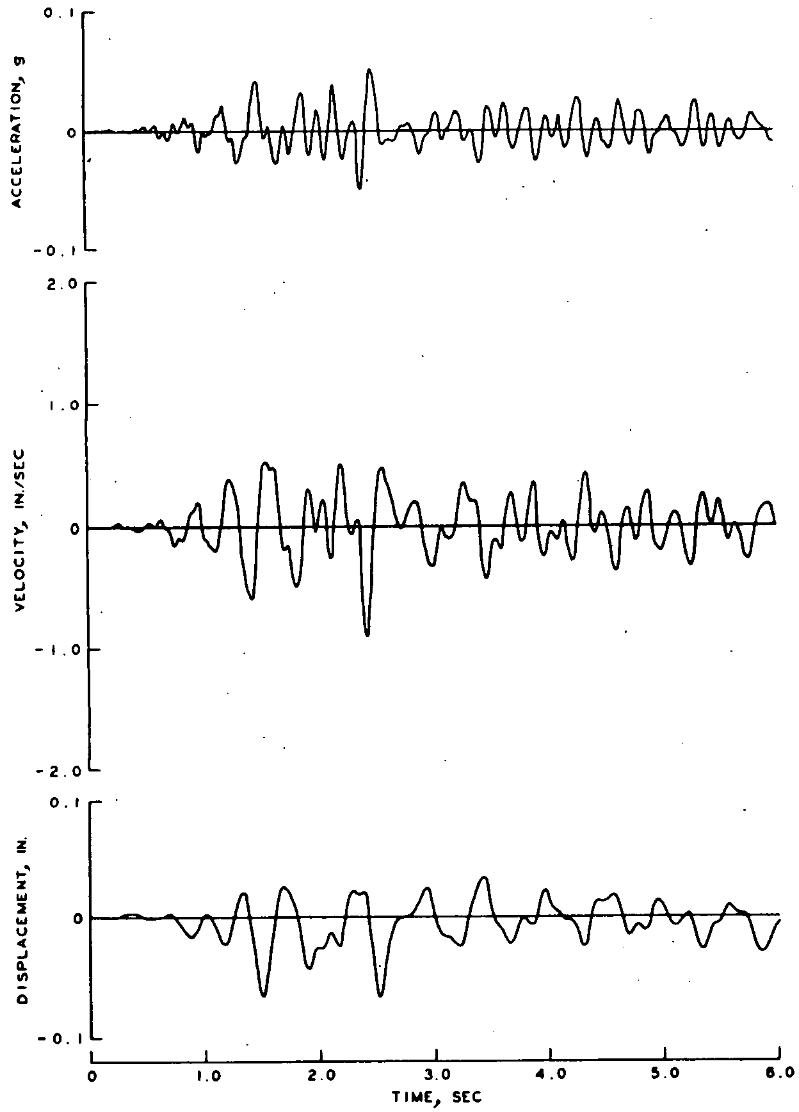
b. ACCELERATION RESPONSE SPECTRUM

NOTE: LOCATION 4 WAS IN THE GATE CHAMBER.

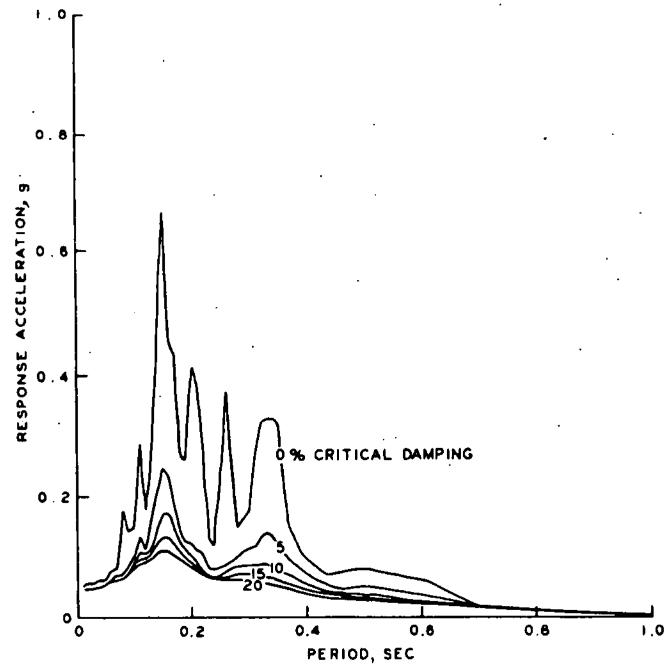
MOTION HISTORY
AND ACCELERATION
RESPONSE SPECTRUM
RADIAL COMPONENT
LOCATION 4



MOTION HISTORY
AND ACCELERATION
RESPONSE SPECTRUM
VERTICAL COMPONENT
LOCATION 4



a. MOTION HISTORY



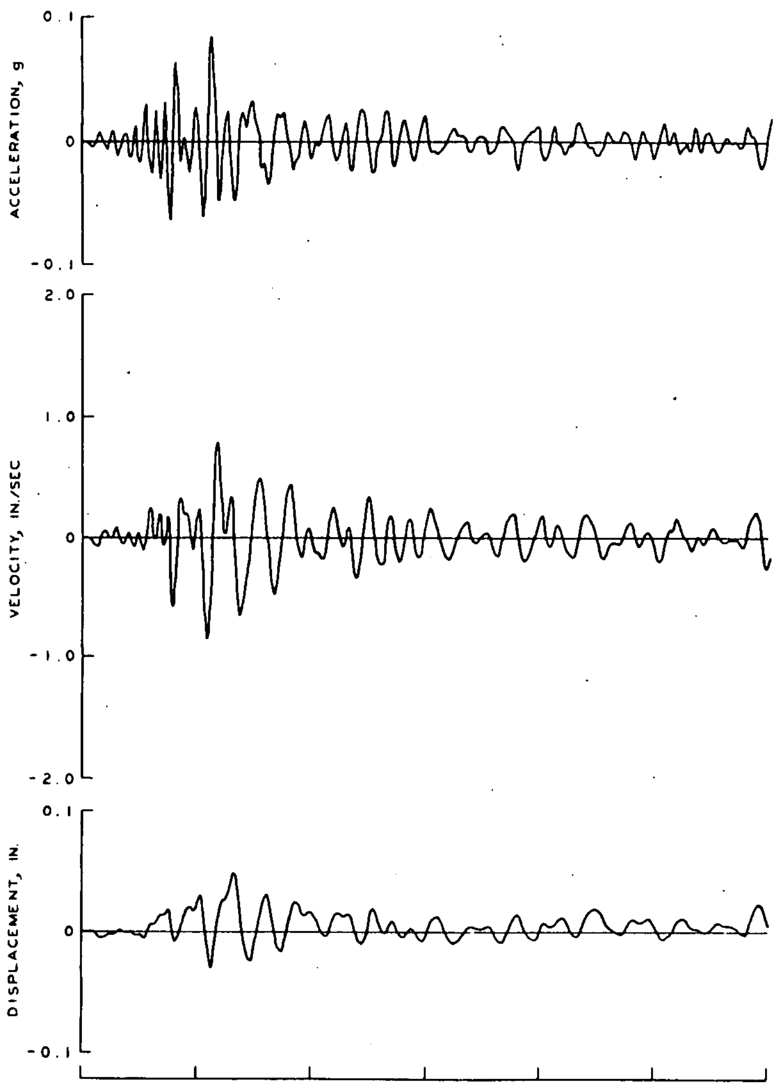
b. ACCELERATION RESPONSE SPECTRUM

NOTE: LOCATION 5 WAS ON THE SURFACE OF THE ALLUVIAL VALLEY.

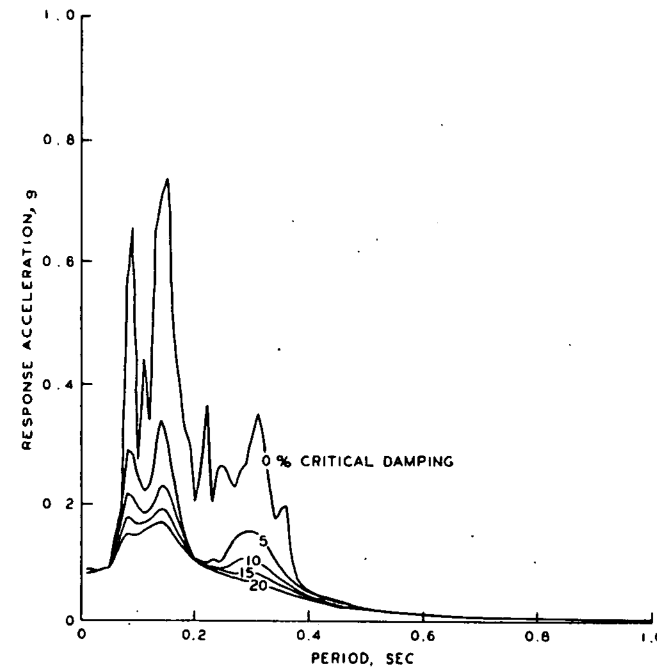
MOTION HISTORY
AND ACCELERATION
RESPONSE SPECTRUM
RADIAL COMPONENT
LOCATION 5

PLATE A9

PLATE A10



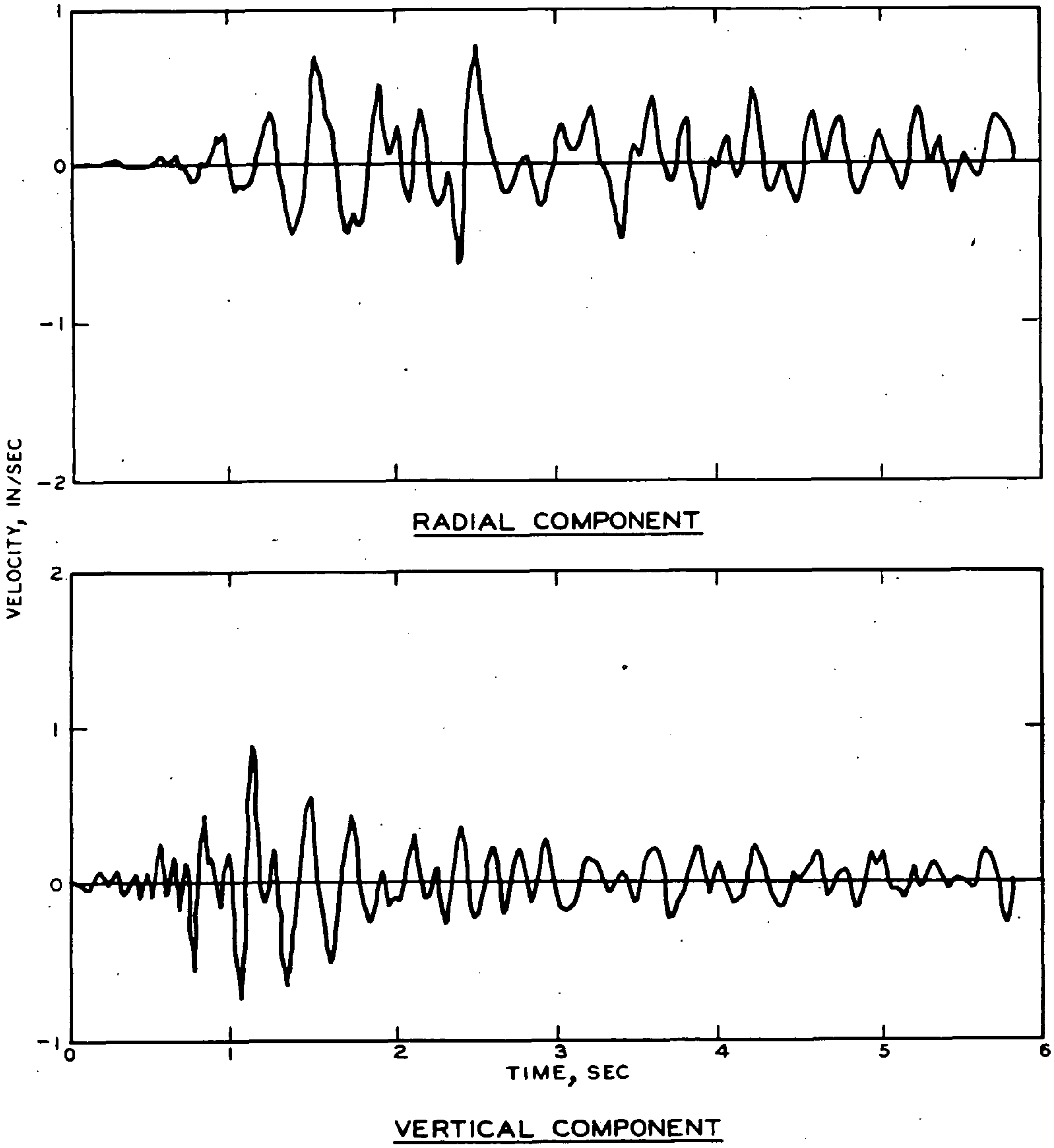
a. MOTION HISTORY



b. ACCELERATION RESPONSE SPECTRUM

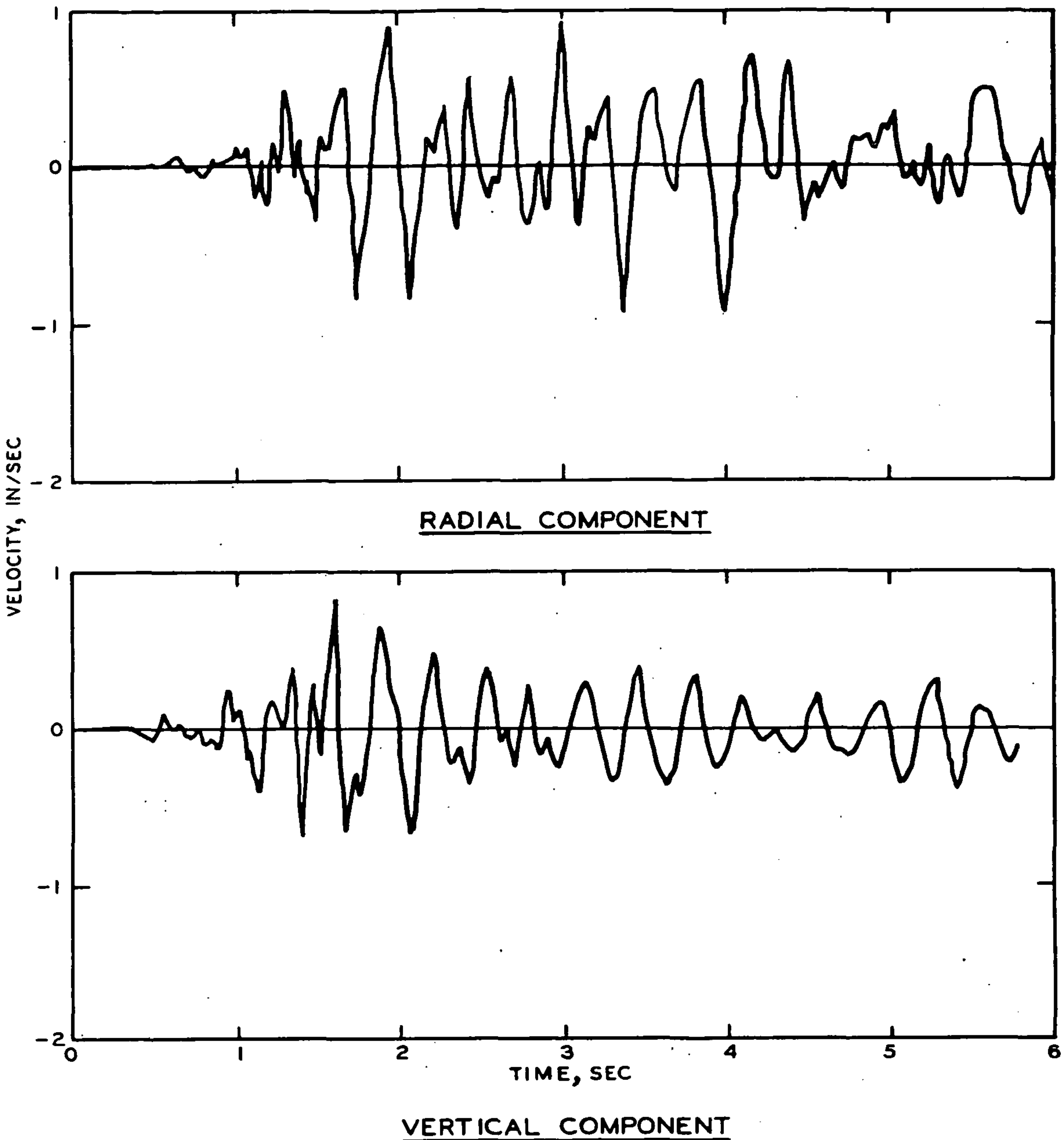
NOTE: LOCATION 5 WAS ON THE SURFACE OF THE ALLUVIAL VALLEY.

MOTION HISTORY
AND ACCELERATION
RESPONSE SPECTRUM
VERTICAL COMPONENT
LOCATION 5



NOTE: LOCATION 6 WAS ADJACENT TO LOCATION 5 (ON THE SURFACE OF THE ALLUVIAL VALLEY APPROXIMATELY 500 FT DOWNSTREAM FROM LOCATION 3)

MOTION HISTORIES
LOCATION 6



NOTE : LOCATION 7 WAS AT A SETTLEMENT MARKER, 200FT WEST OF LOCATION 1, ON THE SURFACE OF THE DAM AT THE CREST.

MOTION HISTORIES
LOCATION 7

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APPENDIX B: ACCELERATION RESPONSE SPECTRA FROM ONE-DIMENSIONAL ANALYSIS

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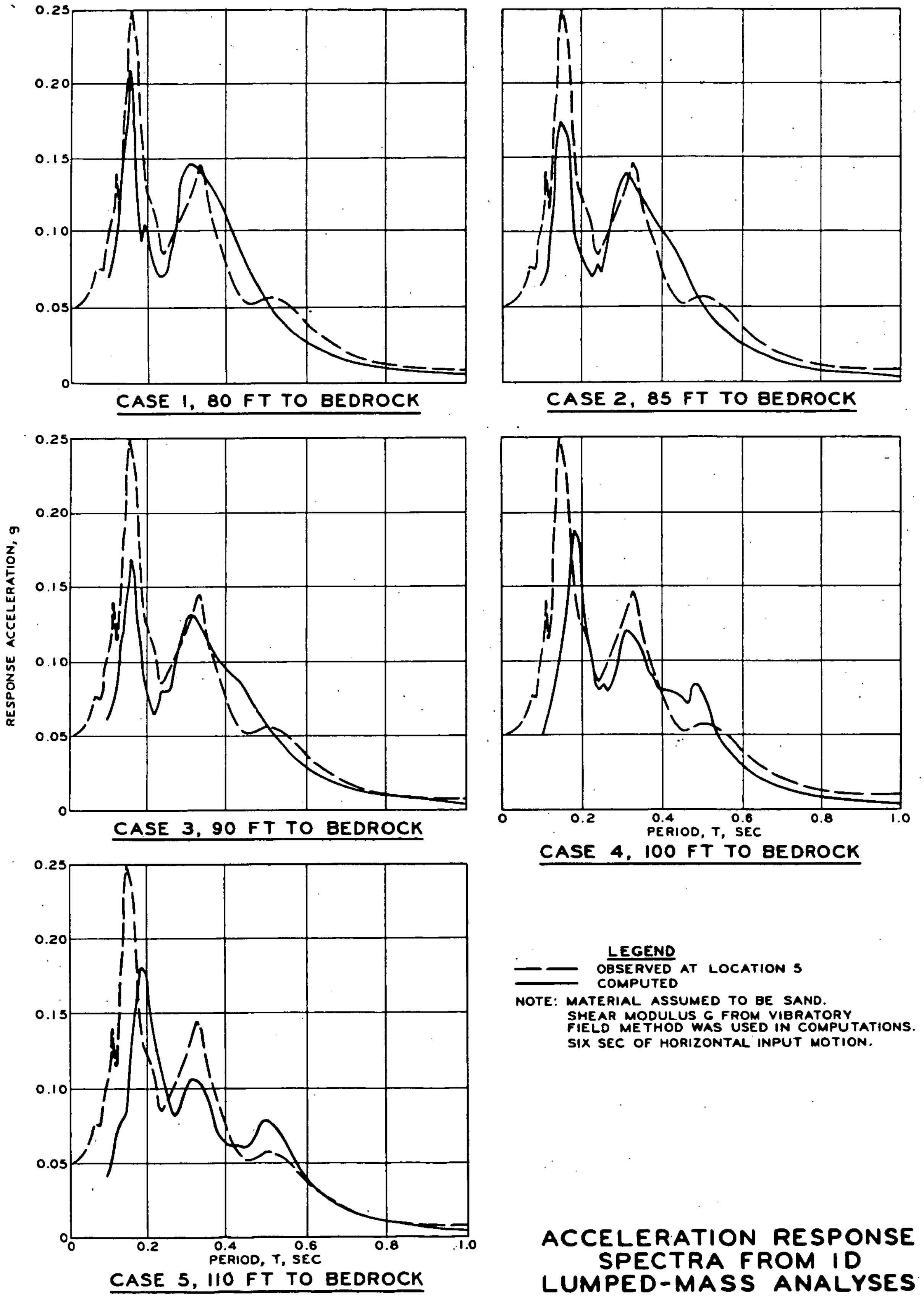
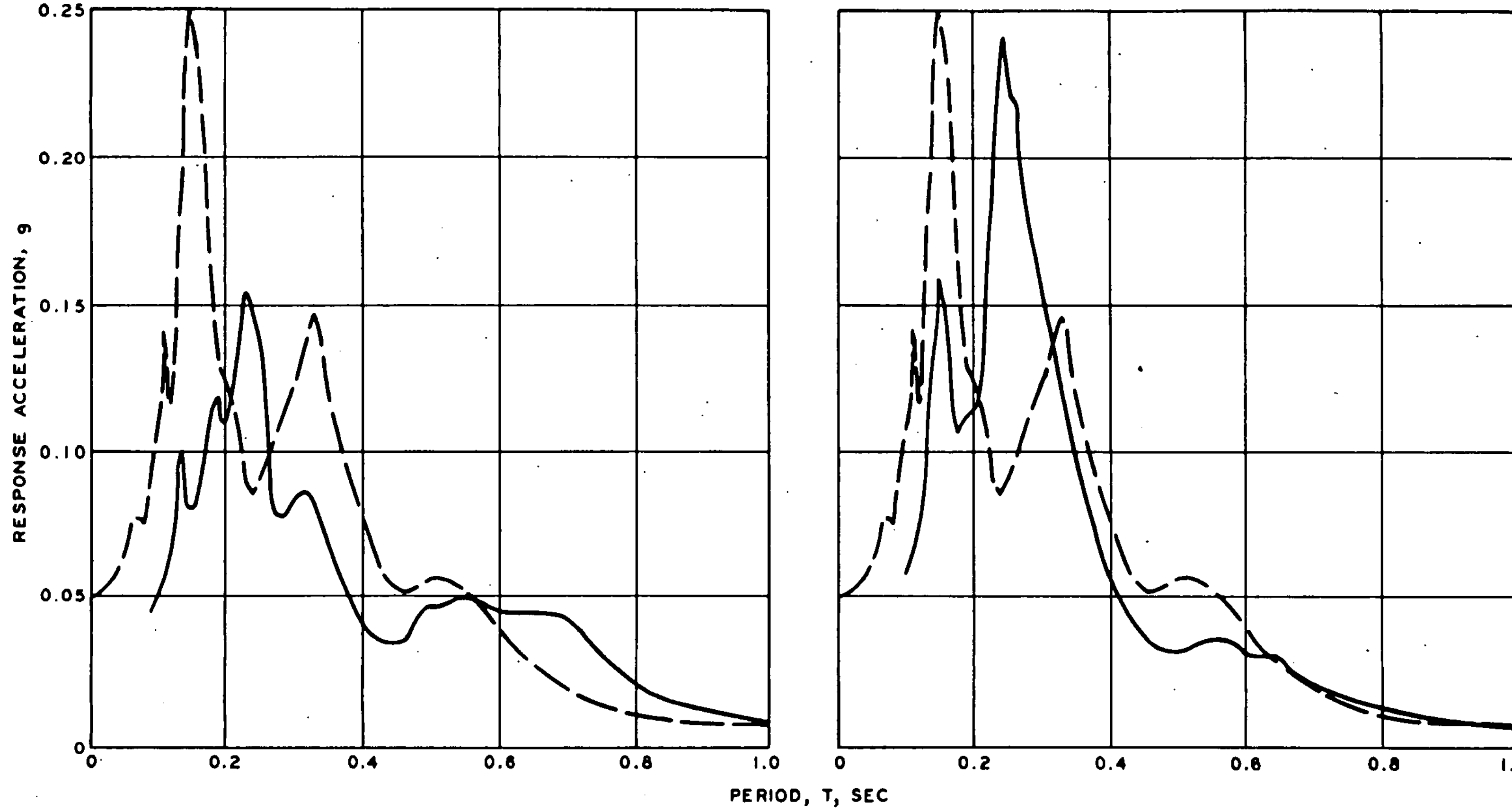
Response spectra as defined in Appendix A are shown in plate B1 for the observed motion at location 5R, which is the radial component of acceleration on the alluvium over 500 ft downstream from Rifle Gap Dam. Acceleration response spectra computed from the 1D analyses for the various cases investigated are shown in plates B2-B6. Spectral damping in all plates is 5 percent.



ACCELERATION RESPONSE SPECTRA FROM 1D LUMPED-MASS ANALYSES
CASES 1, 2, 3, 4, AND 5



CASE 6, 90 FT TO BEDROCK

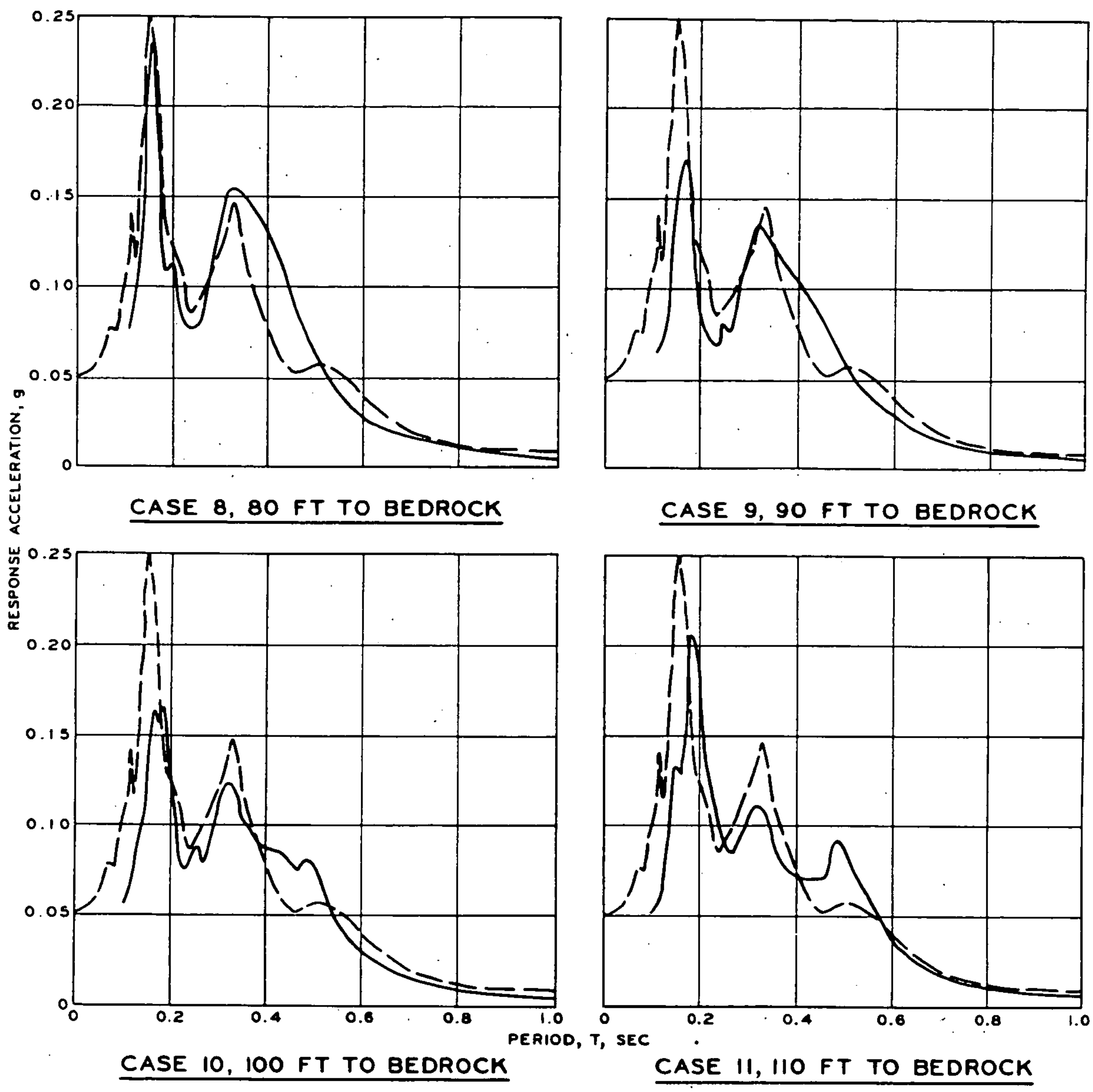
CASE 7, 110 FT TO BEDROCK

LEGEND

— — OBSERVED AT LOCATION 5
——— COMPUTED

NOTE: MATERIAL ASSUMED TO BE CLAY.
SHEAR MODULUS G USED IN COMPUTATIONS.
SIX SEC OF HORIZONTAL INPUT MOTION.

ACCELERATION RESPONSE SPECTRA FROM 1D LUMPED-MASS ANALYSES
CASES 6 AND 7



LEGEND

— — OBSERVED AT LOCATION 5

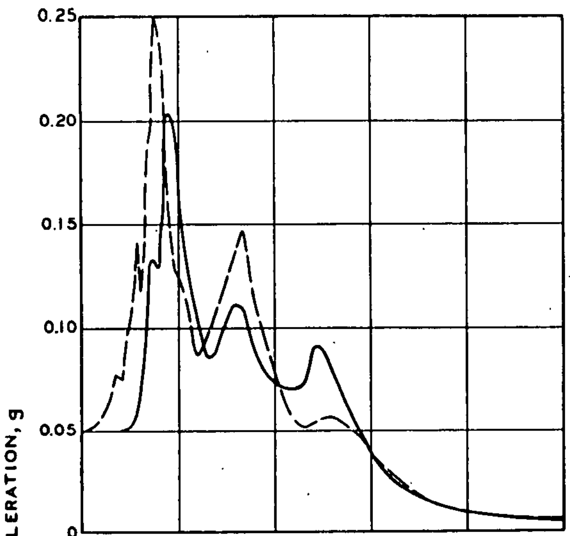
—— COMPUTED

NOTE: MATERIAL ASSUMED TO BE SAND AND CLAY MIXED.
CLAY MODULUS G MULTIPLIED BY 1.875.
SIX SEC OF HORIZONTAL INPUT USED.

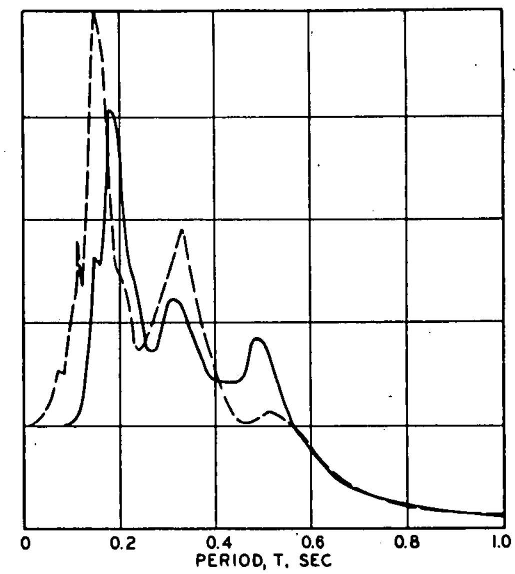
ACCELERATION RESPONSE SPECTRA FROM 1D LUMPED-MASS ANALYSES

CASES 8, 9, 10, AND 11

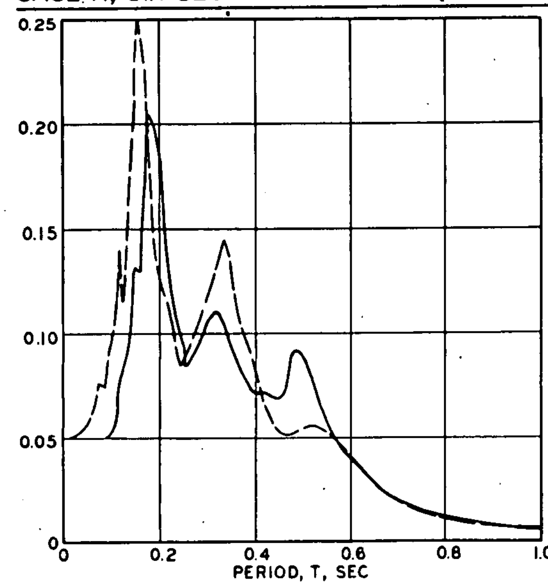
PLATE B4



CASE 11, SIX SECONDS OF INPUT (CORRECTED)



CASE 12, SIX SECONDS OF INPUT



CASE 13, TWELVE SECONDS OF INPUT

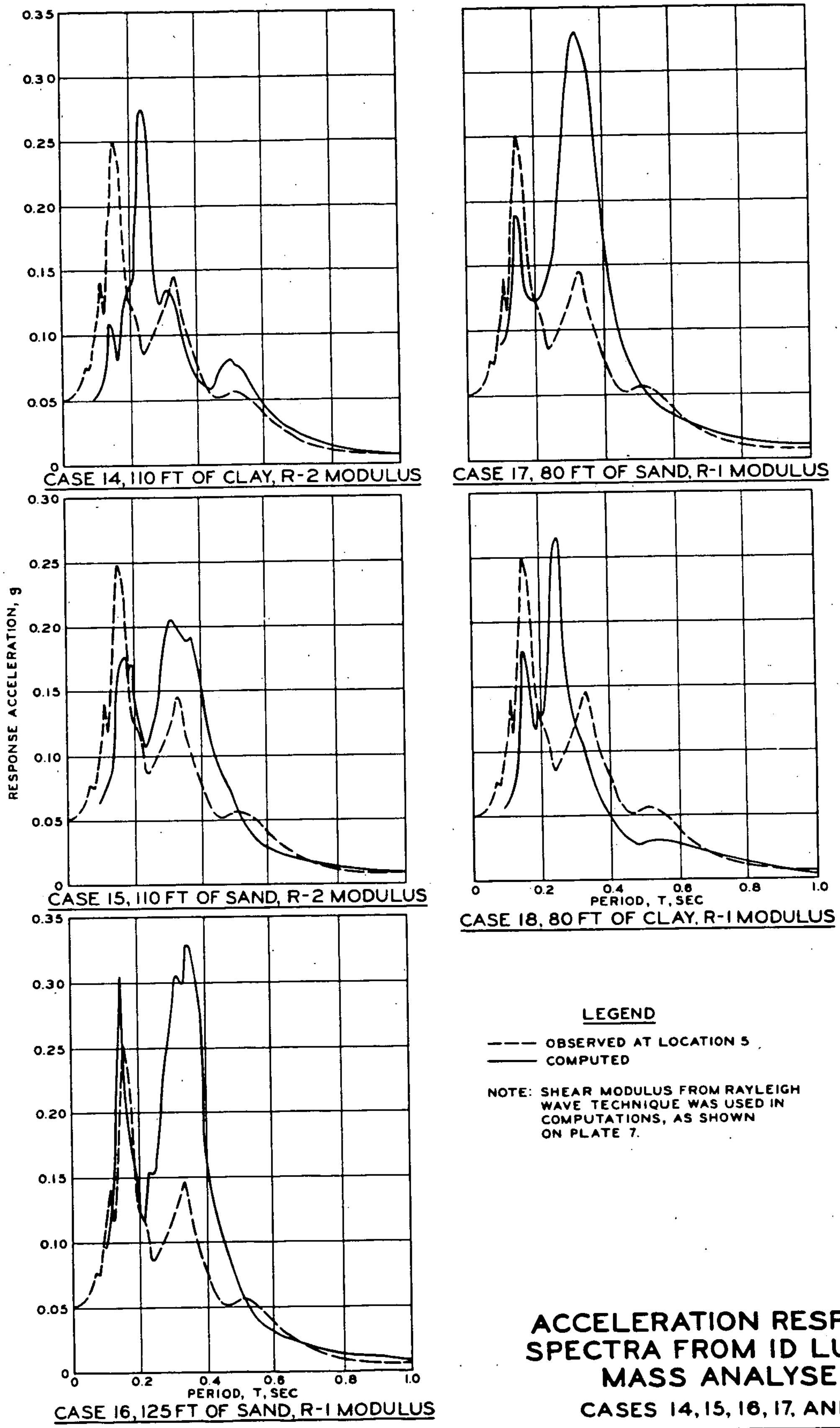
LEGEND

– – – OBSERVED AT LOCATION 5
——— COMPUTED

NOTE: MATERIAL ASSUMED TO BE SAND AND CLAY MIXED. CLAY SHEAR MODULUS G MULTIPLIED BY 1.875. DEPTH TO BEDROCK IS 110 FT.

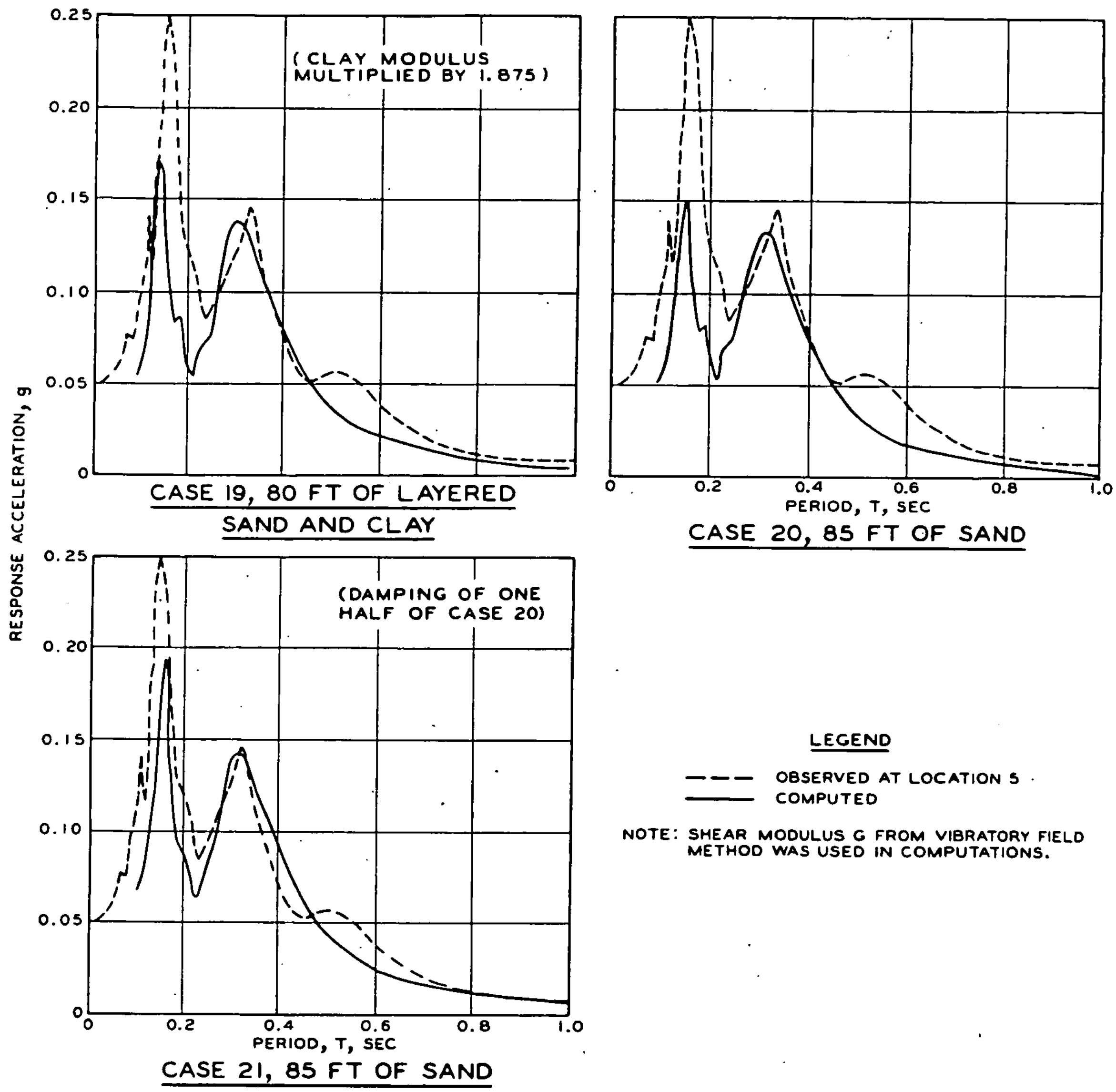
ACCELERATION RESPONSE SPECTRA FROM 1D LUMPED MASS ANALYSES

CASES 11, 12, AND 13



ACCELERATION RESPONSE SPECTRA FROM 1D LUMPED-MASS ANALYSES

CASES 14, 15, 16, 17, AND 18



LEGEND

- - - OBSERVED AT LOCATION 5

——— COMPUTED

NOTE: SHEAR MODULUS G FROM VIBRATORY FIELD METHOD WAS USED IN COMPUTATIONS.

ACCELERATION RESPONSE SPECTRA FROM ID FOURIER ANALYSIS METHOD

CASES 19, 20, AND 21

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APPENDIX C: ACCELERATION RESPONSE SPECTRA FROM TWO-DIMENSIONAL ANALYSIS

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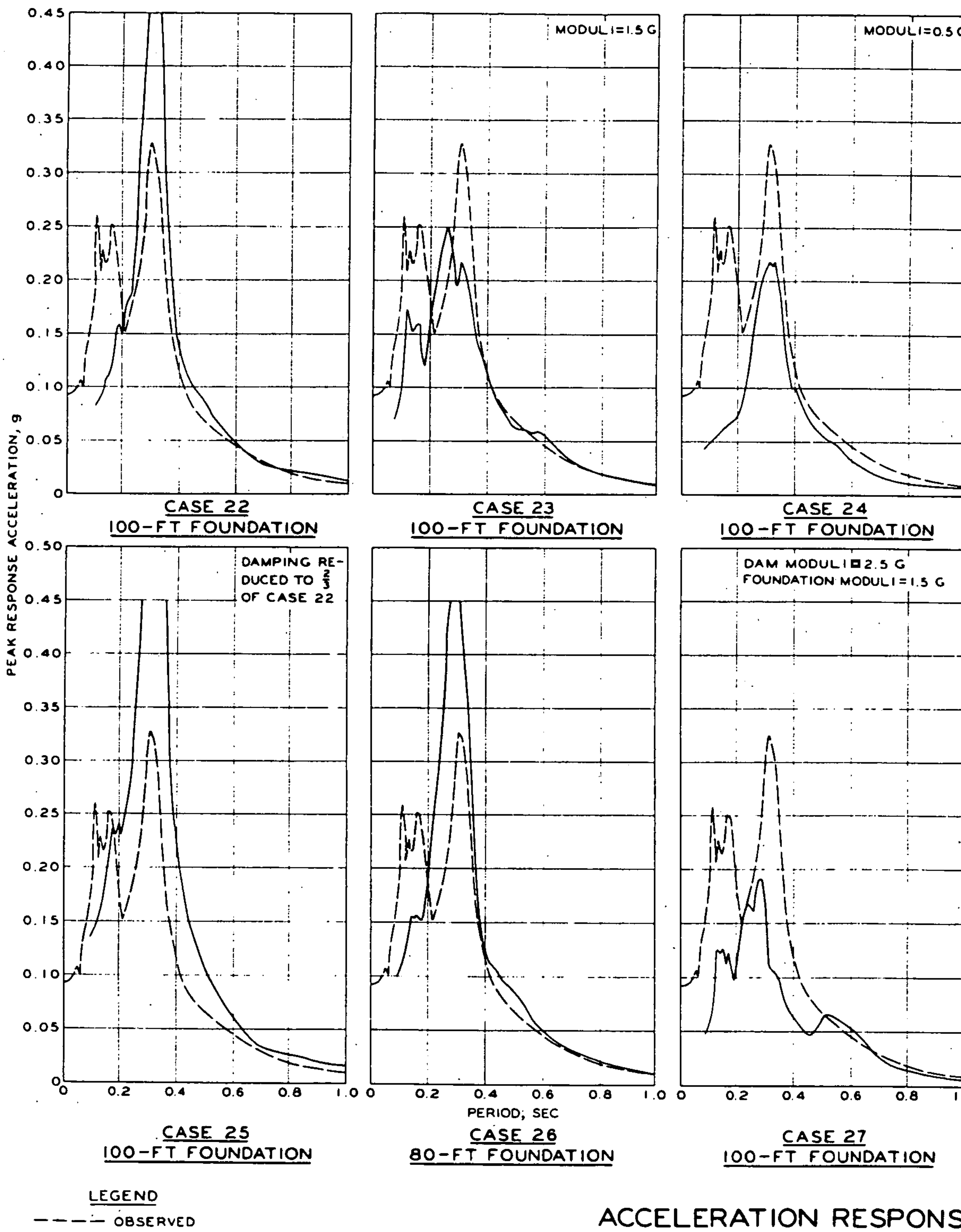
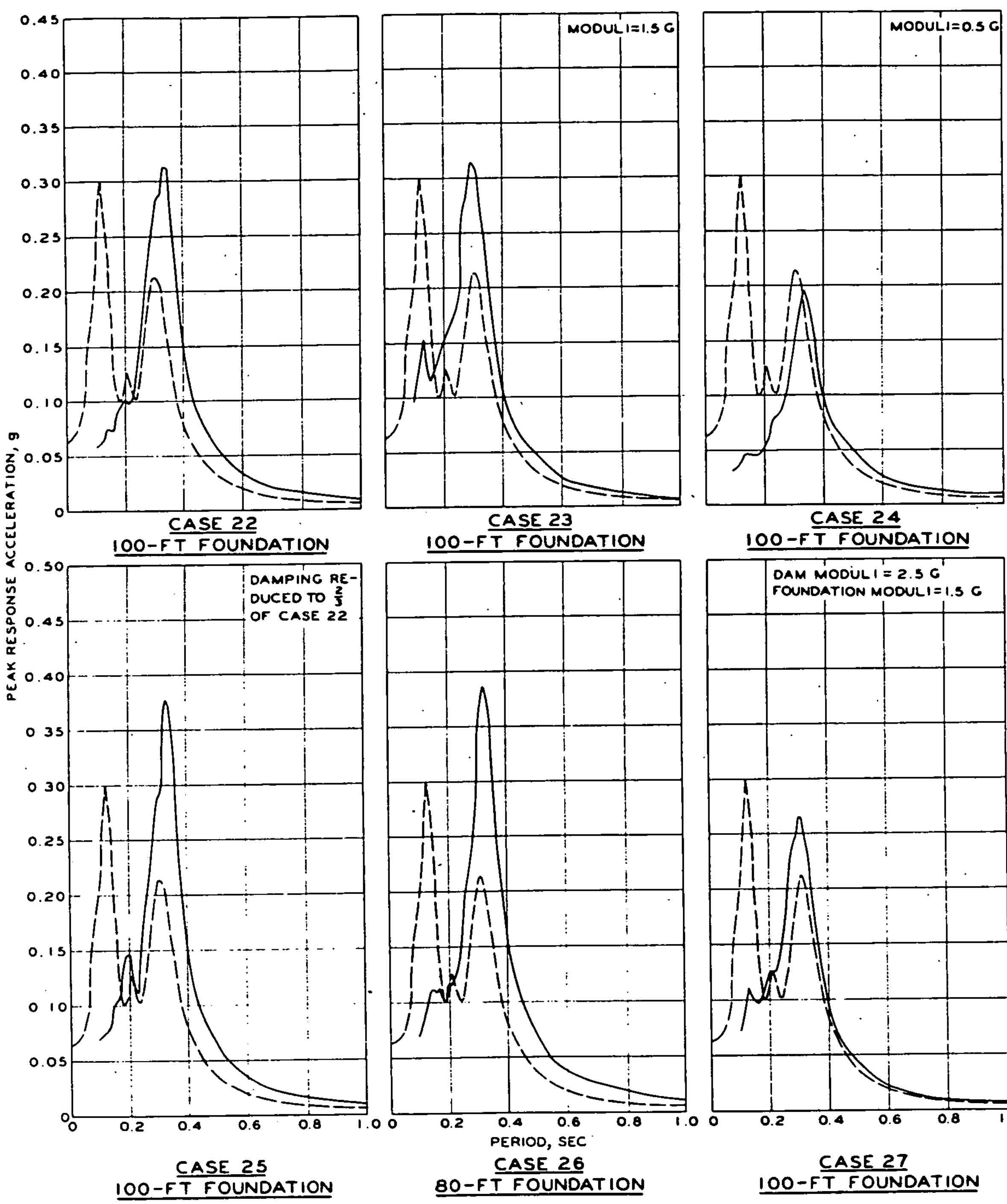
The response spectra, as defined in Appendix A, are given in the plates listed below for the computed and observed responses of the measurement locations at Rifle Gap Dam. The computed spectra are from the 2D analyses, and spectral damping in all plates is 5 percent.

Location	Plate
1R	C1
1V	C2
2R	C3
2V	C4
3R	C5
3V	C6
5R	C7
5V	C8



NOTE: SHEAR MODULUS G FROM VIBRATORY FIELD TECHNIQUE. SAND FOUNDATION WITH 120-FT CLAY EMBANKMENT.

ACCELERATION RESPONSE SPECTRA FROM 2D ANALYSIS
RADIAL COMPONENT
LOCATION 1



LEGEND

---- OBSERVED

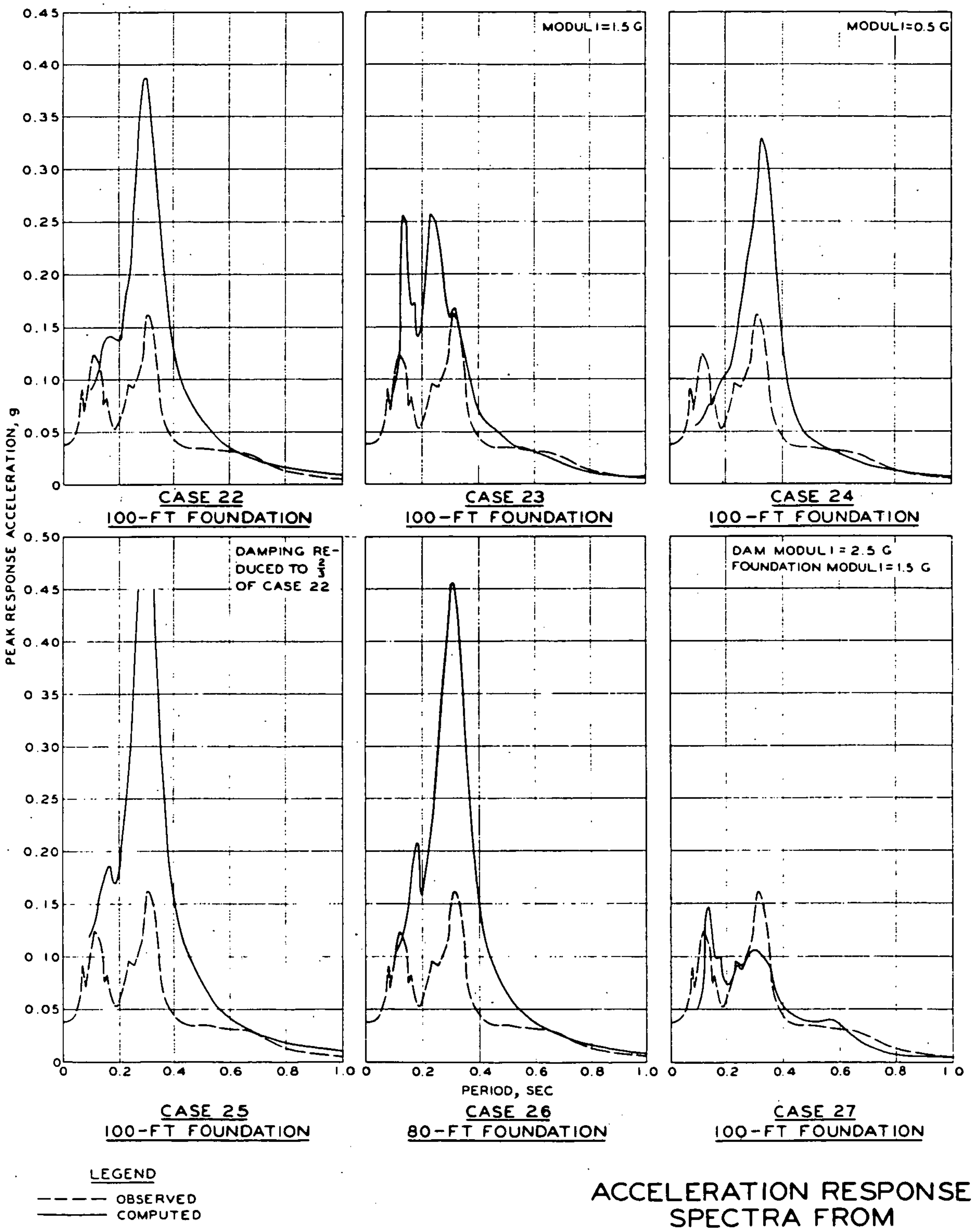
—— COMPUTED

NOTE: SHEAR MODULUS G FROM VIBRATORY FIELD TECHNIQUE. SAND FOUNDATION WITH 120-FT CLAY EMBANKMENT.

ACCELERATION RESPONSE SPECTRA FROM 2D ANALYSIS

VERTICAL COMPONENT

LOCATION 1



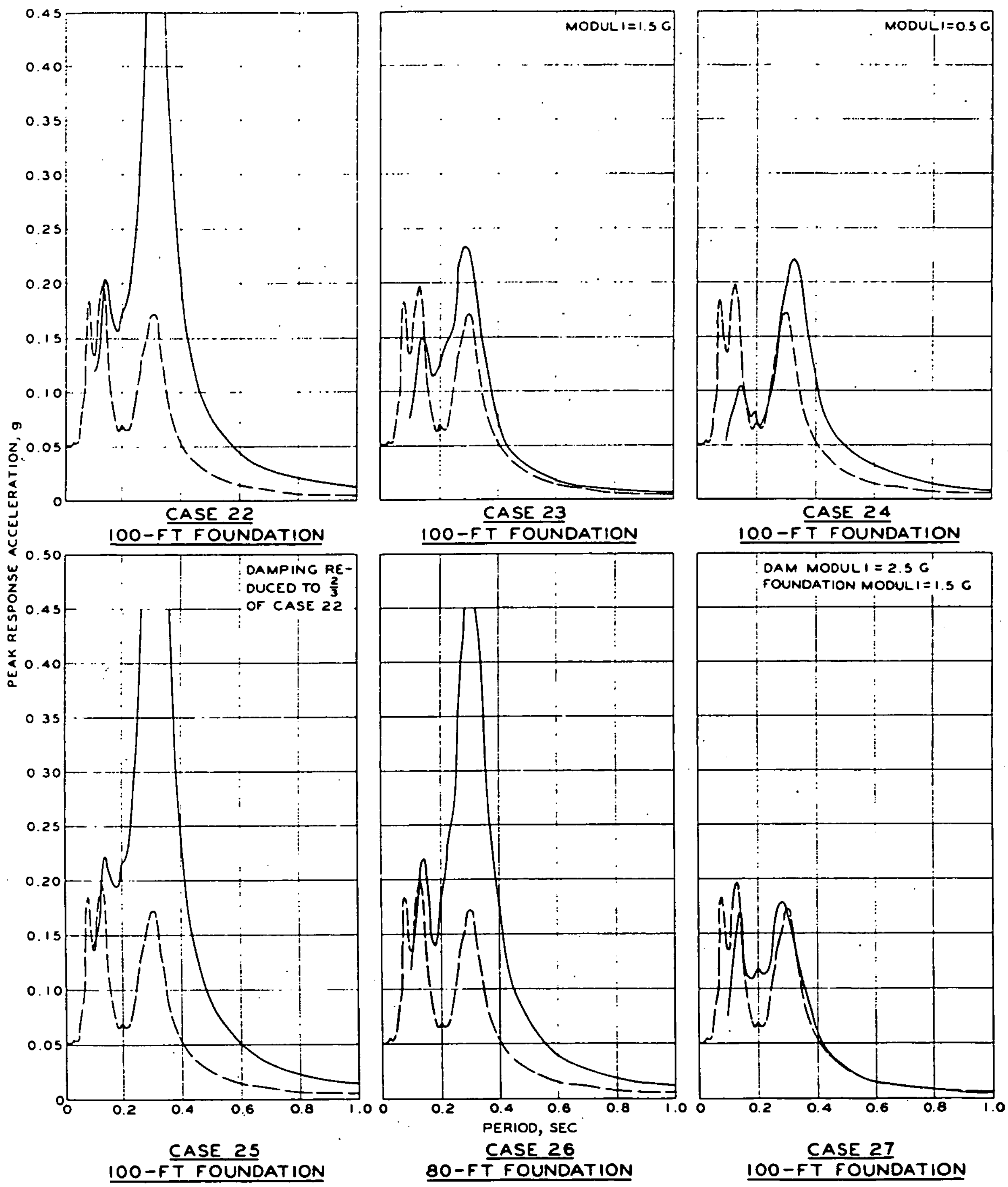
NOTE: SHEAR MODULUS G FROM VIBRATORY FIELD TECHNIQUE. SAND FOUNDATION WITH 120-FT CLAY EMBANKMENT.

ACCELERATION RESPONSE SPECTRA FROM 2D ANALYSIS

RADIAL COMPONENT

LOCATION 2

PLATE C3



LEGEND

- - - - - OBSERVED

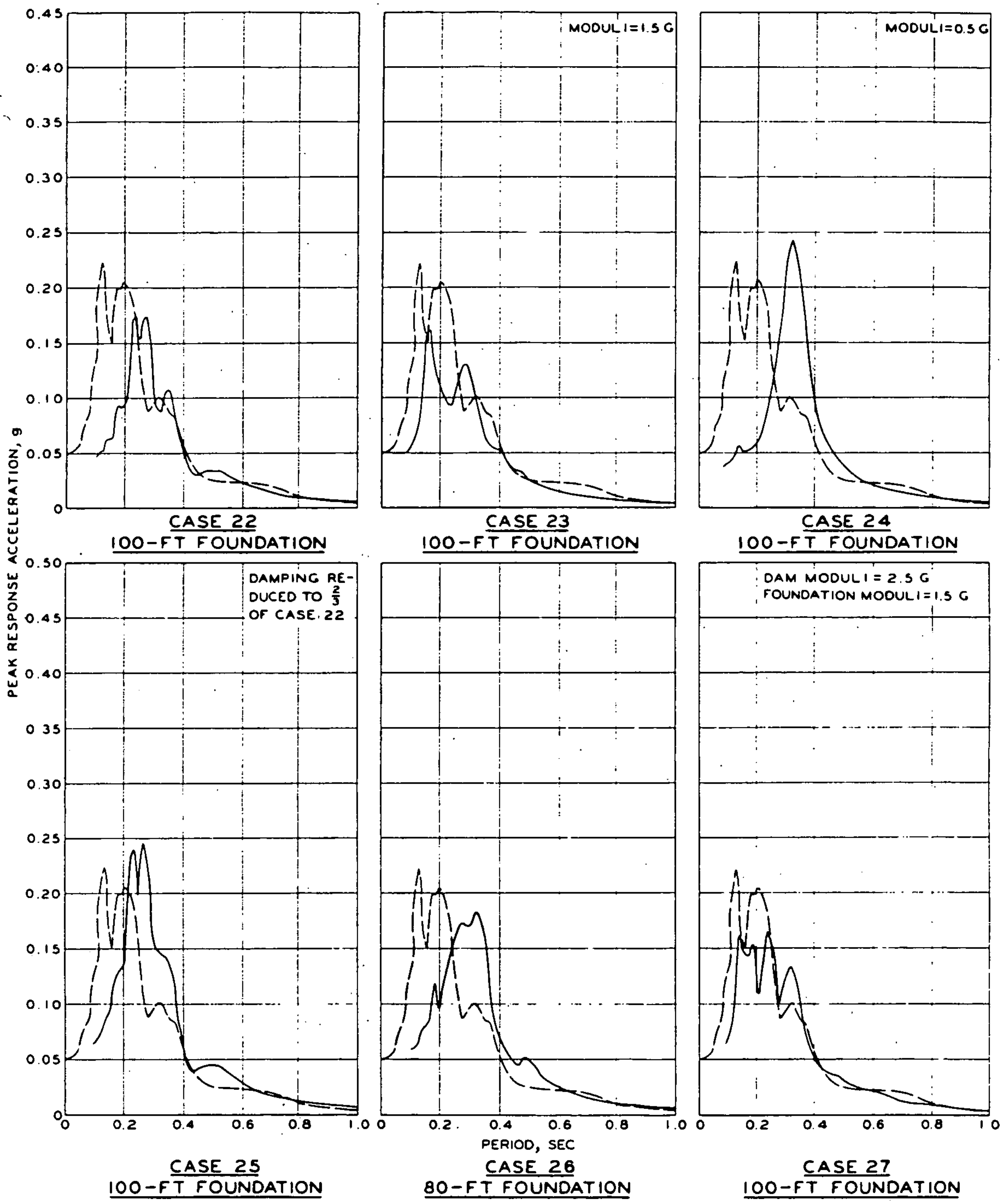
———— COMPUTED

NOTE: SHEAR MODULUS G FROM VIBRATORY FIELD TECHNIQUE. SAND FOUNDATION WITH 120-FT CLAY EMBANKMENT.

ACCELERATION RESPONSE SPECTRA FROM 2 D ANALYSIS

VERTICAL COMPONENT

LOCATION 2



LEGEND

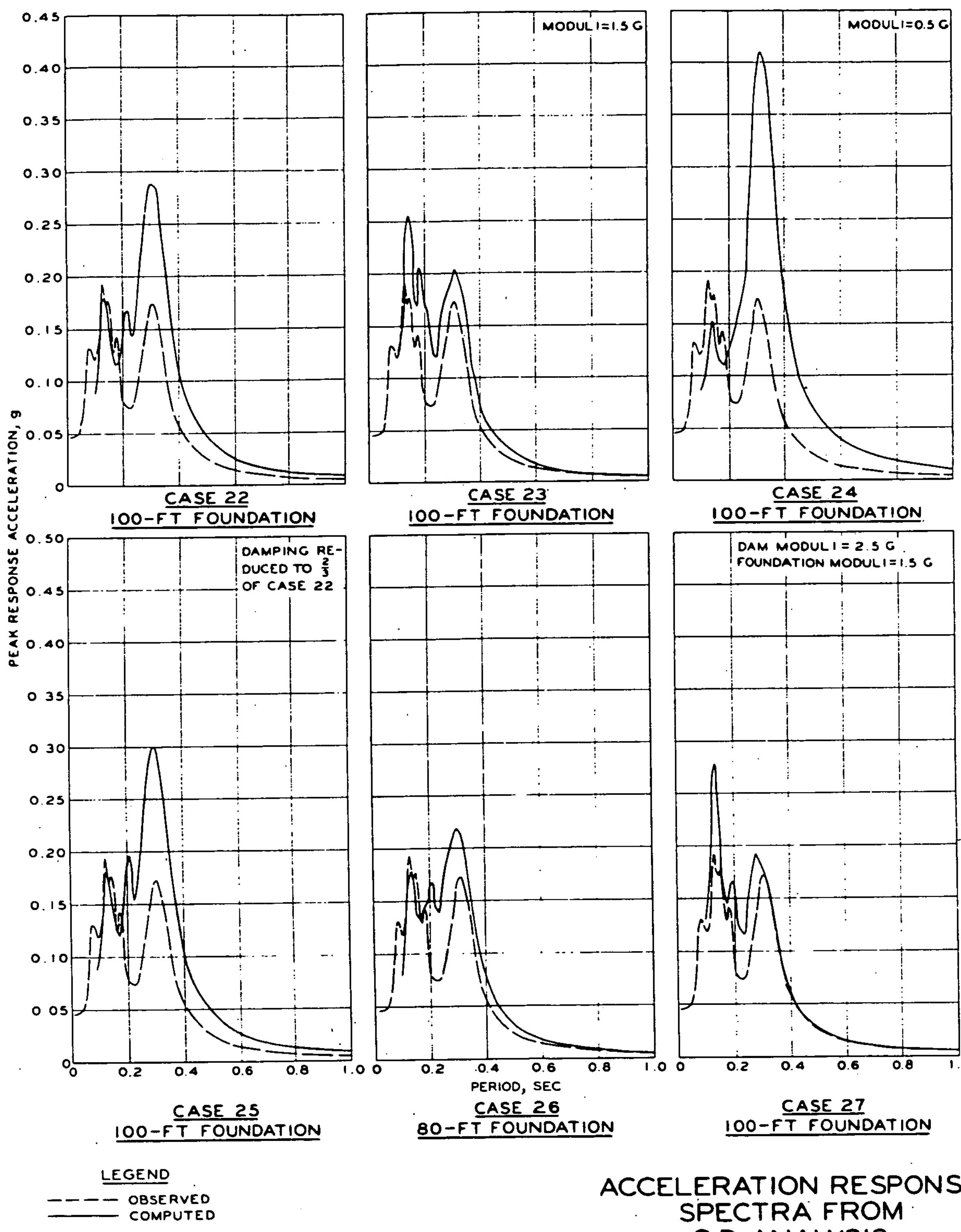
– – – – OBSERVED
——— COMPUTED

NOTE: SHEAR MODULUS G FROM VIBRATORY FIELD TECHNIQUE. SAND FOUNDATION WITH 120-FT CLAY EMBANKMENT.

ACCELERATION RESPONSE SPECTRA FROM 2D ANALYSIS

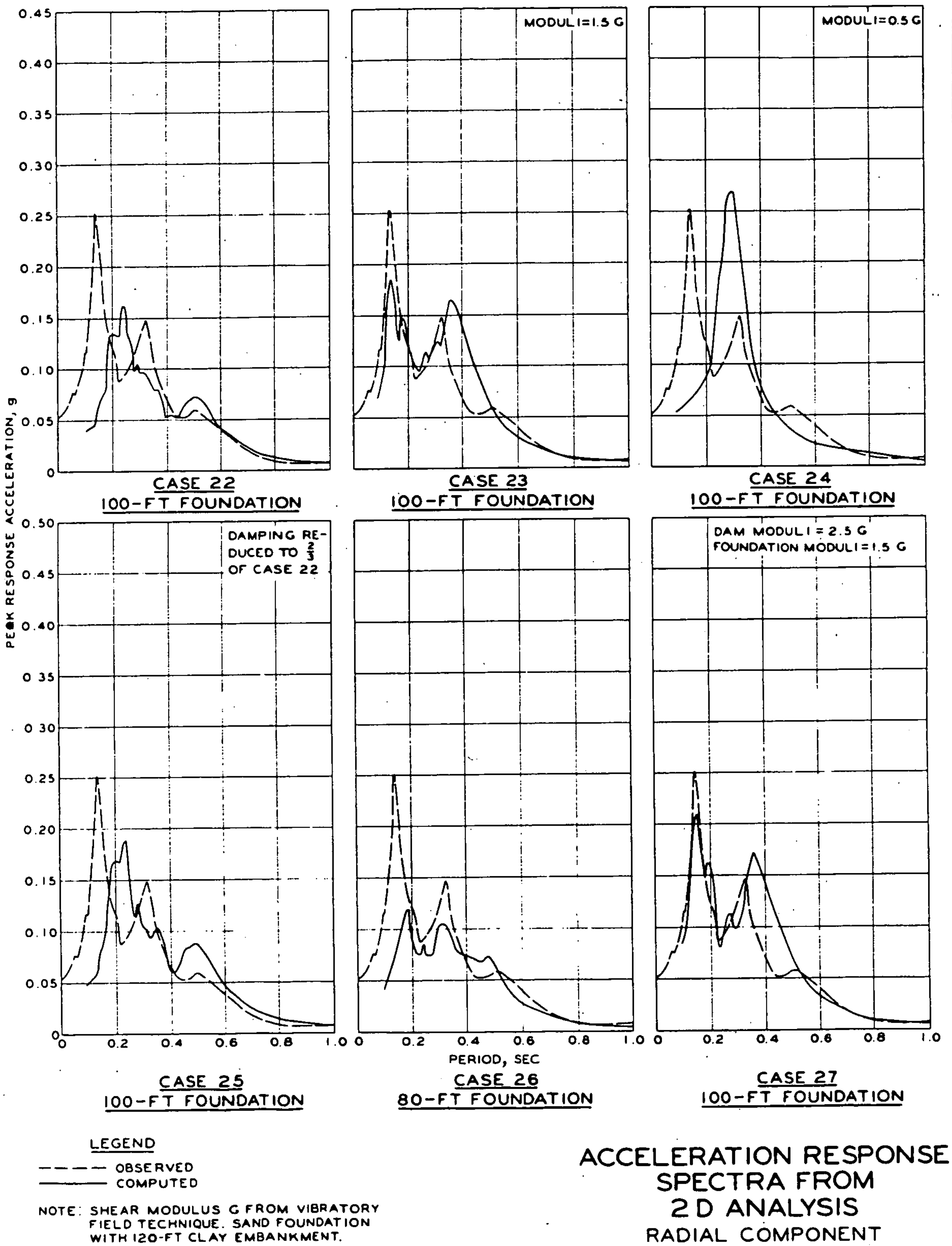
RADIAL COMPONENT

LOCATION 3

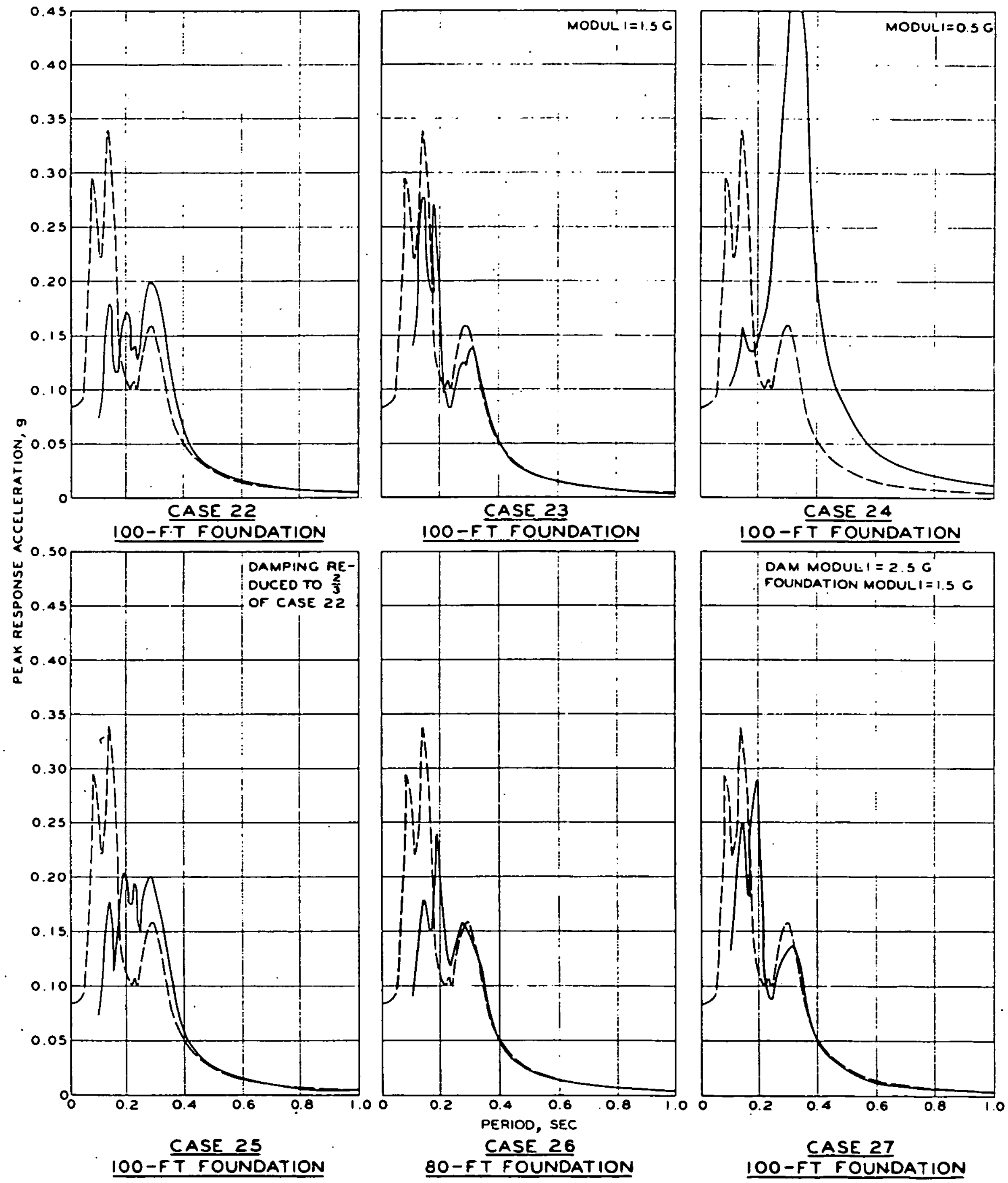


NOTE: SHEAR MODULUS G FROM VIBRATORY FIELD TECHNIQUE. SAND FOUNDATION WITH 120-FT CLAY EMBANKMENT.

ACCELERATION RESPONSE SPECTRA FROM 2D ANALYSIS
VERTICAL COMPONENT
LOCATION 3



ACCELERATION RESPONSE SPECTRA FROM 2D ANALYSIS
RADIAL COMPONENT
LOCATION 5



LEGEND

- - - - OBSERVED
——— COMPUTED

NOTE: SHEAR MODULUS G FROM VIBRATORY FIELD TECHNIQUE. SAND FOUNDATION WITH 120-FT CLAY EMBANKMENT.

ACCELERATION RESPONSE SPECTRA FROM 2D ANALYSIS
VERTICAL COMPONENT
LOCATION 5

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APPENDIX D: SEISMIC FIELD STUDY

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DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION. CORPS OF ENGINEERS
P. O. BOX 631
VICKSBURG. MISSISSIPPI 39180

IN REPLY REFER TO: WESSD

31 March 1971

MEMORANDUM FOR RECORD

SUBJECT: Seismic Field Study, Rifle Gap Dam, Rifle, Colorado, 16-28 November 1970

1. A seismic field study was conducted at Rifle Gap Dam near Rifle, Colorado, during the period 16-28 November 1970. This field study consisted of conventional surface vibratory and refraction seismic tests. In addition to these tests, another seismic test was conducted in which the Rayleigh wave train was recorded. The purpose of these tests was to provide seismic information relative to soil conditions and elastic properties within the Rifle Gap Dam and of the foundation on the downstream side of the dam. Specifically, compression-wave velocities, depth to interfaces, bedrock configuration, and shear wave velocities as a function of depth were to be determined from the field study. Another purpose of the seismic tests in which the Rayleigh wave train was recorded was to determine Rayleigh wave velocities with depth (Rayleigh wave dispersion method) so that a comparison and evaluation of data could be made between the Rayleigh wave dispersion method and the surface vibratory method.

2. Messrs. F. K. Chang, M. M. Carlson, and J. R. Curro, Jr., visited the site to perform the subject study. Prior to arriving at the test site, mechanical difficulties were encountered with the instrumentation vehicle in Denver and Frisco, Colorado. These mechanical breakdowns caused a time loss of some four days. When the instrumentation vehicle was repaired, Messrs. Chang and Curro proceeded to Grand Junction, Colorado, to meet with our contact, Mr. Bill McCleneghan, Bureau of Reclamation. Mr. Carlson drove the instrumentation vehicle to the Rifle Gap Dam test site. Mr. McCleneghan was presented with a plan of tests which were to be performed at the test site. He approved the plan of tests, but stipulated that charge sizes be limited to 2 lb and detonated in shotholes less than 5 ft deep. He also requested that any holes caused by the detonation of explosives be backfilled. During the course of conversation, Mr. Curro asked about water levels in piezometers that were located on the downstream side of the dam about 100 ft from the toe. Mr. McCleneghan stated that approximately six weeks prior to 20 November, all piezometer pipes were overflowing. At the present time (20 November), water levels in the piezometers were about 2-3 ft above the ground surface toward the west end of the dam and 2-3 ft below the ground surface toward the east end of the dam.

Mr. McCleneghan also obtained permission from the Colorado State Highway Department for the conduct of a vibratory test on State Highway 325 which traverses the crest of the dam.

3. After meeting with Mr. McCleneghan, the WES contingent went to the Rifle Gap Dam test site, met Mr. Carlson, and made a visual reconnaissance of the site. No seepage from the embankment and no piping or sand boils from the foundation were observed. Mr. S. W. Guy, Instrumentation Branch, WES, joined the field party 20 November for the conduct of the vibratory tests and returned to WES on 23 November.

4. Seven refraction seismic traverses, seven seismic traverses for the Rayleigh wave dispersion method, and three vibratory traverses were run at the Rifle Gap Dam site. Refraction seismic traverses S-1 through S-6 are shown in Incl 1 and S-7 is shown in Incl 2. The seismic traverses for the Rayleigh wave dispersion method were located in the same position as the refraction seismic traverses. The vibratory traverses (V-1 through V-3) were located as shown in Incl 3.

5. The data obtained from the refraction seismic tests (traverses S-1 through S-7) are shown in the time versus distance plots, Incls 4-7. The time versus distance plots were used to construct subsurface profiles for the seismic data. The subsurface profile is shown in Incl 8 for traverses S-1 and S-2, in Incl 9 for traverses S-3 and S-4, and in Incl 10 for traverses S-5 and S-6. A subsurface profile for traverse S-7 could not be constructed because it was shot in only one direction.

6. The seismic data (Incls 7-10) indicated one minor and three major velocity zones. The minor velocity zone was near the surface of the ground with a maximum thickness of 4 ft and had velocities that ranged from 1000 to 1200 fps. The three major velocity zones were 1400 to 2500 fps with a maximum thickness of about 37 ft, 4800 to 8600 fps with a maximum thickness of about 116 ft, and 10,800 to 12,500 fps for the bedrock velocity zone. Bedrock was encountered at a maximum depth of 130 ft below the ground surface.

7. Data obtained from the vibratory traverses are plotted as number of waves versus distance from which shear (Rayleigh) wave velocities are determined. These plots are shown for traverses V-1 through V-3 in Incls 11-13, respectively. From the data taken from the number of waves versus distance plots, surface wave velocity is simply calculated as wavelength times frequency. Assuming that the surface wave velocity is equal to the shear wave velocity and is applicable at a depth equal to one-half the wavelength, plots of shear wave velocity versus depth were prepared for traverses V-1 through V-3, as shown in Incls 14-16, respectively. For traverse V-1, the shear wave velocity generally

increased from 560 fps at a depth of 7 ft to 750 fps at 93.5 ft. A slightly lower velocity of 495 fps at a depth of 11 ft was noted. The shear wave velocity for traverse V-2 increased from 625 fps at a depth of 6.5 ft to 1085 fps at 108.5 ft. The shear wave velocity for traverse V-3 increased from 700 fps at a depth of 7 ft to 960 fps at 32 ft, then showed a sharp decrease in velocity to 805 fps at 33.5 ft. The shear wave velocity increased again to 1090 fps at 90 ft. The data points at 97 ft and 117.5 ft are questionable because of signal quality.

8. The plots of shear modulus, G, versus depth for traverses V-1 through V-3 are shown in Incls 17-19, respectively. Shear modulus is equal to the shear wave velocity squared times the mass density. For traverses V-1 and V-2, a wet unit weight of 120 pcf was used above 50 ft and 130 pcf below 50 ft. For traverse V-3, a wet unit weight of 135 pcf was used for all depths. The shear modulus ranged from 6350 psi to 33,800 psi as shown in Incls 17-19.

9. Only the results of the Rayleigh wave velocity seismic tests conducted along traverses S-1 and S-2 and along traverse S-7 are presented in this memorandum. The other two seismic tests crossed over a creek about 10 ft deep and the Rayleigh wave energy was not well detected which may indicate that a trench could be detrimental to the normal propagation of Rayleigh wave energy. There were also other problems encountered in obtaining good data from the Rayleigh wave dispersion seismic tests. The most difficult problem was obtaining usable amplitudes of the Rayleigh wave train. Amplifier gains are critcal and must be adjusted for a particular charge size. If the gains are too high, the data traces will exceed the oscillogram width and be lost. Conversely, if the gains are too low, the Rayleigh wave train will not be detected. This procedure caused a number of shots to be repeated. Another problem is the error introduced in the data caused by shooting in or near the original disturbed shothole. When the initial charge is detonated, it produces a cavity in the soil, thus causing another charge detonated in the same shothole to produce data somewhat different from the original shot.

10. The results of the Rayleigh wave seismic test along traverses S-1 and S-2 are shown in Incl 20 along with the data from vibratory traverse V-1. The Rayleigh wave velocity from this test is determined as the distance between two geophones divided by the time required to travel between the same two geophones. The Rayleigh wave velocities are about two to three times higher than the shear wave velocities determined from the vibratory data. It should be noted that data from two boring logs have been included in Incl 20. Data from boring DH21, which indicates two distinct boundaries at depths of 56 and 84 ft,

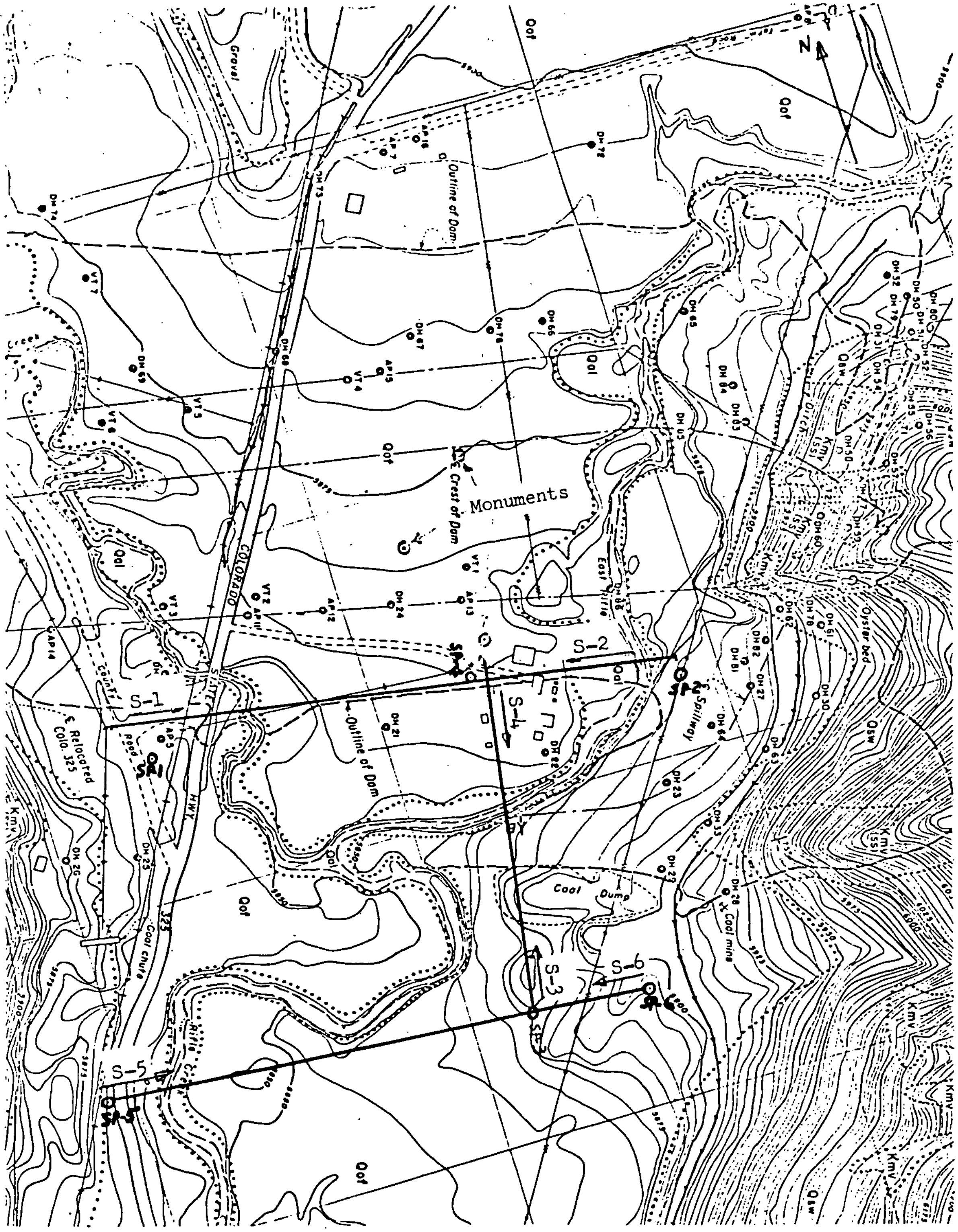
correlate well with the Rayleigh wave velocity profile. However, other borings in the area, such as boring DH22 (Incl 20), indicate a difference as regards depths at which certain materials are encountered. The results of the Rayleigh wave seismic test along traverse S-7, which was shot up the face of the dam, indicated that the direct compression and shear wave velocities obtained were 4000 and 1900 fps, respectively, for the embankment as shown in Incl 21. The average shear wave velocity determined from the vibratory data (traverse V-3) was about 900 fps and the compression-wave velocity was 4800 fps from traverse S-7.

11. Thus far, reasons for differences between surface, vibratory, and Rayleigh wave dispersion data have not been determined. It is suggested that the vibratory shear wave velocity data be used as a lower bound and the maximum Rayleigh wave velocity data be used as the upper bound for the material property description.

R. F. Ballard, Jr. for
J. R. CURRO, JR.
Geophysicist
Vibratory Loads Section

21 Incl
as

CF w/incl:
Mr. S. J. Johnson



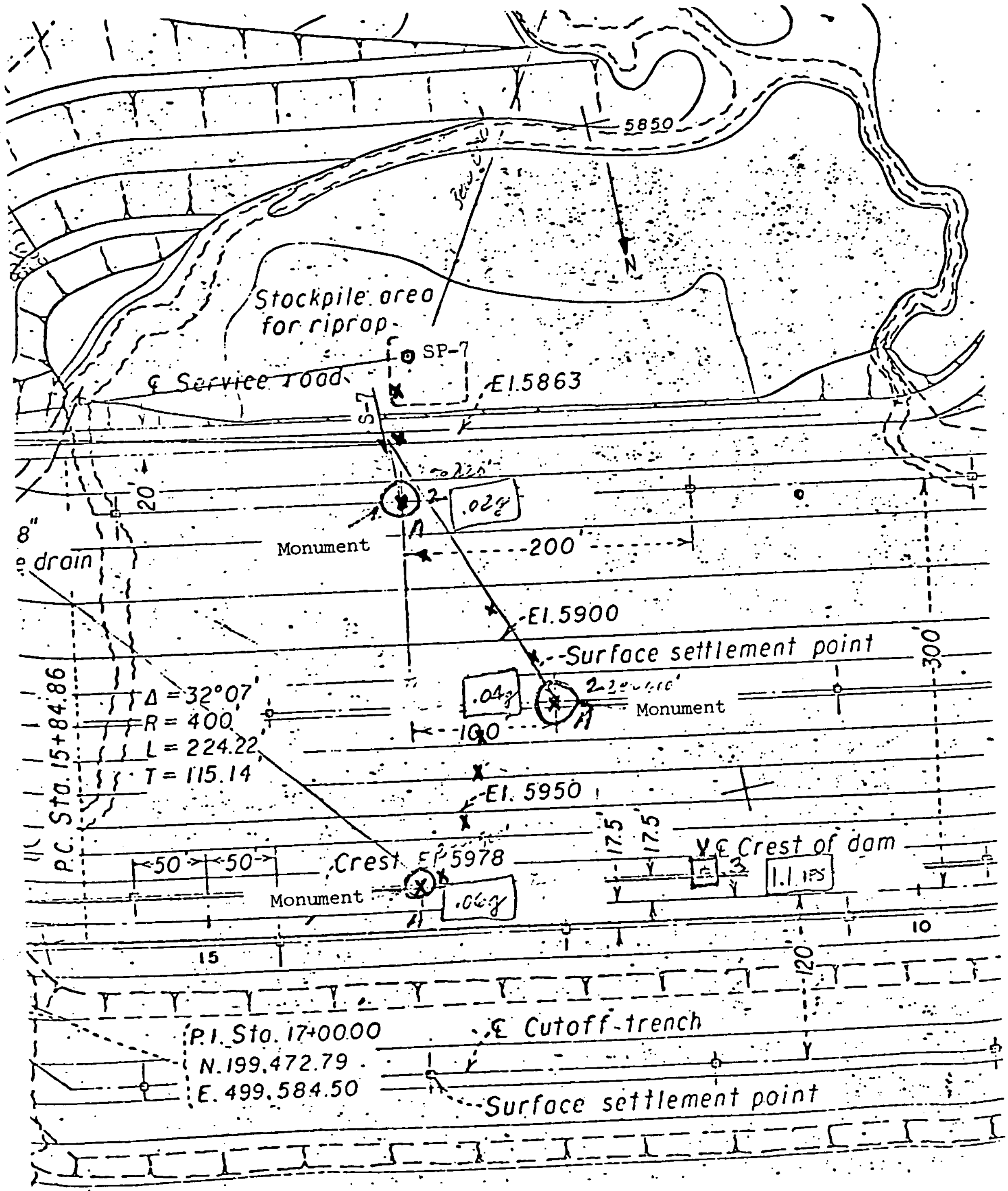
⊙ SP-1 Shotpoint

S-1 ⟶ Seismic traverse no. and direction

1" = 200'

SEISMIC TEST LAYOUT
Rifle Gap Dam, Colorado

Inclosure 1

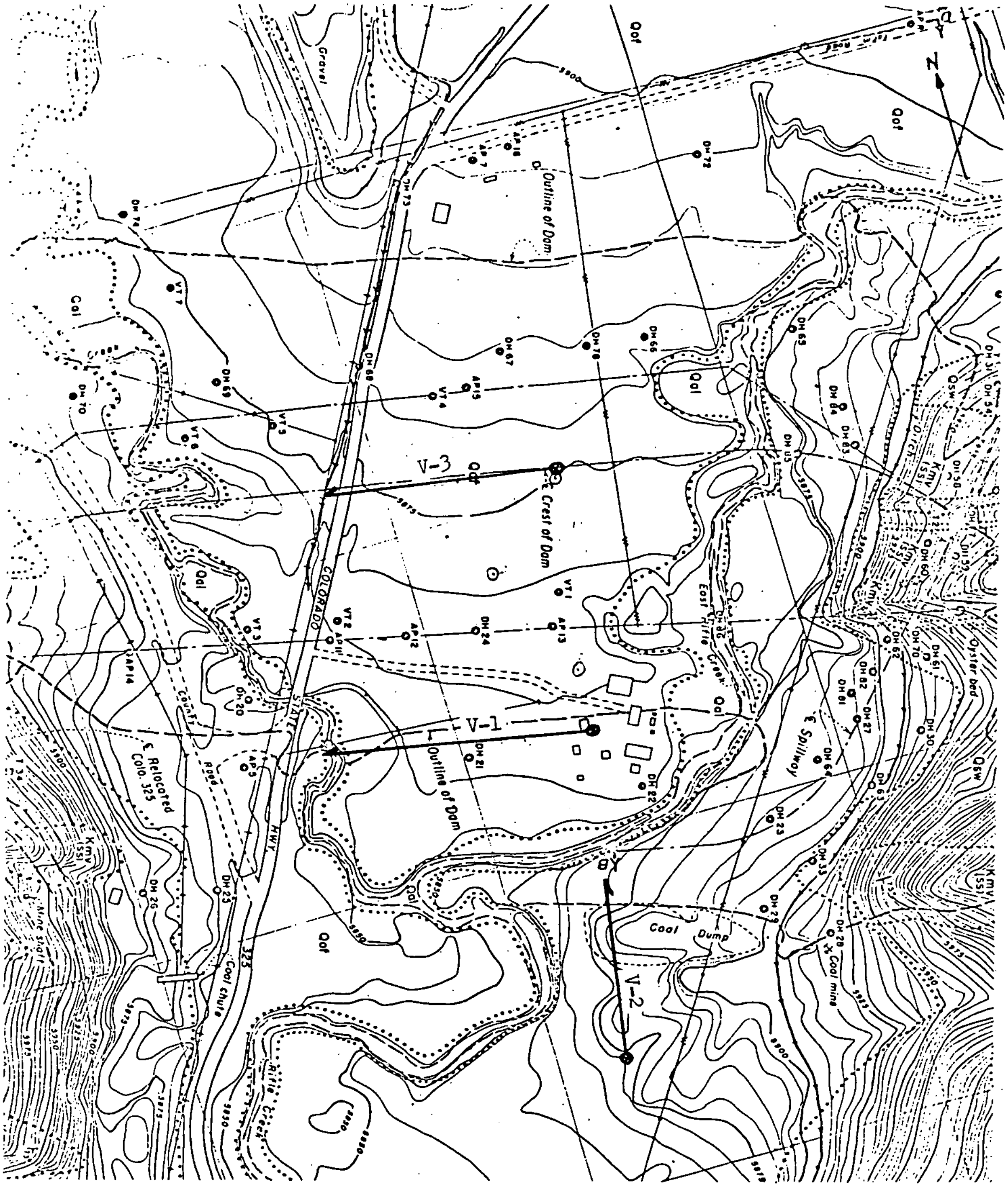


⊙ Shotpoint
X Geophones

1" = 100 ft.

SEISMIC TEST LAYOUT
Traverse S-7 (up face of dam)
Rifle Gap Dam, Colorado

Inclosure 2



⊗ Vibrator location

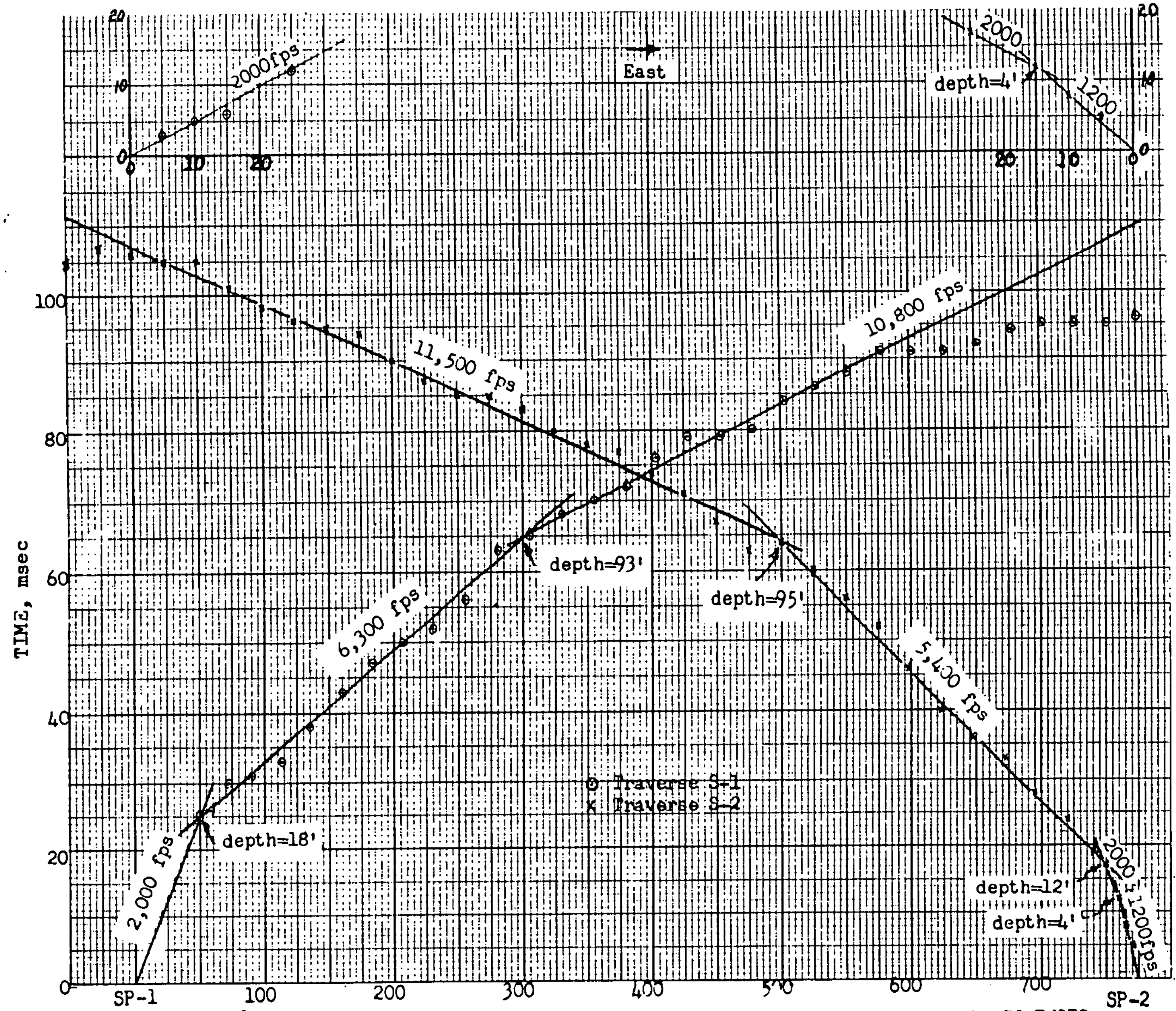
V-1 → Vibration traverse no. and direction

1" = 200 ft.

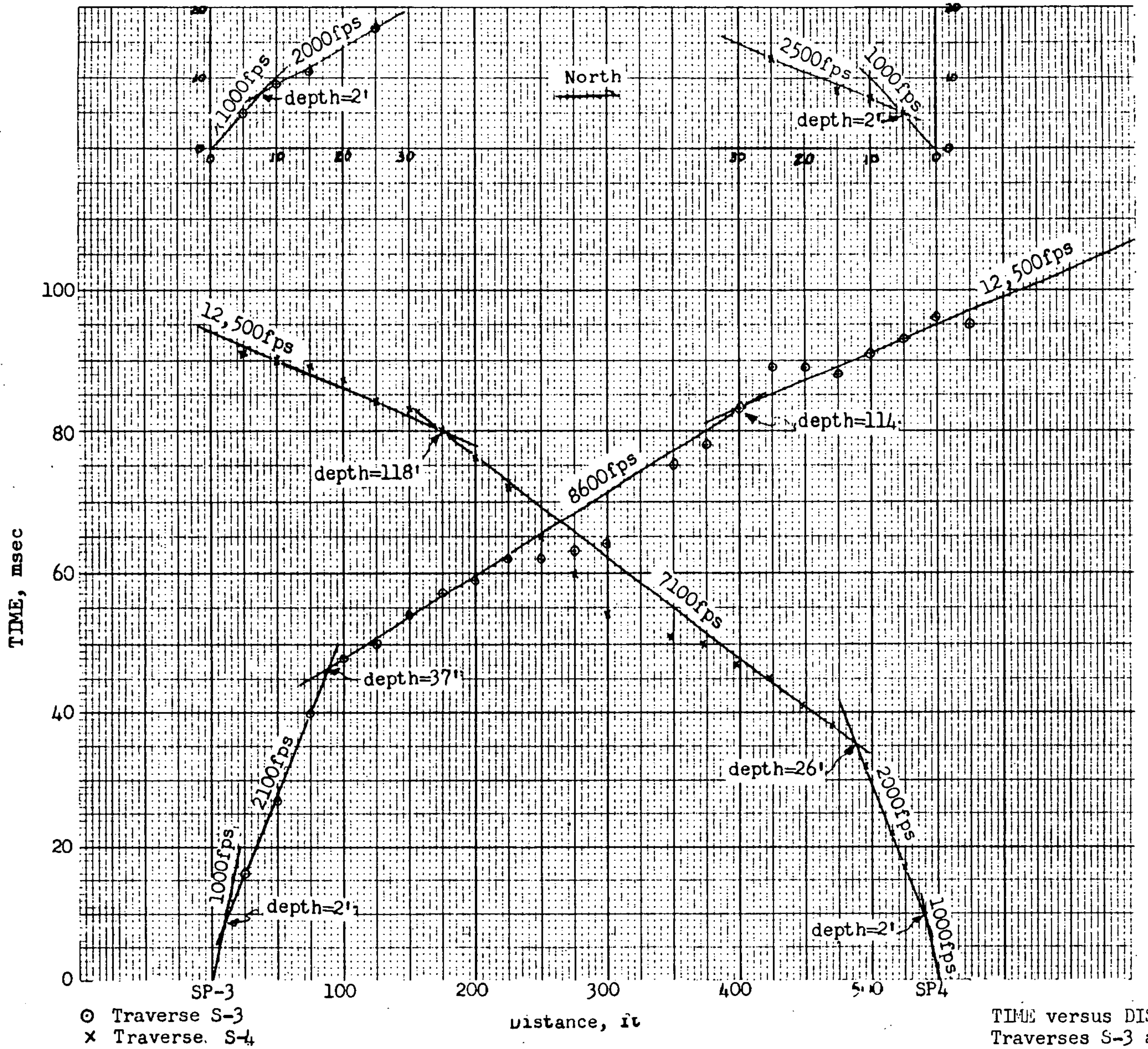
VIBRATORY TEST LAYOUT
Rifle Gap Dam, Colorado

Inclosure 3

Inclosure 4



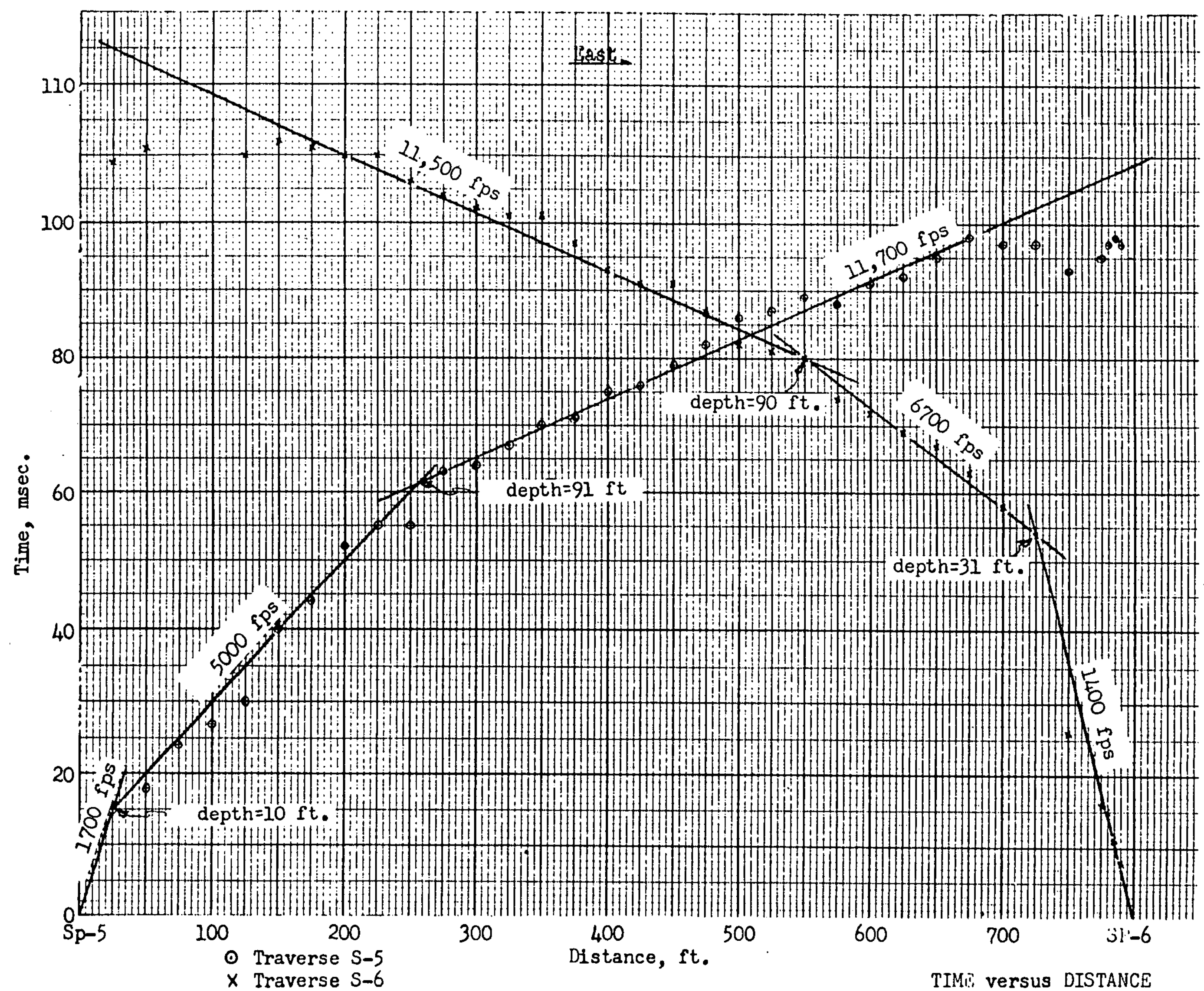
VELOCITY DETERMINATION FROM SEISMIC TESTS
Traverses S-1 and S-2



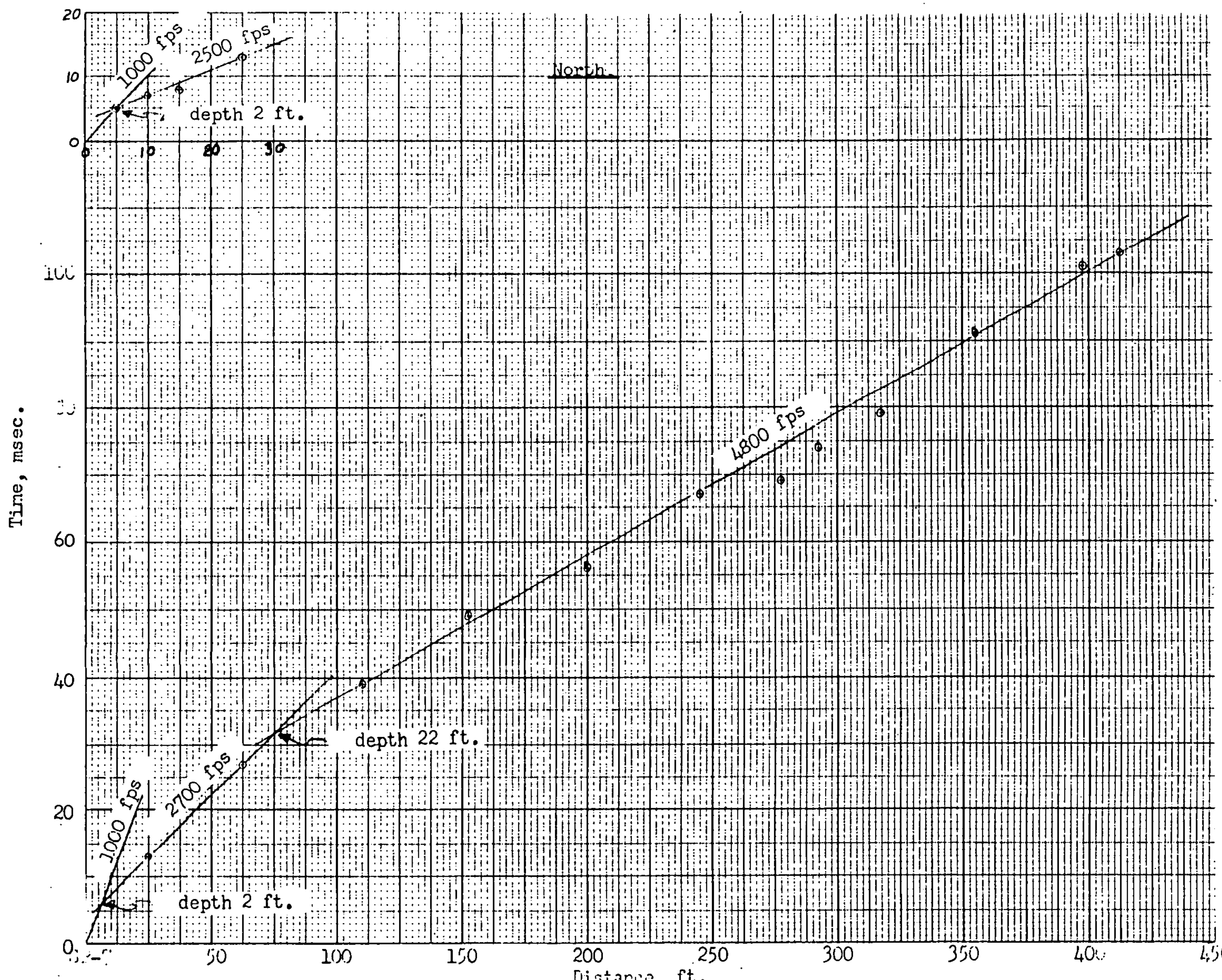
TIME versus DISTANCE
Traverses S-3 and S-4

Inclosure 5

Inclosure 6



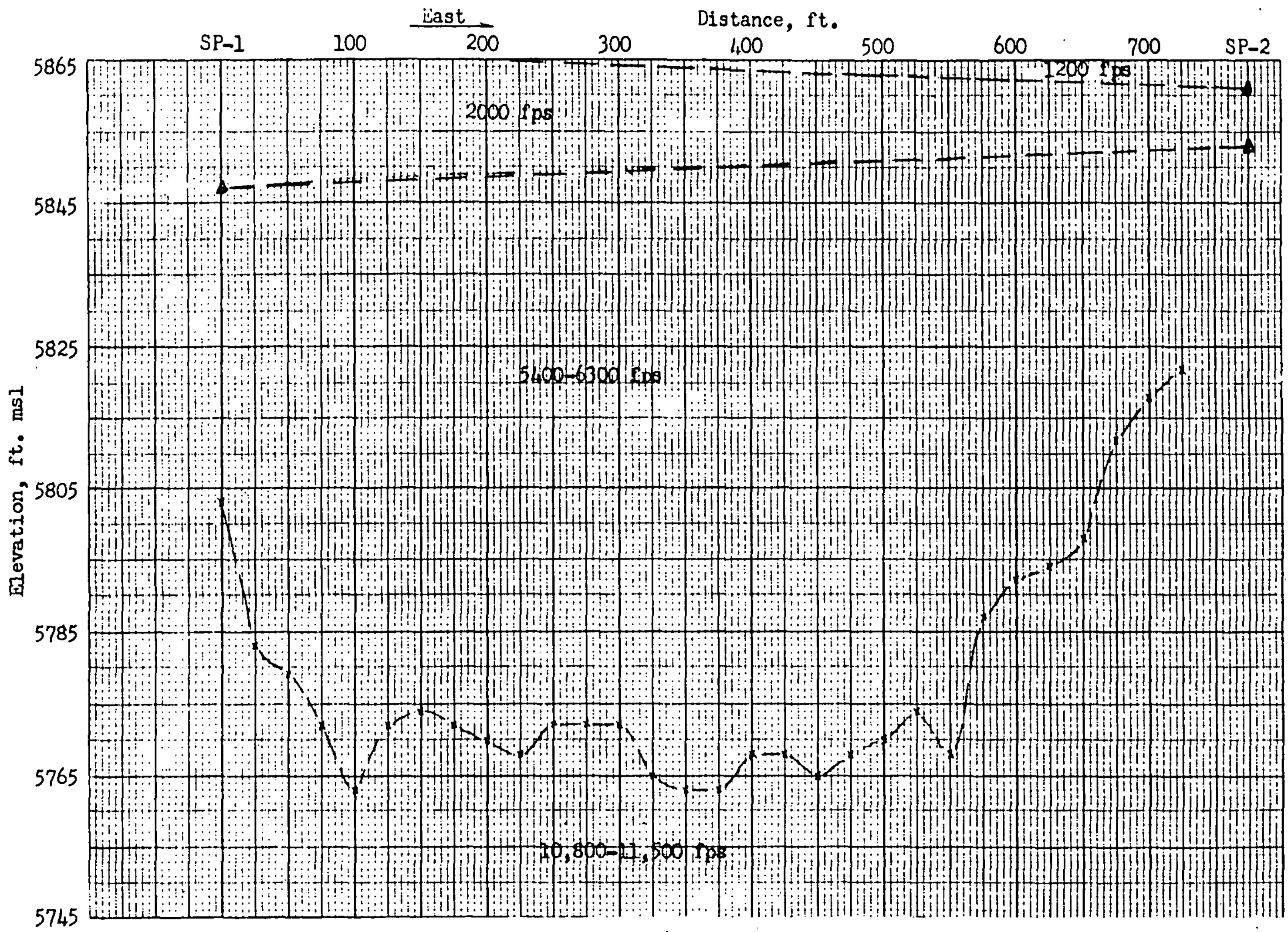
TIME versus DISTANCE
Traverses S-5 and S-6



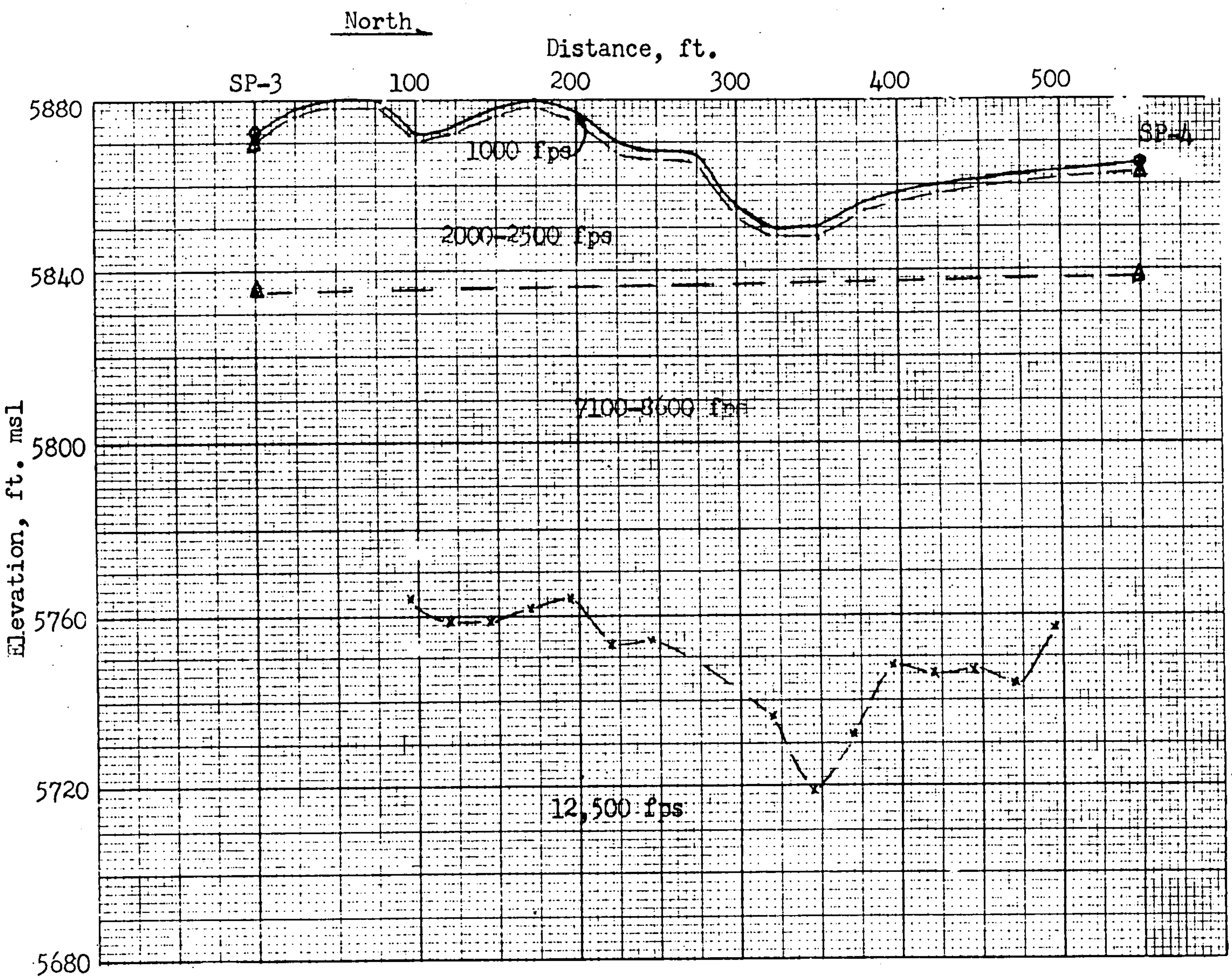
TIME versus DISTANCE
Traverse C-7 (up face of dam)

Inclosure 7

Inclosure 8



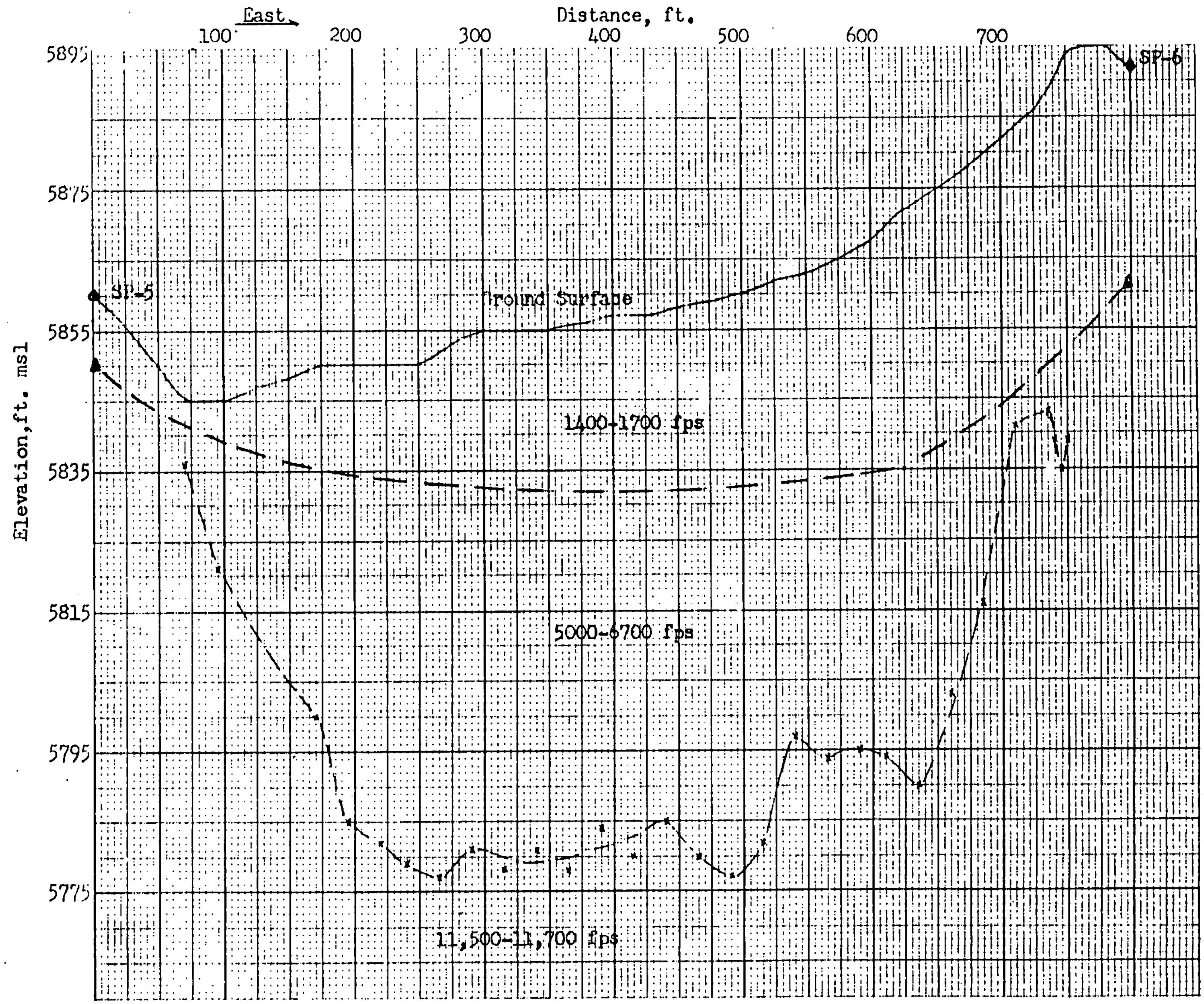
APPROXIMATE PROFILE OF SUBSURFACE MATERIALS
Traverses S-1 and S-2



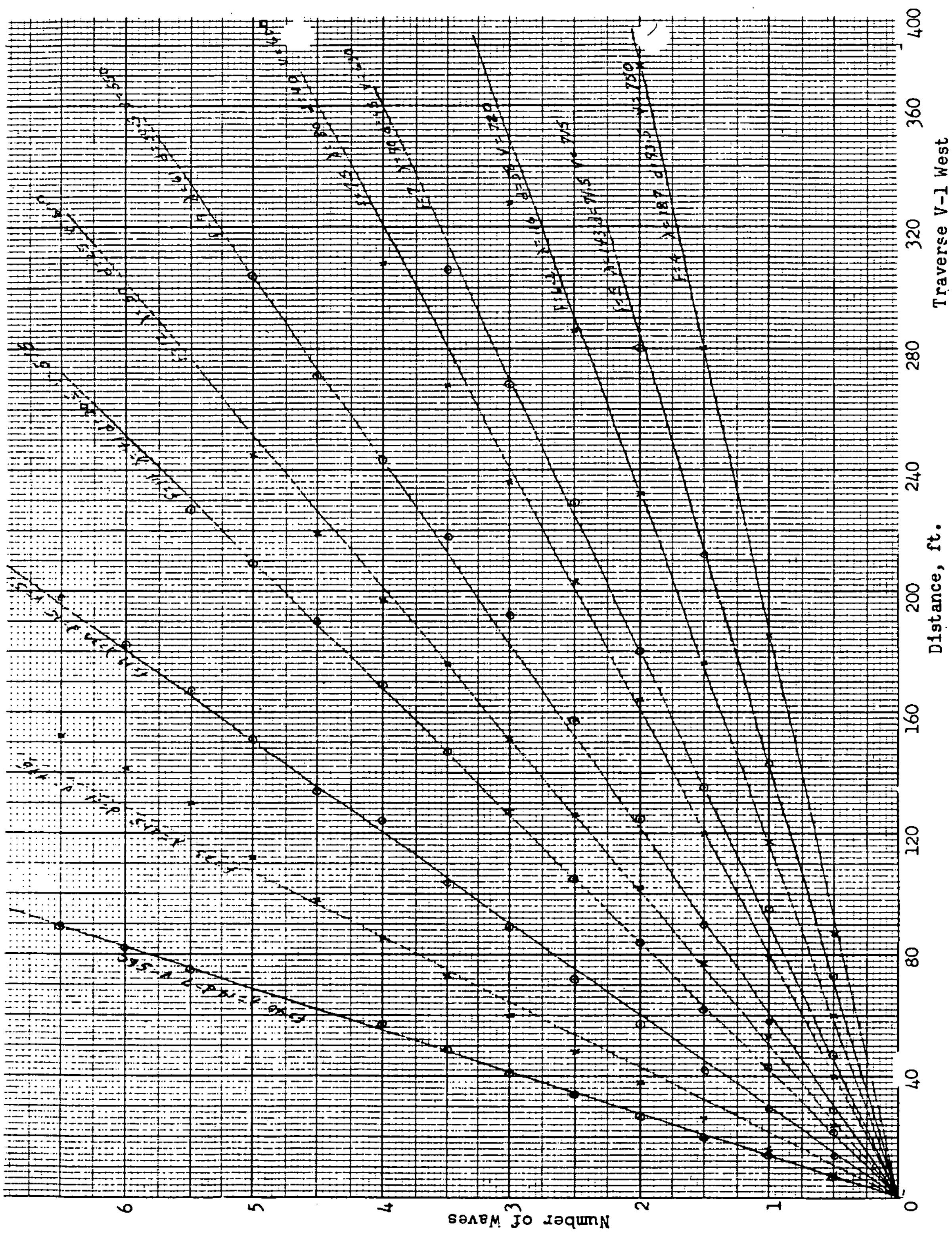
APPROXIMATE PROFILE OF SUBSURFACE MATERIALS
Traverses S-3 and S-4

Inclosure 9

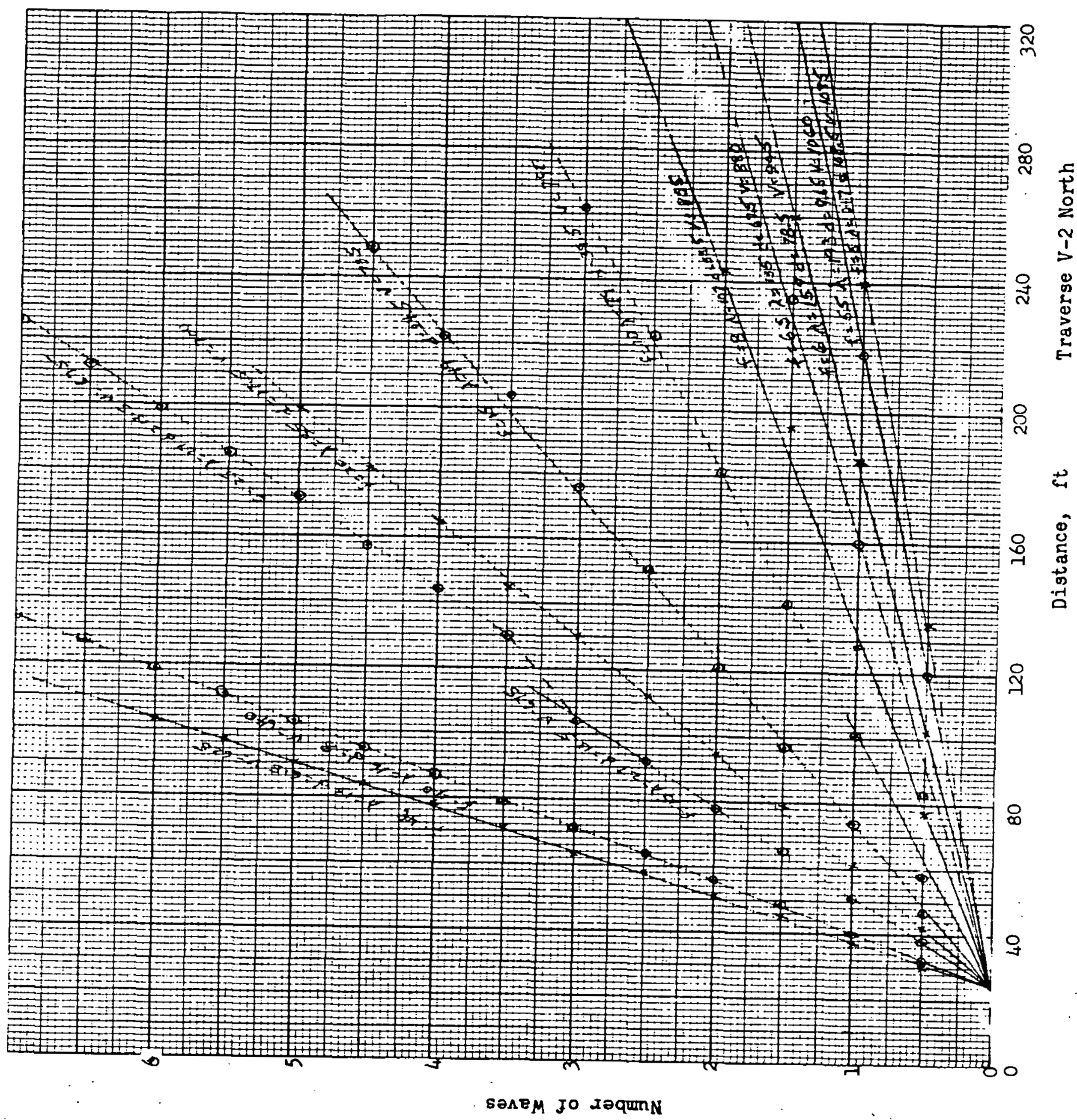
Inclosure 10



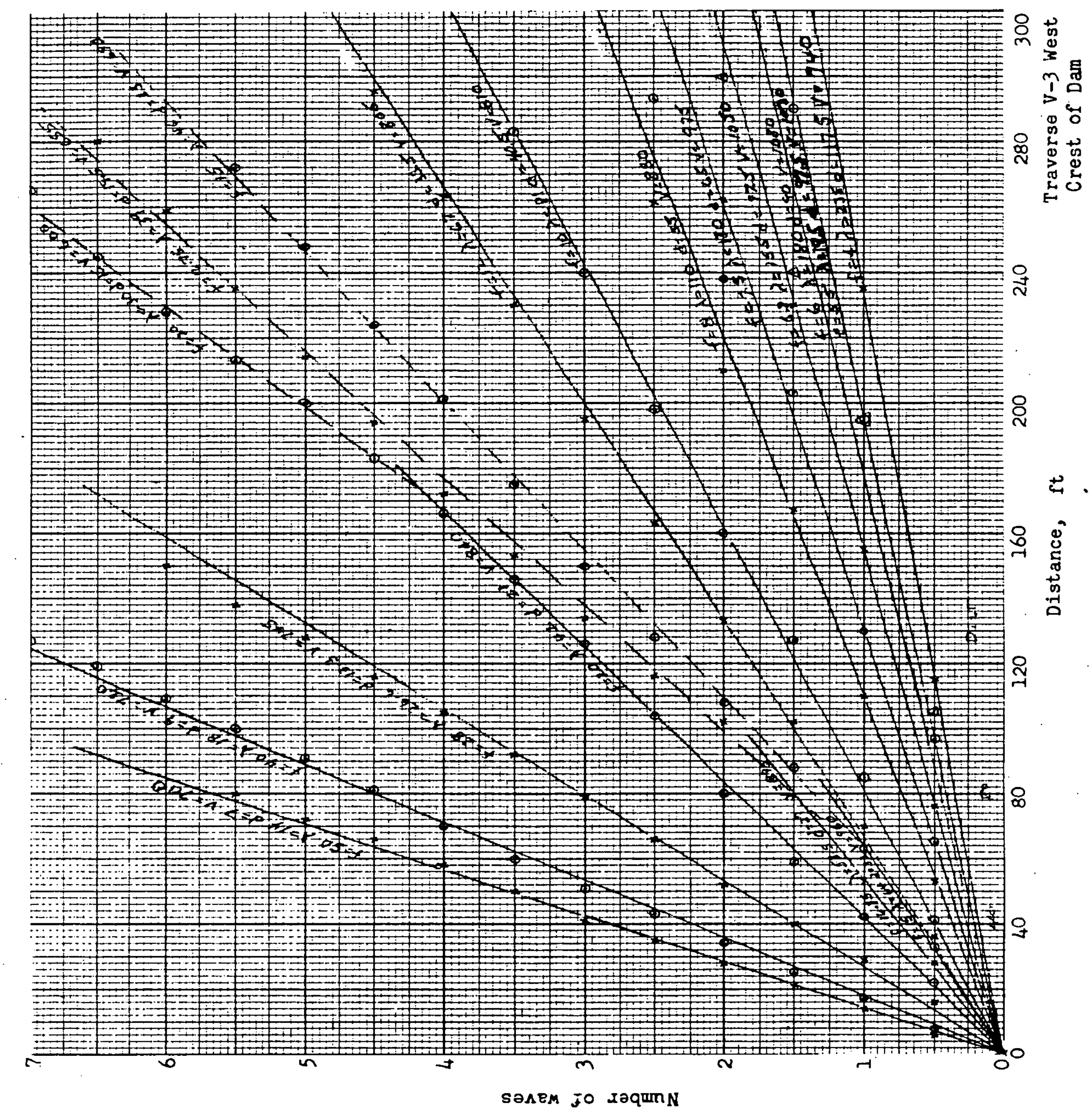
APPROXIMATE PROFILE OF SUBSURFACE MATERIALS
Traverses S-5 and S-6



Inclosure 11

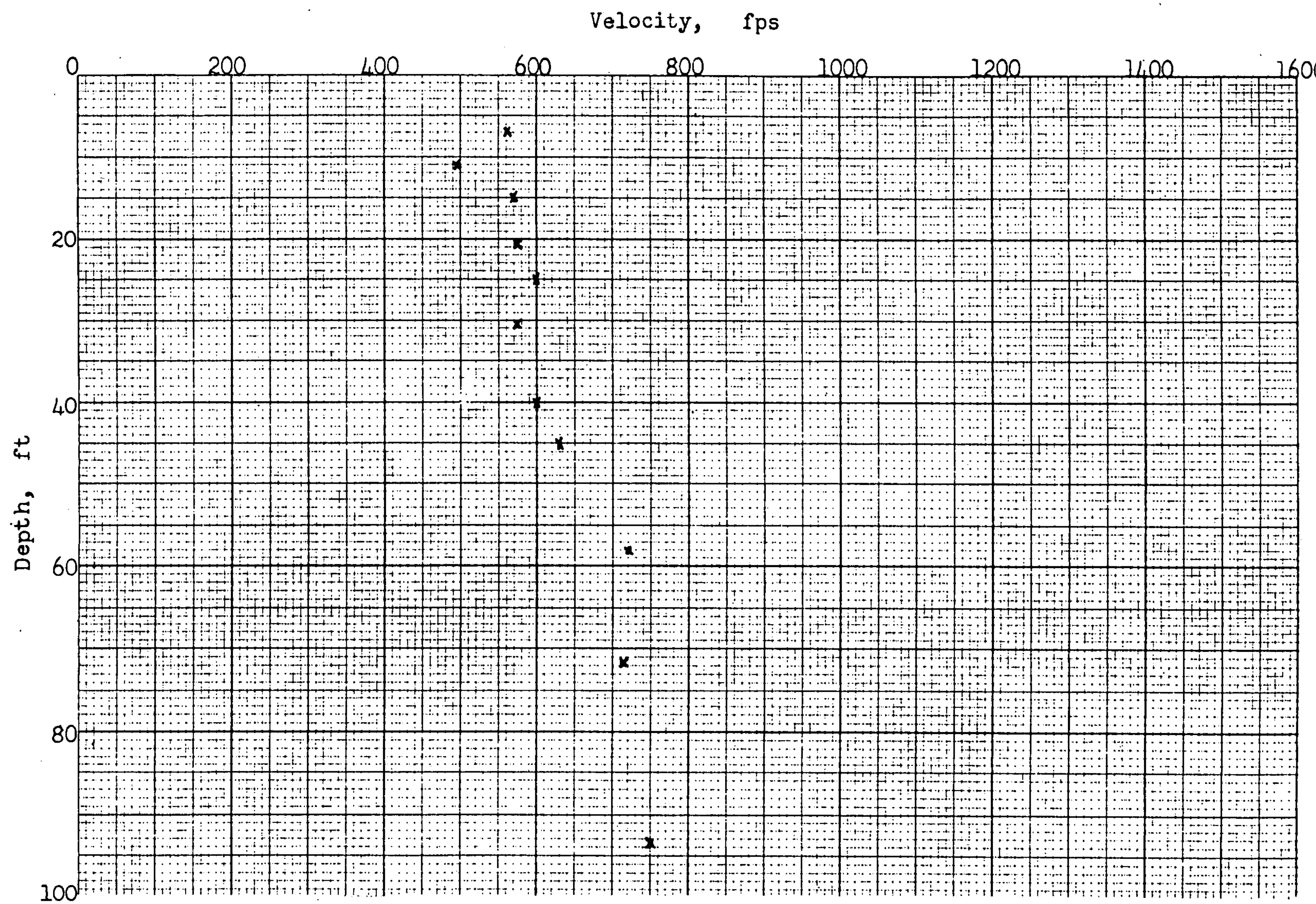


Inclosure 12



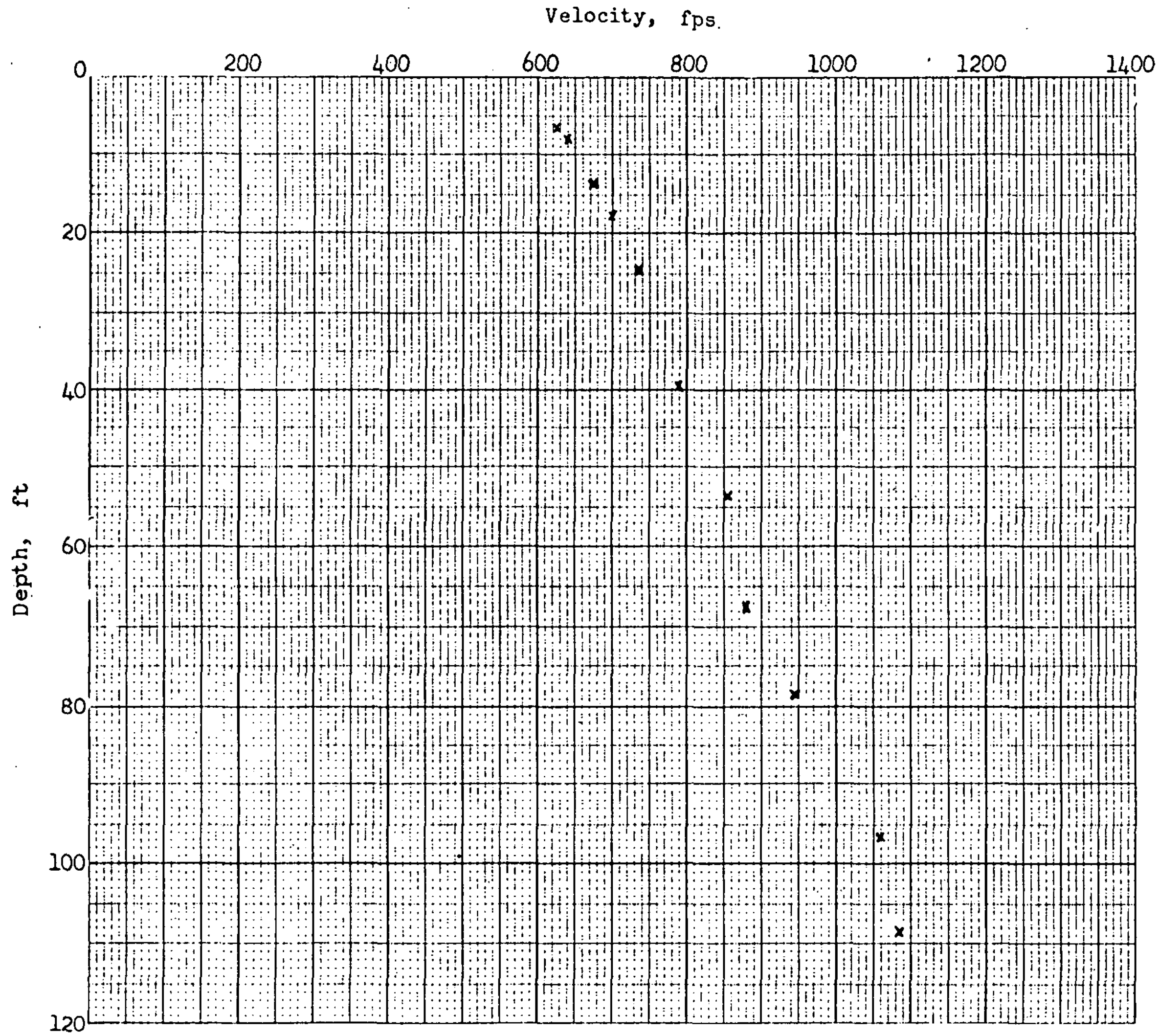
Inclosure 13

Inclosure 14



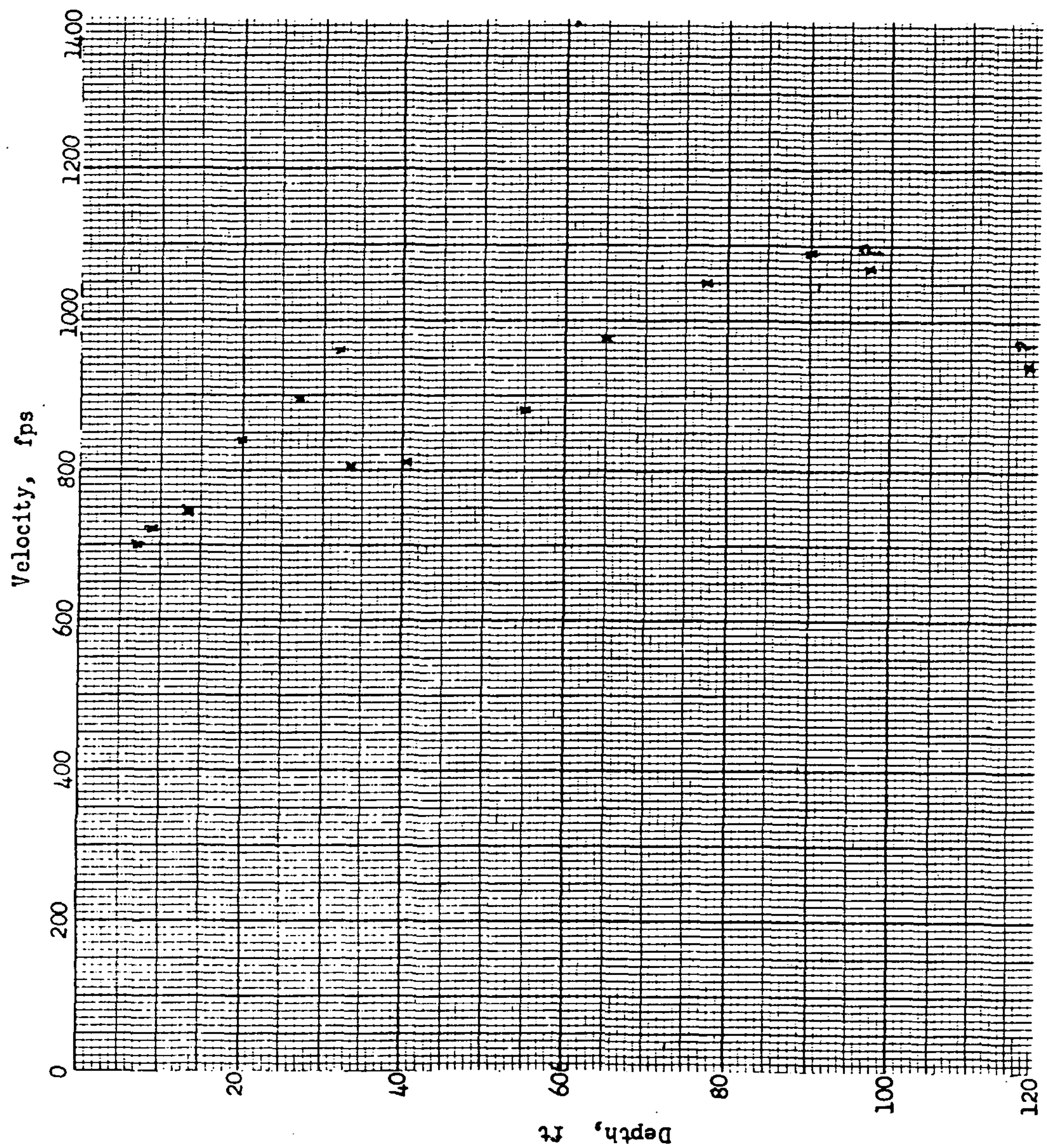
SHEAR WAVE VELOCITY versus DEPTH

Traverse V-1 West



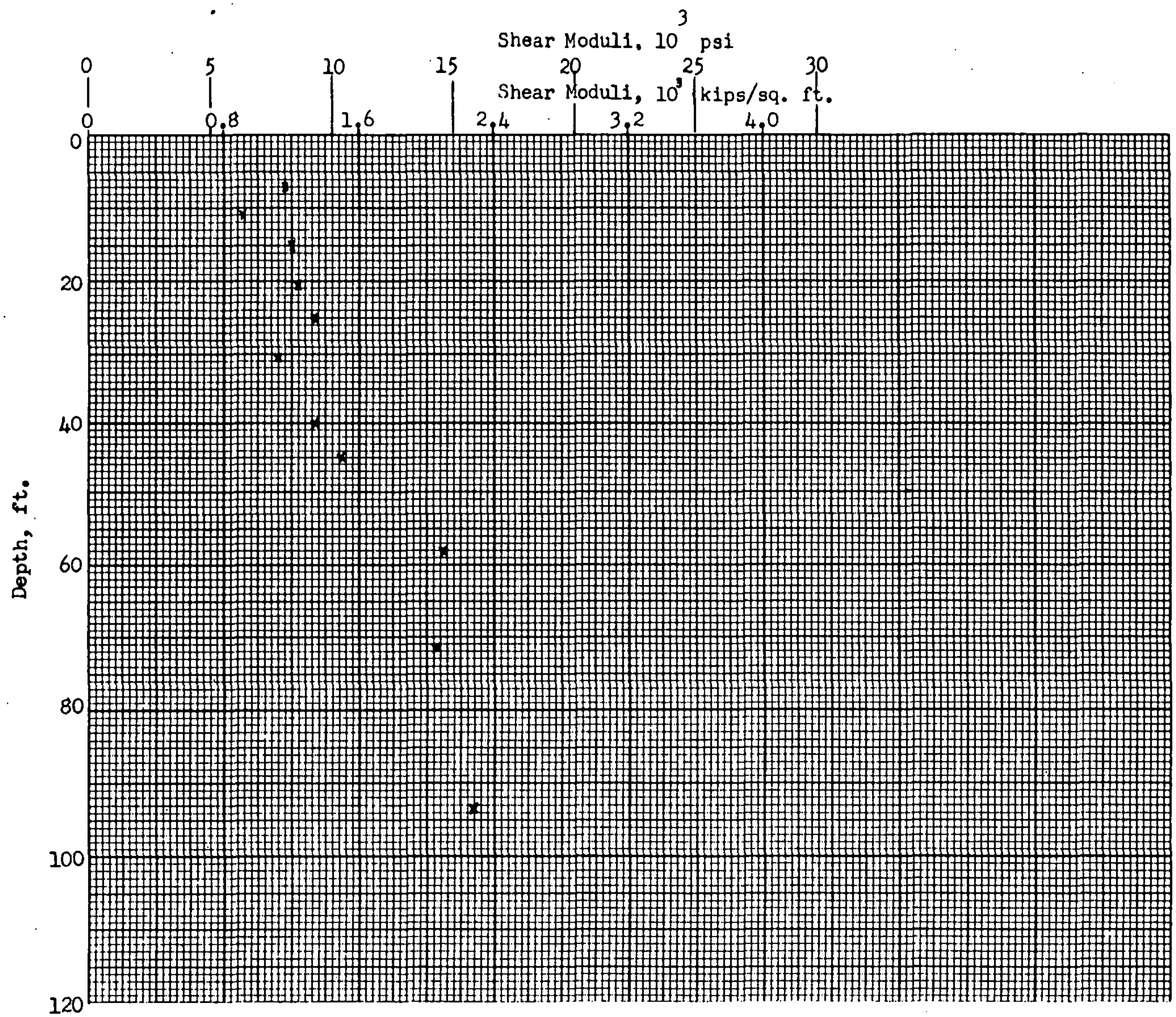
SHEAR WAVE VELOCITY versus DEPTH

Traverse V-2 North



SHEAR WAVE VELOCITY versus DEPTH
Traverse V-3 West
Crest of Dam

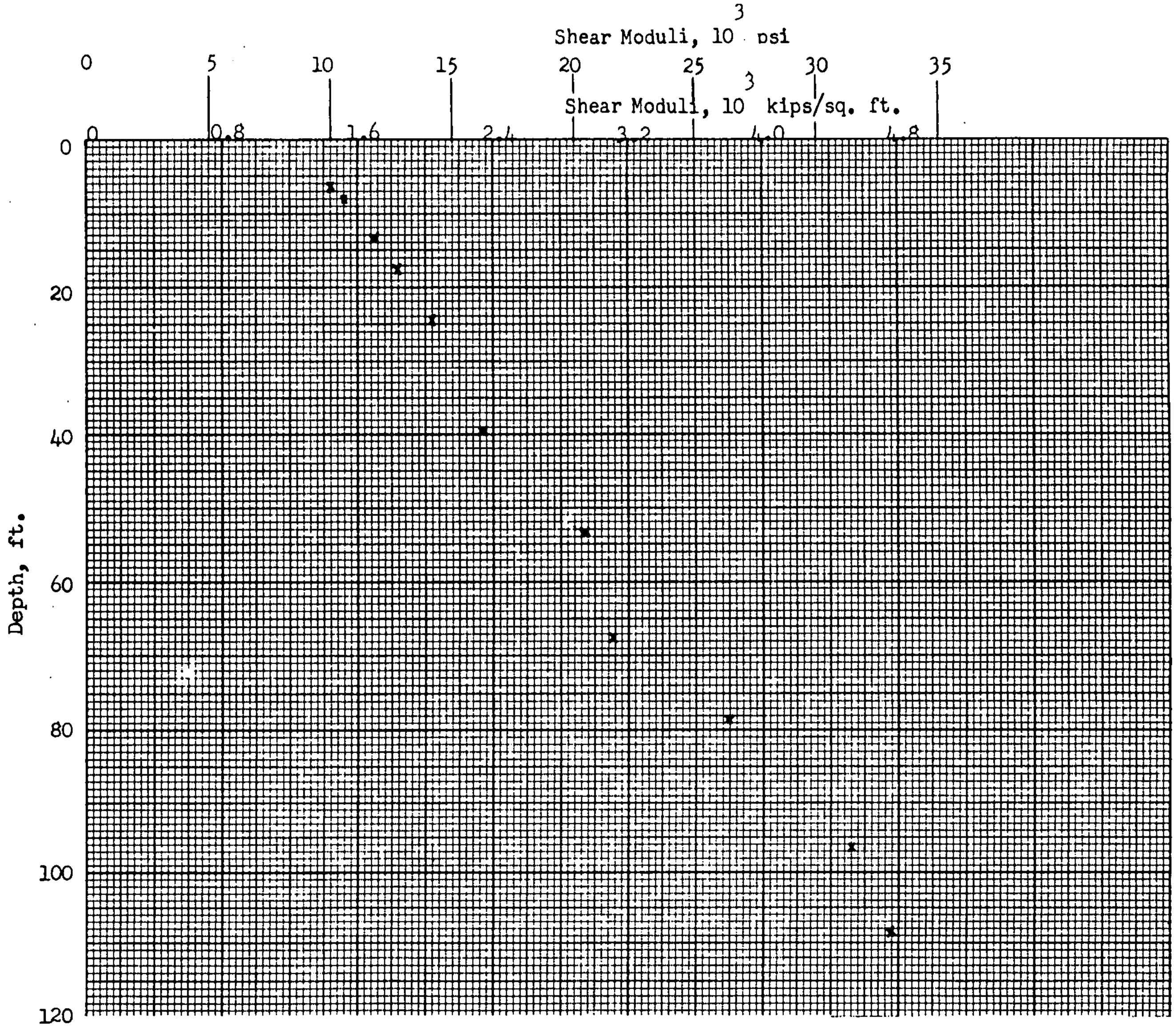
Inclosure 16



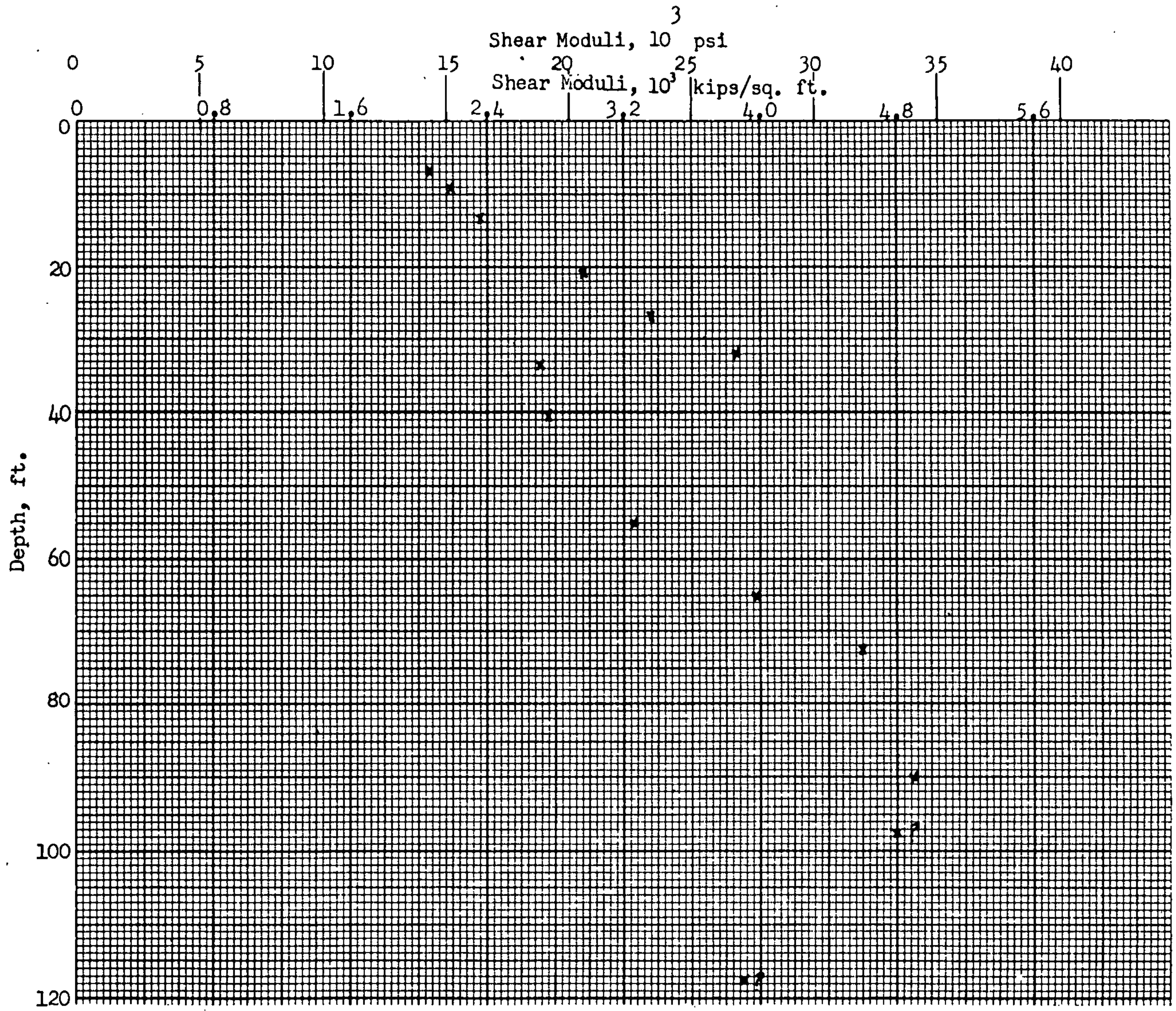
SHEAR MODULI versus DEPTH
Traverse V-1

Inclosure 17

Inclosure 18



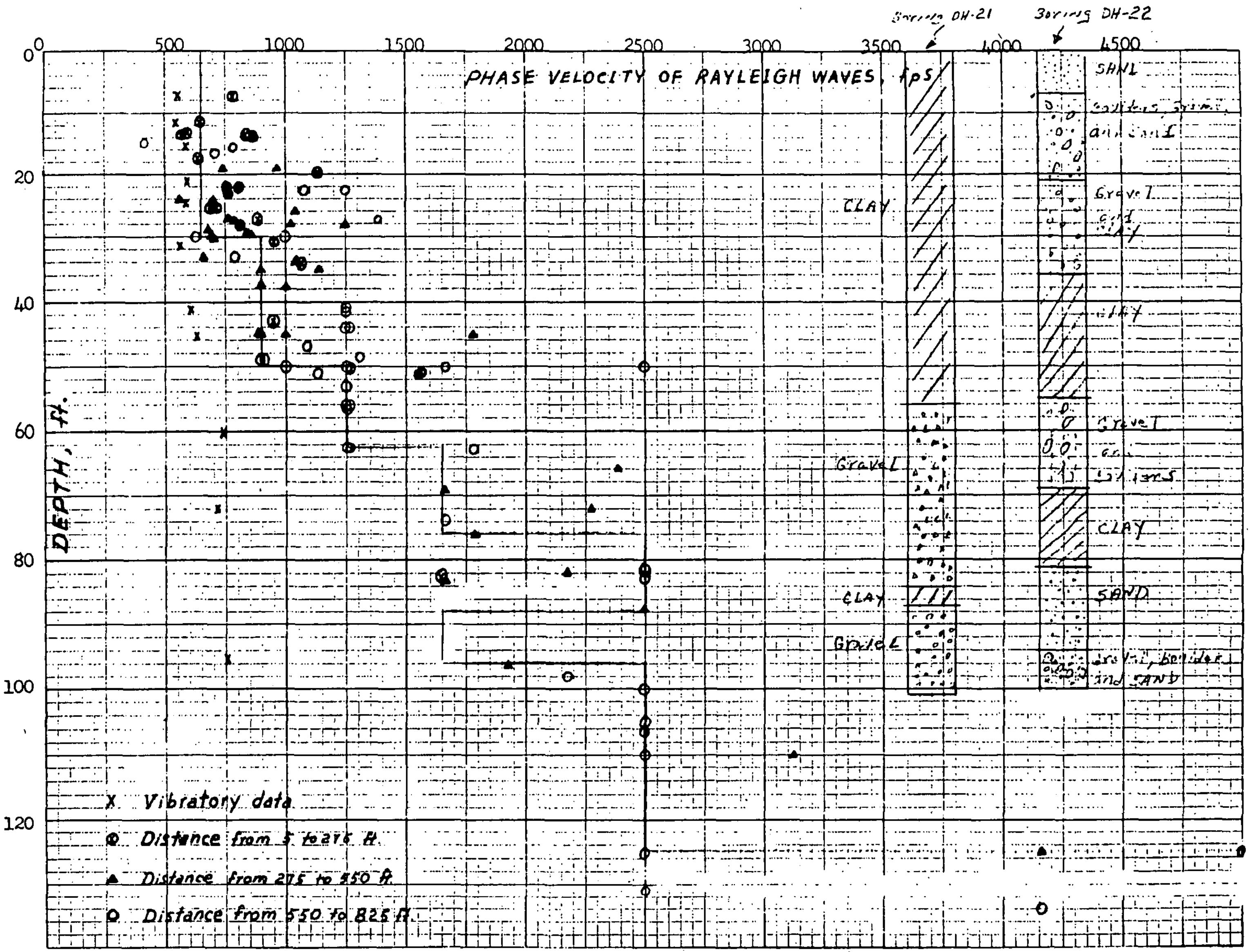
SHEAR MODULI versus DEPTH
Traverse V-2



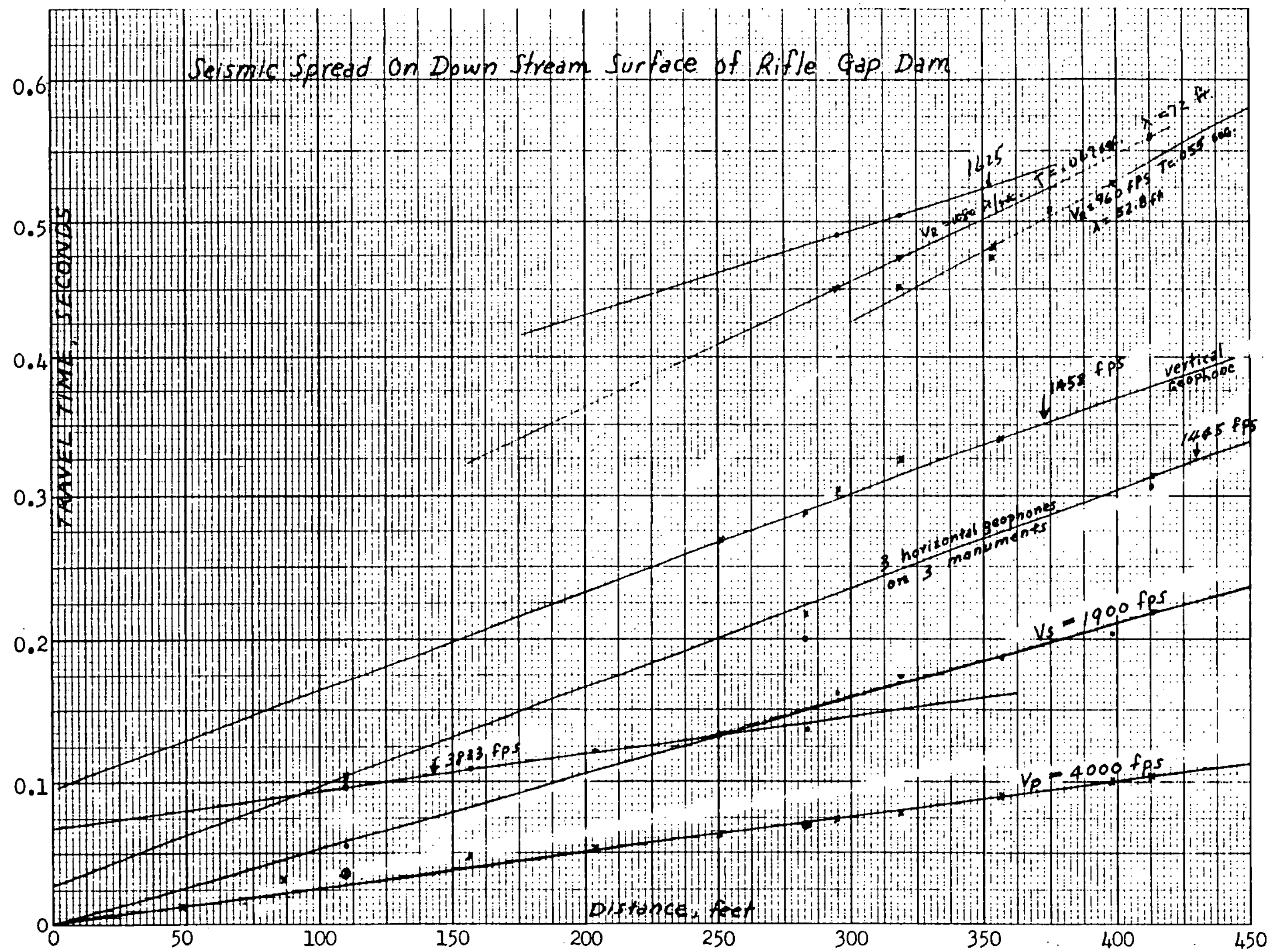
SHEAR MODULI versus DEPTH
Traverse V-3

Inclosure 19

Inclosure 20



Comparison Of Rayleigh Wave Velocities versus Depth(along Traverses S-1 and S-2)
and Shear Wave Velocities versus Depth(Traverse V-1)



TIME versus DISTANCE
Rayleigh Wave Seismic Test(up face of dam)

Inclosure 21

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Security Classification

DOCUMENT CONTROL DATA - R & D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

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4. DESCRIPTIVE NOTES (Type of report and inclusive dates)
Report 2 of a series

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Jack Fowler
Lyman W. Heller

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	Office, Chief of Engineers, U. S. Army Washington, D. C.

13. ABSTRACT
The motion of Rifle Gap Dam was measured in September 1969 during the Project RULISON underground nuclear explosion. The observed response was then compared with the response computed in a mathematical model. Observed and computed responses were similar. From this study it appears that the mathematical models used are applicable to the design and analysis of soil structures, at least for ground motion intensities comparable to those observed at Rifle Gap Dam.

DD FORM 1 NOV 65 1473 REPLACES DD FORM 1473, 1 JAN 64, WHICH IS OBSOLETE FOR ARMY USE.

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14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Earth dams						
Earthquake-resistant structures						
Ground motion						
Mathematical models						
Nuclear explosion effects						
Rifle Gap Dam						
Rock-fill dams						
Rulison (Project)						
Underground explosions						

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