PORTLAND GENERAL ELECTRIC COMPANY
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PORTLAND, OREGON 97204

D. J. BROEHL
ASSISTANT VICE PRESIDENT

October 6, 1978

Trojan Nuclear Plant
Docket 50-344
License NPF-1

Director of Nuclear Reactor Regulation
ATTN: Mr. A. Schwencer, Chief
Operating Reactors Branch #1
Division of Operating Reactors
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dear Sir:

Attached are partial responses to the NRC Staff questions of October 2, 1978 based on information provided by Bechtel in confirmation of telephone conversations between Portland General Electric Company (PGE), Bechtel and the NRC Staff.

This letter and attachments are being served on the Atomic Safety & Licensing Board (ASLB) and all parties to the Control Building Hearings.

Sincerely,

DJBroehl

CLARIFYING INFORMATION TO SUBMITTALS ENTITLED

TROJAN CONTROL BUILDING
SUPPLEMENTAL STRUCTURAL EVALUATION
SEPTEMBER 19, 1978

and

RESPONSE TO QUESTIONS FROM THE
NUCLEAR REGULATORY COMMISSION
DATED AUGUST 30, 1978
September 20, 1978

QUESTION # 1

"On Page B-3 of your submittal, Equation 3 and 4, there is a statement about σ_n being assumed to be equal to v_c'. Justify that statement."

CLARIFICATION # 1

To develop the basic criteria (Figure 4-1 and Appendix B), the empirical relationship obtained by Schneider was used as a basis. Schneider's test specimens (those used for establishing the basic criteria) had overall height equal to overall length The load was applied diagonally to the square specimens which had struts on each side of the pier. The testing mechanism, when studied in detail, showed that each strut received some fraction of the vertical component of the load. The values of this component in these struts are not documented in the testing report. It was assumed that the total vertical component of the diagonally applied load is resisted by the pier tested. Therefore, $\sigma_n = v_u$ (test). It is noted that if the amount of compressive force in each strut were known, then the compression force in the pier would be less than v_u and for the correlation in Figure B-3, the calculated values for Schneider's test specimens would have been lower. Therefore, the assumption $\sigma_n = v_u$ (test) is conservative. For the Berkeley test data, the actual compressive stress is documented and was used for the correlation.

A clarification of the designations used on Page B-3 of Appendix B of the Trojan Control Building Supplemental Structural Evaluation is appropriate. Equation 2 refers to ACI 318-71; v_c' in this equation represents the ultimate shear stress of the concrete without contribution of the reinforcement. Generally, v_u represents the ultimate shear stress capacity including the contribution of the reinforcement. In the basic criteria, the contribution of the reinforcement was not explicitly included as a parameter. Therefore, v_u in equation 5 corresponds to v_c' in equation 2.

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QUESTION # 2

"Provide a basis for breaking up the walls between column lines with the new criteria as opposed to not doing it under the older criteria. Justify why it is different and yet does not violate any conclusions from the old analysis."

CLARIFICATION # 2

In the supplementary evaluation, the determination of shear stress capacity is dependent on height-to-length ratio (H/W) of walls. Therefore, dividing the walls into appropriate segments was required. This division was based on continuity or discontinuity of the horizontal reinforcement in the core. If the steel column was encased in the core with continuous horizontal core reinforcement, then the wall was considered as continuous and the overall length was taken. If the core horizontal reinforcement was interrupted by the encased steel columns, then the wall was divided accordingly. If the walls were not divided into segments, capacity would be higher and thus less conservative.

In the criteria used in the re-evaluation study, the shear capacity is not dependant on H/W ratio as a parameter. Rather, the shear capacity is a function of the concrete strength and the percentage of reinforcement. Therefore, in the re-evaluation study dividing the walls into segments was not necessary.

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QUESTION # 3

"Justify the use of 150 psi on block walls without horizontal reinforcement as an upper limit."

CLARIFICATION # 3

For walls without core steel, the ultimate shear stress capacity is taken as 150 psi. The explanation of this limiting value is as follows:

1. The allowable shear stress, according to the UBC 1967, is 50 psi for members with no shear reinforcement (without 1/3 increase for earthquake loading allowed in the UBC). For masonry-type structures, the minimum factor of safety used in arriving at elastic allowable shear stresses is taken as three*. Therefore, it is reasonable to take the ultimate shear stress as 150 psi.

2. The value 150 psi is approximately $2\sqrt{f'_c}$ (for f'_c = 5000 psi) which is only the contribution of the concrete in the criteria presented in the re-evaluation study, where $v_u = v_c + v_s$.

*See "Building Code Requirements for Concrete Masonry Structures," proposed ACI Standard, ACI Journal (August 1978); and also commentary, ACI Journal (September 1978) p. 485.

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QUESTION # 5

"Justify the use of a factor of 2 on top of the AISC criteria for the joint capacity."

CLARIFICATION # 5

As it is stated in Section 6 of the Trojan Control Building Supplemental Structural Evaluation, the steel beam to column connection capacity is based on twice the AISC Part I allowable capacity. This factor of 2 is based on experiments conducted by Fisher and Beedle* and summarized in ASCE Manual No. 41.** These tests show a factor of safety of 2 to 3.3 for bearing and larger than 3 for shear of bolts.

* Fisher, J. W., and Beedle, L. S., "Criteria for Designing Bearing Type Bolted Joints," ASCE (ST 5), Paper 4511 (October 1965).

** Plastic Design in Steel, A Guide and Commentary, ASCE Manual No. 41, (1971) p. 211.

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QUESTION # 6

"State the number of modes considered in the STARDYNE analysis, the upper frequency cutoff of the significant modes, and why the other higher modes do not have a significant effect on the response. Justify that."

CLARIFICATION # 6

In the fixed-base STARDYNE analysis, the first 30 modes were included in determining the SRSS responses. In combining the modes, closely spaced modes were considered and combined by the "10% grouping method" described in BC-TOP-4A. The highest frequency was 18.7 cps. Since the sum of the effective modal weights of these modes in the N-S and E-W directions are 94% and 91% respectively, of the total weight, the higher modes which have not been included cannot contribute significantly to the global response, and the global response governs the shear force in the walls.

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QUESTION # 7

"Supply the sum of the effective weights in both the north-south and east-west directions for all the modes considered in 6 above."

CLARIFICATION # 7

Please refer to clarification offered in response to Question # 6.

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QUESTION # 9

"Provide the significant natural frequencies of the Turbine Building (for both directions)."

CLARIFICATION TO # 9

The significant natural frequencies of the Turbine Building are as follows:

	1st mode	2nd mode
N - S Direction	1.93 cps	3.96 cps
E - W Direction	0.89 cps	3.63 cps

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QUESTION # 10

"Confirm, that the September 1 submittal just documented the results of the meeting that we had to discuss this new information, that the results were preliminary in nature and the September 1 submittal has been superseded by the "Supplemental Evaluation" submittal.

CLARIFICATION # 10

On August 28, 1978, a meeting was held with the NRC to discuss new information regarding Control Building design. At this meeting, presentations were given by Bechtel engineers on various technical details. Copies of the overhead slides used for these presentations were left with the NRC, and they were attached to the NRC meeting notes. The engineers who gave these presentations emphasized that the technical results presented were preliminary, since Bechtel was still checking data. Subsequent to this meeting, the following two documents were submitted to the NRC attached to PGE's letter of September 1, 1978.

a. Attachment 1, "Preliminary Results of STARDYNE Finite Element Analyses of Trojan Control-Auxiliary-Fuel Building Complex," August 28, 1978.

b. Attachment 2, "Supplementary Information On:

    1. Preliminary Assessment of Fuel Building to Resist Seismic Loads Based on Results of the STARDYNE Finite Element Analysis.

    2. Transferring Lateral Earthquake Force From the Structures to the Rock Subsoil.

CLARIFICATION # 10, continued

3. Evaluation of Deflections and Displacements" dated August 28, 1978.

These two documents summarized the oral presentation given to the NRC on August 28.

The information given at the NRC meeting and in the foregoing two documents was preliminary in nature; it should not be used as a basis for evaluation of the structures, and has now been superseded by the information given in the documents titled "Trojan Control Building, Supplementary Structural Evaluation," September 19, 1978; and "Response to Questions from the Nuclear Regulatory Commission Dated August 30, 1978," September 20, 1978.

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QUESTION # 11

"For Table 2.1 on Page 2-4 of the response to NRC questions specify the moment and the shear capacities of Fuel Building walls according to ACI 318-63. Include a statement that the 1963 code is met in total for those walls."

CLARIFICATION # 11

The additional data requested are incorporated into the expanded Table 2.1 as revised October 1978 and attached to the response to questions 13 and 14. Also, the required statement regarding the ACI 318-63 code is given in the response to questions no. 13 and 14 of this supplemental information.

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QUESTION # 21

Discuss and compare the reinforcement anchorages in the tests that you are using to assure they are continuous. The comparison should be made with steel in the Trojan walls."

CLARIFICATION # 21

In the Trojan Control Building walls, all the reinforcement in the blocks is continuous or anchored. Some of the core horizontal reinforcement is continuous also. In consideration of the reinforcement percentages for applicability of the basic criteria, only the continuous or anchored reinforcement in the block walls over the gross section was considered. For Schneider's test specimens, the report does not give a detailed description of the anchorages. The figures in the report show 180' bends, but the extension of the bend is not shown. The Berkeley test specimens had 90' bends with 9 inches of bars extending vertically upward or downward. The continuous reinforcing bars of the Trojan walls have at least the same effectiveness in anchorage as that of the reinforcing bars of the test specimens.

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QUESTION # 28

With regard to the Service Water lines that pass from the Control Building across the hallway at Turbine Building Elevation 69 ft: show that with displacements (ABS) considered, there is no failure of these lines; or if there is a failure, that the effects of such a failure are acceptable. If the line must be isolated, what other equipment is affected?

CLARIFICATION # 28

In the event of an SSE, the failure of service water lines (2"-HKD-1-71 and 2"-HKD-2-68) at Turbine Building elevation 69 ft has been considered. The subject lines supply cooling water to the room coolers (V-163A, B, C, and V-164) located in the Turbine Building switchgear room. The switchgear room houses only "A" train components. With complete loss of cooling water, the switchgear room temperature would require more than three hours to rise to the maximum postulated temperature limit of 107 F. This permits time not only to achieve a safe shutdown condition, but also time for operator action to achieve alternative cooling means.

Although the lines in question would be overstressed by the displacements resulting from an SSE, it is highly unlikely that an end-to-end break would occur due to the very ductile materials (CuNi) employed. Even assuming that such a break were to occur, the inventory carried in these lines would empty into the Turbine Building. The Control Building areas would not be flooded. The Turbine Building has sufficient drain flow capacity in terms of floor drains and stairwells to accommodate the flow (approximately 300 gpm) due to an end-to-end line break. Therefore, flooding of any safety-related equipment would not occur. Further, the loss of

CLARIFICATION # 28, continued

inventory expected from such a break is insufficient to have any significant effect on the complete Service Water System. These lines can not be isolated; however, their rupture will not affect other safety-related equipment. For these reasons, the postulated failures of these service water lines can be accommodated without affecting the ability to achieve safe shutdown.

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QUESTION # 29

What are the displacements between the Control Building and the Containment at Elevation 77 ft? State the loadings considered for containment deflection.

CLARIFICATION # 29

Relative displacements between the Control Building and Containment at Elevation 77' are given below. The displacements for the Containment are for the SSE of 0.25g.

	Maximum Displacement(inch) SSE 0.25g	
	N-S Direction	E-W Direction
Control Building @ 5% damping	0.72	0.06
Containment @ 5% damping	0.04	0.04
ABS Combination	0.76	0.1
SRSS Combination	0.72	0.072