PSEG Public Service Electric and Gas Company

80 Park Plaza, Newark, NJ 07101 / 201 430-8217 MAILING ADDRESS / P.O. Box 570, Newark, NJ 07101

Robert L. Mittl General Manager
Nuclear Assurance and Regulation

January 26, 1984

Director of Nuclear Reactor Regulation
United States Nuclear Regulatory Commission
7920 Norfolk Avenue
Bethesda, Maryland 20014

Attention: Mr. Albert Schwencer, Chief
Licensing Branch 2
Division of Licensing

Gentlemen:

HOPE CREEK GENERATING STATION
DOCKET NO. 50-354
NRC REVIEW OF STRUCTURAL/GEOTECHNICAL TOPICS

Pursuant to the agreements reached at the meetings held on January 10, 11, and 12, 1984, to review HCGS structural/geotechnical topics with the NRC, attached is one (1) set of responses to those items denoted as Category I target date. A listing of the attached Category I target date items, broken down by date of meeting, is as follows:

Meeting of January 10, 1984:
Items B.6, B.8, B.9, and B.11

Meeting of January 11, 1984:
Items A.7, A.10, A.11, A.15, A.17, B.1, B.2, B.3, B.4, B.5, B.6, B.7, B.8, B.9, B.12, and B.13

Meeting of January 12, 1984:
Items A.1, A.2, and A.3
(Items C.1, C.2, C.4, C.5, C.6, and C.7 will be included as part of Amendment 4 to the FSAR)

In addition, please note that four (4) advance sets of these responses were transmitted to D. Wagner via Federal Express on January 23, 1984.

B001
1/1

Should you have any questions in this regard, do not hesitate to contact us.

Very truly yours,

R. L. Mittl
General Manager -
Nuclear Assurance and Regulation

JES:db

Attachment: Resolution of NRC Comments on Structural/Geotechnical Topics

CC: D. H. Wagner (w/attach.)
USNRC Licensing Project Manager

OQ 14 01/02-A

Response to NRC Audit

Meeting Date: January 10, 1984

Question No.: B.6

QUESTION: Provide tabulations of shear moduli used for:

. soil column model

. design basis model

RESPONSE: The attached Figure 1 shows a typical soil column used for the deconvolution analysis.

The attached Figure 2 shows a schematic representation of the soil model used for the soil-structure interaction analyses.

The attached Table 1 shows a comparison of the final iterated shear moduli used in the deconvolution analysis (which corresponds to free-field conditions) and the final iterated shear moduli used in the SSI analysis. The column of soil from which the SSI values have been extracted corresponds to a location underneath the reactor building, as indicated in Figure 2.

Note that the data provided corresponds to layers 20 to 53 in the soil column's model. The corresponding element numbers at similar depths underneath the reactor building are elements numbers 804 to 837.

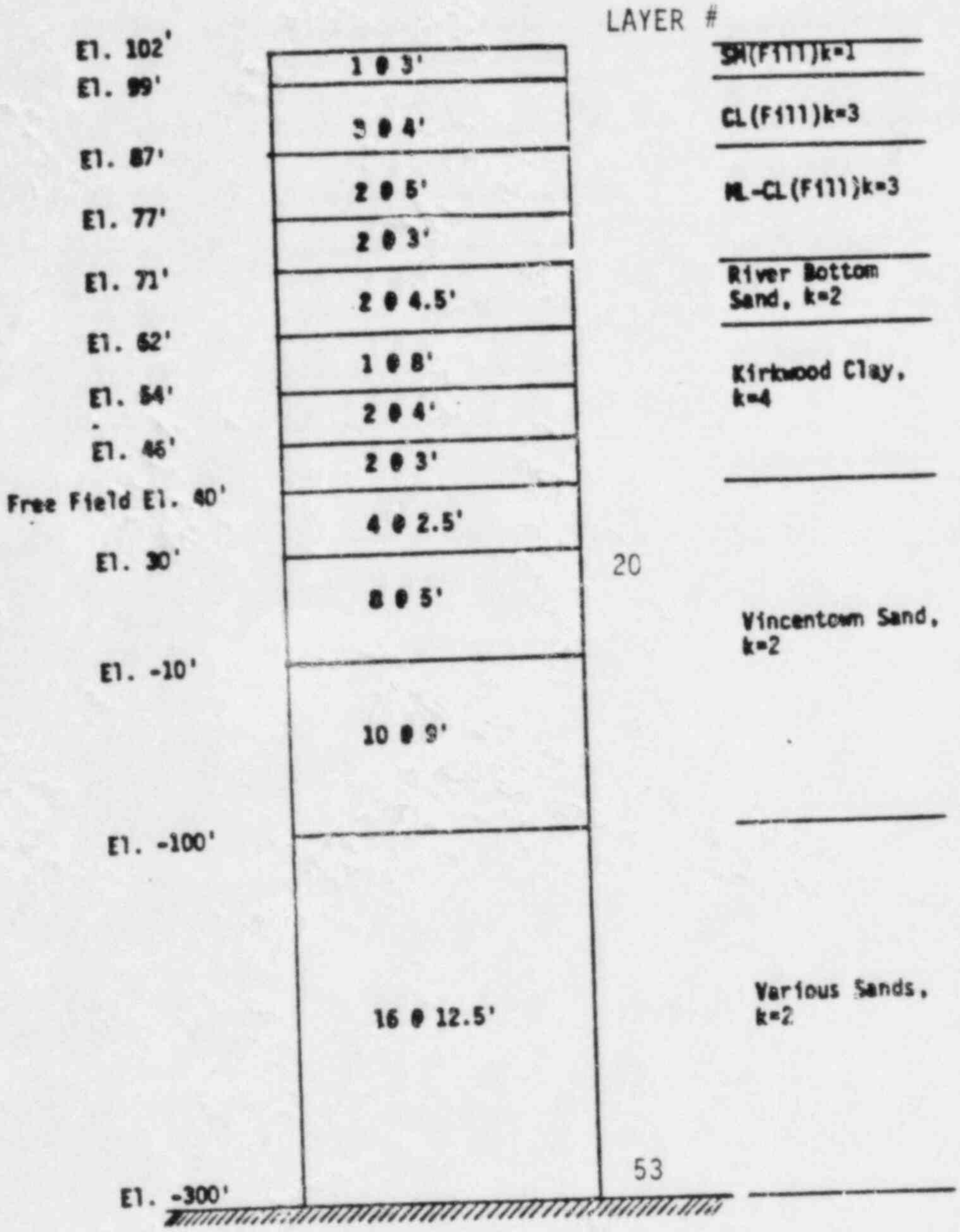


FIGURE 1

IDEALIZED FREE-FIELD SOIL COLUMN FOR DECONVOLUTION
(POWER BLOCK AREA NORTH-SOUTH DIRECTION)

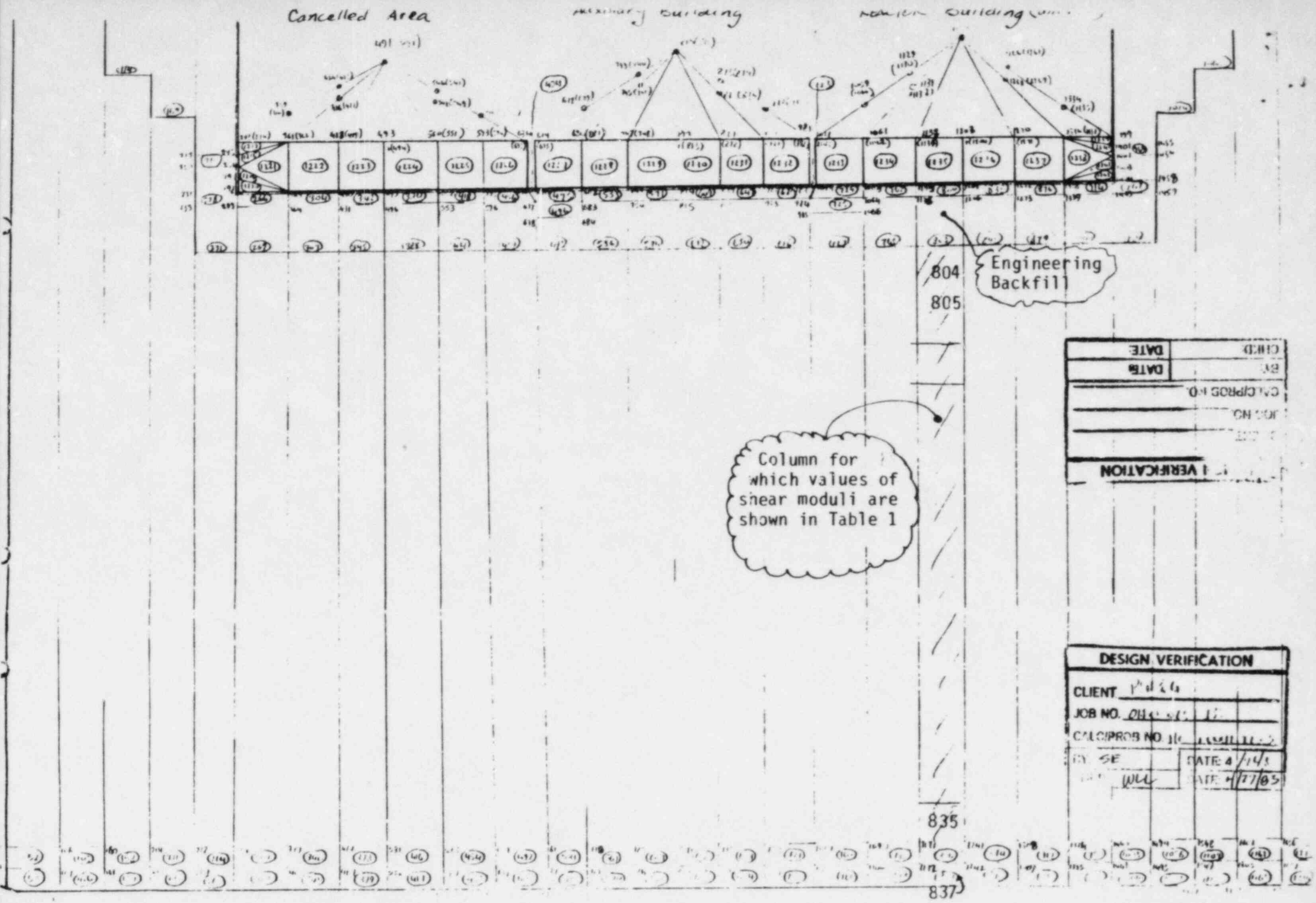


FIGURE 2: SOIL-STRUCTURE INTERACTION MODEL (SCHEMATIC)

Soil Column Model			SSI Model		
LAYER	G-USED (KSF)	EFF. SHEAR STRN	ELEM	G-USED (KSF)	EFF. SHEAR STRN
20	868.	0.8894E-01	804	985.	0.1046E+00
21	1008.	0.7539E-01	805	1295.	0.7926E-01
22	1251.	0.6032E-01	806	1600.	0.6373E-01
23	1395.	0.5456E-01	807	1811.	0.5595E-01
24	1550.	0.5047E-01	808	2003.	0.5043E-01
25	1664.	0.4910E-01	809	2158.	0.4695E-01
26	1762.	0.4892E-01	810	2272.	0.4506E-01
27	1849.	0.4940E-01	811	2370.	0.4399E-01
28	1986.	0.4985E-01	812	2493.	0.4336E-01
29	2121.	0.5108E-01	813	2666.	0.4284E-01
30	2376.	0.4885E-01	814	2879.	0.4195E-01
31	2653.	0.4597E-01	815	3098.	0.4099E-01
32	2836.	0.4449E-01	816	3313.	0.3994E-01
33	3086.	0.4182E-01	817	3564.	0.3834E-01
34	3292.	0.3986E-01	818	3818.	0.3666E-01
35	3595.	0.3702E-01	819	[illegible]092.	0.3485E-01
36	3885.	0.3480E-01	820	4360.	0.3325E-01
37	4109.	0.3355E-01	821	4473.	0.3295E-01
38	3629.	0.3910E-01	832	3917.	0.3843E-01
39	3672.	0.4003E-01	823	3905.	0.3950E-01
40	3682.	0.4136E-01	824	3909.	0.4048E-01
41	3721.	0.4234E-01	825	3921.	0.4148E-01
42	3727.	0.4362E-01	826	3930.	0.4258E-01
43	3771.	0.4434E-01	827	3952.	0.4356E-01
44	3792.	0.4525E-01	828	3970.	0.4459E-01
45	3802.	0.4622E-01	829	3983.	0.4564E-01
46	3815.	0.4712E-01	830	3995.	0.4666E-01
47	3810.	0.4823E-01	831	3998.	0.4770E-01
48	3810.	0.4925E-01	832	4001.	0.4867E-01
49	3801.	0.5039E-01	833	4001.	0.4962E-01
50	3787.	0.5159E-01	834	4002.	0.5052E-01
51	3779.	0.5271E-01	835	4006.	0.5131E-01
52	3758.	0.54[illegible]4E-01	636	4009.	0.5211E-01
53	3731.	0.5549E-01	837	4009.	0.5295E-01

TABLE 1: COMPARISON OF FINAL ITERATED SHEAR MODULI FOR FREEFIELD AND SSI CONDITIONS

Response to NRC Audit

Meeting Date: January 10, 1984

Question No.: B.8

QUESTION: Provide calculation showing how torsional stiffness was established for the reactor building.

RESPONSE: A typical calculation of evaluating torisonal stiffness is provided in the attachment. This calculation is performed for beam element between nodes 101 and 294 of Figure 1.

The procedure consists of assigning a number to each wall contributing to the torsional stiffness of the system. (This is shown in Figure 2). For each wall, axial and shear areas are computed. The total shear area in a given direction is computed by adding the contribution of each individual wall in that particular direction. The contribution of the perpendicular walls is neglected. For example, in determining the total shear area in the Y-direction, wall 2 will contribute but wall 13 will not be considered. For special cases such as wall 1 (circular section), half of the area is contributed to each direction. The procedure for arriving at the total torsional stiffness at the location of interest is shown in the tables attached.

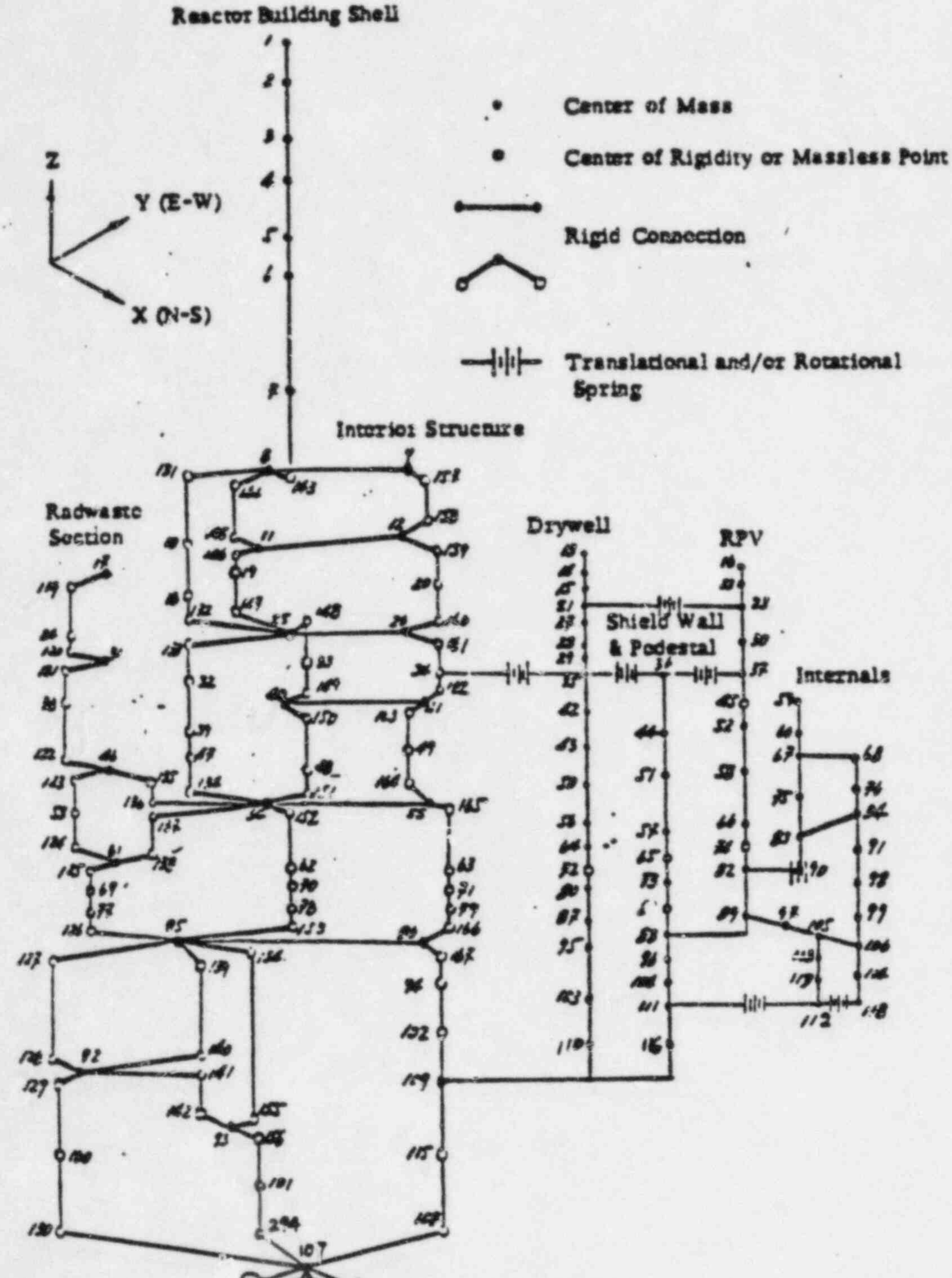


FIGURE 1: REACTOR BUILDING MATHEMATICAL MODEL

ATTACHMENT TO RESPONSE D-3

EDS NUCLEAR
220 MONTGOMERY ST. • SAN FRANCISCO, CALIFORNIA 94104 • (415) 421-5056

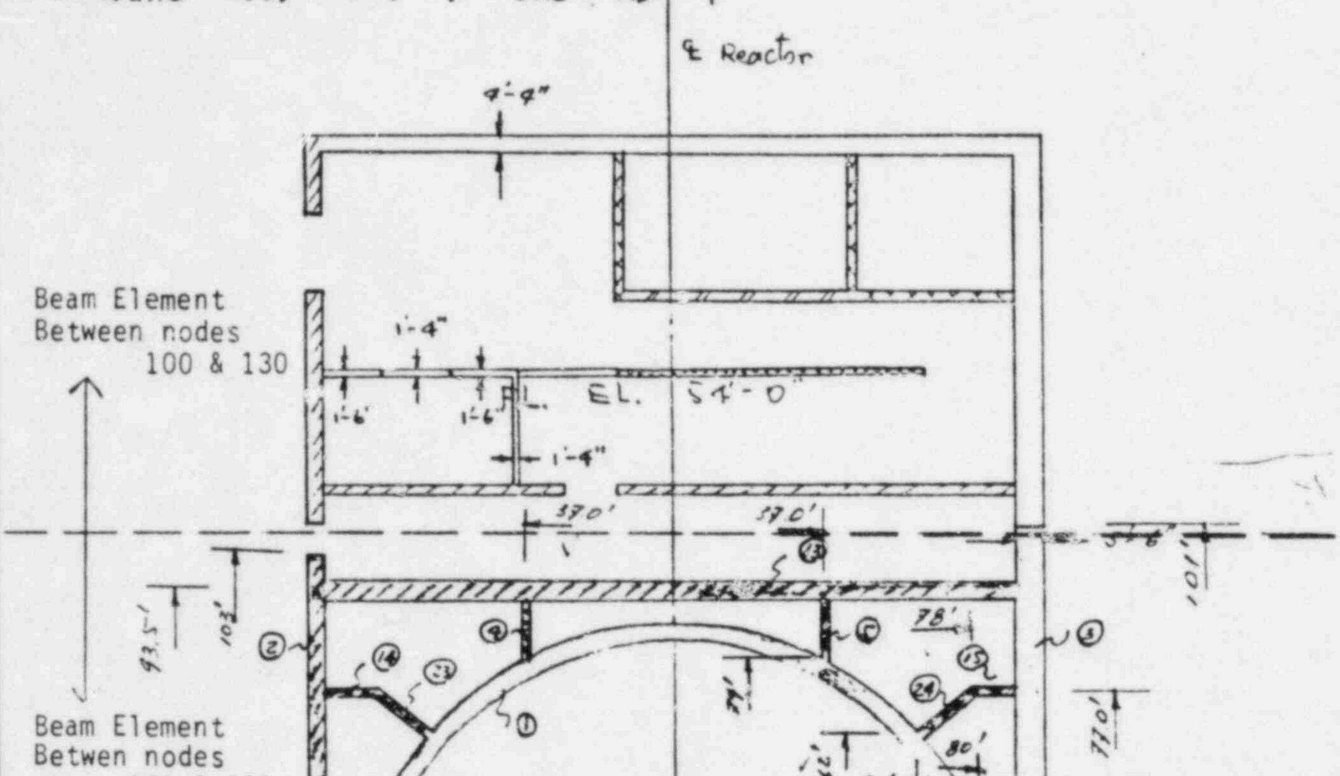
JOB NO.	PROJECT	BY
1600012	Hope Creek Reactor Bldg.	[illegible]
CLIENT	TITLE	DATE
PSEG	Structural Walls @ El. 54'-0"	6/6/75
		CHECKED
		[illegible]

Rev-0

(19)

Structural Walls @ El. 54'-0"

(Str. Walls Bet. 54' & 77'-Shell-A)



Ref: Drawing SK-C-568 (Revision H)

FIGURE 2

EL. 54 WALLS (A)

SECTION	AREA A	A_{sx}	A_{sy}	$x_{CG.}$	$Y_{CG.}$	$A_{sx} \cdot Y_{CG.}$	$A_{sy}\, x_{CG.}$	$\bar{x} = (x_{CG} - x_{sc})$	$\bar{Y} = (Y_{CG} - Y_{sc})$	$A_{sx} \cdot \bar{Y}^2 \times 10^6$	$A_{sy} \cdot \bar{x}^2 \times 10^6$
1	1583.363	791.681	791.582	0.0	0.0	0	0.0	-17.05	32.81	0.8522 ✓	0.2301 ✓
2	687.750		687.75	-93.5			-64304.62	-110.55			8.4032 ✓
3	1249.875		1249.875	94.75			118425.64	77.70			7.5459 ✓
4	13.00		13.0	-37.0			-481.0	-54.05			0.0380 ✓
5	13.00		13.0	37.0			481.0	19.95			0.0052 ✓
6	18.00		18.0	-81.0			-1458.0	-98.05			0.1730 ✓
7	55.00		55.0	-37.0			-2035.0	-54.05			0.1607 ✓
8	40.50		40.50	-37.0			-1498.5	-54.05			0.1183 ✓
9	75.00		75.0	12.0			900.0	-5.05			0.0019 ✓
10	55.0		55.0	37.0			2035.0	+19.95			0.0219 ✓

Rev. 0

EDS NUCLEAR: 220 MONTGOMERY ST. • SAN FRANCISCO, CALIFORNIA 94104

JOB NO. 1400012 | CLIENT PSEG | BY VNC | DATE 6/12/75

PROJECT HOPE CREEK | CKD. GA | DATE 18 JUN 75

ITEM REACTOR BLDG SHELL WALLS. SHEAR AREA, TORSION CONSTANT EL. 54 (A) | SHT. 1 OF 3

EDS NUCLEAR: 220 MONTGOMERY ST. • SAN FRANCISCO, CALIFORNIA 94104

JOB NO. 1400012 | CLIENT PSE G | BY VNC | DATE 9/12/75

PROJECT HOPE CREEK | CKD. GH | DATE 18 JUN 75

ITEM EL. 54 - (A) REACTOR BLDG SHELL WALLS. SHEAR AREA, TORSION CENTER | SHT. 2 OF 3

SECTION	AREA A	A_{sx}	A_{sy}	$x_{CG.}$	$Y_{CG.}$	$A_{sx} \cdot Y_{CG.}$	$A_{sy}\, x_{CG.}$	$\bar{x} = (x_{CG} - x_{sc})$	$\bar{Y} = (Y_{CG} - Y_{sc})$	$A_{sx} \cdot \bar{Y}^2 \times 10^6$	$A_{sy} \cdot \bar{x}^2 \times 10^6$
11	36.00	--	36.00	37.0	——	----	1332.0	19.95	----	----	0.0143 ✓
12	18.00		18.00	81.0			1458.0	63.95			0.0736 ✓
13	564.75	564.75			93.5	52804.13 ✓			126.31	9.0164	
14	28.00	28.0			77.0	2156.0 ✓			109.81	0.3376	
15	28.00	28.0			77.0	2156.0			109.81	0.3376	
16	36.00	36.0			37.0	1332.0 ✓			69.81	0.1754	
17	36.00	36.0			37.0	1332.0 ✓			69.81	0.1754	
18	36.00	36.0			-37.0	-1332.0 ✓			-4.19	0.0006 ✓	
19	36.00	36.0			-37.0	-1332.0			-4.19	0.0006 ✓	
20	100.00	100.0			-101.0	-10100.0 ✓			-68.19	0.4650	

Rev. 0

EDS NUCLEAR: 220 MONTGOMERY ST. • SAN FRANCISCO, CALIFORNIA 94104

JOB NO. 1ACCC12 | CLIENT PSE G | BY VNC | DATE [illegible]

PROJECT HOPE CREEK | CKD. [illegible] | DATE [illegible]

ITEM REACTOR BLDG SHELL WALLS SHEAR AREA, TORSION CONSTANT @ EL. 54 .A | SHT. [illegible] OF [illegible]

Rev. 0

SECTION: 22 $\frac{19}{16}$ 21.922 A_{sx} = 32.0 A_{sy} = 30.0

SECTION	AREA	A_{sx}	A_{sy}	X_{CG}	Y_{CG}	$A_{sx} \cdot Y_{CG}$	$A_{sy}\ X_{CG}$	$\bar{X} = (X_{CG} - X_{sc})$	$\bar{Y} = (Y_{CG} - Y_{sc})$	$A_{sx} \cdot \bar{Y}^2 \times 10^6$	$A_{sy} \cdot \bar{X}^2 \times 10^6$
21	100.00	100.0	---	---	-101.0	-10100.0			-68.19	0.4658	
22	1035.375	1035.375			-126.0	~~-130457.25~~			-93.19	~~8.9916~~ [illegible]15	
23	43.863	32.0	30.0	-70.0	69.5	2224.0	-2100.0	-87.05	102.31	0.3350	0.2273
24	43.863	32.0	30.0	70.0	69.5	2224.0	2100.0	52.95	102.31	0.3350	0.0841
25	72.139	50.0	52.0	-68.5	-79.0	-3950.0	-3562.0	-85.55	-46.19	0.1067	0.3806
26	72.139	50.0	52.0	68.5	-79.0	-3950.0	3562.0	51.45	-46.19	0.1067	0.1376
Σ		2955.806	3216.917			~~-96993.12~~ [illegible]	5484.153			21.~~6945~~ [illegible]	17.6178

$X_{sc} = 17.05$ $Y_{sc} = -32.81$

TORSION CONSTANT $= J + \left| \Sigma A_{sx} \cdot \bar{Y}^2 + \Sigma A_{sy} \cdot \bar{X}^2 \right| = (1483.263)\left(\frac{82.5+86.5}{2}\right)^2 + 21.~~6945~~ \times 10^6 + 17.617 \times 10^6$

$= 50.~~4645~~ \times 10^6$

Response to NRC Audit

Meeting Date: January 10, 1984

Question No.: B.9

QUESTION: Provide calculations showing drywell stick model development (provide for one section only).

RESPONSE: The methodology used in the development of the Drywell stick is covered in detail in Attachment I (Reference: Impell Report No. SED-76-017, Rev. 5).

In summary, equivalent beam properties were obtained based on analyses of the drywell structure using an axisymmetric model. To develop the correct displacement pattern and benchmark both models, an anti-symmetric unit load was applied to the axisymmetric model. As seen by the results in the attached Figure B-228, the equivalent beam model used for subsequent analysis reproduces very closely the displacement pattern obtained by the axisymmetric shell model.

A sample calculation of the equivalent beam properties between elevation 86.92 ft and 91.06 ft is provided in Attachment II.

ATTACHMENT I

9.0 DISCUSSION OF SPECIAL STUDIES

During the course of the Hope Creek seismic analysis, a number of special studies were required in order to evaluate the effect of isolated structural components on the overall structural response of the Power Block area. In addition, work was performed to summarize and envelope the floor response spectra for individual buildings, in order to generate building envelope curves. In addition, a study was performed to investigate effects on seismic response due to cancellation of Unit 2.

9.1 Drywell Study

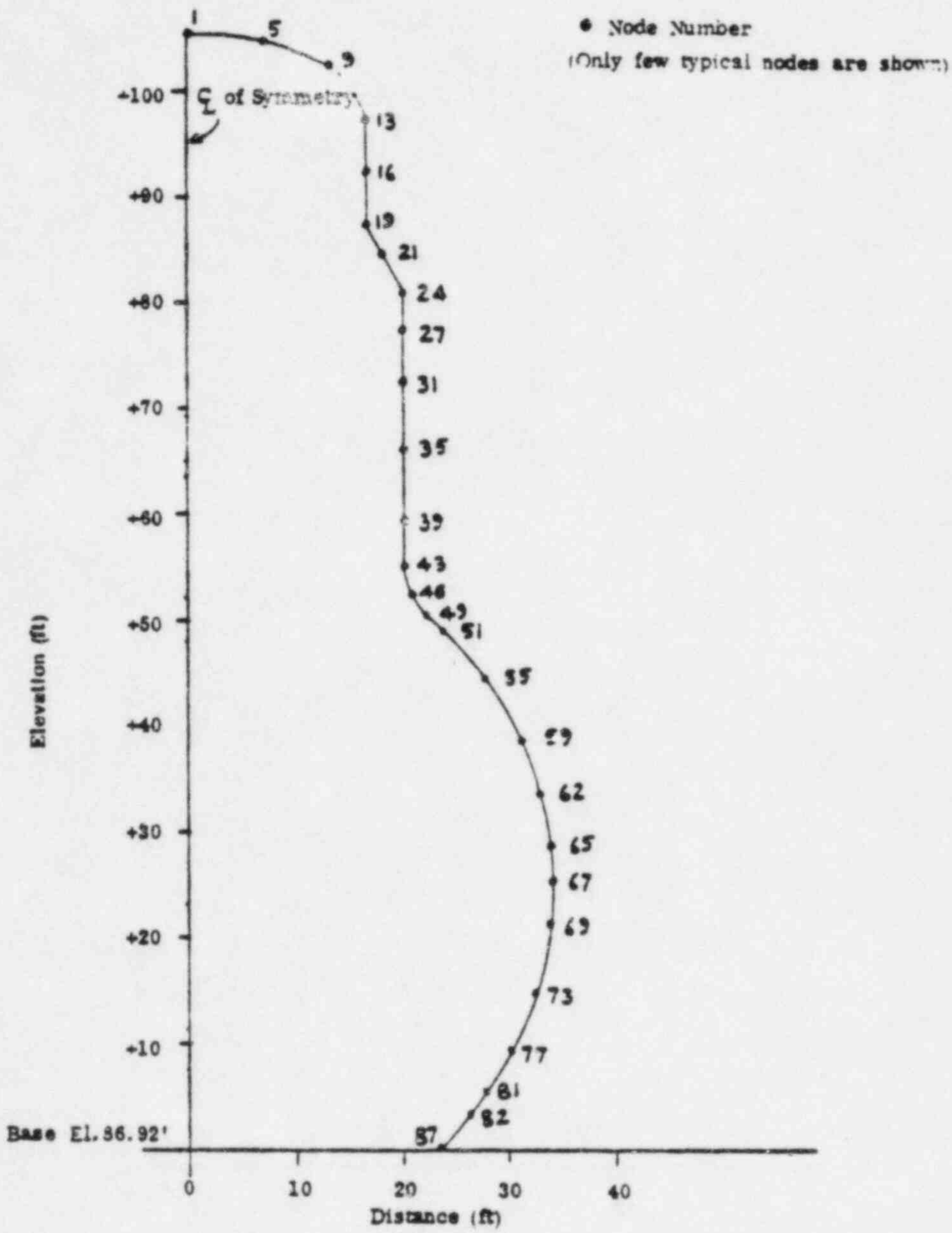
The mathematical model for the Drywell constructed by the conventional approach of classical beam theory, when incorporated in the Reactor Building model, appeared to produce unrealistically high response results for the Drywell base shear and moment. In order to accurately identify the interaction effects between the Drywell and other structural components of the Reactor Building, the Drywell dynamic properties were re-evaluated by means of an axisymmetric shell analysis under static and dynamic loadings.

Initially, a unit static shear was applied at the shear lug location of the axisymmetric shell model (Figure B-225). As can be seen from Figure B-226, the results showed that large longitudinal moments and circumferential forces occurred at those critical areas having abrupt change in geometric shape. An "equivalent" beam model was then constructed to match the static deflection of the shear model. This model had two (2) flexible short segments at the base of the Drywell and at the intersection of the cylindrical shell portion and the spherical shell portion (Figure B-227). As a result of these flexible segments, the model is much more flexible than the "classical" beam model.

A comparison of static displacements between the three models is presented in Figure B-228. As can be seen from the figure, the revised beam model closely matches the displacement pattern of the axisymmetric model, with both being considerably more flexible than the "classical" beam model.

In order to confirm the dynamic properties of the "equivalent" model, frequency and mode shape analyses, followed by a fixed base response spectrum analysis using the Regulatory Guide 1.60 Design Spectrum, were performed for both the axisymmetric shell model and the equivalent beam model. As evidenced by Figures B-229 through B-233, good agreement between the response of these two models was found.

As a result of the above analyses, the Drywell properties for the Reactor Building analyses were based on the equivalent beam model.

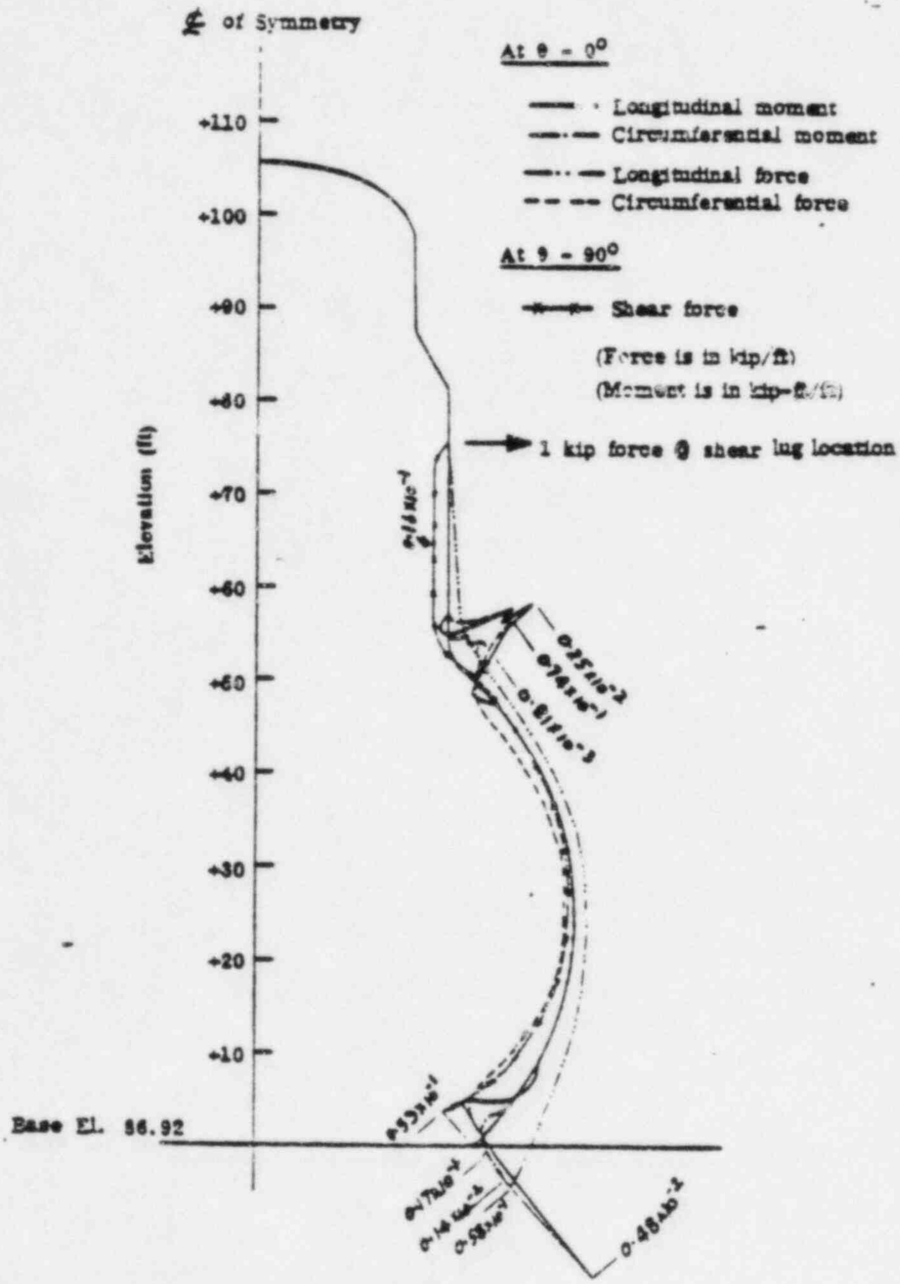


DRYWELL AXISYMMETRIC MODEL

TOTAL NODES = 87

FIGURE B-225

PUBLIC SERVICE ELECTRIC & GAS CO.	SEISMIC STRUCTURAL ANALYSES
HOPE CREEK PROJECT	

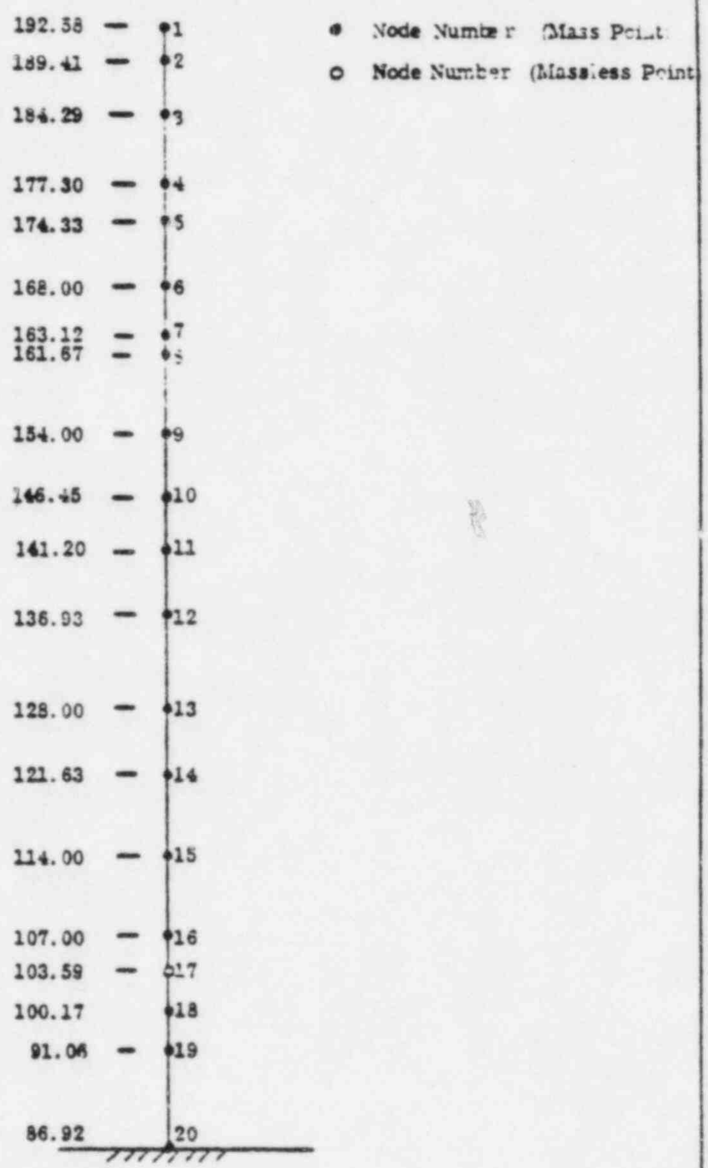


DRYWELL - AXISYMMETRIC SHELL MODEL (87 NODES)

SHELL STRESS

FIGURE B-226

PUBLIC SERVICE ELECTRIC & GAS CO.	SEISMIC STRUCTURAL ANALYSES	
HOPE CREEK PROJECT		



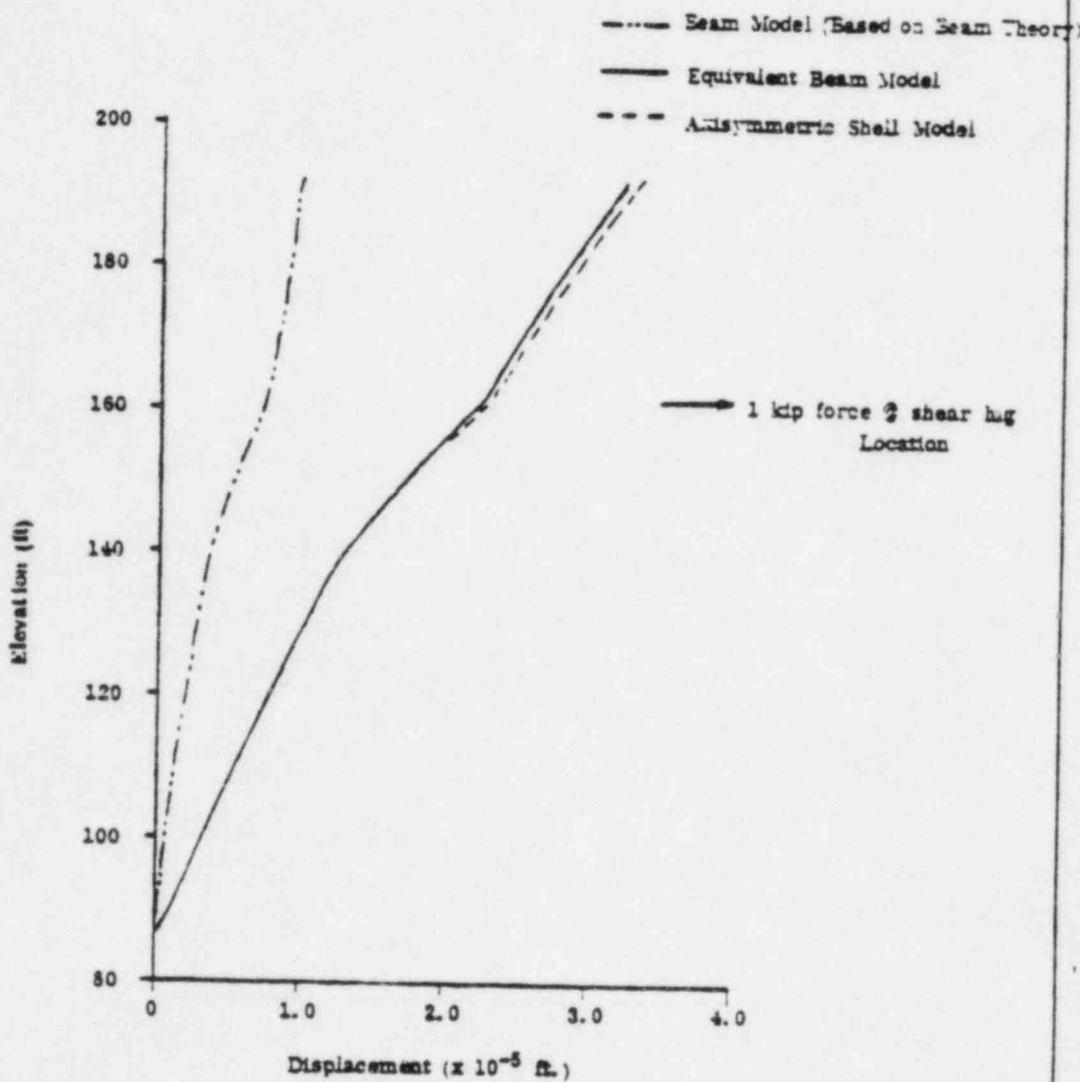
DRYWELL EQUIVALENT BEAM MODEL

TOTAL NODES = 20

FIGURE B-227

PUBLIC SERVICE ELECTRIC & GAS CO.	SEISMIC STRUCTURAL ANALYSES
HOPE CREEK PROJECT	

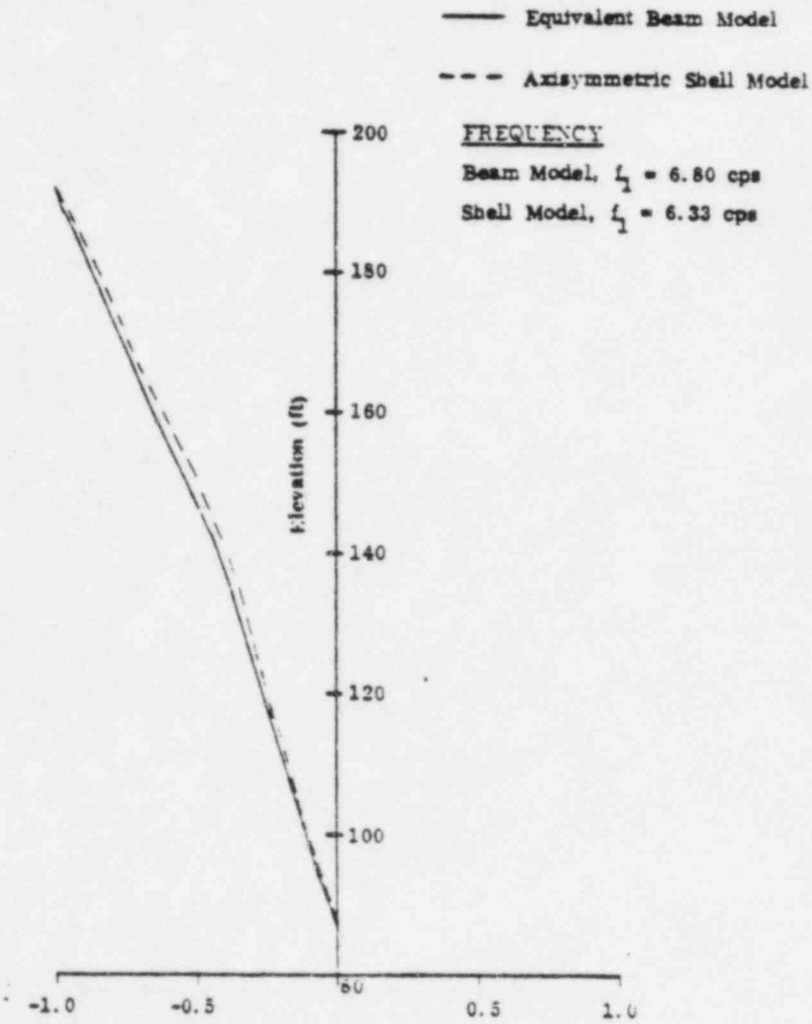
STATIC DISPLACEMENT COMPARISON



DRYWELL (FREE STANDING)

FIGURE B-228

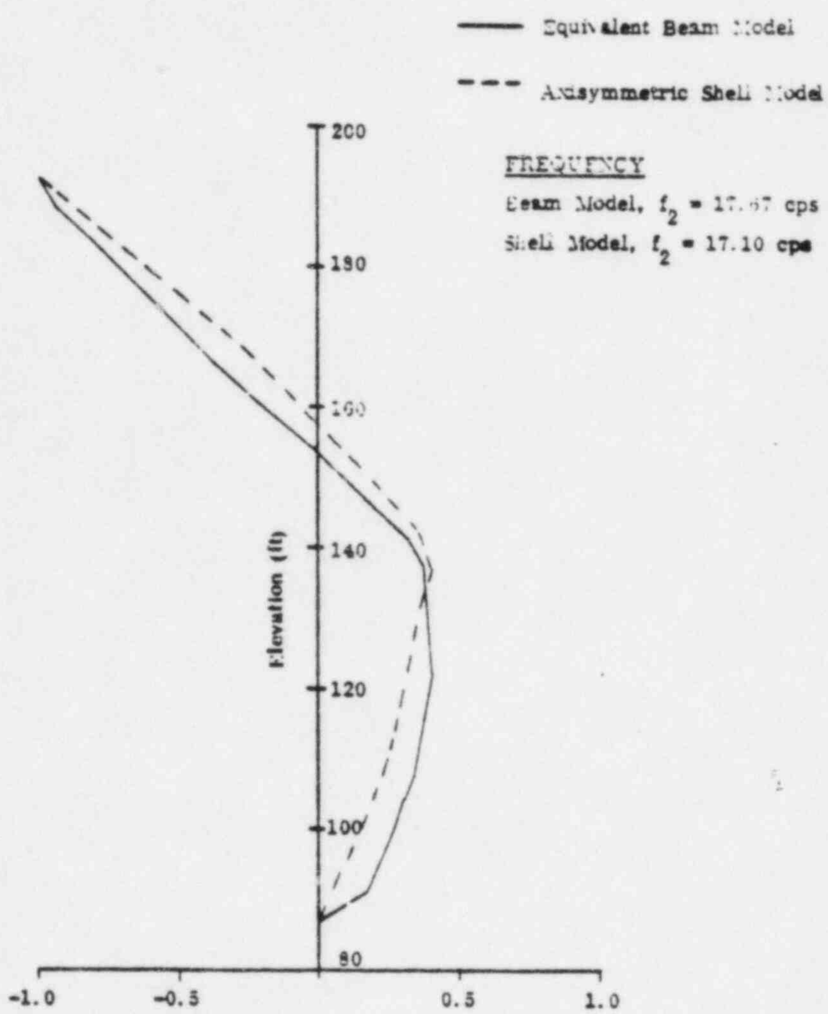
MODE SHAPE COMPARISON - FIRST MODE



DRYWELL (FREE STANDING)

FIGURE B-229

MODE SHAPE COMPARISON - SECOND MODE



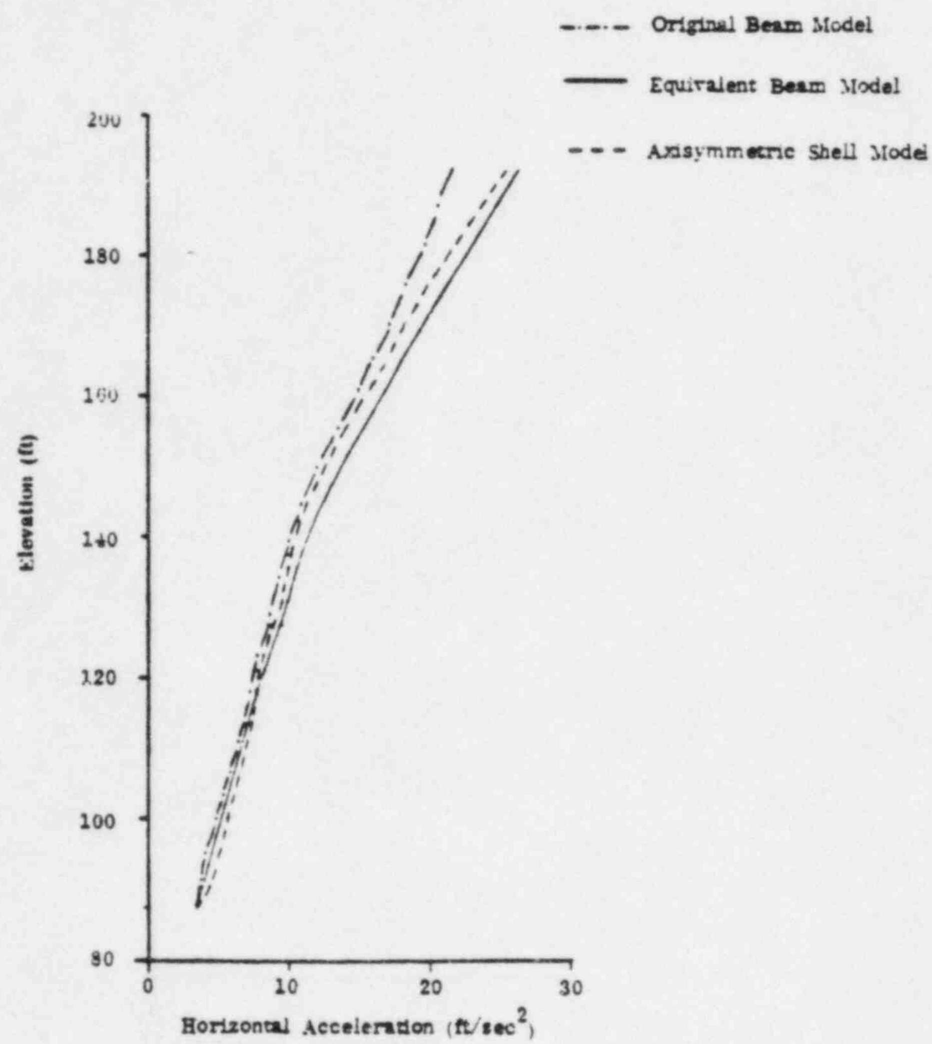
DRYWELL (FREE STANDING)

FIGURE B-230

PUBLIC SERVICE ELECTRIC & GAS CO.	SEISMIC STRUCTURAL ANALYSES	
HOPE CREEK PROJECT		

REGULATORY GUIDE 1.60 RESPONSE SPECTRUM ANALYSIS

MAXIMUM HORIZONTAL ACCELERATION COMPARISON

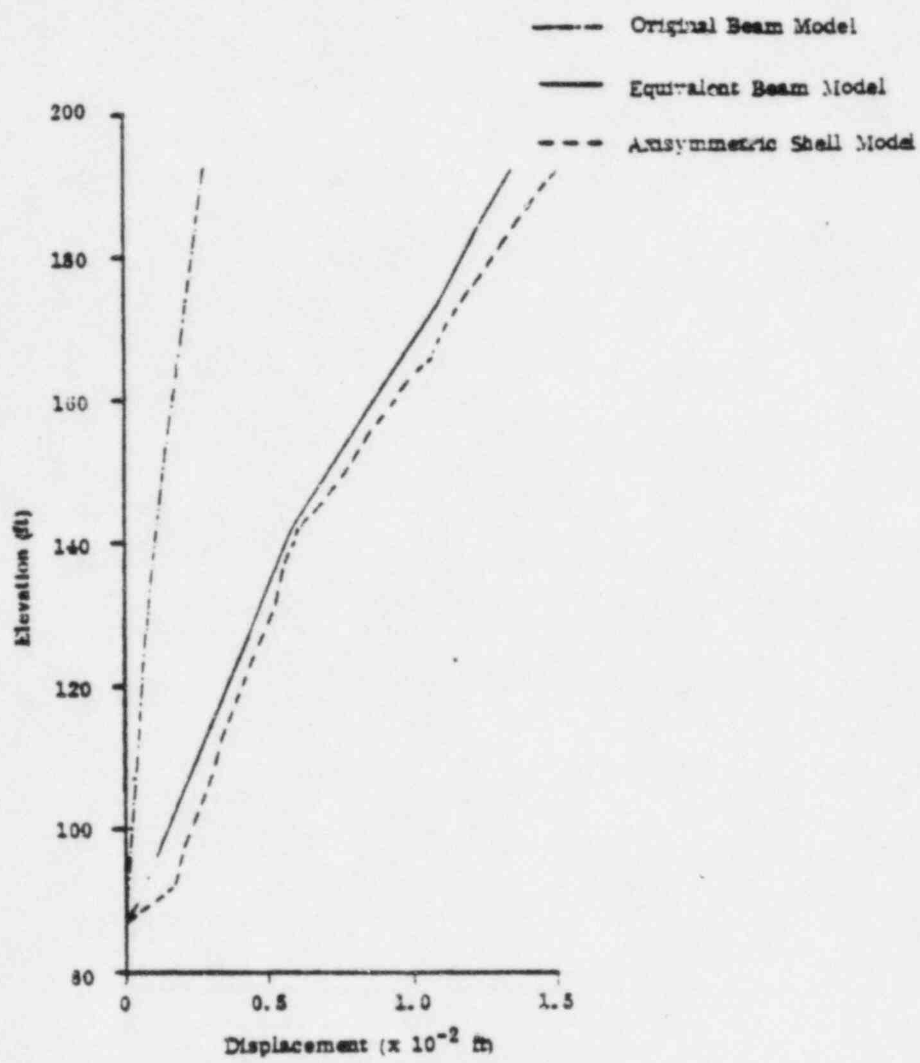


DRYWELL (FREE STANDING)

FIGURE B-231

REGULATORY GUIDE 1.60 RESPONSE SPECTRUM ANALYSIS

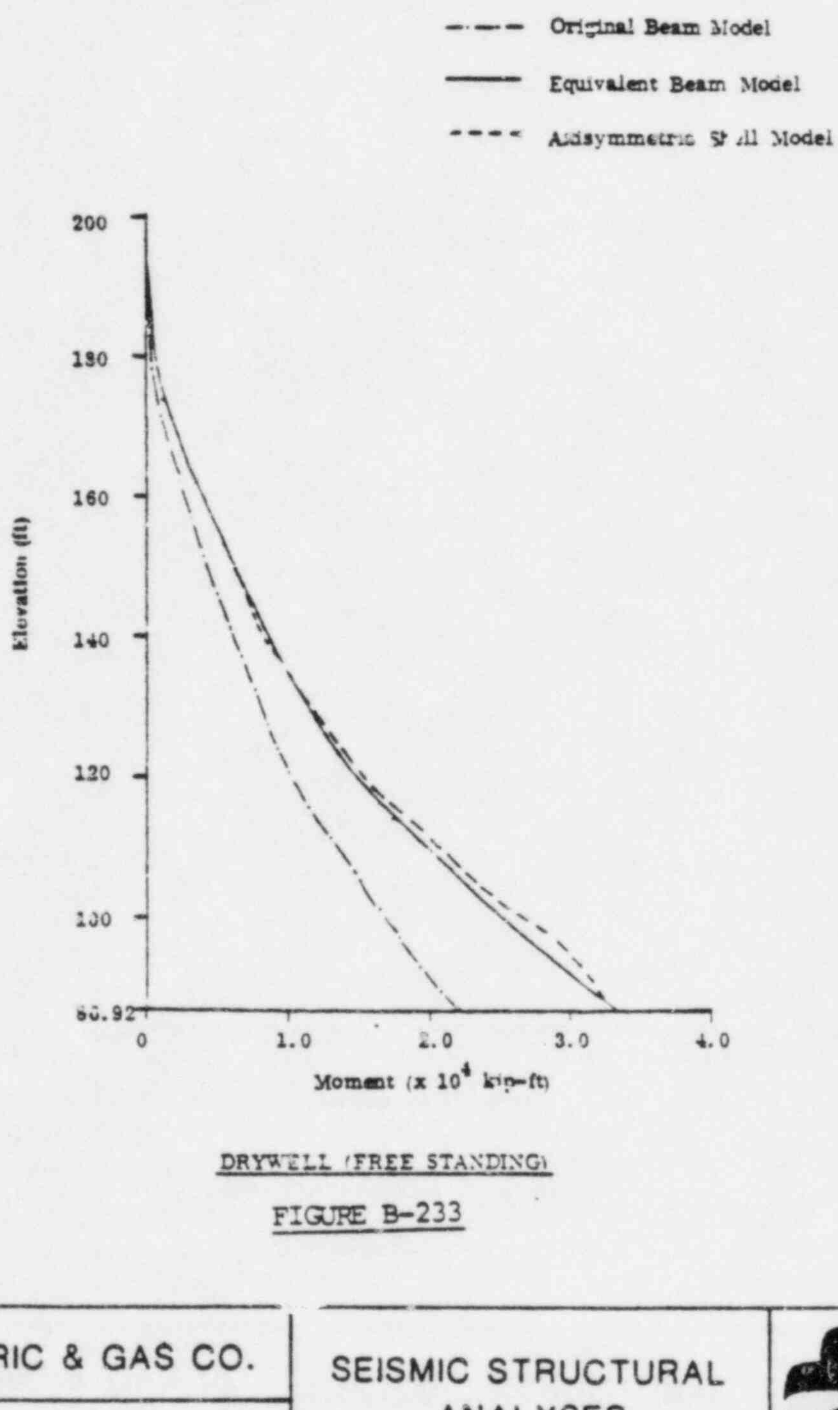
MAXIMUM HORIZONTAL DISPLACEMENT COMPARISON



DRYWELL (FREE STANDING)

FIGURE B-232

REGULATORY GUIDE 1.60 RESPONSE SPECTRUM ANALYSIS
MAXIMUM MOMENT COMPARISON



DRYWELL (FREE STANDING)

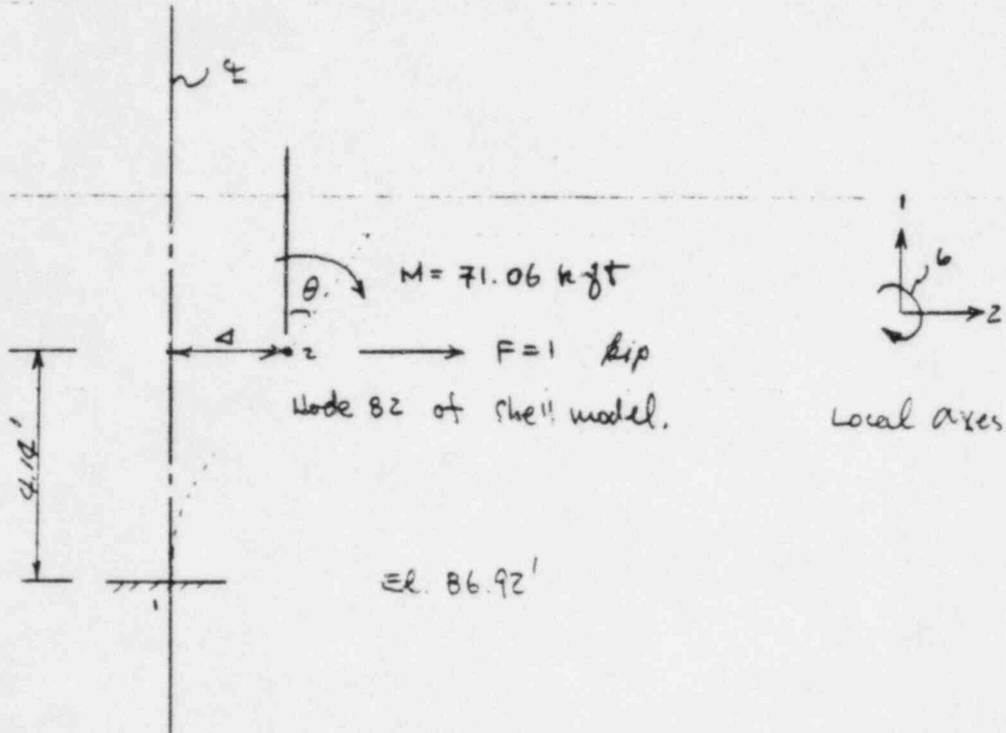
FIGURE B-233

ATTACHMENT II

Development of Equivalent Beam Model Properties

Equivalent Beam Model

Change node 19 coordinate to (0, 0, 91.06).



Δ @ node 82 (90°) = 0.96009×10^{-6} ft (4.14' from drywell base)

Δ @ node 50 (90°) = 0.11921×10^{-4} ft (50.0' from drywell base)

$$\therefore \theta = \frac{0.11921 \times 10^{-4} - 0.96009 \times 10^{-6}}{50 - 4.14} = 2.390 \times 10^{-7} \; *$$

Ref: J.S. Przemieniecki, "Theory of Matrix Structural Analysis", pp79

$$\left.\begin{aligned} S_6 &= \frac{12EI}{l^3(1+\phi)}\Delta - \frac{6EI}{l^2(1+\phi)}\theta \\ S_{12} &= -\frac{6EI}{l^2(1+\phi)}\Delta + \frac{(4+\phi)EI}{l(1+\phi)}\theta \end{aligned}\right\} \quad (1)$$

* if we use 81 & 82, $\theta = \frac{0.11845 \times 10^{-5} - 0.96009 \times 10^{-6}}{5.08 - 4.14} = 2.387 \times 10^{-7}$

82 & 83 $\theta = 2.209 \times 10^{-7}$

ITEM: Drywell Beam Model	CLIENT: PSEG
	PROJECT: Hope Creek
	JOB NO. 140.0012
BY: HH · DATE: 2/10/77 · CKD.: GCH · DATE: 2/11/77	ITEM NO. · SHT. 3 OF 11

$(1)_1 \Rightarrow \quad S_8 = \frac{6EI}{\ell^2(1+\phi)} \left\{ \frac{2}{\ell}\Delta - \theta \right\}$

$$1+\phi = \frac{6EI}{S_8 \ell^2} \left\{ \frac{2}{\ell}\Delta - \theta \right\}$$

or

$$\phi = \frac{6EI}{S_8 \ell^2} \left(\frac{2}{\ell}\Delta - \theta \right) - 1$$

$(1)_2 \Rightarrow \quad S_{12} = -\frac{6EI}{\ell^2(1+\phi)}\Delta + \frac{3EI}{\ell(1+\phi)}\theta + \frac{EI}{\ell}\theta$

$$= -\frac{\ell}{2} S_8 + \frac{EI}{\ell}\theta$$

or

$$I = \frac{\ell}{E\theta} \left(S_{12} + \frac{\ell}{2} S_8 \right)$$

✓

So

$$\left.\begin{aligned} I &= \frac{\ell}{E\theta} \left(S_{12} + \frac{\ell}{2} S_8 \right) \\ \phi &= \frac{6EI}{S_8 \ell^2} \left(\frac{2}{\ell}\Delta - \theta \right) - 1 \end{aligned}\right\} \quad (2)$$

✓

ITEM: Drywell Beam Model	CLIENT: PSEG
	PROJECT: Hope Creek
	JOB NO. 140,0012
BY: HH / DATE: 2/10/77 / CKD. GCH / DATE: 2/11/77	ITEM NO. / SHT. 4 OF 11

EDS NUCLEAR: 220 MONTGOMERY ST. • SAN FRANCISCO, CALIFORNIA 94104

Use $\Delta = 0.960 \times 10^{-6}$ $\theta = 2.35 \times 10^{-7}$

$S_8 = 1.0$ $S_2 = 71.0$

$E = 4.0 \times 10^{c}$ $\nu = 0.3$

Then,

$I = 321.8$

$\Phi = 102.1$

Now,

$$\phi = \frac{12EI}{\ell^2 G A_s} \quad \therefore \quad A_s = \frac{24(1+\nu)I}{\ell^2 \phi}$$

✓

$\therefore$ $A_s = 5.74$

$\therefore$ <u>1st try</u>, $I = 322 \ ft^4$, $A_s = 5.74 \ ft^2$

<u>Check</u>

$$\Phi = \frac{12EI}{\ell^2 G A_s} = \frac{24(1+\nu)I}{\ell^2 A_s} = 102.12$$

✓

$$\therefore \left.\begin{aligned} 0.528(E\Delta) - 1.093(E\theta) &= 1 \\ -1.093(E\Delta) + 80.041(E\theta) &= 71 \end{aligned}\right\}$$

$$\therefore E\Delta = \frac{80.041 + (1.093)(71)}{(0.528)(80.041) - (1.093)^2} = 3.839$$

$$E\theta = \frac{(0.528)(71) + 1.093}{(0.528)(80.041) - (1.093)^2} = 0.939$$

$$\therefore \left.\begin{aligned} \Delta &= 9.597 \times 10^{-7} \\ \theta &= 2.349 \times 10^{-7} \end{aligned}\right\}$$

✓

ITEM: Drywell Beam model				CLIENT: PSEG	
				PROJECT: Hope Creek	
				JOB NO. 140.0012	
BY: HM	DATE: 1/10/77	CKD.: GCH	DATE: 2/11/77	ITEM NO.	SHT. 5 OF 11

Response to NRC Audit

Meeting Date: January 10, 1984

Question No.: B.11

<u>QUESTION</u>: Define location of (x, y, z) "0" reference points for auxiliary building model.

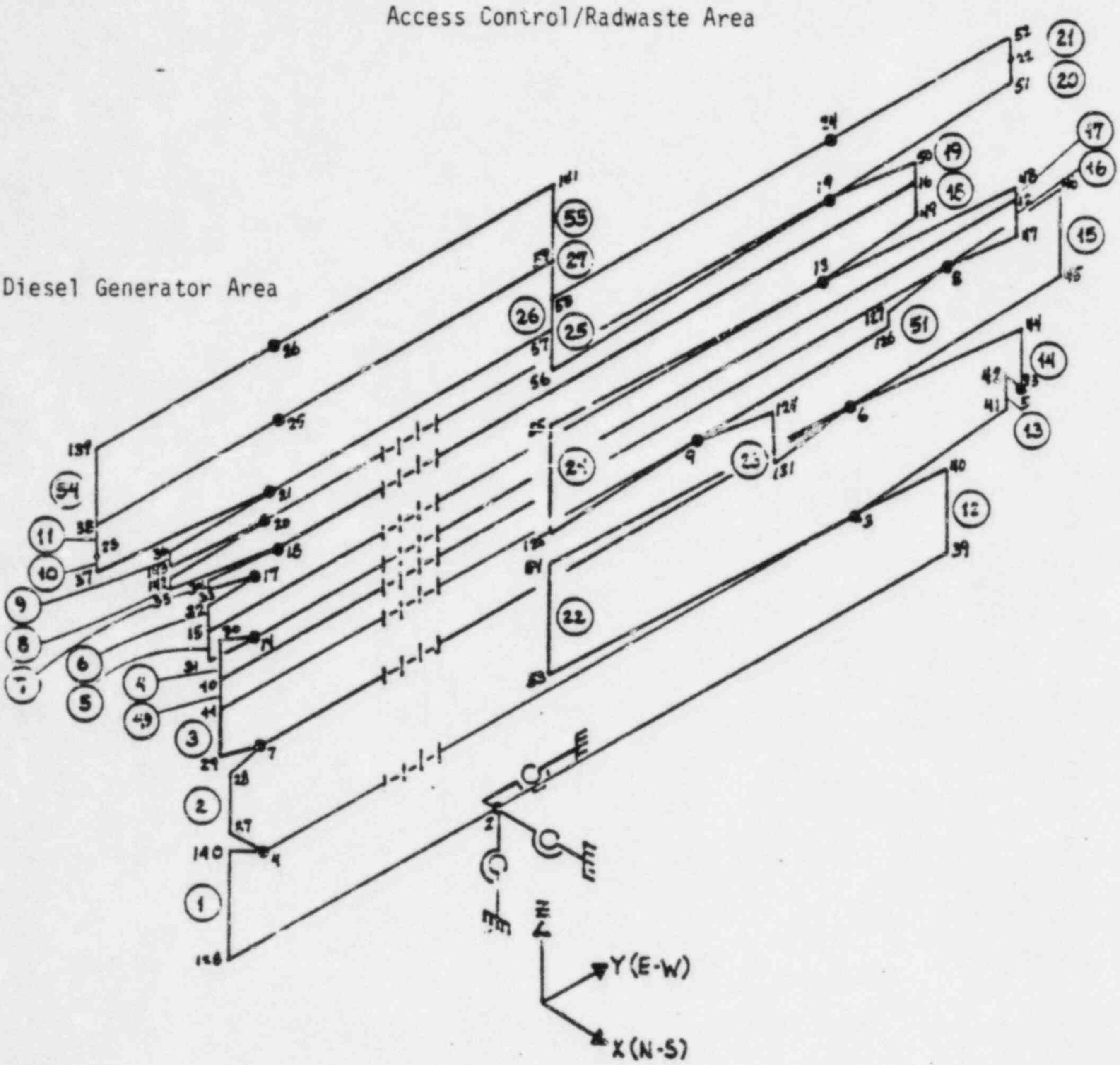
<u>RESPONSE</u>: The attached Figure 1 shows the auxiliary building mathematical model. The auxiliary building plan at the ground elevation (elevation 54.0 ft) is shown in Figure 2.

The reference coordinate system used for the calculations of building properties is shown in Figure 2. Based on this coordinate system the following is calculated:

a. The center of rigidity for the Control/Radwaste area has x,y,z coordinate (in ft.) of (0.84, 114.09, 54.0). This joint corresponds to nodal point 39 in Figure 1.

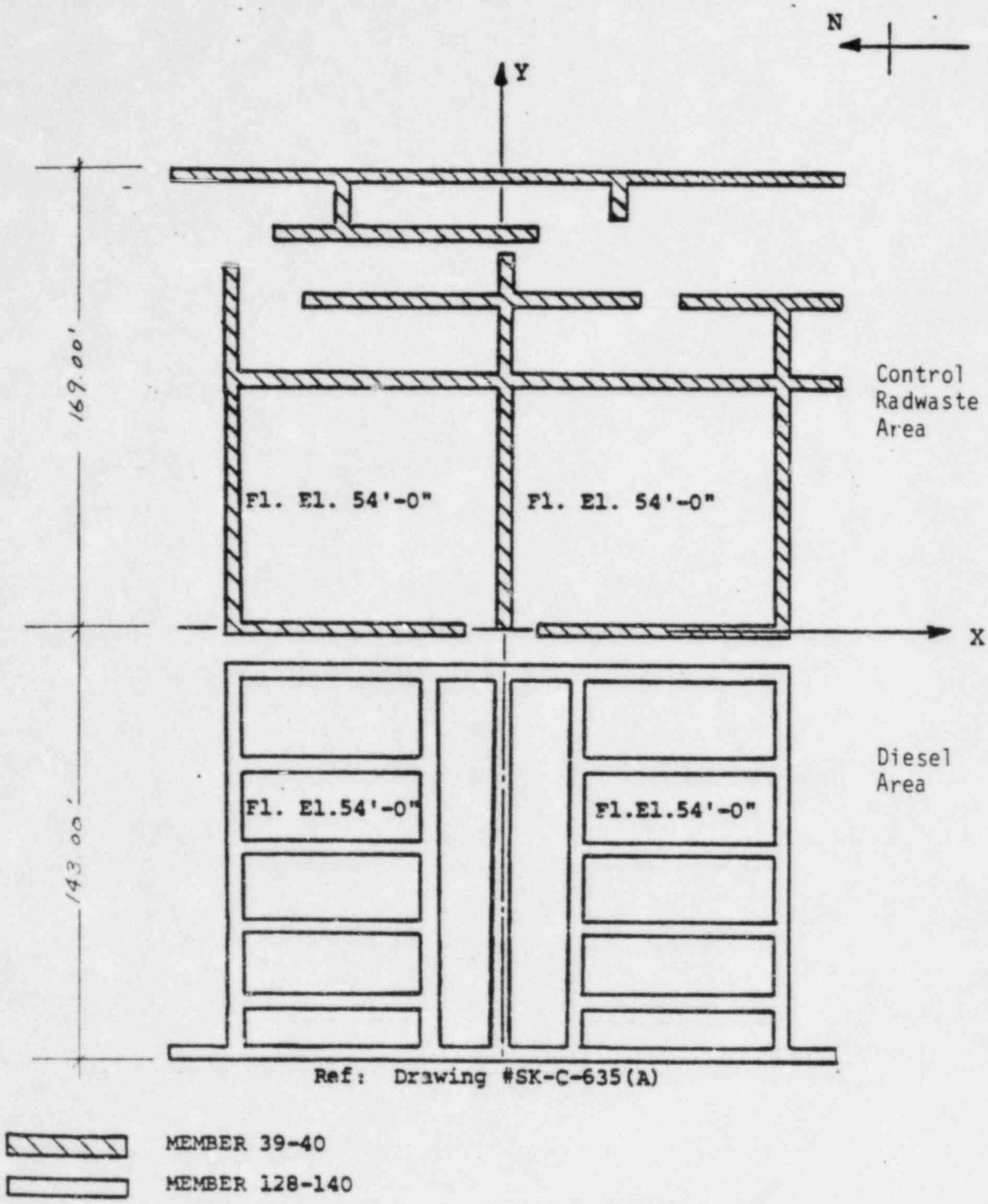
b. The center of rigidity of the Diesel Generator area has x,y,z coordinates (in ft.) of (0.0, -89.70, 54.0). This point corresponds to nodal point 128 in Figure 1.

c. The center of mass of the complete plan system of walls has x,y,z coordinates (in ft.) of (0.0, -13.00, 54.0). This point corresponds to nodal point 2 in Figure 1. This is the node at which the translational and rotational motions are input to the building model.



AUXILIARY BUILDING MATHEMATICAL MODEL

FIGURE 1



AUXILIARY BUILDING PLAN AT ELEVATION 54'-0"

FIGURE 2

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: A.7

QUESTION: Provide a simplified calculation for overturning moment of reactor building foundation mat.

RESPONSE: The factor of safety against overturning for the reactor building as given in FSAR Appendix 3H was based on the energy method approach described in BC-TOP-4A (FSAR Reference 3.7-1). During discussions with the NRC, the NRC requested the factor of safety against overturning be calculated using conventional methods. The factor of safety against overturning using conventional methods is 3.60 which exceeds the minimum safety factor of 1.10 specified by SRP Section 3.8.5-II of NUREG-0800. FSAR Appendix 3H will be revised to indicate the factors of safety against overturning by both the energy method and conventional method.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: A.10

QUESTION: Justify the boundary conditions used at the base of the drywell shield wall.

RESPONSE: The drywell shield wall was modeled using a finite element analysis. The base was assumed to be at el. 83 ft-0 in., since this is the elevation of the junction with the support pedestal.

Calculations and drawings indicate that the reinforcement required at el. 54 ft-0 in. was extended to el. 83 ft-0 in. The reinforcement design was reviewed by Mr. L. Yang of the NRC Staff during the NRC Structural Audit.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: A.11

QUESTION: Justify the boundary conditions used for the analytical model of the reactor building dome.

RESPONSE: As presented to Mr. L. Yang of the NRC Staff, the model boundary is cut off at el. 201 ft. The boundary at el. 201 ft is assumed to be restrained. Our justification is as follows:

- A plot of the deformations under lateral loads shows that most of the displacements die out at the elevation of the corbel (elevation 239 ft), which is well above the cut-off elevation.

- The cylinder wall is continuous below the cut-off elevation to provide for the continuity effect in the vertical (meridional) direction. The cut-off elevation also corresponds to the elevation of the operating floor diaphragm which provides a fixed condition in the radial and circumferential directions.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: A.15

<u>QUESTION</u>: Explain whether a heavy load drop event needs to be considered for the intake structure gantry crane. If so, provide the results of a heavy load drop for the intake structure roof.

<u>RESPONSE</u>: As discussed with Mr. D. Jeng of the NRC Staff, a heavy load drop event need not be considered for the intake structure gantry crane for the reasons stated in HCGS FSAR paragraph 9.1.5.3.3hh.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: A.17

QUESTION: Provide a typical calculation for the knuckle region of the reactor building dome using critical loads.

RESPONSE: As presented to Mr. L. Yang of the NRC Staff, the area of transition from the cylindrical to spherical portion of the dome (knuckle region) sustains the highest moments and axial forces. In the meridional direction, the maximum factored moment of 5375 ft-lbs/ft is combined with 32,580 lbs of axial compression. Meridional reinforcement of No. 7 bars spaced at 18 in. is provided on both faces. The circumferential reinforcement of No. 9 bars at 18 in. on each face is based on the maximum factored tension of 5320 lbs/ft and moment of 1401 ft-lbs/ft.

Response to NRC Audit

Meeting Date: January 11, 1984

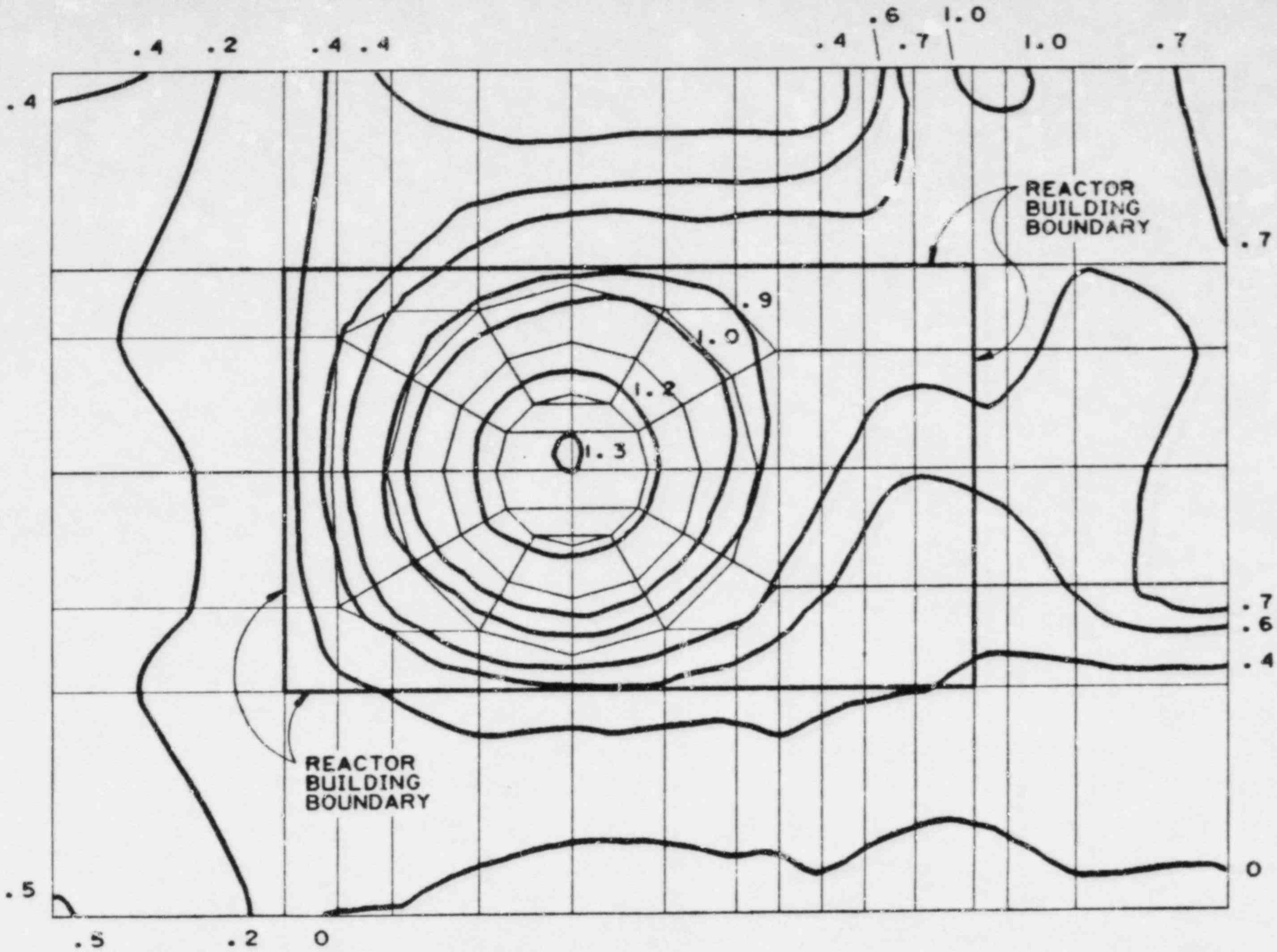
Question No.: B.1

QUESTION: Provide the deflected shape and the soil contact pressures for the reactor building foundation mat (if contact pressures are non-uniform).

RESPONSE: As indicated in FSAR Appendix 3D, Load Combination $1.4D + 1.7L + 1.0F_1$ (F_1 is load due to post-accident containment flooding) governs the flexural reinforcing requirements for the reactor building basemat. This load combination also governs the maximum deflected shape of the reactor building foundation. As shown in Sketch B.1-1, the maximum deflection occurs under the drywell pedestal and radiates outward in an expected dish shape pattern.

Load combination D + L governs the soil contact pressures for the reactor building foundation. As shown in Sketch B.1-2, the maximum soil contact pressure is 8.21 ksf (compression) and does not exceed the estimated allowable static bearing capacity of 12 ksf for the Vincentown Formation as discussed in FSAR Section 2.5.4.10. This maximum contact pressure distribution is assumed to occur during the construction stage when all dead and live loads are applied and the site dewatering system is not turned off.

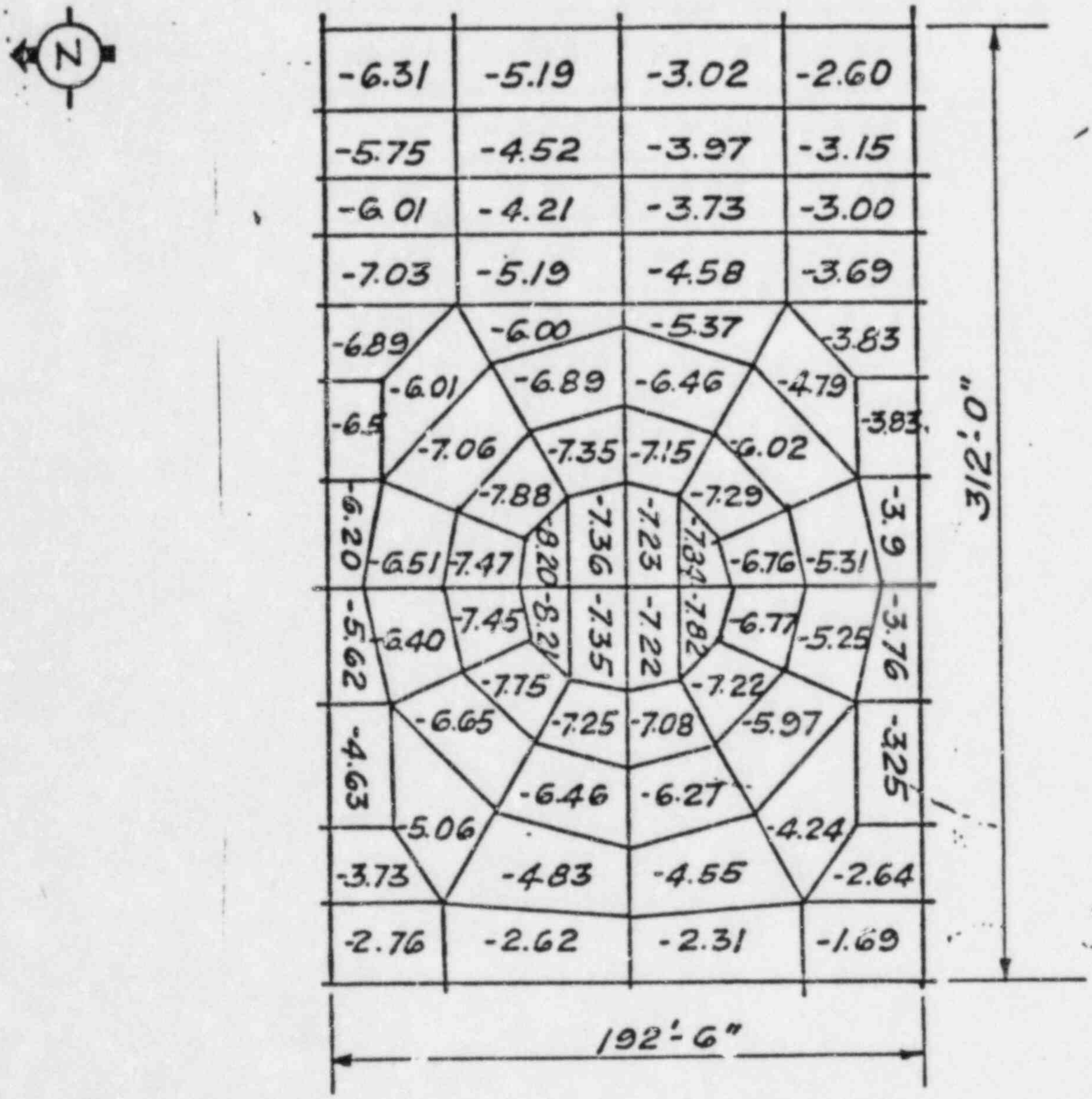
The above was discussed with Mr. L. Yang of the NRC during the NRC Structural Audit.



REACTOR BUILDING FOUNDATION DEFLECTED SHAPE (INCHES)

FOR LOAD COMBINATION 1.4D + 1.7L + F_1

SKETCH B. 1-1



REACTOR BUILDING FOUNDATION SOIL CONTACT PRESSURES (KSF)

FOR LOAD COMBINATION D + L

("-" indicates compression)

SKETCH B.1-2

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: B.2

QUESTION: Provide calculations for the factors of safety against sliding and overturning for OBE and SSE cases for the drywell shield wall.

RESPONSE: The maximum overturning moments at the base of the drywell shield wall are:

410,000 k-ft (OBE case)
820,000 k-ft (SSE case)

The factors of safety against overturning are:

F.S. = 6.42 > 1.5 (OBE case)
F.S. = 2.73 > 1.1 (SSE case)

The maximum shears at the base of the drywell shield wall are:

4580 kips (OBE case)
6380 kips (SSE case)

The factors of safety against sliding are:

F.S. = 3.79 > 1.5 (OBE case)
F.S. = 2.72 > 1.1 (SSE case)

Calculations for the factors of safety against sliding and overturning for OBE and SSE cases for the drywell shield wall were reviewed by Mr. L. Yang of the NRC Staff during the NRC Structural Audit.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: B.3

QUESTION: Provide calculations of seismic shear force distribution in the cylinder wall.

RESPONSE: The seismic shear forces are distributed to the shear walls in proportion to the relative rigidities of the shear walls.

The total seismic shear forces at the base of the reactor building shell are:

Condition	Direction	Shear (kips)
OBE	North-South	27,400
OBE	East-West	30,500
SSE	North-South	48,000
SSE	East-West	71,000

The seismic shear forces at the base of the cylinder wall and the percentages of the total reactor building seismic shear forces are:

Condition	Direction	Shear (kips)	Percentage of Total Shear
OBE	North-South	6,356	23
OBE	East-West	6,482	21
SSE	North-South	11,134	23
SSE	East-West	15,090	21

It should be noted that the values represent the results of the analysis performed prior to the cancellation of Unit 2. The calculations incorporating the result of the Unit 2 cancellation study are in process.

Calculations of seismic shear force distribution were reviewed by Mr. L. Yang of the NRC Staff during the NRC Structural Audit.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: B.4

QUESTION: Provide calculations for overturning of the cylinder wall.

RESPONSE: The maximum overturning moments at the base of the cylinder wall are:

1.792×10^6 k-ft (OBE case)
3.05×10^6 k-ft (SSE case)

The factors of safety against overturning are:

F.S. = 3.8 > 1.5 (OBE case)
F.S. = 2.1 > 1.1 (SSE case)

Calculations for overturning of the cylinder wall were reviewed by Mr. L. Yang of the NRC Staff during the NRC Structural Audit.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: B.5

QUESTION: Explain the assumptions used for tha deep beam design of the fuel pool walls.

RESPONSE: The design of the fuel pool walls is in conformance with the Portland Cement Association (PCA) publication, "Design of Deep Beams". This method considers the nonlinear stress distribution along the height of the wall. The stresses and tensile forces were determined from the appropriate design curves in the PCA publication as a function of the height to span ratio of the wall. The reinforcing steel was designed to resist the tensile forces.

Calculations and drawings of the fuel pool walls were reviewed by Mr. L. Yang of the NRC Staff during the NRC Structural Audit.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: B.6

<u>QUESTION</u>: Explain the method of inputting the loads into the ASHSD dome model.

<u>RESPONSE</u>: As presented to Mr. L. Yang of the NRC Staff, the lateral seismic loads were input into the computer analysis employing a partial Fourier Series representation of the load. A uniform lateral load was applied by combining the second harmonic of the radial and tangential loads.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: B.7

QUESTION: Provide calculations demonstrating that the dynamic effects of tornado depressurization are not governing.

RESPONSE: As presented to Mr. L. Yang of the NRC Staff, the computer analysis is performed in the following manner:

- Dead loads, live loads, seismic loads and tornado depressurization loads are applied to the model individually.
- For each individual loading the shears, moments and axial forces are established for each element.
- The above information is then input into a post processor routine which combines and factors the individual loads into the potentially controlling load combinations.
- The resultant moments an forces are generated for each element.
- To simplify the interpretation of the output, the routine selects the shears, axial forces, and moments from the loading combination that yields either the maximum or minimum value in each direction for each element.
- These maximum and minimum values are then used for the design.

Accordingly, the controlling elements may have loads and moments in various directions conservatively combined from more than one loading combination. Therefore, it is not possible, nor is it necessary to confirm that the effects of tornado depressurization are not governing.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: B.8

QUESTION: Provide worst case calculation of auxiliary building room subject to abnormal pressure.

RESPONSE: A calculation of the design of an auxiliary building room subject to abnormal pressure was shown to Mr. D. Jeng of the NRC during the structural audit.

The following areas of the calculation were reviewed:

- Calculation of forces and moments due to abnormal pressure
- Combination of forces and moments due to abnormal pressure with forces and moments due to other loads
- Determination of reinforcing steel requirements.

All areas of the calculation were described to the satisfaction of Mr. Jeng.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: B.9

<u>QUESTION</u>: Provide maximum tangential shear stresses in the drywell shield wall and the cylinder wall.

<u>RESPONSE</u>: The maximum tangential shear stresses are:

136.2 k/ft for drywell shield wall
148.1 k/ft for cylinder wall

The shear calculations were reviewed by Mr. D. Jeng of the NRC Staff during the NRC Structural Audit.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: B.12

QUESTION: Provide static factor of safety against overturning for intake structure.

RESPONSE: The factor of safety against overturning for the intake structure as given in FSAR Appendix 3G was based on the energy method approach described in BC-TOP-4A (FSAR Reference 3.7-1). During discussions with the NRC, the NRC requested the factor of safety against overturning be calculated using conventional methods. The factor of safety against overturning using conventional methods is 1.12 which exceeds the minimum safety factor of 1.10 specified by SRP Section 3.8.5-II of NUREG-0800. FSAR Appendix 3G will be revised to indicate the factors of safety against overturning by both the energy method and conventional method.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: B.13

QUESTION: Provide two or three calculations illustrating use of 0.9 dead load factors.

RESPONSE: Three calculations illustrating the use of the 0.9 dead load factor for the reactor building were reviewed by Mr. L. Yang of the NRC Staff during the the NRC Structural Audit. Similar calculations exist for other Seismic Category I structures.

Response to NRC Audit

Meeting Date: January 12, 1984

Question No: A.1

Question: Describe the procedures which assure that the post-modification seismic loads for the torus were examined and that the torus structure was found to be adequate to resist the post-modification seismic loads.

Response: The evaluation of post-modification seismic loads for the torus was separated into two parts: An evaluation for horizontal loads and an evaluation for vertical loads. The support design for the torus, i.e. pinned-pinned vertical columns and pinned lateral restraints, assures that horizontal and vertical behavior are uncoupled, thus allowing consideration of them separately. This was confirmed by the results of the seismic analysis of the unmodified structure, which also show that responses in each of the horizontal and vertical directions are dominated by one structural mode.

For horizontal loads, an evaluation was made of the effects of the torus modifications on the horizontal seismic analysis for the unmodified configuration. It was concluded that the effect of the torus modifications on the horizontal seismic response of the torus is negligible. The modifications added to the torus consist mainly of local column connection stiffening which does not significantly change the dominant horizontal torus frequency. The original analysis for horizontal loads is conservative, since the stiffening effect, though insignificant, would tend to increase the dominant frequency, resulting in lower accelerations applied to the torus because of the position of the frequency on the response spectrum curve. The evaluation described above was performed as part of the Hope Creek Plant Unique Analysis.

For vertical loads, a new analysis was performed using a finite element model of the modified torus. The results of this analysis are documented in the Hope Creek Plant Unique Analysis Report.

Response to NRC Audit

Meeting Date: January 12, 1984

Question No: A.2

Question: Justify the methods used to consider fluid structure interaction effects for analysis of the torus.

Response: Fluid structure interaction effects are considered in the torus analysis through the use of a fluid added mass formulation which results in a "consistent" mass matrix, i.e. a mass matrix having off-diagonal terms, representing the mathematical coupling of the fluid. Fluid compressibility and surface gravity effects are neglected. It is also assumed that there is no irrotational flow (viscosity), no steady flow (aerodynamics) and no non-linear effects (cavitation).

The assumption of an incompressible fluid is appropriate if structural frequencies of interest are not in the range of either fluid sloshing frequencies or fluid accoustic frequencies. Fluid sloshing frequencies for the Hope Creek torus are less than 1 hz. Accoustic frequencies of the torus are obtained by dividing the velocity of sound in water by a characteristic length. Using the maximum water depth in the torus as the characteristic length, gives a corresponding accoustic frequency of over 50 hertz.

The dominant structural frequencies of the torus for the loadings defined in NUREG-0661 range from 15 hertz to 35 hertz. They are sufficiently far removed from the sloshing and acoustic frequencies of the fluid, so as to justify the assumption of an incompressible fluid.

Response to NRC Audit

Meeting Date: January 12, 1984

Question No: A.3

<u>Question</u>: Describe how the effects of relative seismic displacements of the torus were considered for the stress evaluation of the vent system.

<u>Response</u>: The effects of relative seismic displacements of the torus on the vent system were considered by approximating the maximum relative support displacements using the floor response spectra. The maximum displacement for each support is predicted by $Sd = Sag/w^2$, where Sa is the spectral acceleration in g's at the high frequency end of the spectrum curve, g is the gravity constant, and w is the fundamental structural frequency of the torus in radians per second.

The resulting displacements from this calculation, 0.01 in. in the horizontal direction and 0.017 in. in the vertical direction, are small compared with those of other major vent system loadings such as SRV discharge and pool swell and have a neglible effect on the vent system.