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SUBJECT: Forwards completed response to 831114 request for addl info re Contention 3 to 831014 testimony.Response submitted on 831117 did not include JW Fisher ltr.

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PACIFIC GAS AND ELECTRIC COMPANY

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J. O. SCHUYLER
VICE PRESIDENT
NUCLEAR POWER GENERATION

December 2, 1983

Mr. George W. Knighton
Licensing Branch No. 3
Division of Licensing
Office of Nuclear Reactor Regulation
U. S. Nuclear Regulatory Commission
Washington, D.C. 20555

Re: Docket No. 50-275, OL-DPR-76
Diablo Canyon Unit 1
Additional Information Regarding Contention 3
To Staff Testimony of October 14, 1983

Dear Mr. Knighton:

On November 17, 1983, PGandE provided information requested in the NRC letter of November 14, 1983.

The enclosed material which was submitted on November 17, 1983 now includes the John W. Fisher letter which was inadvertently omitted.

Sincerely,

J. O. Schuyler

Enclosure

cc: L. Chandler
H. E. Schierling
Service List

B001
1/1

PGandE Responses to Staff Request for Additional Information

(Staff Testimony Regarding Contention 3 - October 14, 1983)

Question - Answer 36:

Provide justification for the assumption that the floor slabs are rigid in the auxiliary building for stick model analysis. (SSER 18, Section 3.2.4.4)

Response:

In order to justify the assumption that the floor slabs of the auxiliary building are sufficiently rigid in their own planes as represented in the stick model analysis, parametric studies were performed by the Diablo Canyon Project. These studies considered the effects of the diaphragm flexibility in the fuel pool area, which is the most flexible floor section in the auxiliary building. The studies were made by developing a three-stick model which allowed for the inclusion of inplane floor flexibility. The details of these studies are given in Section 4.2.15 of ITR 55. A comparison of the results from the one-stick and the three-stick models are summarized in Tables 12 and 13 and Figures 15, 16 and 17. Table 12 compares the frequencies and participation factors. The fundamental frequencies compare within 5 percent and the participation factors are within 13%. The fundamental frequency of the three-stick model showed a reduction as would be expected since additional flexibility exists in the three-stick model, mainly the inplane flexibility of the slabs in the vicinity of and surrounding the fuel pools. Table 13

compares the shear forces at various elevations and the largest variation at any level is about two percent. The response spectra are compared at various locations in Figures 15, 16 and 17. The response spectra at elevations 100 feet from the three-stick model is completely enveloped by the broadened response spectra from the one-stick model at that elevation. Figure 16 compares the response spectra at the mass center of the auxiliary building at elevation 140 feet. The response spectra from the three-stick model exceeded that of the one-stick model by eight percent at the frequency of the peak response. The broadened response spectra from the one-stick model envelopes the response spectra from the three-stick model at all other frequencies. The response spectra at the point farthest from the center of mass is shown in Figure 17. The comparison shown in this figure shows that the broadened response spectra from the one-stick model completely envelopes the response spectra from the three-stick model.

These comparisons indicate that the diaphram flexibility in the fuel pool area, as characterized by the three-stick model, agrees very well with results obtained from the one-stick model. Since the fuel pool zone has greater in-plane flexibility than any of the other slabs, the flexibility of the slabs in other areas should not cause important deviations in the response spectra obtained from the one-stick model. Thus, the assumption of in-plane diaphragm rigidity of the floor slabs in the one-stick model is validated.

<u>Question - Answer 32:</u>

Confirm that separate torsional time histories were input for each column of the fuel handling building. (SSER 18, Section 3.2.5.5)

Response:

As stated on page 2.1.3-7, Revision 2 of PG&E's Phase I Final Report, the dynamic analyses of Models 2.1 and 2.2 of the fuel handling building structure were performed by using a set of time histories (one translation and one rotation) at the center of mass of the auxiliary building at elevation 140 ft. Since the bases of the individual columns of the fuel handling building are connected to the center of mass point of the auxiliary building at elevation 140 ft. by a rigid linkage system which forms a rigid plane connecting the individual columns with the center of mass, the rotational time history is multiplied by the respective distance from the center of mass to each column to provide unique time histories to these individual columns.

Question - Answer 37:

Provide details of bolt bearing capacities evaluated for the bolts in the roof trusses of the turbine building. (SSER 18, Section 3.2.8.4)

Response:

The bolt bearing capacities of the 3-bolt connection in the roof trusses of the turbine building were evaluated by application of the provisions of the AISC Code 8th Edition, Part 2. Page 4-8 of the Hosgri Report states that the AISC Code, 7th Edition, Part 2, is to be used for initial evaluation of steel capacities in the turbine building. Further, the stated section provides for additional evaluation where more severe response results in inelastic deformations of the lateral force resisting elements. For the more severe responses, the allowable stress limitations of AISC Code, 7th Edition need

not be applied. In accordance with this provision of the Hosgri criteria, the 8th Edition provisions were invoked as follows:

Equation 1.16-1 of the code provides the capacity of a bolt with standard spacing, l_e

$$l_e = \frac{2P}{F_u t} \cdot \frac{d}{2}$$

Substituting the values for this connection,

P = 42.5 kips per bolt

This is the capacity of each of the two inside bolts.

Equation 1.16-2 of the code provides the capacity of the third bolt near the edge (edge distance l_b)

$$l_b = \frac{2P}{F_u t}$$

from which, P = 29.75 kips

Therefore, the total working load (P_w) of the 3-bolt connection is:

$$P_w = (2 \text{ x } 42.5) + (29.75)$$

$$= 114.75 \text{ kips}$$

Sections 1.16.4 and 1.16.5 of the commentary to the AISC 8th edition code states that the formulas 1.16-1 and 1.16-2 of the specification are based on a factor of safety of 2.

Therefore, the ultimate capacity of the 3-bolt connection:

P_u = (Factor of Safety) x (Total working load)

P_u = (2)(114.75)

= 229.5 kips

The calculations for the connection capacity were sent to Professor John W. Fisher of Lehigh University for review. His letter providing the review comments is attached which concludes, under paragraph 3, that the capacity thus evaluated has up to 10% additional margin.

LEHIGH UNIVERSITY
Bethlehem, Pennsylvania 18015

Fritz Engineering Laboratory
Building 13

July 15, 1983

Mr. C. H. Moore
Project Engineer - Unit 1
Diablo Canyon Project
Bechtel Power Corporation
P.O. Box 3965
San Francisco, California 94119

Dear Mr. Moore:

I just returned from my lecture tour of Australia and found your April 5, 1983 letter and enclosures regarding the turbine building bottom chord roof bracing connections.

1. I have reviewed the calculations, and it is my opinion that they provide a lower bound estimate of the connection capacity of the three bolt group that is provided by the double angle - gusset connection. The shear capacity of the gusset along the fastener line is reasonably estimated using the capacity for the end distance and fastener spacing.

2. The adequacy of this model can be seen from tests that we carried out on three bolt points in 1977 when the current AISC provisions were being developed. The yield point of the A588 steel plate was 52.8 ksi, and its tensile strength was 75.2 ksi. 7/8 in. A490 bolts in 15/16 in holes were used to connect 3/8 in. plates. Following is a summary of the test data.

Joint	Thickness	L	P_u	σ_b/σ_u^P
IV-IT	0.372 in.	3 in.	238 kips	3.24
IV-2T	0.372 in.	3 in.	240 kips	3.28
IV-2S	0.370 in.	2.25 in.	224 kips	3.07
IV-1S	0.374 in.	1.375 in.	188 kips	2.55

The bolts in these joints were all installed by turn of nut and had good clamping force. Slip loads were between 95 and 108 kips. The fastener spacing was 3 in. for all joints. Note that joints IV-IT and IV-2T both resulted in σ_b/σ_u^P values that exceed 3.0. Hence, the shear model does not hold for the full end distance of 3.5 in. This is the reason an upper bound is placed on bearing stress at $3.0 \times \sigma_u^P$ (at ultimate).

The predicted capacity is given by the following for Joints IV-2S and IV-1S.

$$P_u = 0.7\ (\sigma_u^P)\ (2t)\ (L - \frac{d}{2}) + 2 \times 0.7\ (\sigma_u^P)\ (2t) \cdot (s - d) \qquad (1)$$

For Joint IV-1S this gives 198 K and for Joint IV-2S it gives 229 K. These values are 2 to 5% above the observed experimental values.

If the AISC approximations are used, the predicted capacities are given by:

$$P_u = L F_u t + 2 (e - \frac{d}{2}) F_u t \qquad (2)$$

This yields 202.6 k for Joint IV-2S and 180.7 K for Joint IV-1S. These are 4 to 10% less than the experimental values, and therefore provide a factor of safety greater than 2.

Figures 1a and 1b attached are photographs of the test specimens.

3. The calculated values given on Sheet 5 are about the same as the capacity given by Eq. 2. This results because 2 x 29.75 = 59.5 K which is equal to the value provided by the edge distance calculation on Page 5. Hence, the predicted capacity of 229.5 K for your three bolt joint is a conservative estimate. You can expect up to 10% more capacity because of the bolt tension, and because Eq. 2 was used to estimate capacity. Although not shown on the sketch on Page 3, I have assumed that the end distance for the gusset plate is also 1-3/4 in., as is shown for the angles.

4. It is apparent from Fig. 1b that considerable ductility exists in the connection before failure. The plate shear mode is a ductile condition. Hence, different sized angles will result in redistribution as they deform and transfer load in proportion to the plate shear area.

Please advise if you have any questions on the enclosed material.

Sincerely yours,

John W. Fisher

John W. Fisher
Professor of Civil Engineering

JWF:rag

Enclosure

[illegible]