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APPENDIX C - COMPUTER PROGRAMS

The computer programs referred to in Sections 2.5, 3.7, and 3.8 by their acronyms are described herein. All programs are verified, within the stated assumptions and limitations, for correctness of utilized theory and validity of obtained results for a variety of typical problems. Results are checked against known solutions, solutions obtained from other programs, or hand calculations. Examples of validation problems are included with the program descriptions. Whenever applicable, internal checks, such as equilibrium and orthogonality checks, are included as an aid in checking the validity of the results.

C.1 CBEAM

CBEAM (Reinforced Concrete Beam Design and Schedule) is written to perform the routine work of reinforcement selection for rectangular cross section beams. The program is based on the design methods of the ACI 318-71 Code and Sargent & Lundy's (S&L) structural design standards.

In CBEAM, all beam sections are assumed to be rectangular sections. For stirrup reinforcement, each beam is divided into three portions: left 1/4 length; middle 1/2 length; and, right 1/4 length. The program assumes that constant shear forces are applied within each region. Design forces (bending moments and shear forces) for continuous frames should be obtained from analysis programs such as STRUDL. Design forces for individual members should be obtained by any acceptable analytical procedure.

Required input data includes identification titles, dimensions of the member sections, and design member forces. Output includes a beam schedule suitable for direct release for construction use and a longtudinal bar schedule according to S&L's structural design standards.

To demonstrate the validity of CBEAM, a typical three-span beam design was processed on CBEAM and the results compared to hand calculations.

Tables C-1 through C-3 show the beam characteristics and the resulting output for the three beams. As shown, the results compare very favorably.

C.2 COLOAD

COLOAD (Column Load Computation) computes column loads for power plant buildings. A floor plane is modeled as a mesh grid system consisting of a number of slab elements simply supported by columns. A linear interpolation scheme along with an iteration process is used to distribute floor loadings to columns. The input data consists of floor geometry, loading conditions and locations and column weight information.

COLOAD was developed by Sargent & Lundy in 1972. It is currently maintained on a UNIVAC 1106 operating under EXEC-8.

To demonstrate the validity of COLOAD, a two-story building is analyzed for column loads and the results of COLOAD are compared against hand calculations.

Models of the two floors are shown in Figure C-1. Each slab element in both floors has an area of 100 in^2 with a total weight of 1000 kips. The weight of each column is 50 kips and the column weight due to uniform load is 250 kips.

Table C-4 gives the resulting total load as calculated by COLOAD and hand calculations. The results are identical.

C.3 CONCRETE

CONCRETE (Quality Control Analysis of Concrete) is a computer program used for statistical evaluation of concrete strength. It sorts and analyzes the field data collected on concrete samples and presents it in a convenient-to-interpret form.

The compressive strength test results of concrete cylinders are statistically analyzed to obtain the mean, standard deviation, coefficient of variation, moving averages, and other statistical parameters required in quality appraisal of concrete according to ACI 214-65. The strength results are also compared with the quality control limits fixed according to ACI 318-71 and the ASTM Manual on Quality Control of Materials, 15-C. Violations or inadequacies are clearly pointed out in the output.

CONCRETE was developed and is maintained by Sargent & Lundy. Since 1972, the program has been used at Sargent & Lundy on a UNIVAC 1106 operating under EXEC-8.

To demonstrate the validity of the program, a sample problem was taken from "Notes on ACI 318-71 Building Code Requirements with Design Applications" (Reference 1) and the results compared against those from CONCRETE.

Concrete strength test results from 28 day strength of 46 pairs of cylinders sampled from a particular class of concrete delivered to a site are shown in Table C-5. Figure C-2 shows the results from CONCRETE; Table C-6 shows the results from the ACI notes. The results are identical.

C.4 CSEF-III

CSEF-III (Circular Slab on an Elastic Foundation) analyzes any circular slab for arbitrary load conditions. The analysis is based on the solution to the basic differential equation for a plate on an elastic foundation, modified to include a Fourier series representation for the circumferential variation. For each Fourier harmonic, a matrix of linear equations is formulated using a finite difference representation of the plate equation and boundary equations. The system of equations is then solved by means of a Gauss elimination method.

Arbitrary normal pressure loading, edge moment and/or shear loads and axisymmetric thermal gradient loads may be considered in the analysis. At any radius, the entire circumference may be fixed against rotation or displacement allowing for a variety of support conditions or boundary conditions.

The program output includes deflection, soil pressure, radial moment, hoop moment, twisting moment, radial shear, tangential shear and Kirchhoff shear for each Fourier harmonic. Results from individual harmonics are superimposed and output along the radii at specified angles. Moments may be converted to a Cartesian coordinate system for comparison with other programs.

Version III of CSEF was developed by Sargent & Lundy in 1971. It is currently maintained at Sargent & Lundy on UNIVAC 1100 series hardware operating under EXEC-8.

Two plate analyses are presented as examples of validation.

The first example is a concrete circular plate with

Radius	$a = 50$ ft
Thickness	$h = 5$ ft
Elastic Modulus	$E = 576{,}000$ ksf
Poisson's Ratio	$\mu = 0.17$

resting on an elastic foundation with

Foundation Modulus $k = 518.4$ kip/ft^3.

The plate is analyzed for a uniform pressure load

$p = 50$ kip/ft^2

and a uniform edge load

$M = 1000$ kip-ft/ft.

The results of the CSEF-III analysis are compared with a hand calculated solution. The hand calculations are based on equations presented in Hetenyi, "Beams on Elastic Foundation" (Reference 2). The displacements for the distributed load and edge moments are found independently and superimposed. For the uniform pressure load the displacement is

$$w = p/k$$

and for the edge moment

$$w = D_1 \; Z_1 \; (\nu r) + D_2 \; Z_2 \; (\nu r)$$

where

$$\nu = \sqrt[4]{k / D}$$

$$z_1(x) = R_e \quad J_o \quad \left(x \sqrt{i}\right)$$

$$z_2(x) = I_m \quad J_o \quad \left(x \sqrt{i}\right)$$

J_o = Bessel function of the first kind

and

$$D_1 = \frac{M\nu^2}{k} \frac{z_1'(\nu a)}{z_1(\nu a) z_2'(\nu a) - z_1'(\nu a) \quad z_2(\nu a) + \frac{1-\mu}{\nu a} \quad \left[z_1'^2(\nu a) + z_2'^2(\nu a)\right]}$$

$$D_2 = \frac{M\nu^2}{k} \frac{z_2'(\nu a)}{z_1(\nu a) z_2'(\nu a) - z_1'(\nu a) \quad z_2(\nu a) + \frac{1-\mu}{\nu a} \quad \left[z_1'^2(\nu a) + z_2'^2(\nu a)\right]}$$

$$z_1' \text{ and } z_2' = \frac{dZ(x)}{dx}$$

The results obtained by evaluating these equations for displacement and the appropriate expressions for radial moment are presented along with those obtained from CSEF-III in Figures C-3 and C-4, respectively. As illustrated in these figures, the independent solutions compare favorably.

For a second example a simply supported circular plate under a linearly varying pressure load is presented. The solution is obtained for a plate having the following properties and loading:

Radius	a = 100 in.
Thickness	h = 2 in.
Elastic Modulus	E = 3.0 x 10^7 psi
Poisson's Ratio	υ = 0.3
Loading intensity	$q = \dfrac{pr \quad c_{os} \qquad \phi}{a}$

where

$p = 100$ psi.

Results from CSEF-III are compared with those obtained for Kalnin's Shell of Revolution (Reference 3). The comparison for radial moment and displacement are shown in Figures C-5 and C-6. Solutions obtained from both programs favorably compare with a solution presented by Timoshenko and Woinowsky-Krieger, "Theory of Plates and Shells" (Reference 4).

C.5 DYNAS

DYNAS (Dynamic Analysis of Structures) is designed to perform dynamic analysis of structures which can be idealized as three dimensional space frame and/or rigid slabs connected together by translational or torsional springs. The program considers the combined effects of translational, torsional and rocking motions on the structure. The program uses response spectrum, time history forced vibration, or static method of analysis, depending on the type of forcing function available. All the methods of dynamic analysis use the normal mode approach.

The program is capable of analyzing structures with parts having different associated dampings; composite modal damping values are obtained by weighing the damping factors according to the mass of each element. The option is also available to analyze a large structural system using the modal synthesis technique. By this, the system is divided into subsystems whose modal characteristics are computed separately and then synthesized to obtain the response of the complete system. The base motion can be applied simultaneously in two orthogonal directions. Response spectrum can be generated as specified slabs or joints.

In case of response spectra analysis, various modal responses may be combined to obtain the probable maximum responses by using three different methods: square root of the sum of the squares, doube sum, and absolute double sum method. The responses due to seismic motions from two directions are combined by using the SRSS method.

In case of time-history analysis, the decoupled differential equations of motion are numerically integrated using Newmark's β-method (Reference 5). The response due to translational and rocking components from the same directions are combined algebraically. The response due to seismic motion from two directions may be combined either by SRSS or the algebraic sum method.

The DYNAS program was originally developed by Sargent & Lundy in 1970. The program is currently maintained on a UNIVAC 1106 operating under EXEC-8.

Four of the problems used to validate the program are presented.

In the first problem, a three-story shear building is analyzed and compared to a solution obtained by Biggs (Reference 6). The structure is represented by the closed-coupled system shown in Figure C-7. The masses and stiffness values used are also given in Figure C-7. For the analysis the following response spectra were used:

Frequency	Displacement
1.00 cps	3.30 in.
2.18 cps	1.40 in.
3.18 cps	0.66 in.

The results obtained by Biggs and from DYNAS are compared in Tables C-7 through C-9.

In the second problem, results of DYNAS are compared to those obtained by Wilson, et al. (Reference 7) using the SAPIV program.

At the fixed end of a cantilever beam, shown in Figure C-8, an acceleration, shown in Figure C-8, is applied. The natural periods calculated by both SAPIV and DYNAS are shown in Table C-10. A comparison of the bending moment at the fixed end of the cantilever beam is shown in Figure C-9.

In the third problem, a three-story shear building is analyzed and compared to a solution obtained by Biggs (Reference 6).

The structure is represented by the close-coupled spring-mass system in Figure C-10. The masses, stiffness values, and forcing functions are also given in Figure C-10. The results obtained by Biggs and from DYNAS are compared in Figure C-11.

In the fourth problem, a cantilever shown in Figure C-12, part (a), is analyzed for the first 3 seconds of a Castaic N-21-E earthquake applied along the x-direction at joint 6. The translational and rotational (x and θy response) time histories at joint 4 are applied as simultaneous excitations for the cantilever shown in Figure C-12, part (b). The results of the two models are compared in Table C-11.

As demonstrated in all four problems DYNAS performs an accurate analysis.

C.6 DYNAX

DYNAX (Dynamic Analysis of Axisymmetric Structures) is a finite element program capable of performing both static and dynamic analyses of axisymmetric structures. Its formulation is based on a small displacement theory.

Three types of finite elements are available: quandrilateral, triangular, and shell. The geometry of the structure can be general as long as it is axisymmetric. Both isotropic and orthotropic elastic material properties can be modeled. Discrete and distributed springs are available for modeling elastic foundations, etc.

For static analysis, input loads can be structure weight, nodal forces, nodal displacements, distributed loads, or temperatures. Loads can be axisymmetric or nonaxisymmetric. For the solids or revolution, the program outputs nodal displacements and element and nodal point stresses in the global system (radial, circumferential, and axial). In the case of shells of revolution, the output consists of nodal displacements, and element and nodal point shell forces in a shell coordinate system (meridional, circumferential, and normal).

For dynamic analysis, three methods are available: direct integration method, modal superposition method, and response spectrum method. In the case of dynamic analysis by direct integration method or modal superposition method, a forcing function can be input as (1) nodal force components versus time for any number of nodes, or (2) vertical or horizontal ground acceleration versus time. For nonaxisymmetric loads the equivalent Fourier expansion is used. In the case of dynamic analysis by response spectrum method, spectral velocity versus natural frequency for up to four damping constants is input. The output of dynamic analysis is in terms of nodal displacements, element stresses and resultant forces and moments at specified time steps. When the modal superposition method is used, and in the case of earthquake response analysis, the requested number of frequencies and mode shapes are computed and printed together with the cumulative response of all the specified modes, as computed by the root sum square (RSS) method and the absolute sum method.

DYNAX was originally developed under the acronym ASHAD by S. Ghosh and C. L. Wilson of the University of California, Berkeley, in 1969 (Reference 8). It was acquired by Sargent & Lundy in 1972 and is operating under EXEC-8 on a UNIVAC 1106.

To demonstrate the validity of the major analytical capabilities of DYNAX, documented results and hand calculations for several problems are compared with DYNAX results.

The first problem is taken from S. Timoshenko's book "Theory of Plates and Shells" (Reference 9). A clamped shallow spherical shell, shown in Figure C-13, is analyzed for displacement and stresses produced by a uniform pressure applied on its outside surface. DYNAX and Timoshenko's solutions are compared in Figures C-14 and C-15.

The second problem, taken from "Theory of Elasticity" by Timoshenko and Goodier (Reference 10), is a plane strain analysis of a thick-walled cylinder subjected to external pressure. The finite element idealization and the loading system used for this case are shown in Figure C-16. Results of the DYNAX analysis are compared with the exact solution in Figure C-17. The agreement for both stresses and displacements is excellent.

The third problem was presented in an article by Budiansky and Radkowski in the August 1963 issue of the AIAA Journal (Reference 11). The structure, illustrated in Figure C-18, is a short, wide cylinder with a moderate thickness to radius ratio. The applied loads and the output stresses are pure uncoupled harmonics. For this finite element analysis the cylinder is divided into 50 elements of equal size. This problem checks the harmonic deflections, element stresses, and forces. Figures C-19 and C-20 compare DYNAX results with the results given in the article.

The fourth problem is taken from an article by Reismann and Padlog (Reference 12). A ring (line) load of magnitude P (500 lb) is suddenly applied to the center of a freely supported cylindrical shell. The dimensions of the shell and the time history of load are shown in Figure C-21. Because of symmetry only one-half of the cylinder is modeled using 80 elements of equal size. The time history of radial deflection and meridional moments from DYNAX and from Reismann and Padlog are compared and are shown in Figures C-22 and C-23, respectively.

For the fifth problem, the method of mode superposition is used to solve a shallow spherical cap with clamped support under the action of suddenly applied uniformly distributed load. The dimensions of the shell and the load time history are shown in Figure C-24. The first 12 modes were considered to formulate the uncoupled equations of motion. Each of these equations was solved by the step-by-step integration method using a time step of 0.1×10^{-4} seconds. The results are compared graphically with those obtained by S. Klein (Reference 13) in Figures C-25 and C-26.

The sixth problem is a hyperbolic cooling tower, as shown in Figure C-27. The tower is analyzed for horizontal earthquake motion. A response spectrum for 2% damping, as shown in Figure C-28, was used for this analysis. The RMS values of the meridional force are compared with those obtained by Abel, et al. (Reference 14) in Figure C-29.

The seventh problem demonstrates the validity of the tying routines. A moment of 100 k-ft/ft is applied at the top of a 50-foot cylindrical shell (Figure C-30). Figure C-31 shows the results from DYNAX and an analytical solution (Reference 9).

In the eighth problem, a plate, shown in Figure C-32, is analyzed for cracking due to varying temperature and the results from DYNAX are compared with hand calculations. The finite element model and material properties are also shown in Figure C-32. The temperature gradient is 2.4° F per inch thickness. The strain calculated by DYNAX is ε_o = 1.8936E-5 and the strain calculated by hand is ε_o = 1.81E-5.

For the ninth problem, a cylinder under constant pressure (Figure C-33) is analyzed by DYNAX and SOR-III, a public domain program (Reference 15). The flexibility matrix for the boundary conditions of the top and bottom used in SOR-III is

$$\begin{bmatrix} .33294 \times 10^{-3} & -.55426 \times 10^{-3} \\ -.55426 \times 10^{-3} & .18453 \times 10^{-3} \end{bmatrix} \begin{Bmatrix} 1 \\ 1 \end{Bmatrix} = \begin{Bmatrix} u_r \\ \beta \end{Bmatrix}$$

The inverse of this matrix is then input in DYNAX as

$$\begin{bmatrix} -750.8 & 0. & 0. & -2255.2 \\ 0. & .5 \times 10^{6} & 0. & 0. \\ 0. & 0. & 0. & 0. \\ -2255.2 & 0. & 0. & 1354.7 \end{bmatrix} \begin{Bmatrix} 1 \\ 1 \\ 1 \\ 1 \end{Bmatrix} = \begin{Bmatrix} H \\ V \\ T \\ M \end{Bmatrix}$$

Table C-12 shows a comparison of results for the two programs.

The tenth problem analyzes a cylinder under constant dynamic axial pressure (Figure C-34) for nonreflecting boundaries. The velocity of the waves traveling through the cylinder due to the dynamic load can be calculated as follows:

$$C = \sqrt{\frac{(1-\nu)}{(1+\nu)(1-2\nu)}\frac{E}{\rho}} = \sqrt{\frac{100}{1}} = 10 \text{ ft/sec}.$$

After two seconds the waves will reach the bottom of the cylinder. Since the dynamic load is constant, the velocity of the particles should maintain its value of 10 ft/sec over the entire cylinder for the rest of the load duration.

Table C-13 shows some of the results obtained using the nonreflecting boundary option at the bottom of the cylinder. The velocity in the z-direction at 2.2 seconds and at time 4.0 seconds is in good agreement with the actual velocity of 10 ft/sec.

In the eleventh problem, a cylinder (Figure C-35) is analyzed for frequency, mode shapes, and mixed modal damping obtained from the input material damping constants. The results are obtained based on weighing the damping factors according to the stiffness of each element. Results of this problem are compared to hand calculations in Table C-14.

As shown in these problems, DYNAX is capable of producing accurate results from both static and dynamic analyses of shells.

C.7 FAST

FAST is used primarily for the dynamic analysis of a linear axisymmetric structure subjected to many independent loading cases. The structural model is given as input either in the form of its response for a typical dynamic load, usually a band limited white noise load, or in the form of its eigen values, eigen vectors, participation factors and modal damping ratios. The structural model input required for FAST is obtained from the results of finite element programs. Using the given input parameters, FAST computes transfer functions for various response components. A transfer function expresses the relationship in the frequency domain between a given loading and a specific response component. Using these transfer functions, the response to different loading cases may then be obtained using FAST. This approach reduces the computer analysis time for a structural model which is subjected to different time history motions since only one detailed analysis of the original structure using finite element programs is required; the analysis for the different time histories is then performed using the transfer functions obtained by FAST. The results are given in print and plot forms.

FAST was developed at Sargent & Lundy in 1975. It is currently maintained on a UNIVAC 1106 operating under EXEC-8.

To validate the code a cylindrical concrete tank on soil was analyzed and results were compared to results obtained from the DYNAX program (S&L Program No. 09.7.083-7.0).

The structural model of the tank is shown in Figure C-36, and the concrete and soil properties are shown in Table C-15. Results were compared for a time history pressure load symmetrically applied about the $\theta = 0°$ meridian. The meridional distribution of this pressure is shown in Figure C-36 and the time history distribution is shown in Figure C-37. The Fourier coefficients for circumferential distribution of pressure are shown in Table C-15.

As shown in Tables C-16 and C-17, the results obtained from the two programs for nodal accelerations at nodes A and B in Figure C-36 and for maximum stress resultants at element C in Figure C-36 compare favorably.

Results were also compared for the same load distribution applied about the $\theta = 0°$, $\theta = 120°$ and $\theta = 240°$ meridians simultaneously. Tables C-18 and C-19 show a favorable comparison of the results.

C.8 LAFD

LAFD (Analysis of Liner Anchor Forces and Displacements), calculates the maximum force and displacement of anchors, which result from local buckling of thin plate liners anchored to concrete walls. The solution method used in LAFD is described in Reference 16.

First, displacements are found for an assumed postbuckling load by a relaxation technique. Then, using the maximum displacement, the anchor force and the strain in the buckled plate are calculated. The stress-strain relation given in a paper by Young and Tate (Reference 17) is reestablished in the program. Using the calculated strain, first stress is found and then new load. The new load is then used to find a new set of displacements. The procedure is repeated to find a second new load. This load is then compared to the load used in the previous cycle. The procedure is repeated until the difference between the loads obtained in the last two cycles are approximately equal.

The program is capable of analyzing four types of anchors: Nelson studs of 1/2, 5/8, and 3/4 inch diameter and 3 by 3 by 1/4 inch angle continuous rib anchors. The force-deformation relations of these anchors are obtained from the manufacturer's publication (Reference 18).

The program output includes the maximum anchor force, the maximum anchor deformation, and the postbuckling load of the buckled plate.

LAFD was developed by Sargent & Lundy in 1971 and is currently maintained on a Univac 1106 operating under EXEC-8.

To validate the program, significant calculations were verified with hand calculations. As an example of this validation, a comparison of these calculations is presented for a strip of liner having the following properties:

Strip span a	= 17.5 in.
Plate thickness t	= 0.375 in.
Strip width w	= 9 in.
Modulus of Elasticity E	= 30 x 10^3 ksi
Yield Stress σ_o	= 36 ksi

5/8"Ø Nelson Studs are used as anchors.

The anchor displacement, U_i, the force in the anchor adjacent to the buckled panel, f_1, and the postbuckling load P as calculated by the program are shown in Table C-20. Substituting these displacements into the appropriate force-deformation relationship for 5/8"Ø Nelson Studs yields the anchor forces contained in Table C-21.

The validity of the solution is checked using the displacements and anchor forces given in Tables C-20 and C-21 to verify the equality of the original equations (see Figure C-38):

$$Fo - P = \frac{EA}{a} \left[(U_1 - U_2) \right] + f_1 \qquad \text{(C-1)}$$

$$0 = \frac{EA}{a}\left[\left(2U_n - U_{n-1} - U_{n+1}\right)\right] + f_n \qquad \text{(C-2)}$$

n = 1, 2, 3....N

The postbuckling load, P, as determined by Equation C-1 = 21.864K as compared to 21,978K obtained from the program. Substitution into Equation C-2 yields approximately zero. Equilibrium having been verified, results obtained from the program are valid.

C.9 LUSH

LUSH (Complex Response Analysis of Soil-Structure Interaction) is a finite element program designed for the seismic analysis of soils and structures. Unlike other dynamic soil response programs LUSH has frequency independent damping. This makes it especially appropriate for analyses requiring accuracy in the high frequency range such as generating foundation response spectra.

LUSH solves plane strain problems excited by an acceleration time history specified at the rigid base of the model. The proper strain dependent elastic moduli and damping values of each soil element are determined by iteration until compatibility between maximum principal shear strains and properties is obtained.

Most of the analysis is performed in the frequency domain using the method of complex response with complex moduli. First the stiffness matrix is formulated using the complex moduli given by the equation

$$G^* = G\left(1-2\beta^2 + 2 \quad i \quad \beta \quad \sqrt{1-\beta^2}\right)$$

where for a given element G* is the complex moduli, G is the shear modulus and β is the damping ratio. This accounts for frequency independent viscous damping. Next, using the complex stiffness matrix the nodal point equations of motion are formulated. The input acceleration time history is converted to the frequency domain by a Fourier transform. Then the equations of motion are solved for each term of the Fourier transform and the results superimposed. This gives the Fourier transforms of all the nodal point displacement histories. From these displacements the Fourier transforms for stress, strain, and acceleration histories can be determined. Finally the stresses, strains, and accelerations are converted to the time domain. This process is repeated for each iteration on the soil properties.

The specific output of the analysis is selected by the user. Output relating to elements includes maximum stresses, maximum principal shear strain, strain consistent shear modulus, and strain consistent damping. Nodal point output includes maximum acceleration, acceleration time histories, and response spectra.

LUSH was developed at the University of California, Berkeley by Lysmer, Udaka, Seed, and Hwang (Reference 19). Sargent & Lundy modified the program and now maintains it for use on the EXEC-8 processor of the UNIVAC 1106 computer.

The Sargent & Lundy version of LUSH was validated by comparison with a problem presented in Reference 19. This problem is quite simple but employs all the capabilities of the program.

The problem is outlined in Figure C-39. The strain dependent soil properties used are presented in Table C-22. The comparisons between the S&L version of LUSH results and the results reported in Reference 19 are given in Tables C-23, C-24, and C-25, and Figure C-40. The agreement is very good.

C.10 MESHG

MESHG (Mesh Generator) checks input data for finite element programs. Using the member incidences and node point coordinates as prepared for a finite element program such as SLSAP, PLFEM, DYNAX, and QUAD4, the program produces a CalComp plot of the mesh.

Several isometric views of 3-D data may be obtained, axes may be rotated for 2-D data, and scaling may be specified. Element numbers are plotted proportional in size to element areas for ease in detecting errors in element connectivity or nodal coordinates.

The program was developed by Sargent & Lundy in 1970. It is currently maintained on a Univac 1106 operating on EXEC-8. Plotting is done off-line on a CalComp 905/936 system. Validity of the program is repeatedly verified by inspection of each plotted mesh.

C.11 PCAUC

PCAUC (Portland Cement Association Ultimate Design of Columns) is used to design or to investigate reinforced concrete columns using the ultimate strength theory in accordance with ACI 318-71 Code. The program is capable of designing or investigating tied columns subjected to an axial load combined with unixaial or biaxial bending moment. The program input consists of the dimensions of sections, material properties, reinforcement requirement, and loading data. The slenderness effect is not included in the present program.

Output from the design part of the program includes the steel reinforcement arrangement, ultimate capacity for all loading cases, and interaction control points data. Output from the investigative part of the program either includes biaxial or uniaxial interaction data. Sargent & Lundy has modified the original PCA program to follow the 1971 ACI building code and to provide more design options and greater capacity.

PCAUC is a modified version of the program "Ultimate Strength Design of Concrete Columns," developed by the Portland Cement Association. The program was obtained by Sargent & Lundy in 1972 and modified. It is currently maintained on the UNIVAC 1106 operating under EXEC-8.

To validate PCAUC, documented results from several problems were compared with PCAUC results. Three of these problems are presented here.

The first problem is taken from Wang and Salmon's book "Reinforced Concrete Design" (Reference 20). The reinforcement for a 17-inch by 17-inch square tied column is designed for compression control loads. The loads include a dead-load axial load of 214 kips and bending moment of 47 ft/kips, and a liveload axial load of 132 kips and a bending moment of 23 ft/kips. The reinforcement is designed according to the ACI Code with f_c' = 3,000 psi and f_y = 40,000 psi.

The solution as given in Wang and Salmon's book is identical to the solution obtained from PCAUC, shown in Figure C-41. It should be noted that the ultimate capacity provided by PCAUC has been reduced by a factor of 0.7.

The second problem is also taken from Wang and Salmon's book (Reference 21). The reinforcement for a tied column 14 inches wide and 20 inches deep is designed for tension control loads with a dead-load axial load of 43 kips and bending moment of 96 ft/kips, and a live-load axial load of 32 kips and bending moment of 85 ft/kips. The reinforcement is designed according to the ACI Code using symmetrical reinforcement with respect to its width and with f_c' = 4,500 psi and f_y = 50,000 psi.

The solution as given in Wang and Salmon's book is identical to the solution obtained from PCAUC, shown in Figure C-42.

The third problem is taken from "Notes on ACI 318-71 Building Code Requirements with Design Applications," by the Portland Cement Association (Reference 22). A square tied column 28 inches by 28 inches is designed for biaxial bending loads for the following service loads:

	Dead	Live
Axial	550 kips	300 kips
M_x	320 ft/kips	200 ft/kips
M_y	160 ft/kips	100 ft/kips

The bending is designed according to the ACI Code with f_c' = 5,000 psi and f_y = 60,000 psi.

The selected reinforcement obtained from PCAUC, shown in Figure C-43, is identical to that from Reference 22. It should also be noted that the interaction control points obtained by both show good agreement.

C.12 PIPSYS

PIPSYS (Integrated Piping Analysis System) analyzes piping systems of power plants for static and dynamic loadings, and computes the combined stresses. The following analyses are performed:

a. Static: Analysis of thermal, displacement, distributed, and concentrated weight loadings on piping systems;

b. Dynamic: Analysis of piping system response to seismic and fluid transient loads;

c. Stress Combination: Computes the combined stresses in the piping components in accordance with the ASME Boiler and Pressure Vessel Code, Section III (Reference 23).

The static, dynamic, and stress combination analyses can be performed independently or in sequence. Results of the static and dynamic analyses can be stored on magnetic tape for use at a later date to perform the stress combination analysis. The piping configuration can be plotted on a CalComp plotter.

The input consists of the piping system geometry, material properties, static, and dynamic loadings. Various options exist to control the length of the output. The default option generally prints only the summary of input data and final results.

PIPSYS was developed at Sargent & Lundy in 1972. It is currently maintained on a UNIVAC 1106 operating under EXEC-8.

To demonstrate the validity of the PIPSYS program the following three examples are presented.

To illustrate the validity of the static portion of PIPSYS, the problem shown in Figure C-44 was analyzed and the results compared to those given in Reference 24. Table C-26 shows the comparision of member end moments. As shown, the results from PIPSYS and Reference 24 are in good agreement.

To illustrate the validity of the stress combination analysis portion of PIPSYS, the problem outlined in Reference 25 was reanalyzed on the PIPSYS program. The layout of the piping system is shown in Figure C-45. The stress analysis is performed at location 19. The summary of loads sets and descriptions are presented in Table C-27. The results of the stress analysis are presented in Tables C-28 and C-29. The notations and equation numbers correspond to the ASME Boiler and Pressure Vessel Code (Reference 23).

It is observed that the PIPSYS results are very close to those presented in Reference 25.

To illustrate the validity of the dynamic analysis portion of PIPSYS, a problem was analyzed and the results obtained from PIPSYS were compared with those from two public domain computer programs, DYNAL (Reference 26) and NASTRAN (References 27 and 28).

Figure C-46 shows a schematic representation of the piping system analyzed. The system is modeled with simple beam elements with a total of 136 degrees-of-freedom. Figure C-47 shows the time-dependent blow-down forces at the relief valves locations. Results of PIPSYS are compared with DYNAL and NASTRAN in Table C-30 and Figure C-48. The results from all three programs are quite close.

C.13 PLFEM-II

PLFEM (Plate Finite Element Method) analyzes plane elastic bodies, plates, and shell structures by the stiffness matrix method. The program uses two finite elements, a rectangualar element and a triangular element.

Elastic spring supports and/or an elastic foundation may be considered in the analysis. Orthotropic materials may also be considered in conjuction with the rectangular element. Pressure loads, concentrated forces, nodal displacements and temperature loads may be considered in the analysis. All loading cases may be factored and/or combined in any manner.

The program output includes deflections and rotations of all joints and membrane stresses (normal, shearing, and principal) at the center of each element, the resultant moments (X, Y, twisting principal) and shears and reaction forces. An equilibrium check is made to determine the accurancy of the results.

PLFEM was developed and is maintained by Sargent & Lundy. It was originally developed on a UNIVAC 1108 in 1966. Since May 1972 it has been successfully operating on the Sargent & Lundy UNIVAC 1106 under EXEC-8.

Three sample problems are presented to demonstrate the validity of PLFEM. Plots of the computer results obtained are compared with theoretical results and results by other methods.

The first problem is an analysis of a rectangular tank filled with water which was presented by Y. K. Cheung and J. D. Davies in an article in the May 1967 issue of "CONCRETE" (Reference 29). The finite element used was presented by Zienkiewicz and Cheung in the Proceedings of the Institute of Civil Engineers in August 1964 (Reference 30). Experimental results obtained agreed exactly with the finite element results except at a few isolated points where very small differences were noted. The PLFEM grid and loading for the tank problem are shown in Figure C-49. The grid used is the same size as Cheung and Davies. Moments in three regions of the tank are plotted along with the PLFEM results in Figures C-50 through C-52.

As a second example a rectangular plate subjected to a uniform plane stress and having a circular hole in its center is analyzed. The grid used in the PLFEM analysis is shown in Figure C-53. Because of double symmetry, only one-quarter of the plate is analyzed. Results obtained from the PLFEM analysis are plotted in Figure C-54 against the exact values as given by S. Timoshenko and J. Goodier in "The Theory of Elasticity" (Reference 31).

As a final example, a square plate having a rectangular hole in its center is anlayzed for the effect of a temperature gradient through the plate. The grid used in the PLFEM analysis is shown in Figure C-55. Only one-quarter of the plate is analyzed because of the double symmetry. Moment values obtained by PLFEM are plotted for two regions of the plate in Figure C-56. For comparison, values of the moments obtained by an analysis based on the Hrennekoff frame work analogy are also shown.

C.14 PLGIRD

PLGIRD (Plate Girder Design and Investigation Program) performs design and investigation of the plate girder for axial and transverse loads. Input data for PLGIRD in its design mode consists of design criteria and material properties along with various load combinations. Input data for PLGIRD in its investigation mode consists of two input types. The first type of input data is the same as that for the design mode in order to obtain the configuration of the girder to be investigated. The second input type consists of revised load magnitudes and their locations. Allowable stresses are based on the AISC Manual of Steel Construction (Reference 32).

PLGIRD was developed by Sargent & Lundy in 1968. It is currently maintained on UNIVAC 1100 series hardwater operating under EXEC-8.

The validity of PLGIRD is demonstrated in the following.

Figure C-57 shows a plate girder configuration designed by the program under the load combination shown in Figure C-57. Hand calculations are carried out for the given loading combination and the plate girder configuration. The section properties, the actual and allowable stresses and their interaction as computed by manual methods are compared with the program results.

As illustrated in Table C-31, the solutions compare favorably.

Figure C-57 shows the loading combination for which the plate girder configuration is to be investigated. Hand calculations are carried out for the given loading combination on the plate girder configuration.

The results of the hand calculations are compared with PLGIRD's results. As illustrated in Table C-32, the solutions compare favorably.

C.15 POLSAP4

POLSAP4 (Problem Oriented Language for SLSAP4) is a preprocessor for the SLSAP4 program which allows the user to describe a structural model with commands consistent with engineering terminology. Input data consists of a free-format, self-documenting description of the problem to be solved. It eliminates the need for the normal error-prone, fixed-format numerical input with specific card sequence as described in the SLSAP4 Manual.

POLSAP4 interprets the model decription commands and generates the required fixed-format SLSAP4 input data. Control is then passed to the current operating version of the MESHG program for mesh plotting or the SLSAP4 program for the desired analysis.

The program was developed by Sarqent & Lundy in 1974. It is currently maintained on a Univac 1106 operating under EXEC-8.

Validity of the program is verified by comparison of the POLSAP4 generated data for SLSAP4 with the required input described in the SLSAP4 manual. Figures C-58 and C-60 show the POLSAP input commands for a beam and plate problem, respectively. Figures C-59 and C-61 show the generated SLSAP4 input data.

As shown, the data is correctly generated.

C.16 RSG

RSG (Response Spectrum Generator) generates dynamic response spectra (displacement, velocity, and acceleration) for single-degree-of-freedom elastic systems with various dampings, subjected to a prescribed time dependent acceleration. The program may also be used to obtain a response spectrum consistent acceleration time history in which the response spectrum of the generated acceleration time history closely envelops the given spectrum. The differential equation of motion is solved by using Newmark's β-method of numerical integration (Reference 33).

The program has the capability to apply a baseline correction in an earthquake acceleration time history as well as to obtain and to plot the Fourier transform of the given acceleration time history. Options are available to obtain plots of the given acceleration time history, the generated response spectra and their envelopes. An interpolation option to obtain an acceleration time history at equal intervals or at a smaller time interval is available. The program can also be used as a postprocessor for other programs with all its options and capabilities.

Depending upon the option, the program output includes the response spectrum, the Fourier transform of a given acceleration time history, or the response spectrum consistent acceleration time history.

RSG was developed by Sargent & Lundy in 1969. Since 1972, the program has been maintained on UNIVAC 1100 series hardware operating under EXEC-8.

To illustrate the validity of the program, two sample problems are presented. For the first problem, the response spectrum for a one-degree-of-freedom damped system as presented by Biggs (Reference 34) is determined by using RSG. The system was subjected to the sinusoidal ground acceleration as shown in Figure C-62. The maximum dynamic load factor, $(DLF)_{a,max}$ for the damped steady-state response is plotted for 20% of critical damping; that is, $\beta/\omega = 0.2$. The response spectra obtained by Biggs and from RSG are also shown in Figure C-62. As seen by this comparison, results obtained from RSG are accurate. In addition, a Fourier transform plot of the given sine wave (5 cycle/sec) time history is shown in Figure C-63. As seen from the figure, the Fourier transform shows a peak at 5 cycle/sec. As a second validation problem, a spectrum consistent time history was generated. A comparison between the desired response spectra and the response spectrum of the compatible time history is shown in Figure C-64. As seen from this figure, a good comparison is obtained.

C.17 SETTLE

SETTLE (Settlement Analysis Program) predicts the magnitude of settlements of shallow foundation caused by foundation load. Janbu's tangent modulus method (Reference 35) is used to account for the nonlinear stress-strain behavior of soil.

The distribution of contact pressure taking into consideration the effect of the foundation rigidity is taken into account by considering that the foundation is rigid and, therefore, the settlement distribution profile is linear. The variations in contact pressure can then be determined from the conditions of force equilibrium and compatibility of the foundation and soil settlements.

The foundation settlement calculation is based on the following fundamental assumptions:

1. The soil profile can be divided into homogeneous horizontal layers with uniform thickness.

2. The stress increment caused by the applied loads can be approximated by the Boussinesq formula.

3. At any point, the stress increment contributed by each of the loading areas can be superimposed to calculate the total stress increment at that point.

The settlement at a point is computed by summing the individual settlement of each soil sublayer of a predetermined thickness. The following calculations are performed for each settlement point:

1. The stress increment caused by each loading area is computed and the total influence at the center of each soil sublayer caused by all the loading areas is accumulated.

2. After the stress increments have been accumulated, the settlements they produced are computed and accumulated. Settlements are computed by Janbu's tangent modulus concept.

3. The settlement beneath a point is considered as the total of the individual settlements of each soil sublayer.

4. After the settlement at one point has been obtained, SETTLE proceeds to calculate the settlement for the next point beneath that point.

An iterative procedure is used for rigid foundations so as to make the settlement patterns of the foundation and the subsoil compatible. If settlement patterns are not compatible, the distribution of contact pressure underneath the foundation is recalculated to satisfy the deformation prescribed by the subsoil. The new contact pressure distribution is then used to compute the new settlement pattern of the subsoil. The iteration continues until the predetermined convergence criteria are satisfied.

SETTLE was developed by Sargent & Lundy in 1976. It is currently maintained on UNIVAC 1100 series hardware operating under EXEC-8.

Results of settlement computations have been validated with those of the ICES-SEPOL program (Reference 36), a public domain program, and combined with hand calculations.

The first three problems consist of a soil strata 45 ft thick with two different soil layers loaded by three rectangular loading areas with varying intensities (Figures C-65a, C-65b, C-100 and C-101). Problem 1 is run using Janbu's method, and the results are compared with hand calculation results. Terzaghi's method and the elastic method are illustrated in Problems 2 and 3, respectively, and are compared with the ICES-SEPOL program results. The results shown in Tables C-60 to C-62 indicate that settlements in each case are essentially the same for both methods.

The iteration procedure requires calculation of the movement of rigid foundations. The fourth problem presents this case. The soil properties are the same as those used in the first problem (Figure C-65a). This part of the calculation was validated by analyzing a 20 ft by 30 ft mat foundation loaded by four different loading zones (Figure C-66a). The mat was divided into six elements with a spring under the center of each element (Figure C-66b). The movement of the rigid foundation using a set of initial subgrade moduli was calculated by SETTLE and by hand. As shown in Table C-63, the results are exactly the same.

The foundations for the above-mentioned four problems are at the same elevation. The fifth problem is only used to validate the stress at different depths below the foundation levels. Once the computed stress is correct, it will lead to the exact settlement calculated by the previous approach (equivalent foundation level).

Figure C-102 shows the loading area configurations and the depths from the highest foundation level. The stress calculated by SETTLE for various foundation levels is compared with the hand calculations in Table C-64. The results are exactly the same.

C.18 SHAKE

SHAKE (Soil Layer Properties and Response/Earthquake) is a program which computes response in a horizontally layered semi-infinite system subjected to vertically traveling shear waves. Strain-compatible soil properties are computed within the program. Earthquake motion can be specified at any level of the soil profile and a resulting motion can be computed anywhere else in the profile. The method is based on the continuous solution of the shear wave equation. For soil liquefaction studies, plots of stress time histories at various levels in a soil profile can also be obtained.

The input for the program includes data for the soil profile, curves of strain versus shear moduli and damping ratios, and the input earthquake motion.

The output includes the strain-compatible soil properties, response spectra of object and computed motions and printer and CalComp plots of time histories, Fourier spectra and response spectra. Stress time history plots are also included.

A flowchart listing the various options available in SHAKE is given in Figure C-67. A single SHAKE run may use several options, but the options must be performed in a logical sequence; i.e., an option cannot be used unless all the necessary information needed for that option has been supplied by previously used options.

SHAKE was originally developed by P. B. Schnabel and J. Lysmer of the University of California at Berkeley (Reference 37). It was modified and is now maintained by Sargent & Lundy. It has been used on a UNIVAC 1106 system operating under EXEC-8 at Sargent & Lundy since October 1972.

For verification of the SHAKE program, the results from SHAKE and the public domain program QUAD4 (Reference 38) were compared for a typical problem. QUAD4, a finite element program, uses a step by step integration technique in the time domain to solve the two dimensional discrete equations of motion; SHAKE uses a numerical solution in the frequency domain to solve the one dimensional wave equation. For the comparison, it was necessary to impose suitable boundary conditions on the finite model for the QUAD4 analysis to ensure only one dimensional wave progagation.

The problem solved by SHAKE and compared with the QUAD4 results analyzed the seismic response of a 100 foot layer of dense sand, Figure C-68. The properties of the sand were considered to be as follows:

Total unit weight = 125 pcf

$(K_2)_{max} = 65$

$K_o = 0.5.$

The parameter (K_2)max relates the maximum shear modulus, ^{G}max' and effective mean pressure at any depth, y, below the surface as follows:

$$G_{max} = 1000 \left[(K_2)_{max} \right] \sigma' \, m^{1/2}$$

where

$$\sigma'_m = \left(\frac{1+2\,K_o}{3}\right)\sigma'_n$$

K_o = Coefficient of lateral pressure at rest

σ'_m = Effective vertical pressure at depth y.

Damping values and the variation of modulus values with strain were based on published data for sands (Reference 39).

The response of the sand layer was evaluated using the time history of accelerations recorded at Taft, California during the 1952 Kern County earthquake as base excitation. The ordinates of this time history were adjusted to provide a maximum acceleration of 0.15g.

The results obtained from SHAKE and the QUAD4 results are compared in Figures C-69 and C-70. The maximum shear stresses and accelerations from both solutions are compared in Figure C-69, and the response spectra of the surface motions are compared in Figure C-70. As illustrated in these figures the two solutions compare favorably.

C.19 SLSAP4

SLSAP4 (Sargent & Lundy Structural Analysis Program) performs static and dynamic structural analyses. The structure may consist of any of the following element types: 3-D truss, 3-D beam, plane stress or plane strain, 2-D axisymmetric solid, 3-D solid, thick shell, thin shell, isoparametric shell, boundary spring or pipe. The stiffnesses of the elements are evaluated for linear elastic isotropic or orthotropic materials. The structural stiffness is obtained by assembling all the individual element stiffnesses. In static analysis each load case may include element loadings: thermal loads, pressure loads, gravity loads, and concentrated nodal loads. The program calculates the nodal displacements and forces or stresses in elements for multiple load cases. There are four options available in SLSAP4 dynamic analysis: frequency calculations only, frequency calculations followed by response history analysis, frequency calculations followed by response spectrum analysis, and response history analysis by direct integration. The program performs the solution of eigenvalue/vectors using either the determinant search algorithm or the subspace iteration algorithm depending on the size of the problem. The output for the time history analysis and the response spectrum analysis includes displacement of the nodes and the element stresses.

The postprocessor, developed by Sargent & Lundy, enhances the working application of the static analysis portion of the SLSAP4 program. Its primary purpose is to perform load combination analyses for structures with multiple loading cases. The postprocessor combines files from independent runs into a single file, selects output requested by the user, and checks for the absolute upper limits of the combined element stresses. It also has the capability to calculate the plate/shell minimum required moment capacities in two orthogonal directions or to calculate the principal stresses of the elements. In addition, computer graphic capabilities for contours have been implemented for the mat foundation.

SAP was orginally developed by C. L. Wilson of the University of California at Berkeley in 1968. Sargent & Lundy currently maintains a modified SAPIV version released in 1973 (Reference 40). The program can successfully operate on either the UNIVAC 1106 operating under EXEC-8 or the CDC 7600 computer.

To demonstrate the validity of the major analytical capabilities of SLSAP4, nine of the problems used for validation are presented. These problems are taken from the SAPIV user manual (Reference 40, pp. 43-56), several other static and dynamic computer programs and classical solutions.

In the first problem, the pipe network shown in Figure C-71 is analyzed by SLSAP4 and SAPIV. The static response of the system is calculated under the combined effects of concentrated loads, vertical (y-direction) gravity loads, uniform temperature increase, and non-zero displacements imposed at one support point. The applied loads are shown in Table C-35.

The results from both programs are compared in Table C-36. Also shown are the results from ADLPIPE (Reference 41) as given in the SAPIV user manual. As shown, all of the results compare favorably.

In the second problem, a clamped spherical shell shown in Figure C-72 is analyzed for stresses produced by a uniform pressure applied on its outside surface. The model represents a five degree wedge of the shell with eighteen thin shell elements along the thirty-nine degree meridian.

The curves in Figure C-72 are plots of the meridian (ϕ) and circumferential (θ) direction surface predicted by SAPIV and SLSAP4 at the element centroid. The results are also identical.

In the third problem, a plane frame is analyzed to determine the three lowest frequencies and corresponding mode shapes. The frame and the beam element are shown in Figure C-73.

Results from SLSAP4 and SAPIV are compared in Table C-37. As shown, the results compare favorably.

The fourth problem deals with the response spectrum analysis of a pipe assemblage. This problem was originally presented in the PIPDYN user manual (Reference 42).

The model of the pipe assemblage is shown in Figure C-74. Z-moments are predicted for the local coordinates of the thirteen elements for the five lowest modes.

Table C-38 shows a comparison of the moment predictions from SLSAP4 and SAPIV. The proportional horizontal and vertical spectra are simultaneously specified. PIPDYN results, as documented in the SAPIV user manual, are also shown. All program results are in good agreement.

In the fifth problem, a cantilever beam, shown in Figure C-75, is analyzed under both uniform and concentrated loads. The beam is modeled using ten equal-length beam elements. It has a crosssectional area of 1 by 2 inch, length of 10 inches, and Young's modulus equal to 30 by 10^3 ksi. A uniform load equal to 2 kips/inch and a concentrated load of 10 kips are applied at one end of the beam.

The results from SLSAP4 are compared to analytical results obtained by Timoshenko and Gere (Reference 43). Figure C-75 shows excellent agreement between the bending moments obtained by both solutions.

In the sixth problem, a simply supported square plate under uniform loading is analyzed. A 10 in^2 by 1 inch thick plate with Poisson's ratio equal to 0.3 and Young's modulus equal to 30 x 10^3 ksi is loaded with 1 ksi pressure.

The results obtained are compared to those presented by S. Timoshenko and S. Woinowsky-Krieger (Reference 44). Bending moments M_{xx} and M_{yy} for both x and y symmetry lines obtained in the two solutions are shown in Figure C-76. The maximum bending moment which occurs at the center of the plate differs by only 1.05%.

In the seventh problem, a cantilever beam, shown in Figure C-77, is analyzed for ground acceleration. The response history of eight flexural modes is calculated by mode superposition analysis. The ground acceleration applied at node 1 is shown in Figure C-77.

The natural periods for the eight lowest flexural modes as calculated by SLSAP4 and SAPIV are given in Table C-39. The transverse deflection versus time for nodes 5 and 9 is plotted in Figure C-78. The fixed end moment versus time at element 1 is plotted in Figure C-79. The results show a favorable comparison.

For the eighth problem, the time history response of a cylindrical tube to a suddenly applied load is analyzed by mode superposition and direct integration. Results are compared with SAPIV and solutions by Timoshenko and Love (Reference 45).

One-half of the tube, shown in Figure C-80, is idealized as an assemblage of axisymmetric elements with a total of 61 degrees of freedom. The time variation of the applied load is also shown in Figure C-80.

The twenty lowest modes calculated by SLSAP4 and SAPIV by mode superposition are listed in Table C-40. Figure C-81 shows the radial displacement versus time for SLSAP4 and Timoshenko-Love. Figure C-82 shows the plot for direct time integrations results from SLSAP4 and Timoshenko-Love. As shown, results from SLSAP4 compare favorably with results from both SAPIV and Timoshenko-Love.

In the ninth problem, a circular plate on a rigid foundation, shown in Figure C-83, is analyzed. No-tension (zero stiffness in the region of the plate uplift) boundary elements are used to model the supporting foundation.

Results from SLSAP4 are compared with those from the NOBEC program (Reference 46) in Figure C-84. As shown, the results compare favorably.

C.20 SOR-III

SOR-III (Shell of Revolution) is a computer program used to analyze thin shells of revolution subjected to axisymmetric loading by employing a generalized Adams-Moulton method to numerically integrate the governing differential equations.

Arbitrary distribution of normal, tangential, and moment surface loadings, as well as edge forces and deflections may be analyzed in the axisymmetric loadings. Input of boundary conditions allow for the consideration of elastic support conditions. Temperature variations along the meridian or across the thickness may also be considered.

The program output includes shell displacements, outer fiber stresses and strains, and stress resultants.

SOR-III was developed by Knolls Atomic Power Laboratory for the United States Atomic Energy Commission (Reference 47). Version III was acquired by Sargent & Lundy in 1969 and is currently maintained on Sargent & Lundy's UNIVAC 1106 computer. The Sargent & Lundy version has been modified to punch data for plotting.

Results from this program have been frequently compared with other available solutions and other computer programs to check the validity of the program. One of these comparisons is the analysis of a circular flat reinforced concrete plate. The details of the problem and the boundary conditions are shown in Figure C-85. Results of the SOR-III analysis were compared with the finite element program, SABOR III (Reference 48). Figure C-86 shows the bending moment in the meridional and hoop directions, respectively. Figure C-87 shows the comparison of radial shear. As shown in these figures, results compare favorably.

C.21 STAND SYSTEM

STAND (Structural Analysis and Design) is an integrated system programmed to perform analysis and design of structural steel members according to the 1969 AISC specification. It consists of the following subsystems:

a. Beam Edit,

b. Rolled Beam Design,

c. Composite Beam Design,

d. Plate Girder Design,

e. Column Edit,

f. Column Design, and

g. Column Base Plate Design.

The program input consists of member geometry and basic loadings. The design is performed for specified combinations of basic loadings and overstress factors. For floor framing systems, the program is capable of automatically transferring reactions from tributary beams to supporting members and analyzing and designing for axial and vertical seismic loads. There are many design control parameters available, such as minimum and maximum depth limitations, shape of the rolled section, location of the lateral support of the compression flange, material grade or yield stress, deflection limitations, flange cutoff criterion and location of stiffeners, etc.

For columns, the program is capable of accounting for axial loading as well as uniaxial or biaxial bending.

For column base plate design, only axial load and column combinations are considered.

The program output includes the complete final design and provides the designer with sufficient intermediate information to evaluate the results. For rolled and composite beam designs, complete details of shop welded and field bolted end connections are contained in the output. Supplementary information for economic evaluation of the design is also provided.

STAND was developed and is maintained by Sargent & Lundy. Since May 1972, the program has been extensively used at Sargent & Lundy on Univac 1106 hardware operating under EXEC-8. Some of the principle applications include the design of steel floor framing using various types of horizontal structural elements and the design of columns or beam columns.

To validate STAND, results from the program were compared with results from example design problems in the "Manual of Steel Construction" (Reference 49). Six problems are given.

The first is a rolled beam design problem (Example 1, pages 2-4 and 5). A beam of 36 ksi steel is designed for a 125 kip-ft bending moment, assuming its compression flange is braced at 6 foot intervals. The results, listed in Table C-41, show that STAND selects a more efficient section.

The second is a composite beam design problem (Example 1, pp. 2-143 and 144). A noncoverplated composite interior floor beam is designed. Limits of 1-1/2 inch for dead load deflection and 1-2/10 inch for live load deflection are imposed. The results, shown in Table C-42, are nearly identical.

The third is a column design problem with three examples, (Examples 1, 2, and 5, pages 3-4, 5 and 9).

The first is the design of a W12 column of 36 ksi steel that will support a concentric load of 670 kips. The effective length with respect to its minor axis is 16 feet, and to its major axis, 31 feet.

The second is the design of an 11-foot long W12 interior bay column of 36 ksi steel that will support a concentric load of 540 kips. The column, rigidly framed at the top by 30 foot long W30 by 116 girders connected to each flange, is braced normal to its web at the top and the base.

The third is the design of a W14 column of 36 ksi steel for a tier building, 18 foot story height, that will support a 600 kip gravity load and a 190 kip-ft maximum wind moment, assuming K = 1 relative to both axes and bending is about the major axis.

The results from all three checks are identical to those in the AISC manual, and are shown in Table C-43.

The fourth problem is a plate girder design problem (Example 1, page 2-108). A welded plate girder is designed to support a uniform load of 3 kips/ft and two concentrated loads of 70 kips as shown in Figure C-88. The compression flange of the girder is laterally supported only at points of concentrated load. The close results are shown in Table C-44.

To validate the capability of analyzing and designing for axial and vertical seismic loads for floor framing systems, two examples are given.

In the fifth problem, a beam of 36 ksi steel is designed for transverse loads shown in Figure C-89 and an axial compressive load of 40 kips. The strength of concrete is 3500 ksi. The effective concrete slab width is 40 inches and the thickness of the slab is 4 inches. QL metal deck parallel to the beam is used. The distance between the top of the steel to the top of the slab is 6 inches. As seen in Table C-45 the STAND output is identical with that from hand calculations.

In the sixth problem, a beam of 36 ksi steel is designed for loads shown in Figure C-90. The strength of concrete is 3500 ksi. The effective concrete slab width is 36.2 inches and the thickness of the slab is 5 inches. QL metal deck perpendicular to the beam is used. The distance between the top of the steel and the top of the slab is 8 inches. Vertical seismic excitations are considered in the design. The STAND output is compared with hand calculations in Table C-46.

C.22 STRUDL II

STRUDL II (Structural Design Language) is primarily used for static analysis of frame and truss structures. The program is, however, capable of performing linear, static or dynamic analyses for finite element representations of structures using stiffness matrix methods. Nonlinear static problems and stability problems may also be treated.

The program is capable of analyzing plane trusses and frames, grids and elastic bodies, space trusses and frames, or three dimensional elastic solids subjected to arbitrary loads, temperature changes or specified displacements. Either earthquake accelerations or time history force may be used for dynamic analysis. Anisotropic materials may also be used. In addition to analysis, the program is capable of performing structural steel design according to the AISC Code and reinforced or prestressed concrete design according to the ACI Code.

The program output depends upon the type of finite element used and the analysis that was performed. Included in the output are displacements and member forces and moments or element stresses and moments. Eigen values, eigen vectors, and time history response or nodal response may be obtained for dynamic analyses. Member sizes may be obtained if the design portion is used.

STRUDL II was developed as part of the Integrated Civil Engineering System at the Massachusetts Institute of Technology (Reference 50). It has been in the public domain since 1968. Two versions are currently being used: one maintained by the McDonnell Douglas Automation Company on IBM 370 series hardware (Reference 51), and one maintained by UNIVAC on the 1100 series hardware (Reference 52).

C.23 TEMCO

TEMCO (Reinforced Concrete Sections Under Eccentric Loads and Thermal Gradients) analyzes reinforced concrete sections subjected to separate or combined actions of eccentric loads and thermal loads. The program can also analyze reinforced concrete sections subjected to axial force and biaxial bending. The effect of temperature is induced in the section by reactions created by the deformation restraint. No thermal loads can be specified when analysis under axial force and biaxial bending is desired.

The analysis may be done with either a cracked or an uncracked section. Material properties can be either linear or nonlinear. The program is capable of handling rectangular as well as nonrectangular sections. The effect of thermal expansion on the liner on a concrete section can be determined assuming the liner has no strength.

The program input consists of section dimensions, area and location of each layer of reinforcing steel, loads, load combinations, and material properties.

The deformations corresponding to the given eccentric loads (axial load and bending moments) are determined by an iterative procedure. Thermal load is applied on the section by inducing reactions created by the deformation restraint; i.e., there is no deformation change due to a thermal load on the section. The axial expansion can be assumed to be either free or restrained after thermal gradient is applied. An iterative procedure is employed again for finding the final strain distribution such that equilibrium of internal and external loads is satisfied.

The program output consists of an echo print of the input, combined loads, final location of neutral axis, final stresses in steel and concrete and final internal forces. Similar intermediate results (before thermal load is applied) can also be output if desired.

The program can be used to analyze a wide variety of reinforced concrete beams and columns, slabs, and containment structures subjected to various combinations of nonthermal and thermal loads.

The program was developed and is maintained by Sargent & Lundy. Since February 1972, the program has been extensively used at Sargent & Lundy on UNIVAC 1100 series hardware operating under EXEC-8.

To demonstrate the validity of TEMCO, program results are compared with hand calculated results. Five example problems are presented. The section and material properties for each problem are given in Tables C-47, C-49, and C-51 along with the applied nonthermal and thermal loads.

The first problem involves a section with two layers of steel under the action of a compressive force applied at the centerline of the section, a bending moment and a thermal gradient.

A cracked analysis of the section is required assuming nonlinear material properties.

The second problem involves a section with two layers of steel under the action of a tensile force applied at the centerline of the section, a bending moment and a thermal gradient. A cracked analysis of the section is required assuming linear material properties.

The third problem involves a section with two layers of steel under the action of a tensile force applied at the centerline of the section, a bending moment and a thermal gradient. A cracked analysis of the section is required assuming linear material properties.

The fourth problem involves a section with ten reinforcing steel bars under the action of a tensile force and biaxial bending. A cracked analysis of the section is required assuming nonlinear material properties.

The fifth problem involves a section with two liners (one on each side) under the section of nonthermal and thermal loads. A cracked analysis of the section is required assuming nonlinearmaterial properties.

The hand calculated solutions were obtained according to the following outlined procedure:

a. Assume the location of the neutral axis and the stress distribution to be the same as given by the program under the given mechanical loading.

b. Compute the strain distribution under the given mechanical loading.

c. Compute the stress resultants by integration, using the proper stress-strain relationships.

d. Check for equilibrium with external mechanical loads.

e. If equilibrium is satisfied, compute the deformation imposed on the section by the given thermal load.

f. Compute the final deformations by subtracting the thermal deformations from the mechanical deformations.

g. For free thermal expansion, compute the new axial strain such that equilibrium is satisfied, keeping the curvature constant.

h. Compute the final stress resultants by integration, using the proper stress-strain relationships.

i. Compute the thermal loads.

j. Check for equilibrium and compare program results with hand calculated results.

Results obtained using this procedure together with those computed by TEMCO for all five problems are presented in Tables C-48, C-50, and C-52.

It is concluded that results given by the program agree very well with results obtained by hand calculations and that equilibrium between internal and external forces is satisfied for all five problems.

C.24 NONLIN2

NONLIN2 (Nonlinear Dynamic Analysis of Two-Dimensional Structures) performs an inelastic analysis of plane structures subjected to static and dynamic loadings. The analysis considers the nonlinearity arising from a bilinear stress-strain or moment-curvature relationship. The dynamic analysis is performed using a step-by-step numerical integration of the equations of motion.

The NONLIN2 program has evolved into a family of programs specifically tailored to solve particular structural systems. The NONLINS program is designed especially for steel structures. The NONLINP program is especially oriented to the analysis of piping systems. The NONLINRC program is specialized for reinforced concrete structures.

Input to the program, including options, consists of the following:

a. title, problem size parameters, plasticity information, and type of excitation;

b. material properties, elastic support information, nodal coordinates, nodal load coefficients, and nodal lumped masses;

c. fixed end forces, element connectivity, and element properties; and

d. load or acceleration cards.

Output includes:

a. an echo print of input data;

b. maximum nodal displacements and rotations at specified time intervals; and

c. maximum member forces and moments for specified time intervals.

NONLIN2 was developed at Sargent & Lundy in 1971. It is currently maintained on UNIVAC 1100 series hardware operating under EXEC-8.

The following problem was used to validate the program.

The ten-story shear wall structure shown in Figure C-91 was analyzed using the NONLINRC program. The structure was subjected to the first 4 seconds of the N-S component of the El Centro earthquake (1940). Beam elements with shear deformations were used. A strain hardening of 3% was considered. The time history of the x-displacement of the upper-most node point (node 11) is plotted in Figure C-92. The results obtained from the DRAIN-2 program (Reference 53) are shown in full circles. The time history of the moment at the base is plotted in Figure C-93. The results obtained by the DRAIN-2 program are shown in full circles. The agreement is very close.

C.25 PWRRA

PWRRA (Pipe Whip Restraint Reaction Analysis) computes the maximum response to a time-dependent forcing function of a simplified model of the pipe and restraint system for pipe whip restraint designs.

PWRRA was developed at Sargent & Lundy in 1974. It is currently maintained on UNIVAC 1100 series hardware operating under EXEC-8.

To validate the program, three sample problems were analyzed.

In the first problem, the pipe cantilever beam model of Ma and Bathe (Reference 54) was analyzed using PWRRA. The pipe was modeled with 26 nodes. The pipe and restraint properties are shown in Figure C-94. The program results are compared with the results of Reference 54 and are tabulated in Table C-53.

In the second problem, a longitudinal break at an interior point of a 24-inch pipe (Reference 55, case 3L28) was analyzed using PWRRA. The pipe model, restraint properties and forcing function are shown in Figure C-95. Since the pipe properties were not available from Reference 55, built-in properties (A106 grade B carbon steel) for pipe were used. Table C-54 shows the comparison of the program results and the results given in Reference 55.

A circumferential break of the main steamline (Reference 56, Line B) was analyzed in the third problem. The pipe method is shown in Figure C-96. Built-in material properties for carbon steel A106 grade B were used in the analysis. The restraint properties shown in Figure C-96 were obtained from Reference 57. The comparison of results is tabulated in Table C-55.

The comparisons between the results of the PWRRA program and the published references show good agreement.

C.26 SLOPE

SLOPE (Slope Stability Analysis) utilizes the theory of equilibrium of forces to determine the factor of safety against sliding of any embankment or slope. It contains the Bishop, Fellenius, and Morgenstern-Price methods of two dimensional stability analysis. In the Bishop and Fellenius methods, the factor of safety against failure is estimated along a circular surface of failure, whereas any arbitrary failure surface may be chosen for the Morgenstern-Price method.

The input includes the slope geometry, soil profile, soil properties (density, cohension, and the friction angle) and the piezometric surface(s). The program also has the capability to introduce an earthquake loading assumed as a horizontal gravitational force. Once the problem is input, several execution commands can be used to determine the factor of safety by the various methods. Also, different stages such as end-of-construction, full-lake and sudden-drawdown, can be considered in a single run.

The output includes factors of safety for each trial surface and a printer plot of the slope cross section having slope profile, soil profile, water table conditions, and failure surface for the minimum factor of safety.

SLOPE was developed and put under ICES (Integrated Civil Engineering Systems) by William A. Bailey at the Massachusetts Institute of Technology. It has been in the public domain since 1967. Sargent & Lundy currently uses the SLOPE version maintained by the McDonnell Douglas Automation Company on IBM 370 series hardware (Reference 58).

C.27 SEISHANG

SEISHANG (Seismic Analysis of Hangers) is used for the analysis and design of electrical cable and HVAC duct support systems. The program computes the allowable spans for cable trays and selects the proper member sections for various types of supports. The input load functions can be in the form of dead load, live load, or dynamic response spectra.

Program input consists of geometric data, material properties, member properties, and external loadings. Program output consists of allowable spans, member sizes, and mechanical response.

SEISHANG was developed at Sargent & Lundy in 1976. It is currently maintained on UNIVAC 1100 series hardware under EXEC-8.

To demonstrate the validity of the program, two problems are presented.

A typical cable tray, shown in Figure C-97, is analyzed and compared to the solution obtained by hand calculation. The results obtained from SEISHANG and by hand calculation are compared in Table C-56. The results show good agreement.

Two typical HVAC supports, shown in Figures C-98 and C-99, are analyzed and compared to the solution obtained from the DYNAS (09.7.090-9.0) computer program (Reference 59). The results obtained from SEISHANG and from DYNAS are compared in Tables C-57 and C-58. The HVAC support shown in Figure C-98 is also analyzed by the PIPSYS (09.5.065-3.4) computer program (Reference 60). The results obtained from SEISHANG and from PIPSYS are compared in Table C-59. The results show good agreement.

C.28 VESLFAT

VESLFAT (vessel fatigue) is a computer program used to perform ASME B&PV Code Section III analyses as required by NB-3222.2 and NB-3222.4(e) for Service Levels A and B conditions defined by the user. The VESLFAT program computes primary plus secondary and total stress ranges for all events and performs a correction for elastic-plastic analysis, if appropriate.

VESLFAT is prepared, verified, and validated, controlled and maintained under Structural Integrity Associates' (SI) Quality Assurance (QA) Program, which is in compliance with the requirements of 10 CFR 50, Appendix B, 10 CFR 21, and ANSI/ASME NQA-1-1989, and meets the intent of applicable portions of ANSI N45.2. The SI QA Program was audited by NUPIC in April 2009, as part of NUPIC's triennial requalification audit program, and the QA Program was judged to be acceptable for performing safety-related work for NUPIC member utilities and research institutions.

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TABLE C-1
SPAN 1 CHARACTERISTICS AND OUTPUT RESULTS

		LEFT SIDE	MIDDLE	RIGHT SIDE
Clear Span (ft)			23.0	
Section (in.)			24.0 x 36.0	
Design Moment M_u (kip-ft)		1130.70	650.0	1204.7
Design Shear V_u (kip)		345.4	134.1	230.7
Required Area (in^2)	CBEAM	8.62	4.57	9.31
	Hand Calcs.	8.58	4.72	9.36
Required Bars	CBEAM	2 - #10 4 - #11	3 - #11	2 - #10 5 - #11
	Hand Calcs.	2 - #10 4 - #11	3 - #11	2 - #10 5 - #11
Provided Steel	CBEAM	8.78	4.68	10.34
	Hand Calcs.	8.78	4.68	10.34
Stirrups	CBEAM	#5 - at 7.0 in.**	#4 - at 14.0 in.*	#4 - at 4.0 in.*
	Hand Calcs.	#5 - at 7.0 in.**	#4 - at 14.0 in.*	#4 - at 4.0 in.*

* Note: Type 1 Stirrups -
** Note: Type 2 Stirrups -

TABLE C-2
SPAN 2 CHARACTERISTICS AND OUTPUT RESULTS

		LEFT SIDE	MIDDLE	RIGHT SIDE
Clear Span (ft)			15.5	
Section (in.)			24.0 x 27.0	
Design Moment M_u (kip-ft)		627.4	484.3	543.9
Design Shear V_u (kip)		132.9	70.4	103.6
Required Area (in^2)	CBEAM	6.51	4.77	5.42
	Hand Calcs.	6.69	4.73	5.45
Required Bars	CBEAM	2 - #10 5 - #11	4 - #11	6 - #10
	Hand Calcs.	2 - #10 5 - #11	4 - #11	6 - #10
Provided Steel	CBEAM	10.34	6.24	7.62
	Hand Calcs.	10.34	6.24	7.62
Type 1 Stirrups	CBEAM	#4 - at 6.0 in.	#4 - at 12.0 in.	#4 - at 12.0 in.
	Hand Calcs.	#4 - at 6.0 in.	#4 - at 12.0 in.	#4 - at 12.0 in.

TABLE C-3
SPAN 3 CHARACTERISTICS AND OUTPUT RESULTS

		LEFT SIDE	MIDDLE	RIGHT SIDE
Clear Span (ft)			15.5	
Section (in.)			24.0 x 27.0	
Design Moment M_u (kip-ft)		586.3	503.1	490.4
Design Shear V_u (kip)		111.8	67.6	112.8
Required Area (in^2)	CBEAM	5.88	4.97	4.84
	Hand Calcs.	5.86	4.98	4.86
Required Bars	CBEAM	6 - #10	4 - #11	4 - #10
	Hand Calcs.	6 - #10	4 - #11	4 - #10
Provided Steel	CBEAM	7.62	6.24	5.08
	Hand Calcs.	7.62	6.24	5.08
Type 1 Stirrups	CBEAM	#4 - at 10.0 in.	#4 - at 12.0 in.	#4 - at 9.0 in.
	Hand Calcs.	#4 - at 10.0 in.	#4 - at 12.0 in.	#4 - at 9.0 in.

TABLE C-4
RESULTING TOTAL LOAD

NODE	TOTAL LOAD (kips)	
	COLOAD	HAND CALCULATIONS
100A	600	600
100B	1237.5	1237.5
100C	350	350
200A	1237.5	1237.5
200B	0	0
200C	1237.5	1237.5
300A	350	350
300B	1237.5	1237.5
300C	600	600

TABLE C-5
CONCRETE STRENGTH TEST TALLY SHEET

Developed and distributed cooperatively by the Expanded Shale, Clay and Slate Institute and the National Ready Mixed Concrete Association for use with Form 2 or 3 to calculate strength control parameters.

Concrete source: XYZ CONCRETE CO. Class of Concrete:* Code B ; nominal minimum comp. strength 3450 psi; specified air-dry unit weight - lb. per cu. ft.; specified slump 3-5 in.; min. cement 545 lb. per cu. yd.; specified air content - %; max. net mixing water - lb. per cu. yd.; cement brand, source and type SEVERAL TYPE I ; coarse agg. type, source and max. size NATURAL SAND - PQR CO. ; fine agg. type and source GRAVEL - PQR CO. ; other distinguishing properties

TEST ORDER	SAMPLE ID. NO.	1971 DATE	SLUMP in.	U.W., lb/ cu. ft.	AIR CONT. %	COMPRESSIVE STRENGTH, psi							
						At 7 days				At 28 days			
						Cyl. 1	Cyl. 2	Cyl. 3	Avg.	Cyl. 1	Cyl. 2	Cyl. 3	Avg.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	TC 1	7-6	4			2380			2380	3380	3410		3395
2	TC 2	7-9	4.5			2520			2520	3530	3580		3555
3	TC 3	7-9	3			2500			2500	3535	3555		3545
4	TC 4	7-9	3.5			2300			2300	3095	3125		3110
5	TC 5	7-11	4			2400			2400	3220	3300		3260
6	TC 6	7-12	3			2500			2500	3555	3595		3575
7	TC 7	7-12	4			2820			2820	3960	3990		3975
8	TC 8	7-13	4.5			2900			2900	3755	3795		3775
9	TC 9	7-17	5			2600			2600	3640	3700		3670
10	TC 10	7-17	4.5			2840			2840	3810	3860		3835
11	TC 11	7-17	3.5			2120			2120	2965	2985		2975
12	TC-12	7-17	3.5			2210			2210	3185	3215		3200
13	TC 13	7-17	2.5			2300			2300	3095	3145		3120
14	TC 14	7-19	4			2400			2400	3050	3060		3055
15	TC 15	7-20	5			2390			2390	3470	3530		3500
16	TC 16	7-20	4.5			2790			2790	3820	3860		3840

TABLE C-5 (Cont'd)

CONCRETE CLASSIFICATION CODE____B_____.

TEST ORDER	SAMPLE ID. NO.	1971 DATE	SLUMP in.	U.W., lb/ cu. ft.	AIR CONT. %	COMPRESSIVE STRENGTH, psi							
						At 7 days				At 28 days			
						Cyl. 1	Cyl. 2	Cyl. 3	Avg.	Cyl. 1	Cyl. 2	Cyl. 3	Avg.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
17	TC 17	7-23	4			2310			2310	3035	3075		3055
18	TC 18	7-24	4			2100			2100	2800	2830		2815
19	TC 19	7-25	3			2310			2310	3400	3420		3410
20	TC 20	7-26	3			3000			3000	4200	4240		4220
21	TC 21	7-26	4			2950			2950	3790	3850		3820
22	TC 22	7-27	3			2960			2960	3990	4000		3995
23	TC 23	7-31	3			2510			2510	3660	3690		3675
24	TC 24	8-1	4			2100			2100	3210	3230		3220
25	TC 25	8-1	4.5			2340			2340	3470	3440		3455
26	TC 26	8-2	3.5			2115			2115	2990	2970		2980
27	TC 27	8-3	3.5			2220			2220	3200	3190		3195
28	TC 28	8-3	5			2170			2170	3280	3240		3260
29	TC 29	8-3	4			2215			2215	3390	3400		3395
30	TC 30	8-6	4			2160			2160	2970	2960		2965
31	TC 31	8-7	3			2300			2300	3670	3640		3655
32	TC 32	8-7	4			2800			2800	3830	3800		3815
33	TC 33	8-7	3.5			3110			3110	4470	4490		4480
34	TC 34	8-9	3.5			2560			2560	3660	3640		3650
35	TC 35	8-13	3.5			2140			2140	3390	3380		3385
36	TC 36	8-14	4			2410			2410	3600	3590		3595
37	TC 37	8-15	3.5			2120			2120	3225	3275		3250
38	TC-38	8-15	5			2100			2100	3025	3065		3045
39	TC 39	8-15	5.5			1920			1920	2650	2680		2665
40	TC 40	8-16	3.5			2200			2200	3490	3480		3485
41	TC 41	8-16	3.5			3100			3100	4040	4030		4035

TABLE C-5 (Cont'd)

CONCRETE CLASSIFICATION CODE____B_____.

TEST ORDER	SAMPLE ID. NO.	1971 DATE	SLUMP in.	U.W., lb/ cu. ft.	AIR CONT. %	COMPRESSIVE STRENGTH, psi At 7 days Cyl. 1	Cyl. 2	Cyl. 3	Avg.	At 28 days Cyl. 1	Cyl. 2	Cyl. 3	Avg.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
42	TC 42	8-17	4			2840			2840	3485	3515		3500
43	TC 43	8-20	5			2300			2300	2985	3065		3025
44	TC 44	8-20	3.5			2310			2310	3425	3445		3435
45	TC 45	8-22	5			2410			2410	3585	3615		3600
46	TC 46	8-23	3.5			2390			2390	3530	3500		3515

* Depending upon circumstances and local practice, it may not be necessary to include all of the information on class of concrete -- e.g. air content may not be specified or the data may involve several randomly used sources of cement. Any convenient code designation, letters or numbers, may be used to identify the concrete class and associate it with data on Form 2 or 3.

Comment: Form 1 provides a means of collecting the values to be used on Form 2 or 3 and displaying other data relevant to concrete control. The number of cylinders tested at a given age from the same sample may range from 1 to 3, but is typically 2 under nationally recognized specifications and codes. Thus, in many cases nothing will be recorded in one or more of Columns 7, 8, 9, 10, 11, 12, and 13.

TABLE C-6
CALCULATION OF STRENGTH TEST PARAMETERS

(For use when the number of tests being analyzed is small and, preferably when an accumulating type calculating machine or computer is available for summing strength test results and their squares.)

Developed and distributed cooperatively by the Expanded Shale, Clay and Slate Institute and the National Ready Mixed Concrete Association.

Concrete Classification Code__B__; test age__28__days; no. cylinders per test__2__.

TEST ORDER	COMP. STR., psi, x	x^2	TEST ORDER	COMP. STR., psi, x	x^2	TEST ORDER	COMP. STR., psi, x	x^2
(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)
1	3395	11526025	41	4035	16281225	81		
2	3555	12638025	42	3500	12250000	82		
3	3545	12567025	43	3025	9150625	83		
4	3110	9672100	44	3435	11799225	84		
5	3260	10627600	45	3600	12960000	85		
6	3575	12780625	46	3515	12355225	86		
7	3975	15800625	47			87		
8	3775	14250625	48			88		
9	3670	13468900	49			89		
10	3835	14707225	50			90		
11	2975	8850625	51			91		
12	3200	10240000	52			92		
13	3120	9734400	53			93		
14	3055	9333025	54			94		
15	3500	12250000	55			95		
16	3840	14745600	56			96		
17	3055	9333025	57			97		
18	2815	7924225	58			98		
19	3410	11622810	59			99		
20	4220	17808400	60			100		
21	3820	14592400	61			101		
22	3995	15960025	62			102		
23	3675	13505625	63			103		
24	3220	10368400	64			104		
25	3455	11937025	65			105		
26	2980	8880400	66			106		
27	3195	10208025	67			107		
28	3260	10627600	68			108		
29	3395	11526025	69			109		
30	2965	8791225	70			110		
31	3655	13359025	71			111		
32	3815	14554225	72			112		
33	4480	20070400	73			113		
34	3650	13322500	74			114		
35	3385	11458225	75			115		
36	3595	12924025	76			116		
37	3250	10562650	77			117		
38	3045	9272025	78			118		
39	2665	7102225	79			119		
40	3485	12145225	80			120		
Sum	137870	481053450		21110	74796300		158980 c	555849750 d

TABLE C-6 (Cont'd)

INSTRUCTIONS

1. In Column 2, enter test strengths for the proper age from Column 10 or 14 of Form 1.

2. If an accumulating calculator is not available (NOTE), square the test strengths of Column 2 and enter the results in Column 3.

3. Sum the test strengths from Column 2 and their squares from Column 3 and enter the totals opposite "Sum" at the bottom of the form. To facilitate checking, it is suggested that the partial sums for each of the 3 sections of the table be recorded, and these be added to provide the values used in the calculations. The sums of Column 2 and 3 are, respectively, the values of "C" and "D" to be used in the formulas below. "n" is the total number of tests, or the last ' 'Test Order" number for which a strength test result has been recorded.

4. Average strength, $\overline{X} = C/n = 158980 \ / \ 46$

$$= 3456 \text{ psi}$$

5. Standard deviation, $\sigma = \sqrt{nD - C^2} \ / \ n$

$$= \sqrt{46 \times 555849750 - 158980^2} \ / \ 46$$

$$= \sqrt{25569078500 - 25274640400} \ / \ 46$$

$$= \sqrt{294438100} \ / \ 46$$

$$= \frac{17159}{46}$$

$$= 373 \text{ psi}$$

NOTE: Most desk calculators are capable of accumulating the individual strength test results and their squares, thus providing the values for "C" and "D" in one operation without the necessity for entering the individual values of x^2 in Column 3. In the calculation of σ, a slide rule will not provide the needed accuracy when this method is used.

TABLE C-7
PROBABLE MAXIMUM STORY SHEARS

	PROBABLE MAXIMUM STORY SHEAR, kips	
MODE NUMBER	BIGGS	DYNAS
1	2250	2262
2	1740	1757
3	895	902

TABLE C-8
STRUCTURAL FREQUENCIES

	STRUCTURAL FREQUENCY, CPS	
MODE NUMBER	BIGGS	DYNAS
1	1.00	1.00
2	2.18	2.18
3	3.18	3.18

TABLE C-9
PROBABLE MAXIMUM STORY DISPLACEMENTS

MODE NUMBER	PROBABLE MAXIMUM STORY DISPLACEMENT, in. BIGGS	DYNAS
1	1.50	1.51
2	3.22	3.20
3	4.86	4.68

TABLE C-I0
NATURAL PERIODS FOR THE EIGHT LOWEST FLEXURAL MODES

	PERIODS IN SECONDS	
MODE NUMBER	SAPIV	DYNAS
1	525.79	525.69
2	85.368	85.369
3	30.965	30.964
4	16.059	16.060
5	9.9006	9.9010
6	6.8276	6.8279
7	5.1865	5.1866
8	4.3777	4.3778

TABLE C-11
COMPARISON OF RESULTS IN KIP/FT/SEC UNIT

RESPONSE COMPONENT	MODEL 1	MODEL 2
Maximum lateral displacement of:		
Joint 1	0.170	0.160
Joint 2	0.178	0.166
Joint 3	0.149	0.137
Joint 4	0.131	0.124
Maximum lateral acceleration of:		
Joint 1	-0.112	-0.108
Joint 2	0.268	0.267
Joint 3	0.168	0.166
Joint 4	-0.295	-0.284
Maximum moment in:		
Member 1	-28.2	-28.2
Member 2	29.3	29.2
Member 3	-19.4	-19.5

TABLE C-12
COMPARISON OF DISPLACEMENTS AND FORCES

	SOR-III				DYNAX			
Z	R-Displacement	Rotation	Hoop Force	Meridional Moment	R-Displacement	Rotation	Hoop Force	Meridional Moment
0.	-.3653-1	.6409-1	-913.4	4.43	-.3723-1	-.6511-1	-930.8	6.56
1.	.1982-1	.4196-1	495.6	-42.95	.1952-1	-.4256-1	488.0	-48.63
2.	.46049-1	.1207-1	1151.2	-33.19	.462-1	-.1222-1	1155.0	-32.84
3.	.46049-1	-.1207-1	1151.2	-33.19	.462-1	-.1222-1	1155.0	-32.84
4.	.1982-1	-.4196-1	495.6	-42.95	.1952-1	.4256-1	488.0	-48.63
5.	-.3653-1	-.6409-1	-913.4	4.43	-.3723-1	.6511-1	-930.8	6.56

TABLE C-13
VELOCITY IN THE Z-DIRECTION

AT TIME 2.2 SECONDS			AT TIME 4.0 SECONDS		
NODE	Z-ORDINATE	Z-VELOCITY NONREFLECTING	NODE	Z-ORDINATE	Z-VELOCITY NONREFLECTING
1	0.	-10.1	1	0.	-10.5
3	1.	-12.8	3	1.	-10.7
5	2.	-12.2	5	2.	-8.92
7	3.	-9.22	7	3.	-10.5
11	5.	-10.4	11	5.	-9.44
15	7.	-9.2	15	7.	-9.20
17	8.	-8.92	17	8.	-10.4
19	9.	-11.4	19	9.	-10.5
23	11.	-9.12	23	11.	-10.7
27	13.	-9.14	27	13.	-10.7
31	14.	-10.4	31	14.	-9.75
33	15.	-10.2	33	15.	-9.92
37	17.	-11.0	37	17.	-9.62
41	20.	-11.2	41	20.	-9.58

TABLE C-14
MODEL DAMPING COMPARISON

MODE	DYNAX	HAND CALCULATIONS
1	0.0352	0.0348
2	0.0368	0.0367
3	0.0430	0.0430

TABLE C-15
PROPERTIES OF STRUCTURAL MODEL

(a) MATERIAL PROPERTIES

MATERIAL	DENSITY $\frac{kip \cdot sec^2}{ft^4}$	YOUNG'S MODULUS (kip/ft^2)	POISSON'S RATIO
Concrete	0.00466	584000	0.17
Soil	0.00420	2351.5	0.42

(b) CIRCUMFERENTIAL DISTRIBUTION OF LOAD

FOURIER HARMONIC NUMBER	0	1	2	3	4
Coefficient	.2644	.3927	.1836	.0499	.0386

TABLE C-16
COMPARISON OF NODAL ACCELERATIONS IN G UNITS

(a) MAXIMUM RADIAL ACCELERATION

NODE	DYNAX		FAST	
	TIME (sec)	ACCLN.	TIME (sec)	ACCLN.
A	0.012	8.60	0.012	8.60
B	0.132	-21.09	0.132	-21.10

(b) MAXIMUM VERTICAL ACCELERATION

NODE	DYNAX		FAST	
	TIME (sec)	ACCLN.	TIME (sec)	ACCLN.
A	0.171	-17.20	0.171	-17.21
B	0.135	12.58	0.135	12.58

TABLE C –17
COMPARISON OF MAXIMUM STRESS RESULTANTS
IN K, FT UNITS AT ELEMENT C, θ = 0°

	DYNAX		FAST	
COMPONENT	TIME (sec)	FORCE K, ft	TIME (sec)	FORCE K, ft
Meridional membrane force	0.10	-9.57	0.10	-9.57
Circumferential membrane force	0.10	-9.79	0.10	-9.79
Meridional moment	0.23	15.67	0.23	15.67
Circumferential moment	0.10	-5.97	0.10	-5.97
Meridional transverse shear	0.10	-15.12	0.10	-15.12

TABLE C-18
COMPARISON OF NODAL ACCELERATIONS IN G UNITS

(a) MAXIMUM RADIAL ACCELERATION

NODE	DYNAX		FAST	
	TIME (sec)	ACCLN.	TIME (sec)	ACCLN.
A	0.012	8.73	0.012	8.74
B	0.132	-21.40	0.132	-21.41

(b) MAXIMUM VERTICAL ACCELERATION

NODE	DYNAX		FAST	
	TIME (sec)	ACCLN.	TIME (sec)	ACCLN.
A	0.171	-1745.	0.171	-1747
B	0.135	1276.	0.135	1277.

TABLE C-19
COMPARISON OF MAXIMUM STRESS RESULTANTS
IN K, FT UNITS AT ELEMENT C, $\theta = 0°$

	DYNAX		FAST	
COMPONENT	TIME (sec)	FORCE K, ft	TIME (sec)	FORCE K, ft
Meridional membrane force	0.10	-9.71	0.10	-9.71
Circumferential membrane force	0.10	-9.94	0.10	-9.94
Meridional moment	0.23	15.44	0.23	15.44
Circumferential moment	0.10	-36.81	0.10	-36.81
Meridional transverse shear	0.10	-9.06	0.10	-9.06

TABLE C-20
DISPLACEMENTS

LOCATION	VALUE, INCHES
U_1	.059492
U_2	.045083
U_3	.033292
U_4	.023913
U_5	.016642
U_6	.011246
U_7	.007491
U_8	.004830
U_9	.002874
U_{10}	.001338
f_1	16.293 kips

Post-Buckling Load

P = 21.978 kips

TABLE C-21
ANCHOR FORCES

LOCATION	VALUE, kips
f_1	16.270
f_2	15.430
f_3	14.149
f_4	12.338
f_5	10.935
f_6	9.531
f_7	6.348
f_8	4.093
f_9	2.436
f_{10}	1.134

TABLE C-22
STRAIN-COMPATIBLE SOIL PROPERTIES

EFFECTIVE SHEAR STRAIN γeff(%)	SHEAR MODULUS REDUCTION FACTOR*		FRACTION OF CRITICAL DAMPING (%)	
	CLAY	SAND	CLAY	SAND
$\leq$ 1. x 10^{-4}	1.000	1.000	2.50	0.50
3.16 x 10^{-4}	0.913	0.984	2.50	0.80
1.00 x 10^{-3}	0.761	0.934	2.50	1.70
3.16 x 10^{-3}	0.565	0.826	3.50	3.20
1.00 x 10^{-2}	0.400	0.656	4.75	5.60
3.16 x 10^{-2}	0.261	0.443	6.50	10.0
1.00 x 10^{-1}	0.152	0.246	9.25	15.5
0.316	0.076	0.115	13.8	21.0
1.00	0.037	0.049	20.0	24.6
3.16	0.013	0.049	26.0	24.6
$\geq$ 10.00	0.004	0.049	29.0	24.6

* This is the factor which has to be applied to the shear modulus at low shear strain amplitudes (here defined as 10^{-4} percent) to obtain the modulus at higher strain levels.

TABLE C-23
COMPARISON OF COMPUTED SOIL PROPERTIES
DUE TO HORIZONTAL EXCITATION

ELEMENT	SHEAR MODULUS G ksf		DAMPING RATIO λ%	
NUMBER	REF. 1	LUSH	REF. 1	LUSH
2	1537.	1512.	8.6	8.7
3	1409.	1388.	8.4	8.5
4	840.	828.	7.8	7.9
5	774.	763.	7.8	7.9

TABLE C-24
COMPARISON OF STRESSES DUE TO HORIZONTAL EXCITATION

ELEMENT	σ_x psf		σ_y psf		σ_{xy} psf	
NUMBER	REF. 1	LUSH	REF. 1	LUSH	REF. 1	LUSH
1	110.8	111.4	158.9	157.1	377.1	373.4
2	120.5	118.3	79.8	78.3	509.2	505.8
3	28.5	28.4	29.9	29.8	443.0	440.7
4	15.8	15.3	23.3	22.8	696.8	692.2
5	39.7	38.9	42.1	41.3	648.8	644.6

TABLE C-25
COMPARISON OF NODAL POINT ACCELERATIONS DUE TO HORIZONTAL AND VERTICAL EXCITATIONS

	HORIZONTAL EXCITATION				VERTICAL EXCITATION			
	X Acc. g		Y Acc. g		X Acc. g		Y Acc. g	
NODAL POINT NUMBER	REF. 1	LUSH	REF.1	LUSH	REF. 1	LUSH	REF. 1	LUSH
1	.1849	.1835	Fixed		Fixed		.1642	.1634
2	.2142	.2119	.0121	.0116	.1392	.1370	.2084	.2046
3	.1723	.1715	Fixed		Fixed		.1669	.1659
4	.1444	.1443	.0000	.0000	.0000	.0000	.1322	.1299
5	.1444	.1443	Fixed		Fixed		.1322	.1299
6	.1646	.1630	Fixed		Fixed		.1170	.1165
7	.1708	.1694	.0050	.0049	.0547	.0572	.1101	.1085
8	.1855	.1842	Fixed		Fixed		.1068	.1051

TABLE C-26
COMPARISON OF MOMENTS FOR SELECTED MEMBERS

	MOMENTS FROM REFERENCE 26 (kip-ft)	MOMENTS FROM PIPSYS (kip-ft)
M_{AB}	106.0	102.8
M_{BA}	72.0	72.5
M_{BC}	133.0	131.8
M_{CB}	133.0	131.8
M_{CD}	-133.0	-131.8
M_{DC}	-133.0	-131.8
M_{DE}	133.0	131.8
M_{ED}	86.0	84.2
M_{BE}	-158.0	-156.6
M_{EB}	-158.0	-156.6
M_{FE}	106.0	102.8
M_{EF}	72.0	72.5

TABLE C-27
SUMMARY OF LOAD SETS AT GIRTH BUTT WELD WITH CHANGE IN MATERIAL AND WALL THICKNESS

LOAD SET NO.	LOAD SET DESCRIPTION	NO. OF TRANSIENTS	P	M_x	M_y	M_z	ΔT_1	T_a (VALVE)	T_b (PIPE)	$\Delta T2$
1	Zero	5	0	0	0	0	0	70	70	0
2	Cold Hydro Test		3590	0	0	0	0	70	70	0
3	Hot Hydro Test, Up	40	2200	251.7	141.6	-7.1	2.4	400	400	0.3
4	Hot Hydro Test, Down		0	0	0	0	-2.4	70	94	-0.3
5	Plant Startup	100	2200	337.2	184.9	-936.0	0	70	70	0
6	Plant Shutdown		0	0	0	0	0	70	70	0
7	Plant Loading	18300	2200	381.6	204.4	-1169.6	0	70	70	0
8	Plant Unloading		2200	337.2	184.9	-936.0	0	70	70	0
9	Loss of Load, 4.1	80	2515	384.2	204.4	-1183.4	0	70	70	0
10	Loss of Load, 4.2		1500	345.7	186.4	-1011.4	0	70	70	0
11	N.O. + Earthquake	50	2200	408.6	463.3	-1134.1	0	70	70	0
12	N.O. – Earthquake		2200	265.8	-93.5	-737.9	0	70	70	0

TABLE C-28
SIX HIGHEST VALUES OF STRESS INTENSITY, GIRTH BUTT WELD
WITH CHANGE IN MATERIAL AND WALL THICKNESS

LOAD SET PAIR		VALUES FROM REFERENCE 25				PIPSYS PROGRAM			
		S_n	Eq. (12)	Eq. (13)	K_e	S_n	Eq. (12)	Eq. (12)	K_e
3	4	52549	*	*	1,000	52600	*	*	1,000
3	9	49883	*	*	1,000	49900	*	*	1,000
3	10	49620	*	*	1,000	49600	*	*	1,000
3	6	48013	*	*	1,000	48000	*	*	1,000
1	3	48013	*	*	1,000	48000	*	*	1,000
3	11	47728	*	*	1,000	47700	*	*	1,000

* Because S_n, calculated by Equation (10) is less than $3S_m$ Equations (12) and (13) are satisfied.

TABLE C-29
SUMMARY OF CALCULATIONS OF CUMULATIVE USAGE FACTOR, GIRTH BUTT WELD WITH CHANGE IN MATERIAL AND WALL THICKNESS

LOAD SET PAIR		VALUES BASED ON REFERENCE 25		VALUES FROM PIPSYS PROGRAM	
i	j	$\frac{S_p K_e}{2}$	USAGE FACTOR	$\frac{S_p K_e}{2}$	USAGE FACTOR
3	9	40338	0.0050	40300	0.005
4	9	34400	0.0029	34400	0.003
1	11	29806	0.0002	29800	0.000
6	11	29806	0.0020	29800	0.002
6	7	29163	0.0023	29200	0.002
2	10	26254	0.0002	26300	0.000
10	12	93170	0.0000	93200	0.000
Cumulative Usage Factor			0.0126		0.0124

TABLE C-30
MODAL FREQUENCIES (CYCLES/SEC)

MODE NUMBER	PIPSYS	NATRAN	DYNAL
1	6.07	6.085764	6.0821088
2	10.69	10.94144	10.936468
3	11.48	11.66862	11.666215
4	14.76	15.20947	15.204282
5	20.12	22.25613	22.135260
6	23.87	28.53255	28.505264
7	25.32	30.58105	30.530972
8	28.80	31.22073	31.190062
9	30.00	32.27319	32.199679
10	42.39	43.14653	43.135100
11	42.95	43.50436	43.497053
12	58.02	58.19336	57.991710
13	77.78	76.62025	71.996751
14	90.74	93.69710	92.12974
15	91.8	96.04482	95.167976
16	93.39	97.81956	97.410131
17	96.96	99.40727	98.209594
18	101.42	104.6169	101.64513
19	102.14	105.4910	103.80206
20	103.03	107.7136	107.52304

TABLE C-31
LOADS ON PLATE GIRDER CONFIGURATION

METHOD OF SOLUTION	I_{xx} (in^4)	MAX. BENDING MOMENT (k-ft)	f_b (ksi)	F_b (ksi)	fa (ksi)	Fa (ksi)	INTERACTION RATIO
Hand calculation	24701	2146.72	17.93	21.6	2.691	20.02	0.977
PLGIRD	24702	2146.78	17.92	21.6	2.70	20.00	1.0

TABLE C-32
INVESTIGATION OF PLATE GIRDER

METHOD OF SOLUTION	LEFT REACTION (k)	RIGHT REACTION (k)	BENDING MOMENT (k-ft)	I_{xx} (in^4)	I_{yy} (in^4)	BENDING STRESS f_b (ksi)	BENDING STRESS F_b (ksi)	AXIAL STRESS f_a (ksi)	AXIAL STRESS F_a (ksi)	INTERACTION RATIO	WEB SHEAR STRESS: SHEAR STRESS AT LEFT END (ksi)	WEB SHEAR STRESS: SHEAR STRESS AT RIGHT END (ksi)	WEB SHEAR STRESS: EDGE COMPRESSION ON WEB PLATE (ksi)
HAND CALCULATION	235.36	242.46	2484.65	24701	2563.5	20.74	21.6	3.23	20.02	1.138	15.185	15.64	9.425
PLGIRD	235.63	242.28	2481.24	24702	2564	20.72	21.6	3.23	20.02	1.14	15.20	15.63	9.42

TABLES C-33 AND C-34 HAVE BEEN DELETED.

TABLE C-35
APPLIED LOADS FOR SLSAP4 PIPE NETWORK

LOADING TYPE	DIRECTION X	DIRECTION Y	DIRECTION Z
Concentrated:			
At Node 3		1000.0	
At Node 4		-200.0	
At Node 8	3000.0	1000.0	2000.0
Distributed Weight		-6284.0	
Total	3000.0	-4484.0	2000.0

TABLE C-36
FORCE EQUILIBRIUM REACTIONS

NODE	SLSAP4			SAPIV			ADLPIPE		
	FX	FY	FZ	FX	FY	FZ	FX	FY	FZ
9	5643.5	-	-	5643.51	-	-	5659.	-	-
11	-	-4044.7	-	-	-4044.59	-	-	-4052.	-
12	2350.1	4023.1	-4960.9	2350.08	4023.01	-4960.70	2361.	4026.	-4966.
13	-10993.5	4505.6	2960.6	-10993.59	4505.61	2960.70	-11021.	4509.	2966.
TOTAL	-2999.9	4484.0	-2000.3	-3000.00	4484.03	-2000.00	-3001.	4483.	-2000.

TABLE C-37
PERIODS OF PLANE FRAME

MODE NUMBER	PERIOD (sec) SLSAP4	PERIOD (sec) SAPIV
1	8.182	8.183
2	2.673	2.673
3	1.543	1.543

TABLE C-38
COMPARISON OF MOMENT

ELEMENT NUMBER	MOMENT MZ (kip/in.) IN ELEMENT LOCAL COORDINATES (at element end 1)		
	SLSAP4	SAPIV	PIPDYN
1	376.9	376.9	377.0
2	30.66	30.67	30.68
3	152.9	152.9	152.9
4	100.6	100.6	100.6
5	83.27	83.27	83.27
6	46.17	46.17	46.19
7	1.081	1.081	1.082
8	21.59	21.59	21.81
9	7.052	7.052	7.038
10	7.537	7.537	7.571
11	160.3	160.3	160.4
12	78.07	78.07	78.09
13	26.08	26.08	25.80

TABLE C-39
CANTILEVER BEAM ANALYSIS -
NATURAL PERIODS FOR THE EIGHT
LOWEST FLEXURAL MODES

MODE NUMBER	PERIOD (sec) SLSAP4	PERIOD (sec) SAPIV
1	525.8	525.79
2	85.37	85.368
3	30.96	30.965
4	16.06	16.059
5	9.901	9.9006
6	6.828	6.8276
7	5.186	5.1865
8	4.378	4.3777

TABLE C-40
CYLINDRICAL TUBE ANALYSIS -
SELECTED NATURAL PERIODS

MODE NUMBER	PERIOD (sec x 10^{-3}) SLSAP4	PERIOD (sec x 10^{-3}) SAPIV
1	1.279	1.2788
5	0.6214	0.62140
10	0.3298	0.32983
15	0.1746	0.17463
20	0.1150	0.11497

TABLE C-41
ROLLED BEAM DESIGN PROBLEM

	MAXIMUM MOMENTS (kip-ft)	SECTION SELECTED	SECTION MODULUS (in^3)
AISC	125	W16 x 40	64.6
STAND	125.58	W18 x 40	68.4

TABLE C-42
COMPOSITE BEAM DESIGN PROBLEM

	BENDING MOMENTS(kip-ft)				
	CONSTRUCTION LOAD	DESIGN LOAD	MAXIMUM SHEAR kips)	STEEL SECTION	NUMBER OF SHEAR CONNECTORS
AISC	71.3	237.2	26.4	W21 x 44	42
STAND	71.3	236.5	26.3	W21 x 44	42

TABLE C-43
COLUMN DESIGN PROBLEM

ITEMS	AISC EXAMPLE 1	AISC EXAMPLE 2	AISC EXAMPLE 5
Column Design Parameters	670^k 670^k	540^k 540^k	600 kips 100 k-ft 190 k-ft 600 kips
AISC Solution	W12 x 161	W12 x 99	W14 x 142
STAND Solution	W12 x 161	W12 x 99	W14 x 142

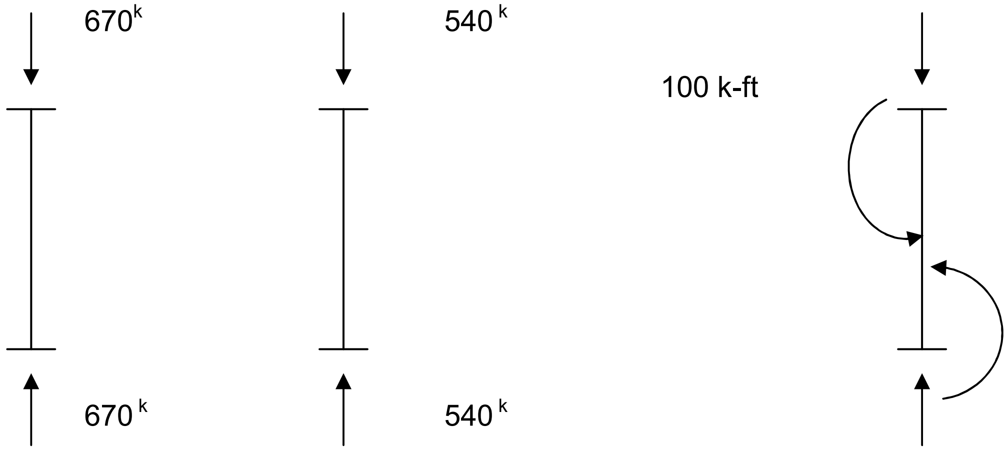


TABLE C-44
PLATE GIRDER DESIGN PROBLEM

RESULTS	AISC	STAND
Maximum Bending Moment (kip-ft)	2054	2045
Maximum Vertical Shear (kips)	142	141.3
Web Section	1 plate, 70 x 5/16	1 plate, 70 x 5/16
Flange Section	2 plates, 18 x 3/4	2 plates, 18 x 3/4
Stiffener End Spacing (ft)	3.5	3.56
Stiffener Intermediate Spacing (ft)	6.75	6.72
Area* of Stiffeners		
Furnished (in^2)	2.0	1.88

* Required area is 1.78 in^2.

TABLE C-45
COMPOSITE BEAM WITH AXIAL LOADS

	ALLOWABLE AXIAL STRESS (ksi)	
	CONSTRUCTION CASE	DESIGN CASE
STAND	20.48	20.98
Hand Calculation	20.48	20.98

TABLE C-46
COMPOSITE BEAM WITH VERTICAL SEISMIC LOADS

	FREQUENCY (Hz)	ACCELERATION		DESIGN MOMENT (k-ft)	
		OBE	SSE	OBE	SSE
STAND	10.5	1.88	2.02	1410.6	1484.2
Hand Calculation	10.49	1.875	2.025	1410.4	1483.9

TABLE C-47
INPUT FOR FIRST THREE CONCRETE SECTION ANALYSIS PROBLEMS

SECTION AND MATERIAL PROPERTIES	PROBLEM 1	PROBLEM 2	PROBLEM 3
Thickness (in.)	42.0	30.0	42.0
Width (in.)	12.0	12.0	12.0
Area of 1st steel layer (in^2)	6.25	2.25	3.12
Distance of lst steel layer (in.)	3.0	3.0	3.0
Area of 2nd steel layer (in^2)	6.25	4.0	3.12
Distance of 2nd steel layer (in.)	37.0	25.0	37.0
Concrete unit weight (lb/ft^3)	150.0	150.0	150.0
Concrete compressive strength (lb/in^2)	4000.0	4000.0	4000.0
Concrete coefficient of thermal expansion (in/in/°F)	5.56×10^{-6}	5.56×10^{-6}	5.56×10^{-6}
Steel yield strength ($kips/in^2$)	45.0	45.0	45.0
Steel modulus of elasticity ($kips/in^2$)	29000.0	29000.0	29000.0
Material properties	Nonlinear	Nonlinear	Linear
Applied axial force (kips)	-38.25	76.53	34.65
Applied bending moment (ft-kips)	129.75	-9.49	206.25
Inside temperature (°F)	82.50	67.50	247.50
Outside temperature (°F)	52.50	0.0	115.50

TABLE C-48
RESULTS OF FIRST THREE CONCRETE SECTION ANALYSIS PROBLEMS

RESULTS	PROBLEM 1	PROBLEM 2	PROBLEM 3
Equilibrating axial force given by TEMCO (kips)	-38.25	76.53	34.65
Equilibrating axial force computed by hand (kips)	-38.253	76.53	34.65
Equilibrating bending moment give by TEMCO (ft-kips)	129.75	-9.49	206.25
Equilibrating bending moment computed by hand (ft-kips)	129.752	-9.493	206.25
Thermal moment given by TEMCO (ft-kips)	-54.58	-21.07	-137.75
Thermal moment computed by hand (ft-kips)	-54.585	-21.071	-137.757

TABLE C-49
INPUT FOR TENSILE FORCE AND BIAXIAL BENDING PROBLEM

SECTION AND MATERIAL PROPERTIES	PROBLEM 4
Thickness (in.)	42.0
Width (in.)	12.0
Area of each steel bar (in^2)	1.25
Number of steel bars	10.0
Concrete unit weight (lb/ft^3)	150.0
Concrete compressive strength (lb/in^2)	4000.0
Steel yield strength ($kips/in^2$)	45.0
Steel modulus of elasticity ($kips/in^2$)	29000.0
Material properties	Nonlinear
Applied axial force (kips)	21.0
Applied x bending moment (ft-kips)	125.0
Applied y bending moment (ft-kips)	125.0

TABLE C-50
RESULTS FROM TENSILE FORCE AND BIAXIAL BENDING PROBLEM

RESULTS	PROBLEM 4
Equilibrating axial force given by TEMCO (kips)	20.999
Equilibrating axial force computed by hand (kips)	22.733
Equilibrating x bending moment given by TEMCO (ft-kips)	125.000
Equilibrating x bending moment computed by hand (ft-kips)	124.630
Equilibrating y bending moment given by TEMCO (ft-kips)	125.000
Equilibrating y bending moment computed by hand (ft-kips)	123.753

TABLE C-51
INPUT FOR NONTHERMAL AND THERMAL LOADS PROBLEM

SECTION AND MATERIAL PROPERTIES	PROBLEM 5
Thickness (in.)	70.92
Width (in.)	12.00
Number of reinforcement layers	6
Area of each reinforcement layer (in^2)	3.96
Concrete unit weight (lb/ft^3)	150.00
Concrete compressive strength (lb/in^2)	4000.00
Concrete coefficient of thermal expansion (in/in/°F)	0.556×10^{-5}
Reinforcing steel yield strength ($kips/in^2$)	30000.00
Material properties	Nonlinear
Number of liners	2
Thickness of each liner (in.)	0.375
Temperature in the first liner (°F)	200.00
Temperature in the second liner (°F)	100.00
Effective eccentricity of the first liner (in.)	20.00
Effective eccentricity of the second liner (in.)	60.00
Liner yield strength ($kips/in^2$)	30.00
Liner modulus of elasticity ($kips/in^2$)	30000.00
Liner coefficient of thermal expansion (in/in/°F)	0.65×10^{-5}
Applied axial force (kips)	165.40
Applied bending moment (ft-kips)	-35.23
Applied thermal axial force (kips)	90.00
Applied thermal bending moment (ft-kips)	900.00
Applied shear force (kips)	160.71

TABLE C –52
RESULTS FROM NONTHERMAL AND THERMAL LOADS PROBLEM

RESULTS	PROBLEM 5
Equilibrating axial force given by program (kips)	293.22
Equilibrating axial force computed by hand (kips)	293.354
Equilibrating bending moment given by program (ft-kips)	161.65
Equilibrating bending moment computed by hand (ft-kips)	161.32
Required shear reinforcement area given by program (in^2)	0.486
Required shear reinforcement area computed by hand (in^2)	0.486

TABLE C-53
COMPARISON OF RESULTS FOR
EXAMPLE 1 OF PWRRA

RESULTS OBTAINED FROM	MAXIMUM TIP DISPLACEMENT (inches)
Ma and Bathe	5.1
PWRRA	5.506

TABLE C-54
COMPARISON OF RESULTS FOR EXAMPLE 2 OF PWRRA

RESULTS OBTAINED FROM	MAXIMUM TIP DISPLACEMENT (inches)	MAXIMUM RESTRAINT LEFT RESTRAINT (inches)	DISPLACEMENT RIGHT RESTRAINT (inches)
Bisconti, et al. (Reference 55)	27.40	5.59	4.60
PWRRA	27.47	5.46	5.32

TABLE C-55

COMPARISON OF RESULTS FOR EXAMPLE 3 of PWRRA

RESULTS OBTAINED FROM	MAXIMUM RESTRAINT DEFLECTIONS (inches)	MAXIMUM RESTRAINT REACTIONS (kips)
GAAA (Reference 56)	6.216	651.78
PWRRA	6.0758	648.48

TABLE C-56
ALLOWABLE SHEAR, MOMENT AND SPAN
OF CABLE TRAY

	SEISHANG	HAND CALCULATION
Vertical shear, static (kip)	16.05	16.05
Postive bending moment, static (k-in.)	50.64	50.83
Negative bending moment, static (k-in.)	57.62	57.64
Vertical shear seismic (kip)	20.84	20.81
Horizontal shear, seismic (kip)	12.84	12.83
Positive bending moment, seismic (k-in.)	67.51	67.61
Negative bending moment, seismic (k-in.)	76.83	76.82
Horizontal bending moment, seismic (k-in.)	153.61	153.59
Span (ft)	20.78	20.75

TABLE C-57
RESPONSE OF THE CEILING MOUNTED SUPPORT

		SEISHANG	DYNAS
Horizontal period (sec)		0.1742	0.1765
Vertical period (sec)		0.0092	0.0093
Forces and moments due to horizontal seismic:			
Vertical element (No. 1)	axial (lb)	1600	1607
	shear (lb)	770	772
	bending (lb-in.)	17100	17208
Horizontal element (No. 9)	axial	25	26
	shear (lb)	302	304
	bending (lb-in.)	10900	10944
Forces and moments due to vertical seismic:			
Vertical element (No. 1)	axial (lb)	383	340
	shear (lb)	0	2
	bending (lb-in.)	30	24
Forces and moments due to dead load:			
Vertical element (No. 1)	axial (lb)	776	774
	shear (lb)	0	0
	bending (lb-in.)	30	0

TABLE C-58
RESPONSE OF THE WALL MOUNTED SUPPORT

		SEISHANG	DYNAS
Horizontal period (sec)		0.0067	0.0067
Vertical period (sec)		0.1065	0.1080
Forces and moments due to horizontal seismic:			
Vertical element (No. 6)	axial (lb)	0	1
	shear (lb)	2	2
	bending (lb-in.)	35	48
Horizontal element (No. 11)	axial (lb)	101	105
	shear (lb)	2	2
	bending (lb-in.)	23	24
Forces and moments due to vertical seismic:			
Vertical element (No. 6)	axial (lb)	39	0
	shear (lb)	131	128
	bending (lb-in.)	2700	2676
Forces and moments due to dead load:			
Vertical element (No. 1)	axial (lb)	717	702
	shear (lb)	303	329
	bending (lb-in.)	4910	5208

TABLE C-59
INTERACTION COEFFICIENTS OF THE
CEILING MOUNTED SUPPORT

INTERACTION COEFFICIENT	SEISHANG	PIPSYS
Vertical Element (No. 2)	0.617	0.620
(No. 5)	0.520	0.516
Horizontal Element (No. 6)	0.683	0.678
Brace Element (No. 3)	0.569	0.553

TABLE C-60
SETTLEMENT FOR PROBLEM 1 OF SETTLE VALIDATION

	MAGNITUDE OF SETTLEMENT (in.)	
SETTLEMENT POINT	JANBU'S METHOD	HAND CALCULATION
1	0.0650	0.0650
2	0.0633	0.0633
3	0.0636	0.0636
4	0.0618	0.0618
5	-0.0737	-0.0737
6	-0.0751	-0.0751
7	-0.0677	-0.0677
8	-0.0731	-0.0731
9	0.0636	0.0636
10	0.0665	0.0665

TABLE C-61
SETTLEMENT FOR PROBLEM 2 OF SETTLE VALIDATION

	MAGNITUDE OF SETTLEMENT (ft.)	
SETTLEMENT POINT	SETTLE	ICES-SEPOL
1	0.0054	0.0054
2	0.0053	0.0053
3	0.0053	0.0053
4	0.0051	0.0051
5	-0.0061	-0.0061
6	-0.0062	-0.0062
7	-0.0056	-0.0056
8	-0.0060	-0.0060
9	0.0053	0.0053
10	0.0055	0.0055

TABLE C-62
SETTLEMENT FOR PROBLEM 3 OF SETTLE VALIDATION

	MAGNITUDE OF SETTLEMENT (ft.)	
SETTLEMENT POINT	ELASTIC METHOD	ICES-SEPOL
1	0.014	0.015
2	0.013	0.013
3	0.013	0.013
4	0.011	0.011
5	-0.001	-0.002
6	-0.004	-0.004
7	-0.001	-0.001
8	-0.004	-0.004
9	0.014	0.014
10	0.015	0.015

TABLE C-63
SETTLEMENT OF RECTANGULAR RIGID MAT
FOUNDATION FOR PROBLEM 4 OF SETTLE VALIDATION

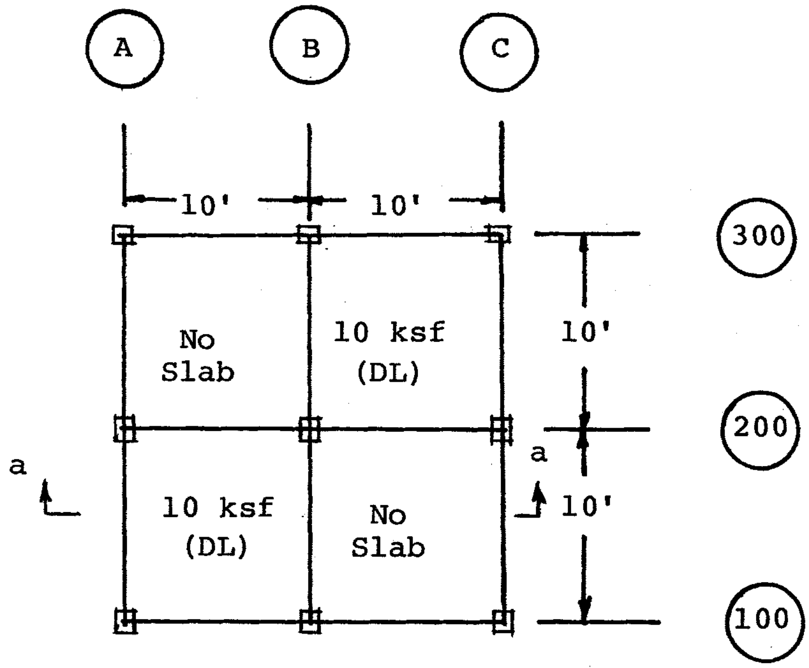
METHOD	δ (ft.)	θ_x (deg.)	θ_y (deg.)
SETTLE	0.01895	0.0408	0.0141
Hand Calculation	0.01895	0.0408	0.0141

NOTES:

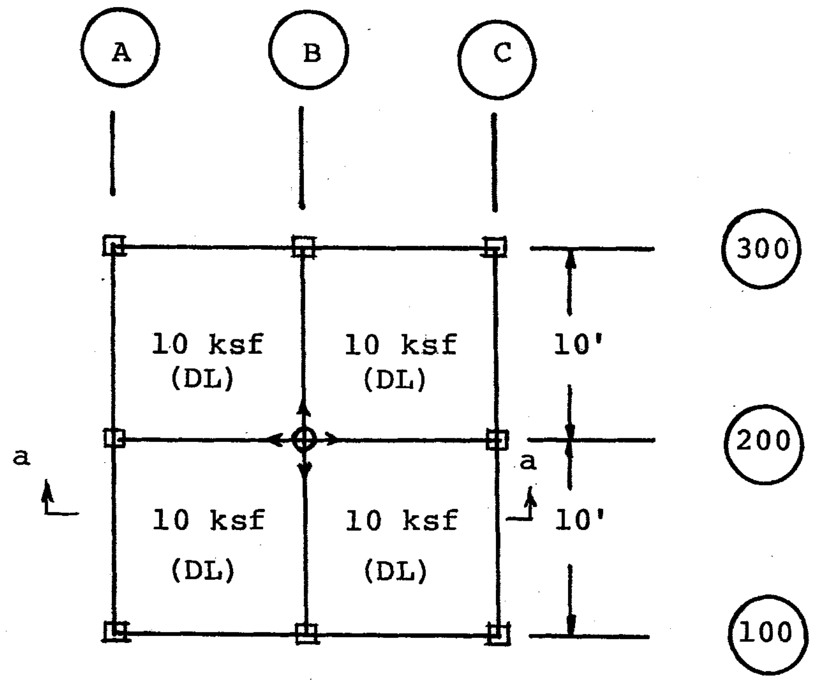
1. δ = Uniform deformation
2. θ_x = Rotation about x-axis
3. θ_y = Rotation about y-axis

TABLE C-64
STRESS FOR PROBLEM 5 OF SETTLE VALIDATION

			STRESS (psf)	
X-AXIS	Y-AXIS	DEPTH FROM HIGHEST FOUNDATION LEVEL (Ft)	HAND CALCULATION	SETTLE
583.125	321.375	13.5	817.7488	817.7487
		27.5	693.7432	693.7433
		42.5	550.6955	550.6955
635.375	321.375	13.5	808.9652	808.9652
		27.5	640.3041	640.3041
		42.5	463.8836	463.8836
583.125	376.125	13.5	817.7235	817.7234
		27.5	675.3727	675.3728
		42.5	492.0424	492.0424
635.375	376.125	13.5	808.9374	808.9375
		27.5	619.8090	619.8090
		42.5	396.4908	396.4908
674.750	321.375	13.5	-815.0380	-815.0379
		27.5	-389.9123	-389.9123
		42.5	-149.2998	-149.2998
674.750	376.125	13.5	-815.0597	-815.0597
		27.5	-405.5816	-405.5816
		42.5	-199.1152	-199.1152
590.950	414.250	13.5	-904.4085	-904.4085
		27.5	-479.3660	-479.3660
		42.5	-166.6742	-166.6743
655.650	414.250	13.5	-959.5652	-959.5651
		27.5	-621.7043	-621.7043
		42.5	-326.0188	-326.0188



(a) Second Floor



200B is an empty node where no column exists.
The load is released in x- and y- directions.

(b) First Floor

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FIGURE C-1

PLAN, ELEVATION, AND LOADING FOR
COLOAD VALIDATION PROBLEM

SAMPLE PROBLEM TO VERIFY PROGRAM 'CONCRETE'

```
NUMBER OF DIFFERENT LOCATIONS  - - -   1
NUMBER OF DIFFERENT CONCRETES USED -   1
CURING PERIODS SPECIFIED           - -   7  28   0
```

SPECIFIED DESIGN STRENGTHS OF CONCRETES

		7-DAY PSI	28-DAY PSI	90-DAY PSI
CONCRETE NO. 1	AA-3	2200.0	3000.0	.0

THE FOLLOWING RESULTS ARE FOR GOOD QUALITY CONTROL WITH

```
EXPECTED COEFFICIENT OF VARIATION       =   15.0 %
EXPECTED WITHIN TEST COEFF. OF VARIATION=    5.0 %
```

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FIGURE C-2

CONSOLIDATED RESULTS FOR DIFFERENT LOCATIONS (CONCRETE)

(SHEET 1 of 3)

* * * * ANALYSIS OF RESULTS FOR SAMPLES FROM LOCATION 1 * * * *

CONCRETE MIX USED - - AA-3

* * * * NO 7 DAY CYLINDER TESTED FROM THIS LOCATION YET

* * * * RESULTS FOR 28-DAY STRENGTH

SL NO	CYL 1 PSI	CYL 2 PSI	AVG PSI	MOVING AVG	RANGE PSI	SAMP NO	CONC	SAMP DATE MO	DY	YR	TEST DATE MO	DY	YR	CEM	SLUMP IN	AIR %	TEMP DEG F	FRACTURE CYL 1	CYL2	CURING PERIOD
1	3380.0	3410.0	3395.0	3395.0	30.0	1	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
2	3530.0	3580.0	3555.0	3475.0	50.0	2	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
3	3535.0	3555.0	3545.0	3498.3	20.0	3	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
4	3095.0	3125.0	3110.0	3403.3	30.0	4	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
5	3220.0	3300.0	3260.0	3305.0	80.0	5	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
6	3555.0	3595.0	3575.0	3315.0	40.0	6	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
7	3960.0	3990.0	3975.0	3603.3	30.0	7	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
8	3755.0	3795.0	3775.0	3775.0	40.0	8	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
9	3640.0	3700.0	3670.0	3806.7	60.0	9	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
10	3810.0	3860.0	3835.0	3760.0	50.0	10	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
11	2965.0	2985.0	2975.0	3493.3	20.0	11	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
12	3185.0	3215.0	3200.0	3336.7	30.0	12	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
13	3095.0	3145.0	3120.0	3098.3	50.0	13	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
14	3050.0	3060.0	3055.0	3125.0	10.0	14	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
15	3470.0	3530.0	3500.0	3225.0	60.0	15	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
16	3820.0	3860.0	3840.0	3465.0	40.0	16	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
17	3035.0	3075.0	3055.0	3465.0	40.0	17	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
18	2800.0	2830.0	2815.0	3236.7	30.0	18	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
19	3400.0	3420.0	3410.0	3093.3	20.0	19	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
20	4200.0	4240.0	4220.0	3481.7	40.0	20	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
21	3790.0	3850.0	3820.0	3816.7	60.0	21	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
22	3990.0	4000.0	3995.0	4011.7	10.0	22	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
23	3660.0	3690.0	3675.0	3830.0	30.0	23	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
24	3210.0	3230.0	3220.0	3630.0	20.0	24	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
25	3470.0	3440.0	3455.0	3450.0	30.0	25	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
26	2990.0	2970.0	2980.0	3218.3	20.0	26	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
27	3200.0	3190.0	3195.0	3210.0	10.0	27	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
28	3280.0	3240.0	3260.0	3145.0	40.0	28	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
29	3390.0	3400.0	3395.0	3283.3	10.0	29	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
30	2970.0	2960.0	2965.0	3206.7	10.0	30	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
31	3670.0	3640.0	3655.0	3338.3	30.0	31	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
32	3830.0	3800.0	3815.0	3478.3	30.0	32	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
33	4470.0	4490.0	4480.0	3983.3	20.0	33	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
34	3660.0	3640.0	3650.0	3981.7	20.0	34	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
35	3390.0	3380.0	3385.0	3838.3	10.0	35	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
36	3600.0	3590.0	3595.0	3543.3	10.0	36	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
37	3225.0	3275.0	3250.0	3410.0	50.0	37	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
38	3025.0	3065.0	3045.0	3296.7	40.0	38	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
39	2650.0	2680.0	2665.0	2986.7	30.0	39	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
			*** BAD CONTROL-MOVING AVERAGE																	
40	3490.0	3480.0	3485.0	3065.0	10.0	40	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO [illegible]	28

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FIGURE C-2

CONSOLIDATED RESULTS FOR DIFFERENT LOCATIONS (CONCRETE)

(SHEET 2 of 3)

41	4040.0	4030.0	4035.0	3395.0	10.0	41	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
42	3485.0	3515.0	3500.0	3673.3	30.0	42	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
43	2985.0	3065.0	3025.0	3520.0	80.0	43	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
44	3425.0	3445.0	3435.0	3320.0	20.0	44	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
45	3585.0	3615.0	3600.0	3353.3	30.0	45	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28
46	3530.0	3500.0	3515.0	3516.7	30.0	46	1	0	0	0	0	0	0	0	.0	.0	0	NO INF	NO INF	28

```
**** **** **** ****

NUMBER OF 28-DAY SAMPLES COLLECTED FOR THIS LOCATION - - - - - 46

REQUIRED MEAN OBSERVED STRENGTH- - - 3501.0
MEAN STRENGTH OF THE CONCRETE  - - - 3456.1    ALLOW. DESIGN STR. - 2955.1    SPEC. DESIGN STR. - 3000.0
STANDARD DEVIATION OF THE STRENGTH -  373.0    C.O.V.  PERCENT    -   10.8    EXPECTED C.O.V.   -   15.0
WITHIN TEST STANDARD DEVIATION - - -   28.1    C.O.V.  PERCENT    -     .8    EXPECTED C.O.V.   -    5.0
AVERAGE OBSERVED RANGE - - - - - - -   31.7

NUMBER OF BAD SAMPLES - AVERAGE STRN  - - - -  0
NUMBER OF BAD SAMPLES - MOVING RANGE  - - - -  0
NUMBER OF TIMES INEFFICIENT TESTING NOTICED -  0

*** RESULTS OF CONTROL ACCORDING TO ACI MANUAL ***

NUMBER OF SAMPLES FALLING BELOW FC - - - - - - - -  5    PERCENT OF SAMPLES COLLECTED -  10.87
NUMBER OF SAMPLES FALLING BELOW  FC-500  - - - - -  0    PERCENT OF SAMPLES COLLECTED -    .00
NUMBER OF TIMES MOVING AVG FELL BELOW FC    - - - -  1    PERCENT OF SAMPLES COLLECTED -   2.17

* * * * NO  0 DAY CYLINDER TESTED FROM THIS  LOCATION YET
```

Note: Scanned image of computer print-out

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FIGURE C-2

CONSOLIDATED RESULTS FOR DIFFERENT LOCATIONS (CONCRETE)

(SHEET 3 of 3)

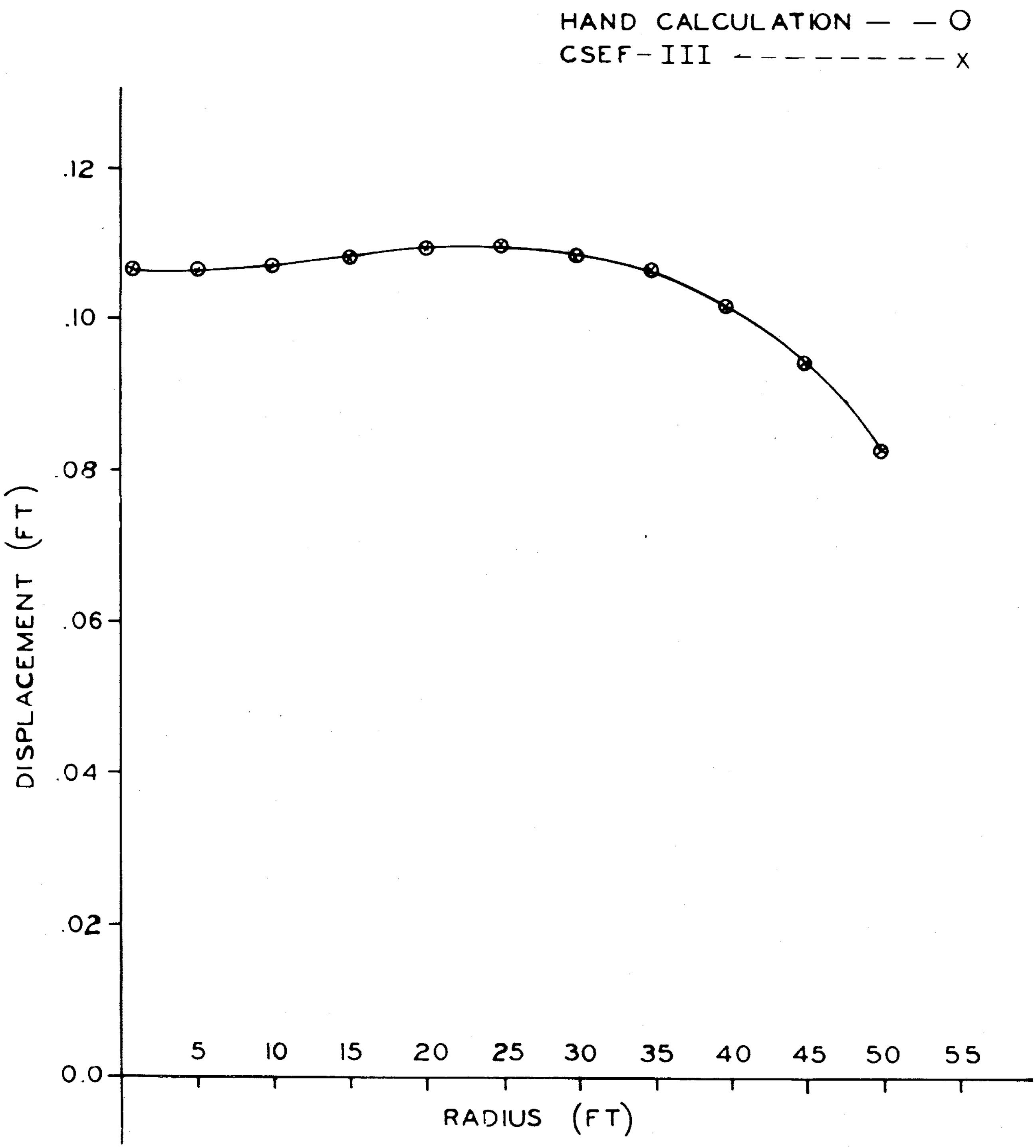

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FIGURE C-3

COMPARISON OF DEFLECTION OF A CIRCULAR PLATE DUE TO UNIFORM PRESSURE AND AXI-SYMMETRIC EDGE MOMENT (CSEF-III VS. HAND CALCULATION)

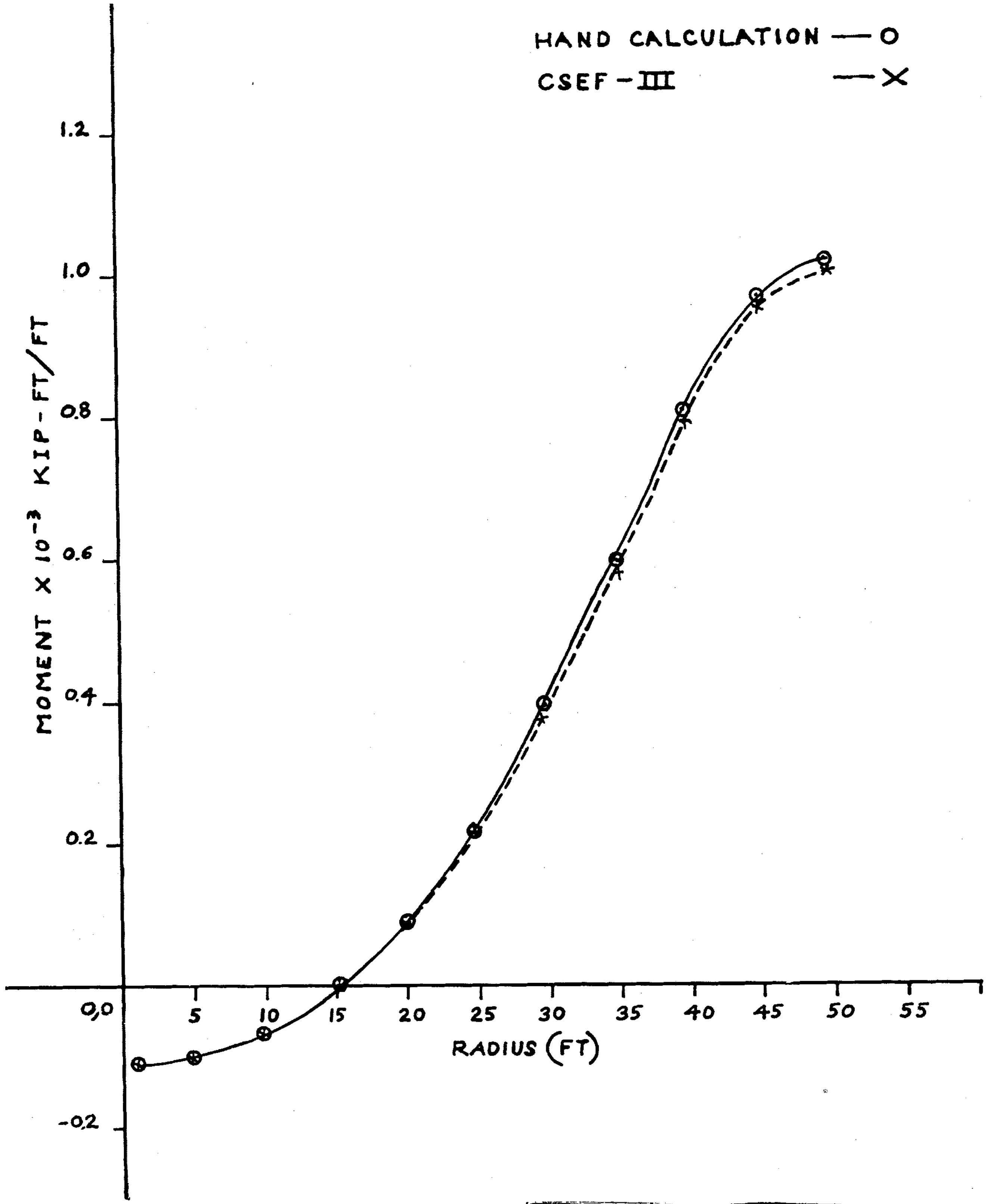

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FIGURE C-4

RADIAL MOMENT DUE TO UNIFORM PRESSURE
AND AXISYMMETRIC EDGE MOMENT
(CSEF-III VS. HAND CALCULATION)

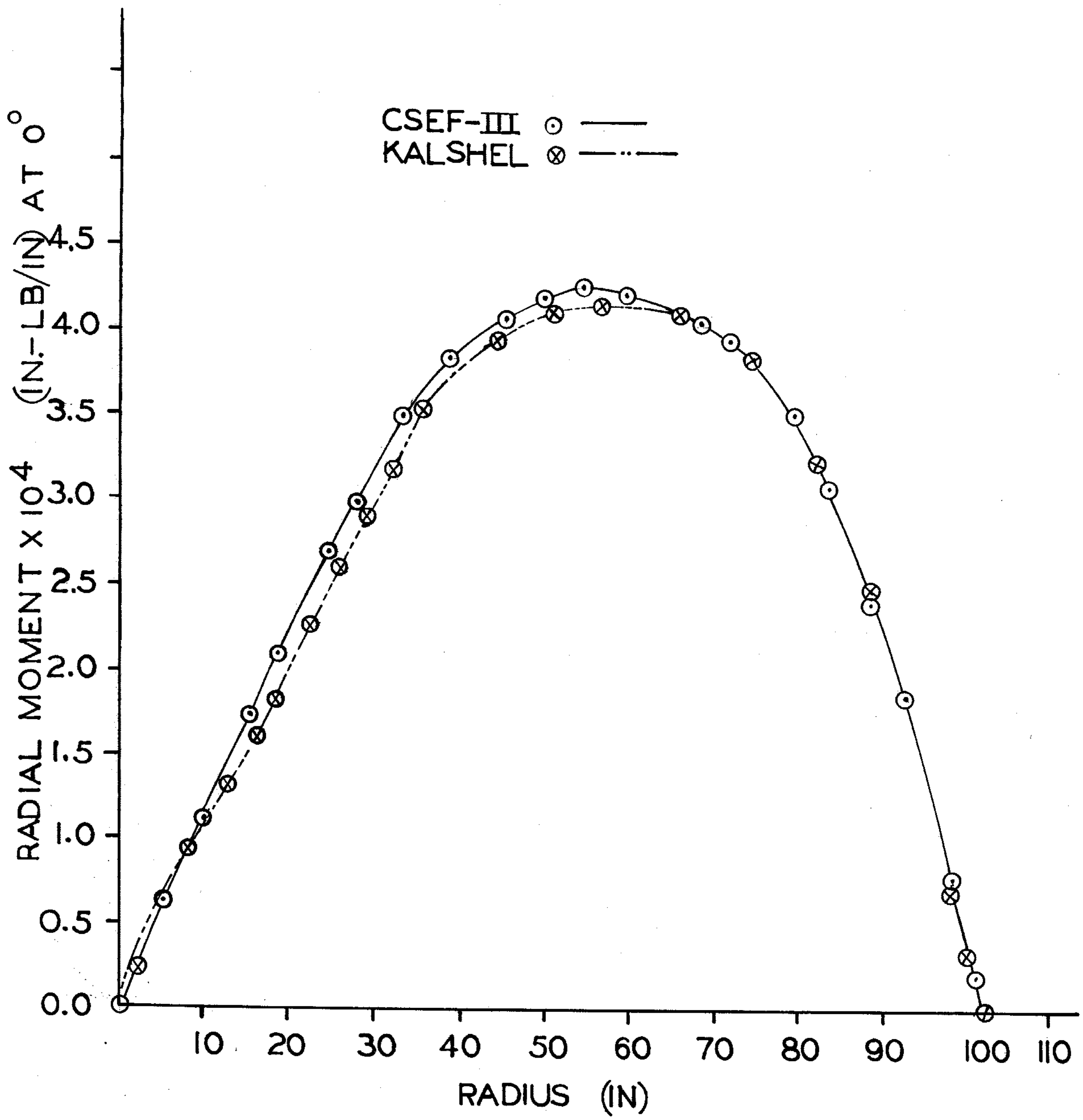

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FIGURE C-5

SIMPLY SUPPORTED CIRCULAR PLATE,
LINEARLY VARYING PRESSURE LOAD
FOR RADIAL MOMENT

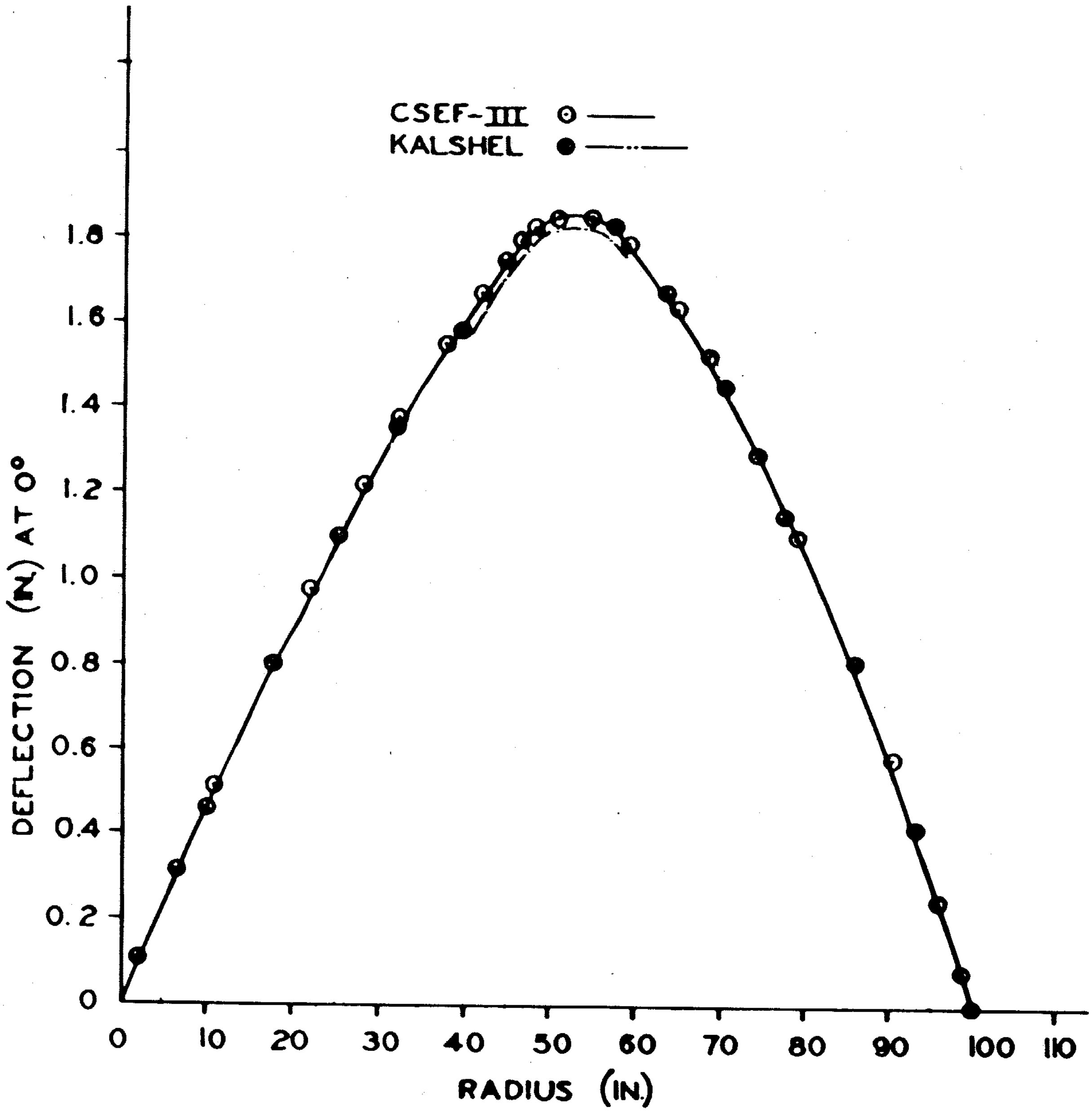

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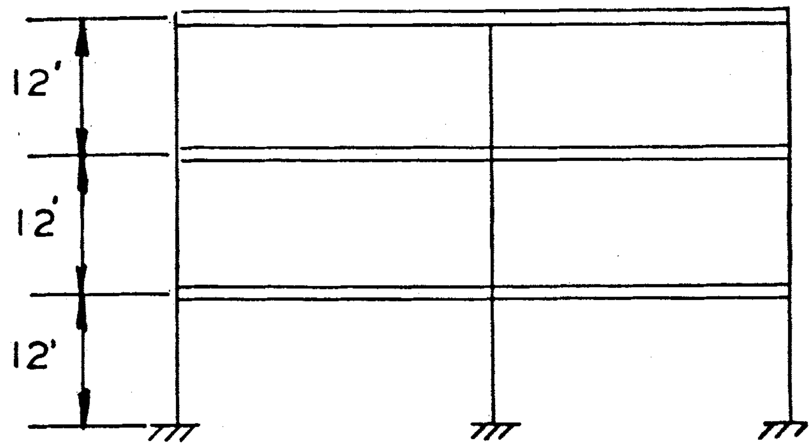
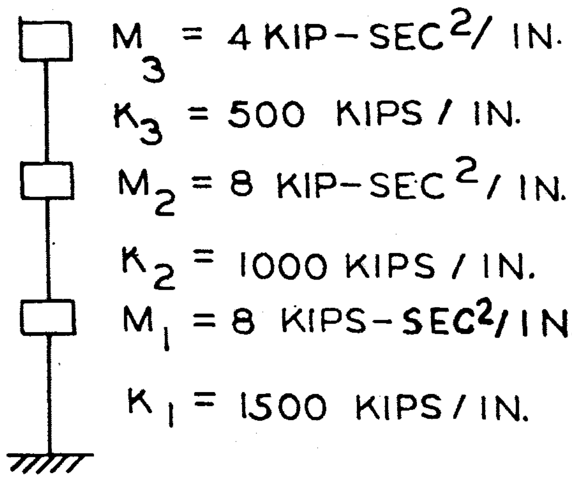
FIGURE C-6

SIMPLY SUPPORTED CIRCULAR PLATE,
LINEARLY VARYING PRESSURE LOAD
FOR DEFLECTION (CSEF-III VS. KALSHEL)



(A)



(B)

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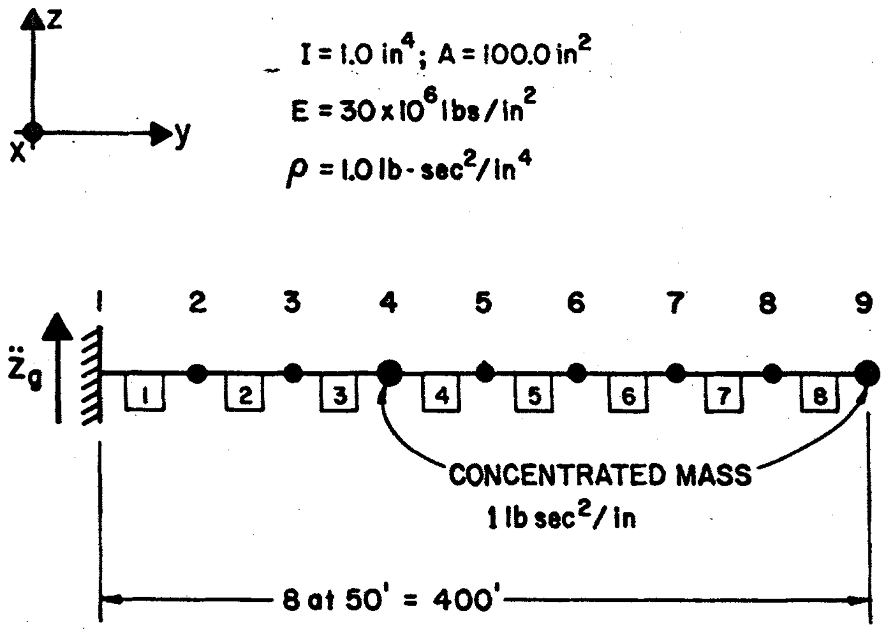
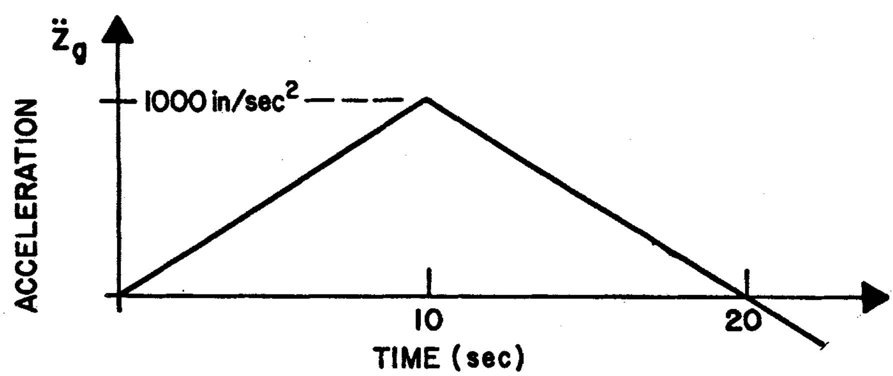
FIGURE C-7

THREE-STORY SHEAR BUILDING MODEL FOR DYNAS



(a) NODE AND BEAM NUMBER ASSIGNMENTS FOR THE CANTILEVER MODEL



(b) GROUND ACCELERATION APPLIED AT NODE 1

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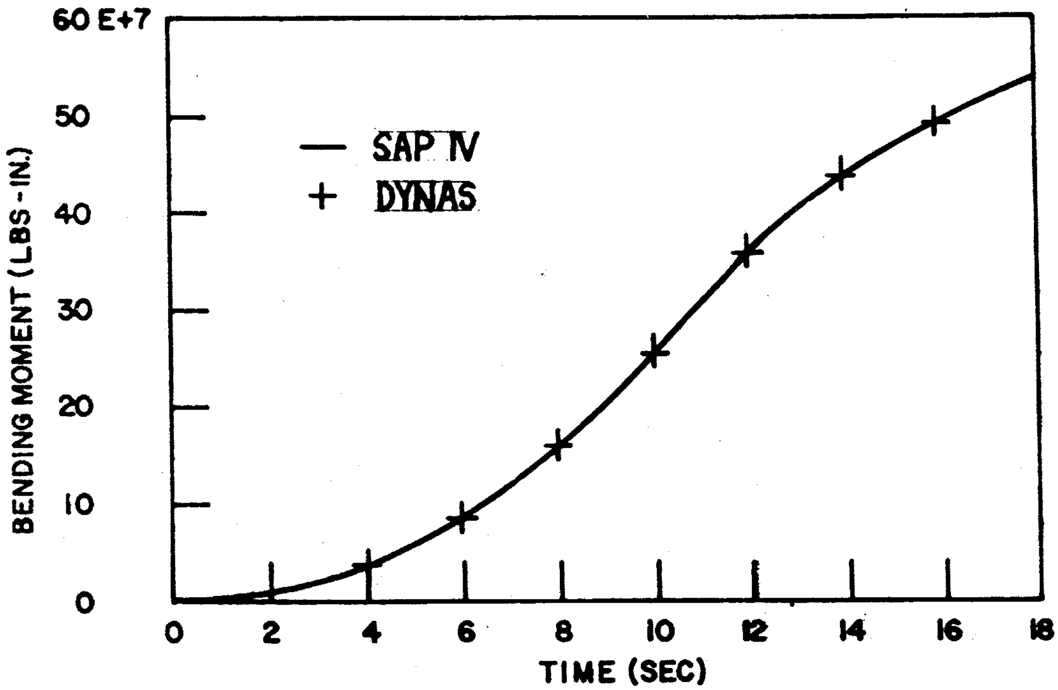
FIGURE C-8

RESPONSE HISTORY ANALYSIS OF CANTILEVER BEAM FOR DYNAS VALIDATION PROBLEM NO. 2



MOMENT AT NODE 1
(FIXED END OF CANTILEVER)

FIGURE C-9

COMPARISON OF CANTILEVER RESPONSES FROM DYNAS AND SAPIV (DYNAS)

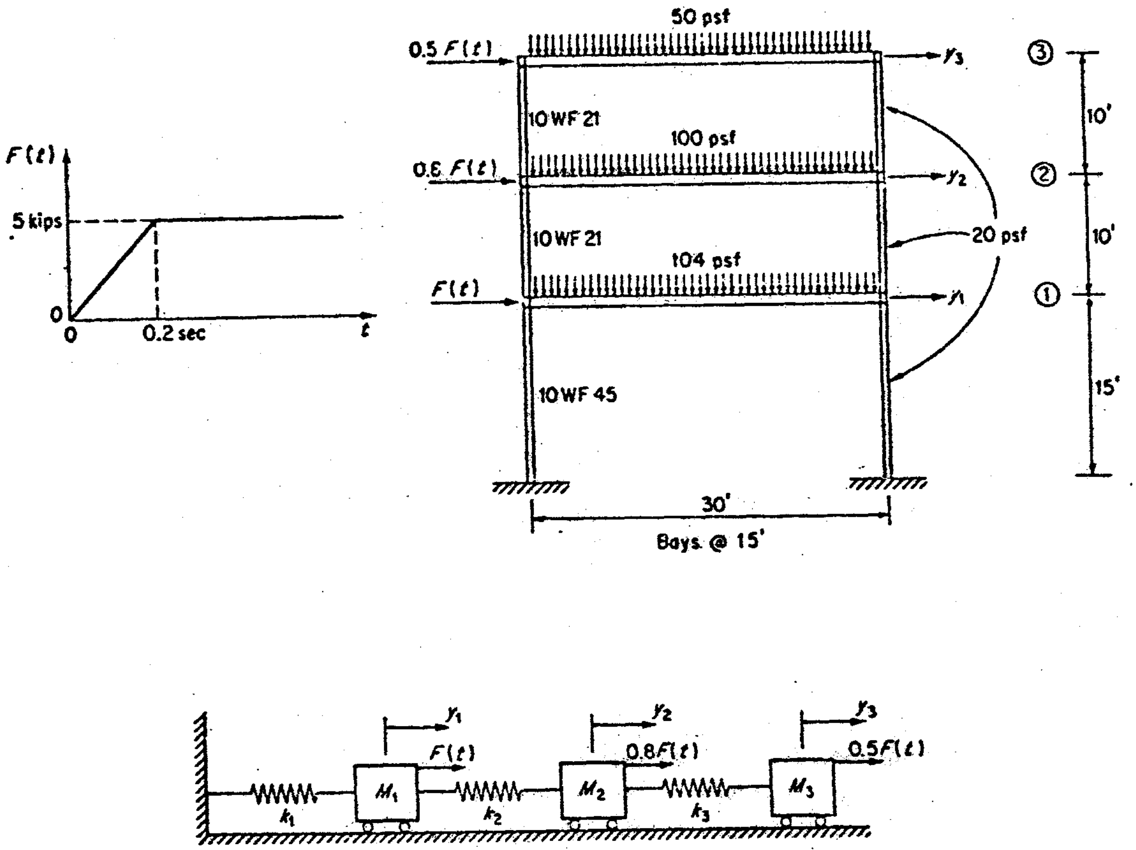


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FIGURE C-10

STEEL FRAME WITH RIGID GIRDERS FOR DYNAS VALIDATION PROBLEM NO. 3

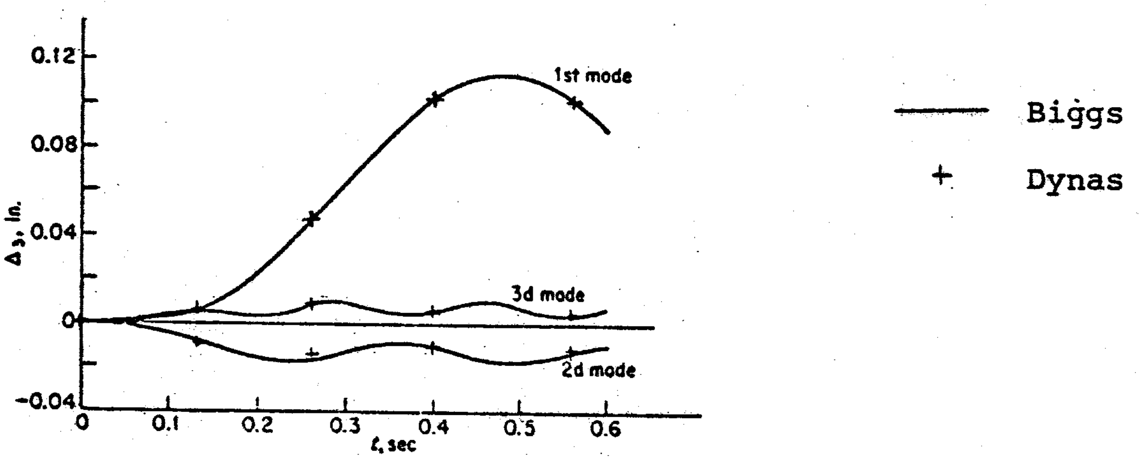

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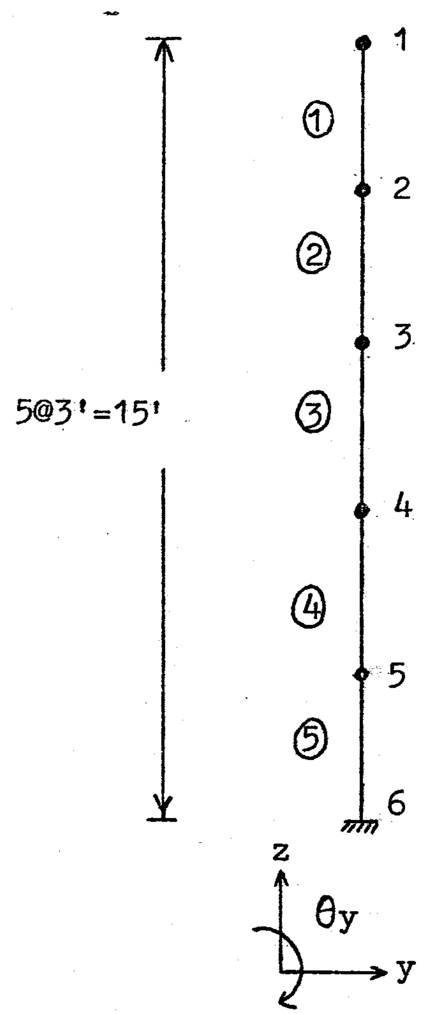
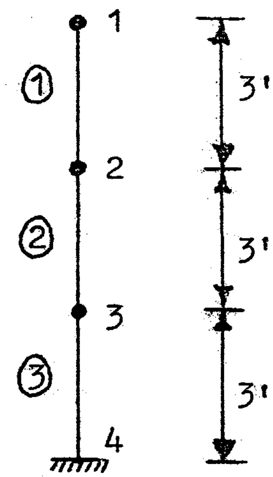
FIGURE C-11

COMPARISON OF MODAL COMPONENTS OF TOP-STORY DISTORTION FROM DYNAS AND REFERENCE 6



(a) Model 1



(b) Model 2

Members

A = 0.388 sq.ft.
I = 0.091 $ft.^4$

Damping ratio = 0.02

Masses

Along x = 20 k
Along θ_y = 100 k/ft^2

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FIGURE C-12

2-D CANTILEVER MODELS FOR DYNAS VALIDATION PROBLEM NO. 4

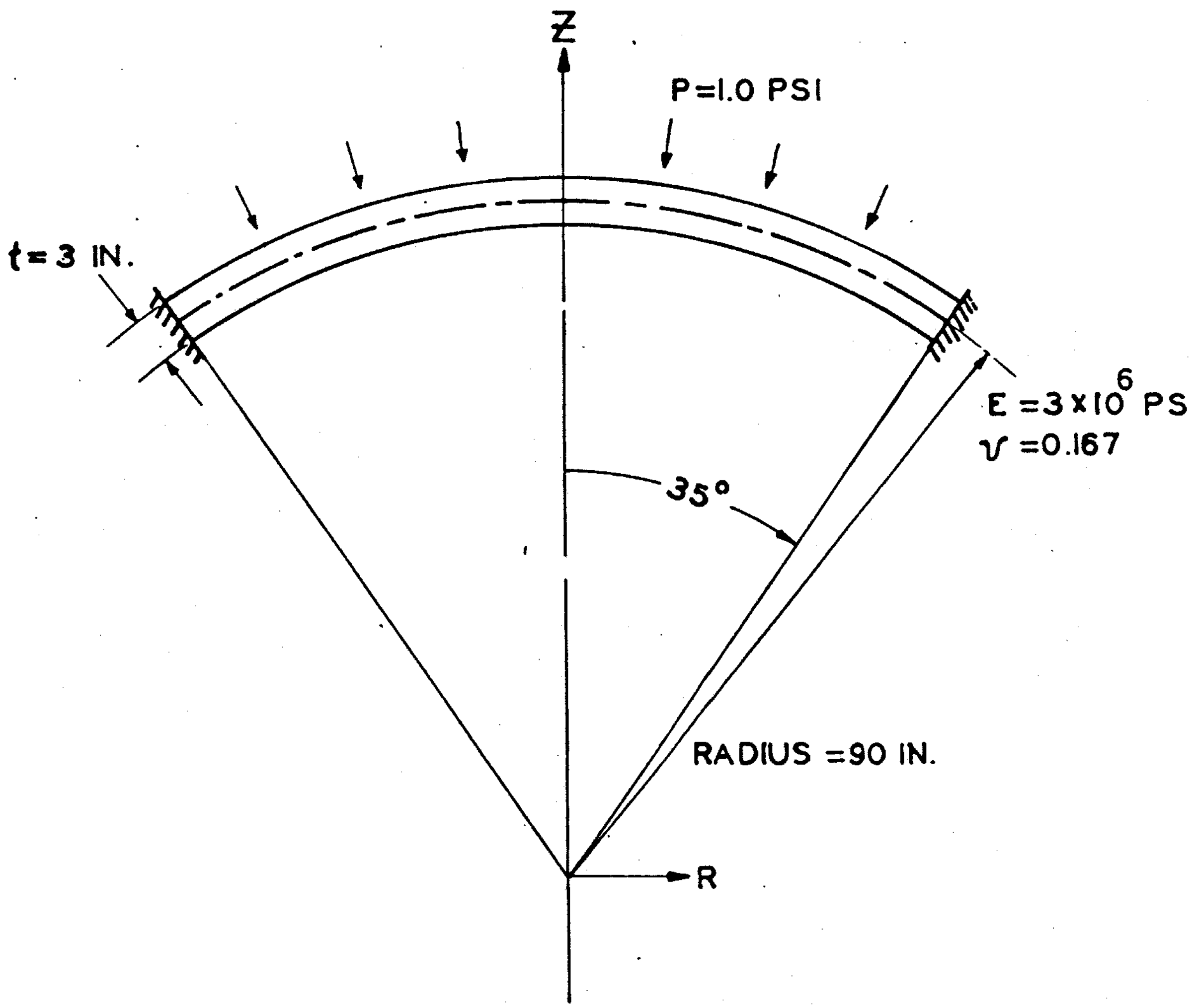

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FIGURE C-13

SHALLOW SPHERICAL SHELL ANALYZED BY DYNAX - VALIDATION PROBLEM 1

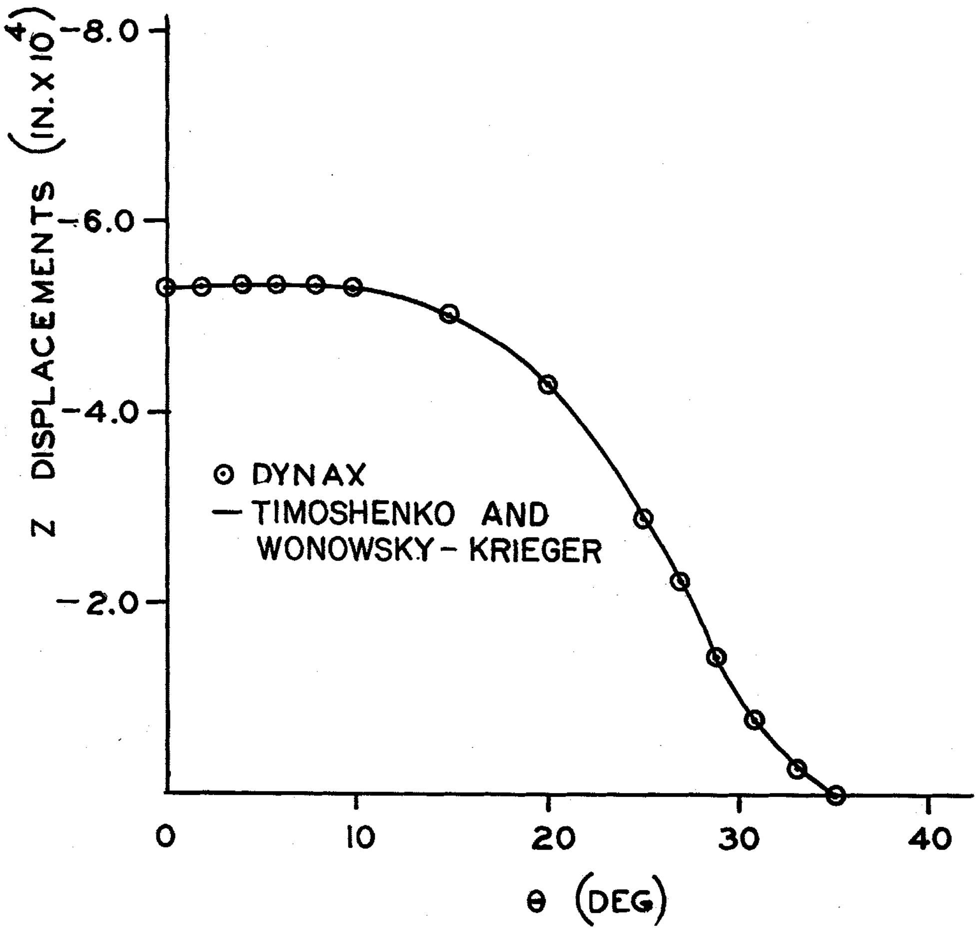

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FIGURE C-14

AXIAL DISPLACEMENT
SHALLOW SPHERICAL SHELL

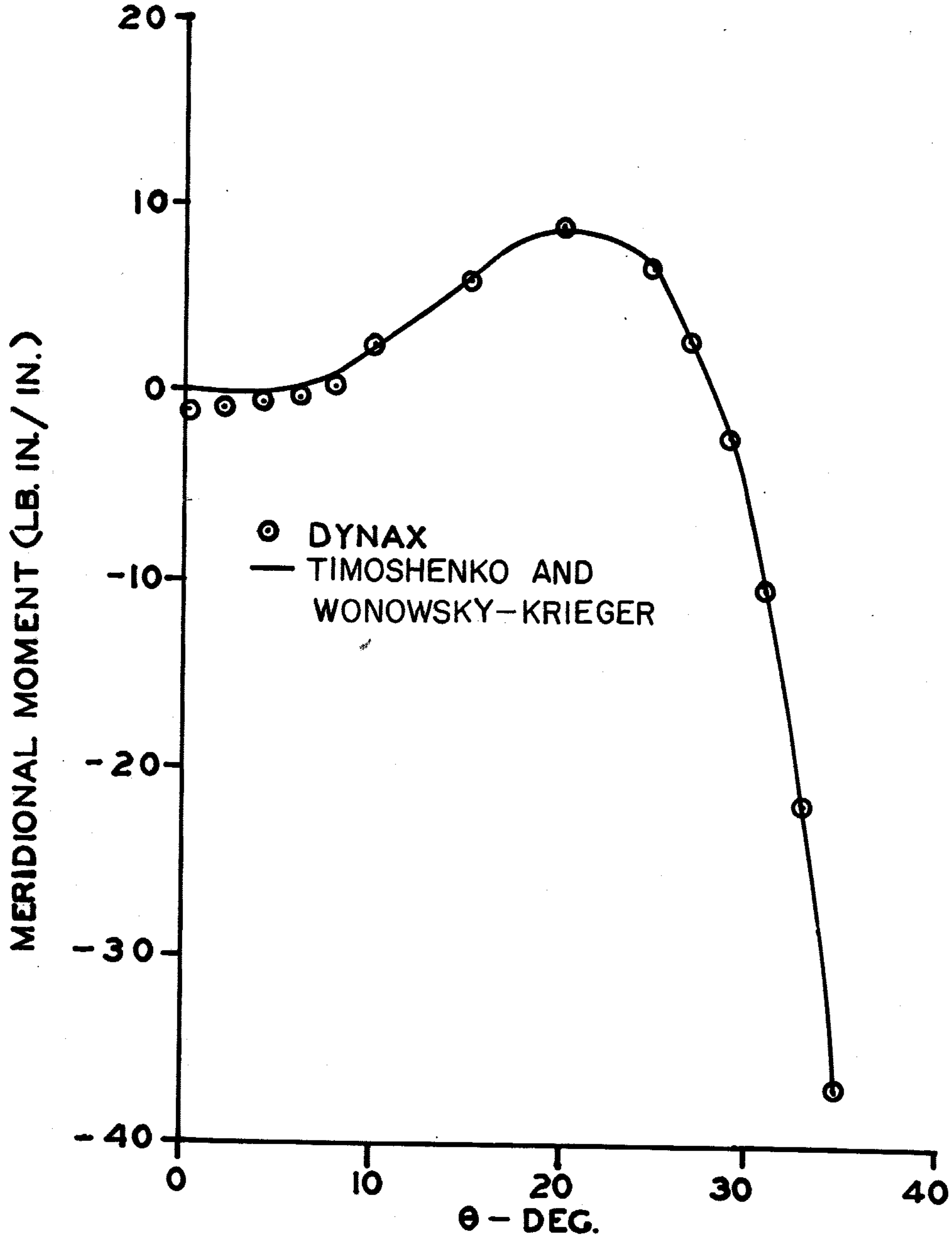

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FIGURE C-15

MERIDIONAL MOMENT SHALLOW SPHERICAL SHELL

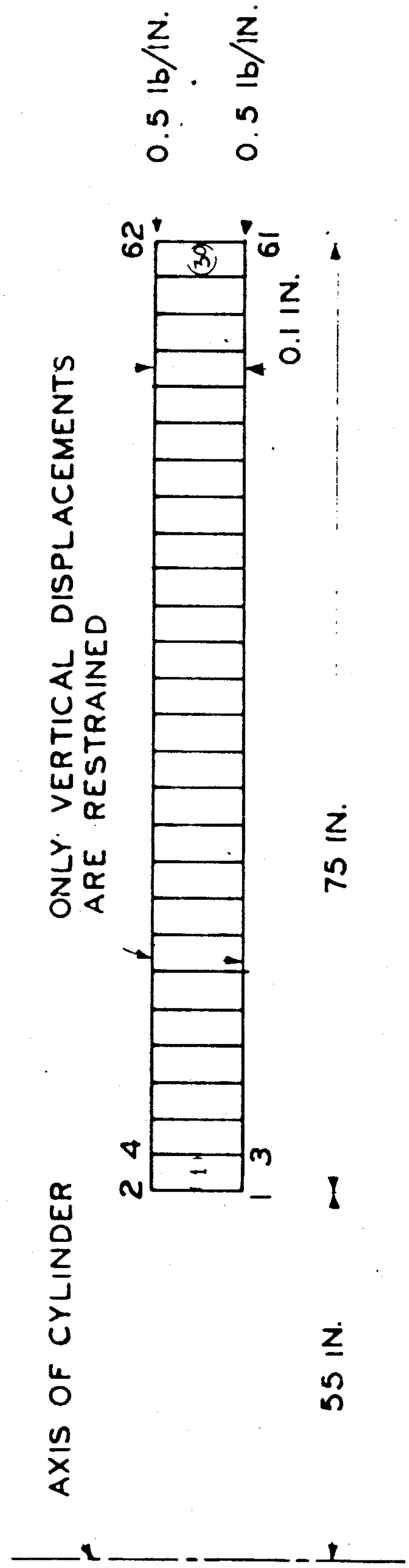

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FIGURE C-16

FINITE ELEMENT IDEALIZATION OF THICK-WALLED CYLINDER FOR DYNAX VALIDATION PROBLEM 2

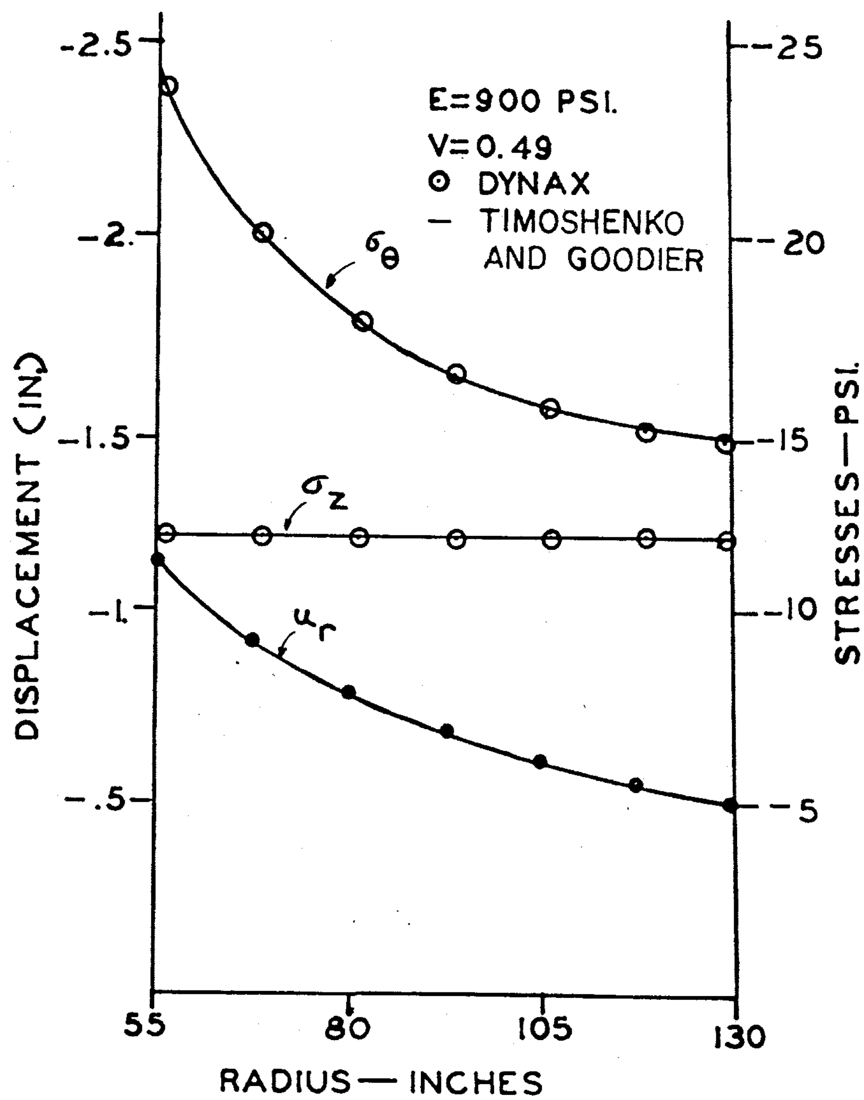

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FIGURE C-17

COMPARISON OF STRESSES AND DISPLACEMENTS THICK-WALLED CYLINDERS FROM DYNAX AND REFERENCE 10

T= SHELL THICKNESS = 1 IN.
M= 1 LB.-IN./IN.
E= 91. LB/IN^2
ν=.3
N= FOURIER HARMONIC NUMBER

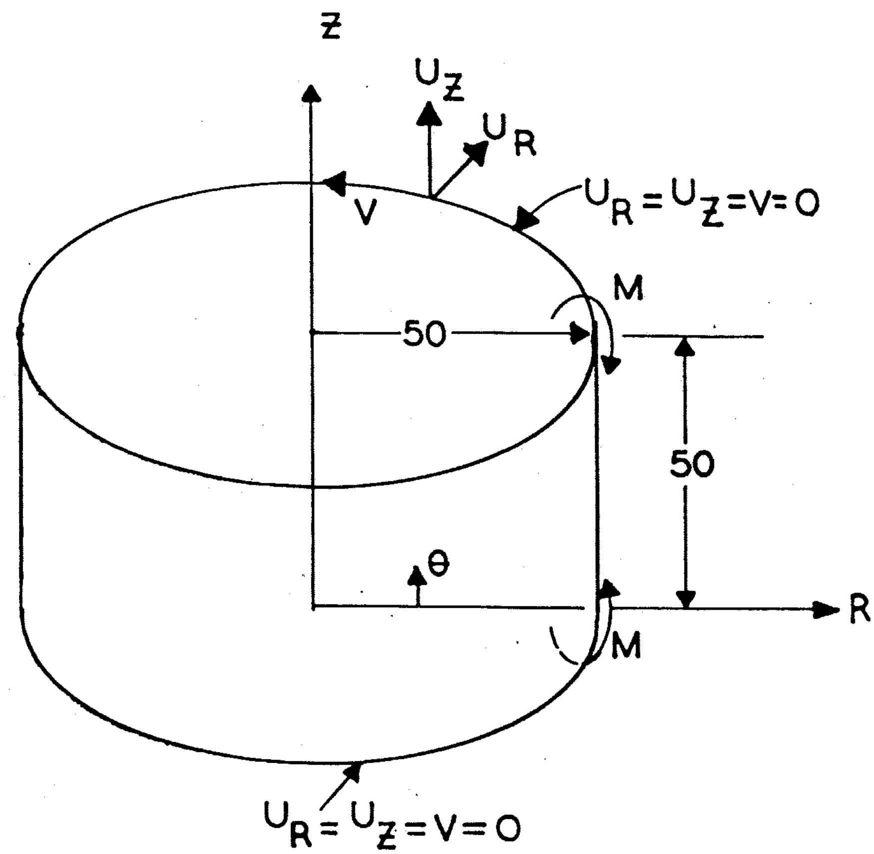

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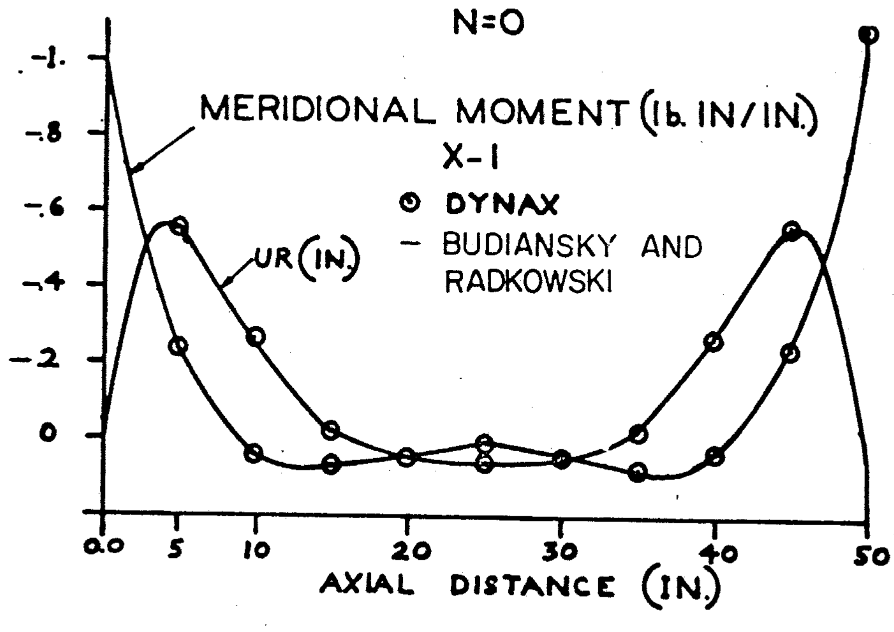
FIGURE C-18

CYLINDER UNDER HARMONIC LOADS ANALYZED BY DYNAX - VALIDATION PROBLEM 3



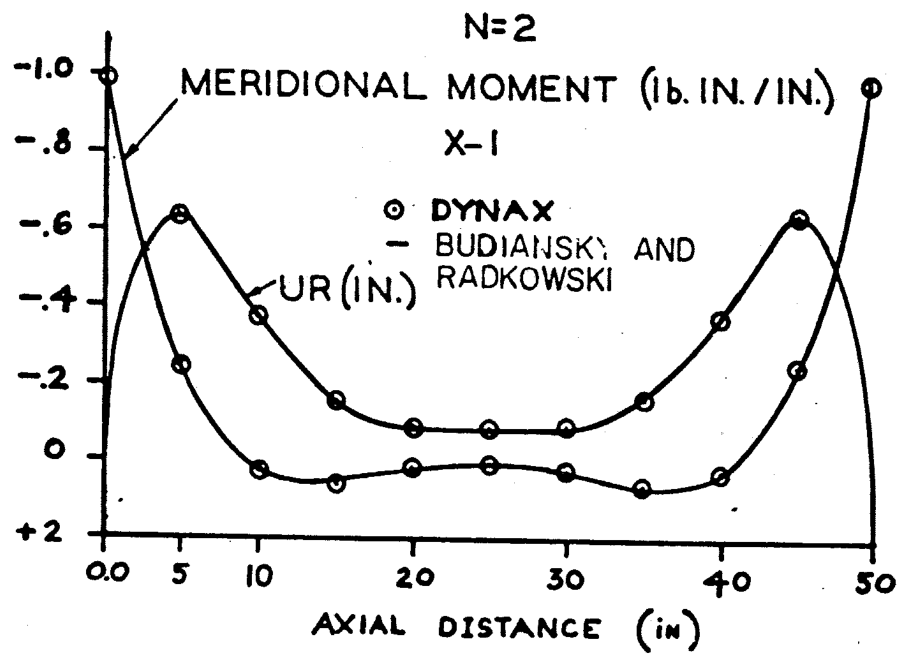


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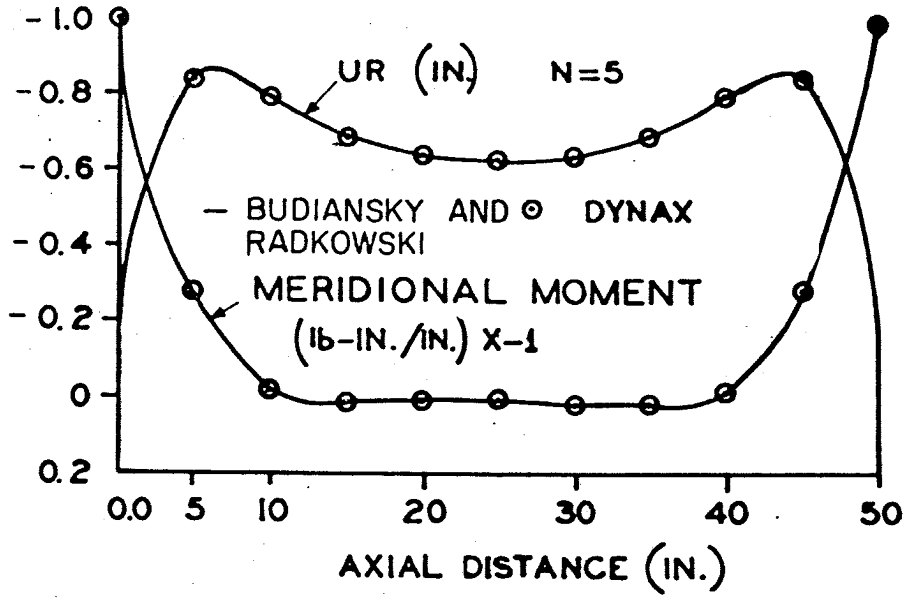
FIGURE C-19 COMPARISON OF RESULTS FROM DYNAX AND REFERENCE 11 OF MERIDIONAL MOMENTS AND DEFLECTIONS OF CYLINDER - (N=0, N=2)



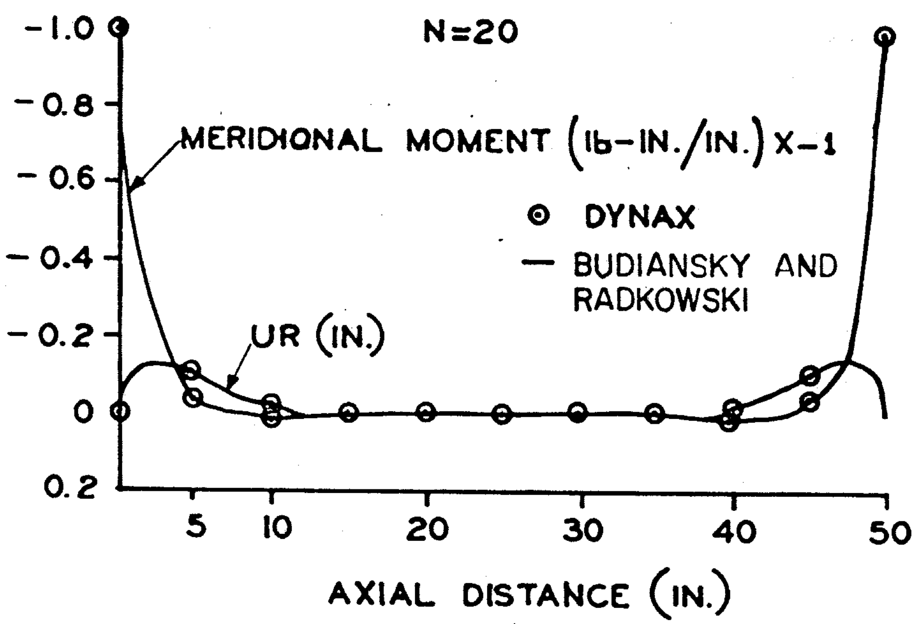


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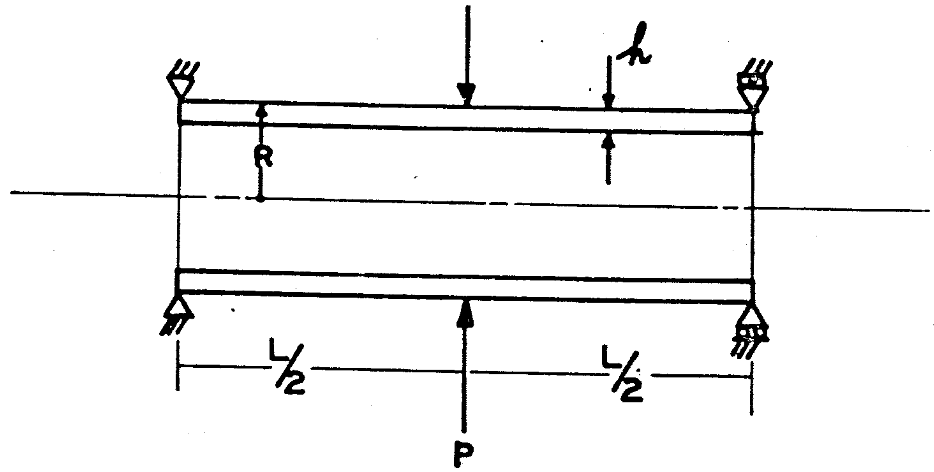
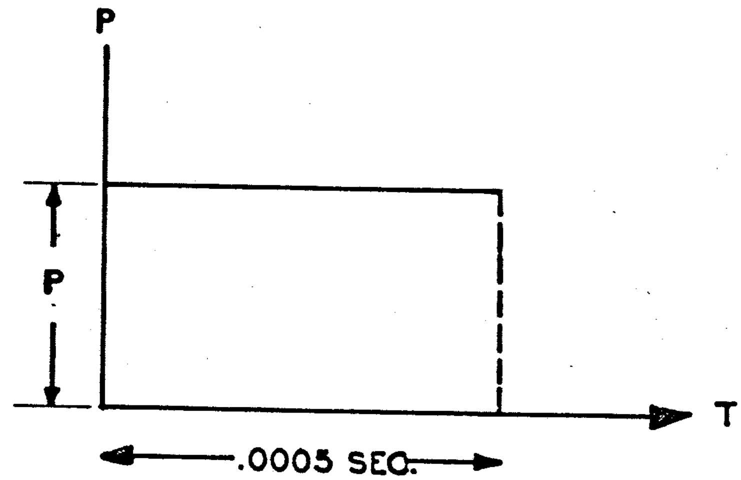
FIGURE C-20
COMPARISON OF RESULTS FROM DYNAX AND REFERENCE 11 OF MERIDIONAL MOMENTS AND DEFLECTIONS OF CYLINDER
(N=5, N=20)





TIME HISTORY LOADING

L = 18 IN.

P = 500 lb.

R = 3 IN.

h = 0.3 IN.

E = 30×10^6 lb./IN.^{2}

MASS DENSITY (ρ) = 0.0187 $\frac{\# \text{ SEC.}^2}{\text{IN.}^4}$

ν = 0.3

TIME STEP = .000005 SEC.

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FIGURE C-21

SUDDENLY APPLIED RING LINE LOAD ANALYZED BY DYNAX - VALIDATION PROBLEM 4

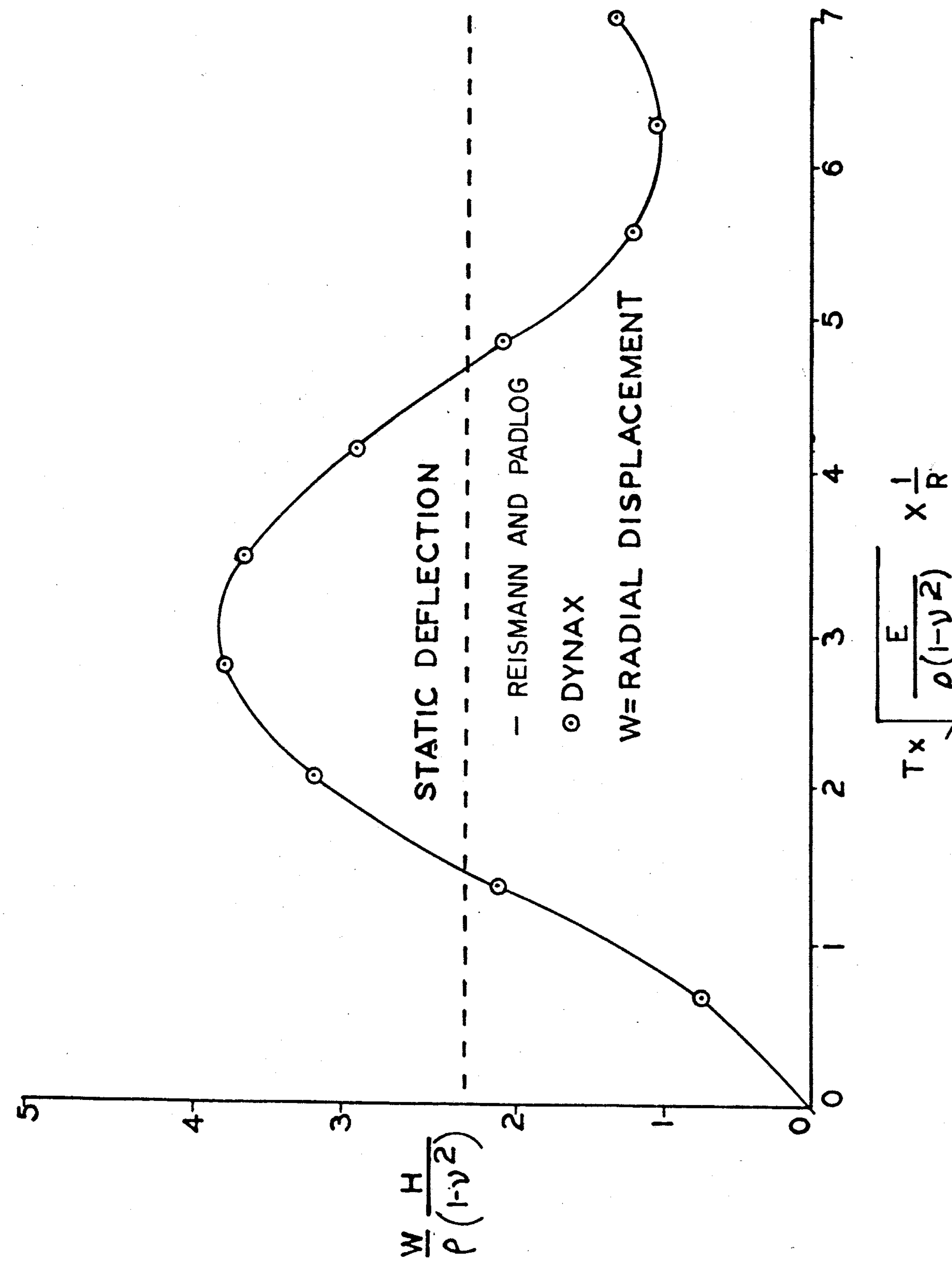

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FIGURE C-22

RADIAL DISPLACEMNET VS. TIME COMPARISON OF RESULTS FROM DYNAX AND REFERENCE 12

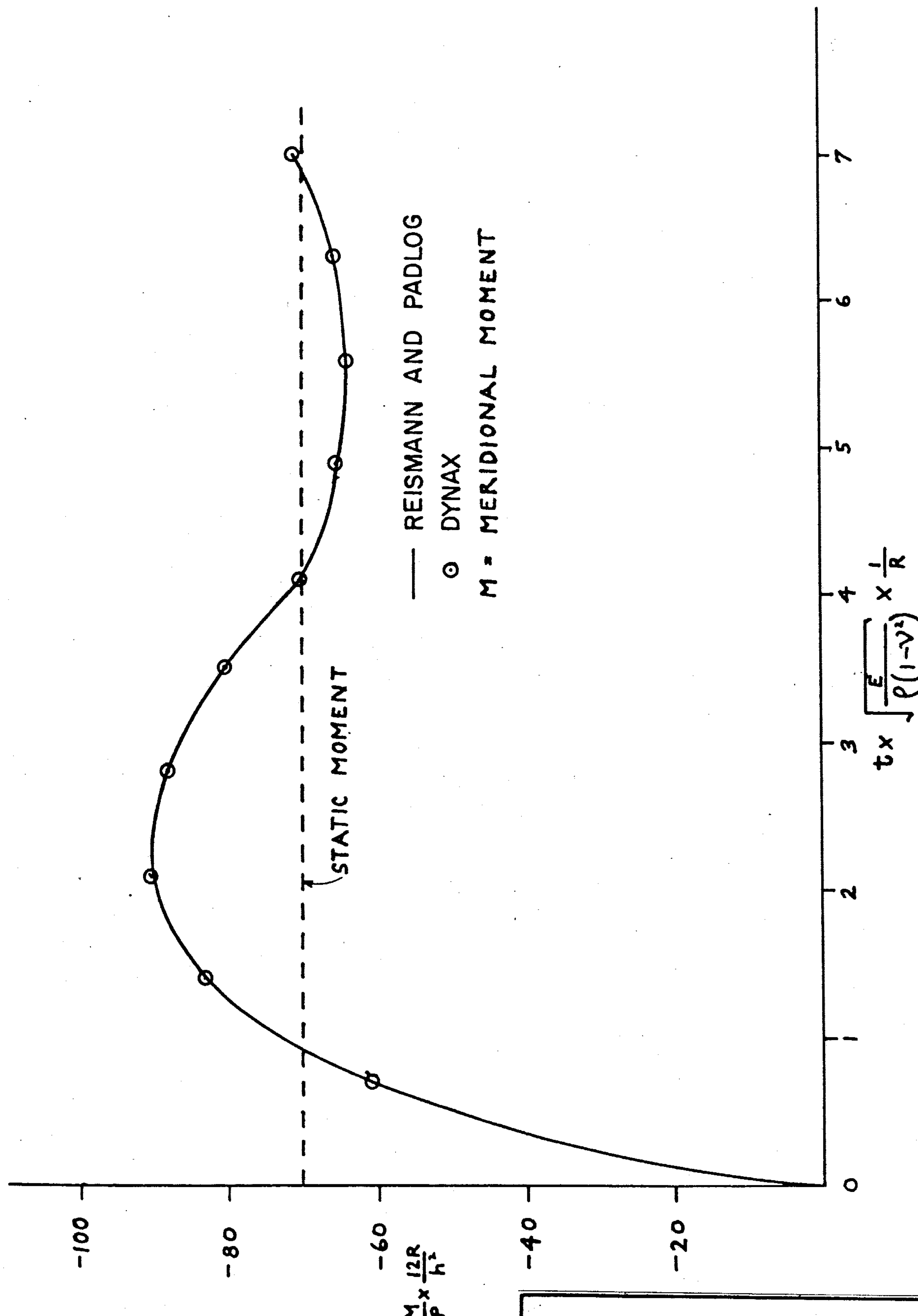

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FIGURE C-23

BENDING MOMENT VS. TIME - SUDDENLY APPLIED RING (LINE) LOAD - COMPARISON OF RESULTS FROM DYNAX AND REFERENCE 12

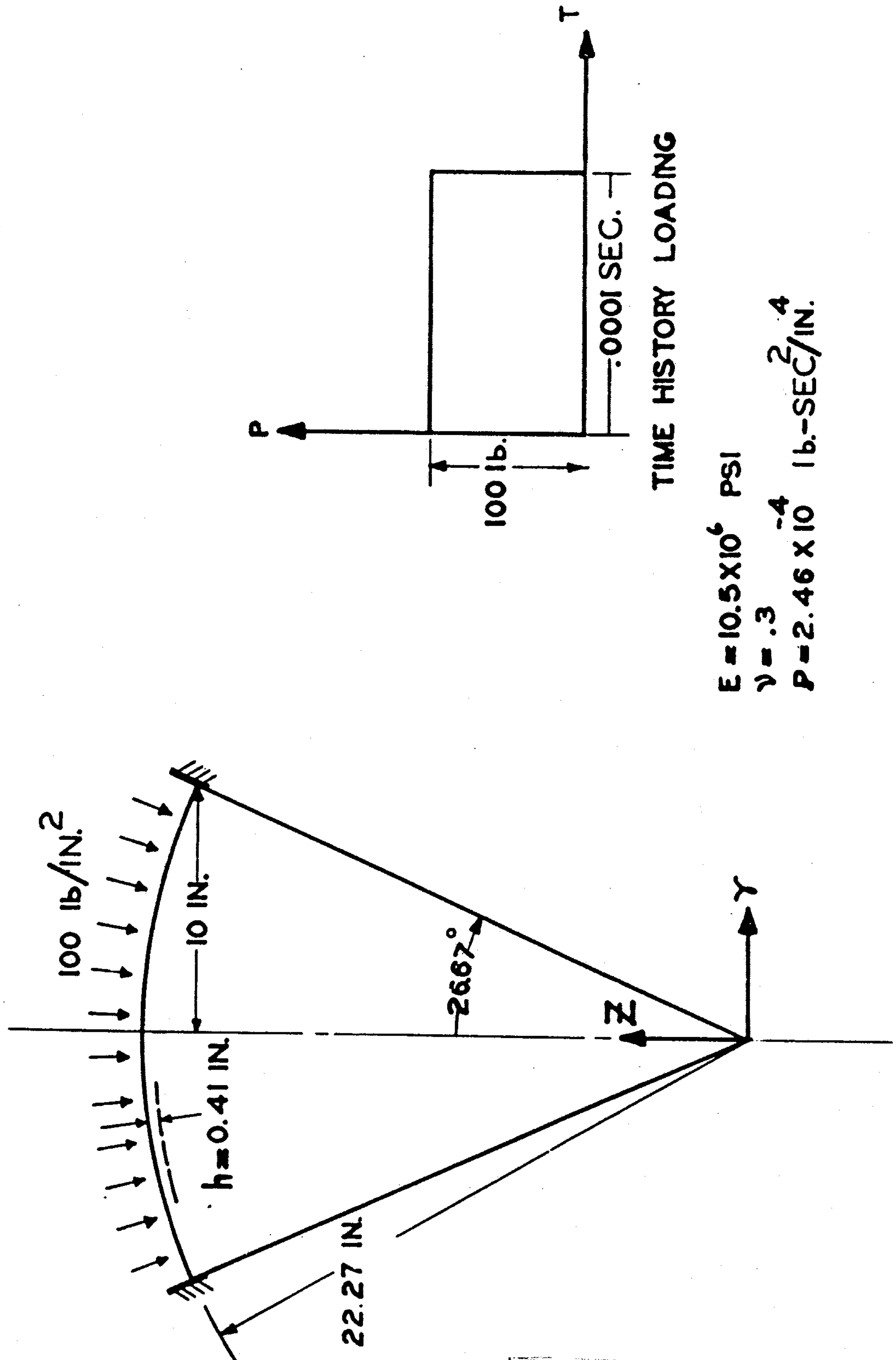

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FIGURE C-24

SPHERICAL CAP ANALYZED BY DYNAX - VALIDATION PROBLEM 5

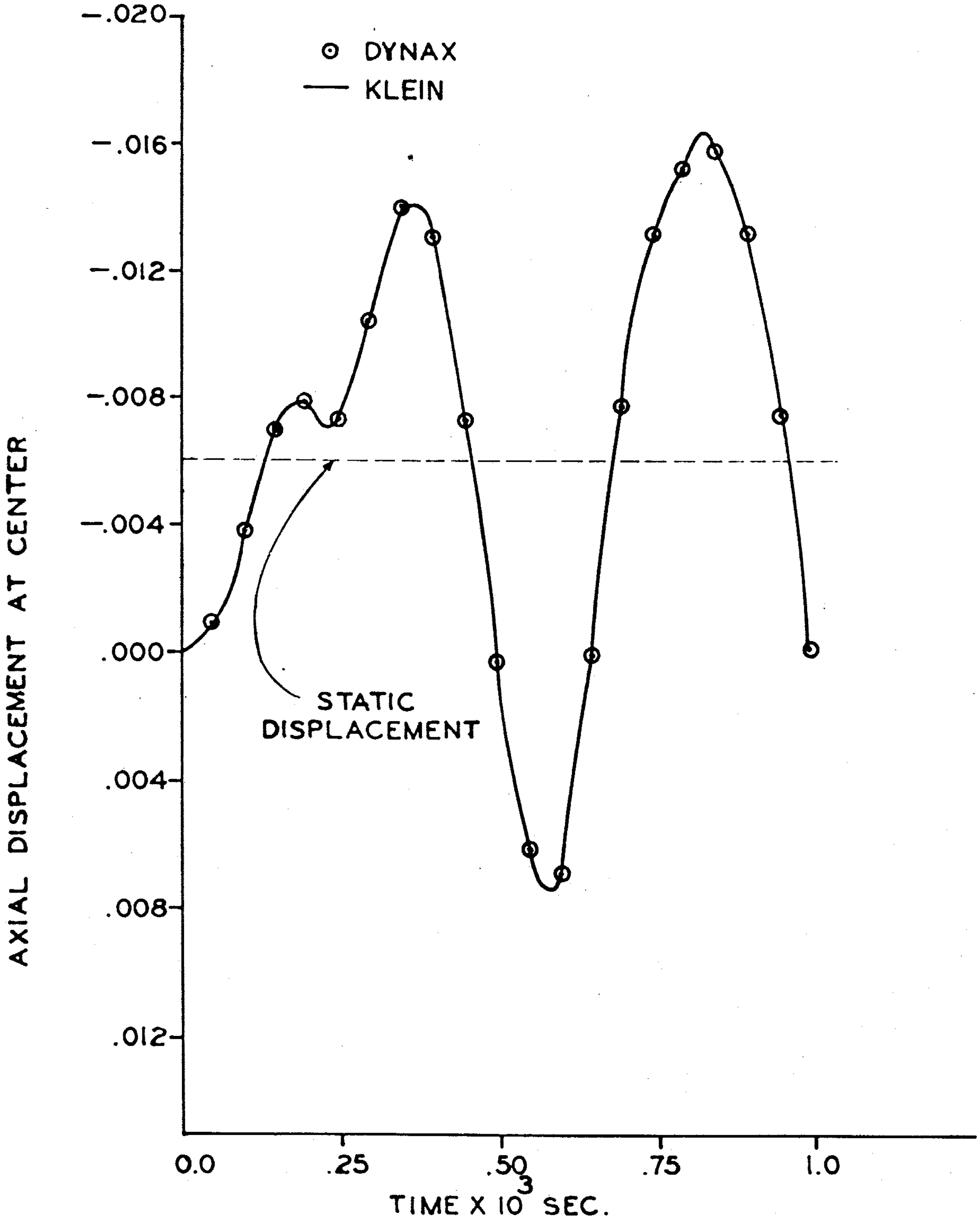

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FIGURE C-25

COMPARISON OF RESULTS FROM DYNAX AND REFERENCE 13 OF AXIAL DISPLACEMENT OF SPHERICAL CAP UNDER DYNAMIC LOAD

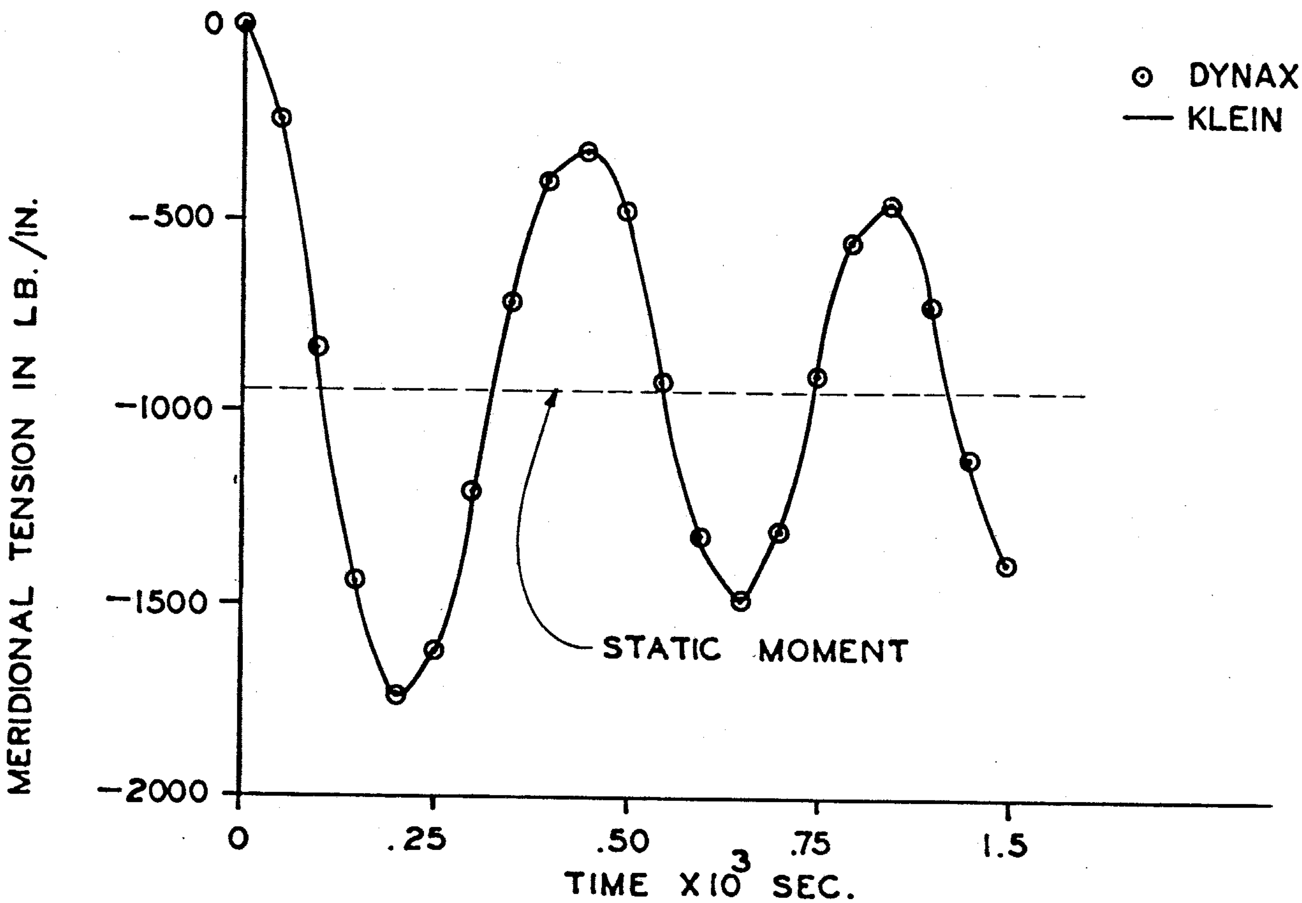

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FIGURE C-26

COMPARISON OF RESULTS FROM DYNAX AND REFERENCE 13 OF MERIDIONAL TENSION OF SPHERICAL CAP UNDER DYNAMIC LOAD

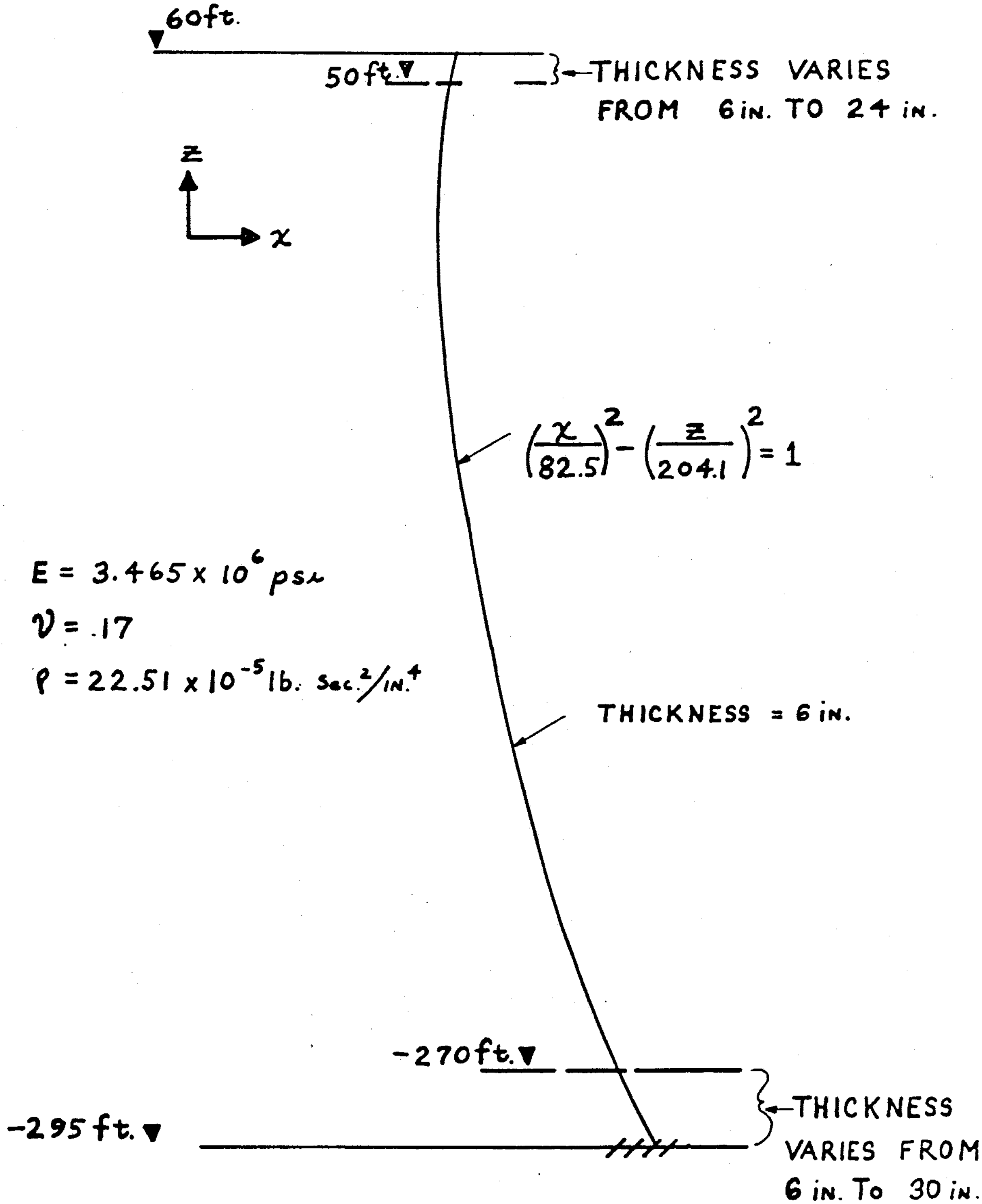

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FIGURE C-27

HYPERBOLIC COOLING TOWER ANALYZED BY DYNAX - VALIDATION PROBLEM 6

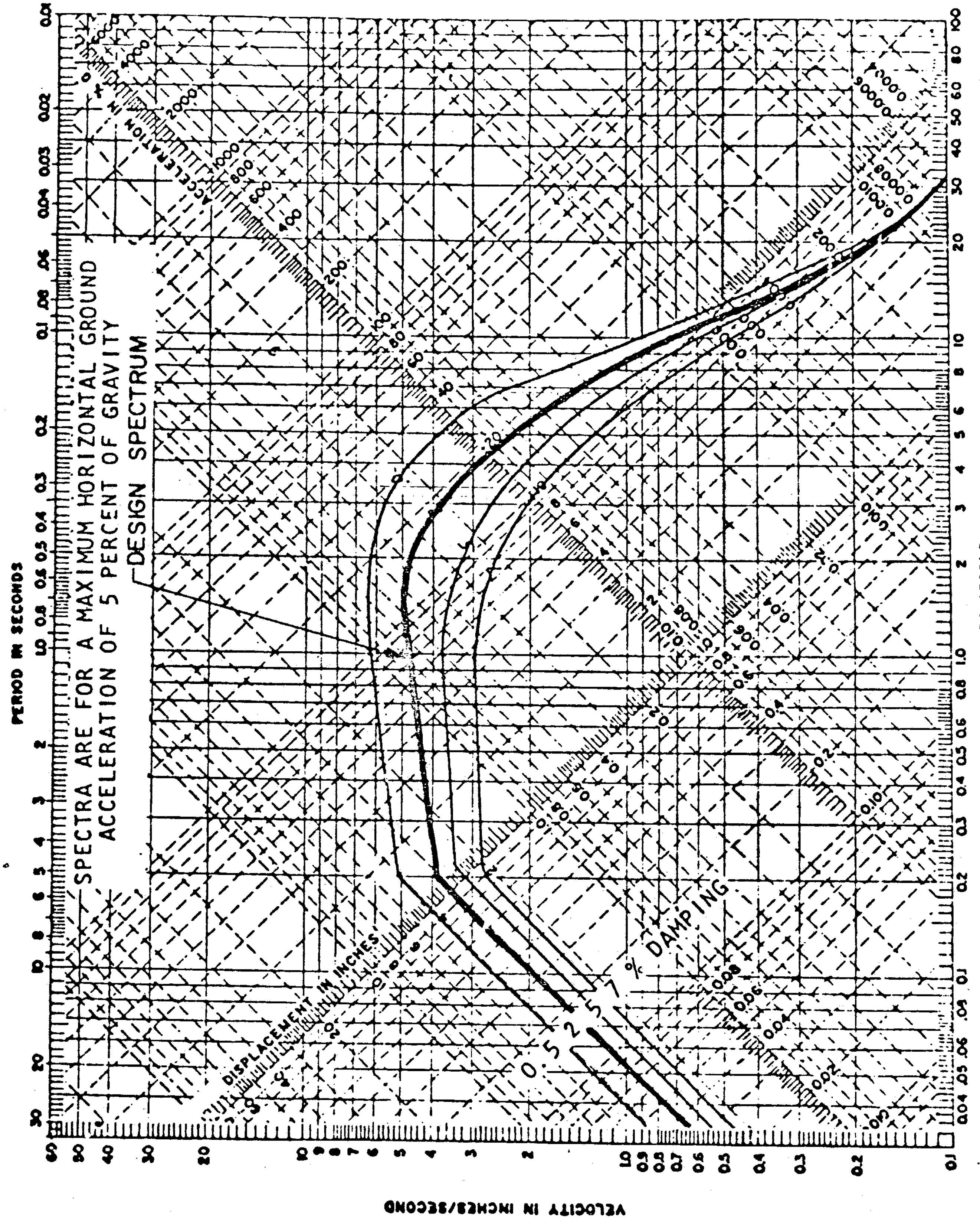

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FIGURE C-28

SPECTRUM OF DESIGN EARTHQUAKE USED FOR DYNAX - VALIDATION PROBLEM 6

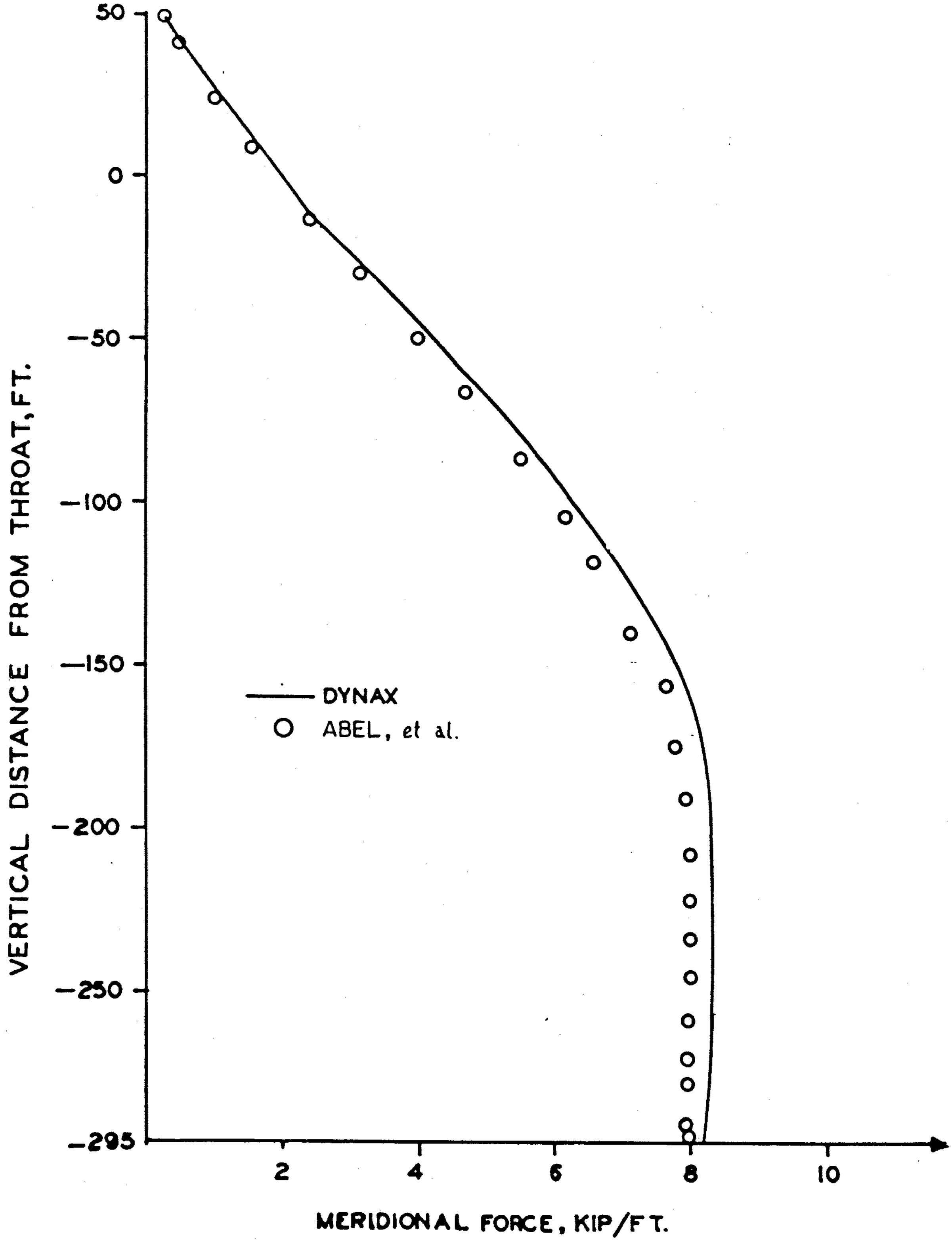

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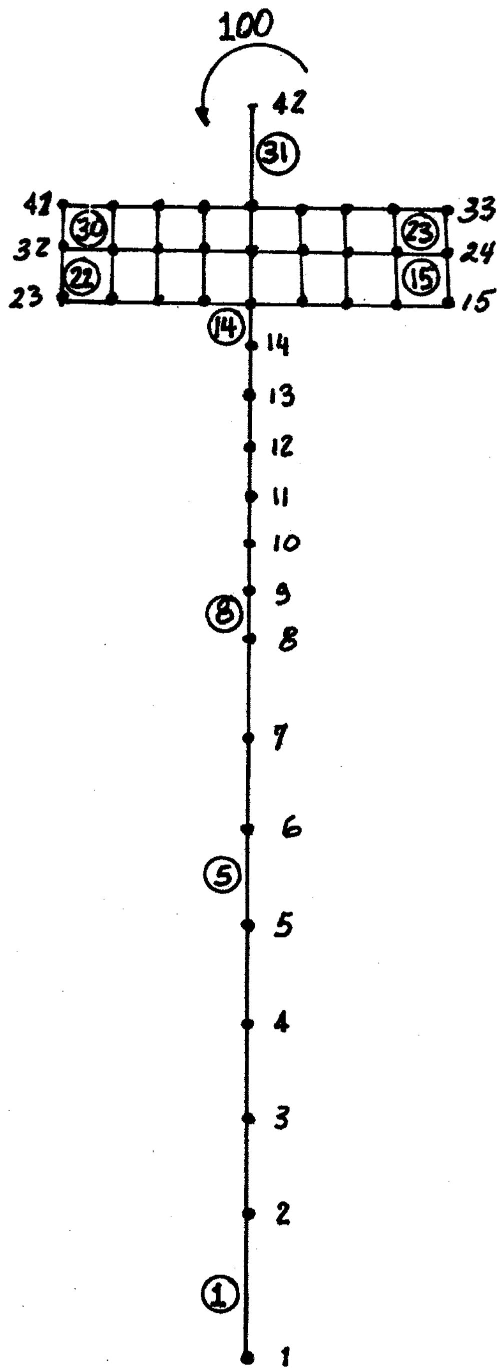
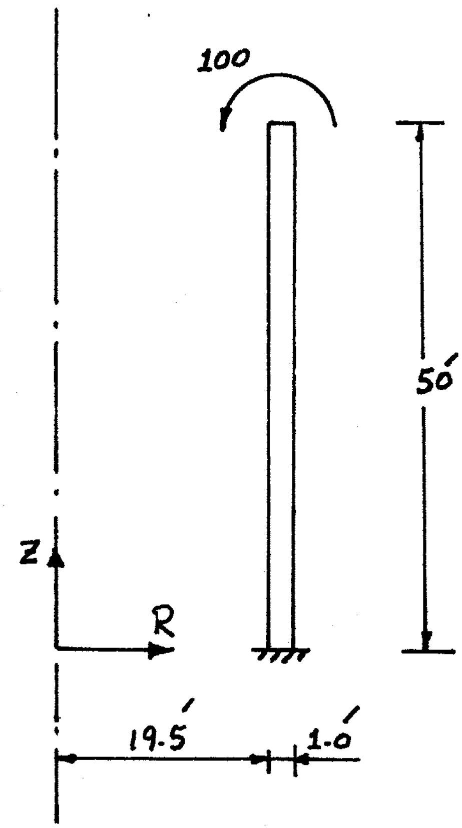
FIGURE C-29

COMPARISON OF COOLING TOWER MERIDIONAL FORCES OBTAINED BY DYNAX AND REFERENCE 14



Finite Element Model



Structure

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FIGURE C-30

TYING OF SOLID AND SHELL ELEMENTS
ANALYZED BY DYNAX - VALIDATION PROBLEM 7

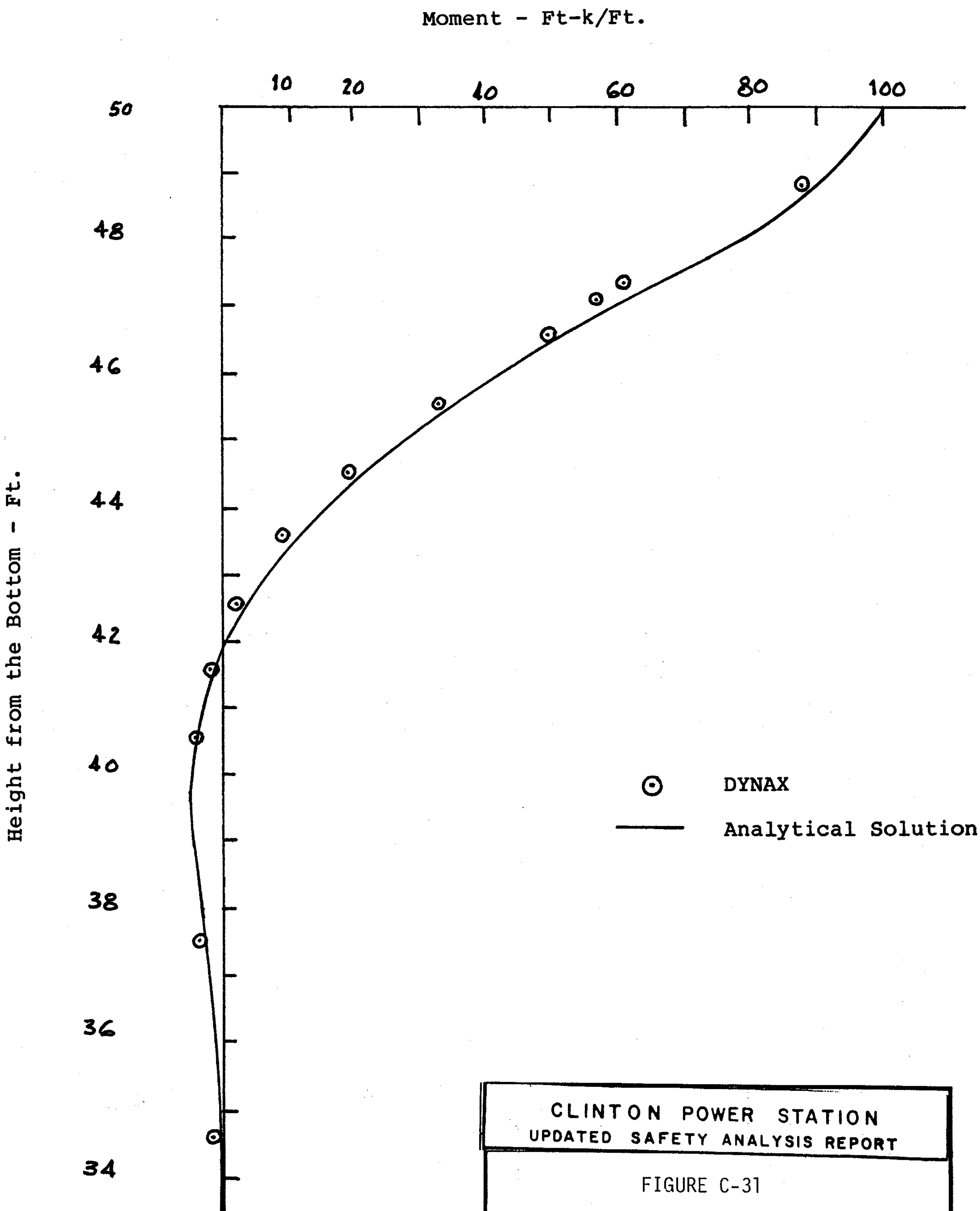

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FIGURE C-31

MOMENT DIAGRAM OF RESULTS FROM DYNAX AND ANALYTICAL SOLUTION

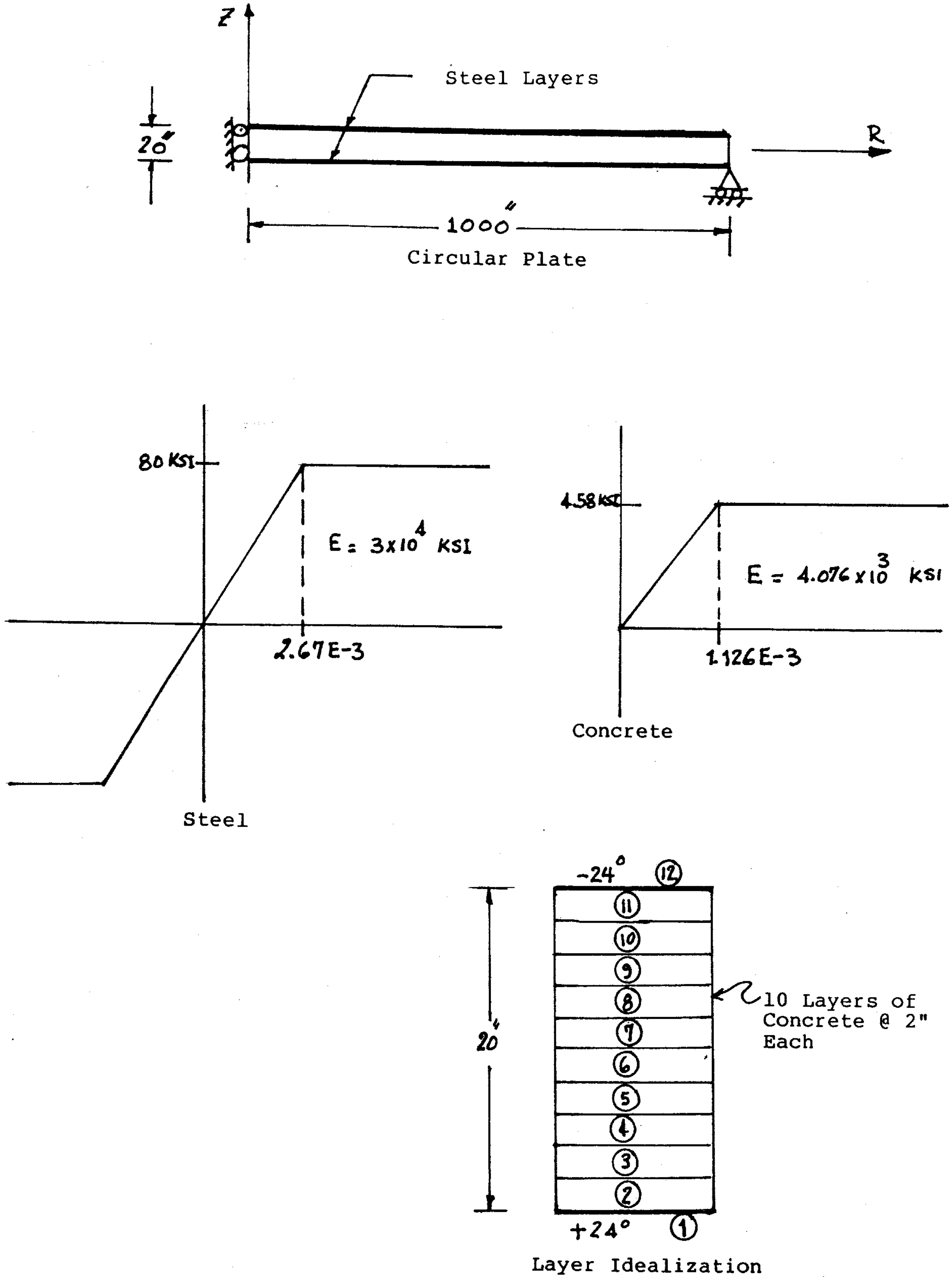

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FIGURE C-32

CIRCULAR PLATE ANALYZED BY DYNAX -
VALIDATION PROBLEM 8

R = 10 ft.
L = 5 ft.
t = 0.25 ft.

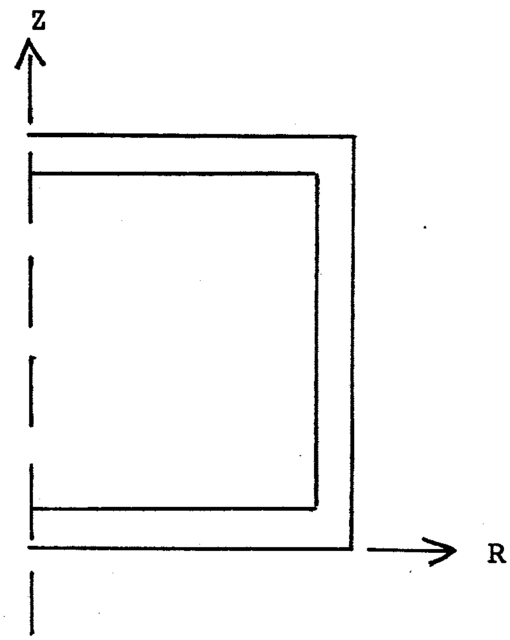

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FIGURE C-33

CYLINDER UNDER CONSTANT PRESSURE ANALYZED BY DYNAX AND SOR-III (DYNAX)

$E = 100$

$\rho = 1$

$\nu = 0$

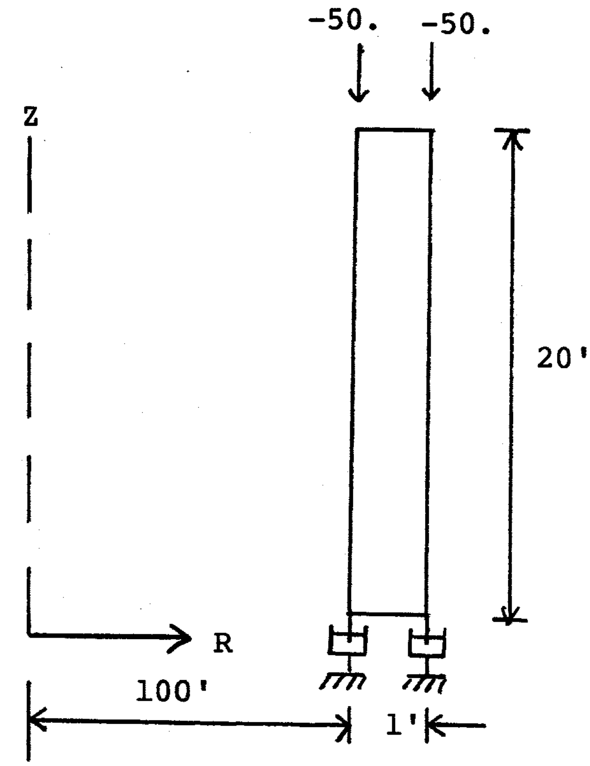

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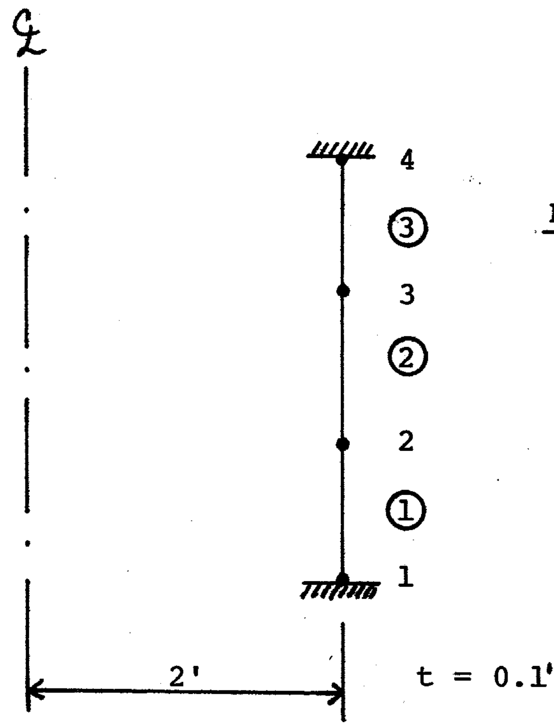
FIGURE C-34

CYLINDER UNDER DYNAMIC AXIAL PRESSURE FOR NON-REFLECTING BOUNDARIES ANALYZED BY DYNAX - VALIDATION PROBLEM 10



Element	Material Damping Coefficient β
1	0.04
2	0.05
3	0.03

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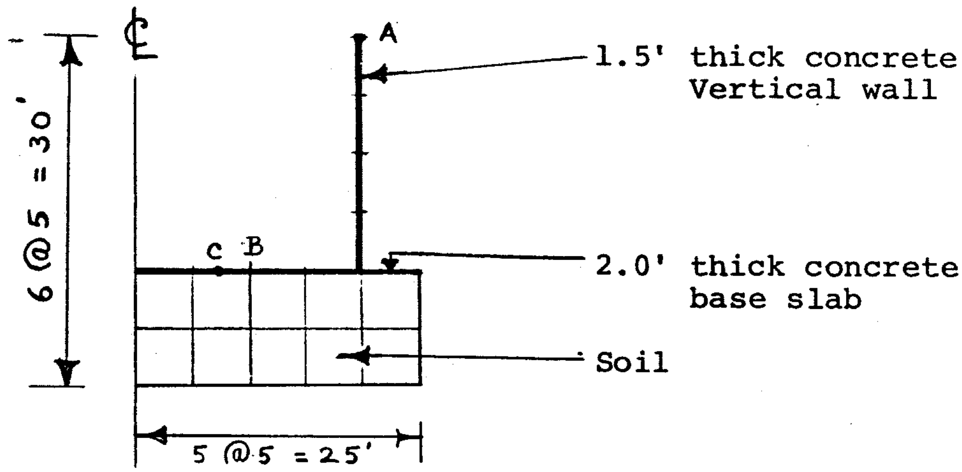
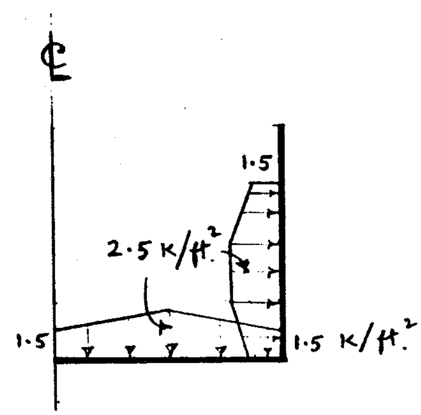
FIGURE C-35

FINITE ELEMENT MODEL AND MATERIAL DAMPING COEFFICIENTS FOR CYLINDER ANALYZED BY DYNAX - VALIDATION PROBLEM 11



(a) Model



(b) Meridional Load Distribution

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FIGURE C-36

MODELING AND LOAD DISTRIBUTION

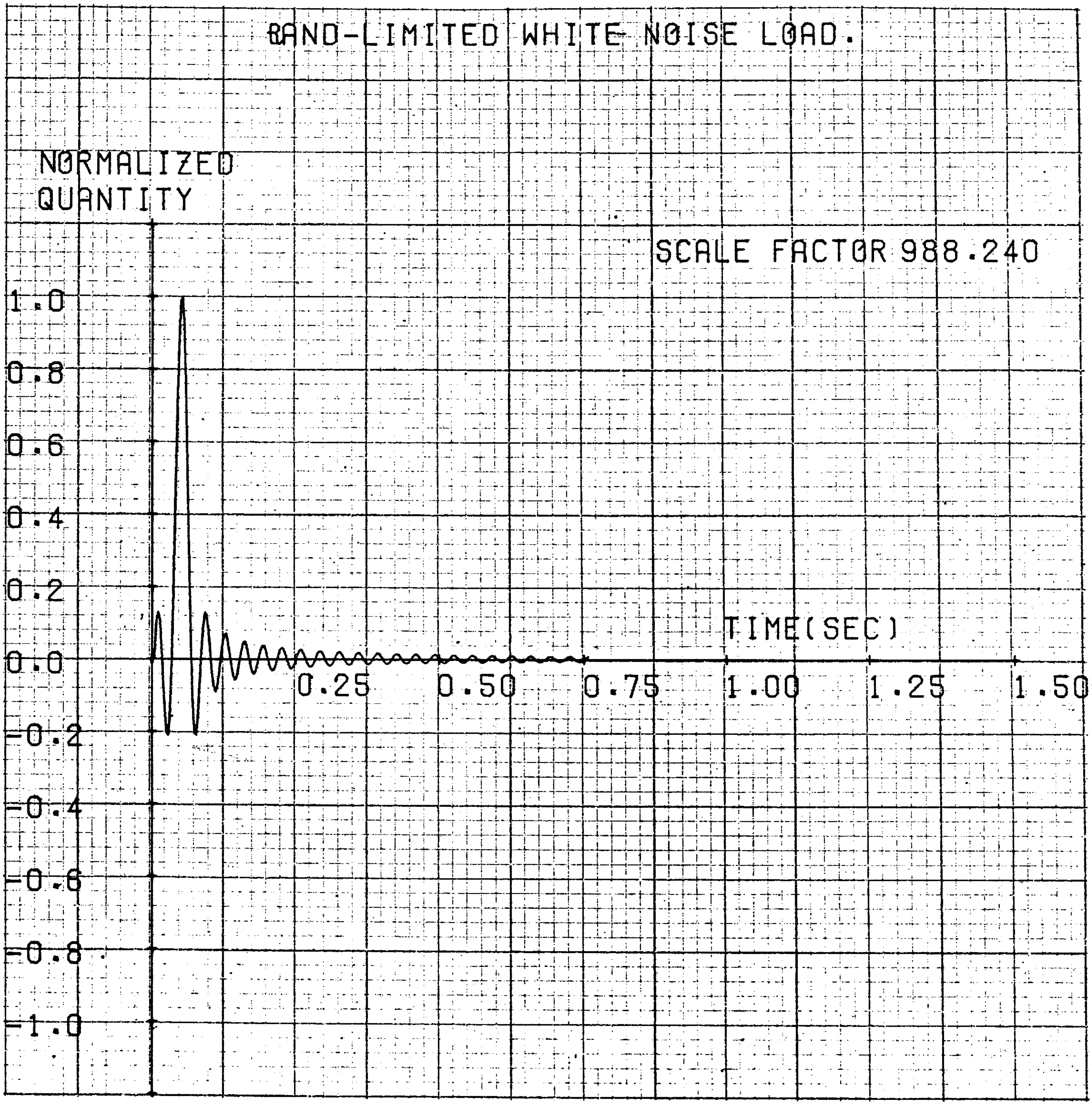

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FIGURE C-37

TIME HISTORY OF LOAD ω (t)

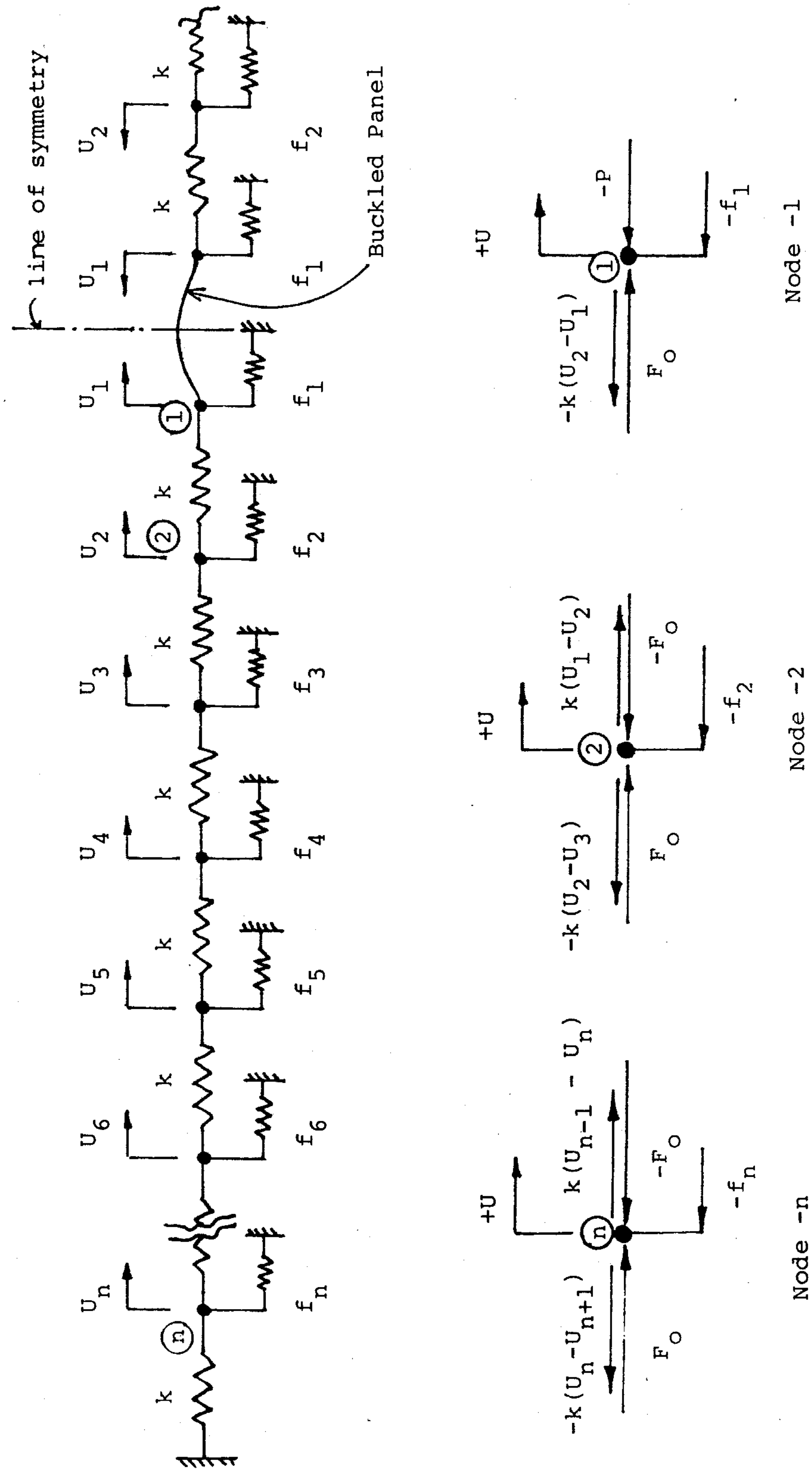

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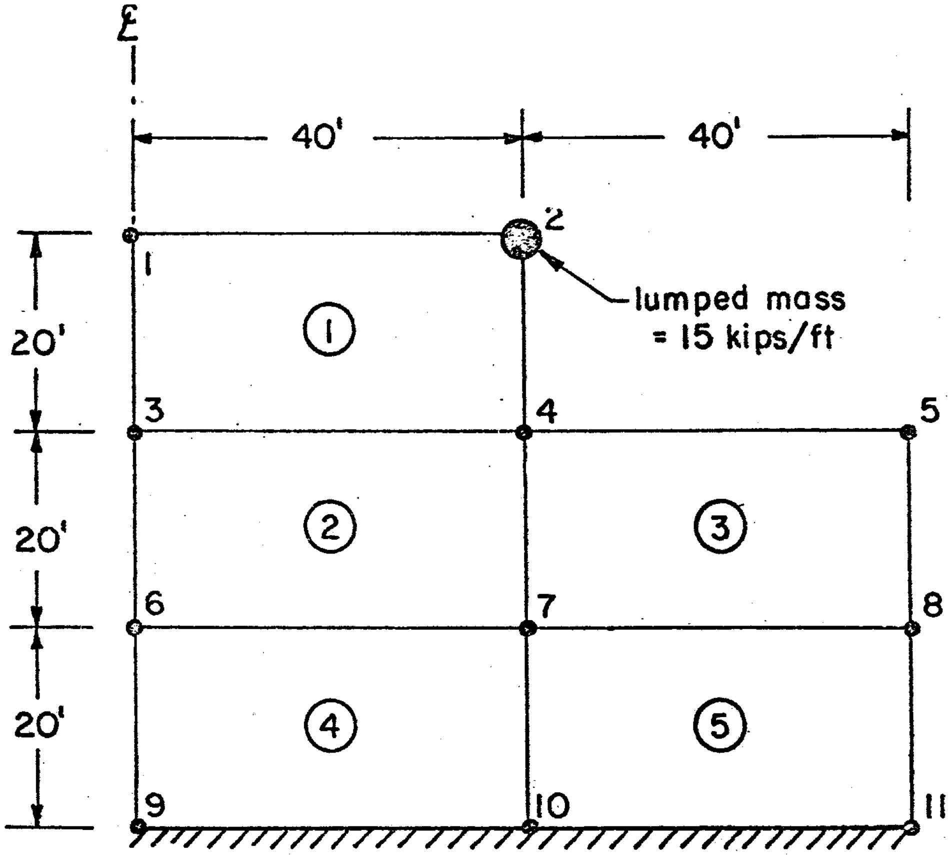
FIGURE C-38

IDEALIZED MODEL OF ANCHOR-PANEL SYSTEM
USED IN LAFD VALIDATION PROBLEM



Nodal points 4 and 5 linked by rigid element

Element No.	Material	Unit Wt. pcf	Poissons Ratio	Shear Modulus at 10^{-4}% Strain ksf
1	Steel	150	0.30	6000
2	Sand	115	0.45	3000
3	Sand	115	0.45	2700
4	Clay	120	0.45	4000
5	Clay	120	0.45	3700

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FIGURE C-39

LUSH VALIDATION PROBLEM

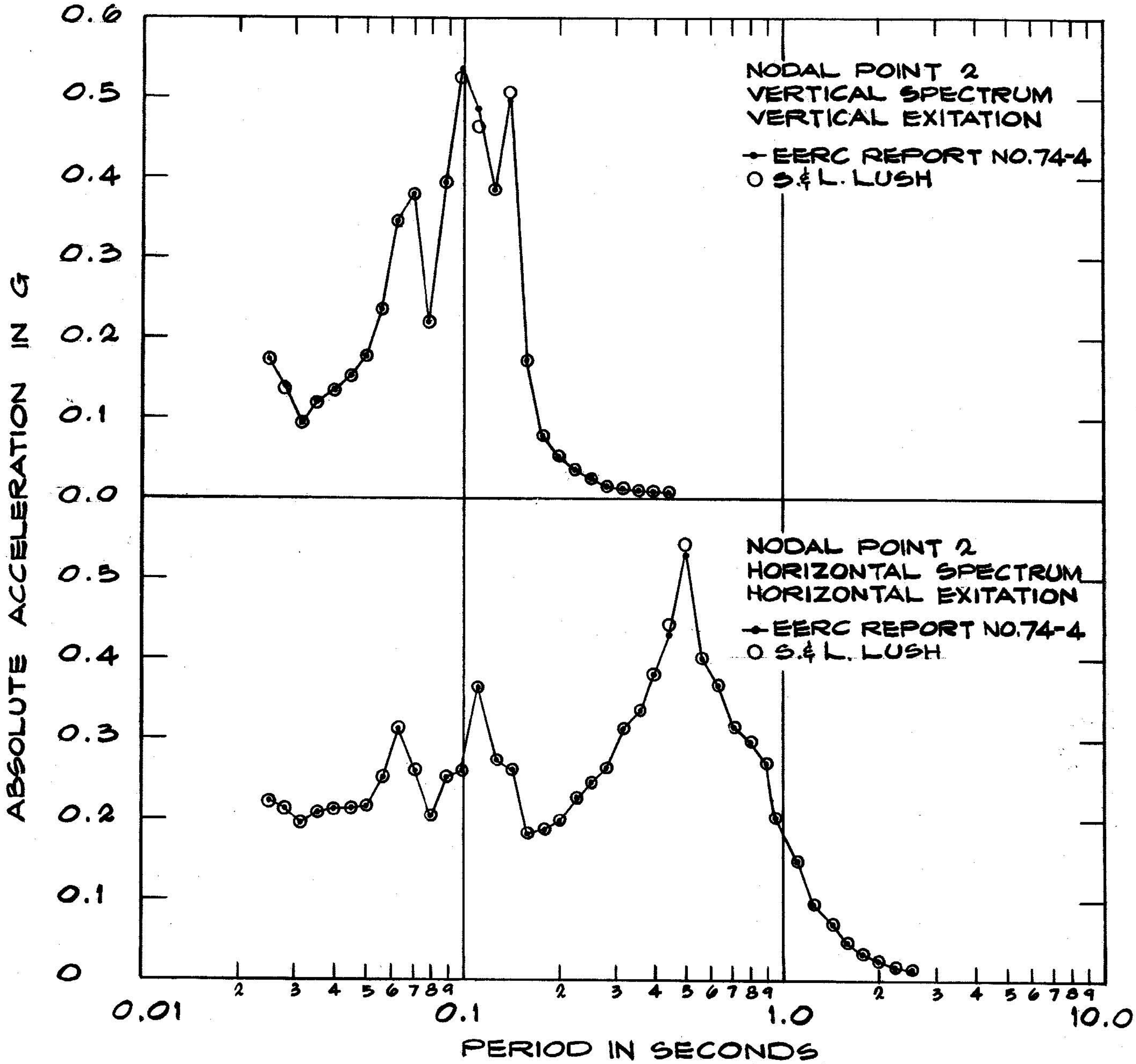

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FIGURE C-40

COMPARISON OF RESPONSE SPECTRUM
(EERC REPORT NO. 74-4 AND
S&L LUSH)

P.C.A. - U.S.D. OF R.C. COLUMNS

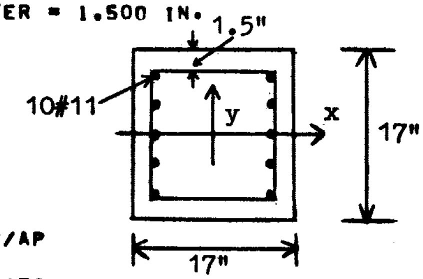
VALIDATION PROBLEM NO.1 - DESIGN OF A TIED COLUMN -COMPRESSION CONTROL

DESIGN OF
TIED COLUMN

B= 17.00 T= 17.00 FC= 3.000 FY=40.000 PHIC= .700 PHIB= .900

USE- 10 NO. 9 BARS. AST = 10.00 SQ.IN. = 3.47 PCT. COVER = 1.500 IN.

	ROW 1	ROW 2	ROW 3	ROW 4
NO. OF BARS	2	2	3	3
COVER	1.500	1.500	1.500	1.500



LOAD CASE	APPLIED FORCES AP	AMX	AMY	ULTIMATE CAPACITY UP	UMX	UMY	UP/AP
1	525.	0.	105.	563.	0.	113.	1.072
2	525.	75.	0.	603.	86.	0.	1.148

INTERACTION CONTROL POINTS REQUESTED

	PZ	PB	MB	MZ
X -AXIS	778.0	304.7	166.2	176.2
Y -AXIS	778.0	245.8	234.6	199.7
Z -AXIS	778.0	314.6	167.2	193.7

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FIGURE C-41

DESIGN OF TIED COLUMN -
COMPRESSION CONTROLS

P.C.A. - U.S.D. OF R.C. COLUMNS

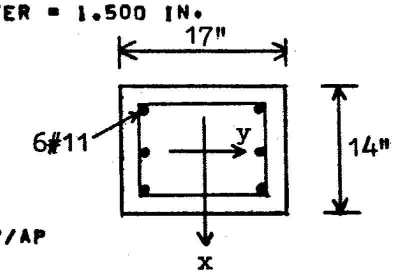
VALIDATION PROBLEM NO. 2 - DESIGN OF A TIED COLUMN - TENSION CONTROLS

DESIGN OF
TIED COLUMN

B= 14.00 T= 20.00 FC= 4.500 FY=50.000 PHIC= .700 PHIB= .900

USE= 6 NO.11 BARS. AST = 9.36 SQ.IN. = 3.35 PCT. COVER = 1.500 IN.

	ROW 1	ROW 2	ROW 3	ROW 4
NO. OF BARS	3	3	0	0
COVER	1.500	1.500	1.500	1.500



LOAD CASE	APPLIED FORCES AP	AMX	AMY	ULTIMATE CAPACITY UP	UMX	UMY	UP/AP
1	115.	279.	0.	122.	295.	0.	1.057
2	115.	0.	14.	801.	0.	94.	6.966

INTERACTION CONTROL POINTS REQUESTED

	PZ	PB	MB	MZ
X -AXIS	1052.2	317.9	353.8	282.8
Y -AXIS	1052.2	315.4	187.2	180.3
Z -AXIS	1052.2	310.9	231.3	254.0

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FIGURE C-42

DESIGN OF TIED COLUMN -
TENSION CONTROLS

P.C.A. - U.S.D. OF R.C. COLUMNS

VALIDATION PROBLEM NO.3 - DESIGN OF A TIED COLUMN-BIAXIAL BENDING

DESIGN OF
TIED COLUMN

B= 28.00 T= 28.00 FC= 5.000 FY=60.000 PHIC= .700 PHIB= .900

USE- 12 NO.11 BARS. AST = 18.72 SQ.IN. = 2.39 PCT. COVER = 1.500 IN.

	ROW 1	ROW 2	ROW 3	ROW 4
NO. OF BARS	4	4	2	2
COVER	1.500	1.500	1.500	1.500

LOAD	APPLIED FORCES			ULTIMATE CAPACITY			
CASE	AP	AMX	AMY	UP	UMX	UMY	UP/AP
1	1330.	790.	0.	1626.	966.	0.	1.223
2	1330.	0.	394.	2216.	0.	655.	1.666
3	1330.	790.	394.	1388.	824.	411.	1.044

INTERACTION CONTROL POINTS REQUESTED

	PZ	PB	MB	MZ
X -AXIS	3062.9	983.0	1167.4	999.1
Y -AXIS	3062.9	983.0	1167.4	999.1
Z -AXIS	3062.9	910.2	949.7	947.4

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FIGURE C-43

DESIGN OF TIED COLUMN -
BIAXIAL BENDING

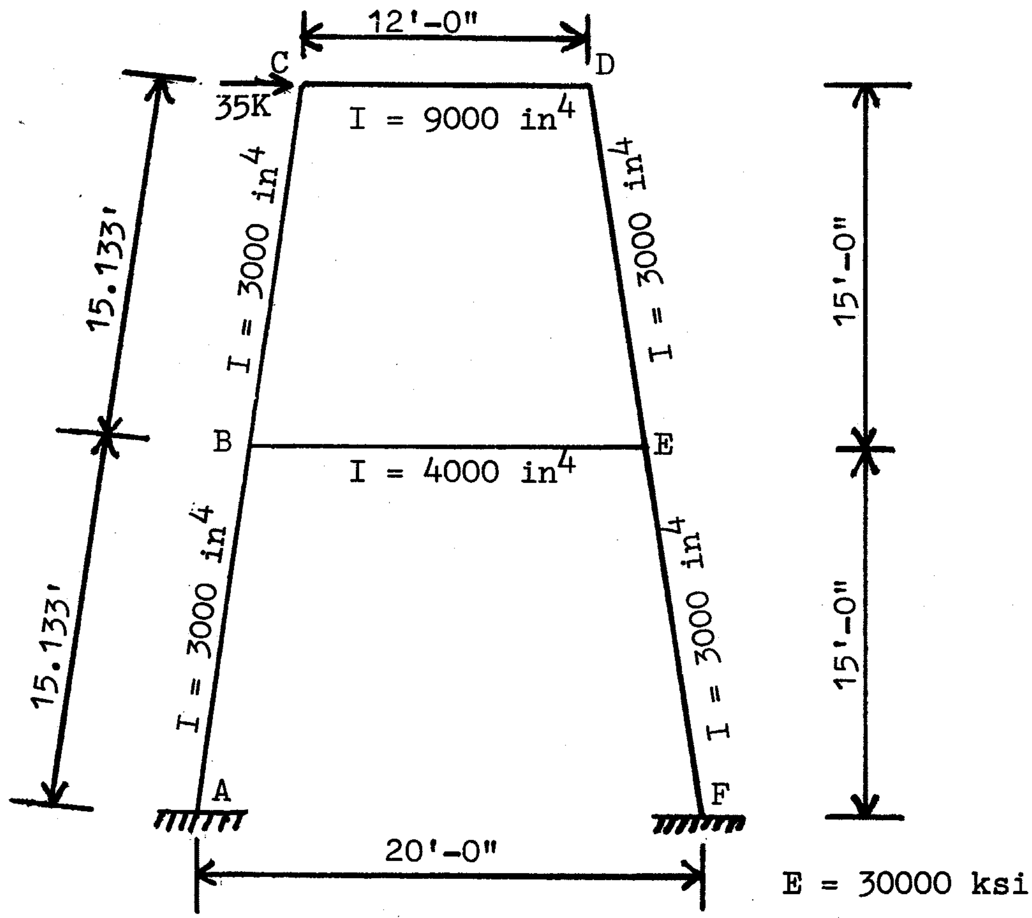

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FIGURE C-44

EXAMPLE FRAME FOR PIPSYS STATIC ANALYSIS

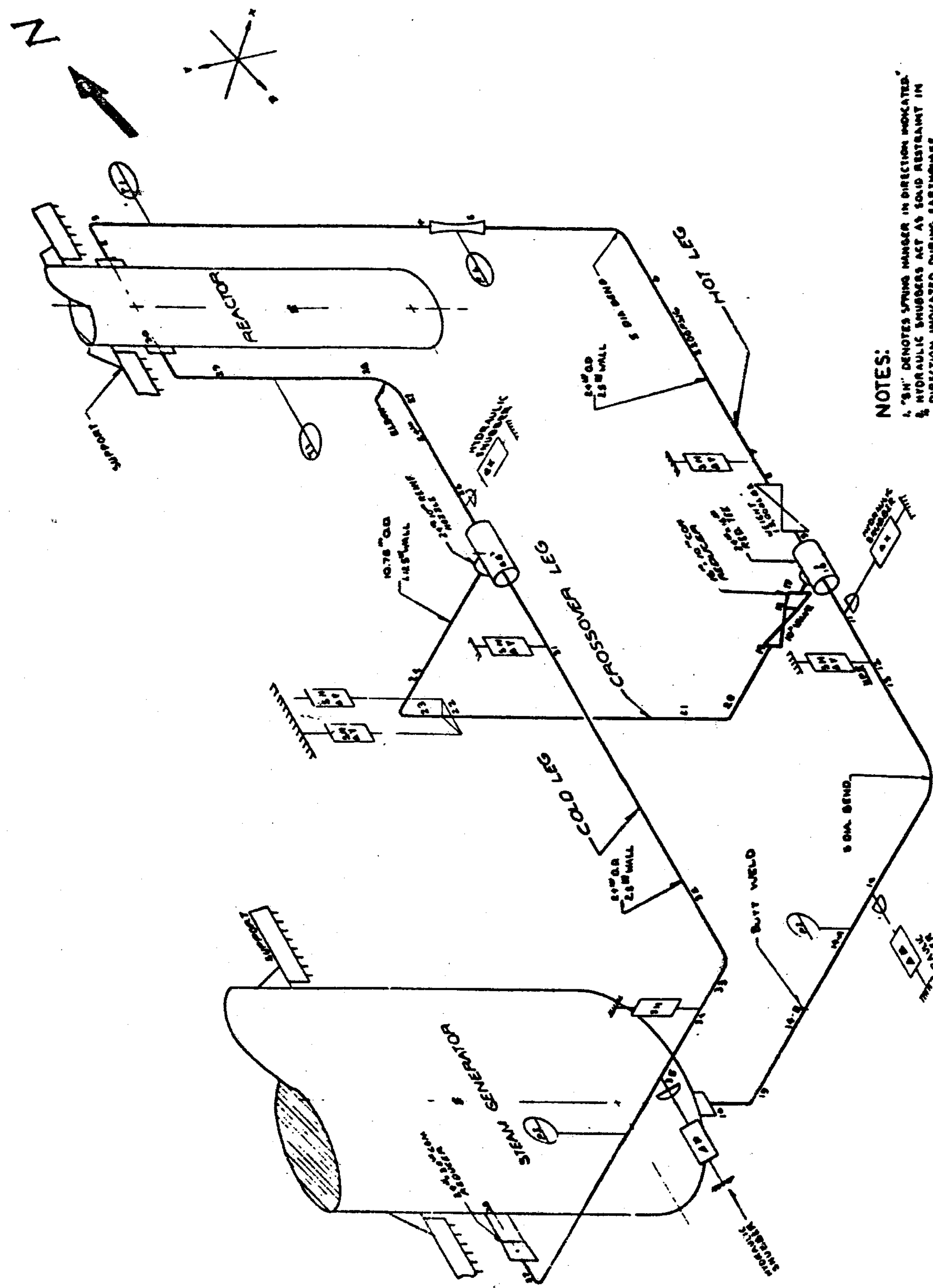

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FIGURE C-45

PIPING SYSTEM FOR COMBINED STRESS ANALYSIS (PIPSYS)

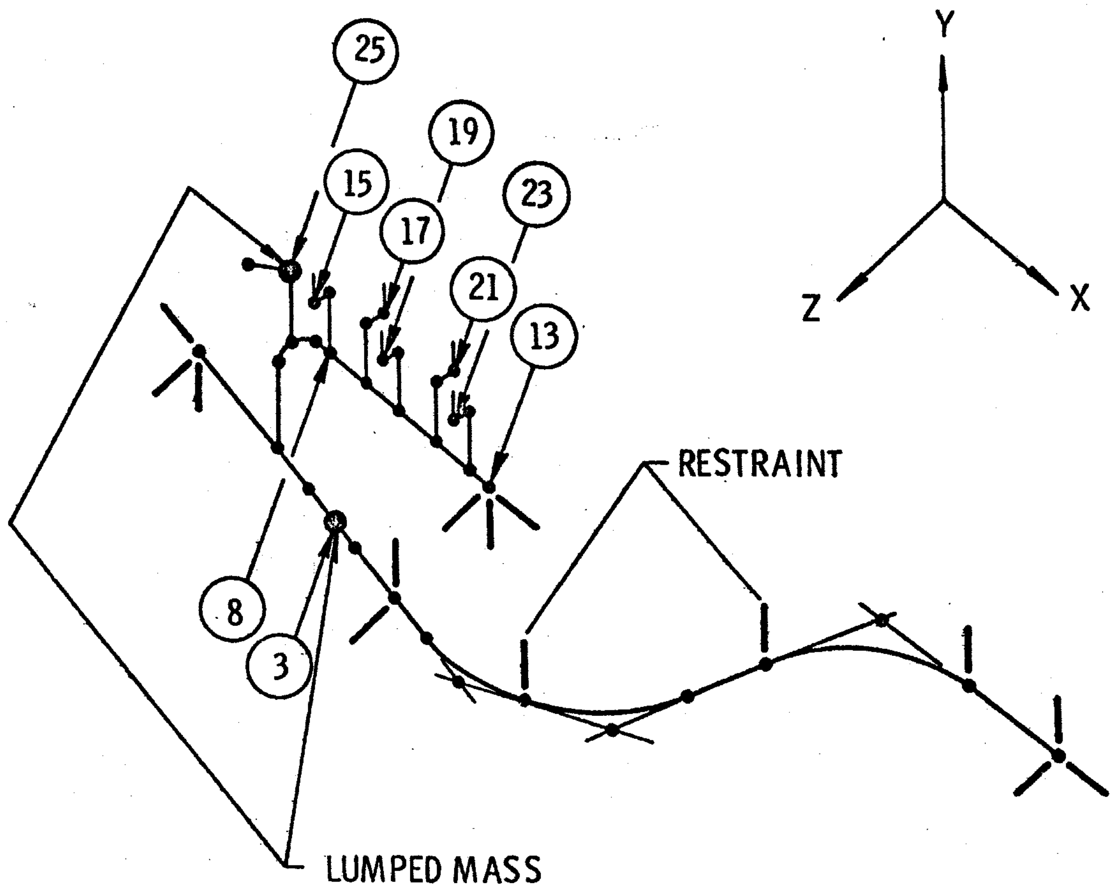

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FIGURE C-46

STRUCTURAL MODEL OF PIPING SYSTEM (PIPSYS)

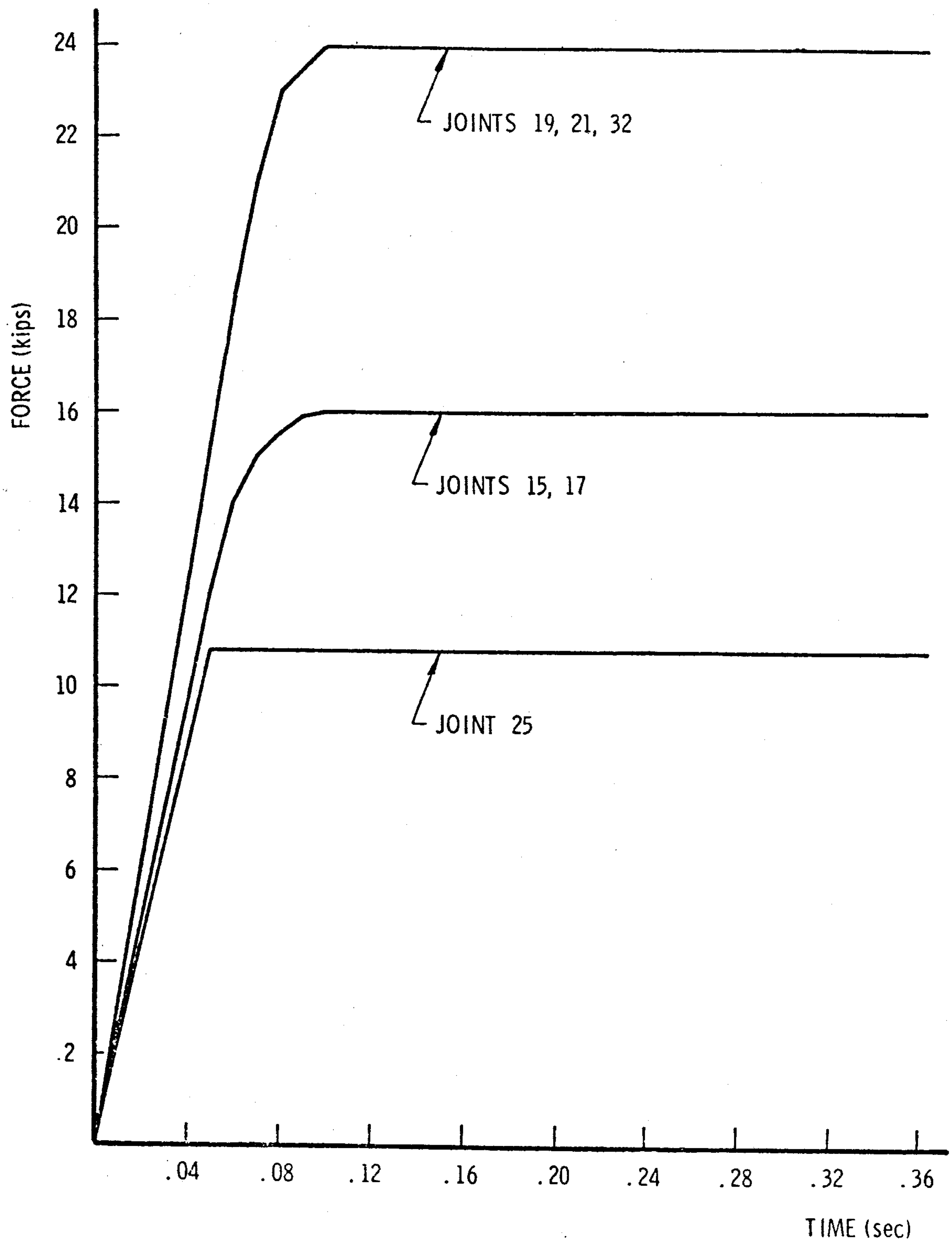

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FIGURE C-47

LOAD TIME HISTORY (PIPSYS)

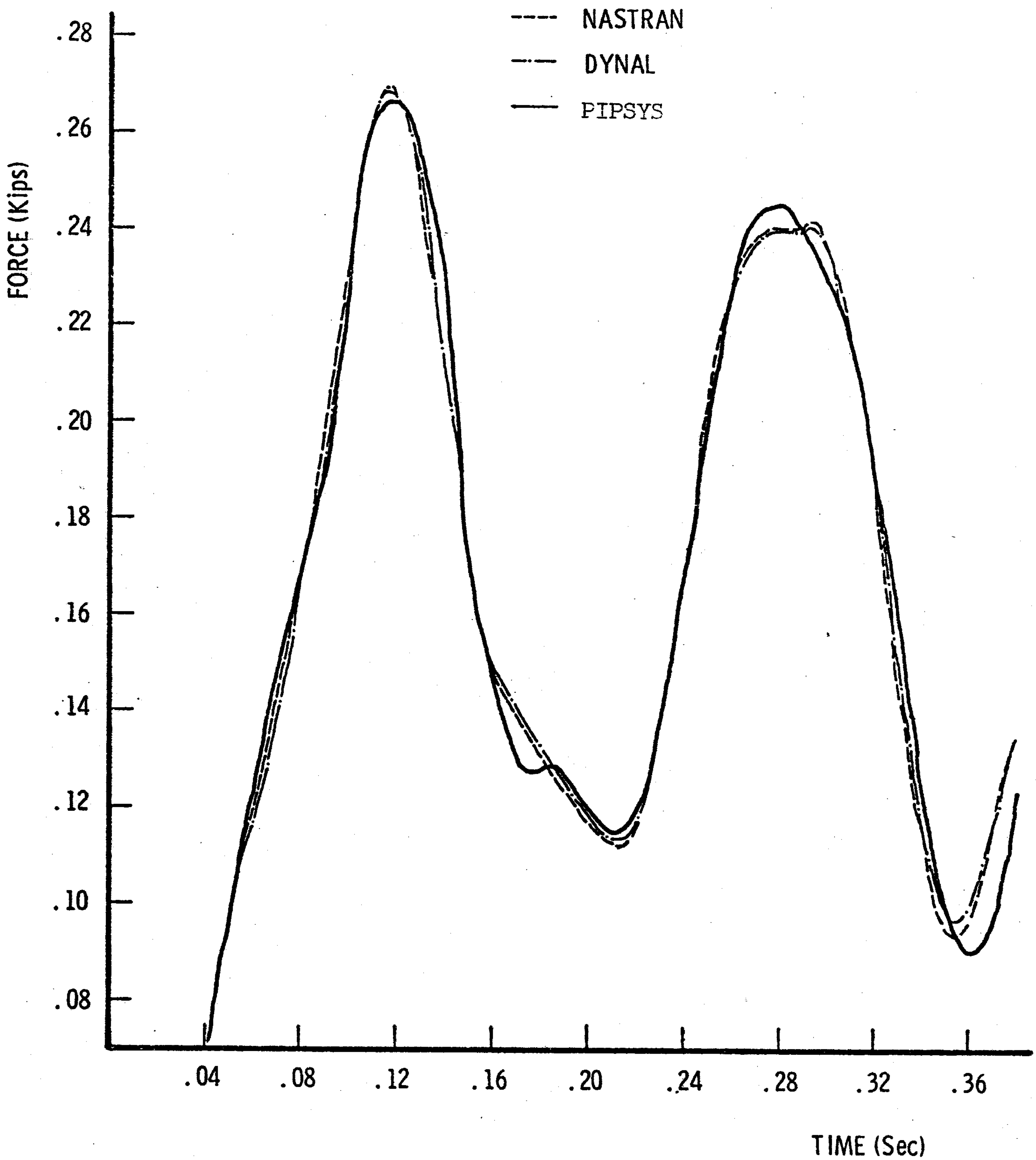

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FIGURE C-48

DISPLACEMENT VS. TIME JOINT 8
Z DIRECTION (PIPSYS)

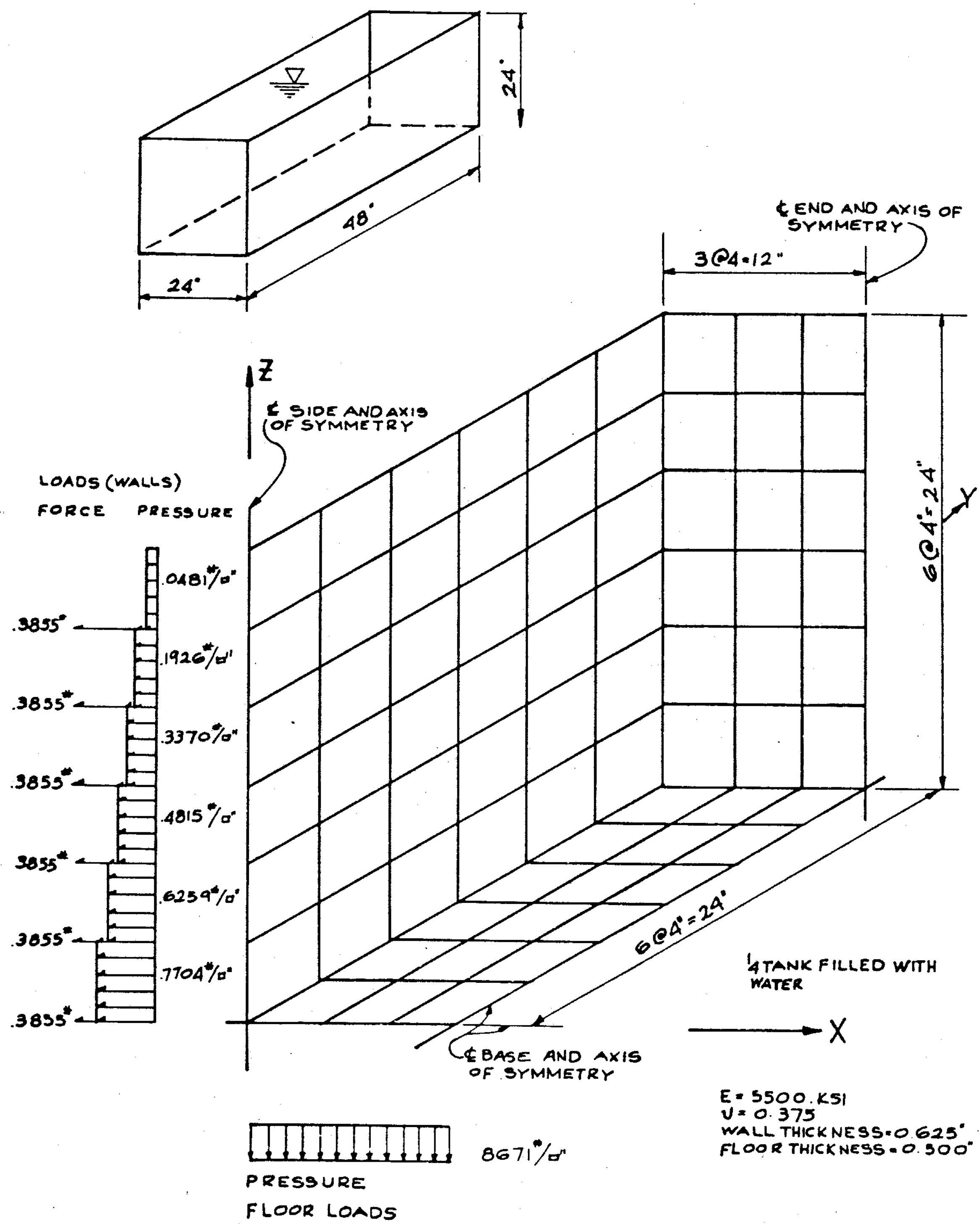

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FIGURE C-49

RECTANGULAR TANK FILLED WITH WATER (PLFEM-II)

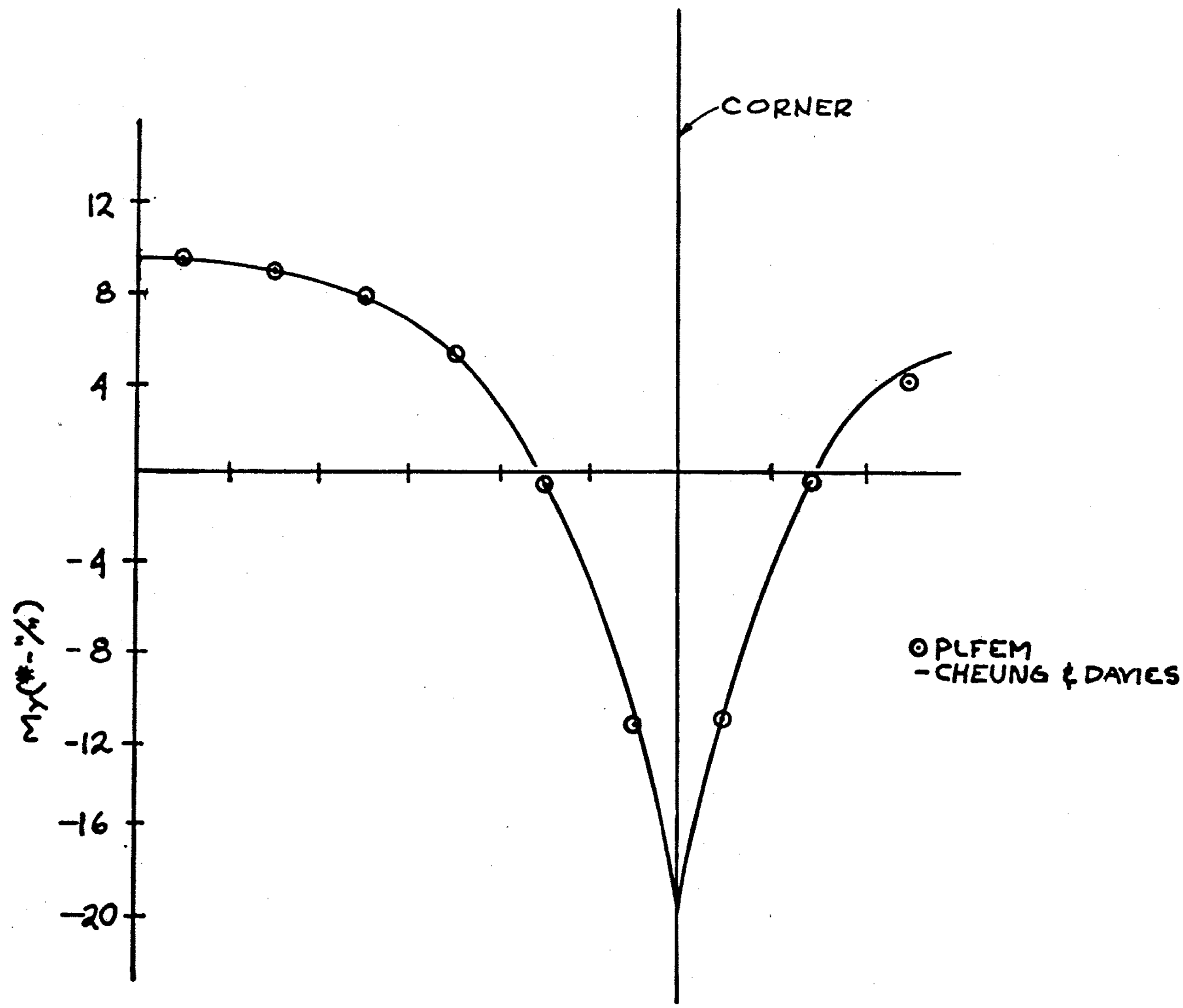

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FIGURE C-50

MOMENT OF M_y AT HORIZONTAL ℄
OF WALLS (PLFEM-II)

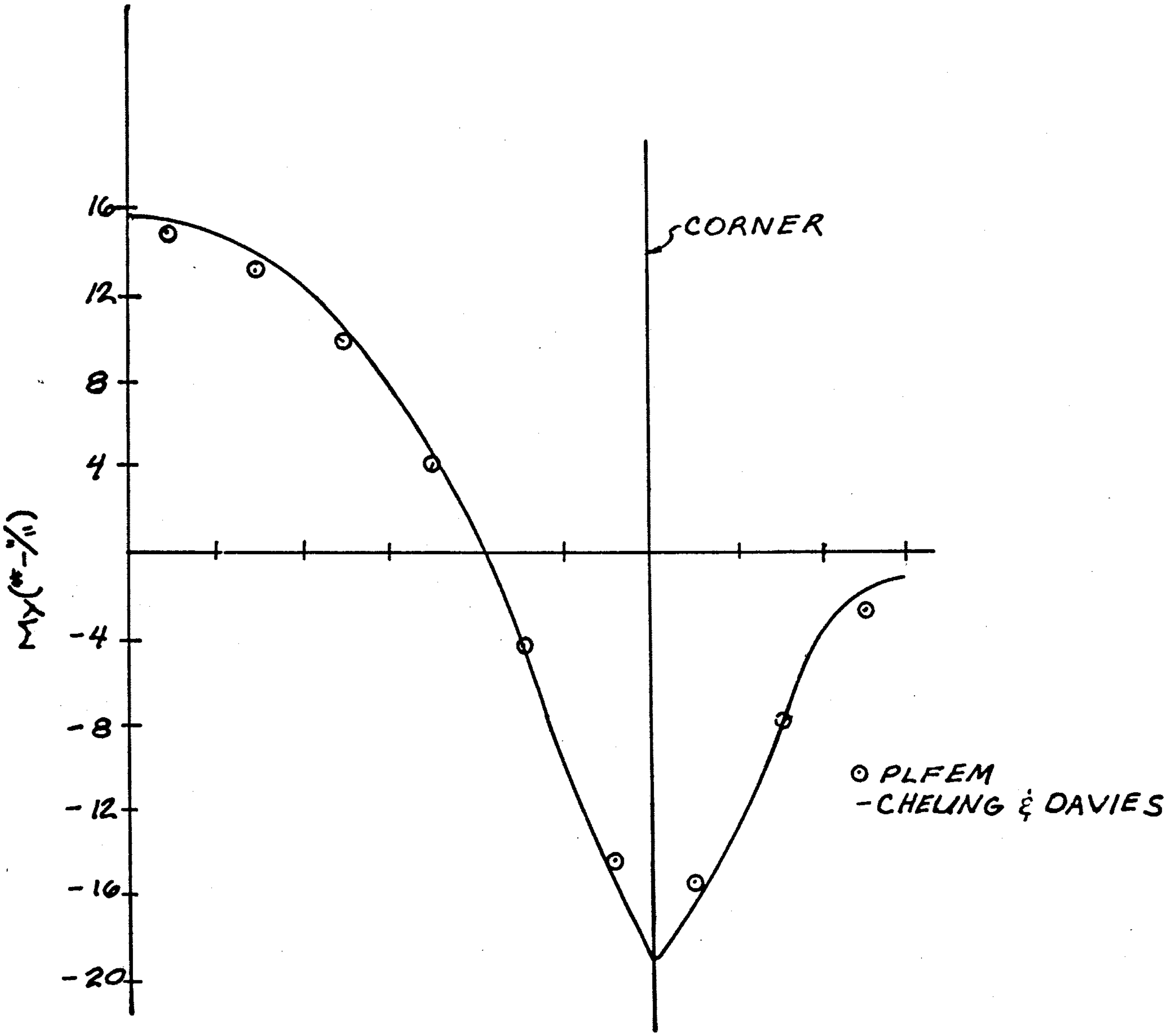

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FIGURE C-51

MOMENT M_y AT TOP OF WALL (PLFEM-II)

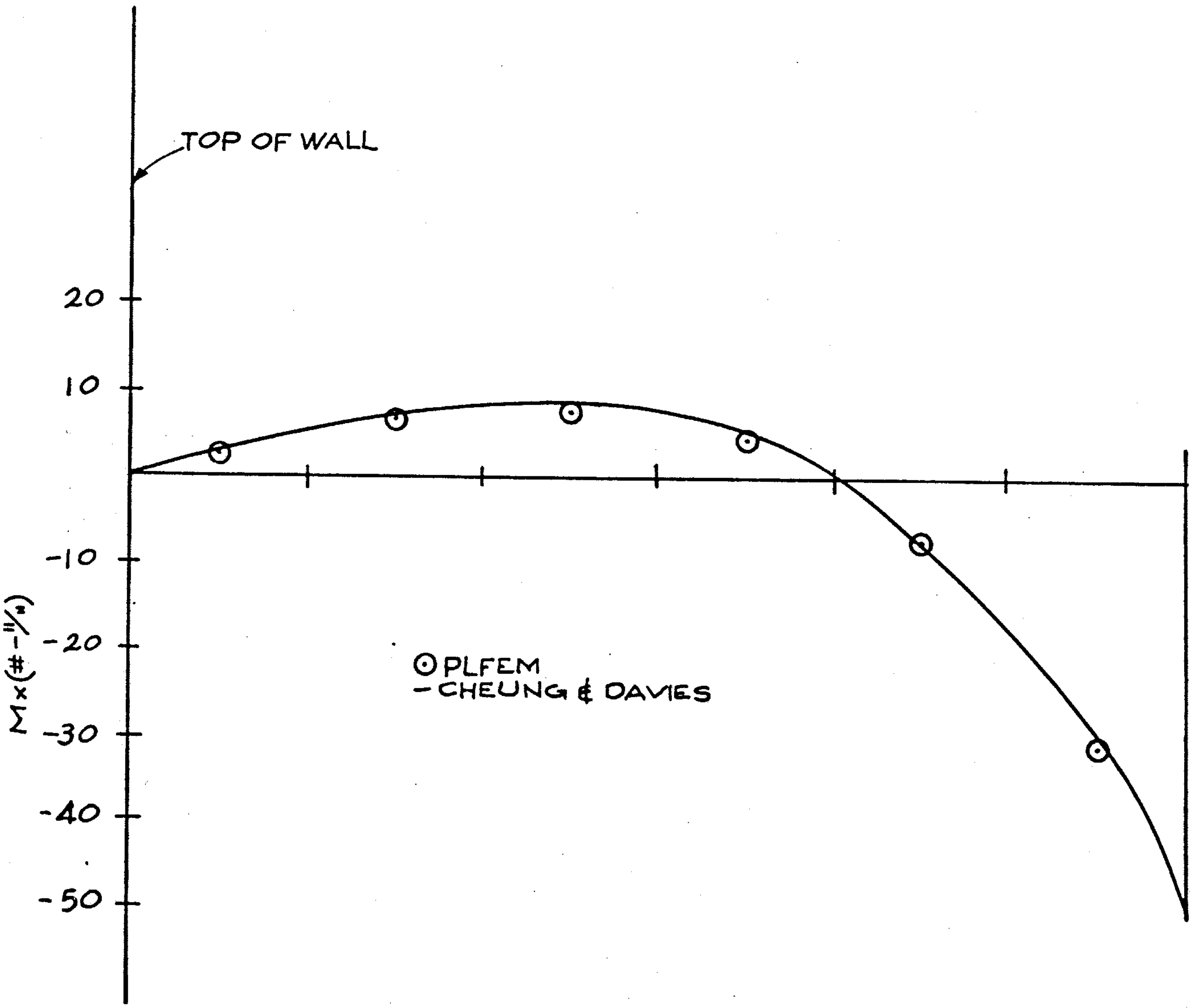

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FIGURE C-52

M_x MOMENT ALONG ℄ OF LONG WALL
(PLFEM-II)

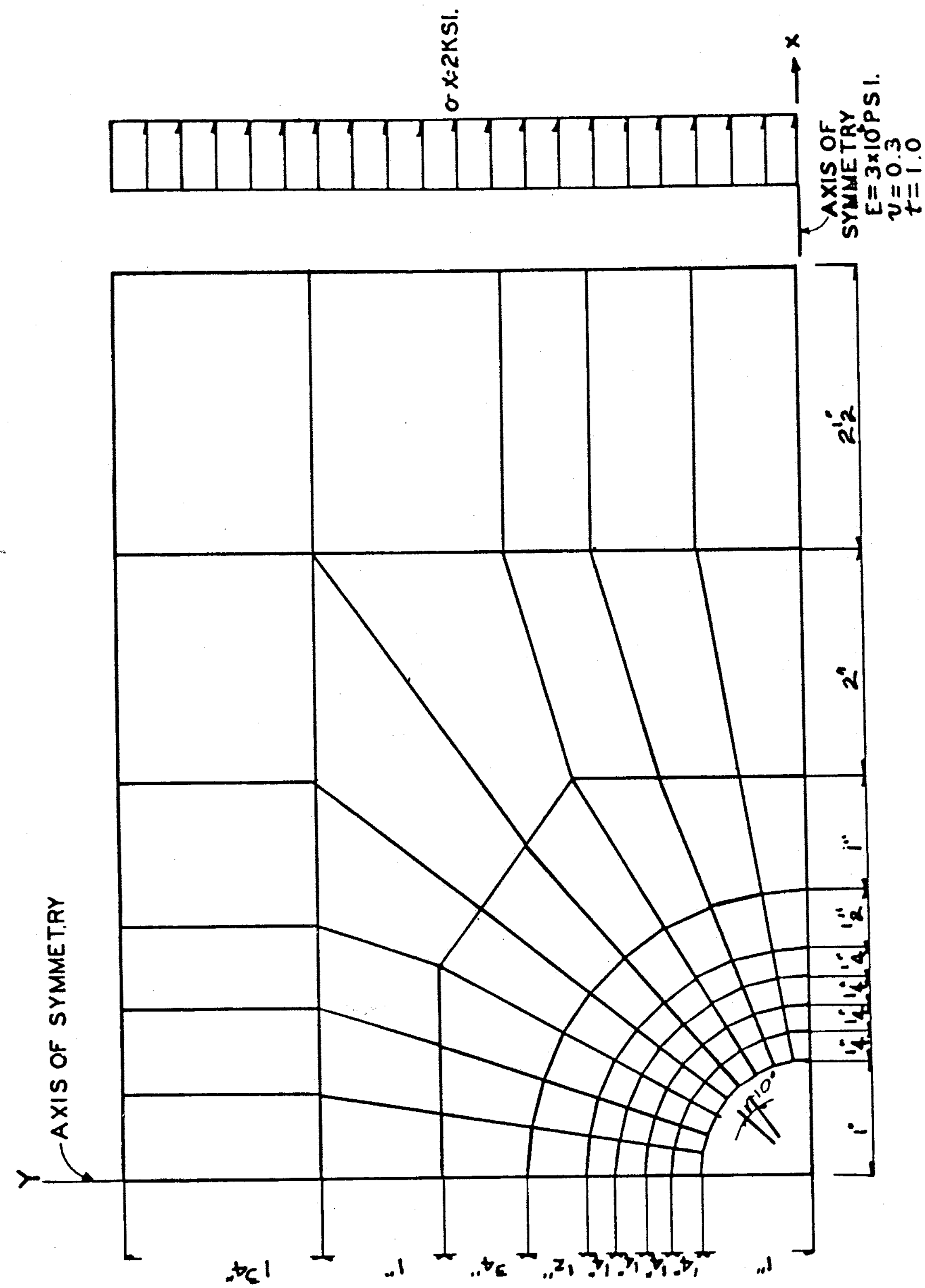

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FIGURE C-53

PLATE WITH CIRCULAR HOLE UNDER
UNIFORM TENSION (PLFEM-II)

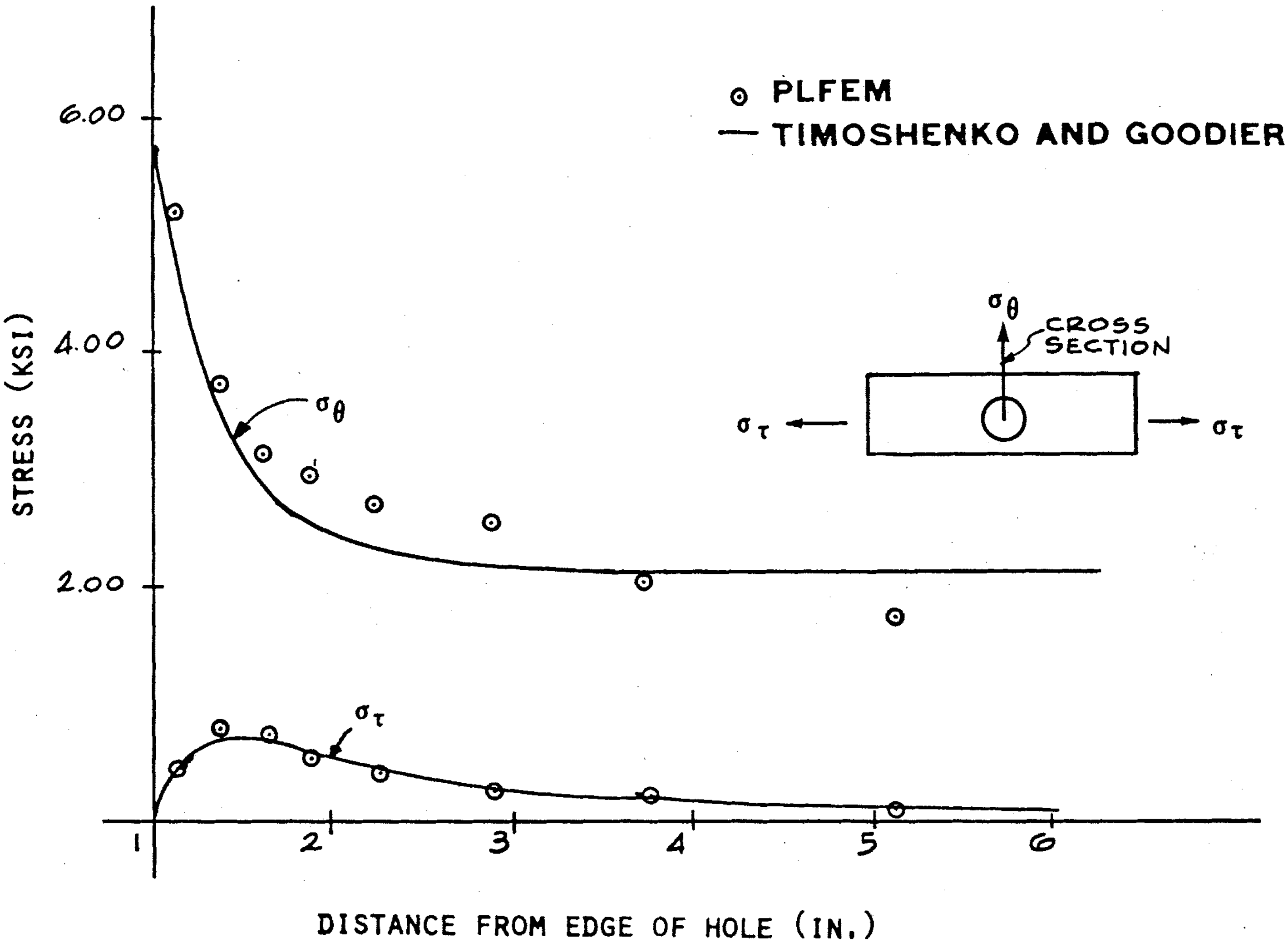

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FIGURE C-54

STRESSES IN PLATE WITH CIRCULAR HOLE UNDER UNIFORM TENSION (PLFEM-II)

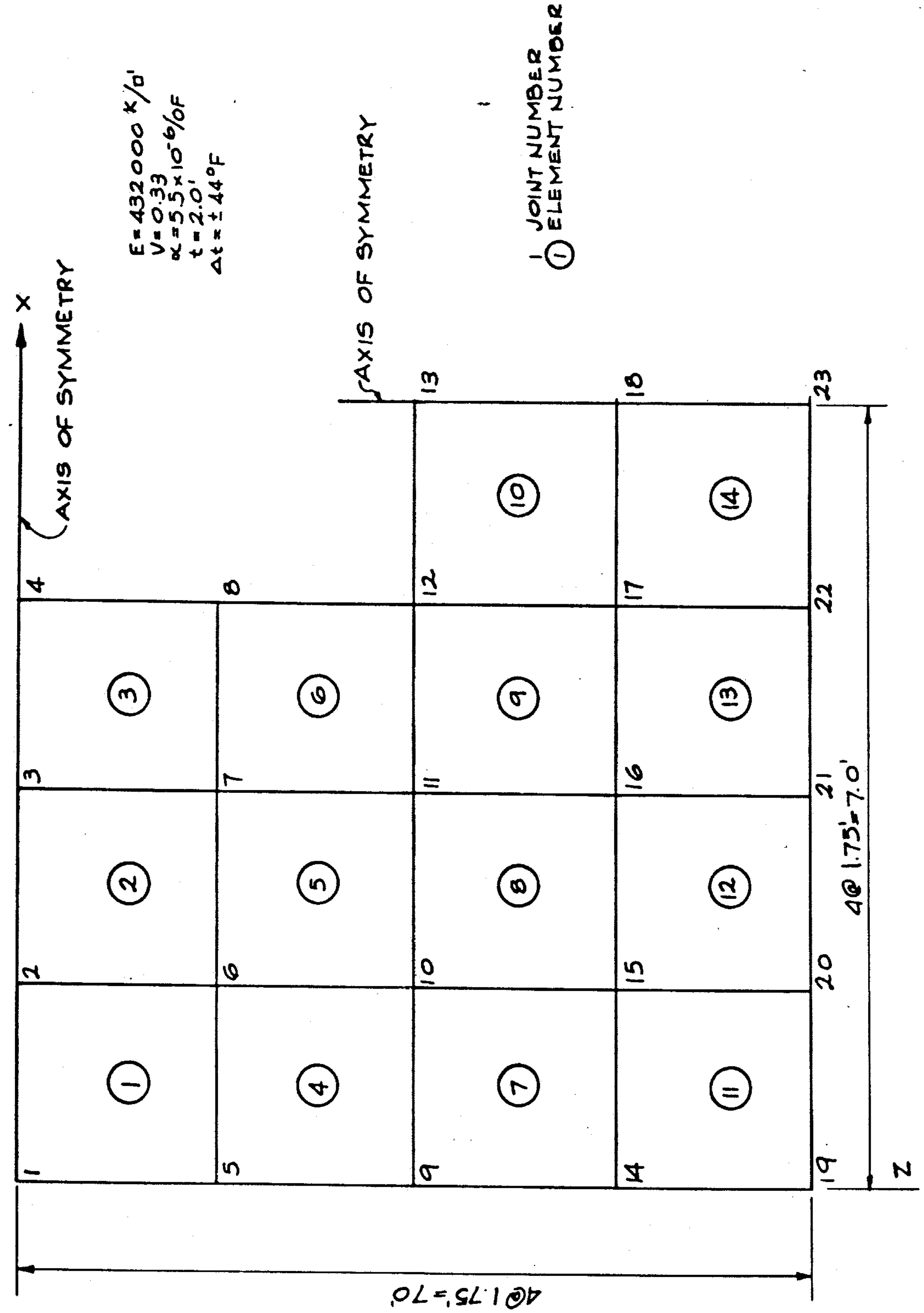

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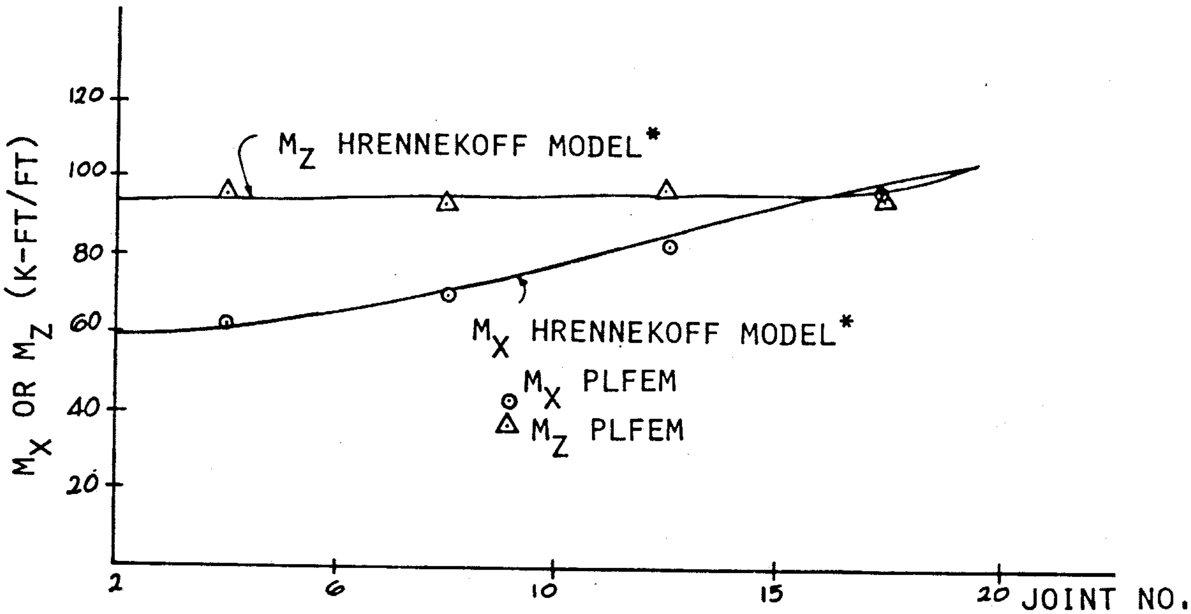
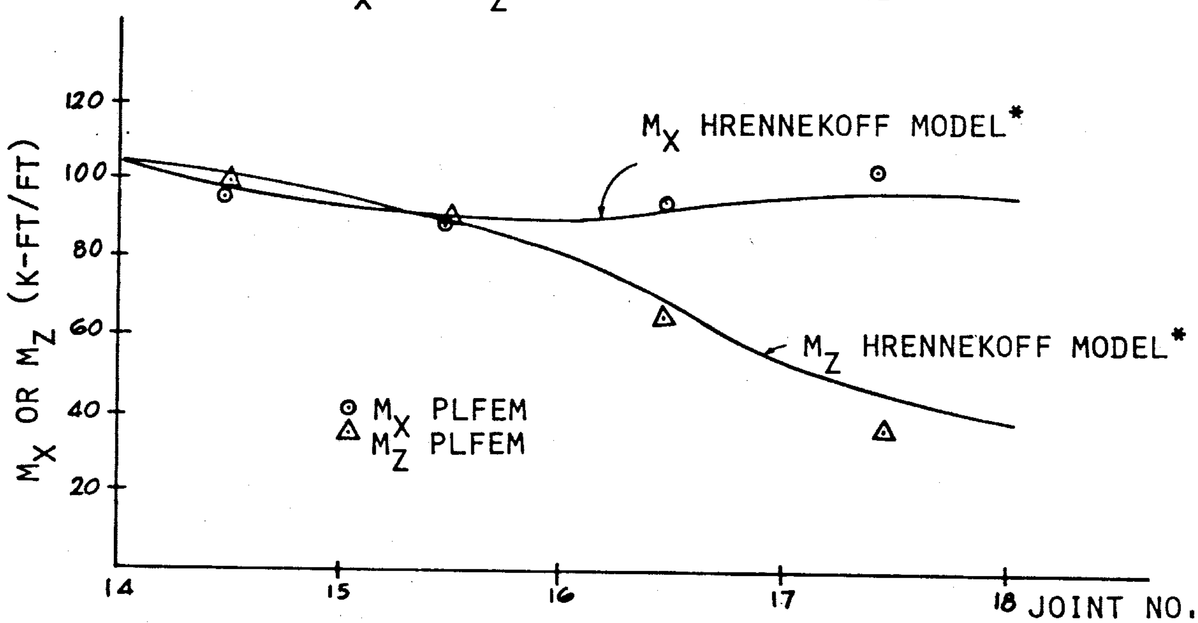
FIGURE C-55

SQUARE PLATE WITH RECTANGULAR
HOLE SUBJECTED TO TEMPERATURE
VARIATION (PLFEM-II)



M_X AND M_Z ALONG THE 2-20 LINE



M_X AND M_Z ALONG THE 14-18 LINE

* HRENNEKOFF MODEL BASED ON A FRAMEWORK ELEMENT 0.875'SQUARE

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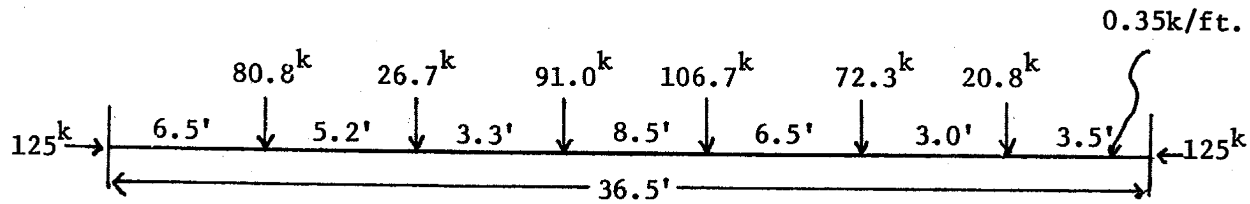
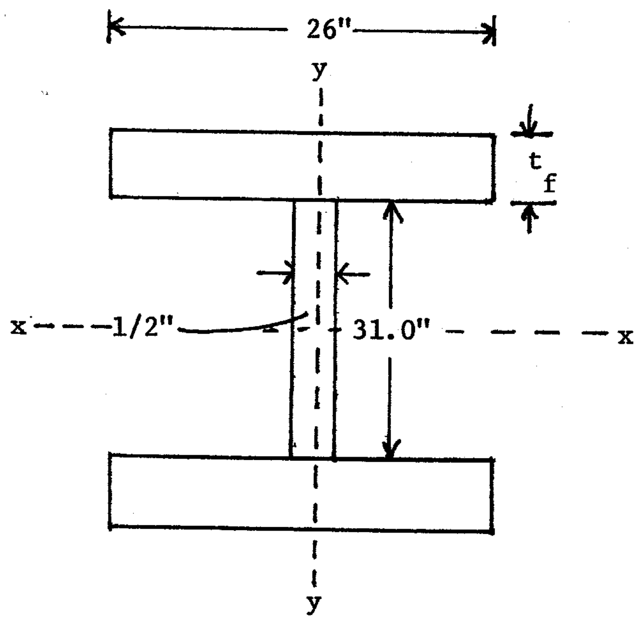
FIGURE C-56

MOMENTS IN PLATE DUE TO TEMPERATURE VARIATION (PLFEM-II)



(a) Loading Combination

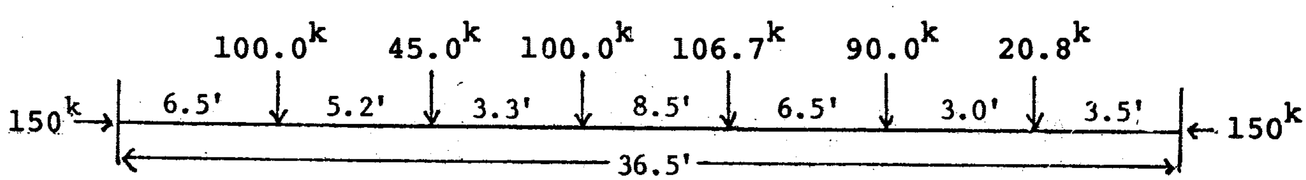


t_f = 0.875" in 5.21' from left end

= 1.6875" from 5.21' to 31.54' from left end

= 0.875" in 4.96' from right end

(b) Designed Plate Girder Configuration



(c) Revised Load Combination

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FIGURE C-57

LOADS AND CONFIGURATION FOR PLGIRD SAMPLE PROBLEM

```
     15
SAPIV  '2D CANTILEVER BEAM TEST '
NUMBER OF NODES 7 ELEMENT GROUPS 1 LOADINGS 5 NODAL LOADS 15
NODE COORDINATES
1 X 0.0 Y 0.0 Z 0.0 SUPPORT
2 X 0.0 Y 5.0 Z 0.0
3 X 0.0 Y 1.0 Z 0.0
GENERATE TO 6 Y 4.0
7 X 10.0 Y 0.0 Z 0.0 SUPPORT
C
ELEMENT GROUP 1 TYPE BEAM
NUMBER OF ELEMENTS 5 MATERIALS 1 SECTIONS 1
C
ELEMENT INCIDENCE
1 I 1 J 3 K 7
2 I 3 J 4 K -7
GEN TO 4
5  I 6 J 2 K 7
C
MATERIAL PROPERTIES
TYPE 1 E 1000.
C
SECTION PROPERTIES
TYPE 1 A1 1.E6 J1 1. I2 100. I3 1000.
C
ELEMENT INDEX MT 1 ST 1
ALL
C
RETURN
C
LOADING 1 'CONCENTRATED LOAD AT NODE 2'
NODAL LOADS
2 FORCE X 1.
LOADING 2 'CONCENTRATED LOAD AT NODES 2 + 4'
NODAL LOADS
2  FORCE X 1.
4  FORCE  X -1.
LOADING 3 'CONCENTRATED LOAD AT NODES 2,3+5'
NODAL LOADS
2,3  FORCE X 1.
5  FORCE X -2.
LOADING 4 'CONCENTRATED LOAD AT NODES 2,3,4+6'
NODAL LOADS
2,4 FORCE X 1.
3,6  FORCE X -1.
LOADING 5 'CONCENTRATED LOADS AT NODES 2,3,4,5,+6'
NODAL LOADS
2,3,5 FORCE X 2.
4 FORCE X -3.
6 FORCE X -3.
SOLVE SLSAP4
FINISH
```

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FIGURE C-58

POLSAP4 INPUT COMMANDS FOR BEAM PROBLEM

```
SLSAP4 1106
S
F
F
F
 2D CANTILEVER BEAM TEST
    7    1    5    0    0    0    0                    0000 .00000    0
    1    1    1    1    1    1    1    .00000    .00000    .00000          .00000
    2    0    0    0    0    0    0    .00000   5.00000    .00000          .00000
    3    0    0    0    0    0    0    .00000   1.00000    .00000          .00000
    4    0    0    0    0    0    0    .00000   2.00000    .00000          .00000
    5    0    0    0    0    0    0    .00000   3.00000    .00000          .00000
    6    0    0    0    0    0    0    .00000   4.00000    .00000          .00000
    7    1    1    1    1    1    1  10.00000    .00000    .00000          .00000
    2    5    1    0    1    0
    1     1000. .00000000   .000000   .000000
    1 .10000+07 .00000    .00000    .10000+01 .10000+03 .10000+04   .000   .000
    .00000    .00000    .00000    .00000
    .00000    .00000    .00000    .00000
    .00000    .00000    .00000    .00000
    .00000    .00000    .00000    .00000
    1    1    3    7    1    1    0    0    0    0000000000000        0 0 0 0 1
    .00000    .00000    .00000
    2    3    4    7    1    1    0    0    0    0000000000000        0 0 0 0 1
    .00000    .00000    .00000
    3    4    5    7    1    1    0    0    0    0000000000000        0 0 0 0 1
    .00000    .00000    .00000
    4    5    6    7    1    1    0    0    0    0000000000000        0 0 0 0 1
    .00000    .00000    .00000
    5    6    2    7    1    1    0    0    0    0000000000000        0 0 0 0 1
    .00000    .00000    .00000
    2    1    1.0000    .0000    .0000    .0000    .0000    .0000
    2    2    1.0000    .0000    .0000    .0000    .0000    .0000
    2    3    1.0000    .0000    .0000    .0000    .0000    .0000
    2    4    1.0000    .0000    .0000    .0000    .0000    .0000
    2    5    2.0000    .0000    .0000    .0000    .0000    .0000
    3    3    1.0000    .0000    .0000    .0000    .0000    .0000
    3    4   -1.0000    .0000    .0000    .0000    .0000    .0000
    3    5    2.0000    .0000    .0000    .0000    .0000    .0000
    4    2   -1.0000    .0000    .0000    .0000    .0000    .0000
    4    4    1.0000    .0000    .0000    .0000    .0000    .0000
    4    5   -3.0000    .0000    .0000    .0000    .0000    .0000
    5    3   -2.0000    .0000    .0000    .0000    .0000    .0000
    5    5    2.0000    .0000    .0000    .0000    .0000    .0000
    6    4   -1.0000    .0000    .0000    .0000    .0000    .0000
    6    5   -3.0000    .0000    .0000    .0000    .0000    .0000
    0
    .00000    .00000    .00000    .00000
    .00000    .00000    .00000    .00000
    .00000    .00000    .00000    .00000
    .00000    .00000    .00000    .00000
    .00000    .00000    .00000    .00000
     0
     0
```

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FIGURE C-59

GENERATED SLSAP4 DATA FOR BEAM PROBLEM

```
000        5
000   SAPIV ' SQUARE PLATE TEST - 4 TRIANGLES '
000   C
000   NUMBER OF NODES 5 ELEMENT GROUPS 2 LOADINGS 4 NODAL LOADS 1
000   C
000   NODE COORDINATES
000   1 X 0.0  Y 0.0  SUPPORT
000   2 X 0.5  Y 0.0   FREE
000   GEN TO 4  X 0.0  Y 0.5
000   5  X 0.5  Y 0.5  SUPPORT
000   C
000   NODE RELEASE  FORCE Z
000   1,5
000   NODE CONSTRAINT
000   2  FORCE  X Y  MOMENT Y Z
000   3  MOMENT Z
000   4  FORCE X  Y  MOMENT X Z
000   C
000   ELEMENT GROUP 1 TYPE PLATE
000   C
000   NUMBER OF ELEMENTS 4 MATERIALS 1
000   C
000   ELEMENT INCIDENCE
000   1  I 4  J 1  K 3
000   2  I 1  J 2  K 3
000   3  I 2  J 5  K 3
000   4  I 5  J 4  K 3
000   C
000   MATERIAL PROPERTIES
000   TYPE  1  XCTE 1. YCTE 1. THICKNESS 0.01 DENSITY 1.  #
000   CXX  1.0989  CXY  0.32967  CYY  1.0989  GXY  0.3846
000   C
000   ELEMENT INDEX  MT 1
000   ALL
000   C
000   ELEMENT LOAD MULTIPLIERS
000   CASE B  PRESSURE  1.
000   CASE C  GRAVITY Z 1.
000   CASE D  TEMPERATURE 1.
000   C
000   PRESSURE LOAD 1.
000   ALL
000   TEMPERATURE VARIATION 1.
000   ALL
000   TEMPERATURE GRADIENT 1.
000   ALL
000   C
000   RETURN
000   C
000   ELEMENT GROUP 2 TYPE BOUNDARY
000   C
000   NUMBER OF ELEMENTS 3
000   C
000   ELEMENT INCIDENCE
000   1  NODE 1  DIRECTION  I 1  J 2  K 1  L 4
000   2  NODE 2  DIRECTION  I 1  J 2  K 1  L 4
000   3  NODE 4  DIRECTION  I 1  J 2  K 1  L 4
000   C
000   ELEMENT INDEX  DISPLACEMENT
000   ALL
000   C
000   SPECIFIED SPRING STIFFNESS  1.E+10
000   ALL
000   C
000   RETURN
000   C
000   LOADING 1 ' NODAL LOAD ONLY '
000   NODAL LOADS
000   5  FORCE Z 100.
000   LOADING 2 ' LOAD CASE B ONLY '
000   COMBINE CASE B 100.
000   LOADING 3 ' LOAD CASE C ONLY '
000   COMBINE CASE C 100.
000   LOADING 4 ' LOAD CASE D ONLY '
000   COMBINE CASE D 10.
000   C
000   PLOT MESH
000   FRAME  WIDTH 11.  LENGTH 8.
000   C
000   SOLVE  SLSAP4
000   C
000   FINISH
```

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FIGURE C-60

POLSAP4 INPUT FOR PLATE PROBLEM

```
000   SLSAP4 1106
000   S
000   F
000   F
000   F
000     SQUARE PLATE TEST - 4 TRIANGLES
000       5    2    4    0    0    0    0                    0000 .00000      0
000       1    1    1    0    1    1    1    .00000      .00000      .00000          .00000
000       2    1    1    0    0    1    1    .50000      .00000      .00000          .00000
000       3    0    0    0    0    0    1    .25000      .25000      .00000          .00000
000       4    1    1    0    1    0    1    .00000      .50000      .00000          .00000
000       5    1    1    0    1    1    1    .50000      .50000      .00000          .00000
000       6    4    1
000            1                        1.000000  1.000000  1.000000   .010000
000        1.099       .330       .000     1.099      .000       .385        0.    .000000
000      .00000   1.00000     .00000     .00000
000      .00000    .00000     .00000    1.00000
000      .00000    .00000     .00000     .00000
000      .00000    .00000     .00000     .00000
000      .00000    .00000    1.00000     .00000
000       1    4    1    3    0         1          .01000    1.00000    1.00000    1.00000
000       2    1    2    3    0         1          .01000    1.00000    1.00000    1.00000
000       3    2    5    3    0         1          .01000    1.00000    1.00000    1.00000
000       4    5    4    3    0         1          .01000    1.00000    1.00000    1.00000
000       7    3
000      .00000    .00000     .00000     .00000
000       1    1    2    1    4    1    0         .000000    .000000 .10000+11
000       2    1    2    1    4    1    0         .000000    .000000 .10000+11
000       4    1    2    1    4    1    0         .000000    .000000 .10000+11
000       5    1     .0000      .0000   100.0000     .0000      .0000      .0000
000       0
000      .00000    .00000     .00000     .00000
000      .00000 100.00000     .00000     .00000
000      .00000    .00000  100.00000     .00000
000      .00000    .00000     .00000   10.00000
000        0
000        0
```

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CLINTON POWER STATION
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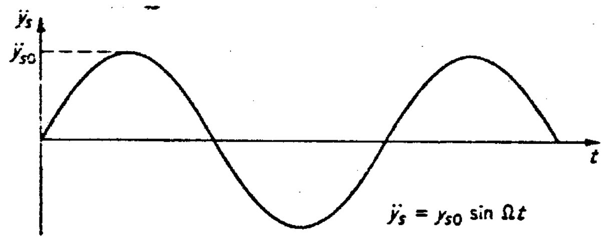
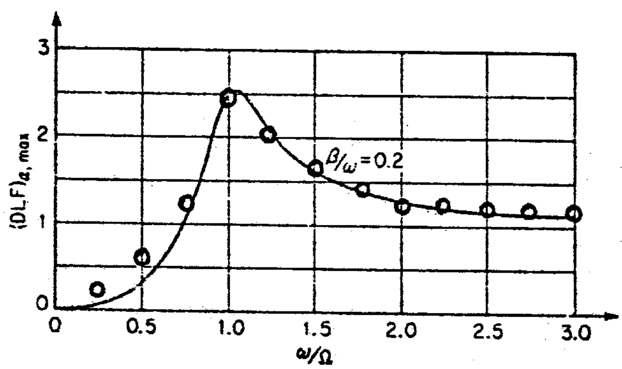
FIGURE C-61

GENERATED SLSAP4 INPUT FOR PLATE PROBLEM



Acceleration Time History
$\Omega = 2\pi \cdot (5)$ radians/sec.



Response Spectrum

$\beta/\omega = 0.2$

Ω = Forcing frequency in radians/second

$(DLF)_{a,max}$ = Maximum Dynamic Load Factor

___ Biggs

ooo RSG

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FIGURE C-62

RESPONSE SPECTRUM FOR SINUSOIDAL VARIATION OF GROUND MOTION

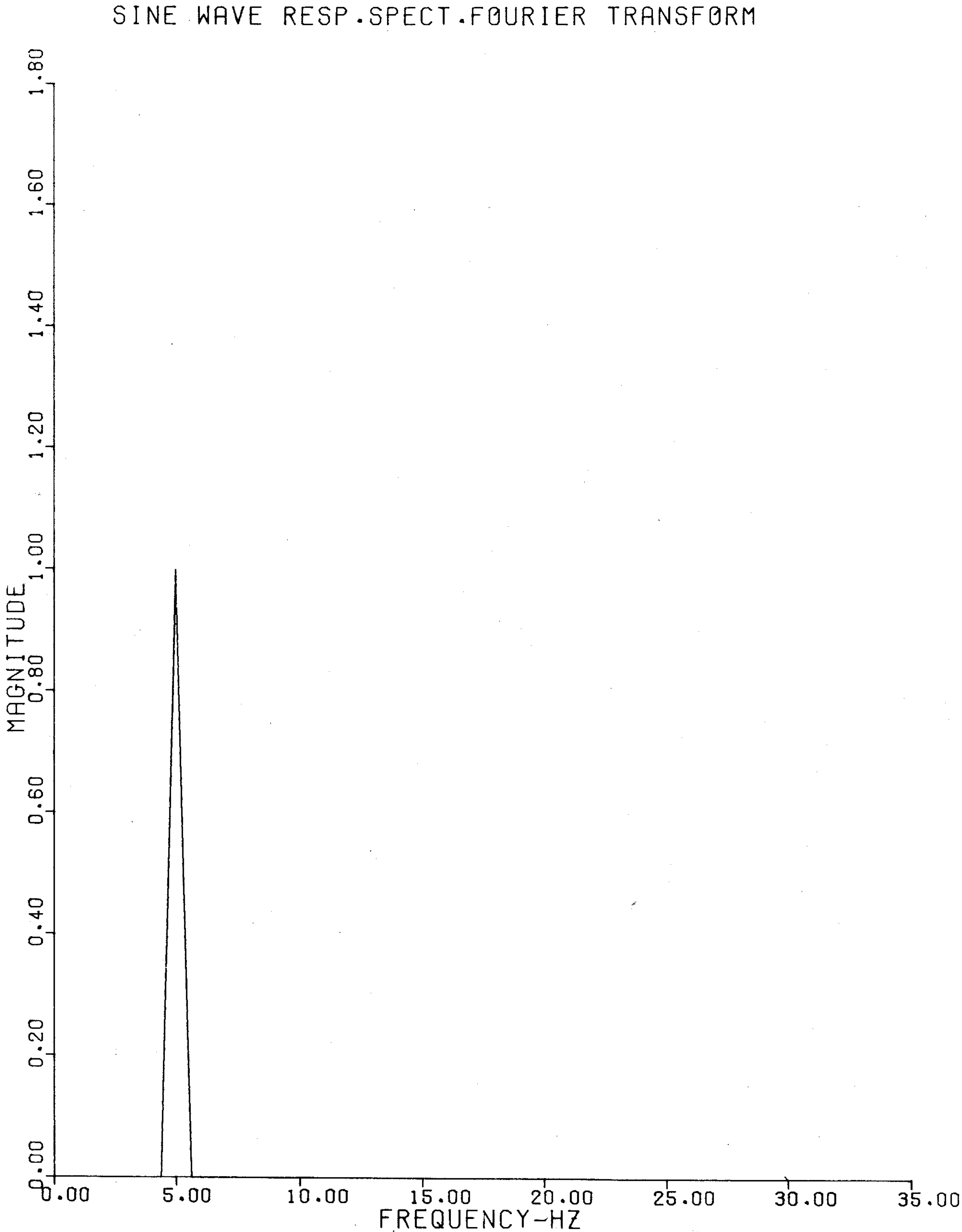

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FIGURE C-63

FOURIER TRANSFORM PLOT FROM RSG FOR A 5 CYCLE/SEC SINE WAVE TIME HISTORY

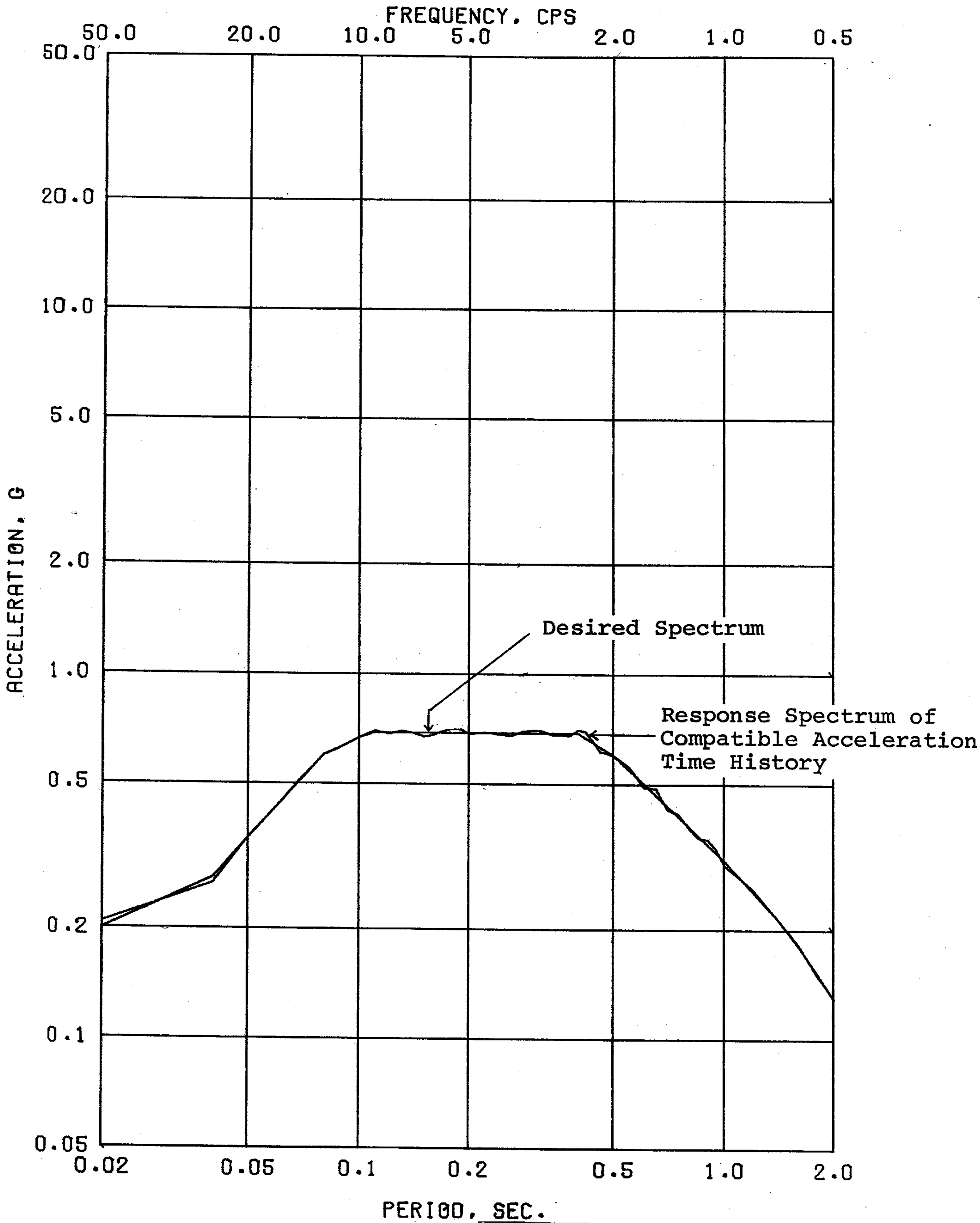

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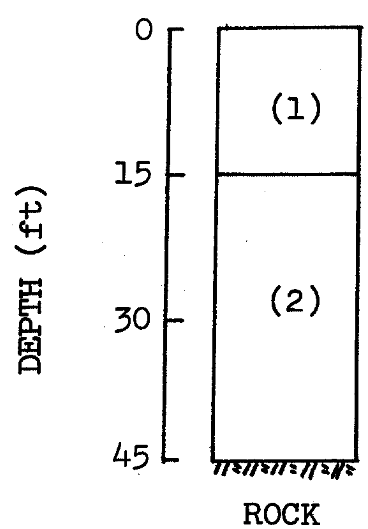
FIGURE C-64

COMPARISON OF DESIRED RESPONSE SPECTRUM AND RESPONSE SPECTRUM OF COMPATIBLE ACCELERATION TIME HISTORY (DAMPING = 0.02) FROM RSG

Excavated Effective
Soil Pressure
= 4232.8 psf



UNIT WT. (pcf)	P > Pc m	P > Pc a	P < Pc m	P < Pc a	Pc(psf)
85.6	57.565	0	575.646	0	20,000
81.6	230.759	0	2302.585	0	20,000

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FIGURE C-65a

SOIL PROFILE AND PROPERTIES FOR CONSOLIDATION SETTLEMENT COMPUTATION USING JANBU'S METHOD (SETTLE VALIDATION PROBLEMS 1 AND 4)

Loading Area	LENGTH (ft)	WIDTH (ft)	LOAD (psf)	DEPTH (ft)
(1)	104.5	109.5	852.0	0
(2)	26.5	109.5	-1013.0	0
(3)	129.4	21.5	-1013.0	0

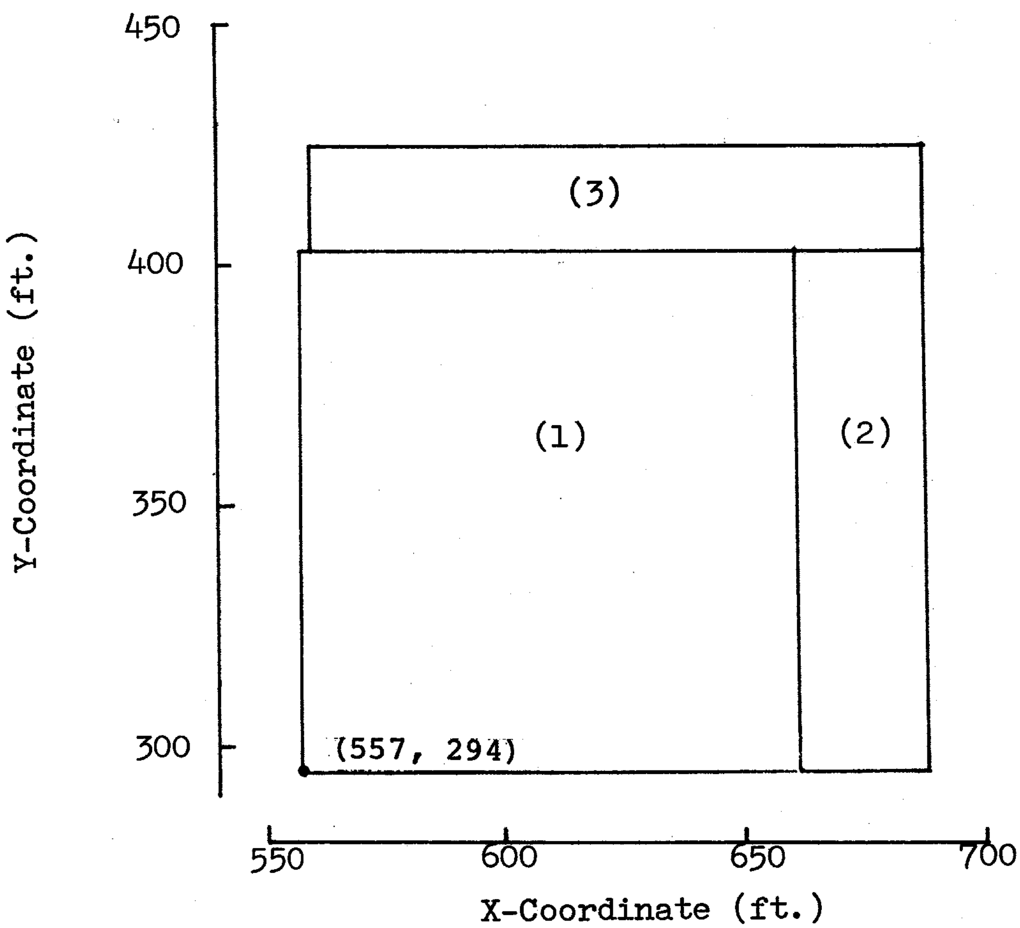

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FIGURE C-65b

LOADING AREA ON SOIL FOR SETTLE VALIDATION PROBLEMS 1 TO 3

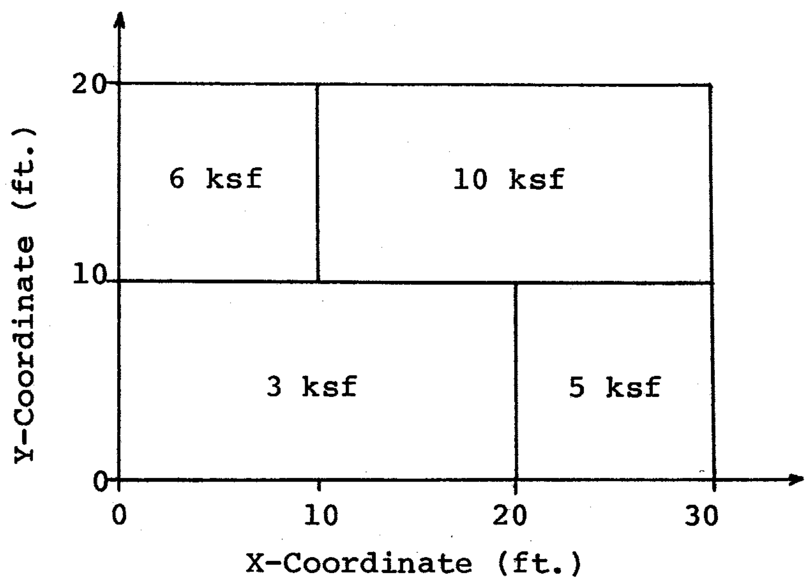

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FIGURE C-66a

LOADING AREA USED FOR CALCULATING RIGID FOUNDATION MOVEMENT FOR SETTLE VALIDATION PROBLEM 4

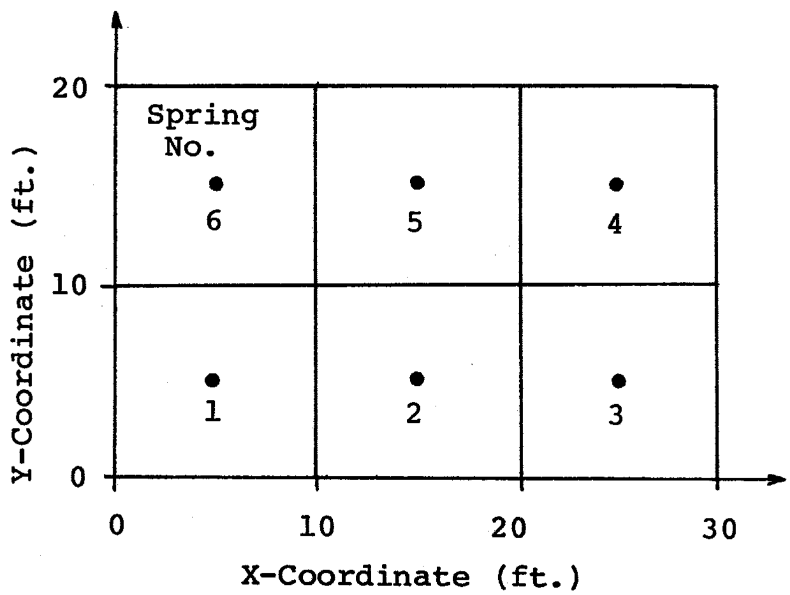

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FIGURE C-66b

LOCATION OF SPRING FOR CALCULATING RIGID FOUNDATION MOVEMENT FOR SETTLE VALIDATION PROBLEM 4

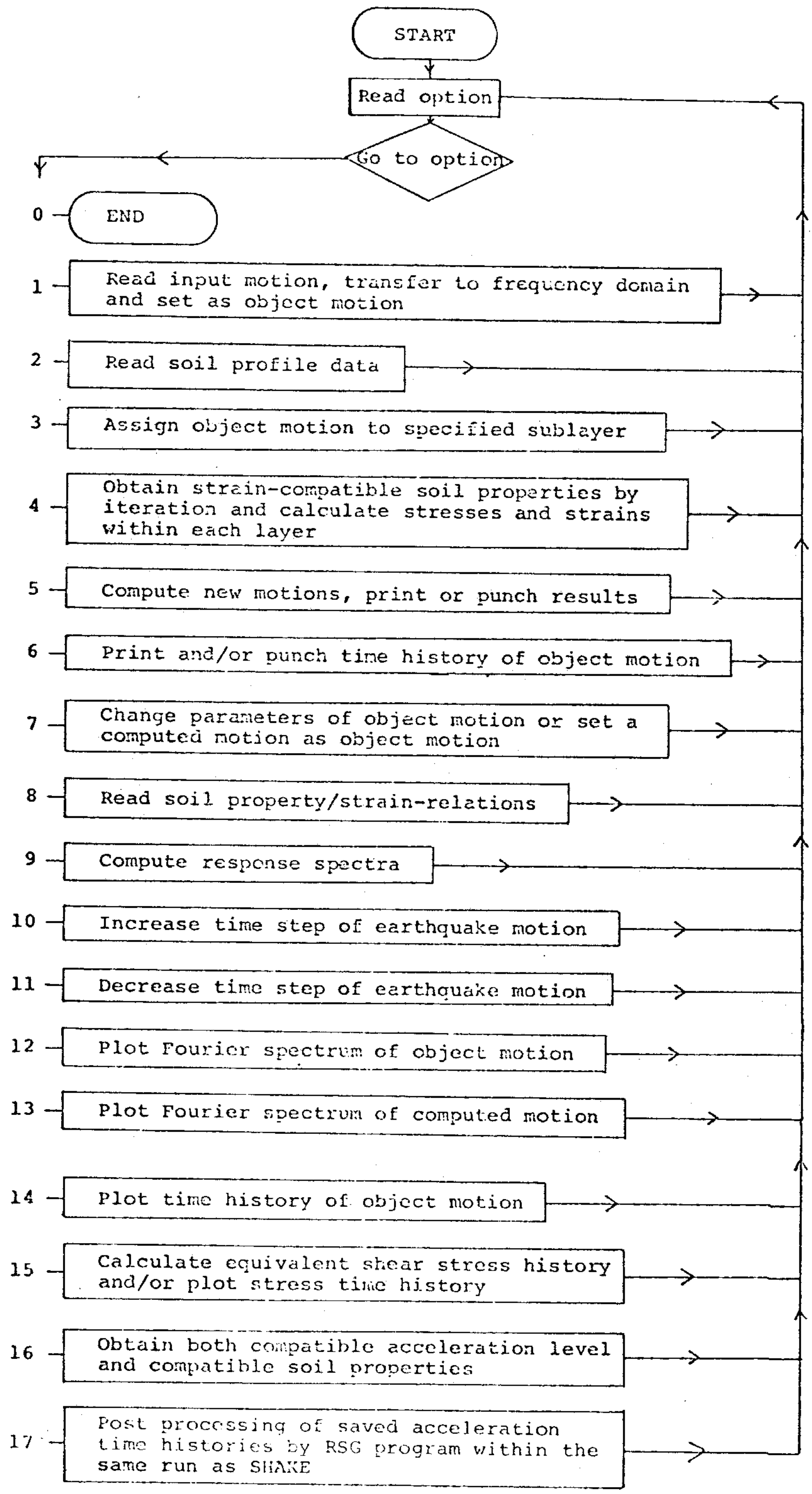

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FIGURE C-67

FLOW CHART OF SHAKE

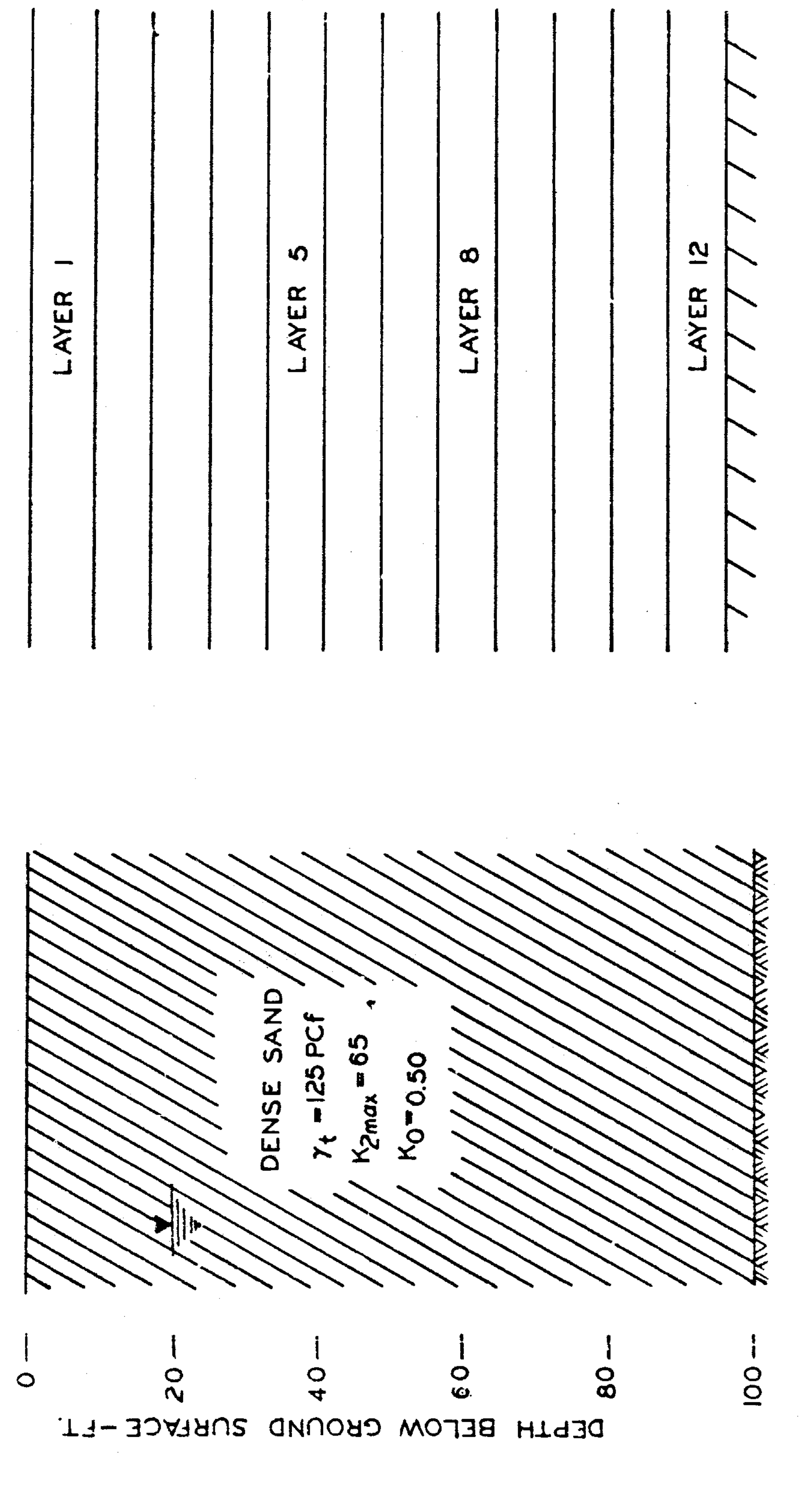

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FIGURE C-68

SOIL PROFILE AND LAYERED REPRESENTATION USED FOR THE SHAKE SAMPLE PROBLEM

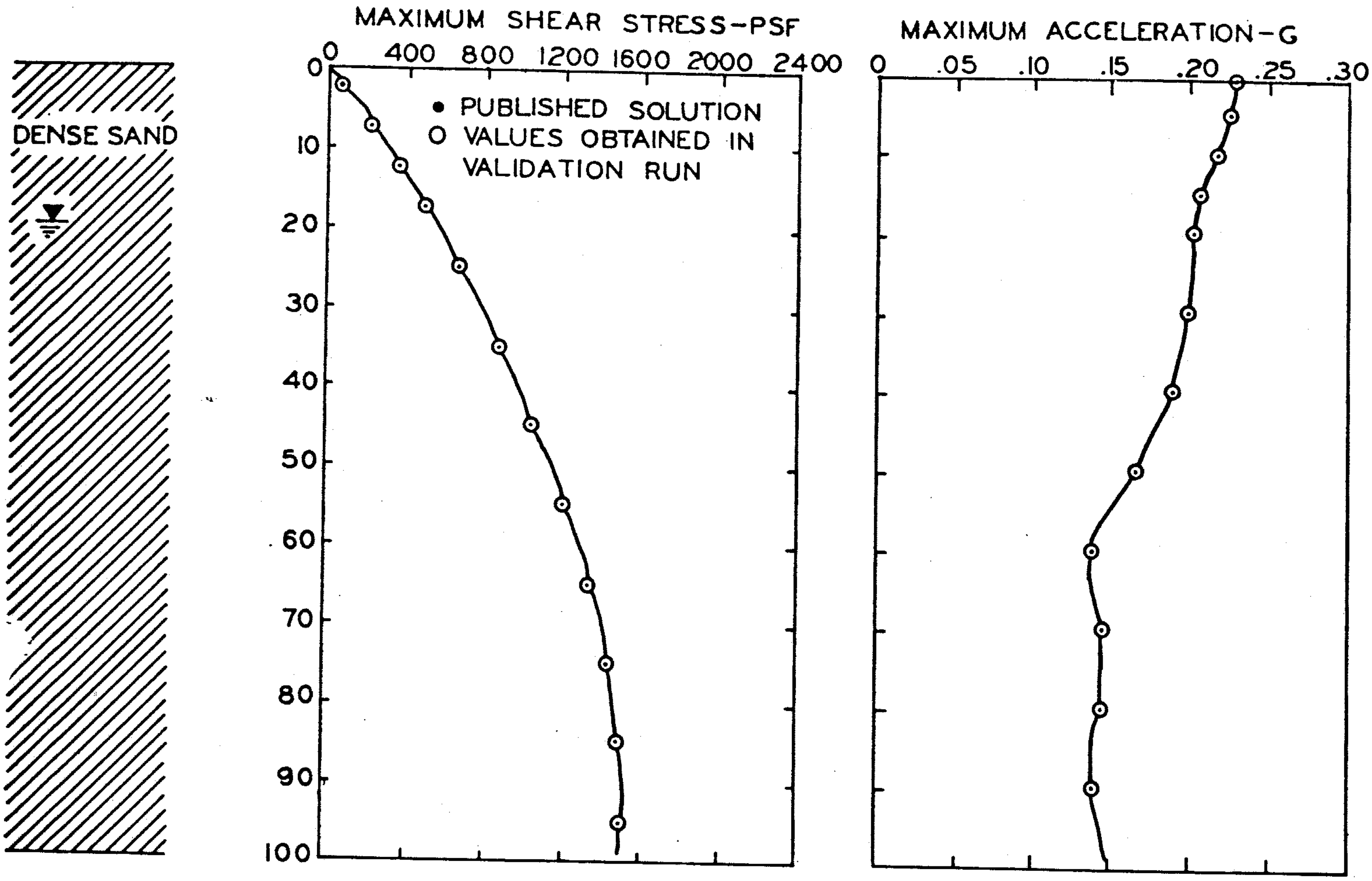

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FIGURE C-69

COMPARISON OF SHEAR STRESSES AND ACCELERATIONS COMPUTED BY SHAKE AND QUAD4 (SHAKE)

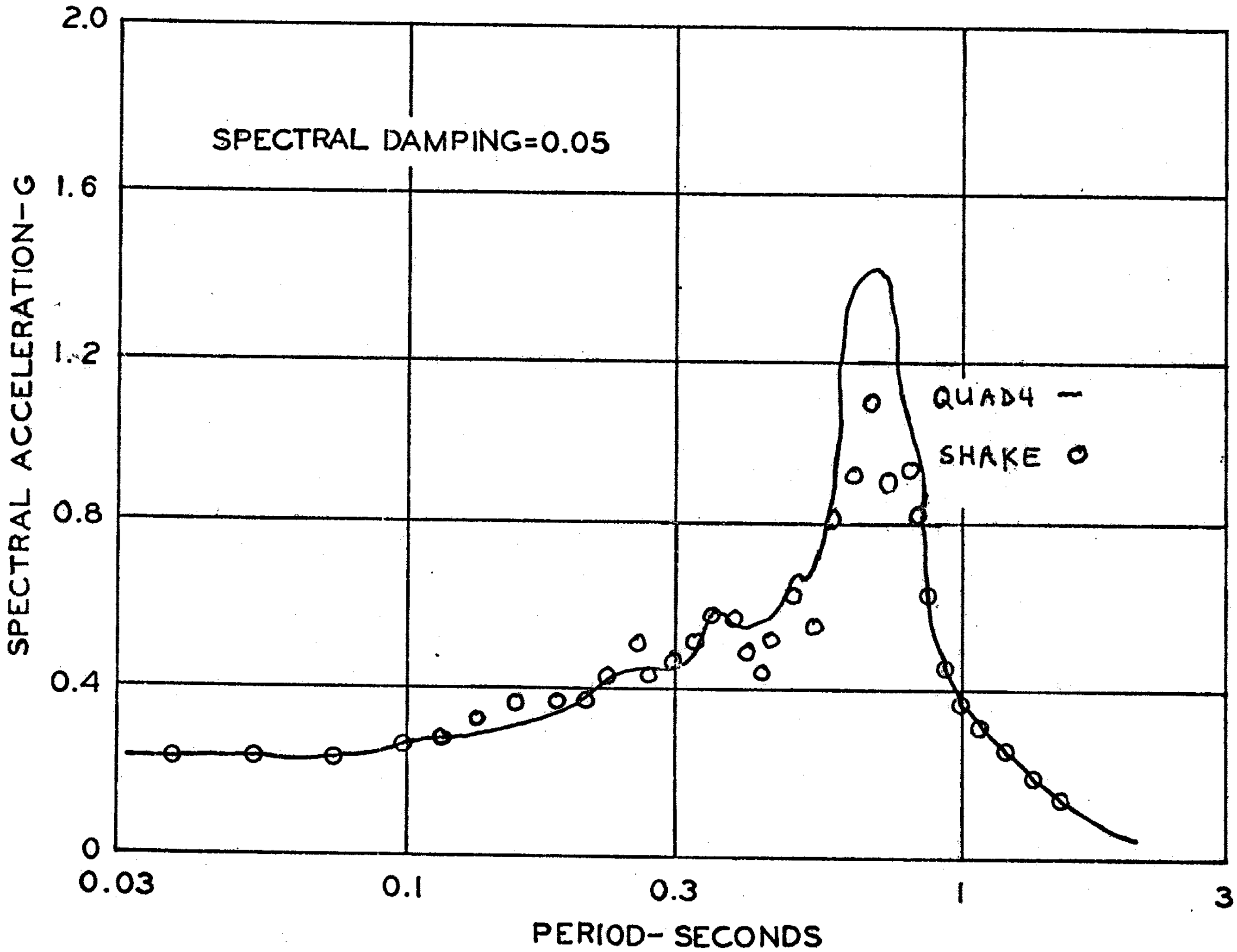

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FIGURE C-70

COMPARISON OF SPECTRAL VALUES FOR SURFACE MOTIONS COMPUTED BY SHAKE AND QUAD4 (SHAKE)

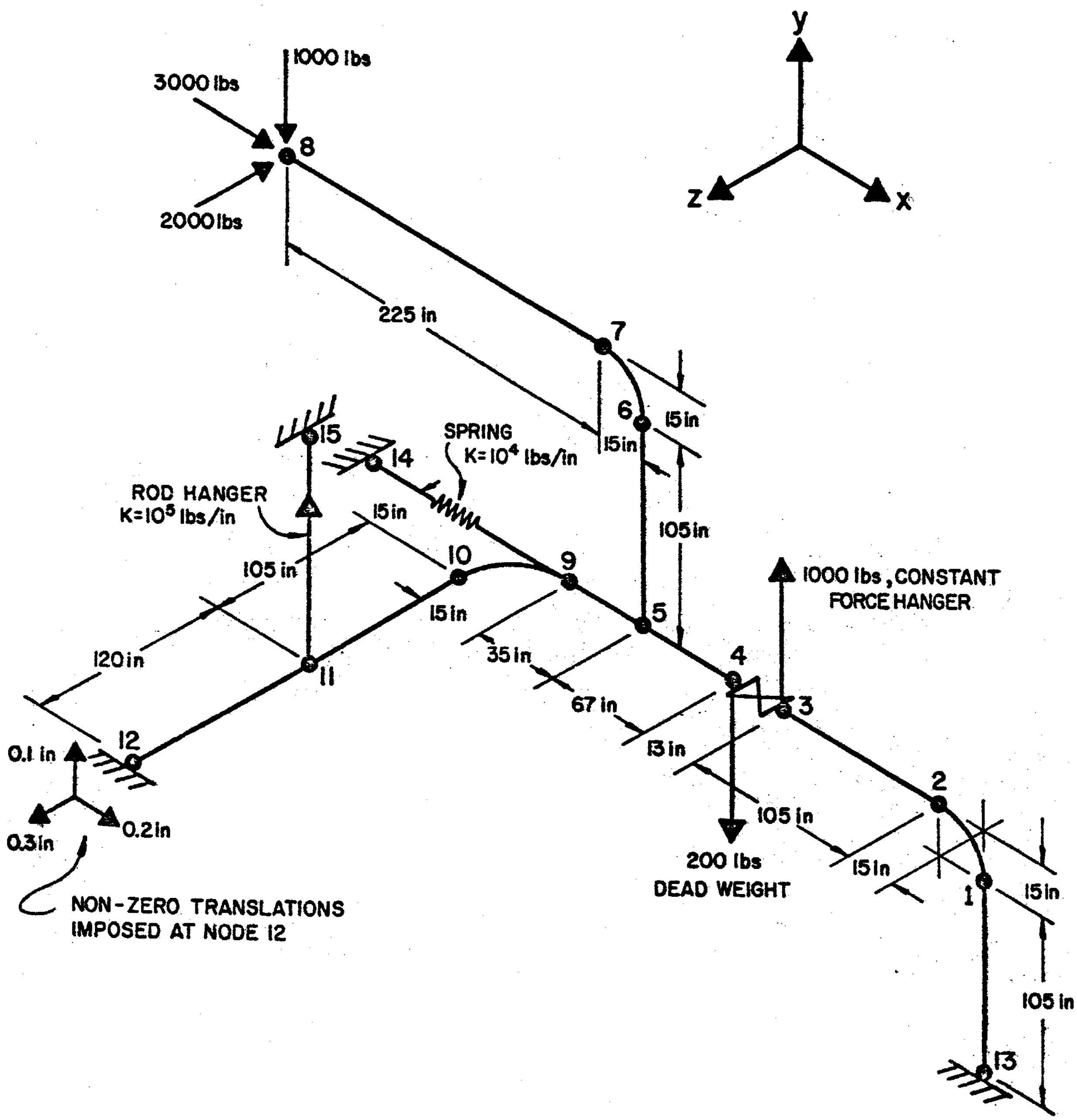

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FIGURE C-71 MODEL OF PIPE NETWORK FOR SLSAP AND SAPIV (SLSAP VALIDATION PROBLEM 1)

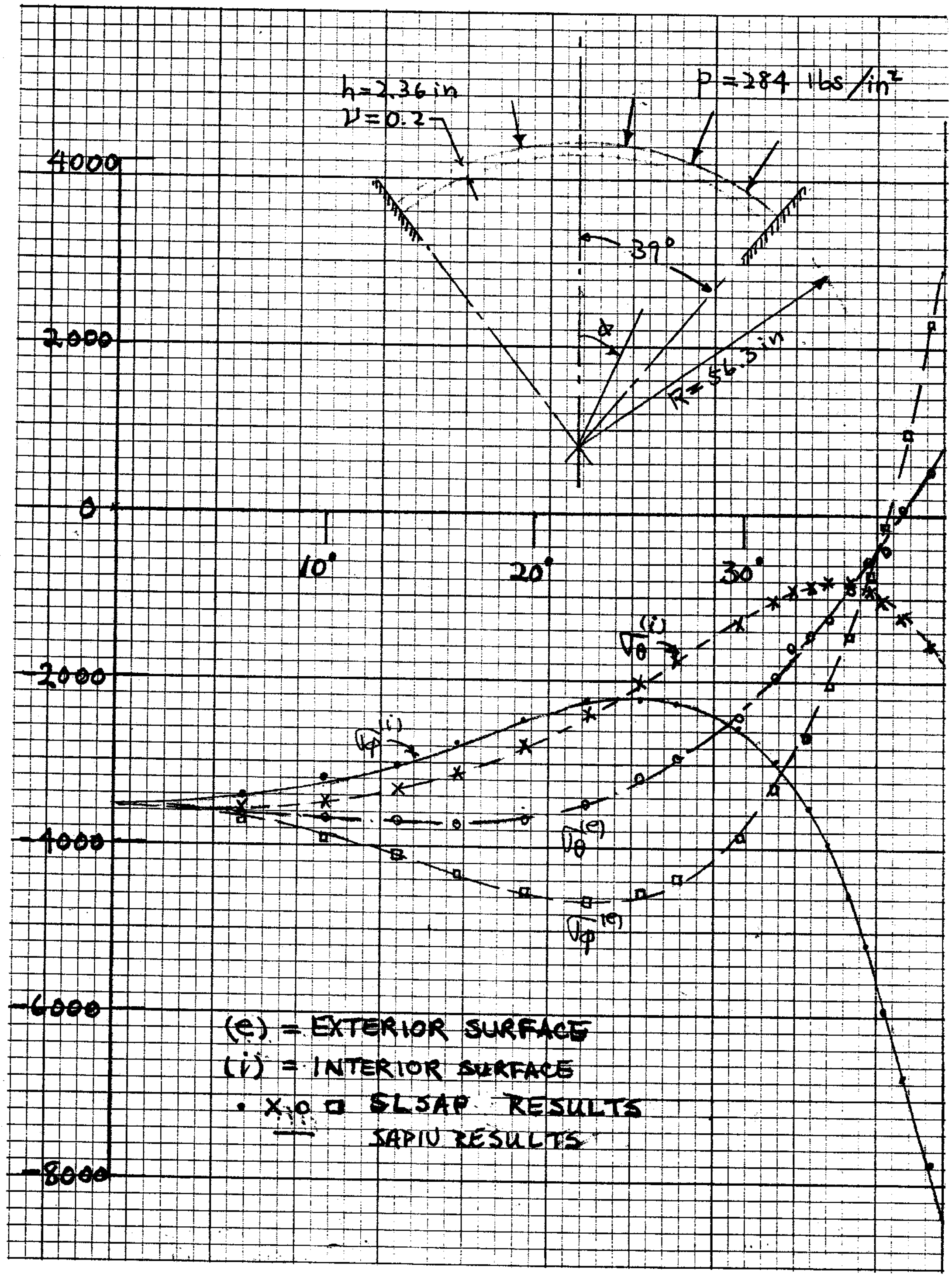

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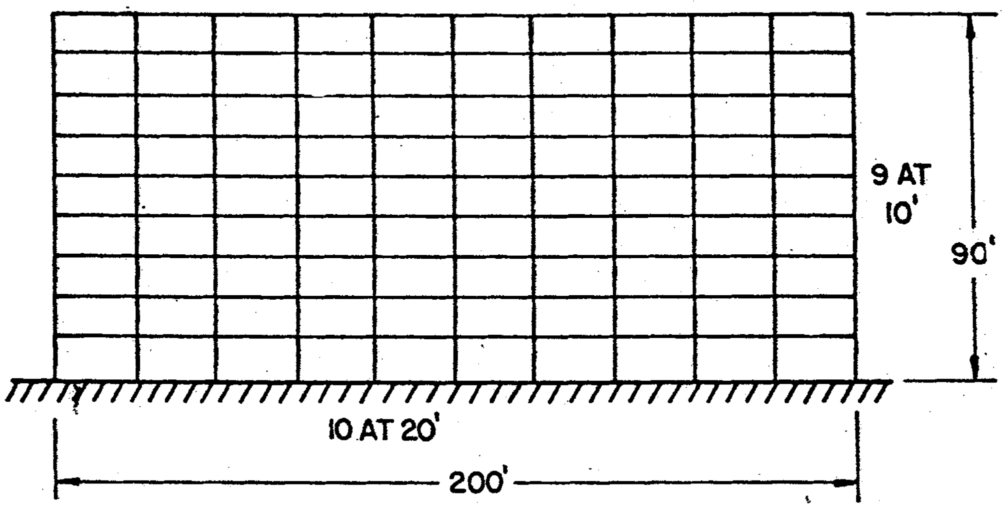
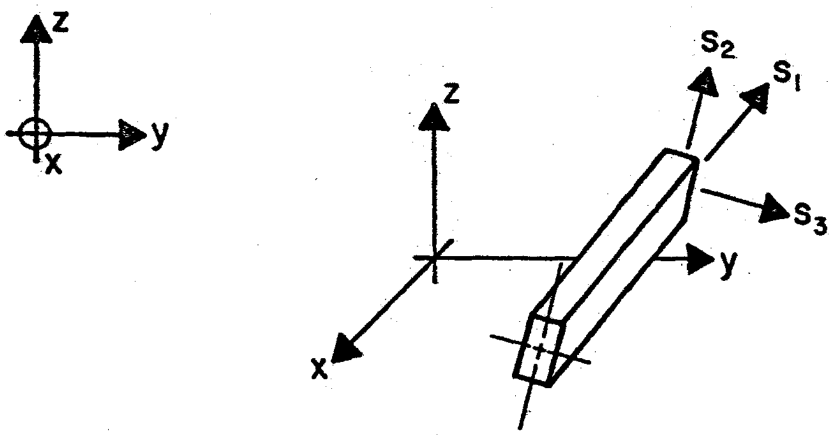
FIGURE C-72

COMPARISON OF SURFACE STRESSES IN A CLAMPED SPHERICAL SHELL UNDER EXTERNAL PRESSURE FOR SLSAP AND SAPIV (SLSAP VALIDATION PROBLEM 2)



(a) ELEVATION OF FRAME

DATA : YOUNG'S MODULUS = 432000, MASS DENSITY = 1.0
FOR ALL BEAMS AND COLUMNS $A_1 = 3.0$, $I_1 = I_2 = I_3 = 1.0$
UNITS : FT, KIPS



(b) BEAM ELEMENT DEFINITION

S_1, S_2 AND S_3 = BEAM LOCAL AXES
I_1, I_2 AND I_3 = FLEXURAL INERTIA ABOUT S_1, S_2, AND S_3
A_1 = AREA ASSOCIATED WITH S_1

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FIGURE C-73

MODEL OF PLANE FRAME FOR SLSAP AND SAPIV (SLSAP VALIDATION PROBLEM 3)

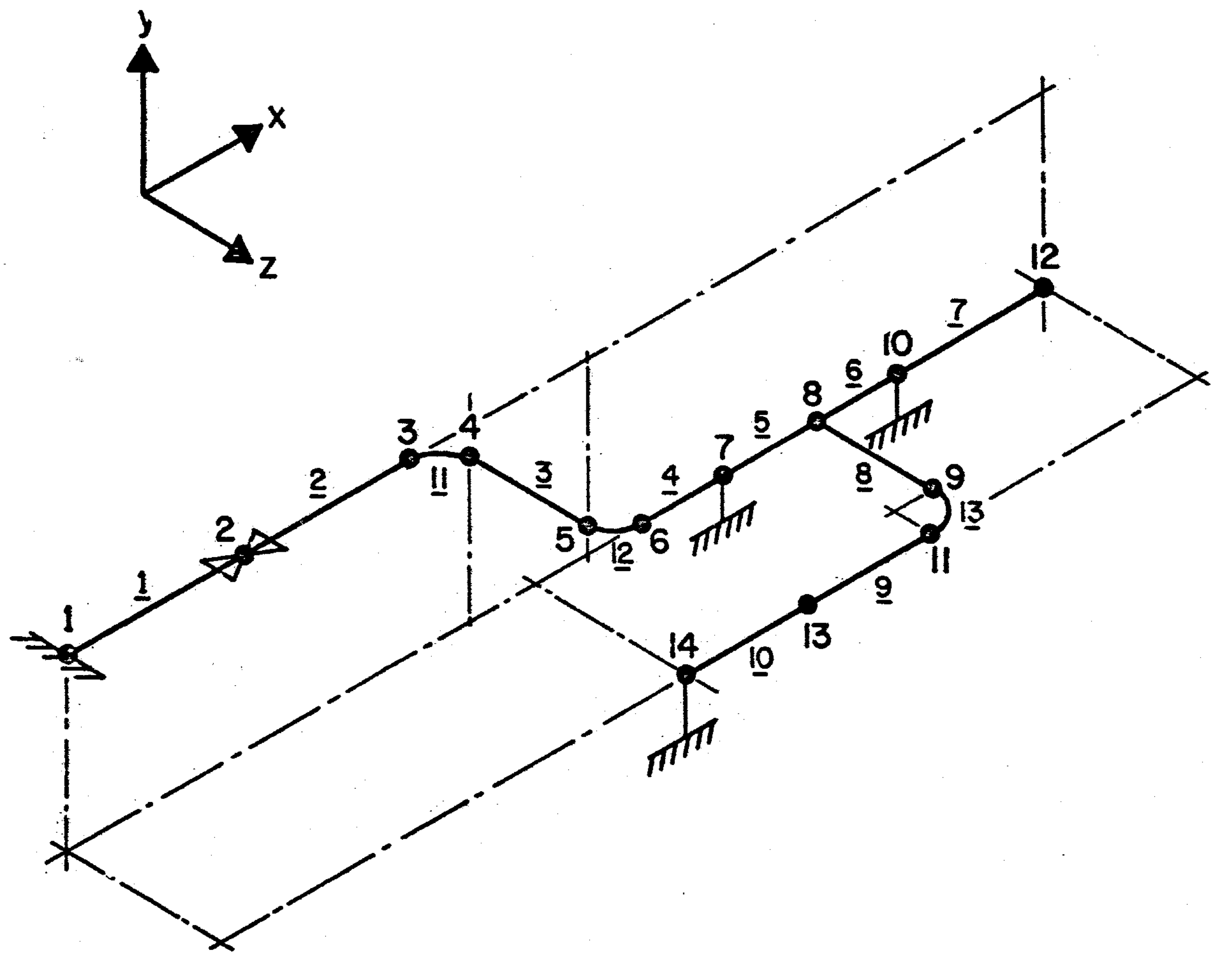

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FIGURE C-74

MODEL OF PIPE ASSEMBLAGE FOR SLSAP AND SAPIV (SLSAP VALIDATION PROBLEM 4)

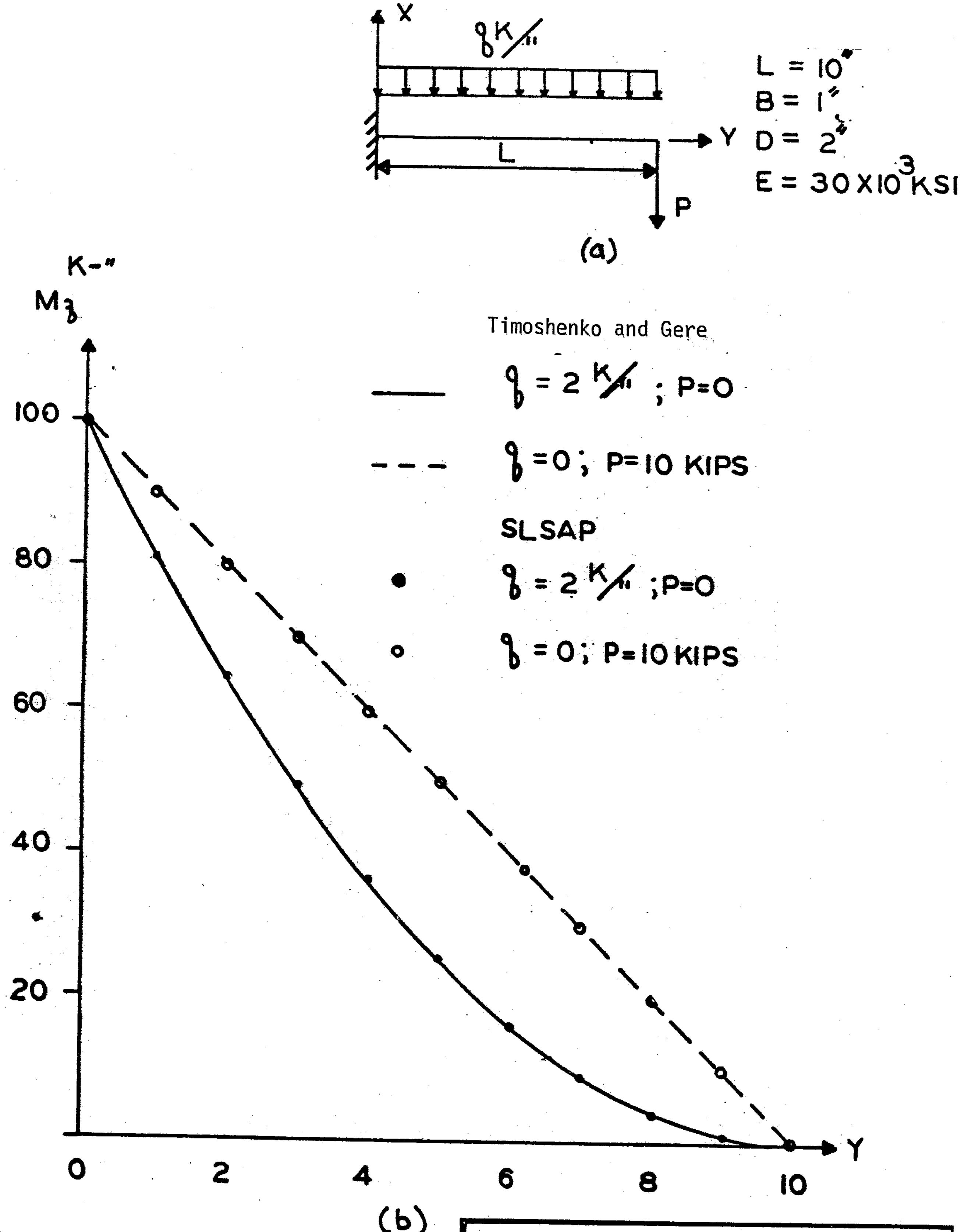

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FIGURE C-75

COMPARISON OF BENDING MOMENTS IN A CANTILEVER BEAM FOR SLSAP AND REFERENCE 43 (SLSAP VALIDATION PROBLEM 5)

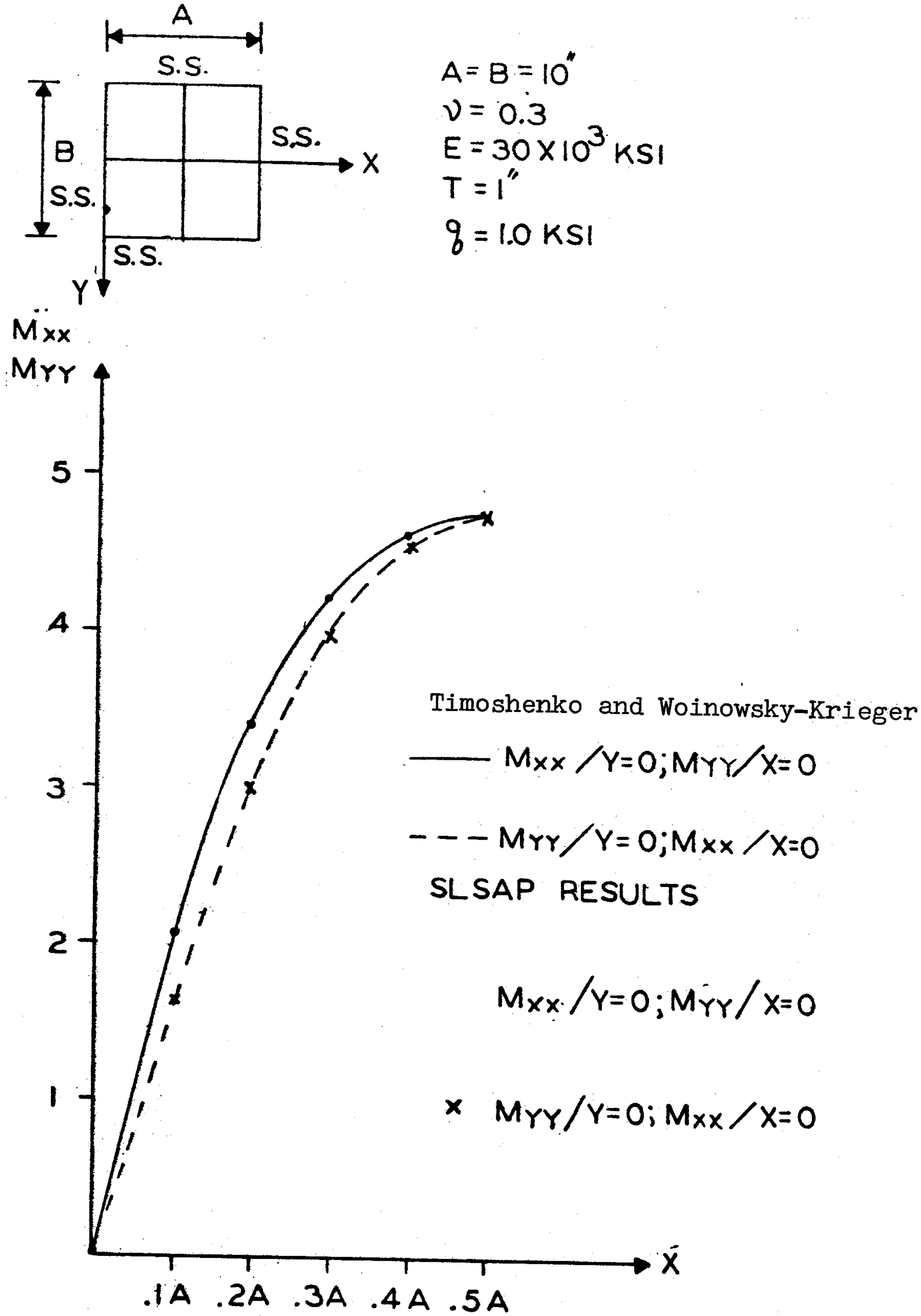

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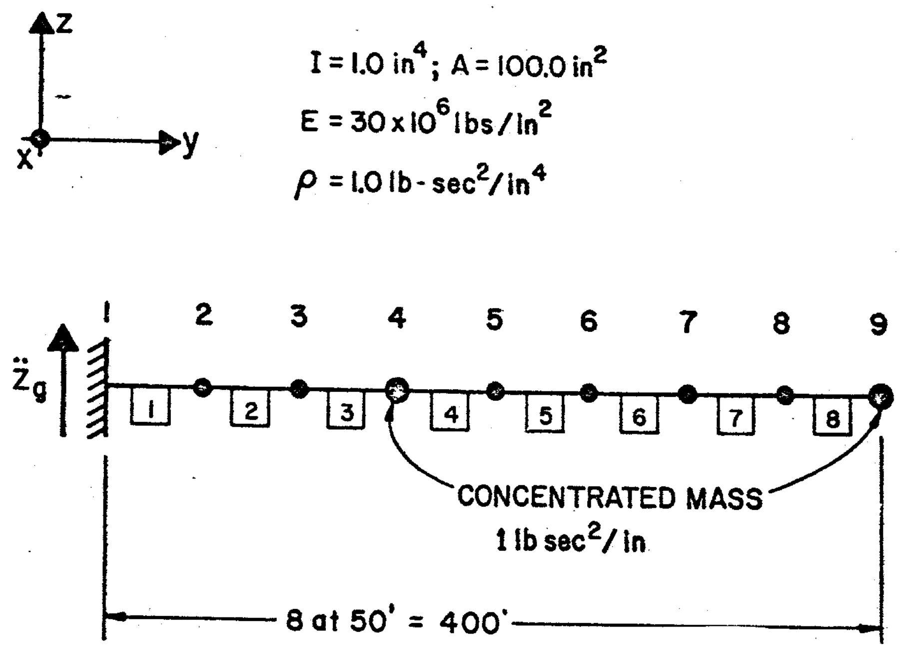
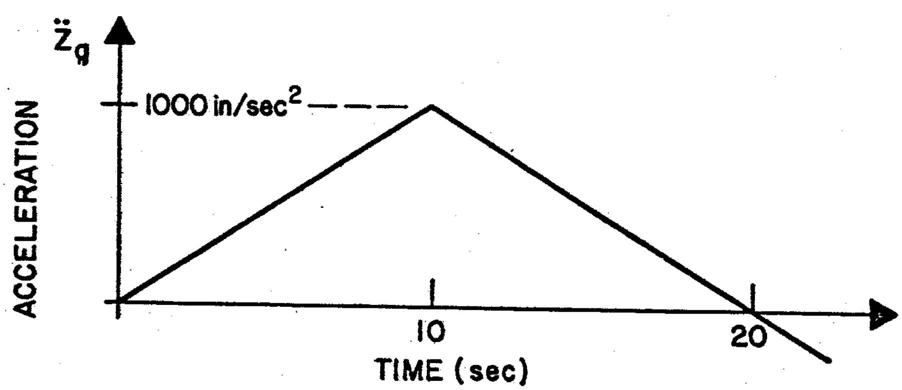
FIGURE C-76

COMPARISON OF BENDING MOMENTS IN A SIMPLY SUPPORTED PLATE FOR SLSAP AND REFERENCE 5.44 (SLSAP VALIDATION PROBLEM 6)



(a) NODE AND BEAM NUMBER ASSIGNMENTS FOR THE CANTILEVER MODEL



(b) GROUND ACCELERATION APPLIED AT NODE 1

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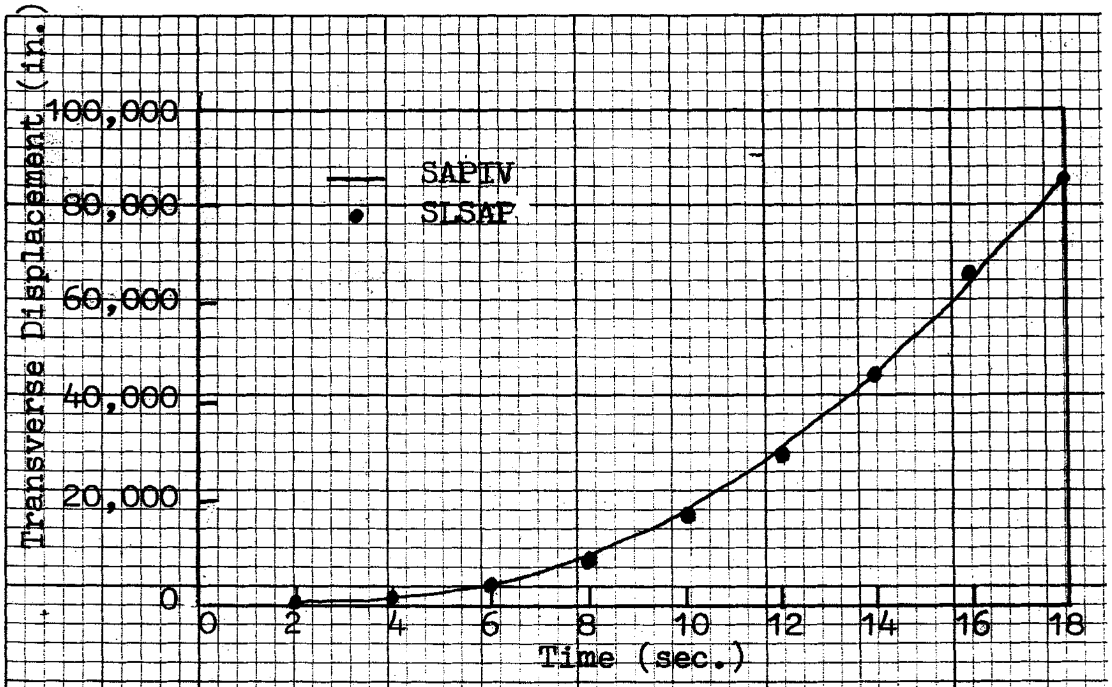
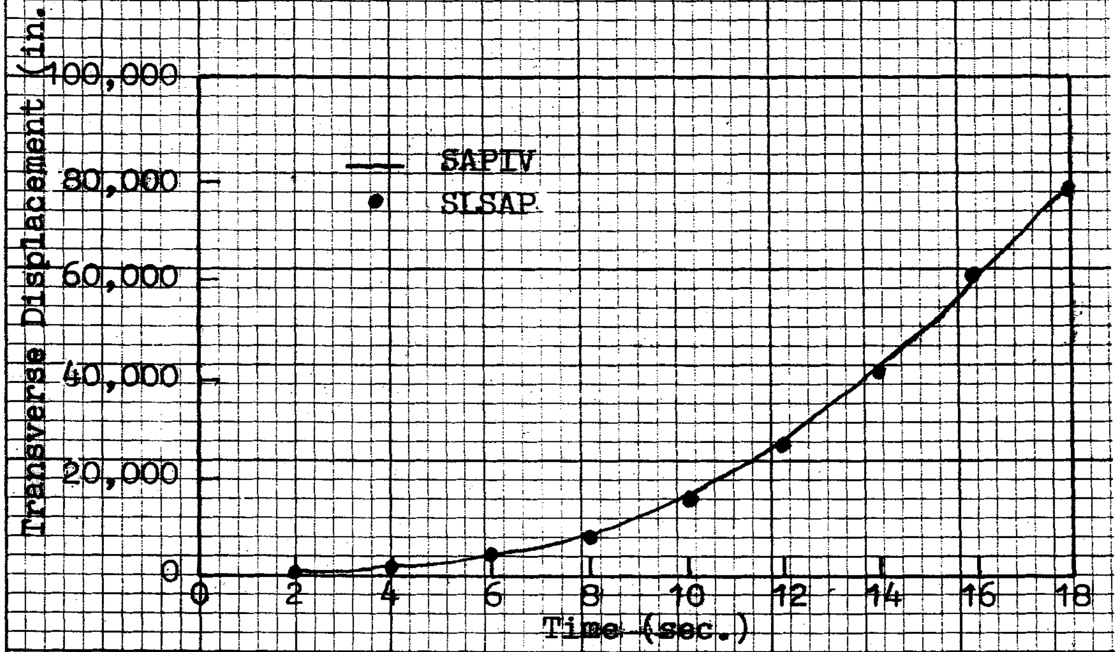
FIGURE C-77 MODEL FOR RESPONSE HISTORY ANALYSIS FOR SLSAP AND SAPIV (SLSAP VALIDATION PROBLEM 7)



(a) Node 5



(b) Node 9

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FIGURE C-78

COMPARISON OF SLSAP AND SAPIV TRANSVERSE DEFLECTIONS OF THE CANTILEVER BEAM (SLSAP VALIDATION PROBLEM 7)

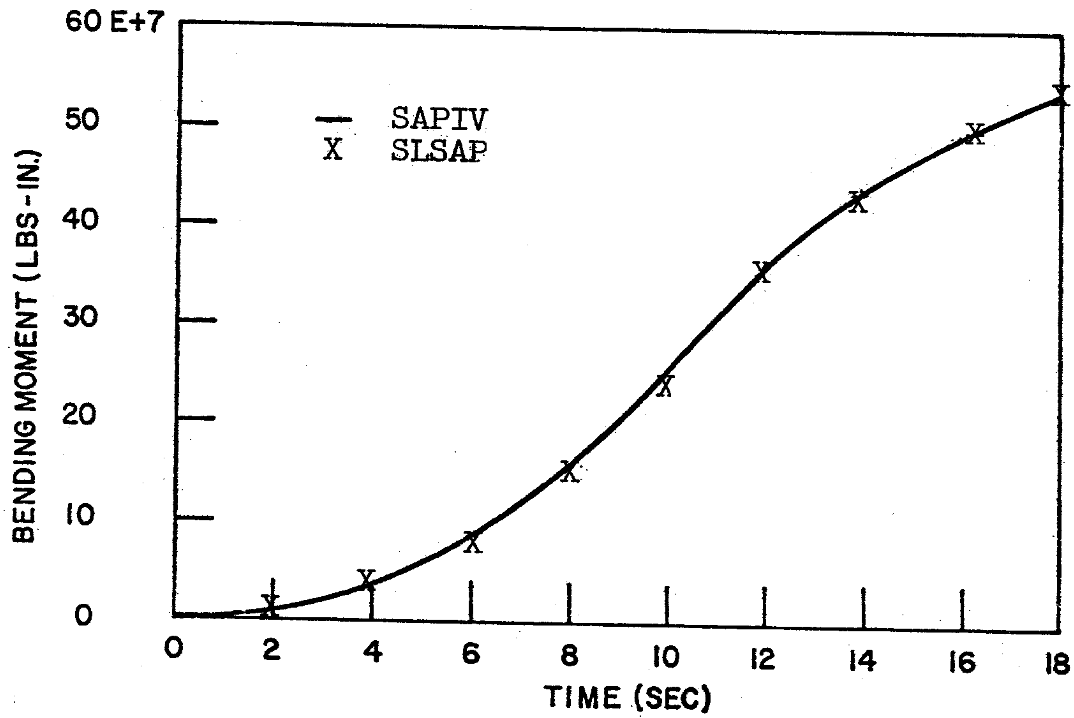

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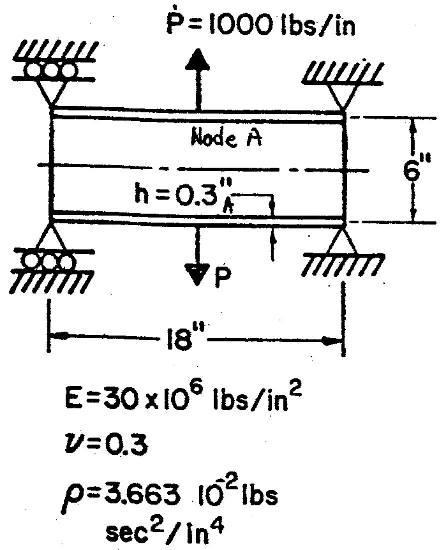
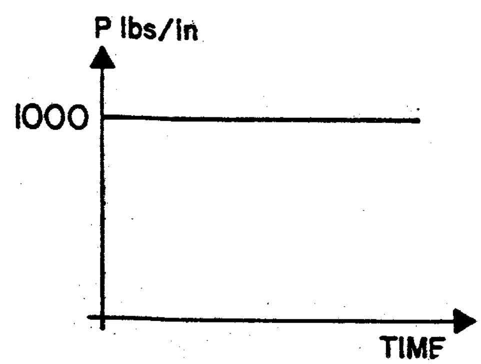
FIGURE C-79

COMPARISON OF SLSAP AND SAPIV BENDING MOMENTS FOR THE CANTILEVER BEAM (SLSAP VALIDATION PROBLEM 7)



a) CYLINDRICAL TUBE



b) TIME VARIATION OF LOAD

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FIGURE C-80

CYLINDRICAL TUBE AND LOAD HISTORY FOR SLSAP AND SAPIV MODE SUPERPOSITION AND DIRECT INTEGRATION ANALYSES (SLSAP VALIDATION PROBLEM 8)

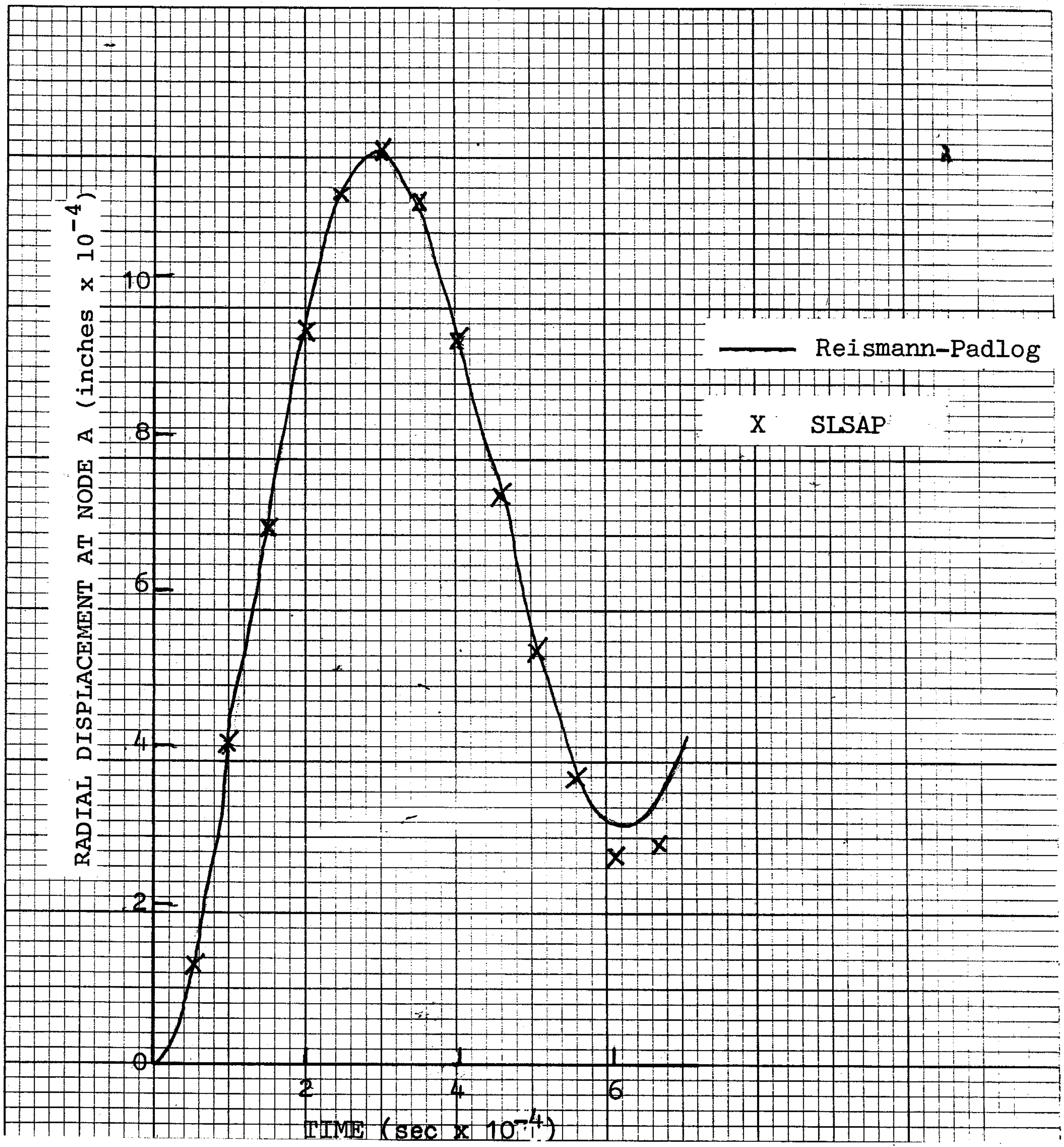

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FIGURE C-81

DISPLACEMENT COMPARISON OF SLSAP MODE SUPERPOSITION AND REFERENCE 45 FOR THE CYLINDRICAL TUBE (SLSAP VALIDATION PROBLEM 8)

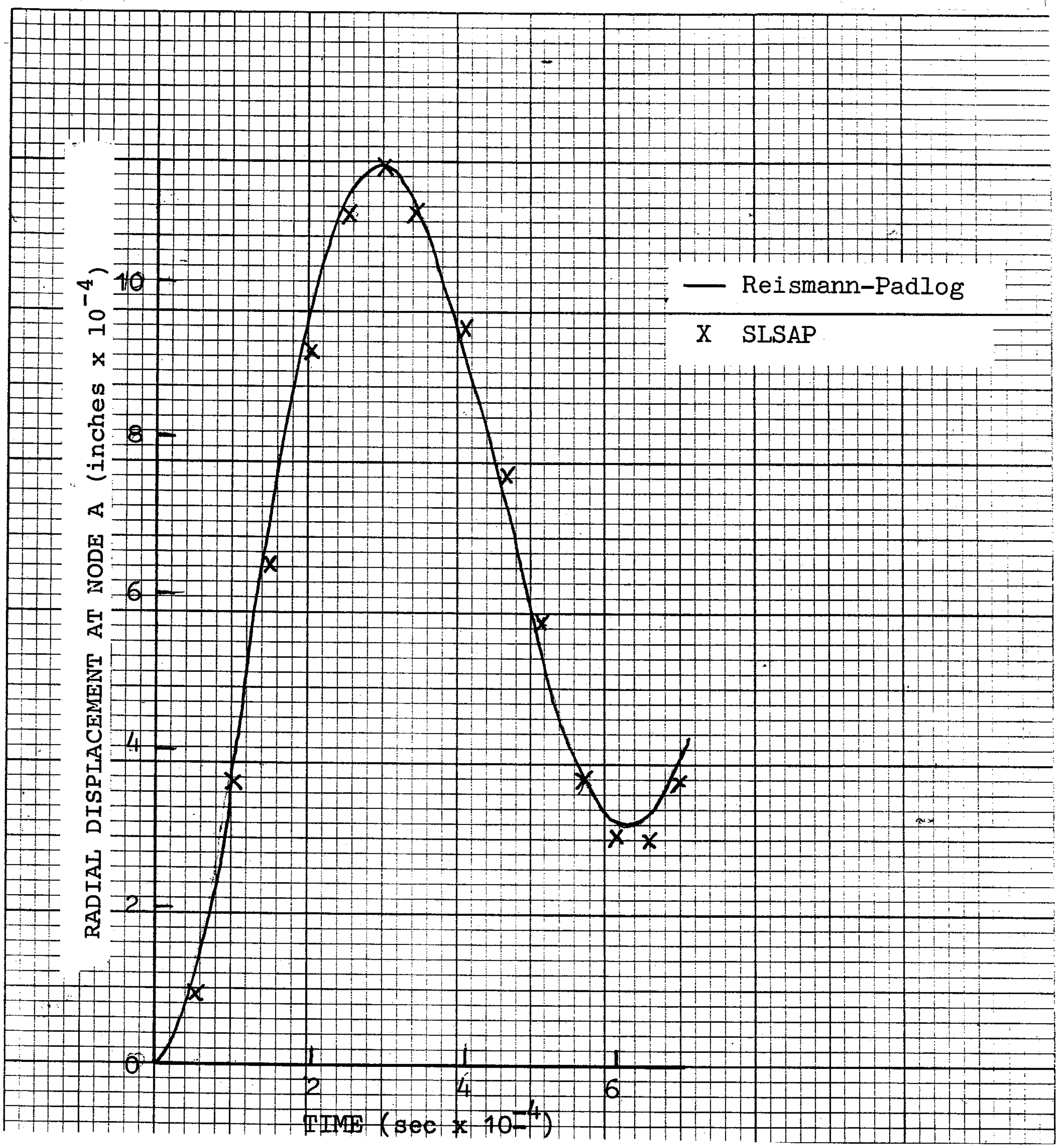

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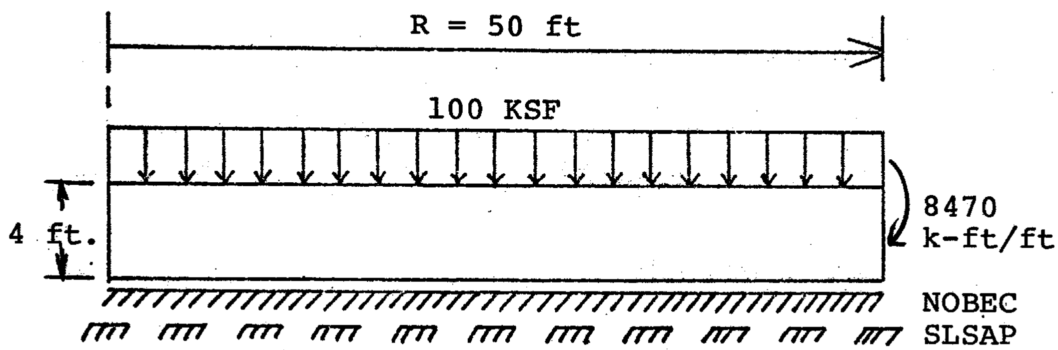
FIGURE C-82

DISPLACEMENT COMPARISON OF SLSAP DIRECT INTEGRATION AND REFERENCE 45 FOR THE CYLINDRICAL TUBE (SLSAP VALIDATION PROBLEM 8)



$E = 593141.8$ kips/ft^2

$\nu = 0.205$

$\rho = 0.15$ kips sec^2/ft^4

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FIGURE C-83

CIRCULAR PLATE ON A RIGID FOUNDATION FOR SLSAP AND NOBEC (SLSAP VALIDATION PROBLEM 9)

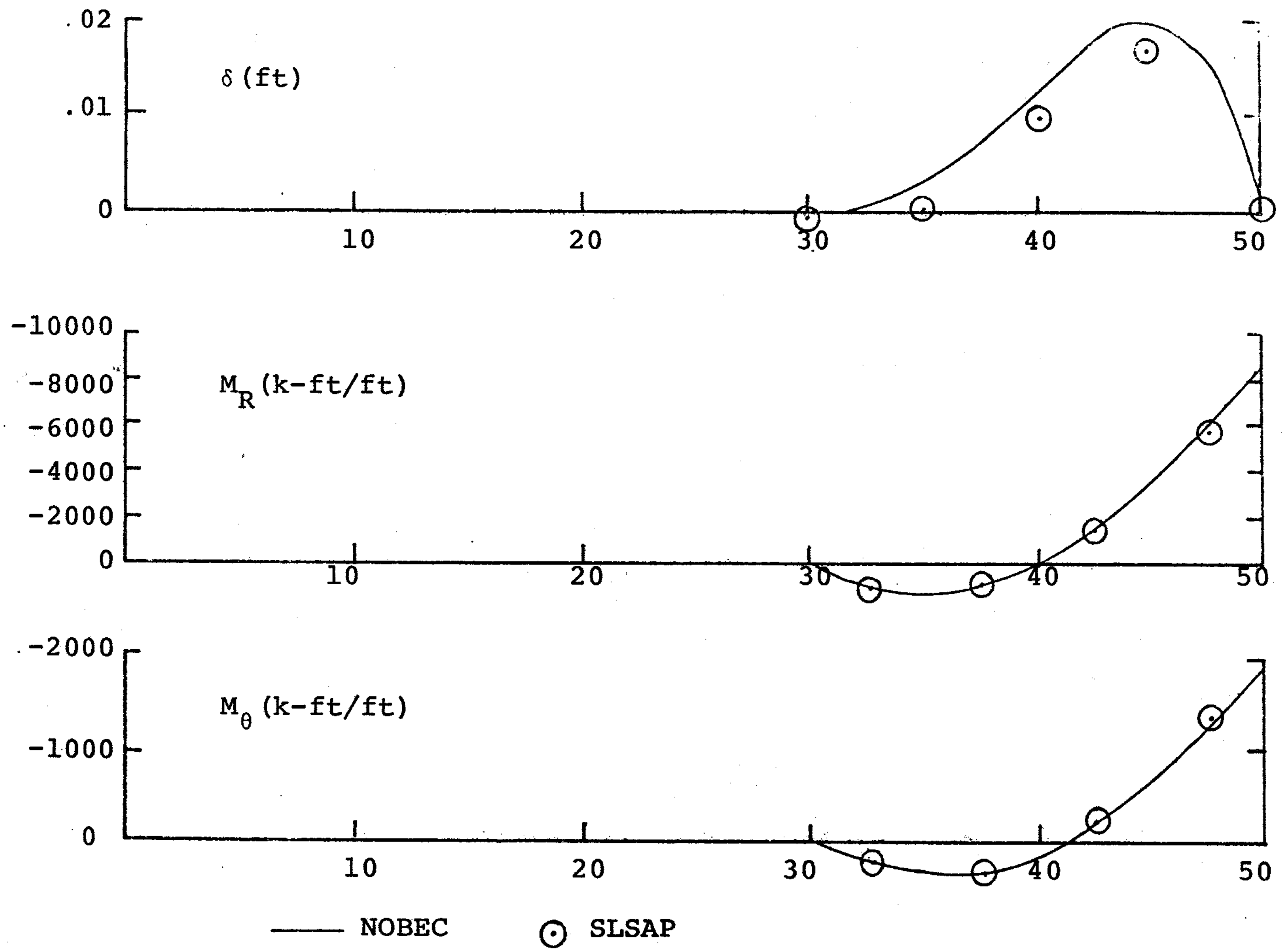

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FIGURE C-84 COMPARISON OF DISPLACEMENT AND MOMENT VARIATION OF CIRCULAR PLATE FROM SLSAP AND NOBEC (SLSAP VALIDATION PROBLEM 9)

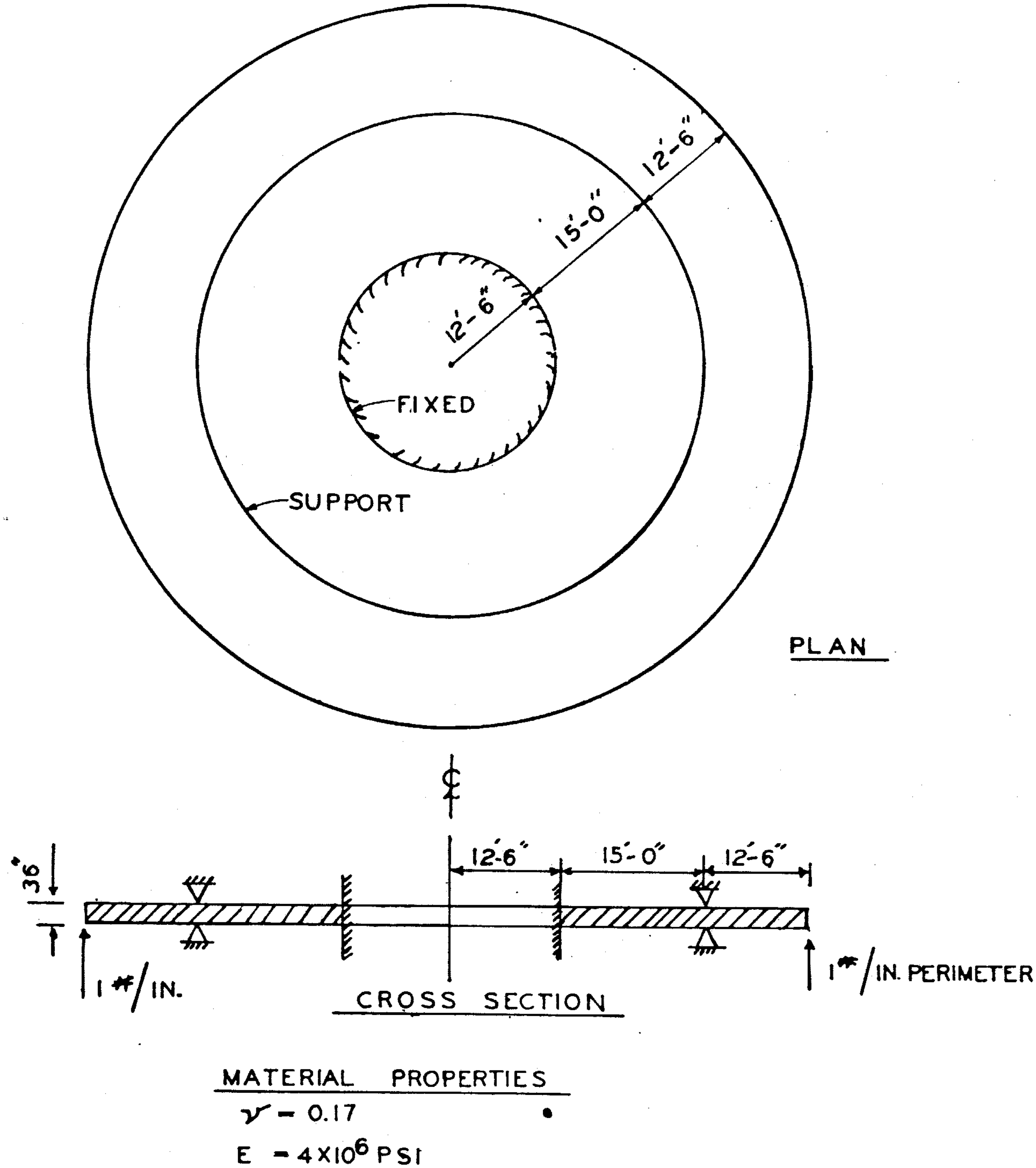

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FIGURE C-85

CIRCULAR PLATE FOR SOR-III EXAMPLE

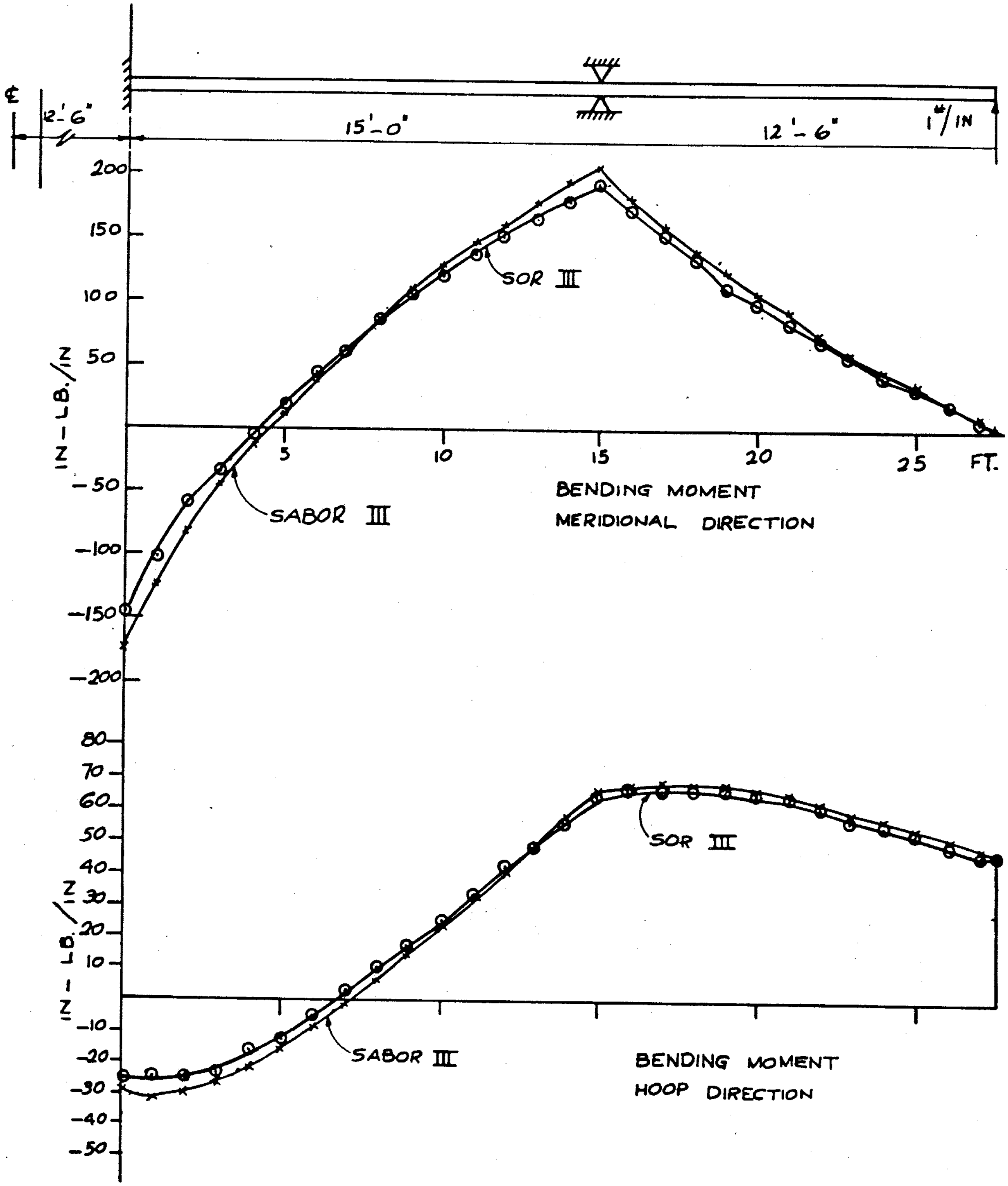

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FIGURE C-86

MOMENT COMPARISON SABOR-III AND SOR-III

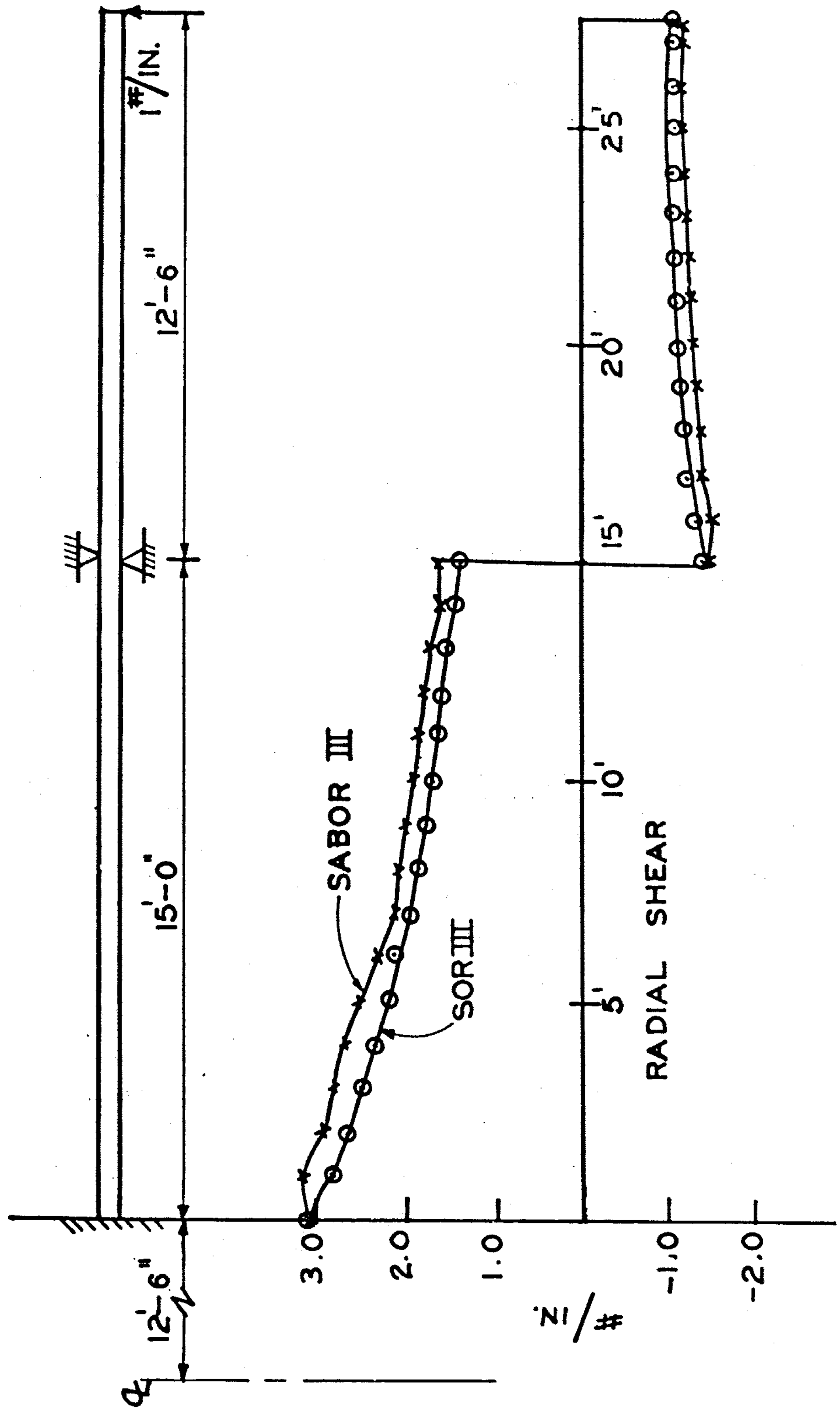

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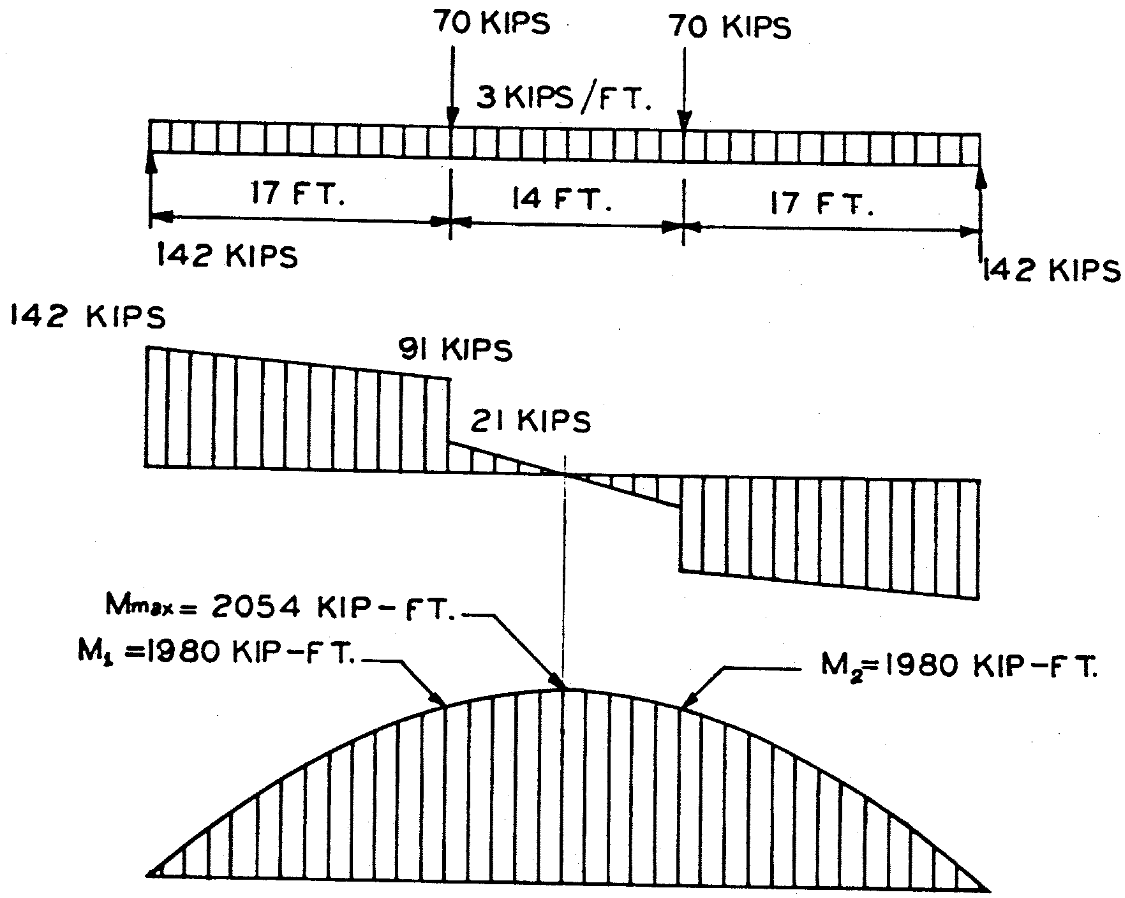
FIGURE C-87

RADIAL SHEAR COMPARISON FOR
SABOR-III AND SOR-III



SHEAR AND MOMENT DIAGRAMS

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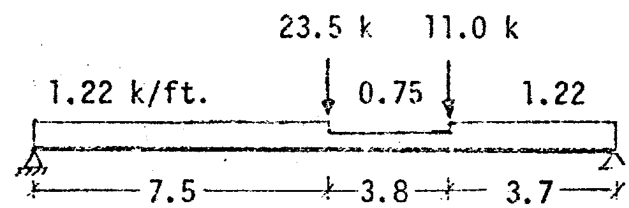
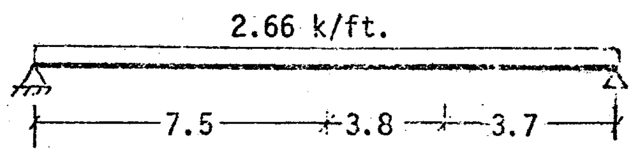
FIGURE C-88

LOADS ON BEAM (STAND VALIDATION PROBLEM 4)



Basic Loading 1



Basic Loading 2

Beam No. 2, The Compression flange is continuously supported.

FIGURE C-89

TRANSVERSE LOADS ON BEAM
(STAND VALIDATION PROBLEM 5)

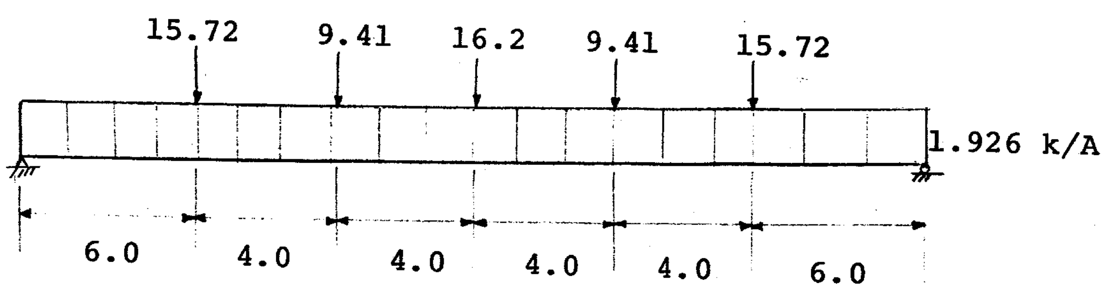


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FIGURE C-90

TRANSVERSE LOADS ON BEAM
(STAND VALIDATION PROBLEM 6)

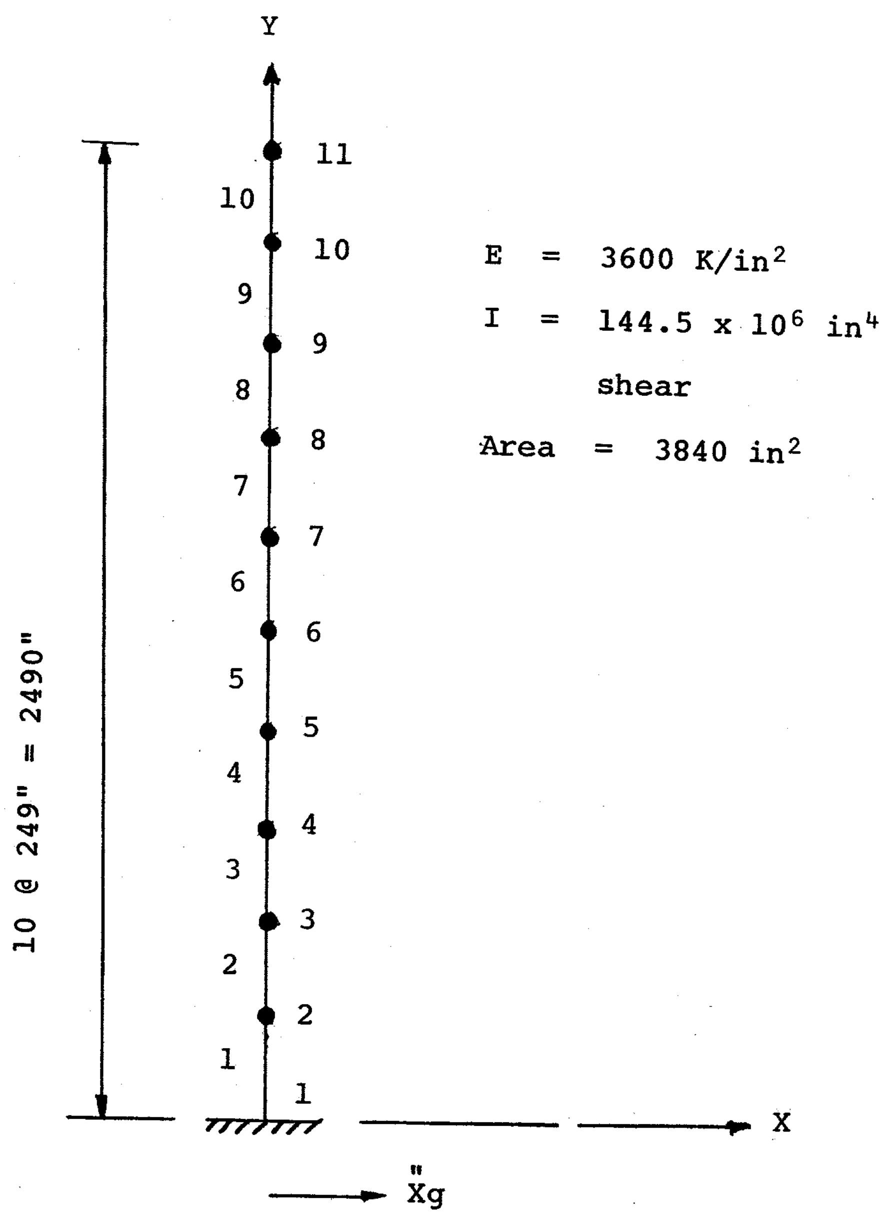

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FIGURE C-91

TEN-STORY SHEAR WALL MODEL
FOR NONLIN2 PROGRAM

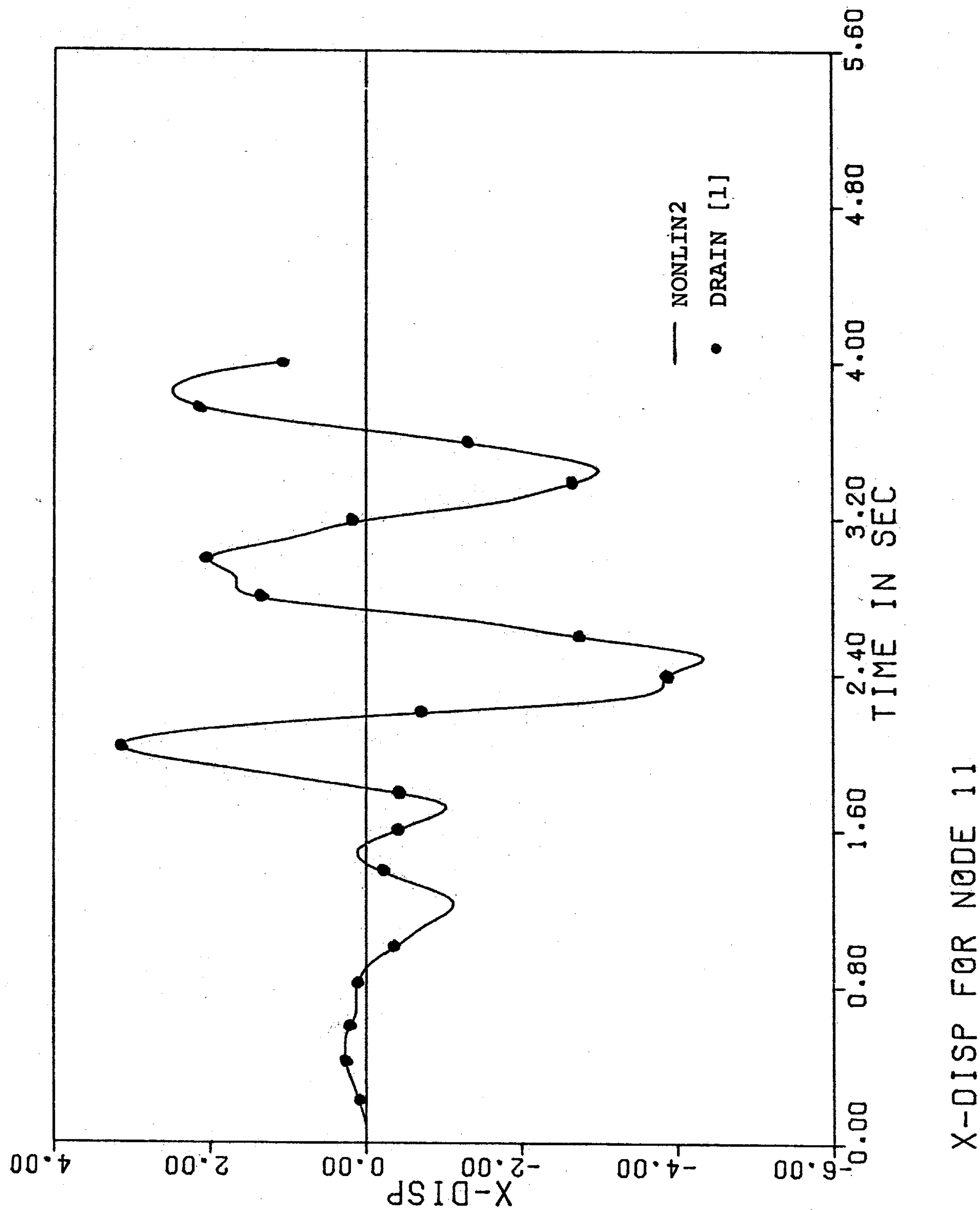

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FIGURE C-92

COMPARISON OF DISPLACEMENT FOR
NODE 11 (NONLIN2)

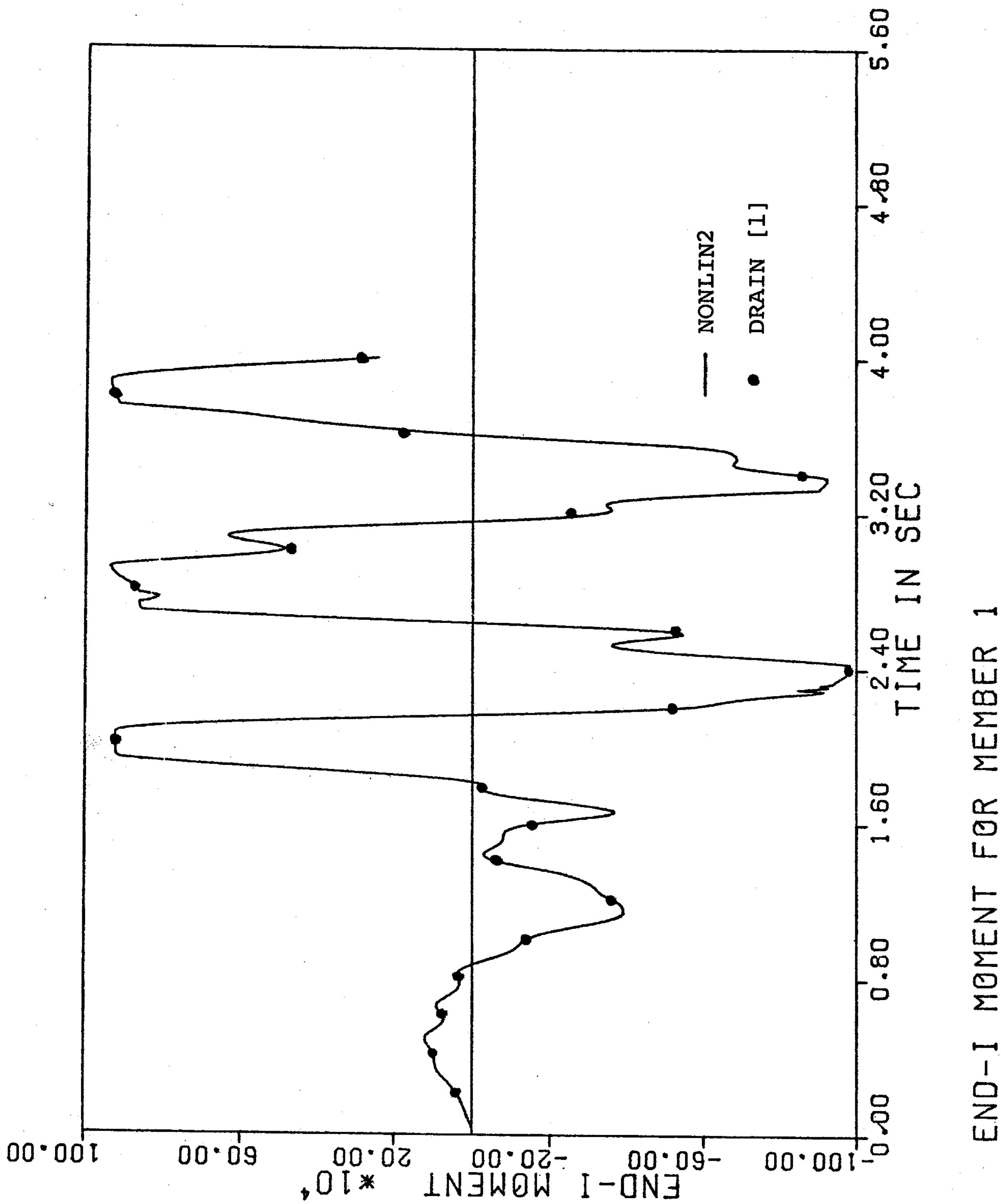

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FIGURE C-93

COMPARISON OF MOMENT FOR MEMBER 1
(NONLIN2)

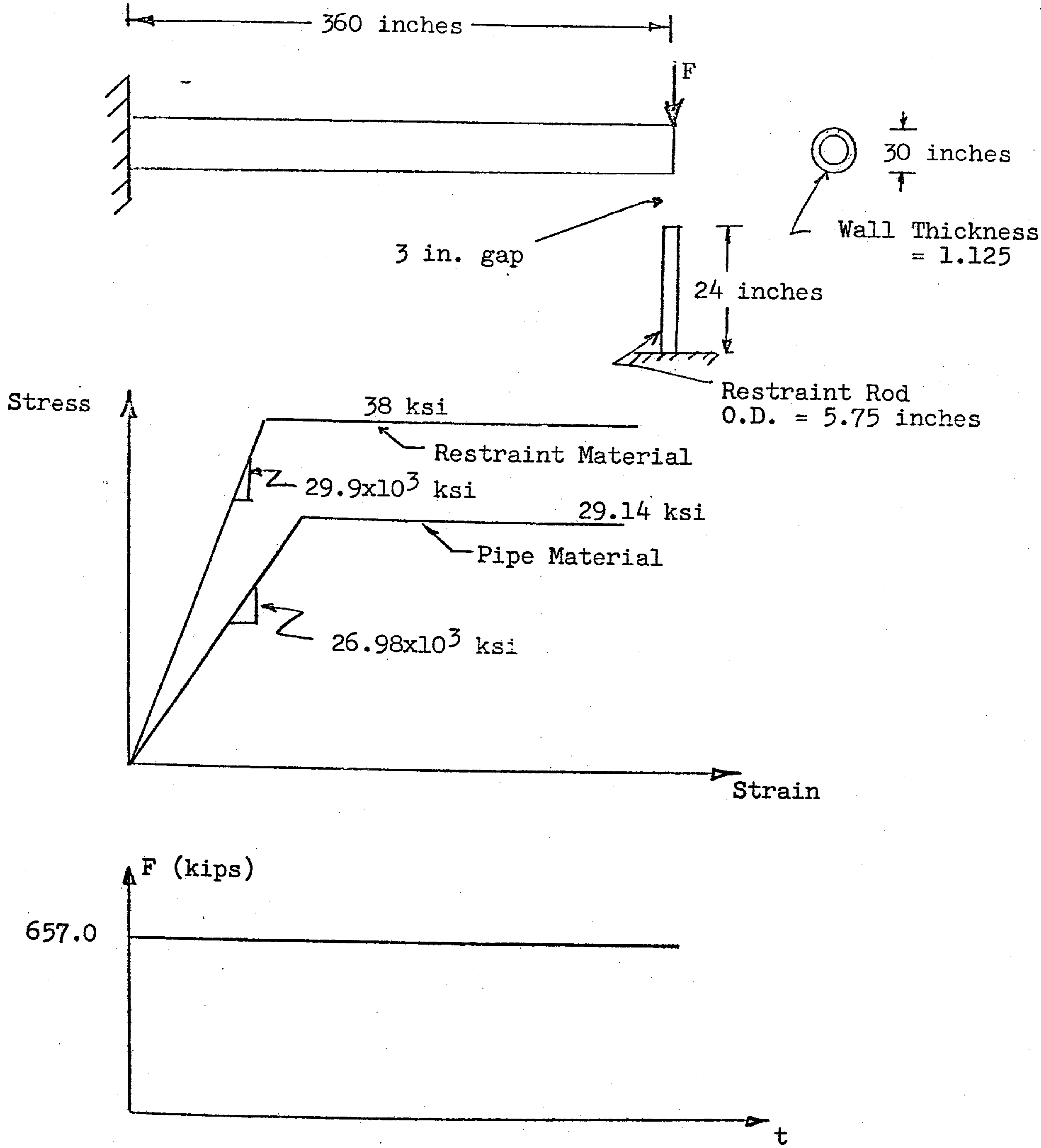

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FIGURE C-94

PIPE WHIP MODEL FOR EXAMPLE 1 OF PWRRA

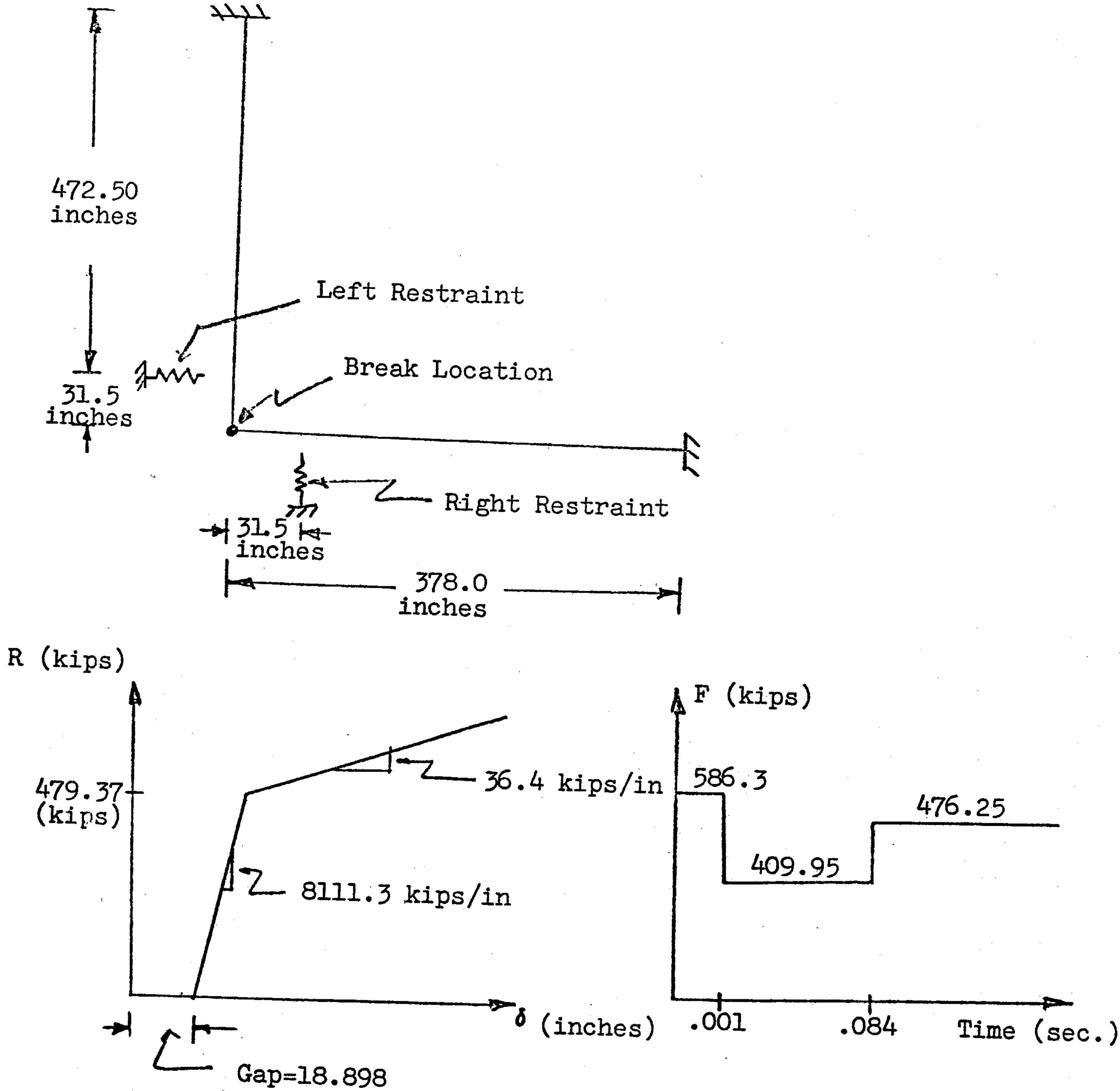

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FIGURE C-95

PIPE WHIP MODEL FOR EXAMPLE 2 OF PWRRA

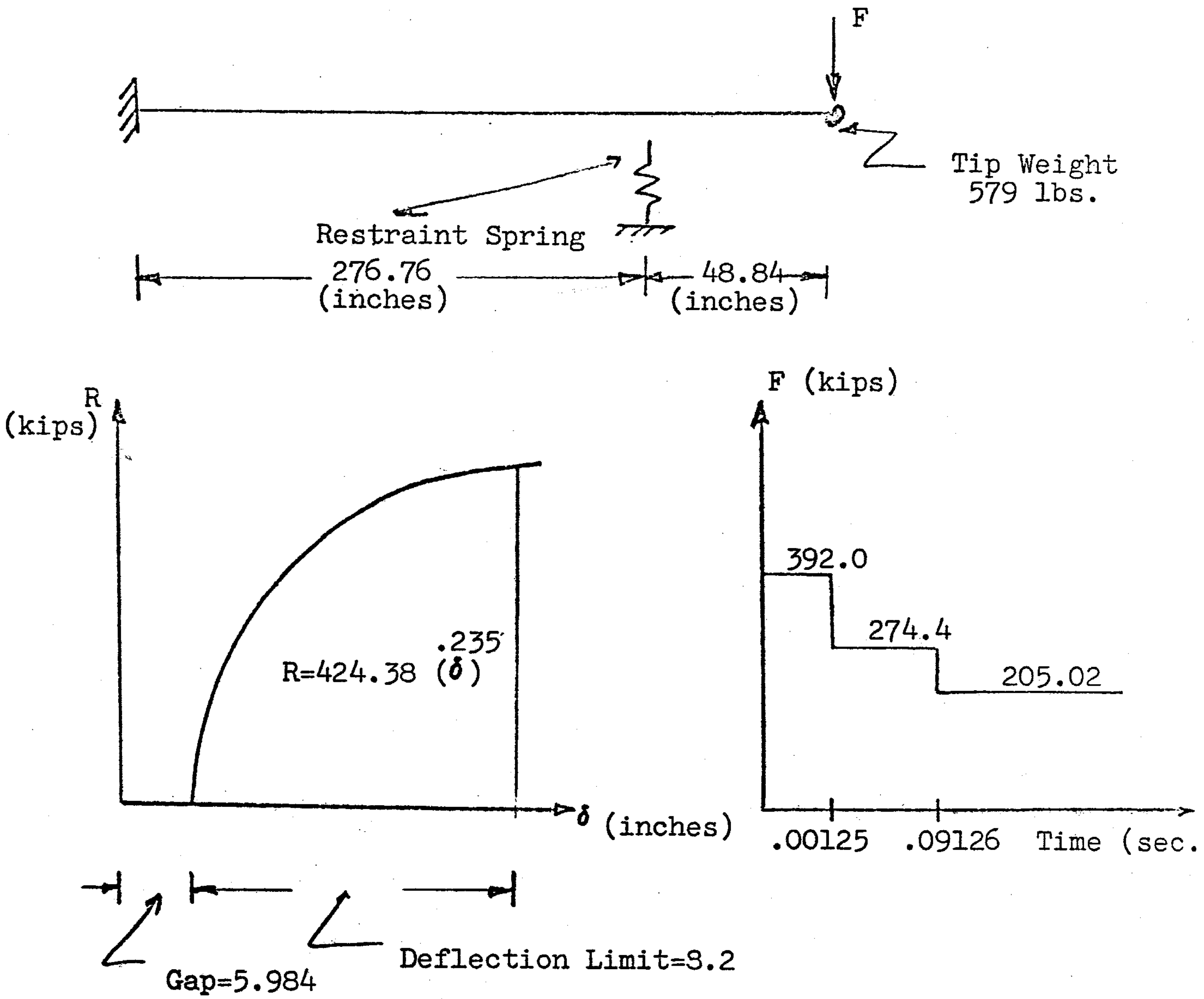

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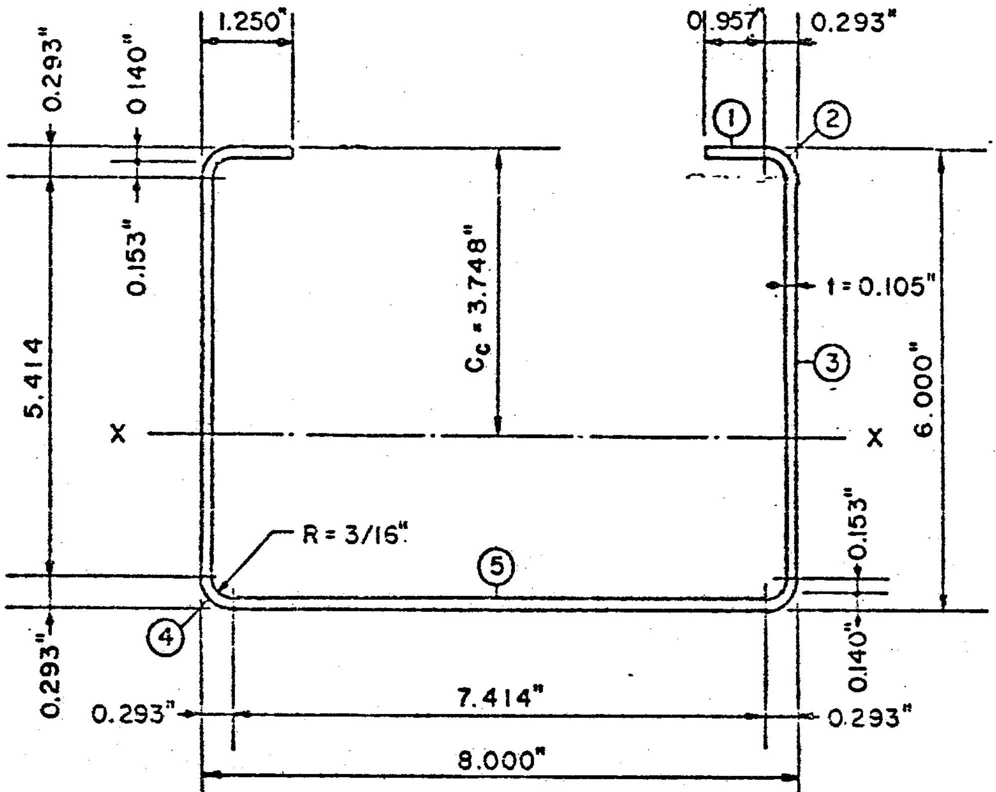
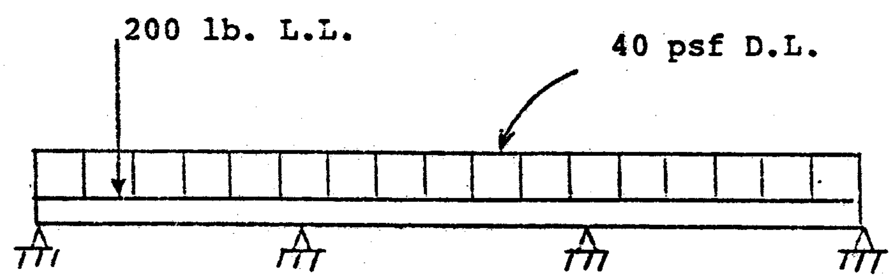
FIGURE C-96

PIPE WHIP MODEL FOR EXAMPLE 3 OF PWRRA



(a)



vertical seismic design load = 1.5g

horizontal seismic design load = 4.5g

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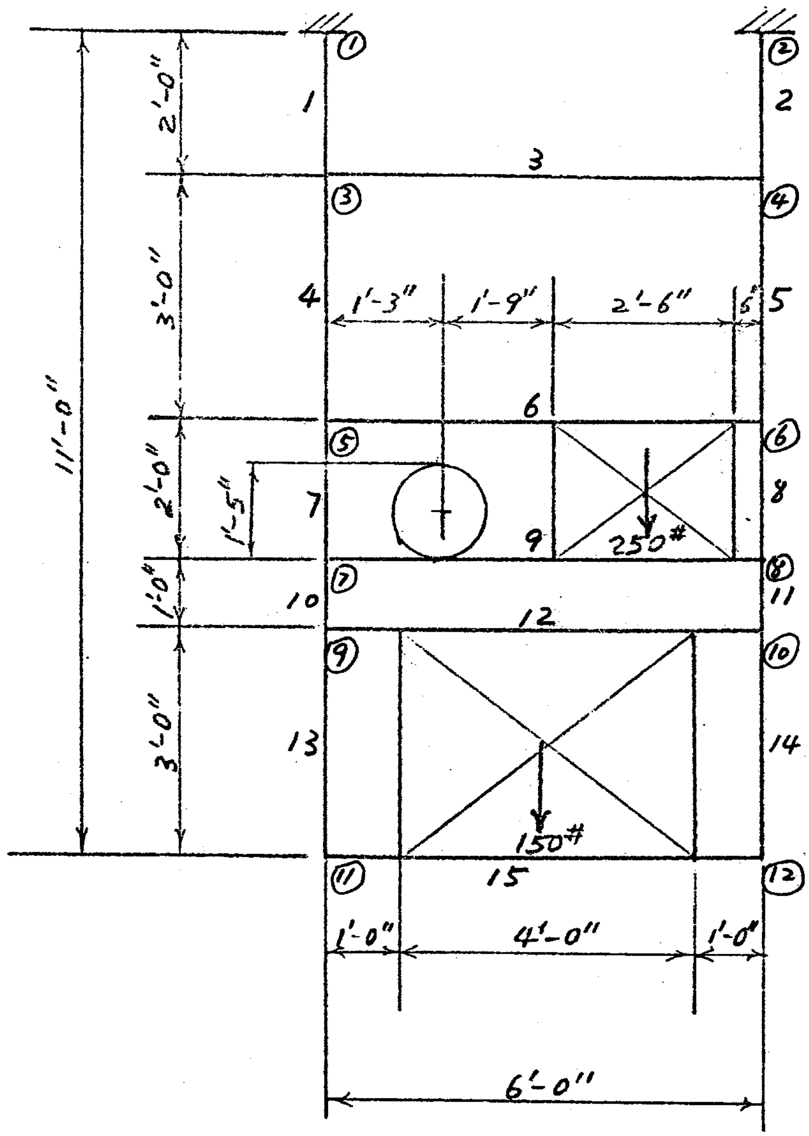
FIGURE C-97

CABLE TRAY MODEL FOR SEISHANG PROGRAM



I = 2.22 in^4

A = 2.75 in^2

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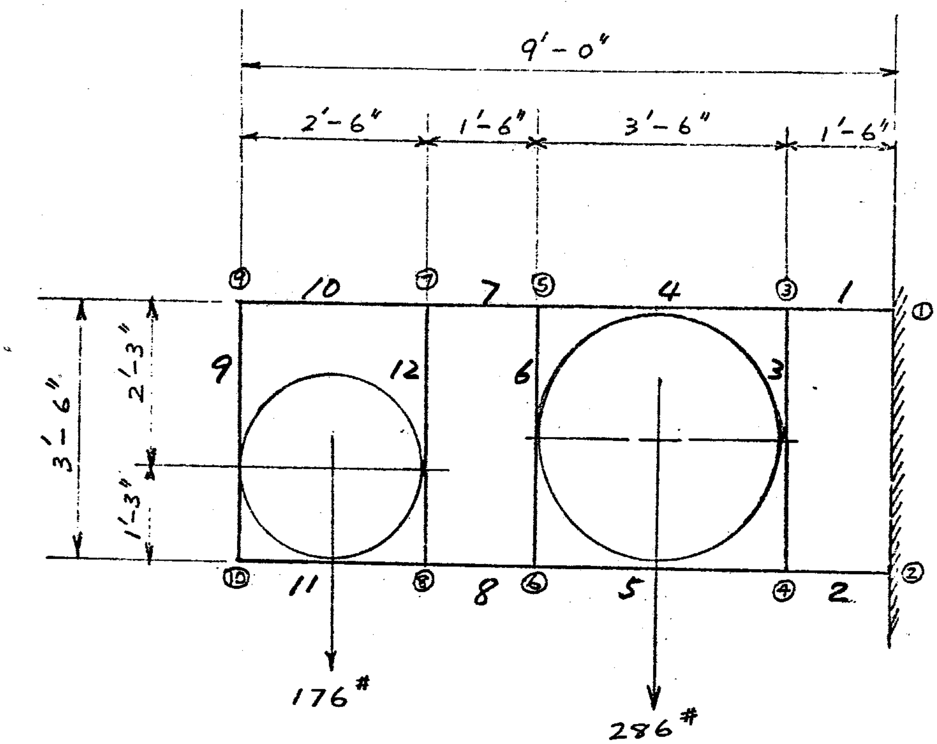
FIGURE C-98

CEILING MOUNTED SUPPORT MODEL
FOR SEISHANG PROGRAM



$I = 1.24\ in^4$

$A = 1.44\ in^2$

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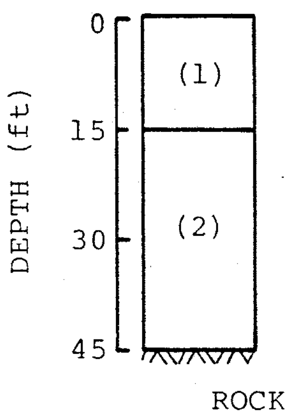
FIGURE C-99

WALL MOUNTED SUPPORT MODEL FOR SEISHANG PROGRAM

Excavated Effective Soil Pressure = 4232.8 psf



Layer	UNIT WT. (pcf)	P>Pc $\frac{C_c}{1+e_o}$	P<Pc $\frac{C_r}{1+e_o}$	Pc (psf)
(1)	85.6	0.04	0.004	20,000
(2)	81.6	0.01	0.001	20,000

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FIGURE C-100

SOIL PROFILE AND PROPERTIES FOR CONSOLIDATION SETTLEMENT COMPUTATION USING TERZAGHI'S METHOD (SETTLE VALIDATION PROBLEM 2)

Soil Profile	Young's Modulus (psf)	Poisson's Ratio
//≡//≡//≡//		
Semi-infinite Elastic body	5×10^{6}	0

//≡//≡//≡// <= NOTE: This is a soil formula

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FIGURE C101

SOIL PROFILE AND PROPERTIES FOR
ELASTIC SETTLEMENT COMPUTATION
(SETTLE VALIDATION PROBLEM 3)

LOADING AREA	LENGTH (ft)	WIDTH (ft)	LOAD (psf)	DEPTH (ft)
(1)	104.5	109.5	852.0	0.0
(2)	26.5	109.5	-1013.0	3.0
(3)	129.4	21.5	-1013.0	12.0

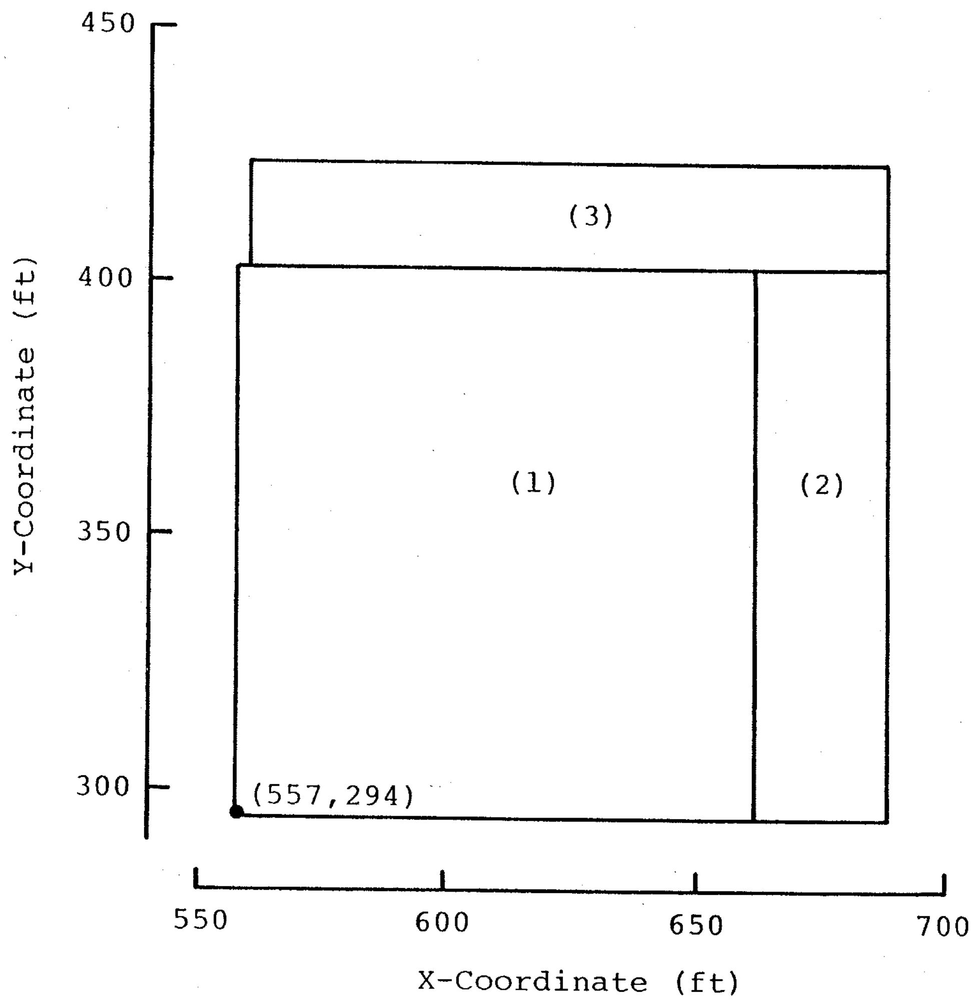

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FIGURE C-102

LOADING AREA ON SOIL FOR SETTLE VALIDATION PROBLEM 5