Department of Energy
Washington, D.C. 20545

Docket No. 50-537
HQ:S:82:044

JUN 08 1982

Mr. Paul S. Check, Director
CRBR Program Office
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dr. Mr. Check:

RESPONSES TO REQUEST FOR ADDITIONAL INFORMATION - STRUCTURAL ENGINEERING

Reference: Letter, P. S. Check to J. R. Longenecker, "CRBRP Request for Additional Information," dated February 26, 1982

This letter formally responds to your request for additional information contained in the reference letter.

Enclosed are responses to Questions CS 220.2, 6, 20, 21, 22, 24, 27, and 39 that will also be incorporated into the PSAR Amendment 69; scheduled for submittal later in June.

Sincerely,

John R. Longenecker

John R. Longenecker
Acting Director, Office of the
Clinch River Breeder Reactor
Plant Project
Office of Nuclear Energy

Enclosures

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Question CS220.2 (3.5.4.1 & 3.5.4.2)

The revised Petry equation for penetration depth as a function of velocity seems to have been copied incorrectly in that the term in the exponential is dimensionally incorrect and the term V', which is a logarithm, is given with a dimension (the velocity dimension should be incorporated in K). Further the K in the text does not agree with the K in Figure 3.5-1. It is requested that corrections be made. Indicate how you calculate d_m for a noncylindrical or nonspherical projectile?

Also indicate if the wall thicknesses you determined meet the requirements as shown in Table 1 on Page 3.5.3-6 of the revised SRP Section 3.5.

Response:

PSAR Section 3.5.4.1 will be revised to show that the term V' does not have the dimension ft/sec and that the term a should be equated to Tp/KApV'. The value of K in PSAR Figure 3.5-1 will be corrected to 2.76×10^{-3} to agree with PSAR Section 3.5.4.1.

For a noncylindrical or nonspherical projectile or missile, d_m is calculated by determining the equivalent diameter of a noncircular missile:

$$d_m = \sqrt{\frac{4A}{\pi}}$$

where A = cross-sectional area of missile

The wall thicknesses determined in the CRBRP design are consistent with the requirements shown in Table 1 on page 3.5.3-6 of the revised SRP. For concrete strength of 4000 psi, the minimum wall thickness designed is 27" which is greater than the required minimum thickness of 20". The minimum roof thickness of 27 as designed is more than the required minimum thickness of 16".

3.5.4 Barrier Design Procedures

Missile resistant barriers and structures will be designed to withstand and absorb missile impact loads without being fully perforated in order to prevent damage to protected components. In addition, the overall structural response will be evaluated to assure the structural integrity due to missile impact loads. For concrete missile barriers, the possibility of generation of secondary missiles due to spalling or scabbing will also be taken into consideration so that protective measures can be provided.

The design procedures are described below.

3.5.4.1 Penetration Into Concrete Target Structures

To arrive at a method for computing the penetration into concrete walls, formulas reviewed in ORNL-NSIC-22 (Ref. 1) were studied. Four equations were studied in ORNL-NSIC-22. Two of these, the Army Corps of Engineers formula and the National Defense Research Committee formula, do not apply for impact velocities under 500 ft/sec. The remaining two equations are the modified
15 Petry formula and the Ballistic Research Laboratory formula. These two formulas were compared by determining the depths of penetration for a 6-inch-diameter missile of 100 pounds and a 16-inch-diameter missile of 2,500 pounds
6 with velocities in the range of 0 to 500 ft/sec. As seen in Figures 3.5-1 and 3.5-2, the Petry formula is the more conservative for velocities greater than
6 150 and 200 ft/sec. respectively.

Therefore, the depth of a concrete wall or slab to which a missile can penetrate is estimated by use of the modified Petry formula:

$$D' = KA_pV' \left[1 + e^{-4(a-2)}\right]$$

where

6 D' = depth of penetration (ft.)

61 K, a material constant = 2.76×10^{-3} $\dfrac{(ft^2 - sec.)}{lb}$

$A_p = \dfrac{\text{missile weight}}{\text{maximum cross-sectional area}}$ (psf)

V = impact velocity (ft./sec.)

$V' = \log_{10}\left(1 + \dfrac{V^2}{215000}\right)$

61 $a = \dfrac{T_p}{KA_pV'}$

T_p = wall or slab thickness (ft.)

For design purpose, all Category I concrete structures will satisfy the requirement; $T_p \geq 2D'$

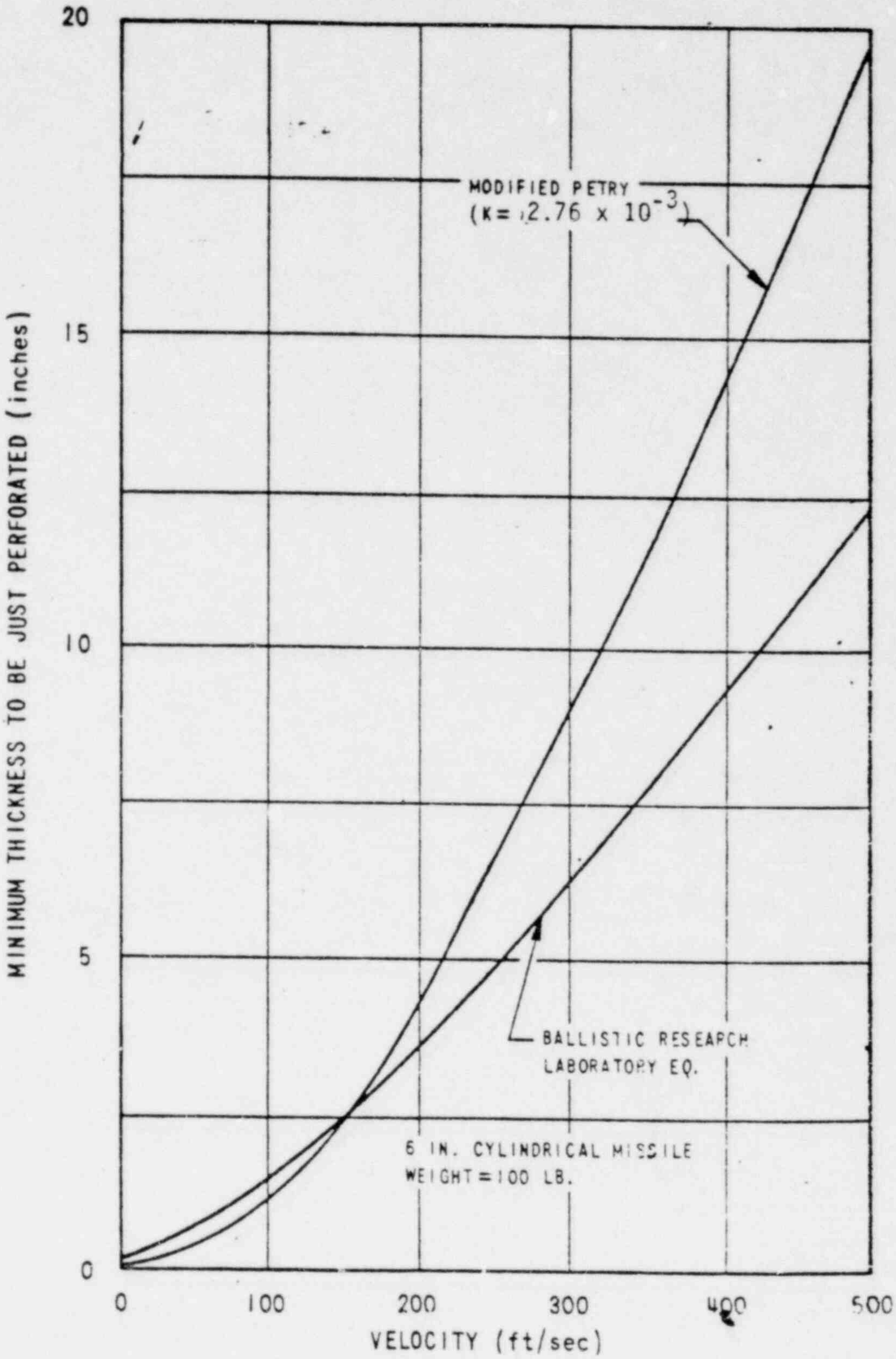


Figure 3.5-1. Comparison of Missile Penetration Formulas for Concrete

6656-1

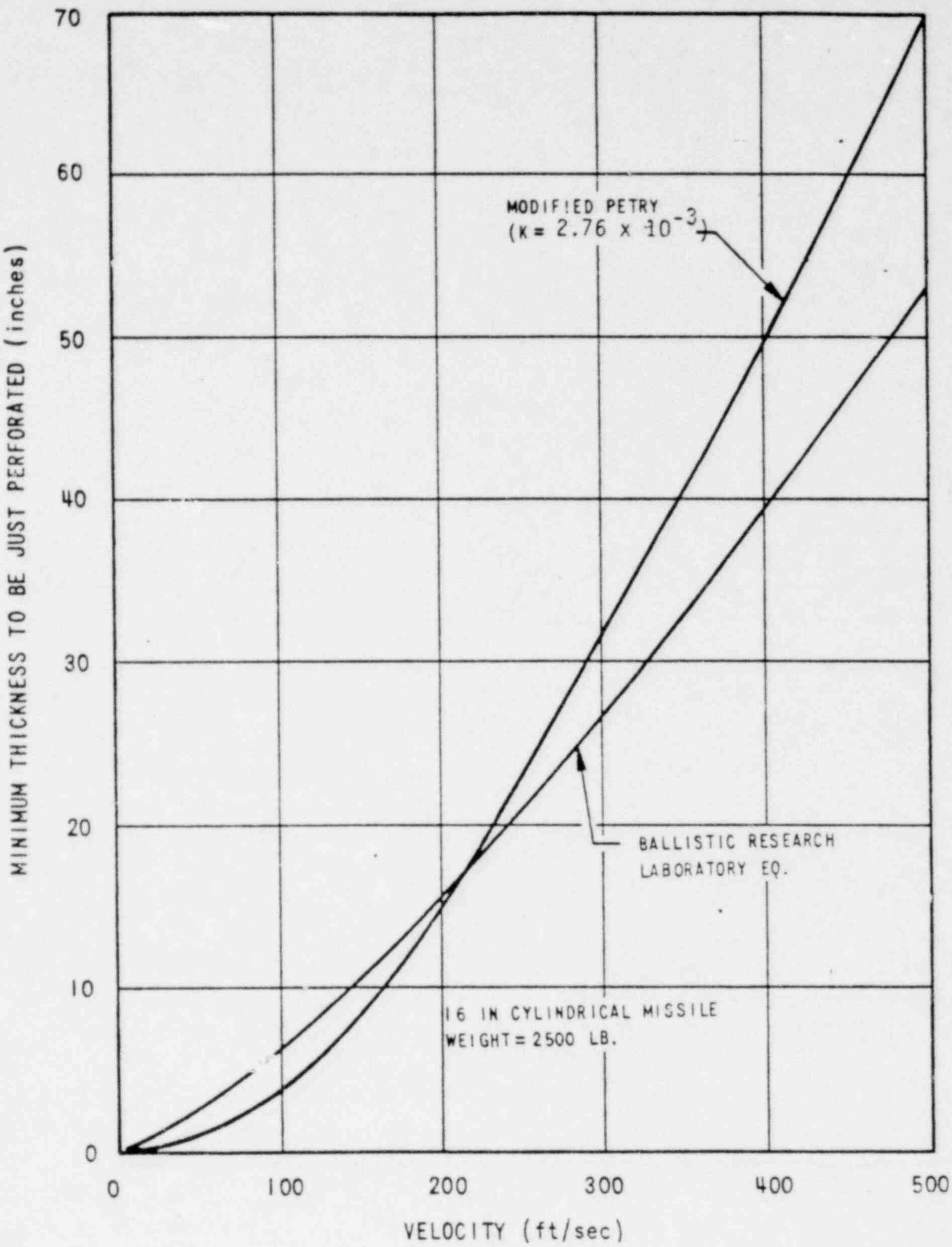


Figure 3.5-2. Comparison of Missile Penetration Formulas for Concrete

6656-2

Question CS220.6 (3.7.1.1)

It is stated that for a lumped-mass-spring type of models the seismic design response spectra will be applied at the foundation. The mathematical models as shown in Figures 3.7-16, 3.7-16A, and 3.7-16B are the lumped-mass-spring type. Indicate how the springs and dashpots representing soil are derived from a static finite element model. Provide a description in detail.

Further, the mathematical models in Figures 3.7-16, 3.7-16A, and 3.7-16B lack numerical details. No one could judge the adequacy of plans for a plant model based on the material given on these diagrams. A full discussion with tables should be provided delineating the numbers, their meanings, etc.

Response:

A detailed description on how foundatin springs and dashpots were calculated is given in Section 3.7.1.6 of the PSAR (Pages 3.7-3a, 3.7-3b, 3.7-3c, and 3.7-4.)

The attached diagrams are the updated mathematical models used in the seismic analysis of the Nuclear Island buildings. Figures 3.7-16, 3.7-16a and 3.7-16b show respectively the mathematical models for the analyses in the North-South, East-West and Vertical directions.

The mathematical models consist of four main parts:

1) The Reactor Service Building

2) The Confinement Structure

3) The Reactor Containment Building

4) The Steam Generator, Electrical Equipment and Control Buildings

The Reactor Vessel and the polar crane are coupled by means of simplified lumped-mass models.

The nodes or mass points correspond to the locations of centers of mass and were selected in general, at the floor elevations. For each of the horizontal analyses (North-South or East-West) three dynamic degrees of freedom per node were allowed (translation, rotation and torsion). For the vertical analysis, one dynamic degree of freedom (translation) was allowed. The beam elements which connect the different nodes vertically are located at the shear centers of their sections and are characterized by areas, shear areas and moments of

inertia (for bending and torsion) of the members and by the modulus of elasticity and Poisson's ratios of the material. The ends of beam elements are connected to the mass points by horizontal rigid members. The four parts of the model are supported by the foundation mat which is assumed to be rigid. This assumption is justified because the mat acts as a diaphragm and is stiffened by the vertical walls of the buildings. The buildings above the mat are interconnected by flexible ties which include cross-coupling between the interconnected nodes. The Reactor Containment Building is connected to the other elements only at the mat and operating floor levels. In the analysis for the vertical direction, the steel containment dome was idealized by using equivalent springs which account for the "breathing" of the dome during a vertical vibration. "Breathing" is a shell mode of vibration that the dome experiences under a vertical motion.

Figure 3.7-16c shows a plan of the Nuclear Island and the system of coordinates. Table 3.7-7 gives the coordinates of the mass points (nodes).

PSAR Section 3.7.2.1.1 and the referenced tables and figures have been updated to include the design information discussed above.

3.7.2.1 Seismic Analysis Method

3.7.2.1.1 Category I Structures

A complete analysis will be performed on each of the Seismic Category I structures to predict its behavior during an earthquake. The SSE and OBE will be considered; each of two orthogonal horizontal directions and the vertical direction will be treated separately and the results combined. The input motions are described in Section 3.7.1.

3.7.2.1.1.1 Nuclear Island

A lumped-mass formulation, with direct integration of the coupled equations of motion will be used.

The Buildings of the Nuclear Island: Reactor Containment (RCB), Confinement, Reactor Service (RSB), Steam Generator (SGB), Electrical Equipment (EEB) and Control Building (CB) have a common foundation mat with the bottom at Elevation 715'-0"; finished grade is at El. 815'-0".

Except for the RCB interior structure, the Category I structures are tied together up to the roof level. The RCB interior structure is tied to the adjacent structures at the mat level and at the operating floor level (Elev. 816'-0").

The structures and foundation materials will be represented in terms of lumped-masses amd massless springs and dashpots.

The inertial properties are characterized by the masses and mass moments of inertia which will be lumped at points selected to assure proper representation of the dynamic behavior of the structures.

The mass points will be, in general, at the elevation of the floors and will be located at the center of mass of the contributing elements.

The stiffness properties are characterized by the areas, shear areas and moments of inertia (for bonding and torsion) of the members and by the moduli of elasticity and Poisson's ratios. The flexible members (beam elements) between floor levels will be assumed located at the shear center of their sections and their ends are defined by the elevations of the mass points. The ends of beam elements are connected to the mass points by horizontal rigid elements.

The soil (rock) structure interaction will be represented by equivalent springs and dashpots. The stiffnesses of the foundation springs will be calculated as described in Section 3.7.1.6.

The damping values to be used for the structures in terms of percent of critical damping are given in Table 3.7-2; the combined damping ratios for the structures (steel containment and concrete buildings) will be calculated based on the equation:

$$\bar{\beta}_j = \frac{\{\phi\}^T (\bar{K}) \{\phi\}}{\{\phi\}^T (K) \{\phi\}}$$

Where:

(K) = assembled stiffness matrix for fixed base structure

$\overline{\beta}_j$ = equivalent modal damping ratio of the jth mode for the fixed base structure

$(\overline{K})$ = modified stiffness matrix for the fixed base structure constructed from element matrices formed by the product of the damping ratio for the element and its stiffness matrix.

ϕ = jth normalized modal vector for the fixed base structure

These damping ratios together with the damping coefficients associated with the foundation dampers will be used in the formulation of the damping matrix for the soil-structure system.

The damping coefficients for the foundation dampers (translational, rocking and torsional) will be calculated based on the equations for geometrical damping in an elastci half-space using equivalent half-space dynamic properties derived from the spring stiffness. The equations of Appendix 3.7-A, Sections C.3.1 and C.3.2 will be used.

Three basic mathematical models will be used, one for each directional component of the earthquake. Figures 3.7-16, 3.7-16A and 3.7-16B respectively show sketches of the mathematical models for the North-South, East-West and Vertical directions. Figure 3.7-16c shows a plan of the Nuclear Island and the system of coordinates. Table 3.7-7 gives the coordinates of the mass points (nodes).

The mathematical model consists of four main parts:
1) The RSB; 2) The Confinement; 3) The RSB; 4) The EEB, CB and SGB.

The four parts of the model are supported by the foundation mat which is assumed to be rigid. This assumption is justified because the mat acts as a diaphragm and is stiffened by the vertical walls of the buildings.

The buildings above the mat are interconnected by flexible ties which include cross coupling between interconnected nodes.

The stiffness of the flexible ties that interconnect the nodal points of the four main parts of the structure will be calculated by finite element analysis with the computer program MRI/STARDYNE.

Three dynamic degrees of freedom per node will be allowed on the mathematicla models for the two horizontal components of the earthquake: translational and rotational along the direction of the earthwuake and rotational (torsional) about a vertical axis.

Each of the models for the horizontal components (North-South and East-West) has three foundation springs: translational and rocking along the direction of the motion and torsional about the vertical axis through the mat centroid and the corresponding dashpots (dampers).

The model for the vertical direction will allow one dynamiv degree of freedom per node and has only one foundation spring and dashpot (vertical); in this model the dome of the steel containment has been idealized using equivalent springs which account for the "breathing" (shell type of vibration) of the dome during a vertical vibration.

To account for "breathing," a stiffness matrix of the dome with cross-coupling terms was derived from an axiasymmetrical shell model of the dome, using the KALNINS computer program the equivalent springs represent the terms on the stiffness matrix.

The Reactor Vessel has been coupled to the models.

Two computer programs: HETHA for horizontal motions and VETHA for vertical motions, will be used to calculate the structural responses. Using a formulation for the equations of motion similar to that proposed by TSAI (Reference 5), the programs solve the coupled equations of motion by direct integration to obtain acceleration time-histories at each one of the mass-points for the dynamic degress of freedom assumed in the model.

With the acceleration time-histories, floor response spectra will be calculated.

Spectral values will be computed for the set of frequencies given in Table 3.7-1. In addition, spectral values will be calculated at the natural frequencies of the structures.

Response spectra will be computed for critical equipment dampings of 2%, 3%, 4%, and 7% for the SSE and 1%, 2%, and 4% for the OBE.

In addition, to account for the effect of possible variations of the structural material properties and damping, and for the relative accuracy of the dynamic calculations, the computed floor response spectra will be smoothed and peaks will be widened within a ±10% band.

The responses will be calculated for the upper and lower bounds of the range of foundation material properties; the design response spectra will be the envelope of the corresponding widened spectra for the upper and lower bounds.

The responses will be calculated for nodal points which correspond to centers of mass. To find the response at points away from the nodal points, additional linear accelerations caused by rotational and torsional accelerations will be added.

The effects of the three earthquake directions will be combined by the rule of the square root of the sum of the squares.

The time-history of the forces acting on the structures (shears and moments) were calculated using the computer program STARDYNE using mathematic!a models similar to those of HETHA and VETHA. The soil spacings and dampers were eliminated and teh acceleration time-histories calculated by HEATHA and VETHA at the foundation mat were used as imput. The spectra at different locations were calculated to check against those calculated by HEATHA and VETHA. Peak values of the forces are identified and envelopes of maximum forces were constructed for the design of the structures. The envelopes will be based on the results of the anlayses for the upper and lower bound of the range of the foundation material properties.

3.7.2.1.1.2 Emergency Cooling Tower

Another Category I structure independent of the Nuclear Island and founded on rock is the Emergency Cooling Tower (ECT). A description of this structure is given in Section 3.8.4.1.5 of the PSAR.

The conditions of the foundation material under the ECT are similar to those under the Nuclear Island, i.e., inclined layers of silstone and limestone, with siltstone directly under the foundation mat.

A lumped mass analysis of this structure will be performed. Since the analysis of the Nuclear Island showed a good correlation between the spring constants for the rock/structure interaction calculated by the static finite element method and elastic half-space theory (with the properties of siltstone), the springs and dampers for ECT will be calculated by elastic half-space theory in a similar manner. Analysis will be performed for the Upper Bound and Lower Bound of the rock properties and also for a "fixed" base and the results for the three cases will be enveloped. The embedment springs will be connected at appropriate nodes of the structural model. The three input motions, (North-South, East-West and vertical) will be applied simultaneously on the three-dimensional, lumped mass model with six degrees of freedom at each node. The mass points will be located at the center of mass of the corresponding sections (in general, at floor locations). The beam elements, between mass points, will represent the axial, bending, shear and torsional stiffness of the structure and will be located at the corresponding shear centers. Time-history modal super-position analysis will be used with composite modal dampings calculated by the equation:

$$\bar{\beta}_j = \frac{\{\phi\}^T (\bar{K}) \{\phi\}}{\{\phi\}^T (K) \{\phi\}}$$

where:

(K) = assembled stiffness matrix for structure

$\bar{\beta}_j$ = equivalent modal damping ratio of the jth mode

$(\bar{K})$ = modified stiffness matrix constructed from element matrices formed by the product of the damping ratio for the element and its stiffness matrix.

ϕ = jth normalized modal vector

The fluid in the ECT will be treated in accordance with Housner's theory (Ref. 12).

The analysis will result in acceleration time histories at the different nodal points, forces in the structural members and floor acceleration response spectra. (One for each of the six degress of freedom). Analysis will be done for both OBE and SSE conditions.

3.7.2.1.1.3 Diesel Generator Buildings

The Diesel Generator Building (DGB) is the only major Category I structure founded on soil. This structure is described in Section 3.8.4.1.4 of the PSAR. The soil-structure interaction will be treated as described in Section 3.7.1.6 using the computer program FLUSH. A three-dimensional lumped mass model of the structure will be generated using the computer program STARDYNE. Condensed mass and stiffness matrices consistent with the two-dimensional formulation of FLUSH, will be calculated from this model and used as superelements in FLUSH. Two separate mathematical models for the North-South and East-West directions will be used. The FLUSH analysis will provide seismic responses at different points in the structure. A range of soil properties will be used in the analysis and these responses will be enveloped. The enveloped response spectra at the foundation level will also envelope the design response spectra for the site.

Since FLUSH is two-dimensional, to do a more detailed analysis of the structure, the acceleration time-histories calculated by FLUSH at the DGB foundation will be used for a three-dimensional analysis with six degrees of freedom per mode using the STARDYNE model. The acceleration in the STARDYNE model. The responses from this calculation will be verified against those produced by FLUSH. Design response spectra and forces on the structure will be produced from this STARDYNE analysis. Analysis will be done for both OBE and SSE conditions.

3.7.2.1.1.4 Miscellaneous Category I Structures

Other Category I structures supported on soil are as follows:

Diesel Fuel Storage Tanks (described in Section 3.8.4.1.6), Electrical Manholes (described in Section 3.8.4.1.7)

Emergency Plant Service Water (EPSW) Pipes (described in Section 9.9.4) and Class 1E Duct Banks (described in Section 3.10).

The seismic design of these buried structures will be in accordance with the method described in Section 3.7.3.12.

3.7.2.1.1.5 Category III Structures

Two Category III structures, the Turbine Generator (TGB) and Radwaste (RWB) buildings, because of their proximity to the Category I structures, were designed to withstand the effects of the SSE. The TGB and RWB are supported on soil, and the soil-structure interaction approach is described in Section 3.7.1.6.

Three dimensional models of the buildings were constructed using the computer program STARDYNE. Condensed mass and stiffness matrices consistent with the two-dimensional formulation of FLUSH were calculated from the models and used as supplements in FLUSH. FLUSH analyses were performed with models for the North-South and East-West directions, for upper and lower bound of soil properties. Response spectra at the foundations were produced and enveloped.

With enveloped response spectra applied at the foundation and using the three-dimensional STARDYNE models, the forces on the members of the structure were calculated by the response spectrum modal analysis method.

The response spectra used in the three-dimensional analysis were the envelopes of the spectra at the foundation level from 1) FLUSH analysis of the buildings 2) Seismic analysis of the Nuclear Island. The latter was done to account for the effects of the motions of the massive Nuclear Island on the adjacent buildings.

The spectral envelopes were above the Design Response Spectra for the site. Accelerations at the buildings calculated by the spectrum analysis were compared with those of the FLUSH analysis as a verification of the analysis.

TABLE 3.7-2A

DAMPING RATIOS FOR FOUNDATION MATERIALS

(Internal Damping)

CLASS I FILL

Shear Strain γ-in./in.	Damping %
5×10^{-6}	5.6
1×10^{-5}	5.6
2×10^{-5}	5.6
5×10^{-5}	5.6
1×10^{-4}	5.6
2×10^{-4}	5.7
5×10^{-4}	6.7
1×10^{-3}	8.0
4×10^{-3}	14.8
1×10^{-2}	17.2

ROCK

SSE: 2% Damping

OBE: 1% Damping

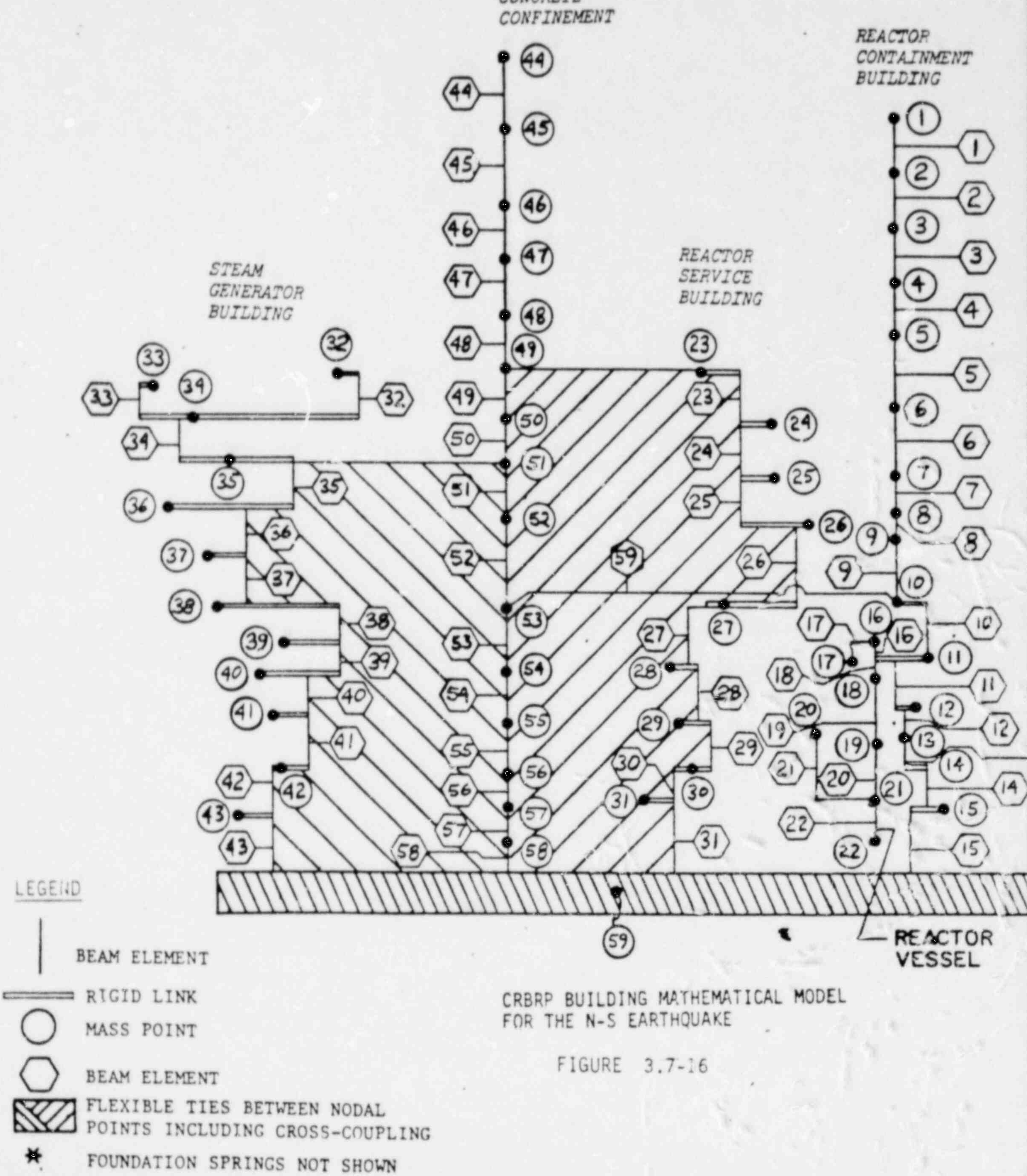
TABLE 3.7-7

LOCATION OF NODE POINTS

Node No. for Horizontal Directions	Corresponding Node No. for Vertical Direction	Distance from Center of Containment Building (Ft.)		Elevation (Ft.)	Applicable Nuclear Island Building
		x (E-W)	y (N-S)		
1	2	0.0	0.0	965.72	Reactor Containment
2	3	0.0	0.0	948.57	Reactor Containment
3	4	0.0	0.0	931.43	Reactor Containment
4	5	0.0	0.0	915.21	Reactor Containment
5	6	0.0	0.0	899.00	Reactor Containment
6	7	0.0	0.0	876.00	Reactor Containment
7	8	0.0	0.0	856.00	Reactor Containment
8	9	0.0	0.0	842.00	Reactor Containment
9	10	0.0	0.0	836.00	Reactor Containment
10	11	0.43	2.78	816.00	Reactor Containment
11	12	1.57	- 2.22	800.00	Reactor Containment
12	13	1.52	- 2.06	783.75	Reactor Containment
13	14	0.67	6.8	774.00	Reactor Containment
14	15	2.38	- 3.11	766.00	Reactor Containment
15	16	4.62	- 1.48	752.66	Reactor Containment
23	24	26.37	-161.64	884.00	Reactor Service
24	25	40.28	-157.55	869.00	Reactor Service
25	26	39.87	-157.86	857.00	Reactor Service
26	27	42.35	-170.03	840.00	Reactor Service
27	28	15.39	-160.41	816.00	Reactor Service
28	29	10.69	-160.57	797.00	Reactor Service
29	30	12.40	-159.35	779.00	Reactor Service

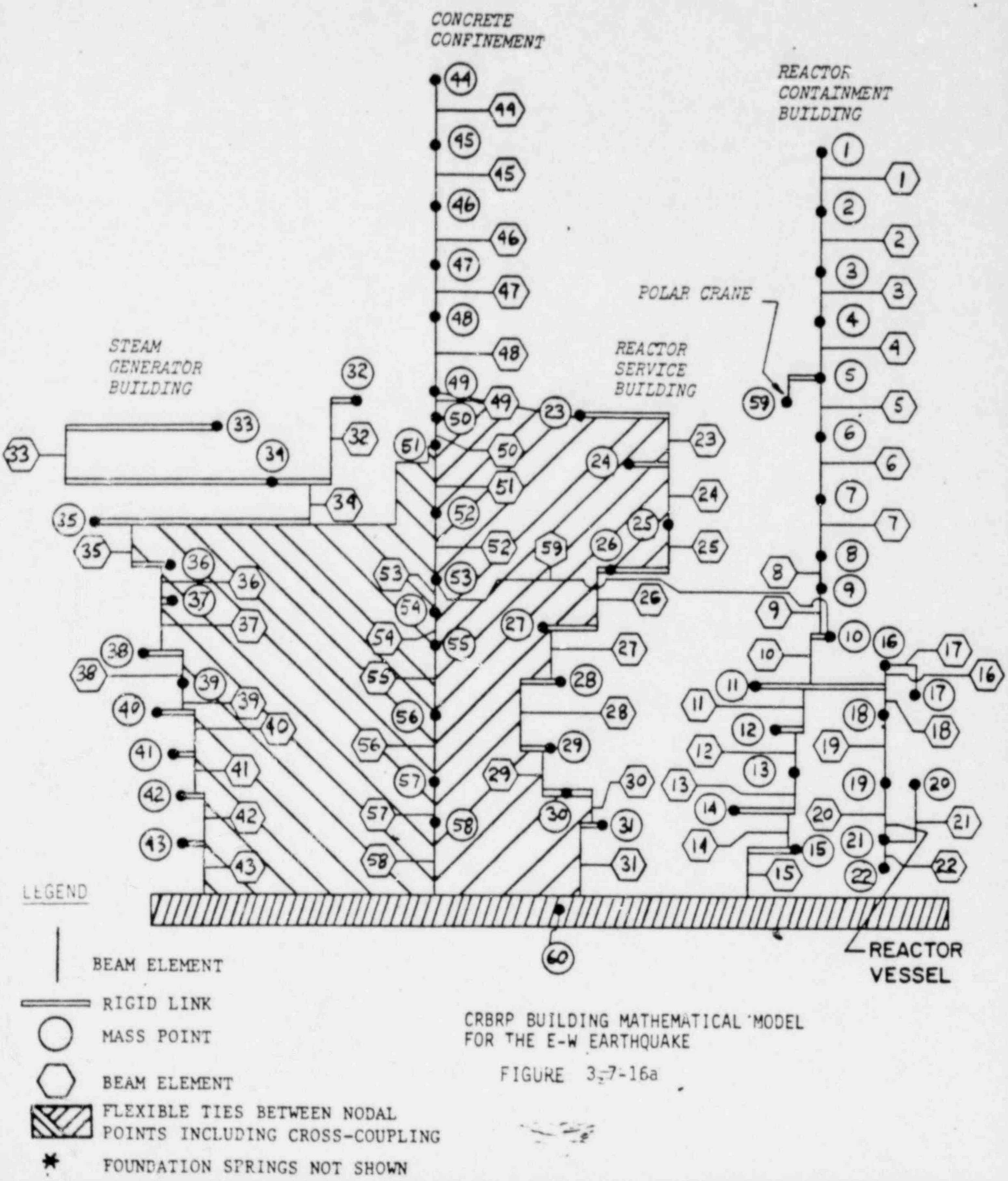
TABLE 3.7-7 (Continued)
LOCATION OF NODE POINTS (Cont.)

Node No. for Horizontal Directions	Corresponding Node No. for Vertical Direction	Distance from Center of Containment Building (Ft.)		Elevation (Ft.)	Applicable Nuclear Island Building
		x (E-W)	y (N-S)		
30	31	18.13	-156.74	765.00	Reactor Service
31	32	4.23	-150.01	755.00	Reactor Service
32	33	20.72	209.78	886.00	Steam Generator
33	34	-162.79	132.76	883.00	Steam Generator
34	35,60	- 26.68	155.13	873.00	Steam Generator
35	36	- 34.93	112.82	857.00	Steam Generator
36	37	-103.75	129.68	846.00	Steam Generator
37	38,61	- 47.77	127.34	837.00	Steam Generator
38	39	- 53.74	119.41	816.00	Steam Generator
39	40,62	- 13.01	148.39	806.00	Steam Generator
40	41	- 38.68	117.15	794.00	Steam Generator
41	42,63	- 21.94	131.55	787.00	Steam Generator
42	43,64	- 21.58	127.52	765.00	Steam Generator
43	44	- 37.29	139.32	746.00	Steam Generator
59 (N-S)	67	- 18.11	30.78	733.00	Common Base Mat
60 (E-W)	67	- 18.11	30.78	733.00	Common Base Mat



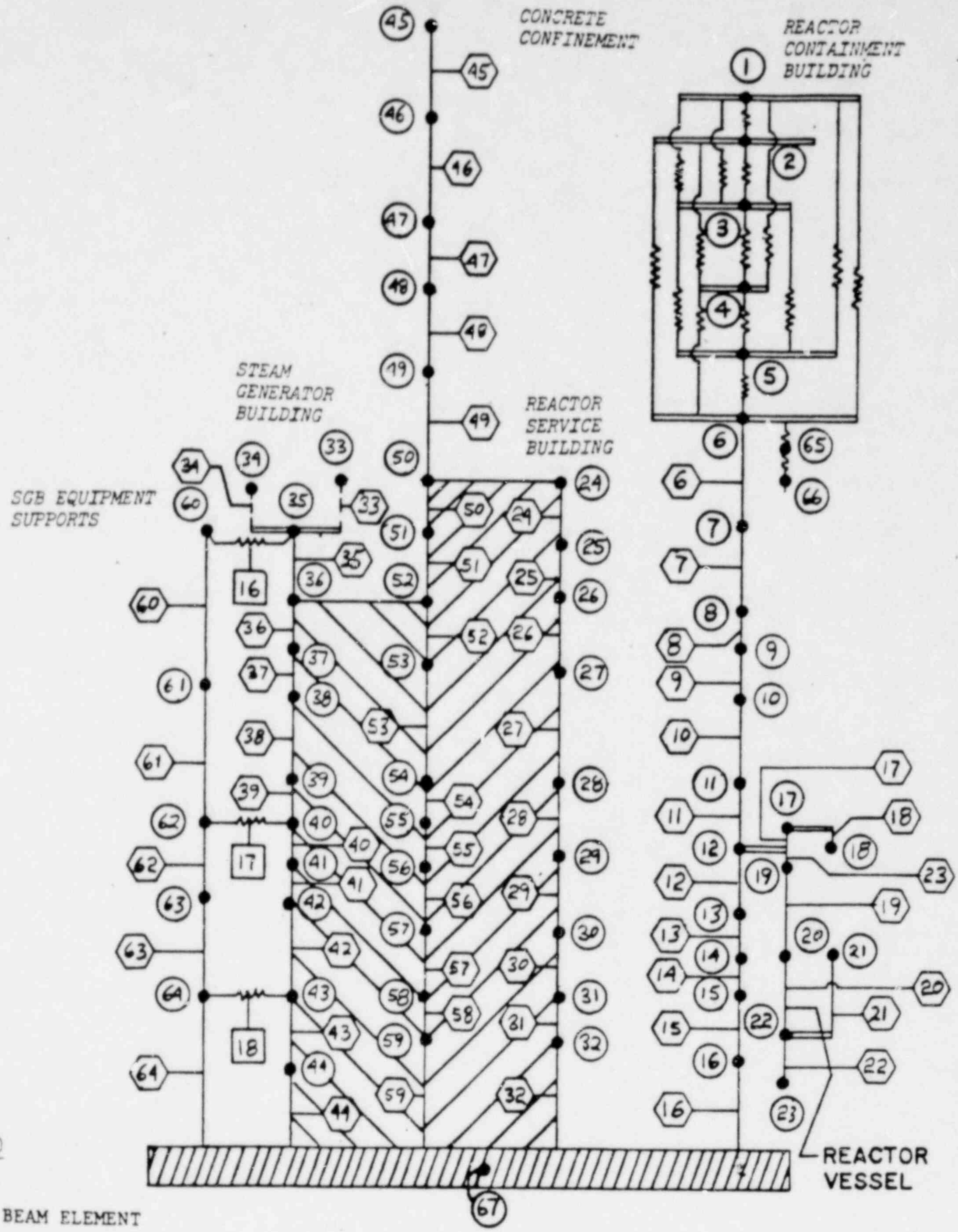
CRBRP BUILDING MATHEMATICAL MODEL FOR THE N-S EARTHQUAKE

FIGURE 3.7-16



CRBRP BUILDING MATHEMATICAL MODEL
FOR THE E-W EARTHQUAKE

FIGURE 3.7-16a



LEGEND

BEAM ELEMENT

RIGID LINK

MASS POINT

BEAM ELEMENT

FLEXIBLE TIES BETWEEN NODAL POINTS INCLUDING CROSS-COUPLING

* FOUNDATION SPRINGS NOT SHOWN

CRBRP BUILDING MATHEMATICAL MODEL FOR THE VERTICAL EARTHQUAKE

FIGURE 3.7-16b

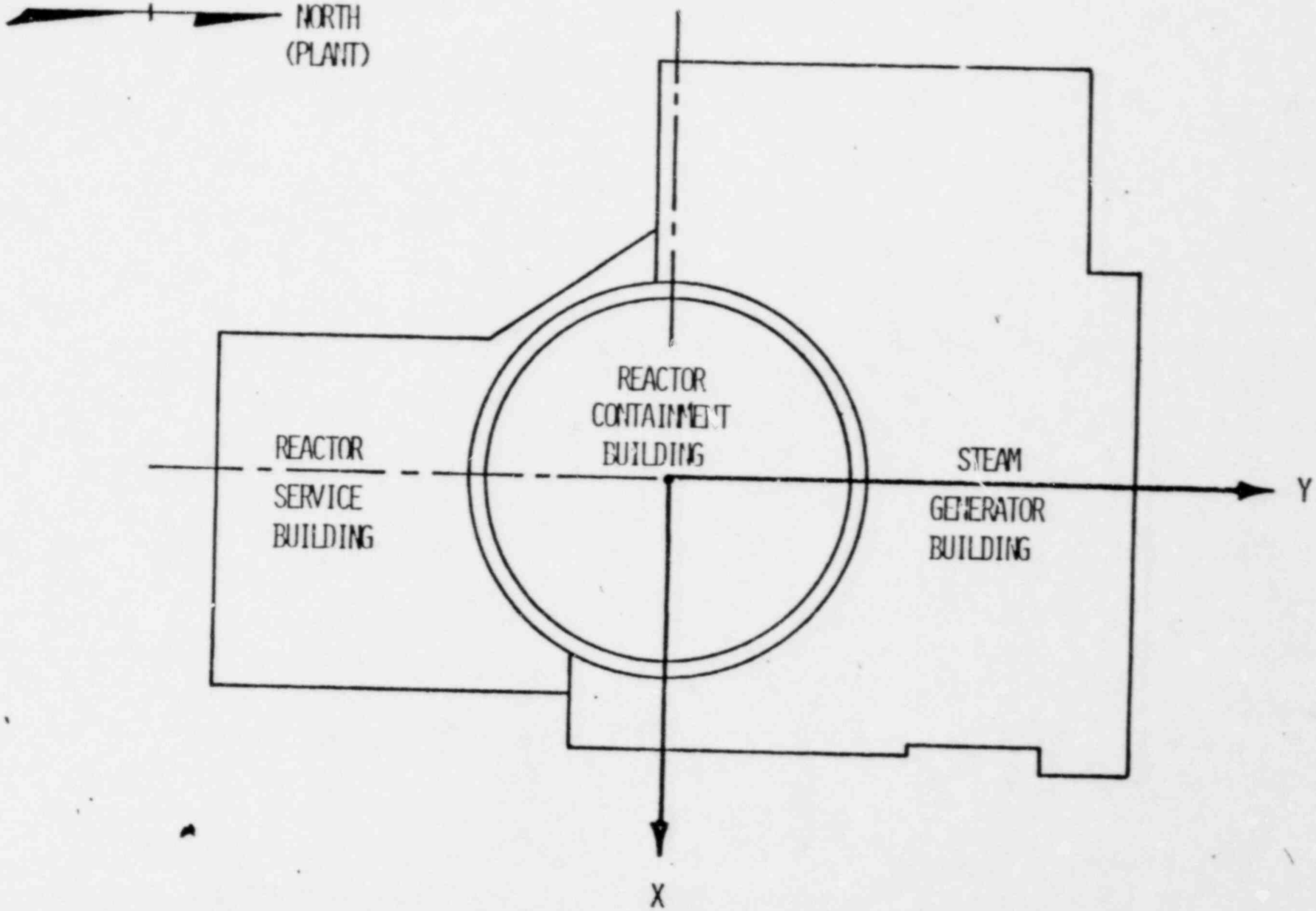


FIGURE 3.7-16c CRBRP BUILDING NODES COORDINATE SYSTEM - PLAN

Question CS 220.20

In Section 6.1 on page 3.7-A.8, your definition of significant dynamic modes is not consistent with that in SRP Section 3.7.2 and should be revised. Further, the sentence before the last sentence in the third paragraph stated that different response spectra will be applied for the particular support location. An explanation should be given to this statement.

Response

The definition of significant dynamic modes has been revised, as shown marked on PSAR Page 3.7-A.8.

The different response spectra are the spectra derived and applicable to the various supports of structural systems (such as piping) which are not at the same location. The spectrum at the first support may not be the same as the spectrum at an intermediate support or at the last support In this case, all the different support response spectra are superimposed to yield an envelope response spectrum for input to the response spectrum analysis.

6.1 Dynamic Analyses

Seismic Category I and II structures, systems, and components shall be analyzed by a detailed dynamic analysis using either time history methods or the response spectrum method. Other methods of dynamic analysis which provide an acceptable solution may be used but the justifications and procedures shall be submitted to W-LRM for review and approval. A simplified analysis such as that based on an equivalent static load method may be used if it can be demonstrated that the simplified method provides adequate conservatism.

Analytical procedures of the detailed dynamic analysis and of a simplified analysis are given in Attachment A.

The analysis will include the effects of the dynamic coupling between major components of the system and the soil-structure interaction effects. A sufficient number of modes of the mathematical model which represents the structural system shall be included in the analysis to assure participation of all significant modes. The criterion for sufficiency is that the inclusion of additional modes does not result in more than a 10% increase in responses. Where the response spectrum method is used, the individual modal responses shall be combined by the square root of the sum of the squares, except for closely spaced modes (frequencies 10% or less apart) where the modal responses shall be combined by the absolute sum (see Attachment A). Supports of structural systems (such as piping) may be subjected to different accelerations; i.e., different response spectra for the particular support location. In this case, the different response spectra should be super imposed to yield an envelope response spectrum to be used in the response spectrum analysis.

The system will also be analyzed to determine the effects of the relative displacements at their supports.

The relative displacements should be imposed on the mathematical model as separate static displacements unless other conservative methods of analysis are employed or a time history analysis is performed. In the latter case, the appropriate time histories are used at each anchor point location so that the differential effects are inherently included in the analysis. The relative displacements are imposed in the most unfavorable manner to satisfy input motions which may be out of phase from each other. The effects resulting from the separate analysis for relative displacements should be combined absolutely with those resulting from the seismic-induced inertial loadings.

Time history analyses of the supporting structure are required to determine the response motin at the location on the structure of the supported component. This motion is used as input to the analysis of the supported component and may be in the form of motion time history or floor response spectra.

Question CS220.21

In Sections 8.1.1.1 and 8.1.1.2 on page 3.7-A, the listed load combinations contain the term "operating." Define specifically what are the loads included in this term.

Response:

The loading components included in the term "Operating" are given in its definition in Section 7.1.1, Page 3.7-A.11 of the PSAR.

Question CS220.22(a)

There are a number of misprints, unclear statements and typographical errors which need your correction and/or clarification.

o Section 3.7.1.6 page 3.7-3C misprints in both items 2) and 3).

Response:

The word "p[rocedure" should be "procedure".

The word "axiaymmetrical" should be "axisymmetrical".

PSAR Page 3.7-3C has been revised.

Vertical spring constant: $k'_v = \frac{G B_z (LW)^{1/2}}{W(1-v)}$ (3)

Horizontal spring constant: $k'_v = \frac{2(1+v) G B_x (LW)^{1/2}}{W}$ (4)

Rocking spring constant: $k' = \frac{G B_\phi LW^2}{W(T-V)}$ (5)

Where G = shear modulus of soil

v = Poisson's ratio of soil

The parameters B_z, B_x, $B\phi$ are a function of the ratio $\frac{W}{L}$ and are given in Figure 10-16 of Reference 11.

c) The ratios between the values calculated in (b) and those of (a) were calculated (k'/k).

Table 3.7-6 shows the results of (a) and (b) and (c) for the horizontal translational spring and Figure 3.7-22 shows a plot of $k'_{H/kH}$ as a function of the aspect ratio of the foundation, (W/L).

It can be seen that as the ratio W/L increases, the ratio k'_{HkH} approaches unity. Since a large W/L ratio approaches the plane strain condition of the finite element calculation and for large W/L ratios the two methods give similar result, a good correlation has been proven.

From the curve of Figure 3.7-22 for the aspect of the foudnation $\frac{W}{L}$ = 0.67 the ratio k'_{HkH} = 1.33. This value gives the correction factor for three dimensional effects to be applied to the previous calculated stiffness k_H. Similar calculations were performed for the rotational and the vertical springs.

2) Model "B" (East-West direction)

This is a plane strain model similar to "A" but based on a section through an East-West plane (Figure 2.7-23) and was used to calculate the horizontal translational and rocking springs for the East-West direction and the vertical translational spring. The procedure was similar to that described in (1) above.

Question 220.22(b)

The equation for the dampi~~ ~~ues β_j in Section 3.7.2.1.1 is in error. (Probably misprint)

Response:

The correct equation is:

$$\bar{\beta}_j = \frac{\{\phi\}^T (\bar{K}) \{\phi\}}{\{\phi\}^T (K) \{\phi\}}$$

There is a misprint in the PSAR. $(\bar{K})$ in denominator should have been (K).

PSAR page 3.7-5 has been revised.

3.7.2.1 Seismic Analysis Method

3.7.2.1.1 Category I Structures

A complete analysis will be performed on each of the Seismic Category I structures to predict its behavior during an earthquake. The SSE and OBE will be considered; each of two orthogonal horizontal directions and the vertical direction will be treated separately and the results combined. The input motions are described in Section 3.7.1.

3.7.2.1.1.1 Nuclear Island

A lumped-mass formulation, with direct integration of the coupled equations of motion will be used.

The Buildings of the Nuclear Island: Reactor Containment (RCB), Confinement, Reactor Service (RSB), Steam Generator (SGB), Electrical Equipment (EEB) and Control Building (CB) have a common foundation mat with the bottom at Elevation 715'-0"; finished grade is at El. 815'-0".

Except for the RCB interior structure, the Category I structures are tied together up to the roof level. The RCB interior structure is tied to the adjacent structures at the mat level and at the operating floor level (Elev. 816'-0").

The structures and foundation materials will be represented in terms of lumped-masses amd massless springs and dashpots.

The inertial properties are characterized by the masses and mass moments of inertia which will be lumped at points selected to assure proper representation of the dynamic behavior of the structures.

The mass points will be, in general, at the elevation of the floors and will be located at the center of mass of the contributing elements.

The stiffness properties are characterized by the areas, shear areas and moments of inertia (for bonding and torsion) of the members and by the moduli of elasticity and Poisson's ratios. The flexible members (beam elements) between floor levels will be assumed located at the shear center of their sections and their ends are defined by the elevations of the mass points. The ends of beam elements are connected to the mass points by horizontal rigid elements.

The soil (rock) structure interaction will be represented by equivalent springs and dashpots. The stiffnesses of the foundation springs will be calculated as described in Section 3.7.1.6.

The damping values to be used for the structures in terms of percent of critical damping are given in Table 3.7-2; the combined damping ratios for the structures (steel containment and concrete buildings) will be calculated based on the equation:

$$\bar{\beta}_j = \frac{\{\phi\}^T(\bar{K})\{\phi\}}{\{\phi\}^T(K)\{\phi\}}$$

Question CS220.22(c)

In Section 3.7.2.6.1 on page 3.7-9, it is stated that each node has three degrees of freedom in the horizontal directions. This is a wrong statement, since each node should have six degrees of freedom. A correction of this statement should be made.

Response:

The first paragraph of Section 3.7.2.6.1 has been revised to clarify the statement.

3.7.2.2 Natural Frequencies and Response Loads

This section will be discussed in the FSAR.

3.7.2.3 Procedures Used to Lump Masses

This will be more completely addressed in the FSAR, consistent with Reg. Guide 1.70. However, the following preliminary information is provided at this time.

In a lumped mass seismic system analysis, the uniform masses of the elements are concentrated at a series of mass-points, or nodes. The nodes are selected at change of sections, at locations of equipment support where the equipment's mass is lumped, at locations where the dynamic responses is desired, and at intermediate locations to limit the length of the elements so that the mathematical model will adequately represent the actual system. Consideration is also given to a lower limit for the element's length, so that a convergent solution will be obtained without requiring a prohibitively small value of the integration time interval in a time history analysis. Along with this procedure, the total number of masses chosen will be such that additional masses, or degrees of freedom, do not result in more than a 10% increase in responses. Alternately, the number of masses, or degrees of freedom, will be taken equal to twice the number of modes with frequencies less than 33 H_z.

3.7.2.4 Rocking and Translational Response Summary

This section will be discussed in the FSAR.

3.7.2.5 Methods Used to Couple Soil With Seismic-System Structures

Seismic Category I structures supported on soil as identified in Section 3.7.1.5 will be analyzed in accordance with the procedures described in Section 3.7.1.6.

3.7.2.6 Development of Floor Response Spectra

Section 3.7.2.1.1 describes the method for calculating floor response spectra which shall be based on direct integration of the coupled equations of notion.

3.7.2.6.1 Design Response Spectra for Major Components

The building analysis for the Nuclear Island gives floor response spectra at nodes located at the center of mass of the applicable floors identified in the mathematical models on Figures 3.7-16, 3.7-16A and 3.7-16B. Since each node has three degrees of freedom (translation, torsion, rotation) in each horizontal direction (East-West, North-South) and one degree of freedom in the vertical direction (translation) seven response spectra are developed at each node for each of the two earthquakes (OBE and SSE). These are:

Question CS 220.22 (d.)

There are a number of misprints, unclear statements and typographical errors which need your correction and/or clarification.

d) Section 3.7.2.7 on page 3.7-9b, the last three sentences need some correction or clarification in order to be understandable.

Response

d) A missing word has been properly inserted in Section 3.7.2.7, Page 3.7-9b of the PSAR. This will clarify the meaning of the last three sentences.

Two methods of combining the seven spectra are given in Appendix 3.7-A. The general formulation requires all seven resulting spectra to be applied individually, and the similar effects, such as deflections and stresses, which result from the individual application of the seven spectra are combined by the square root of the sum of the squares. This combinatin is performed as a last step after the modal combination for each of the three earthquake directions. The other method utilizes a simplified procedure which combines the seven response spectra and reduces them to three response spectra (two horizontal and one vertical). However, when this simplified procedure is used, the similar effects obtained for each of the three spectral input directions are combined absolutely. The full details of these two methods of spectra combination and application are given in Appendix 3.7-A.

3.7.2.7 Differential Seismic Movement of Interconnect Components

The effects of differential seismic movements of support points and of interconnected components between floor are considered in the analysis. The relative displacements are imposed on the seismic mathematical model as separate static displacements unless other conservative methods of analysis are employed or a time history analysis is performed. In the latter case, the appropriate time histories are used at each anchor point location so that the differential effects are inherently included in the analysis. The relative displacements are imposed in the most unfavorable manner to satisfy input motions which may be out of phase from each other. For effects resulting from the separate analysis for relative displacements are combined absolutely with those resulting from the seismic-induced inertial loadings. The loading combinations and stress criteria shall be those given in Tables 3.9-2 and 3.9-3, respectively

Question CS220.22(e)

Section 3.7.2.14, misprint in the fourth paragraph.

Response:

The word "requesented" should be "represented". PSAR page 3.7-11 has been revised.

The most severe load combination will be considered.

The maximum soil (rock) pressure under foundation mat will be calculated under the assumption that the soil pressures vary linearly and that the soil cannot develop tensile stresses. If tension is obtained in the original calculation, the soil pressure distribution will be adjusted such that an all compressive linear distribution which balances the applied forces and moments is obtained.

3.7.2.14 Analysis Procedure for Damping

The damping ratios of Table 3.7-2 will be used for structures, systems and components; the damping coefficients for the dashpots associated with the soil-structure interaction will be calculated as described in Section 3.7.2.1.1.

For structures with different elements, the composite damping will be calculated based on the modes of vibration for a fixed base condition with modal damping ratios evaluation by a weighted average based on the relative strain energy of the different elements as described in Section 3.7.2.1.1.

The damping ratios for the fixed base structure and damping coefficients of the foundation dampers are used to formulate the damping matrix of the equations of motion.

For seismic system analysis utilizing response spectrum techniques the applicable response spectrum for the appropriate damping value of Table 3.7-2 is used in the analysis. For the time history analyses, the damping matrix is represented as proportional damping with a linear combination of the mass and stiffness matrices. The mass and stiffness matrices coefficients are calculated as a function of frequency by establishing a predominant frequency range of the system. In a coupled system with different structural elements, either the lowest damping value is used for all nodes or individual modes are identified and associated with a particular element and damping value for the element.

3.7.3 Seismic Subsystem Analysis

3.7.3.1 Determination of Number of Earthquake Cycles

During the design life of the plant (30 years) the occurrence of approximately one to five operating basis earthquakes and one SSE, each having 12 to 15 seconds average duration of strong motion excitation, is postulated.

For fatigue analysis of mechanical systems and components, a total of 50 earthquake cycles will be used for the OBE. This number is the product of 10 equivalent maximum peak response cycles for one seismic event and a conservative number of 5 OBE occurrences. Seismic response time histories contain a very small number of maximum peak cycles and several other cycles of smaller amplitude. These may be derived from the seismic response of the

Question CS220.22(f)

Section 3.7.3.1, in the last sentence, is the word "assured" used for "assumed"?

Response:

The word "assured" should be "assumed". PSAR page 3.7-11a has been revised.

particular structural system, which is damping dependent. The smaller cycles may be converted into a smaller number of equivalent maximum peak cycles with the aid of fatigue curves. For the postulated earthquake characteristics given above, a total of 10 equivalent maximum peak response cycles are considered sufficient for each OBE and SSE occurrence. (See Appendix 3.7-A for conditions under which the OBE and SSE are assumed to occur).

3.7.3.2 Basis for Selection for Forcing Frequencies

Forcing frequencies are not selected. The frequency content of the seismic motion is reflected in the design ground-time histories and in response-time histories as modified by the structure and component characteristics.

Question CS220.22(g)

Section 3.7.4.2, next to last paragraph under 1, on page 3.7-17, the word "Time-History".

Response:

The word "Time-hisotry" should be "time-history". PSAR page 3.7-17 has been revised.

c) On the structure of the Reactor Containment Building, at a higher elevation and as near as practicable vertically above the sensor in b) above.

d) At the most pertinent location on one of the independent seismic Category I structures where the response is different from that of the Reactor Containment Structure.

The strong motion triaxial sensors will have a dynamic range of 100:1 from zero to peak, a flat frequency range from 0.1 Hz to 30.0 Hz and velocity proportional damping adjustable between 55% and 70% critical. They will not have spurious resonances within the frequency range of interest and the sensitivity to acceleration of components orthogonal to the sensitive axix shall not exceed 0.03 g/g.

The triaxial seismic trigger will be placed near the "free field" acceleration sensor; it will simultaneously activate all recording instruments of the Time-History Accelerograph when a threshold ground acceleration is reached. The triaxial seismic trigger will be adjustable from a minimum of 0.005 g to 0.02 g; with a flat frequency range from 1 Hz to 10 Hz.

The multi-channel recorder and control unit will be located in the Control Room; it will be battery operated and will record the time-correlated acceleration from the four (4) triaxial acceleration sensors on magnetic tape. It will have sufficient number of active channels to record the data of the acceleration sensors plus at least one separate timing reference trace. The recording speed will be sufficient to resolve a frequency of 30.0 Hz; the timing marks will consist of at least two pulses or marks per second with a +0.2% internal timing accuracy; the dynamic range will be 100:1 for combined record and playback.

Upon activation by the seismic trigger the time-history accelerographs will achieve full operational status within 0.1 seconds; the system will continue to operate for at least 5 seconds after the strong motion acceleration falls below the trigger threshold level. The recording time will not be less than 25 seconds. The design will provide for annunciation on the control room panel, upon activation of time-history accelerographs.

The tape playback unit will be designed to transcribe the signals recorded on magnetic tape by the central recorder and control unit into graphic form for a quick look analysis of the earthquake.

2. <u>Triaxial Seismic Switches</u>

Question CS220.22(h)

In Table 3.7-5, you used a poisson ratio of 0.3 for both limestone and siltstone. Indicate how this value is obtained and why it is the same for both.

Response:

The derivation of Poisson's Ratio for siltstone and limestone has been based on detailed analysis of a significant number of geophysical measurements as obtained from cross-hole, up-hole and continuous velocity methods. Similar ranges were obtained by the different methods for each rock type, resulting in the selection of 0.3 as the design value. Details are provided in Section 2.5.4.2.2 of the PSAR.

Question CS 220.22 (I)

There are a number of misprints, unclear statements and typographical errors which need your correction and/or clarification

I) The Figures 3.7.17D and 3.7.18 have the same title, but it is not clear how they are related. Clarify.

Response

I) The title of Figure 3.7.17D has been changed to "Schematic of Reactor System Finite Element Model." In addition, an introductory sentence has been added to Section 3.7.3.15.2 to identify this figure.

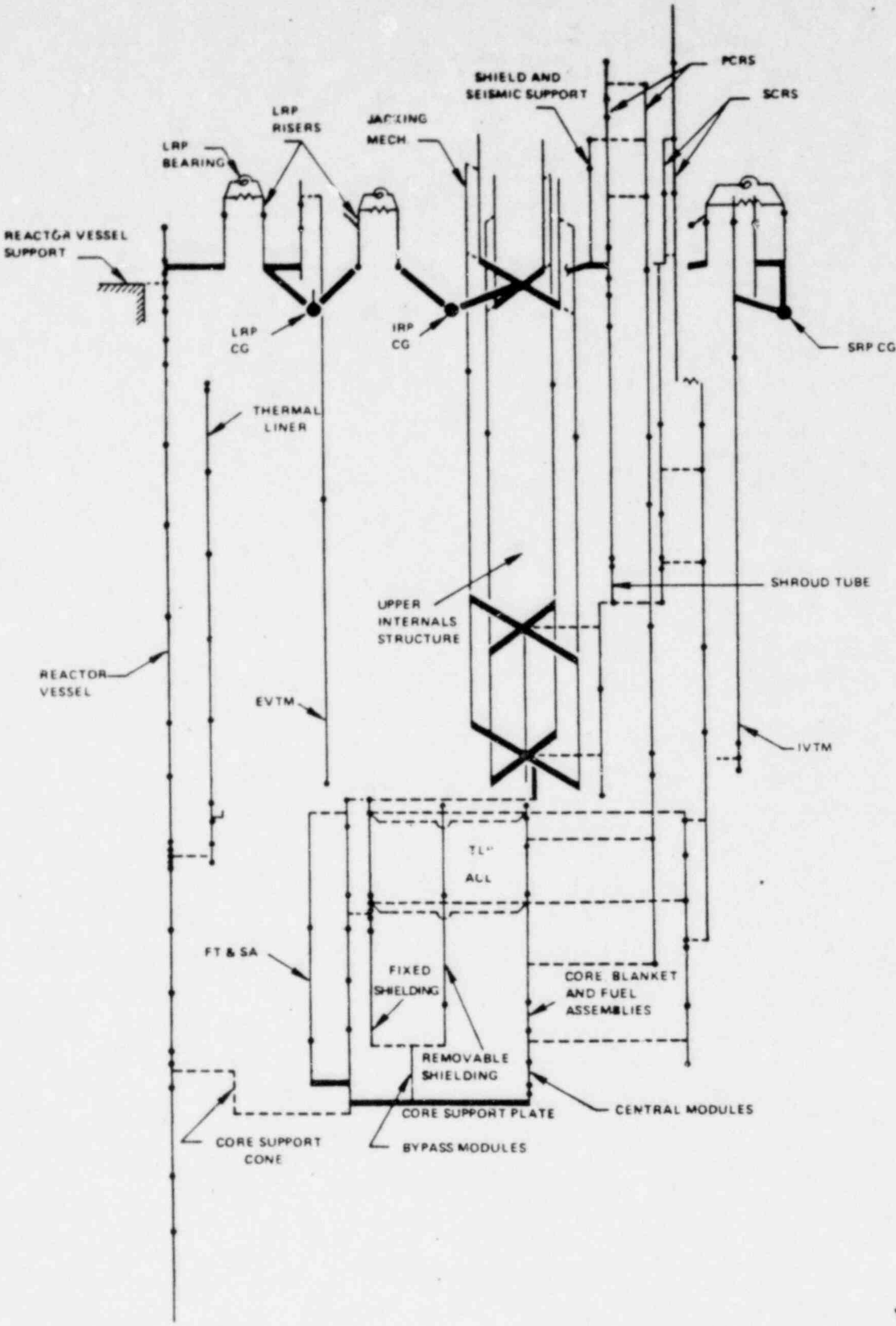


Figure 3.7-17D Schematic of Reactor System Finite Element Model

9859-5

Question CS220.22(J)

J) Appendix 3.7 in "Attachment A", the equation for a simplified analysis appears to have incorrect units. Also the transformation equations appear to be incorrect.

Response:

The corrections to the errors in the transformation operations sited above appear in the revised PSAR page 3.7-A-A1. The errors were primarily typographical in nature.

With regard to the simplified analysis equation, (Equation 19) it should be noted that the spectral accelerations are given in "g" units. The multiplication of the weight by "g" gives a force. Therefore, the equation is dimensionally correct. A revision will be made to clarify the units for "A"s.

ATTACHMENT A

ANALYTICAL PROCEDURES

A.1 Detailed Dynamic Analyses

Two of the methods used to perform a suitable, detailed dynamic analysis of a structural system are the time history analysis and the response spectrum analysis. The equations of motion are solved either by direct integration or modal superposition. Using modal superposition th normal modes of the system are obtained along with the natural frequencies, mode shapes and participation factors. For the time history analysis, the forcing function consists of the earthquake motion as a function of time. For the response spectrum analysis, the input excitation is provided in the form of response spectra which give the spectral response motions (displacement, velocity, acceleration) as a function of the natural frequencies for the appropriate damping values.

A.1.1 Equations of Motion

The equations of motion of a multi-degree-of-freedom discrete-mass damped system subjected to an arbitrary support motion $\ddot{Y}_s(t)$, can be written as:

$$M\ddot{X}(t) + C\dot{X}(t) + KX(t) = -MI\ddot{Y}_s(t) \qquad (1)$$

where:

M = Mass matrix

C = Damping matrix which can be expressed as a linear combination of the mass and stiffness matrices

K = Stiffness matrix

I = Unit vector in direction parallel to support motion

$\ddot{Y}_s(t)$ = Time dependent support acceleration

$X(t)$, $\dot{X}(t)$ and $\ddot{X}(t)$ = Time dependent displacement, velocity and acceleration vectors, respectively.

In the direct integration approach, the coupled equations of motion are solved directly. In the modal analysis, using the orthogonality relations and expressing the displacements, velocities and accelerations in normal coordinates; i.e., $X(t) = \phi A(t)$, $\dot{X}(t) = \phi\dot{A}(t)$, and $\ddot{X}(t) = \phi\ddot{A}(t)$, the above coupled equations of motion (Eq. 1) may then be rewritten as the following uncoupled, normal equations of motion:

$$M_r\ddot{A}_r(t) + 2M_r\omega_r\xi_r\dot{A}_r(t) + K_rA_r(t) = -M_r\Gamma_r\ddot{Y}_s(t)$$

F_s = Equivalent static force distributed proportional to the mass of the system.

W = Weight of the system including liquid contents.

A_s = Maximum peak acceleration of the response spectra which apply at the points of support of the structural system. (In "g" units)

A factor less than 1.5 in Equation 19 may be used if adequate justification is provided.

Question CS 220.24

The containment description should include basic shell thickness and state if the shell is stiffened. The containment vent and purge system should be mentioned in the list of components.

Response

Horizontal ring stiffeners are provided at elevations 856 and 839. In addition the crane girder also functions as a horizontal ring stiffener in the elevation 870 to 890.

The PSAR has been augmented with the requested information in 3.8.2.1 and also Figure 3.8-3.

The containment vent and purge systems are features provided to manage the hypothetical event beyond the design basis. These systems are discussed in CRBRP 3, Vol. 2, Figure 1-1. Only the penetrations for the vent and purge systems are a part of the containment boundary and these penetrations are designed to the same criteria as all other penetrations. The vent and purge system is not a part of the containment system and therefore should not be included in the list of containment system components.

3.8 DESIGN OF CATEGORY I STRUCTURES

3.8.1 Concrete Containment (Not Applicable)

3.8.2 Steel Containment System

3.8.2.1 Description of the Containment

The Containment Vessel is a low leakage, free-standing, all welded steel vessel anchored to the base mat with a steel lined concrete bottom in the form of a vertical right cylinder having an inside diameter of 186 feet and with side walls extending approximately 169 feet from the flat bottom liner at the base to the spring line of the ellipsoidal-spherical dome. The cylindrical shell is embedded in concrete up to the elevation of the operating floor. On the inside of the Containment Vessel, there is the continuous reinforced concrete wall comprising the peripheral boundary of the internal concrete structure. Butting against the outside face of the steel shell from elevation 733 feet up to the elevation of the underside of the operating floor, there is another reinforced concrete wall of sufficient thickness designed to prevent buckling of the steel shell. Neither of the two concrete walls are considered part of the containment vessel. Alumina-silica insulation is attached to the inside surface of the Containment Vessel from elevation 816 feet to elevation 823 feet. The insulatin is 3 inches thick and has a viue of 0.0267 Btu/hr - ft-°F. Its purpose is to limit the shell temperature at elevation 816 feet during Design Basis Accidents to less than 130°F.

The vessel includes: its shell, one horizontal gland girder, two horizontal stiffeners, a 1/4" bottom liner plate, one access airlock, one emergency egress airlock, vacuum relief system, one equipment hatch, penetrations, inspection ladders, miscellaneous appurtenances and attachments. The configuration of the Containment Building is shown in figures in Section 1.2 and the configuration of the shell is shown in Figure 3.8-3. The design lifetime of the containment vessel shall be 30 years.

3.8.2.2 Applicable Codes, Standards and Specifications

3.8.2.2.1 Codes

The Containment Vessel will be designed, material procured, fabricated, installed and tested in accordance with the requirements of the ASME B&PV Code, Section III, Division 1, 1974 Edition with Addenda through Winter 1974 and Code cases 1713, 1714, 1809, 1682 and 1785 and ASME-III, Division 2, 1975 Edition, Subsection cc, for the steel lined concrete containment bottom. The design shall also meet the requirements of the Class MC Section of RDT Standard E15-2T, "Requirements for Nuclear Components".

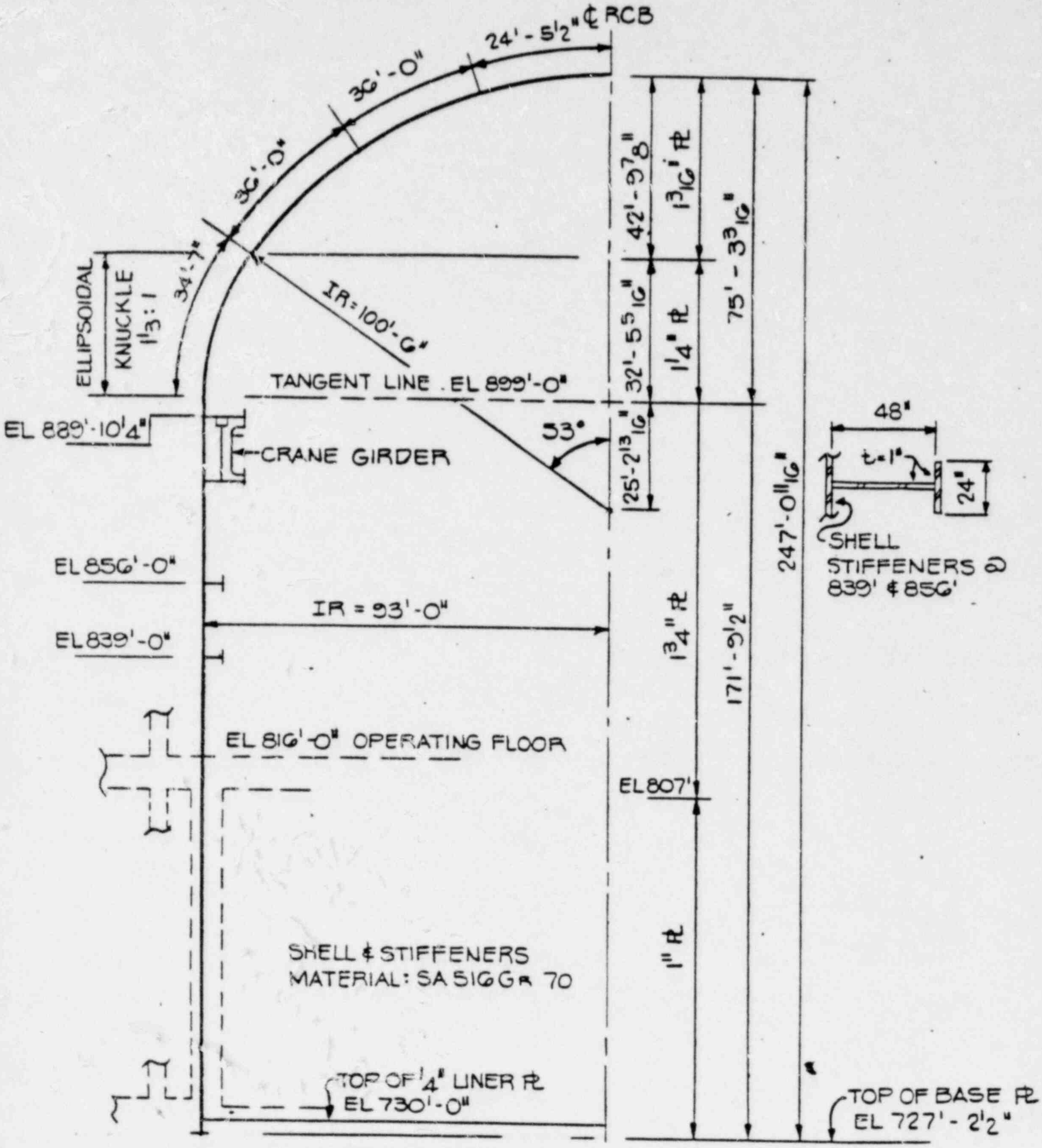


FIGURE 3.8-3 CRBRP CONTAINMENT VESSEL

Question CS 220.27

The design temperature of 250°F must not apply to the complete containment including that portion embedded in concrete. The PSAR should define the design temperature distribution for the complete containment.

Also the Symbol "W" used in Table 3.8-1 is not defined in test.

Response

The Design Temperature of 250°F is a conservative value, well above the maximum temperature of the shell calculated under DBA. For Design Conditions (Load Combinations 4 to 9, Table 3.8-1) the material properties of the steel shell were based on 250°F. Secondary stress (thermal) verification is not required under Design Conditions (PSAR Table 3.8-3). Therefore, temperature distributions are not required. Thermal buckling was verified based on the temperature distribution (axisymmetrical) given in Figure 6.2.11. This temperature distribution accounts for the effects of the insulation blanket between Elevations 825 feet and 816.0 feet and the embedment in concrete below Elevation 816.0 feet.

The definition of "W" wind level was erroneously omitted from the PSAR. "W" is now defined in the modified PSAR 3.8.2.3.1 attached.

3.8.2.3 Loads and Loading Combinations

3.8.2.3.1 Design Loadds

The following loads shall be used in the design of the Containment Vessel and Appurtenances.

D - Dead Load, including the weight of the steel containment vessel, penetration sleeves, equipment and personnel access hatches, and other attachments supported by the vessel, plus loads due to concrete shrinkage.

L - Live Loads, as applicable, including:

1. Penetration Loads (including seismic), as applicable
2. Floor Loads - 100 PSF
3. Walkways - 200 lbs. per linear foot
4. Equipment and Personnel Airlock Floor Load - 300 PSF or 40,000 lbs. moving concentrated load
5. Emergency Airlock Floor Load - 200 PSF or 10,000 lbs.
6. Polar Crane Loads (Ref. 1)
7. Construction Loads*
8. Painters Line Anchor - 2,000 lbs. in any horizontal direction
9. Interior Scaffold - 2,000 lbs. each on any 2 adjacent clips Support Clips - combined with a Dead Load on all clips of 200 lbs. each.

W - Wind Loads at 80 mph (ANSI A58.1-1972)

P_i - Internal Design Pressure (or Transient Pressure Loads)

P_e - External Design Pressure

P_t - Testing Pressure

T_o - Thermal loads due to temperature gradient through walls under normal operating conditions.

T' - Thermal loads due to temperature gradient through walls from accidents, such as major sodium fires.

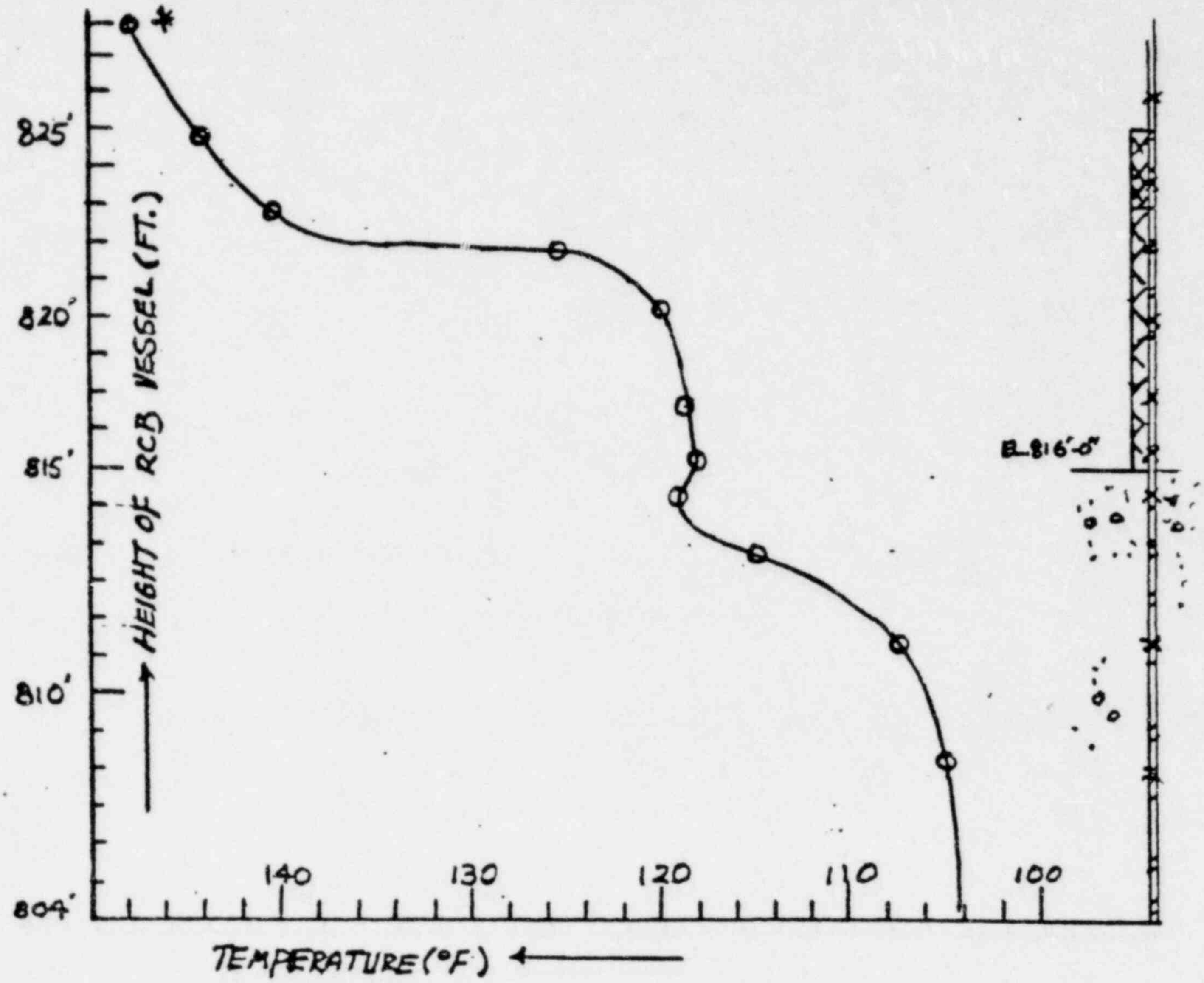
T_t - Thermal load under testing temperature conditions.

E - Loads resulting from an Operating Basis Earthquake (OBE)

E' - Loads resulting from a Safe Shutdown Earthquake (SSE)

* A concrete placement load, resulting from using the vessel shell below operating floor elevation as the formwork for placing the reinforced concrete walls, and loads that are imposed by concrete forms when constructing the confinement shell. A snow load will be considered also during the construction period.

CONTAINMENT VESSEL AXIAL TEMP. PROFILE



* THE TEMP. PROFILE WILL BE EXTRAPOLATED LINEARLY TO 250°F AT ELEVATION 835'.

figure 6.2-11

Question CS 220.39

Information detailing the current version of the MPHI computer code must be provided along with benchmark information for validation of the code and its use in this particular application.

Response

MPHI is a computer program that calculates moment-curvature (M-ϕ) relationships for reinforced concrete sections under thermal gradients, and with an axial force P. The axial force P may be zero. The program has capabilities to account for non-linear material properties, variation of material properties with temperature, non-linear thermal gradients, tensile cracking, and compressive crushing. The M-ϕ relationship provides information on the capacity of a section under a temperature gradient. The moment corresponding to zero curvature is also of interest because it represents the thermal moment for a section restrained against rotation.

For a given axial force P and temperature distribution, the moment-curvature relationship is determined using a numerical procedure that involves the following:

1. The section under consideration (Figure Q220.39-1) is divided into a number of elements by nodal points. The thermal strain ε_t is calculated at each node, I, as:

   $$(\varepsilon_t)_I = (T_I - T_{ref})\ \alpha_{(T\ )}$$

   where

   T_I is the nodal temperature

   T_{ref} is the reference temperature

   $\alpha_{(T\ )}$ is the average coefficient of thermal expansion at T_I

2. A plane section is passed at a curvature ϕ with strains ε_a and ε_b at the two edges of the section. At node I the strain ε_i is:

   $$\varepsilon_i = \left(\frac{\varepsilon_a - \varepsilon_b}{H}\right) h_i + \varepsilon_b$$

   where

   H is the total depth of the section

   h_I is the coordinate of node I

   The mechanical strain at node I is:

   $$(\varepsilon_s)_i = \varepsilon_i - (\varepsilon_t)_i$$

3. Stresses, σ_i, are calculated at each node based on (ε_s) I and a stress-strain relationship which is defined in the input of the problem. Element forces are calculated based on the average of the stresses in the two nodes defining the element.

4. The axial force, P', and the moment, M, are calculated by summation of the element forces and their moments about the centroidal axis.

5. The value of P' is compared with the force under consideration (P), and if different the plane section is moved to a new position maintaining the same curvature ϕ and the process is repeated until convergence.

6. A new curvature is selected and steps 2 to 5 are repeated.

7. In this manner a ϕ versus M relationship is developed for a given P.

The program has options to develop the moment-curvature relationship for the following axial force, P, or restraint conditions.

a. Axial force specified in the input.
b. No axial restraint (P = 0, displ. $\neq$ 0)
c. Full axial restraint (P $\neq$ 0, displ. = 0)

The last two options are special cases of the first and provide the capability to develop mement curvature relationships for two extreme cases of axial restraint. The first option may be used for specific axial force values.

Tensile cracking of the concrete is accounted for automatically by the input σ-ε relations. Compressive crushing of a concrete element is assumed to occur in the part of the section where the strains exceed a limiting value which is input to the program. Degradation of a concrete element is assumed to occur in the part of the section where the temperature of the element exceeds a limiting value specified in the program. The part of the section where the strain or the temperature exceeds the limiting value is automatically removed from the section.

Availability

The MPHI program was developed by Burns and Roe and is available as a Burns and Roe in-house program in the Burns and Roe computer and in time sharing CDC computers.

Verification

The program was verified by hand calculations. For this purpose a reinforced concrete section was considered under a temperature distribution and was

divided into elements (Figure Q220.39-2). Material properties, for the purpose of this calculation, were assumed to be those shown in Figures Q220.39-3 to Q220.39-5. The M-ϕ relationship was developed for the case of full axial restraint and then, selected points on the relationship were calculated by hand. The moment curvature points obtained by hand calculations are in complete agreement as shown in Table Q220.39-1. It should be pointed out that in addition to the values in Table Q220.39-1 the hand calculations provided a detailed check for the intermediate steps of the computer program such as strains and stresses to ensure that there are no errors that might affect the results under a different set of variables.

Further verification of the program was performed using the computer program ANSYS (Reference Q220.39-1). Limited analysis by ANSYS provided information for the moment corresponding to zero curvature (thermal moment) for the section in Figure Q220.39-2 under full axial and no axial restraint. The model is shown in Figure Q220.39-6 and the properties in Figures Q220.39-3 to Q220.39-5. A comparison of the MPHI and ANSYS results is given in Table Q220.39-2.

Application

MPHI is used to calculate the moment-curvature relationship of reinforced concrete sections under temperature distribution and axial force. The moment capacity may be obtained from the M-ϕ relationship. In addition the thermal moment of a section restrained against rotation may be obtained as that corresponding to zero curvature. (Figure CSQ220.39-7)

Reference

QCS 220.39-1 Computer Program ANSYS, Revision 3, Swanson Analysis Systems, Inc., Houston Pennsylvania.

TABLE QCS220.39-1

MPHI VERIFICATION RESULTS

Curvature	Computer Results		Hand Calculations	
1/in.	Force lbs	Moment lbs-in	Force lbs	Moment lbs-in
0	-53,725	-27,327	-53,725	-27,327
+.00002	-53,892	+27,268	-53,892	+27,268
-.00002	-52,879	-76,738	-52,879	-76,738

Moment is positive when it creates tension on top (Node No. 1).
Negative axial force causes compression on the section.

TABLE QCS220.39-2

COMPARISON OF RESULTS FROM MPHI AND ANSYS

CASE	MPHI RESULTS FORCE KIPS	MPHI RESULTS MOMENT k-in	ANSYS RESULTS FORCE KIPS	ANSYS RESULTS MOMENT k-in
Curvature $\phi = 0$				
Full Axial Restraint	-53.7	-27	-53.2	-31
No Axial Restraint	0	-193	0	-188

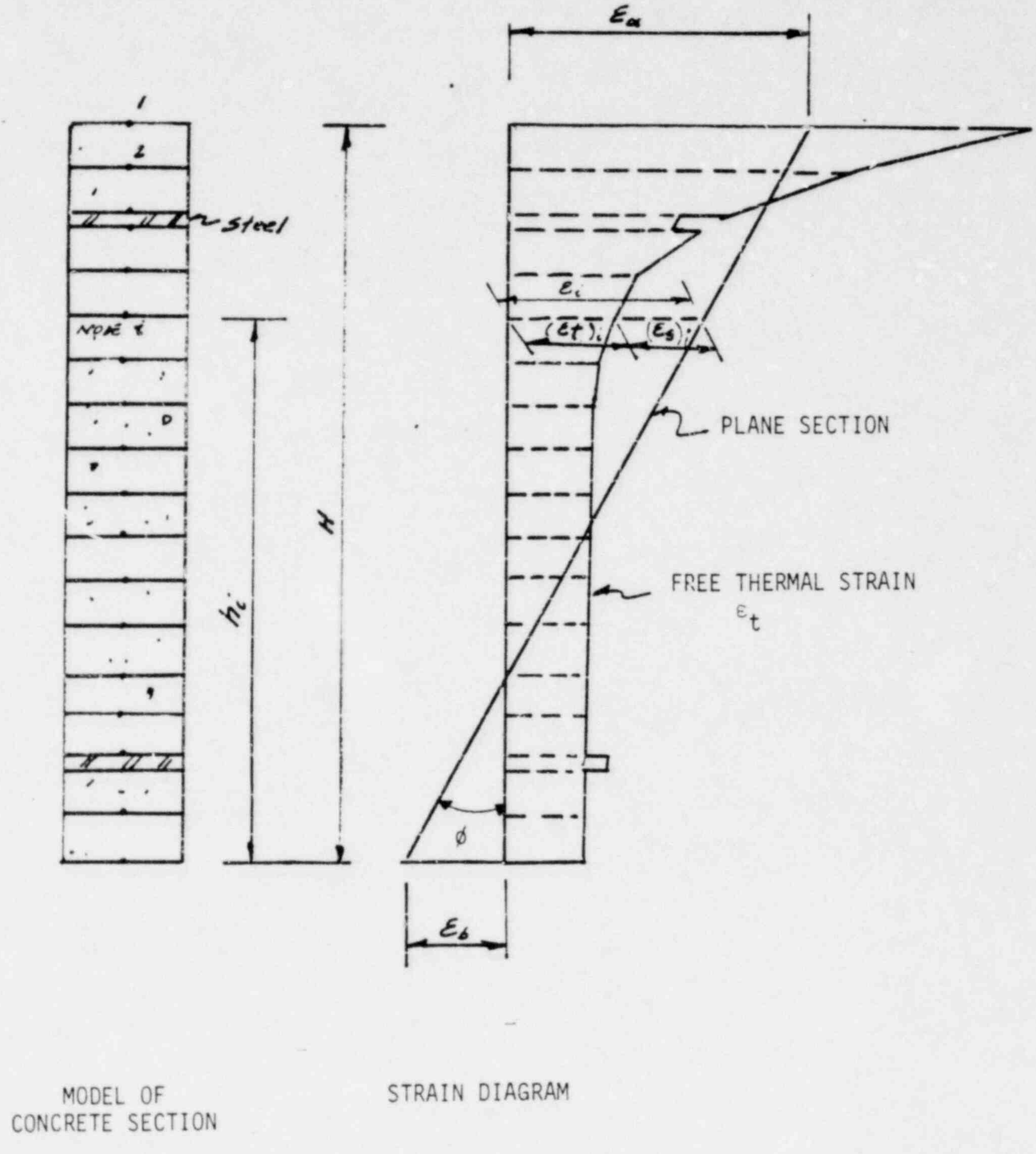


FIGURE Q220.39-1 TYPICAL MPHI MODEL AND STRAIN DIAGRAM

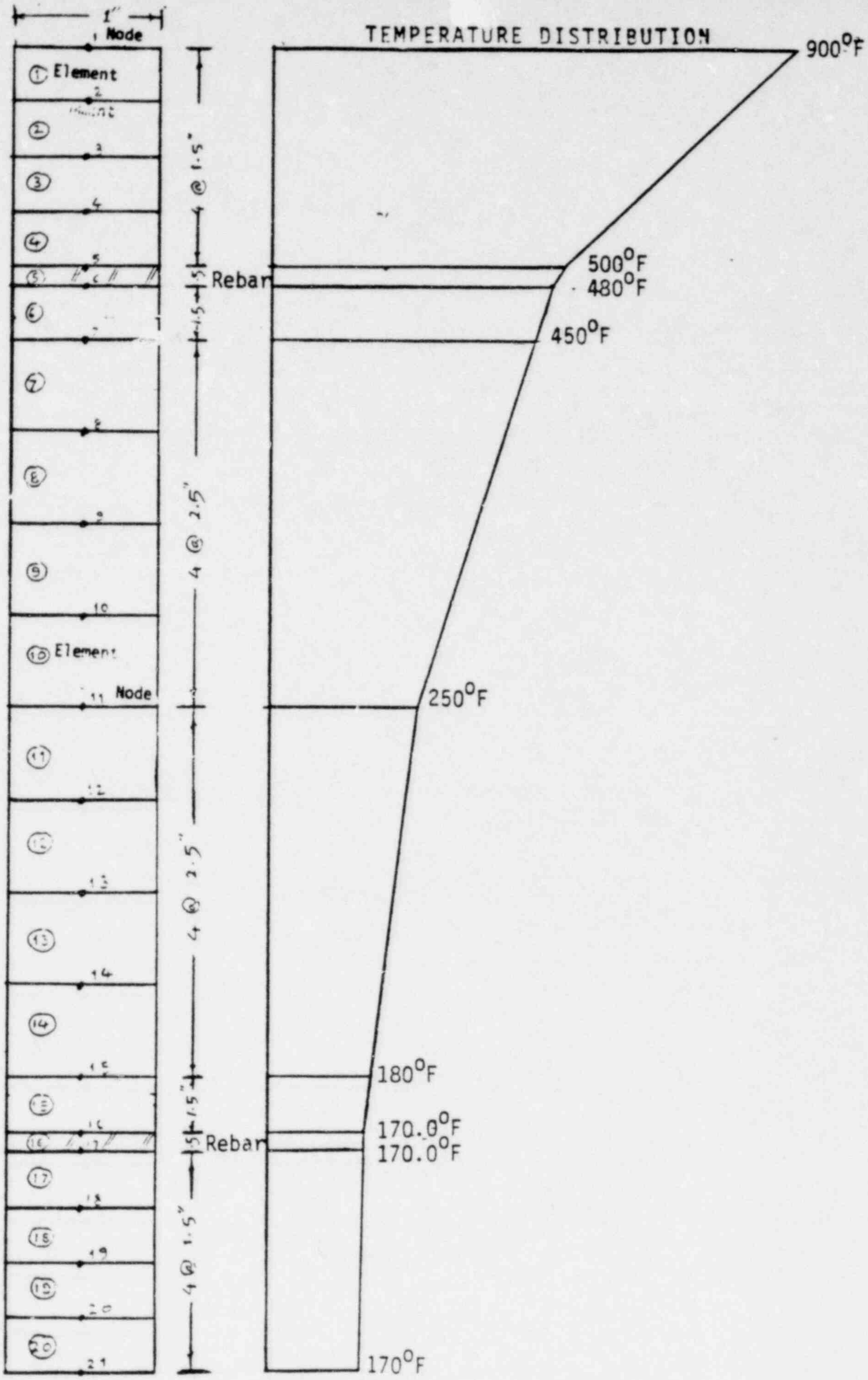


FIGURE Q220.39-2 MODEL AND TEMPERATURES FOR VERIFICATION PROBLEM

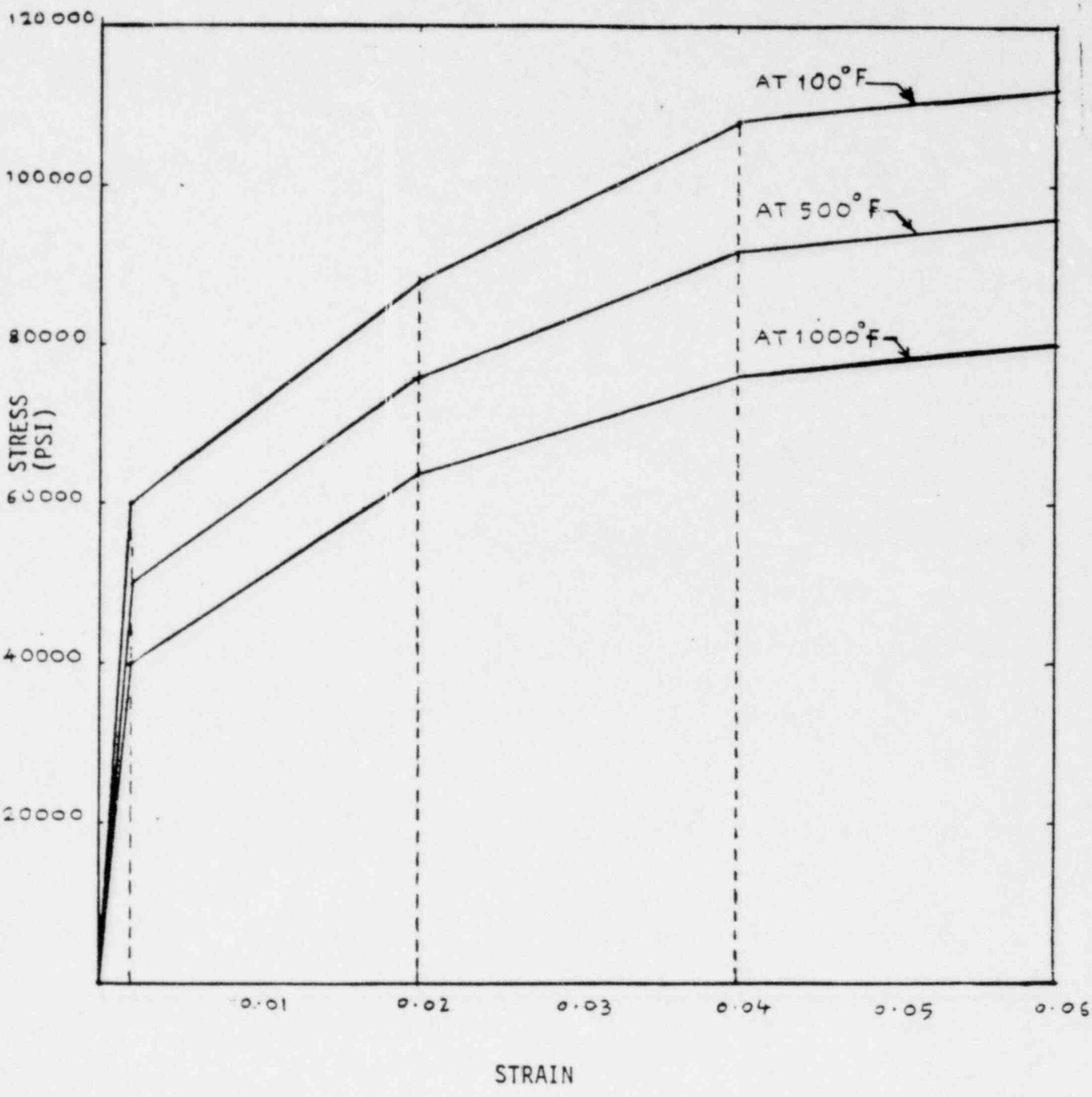
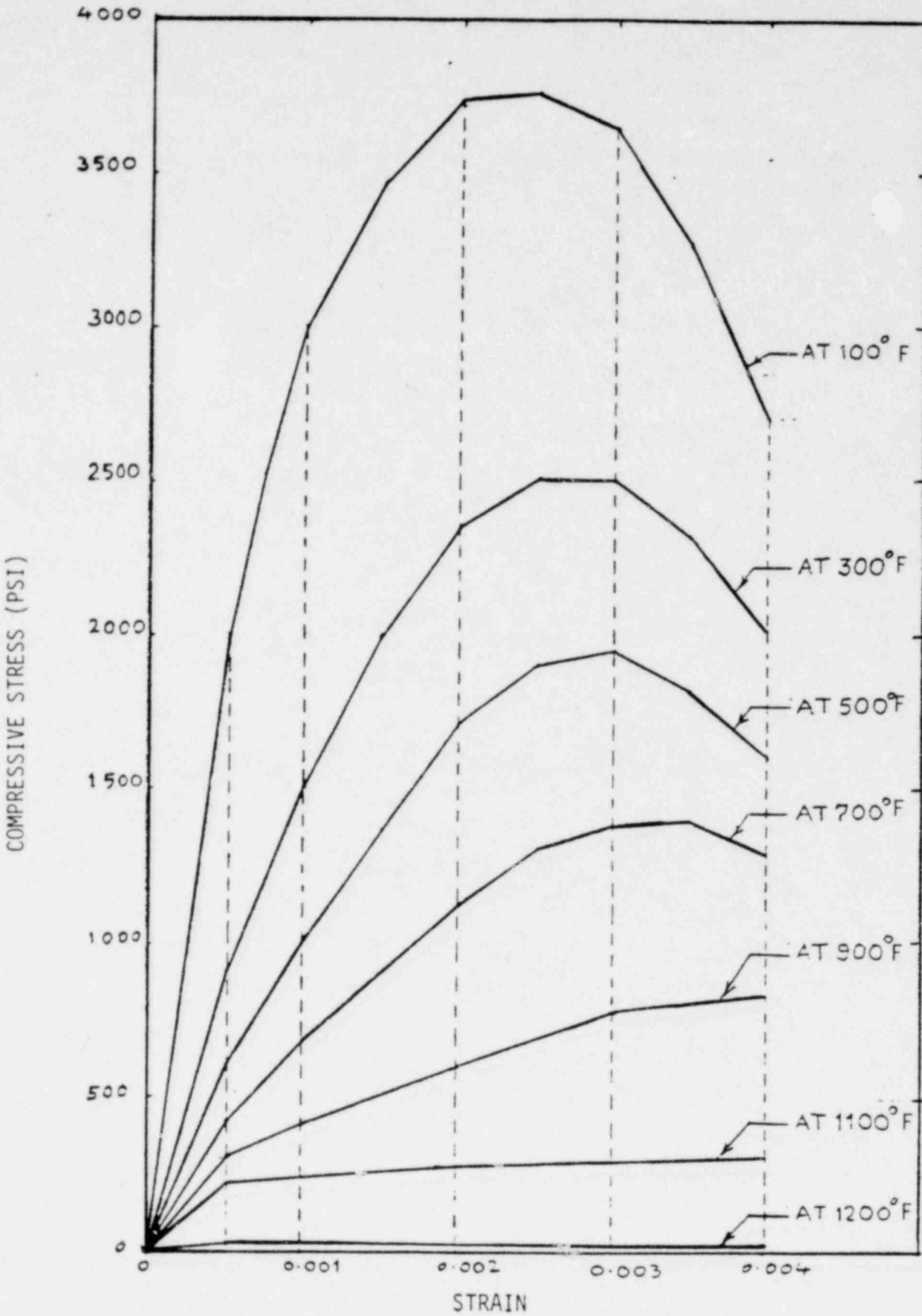


FIGURE Q220.39-3 STRESS-STRAIN CURVES FOR REINFORCING BARS - VERIFICATION PROBLEM



NOTE: Stresses in concrete are zero for tensile strains

FIGURE Q220.39-4 STRESS-STRAIN CURVES FOR CONCRETE - VERIFICATION PROBLEM

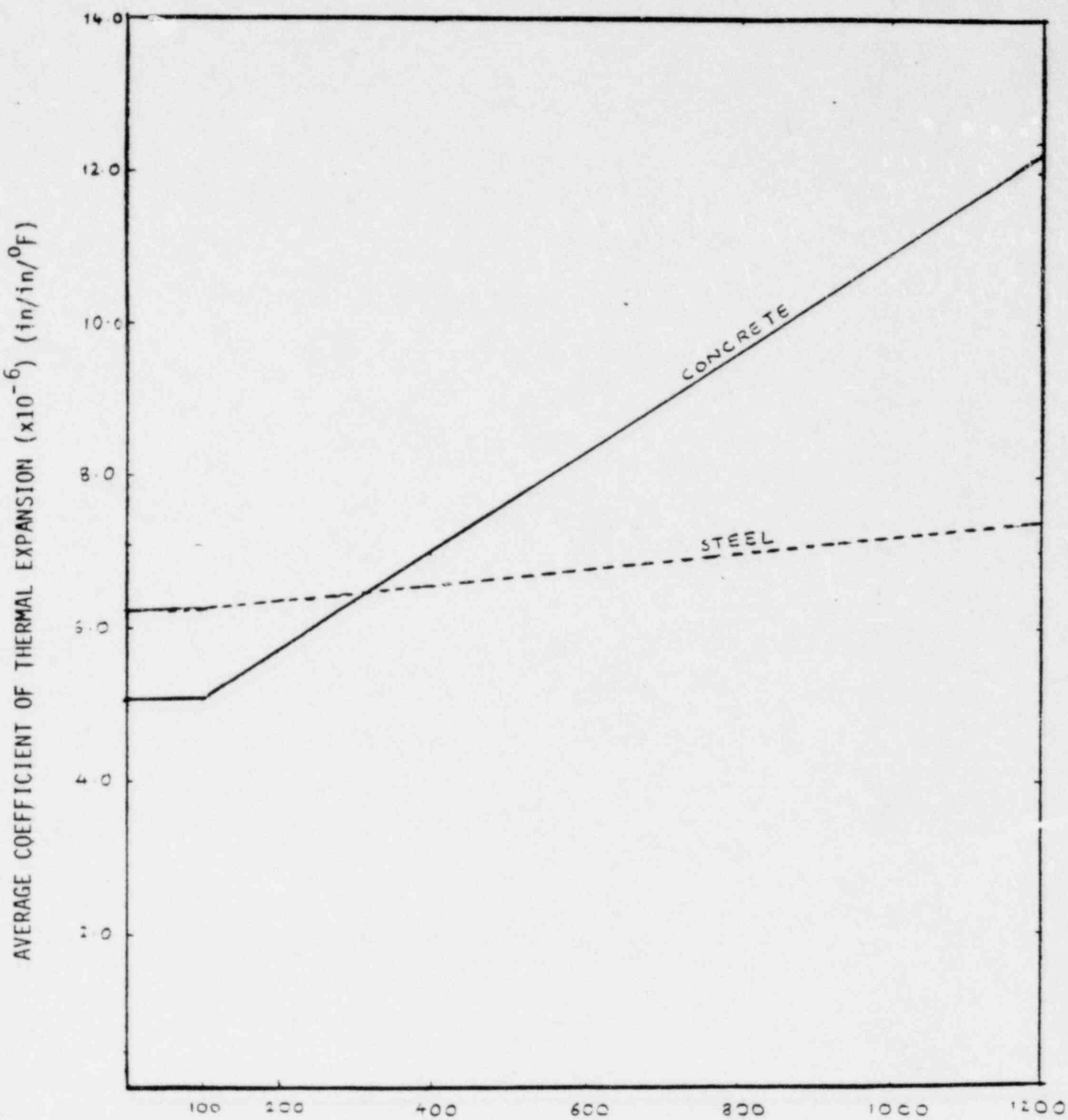


FIGURE Q220.39-5 COEFFICIENT OF THERMAL EXPANSION - VERIFICATION PROBLEM

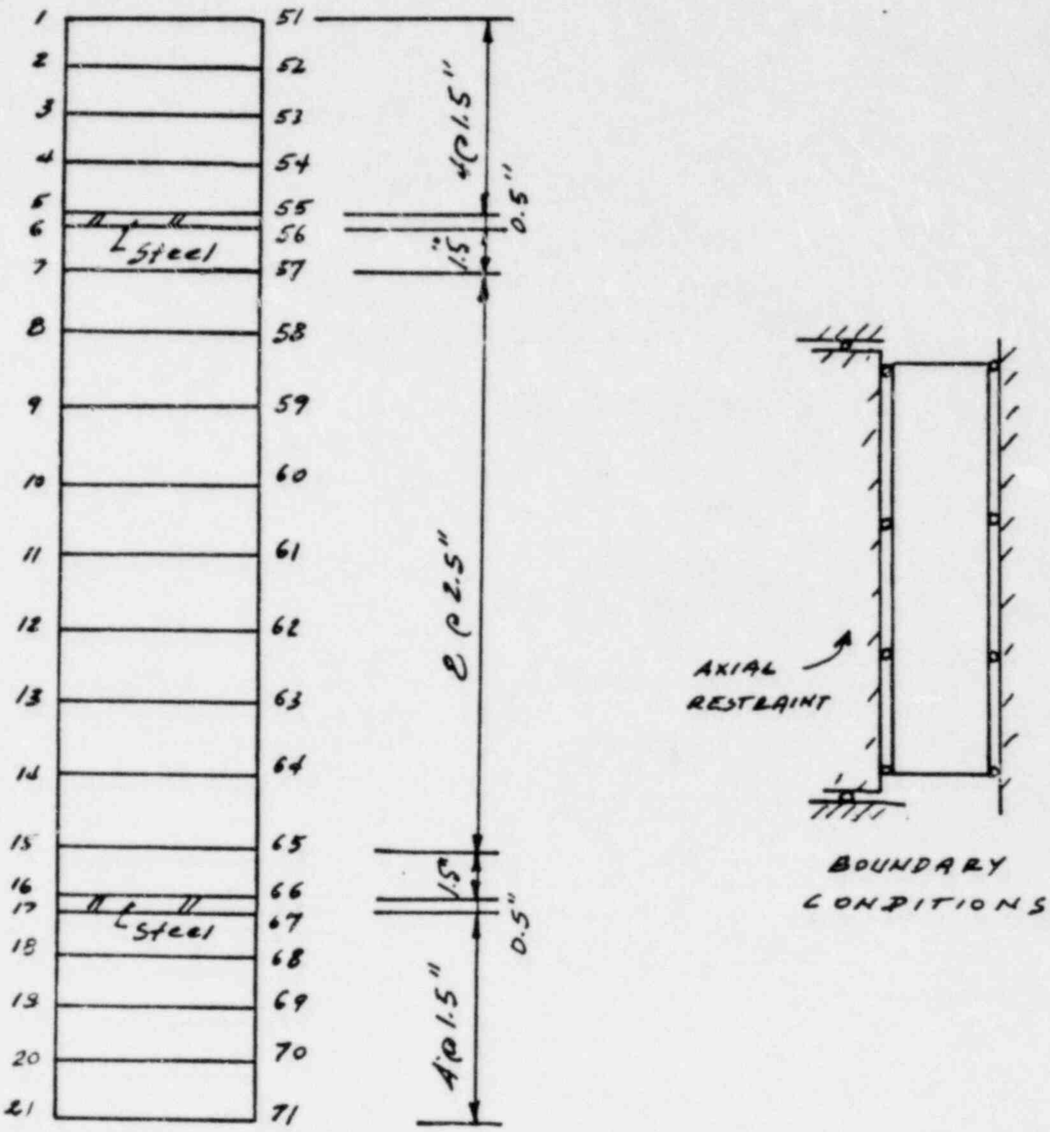


FIGURE Q220.39-6 ANSYS MODEL OF REINFORCED CONCRETE - ϕ = 0

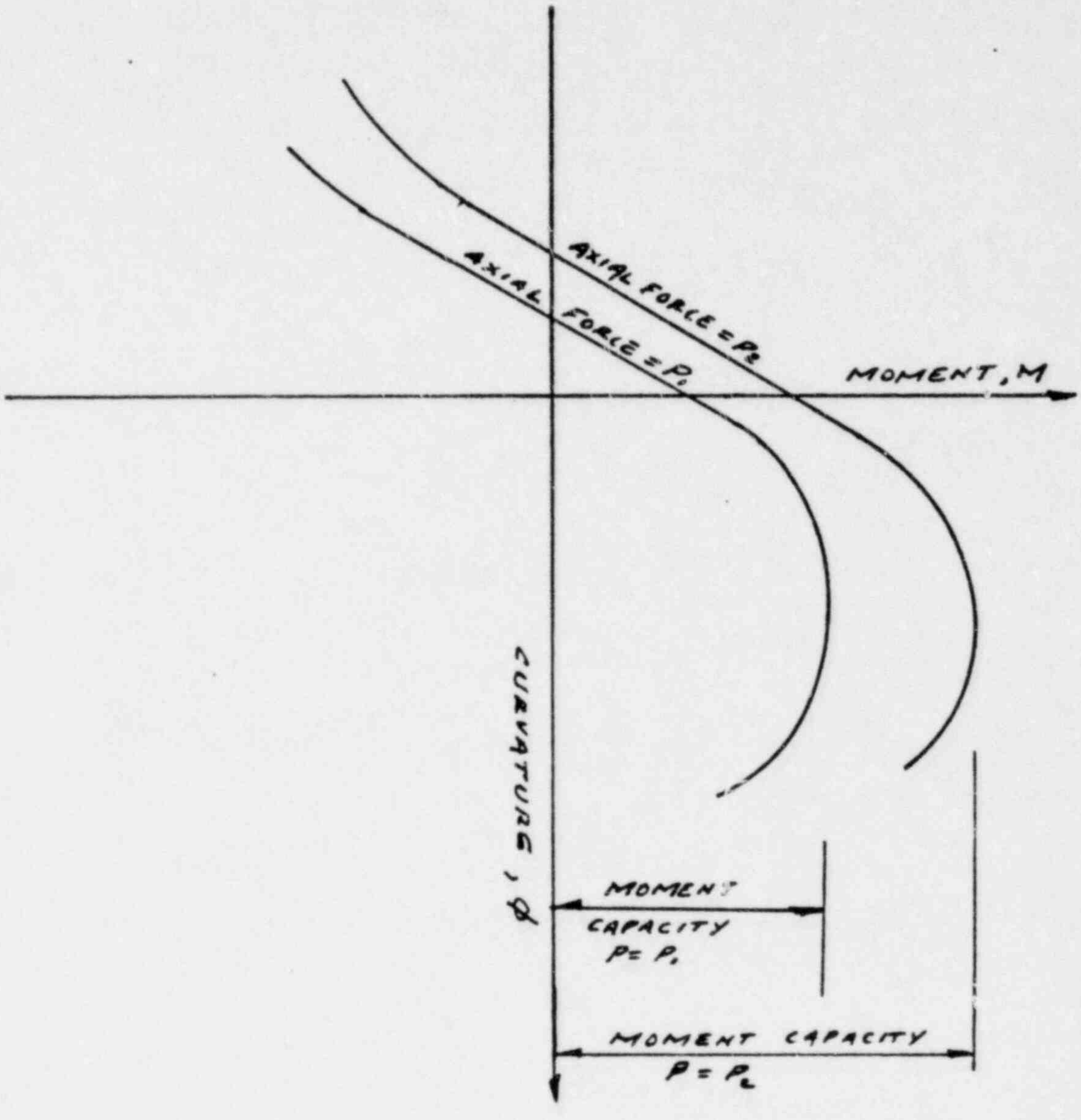


FIGURE Q220.39-7 TYPICAL MOMENT-CURVATURE DIAGRAMS