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SUBJECT:
RESPONSE TO REQUEST FOR ADDITIONAL INFO. ON DESIGN CRITERIA FOR STRUCTURES, COMPONENTS, EQUIPMENT, & SYSTEMS INCLUDED IN AMEND. NO. 50.

PLANT NAME: DIABLO CANYON - UNIT 1
DIABLO CANYON - UNIT 2

REVIEWER INITIAL: XRL
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February 2, 1978

RECEIVED FEB 7, 1978 U.S. NUCLEAR REGULATORY COMMISSION Mail Section

Mr. John F. Stolz, Chief
Light Water Reactors Branch No. 1
Division of Project Management
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Re: Docket No. 50-275-OL
Docket No. 50-323-OL
Diablo Canyon Units 1 & 2

Dear Mr. Stolz:

The enclosure to your letter dated August 24, 1977 included a series of questions covering additional information on design criteria for structures, components, equipment, and systems included in Amendment No. 50. Enclosed are 20 copies of the answer to Question 3.67A(d). The answer was inadvertently omitted from our response dated November 2, 1977. Also enclosed are updated and expanded responses to Questions 3.54, 3.67A, and 3.98.

Kindly acknowledge receipt of the above material on the enclosed copy of this letter and return it to me in the enclosed addressed envelope.

Very truly yours,

Philip A. Crane, Jr.

Enclosures
CC w/enc.: Service List

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3.67A On Page 4-9, Section 4.2.1, please clarify the following:

(d) How were the torsional stiffnesses in the coupled analysis computed?

Response:

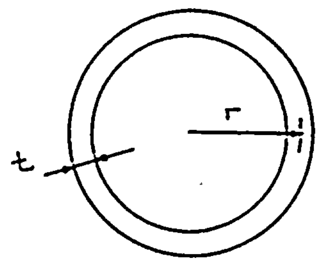
The torsional stiffness of a segment of the containment was computed on the basis of the following equations:

I Containment Exterior Shell

$$J_h = 2\pi r^3 t \quad (1)$$

$$\phi = \int_{h_1}^{h_2} \frac{T}{J_h G}\, dh \quad (2)$$

$$K_T = \frac{T}{\phi} \quad (3)$$



where

K_T = torsional stiffness

G = shear modulus

ϕ = torsional rotation due to torque T

h = length of segment considered

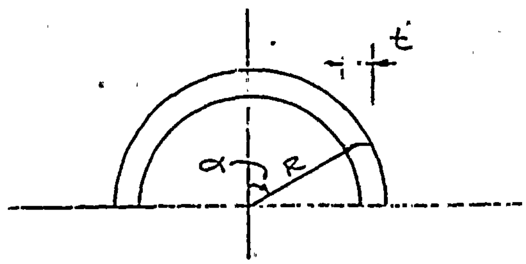
In the cylindrical portion of the shell, equation (3) becomes

$$K_T = \frac{GJ}{h}$$

In the dome section equation (2) becomes

$$\phi = \int_{\alpha_1}^{\alpha_2} - \frac{TR\sin\alpha d\alpha}{2\pi(R^3\sin^3\alpha)\,\frac{2.5}{\sin\alpha}\,G}\,d\alpha$$

where α and R are as shown below:



II Containment Interior

The torsional stiffness of the interior shell is the same as for the containment exterior shell. The contribution of the walls is as follows:

$$(K_T)_{wall} = K_\theta + K_{TR}\, d^2$$

where

K_θ = torsional stiffness of wall about its own axis $\frac{GJ}{h}$

$$K_{TR} = \frac{1}{1/K_b + 1/K_s}$$

$$K_b = \frac{12EI}{h^3}$$

$$K_s = \frac{A_{sh}G}{h}$$

d = distance between center of rigidity of Containment Interior and wall in question

h = height of wall

I = moment of inertia of wall

A_{sh} = shear area of wall

J = polar moment of inertia of wall

G = shear modulus

E = modulus of elasticity

3:54 On Page 2-6, Par. 1, two statements are made that "this combination was obtained by summing the vertical and the most critical horizontal loads within each mode. These summed horizontal. . . . . by the square root sum of the squares (SRSS) method." Since the natural frequencies and mode shapes of the structures are not the same in separate horizontal and vertical directions, the responses resulting from these analyses should not be combined on a modal basis. Indicate if this same technique was used in the re-analysis. If so, justification should be provided.

Response:

The statements on Page 2-6 to which this question refers are most descriptive of the method used to evaluate the NSSS for the DDE. The techniques used to analyze structures, systems, and components for the postulated Hosgri event are explained in detail in Chapter 4 through 12 of the Hosgri Evaluation Report. It is believed that the information in these chapters, particularly Chapter 4 for structures, provides the answer to this question for the Hosgri evaluation.

For the evaluation of the Diablo Canyon major structures for the DDE, individual responses to one horizontal and one vertical component were combined on an absolute sum basis. For the evaluation of the structures for the Hosgri earthquake, individual responses to two horizontal and one component were combined on the SRSS basis.

3.98 In addition to the shears included in Table 4-11, provide the shear forces in the dome shell due to the vertical input. In what directions are the shears in the shell computed and how are they combined with the net shear forces on the wall.

Response:

Calcuation of shear forces in the dome due to the vertical input is possible using the data provided in the analysis. These forces would then be combined with dead and live load forces to obtain the total shear forces in the dome.

Vertical input produces only out of plane (radial) shear, but no membrane shear. Calculated values of radial shear are:

Node	El.	Shear Force	Shear Stress
2	301.64	0.77 k/ft	2.00 PSI
8	274.37	0.50	1.30 PSI
10	258.27	0.65	1.69 PSI
14	231.00	0.68	1.29 PSI

These shears are negligible when compared to membrane shears:

Node	Membrane Shear Force
2	6.85 k/ft
8	44.80
10	63.15
14	101.00

3.67A

The question was raised as to whether the finite element model of the containment structure (Fig. 4.24) used to compute responses due to the horizontal component of the ground motion is equivalent to the lumped mass model (Fig. 4.26) used to account for the 5% eccentricity. The two models are considered equivalent based on the dynamic characteristics of the two models and the absolute horizontal acceleration responses of the two models due to the same horizontal ground motion as is shown in the following tables:

Containment Exterior Structure

Mode No.	F. E. Model (Fig. 4.24) Period (sec)	F. E. Model (Fig. 4.24) % Participation Factor	Lumped Mass Model (Fig. 4.26) Period (sec)	Lumped Mass Model (Fig. 4.26) % Participation Factor
1	.225	44.97	.217	58.85
2	.081	22.75	.109	0.32
3	.053	3.7	.074	25.86
4	.053	7.88	.041	12.35
5	.047	1.96	.039	5.63
6	.045	0.43		

Notice that the second mode of the lumped mass model is predominantly a torsional mode and has a very small participation factor. The third mode of the lumped mass model is predominantly a transitional mode and should be compared with the second mode of the finite element model. Based on this table the dynamic characteristics of the two models show good agreement.

Containment Exterior Structure
Maximum Absolute Horizontal Acceleration

Node No.	F. E. Model (Fig. 4.24) Acceleration (g)	Lumped Mass Model (Fig. 4.26) Acceleration (g)
2	2.2	2.4
8	2.0	2.2
10	1.9	1.9
14	1.6	1.7
17	1.4	1.5
19	1.2	1.2
20	1.0	1.0
22	0.8	0.9
23	0.7	0.8

The responses in the above table were for the Blume-Hosgri, $\tau = 0.04$ time history and the two models show good agreement.

The above question was also raised regarding the Auxiliary Building. Following is a comparison of the dynamic characteristics and the horizontal response of Auxiliary Building model (Fig. 3.108) in the N-S direction with and without accidental eccentricity. In the analysis presented in Amendment 50, accidental eccentricities were added to the geometric eccentricities in the

model. This model is shown to be equivalent to the model where the geometric eccentricities alone are included in the model.

Auxiliary Building
Dynamic Characteristics in the N-S Direction

Mode No.	Model used in Amendment 50 (Fig. 4.108) Period	Model used in Amendment 50 (Fig. 4.108) % Participation Factor	Model using Geometric Eccentricities Only Period	Model using Geometric Eccentricities Only % Participation Factor
1	.454	6.48	.454	6.71
2	.356	0.05	.356	0.02
3	.099	27.53	.096	21.01
4	.084	36.69	.088	42.60
5	.043	17.96	.042	18.44

Auxiliary Building
Maximum Absolute Horizontal Acceleration

Node No.	Model used in Amendment 50 Acceleration (g)	Model using Geometric Eccentricities Only Acceleration (g)
1	1.5	1.4
2	1.0	1.1
3	0.8	0.9
4	0.7	0.7
6	1.6	1.6

The above responses were computed for the Blume-Hosgri horizontal ground motion (τ = .052). The dynamic characteristics and responses of the two models show good agreement.

In our response to question 3.67A (b), a comparison was presented for the Containment Exterior Structure between the torsion due to the accidental eccentricity as computed in our analysis and the torsion as computed by the procedure outlined in the question. Following are two tables giving this comparison for the containment Exterior [This table is same as presented in our reponse to question 3.67A (b)] and the Auxiliary Building.

Containment Exterior Structure

Nodal Point	Elevation (ft)	Torsion as Computed in Amendment 50 (k-ft) x 10^3	Torsion as Computed by Procedure Outlined in Question 3.67A (b) (k-ft) x 10^3
2	301.64	5.49	0.32
8	274.37	67.61	59.33
10	258.27	136.94	104.89
14	231.00	216.36	169.95

Containment Exterior Structure (cont'd)

Nodal Point	Elevation (ft)	Torsion as Computed in Amendment 50 (k-ft) x 10^3	Torsion as Computed by Procedure Outlined in Question 3.67A (b) (k-ft) x 10^3
17	205.58	293.17	233.28
19	181.08	353.32	283.18
20	155.83	400.00	325.90
22	130.58	427.78	356.85
23	109.67	439.99	395.26

Auxiliary Building

Element No.	Elevation (ft)	Torsion as Computed in Amendment 50 (k-ft) x 10^5	Torsion as Computed by Procedure Outlined in Question 3.67A (b) (k-ft) x 10^5
1	163.0	1.7	1.5
2	140.0	50.6	37.3
3	115.0	76.6	53.6
4	100.0	71.1	51.2
5	188.0	0.08	0.03

The above two tables show that the torsion as computed by procedures described in Amendment 50 is consistantly higher than the torsion as computed by procedures outlined in Question 3.67A (b).