PORTLAND GENERAL ELECTRIC COMPANY
121 S. W. SALMON STREET
PORTLAND, OREGON 97204

C. J. BROEHL
ASSISTANT VICE PRESIDENT

October 4, 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,

C J Broehl

DJB/TEB/jf/Al
Attachments

NRC Staff Questions of October 2, 1978 and Licensee Responses

Question 13

Relative to the Fuel Building walls where you used ACI 318-77, are all provisions of that code met?

Question 14

With regard to the Fuel Building walls, provide the details of not only how the shear capacities for the walls were calculated but also how the moment capacities for the Fuel Building walls were calculated.

Response to Questions 13 and 14

In the response to Question 2, Item 1 "Responses to Questions from Nuclear Regulatory Commission, dated August 30, 1978" (submitted on September 20, 1978), the integrity of the Fuel Building was addressed. The response included a table (Table 2.1) with information on loads vs. capacities. A revised table (Table 2.1, revised October 1, 1978) giving additional requested data is attached to this supplementary response. We include herein an explanation of the development of the additional information and our conclusion regarding the seismic capability of the Fuel Building.

Information on factored OBE loads (0.15g with 2% damping), derived from the original stick model and the STARDYNE finite element analyses, as well as the values of "Shear Capacity" given in the original Table 2.1, are unchanged. However, the heading "Shear Capacity" was revised to read "Design Shear Strength" in accordance with the definition given in the ACI 318-77 code. As was explained in the original response, the values in this column are based, conservatively, on $2(f'_c)^{1/2}$ shear stress. Formulae (11-33) or (11-34) of the ACI 318-77 code were not used. Calculations based on these two formulae would have resulted in considerably higher "Design Shear Strength". The "Ultimate Shear Strength" of each wall as defined by the ACI 318-63 code has been calculated and added to the table. It should be noted that the meaning of "Design Shear Strength" defined by the ACI 318-77 code and "Ultimate Shear Strength" defined by the ACI 318-63 code are

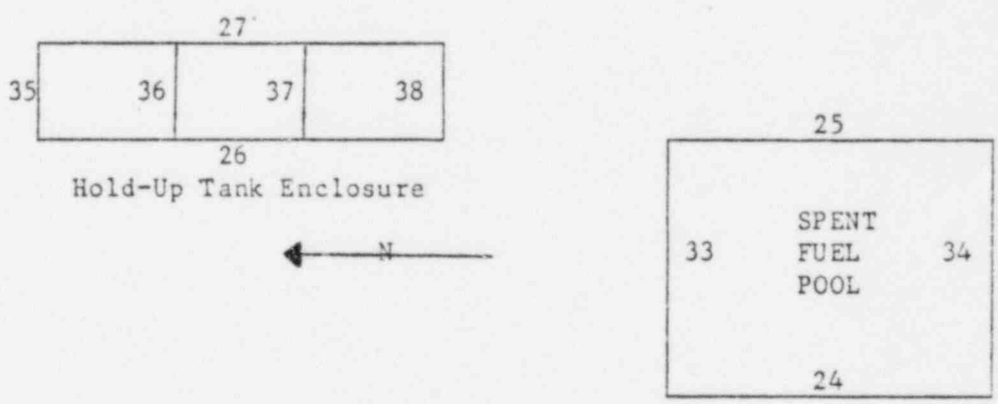
TABLE 2.1

(As Revised on October 1, 1978)

LOAD VS STRENGTH OF EACH WALL OF THE HOLD-UP TANK ENCLOSURE AND THE SPENT FUEL POOL (KIPS)

	WALL ELEMENT	Shear Strength Governed by Ultimate Resisting Moment	"Ultimate Shear Strength" ACI STD 318-63	"Design Shear Strength" ACI STD 318-77	1.4 OBE (.15 g)			
					Orig. Stick Model		Supplemental*	
					N-S	E-W	N-S	E-W
	26	8900	6600	7363	1320		2130	349
HOLD-UP TANK ENCLOSURE	27	8600	6600	7363	1320		1764	878
	35	2710	2630	2718		1360	956	1890
	36	2190	1730	1616		1360	193	729
	37	2370	1730	1616		1360	60	671
	38	1830	2630	2718		1360	140	596
SPENT FUEL POOL	24	17200	10400	11124	4500		1703	823
	25	14700	12900	13792	4500		1465	515
	33	13700	13400	14352		4340	1304	2834
	34	15700	15200	16561		2980	555	1346

* STARDYNE finite element/analysis



Key to Wall Numbers

3jr/2A11

identical. The ACI 318-63 code did not include specific provisions for calculating shear strength of shear walls. The values in the table under "Ultimate Shear Strength" are based on $2(f_c')^{1/2}$ shear stress, 0.85 capacity reduction factor and specified design strength (not "as-built" strength) of materials. In computing the "d" value (distance from extreme compression fiber to the centroid of tension reinforcement), a flange width extended to the centerline between two walls, and the reinforcement in the tensile flange and two-thirds of the reinforcement of the web were considered. Since formulae (17-2) and (17-3) of the ACI 318-63 code have not been used, additional conservatism was introduced. Based on these parameters, most of the "Ultimate Design Strengths" as defined by the ACI 318-63 code are slightly lower than the "Design Shear Strength" based on the ACI-318-77 code. However, even these lower strengths are higher than the loads derived from both the original stick model analysis and the STARDYNE analysis based on the factored load condition.

The revised table also provides information on shear strengths governed by ultimate resisting moments. The calculation of these values has been based on the ACI 318-63 code as described above. Since both the holdup tank enclosure and the fuel pool are "box" type reinforced concrete structures, with 0.6 to 0.8 percent reinforcing steel, the moment capacities of these walls are high. There is only one wall for which the shear strength is governed by ultimate resisting moment.

The results of the investigation demonstrate that the Fuel Building resists the SSE and factored OBE loads well within the FSAR criteria, based on both the ACI 318-63 and ACI 318-77 codes.

<u>Question 19</u>

With regard to the foundation:

a. Describe the foundation and any peculiarities, such as mud sills, water-proofing, etc., that would influence the values of C and μ at the interface of the foundation with the bedrock. Discuss the construction technique, etc., that was used to assure that there are no peculiarities, such as some type of weakening of that interface, through the construction technique used and weakening of the bond value. Justify that bond value between the foundation and that interface is such that the values used for the tuff material are indeed the weakest link.

b. Describe in detail how the dead loads and earthquake loads were considered in arriving at a pressure distribution for the foundation.

c. Describe your conformance to Standard Review Plan Sections 2.5.4 and 3.8.5 and justify any deviations from these two sections.

d. Explain how with a wide range of velocities, it is still conservative to use an average value.

Also explain the obvious discrepancy between the shear wave velocities used in your analysis and the shear wave velocities quoted in the FSAR.

In detail, justify the values for the tuff considering that we heard that some tests at that time were not performed and that a lot of this was extrapolation through current knowledge.

e. Also expand on the discussions such that you are assured that there are no seams, fissures, etc., in the rock that would affect the values you are using.

RESPONSE TO QUESTION 19

The footings and grade beams for the Control Building were constructed by placing concrete directly on the rock foundation. There are no mudsills or waterproofing materials under the footings or grade beams of the Control Building.

Structural excavations for footings and grade beams were made into competent rock. These excavations were scaled and cleaned of loose rock and other debris by the structural excavation contractor.

Prior to placing concrete on the foundation, debris were removed and the rock was thoroughly wetted. The foundation was inspected by the contractor's Q.C. personnel.

Foundation materials underlying the structure are volcanic rocks, primarily basalts and tuffs. The geologic mapping during the excavation phase of construction revealed the rocks are gently folded, dipping 15° to 20°, discontinous masses of basalts and tuffs with some volcanic agglomerates. These rocks have been intruded by igneous dikes which interrupt the stratification of the other rock units. Measured angles of dip of joint planes ranged from 30° to 80°. Seismic P (compressional) and S (shear) wave velocity measurements were made as the excavation neared final grade. Four geophysical survey lines were made in the power block area; these measurements indicated that P wave velocities ranged from 8,200 to 10,600 fps and the S wave velocities from 4,500 to 5,000 fps., as shown in Table A. These values are actual in-place measurements of the foundation rocks, near final grade, and include the effects of any fractures, weathered zones and any other weakness the rock may have. The attached Table B, showing static properties, is a summary of the laboratory test results of core samples obtained during the exploration program. The compressive strengths ranged from 360 to 13,150 psi and averaged 2,497 psi.

Photographic evidence exists to demonstrate that the foundation conditions as exposed during final excavations are competent rock masses as described in the FSAR. Although there are fractures, shears, and joints in the foundation, there are no seams or fissures in the rock which would affect the results of the analysis.

In order to provide very conservative estimates of the shear strength of the foundation rocks and their resistance to sliding, the following assumptions were made:

The foundation for the structure was assumed to be composed entirely of tuff, the weakest rock unit present at the site. See FSAR Sect. 2.5.1.5 and Table 2.5-1. As described above, this is a highly conservative assumption with regard to known foundation conditions.

This hypothetical mass is assumed to have a compressive strength of 1,225 psi, the average of the unconfined compressive laboratory test results on the tuff. This value is also conservative because the P and S wave velocities measured near foundation grade indicate a much stronger rock than one having 1,225 psi compressive strength. A value of 3,000 to 5,000 psi is more realistic.

The coefficient of friction μ is estimated to be at least 0.7. This is a reasonable value frequently used in analysis of sliding for concrete dams on similar rocks. The U.S. Bureau of Reclamation in their Design Standards No. 2, "Treatise on Dams", Chapter 9 suggests a value 0.8 should be used. The "Handbook of Applied Hydraulics", C. V. Davis, Editor, published by McGraw-Hill quotes values ranging 0.65 to 0.75.

Rock test results of tuffs from the Howard Prairie Dam Site east of Medford, Oregon, showed a coefficient of friction of 0.9 based on the tests of 15 specimens. These are believed to be weaker rocks than the Trojan tuffs since the compressive strengths averaged only 530 psi, compared with the 1,225 psi for 41 samples at Trojan.

The shear strength "C" of this weaker hypothetical rock mass is estimated to be in excess of 130 psi. One method of estimating the shear strength of rock is to divide the unconfined compressive strength by three. For this material this would be 1,225 divided by 3 which equals 408 psi. If we use a safety factor of three, a conservative shear strength value would be about 130 psi.

Another "check method" frequently used by rock mechanics and foundation specialists is to take 10% of the average unconfined compressive strength test results; 10% of 1,225 which is 122.5, which is in good agreement with the value of 130 psi.

Additional conservatism is included in this estimated 130-psi shear strength in that these values are based on the assumption of zero confining load - clearly not the case at Trojan; and secondly, all of this evaluation of rock properties is based on normal stress conditions. These values are used for normal design conditions. Much higher strengths of the rocks are normally used for SSE conditions. For example, normal bearing capacity values are frequently increased by at least a factor of two, often three or more for SSE conditions.

The stress distribution at the foundation level caused by gross moments due to lateral seismic loads is determined from the STARDYNE analysis output. The stresses due to the dead load and the vertical seismic acceleration are then superimposed, and the total stress distribution pattern under the foundation is established. This stress distribution indicates the areas of the foundation that may be subject to uplift. These areas are excluded in evaluating the contribution from the cohesion, C.

The stress distribution has no influence on the frictional contribution, μ, because the net vertical load, D, is unchanged by the non-uniform distribution of load on the foundation level.

It should be emphasized that the Standard Review Plans (SRP) were not issued at the time when the Trojan Nuclear Plant was being designed and constructed.

Evaluation of Section 2.5.4, Stability of Subsurface Materials and Foundations, (May 1975) has shown general compliance with the SRP.

Evaluation of Section 3.8.5, Foundations, (11-24-75) shows that the design and construction of the Control Building generally complies with the SRP with the following two clarifications. First, some of the load factors and load combinations are as defined in the Trojan FSAR and differ from the SRP. Second, the SRP lists more updated versions of some codes and some standards that were not in existence at the time the Trojan Nuclear Plant was being designed and constructed.

Using the average compressive strength of 1,225 psi for the tuff is reasonable because the confining load is assumed to be zero. Additional conservatism is obtained by using a factor of safety equal to 3 and 10% of the unconfined compressive strengths as discussed above.

The shear wave velocity used in the STARDYNE flexible base analysis was computed by using the rock properties as determined by laboratory testing and the following formula:

where, $$v = \left(\frac{Eg}{2\gamma(1+\sigma)} \right)^{1/2}$$

V = shear wave velocity, ft/sec.

E = dynamic modulus of elasticity, $lbs/in.^2$

g = acceleration due to gravity, ft/sec^2

γ = unit weight, lbs/ft^3

σ = Poisson's ratio

Using the numerical values given in the FSAR, the resulting shear wave velocity is 5,473 ft/sec.

The strength values of tuffs are discussed in Section 2.5.1.5 of the FSAR.

TEB/jf/A2

TABLE A

Line	V_P	V_S	ν	E	μ	K
A	8,500	4,500	.31	1.75	.62	1.52
B	8,500	4,500	.31	1.75	.62	[illegible]
C	8,200	4,500	.32	1.6	.62	.45
D	10,600	5,000	.35	2.3	.75	2.55

V_P = Compressional wave velocity ft/sec

V_S = Shear wave velocity ft/sec

ν = Poisson's ratio

E = Young's modulus (Compression) x 10^6 psi

μ = Shear modulus x 10^6 psi

K = Bulk modulus x 10^6 psi

anb/3jr/2A12

TABLE B

DATA FOR ALL ROCK SAMPLES TESTED

Test	Number of Tests	High	Low	Average
Specific Gravity	34	2.51	1.84	2.13
Porosity, %	34	32.4	9.3	20.15
Absorption, %	34	17.3	3.7	9.71
Unconfined Compressive Strength, psi	55	13,510	360	2,497
Modulus of Elasticity at 150 psi, psi	21	$7.5x10^6$	$0.09x10^6$	$1.7x10^6$

TEB/3anb/jf2A16