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2NRC-5-015

February 1, 1985

United States Nuclear Regulatory Commission
Washington, DC 20555

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

SUBJECT: Beaver Valley Power Station - Unit No. 2
Docket No. 50-412
Response to NRC Structural and Geotechnical Engineering Branch's
Draft SER Open Item on Soil-Structure Interaction

Gentlemen:

This letter provides our response to the NRC Structural and Geotechnical Engineering Branch's (SGEB) Draft SER open item on Soil-Structure Interaction (Item SRP 3.7.3 [Audit Action Items 4, 7, and 23]). This submittal supplements our response to NRC Structural Design Audit Action Item 7, which was provided in Reference (a), and addresses the discussion of that response at our November 30, 1984, meeting with the SGEB (Reference [b]).

In Action Item 7, the SGEB reviewers requested that additional soil-structure interaction analyses be performed for the containment and intake structures in order to demonstrate that BVPS-2 meets the intent of SRP 3.7.2.II.4. No further analyses were performed for the intake structure because, as stated in References (a) and (b), the adequacy of this structure was addressed under the BVPS-1 docket.

To demonstrate that BVPS-2 meets the intent of SRP 3.7.2.II.4, DLC's response to Action Item 7 provided an alternate soil-structure interaction analysis for the containment structure. As discussed in FSAR Section 3.7.2, the original soil-structure interaction analysis for the containment used the finite element method (PLAXLY computer code), in which the soil was modeled as finite elements and the structure as a lumped mass elastic beam. The alternate soil-structure interaction analysis, provided in the Action Item 7 response, was based on the three-step solution developed by Kausel and Whitman. This analysis used the same lumped mass elastic beam model to represent the containment structure; the soil was modeled as a half-space using the frequency-dependent compliance function method of anlaysis. The design earthquake input motion was defined to occur at the ground surface in the free field. Kinematic interaction was used to transform the purely translational motion at the ground surface into combined translational and rotational motion at the foundation level.

At our meeting with the SGEB on November 30, 1984, our response to Action Item 7 was discussed. The SGEB reviewers requested that a further soil-structure interaction analysis be performed by using either a simplified

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Whitman-type soil spring approach or a frequency-dependent impedance approach. For either approach, the SGEB reviewers specified that the free-field ground surface earthquake input motion was to be applied at the foundation level of the structure. DLC agreed to perform an analysis for the containment structure that uses the frequency-dependent impedance approach with the free-field ground surface motion applied at the foundation level.

Upon further consideration of the SGEB reviewers' request, we conclude that such an analysis would yield results which are neither physically representative of the actual site conditions nor technically appropriate. The SGEB reviewers' suggested analysis neglects two physical phenomena which are well-recognized by professionals in the field of seismic analysis (Reference [c]) and are very important to specifying the appropriate vibratory ground motion to be applied at the foundation level of the structure, consistent with the requirements of 10CFR100, Appendix A. These two phenomena are:

(1) the soil layer between the ground surface and foundation level modifies the foundation level vibratory motion compared with the ground surface vibratory motion; and

(2) the geometric effects of the structure also modify the vibratory motion at the foundation level relative to the ground surface vibratory motion.

The Kausel-Whitman three-step analytical method, used in our Action Item 7 response, has a sound engineering basis and accounts for both the effects of the soil layer and the geometric effects of the structure on the vibratory motion at the foundation level compared with the ground surface vibratory motion. We believe that the results of the analysis presented in our Action Item 7 response are physically consistent with these well-recognized principles of soil-structure interaction and are therefore technically appropriate. Attachment A provides a detailed description of the Kausel-Whitman three-step analytical method.

Attachment B presents a comparison of the one-percent damping curves of both the BVPS-2 design response spectra and those resulting from the Kausel-Whitman three-step method for several typical locations. Both spectra compare favorably with only minor exceedances which are insignificant considering the conservative value (one percent) used for equipment damping. This demonstrates that BVPS-2 meets the intent of SRP 3.7.2.II.4.

In DLC's application for the BVPS-2 Construction Permit, the soil-structure interaction was analyzed as directed by your staff. (See PSAR Question 3.19, Amendment 7, July 9, 1973.) In the course of the present Operating License Application review, the docket has been augmented with a responsive, technically appropriate analysis which indicates that BVPS-2 meets the intent of SRP 3.7.2.II.4 of NUREG-0800, the most recent formal NRC guidance concerning this issue. Therefore, we believe that the BVPS-2 PSAR, FSAR, the supplemental information provided in the response to NRC Structural Design Audit Action Item 7, and this letter provide a complete record for the satisfactory closure of this issue. DLC is willing to again meet with the

SGEB staff to clarify any points in this or previous submittals. If such a meeting is desired, DLC requests that the Assistant Director for Licensing participates to ensure involvement of appropriate NRC management personnel. However, DLC believes that further requests for analyses utilizing alternative methodologies will offer no meaningful additions to the existing record. Therefore, DLC is requesting that further requests for information on this issue be addressed by the NRC staff as a backfit in accordance with the provisions of 10CFR50.109, GNLR 84-08, and NRC Manual Chapter 0514.

DUQUESNE LIGHT COMPANY

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

JDO/wjs
Attachments

cc: Mr. B. K. Singh, Project Manager (w/a)
Mr. G. Walton, NRC Resident Inspector (w/a)
Mr. G. E. Lear, NRC SGEB Chief (w/a)

References:
(a) Letter 2NRC-4-080, dated June 15, 1984
(b) Letter 2NRC-4-207, dated December 17, 1984
(c) NUREG/CR-1780, "Soil-Structure Interaction: The Status of Current Analysis Methods and Research" prepared for U.S. Nuclear Regulatory Commission by Lawrence Livermore Laboratory, October 1980

COMMONWEALTH OF PENNSYLVANIA)
) SS:
COUNTY OF ALLEGHENY)

On this 1st day of February, 1985, before me, a Notary Public in and for said Commonwealth and County, personally appeared E. J. Woolever, who being duly sworn, deposed and said that (1) he is Vice President of Duquesne Light, (2) he is duly authorized to execute and file the foregoing Submittal on behalf of said Company, and (3) the statements set forth in the Submittal are true and correct to the best of his knowledge.

Anita Elaine Reiter
Notary Public

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

ATTACHMENT A

I. DESCRIPTION OF THE THREE-STEP ANALYSIS

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

1. Calculations of frequency-dependent soil stiffnesses

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

3. Interaction Analysis

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

I.1.1 Frequency-Dependent Soil Stiffness

The frequency-dependent stiffnesses of a rectangular footing founded at the surface of a layered medium are computed with the program REFUND, discussed in Section II. The program solves the problem of forced vibration of a rigid plate on a viscoelastic, layered stratum using numerical solutions to the generalized problems of Cerruti and Boussinesq (see Figure I-2). The effects of unit harmonic horizontal and vertical point loads are combined by superposition to produce the behavior of a rectangular plate.

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

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

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

$$U_r = \sum_0^\infty u_r^n \cos n\theta \qquad P_r = \sum_0^\infty p_r^n \cos n\theta$$

$$U_y = \sum_0^\infty u_y^n \cos n\theta \qquad P_y = \sum_0^\infty p_y^n \cos n\theta$$

$$U_\theta = \sum_0^\infty -u_\theta^n \sin n\theta \qquad P_\theta = \sum_0^\infty -p_\theta^n \sin n\theta$$

For the problem at hand, only the first two components of the series are needed. The (unit) vertical force case corresponds to the Fourier component of order zero (n = 0), and the horizontal unit force case corresponds to the Fourier component of order one (n = 1). The cartesian displacement (flexibility) matrix (F) at a point then follows from the cylindrical displacement components.

$$F = \begin{Bmatrix} \frac{1}{2}(u_r^1+u_\theta^1)+\frac{1}{2}(u_r^1-u_\theta^1)\cos 2\theta & u_r^0 \cos\theta & \frac{1}{2}(u_r^1-u_\theta^1)\sin 2\theta \\ u_y^1 \cos\theta & u_y^0 & u_y^1 \sin\theta \\ \frac{1}{2}(u_r^1-u_\theta^1)\sin 2\theta & u_r^0 \sin\theta & \frac{1}{2}(u_r^1+u_\theta^1)-\frac{1}{2}(u_r^1-u_\theta^1)\cos 2\theta \end{Bmatrix}$$

and the displacement vector for arbitrary loading is

U = FP

where

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

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

For points along the free surface, the reciprocity theorem requires that $U_r^0 = U_y^1$. Hence, F is chessboard symmetric/antisymmetric. REFUND then computes the cylindrical displacement components for the two loading cases, and determines the cartesian flexibility matrix F under the load (axis) at the boundary and at selected points beyond the boundary.

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

$$FP = U$$

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

$$U = TV$$

where V is a (6 x 1) vetor containing the rigid body translations or rotations of the plate and T is the linear transformation matrix assembled with the coordinates of the nodal points. The stiffness functions are then obtained from

$$Z = T^T P$$

which corresponds formally to

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

A comparison of REFUND results with another method is shown in Section II.1.

I.1.2 Embedment Correction

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

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

in which

C_R = correction factor

R = foundation radius

E = embedment depth

H = depth to bedrock

C_1 = constants, different values for each degree of freedom.

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

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

I.1.3 Kinematic Interaction

In the second step of the analysis shown in Figure I-1, "kinematic interaction" modifies the purely translational input specified at the surface of the stratum to both a translational and rotational motion at the base of the rigid, massless foundation. The existence of the additional input can be inferred from Figure I-5. In a stratum undergoing translational motion only, the boundary conditions at the "excavation" require the foundation to rotate. Ignoring the rotational component would result in an unconservative solution. Note that the modified motion at the base of the foundation is not equivalent to a deconvolution.

The solution to the kinematic interaction portion of the analysis is based on Kausel's adaptation of Iguchi's (1982) generalized weighted averaging technique. In essence, the method requires solving the 6 x 1 equation

$$U_f = H^{-1} \int_A T^T U^* dA + K^{-1} \int_A T^T S^* dA$$

where:

$$T = \begin{Bmatrix} 1 & 0 & 0 & 0 & (Z - Z_0) & -(Y - Y_0) \\ 0 & 1 & 0 & -(Z - Z_0) & 0 & (X - X_0) \\ 0 & 0 & 1 & (Y - Y_0) & -(X - X_0) & 0 \end{Bmatrix}$$

(X_0, Y_0, Z_0) = coordinates of the centroid of the foundations contact area

(X,Y,Z) = coordinates of foundation/soil interface

A = surface area of foundation

U^* = $U^*(X,Y,Z)$ = the free field displacement vector along the foundation/soil interface (before excavation)

S = $S^*(X,Y,Z)$ - the free field tractions vector at the foundation/soil interface

H = $\int_A T^T T \, dA$

K = Foundation impedance matrix

U_f = matrix of transfer functions for motion of the massless foundation

To obtain the actual motion to be used as support motion in the three-step method, the transfer functions must be convolved with the Fourier transforms of the accelerations of the surface earthquakes, resulting in the following solution:

$$\ddot{x}(t) = \text{IFT} \left\{ F(\Omega)\ u \right\}$$

$$\Phi_1(t) = \text{IFT} \left\{ F(\Omega)\ \Phi \right\}$$

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

IFT = Inverse Fourier transform

$$u = \frac{\cos(PE)\ \frac{2E}{R}\left(\frac{\sin(PE)}{PE}\right) - \frac{\pi PR}{4}\left(1 - \frac{v}{2}\right)}{1 + \frac{2E}{R}} \quad \frac{\sin(PE)}{\left(\frac{G_f}{G_1} + \frac{2E}{3R}\right)\left(1 + 0.6 i PR\right)}$$

$$\phi = \frac{4\left\{\left(\frac{h}{R}\cos(PE)\right)+\left(\frac{2hE}{R^2}\frac{\sin(PE)}{PE}\right)-\left(\frac{E}{R}\right)^2\left(\frac{\sin^2\left(\frac{PE}{2}\right)}{\left(\frac{PE}{2}\right)^2}\right)\right\}}{\left\{1+\frac{4E}{R}+\frac{8}{3}\left(\frac{E}{R}\right)^3-\frac{4\left(\frac{E}{R}\right)^4}{1+\frac{2E}{R}}\right\}} + \frac{3\pi(1-\nu)}{8}\left\{\frac{(1-\cos(PE)-ph\sin(PE))}{\left(\frac{G_2}{G_1}+\frac{2E}{R}\right)\left(1+\frac{0.3i\,(PR)^3}{1+(PR)^2}\right)}\right\}$$

R = foundation radius

$P = \frac{\Omega}{c_s}(1-iB)$

E = foundation embedment depth

V = Poisson's ratio

G_1 = shear modulus of soil adjacent to foundation

G_2 = shear modulus of soil below foundation

h = height of the foundation's area center of gravity above the base of the foundation

c_s = shear wave velocity

I.1.4 Interaction Analysis

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

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

$$M\ddot{U} + C\dot{Y} + KY = 0 \qquad (1)$$

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

These two vectors are related by:

$$U = Y + EU_g \qquad (2)$$

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

$$
\left\{ \begin{matrix} I & & T_1 \\ O & & I \\ I & & T_2 \\ O & & I \\ & \vdots & \\ I & & T_n \\ O & & I \end{matrix} \right\} \tag{3}
$$

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

$$
T_i = \left\{ \begin{array}{c|c|c} O & Z_i \;\; Z_o & -(Y_i - Y_o) \\ \hline -(Z_i - Z_o) & o & X_i - X_o \\ \hline Y_i - Y_o & -(X_i - X_o) & o \end{array} \right\}
$$

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

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

$$\ddot{U} = H_j e^{iwt} \qquad \ddot{Y} = (H_j - E_j) e^{iwt}$$

$$\dot{U} = \frac{1}{iw} H_j e^{iwt} \qquad \dot{Y} = \frac{1}{wi}(H_j - E_j) e^{iwt}$$

$$U = -\frac{1}{w^2} H_j e^{iwt} \qquad Y = -\frac{1}{w^2}(H_j - E_j) e^{iwt} \tag{4}$$

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

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

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

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

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

Defining the dynamic stiffness matrix:

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

The solution for the transfer functions follows formally from:

$$H_j = -K_d^{-1}(K + iD)E_j$$

$$= -(I + \omega^2 K_d^{-1} M)E_j$$

(8)

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

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

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

(9)

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

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

(10)

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

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

The dynaimc equations are solved in FRIDAY by Gaussian elimination, and the Fourier transforms are computed by subroutines using the Cooley-Tuckey FFT (fast Fourier transform) algorithm. A comparison of the results of FRIDAY with another solution is shown in Section II.3.

I.2 REFERENCES

1. Timoshenko & Goodier, Theory of Elasticity, Third Edition, McGraw-Hill Book Co., pp. 97-109.

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

3. Michio Iguchi, An Approximate Analysis of Input Motions for Rigid Embedded Foundations. Trans of A.I.J. No. 315 May 1982.

II. DESCRIPTION OF COMPUTER PROGRAMS

II.1 REFUND AND EMBED

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

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

Since REFUND is restricted to surface-founded plates, the effects of embedment are included by adjusting the REFUND results with the program EMBED. The theoretical bases of these programs and their application to the solution methodology are described in Section I.1.2.

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

	Layer 1	Layer 2
Shear wave velocity	1	1.25
Specific weight	1	1.1764
Poisson's ratio	0 25	0.25

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

$$K = Ko \ [k + iao\ c]$$

in which ao is a dimensionless measure of frequency and Ko is a zero-frequency stiffness.

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

II.2 KINACT

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

The theoretical basis for the program is derived from wave propagation theory as described in Section I.1.3.

II.3 FRIDAY

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

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

The theoretical basis and implementation of the program is described in Section I.1.4. A comparison of FRIDAY with a public domain program, STAR-DYNE, for the seismic response of a fixed-base, multi-mass, cantilever model is shown in Figure II.3.-1. The model is shown in Figure II.3-2.

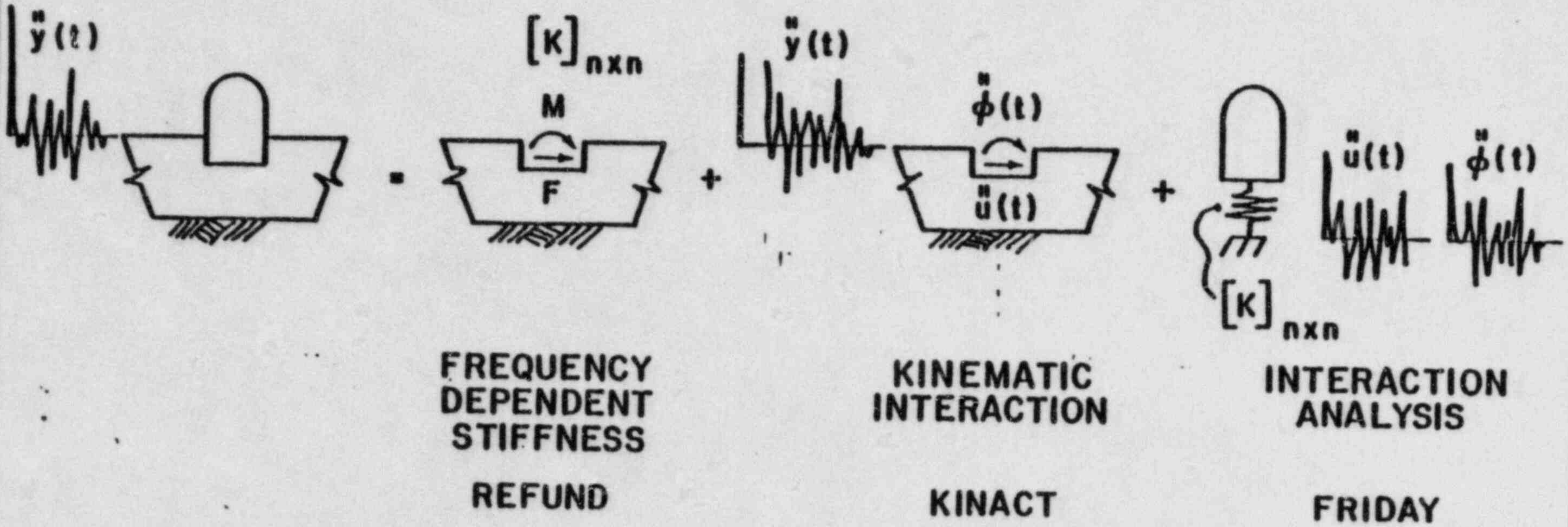


FIGURE I-1
THE THREE STEP SOLUTION

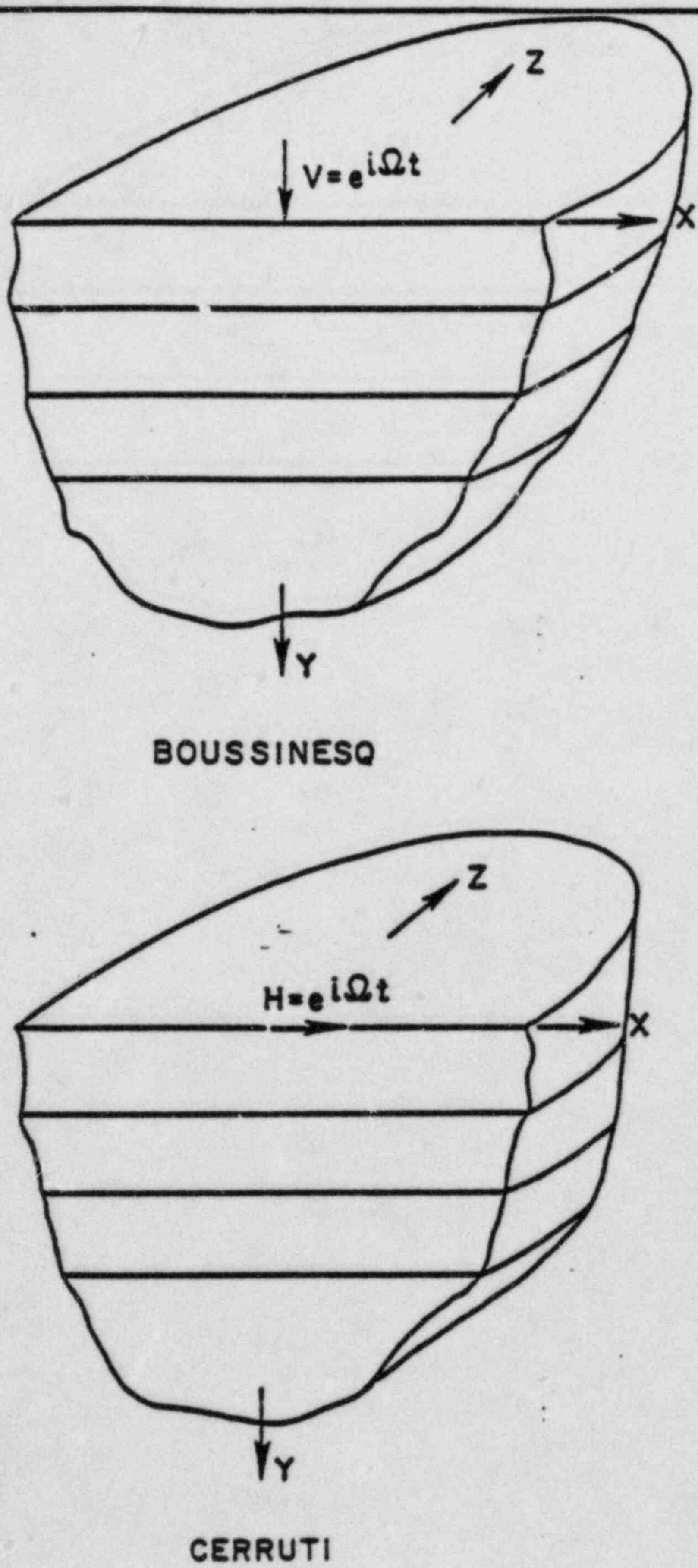


FIGURE I-2
THE BOUSSINESQ AND CERRUTI PROBLEMS

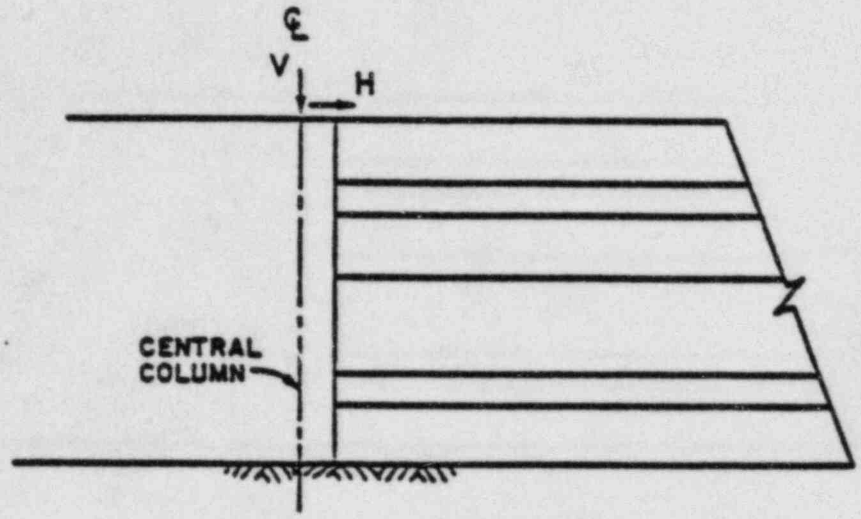


FIGURE I-3
IDEALIZATION OF THE BASIC 'REFUND' SOLUTION FOR CONCENTRATED LOADS

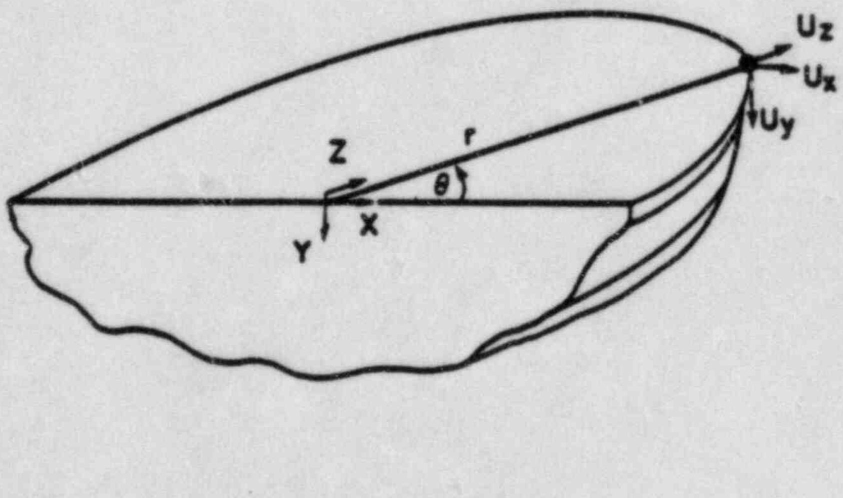
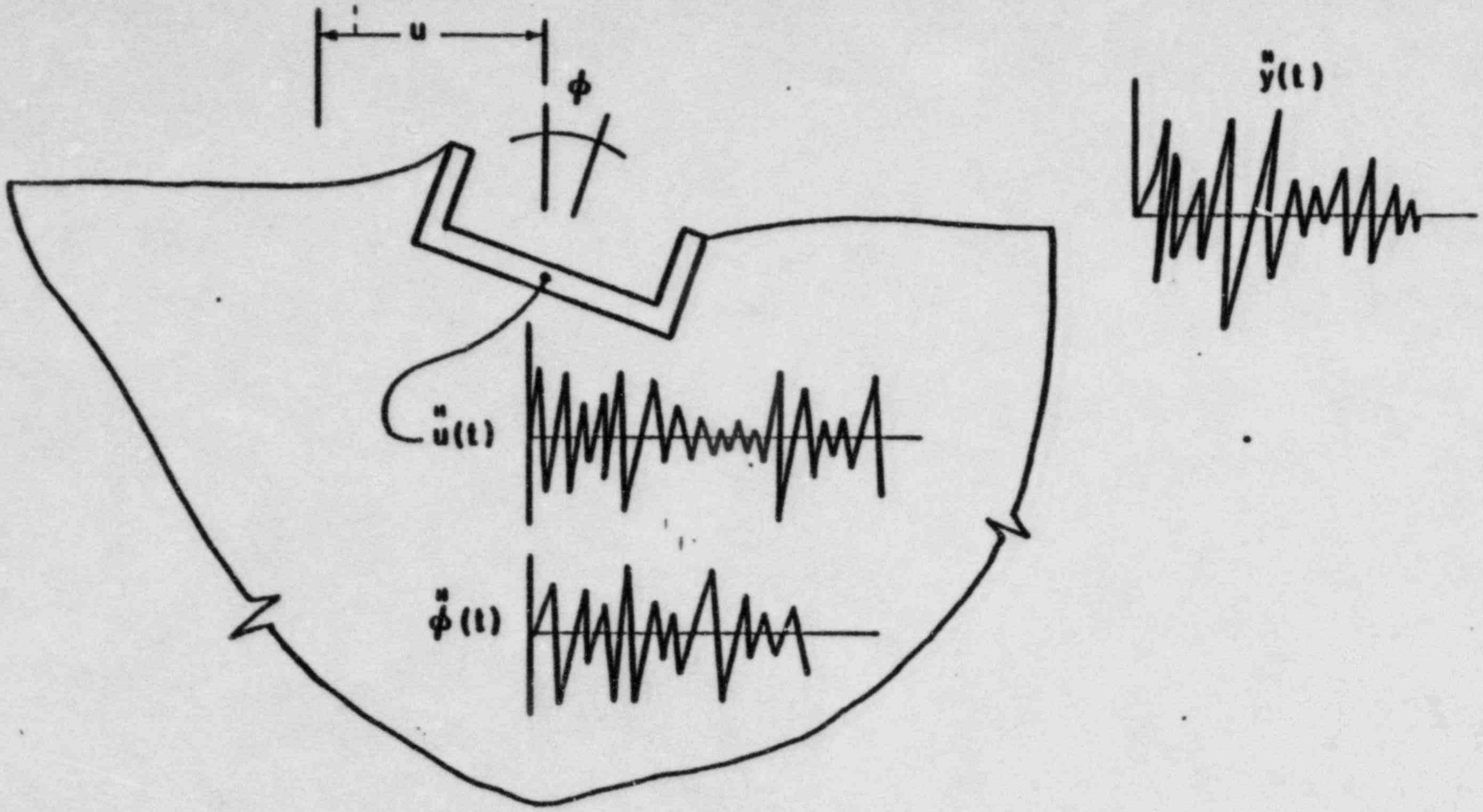


FIGURE I-4
'REFUND' COORDINATE SYSTEM



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

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

FIGURE I-5
KINEMATIC INTERACTION

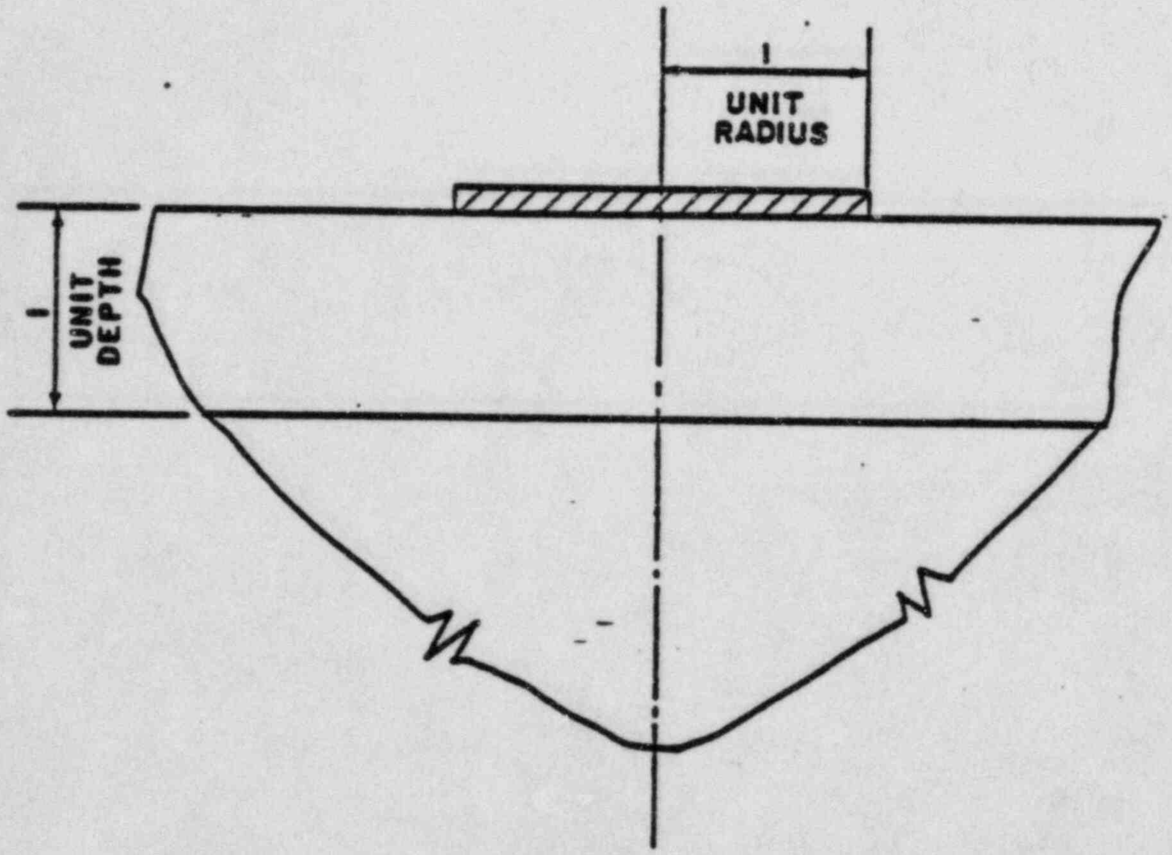


FIGURE II.1-1
LUCO'S TWO-LAYER PROBLEM

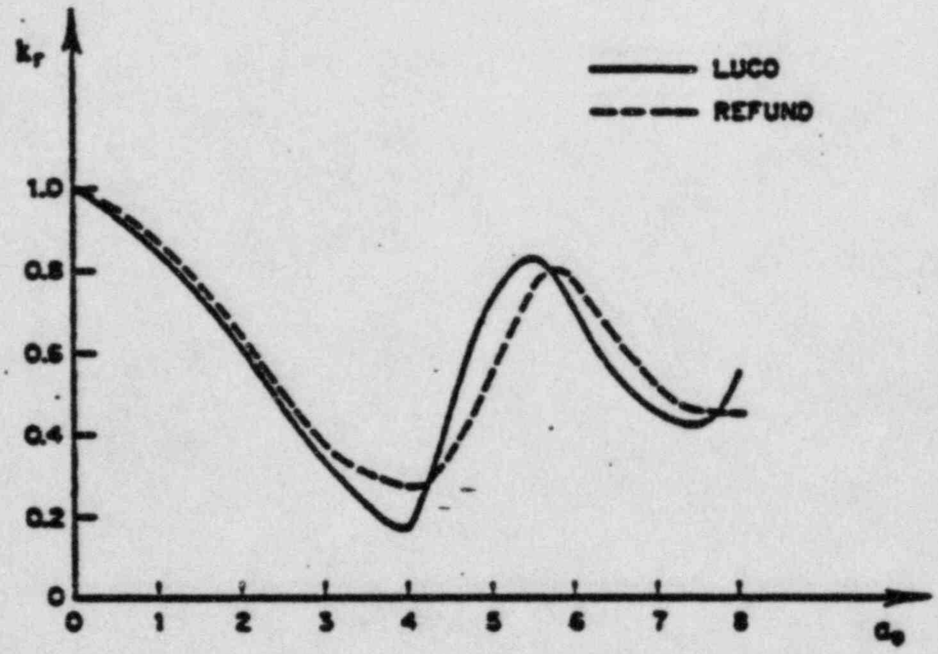


FIGURE II.1-2
ROCKING STIFFNESS COMPARISON -
REAL PART

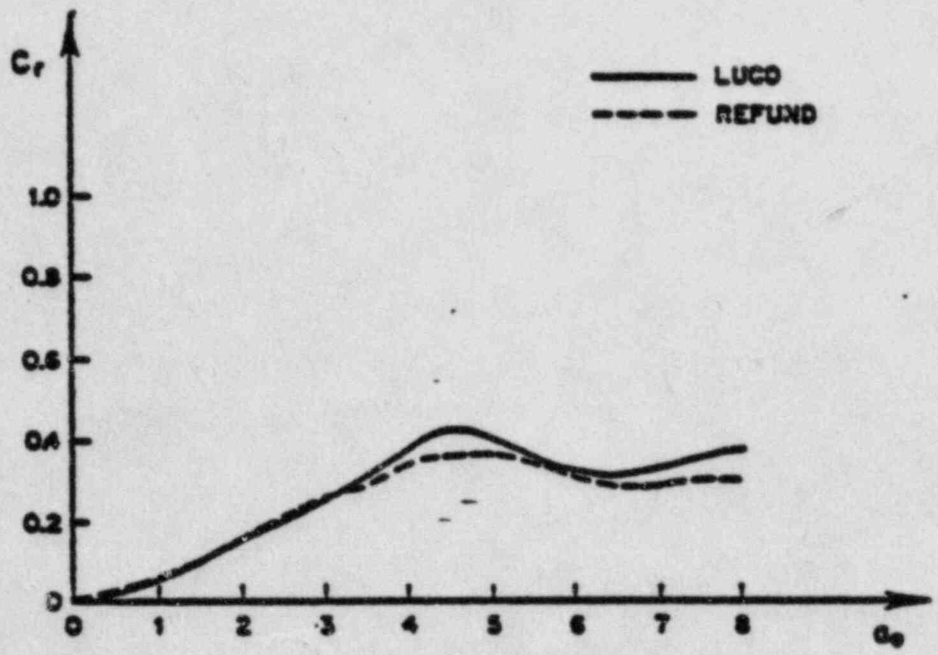


FIGURE II.1-3
ROCKING STIFFNESS COMPARISON -
IMAGINARY PART

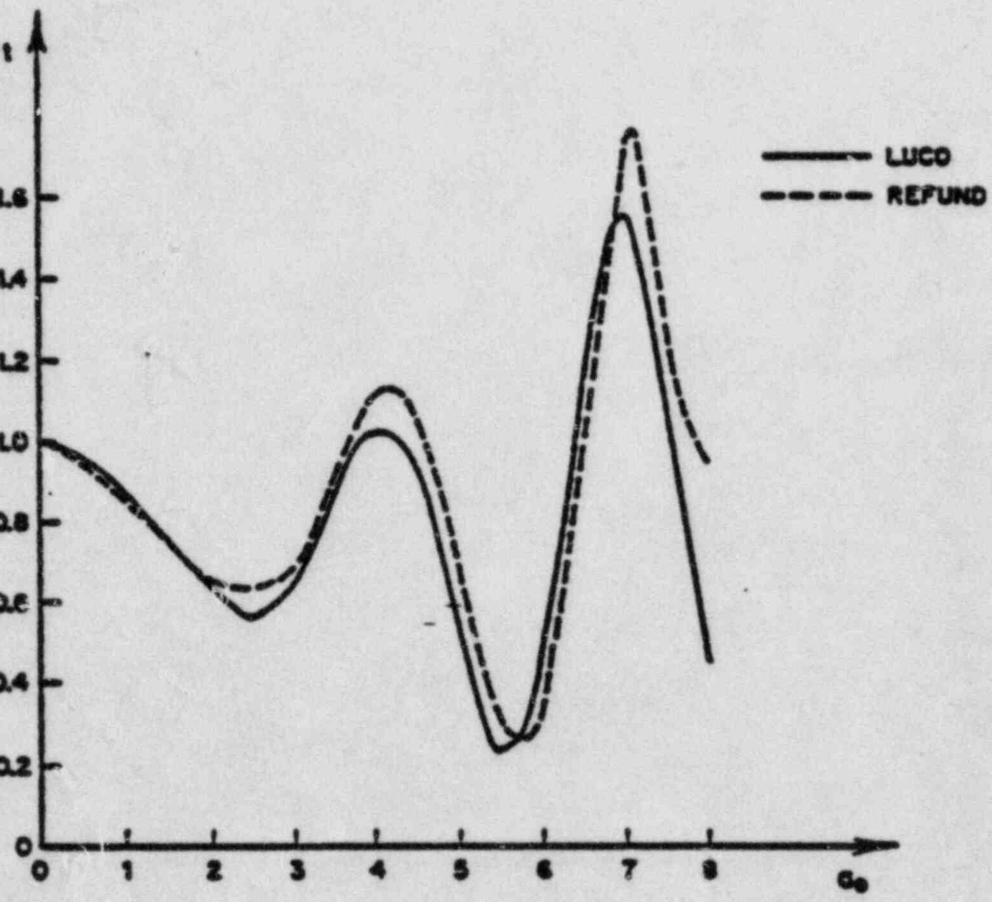


FIGURE II.1-4
HORIZONTAL STIFFNESS COMPARISON -
REAL PART

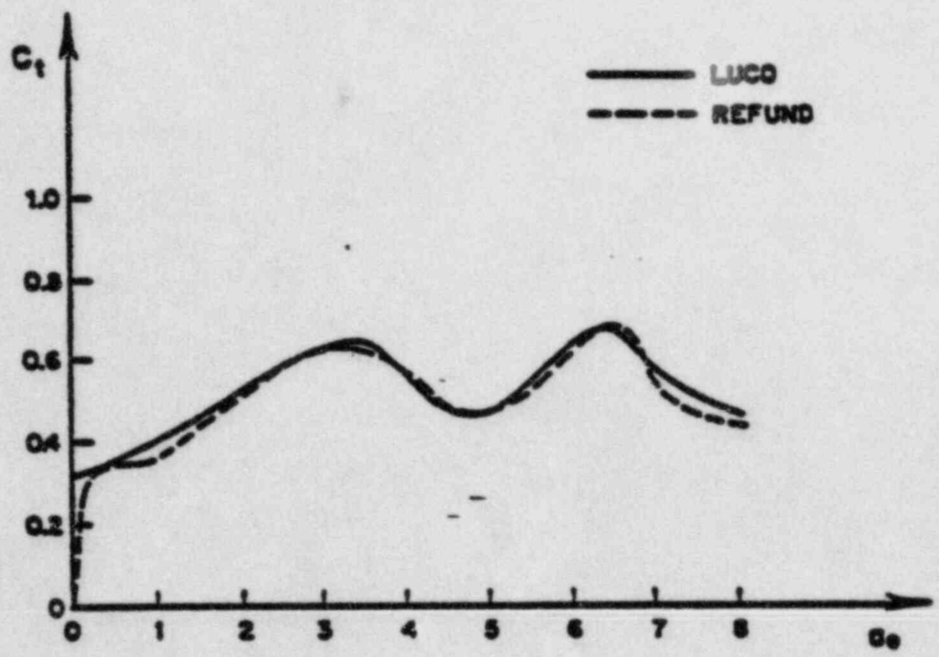


FIGURE II.1-5
HORIZONTAL STIFFNESS COMPARISON - IMAGINARY PART

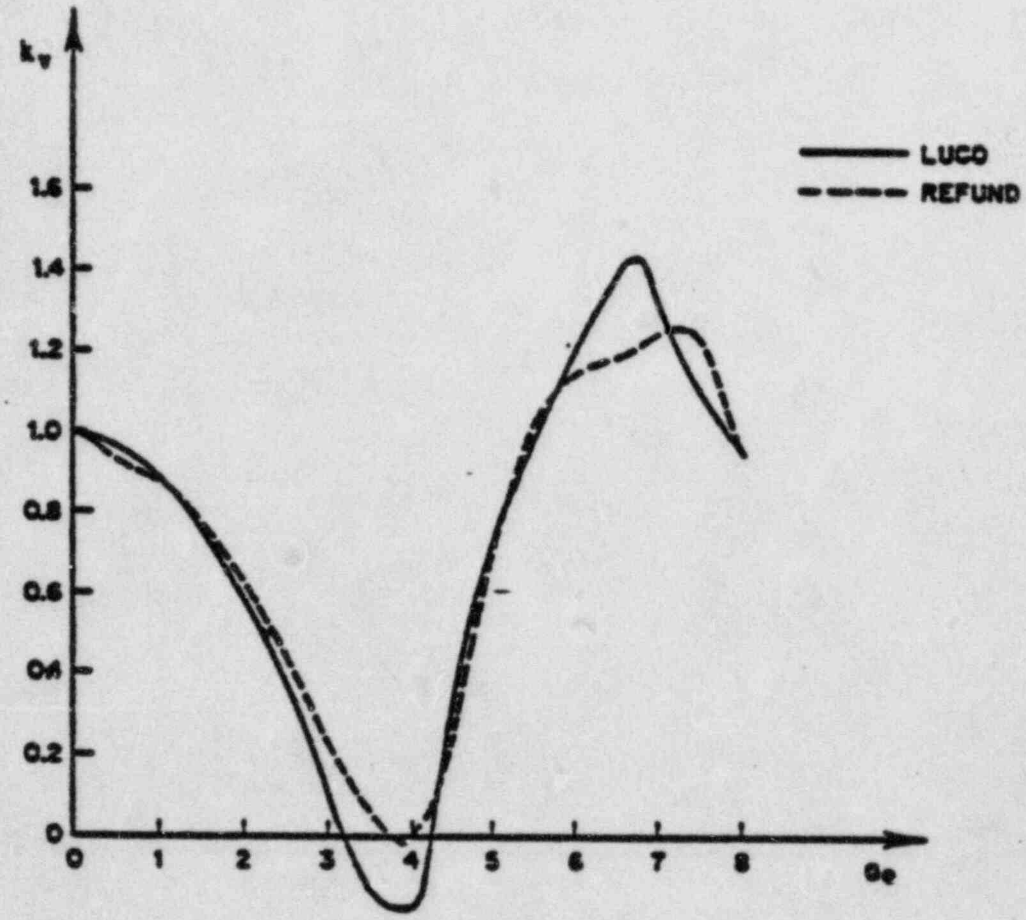


FIGURE II.1-6
VERTICAL STIFFNESS COMPARISON -
REAL PART

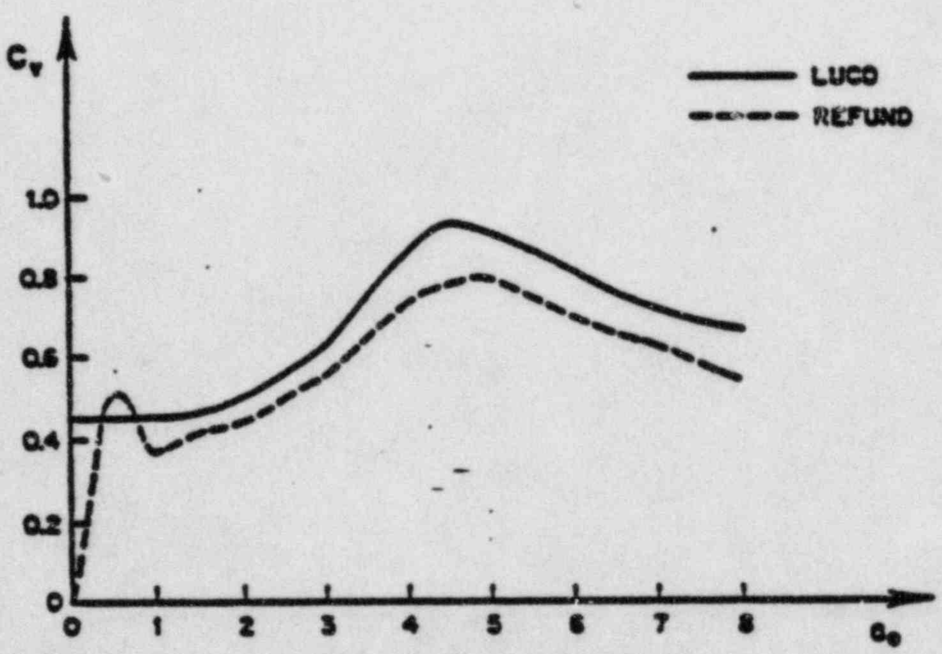


FIGURE II.1-7
VERTICAL STIFFNESS COMPARISON -
IMAGINARY PART

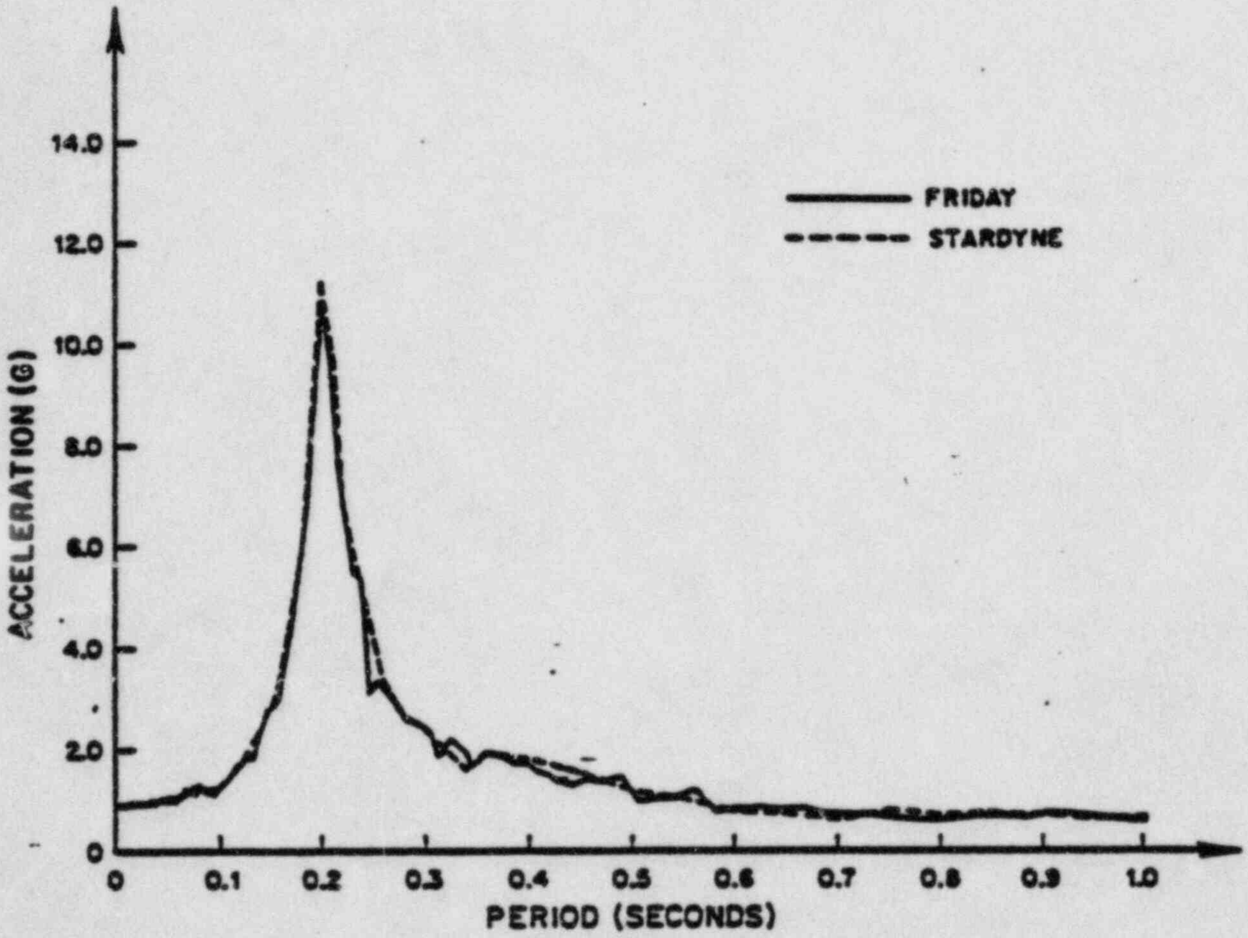


FIGURE II.3-1
COMPARISON OF 'FRIDAY' AND
'STARDYNE'-ARS AT THE ROOF

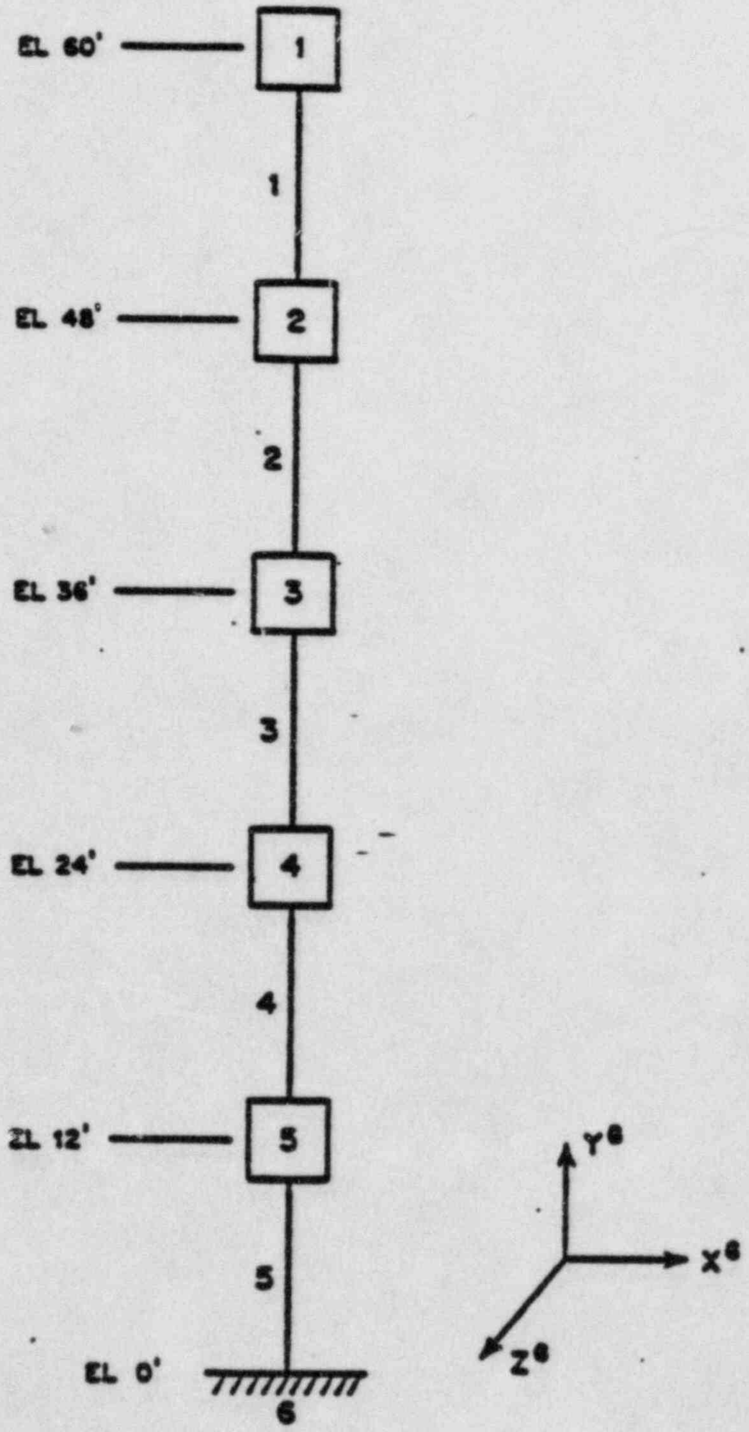
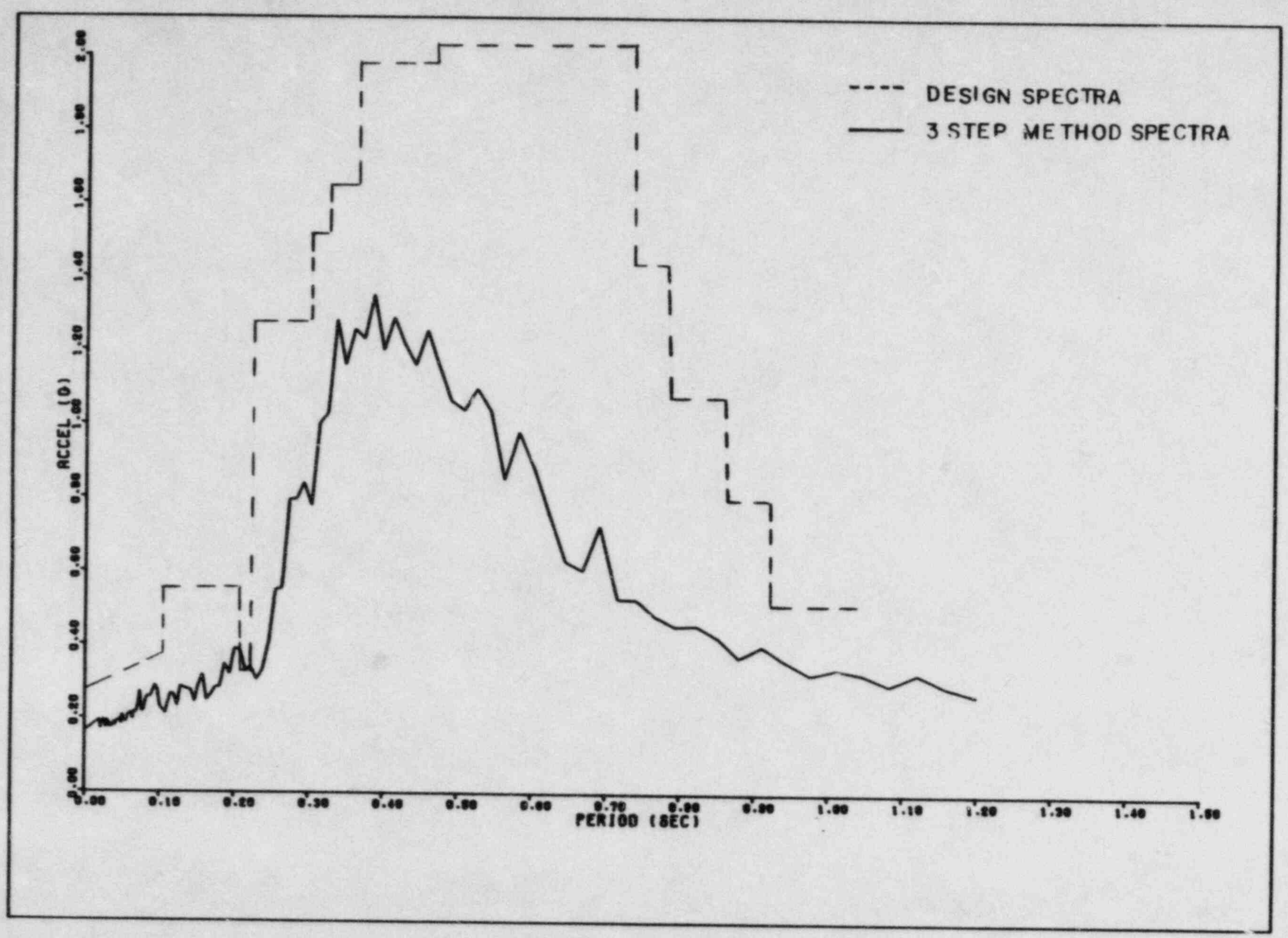


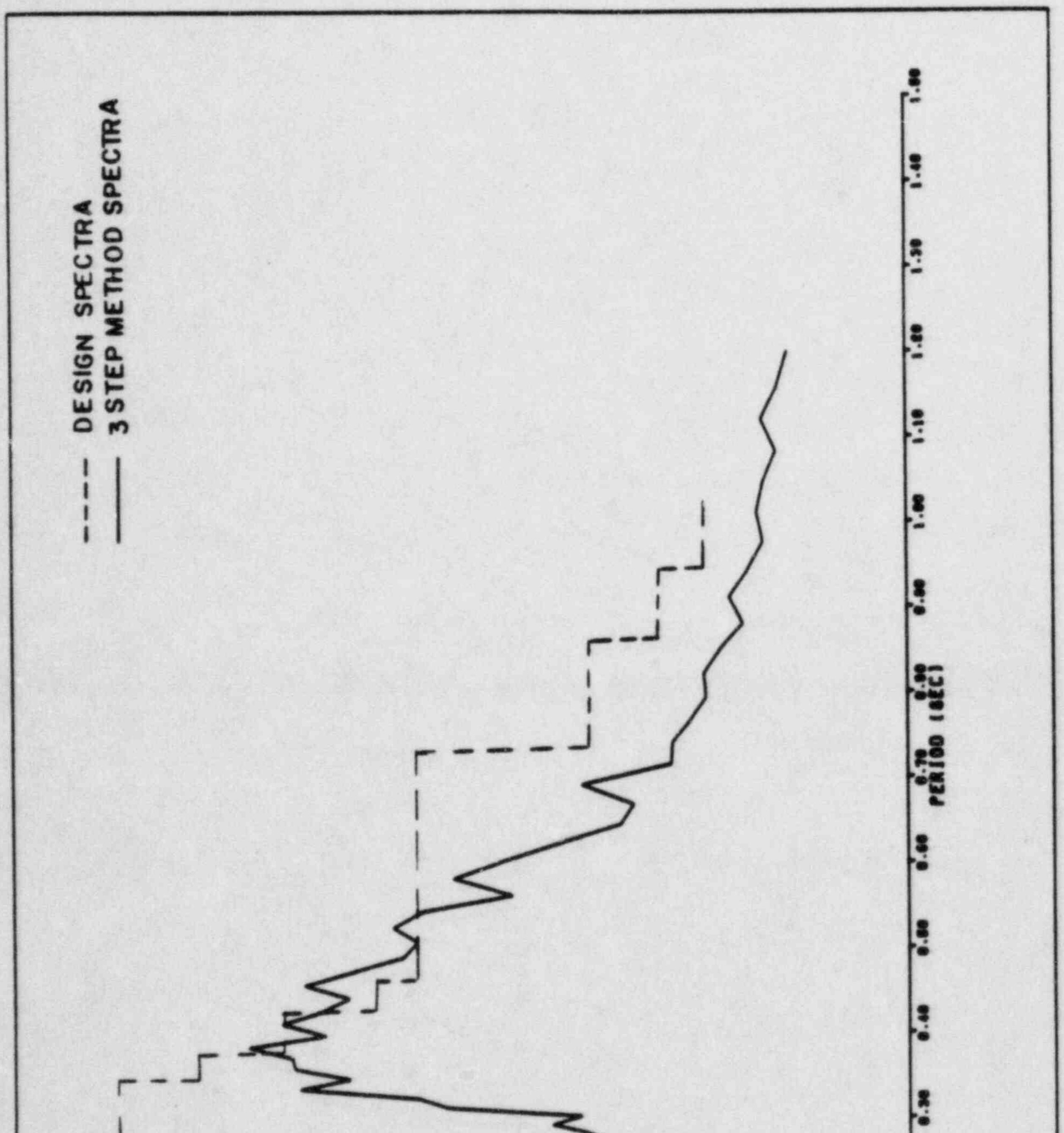
FIGURE II.3-2
'STARDYNE' MODEL

ATTACHMENT B
FIG. B.1



BVPS-2 CONTAINMENT STRUCTURE EL 854.0 (TOP OF DOME)
HORIZONTAL SSE 1% EQUIPMENT DAMPING

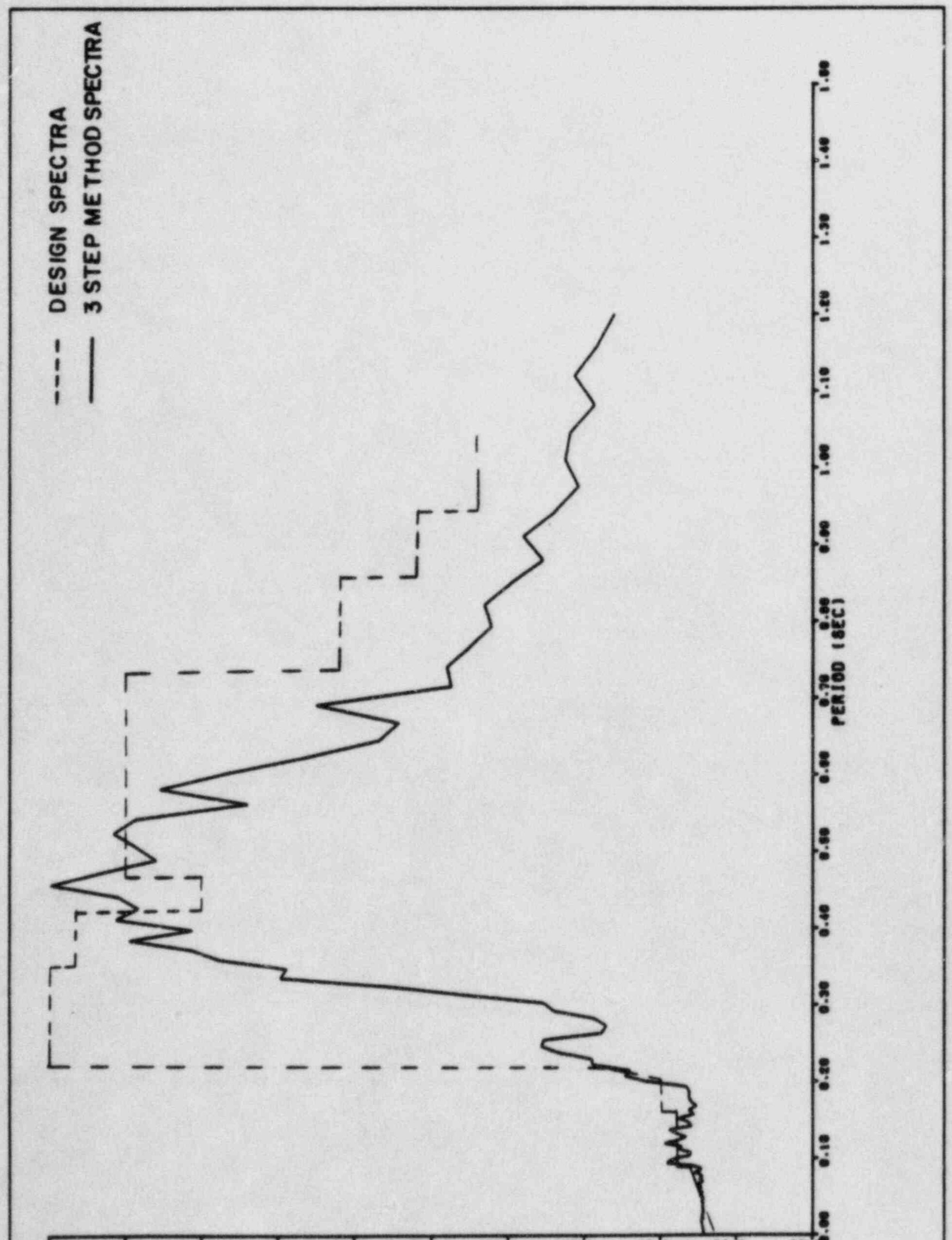
ATTACHMENT B
FIGURE B.2



BVPS-2 CONTAINMENT STRUCTURE EL 818.0 (TOP OF CRANEWALL)
HORIZONTAL SSE 1% EQUIPMENT DAMPING

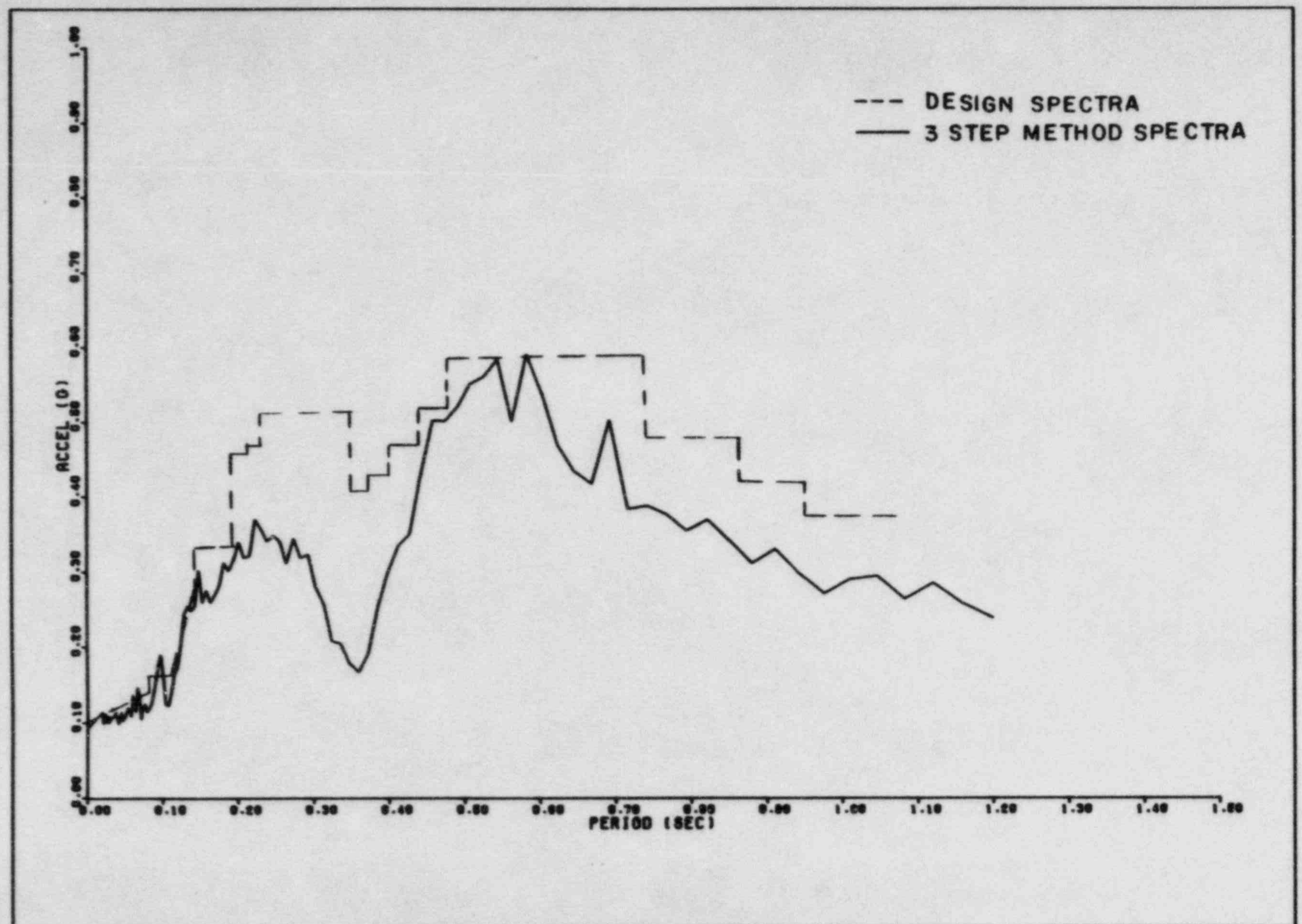
ATTACHMENT B

FIGURE B.3



BVPS-2 CONTAINMENT STRUCTURE EL 738.0
HORIZONTAL SSE 1% EQUIPMENT DAMPING

ATTACHMENT B
FIGURE B. 4



BVPS-2 CONTAINMENT STRUCTURE EL 681.0 (BASE MAT)
HORIZONTAL SSE 1% EQUIPMENT DAMPING