TECHNICAL CRITERIA AND PROCEDURES USED FOR THE GENERATION OF IN-STRUCTURE RESPONSE SPECTRA

FOR

BALTIMORE GAS AND ELECTRIC COMPANY

CALVERT CLIFFS NUCLEAR POWER PLANT

ABSTRACT

ATTACHMENT 1 is a Technical Discussion which presents the detailed information as to the technical criteria and procedures used to generate the In-Structure Response Spectra at Calvert Cliffs Nuclear Power Plant (CCNPP). This technical discussion has been prepared for use in resolving Unresolved Safety Issue (USI) A-46 as requested by the NRC in Supplemental Safety Evaluation Report No. 2 (SSER No. 2) on SQUG Generic Implementation Procedure, Revision 2 as corrected on February 14, 1992 (GIP-2). SSER-2 has been transmitted to (USI) A-46 Plant Licensees who are Members of the Seismic Qualification Utility Group (SQUG) by the NRC via Supplement No. 1 to Generic Letter (GL) 87-02. This generic letter was issued by the NRC to implement resolution to (USI) A-46 which concluded that the seismic adequacy of certain equipment in operating nuclear power plants should be reviewed against seismic criteria not in use when these plants were licensed.

ATTACHMENT 1 also presents a brief discussion of the methods used for performing the seismic analyses, mathematical model formulation, soil structure interaction considerations and seismic analysis results. The analytical results include frequencies and mode shapes and the In-Structure Response Spectra. This technical discussion presents a summary of the parameters for the significant modes of vibration, specifies the design basis earthquakes and shows sketches of the various models used for seismically analyzing the Class I structures along with a tabulation of peak component responses as well as the floor acceleration.

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1.0 INTRODUCTION

This technical discussion presents the criteria and procedures used for the generation of the In-Structure (Floor) Response Spectra (ISRS) which defines the seismic requirements for equipment installed within the Calvert Cliffs Nuclear Power Plant Class I Structures. This technical discussion also describes the considerations and concepts utilized for performing the seismic analysis for the Class I Structures, namely the Containment, Auxiliary and Intake Structures.

In-Structure Response Spectra, otherwise known as Floor Response Spectra (FRS) have been developed in order to specify the seismic requirements for "Safety-Related" (SR) equipment associated with Class I structures. These spectra have been generated using lumped-parameter analytical techniques. The considerations and assumptions used for formulating the building models and conducting the seismic analyses are described.

Both the Spectral Response and Time-History types of analyses have been performed utilizing the Class I structural models. The Spectral Response analysis results have been used for defining the seismic loads required for design of the Class I structures. Using the Time-History method of analysis, floor level acceleration responses versus time have been computed from which the ISRS or FRS have been generated. Criteria and procedures used for the development of the ISRS are described. Reference documents are cited which contain the complete Calvert Cliffs Nuclear Power Plant In-Structure (FRS) Floor Response Spectra.

2.0 BACKGROUND

Safety-related Class I (SR-Class I) structures which have been designed to withstand the effects of the design earthquake include:

- o Containment Building
- o Auxiliary Building
- o Intake Structure
- o Condensate Storage Tank
- o Diesel Generator Fuel Oil Storage Tank

Systems and components located within these structures are also designed to withstand the effects of the design earthquake, i.e., whether only structural adequacy is required, or whether in addition, functionality is to be maintained. In order to withstand the loads from the design earthquake, the design of SR-Class I structures has been based on the analytical techniques cited in:

- o TID 7024, "Nuclear Reactors and Earthquakes", USAEC-1063, [Ref. 1].

- o BC-TOP-4A, "Seismic Analysis of Structures and Equipment for Nuclear Power Plants", Bechtel, [Ref. 2].

Those structures not designated as Class I are classified as Class II and have been designed to seismic criteria as specified in the 1967 (UBC) Uniform Building Code.

To implement the analytical techniques cited in References 1 and 2, lumped-mass mathematical models of major SR-Class I structures have been created for performing both the Spectral Response and Time-History types of seismic analysis. Seismic forces for the design of the SR-Class I buildings have been obtained using the Spectral Response analysis method.

For defining the seismic requirements of equipment located in the SR-Class I structures, the Time-History analysis method has been used for developing the In-Structure (Floor) Response Spectra (ISRS). The structures for which ISRS have been generated include the Containment Building, Auxiliary Building and Intake Structure.

3.0 METHODS OF ANALYSIS

For each of the Class I structures, the seismic analysis was conducted in five successive steps:

1) Formulating mathematical models
2) Determining natural frequencies and mode shapes
3) Selecting appropriate modal damping values
4) Appropriately describing earthquake parameters
5) Determining response of structure and generating in-structure (floor) response spectra

The mathematical model of each structure was formulated using finite elements which idealized the structure into a system of lumped masses and stiffness coefficients. At appropriate locations within the building, such as at floor slabs, mass points were chosen to lump the contributing weights of the structure. Between these points, the flexibility of the member was calculated considering the member parameters such as length, cross-sectional area, effective shear area and area moment of inertia.

In order to obtain the natural frequencies and mode shapes of the structure, Bechtel Computer Program CE-617 was used. This program utilized the lumped weights of the model and the element flexibilities to formulate the stiffness and mass matrices of the idealized system. The technique of diagonalization by successive rotations was used to obtain the natural frequencies and mode shapes.

Damping values for the structural system were based on the type of material and the mode shape. Table 1.0 shows the damping values associated with the various types of material configurations. Damping for each mode considered the damping properties of the individual elements and the mode shapes of the system. The effective modal damping was computed as presented in Appendix B.

3.1 SPECTRAL RESPONSE

For determining the response of the building for purposes of calculating the overall building design loads due to seismic excitation, the Spectral Response method of analysis was applied. The earthquake magnitude was described by the Design (Ground) Response Spectra which portrays the amplification of the earthquake as a single degree-of-freedom oscillator versus frequency. Design (Ground) Response Spectra were established for both Operating Basis

Earthquake (OBE) and Design Basis Earthquake (DBE) where the DBE is synonymous with the current designation for Safe Shutdown Earthquake (SSE). These Design (Ground) Response Spectra from Reference 9, UFSAR Appendix 5A are shown in Figures 1.1 and 1.2. The OBE has a maximum ground acceleration of 0.08 G horizontally and 0.053 G vertically. The DBE earthquake has a maximum ground acceleration of 0.15 G horizontally and 0.10 G vertically. For Calvert Cliffs Nuclear Power Plant (CCNPP), one horizontal earthquake component and a vertical earthquake component are specified to act simultaneously.

From the Design (Ground) Response Spectra, acceleration values for each mode were selected as associated with each natural frequency and modal damping[1] value. The standard Spectral Response method of analysis was used to obtain the structural inertial forces and displacements at the mass points and resulting member shears and moments per mode. Modal results were then combined using a modal synthesis technique[2] to obtain the total structural response.

3.2 TIME HISTORY

The time-history method of an lysis has been utilized to analyze the SR-Class I structures for developing the in-structure response spectra (ISRS). These ISRS are used for seismically evaluating equipment installations. Mathematical models idealizing the structures have been generated to determine the time-history response of the buildings when subjected to the design earthquake time-history. For each building, all modes of vibration below 33 Hz have been considered for modal synthesis in each direction of excitation. A set of uncoupled modal equations, representing the idealized system under dynamic loading, has been solved using a mathematical routine such as the Runge-Kutta Fourth-Order stepwise numerical method. By algebraically combining the modal responses at each time increment, acceleration time-histories at the various floor elevations have been calculated.

This analysis has utilized the east-west component of the earthquake as recorded in 1940 at El Centro, California . This accelerogram was scaled to 8 percent gravity for

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1 See Appendix B for the formulation for obtaining Modal Damping.

2 See Appendix C for the formulation of the Modal Synthesis Technique.

maximum OBE. Figures 1.3, 1.4 and 1.5 show comparisons of the El Centro Response Spectra versus the CCNPP Design (Ground) Response Spectra for 1/2, 2 and 5 percent damping respectively. As can be seen, the response spectra of the El Centro east-west component envelope the CCNPP Design Response Spectra with a considerable margin at or near the natural frequencies of all Class I structures, thus assuring conservatism.

For OBE, the vertical in-structure response spectra for each building were generated by using the time-history analysis method where the vertical component of the 1940 El Centro earthquake was scaled to 5.3 percent gravity. DBE vertical response spectra were obtained as 1.875 times that of OBE.

The in-structure response spectra (ISRS) have been generated by first applying the recorded earthquake to the structural model and obtaining the time-history response at selected mass points such as floor elevations. Each of the floor time histories was then applied to a single-degree-of-freedom system whose natural frequency and damping value were varied. The calculations to construct the ISRS curves were accomplished using Bechtel Computer Program CE-611. Computed response values were enveloped to smooth out the erratic response of the earthquake's random behavior. At the high frequency end of the ISRS, the acceleration level converged to a value which is the building's floor acceleration, otherwise known as zero-period acceleration (ZPA).

These time-history records were used to develop the in-structure response spectra for several values of component damping. The in-structure response spectra were constructed by plotting the maximum response of interest at a selected set of frequency[3] data points. Response spectra peaks associated with structural resonances were broadened by +/- 10 percent of the peak frequency value and subsequently smoothed to account for uncertainties in the model representations.

For CCNPP, in-structure response spectra (ISRS), or floor response spectra (FRS) have been generated for all floor elevations of the auxiliary building, and at various elevations within the containment and intake structures. Spectra are available for both OBE and DBE for three directions of seismic excitation, and component damping

[3] Table 2.0 lists the frequencies at which response was determined.

values ranging from 1/2 percent to 7 percent. In-structure response spectra for DBE (15 percent gravity) were obtained by increasing the OBE spectra by a factor of 1.875; [0.15/0.08 = 1.875].

During the design of the high density spent fuel racks (August, 1976) for the Auxiliary Building Spent Fuel Pool, additional Auxiliary Building in-structure response spectra were generated. The natural frequencies and mode shapes of the Auxiliary Building, as obtained earlier, were used for this ISRS regeneration. These new ISRS were obtained for additional values of component damping using two horizontal directions of Auxiliary Building seismic excitation. These ISRS have also been generated for use in any future analysis of Class I equipment installed in the Auxiliary Building.

4.0 MATHEMATICAL MODELS

For the Containment Building, Auxiliary Building, and Intake Structure, the mathematical models have idealized these structures into lumped parameter systems of mass, stiffness and damping assuming linear elastic behavior where the stiffness is modeled using plane frame beam elements.

4.1 CONTAINMENT STRUCTURE

The Containment Structure is a prestressed cylindrical shell with a hemispherical dome and a reinforced concrete flat circular bottom slab. The interior structures consist of two secondary shield walls, two steam generator pedestals supporting the steam generators and a primary shield wall housing the reactor which is supported by the reactor cavity wall. The interior structures are not attached to the cylindrical shell but are rigidly supported by the base slab. For the mathematical model of the Containment Structure, the exterior as well as internal structures are treated as an integral unit.

The Containment Structure has been idealized (Figure 2.1) into a plane frame representing all the mass and structural properties of the building. The plane coincides with the East-West centerline of the Containment Structure. Previous preliminary analysis demonstrated that the responses are nearly equal for the North-South and the East-West directions.

The containment shell and the interior structures have been integrally modeled by considering them both fixed to the rigid base slab. The horizontal distortions of the cylindrical shell have been neglected and thus the shell has been idealized into a solid vertical beam cantilevered from the base slab. The containment interior structures have been idealized as a plane frame consisting of vertical and horizontal elastic members supported by the base slab. These members geometrically coincide with the East-West cross section of the structure, and each member has the same propeties, (i.e.: cross section area, effective shear area, moment of inertia and the modulus of elasticity) as the actual structural components being represented.

In the mathematical model, the base slab has been idealized to rest on equivalent vertical and horizontal beams representing the soil characteristics under the slab. All joints on the base slab have been considered constrained.

That is, their lateral displacements have been assumed to be equal. However, the base slab has been allowed to rotate on the soil springs as a rigid body. All masses considered to be effective at these constrained joints have been considered additive. The section of this technical discussion related to soil structure interaction presents additional soil characteristics information.

4.1.1 Snubbers

Steam generator snubbers are located at Elevation 75'-00" between the top portion of the steam generators and the surrounding concrete walls. The purpose of these snubbers is to limit the deflection of the upper portion of the steam generators during any kind of shock. For a shock condition producing a velocity in excess of 0.5 inch/minute, their inherent stiffness is infinite. Therefore, the stiffness due to the snubber axial deformation has been considered in the seismic model.

4.1.2 Secondary Shield Walls, Buttresses and Refueling Canal Wall

Five secondary shield walls, four buttresses and one refueling canal wall on each side of the containment have been represented in the model using a vertical member located at the shear center of the interior structural elements. These structural elements have been combined into a single model member because, under vibration, they have been considered to act in phase. Therefore, two vertical members, one on each side of the centerline of containment, represent the main structural resistance of containment interior structures.

4.1.3 Flexibility Coefficients

For each model beam element, three planar degrees of freedom have been considered when formulating the element flexibility matrix. The total model flexibility matrix was formulated by combining these element flexibilities, either in series or in parallel, depending on their structural function.

For the Containment Structure model, the flexibility matrix was obtained by applying only a unit lateral load at each of the free joints and computing the

displacements at all other joints. This in effect has allowed only lateral dynamic degrees of freedom at each mass point, eliminating the vertical and rotational dynamic degrees of freedom. This reduced flexibility matrix was then inverted to obtain the stiffness matrix used for computing natural frequencies and mode shapes.

A separate vertical model has been used to evaluate the Containment Structure vertical seismic response.

4.1.4 Soil Damping for Model

During the analysis of the Containment Structure for horizontal seismic excitation, it was observed that the total response was approximated to a 99.98% accuracy (Table 3.0) by only considering the first three modes. The contributing modes were due to vibrations purely on the soil (i.e.; shearing translation, rocking and a combination of translation and rocking respectively). This phenomenon has been attributed mainly to the very soft soil at the Calvert Cliffs site.

The damping for the horizontal translation of a structure on soil is due principally to internal friction losses of soil. As shown in Table 1 and as given in the Calvert Cliffs UFSAR [Ref. 9], the percent of critical damping for soil is specified as 2 to 3 percent for a rigid body translation on soil and 5 to 7 percent for rocking motion on soil. These values were used to determine the modal damping for the dominant modes of the Containment Structure in accordance with the techniques described in Appendix B. Modal damping for the significant modes of vibration are given in Table 3.0.

The vertical motion on soil is also damped by radiation losses in addition to internal friction losses. This radiation loss is the energy in the form of compression waves created by the vertical vibration of the structure, which propagate away from the foundation and are eventually dissipated. A conservative value for radiation damping is given in Reference 7 as 5% of the critical damping.

Therefore, the combination of internal and radiation damping gives values of 7% to 8% which have been used for determining the seismic response due to the vertical vibration of the structure on soil.

It should be added that the Containment Structure is embedded approximately two-thirds of its height up to Elev. 45'-00" with soil fill. The effect of this embedment would be to increase the system damping and thereby reduce its response. Reference 6 indicates that the vertical damping value on soil may be increased by 3 to 3.5 times for foundations buried one-half to two-thirds of their height in soil. On this basis, a value of soil damping of 21 to 24 percent of the critical damping could have been used.

4.1.5 Vertical Model

For vertical response, the Containment Structure has been idealized as having only one translational degree of freedom. This idealization was based on preliminary studies which showed that because of the thick concrete members, the predominant flexibility of the system was due to the soil. The idealized system (Figure 5.1) consists of the Containment Structure mass acting on the soil (spring) stiffness. Estimation of the total vertical response using only a single mode has been justified since the stiffnesses of the containment shell and the interior structures are much greater in the vertical direction than in the horizontal direction. Therefore, the vertical response is assumed constant throughout the structure.

Since the model is a single degree-of-freedom system, the mode shape has a value of unity. This single degree-of-freedom (F_n = 2.62 Hz) vertical model of the Containment Structure with the above cited properties has been utilized for computing the vertical In-Structure (Floor) Response Spectra by the Time-History analysis method.

The idealized models of the Containment Structure are shown in Figure 2.1 and Figure 5.1. Modal parameters for the significant modes of vibration are given in Table 3.0. Additional details of the model, the seismic analysis and the generation of the floor response spectra are presented in Reference 4.

4.2 AUXILIARY BUILDING

The Auxiliary Building is located between the Containment Structures for Units 1 and 2. The foundation of the Auxiliary Building is rectangular with average dimensions of 120 feet in the North-South direction and 240 feet in the East-West direction.

The Auxiliary Building is divided into three separate structures, namely:

- o The main building which includes the diesel generator area, the spent fuel pool and fuel-handling areas and,
- o Two penetration buildings housing the fuel transfer tubes from each unit.

For horizontal seismic excitation, mathematical models have been generated using a cantilevered "stick" idealization. Lumped masses have been concentrated at the various floor elevations and are interconnected by stiffnesses representing the shear and flexural characteristics of the building walls. Also, included in the main building is the steel frame structure over the spent fuel pool and fuel-handling areas. The base slabs of the main auxiliary building and the penetration areas have been assumed to be rigid and are supported by horizontal and vertical beams which simulate the soil translational and rocking characteristics. Two horizontal models have been generated to determine the response to both north-south and east-west seismic excitation. A single lumped mass model has been used to evaluate response for vertical seismic excitation.

The Auxiliary Building has been constructed into three separate structures by introducing seismic joints. These structures are designated as the main building which includes the spent fuel pool, the diesel generator area and the fuel handling area, and two penetration buildings. Essentially, the design of the building consists of a reinforced concrete structure with walls interconnected by floors. The portion of the main building housing the spent fuel pool and fuel-handling area consists of a steel-framed structure supported by a concrete structure at the operating floor level (Elevation 69'-0"). The Auxiliary Building is supported by soil which supplies an elastic foundation.

To perform the seismic analysis for horizontal seismic excitation, each portion of the Auxiliary Building was

idealized into a lumped parameter system. The idealization was essentially a cantilevered beam representing the structure's mass and stiffness characteristics. Figures 3.1, 3.2, 3.3 and 3.4 show the models for the main building and the penetration area. The structural mass including significant equipment items, was lumped at the various floor elevations. The shear area between mass points was computed by only considering those elements which were connected to the main structural members by the shear transmitting walls. The remaining elements of the section area were regarded as columns, and their moment of inertias were additive to form the beam properties of the idealized member. The steel frame, over the spent fuel pool and fuel-handling area of the main building, was also included in the mathematical model.

The soil under the slabs of the main auxiliary building and the penetrations areas has been represented in the models by horizontal and vertical beams. These equivalent beam properties represent the soil translational and rocking stiffness characteristics and are shown in the mathematical models.

To perform the seismic analyses of the Auxiliary Building in the vertical direction, a single lumped mass system was formulated (Figure 5.2) where the total mass was based on the building weight and the soil characteristics provided the system's stiffness.

4.2.1 Steel Frame over Spent Fuel Pool Area

The steel frame structure between Elevation 69'-00" and Elevation 117'-00" was idealized into an equivalent single concrete element. For each direction of seismic excitation, the frame elements in this area of the Auxiliary Building together with the concrete wall at column line M_d, were evaluated separately to determine their individual flexibility. The total flexibility of this structure was obtained by combining the individual element flexibilities, either as elements in parallel or in series depending on the manner in which they performed their structural design function.

Once the total flexibility was obtained, a concrete beam element was calculated such that its area and moment of inertia would produce the equivalent deflection for a unit load. These concrete element properties were then incoporated into the total

Auxiliary Building model as the idealization of the frame structure over the spent fuel pool area.

4.2.2 Soil Damping for Model

For horizontal translation of a structure on soil, the damping is due to the internal friction losses in the soil and is given as 2 to 3 percent of the critical damping in the Calvert Cliffs Nuclear Power Plant UFSAR [Ref. 9]. For rigid body rocking on soil, the damping due to the internal friction within the soil is given as 5 to 7 percent. These values were used to determine the modal damping for the dominant modes of the Auxiliary Building in accordance with the techniques described in Appendix B. Modal damping for the significant modes of vibration of the Auxiliary Building are given in Table 4.0.

The Auxiliary Building, like the Containment Structure, is surrounded by soil fill up to El. 45'-00", and therefore is embedded halfway in the soil. The effect of soil embedment would be to increase the system damping. It is stated in Ref. 3 that the vertical damping is increased by 3 to 3.5 times for structures buried in soil one-half to two-thirds of their height. However, for these analyses, the damping effects due to embedment have conservatively been neglected.

4.2.3 Vertical Model

The soil parameters, as described in Section 5.1, have been used to determine the soil-spring constant [$K_{z(soil)}$] for formulating the vertical model of the Auxiliary Building as shown in Figure 5.2.

The Modulus of Elasticity [$E_{SoilDyn}$] for soil is 40,000 psi or 5760 kip/ft^2. Poisson's ratio is 0.4. References 6, 7 and 8 have been utilized for computing the soil stiffness characteristics, whereby:

[$K_{z(soil)}$] = 1.275×10^6 kip/ft]

The total weight of 142,000 kips for the Auxiliary Building has been included in the horizontal seismic analysis, including major equipment items. This value has also been the weight used for the vertical model.

[W_z = 142,000 kips]

The vertical motion is also damped by radiation losses in addition to frictional losses. The radiation losses are due to the vertical compression waves created by the vibration of the structure which propagate down into the soil and are eventually absorbed. The value given for radiation damping in Reference 2 is 5 percent. Therefore, the damping value used for vertical vibration of the Auxiliary Building is 10 percent and is obtained by combining the radiation and internal damping in the soil.

$[C/C_c = 0.10]$

For specifying the time history seismic input excitation in the vertical direction, the vertical component of the 1940 El-Centro earthquake was chosen.

The horizontal time history was specified by the East-West horizontal component of the 1940 El Centro earthquake. When the time-history analysis was performed, the accelerogram for the El Centro earthquake was amplified by the structure, utilizing the computed natural frequencies and modal damping values. The resultant amplified time-histories at each floor were then transformed into In-Structure (Floor) Response Spectra by using Bechtel Computer Program CE 611. These ISRS were generated for application to the seismic design of systems, equipment and piping with damping ratios of 1/2%, 1%, 2%, 2-1/2%, 3% and 5% as required to suit the material characteristics of the component being examined.

The idealized models of the Auxiliary Building are shown in Figure 3.1 through 3.4 and Figure 5.2. Modal parameters for the significant modes of vibration are given in Table 4.0. Additional details of the model, the seismic analysis and the generation of the floor response spectra are presented in Reference 3.

4.3 INTAKE STRUCTURE

The Intake Structure is a reinforced concrete structure which houses the circulating water and salt water pumps. The structure is approximately 384 feet long, 90 feet wide, and 45 feet deep and consists of interconnecting floors and walls supported by an invert slab. This slab (bottom floor) is continuous while the structure above is divided by two vertical expansion joints.

For the purpose of seismic analysis in the horizontal direction, the structure has been idealized into three vertical "stick" cantilevered beams supported by a common slab. Figures 4.1 and 4.2 show the Intake Structure models. The shear and flexural stiffnesses of the Intake Structure walls have been idealized into beam elements with the building masses being lumped at the floor elevations. The invert slab has been idealized as a rigid beam supported by springs which represent the stiffness of the soil below the slab. Two horizontal models have been formulated representing the structural characteristics for seismic excitation in both the east-west and north-south directions. For vertical seismic excitation, the structure has been idealized as a single-degree-of-freedom system (Figure 5.3) where the main flexibility is contributed by the soil under the structure.

The idealized models of the Intake Structure are shown in Figure 4.1 and 4.2 and Figure 5.3. Modal parameters for the significant modes of vibration are given in Table 5.0. Additional details of the intake structure model, the seismic analysis results and the floor response spectra are given in the Intake Structure Seismic Analysis Report, January 1972, [Ref. 5].

5.0 SOIL-STRUCTURE INTERACTION

The stiffness and damping effects of the supporting soil for the Containment Structure, Auxiliary Building and Intake Structure have been considered in each of the seismic analyses. The following describes those considerations used for the Auxiliary Building, but analogous considerations are also applicable to the other two primary structures.

Since the base slab of the Auxiliary Building varies in thickness between 4 and 10 feet, the average thickness of the slab at elevation -15.00 feet, used for analysis purposes, is approximately 10 feet. The base dimensions of the Auxiliary Building are approximately 120 feet by 240 feet. The base slab can be considered to be infinitely rigid compared to the supporting soil medium. However, because the soil under the base slab will affect the system natural frequencies and hence the response of the structure, its flexibility and damping properties are incorporated in the model.

The effects of the foundation soil medium has been considered in the following manner:

a. Stiffness coefficients were computed from the soil properties for horizontal translation, vertical translation and rocking, (i.e., rotation of the foundation about its owns axis.)

b. Members were attached to the base slab having properties and a configuration which would produce equivalent stiffness coefficients of the soil and simulate equivalent rigid body degrees of freedom of the slab.

In the original seismic analysis of the Containment and the Auxiliary Building, embedment up to elevation 45'-00" was neglected because:

a. The applied contact pressure was assumed to be negligible.

b. During any vibratory motion, the fill at the sides of the building was assumed not to create any elastic rebound.

c. The embedment effect would tend to increase the energy dissipation under seismic excitation.

Therefore, neglecting it was considered conservative.

d. No effective mass of soil was assigned to the model.

5.1 SOIL PARAMETERS

The type of soil at the Calvert Cliffs site can be described as "sandy silt or silty sand" per data from soil borings[4] under each containment site. The data from these borings yields the average value of soil weight density (w) to be:

$$w = 114.2 \text{ pcf}$$

The dynamic modulus of elasticity ($E_{Soil\ Dyn}$) was obtained from the dynamic triaxial test of soil samples[5] as:

$$E_{Soil\ Dyn} = 40{,}000 \text{ psi}$$

Since the static modulus of elasticity (E_{Static}) is a function of the contact pressure between the foundation and the underlying soil, an empirical formula[6] was established to define this modulus as:

$$E_{Static} = 2000 + 0.625 \times \sigma_v$$

where σ_v is the contact pressure.

The dynamic triaxial tests were conducted on undisturbed soil samples, under a lateral pressure equivalent to the confining pressure, by vibrating the sample through various ranges of input amplitude. Results are given in Figure 6.1 which show $E_{Soil\ Dyn}$ versus strain amplitude.

Poisson's ratio (μ) for soil[6] for static as well as dynamic response was established as:

$$\mu_{static} = 0.30$$

$$\mu_{dynamic} = 0.40$$

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[4] Reference 9, Appendix 2E

[5] Reference 9, Dames & Moore Test Reports

[6] As recommended by Bechtel Geotechnical Group

6.0 SEISMIC ANALYSIS RESULTS

6.1 FREQUENCIES AND MODE SHAPES

The natural frequencies and associated mode shapes have been obtained using Bechtel Computer Programs CE-617 and CE-917. These programs solved the free-vibration problem for the structural model using the method of diagonalization by successive rotations. The solution has resulted in up to ten modes of vibration for each direction of lateral excitation. The mode shapes indicate how the mass points in the model are displaced with respect to each other for a particular natural frequency of vibration.

An examination of these modes reveals that the effects of soil and its associated damping is a critical factor in the dynamic response of the structural system. The percentages of critical damping for soil conform to those presented in the UFSAR, and are considered extremely conservative in comparison to those presented in current literature and research.

Tables 3.0 through 5.0 summarize the modal parameters for each of the primary Class I structures by giving the dominant natural frequencies, modal participation factors and modal damping values for three directions of seismic excitation.

6.2 BUILDING RESPONSE

The acceleration inputs for each natural frequency were obtained from the Design (Ground) Response Spectra for OBE (8 percent gravity maximum acceleration) and DBE (15 percent gravity maximum acceleration) as shown in Fig. 1.1 and Fig. 1.2 respectively as given in Reference 9.

Utilizing the Design (Ground) Response Spectra, the seismic response of the structure was computed by the normal-mode technique. This normal-mode technique computed the probable maximum accelerations, displacements, and inertia forces at the model mass points for each mode. The total seismic response was obtained by Bechtel Computer Program CE-641 using a modal synthesis technique.

6.3 IN-STRUCTURE (FLOOR) RESPONSE SPECTRA

In-Structure Floor Response Spectra (ISRS) or Floor Response Spectra (FRS) for Calvert Cliffs Nuclear Power Plant, Units 1 & 2 are given in References 3, 4 and 5. Selected representative ISRS for the primary Class I structures are included in Appendix A of this Attachment.

The ISRS are to be used for the seismic design of Class I equipment and piping at various elevations within the Class I Buildings. For the seismic design of equipment subjected to seismic excitation, the DBE values of FRS are obtained by multiplying the OBE values by 1.875. The vertical seismic forces should be assumed to act either upward or downward simultaneously with the horizontal seismic force acting in the most unfavorable direction.

To determine seismic loads on Class I systems and equipment, the in-structure response spectra shall be applied, in a simplified single degree-of-freedom fashion, as follows:

a. Determine which response spectra is applicable, using information from the specification for equipment location and the appropriate damping value.

b. Evaluate the natural frequency of the equipment in both the horizontal and the vertical directions. The horizontal frequency shall be computed in the direction that will give the highest stresses.

c. Utilizing the natural frequency, enter the applicable response spectra (ISRS) to obtain the maximum response acceleration.

d. The horizontal and the vertical seismic forces are equal to the mass of the equipment times the respective response acceleration.

e. Both a horizontal and a vertical force shall be applied simultaneously at the center of gravity of the equipment to evaluate structural adequacy.

For seismic analyses, the Damping Values (Percent of Critical Damping) as given in Table 1.0 shall be used.

Test data may be used to determine damping used for performing a dynamic analysis. Such data may originate from information such as dynamic environments encountered in equipment transportation or actual dynamic tests.

The results of the seismic analysis are summarized in the Tables of Appendix A for selected points in each primary Class I structure. For each of the primary Class I structures, the floor acceleration and the peak response acceleration (from the ISRS at the resonant frequency) for selected values of component damping, are listed for several building elevations due to the Operating Basis Earthquake (OBE) and the Design Basis Earthquake (DBE). Appendix A also gives In-Structure Response Spectra for selected locations of the Class I structures. As can be seen, these spectra are all considered narrow banded.

To summarize, In-structure Response Spectra (ISRS or FRS) for the Containment, Auxiliary and Intake Structures are given in Appendix A and in References 3, 4 and 5, respectively. Each of the references contains all of the ISRS used at CCNPP and define the licensing commitment with respect to the seismic requirements for equipment installed within these Class I buildings.

7.0 REFERENCES

1 "Nuclear Reactors and Earthquakes"; TID-7024; by Lockheed Aircraft Corp. and Holmes and Narver, Inc.; prepared for USAEC; August 1963.

2 "Seismic Analysis of Structures and Equipment for Nuclear Power Plants"; Bechtel Design Guide 2.44; Bechtel Power Corp. 1980 (Topical Report BC-TOP-4-A, Rev. 3, November 1974).

3 "Auxiliary Building Updated Seismic Analysis for Job 6750"; for CCNPP, BG&E; by Bechtel Associates; 1976 (Bechtel Calculation No. C-1007.1).

4 "Containment Building Updated Seismic Analysis for Job 6750"; for CCNPP, BG&E; by Bechtel Associates; July 1970, Rev. 1; June 1982 (Bechtel Calculation No. C-1007.2).

5 "Intake Structure Seismic Analysis for Job 6750"; for CCNPP, BG&E; by Bechtel Associates; January 1972 (Bechtel Calculation No. C-1007.3).

6 "Vibration Analysis and Design of Foundations for Machines and Turbines"; by A. Major; Collet's, London; 1962.

7 "Design Procedures for Dynamically Loaded Foundations"; by R. V. Whitman and F. E. Richart; ASCE Soil Division Journal; November, 1967.

8 "Design Criteria for Nuclear Reactors Subjected to Earthquake Hazards"; by N. M. Newmark; Proceedings of the International Atomic Panel on Aseismic Design and testing of Nuclear Facilities; Tokyo, Japan; May 25, 1967.

9 "Updated Final Safety Analysis Report, Calvert Cliffs Nuclear Power Plant"; Revision 12; Baltimore Gas and Electric Company; April 1992.

TABLES

TABLE 1.0

MATERIAL DAMPING VALUES
(Percent of Critical Damping)

STRUCTURE OR COMPONENT	CALVERT CLIFFS FSAR OBE	CALVERT CLIFFS FSAR DBE	NRC REG. GUIDE 1.61 OBE	NRC REG. GUIDE 1.61 SSE
Steel piping & Equipment (pipe diam. > 12")	0.5	0.5	2.0	3.0
Small Diameter Piping (pipe diam. < 12")	0.5	0.5	1.0	2.0
Welded Steel Plate Assemblies	1.0	1.0	--	--
Welded Structural Steel	2.0	2.0	2.0	4.0
Bolted Structural Steel	2.5	2.5	4.0	7.0
Reinforced Concrete Structure	3.0	5.0	4.0	7.0
Rigid Body Translation on Soil	3.0	3.0	--	--
Rigid Body Rocking on Soil	5.0	7.0	--	--

TABLE 2.0

FREQUENCIES FOR GENERATING
IN-STRUCTURE RESPONSE SPECTRA VALUES

Frequency (Hz)	Frequency (Hz)
0.20	4.00
0.30	4.40
0.40	4.70
0.50	5.00
0.60	5.50
0.70	6.00
0.80	6.50
0.90	7.00
1.00	7.50
1.10	8.00
1.20	8.50
1.30	9.00
1.40	10.00
1.50	11.00
1.60	12.00
1.70	13.00
1.80	14.00
2.00	15.00
2.20	16.50
2.40	18.00
2.60	20.00
2.80	22.00
3.00	25.00
3.30	28.00
3.60	33.00

In addition to the above frequencies, response was computed at all system natural frequencies below 33 Hz.

Table 3.0

CONTAINMENT STRUCTURE

MODAL PARAMETERS FOR SIGNIFICANT MODES

East-West Motion

Mode No.	Frequency	Modal Weight	Participation	Modal Damping OBE	Modal Damping DBE
	(Hz)	(Kips)	(Percent)	(Percent)	(Percent)
1	0.97	63745.0	59.91	4.46	6.26
2	3.21	41109.0	38.64	2.33	3.65
3	5.84	1521.2	1.43	3.15	4.59
SUM =			99.98		

Vertical Motion

Mode No.	Frequency	Modal Weight	Participation	Modal Damping OBE	Modal Damping DBE
	(Hz)	(Kips)	(Percent)	(Percent)	(Percent)
1	2.75	106400.	100.0	7.0	8.0
SUM =			100.0		

Table 4.0

AUXILIARY BUILDING

MODAL PARAMETERS FOR SIGNIFICANT MODES

North-South Motion

Mode No.	Frequency	Modal Weight	Participation	Modal Damping OBE	Modal Damping DBE
	(Hz)	(Kips)	(Percent)	(Percent)	(Percent)
1	1.57	21368.0	15.05	3.16	4.73
2	2.08	113880.	80.20	3.11	4.24
3	6.11	6617.6	4.66	----	----
SUM =			99.91		

East-West Motion [1985 Reanalysis]

Mode No.	Frequency	Modal Weight	Participation	Modal Damping OBE	Modal Damping DBE
	(Hz)	(Kips)	(Percent)	(Percent)	(Percent)
1	2.11	134313.	94.59	25.92	----
2	4.53	6640.9	4.68	13.07	----
SUM =			99.27		

Vertical Motion

Mode No.	Frequency	Modal Weight	Participation	Modal Damping OBE	Modal Damping DBE
	(Hz)	(Kips)	(Percent)	(Percent)	(Percent
1	2.58	142000.	100.0	10.0	10.0
SUM =			100.0		

Table 5.0

INTAKE STRUCTURE

MODAL PARAMETERS FOR SIGNIFICANT MODES

North-South Motion

Mode No.	Frequency	Modal Weight	Participation	Modal Damping OBE	Modal Damping DBE
	(Hz)	(Kips)	(Percent)	(Percent)	(Percent)
1	2.51	136600.	99.98	2.04	3.06
2	21.93	21.6	0.00	3.41	4.90
SUM =			99.98		

East-West Motion

Mode No.	Frequency	Modal Weight	Participation	Modal Damping OBE	Modal Damping DBE
	(Hz)	(Kips)	(Percent)	(Percent)	(Percent)
1	2.43	135560.	99.22	2.25	3.31
2	6.61	521.4	0.00	3.41	4.56
3	7.44	292.8	0.00	3.17	4.75
SUM =			99.22		

Vertical Motion

Mode No.	Frequency	Modal Weight	Participation	Modal Damping OBE	Modal Damping DBE
	(Hz)	(Kips)	(Percent)	(Percent)	(Percent)
1	3.05	136625.	100.0	5.00	5.00
SUM =			100.0		

FIGURES

FIGURE 1.1
DESIGN (GROUND) RESPONSE SPECTRA
HORIZONTAL OBE

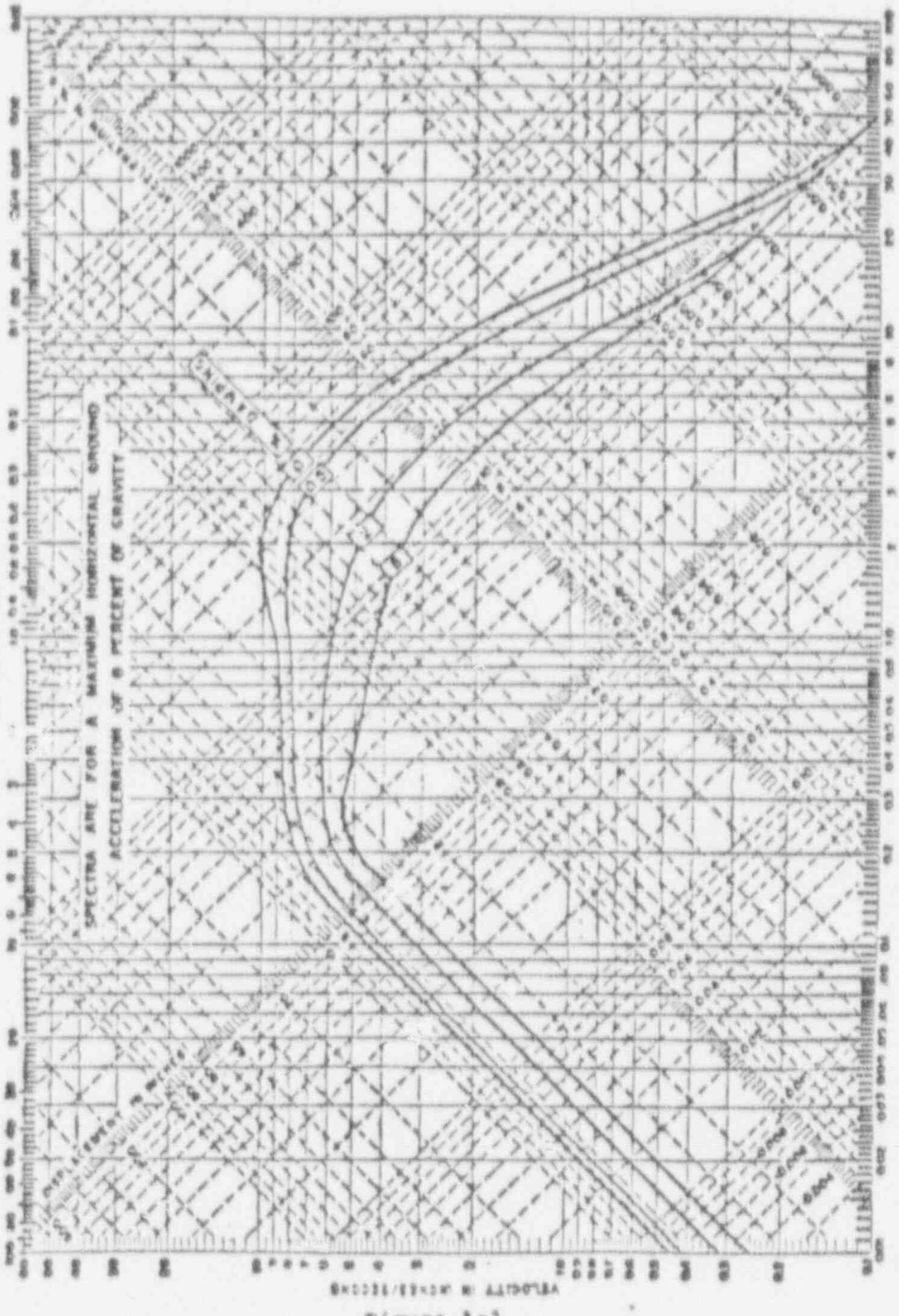


Figure A-1
RESPONSE SPECTRA
OPERATING BASIS EARTHQUAKE

DAMES & MOORE

FIGURE 1.2
DESIGN (GROUND) RESPONSE SPECTRA
HORIZONTAL DBE

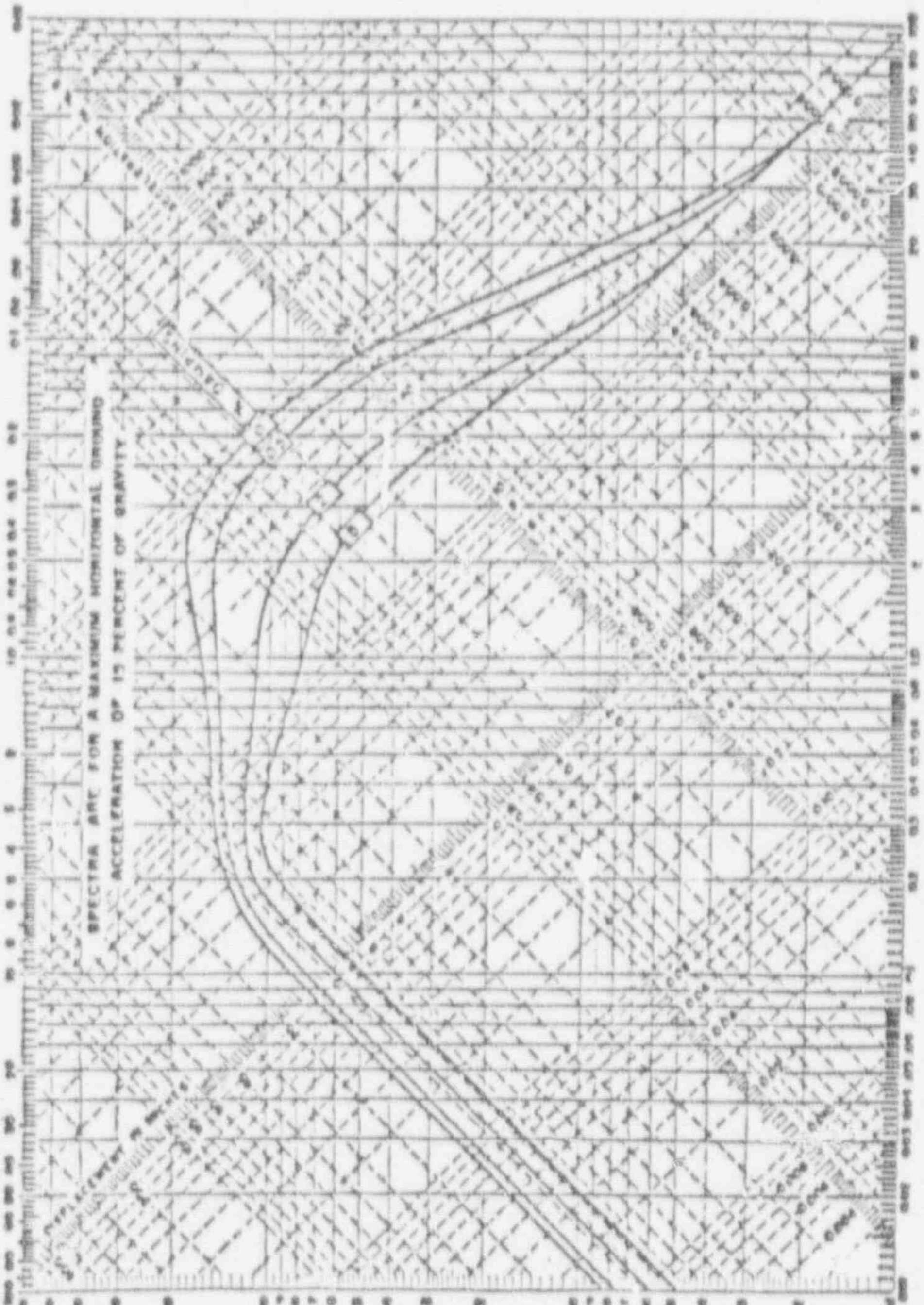


Figure A-2
RESPONSE SPECTRA
DESIGN BASIS EARTHQUAKE

DAMES & MOORE

FIGURE 1.3
DESIGN (GROUND) RESPONSE SPECTRUM COMPARISON
1/2 PERCENT DAMPING
EAST-WEST COMPONENT OF EL CENTRO EARTHQUAKE

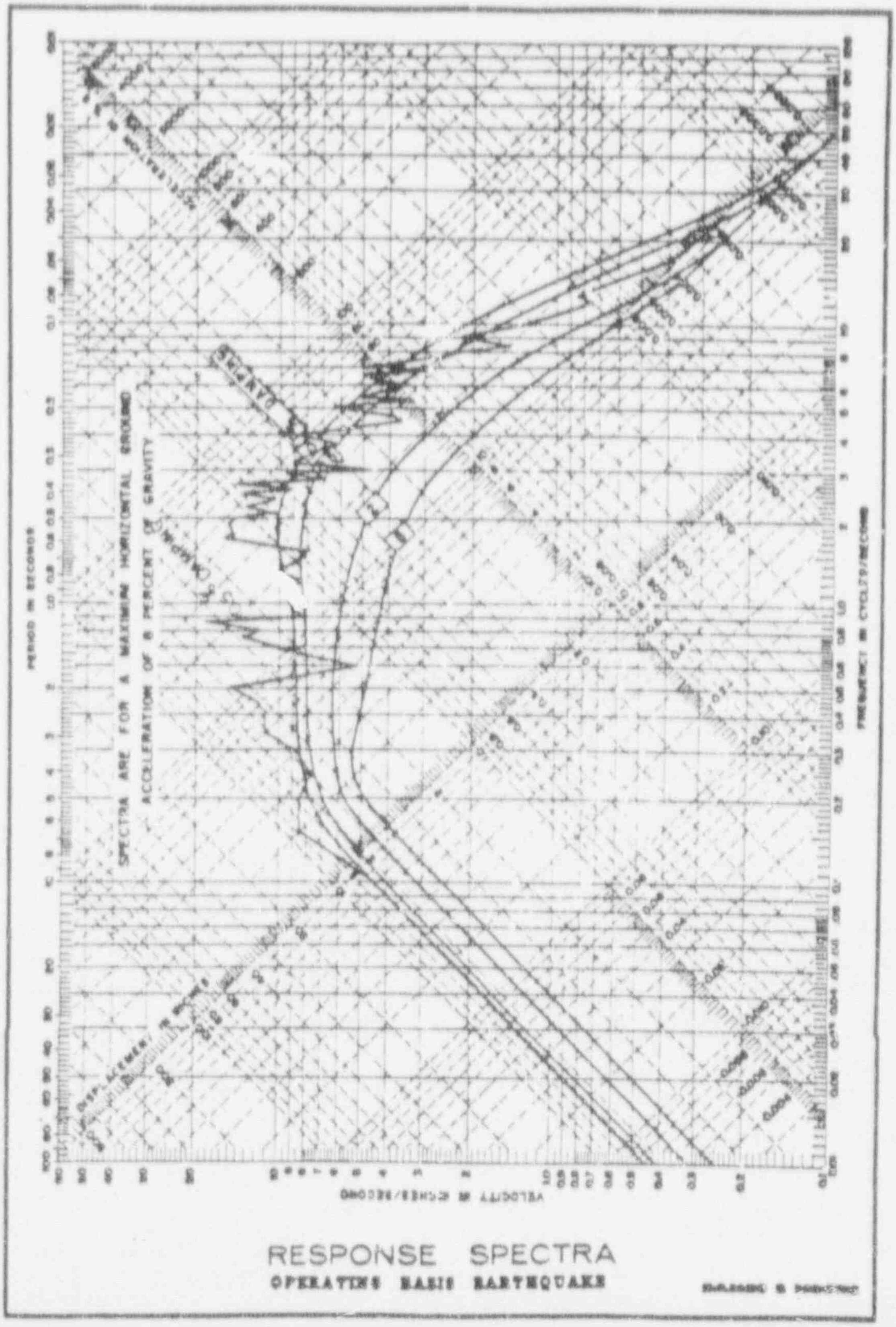


FIGURE 5A-2

FIGURE 1.4
DESIGN (GROUND) RESPONSE SPECTRUM COMPARISON
2 PERCENT DAMPING
EAST-WEST COMPONENT OF EL CENTRO EARTHQUAKE

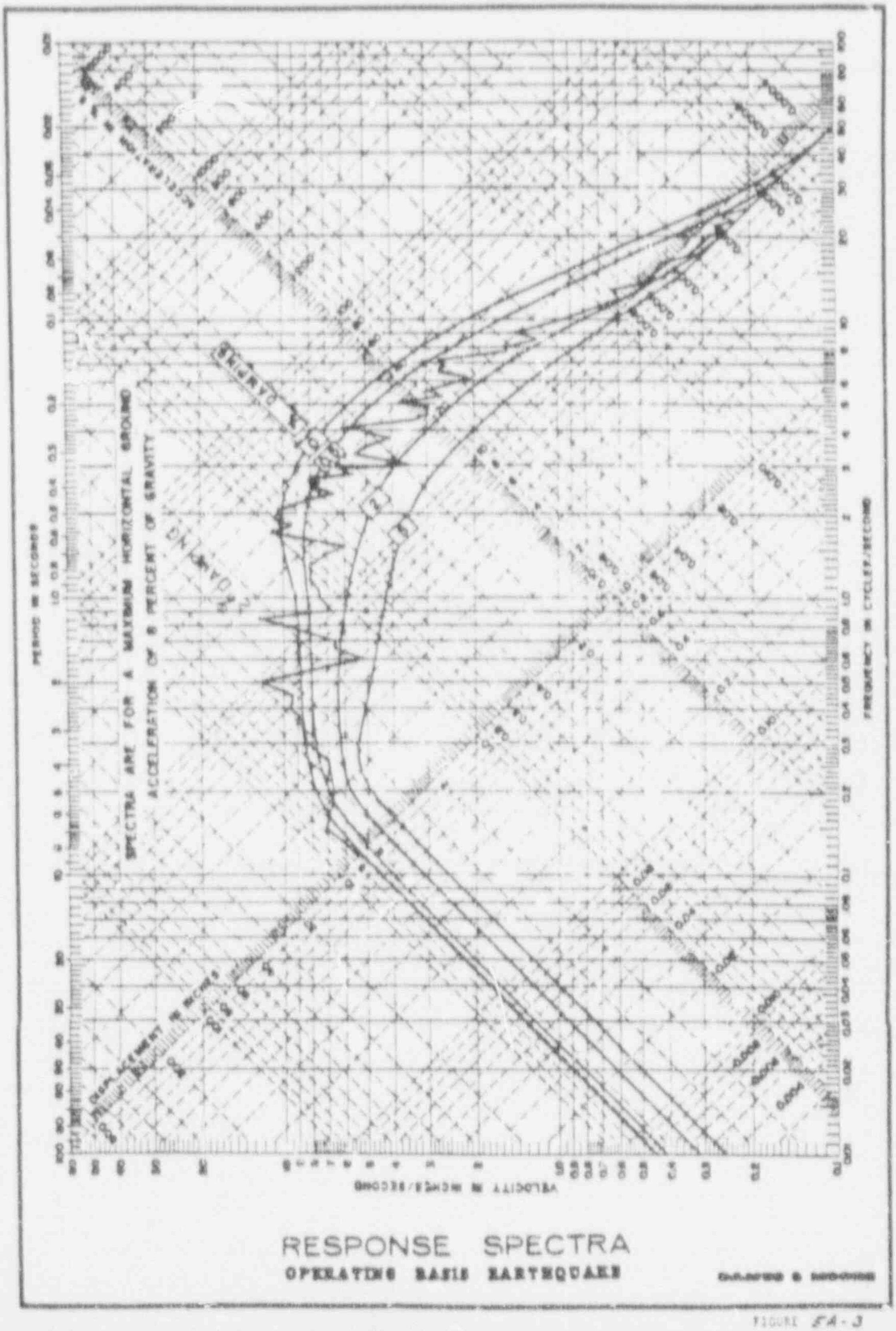


FIGURE 5A-3

FIGURE 1.5
DESIGN (GROUND) RESPONSE SPECTRUM COMPARISON
5 PERCENT DAMPING
EAST-WEST COMPONENT OF EL CENTRO EARTHQUAKE

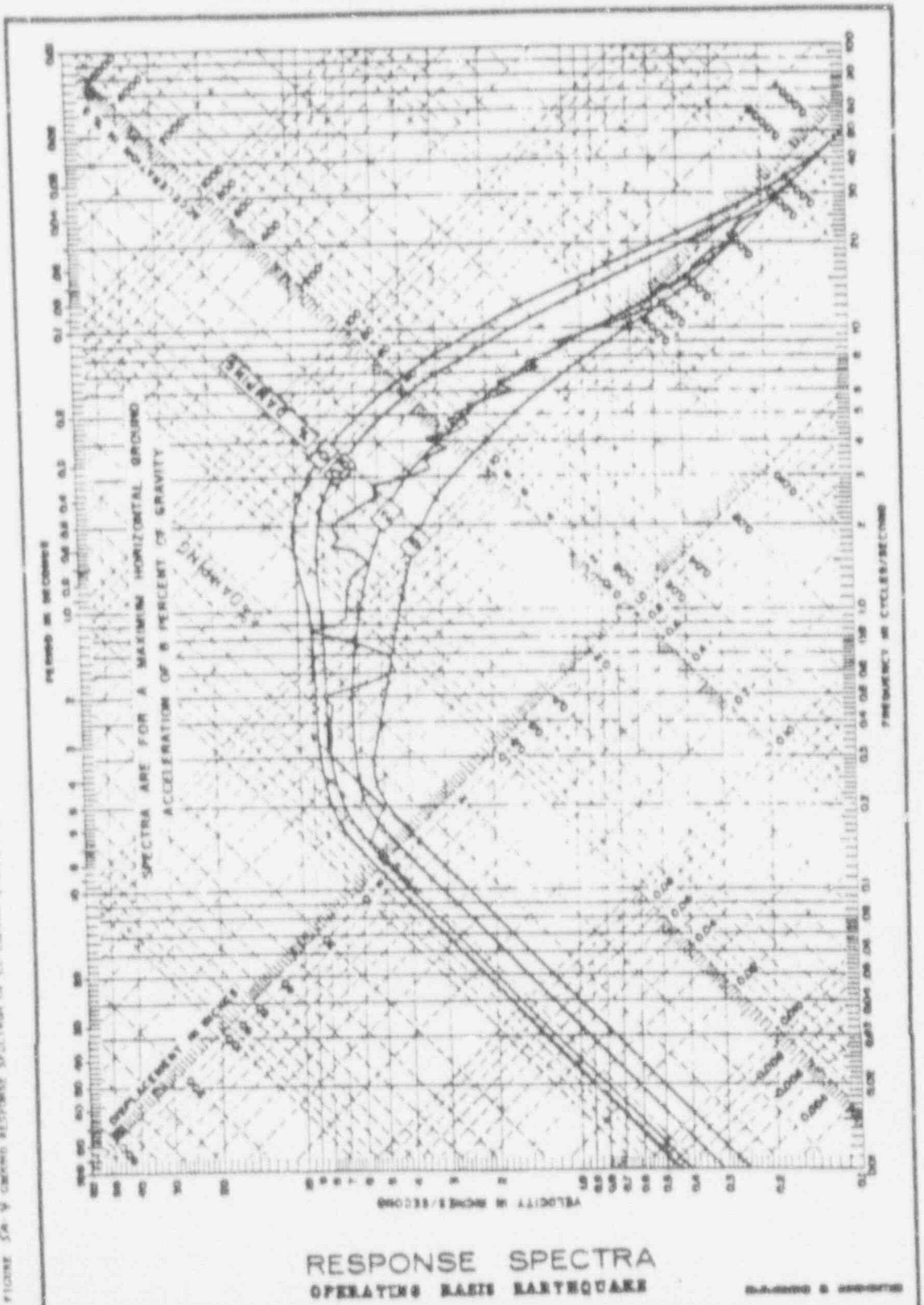
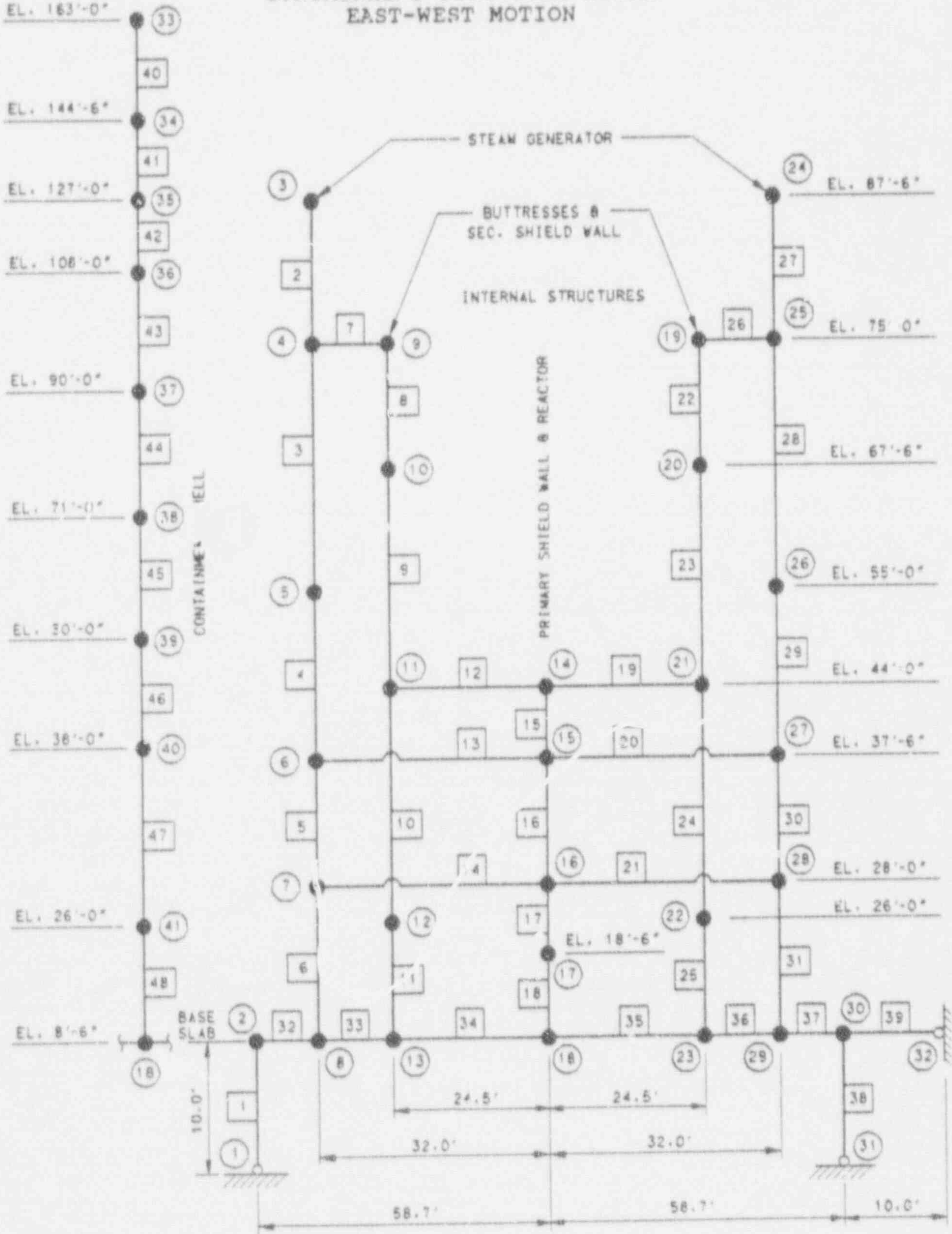


FIGURE 5A-4

FIGURE 2.1
CONTAINMENT STRUCTURE MODEL
EAST-WEST MOTION

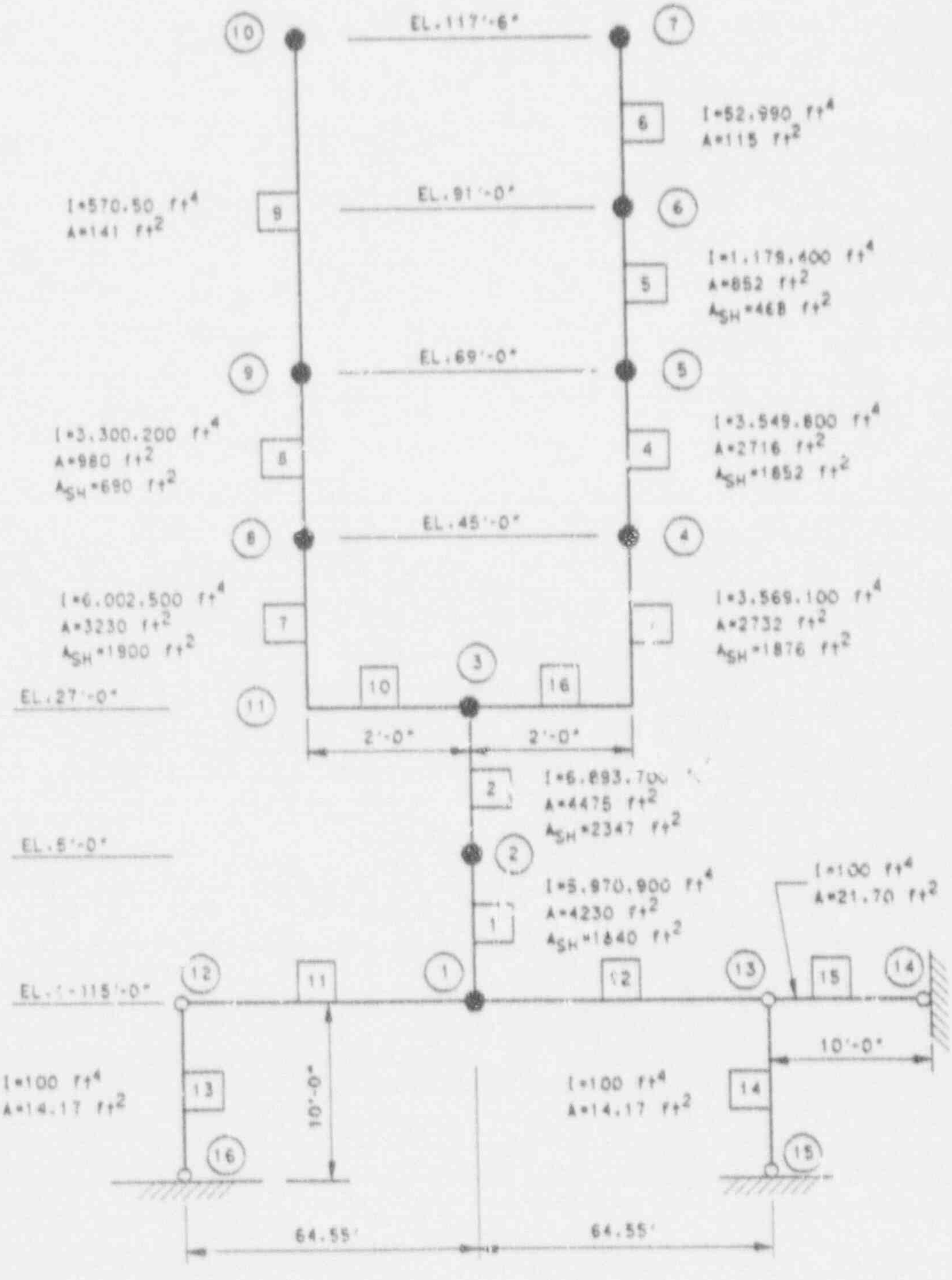


MEMBERS 32,33,34,35,36 & 37 ARE "RIGID"

FIGURE 2.2
CONTAINMENT STRUCTURE MODEL
ELEMENT PROPERTIES FOR EAST-WEST MOTION

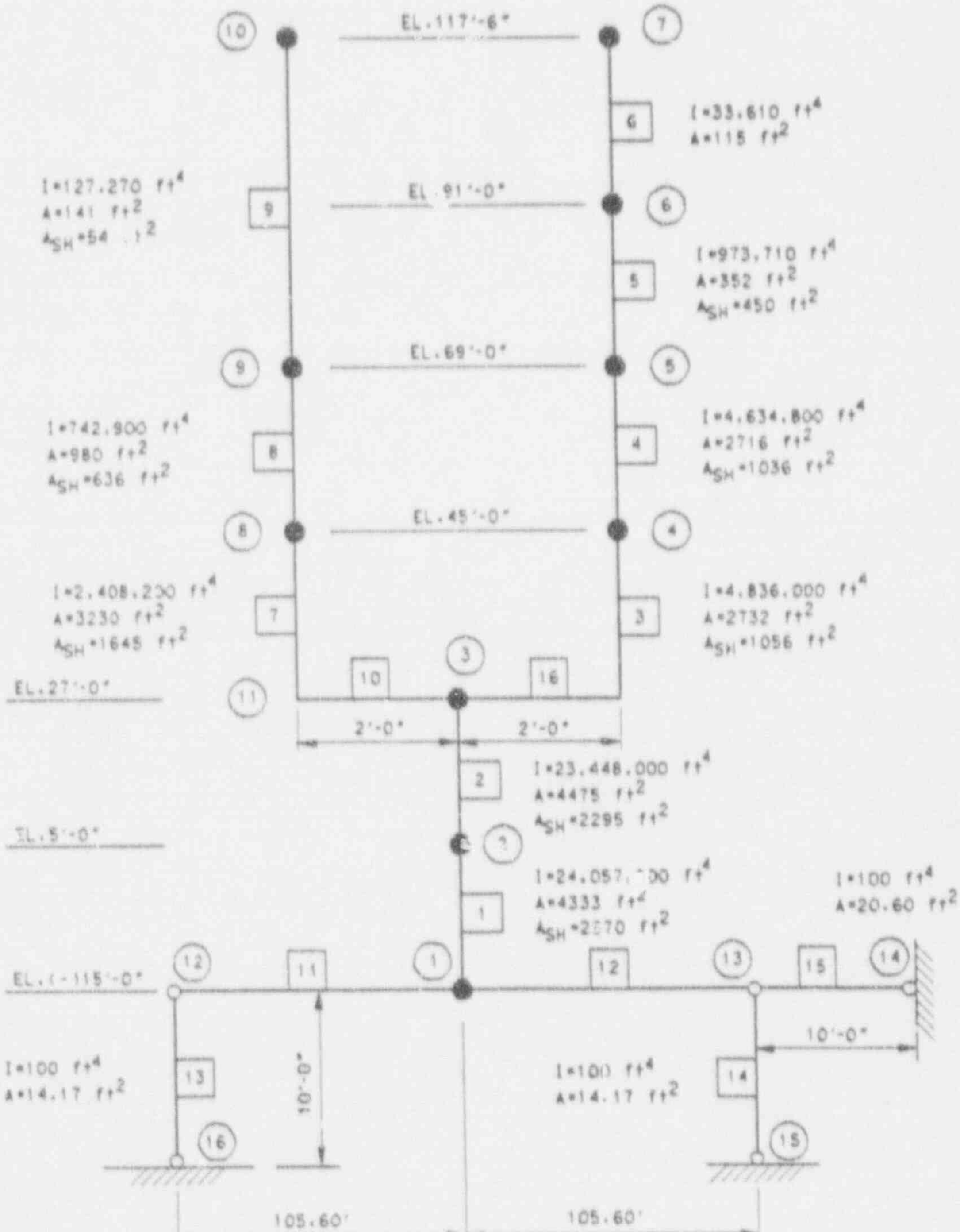
Member Number	Area	Shear Area	Moment of Inertia	Model Component
	(Ft^2)	(Ft^2)	(Ft^4)	
1, 38	8.427	1.0	1.0	Soil
2, 27	24.7	12.3	1185.0	Steam Generator
3, 28	18.5	9.3	462.6	Steam Generator
4, 29	15.3	7.7	343.0	Steam Generator
5, 30	9.1	6.2	83.0	Steam Generator
6, 31	144.0	96.0	1728.0	Steam Generator
7, 26	1.5	1.0	10.0	SG Snubbers
8, 22	398.0	265.5	733.0	Secondary Shield Walls
9, 23, 10, 24	1065.0	710.0	4484.0	Secondary Shield Walls
11, 25	200.0	131.2	1196.0	Secondary Shield Walls
12, 19	325.0	217.0	8770.0	Int. Structure
13, 20	8.1	4.1	14.9	Main Piping
14, 21	135.5	90.4	406.6	Int. Structure
15	601.0	401.0	58691.0	Primary Shield Wall & Reactor
16, 17, 18	672.0	448.0	64125.0	Primary Shield Wall & Reactor
32, 37	1118.0	745.0	9313.0	Base Slab
33, 36	1333.0	889.0	11083.0	Base Slab
34, 35	1419.0	946.0	11825.0	Base Slab
39	12.0	10.0	10.0	Soil
40	4348.0	2865.0	9073000.0	Containment Dome
41 thru 48	1576.0	1051.0	3526000.0	Containment Shell

FIGURE 3.1
AUXILIARY BUILDING MODEL
NORTH-SOUTH MOTION



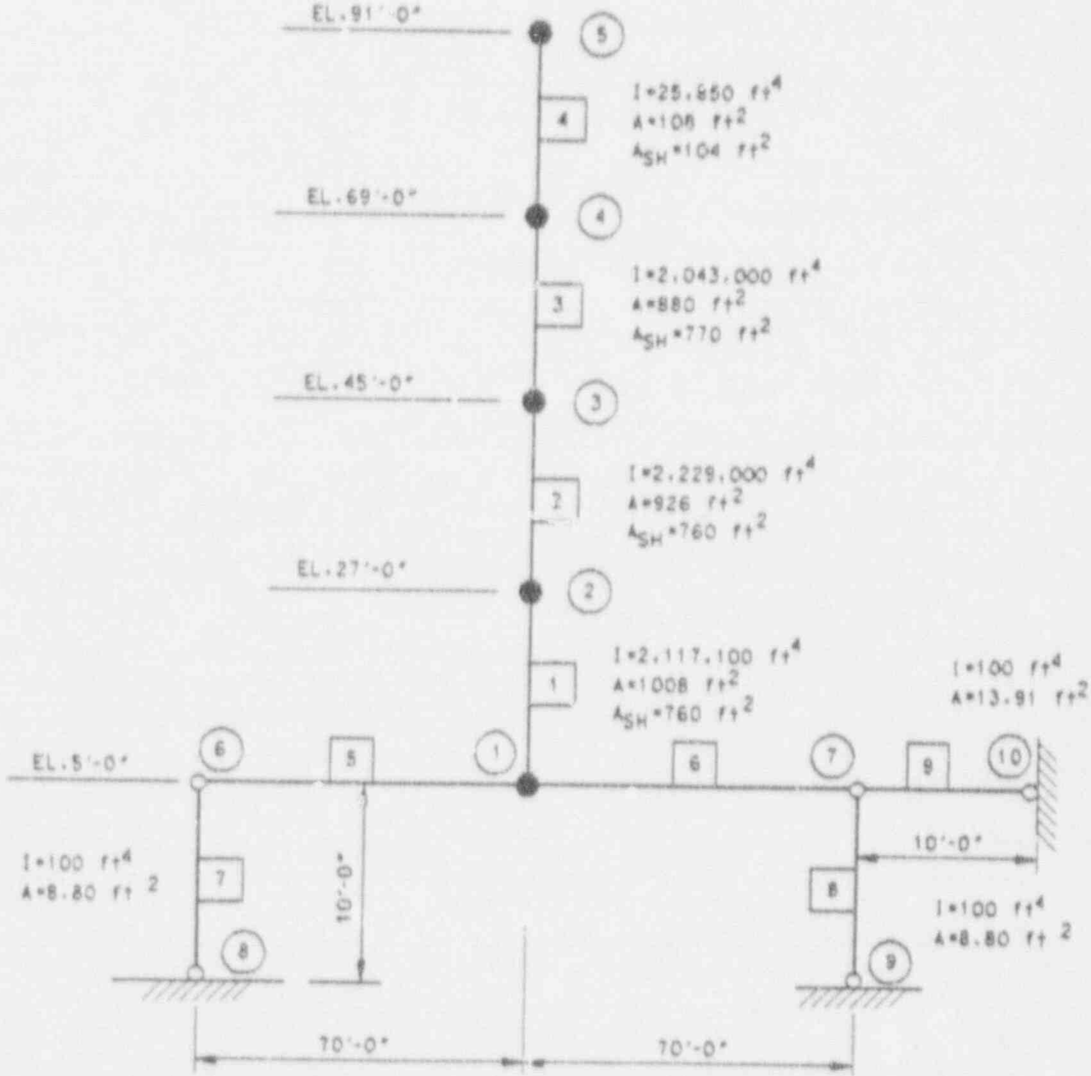
MEMBERS 10,11,12 & 16 ARE "RIGID"

FIGURE 3.2
AUXILIARY BUILDING MODEL
EAST-WEST MOTION



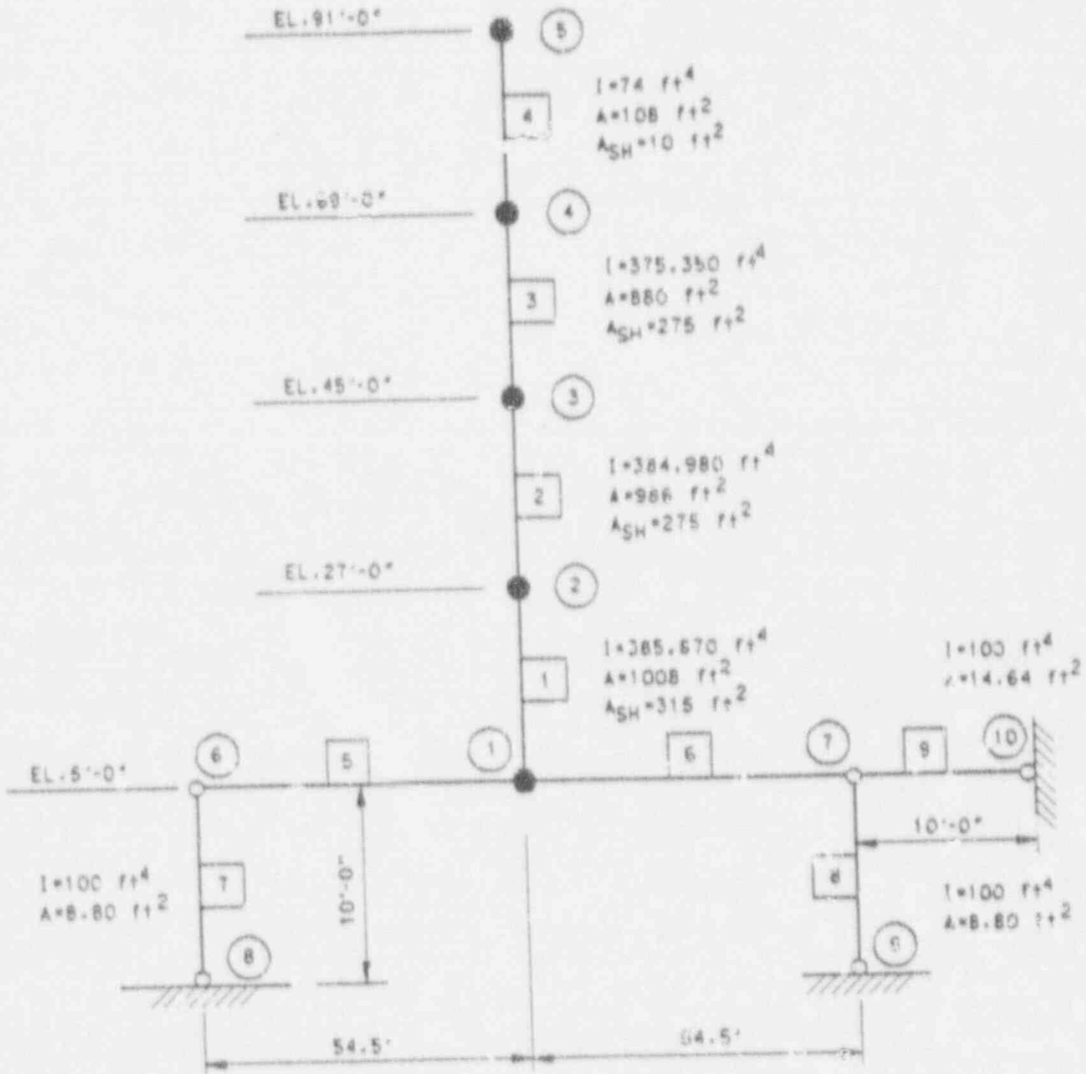
MEMBERS 10,11,12 & 16 ARE "RIGID"

FIGURE 3.3
AUXILIARY BUILDING MODEL
PENETRATION AREA
NORTH-SOUTH MOTION



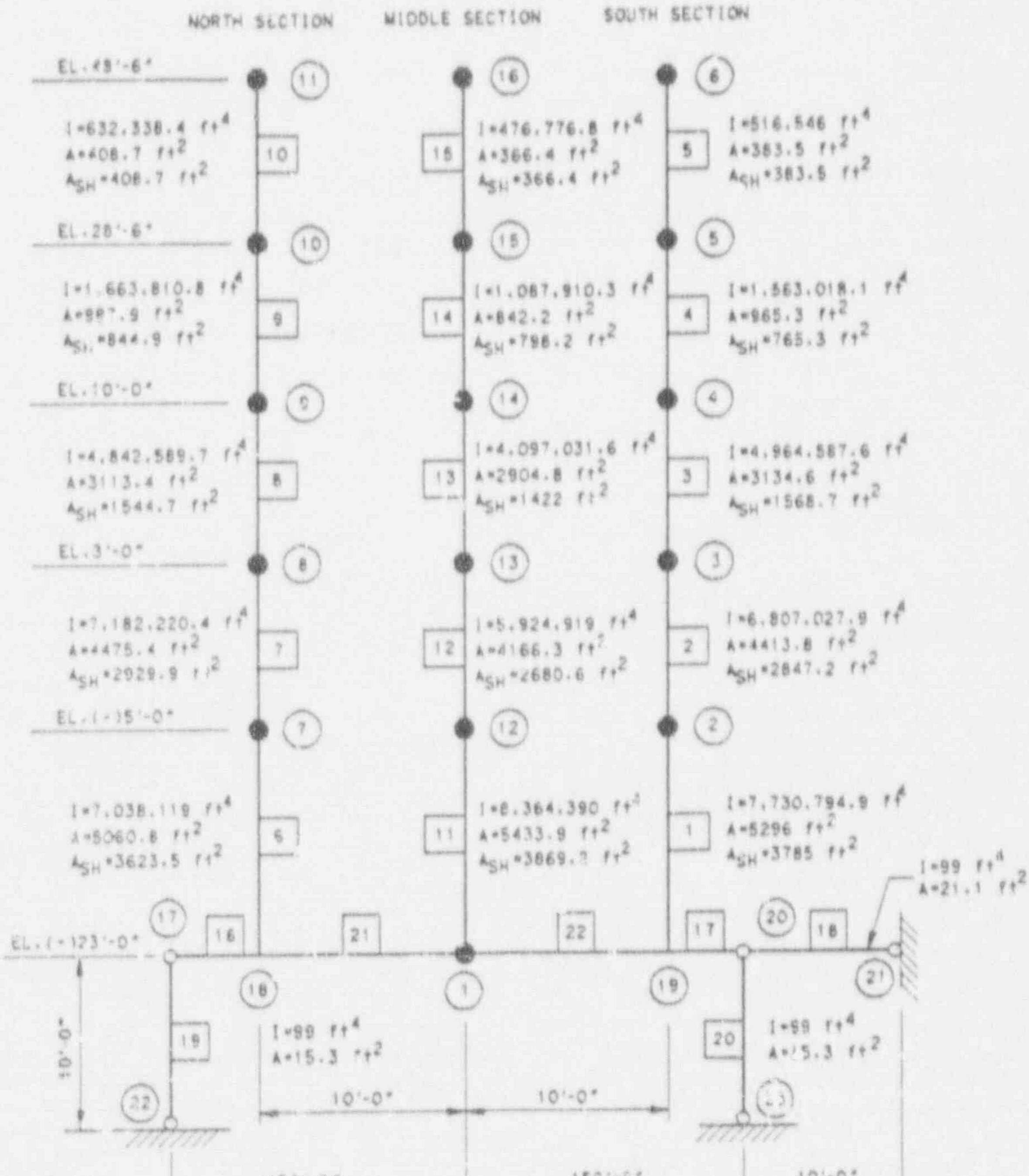
MEMBERS 5 & 6 ARE "RIGID"

FIGURE 3.4
AUXILIARY BUILDING MODEL
PENETRATION AREA
EAST-WEST MOTION



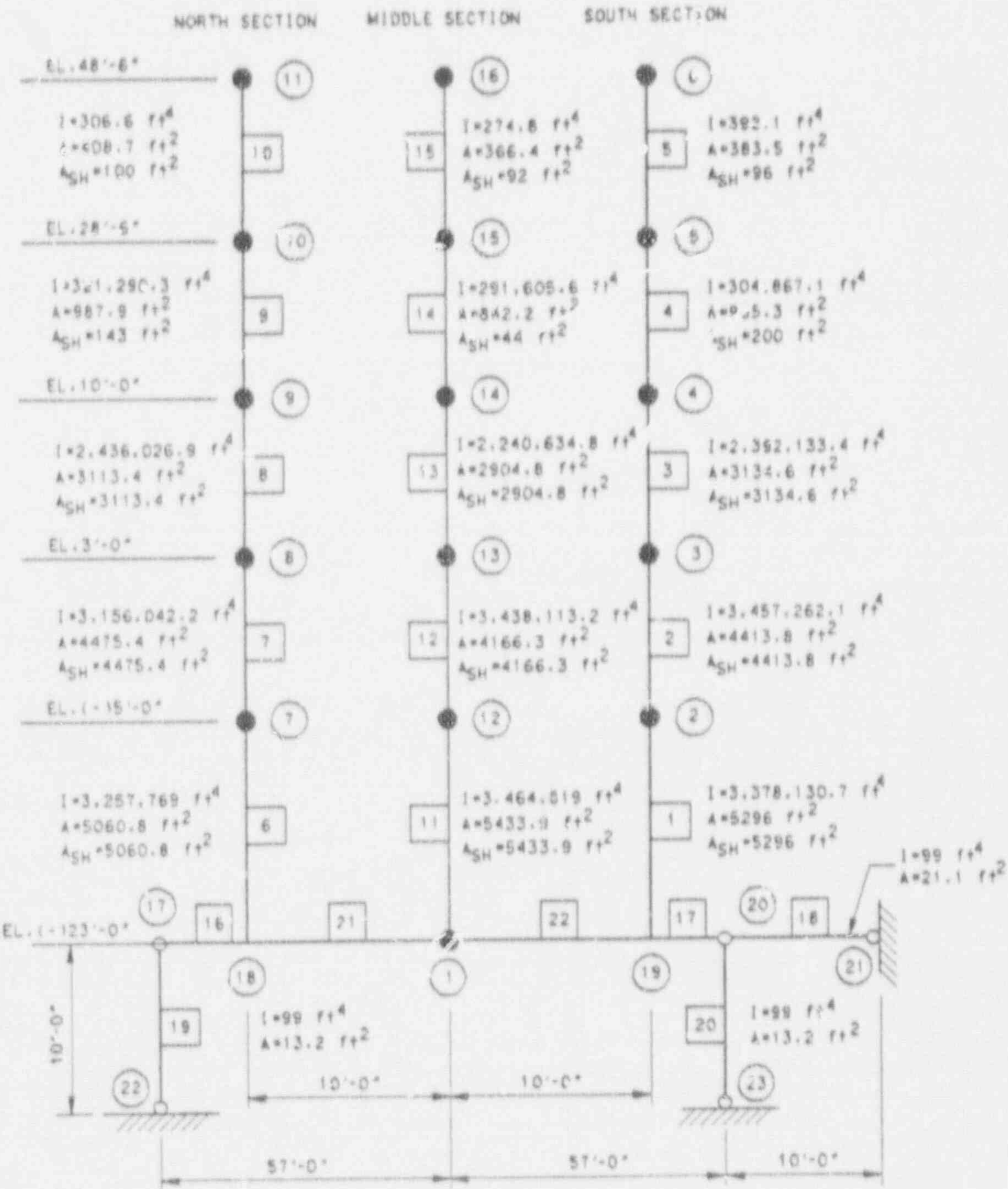
MEMBERS 5 & 6 ARE "RIGID"

FIGURE 4.1
INTAKE STRUCTURE MODEL
NORTH-SOUTH MOTION



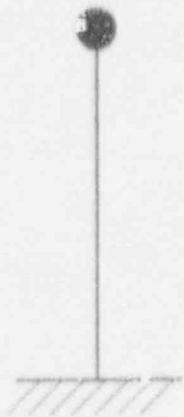
MEMBERS 16,17,21 & 22 ARE "RIGID"

FIGURE 4.2
INTAKE STRUCTURE MODEL
EAST-WEST MOTION



MEMBERS 16,17,21 & 22 ARE "RIGID"

FIGURE 5.1
CONTAINMENT STRUCTURE
VERTICAL MODEL



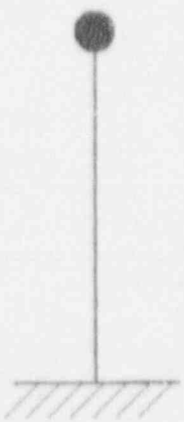
K_z = 988.8 x 10^3 kip/ft

W = 106400 kips

β = 0.08

f_n = 2.62 Hz

FIGURE 5.2
AUXILIARY BUILDING
VERTICAL MODEL



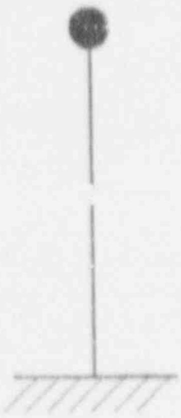
K_z = 1.275 x 10^6 kip/ft

W = 142,000 kips

β = 0.10

f_n = 2.58 Hz

FIGURE 5.3
INTAKE STRUCTURE
VERTICAL MODEL



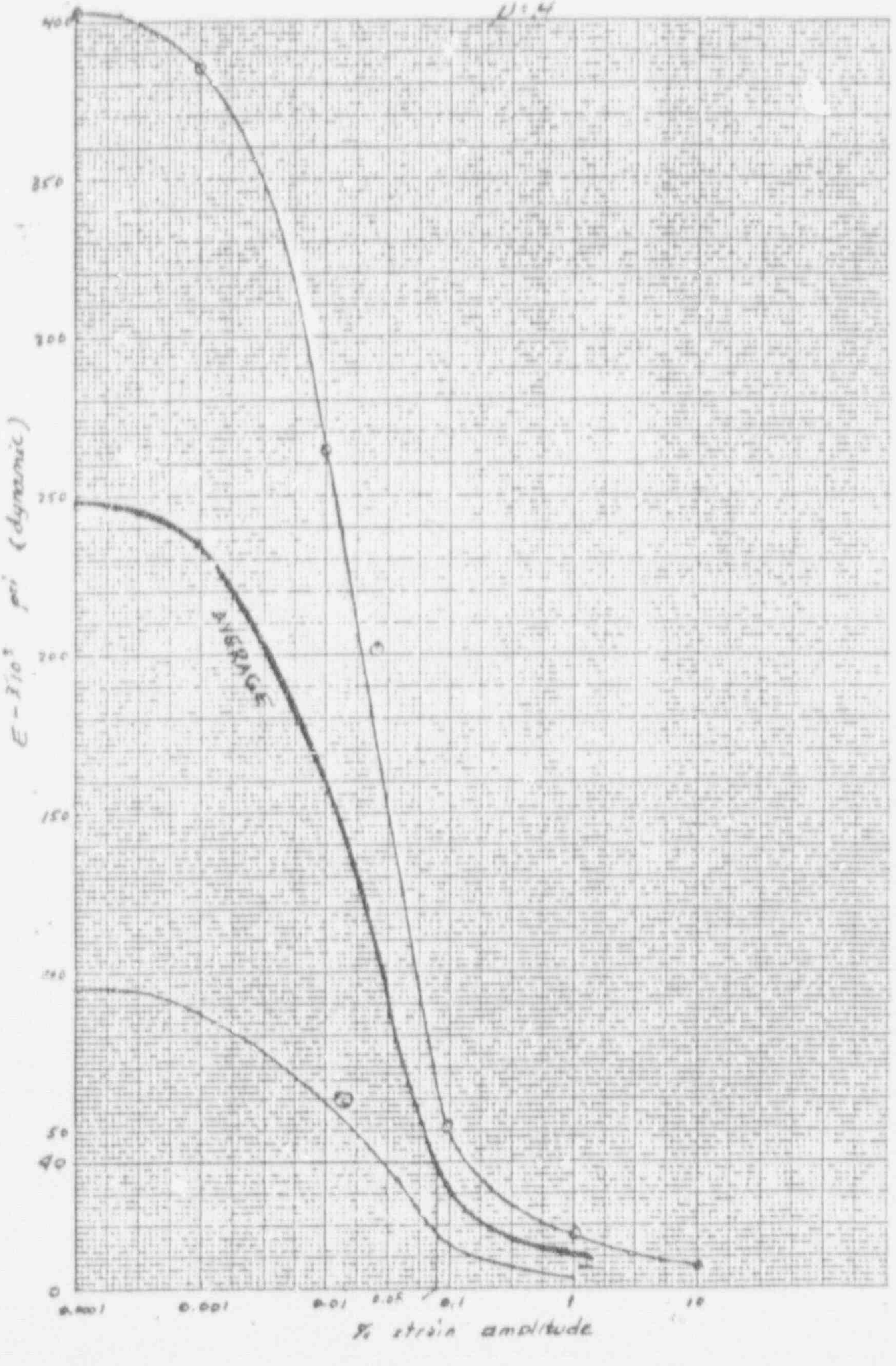
K_z = 1.56 x 10^6 kip/ft

W = 136,625 kips

ß = 0.05

f_n = 3.05 Hz

FIGURE 6.1
DYNAMIC MODULUS OF ELASTICITY FOR SOIL



APPENDICES

APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA

SELECTED

IN-STRUCTURE RESPONSE SPECTRA

FOR

BALIMORE GAS AND ELECTRIC COMPANY

CALVERT CLIFFS NUCLEAR POWER PLANT

APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA

TABLE OF CONTENTS

APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA

Summary

The following pages contain selected In-Structure Response Spectra (ISRS) for the Containment Building, Auxiliary Building and Intake Structure. These ISRS are presented for elevations which have significant equipment installations.

Three directions of seismic excitation, namely North-South, East-West and Vertical, are cited for both Operating Basis Earthquake and Design Basis Earthquake. ISRS are given for three values of critical damping for components.

APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA

TABLE A-A

CONTAINMENT BUILDING

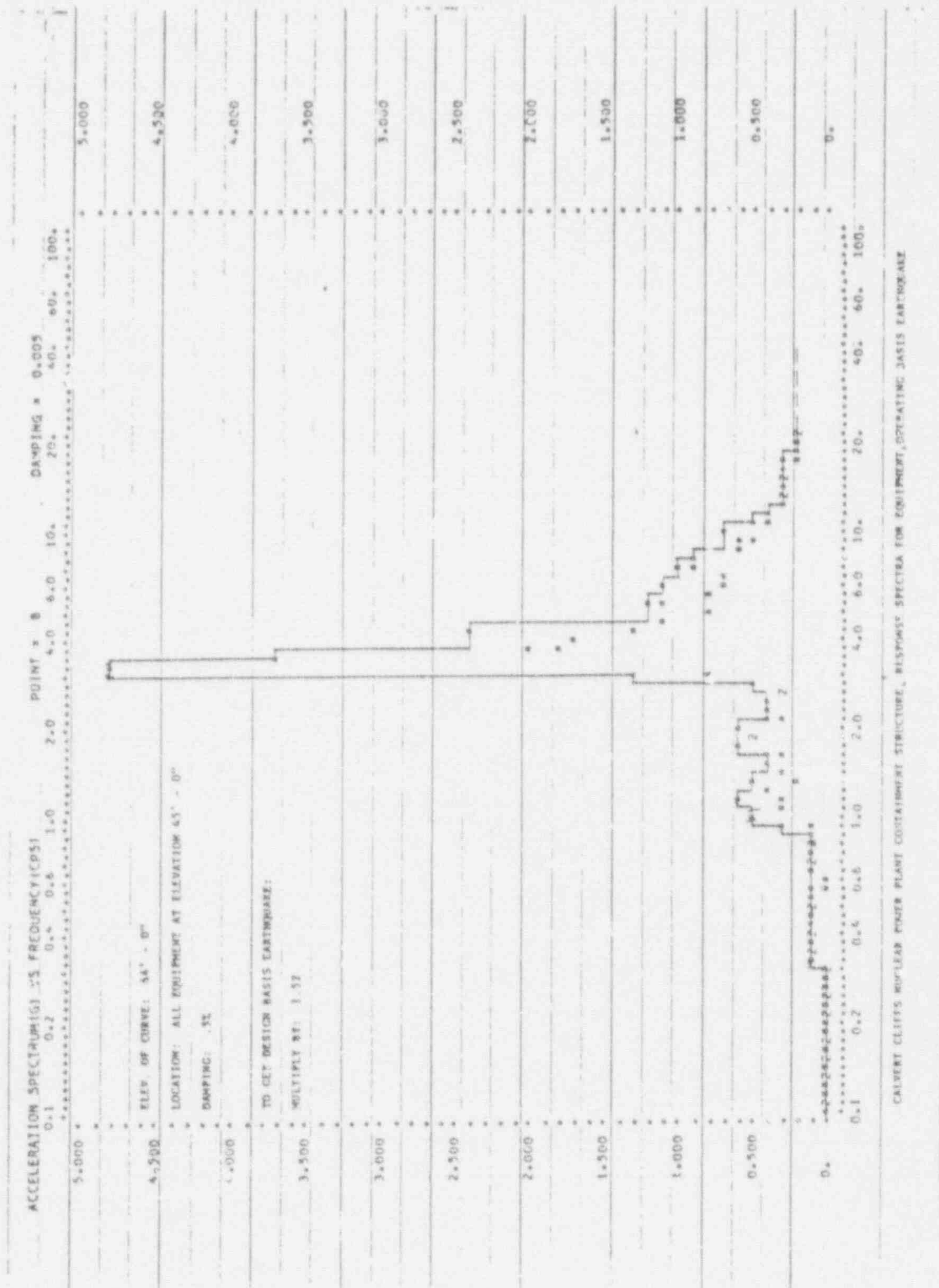
FLOOR ACCELERATIONS AND PEAK FLOOR RESPONSE ACCELERATIONS
for
Operating Basis Earthquake (OBE)
and
Design Basis Earthquake (DBE)

ELEVATION	PEAK FLOOR RESPONSE ACCELERATIONS[1] for component damping of 1/2 %	1 %	2.5 %	FLOOR ACCELERATION (ZPA)
(ft.)	(g's)	(g's)	(g's)	(g's)
East-West Motion				
44' (OBE)	4.80	3.20	1.76	0.240
(DBE)	7.30	4.90	2.70	0.360
Vertical Motion				
All (OBE)	0.92	0.80	0.59	0.140
(DBE)	1.73	1.50	1.11	0.260

[1] Acclelerations at Containment Building Natural Frequencies of 3.21 Hz for East-West Motion and 10.00 Hz for Vertical Motion.

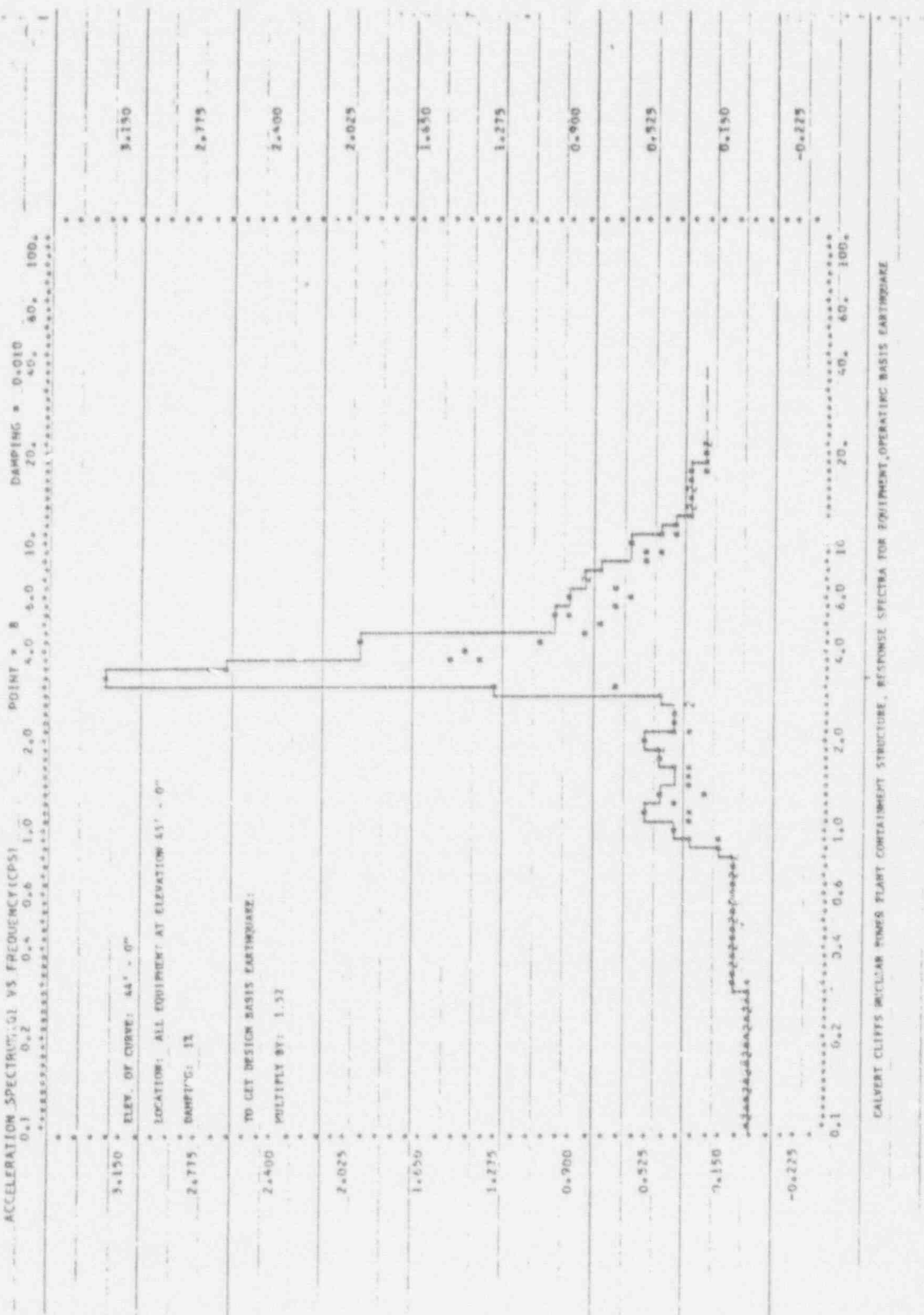
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



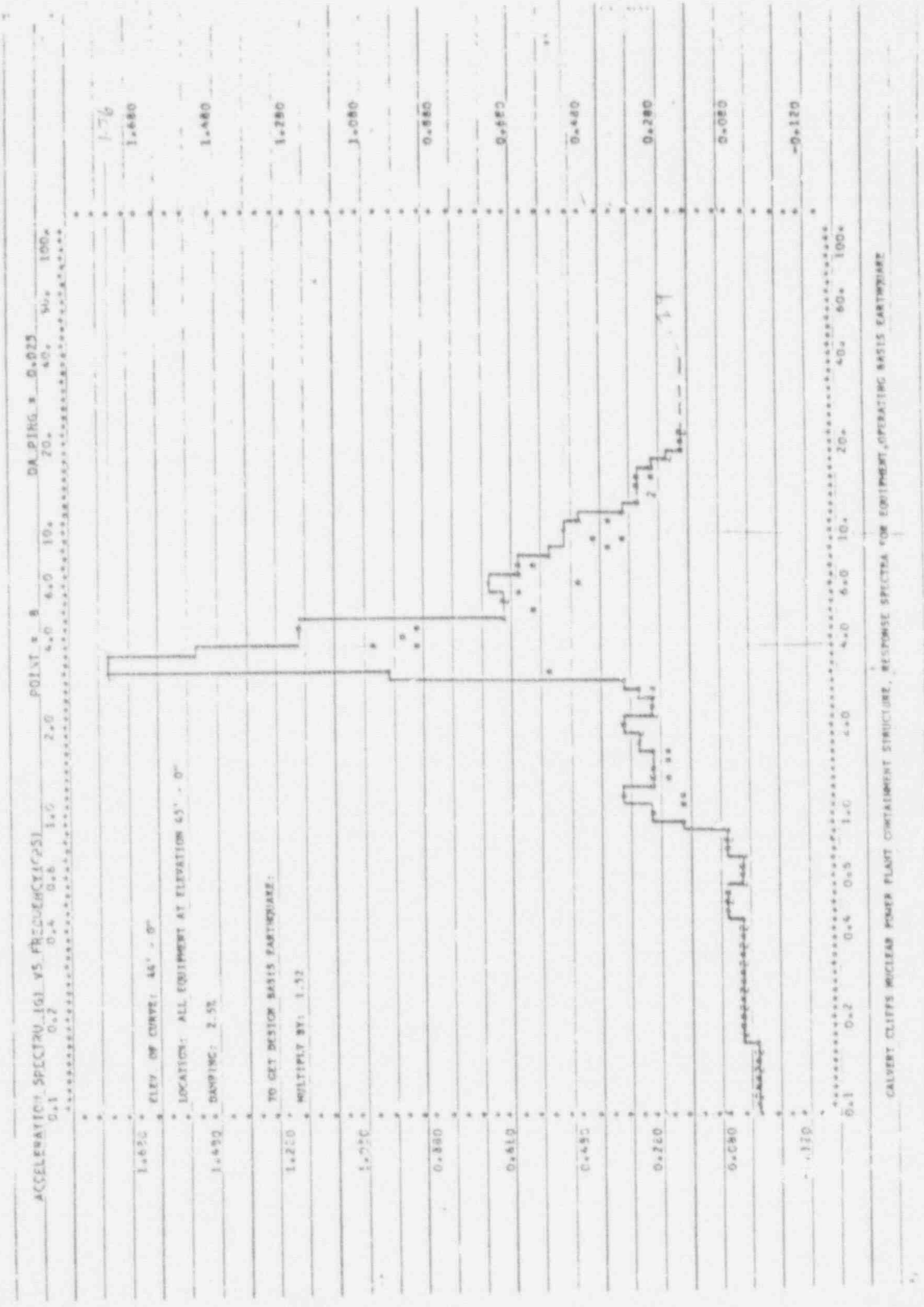
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



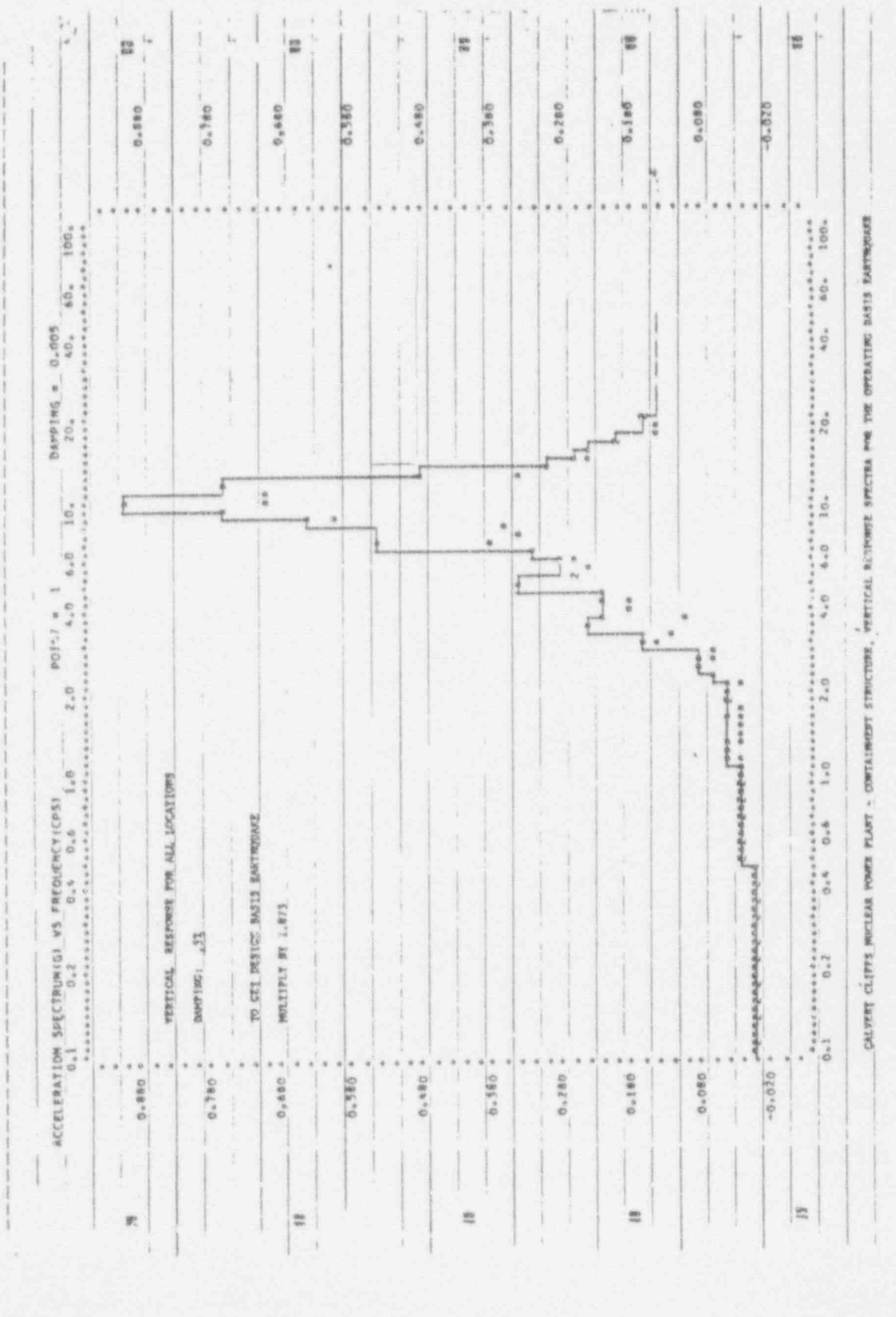
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



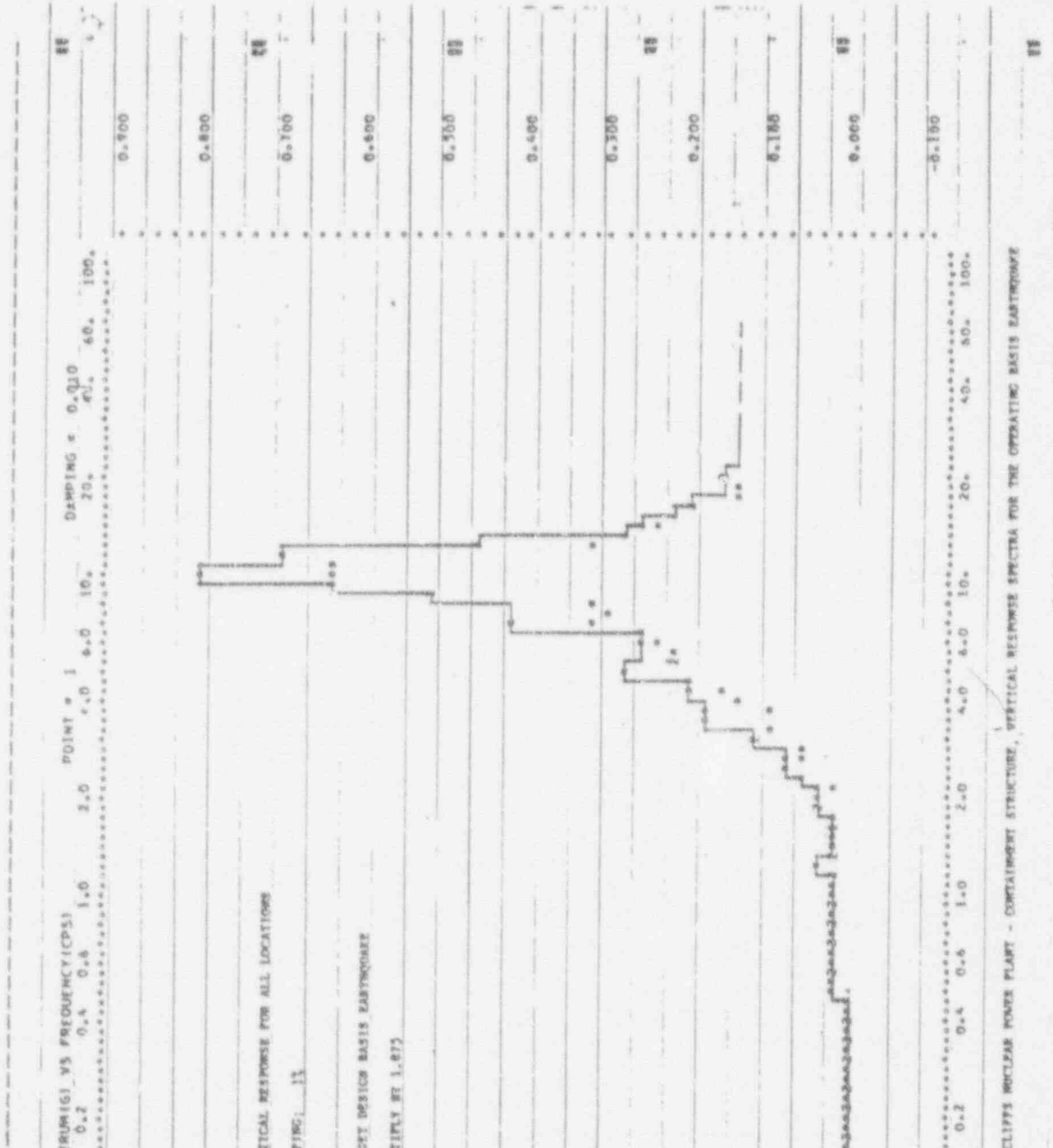
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



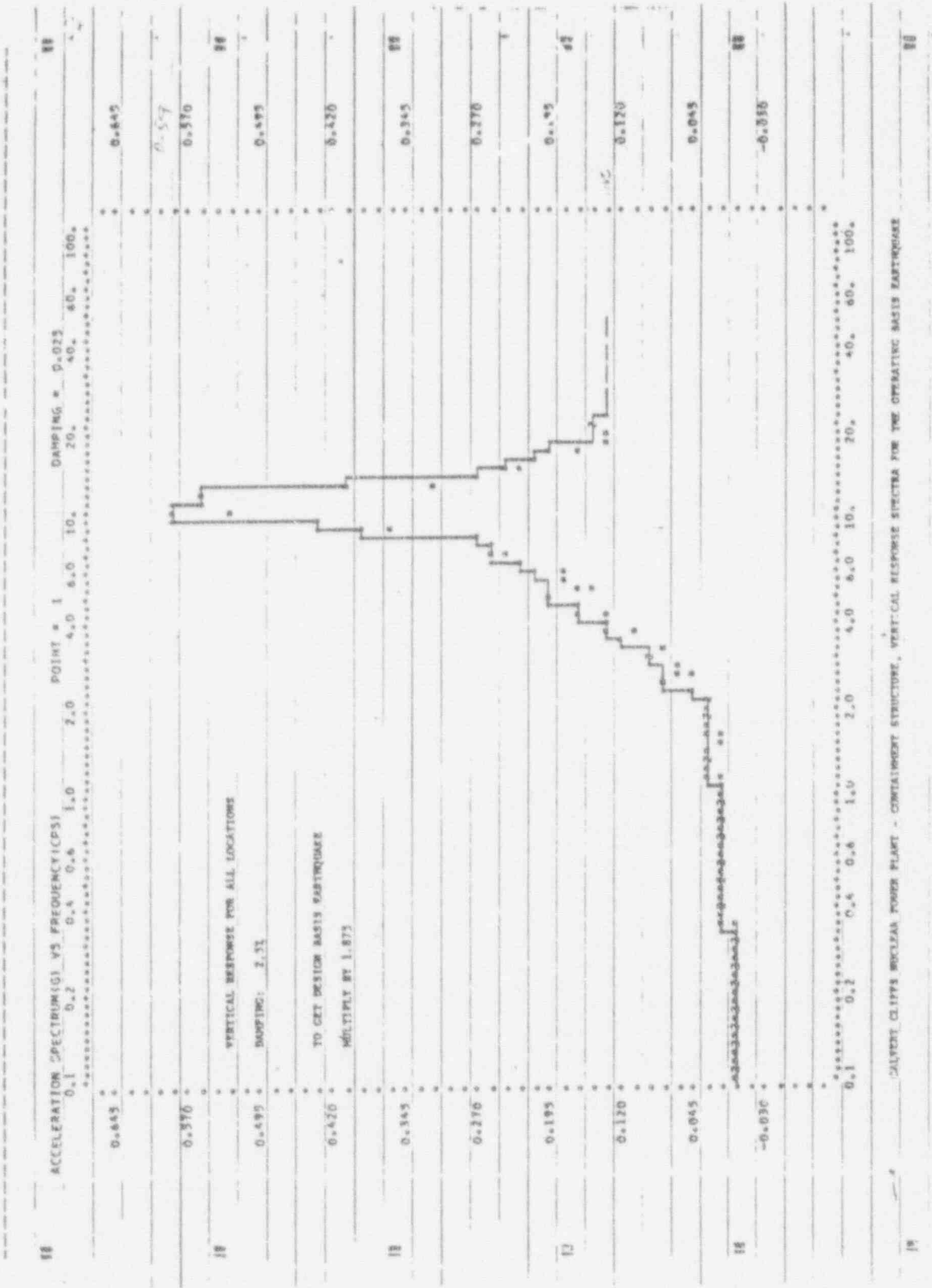
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA

TABLE A-B
AUXILIARY BUILDING
FLOOR ACCELERATIONS AND PEAK FLOOR RESPONSE ACCELERATIONS
for
Operating Basis Earthquake

ELEVATION	PEAK FLOOR RESPONSE ACCELERATIONS[2] for component damping of			FLOOR ACCELERATION [ZPA]
	1/2 %	2 %	5 %	
(ft.)	(g's)	(g's)	(g's)	(g's)
North-South Motion				
27'	2.420	1.442	0.809	0.146
45' E. of SFP	2.682	1.593	0.896	0.167
W. of SFP	2.668	1.585	0.892	0.166
69' E. of SFP	3.263	1.936	1.094	0.212
W. of SFP	2.985	1.769	0.998	0.192
East-West Motion				
27'	3.320	1.335	0.980	0.173
45' E. of SFP	3.550	1.418	1.040	0.186
W. of SFP	3.520	1.406	1.030	0.184
69' E. of SFP	3.840	1.516	1.120	0.201
W. of SFP	3.780	1.504	1.110	0.200
Vertical Motion				
All	0.930	0.650	0.560[3]	0.135

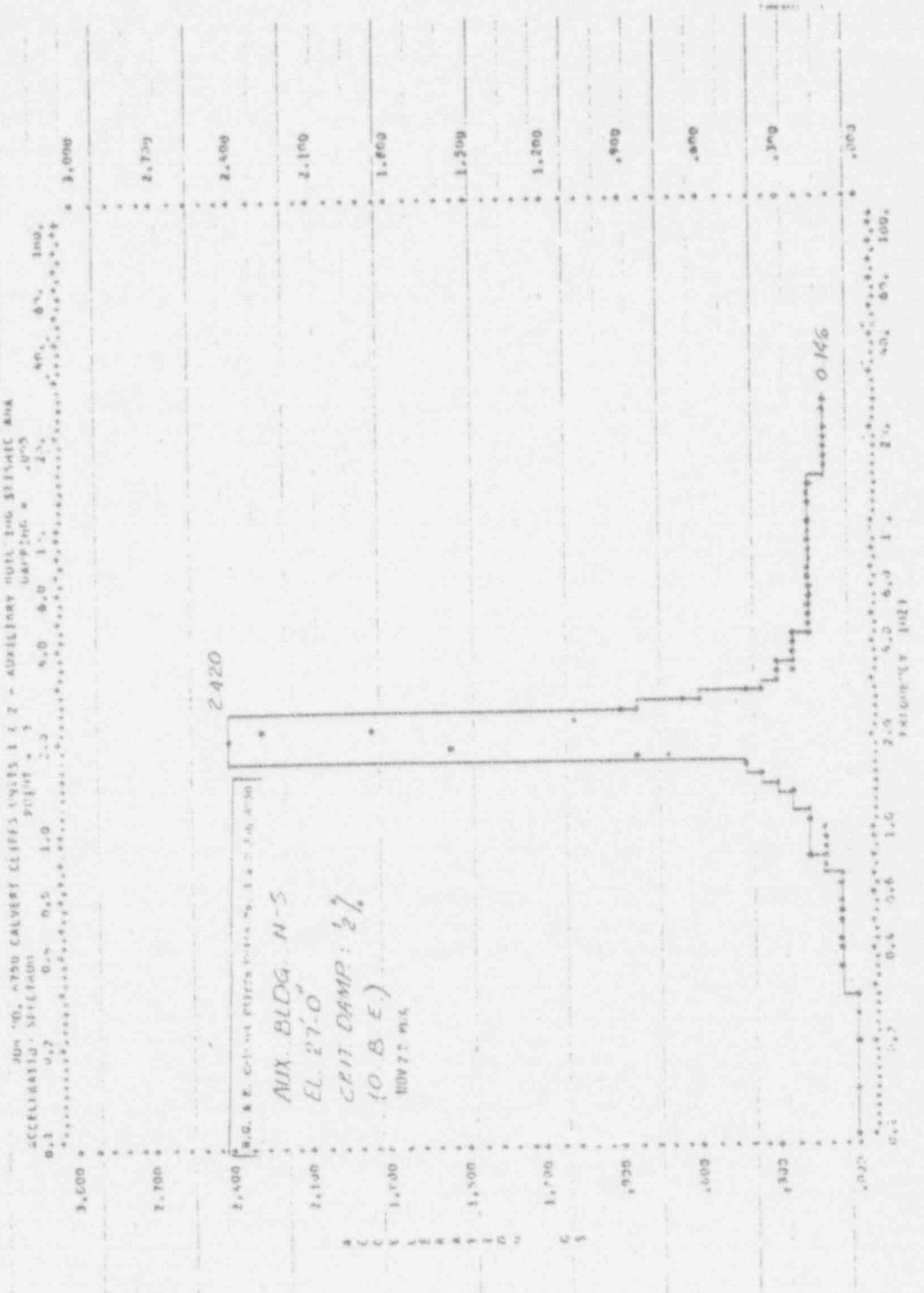
---

[2] Acclelerations at Auxiliary Building Natural Frequencies of 1.57 Hz for North-South Motion, 2.11 Hz for East-West Motion and 10.00 Hz for Vertical Motion.

[3] Peak Floor Response Acceleration at 3 percent critical damping.

APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



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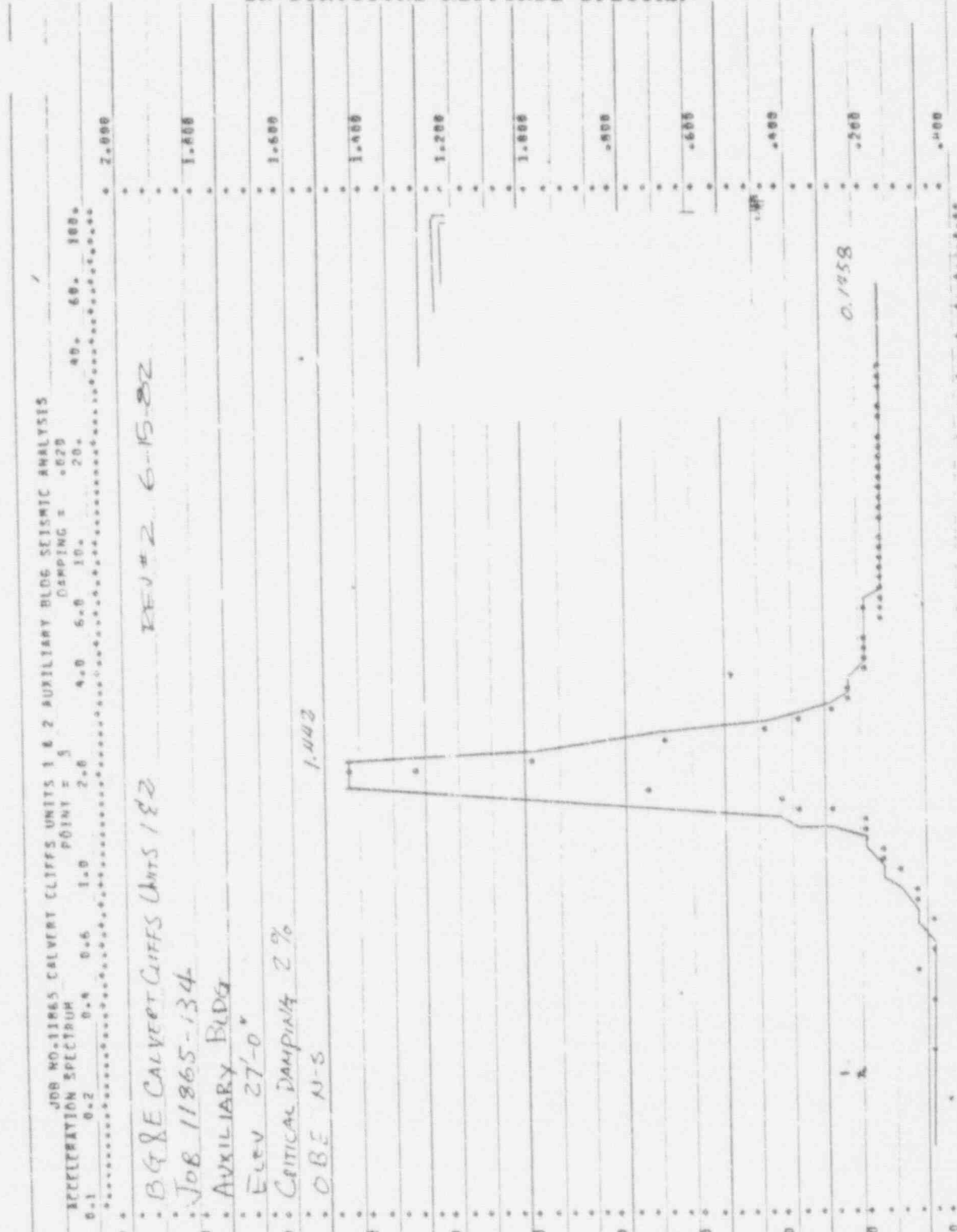
IN-STRUCTURE RESPONSE SPECTRA



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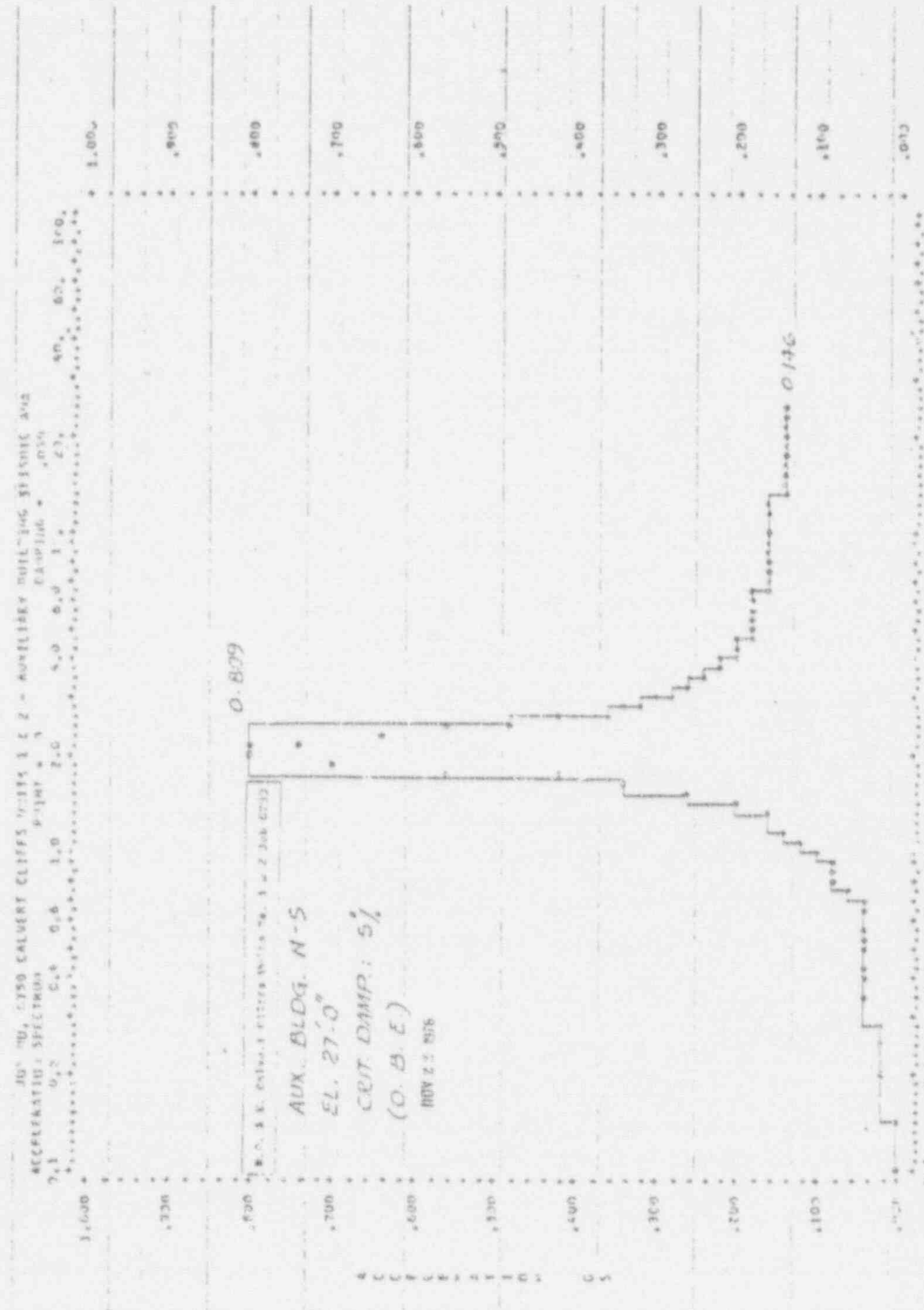
IN-STRUCTURE RESPONSE SPECTRA



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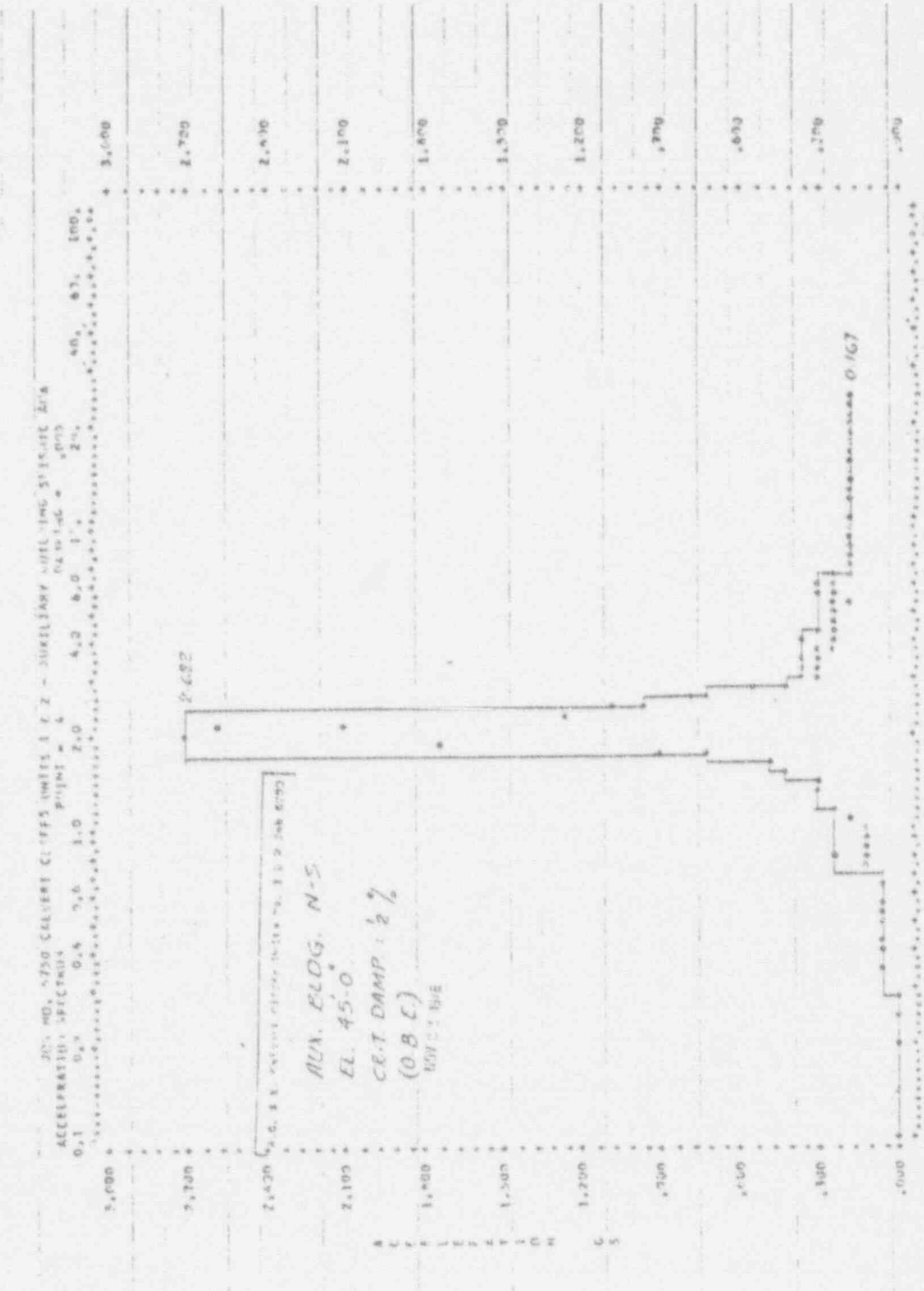
IN-STRUCTURE RESPONSE SPECTRA



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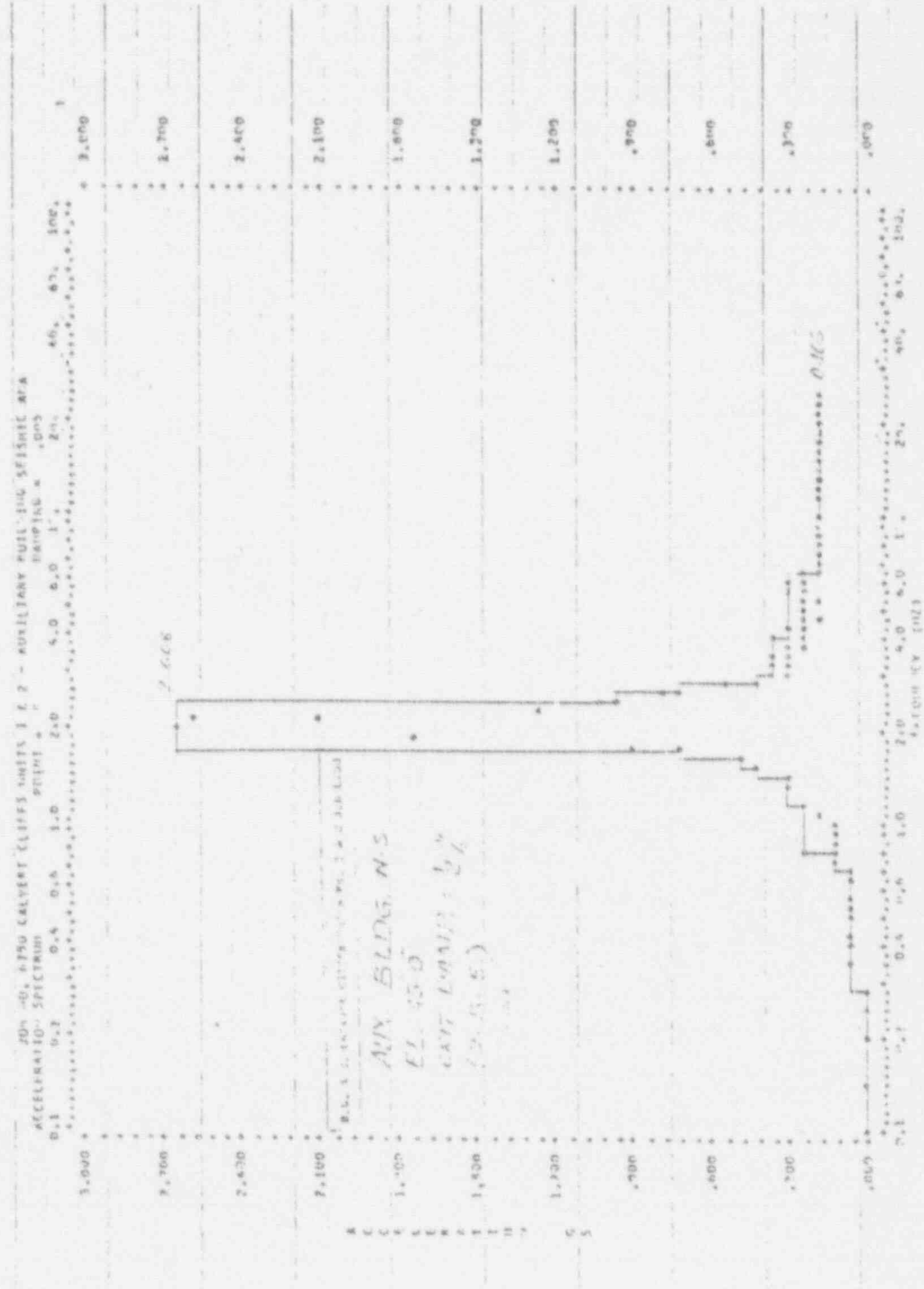
IN-STRUCTURE RESPONSE SPECTRA



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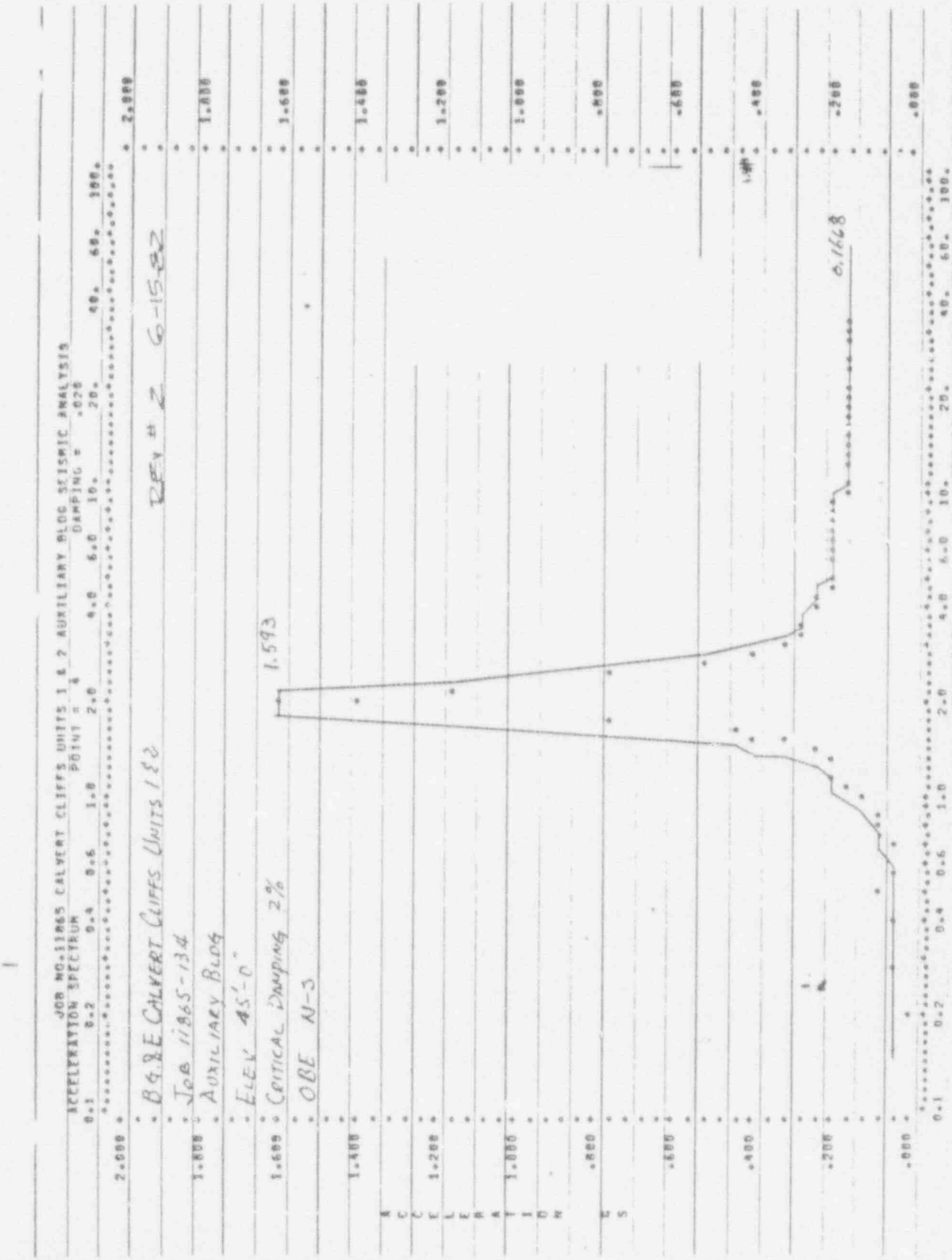

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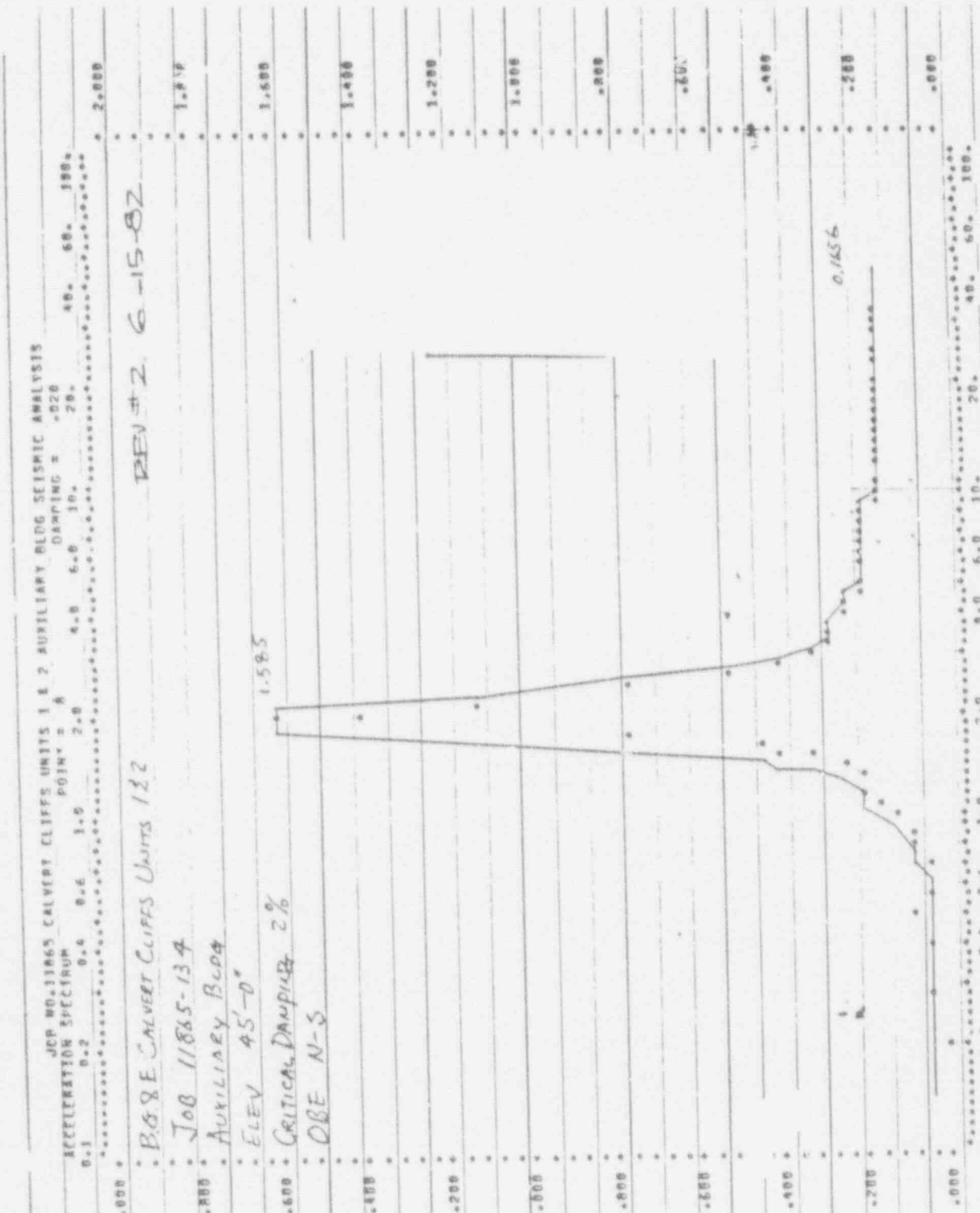

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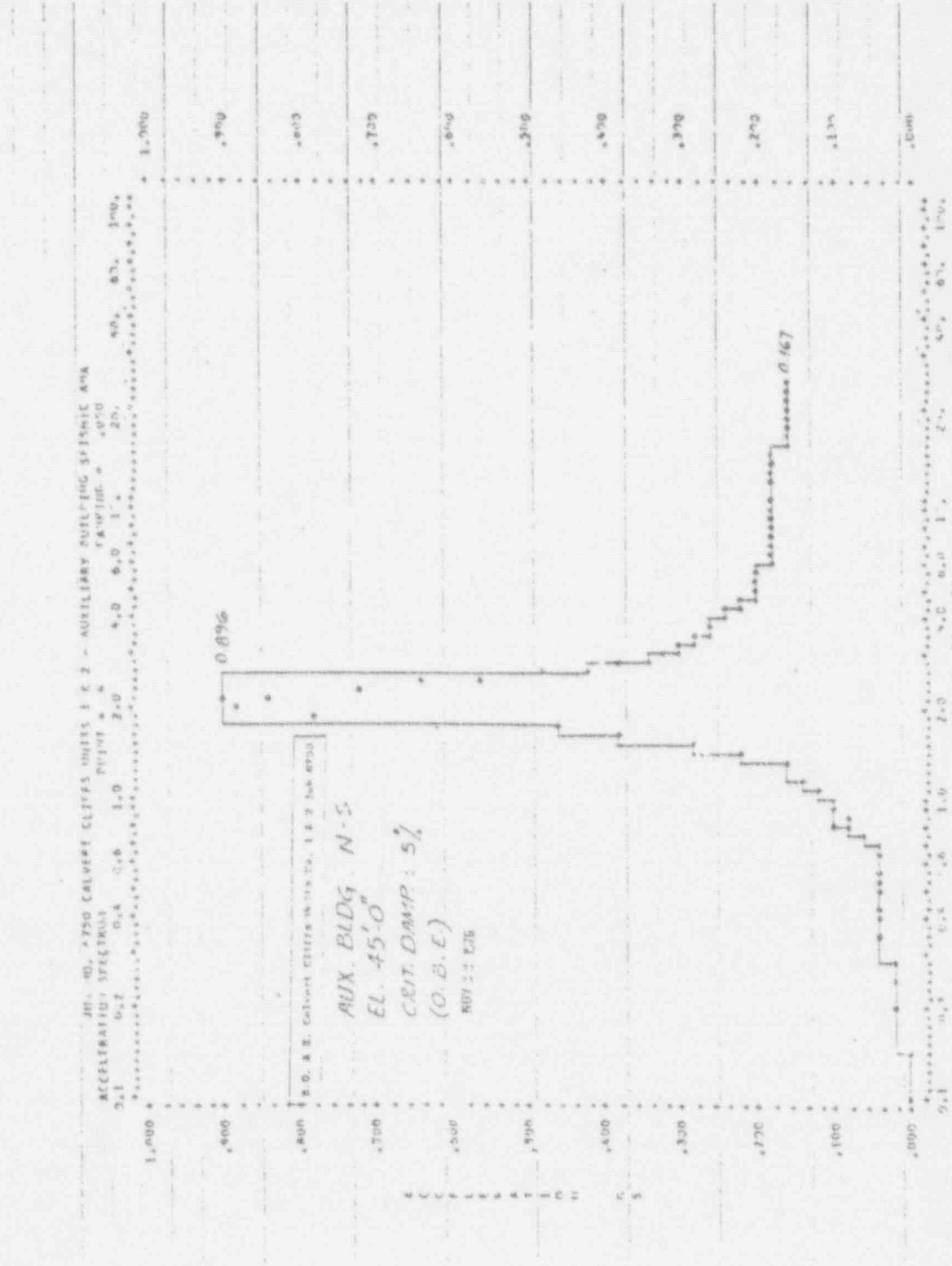
IN-STRUCTURE RESPONSE SPECTRA



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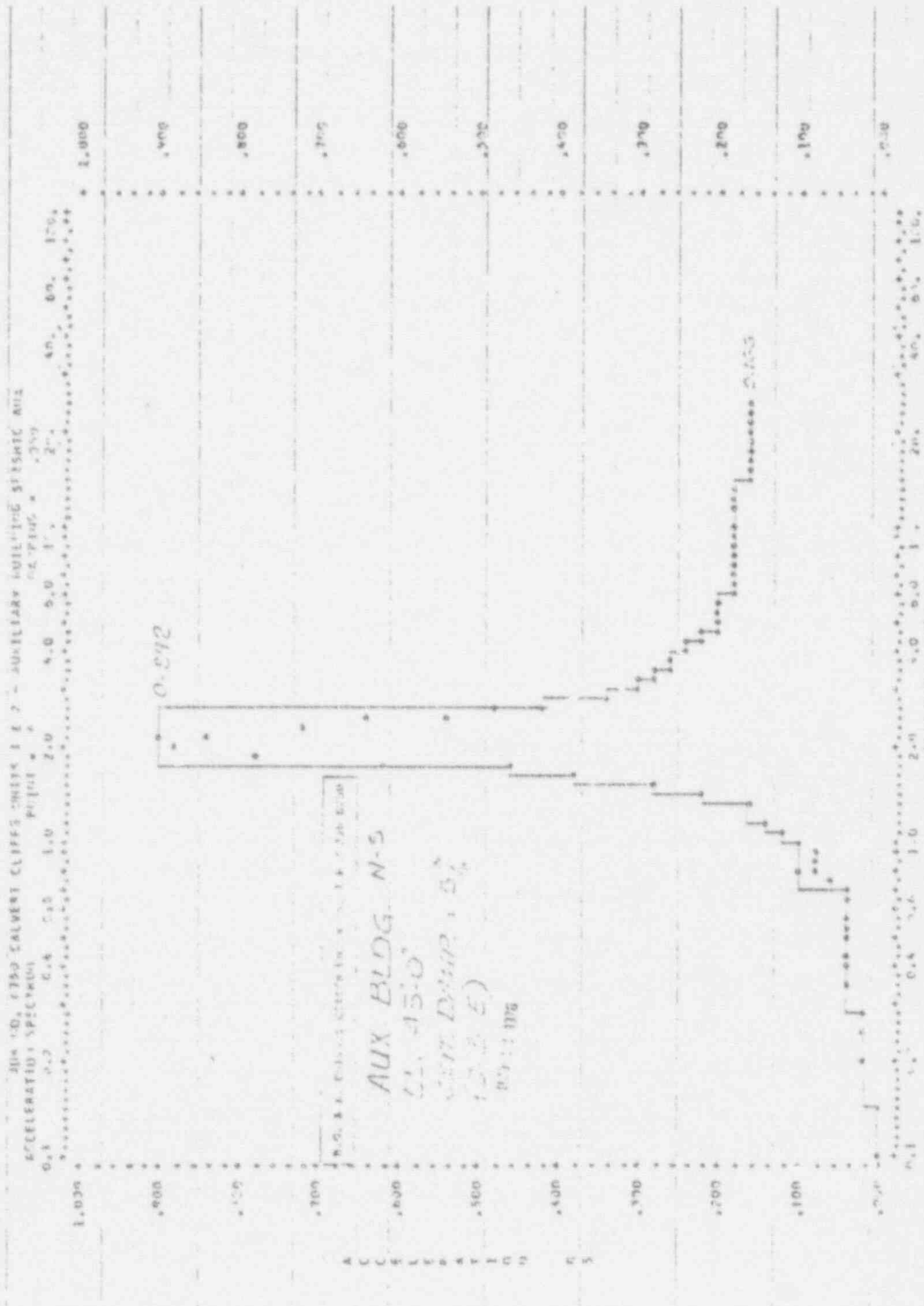
IN-STRUCTURE RESPONSE SPECTRA



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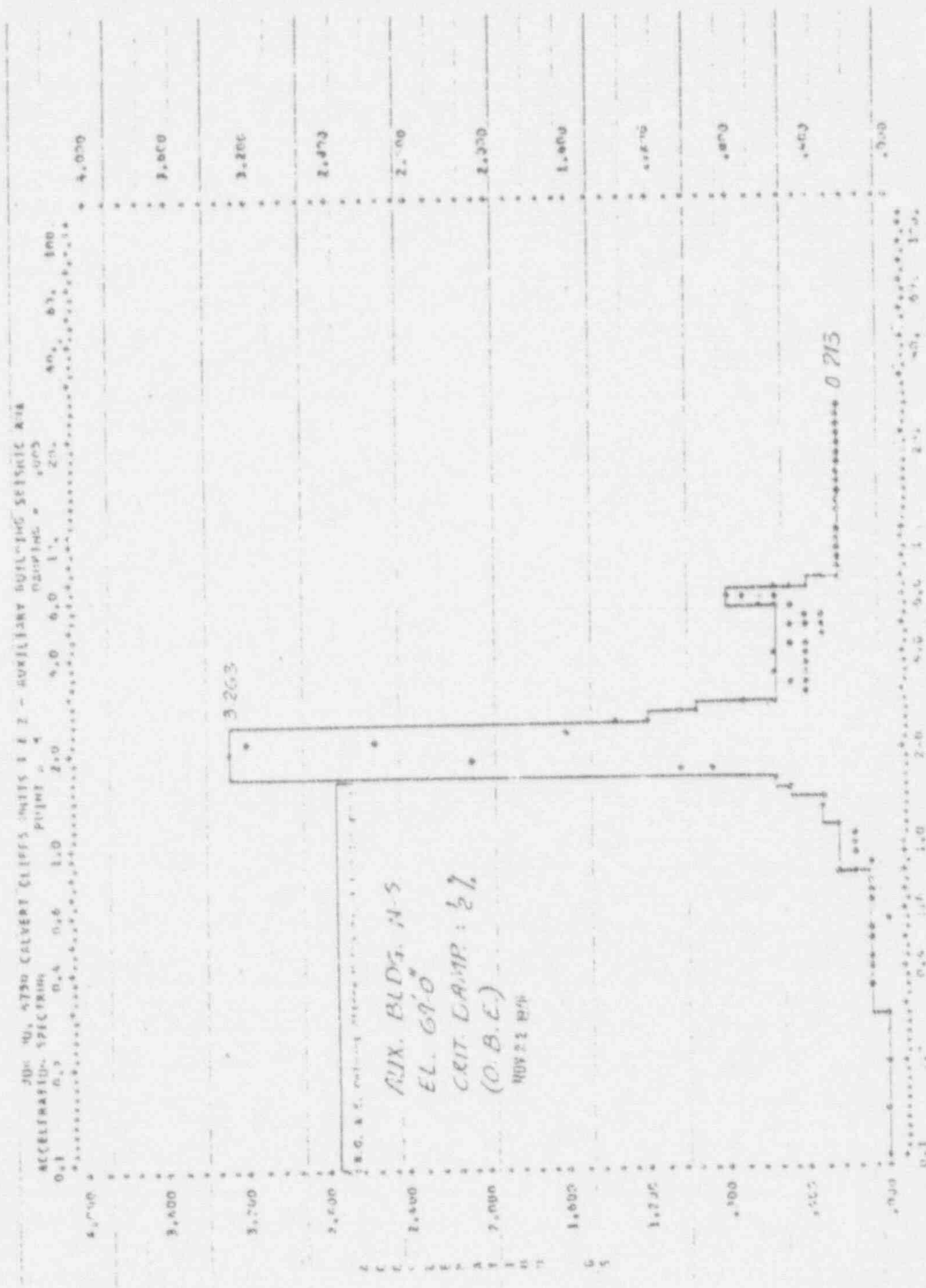
IN-STRUCTURE RESPONSE SPECTRA



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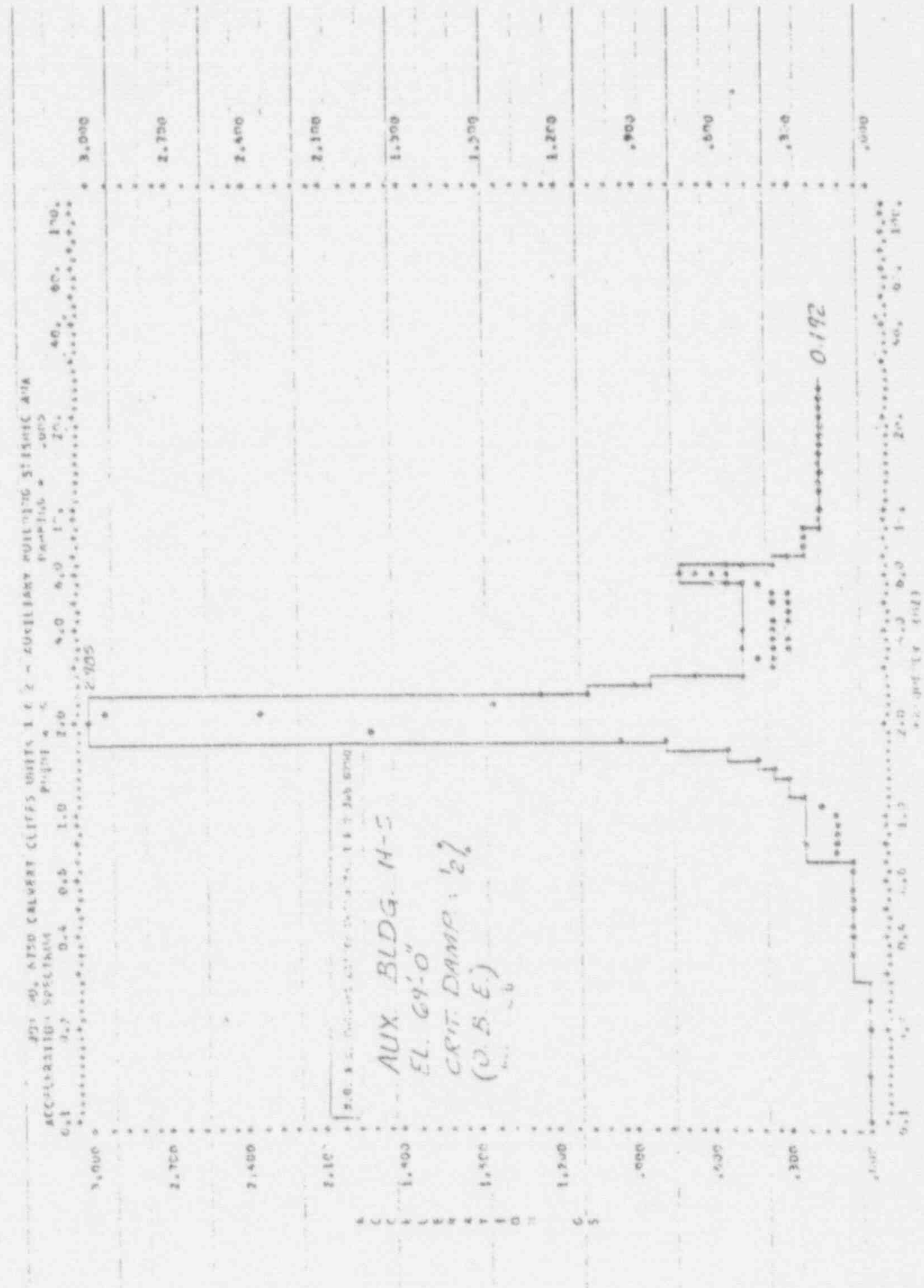
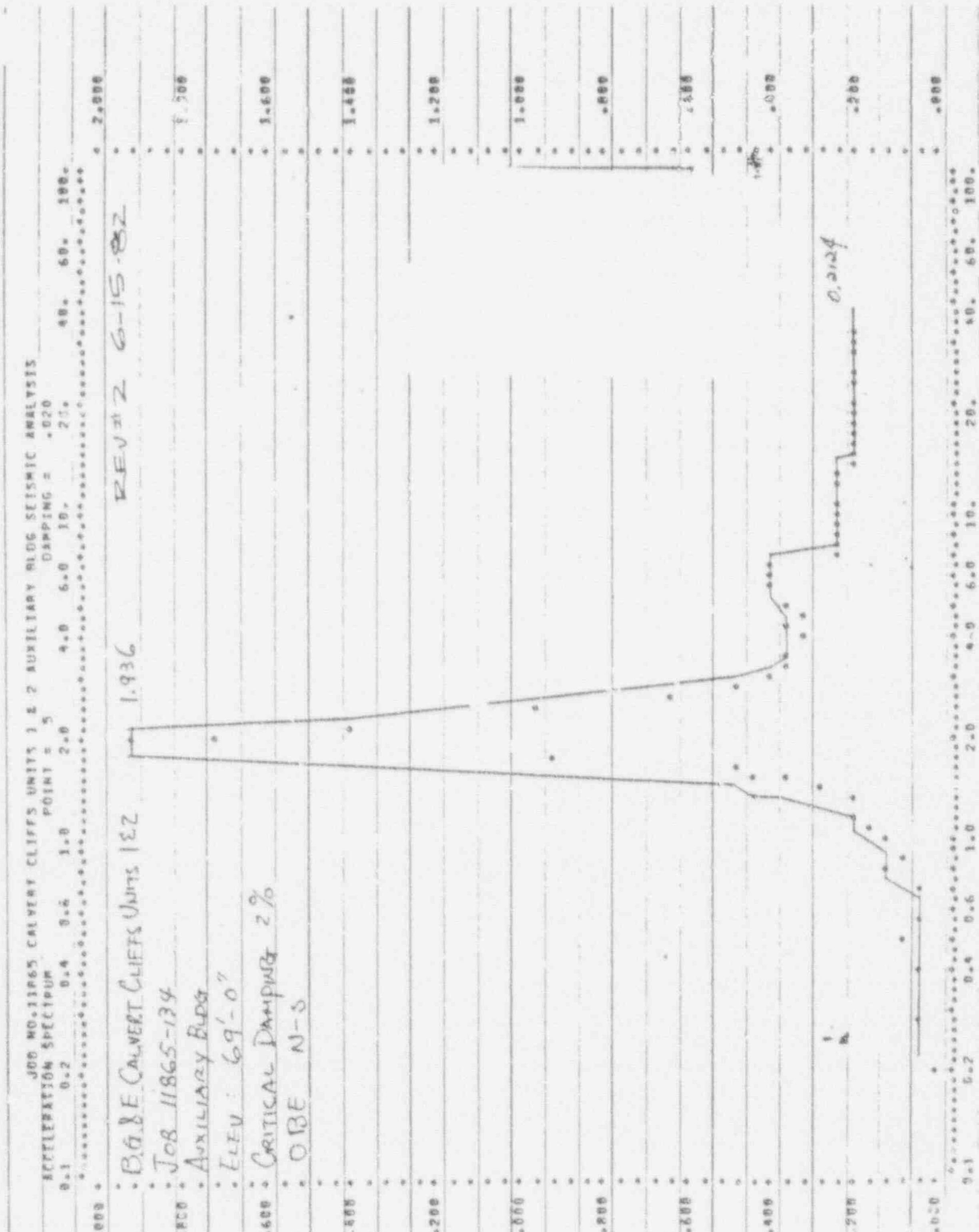
IN-STRUCTURE RESPONSE SPECTRA



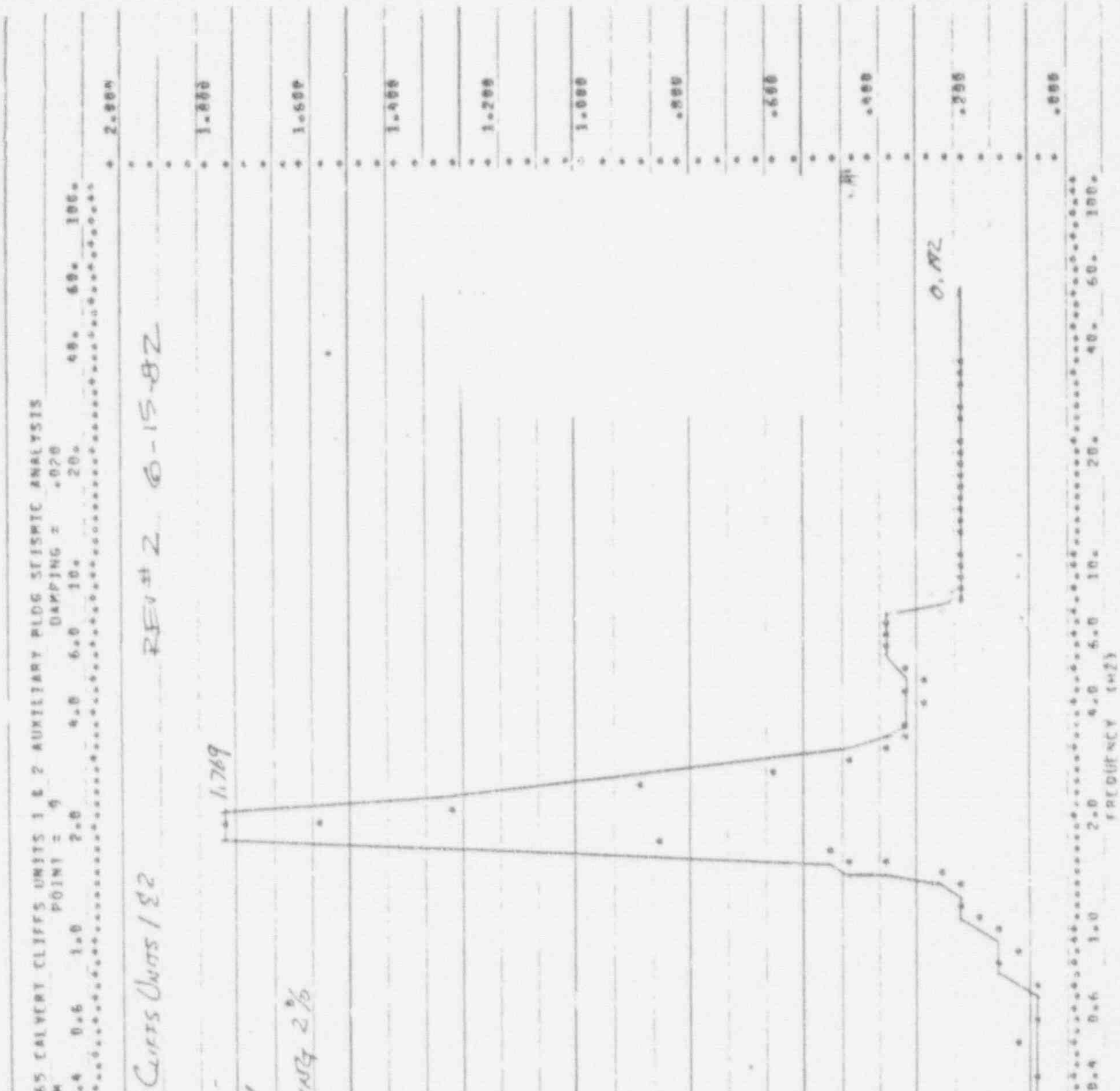
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



APPENDIX A

IN-STRUCTUR RESPONSE SPECTRA



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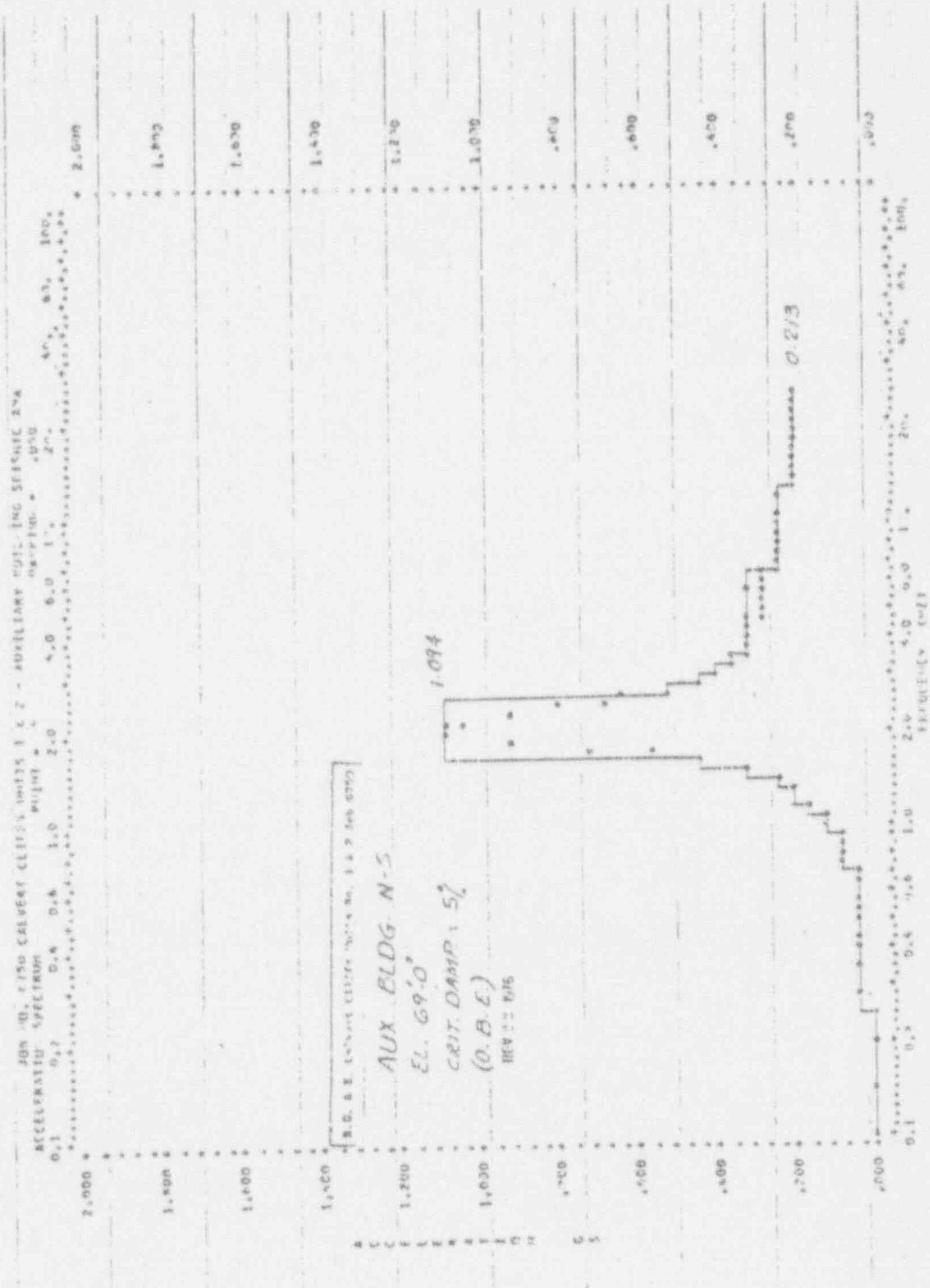
IN-STRUCTURE RESPONSE SPECTRA



APPENDIX A

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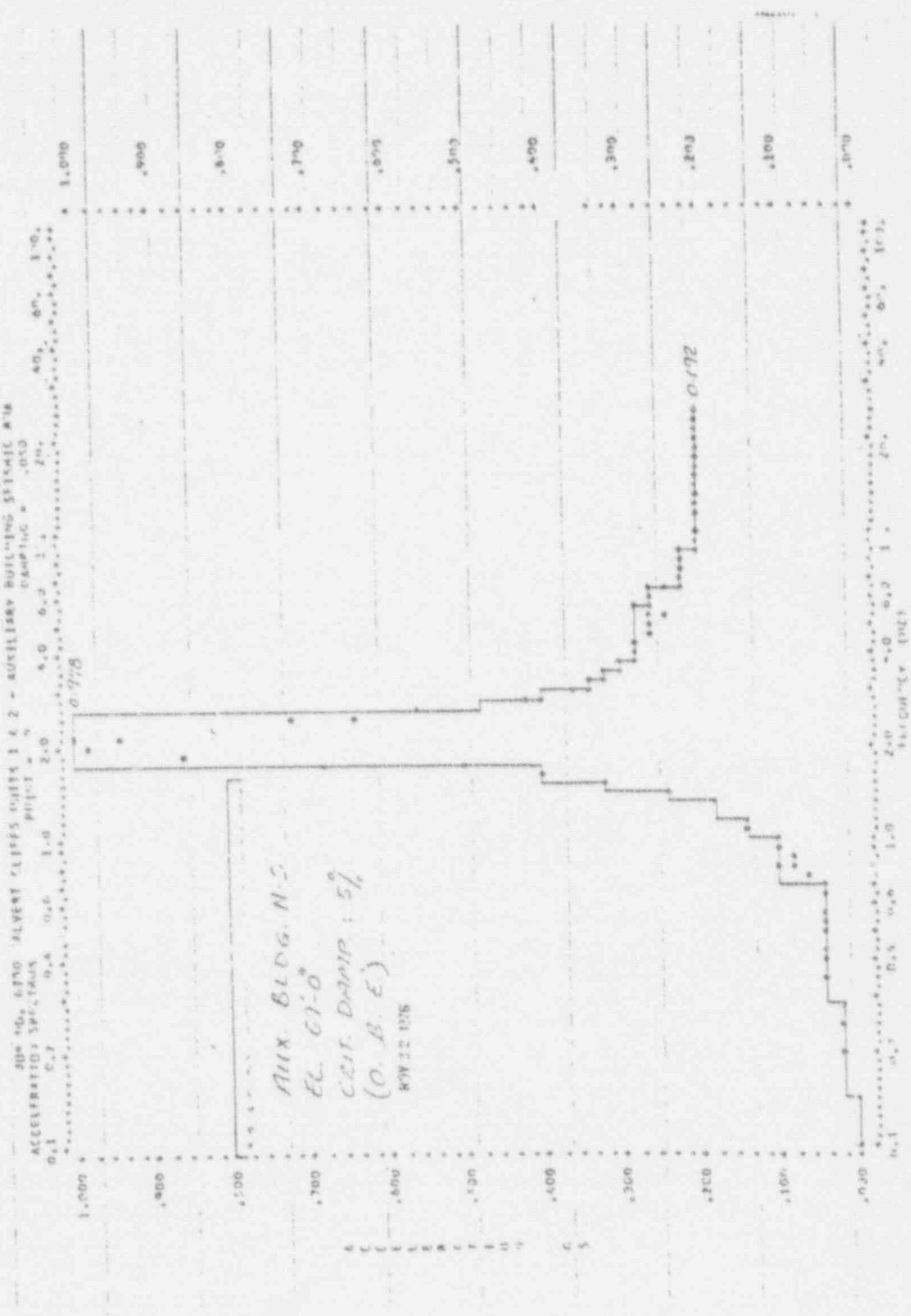

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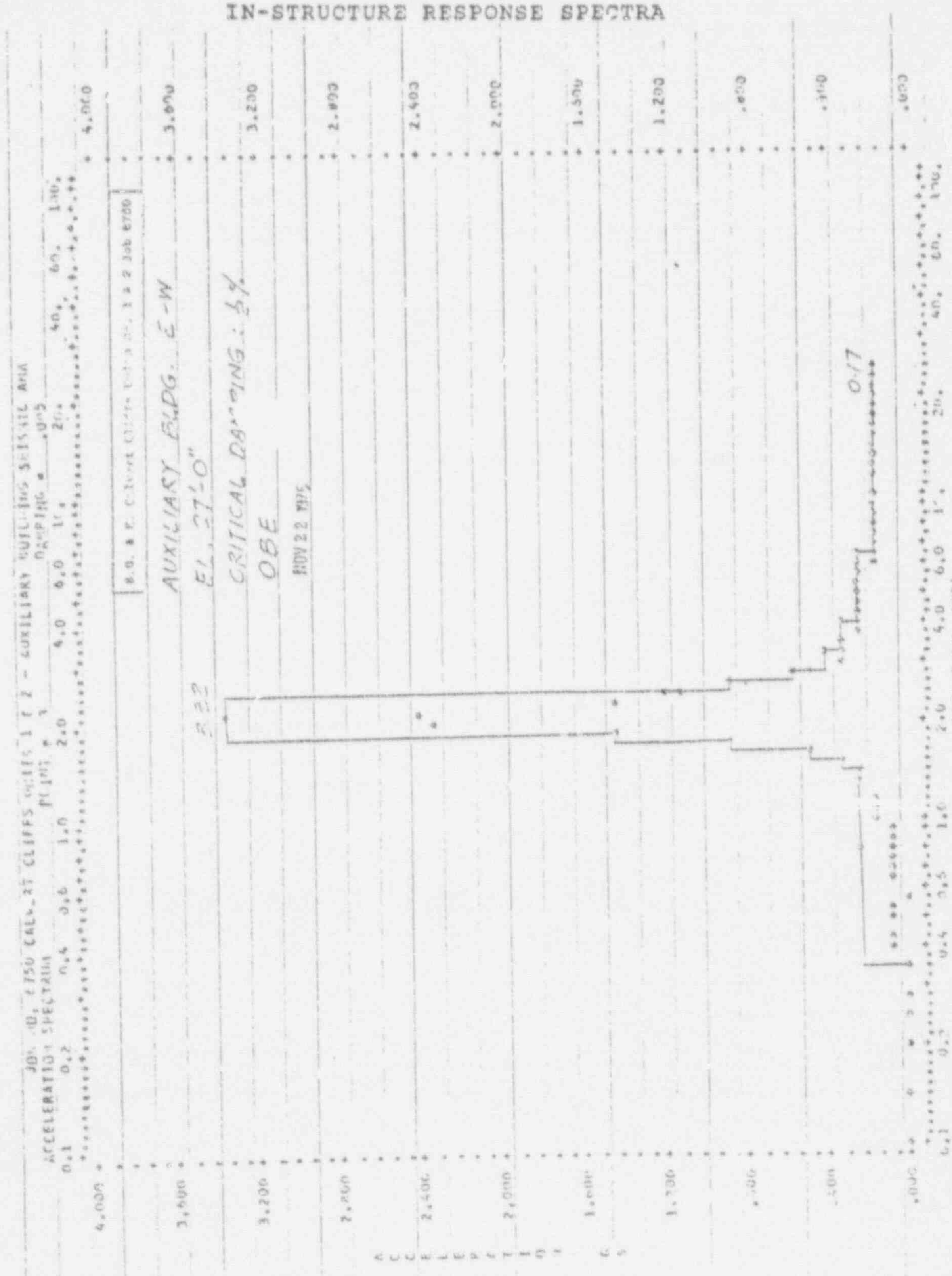

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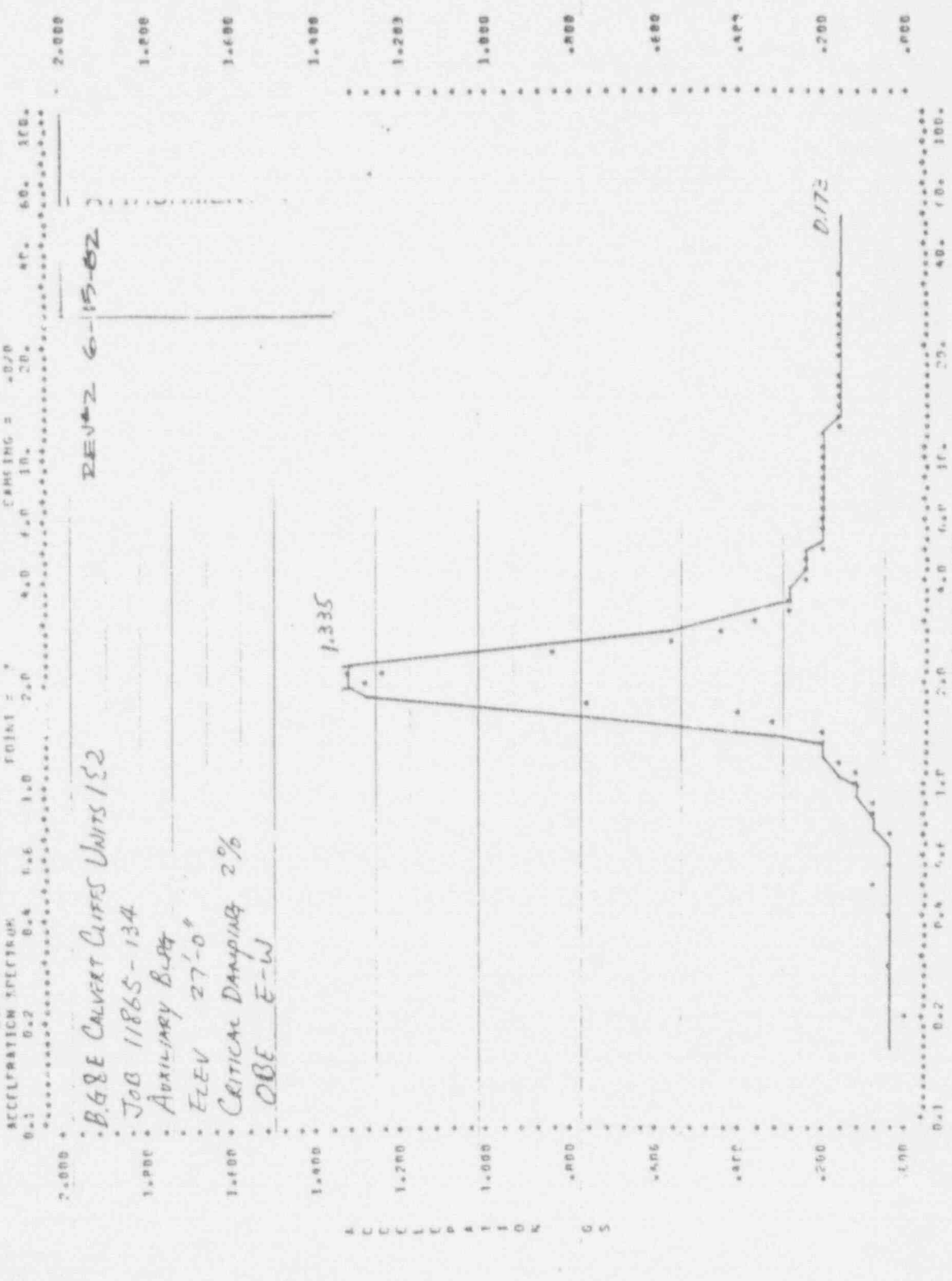
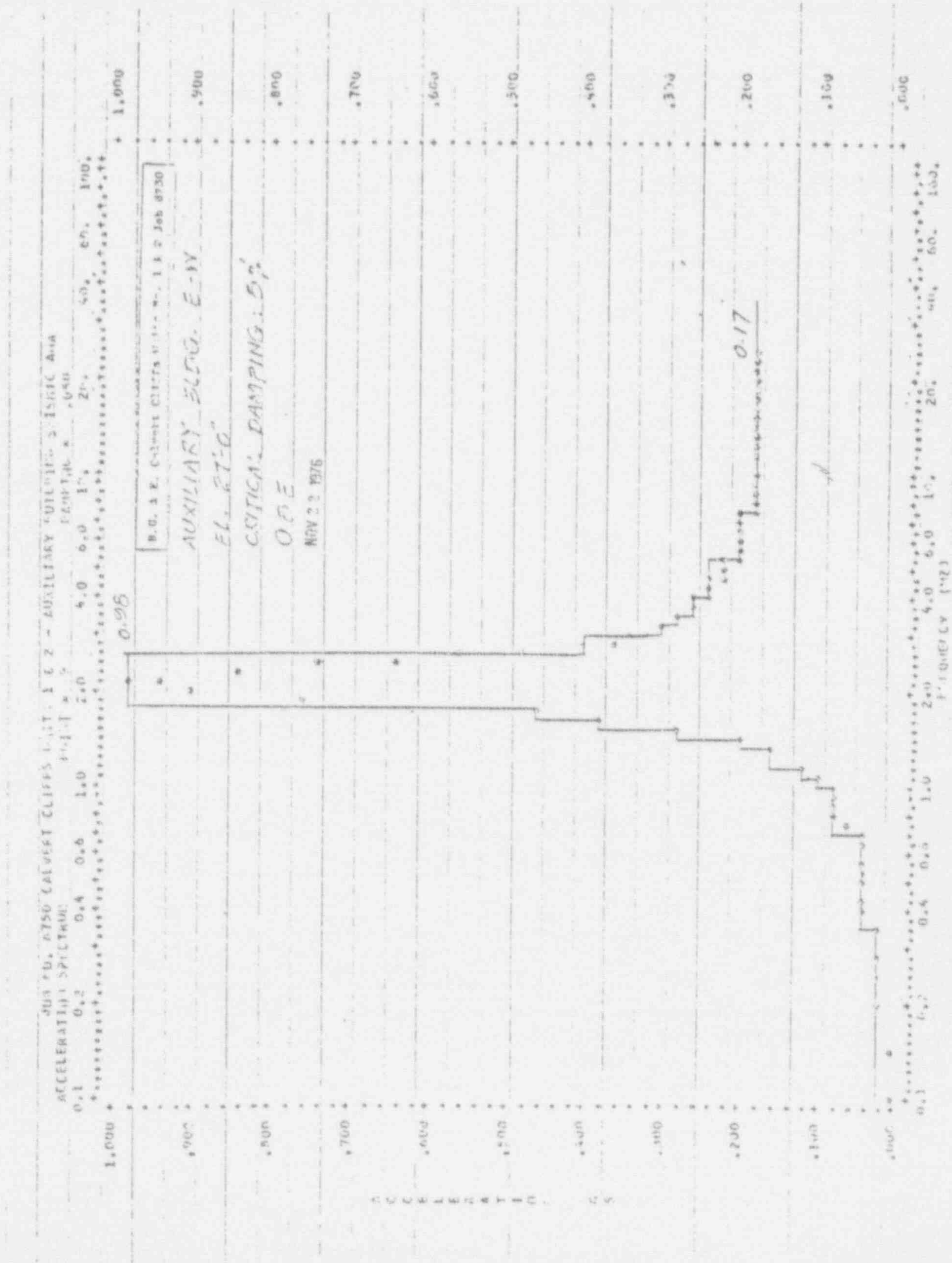


APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



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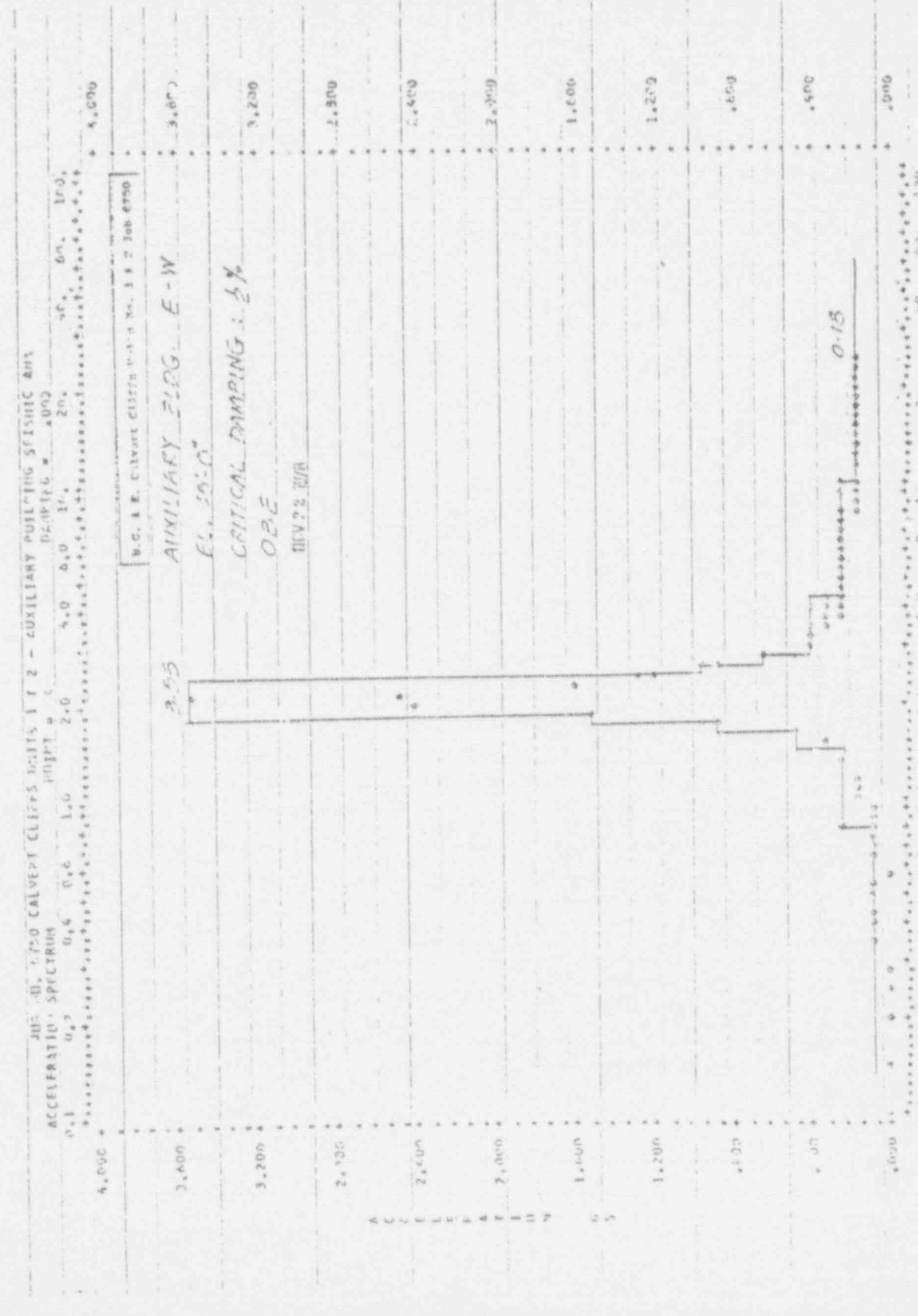
IN-STRUCTURE RESPONSE SPECTRA



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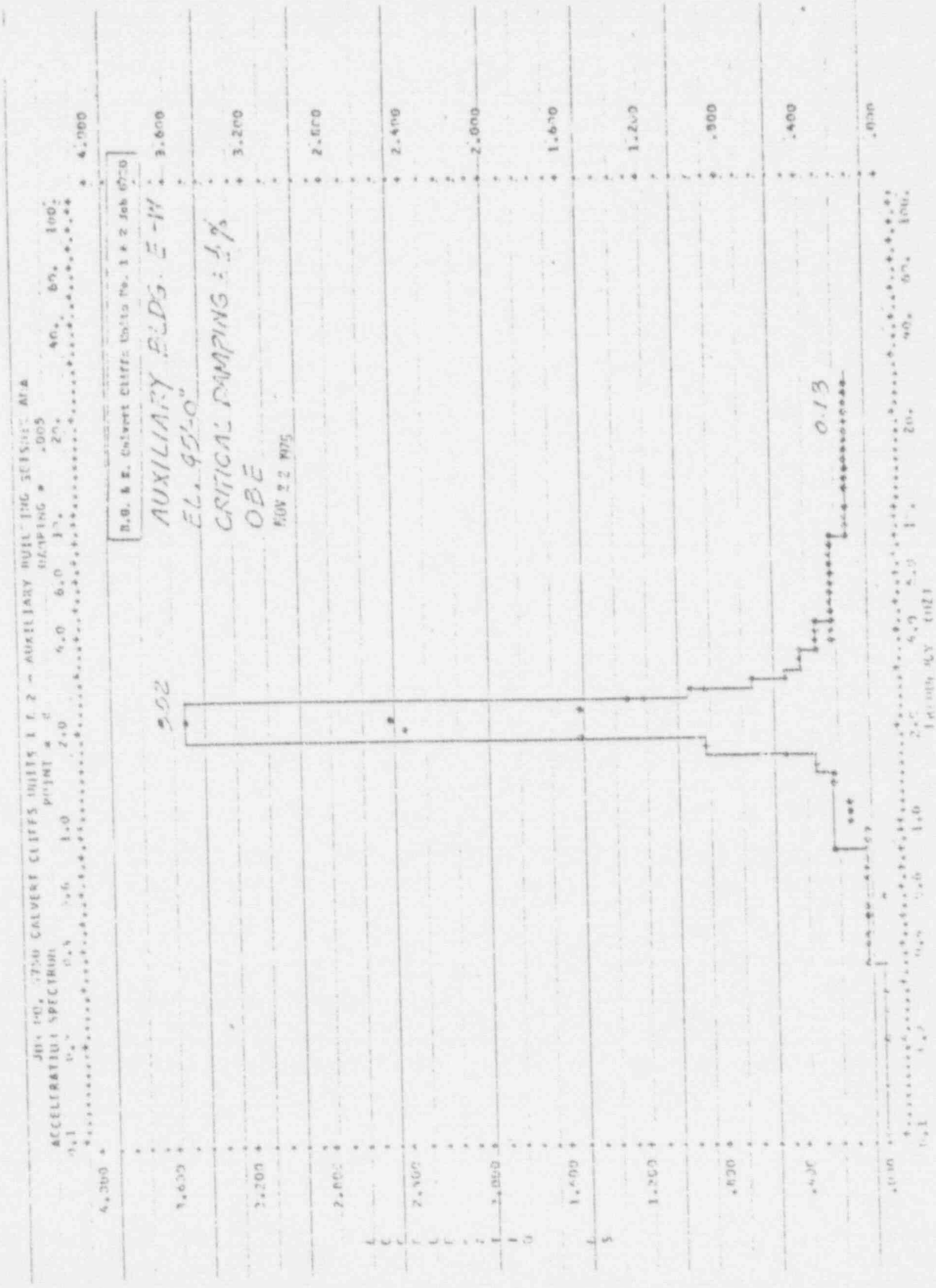
IN-STRUCTURE RESPONSE SPECTRA



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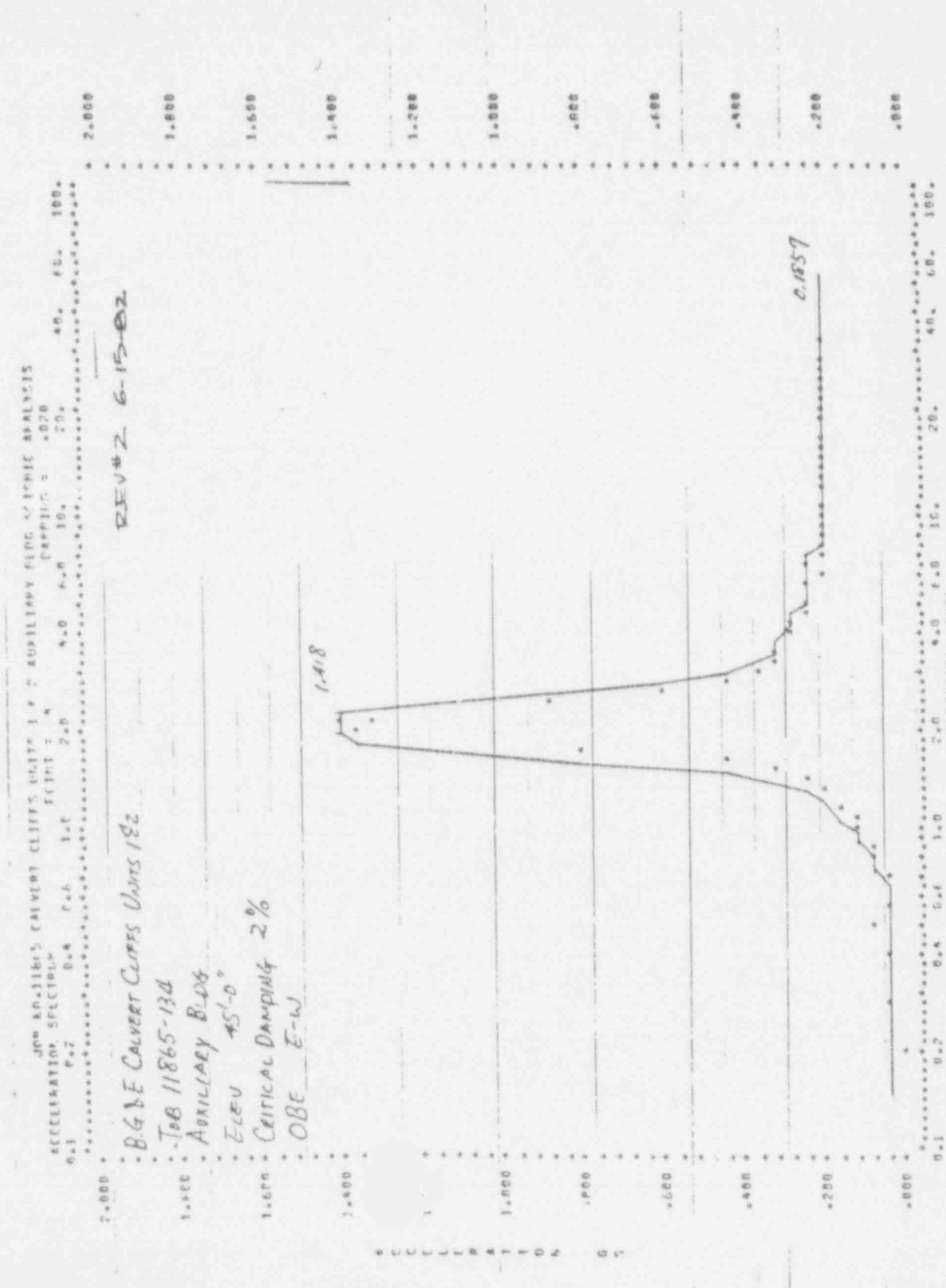

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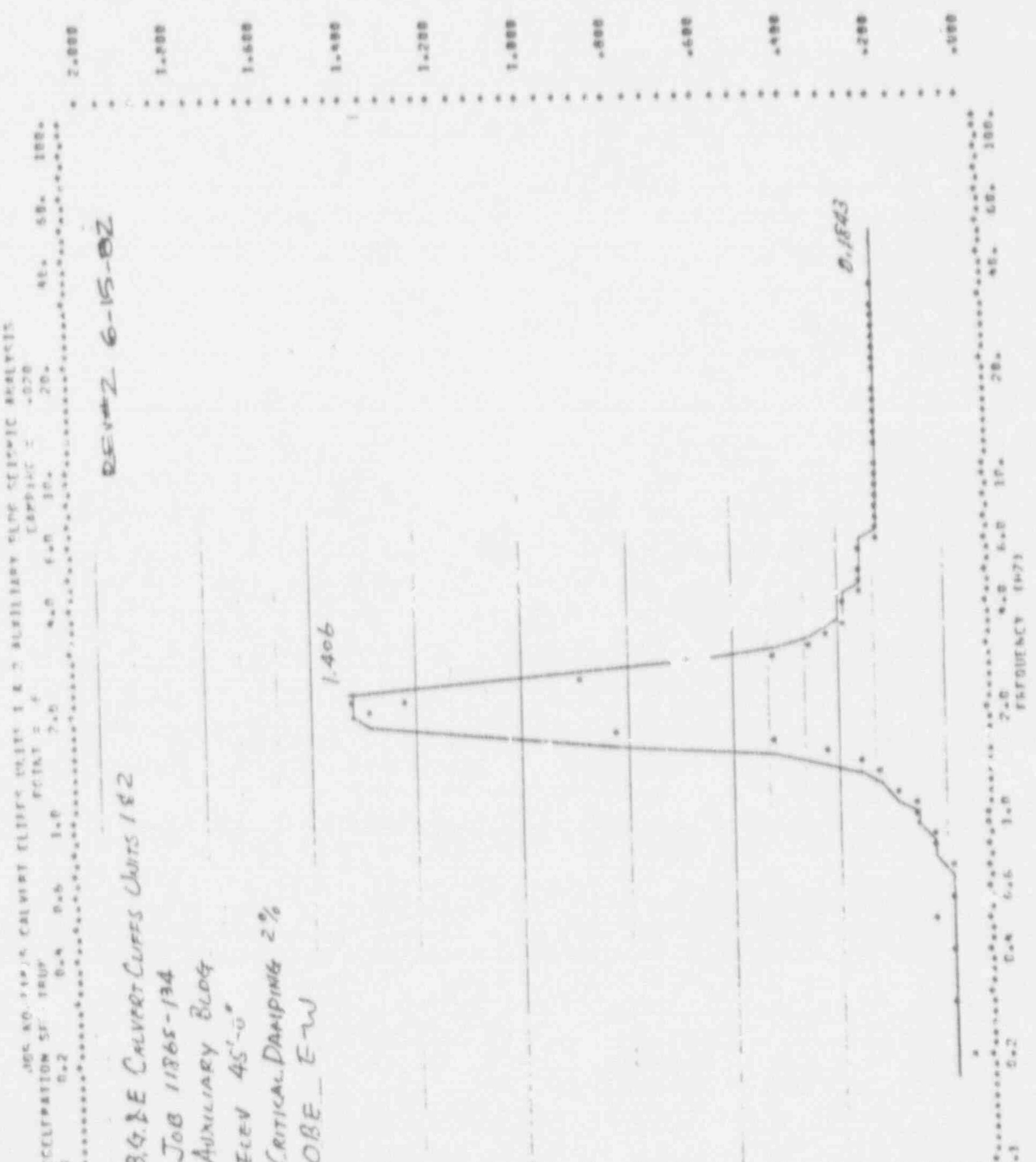
IN-STRUCTURE RESPONSE SPECTRA



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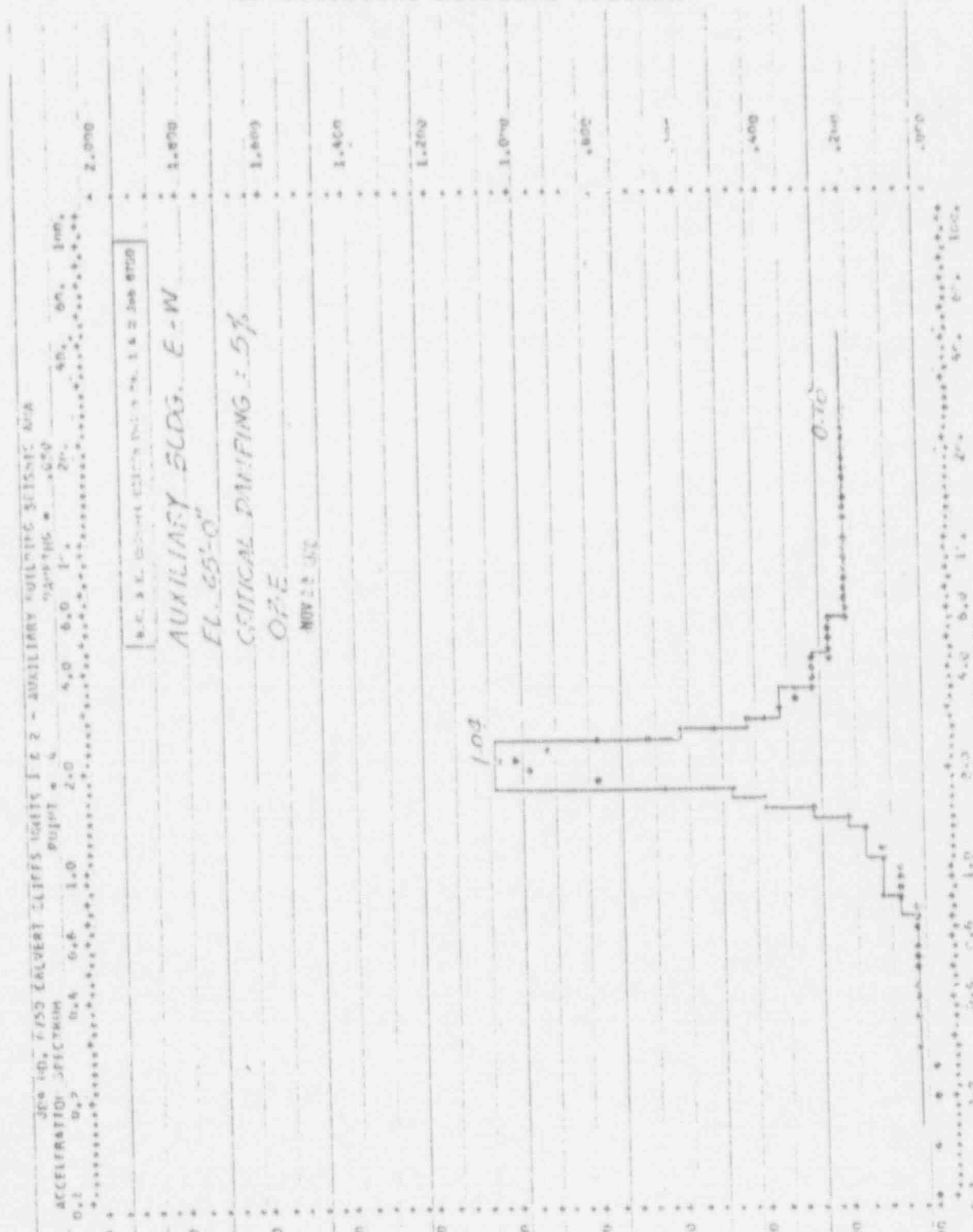

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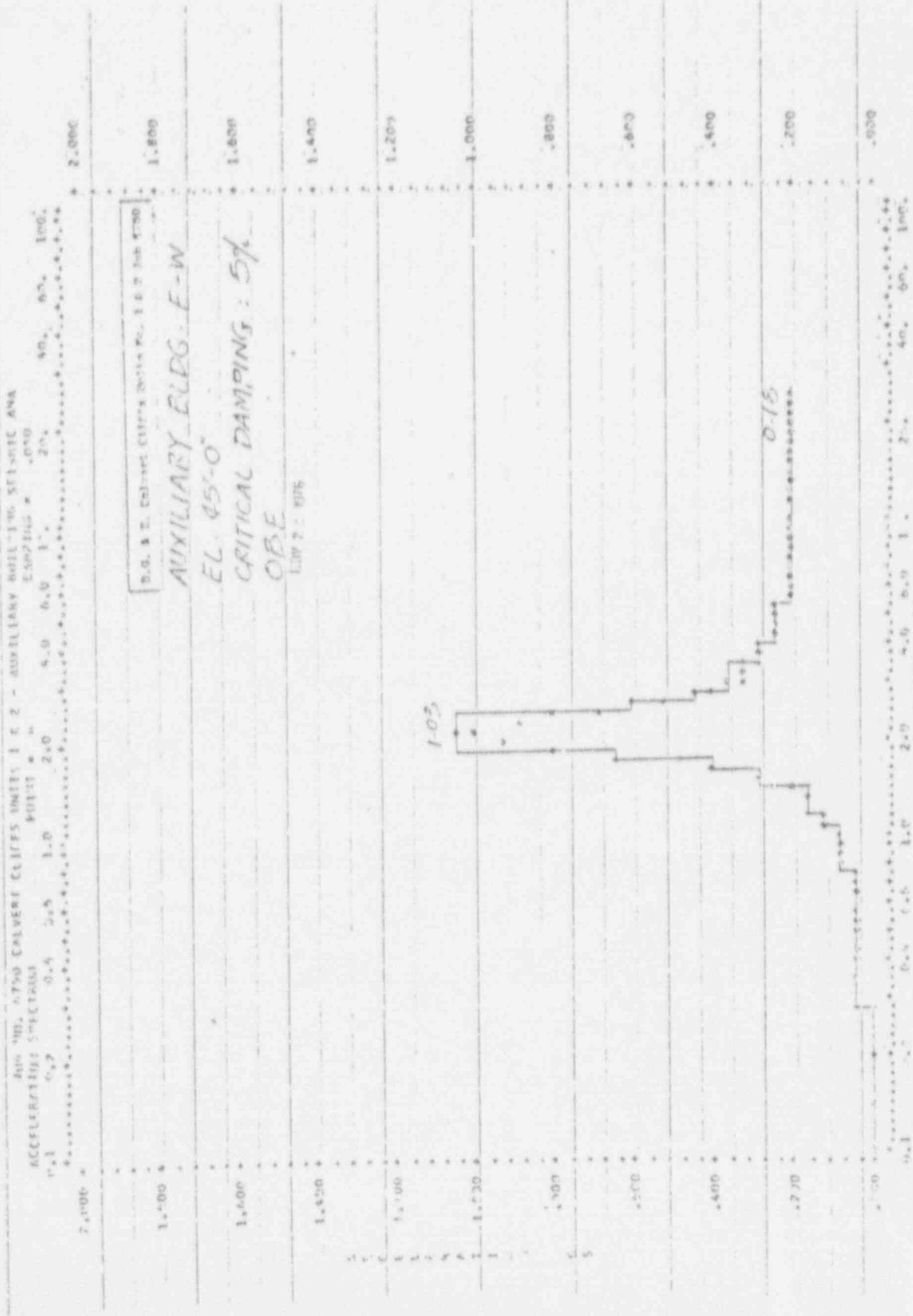

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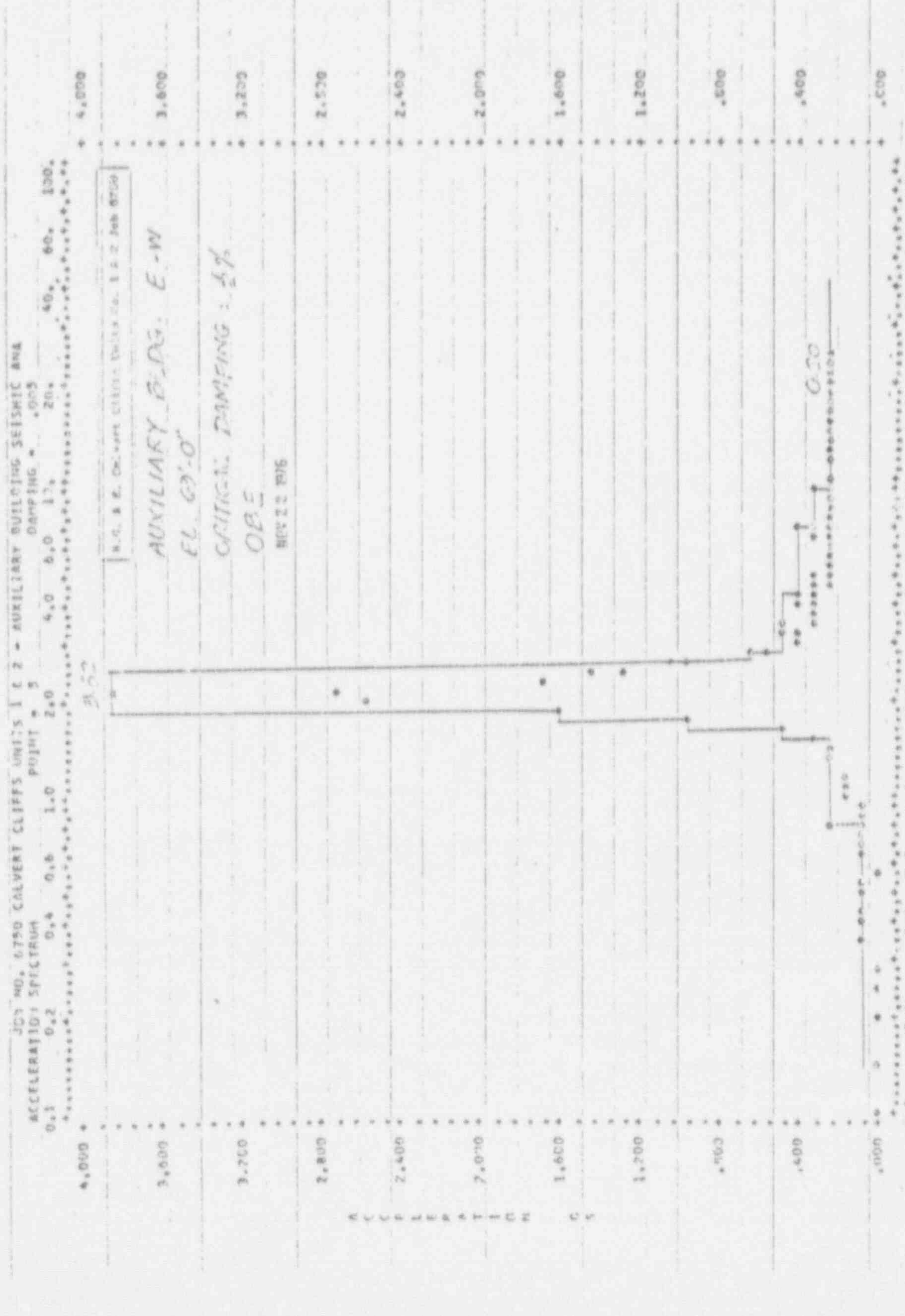
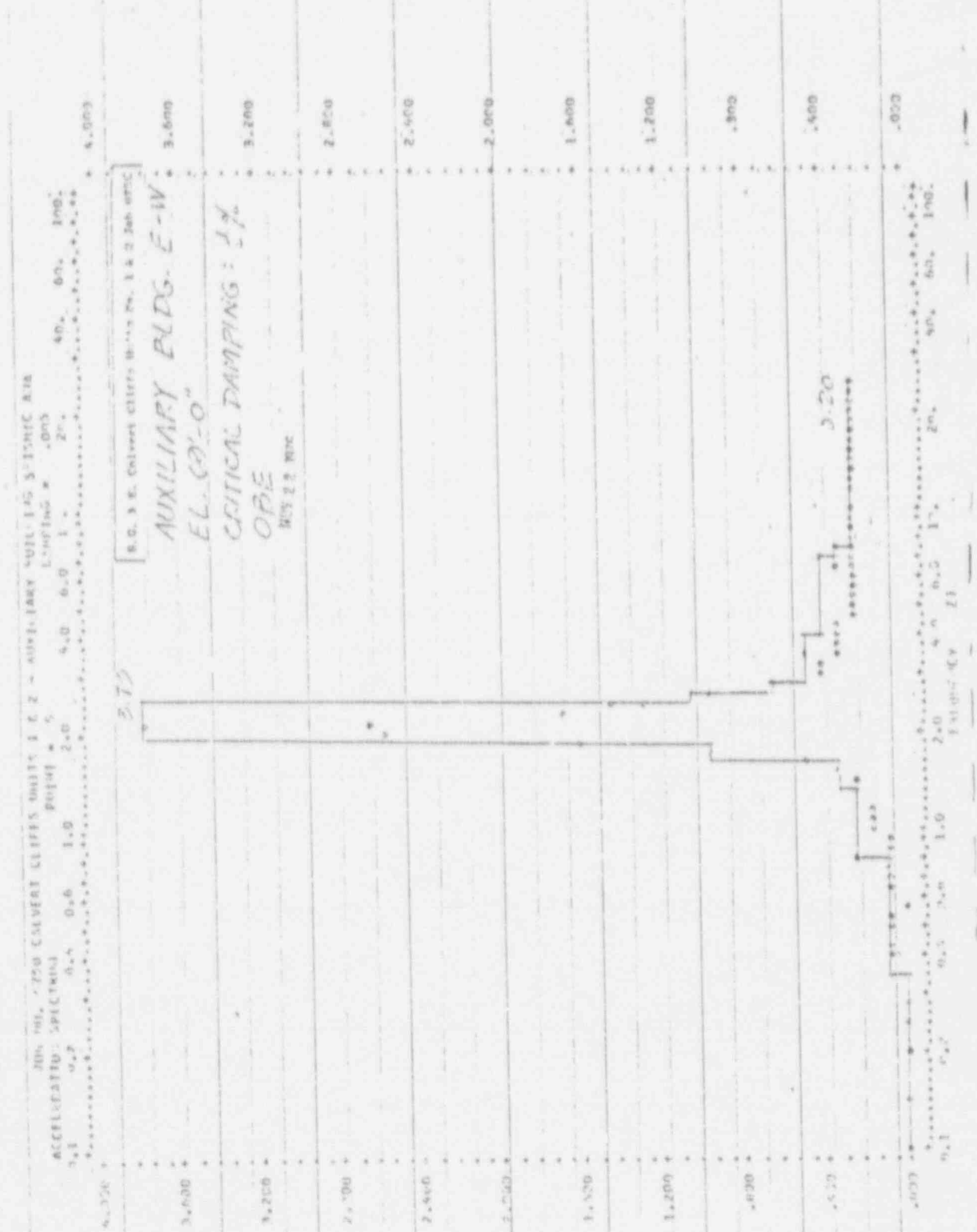
IN-STRUCTURE RESPONSE SPECTRA



APPENDIX A



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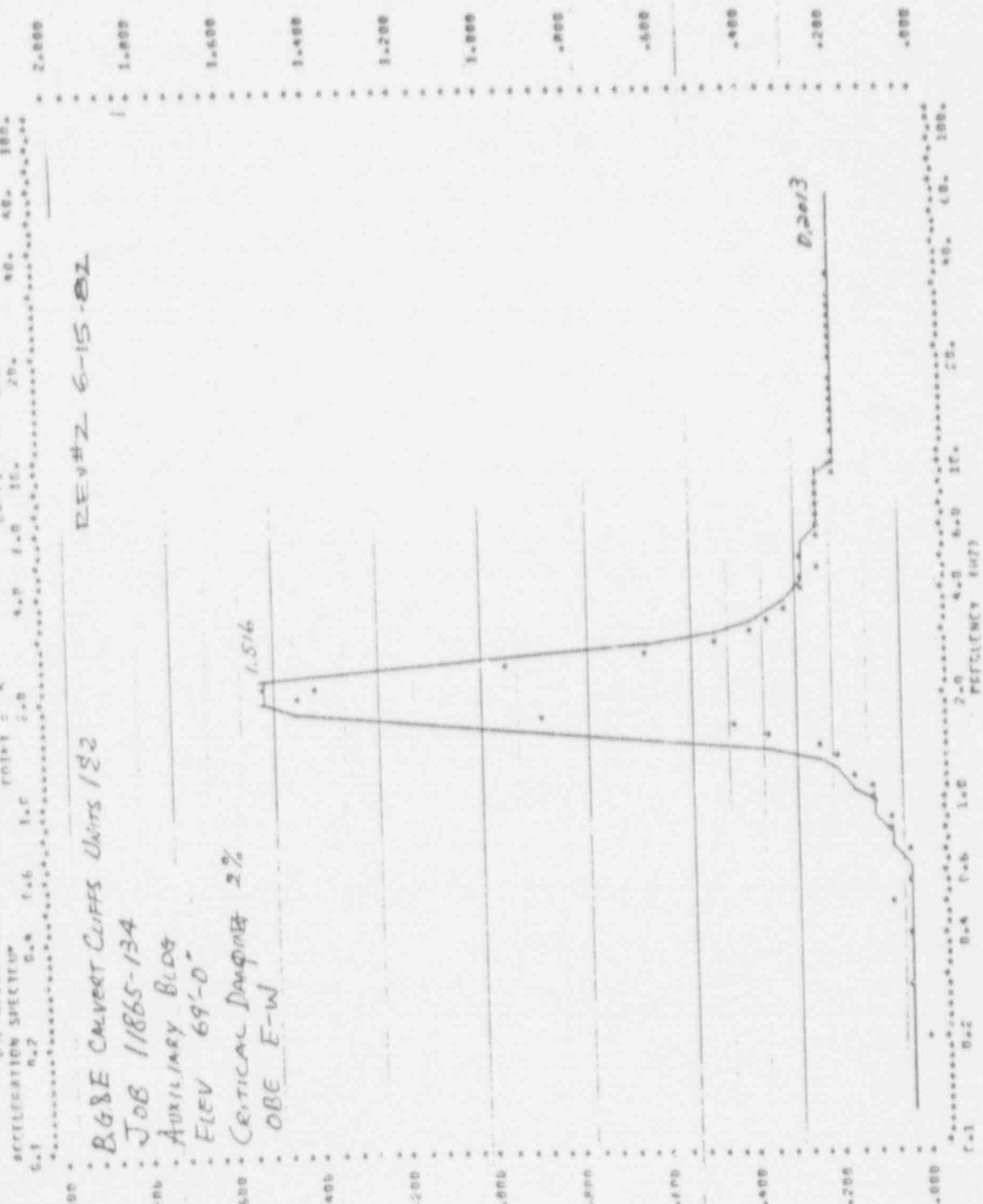
IN-STRUCTURE RESPONSE SPECTRA



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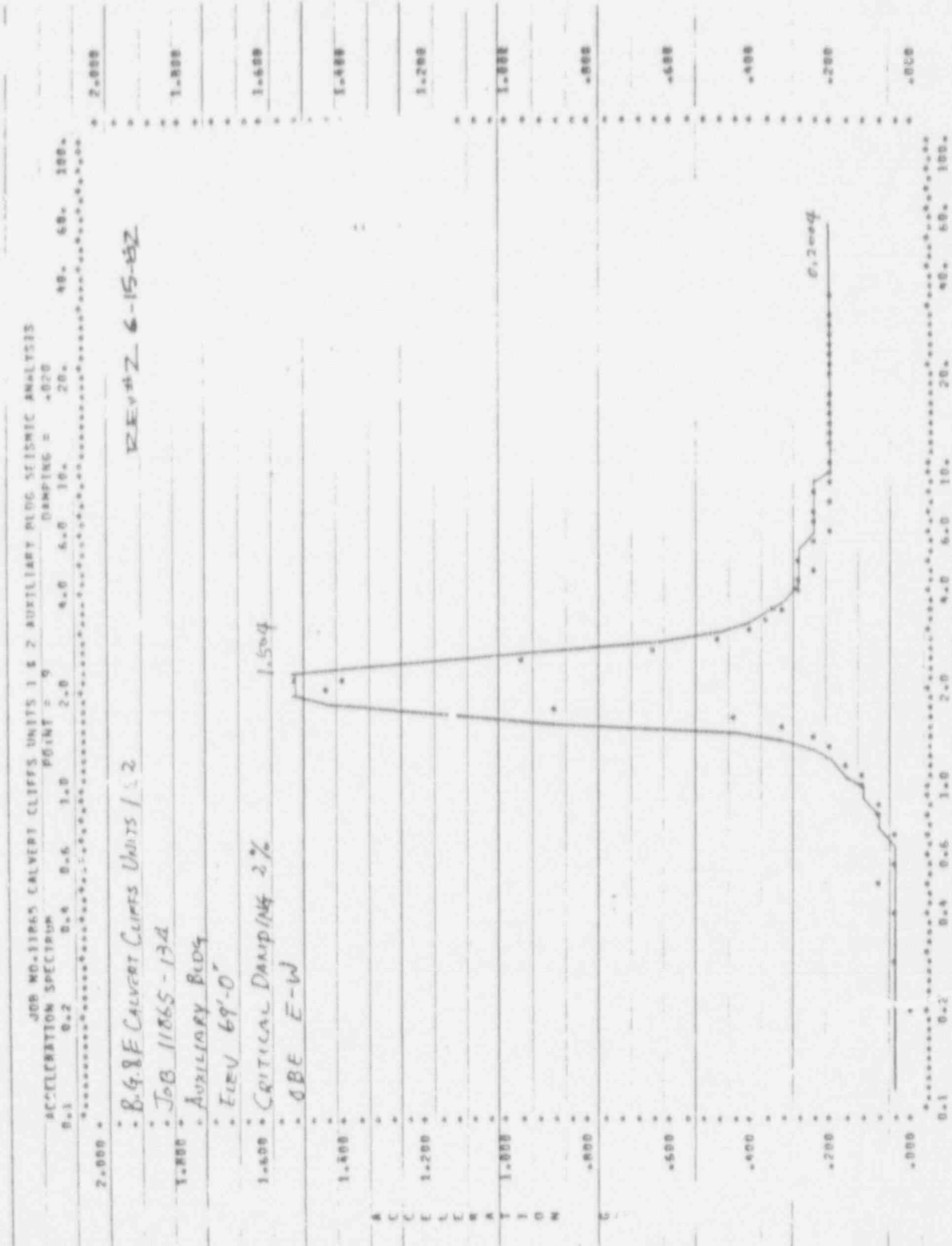
IN-STRUCTURE RESPONSE SPECTRA



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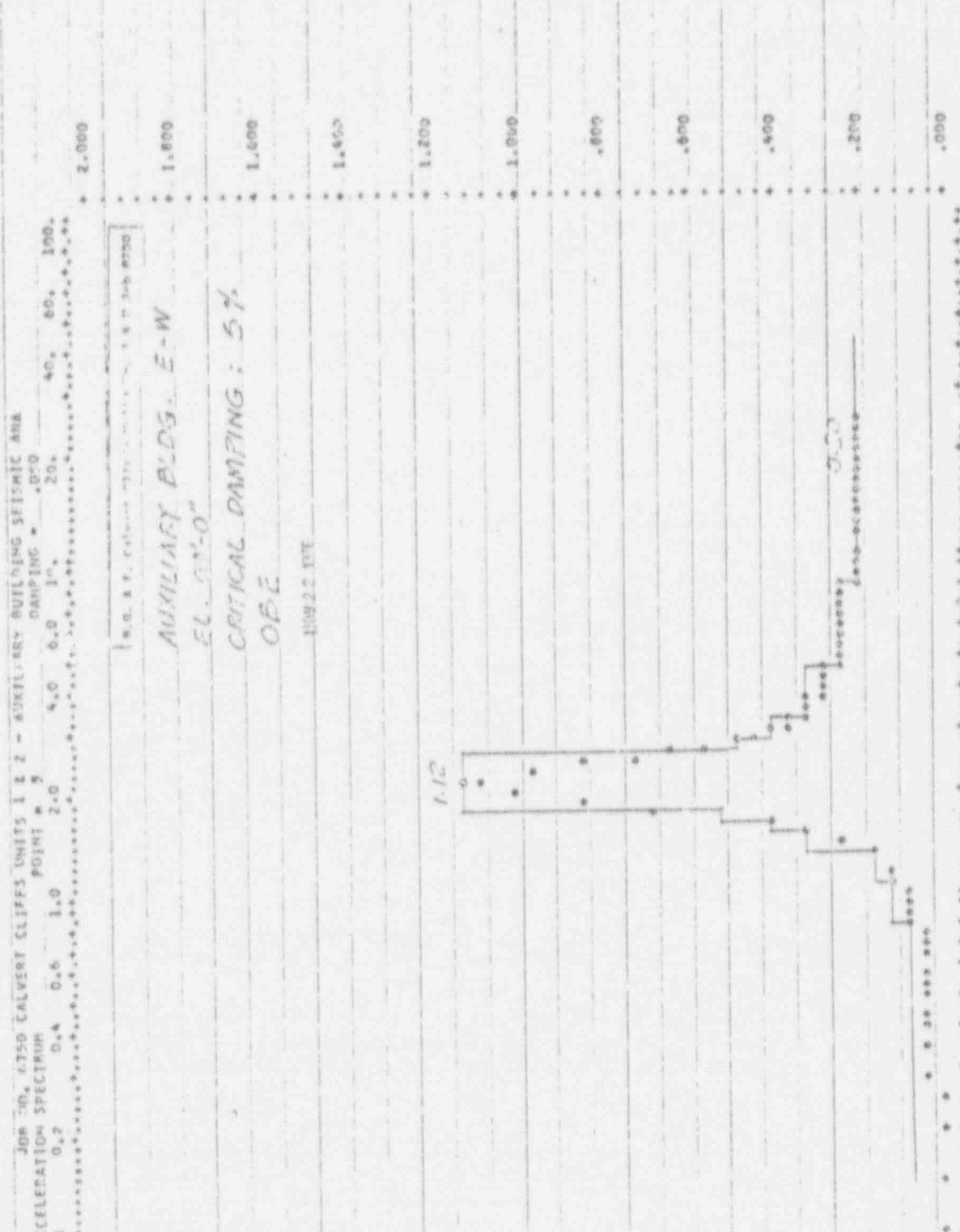
IN-STRUCTURE RESPONSE SPECTRA



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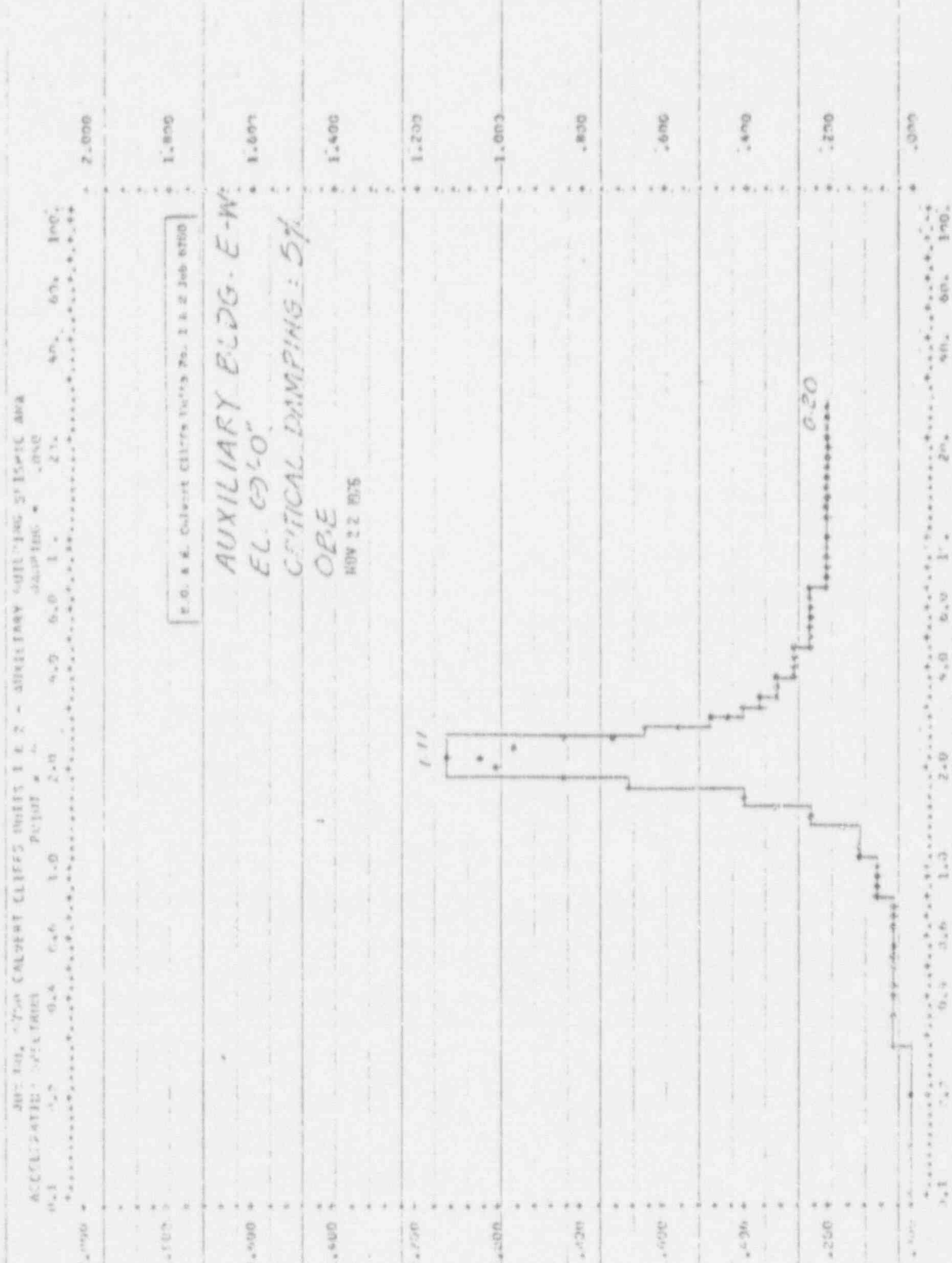
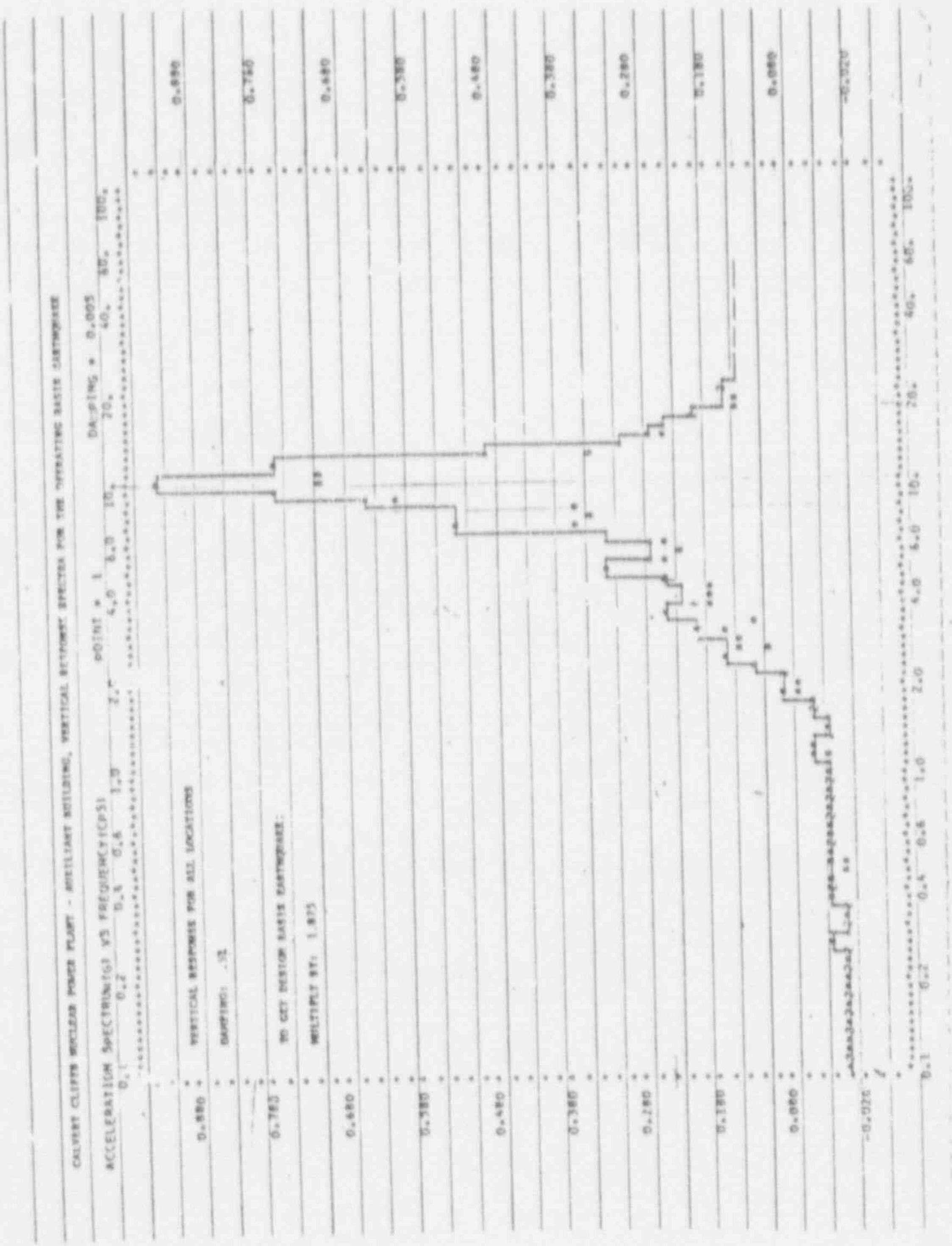
IN-STRUCTURE RESPONSE SPECTRA



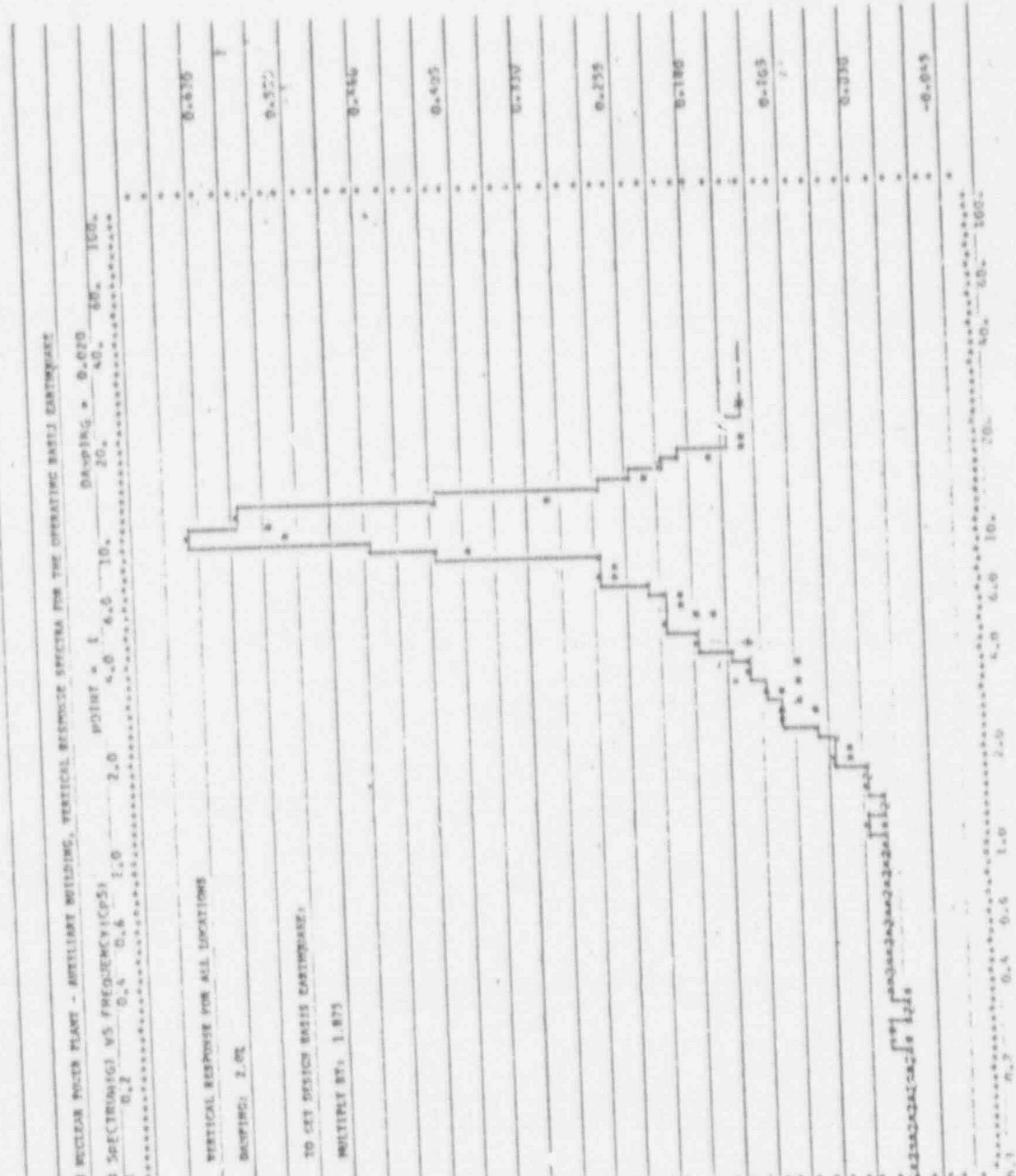
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



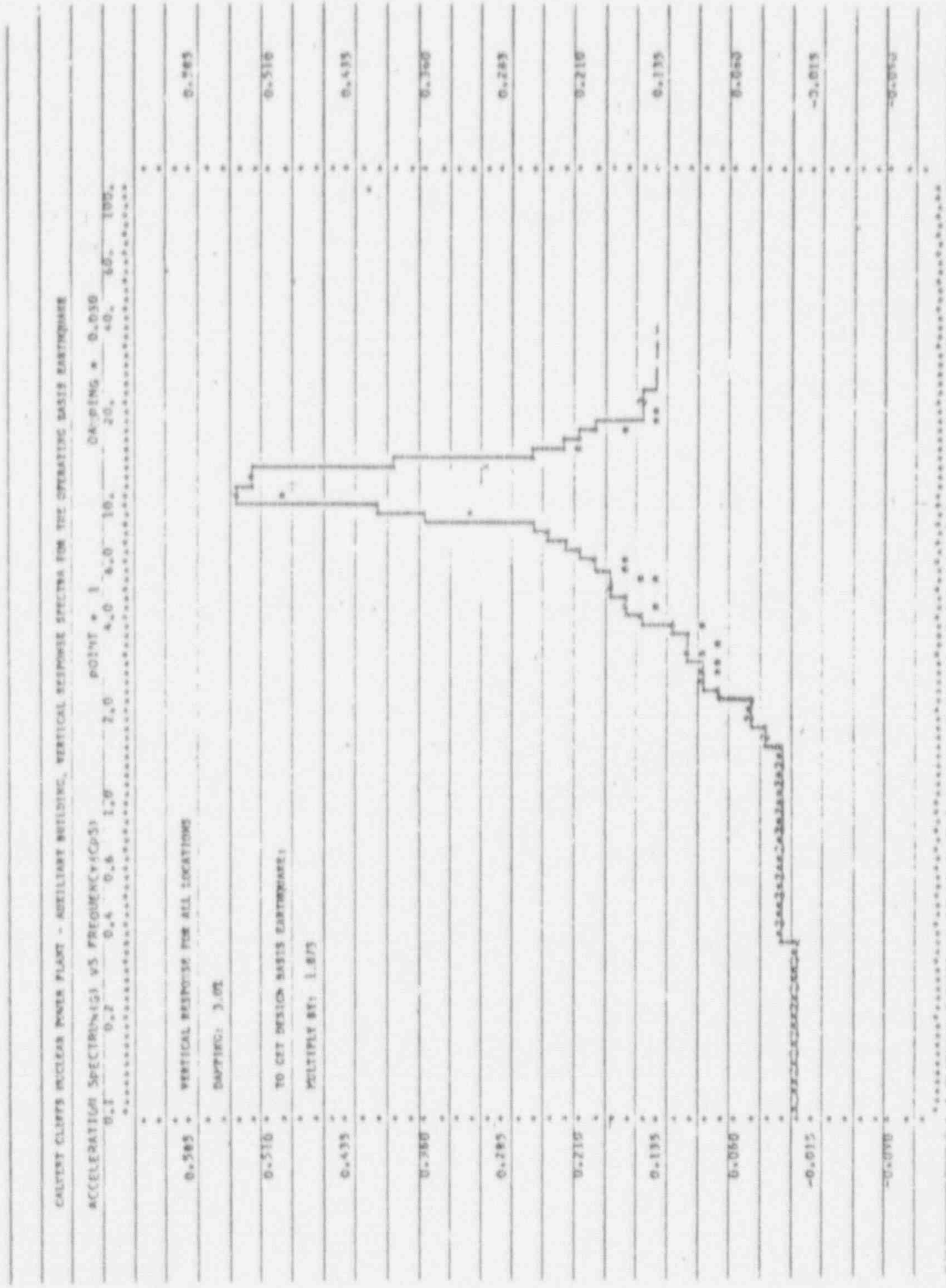
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA

TABLE A-C
AUXILIARY BUILDING
FLOOR ACCELERATIONS AND PEAK FLOOR RESPONSE ACCELERATIONS
for
Design Basis Earthquake

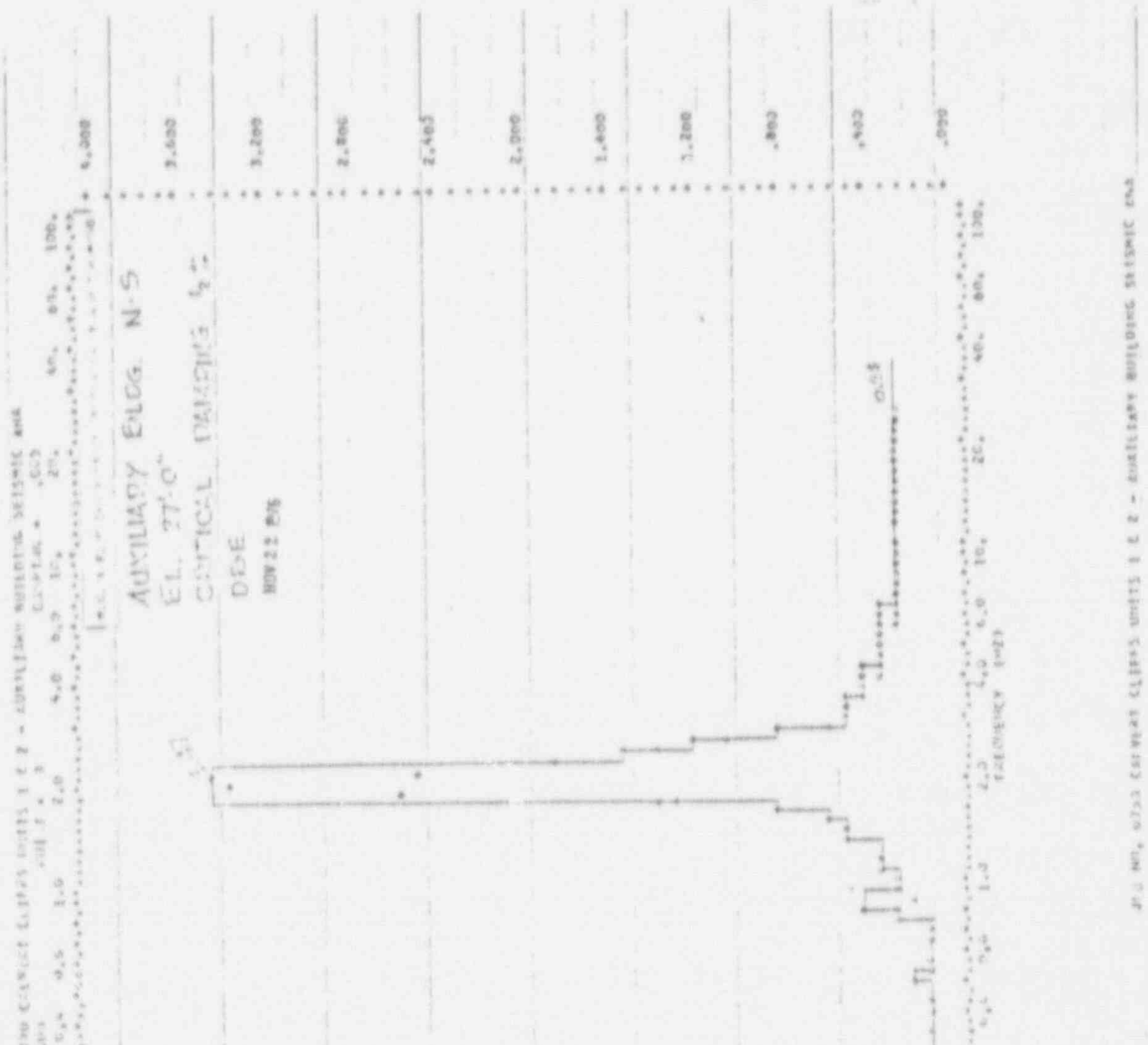
ELEVATION	PEAK FLOOR RESPONSE ACCELERATIONS[4] for component damping of 1/2 %	2 %	5 %	FLOOR ACCELERATION [ZPA]
(ft.)	(g's)	(g's)	(g's)	(g's)
North-South Motion				
27'	3.470	2.090	1.210	0.244
45' E. of SFP	3.910	2.180	1.340	0.273
W. of SFP	3.890	2.300	1.330	0.271
69' E. of SFP	4.900	2.790	1.630	0.349
W. of SFP	4.450	2.750	1.490	0.315
East-West Motion				
27'	4.770	2.450	1.440	0.282
45' E. of SFP	5.020	2.640	1.540	0.303
W. of SFP	4.980	2.600	1.520	0.301
69' E. of SFP	5.400	2.806	1.650	0.328
W. of SFP	5.350	2.790	1.640	0.327
Vertical Motion				
All	1.744	1.219	1.030[5]	0.253

4 Accelerations at Auxiliary Building Natural Frequencies of 1.57 Hz for North-South Motion, 2.11 Hz for East-West Motion and 10.00 Hz for Vertical Motion.

5 Peak Floor Response Acceleration at 3 percent critical damping.

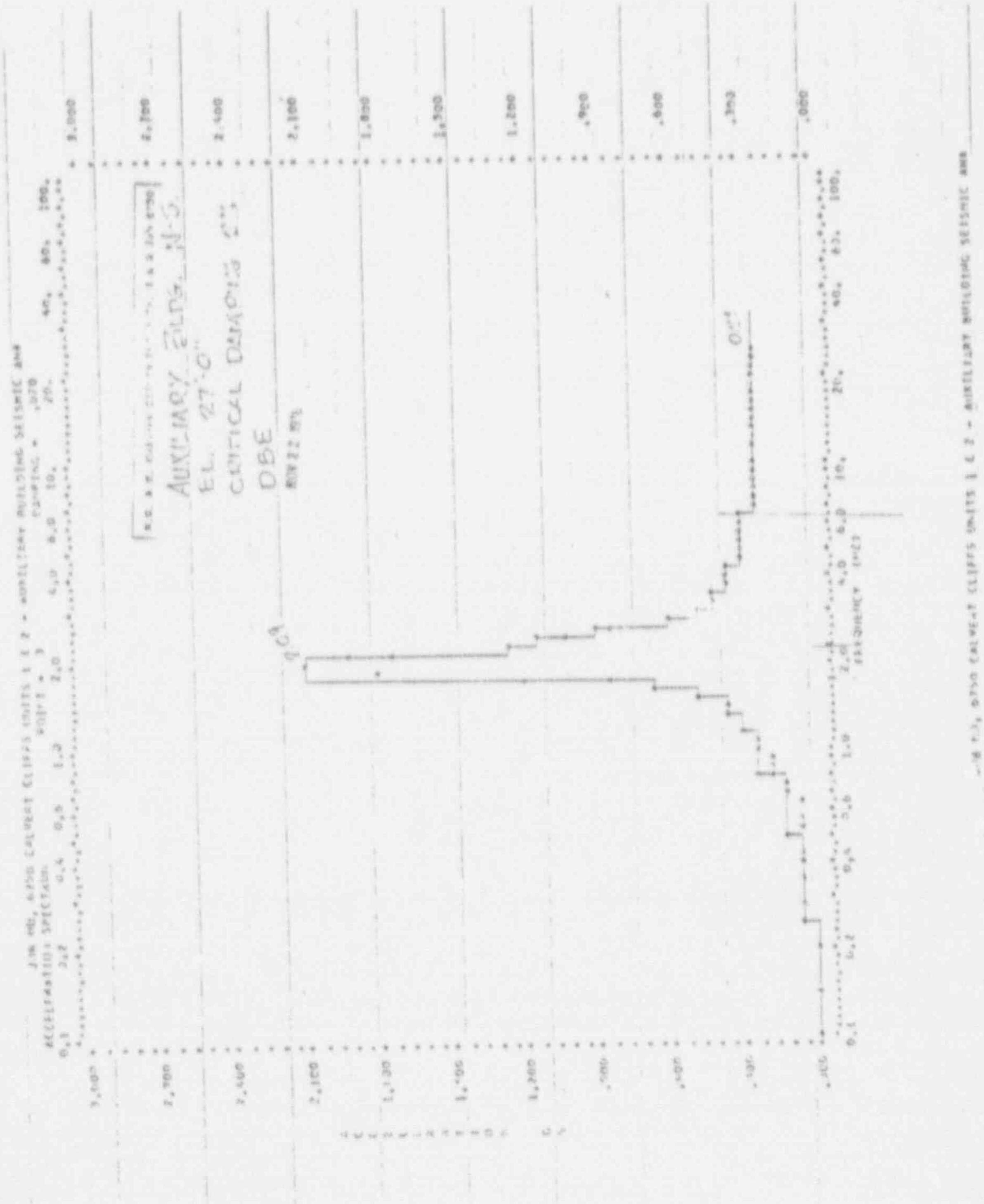
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



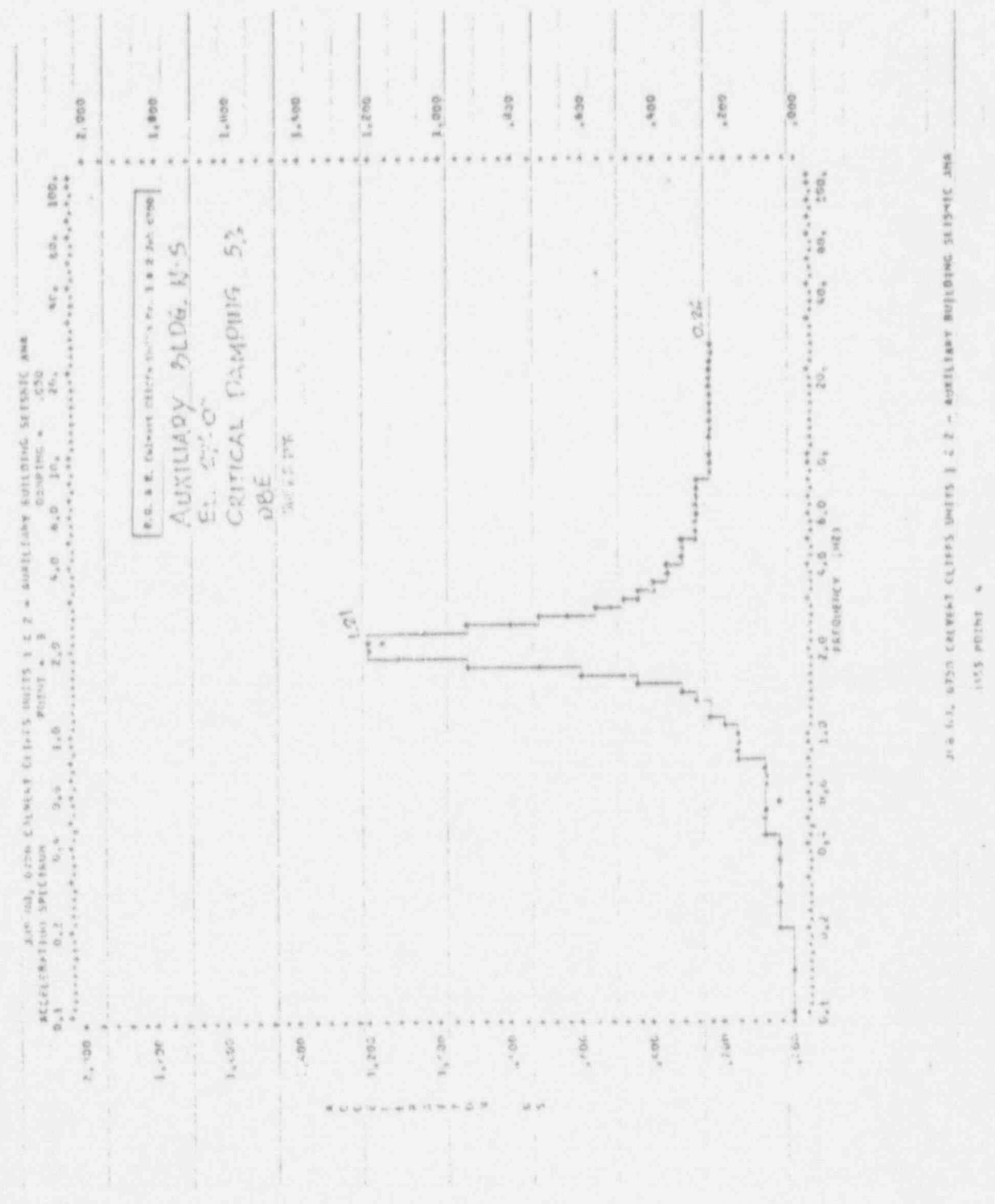
APPENDIX 1

IN-STRUCTURE RESPONSE SPECTRA



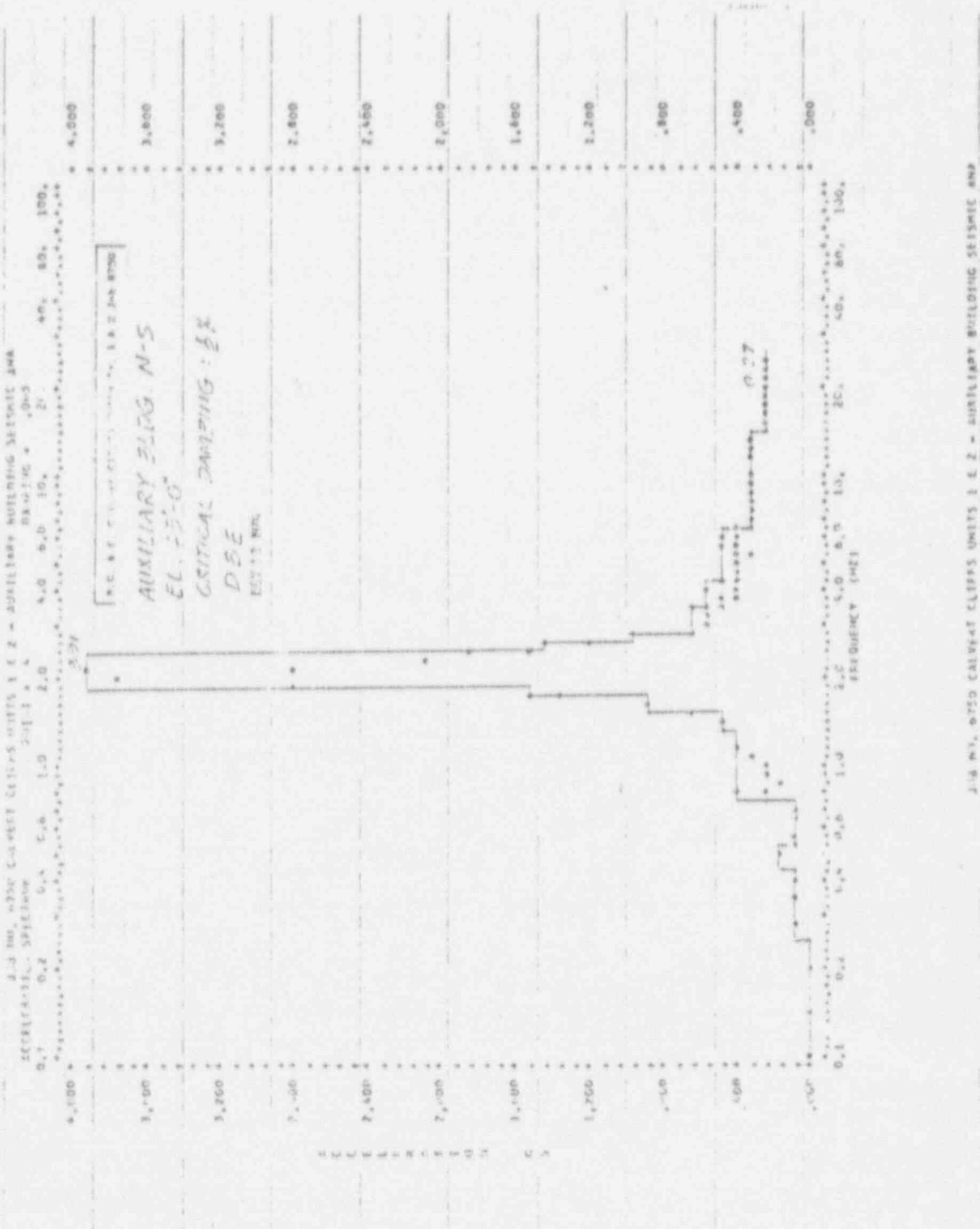
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



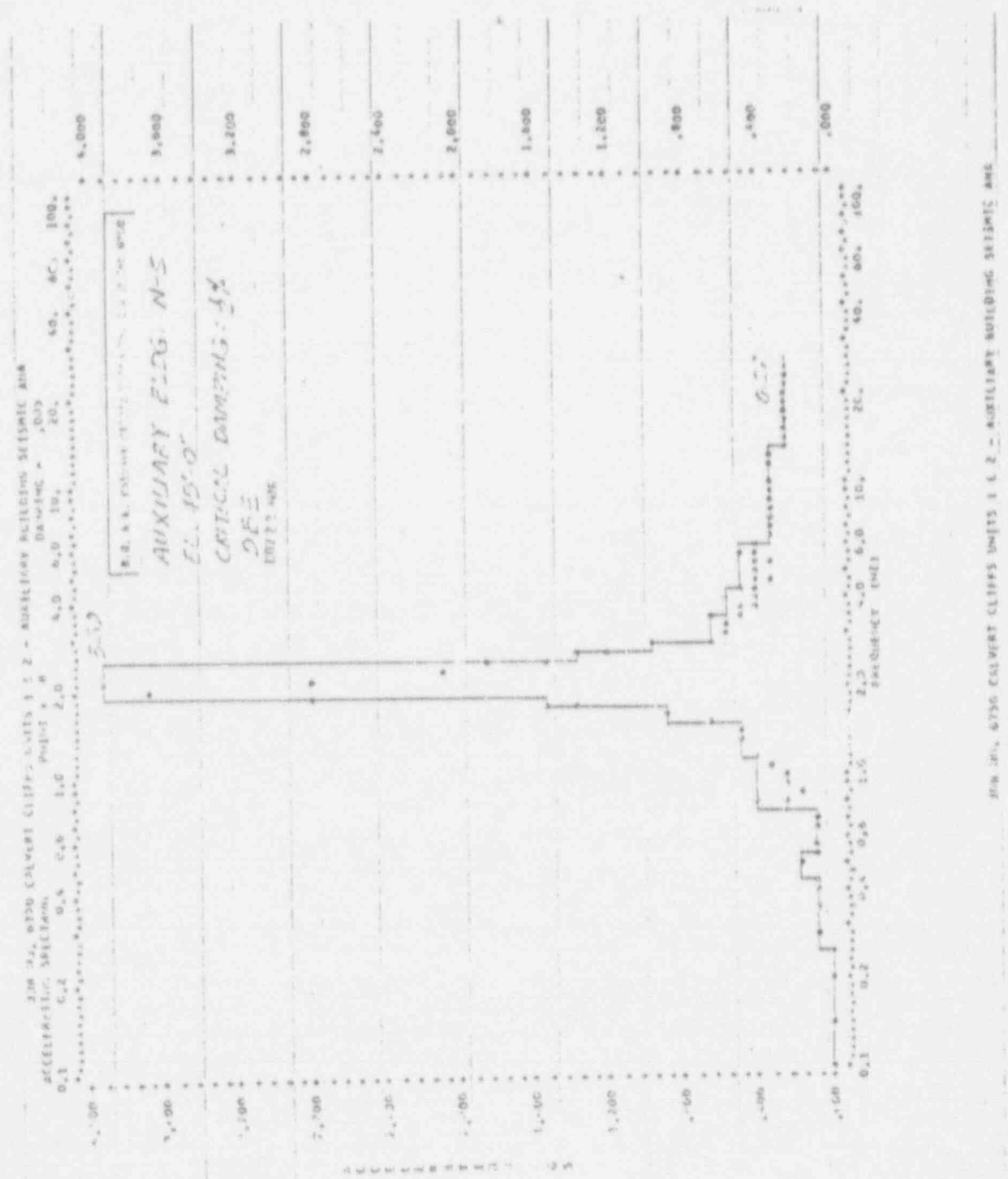
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



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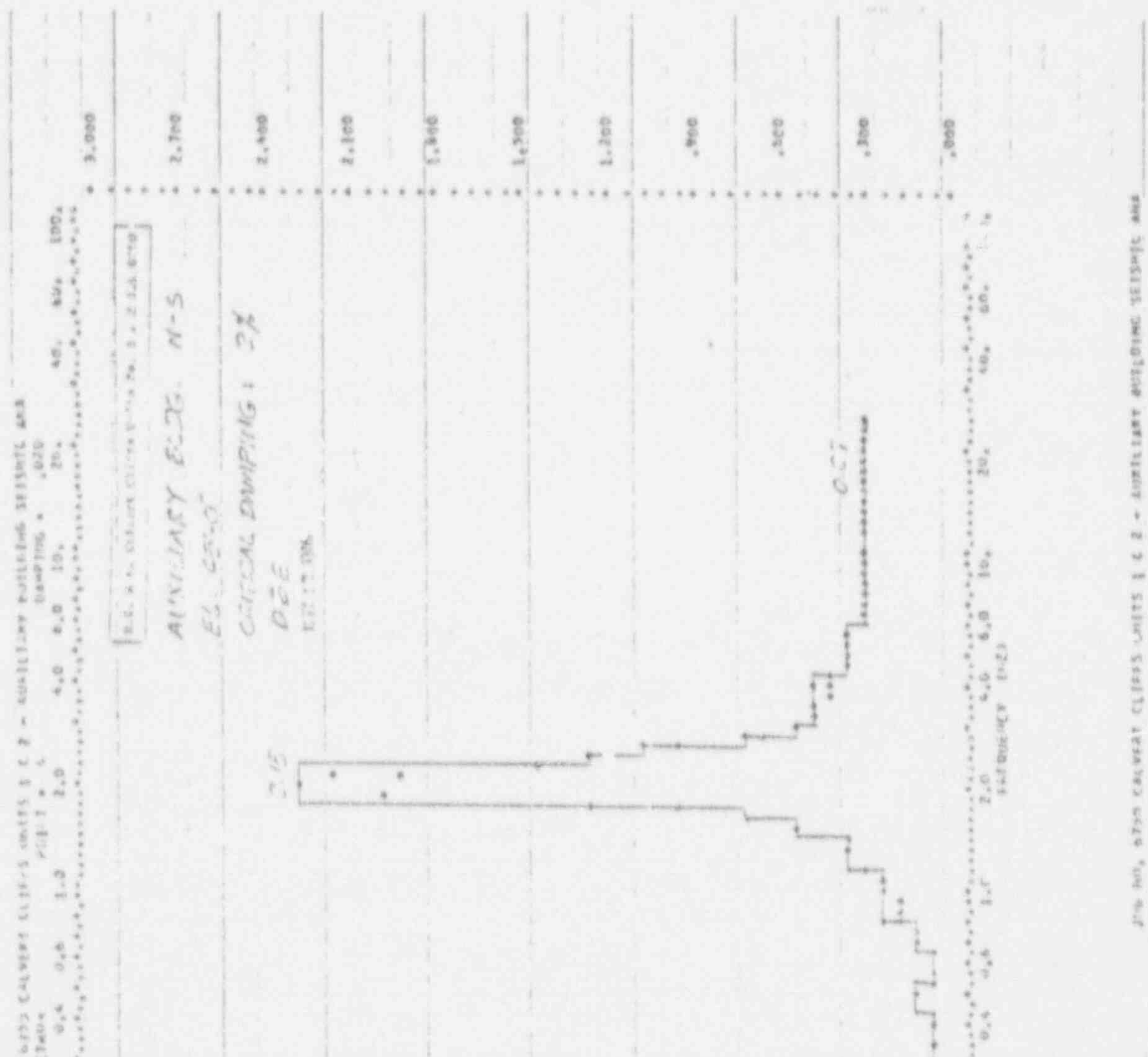

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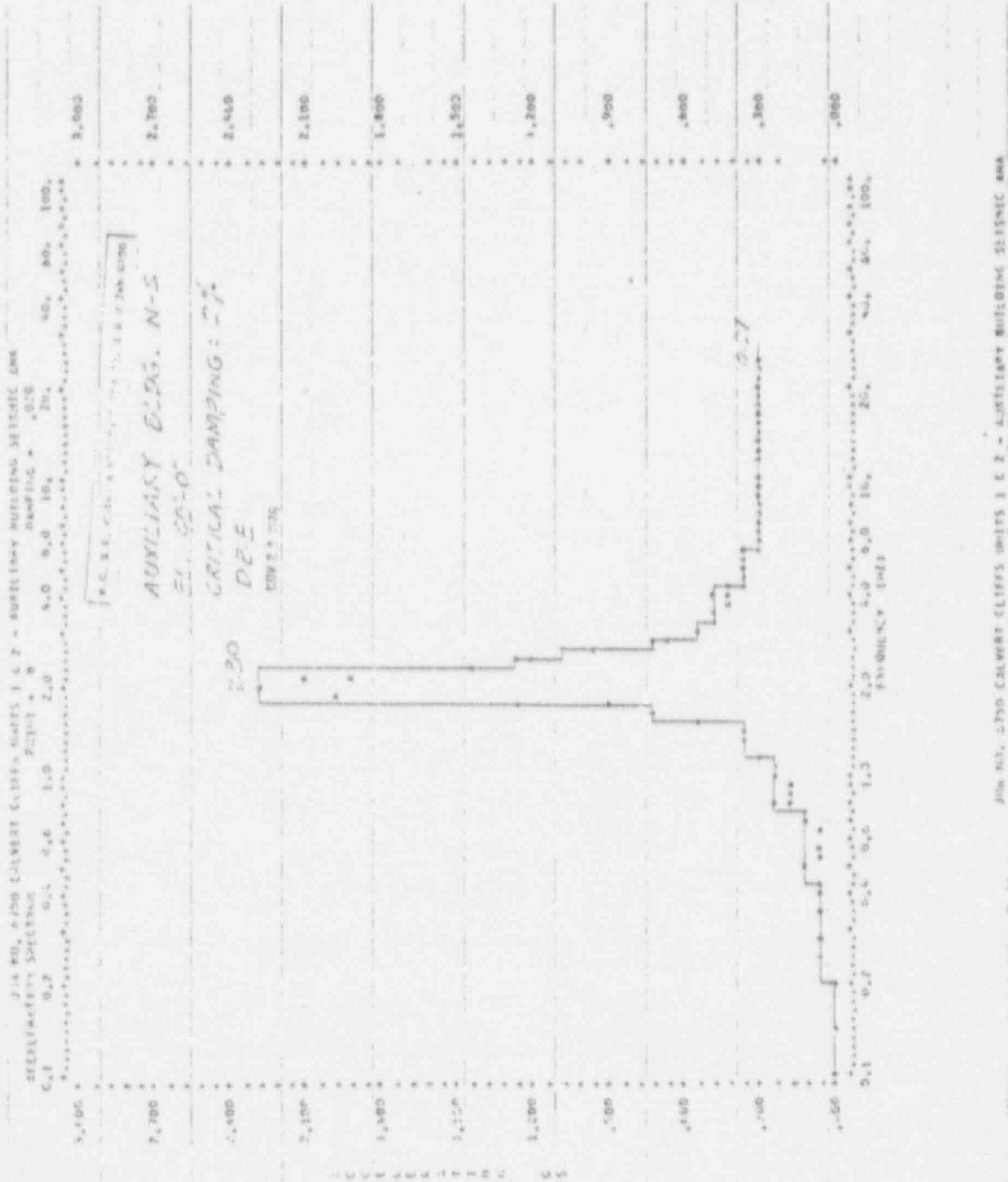
IN-STRUCTURE RESPONSE SPECTRA



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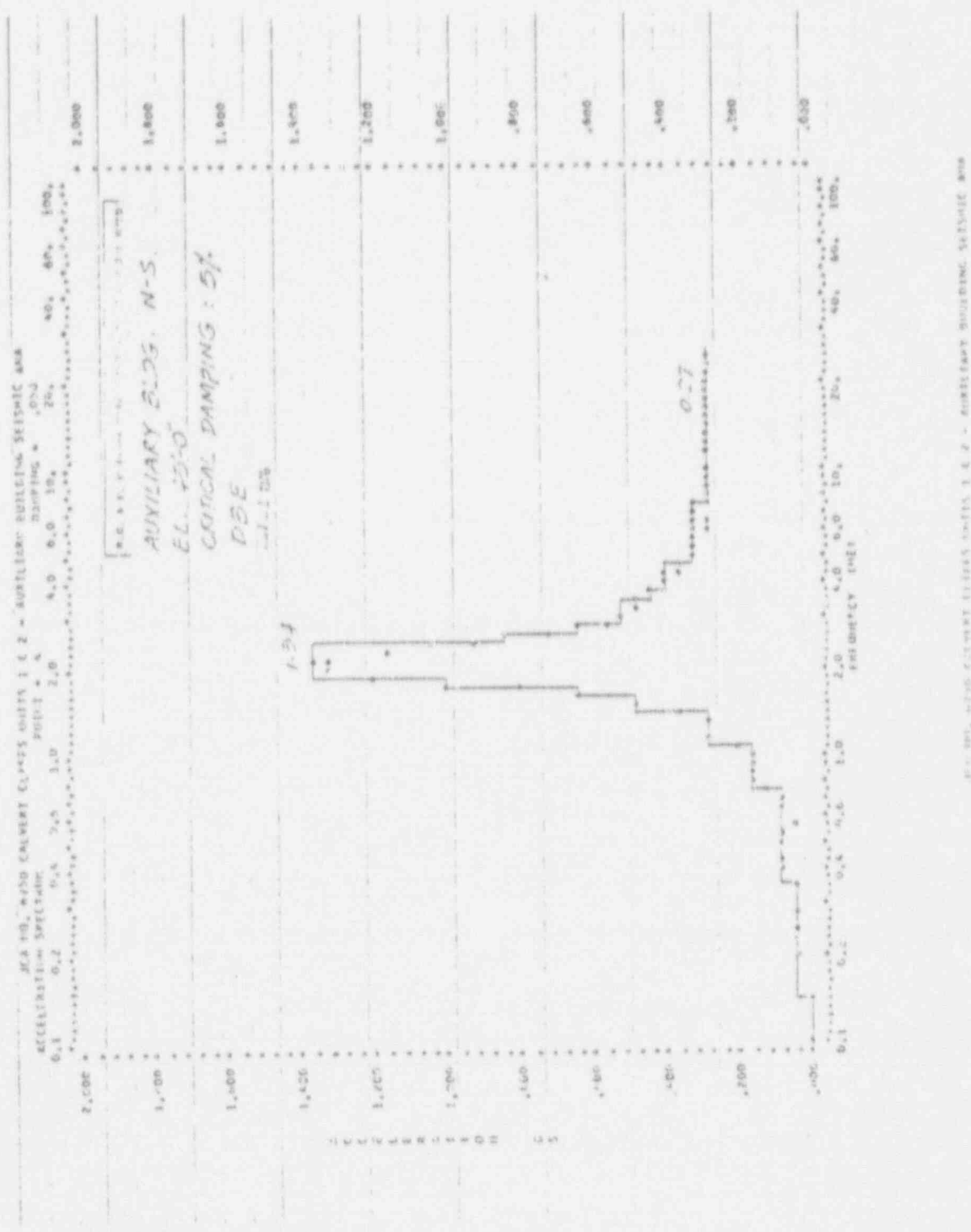
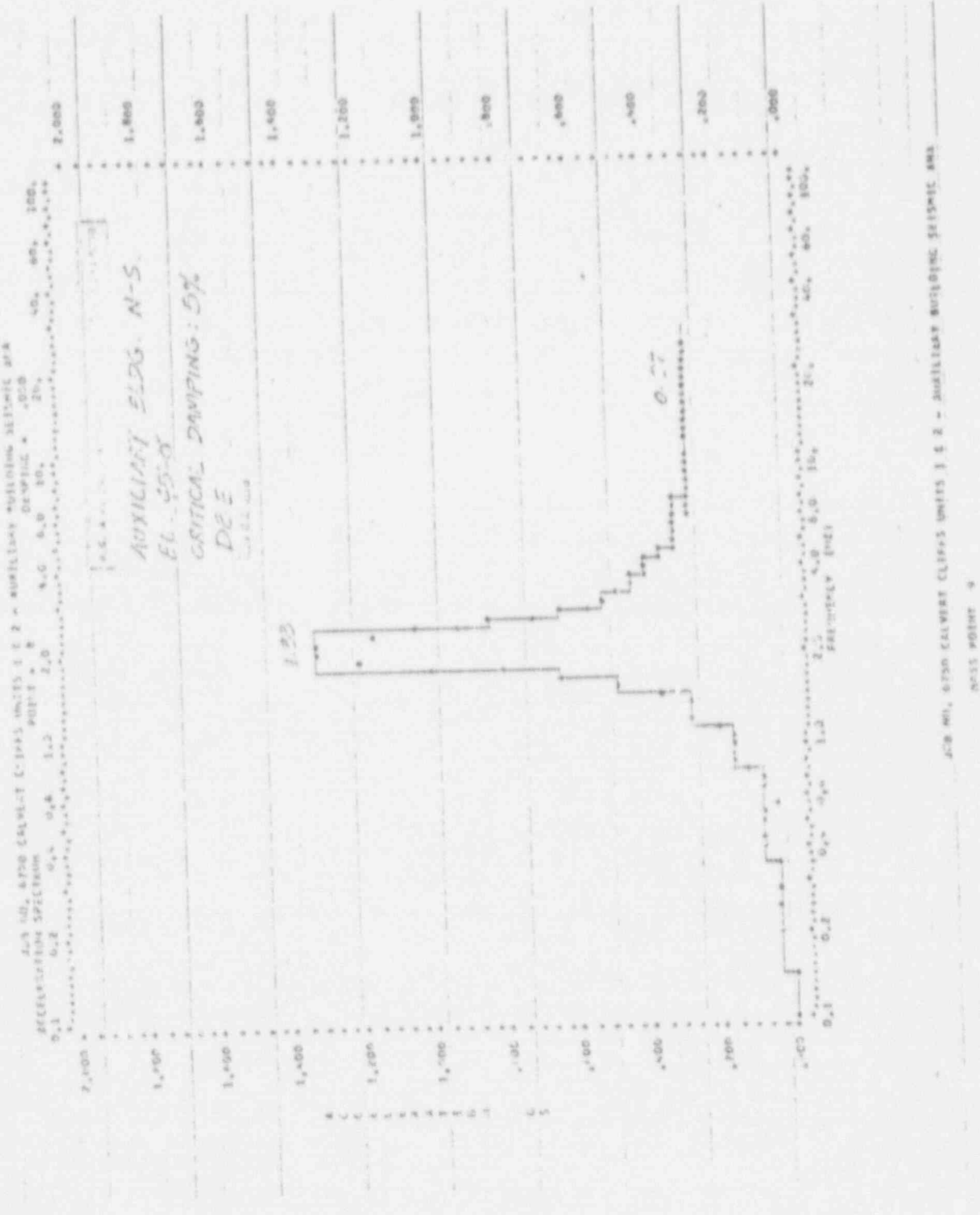


APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



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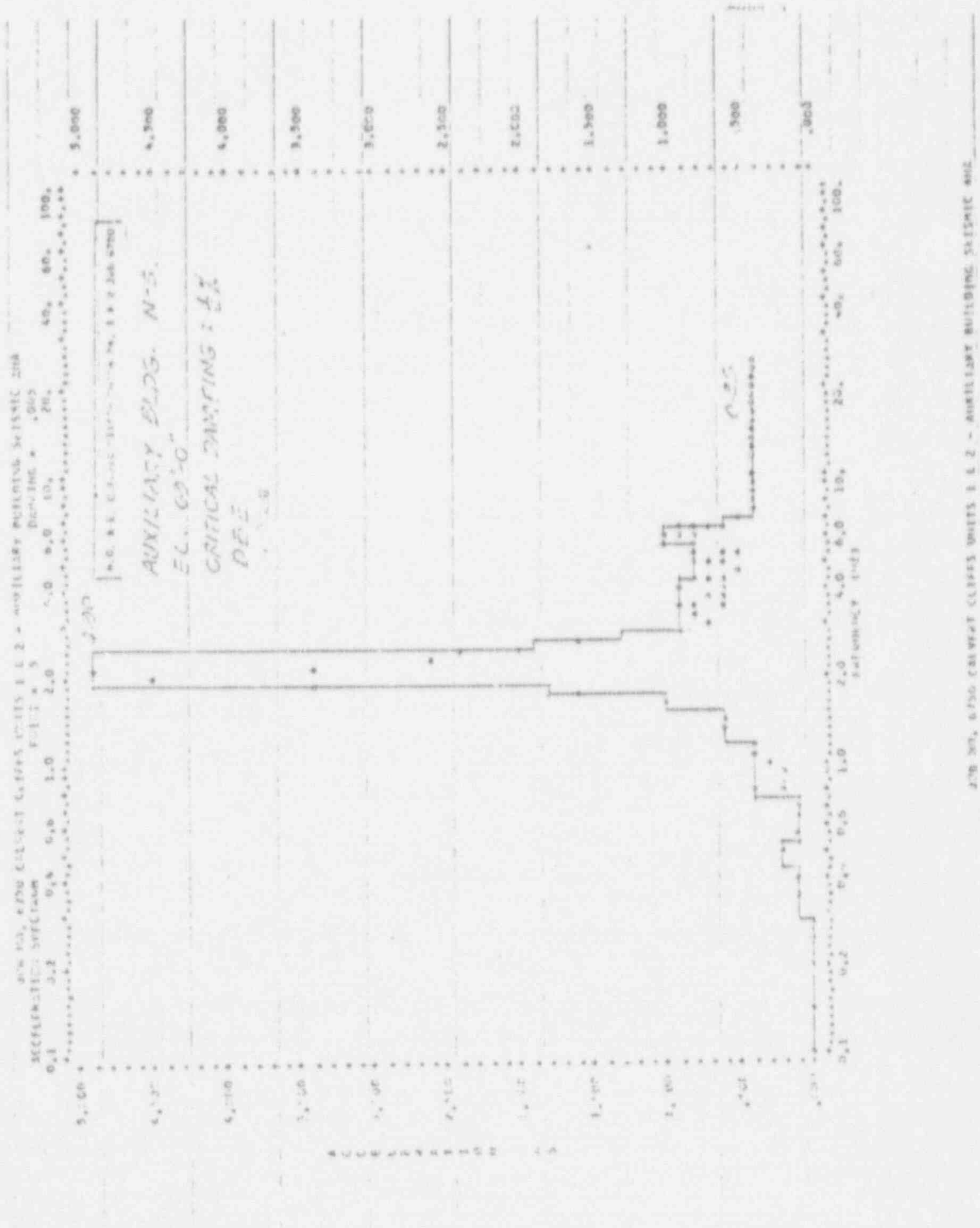
IN-STRUCTURE RESPONSE SPECTRA



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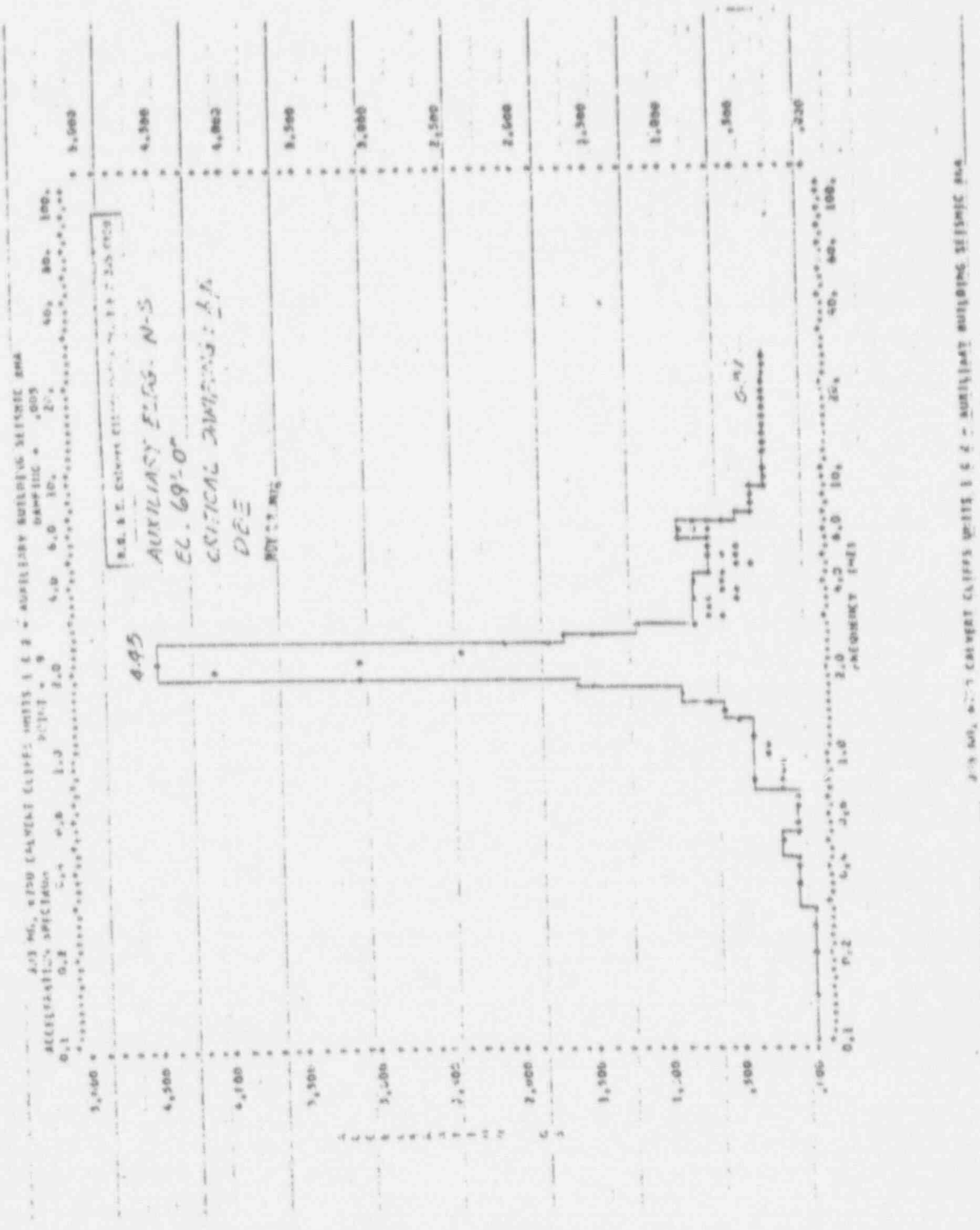
IN-STRUCTURE RESPONSE SPECTRA



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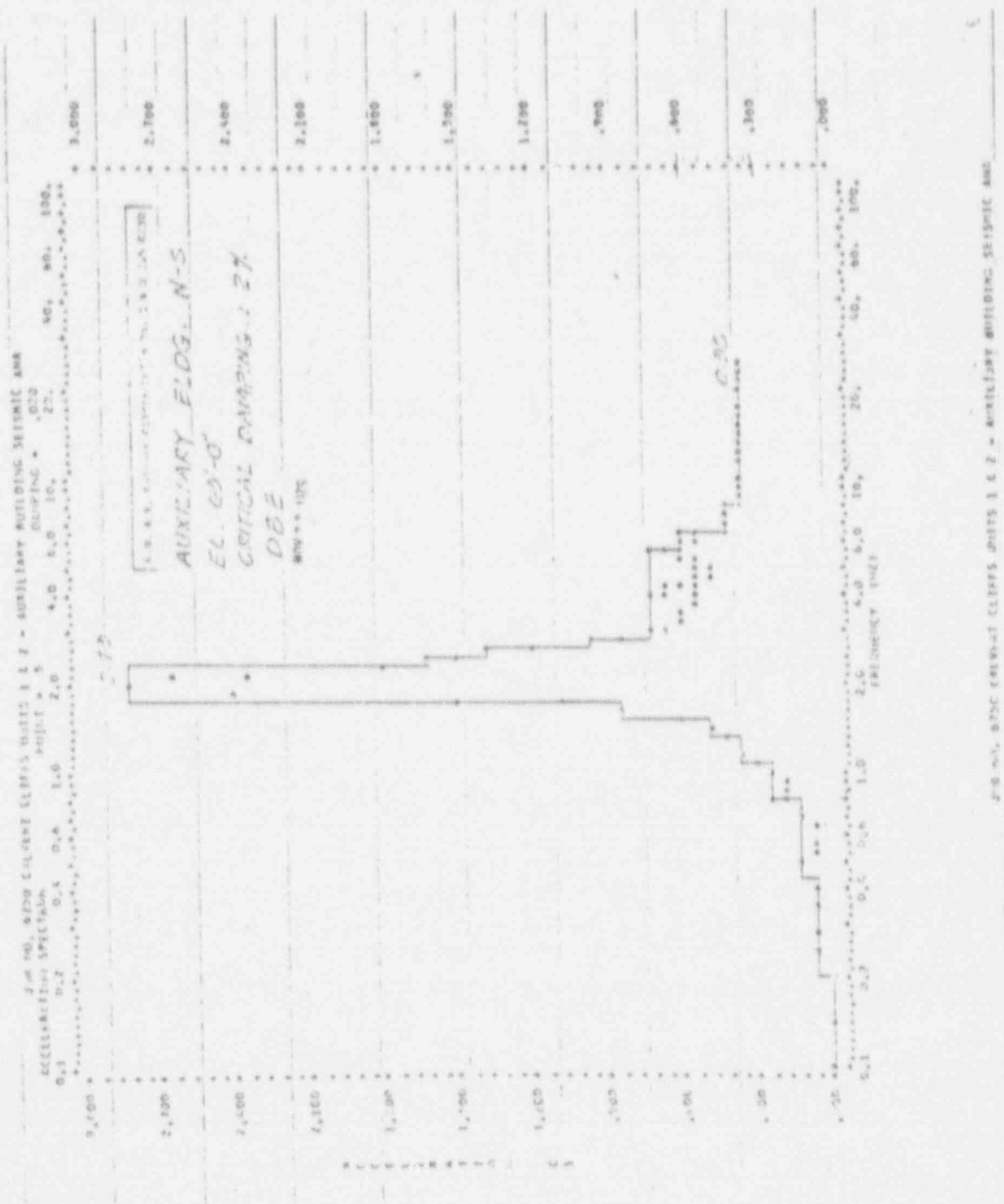
IN-STRUCTURE RESPONSE SPECTRA



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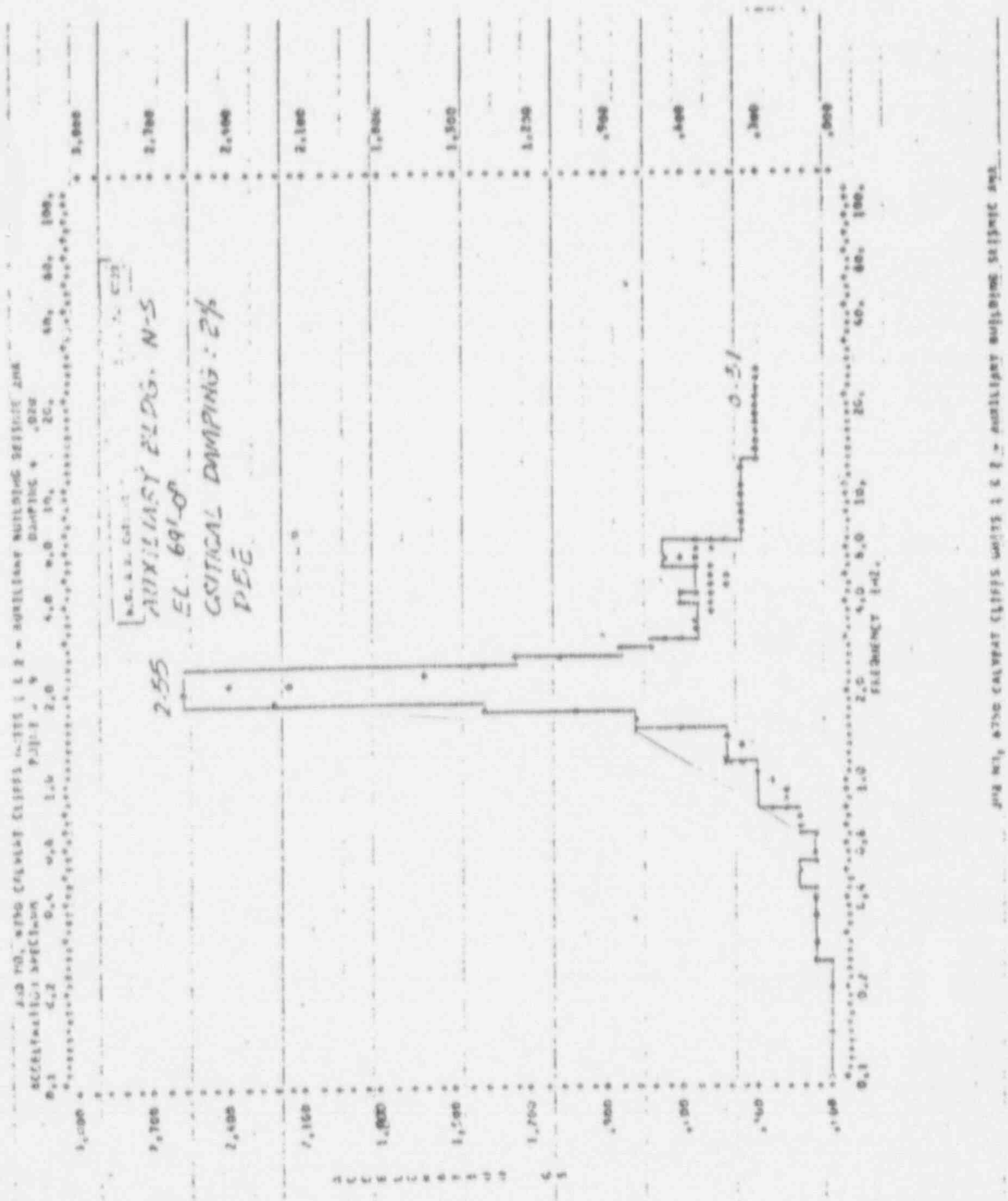
IN-STRUCTURE RESPONSE SPECTRA



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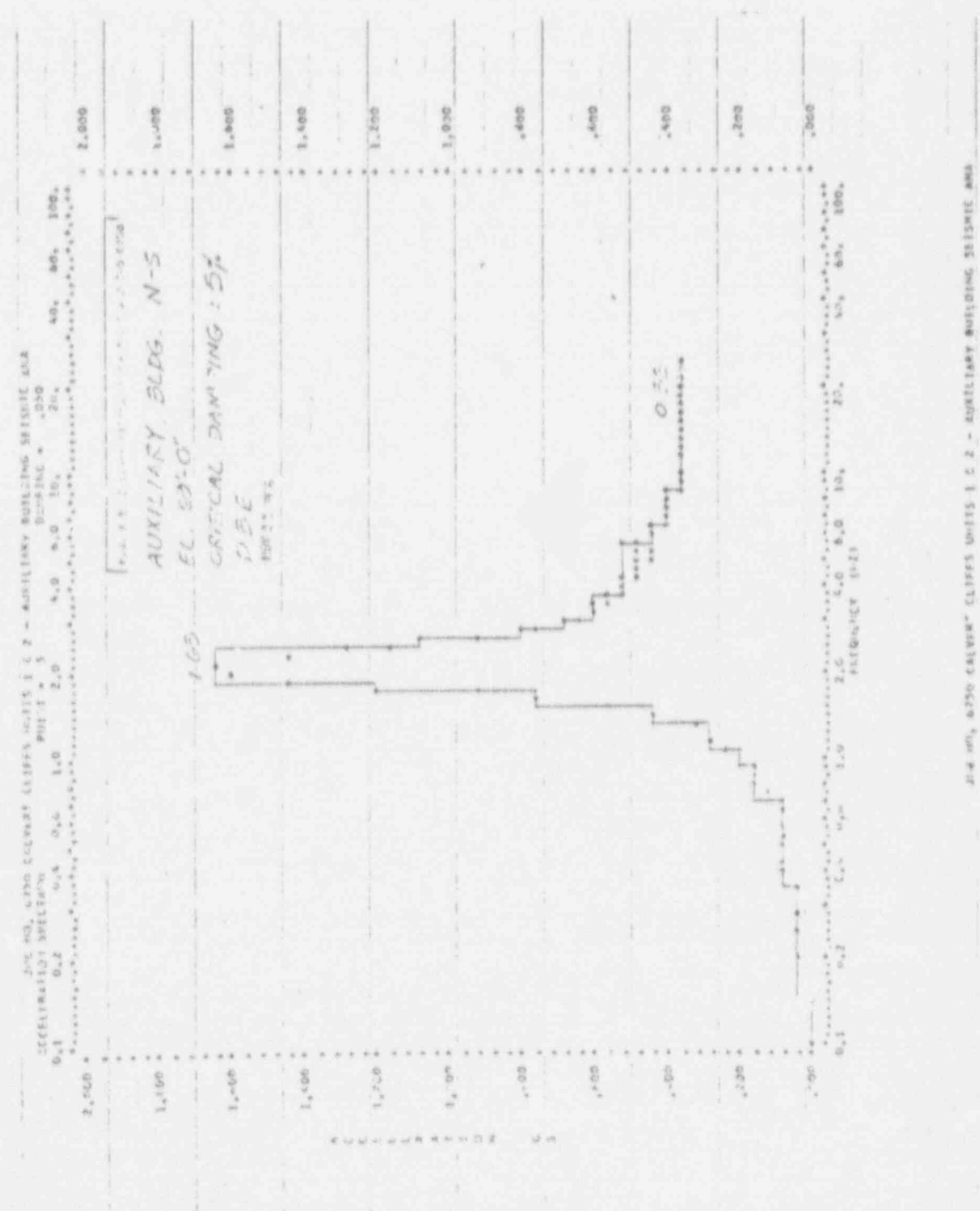
IN-STRUCTURE RESPONSE SPECTRA



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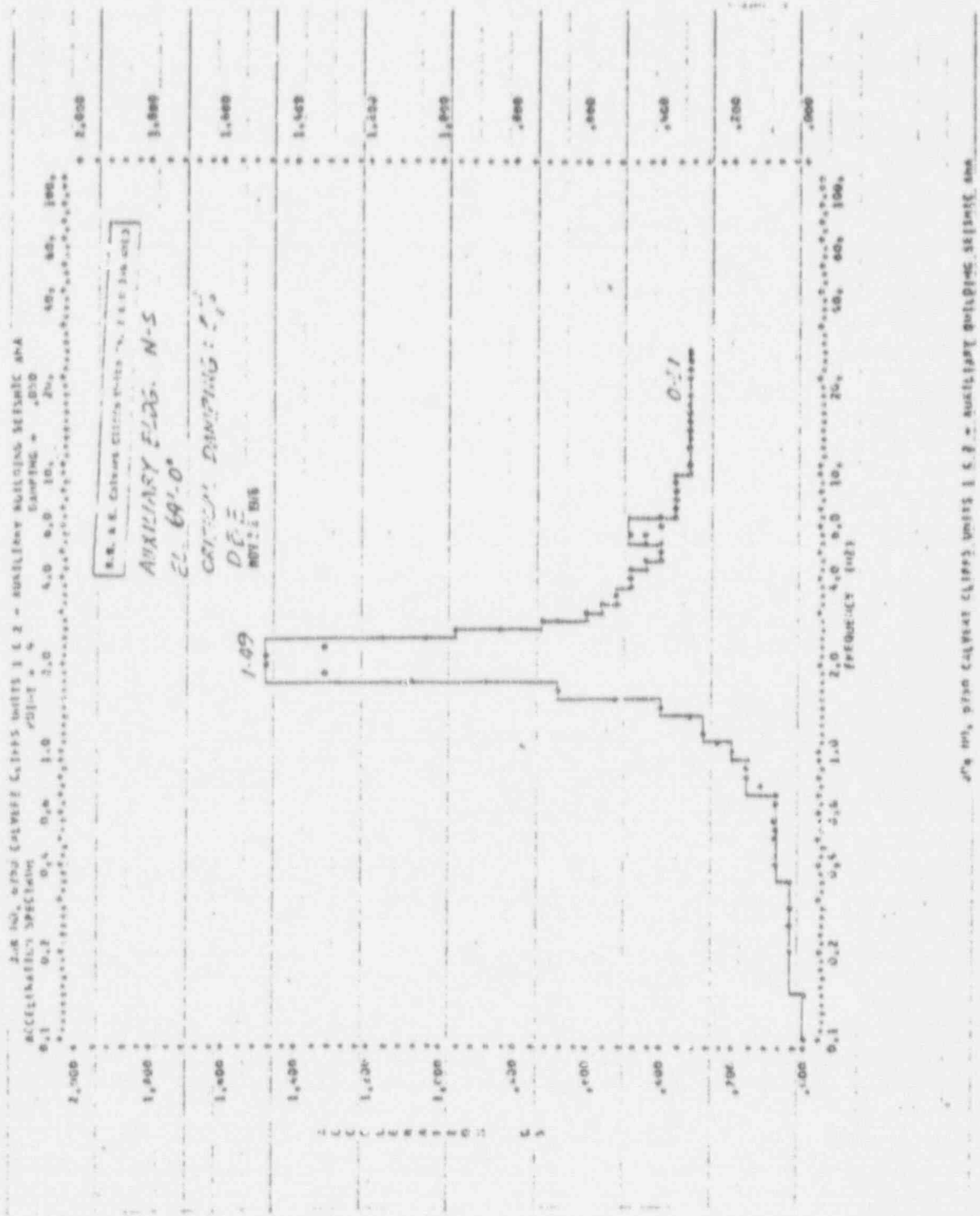

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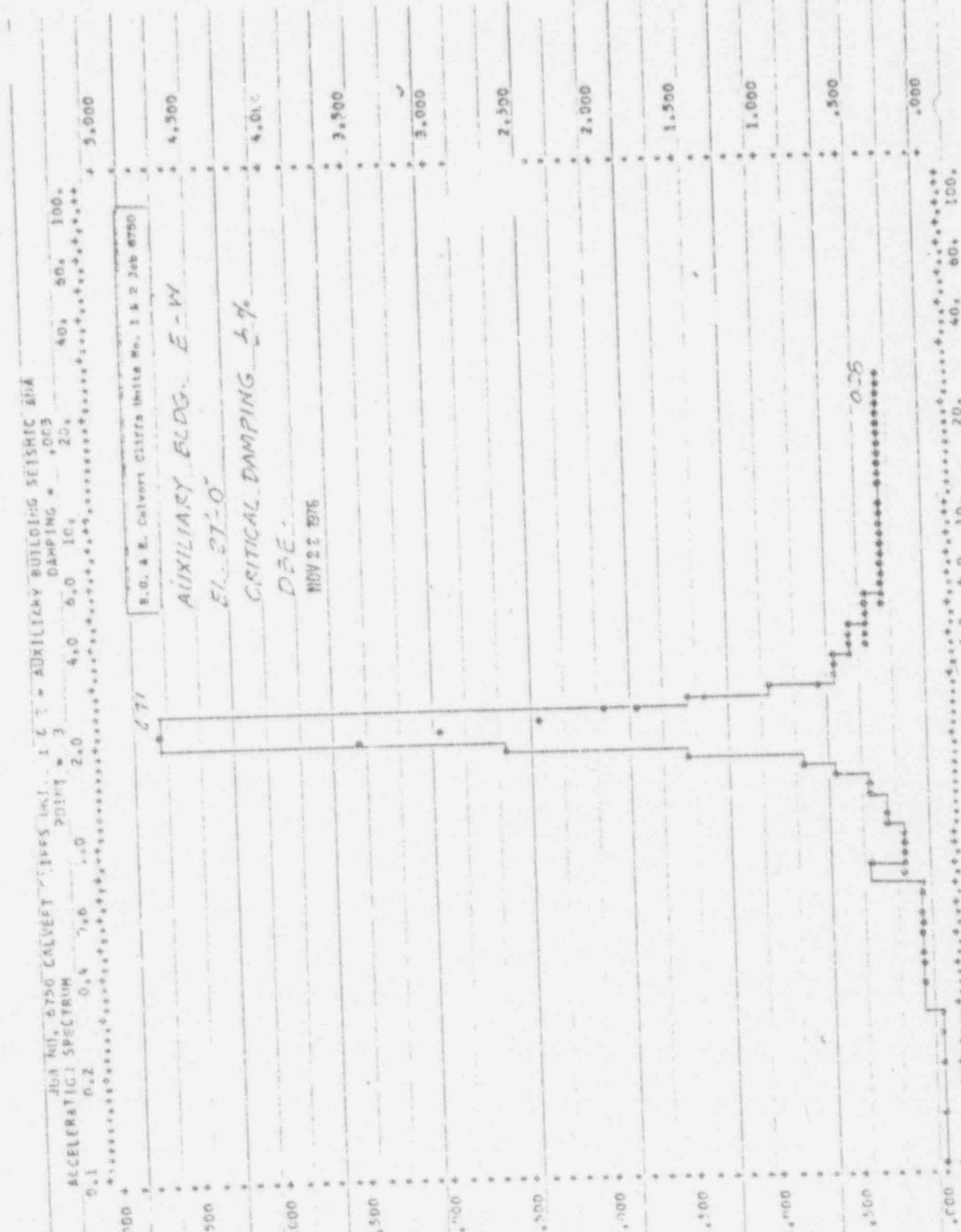

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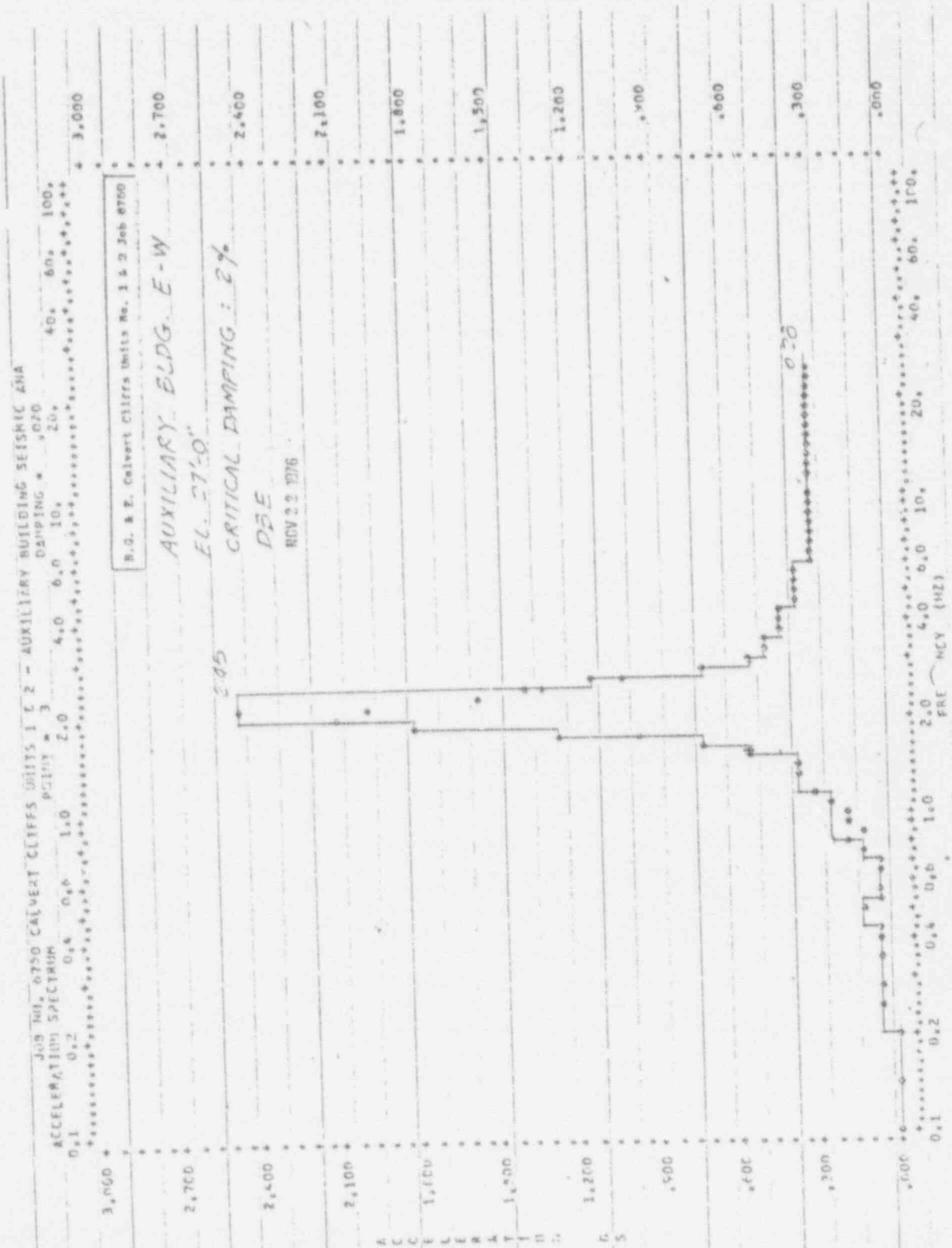

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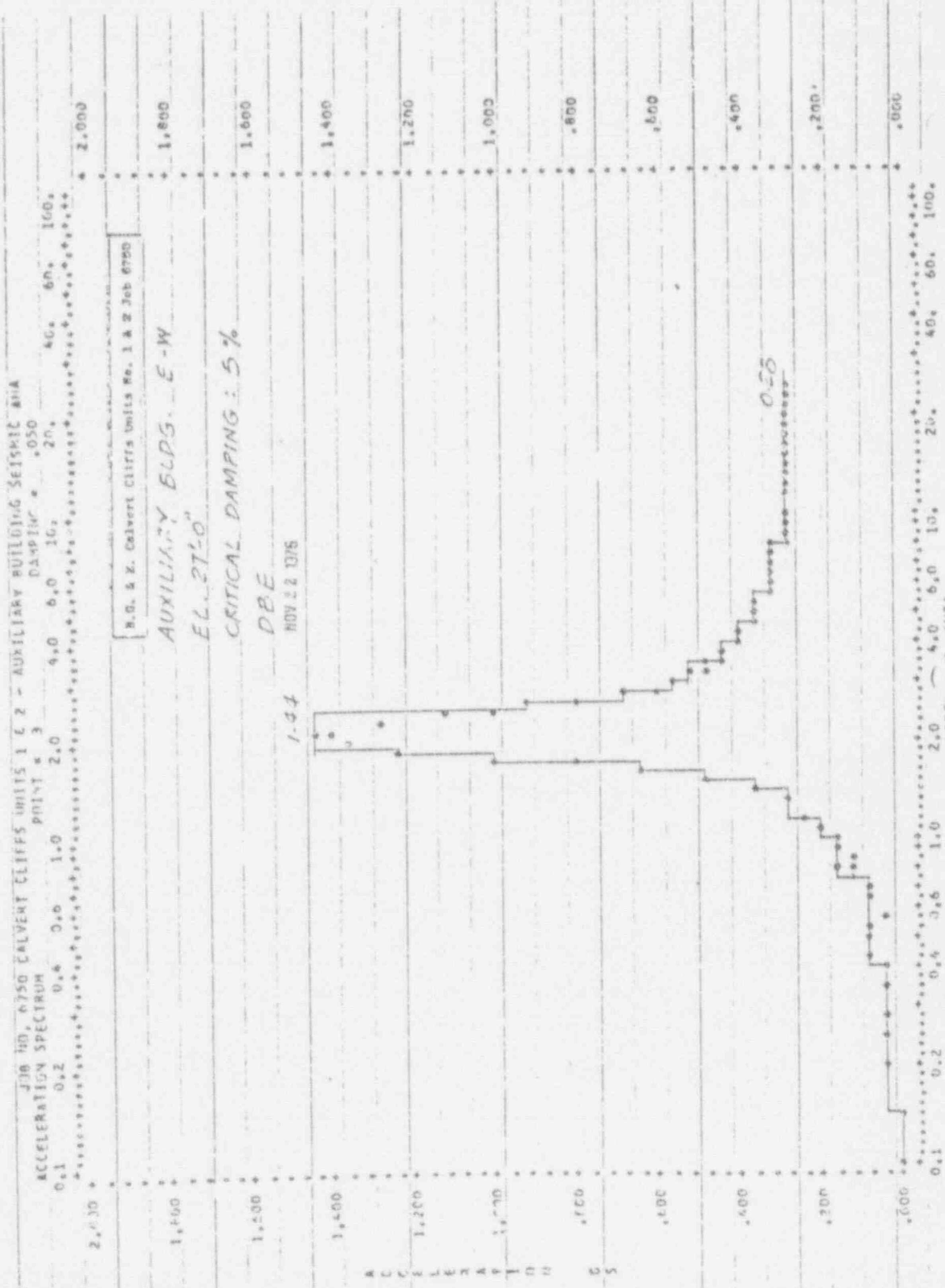

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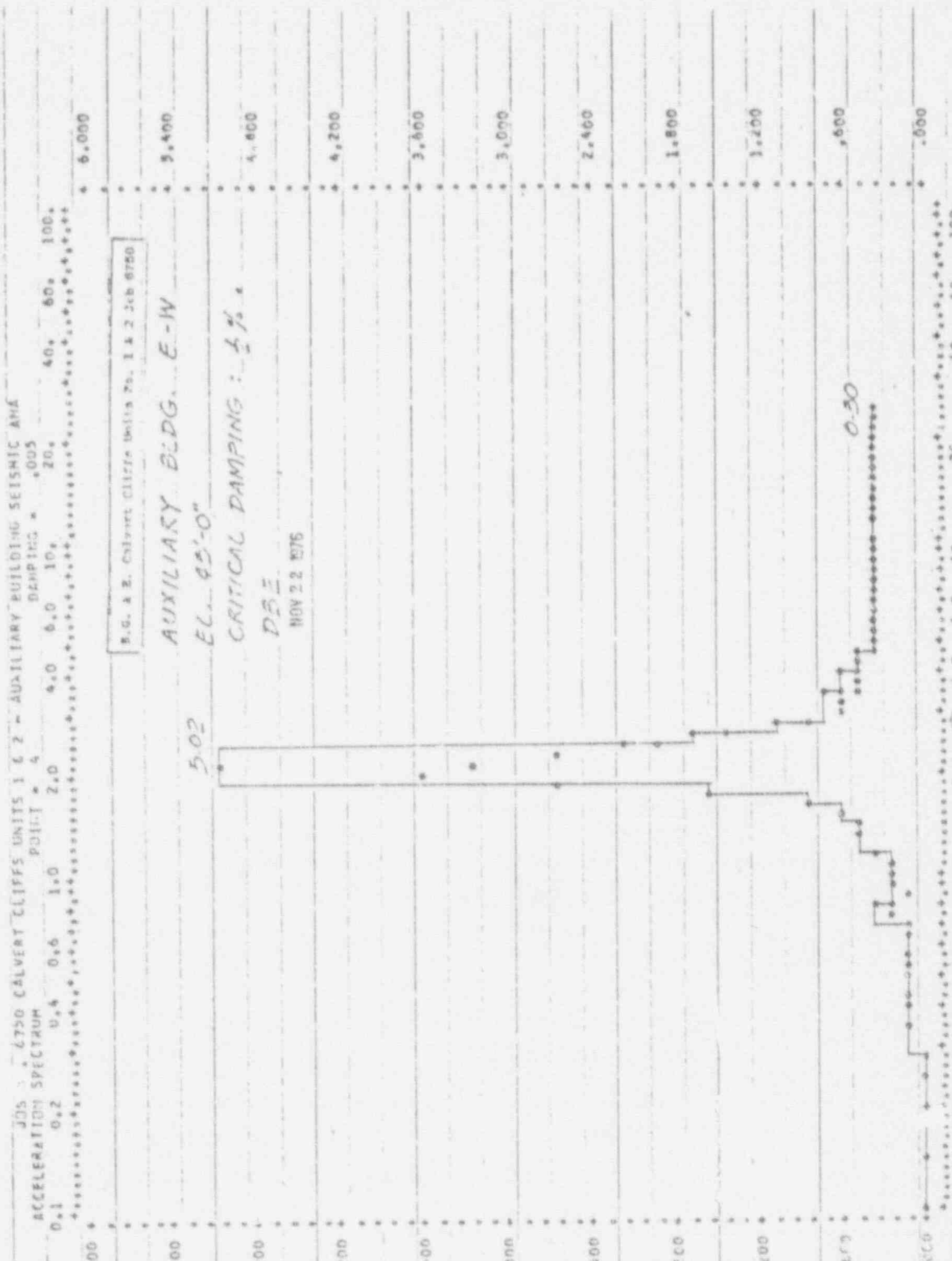
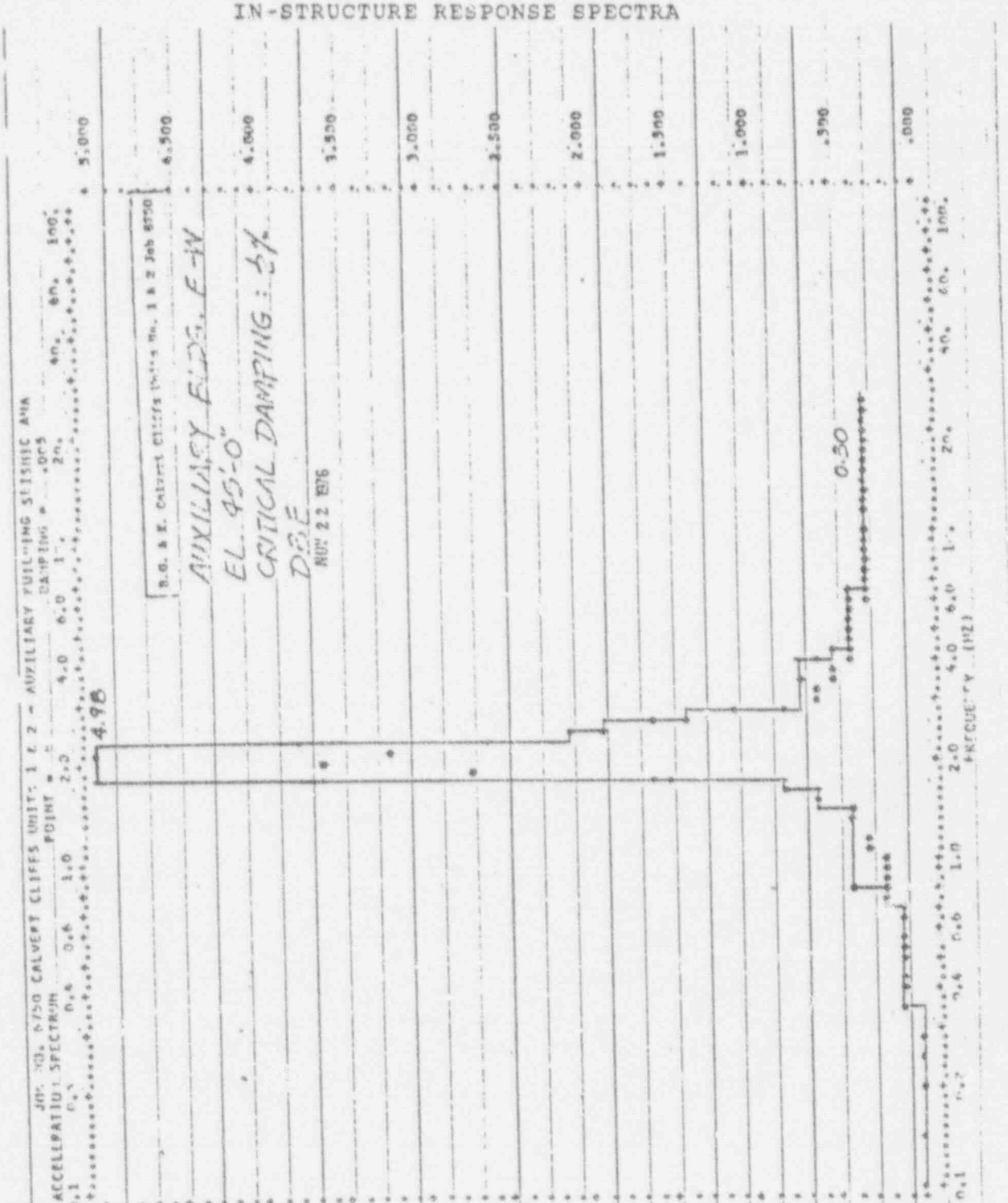

CALVERT CLIFFS UNITS 1 & 2 - AUXILIARY BUILDING SEISMIC ANA
ACCELERATION SPECTRUM POINT = 4 DAMPING = .005
AUXILIARY BLDG. E-W
EL. 45'-0"
CRITICAL DAMPING: 1/2%
DBE
NOV 22 1976
5.02
0.30
FREQ. CY (HZ)
ACCELERATION G'S
0.1 0.2 0.4 0.6 1.0 2.0 4.0 6.0 10. 20. 40. 60. 100.
6.000
5.400
4.800
4.200
3.600
3.000
2.400
1.800
1.200
.600
.000

APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



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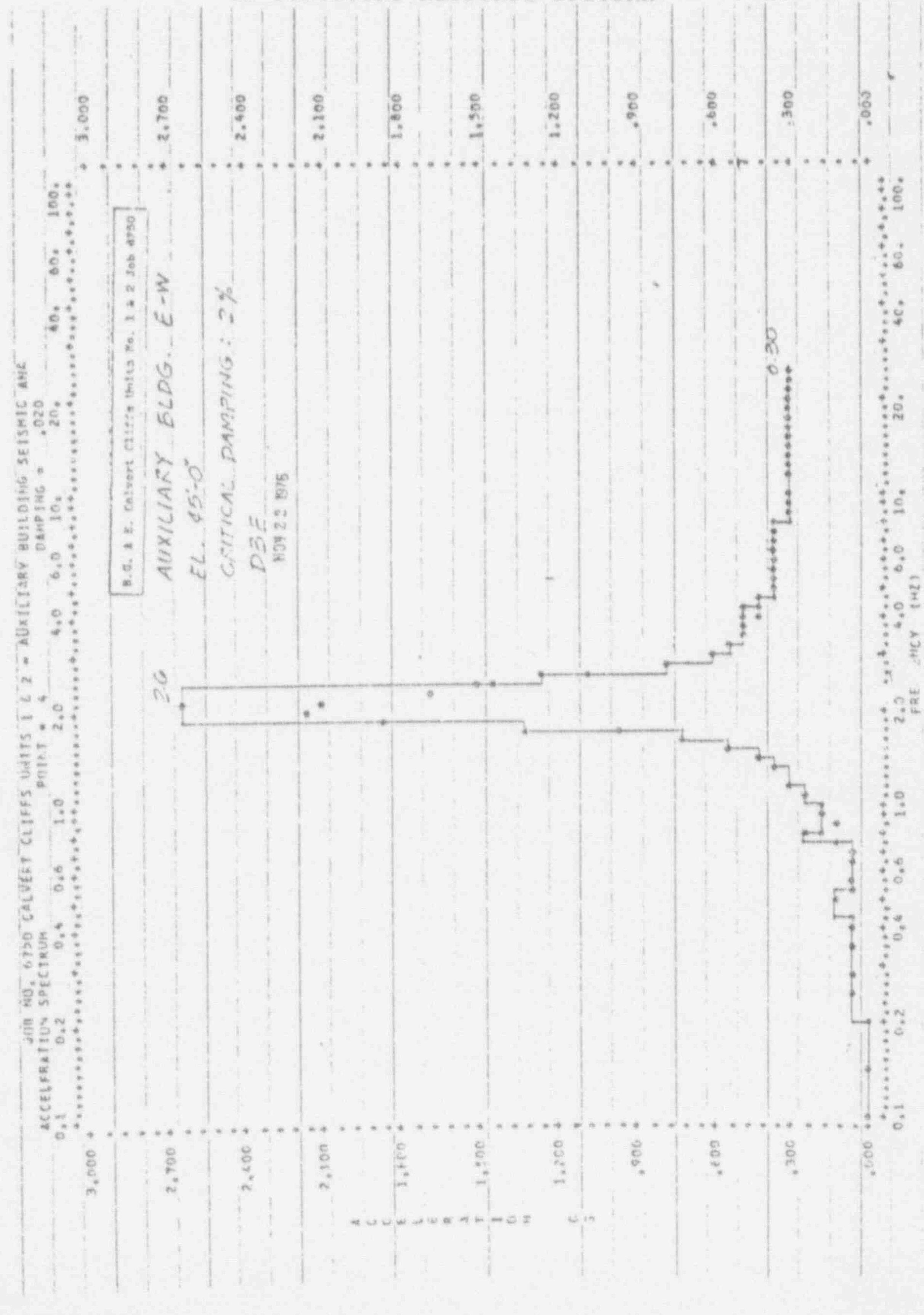

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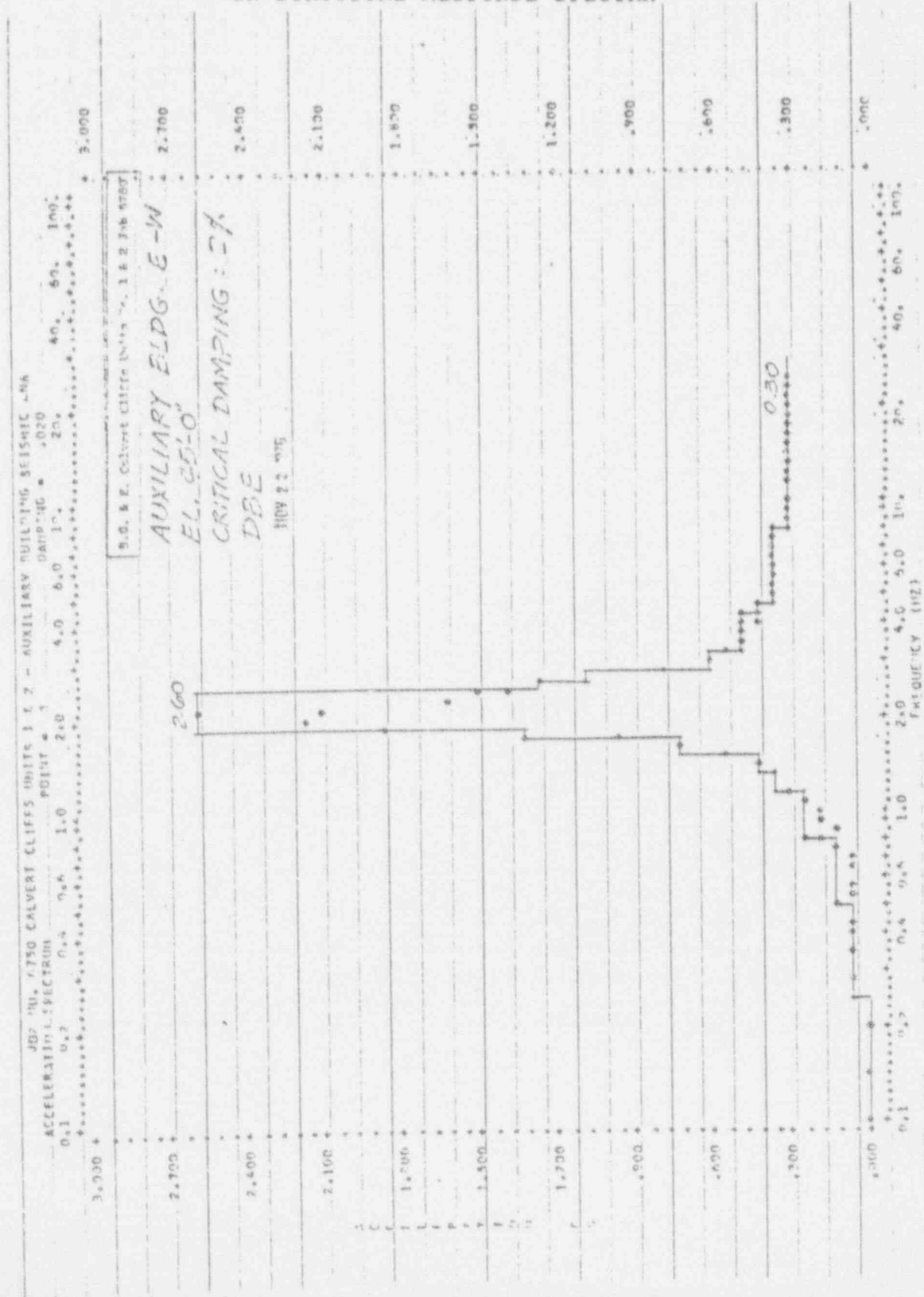

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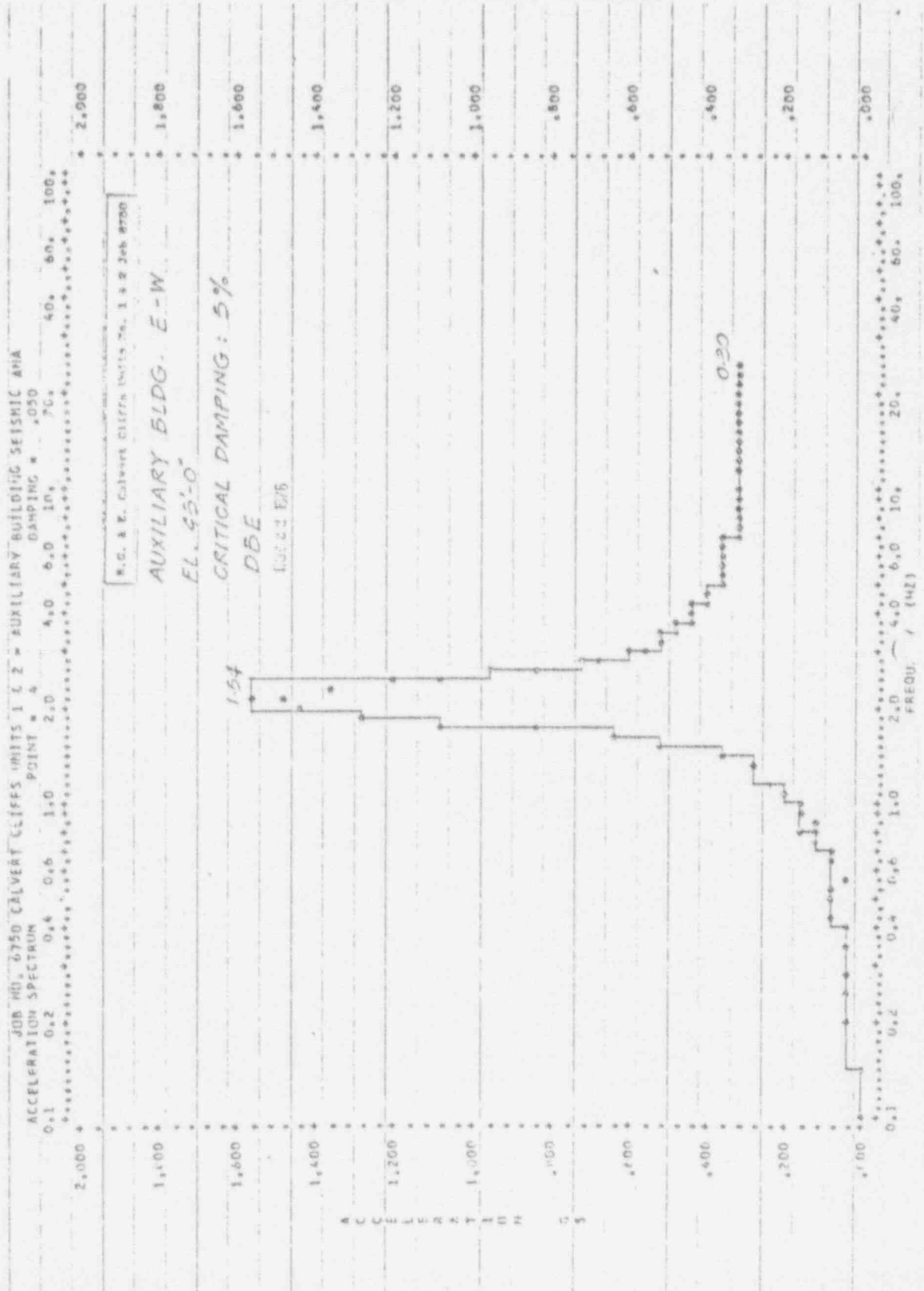

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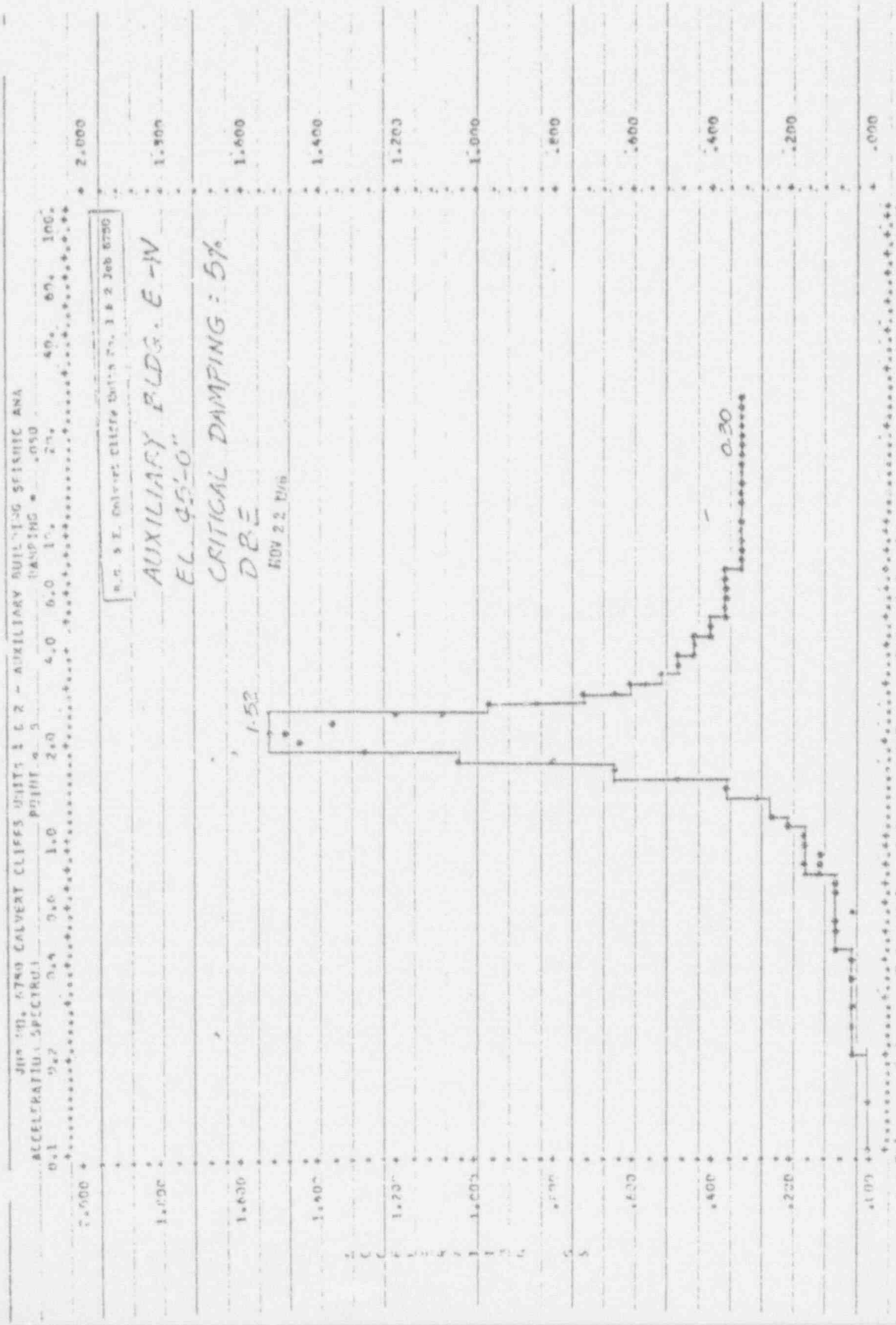

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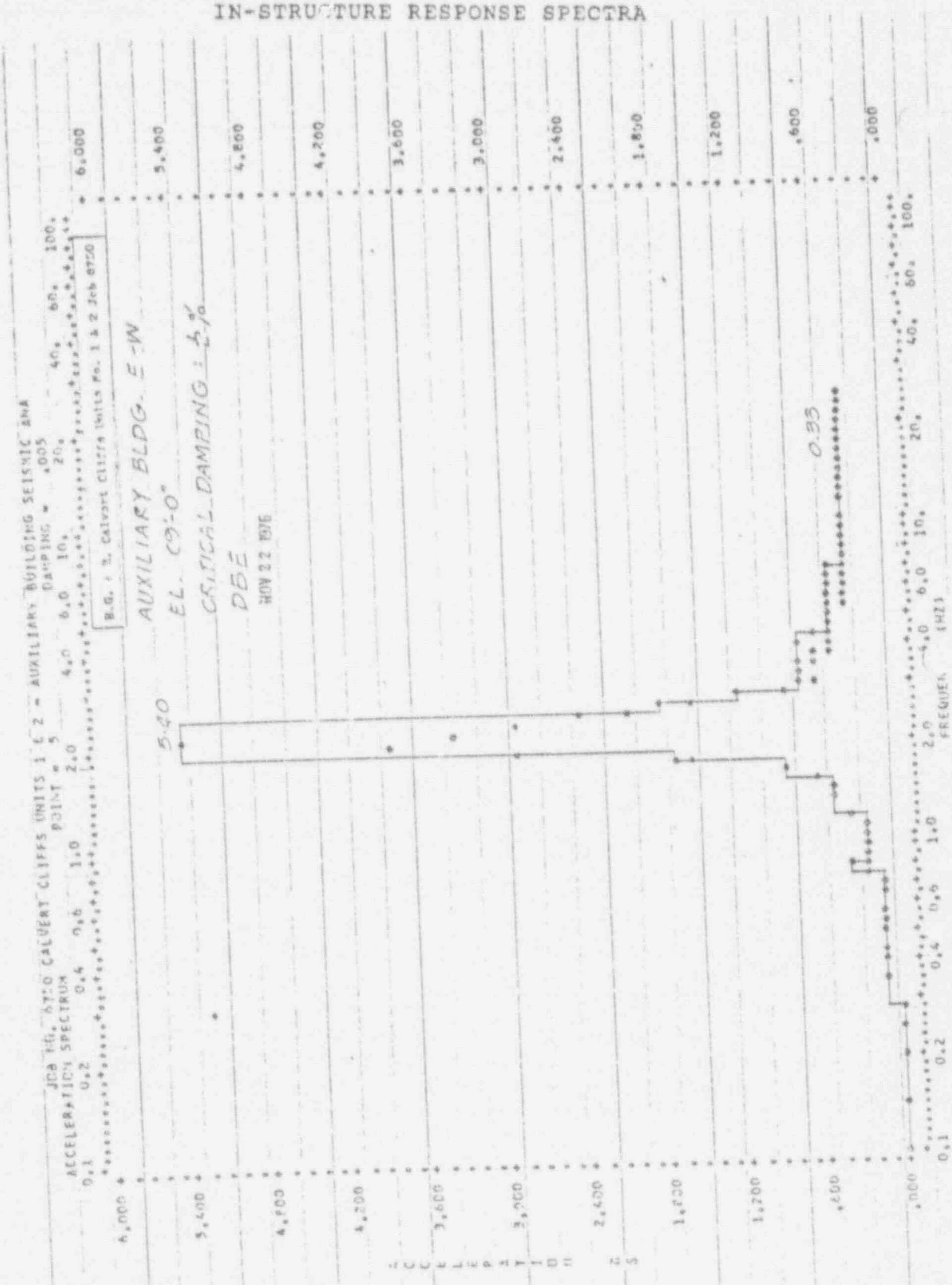

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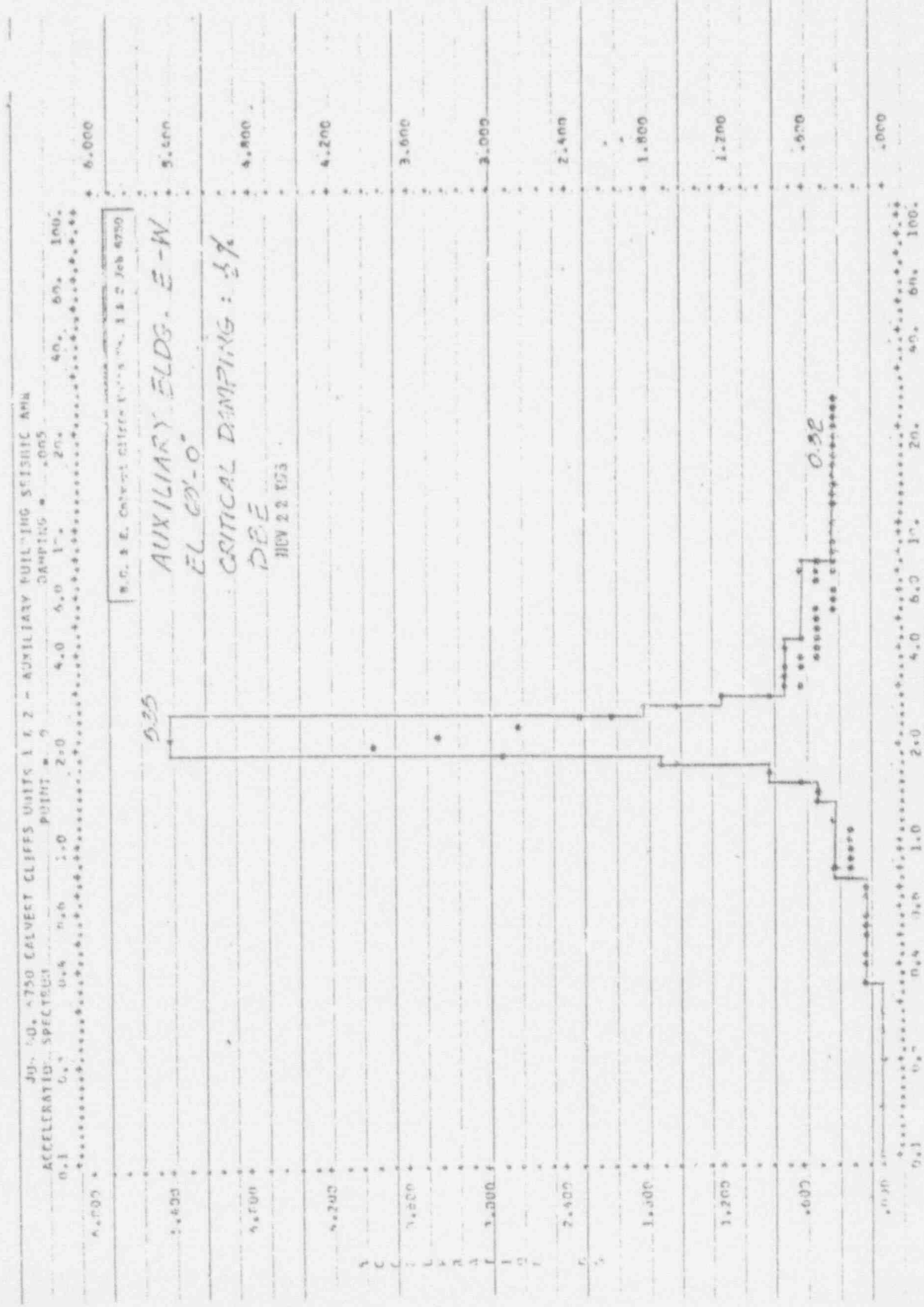

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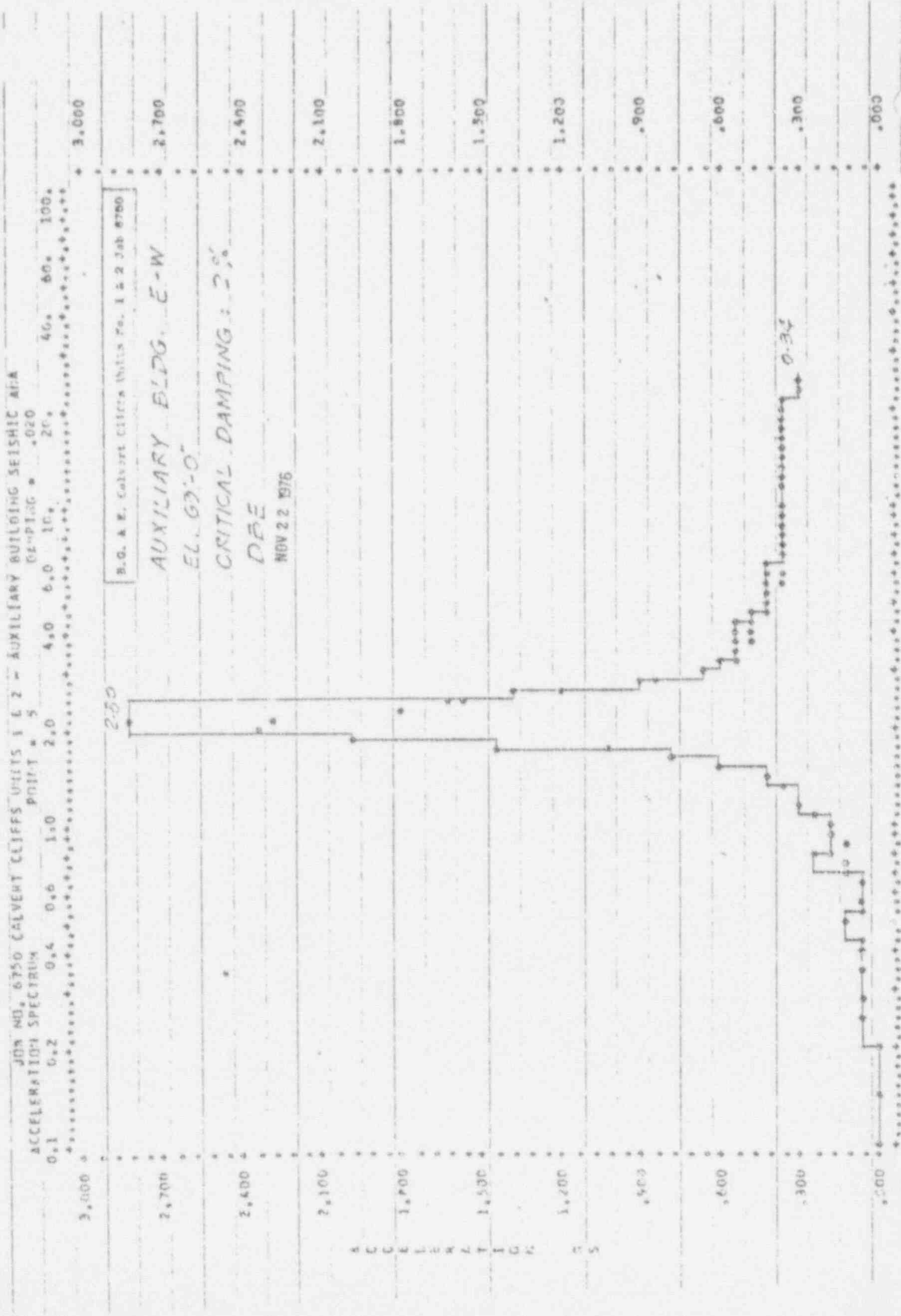

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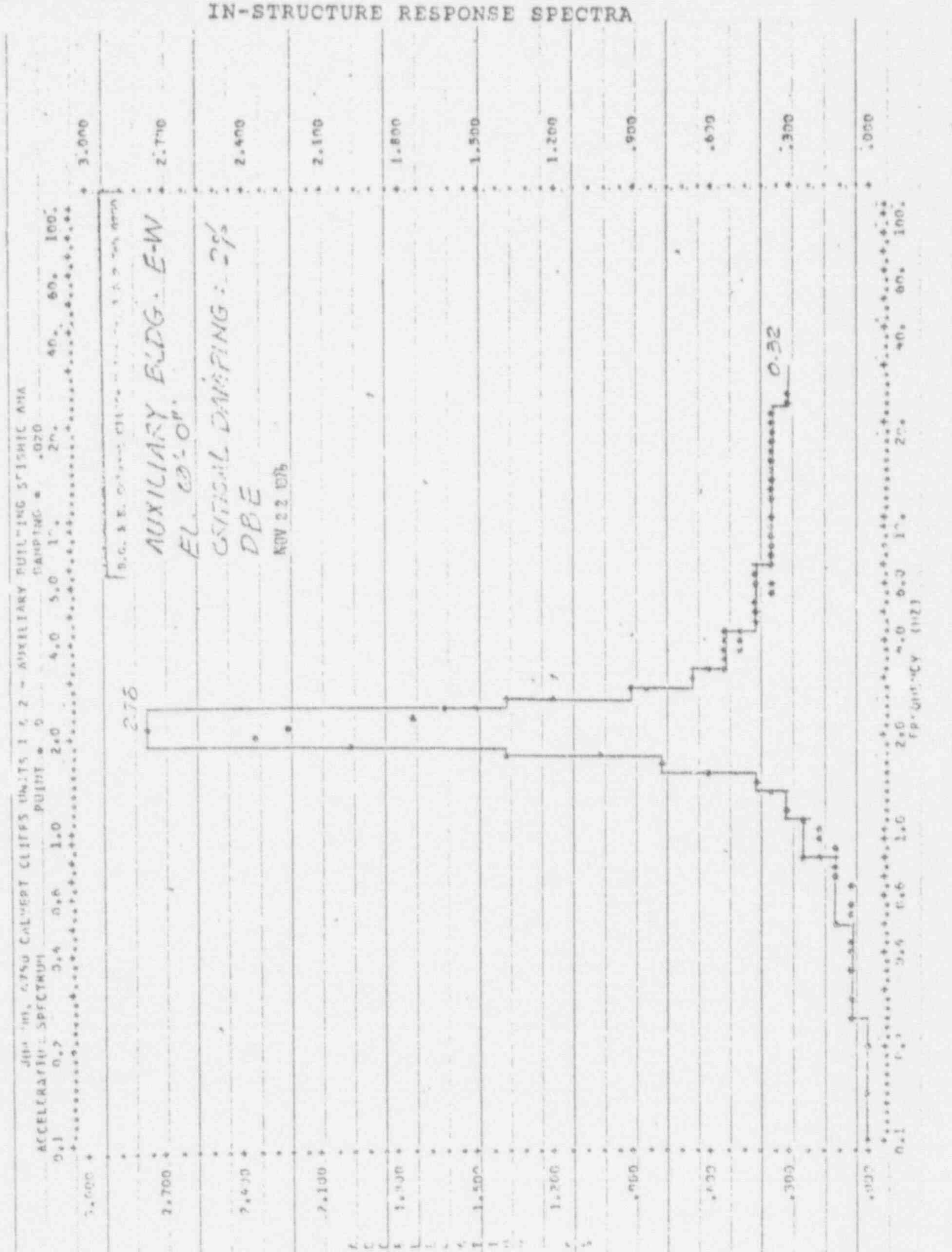

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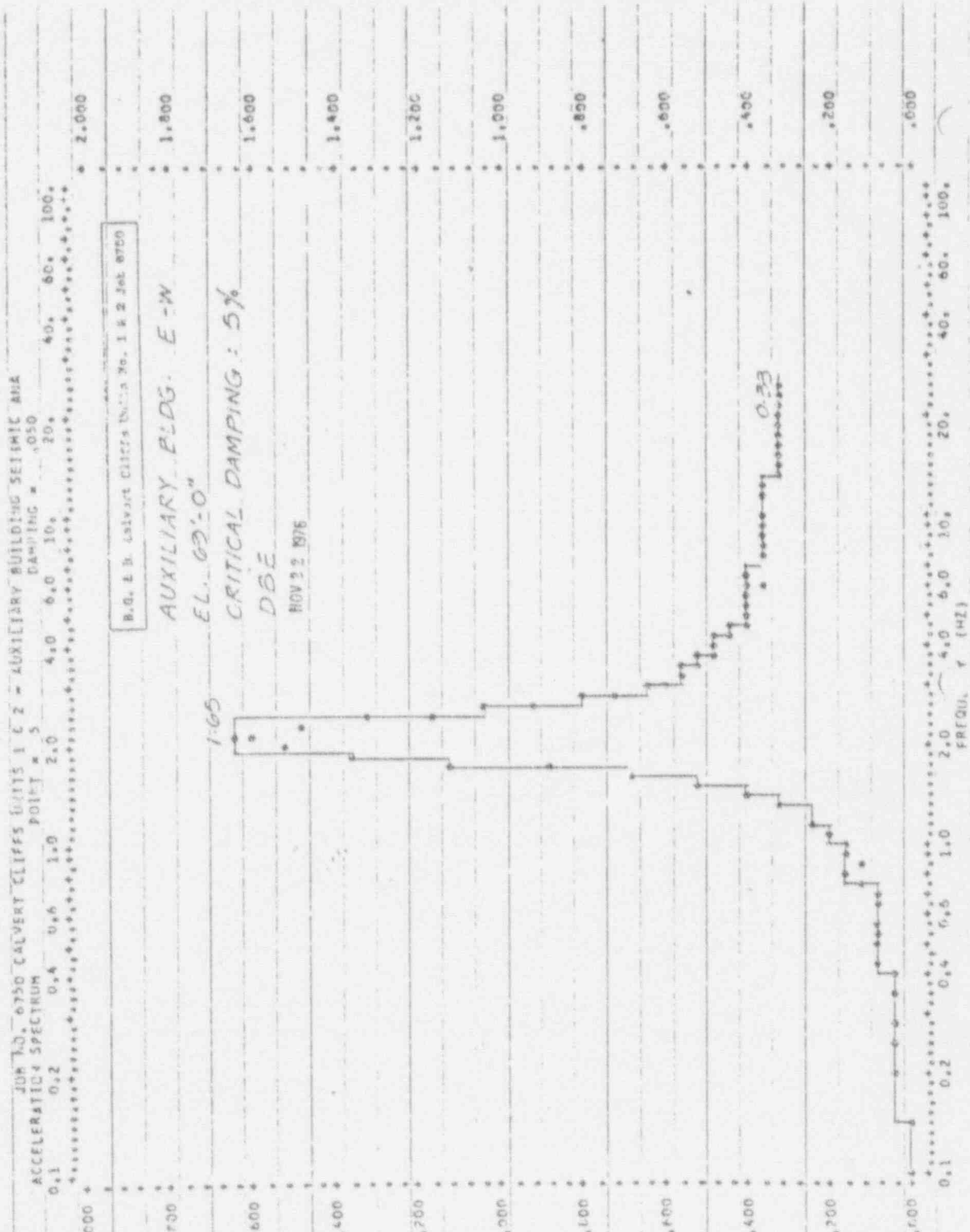

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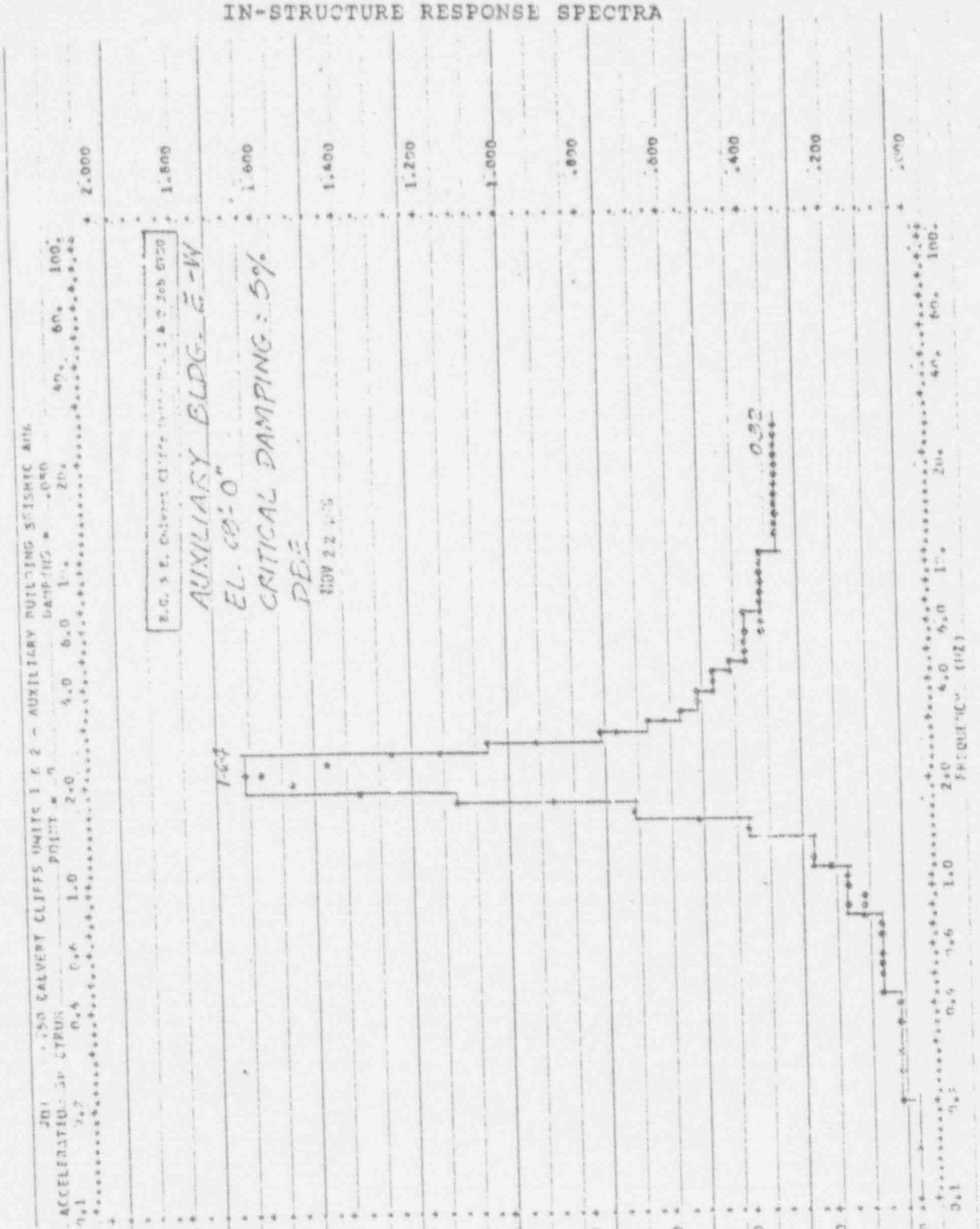


APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA

TABLE A-D

INTAKE STRUCTURE

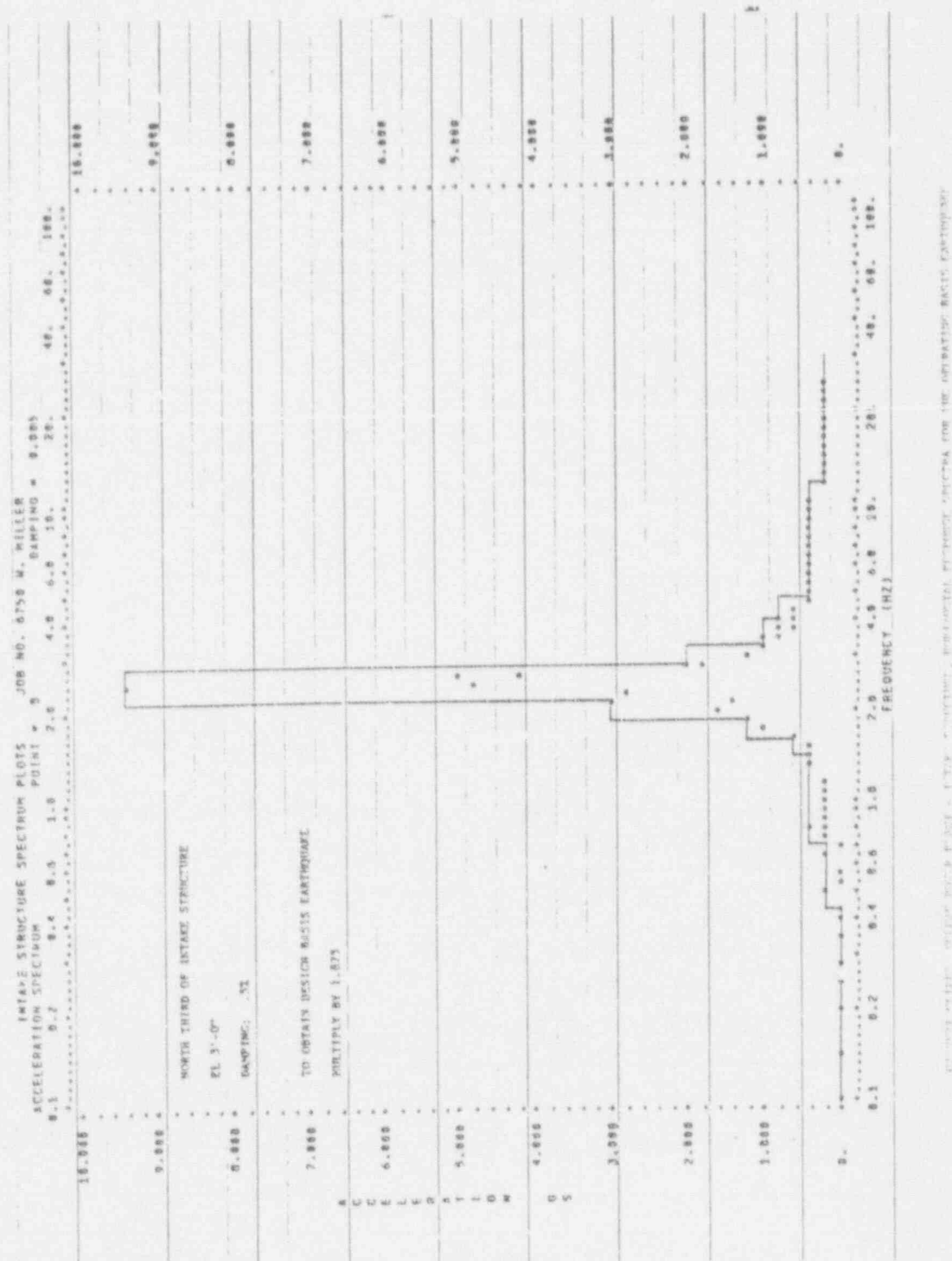
FLOOR ACCELERATIONS AND PEAK FLOOR RESPONSE ACCELERATIONS
for
Operating Basis Earthquake (OBE)
and
Design Basis Earthquake (DBE)

ELEVATION	PEAK FLOOR RESPONSE ACCELERATIONS[6] for component damping of 1/2 %	1 %	2.5 %	FLOOR ACCELERATION [ZPA]
(ft.)	(g's)	(g's)	(g's)	(g's)
Horizontal East-West Motion				
(Values for North-Third, Middle-Third and South-Third of Intake Structure)				
3'-0" (OBE)	9.50	6.20	3.50	0.169
(DBE)	17.80	11.60	6.60	0.317
Vertical Motion				
All (OBE)	0.65	0.50	0.32	0.060
(DBE)	1.22	0.94	0.60	0.113

[6] Acclelerations at Intake Structure Natural Frequencies of 3.05 Hz for East-West Motion and 2.43 Hz for Vertical Motion.

APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



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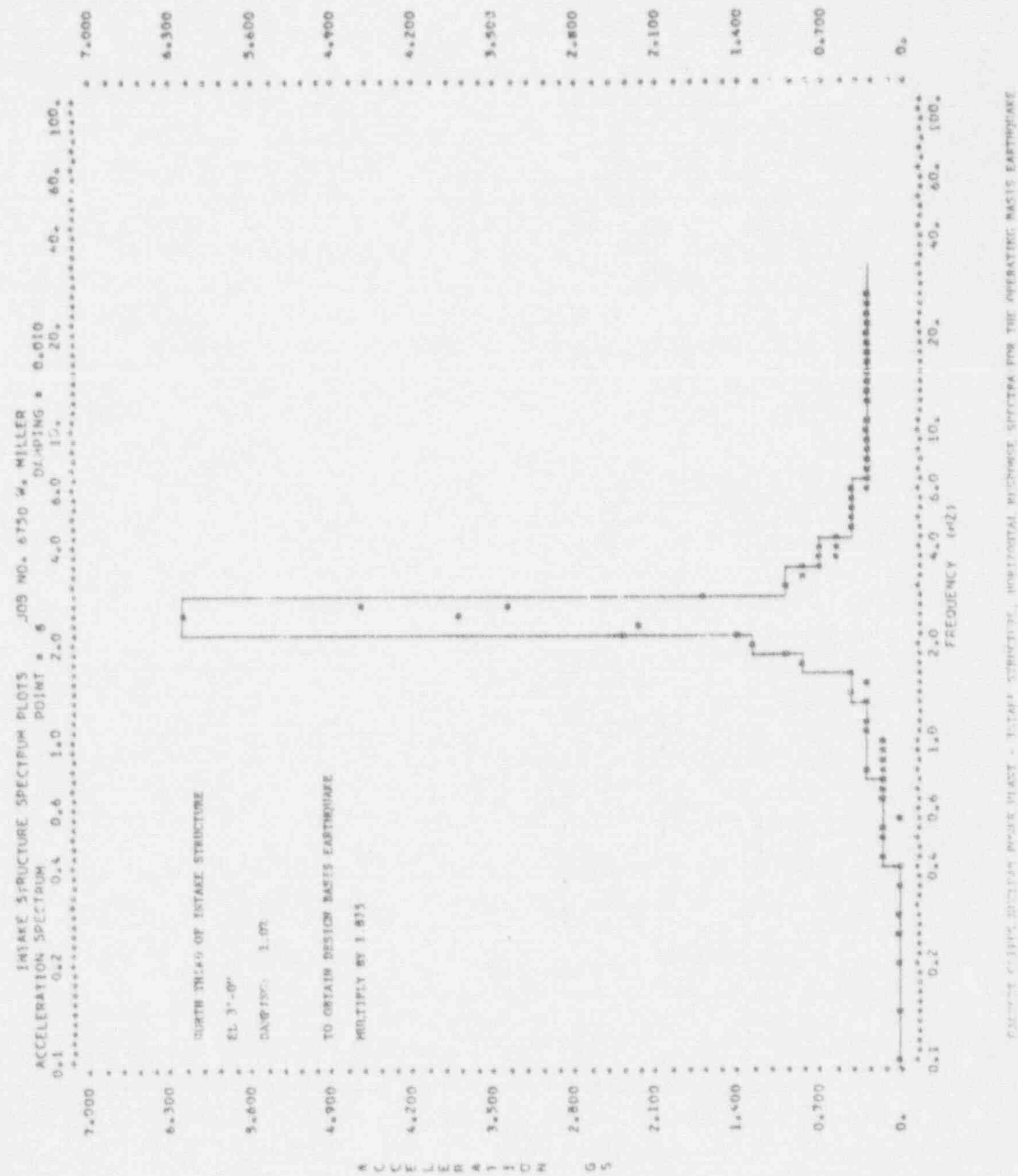
IN-STRUCTURE RESPONSE SPECTRA



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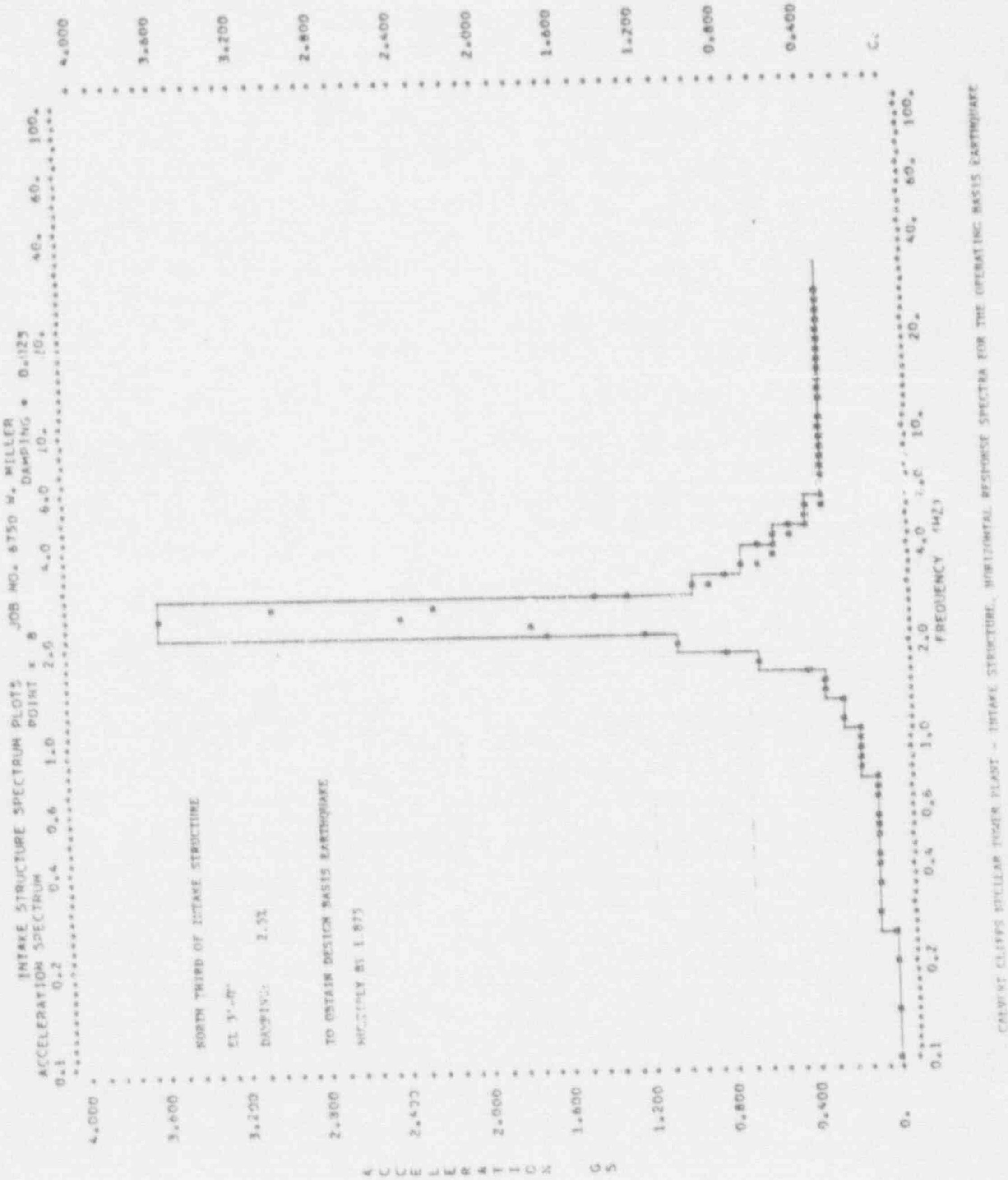

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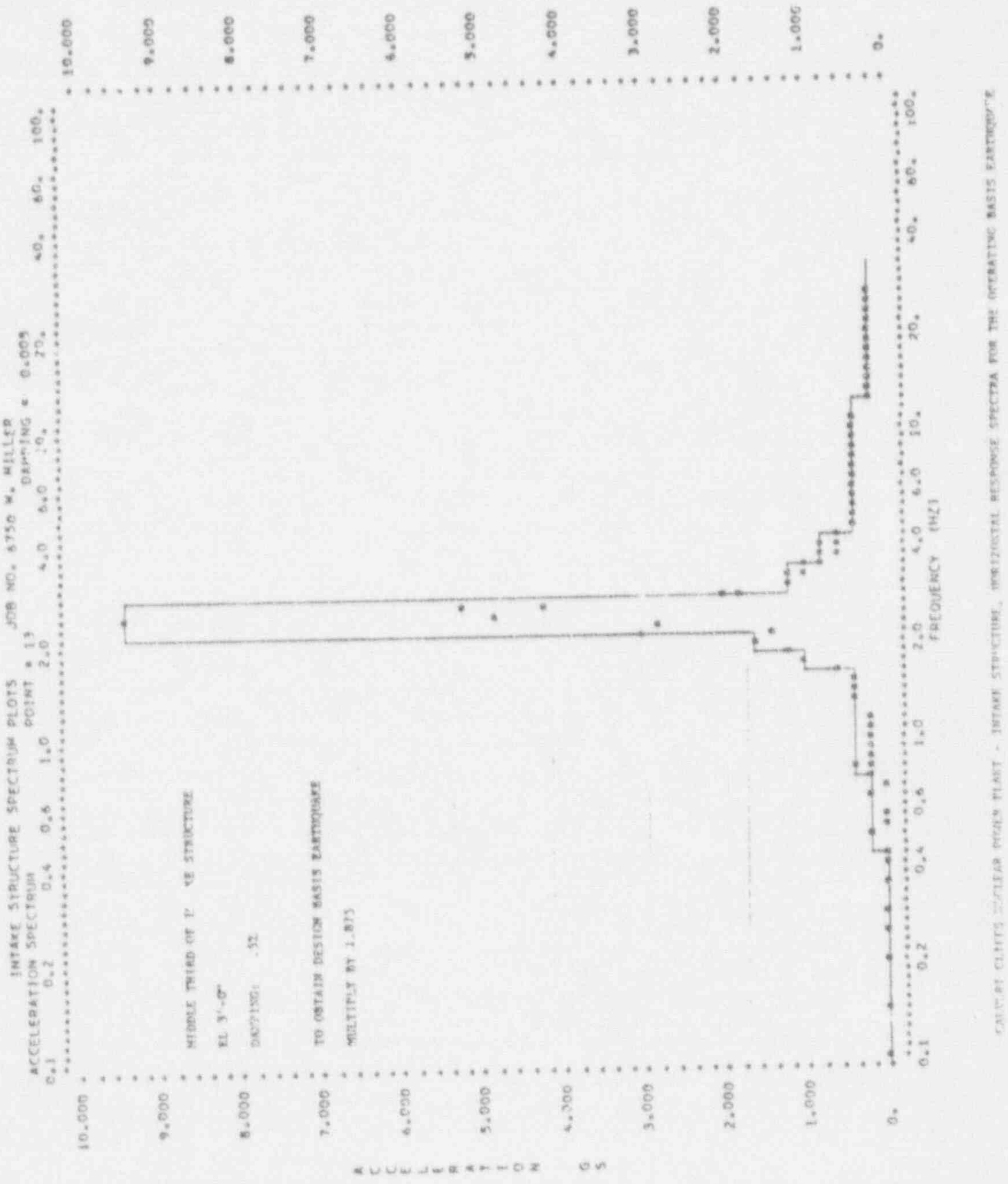

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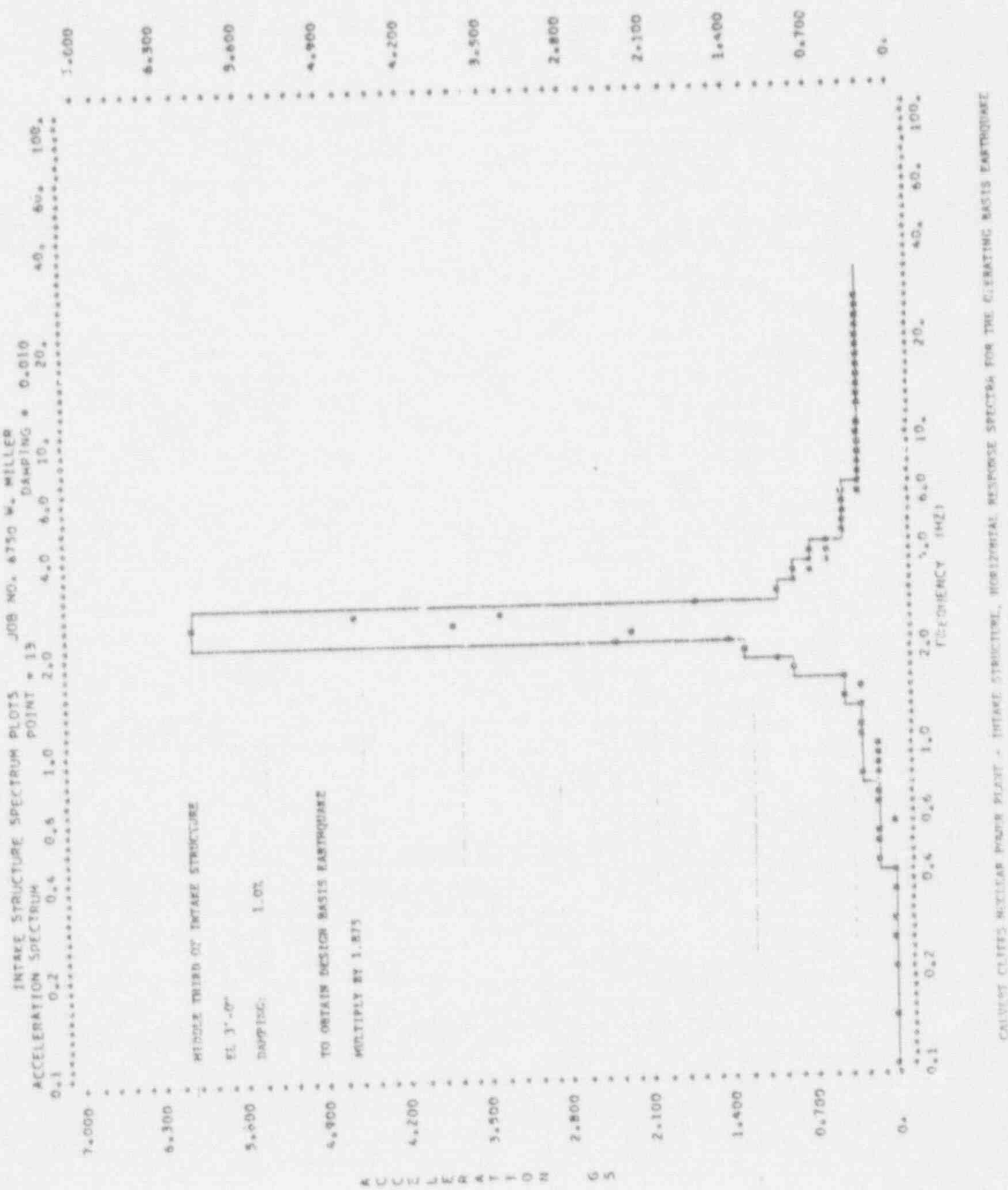
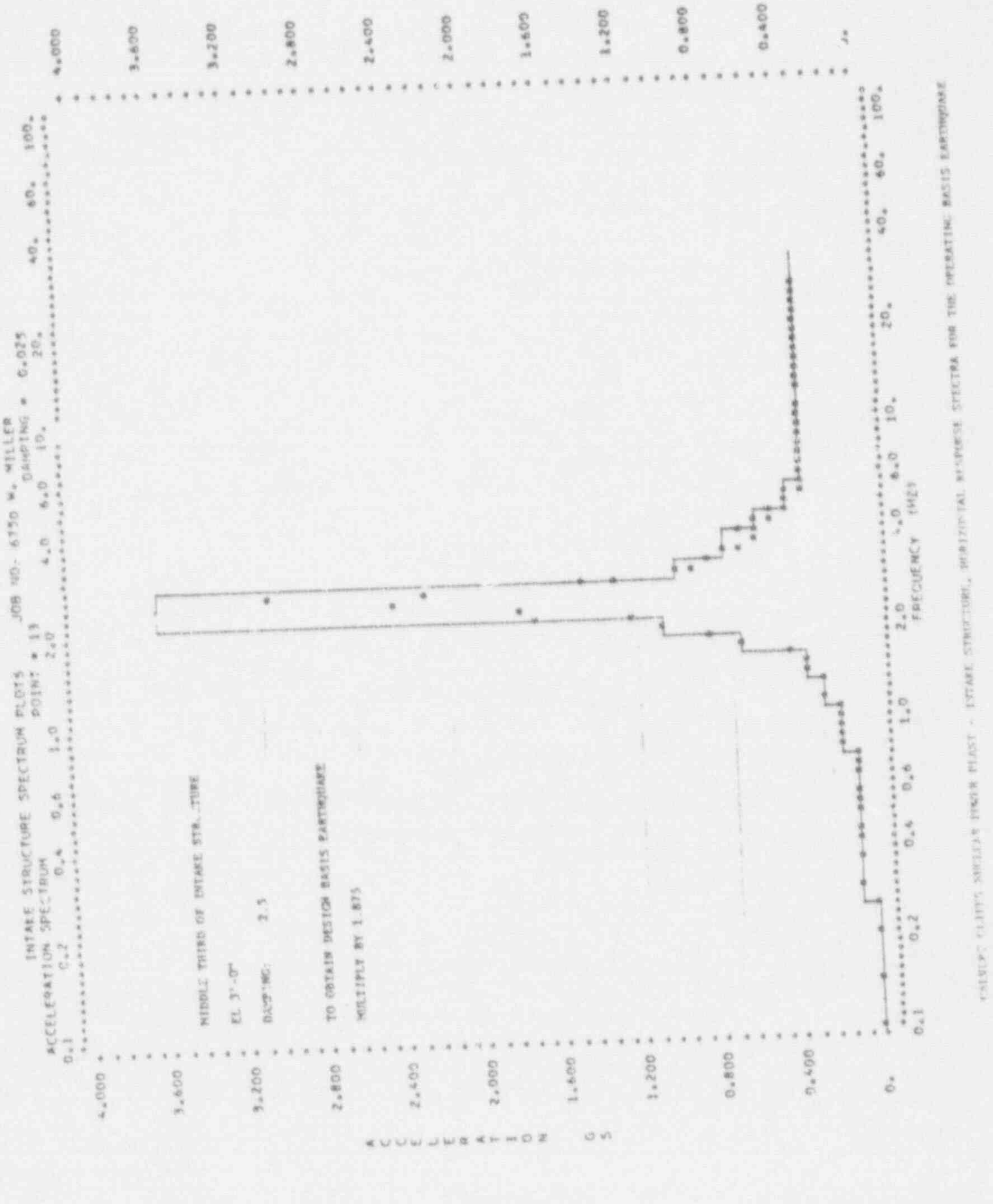


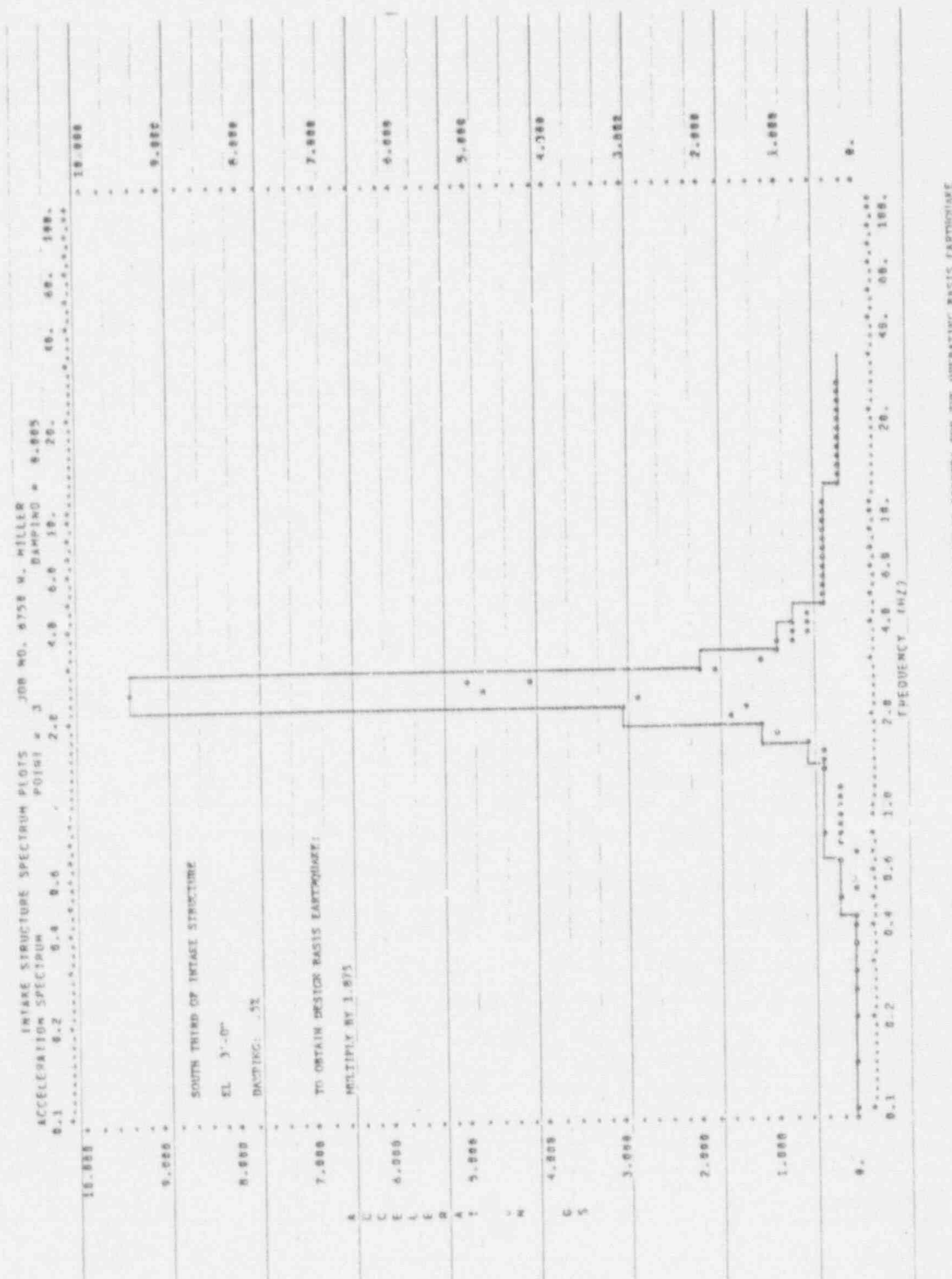
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



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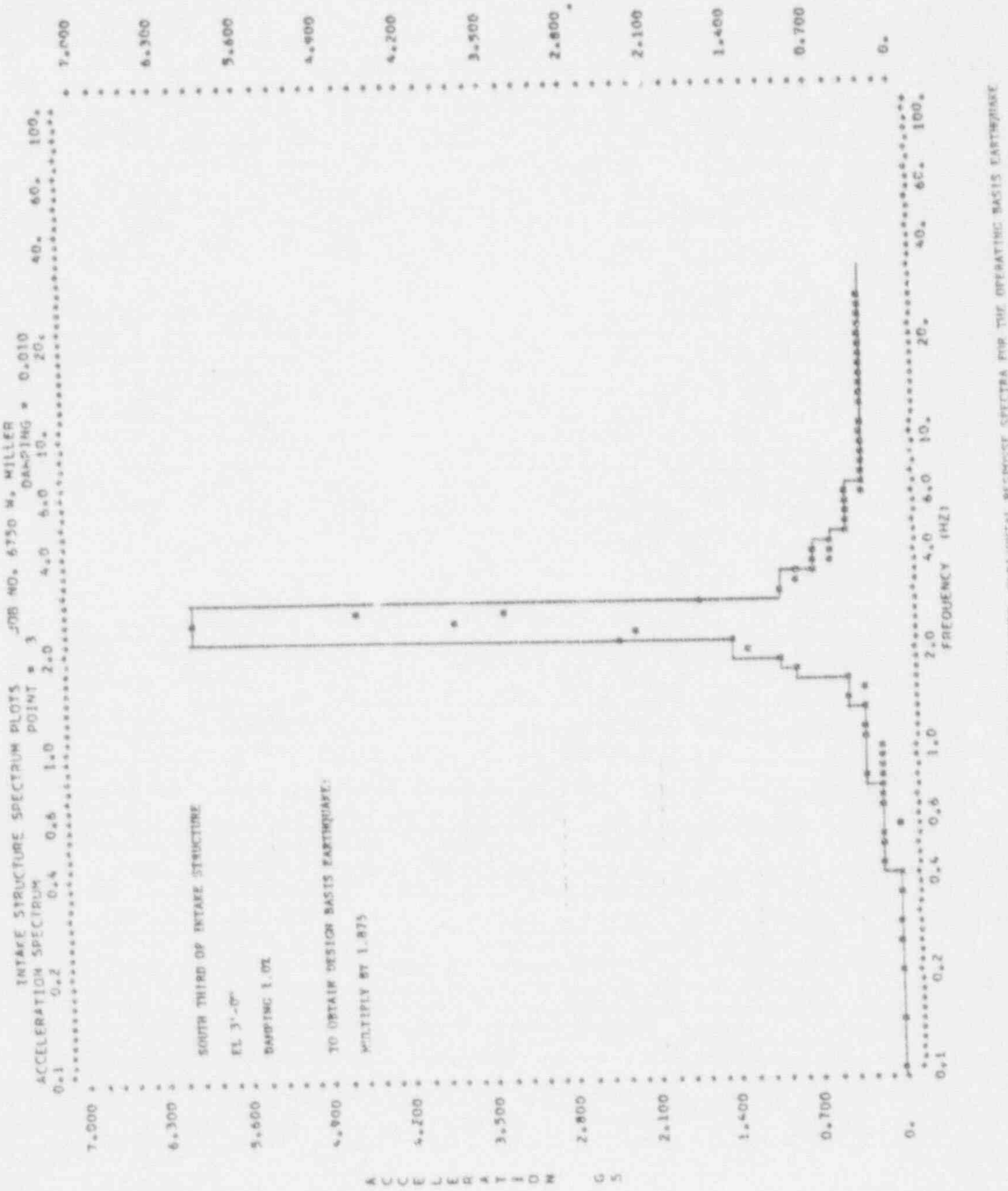
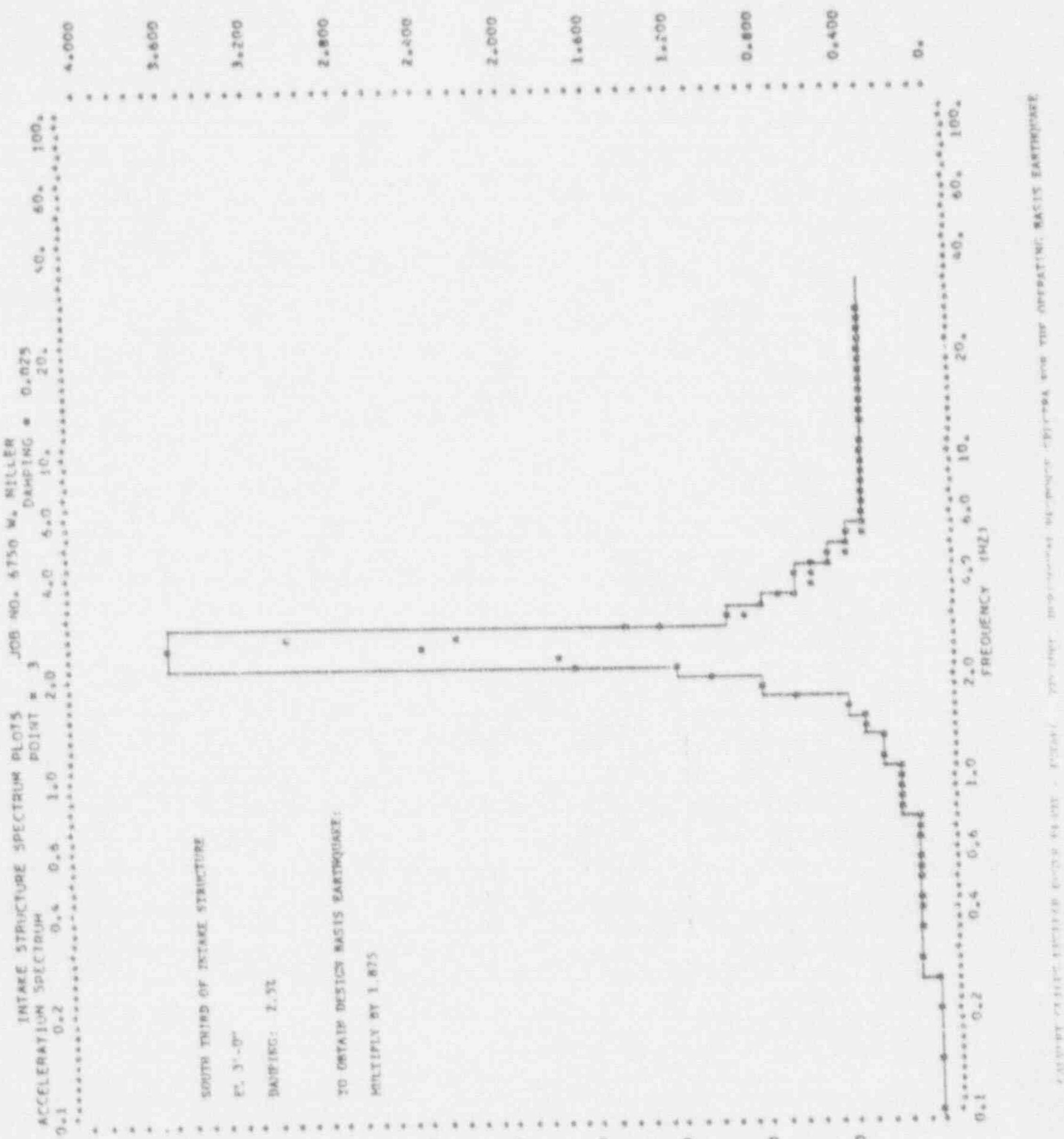


APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



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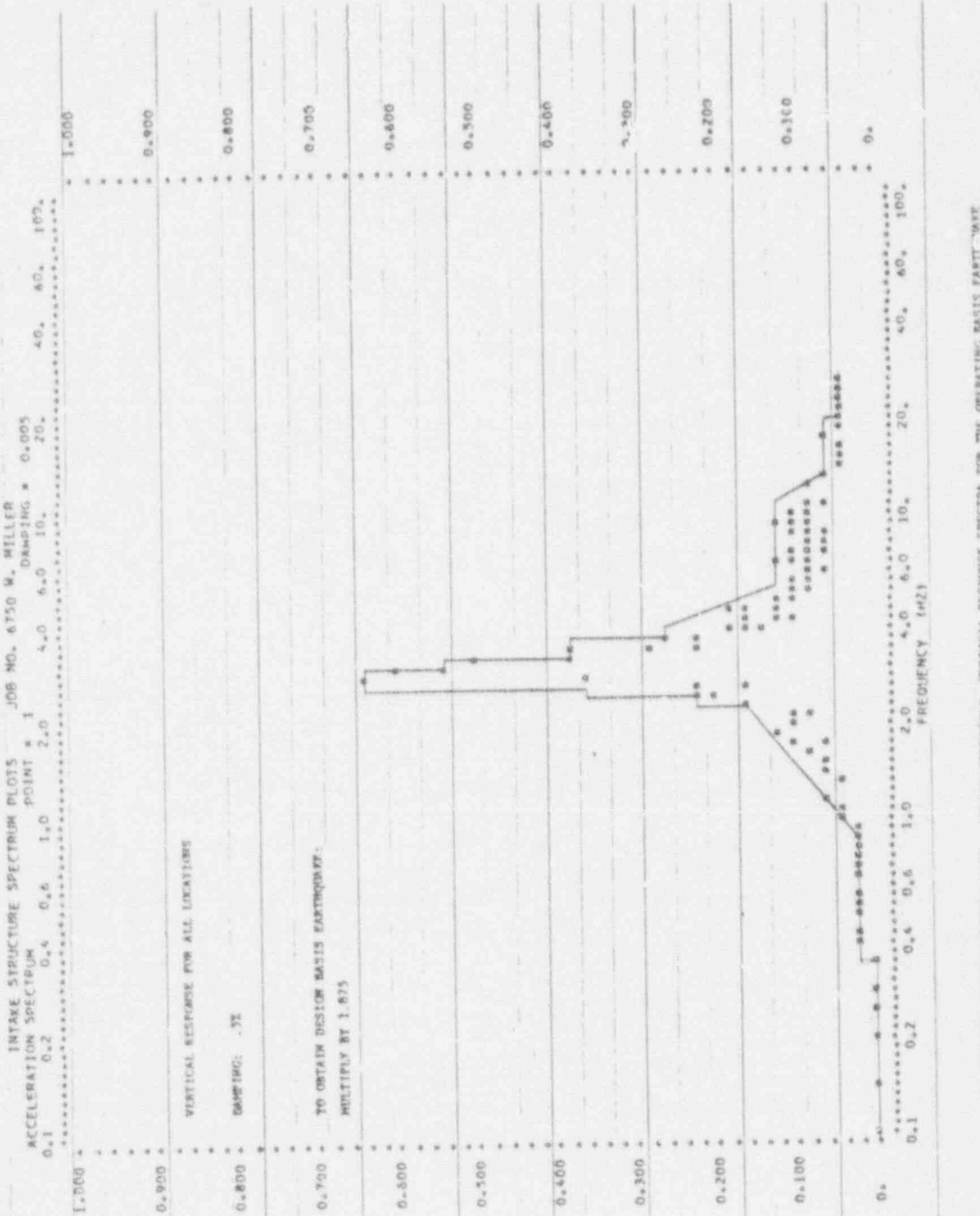
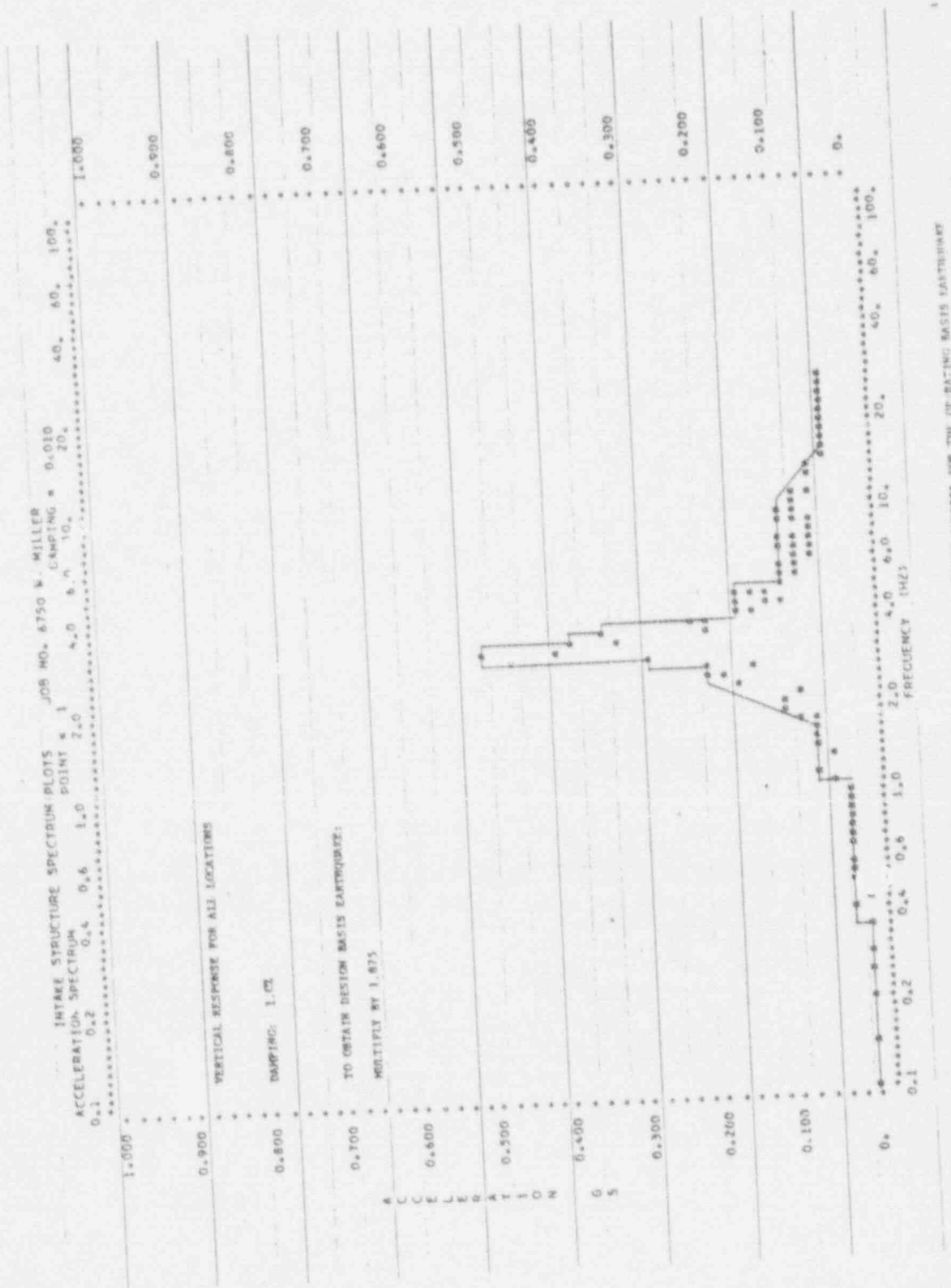


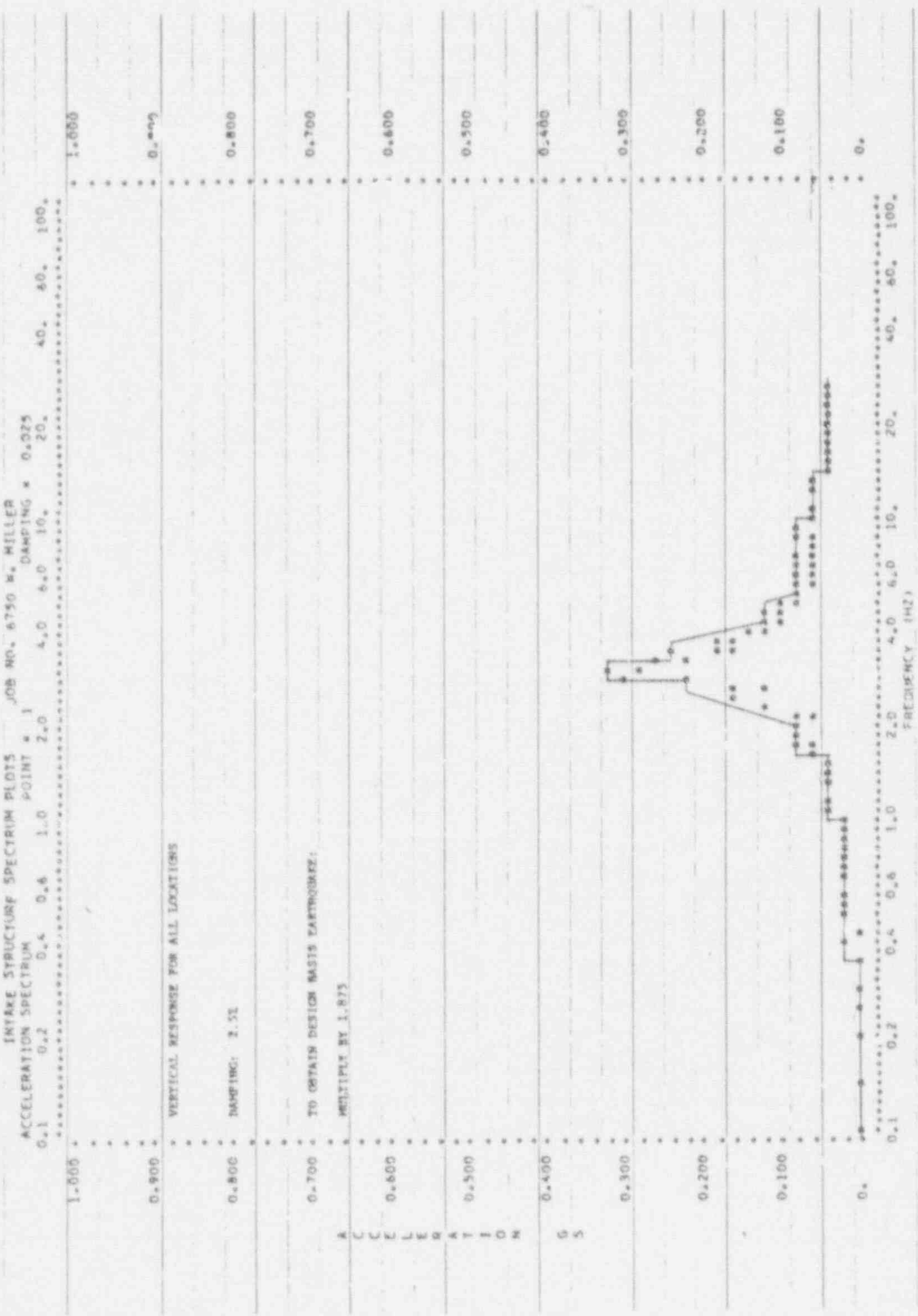
APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



APPENDIX A

IN-STRUCTURE RESPONSE SPECTRA



APPENDIX B

MODAL DAMPING

Modal damping values have been obtained by using a mass-mode weighing technique. The following is a brief description of the formulation of this technique.

For a particular mode, the damping for each mass point is proportional to the modal displacement. The total modal displacement at any mass point is comprised of three components, namely,

a. Rigid-Body horizontal translation on soil
b. Rigid-body rotation on soil
c. Deflection due to deformation of structure

The modal displacement parameters used in conjunction with the above shown figure are as follows:

ϕ_i = modal displacement = $\phi_T + \phi_R + \phi_S$

where ϕ_T = modal displacement due to horizontal translation of soil

ϕ_R = modal displacement due to rotation of soil

ϕ_S = modal displacement due to deformation of structure

The following damping parameters can be defined:

β_{mode} = critical damping to be applied to a mode

and

β_i = critical damping to be applied at one mass point

The damping at a particular mass point for a particular mode can be formulated as follows:

APPENDIX B

$$\beta_i = \frac{\phi^i_T\ \beta_T + \phi^i_R\ \beta_R + \phi^i_S\ \beta_S}{\phi_i}$$

where β_T = critical damping for the translation of soil

β_R = critical damping for the rotation on soil

β_S = critical damping for the deformation of structure

Thus, each displacement component is damped by its associated factor.

For a particular mode, the modal damping (percent of critical) can be obtained as shown below:

$$\beta_{mode} = \frac{\Sigma^i_1\ m_i\ \beta_i\ \phi_i}{\Sigma^i_1\ m_i\ \phi_i}$$

Thus, the modal damping shows that each mass point is damped proportionally to both mass (material) and type of component displacement.

The following is a tabulation of the damping values used for computing the (β_{mode}) modal damping.

DAMPING VALUES: (Percent of Critical Damping)

		OBE	DBE
Rigid Body Translation	(β_T)	2.0	3.0
Rigid Body Rotation	(β_R)	5.0	7.0
Reinforced Concrete Structure	(β_S)	3.0	5.0
Equipment Supports		2.0	3.0
Piping		0.5	0.5

APPENDIX C

MODAL SYNTHESIS TECHNIQUE

The Modal Synthesis Technique is a formulation which determines the "effectiveness" of each mode in order to obtain the total system response. The formulation required to obtain the Modal Participation Factor, which indicates the modal "effectiveness", is given. The following is a brief description the Modal Synthesis Technique.

Notations

N = Number of modes taken into consideration (Maximum N is the number of Masses)

P = Number of structural joints with masses

m = Number of lumped masses

i = Index showing a mass under consideration

j = Index showing a mode under consideration

ϕ_{ij} = Normalized mode shape for mass i in mode j

ϕ_{ij} is a shape factor for a certain mode with frequency f_j. ϕ_{ij} defines the relative position of joint i in the j^{th} mode of vibration. They have no defined amplitude.

m_i = Lumped mass at point i

Γ_j = Participation factor of mode j

f_j = Frequency of mode j

A_j = Acceleration from Recommended Response Spectra for f_j, or the acceleration amplitude of mode j

a_{ij} = Absolute acceleration at point i for mode j

APPENDIX C

With a known frequency for a given mode f_j and a given modal damping value β_j, an input acceleration amplitude A_j can be obtained from the Recommended (Ground) response Spectra. This ground-spectra acceleration A_j response for a mode, multiplied by its participation factor Γ_j, gives its modal acceleration amplitude.

The Participation factor for mode j is defined as:

$$\Gamma_j = \frac{\Sigma^{P}_{i=1} \quad \phi_{ij} \quad m_i}{\Sigma^{P}_{i=1} \quad (\phi_{ij})^2 \quad m_i}$$

The modal amplitude for mode j is obtained as:

$$A_j' = \Gamma_j \quad A_j$$

and the acceleration for mode j and at point i is

$$a_{ij} = A_j' \quad \phi_{ij} = \Gamma_j \quad \beta_j \quad \phi_{ij}$$

The inertia force for mode j at point i can then be computed as:

$$F_{ij} = m_i \quad a_{ij} = m_i \quad \Gamma_j \quad A_j \quad \phi_{ij}$$

The inertia force for each mode j can have a positive or negative direction, yet this direction is not known during the combination of the modal forces. The probable value is

$$F_i = [\quad \Sigma^{N}_{j=1} \quad (F_{ij})^2 \quad]^{1/2}$$

However,

$$F_{i(max)} \leq \Sigma \quad |F_{ij}|$$

APPENDIX C

In the computer output, the mode shapes have been normalized (adjusted) such that

$$\{\phi'\}^T \quad [M] \quad \{\phi'\} \quad = \quad [1]$$

This matrix normalization is equivalent to

$$\Sigma^{p}_{i=1} \quad \{\phi_{ij}\}^2 \quad m_i \quad = \quad 1$$

Hence, the participation factor in this case can be obtained by:

$$\Gamma_j \quad = \quad \Sigma^{p}_{i=1} \quad \phi_{ij} \quad m_i$$

By computing the modal participation factors for each mode of vibration, the principal modes which contribute to the total system response can be readily identified.