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SEISMIC REVIEW OF THE BIG ROCK POINT NUCLEAR POWER PLANT, CONTAINMENT SHELL STRUCTURE, AS PART OF THE SYSTEMATIC EVALUATION PROGRAM

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1. SUMMARY

As part of the safety assessments of Big Rock Point Nuclear Power Plant facilities, the containment shell structure of Big Rock Point was analyzed using site-specific ground-response spectra with peak ground accelerations of 0.11 g in the horizontal direction and 0.07 g in the vertical direction. The preliminary results of the seismic and dead-load analyses are included in this report. Results of this independent evaluation were compared with the licensee's analyses. Similiar conclusions were obtained which include the following:

a. Stresses in the shell structure due to seismic load are much lower than the allowable tensile stress of the steel.

b. The containment structure has sufficient margin of safety against overturning, sliding, and twisting.

c. The problem of elastic stability of the shell is, as expected, the more critical factor under seismic consideration. However, sufficient safety margins are available to resist the combined compressive stresses induced by seismic and dead loads.

d. It is important to point out that these conclusions were drawn from the study of dead and seismic loads only and no other loads, such as design basis accident (DBA), were considered.

2. ANALYSIS OF THE CONTAINMENT-SHELL STRUCTURE

The objective of this analysis is to perform an independent seismic evaluation of the containment-shell structure using site-specific spectra of the Big Rock Point site. The spectra were based on those recommended in Reference 1 with peak ground accelerations of 0.11 g in the horizontal directions and 0.07 g in the vertical direction.

In accordance with the intent of the Systematic Evaluation Program (SEP), the structural review is not based on demonstrating compliance with specific criteria in the Standard Review Plan or Regulatory Guides, but rather the seismic resistance of the structure is compared qualitatively to the intent of today's licensing criteria in order to determine acceptable levels of safety and reliability.

2.1 DESCRIPTION OF STRUCTURE

The containment-shell structure,[2] which is part of the reactor building, is a spherical steel shell, 130 ft in diameter and having a thickness varying from 0.702 in. to 0.774 in. The containment shell encloses the reinforced-concrete internal structure which houses the nuclear steam supply system (NSSS), spent-fuel storage pool and emergency condenser, as shown in Figure 1.

The lower portion of the steel containment is embedded in the concrete foundation of the internal structure. The foundation has the shape of an inverted spherical segment approximately 7-ft thick. To provide smooth transition of the supporting edge where the steel shell is embedded in the concrete foundation, an 8-ft-deep sand-filled cavity was constructed around the edge of the foundation.

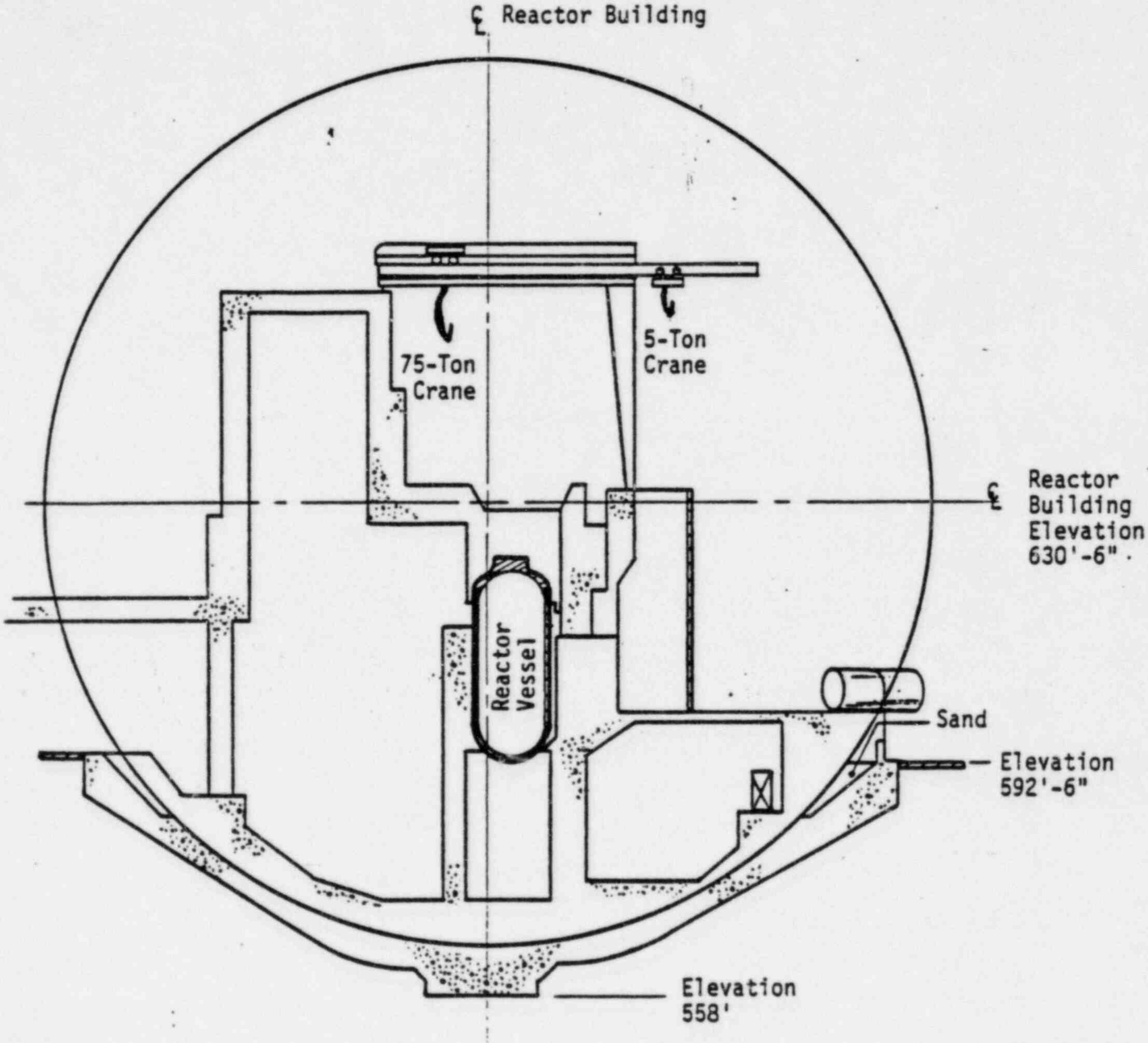


Figure 1. Containment building.

The Big Rock Point site is situated in a limestone area. The soil foundation of the reactor building can be idealized as being composed of a layer of very dense glacial till on top of several layers of limestone.[3]

2.2 MATHEMATICAL MODEL

The major concern of this analysis is the structural integrity of the containment shell under seismic conditions. However, since the shell structure and the internal structure share a common foundation on soil, the structure-to-structure effect through soil structure interaction is an important consideration. The shell and the massive internal structure cannot be separated into two independent structures. Therefore, the modeling approach adopted in this analysis is to include a simplified model of the internal structures in the same model with a detailed representation of the shell to calculate the shell's seismic and dead-load responses. The internal structure was modeled in sufficient detail to include its interaction effects with the shell.

The containment shell is an axisymmetric structure, but the internal structure is asymmetric with an eccentric mass and stiffness distribution which may cause significant torsional responses under seismic loads. It was therefore decided to model the coupled shell-internal structure as a three-dimensional structure without utilizing the benefit of the shell symmetry.

In determining a proper representation of the shell structure, the fact that under seismic and dead loads, the stress in this kind of shell is normally very low, well below the yield stress of the material, was considered. The compressive membrane stress under elastic stability criteria is a more critical consideration. Therefore, a three-dimensional shell element mesh was constructed with elements sized to capture mainly membrane behavior. A uniform shell thickness of 3/4 in. was assumed for the model used in the analysis. When the information about the exact shell thickness was available later, a new model incorporating these thicknesses was constructed and a confirmatory eigen-value analysis was performed. The discrepancy between the two models in terms of modal frequencies was less than 2%; therefore, the original model based on 3/4-in. thickness was considered to be adequate.

As mentioned previously, the idea of modeling the internal structure is to capture its interaction effects on the shell. Lumped masses connected by beam elements are adequate for the purpose of including the mass and stiffness effects. The properties of the internal structure are based on information available in Reference 3.

The foundation of the reactor building is an inverted spherical concrete dome, approximately 7-ft thick, embedded in the soil. The concrete foundation is modeled as a rigid disk connected to the shell around the edge and to the internal structure at the center at elevation 584.5 ft. Six springs representing the three translational and three rotational degrees of freedom of the concrete foundation are attached to the rigid disk to simulate soil-structure interaction. The spring constants and the associated radiation damping coefficients were estimated based on an elastic half space assumption for the soil. The damping values for horizontal and vertical directions were taken as 75% of the theoretical values in keeping with the recommendations of the Senior Seismic Review Team.[4]

The sand cushion around the shell edge was modeled by equivalent elastic springs in the direction normal to the shell surface. The equivalent spring constants were calculated from the data given in Figure A1-7 of Reference 3.

The structural damping ratios were assumed to be 2% and 3% for the steel shell and concrete internal structure, respectively. These values are those suggested in NUREG/CR-0098[5] for welded assemblies and reinforced-concrete structures subjected to stresses below one-half the yield point.

Figure 2 shows the mathematical model for the coupled shell and internal structure.

Also, according to Reference 4, three different soil modulus conditions are to be considered in the structural analysis because of uncertainty in soil properties. The suggested three conditions are: 1) a best-estimate large-strain shear modulus, 2) 50% of the modulus corresponding to the best estimate of the large-strain condition, and 3) 90% of the modulus corresponding to the best estimate of the low-strain condition.

The subsurface conditions of the Big Rock Site may be idealized as being composed of approximately 30 to 40 ft of medium dense to very dense glacial till on top of several layers of limestone.[3] The glacial till has a shear modulus of 14.2×10^6 psf, based on a shear-wave velocity survey.[3] The limestone layers have moduli varying from 79.6×10^6 psf to 138.5×10^6 psf, between the elevations of 553 ft and 413 ft. The surface grade is at elevation 593 ft.

To properly represent the soil-structure interaction effect in the model, several approaches were considered. The first approach used the spring constants and the associated radiation damping coefficients based on an elastic half-space assumption for the soil.[6] A weighted average shear modulus according to the thickness of soil layers was used to represent the layer structure which included one layer of glacial till and three layers of limestone. The shear modulus for low strain was estimated to be 68.2×10^6 psf for this case. For the large-strain condition case ($\gamma = 5 \times 10^{-5}$), a reduction factor of 82% was estimated for hard glacial till[7] which gave a shear modulus of 55.8×10^6 psf. The corresponding 50% best estimate of the large-strain case had a shear modulus of 27.9×10^6 psf.

The second approach was based on the method suggested in Reference 8, which gave equivalent shear modulus of elastic half-space solution for circular footing embedded in a layer of soil on top of the bedrock. In this case the three layers of limestone are relatively stiff and can be considered to be the bedrock. The top layer of glacial till is treated as

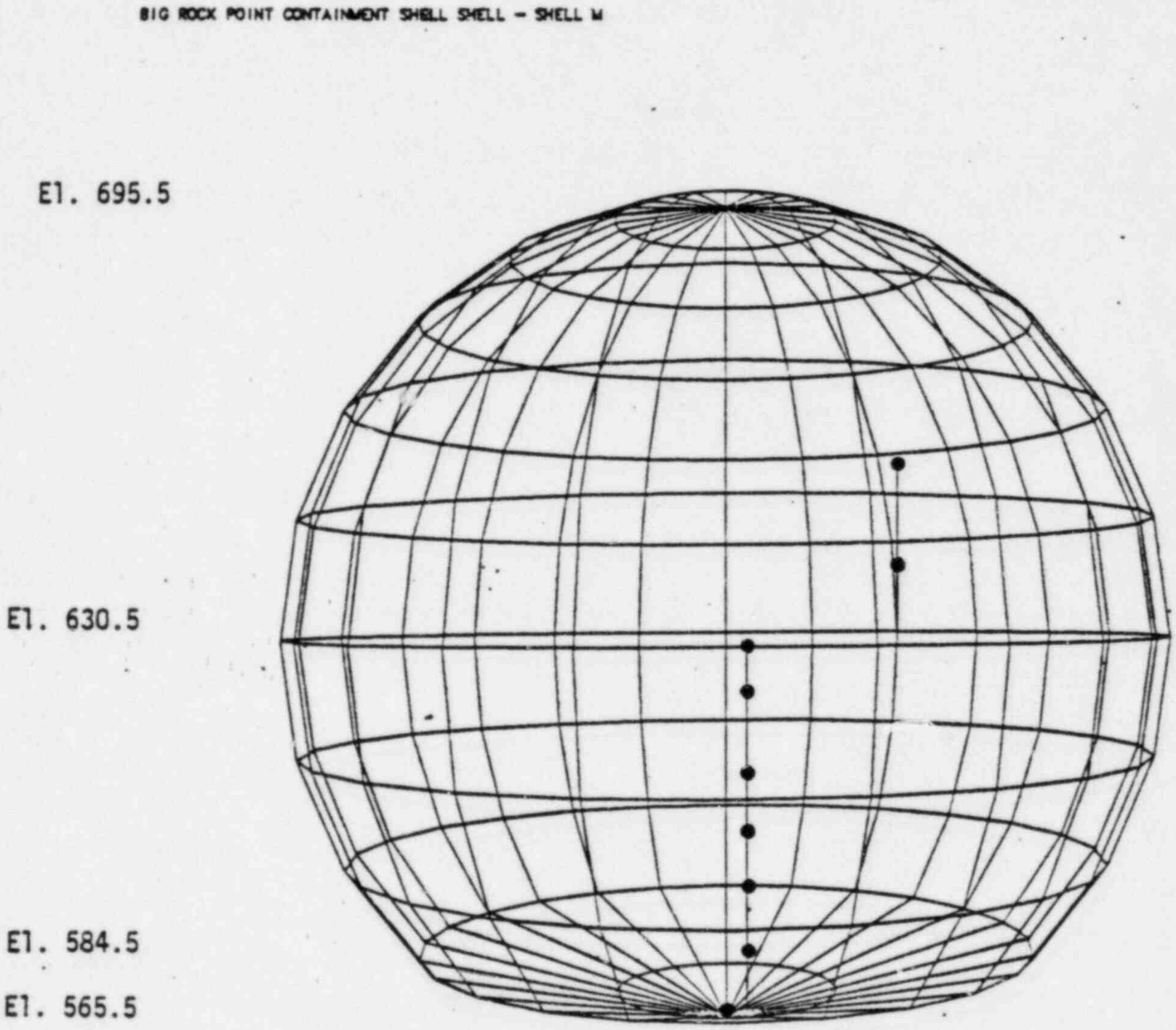


Figure 2. Mathematical model for Big Rock Point containment shell and reactor building.

the soil layer. The equivalent low-strain shear modulus for the elastic half-space medium in this case was calculated to be 49.7×10^6 psf. The corresponding large-strain and 50% large-strain cases then have shear moduli of 40.8×10^6 psf and 20.4×10^6 psf, respectively.

From the above two approaches, a set of three shear moduli were selected. Their values and the corresponding spring constants and damping ratios together with mass density and Poisson's ratio are listed in Table 1. Case A has the lowest shear modulus while Case B has the highest shear modulus of the two approaches described above. A shear modulus in between Case A and Case B is selected as equal to 34.1×10^6 psf for the third case, Case C. The soil spring constants and damping values were calculated based on the elastic half-space theory.

Another approach, suggested in Reference 9, gave the spring constants. The horizontal spring constant was calculated to be 1.13×10^6 k/in. which is close to Case B, the rocking spring constant was 1.53×10^6 kin./rad which is close to Case C.

Based on the above discussion on these different approaches, the values listed in Table 1 can be considered as a reasonable bound for the soil springs. Therefore, three models were constructed using the listed values.

2.3 METHOD OF ANALYSIS

The computer code used for the analysis is a version of SAPIV modified by Lawrence Livermore National Laboratory. The seismic responses were computed by the response-spectrum method. The undamped natural frequencies and mode shapes of the soil-structure system were calculated, then the composite-modal-damping ratios of each mode were computed using the stiffness-proportion damping method. Site-specific spectra based on the computed composite-modal-damping ratio were input in two horizontal and the vertical directions. Table 2 lists the horizontal spectral values at

Table 1. Properties of soil springs.

CASE	A	B	C
G (psf)	20.4×10^6	68.2×10^6	34.1×10^6
SPRING CONSTANTS			
Vertical (k/in.)	0.57×10^6	1.90×10^6	0.95×10^6
Horizontal (k/in.)	0.40×10^6	1.35×10^6	0.68×10^6
Rocking (kin./rad)	1.16×10^{11}	3.86×10^{11}	1.93×10^{11}
Torsion (kin./rad)	1.27×10^{11}	4.25×10^{11}	2.12×10^{11}
DAMPING RATIOS (%)			
Vertical	57	57	57
Horizontal	33	33	33
Rocking	11	11	11
Torsion	9	9	9

SOIL MASS DENSITY: 2.18×10^{-7} lb-sec^2/in.^{4}

POISSON'S RATIO: 0.45

5% damping for the site-specific spectrum of the Big Rock Site. The spectral values at different frequencies for different damping values were calculated according to the formula given in Reference 1. The vertical spectral value was taken to be two-thirds of the horizontal value. The peak ground acceleration was 0.11 g for the horizontal directions and 0.07 g for the vertical direction. The modal responses for each direction were combined by the square-root-of-sum-of-squares (SRSS) method. The total responses of the structure were obtained by combining the absolute values of responses to each input direction with combination factors of 1.0 for the major direction and 0.4 for the other two directions as suggested in Reference 5. The major direction (i.e., the one with the factor 1.0) was input in each of the two horizontal directions (which are perpendicular to one another) and the vertical direction with respect to the structure, and the combination with higher response was used in the final results.

The above analytical procedure was repeated for the three cases with different soil-shear moduli.

2.4 RESULTS

Twenty modes for each of the three soil cases were extracted and included in the seismic analyses. The modal frequencies and the composite modal damping ratios of the first ten modes and the twentieth mode are presented in Table 3. The modal damping values were reduced to 20% if the calculated value exceeds that limit in accordance with SSRT's guideline on soil-structure interaction.[4] Among the three models which have different soil-shear modulus, the first five modes are very similar. The first mode is an internal-structure mode, with the major response occurring at the steam-drum enclosure. The second and third modes, which have nearly the same frequencies are containment-shell modes in two orthogonal directions. The fourth and fifth modes, also with very close frequencies, are combined shell and internal-structure modes. The sixth mode is a vertical mode and the seventh mode is a torsional mode of the shell structure. Several typical mode shapes of Model B are shown in Figures 3 through 7.

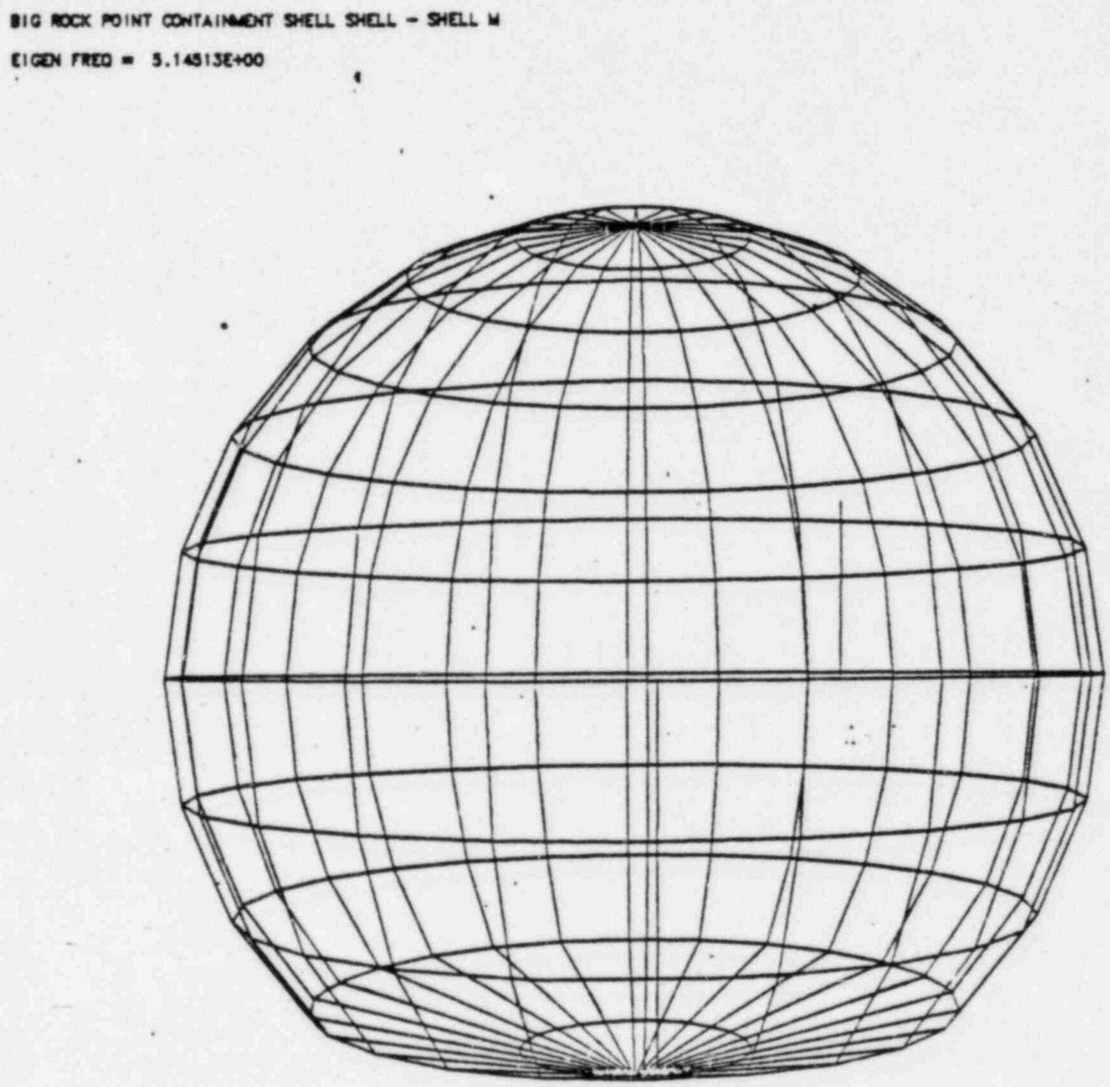


Figure 3. Mode shape--first mode, Model B.

BIG ROCK POINT CONTAINMENT SHELL SHELL - SHELL M

EIGEN FREQ = 7.97496E+00

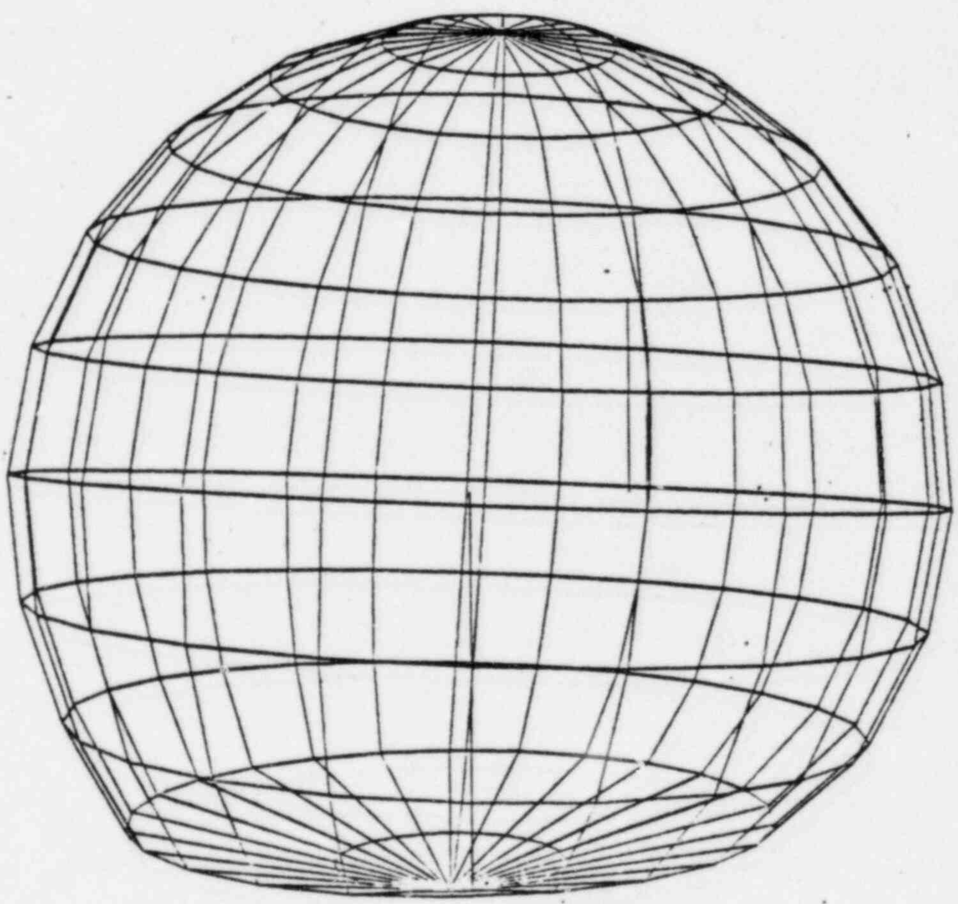


Figure 4. Mode shape--second mode, Model B.

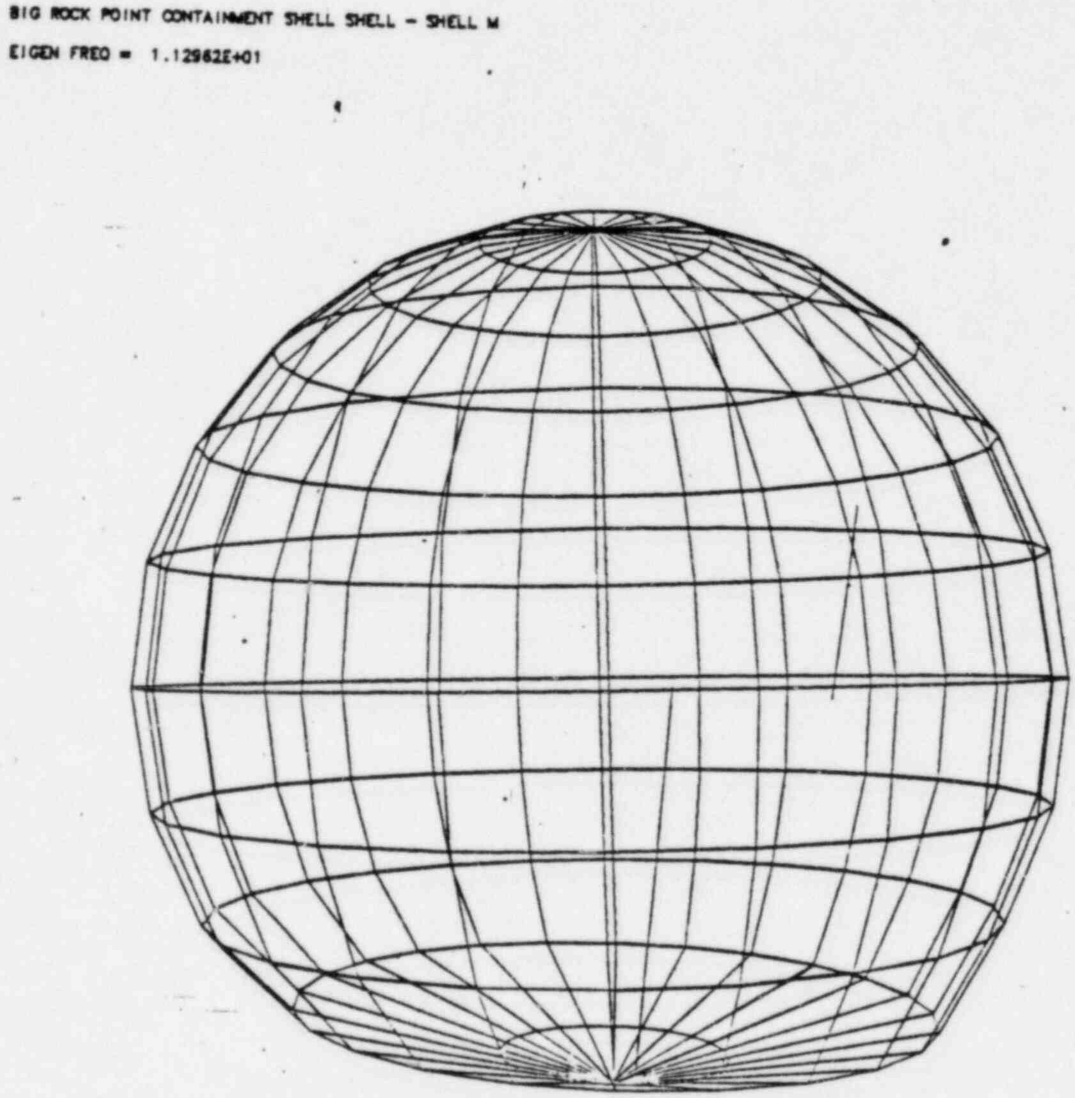


Figure 5. Mode shape--fourth mode, Model B.

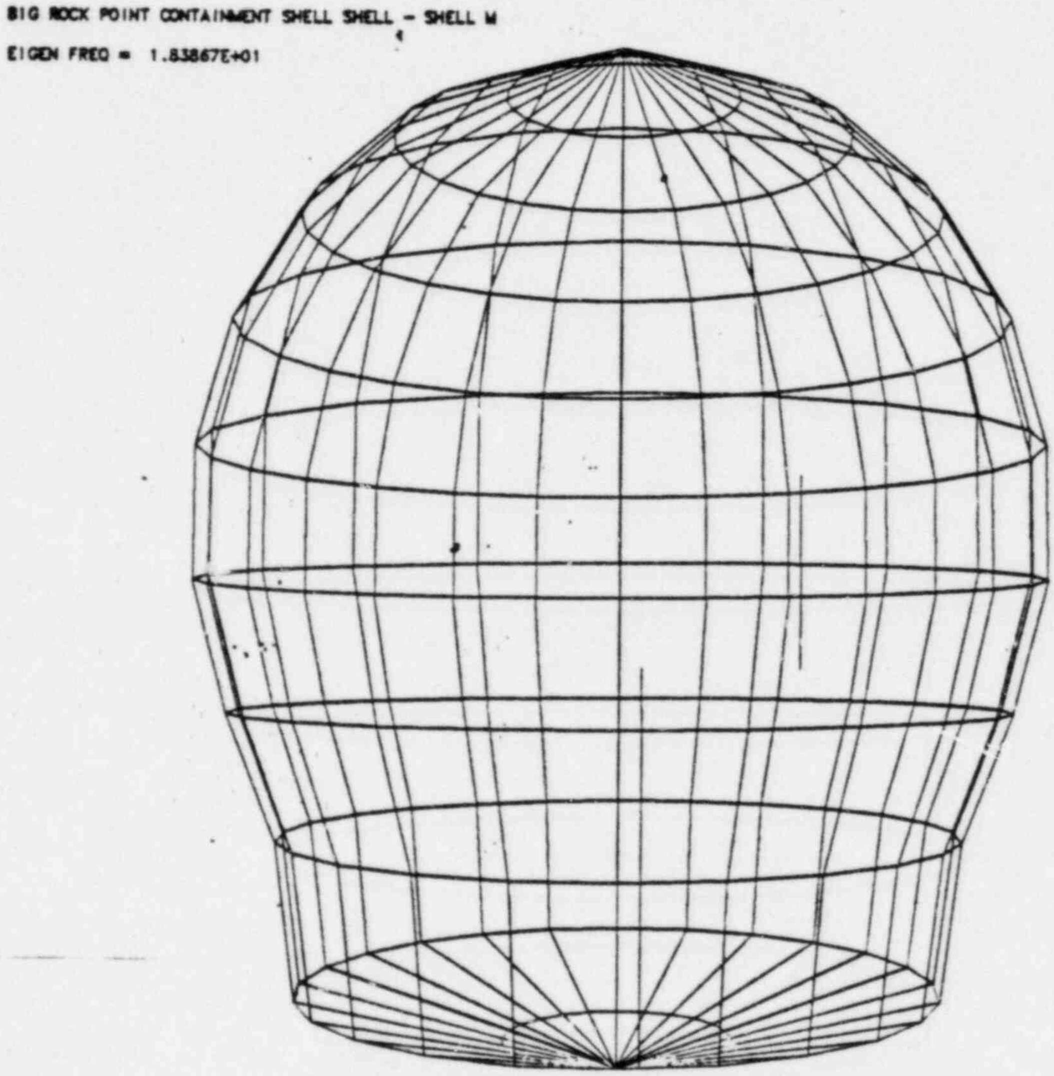
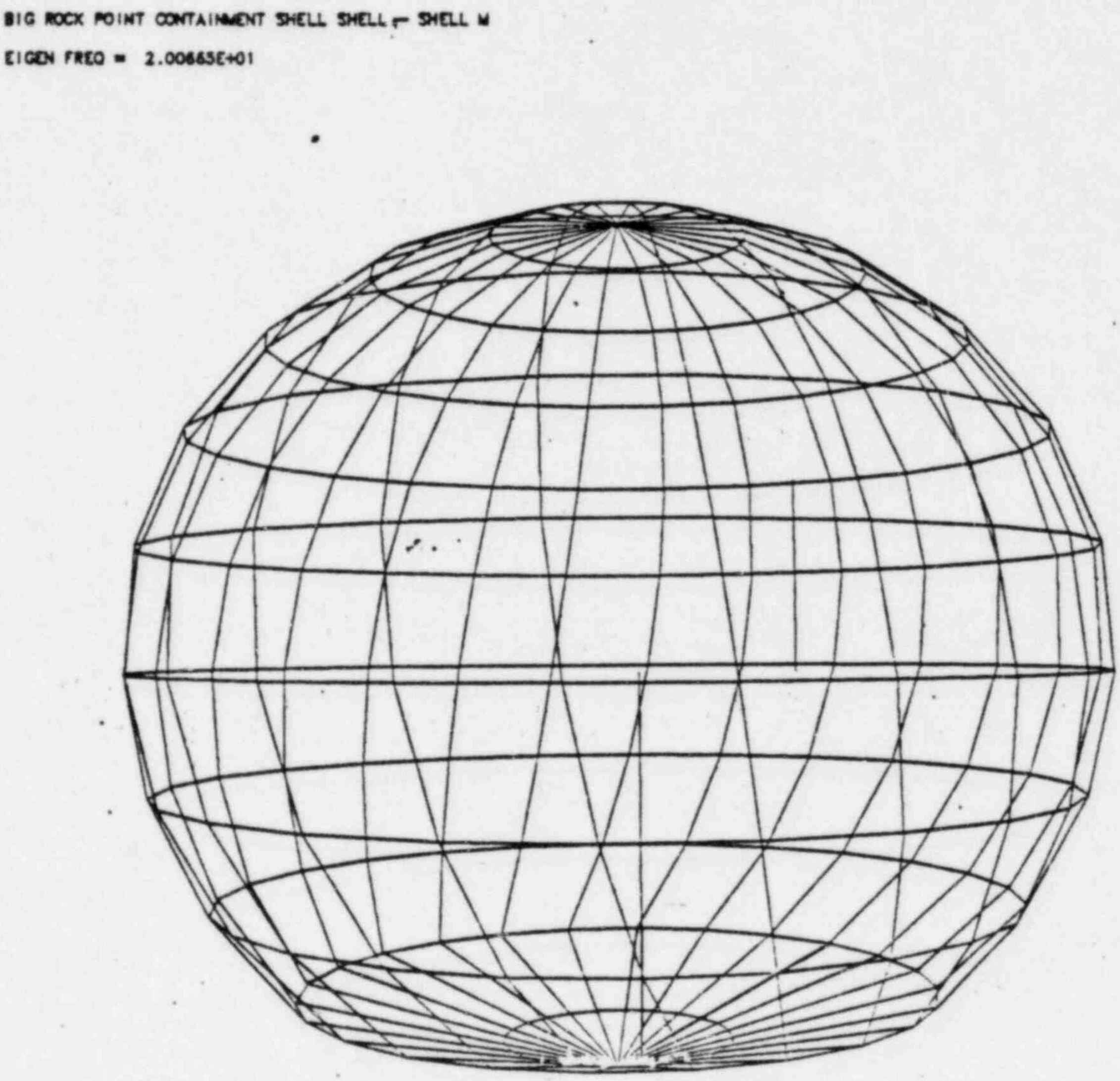


Figure 6. Mode shape--sixth mode, Model B.



'Figure 7. Mode shape--seventh mode, Model B.

Table 2. Horizontal site-specific spectrum.

PERIOD	PSEUDO SPECTRAL ACCELERATION (cm/sec^2) (5% DAMPING)
0.03	102.50
0.04	122.29
0.05	130.19
0.08	152.05
0.10	179.69
0.20	213.50
0.30	201.96
0.40	171.68
1.00	122.90

Table 3. Modal frequencies and composite-modal-damping ratios.

MODE	MODEL A		MODEL B		MODEL C	
	FREQ. (Hz)	DAMPING RATIO(%)	FREQ. (Hz)	DAMPING RATIO (%)	FREQ. (Hz)	DAMPING RATIO (%)
1	5.00	4	5.15	3	5.08	3
2	7.13	13	7.98	2	7.52	3
3	7.22	11	7.98	2	7.54	3
4	8.85	6	11.30	11	9.60	14
5	8.96	8	11.71	11	9.85	14
6	12.51	20	18.39	10	15.48	20
7	15.91	17	20.07	3	18.34	8
8	16.48	20	21.83	20	19.06	11
9	16.56	12	24.47	16	20.00	17
10	19.58	5	25.26	17	20.36	20
20	32.07	3	32.14	6	32.12	5

The maximum shell stresses due to seismic and dead loads are listed in Table 4. The maximum stresses are quite uniform in the hoop direction. In the meridional direction, they have typical distribution as shown in Figure 8 for Model B.

Table 4. Maximum membrane and shear (psi) stresses in the steel shell.

	SEISMIC LOAD			DEAD LOAD
	MODEL			
STRESS (psi)	A	B	C	
Hoop	±341	±246	±455	193
Meridional	±687	±457	±744	-471
Shear	±733	±469	±789	0

The maximum combined seismic and dead-load stresses in the shell are in the order of 650 psi for tension, 1200 psi for compression, and 800 psi for shear. They are all very low compared to the material-yield stress. Therefore, the only critical condition for an Safe-Shutdown Earthquake event is either compressive or torsional shell buckling.

There are very few generally accepted buckling criteria for a sphere under seismic loads. Reference 10 gives a critical load on a thin truncated conical shell under axial load,

$$F = 0.277\ (2\ Et^2 \cos^2 \alpha)$$

which is based on 170 tests and will give 95% confidence in at least 90% of the cones carrying more than this critical load. If applied to this shell, the formula gives an equivalent uniform meridional compressive stress of 5460 psi.

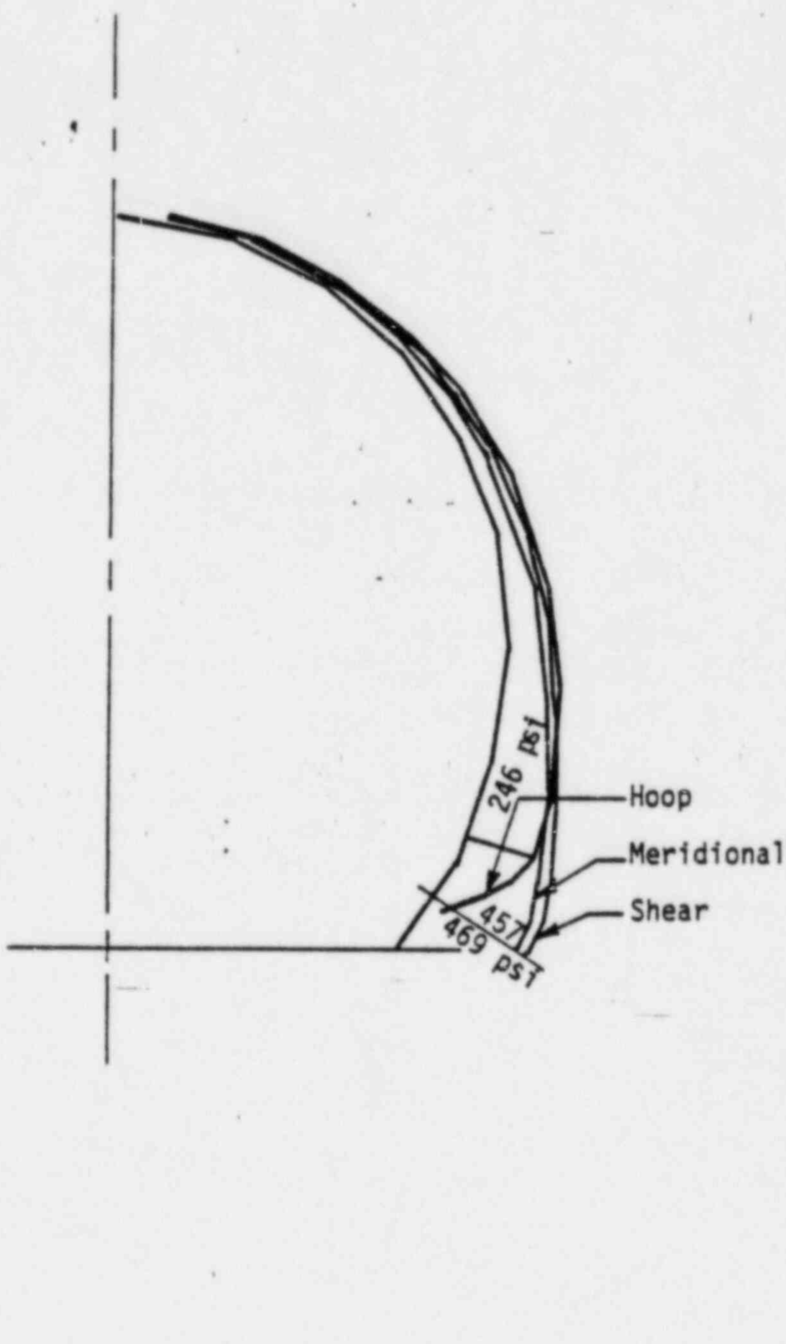


Figure 8. Stress distribution in the meridional direction, Model B.

Reference 11 suggests a critical pressure for a sphere under vacuum.

$$P = P_{cl} \left[0.14 + 3.2/\lambda^2\right] \qquad \lambda > 2$$

$$\lambda = \left[12(1-\nu^2)\right]^{1/4} (R/t)^{1/2} 2\sin \phi/2$$

$$P_{cl} = 2/\left[3(1-\nu^2)\right]^{1/2} E(t/R)^2$$

In this case, P = 4.55 psi which gives an allowable shell compressive stress of 2364 psi.

The two allowable compressive stresses (5460 psi and 2364 psi) above are for the case where the shell is under uniform compression. Under seismic excitation, only a portion of the shell will be subjected to compression. These criteria, if applied to this analysis, are very conservative. Therefore, the shell is considered to have a factor of safety at least 1.9 (2364 psi/1200 psi) against buckling.

For torsional buckling, Reference 10 gives a formula which is applicable to a thin truncated conical shell under torsion. If it is applied to this case, the critical shear stress is 4780 psi, which would result in a safety factor of 6 since the maximum sheer stress is 789 psi. Again, this is a conservative estimate.

The overall stability of the shell structure is also of concern during an SSE. From the response-spectrum analysis, the maximum overturning moment, torsional moment, and sliding force can be obtained from the soil spring forces. From the structural dead weight, the vertical seismic force, and an assumed friction coefficient of 0.45 (Reference 3), the resisting moment and forces can be calculated. The resulting factor of safety from these calculations is 7 for overturning and torsion, and 4 for sliding.

3. COMPARISON WITH LICENSEE'S ANALYSIS

The major differences between the licensee's analysis[3] for the containment structure and this analysis are listed in Table 5. The licensee's analysis used the R.G.1.60 spectra with higher peak ground and spectral accelerations than the site-specific spectrum.

The licensee's model for the containment structure included two parts: 1) a concrete internal structure with soil springs and a single lumped mass for the shell, and 2) a finite-element shell model for the containment shell without the internal structure and soil springs. The analysis was performed in two steps. First, Model 1 was analyzed by the time-history method using an artificial earthquake generated from the R.G. 1.60 spectrum, including soil-structure interaction. Second, the floor-response spectra developed from Model 1 at the shell base were input to Model 2, and a response-spectrum analysis of Model 2 was performed. The responses of Model 2 were used as the basis for structural evaluation. However, it is not clear how the effects of the base rocking and torsional motion were included in the response-spectrum analysis of Model 2. In estimating the soil-spring coefficients, the licensee's analysis considered the embedment effect of the foundation and included some correction due to the frequency dependent characteristics of the soil springs. However, only one set of spring constants were used in the licensee's analysis. The radiation damping values of soil were also reduced by a factor of one-half in the licensee's analysis. In this analysis, the theoretical damping values for translational motions were reduced to 75%.

Both analyses predict frequencies of the dominant shell modes at about 8 Hz. The membrane seismic stresses from both analyses are in proportion to their spectral values, but the licensee's analysis predicted slightly higher bending stress because of their more refined finite-element model. However, the critical condition of the shell is controlled by its elastic stability which depends on the shell-membrane stress.

Table 5. Differences between the EG&G and Licensee's analysis.

	EG&G ANALYSIS SITE-SPECIFIC SPECTRUM	LICENSEE'S ANALYSIS R.G. 1.60 SPECTRUM
INPUT		
Peak Ground Acceleration	0.11 g (H) 0.07 g (H)	0.12 g (H) 0.08 g (V)
MODEL		
Structure Model	ONE MODEL 3-D shell elements for the shell. Lumped mass-stick beam model for the internal structure.	TWO MODELS 1. Single lumped mass for the shell. Lumped mass-stick beam model for the internal structure. 2. Refined axisymmetric shell model.
Soil Spring	No embedment effect. Variation of shear modulus.	Embedment and frequency dependent effects, damping reduction. No variation of shear modulus.
MATERIAL DAMPING		
Steel	2%	4%
Concrete	3%	7%
METHOD OF ANALYSIS	Response-spectrum analysis (RSA).	Modal time-history method for Model 1. RSA for Model 2.

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