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December 17, 1984

United States Nuclear Regulatory Commission
Washington, DC 20555

ATTENTION: Mr. George W. Knighton, Chief
Licensing Branch 3
Office of Nuclear Reactor Regulation

SUBJECT: Beaver Valley Power Station - Unit No. 2
Docket No. 50-412
Responses to NRC Structural Engineering Section's Draft SER
Open and Confirmatory Items

Gentlemen:

In three separate submittals dated February 27, 1984; April 27, 1984; and June 15, 1984, DLC provided responses to all 28 of the NRC Structural Engineering Section's Structural Design Audit Action Items. In an NRC letter dated March 1, 1984, DLC received the first portion of the draft SER which included the Structural Engineering Section's input. The Structural Engineering Section identified 13 open items and 2 confirmatory items in this draft SER. Each of these items was directly related to one or more of the 28 Structural Design Audit action items and, therefore, these items were addressed by DLC in the responses provided for the audit action items.

On November 21, 1984, the NRC informally provided DLC with a written summary of the Structural Engineering Section's remaining open and confirmatory items. On November 30, 1984, DLC met with the Structural Engineering Section to discuss resolution of these items which are listed in Attachment 1. This list includes the current status of each item, based on the results of the November 30, 1984, meeting. At this meeting, DLC committed to providing a response to Item SRP 3.8.5 (Audit Action Items 6 and 7) and a schedule for submitting the remaining responses as soon as possible.

Attachment 2 provides responses to the following items:

SRP 3.7.1 (Audit Action Item 1)
SRP 3.8.3 (Audit Action Item 15)
SRP 3.8.3 (Audit Action Item 13)
SRP 3.8.4 (Audit Action Item 19)
SRP 3.8.4 (Audit Action Item 22)
SRP 3.8.5 (Audit Action Items 6 and 7)

The following responses will be submitted by December 21, 1984:

SRP 3.8.1 (Audit Action Item 10)
SRP 3.8.3 (Audit Action Item 28)
SRP 3.8.3 (Audit Action Item 27)

B001
1/1

The response to Item SRP 3.8.5 (Audit Action Items 4, 7, and 23) will be submitted by January 18, 1984. As stated in the November 30, 1984, meeting, DLC will respond to this item by providing an analysis for the containment structure that uses the frequency-dependent impedance approach with the earthquake input motion applied at the foundation level. A discussion of the computer program (FRIDAY) used in this analysis will be provided along with verification examples. Comparisons of floor amplified response spectra will be performed if this analysis indicates that they are necessary.

Attachment 3 provides a copy of the report entitled "Soil-Structure Interaction in the Development of Amplified Response Spectra for Beaver Valley Power Station, Unit 1" which was submitted by DLC to NRC on June 11, 1979, in response to the NRC's March 13, 1979, Order to Show Cause. This report is being provided in accordance with the Structural Engineering Section's request made at the November 30, 1984, meeting concerning the intake structure. As stated in this meeting, the original design of the intake structure and the 1979 Order to Show Cause response included consideration of the future BVPS-2 intake structure loads. As further stated in this meeting, it is DLC's position that the intake structure is a BVPS-1/BVPS-2 shared facility that was previously addressed by DLC and reviewed and approved by the NRC under the BVPS-1 docket. Any further analysis of the intake structure requested by the NRC under the BVPS-2 docket will be addressed by DLC as a backfit item under Generic Letter 84-08.

DUQUESNE LIGHT COMPANY

By E J Woolever
E. J. Woolever
Vice President

JDO/wjs
Attachments

cc: Mr. B. K. Singh, Project Manager (w/attachments 1 & 2)
Mr. G. Walton, NRC Resident Inspector (w/attachments 1 & 2)

SUBSCRIBED AND SWORN TO BEFORE ME THIS
17th DAY OF December, 1984.

Anita Elaine Reiter
Notary Public

ANITA ELAINE REITER, NOTARY PUBLIC
ROBINSON TOWNSHIP, ALLEGHENY COUNTY
MY COMMISSION EXPIRES OCTOBER 20, 1986

ATTACHMENT 1

Structural & Geotechnical Engineering Branch
Remaining Open and Confirmatory Items
from Structural Engineering Section

SRP 3.7.1

Action Item 1 (Confirmatory)

A. Exceedance of vertical resp. spectra (7-10Hz, Fig. 9-1)

B. Subject to Geosciences Branch approval (12/7/84 meeting)

SRP 3.7.3

Action Items 4, 7, and 23 (Open)

A. Performance of the half-space (unbounded) SSI method for the containment and intake structures

B. Use input from (A) to address Items 4, 7, and 23. There is no need to address aux. bldg. again.

SRP 3.8.1

Action Item 10 (Confirmatory)

For different load category, use direct comparison to show that BV-2 meets the criteria of liner strain allowable and the liner anchor allowable as specified in ASME Div. 2 Tables CC-3720-1 and CC-3730-1, respectively.

SRP 3.8.3

A. Action Item 15 (Confirmatory)

Provide the governing load combinations for different parts of the internal structures and show they are compatible to those of ACI-349 and R.G. 1.142.

B. Action Item 13 (Confirmatory)

Submission of final results of the confirmation program to verify the design of the containment's internal structure (including steam generator cubicles) for final pressure loads.

C. Action Item 28 (Confirmatory)

Clarification is needed to explain why the effects of Ta are of no importance. You have indicated in your thermal transient analysis that thermal strain = 0.001 and concrete allowable strain = 0.003.

D. Action Item 27 (Confirmatory)

(1) ABS of two seismic components vs. SRSS of three seismic components on polar crane. Need to show ABS is equally conservative as SRSS.

(2) Use the input from SRP 3.7.3.A, B in (1).

SRP 3.8.4

A. Action Item 19 (Confirmatory)

Submit a typical calculational analysis to show the seismic wave effect on conduits at bends and tees.

B. Action Item 22 (Confirmatory)

Provide documents to show that the existing design margins with the refueling water storage tank and the primary demineralized water storage tank can accommodate the increased loads due to tank wall flexibility.

SRP 3.8.5

A. Action Items 6 and 7 (Open; See SRP 3.7.3)

Safety factors against sliding and overturning for the containment and intake structures. Three componnent earthquake input from 3.7.3.A should be used in the assessment.

ATTACHMENT 2

SRP 3.7.1 (Audit Action Item 1)

A. Exceedance of vertical response spectra (7-10 Hz.; Fig. 9-1 of SWEC, June 1984, report entitled "Seismic Design Response Spectra, BVPS-2").

B. Subject to Geosciences Branch approval (December 7, 1984, meeting).

Response:

DLC submitted a response to NRC Structural Design Audit Action Item 1 in letter 2NRC-4-047, dated April 27, 1984. In this response, DLC indicated that a separate report, entitled "Seismic Design Response Spectra, BVPS-2," would be submitted by June 1, 1984. This was provided in letter 2NRC-4-072, dated June 1, 1984.

Based on our meeting with the NRC Structural Engineering Section on November 30, 1984, it is DLC's understanding that this item will be closed upon the Geosciences Branch approval of the BVPS-2 seismic design response spectra.

SRP 3.8.3 (Audit Action Item 15)

Provide the governing load combinations for different parts of the internal structures and show they are compatible to those of ACI-349 and Reg. Guide 1.142.

Response:

This response supplements our response to NRC Structural Design Audit Action Item 15 provided in letter 2NRC-4-047, dated April 27, 1984, and is being provided in accordance with our November 30, 1984, meeting with the NRC Structural Engineering Section.

The governing load combinations for elements of the internal structure of the containment are equations 7, 8, and 9 of Table 15.1 from our April 27, 1984, response. The following Table 15.1A compares equations 7, 8, and 9 with the corresponding equations of ACI 349-76 and Regulatory Guide 1.142 (also given in the April 27, 1984, response) and demonstrates that they are identical.

TABLE 15.1A

Load Combination Comparison for Design of Containment Internal Structure

Controlling BVPS-2 Load Equations	Corresponding ACI 349-76/ Regulatory Guide 1.142 Load Equations
(7) U=D+L+Ta+Ra+1.5Pa	(6) U=D+L+Ta+Ra+1.5Pa
(8) U=D+L+Ta+Ra+1.25Pa+1.0(Yr+Yj+Ym)+1.25Eo	(7) U=D+L+Ta+Ra+1.25Pa+1.0(Yr+Yj+Ym)+1.25Eo
(9) U=D+L+Ta+Ra+Pa+1.0(Yr+Yj+Ym)+Ess	(8) U=D+L+Ta+Ra+1.0Pa+1.0(Yr+Yj+Ym)+1.0Ess

SRP 3.8.3 (Audit Action Item 13)

Submission of final results of the confirmation program to verify the design of the containment's internal structure (including steam generator cubicles) for final pressure loads.

Response:

This response supplements our response to the NRC Structural Design Audit Action Item 13 provided in letter 2NRC-4-047, dated April 27, 1984, and is being provided in accordance with our November 30, 1984, meeting with the NRC Structural Engineering Section.

The confirmation program, referenced in our original respose to Action Item 13, for the final pressure loads in the steam generator cubicles has been completed. As discussed in the November 30, 1984, meeting, the pressure time histories used as a basis for the confirmation are presented in FSAR Section 6.2, Figures 6.2-50 through 6.2-97. Dynamic load factors (DLF) were calculated in the same manner as described in our April 27, 1984, response for these pressure time histories, and resulted in DLF's ranging from 1.0 to 1.35. Using these DLF's, the cubicles were determined to be structurally adequate.

SRP 3.8.4 (Audit Action Item 19)

Submit a typical calculational analysis to show the seismic wave effect on conduits at bends and tees.

Response:

DLC submitted a response to NRC Structural Design Audit Action Item 19 in letter 2NRC-4-080, dated June 15, 1984. At our meeting on November 30, 1984, DLC provided the NRC Structural Engineering Section with a copy of the requested calculation for their review as an extension of the Structural Design Audit. It is DLC's understanding that this item will be closed upon the Structural Engineering Section's approval of this calculation.

SRP 3.8.4 (Audit Action Item 22)

Provide documents to show that the existing design margins with the refueling water storage tank and the primary demineralized water storage tank can accommodate the increased loads due to tank wall flexibility.

Response:

DLC submitted a response to NRC Structural Design Audit Action Item 22 in letter 2NRC-4-047, dated April 27, 1984. At our meeting on November 30, 1984, with the NRC Structural Engineering Section, DLC summarized the results of the tank wall flexibility analysis for the refueling water storage tank and the primary demineralized water storage tank and provided a copy of the calculation which addresses the tank wall flexibility question for Structural Engineering Section review as an extension of the Structural Design Audit. It is DLC's understanding that this item will be closed upon the Structural Engineering Section's approval of this calculation.

SRP 3.8.5 (Audit Action Items 6 and 7)

Safety factors against sliding and overturning for the containment and intake structures. Three component earthquake input from Item SRP 3.7.3(A) should be used in the assessment.

Response:

This response supplements our response to NRC Structural Design Audit Action Item 6 provided in letter 2NRC-4-047, dated April 27, 1984, and is being provided in accordance with our November 30, 1984, meeting with the NRC Structural Engineering Section. Based on this meeting, it is DLC's understanding that submitting this response makes this a confirmatory item which will be closed upon NRC Structural Engineering Section approval of DLC's response to Item SRP 3.7.3 (Action Items 4, 7, and 23).

Factors of safety against sliding and overturning that account for three-component earthquake input have been calculated for the containment structure (Table 6.1A). The methodology used was discussed and agreed upon at the November 30, 1984, meeting and is described below:

Maximum floor acceleration responses were calculated in accordance with the acceptance criteria of SRP 3.7.2 (that is, the maximum structural responses due to each of the three components of earthquake motion should be combined by taking the square root of the sum of the squares of the maximum codirectional responses of earthquake motion at a particular point of the structure).

From the calculated floor accelerations, the shear and overturning moments at the base of the containment were obtained by summation of inertia forces.

Sliding forces were resisted by friction at the soil/mat interface. Overturning was resisted by a bearing under the foundation mat.

The calculated factors of safety are well above the specified allowable values as seen in Table 6.1A.

During the November 30, 1984, meeting, the NRC Structural Section raised a concern regarding the effect further consideration of Structural Design Audit Action Item 7 would have on the factors of safety for sliding and overturning. It is our judgement that, while the factors of safety may change as a result of further consideration of Action Item 7, the resulting values would not fall below the specified allowables.

TABLE 6.1A

Factors of Safety (SSE Earthquake)

CONDITION	CALCULATED FACTOR OF SAFETY	SRP 3.8.5 LIMITS	BVPS-2 FSAR LIMIT
PLAXLY ANALYSIS			
Overturning (SSE)	4.3	1.1	1.1
Sliding (SSE)	3.2	1.1	1.1
FRIDAY ANALYSIS			
Overturning (SSE)	4.2	1.1	1.1
Sliding (SSE)	3.1	1.1	1.1

COMMONWEALTH OF PENNSYLVANIA)
) SS:
COUNTY OF ALLEGHENY)

On this 17th day of December, 1984, before me, a Notary Public in and for said Commonwealth and County, personally appeared E. J. Woolever, who being duly sworn, deposed and said that (1) he is Vice President of Duquesne Light, (2) he is duly authorized to execute and file the foregoing Submittal on behalf of said Company, and (3) the statements set forth in the Submittal are true and correct to the best of his knowledge.

Anita Elaine Reiter
Notary Public

ANITA ELAINE REITER, NOTARY PUBLIC
ROBINSON TOWNSHIP, ALLEGHENY COUNTY
MY COMMISSION EXPIRES OCTOBER 20, 1986

ATTACHMENT 3

Duquesne Light

435 Sixth Avenue
Pittsburgh, Pennsylvania
15219

(412) 456-6000

August 29, 1979

Mr. Harold R. Denton
Director of Nuclear Reactor Regulation
United States Nuclear Regulatory Commission
Washington, D.C. 20555

ATTENTION: Mr. A. Schwencer, Chief
Operating Reactors Branch No. 1
Division of Operating Reactors

SUBJECT: Beaver Valley Power Station - Unit No. 1
Docket No. 50-334
Soil-Structure Interaction in the
Development of Amplified Response Spectra

Dear Mr. Denton:

Enclosed are 40 copies of errata sheets (1 and 2) for Report on Soil-Structure Interaction in the Development of Amplified Response Spectra for Beaver Valley Power Station, Unit No. 1.

The report on Soil-Structure Interaction in the Development of Amplified Response Spectra for Beaver Valley Power Station, Unit No. 1 was submitted on June 11, 1979.

DUQUESNE LIGHT COMPANY

By E. J. Woolever
E. J. Woolever
Vice President

Enclosure

~~7909050237~~

(CORPORATE SEAL)

Attest:

H. W. Staas
Secretary

COMMONWEALTH OF PENNSYLVANIA)
) SS:
COUNTY OF ALLEGHENY)

On this 29th day of August, 1979, before me, Robert J. Monroe, a Notary Public in and for said Commonwealth and County, personally appeared E. J. Woolever, who being duly sworn, deposed, and said that (1) he is Vice President of Duquesne Light, (2) he is duly authorized to execute and file the foregoing Submittal on behalf of said Company, and (3) the statements set forth in the Submittal are true and correct to the best of his knowledge, information and belief.

Robert J Monroe

ROBERT J. MONROE, Notary Public
PITTSBURGH, ALLEGHENY COUNTY, PA.
MY COMMISSION EXPIRES
FEBRUARY 7, 1983

Beaver Valley Power Station, Unit 1

Errata Sheet
for
Report on
Soil-Structure Interaction in the Development
of Amplified Response Spectra

Duquesne Light Company

June 11, 1979

Please refer to the above report prepared by Stone & Webster Engineering Corp. and make the following changes:

SECTION 2:

Refer to page 2-3 and change equation $\gamma_T = \frac{1 + Wn}{1 + e}$ (SG) $\gamma_w = 126$ pcf

to read $\gamma_T = \frac{1 + Wn}{1 + e}$ (SG) $\gamma_w = 126$ pcf

Change equation $\gamma_T = \frac{SG + (s/100)\ e}{1 + e}\ \ {}_w = 136$ pcf

to read $\gamma_T = \frac{SG + (s/100)\ e}{1 + e} \gamma_w = 136$ pcf

Refer to Figure 2-14 and

Change γ_T 2 136 pcf

to read $\gamma_T = 136$ pcf

Refer to Figure 2-15 and

Change γ_T 2 136 pcf

to read $\gamma_T = 136$ pcf

Refer to Figure 2-16 and

Change γ_T 2 63 pcf

to read $\gamma_T = 63$ pcf

SECTION 5:

Refer to page 5-4 and change the first sentence to read:

"The ARS for cases 1, 2 and 5 are compared in Figures 5-7 through 5-15 for piping damping ratios of .005, .010, and .030."

Duquesne Light
435 Sixth Avenue
Pittsburgh, Pennsylvania
15219

(412) 471-4300

June 11, 1979

Mr. Harold R. Denton
Director of Nuclear Reactor Regulation
United States Nuclear Regulatory Commission
Attention: A. Schwencer, Chief
Operating Reactors Branch No. 1
Division of Operating Reactors
Washington, DC 20555

Reference: Beaver Valley Power Station, Unit No. 1
Docket No. 50-334
Soils-Structure Interaction in the Development of
Amplified Response Spectra.

Gentlemen:

Enclosed are three (3) signed originals and thirty-seven (37) copies of a report which describes the use of "Soils-Structure Interaction in the Development of Amplified Response Spectra for Beaver Valley Power Station, Unit No. 1".

The amplified response spectra, developed in accordance with the methods described in the report, are presently being utilized to perform a reanalysis of a majority of the plant piping systems identified in our April 2, 1979 reply to the Nuclear Regulatory Commission's March 13, 1979 "Order to Show Cause".

The information requested in item 1 of Mr. Eisenhut's letter of May 25, 1979, which confirmed that soil structure interaction methodology may be used on Beaver Valley, is included as Section 8 of the enclosed report.

Very truly yours,

E J Woolever

E. J. Woolever
Vice President, Engineering
and Construction

Enclosure

~~7906220155~~

(CORPORATE SEAL)

Attest:

N W Staas

H. W. Staas
Secretary

COMMONWEALTH OF PENNSYLVANIA)
) SS:
COUNTY OF ALLEGHENY)

On this 11th day of June, 1979, before me, DONALD W. SHANNON, a Notary Public in and for said Commonwealth and County, personally appeared E. J. Woolever, who being duly sworn, deposed, and said that (1) he is Vice President of Duquesne Light, (2) he is duly authorized to execute and file the foregoing Submittal on behalf of said Company, and (3) the statements set forth in the Submittal are true and correct to the best of his knowledge, information and belief.

Donald W. Shannon

DONALD W. SHANNON, NOTARY PUBLIC
PITTSBURGH, ALLEGHENY COUNTY
MY COMMISSION EXPIRES JUNE 7, 1983
Member, Pennsylvania Association of Notaries

SOIL-STRUCTURE INTERACTION IN THE DEVELOPMENT OF AMPLIFIED RESPONSE SPECTRA FOR BEAVER VALLEY POWER STATION, UNIT 1

DUQUESNE LIGHT COMPANY

June 11, 1979

Stone & Webster Engineering Corporation
Boston, Massachusetts

~~7906080158~~

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LIST OF FIGURES (Cont)

1.0 INTRODUCTION

On March 13, 1979 the Nuclear Regulatory Commission (NRC) issued an Order to Show Cause to the Duquesne Light Company. The order required shutdown of the Beaver Valley Power Station Unit 1 within 48 hours after receipt of the order.

The order required all piping systems originally seismically analyzed using algebraic summation of intramodal responses to be reanalyzed using methodology currently acceptable to the NRC staff. In carrying out this reanalysis, amplified response spectra, developed using soil-structure interaction (SSI) techniques, have been used.

Soil-structure interaction has been the subject of much dialogue between the Staff, DLC, and Stone & Webster since the Order, the fundamental purpose of which was to agree on the details of the SSI methodology for use in developing suitable amplified response spectra and subsequent pipe stress analysis.

Over the course of numerous discussions, the NRC staff asked for documentation in a number of areas, and it is the purpose of this report to reply in detail to the NRC staff's requests.

This report describes the basis for performing soil-structure interaction analyses to develop amplified response spectra for use in reevaluating the

pipe stress and support loads. The soil properties are developed from subsurface data into a soil profile, in which each stratum has its own soil parameters. The required dynamic properties in each layer are described first by the small strain values of shear modulus, and then site response analysis is used to develop values of damping and shear modulus that are compatible with the strains to be expected during an earthquake. The design basis earthquake (DBE) and the operating basis earthquake (OBE) are described by ground response spectra and by artificial time histories that give response spectra enveloping the ground response spectra. The analysis of soil-structure interaction is performed by two methods: a one-step, finite element method, and a three-step, analytically based method. The report describes how these methods, including the structural representation, are derived and how they are used in the present case.

Results for different methods and for different input are compared, and their application to pipe stress analysis is discussed.

The results show that the three-step (REFUND/FRIDAY) method gives conservative results that are consistent with the present state-of-the-art of soil-structure interaction.

2.0 SOIL PROPERTIES

The soil properties developed for use in the soil-structure interaction analyses are presented in this section of the report. The computer program SHAKE developed by Schnabel, Lysmer, and Seed[1] and discussed in Section 10.1 was used to calculate strain compatible shear moduli and damping from low strain values determined from field testing and empirical formulae based on laboratory test data. Although most of the data are included in reports previously submitted to the NRC for completeness, the data are summarized below.

2.1 SUBSURFACE DATA

Subsurface information was obtained from several sources, which include the Beaver Valley Power Station (BVPS) Unit 1 FSAR,[2] the Geotechnical Design Criteria for Unit 2,[3] and the report on the Soil Densification Program for Unit 2.[4] The pre-construction borings under the Category 1 structures are located as shown in Figure 2-1. The logs for these borings are included in Appendix 2F of the Unit 1 FSAR. Two seismic cross-hole surveys were performed by Weston Geophysical Laboratory, the first in 1968 and the second in 1977, in conjunction with the Unit 2 Soil Densification Program. The 1977 report summarizing both cross-hole surveys is included as Appendix 10.6 of this report.

2.2 SOIL PROFILES

The soil beneath the plant consists of medium to dense sand and gravels that extend from the shale bedrock at El. 620 to plant grade at El. 735. The interbedded sands and gravels are alluvial deposits that occur as terraces along the Ohio River Valley. The terraces are the result of cyclic aggradation and degradation of local materials and glacial outwash by the Ohio River drainage system during the Pleistocene Epoch. Normal groundwater at the site is at El. 665 and closely reflects the pool elevation of the Ohio River. The groundwater elevation chosen for analysis is El. 675, some 10 feet higher than normal. Figures 2-2 and 2-3 show two typical soil profiles through the site area. Figure 2-2 is an east-west profile encompassing Borings 114, 104, 103, 115, and 116; Figure 2-3 is a north-south profile emcompassing Borings 102, 115 and 101. Included on the profiles are the Standard Penetration Test (SPT) blow counts (N) and Unified Soil Classification System (USCS) symbols for each sample.

2.3 SOIL PARAMETERS

The soil underlying BVPS is granular. From the surveys and field investigations mentioned above, various soil parameters have been calculated. USCS classifications and SPT blow counts are presented on each boring log in Appendix 2F of the Unit 1 FSAR. Values of void ratio (e), specific gravity

(SG), and in-situ dry densities (γd) are those presented for in situ soil in the Beaver Valley Unit 2 Geotechnical Design Criteria.[*] Values of γd and e were calculated from the results of in situ soil testing performed during Unit 1 construction. These values are:

$$\gamma_d = 117 \text{ PCF}$$

$$e = 0.40$$

$$SG = 2.65$$

To calculate the values of total density (γ_T) above the water table an average water content (W_n) of 6.5 percent was assumed.

$$\gamma_T = \frac{1 + W_n}{1 + e} (SG) \ \gamma w = 126 \text{ PCF}$$

Below the water table, 100 percent saturation (S) was assumed giving

$$\gamma_T = \frac{SG + (s/100)e}{1 + e} \ w = 136 \text{ PCF}$$

Values of shear wave velocity (V_s) and compression wave velocity (V_p) for low strain were based on the 1977 Weston Geophysical Laboratory Survey (Appendix 10.6, Table I).

Values of Poisson's ratio (μ) were calculated both above and below the water table based on the relationship between V_s and V_ρ.

Above the water table, an average V_s = 800 fps and an average V_ρ = 2,500 fps were used.

Since:

$$\mu = \frac{1 - 2r}{2 - 2r}$$

where

$$r = \left(V_s/V_p\right)^2$$

then

$$\mu = 0.44$$

Values of dynamic Poisson's ratio below the water table for both the Operating Basis Earthquake (OBE) and Design Basis Earthquake (DBE) were calculated using the free-field strain compatible shear modulus (Gave) determined from the

SHAKE analyses discussed in the next section and the shear modulus values (Gmax) as determined from Weston's cross hole surveys.

For the DBE:

$$G_{ave} = 3{,}000 \text{ KSF}$$

and

$$V_s = \left(\frac{3{,}000}{\rho}\right)^{0.5} \text{ fps}$$

where

$$\rho = 0.136/32.2 \text{ Kip-Sec/ft}^4$$

giving

$$V_s = 843 \text{ fps}$$

Below the water table, the strain compatible compression wave velocity (V_p) was calculated as either the compressional wave velocity of water

$$V_p = 5{,}000 \text{ fps}$$

or as

$$V_p = \overline{V}_p \left(\frac{G_{ave}}{G_{max}}\right)^{0.5}$$

where

$$\overline{V}_p = 6400 \text{ fps (from Weston's report)}$$

If the value of V_p calculated from Gave, Gmax and $\overline{V}_p$ was less than 5,000 fps, the compressional wave velocity of water was used.

Therefore for the DBE:

$$r = (V_s/V_p)^2 = 0.028$$

where

$$V_p = 5{,}000 \text{ fps}$$

and

$$\mu = \frac{1 - 2r}{2 - 2r} = 0.49$$

Table 2-1 presents the strain compatible Poisson's ratio for each layer for Gmax, Gmax plus 50 percent and Gmax minus 50 percent.

Analyses have shown that there is an adequate factor of safety against liquefaction of the granular materials beneath the site. The results of the liquefaction analyses are presented in reports previously submitted to the NRC.[5,6,7]

2.4 MODULUS AND DAMPING PROFILES

Soil profiles were developed for the free-field and under each Category 1 structure. These profiles, presented in Tables 2-2 through 2-16, are based on the soil profiles discussed in Section 2.2. Small strain values of modulus and damping were developed from cross-hole seismic surveys conducted at both Units 1 and 2. Additional analyses were conducted at Gmax values of plus and minus (±) 50 percent. The results of these studies are discussed in Section 2.4.3.

2.4.1 Small Strain Modulus and Damping

The values of small strain shear modulus and damping were based on the results of the Weston Geophysical's Survey (Appendix 10.6). The results of the study were analyzed by Dr. R.V. Whitman and his recommendations are included in his report in Appendix 2D to the FSAR. In summary, Dr. Whitman compared the results of the Weston survey with values of V_s calculated using the Hardin and Black[9] relationship. As can be seen in Figure 2-4, the results from both methods agree closely. The shear wave velocity profile for small strain values used in the present free-field SHAKE analysis is shown in Figure 2-6 and is basically the same as that presented by Whitman in the curve on Figure 2-5.

2.4.2 Strain Dependent Modulus and Damping

The calculation of strain dependent modulus and damping profiles is discussed in detail in the following sections.

2.4.2.1 Summary of SHAKE Analysis

The computer program SHAKE[1] was used to obtain values of shear modulus and damping at strain levels compatible with those induced during the DBE and OBE. The time histories from the El Centro 1940 (North-South component) and Kern

County (Taft S69E) earthquakes were normalized to a peak acceleration of .125g and .06g for the DBE and OBE, respectively. These motions were input at the ground surface (El 735 feet) and deconvolved in the free field down to bedrock through the soil profile in Figure 2-6. The deconvolved time history was then amplified up through the soil profile to the base of the structure. Iterations of shear modulus and damping with strain were performed internally by SHAKE in both the free field and under the structures. The values obtained from the final iteration were tabulated for each layer in the soil profile, and the average values of shear modulus and damping using El Centro and Taft accelerograms as input were used in soil-structure interaction calculations. Strain compatible shear modulus and damping values for the DBE and OBE are included in Tables 2-2 to 2-16.

2.4.2.2 Earthquake Accelerograms

Two strong motion time history accelerograms were used in the SHAKE analyses to determine strain compatible soil properties: the 1940 El Centro earthquake (North-South component) and the 1952 Kern County earthquake (S69E component of the Taft record). The El Centro earthquake record was chosen because it is representative of the strongest motions available from deep soil sites, whereas Taft was chosen because of its wide frequency range and strong motion characteristics.

The Taft S69E record, from the 1952 Kern County earthquake, has a maximum acceleration of .179g at a time of 3.70 sec and a mean square frequency of 2.95 Hz. Each value of the accelerogram was multiplied by a factor of .698 to scale the record to a peak acceleration of .125g for the DBE at Beaver Valley. A similar scaling technique was used to obtain the Taft record for the OBE. Frequencies over 20 Hz were excluded from the time history input at ground surface in order to allow convergence of the iterations when deconvoluting in the free field and to maintain deconvoluted time histories with mean square frequencies close to the original Taft record in each of the layers of the soil profile. The time history at the base layer, El 620 feet in this analysis, was stored for later use in amplification analyses under each of the structures. The peak acceleration of the Taft record at the base layer after deconvolution to El 620 feet was .059g.

The 1940 El Centro earthquake, North-South record, was also used in the SHAKE analyses. The maximum recorded acceleration at El Centro was .349g at a time of 2.12 sec, with a mean square frequency of 3.18 Hz. Each value of the accelerogram was multiplied by a factor of .358 to scale the El Centro record to the Beaver Valley DBE. Frequencies above 20 Hz were cut off the El Centro record. The peak acceleration of the El Centro record at the base layer after deconvolution to El 620 feet was .090g.

2.4.2.3 Soil Profile

A horizontally layered, idealized soil profile was established for the SHAKE analysis based on previous studies discussed in Section 2.2. A description of the profile and relevant soil properties for each layer are included in Figures 2-2 to 2-16 for the free field case and for each structure.

The structures themselves have been represented as "pseudosoils" in the SHAKE analysis. These soils are described by unit weights and shear wave velocities that are compatible with the structure. For the equivalent unit weight, the total weight of the structure was divided by the volume of the pseudosoil layer. The shear wave velocity was computed using the equation for the first harmonic natural period of the structure, which is:

$$V_s = \frac{4H}{T}$$

where:

V_s = equivalent shear wave velocity for structure

H = thickness of pseudosoil layer

T = natural period of the structure

2.4.2.4 Strain Dependency Relationships

The variation of shear modulus with strain is input into SHAKE using the shear modulus factor K varying with strain. For Beaver Valley, K is an empirical factor relating shear modulus to confining stress for the underlying granular material. The shear modulus is calculated from the shear modulus factor K by the following equations:

For sands:

$$G = 1000K\ (\bar{\sigma}_o)^{0.5}\ F$$

where:

G = shear modulus in ksf

K = shear modulus factor for sands

$\bar{\sigma}_o$ = effective octahedral stress in ksf

F_G = scaling factor of low strain shear modulus value

The decrease of shear modulus with increasing shear strain is presented in terms of K to conform with the input format required in the SHAKE program. The strain dependency relationships of K, plotted with shear strain, are presented in SW-AJA,[8] specifically Figure 5-2. These curves are based on

empirical data plotted by Seed and Idriss. The factor F is calculated internally by the program, using the small strain values of shear modulus and K input into the program. This calculated value of F is used in subsequent iterations to compute the new shear modulus based on a K vs shear strain curve that has been shifted from the empirical curve by the factor F to account for site conditions as defined by Gmax.

The increase of damping ratio for sands with increasing shear strain is based on Figure 5-9 of the Shannon and Wilson Report.[8] This curve is based on data plotted by Seed and Idriss. The curves were modified by the use of a damping correction factor,[1] which accounts for the variability of damping with depth:

$$F_D = 2.53 - 0.45 \log \bar{\sigma}v$$

where:

F_D = factor modifying damping curves

$\bar{\sigma}v$ = vertical effective overburden stress in psf

2.4.2.5 Strain Compatible Shear Moduli and Damping

The shear moduli and damping values corresponding to the shear strain induced by the DBE and OBE are presented in tabular form for each structure analyzed and for the free field case in Tables 2-2 through 2-16. The results represent values obtained from the last iteration of shear moduli and damping. Criteria for convergence of iterations were established at plus or minus 5 percent of the previously iterated value. The data include strain-compatible moduli and damping ratios calculated from the two earthquake accelerograms described in Section 2.4.2.2, i.e., El Centro North-South and Taft S69E. An average value was calculated for each soil layer and used to model the soil in subsequent soil-structure interaction analyses.

2.4.2.6 Variation of Shear Modulus

The effect of increasing and decreasing the low strain shear moduli (G_{max}) by 50 percent was evaluated using SHAKE. The El Centro and Taft earthquake records, normalized for the DBE, were input at the ground surface in the free field, deconvoluted to the base layer and then amplified up through the soil to the containment structure. All soil parameters other than the low strain shear moduli remained unchanged.

The strain compatible soil properties for G_{max} plus 50 percent and G_{max} minus 50 percent are listed on Tables 2-13 through 2-16, for the free field and under the reactor containment, respectively. Poisson's ratio, calculated for these cases using small strain values and strain compatible values from the DBE and OBE, are listed on Table 2-1. Strain compatible soil properties for G_{max} are included in Tables 2-2 and 2-3 for the free field and containment, respectively.

To determine the variation of G, which is a function of the product of Gmax and G/G_{max}, it is assumed that Gmax and G/G_{max} are uncorrelated. Thus

$$V_G^2 = V_{Gmax}^2 + V_{G/Gmax}^2 + V_{Gmax}^2 \; V_{G/Gmax}^2$$

where

V_{Gmax} = coefficient of variation of in situ Gmax values from shear wave velocities determined from cross-hole data

$V_{G/Gmax}$ = coefficient of variation of G/Gmax from SW-AJA curves (Figure 5-2)

V_G = coefficient of variation of G values at various shear strain levels

From V_G, the expected variation as a percentage of the average G value for a particular shear strain level can be estimated. This variation was ±8.4 percent at low shear strains and ranged from ±46.1 to ±77.8 percent of the average shear modulus at a shear strain level of 2×10^{-2} to 6×10^{-1} percent, the range of shear strain levels generated by the DBE and OBE at the site. Although the percentage variation of the average G value is higher at higher shear strain levels, the actual range of moduli values is approximately the same as at low strain levels.

2.5 SUMMARY ON SOIL PROPERTIES

Procedures followed to obtain soil properties for the soil-structure interaction analyses and their use in developing amplified response spectra are summarized as follows.

First, a small strain soil profile was developed from the best available soil data, including cross hole seismic shear wave velocity measurements, as well as data from borings and samples.

Second, the effect of an earthquake in the free field was evaluated using the SHAKE computer program. The control motion was specified at the surface of the free field; two real records were used - El Centro and Taft - normalized to the acceleration level of the specified design earthquake (OBE or DBE). The program iterated to obtain values of shear modulus and damping compatible with the levels of strain developed during the earthquake. The average of the results from the two records was used in further analyses and is here called the strain compatible, free field profile.

Third, the moduli and material damping for the strain compatible, free field profile were used for the REFUND/FRIDAY analyses.

Fourth, the motion at the base of the profile obtained in the SHAKE analysis of the free field was input to several profiles representing the soil column under the Category I buildings. The top layers of these profiles had masses and fundamental periods equivalent to those of the corresponding buildings. The small strain values of soil shear moduli were adjusted to account for the additional static stresses imposed by the buildings. The computer program SHAKE was run to obtain strain compatible moduli and damping values for each building profile. The average of results for the two time histories established each profile.

Fifth, the strain compatible properties under each building were used in the finite element dynamic analyses as soil properties directly under the corresponding buildings. The strain compatible, free field soil properties were used for the elements representing the free field. Strain compatible soil properties were interpolated between these values for two columns of elements adjacent to the building.

Sixth, no further iteration on soil properties was performed in either the REFUND/FRIDAY or the finite element analysis.

2.6 REFERENCES

1. Schnabel, P.B., Lysmer, J., and Seed, H.B. SHAKE, A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites. Earthquake Engineering Research Center, Report No. EERC72-12 December 1972 (as modified for SWEC Computer System in Program ST211 Version 2 Level 0).

2. Beaver Valley Power Station-Unit 1, Final Safety Analysis Report, Appendix B, Section B.1.2, Seismic Design.

3. Geotechnical Design Criteria, BVPS Unit 2. 2BVM-80, Stone & Webster Engineering Corp, Boston, issued 6/23/77, revised 3/15/78, p. 4.

4. Report of Soil Densification Program, BVPS Unit 2. Stone & Webster Engineering Corp, Boston, 9/23/76.

5. Supplement to Soil Study - Category I Structures, BVPS Unit 1. Duquesne Light Co., Pittsburgh, January 13, 1976.

6. Supplement No. 2 to Soil Study - Category I Structures, BVPS Unit 1. Duquesne Light Co., Pittsburgh, April 30, 1978.

7. Soil Analysis of Turbine Building and Northern Yard Area, BVPS Unit No. 1. Duquesne Light Co., Pittsburgh, April 30, 1979.

8. Soil Behavior Under Earthquake Loading Conditions. USAEC SW-AJA, January 1972.

9. Hardin, B.O. and Black, W.L., Closure to Vibration Modulus of Normally Consolidated Clays. Journal of Soil Mechanics and Foundations Division, ASCE Volume 95, SM 6, November 1969.

TABLE 2-1

STRAIN COMPATIBLE POISSON'S RATIOS
Free Field - El. 735

Layer No.	Top of Layer El.	Gmax DBE	OBE	Gmax + 50% DBE	Gmax - 50% DBE
1	735	0.440	0.440	0.440	0.440
2	725	0.440	0.440	0.440	0.440
3	715	0.440	0.440	0.440	0.440
4	705	0.440	0.440	0.440	0.440
5	695	0.440	0.440	0.440	0.440
6	690	0.440	0.440	0.440	0.440
7	685	0.440	0.440	0.440	0.440
8	675	0.490	0.480	0.473	0.493
9	665	0.490	0.480	0.473	0.493
10	655	0.490	0.480	0.473	0.493
11	645	0.490	0.480	0.473	0.493
12	635	0.490	0.480	0.473	0.493
13	625	0.490	0.480	0.473	0.493
14	620				

TABLE 2-2

STRAIN COMPATIBLE SOIL PROPERTIES

Free Field - Elevation 735

Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values Cs (fps)	Total Unit Wt (kcf)	Material	DBE = .125 g Shear Modulus (ksf) Taft S69E	DBE Shear Modulus (ksf) ElCentro 1940 NS	DBE Shear Modulus (ksf) Average	DBE Damping Ratio Taft S69E	DBE Damping Ratio ElCentro 1940 NS	DBE Damping Ratio Average	OBE = 0.06 g Shear Modulus (ksf) Taft S69E	OBE Shear Modulus (ksf) ElCentro 1940 NS	OBE Shear Modulus (ksf) Average	OBE Damping Ratio Taft S69E	OBE Damping Ratio ElCentro 1940 NS	OBE Damping Ratio Average
1	10	735	600	.125	Sand	1095	1091	1093	.041	.041	.041	1242	1240	1241	.025	.026	.026
2	10	725	800	.125	Sand	1728	1701	1715	.054	.055	.055	2074	2062	2068	.033	.034	.034
3	10	715	950	.125	Sand	2369	2285	2327	.057	.060	.059	2859	2813	2836	.036	.038	.037
4	10	705	950	.125	Sand	2114	1979	2047	.068	.074	.071	2685	2624	2655	.043	.046	.045
5	5	695	1100	.125	Sand	3024	2820	2922	.062	.069	.066	3724	3612	3668	.040	.043	.042
6	5	690	1100	.125	Sand	2880	2696	2788	.066	.073	.070	3637	3529	3583	.042	.046	.044
7	10	685	1100	.125	Sand	2694	2564	2629	.073	.077	.075	3520	3434	3477	.046	.049	.048
8	10	675	1100	.136	Sand	2837	2714	2775	.076	.079	.078	3766	3702	3734	.048	.050	.049
9	10	665	1200	.136	Sand	3490	3417	3454	.073	.075	.074	4538	4471	4505	.046	.048	.047
10	10	655	1200	.136	Sand	3327	3267	3297	.077	.079	.078	4428	4396	4412	.049	.050	.050
11	10	645	1200	.136	Sand	3207	3192	3200	.080	.080	.080	4342	4322	4332	.051	.052	.052
12	10	635	1200	.136	Sand	3124	3142	3133	.082	.082	.082	4277	4270	4274	.053	.053	.053
13	5	625	1200	.136	Sand	3083	3119	3101	.083	.082	.083	4239	4257	4248	.054	.053	.054
14		620	5000	.160	Rock												

NOTE:

Ground water table at El. 675

IMAGE EVALUATION
TEST TARGET (MT-3)

150mm

6"

TABLE 2-3

STRAIN COMPATIBLE SOIL PROPERTIES

Reactor Building

						DBE = .125 g						OBE = .06 g					
						Shear Modulus (ksf)			Damping Ratio			Shear Modulus (ksf)			Damping Ratio		
Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values Cs (fps)	Total Unit Wt (kcf)	Material	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average
1	10	735	1091	.138	Reactor Building Pseudo-soil												
2	10	725	1091	.138	Reactor Building Pseudo-soil												
3	10	715	1091	.138	Reactor Building Pseudo-soil												
4	10	705	1091	.138	Reactor Building Pseudo-soil												
5	10	695	1091	.138	Reactor Building Pseudo-soil												
6	4	685	1091	.138	Reactor Building Pseudo-soil												
7	6	681	1100	.125	Sand	2416	2111	2264	.082	.092	.087	3319	3232	3276	.052	.055	.054
8	10	675	1100	.136	Sand	2651	2366	2509	.081	.090	.086	3616	3529	3573	.052	.055	.054
9	10	665	1200	.136	Sand	3352	3090	3221	.076	.083	.080	4416	4295	4356	.049	.052	.051
10	10	655	1200	.136	Sand	3192	3030	3111	.080	.085	.083	4340	4207	4274	.051	.054	.053
11	10	645	1200	.136	Sand	3045	3030	3037	.084	.085	.085	4266	4133	4200	.053	.056	.055

TABLE 2-3 (Cont)

Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values Cs (fps)	Total Unit Wt (kcf)	Material	DBE = .125 g Shear Modulus (ksf) Taft S69E	DBE = .125 g Shear Modulus (ksf) ElCentro 1940 NS	DBE = .125 g Shear Modulus (ksf) Average	DBE = .125 g Damping Ratio Taft S69E	DBE = .125 g Damping Ratio ElCentro 1940 NS	DBE = .125 g Damping Ratio Average	OBE = 0.06 g Shear Modulus (ksf) Taft S69E	OBE = 0.06 g Shear Modulus (ksf) ElCentro 1940 NS	OBE = 0.06 g Shear Modulus (ksf) Average	OBE = 0.06 g Damping Ratio Taft S69E	OBE = 0.06 g Damping Ratio ElCentro 1940 NS	OBE = 0.06 g Damping Ratio Average
12	10	635	1200	.136	Sand	2945	3039	2992	.087	.084	.086	4212	4079	4146	.054	.057	.056
13	5	625	1200	.136	Sand	2908	3008	2958	.088	.085	.087	4182	4071	4127	.055	.058	.057
14		620	5000	.160	Rock												

NOTE:

Ground water table at El. 675

TABLE 2-4

STRAIN COMPATIBLE SOIL PROPERTIES

Safeguards Building

						DBE = 0.125 g						OBE = 0.06 g					
						Shear Modulus (ksf)			Damping Ratio			Shear Modulus (ksf)			Damping Ratio		
Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values Cs (fps)	Total Unit Wt (kcf)	Material	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average
1	11.5	735	1243	.137	Safeguard Bldg. Pseudo-Soil												
2	8.5	723.5	800	.125	Sand	1654	1587	1621	.058	.062	.060	2007	1961	1984	.037	.040	.039
3	10	715	950	.125	Sand	2292	2141	2216	.060	.067	.064	2801	2708	2755	.039	.042	.041
4	10	705	950	.125	Sand	2008	1826	1917	.073	.081	.077	2629	2521	2575	.046	.050	.048
5	5	695	1100	.125	Sand	2629	2675	2802	.065	.073	.069	3663	3504	3584	.041	.046	.044
6	5	690	1100	.125	Sand	2794	2547	2671	.069	.078	.074	3588	3419	3504	.044	.049	.047
7	10	685	1100	.125	Sand	2624	2420	2522	.075	.082	.079	3479	3322	3401	.047	.052	.050
8	10	675	1100	.136	Sand	2786	2601	2694	.077	.083	.080	3734	3594	3664	.049	.053	.051
9	10	665	1200	.136	Sand	3460	3354	3407	.074	.076	.075	4504	4356	4430	.047	.051	.049
10	10	655	1200	.136	Sand	3309	3223	3266	.077	.080	.079	4394	4283	4339	.050	.053	.052
11	10	645	1200	.136	Sand	3204	3194	3199	.080	.080	.080	4308	4213	4261	.052	.054	.053
12	10	635	1200	.136	Sand	3138	3135	3137	.082	.082	.082	4244	4172	4208	.053	.055	.054
13	5	625	1200	.136	Sand	3114	3144	3129	.083	.082	.082	4210	4170	4190	.054	.055	.055
14		620	5000	.160	Rock												

NOTE:

Ground water table at El. 675

TABLE 2-5

STRAIN COMPATIBLE SOIL PROPERTIES

Auxiliary Building

						DBE = .125 g						OBE = 0.06 g					
			Low Strain Values			Shear Modulus (ksf)			Damping Ratio			Shear Modulus (ksf)			Damping Ratio		
Layer No.	Thick-ness (ft)	Top of Layer Elev.	Cs (fps)	Total Unit Wt (kcf)	Material	Taft S69E	ElCentro 1940 NS	Aver-age	Taft S69E	ElCentro 1940 NS	Aver-age	Taft S69E	ElCentro 1940 NS	Aver-age	Taft S69E	ElCentro 1940 NS	Aver-age
1	10	735	1105	.179	Auxiliary Building Pseudo-soil												
2	11	725	1105	.179	Auxiliary Building Pseudo-soil												
3	9	714	950	.125	Sand	2024	1822	1923	.072	.081	.077	2594	2539	2567	.047	.050	.049
4	10	705	950	.125	Sand	1786	1578	1682	.083	.092	.088	2453	2393	2423	.053	.056	.055
5	5	695	1100	.125	Sand	2707	2444	2576	.072	.081	.077	3465	3364	3415	.048	.051	.050
6	5	690	1100	.125	Sand	2626	2369	2498	.075	.084	.080	3413	3296	3355	.049	.053	.051
7	10	685	1100	.125	Sand	2518	2272	2395	.079	.087	.083	3348	3215	3282	.051	.055	.053
8	10	675	1100	.136	Sand	2755	2577	2666	.078	.084	.081	3654	3501	3578	.051	.055	.053
9	10	665	1200	.136	Sand	3467	3347	3407	.073	.076	.075	4488	4268	4378	.048	.053	.051
10	10	655	1200	.136	Sand	3336	3302	3319	.077	.078	.078	4436	4212	4324	.049	.054	.052
11	10	645	1200	.136	Sand	3247	3234	3241	.079	.079	.079	4376	4171	4274	.050	.055	.052
12	10	635	1200	.136	Sand	3189	3212	3201	.081	.080	.081	4314	4142	4228	.052	.056	.054
13	5	625	1200	.136	Sand	3152	3153	3153	.082	.082	.082	4280	4147	4214	.053	.056	.055
14		620	5000	.160	Rock												

TABLE 2-6

STRAIN COMPATIBLE SOIL PROPERTIES

Service Building

						DBE = .125 g						OBE = 0.06 g					
						Shear Modulus (ksf)			Damping Ratio			Shear Modulus (ksf)			Damping Ratio		
Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values Cs (fps)	Total Unit Wt (kcf)	Material	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average
1	10	735	2372	.072	Service Building Pseudo-soil												
2	15.5	725	2372	.072	Service Building Pseudo-soil												
3	4.5	709.5	950	.125	Sand	2525	2551	2538	.050	.049	.050	2980	2993	2987	.031	.031	.031
4	10	705	950	.125	Sand	2247	2315	2281	.062	.059	.061	2768	2826	2797	.040	.038	.039
5	5	695	1100	.125	Sand	3099	3189	3144	.059	.056	.058	3772	3881	3827	.038	.035	.037
6	5	690	1100	.125	Sand	2918	3049	2984	.065	.061	.063	3672	3787	3730	.041	.038	.040
7	10	685	1100	.125	Sand	2697	2835	2766	.073	.068	.071	3553	3675	3614	.045	.041	.043
8	10	675	1100	.136	Sand	2782	3011	2897	.077	.070	.074	3816	3978	3897	.046	.042	.044
9	10	665	1200	.136	Sand	3364	3727	3546	.076	.067	.072	4624	4768	4696	.044	.041	.043
10	10	655	1200	.136	Sand	3136	3555	3346	.082	.071	.077	4536	4607	4572	.046	.045	.046
11	10	645	1200	.136	Sand	2974	3403	3189	.086	.075	.081	4476	4482	4479	.048	.048	.048
12	10	635	1200	.136	Sand	2870	3279	3075	.089	.078	.084	4422	4386	4404	.049	.050	.050
13	5	625	1200	.136	Sand	2825	3231	3028	.090	.079	.085	4398	4350	4374	.050	.051	.051
14		620	5000	.160	Rock												

TABLE 2-7

STRAIN COMPATIBLE SOIL PROPERTIES

Cable Vault (Main Steam Building)

						DBE = 0.125 g						OBE = 0.06 g					
						Shear Modulus (ksf)			Damping Ratio			Shear Modulus (ksf)			Damping Ratio		
Layer No.	Thick-ness (ft)	Top of Layer Elev.	Low Strain Values Cs (fps)	Total Unit Wt (kcf)	Material	Taft S69E	ElCentro 1940 NS	Aver-age	Taft S69E	ElCentro 1940 NS	Aver-age	Taft S69E	ElCentro 1940 NS	Aver-age	Taft S69E	ElCentro 1940 NS	Aver-age
1	10	735	1135	.101	Cable Vault Pseudo-Soil												
2	12	725	1135	.101	Cable Vault Pseudo-Soil												
3	8	713	950	.125	Sand	2399	2419	2409	.055	.055	.055	2884	2986	2935	.035	.031	.033
4	10	705	950	.125	Sand	2100	2085	2093	.069	.069	.069	2691	2775	2733	.043	.039	.041
5	5	695	1100	.125	Sand	2975	2969	2972	.063	.063	.063	3723	3787	3755	.040	.037	.039
6	5	690	1100	.125	Sand	2829	2834	2832	.068	.068	.068	3646	3682	3664	.042	.040	.041
7	10	685	1100	.125	Sand	2638	2641	2640	.075	.075	.075	3544	3562	3553	.045	.044	.045
8	10	675	1100	.136	Sand	2767	2798	2783	.078	.077	.078	3788	3800	3794	.047	.046	.047
9	10	665	1200	.136	Sand	3403	3518	3461	.075	.072	.074	4562	4591	4577	.046	.045	.046
10	10	655	1200	.136	Sand	3236	3361	3299	.079	.076	.078	4439	4520	4480	.049	.046	.048
11	10	645	1200	.136	Sand	311[illegible]	3274	3193	.083	.078	.081	4339	4427	4383	.051	.049	.050
12	10	635	1200	.136	Sand	3033	3172	3103	.085	.081	.083	4266	4390	4328	.053	.049	.051
13	5	625	1200	.136	Sand	3003	3158	3081	.085	.081	.083	4228	4386	4307	.054	.049	.052
14		620	5000	.160	Rock												

TABLE 2-8

STRAIN COMPATIBLE SOIL PROPERTIES

Diesel Generator Building

						DBE = .125 g						OBE = 0.06 g					
						Shear Modulus (ksf)			Damping Ratio			Shear Modulus (ksf)			Damping Ratio		
Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values Cs (fps)	Total Unit Wt (kcf)	Material	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average
1	4.5	735	514	.357	Diesel Generator Building Pseudo-soil												
2	5.5	730.5	600	.125	Sand	643	592	618	.091	.098	.095	975	938	957	.053	.057	.055
3	10	725	800	.125	Sand	1360	1223	1292	.077	.086	.082	1837	1760	1799	.047	.052	.050
4	10	715	950	.125	Sand	2016	1831	1924	.072	.081	.077	2633	2496	2565	.046	.051	.049
5	10	705	950	.125	Sand	1797	1650	1724	.082	.089	.086	2503	2345	2424	.051	.058	.055
6	5	695	1100	.125	Sand	2726	2511	2619	.072	.079	.076	3509	3301	3405	.046	.053	.050
7	5	690	1100	.125	Sand	2637	2446	2542	.075	.081	.078	3453	3231	3342	.048	.055	.052
8	10	685	1100	.125	Sand	2530	2376	2453	.078	.083	.081	3384	3147	3266	.050	.058	.054
9	10	675	1100	.136	Sand	2791	2668	2730	.077	.081	.079	3685	3418	3552	.050	.058	.054
10	10	665	1200	.136	Sand	3554	3366	3460	.071	.076	.074	4513	4179	4346	.047	.055	.051
11	10	655	1200	.136	Sand	3457	3287	3372	.074	.078	.076	4462	4116	4289	.048	.057	.053
12	10	645	1200	.136	Sand	3382	3237	3310	.076	.079	.078	4393	4086	4240	.050	.057	.054
13	10	635	1200	.136	Sand	3325	3206	3266	.077	.080	.079	4347	4076	4212	.051	.057	.054
14	5	625	1200	.136	Sand	3293	3210	3252	.078	.080	.079	4324	4078	4201	.052	.057	.055
15		620	5000	.160	Rock												

TABLE 2-9

STRAIN COMPATIBLE SOIL PROPERTIES

Fuel Building

						DBE = .125 g						OBE = 0.06 g					
						Shear Modulus (ksf)			Damping Ratio			Shear Modulus (ksf)			Damping Ratio		
Layer No.	Thick-ness (ft)	Top of Layer Elev.	Low Strain Values Cs (fps)	Total Unit Wt (kcf)	Material	Taft S69E	ElCentro 1940 NS	Aver-age	Taft S69E	ElCentro 1940 NS	Aver-age	Taft S69E	ElCentro 1940 NS	Aver-age	Taft S69E	ElCentro 1940 NS	Aver-age
1	13	735	1092	.296	Fuel Bldg. Pseudo-Soil												
2	7	722	800	.125	Sand	1093	1063	1078	.094	.097	.096	1702	1531	1617	.055	.066	.061
3	10	715	950	.125	Sand	1738	1678	1708	.085	.088	.087	2502	2322	2412	.051	.059	.055
4	10	705	950	.125	Sand	1563	1501	1532	.093	.097	.095	2404	2170	2287	.055	.066	.061
5	5	695	1100	.125	Sand	2430	2254	2342	.082	.088	.085	3408	3149	3279	.049	.057	.053
6	5	690	1100	.125	Sand	2361	2185	2273	.084	.090	.087	3371	3100	3236	.051	.059	.055
7	10	685	1100	.125	Sand	2275	2110	2193	.087	.092	.090	3322	3007	3165	.052	.062	.057
8	10	675	1100	.136	Sand	2523	2310	2417	.085	.092	.089	3623	3293	3458	.052	.062	.057
9	10	665	1200	.136	Sand	3247	2855	3051	.079	.089	.084	4443	4114	4279	.049	.057	.053
10	10	655	1200	.136	Sand	3161	2681	2921	.081	.094	.088	4400	4070	4235	.050	.058	.054
11	10	645	1200	.136	Sand	3075	2573	2824	.084	.098	.091	4373	4055	4214	.050	.058	.054
12	10	635	1200	.136	Sand	3002	2510	2756	.085	.102	.094	4358	4033	4193	.051	.059	.055
13	5	625	1200	.136	Sand	2943	2485	2714	.087	.103	.095	4353	4016	4185	.051	.059	.055
14		620	5000	.160	Rock												

TABLE 2-10

STRAIN COMPATIBLE SOIL PROPERTIES

Free Field - Elevation 645
(North of Intake Structure)

						DBE = .125 g						OBE = .06 g					
						Shear Modulus (ksf)			Damping Ratio			Shear Modulus (ksf)			Damping Ratio		
Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values Cs (fps)	Total Unit Wt (kcf)	Material	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average
1	5	645	1200	.136	Sand	5917	5853	5885	.014	.015	.015	6052	5925	5989	.012	.014	.013
2	10	640	1200	.136	Sand	5402	5211	5307	.025	.030	.028	5804	5465	5635	.016	.024	.020
3	10	630	1200	.136	Sand	4911	4635	4773	.037	.044	.041	5535	5124	5330	.022	.032	.027
4		620	5000	.160	Rock												

NOTE:

Ground water table at El. 675

TABLE 2-11

STRAIN COMPATIBLE SOIL PROPERTIES

Free Field - Elevation 675
(South of Intake Structure)

						DBE = .125 g						OBE = 0.06 g					
						Shear Modulus (ksf)			Damping Ratio			Shear Modulus (ksf)			Damping Ratio		
Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values Cs (fps)	Total Unit Wt (kcf)	Material	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average
1	10	675	600	.136	Sand	1028	960	994	.057	.064	.061	1167	1172	1170	.043	.043	.043
2	10	665	1100	.136	Sand	3642	3489	3566	.051	.056	.053	4095	4238	4167	.038	.034	.036
3	10	655	1100	.136	Sand	3162	2832	2977	.066	.076	.071	3685	3919	3802	.050	.043	.047
4	10	645	1200	.136	Sand	3651	3274	3463	.069	.078	.074	4301	4583	4442	.052	.045	.049
5	10	635	1200	.136	Sand	3413	3013	3213	.075	.085	.080	4142	4436	4289	.056	.049	.053
6	5	625	1200	.136	Sand	3311	2868	3090	.077	.089	.083	4083	4388	4236	.057	.050	.054
7		620	5000	.160	Rock												

NOTE:

Ground water table at El. 675

TABLE 2-12

STRAIN COMPATIBLE SOIL PROPERTIES

Main Intake Structure

						DBE = .125 g						OBE = 0.06 g					
						Shear Modulus (ksf)			Damping Ratio			Shear Modulus (ksf)			Damping Ratio		
Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values Cs (fps)	Total Unit Wt (kcf)	Material	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average	Taft S69E	ElCentro 1940 NS	Average
1	55	730	1910	.063	Intake Structure - Pseudo-soil												
2	40.5	675	1910	.063	Intake Structure - Pseudo-soil												
3	9.5	634.5	1200	.136	Sand	2781	2720	2751	.091	.093	.092	4063	3788	3926	.058	.065	.062
4	5	625	1200	.136	Sand	2676	2630	2653	.094	.095	.095	4011	3701	3856	.059	.067	.063
5		620	5000	.160	Rock												

NOTE:

Ground water table at El. 675

TABLE 2-13

STRAIN COMPATIBLE SOIL PROPERTIES - DBE

Gmax + 50% - Free Field - Elevation 735

Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values G(ksf)	Total Unit Wt (kcf)	Material	Shear Modulus (ksf) Taft S69E	ElCentro 1940 NS	Average	Damping Ratio Taft S69E	ElCentro 1940 NS	Average
1	10	735	2097	.125	Sand	1791	1794	1793	0.031	0.030	0.031
2	10	725	3726	.125	Sand	2870	2882	2876	0.043	0.042	0.043
3	10	715	5255	.125	Sand	3891	3947	3911	0.047	0.046	0.047
4	10	705	5255	.125	Sand	3586	3670	3628	0.056	0.053	0.055
5	5	695	7046	.125	Sand	4977	5096	5037	0.052	0.050	0.051
6	5	690	7046	.125	Sand	4838	4985	4912	0.055	0.052	0.054
7	10	685	7046	.125	Sand	4646	4843	4745	0.059	0.055	0.057
8	10	675	7667	.136	Sand	4940	5159	5050	0.062	0.057	0.061
9	10	665	9123	.136	Sand	6054	6195	6125	0.058	0.056	0.057
10	10	655	9123	.136	Sand	5798	5973	5886	0.063	0.060	0.062
11	10	645	9123	.136	Sand	5604	5749	5677	0.066	0.064	0.065
12	10	635	9123	.136	Sand	5484	5566	5525	0.068	0.067	0.068
13	5	625	9123	.136	Sand	5394	5452	5423	0.070	0.069	0.070
14		620			Rock						

NOTE:

Ground water table at El. 675

TABLE 2-14

STRAIN COMPATIBLE SOIL PROPERTIES - DBE

Gmax Minus 50% - Free Field - Elevation 735

Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values G(ksf)	Total Unit Wt (kcf)	Material	Shear Modulus (ksf) Taft S69E	ElCentro 1940 NS	Average	Damping Ratio Taft S69E	ElCentro 1940 NS	Average
1	10	735	699	.125	Sand	450	454	452	.062	.061	.062
2	10	725	1242	.125	Sand	605	653	629	.086	.080	.083
3	10	715	1752	.125	Sand	796	883	840	.092	.084	.088
4	10	705	1752	.125	Sand	684	722	703	.109	.102	.106
5	5	695	2349	.125	Sand	1028	1069	1049	.094	.092	.093
6	5	690	2349	.125	Sand	988	1014	1001	.099	.096	.098
7	10	685	2349	.125	Sand	960	968	964	.103	.102	.103
8	10	675	2556	.136	Sand	1073	1070	1072	.099	.100	.100
9	10	665	3041	.136	Sand	1436	1422	1429	.089	.090	.090
10	10	655	3041	.136	Sand	1442	1435	1439	.089	.089	.089
11	10	645	3041	.136	Sand	1482	1470	1476	.086	.087	.087
12	10	635	3041	.136	Sand	1475	1494	1485	.087	.086	.087
13	5	625	3041	.136	Sand	1411	1494	1453	.090	.086	.088
14		620			Rock						

NOTE:

Ground water table at El. 675

TABLE 2-15

STRAIN COMPATIBLE SOIL PROPERTIES - DBE

Gmax Plus 50% - Reactor Building

Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values G (ksf)	Total Unit Wt (kcf)	Material	Shear Modulus (ksf) Taft S69E	Shear Modulus (ksf) ElCentro 1940 NS	Shear Modulus (ksf) Average	Damping Ratio Taft S69E	Damping Ratio ElCentro 1940 NS	Damping Ratio Average
1	10	735	5101	.138	Reactor Building Pseudo-soil						
2	10	725	5101	.138	Reactor Building Pseudo-soil						
3	10	715	5101	.138	Reactor Building Pseudo-soil						
4	10	705	5101	.138	Reactor Building Pseudo-soil						
5	10	695	5101	.138	Reactor Building Pseudo-soil						
6	4	685	5101	.138	Reactor Building Pseudo-soil						
7	6	681	7046	.125	Sand	4215	4103	4159	.069	.071	.070
8	10	675	7667	.136	Sand	4564	4435	4500	.069	.072	.071
9	10	665	9123	.136	Sand	5658	5487	5573	.065	.068	.067
10	10	655	9123	.136	Sand	5380	5231	5306	.070	.073	.072
11	10	645	9123	.136	Sand	5171	5019	5095	.074	.076	.075

TABLE 2-15 (Cont)

Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values G (ksf)	Total Unit Wt (kcf)	Material	Shear Modulus (ksf) Taft S69E	Shear Modulus (ksf) ElCentro 1940 NS	Shear Modulus (ksf) Average	Damping Ratio Taft S69E	Damping Ratio ElCentro 1940 NS	Damping Ratio Average
12	10	635	9123	.136	Sand	5058	4850	4954	.076	.079	.078
13	5	625	9123	.136	Sand	4943	4743	4843	.078	.081	.080
14		620		.160	Rock						

NOTE:

Ground water table at El. 675

TABLE 2-16

STRAIN COMPATIBLE SOIL PROPERTIES - DBE

Gmax Minus 50% - Reactor Building

Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values G (ksf)	Total Unit Wt (kcf)	Material	Shear Modulus (ksf) Taft S69E	Shear Modulus (ksf) ElCentro 1940 NS	Shear Modulus (ksf) Average	Damping Ratio Taft S69E	Damping Ratio ElCentro 1940 NS	Damping Ratio Average
1	10	735	5101	.138	Reactor Building Pseudo-soil						
2	10	725	5101	.138	Reactor Building Pseudo-soil						
3	10	715	5101	.138	Reactor Building Pseudo-soil						
4	10	705	5101	.138	Reactor Building Pseudo-soil						
5	10	695	5101	.138	Reactor Building Pseudo-soil						
6	4	685	5101	.138	Reactor Building Pseudo-soil						
7	6	681	2349	.125	Sand	859	972	916	.116	.101	.109
8	10	675	2556	.136	Sand	972	1058	1015	.112	.101	.107
9	10	665	3041	.136	Sand	1290	1341	1316	.098	.094	.096
10	10	655	3041	.136	Sand	1332	1286	1309	.094	.098	.096
11	10	645	3041	.136	Sand	1334	1250	1292	.094	.102	.098

TABLE 2-16 (Cont)

Layer No.	Thickness (ft)	Top of Layer Elev.	Low Strain Values G (ksf)	Total Unit Wt (kcf)	Material	Shear Modulus (ksf) Taft S69E	ElCentro 1940 NS	Average	Damping Ratio Taft S69E	ElCentro 1940 NS	Average
12	10	635	3041	.136	Sand	1312	1241	1277	.096	.103	.100
13	5	625	3041	.136	Sand	1268	1245	1257	.100	.103	.102
14		620			Rock						

NOTE:

Ground water table at El. 675

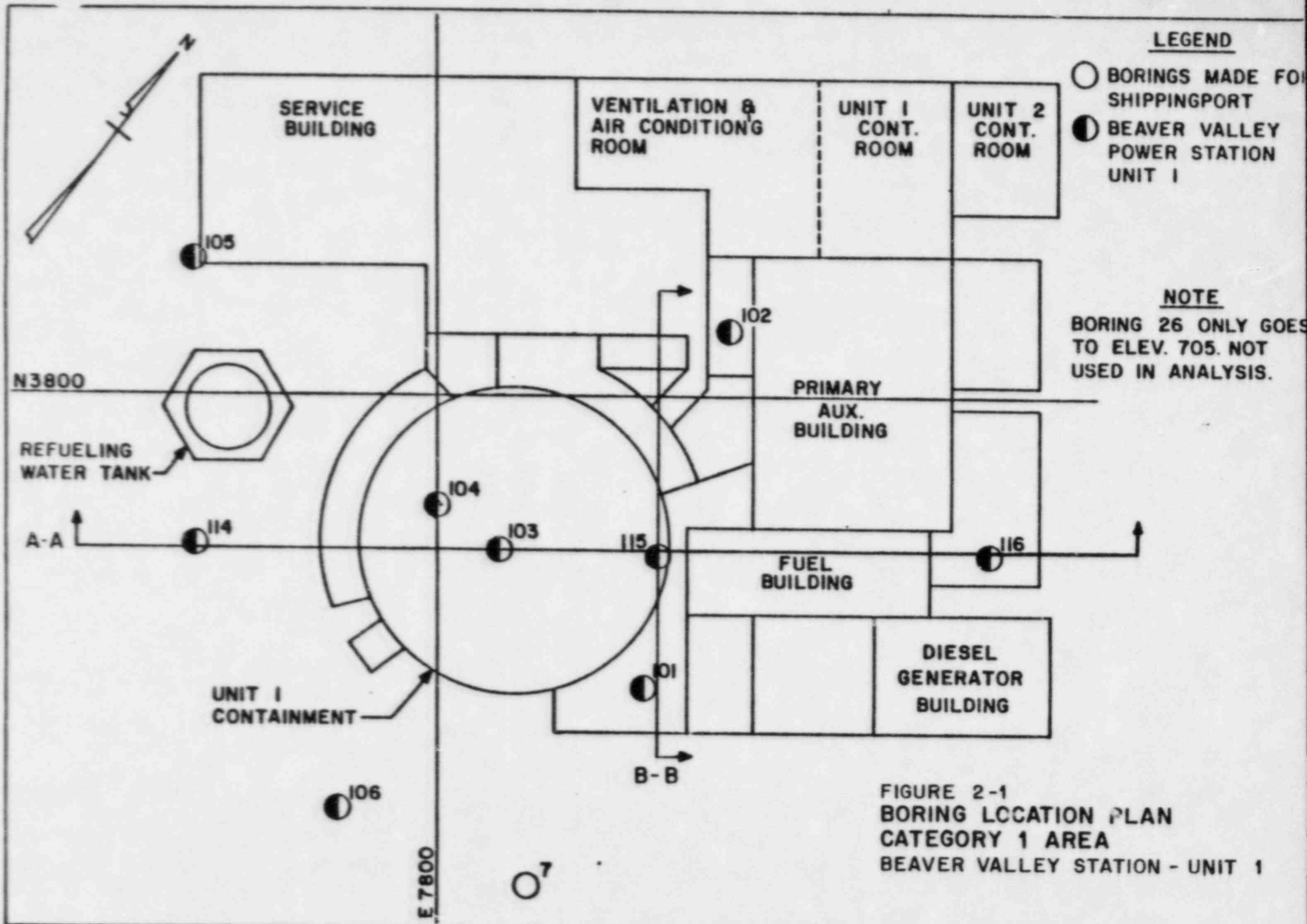


FIGURE 2-1
BORING LOCATION PLAN
CATEGORY 1 AREA
BEAVER VALLEY STATION - UNIT 1

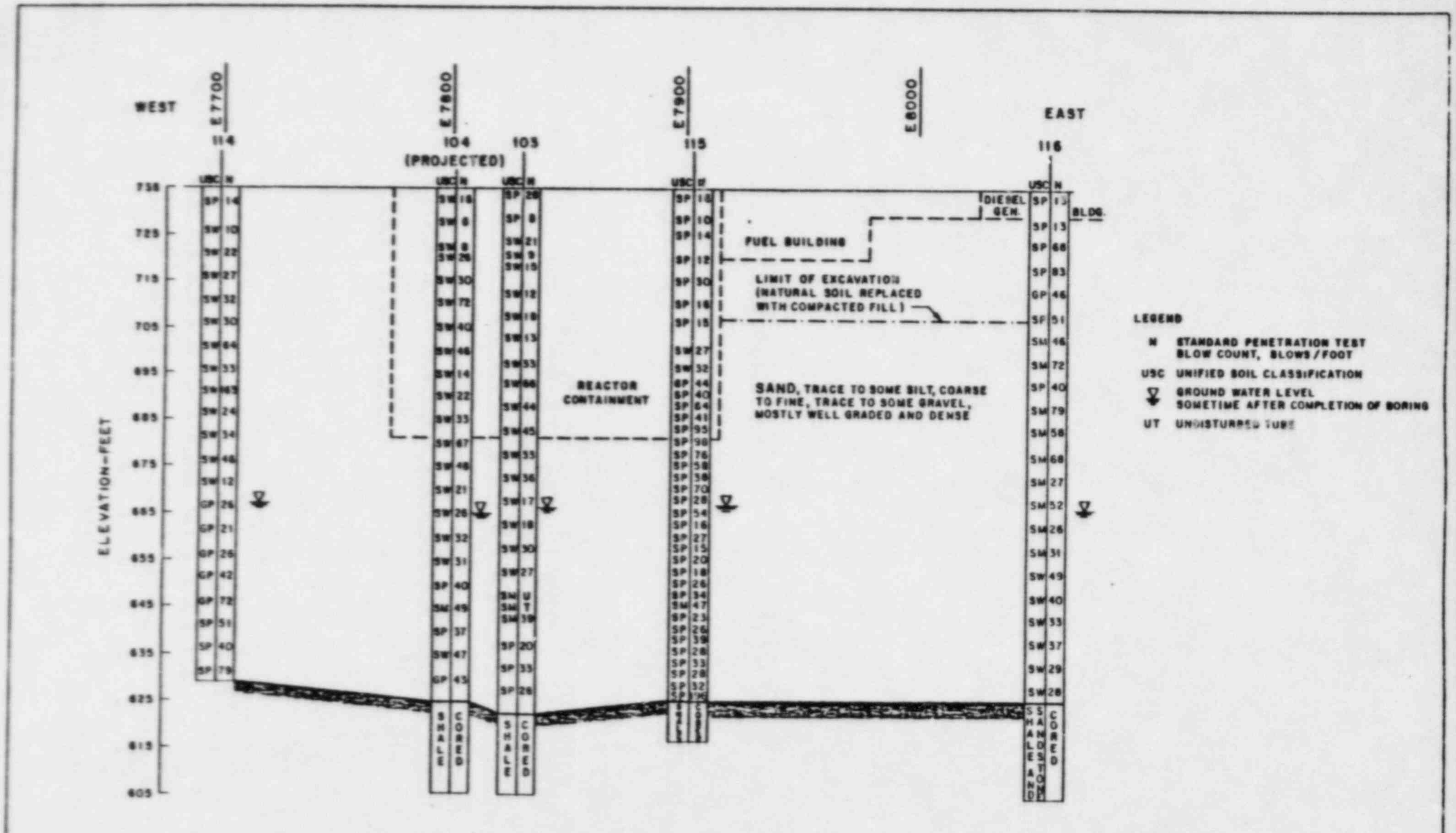


FIGURE 2-2
EAST-WEST SOIL PROFILE
SECTION 1-1
BEAVER VALLEY POWER STATION-UNIT 1

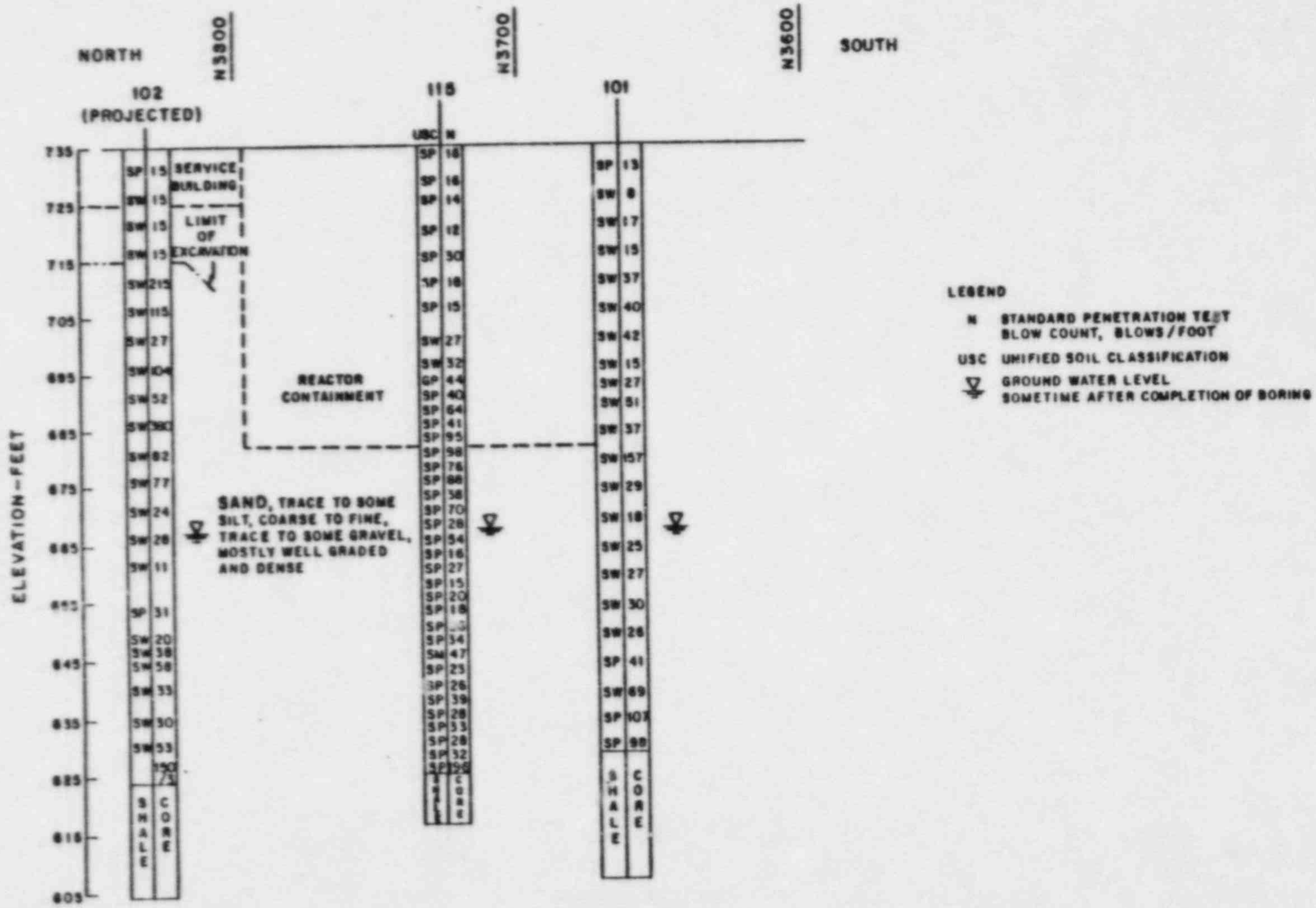
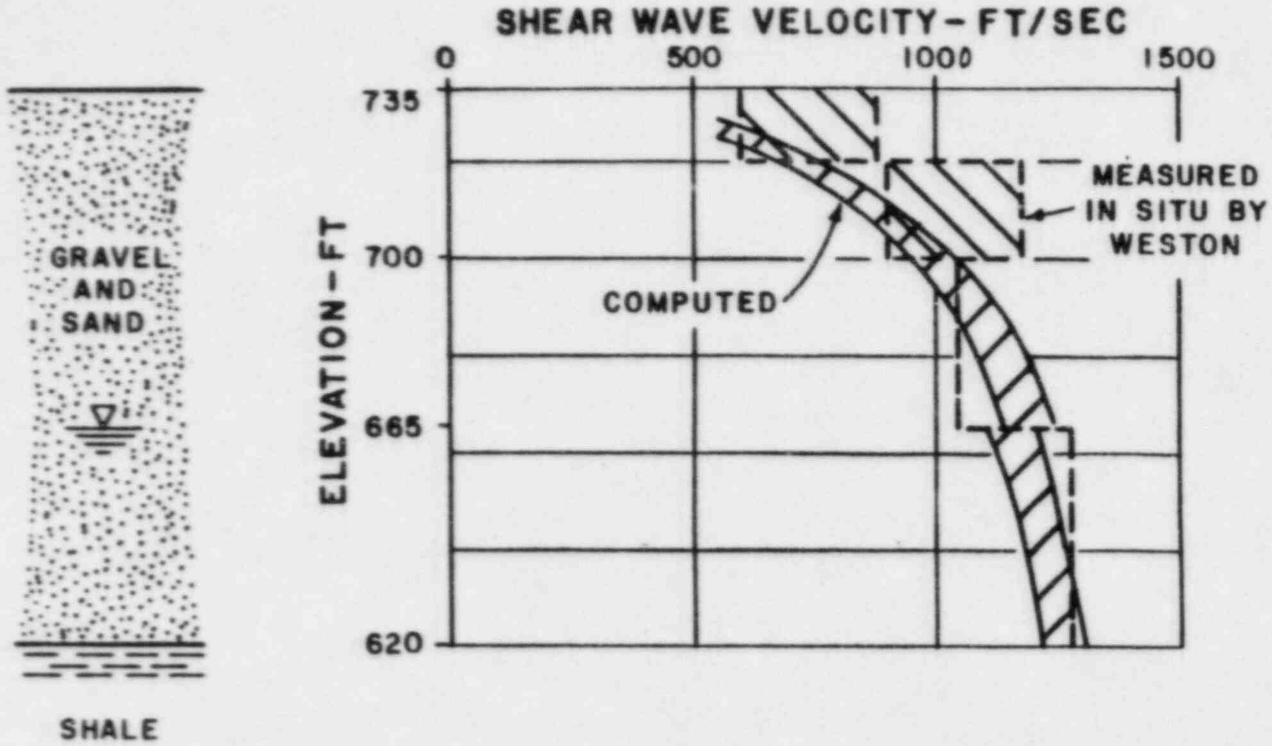


FIGURE 2-3
NORTH-SOUTH SOIL PROFILE
SECTION 2-2
BEAVER VALLEY POWER STATION-UNIT 1



VALUES ASSUMED FOR COMPUTING C_S

$e = 0.58$	$e = 0.46$
$S = 50\%$ $\gamma_t = 120$ pcf	$S = 50\%$ $\gamma_t = 125$ pcf
$S = 100\%$ $\gamma_t = 130$ pcf	$S = 100\%$ $\gamma_t = 135$ pcf
$G_s = 2.70$	

FIGURE 2-4
MEASURED AND COMPUTED VALUES OF SHEAR WAVE VELOCITY
BEAVER VALLEY POWER STATION - UNIT 1

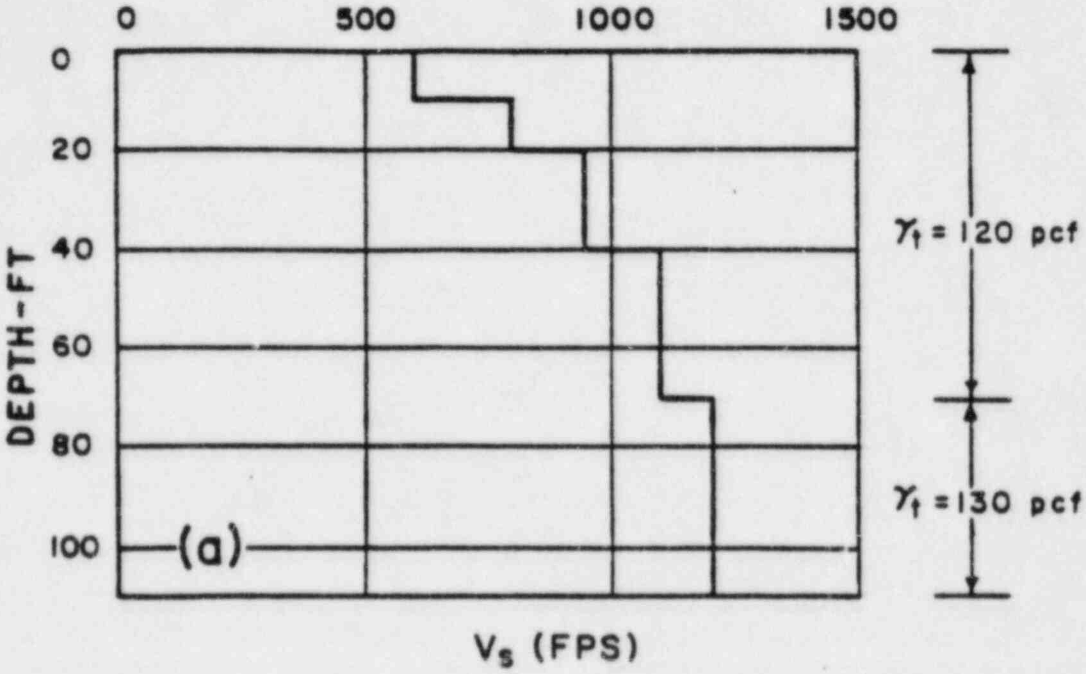


FIGURE 2-5
PROPERTIES USED FOR
WHITMAN'S ANALYSIS
BEAVER VALLEY POWER STATION-UNIT 1

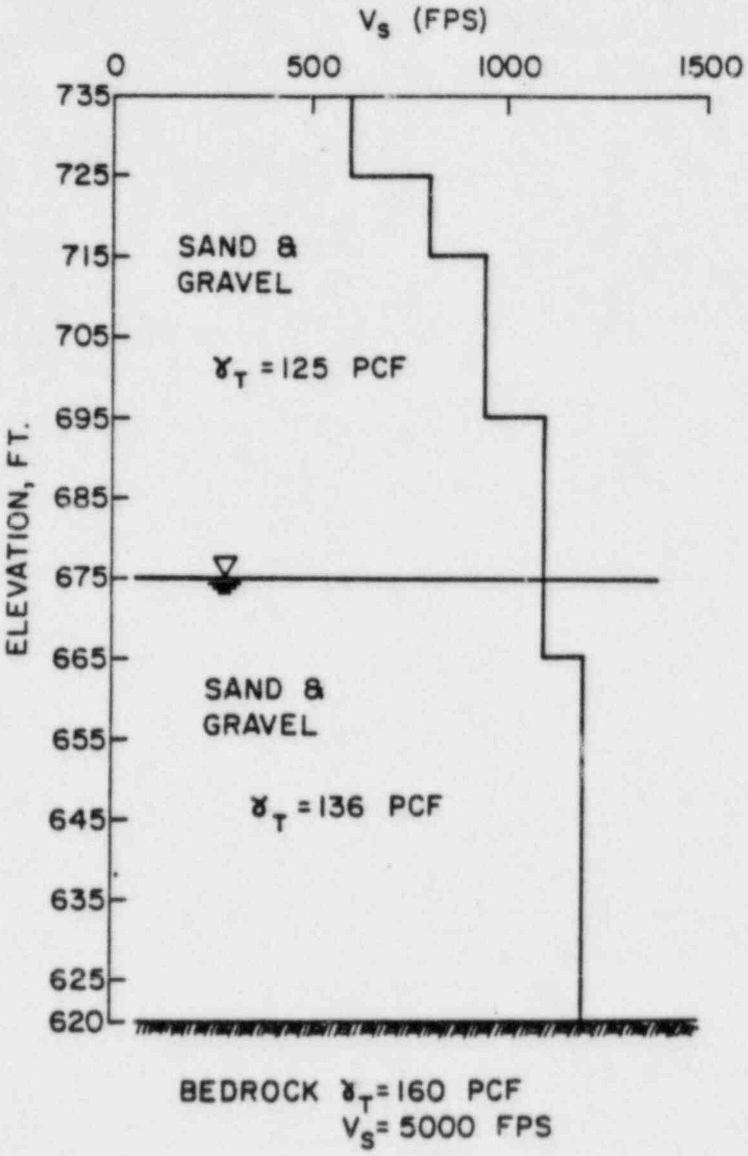


FIGURE 2-6
FREE FIELD SOIL PROFILE
BEAVER VALLEY POWER STATION-UNIT 1

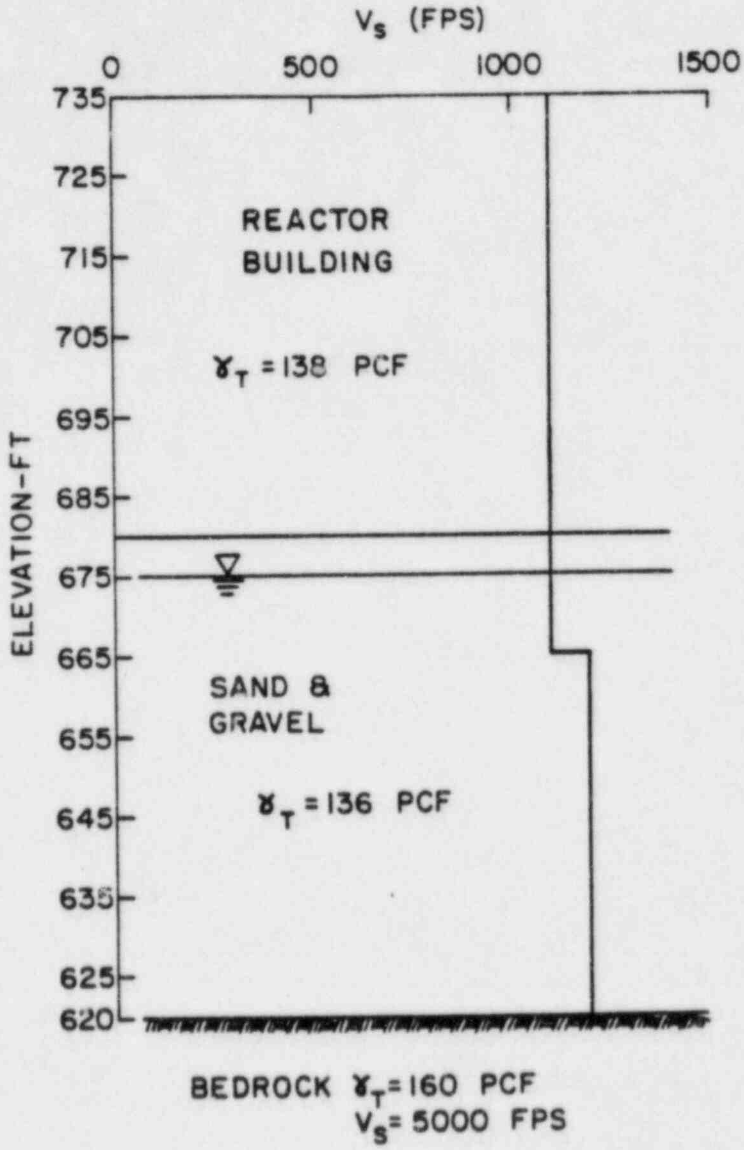


FIGURE 2-7
REACTOR BUILDING SOIL PROFILE
BEAVER VALLEY POWER STATION-UNIT 1

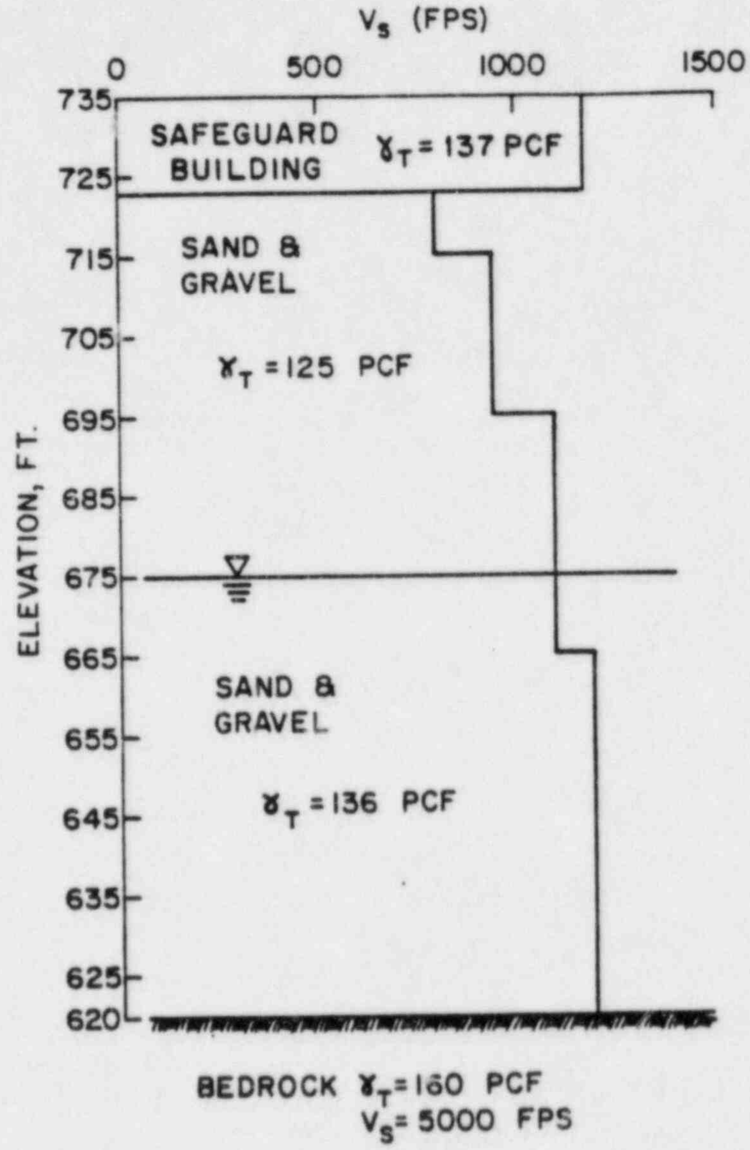


FIGURE 2-8
SAFEGUARD BUILDING SOIL PROFILE
BEAVER VALLEY POWER STATION-UNIT 1

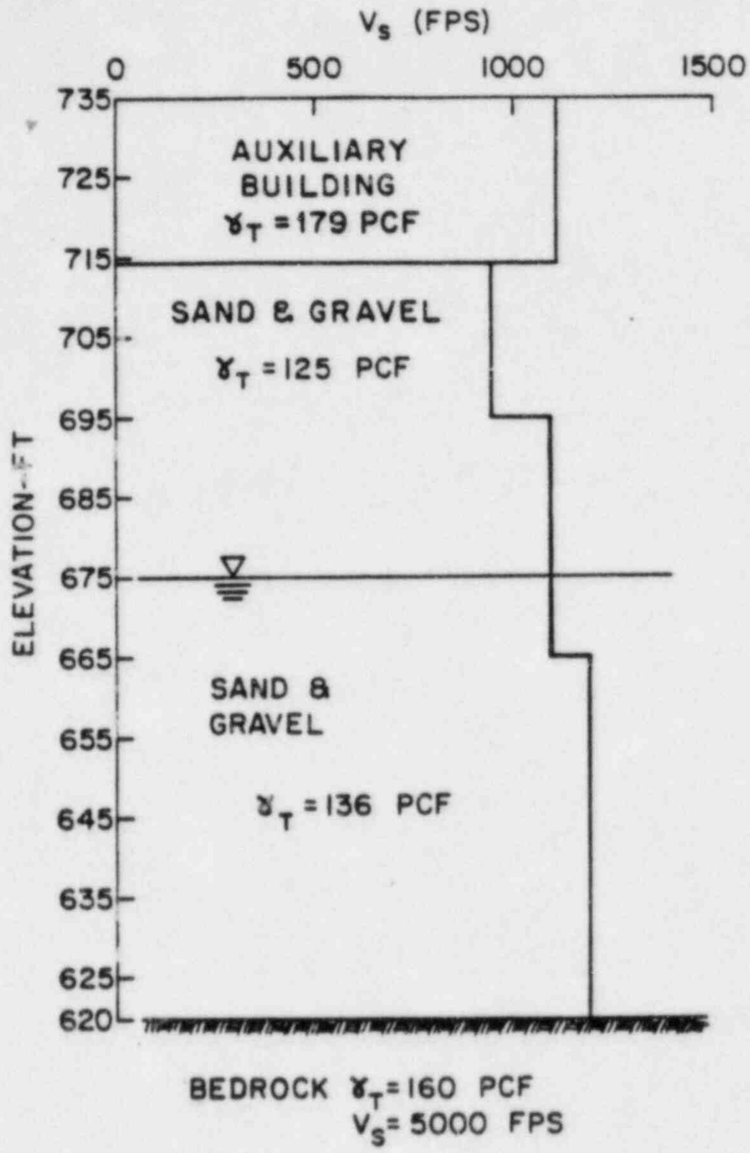


FIGURE 2-9
AUXILIARY BUILDING SOIL PROFILE
BEAVER VALLEY POWER STATION-UNIT 1

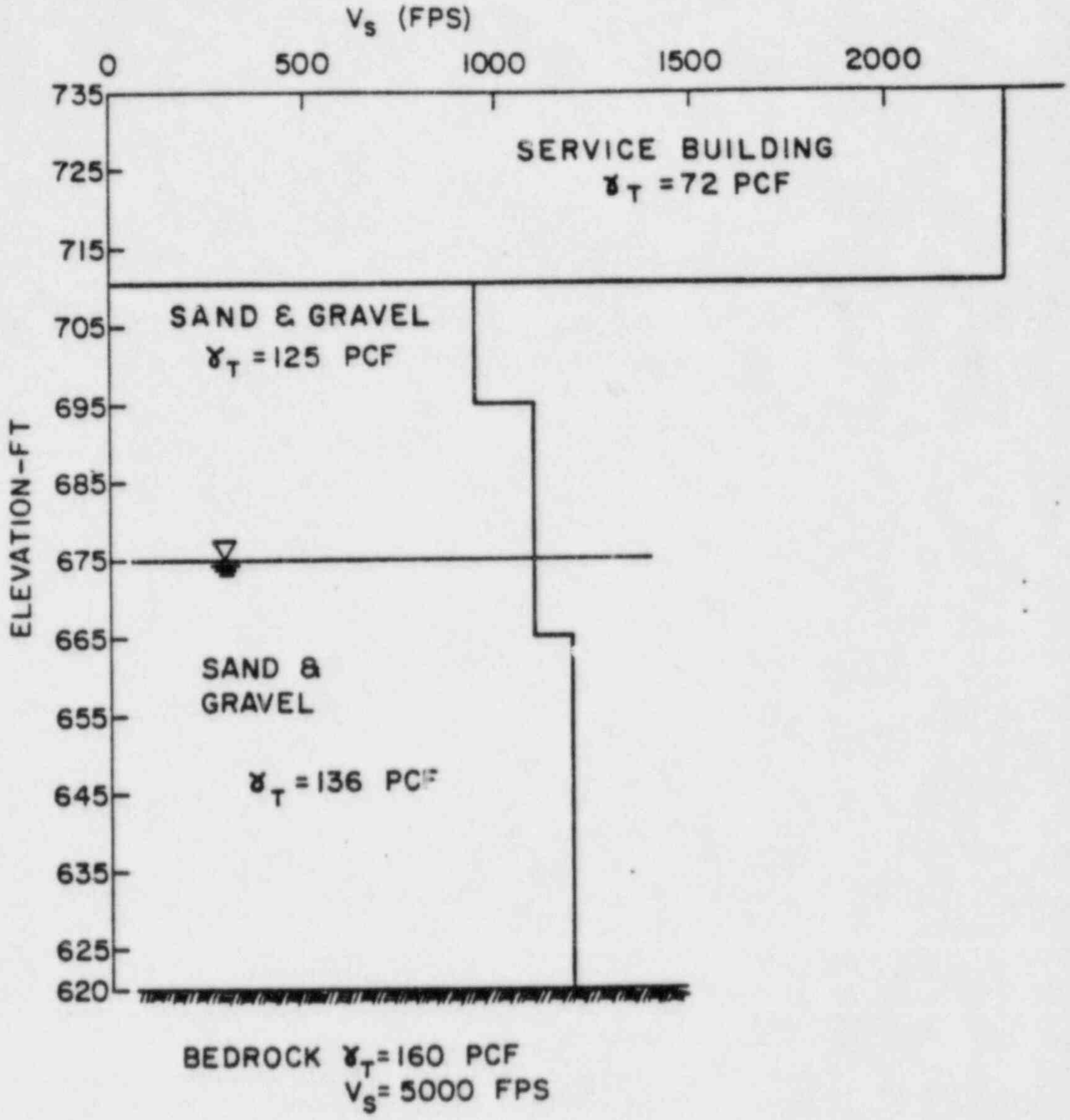


FIGURE 2-10
SERVICE BUILDING SOIL PROFILE
BEAVER VALLEY POWER STATION-UNIT 1

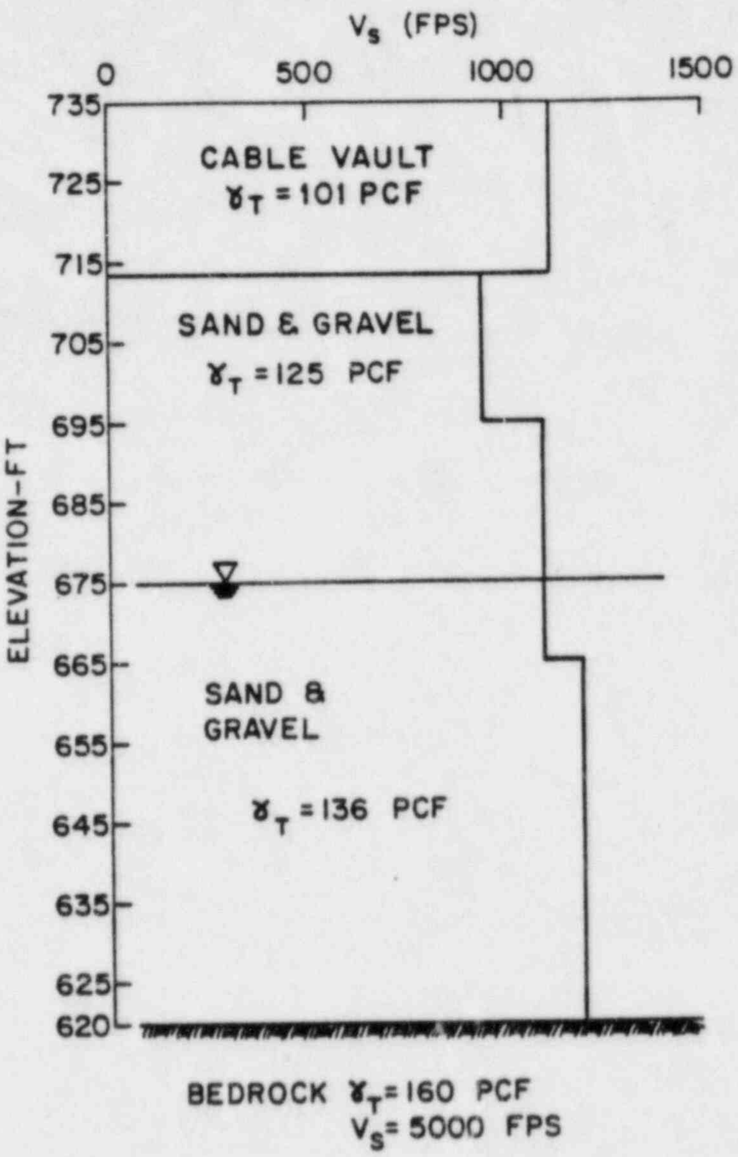


FIGURE 2-11
CABLE VAULT SOIL PROFILE
BEAVER VALLEY POWER STATION-UNIT 1

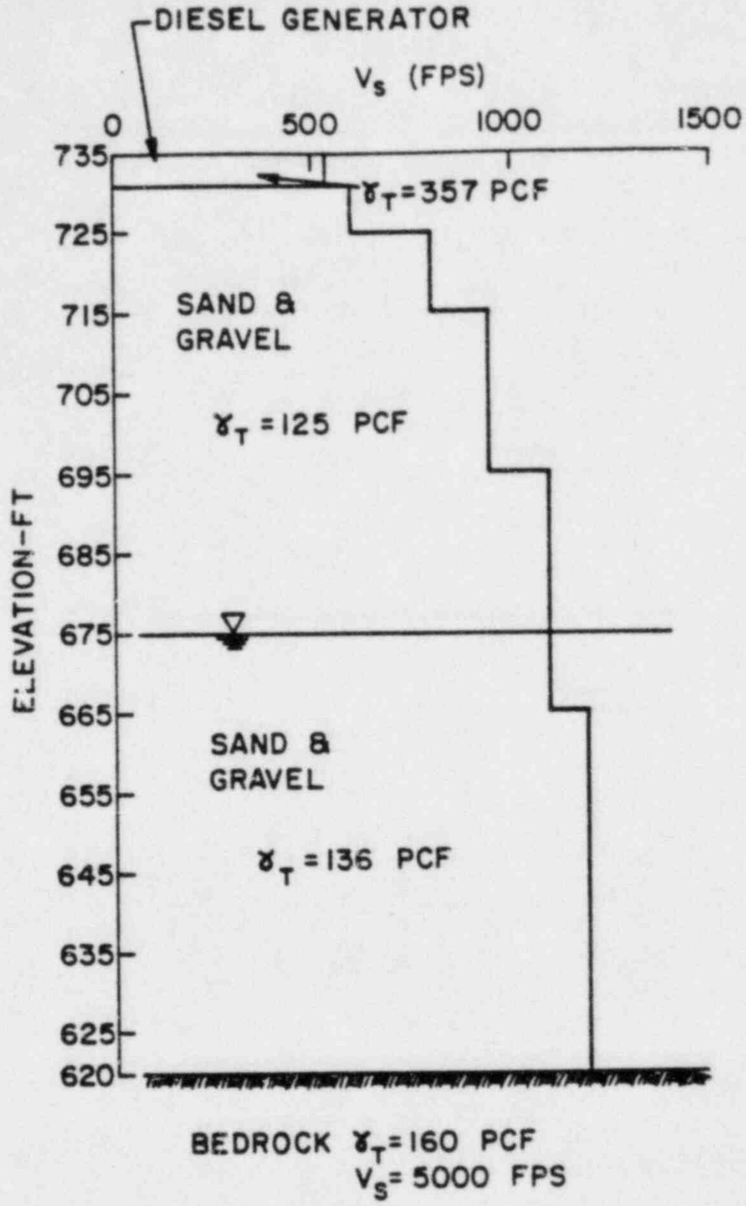


FIGURE 2-12
DIESEL GENERATOR BUILDING
SOIL PROFILE
BEAVER VALLEY POWER STATION-UNIT 1

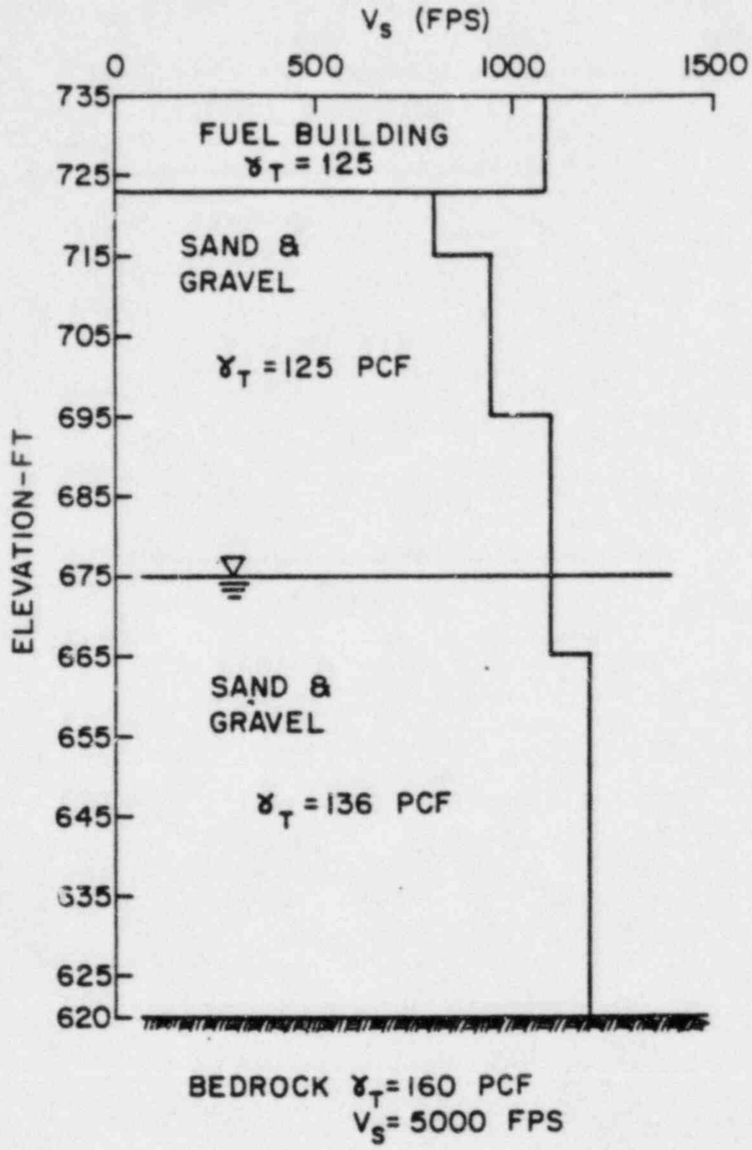


FIGURE 2-13
FUEL BUILDING SOIL PROFILE
BEAVER VALLEY POWER STATION-UNIT 1

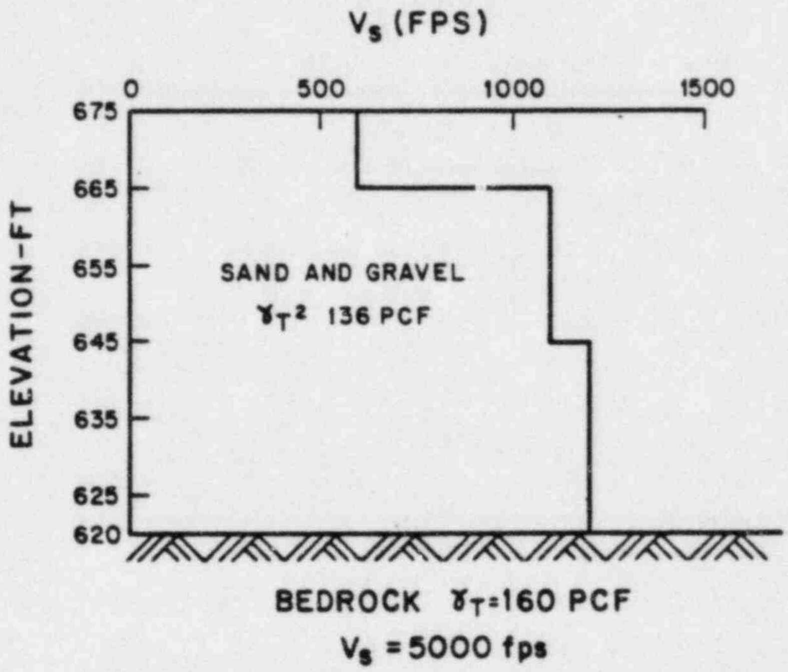


FIGURE 2-14
FREE FIELD SOIL PROFILE
SOUTH OF INTAKE STRUCTURE
BEAVER VALLEY POWER STATION-UNIT 1

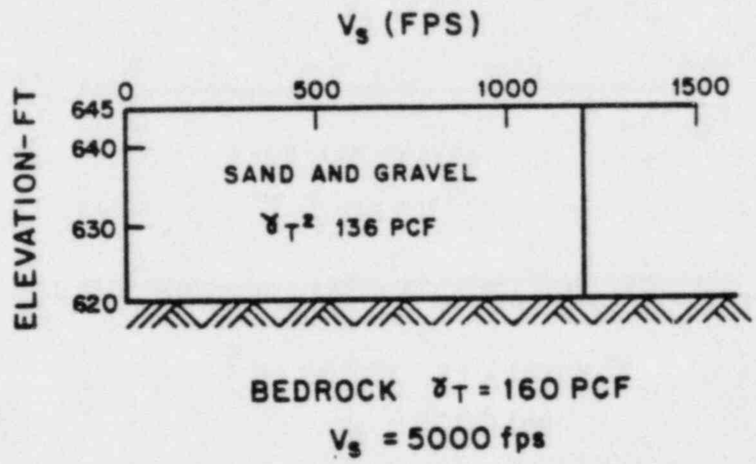


FIGURE 2-15
FREE FIELD SOIL PROFILE
NORTH OF INTAKE STRUCTURE
BEAVER VALLEY POWER STATION-UNIT 1

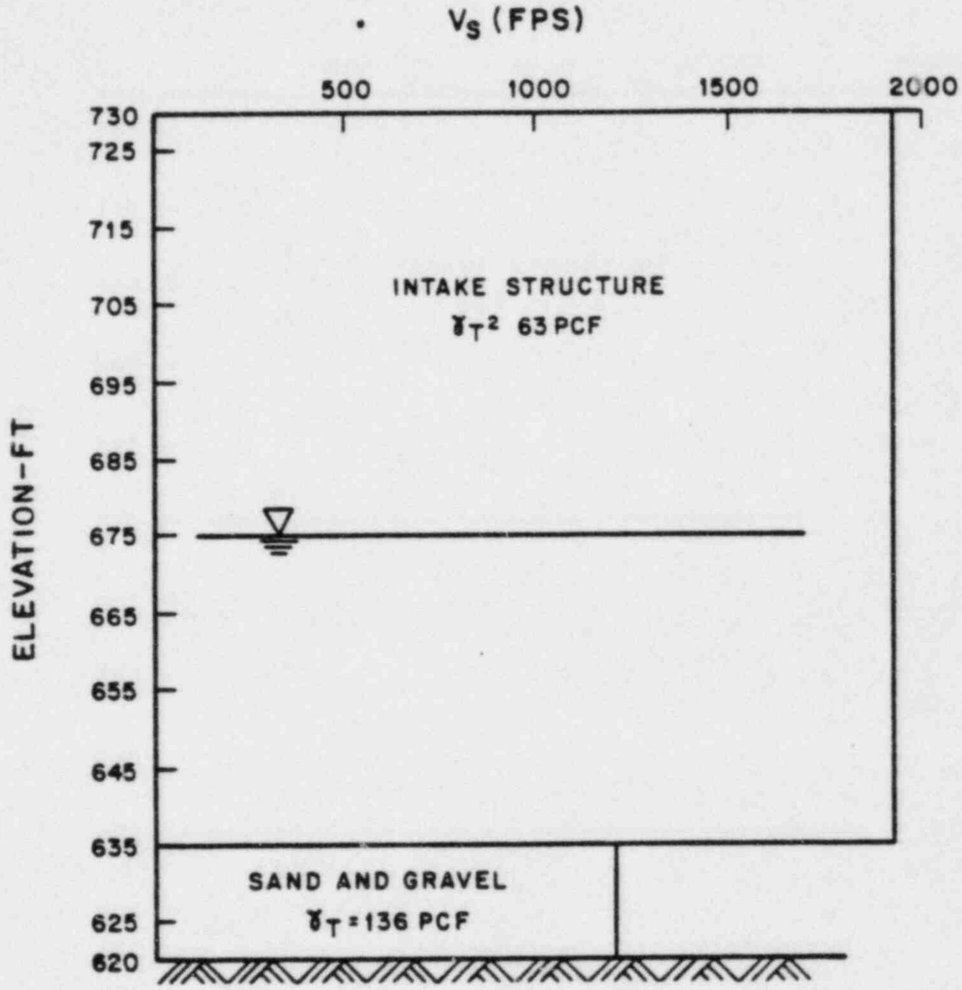


FIGURE 2-16
INTAKE STRUCTURE SOIL PROFILE
BEAVER VALLEY POWER STATION-UNIT 1

3.0 GROUND RESPONSE

The selection of seismic design parameters has been discussed in detail in the Beaver Valley Unit 1 FSAR. This section discribes the smoothed ground response spectra.

3.1 DESIGN BASIS EARTHQUAKE (DBE) AND OPERATIONAL BASIS EARTHQUAKE (OBE)

The design basis earthquake (DBE) for the Beaver Valley site has a peak acceleration of 0.125g at the ground surface elevation of 735 feet. This acceleration level was established by considering an intensity V to VI earthquake with peak bedrock acceleration of 0.035g amplified from bedrock elevation 620 feet through the overlying soil to elevation 735 feet. The amplification factor is 3.5. Smoothed response spectra were then normalized to the amplified acceleration.

Accordingly, the design is based on a DBE normalized to 0.125 g at the ground surface (El. 735) and for the OBE normalized to 0.06 g at the same elevation. Vertical accelerations are taken as two-thirds of the horizontal accelerations.

3.2 GROUND RESPONSE SPECTRA

The ground response spectra shown in Figure 3-1 (DBE) and Figure 3-2 (OBE) are the bases for the design of all ground supported structures, equipment, and piping. The design is based on a DBE normalized to 0.125 g and for the OBE normalized to 0.06 g. Dynamic amplification factors used for these spectra are such as to give a maximum spectral acceleration of 0.45 g for two percent damping for the DBE with appropriate relative values for other amounts of damping. The spectra are flat from 2 to 5 Hz (0.2 to 0.5 sec period) and reduce to an amplification ratio of unity for frequency exceeding 20 Hz. Amplified response spectra are used for the design of equipment, piping, and instrumentation supported from structures.

3.3 ARTIFICIAL TIME HISTORY

The artificial time history has a total duration of 15 seconds, with about 3.5 seconds each of rise and fall time, whose ground response spectra are forced to fit the specified site spectrum. An artificial accelerogram which reproduces the frequency content displayed either in a response spectrum or in a power spectral density function is simulated statistically by using a stochastic model as described in Reference 1. In this model, the earthquake motion is considered to be a wide-band stationary process whose spectral density function, duration, and maximum acceleration are specified. The

artificial motion is generated by matching the target or site spectrum for several specified percentages of critical damping at 125 oscillator periods distributed from 0.0204 (49 Hz) to 5.0 (0.2 Hz) seconds. For a detailed treatment of the modeling procedure, see References 2 and 3.

The acceleration time history yields ground response spectra at damping values of 0.5, 1, 2, 5, 7, and 10 percent that envelop the smoothed site design ground response spectra for those damping values (see Figure 3-3, for example).

3.4 GROUND RESPONSE SPECTRA AT BASE OF CONTAINMENT

The ground response spectra at the base of the reactor containment structure were calculated and plotted using SHAKE. The artificial earthquake developed for the Beaver Valley site was normalized to the DBE maximum acceleration of 0.125 g and input at the ground surface of the free-field profile. The earthquake motion was deconvoluted to the base of the profile and the computed motion at the El 681 feet, the containment founding grade, was used to compute the real velocity and acceleration response spectra and the tripartite plot of real displacement, pseudovelocity, and pseudoacceleration vs. frequency. These spectra are plotted for damping ratios of .5, 1.0, and 3.0 percent.

Response spectra were calculated for three soil profiles, represented by the shear modulus (Gmax) calculated from seismic cross-hole surveys, Gmax plus 50 percent, and Gmax minus 50 percent. The spectra for each soil profile are plotted on Figures 3-4, 3-5, and 3-6, respectively. Also plotted on these figures is the ground response spectrum for .5 percent damping presented in the Beaver Valley Unit 1 FSAR.

3.5 REFERENCES

1. Hou, S.N., Earthquake Simulation Models and their Applications. Research Report R68-17, Department of Civil Engineering, MIT, 1968.

2. Rascon, O.A. and Cornell, C.A., Strong Motion Earthquake Simulation. Research Report R68-15, Department of Civil Engineering, MIT, 1968.

3. Tsai, N.C., Spectrum Compatible Motions for Design Purposes. Journal of Engineering Mechanics Division, ASCE, Vol 98, No. EM2, Rev. 4, Paper 8807, April 1972, p 345-356.

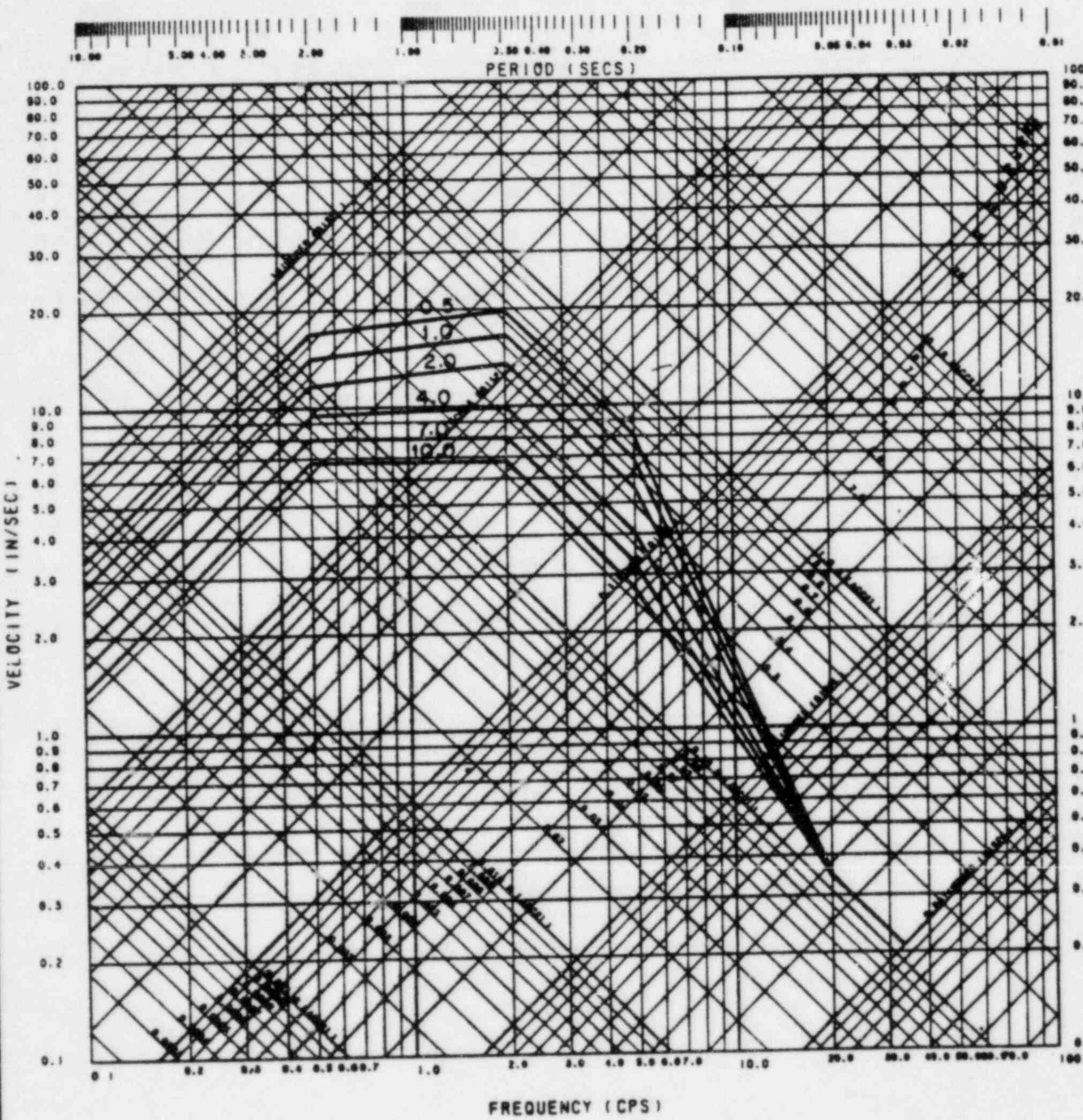


FIGURE 3-1
RESPONSE SPECTRA 0.125 G DBE
BEAVER VALLEY POWER STATION - UNIT 1

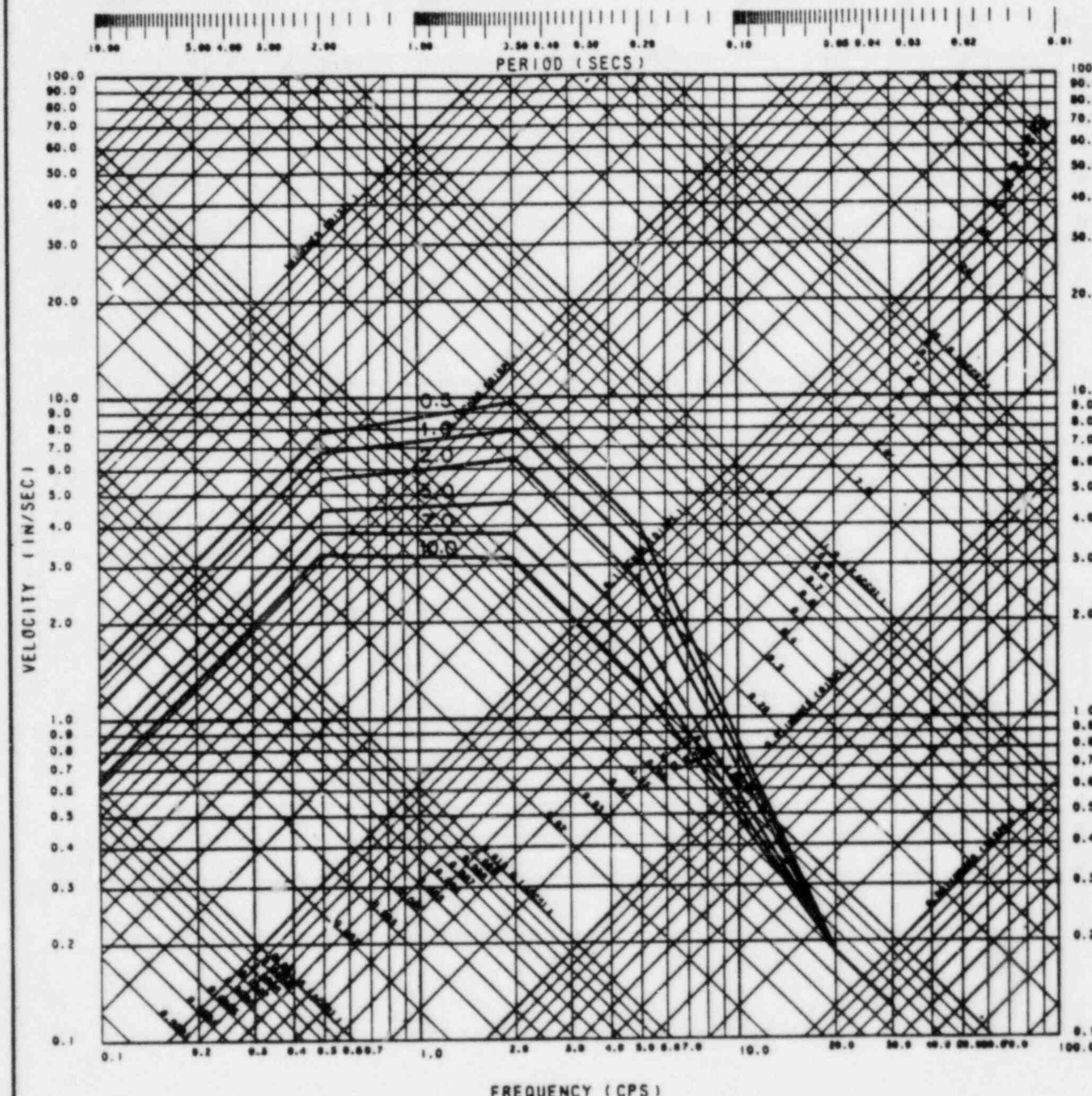


FIGURE 3-2
RESPONSE SPECTRA 0.06G OBE
BEAVER VALLEY POWER STATION-UNIT 1

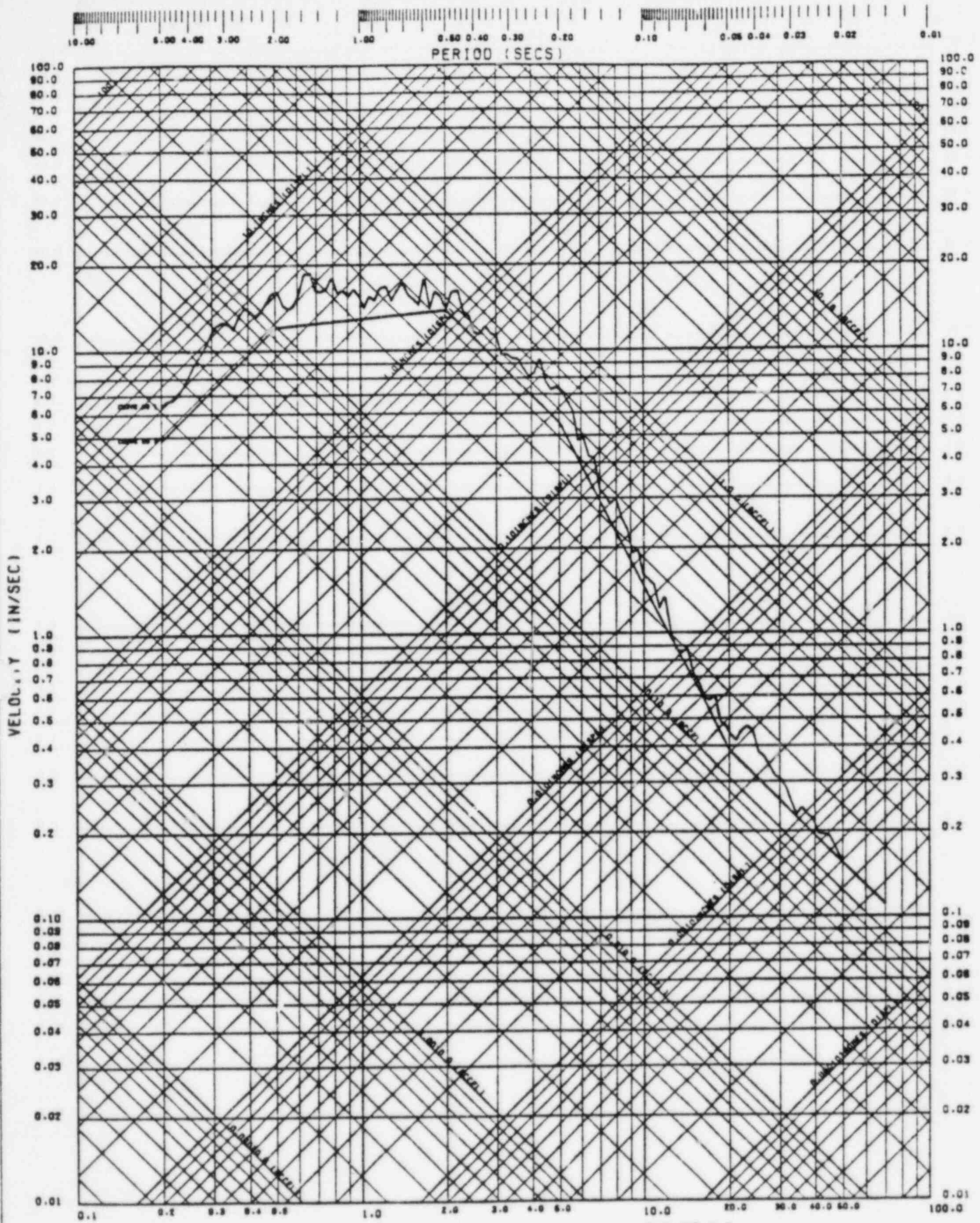
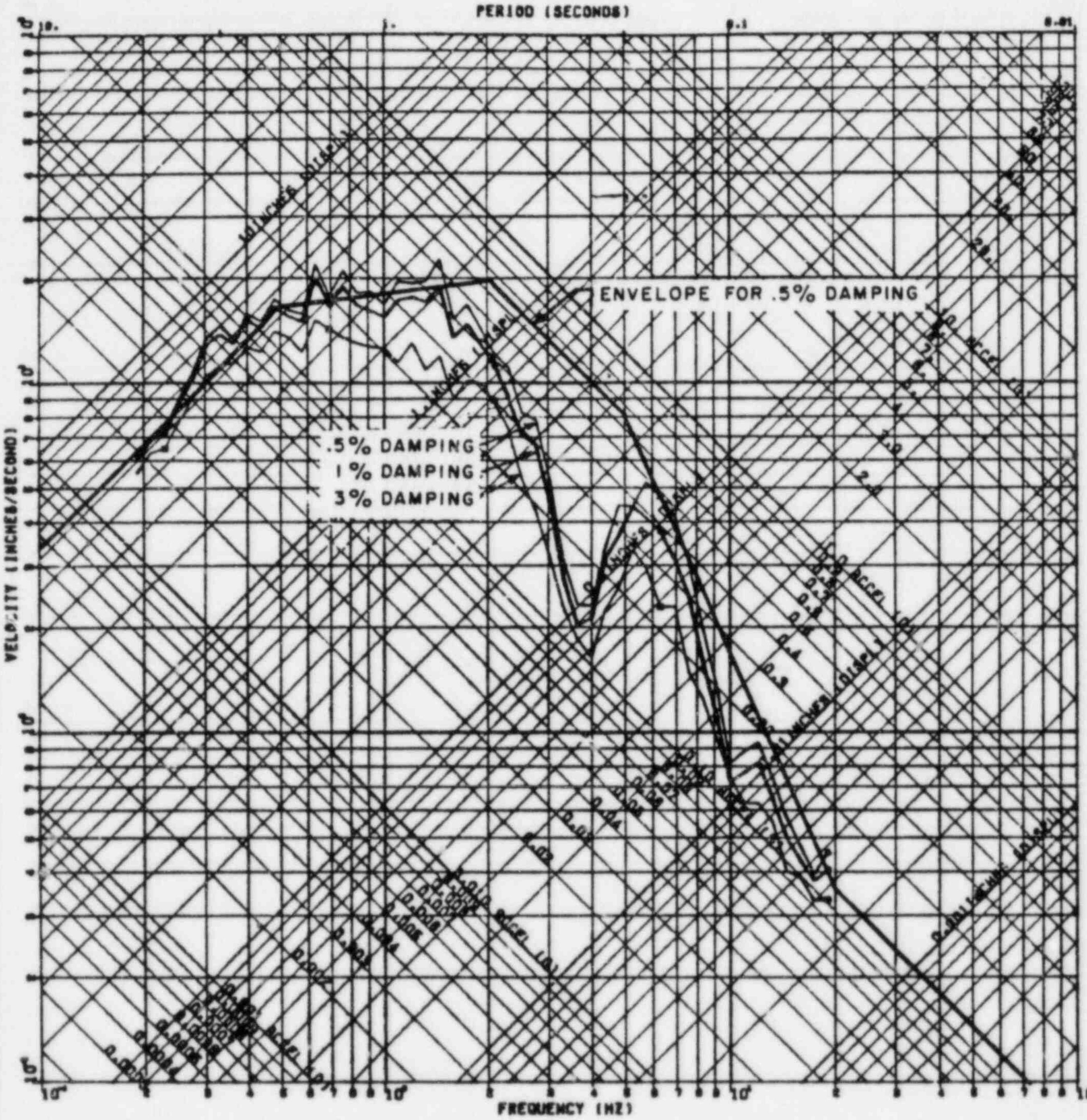


FIGURE 3-5
RESPONSE SPECTRA
ARTIFICIAL EARTHQUAKE-2% DAMPING
BEAVER VALLEY POWER STATION-UNIT 1

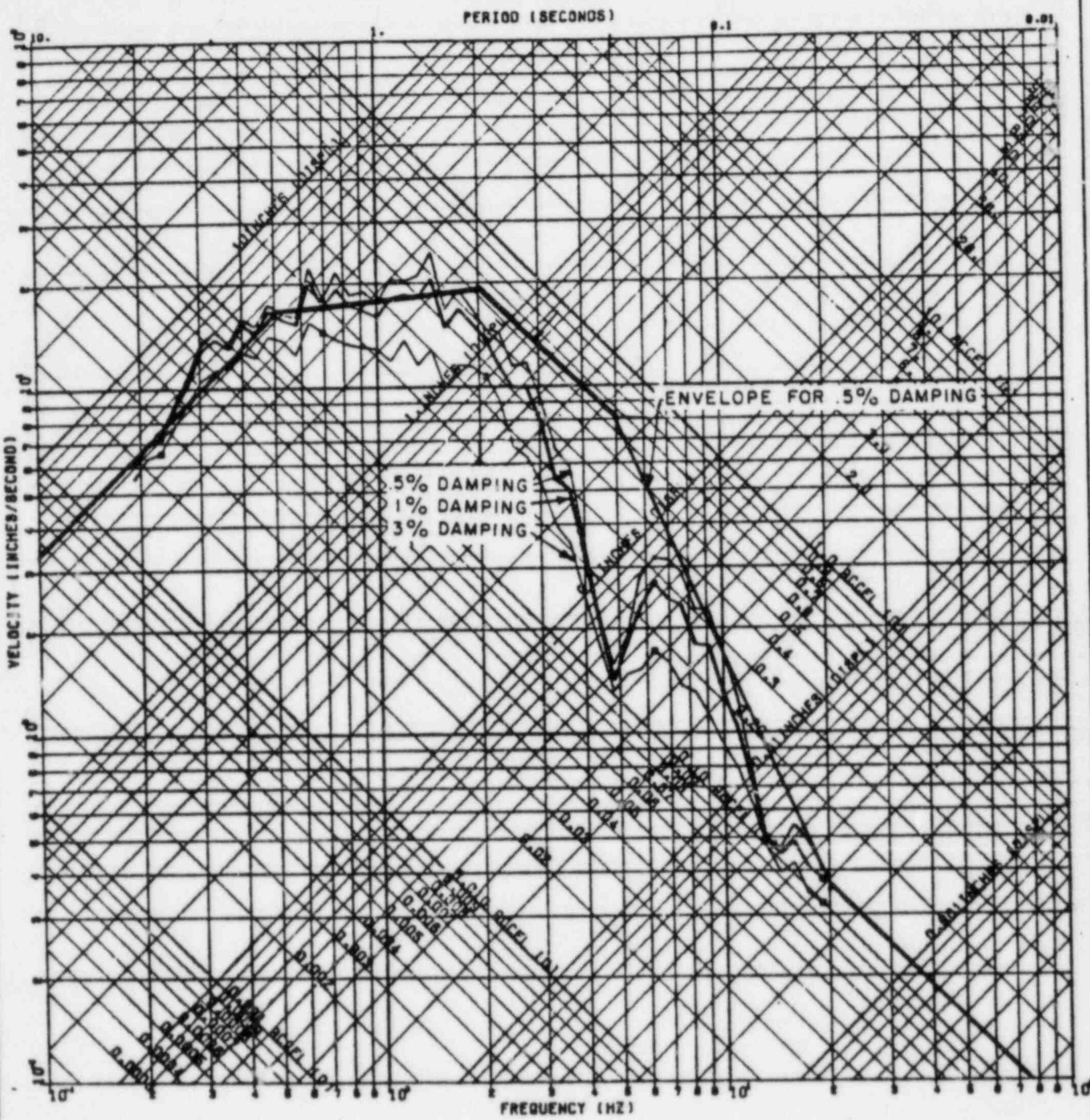
RUN NUMBER ARTIFICIAL EARTH



ARTIFICIAL EARTHQUAKE SSE BVPS FREE FIELD 735-620 DECONVOLUT
SPECTRA FOR TOP OF LAYER 8
DAMPING VALUES
0.030
0.010
0.005

FIGURE 3-4
GROUND RESPONSE SPECTRA
AVERAGE Gmax
BEAVER VALLEY POWER STATION-UNIT I

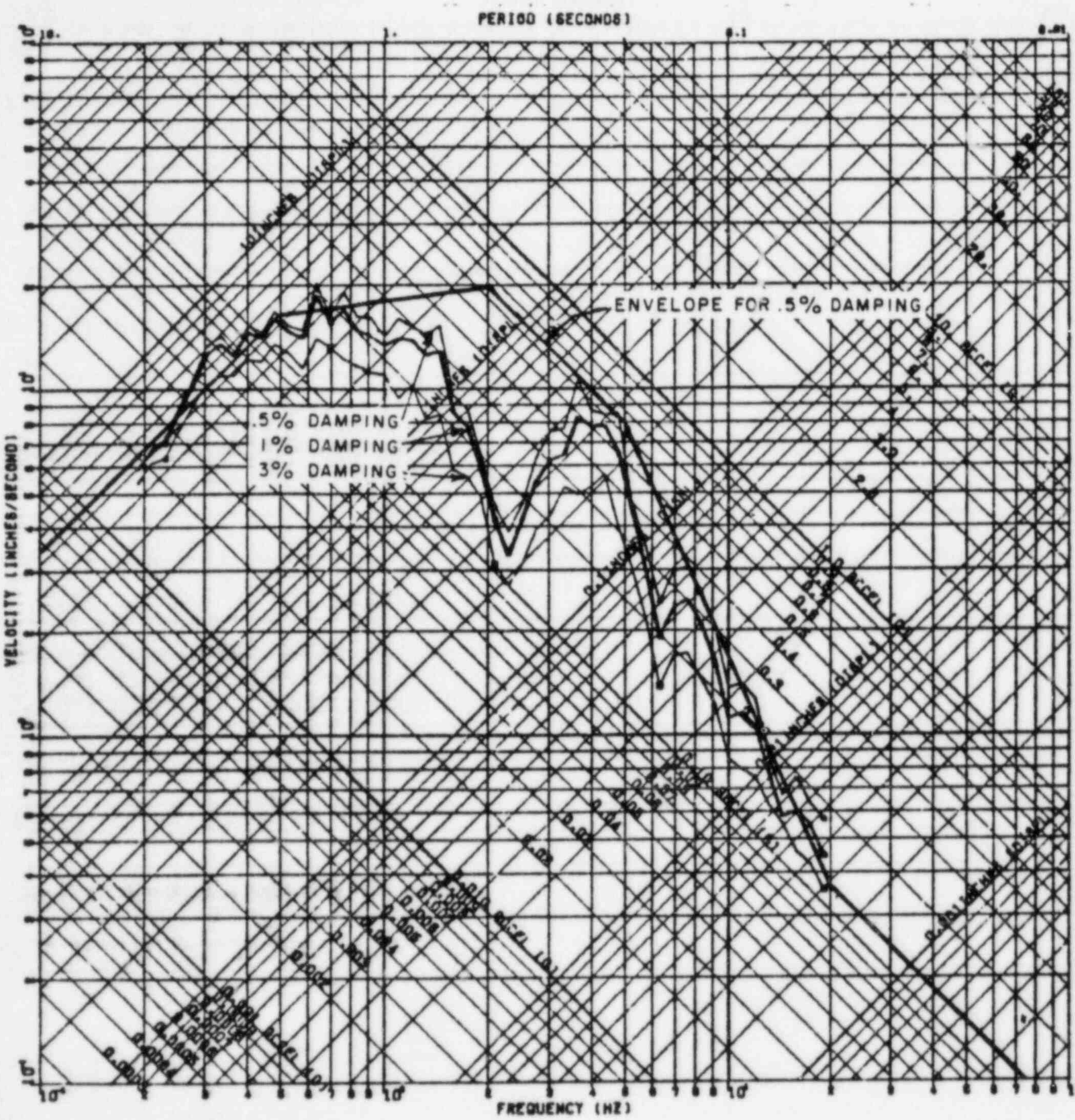
RUN NUMBER ARTIFICIAL EARTH



ARTIFICIAL EARTHQUAKE SSE BVPS FREE FIELD 735-620 DECONVOLUT
SPECTRA FOR TOP OF LAYER 6
DAMPING VALUES
⊠ 0.030
⊕ 0.010
▲ 0.005

FIGURE 3-5
GROUND RESPONSE SPECTRA
AVERAGE Gmax + 50%
BEAVER VALLEY POWER STATION-UNIT I

RUN NUMBER ARTIFICIAL EARTH



ARTIFICIAL EARTHQUAKE SSE BVPS FREE FIELD 735-620 DECONVOLUT
SPECTRA FOR TOP OF LAYER 8
DAMPING VALUES
□ 0.030
○ 0.010
▲ 0.005

FIGURE 3-6
GROUP RESPONSE
SPECTRA AVERAGE Gmax - 50%
BEAVER VALLEY POWER STATION-UNIT I

4.0 AMPLIFIED RESPONSE ANALYSIS

Soil-structure interaction analysis can be performed using a direct finite element solution in which the dynamic model is composed of detailed representations of both the structure and the supporting medium. In a direct interaction analysis, the effects of embedment upon stiffness and control motion are automatically included. Although such a procedure may appear to be efficient, analyses become more difficult to manage when large, complex structures are founded upon stratified media. Also, this procedure does not produce any intermediate results, which are often useful in making engineering assessments.

Many different procedures may be used to reduce such an analysis to more manageable steps. For example, a detailed finite element soil model can be used to compute frequency-dependent stiffnesses that are then used in a second step for seismic analysis of a detailed structural model. For embedded structures, however, some method that redefines the control motion must be included. An earthquake with a specified amplitude and frequency content at the site surface is not necessarily a reasonable input to the detailed model in the second step.

A multiple-step analysis need not rely upon finite element representations of soil. The three-step solution described below is based upon the theory of

elasticity, and includes a solution for the problem of definition of the control motion in the case of embedded structures.

4.1 DESCRIPTION OF THE THREE-STEP ANALYSIS

The solution of soil-structure interaction problems can be reduced to the following three steps:

1. calculations of frequency-dependent soil stiffnesses

2. modification of the specified surface motion to account for structure embedment

3. interaction analysis

These steps are illustrated in Figure 4-1 (see Reference 2).

4.1.1 Frequency-Dependent Soil Stiffness

The frequency-dependent stiffnesses of a rectangular footing founded at the surface of a layered medium are computed with the program REFUND, discussed in Section 10.3. The program solves the problem of forced vibration of a rigid plate on a viscoelastic, layered stratum using numerical solutions to the

generalized problems of Cerruti and Boussinesq (see Figure 4-2). The effects of unit harmonic horizontal and vertical point loads are combined by superposition to produce the behavior of a rectangular plate.

Solutions to the problem of a point load on the surface of continuum require an assumption about the behavior of the medium directly under the load; for example, see Timoshenko and Goodier.[1] In REFUND, a solution directly under the load is achieved by employing a column of elements for which a linear displacement function is assumed. Away from this central column, in the "far-field," the solution for a viscoelastic layered medium is obtained (see Figure 4-3).

If the central column under the point load is removed and replaced by equivalent distributed forces corresponding to the internal stresses, the dynamic equilibrium of the far field is preserved. Since no other prescribed forces act on the far field, the displacements at the boundary (and any other point in the far field) are uniquely defined in terms of these boundary forces. The problem is thus to find the relations between these boundary forces and the corresponding boundary displacements.

It is always possible to express the displacements in the far field in terms of eigenfunctions corresponding to the natural modes of wave propagation in the stratum, each having a characteristic wave number k. In an unbounded

medium, any value of the wave number k, and hence any wavelength, is admissible; for a layered stratum, however, only a numerable set of values of k (each one with a corresponding propagation mode) satisfies the boundary conditions. There are thus, at a given frequency, an infinite but numerable set of propagation modes and wave numbers k that can be found by solving a transcendental eigenvalue problem. For each eigenfunction the distribution of stresses can be determined up to a multiplicative constant, the participation factor of the mode. By combining these modal stresses to match any given distribution of stresses at the boundary, the participation factors and the corresponding dynamic stiffness function relating boundary stresses to boundary displacements can be determined.

In REFUND's cylindrical coordinates, loads and displacements are expanded in Fourier series around the axis:

$$u_r = \sum_{0}^{\infty} u_r^n \cos n\theta \qquad p_r = \sum_{0}^{\infty} p_r^n \cos n\theta$$

$$u_y = \sum_{0}^{\infty} u_y^n \cos n\theta \qquad p_y = \sum_{0}^{\infty} p_y^n \cos n\theta$$

$$u_\theta = \sum_{0}^{\infty} -u_\theta^n \sin n\theta \qquad p_\theta = \sum_{0}^{\infty} -p_\theta^n \sin n\theta$$

For the problem at hand, only the first two components of the series are needed. The (unit) vertical force case corresponds to the Fourier component of order zero (n=0), and the horizontal unit force case corresponds to the

Fourier component of order one (n=1). The cartesian displacement (flexibility) matrix (F) at a point then follows from the cylindrical displacement components:

$$\left\{\begin{array}{c|c|c} \frac{1}{2}(u_r^1+u_\theta^1)+\frac{1}{2}(u_r^1-u_\theta^1)\cos 2\theta & u_r^o\cos\theta & \frac{1}{2}(u_r^1-u_\theta^1)\sin 2\theta \\ \hline u_y^1\cos\theta & u_y^o & u_y^1\sin\theta \\ \hline \frac{1}{2}(u_r^1-u_\theta^1)\sin 2\theta & u_r^o\sin\theta & \frac{1}{2}(u_r^1+u_\theta^1)-\frac{1}{2}(u_r^1-u_\theta^1)\cos 2\theta \end{array}\right\}$$

and the displacement vector for arbitrary loading is

$$U = FP$$

where

$$U = \begin{Bmatrix} u_x \\ u_y \\ u_z \end{Bmatrix} \qquad P = \begin{Bmatrix} p_x \\ p_y \\ p_z \end{Bmatrix}$$

U is the displacement vector at a point (x,0,z) while P is the load vector at (0,0,0). The coordinate system is illustrated in Figure 4-4.

For points along the free surface, the reciprocity theorem requires that $U_r^o = U_y'$. Hence, F is chessboard symmetric/antisymmetric. REFUND then

computes the cylindrical displacement components for the two loading cases, and determines the cartesian flexibility matrix F under the load (axis), at the boundary, and at selected points beyond the boundary.

To compute the subgrade stiffness functions for a rigid, rectangular plate, the program discretizes the foundation into a number of points and computes the global flexibility matrix F from the nodal submatrices F using the technique just described. Imposing then the conditions of unit rigid body displacements and rotations, it is possible to solve for the global load vector from the equation

$$FP = U$$

where U is the global displacement vector satisfying the rigid body condition. It follows that U is of the form

$$U = TV$$

where V is a (6x1) vector containing the rigid body translations or rotations of the plate and T is linear transformation matrix assembled with the

coordinates of the nodal points. The stiffness functions are then obtained from

$$Z = T^T P$$

which corresponds formally to

$$Z = T^T F^{-1} T V$$

A comparison of REFUND results with another method is shown in Section 10.3.

4.1.2 Embedment Correction

The effects of foundation embedment on the impedances are included by employing correction factors described by Kausel et al.[2] These correction factors are determined from parametric studies of embedded foundations and are of the form

$$C_R = (1 + C_1 \frac{R}{H})(1 + C_2 \frac{E}{R})(1 + C_3 \frac{E}{H})$$

in which

C_R = correction factor

R = foundation radius

E = embedment depth

H = depth to bedrock

C_i = constants, different values for each degree of freedom.

The frequency dependent stiffnesses, K, determined by REFUND are modified to become

$$K^1 = K \times C_R$$

4.1.3 Kinematic Interaction

In the second step of the analysis shown in Figure 4-1, "kinematic interaction" modifies the purely translational input specified at the surface of the stratum to both a translational and rotational motion at the base of the rigid, massless foundation. The existence of the additional input can be inferred from Figure 4-5. In a stratum undergoing translational motion only,

the boundary conditions at the "excavation" require the foundation to rotate. Ignoring the rotational component would result in an unconservative solution.

Note that the modified motion at the base of the foundation is not equivalent to a deconvolution. The specified surface motion is modified so that

$$\ddot{y}_1(t) = \mathrm{IFT}\begin{cases} F(\Omega)\left[\cos\left(\frac{\pi f}{2f_n}\right)\right], & f \le 0.7 f_n \\ F(\Omega)\left[0.453\right], & f > 0.7 f_n \end{cases}$$

and

$$\ddot{\phi}_1(t) = \mathrm{IFT}\begin{cases} F(\Omega)\left[0.257\left(1 - \cos\frac{\pi}{2}\frac{f}{f_n}\right)/R\right], & f \le f_n \\ F(\Omega)\left[0.257/R\right], & f > f_n \end{cases}$$

$F(\Omega)$ = Fourier Transform of surface motion

IFT = inverse transform

R = foundation radius

f_n = fundamental shear beam frequency of the column of soil between the embedment level and the free surface

These relationships are taken from Kausel et al.[2]

A finite element analysis of a rigid, massless, embedded foundation provides a demonstration that the relations above are reasonable and conservative. Such a comparison is shown in Section 10.4 (KINACT).

4.1.4 Interaction Analysis

The third step of the procedure illustrated schematically in Figure 4-1 is the analysis of the structural model supported on the frequency-dependent springs from Step 1 for the modified seismic input from Step 2. The solution is achieved using the program FRIDAY.

FRIDAY evaluates the dynamic response of an assembly of cantilever structures supported by a common mat and subjected to a seismic excitation. The support of the mat can be rigid, or it can consist of frequency-dependent/independent springs and dashpots (subgrade stiffnesses). The equations of motion are solved in the frequency domain, determining response time histories by convolution of the transfer functions and the Fourier transform of the input excitation. The dynamic equilibrium equations can be written in matrix notation as:

$$M\ddot{U} + C\dot{Y} + KY = 0 \tag{1}$$

where M, C, K are the mass, damping and stiffness matrices, respectively, and U, Y are the absolute and relative (to the moving support) displacement vectors.

These two vectors are related by:

$$U = Y + EU_g \tag{2}$$

where U is the base excitation vector (3 translations and 3 rotations), and E is the matrix:

$$E = \begin{Bmatrix} I & T_1 \\ O & I \\ I & T_2 \\ O & I \\ & \vdots \\ I & T_n \\ O & I \end{Bmatrix} \tag{3}$$

where I is the (3x3) identity matrix, 0 is the null matrix, and

$$T_i = \left\{ \begin{array}{c|c|c} 0 & Z_i - Z_0 & -(Y_i - Y_0) \\ -(Z_i - Z_0) & 0 & X_i - X_0 \\ Y_i - Y_0 & -(X_i - X_0) & 0 \end{array} \right\}$$

with x_i, y_i, z_i being the coordinates of the corresponding mass point; x_o, y_o, z_o are the coordinates of the common support.

In the frequency response method, the transfer functions are determined by setting, one at a time, the ground motion components equal to a unit harmonic of the form $u_i = e^{iwt}$. It follows then that U, Y are also harmonic:

$$\begin{aligned} \ddot{U} &= H_j e^{i\omega t} & \ddot{Y} &= (H_j - E_j) e^{i\omega t} \\ \dot{U} &= \frac{1}{i\omega} H_j e^{i\omega t} & \dot{Y} &= \frac{1}{i\omega}(H_j - E_j) e^{i\omega t} \\ U &= -\frac{1}{\omega^2} H_j e^{i\omega t} & Y &= -\frac{1}{\omega^2}(H_j - E_j) e^{i\omega t} \end{aligned} \tag{4}$$

where $H_i = H_i(\omega)$ is the vector containing the transfer functions for the j^{th} input ground motion, and E_i is the j^{th} column of E in Eq. 3. Substitution of Eq 4 into Eq 1 yields

$$(-\omega^2 M + i\omega C + K)H_j = (i\omega C + K)E_j \tag{5}$$

If the damping matrix is of the form $C = \frac{1}{\omega} D$, which corresponds to a linear hysteretic damping situation, the equation reduces to

$$(-\omega^2 M + K + iD)H_j = (K + iD)E_j \tag{6}$$

In view of the correspondence principle, it is possible to generalize the equation of motion allowing at this stage elements in the stiffness matrix K with an arbitrary variation with frequency. This enables the use of frequency-dependent stiffness functions or impedance (inverse of flexibility functions or compliances).

Defining the dynamic stiffness matrix:

$$K_d = K + iD - \omega^2 M \tag{7}$$

The solution for the transfer functions follows formally from:

$$H_j = -K_d^{-1}(K + iD)E_j$$
$$= -(I + \omega^2 K_d^{-1} M)E_j \tag{8}$$

Note that the dynamic stiffness matrix K does not depend on the loading condition E_i. Also, for $\omega = 0$, $H_i(0) = E_i$.

Having found the transfer functions, the acceleration time-histories follow then from the inverse Fourier transformation:

$$\ddot{U} = \frac{1}{2\pi}\int_{-\infty}^{\infty}\left\{\sum_{j=1}^{j=6} H_j f_j\right\} e^{i\omega t} d\omega \tag{9}$$

where, $f_i = f_i(\omega)$ is the Fourier transform of the j^{th} input acceleration component:

$$f_j = \int_0^T \ddot{u}_j e^{-i\omega t} dt \tag{10}$$

The procedure consists then of determining the dynamic stiffness matrix K_d, solving Eq 6 for the six loading conditions $H = \left\{H_j\right\}$, determining the six Fourier transforms of the input components $F = \left\{f_j\right\}$, and performing the inverse transformation (Eq 9), which corresponds formally to:

$$\ddot{U} = \frac{1}{2\pi}\int_{-\infty}^{\infty} HF\, e^{i\omega t}\, d\omega$$

The dynamic equations are solved in FRIDAY by Gaussian elimination, and the Fourier transforms are computed by subroutines using the Cooley-Tuckey FFT (fast Fourier transform) algorithm. A comparison of the results of FRIDAY with another solution is shown in Section 10.5.

4.2 STRUCTURAL MODELING

The level of detail in mathematical models of structures is determined by consideration of the following:

1. distribution of mass in the building
2. symmetry/asymmetry of building arrangement
3. locations at which output is required
4. approximate frequency content of input

The models used in the analysis, typically, are generalized, three-dimensional, multi-mass representations. The total number of degrees of freedom included is more than sufficient to encompass all significant frequencies; the number of masses being governed, as a practical matter, by the locations at which amplified response spectra (ARS) are required.

Eccentricity between the center of mass and center of stiffness at every level is included, except where insignificant. As a result, the effects of torsion upon the modes and frequencies is automatically determined. A typical model is shown in Figure 4-6. The generalized dynamic members connecting the centers of mass have stiffness matrices determined by tensor transformation from the matrices of the structural elements connecting the centers of stiffness.

To demonstrate the effects of torsion on the results, a comparison was made between the analyses using a generalized three-dimensional model and a planar model of the main steam valve building. This building has one open side and relatively large eccentricities between centers of mass and rigidity. The results of this comparison are shown in Figures 4-7 to 4-10 and indicate that for this site the effects of torsion are not significant.

4.3 RESULTS

Output from the third step FRIDAY includes structural response as well as ARS for all coordinates in each structure analyzed. In general, a structural coordinate coincides with a building floor level. Typical structural acceleration and displacement profiles are shown in Figures 4-11 and 4-12. ARS are generated for two orthogonal horizontal and the vertical directions at each structural coordinate for both OBE and DBE earthquakes. Typical ARS are shown in Section 5. For use in pipe stress problems, ARS peaks are automatically broadened ±25 percent to account for variations in soil and structural material properties.

Comparisons of ARS generated by the three-step REFUND/FRIDAY method and the finite element PLAXLY method as well as those based on the FSAR earthquake and the Regulatory Guide 1.60 earthquakes were made at the request of the NRC. The ARS generated for these comparisons used strain compatible soil parameters from the last iteration of the SHAKE program.

Comparisons were also made of ARS generated from the REFUND/FRIDAY programs for a variety of soil parameters as requested by the NRC.

All ARS comparisons are described in Section 5.

4.4 REFERENCES

1. Timoshenko & Goodier, Theory of Elasticity, 3rd Edition. McGraw-Hill Book Co., p 97-109.

2. Kausel, Whitman, Morray, & Elsabee, The Spring Method for Embedded Foundations. Nuclear Engineering and Design 48(1978): 377-392.

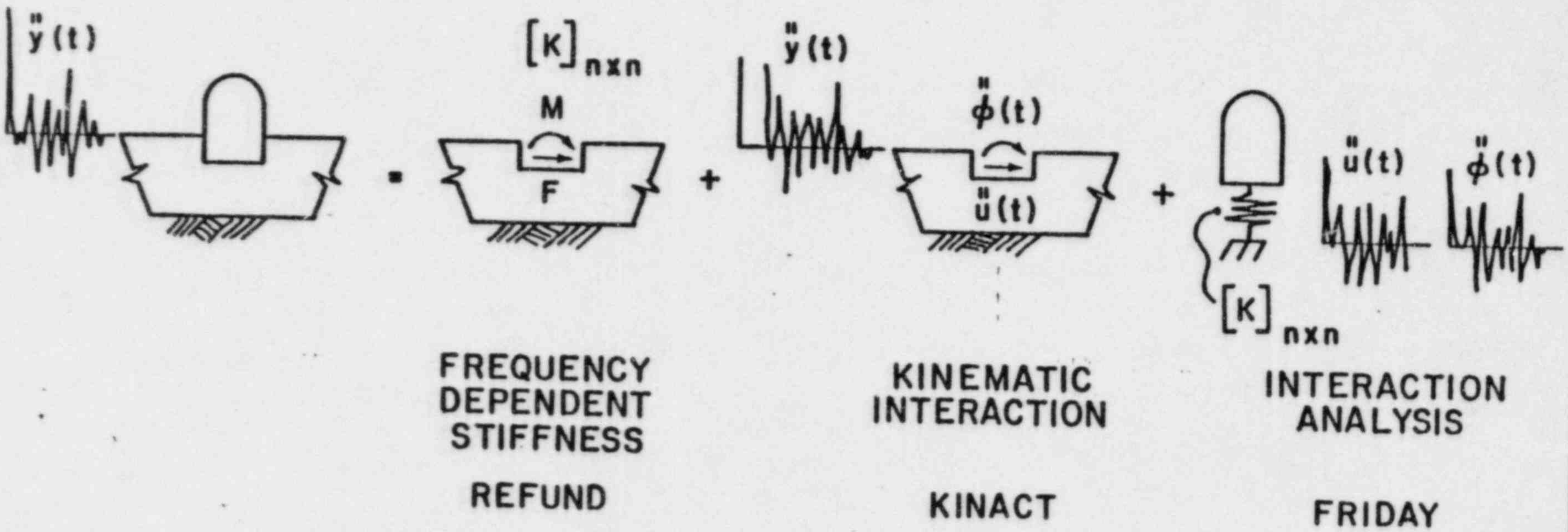


FIGURE 4-1
THE THREE STEP SOLUTION
BEAVER VALLEY POWER STATION - UNIT 1

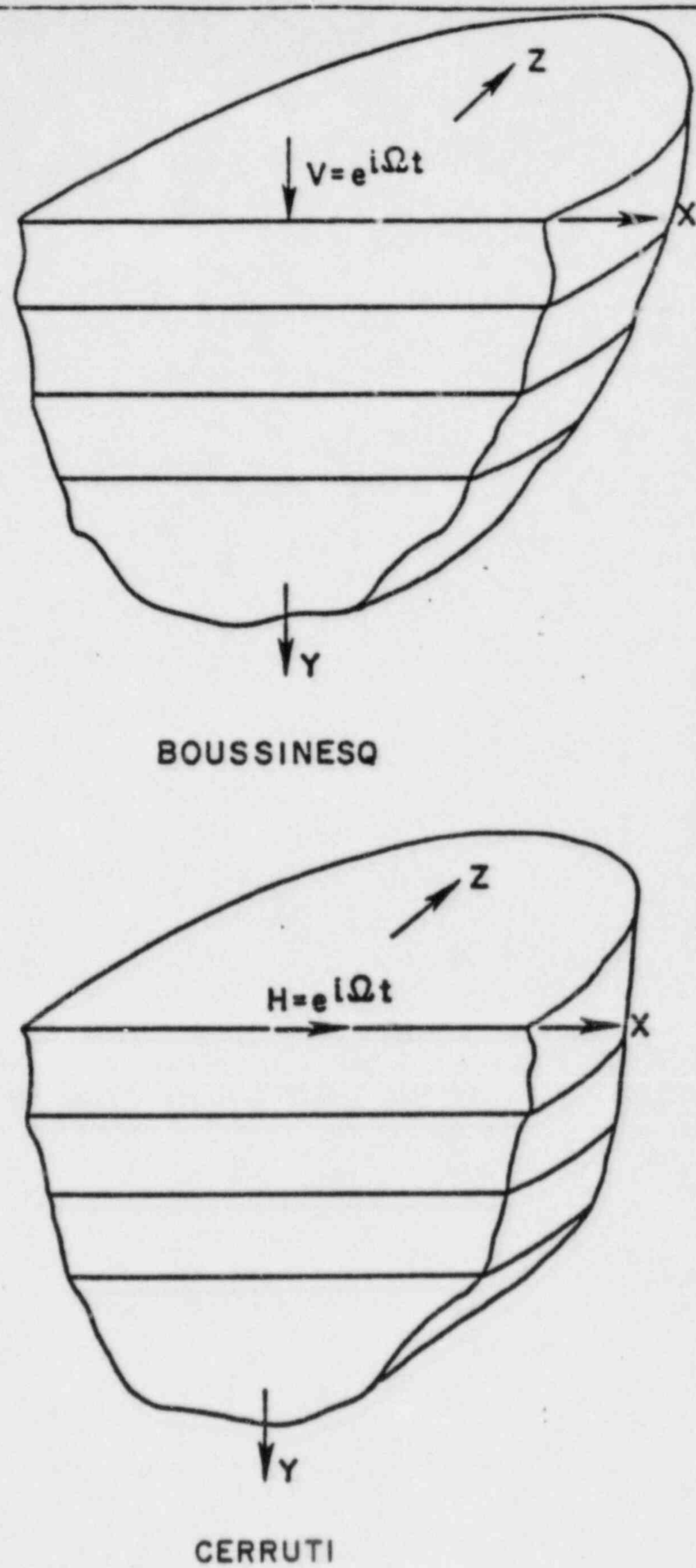


FIGURE 4-2
THE BOUSSINESQ AND CERRUTI PROBLEMS
BEAVER VALLEY POWER STATION - UNIT 1

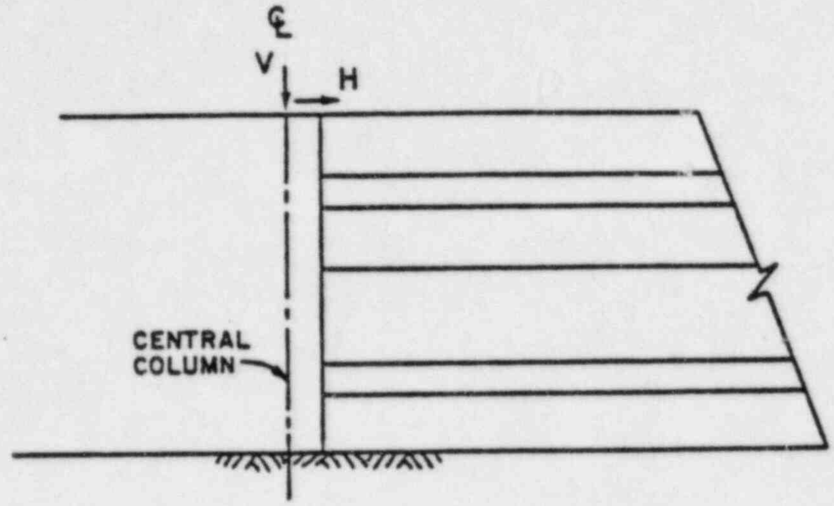


FIGURE 4-3
IDEALIZATION OF THE BASIC 'REFUND'
SOLUTION FOR CONCENTRATED LOADS
BEAVER VALLEY POWER STATION - UNIT 1

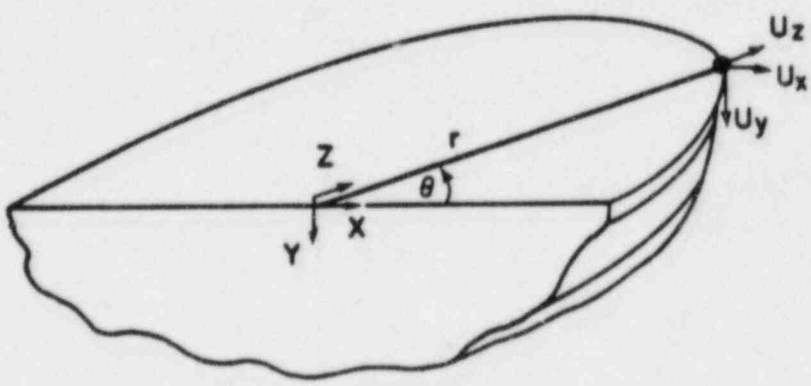
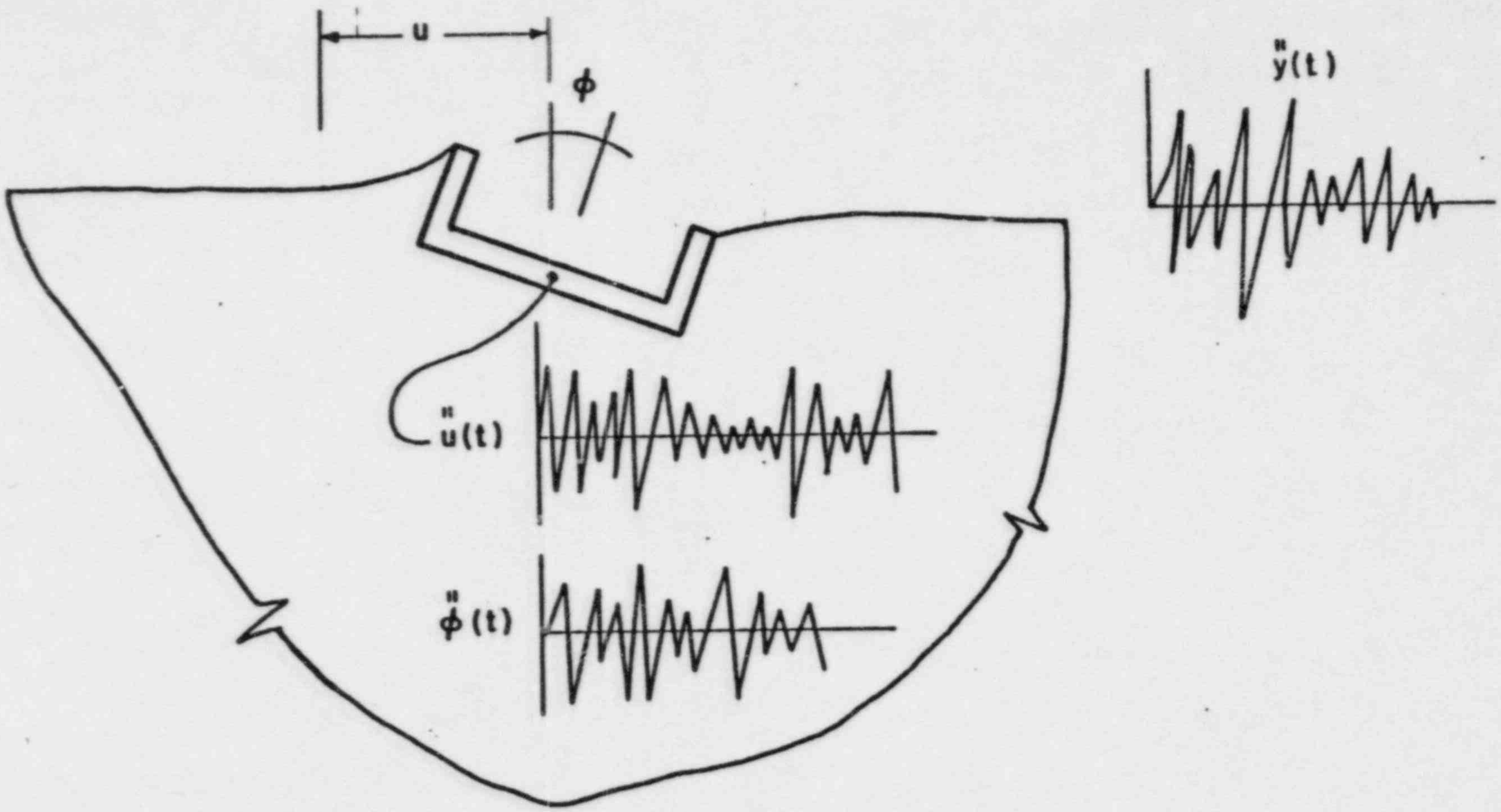


FIGURE 4-4
'REFUND' COORDINATE SYSTEM
BEAVER VALLEY POWER STATION - UNIT 1



$\ddot{u}(t)$ = TRANSLATIONAL ACCELERATION AT BASE OF RIGID, MASSLESS FOUNDATION

$\ddot{\phi}(t)$ = ROTATIONAL ACCELERATION

FIGURE 4-5
KINEMATIC INTERACTION
BEAVER VALLEY POWER STATION - UNIT 1

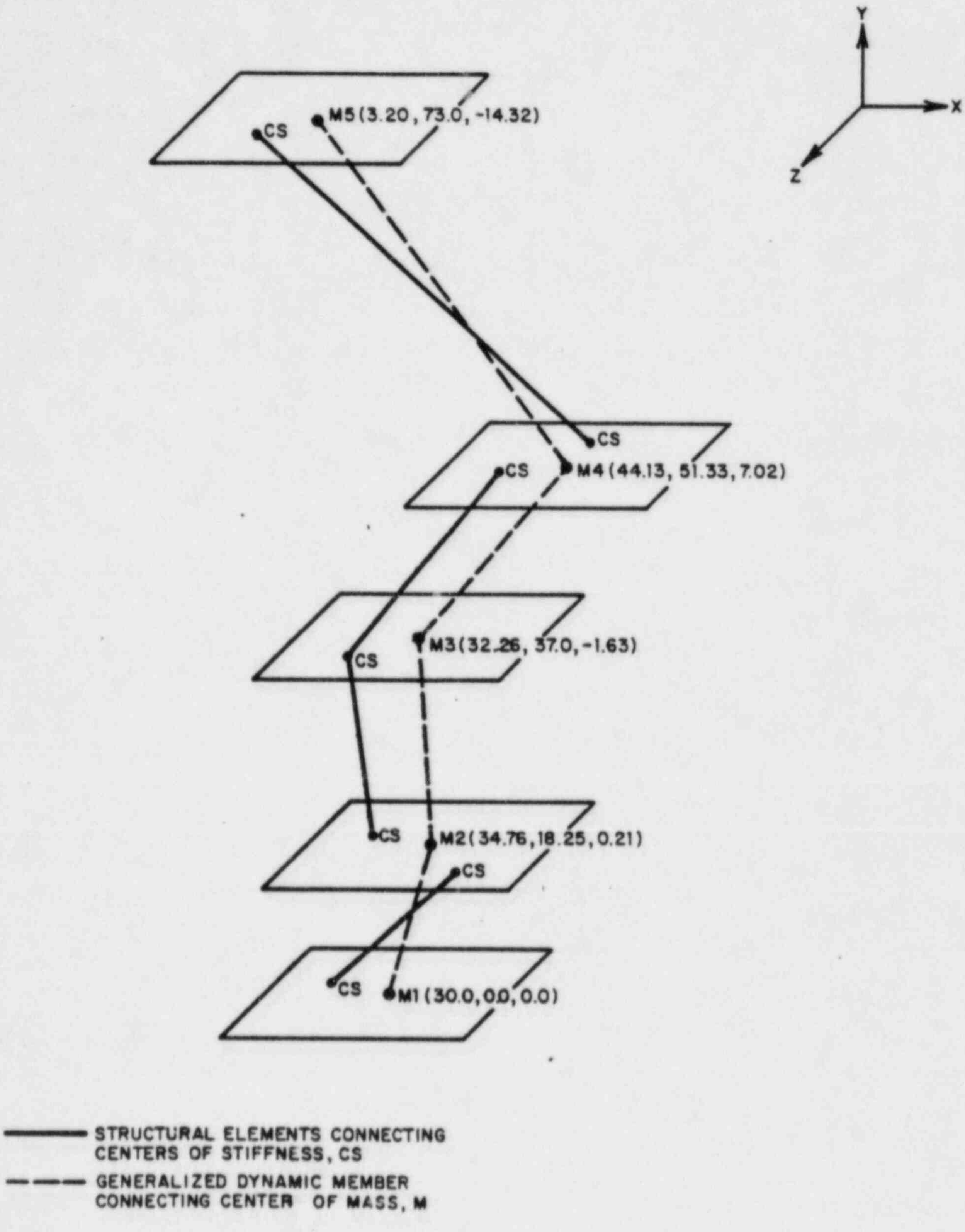


FIGURE 4-6
GENERALIZED DYNAMIC MODEL OF A CATEGORY 1 STRUCTURE
BEAVER VALLEY POWER STATION - UNIT 1

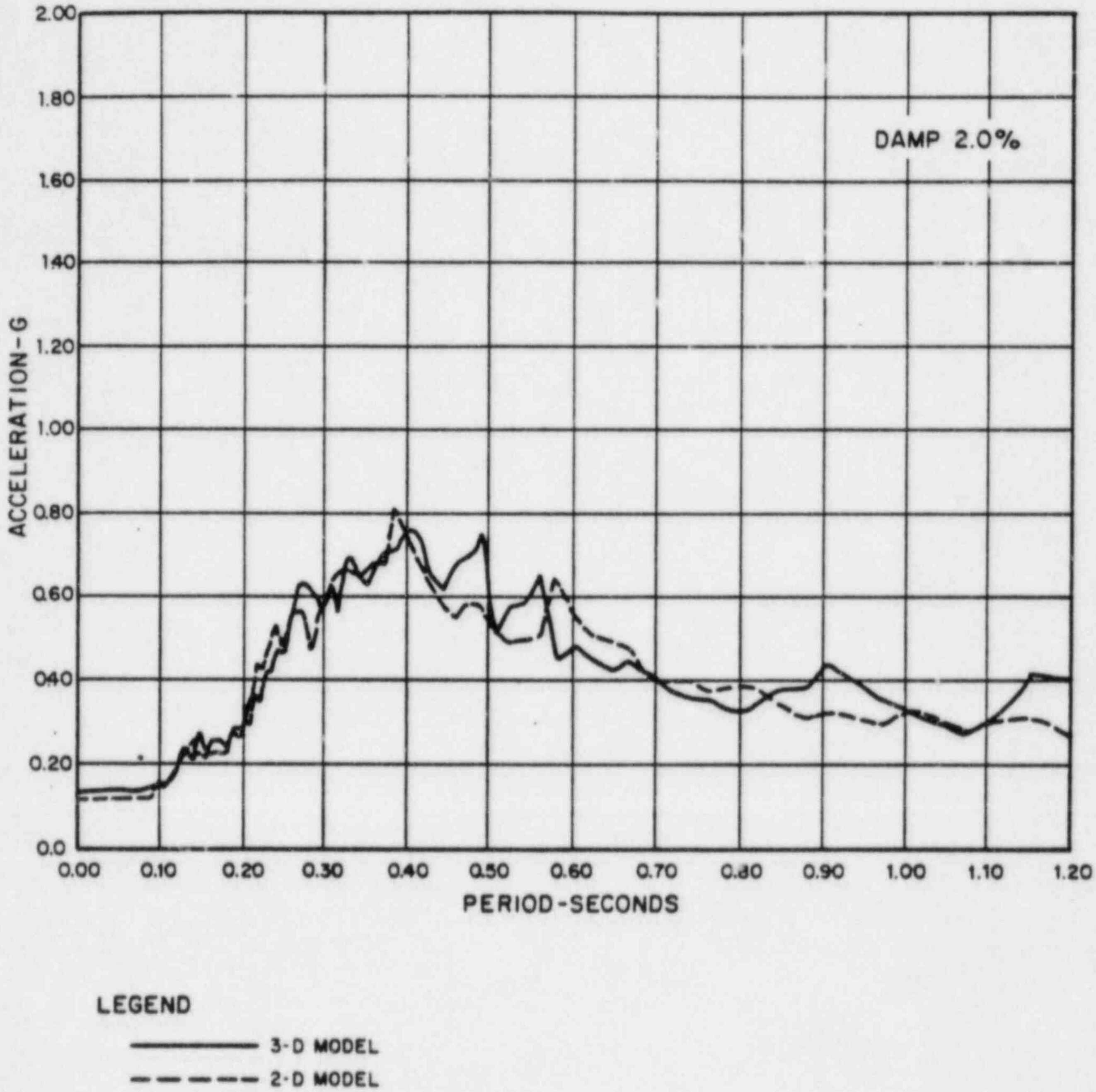


FIGURE 4-7
SEISMIC ANALYSIS OF
MAIN STEAM VALVE BUILDING
HORIZONTAL SSE
EW HORIZONTAL RESPONSE SPECTRUM
AT MAT
BEAVER VALLEY POWER STATION-UNIT 1

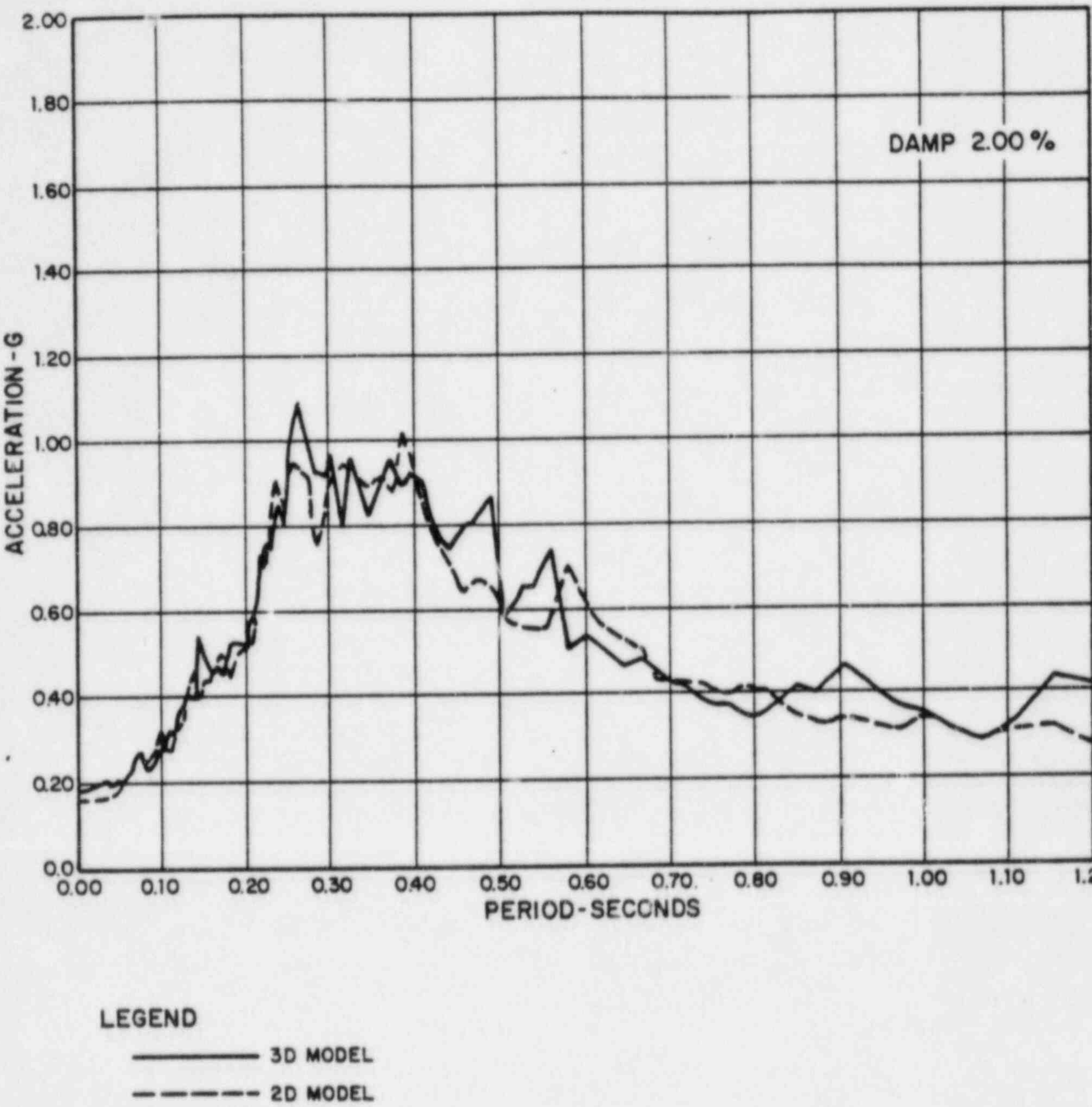


FIGURE 4-8
SEISMIC ANALYSIS OF
MAIN STEAM VALVE BUILDING
HORIZONTAL SSE
EW HORIZONTAL RESPONSE SPECTRUM AT TOP
BEAVER VALLEY POWER STATION - UNIT 1

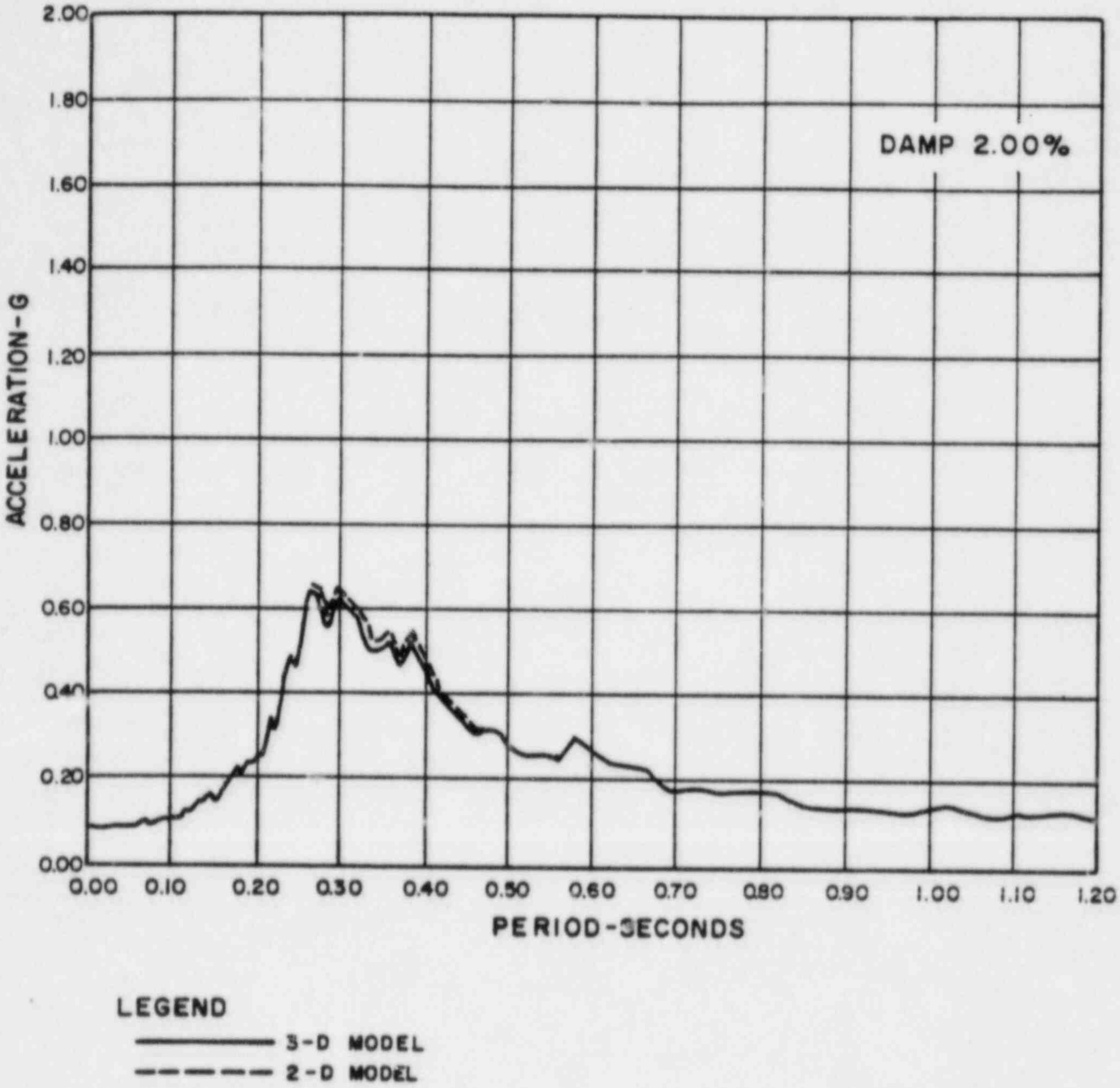


FIGURE 4-9
SEISMIC ANALYSIS OF
MAIN STEAM VALVE BUILDING
HORIZONTAL SSE
NS HORIZONTAL RESPONSE SPECTRUM AT MAT
BEAVER VALLEY POWER STATION-UNIT 1

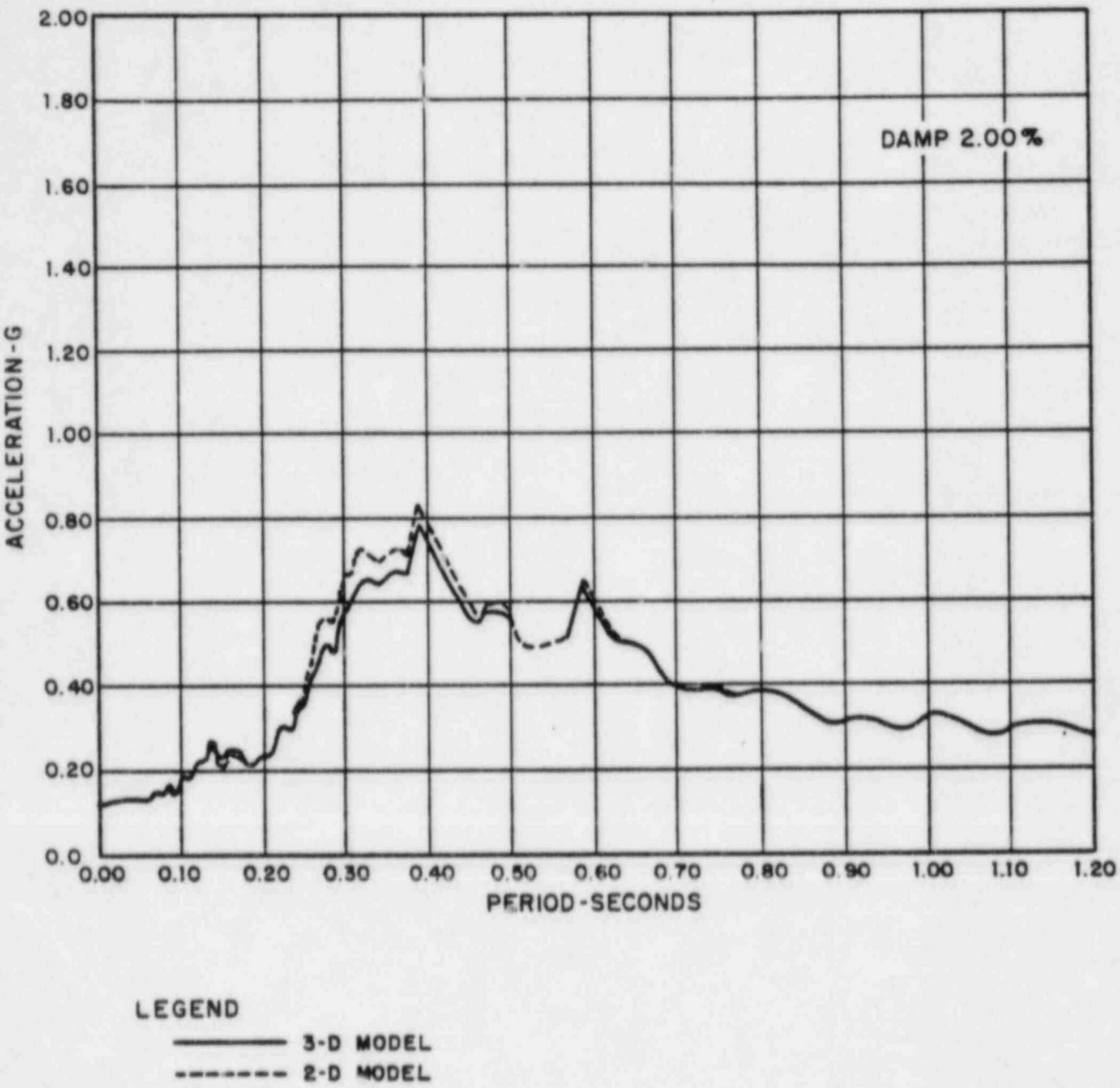


FIGURE 4-10
SEISMIC ANALYSIS OF
MAIN STEAM VALVE BUILDING
HORIZONTAL SSE
NS HORIZONTAL RESPONSE SPECTRUM AT TOP
BEAVER VALLEY POWER STATION - UNIT 1

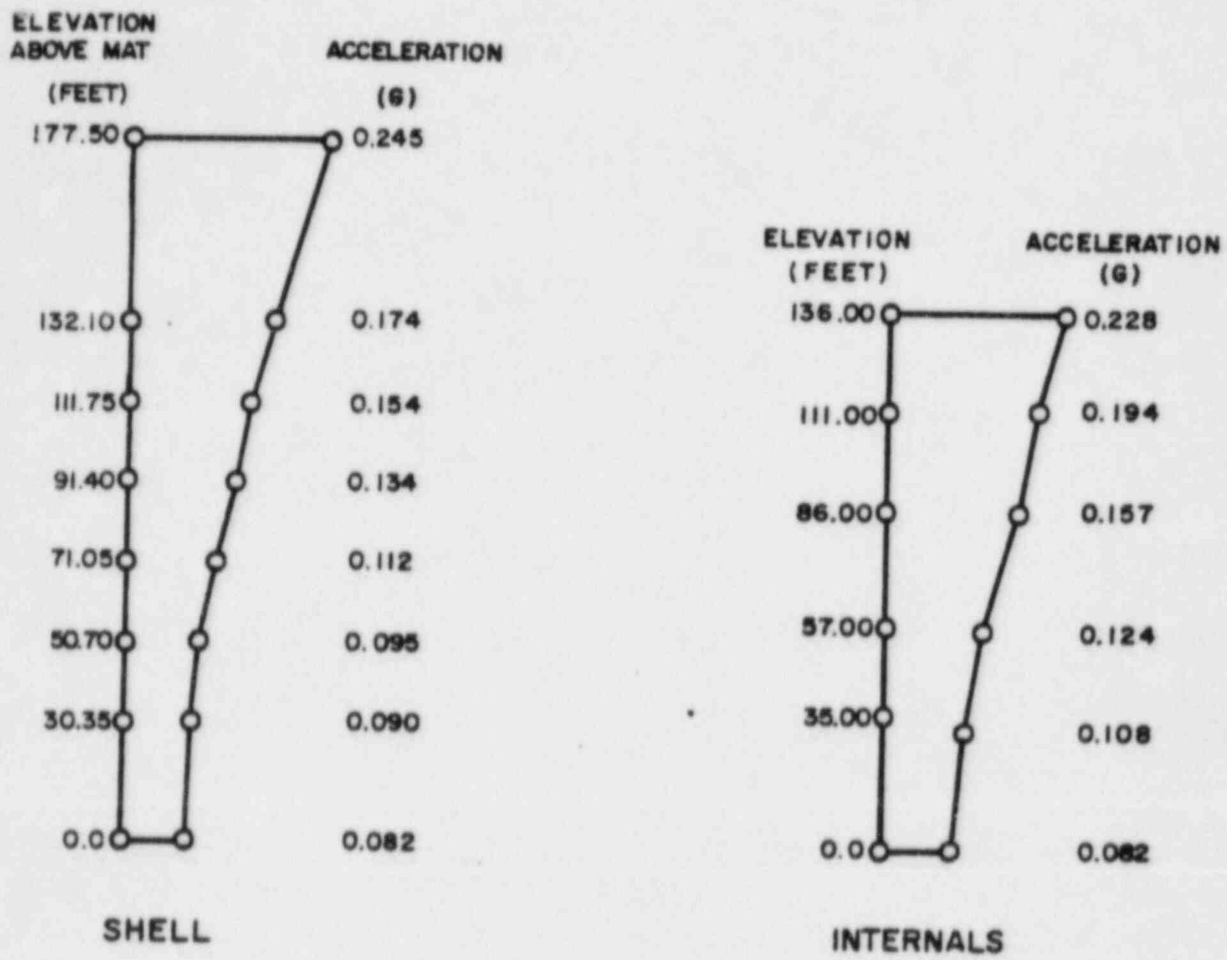


FIGURE 4-11

TYPICAL ACCELERATION PROFILES

BEAVER VALLEY POWER STATION - UNIT 1

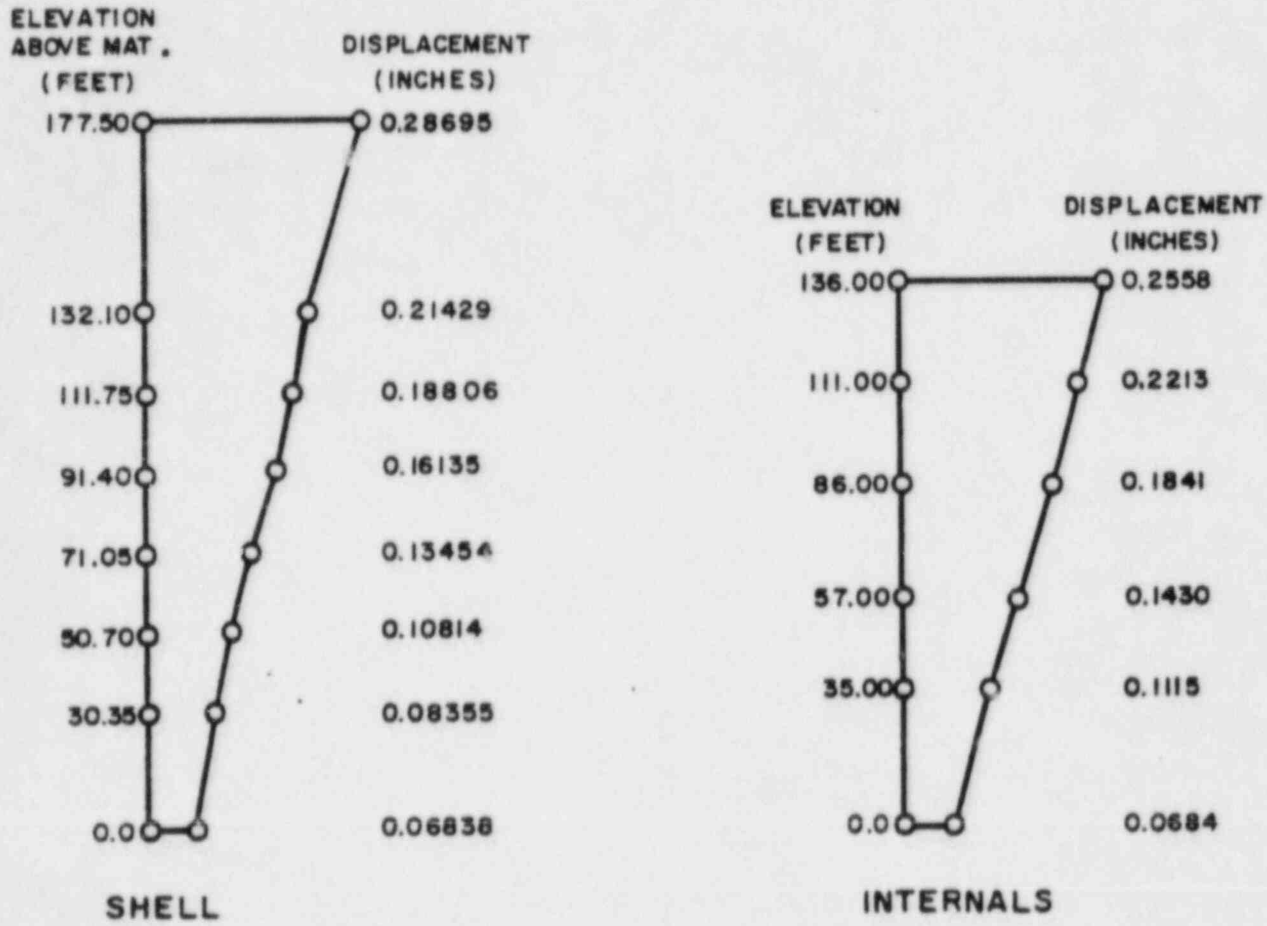


FIGURE 4-12
TYPICAL DISPLACEMENT PROFILES
BEAVER VALLEY POWER STATION - UNIT 1

5.0 COMPARISONS OF RESULTS

Comparisons of amplified response spectra (ARS) for the DBE were prepared for the following cases:

1. Methodology - REFUND/FRIDAY vs PLAXLY

2. Earthquake - FSAR vs Regulatory Guide 1.60

3. Soil Parameter Variation - low strain, first and last iterations from SHAKE; ±50 percent variation of low strain input to SHAKE.

5.1 REFUND/FRIDAY VS PLAXLY

The containment structure was analyzed two ways for purposes of comparison using strain compatible soil parameters from the SHAKE program.

1. A one-step analysis using the finite element program PLAXLY

2. A three-step analysis using the methodology described in Section 4.1

The following observations can be made about the ARS shown in Figures 5-1 through 5-3.:

1. At the mat level, the results of the two methods are very close.

2. With increasing elevation, the REFUND/FRIDAY results become more conservative with respect to the PLAXLY results. This is a consequence of the conservative assumption made about the rotational part of the input in the kinematic interaction step (see, for example, Figure 10.4-2).

5.2 FSAR EARTHQUAKE VS REGULATORY GUIDE 1.60 EARTHQUAKE

Additional analyses were performed at the request of the NRC using the three-step method (REFUND/FRIDAY) to compare the design earthquake in the FSAR to that specified by Regulatory Guide 1.60. The ARS shown in Figures 5-4 through 5-6 are comparisons of consistent piping analysis bases; that is, the spectra for equipment dampings associated with the Regulatory Guide 1.60 earthquake (2 and 3 percent) are displayed with the 1 percent spectra for the FSAR earthquake. The soil shear moduli and damping used for these analyses are from the last iteration of the SHAKE program.

Even though the Regulatory Guide 1.60 earthquake is significantly more energetic than the FSAR earthquake, the results are very close.

5.3 VARIATION OF SOIL PROPERTIES

At the request of the NRC, ARS were generated for a range of soil shear modulus and damping ratio:

1. The low-strain soil shear modulus (Gmax) with soil damping ratio equal to 0.05.

2. Shear modulus and damping after one iteration in SHAKE, starting from the low-strain modulus (Gmax).

3. Shear modulus and damping consistent with earthquake amplitude, but calculated by the program SHAKE starting from 1 1/2 times the low-strain modulus Gmax +50 percent.

4. Shear modulus and damping consistent with earthquake amplitude, but calculated by SHAKE starting from 1/2 times the low-strain modulus Gmax -50 percent.

5. Shear modulus and damping from the last iteration of SHAKE, starting with the low-strain modulus (Gmax).

The ARS for Cases 1, 2, and 5 are compared in Figures 5-7 through 5-16 for piping damping ratios of .005, .010, and .030. They indicate that the analysis is sensitive to extreme variations in parameters but that, within the limits of the iterations of SHAKE, both the amplitudes and frequency content are well-behaved.

The ARS for Cases 3, 4 and 5 are shown in Figures 5-16 through 5-24 for piping damping ratios of .005, .010, and .030. Beginning the SHAKE analysis with 1/2 the low-strain modulus results in extremely low moduli for the final iteration. Again, while apparently sensitive to extreme variations of input parameters, the amplified response analysis is relatively insensitive to variations of modulus and damping in the reasonable middle range of values.

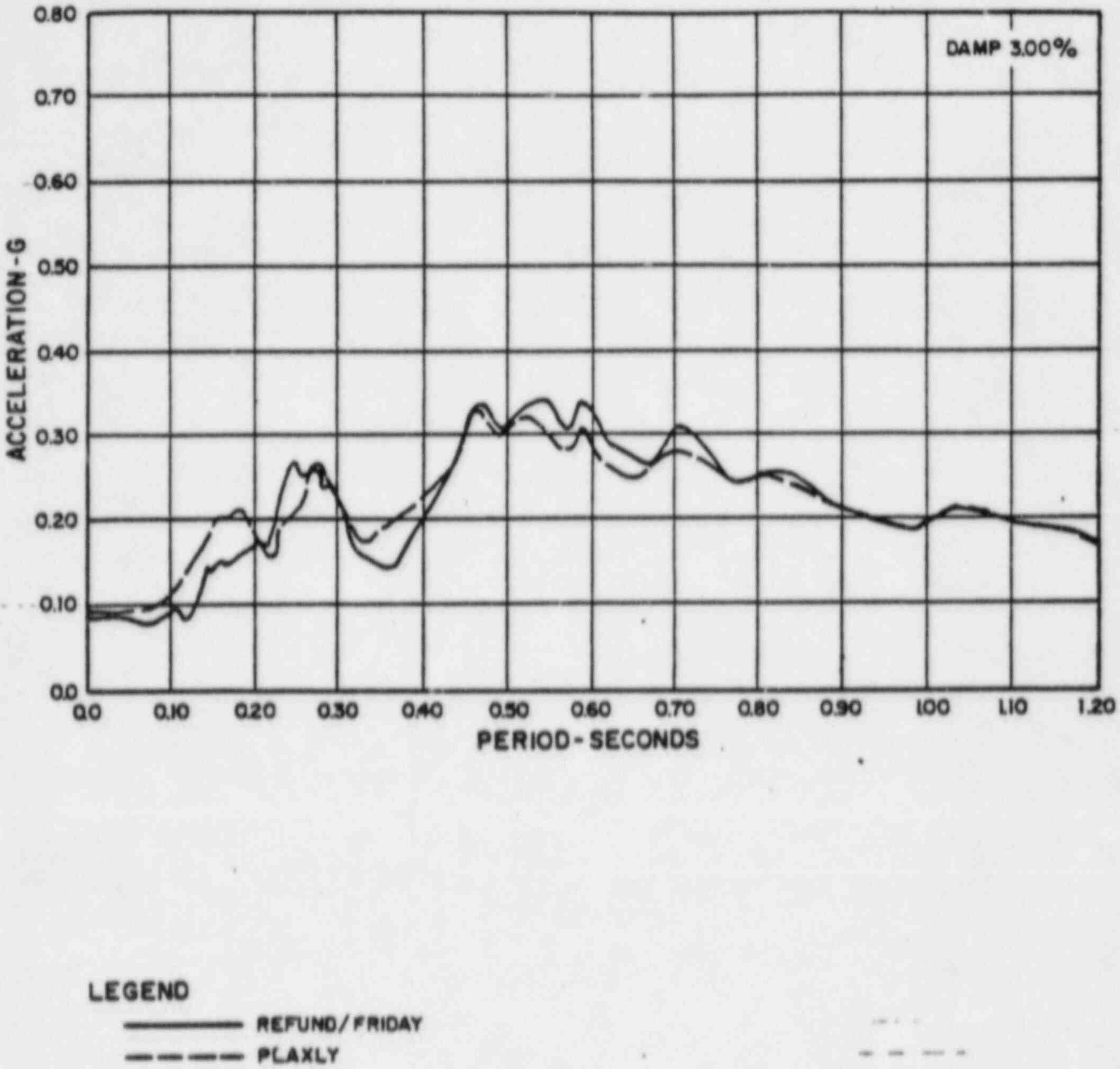


FIGURE 5-1

COMPARISON OF REFUND/FRIDAY AND PLAXLY - ARS AT MAT

BEAVER VALLEY POWER STATION - UNIT 1

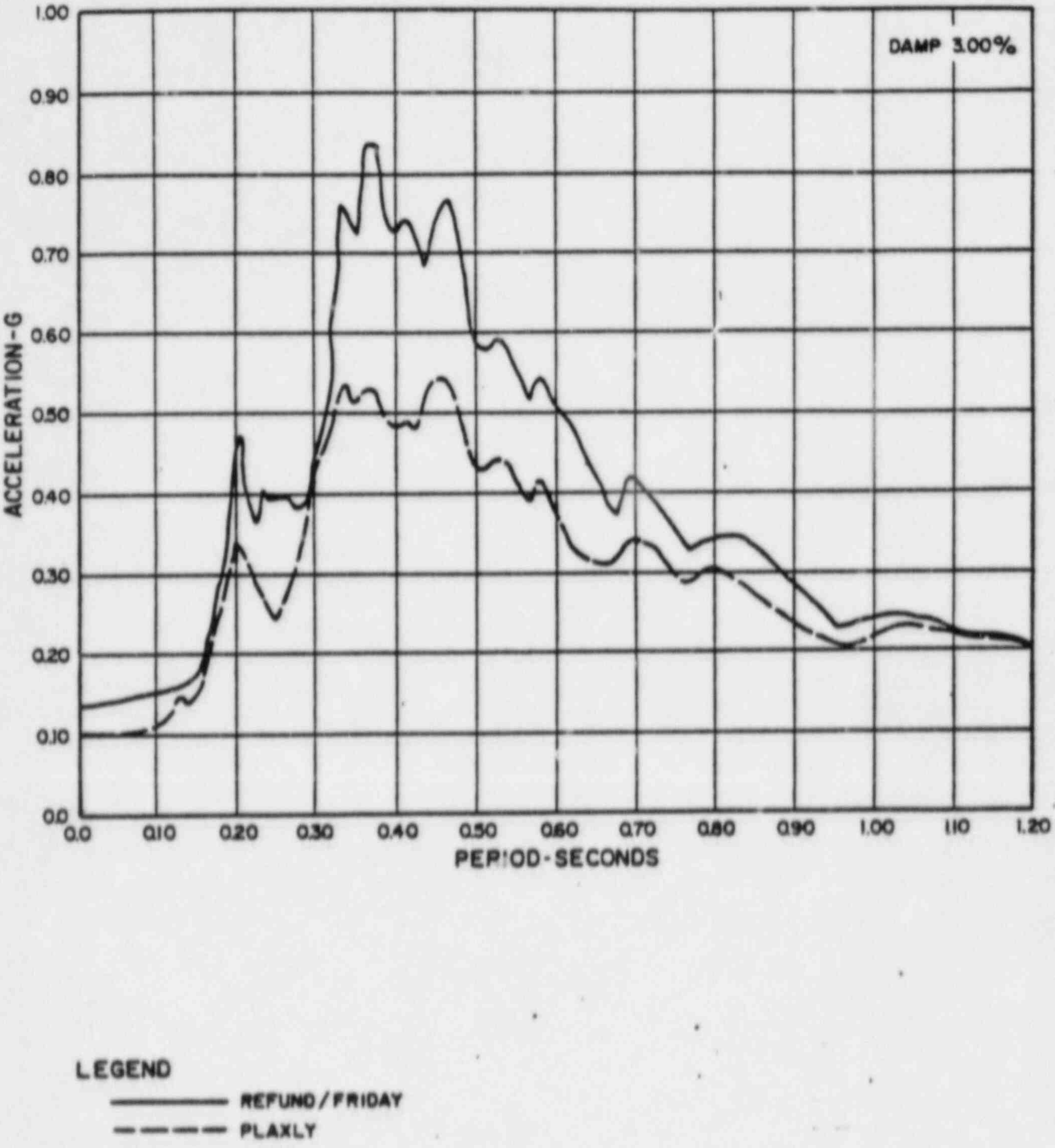


FIGURE 5-2
COMPARISON OF REFUND/ FRIDAY AND PLAXLY-ARS AT OPERATING FLOOR
BEAVER VALLEY POWER STATION-UNIT 1

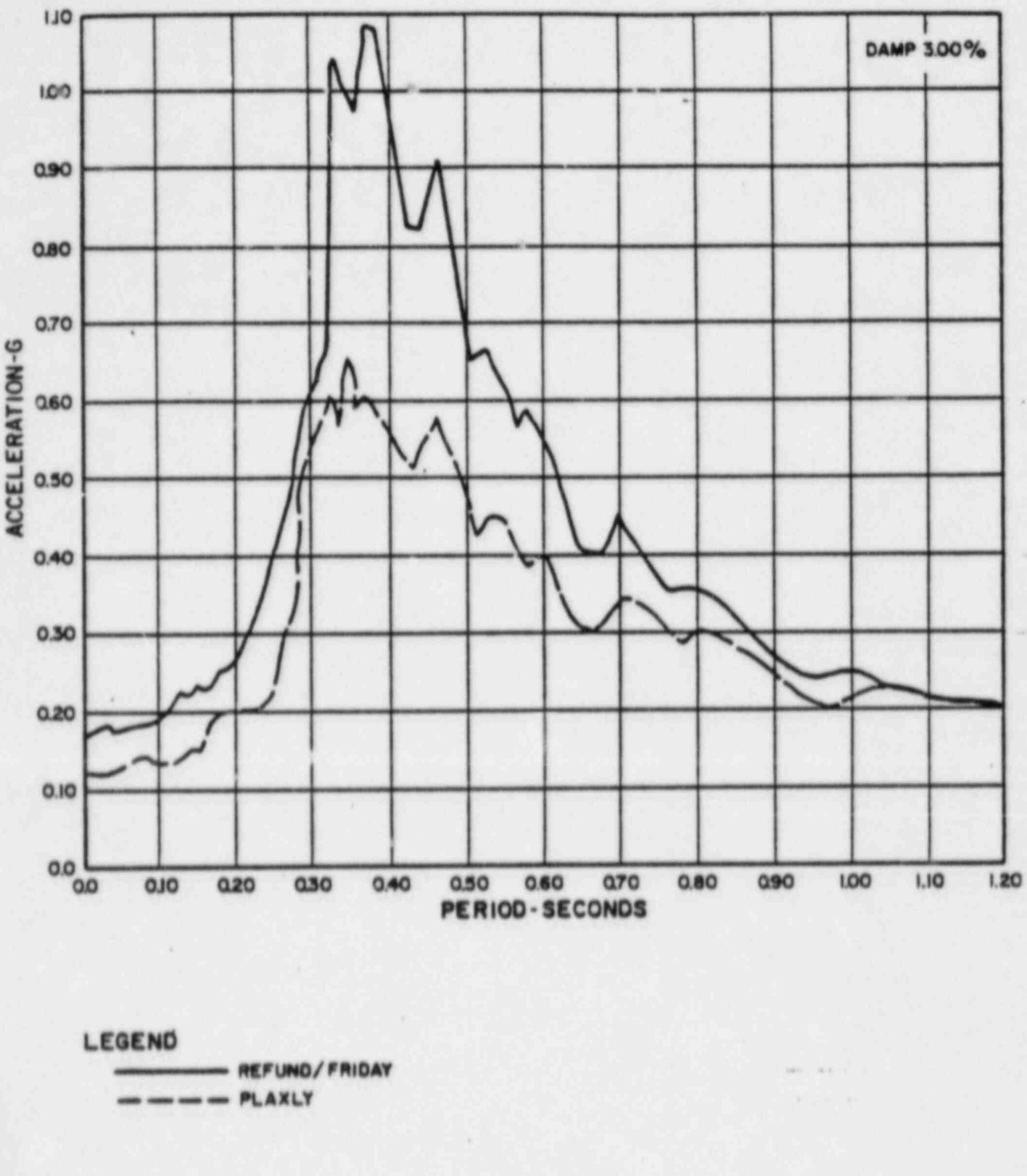


FIGURE 5-3

COMPARISON OF REFUND/FRIDAY AND PLAXLY-ARS AT SPRINGLINE

BEAVER VALLEY POWER STATION-UNIT 1

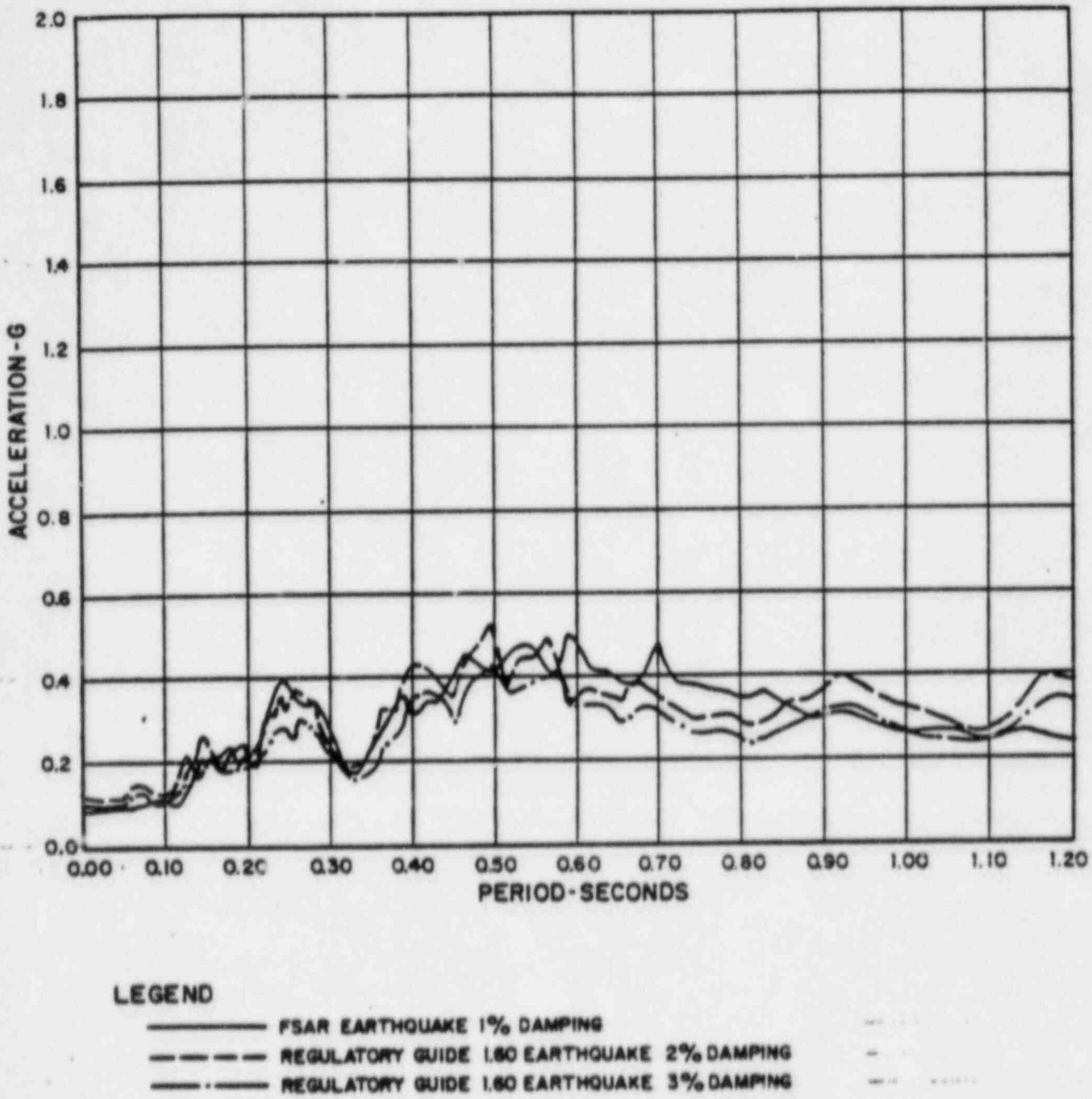


FIGURE 5-4
COMPARISON OF FSAR AND
REGULATORY GUIDE 1.60
EARTHQUAKES-ARS AT MAT
BEAVER VALLEY POWER STATION - UNIT 1

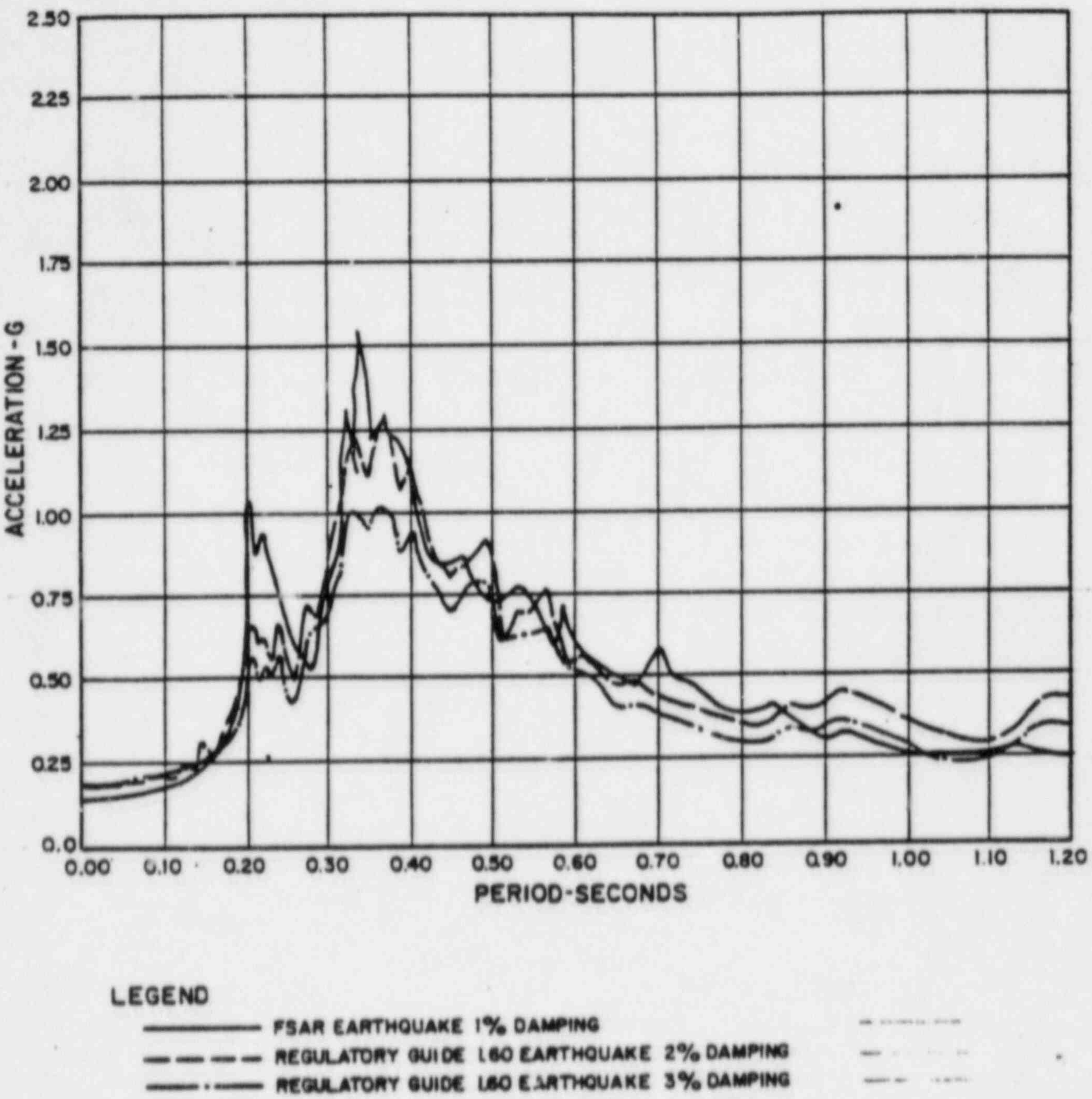
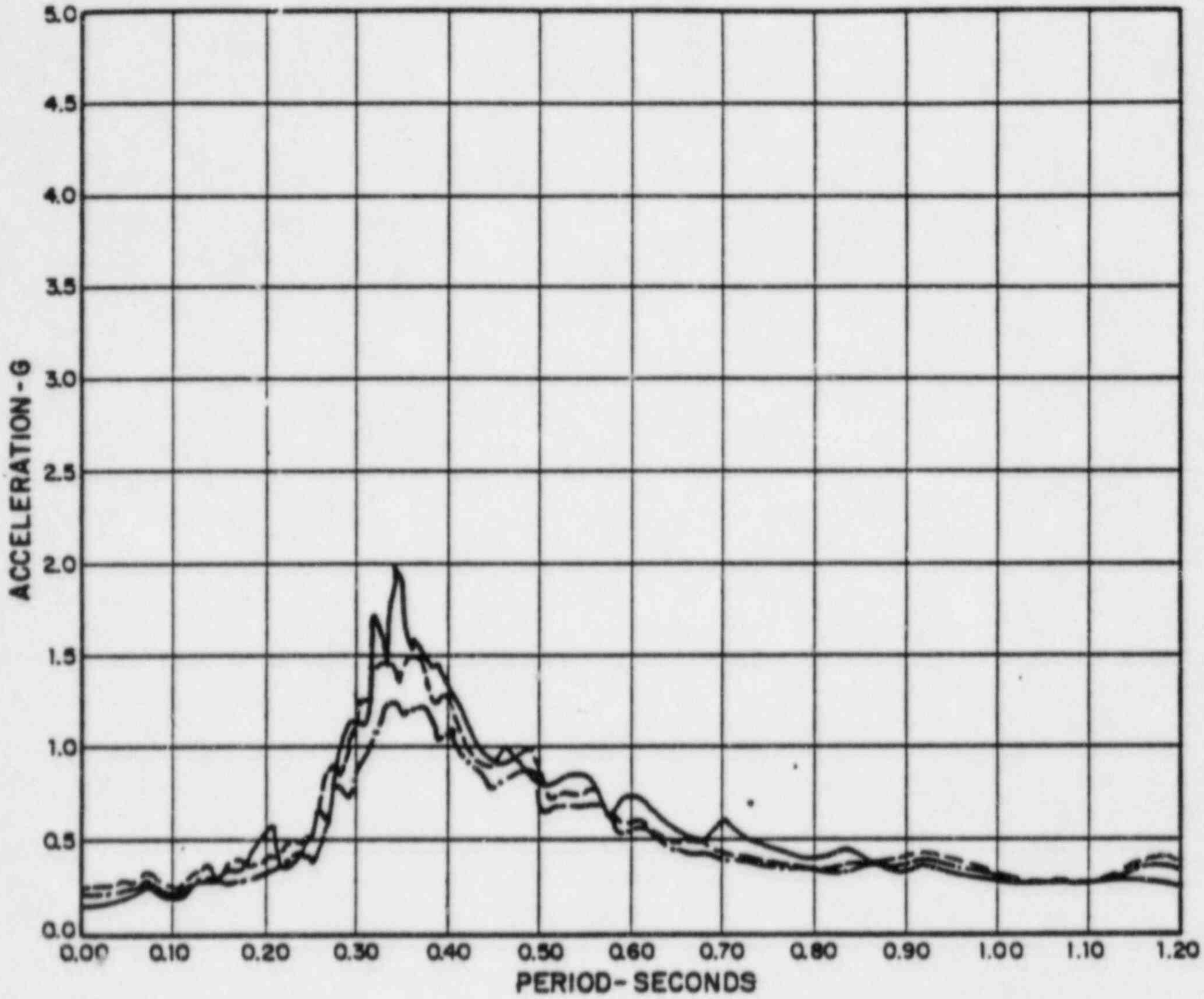


FIGURE 5-5

COMPARISON OF FSAR AND REGULATORY GUIDE 1.60 EARTHQUAKES-ARS AT OPERATING FLOOR

BEAVER VALLEY POWER STATION - UNIT 1



LEGEND

FSAR EARTHQUAKE 1% DAMPING

REGULATORY GUIDE 1.60 EARTHQUAKE 2% DAMPING

REGULATORY GUIDE 1.60 EARTHQUAKE 3% DAMPING

FIGURE 5-6

COMPARISON OF FSAR AND REGULATORY GUIDE 1.60 – ARS AT SPRINGLINE

BEAVER VALLEY POWER STATION - UNIT 1

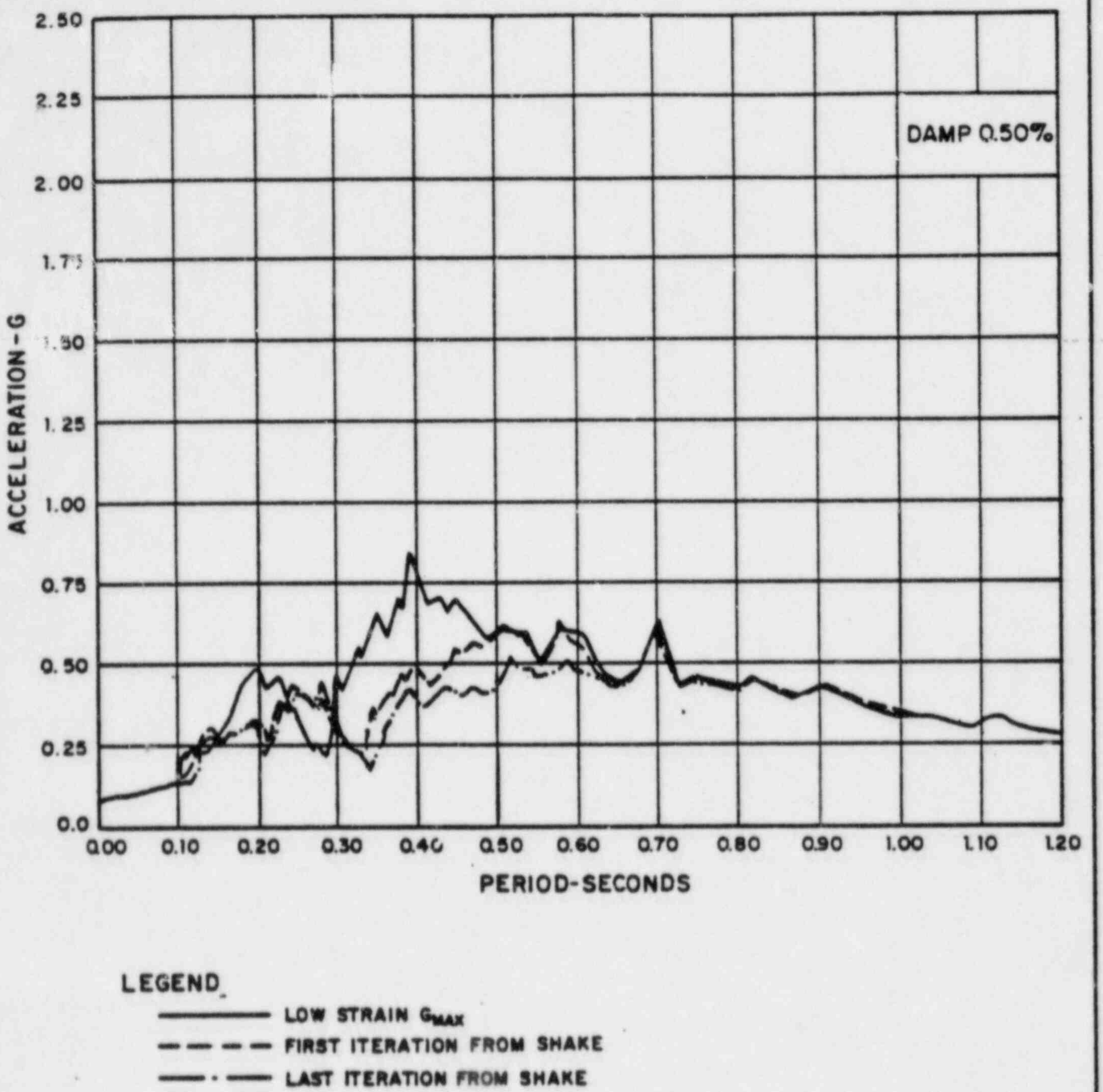


FIGURE 5-7
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM AT MAT
BEAVER VALLEY POWER STATION - UNIT 1

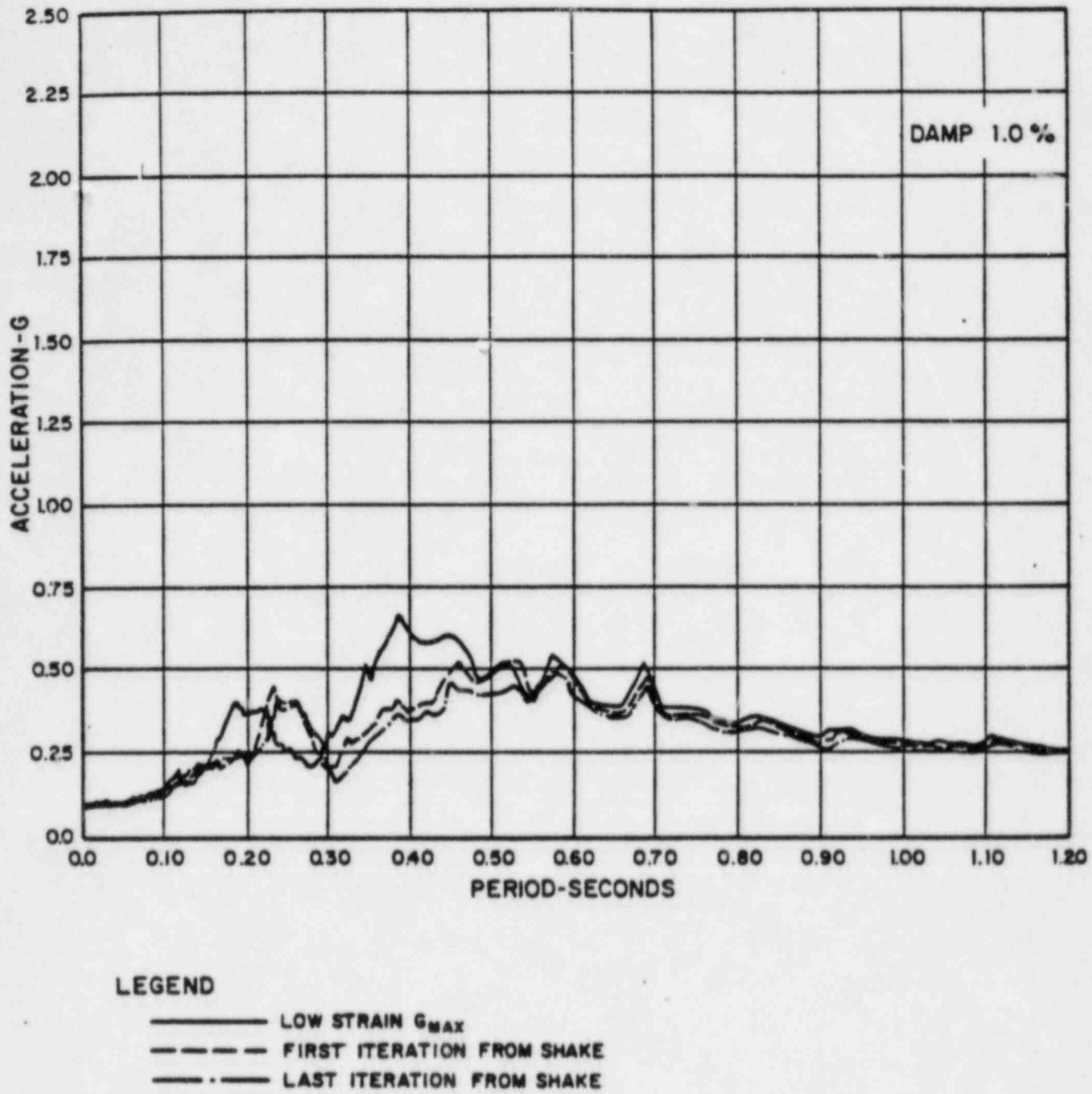
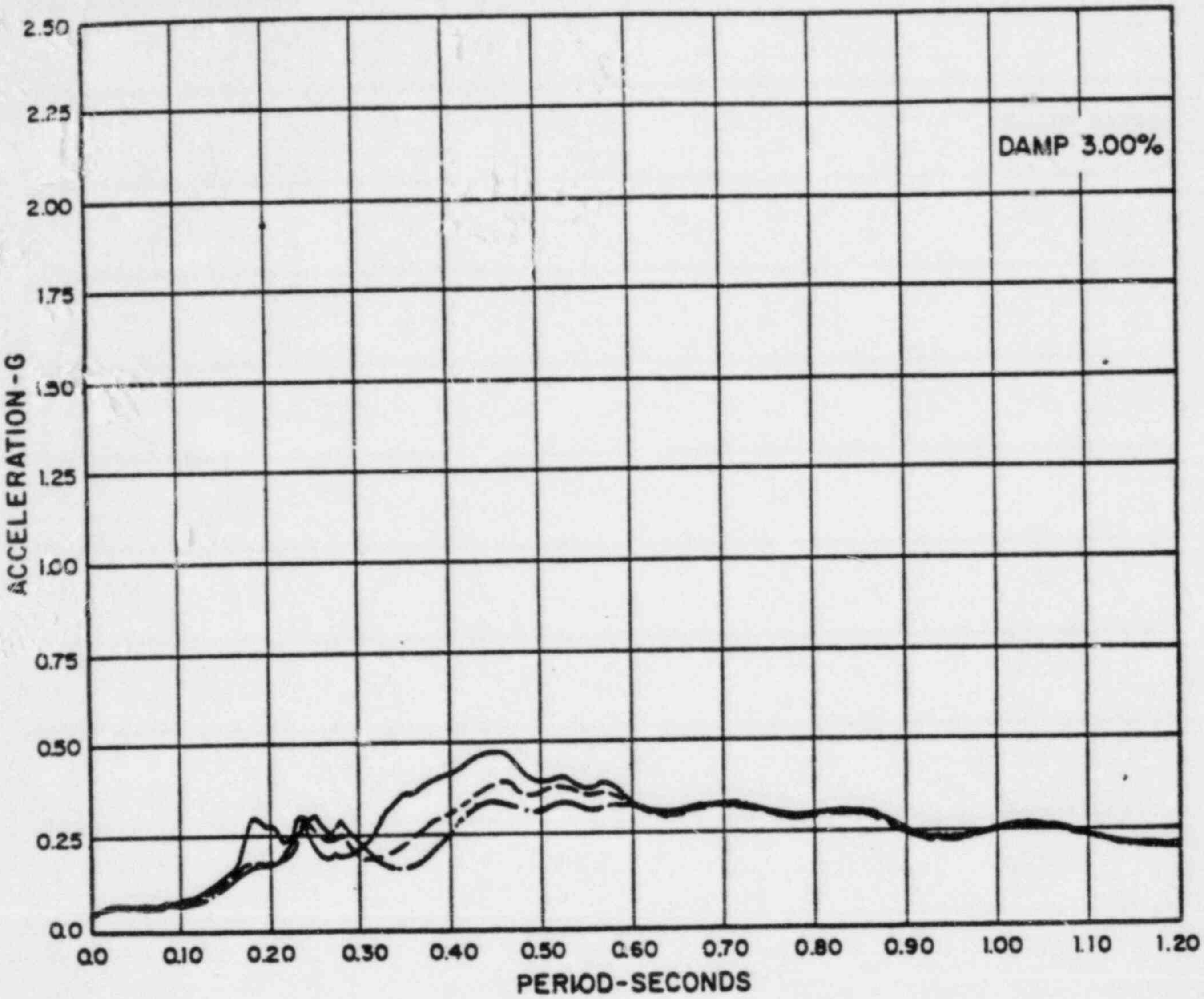


FIGURE 5-8
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM AT MAT
BEAVER VALLEY POWER STATION-UNIT 1



LEGEND

- LOW STRAIN G_{MAX}
- FIRST ITERATION FROM SHAKE
- LAST ITERATION FROM SHAKE

FIGURE 5-9
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM AT MAT
BEAVER VALLEY POWER STATION - UNIT 1

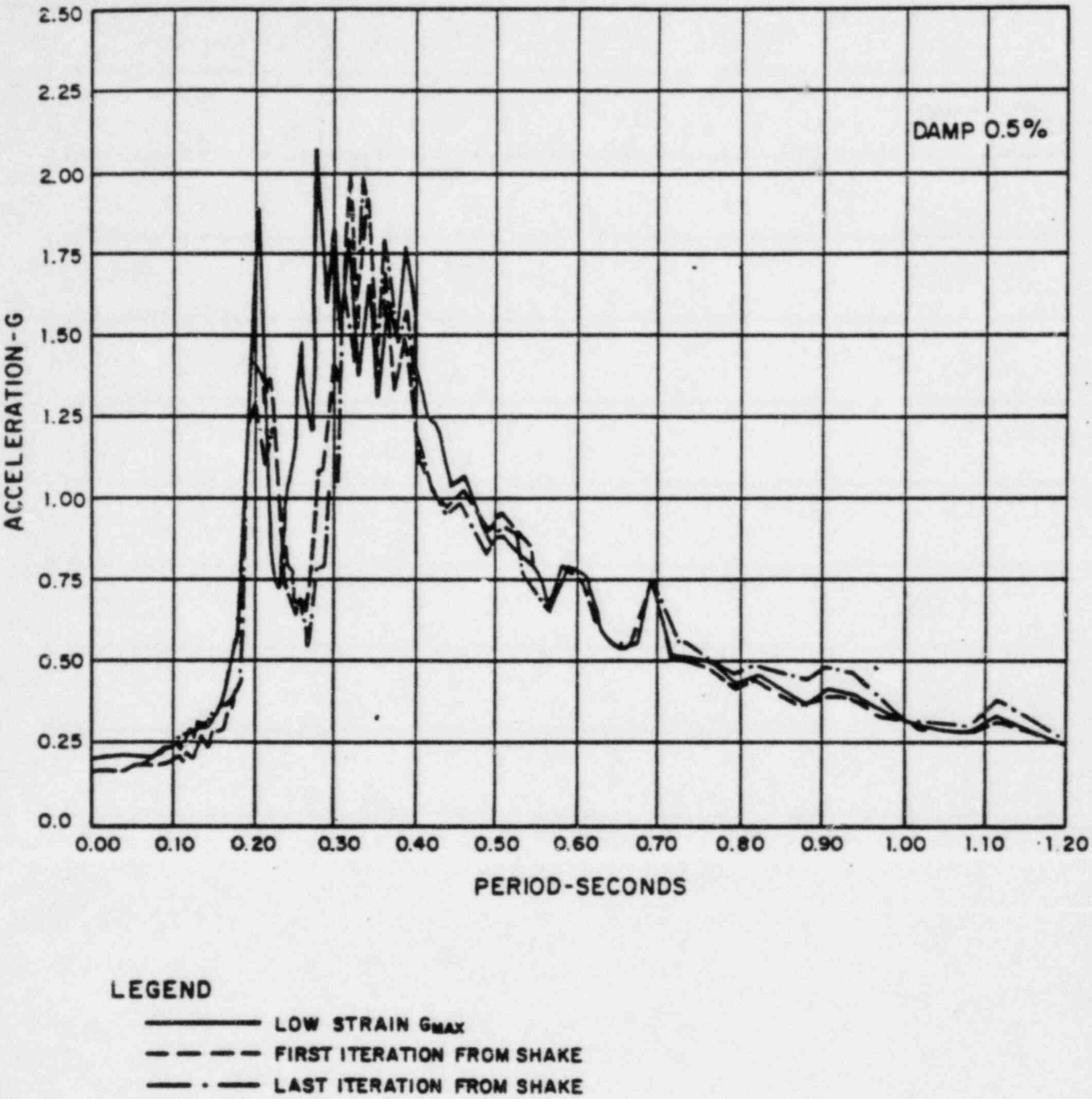


FIGURE 5-10
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM
AT OPERATING FLOOR
BEAVER VALLEY POWER STATION-UNIT 1

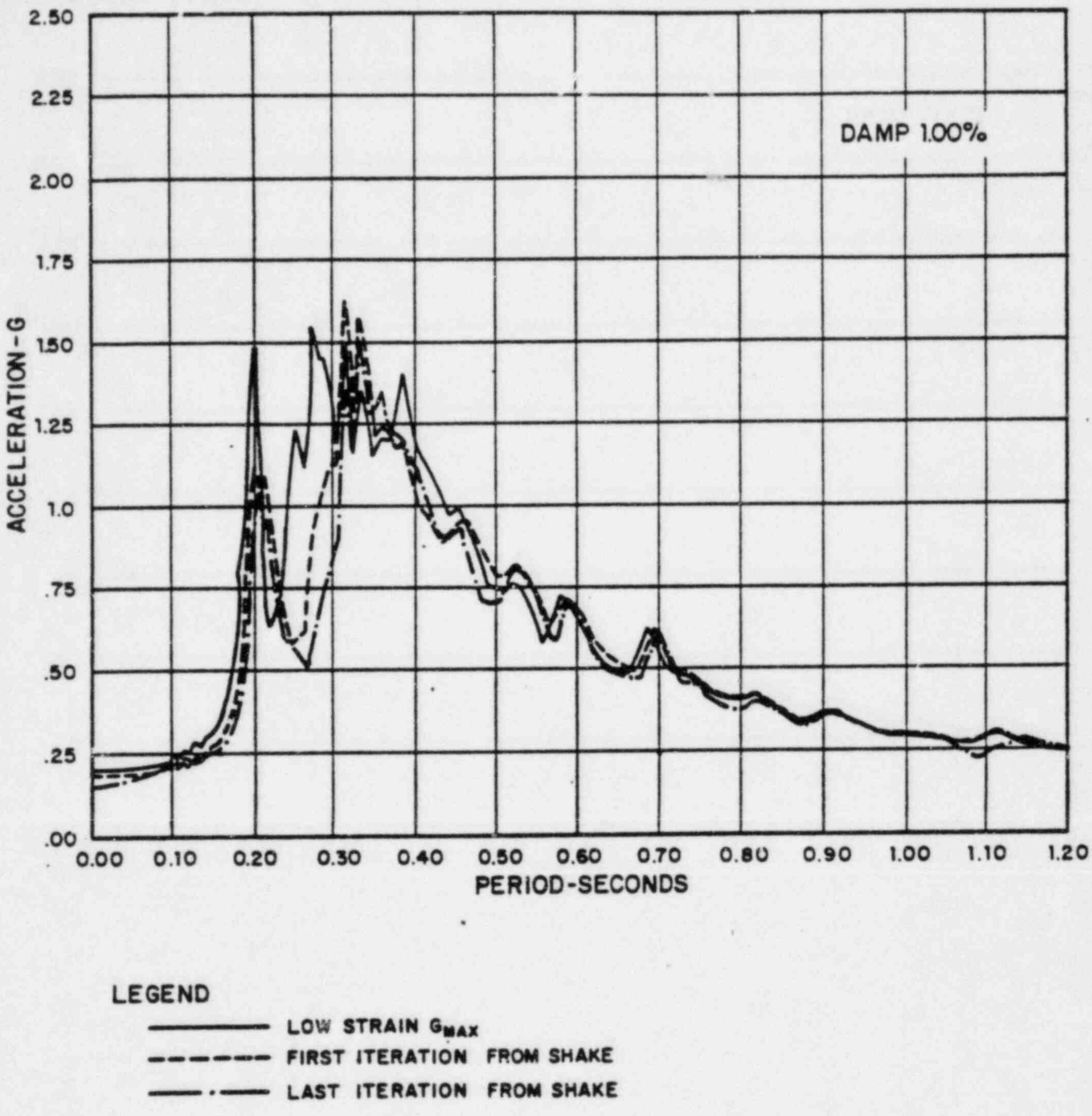


FIGURE 5-11
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM AT
OPERATING FLOOR
BEAVER VALLEY POWER STATION - UNIT 1

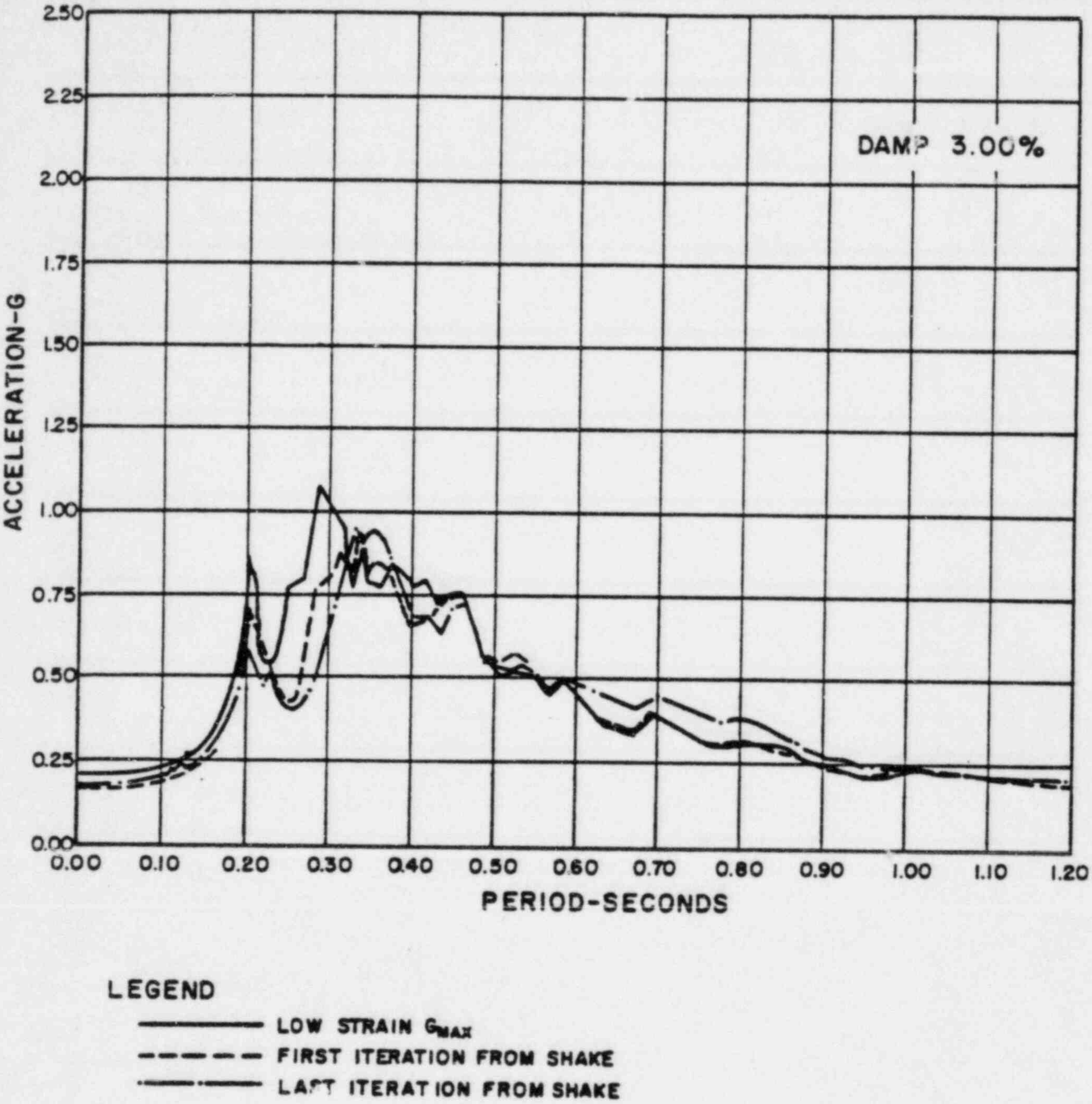


FIGURE 5-12
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM
AT OPERATING FLOOR
BEAVER VALLEY POWER STATION-UNIT 1

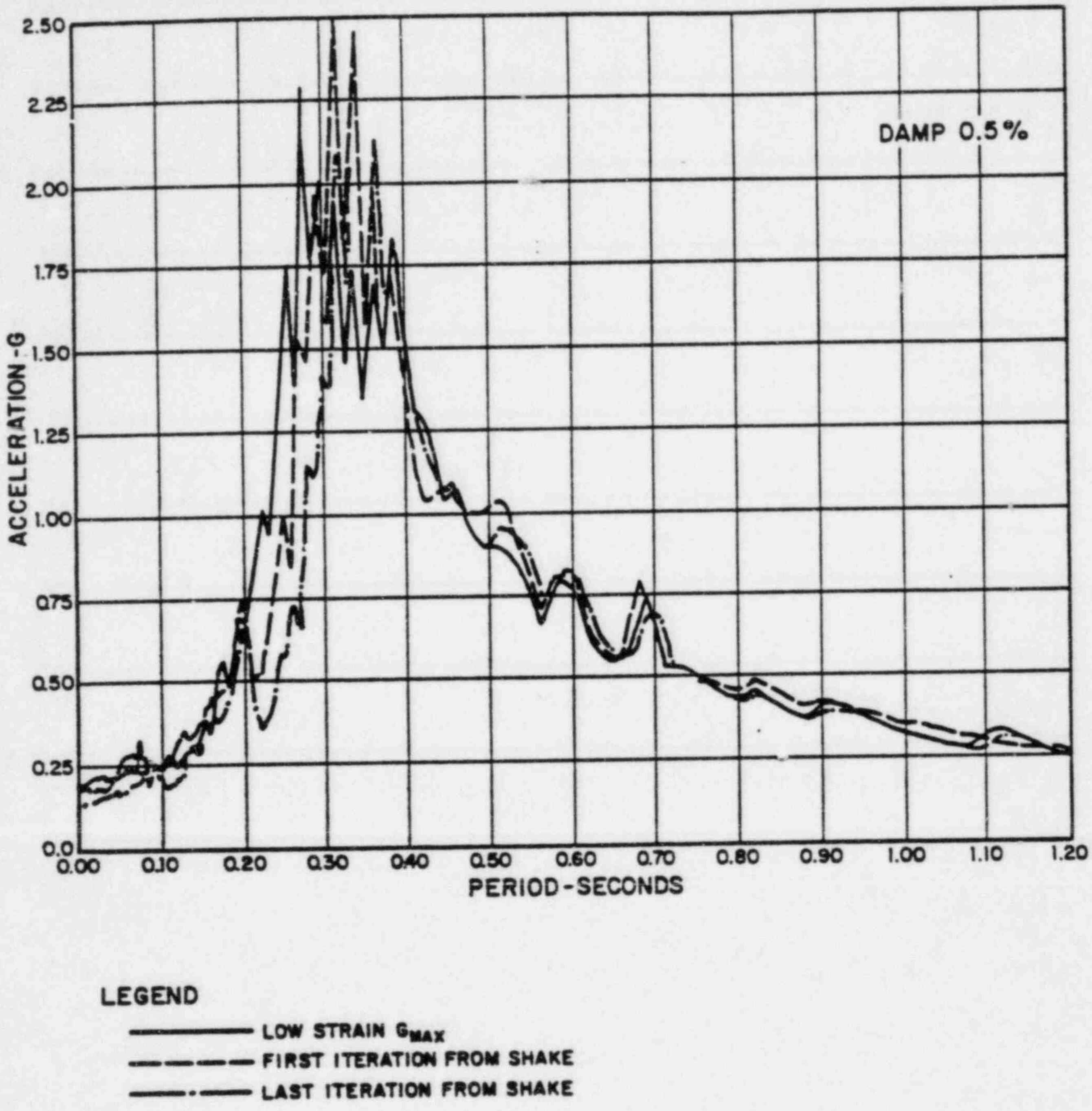


FIGURE 5-13
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM
AT SPRINGLINE
BEAVER VALLEY POWER STATION - UNIT 1

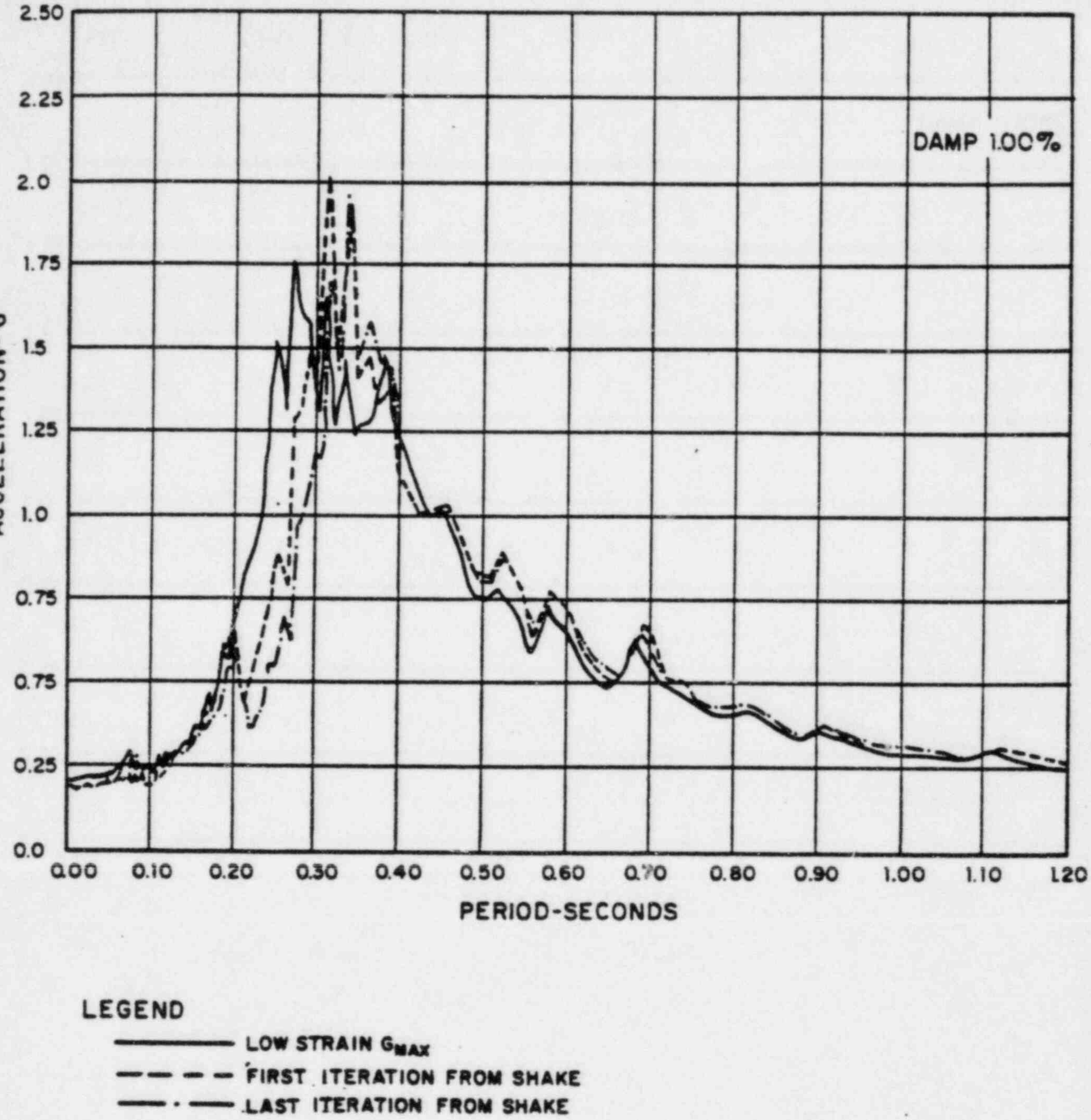


FIGURE 5-14
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM
AT SPRINGLINE
BEAVER VALLEY POWER STATION - UNIT 1

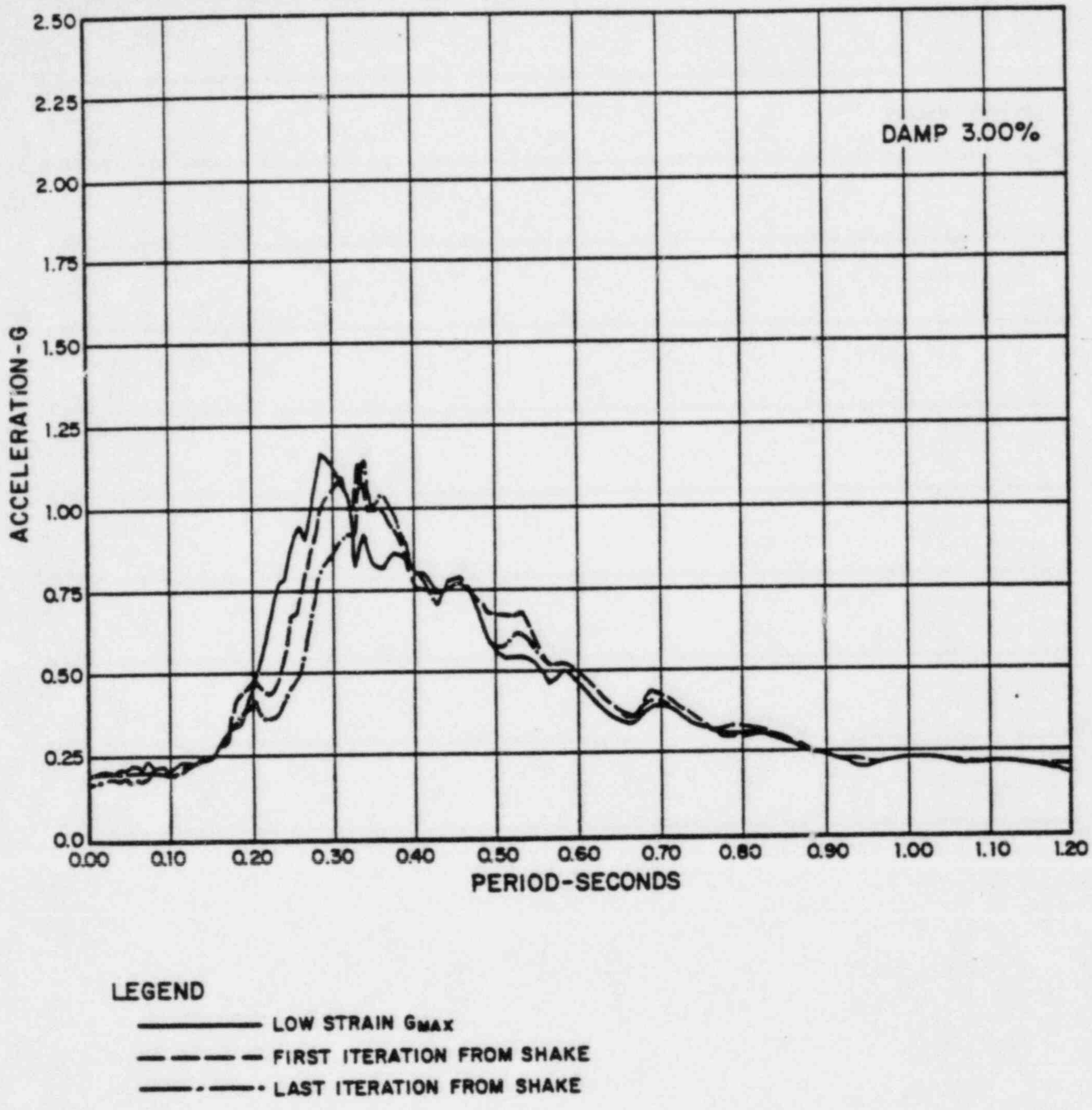


FIGURE 5-15
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM
AT SPRINGLINE
BEAVER VALLEY POWER STATION - UNIT 1

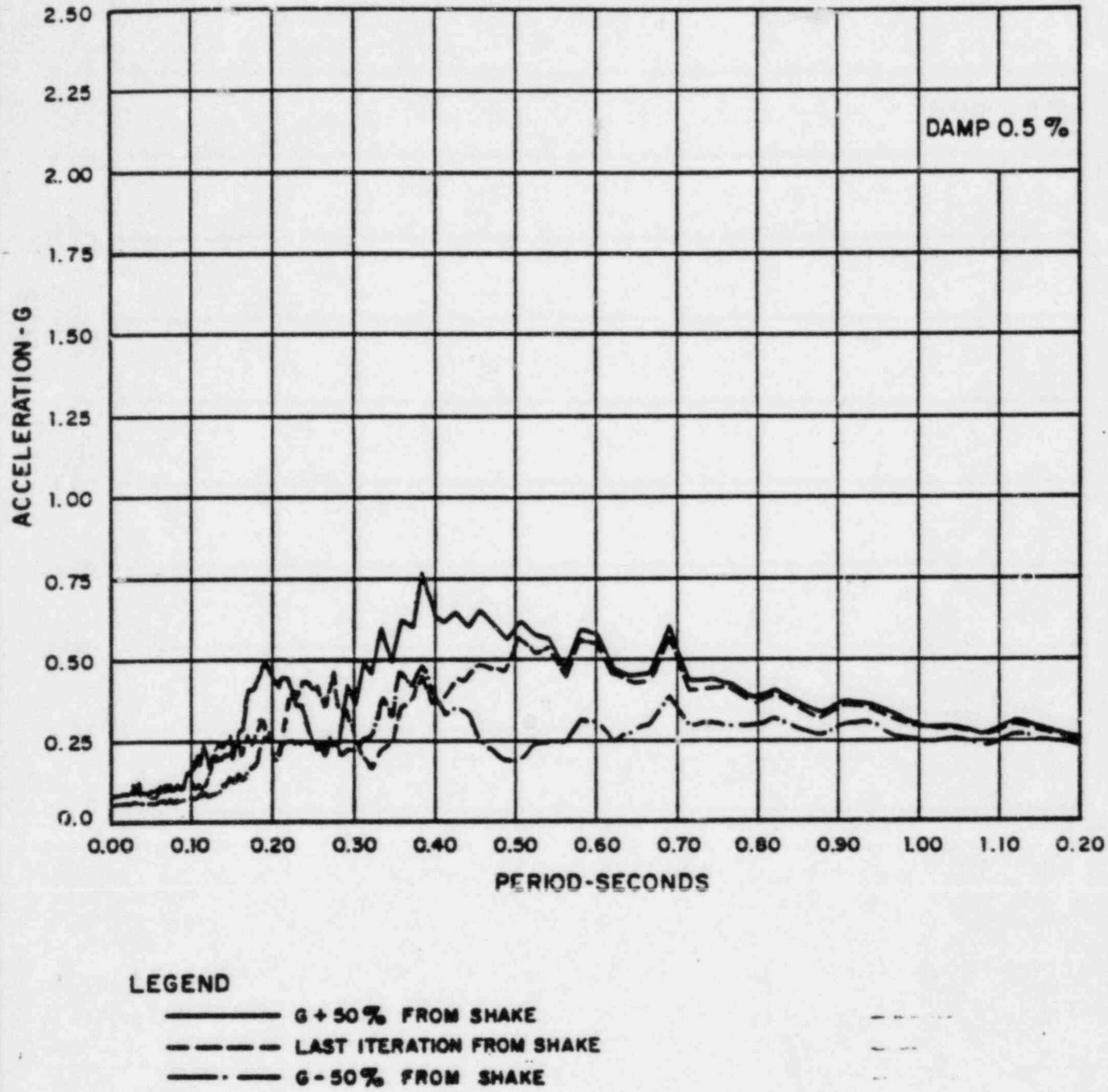


FIGURE 5-16
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM AT MAT
BEAVER VALLEY STATION-UNIT 1

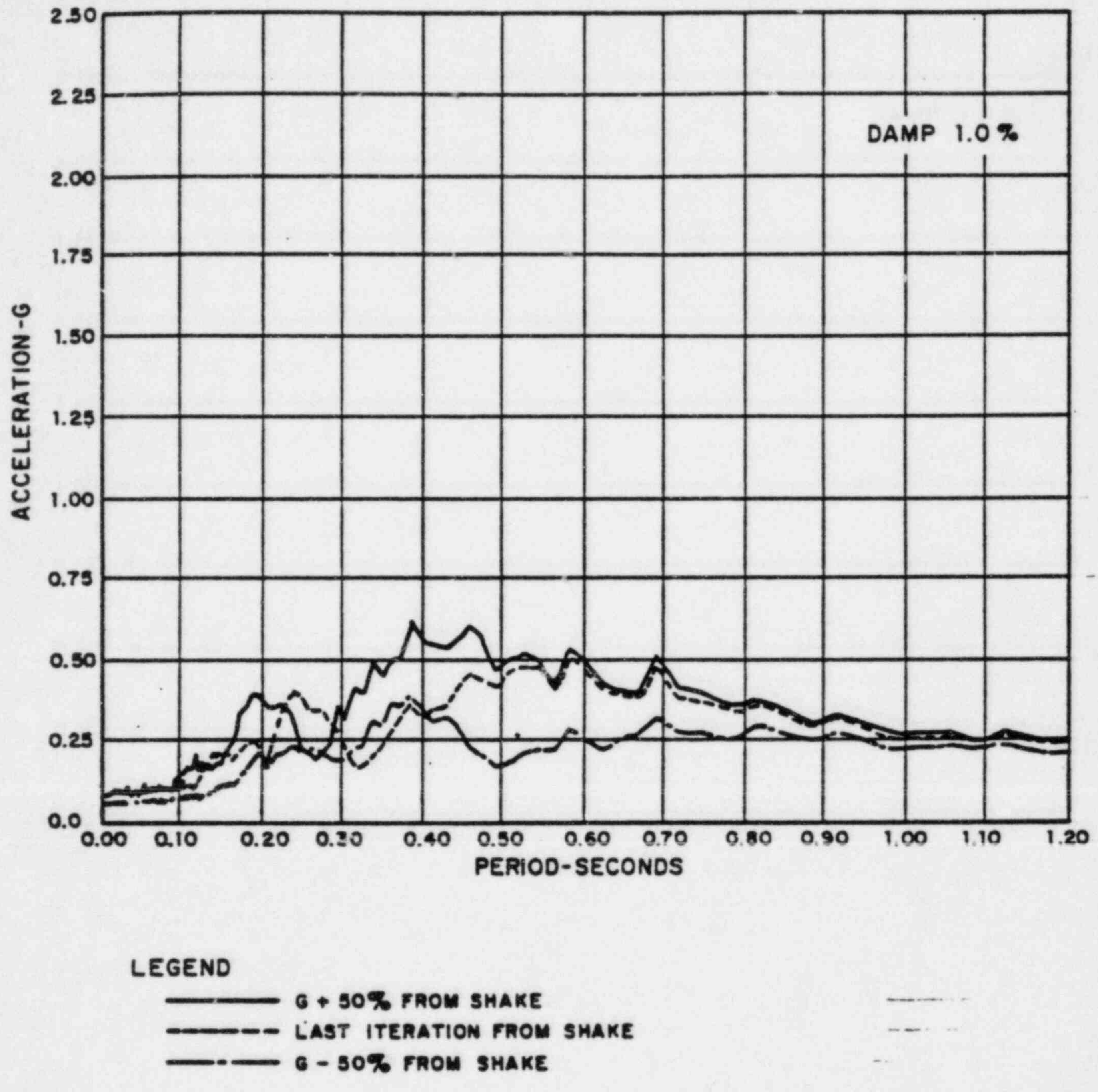


FIGURE 5-17
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM AT MAT
BEAVER VALLEY POWER STATION - UNIT 1

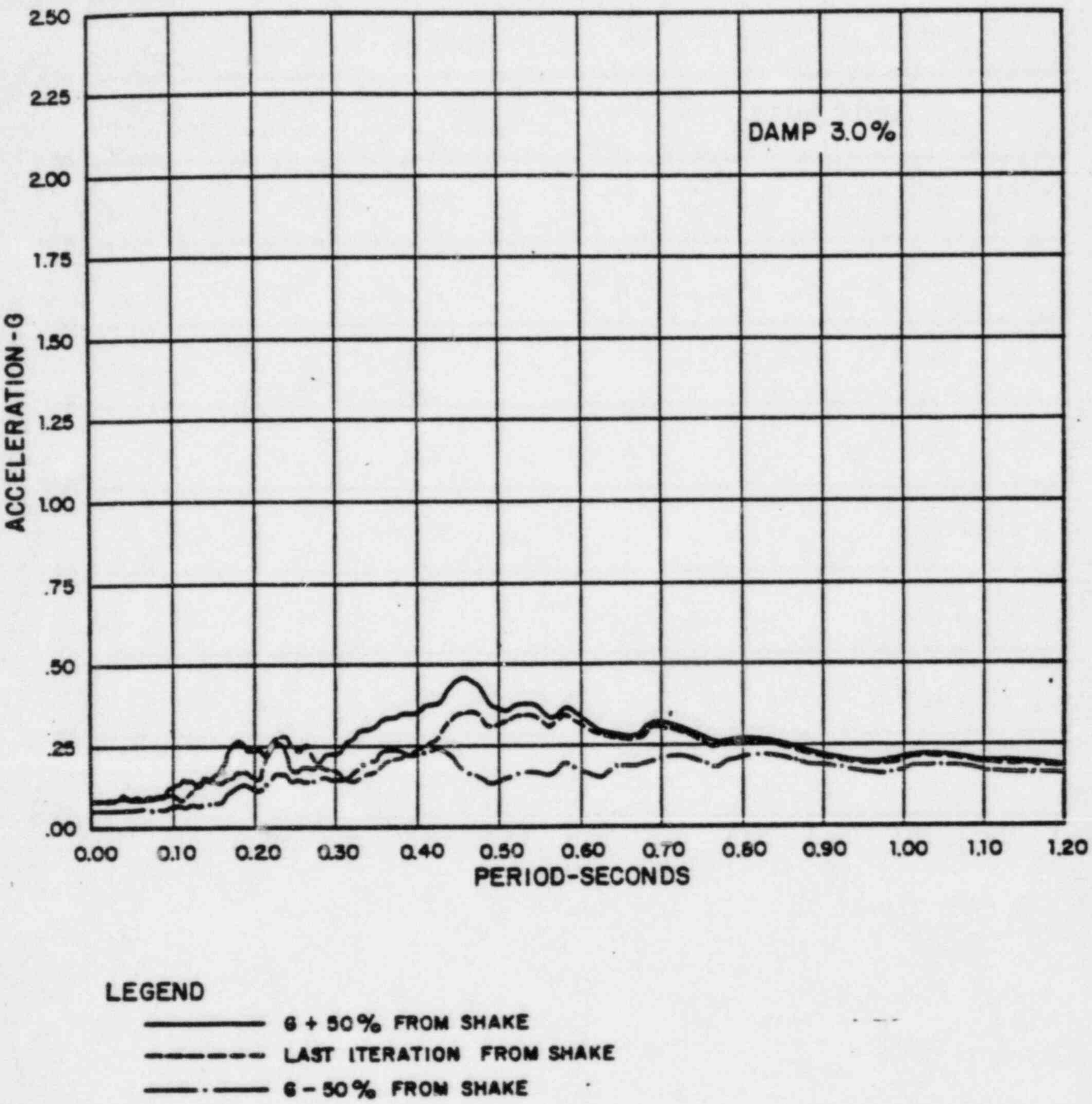


FIGURE 5-18
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM AT MAT
BEAVER VALLEY POWER STATION - UNIT 1

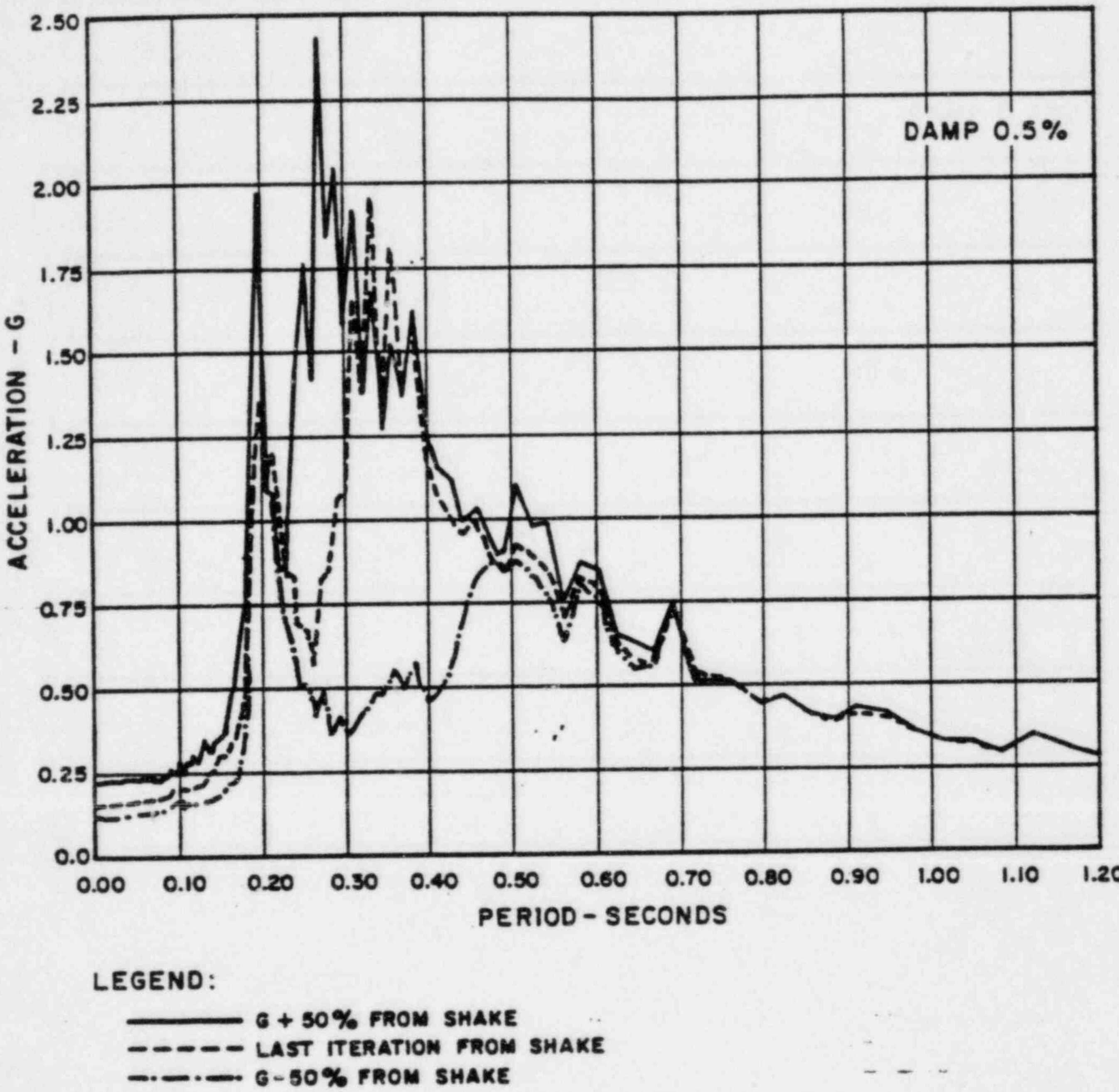


FIGURE 5-19
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM
AT OPERATING FLOOR
BEAVER VALLEY POWER STATION - UNIT 2

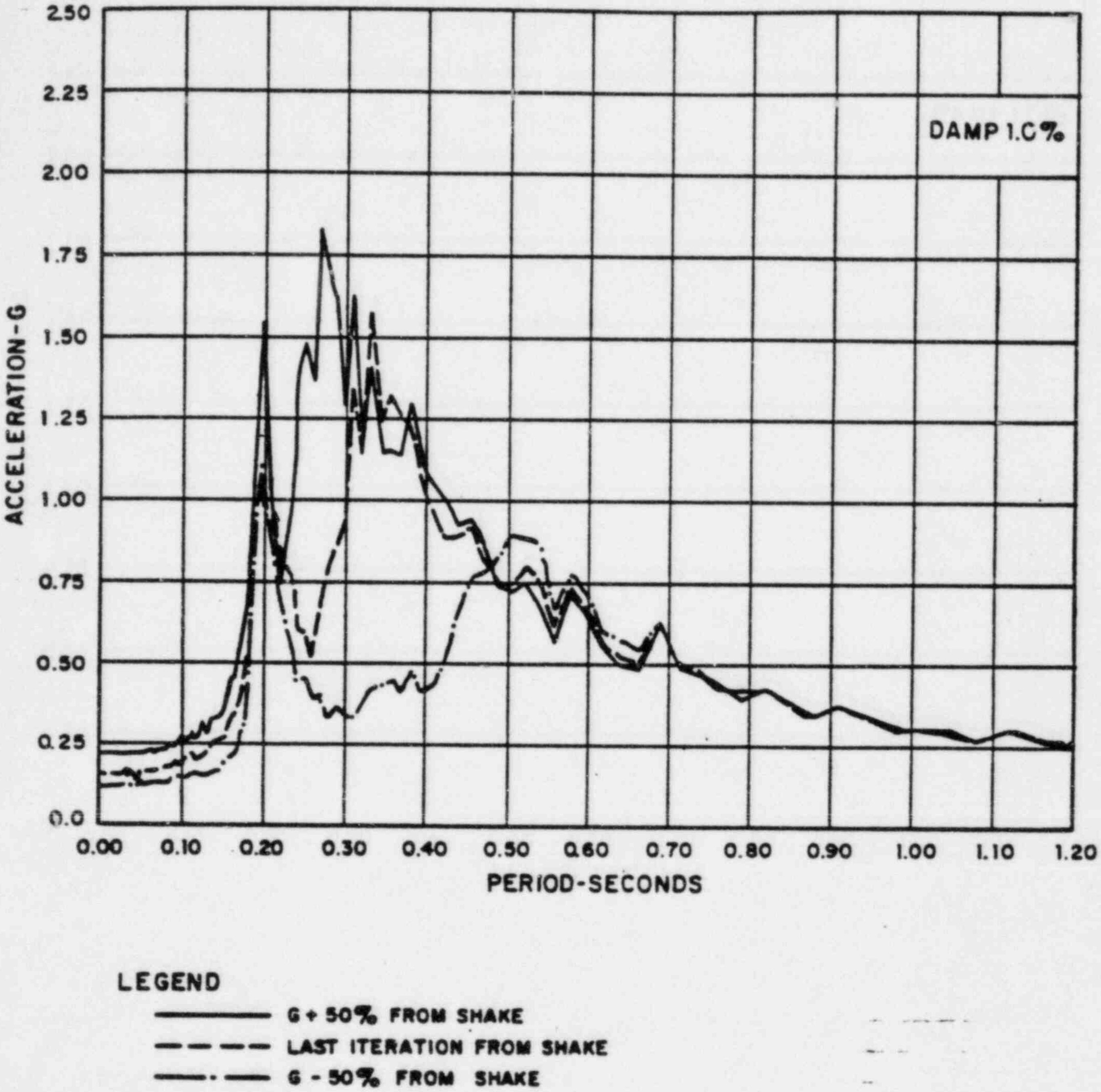


FIGURE 5-20
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM
AT OPERATING FLOOR
BEAVER VALLEY POWER STATION-UNIT 1

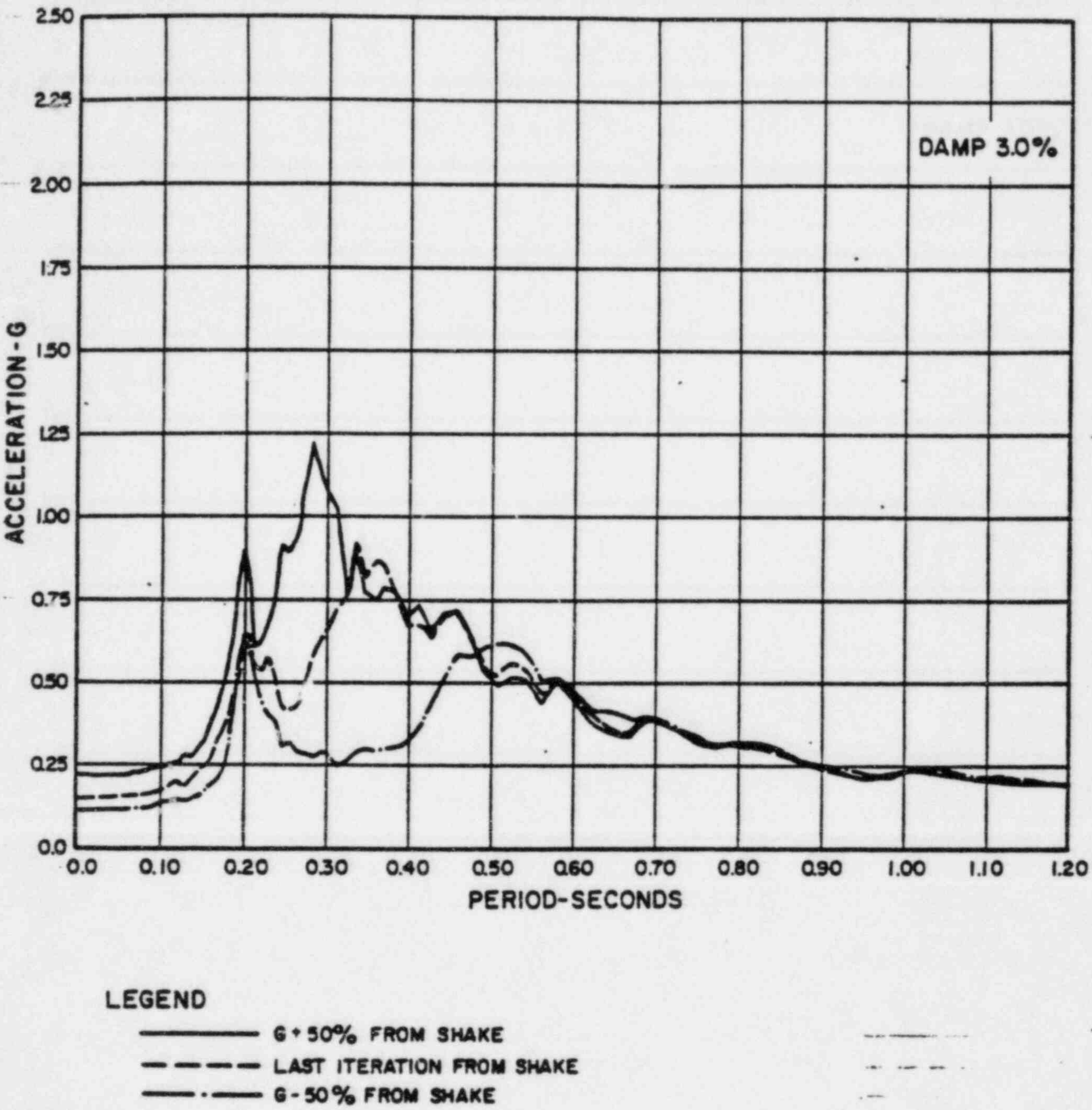


FIGURE 5-21
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM AT
OPERATING FLOOR
BEAVER VALLEY POWER STATION - UNIT 1

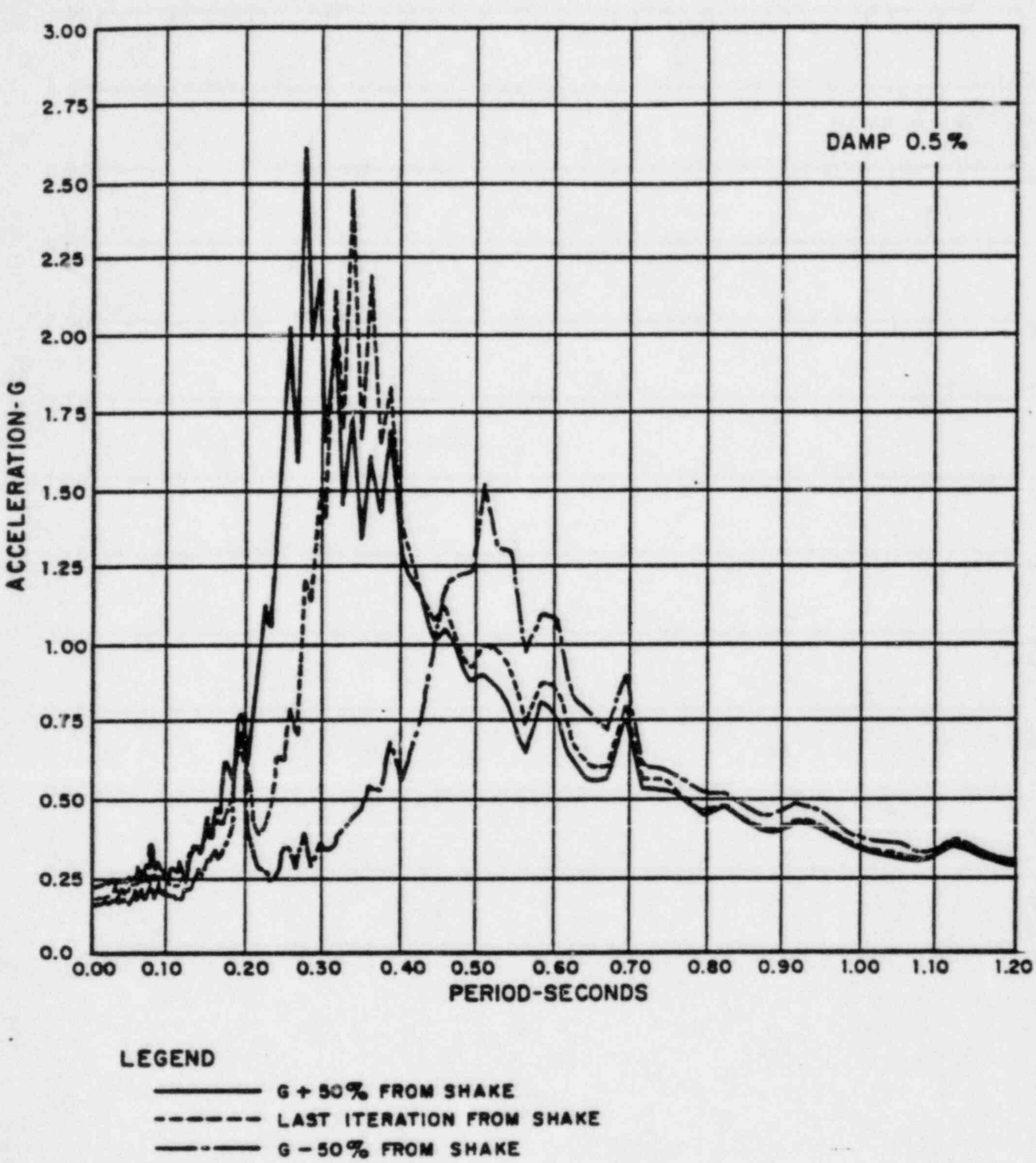


FIGURE 5-22
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM
AT SPRINGLINE
BEAVER VALLEY POWER STATION - UNIT 1

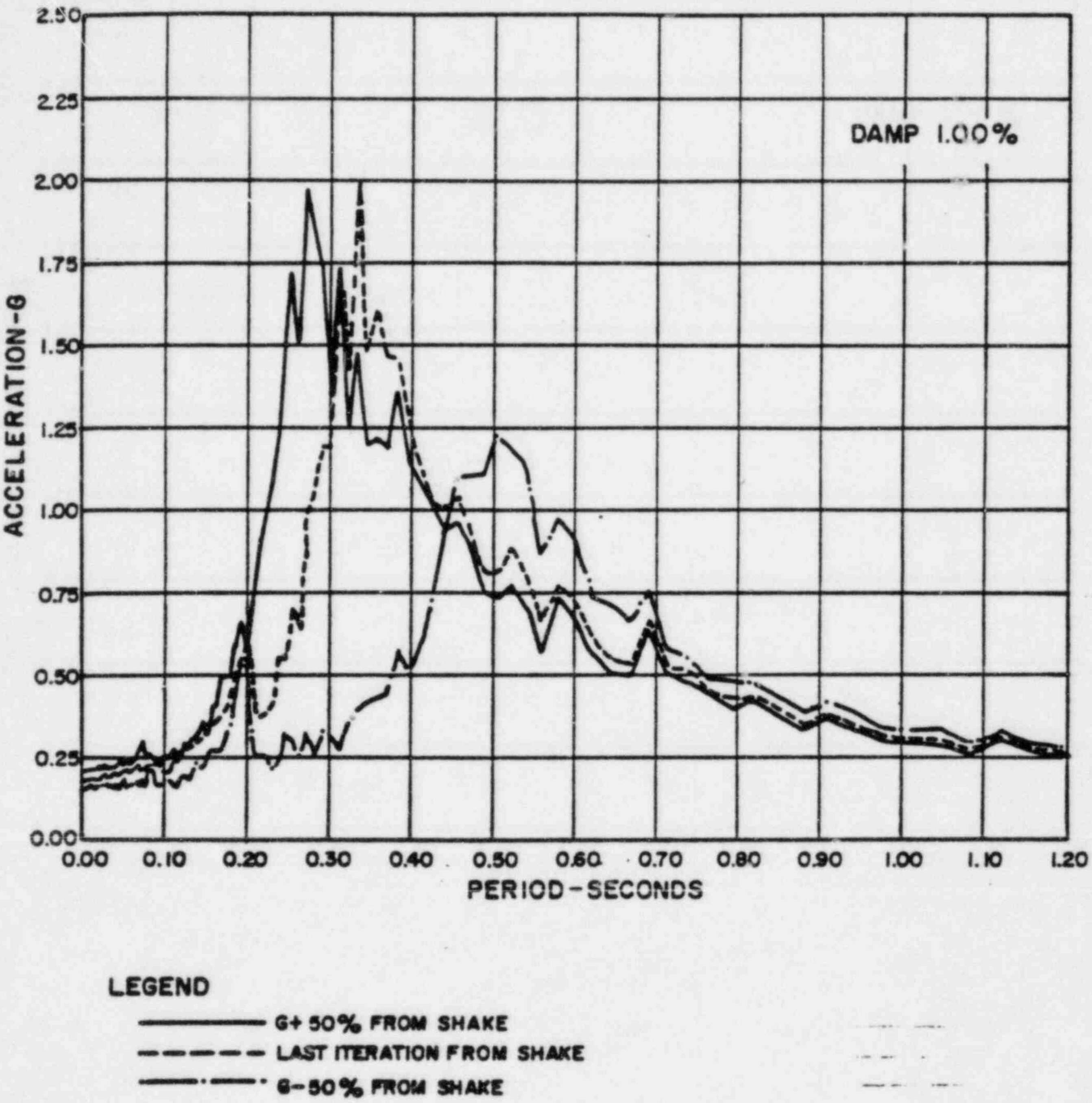


FIGURE 5-23
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM AT SPRINGLINE
BEAVER VALLEY POWER STATION-UNIT 1

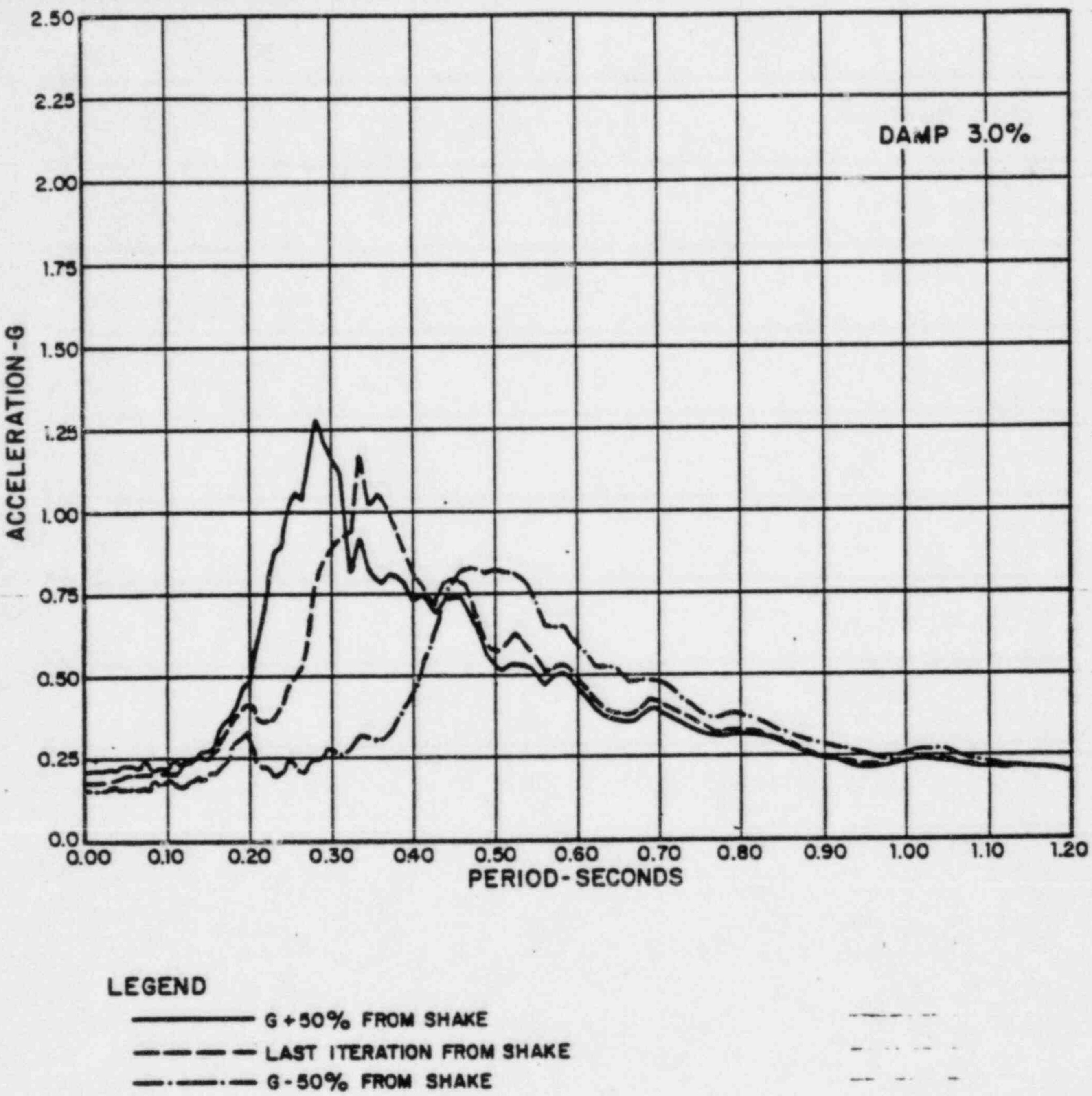


FIGURE 5-24
COMPARISON OF ARS FOR
SOIL PARAMETER VARIATIONS
HORIZONTAL RESPONSE SPECTRUM
AT SPRINGLINE
BEAVER VALLEY POWER STATION-UNIT 1

6.0 APPLICATION OF SEISMIC INPUT TO PIPE STRESS ANALYSIS

In general, seismic input to pipe stress analysis consists of inertia loads obtained through the application of amplified response spectra, and building seismic displacements applied appropriately at support points in accordance with the design load combinations for each piping system.

6.1 AMPLIFIED RESPONSE SPECTRA

Amplified response spectra for pipe stress analysis are developed and peak broadened as described in Section 4 of this report. Damping values for piping systems are 0.5 percent for the OBE and 1.0 percent for the DBE.

When all support points of a piping problem are located within the same structure, the amplified response spectrum which is closest to and higher in elevation than the center of mass of the piping system is applied in the analysis. For piping routed between buildings, an enveloped response spectrum representing the highest accleration for all periods is used.

6.2 BUILDING DISPLACEMENTS

Relative seismic structural displacements within a building, as determined from the building seismic analysis, are used as inputs to support motion of piping systems and are considered as static boundary displacements in the piping analysis. For piping running between buildings, the relative support motion includes the effect of each building's motion taken out of phase; this is the most conservative approach.

7.0 SOIL STRUCTURE INTERACTION ANALYSIS IN THE ORIGINAL DESIGN

This section is included because of a request by the Nuclear Regulatory Commission, Division of Operating Reactors, during meetings held with the Stone & Webster Engineering Corporation at Bethesda, Md. on March 16 and 17, 1979. The basis for material presented in this section is described in Section B.1.2, entitled "Seismic Design" of Appendix B of the Beaver Valley Power Station FSAR.[1]

This section provides comparisons of ARS for the containment structure at the operating floor (Figure 7-1) and springline (Figure 7-2) calculated by (1) the time history method using a maximum modal damping of 7 percent, shown by the dashed line, and (2) by the time history method using modal damping but including radiation damping due to soil structure interaction, shown by the solid line. The analysis used to compute radiation damping is in accordance with procedures described by Whitman.[2]

Table 7-1 shows weighted modal damping values used in the time history analyses which were performed to calculate the ARS. These modal damping values were calculated in the manner suggested by Biggs and Whitman.[3]

The spring connected lumped mass model (including soil springs) used in these analyses is similar to that depicted in the FSAR,[1] Figure B.1-1.

The Helena E-W earthquake record normalized to .125 g was used as input in accordance with studies referenced in the FSAR,[1] Appendix B.

7.1 REFERENCES

1. Beaver Valley Power Station Unit 1 Final Safety Analysis Report, Appendix B, Section B.1.2, Seismic Design.

2. Whitman, R. V. Vibrations in Civil Engineering. Proceedings of a Symposium Organized by the British National Section of the International Association for Earthquake Engineering.

3. Biggs, J. M. and Whitman, R.V. Soil Structure Interaction in Nuclear Power Plants, Fluid Japanese Symposium on Earthquake Engineering, Nov. 1970.

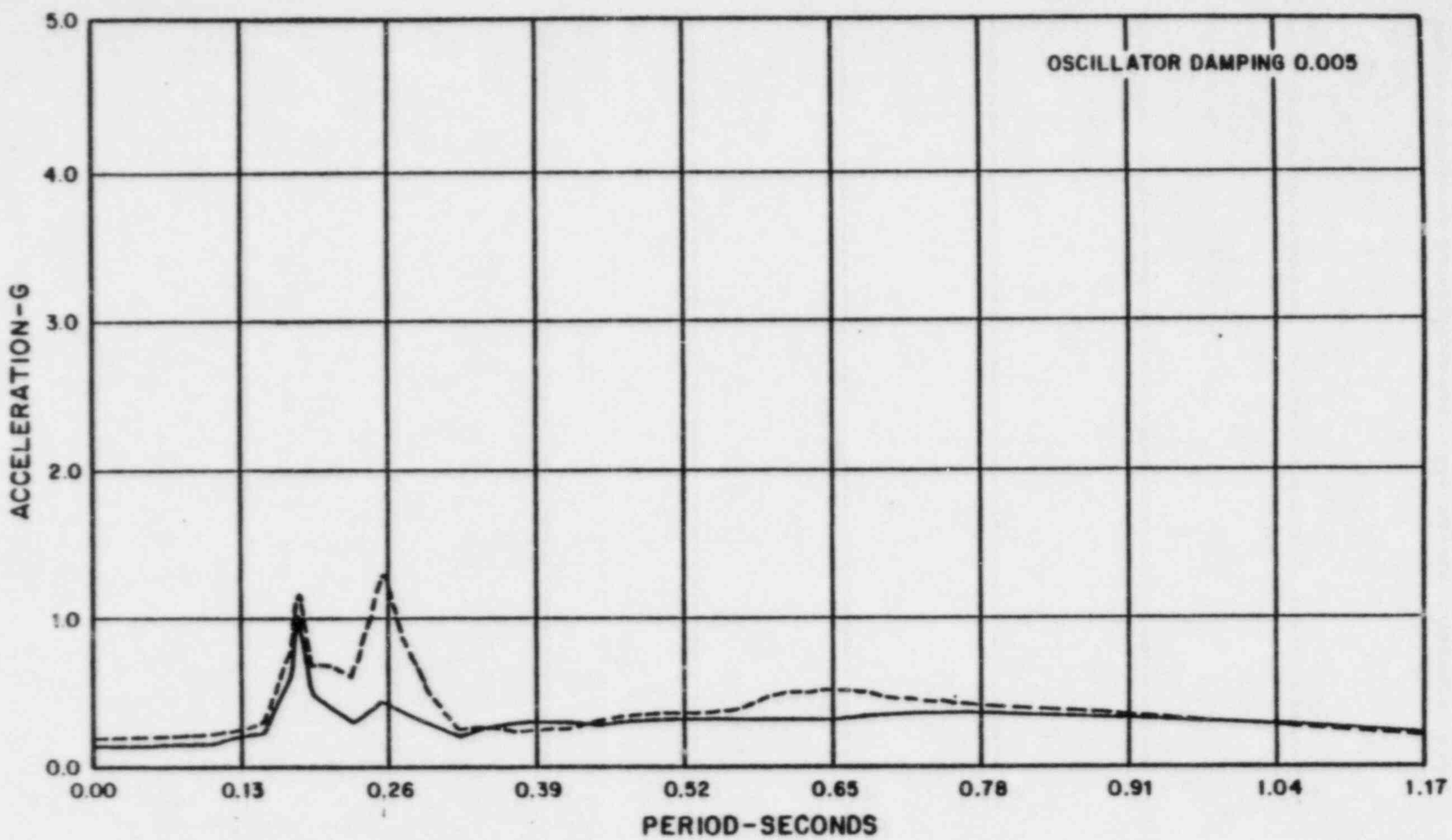
TABLE 7-1

MODAL DAMPING RATIOS USED IN THE TIME HISTORY ANALYSIS
USING SOIL SPRING STIFFNESSES*

Mode	Freq (cps)	Period (sec)	Modal Damping Case I	Modal Damping Case II	
1	1.61	0.622	0.07	0.1565	
2	2.847	0.3512	0.07	0.6536	Vertical
3	3.83	0.2611	0.07	0.3059	
4	5.525	0.1843	0.041	0.041	
5	11.933	0.0838	0.022	0.022	
6	14.255	0.0701	0.0224	0.0224	
7	14.952	0.0669	0.0207	0.0207	
8	18.873	0.0530	0.02703	0.02703	
9	22.013	0.0454	0.0279	0.0279	Vertical
10	22.293	0.0448	0.0213	0.0213	
11	26.015	0.03844	0.0207	0.0207	
12	26.749	0.03738	0.0202	0.0202	
13	30.226	0.0331	0.0249	0.0249	Vertical
14	31.037	0.0322	0.0200	0.0200	
15	34.511	0.0289	0.0206	0.0206	

NOTE:

* Computed according to BV1 FSAR, Appendix B, page B.1-4.



LEGEND

——— TIME HISTORY METHOD WEIGHTED MODEL DAMPING INCLUDING RADIATION DAMPING

– – – – TIME HISTORY METHOD WEIGHTED MODEL DAMPING 0.07 MAXIMUM

FIGURE 7-1
COMPARISON OF ARS AT
THE OPERATING FLOOR
BY TIME HISTORY METHOD
BEAVER VALLEY POWER STATION-UNIT 1

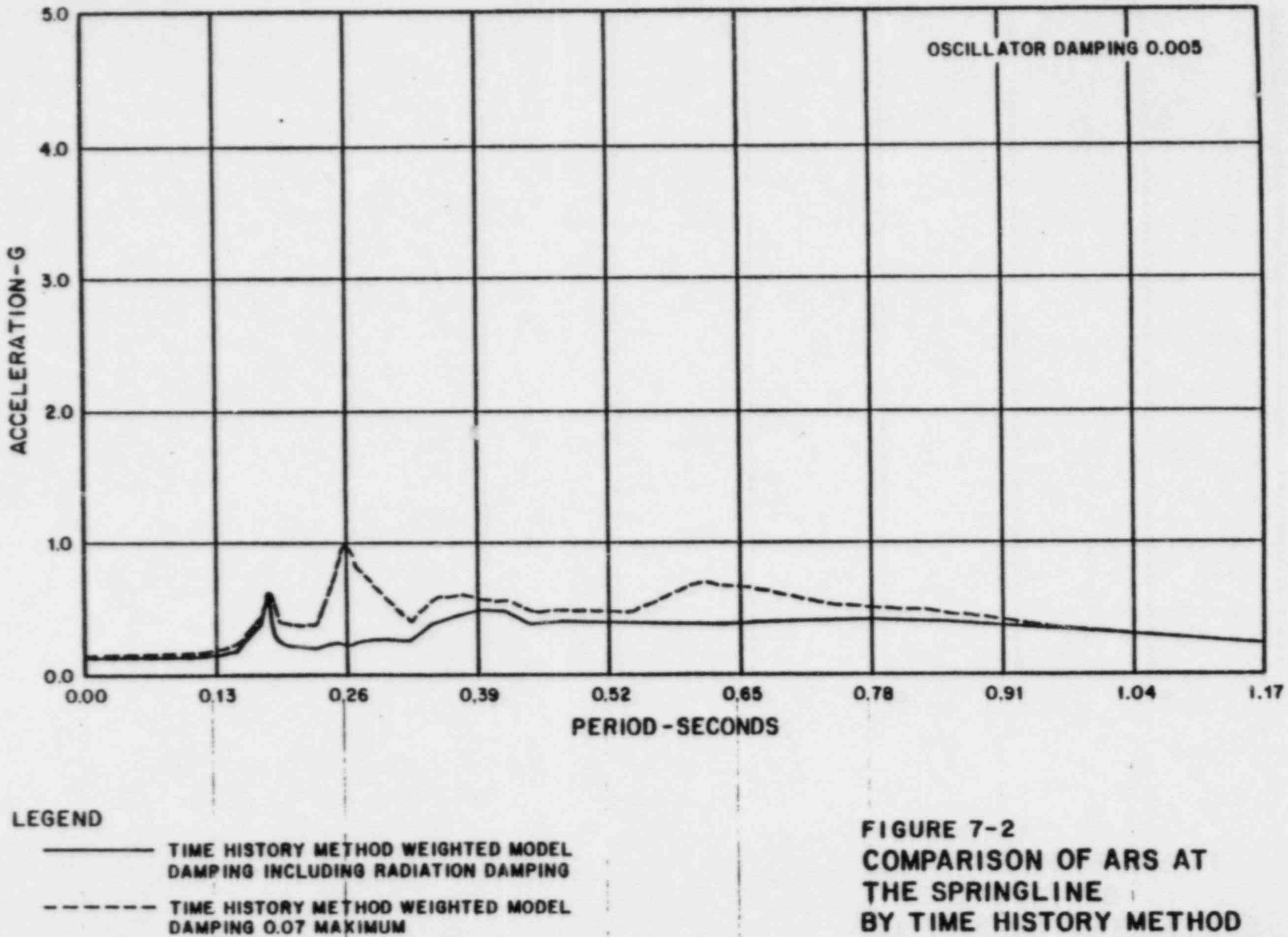


FIGURE 7-2
COMPARISON OF ARS AT
THE SPRINGLINE
BY TIME HISTORY METHOD
BEAVER VALLEY POWER STATION-UNIT 1

8.0 INVESTIGATION OF THE EFFECTS OF EARTHQUAKES SMALLER THAN THE DBE

Because the soil shear moduli used in the generation of ARS are functions of strain, the ARS are not direct linear functions of maximum ground acceleration. Therefore, it is theoretically possible that at some frequencies the ARS for some smaller earthquake exceed those of the DBE.

For the purpose of this study, an average strain compatible shear modulus for a range of peak horizontal ground accelerations from 0.01 g to 0.125 g was determined using SHAKE. The analyses were conducted for the free field profile using the Taft and El Centro accelerograms and Gmax values. The average shear modulus corresponding to each peak horizontal ground acceleration was determined by first averaging the shear moduli from the last iteration of SHAKE for the two accelerograms, then calculating the average value over the full depth of soil below the containment foundation elevation. The variation in average shear modulus versus peak horizontal ground acceleration is given in Figure 8-1.

The ARS generated for a range of soil moduli provide a basis for estimating the ARS for earthquakes smaller than the DBE. For example, the DBE shear moduli for the first iteration of SHAKE are actually consistent with a smaller earthquake. The maximum ground acceleration consistent with those moduli,

divided by 0.125 g yields a ratio which can be applied to the ARS resulting from analysis using the first iteration SHAKE moduli for the DBE.

The resulting family of ARS at the operating floor are enveloped by the DBE spectrum, demonstrating that the effects of the DBE are not exceeded by those of smaller earthquakes (Figure 8-2). Therefore, it can be concluded that the stresses in piping due to the DBE are not exceeded by those due to smaller earthquakes.

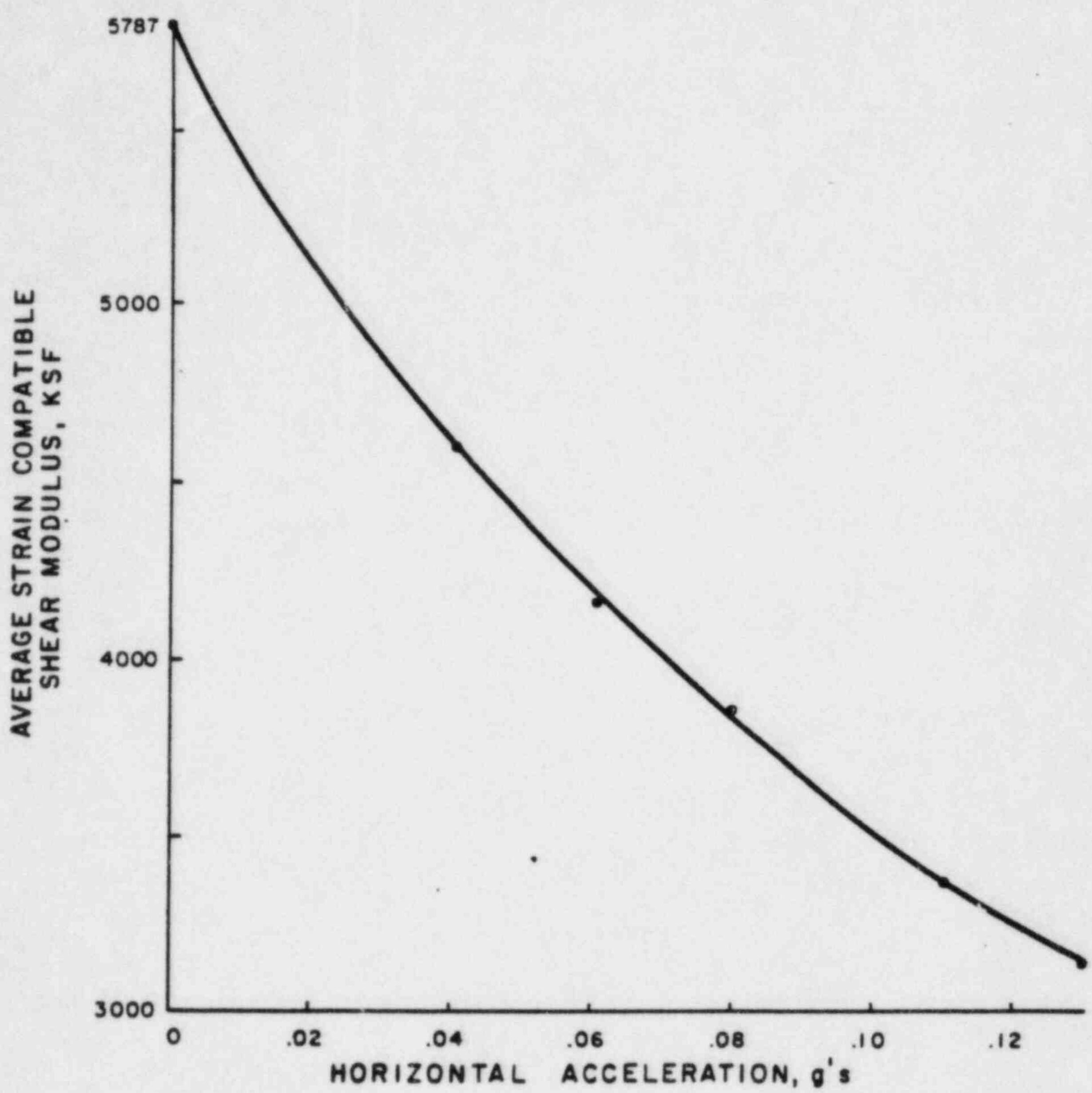


FIGURE 8-1
VARIATION OF SHEAR MODULUS
WITH GROUND ACCELERATION
BEAVER VALLEY POWER STATION-UNIT 1

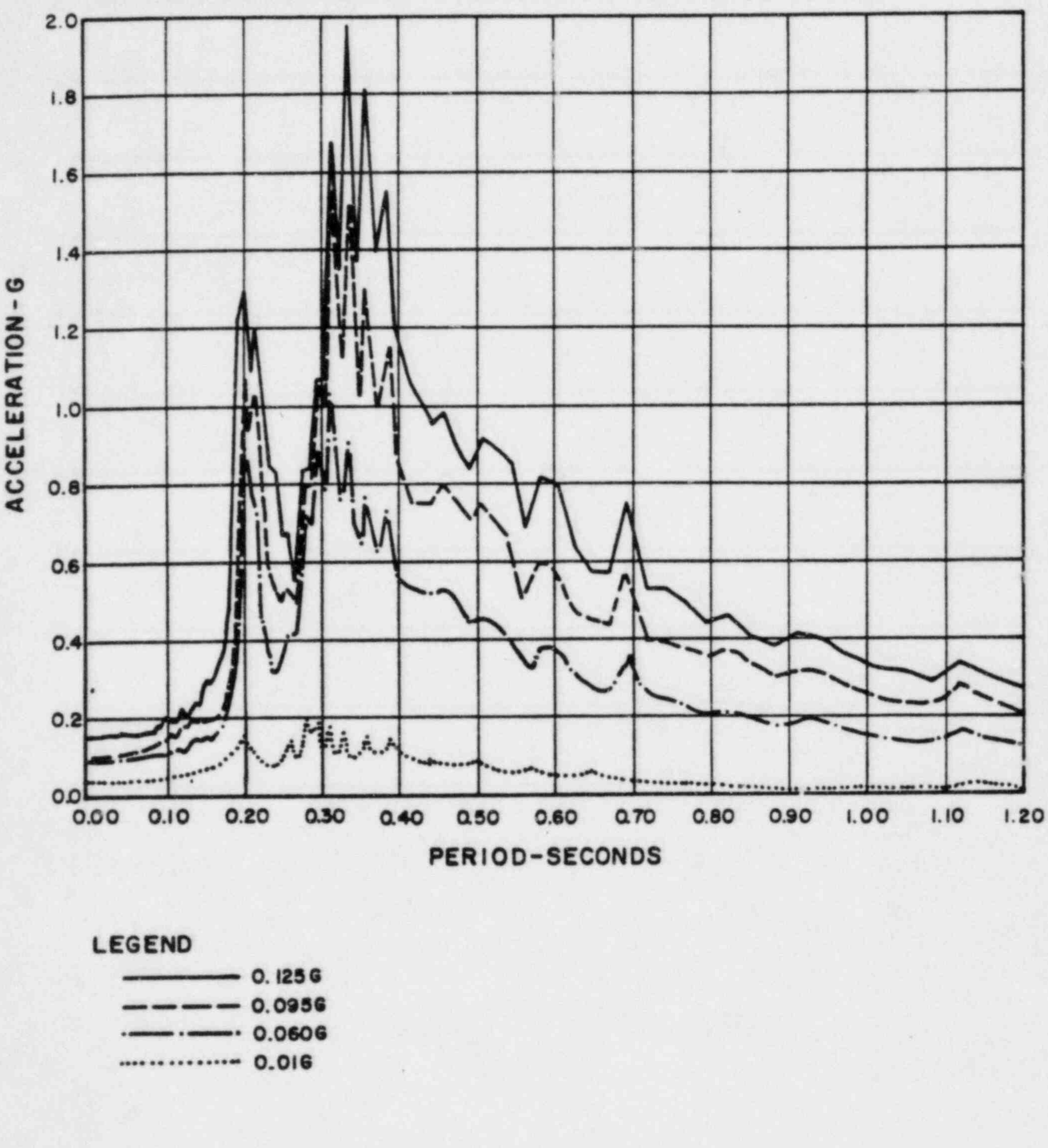


FIGURE 8-2
SEISMIC ANALYSIS OF CONTAINMENT
HORIZONTAL SSE

HORIZONTAL RESPONSE SPECTRUM
AT OPERATING FLOOR
BEAVER VALLEY POWER STATION-UNIT 1

9.0 CONCLUSIONS

Based upon the data and studies in this report, the following conclusions can be drawn about the effects of soil-structure interaction (SSI) analysis on amplified response spectra (ARS) at the Beaver Valley Power Station site.

9.1 USE OF SOIL-STRUCTURE INTERACTION

The principles and the methodology of SSI used to develop ARS are applicable to the Beaver Valley site and can be used with confidence to conservatively predict the seismic forces on piping systems.

9.2 SOIL PROPERTIES

The soil investigations made at the site to provide information for the licensing and design of Unit 1 are summarized in Section 2 of this report. The data from these investigations provide an adequate basis for the development of strain compatible soil properties for use in the SSI analysis.

Soil shear moduli values derived from in situ measurements at the Beaver Valley site are consistent with those obtained from empirical relationships.

The use of low strain shear moduli Gmax values for soil is not appropriate in developing ARS because earthquake-induced soil strain levels are approximately 2 orders of magnitude higher than low strain levels.

The use of low strain shear moduli values equal to ±50 percent of the Gmax to serve as a basis for developing a range of values in the strain compatible free-field soil profile is excessive. A more meaningful range would be a variation of the iterated strain compatible soil shear moduli values by ±50 percent of the mean value.

9.3 GROUND RESPONSE

Licensed ground response spectra and an enveloping artificial time history as input motion at the ground surface in the free field are appropriate for use in the SSI-ARS analysis.

9.4 AMPLIFIED RESPONSE ANALYSIS

The use of the multi-step analysis procedure described in Section 4 of this report provides an approach that includes conservatisms in stating the magnitude of the amplified acceleration values and allows development of the analysis in a series of logical steps convenient for an engineering evaluation of results.

9.5 COMPARISON OF RESULTS

The results of comparing the different methodologies and the FSAR earthquake with the Regulatory Guide 1.60 earthquake, and the effect of varying soil parameters lead to the following conclusions:

1. Comparison of ARS shown in Figures 5-1 through 5-3, calculated using the three-step analysis (REFUND/FRIDAY) and the one-step analysis (PLAXLY) show good agreement at all building levels with respect to frequencies at which peaks occur. The magnitudes of amplified acceleration agree reasonably well at lower levels in the structure. At higher levels, the REFUND/FRIDAY results generally exceed the PLAXLY results. At some frequencies, the ARS calculated for the base mat by REFUND/FRIDAY have amplitudes less than those obtained from PLAXLY. Since the spectral amplitudes involved are small fractions of 1.0 g, there would be no serious consequences in using these spectra in pipe stress analysis. Nevertheless, it is concluded that base mat spectra will not be used in pipe stress analyses.

2. Comparisons of ARS shown in Figures 5-4 thru 5-6 made from Regulatory Guides 1.60 ground response spectra and 1.61 damping values and ARS calculated on the basis of the FSAR committed ground response spectra

and damping values indicate good agreement in amplitude and frequencies of the peaks.

3. A comparison of ARS for soil parameter variations in Figures 5-7 through 5-15 using low strain shear modulus (Gmax), first iteration SHAKE, and last iteration SHAKE soil properties shows little variation in amplitude and frequency of peaks.

4. Comparisons of ARS for soil parameter variations in Figures 5-16 through 5-24 using strain compatible soil properties from the last iteration of SHAKE based upon (a) the low strain shear modulus (Gmax) input to SHAKE, (b) Gmax plus 50 percent input to SHAKE, and (c) Gmax minus 50 percent input to SHAKE show some variation in amplitude and frequency of the maximum response.

5. Changes in the shear modulus of the soil change the frequencies at which the amplification function has its peaks. This shift in frequency is evident in the general shapes of the response spectra for different values of G. The exact frequencies of the specific individual peaks are influenced by the frequency content of the artificial earthquake, so that each individual peak appears in all spectra. However, the essential phenomenon displayed is a shift in

frequency of the amplification function, causing different pre-existing peaks to be selected for amplification.

6. The results show that ARS are not sensitive to torsion in the structure.

7. Spectra calculated using the three-step method, the FSAR earthquake, and the strain compatible free-field soil properties are an adequate basis for analysis of piping systems when peak broadened ±25 percent. Additional conservatism was directed by the NRC in the period range from .4 sec to .55 sec where amplitudes will be increased by 20 percent in accordance with their position confirmed in a letter dated May 25, 1979.

9.6 APPLICATION OF SEISMIC INPUT TO PIPE STRESS ANALYSIS

The application of seismic input to pipe stress analysis as defined in Section 6 of this report is conservative and serves as an adequate basis for reevaluation of the designated piping systems.

9.7 SOIL-STRUCTURE INTERACTION ANALYSIS

The effects of radiation damping due to soil-structure interaction analysis, as shown in Section 7 of this report, generally decreases the amplified acceleration valves, as discussed in Appendix B of the FSAR.

9.8 EFFECTS OF GROUND ACCELERATION ON ARS

The ARS resulting from the DBE are not exceeded by those of smaller earthquakes. Therefore, the inertial pipe stresses due to the DBE are an adequate basis for qualification of piping.

9.9 COMPUTER PROGRAM VERIFICATION

The computer programs used to generate the SSI ARS have been qualified by (1) comparison of results to those obtained from similar programs which are recognized and widely used; or (2) comparison of program results to those obtained by hand calculations or analytical results published in technical literature. These comparisons are shown for the SHAKE, PLAXLY, REFUND, KINACT, and FRIDAY programs in Section 10 of this report. Reasonable agreement is demonstrated for these computer programs.

10.1 SHAKE

SHAKE is a public domain computer program developed at the University of California and described by Schnabel, Lysmer, and Seed.[1] Stone & Webster has made a few changes in the program, principally the addition of plotter capability and improvement of some of the output, but the program in use for this work is essentially that described by Schnabel, et al.

The program solves the problem of vertically propagating shear waves in a layered medium. The values of shear modulus and damping for a particular layer depend on the average shear strain induced in that layer by the earthquake. The program iterates to obtain values of modulus and damping that are compatible with the strains and with curves of modulus and damping versus strain.

Although the program is well known and widely used, Stone & Webster has checked the results computed by the program against those developed independently by Roesset[2] and has also checked that the calculations of modulus and damping are internally consistent. For example, Figure 10.1-1 shows the comparison of the amplification functions from SHAKE and Roesset's analysis for the first iteration on the soil profile in Figure 10.1-2.

REFERENCES

1. Schnabel, P.B.; Lysmer, J.; and Seed, H.B. SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites, Earthquake Engineering Center, Report No. EERC 72-12, University of California, Berkeley, California, December 1972.

2. Roesset, J.M., Fundamentals of Soil Amplification. In: Seismic Design for Nuclear Power Plants, R.J. Hansen, ed., M.I.T. Press, Cambridge, Mass., 1970, pp 183-244.

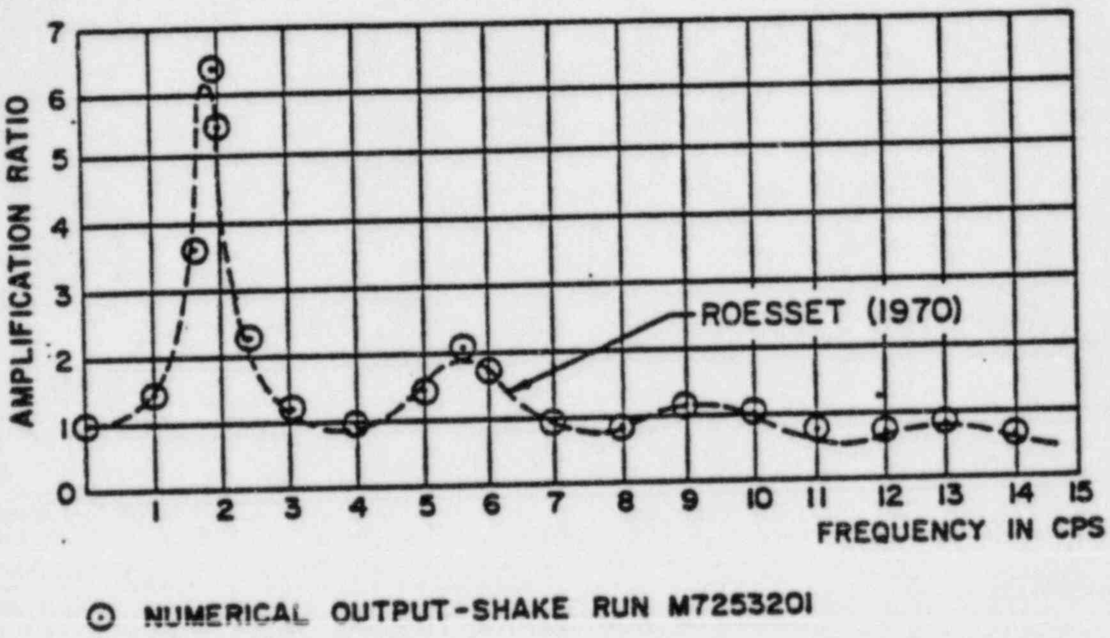


FIGURE 10.1-1
AMPLIFICATION FUNCTION OF SOIL
BEAVER VALLEY POWER STATION - UNIT 1

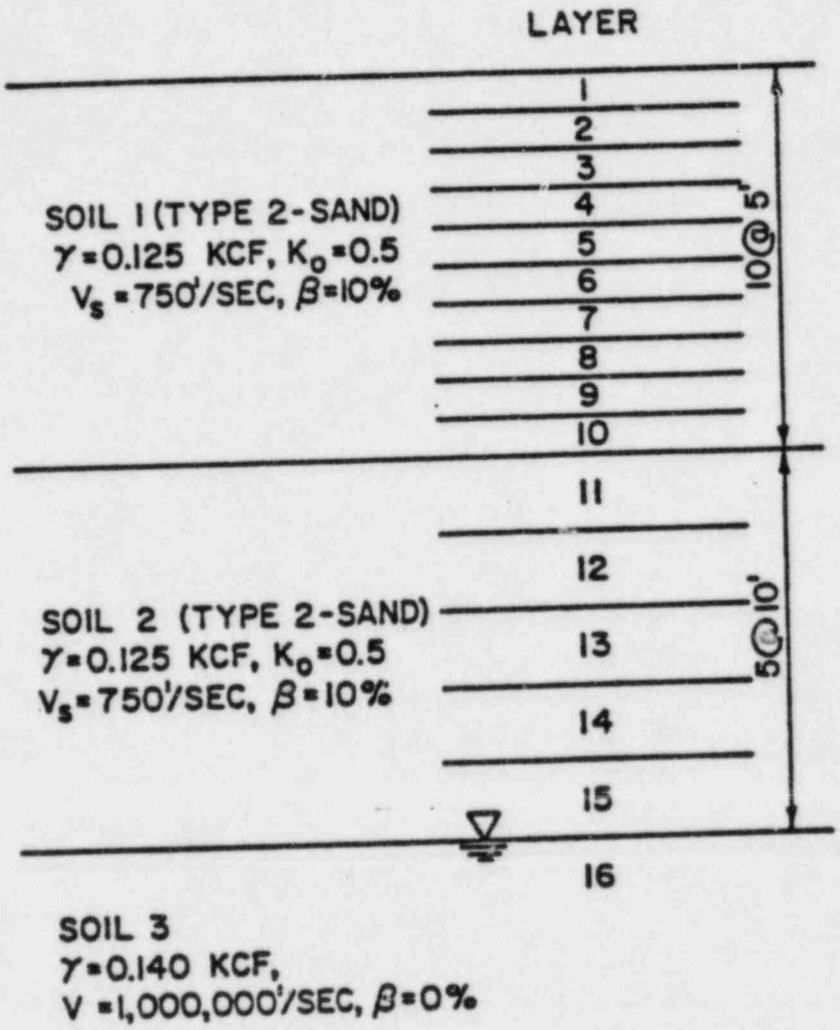


FIGURE 10.1-2
SOILS PROFILE
BEAVER VALLEY POWER STATION - UNIT 1

10.2 PLAXLY

PLAXLY is an isoparametric, plane-strain, finite element computer program used in seismic soil-structure analysis. The equations of motion are solved in the frequency domain.

A primary element in the PLAXLY solution is the consistent transmitting boundary modeling the layered far-field. This boundary avoids the unrealistic reflections associated with more simplistic "free" or "roller" lateral boundary conditions.

The principal limitations upon the program and its application are the following:

1. Geometry and material properties must be such that they can be satisfactorily modeled in two dimensions.

2. Properties of the layered far-field cannot change horizontally.

3. Base rock is assumed to be infinitely stiff.

4. Material properties are isotropic, linearly elastic.

For purposes of comparison, the results of PLAXLY and those of a similar program in the public domain, FLUSH (CDC Version 2.2), are shown in Figure 10.2-1. The PLAXLY flow diagram is shown in Figure 10.2-2.

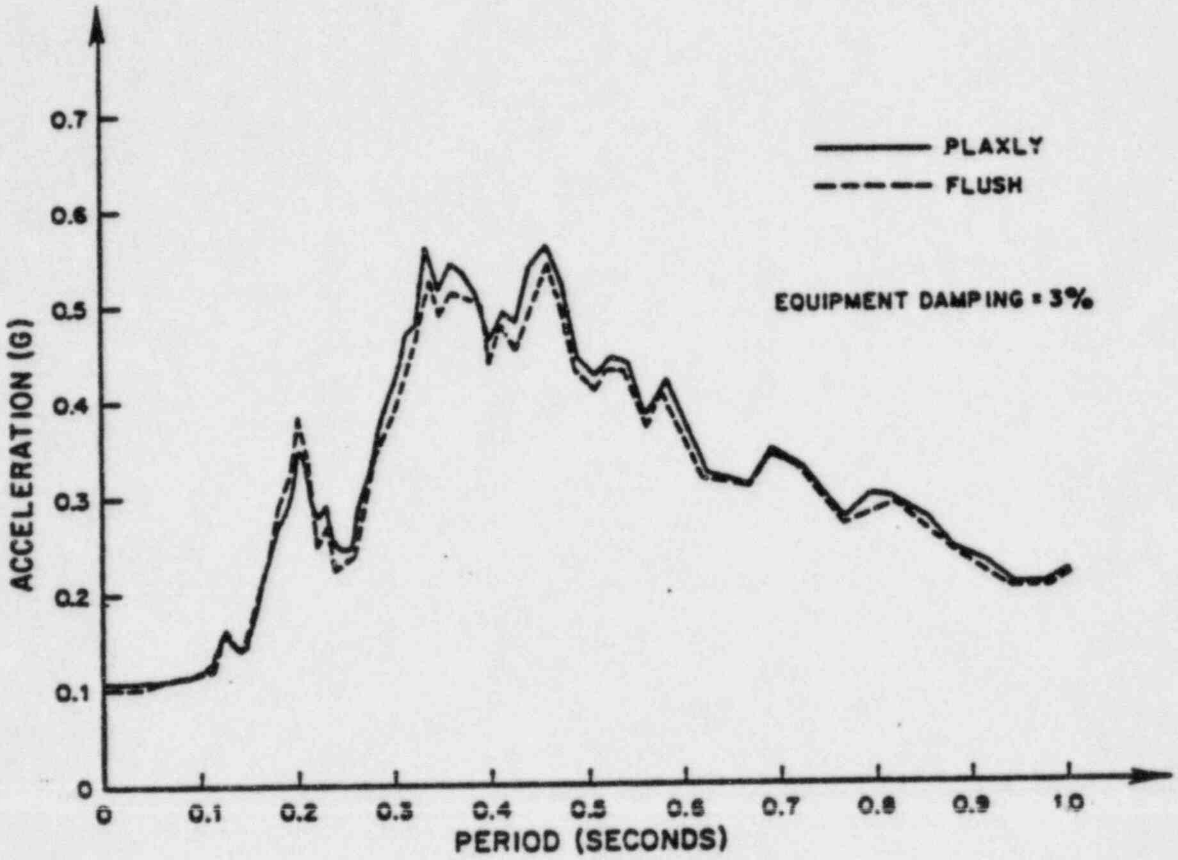


FIGURE 10.2-1
COMPARISON OF ARS BY PLAXLY
AND FLUSH AT OPERATING FLOOR
BEAVER VALLEY POWER STATION - UNIT 1

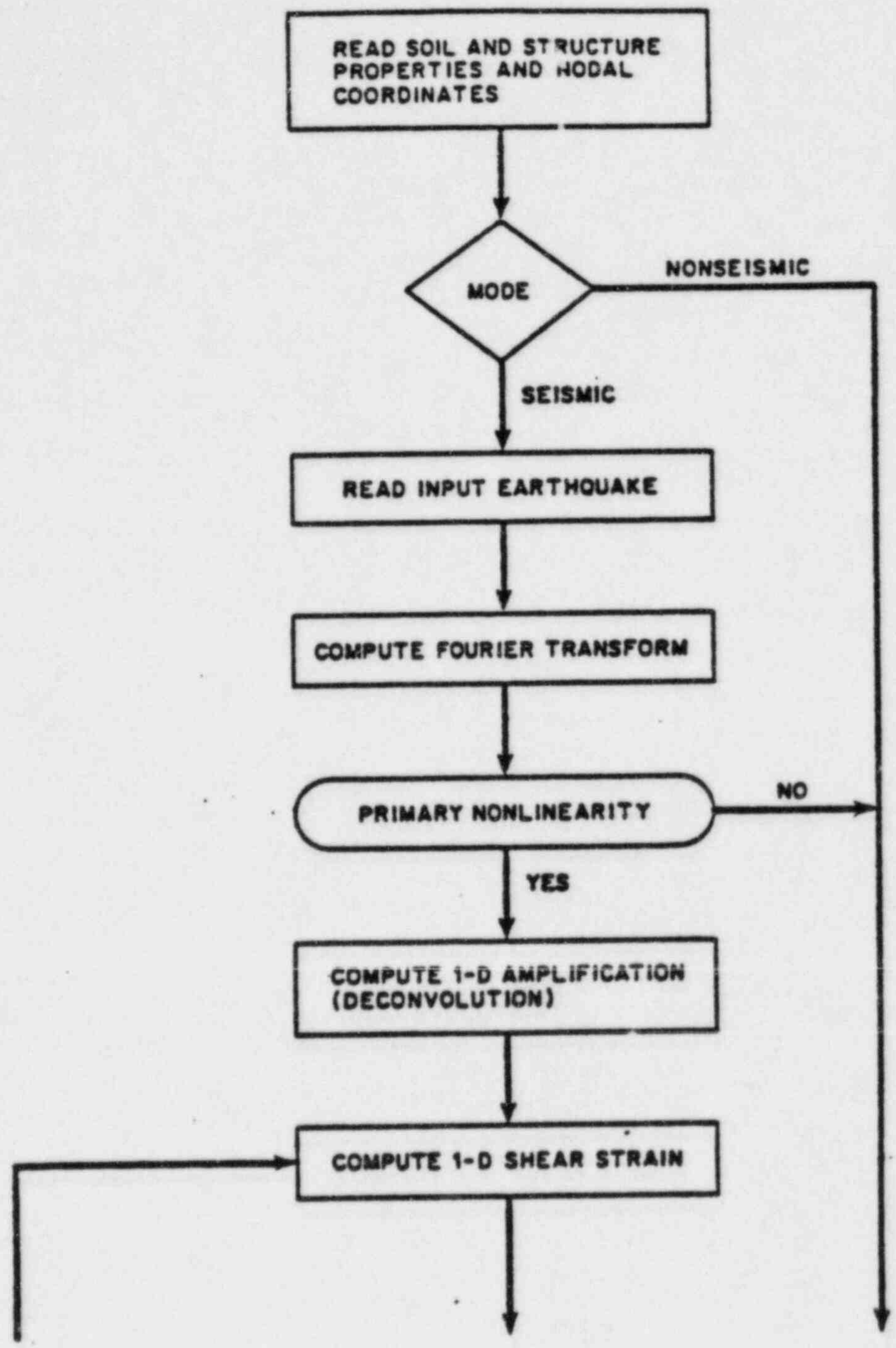


FIGURE 10.2-2 (SH. 1 OF 3)
'PLAXLY' FLOW DIAGRAM
BEAVER VALLEY POWER STATION - UNIT 1

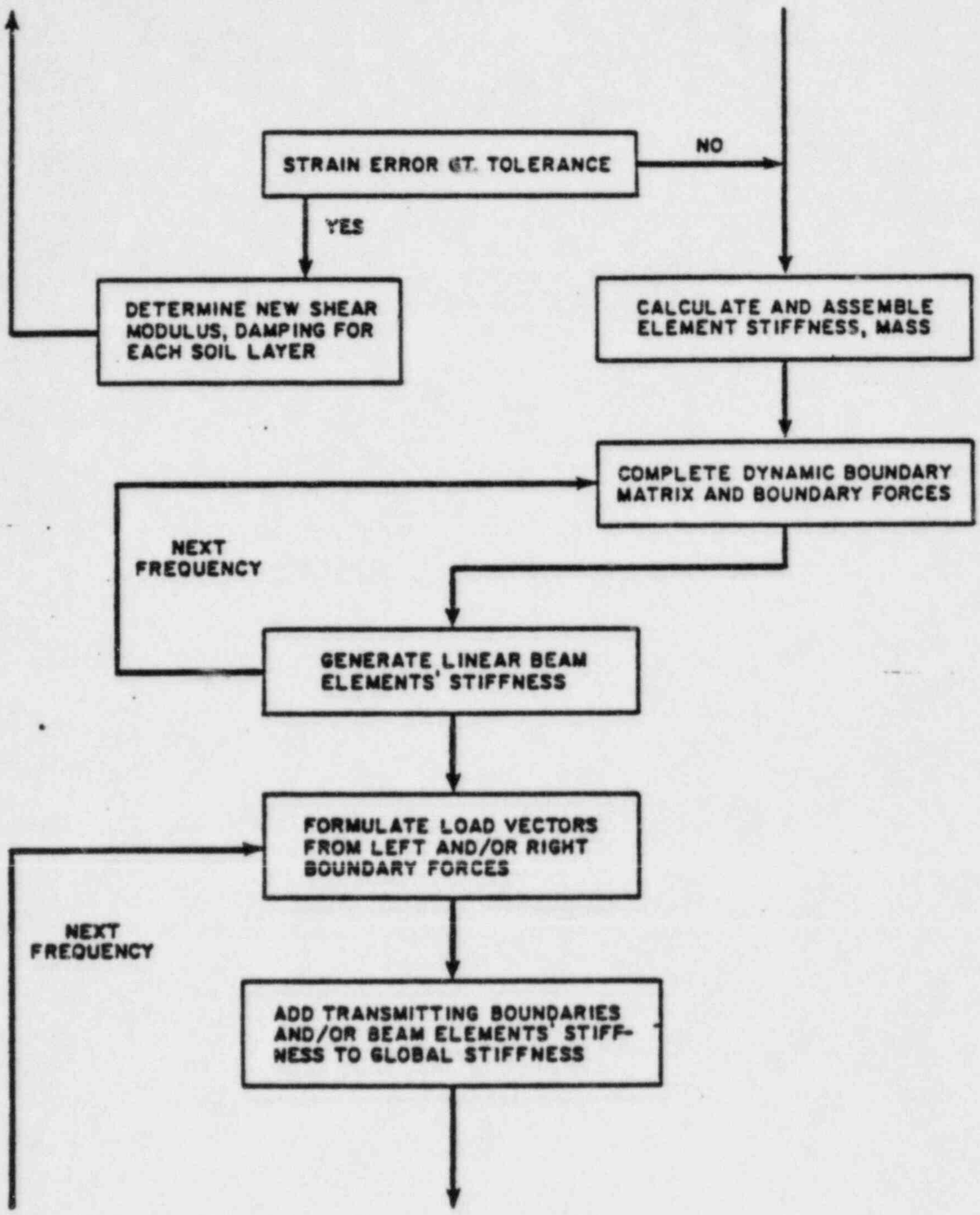


FIGURE 10.2-2 (SH. 2 OF 3)
'PLAXLY' FLOW DIAGRAM
BEAVER VALLEY POWER STATION - UNIT 1

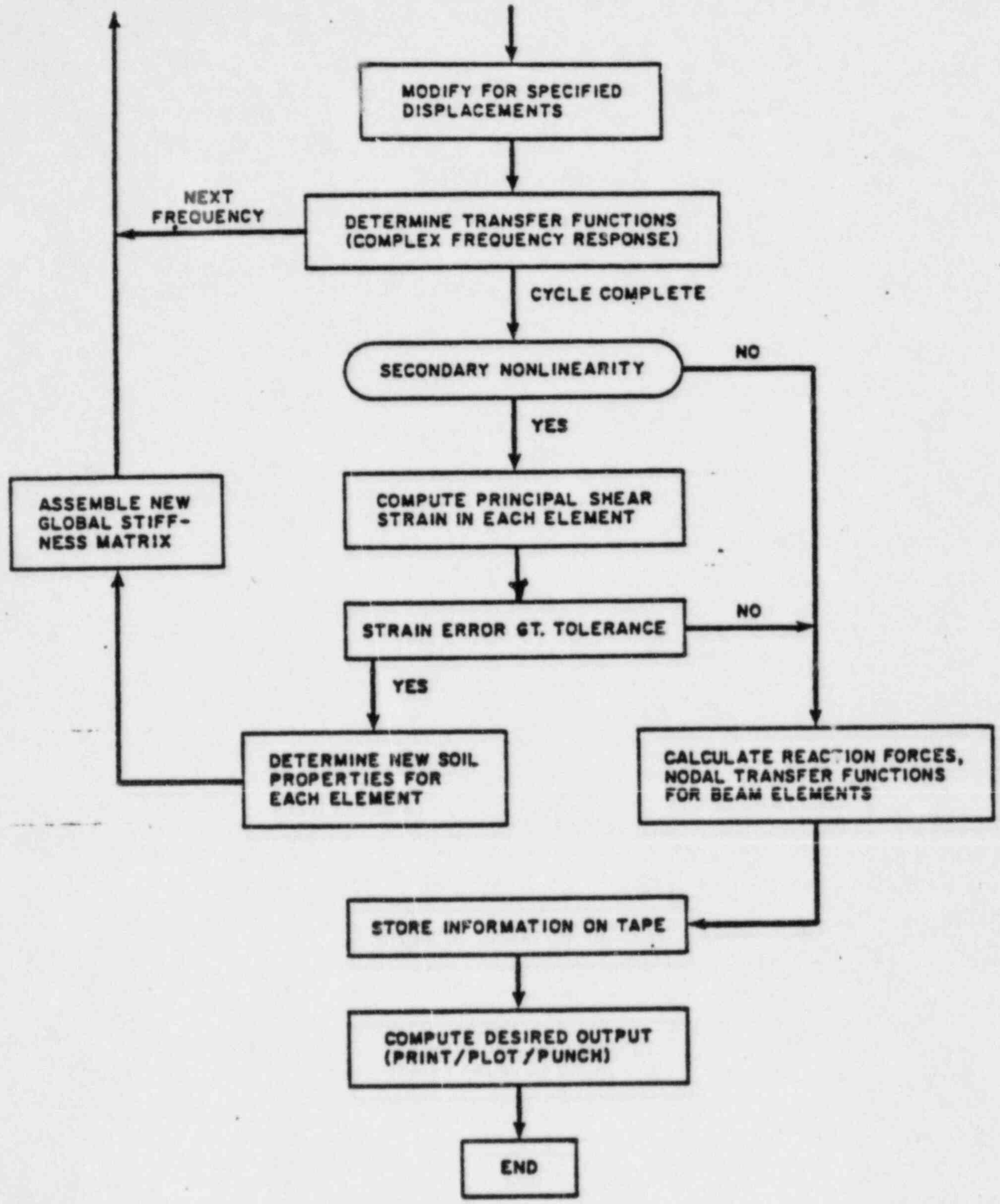


FIGURE 10.2-2 (SH. 3 OF 3)
'PLAXLY' FLOW DIAGRAM
BEAVER VALLEY POWER STATION - UNIT 1

10.3 REFUND AND EMBED

The computer program REFUND is used for computation of the dynamic stiffness functions (impedance functions) of a rigid, massless, rectangular plate welded to the surface of a viscoelastic, layered stratum. The subgrade stiffness matrix is evaluated for all six degrees of freedom for the range of frequencies specified by the user. Embedment effects are applied subsequently by the program EMBED.

The program reads the topology and material properties, assembles the subgrade flexibility matrix, and determines the foundation impedances by inversion. The subgrade flexibility matrix is determined with discrete solutions, to the problems of Cerruti and Boussinesq. A cylindrical column of linear elements is joined to a consistent transmitting boundary, and the flexibility coefficients found by applying unit horizontal and vertical loads at the axis. The rectangular plate is discretized into a number of nodal points, and the global flexibility matrix found using the technique just described. The foundation stiffnesses are then determined solving a set of linear equations which result from imposing unit rigid body translations and rotations to the plate.

Since REFUND is restricted to surface-founded plates, the effects of embedment are included by adjusting the REFUND results with the program EMBED. The

theoretical bases of these programs and their application to the solution methodology are described in Section 4.2.

The results of REFUND compare very well with published results. The comparisons shown in Figures 10.3-2 through 10.3-7 are based upon "Impedance Functions for a Rigid Foundation on a Layered Medium", J.E. Luco, Nuclear Engineering and Design, Vol 2, 1974. Of the various solutions presented by Luco, the following was selected for comparison (see Figure 10.3-1):

	Layer 1	Layer 2
Shear wave velocity	1	1.25
Specific weight	1	1.1764
Poisson's ratio	0.25	0.25

The comparisons shown are of the coefficients k and c from which the vertical, translational, and rocking impedances can be expressed:

$$K = K_0 \; [k + i a_0 \, c]$$

in which a_0 is a dimensionless measure of frequency and K_0 is a zero-frequency stiffness.

The minor differences shown between the REFUND result and Luco's analysis can be attributed to the use of an "equivalent" rectangular plate in the REFUND analysis (Luco's is circular) and differences in boundary conditions at the footing (rough vs. smooth).

The REFUND and EMBED flow diagrams are shown in Figure 10.3-8.

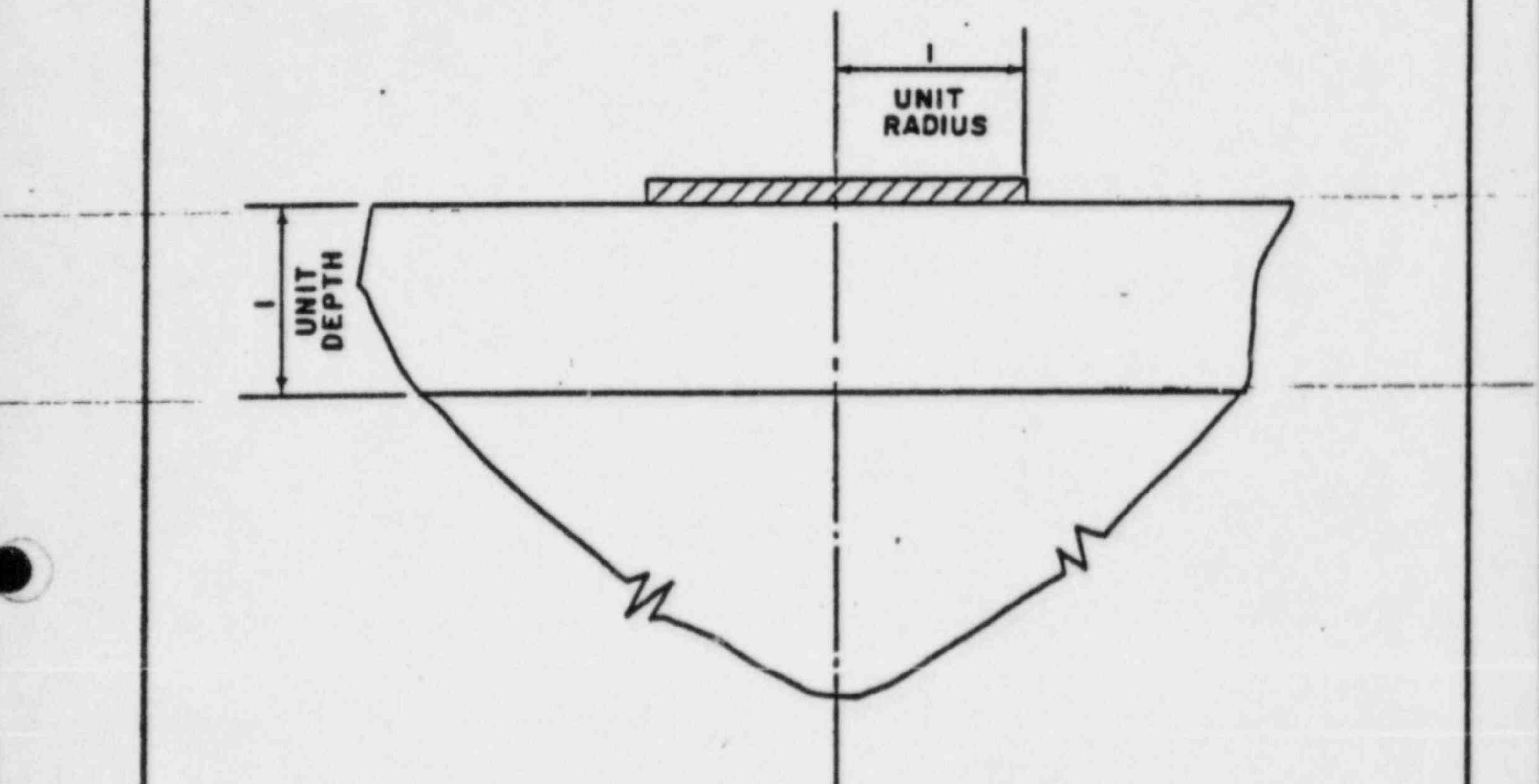


FIGURE 10.3-1
LUCO'S TWO-LAYER PROBLEM
BEAVER VALLEY POWER STATION - UNIT 1

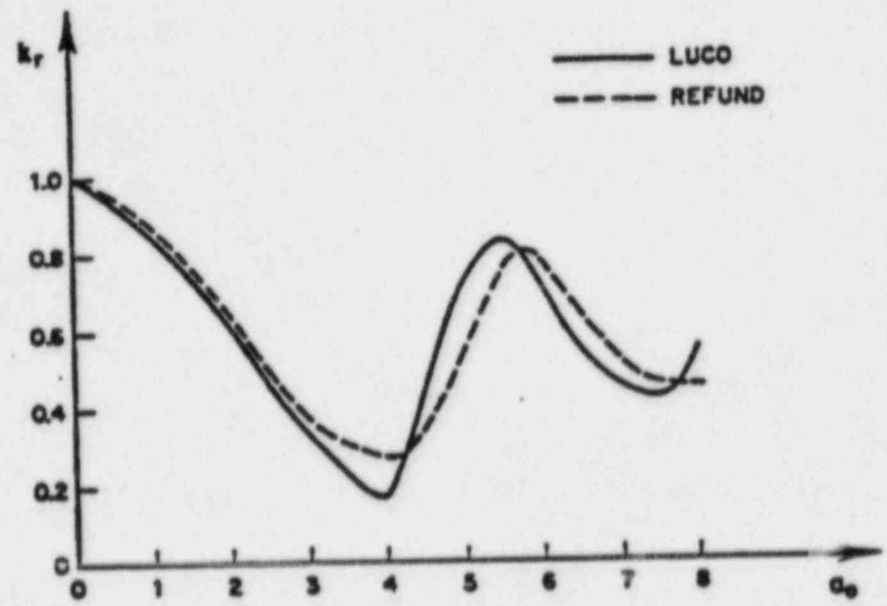


FIGURE 10.3-2
ROCKING STIFFNESS COMPARISON -
REAL PART
BEAVER VALLEY POWER STATION - UNIT 1

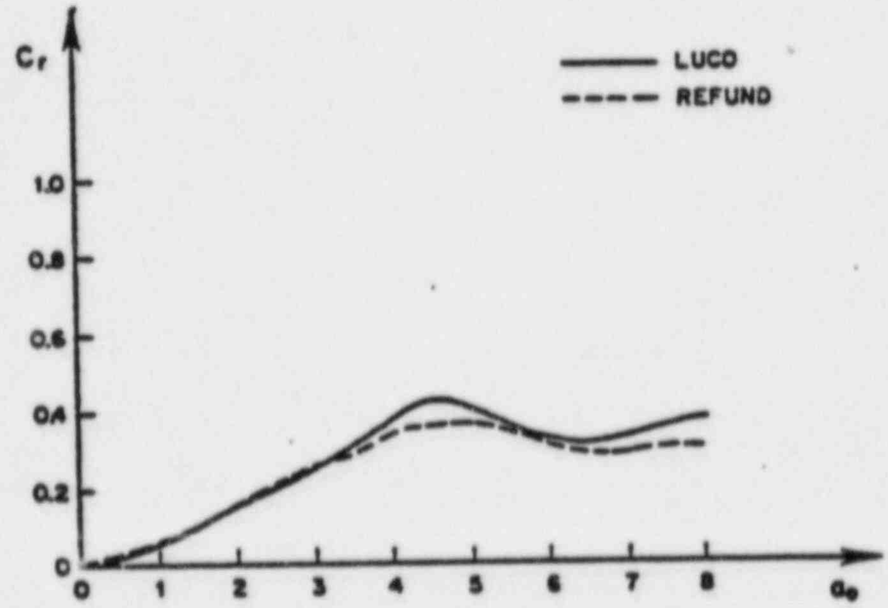


FIGURE 10.3-3
ROCKING STIFFNESS COMPARISON -
IMAGINARY PART
BEAVER VALLEY POWER STATION - UNIT 1

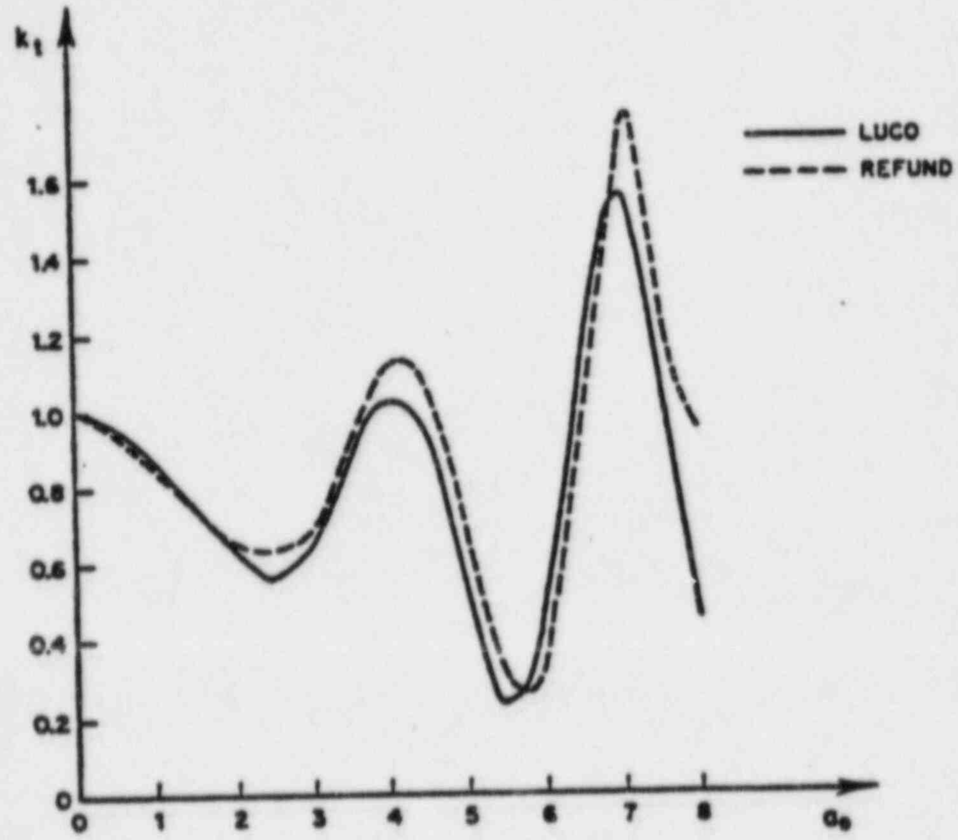


FIGURE 10.3-4
HORIZONTAL STIFFNESS COMPARISON -
REAL PART
BEAVER VALLEY POWER STATION - UNIT 1

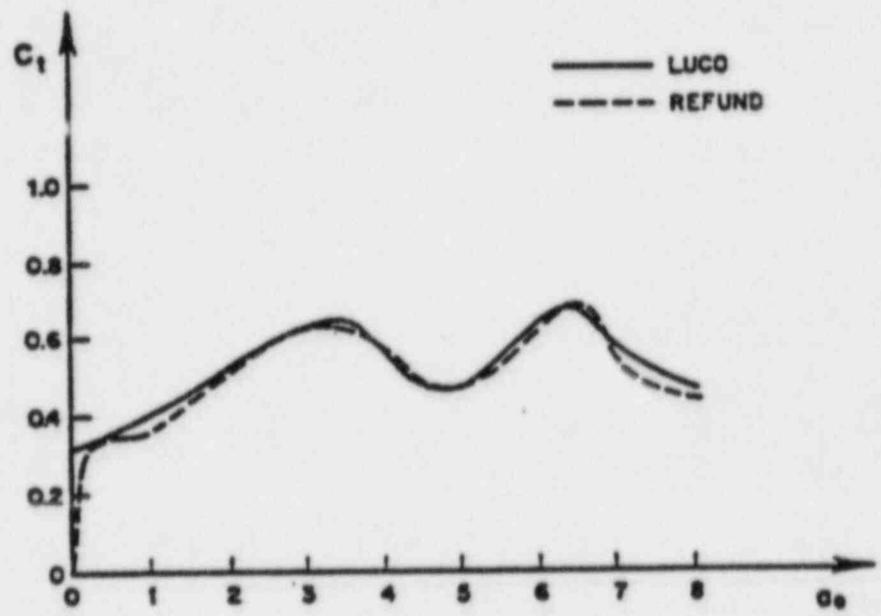


FIGURE 10.3-5
HORIZONTAL STIFFNESS COMPARISON -
IMAGINARY PART
BEAVER VALLEY POWER STATION - UNIT 1

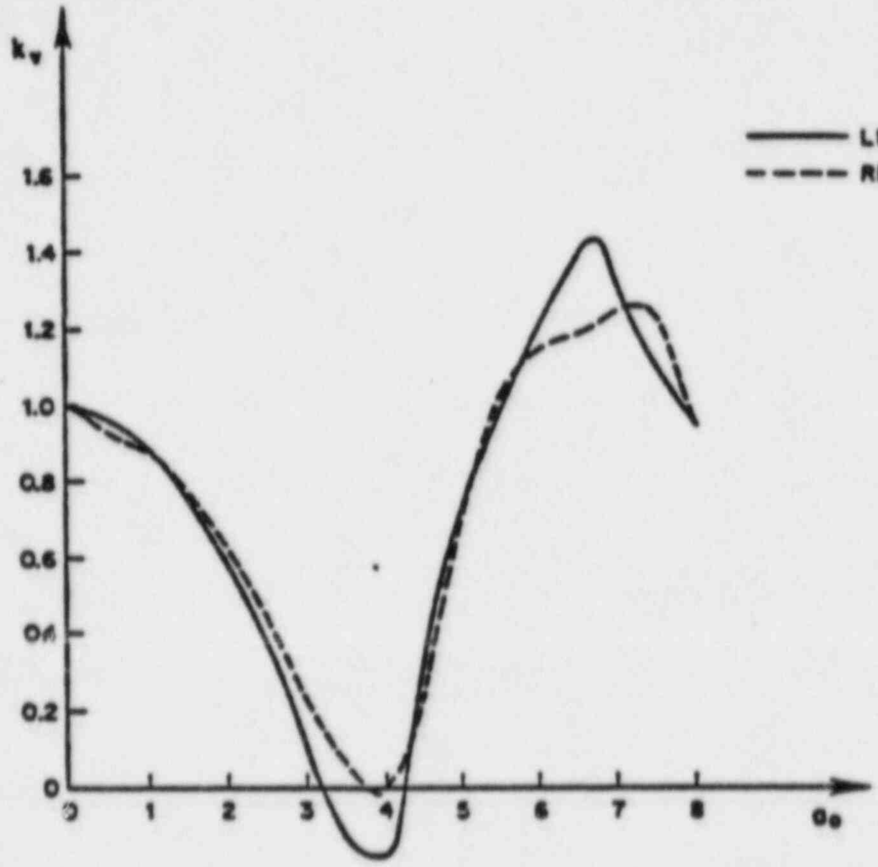


FIGURE 10.3-6
VERTICAL STIFFNESS COMPARISON -
REAL PART
BEAVER VALLEY POWER STATION - UNIT 1

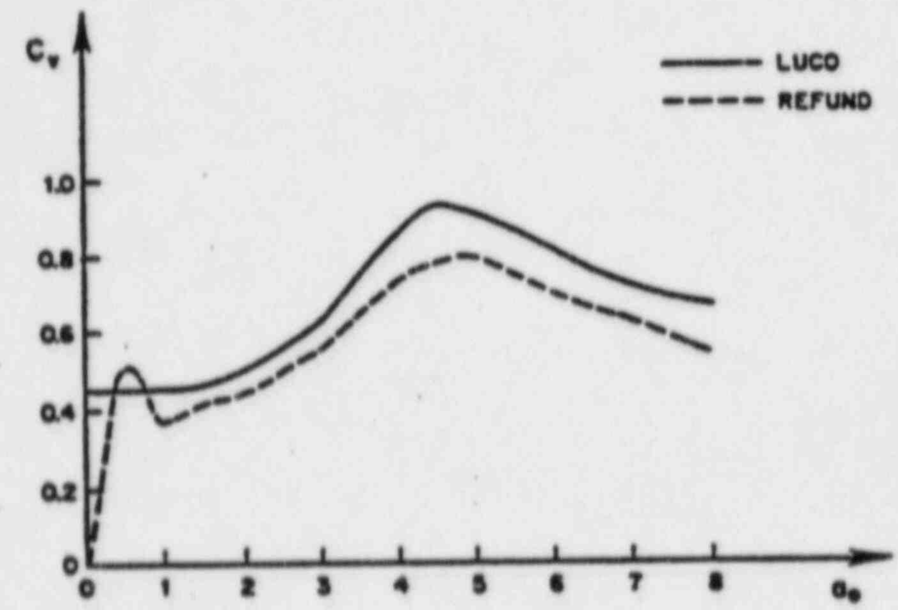


FIGURE 10.3-7
VERTICAL STIFFNESS COMPARISON -
IMAGINARY PART
BEAVER VALLEY POWER STATION - UNIT 1

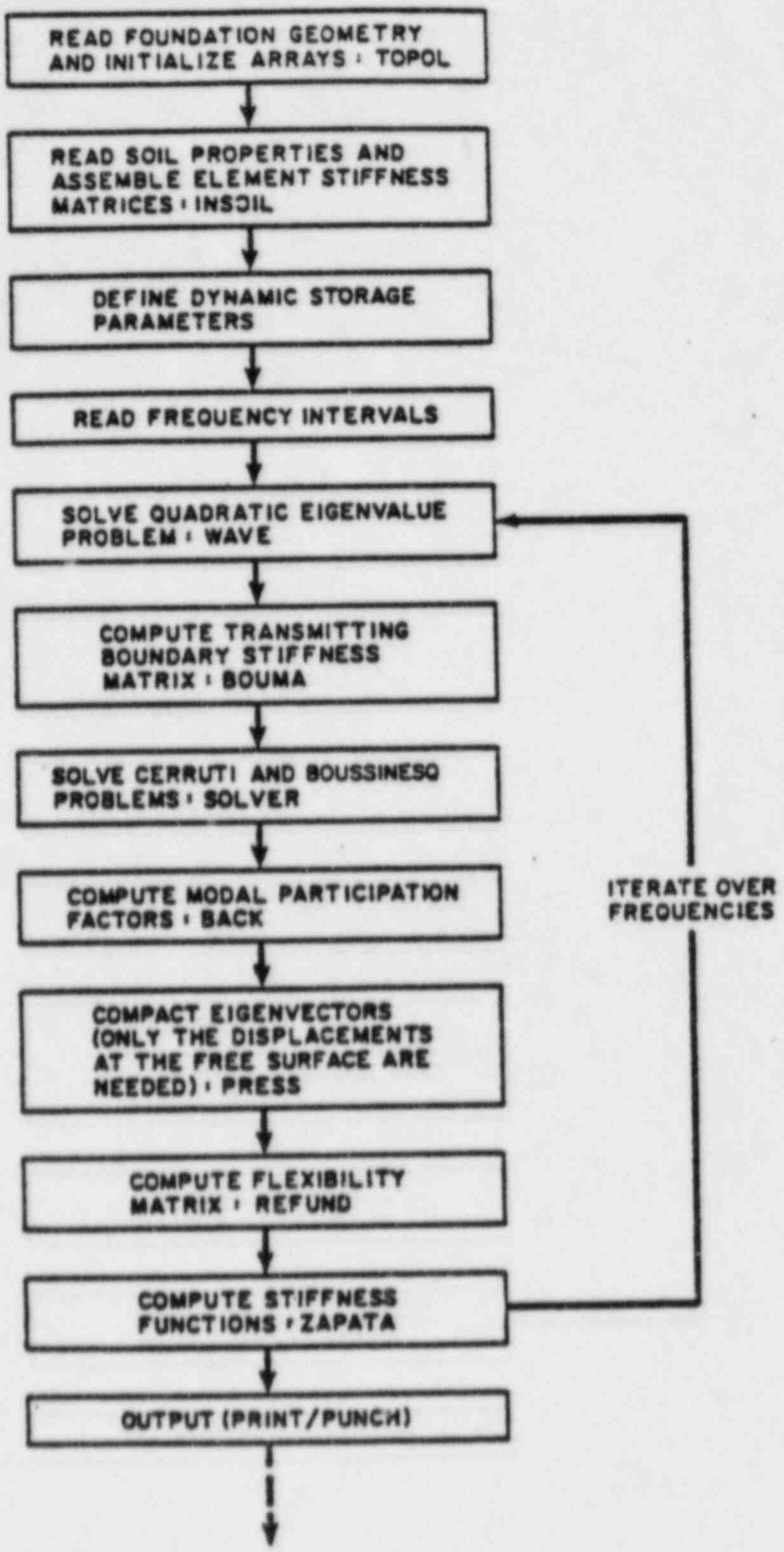


FIGURE 10.3-8 (SH. 1 OF 2)
'REFUND' AND 'EMBED'
FLOW DIAGRAMS
BEAVER VALLEY POWER STATION - UNIT 1

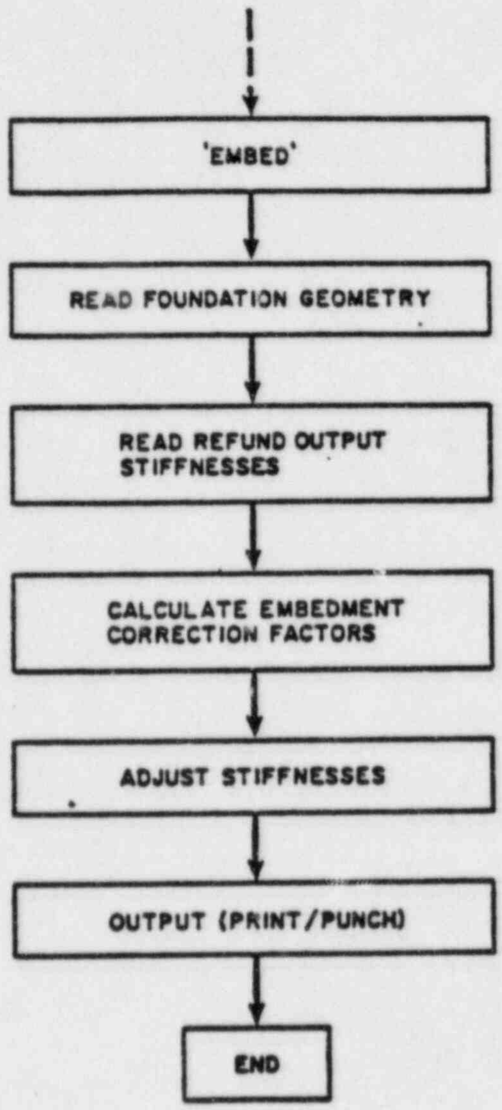


FIGURE 10.3-8 (SH. 2 of 2)
'REFUND' AND 'EMBED'
FLOW DIAGRAMS
BEAVER VALLEY POWER STATION - UNIT 1

10.4 KINACT

KINACT is a computer program used in the three-step solution of soil-structure interaction problems. Briefly, the program modifies the specified translational time history at the surface to translational and rotational time histories at the base of a rigid, massless foundation.

The theoretical basis for the program is derived from wave propagation theory and parametric studies of finite element solutions, described in more detail in Section 4.1.3. Comparisons of the spectra of translational and rotational motion predicted by KINACT and by PLAXLY are shown in Figures 10.4-1 and 10.4-2.

As the figures indicate, KINACT slightly underestimates the translational part of the motion, but significantly overstates the rotational part. This condition results from the dependence of the two variables U and ϕ

$$\bar{\phi} = C \, \frac{(\bar{\bar{U}}_S - \bar{\bar{U}}_B)}{E}$$

where

$\bar{U}_S$ = surface translational acceleration

$\bar{U}_B$ = translational acceleration of rigid massless foundation

C = constant

E = embedment

This self-compensating feature of the formulation is insurance against an unconservative result.

The KINACT flow diagram is shown in Figure 10.4-3.

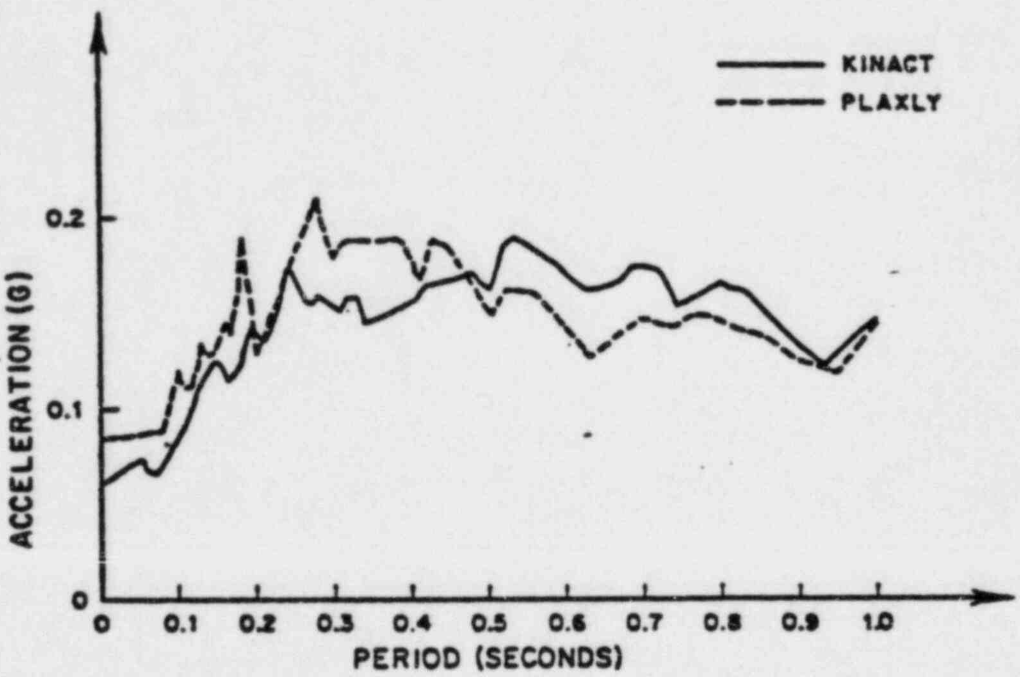


FIGURE 10.4-1
TRANSLATIONAL RESPONSE SPECTRA AT
BASE OF RIGID, MASSLESS FOUNDATION
BEAVER VALLEY POWER STATION - UNIT 1

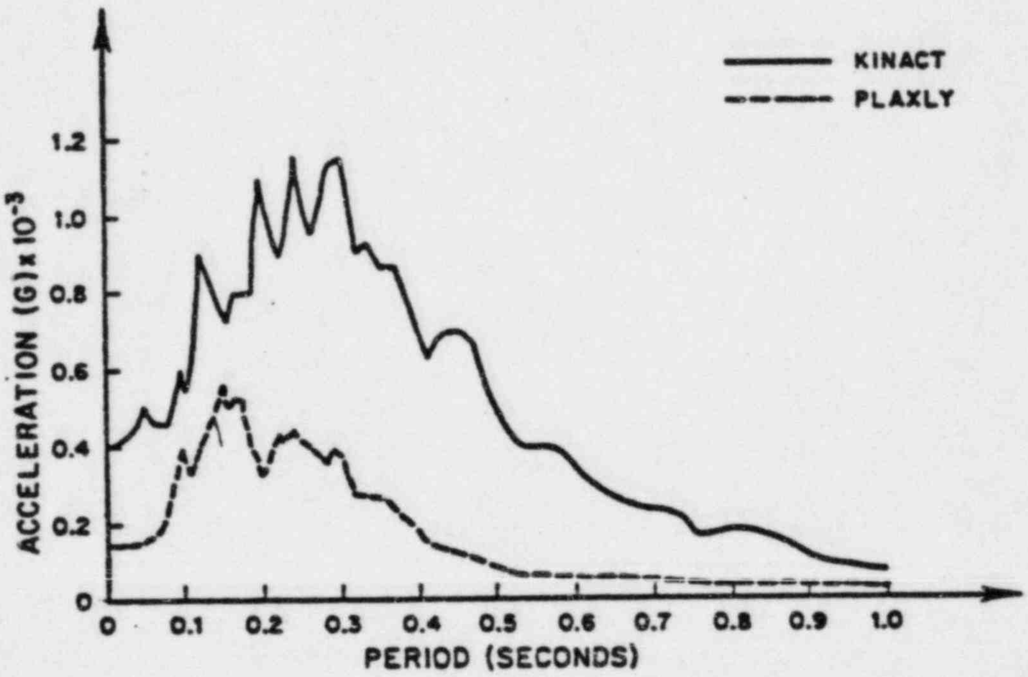


FIGURE 10.4-2
ROTATIONAL RESPONSE SPECTRUM AT
BASE OF RIGID, MASSLESS FOUNDATION
BEAVER VALLEY POWER STATION - UNIT 1

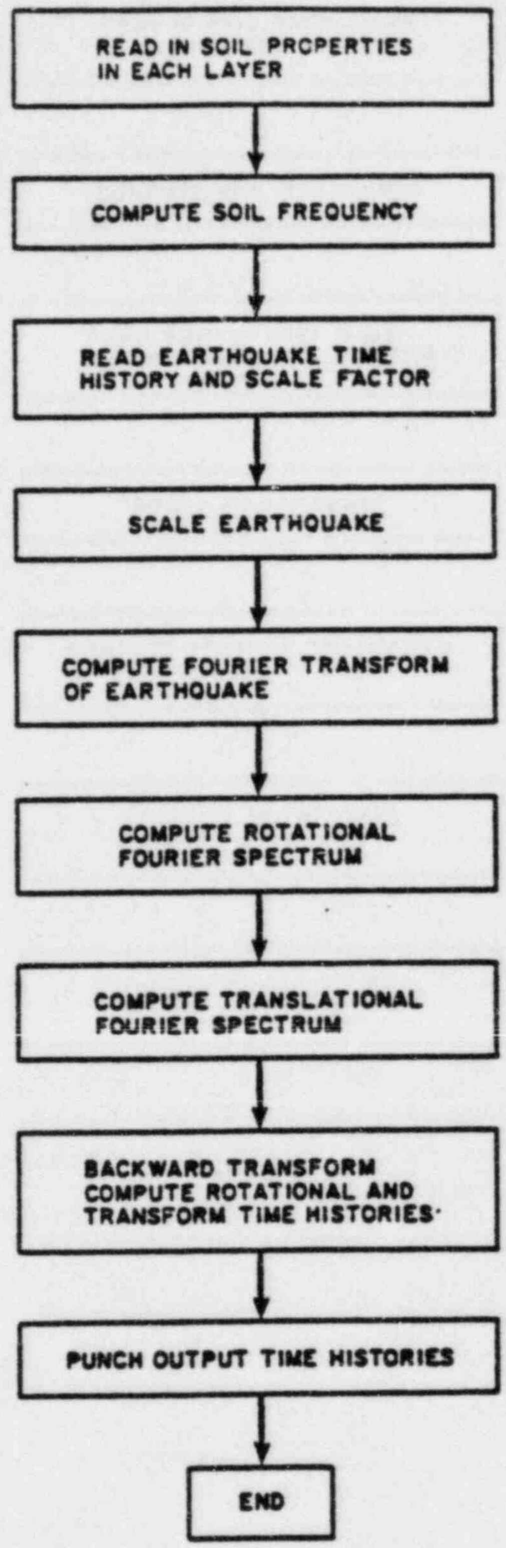


FIGURE 10.4-3
'KINACT' FLOW DIAGRAM
BEAVER VALLEY POWER STATION - UNIT 1

10.5 FRIDAY

The computer program FRIDAY is used for dynamic analysis of structures subjected to seismic loads, accounting for soil-structure interaction by means of frequency-dependent complex soil springs.

The structure is idealized as a set of lumped masses connected by springs or linear members, and attached to a common support, the mat. The latter is supported by soil springs or impedances, which may or may not be frequency-dependent. Alternatively, the mat may rest on a rigid subgrade. The structure may be three-dimensional, but cannot be interconnected; each structure has to be simply connected. Fourier transform techniques are used to determine time histories; cutoff frequency is prescribed internally to 15 Hz.

The theoretical basis and implementation of the program is described in Section 4.1.4. A comparison of FRIDAY with a public domain program, STARDYNE, for the seismic response of a fixed base, multi-mass, cantilever model is shown in Figure 10.5-1. The model is shown in Figure 10.5-2.

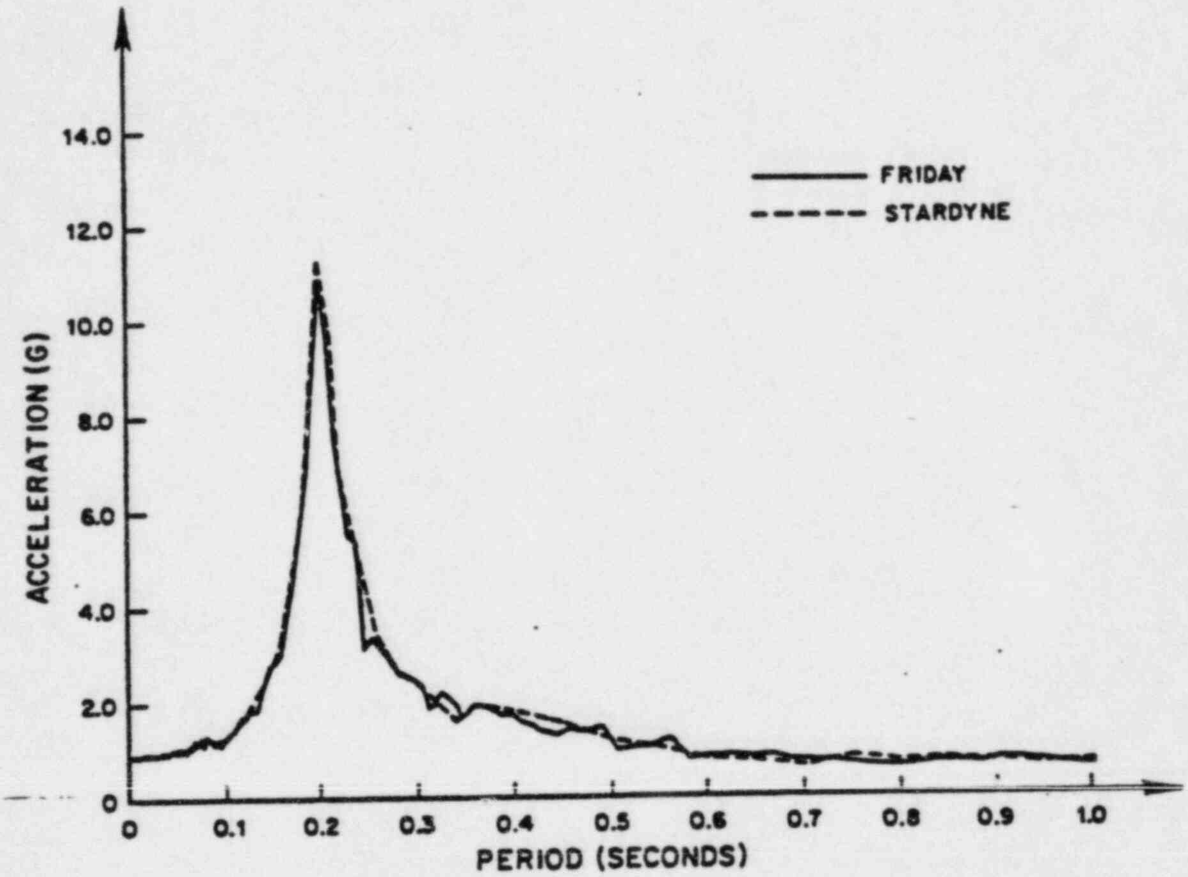


FIGURE 10.5-1
COMPARISON OF 'FRIDAY' AND
'STARDYNE'-ARS AT THE ROOF
BEAVER VALLEY POWER STATION - UNIT 1

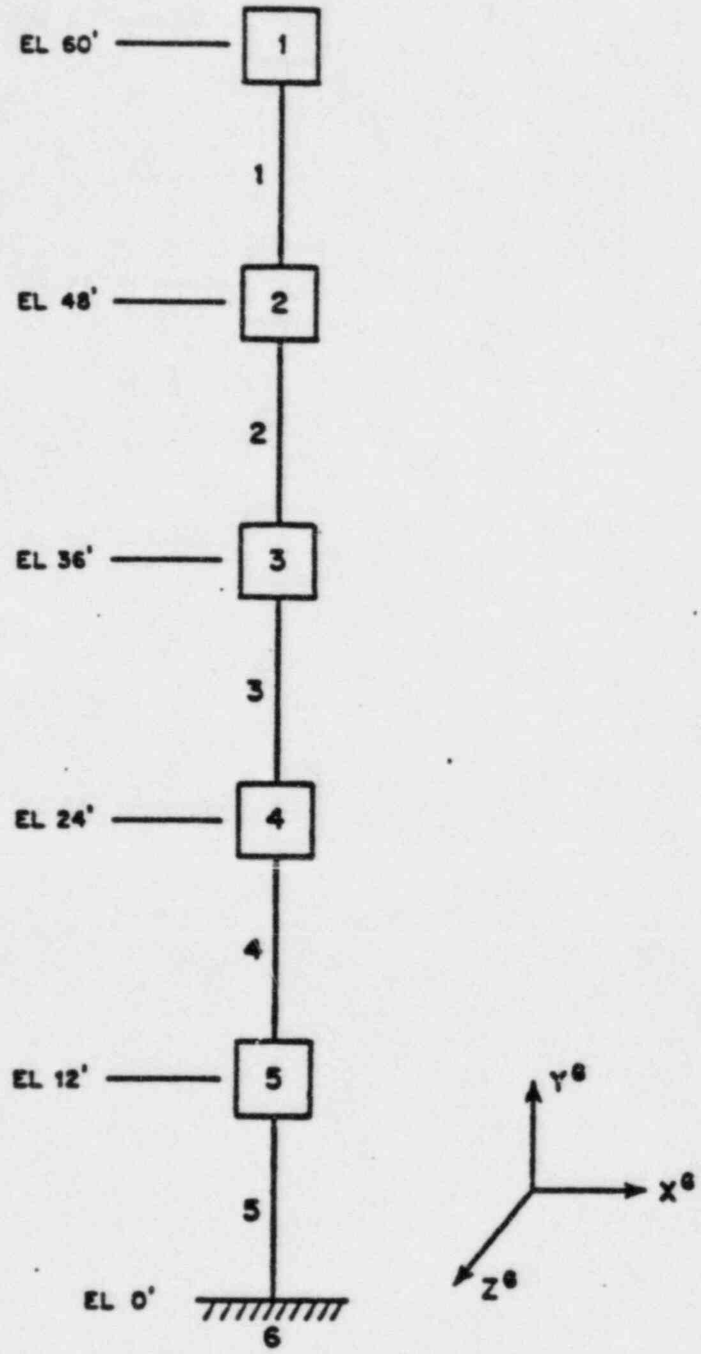


FIGURE 10.5-2
'STARDYNE' MODEL
BEAVER VALLEY POWER STATION - UNIT 1

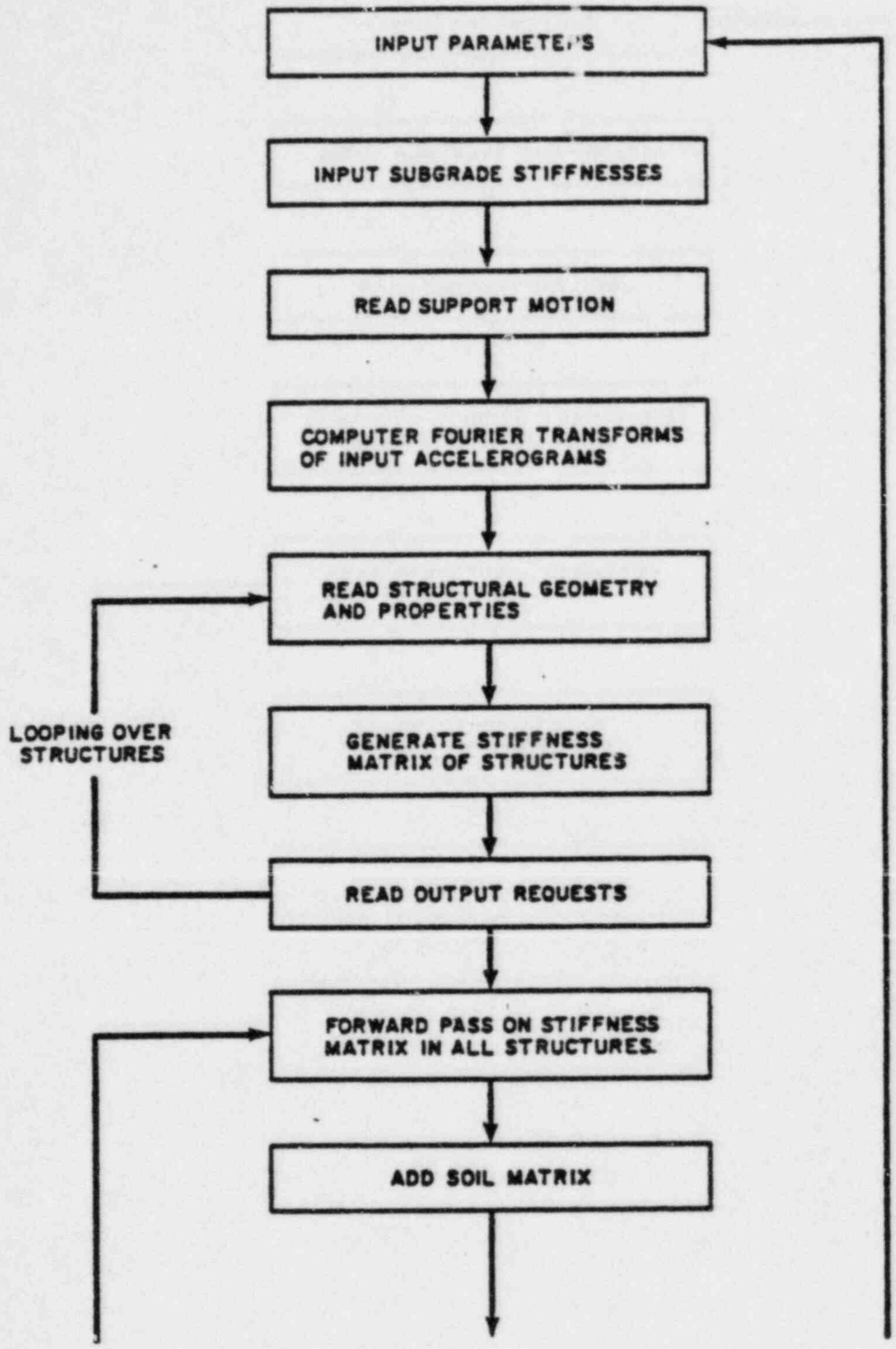


FIGURE 10.5-3 (SH. 1 OF 2)
'FRIDAY' FLOW DIAGRAM
BEAVER VALLEY POWER STATION - UNIT 1

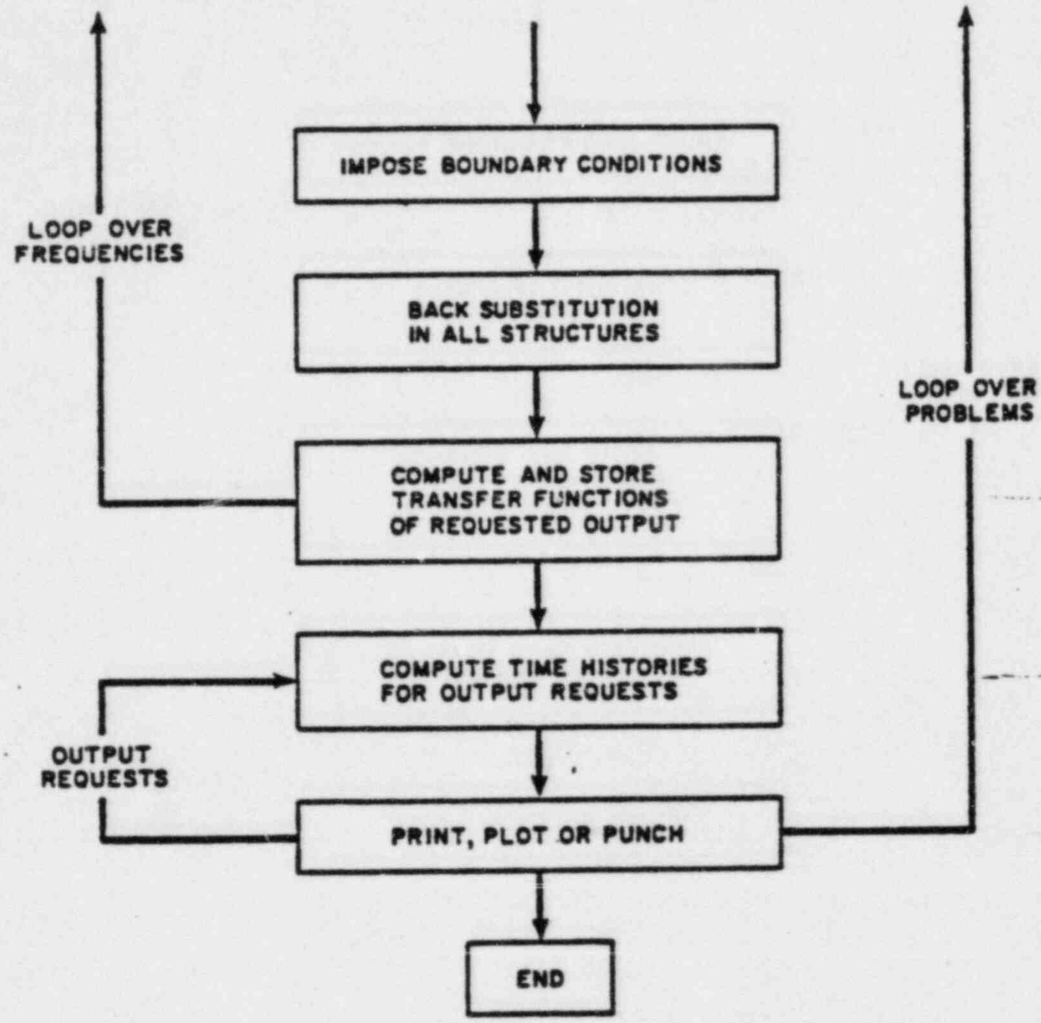


FIGURE 10.5-3 (SH. 2 OF 2)
'FRIDAY' FLOW DIAGRAM
BEAVER VALLEY POWER STATION - UNIT 1

APPENDIX 10.6

IN SITU SEISMIC VELOCITY MEASUREMENTS

WESTON GEOPHYSICAL ENGINEERS, INC.
Post Office Box 550 • Westboro, Massachusetts 01581 • (617) 366-9191

September 6, 1977

Stone & Webster Engineering Corporation
245 Summer Street
Boston, Massachusetts 02107

Gentlemen:

In accordance with your Contract No. 2BVC-52035, dated June 6, 1977, a seismic cross-hole study was conducted in the vicinity of the Beaver Valley Power Station, Unit 2. Fieldwork was conducted during the period of June 9 to June 22, 1977.

Preliminary data have been previously submitted; this is a formal presentation of our findings.

Very truly yours,

WESTON GEOPHYSICAL ENGINEERS, INC.

Vincent J. Murphy

VJM:df

IN-SITU SEISMIC VELOCITY MEASUREMENTS

BEAVER VALLEY POWER STATION

UNIT NO. 2

DUQUESNE LIGHT COMPANY

WESTON GEOPHYSICAL ENGINEERS, INC.

IN-SITU SEISMIC VELOCITY MEASUREMENTS

BEAVER VALLEY POWER STATION

UNIT NO. 2

INTRODUCTION AND PURPOSE

A seismic cross-hole study was conducted in the vicinity of the Beaver Valley Power Station, Unit 2, of the Duquesne Light Company between June 9 and June 22, 1977. The purpose of the study was to measure the in-situ compressional ("P") and shear ("S") wave velocity values for a layer of granular material which was densified by Franki Pressure Injected Footing (PIF) compaction piles. The in-situ "P" and "S" wave velocity values measured in this study were used to calculate the elastic moduli values for those materials encountered within the seismic cross-hole array.

The field effort at the site was expedited by Mr. J. W. Williams, the Superintendent of Construction for the construction division of Stone & Webster Engineering Corporation. Survey requirements were outlined and the project was coordinated by Mr. D. Campbell, the Lead Geotechnical Engineer for the geotechnical division of Stone & Webster at the Boston office. Weston Geophysical's project geophysicist for this study is V. J. Murphy, and the assistant project geophysicist is R. P. Allen.

LOCATION

The area of investigation (Sheet 1) is on a high-level terrace on the south bank of the Ohio River (former elevation 735 feet) approximately 25 miles northwest of Pittsburgh. Sheet 1 is a section of the Hookstown, Pennsylvania and Midland, Pennsylvania, United States Geological Survey topographic quadrangle maps (1:24,000). The seven boreholes used for the in-situ velocity measurements are shown on Sheet 2, which was prepared from a map provided by Stone & Webster.

IN-SITU VELOCITY MEASUREMENTS - CROSS-HOLE PROCEDURE

Cross-hole velocity measurements are made using geophones containing three orthogonal elements (one vertical and two horizontal). Recordings are obtained using a 12- to 16-channel seismograph that contained a two-millisecond timing system. Seismic energy is generated with the energy source in one hole and detected in the geophone holes with the seismic source and the geophones at the same elevation levels. The energy source(s) for this survey included both small explosive charges and an air gun.

The "P" wave and "S" wave velocity data were obtained at 5-foot intervals within the densified zone and at 10-foot intervals above and below the zone.

RESULTS

Generalized results of the survey are presented in Table 1, which also lists the results of a previous Weston survey (in this general vicinity) reported to Stone & Webster in 1968. Table 2 lists the specific velocity values measured at each elevation for the present survey and the corresponding elastic moduli values. The various combinations of shothole and recording (detector) holes that were used are also noted on Table 2.

The densified zone occurs generally between Elevations 670 and 640. It is interesting to note that complete saturation, as evidenced by seismic velocities of about 5,000 ft/sec or greater, does not occur above Elevation 652, although the water table elevation has been observed by geotechnical personnel in the field at Elevation 667. This apparent discrepancy can be explained by the possible injection of small amounts of air into the surrounding sediments during the densification process, (a minute percentage of air in an otherwise fully saturated layer can lower the seismic velocity value significantly).

No anomalous conditions, other than that mentioned above, were observed during the cross-hole study.

TABLE I

GENERALIZED "P"- AND "S"-WAVE VELOCITY VALUES
1968 SURVEY AND PRESENT SURVEY *

1968 SURVEY

	"P" WAVE VELOCITY (FT./SEC.)	"S" WAVE VELOCITY (FT./SEC.)
EL. 740'		
APPROX. GROUND SURFACE		
	1500 (SOME 1000)	900- (SOME 600)
720'		
	2000	900-1200-
700'		
	2000-	1050±
680'		
APPROX. WATER TABLE		
660'	6000	1300-
	6000	1300-
640'		
620'		
	12,000	6000-
600'		

PRESENT SURVEY

	"P" WAVE VELOCITY (FT./SEC.)	"S" WAVE VELOCITY (FT./SEC.)
EL. 740'		
720'		
APPROX. GROUND SURFACE		
700'		
NOTE: NO SEISMIC MEASUREMENTS TAKEN ABOVE EL. 685 IN PRESENT SURVEY		
	2000-2500	700-800?
680'	? ?	? ?
	2400-2500	1000
660'	3000	1000-1200
APPROX. WATER TABLE †		
640'	6300-6500	1500-1800
620'		
	12,000	4400-5800?
600'		

* CF TEXT

† CF TEXT FOR DISCUSSION OF THE WATER TABLE ELEVATION FOR PRESENT SURVEY.

TABLE 2

IN-SITU VELOCITY MEASUREMENTS

Elevation	Density*	"P" Wave Velocity (ft./sec.)†	"S" Wave Velocity (ft./sec.)†	Poisson's Ratio	Shear Modulus (x 10^5 lbs./in.^{2})	Young's Modulus (x 10^5 lbs./in.^{2})	Bulk Modulus (x 10^5 lbs./in.^{2})
685	123	2,000-2,500	700-800	.438	.149	.429	1.14
675	123	2,500	1,000	.405	.265	.746	1.31
670	123	2,400	1,000	.395	.265	.741	1.18
665	135.9	2,800	1,000-1,100	.418	.323	.917	1.87
660	135.9	3,000	1,000-1,100	.430	.323	.925	2.21
655	135.9	3,000	1,100-[illegible]	.414	.388	1.10	2.12
650	135.9	6,300-6,400	1,600-1,700	.464	.799	2.34	10.76
645	135.9	6,500	1,500	.472	.660	1.94	11.51
635	135.9	6,400	1,800	.457	.950	2.77	10.75
625	155	12,000	4,400-5,800	.390	8.70	24.19	36.57

Note: There were five seismic arrays utilized during the cross-hole survey. They are listed below:

Shot Hole	Recording Holes
1	2, 3, 4 and 5
1	3, 4, 7 and 6
2	3, 4, 7 and 6
5	7, 4, 3 and 2
6	7, 4, 3 and 2

*Provided by Stone & Webster Engineering Corporation.

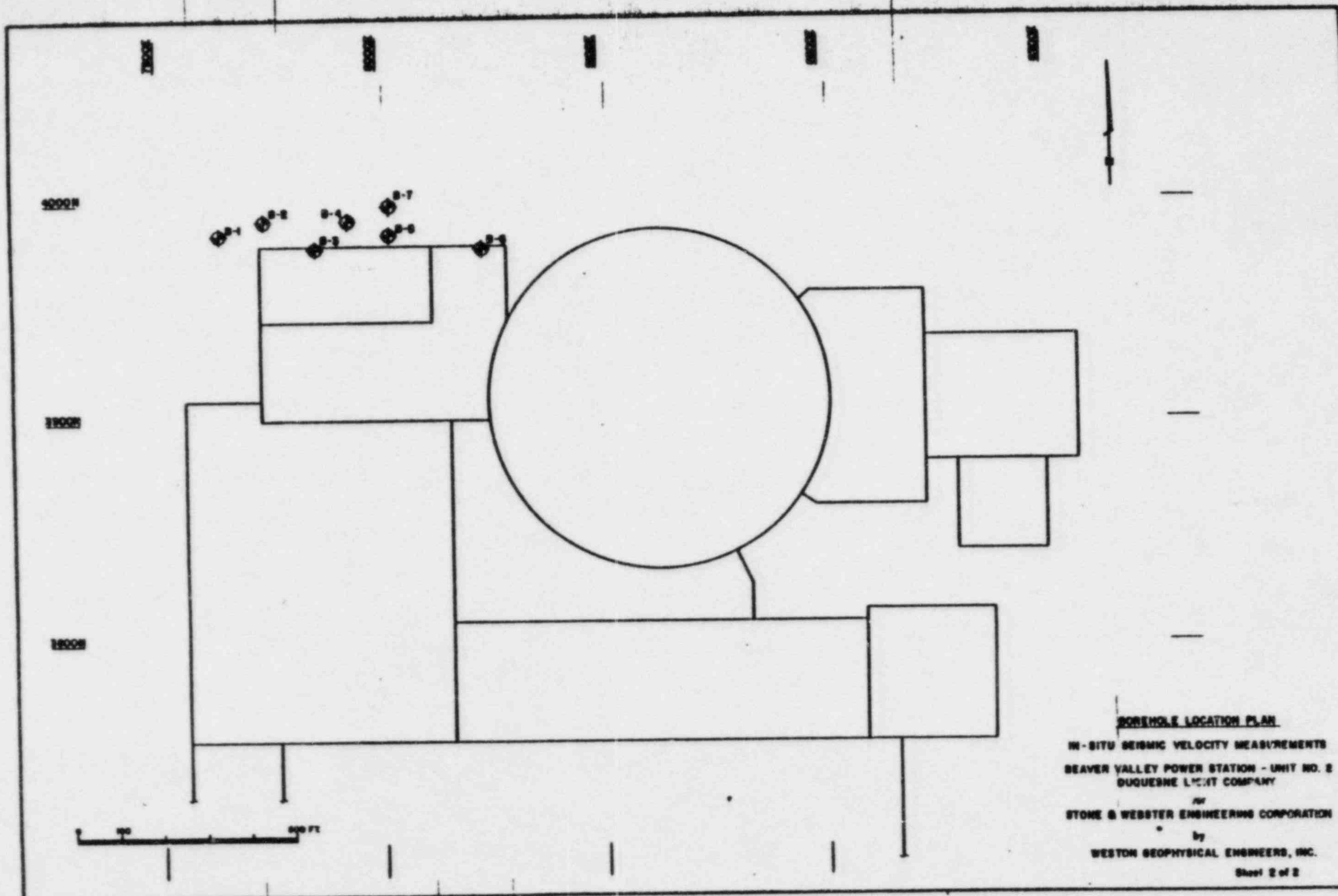
†Where a range of velocity values is given, the average of that range was used in the moduli calculations.

AREA OF INVESTIGATION

IN-SITU SEISMIC VELOCITY MEASUREMENTS

BEAVER VALLEY POWER STATION - UNIT NO. 2
DUQUESNE LIGHT COMPANY
for
STONE & WEBSTER ENGINEERING CORPORATION
by
WESTON GEOPHYSICAL ENGINEERS, INC.

Sheet 1 of 2



BOREHOLE LOCATION PLAN

IN-SITU SEISMIC VELOCITY MEASUREMENTS

BEAVER VALLEY POWER STATION - UNIT NO. 2
DUQUESNE LIGHT COMPANY

for

STONE & WEBSTER ENGINEERING CORPORATION

by

WESTON GEOPHYSICAL ENGINEERS, INC.

Sheet 2 of 2