

CPS/USAR

APPENDIX C - COMPUTER PROGRAMS

TABLE OF CONTENTS

| | | <u>PAGE</u> |
|------|--------------------------|-------------|
| C - | <u>COMPUTER PROGRAMS</u> | C-1 |
| C.1 | CBEAM | C-2 |
| C.2 | COLOAD | C-3 |
| C.3 | CONCRETE | C-4 |
| C.4 | CSEF-III | C-5 |
| C.5 | DYNAS | C-8 |
| C.6 | DYNAX | C-10 |
| C.7 | FAST | C-13 |
| C.8 | LAFD | C-14 |
| C.9 | LUSH | C-16 |
| C.10 | MESHG | C-17 |
| C.11 | PCAUC | C-18 |
| C.12 | PIPSYS | C-20 |
| C.13 | PLFEM-II | C-22 |
| C.14 | PLGIRD | C-23 |
| C.15 | POLSAP4 | C-24 |
| C.16 | RSG | C-25 |
| C.17 | SETTLE | C-26 |
| C.18 | SHAKE | C-28 |
| C.19 | SLSAP4 | C-30 |
| C.20 | SOR-III | C-33 |
| C.21 | STAND SYSTEM | C-34 |
| C.22 | STRUDL II | C-36 |
| C.23 | TEMCO | C-37 |
| C.24 | NONLIN2 | C-39 |
| C.25 | PWRRRA | C-40 |
| C.26 | SLOPE | C-41 |
| C.27 | SEISHANG | C-42 |
| C.28 | VESLFAT | C-42 |
| C.29 | References | C-43 |

CPS/USAR

APPENDIX C - COMPUTER PROGRAMS

LIST OF TABLES

| <u>NUMBER</u> | <u>TITLE</u> | <u>PAGE</u> |
|---------------|---|-------------|
| C-1 | Span 1 Characteristics and Output Results | C-47 |
| C-2 | Span 2 Characteristics and Output Results | C-48 |
| C-3 | Span 3 Characterisitcs and Output Results | C-49 |
| C-4 | Resulting Total Load | C-50 |
| C-5 | Concrete Strength Test Tally Sheet | C-51 |
| C-6 | Calculation of Strength Test Parameters | C-54 |
| C-7 | Probable Maximum Story Shears | C-56 |
| C-8 | Structural Frequencies | C-57 |
| C-9 | Probable Maximum Story Displacements | C-58 |
| C-10 | Natural Periods for the Eight Lowest Flexural Modes | C-59 |
| C-11 | Comparison of Results in kip/ft/sec Unit | C-60 |
| C-12 | Comparison of Displacements and Forces | C-61 |
| C-13 | Velocity in the Z-Direction | C-62 |
| C-14 | Model Damping Comparison | C-63 |
| C-15 | Properties of Structural Model | C-64 |
| C-16 | Comparison of Nodal Accelerations in G Units | C-65 |
| C-17 | Comparison of Maximum Stress Resultants in K, ft Unit atElement C, $\theta = 0^\circ$ | C-66 |
| C-18 | Comparison of Nodal Accelerations in G Units | C-67 |
| C-19 | Comparison of Maximum Stress Resultants in K, ft Units at Element C, $\theta = 0^\circ$ | C-68 |
| C-20 | Displacements | C-69 |
| C-21 | Anchor Forces | C-70 |
| C-22 | Strain-Compatible Soil Properties | C-71 |
| C-23 | Comparison of Computed Soil Properties Due to Horizontal Excitation | C-72 |
| C-24 | Comparison of Stresses Due to Horizontal Excitation | C-73 |
| C-25 | Comparison of Nodal Point Accelerations Due to Horizontal and Vertical Excitations | C-74 |
| C-26 | Comparison of Moments for Selected Members | C-75 |
| C-27 | Summary of Load Sets at Girth Butt Weld With Change in Material and Wall Thickness | C-76 |
| C-28 | Six Highest Values of Stress Intensity, Girth Butt Weld With Change in Material and Wall Thickness | C-77 |
| C-29 | Summary of Calculations of Cumulative Usage Factor, Girth Butt Weld With Change in Material and Wall Thickness | C-78 |
| C-30 | Modal Frequencies (cycles/sec) | C-79 |
| C-31 | Loads on Plate Girder Configuration | C-80 |
| C-32 | Investigation of Plate Girder | C-81 |
| C-33 | Deleted | C-82 |
| C-34 | Deleted | C-82 |
| C-35 | Applied Loads for SLSAP4 Pipe Network | C-83 |
| C-36 | Force Equilibrium Reactions | C-84 |
| C-37 | Periods of Plane Frame | C-85 |

CPS/USAR

LIST OF TABLES (Cont'd)

| <u>NUMBER</u> | <u>TITLE</u> | <u>PAGE</u> |
|---------------|--|-------------|
| C-38 | Comparison of Moment | C-86 |
| C-39 | Cantilever Beam Analysis – Natural Periods for the Eight Lowest Flexural Modes | C-87 |
| C-40 | Cylindrical Tube Analysis – Selected Natural Periods | C-88 |
| C-41 | Rolled Beam Design Problem | C-89 |
| C-42 | Composite Beam Design Problem | C-90 |
| C-43 | Column Design Problem | C-91 |
| C-44 | Plate Girder Design Problem | C-92 |
| C-45 | Composite Beam With Axial Loads | C-93 |
| C-46 | Composite Beam With Vertical Seismic Loads | C-94 |
| C-47 | Input for First Three Concrete Section Analysis Problems | C-95 |
| C-48 | Results of First Three Concrete Section Analysis Problems | C-96 |
| C-49 | Input for Tensile Force and Biaxial Bending Problem | C-97 |
| C-50 | Results from Tensile Force and Biaxial Bending Problem | C-98 |
| C-51 | Input for Nonthermal and Thermal Loads Problem | C-99 |
| C-52 | Results from Nonthermal and Thermal Loads Problem | C-100 |
| C-53 | Comparison of Results for Example 1 of PWRRA | C-101 |
| C-54 | Comparison of Results for Example 2 of PWRRA | C-102 |
| C-55 | Comparison of Results for Example 3 of PWRRA | C-103 |
| C-56 | Allowable Shear, Moment and Span of Cable Tray | C-104 |
| C-57 | Response of the Ceiling Mounted Support | C-105 |
| C-58 | Response of the Wall Mounted Support | C-106 |
| C-59 | Interaction Coefficients of the Ceiling Mounted Support | C-107 |
| C-60 | Settlement for Problem 1 of SETTLE Validation | C-108 |
| C-61 | Settlement for Problem 2 of SETTLE Validation | C-109 |
| C-62 | Settlement for Problem 3 of SETTLE Validation | C-110 |
| C-63 | Settlement of Rectangular Rigid Mat Foundation for Problem 4 of SETTLE Validation | C-111 |
| C-64 | Stress for Problem 5 of SETTLE Validation | C-112 |

CPS/USAR

APPENDIX C - COMPUTER PROGRAMS

LIST OF FIGURES

| <u>NUMBER</u> | <u>TITLE</u> |
|---------------|---|
| C-1 | Plan, Elevation and Loading for COLOAD Validation Problem |
| C-2 | Consolidated Results for Different Locations (CONCRETE) |
| C-3 | Comparison of Deflection of a Circular Plate Due to Uniform Pressure and Axisymmetric Edge Moment (CSEF-III vs. Hand Calculation) |
| C-4 | Radial Moment Due to Uniform Pressure and Axisymmetric Edge Moment (CSEF-III vs. Hand Calculation) |
| C-5 | Simply Supported Circular Plate, Linearly Varying Pressure Load for Radial Moment |
| C-6 | Simply Supported Circular Plate, Linearly Varying Pressure Load for Deflection (CSEF-III vs. KALSHEL) |
| C-7 | Three-Story Shear Building Model for DYNAS |
| C-8 | Response History Analysis of Cantilever Beam for DYNAS Validation Problem No. 2 |
| C-9 | Comparison of Cantilever Responses from DYNAS and SAPIV (DYNAS) |
| C-10 | Steel Frame with Rigid Girders for DYNAS Validation Problem No. 3 |
| C-11 | Comparison of Modal Components of Top-Story Distortion from DYNAS and Reference 6 |
| C-12 | 2-D Cantilever Models for DYNAS Validation Problem No. 4 |
| C-13 | Shallow Spherical Shell Analyzed by DYNAX - Validation Problem 1 |
| C-14 | Axial Displacement Shallow Spherical Shell |
| C-15 | Meridional Moment Shallow Spherical Shell |
| C-16 | Finite Element Idealization of Thick-Walled Cylinder for DYNAX Validation Problem 2 |
| C-17 | Comparison of Stresses and Displacements in Thick-Walled Cylinders From DYNAX and Reference 10 |
| C-18 | Cylinder Under Harmonic Loads Analyzed by DYNAX - Validation Problem 3 |
| C-19 | Comparison of Results From DYNAX and Reference 11 of Meridional Moments and Deflections of Cylinder - (N=0, N=2) |
| C-20 | Comparison of Results From DYNAX and Reference 11 of Meridional Moments and Deflections of Cylinder - (N=5, N=20) |
| C-21 | Suddenly Applied Ring Line Load Analyzed by DYNAX - Validation Problem 4 |
| C-22 | Radial Displacement vs. Time - Comparison of Results from DYNAX and Reference 12 |
| C-23 | Bending Moment vs. Time - Suddenly Applied Ring (Line) Load - Comparison of Results from DYNAX and Reference 12 |
| C-24 | Spherical Cap Analyzed by DYNAX - Validation Problem 5 |
| C-25 | Comparison of Results From DYNAX and Reference 13 of Axial Displacement of Spherical Cap Under Dynamic Load |
| C-26 | Comparison of Results From DYNAX and Reference 13 of Axial Displacement of Spherical Cap Under Dynamic Load |
| C-27 | Hyperbolic Cooling Tower Analyzed by DYNAX - Validation Problem 6 |
| C-28 | Spectrum of Design Earthquake Used for DYNAX - Validation Problem 6 |
| C-29 | Comparison of Cooling Tower Meridional Forces Obtained by DYNAX and Reference 14 |

CPS/USAR

LIST OF FIGURES (Cont'd)

| <u>NUMBER</u> | <u>TITLE</u> |
|---------------|---|
| C-30 | Tying of Solid and Shell Elements Analyzed by DYNAX - Validation Problem 7 |
| C-31 | Moment Diagram of Results from DYNAX and Analytical Solution |
| C-32 | Circular Plate Analyzed by DYNAX - Validation Problem 8 |
| C-33 | Cylinder Under Constant Pressure Analyzed by DYNAX and SOR-III (DYNAX) |
| C-34 | Cylinder Under Dynamic Axial Pressure for Non-Reflecting Boundaries Analyzed by DYNAX - Validation Problem 10 |
| C-35 | Finite Element Model and Material Damping Coefficients for Cylinder Analyzed by DYNAX - Validation Problem 11 |
| C-36 | Modeling and Load Distribution |
| C-37 | Time History of Load (t) |
| C-38 | Idealized Model of Anchor-Panel System Used in LAFD Validation Problem |
| C-39 | LUSH Validation Problem |
| C-40 | Comparison of Response Spectrum (EERC Report No. 74-4 and S&L LUSH) |
| C-41 | Design of Tied Column - Compression Controls |
| C-42 | Design of Tied Column - Tension Controls |
| C-43 | Design of Tied Column - Biaxial Bending |
| C-44 | Example Frame for PIPSYS Static Analysis |
| C-45 | Piping System for Combined Stress Analysis (PIPSYS) |
| C-46 | Structural Model of Piping System (PIPSYS) |
| C-47 | Load Time History (PIPSYS) |
| C-48 | Displacement vs. Time Joint 8 Z Direction (PIPSYS) |
| C-49 | Rectangular Tank Filled With Water (PLFEM-II) |
| C-50 | Moment of M_y at Horizontal Centerline of Walls (PLFEM-II) |
| C-51 | Moment M_y At Top of Wall (PLFEM-II) |
| C-52 | M_x Moment Along Centerline of Long Wall (PLFEM-II) |
| C-53 | Plate With Circular Hole Under Uniform Tension (PLFEM-II) |
| C-54 | Stresses in Plate with Circular Hole Under Uniform Tension (PLFEM-II) |
| C-55 | Square Plate With Rectangular Hole Subjected to Temperature Variation (PLFEM-II) |
| C-56 | Moments in Plate Due to Temperature Variation (PLFEM-II) |
| C-57 | Loads and Configuration for PLGIRD Sample Problem |
| C-58 | POLSAP4 Input Commands for Beam Problem |
| C-59 | Generated SLSAP4 Data for Beam Problem |
| C-60 | POLSAP4 Input for Plate Problem |
| C-61 | Generated SLSAP4 Input for Plate Problem |
| C-62 | Response Spectrum for Sinusoidal Variation of Ground Motion |
| C-63 | Fourier Transform Plot from RSG for a 5 Cycle/Sec Sine Wave Time History |
| C-64 | Comparison of Desired Response Spectrum and Response Spectrum of Compatible Acceleration Time History (Damping = 0.02) from RSG |
| C-65a | Soil Profile and Properties for Consolidation Settlement Computation Using Janbu's Method (SETTLE Validation Problems 1 and 4) |
| C-65b | Loading Area on Soil for SETTLE Validation Problems 1 to 3 |
| C-66a | Loading Area Used for Calculating Rigid Foundation Movement for Settle Validation Problem 4 |

CPS/USAR

LIST OF FIGURES (Cont'd)

| <u>NUMBER</u> | <u>TITLE</u> |
|---------------|---|
| C-66b | Location of Spring for Calculating Rigid Foundation Movement for SETTLE Validation Problem 4 |
| C-67 | Flow Chart for SHAKE |
| C-68 | Soil Profile and Layered Representation Used for the SHAKE Sample Problem |
| C-69 | Comparison of Shear Stresses and Accelerations Computed by SHAKE and QUAD4 (SHAKE) |
| C-70 | Comparison of Spectral Values for Surface Motions Computed by SHAKE and QUAD4 (SHAKE) |
| C-71 | Model of Pipe Network for SLSAP and SAPIV (SLSAP Validation Problem 1) |
| C-72 | Comparison of Surface Stresses in a Clamped Spherical Shell Under External Pressure for SLSAP and SAPIV (SLSAP Validation Problem 2) |
| C-73 | Model of Plane Frame for SLSAP and SAPIV (SLSAP Validation Problem 3) |
| C-74 | Model of Pipe Assemblage for SLSAP and SAPIV (SLSAP Validation Problem 4) |
| C-75 | Comparison of Bending Moments in a Cantilever Beam for SLSAP and Reference 43 (SLSAP Validation Problem 5) |
| C-76 | Comparison of Bending Moments in a Simply Supported Plate for SLSAP and Reference 44 (SLSAP Validation Problem 6) |
| C-77 | Model for Response History Analysis for SLSAP and SAPIV (SLSAP Validation Problem 7) |
| C-78 | Comparison of SLSAP and SAPIV Transverse Deflections of the Cantilever Beam (SLSAP Validation Problem 7) |
| C-79 | Comparison of SLSAP and SAPIV Bending Moments for the Cantilever Beam (SLSAP Validation Problem 7) |
| C-80 | Cylindrical Tube and Load History for SLSAP and SAPIV Mode Superposition and Direct Integration Analyses (SLSAP Validation Problem 8) |
| C-81 | Displacement Comparison of SLSAP Mode Superposition and Reference 45 for the Cylindrical Type (SLSAP Validation Problem 8) |
| C-82 | Displacement Comparison of SLSAP Direct Integration and Reference 45 for the Cylindrical Type (SLSAP Validation Problem 8) |
| C-83 | Circular Plate on a Rigid Foundation for SLSAP and NOBEC (SLSAP Validation Problem 9) |
| C-84 | Comparison of Displacement and Moment Variation of Circular Plate from SLSAP and NOBEC (SLSAP Validation Problem 9) |
| C-85 | Circular Plate for SOR-III Example |
| C-86 | LMoment Comparison SABOR-III and SOR-III |
| C-87 | Radial Shear Comparison for SABOR-III and SORLK-III |
| C-88 | Loads on Beam (STAND Validation Problem 4) |
| C-89 | Transverse Loads on Beam (STAND Validation Problem 5) |
| C-90 | Transverse Loads on Beam (STAND Validation Problem 6) |
| C-91 | Ten-Story Shear Wall Model for NONLIN 2 Program |
| C-92 | Comparison of Displacement for Node 11 (NONLIN2) |
| C-93 | Comparison of Moment for Member 1 (NONLIN2) |
| C-94 | Pipe Whip Model for Example 1 of PWRRA |
| C-95 | Pipe Whip Model for Example 2 of PWRRA |
| C-96 | Pipe Whip Model for Example 3 of PWRRA |

CPS/USAR

LIST OF FIGURES (Cont'd)

| <u>NUMBER</u> | <u>TITLE</u> |
|---------------|--|
| C-97 | Cable Tray Model for SEISHANG Program |
| C-98 | Ceiling Mounted Support Model for SEISHANG Program |
| C-99 | Wall Mounted Support Model for SEISHANG Program |
| C-100 | Soil Profile and Properties for Consolidation Settlement Computation Using Terzaghi's Method (SETTLE Validation Problem 2) |
| C-101 | Soil Profile and Properties for Elastic Settlement Computation Using Terzaghi's Method (SETTLE Validation Problem 3) |
| C-102 | Loading Area on Soil for SETTLE Validation Problem 5 |

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.

CPS/USAR

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 longitudinal 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 in² 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

$$\text{Foundation Modulus } k = 518.4 \text{ kip/ft}^3.$$

The plate is analyzed for a uniform pressure load

$$p = 50 \text{ kip/ft}^2$$

and a uniform edge load

$$M = 1000 \text{ kip-ft/ft.}$$

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

CPS/USAR

$$w = p/k$$

and for the edge moment

$$w = D_1 Z_1(vr) + D_2 Z_2(vr)$$

where

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

$$z_1(x) = R_e J_0(x\sqrt{i})$$

$$z_2(x) = I_m J_0(x\sqrt{i})$$

J_0 = Bessel function of the first kind

and

$$D_1 = \frac{Mv^2}{k} \frac{z_1'(va)}{z_1(va)z_2'(va) - z_1'(va)z_2(va) + \frac{1-\mu}{va} [z_1'^2(va) + z_2'^2(va)]}$$

$$D_2 = \frac{Mv^2}{k} \frac{z_2'(va)}{z_1(va)z_2'(va) - z_1'(va)z_2(va) + \frac{1-\mu}{va} [z_1'^2(va) + z_2'^2(va)]}$$

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

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 | $a = 100 \text{ in.}$ |
| Thickness | $h = 2 \text{ in.}$ |
| Elastic Modulus | $E = 3.0 \times 10^7 \text{ psi}$ |
| Poisson's Ratio | $\nu = 0.3$ |
| Loading intensity | $q = \frac{pr}{a} \cos \phi$ |

CPS/USAR

where

$$p = 100 \text{ psi.}$$

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, double 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.

CPS/USAR

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 θ_y 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: quadrilateral, 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,

CPS/USAR

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 $\epsilon_o = 1.8936E-5$ and the strain calculated by hand is $\epsilon_o = 1.81E-5$.

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

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

CPS/USAR

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:

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

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 $\theta = 0^\circ$ 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 $\theta = 0^\circ$, $\theta = 120^\circ$ and $\theta = 240^\circ$ 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_0 | = 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):

$$F_0 - P = \frac{EA}{a} [(U_1 - U_2)] + f_1 \quad (C-1)$$

CPS/USAR

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

$$n = 1, 2, 3, \dots, N$$

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

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

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 uniaxial 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:

CPS/USAR

| | <u>Dead</u> | <u>Live</u> |
|-------|-------------|-------------|
| 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 comparison 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

CPS/USAR

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 rectangular 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 conjunction 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 accuracy 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 analyzed 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 hardware 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 description 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.

CPS/USAR

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 propagation.

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:

$$\text{Total unit weight} = 125 \text{ pcf}$$

$$(K_2)_{\max} = 65$$

$$K_o = 0.5.$$

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:

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

CPS/USAR

where

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

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 originally 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.

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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 cross-sectional area of 1 by 2 inch, length of 10 inches, and Young's modulus equal to 30×10^3 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 in² by 1 inch thick plate with Poisson's ratio equal to 0.3 and Young's modulus equal to 30×10^3 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).

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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.

CPS/USAR

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.

CPS/USAR

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 nonlinear material 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 PWRRRA

PWRRRA (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.

PWRRRA 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 PWRRRA. 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 PWRRRA. 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 PWRRRA 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, cohesion, 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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CPS/USAR

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TABLE C-1
SPAN 1 CHARACTERISTICS AND OUTPUT RESULTS

| | | <u>LEFT SIDE</u> | <u>MIDDLE</u> | <u>RIGHT SIDE</u> |
|----------------------------------|----------------|--------------------|-------------------|--------------------|
| 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 -

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TABLE C-2
SPAN 2 CHARACTERISTICS AND OUTPUT RESULTS

| | | <u>LEFT SIDE</u> | <u>MIDDLE</u> | <u>RIGHT SIDE</u> |
|----------------------------------|-------------|--------------------|------------------|-------------------|
| 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. |

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TABLE C-3
SPAN 3 CHARACTERISTICS AND OUTPUT RESULTS

| | | <u>LEFT SIDE</u> | <u>MIDDLE</u> | <u>RIGHT SIDE</u> |
|----------------------------------|-------------|------------------|------------------|-------------------|
| 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. |

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TABLE C-4
RESULTING TOTAL LOAD

| <u>NODE</u> | <u>TOTAL LOAD (kips)</u> | |
|-------------|--------------------------|--------------------------|
| | <u>COLOAD</u> | <u>HAND CALCULATIONS</u> |
| 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 |

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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 |

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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 |

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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) |
| 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.

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**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 | 555849750 |
| | | | | | | | c | d |

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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, $\bar{X} = C/n = 158980 / 46$
 $= 3456 \text{ psi}$

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.

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TABLE C-7
PROBABLE MAXIMUM STORY SHEARS

PROBABLE MAXIMUM STORY SHEAR, kips

| <u>MODE NUMBER</u> | <u>BIGGS</u> | <u>DYNAS</u> |
|------------------------|--------------|--------------|
| 1 | 2250 | 2262 |
| 2 | 1740 | 1757 |
| 3 | 895 | 902 |

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TABLE C-8
STRUCTURAL FREQUENCIES

STRUCTURAL FREQUENCY, CPS

| <u>MODE NUMBER</u> | <u>BIGGS</u> | <u>DYNAS</u> |
|------------------------|--------------|--------------|
| 1 | 1.00 | 1.00 |
| 2 | 2.18 | 2.18 |
| 3 | 3.18 | 3.18 |

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TABLE C-9
PROBABLE MAXIMUM STORY DISPLACEMENTS

PROBABLE MAXIMUM STORY DISPLACEMENT, in.

| <u>MODE NUMBER</u> | <u>BIGGS</u> | <u>DYNAS</u> |
|------------------------|--------------|--------------|
| 1 | 1.50 | 1.51 |
| 2 | 3.22 | 3.20 |
| 3 | 4.86 | 4.68 |

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TABLE C-10
NATURAL PERIODS FOR THE EIGHT LOWEST
FLEXURAL MODES

| <u>MODE</u> <u>NUMBER</u> | <u>PERIODS IN SECONDS</u> | |
|------------------------------|---------------------------|--------------|
| | <u>SAPIV</u> | <u>DYNAS</u> |
| 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 |

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

| <u>RESPONSE COMPONENT</u> | <u>MODEL 1</u> | <u>MODEL 2</u> |
|----------------------------------|----------------|----------------|
| 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 |

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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 |

CPS/USAR

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 |

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TABLE C-14
MODEL DAMPING COMPARISON

| <u>MODE</u> | <u>DYNAX</u> | <u>HAND CALCULATIONS</u> |
|-------------|--------------|--------------------------|
| 1 | 0.0352 | 0.0348 |
| 2 | 0.0368 | 0.0367 |
| 3 | 0.0430 | 0.0430 |

CPS/USAR

TABLE C-15
PROPERTIES OF STRUCTURAL MODEL

(a) MATERIAL PROPERTIES

| MATERIAL | DENSITY | YOUNG'S MODULUS (kip/ft ²) | POISSON'S RATIO |
|----------|---|--|--------------------|
| | $\frac{\text{kip} \cdot \text{sec}^2}{\text{ft}^4}$ | | |
| 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 |

CPS/USAR

TABLE C-16
COMPARISON OF NODAL ACCELERATIONS IN G UNITS

(a) MAXIMUM RADIAL ACCELERATION

| NODE | DYNAX | | FAST | |
|------|-------------------|---------------|-------------------|---------------|
| | <u>TIME (sec)</u> | <u>ACCLN.</u> | <u>TIME (sec)</u> | <u>ACCLN.</u> |
| 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 | |
|------|-------------------|---------------|-------------------|---------------|
| | <u>TIME (sec)</u> | <u>ACCLN.</u> | <u>TIME (sec)</u> | <u>ACCLN.</u> |
| A | 0.171 | -17.20 | 0.171 | -17.21 |
| B | 0.135 | 12.58 | 0.135 | 12.58 |

CPS/USAR

TABLE C -17
COMPARISON OF MAXIMUM STRESS RESULTANTS
IN K, FT UNITS AT ELEMENT C, $\theta = 0^\circ$

| <u>COMPONENT</u> | <u>DYNAX</u> | | <u>FAST</u> | |
|--------------------------------|-----------------------------|------------------------------|-----------------------------|------------------------------|
| | <u>TIME</u> <u>(sec)</u> | <u>FORCE</u> <u>K, ft</u> | <u>TIME</u> <u>(sec)</u> | <u>FORCE</u> <u>K, ft</u> |
| 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 |

CPS/USAR

TABLE C-18
COMPARISON OF NODAL ACCELERATIONS IN G UNITS

(a) MAXIMUM RADIAL ACCELERATION

| <u>NODE</u> | <u>DYNAX</u> | | <u>FAST</u> | |
|-------------|-----------------------------|---------------|-----------------------------|---------------|
| | <u>TIME</u> <u>(sec)</u> | <u>ACCLN.</u> | <u>TIME</u> <u>(sec)</u> | <u>ACCLN.</u> |
| A | 0.012 | 8.73 | 0.012 | 8.74 |
| B | 0.132 | -21.40 | 0.132 | -21.41 |

(b) MAXIMUM VERTICAL ACCELERATION

| <u>NODE</u> | <u>DYNAX</u> | | <u>FAST</u> | |
|-------------|-----------------------------|---------------|-----------------------------|---------------|
| | <u>TIME</u> <u>(sec)</u> | <u>ACCLN.</u> | <u>TIME</u> <u>(sec)</u> | <u>ACCLN.</u> |
| A | 0.171 | -1745. | 0.171 | -1747 |
| B | 0.135 | 1276. | 0.135 | 1277. |

CPS/USAR

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

| <u>COMPONENT</u> | <u>DYNAX</u> | | <u>FAST</u> | |
|--------------------------------|-----------------------------|------------------------------|-----------------------------|------------------------------|
| | <u>TIME</u> <u>(sec)</u> | <u>FORCE</u> <u>K, ft</u> | <u>TIME</u> <u>(sec)</u> | <u>FORCE</u> <u>K, ft</u> |
| 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 |

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TABLE C-20
DISPLACEMENTS

| <u>LOCATION</u> | <u>VALUE, INCHES</u> |
|-----------------|----------------------|
| 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

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TABLE C-21
ANCHOR FORCES

| <u>LOCATION</u> | <u>VALUE, kips</u> |
|-----------------|--------------------|
| 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 |

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TABLE C-22
STRAIN-COMPATIBLE SOIL PROPERTIES

| EFFECTIVE SHEAR STRAIN $\gamma_{eff}(\%)$ | SHEAR MODULUS REDUCTION FACTOR* | | FRACTION OF CRITICAL DAMPING (%) | |
|--|------------------------------------|-------------|-------------------------------------|-------------|
| | <u>CLAY</u> | <u>SAND</u> | <u>CLAY</u> | <u>SAND</u> |
| $\leq 1. \times 10^{-4}$ | 1.000 | 1.000 | 2.50 | 0.50 |
| 3.16×10^{-4} | 0.913 | 0.984 | 2.50 | 0.80 |
| 1.00×10^{-3} | 0.761 | 0.934 | 2.50 | 1.70 |
| 3.16×10^{-3} | 0.565 | 0.826 | 3.50 | 3.20 |
| 1.00×10^{-2} | 0.400 | 0.656 | 4.75 | 5.60 |
| 3.16×10^{-2} | 0.261 | 0.443 | 6.50 | 10.0 |
| 1.00×10^{-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 |
| ≥ 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.

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

| <u>ELEMENT</u> <u>NUMBER</u> | <u>SHEAR MODULUS G ksf</u> | | <u>DAMPING RATIO $\lambda\%$</u> | |
|---------------------------------|----------------------------|-------------|---|-------------|
| | <u>REF. 1</u> | <u>LUSH</u> | <u>REF. 1</u> | <u>LUSH</u> |
| 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 |

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TABLE C-24
COMPARISON OF STRESSES DUE TO HORIZONTAL EXCITATION

| ELEMENT NUMBER | σ_x psf | | σ_y psf | | σ_{xy} psf | |
|-------------------|----------------|-------------|----------------|-------------|-------------------|-------------|
| | <u>REF. 1</u> | <u>LUSH</u> | <u>REF. 1</u> | <u>LUSH</u> | <u>REF. 1</u> | <u>LUSH</u> |
| 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 |

CPS/USAR

TABLE C-25

COMPARISON OF NODAL POINT ACCELERATIONS DUE TO HORIZONTAL AND VERTICAL EXCITATIONS

| NODAL POINT NUMBER | HORIZONTAL EXCITATION | | | | VERTICAL EXCITATION | | | |
|--------------------------|-----------------------|-------------|--------------|-------------|---------------------|-------------|---------------|-------------|
| | X Acc. g | | Y Acc. g | | X Acc. g | | Y Acc. g | |
| | <u>REF. 1</u> | <u>LUSH</u> | <u>REF.1</u> | <u>LUSH</u> | <u>REF. 1</u> | <u>LUSH</u> | <u>REF. 1</u> | <u>LUSH</u> |
| 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 |

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TABLE C-26
COMPARISON OF MOMENTS FOR SELECTED MEMBERS

| | <u>MOMENTS FROM REFERENCE 26 (kip-ft)</u> | <u>MOMENTS FROM PIPSYS (kip-ft)</u> |
|-----------------|---|---|
| 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 |

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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) | ΔT ₂ |
|--------------|----------------------|-------------------|------|----------------|----------------|----------------|-----------------|---------------------------|--------------------------|-----------------|
| 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 |

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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.

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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 | |
|-------------------------|----|------------------------------|--------------|----------------------------|--------------|
| | | $\frac{S_p K_e}{2}$ | USAGE FACTOR | $\frac{S_p K_e}{2}$ | USAGE FACTOR |
| i | j | | | | |
| 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 |

CPS/USARTABLE 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 |

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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) | f_a (ksi) | F_a (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 |

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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 | | AXIAL STRESS | | INTERACTION RATIO | WEB SHEAR STRESS | | |
|--------------------|-------------------|--------------------|-----------------------|-----------------------------|-----------------------------|----------------|-------------|--------------|-------------|-------------------|-------------------|-----------------|--------------------|
| | | | | | | f_b (ksi) | F_b (ksi) | f_a (ksi) | F_a (ksi) | | SHEAR STRESS | SHEAR STRESS AT | EDGE COMPRESSION |
| | | | | | | | | | | | AT LEFT END (ksi) | RIGHT END (ksi) | 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 |

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

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TABLE C-35
APPLIED LOADS FOR SLSAP4 PIPE NETWORK

| LOADING TYPE | DIRECTION | | |
|--------------------|-----------|---------|--------|
| | X | Y | 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 |

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TABLE C-36
FORCE EQUILIBRIUM REACTIONS

| <u>NODE</u> | 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. |

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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 |

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TABLE C-38
COMPARISON OF MOMENT

MOMENT MZ (kip/in.) IN ELEMENT LOCAL
COORDINATES (at element end 1)

| ELEMENT NUMBER | 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 |

CPS/USAR

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

| <u>MODE NUMBER</u> | <u>PERIOD (sec) SLSAP4</u> | <u>PERIOD (sec) SAPIV</u> |
|------------------------|--------------------------------|-------------------------------|
| 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 |

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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 |

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TABLE C-41
ROLLED BEAM DESIGN PROBLEM

| | <u>MAXIMUM MOMENTS (kip-ft)</u> | <u>SECTION SELECTED</u> | <u>SECTION MODULUS (in³)</u> |
|-------|---|-----------------------------|---|
| AISC | 125 | W16 x 40 | 64.6 |
| STAND | 125.58 | W18 x 40 | 68.4 |


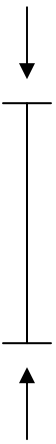
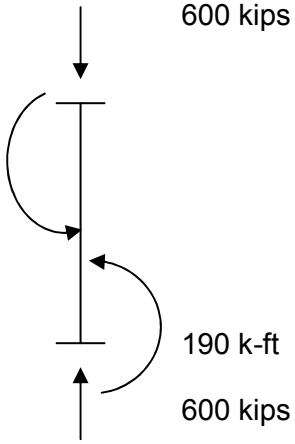
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TABLE C-42
COMPOSITE BEAM DESIGN PROBLEM

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

CPS/USAR

TABLE C-43
COLUMN DESIGN PROBLEM

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

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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 in².

CPS/USAR

TABLE C-45
COMPOSITE BEAM WITH AXIAL LOADS

| | <u>ALLOWABLE AXIAL STRESS (ksi)</u> | |
|------------------|-------------------------------------|--------------------|
| | <u>CONSTRUCTION CASE</u> | <u>DESIGN CASE</u> |
| STAND | 20.48 | 20.98 |
| Hand Calculation | 20.48 | 20.98 |

CPS/USAR

TABLE C-46
COMPOSITE BEAM WITH VERTICAL SEISMIC LOADS

| | FREQUENCY (Hz) | <u>ACCELERATION</u> | | <u>DESIGN MOMENT (k-ft)</u> | |
|---------------------|-------------------|---------------------|-------|-----------------------------|--------|
| | | 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 |

CPS/USAR

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

| SECTION AND MATERIAL PROPERTIES | PROBLEM | | |
|--|-------------------------|-------------------------|-------------------------|
| | 1 | 2 | 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 1st 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 x 10 ⁻⁶ | 5.56 x 10 ⁻⁶ | 5.56 x 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 |

CPS/USAR

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

| <u>RESULTS</u> | <u>PROBLEM</u> | | |
|---|----------------|---------|----------|
| | 1 | 2 | 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 |

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

| <u>SECTION AND MATERIAL PROPERTIES</u> | <u>PROBLEM 4</u> |
|---|------------------|
| 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 |

CPS/USAR

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

| <u>RESULTS</u> | <u>PROBLEM 4</u> |
|--|------------------|
| 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 |

CPS/USAR

TABLE C-51
INPUT FOR NONTHERMAL AND THERMAL LOADS PROBLEM

| <u>SECTION AND MATERIAL PROPERTIES</u> | <u>PROBLEM 5</u> |
|--|--------------------------|
| 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 x 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 x 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 |

CPS/USAR

TABLE C -52
RESULTS FROM NONTHERMAL AND THERMAL LOADS PROBLEM

| <u>RESULTS</u> | <u>PROBLEM 5</u> |
|---|------------------|
| 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 |

CPS/USAR

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

| <u>RESULTS OBTAINED FROM</u> | <u>MAXIMUM TIP DISPLACEMENT (inches)</u> |
|------------------------------|--|
| Ma and Bathe | 5.1 |
| PWRRA | 5.506 |

CPS/USAR

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

| 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 |
| PWRRRA | 27.47 | 5.46 | 5.32 |

CPS/USAR

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 |

CPS/USAR

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

| | <u>SEISHANG</u> | <u>HAND</u> <u>CALCULATION</u> |
|--|-----------------|-----------------------------------|
| 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 |

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TABLE C-57
RESPONSE OF THE CEILING MOUNTED SUPPORT

| | | <u>SEISHANG</u> | <u>DYNAS</u> |
|---|------------------|-----------------|--------------|
| 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 |

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TABLE C-58
RESPONSE OF THE WALL MOUNTED SUPPORT

| | | <u>SEISHANG</u> | <u>DYNAS</u> |
|---|------------------|-----------------|--------------|
| 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 |

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TABLE C-59
INTERACTION COEFFICIENTS OF THE
CEILING MOUNTED SUPPORT

| <u>INTERACTION COEFFICIENT</u> | <u>SEISHANG</u> | <u>PIPSYS</u> |
|--------------------------------|-----------------|---------------|
| 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 |

CPS/USAR

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 |

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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 |

CPS/USAR

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 |

CPS/USAR

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 |

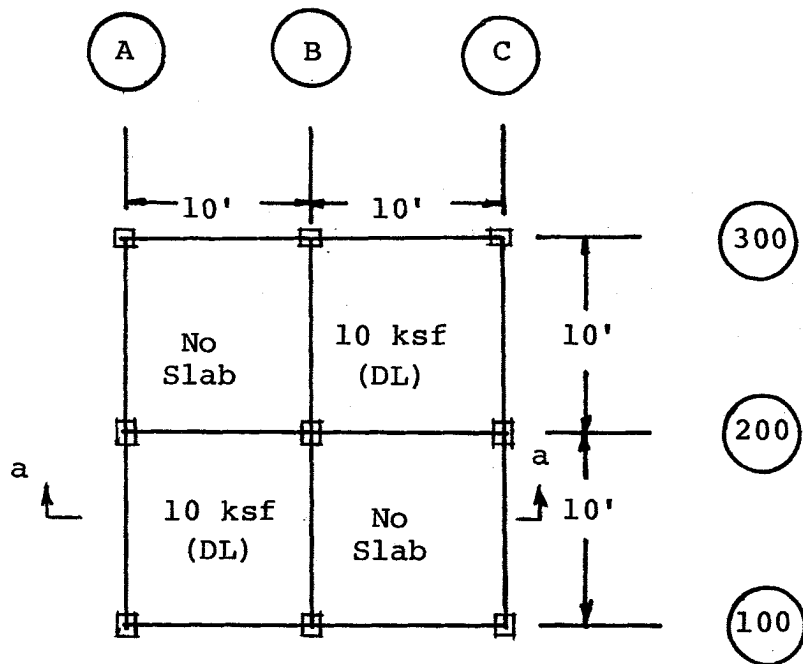
NOTES:

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

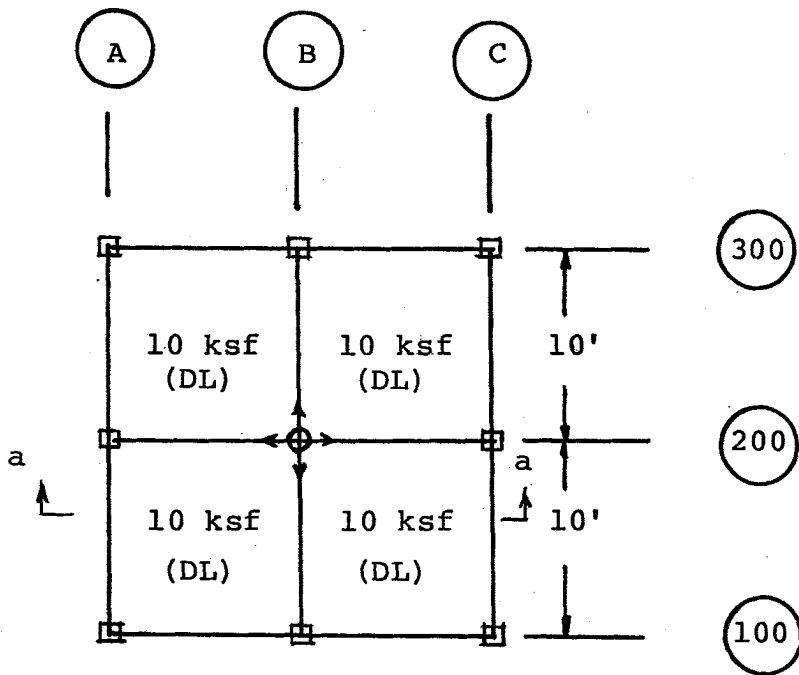
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TABLE C-64
STRESS FOR PROBLEM 5 OF SETTLE VALIDATION

| X-AXIS | Y-AXIS | DEPTH FROM HIGHEST FOUNDATION LEVEL (Ft) | STRESS (psf) | |
|---------|---------|---|---------------------|-----------|
| | | | 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

| |
|---|
| <p>CLINTON POWER STATION UPDATED SAFETY ANALYSIS REPORT</p> |
| <p>FIGURE C-1</p> <p>PLAN, ELEVATION, AND LOADING FOR COLOAD VALIDATION PROBLEM</p> |

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 %

Note: Scanned image of computer print-out

**CLINTON POWER STATION
UPDATED SAFETY ANALYSIS REPORT**

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 AVERAGE | RANGE PSI | SAMP NO | CONC | SAMP DATE MO DY YR | TEST DATE MO DY YR | CEM | SUMP IN | AIR % | TEMP DEG F | FRACTURE CYL 1 | CYL 2 | 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 |
| 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 INF | 28 |

*** BAD CONTROL-MOVING AVERAGE

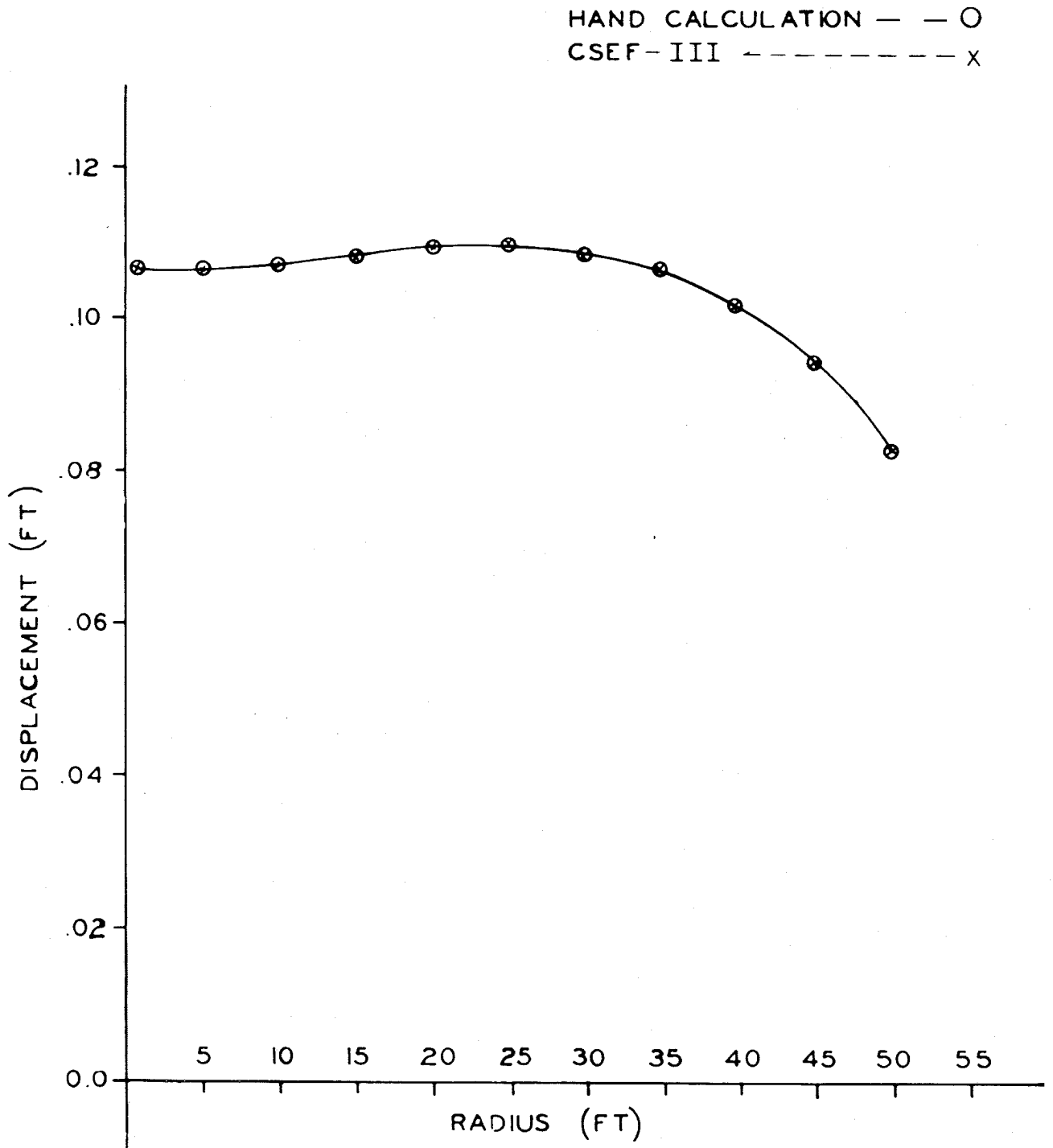
Note: Scanned image of computer print-out

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FIGURE C-2

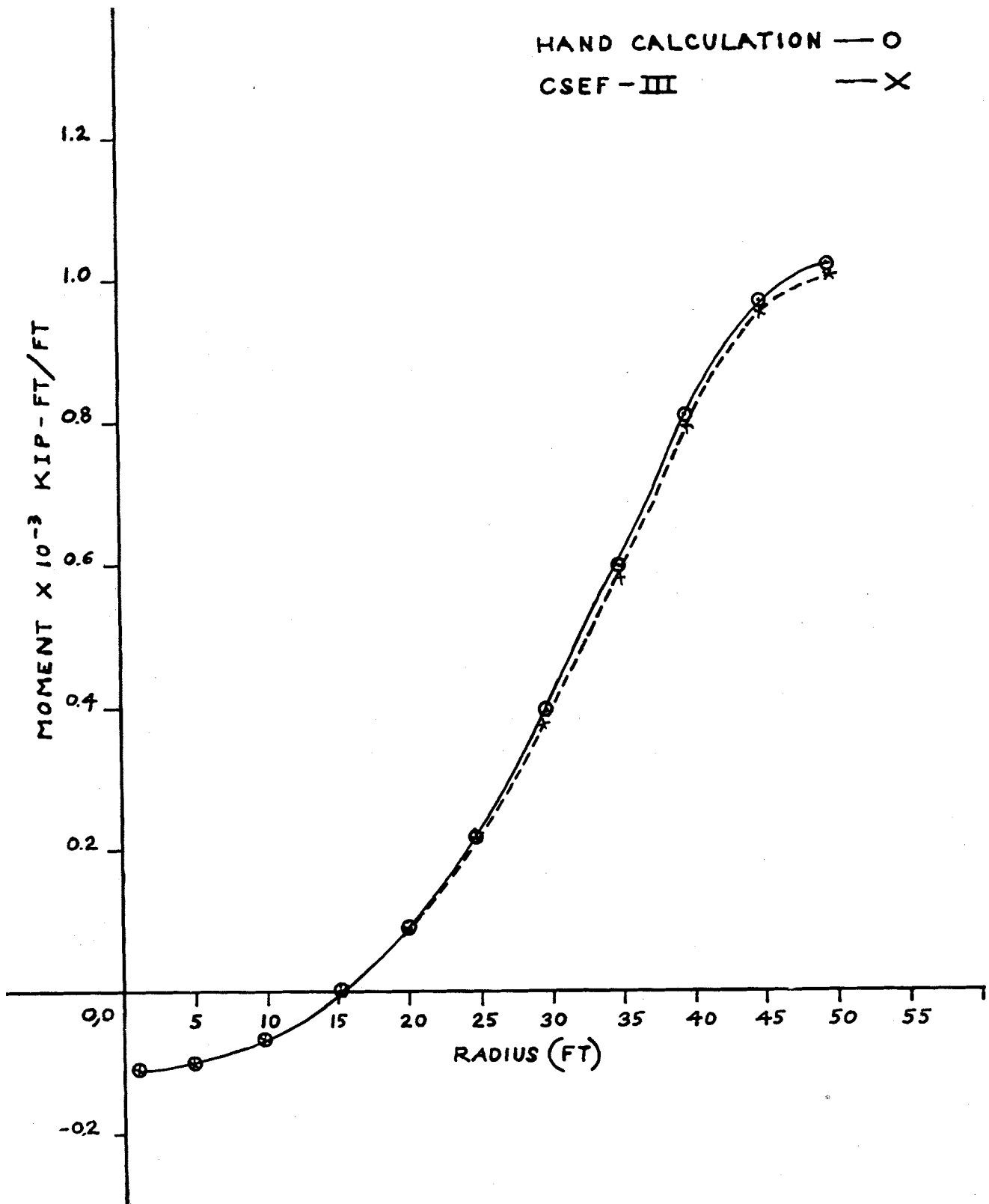
CONSOLIDATED RESULTS FOR DIFFERENT
 LOCATIONS (CONCRETE)

(SHEET 2 of 3)



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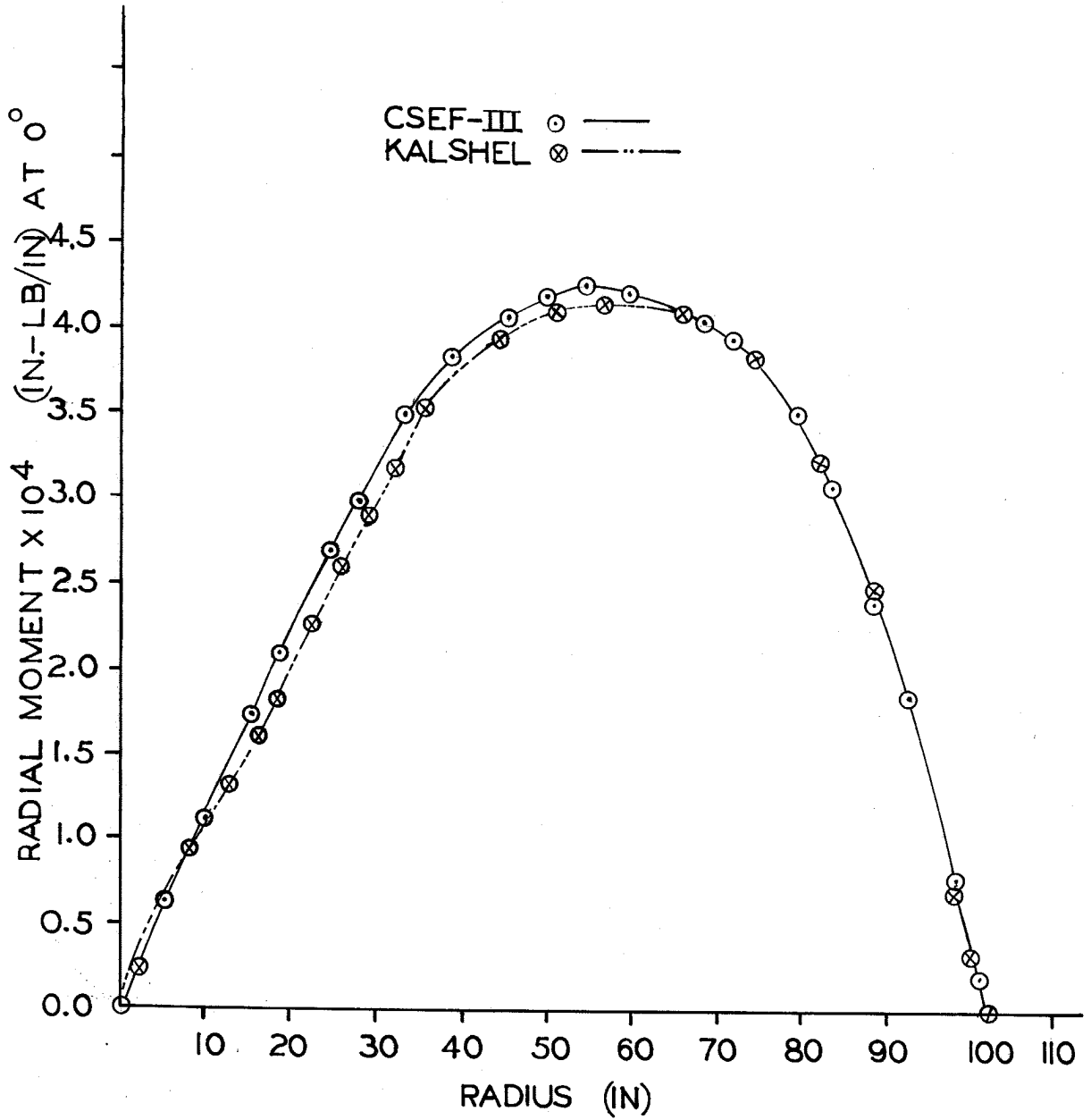
FIGURE C-3
COMPARISON OF DEFLECTION OF A CIRCULAR
PLATE DUE TO UNIFORM PRESSURE AND AXI-
SYMMETRIC EDGE MOMENT (CSEF-III VS.
HAND CALCULATION)



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FIGURE C-4

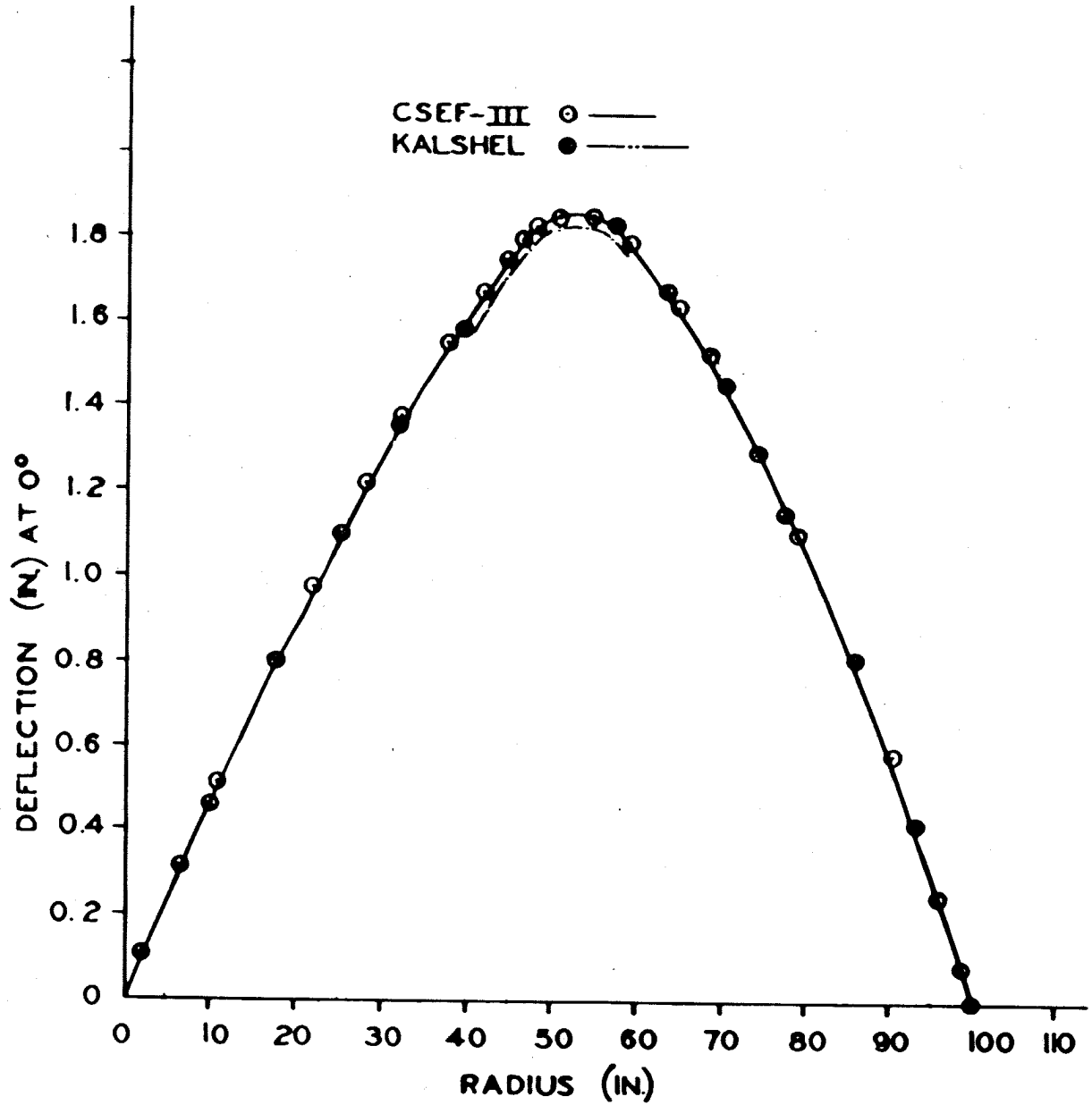
RADIAL MOMENT DUE TO UNIFORM PRESSURE
 AND AXISYMMETRIC EDGE MOMENT
 (CSEF-III VS. HAND CALCULATION)



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FIGURE C-5

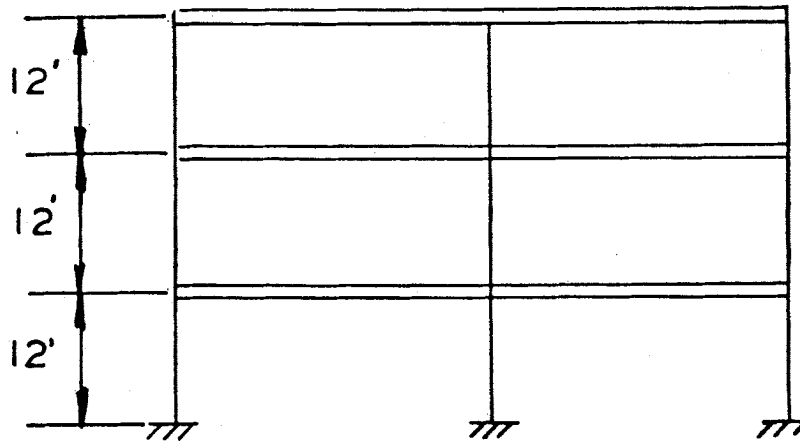
SIMPLY SUPPORTED CIRCULAR PLATE,
 LINEARLY VARYING PRESSURE LOAD
 FOR RADIAL MOMENT



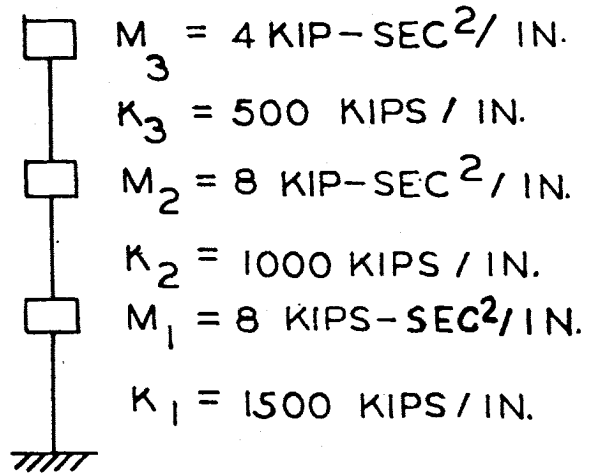
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FIGURE C-6

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



(A)

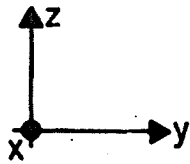


(B)

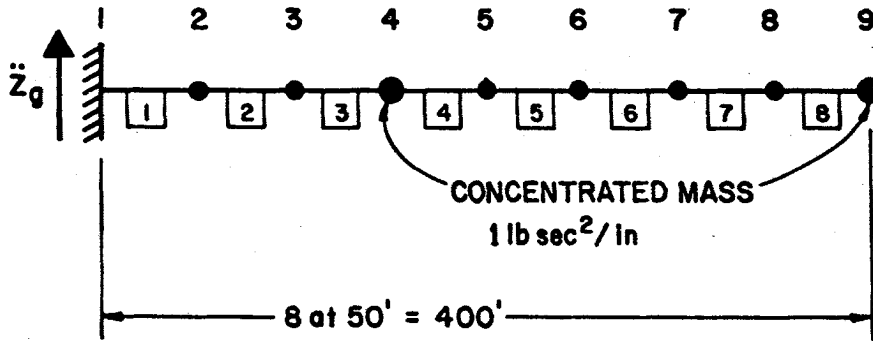
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FIGURE C-7

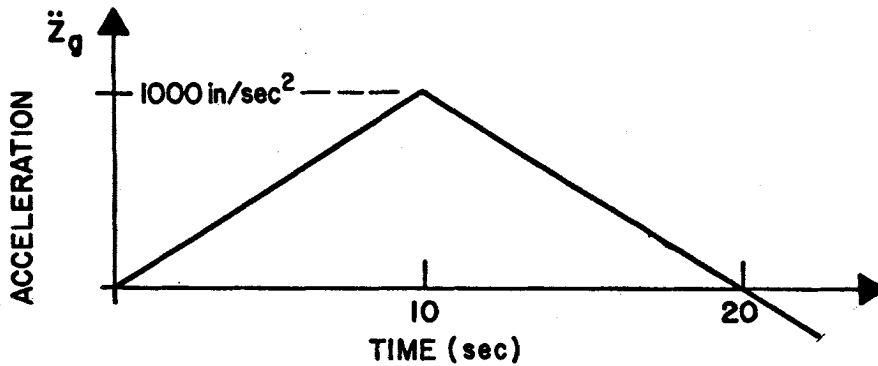
THREE-STORY SHEAR BUILDING
 MODEL FOR DYNAS



$I = 1.0 \text{ in}^4$; $A = 100.0 \text{ in}^2$
 $E = 30 \times 10^6 \text{ lbs/in}^2$
 $\rho = 1.0 \text{ lb-sec}^2/\text{in}^4$

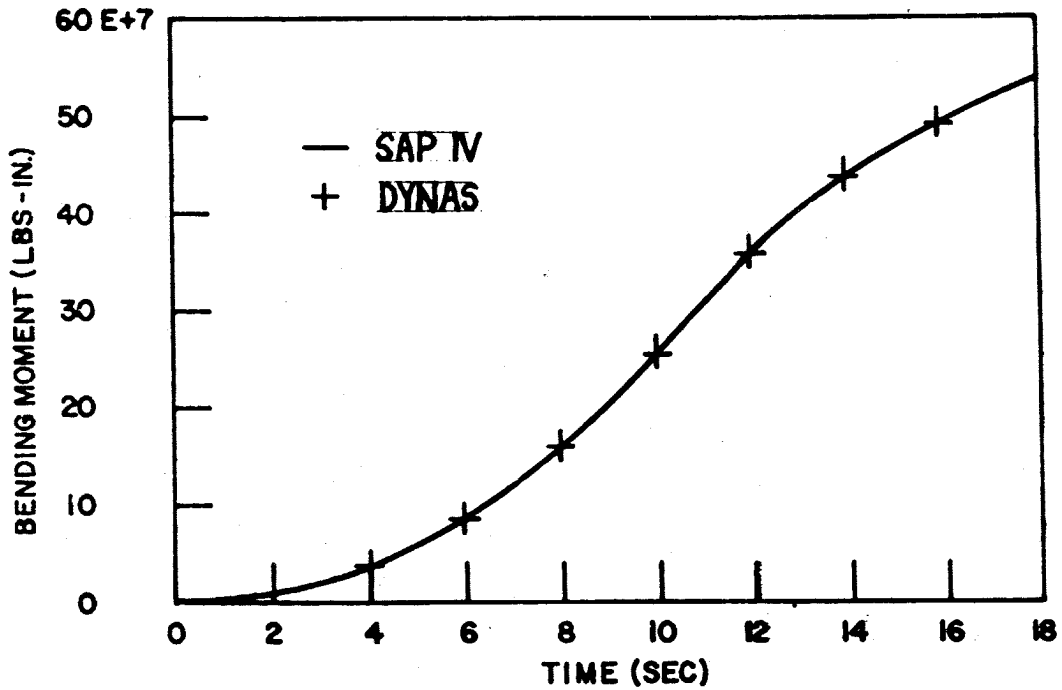


(a) NODE AND BEAM NUMBER ASSIGNMENTS FOR THE CANTILEVER MODEL



(b) GROUND ACCELERATION APPLIED AT NODE 1

| |
|--|
| CLINTON POWER STATION UPDATED SAFETY ANALYSIS REPORT |
| FIGURE C-8 RESPONSE HISTORY ANALYSIS OF CANTILEVER BEAM FOR DYNAS VALIDATION PROBLEM NO. 2 |

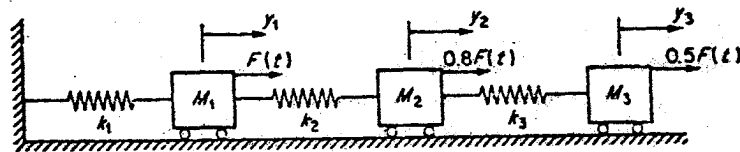
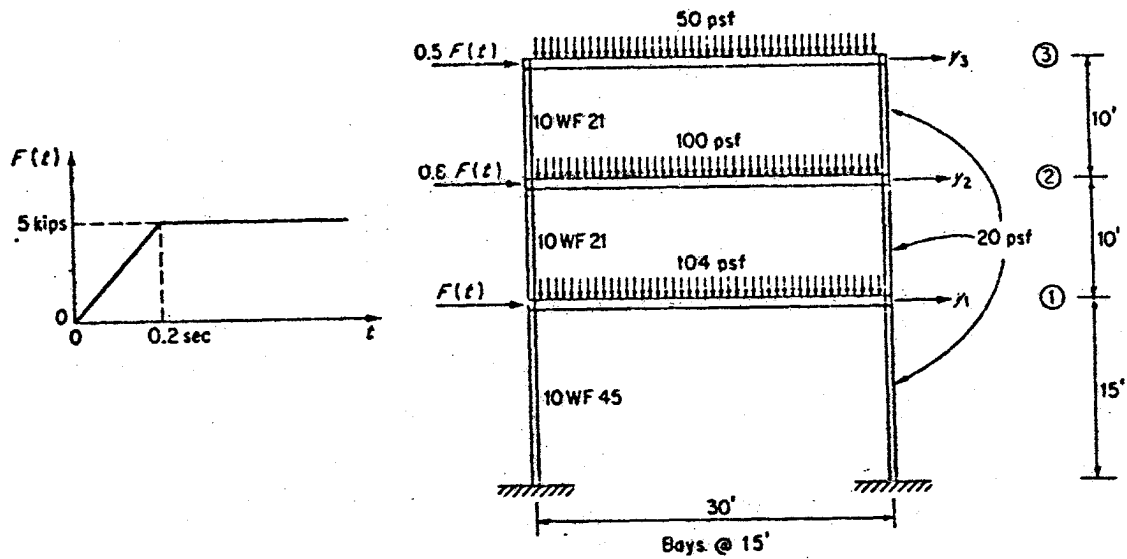


**MOMENT AT NODE 1
(FIXED END OF CANTILEVER)**

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FIGURE C-9

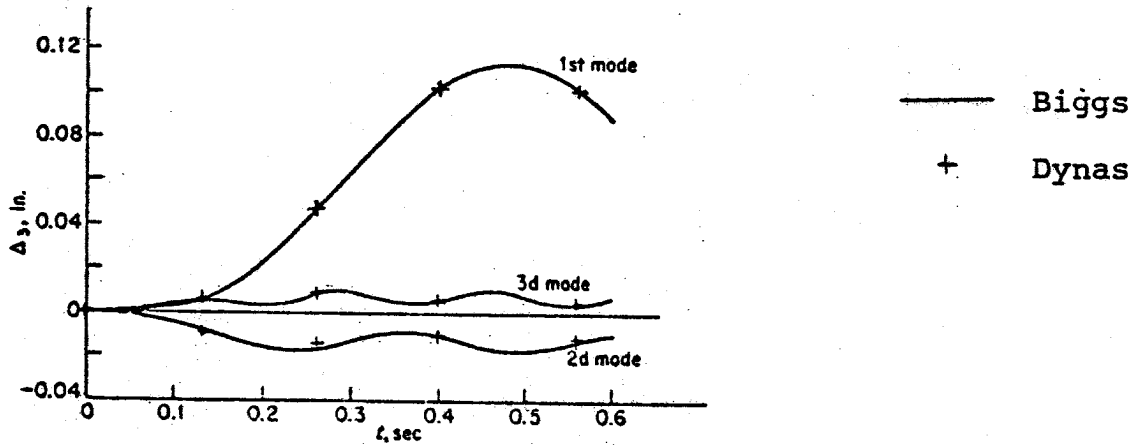
COMPARISON OF CANTILEVER RESPONSES FROM
DYNAS AND SAPIV (DYNAS)



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FIGURE C-10

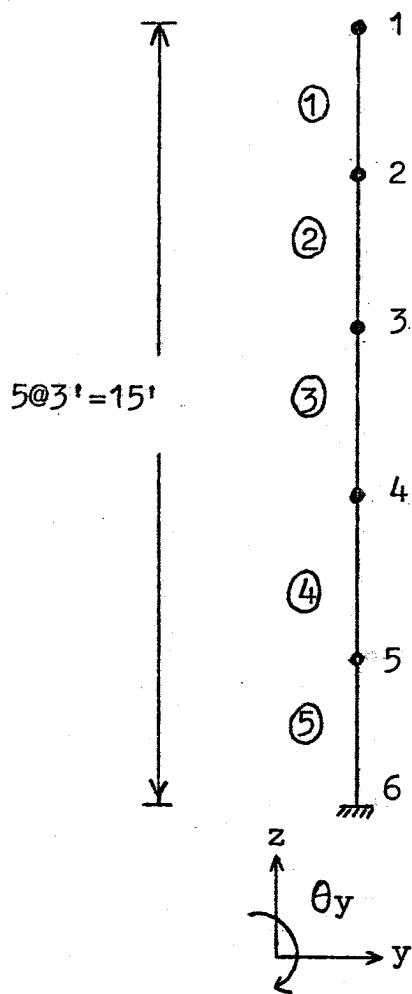
STEEL FRAME WITH RIGID GIRDERS FOR
 DYNAS VALIDATION PROBLEM NO. 3



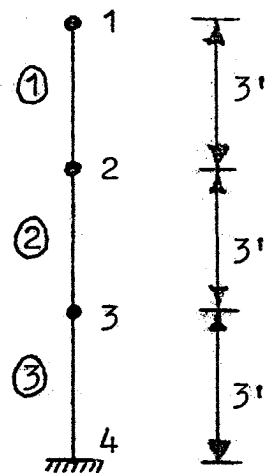
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FIGURE C-11

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



(a) Model 1



(b) Model 2

Members

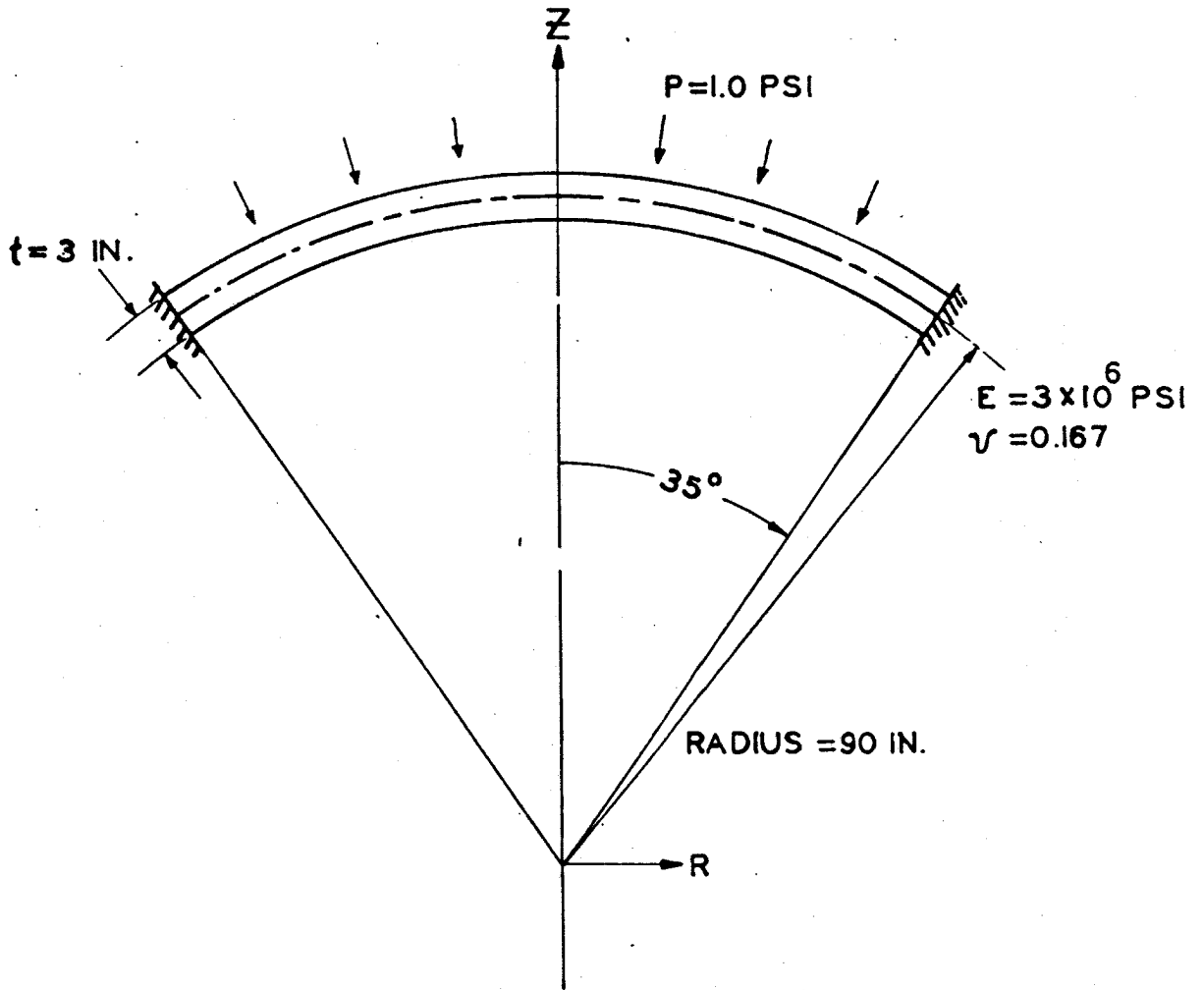
$A = 0.388 \text{ sq. ft.}$
 $I = 0.091 \text{ ft.}^4$

Damping ratio = 0.02

Masses

Along x = 20 k
 Along $\theta_y = 100 \text{ k/ft}^2$

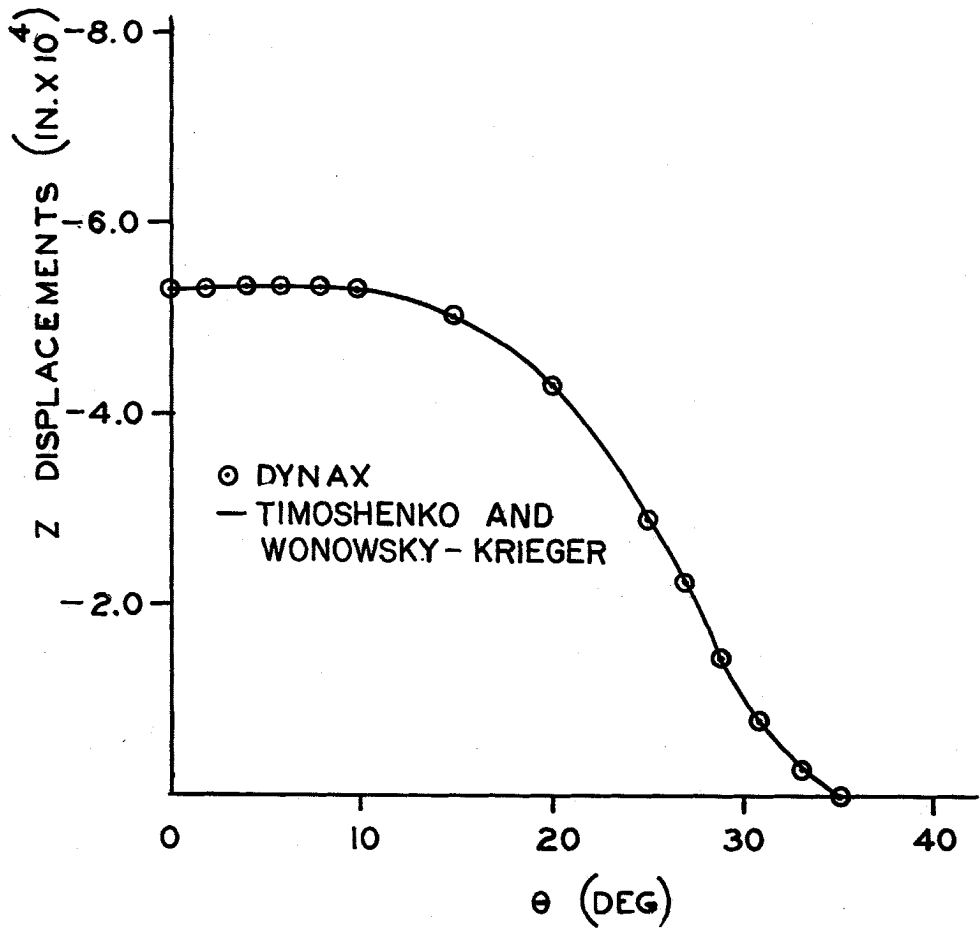
| |
|---|
| CLINTON POWER STATION UPDATED SAFETY ANALYSIS REPORT |
| FIGURE C-12 |
| 2-D CANTILEVER MODELS FOR DYNAS VALIDATION PROBLEM NO. 4 |



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FIGURE C-13

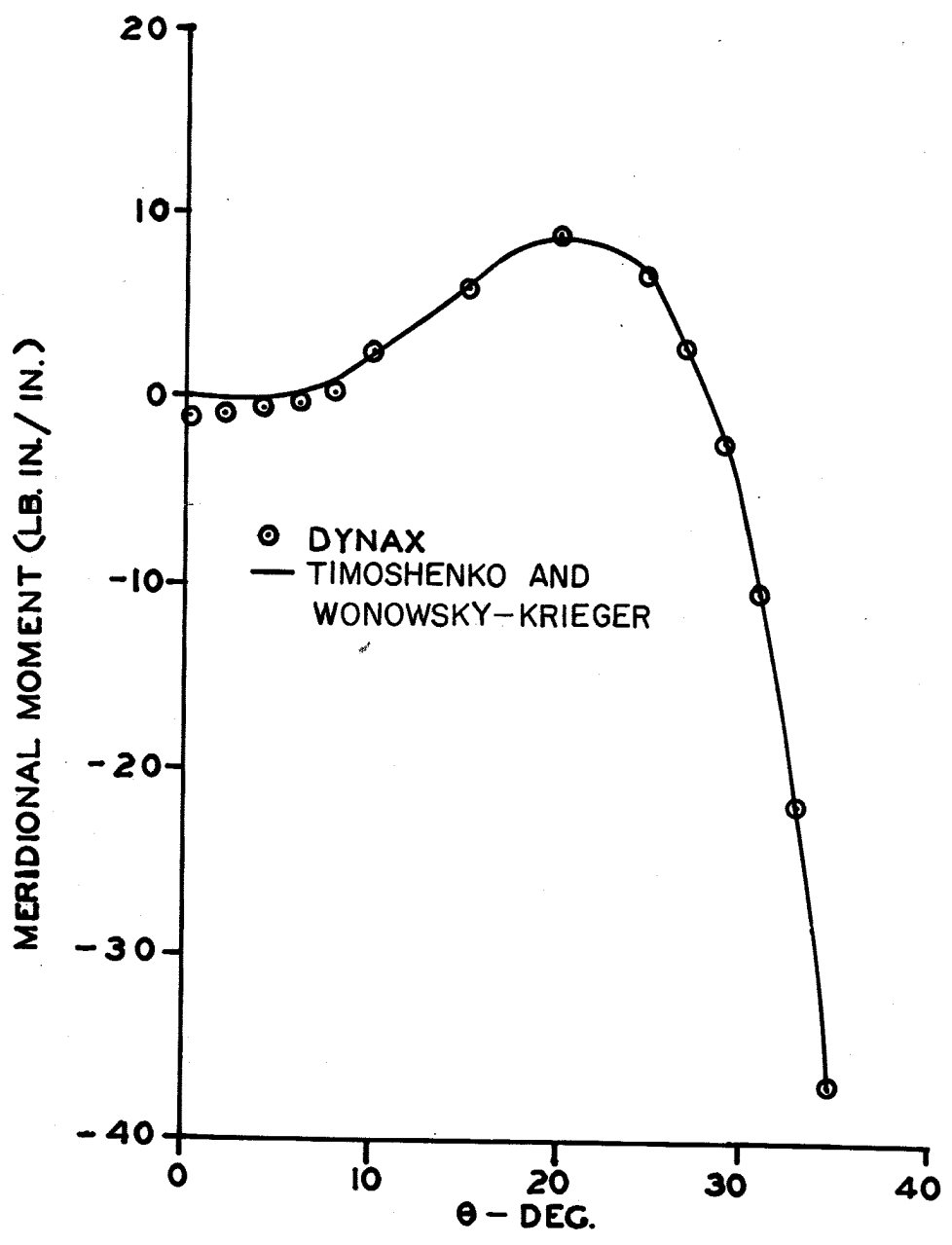
SHALLOW SPHERICAL SHELL ANALYZED BY
DYNAX - VALIDATION PROBLEM 1



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FIGURE C-14

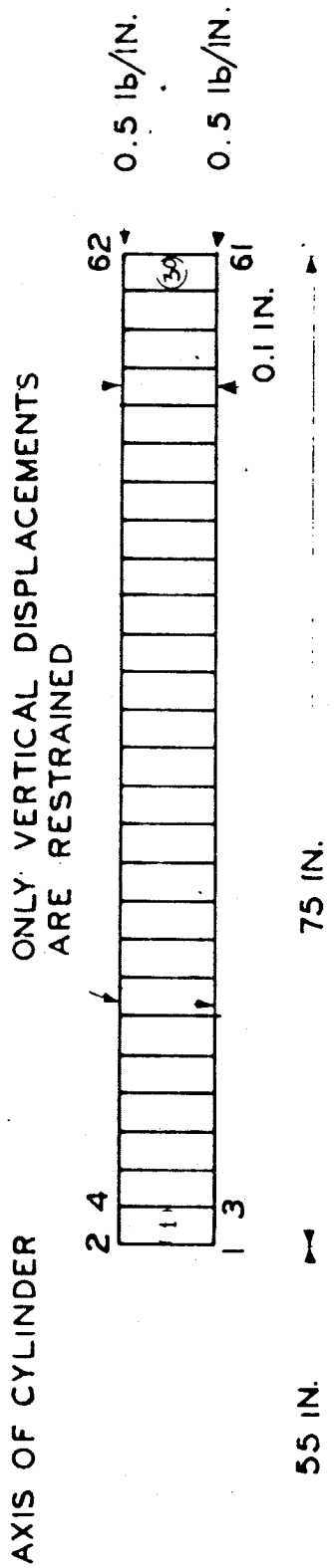
AXIAL DISPLACEMENT
 SHALLOW SPHERICAL SHELL



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FIGURE C-15

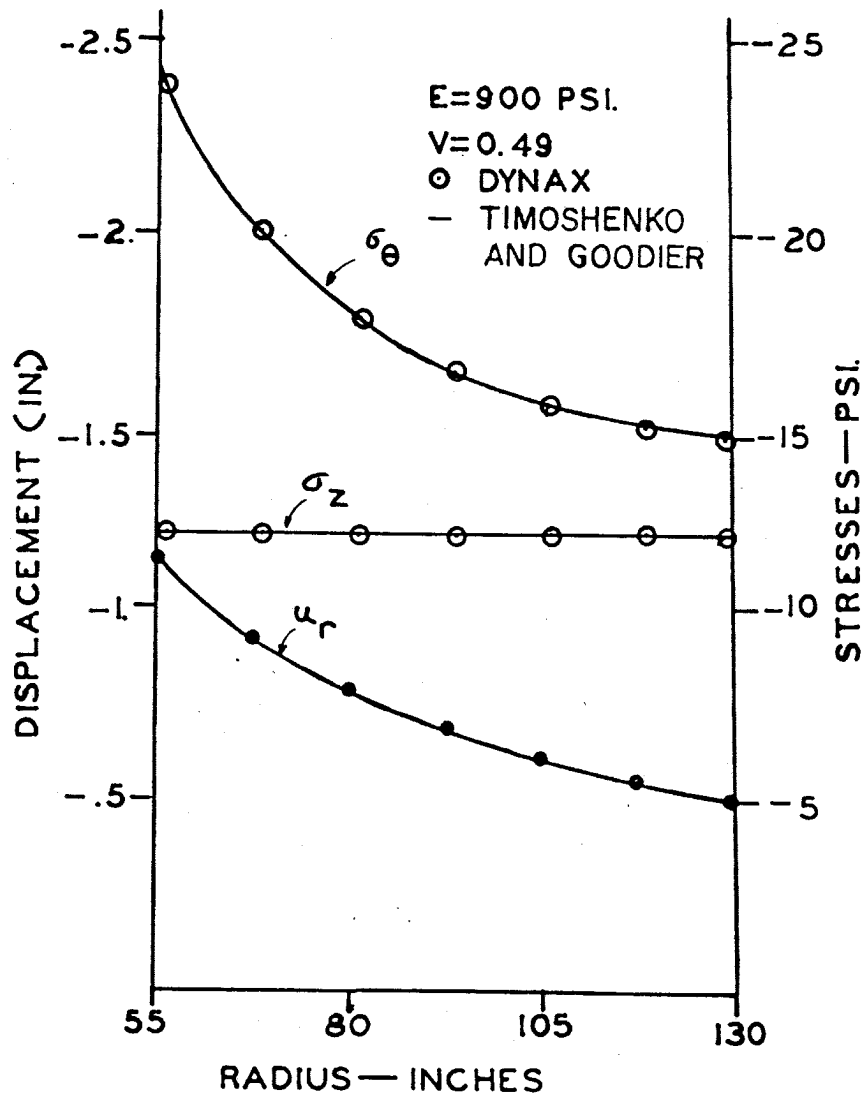
MERIDIONAL MOMENT SHALLOW SPHERICAL SHELL



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FIGURE C-16

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

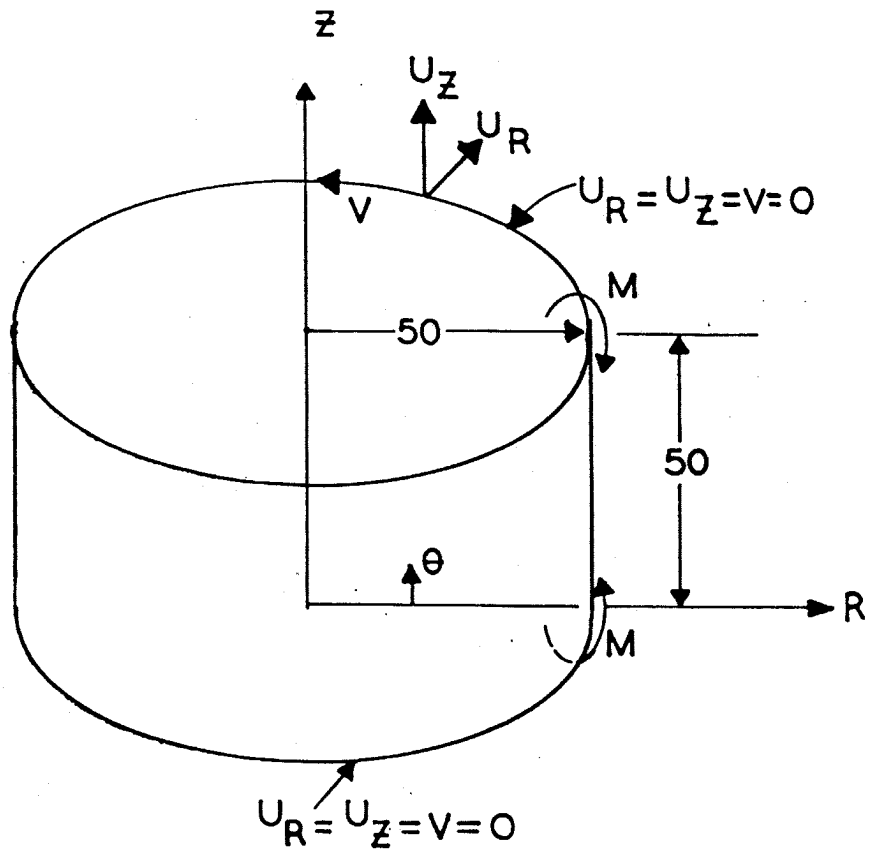


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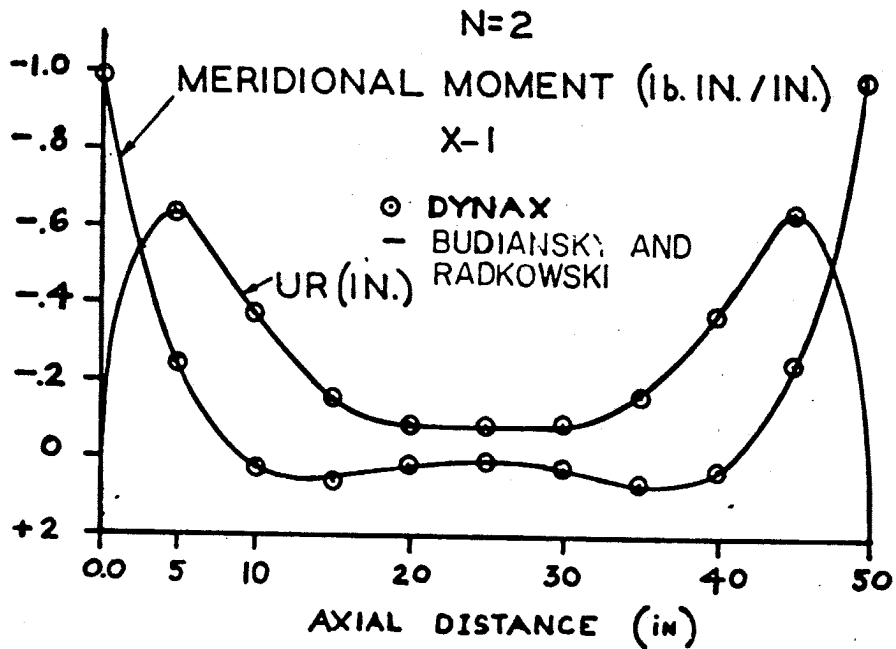
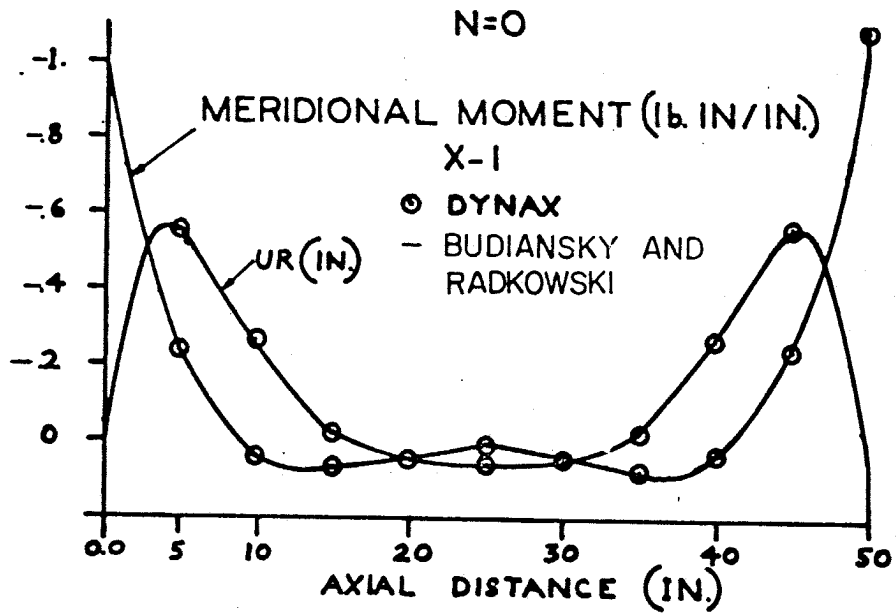
FIGURE C-17

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

$T = \text{SHELL THICKNESS} = 1 \text{ IN.}$
 $M = 1 \text{ LB.-IN./IN.}$
 $E = 91. \text{ LB/IN}^2$
 $\nu = .3$
 $N = \text{FOURIER HARMONIC NUMBER}$



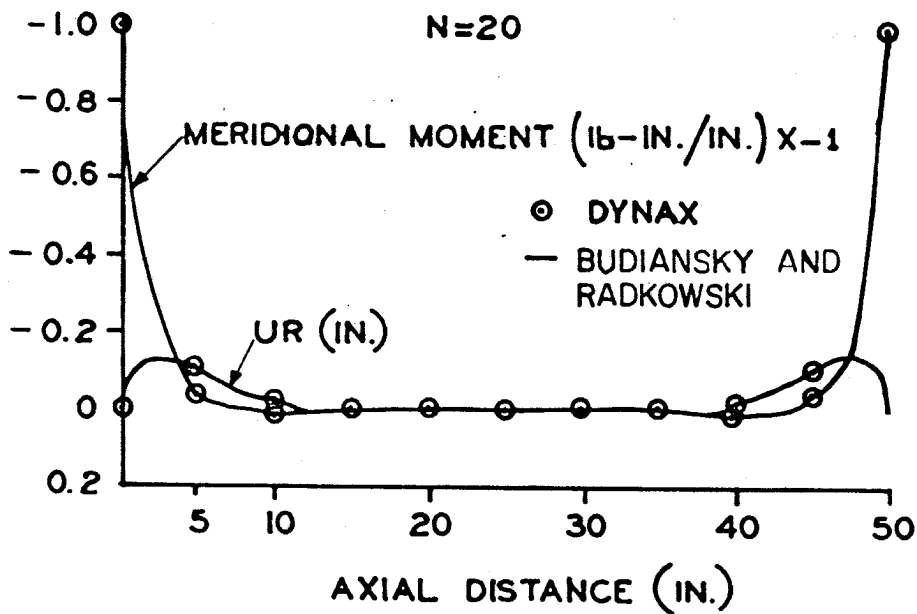
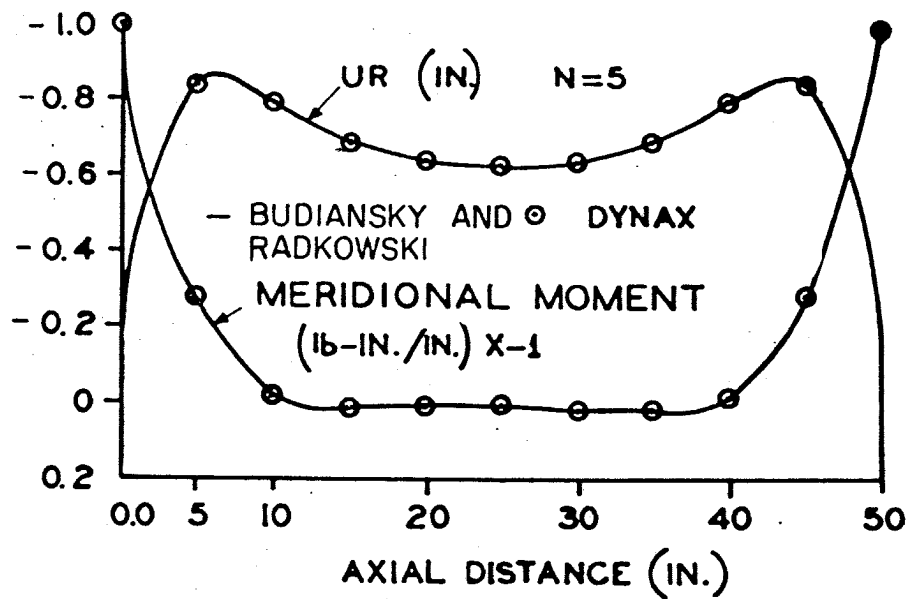
| |
|---|
| CLINTON POWER STATION UPDATED SAFETY ANALYSIS REPORT |
| FIGURE C-18 CYLINDER UNDER HARMONIC LOADS ANALYZED BY DYNAX - VALIDATION PROBLEM 3 |



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UPDATED SAFETY ANALYSIS REPORT

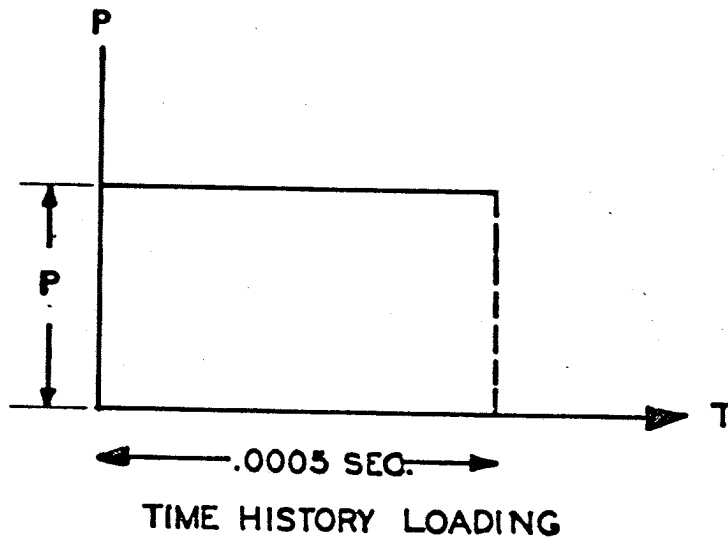
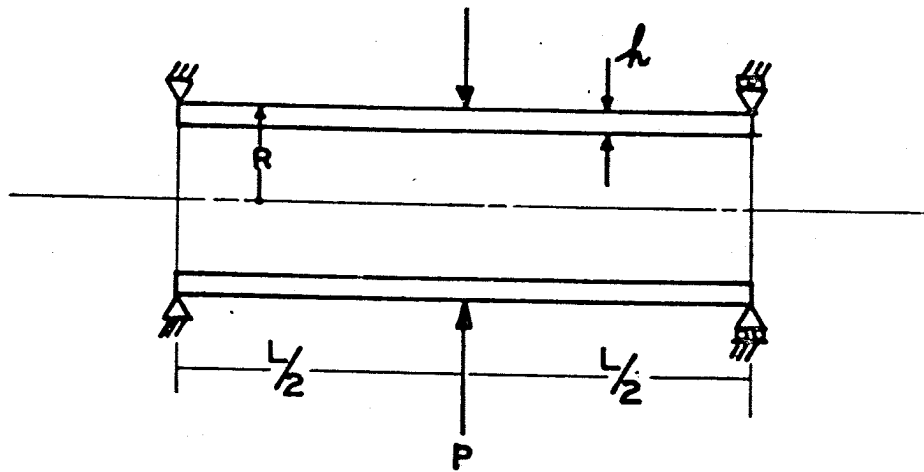
FIGURE C-19

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)

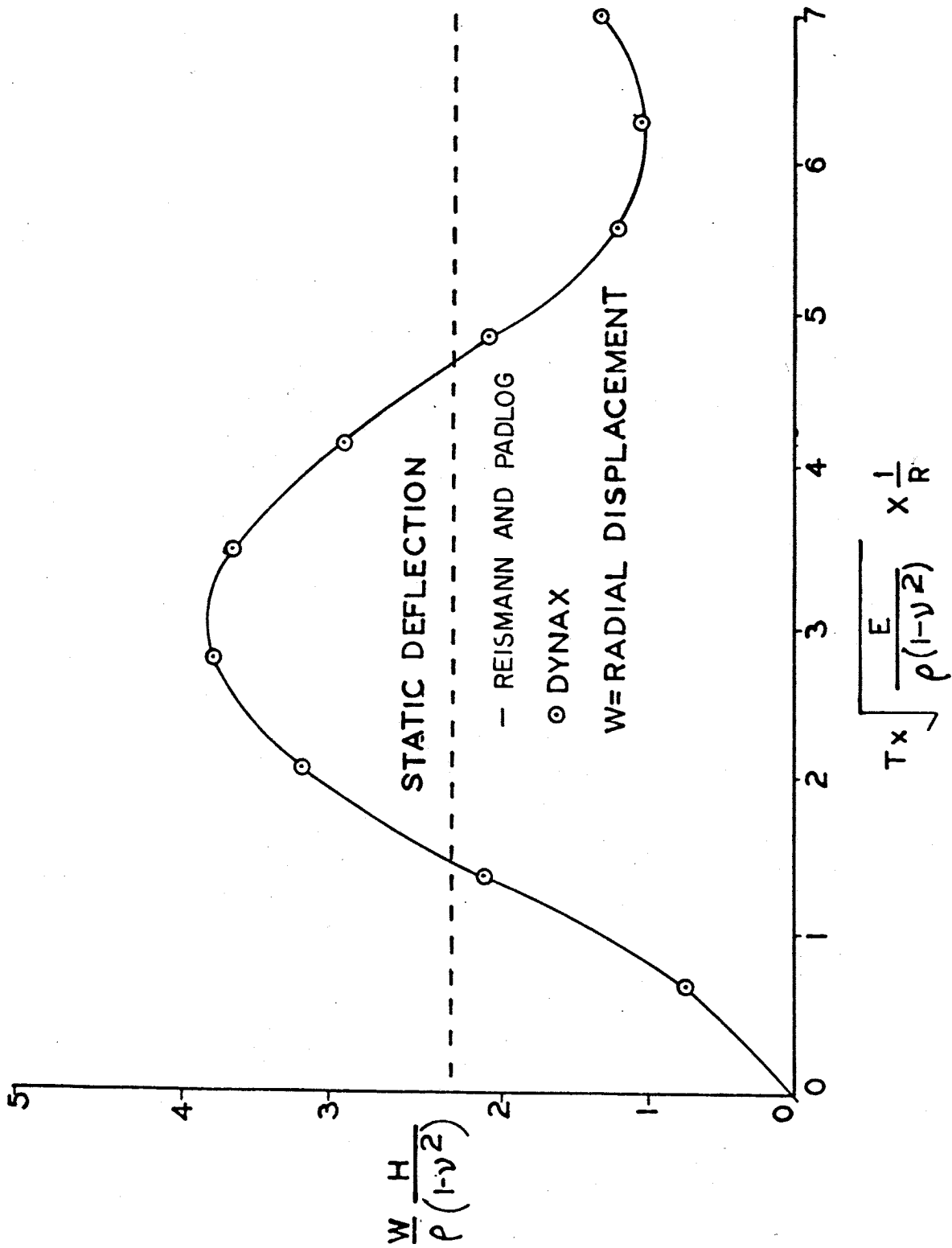


$L = 18 \text{ IN.}$
 $P = 500 \text{ lb.}$
 $R = 3 \text{ IN.}$
 $h = 0.3 \text{ IN.}$
 $E = 30 \times 10^6 \text{ lb. / IN.}^2$

MASS DENSITY (ρ) = $0.0187 \frac{\text{# SEC.}^2}{\text{IN.}^4}$
 $\nu = 0.3$
 TIME STEP = $.000005 \text{ SEC.}$

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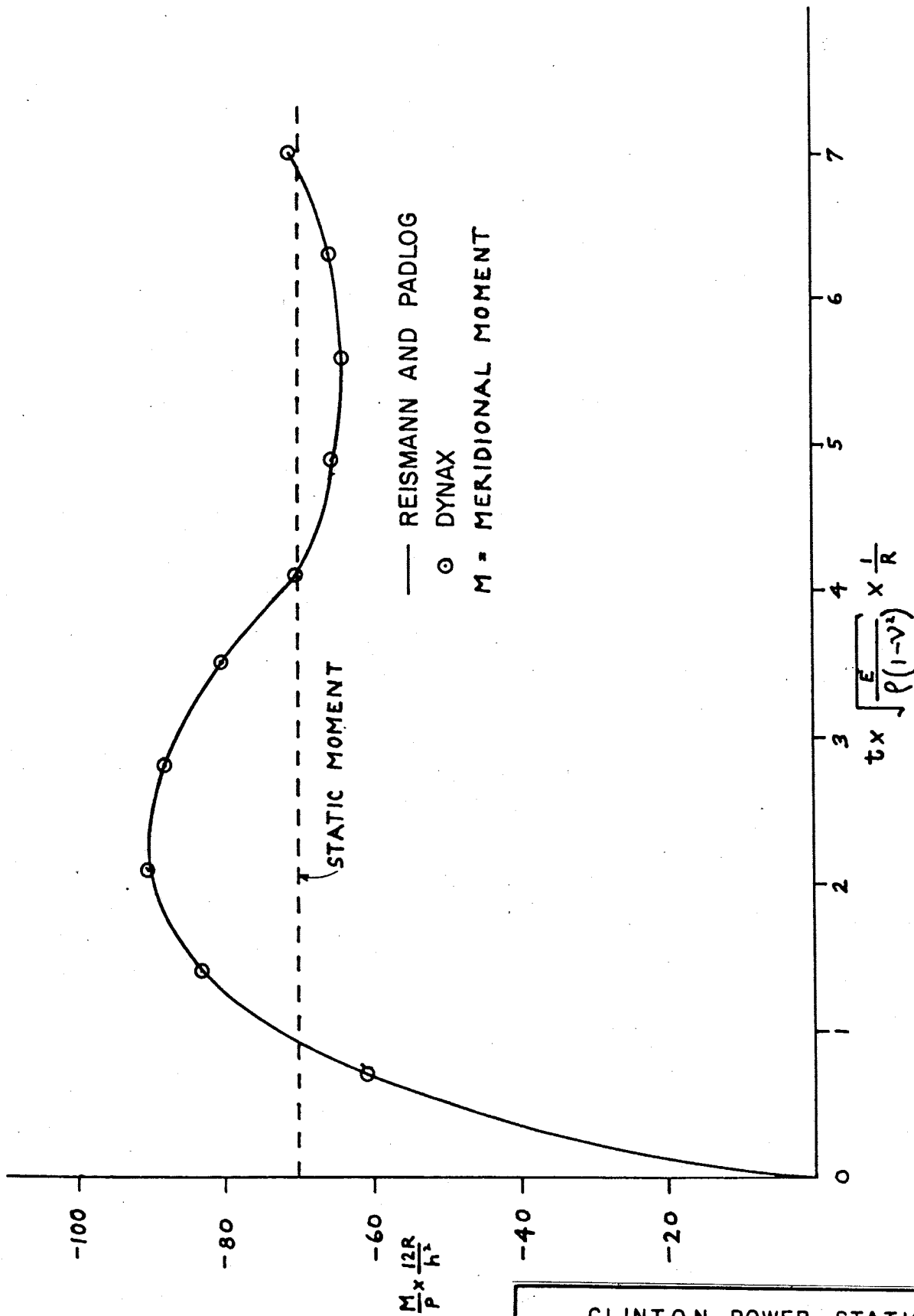
FIGURE C-21
 SUDDENLY APPLIED RING LINE LOAD
 ANALYZED BY DYNAX - VALIDATION PROBLEM 4



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FIGURE C-22

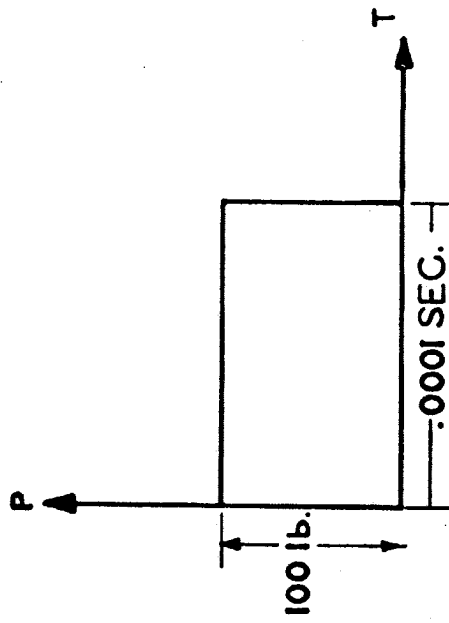
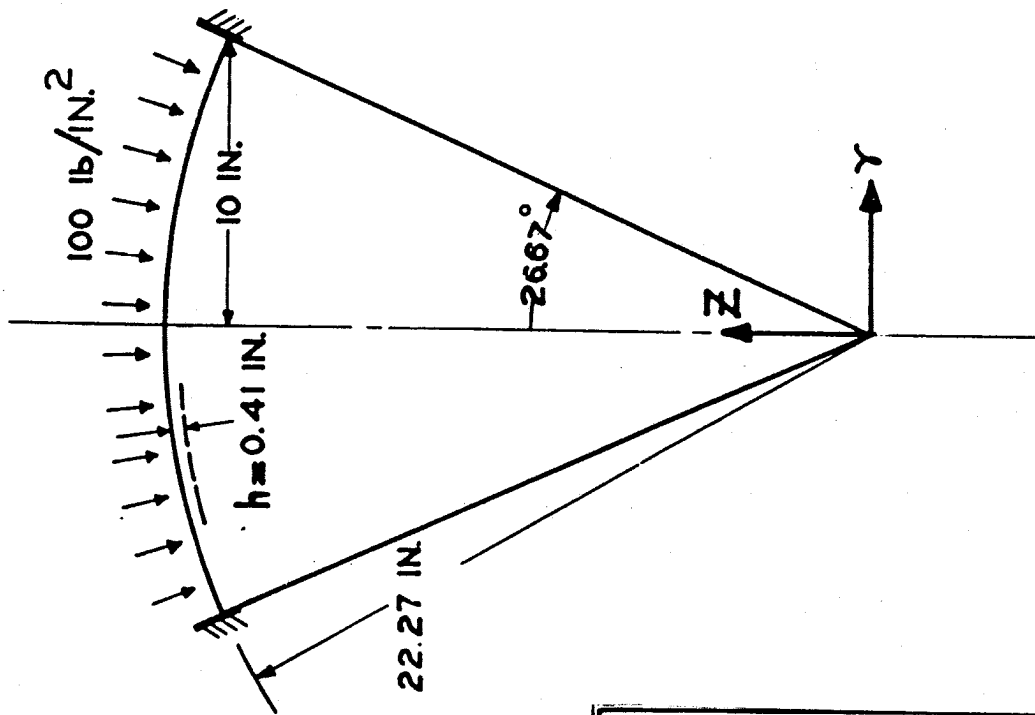
RADIAL DISPLACEMENT VS. TIME COMPARISON
 OF RESULTS FROM DYNAX AND REFERENCE 12



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FIGURE C-23

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



TIME HISTORY LOADING

$$E = 10.5 \times 10^6 \text{ PSI}$$

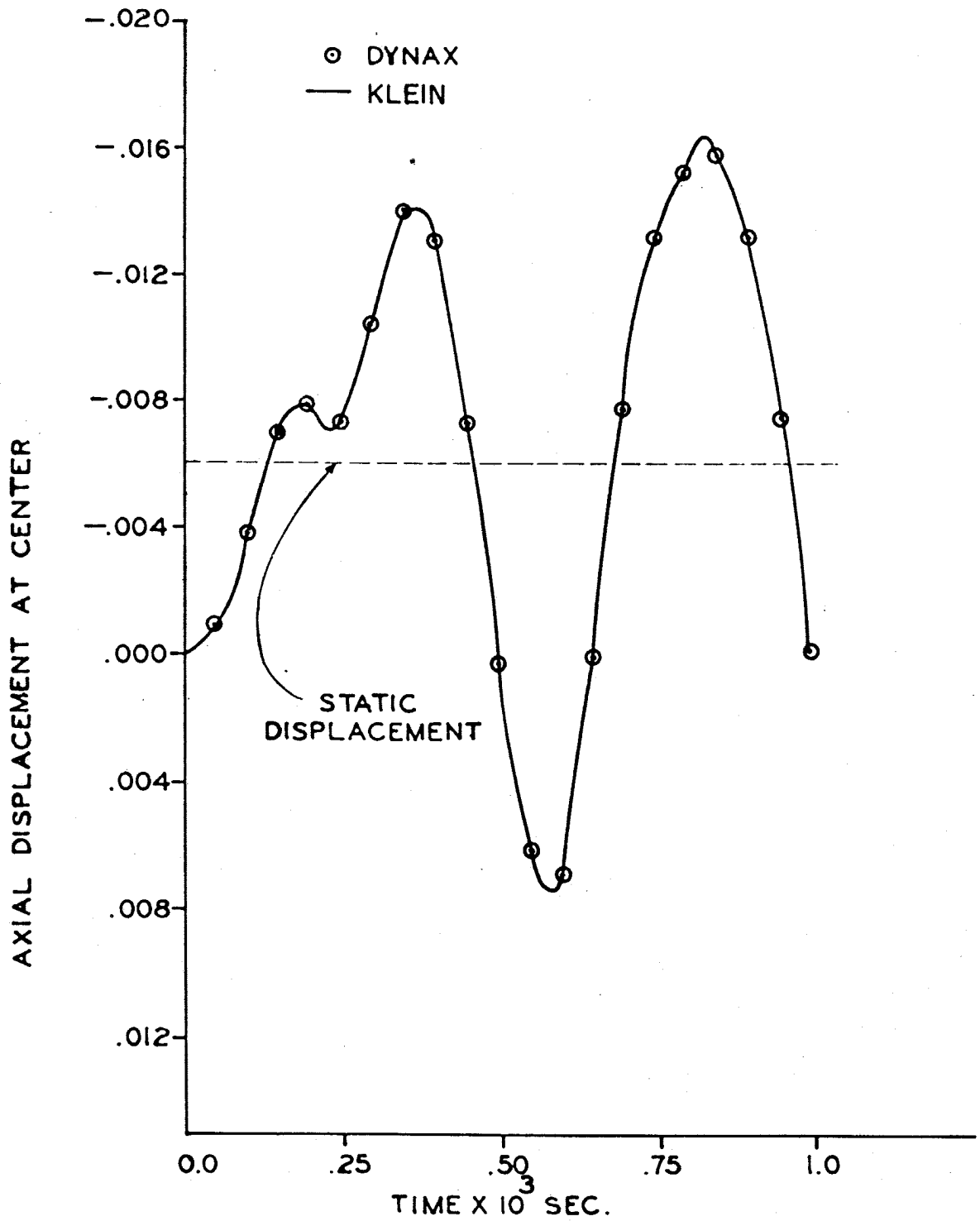
$$\nu = .3$$

$$P = 2.46 \times 10^{-4} \text{ lb.-SEC}^2/\text{IN.}$$

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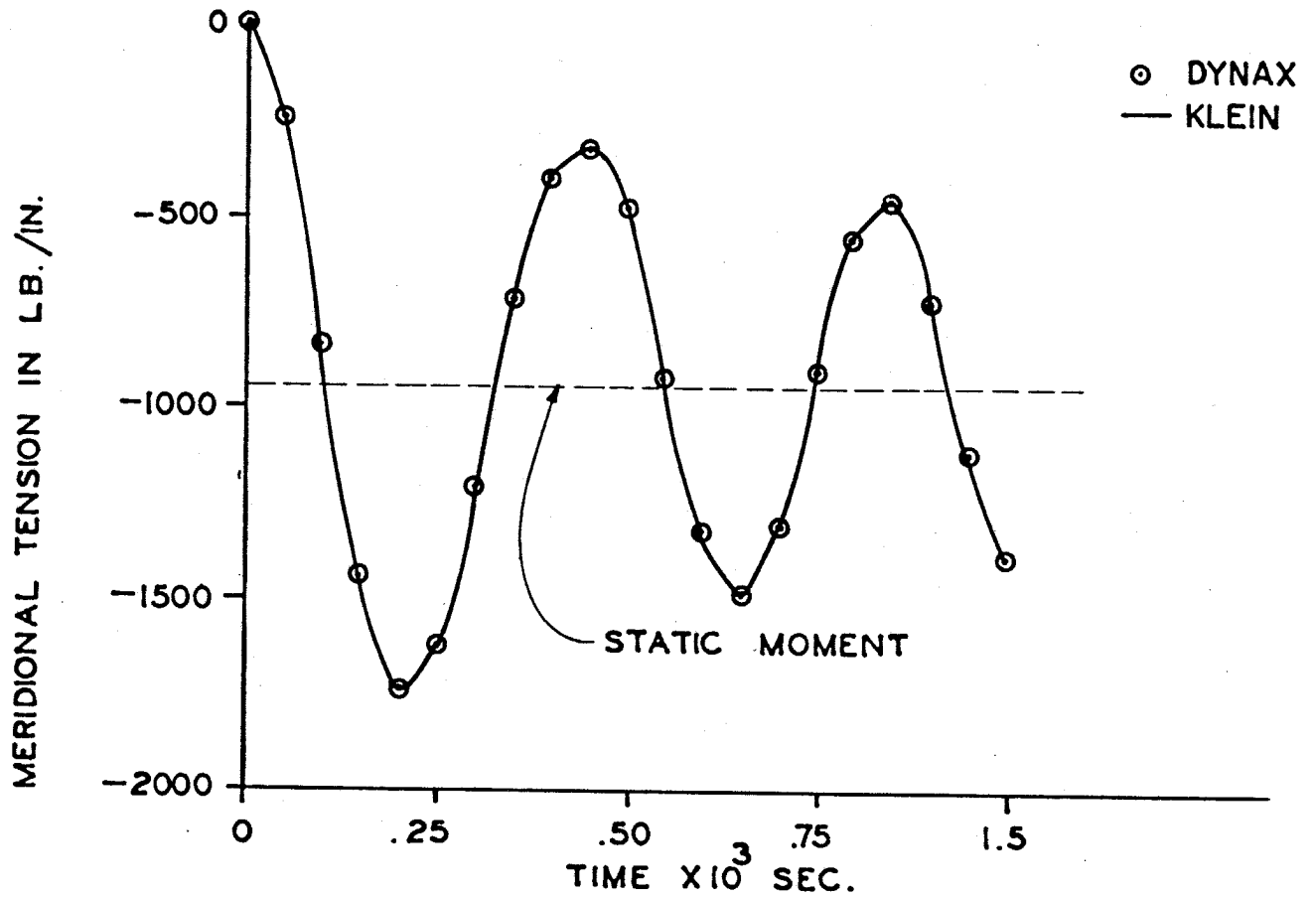
FIGURE C-24

SPHERICAL CAP ANALYZED BY DYNAX -
 VALIDATION PROBLEM 5



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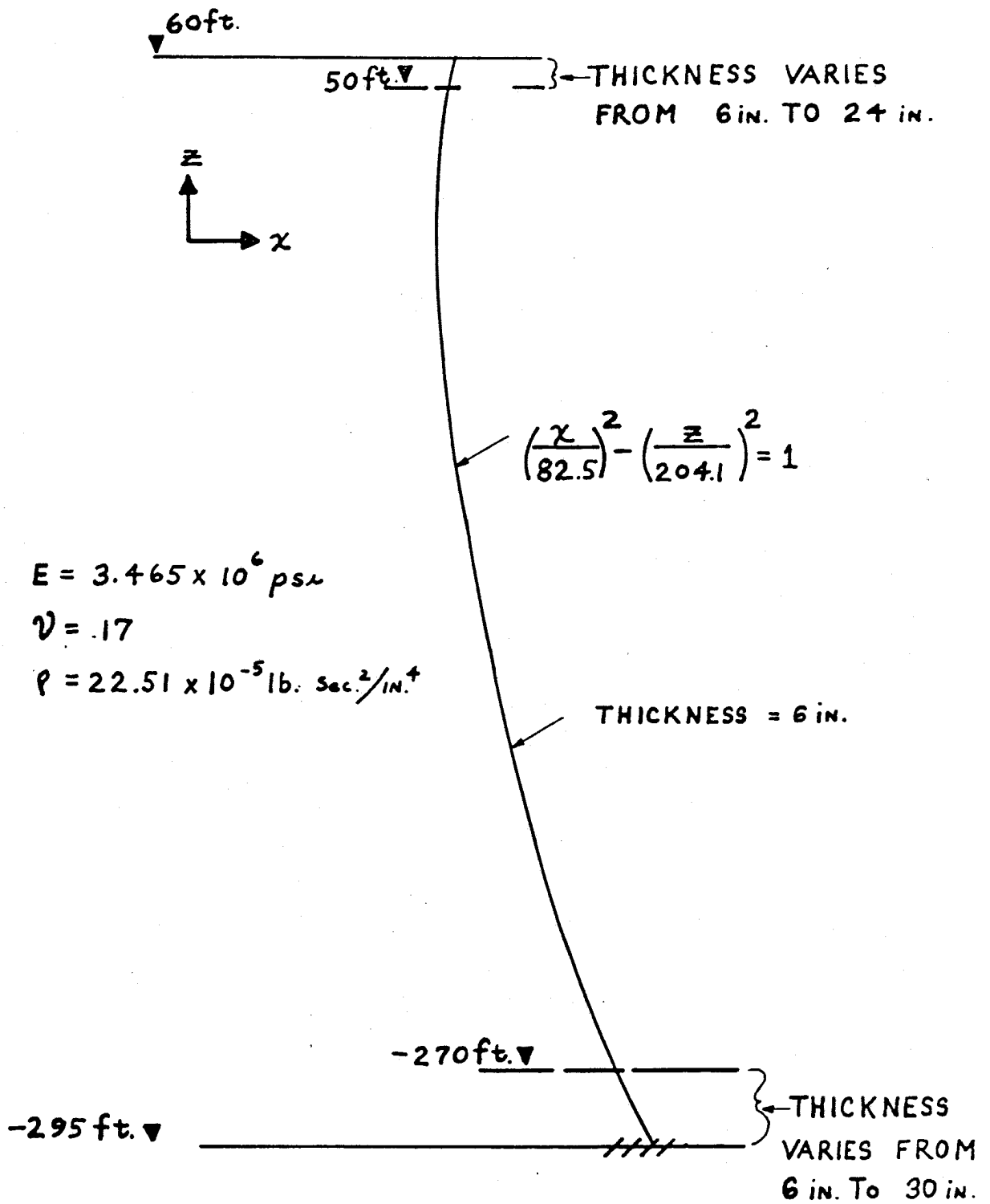
FIGURE C-25
 COMPARISON OF RESULTS FROM DYNAX AND
 REFERENCE 13 OF AXIAL DISPLACEMENT
 OF SPHERICAL CAP UNDER DYNAMIC LOAD



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FIGURE C-26

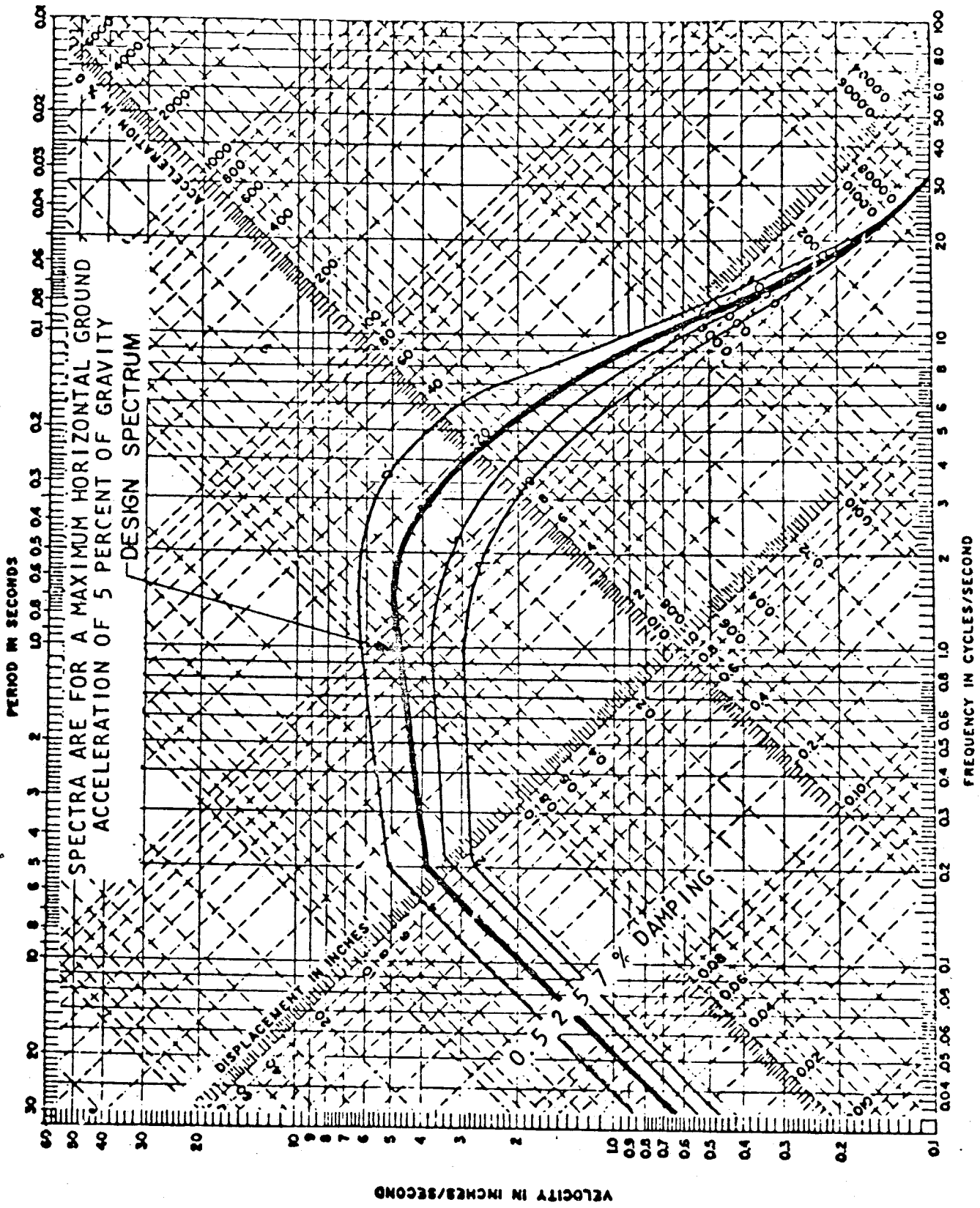
COMPARISON OF RESULTS FROM DYNAX AND
REFERENCE 13 OF MERIDIONAL TENSION
OF SPHERICAL CAP UNDER DYNAMIC LOAD



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FIGURE C-27

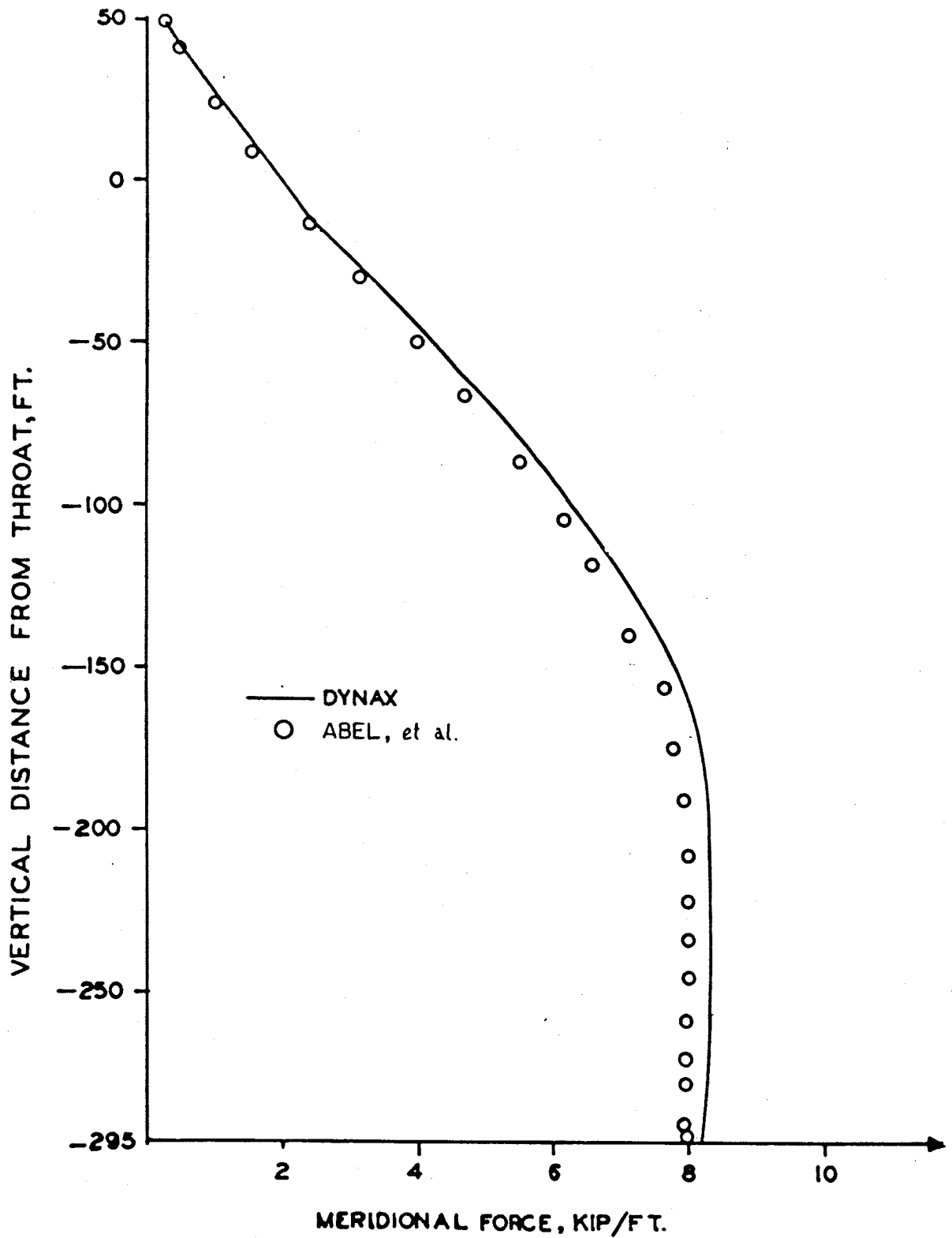
HYPERBOLIC COOLING TOWER ANALYZED BY
 DYNAX - VALIDATION PROBLEM 6



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FIGURE C-28

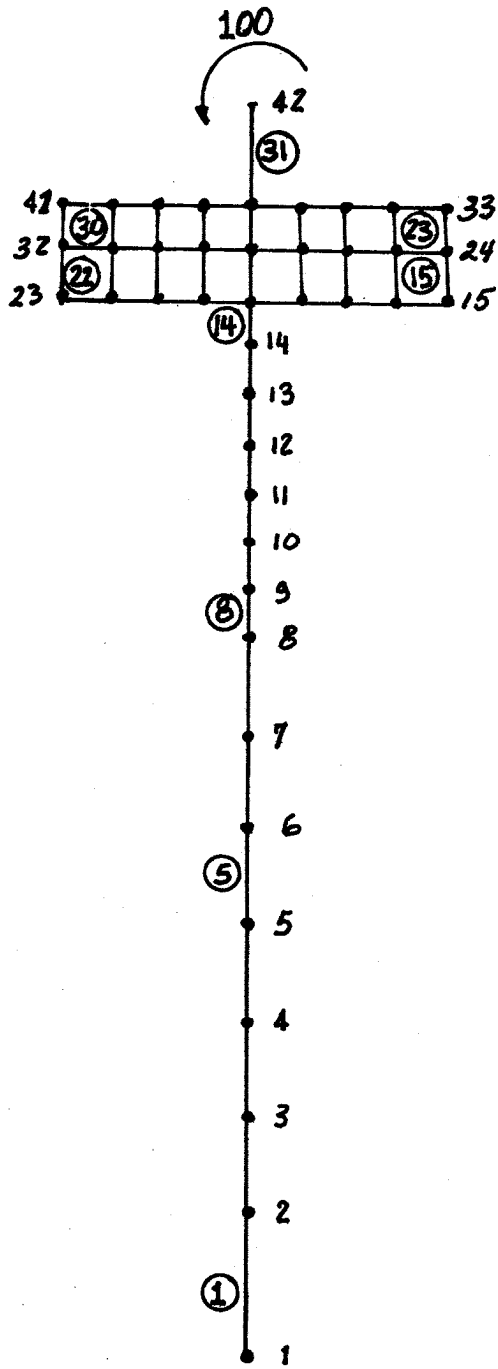
SPECTRUM OF DESIGN EARTHQUAKE USED FOR
 DYNAX - VALIDATION PROBLEM 6



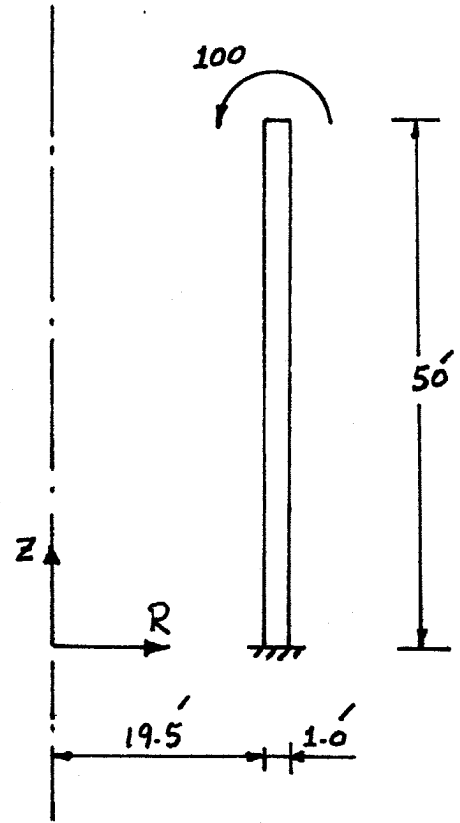
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FIGURE C-29

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



Finite Element Model

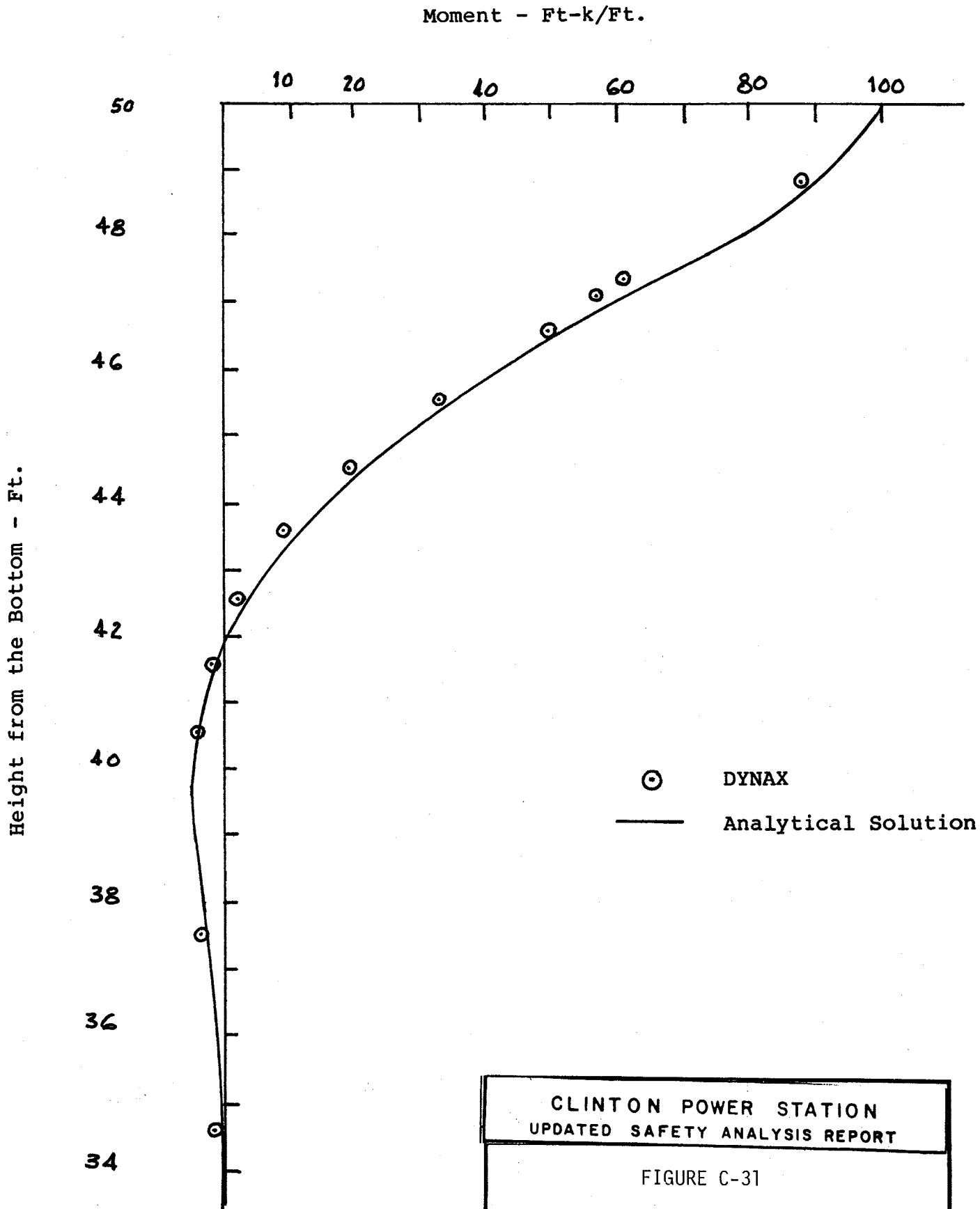


Structure

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FIGURE C-30

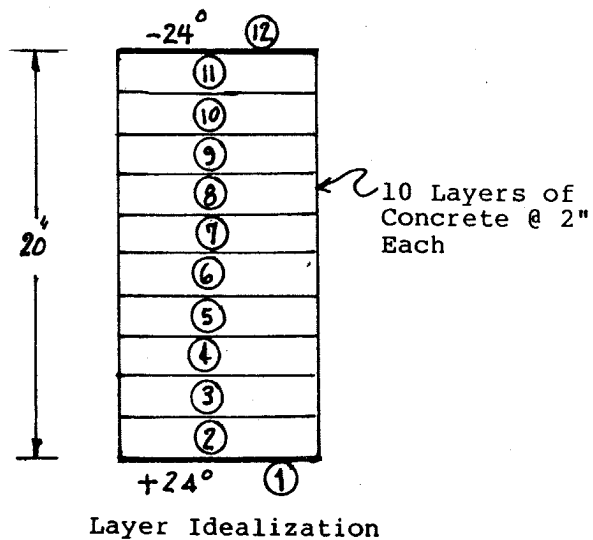
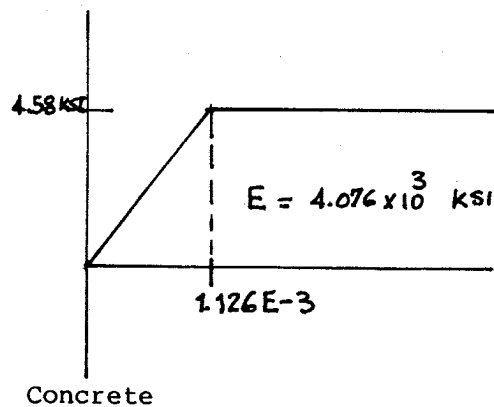
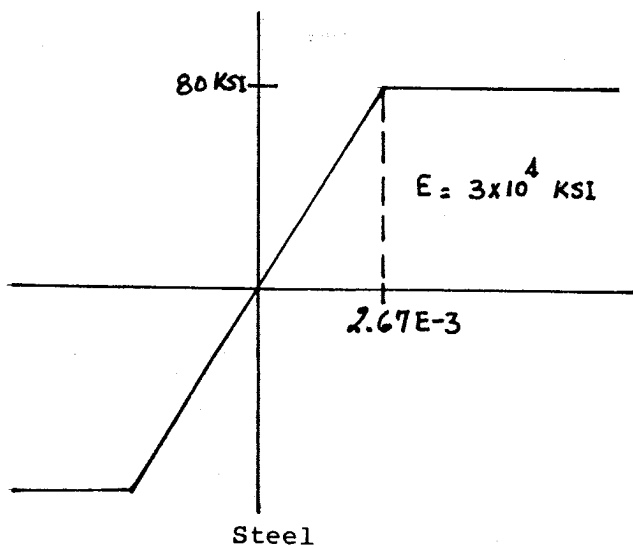
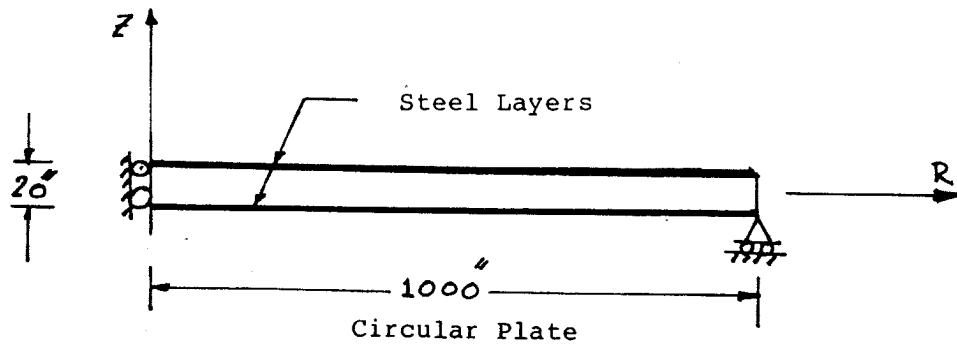
TYING OF SOLID AND SHELL ELEMENTS
 ANALYZED BY DYNAX - VALIDATION PROBLEM 7



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FIGURE C-31

MOMENT DIAGRAM OF RESULTS FROM DYNAX
 AND ANALYTICAL SOLUTION

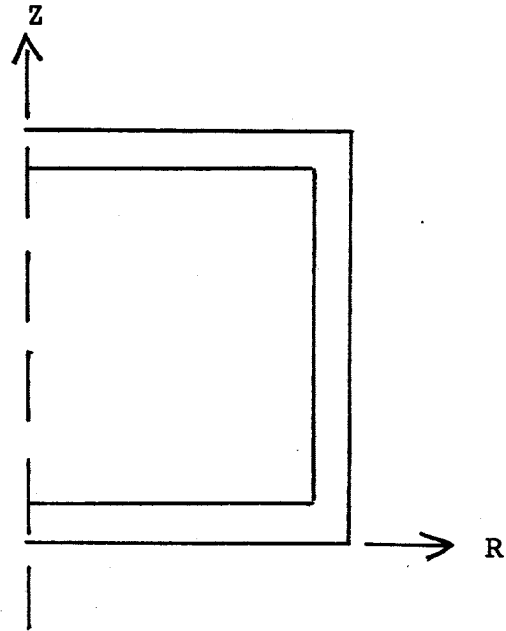


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FIGURE C-32

CIRCULAR PLATE ANALYZED BY DYNAX -
 VALIDATION PROBLEM 8

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

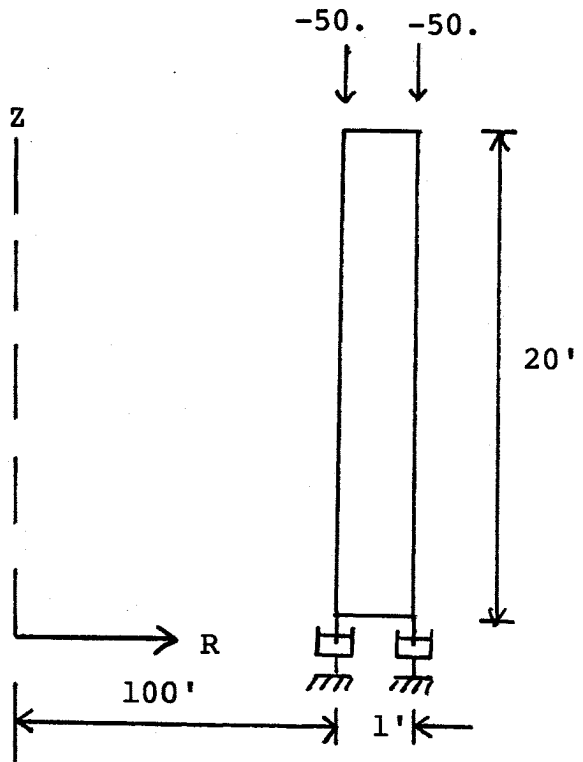


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FIGURE C-33

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

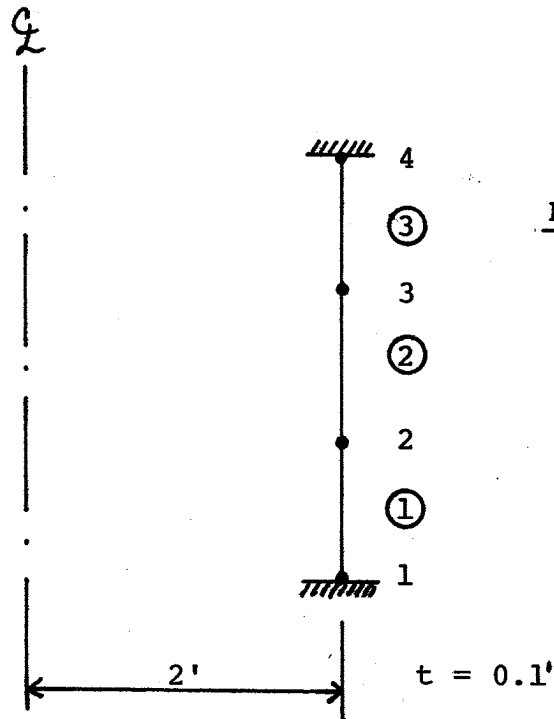
E = 100
ρ = 1
ν = 0



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FIGURE C-34

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

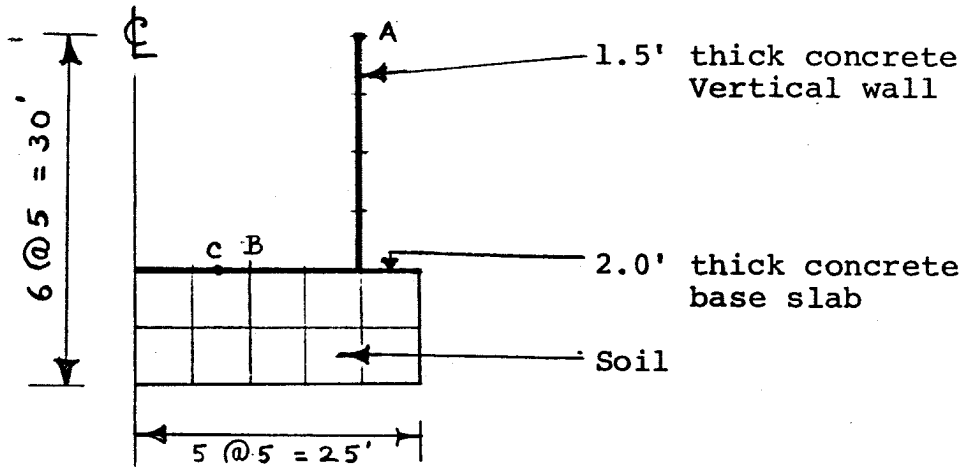


| <u>Element</u> | <u>Material Damping Coefficient, β</u> |
|----------------|---|
| 1 | 0.04 |
| 2 | 0.05 |
| 3 | 0.03 |

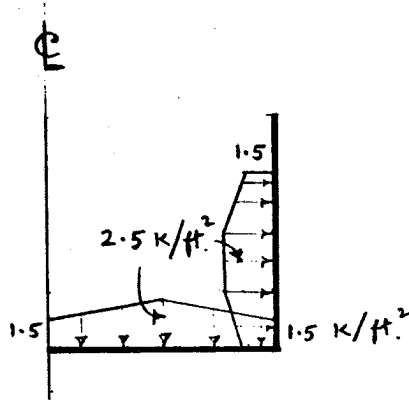
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FIGURE C-35

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



(a) Model

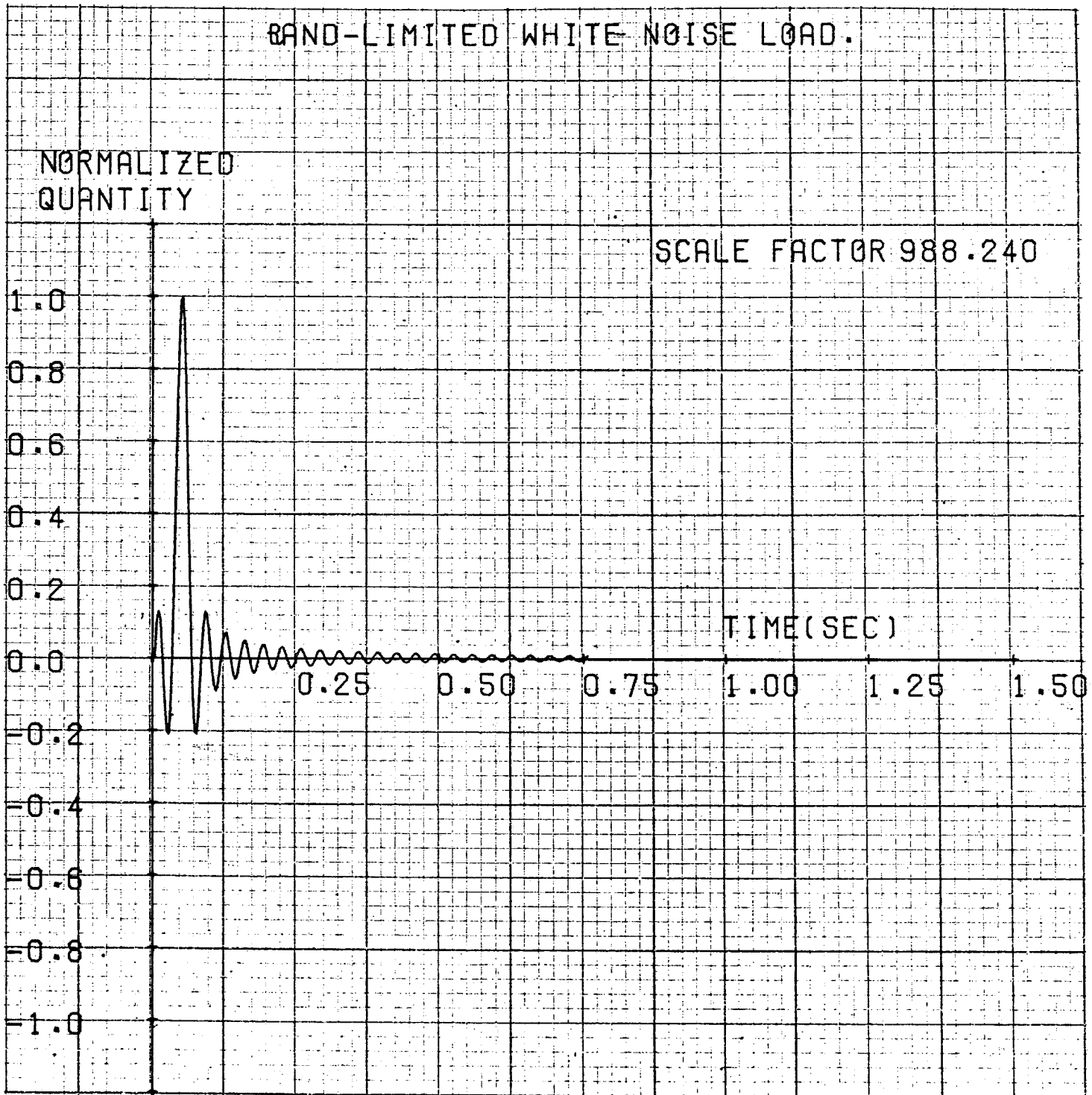


(b) Meridional Load Distribution

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FIGURE C-36

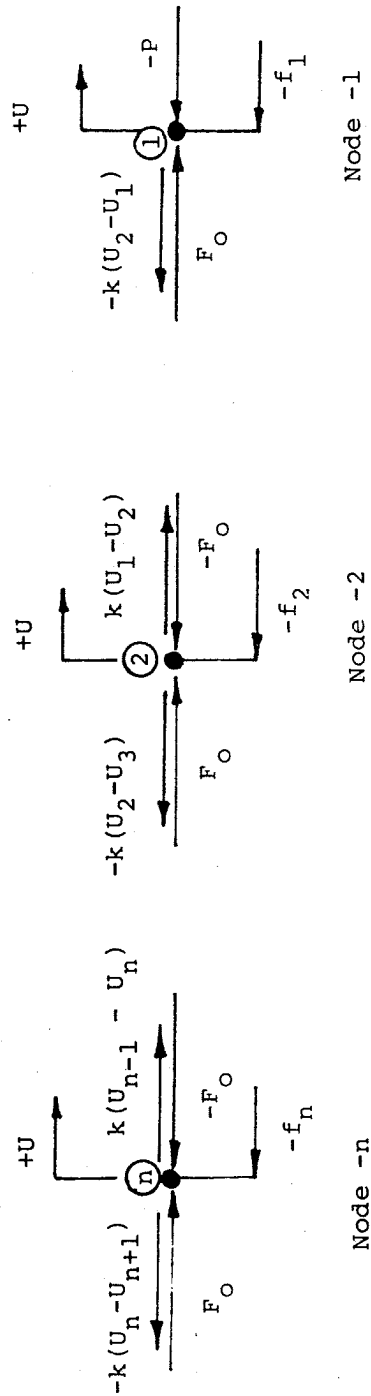
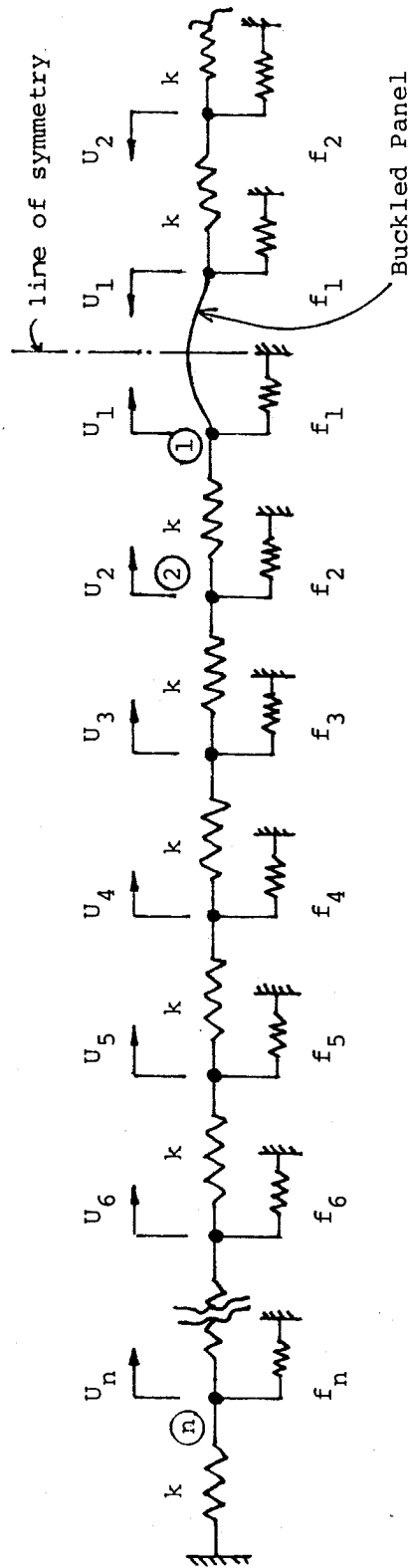
MODELING AND LOAD DISTRIBUTION



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FIGURE C-37

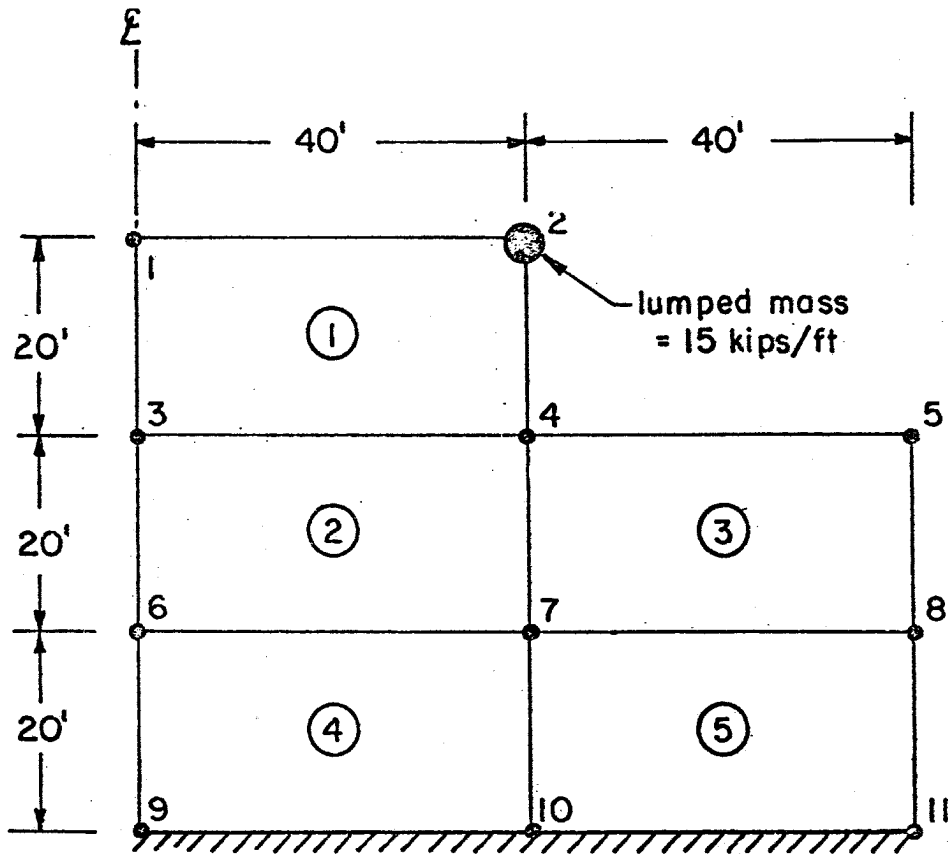
TIME HISTORY OF LOAD $\omega(t)$



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FIGURE C-38

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



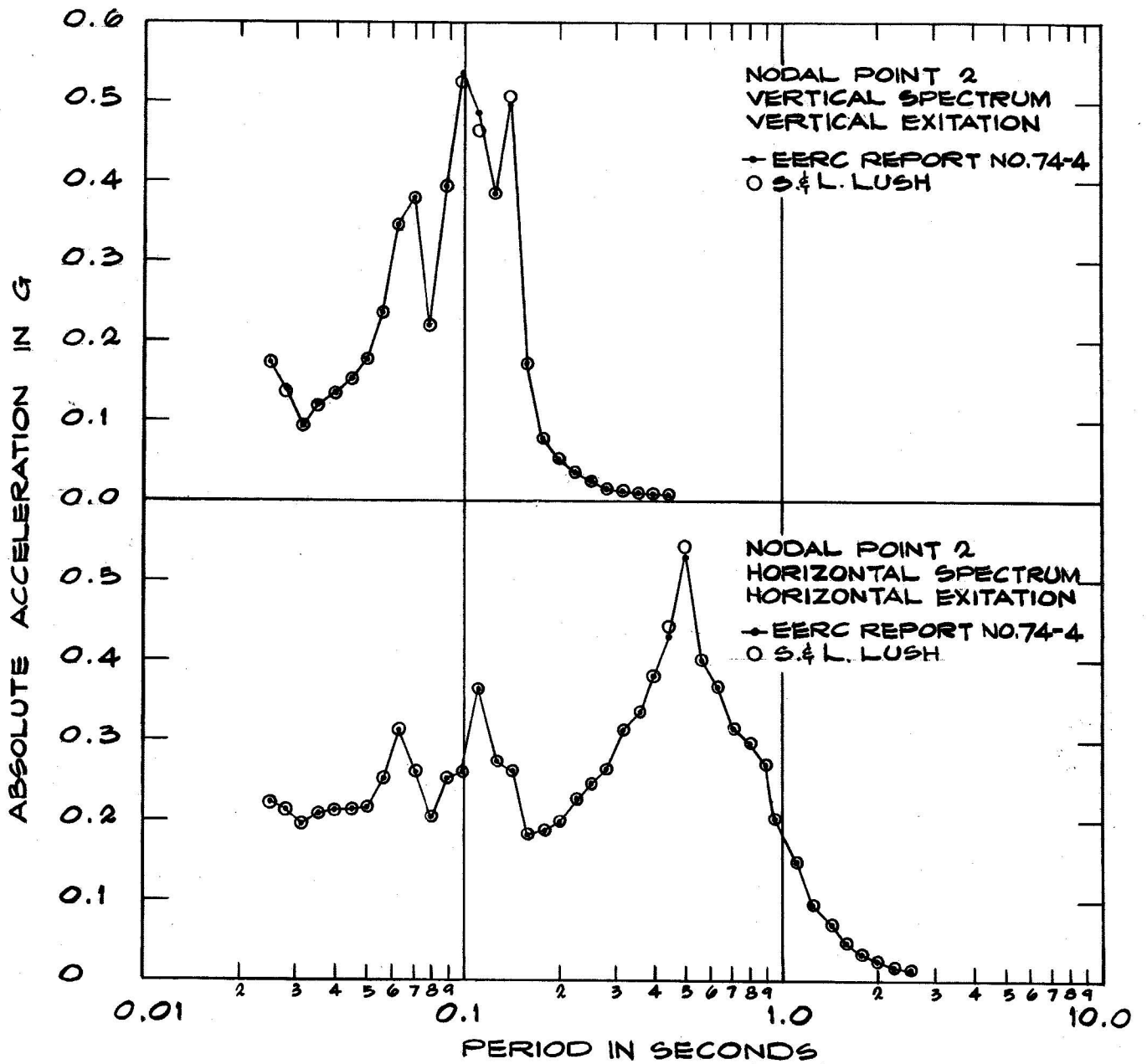
Nodal points 4 and 5 linked by rigid element

| 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

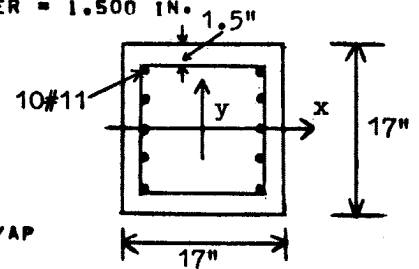
VALIDATION PROBLEM NO.1 - DESIGN OF A TIED COLUMN -COMPRESSION CONTROL

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 | | | ULTIMATE CAPACITY | | | UP/AP |
|-----------|----------------|-----|------|-------------------|-----|------|-------|
| | AP | AMX | AMY | UP | UMX | UMY | |
| 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

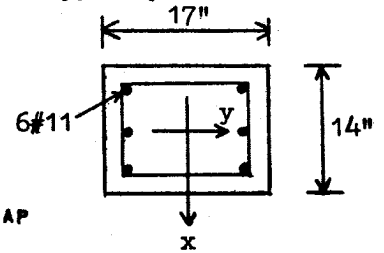
VALIDATION PROBLEM NO. 2 - DESIGN OF A TIED COLUMN - TENSION CONTROLS

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 | | | ULTIMATE CAPACITY | | | UP/AP |
|-----------|----------------|------|-----|-------------------|------|-----|-------|
| | AP | AMX | AMY | UP | UMX | UMY | |
| 1 | 115. | 279. | 0. | 122. | 295. | 0. | 1.057 |
| 2 | 115. | 0. | 14. | 801. | 0. | 94. | 6.966 |

INTERACTION CONTROL POINTS REQUESTED

| | PZ | PR | MR | 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.O. 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 CASE | APPLIED FORCES | | | ULTIMATE CAPACITY | | | UP/AP |
|--------------|----------------|------|------|-------------------|------|------|-------|
| | AP | AMX | AMY | UP | UMX | UMY | |
| 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 |

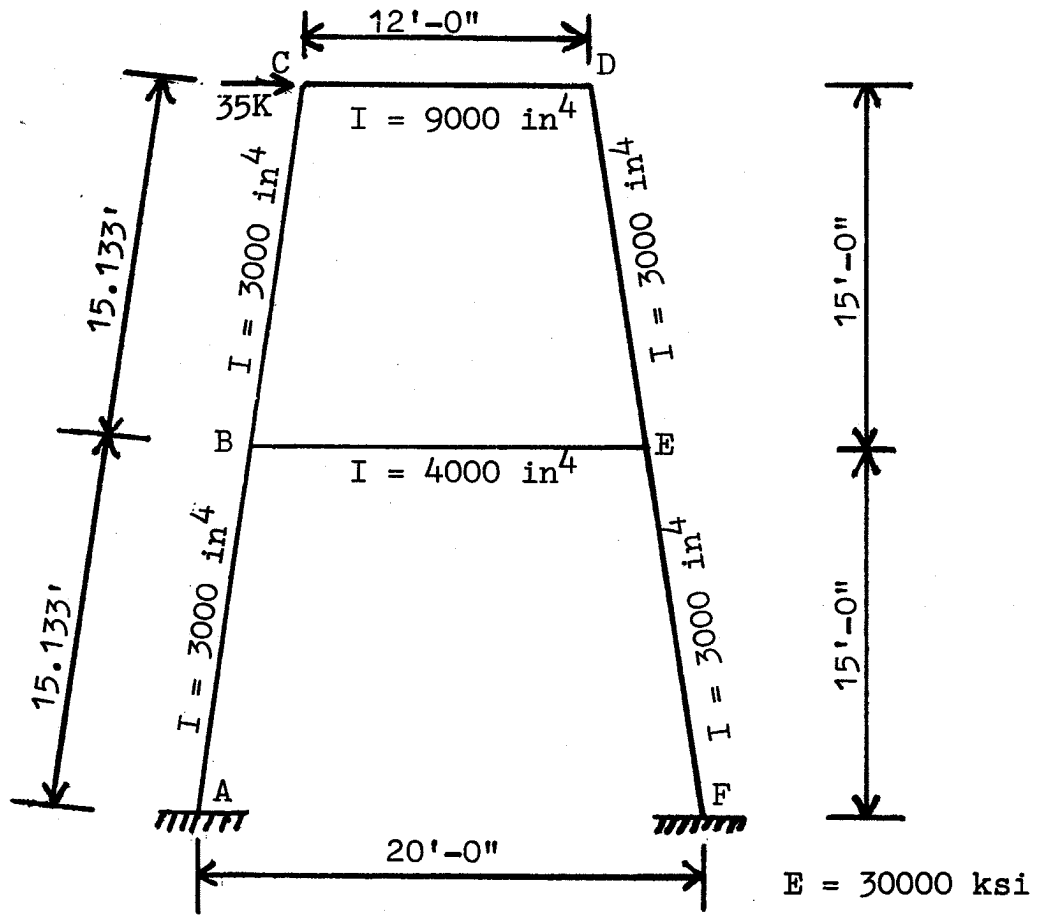
INTERACTION CONTROL POINTS REQUESTED

| | 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 |

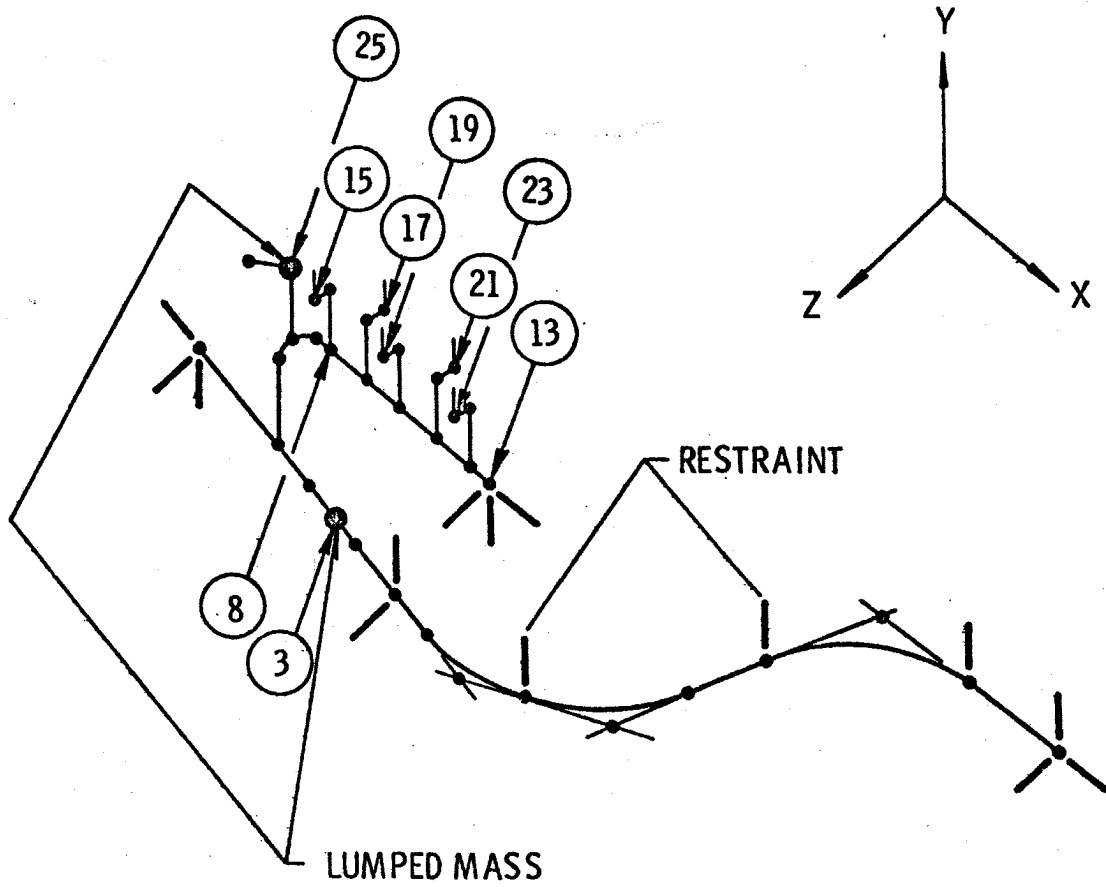
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FIGURE C-43

DESIGN OF TIED COLUMN -
BIAXIAL BENDING



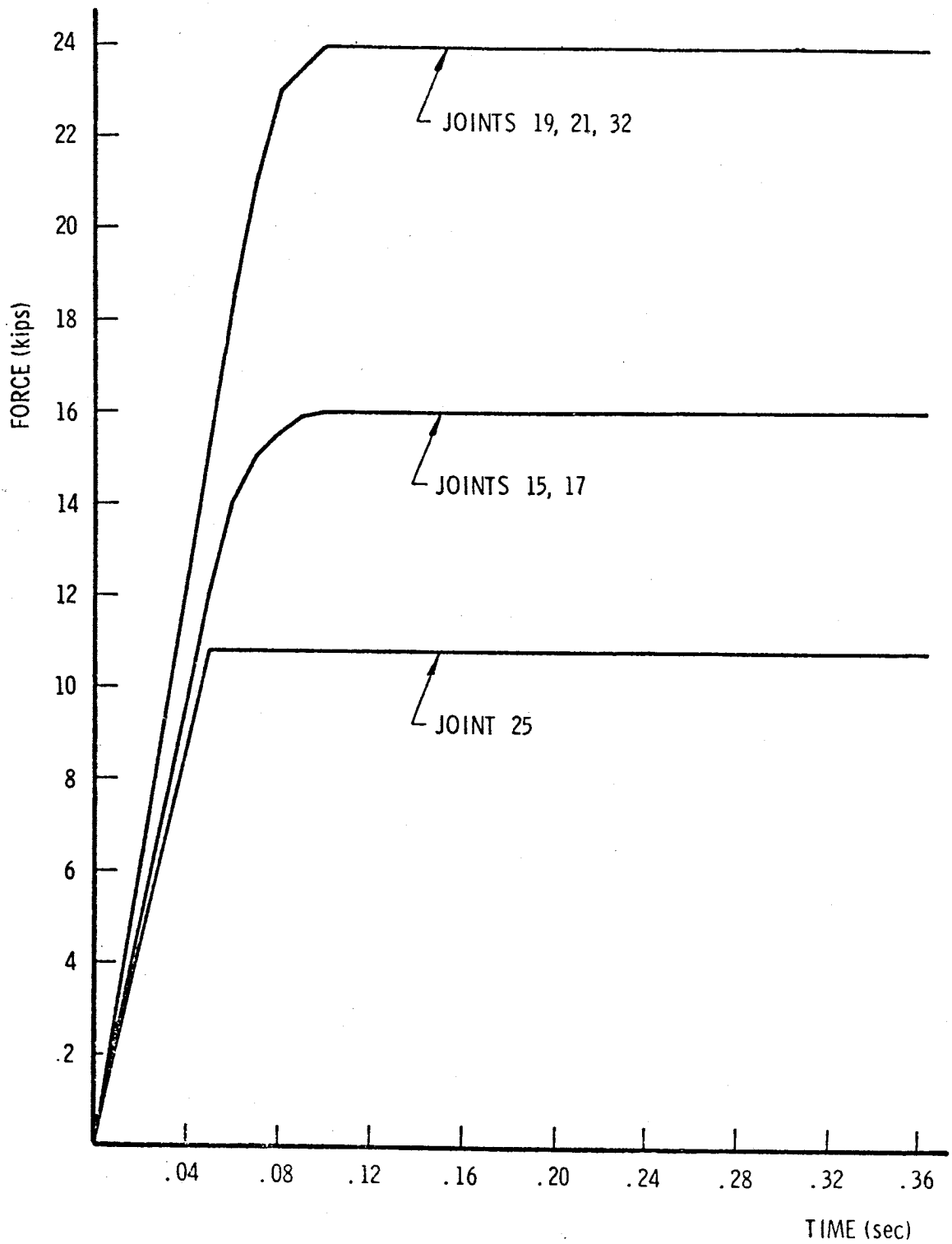
| |
|---|
| CLINTON POWER STATION UPDATED SAFETY ANALYSIS REPORT |
| FIGURE C-44 EXAMPLE FRAME FOR PIPSYS STATIC ANALYSIS |



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FIGURE C-46

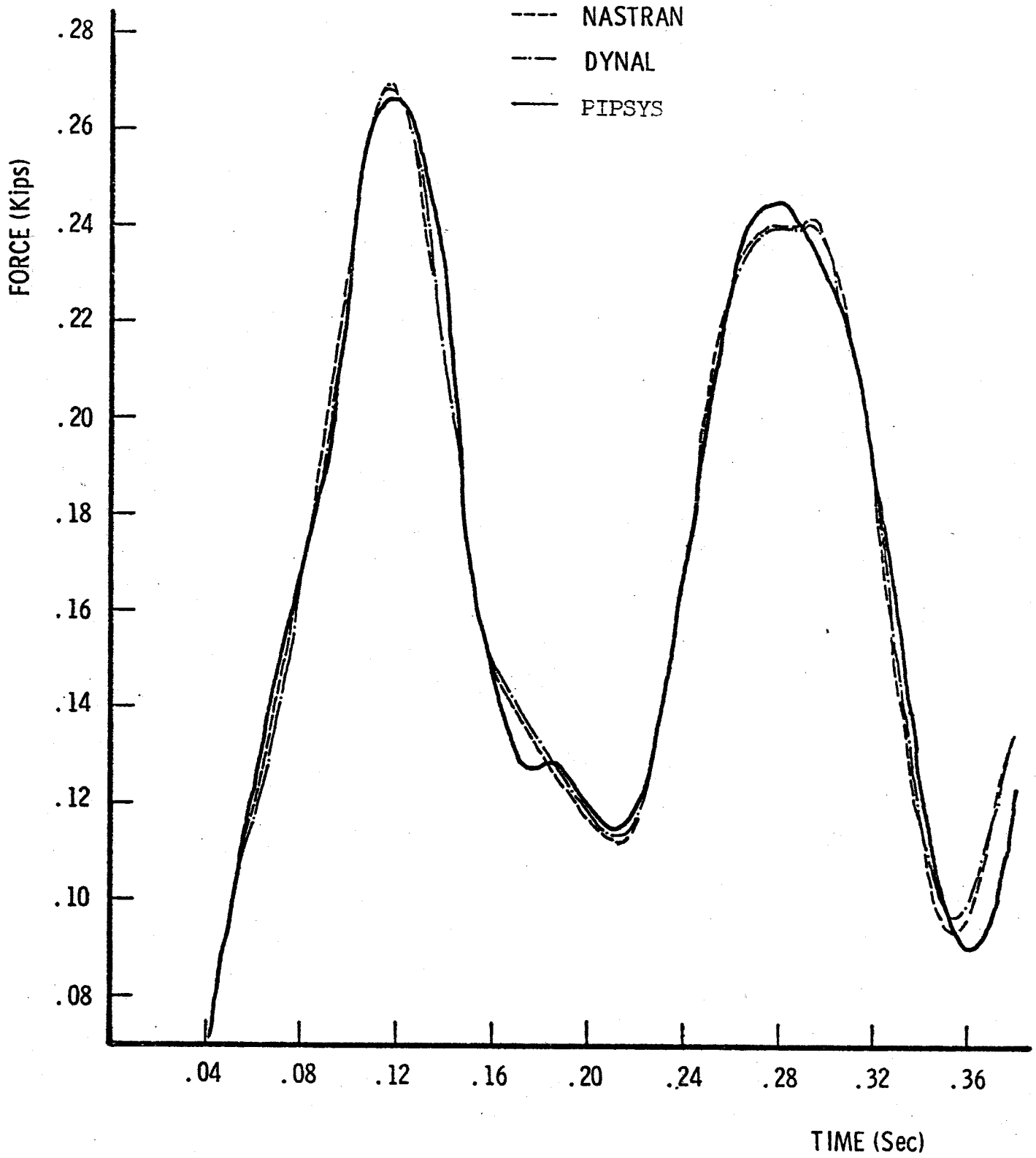
STRUCTURAL MODEL OF PIPING
 SYSTEM (PIPSYS)



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FIGURE C-47

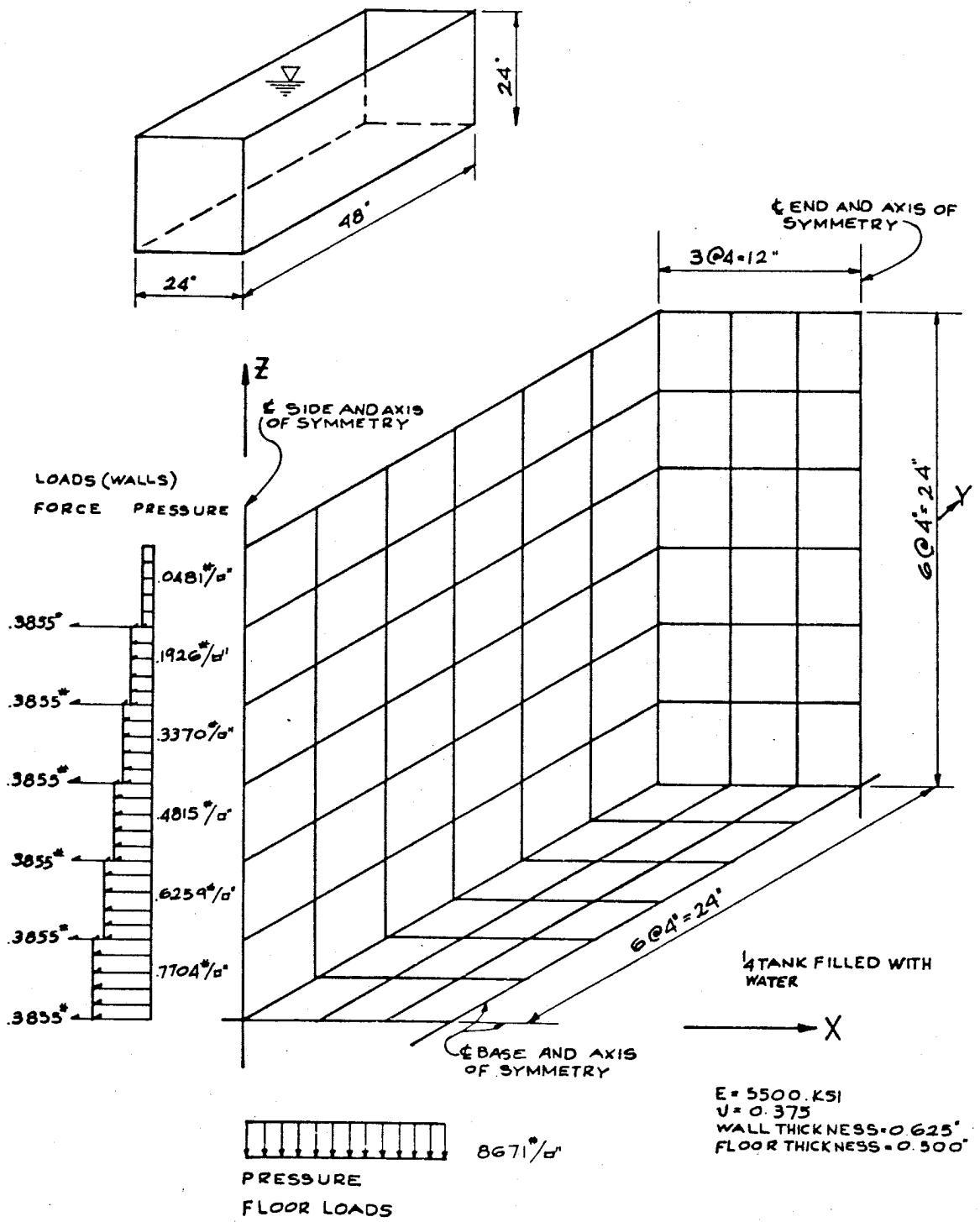
LOAD TIME HISTORY (PIPSYS)



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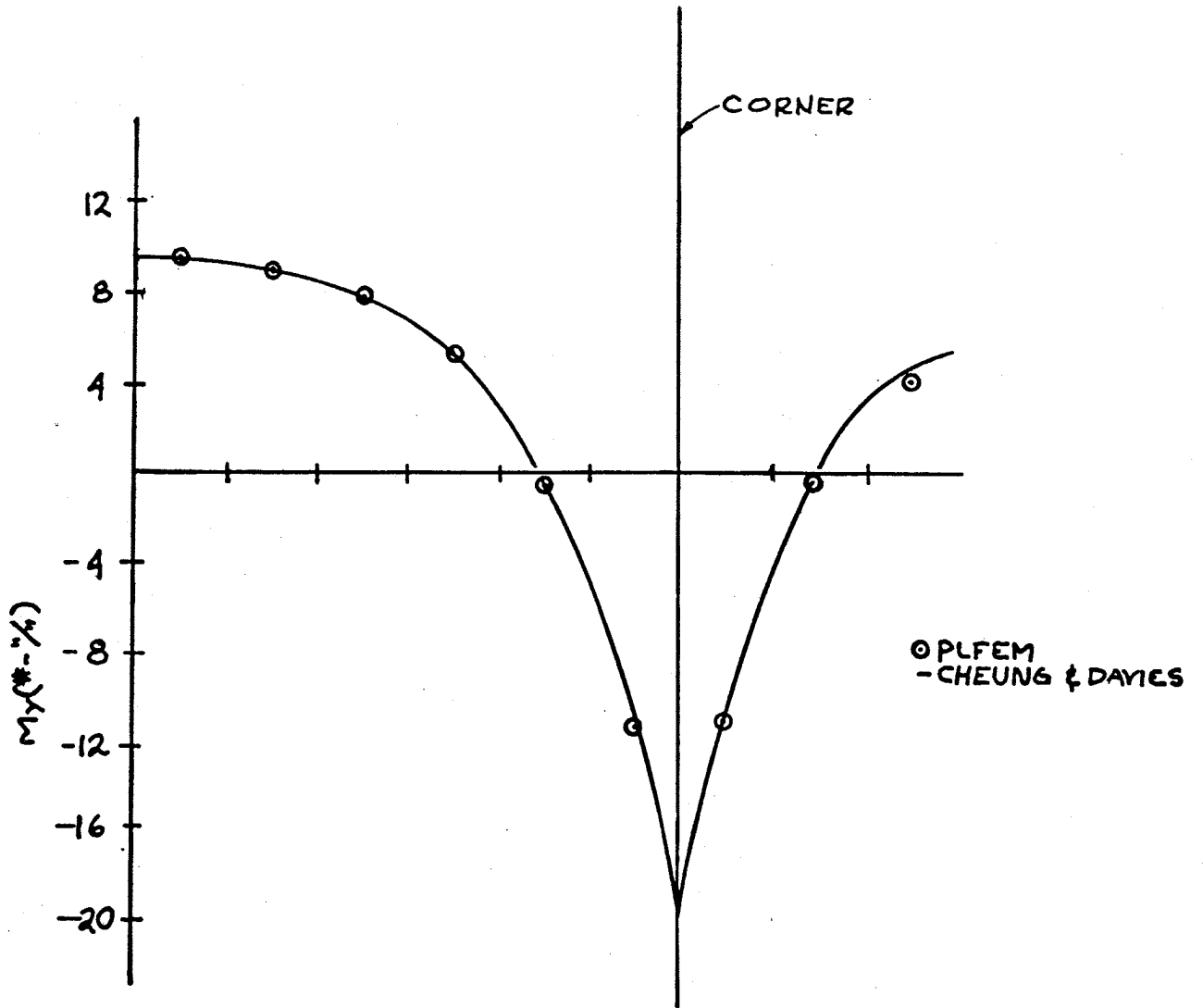
FIGURE C-48

DISPLACEMENT VS. TIME JOINT 8
 Z DIRECTION (PIPSYS)



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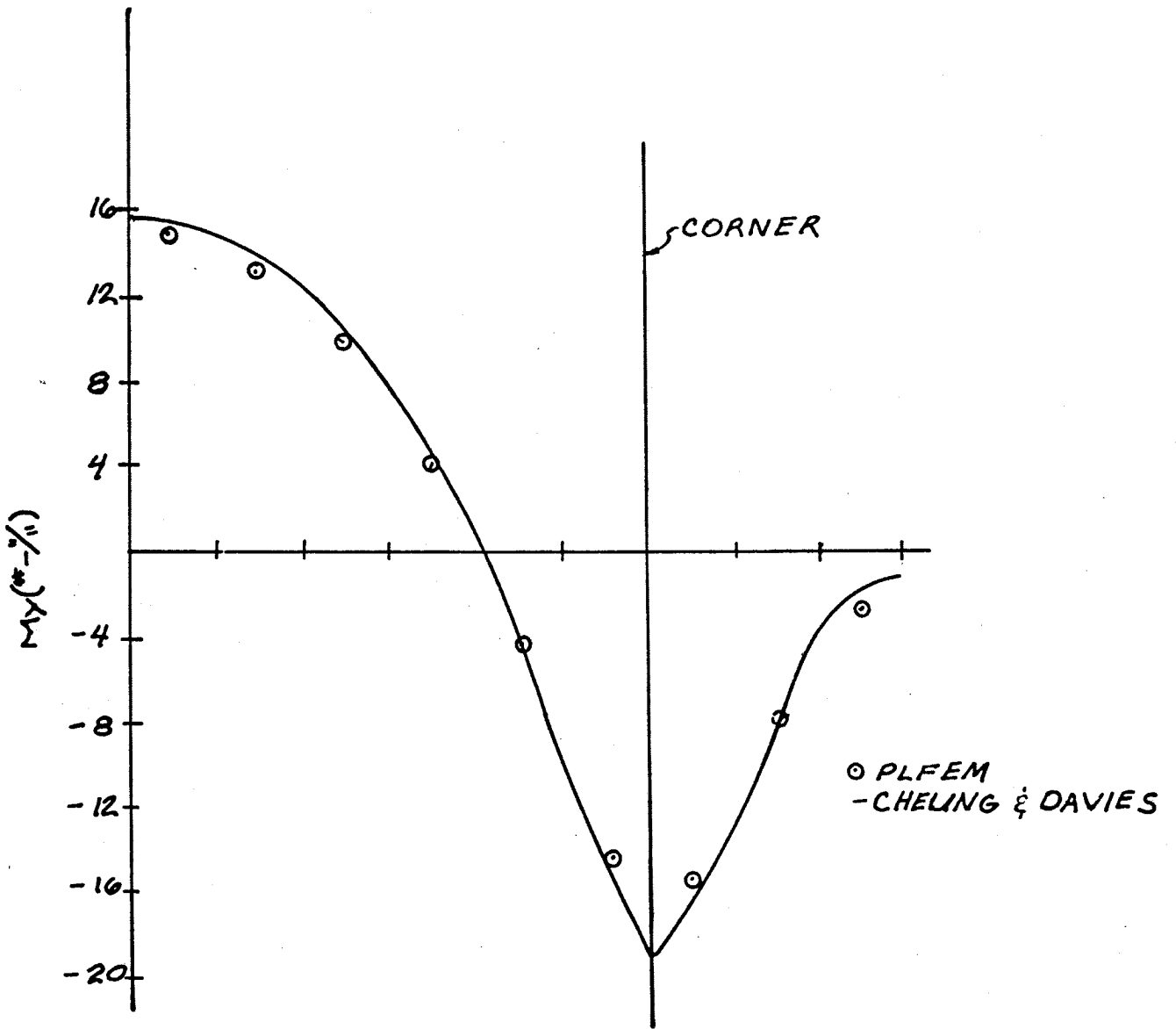
FIGURE C-49
 RECTANGULAR TANK FILLED WITH
 WATER (PLFEM-II)



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FIGURE C-50

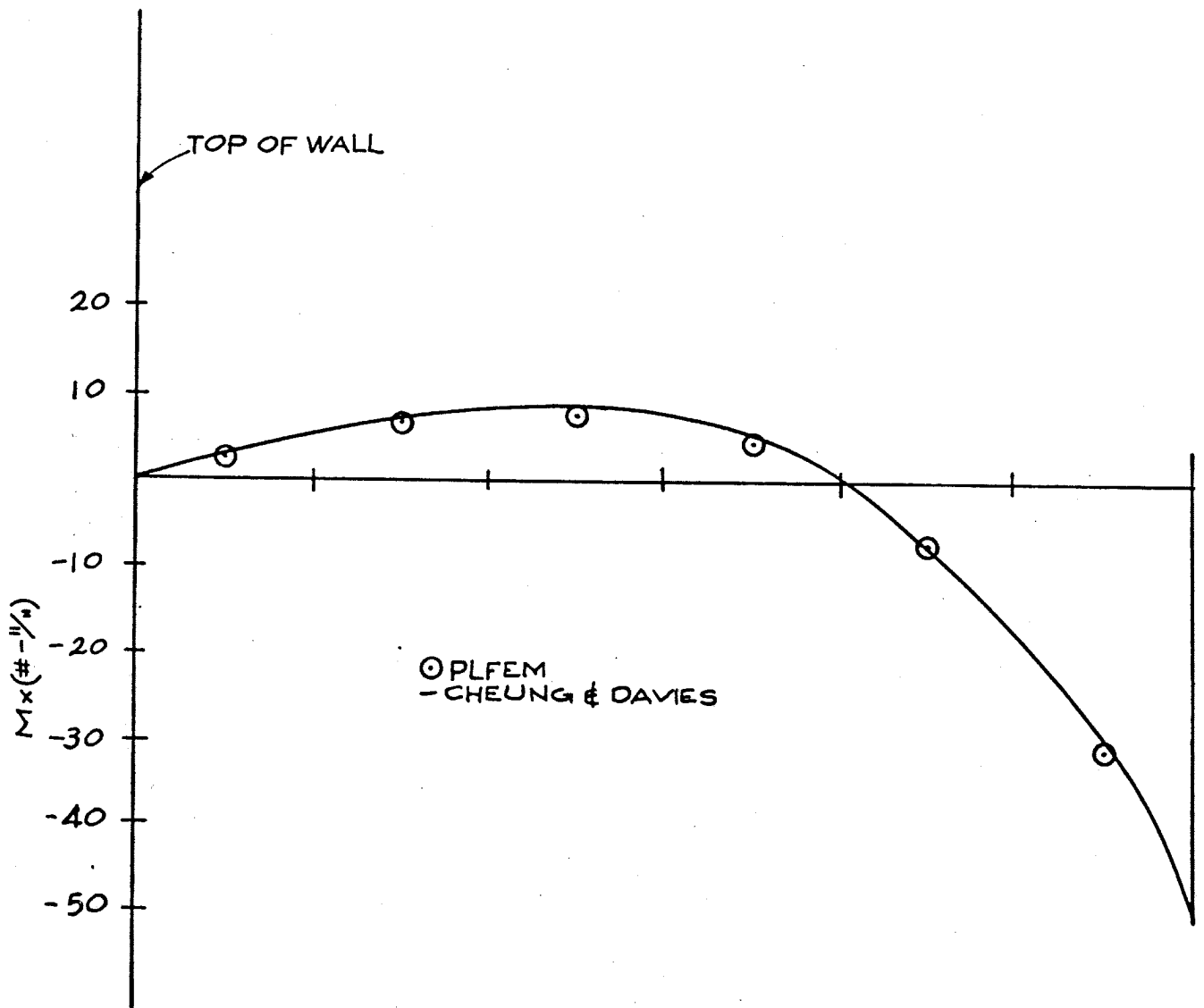
MOMENT OF M_y AT HORIZONTAL Q_L
 OF WALLS (PLFEM-II)



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FIGURE C-51

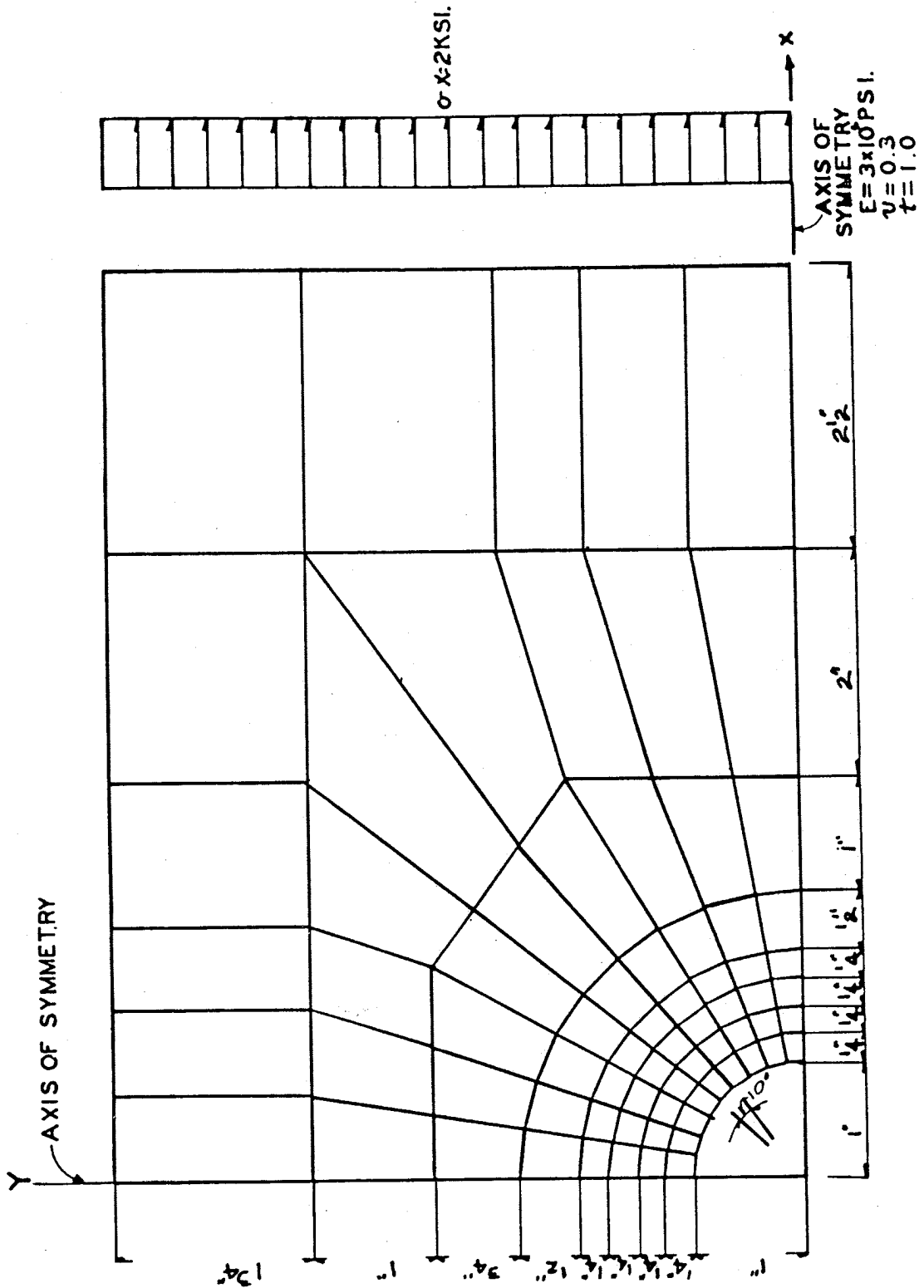
MOMENT M_y AT TOP OF WALL (PLFEM-II)



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FIGURE C-52

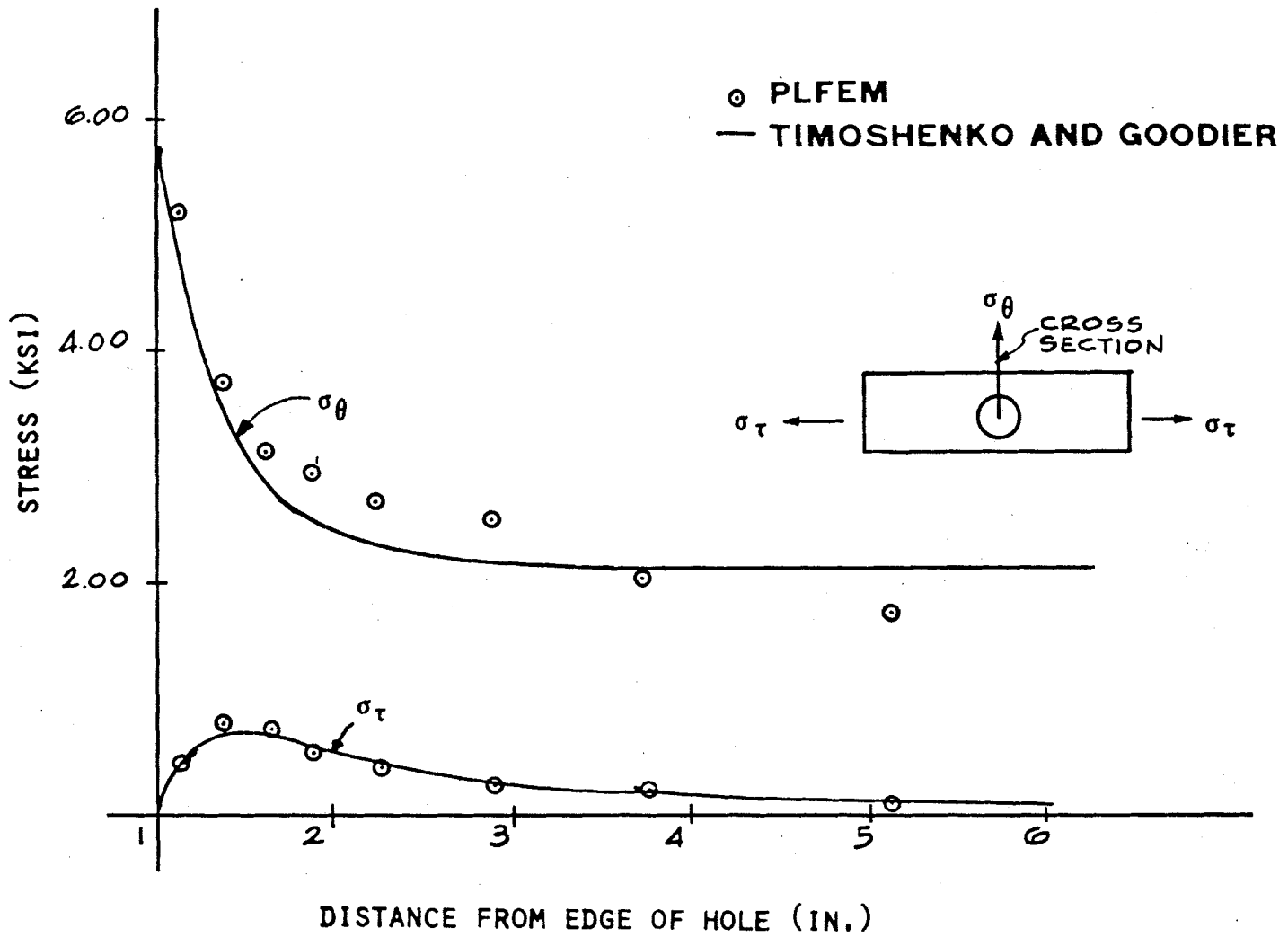
M_x MOMENT ALONG Q_c OF LONG WALL
 (PLFEM-II)



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FIGURE C-53

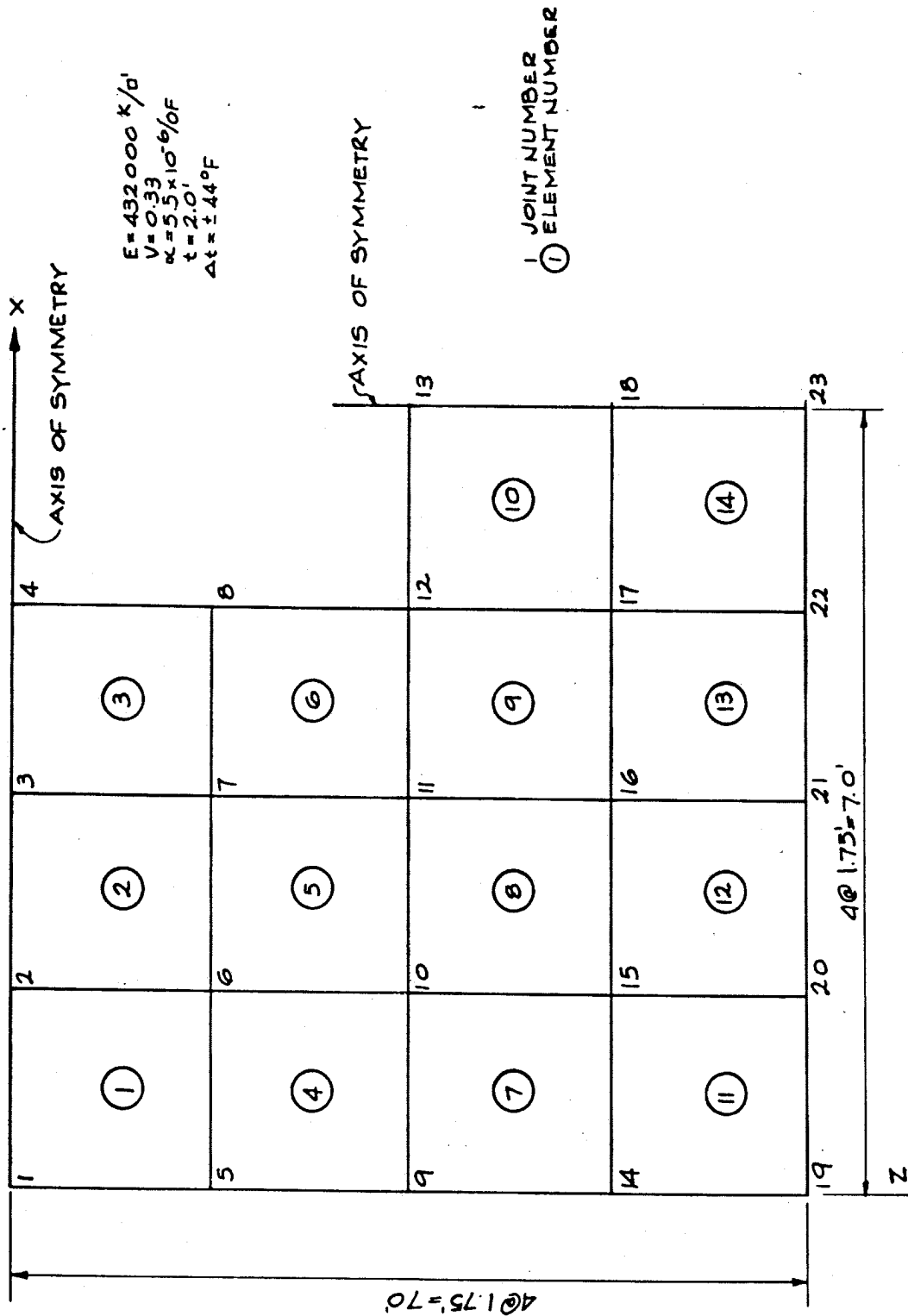
PLATE WITH CIRCULAR HOLE UNDER
 UNIFORM TENSION (PLFEM-II)



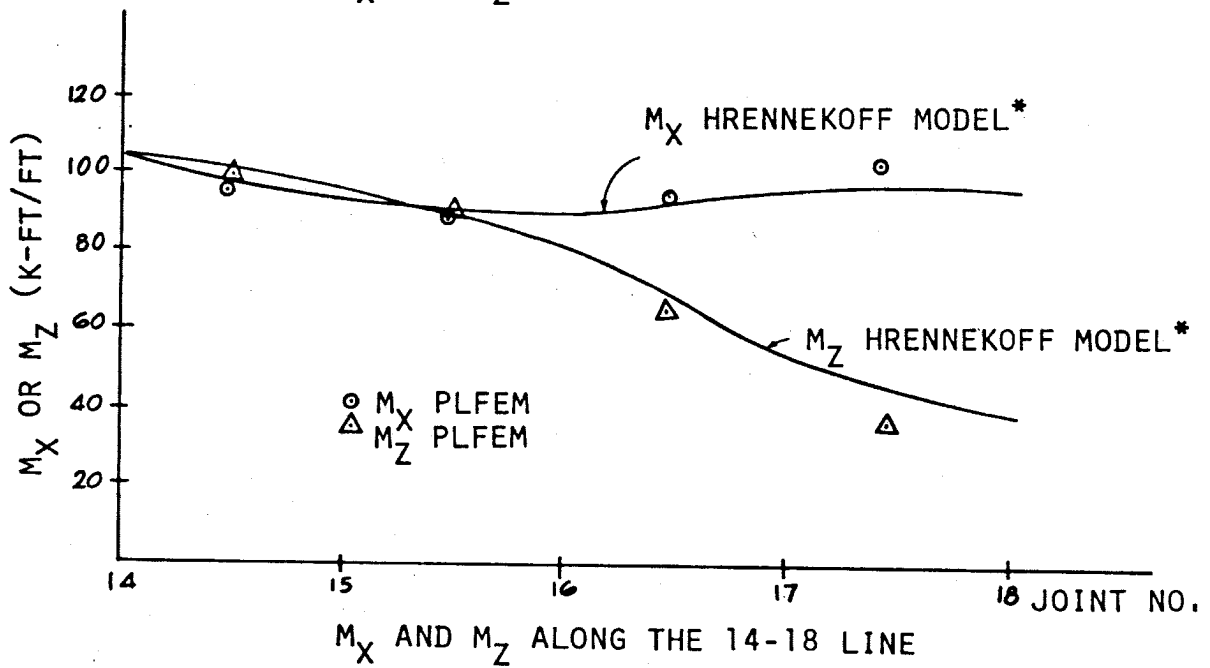
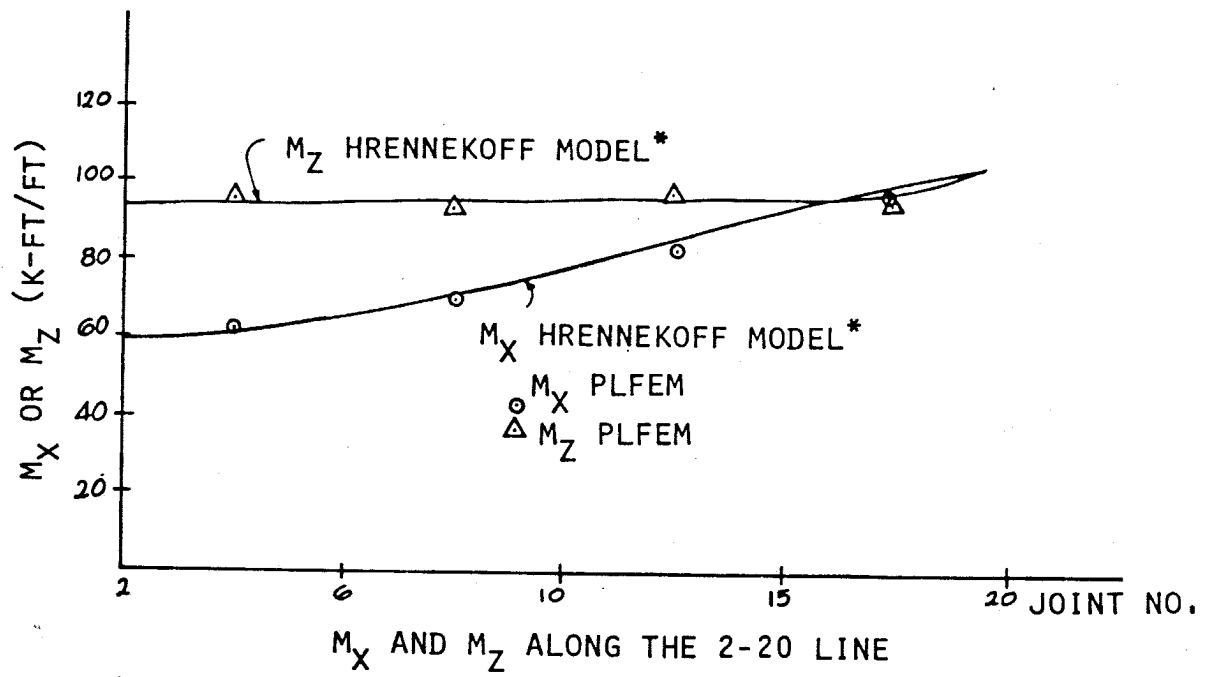
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FIGURE C-54

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)

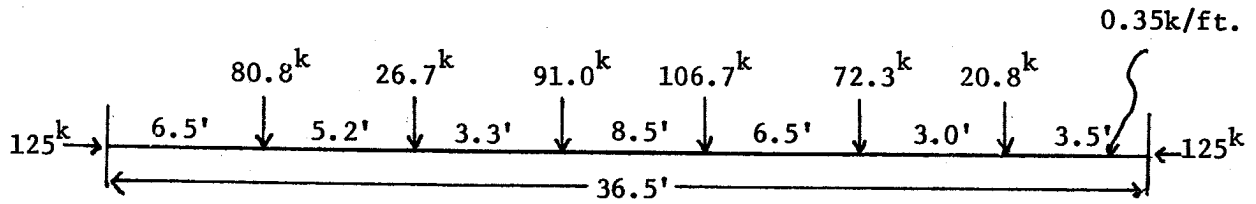


* HRENNEKOFF MODEL BASED ON A FRAMEWORK ELEMENT 0.875' SQUARE

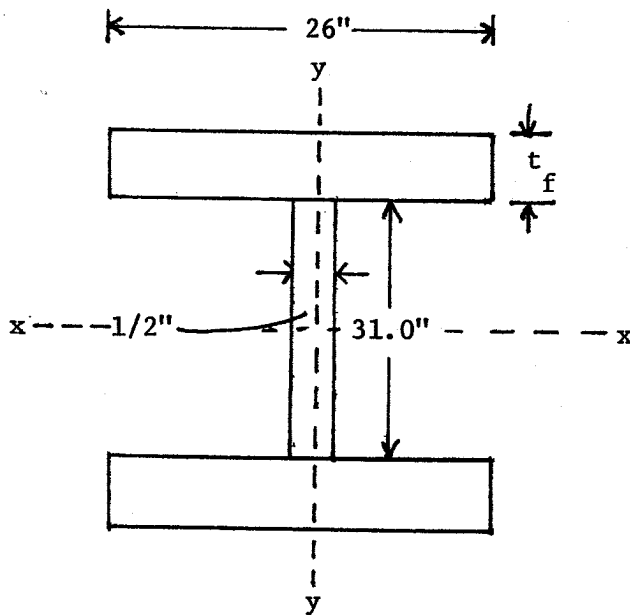
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FIGURE C-56

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

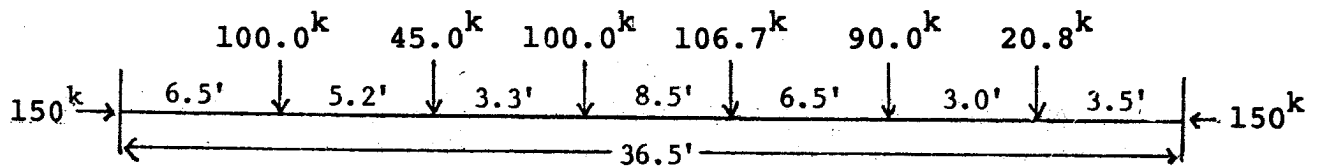


(a) Loading Combination



$t_f = 0.875''$ 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 | .000000 | | | | | | | |
| 1 | .10000+07 | .00000 | .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 | .00000 | .00000 | .00000 | .00000 | | | | | | | | |
| 1 | 1 | 3 | 7 | 1 | 1 | 0 | 0 | 0 | 000000000000 | | | 0 0 0 0 1 |
| .00000 | .00000 | .00000 | .00000 | .00000 | | | | | | | | |
| 2 | 3 | 4 | 7 | 1 | 1 | 0 | 0 | 0 | 000000000000 | | | 0 0 0 0 1 |
| .00000 | .00000 | .00000 | .00000 | .00000 | | | | | | | | |
| 3 | 4 | 5 | 7 | 1 | 1 | 0 | 0 | 0 | 000000000000 | | | 0 0 0 0 1 |
| .00000 | .00000 | .00000 | .00000 | .00000 | | | | | | | | |
| 4 | 5 | 6 | 7 | 1 | 1 | 0 | 0 | 0 | 000000000000 | | | 0 0 0 0 1 |
| .00000 | .00000 | .00000 | .00000 | .00000 | | | | | | | | |
| 5 | 6 | 2 | 7 | 1 | 1 | 0 | 0 | 0 | 000000000000 | | | 0 0 0 0 1 |
| .00000 | .00000 | .00000 | .00000 | .00000 | | | | | | | | |
| 2 | 1 | 1.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 2 | 2 | 1.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 2 | 3 | 1.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 2 | 4 | 1.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 2 | 5 | 2.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 3 | 3 | 1.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 3 | 4 | -1.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 3 | 5 | 2.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 4 | 2 | -1.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 4 | 4 | 1.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 4 | 5 | -3.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 5 | 3 | -2.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 5 | 5 | 2.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 6 | 4 | -1.0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | .0000 | |
| 6 | 5 | -3.0000 | .0000 | .0000 | .0000 | .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 | .00000 | .00000 | .00000 | .00000 | |
| .00000 | .00000 | .00000 | .00000 | .00000 | .00000 | .00000 | .00000 | .00000 | .00000 | .00000 | .00000 | |
| .00000 | .00000 | .00000 | .00000 | .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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|---|
| CLINTON POWER STATION UPDATED SAFETY ANALYSIS REPORT |
| FIGURE C-59 |
| GENERATED SLSAP4 DATA FOR BEAM PROBLEM |

```

000001      000      5
000002      000      SAPIV  SQUARE PLATE TEST - 4 TRIANGLES
000003      000      C
000004      000      NUMBER OF NODES 5 ELEMENT GROUPS 2 LOADINGS 4 NODAL LOADS 1
000005      000      C
000006      000      NODE COORDINATES
000007      000      1 X 0.0 Y 0.0 SUPPORT
000008      000      2 X 0.5 Y 0.0 FREE
000009      000      GEN TO 4 X 0.0 Y 0.5
000010      000      5 X 0.5 Y 0.5 SUPPORT
000011      000      C
000012      000      NODE RELEASE FORCE Z
000013      000      1,5
000014      000      NODE CONSTRAINT
000015      000      2 FORCE X Y MOMENT Y Z
000016      000      3 MOMENT Z
000017      000      4 FORCE X Y MOMENT X Z
000018      000      C
000019      000      ELEMENT GROUP 1 TYPE PLATE
000020      000      C
000021      000      NUMBER OF ELEMENTS 4 MATERIALS 1
000022      000      C
000023      000      ELEMENT INCIDENCE
000024      000      1 I 4 J 1 K 3
000025      000      2 I 1 J 2 K 3
000026      000      3 I 2 J 5 K 3
000027      000      4 I 5 J 4 K 3
000028      000      C
000029      000      MATERIAL PROPERTIES
000030      000      TYPE 1 XCTE 1. YCTE 1. THICKNESS 0.01 DENSITY 1. #
000031      000      CXX 1.0989 CXY 0.32967 CYY 1.0989 GXY 0.3846
000032      000      C
000033      000      ELEMENT INDEX MT 1
000034      000      ALL
000035      000      C
000036      000      ELEMENT LOAD MULTIPLIERS
000037      000      CASE B PRESSURE 1.
000038      000      CASE C GRAVITY Z 1.
000039      000      CASE D TEMPERATURE 1.
000040      000      C
000041      000      PRESSURE LOAD 1.
000042      000      ALL
000043      000      TEMPERATURE VARIATION 1.
000044      000      ALL
000045      000      TEMPERATURE GRADIENT 1.
000046      000      ALL
000047      000      C
000048      000      RETURN
000049      000      C
000050      000      ELEMENT GROUP 2 TYPE BOUNDARY
000051      000      C
000052      000      NUMBER OF ELEMENTS 3
000053      000      C
000054      000      ELEMENT INCIDENCE
000055      000      1 NODE 1 DIRECTION I 1 J 2 K 1 L 4
000056      000      2 NODE 2 DIRECTION I 1 J 2 K 1 L 4
000057      000      3 NODE 4 DIRECTION I 1 J 2 K 1 L 4
000058      000      C
000059      000      ELEMENT INDEX DISPLACEMENT
000060      000      ALL
000061      000      C
000062      000      SPECIFIED SPRING STIFFNESS 1.E+10
000063      000      ALL
000064      000      C
000065      000      RETURN
000066      000      C
000067      000      LOADING 1  NODAL LOAD ONLY
000068      000      NODAL LOADS
000069      000      5 FORCE Z 100.
000070      000      LOADING 2  LOAD CASE B ONLY
000071      000      COMBINE CASE B 100.
000072      000      LOADING 3  LOAD CASE C ONLY
000073      000      COMBINE CASE C 100.
000074      000      LOADING 4  LOAD CASE D ONLY
000075      000      COMBINE CASE D 10.
000076      000      C
000077      000      PLOT MESH
000078      000      FRAME WIDTH 11. LENGTH 8.
000079      000      C
000080      000      SOLVE SLSAP4
000081      000      C
000082      000      FINISH

```

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FIGURE C-60

POLSAP4 INPUT FOR PLATE PROBLEM

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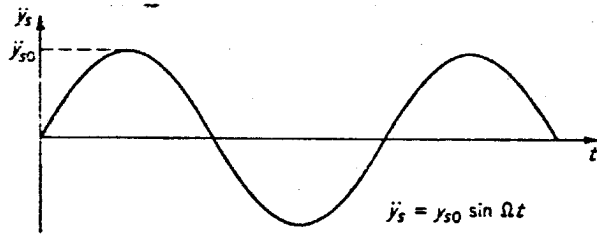
```

000001 000 SLSAP4 1106
000002 000 S
000003 000 F
000004 000 F
000005 000 F
000006 000
000007 000 SQUARE PLATE TEST - 4 TRIANGLES
000008 000 5 2 4 0 0 0 0 0000 .00000 0
000009 000 1 1 1 0 1 1 1 .00000 .00000 .00000 .00000
000010 000 2 1 1 0 0 1 1 .50000 .00000 .00000 .00000
000011 000 3 0 0 0 0 0 1 .25000 .25000 .00000 .00000
000012 000 4 1 1 0 1 0 1 .00000 .50000 .00000 .00000
000013 000 5 1 1 0 1 1 1 .50000 .50000 .00000 .00000
000014 000 6 4 1
000015 000 1 1.00000 1.00000 1.00000 .010000
000016 000 1.099 .330 .000 1.099 .000 .385 0. .000000
000017 000 .00000 1.00000 .00000 .00000
000018 000 .00000 .00000 .00000 1.00000
000019 000 .00000 .00000 .00000 .00000
000020 000 .00000 .00000 1.00000 .00000
000021 000 1 4 1 3 0 1 .01000 1.00000 1.00000 1.00000
000022 000 2 1 2 3 0 1 .01000 1.00000 1.00000 1.00000
000023 000 3 2 5 3 0 1 .01000 1.00000 1.00000 1.00000
000024 000 4 5 4 3 0 1 .01000 1.00000 1.00000 1.00000
000025 000 7 3
000026 000 .00000 .00000 .00000 .00000
000027 000 1 1 2 1 4 1 0 .000000 .000000 .10000+11
000028 000 2 1 2 1 4 1 0 .000000 .000000 .10000+11
000029 000 4 1 2 1 4 1 0 .000000 .000000 .10000+11
000030 000 5 1 .0000 .0000 100.0000 .0000 .0000 .0000
000031 000 0
000032 000 .00000 .00000 .00000 .00000
000033 000 .00000 100.00000 .00000 .00000
000034 000 .00000 .00000 100.00000 .00000
000035 000 .00000 .00000 .00000 10.00000
000036 000 0
000037 000 0

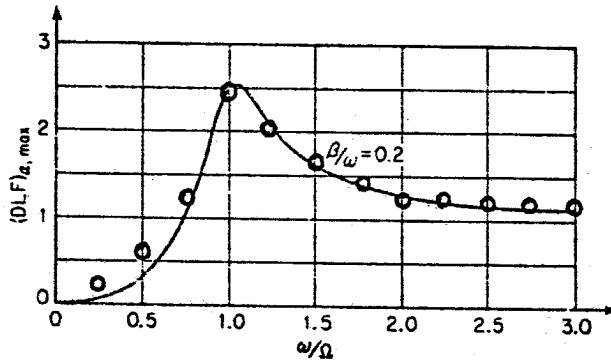
```

Note: Scanned image of computer print-out

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|--|
| <p>CLINTON POWER STATION UPDATED SAFETY ANALYSIS REPORT</p> |
| <p>FIGURE C-61</p> |
| <p>GENERATED SLSAP4 INPUT FOR PLATE PROBLEM</p> |



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



Response Spectrum

$\beta/\omega = 0.2$

Ω = Forcing frequency in radians/second

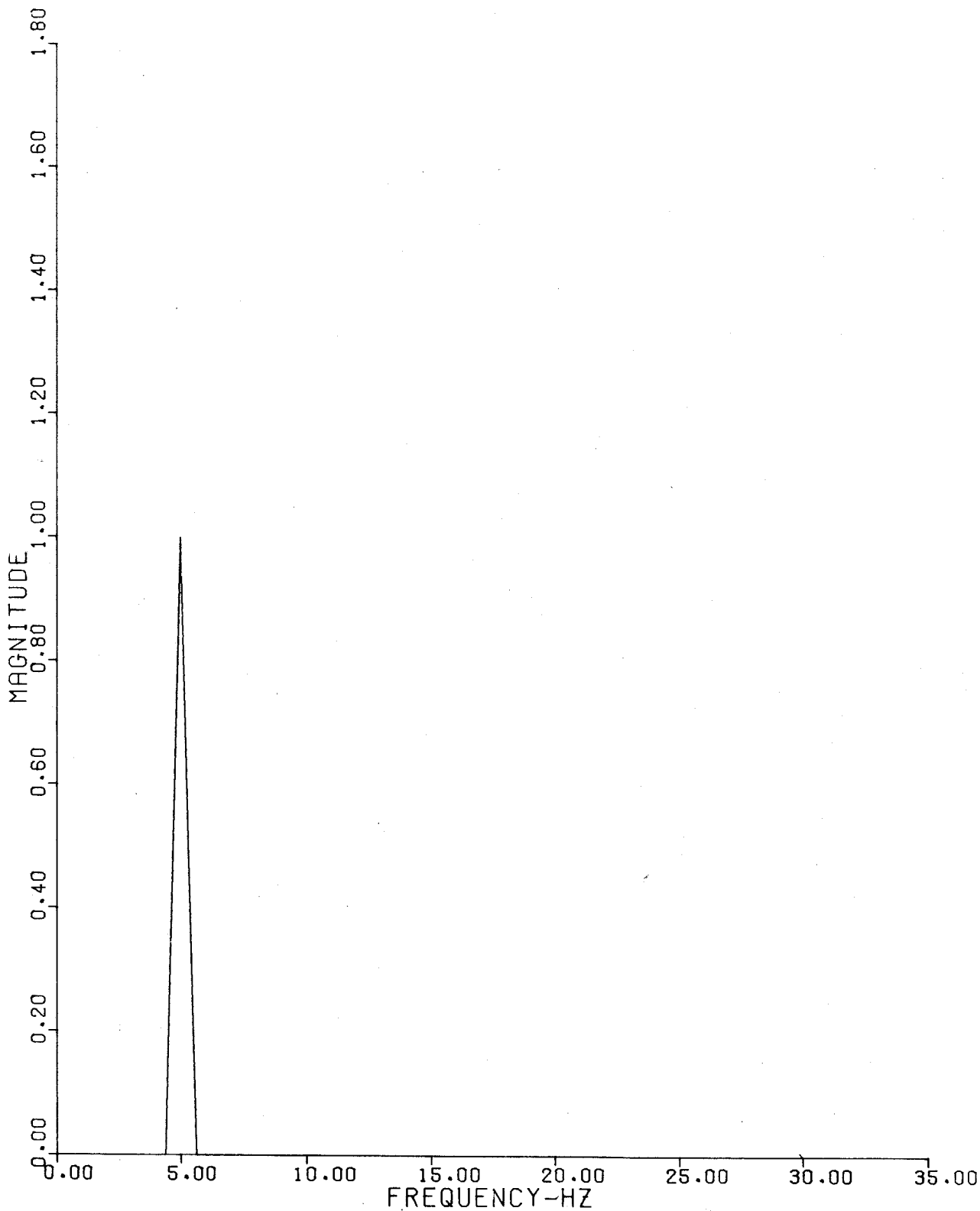
$(DLF)_a, \text{max}$ = Maximum Dynamic Load Factor

— Biggs

ooo RSG

| |
|---|
| CLINTON POWER STATION UPDATED SAFETY ANALYSIS REPORT |
| FIGURE C-62 RESPONSE SPECTRUM FOR SINUSOIDAL VARIATION OF GROUND MOTION |

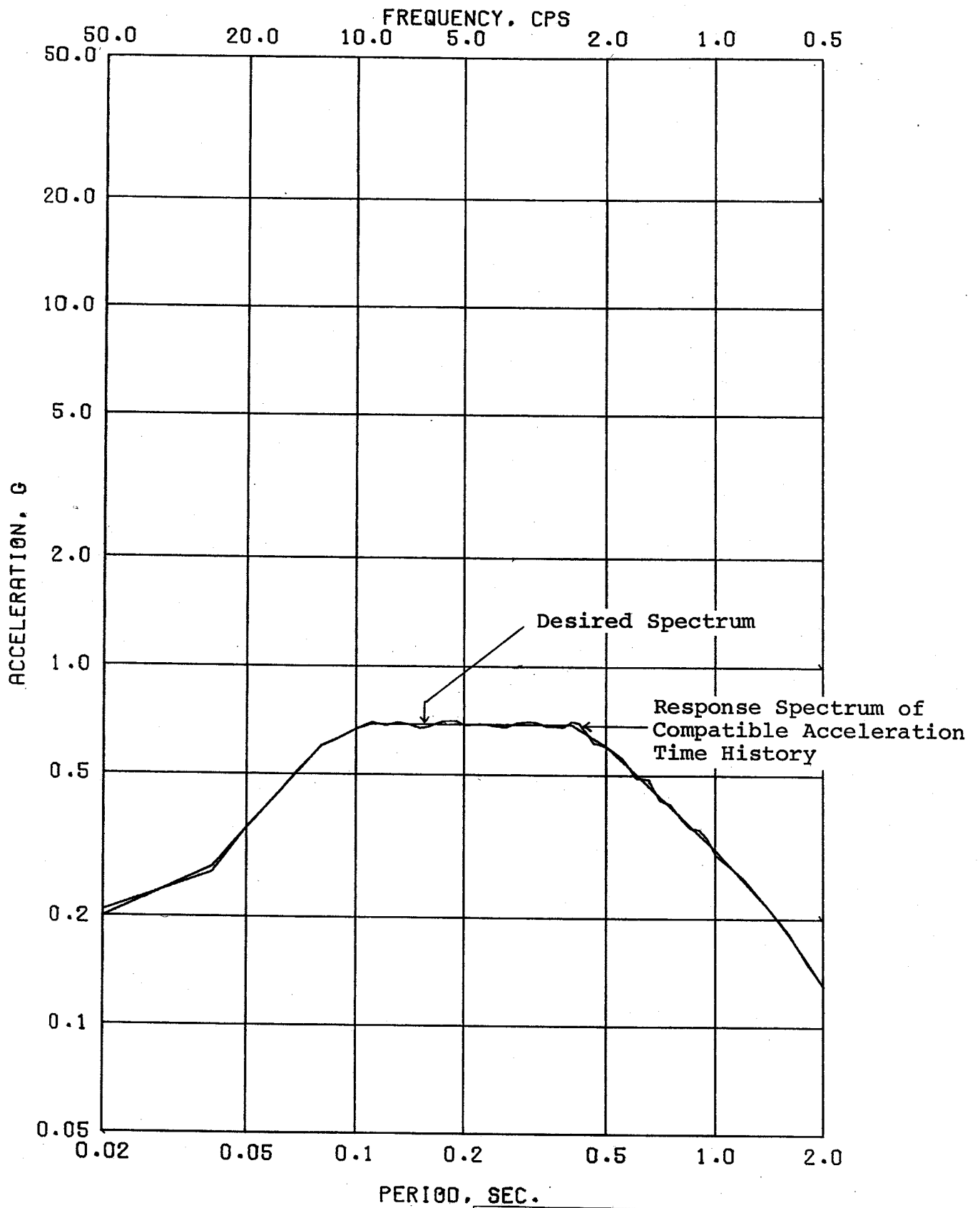
SINE WAVE RESP.SPECT.FOURIER TRANSFORM



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UPDATED SAFETY ANALYSIS REPORT

FIGURE C-63

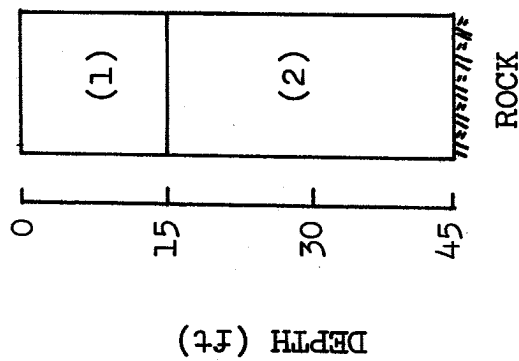
FOURIER TRANSFORM PLOT FROM RSG FOR A
5 CYCLE/SEC SINE WAVE TIME HISTORY



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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

$$\text{UNIT WT. (pcf)} \quad \frac{P > P_c}{m} \quad \frac{P < P_c}{m} \quad P_c (\text{psf})$$



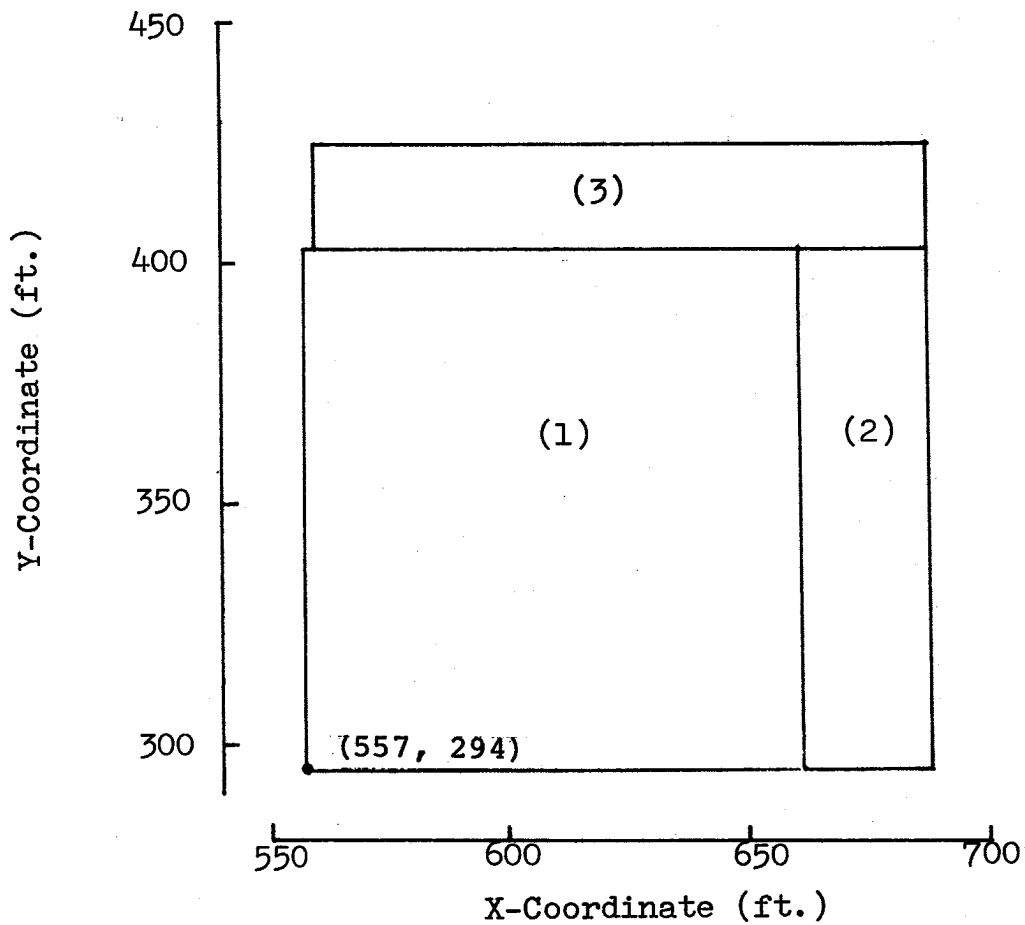
| UNIT WT. (pcf) | $\frac{P > P_c}{m}$ | $\frac{P < P_c}{m}$ | P_c (psf) |
|----------------|---------------------|---------------------|-------------|
| 85.6 | 57.565 | 0 | 575.646 |
| 81.6 | 230.759 | 0 | 2302.585 |
| | | | 20,000 |
| | | | 20,000 |

CLINTON POWER STATION
UPDATED SAFETY ANALYSIS REPORT

FIGURE C-65a

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

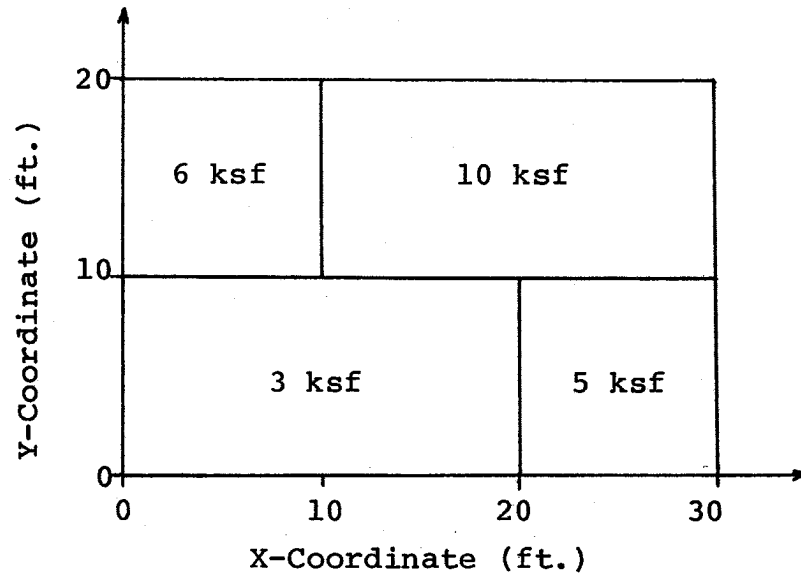
| Loading Area | LENGTH (ft) | WIDTH (ft) | LOAD (psf) | DEPTH (ft) |
|--------------|-------------|------------|------------|------------|
| (1) | 104.5 | 109.5 | 852.0 | 0 |
| (2) | 26.5 | 109.5 | -1013.0 | 0 |
| (3) | 129.4 | 21.5 | -1013.0 | 0 |



**CLINTON POWER STATION
UPDATED SAFETY ANALYSIS REPORT**

FIGURE C-65b

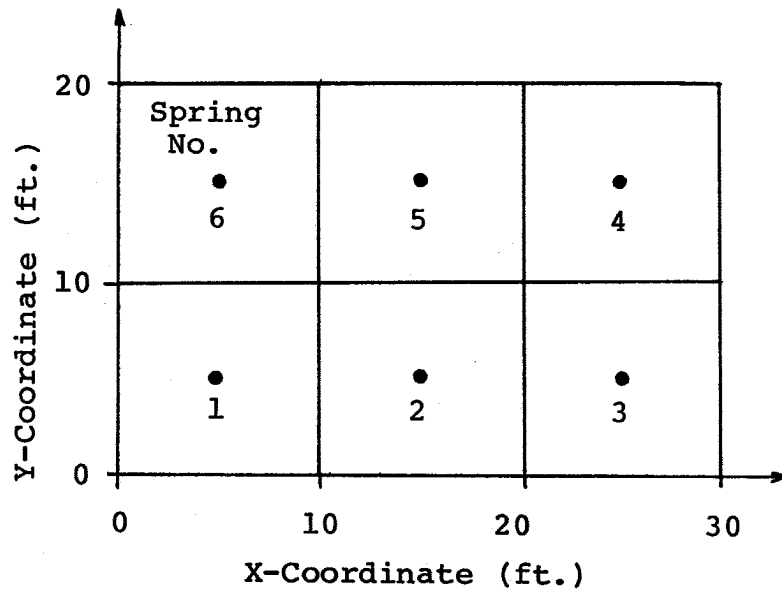
LOADING AREA ON SOIL FOR SETTLE
VALIDATION PROBLEMS 1 TO 3



CLINTON POWER STATION
UPDATED SAFETY ANALYSIS REPORT

FIGURE C-66a

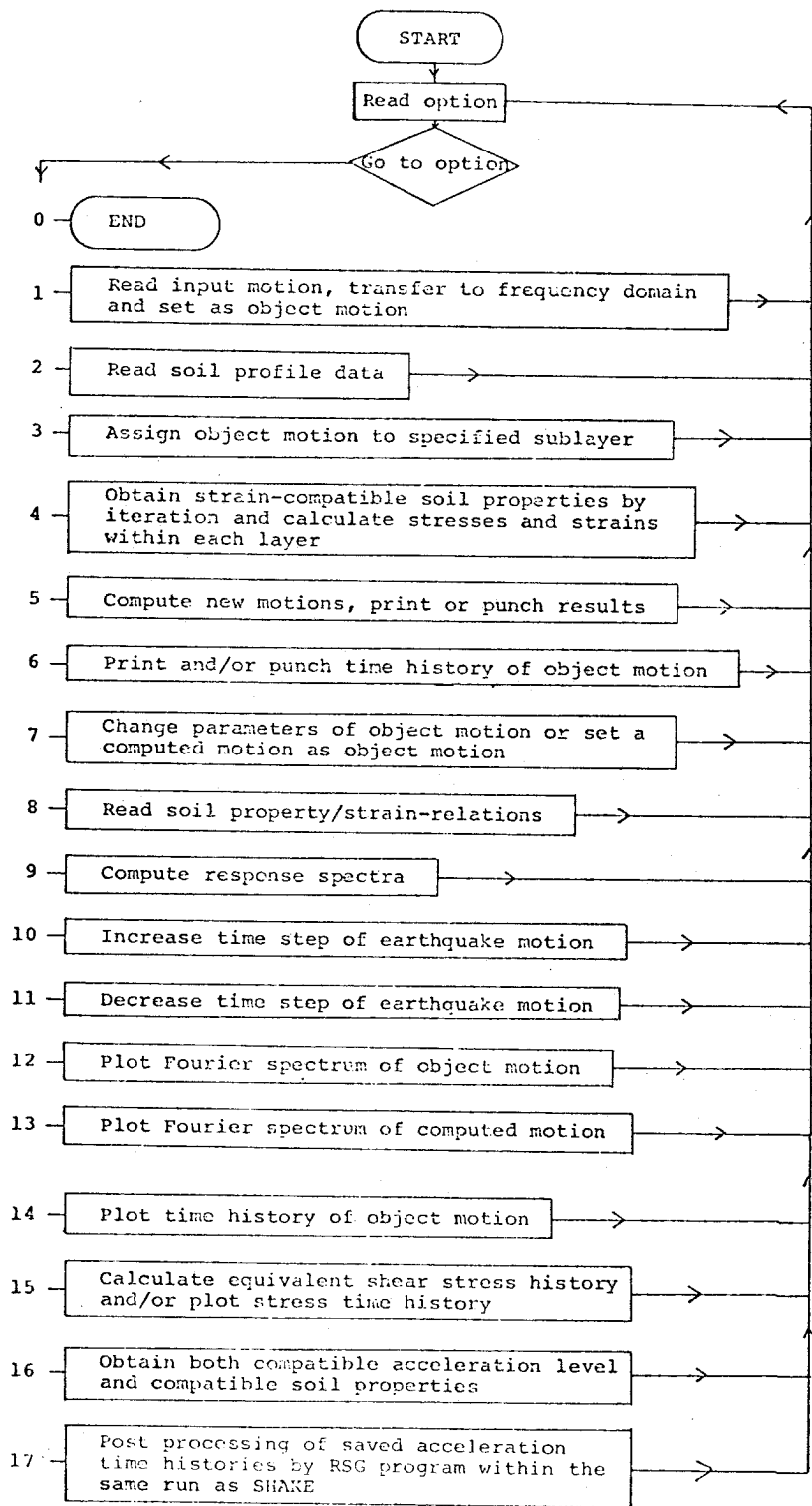
LOADING AREA USED FOR CALCULATING RIGID
FOUNDATION MOVEMENT FOR SETTLE
VALIDATION PROBLEM 4



CLINTON POWER STATION
UPDATED SAFETY ANALYSIS REPORT

FIGURE C-66b

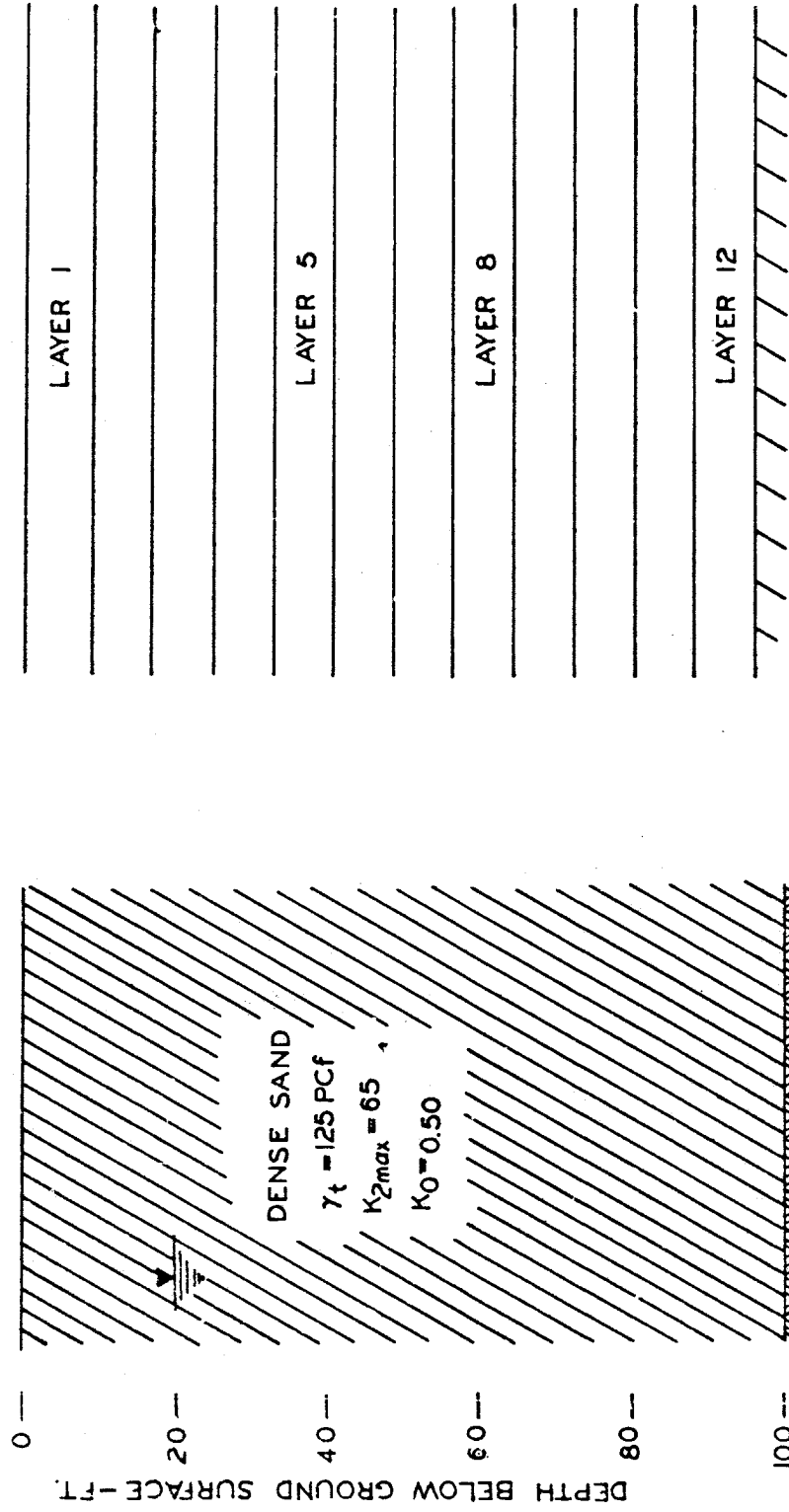
LOCATION OF SPRING FOR CALCULATING
RIGID FOUNDATION MOVEMENT FOR SETTLE
VALIDATION PROBLEM 4



**CLINTON POWER STATION
UPDATED SAFETY ANALYSIS REPORT**

FIGURE C-67

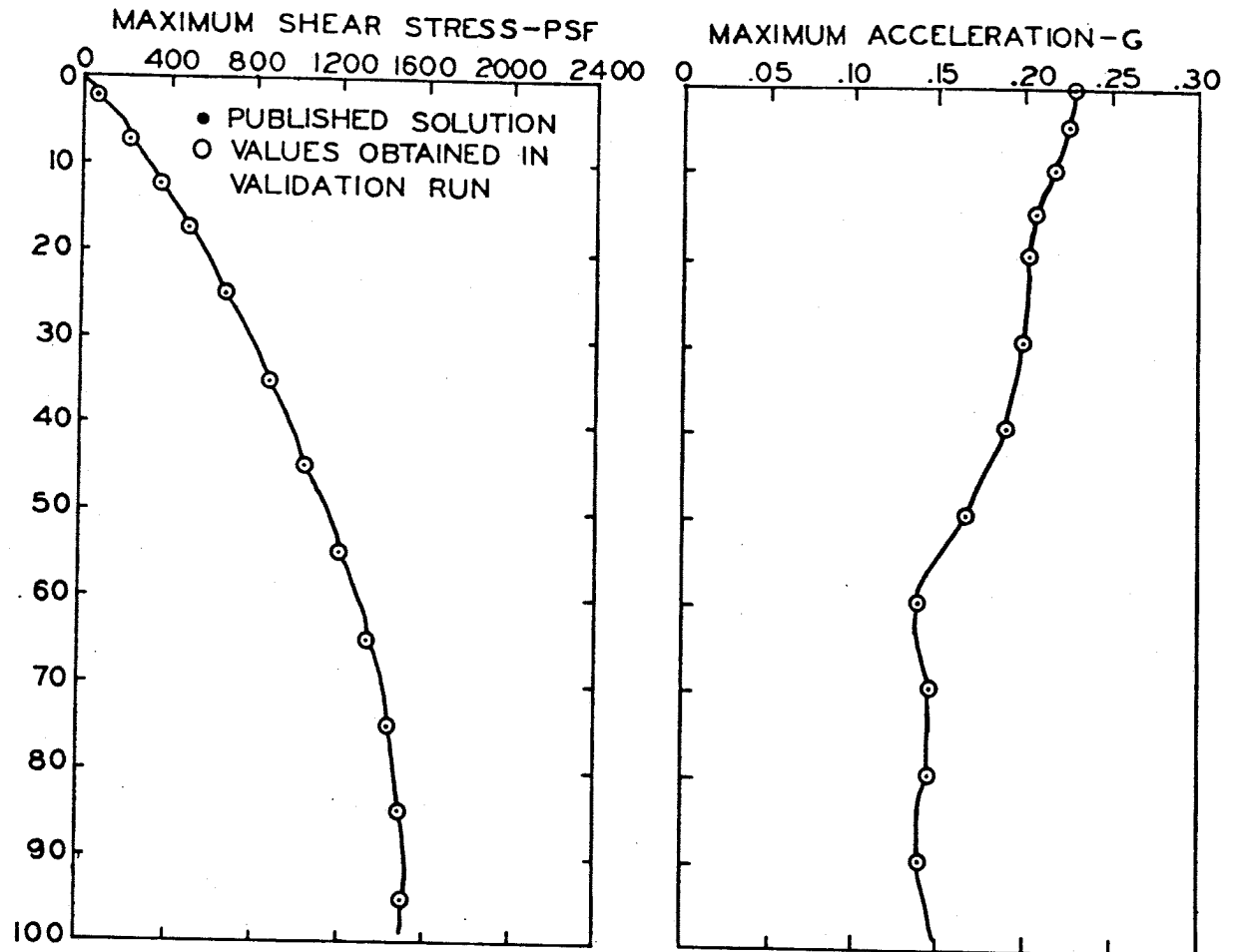
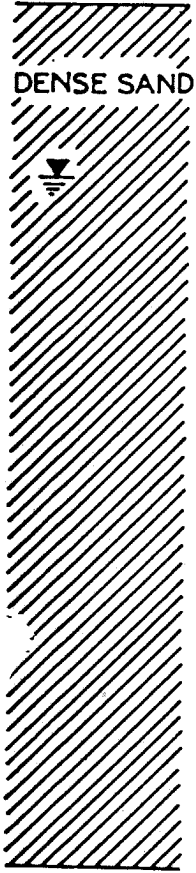
FLOW CHART OF SHAKE



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FIGURE C-68

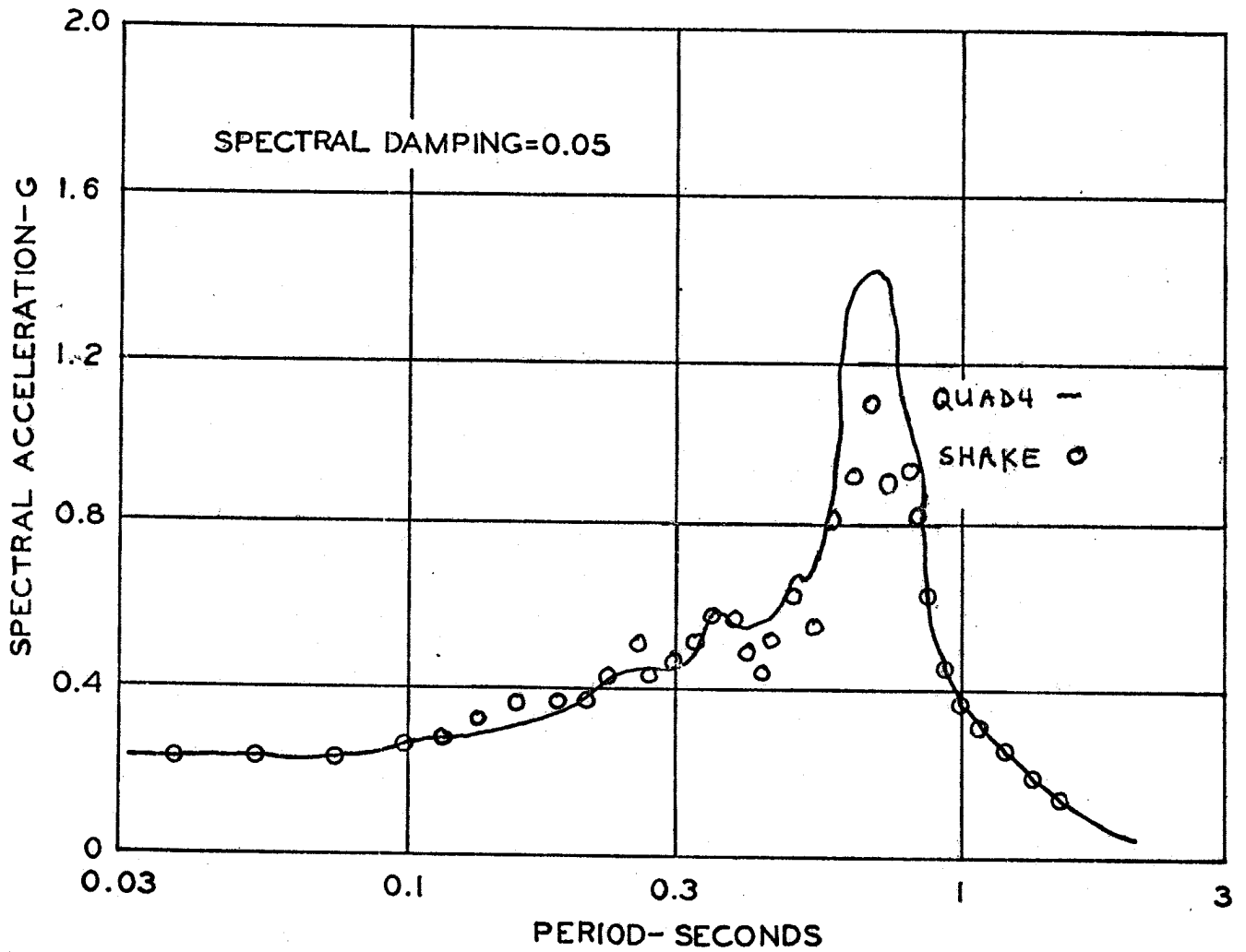
SOIL PROFILE AND LAYERED REPRESENTATION
 USED FOR THE SHAKE SAMPLE PROBLEM



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FIGURE C-69

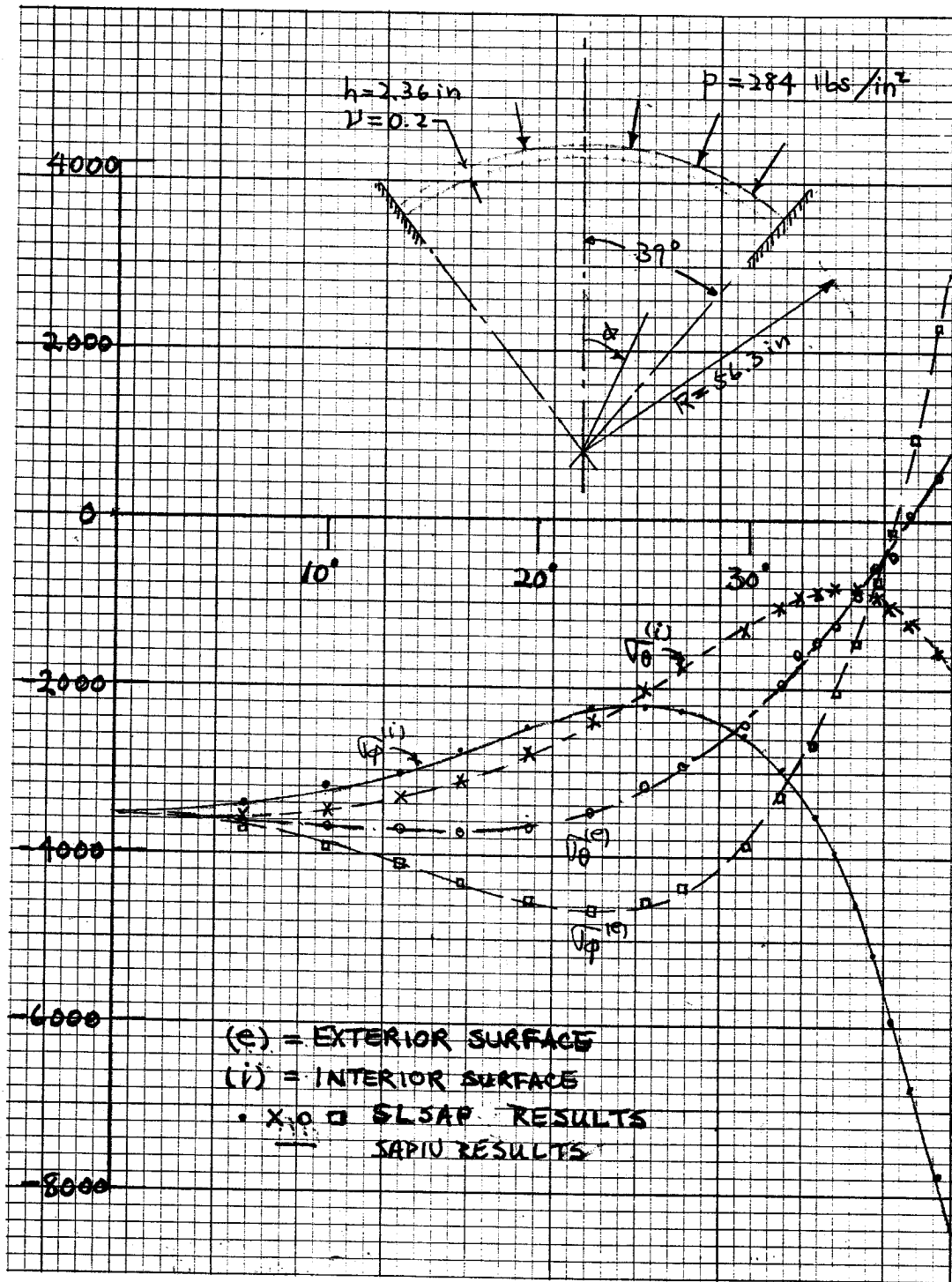
COMPARISON OF SHEAR STRESSES AND
 ACCELERATIONS COMPUTED BY
 SHAKE AND QUAD4 (SHAKE)



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 UPDATED SAFETY ANALYSIS REPORT

FIGURE C-70

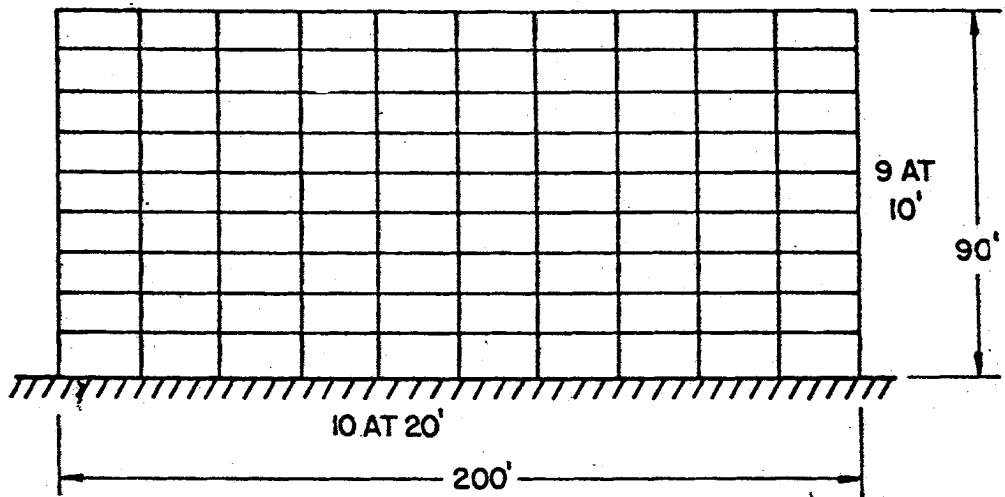
COMPARISON OF SPECTRAL VALUES FOR
 SURFACE MOTIONS COMPUTED BY SHAKE
 AND QUAD4 (SHAKE)



CLINTON POWER STATION
 UPDATED SAFETY ANALYSIS REPORT

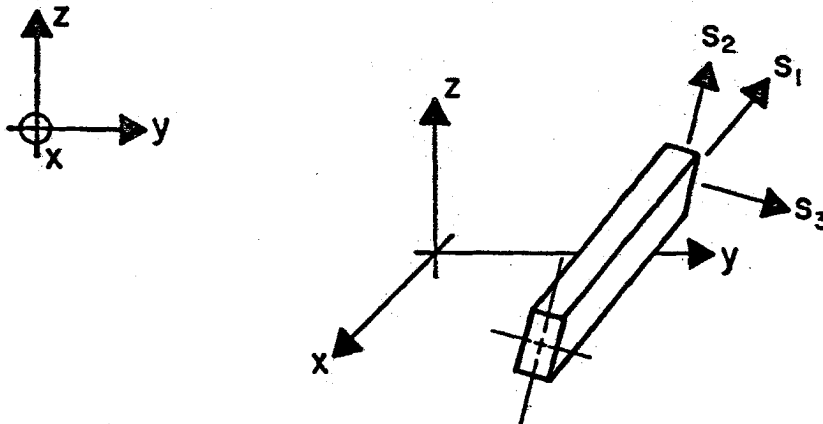
FIGURE C-72

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



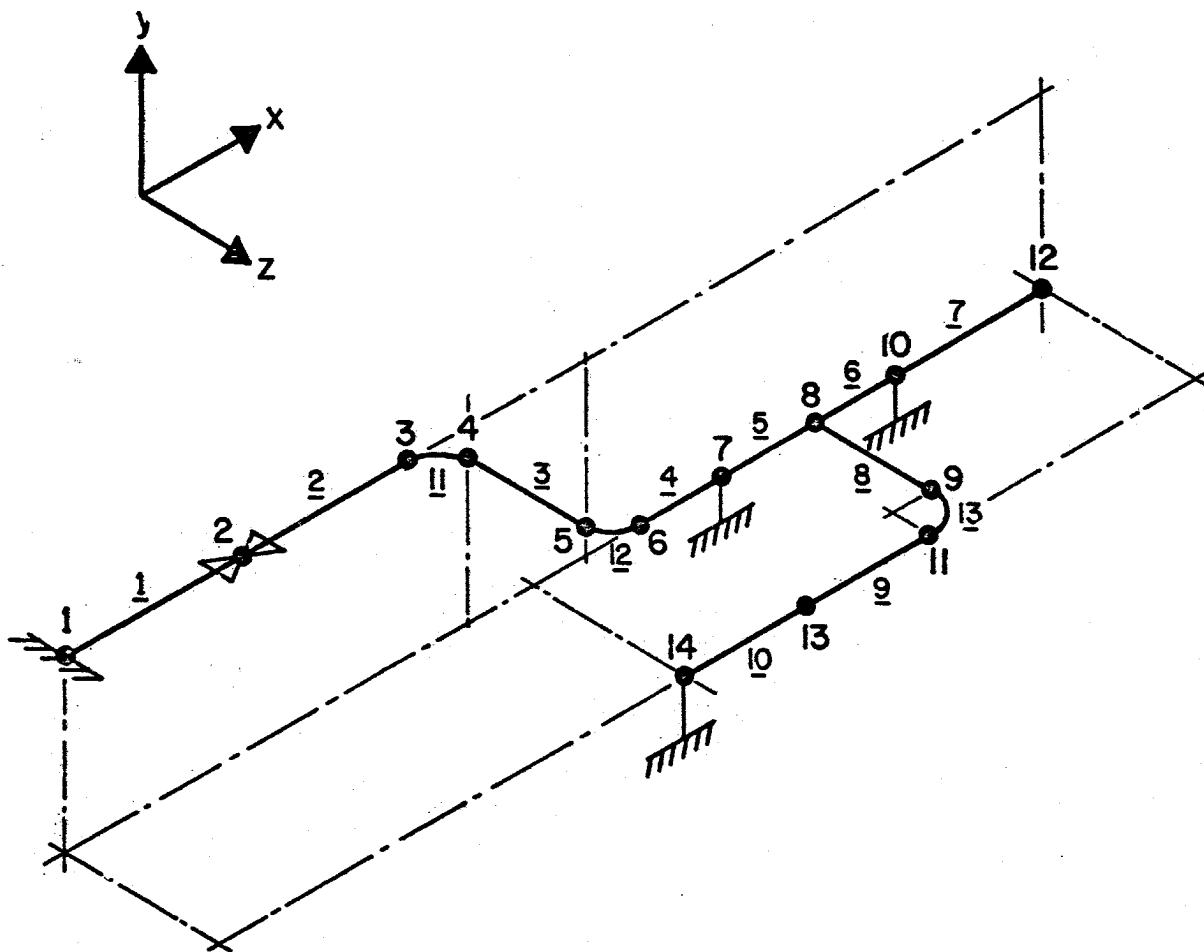
(b) BEAM ELEMENT DEFINITION

S_1, S_2 AND S_3 = BEAM LOCAL AXES

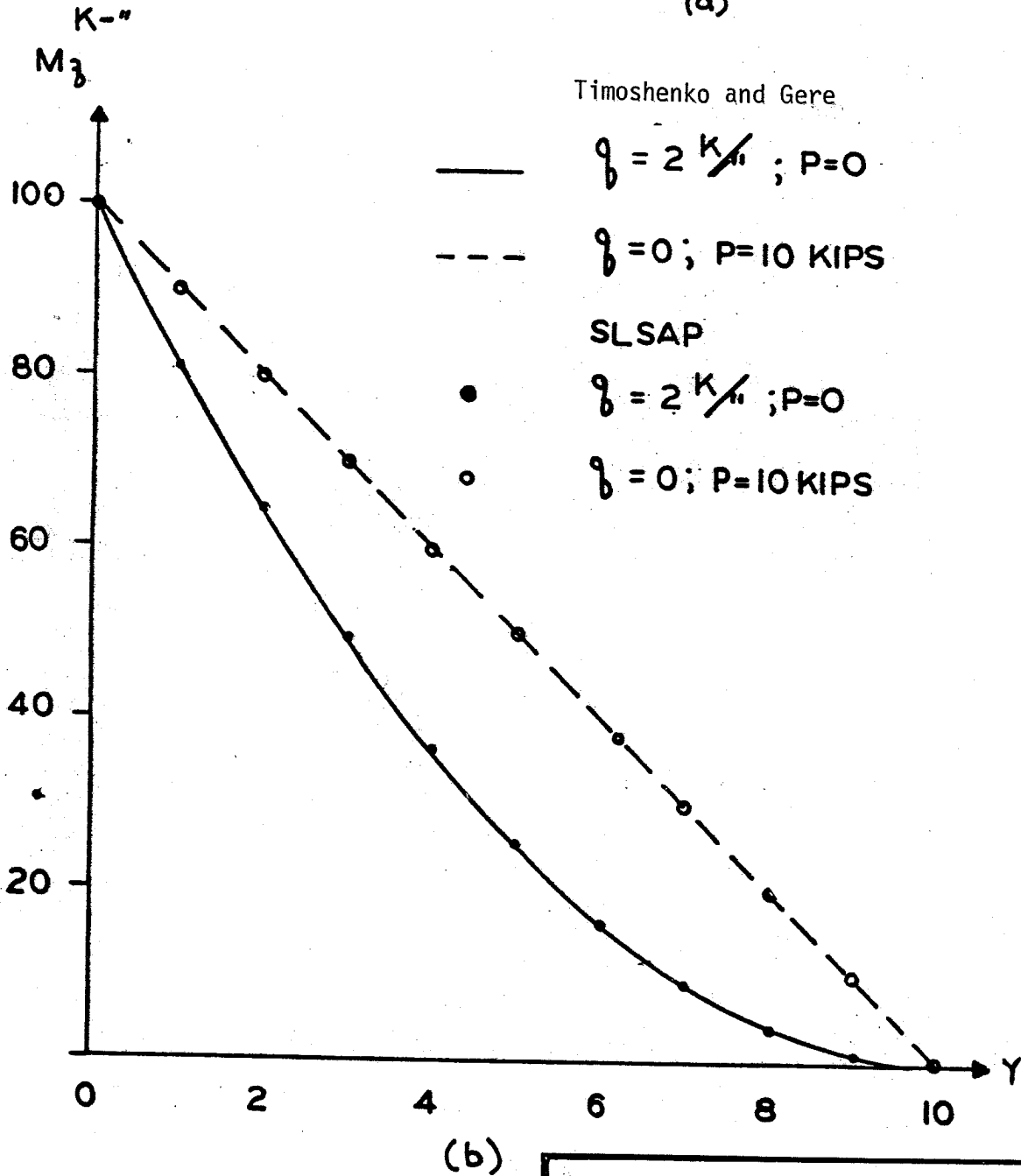
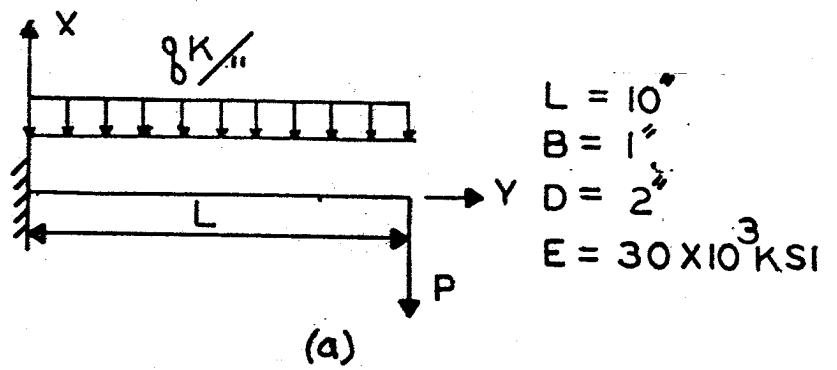
I_1, I_2 AND I_3 = FLEXURAL INERTIA ABOUT S_1, S_2 , AND S_3

A_1 = AREA ASSOCIATED WITH S_1

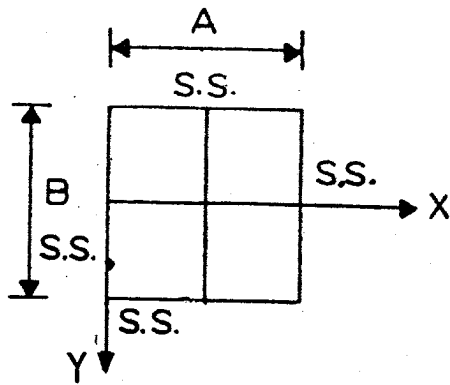
| |
|---|
| CLINTON POWER STATION UPDATED SAFETY ANALYSIS REPORT |
| FIGURE C-73 MODEL OF PLANE FRAME FOR SLSAP AND SAPIV (SLSAP VALIDATION PROBLEM 3) |



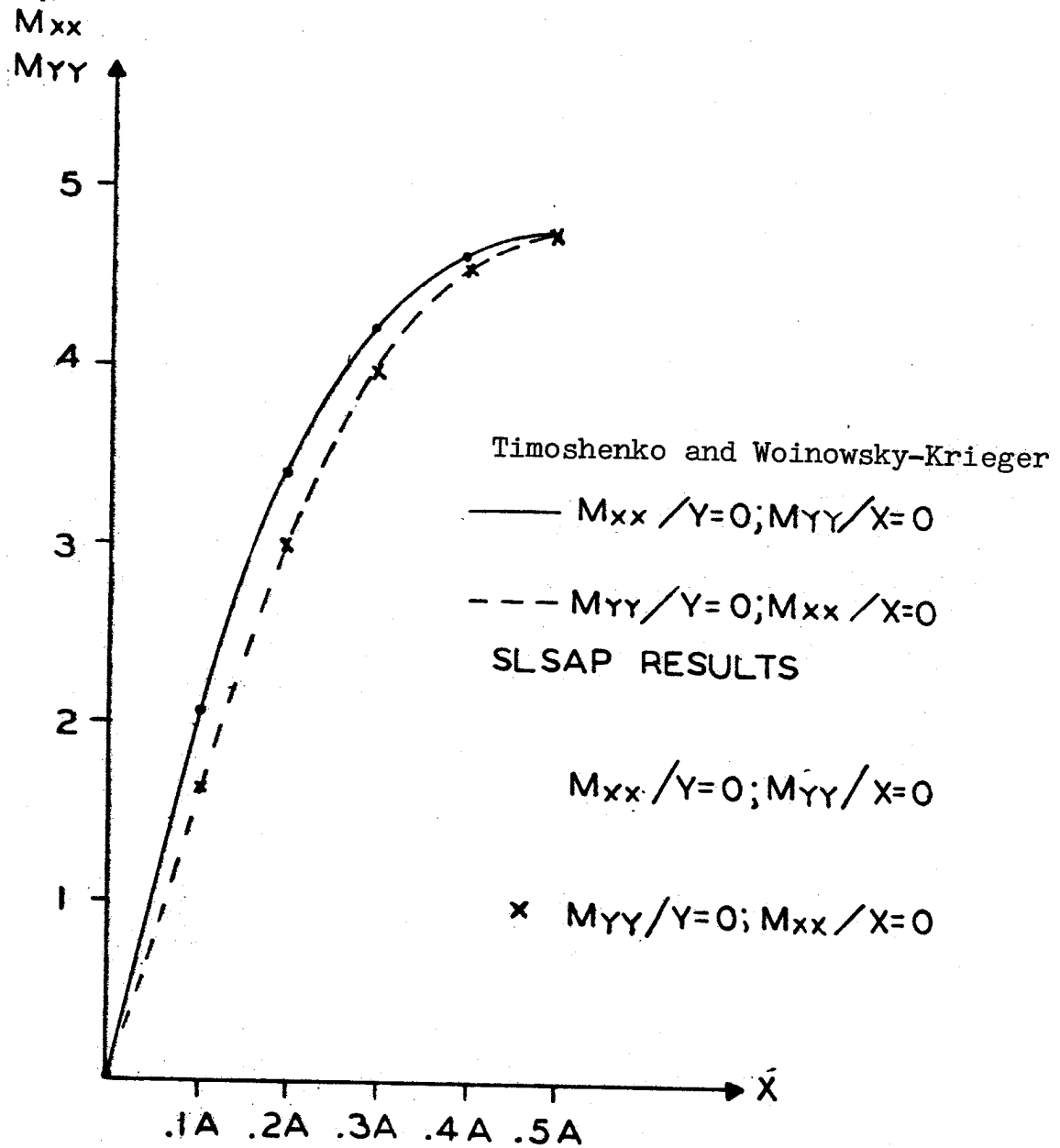
| |
|--|
| <p>CLINTON POWER STATION UPDATED SAFETY ANALYSIS REPORT</p> |
| <p>FIGURE C-74</p> |
| <p>MODEL OF PIPE ASSEMBLAGE FOR SLSAP AND SAPIV (SLSAP VALIDATION PROBLEM 4)</p> |



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 FIGURE C-75
 COMPARISON OF BENDING MOMENTS IN A
 CANTILEVER BEAM FOR SLSAP AND
 REFERENCE 43 (SLSAP VALIDATION PROBLEM 5)



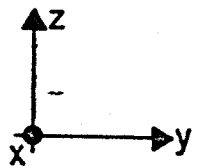
$A = B = 10''$
 $\nu = 0.3$
 $E = 30 \times 10^3 \text{ KSI}$
 $T = 1''$
 $q = 1.0 \text{ KSI}$



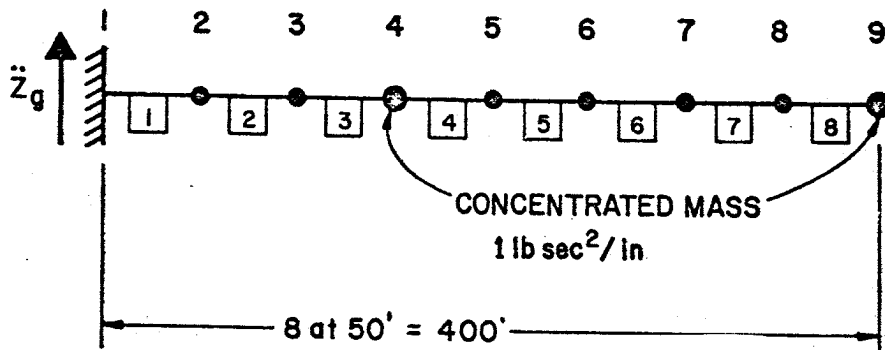
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FIGURE C-76

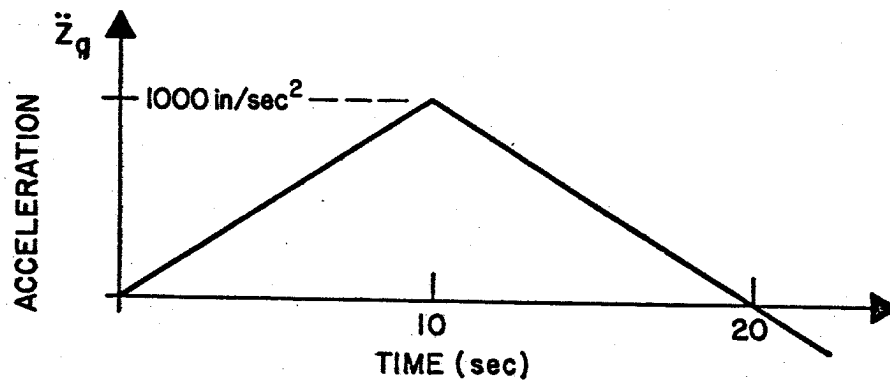
COMPARISON OF BENDING MOMENTS IN A SIMPLY
 SUPPORTED PLATE FOR SLSAP AND REFERENCE 15
 44 (SLSAP VALIDATION PROBLEM 6)



$I = 1.0 \text{ in}^4$; $A = 100.0 \text{ in}^2$
 $E = 30 \times 10^6 \text{ lbs/in}^2$
 $\rho = 1.0 \text{ lb-sec}^2/\text{in}^4$



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

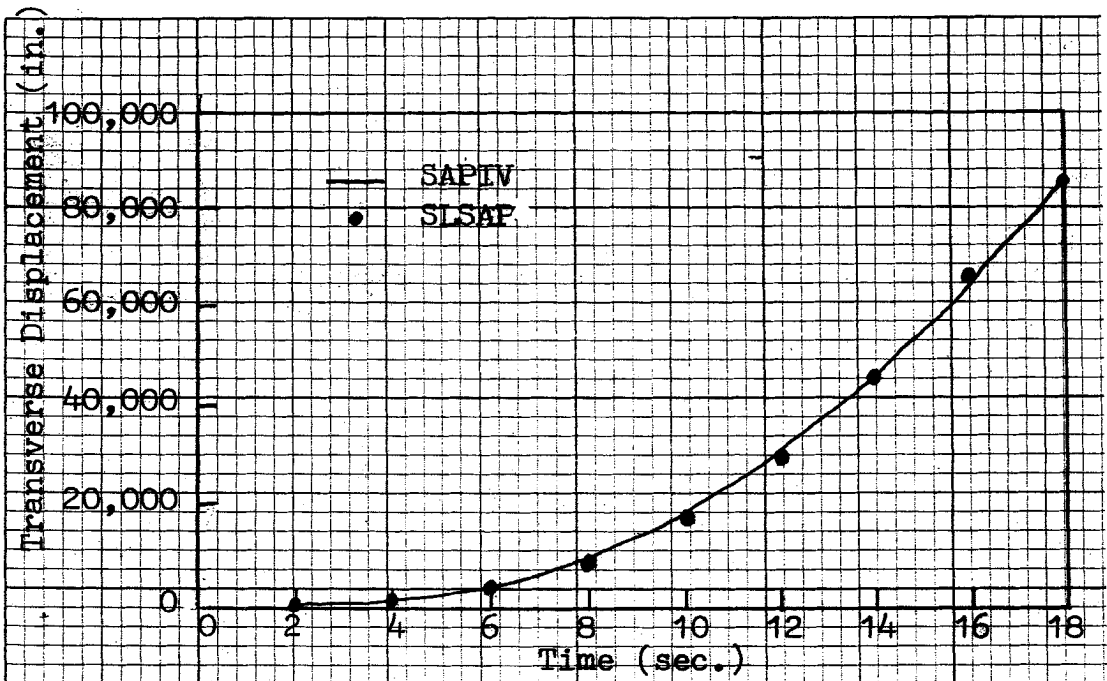


(b) GROUND ACCELERATION APPLIED AT NODE 1

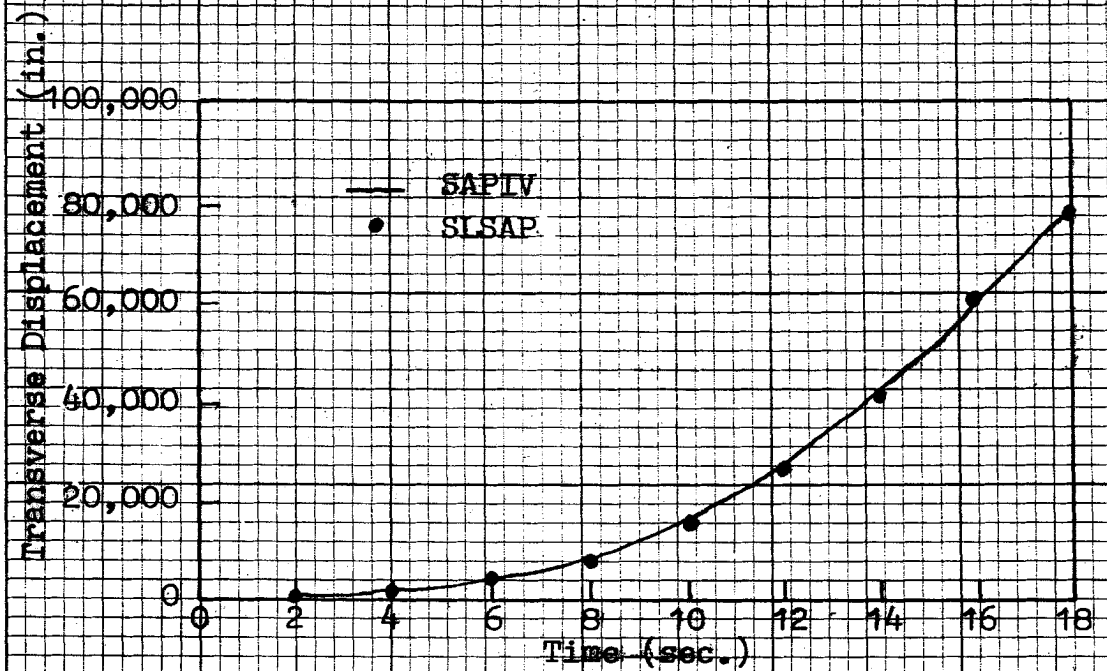
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FIGURE C-77

MODEL FOR RESPONSE HISTORY ANALYSIS
 FOR SLSAP AND SAPIV (SLSAP
 VALIDATION PROBLEM 7)



(a) Node 5

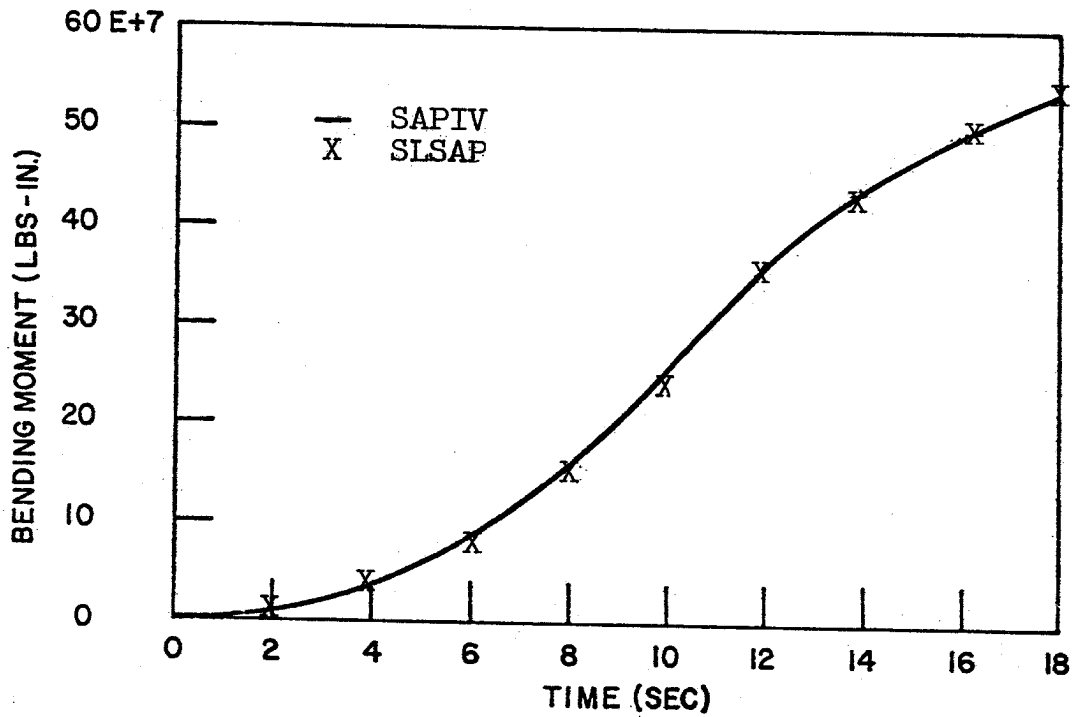


(b) Node 9

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FIGURE C-78

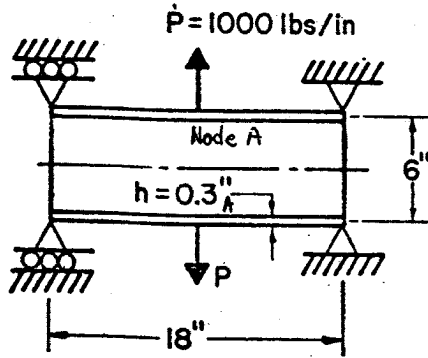
COMPARISON OF SLSAP AND SAPIV TRANSVERSE
 DEFLECTIONS OF THE CANTILEVER BEAM
 (SLSAP VALIDATION PROBLEM 7)



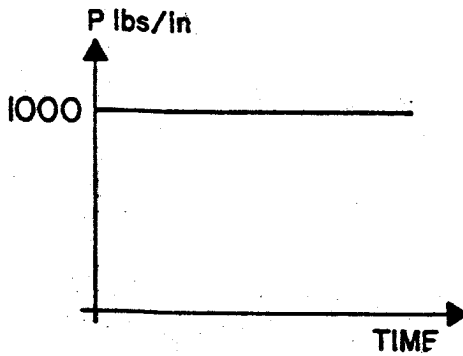
CLINTON POWER STATION
UPDATED SAFETY ANALYSIS REPORT

FIGURE C-79

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

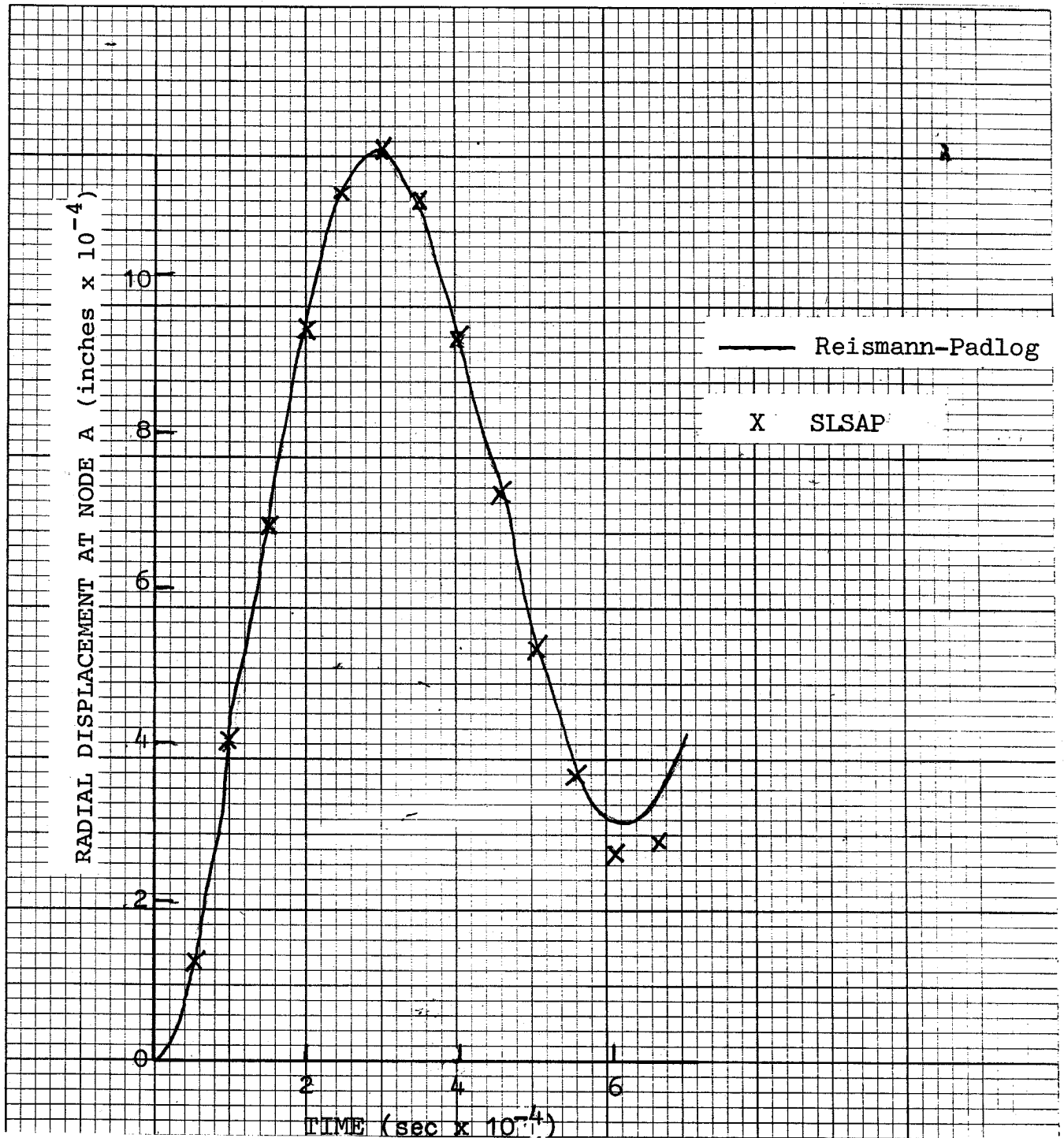


a) CYLINDRICAL TUBE



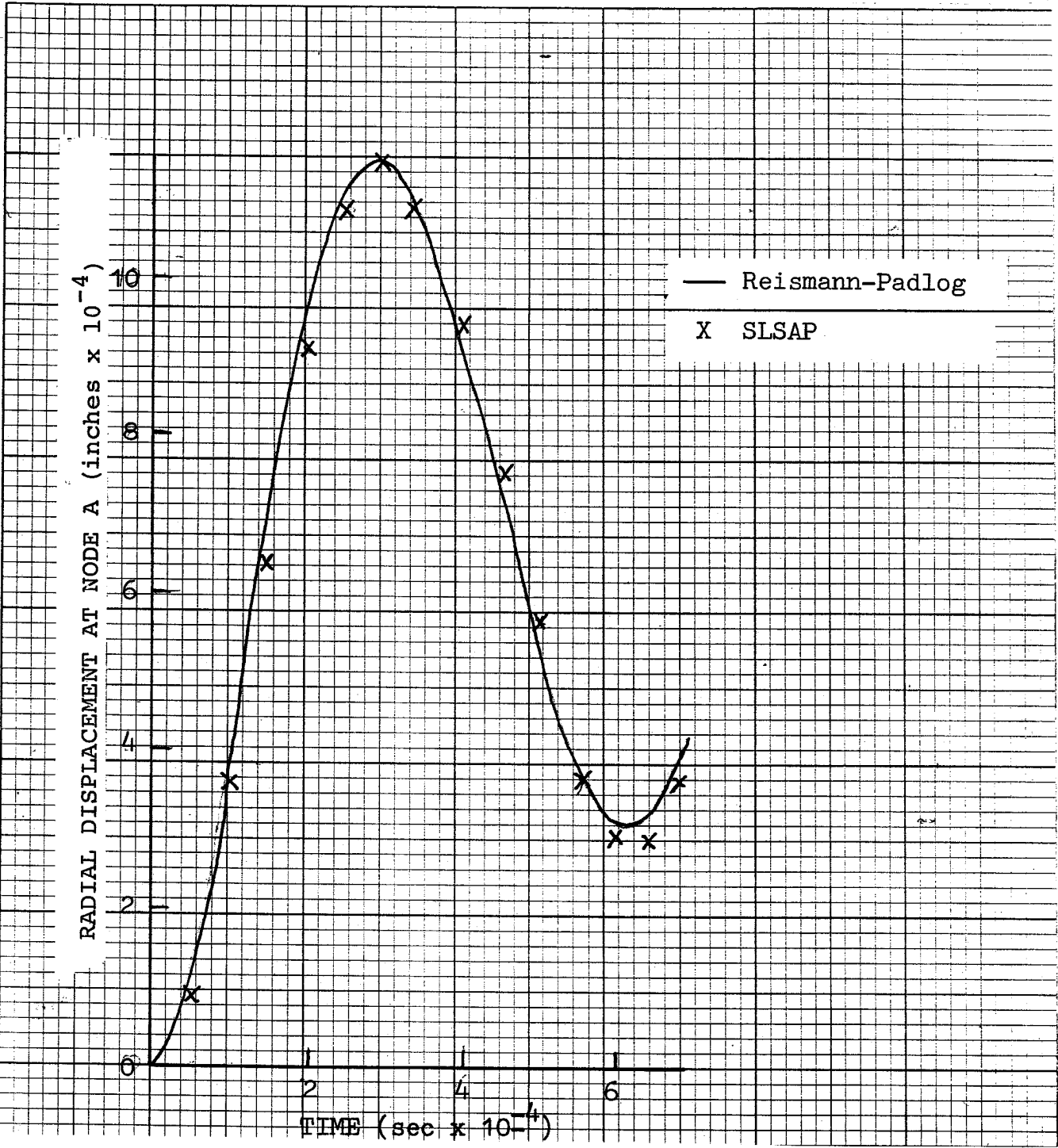
b) TIME VARIATION OF LOAD

| |
|--|
| <p>CLINTON POWER STATION UPDATED SAFETY ANALYSIS REPORT</p> |
| <p>FIGURE C-80 CYLINDRICAL TUBE AND LOAD HISTORY FOR SLSAP AND SAPIV MODE SUPERPOSITION AND DIRECT INTEGRATION ANALYSES (SLSAP VALIDATION PROBLEM 8)</p> |

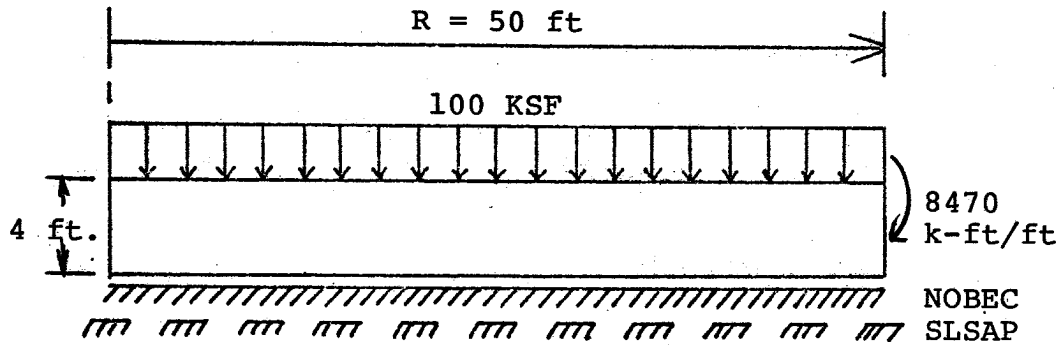


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FIGURE C-81
 DISPLACEMENT COMPARISON OF SLSAP MODE
 SUPERPOSITION AND REFERENCE 45 FOR THE
 CYLINDRICAL TUBE
 (SLSAP VALIDATION PROBLEM 8)



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 FIGURE C-82
 DISPLACEMENT COMPARISON OF SLSAP DIRECT
 INTEGRATION AND REFERENCE 45 FOR THE
 CYLINDRICAL TUBE (SLSAP VALIDATION
 PROBLEM 8)



$$E = 593141.8 \text{ kips/ft}^2$$

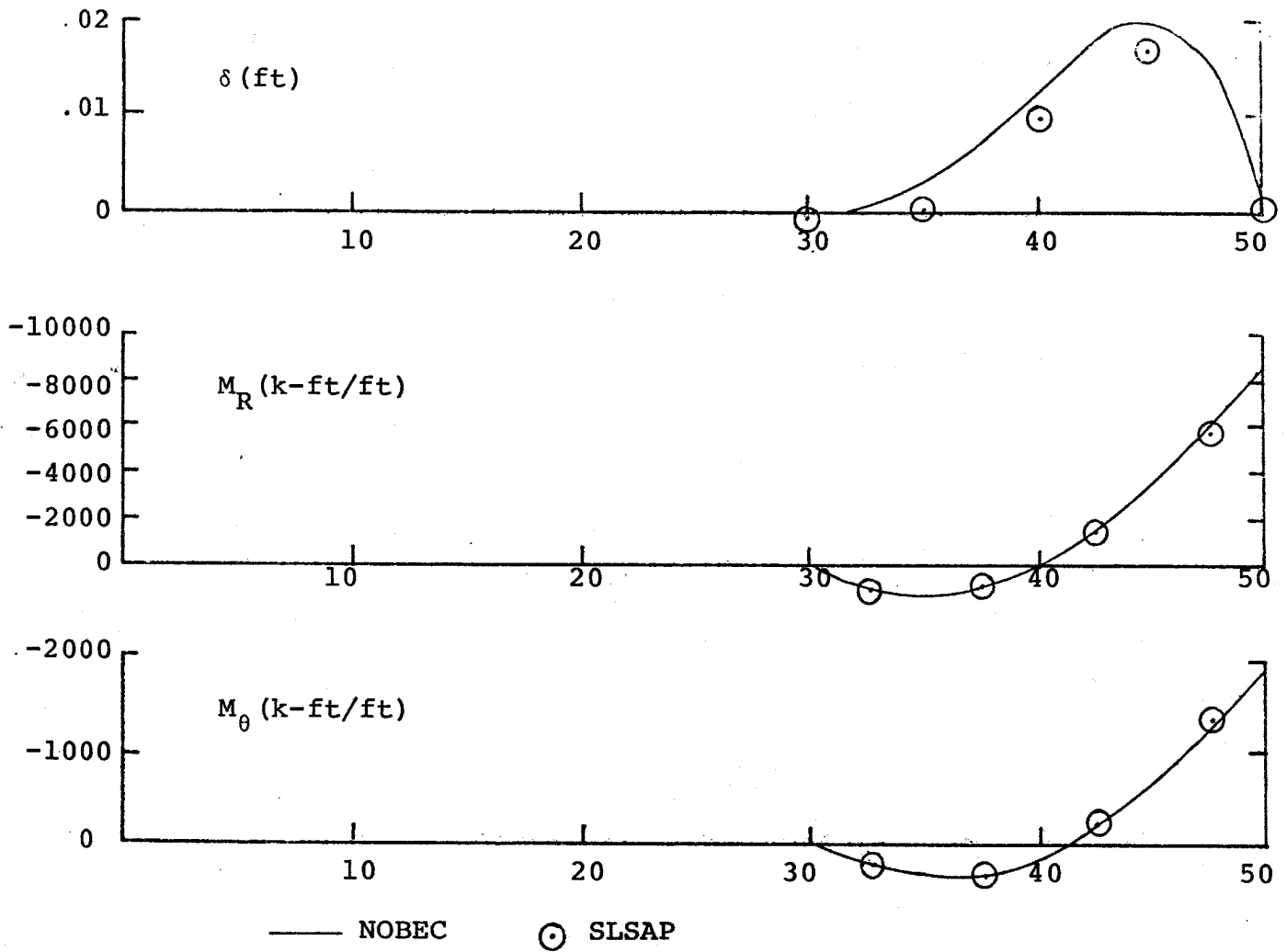
$$\nu = 0.205$$

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

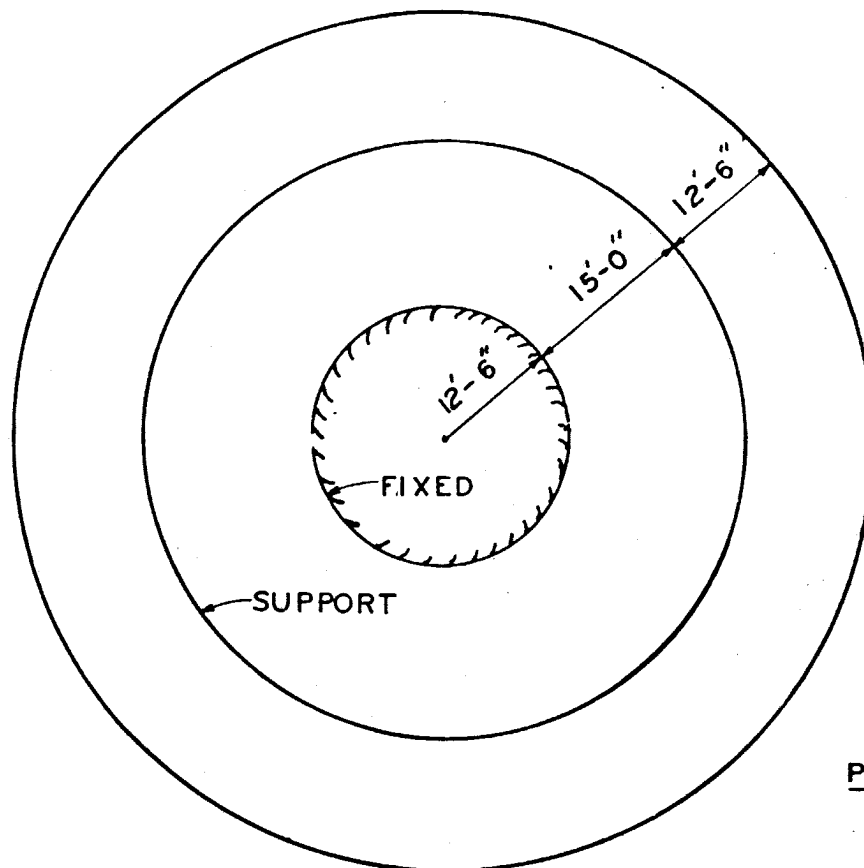
CLINTON POWER STATION
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FIGURE C-83

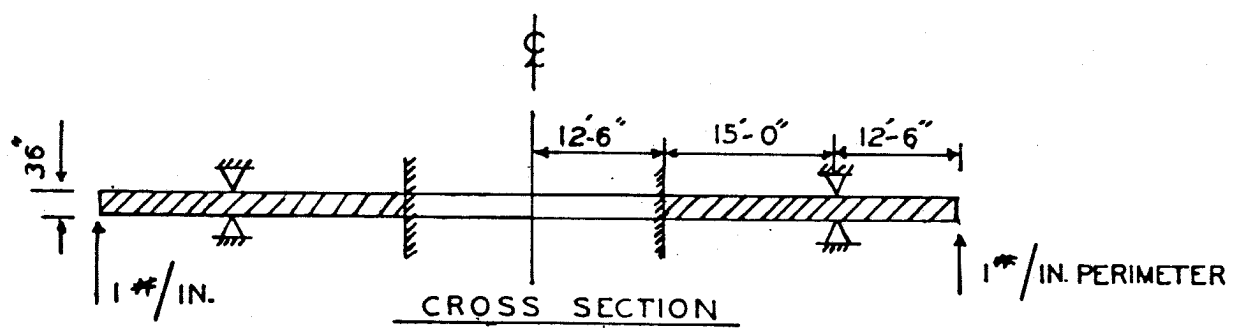
CIRCULAR PLATE ON A RIGID FOUNDATION
 FOR SLSAP AND NOBEC (SLSAP VALIDATION
 PROBLEM 9)



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



PLAN



CROSS SECTION

MATERIAL PROPERTIES

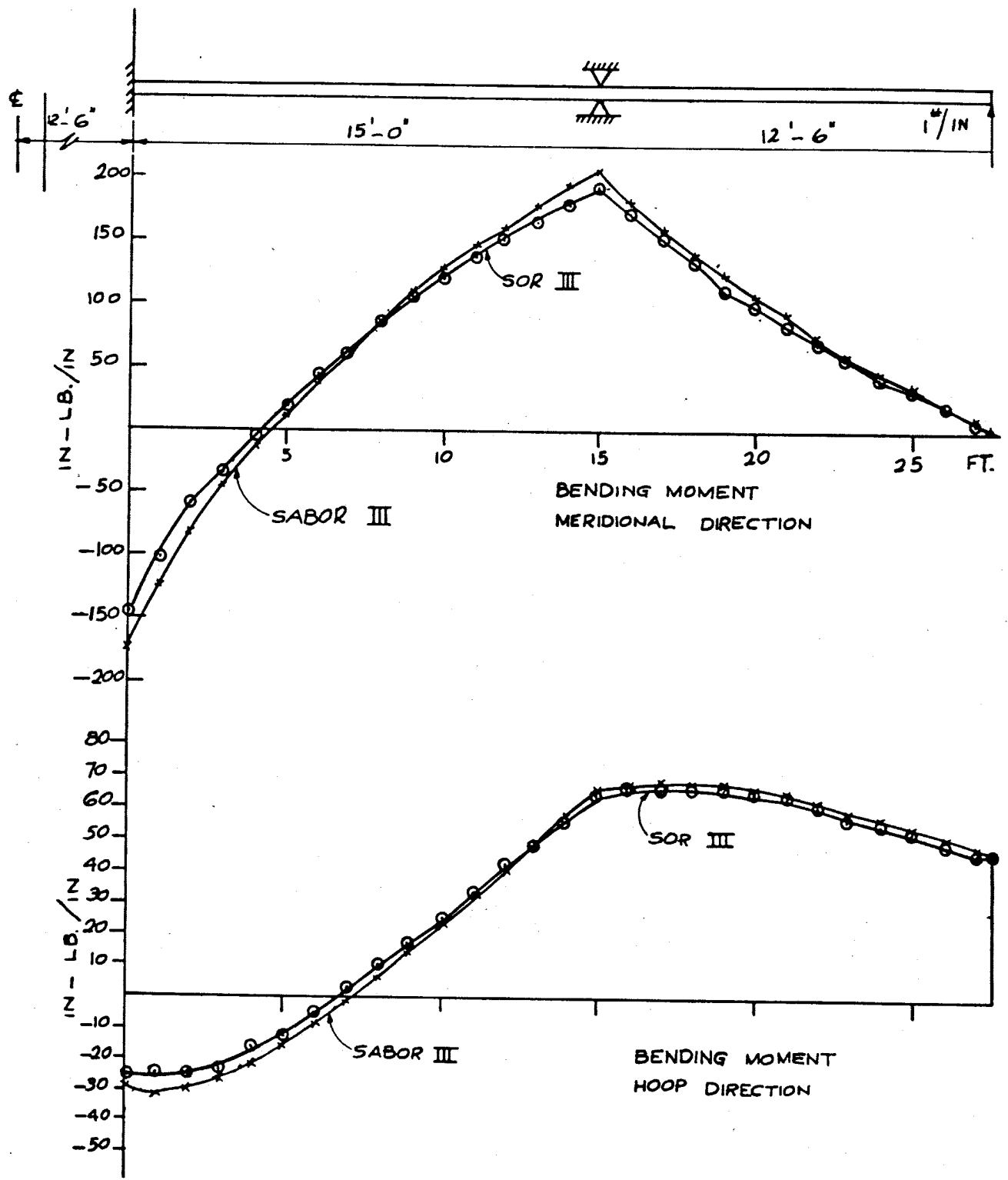
$\nu = 0.17$

$E = 4 \times 10^6 \text{ PSI}$

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FIGURE C-85

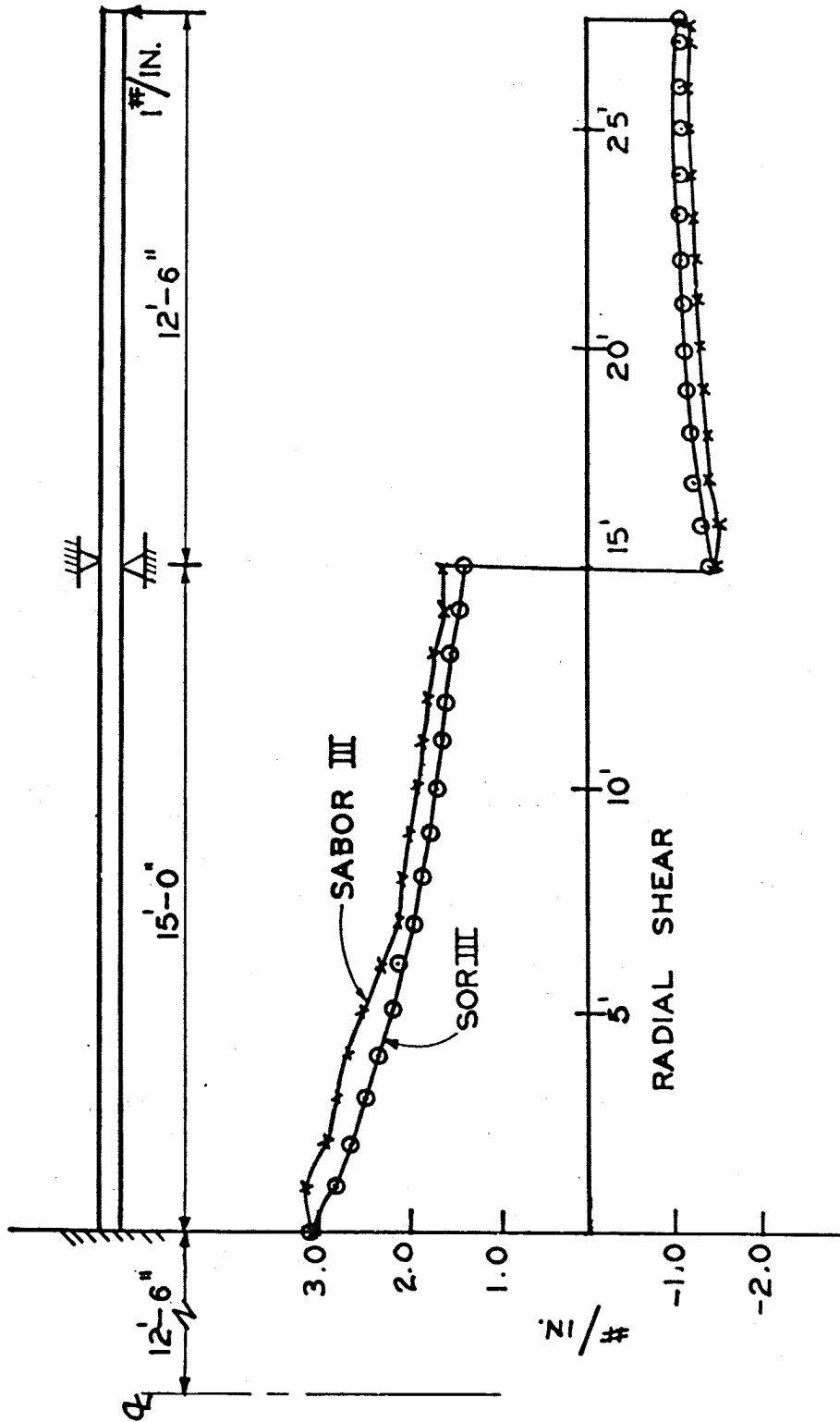
CIRCULAR PLATE FOR SOR-III EXAMPLE



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FIGURE C-86

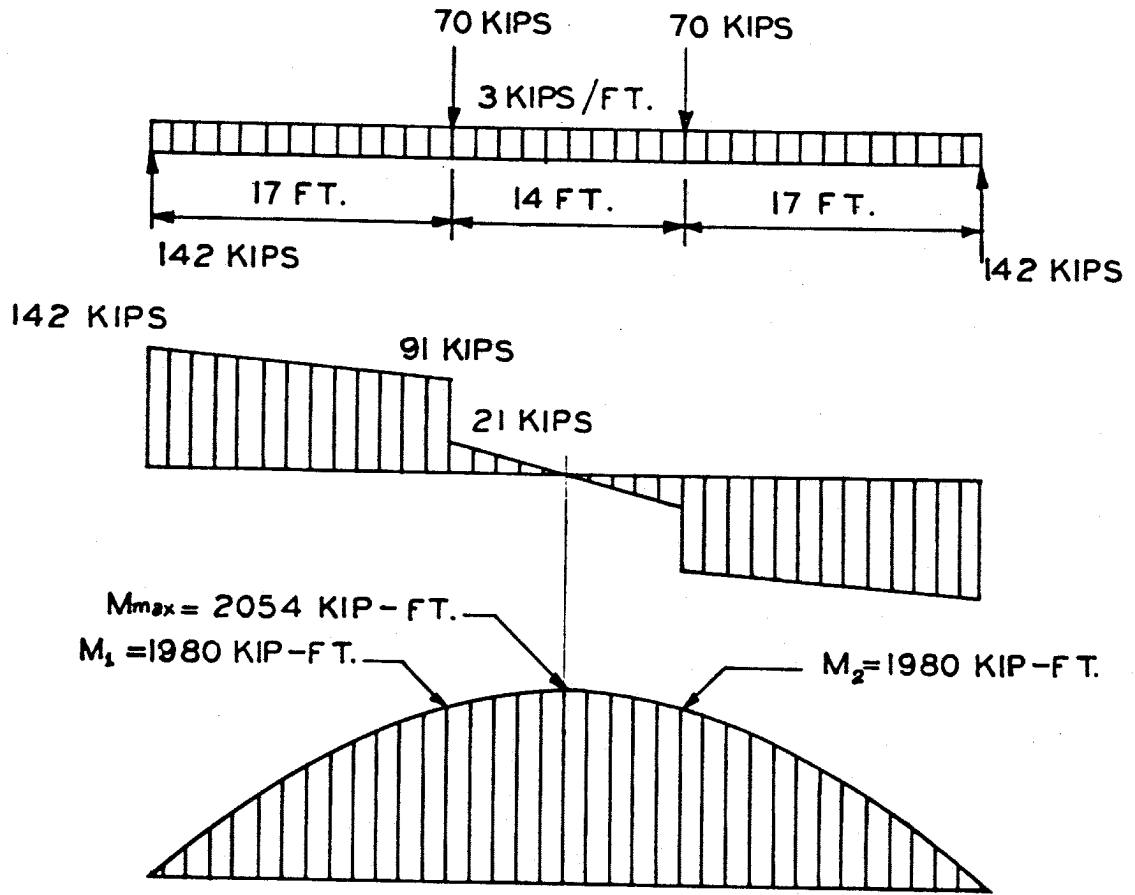
MOMENT COMPARISON SABOR-III AND SOR-III



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FIGURE C-87

RADIAL SHEAR COMPARISON FOR
 SABOR-III AND SOR-III

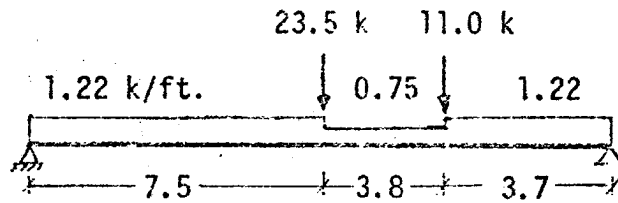


SHEAR AND MOMENT DIAGRAMS

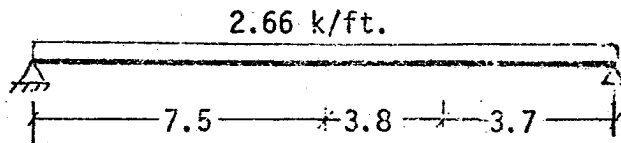
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FIGURE C-88

LOADS ON BEAM (STAND VALIDATION
 PROBLEM 4)



Basic Loading 1



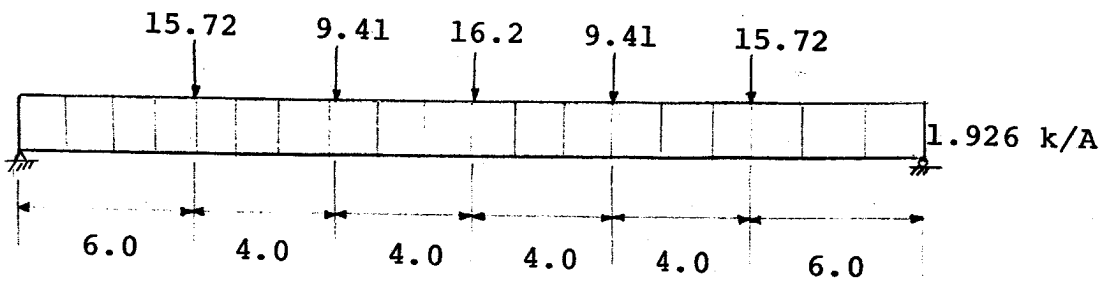
Basic Loading 2

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

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FIGURE C-89

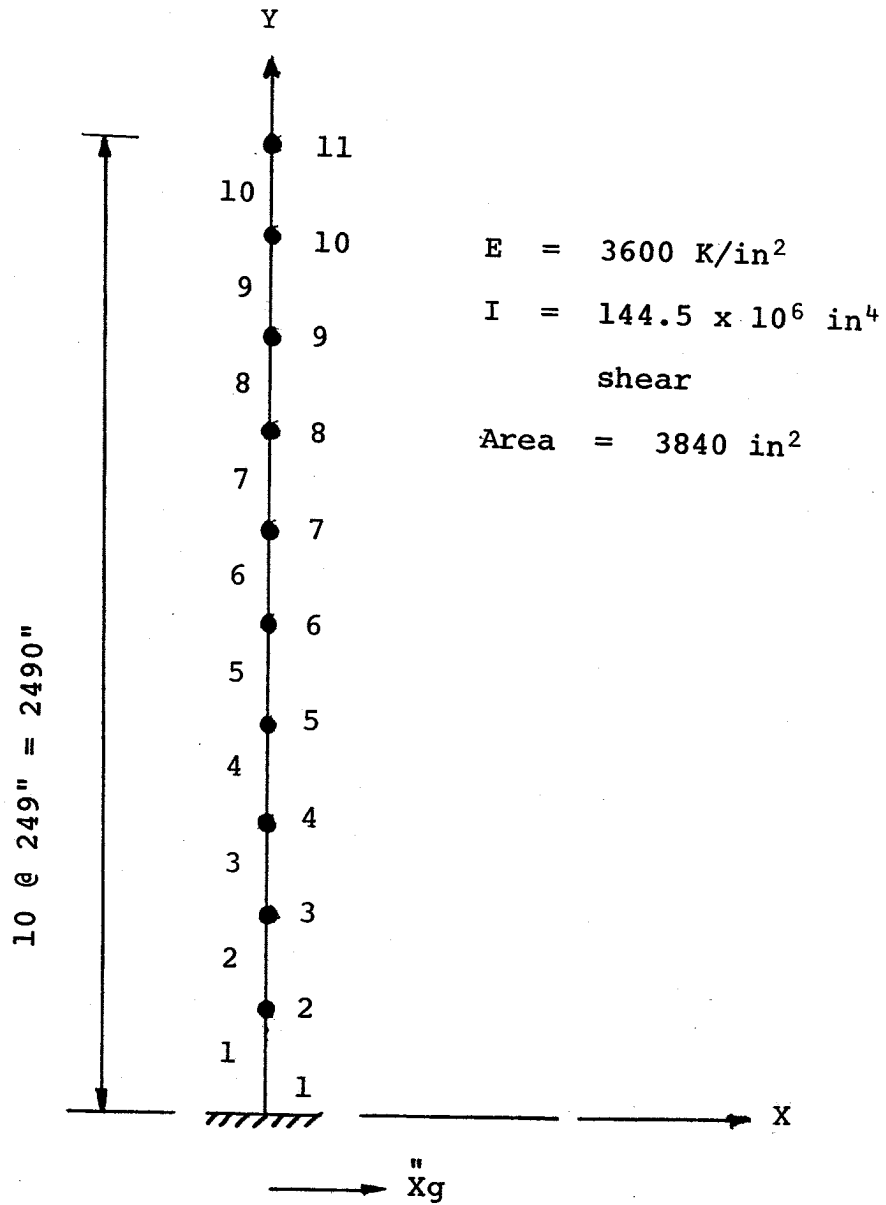
TRANSVERSE LOADS ON BEAM
 (STAND VALIDATION PROBLEM 5)



CLINTON POWER STATION
 UPDATED SAFETY ANALYSIS REPORT

FIGURE C-90

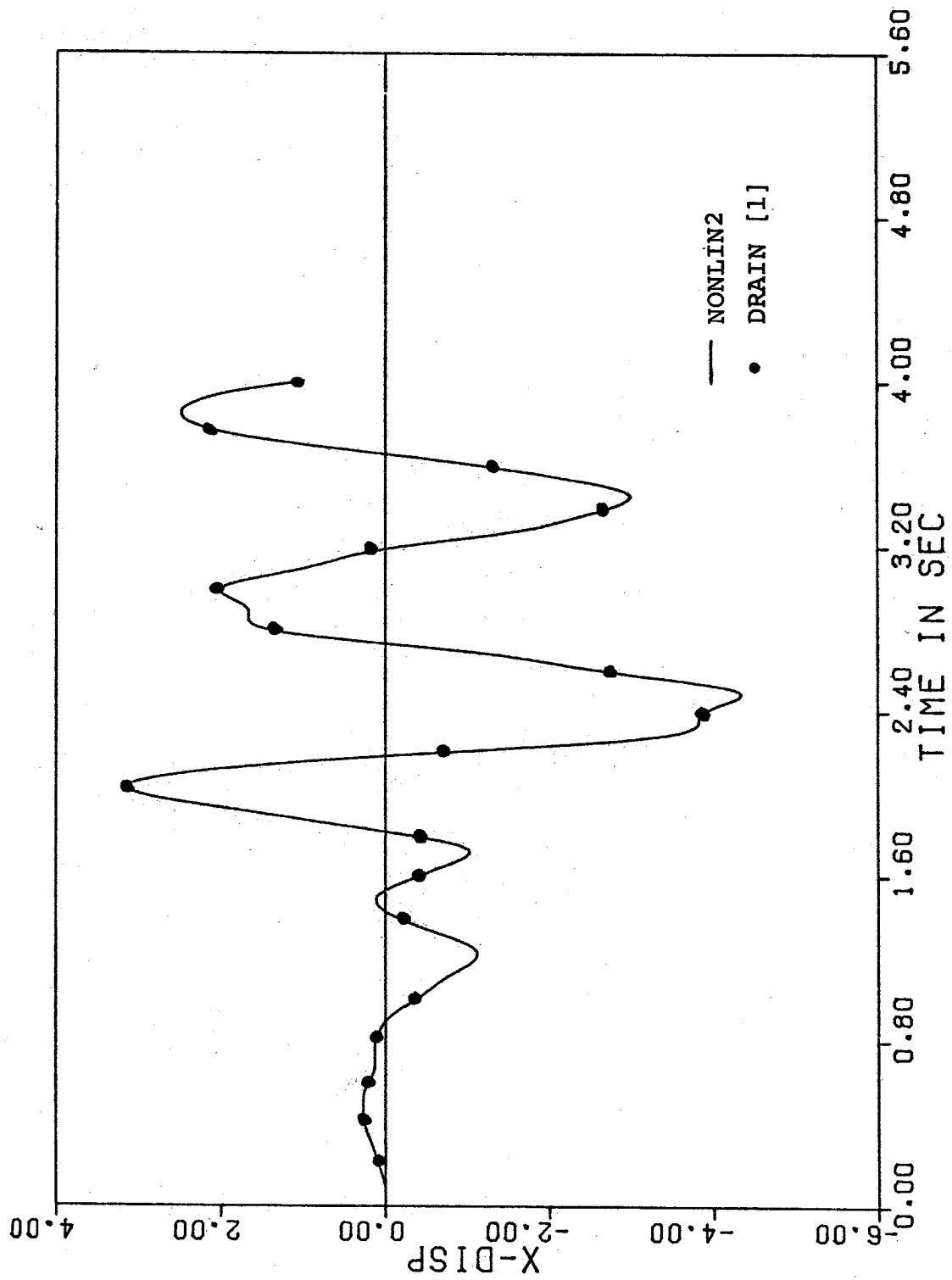
TRANSVERSE LOADS ON BEAM
 (STAND VALIDATION PROBLEM 6)



CLINTON POWER STATION
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FIGURE C-91

TEN-STORY SHEAR WALL MODEL
 FOR NONLIN2 PROGRAM

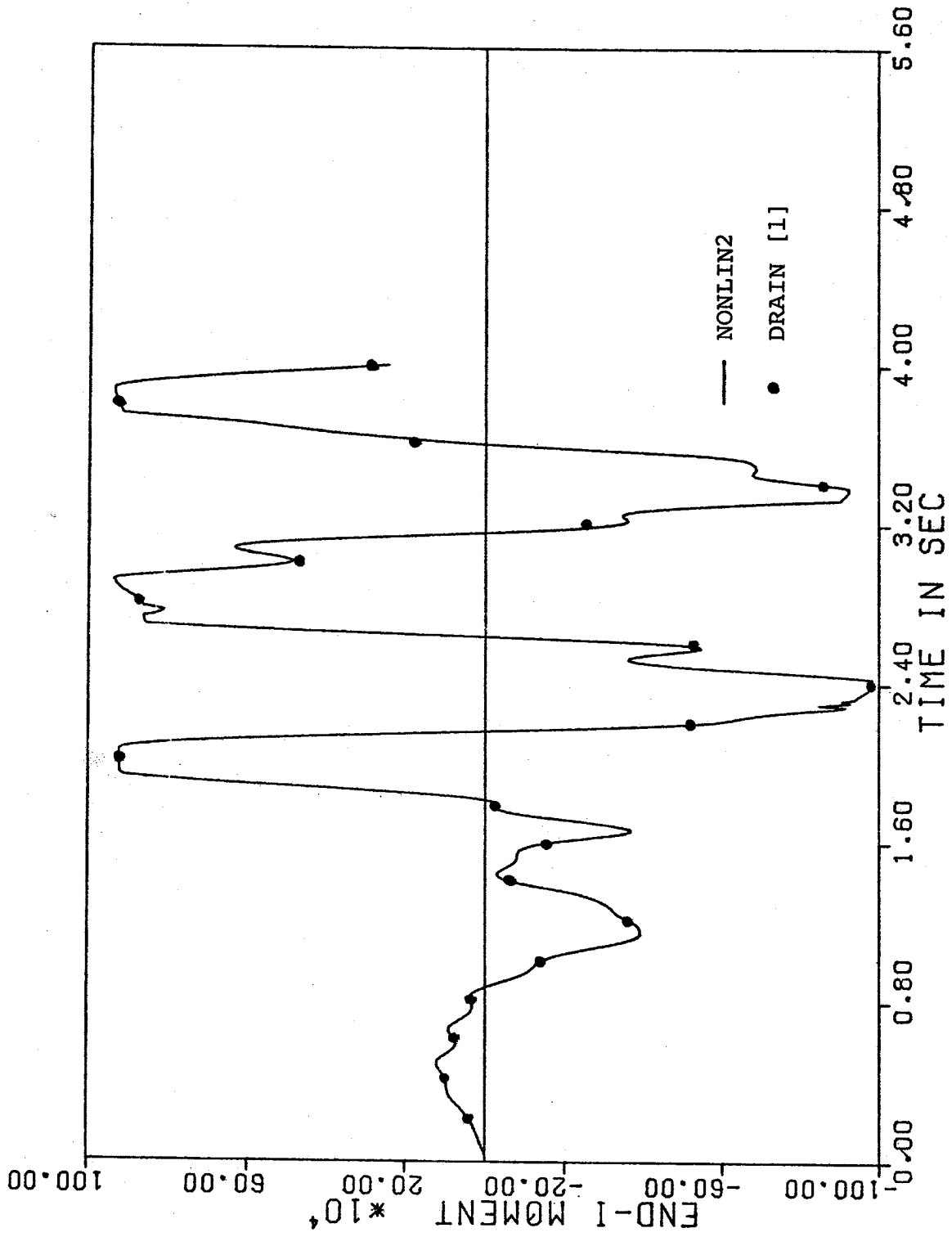


X-DISP FOR NODE 11

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FIGURE C-92

COMPARISON OF DISPLACEMENT FOR
 NODE 11 (NONLIN2)

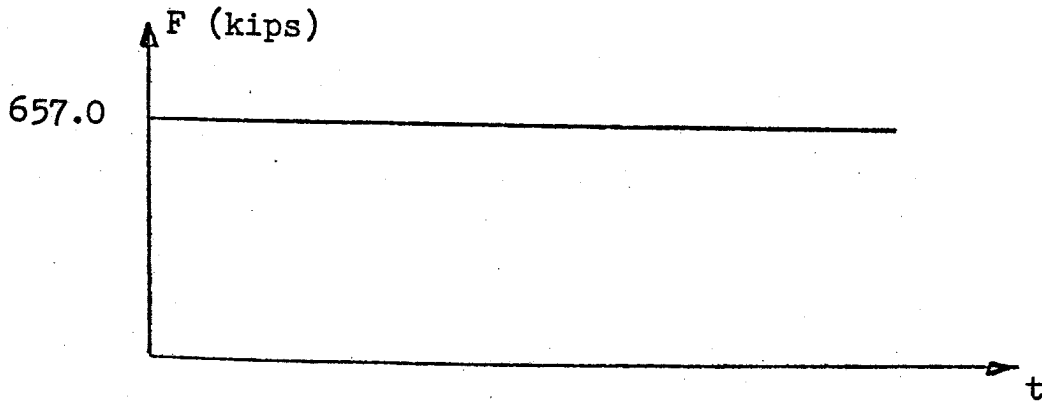
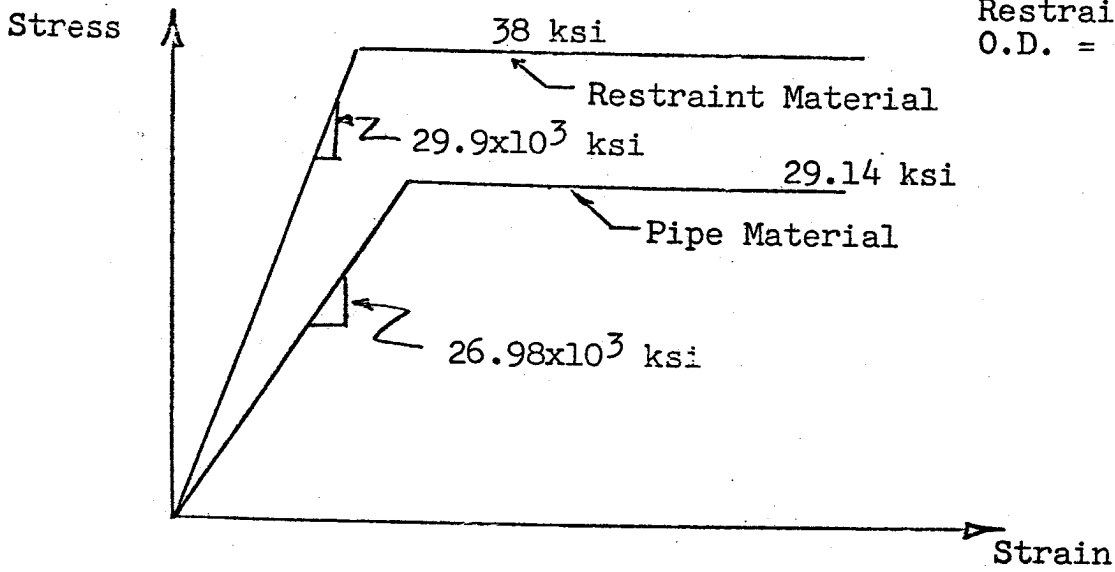
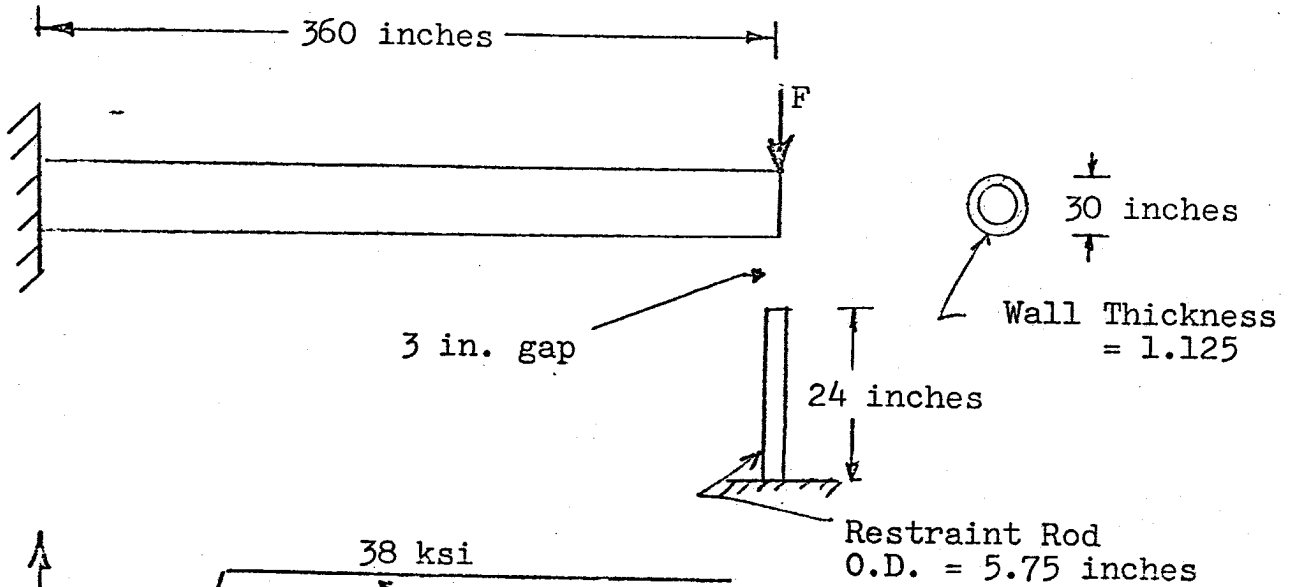


END-I MOMENT FOR MEMBER 1

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FIGURE C-93

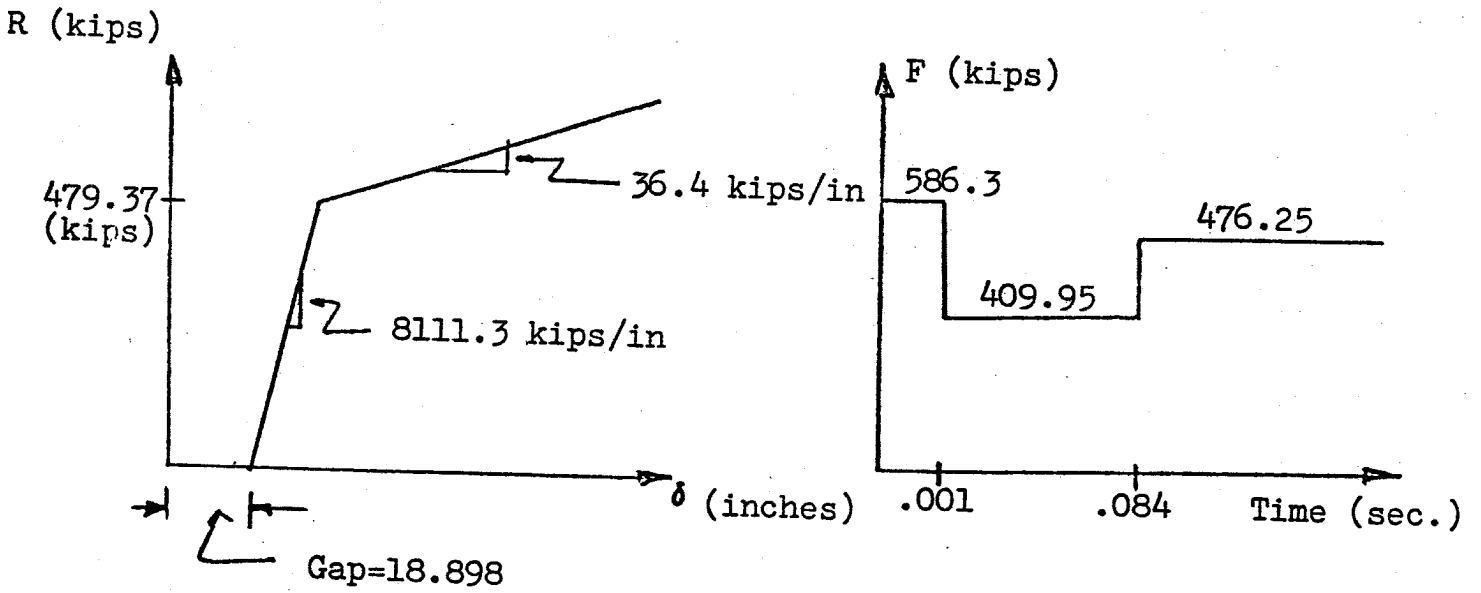
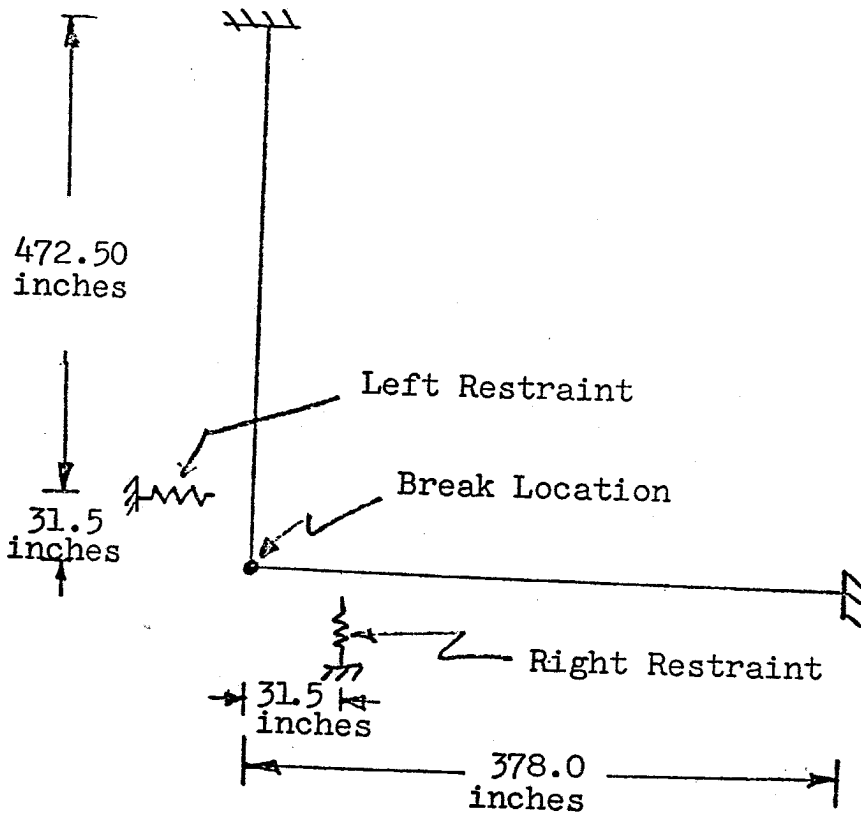
COMPARISON OF MOMENT FOR MEMBER 1
 (NONLIN2)



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FIGURE C-94

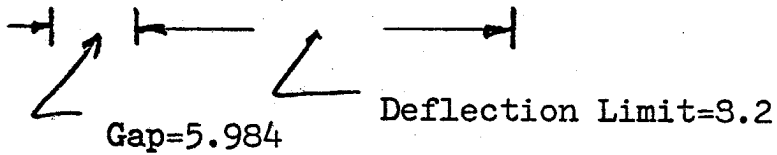
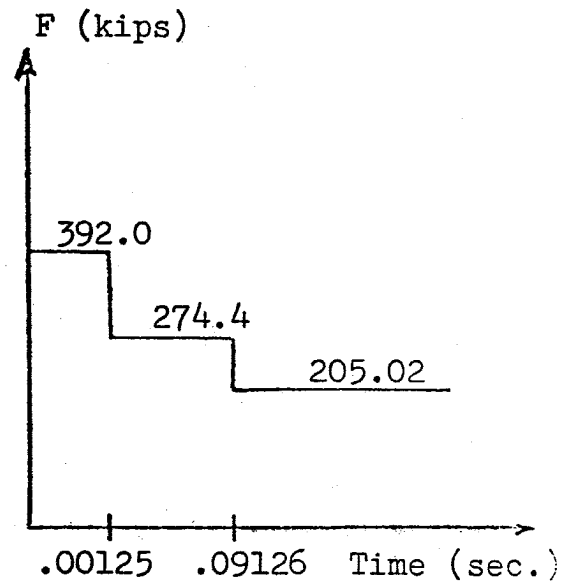
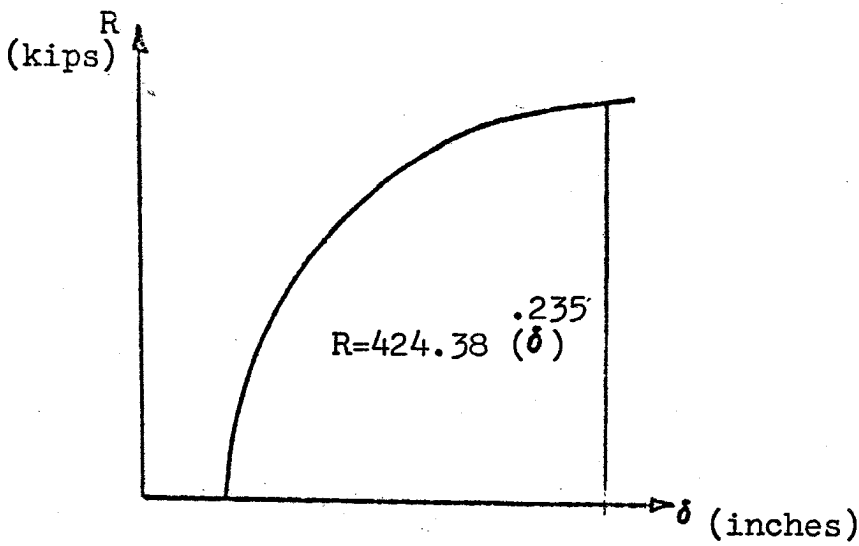
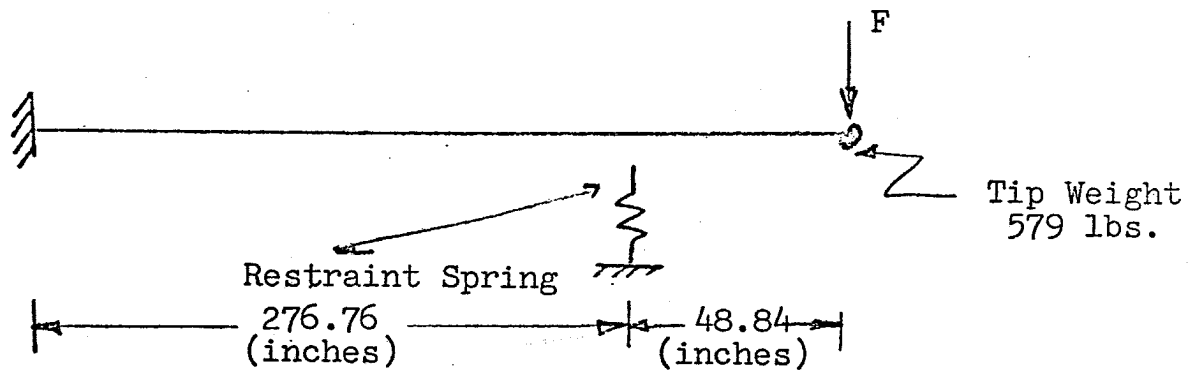
PIPE WHIP MODEL FOR EXAMPLE 1 OF PWRA



CLINTON POWER STATION
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FIGURE C-95

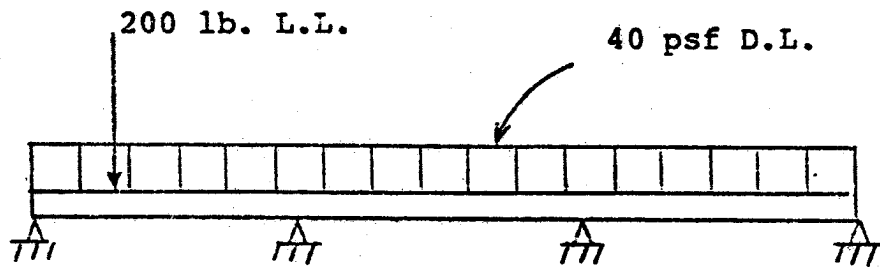
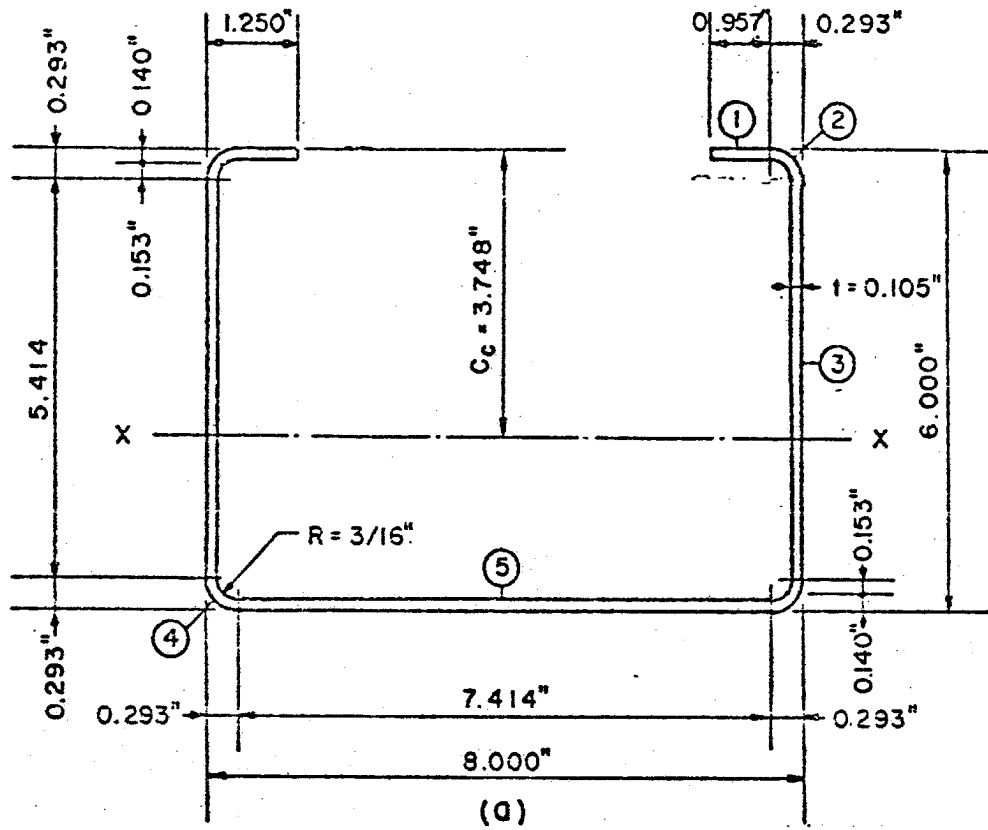
PIPE WHIP MODEL FOR EXAMPLE 2 OF PWRA



CLINTON POWER STATION
 UPDATED SAFETY ANALYSIS REPORT

FIGURE C-96

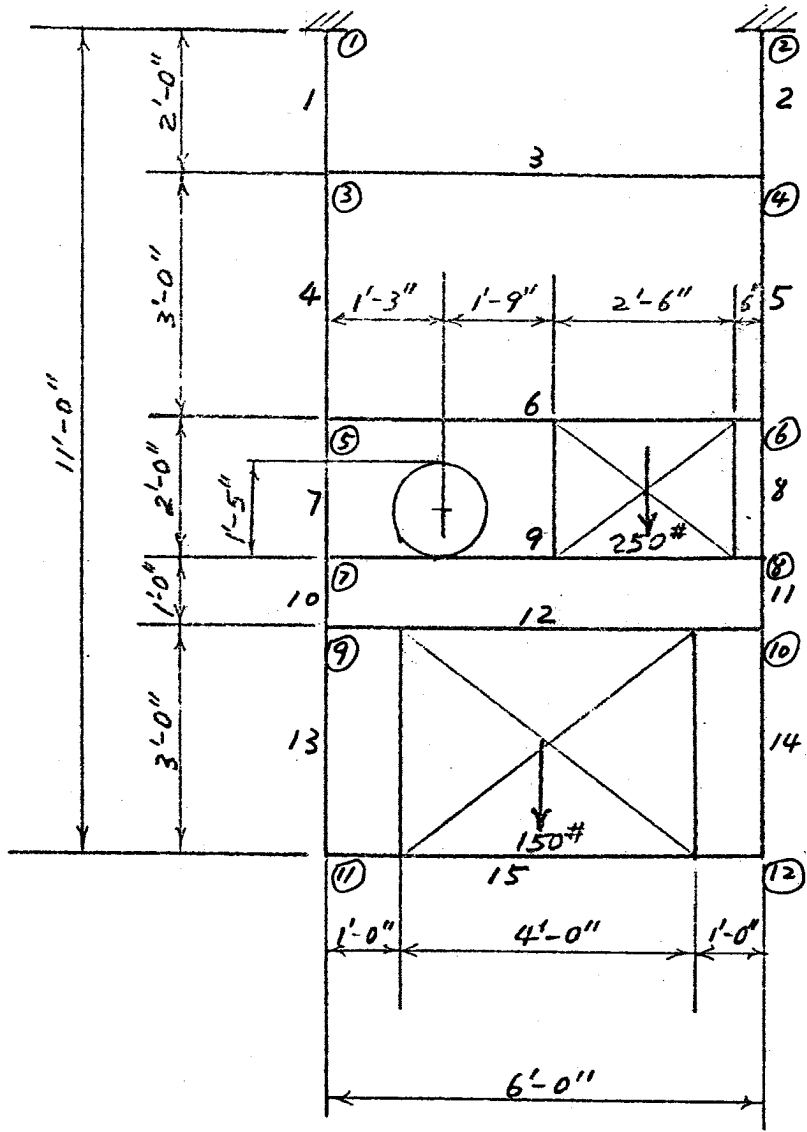
PIPE WHIP MODEL FOR EXAMPLE 3 OF PWRA



vertical seismic design load = 1.5g

horizontal seismic design load = 4.5g

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| <p>CLINTON POWER STATION UPDATED SAFETY ANALYSIS REPORT</p> |
| <p>FIGURE C-97</p> |
| <p>CABLE TRAY MODEL FOR SEISHANG PROGRAM</p> |



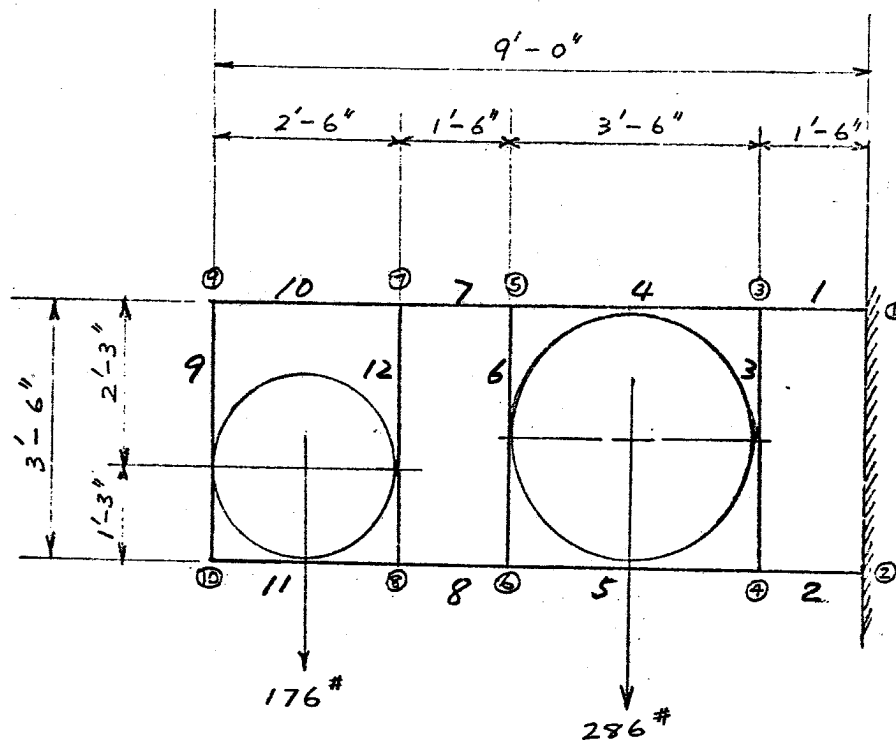
$I = 2.22 \text{ in}^4$

$A = 2.75 \text{ in}^2$

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FIGURE C-98

CEILING MOUNTED SUPPORT MODEL
 FOR SEISHANG PROGRAM



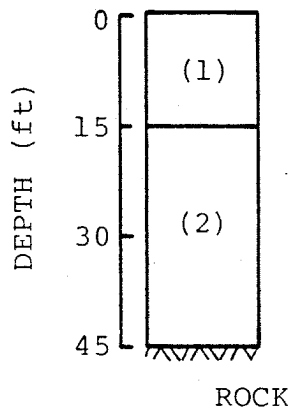
$$I = 1.24 \text{ in}^4$$

$$A = 1.44 \text{ in}^2$$

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FIGURE C-99
 WALL MOUNTED SUPPORT MODEL FOR
 SEISHANG PROGRAM

Excavated Effective
Soil Pressure
= 4232.8 psf



| <u>UNIT WT.</u> (pcf) | $\frac{P > P_c}{C_c}$ $\frac{1}{1+e_o}$ | $\frac{P < P_c}{C_r}$ $\frac{1}{1+e_o}$ | <u>P_c (psf)</u> |
|--------------------------|--|--|----------------------------|
|--------------------------|--|--|----------------------------|

| | | | |
|------|------|-------|--------|
| 85.6 | 0.04 | 0.004 | 20,000 |
| 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

=====
=====

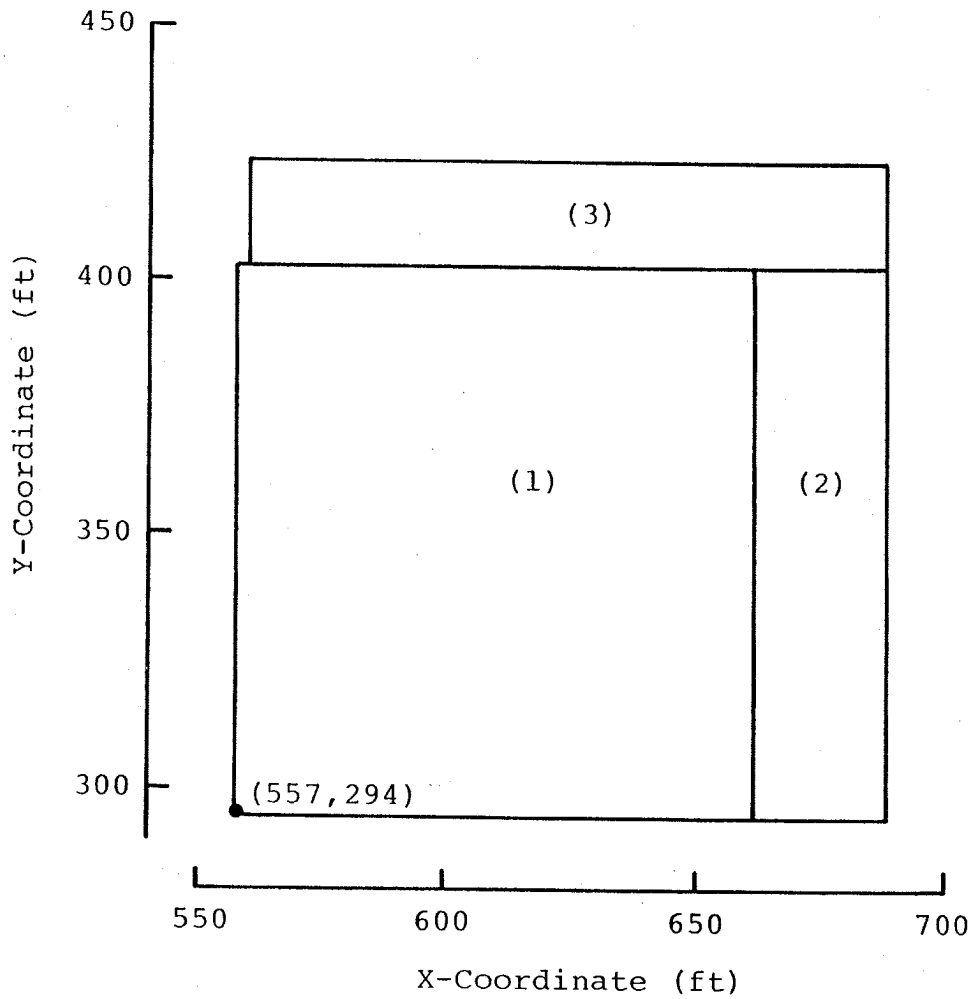
<= NOTE: This is a soil formula

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FIGURE C101

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

| <u>LOADING AREA</u> | <u>LENGTH (ft)</u> | <u>WIDTH (ft)</u> | <u>LOAD (psf)</u> | <u>DEPTH (ft)</u> |
|---------------------|--------------------|-------------------|-------------------|-------------------|
| (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