ILLINOIS POWER COMPANY IP

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500 SOUTH 27TH STREET, DECATUR, ILLINOIS 62525

December 3, 1981

Mr. James R. Miller, Chief
Standardization & Special Projects Branch
Division of Licensing
Office of Nuclear Reactor Regulation
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dear Mr. Miller:

Clinton Power Station Unit 1
Docket No. 50-461

In order to address the issues raised by Questions 220.14, 220.15, 220.121, and 220.26 we met with the NRC staff on October 15, 1981. It was agreed in that meeting that a site specific spectrum for the Clinton site will be developed and used for a new soil spring SSI analysis. The effort to develop the site specific spectrum is in progress and is expected to be completed by early January 1982. Based on our initial evaluation it was determined that a 0.17 g Regulatory Guide 1.60 spectrum will envelope the site specific spectrum for Clinton.

In a meeting on November 19, 1981 with the staff we presented our evaluation of Clinton plant design using a 0.17 g Regulatory Guide 1.60 spectrum. We showed that the new soil spring SSI responses including soil properties variation, when combined with SRV and LOCA loads, yield responses which are within code allowables.

The enclosure summarizes our evaluation of the plant design for 0.17 g Regulatory Guide 1.60 spectrum, including soil properties considered, methodology used and the results obtained.

It is also our judgment that when advantage is taken of the inherent conservatisms in the present seismic design of Clinton, the plant is capable of withstanding a seismic input of 0.20 g Regulatory Guide 1.60 spectrum.

B001
S1/1

Sincerely,

J.D. Geier
Manager, Nuclear Station Engineering

Attachments

cc: J. H. Williams, NRC Clinton Project Manager
H. H. Livermore, NRC Resident Inspector
N. C. Chokshi, NRC/SEB

Question 220.14

Our examination of Figures 3.7-12 through 3.7-27 indicate that at some frequency the free-field foundation level spectra is considerably less than the design response spectra as defined in FSAR, Section 3.7.1.1. Such reduction in design response spectra at foundation level is not acceptable to the staff. Either provide justification or use free-field motion at foundation level.

Question 220.15

Indicate how this motion accounts for variation in the soil properties at the site.

Question 220.21

You have stated that strain-compatible shear modulus and damping values for each layer for both OBE and SSE earthquakes are obtained from "SHAKE" analysis. Indicate, for each layer, to which strain-levels these values correspond to. In addition, give numerical values of these soil properties and corresponding strain levels. Provide this information for both vertical and horizontal analysis. In the staff's opinion, the soil properties used in the soil-structure interaction analysis should be those corresponding to the low strain levels, which are consistent with the realistic soil strain developed during the design earthquake. Use of high strain parameters need to be adequately justified. Discuss, how did you account for variation in material properties in your soil-structure interaction analysis.

Question 220.26

The current position of the Regulatory staff regarding the soil-structure interaction is contained in Attachment 2. Note that this position, in addition to the finite element method, requires the use of elastic half space approach. Provide your preocedure and results from the elastic half space approach for the staff's review. Also, indicate whether you will comply with the rest of the requirements of this position or not.

RESPONSE

The Clinton design basis SSI analysis was based on a 0.26g RG 1.60 spectra specified at the grade elevation. A one dimensional deconvolution analysis was used to determine the base rock motion for use in the finite element soil structure interaction analysis. This deconvolution analysis resulted in the free field foundation elevation spectra lower than the RG 1.60 spectra in several frequency ranges. The NRC staff has stated that this reduction in ground motion with depth is not acceptable and that in the new soil spring analysis, the wide band response spectra should be applied at the foundation elevation. In our opinion the reduction in motion with depth is supported by measured data, Seed (1). In addition, the application of a 0.26g RG 1.60 spectra at the foundation elevation together with the consideration of soil properties variation will be an extremely conservative definition of the seismic hazard at the Clinton site. To resolve these issues, we met with the NRC Staff on October 15, 1981 and it was agreed that the 0.26g RG 1.60 spectra is very conservative and a site specific spectra consistent with the seismic hazard and site conditions at the Clinton site will be developed for use with the new soil spring soil structure interaction analysis including the variation in soil properties.

The effort to develop the site specific spectra is in progress and is expected to be completed by January 4, 1982. Based on our initial evaluation it was determined that the site specific spectra at Clinton may be approximated by a 0.17g RG 1.60 spectra. This 0.17g RG 1.60 spectra specified at the foundation elevation was used in evaluation performed as part of this response.

Based on the evaluation presented here it can be concluded that the stresses in plant structures and piping using the new seismic response when combined with SRV and LOCA loads are well within code allowables based on minimum specified strengths. When advantage is taken of actual measured material strength, the present evaluation also shows that the plant structures [redacted] can be judged to be capable of withstanding a 0.20g RG 1.60 spectra input.

The following paragraphs provide the details of the soil properties considered, the method of determining soil impedances, the method of soil structure interaction analysis and the comparison of new responses to the design basis responses.

Soil Properties

Typical Subsurface profiles underneath the plant structures is shown in Figure 220.15-1. It can be seen from this figure that the plant structures are underlain by 20 feet of structural fill followed by 105 feet of Illinoian Till. The Illincian Till layer is underlain by Lacustrine Deposit and the Pre-Illinoian Till followed by bed rock. The dynamic soil properties considered for the design basis analysis is presented in FSAR Table 2.5-48. To determine the appropriate upper and lower bound of dynamic soil properties for use in the new soil spring soil structure interaction analysis, the dynamic triaxial test results on samples from each of the deposits underlying the plant were replotted and evalauted. Based on this evaluation the upper and lower bound soil dynamic shear modulus values were determined and are presented in Figures 220.15-2 through 220.15-5 for the sturctural fill, Illinoian Till, Lacustrine Deposit and Pre-Illinoian Till layers respectively. These upper and lower

bound properties were used in the soil spring soil structure interaction analysis (SSI).

The soil shear modulus is dependent on the maximum effective shear strains expected during the SSE. The range of soil strain expected during strong motion earthquakes is presented in Figure 220.15-6 which is reproduced from Reference (2). This range varies from 1.5×10^{-2} to 1.5×10^{-1} percent strain. For the Clinton SSI analysis a low 2.0×10^{-2}% strain level is considered. As lower strain levels lead to higher shear modulus values and higher shear modulus values lead to higher structural response, the 2.0×10^{-2}% shear strain level is considered to be conservative.

Based on the above considerations and data presented in Figures 220.15-2 through 220.15-5, the upper bound, mean and the lower bound soil properties used in the soil spring soil structure interaction analysis are presented in Table 220.15-1. It can be observed that a wide variation in soil shear modulus values is considered. No variation in the soil damping values was considered because the 6% to 10% of critical damping values used are considered to be conservatively low soil damping values when compared to 7% of critical damping recommended for reinforced concrete at or near yield in RG 1.61.

Soil Impedance Function Computation

The soil media is modeled by a visco-elastic layered half space. The soil spring and dashpot constants are obtained in terms of frequency dependent impedance functions. These impedance functions are computed using Sargent & Lundy's DIMFU program. The DIMFU program computes the impedance functions for a rigid circular

foundation placed on the surface of a layered visco-elastic half space using Luco's (3) method.

To validate the program, the impedance functions for vertical ($k_{vv}+ic_{vv}$), rocking ($K_{mm}+iC_{mm}$) and Horizontal ($K_{HH}+iC_{HH}$) motions were obtained by DIMFU for the two example problems for which impedance functions were presented in the original paper by Luco (3).

In the first example, a soil medium of uniform visco-elastic half-space with voigt-type damping was considered. The impedance functions for a_o = 1 and 4; are obtained. These impedance functions are shown in Figure 220.15-7 for rocking, Figure 220.15-8 for horizontal and Figure 220.15-9 for vertical vibrations for σ = 1/3 and a'/a = 0.3, where a'/a is the relative viscosity coefficient of the medium. The impedance functions obtained by DIMFU compare well with those obtained by Luco (3) and Veletsos and Vebric (4).

In the second validation example, a hysteretically damped soil medium consisting of a viscoelastic layer of thickness ℓ (=150 ft.) and properties β_1, σ_1, ρ_1 (mass density) and ξ_1 (hysterestic damping constant) resting on a viscoelastic half-space with properties ξ_2, β_2, σ_2, and ρ_2 was considered. The radius of the rigid disc is a (=50 ft). The impedance functions are obtained for a_o = 1 and 5 for $\beta_1 = 0.8\rho_2$, $\rho_1 = 0.85\rho_2$, $\sigma_1 = \sigma_2 = 0.25$; $\xi_1 = \xi_2 = 0.05$; $\ell/a = 3$. These impedance functions are shown in Figure 220.15-10 for rocking, Figure 220.15-11 for horizontal, and Figure 220.15-12 for vertical vibrations. The results obtained from DIMFU compare well with those obtained by Luco (3).

Based on these two examples, it can be concluded that DIMFU correctly computes soil impedance functions.

Soil Structure Interaction Analysis

An extension of the component mode substitution approach suggested by Benfield and Hruda [5] is used to compute the soil structure interaction response. The soil medium is one substructure and the structure represents the other substructure. The structure is represented by its fixed base modal characteristics, whereas the soil medium is represented by frequency dependent impedance functions. The two substructures are then coupled to obtain the structural response, including the effects of soil-structure interaction. The response is obtained using the frequency response method [7].

Figure 220.15-13 shows the schematic details of the Clinton Unit 1 model. This model is consistent with the one unit model used in the design basis Finite Element Method Soil Structure Interaction analysis. The Main Building is modeled by shear beam-slab system and the Containment, Drywell, Pedestal, Shield Wall and RPV are modeled by beam elements. The masses are lumped at slab locations for the main building model. For the portions of the structure modeled by the beam elements, the locations of the lumped masses are shown in Figure 220.15-13.

The soil is modeled by frequency dependent spring dash pot system for horizontal, rocking and vertical motions. These soil springs are connected to the structural model at the common boundary interface node labeled 12.

For the soil-structure system shown in Figure 220.15-13, the equations of motion for a given earthquake excitation $\ddot{Y}(t)$ are:

$$\begin{bmatrix} m_{ii} & 0 \\ 0 & m_{bb} \end{bmatrix} \begin{Bmatrix} \ddot{x}_i \\ \ddot{x}_b \end{Bmatrix} + \begin{bmatrix} c_{ii} & c_{ib} \\ c_{bi} & c_{bb}+c_{ss} \end{bmatrix} \begin{Bmatrix} \dot{x}_i \\ \dot{x}_b \end{Bmatrix} + \begin{bmatrix} k_{ii} & k_{ib} \\ k_{bi} & k_{bb}+k_{ss} \end{bmatrix} \begin{Bmatrix} x_i \\ x_b \end{Bmatrix} = - \begin{bmatrix} M_{ii} & 0 \\ 0 & M_{bb} \end{bmatrix} \begin{Bmatrix} \phi_{ri} \\ \phi_{rb} \end{Bmatrix} \ddot{y} \quad (1)$$

where the subscripts i, and b refer to the structure interior and the common boundary interface and the subscript s refers to the soil respectively; m, c, and k are the mass, the damping, and the stiffness matrix components, respectively; and $\ddot{x}$, $\dot{x}$, and x are the acceleration, velocity, and displacement vectors relative to the base motion. ϕ_{ri} , and ϕ_{rb} are the rigid body vectors with components equal to unity in the direction parallel to the support motion, and zero otherwise. $\ddot{y}(t)$ is the prescribed acceleration motion.

The equations of motion for free vibration of the substructure representing the structure are given by:

$$\begin{bmatrix} m_{ii} & 0 \\ 0 & m_{bb} \end{bmatrix} \begin{Bmatrix} \ddot{x}_i \\ \ddot{x}_b \end{Bmatrix} + \begin{bmatrix} k_{ii} & k_{ib} \\ k_{bi} & k_{bb} \end{bmatrix} \begin{Bmatrix} x_i \\ x_b \end{Bmatrix} = 0 \quad (2)$$

For linear problems, the displacement of an internal degree of freedom can be expressed as:

$$\{x_i\} = \{x_i\}_c + [\phi_c]\{x_b\} \quad (3)$$

where $\{x_i\}_c$ is the displacement of internal degrees of freedom when common boundary nodes are assumed fixed and $[\phi_c]$ are the constrained modes for the substructure representing the displacement of internal degrees of freedom induced by a unit displacement of the common

boundary degrees of freedom with all other boundary freedoms fixed. The constrained modes are computed by solving for each substructure a set of linear equations given by

$$[k_{ii}][\Phi_c] = -[k_{ib}] \tag{4}$$

Thus, for free vibrations,

$$[m_{ii}]\{\ddot{x}_i\}_c + [k_{ii}]\{x_i\}_c = 0 \tag{5}$$

Expressing $\{x_i\}_c$ in terms of normalized mode shapes of equation 5,

$$\{x_i\}_c = [\phi]\{p\} \tag{6a}$$

where

$$[\phi]^T[k_{ii}][\phi] = [\omega_n^2] \tag{6b}$$

and

$$[\phi]^T[m_{ii}][\phi] = [I] \tag{6c}$$

$\{p\}$ represents the generalized normal coordinates and ω_n is the frequency of the constrained substructure in mode n. Based on equations 3 and 6, the displacement in the soil-structure system can be expressed as

$$\begin{Bmatrix} x_i \\ x_b \end{Bmatrix} = \begin{bmatrix} \phi & \Phi_c \\ 0 & I \end{bmatrix} = \begin{Bmatrix} p \\ x_b \end{Bmatrix} \tag{7}$$

Transformation of equation 1 by equation 7 yields the equations of motion in terms of the generalized coordinates, as follows:

$$\begin{bmatrix} I & \bar{m}_{ib} \\ \bar{m}_{bi} & \bar{m}_{bb} \end{bmatrix} \begin{Bmatrix} \ddot{p} \\ \ddot{x}_b \end{Bmatrix} + \begin{bmatrix} \bar{c}_{ii} & \bar{c}_{ib} \\ \bar{c}_{bi} & \bar{c}_{bb} \end{bmatrix} \begin{Bmatrix} \dot{p} \\ \dot{x}_b \end{Bmatrix} + \begin{bmatrix} \omega^2 & 0 \\ 0 & \bar{k}_{bb} \end{bmatrix} \begin{Bmatrix} p \\ x_b \end{Bmatrix}$$

$$= -\begin{Bmatrix} \gamma_i \\ \gamma_b \end{Bmatrix} \ddot{Y}(t) \tag{8}$$

where

$$\bar{m}_{ib} = \bar{m}_{bi}^T = \phi^T m_{ii} \phi_c \quad ; \quad \bar{m}_{bb} = m_{bb} + \phi_c^T m_{ii} \phi_c ;$$

$$\bar{k}_{bb} = k_{bb} - \phi_c^T k_{ii} \phi_c + k_{ss} ; \quad \bar{c}_{ib} = \bar{c}_{bi}^T = \phi^T c_{ii} \phi_c + \phi^T c_{ib} ;$$

$$\bar{c}_{ii} = \phi^T c_{ii} \phi \quad ; \quad \bar{c}_{bb} = c_{bb} - \phi_c^T c_{ii} \phi_c + c_{ss} ;$$

$$\gamma_i = \phi^T m_{ii} \phi_{ri} \quad ; \text{ and } \gamma_b = \phi_c^T m_{ii} \phi_{ri} + m_{bb} \phi_{rb}$$

The soil spring and dash pot constants k_{ss} and c_{ss} are frequency-dependent. The structural damping is specified in terms of modal damping. Under these conditions, equation 8 is best solved using the frequency response method (7). The following formulation for the damping matrix greatly reduce the computations:

$$\bar{c}_{ii} = \phi^T c_{ii} \phi = [2 \beta_n \frac{\omega_n^2}{\Omega}] \tag{9a}$$

$$\bar{c}_{ib} = \bar{c}_{bi}^T = 0 \tag{9b}$$

$$c_{bb} = \frac{2\beta}{\Omega} k_{bb} \tag{9c}$$

where β_n is the modal damping ratio for mode n of the structure and Ω is the frequency for which the response is being computed. c_{bb} is the damping matrix of structural elements which connect to the common interface nodes having a modal damping ratio of β. Expressing the input motions and response in terms of a truncated Fourier Series yields

$$\ddot{y}(t) = \text{Re} \sum_{j=0}^{N/2} \ddot{y}(\Omega_j) \exp(i\Omega_j t) \tag{10a}$$

$$x(t) = \text{Re} \sum_{j=0}^{N/2} x(\Omega_j) \exp(i\Omega_j t) \tag{10b}$$

where N is the number of digitized points in the input motion. The amplitudes $\ddot{y}(\Omega_j)$ and $x(\Omega_j)$ can be found by the Fast Fourier Transform algorithm (6). Substitution of equations 9 and 10 into 8 yields

$$(-\Omega_j^2[\bar{M}]+[K^*]) \quad x(\Omega_j) = -\{Q\}\ddot{y}(\Omega_j) \tag{11}$$

where

$$[\bar{M}] = \begin{bmatrix} I & \bar{m}_{ib} \\ \bar{m}_{bi} & \bar{m}_{bb} \end{bmatrix} \qquad \{Q\} = \begin{Bmatrix} \gamma_i \\ \gamma_b \end{Bmatrix}$$

$$[\bar{K}] = \begin{bmatrix} [\omega_n^2(1+i2\beta_n)] & 0 \\ 0 & \bar{K}^*_{bb} \end{bmatrix}$$

in which

$$\bar{K}^*_{bb} = -\phi_c^T(1+2i\,\beta_n)k_{ii}\phi_c + (1+2i\beta)k_{bb} + (k_{ss}+i\Omega_j c_{ss})$$

The linear set of equations represented by equation 11 determines the displacement amplitudes $x(\Omega_j)$ at each frequency Ω_j, j=0,1...N/2. The equations can be solved by Gaussian elimination. The displacements in the time domain follow from equation 10 by the inverse Fast Fourier Transform.

In the design basis Finite Element Method (FEM) SSI analysis, analysis was performed for one unit and two unit configurations. In the soil spring analysis presented here only the one unit configuration was considered based on our experience with the FEM SSI analysis which showed that the one unit and the two unit responses are very close with the one unit configuration giving a slightly higher response at many locations. It is our judgment that the two unit soil spring SSI result will also lead to similar results.

As stated earlier, the horizontal and vertical models used in the soil spring analysis are consistent with those used in the design basis FEM SSI analysis. However, for the design basis response calculation, the interacted base mat ~~spectra~~ motions together with a more detailed decoupled fixed base structural model was used to compute design responses. As shown in our response to Question 220.25 the responses obtained from the SSI model and the decoupled model are very close. Based on this, ~~for~~ the soil spring responses presented here are based on the SSI model. The decoupled model responses were not computed because they are expected to be very close to those obtained from the SSI model.

Comparison of Structural Responses

Table 220.15-2 presents a comparison of shear wall forces from the design basis FEM analysis and the new soil spring analysis. This response comparison is typical of forces in major structural elements. It can be observed from this comparison that the new forces are 1% to 10% lower than design basis loads with an average margin of approximately 5%.

Clinton design criteria for reinforced concrete member requires that the rebar stresses be limited to 0.9 f_y, where f_y is the minimum specified yield strength. Table 220.15-3 presents the actual average material strength at the Clinton plant. It can be observed that this provides an additional 17% load carrying capacity. Based on the above it can be concluded that the structures have a capacity to resist an earthquake at least 1.17 times the 0.17g RG 1.60 spectrum considered i.e., 0.20 RG 1.60 spectra when elastic analysis is used and advantage

is taken of actual material strength [redacted]. The ~~time~~ true ultimate capacity, however, will be even higher because of significant yielding which would occur prior to failure.

Comparison of Floor Response Spectra

The floor response spectra defines the input excitation for piping and equipment supported within the plant structures. To assess the effect of the soil spring SSI analysis, the floor response spectra from the new SSI analysis are compared to those obtained from the design basis FEM analysis at typical locations throughout the structure. These responses are compared for the two horizontal (North-South and East-West) and one vertical response component at each selected location as follows:

a. Base Mat Elevation 712' for all buildings: comparison is shown in Figures 220.15-14, 220.15-15 and 220.15-16.

b. Main Building Floor Elevation 762': comparison is shown in Figures 220.15-17, 220.15-18 and 220.15-19.

c. Containment Shell Elevation 803': comparison is shown in Figures 220.15-20, 220.15-21 and 220.15-22.

d. Sheild Wall and Pedestal at RPV Support Elevation 743': comparison is shown in Figures 220.15-23, 22015-24 and 220.15-25.

These Figures, 220.15-14 through 220.15-25, present the comparison of unwidened floor response spectra obtained from the new soil spring SSI analysis to the design basis FEM analysis widened spectra for

a 2% oscillator damping. It can be observed that the design basis responses in general, envelope the new soil spring responses.

The design of Clinton piping and equipment are based on the combined SRV+SSE+LOCA loadings. Thus, a comparison of the floor response spectra for the combined SRV+SSE+LOCA loading using the FEM and the soil spring SSE is more indicative of the effect of the new SSI analysis on piping and equipment. This comparison for the two horizontal and one vertical response components at each selected location is presented as follows:

a. Base Mat Elevation 712' for all buildings: comparison of combined spectra is shown in Figures 220.15-26, 220.15-27 and 220.15-28.

b. Main Building Floor Elevation 762': comparison of the combined spectra is shown in Figures 220.15-29, 220.15-30 and 220.15-31.

c. Containment Shell Elevation 803': comparison the combined spectra is shown in Figures 220.15-32, 220.15-33 and 220.15-34.

d. Shield Wall and Pedestal at RPV Support Elevation 743': comparison of the combined spectra is shown in Figures 220.15-35 220.15-36 and 220.15-37.

The above Figures, 220.15-26 through 220.15-37, present the comparison of the combined SSE+SRV+LOCA floor response spectra obtained from the new soil spring SSI analysis to the combined spectra used for design. The spectra are for a 2% oscillator damping and are widened ± 15%. From these comparisons it can be concluded that:

a. The new combined spectra for the vertical component ~~is~~ are enveloped by the design basis spectra.

b. For the Containment ~~[illegible]~~ the horizontal ~~[illegible]~~ combined new spectra ~~and~~ are essentially enveloped by the design basis spectra. ~~[illegible]~~

b. For the Main Building and the Pedestal ~~[illegible]~~ the new combined horizontal spectra ~~is~~ are higher than the design basis in the 2-6 Hz frequency range by approximately 10% – 20%.

Based on the above comparison it can be judged that the new soil spring responses will not have any significant effect on design. To confirm this judgment, four affected subsystems were analyzed to the new combined spectra. The four selected subsystems were chosen so that ~~to have~~ the new spectra are higher than those corresponding ~~to the~~ design basis spectra at the subsystem vibration frequencies. The results can be summarized as follows:

a. Subsystem LP-3: The input response spectra are: Auxiliary Building at Elevation 712' and Elevation 737' and Containment Building at Elevation 720'. Although some of the directional input response accelerations are higher for the site specific (Elevation 737') the envelope response using all the applicable locations is lower for the site specific response spectra compared to the ~~Regulatory Guide~~ Design Basis response spectra, and the resulted stresses, support loads and equipment reactions are also lower. The piping stresses, support reactions and equipment reactions are summarized in Table 220.15-4.

b. Subsystem SX-16: The input response spectra are, Auxiliary Building Elevation 712' and Control Building Elevation 737'. The site specific response spectra are higher than the Design Basis response at some of the subsystem vibration mode frequencies. The results show that the new stresses (due to the site specific responses) are higher at one location by 6% and one support load increased by 21%. However, the increased stresses and support load values are still below the allowables and support rated capacity. The rest of the subsystem stresses and support loads are lower for the site specific response spectra. The piping stresses, support reactions and equipment reactions are summarized in Table 220.15-5.

c. Subsystem RT-1: The input response spectra are, Containment Elevation 755', Drywell Elevation 737', Sheild Wall Elevation 759' and the RPV Elevation 743'. The envelope of the site specific response spectra are higher at some of the subsystem vibration mode frequencies. The analysis results show both increase and decrease in the stresses and support reactions throughout the subsystem. However, in the case of load increase, the site specific response spectra analysis produced less than 5% increase in the stresses and support loads, and the new values remain very much below the allowables and the rated capacities. The piping stresses, support reactions and equipment reactions are summarized in Table 220.15-6.

d. Subsystem RH-34: The input response spectra are, Containment Elevation 755', Drywell Elevation 755', Shield Wall Elevation

743' and RPV Elevation 759'. The site specific response spectra are higher at very few [redacted] the subsystem vibration mode frequencies. However, the analysis results show that, the site specific spectra gives lower stresses and support loads at all the subsystem locations. The piping stresses, support reactions and equipment reactions are summarized in Table 220.15-7.

Circulating Water Screen House

No soil spring SSI analysis for the circulating water screen house was performed because the founding conditions and the design basis analysis at the screen house are very similar to that in the main plant area. It is our judgment that the comparison of the new soil spring and the design basis responses at the river screen house would lead to the same conclusions as for the main plant i.e., that the design basis is conservative.

Conclusions

New analysis based on a 0.17g RG 1.60 spectra together with soil spring SSI analysis, soil properties variation and a foundation

level input of the wide band spectra leads us to conlcude that the new forces and stresses are well below design allowables. Based on this new analysis it is our judgement that Clinton structures [redacted] can withstand earthquakes defined by a 0.20g RG 1.60 when advantage is taken of actual measured yield strength and sections are stressed to the theoretical ultiamte capacities. The true ultimate capacity however, will be higher because of the significant yielding which would occur prior to failure.

References

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3. J. E. Luco, "Vibration of A Rigid Disc on A Layered Visco-elastic Medium," Nuclear Engineering and Design, No. 35, 1976, pp. 325-340.

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7. Hurty, W. C., and Rubinstein, M. F., Dynamics of Structures, Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 1964.

TABLE 220.15-1
DYNAMIC SOIL PROPERTIES USED FOR SSI ANALYSIS

SOIL LAYER No.	LAYER DEPTH (FT)	WEIGHT DENSITY (K/FT^3)	POISSON'S RATIO	UPPER BOUND SOIL		MEAN VALUE SOIL		LOWER BOUND SOIL	
				SHEAR MODULUS (K/FT^2)	DAMPING RATIO	SHEAR MODULUS (K/FT^2)	DAMPING RATIO	SHEAR MODULUS (K/FT^2)	DAMPING RATIO
1	20.0	0.132	0.40	6063.0	0.084	4547.0	0.081	3032.0*	0.081
2	105.0	0.150	0.35	7000.0	0.101	5250.0	0.101	3500.0	0.101
3	10.0	0.134	0.35	5500.0	0.059	4125.0	0.059	2750.0	0.059
4	75.0	0.145	0.40	5500.0	0.080	4125.0	0.080	2750.0	0.080
5	HALF-SPACE	0.159	0.29	300000.0	0	300000.0	0	300000.0	0

* Conservatively assumed as 1/2 of the upper bound soil shear modulus.

TABLE 220.15-2
COMPARISON OF SHEAR WALL FORCES FROM
FINITE ELEMENT AND SOIL SPRING APPROACHES

SHEAR WALL SPRING No.	FINITE ELEMENT METHOD (KIPS)	SOIL SPRING METHOD (KIPS)
X-1101003	7,466	7,361
X-1101005	8,071	7,874
X-1101009	10,005	9,423
X-1101011	11,346	10,634
X-1101019	13,427	12,235
X-1101027	15,449	13,949
Y-1101014	11,299	11,080
Y-1101018	14,298	13,150
Y-1101022	10,966	10,047
Y-0102002	12,484	10,625
Y-1102006	11,986	11,237
Y-1102010	11,220	10,374

TABLE 220.15-3
ACTUAL AND MINIMUM SPECIFIED YIELD STRENGTH

GRADE	MINIMUM SPECIFIED KSI	ACTUAL AVERAGE KSI
A36	36.0	44.3
A572 Grade 50	50.0	60.1
A588 Grade 50	50.0	58.2
Rebars Sizes #8 thru #18	60.0	70.3

TABLE 220.15-4
SUMMARY OF STRESSES, SUPPORT REACTIONS,
AND EQUIPMENT REACTIONS FOR SUBSYSTEM LP-3

STRESSES/NODE	SITE SPEC. RS	DESIGN RS	% CHANGE
17 UFT	10,356	10,727	-3.46
33 UFT	12,043	12,760	-5.62
35 UFT	11,646	12,331	-5.56
5	1,956	2,034	-3.83
30	4,018	4,205	-4.45
65 UFT	14,347	14,842	-3.34
AOB	3,192	3,315	-3.71
175A	2,026	2,240	-9.55
R5	2,875	2,937	-2.11
180 UFT	3,765	3,925	-4.08

SUPPORTS/NODE	SITE SPEC. RS	DESIGN RS	% CHANGE
5 Y-DIR	18,378	19,399	-5.26
5 Z-SKEW	23,223	23,287	-0.28
19 Z-SKEW	4,995	5,177	-3.52
40B Y-DIR	6,165	6,593	-6.49
50 Y-DIR	12,193	12,400	-1.67
52 Z-SKEW	8,147	8,242	-1.15
67 X-DIR	8,102	8,391	-3.44
70B Y-DIR	5,408	5,476	-1.24
C75 X-SKEW	3,661	3,783	-3.22
76 X-SKEW	4,981	5,027	-0.92
175A X-SKEW	12,833	12,947	-0.88
175A Z-SKEW	2,773	2,959	-6.29
80A Y-DIR	9,499	9,549	-0.52

EQUIPMENT/NODE	SITE SPEC. RS		DESIGN RS		% CHANGE	
	F_1	M_1	F_o	M_o	AF	M
120	4,717	17,809	4,769	17,872	-1.09	-.35

TABLE 220.15-5
SUMMARY OF STRESSES, SUPPORTS REACTIONS,
AND EQUIPMENT REACTIONS FOR SUBSYSTEM SX-16

STRESSES/NODE	SITE SPEC. RS	DESIGN RS	% CHANGE
7	3102	3594	-13.70
18	4516	5585	-19.14
25B	6532	9059	-27.89
42	4230	4939	-14.36
75A	5395	6838	-21.10
127	3699	4187	-11.66
135B	6301	5965	+ 5.63
156	3420	3914	-12.62
190	3659	4178	-12.42
203	5839	7415	-21.25

SUPPORTS/NODE	SITE SPEC. RS	DESIGN RS	% CHANGE
Ry 18	6388	8089	-21.03
Rx 30	1477	2008	-26.44
Sz 60	1247	2105	-40.76
Ry 73	6919	9056	-23.60
Ry 77	5363	8783	-38.94
Sy 80	2737	4662	-41.29
Sx 90	3906	6279	-37.79
Rz 103	2632	4115	-36.04
Ry 165A	4486	5619	-20.16
Rx 170	2986	2488	+20.02
Ry 190	6539	8383	-22.00

EQUIPMENT/NODE	SITE SPEC. RS		DESIGN RS		% CHANGE	
	F_1	M_1	F_o	M_o	AF	M
No Equipment						

Table 220.15-6: Summary of Stresses, Support Reactions and Equipment Reaction for Subsystem RT-01

Service Level	Stresses Node No.	Site Spec. R.S.	Design R.S.	Change %
C	65A	6945	6773	+2.5%
C	140B	6808	6741	+1.1%
C	390B	11142	11404	-2.3%
C	440	5830	5836	-0.1%
C	596	3955	4026	-1.7%
C	A660	7597	7645	-0.6%
D	265A	18454	18167	+1.6%
D	470	6000	6369	-5.6%
D	627	5904	6339	-6.9%

Service Level	Supports Node No.	Site Spec. R.S.	Design R.S.	Change %
C	70B	1271	1248	+1.8%
C	156	1294	1283	+0.9%
C	395	3811	3839	-0.7%
C	463	1286	1277	+0.7%
C	490	306	299	+2.3%
C	535B	1894	1902	-0.4%
C	607	429	430	-0.2%
DD	247	131	138	-5.1%
D	490	207	223	-7.1%
D	620	567	543	+4.4%

Service Level	Equip. Node No.	Site Spec. R.S.		Design R.S.		Change %	
		F_1	M_1	F_o	M_o	AF	AM
C	665	401	1072	404	1073	-0.7%	-0.1%
D	665	92	118	119	129	-22.1%	-8.5%

Table 220.15-7 Sumamry of Stresses, Support Reactions and Equipment reactions for Subsystem RH-34

	Stresses Node No.	Site Spec. R.S.	Design R.S.	Change %
LEVEL-C	215	8475	8550	-0.88
	250A	7669	7717	-0.62
	295B	6316	6359	-0.68
	315	5890	5918	-0.47
LEVEL-D	190	11677	11781	-0.88
	250B	8019	8079	-0.74
	295A	6110	6182	-1.16
	295[illegible]	7149	7193	-0.61

	Supports Node No.	Site Spec. R.S.	Design R.S.	Change %
LEVEL-C	200	4712	4834	-2.46
	205	4124	4237	-2.67
	220	8823	9029	-2.28
	240	10234	10569	-3.17
	263	8444	8720	-3.17
LEVEL-D	220	9697	9840	-1.45
	225	10696	10846	-1.38
	240	5623	5920	-5.01
	300	14244	14825	-3.9
	303	18330	19009	-3.57

Equip. Node No.	Site Spec R.S.		Design R.S.		Change %	
	F_1	M_1	F_o	M_o	AF	AM
	NONE					

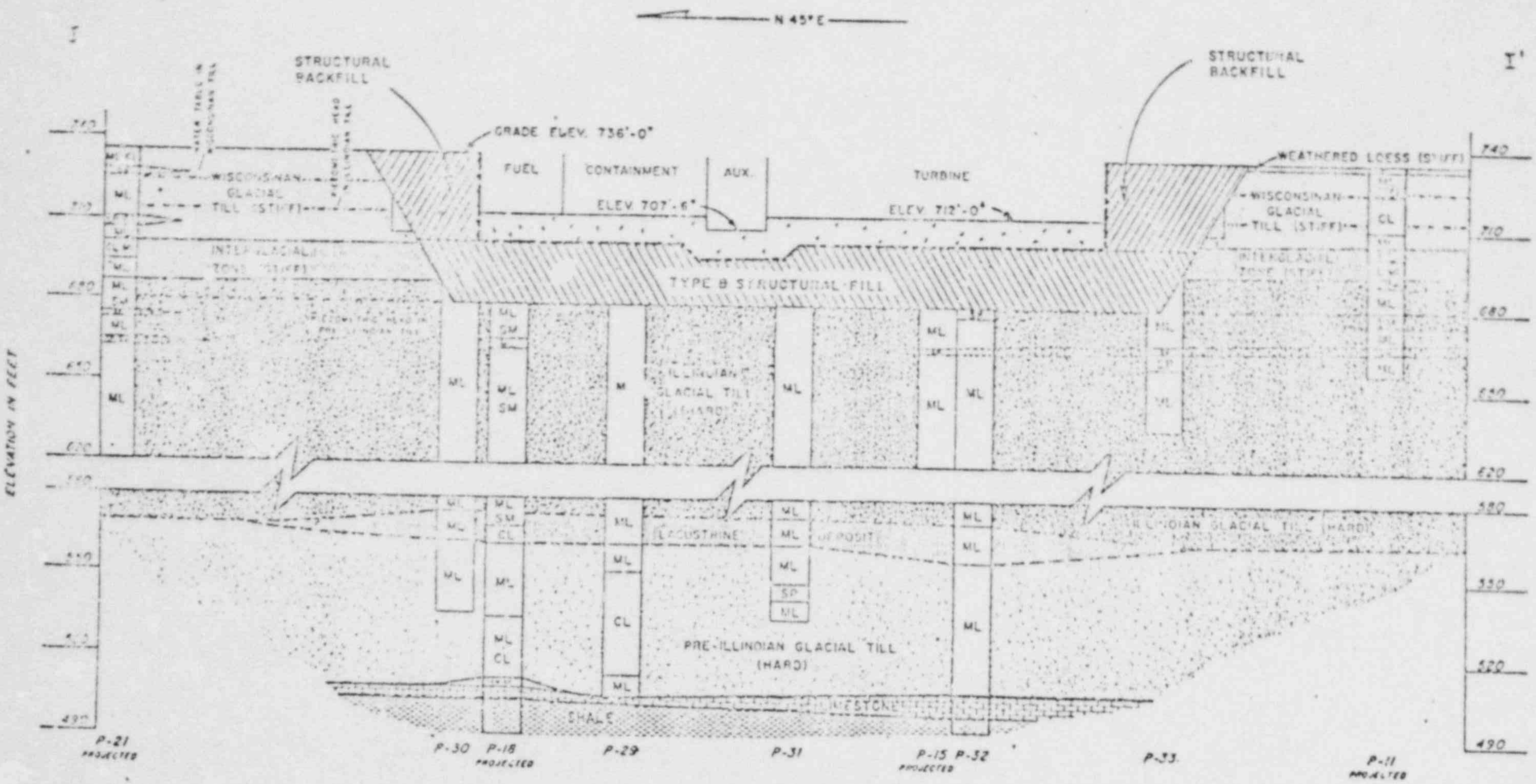

Figure 220.15-1
TYPICAL SUBSURFACE PROFILE

BY T. Parber DATE 11.25.81
CHECKED BY M. Kothawala 11-25-81
FILE ______

REVISIONS
BY ______ DATE ______

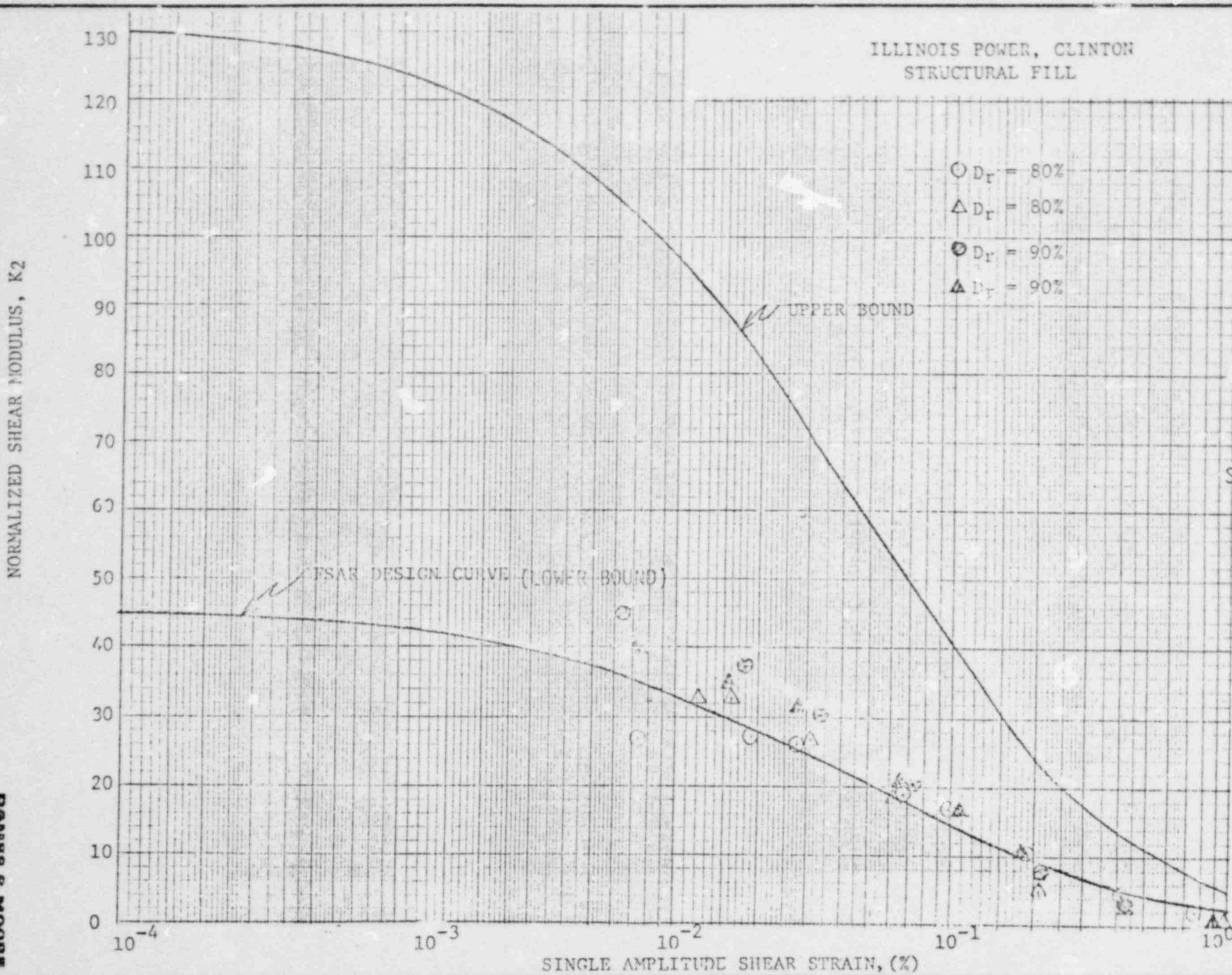

Figure 220.15-2
Structural Fill
Properties

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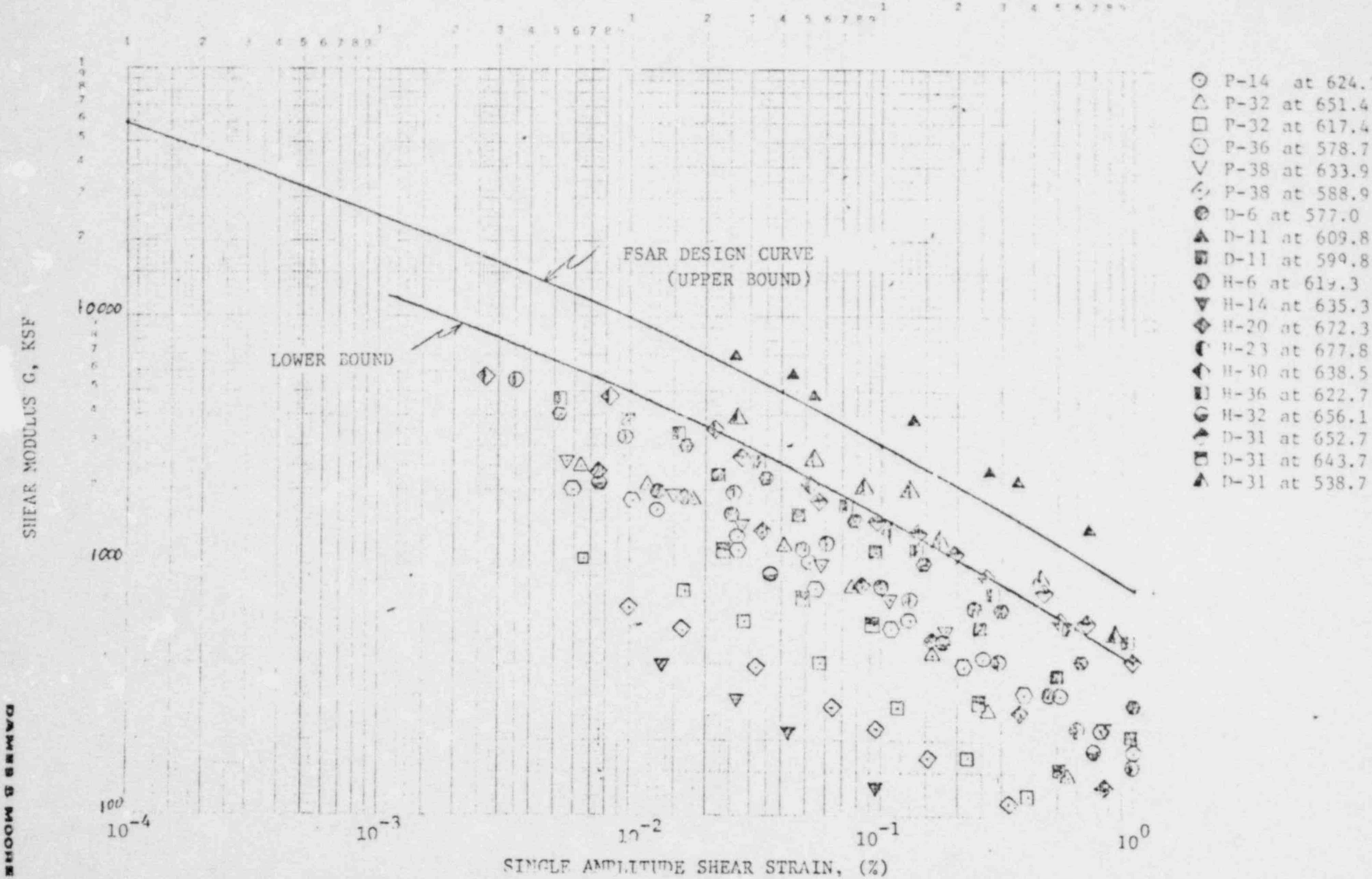
FIGURE 220.15-3
ILLINOIAN TILL PROPERTIES



DAMES & MOORE

BY J. Pulley DATE 11/21/81 CHECKED BY M. K[illegible] 11-25-81 FILE ______

REVISIONS BY ______ DATE ______

FIGURE 220.15-4
LACUSTRINE DEPOSIT PROPERTIES

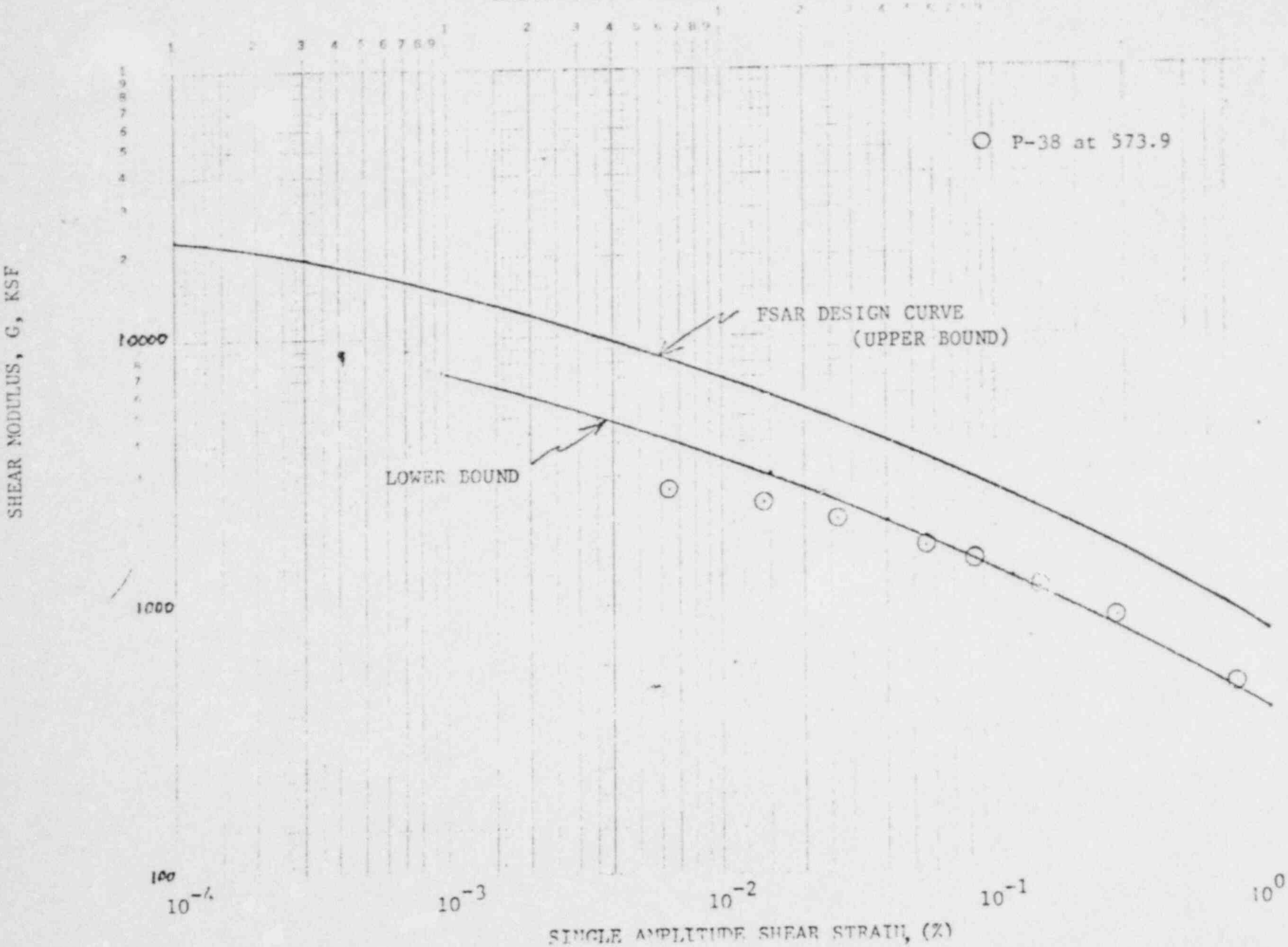


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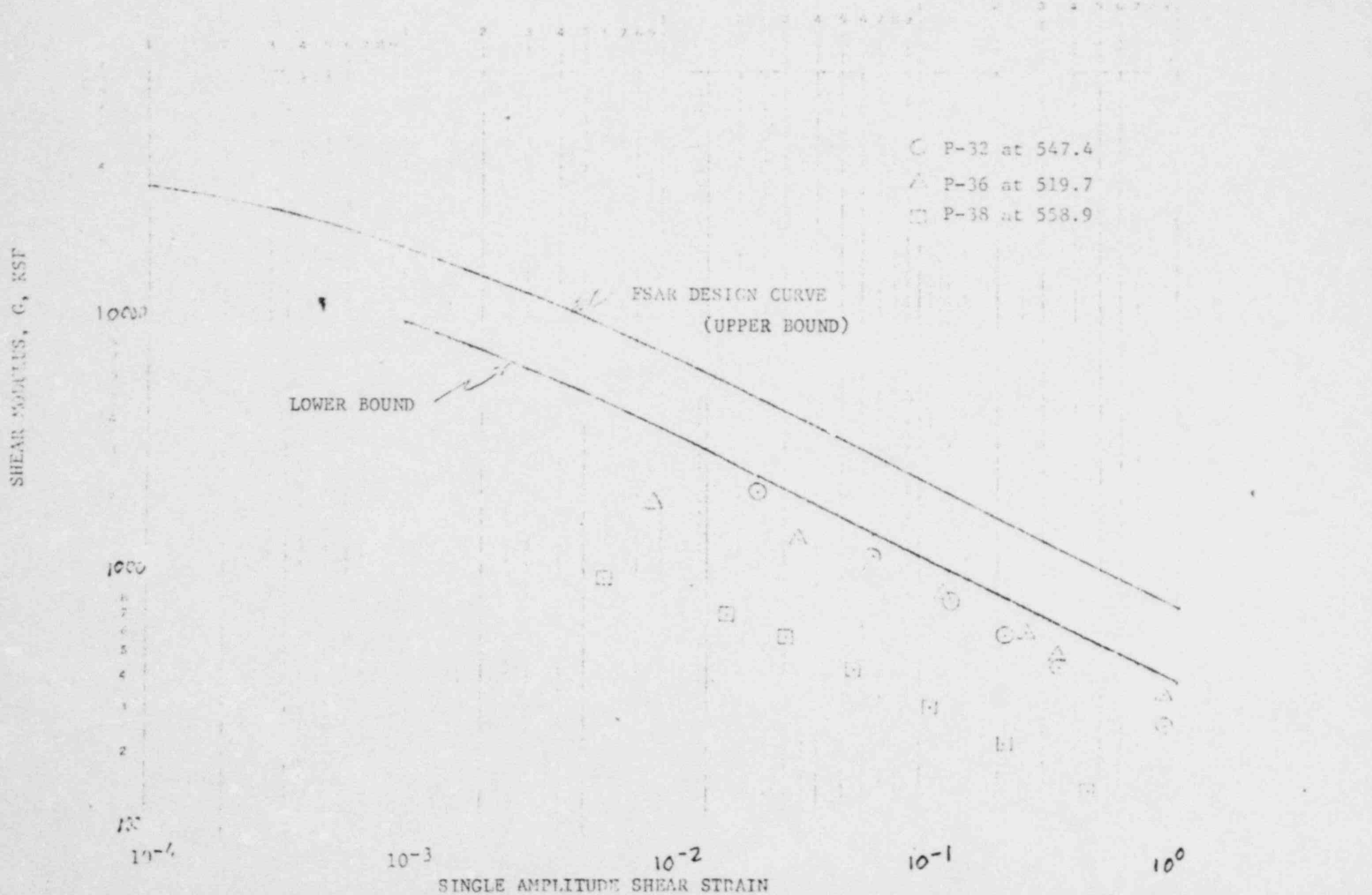
FIGURE 220.15-5
PRE-ILLINOIAN DEPOSIT PROPERTIES



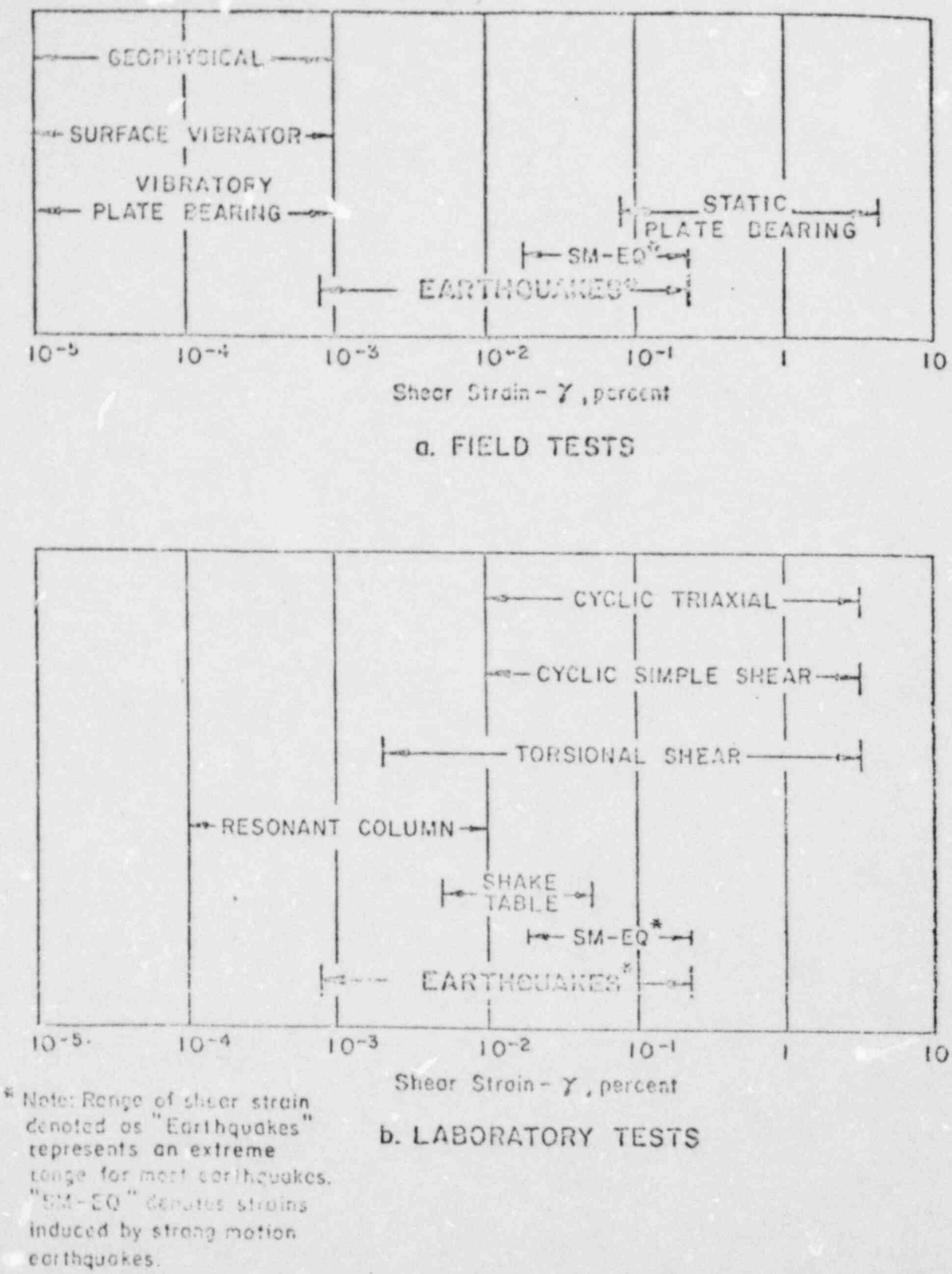


FIGURE 220.15-6
SOIL STRAIN RANGE FOR STRONG MOTION EARTHQUAKES

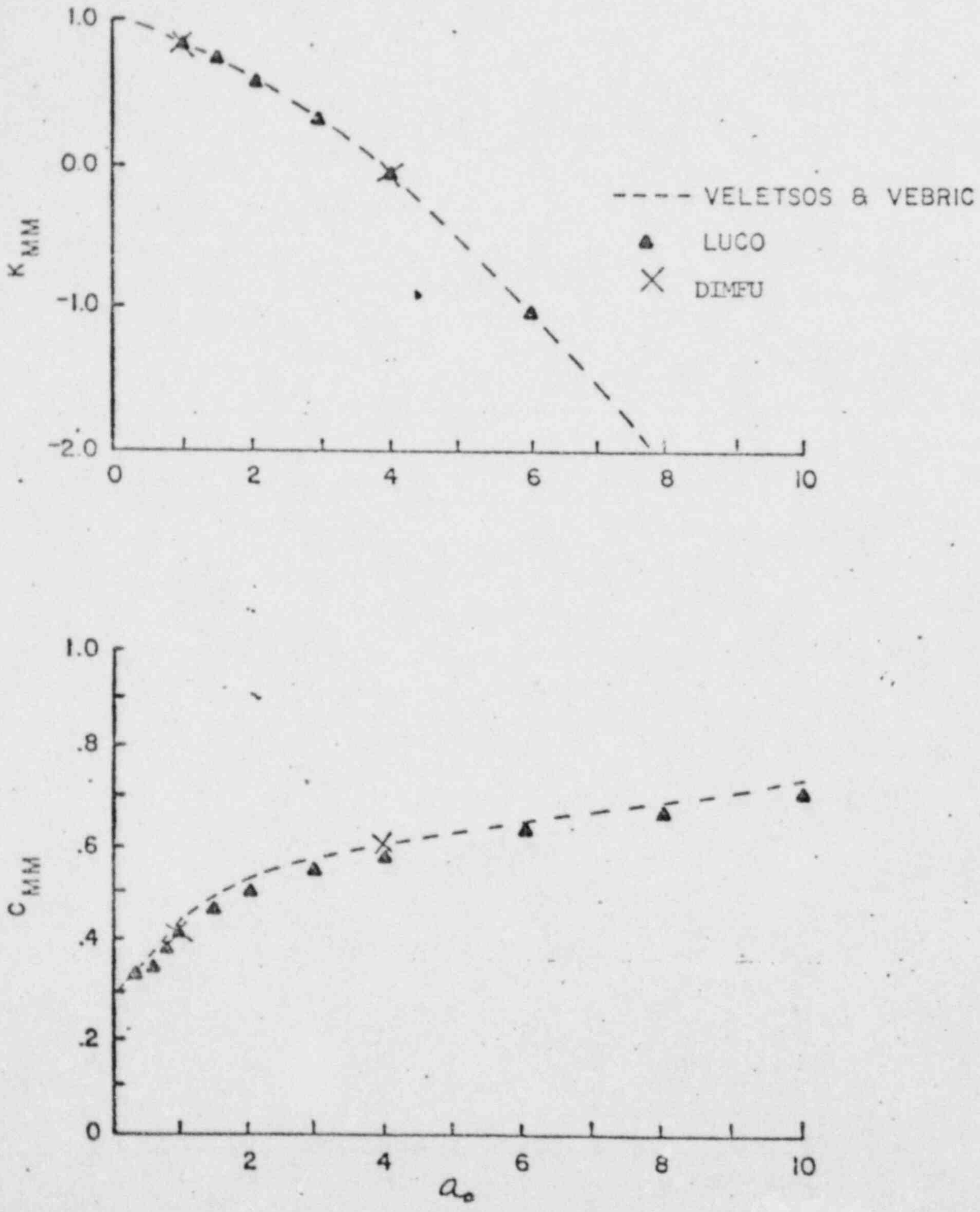


FIGURE 220.15-7
ROCKING IMPEDANCE FUNCTION FOR
VISCOELASTIC HALF-SPACE (VOIGT MODEL)

DIMFU VS. REFS. 3 AND 4

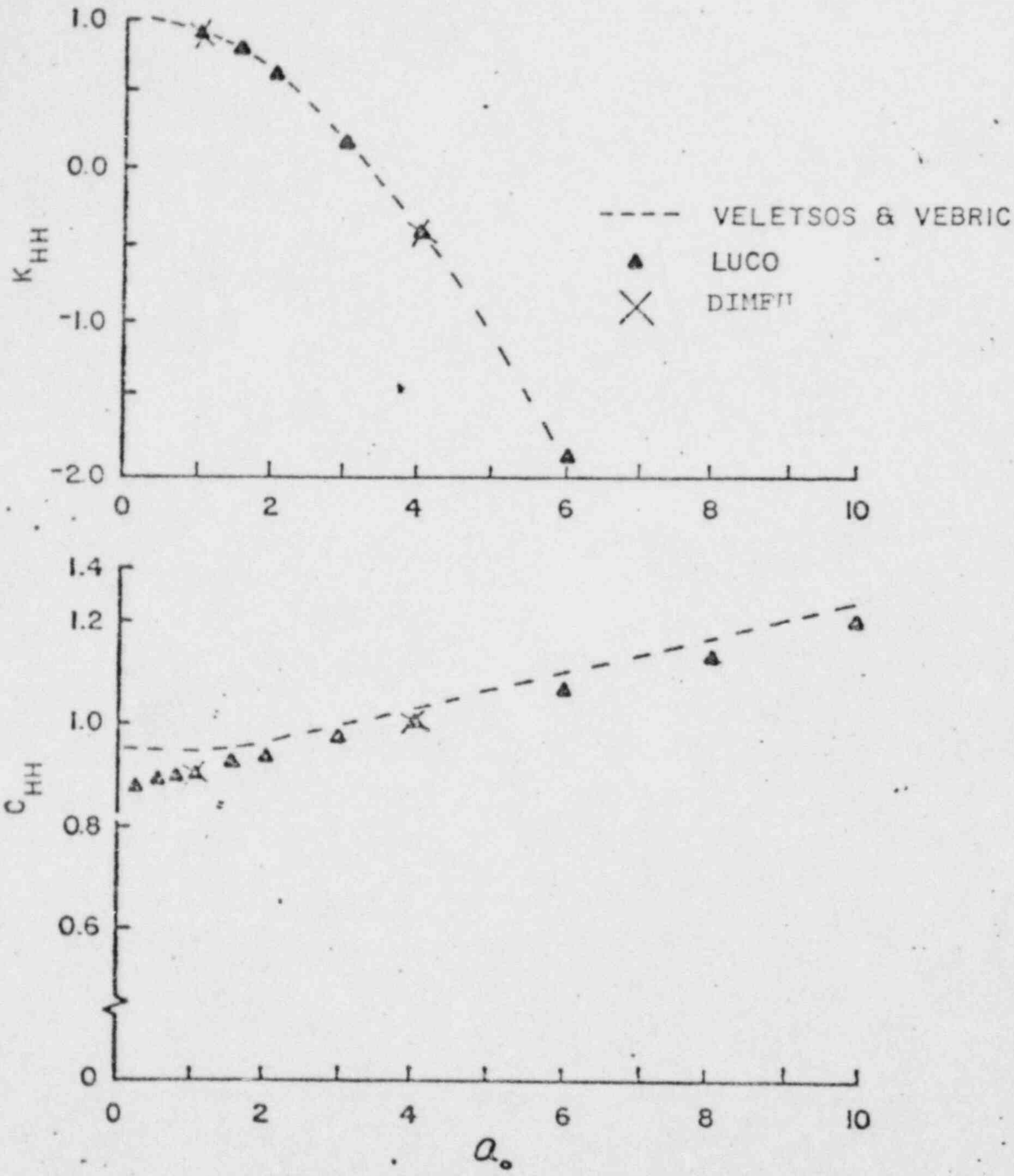


FIGURE 220.15-8

FIGURE 220.15-8
HORIZONTAL IMPEDANCE FUNCTION FOR
VISCOELASTIC HALF SPACE (VOIGT MODEL)
DIMFU VS. REFS. 3 AND 4

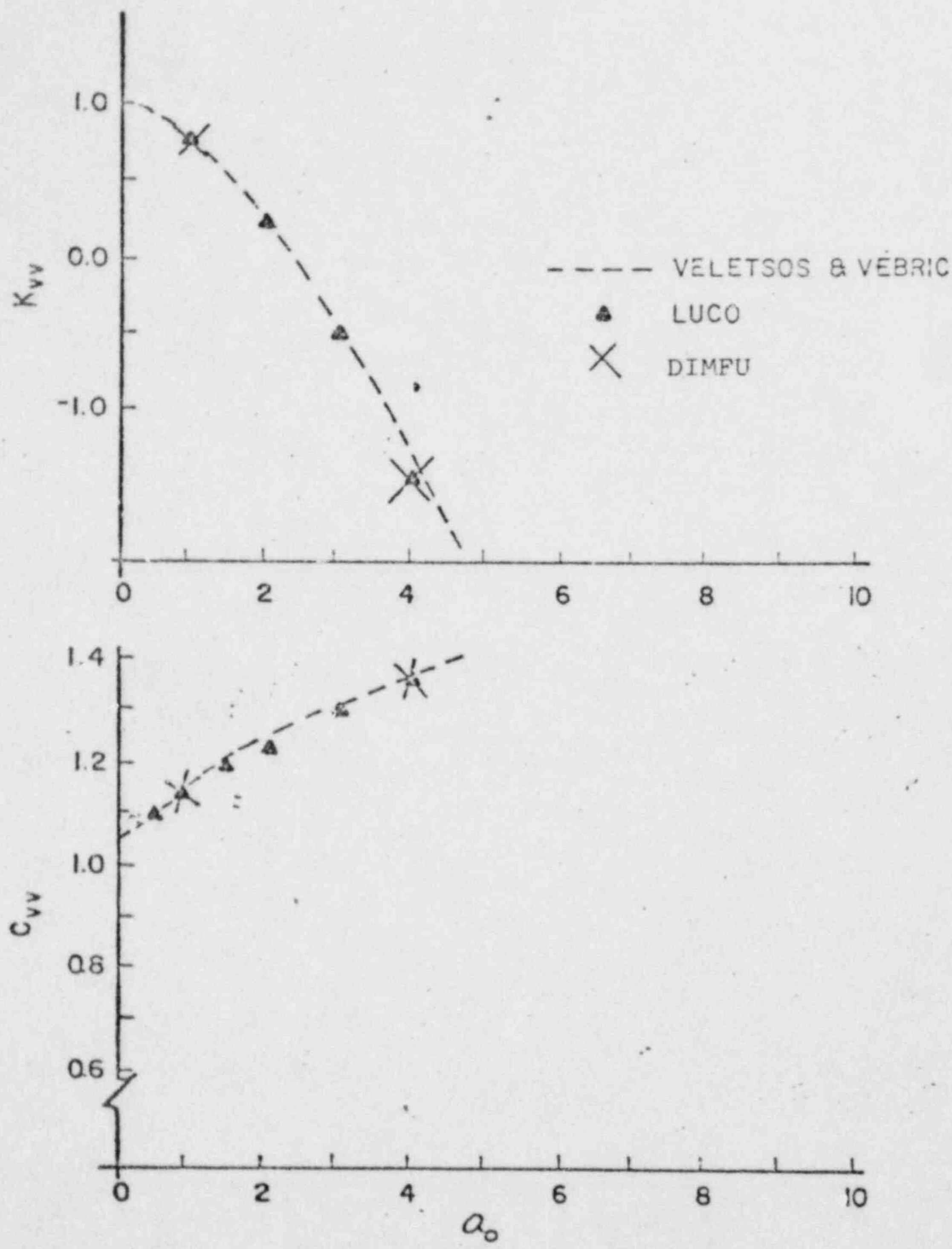


FIGURE 220.15-9
VERTICAL IMPEDANCE FUNCTION FOR
VISCOELASTIC HALF SPACE (VOIGT MODEL)

DIMFU VS. REFS. 3 AND 4

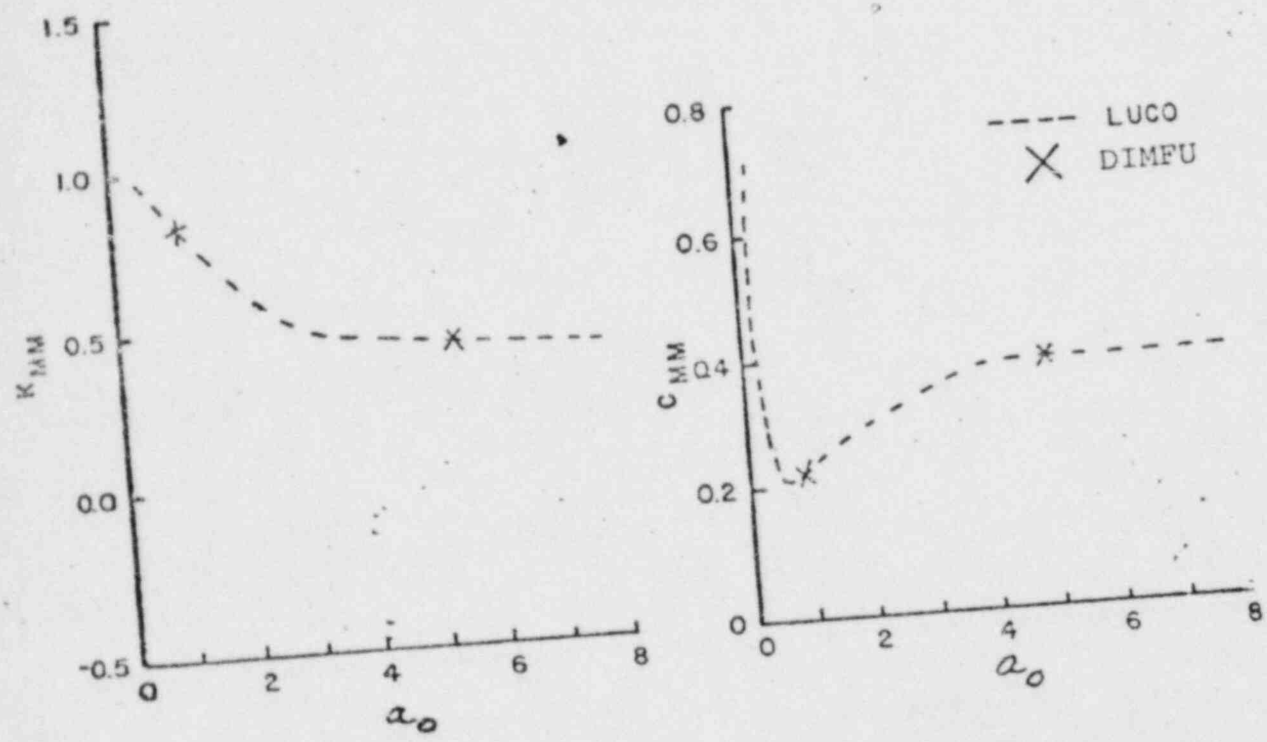


FIGURE 220.15-10
ROCKING IMPEDANCE FUNCTION FOR A
LAYERED HYSTERETICALLY DAMPED MEDIUM
DIMFU VS. REFS. 3 AND 4

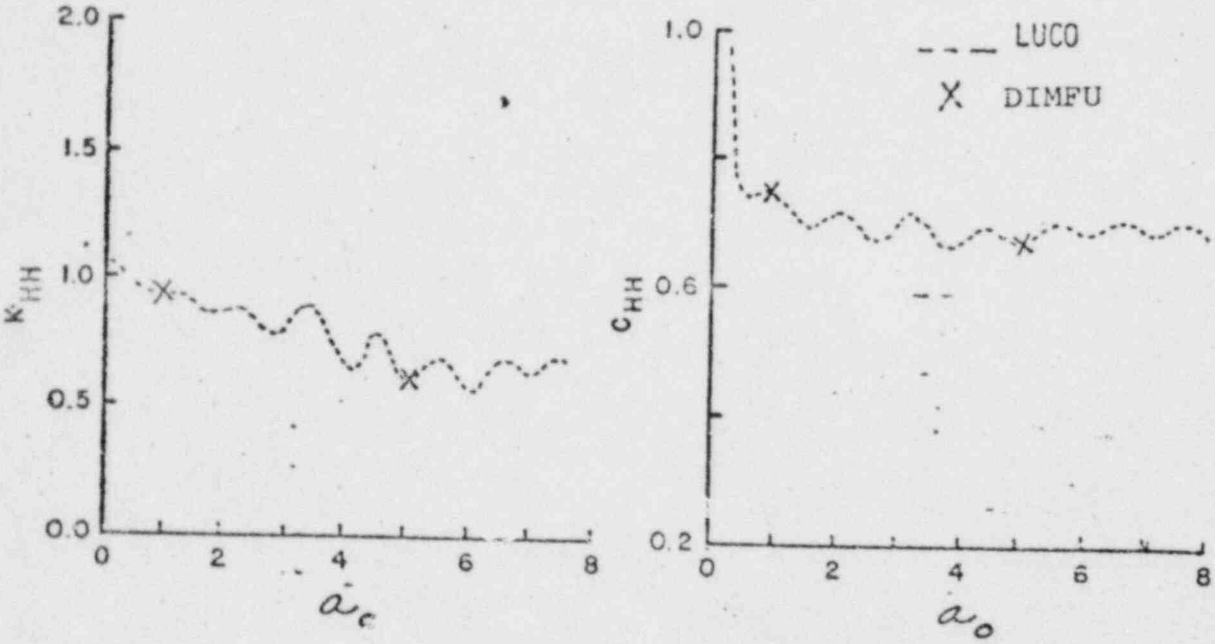


FIGURE 20.15-11
HORIZONTAL IMPEDANCE FUNCTION FOR A
LAYERED HYSTERETICALLY DAMPED MEDIUM

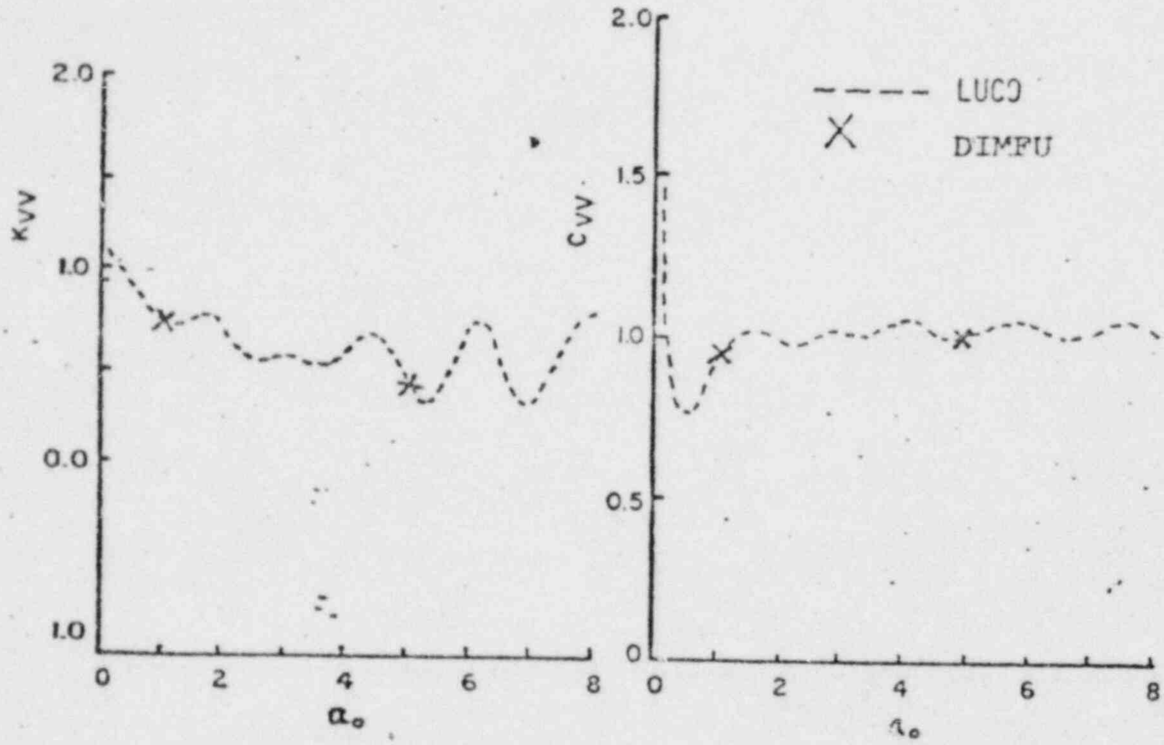


FIGURE 220.15-12
VERTICAL IMPEDANCE FUNCTION FOR A
LAYERED HYSTERETICALLY DAMPED MEDIUM
DIMFU VS REFS 3 AND 4

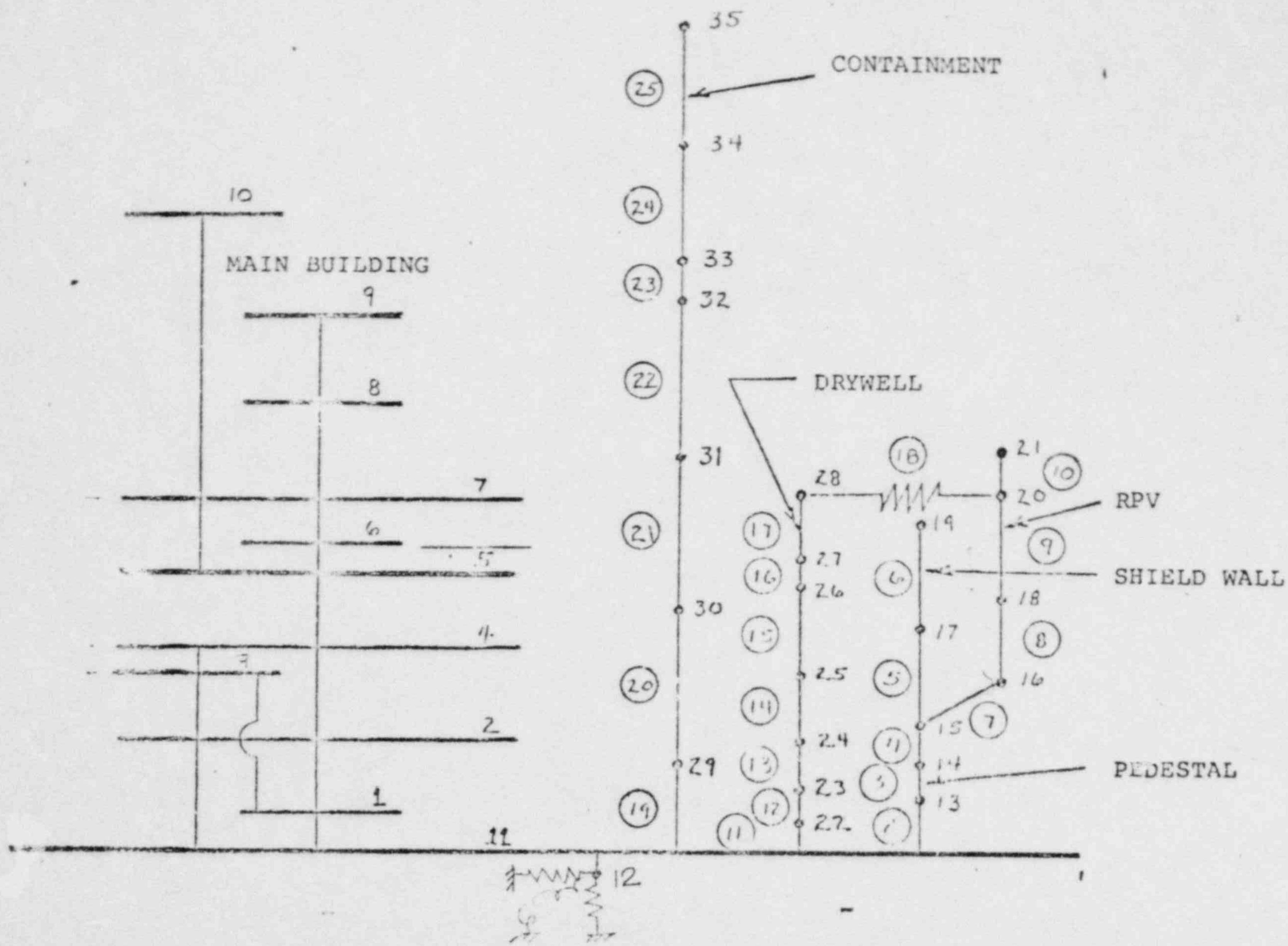


FIGURE 220.15-13
HORIZONTAL STRUCTURAL MODEL

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SOIL SPRING SSI NORTH-SOUTH S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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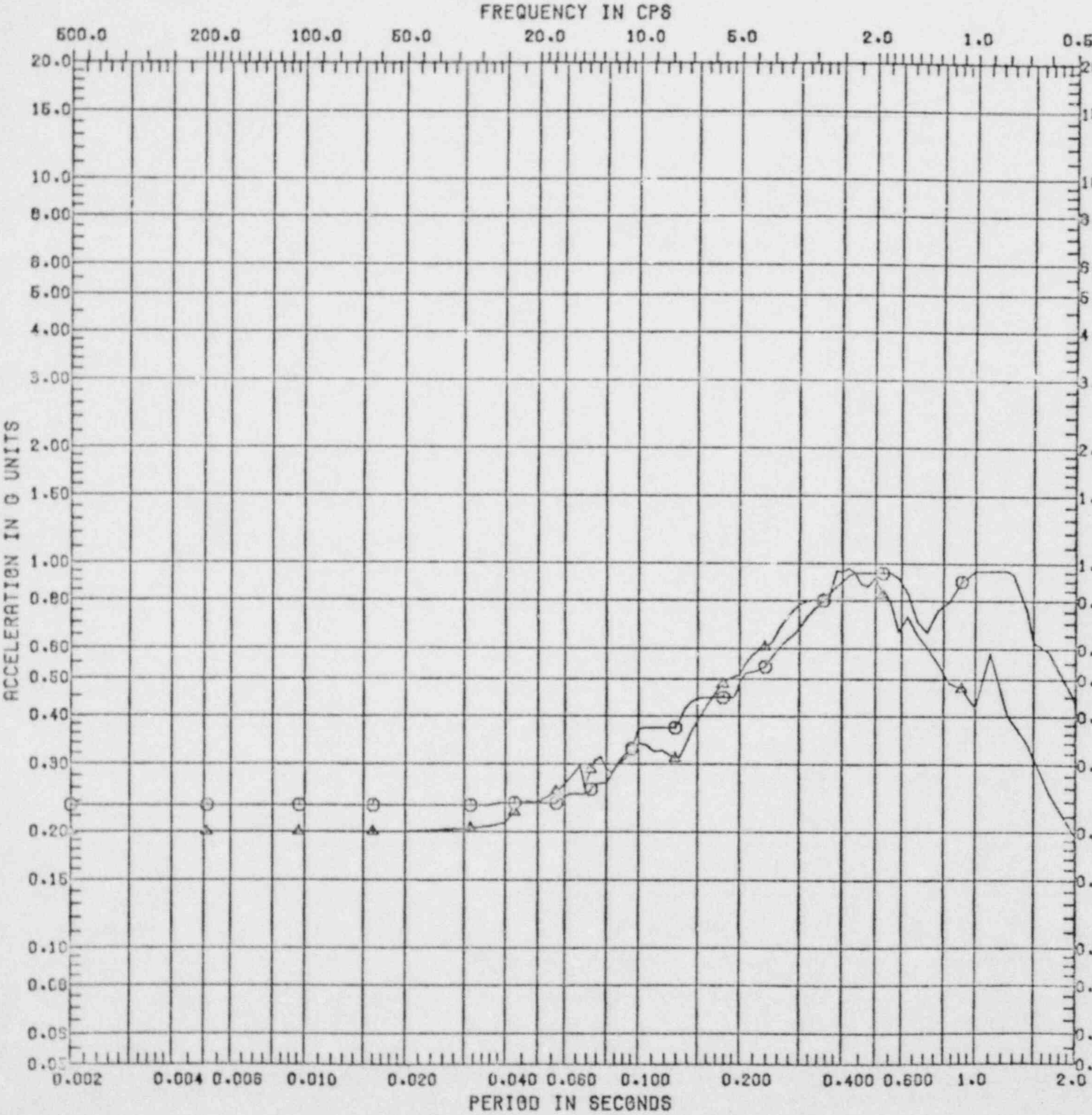

○ WIDENEDSSE DESIGN BASIS SSE INPUT WID
▲ NEWSSE NEW SSE LOAD 4536-29

FIGURE 220.15-14

AUX-FUEL BLDG ELEV 712 HORIZONTAL WALL 1712HB

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SOIL SPRING SSI EAST-WEST S/R SRD
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PROJECT NO. 4536-29
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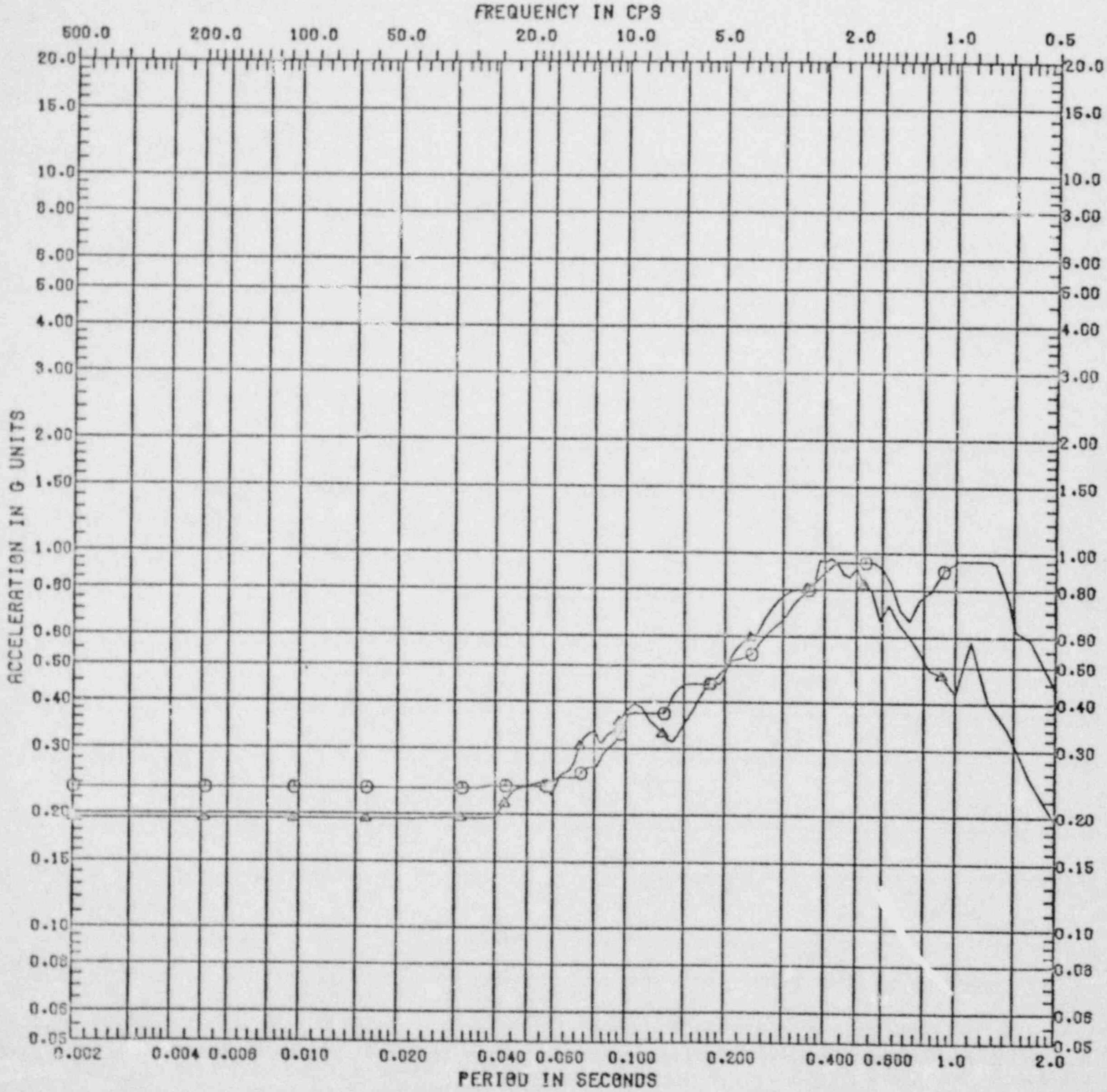

⊙ WIDENEDSSE DESIGN BASIS SSE INPUT WID

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FIGURE 220.15-15

AUX-FUEL BLDG ELEV 712 HORIZONTAL WALL 1712H0

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CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO 4536-29
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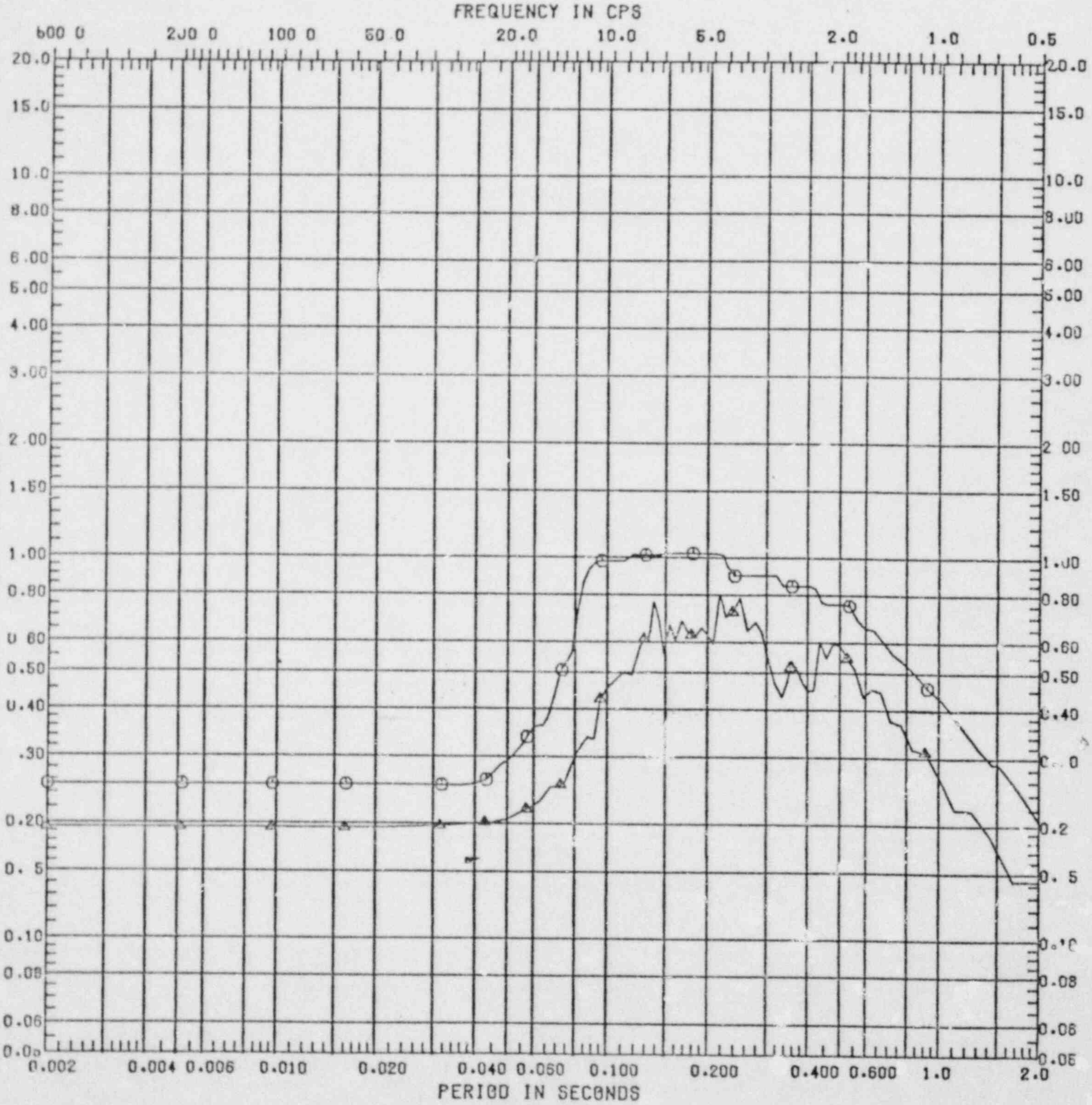

⊙ WIDENEDSSE DESIGN BASIS SSE INPUT WID

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FIGURE 220.15-16

AUX-FUEL BLDG ELEV 712 VERTICAL WALL 1712VH

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SOIL SPRING SSI NORTH-SOUTH S/R SAD
ENVELOPED SPECTRA
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PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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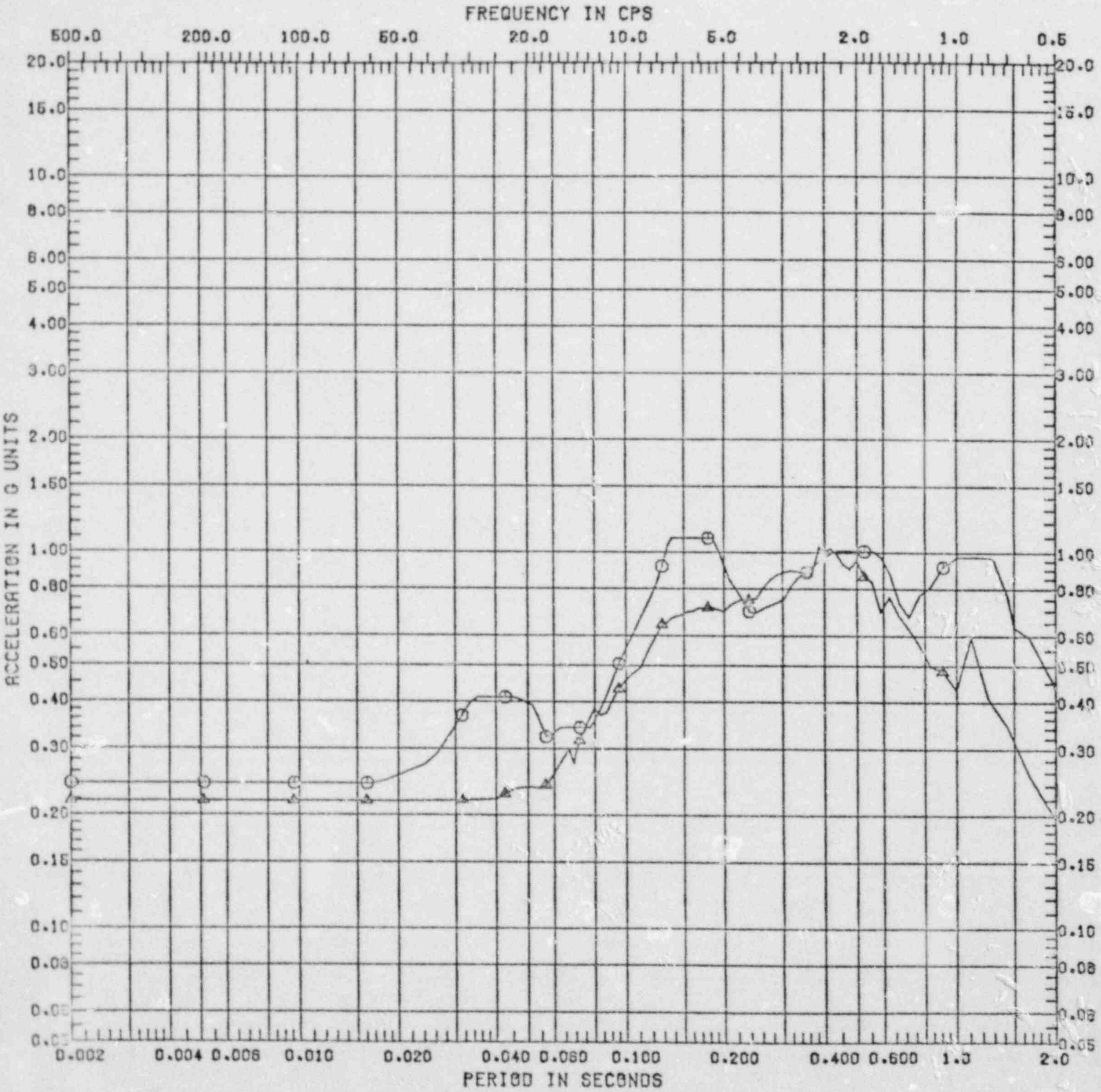

○ WIDENEDSSE DESIGN BASIS SSE INPUT WID
▲ NEWSSE NEW SSE LOAD 4536-29

FIGURE 220.15-17

AUX-FUEL BLDG ELEV 762 HORIZONTAL WALL 1762H8

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ENGINEERS

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SOIL SPRING SSI EAST-WEST S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
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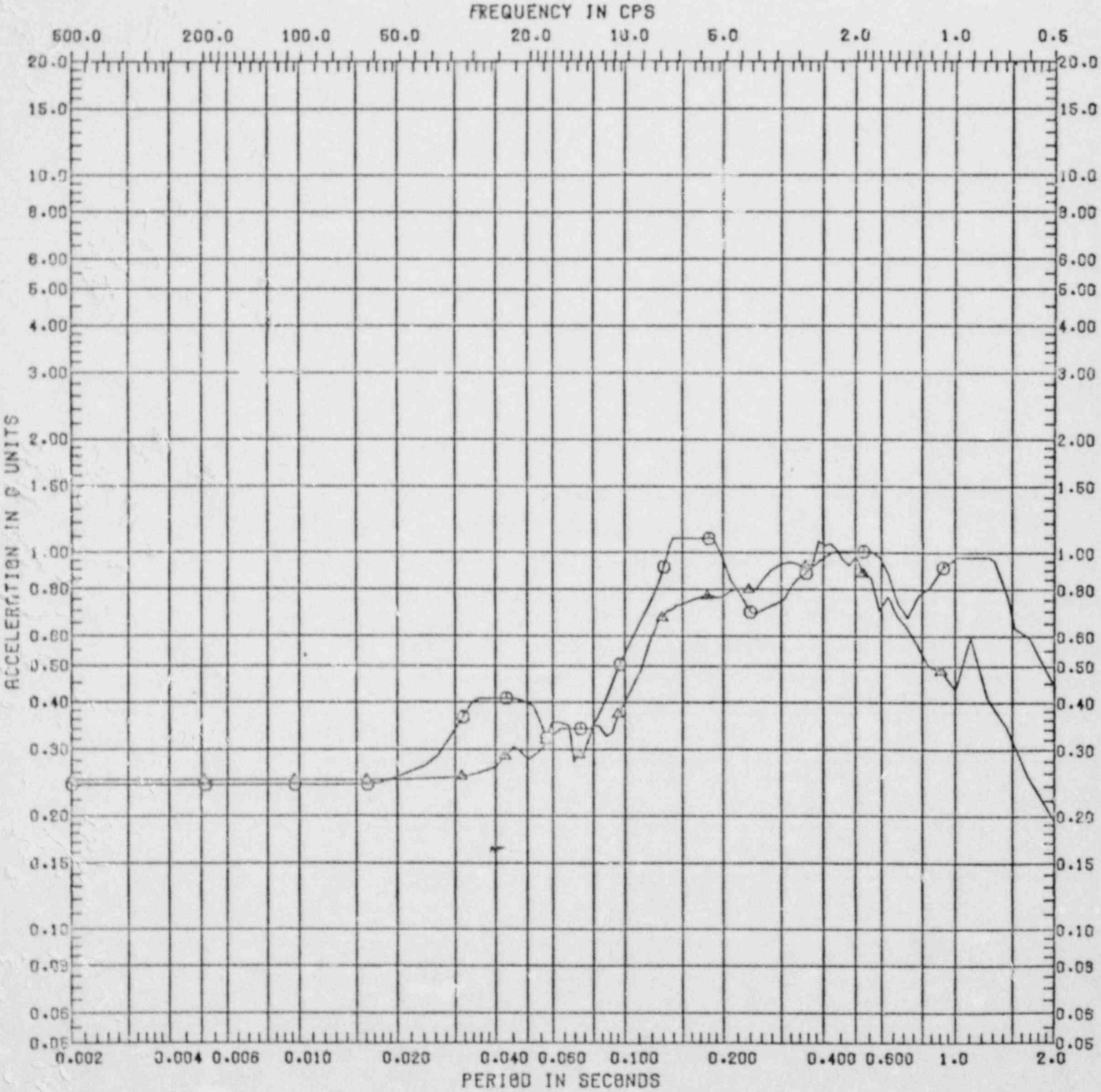

O WIDENEDSSE DESIGN BASIS SSE INPUT WID
▲ NEWSSE NEW SSE LOAD 4536-29

FIGURE 220.15-18

AUX-FUEL BLDG ELEV 762 HORIZONTAL WALL 1762H8

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SOIL SPRING SSI EAST-WEST S/R SAU
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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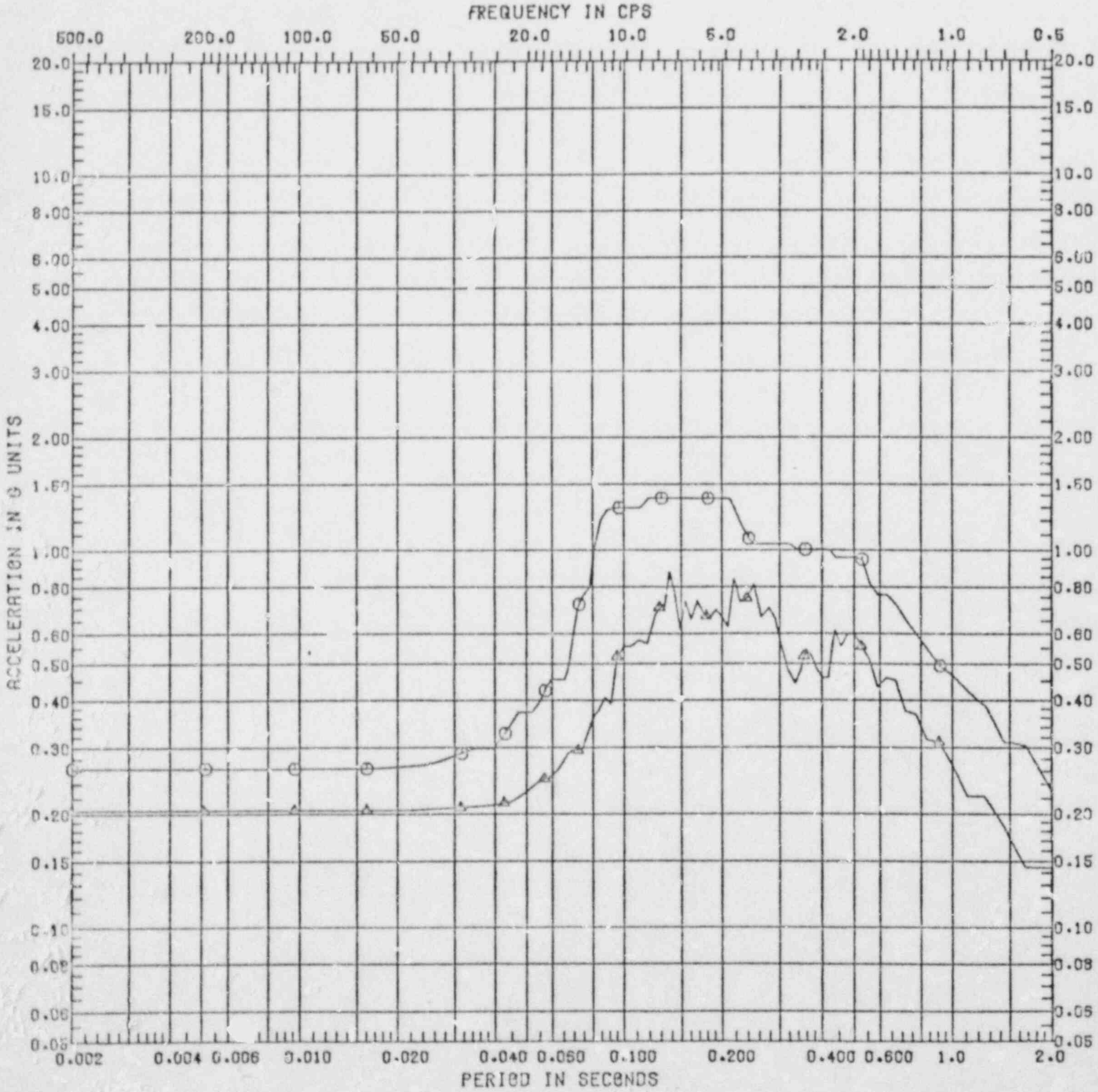

⊙ WIDENEDSSE DESIGN BASIS SSE INPUT WID
▲ NEWSSE NEW SSE LOAD 4536-29

FIGURE 220.15-19

AUX-FUEL BLDG ELEV 762 VERTICAL WALL 1762VH

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SOIL SPRING SSI NORTH-SOUTH S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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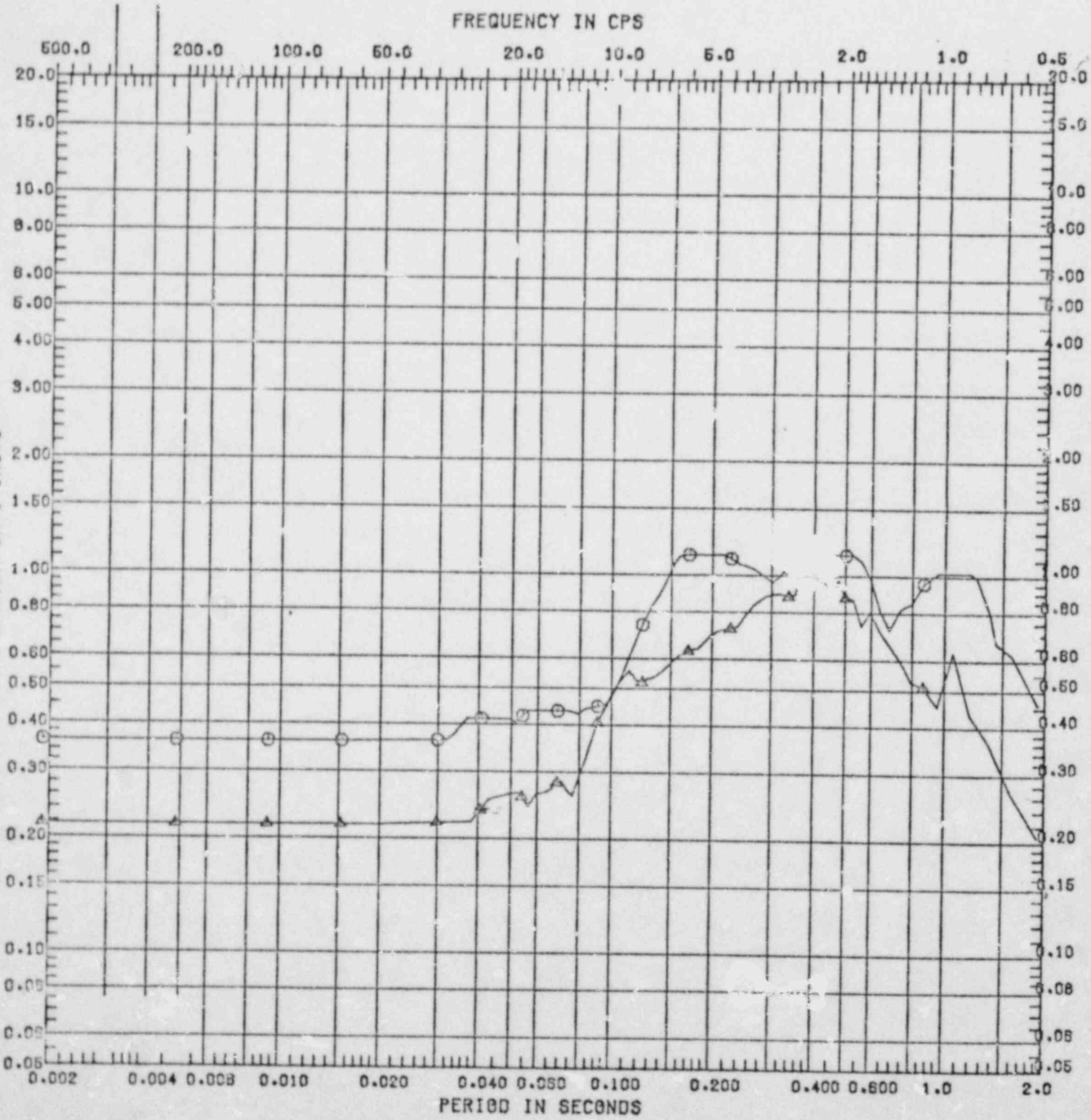

⊙ WIDENEDSSE DESIGN BASIS SSE INPUT WID

▲ NEWSSE NEW SSE LOAD 4536-29

FIGURE 220.15-20

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ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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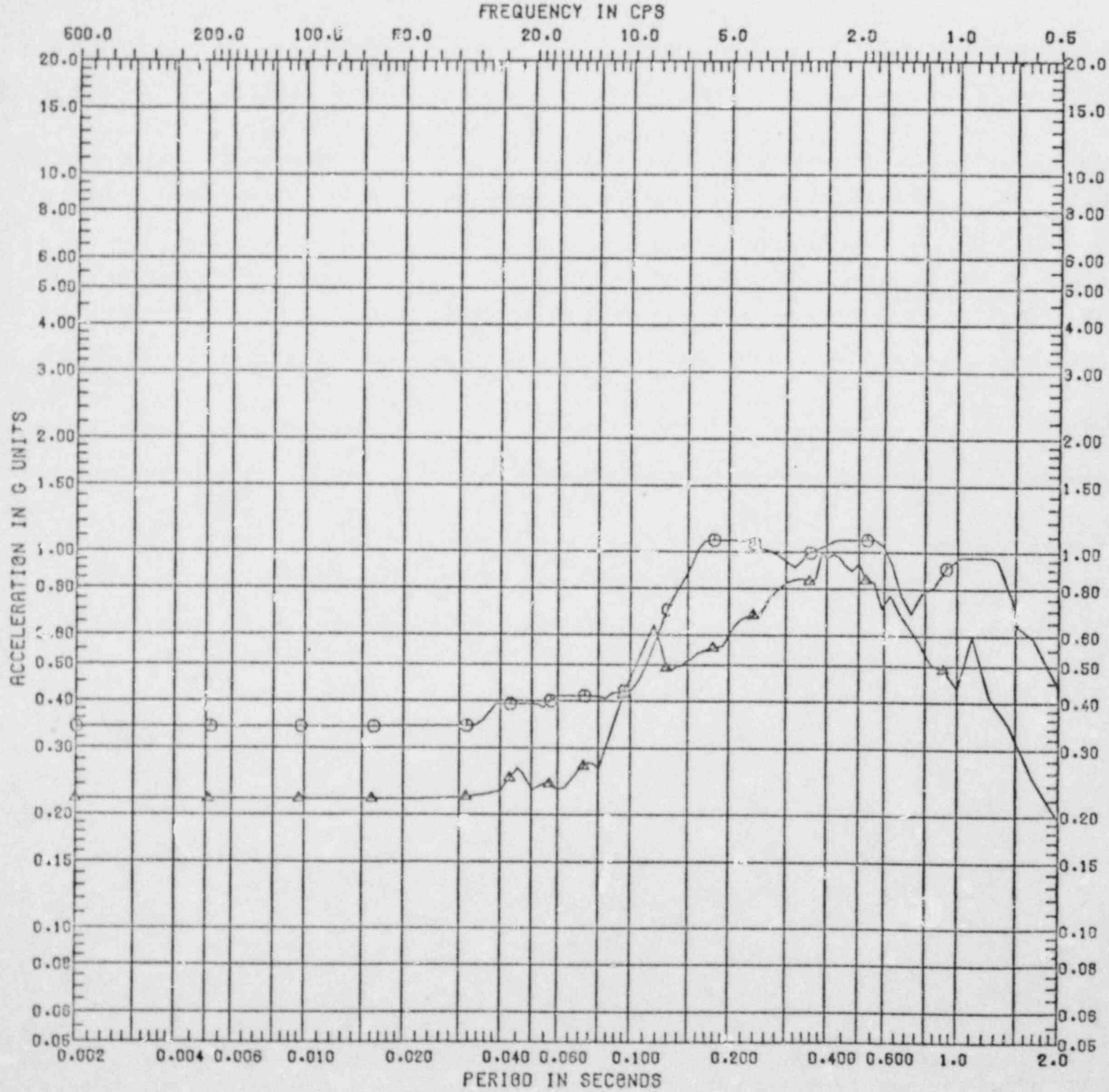

⊙ WIDENEDSSE DESIGN BASIS SSE INPUT WID

▲ NEWSSE NEW SSE LOAD 4536-29

FIGURE 220.15-21

CONTAINMENT WALL ELEV 803 HORIZONTAL WALL 2803H8

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SOIL SPRING SSI EAST-WEST S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
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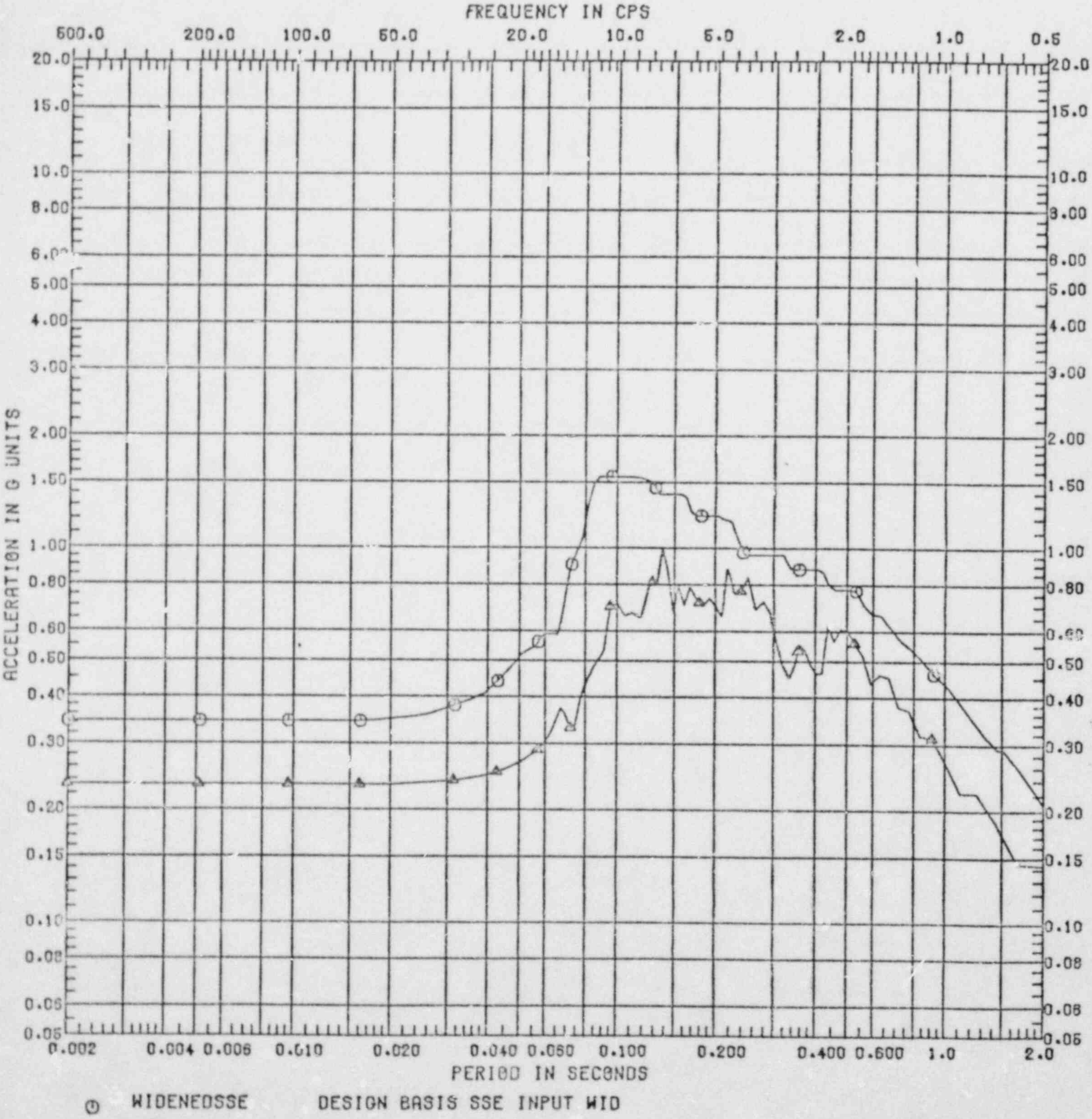

⊙ WIDENEDSSE DESIGN BASIS SSE INPUT WID

▲ NEWSSE NEW SSE LOAD 4536-29

FIGURE 220.15-22

CONTAINMENT WALL ELEV 803 VERTICAL WALL 2803VW

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SOIL SPRING SSI NORTH-SOUTH S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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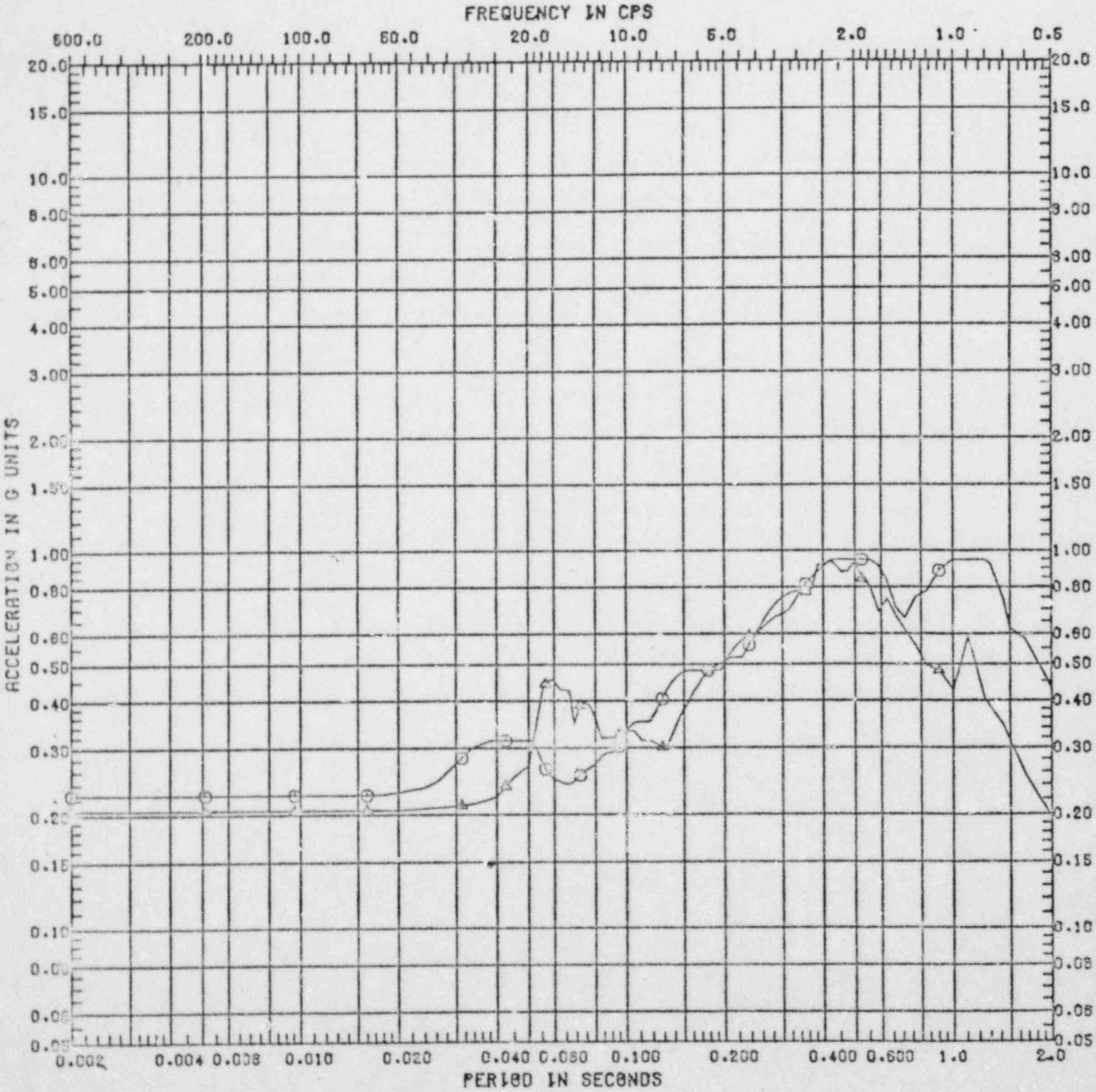

⊙ WIDENEDSSE DESIGN BASIS SSE INPUT WID
△ NEWSSE NEW SSE LOAD 4536-29

FIGURE 220.15-23

SHIELD WALL ELEV 743 HORIZONTAL WALL 4743HB

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ENGINEERS

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SOIL SPRING SSI EAST-WEST S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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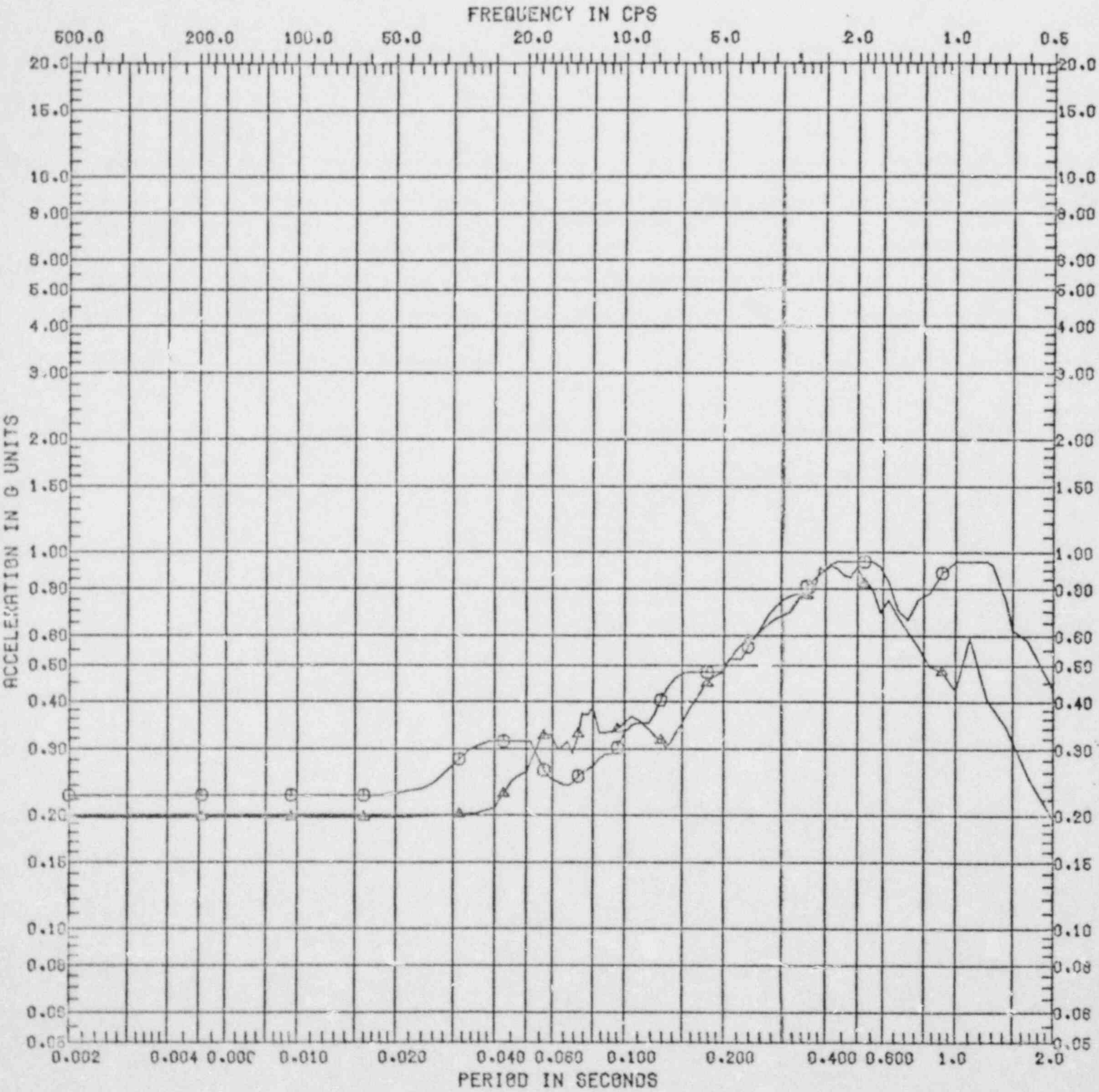

⊙ WIDENEDSSE DESIGN BASIS SSE INPUT WID
▲ NEWSSE NEW SSE LOAD 4536-29

FIGURE 220.15-24

SHIELD WALL ELEV 743 HORIZONTAL WALL 4743H8

SARGENT & LUNDY
ENGINEERS

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SOIL SPRING SSI EAST-WEST S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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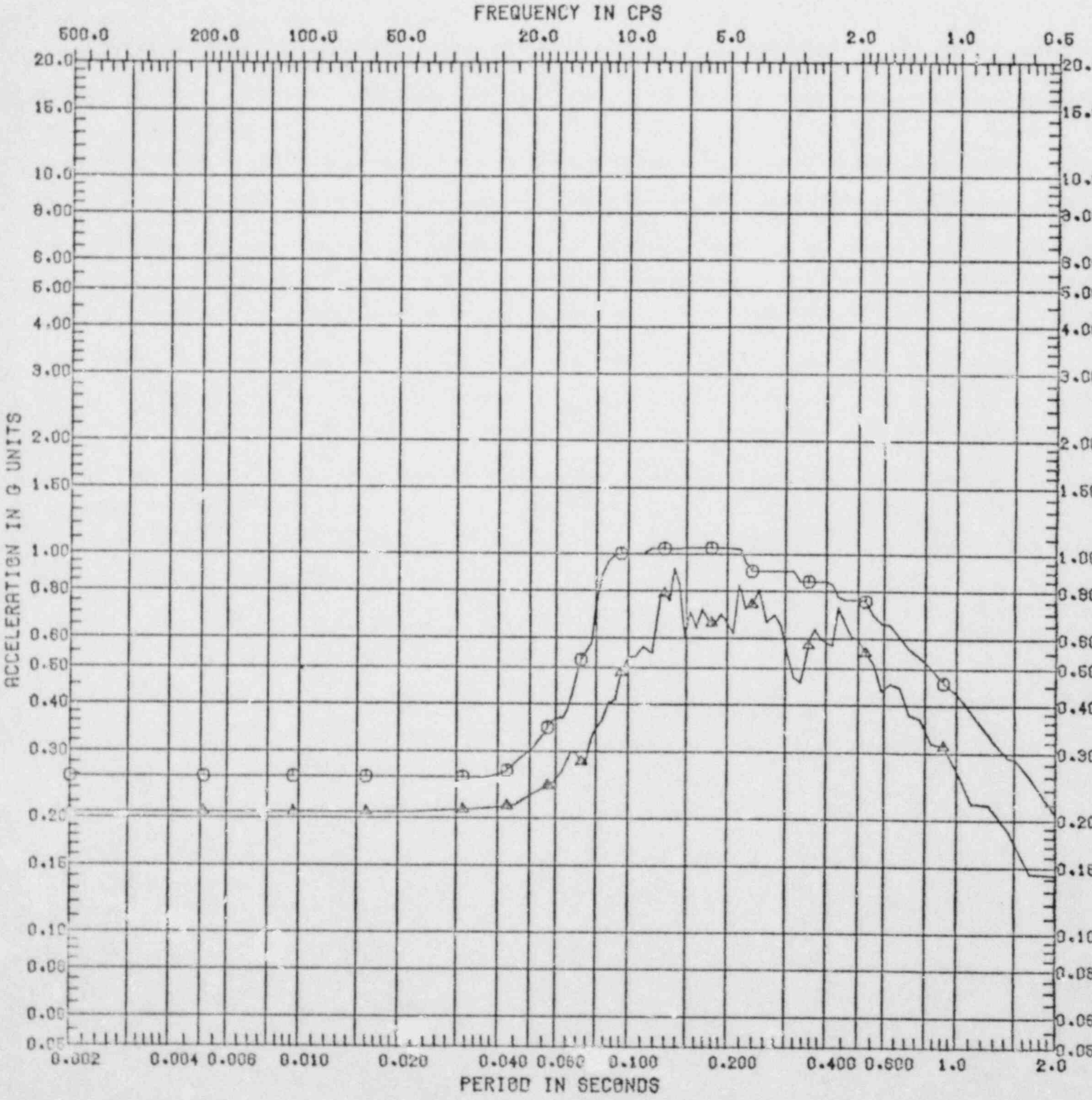

⊙ WIDENEDSSE DESIGN BASIS SSE INPUT WID

▲ NEWSSE NEW SSE LOAD 4536-29

FIGURE 220.15-25

SHIELD WALL ELEV 743 VERTICAL WALL 4743VW

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ENGINEERS

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SOIL SPRING SSI NORTH-SOUTH S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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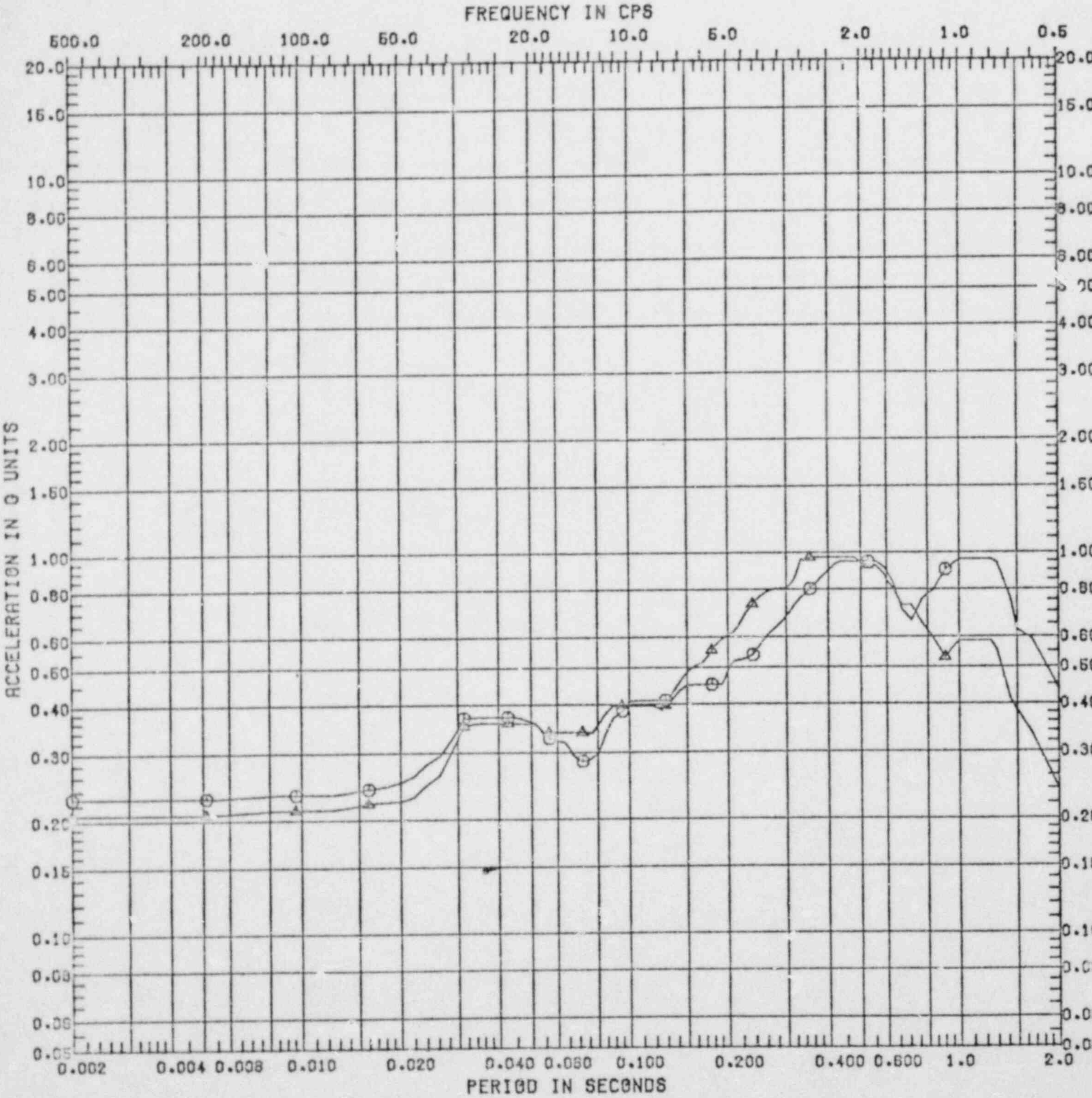

⊙ OBLEVELC LEVEL C

▲ NEWLEVELC LEVEL C

FIGURE 220.15-26

AUX-FUEL BLDG ELEV 712 HORIZONTAL WALL 1712HB

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ENGINEERS

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SOIL SPRING SSI EAST-WEST S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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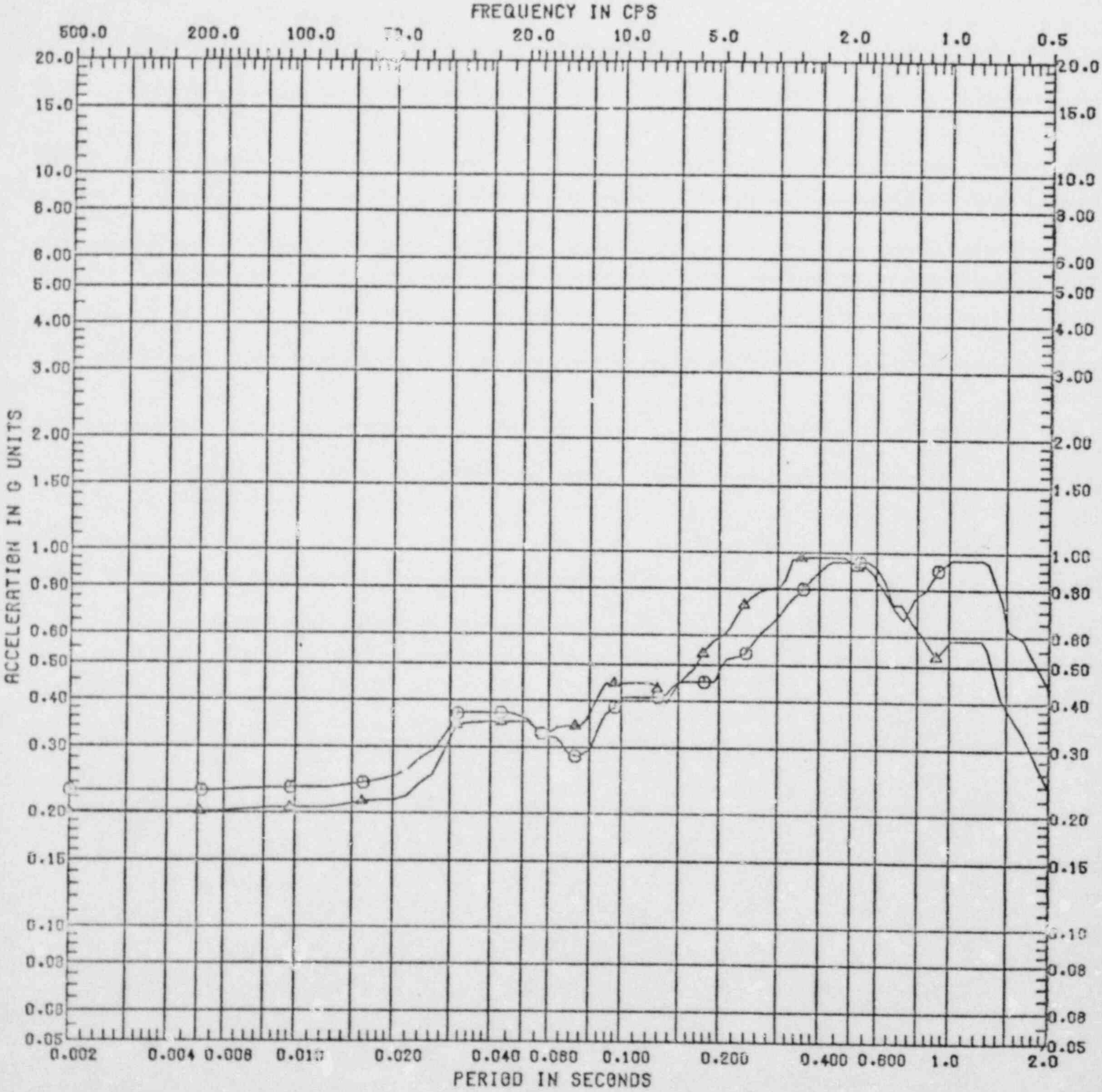

O OBLEVELC LEVEL C
▲ NEWLEVELC LEVEL C

FIGURE 220.15-27

AUX-FUEL BLDG ELEV 712 HORIZONTAL WALL 1712H8

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ENGINEERS

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SOIL SPRING SSI EAST-WEST S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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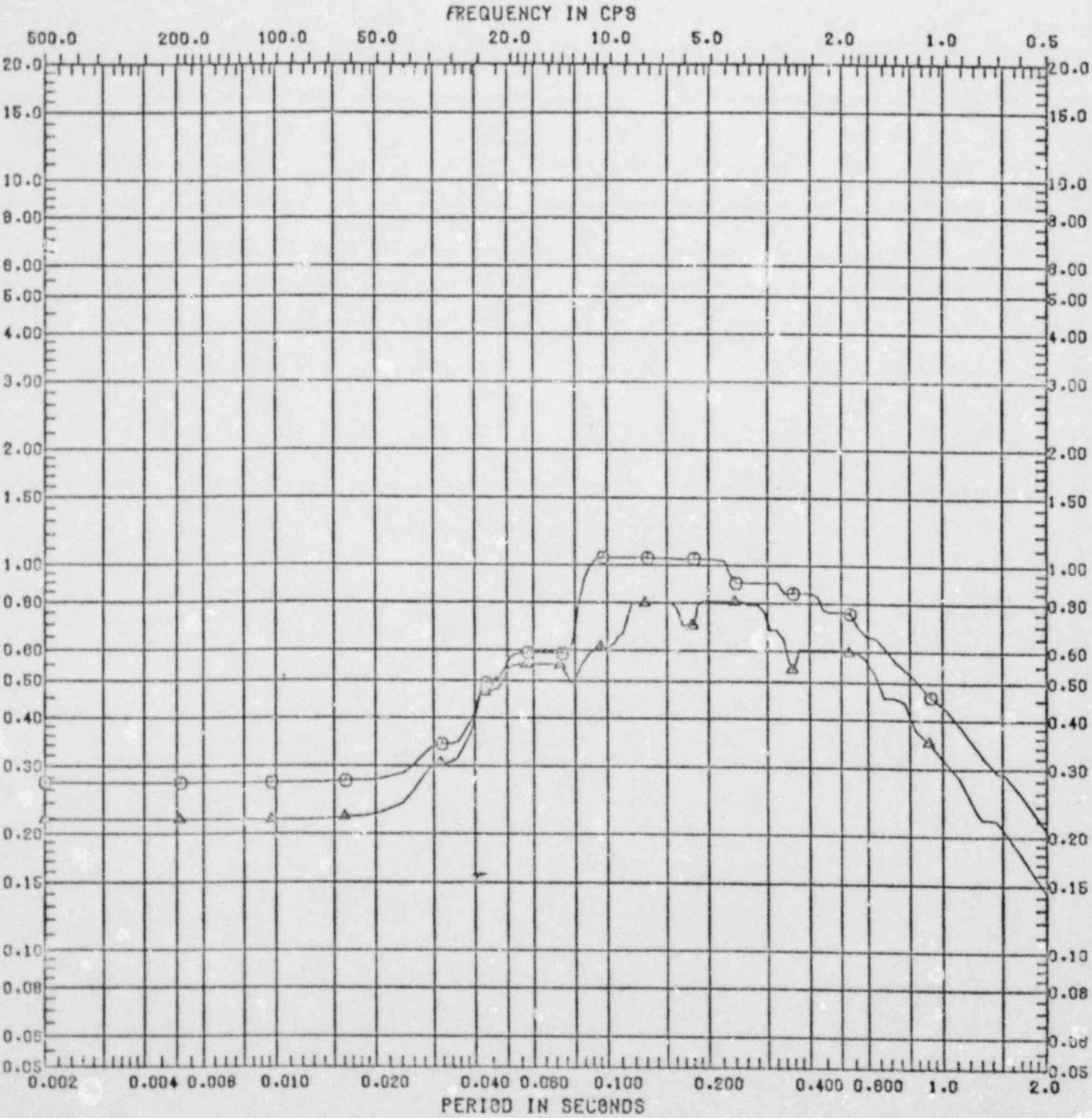

O	OBLEVELC	LEVEL C
▲	NEWLEVELC	LEVEL C

FIGURE 220.15-23

AUX-FUEL BLDG ELEV 712 VERTICAL WALL 1712VW

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ENGINEERS

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SOIL SPRING SSI NORTH-SOUTH S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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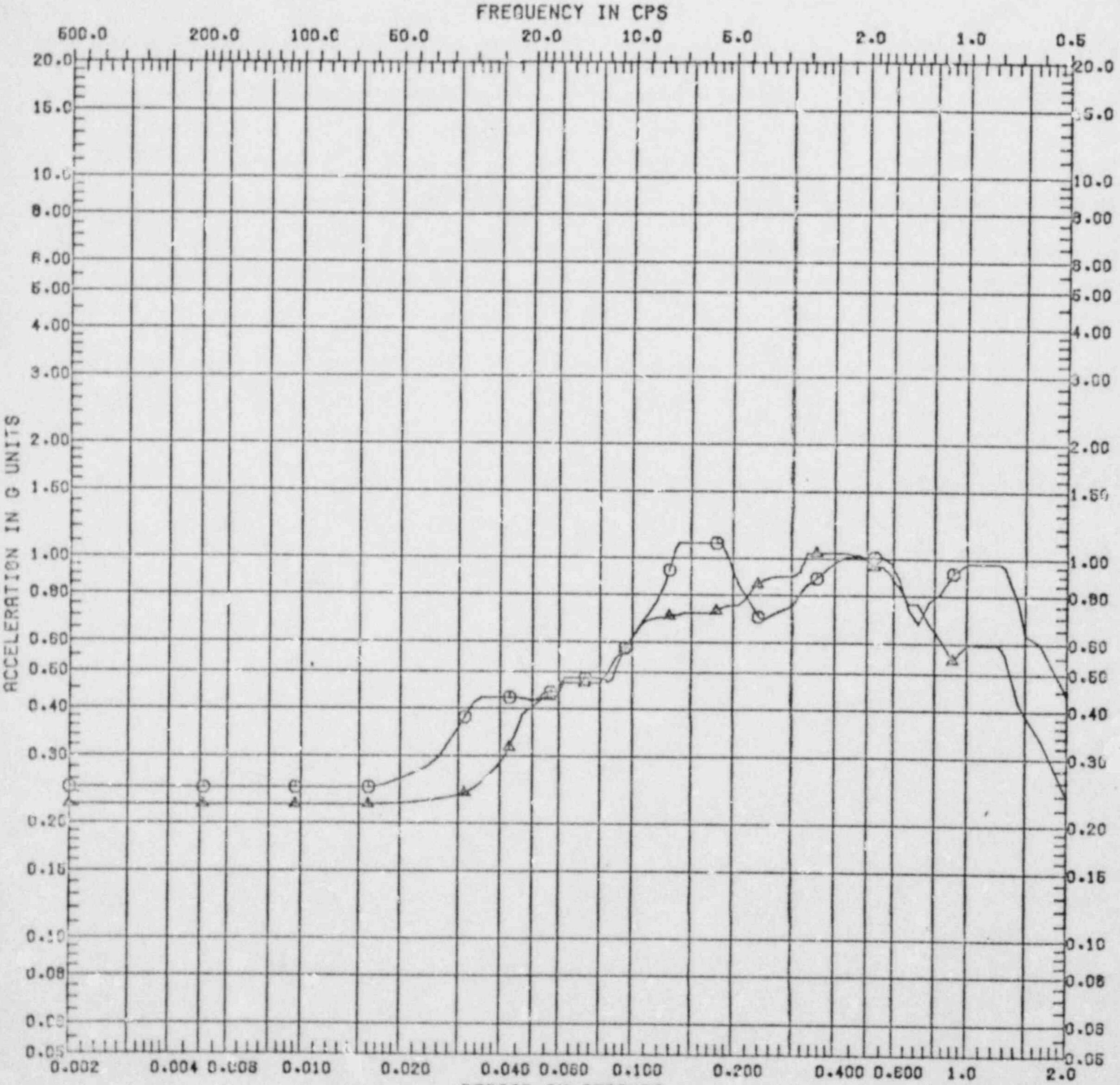

⊙	OBLEVELC	LEVEL C
▲	NEWLEVELC	LEVEL C

FIGURE 220.15-29

AUX-FUEL BLDG ELEV 762 HORIZONTAL WALL 1762H8

SARGENT & LUNDY
ENGINEERS

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A522JK

SOIL SPRING SSI EAST-WEST S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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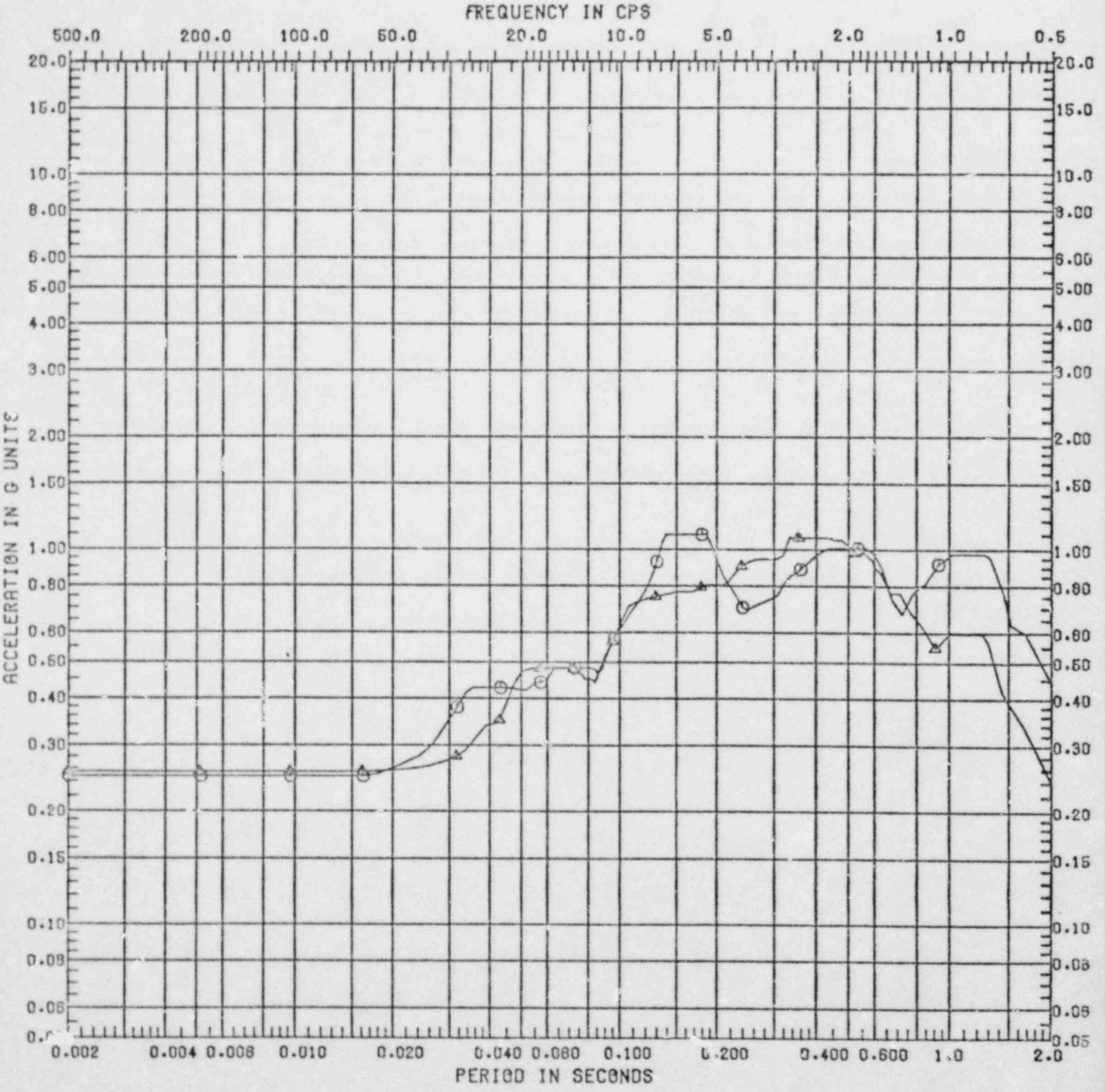

O DBLEVELC LEVEL C

▲ NEWLEVELC LEVEL C

FIGURE 220.15-30

AUX-FUEL BLDG ELEV 762 HORIZONTAL WALL 1762H8

SARGENT & LUNDY ENGINEERS

12 NOV 81
A522JK

SOIL SPRING SSI EAST-WEST S/R SAS
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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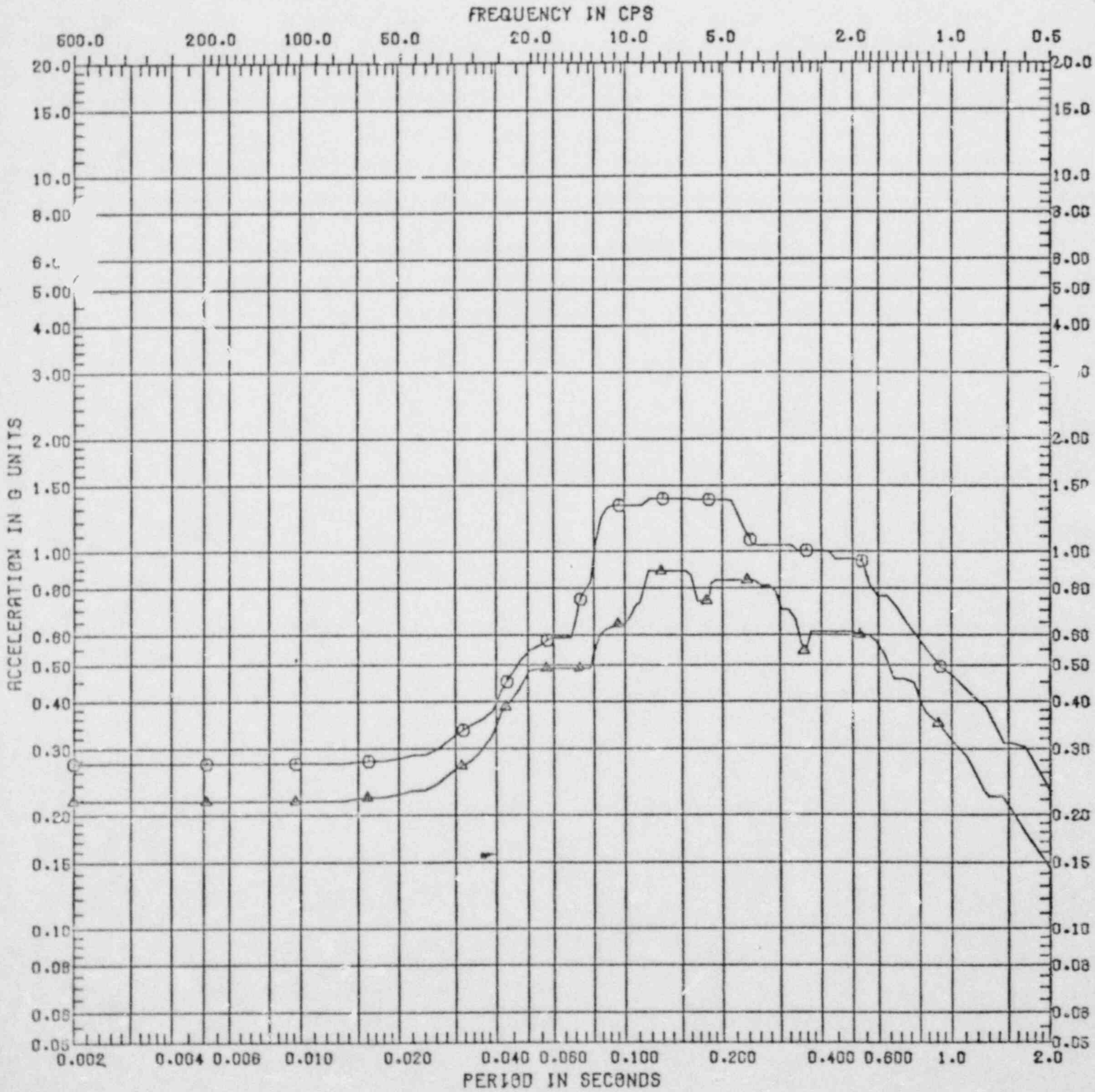

⊙ OBLEVELC LEVEL C

▲ NEWLEVELC LEVEL C

FIGURE 220.15-31

AUX-FUEL BLDG ELEV 762 VERTICAL WALL 1762VW

SARGENT & LUNDY
ENGINEERS

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SOIL SPRING SSI NORTH-SOUTH S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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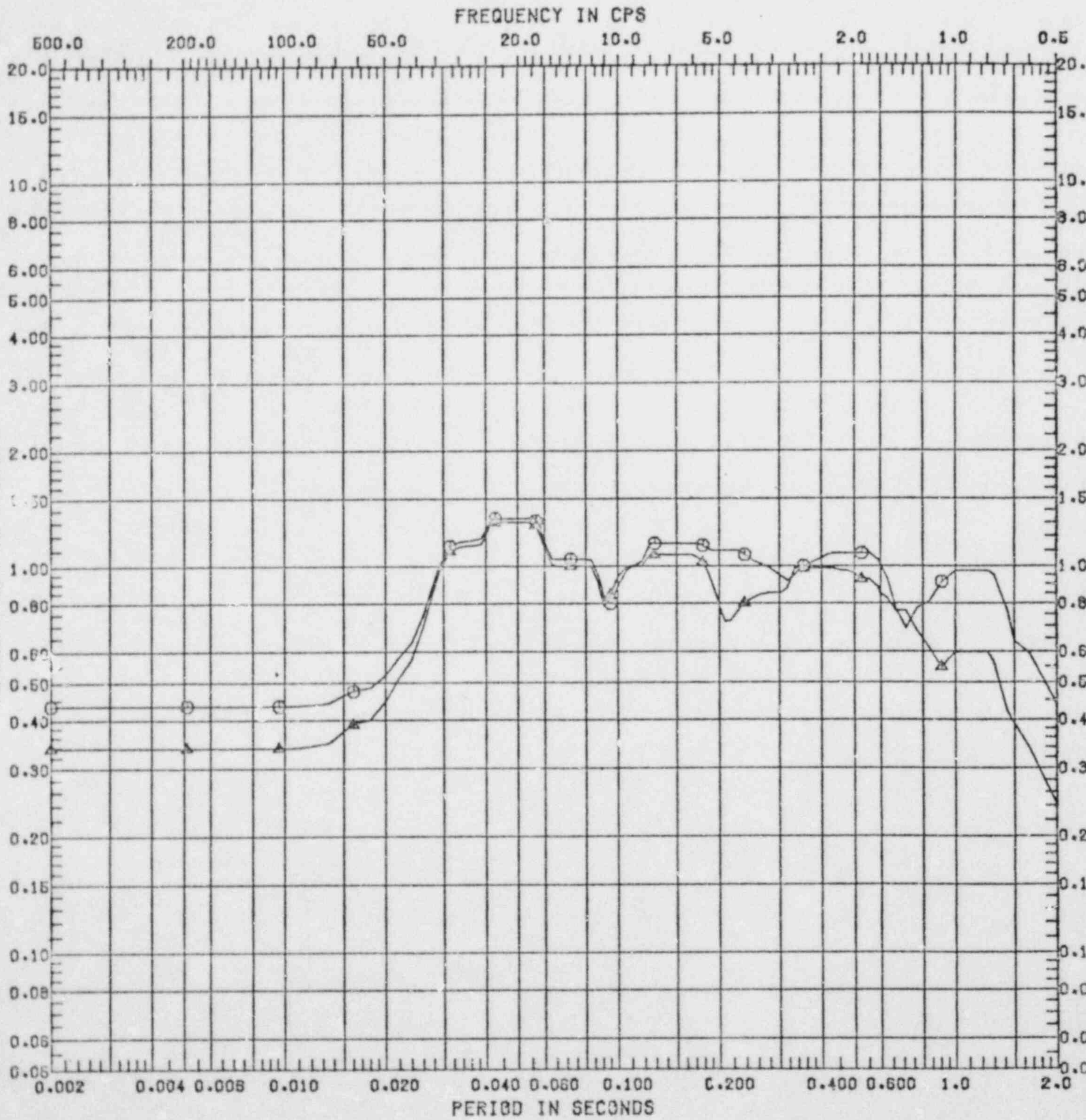

DBLEVELC LEVEL C

NEWLEVELC LEVEL C

FIGURE 220.15-32

CONTAINMENT WALL ELEV 803 HORIZONTAL WALL 2803H8

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ENGINEERS

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SOIL SPRING SSI EAST-WEST S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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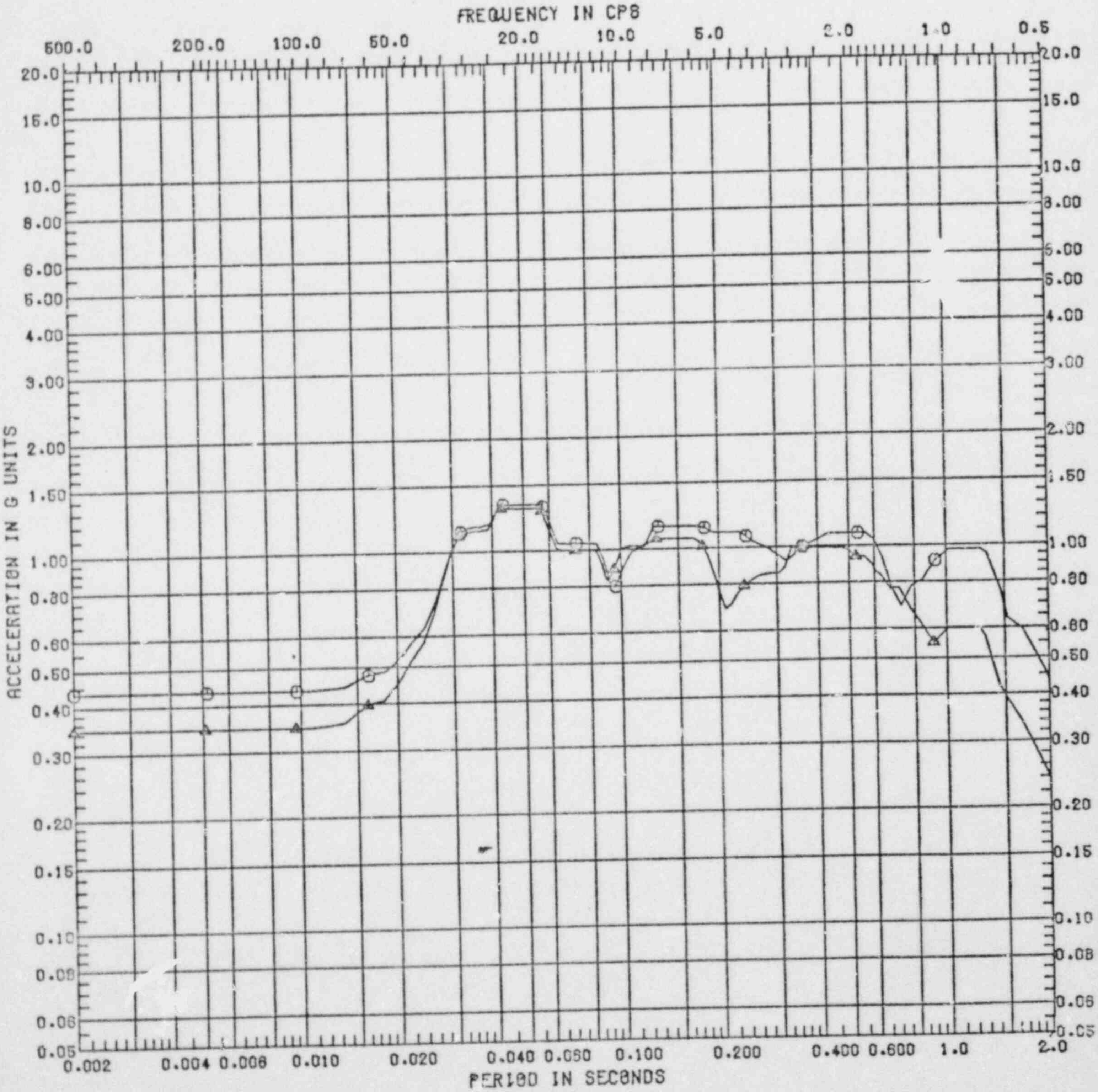

⊙ OBLEVELC — LEVEL C

▲ NEWLEVELC — LEVEL C

FIGURE 220.15-33

CONTAINMENT WALL ELEV 803 HORIZONTAL WALL 2803H8

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ENGINEERS

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SOIL SPRING SSI EAST-WEST S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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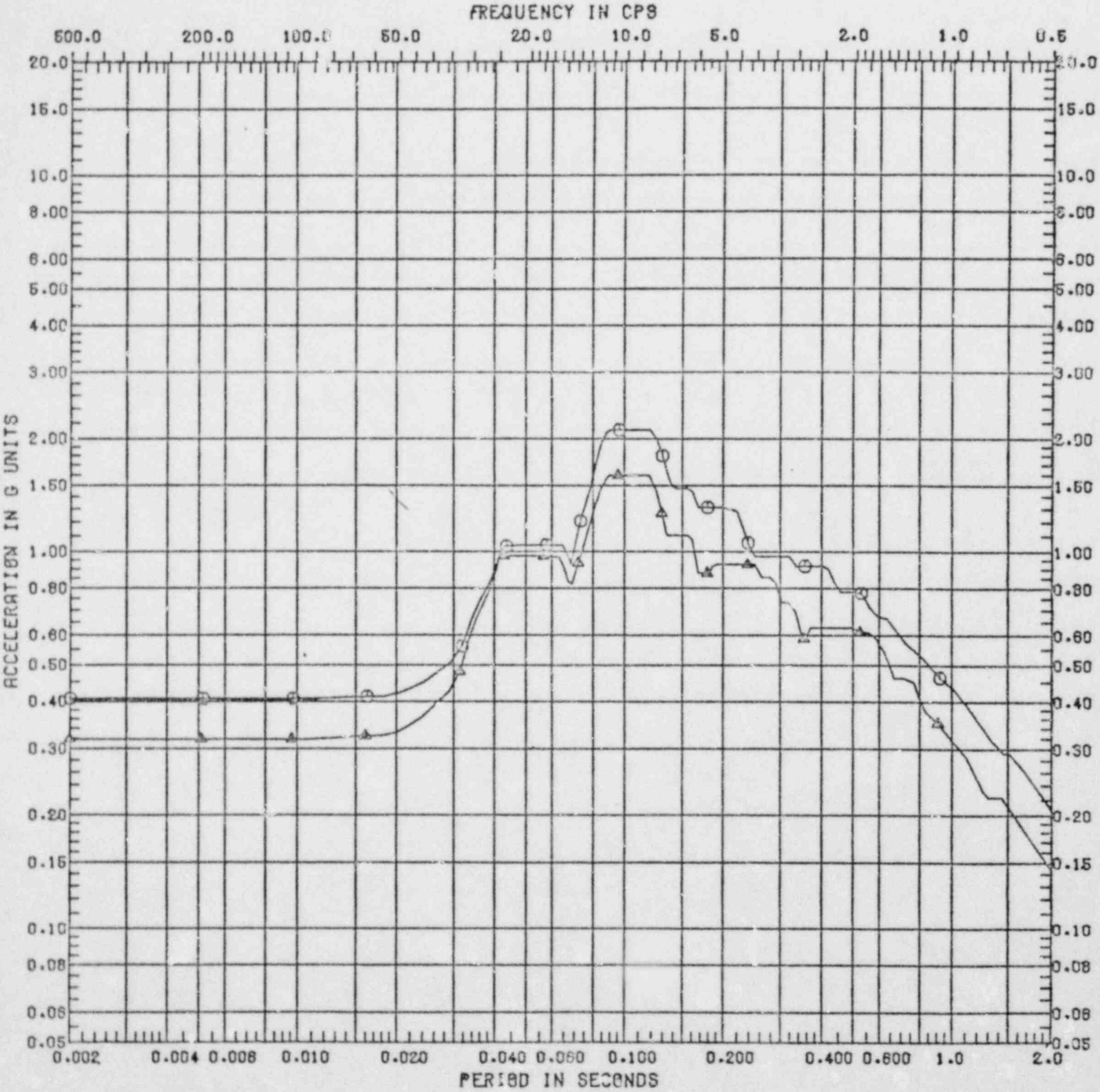

○ OBLEVELC LEVEL C

▲ NEWLEVELC LEVEL C

FIGURE 220.15-34

CONTAINMENT WALL ELEV 803 VERTICAL WALL 2803VW

SARGENT & LUNDY
ENGINEERS

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A817JK

SOIL SPRING SSI NORTH-SOUTH S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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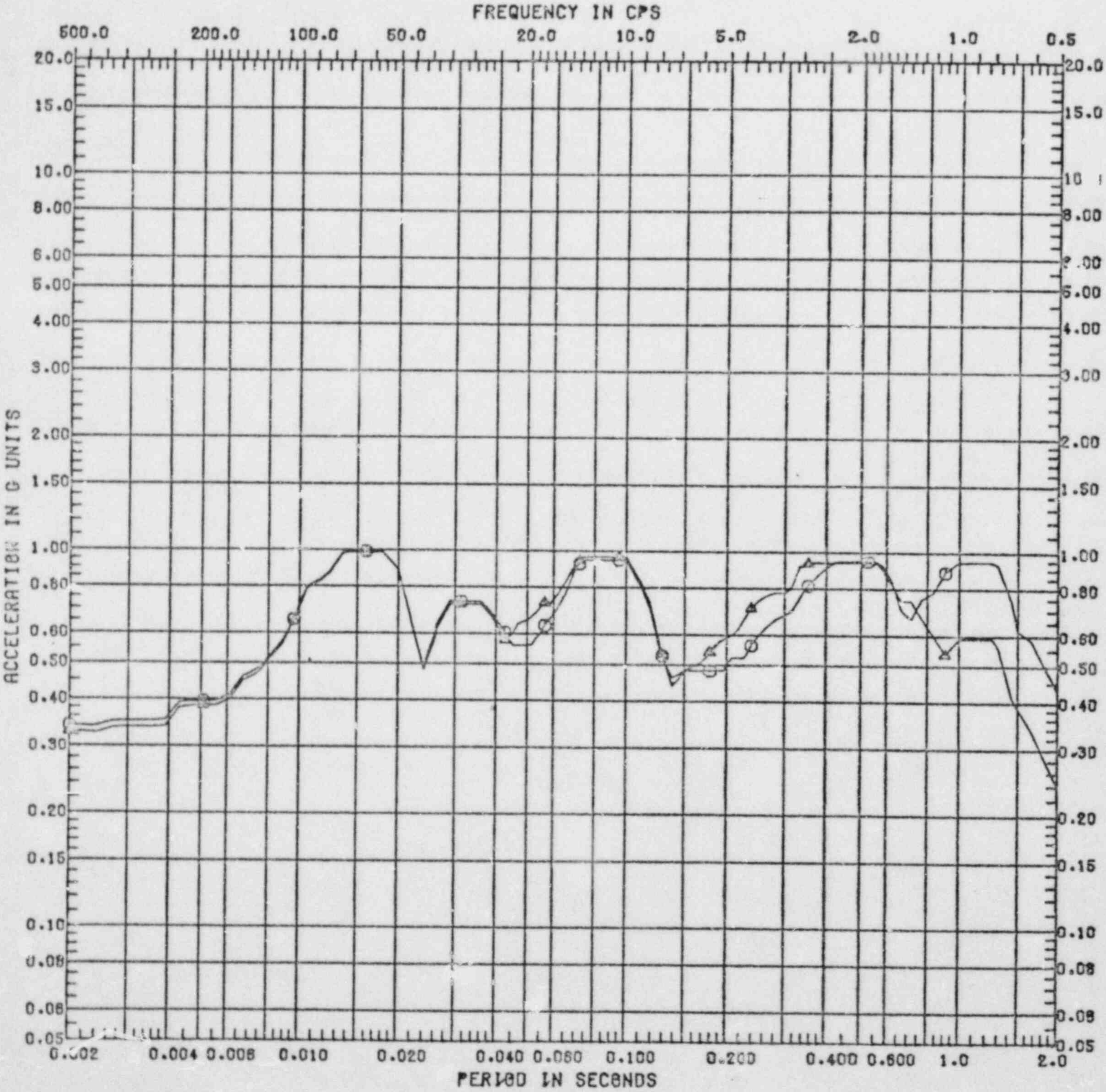

○	DBLEVELC	LEVEL C
▲	NEWLEVELC	LEVEL C

FIGURE 220.15-35

SHIELD WALL ELEV 743 HORIZONTAL WALL 4743H8

SARGENT & LUNDY ENGINEERS

13 NOV 81
A954JK

SBIL SPRING SSI EAST-WEST S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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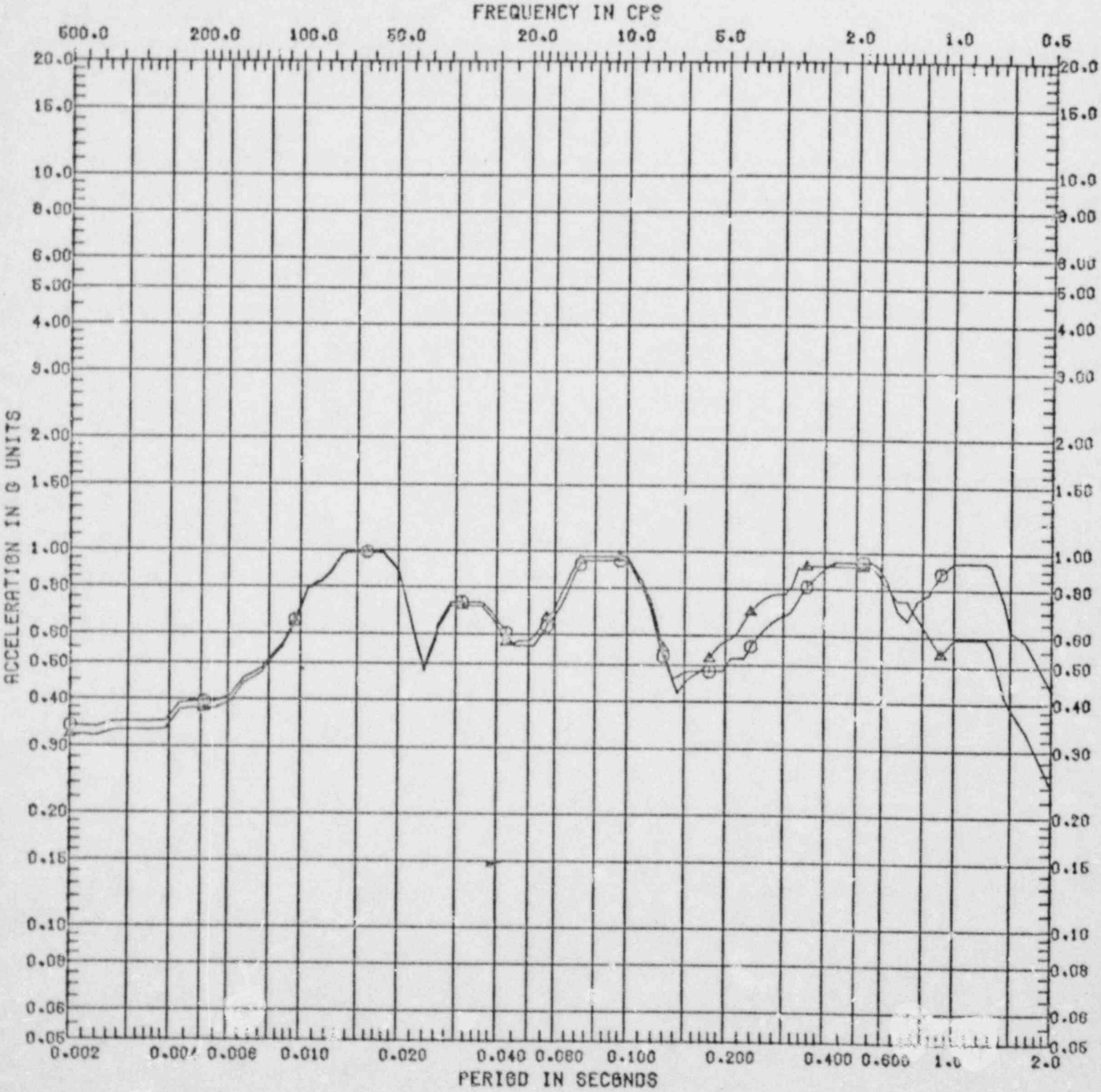

O DBLEVELC LEVEL C
▲ NEWLEVELC LEVEL C

FIGURE 220.15-36

SHIELD WALL ELEV 743 HORIZONTAL WALL 4743H8

SARGENT & LUNDY
ENGINEERS

13 NOV 81
A954JK

SOIL SPRING SSI EAST-WEST S/R SAD
ENVELOPED SPECTRA
CALC NO. 8.11.4-1
PROJECT CLINTON-1 REV 0
PROJECT NO. 4536-29
DAMPING 2.000

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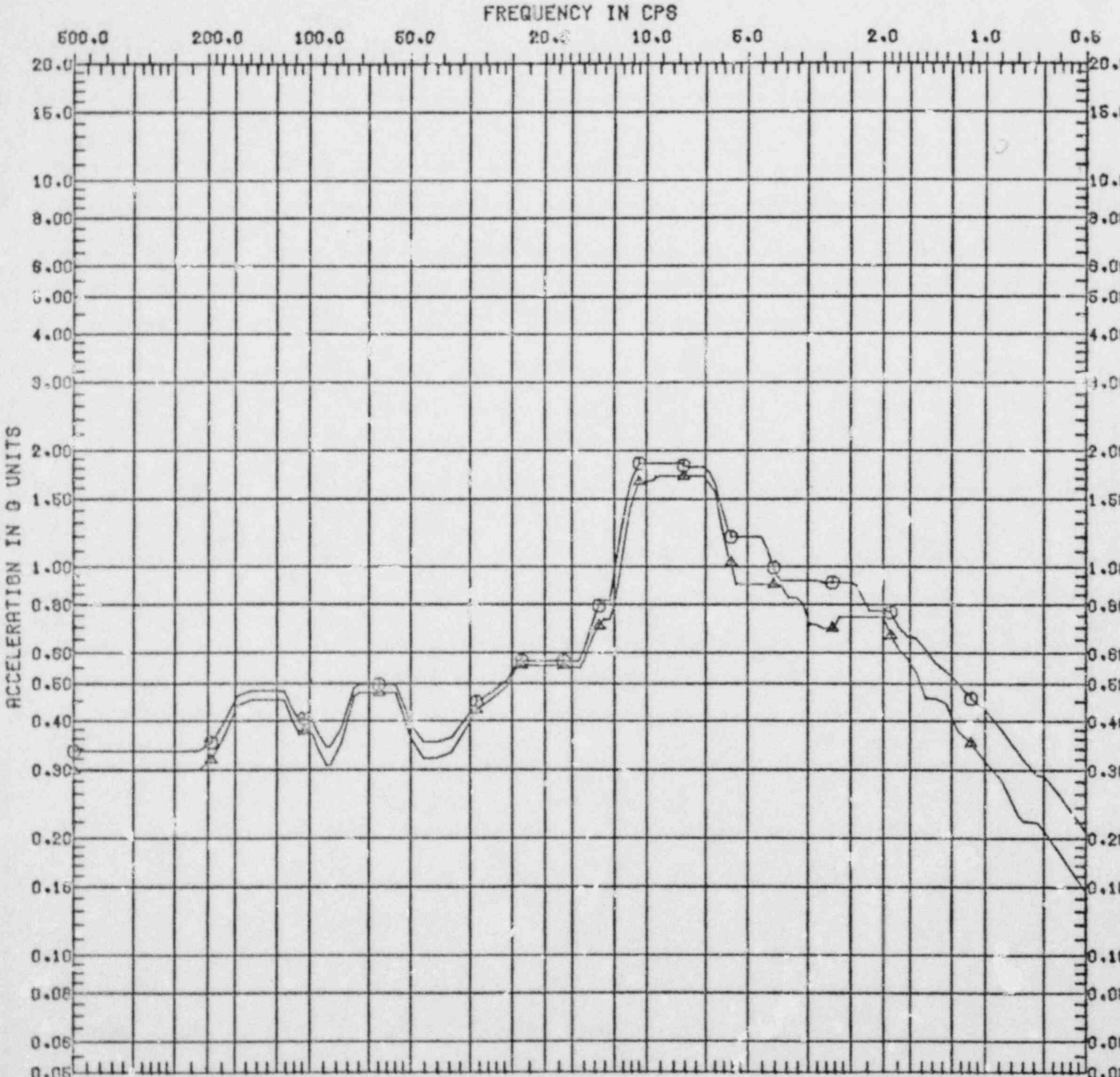

○ OBLEVELC LEVEL C
▲ NEWLEVELC LEVEL C

FIGURE 220.15-37

SHIELD WALL ELEV 743 VERTICAL WALL 4743VW