SEISMIC DESIGN RESPONSE SPECTRA

BEAVER VALLEY POWER STATION - UNIT 2

PREPARED FOR

DUQUESNE LIGHT COMPANY
PITTSBURGH, PENNSYLVANIA

JUNE 1984

By
STONE & WEBSTER ENGINEERING CORPORATION
BOSTON, MASSACHUSETTS

TABLE OF CONTENTS

Section	Title	Page
1	INTRODUCTION	1-1
2	BACKGROUND	2-1
2.1	Beaver Valley Power Station - Unit 1	2-1
2.2	Beaver Valley Power Station - Unit 2	2-3
3	MAXIMUM EARTHQUAKE POTENTIAL	3-1
4	SITE INDEPENDENT RESPONSE SPECTRA	4-1
5	SITE DEPENDENT RESPONSE SPECTRA METHODOLOGY	5-1
6	SITE MATCHED RESPONSE SPECTRA ANALYSIS	6-1
6.1	Site Conditions at BVPS	6-1
6.2	Site Matching Procedure	6-1
6.3	Site Matched Response Spectra	6-3
6.3.1	Site Matched Response Spectra Without Scaling	6-3
6.3.2	Site Matched Response Spectra With Scaling	6-4
7	SITE DEPENDENT RESPONSE SPECTRA FROM SOIL RESPONSE ANALYSIS	7-1
7.1	Soil Model	7-1
7.2	Rock Outcrop Motions	7-1
7.3	Soil Response Analysis Without Scaling	7-2
7.4	Soil Response Analysis With Scaling	7-2
8	SUMMARY OF HORIZONTAL RESPONSE SPECTRA	8-1
9	VERTICAL RESPONSE SPECTRA	9-1
10	REFERENCES	10-1

APPENDIX 1 - NUCLEAR REGULATORY COMMISSION SEISMOLOGY QUESTIONS - BEAVER VALLEY POWER STATION - UNIT 2

APPENDIX 2 - BORING LOGS AND SHEAR WAVE VELOCITY DATA FOR SITE MATCHED ACCELEROGRAPH STATIONS

APPENDIX 3 - BORING LOGS AND SHEAR WAVE VELOCITY DATA FOR ROCK OUTCROP ACCELEROGRAPH STATIONS

LIST OF TABLES

Table	Title	Page
2-1	Response Spectra Beaver Valley Power Station - Unit 1	2-4
2-2	Response Spectra Beaver Valley Power Station - Unit 2	2-5
6-1	Site Matched Accelerograph Stations	6-5
6-2	Site Matched Ground Surface Records	6-6
7-1	Rock Outcrop Motions Without Scaling	7-4
7-2	Rock Outcrop Motions Scaled to M_L = 4.95	7-5
9-1	Eastern United States Earthquake Data	9-4

LIST OF FIGURES

Figure	Title	Page
2-1	Horizontal Response Spectra BVPS - Unit 2	2-6
2-2	Earthquake Intensity - Acceleration Relationships	2-7
2-3	Housner Response Spectra BVPS - Unit 1	2-8
2-4	Horizontal Response Spectra BVPS - Unit 1	2-9
2-5	Horizontal Response Spectra BVPS - Unit 2	2-10
2-6	Accleration Amplification Factors	2-11
2-7	Relationship Between Displacement and Duration	2-12
2-8	Displacement Amplification Factors	2-13
4-1	Site Independent Response Spectrum	4-2
5-1	Comparison of Scaled and Unscaled Site Matched Response Spectra	5-8
6-1	Comparison of In Situ Shear Wave Velocities Before and After Densification	6-7
6-2	Generalized Shear Wave Velocity Profiles	6-8
6-3	Site Matched Response Spectra (Unscaled)	6-9
6-4	Mean and Mean-Plus-One Standard Deviation Site Matched Response Spectra (Unscaled)	6-10
6-5	Site Matched Response Spectra (Unscaled)	6-11
6-6	Site Matched Response Spectra (Unscaled)-Effect of Federal Building Records	6-12
6-7	Site Matched Response Spectra (Scaled)	6-13
6-8	Site Matched Response Spectra (Scaled) - Effect of Federal Building Records	6-14

Figure	Title	Page
6-9	Comparison of Scaled and Unscaled Site Matched Response Spectra	6-15
6-10	Site Matched Response Spectrum	6-16
7-1	Soil Response Analysis	7-6
7-2	Soil Model	7-7
7-3	Strain Dependent Soil Parameters	7-8
7-4	Response Spectra From Soil Response Analysis: In Situ (Unscaled)	7-9
7-5	Response Spectra From Soil Response Analysis: Densified Area (Unscaled)	7-10
7-6	Comparison of Response Spectra From Soil Response Analysis (Unscaled)	7-11
7-7	Response Spectra From Soil Response Analysis: In Situ (Scaled)	7-12
7-8	Response Spectra From Soil Response Analysis: Densified Area (Scaled)	7-13
7-9	Comparison of Response Spectra From Soil Response Analysis (Scaled)	7-14
7-10	Site Dependent Response Spectra From Soil Response Analysis (Scaled)	7-15
8-1	Summary of Response Spectra	8-2
9-1	Comparison of Vertical Response Spectra	9-5
9-2	New Brunswick Earthquake Aftershock: March 31, 1982	9-6

SECTION 1

INTRODUCTION

This report describes the development during the early 1970's of the Beaver Valley Power Station - Unit No. 2 (BVPS-2) horizontal and vertical design response spectra. It also presents the results of a reevaluation of these response spectra by comparison with response spectra determined using current state-of-the-art procedures and available earthquake data. The BVPS-2 horizontal and vertical response spectra compare favorably with the state-of-the-art response spectra and provide an acceptable degree of conservatism for plant design.

This report was prepared to respond to several questions raised by the Nuclear Regulatory Commission (NRC) during their review of the BVPS-2 Final Safety Analysis Report (FSAR). The texts of the questions are provided in Appendix 1.

The Safe Shutdown Earthquake (SSE) horizontal response spectrum for 5-percent damping was reevaluated by comparison with response spectra determined using two basic approaches:

- Site Independent Approach
  Regulatory Guide 1.60 (USAEC 1973)

- Site Dependent Approach
  Site matched response spectra analysis and
  Soil response analysis

Regulatory Guide 1.60 is site independent because it is based on standardized response spectrum shapes developed from statistical analyses of response spectra determined from past earthquake records, largely without regard to the characteristics of the site at which the records were obtained. A response spectrum determined according to Regulatory Guide 1.60 is discussed in Section 4.0.

The site dependent approach was further divided into two basic approaches: site matching and soil response analysis. In the site matching approach, response spectra were determined directly from real earthquake records of the appropriate magnitude that were recorded at accelerograph stations having soil profiles and engineering properties similar to BVPS. In the soil response analysis, earthquake records taken from accelerograph stations on rock outcrops were amplified through the BVPS-2 soil profile to determine the earthquake induced ground surface motions from which response spectra were determined. The amplification analyses were performed using the computer program SHAKE (Schnabel et al 1972). Response spectra determined by using the site matching and the soil response analysis approaches are discussed in Sections 6.0 and 7.0, respectively.

A summary discussion of the comparison of the horizontal response spectra is provided in Section 8.0.

The BVPS-2 vertical response spectra are taken as two-thirds of the corresponding horizontal response spectra. This criterion was found to be consistent with currently available western and eastern United States earthquake data. A comparison was made between the BVPS-2 vertical response spectrum for 5-percent damping and a Regulatory Guide 1.60 vertical response spectrum corresponding to the BVPS-2 SSE Intensity VI(MM). They were found to be approximately equivalent. Further discussion of vertical response spectra is provided in Section 9.0.

SECTION 2
BACKGROUND

The Beaver Valley Power Station (BVPS) has been designed for a Safe Shutdown Earthquake (SSE) and an Operating Basis Earthquake (OBE) corresponding to horizontal ground surface accelerations of 0.125g and 0.06g, respectively. Vertical accelerations were taken as two-thirds of the horizontal accelerations.

The BVPS-2 horizontal response spectra for the SSE are shown in Figure 2-1. When originally presented in the Preliminary Safety Analysis Report (PSAR) (SWEC 1972), the BVPS-2 response spectra were identical to those presented for BVPS-1. They were later revised as dictated by USAEC Position 3 (SWEC 1973). Since BVPS-2 made use of much of the work that had previously been done for BVPS-1, the development of the BVPS-1 response spectra must be first described in order to describe the origins of the BVPS-2 response spectra.

2.1 BEAVER VALLEY POWER STATION - UNIT 1

The historical seismicity of the site area was first investigated by Weston Geophysical Research (WGR) of Weston, Massachusetts, for BVPS-1 (WGR 1968). Based upon their investigations WGR concluded that the maximum historical intensity for the site area corresponded to an Intensity IV(MM) in the upland areas with relatively shallow soil deposits overlying bedrock and possibly low to middle V(MM) for the alluvial deposits along the Ohio River. Accordingly, they recommended an OBE of 0.05g, corresponding to an Intensity V-VI(MM), and an SSE (originally called the Design Basis Earthquake) of 0.1g or about Intensity VI-VII(MM). Intensity was correlated with acceleration using the relationship reproduced in Figure 2-2. It represents an approximate upper bound of a number of empirical correlations in existance at the time and it compares quite well with the later relationships developed by Trifunac and Brady (1975) and Murphy and O'Brien (1977).

Further investigation by Whitman (1968) to evaluate the seismic design basis for the site used an approach similar to the soil response analysis described in Section 7 of this report. The maximum historic intensity in the site area on firm ground or rock had been determined by WGR (1968) as IV(MM). Whitman (1968) conservatively assumed an SSE on bedrock of Intensity V(MM) or, at most, low VI(MM) with a bedrock acceleration of 0.035g. Using a computer program called DYALS, which is similar in concept to SHAKE (Schnabel et al 1972), he performed an analysis in which several earthquake records, normalized to 0.035g, were input at the bedrock surface and amplified through the free-field soil profile. Whitman (1968) compared response spectra at the bedrock with those at the ground surface. The comparison indicated a maximum amplification factor of 3.5, thereby giving a ground surface acceleration of about 0.125g.

The original BVPS-1 SSE horizontal response spectra, shown in Figure 2-3, were based upon the Housner (1963) average response spectra normalized to 0.125g. At the time of the ACRS hearings in early 1970, the Housner spectra were no longer considered acceptable by the Atomic Energy Commission (AEC).

The AEC required that a new response spectra be developed which incorporated an acceleration amplification factor of 3.6 for 2-percent damping. No other criterion was imposed by the AEC. Accordingly, using earthquake data that was then available, the acceleration and displacement amplification factors given in Table 2-1 were developed, as will be described below. To be consistent with the response spectra proposed by Newmark and Hall (1969), smoothed straight line spectra were constructed as shown in Figure 2-4 with frequency break points at 0.5, 2, 5 and 20 Hz.(1)

As stated previously, the only criterion given by the AEC for the revised BVPS-1 response spectra was that the acceleration amplification factor for 2-percent damping should be 3.6. Acceleration amplification factors suggested by Newmark and Hall (1969) are shown in Figure 2-6 as a function of percent damping. Amplification factors for the Housner response spectra (Figure 2-3) are also shown. The amplification factors used for the BVPS-1 response spectra were determined by drawing a line roughly parallel to the Newmark and Hall values through 3.6 at 2-percent damping. Amplification factors and the corresponding spectral accelerations are summarized in Table 2-1.

At the time that the revised BVPS-1 response spectra were being developed, there were few strong ground motion records available. This was before the 1971 San Fernando earthquake had occurred. Using computed response spectra that could be obtained, a study was made which indicated that an approximate correlation existed between the duration of strong ground motion and displacement. From the response spectrum for 5-percent damping, the displacement and velocity at a structural period of 2.5 seconds were determined. The displacement was then normalized to a peak velocity of 4 in/sec. Velocity was used for normalizing displacements because blasting data had indicated that levels of structural damage correlated better with velocity than with acceleration. Duration, defined as the time during which acceleration exceded 0.03g, was determined from the earthquake records and a relationship between normalized displacement and duration was established as shown in Figure 2-7. Assuming a duration of 10 seconds for BVPS gave the corresponding displacement at 5-percent damping as 3 inches. From the

---

(1) The response spectra shown in Figure 2-5 were presented in the BVPS-1 PSAR (SWEC 1970) and they are also the same response spectra that were originally proposed for use at BVPS-2 (SWEC 1972). An inconsistency was noted in the BVPS-1 response spectra shown in Figure 2-5 in 1979 and was corrected as shown in Figure 2-4 to agree with the accelerations and displacements presented in Table 2-1 (SWEC 1979). The inconsistency had no impact on the design of BVPS-1 since the computed values for the response spectra, rather than the curves shown in Figure 2-5, were used for design purposes. The inconsistency that was observed can be described as follows. The spectra for 0-percent and 0.5-percent damping in Figure 2-5 are actually closer to the computed spectra for 0.5-percent and 1-percent damping respectively. In the frequency range of primary interest to BVPS, between about 2 and 10 Hz, the spectra for 2, 5, 7 and 10-percent damping in Figures 2-4 and 2-5 are not significantly different.

present study, the spectral displacement at 5-percent damping for frequencies less than 0.5 Hz was found to be less than one inch.)

The displacement amplification factors and the corresponding spectral displacements are summarized in Table 2-1. They are shown in Figure 2-8 to be somewhat larger than those suggested by Newmark and Hall(1969). Since the spectral displacement at 5-percent damping had been determined, and the corresponding amplification factor was known, the ground displacement was determined as:

$$d_g = \frac{3 \text{ in.}}{1.6} = 1.875 \text{ in.} \qquad \text{(Eq 2-1)}$$

This implies a peak ground displacement of 15 inches for 1g ground acceleration. The remaining spectral displacements were then computed as:

$$d = 1.875 \text{ X (DAF)}$$

where:

d = spectral displacement for a given value of percent damping

DAF = displacement amplification factor

The response spectra, constructed from the data given in Table 2-1, that were used in the design of BVPS-1 are shown in Figure 2-4.

2.2 BEAVER VALLEY POWER STATION - UNIT 2

The SSE response spectra presented in the BVPS-2 PSAR in November 1972 were the same as those shown in Figure 2-5 (SWEC 1972). After review of these response spectra by the AEC, Regulatory Position 3 dated May 25, 1973, dictated certain changes to the response spectra that were required before they could be accepted for the design of BVPS-2 (SWEC 1973). The changes included the following:

1. Between 0.5 and 2 Hz, velocities were made constant, and
2. The constant acceleration region was extended from 5 to 6 Hz.

The revised response spectra within the velocity region were determined by holding constant the spectral velocity value at 2 Hz and extending the plot horizontally to the 0.5 Hz breakpoint, thereby establishing a new spectral displacement somewhat higher than before. The response spectra within the constant acceleration region were extended to 6 Hz and then intersected at 20 Hz. The response spectra finally accepted by the AEC and used for the design of BVPS-2 incorporated the changes described above and are shown in Figure 2-1. A summary of the spectral accelerations and displacements that were taken from Figure 2-1 is provided in Table 2-2.

TABLE 2-1

RESPONSE SPECTRA
BEAVER VALLEY POWER STATION - UNIT 1

Damping (%)	Acceleration Amplification Factor	Acceleration[1] (g)	Displacement Amplification Factor	Displacement[2] (in.)
0.5	5.2	0.65	2.75	5.16
1	4.4	0.55	2.4	4.50
2	3.6	0.45	2.05	3.84
5	2.6	0.33	1.6	3.00
7	2.1	0.26	1.4	2.63
10	1.8	0.23	1.25	2.34

([1]) Acceleration = (0.125g) x Amplification Factor

([2]) Displacement = (1.875 in.) x Amplification Factor

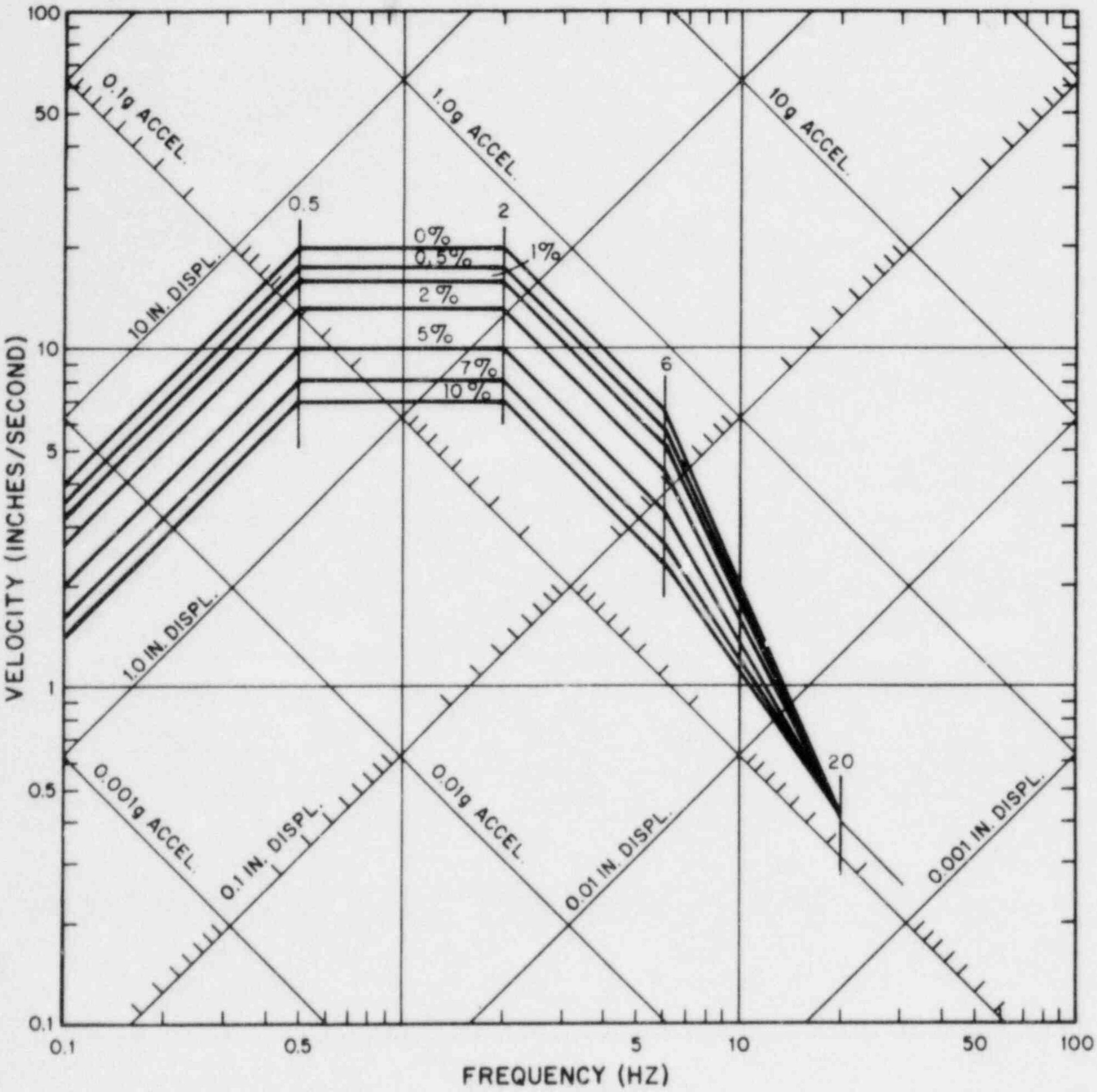
TABLE 2-2

RESPONSE SPECTRA
BEAVER VALLEY POWER STATION - UNIT 2

Damping (%)	Acceleration[1] (g)	Displacement[2] (in.)
0	0.650	6.37
0.5	0.564	5.52
1	0.518	5.07
2	0.427	4.18
5	0.322	3.15
7	0.262	2.56
10	0.225	2.20

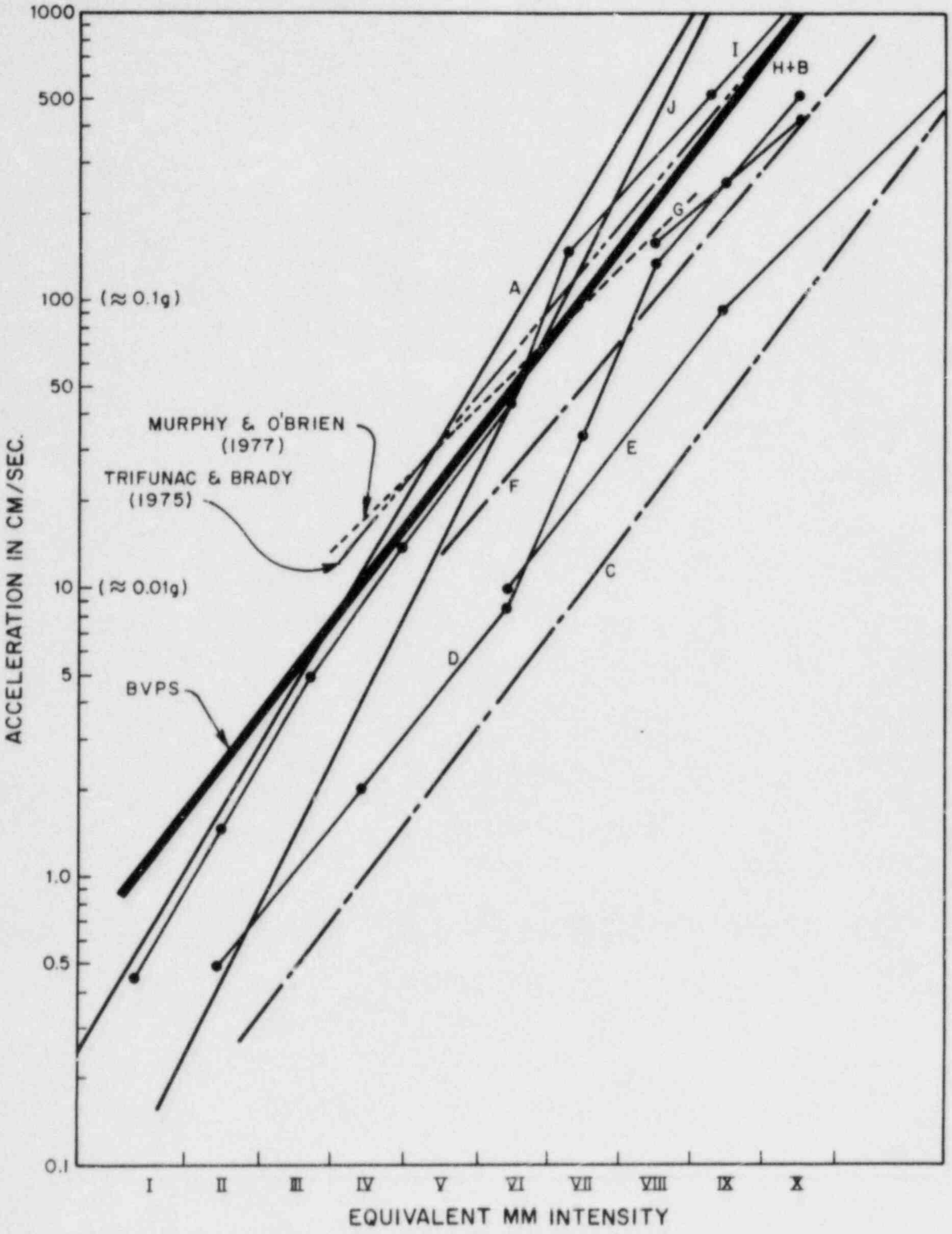
NOTES:

(1) Taken from Figure 2-5
(2) Computed



NOTE
SAFE SHUTDOWN EARTHQUAKE: a_h = 0.125g.

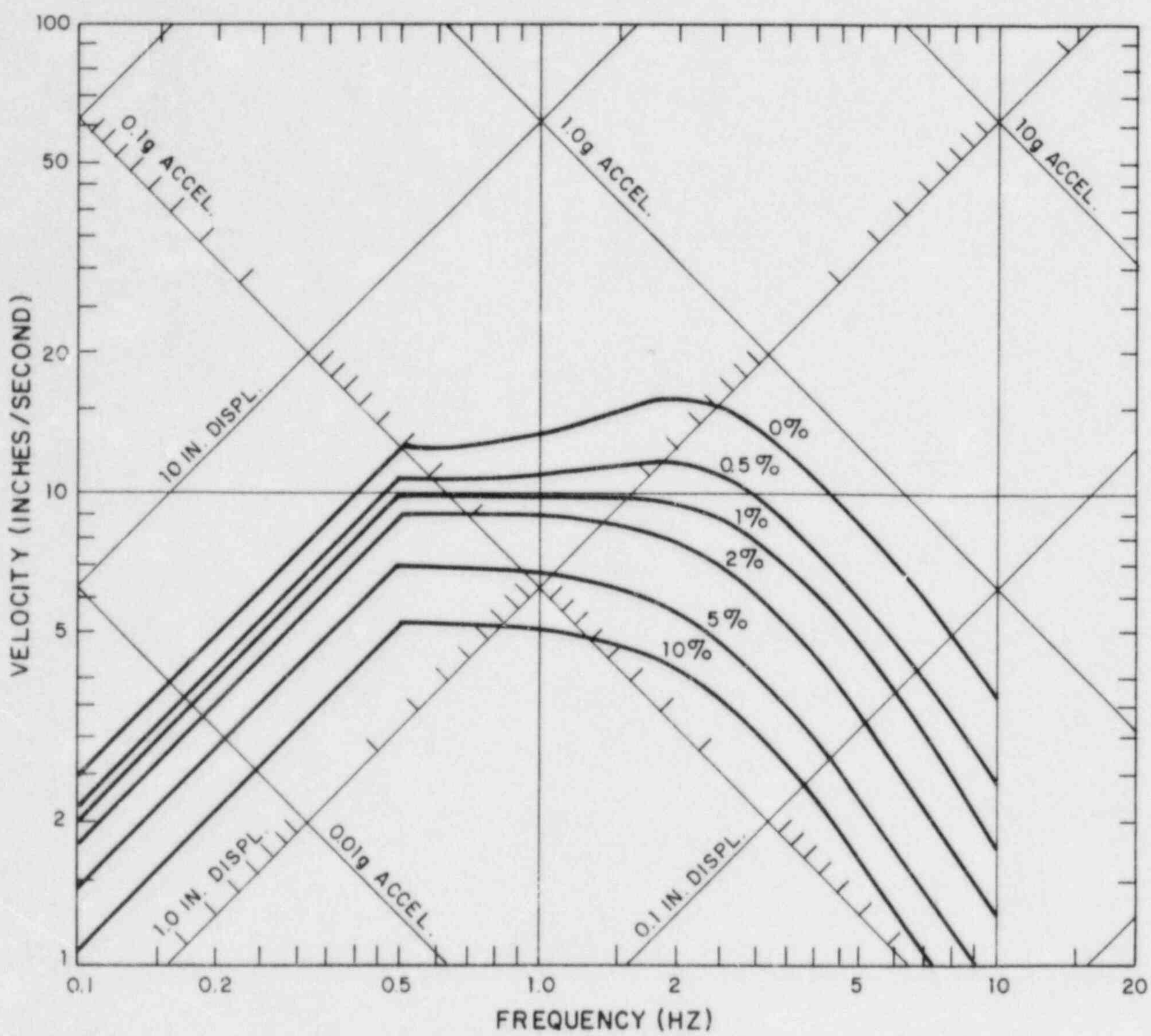
FIGURE 2-1
HORIZONTAL RESPONSE SPECTRA
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



A - HERSHBERGER (1956)
B - GUTENBERG & RICHTER (1942)
* C - CANCANI (1904)
* D - ISHIMOTO (1932)
* E - SAVARENSKY & KIRNOS (1955)
* F - MEDVEDEV ET AL. (1963)
* G - N.Z. DRAFT BY-LAW
H - TID-7024 (1963)
* I - KAWASUMI (1951)
* J - PETERSCHMITT (1951)

* DATA FROM G.A. EIBY (1965)
REFERENCE: WGR (1968)

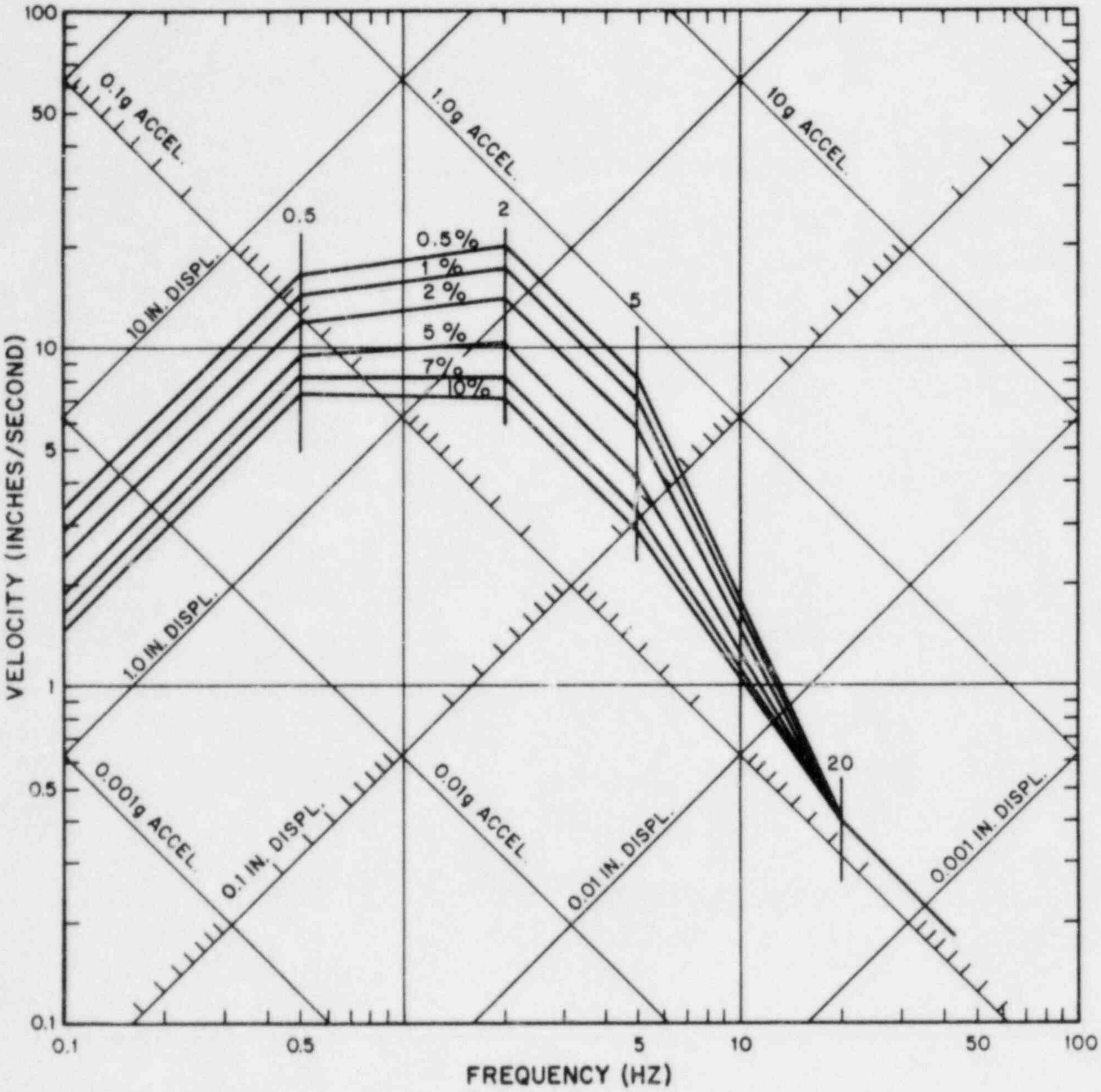
FIGURE 2-2
EARTHQUAKE INTENSITY-ACCELERATION RELATIONSHIPS
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



NOTES

1. SAFE SHUTDOWN EARTHQUAKE: $a_h = 0.125g$.
2. ORIGINALLY PROPOSED RESPONSE SPECTRA SHOWN IN BVPS-1 PSAR (SWEC, 1968).

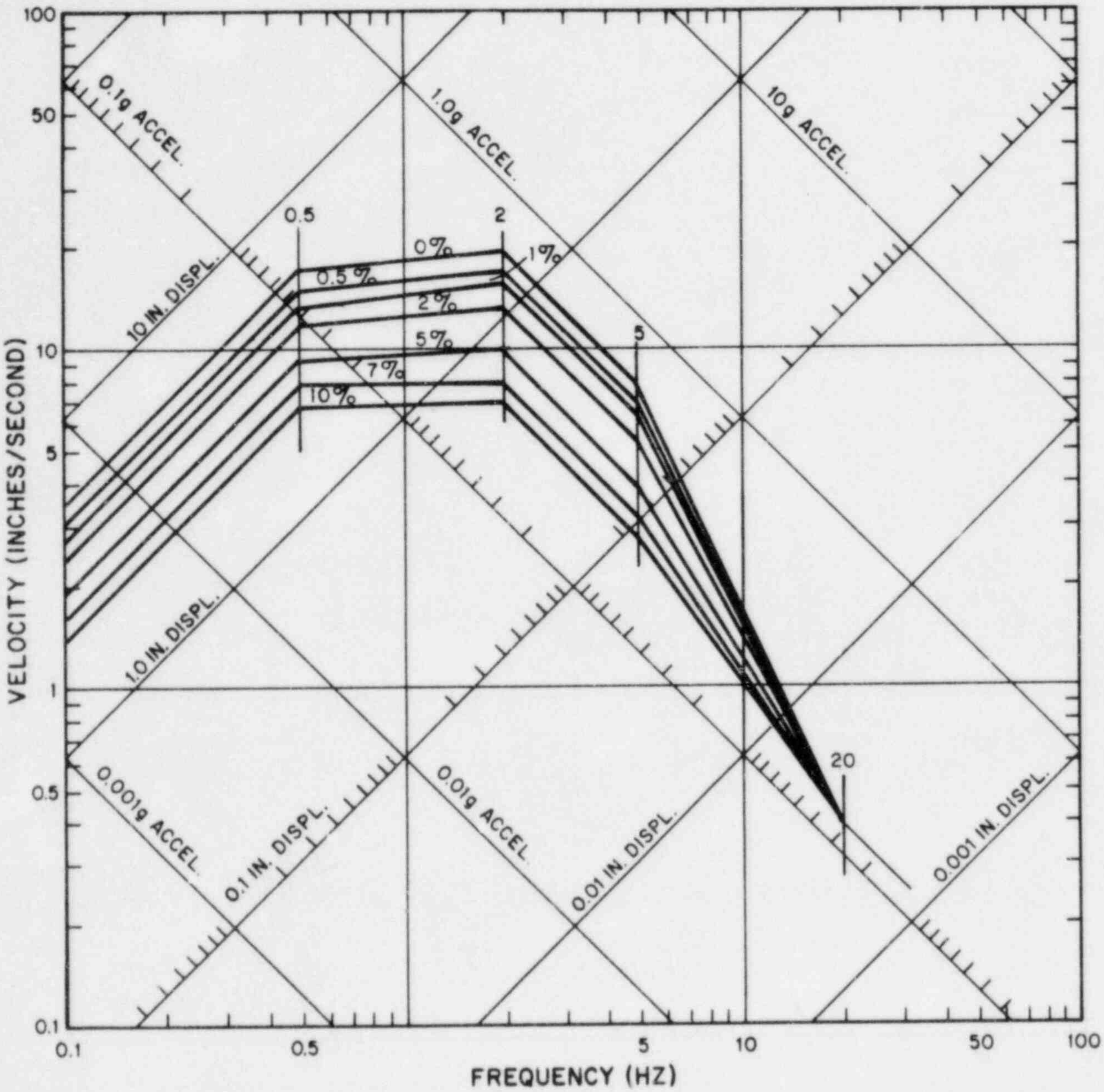
FIGURE 2-3
HOUSNER RESPONSE SPECTRA
BEAVER VALLEY POWER STATION-UNIT 1
STONE & WEBSTER ENGINEERING CORPORATION



NOTES

1. SAFE SHUTDOWN EARTHQUAKE: a_h = 0.125g.
2. FINAL BVPS-1 RESPONSE SPECTRA (SWEC, 1979).

FIGURE 2-4
HORIZONTAL RESPONSE SPECTRA
BEAVER VALLEY POWER STATION-UNIT 1
STONE & WEBSTER ENGINEERING CORPORATION



NOTES

1. SAFE SHUTDOWN EARTHQUAKE: $a_h = 0.125g$
2. ORIGINALLY PROPOSED RESPONSE SPECTRA SHOWN IN BVPS-2 PSAR (SWEC, 1972).
3. RESPONSE SPECTRA SHOWN IN BVPS-1 PSAR (SWEC, 1970).

FIGURE 2-5
HORIZONTAL RESPONSE SPECTRA
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION

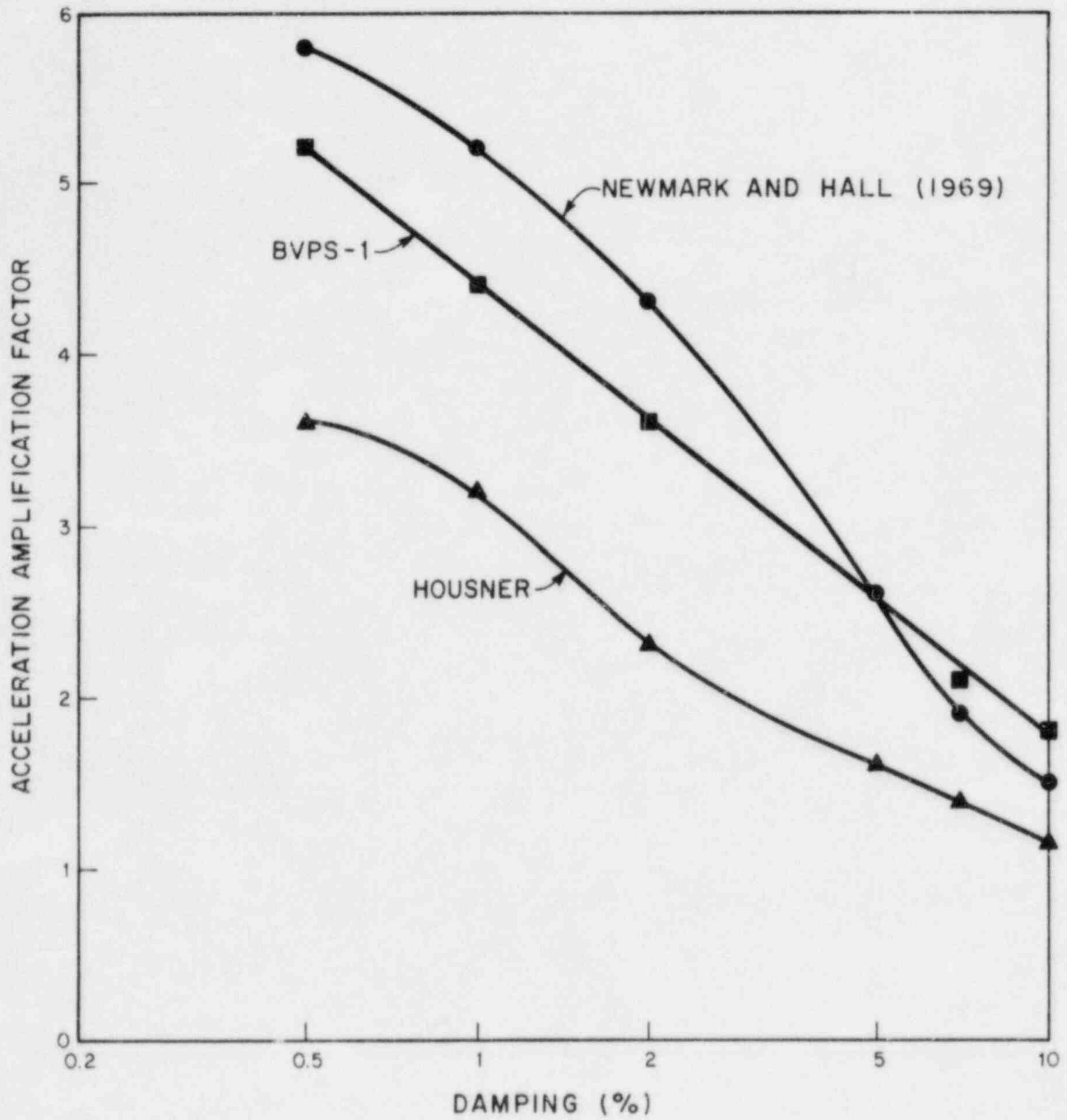
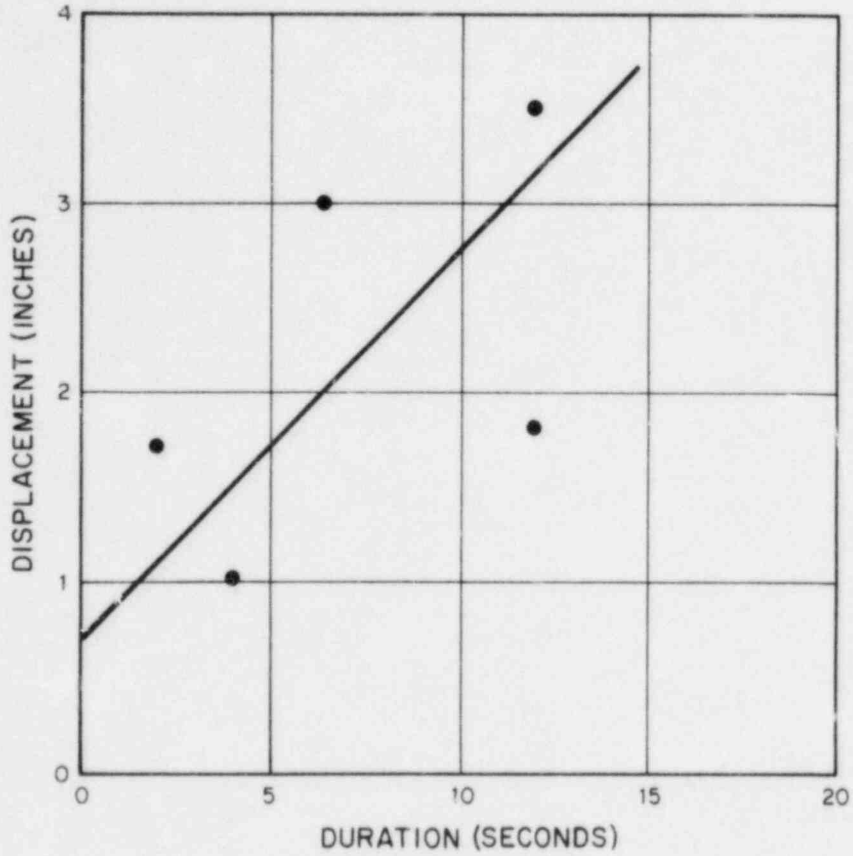


FIGURE 2-6
ACCELERATION AMPLIFICATION FACTORS
BEAVER VALLEY POWER STATION-UNIT 1
STONE & WEBSTER ENGINEERING CORPORATION



DEFINITIONS

DISPLACEMENT: DISPLACEMENT AT 2.5 SEC. PERIOD FOR 5% DAMPING, NORMALIZED TO 4 IN./SEC. VELOCITY.

DURATION: CUMULATIVE TIME THAT ACCELERATION IS > 0.03g

FIGURE 2-7
RELATIONSHIP BETWEEN DISPLACEMENT AND DURATION
BEAVER VALLEY POWER STATION-UNIT 1
STONE & WEBSTER ENGINEERING CORPORATION

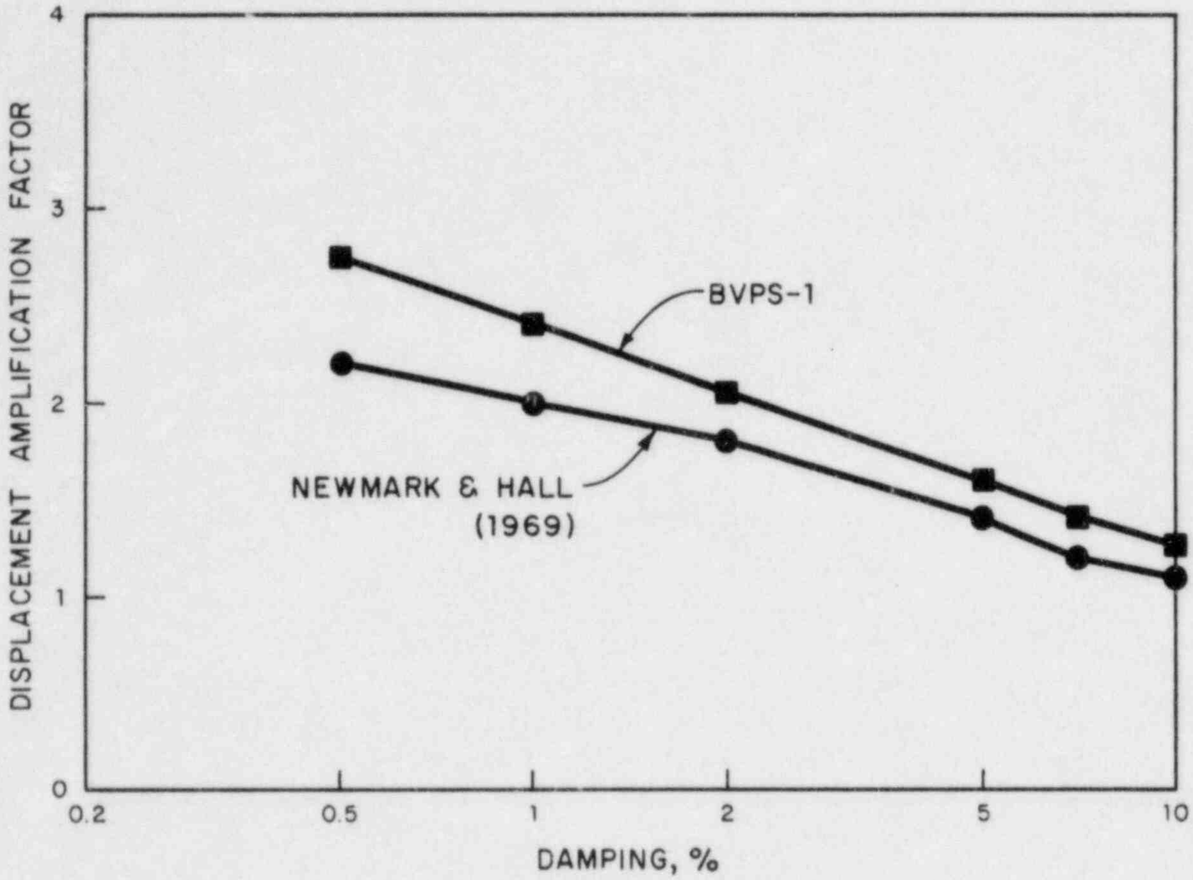


FIGURE 2-8
DISPLACEMENT AMPLIFICATION FACTORS
BEAVER VALLEY POWER STATION-UNIT 1
STONE & WEBSTER ENGINEERING CORPORATION

SECTION 3

MAXIMUM EARTHQUAKE POTENTIAL

The maximum earthquake potential for the site was evaluated for two different sets of conditions. First, actual site intensities resulting from historical earthquakes were determined. Second, the potential site intensities were determined for hypothetical events specified as arising from the largest known earthquakes in each adjoining tectonic province, postulated to occur at the point where the province most closely approached the site. The seismisity of the site region is more completely described in (SWEC 1983).

The BVPS site region lies near the center of the Appalachian Plateau tectonic province and is characterized by a low level of earthquake activity. During the past 180 years, there have been few earthquakes within 200 miles of the site and only three within 50 miles. There are two areas of localized earthquake activity in the site region, one at Anna, Ohio, and one at Attica, New York. The largest earthquakes at each of these sources, an Intensity VI-VIII(MM) at Anna, Ohio, and an Intensity VIII(MM) at Attica, New York, were barely perceptible at the site. An examination of the ground motion effects on BVPS of earthquakes within 200 miles indicated that the site has not experienced ground motions exceeding Intensity III-IV(MM). The site has, however, experienced more severe ground motions from larger and more distant earthquakes. Examination of isoseismal maps of larger earthquakes felt in the eastern United States showed that the New Madrid events of 1811-1812 probably caused a maximum historic ground motion at the site corresponding to an intensity of low to middle V(MM).

The maximum earthquake potential for the site has been estimated from the tectonic province approach to be equivalent to an event of epicentral Intensity VI(MM) occurring within the Appalachian Plateau tectonic province near the site.

Trifunac and Brady (1975) developed the following relationship between intensity and acceleration:

$$\log a = 0.3I_{MM} + 0.014 \qquad \text{(Eq 3-1)}$$

where:

a = peak horizontal acceleration (cm/sec^2)
I_{MM} = Modified Mercalli Intensity

Using this relationship, an Intensity VI(MM) earthquake would produce a horizontal ground surface acceleration of 0.07g.

An empirical correlation between the body wave magnitude of central United States earthquakes and epicentral intensity was given by Nuttli and Herrmann (1978) as:

$$m_b = 0.5I_o + 1.75 \qquad (m_b \pm 0.5 \text{ units}) \qquad \text{(Eq 3-2)}$$

where:

m_b = body wave magnitude

I_o = epicentral intensity, Modified Mercalli

Using this relationship, an Intensity VI(MM) event is estimated to have a magnitude, m_b, of 4.75 ± 0.5.

SECTION 4

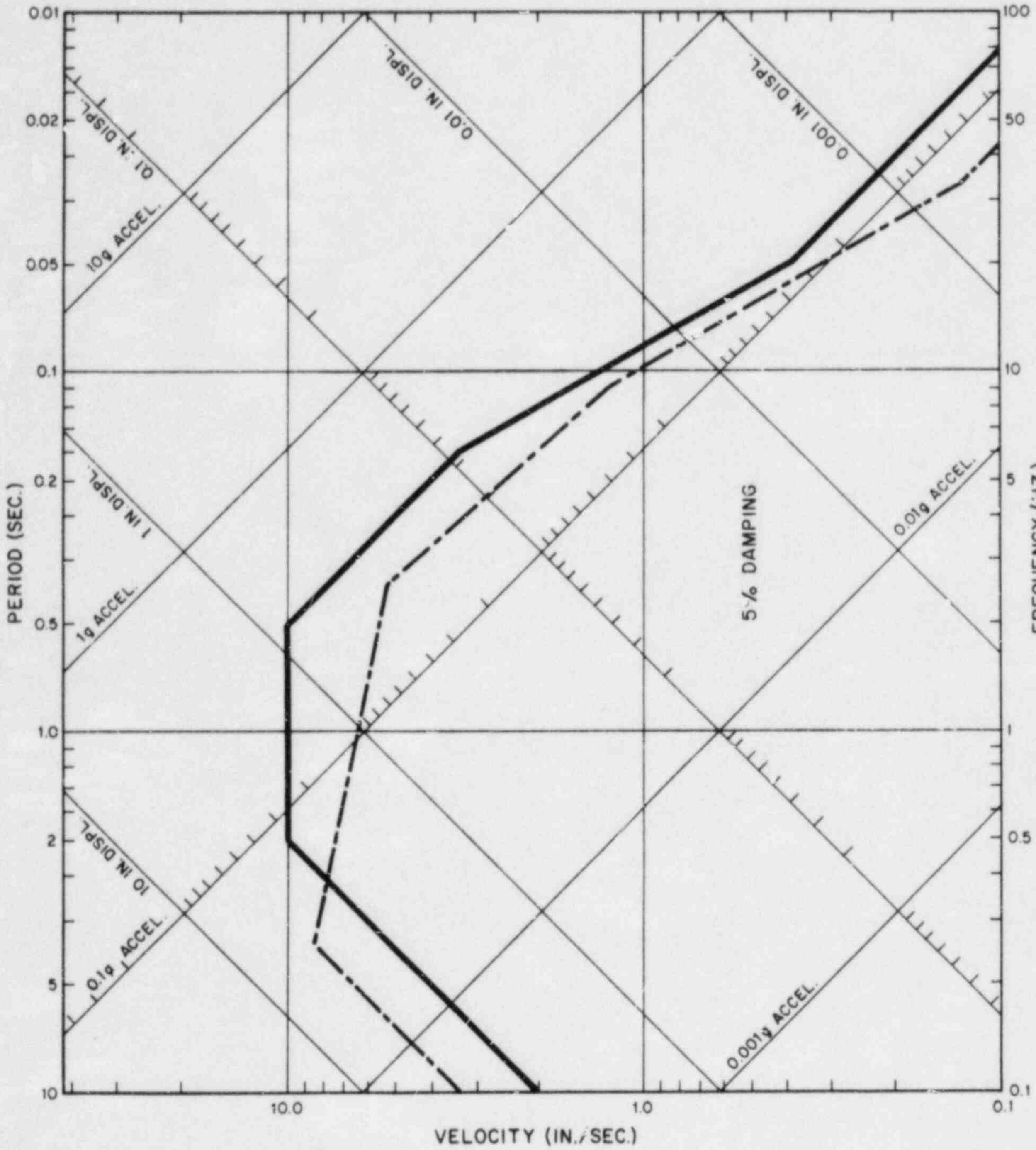
SITE INDEPENDENT RESPONSE SPECTRA

Site independent response spectra are constructed using the amplification factors and standard spectrum shapes given in Regulatory Guide 1.60 (USAEC 1973). The determination of the ground surface acceleration is the only site specific data required. The Regulatory Guide 1.60 standardized response spectra were developed from a statistical analysis of a suite of response spectra computed from actual earthquake time histories. Since the earthquake records were obtained from accelerograph stations covering a wide range of geological, seismological and local soil conditions, the Regulatory Guide 1.60 response spectra construction procedure is considered to be independent of site specific characteristics (Hays 1980).

A Regulatory Guide 1.60 horizontal response spectrum determined for 5-percent damping at a peak ground surface acceleration of 0.07g is shown in Figure 4-1. A response spectrum anchored to 0.07g corresponds to an SSE equivalent to intensity VI(MM) at the site (Section 2).

For comparison, the BVPS-2 SSE horizontal design response spectrum for 5-percent damping is also shown in Figure 4-1. As discussed in Section 2, it is anchored to a peak ground acceleration of 0.125g. In the frequency range of interest to the design of BVPS-2 structures (i.e., 1 to 10 Hz), the BVPS-2 response spectrum conservatively envelopes the Regulatory Guide 1.60 response spectrum anchored to 0.07g. For frequencies less than 0.5 Hz in the constant displacement region, the BVPS-2 response spectrum exhibits spectral displacements that are lower than the Regulatory Guide 1.60 spectrum. However, since there are no BVPS-2 plant structures with natural frequencies less than 0.5 Hz, the differences between the BVPS-2 response spectrum and the Regulatory Guide 1.60 response spectrum are not significant. Also, the site dependent response spectra determined for this study and summarized in Figure 8-1 indicate that spectral displacements are much lower than those of the Regulatory Guide 1.60 response spectrum. They are also significantly lower than those of the BVPS-2 response spectrum.

It is therefore concluded that the BVPS-2 response spectra are reasonable and conservative when compared with response spectra determined using a site independent approach and Regulatory Guide 1.60.



LEGEND

BVPS-2

REG. GUIDE 1.60 (0.07g)

FIGURE 4-1
SITE INDEPENDENT RESPONSE SPECTRUM
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION

SECTION 5

SITE DEPENDENT RESPONSE SPECTRA METHODOLOGY

In contrast to the site independent approach to response spectra development, the site dependent approach attempts to consider the effect of local geologic, seismologic, and soil conditions. The site dependent approach can be broken down into two basic types of analyses:

- Site matched response spectra analysis
- Soil response analysis

The site matched response spectra analysis uses records from earthquakes of similar size and epicentral distance as the site design earthquake that were obtained at accelerograph stations having site subsurface conditions matched as closely as possible to the site under study. Response spectra are then computed directly from these earthquake records.

The soil response analysis uses records from earthquakes of similar size and epicentral distance as the site design earthquake that were obtained from accelerograph stations on rock outcrops. The records are amplified through the site soil profile using an appropriate mathematical model to obtain ground surface response spectra. In either method of analysis, a site response spectrum for a given percent damping is determined as a mean or mean-plus-one standard deviation (84th percentile) of the suite of response spectra computed from the earthquake records.

The following items should be considered when selecting appropriate earthquake records to use in the site dependent method of analysis:

- Site conditions at the accelerograph station

  subsurface profile
  engineering properties
  geologic setting

- Size of the earthquake recorded

  magnitude
  intensity

- Source characteristics of the earthquake

  source mechanism
  transmission path
  epicentral distance
  focal depth

Ideally, records should be selected in which each of these items are matched for the site and its design earthquake. However, the limited number of strong motion records that are currently available requires the relaxation of some of these site-specific requirements. Therefore, judgment becomes important in first defining those particular factors that might have the greatest effect on the ground response at the site, and then selecting those available records that best match the site requirements.

Magnitude, rather than intensity, is a more reliable estimator of earthquake source strength for defining the size of an earthquake. Magnitude is a quantity related to the total energy released by an earthquake. It is determined from instrumental measurements and is independent of the observation point. Intensity is a subjective description of the earthquake size related to damage effects and felt reports and is heavily dependent upon the age of the buildings, local soil conditions, and population distribution. In the eastern United States, however, the general lack of instrumental magnitude data necessitates the use of empirical correlations with intensity in order to determine an appropriate site earthquake magnitude. As discussed in Section 3, the maximum potential ground motion at the BVPS-2 site is estimated to be an event of intensity VI(MM) occurring near the site. Using the Nuttli and Herrmann (1978) relationship, it is estimated that this event has a magnitude, m_b, of 4.75 ± 0.5.

Most of the processed strong motion earthquake records currently available are for western United States earthquakes.(1) Chung and Bernreuter (1980) noted that a direct comparison of body wave magnitudes of western and eastern United States earthquakes may be inappropriate since body wave magnitude is affected by certain regional characteristics. They found that the body wave magnitudes of western United States earthquakes were about 0.3 magnitude units lower than similar eastern United States earthquakes. Furthermore, most western United States earthquakes use local magnitude, M_L, instead of body wave magnitude, m_b, as an indicator of earthquake source strength. Chung and Bernreuter (1980), however, provide an empirical relationship between the m_b of an eastern earthquake and the M_L of an equivalent western earthquake as:

$$M_L \text{ (west)} = 0.57 + 0.92\, m_b \text{ (east)} \qquad \text{(Eq 5-1)}$$

This relationship was used to select a set of western United States earthquakes to represent eastern United States earthquakes. The SSE at BVPS with a body wave magnitude, m_b, of 4.75 ± 0.5 will be the equivalent to a western United States earthquake with a local magnitude, M_L, of about 4.95 ± 0.5. Therefore, the appropiate size earthquake to be used for site dependent response spectra analysis at BVPS-2 should fall within the following magnitude ranges:

- For eastern U.S. earthquakes: $4.25 \leq m_b \leq 5.25$
- For western U.S. earthquakes: $4.5 \leq M_L \leq 5.4$

---

(1)The western United States is defined as the "conterminous United States west of the Rocky Mountains" and, similarly, the eastern United States is the "conterminous United States east of the Rocky Mountains."

The distance from the epicenter of the earthquake to the recording station must also be considered when selecting earthquake records. The SSE for BVPS-2 is an event of intensity VI(MM) occurring near the site. It was necessary, therefore, to select records obtained close enough to the epicenter so that they were not greatly affected by attenuation effects.

For earthquakes in the central United States between the Rocky and the Appalacian Mountains, Gupta and Nuttli (1976) present the following attenuation relation:

$$I(R) = I_o + 3.7 - 0.0011R - 2.7 \log R \qquad \text{(Eq 5-2)}$$

where:

R = epicentral distance, km ($R \geq 20$ km)

I_o = epicentral intensity (MM)

By setting $I(R) - I_o$ to zero and solving for R, the relation shows that there is little attentuation of intensity for epicentral distances less than 25 kilometers. At these close distances, the differences in seismic wave attentuation behavior between earthquakes east and west of the Rocky Mountains do not affect the ground motions and direct use of strong motion records from the west for simulating ground motion in the east is acceptable (USNRC 1979). Therefore, to conform with the site SSE criteria, earthquake records selected were recorded at accelerograph stations located at epicentral distances of about 25 kilometers or less.

The number of available western earthquake records that fell within the magnitude limits discussed previously was small. To increase the number of records available to use in the study, a scaling procedure was developed so that earthquake records which fell outside the magnitude limits could be used. The rational behind this procedure is discussed below.

The shapes of response spectra developed from ground motion records change with changing magnitude, epicentral distance, and site conditions. However, for similar epicentral distances and site conditions, the statistical response spectra for various magnitudes have similar shapes but different spectral amplitudes (SW-AA 1979). This is a result of the shapes of response spectra being more affected by the frequency content of the ground motion record than by the magnitude of the ground motion. The frequency content of the ground motion is highly dependent upon the source-site transmission path and site conditions. Chang and Krinitzsky (1977) studied the duration, spectral content, and predominant period of strong motion earthquake records from the western United States and found that there was no relationship between frequency content and magnitude. It is believed, therefore, that epicentral distance and site conditions are more critical than the magnitude for determining the shapes of response spectra.

Since peak ground acceleration is now the most widely used measure of the strength of ground motions, and since most of the relatively scarce data regarding strength levels of past earthquakes are provided in terms of peak accelerations, scaling for this study was performed through a peak

acceleration vs. magnitude relationship. Herrmann and Nuttli (1980) gave a relation between body wave magnitudes, m_b, and far-field ground acceleration derived from theoretical calculations as:

$$\text{Log } a_h = A + 0.50\, m_b - 0.83 \text{ Log } (R^2 + h^2)^{\frac{1}{2}} - BR \qquad \text{(Eq 5-3)}$$

where:

a_h = horizontal peak acceleration in cm/sec^2

m_b = earthquake body wave magnitude

R = epicentral distance in km

h = focal depth in km

A & B = empirical constants

For any two given earthquakes, assuming that all of the variables are constant except for magnitude:

$$\Delta(\log a_h) = 0.5\, \Delta m_b \qquad \text{(Eq 5-4)}$$

which can be rearranged to give:

$$a_{h_2} = a_{h_1} \times 10^{-\frac{1}{2}(m_{b_1} - m_{b_2})} \qquad \text{(Eq 5-5)}$$

or

$$a_{h_2} = a_{h_1} \times 10^{-\frac{1}{2}\Delta m_b}$$

The scaling law represented by equation 5-4 was used by Nuttli (1979) in the study of the relationship between sustained maximum ground acceleration and velocity to earthquake intensity and magnitude.

Comparing two eastern events with two equivalent western events using equation 5-1 gives the following expression:

$$\Delta m_b \text{ (east)} = 1.09\, \Delta M_L \text{ (west)} \qquad \text{(Eq 5-6)}$$

Substituting equation 5-6 into equation 5-5 leads to the scaling law shown below:

$$a_{h_2} = a_{h_1} \times 10^{-0.54\, \Delta M_L}$$

or

$$\Delta(\log a_h) = 0.54\, \Delta M_L \qquad \text{(Eq 5-7)}$$

The scaling law given in equation 5-7 was used to scale selected western earthquake records to an M_L of 4.95, which is the equivalent western earthquake local magnitude for the BVPS-2 SSE.

Krinitzsky and Chang (1979) proposed a limitation on scaling factors to less than 4 if maximum motion levels (acceleration or velocity) were used as a basis for scaling. Applied to acceleration, this limitation corresponds to a limit of ± 1 unit for body wave magnitude as will be explained below.

Using the Trifunac and Brady (1975) relationship between peak acceleration and intensity (Eq 3-1) for two earthquakes with accelerations a_{h_1} and a_{h_2}:

$$\log a_{h_1} = 0.014 + 0.30 \, (I_{MM})_1 \qquad \text{(Eq 5-8)}$$

$$\log a_{h_2} = 0.014 + 0.30 \, (I_{MM})_2 \qquad \text{(Eq 5-9)}$$

Substituting $a_{h_2} = 4a_{h_1}$

in equation 5-9 and then substracting equation 5-8 from 5-9 gives:

$$\log 4a_{h_1} - \log a_{h_1} = 0.30 \, \{(I_{MM})_2 - (I_{MM})_1\}$$

and $\therefore \Delta I_{MM} = 2.0$

Since $m_b = 0.5I_o + 1.75$ (Eq 3-2)

$$\therefore \Delta m_b = 0.5 \, \Delta I_o = 1$$

This scaling factor limitation was also applied to the local magnitude, M_L, of western earthquakes. For this study, the limit of ±1 magnitude unit was applied to the upper and lower bounds of the BVPS-2 SSE magnitude range rather than to the midpoint of the range.

The viability of the scaling procedure was established through a comparison of scaled and unscaled response spectra computed for two shocks of the 1957 San Francisco earthquake. The local magnitudes, M_L, of the earthquakes were 4.4 and 5.3. (Earthquake data is provided in Table 6-2, Ref. Nos. 3-8.) Recordings were obtained at the Alexander Building and the State Building at epicenter distances between about 13 and 16 kilometers. In fact, the two recording stations are only about 0.8 kilometers apart. Since the two earthquakes occurred in the same geographical area and were actually part of the same overall event, they were presumed to have similar source characteristics. Also, since records were obtained at about the same epicentral distance at stations which had very similar site characteristics, the magnitudes of the earthquakes were considered to be the only major variable affecting the recorded ground motion.

Mean and mean-plus-one standard deviation (MSD)(2) response spectra for 5-percent damping computed with and without scaling for the six components

---

(2)For the remainder of this report, mean-plus-one standard deviation response spectra will be called MSD response spectra.

of recorded ground motion are shown in Figure 5-1. By comparing the MSD response spectra, it is apparent that scaling has had the expected effect of reducing the scatter of the individual response spectra.

The average local magnitude, $\bar{M}_L$, of the six unscaled records is 5.0. This is very close to the target magnitude of 4.95, to which the same records were scaled. If the scaling function is valid, the mean unscaled response spectrum ($\bar{M}_L = 5.0$) should be close to but slightly higher than the mean scaled response spectrum ($M_L = 4.95$). This is shown to be true in Figure 5-1. Some deviation from the results presented above is to be expected when the earthquakes and recording stations are not as closely matched.

In summary, the criteria used for selecting the site dependent strong motion records used in this study from the current library of western United States earthquake records was as follows:

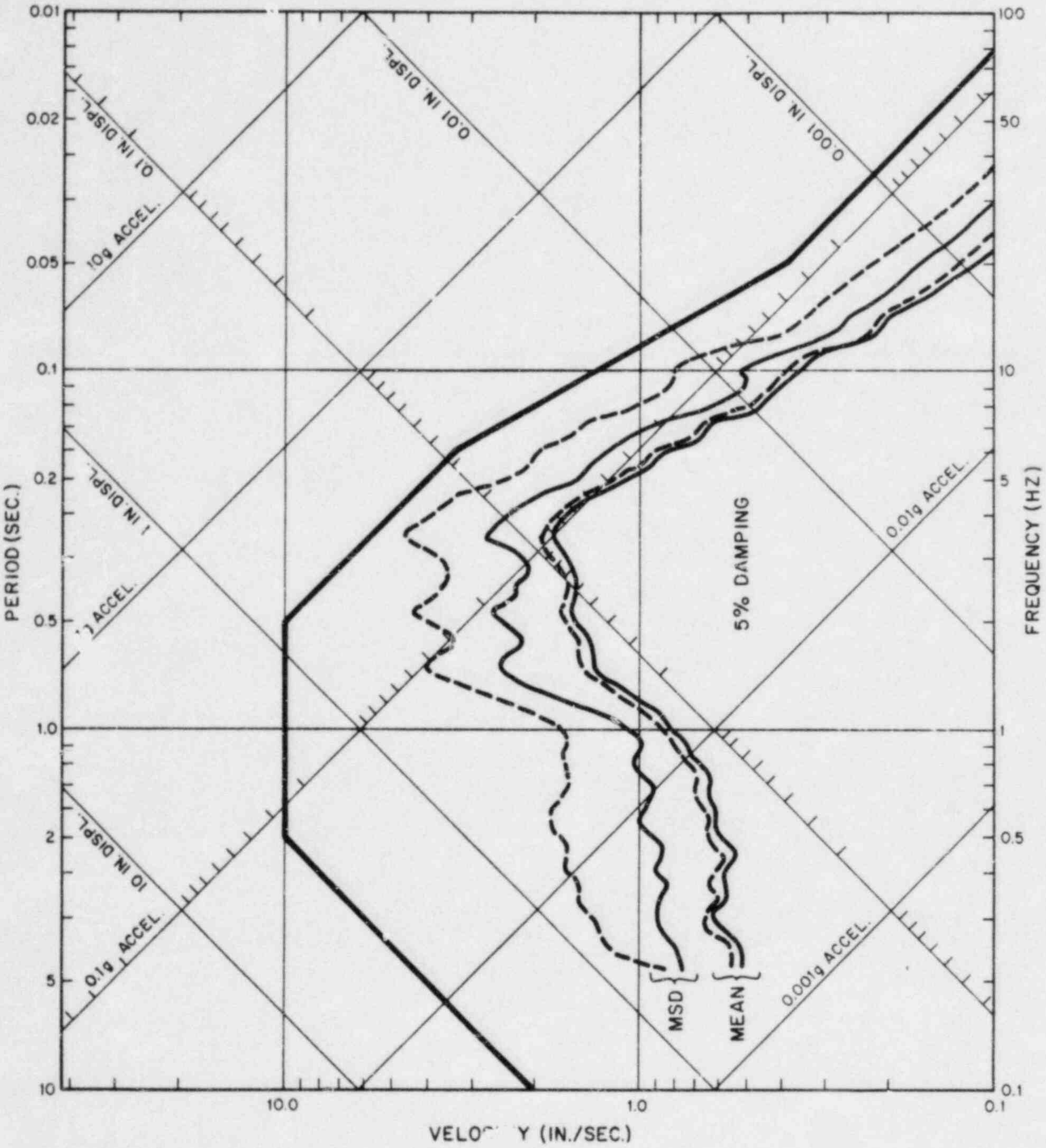
- Epicentral distance of about 25 km or less
- Earthquake records used without scaling

  $4.5 \leq M_L \leq 5.4$

- Earthquake records scaled to $M_L = 4.95$

  $3.5 \leq M_L \leq 6.4$

Accelerograph station data and earthquake data (epicentral location, magnitude, etc.) were obtained from the series of summary reports prepared by Shannon and Wilson, Inc. and Agbabian Associates (SW-AA 1976, 1978 a-c, 1980 a-g). The earthquake time histories used were obtained from the California Institute of Technology baseline corrected data tape (Trifunac and Lee 1973). The response spectra were computed using the computer program SHAKE (Schnabel et al 1972).



LEGEND

- BVPS-2
- SCALED
- UNSCALED

NOTE
REF. Nos. 3 THRU 8 (TABLE 6-2).

FIGURE 5-1
COMPARISON OF SCALED AND UNSCALED SITE MATCHED RESPONSE SPECTRA
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION

SECTION 6

SITE MATCHED RESPONSE SPECTRA ANALYSIS

As described in Section 5, the site matched response spectra analysis used earthquake records with magnitudes and epicentral distances corresponding to the BVPS-2 SSE that were obtained from accelerograph stations with site conditions matched as closely as possible to those of BVPS. Response spectra computed directly from these records were therefore considered to be applicable to BVPS-2.

6.1 SITE CONDITIONS AT BVPS

The major plant structures of BVPS-2 are founded on an alluvial terrace consisting of about 115 feet of interbedded sands, gravels and silty sands and gravels. Ground surface elevation in the main plant area is el 735 feet. Bedrock directly beneath BVPS-2 is a dark, gray carboneaceous shale with a top of rock elevation at about el 620 feet. Site geology and soil conditions are discussed in much greater detail in Sections 2.5.1 and 2.5.4, respectively, of the BVPS-2 FSAR.

Measurements were made of the in situ shear wave velocity of the terrace soils and the underlying bedrock, the results of which are shown in Figure 6-1. The 1968 survey was performed in the vicinity of the BVPS-1 reactor containment structure, and the 1977 survey was performed in the vicinity of the BVPS-2 fuel building after the soil densification program at BVPS-2. The soil conditions at the two locations are similar. The soil densification program is fully described in SWEC (1976) and the limits of densification in the main plant area are shown in BVPS-2 FSAR Figure 2.5.4-15.

A generalized shear wave velocity profile is shown in Figure 6-2. For in situ terrace soils outside the densification area, the shear wave velocity profile is identical to that suggested by Whitman (1968). In the area of the BVPS-2 fuel building, densification was performed between about el 670 feet and el 646 feet as shown in Figure 6-1. To account for the apparent increase in hear wave velocity after densification that is shown in Figure 6-1, the generalized profile for soils within the densified area shows an increased shear wave velocity between el 650 feet and el 620 feet.

6.2 SITE MATCHING PROCEDURE

The site matching procedure involved two steps. The first was to identify the accelerograph stations which had characteristics similar to BVPS-2. The two most important considerations were the soil and shear wave velocity profiles. A soil depth range of between 50 and 200 feet was used as a basic screening criterion in accordance with the classification of stiff soil sites established by SW-AA (1980a). The second step was to identify those site matched accelerograph stations from which earthquake records were available that met the magnitude and epicentral distance criteria established in Section 5.

A list of the accelerograph stations selected for this study, as well as a brief description of site conditions at the stations, is provided in Table 6-1. Available soil and shear wave velocity data for each station are provided in Appendix 2, and for comparison, the BVPS-2 measured shear wave velocity profile has been superimposed on the figures. The set of earthquake records recorded at these stations, which met the criteria of Section 5 and were used to determine site matched response spectra, is presented in Table 6-2.

Brief descriptions of each station are given below:

Alexander Building, San Fracisco, CA

This station is considered to be the best match to BVPS-2. The soil description and the depth to rock of 140 feet are a good match to BVPS-2. The shear wave velocity profile is almost identicial to the measured velocity profile at BVPS-2.

State Building, San Francisco, CA

Although the depth to bedrock at the station is greater than that at BVPS-2, the soil descriptions are similar. The shear wave velocity profiles match for the top 100 feet of the station profile.

City Hall, Oakland, CA

The depth to bedrock at this station is not known but the soil descriptions down to a depth of about 92 feet are similar to BVPS-2. There is also a close match of the shear wave velocity profiles within the depth range of the measurements.

Old Ridge Route, Castaic, CA

This station is classified as a stiff soil site by SW-AA (1980a) and the profile is described as a weathered sandstone. The shear wave velocity profile for the upper 50 to 70 feet is similar to that of BVPS-2. Between 50 and 70 feet there is an increase in shear wave velocity to about 2000 ft/sec and at 110 feet to about 2500 ft/sec, which is indicative of a sounder material. The profile was assumed to be similar to BVPS-2 in terms of amplification effects from earthquake ground motions, with about 50 to 70 feet of soil-like material overlying a more competent base.

6074 Park Drive, Wrightwood, CA

The station was selected based on the soil description as a sandy gravel, although the depth to bedrock was not known.

Federal Building, Eureka, CA

This station was included solely on the basis of the match between shear wave velocities in the upper 115 feet. There are over 350 feet of quaternary sediments (sand) at the station and, as such, it cannot be considered to be matched to BVPS-2 in terms of depth of soil overlying bedrock. However, in terms of shear wave ve` -ity in the upper 100 feet of soil at the

station, there is a close match to BVPS-2. The soil below 140 feet is apparently uniform and very dense with a constant shear wave velocity of about 2000 feet/sec. This dense material can be considered to be a base layer and, consequently, the site conditions here are effectively matched to BVPS-2.

6.3 SITE MATCHED RESPONSE SPECTRA

Site matched response spectra were computed for the earthquake records listed in Table 6-2. Two separate analyses were performed. The first involved computing the response spectra directly, without scaling, for the set of earthquakes with magnitudes that fell within the range of magnitudes corresponding to the BVPS-2 SSE ($M_L = 4.95 \pm 0.5$). The second analysis scaled all of the earthquake records to a target magnitude, M_L, of 4.95 prior to computing the response spectra. For each set of response spectra, mean and MSD response spectra were computed assuming a log-normal distribution.

6.3.1 Site Matched Response Spectra Without Scaling

The closest match to BVPS-2 site conditions was found to be at the two San Francisco accelerograph stations listed in Table 6-1: the Alexander Building and the State Building. As given in Table 6-2, two shocks of the 1957 San Francisco earthquake were recorded at these two stations with a total of six components recorded. Although the shock with an M_L of 4.4 was slightly outside the established range of magnitudes, it was considered acceptable for use without scaling. Response spectra at 5-percent damping were determined and are shown in Figure 6-3. The mean and MSD response spectra are shown in Figure 6-4. As shown, the BVPS-2 SSE response spectrum was found to conservatively envelope the MSD site matched response spectrum for all frequencies.

By considering the remaining accelerograph stations that were not as closely matched to BVPS-2 site conditions, two additional earthquakes (V-330 and W-334) and one additional record of the 1957 San Francisco earthquake (A-017) could be used. Response spectra computed from the twelve components recorded are shown in Figure 6-5. Mean and MSD response spectra are shown in Figure 6-6.

Initially, it was thought that the accelerograph station at the Federal Building in Eureka, California, should not be included in this study. However, as will be explained below, the Federal Building accelerograph station can be considered to be matched to the average site condition of the set of accelerograph stations and, thereby, to BVPS-2, also.

Mean and MSD response spectra shown in Figure 6-6 were determined for two cases, one for which the recordings of the 1962 earthquake (V-330) obtained at the Federal Building were included and one for which they were not. There is over 350 feet of soil at the location of the Federal Building and the accelerograph station was included in the set of site matched stations on the basis of the shear wave velocity match for the first 140 feet of material. The mean M_L is 4.96 for the four earthquakes not including the recordings at the Federal Building. The mean epicentral distance is 16.3 kilometers. By comparison, the 1962 earthquake recorded at the Federal

Building had an M_L of 5.0 and was recorded at a distance of 17.5 kilometers. Since the addition of the response spectra computed from the Federal Building did not signficantly change the mean and MSD response spectra shown in Figure 6-6, the Federal Building accelerograph station was considered to be matched to BVPS-2.

As can be seen in Figure 6-6, the BVPS-2 response spectrum for 5-percent damping conservatively envelopes the mean and MSD site matched response spectra computed for earthquake records without scaling. For frequencies less than about 2 Hz, the spectral displacements indicated by the BVPS-2 response spectrum are considerably higher than those of the mean and MSD site matched response spectra.

6.3.2 Site Matched Response Spectra With Scaling

Response spectra for all 18 of the component recordings listed in Table 6-2 and scaled to an M_L of 4.95 are shown in Figure 6-7. For this set of records, the mean and MSD response spectra are presented in Figure 6-8. Also shown in Figure 6-8 is the effect of including the four records obtained at the Eureka Federal Building. As was noted previously, the results are not significantly changed by the inclusion of the Eureka accelerograph records as site matched.

In Figure 6-9 the unscaled mean and MSD response spectra for the 12 unscaled component records (including Eureka) that were previously presented in Figure 6-6 are compared with the corresponding response spectra for the 18 component records scaled to an M_L of 4.95. The mean local magnitude, $\bar{M}_L$, of the 12 unscaled records was 4.97. The two mean response spectra are very similar, although in the low frequency range the unscaled mean response spectrum is somewhat lower than the scaled mean response spectrum.

The BVPS-2 response spectrum as seen in Figure 6-9 conservatively envelopes both the scaled and unscaled site matched response spectra for all frequency ranges. Spectral displacements shown in the BVPS-2 response spectrum for frequencies less than 2 Hz are considerably higher than those of the site matched response spectra. The MSD site matched response spectra indicate a zero period or ground acceleration of about 0.07g. This is in agreement with the estimated ground surface acceleration for an SSE of Intensity VI (MM).

The scaling procedure has been shown to be an effective means of increasing the number of earthquake records that can be used to determine site matched response spectra. Scaling to a target magnitude is more reasonable than using a range about a target magnitude of ±0.5 magnitude units, since this amounts to a difference in energy release between the lowest and the highest magnitude of about 10 times. A certain degree of conservatism is still maintained if the site matched response spectrum is specified as the MSD response spectrum.

Therefore, the BVPS-2 SSE site matched response spectrum for 5-percent damping is shown in Figure 6-10, and it corresponds to the MSD response spectrum for the 18 recorded components scaled to an M_L of 4.95. The BVPS-2 design response spectrum for 5-percent damping is seen to conservatively envelope the site matched response spectra for all frequency ranges.

TABLE 6-1

SITE MATCHED ACCELEROGRAPH STATIONS[1]

Station	Location	Site Conditions[2]
Alexander Building	San Francisco, CA	140 ft of Quaternary sediments (sand) overlying Franciscan Assembledge (bedrock)
State Building	San Francisco, CA	211 ft of Quaternary sediments (sand) overlying Franciscan Assembledge (bedrock)
City Hall	Oakland, CA	>92 ft of Quaternary sediments (layered sand and clay). Depth to rock not known.
Oak Ridge Route	Castaic, CA	Castaic formation. Weathered sandstone.
6074 Park Drive	Wrightwood, CA	>90 ft of alluvial gravel. Depth to rock not known.
Federal Building	Eureka, CA	>350 ft of Quaternary and Pleistocene sediments (sand). Depth to rock not known.

(1) Reference: SW-AA (1980)

(2) Available site data are provided in Appendix 2

TABLE 6-2

SITE MATCHED GROUND SURFACE EARTHQUAKE RECORDS

Date: Year	Date: Month	Date: Day	Epicenter Location	Magnitude (M_L)	Recording Station	Epicenter Distance D (km)	Component	Peak Acceleration (g): Unscaled	Peak Acceleration (g): Scaled	Scaling Factor to M_L = 4.95	CIT Record No.[2]	Ref. No.
1954	12	21	Eureka, CA	6.5	Federal Building, Eureka, CA	6.4	N79E S11E	0.257 0.168	0.038 0.024	0.146	A-008	1 2
1957	03	22	San Francisco, CA	5.3[1]	State Bldg., San Francisco, CA	12.8	N09E S81W	0.085 0.056	0.056 0.036	0.65	A-016	3 4
					Alexander Bldg., San Francisco, CA	14.4	N09W N81E	0.043 0.046	0.028 0.030	0.65	A-014	5 6
1957	03	22	San Francisco, CA	4.4[1]	Alexander Bldg., San Francisco, CA	16.0	N09W N81E	0.019 0.016	0.038 0.032	1.98	V-323	7 8
					City Hall, Oakland, CA	24.0	N26E S64E	0.040 0.024	0.079 0.049	1.98	A-017	9 10
1962	09	04	Northern, CA	5.0[1]	Federal Bldg., Eureka, CA	17.6	N79E S11E	0.046 0.048	0.046 0.048	1.0	V-330	11 12
1965	07	15	Southern, CA	4.0	Oak Ridge Rte., Castaic, CA	14.4	E S	0.037 0.041	0.119 0.134	3.26	V-331	13 14
1970	09	12	Lytle Creek, CA	5.4[1]	6074 Park Dr., Wrightwood, CA	14.4	S65E S25W	0.142 0.198	0.081 0.113	0.572	W-334	15 16

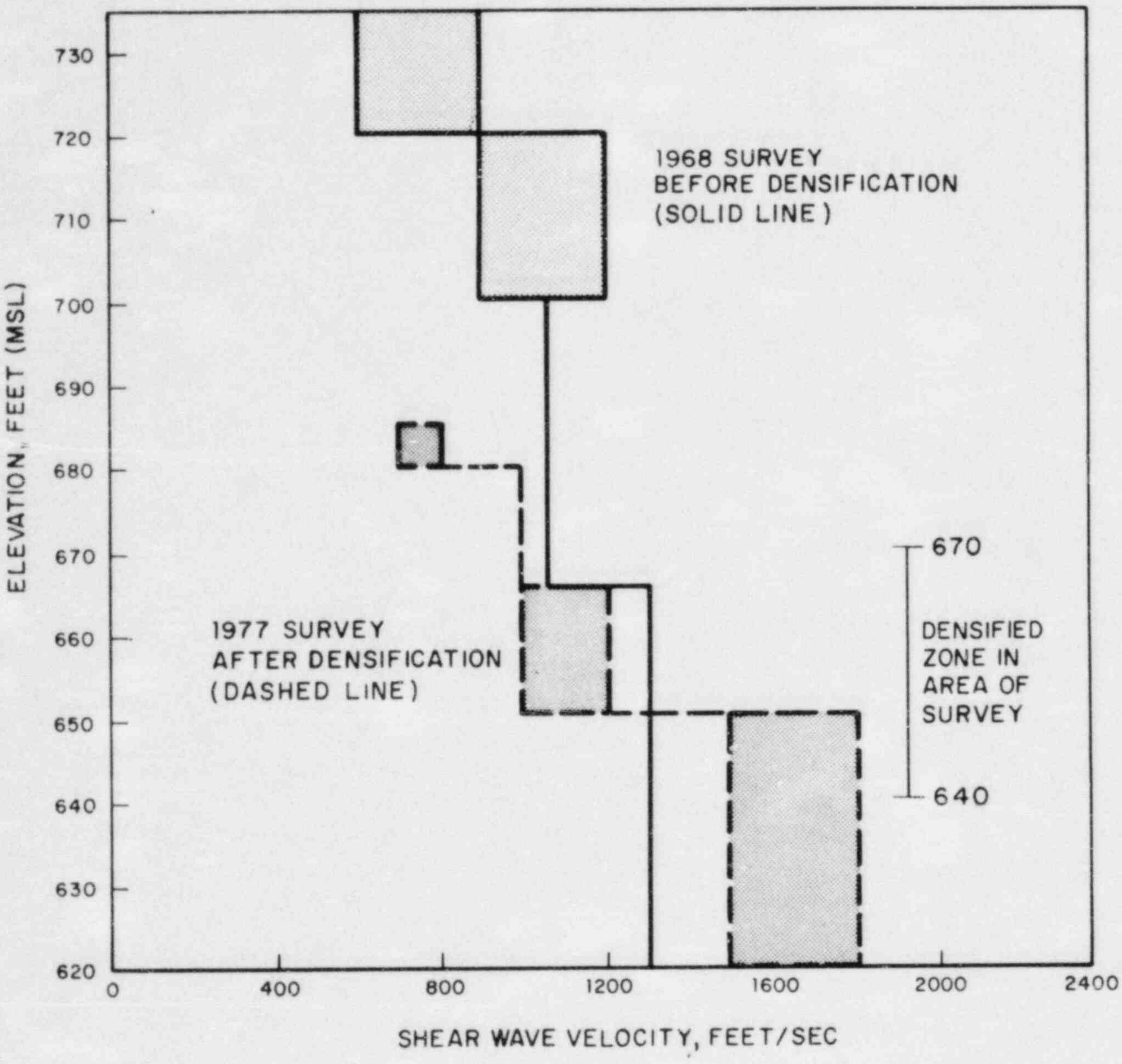
TABLE 6-2 (Cont)

Date			Epicenter Location	Magnitude (M_L)	Recording Station	Epicenter Distance D (km)	Component	Peak Acceleration (g)		Scaling Factor to M_L = 4.95	CIT Record No.(2)	Ref. No.
Year	Month	Day						Unscaled	Scaled			
1971	02	09	San Fernando, CA	6.4	Oak Ridge Rte., Castaic, CA	28.8	N21E N69W	0.315 0.270	0.052 0.045	0.165	D-056	17 18

Notes:

(1)Response spectra were computed directly, without scaling, from records obtained for these earthquakes.

(2)California Institute of Technology record number in accordance with Trifunac and Lee (1973).

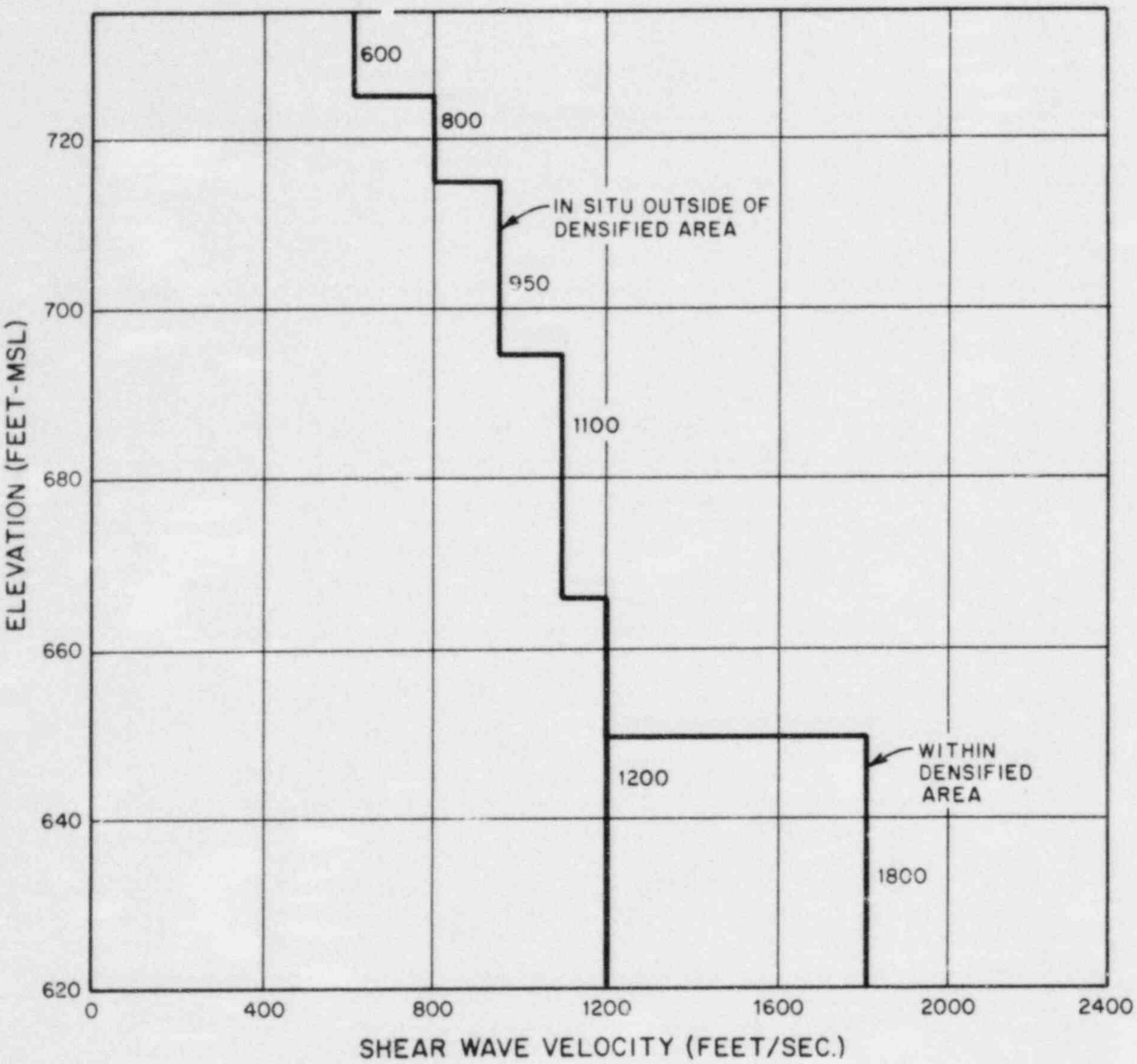


NOTE
GROUND SURFACE AT EL. 735 FT. FOR 1968 SURVEY AND AT EL. 715 FT. FOR 1977 SURVEY.

DLC 1976

REFERENCE
SWEC, 1976

FIGURE 6-1
COMPARISON OF IN SITU SHEAR WAVE VELOCITIES BEFORE AND AFTER DENSIFICATION
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION

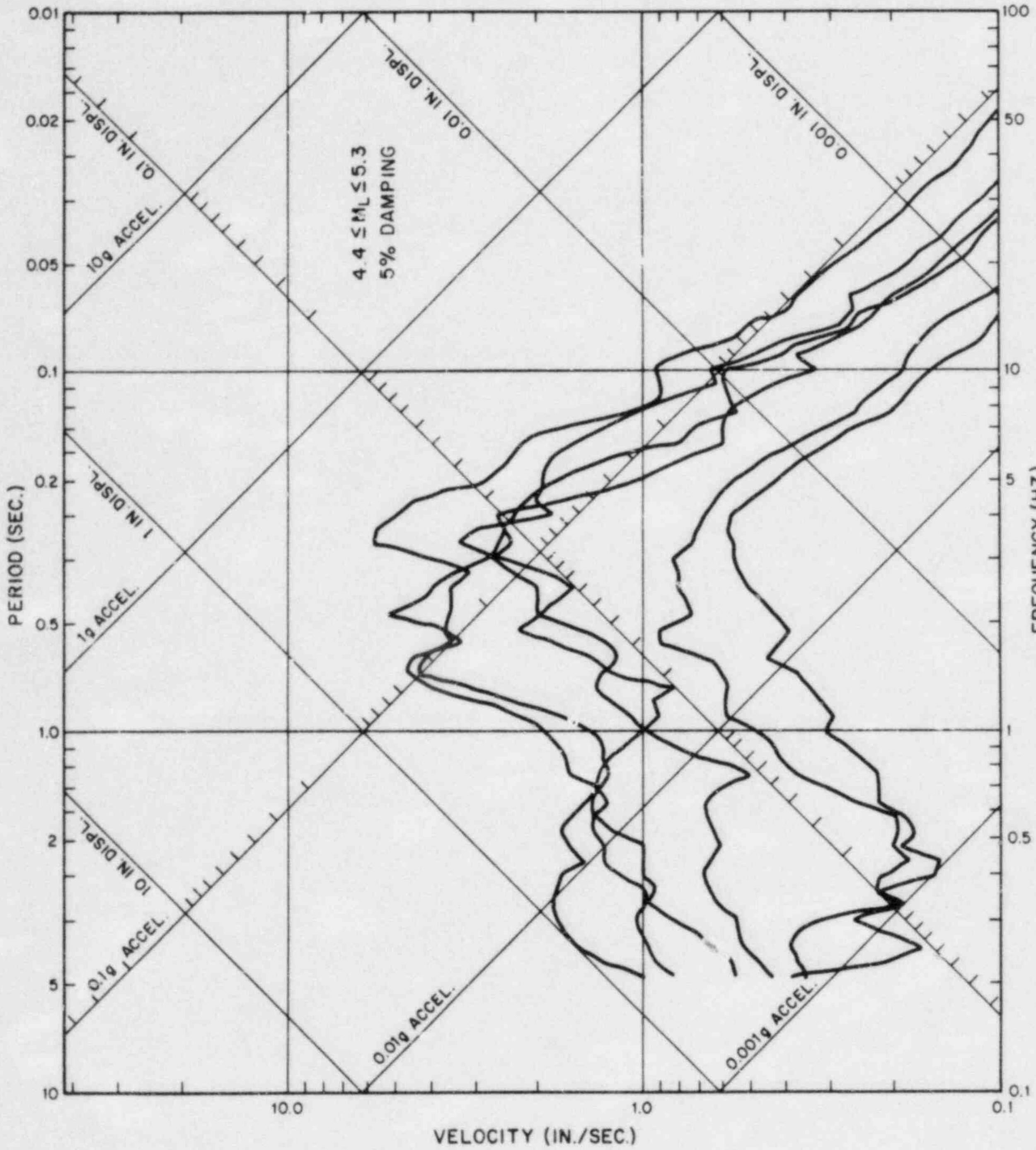


$T = 4H/V_S$; $f = 1/T$

WHERE: T = NATURAL PERIOD OF SOIL PROFILE (SEC.).
f = NATURAL FREQUENCY (H_Z).
H = DEPTH OF SOIL PROFILE (FT.).
V_S = LAYER THICKNESS WEIGHTED AVERAGE SHEAR WAVE VELOCITY.

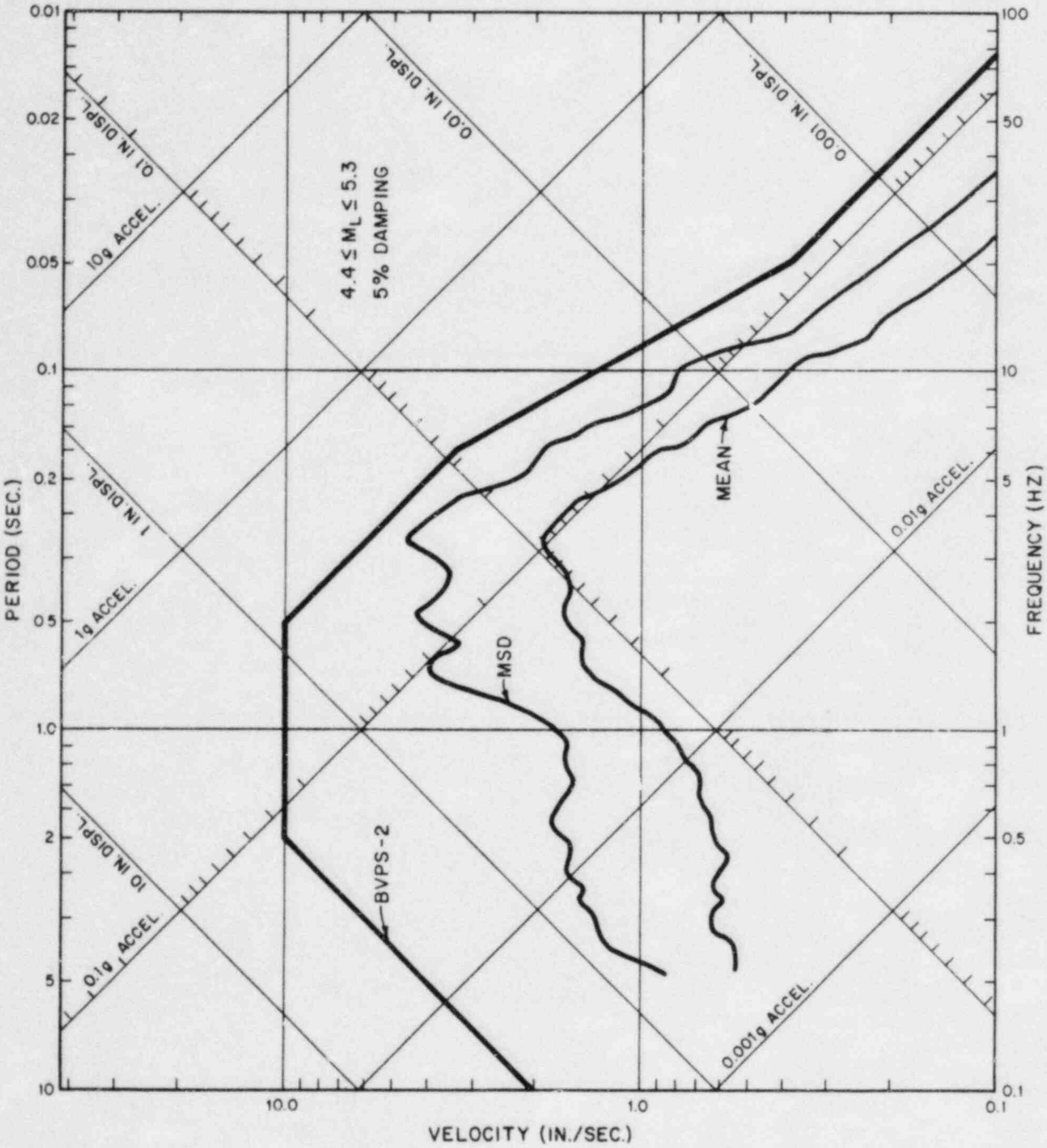
IN SITU SOIL PROFILE: f = 2.3 H_Z.
DENSIFIED AREA PROFILE: f = 2.6 H_Z.

FIGURE 6-2
GENERALIZED SHEAR WAVE VELOCITY PROFILES
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



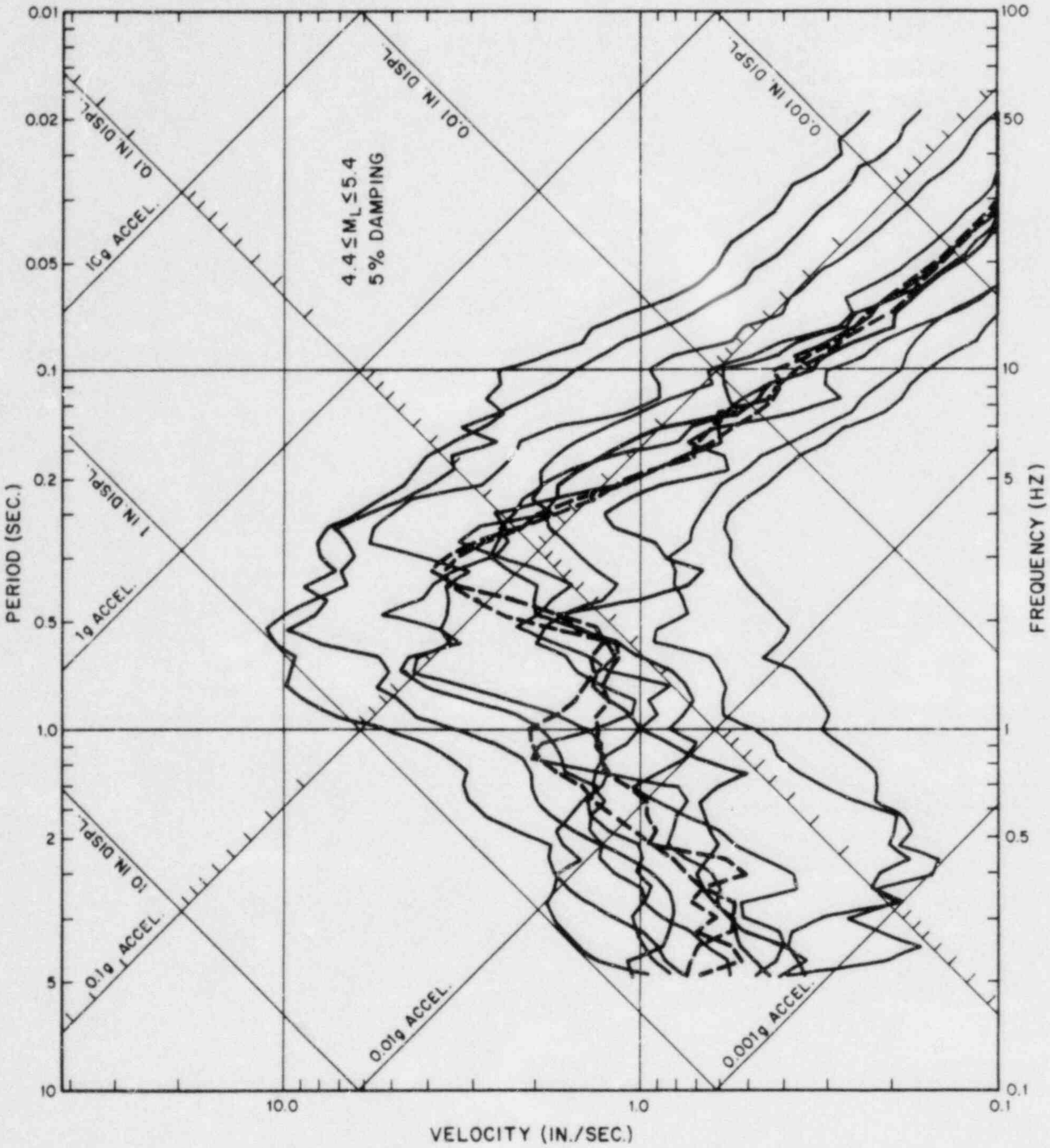
NOTE
REF. Nos. 3 THRU 8 (TABLE 6-2).

FIGURE 6-3
SITE MATCHED RESPONSE SPECTRA (UNSCALED)
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



NOTE
REF. Nos. 3 THRU 8 (TABLE 6-2).

FIGURE 6-4
MEAN & MEAN PLUS ONE STANDARD DEVIATION SITE MATCHED RESPONSE SPECTRA (UNSCALED)
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



LEGEND

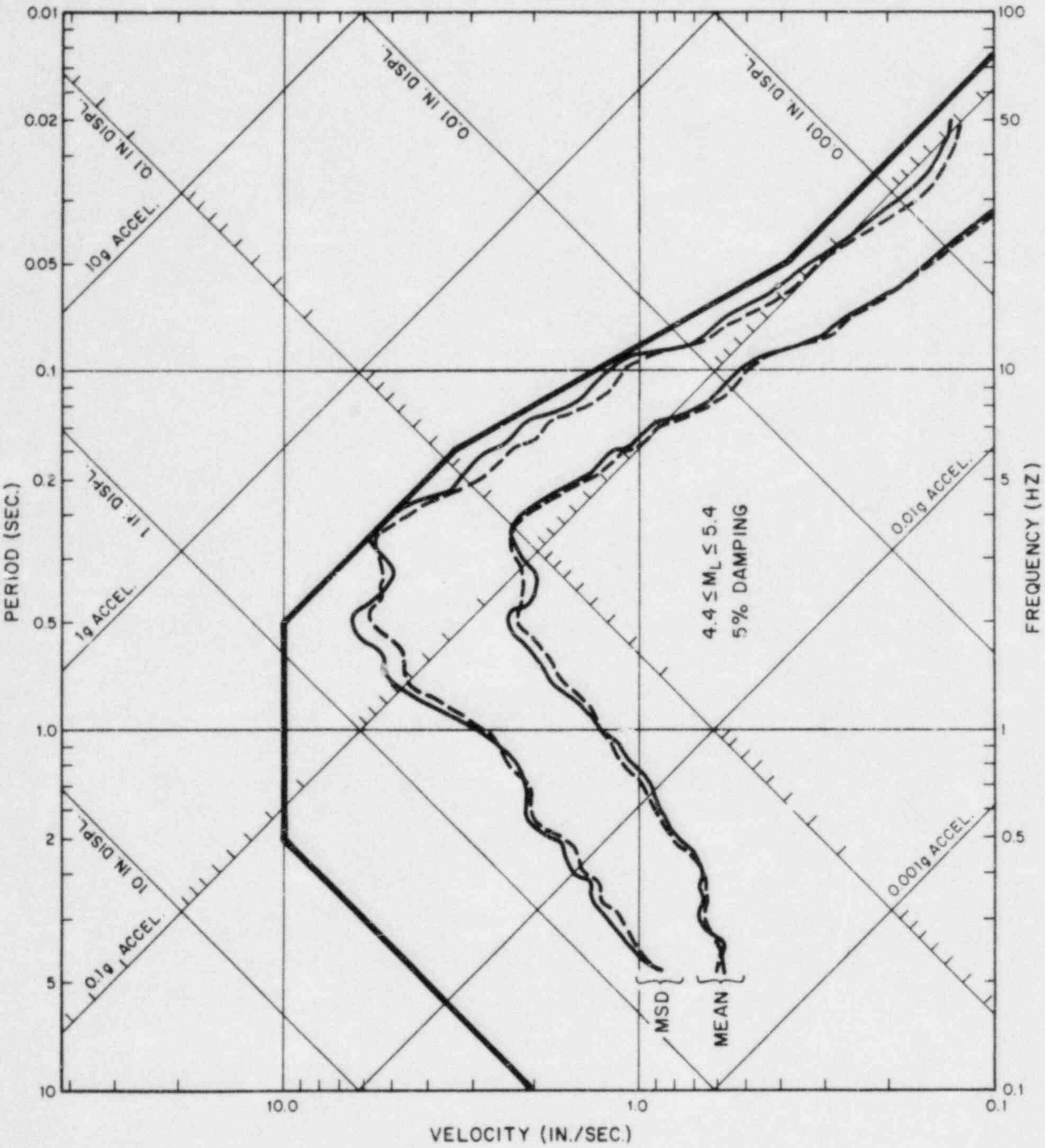
FEDERAL BUILDING-EUREKA, CA. (REF. Nos. 11 & 12)

NOTES

REF. Nos. 3 THRU 12, 15 & 16. (TABLE 6-2).

AVERAGE M_L = 4.97.

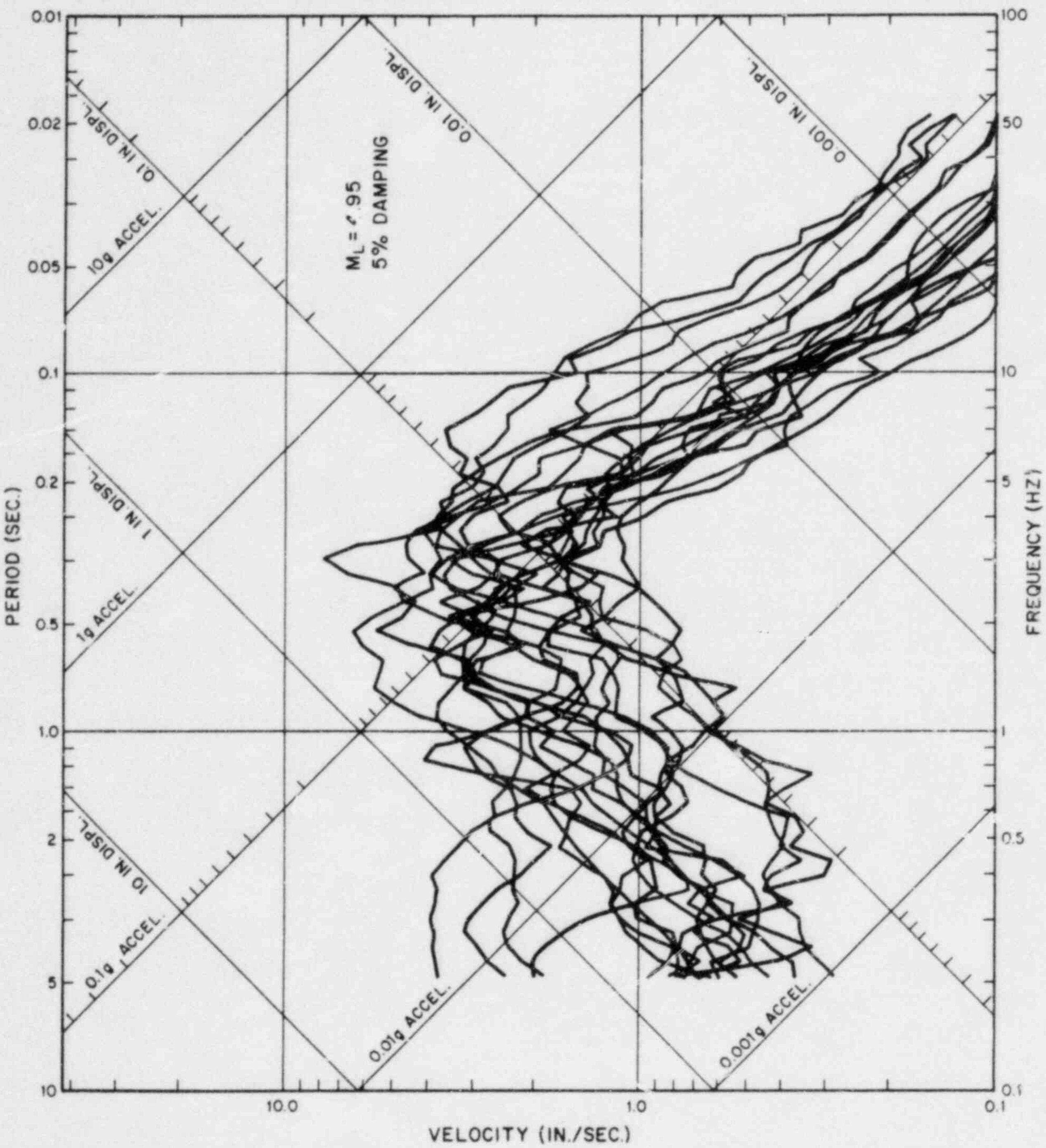
FIGURE 6-5
SITE MATCHED RESPONSE SPECTRA (UNSCALED)
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



LEGEND

- BVPS-2 (heavy solid line)
- WITHOUT FEDERAL BUILDING RECORDS (REF. Nos. 3 THRU 10, 15 & 16) (solid line)
- WITH FEDERAL BUILDING RECORDS (REF. Nos. 3 THRU 12, 15 & 16) (dashed line)

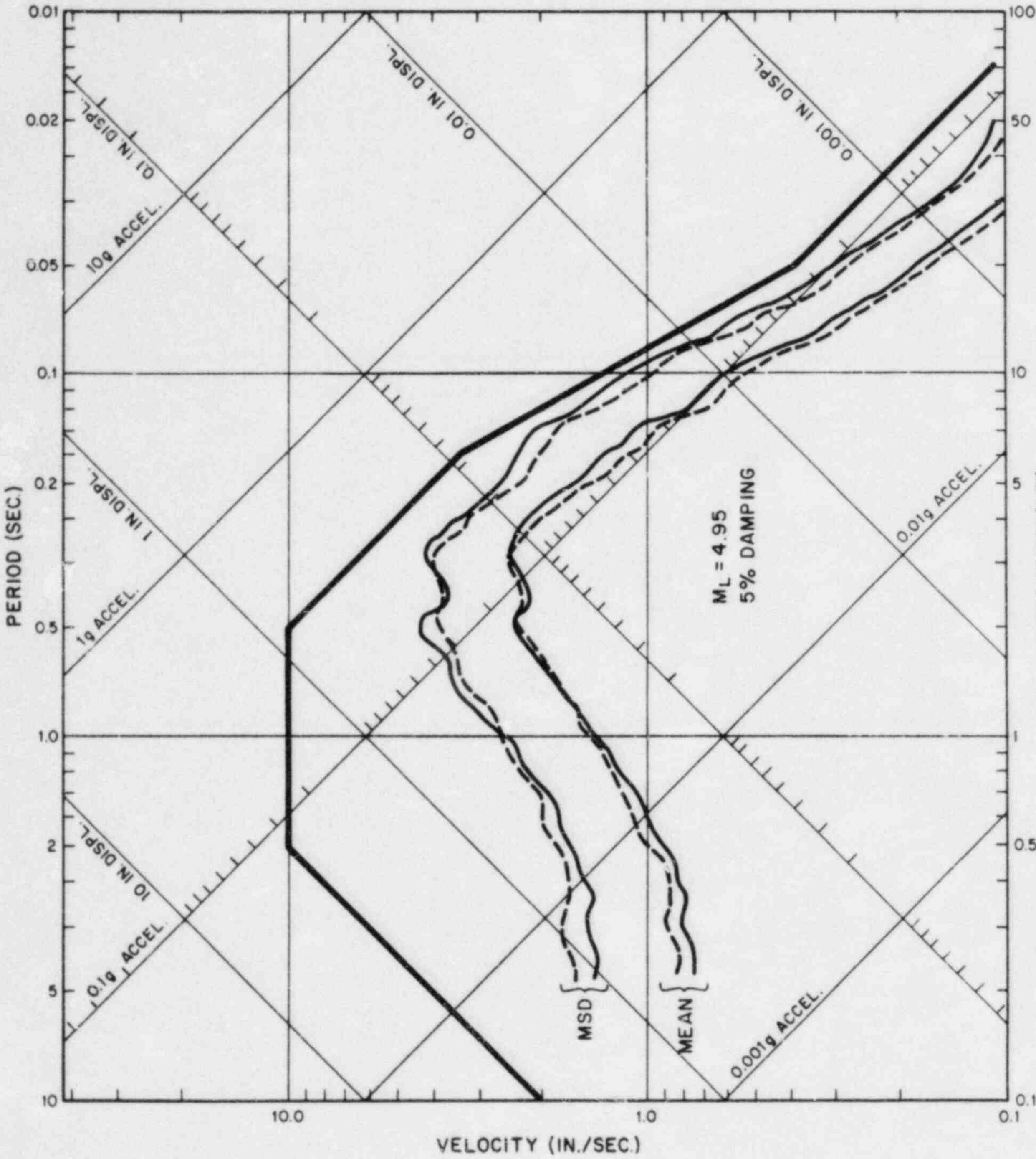
FIGURE 6-6
SITE MATCHED RESPONSE SPECTRA (UNSCALED)-EFFECT OF FEDERAL BUILDING RECORDS
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



NOTE

REF. Nos. 1 THRU 18 (TABLE 6-2).

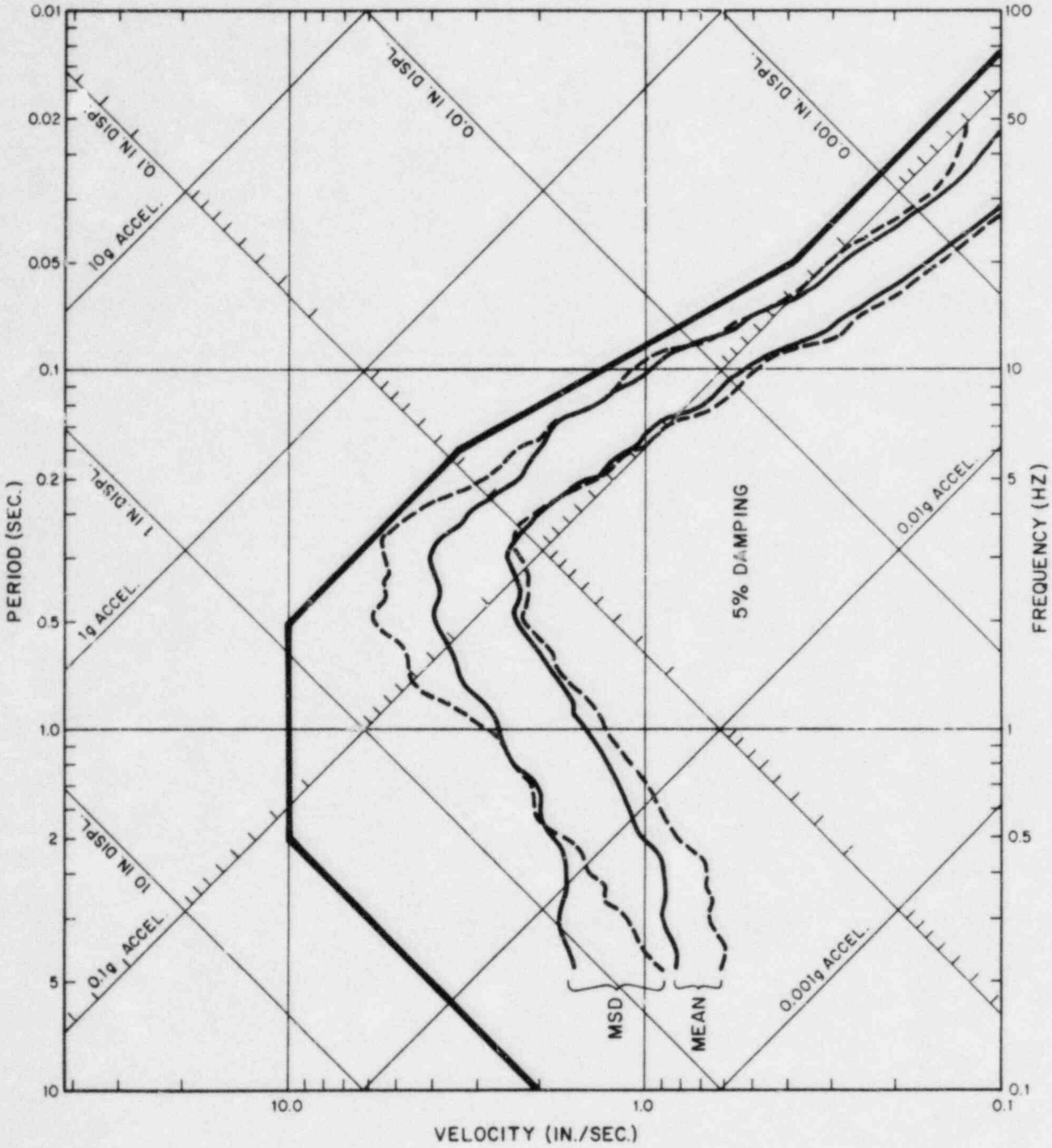
FIGURE 6-7
SITE MATCHED RESPONSE SPECTRA (SCALED)
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



LEGEND

- BVPS-2
- WITHOUT FEDERAL BUILDING RECORDS (REF. Nos. 3 THRU 10 & 13 THRU 18)
- WITH FEDERAL BUILDING RECORDS (REF. Nos. 1 THRU 18)

FIGURE 6-8
SITE MATCHED RESPONSE SPECTRA (SCALED)-EFFECT OF FEDERAL BUILDING RECORDS
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



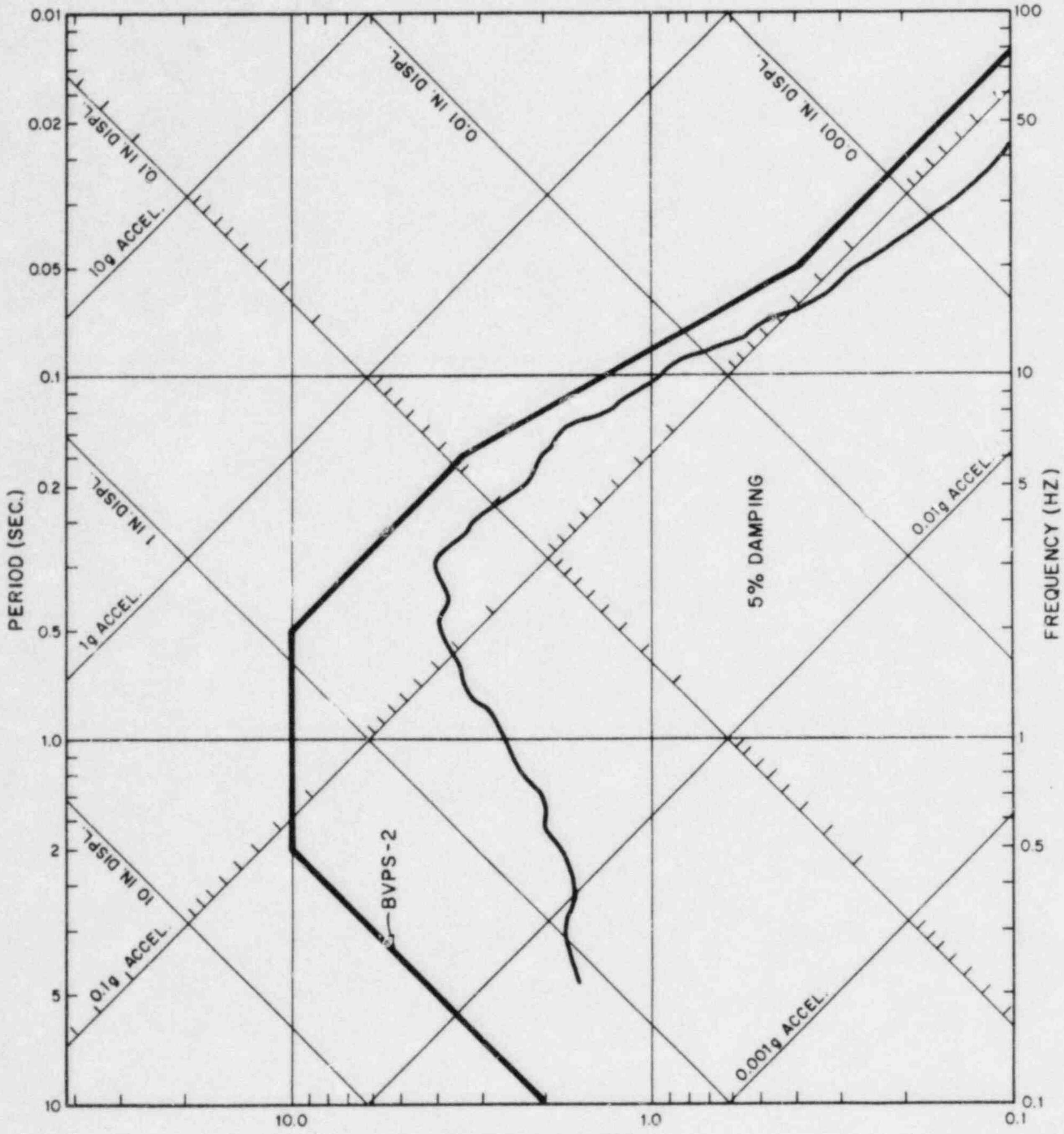
LEGEND

BVPS-2

SCALED - $M_L = 4.95$; $\bar{D} = 16.5$ KM
(REF. Nos. 1 THRU 18)

UNSCALED - $\bar{M}_L = 5.0$; $\bar{D} = 16.5$ KM
(REF. Nos. 3 THRU 12, 15 & 16)

FIGURE 6-9
COMPARISON OF SCALED AND UNSCALED SITE MATCHED RESPONSE SPECTRA
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



NOTE

THE SITE MATCHED RESPONSE SPECTRA SHOWN REPRESENTS THE MSD RESPONSE SPECTRA FOR THE NINE EARTHQUAKES IN TABLE 6-2 SCALED TO A M_L = 4.95.

FIGURE 6-10
SITE MATCHED RESPONSE SPECTRUM
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION

SECTION 7

SITE DEPENDENT RESPONSE SPECTRA FROM SOIL RESPONSE ANALYSIS

Further study to evaluate site dependent response spectra was undertaken using the approach of soil response analysis. Basically, the computer program SHAKE (Schnabel et al 1972) was used to determine the ground surface response of the soil profile in the main plant area as the result of earthquake records input at the bedrock level. SHAKE is a one-dimensional wave propagation model that computes the horizontal response of a horizontally layered, visco-elastic system to vertically propagating shear waves, accounting for the shear strain dependency of soil properties. The soil response analysis consisted of the four basic steps outlined below and illustrated in Figure 7-1 (SW-AA 1975):

Step A: Soil Model: An appropriate soil profile model was developed for soil conditions in the main plant area.

Step B: Rock Outcrop Motions: A suitable ensemble of earthquake records from accelerograph stations on rock outcrops was selected.

Step C: Subsurface Rock Motions: Using SHAKE, the outcrop motions were transformed into an ensemble of subsurface rock motions.

Step D: Ground Surface Motions and Response Spectra: Using SHAKE, the ensemble of subsurface rock motions from Step C were amplified through the soil profile model to determine the ground surface motion for which response spectra were determined.

7.1 SOIL MODEL

Soil conditions in the BVPS-2 main plant area were briefly described in Section 6.1. The model used for the soil response analysis is shown in Figure 7-2. Two shear wave velocity profiles were considered, one for in situ soils within the densified area and one for in situ soils outside the densified area. Shear wave velocities and layer thicknesses for the model correspond to those shown for the generalized profile in Figure 6-2. The elevation of the ground water was assumed to correspond to the normal water level of el 665 feet.

To compute soil response, the program SHAKE iterates to obtain values of soil shear modulus and damping that are compatible with the strain levels induced by earthquake motions. Shear moduli corresponding to the shear wave velocities shown in Figure 7-2 represent low strain or maximum values. The strain dependent variations of shear modulus and damping used for this analysis were based upon the data presented by Seed and Idress (1970), and are presented in Figure 7-3.

7.2 ROCK OUTCROP MOTIONS

Suitable rock outcrop motion recordings were identified using the series of summary reports prepared by Shannon and Wilson, Inc. and Agbabian Associates (SW-AA 1976, 1978 a-c, 1980 a-g). Table 7-1 lists those earthquake records

which had local magnitudes in the range of 4.95 ± 0.5 which were used without scaling. Table 7-2 lists those earthquake records which were scaled to a local magnitude of 4.95. Available boring data at the accelerograph stations from which records were obtained are given in Appendix 3.

Four earthquakes are identified in Table 7-2 as having occurred near Helena, Montana, in 1935. SW-AA (1980a) assigned a local magnitude only to the first earthquake listed. The local magnitude of the remaining three Helena earthquakes were estimated by Kanamori and Jennings (1978). They used strong motion accelerograph records to synthesize Wood-Anderson torsion seismograph records, which were then read in the normal manner to determine local magnitudes. Data was presented which showed that, on the average, local magnitudes which were determined using their procedure agreed quite well with those determined from Wood-Anderson seismograph records. However, the local magnitude for a given earthquake determined using their procedure from any one particular strong motion accelerogram could be in error by as much as 0.3 to 0.5 magnitude units. The local magnitude of the first earthquake in Table 7-2 was 6.0. Kanimori and Jennings (1978) estimated that the local magnitudes of the component recordings of this earthquake were 5.3 and 5.7 and averaged 5.5, a difference of 0.5 magnitude units. All of the earthquakes listed in Table /-2 were scaled to a target magnitude of 4.95. If it is assumed that the magnitudes of the three Helena earthquakes that were determined by Kanimori and Jennings (1978) were also underestimated by 0.5 magnitude units, the effect would be that the scaled records used were actually too large in terms of magnitude. It is possible that the two earthquakes with local magnitudes less than 4.95 should have used a smaller scaling factor and the earthquake with a local magnitude of 5.0 should have used a scaling factor less than 1.0. For the six component recordings of these three Helena earthquakes, the computed ground response would be smaller. This is discussed further in Section 7.4.

7.3 SOIL RESPONSE ANALYSIS WITHOUT SCALING

Only three earthquakes with six outcrop recordings were identified for use directly without scaling. The three earthquakes are listed in Table 7-1. Response spectra which were determined from the ground surface motions computed for the outcrop motions amplified through the in situ soil profile outside of the densified area (hereafter called the in situ soil profile) are shown in Figure 7-4. Similarly, response spectra for the area affected by the densification program (hereafter called the densified area) are shown in Figure 7-5. A comparison of the mean-plus-one standard deviation (MSD) response spectra for the two soil profiles is provided in Figure 7-6. (The mean response spectra are fairly close to the MSD response spectra and are not shown.) The MSD response spectra are almost identical until 2 Hz. For frequencies greater than 2 Hz, the MSD response spectra for the densified area soil profile are higher than those for the in situ profile. Also, for frequencies greater than 2 Hz, both MSD response spectra exceed the BVPS-2 response spectra.

7.4 SOIL RESPONSE ANALYSIS WITH SCALING

Since the number of unscaled outcrop recordings found suitable was small, the scaling law described in Section 5 was used. The ground surface response spectra determined for the 18 scaled outcrop recordings listed

in Table 7-2 are presented in Figures 7-7 and 7-8 for the in situ and densified area profiles, respectively. A comparison of the mean and MSD response spectra for the in situ and densified area profiles is shown in Figure 7-9. Since the MSD response spectra for the two soil profile models were not significantly different, a response spectrum based upon the average of the MSD response spectra for the two soil profile models was determined and is shown in Figure 7-10. The scaled MSD response spectra were determined from 18 outcrop recordings, while the unscaled MSD response spectra were determined from only 6 recordings. Statistically, with the larger data base, the scaled MSD response spectra are considered more appropriate.

As can be seen from Figure 7-10, the response spectrum of the soil profile model exceeds the BVPS-2 response spectrum for frequencies greater than 5 Hz. However, the response of the soil profile model may be overestimated for reasons discussed below.

The magnitudes of the three Helena earthquakes listed in Table 7-2 that were determined by Kanimori and Jennings (1978) may have been underestimated (see Section 7.2). Consequently, the response spectra determined from the scaled outcrop recordings for these earthquakes may have spectral amplitudes that are too large. If true, than the mean and MSD response spectra may actually be somewhat lower.

The mathematical model used in SHAKE is an equivalent linear elastic model. To simulate the nonlinear, elasto-plastic behavior of the real soil deposit under earthquake loading, the equivalent linear elastic model has a tendency to accentuate soil response near the fundamental frequency as well as at the higher frequency multiples of the fundamental frequency. For frequencies less than the fundamental frequency, the equivalent linear elastic model has a tendency to underestimate the displacement response. (The fundamental frequency of the BVPS-2 soil profile model is approximately 2-3 Hz).

SHAKE may overestimate the amplitude of subsurface rock motion, with the subsequent effect that the response of the soil deposit is also overestimated, as will be explained below. Rock outcrop recordings of earthquake motions were used in this analysis to compute the response of the soil deposit caused by earthquake shaking in the base rock. SHAKE transformed the rock outcrop free surface motions to base rock motions; i.e., motions within a half space overlain by soil layers. The procedure used by SHAKE considers only the theoretical boundary condition effects on seismic wave propagation and is independent of rock properties. The transformation results in a reduction to the peak accelerations of the rock outcrop time history. The procedure is also independent of topography which is known to have a pronounced effect on the amplitude of rock motions (Boore 1973). Studies of rock motions measured during Japanese earthquakes have indicated peak accelerations on locally weathered and cracked rock to be considerably higher than on sound bedrock (Okamoto and Mizukoshi 1967). In general, it is expected that subsurface rock would be less weathered and cracked than that of the rock outcrops and that local topography effects would be more pronounced for outcrop motions than for subsurface rock motions. Consequently, the computed subsurface rock motions may be too large.

TABLE 7-1

ROCK OUTCROP MOTIONS WITHOUT SCALING

Date Year	Mo.	Day	Epicenter Location	Magnitude M_L	Recording Station	Epicenter Distance (km)	Component	Acceleration (g) Rock	Ground Surface In Situ	Densified	CIT Record No.	Ref. No.
1935	11	28	Helena, MT	5.0[1]	Federal Bldg. Helena, MT	5.8	NS	0.076	0.121	0.148	U-297	1
							EW	0.084	0.134	0.162		2
1957	03	22	SanFrancisco, CA	5.3	Golden Gate Pk, San Francisco, CA	11.2	S80E	0.105	0.214	0.187	A-015	3
							N10E	0.083	0.150	0.198		4
1970	09	12	Lytle Creek, CA	5.4	Allen Ranch Cedar Springs, CA	19.2	S85E	0.071	0.148	0.163	W-335	5
							S05W	0.056	0.150	0.164		6

NOTES:

(1) Magnitude estimated by Kanamori and Jennings (1978)

TABLE 7-2

ROCK OUTCROP MOTIONS
SCALED TO M_L = 4.95

Year	Mo.	Day	Epicenter Location	Magnitude M_L	Recording Station	Epicenter Distance (km)	Component	Initial Rock Accel. (g)	Scaling Factor	Scaled Peak Acceleration (g) Rock	Scaled Peak Acceleration (g) Ground Surface In Situ	Scaled Peak Acceleration (g) Ground Surface Densfied	CIT Record No. (2)	Ref. No.
1935	10	31	Helena, MT	6.0	Carol College,	6.6	EW	0.145	0.271	0.039	0.073	0.099	B-025	1
					Helena, MT		NS	0.146		0.040	0.075	0.083		2
1935	10	31	Helena, MT	4.0 (1)	Federal Bldg.,	5.8	NS	0.030	3.26	0.098	0.160	0.249	U-295	3
					Helena, MT		EW	0.026		0.084	0.116	0.202		4
1935	11	21	Helena, MT	3.8 (1)	Federal Bldg.,	5.8	EW	0.011	4.18	0.047	0.074	0.094	U-296	5
					Helena, MT		NS	0.007		0.031	0.059	0.077		6
1935	11	28	Helena, MT	5.0 (1)	Federal Bldg.,	5.8	NS	0.076	1.0	0.076	0.121	0.148	U-297	7
					Helena, MT		EW	0.085		0.085	0.134	0.162		8
1957	03	22	San Francisco,CA	5.3	Golden Gate Pk.	11.2	S80E	0.105	0.65	0.068	0.137	0.142	A-015	9
					San Francisco,CA		N10E	0.083		0.054	0.101	0.138		10
1970	09	12	Lytle Creek, CA	5.4	Allen Ranch,	19.2	S05W	0.056	0.572	0.032	0.096	0.090	W-335	11
					Cedar Springs,CA		S85E	0.071		0.041	0.087	0.103		12
1971	02	09	San Fernando, CA	6.4	Array No. 4	28.8	S69E	0.171	0.165	0.028	0.056	0.081	J-142	13
					Lake Hughes, CA		S21W	0.146		0.024	0.067	0.055		14
					Array No. 9	28.6	N21E	0.122	0.165	0.020	0.061	0.053	J-143	15
					Lake Hughes, CA		N69W	0.111		0.018	0.044	0.056		16
					Array No. 12	24.0	N69W	0.283	0.165	0.047	0.086	0.086	J-144	17
					Lake Hughes, CA		N21E	0.353		0.058	0.126	0.147		18

NOTES:

(1) Estimated by Kanamori and Jennings (1978)

(2) California Institute of Technology reference number, Trifunac and Lee (1973)

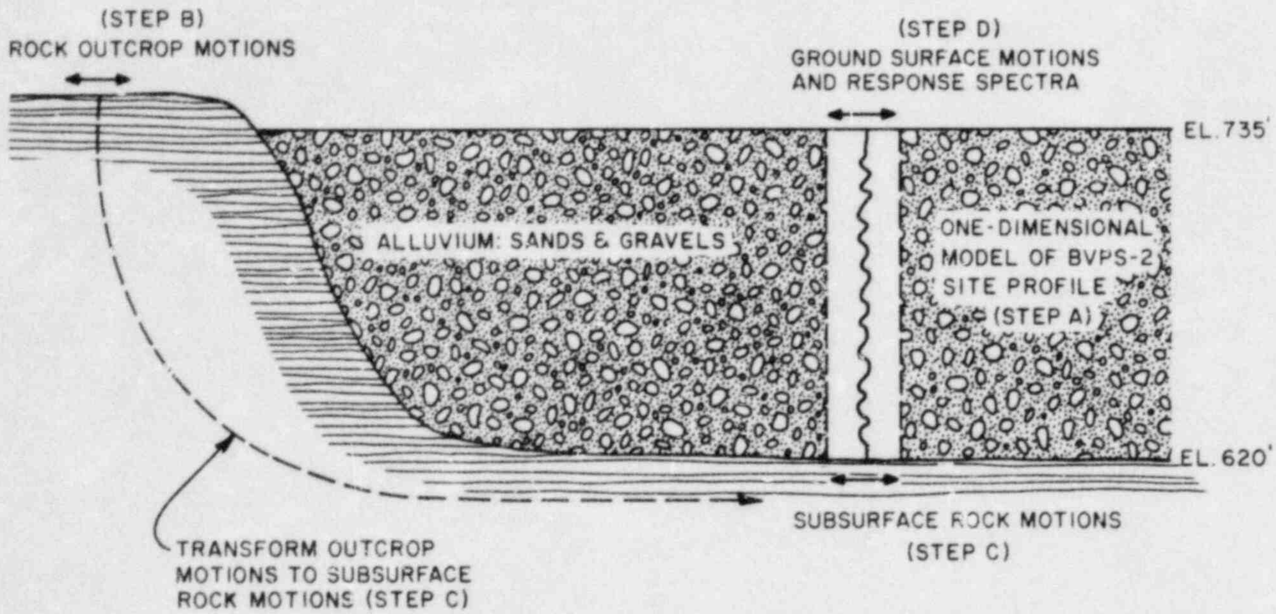


FIGURE 7-1
SOIL RESPONSE ANALYSIS
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION

REFERENCE: SW-AA (1975)

LAYER No.	LAYER THICKNESS (FT.)	UNIT* WEIGHT FT. (RCF)	SHEAR WAVE VELOCITY** (FT./SEC.)	
			IN SITU	DENSIFIED
1	10	125	600	600
2	10	125	800	800
3	10	125	950	950
4	10	125	950	950
5	10	125	1100	1100
6	10	125	1100	1100
7	10	125	1100	1100
8	7.5	136	1200	1200
9	7.5	136	1200	1200
10	10	136	1200	1800
11	10	136	1200	1800
12	10	136	1200	1800
BASE LAYER	HALF SPACE	160	5000	5000

ELEV. (FT.)
735
725
715
705
695
685
675
665
657.5
650
640
630
620

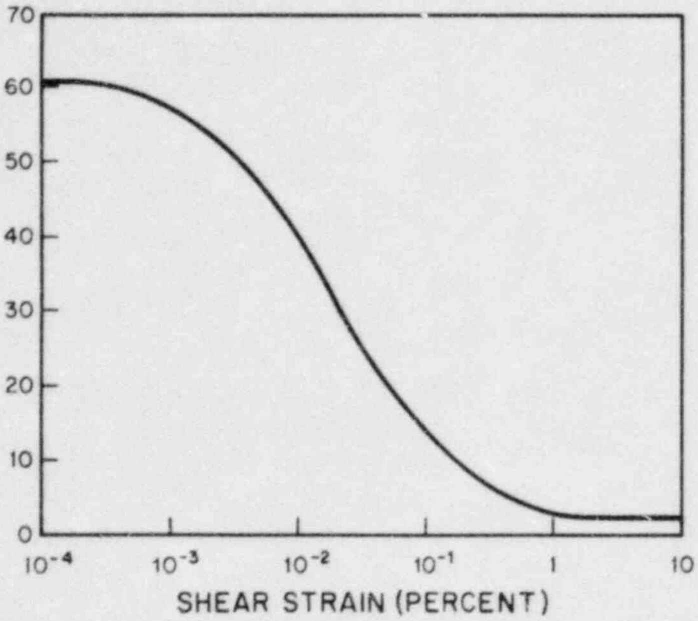
DEPTH (FT.)
0
10
20
30
40
50
60
70
77.5
85
95
105
115

SAND & GRAVEL

ROCK

NOTES
* UNIT WEIGHT FROM BVPS-2 FSAR SECTION 2.5.4.
** SHEAR WAVE VELOCITY FROM FIGURE 6-2.
IN SITU: NATURAL FREQUENCY = 2.3 H_Z
DENSIFIED: NATURAL FREQUENCY = 2.6 H_Z

FIGURE 7-2
SOIL MODEL
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION

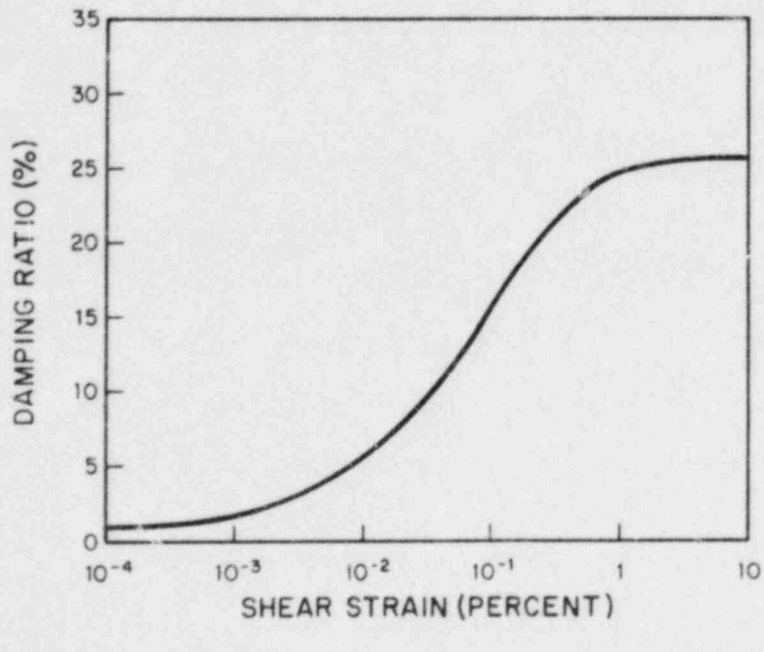


$G = 1000 K_S (\sigma'_m)^{0.5}$ (PSF)

WHERE G = SHEAR MODULUS

σ'_m = MEAN EFFECTIVE SOIL PRESSURE

a. VARIATION OF SHEAR MODULUS OF SAND WITH STRAIN

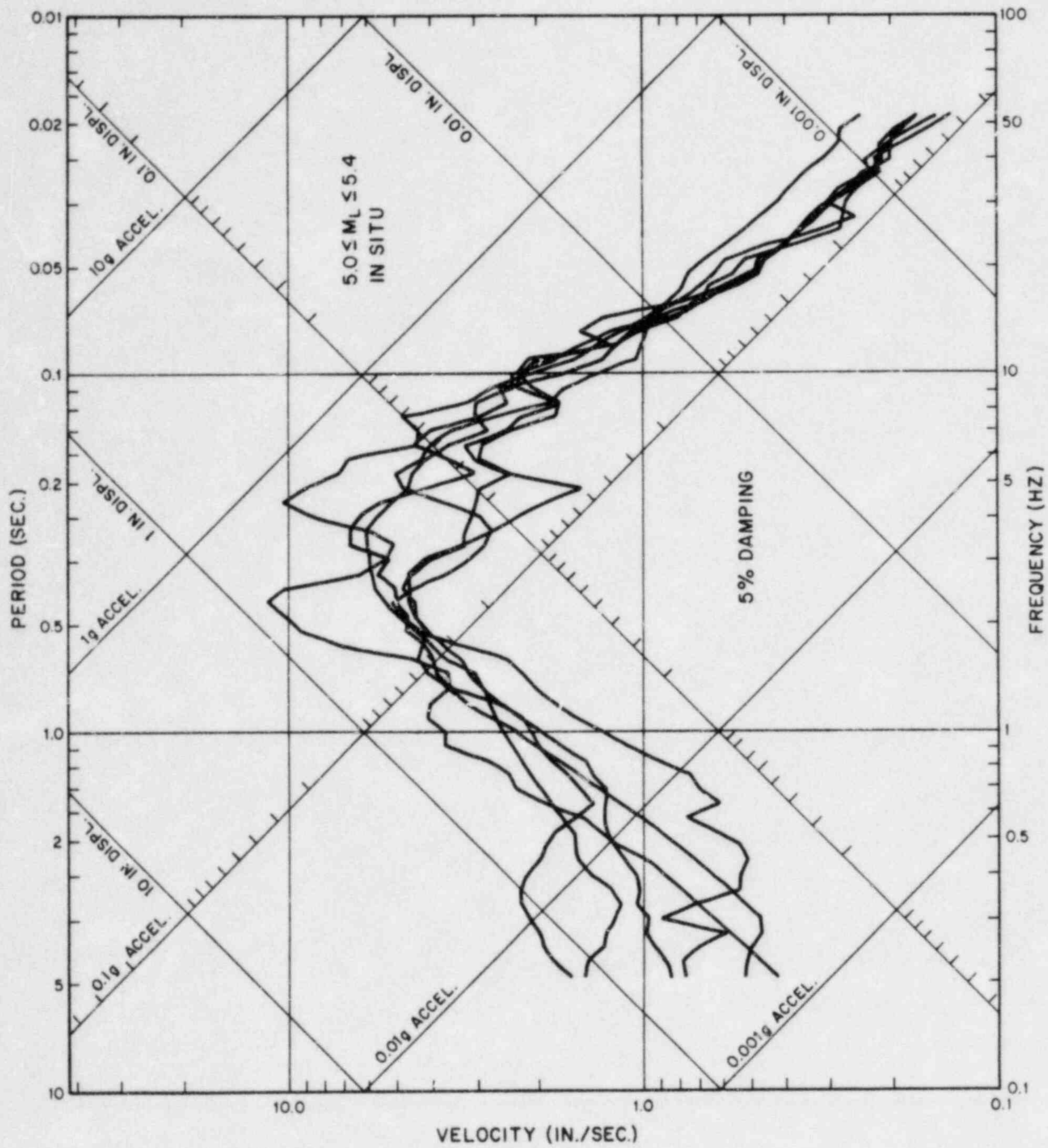


b. VARIATION OF DAMPING RATIO OF SAND WITH STRAIN

FIGURE 7-3
STRAIN DEPENDENT SOIL PARAMETERS

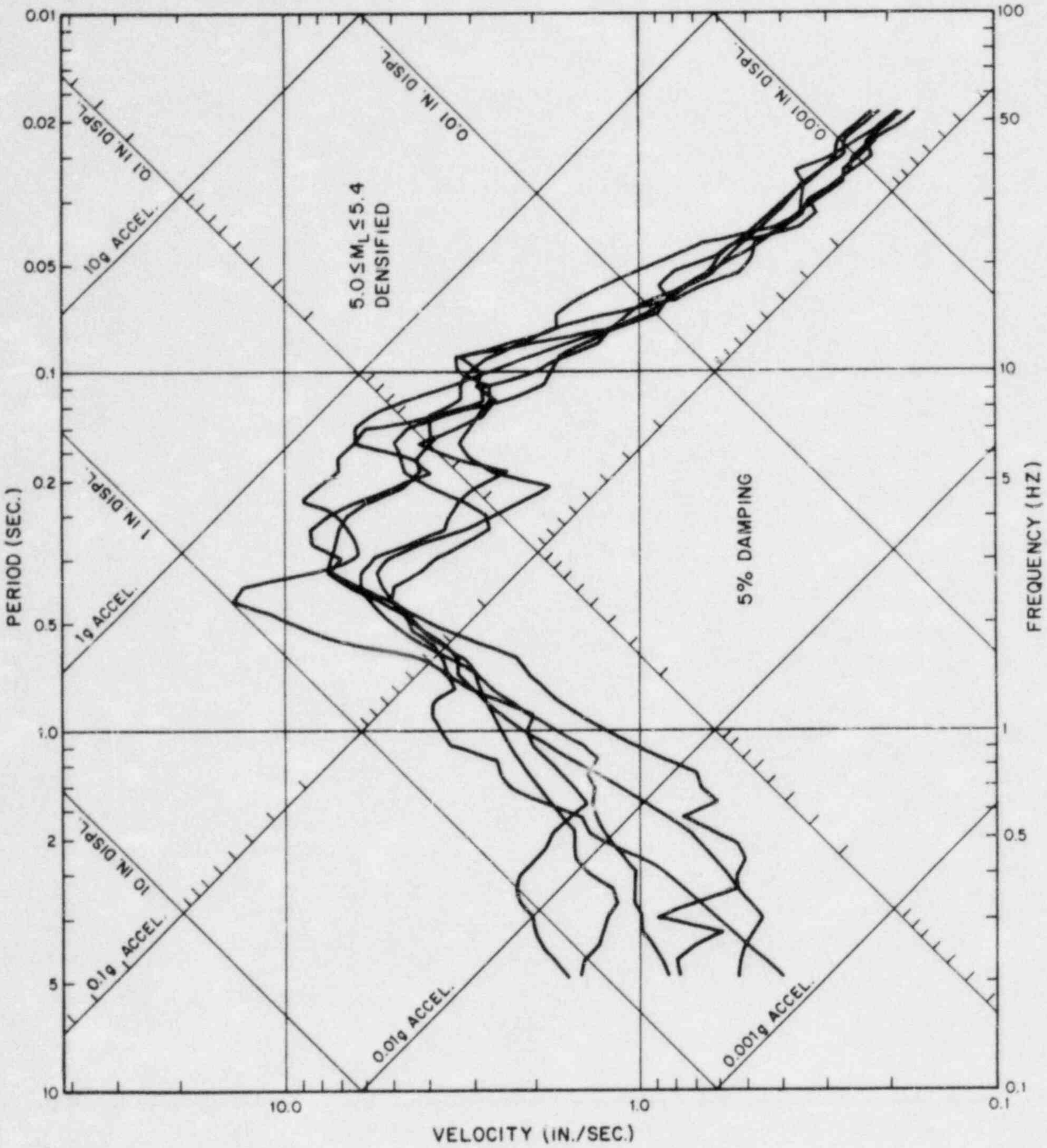
BEAVER VALLEY POWER STATION-UNIT 2

STONE & WEBSTER ENGINEERING CORPORATION



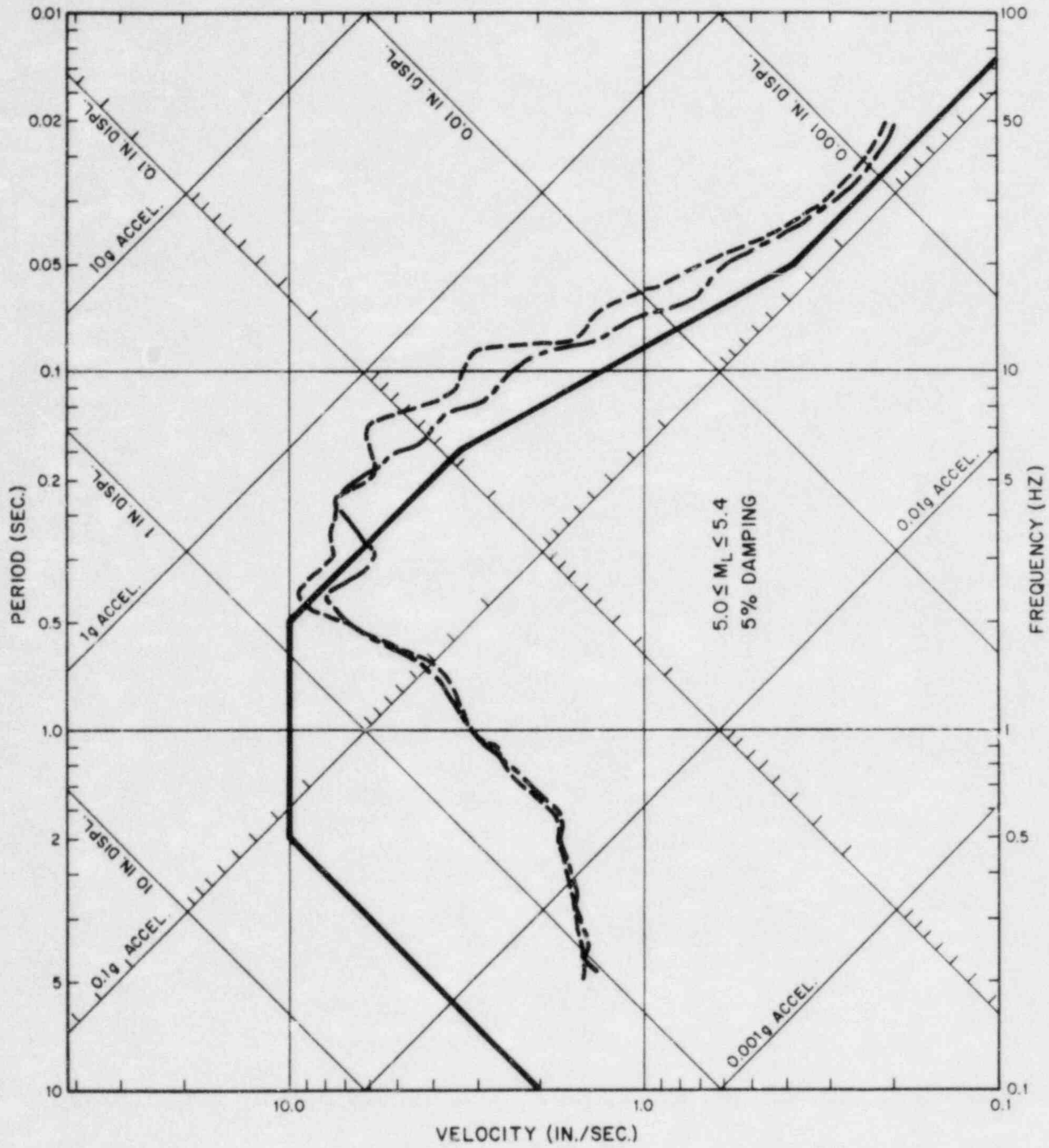
NOTE
REF. Nos. 1 THRU 6 (TABLE 7-1).

FIGURE 7-4
RESPONSE SPECTRA FROM SOIL RESPONSE ANALYSIS: IN SITU (UNSCALED)
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



NOTE
REF Nos. 1 THRU 6 (TABLE 7-1).

FIGURE 7-5
RESPONSE SPECTRA FROM SOIL RESPONSE ANALYSIS: DENSIFIED AREA (UNSCALED)
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION

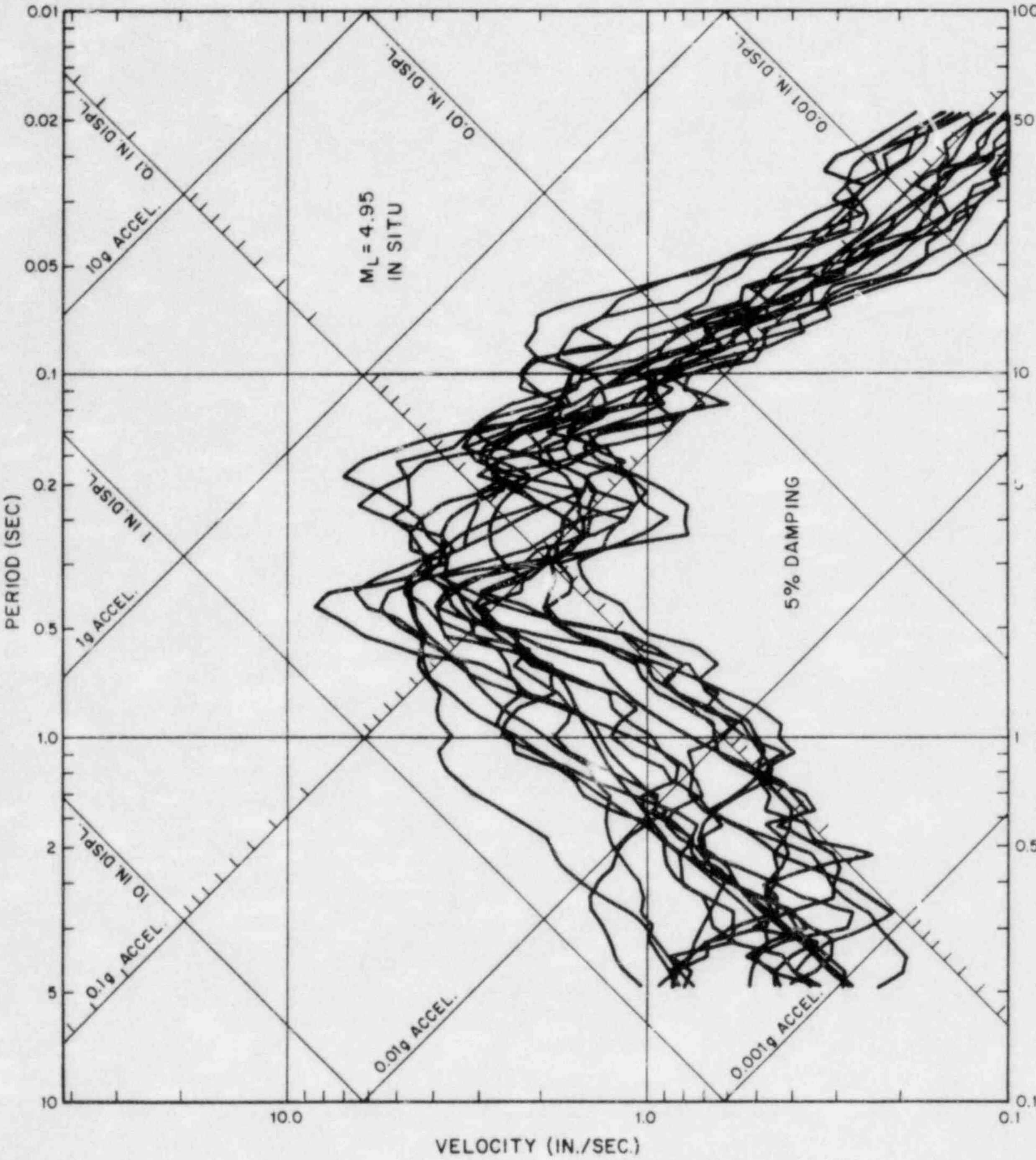


LEGEND

- BVPS-2
- MSD IN SITU
- MSD DENSIFIED AREA

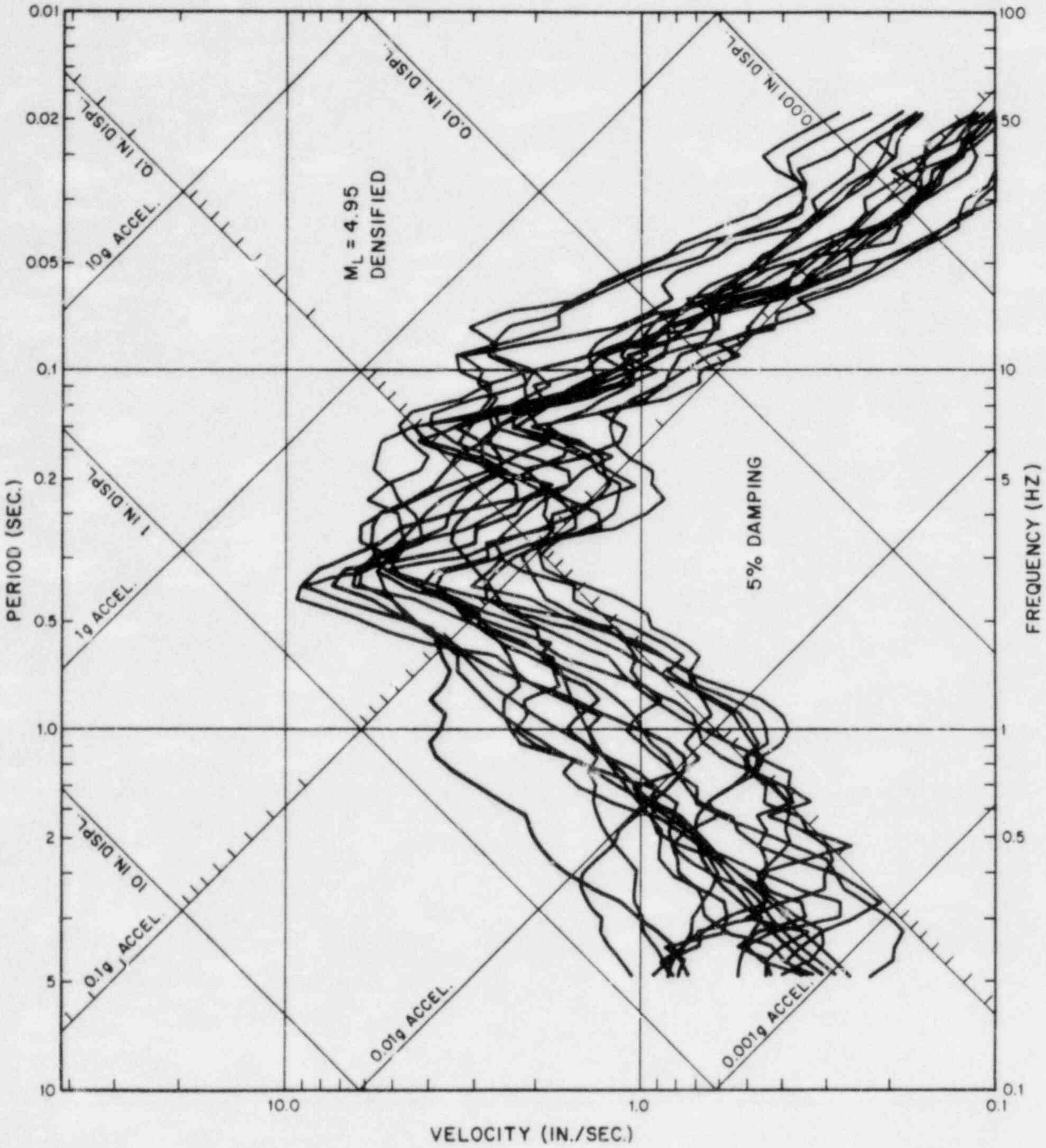
NOTE
REF. Nos. 1 THRU 6 (TABLE 7-1).

FIGURE 7-6
COMPARISON OF RESPONSE SPECTRA FROM SOIL RESPONSE ANALYSIS (UNSCALED)
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



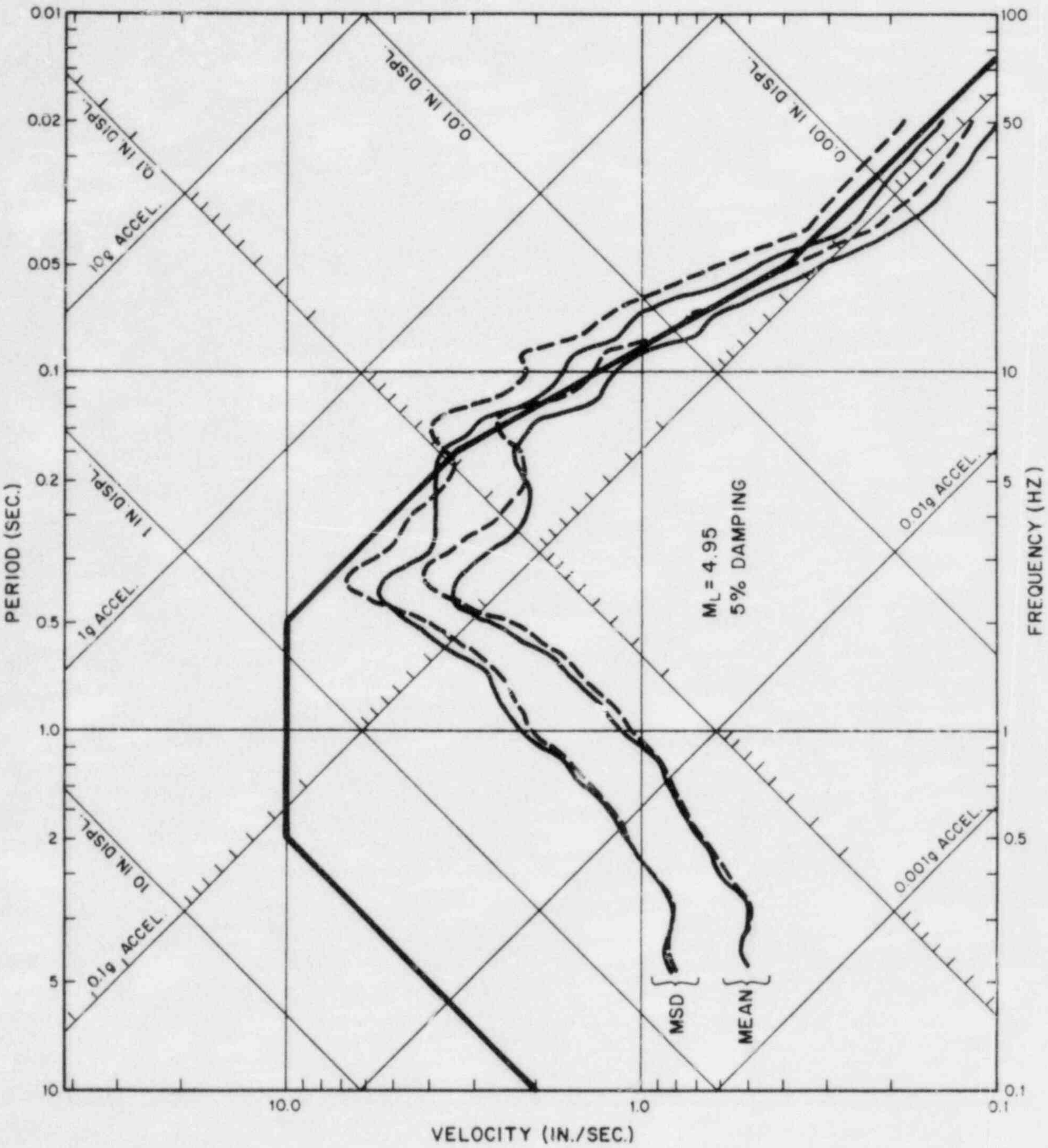
NOTE
REF. Nos. 1 THRU 18 (TABLE 7-2)

FIGURE 7-7
RESPONSE SPECTRA FROM SOIL RESPONSE ANALYSIS: IN SITU (SCALED)
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



NOTE
REF. Nos. 1 THRU 18 (TABLE 7-2)

FIGURE 7-8
RESPONSE SPECTRA FROM SOIL RESPONSE ANALYSIS: DENSIFIED AREA (SCALED)
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION

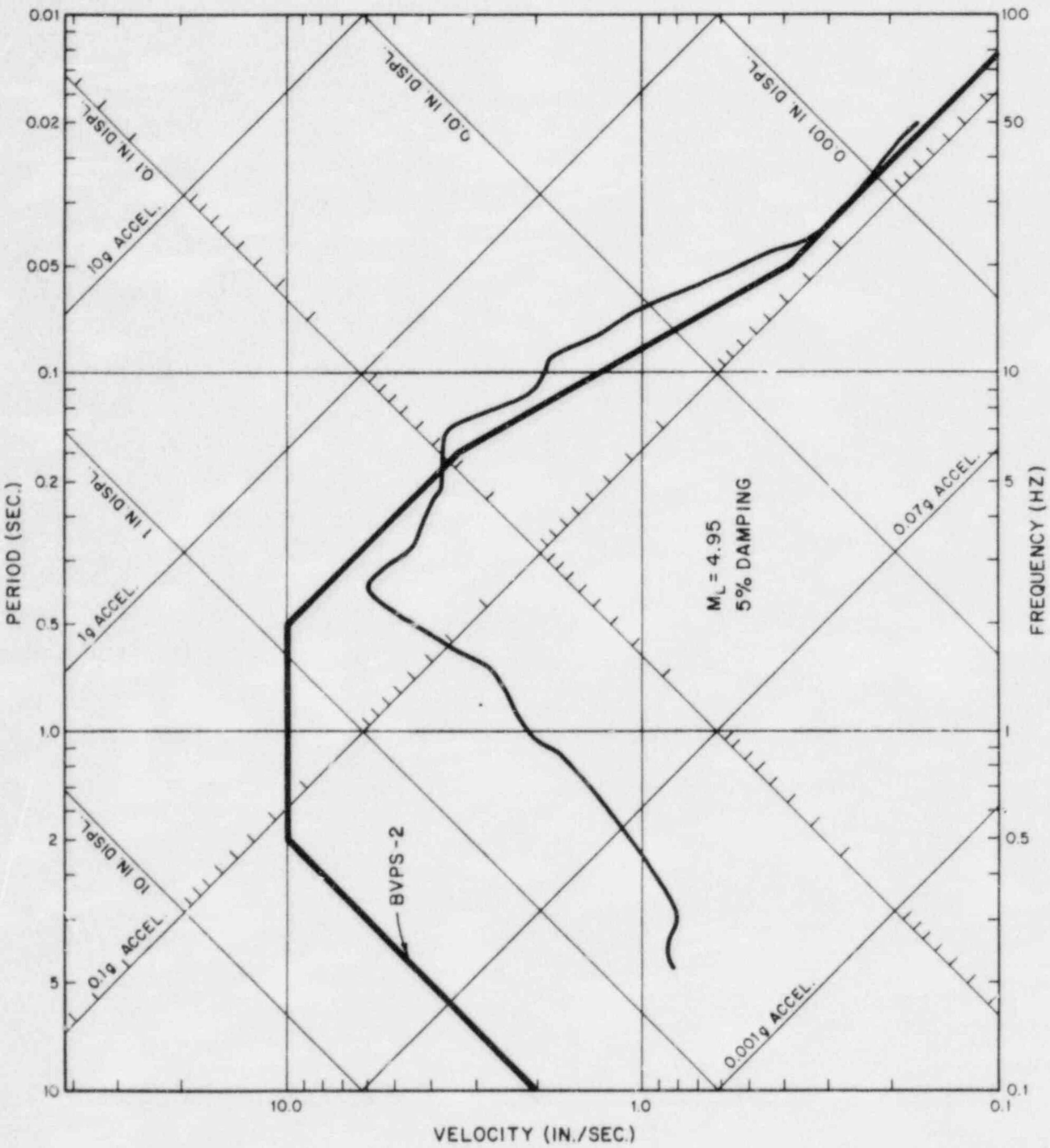


LEGEND

- BVPS-2
- IN SITU
- DENSIFIED AREA

NOTE
REF. Nos. 1 THRU 18 (TABLE 7-2).

FIGURE 7-9
COMPARISON OF RESPONSE SPECTRA FROM SOIL RESPONSE ANALYSIS (SCALED)
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



NOTE

RESPONSE SPECTRA SHOWN IS THE AVERAGE OF THE MSD RESPONSE SPECTRA FOR IN SITU AND DENSIFIED AREA PROFILE SHOWN IN FIGURE 7-9.

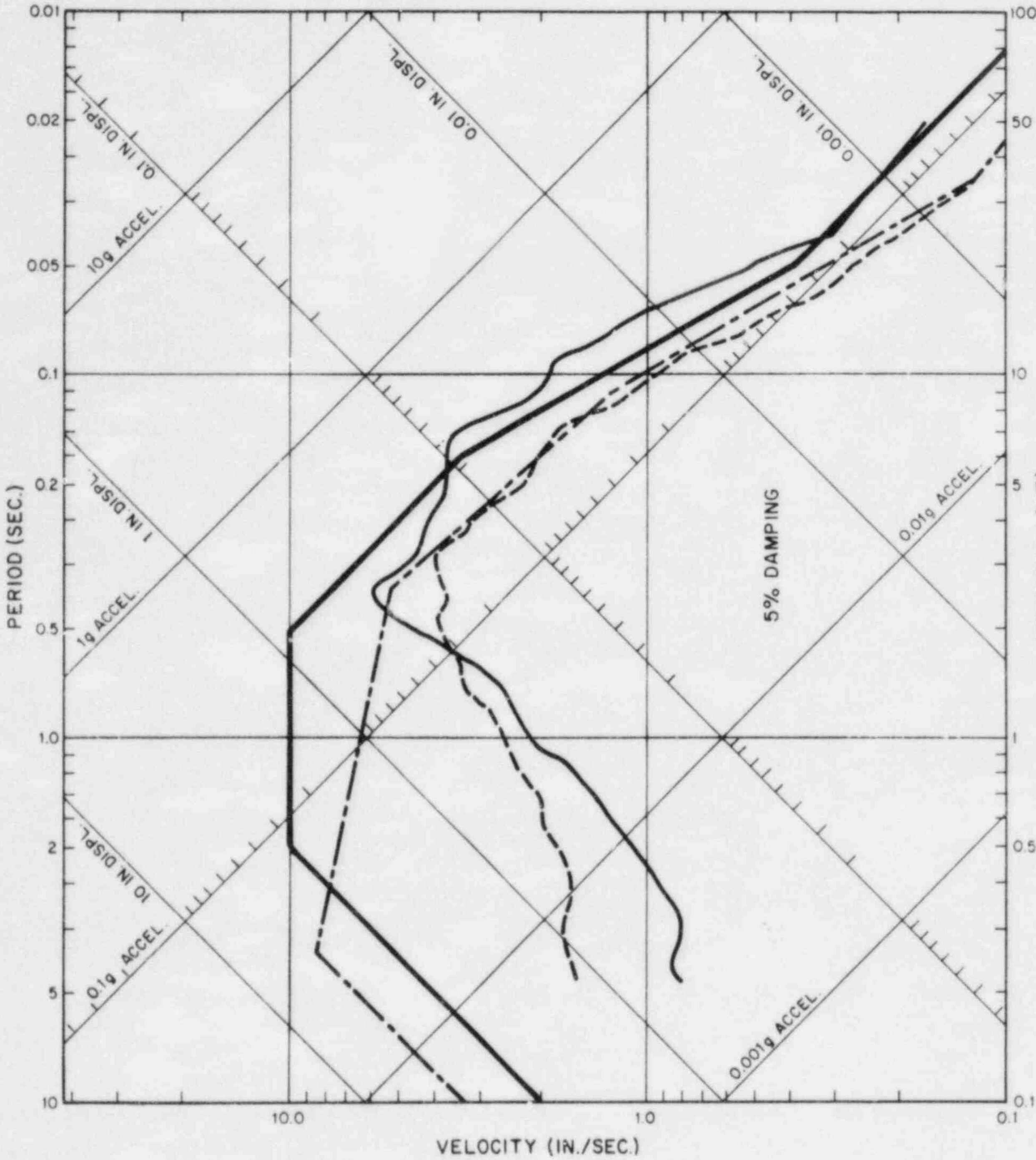
FIGURE 7-10
SITE DEPENDENT RESPONSE SPECTRA FROM SOIL RESPONSE ANALYSIS (SCALED)
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION

SECTION 8

SUMMARY OF HORIZONTAL RESPONSE SPECTRA

The site independent and site dependent response spectra determined for this study are shown in Figure 8-1. For comparison, the BVPS-2 SSE design response spectrum is also shown. The BVPS-2 response spectrum was found to be reasonable and acceptable when compared to response spectra determined using current state-of-the-art procedures.

The frequency range of interest to typcial BVPS-2 plant structures is between approximately 2 Hz and 10 Hz. Within this frequency range, the BVPS-2 response spectrum is seen to conservatively envelope both the Regulatory Guide 1.60 response spectrum and the site matched response spectrum. For frequencies greater than about 6 Hz, the soil response analysis response spectrum exhibits spectral amplitudes somewhat higher than those of BVPS-2. (Explanation of the exceedences is provided in Section 7.4.) However, the BVPS-2 response spectrum is seen to represent a reasonable average of the site independent and site dependent response spectra.



LEGEND

- BVPS-2
- REG. GUIDE 1.60 (0.07g)
- SITE MATCHED RESPONSE SPECTRUM
- RESPONSE SPECTRUM FROM SOIL RESPONSE ANALYSIS

FIGURE 8-1
SUMMARY OF RESPONSE SPECTRA
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION

SECTION 9

VERTICAL RESPONSE SPECTRA

The SSE horizontal ground surface acceleration for BVPS-2 is specified as 0.125g. The vertical acceleration is taken as two-thirds of the horizontal or 0.083g. The BVPS-2 SSE vertical response spectrum for 5 percent damping is shown in Figure 9-1. Its spectral amplitudes are two-thirds of those of the SSE horizontal response spectrum for 5 percent damping shown in Figure 2-1.

The maximum earthquake potential for BVPS-2 has been determined to be Intensity VI(MM), with a ground surface acceleration of 0.07 g (Section 3.0). For comparison with the BVPS-2 vertical response spectrum, a Regulatory Guide 1.60 vertical response spectrum anchored to 0.07g is also shown in Figure 9-1. The BVPS-2 vertical response spectrum is seen to be approximately equivalent to the Regulatory Guide 1.60 response spectrum.

The Regulatory Guide 1.60 response spectra rules are based predominantly on data from western United States earthquakes. The vertical response spectra are two-thirds of the horizontal for frequencies less than 0.25 Hz; above 3.5 Hz the vertical and horizontal response spectra are the same; and between 0.25 Hz and 3.5 Hz, the ratio varies between one and two-thirds. Statistical analyses of earthquake ground motion parameters by SW-AA (1979) have confirmed that the ratio of vertical to horizontal response spectra of two-thirds is conservative for all frequencies for western United States earthquakes. According to Chang (1983), the average ratio of peak vertical to peak horizontal acceleration, a_v/a_h, for Japanese and western United States earthquakes is 0.33 and 0.48, respectively. The V. C. Summer Nuclear Station FSAR (SCE 1981) presents data recorded from a number of western United States and Japanese earthquakes which show peak vertical accelerations generally less than peak horizontal accelerations for moderate size earthquakes with local magnitudes, M_L, less than 6. It was concluded that an appropriate ratio of peak vertical to peak horizontal acceleration was 0.5 for local magnitudes less than about 5.5. The equivalent western earthquake local magnitude is 4.95 for the BVPS-2 SSE of Intensity VI (MM), as discussed in Section 5.0. Therefore, specification of the BVPS-2 vertical acceleration as two-thirds of the horizontal acceleration is conservative considering the available data from the western United States and Japan.

A number of earthquakes records, listed in Table 9-1, have been obtained in recent years from several eastern United States earthquakes. Also presented in Table 9-1 are the earthquake source parameters and the recorded acceleration data obtained from the references cited.

The ratio of the peak vertical acceleration to the average peak horizontal acceleration, shown in Table 9-1, varies widely from site to site for a given earthquake as well as from earthquake to earthquake. For instance, the ratio ranges between 0.43 and 4.04 and averages 1.18 for the aftershocks of the 1982 New Brunswick earthquake. Similarly, for

the 1982 New Hampshire earthquake, the ratio varies between 0.42 and 1.26, with an average of 0.77. On the other hand, the vertical to horizontal acceleration ratio is only 0.34 for the 1978 Monticello Dam earthquake in South Carolina. For all 38 of the records listed in Table 9-1, the average ratio is 0.86. Four of the acceleration ratios for the 1982 New Brunswick aftershocks (denoted by * in Table 9-1) may be questionable as will be explained below. Eliminating these four data points reduces the average ratio of vertical to horizontal acceleration to 0.71.

Table 9-1 lists the frequencies corresponding to the peak accelerations for several of the 1982 New Hampshire earthquake records and all but one of the 1982 New Brunswick earthquake records. In general, the peak horizontal and vertical accelerations are associated with high frequencies in the range of 18 to 47 Hz. Four of the New Brunswick recordings, (denoted by * in Table 9-1) listed peak vertical and horizontal accelerations that occurred at very different frequencies and, as a result, the ratio of vertical to horizontal acceleration may not be appropriate. Ideally, to compare the vertical acceleration corresponding to the peak horizontal acceleration, the comparison should be made at about the same frequency.

The corrected acceleration time histories for the March 31, 1982, New Brunswick aftershock recorded at the Mitchell Lake recording station are presented in Figure 9-2. Comparing peak accelerations, the acceleration ratio, a_v/a_h, is 3.01 (Table 9-1). Examination of the time histories, however, shows that the peak vertical acceleration of 570.9 cm/sec^2 occurred as a single, high frequency spike. A value more representative of a sustained vertical acceleration is about 200 cm/sec^2. The peak horizontal accelerations, on the other hand, are more representative of a sustained acceleration. Comparing the sustained vertical acceleration with the average of the peak horizontal accelerations reduces the acceleration ratio to about 1.1. The acceleration time histories presented by Weichert et al (1982) for the Holmes Lake and Leggie Lodge recordings of the March 31 aftershock were similar in character to the Mitchell Lake recordings described above. Time histories were not provided for the Mitchell Lake recordings of the May 5 event. Since there was some question as to the validity of the acceleration ratio computed using peak accelerations from these four records, they were excluded from the data set when computing the overall average acceleration ratio.

In summary, available data from eastern United States earthquakes suggest that the average ratio of the peak vertical to the peak horizontal acceleration is about 0.7. Data from the western United States and Japan indicate that for earthquakes with local magnitudes less than 5.5 the ratio is about 0.5. Therefore, the specification of the vertical acceleration for BVPS-2 as two-thirds of the horizontal is consistent with available data.

The BVPS-2 vertical response spectrum for 5 percent damping is shown in Figure 9-1. It is seen to be approximately equivalent to a Regulatory Guide 1.60 vertical response spectrum corresponding to the BVPS-2 site Intensity VI(MM). Available data indicate that the 1 to 1 ratio of

vertical to horizontal acceleration used by Regulatory Guide 1.60 for frequencies greater than 3.5 Hz is somewhat too conservative. Therefore, it is concluded that the BVPS-2 vertical response spectrum is appropriate.

TABLE 9-1

EASTERN UNITED STATES EARTHQUAKE DATA

Date Year	Date Month	Date Day	Epicenter Location	Depth km	Magnitude	Recording Station	Epicenter Distance km	Peak Acceleration, g (1) Horizontal H_1	Peak Acceleration, g (1) Horizontal H_2	Peak Acceleration, g (1) Vertical	a_v/a_h (2)
1973	07	30	Blue Mtn. Lake, N.Y.	1.2	$m_b = 2.7$	Blue Mtn. Lake No. 1	0.7	0.018	0.017	0.010	0 57
1973	08	03	Blue Mtn. Lake, N.Y.	1.0	$m_b = 2.6$	Blue Mtn. Lake No. 1	0.3	0.031	0.034	0.019	0.58
1975	06	13	36.53°N, 89.66°W	9.0	$m_b = 4.25$	New Madrid, MO	9.0	0.043	0.064	0.031	0.58
1976	03	25	35.6°N, 90.5°W	12.0	$m_b = 5.0$	Arkabutla Dam, MS.	99				
						Left Toe		0.041	0.022	0.010	0.32
						Left Crest		0.021	0.010	0.006	0.39
						Right Abutment		0.011	0.011	0.006	0.55
						Tiptonville, TN	130	0.011	0.017	0.012	0.86
						New Madrid, MO	131	0.013	0.011	0.010	0.83
						Wappapello Dam, MO	150				
						Right Toe		0.010	0.012	0.005	0.45
						Right Crest		0.006	0.006	0.005	0.83
1976	03	25	35.6°N,90.5°W	14.0	$m_b = 4.5$	Arkabutla Dam, MS	99				
						Left Toe		0.010	0.005	0.004	0.53
1978	08	27	Monticello, SC	-	$M_L = 2.7$	-	-	-	-	-	0.34
1982	01	18	Franklin, NH 42.5°N, 71.6°W	4.5 to 8.0	$m_b = 4.4$	Franklin Falls Dam	8				
						Downstream		0.144(21)	0.386(16)	0.277(21)	1.05
						Rt. Abutment		0.294(14)	0.551(14)	0.176(20)	0.42
						Crest		0.127(11.4)	0.313(14)	0.117(11.4)	0.53
						Union Village Dam	60				
						Crest		0.023	0.026	0.024	0.98
						Lt. Abutment		0.010	0.007	0.006	0.71
						Downstream		0.038	0.023	0.030	0.98
						North Hartland Dam	61				
						Lt Abutment		0.011	0.007	0.004	0.44
						Crest		0.038	0.039	0.017	0.44
						North Springfield Dam	76				
						Crest		0.025	0.022	0.023	0.98
						Downstream		0.032	0.023	0.014	0.51

TABLE 9-1 (CONT'D)

Date Year	Month	Day	Epicenter Location	Depth km	Magnitude	Recording Station	Epicenter Distance km	Peak Acceleration, g (1) Horizontal H_1	H_2	Vertical	a_v/a_h (2)
						Ball Mountain Dam Crest	103	0.009	0.010	0.012	1.26
						White River Junction, VT VA Hospital	60	0.015	0.032	0.022	0.94
1982	01	17	New Brunswick (3)	3.6	3.5	7A	8	0.080	0.058	0.082	1.19
						8A	10	0.016	0.015	0.016	1.03
1982	03	31	Holmes Lake, New Brunswick	4	M_N = 4.8	Holmes Lake	6	0.181(18)	0.347(41)	0.154(37)	0.58*
						Mitchell Lk Rd	4	0.152(18/25)	0.236(22)	0.583(37/43)	3.01*
						Loggie Lodge	6	0.298(22)	0.576(28/35)	0.308(47)	0.70*
						Indian Brook	3	0.426(24)	0.413(24)	0.146(25/40)	0.35
1982	04	02	Holmes Lake, New Brunswick		M_N = 4.3	Mitchell Lk Rd.	4	0.067(33)	0.079(25)	0.555(33)	0.75
1982	04	28	Holmes Lake, New Brunswick		M_N = 3.4	Holmes Lake	6	0.076(40)	0.057(31)	0.042(33)	0.63
1982	05	06	Holmes Lake, New Brunswick		M_N = 4.0	Holmes Lake	6	0.043(25)	0.072(17)	0.025(20)	0.43
						Mitchell Lake Rd.	4	0.055(23)	0.034(23)	0.180(45)	4.04*
						Loggie Lodge	7	0.117(10/25)	0.149(13)	0.067(19)	0.50
1982	07	28	New Brunswick		M_N = 3.7	Indian Brook	1	0.306(25)	0.235(25/30)	0.184(25)	0.68
1982	06	16	Near Trousers Lake, New Brunswick		M_N = 4.6	Mitchell Lake Rd.	25	0.049(25)	0.011(20)	0.027(25)	0.90
						Indian Brook	27	0.015(13)	0.017(20)	0.028(20)	1.75

NOTES:

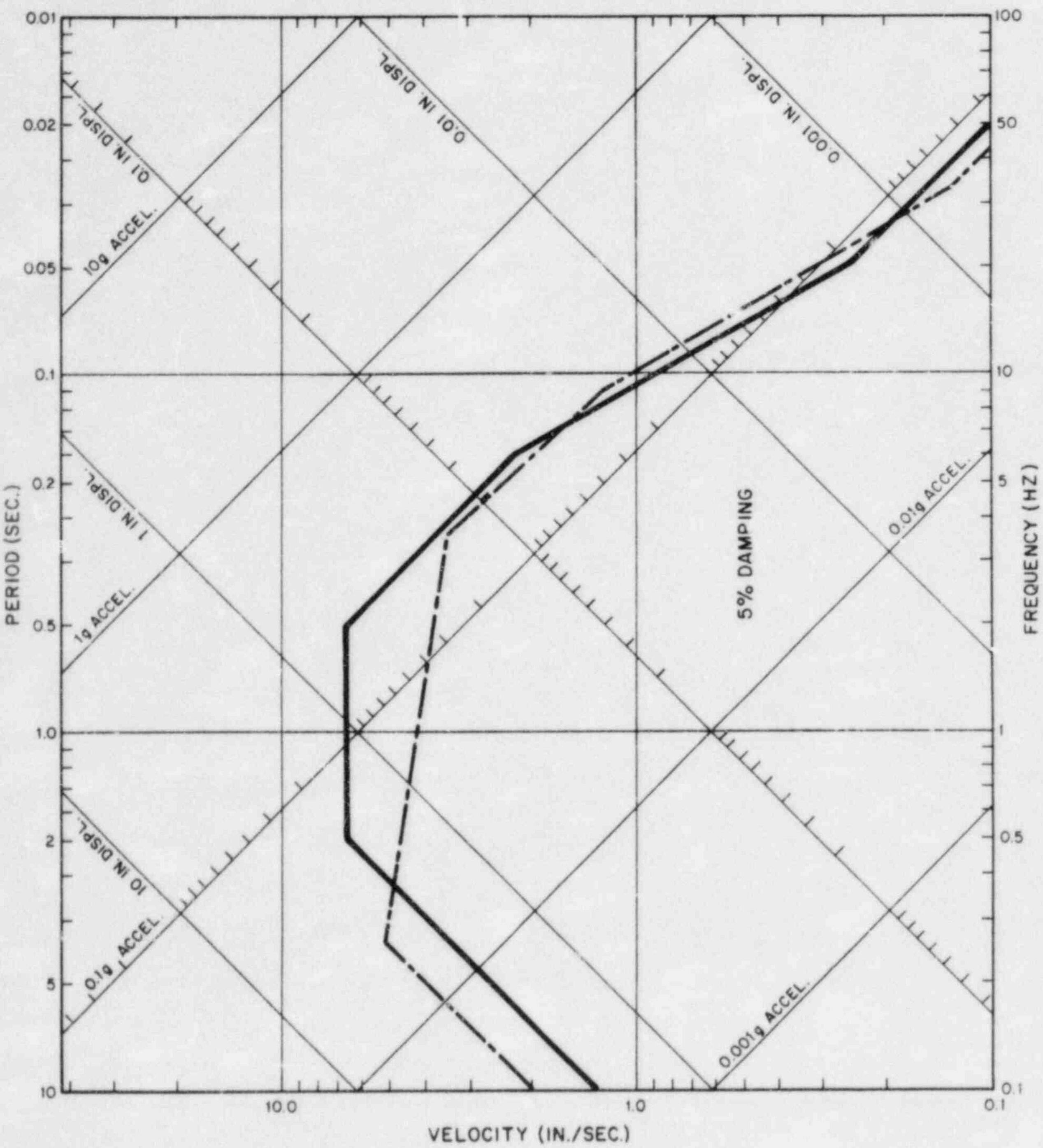
(1) Numbers in parentheses are frequencies for peak accelerations.

(2) Ratio a_v/a_h determined as the peak vertical acceleration divided by the average peak horizontal acceleration. * indicates questionable values as explained in text.

(3) A number of recordings were made for small aftershocks. Listed are only the accelerations scaled from corrected acceleration time histories of the event for which magnitude was estimated.

(4) References

Change 1983
Cranswick et al 1982
Fletcher and Anderson 1974
Hermann 1977
South Carolina Electric 1982
Weichert et al 1982

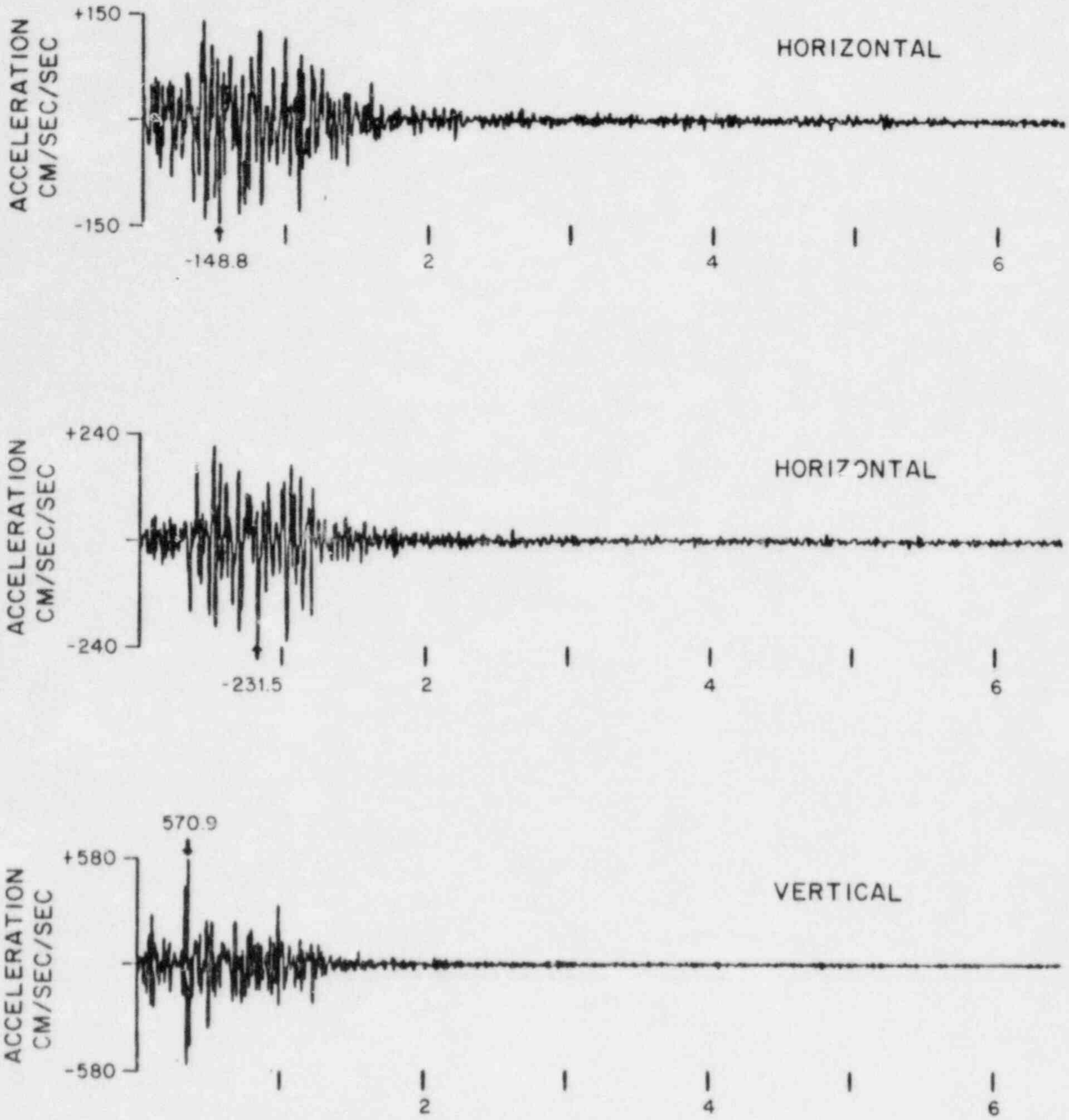


LEGEND

BVPS-2

REG. GUIDE 1.60 (0.07g)

FIGURE 9-1
COMPARISON OF VERTICAL RESPONSE SPECTRA
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION



CORRECTED ACCELERATION TIME HISTORIES

NOTE
RECORDING STATION: MITCHELL LAKE ROAD.
REF: WEICHERT ET AL, 1982.

FIGURE 9-2
NEW BRUNSWICK EARTHQUAKE
AFTERSHOCK: MARCH 31, 1982
BEAVER VALLEY POWER STATION-UNIT 2
STONE & WEBSTER ENGINEERING CORPORATION

SECTION 10
REFERENCES

Boore, D. M. The Effects of Simple Topography on Seismic Waves: Implications for the Accelerations Recorded at Pacoima Dam, San Fernando Valley California. Bulletin of the Seismological Society of America, Vol. 63, No. 5. October 1973.

Chung, D. H. and Bernreuter, D. L. Regional Relationship Among Earthquake Magnitude Scales. Lawrence Livermore Laboratory Report 52745, NUREG/CR-1457. Prepared for United States Nuclear Regulatory Commission, 1980.

Chang, F. K. Analysis of Strong Motion Data from the New Hampshire Earthquake of January 18, 1982. NUREG/CR-3327. Report to U.S. Nuclear Regulatory Commission, 1983.

Chang, F. K. and Krinitzsky, E. L. Duration, Spectral Content, and Predominant Period of Strong Motion Earthquake Records from Western United States. Miscellaneous Paper S-73-1, Report 8. U.S. Army Engineer, Waterways Experiment Station, Vicksburg, MI, 1977.

Cranswick, E.; Mueller, C.; Wetmiller, R.; and Sembera, E. Local Multi-Station Digital Recordings of Aftershocks of the January 9, 1982 New Brunswick Earthquake. U.S. Geological Survey Open File Report No. 82-777, 1982.

Epply, R. A. Earthquake History of the United States, Part 1. U.S. Department of Commerce, 1965.

Fletcher, J. P. and Anderson, J. G. The First Strong Motion Records from a Central or Eastern United States Earthquake. Bulletin of the Seismological Society of America, Vol. 64, No. 5. October 1974.

Gupta, I. N. and Nuttli, O. W. Spatial Attenuation of Intensities for Central U.S. Earthquakes. Bulletin of the Seismological Society of America, Vol. 66, No. 3, June 1976.

Hays, W. Procedures for Estimating Earthquake Ground Motions. Geological Survey Professional Paper 1114. U. S. Government Printing Office, Washington, DC, 1980.

Herrmann, R. B. Analysis of Strong Motion Data from the New Madrid Seismic Zone: 1975-1976. Department of Earth and Atmospheric Sciences. St. Louis University, August 1977.

Herrmann R. B. and Nuttli, O. W. Strong Motion Investigations in the Central United States. Proceedings of Seventh World Conference on Earthquake Engineering, Geoscience Aspects, Part II, Instanbul, Turkey, 1980.

Housner, G. W. Nuclear Reactors and U.S. Earthquakes. Rpt. No. TID-7024. USAEC Division of Technical Information, 1963.

Kanamori, H. and Jennings, P. C. Determination of Local Magnitude, M_L, from Strong Motion Accelerograms. Bulletin of the Seismological Society of America, Vol. 68, No. 2, April 1978.

Krinitzsky, E. L. and Chang, F. K. State-of-the-Art for Assessing Earthquake Hazards in the United States; Specifying Peak Motions for Design Earthquakes. Miscellaneous Paper S-73-1, Report 7. U.S. Army Engineers, Waterways Experiment Station, Vicksburg, MI, 1979.

Murphy, J. R. and O'Brien, L. J. The Correlation of Peak Ground Acceleration Amplitude with Seismic Intensity and Other Physical Parameters. Bulletin of the Seismological Society of America, Vol. 67, No. 3, 1977.

Newmark, N. M.; Blume, J. A.; and Kapur, K. K. Seismic Design Spectra for Nuclear Power Plants. Journal of the Power Division, Vol. 99, No. PO2. American Society of Civil Engineers, November 1973.

Newmark, N. M. and Hall, W. J. Seismic Design for Nuclear Reactor Facilities. Fourth World Conference on Earthquake Engineering, Santiago, Chile, 1969.

Nuttli, O. W. The Relationship of Sustained Maximum Ground Acceleration and Velocity to Earthquake Intensity and Magnitude. Miscellaneous Paper S-73-1, Report 16. U.S. Army Engineers, Waterways Experiment Station, Vicksburg, MI, 1979.

Nuttli, O. W. and Herrmann, R. B. State-of-the-Art for Assessing Earthquake Hazards in the United States: Credible Earthquakes for the Central United States. Miscellaneous Paper S-73-1, Report No. 12. U.S. Army Engineers, Waterways Experiment Station, Vicksburg, MI, 1978.

Nuttli, O. W. and Herrmann, R. B. Earthquake Magnitude Scales. Journal of the Geotechnical Engineering Division, Vol. 108, No. GT5. American Society of Civil Engineers, May 1982.

Okamoto, S. and Mizukoshi, J. Earthquake Ground Motions Observed on Rock Foundations. Proceedings of the IAEA Panel on Aseismic Deisgn and Testing of Nuclear Facilities, Tokyo, Japan, 1967.

Schnabel, P. B.; Lysmer, J.; and Seed, H. B. SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites. Report EERC-72-12. University of California at Berkeley, 1972.

Seed, H. B. and Idress, I. M. Soil Moduli and Damping Factors for Dynamic Response Analysis. Report EERC 70-10. College of Engineering, University of California at Berkeley, 1970.

Shannon and Wilson, Inc. and Agbabian Associates (SW-AA). Procedures for Evaluation of Vibratory Ground Motions of Soil Deposits at Nuclear Power Plant Sites. NUREG-75/072. Prepared for U.S. Nuclear Regulatory Commission 1975.

Shannon and Wilson, Inc. and Agbabian Associates (SW-AA). Geotechnical and Strong Motion Earthquake Data from U.S. Accelerograph Stations, Vol. 1.

Fernadele, Chalame, and El Centro, California. NUREG-0029, Vol. 1, NRC-6. Report to the U.S. Nuclear Regulatory Commission, 1976.

Shannon and Wilson, Inc. and Agbabian Associates (SW-AA). Geotechnical and Strong Motion Earthquake Data from U.S. Accelerograph Stations. Pasadena (CIT Millikan Library), Santa Barbara (County Courthouse), Taft (Lincoln School Tunnel) and Hollister (Melendy Ranch Barn), Calfornia. NUREG-0029, Vol. 2., NRC-6A. Report to U.S. Nuclear Regulatory Commission, 1978a.

Shannon and Wilson, Inc. and Agbabian Associates (SW-AA). Verification of Subsurface Conditions of Selected Rock Accelerograph Stations in California. NUREG/CR-0055, Vol. 1, R6A. Prepared for U.S. Nuclear Regulatory Commission. 1978b.

Shannon and Wilson, Inc. and Agbabian Associates (SW-AA). Data from Selected Accelerograph Stations at Wilshire Boulevard, Century City, and Ventura Boulevard, Los Angeles, California. NUREG/CR-0074, NRC-6A. Report to U.S. Nuclear Regulatory Commission, 1978c.

Shannon and Wilson, Inc. and Agbabian Associates (SW-AA). Statistical Analysis of Earthquake Ground Motion Parameters. NUREG/CR-1175. Report to U.S. Nuclear Regulatory Commission, 1979.

Shannon and Wilson, Inc. and Agbabian Associates (SW-AA). Geotechnical Data from Accelerograph Stations Investigated During the Period 1975-1979, Summary Report. NUREG/CR-1643. Prepared for U.S. Nuclear Regulatory Commission, 1980a.

Shannon and Wilson, Inc. and Agbabian Associates (SW-AA). Geotechnical and Strong Motion Earthquake Data fr[illegible] U.S. Accelerograph Stations, Vol. 3, Gilroy, CA (Gavilan College - [illegible]6); Logan, UT (Utah State University); Bozeman, MT (Montana State University); Tacoma, WA (County-City Building); Helena, MT (Federal Building and Carroll College). NUREG/CR-0985, Vol. 3. Report to U.S. Nuclear Regualtory Commission, 1980b.

Shannon and Wilson, Inc. and Agbabian Associates (SW-AA). Geotechnical and Strong Motion Earthquake Data from U.S. Accelerograph Stations, Vol. 4, Anchorage, AK (AMU Gould Hall); Seattle, WA (Federal Office Building); Olympia, WA (Highway Test Laboratory); Portland, OR (State Office Building and PSU Cramer Hall). NUREG/CR-0985. Report to U.S. Nuclear Regulatory Commission, 1980c.

Shannon and Wilson, Inc. and Agbabian Associates (SW-AA). Geotechnical and Strong Motion Earthquake Data from U.S. Accelerograph Stations, Vol. 5, Fairbanks, AK (UA Duckering Hall); Petrolia, CA (General Store); Holliston, CA (City Hall); Los Angeles, CA (Hollwood Storage Building); and New Madrid, MO (Noranda Aluminum Plant). NUREG/CR-0985. Report to U.S. Nuclear Regulatory Commission, 1980d.

Shannon and Wilson, Inc. and Agbabian Associates (SW-AA). Verification of Subsurface Conditions at Selected "Rock" Accelerograph Station in California, Vol. 2. NUREG/CR-0055. Report to U.S. Nuclear Regulation Commision, 1980e.

Shannon and Wilson, Inc. and Agbabian Asosciates (SW-AA). Verification of Subsurface Conditions at Selected "Rock" Accelerograph Station in California, Vol. 2, Appendix, Earthquake Records. NUREG/CR-055. Report to U.S. Nuclear Regulatory Commission, 1980f.

Shannon and Wilson, Inc. and Agbabian Associates (SW-AA). Verification of Subsurface Conditions at Selected "Rock" Acceleograph Station in California, Vol. 3. NUREG/CR-0055. Report to U.S. Nuclear Regulatory Commission, 1980g.

South Carolina Electric (SCE). Virgil C. Summer Nuclear Station - Unit 1, Final Safety Analysis Report. Amendment 23. Response to NRC Question 361.17(4). January 1981.

South Carolina Electric (SCE). Virgil C. Summer Nuclear Station - Unit 1, Applicant Evaluation of Trifunac Report on Virgil C. Summer Nuclear Station Seismicity Studies. Prepared by M. R. Somerville. January 1982.

Stone and Webster Engineering Corporation (SWEC). Beaver Valley Power Station - Unit 1, Preliminary Safety Analysis Report. Docket No. 50-334. Prepared for Duquesne Light Company, Pittsburgh, PA, 1968.

Stone and Webster Engineering Corporation (SWEC). Beaver Valley Power Station - Unit 1, Preliminary Safety Analysis Report, Amendment 15. Prepared for Duquesne Light Company, Pittsburgh, PA, 1970.

Stone and Webster Engineering Corporation (SWEC). Beaver Valley Power Station - Unit 2, Preliminary Safety Analysis Report. Docket No. 50-334. Prepared for Duquesne Light Company, Pittsburgh, PA. November 10, 1972.

Stone and Webster Engineering Corporation (SWEC) 1973. Beaver Valley Power Station - Unit 2, Preliminary Safety Analysis Report, Amendment 7. Prepared for Duquesne Light Company, Pittsburgh, PA. July 9, 1973.

Stone and Webster Engineering Corporation (SWEC). Report on the Soil Densification Program, Beaver Valley Power Station - Unit 2. 1976.

Stone and Webster Engineering Corporation (SWEC). Soil Structure Interaction in the Development of Amplified Response Spectra for Beaver Valley Power Station - Unit 1. Prepared for Duquesne Light Company, Pittsburgh, PA, 1979.

Stone and Webster Engineering Corporation (SWEC). Beaver Valley Power Station - Unit 2, Final Safety Analysis Report (1983), Section 2.5.2 as revised in Amendment 6. April 1984.

Trifunac, M. D. and Brady, A. G. On the Correlation of Seismic Intensity Scales with Peaks of Recorded Strong Ground Motion. Bulletin of the Seismological Society of America, Vol. 65, No. 1, 1975.

Trifunac, M. D. and Lee, V. Routine Computer Processing of Strong Motion Accelerograms, Volume II, Strong Motion Earthquake Accelerograms, Digitized and Plotted Data. Report EERL-73-03. California Institute of Technology Earthquake Engineering Research Laboratory, 1973.

U. S. Atomic Energy Commission. Design Response Spectra for Seismic Design of Nuclear Power Plants. Regulatory Guide 1.60, Rev. 1. Washington, DC, Directorate of Regulatory Standards, 1973.

U. S. Nuclear Regulatory Commission. Safety Evaluation Report Related to the Operation of the Sequoyah Nuclear Plant Units 1 and 2. NUREG-0011. 1979.

Weichert, D. H. ; Pomeroy, P. W.; Munro, P. S.; and Mork, P. N. Strong Motion Records from the Miramichi, New Brunswick, 1982 Aftershocks. Earth Physics Branch Open File Report 82-31. Ottawa, Canada, 1982.

Weston Geophysical Research. Seismicity Analysis, Beaver Valley Power Station, Appendix 2C. 1968. (See SWEC 1968).

Whitman, R. V. Effect of Local Soil Conditions upon Seismic Threat to Beaver Valley Power Station, Appendix 2D. 1968. (See SWEC 1968).

APPENDIX 1

NUCLEAR REGULATORY COMMISSION

SEISMOLOGY QUESTIONS

BEAVER VALLEY POWER STATION

UNIT 2

Question 220.4 (SRP 3.7.1.II.1a FSAR 3.7B.1.1)	Referring to Item 3.7.1.II.1a discussed in "Priority Review of Beaver Valley 2 Standard Review Plan Differences", the site specific response spectra and the values of vertical design response spectra should be addressed in Section 2.5.
Question 230.2 (SRP Sections 2.5.2.3, 2.5.2.4 2.5.2.5 and 2.5.2.6)	According to the FSAR, the Beaver Valley Power Station Unit 2 (BVPS-2) seismic design parameters are based upon a response spectrum anchored to a 0.125g ZPA which is different from the Regulatory Guide 1.60 spectrum. The documents quoted for the development of the above seismic design spectrum are BVPS-2 PSAR Appendices 2C and 2D. BVPS-2 Appendix 2C recommends a 0.10g design earthquake. BVPS-2 Appendix 2D recommends a Housner response spectrum normalized to 0.125g which was obtained from an estimated amplification factor of 3.5 combined with a maximum ground acceleration of 0.035g. In addition to these multiple assumptions, there are several factors mentioned in the FSAR which have not been adequately discussed with respect to the influence that these factors have on the seismic design criteria for the plant. For example FSAR Table 3.7B-2 indicates varations in depth of soil over bedrock from 35 feet to 110 feet. FSAR Figures 2.5.4-2 through 2.5.4-9 indicates significant differences in density of soils underlying the Category I structures. Describe how the above information was used to determine the seismic design criteria for each of the Category I structures. For example, describe the free field foundation acceleration assumed for the seismic design of Category I structures. Described how the established free field foundation acceleration was augmented to accommodate for soil amplification or reduction.
Question 230.3 (SRP Sections 2.5.2.3, 2.5.2.4 2.5.2.5 and 2.5.2.6)	The site is considered to be located in the Appalachian Plateau Tectonic Province. The largest historic earthquake in this tectonic province was determined to be the November 6, 1926 S.E. Ohio earthquake. This determination was obtained from intensity listings shown in FSAR table 2.5.2-2. Using the Standard Review Plan procedure for deriving seismic design criteria from intensity data, the MMI=VI-VII intensity listed for the 1926 earthquake would indicate a Regulatory Guide 1.60 response spectrum anchored to 0.10g zero period acceleration, which may be modified to reflect local site conditions. In recent safety reviews the staff has relied upon site specific spectra to evaluate the seismic design criteria. The reason being that site specific spectra are more in accord with the controlling earthquake size, frequency spectrum and local site conditions. For example, using the Nuttli/Herrmann (1978) relationship,

the site specific spectra for a MMI=VI-VII intensity earthquake could be developed from the 84th percentile spectra of a suite of appropriate earthquake records of magnitude $m_b = 5.0 + 0.5$. In addition a direct estimate of magnitude may be obtained from the information listed in the updated FSAR Table 2.5.2-2. In the event that appropriate records are not currently available, a site specific spectrum may be determined by modifying a rock site specific spectrum to account for local soil amplification characteristics of the site (cf Midland OL-SER, Clinton OL-SER).

1) Using the guidelines described in the Standard Review Plan, (1981) compare the BVPS-2 design spectra to the appropriate intensity based Regulatory Guide 1.60 spectra. Described the effects of local site conditions and discuss exceedences, if any.

2) Based upon your estimate of the appropriate magnitude of the Safe Shutdown Earthquake prepare Site Specific Spectra in accordance with guidelines described above. Compare these spectra with the design spectra for the plant and discuss exceedences, if any. Include in your discussion the effects of the following variations in parameters which influence the ground motion estimates.

i) Variation in shear velocity in the soil-layers which depend upon the composition, depth and/or densification of soil layers under the Category I structures.

ii) If appropriate, compare results of layered soil analysis programs to methods other than those used in Appendix 2D, such as SHAKE.

230.6 According to the FSAR section 3.7B.1.1 the vertical design response spectra are taken to be two-thirds of the horizontal design response spectra. Discuss the adequacy of the vertical response spectra with respect to the Regulatory Guide 1.60 procedures for determining the vertical response spectra (Reference Regulatory Guide 1.60, Table II). Include in your discussion relevant information obtained from the recent eastern U.S. and Canada earthquake records (1982 New Hampshire and New Brunswick earthquakes).

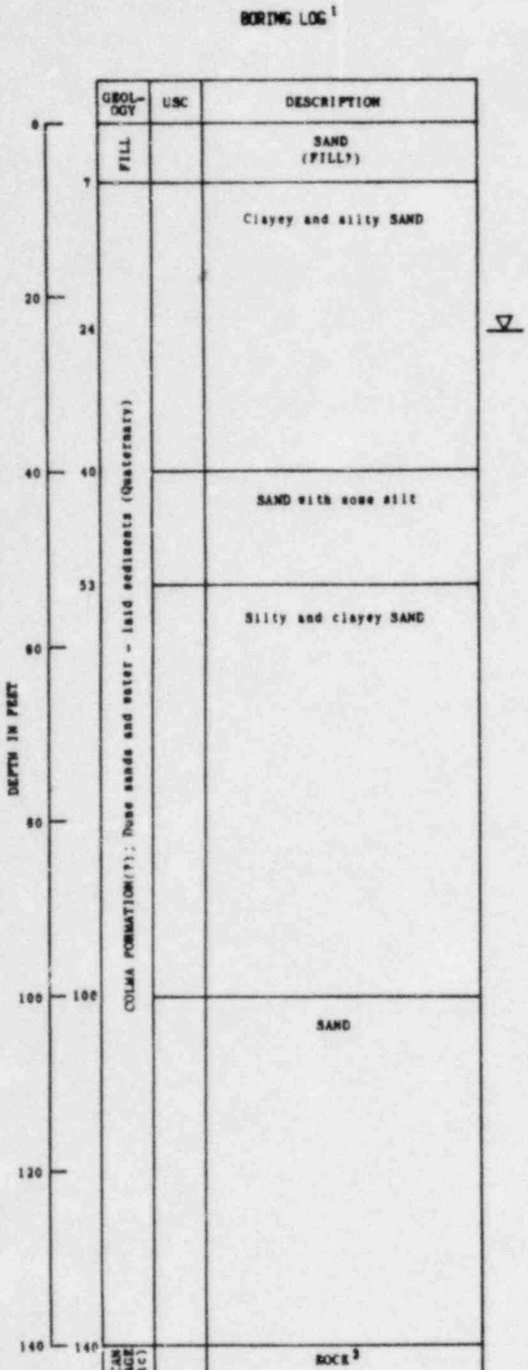
APPENDIX 2

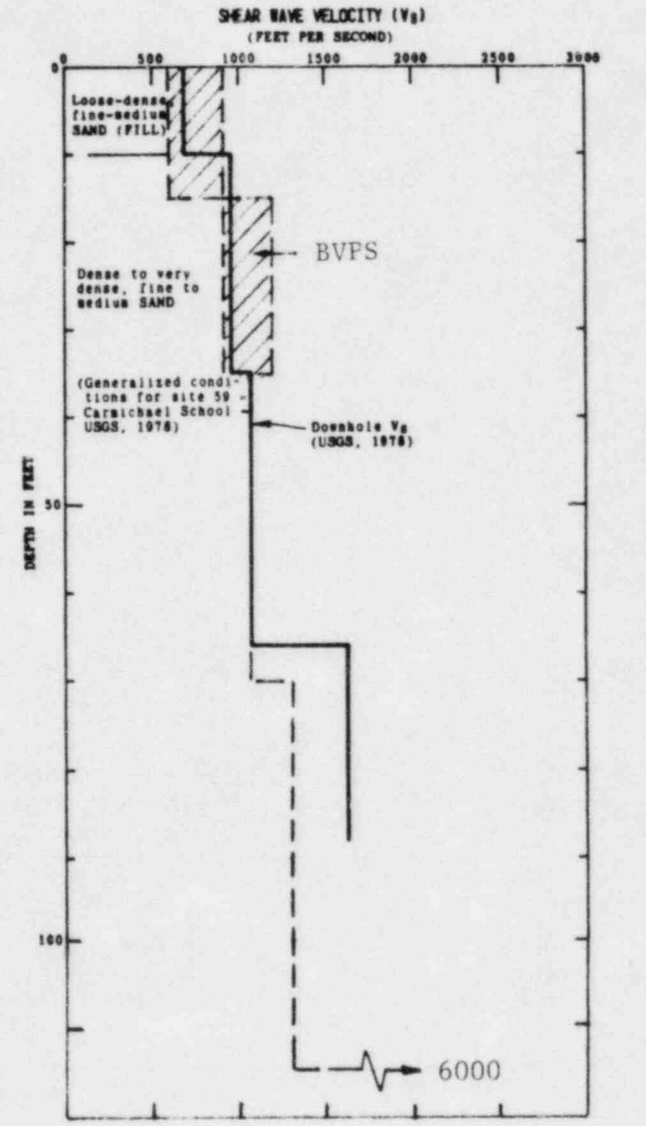
BORING LOGS AND SHEAR WAVE VELOCITY DATA

FOR

SITE MATCHED ACCELEROGRAPH STATIONS

BORING LOG[1]





NOTES:
1) Source of boring information not identified in Seed and Idriss (1969)
2) About 10 feet of fill underlies the site (USGS, unpub.).
3) Bedrock is at Elevation -100 ft. (depth of 135 ft.) based on Schlocker (1974). Depth to bedrock is quite variable in the site area.

BORING

Elevation: Approximately 35 feet MSL (USGS Topographic Quad.)
Data Source: Seed and Idriss (1969)[1]

VELOCITIES

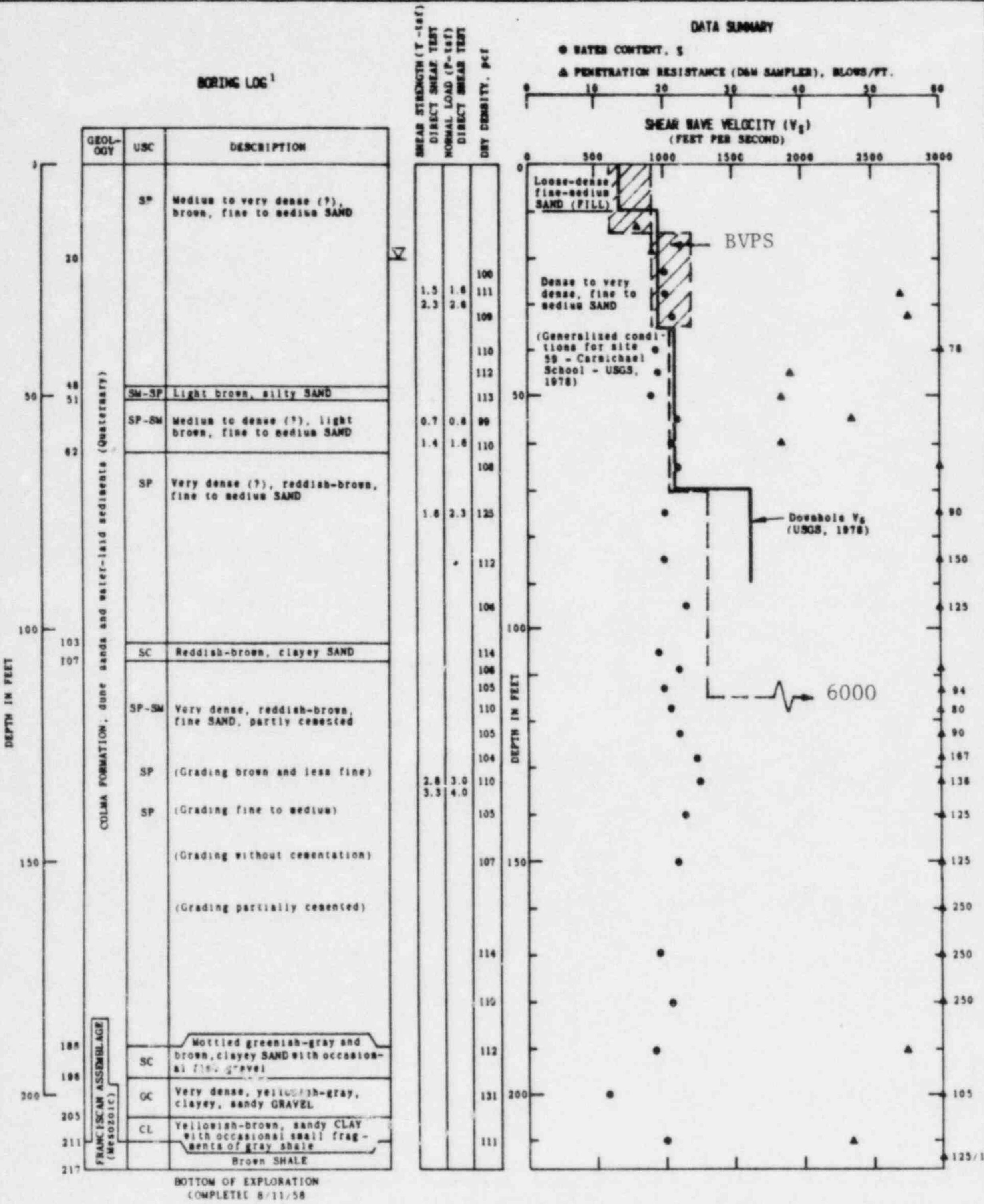
Source: USGS (1977, 1978)
Location: Site 59, Carmichae. ʻhool (CAR) 5300 feet south-sou. .est of Alexander Building.
Date: May 6, 1976

REPORT: SW-AA (1977b)

BUILDING: The Alexander Building (155 Montgomery) is a 16-story high-rise with a basement. The lowest level accelerograph is in the basement, about 10 feet below grade.

SUMMARY LOG
ALEXANDER BUILDING
SAN FRANCISCO, CALIFORNIA

FIGURE A2-1



BORING

Elevation: 67.6 feet

Location: East side of Polk Street, between Turk Street and Golden Gate Avenue. Approximately 300 feet North of State Building.

Data Source: Dames & Moore, 1958 Foundation Investigation for Federal Building Boring No. 4 in report (reprinted in Woodward - Lundgren & Assoc., 1973).

VELOCITIES

Source: USGS (1977 and 1978)

Location: Site 59, Carmichael School (CAR) 4000 feet southeast of State Building.

Date: May 6, 1976

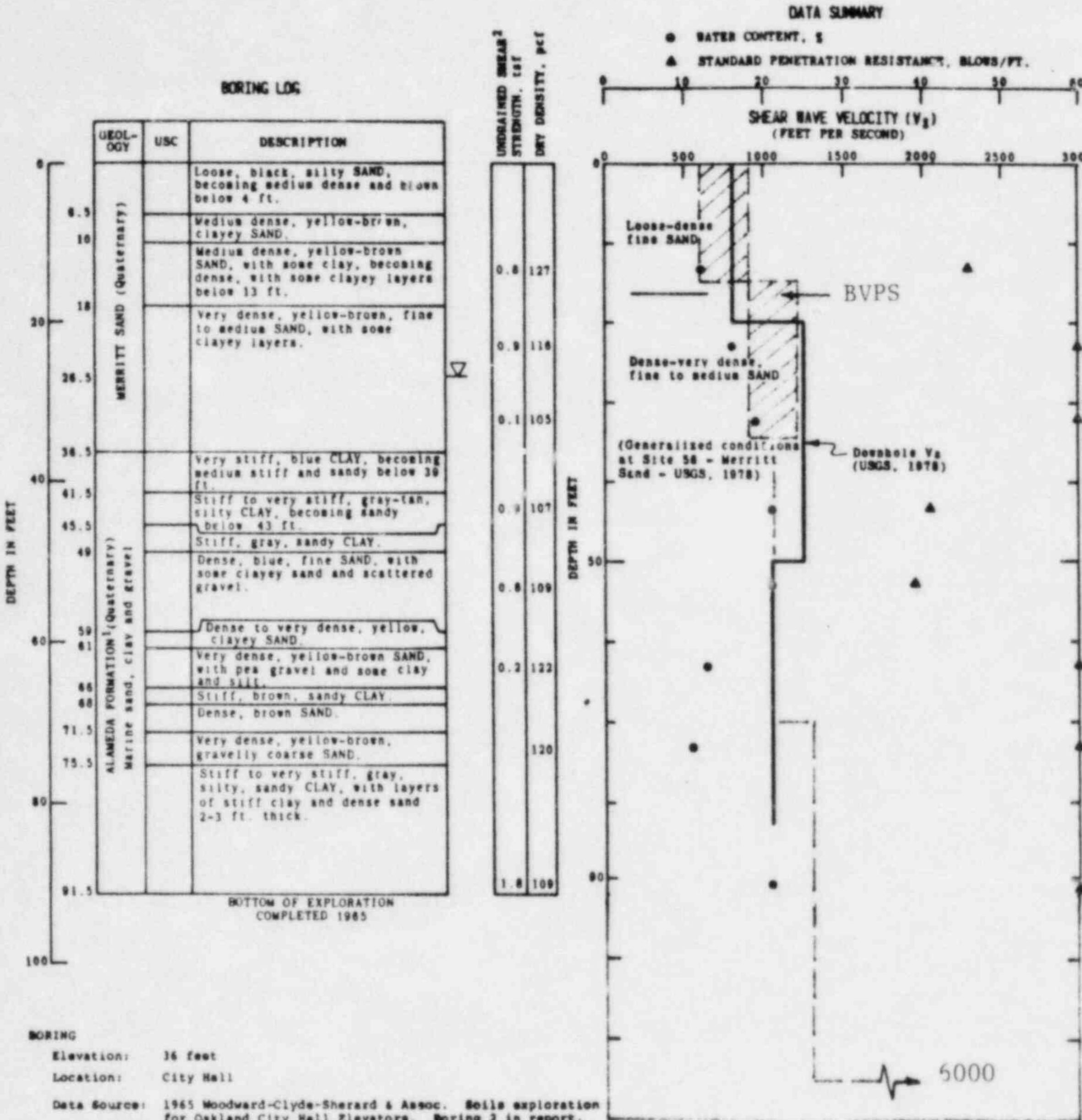
REPORT SM-AA (1977b)

BUILDING The State Bldg. (350 McAllister) is a 7-story structure with a basement (extends about 10 feet below grade) where the accelerograph was located.

NOTE: 1) Depth to bedrock in the vicinity of the State Bldg. is quite variable (Schlocker, 1974).

SUMMARY LOG
STATE BUILDING
SAN FRANCISCO, CALIFORNIA

FIGURE A2-2



BORING

Elevation: 36 feet

Location: City Hall

Data Source: 1965 Woodward-Clyde-Sherard & Assoc. Soils exploration for Oakland City Hall Elevators. Boring 2 in report, reprinted in Woodward - Lundgren & Assoc. (1973).

VELOCITIES

Source: USGS (1977, 1978)

Location: Site 56, Merritt Sand (MER, MS) 2500 feet northwest of Oakland City Hall.

Date: May 3, 1976

REPORT SW-AP (1977b)

BUILDING The Oakland City Hall (1421 Washington Street) is a 15-story structure with a basement. The lowest level accelerograph is in the basement, about 10 feet below grade.

NOTES: 1) The upper section of the Alameda Formation consists of sandy, silty clay. The lower part consists of continental and marine sand, clay and gravel. Maximum known thickness, 1,050 feet. (Woodward-Lundgren and Associates, 1973).

2) Shear strengths are assumed to correspond to U-U tests run at overburden pressure.

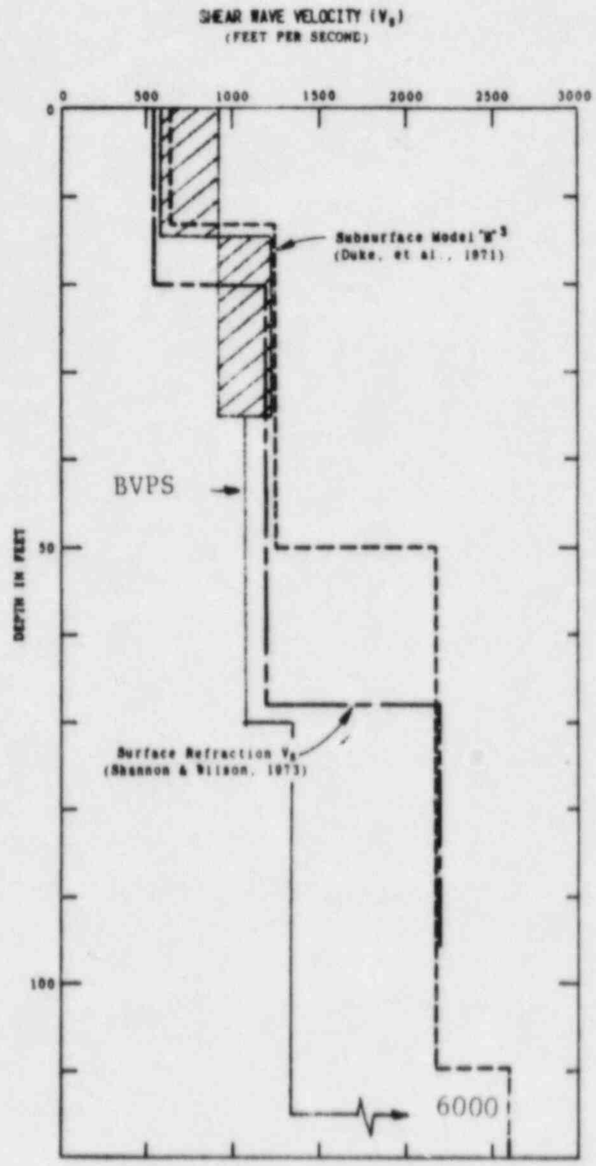
SUMMARY LOG
CITY HALL
OAKLAND, CALIFORNIA

FIGURE A2-3

LOG [1,2]

GEOLOGY	USC	DESCRIPTION
CASTAIC FORMATION; soft shale, siltstone and sandstone (Miocene)		Medium to moderately hard, weathered SANDSTONE

DEPTH IN FEET: 0, 10, 20



VELOCITIES

Data Source:	Shannon & Wilson, Inc. (1973)	– Duke, et al. (1971); Site No. 27, subsurface model "B"
Location:	Two perpendicular lines intersecting at Southeast Corner of Accelerograph Station[2]	– 700 feet Northwest of Accelerograph Station (line elevation 2475 feet, MSL)
Date:	September 1973	– October 4, 1971
Results:	Indicated V_s values are the averages from the two lines	

REPORT SW-AA (1977b)

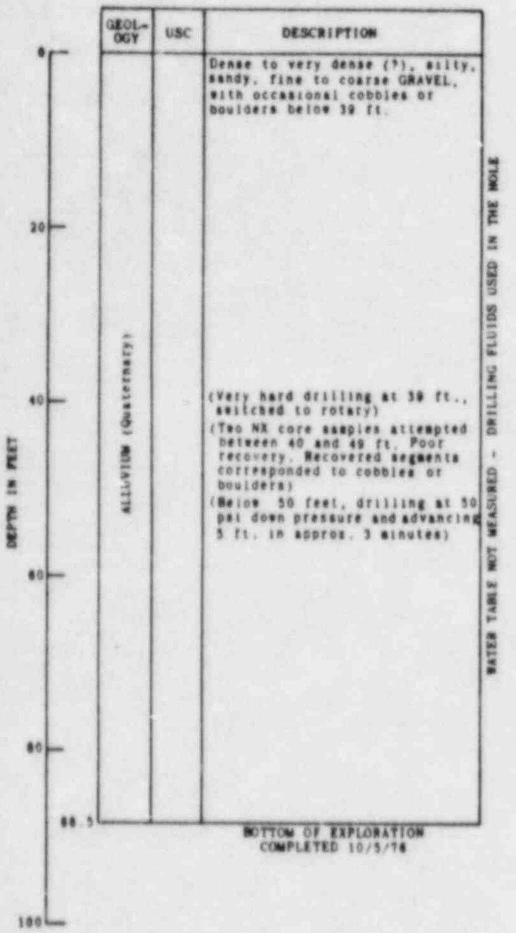
BUILDING The accelerograph, originally located in a small instrument shelter founded at grade, has been relocated to Castaic Dam.

NOTES:
1) No borings were made at the site. Material denoted on log corresponds to that observed during our site reconnaissance (SW-AA, 1977b).
2) Surface elevation approximately 2560 feet, MSL (USGS topographic quad.).
3) Velocities based on refraction measurements to 50 feet. Below 50 feet, velocities were estimated based on site geology.

SUMMARY LOG
OLD RIDGE ROUTE
CASTAIC, CALIFORNIA

FIGURE A2-4

BORING LOG[1]



BORING

Elevation:	Approximately 5930 feet, MSL (USGS topographic quad.)
Location:	15 feet from accelerograph station
Data Source:	SW-AA (1977b)
Equipment:	Mobile B-61 hollow stem auger Augered 0-39 feet sampling only cutting returns. Switched to rotary below 39 feet and sampled cutting returns and attempted 2-NX cores between 40 and 49 feet.

VELOCITIES: Not Available

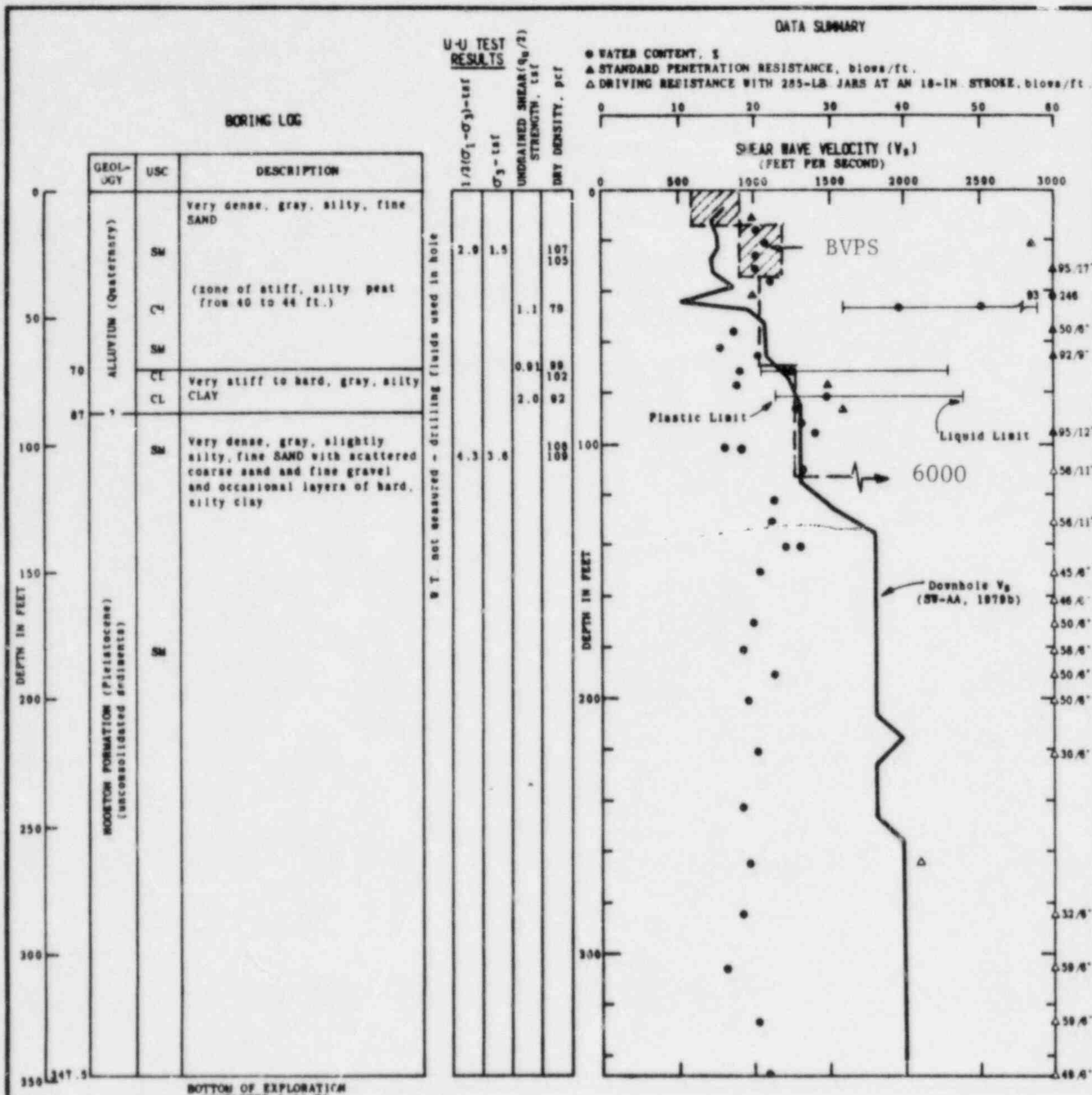
REPORT: SW-AA (1977b)

BUILDING: Although the station has been discontinued, the accelerograph at the United California Bank Building (6074 Park Drive) was located in the daylight basement (grade level) of a one story structure.

NOTE: 1) Bedrock was not encountered within the depth of the boring.

SUMMARY LOG
6074 PARK DRIVE
WRIGHTWOOD, CALIFORNIA

FIGURE A2-5



BORING

Elevation:	40 feet, MSL (interpolated from USGS topo quad)
Location:	12 feet south of the Federal Building
Data Source:	SW-AA (1979b)
Equipment:	Failing 1500 rotary drill; sampling with a 2" O.D. split spoon drive barrel and a Pitcher barrel.

VELOCITIES

Source:	SW-AA (1979b)
Location:	Downhole testing performed in above boring
Date:	May 31, 1979

REPORT SW-AA (1977b, 1979b)

BUILDING The Federal Building (500 "H" St.) is a three-story structure with one basement. The accelerograph is located in the basement, about 10 feet below grade.

SUMMARY LOG
FEDERAL BUILDING
EUREKA, CALIFORNIA

FIGURE A2-6

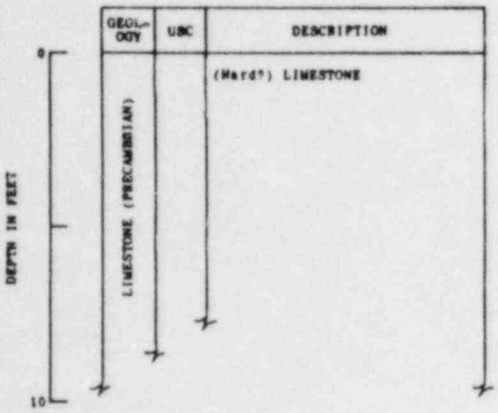
APPENDIX 3

BORING LOGS AND SHEAR WAVE VELOCITY DATA

FOR

ROCK OUTCROP ACCELEROGRAPH STATIONS

LOG [1,2]



VELOCITIES: Not available within vicinity of site

REPORT: SW-AA (1977a)

BUILDING: The Science and Library Building at Carroll College is a 4-story structure with a daylight basement (extends about 5 feet below grade.) The accelerograph was originally located in the basement of the Administration Building. On July 16, 1958, it was moved to the first floor of the Science & Library Building.

NOTES:

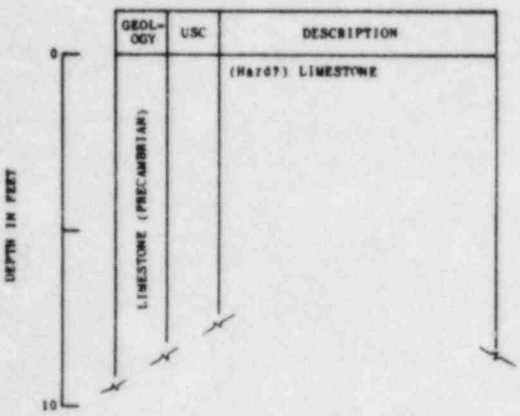
1) No borings were made at the site. Materials indicated on the log were observed during our site reconnaissance (SW-AA, 1977a).

2) Site elevation approximately 4000 feet, MSL (USGS topographic quad.)

SUMMARY LOG
CARROLL COLLEGE
HELENA, MONTANA

FIGURE A3-1

LOG[1,2]



VELOCITIES: Not available within vicinity of the site

REPORT: SW-AA (1977a)

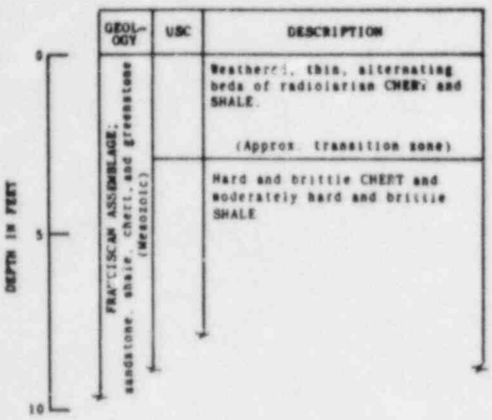
BUILDING Although the station has been discontinued, the accelerograph at the Federal Building was located in the partial basement (5 to 10 feet below grade) of a 4-story structure.

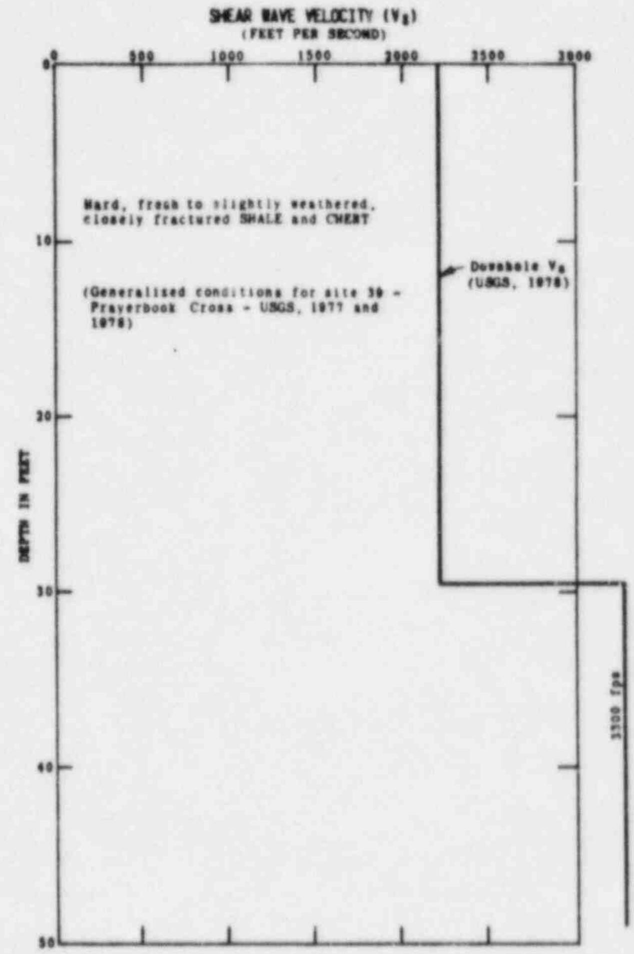
NOTES:
1) No borings were made at the site. Materials indicated on the log were observed during our site reconnaissance. (SW-AA, 1977a).
2) Site Elevation approximately 4000 feet, MSL (USGS topographic quad.).

SUMMARY LOG
FEDERAL BUILDING
HELENA, MONTANA

FIGURE A3-2

LOG[1]





VELOCITIES

Source: USGS (1977 & 1978)

Location: Site 39, Prayerbook Cross (PBR) (site of Golden Gate Park accelerograph station).

Date: March 31, 1976

REPORT SW-AA (1977b)

BUILDING The accelerograph is housed in a small building founded at grade at the base of Prayerbook Cross.

NOTE: 1) No borings were made at the site. The materials indicated on the log were observed during our site reconnaissance. (SW-AA, 1977b).

SUMMARY LOG
GOLDEN GATE PARK
SAN FRANCISCO, CALIFORNIA

FIGURE A3-3

LOG[1,2]

DEPTH IN FEET

GEOLOGY	USC	DESCRIPTION
Jointed quartz monzonitic to dioritic rock (Mesozoic)		Hard, fresh GRANITIC ROCK, jointed

0

10

VELOCITIES Not available

BUILDING The Miller Canyon Guard Station (formerly the Allen Ranch) is a one-story garage with a daylight basement. The accelerograph is located at grade level in the basement.

REPORT SW-AA (1977b)

NOTES: 1) No borings were made at the site. Material descriptions denoted on the log correspond to that observed during our reconnaissance. (SW-AA, 1977b).

2) Site elevation approximately 3490 feet MSL (USGS topographic quad.).

SUMMARY LOG
MILLER CANYON GUARD STATION
CEDAR SPRINGS, CALIFORNIA

LOG[1,2]

DEPTH IN FEET: 0 – 10

GEOLOGY	USC	DESCRIPTION
FOLIATED GRANODIORITIC TO QUARTZ MONZONITIC ROCK (MESOZOIC)		Moderately weathered and decomposed GRANITIC ROCK becoming fresh at depth (probably 15 to 20 feet)

VELOCITIES Not Available

REPORT SW-AA (1977b)

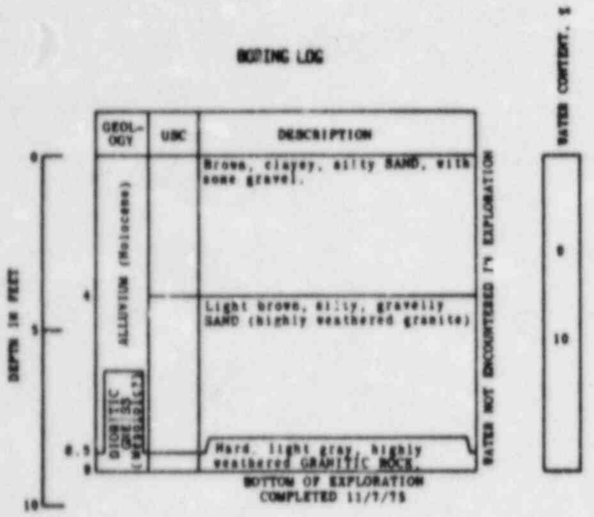
BUILDING The accelerograph is located in a small instrument shelter with a slab-on-grade foundation.

NOTES: 1) No borings were made at the site. Depth of weathering is based upon a rock exposure in a nearby road cut. (SW-AA, 1977b).

2) Site Elevation approximately 3040 feet, MSL (USGS topographic quad.).

SUMMARY LOG
STATION 4
LAKE HUGHES ARRAY
CALIFORNIA

FIGURE A3-5



BORING

Elevation:	Approximately 2080 feet, MSL (USGS topographic quad.)
Location:	5 feet South of Accelerograph station
Data Source:	SW-AA (1976b)
Equipment:	Hand auger

VELOCITIES — Not available within vicinity of the site

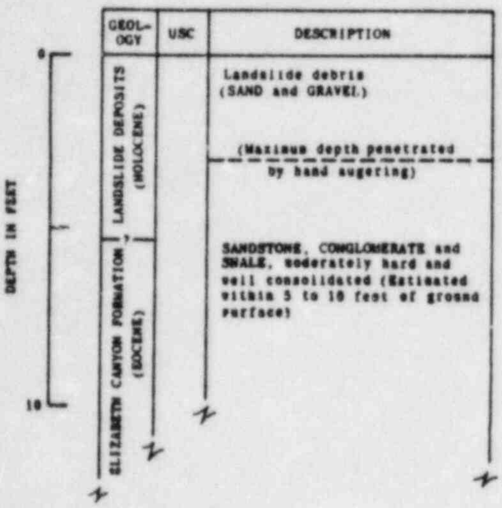
REPORT — SW-AA (1976b)

BUILDING — The accelerograph is located in a one-room shed with a concrete slab-on-grade foundation. The soil beneath a portion of the shed has been eroded, leaving the slab partially suspended.

SUMMARY LOG
STATION 9
LAKE HUGHES ARRAY
CALIFORNIA

FIGURE A3-6

BORING LOG



BORING

Elevation:	Approximately 1640 feet, MSL (USGS topographic quad.)
Location:	Adjacent to accelerograph station
Source:	SW-AA (1977b)
Equipment:	Hand augered to 3 or 4 feet

VELOCITIES	Not available
REPORT	SW-AA (1977b)
BUILDING	The accelerograph is located in a small instrument shelter founded at grade.

SUMMARY LOG
STATION 12
LAKE HUGHES ARRAY
CALIFORNIA

FIGURE A3-7