11/20/78

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of PORTLAND GENERAL ELECTRIC COMPANY, ET AL. (Trojan Nuclear Plant)	))))))	Docket No. 50-344 (Control Building)

NRC STAFF'S PARTIAL PROPOSED FINDINGS OF FACT ON ISSUE OF INTERIM OPERATION OF TROJAN FACILITY PRIOR TO MODIFICATIONS TO CONTROL BUILDING

I. INTRODUCTION AND BACKGROUND

1. On May 26, 1978, the NRC's Director of Nuclear Reactor Regulation issued an Order for Modification of License (Order) directing Portland General Electric Company (PGE), the City of Eugene, Oregon and Pacific Power and Light Company (Licensees), to perform modifications to the Control Building at the Trojan Nuclear Plant (facility) to bring that structure into substantial compliance with the requirements of facility Operating License NPF-1.[1/] That Order resulted from the discovery by PGE and its agent, the Bechtel Corporation, of several design errors with respect to the shear walls in the Control Building at the facility. The facility has been shutdown since March 17, 1978.

2. The Order for Modification of License set forth findings that the design errors reduced the seismic capability of the Control Building,

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1/ Order for Modification of License, May 26, 1978, published at 43 F.R. 23768 (June 1, 1978).

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that the originally intended seismic capability and safety margins should be substantially restored by modifications to that structure and that operation of the facility with the Control Building in its as-built condition would violate the existing facility license. Based on the related safety evaluation by the NRC Staff, however, the Order stated that the Control Building nevertheless had adequate structural capacity to resist the licensed Safe Shutdown Earthquake (SSE) and that the facility operating license should be modified to permit operation, with conditions,[2/] in the interim period prior to approval and completion of the modifications required by the Order.

3. The Order for Modification of License also provided that the Licensees could request a hearing and that any person whose interests might be affected by the Order could file a request for hearing setting forth that person's interests and the manner in which those interests might be affected by the proceeding. Such requests were to be filed by June 26, 1978 and, in the event that a hearing were ordered, the terms

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[2/] The conditions under which interim operation would be permitted are that:

(1) no modification which may in any way reduce the strength of the existing shear walls shall be made without prior NRC approval; and

(2) in the event that an earthquake occurs that exceeds the facility criteria for a 0.11g peak ground acceleration at the plant site, the facility shall be brought to a cold shutdown condition and inspected to determine the effects of the earthquake on the facility. Operation cannot resume under these circumstances without prior NRC approval.

Order for Modification of License, May 26, 1978, p.9.

of the Order for Modification of License would not become effective until a date specified in an order made following the hearing.

4. Pursuant to the opportunity for hearing provided by the Order, timely requests for hearing were filed by the Columbia Environmental Council (CEC), Eugene Rosolie, acting on his own behalf and as representative of the Coalition for Safe Power (CFSP), and by Bonnie Hill, John A. Kullberg, Stephen M. Willingham, David B. McCoy, C. Gail Parson and Nina Bell, all as individuals. These requests for hearing were considered by an Atomic Safety and Licensing Board established to rule on requests for hearing[3/] at a Special Prehearing Conference held in Portland, Oregon on July 26 and 27, 1978.

5. Based on this consideration, the requests for hearing of CEC, Eugene Rosolie and CFSP, Mr. Willingham, Mr. McCoy, Ms. Parson and Ms. Bell were granted.[4/] CEC, Eugene Rosolie/CFSP and Mr. Willingham were admitted as individual parties and Mr. McCoy, Ms. Parson and Mr. Bell were consolidated pursuant to 10 CFR §2.714(e) and admitted as a single party.[5/] Mr. Kullberg withdrew his request for hearing and, therefore, was not admitted as a party. In addition, the State of Oregon was granted leave to participate, through its designated agencies, as an

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3/ See Establishment of Atomic Safety and Licensing Board to Rule on Petitions, June 29, 1978.

4/ Order Concerning Requests for Hearing and Intervention Petitions, July 27, 1978.

5/ Id.

interested state pursuant to 10 CFR §2.715(c).[6/] Subsequently, an untimely petition filed by the Bonneville Power Administration (BPA) was granted and BPA was consolidated with the Licensees as a party to the proceeding.[7/]

6. A Notice of Evidentiary Hearing was issued on August 1, 1978. Pursuant to the Order for Modification of License of May 26, 1978, that Notice set forth the scope of the evidentiary hearing as limited to the following two issues:

(1) whether interim operation prior to modifications required by the May 26, 1978 Order for Modification of License should be permitted; and

(2) whether the scope and timeliness of the modifications required by the May 26, 1978 Order to bring the facility into substantial compliance with the license are adequate from a safety standpoint.

Subsequently, by Order dated August 25, 1978, the Atomic Safety and Licensing Board (Board) granted the Licensees' motion to bifurcate the proceeding into two phases, the first of which would involve consideration of, and a decision on, interim operation prior to Control Building modifications and the second of which would consider the proposed modifications themselves. The hearing was originally scheduled for September 6, 1978 but was rescheduled after the Licensees informed the Licensing Board and parties of additional information that became available in August 1978. This Initial Decision addresses the first issue of interim operation of the Trojan facility with the Control Building in its as-built condition.

6/ Id.

7/ See Order dated August 25, 1978.

7. The evidentiary hearing on the issue of interim operation commenced on October 23, 1978 in Salem, Oregon. The evidentiary hearings were held from October 23 through October 25, October 30 through November 3, and December 11 through December___, 1978.[8/] There were_____ pages of transcript. Witnesses were presented by the Licensees, the State of Oregon and the NRC Staff. (Appendix A hereto lists the exhibits admitted into evidence).

II.
FINDINGS OF FACT ON THE MATTER OF INTERIM OPERATION

A. Control Building Description

8. The Control Building at the Trojan Nuclear Plant is made up of a structural steel frame with steel beams and columns which support concrete floor slabs. Shear walls form the periphery of the Control Building (PGE Exh. 10, pp.2, 3). The Control Building shear walls are composite walls consisting of a reinforced concrete inner core of varying thickness sandwiched between 8-inch thick reinforced concrete masonry (Staff Exh.5, p.1). The masonry block cells are completely filled with concrete grout (Tr.1737, 2223) which has a compressive strength similar to that of the inner concrete core (Staff Exh.5, pp.1, 2). The steel frame and columns carry the vertical loads in the structure (PGE Exh.10, p.4), while the shear walls carry the lateral loads resulting from earthquakes (Tr. 738; PGE Exh.10, p.4).

9. The Control Building, Auxiliary Building and Fuel Building are interconnected by foundation and floor slabs. These floor slabs act as

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8/ Limited appearance statements from members of the public were heard in Portland, Oregon on October 26 and 27, 1978.

diaphragms to transfer lateral loads which are resisted by the Control Building and Fuel Building. The Auxiliary Building, provides some resistance to lateral loads and is further supported laterally by the Control and Fuel Buildings which are situated on either end of the Auxiliary Building (PGE Exh.10, p.4; Tr.741).

B. Design Deficiencies in Control Building Shear Walls

10. During an evaluation of the Control Building shear walls in April 1978 in conjunction with proposed security-related modifications to those walls, the Bechtel Corporation[9/] identified a potential nonconformance with the design criteria stated in the Trojan Final Safety Analysis Report (FSAR) (PGE Exh.13, p.2). This potential nonconformance was reported to the NRC's Office of Nuclear Reactor Regulation which, after evaluation of the matter, issued the Order for Modification of License discussed above.

11. The nonconformances with criteria, identified as design deficiencies, fall into two major categories. First, both the horizontal and vertical reinforcing steel embedded in the inner concrete core of the Control Building shear walls is generally discontinuous, rather than continuous as assumed in design, in that it is not anchored to the steel beams and columns of the Control Building's steel frame (Staff Exh.5, p.2; PGE Exh.10, p.5). Under the applicable codes and standards with which the Control Building is to comply, steel reinforcement must be adequately anchored by bonds, hooks or mechanical anchors (Staff Exh.5,

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9/ Bechtel Corporation was the Licensees' architect/engineer for the Trojan Control Building and executed the design of that structure (PGE Exh.13, p.1).

p.3) or otherwise anchored by being welded to or run through the steel beams and columns (PGE Exh.10, p.5). This discontinuity in the steel reinforcing bars resulted from interruption of the reinforcement by the steel frame members embedded in the concrete and from the fact that construction drawing details failed to show the proper anchoring requirements at all areas where the steel frame intersects the reinforcement in the shear walls (Staff Exh.5, p.3).

12. The second design deficiency was an underestimation of the amount of reinforcing steel required in the Control Building shear walls caused by a misapplication of the applicable Code[10/] shear design formulae (Staff Exh.5, pp.3, 4) and an algebraic error in the manipulation of equations used to determine the required amount of reinforcing steel (Staff Exh.5, pp.4, 5; Tr.853). In calculating the amount of reinforcing steel required in the shear walls, the designer used an allowable concrete shear stress of 3.5 $\phi\sqrt{f'_c}$ rather than the appropriate value of 2.0 $\phi\sqrt{f'_c}$ [11/] (Staff Exh.5, p.4; PGE Exh.10, p.6, n.3). In addition, in solving the equation for the required area of reinforcing steel, the designer multiplied the concrete shear force capacity per unit length by a factor of 1.4 when the correct factor was 1.0 (Staff Exh.5, p.5). The combined effect of these errors was that the concrete contribution in the original design calculations was approximately 2.5 times what it should have been

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10/ Building Code Requirements for Reinforced Concrete, ACI 318-63 (Staff Exh.5, pp.2, 3).

11/ ϕ is a capacity reduction factor defined by ACI 318-63 (Staff Exh.5, p.5) and f'_c is the compressive strength of concrete (Staff Exh.5, p.2).

under the applicable design criteria resulting in too little steel reinforcement being placed in the Control Building shear walls (Staff Exh.5, p.6; Tr.2163; PGE Exh.10, p.6, n.3).

13. As a result of the aforementioned design deficiencies, the capacity of the Trojan Control Building and related structures, systems and components to withstand the seismic events for which the facility was licensed is lower than that capacity intended when the facility was licensed.[12/] The reduction in seismic capacity due to the design deficiencies has been estimated to range from about 30% (Tr.1583, Tr.574-75) to roughly on the order of 50% (Tr.2129). While such estimates provied an indication of the effects of the design deficiencies, they are

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[12/] The basic parameter for seismic design is the ground acceleration expressed as a fraction of "g" level or the gravity acceleration of the earth. From an evaluation of the maximum earthquake potential for the site, a maximum vibratory ground motion which safety-related structures, systems and components must withstand is defined and such safety-related features are designed accordingly (PGE Exh.10, pp.7, 8). This maximum earthquake, termed the Safe Shutdown Earthquake (SSE), is defined for Trojan as 0.25g for zero period acceleration (PGE Exh.10, p.8; Tr.1838, 1841-42) with the corresponding licensed SSE design response spectra which describes the vibratory ground motion associated with the SSE (PGE Exh.17, FSAR fig.3.7-2). For an SSE thus defined, safety-related structures, components and systems must be designed to remain functional assuming 5% structural damping (Tr.1865).

Apart from the SSE requirements, the seismic criteria for the Trojan facility also require the establishment of an Operating Basis Earthquake (OBE). This earthquake is defined for Trojan as having a 0.15g ground acceleration with the corresponding OBE design response spectra and 2% structural damping for design purposes (PGE Ex.10, pp.20, 21; PGE Exh.17, FSAR fig.3.7-1). If vibratory ground motion exceeding that of the OBE occurs, the facility must be shut down until it is demonstrated to the Commission that no functional damage has occurred to those features necessary for continued operation without undue risk to the public health and safety (PGE Exh.10, p.19). OBE "capacity", in effect, is the ability to withstand the OBE without functional damage to safety-related structures, systems and components which would bring into question the safety of resumed operation.

not necessarily determinative of the actual seismic capacity of the Trojan Control Building in its as-built condition. Thus, it is necessary to ascertain whether the Control Building and related structures, systems and components are capable of safely withstanding the SSE despite the design deficiencies so that interim operation would not pose an undue risk to the public health and safety and whether the design deficiencies necessitate the imposition of any operating conditions if it is determined that the facility can safely operate in the interim period prior to modifications.

C. Evaluation of Seismic Capacity of Control Building Complex

14. An extensive evaluation of the Trojan Control Building complex (Control Building, Auxiliary Building and Fuel Building) in its as-built condition was performed. The analyses included an initial reevaluation of the Control Building complex based on modifications of the original analysis performed in 1970 and 1971 when the structures were designed (Tr. 914, 915), a finite element analyses using the TABS computer program (Tr. 915, 916), and a three-dimensional finite element analysis using the STARDYNE computer program (Tr.917).

(i) Initial Reevaluation Using Original Design Analyses

15. The original 1970-71 seismic analyses of the Trojan Control Building complex utilized a fixed-base beam-stick model in which all the mass associated with each level of the structure was lumped into a single concentrated mass and the concentrated masses were interconnected by vertical sticks and horizontal beams representing the stiffness characteristics of the walls and floors (PGE Exh.10, p.9). The elastic member stiffnesses were derived using uncracked concrete properties (Staff Exh.5,

p.7) and the mass was taken as the structure's own weight (dead weight) plus 50 percent of the specified floor live load (PGE Exh.6, Attachment 1, p.2). Inertial loads were determined using the modal analysis spectrum response technique and modal responses were combined using the absolute sum value technique (PGE Exh.6, Attachment 1, p.3). The Control Building complex modeled in this manner then was represented by three primary elements: the Control Building at the west end of the complex and the spent fuel pool and holdup tank enclosure at the east end of the complex. These elements provided the lateral resistance to seismic forces with the Auxiliary Building between them assumed to provide essentially no lateral resistance (Tr.914). Based on this analysis, forces in the structures were derived.

16. In the initial reevaluation (performed using the stick model and hereinafter referred to as "initial re-evaluation") of the Trojan Control Building, the results of the original seismic analysis were utilized since the original analysis considered the shear walls as having uncracked concrete properties without accounting for reinforcement and would, therefore, be unaffected by the design deficiencies identified for the Control Building (Staff Exh.5, p.7). These results were then modified to account for the design deficiencies and to more realistically represent the structure in its as-built condition.

17. The modifications to the original analysis consisted of the following:

(a) The original seismic loadings were conservatively derived by combining modal responses by the absolute sum technique, although the original seismic analysis criteria allow modal response combination either by this technique or by the square root of the sum of the squares (SRSS) method. The SRSS method was used in the initial reevaluation (Staff Exh.5, p.7; PGE Exh.10, p.11). The SRSS method was acceptable under the original seismic design criteria and is acceptable under current criteria (Staff Exh.5, p.16; Tr.700). It provides a more realistic method of modal response combination (Tr.640, 641), is commonly used in the type of structural analysis involved here (Tr.1594) and is generally regarded as acceptable practice (Tr.2109). Utilization of the SRSS technique resulted in loads which are 80% of those computed by the absolute sum technique (Staff Exh.5, p.7).

(b) The masses used in the original seismic analysis at various elevations were conservatively computed by considering the dead load and 50% of the live load since, at the time of that analysis, the final weights for the structure and its contents were not defined precisely (Staff Exh.5, p.8; Tr.638). Actual masses, based on as-built weight information, were utilized in the initial reevaluation.[13/] Use of as-built masses, which were not known at the time of the original analysis, is reasonable and acceptable (Staff Exh.5, p.16; Tr.2108). It resulted in calculated loads which are 87% of the original design loads (Staff Exh.5, p.8; PGE Exh.10, p.11).

(c) In the shear wall capacity determination of the original seismic analysis, the yield strength for reinforcing steel was taken to be 40,000 psi as set forth in the FSAR (Staff Exh.5, p.8; PGE Exh.10, p.10). However, mill certificates for the steel used in the shear walls show that the minimum yield strength is actually 45,000 psi. (PGE Exh.6, Attach.

13/ A further detailed refinement of the masses was used in the STARDYNE analyses (Tr.1607-09).

C, Table C-1; Staff Exh.5, p.8). Thus, in the capacity determination for the reevaluation, a yield strength of 45,000 psi was used for the reinforcement. Since the actual steel strength is now known from tests, use of the certified minimum values for determining shear wall capacity is reasonable and justified (Staff Exh.5, p.17; Tr.1768; Tr.2108).

(d) In the shear wall capacity determination of the original seismic analysis, the design compressive strength for concrete and grout was taken to be 5000 psi as set forth in the FSAR (Staff Exh.5, p.8; PGE Exh.10, p.10). However, the concrete cylinder tests for the material actually used in the Control Building show that the 90-day compressive strength for the concrete and the 28-day compressive strength for the grout are in excess of 6000 psi (Staff Exh.5, p.8; PGE Exh.10, p.10; PGE Exh.6, Attach. C, Table C-2; Tr.707). Thus, in the capacity determination for the reevaluation, a concrete compressive strength of 6000 psi was used. Since the actual material strengths are now known from tests, use of this value for determining shear wall capacity is reasonable and justified (Staff Exh.5, p.17; Tr.1767; Tr.2108). In fact, the evidence indicates that the compressive strength of the concrete may be 10 to 20% higher than the value derived from the cylinder tests since concrete compressive strength increases with time and the concrete in the structure has aged several years since the cylinder tests were performed (Staff Exh.5, p.17).

(e) The initial reevaluation also modified the original analysis in that the shear capacity of the composite shear walls used a correct application of the ACI-318-71 shear formulae with the appropriate and conservative concrete shear strength of $2\sqrt{f'_c}$ rather than $3.5\sqrt{f'_c}$ erroneously used in the original seismic analysis (Staff Exh.5, p.9). Use of a concrete shear strength of $2\sqrt{f'_c}$ is conservative for the shear walls in the Trojan Control Building since tests have demonstrated that the shear strengths are considerably higher (Staff Exh.5, pp.19-23; Tr.583, 596-97) and current design codes would allow the use of $3.3\sqrt{f'_c}$ (Tr.584-85). In addition, the shear capacity of the composite shear walls was computed by considering an equivalent wall thickness equal to the thickness of the concrete core and half the grouted masonry blocks (Staff Exh.5, p.9). These assumptions are reasonable since the grout, with strength comparable to that of the concrete core, infills approximately half the block area (Staff Exh.5, p.18). Only the continuous and adequately embedded reinforcing steel was relied upon in the capacity determination (Staff Exh.5, p.9).

(f) The initial reevaluation of seismic capacity also differs from the original analysis in the method used to determine the moment resisting capacity of shear wall piers. In the reevaluation, this determination was based upon use of the equivalent wall thickness described above, limiting the concrete strain to 0.002 in/in and limiting the strain in the outer reinforcing steel in the tension side of the deflecting wall to twice the yield strain, which is well below the ultimate strain for steel (Staff Exh.5, p.9). This criteria with regard to concrete strain is reasonable since such strain is thereby limited to somewhat below the crushing strain. The criteria for reinforcing steel strain is also reasonable since, because there is additional moment capacity from other reinforcing steel uniformly distributed in the wall which is not at yield, it results in moment capacity which is somewhat below the actual ultimate moment capacity (Staff Exh.5, p.18).

(g) In the original design, it was assumed that the Control and Fuel Buildings provided most of the lateral support for the Auxiliary Building. Only a few Auxiliary Building walls were treated as structural members. In the reevaluation, certain additional walls in the Auxiliary Building are relied upon to carry some of the lateral load originally assumed to be carried by the Control Building. Only the reinforcing steel in these additional walls was considered to carry loads by dowel action[14/] (Staff Exh.5, pp.9, 10; PGE Exh.10, p.11). Reliance on the capacity of these additional capable shear walls in the Auxiliary Building is reasonable and acceptable (Staff Exh.5, p.17; Tr.1768-69; Tr.2109). Moreover, limiting the capacity of these walls to that provided by dowel action gives a conservatively lower capacity than actual full capacity and is acceptable (Staff Exh.5, p.17).

18. With the assumptions and modifications delineated above, the reevaluation analysis of the Control Building complex derived seismically-induced forces and structural capacities based on the original analysis of the complex. The reevaluation utilized the design bases SSE criteria of 0.25g, the design response spectra, damping values, methodologies and, with the exceptions noted above, the general structural criteria, load

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[14/] Dowel action is a structural mechanism by which lateral forces are resisted by vertical rods such as reinforcing steel. Dowel action is illustrated by two concrete blocks, one on top of the other, with vertical pieces of steel connecting the blocks together. If horizontal forces are applied which would tend to push the upper block in one direction and the lower block in the opposite horizontal direction, resistance to these forces and movement of the blocks would be provided by the vertical steel acting as a dowel (Tr.751-52).

factors and load combinations originally used in assessing the adequacy of the Control Building design and set forth in the FSAR (PGE Exh.10, p.9).

19. Individual wall capacities were calculated for each wall at each elevation in the Control and Auxiliary Buildings and for each elevation, the capacities for all walls parallel to the North-South direction were added. Similarly, the capacities for all walls in the East-West direction were added. These capacities were then compared to the appropriate seismic shear forces determined from the reevaluation to determine the seismic capability of the structure (Staff Exh.5, pp.10).

20. The results of this initial reevaluation demonstrate that the Trojan Control Building in its as-built condition can safely withstand the SSE peak ground acceleration of 0.25g and that the structure has an OBE peak ground acceleration capacity of 0.11g, although the original OBE acceleration level approved for the Trojan Nuclear Plant site is 0.15g (Staff Exh.5, p.28; PGE Exh.6, pp.1, 2; PGE Exh.10, p.12).

21. The initial reevaluation also included a calculation of structural capacity based on dowel action of the reinforcing steel and columns in the structure. These calculations assumed that horizontal cracks in the shear walls extended through the structure at various elevations in the Control Building. The horizontal shear force was assumed to be carried in dowel action only by the continuous and adequately embedded steel reinforcement and the columns fully embedded in the shear walls. The shear friction contribution of the concrete core and masonry

block in the shear walls was conservatively neglected (Staff Exh.5, pp.14-15, 27).

22. This evaluation demonstrated that the structure had a minimum shear resistance capacity approximately 1.4 times the required SSE capacity at a given elevation, giving further confidence in the capability of the structure to resist SSE loadings (Staff Exh.5, p.27; PGE Exh.10, pp.12, 13).

(ii) TABS Analysis

23. As additional confirmation of the results of the initial re-evaluation, the Control Building complex in its as-built condition was analyzed using the TABS (Three-dimensional Analysis of Building Systems) computer program (Tr.642). Rather than consisting of a modification of some previous structural evaluation, TABS was a new and independent evaluation (Tr.919-20). In the TABS evaluation, the walls are modeled in their actual positions (Tr.917) and wall stiffnesses are represented by finite elements (Tr.915). The model represents the building system as an assembly of independent plane frame and shear wall elements interconnected by rigid floor diaphragms (PGE Exh.10, p.12, n.7).

24. The results of the TABS program are moments and shear forces on the walls (PGE Exh.10, p.12, n.7). For the SSE condition, the TABS analysis predicts total shear forces for the Control Building complex which are substantially lower than those predicted by the initial re-evaluation (PGE Exh.10, Ref.3, Appendix A, Table A-1). However, the TABS program is somewhat limited in that floor motion can only be represented by three parameters (displacement in two directions and a single

rotation) and the mass distribution representation is similarly limited (Tr.916). Because of this, the adequacy of TABS to represent box-type wall systems such as those found in the Control Building complex is questionable (Staff Exh.6, p.8; Tr.916; Tr.970; Tr.1587-88). No reliance, therefore, was placed on the TABS analyses, though that analysis would indicate that the structure would experience loads less than those predicted by the other evaluations.

(iii) <u>STARDYNE Three-Dimensional Finite Element Analysis</u>

25. One further supplemental confirmatory analysis was performed - a response spectrum modal analysis with a three-dimensional finite element representation of the Control Building complex using the STARDYNE computer program (PGE Exh.10, p.13; Staff Exh.6, p.1). Input to the program included geometry of the building complex, wall and slab thicknesses, material properties, stiffnesses, support conditions and mass distributions (Tr.1492-98). The STARDYNE model represented the building complex using 460 nodal points tied together by 685 plate elements representing walls and floors and 56 beam elements (PGE Exh.10, p.15), modeling the major shear walls and floor slabs with elastic properties representative of the actual walls and floor slabs (Staff Exh.6, p.7). This provides a more realistic representation of the building complex and of force distributions therein than any of the previous analyses (Staff Exh.6, pp.2-3, 7; Or. Exh.1, p.10; Tr.2146). The evidence indicates, in fact, that the STARDYNE model is the most comprehensive technique available for analyzing highly indeterminate structural problems such as those involved here and should represent the building complex in a

manner as close to reality as is possible (Tr. 1002-03, 1488-91, 1774-78, 2095-96). Of all the evaluations performed, only STARDYNE accounts directly for all aspects of distributed stiffness and mass and it predicts the most reliable mode shapes (PGE Exh.12, p.3).

26. Because the STARDYNE analysis for the Control Building complex used linear elastic properties, the resulting forces, force distributions and accelerations are upper bound limits (PGE E n.12, p.3; Tr.850-52, 1554). For the North-South direction, where there is higher loading relative to capacity (Staff Exh.6, p.7), the total base shear[15/] for the Control Building from the STARDYNE analysis is about 20% greater than that predicted by the initial reevaluation using the "stick" model (Staff Exh.6, p.8; PGE Exh.10, p.15). The STARDYNE-predicted base shear for the Fuel Building decreased by 28% (Staff Exh.6, p.8; Tr.639). In addition, the distribution of seismic forces to the shear walls differed from that previously predicted (Staff Exh.6, p.1). Nevertheless, a comparison of the total base shear for the entire building complex from the STARDYNE fixed-base model,[16/] 19600 kips, to that from the initial reevaluation, 18500 kips (PGE Exh.10, Ref.3, App. A, Table A-1), shows

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15/ Base shear is the lateral force that would be applied to the structure by an SSE (Tr.641).

16/ The STARDYNE analysis used two separate sets of foundation assumptions, one in which the building complex base where the structures are secured to the underlying rock was assumed to be flexible and one in which it was assumed to be rigid or fixed (Tr.740-41). The difference between resulting base shears from the flexible base analysis and the fixed-base analysis is insignificant and use of the fixed base model is appropriate based on the foundation rigidity of the Trojan site indicated by the foundation shear wave velocity (PGE Exh.10, Ref.3, App. A, p.A-9; Staff Exh.6, p.7).

good agreement and differences in predicted response between the models are to be expected based on differences in structural representations (Staff Exh.6, p.8; Tr.973; PGE Exh.10, Ref.3, App.A, p.A-11).

27. Given the better defined yet higher wall loadings from STARDYNE, a reassessment of the actual behavior and capacity of the Control Building shear walls was performed using test results from reinforced grouted masonry shear piers and reinforced concrete shear panels. This reassessment resulted in a set of criteria which was used to determine wall capacities and structural behavior based on STARDYNE-predicted loads (Staff Exh.6, p.3; PGE Exh.10, pp.15-16). The capacity criteria were derived from tests which used concrete having only half the strength of that present in the Control Building shear walls (Tr.977; Tr.2165). Consequently, the actual capacity of the Trojan shear walls will exceed that predicted by the criteria and the criteria are thus conservative (Staff Exh.6, p.9; PGE Exh.10, Ref.3, App.B, pp.B-7, B-10; PGE Exh.12, p.4; Tr.1739-40). Confirmation that these criteria conservatively underpredict shear capacity is provided by recent tests performed by the Portland Cement Association (PGE Exh.10, Ref.3, App.B, p.B-6).

28. The wall capacity criteria were derived to predict shear compression capacity, flexure or bending moment capacity and dowel action capacity. Wall capacities were computed based on these criteria and the lowest value for a given wall was taken as the capacity of that wall (PGE Exh.12, p.4). The capacities were then compared to the loads derived from the STARDYNE elastic analysis which allowed each member to reach its total elastic (upper bound) loading irrespective of capacity

(Staff Exh.6, p.9). Where a STARDYNE predicted load exceeds the shear compression, bending moment or dowel action capacity of a particular member, the loads redistribute, through the floor slabs, to other walls whose capacities are not yet exceeded (Tr.1526-27). The wall whose capacity is exceeded will continue to bear some load (Tr.2100-01). The STARDYNE evaluation which limited loads in the bending-moment-controlled walls shows that there are a few small walls in the Control Building for which calculated elastic loads exceed capacity.[17/] However, these walls contribute only a very small part of the total building shear resistance and other walls will accept the 10 to 15% excess load over capacity from the small walls whose capacities are exceeded (Tr.1742-43) since analysis shows that the Control Building has good ability to redistribute loads (Staff Exh.6, p.10; PGE Exh.10, Ref.3, p.7-7; Tr.2191-93).

29. Since loads on walls whose capacities are exceeded will redistribute to walls with excess capacity, the important parameter for determining structural capability is the total capacity of all walls rather than the capacity of any single wall (PGE Exh.10, Ref.3, p.5-1). A comparison of total capacity to total load for the STARDYNE analysis which accounted for load redistribution demonstrates that for an SSE,

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17/ In the North-South direction, five small walls have elastic loads greater than capacity at elevation 45' to 61' and capacity is exceeded in 1 small wall at elevation 61' to 77'. In the East-West direction, a lesser number of small walls have capacities less than elastic loads (PGE Exh.10, Ref.3, p.5-1). For the most limiting major wall in the Control Building, wall capacity exceeds total wall loading (including redistributed loads) by more than 15% (Staff Exh.6, p.10; Tr.2191-92).

total capacity exceeds total load at each elevation by at least 27% (PGE Exh.10, Ref.3, Tables 5-1 to 5-4). Thus, as is the case for the other analyses previously discussed, the STARDYNE evaluation demonstrates that the Control Building in its as-built condition has adequate structural capacity to withstand the required SSE (Staff Exh.6, p.11; Or. Exh.1, p.12).

(iv) OBE Capacity

30. While all the evaluations of the structural capacity of the as-built Control Building show that the structure has adequate capacity to safely withstand the licensed SSE, the analyses indicate that the design deficiencies have reduced the capacity of the structure in such a manner that it does not meet the license criteria for an OBE of 0.15g[18/] (PGE Exh.10, p.20).

31. The NRC Staff has expressed a concern that because the Trojan FSAR criteria with regard to fatigue from an earthquake and the stress levels at which such fatigue becomes important are not met with the Control Building in its as-built condition, cyclic degradation of the structure must be considered (Staff Exh.5, p.29). Though cyclic degradation of the steel reinforcement is not a concern (Staff Exh.5, p.29), such degradation of the concrete and masonry portions of Control Building shear walls could occur with earthquakes at or near the licensed OBE

18/ For the Trojan facility, the OBE loads govern design (Staff Exh.5, pp.4-5, n.2).

level of 0.15g.[19/] Such degradation could impair the structure's ability to withstand a subsequent SSE (Staff Exh.5, pp.30-31; Tr.1438-40).

32. The initial reevaluation of the Trojan Control Building indicated that while some nonlinear behavior or degradation could occur even at an OBE level of 0.11g recommended as a condition for interim operation in the Order for Modification of License of May 26, 1978, such degradation would be sufficiently small that the occurrence of one earthquake at or below that level during a one year period of interim operation would not significantly affect the ability of the Control Building to withstand a subsequent SSE (Staff Exh.5, pp.30-31). The STARDYNE analysis, however, shows an increase in shear wall forces of as much as 20% relative to those predicted in the initial reevaluation. That analysis also indicates, in the Staff's estimation, that nonlinear behavior in the most highly loaded major shear wall, the west wall of the Control Building, will begin to occur at an OBE level of 0.087g[20/] (Staff Exh.6, pp.11-12; Tr.2273). The Staff thus recommends that for interim operation, the OBE level at which the facility should be shutdown and inspected be set at the onset of nonlinear behavior in a major shear wall, conservatively predicted as 0.08g (Staff Exh.6, p.12). For OBE levels less than 0.08g,

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19/ Because of the design deficiencies, more local cracking and distress will occur at this earthquake level than would occur if the design deficiencies did not exist (Tr.1468).

20/ The Staff's estimate in this regard is based on the assumption that first shear cracking in concrete begins to occur at about half the ultimate stress or around 130 psi. For the highly loaded west wall of the Control Building, this stress level would occur at an OBE of 0.087g. Test data indicate, in fact, that shear cracking could begin even sooner - at stress levels of 100 psi (Tr.2131-34, 2271-73).

no cyclic degradation will occur (Tr.2255). This OBE level will also assure that the structure remains essentially elastic at the OBE level as required by Trojan FSAR Sections 3.8.1.3.3 and 3.7.2.18 (Staff Exh.5, p.29).

33. The Licensees, on the other hand, contend that the interim operation OBE level should be set at 0.11g based on the load equivalency between the factored 0.11g OBE load with 2% damping and the SSE load at 5% damping (Tr.852; 861-64).[21/] The Licensees' expert witnesses expressed the view that such an approach is reasonable and sufficiently conservative to avoid significant cyclic degradation, although some concrete cracking would not be precluded at a 0.11g OBE level[22/] (Tr.1448, 1450-54, 1457-58). Nevertheless, the Licensees have committed to accept a 0.08g OBE level for interim operation (Tr.1807-08).

34. The Staff's approach in this regard is more conservative than that expoused by the Licensees (Tr.1459). The basis for the Staff's approach in setting a 0.08g OBE level is that it provides assurance that the facility will be shutdown and inspected at an earthquake level before significant cyclic degradation will occur. In view of this and

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21/ This does not imply, based on the Staff's recommendation of a 0.08g OBE level, that the Control Building's SSE capability has been reduced below 0.25g (Tr.859). Instead, it reflects a difference in technique between the Staff and the Licensee in setting the OBE level for the as-built Control Building (Tr.861-64).

22/ The evidence shows that at an OBE level of 0.08g, some minor cracking of shear walls would occur. At 0.11g, there would be somewhat greater yielding, cracking and inelastic behavior (Tr. 1762-64).

of the Licensees' commitment described above, we find that the appropriate "OBE level"[23/] for interim operation should be 0.08g.

(v) Effects of Structural Displacement

35. In addition to a determination of actual Control Building strength to resist seismic loadings, consideration must be given to the effects of structural displacements on equipment, components and systems and on the overall ability to bring the facility to the safe shutdown condition under the seismic events it is required to withstand. For this purpose, the maximum amounts of interstory displacements (i.e. displacement of one floor relative to the displacement of an adjacent floor) within the Control Building and interstructure displacements between the Control Building and Containment and Control Building and Turbine Building were conservatively estimated considering increased nonlinear behavior of the Control Building (Staff Ex.6, p.4). Turbine Building displacements were refined from those found in the original analyses considering the as-built configuration of the Turbine Building (Staff Exh.6, p.4; Tr.831-32, 1755). The Control Building displacements were conservatively estimated using the stress results from the STARDYNE elastic analysis for the west wall of the Control Building (the major and most highly loaded wall relative to its capacity) and a shear stress vs. shear strain curve derived from concrete and masonry test data (Staff [illegible], p.4).

23/ [illegible] level" in this context refers to that level at which the facility [illegible] be shut down and inspected with operation to resume only after [illegible]roval.

36. These displacement evaluations indicated that the maximum interstory displacement that could occur would be 0.53 inches in the North-South direction between elevations 45' and 61' in the Control Building. This is the upper limit on such displacement (Staff Exh.6, pp.4, 13; PGE Exh.10, p.30). The maximum displacement of the Control Building itself occurs at the top of the structure and was determined to be 0.9 inches[24/] in the North-South direction and 0.09 inches in the East-West direction (Staff Exh.6, pp.4, 5; PGE Exh.10, Ref.3, App.D, pp.D-1 to D-3). These values also must be considered as upper-bound displacements (Staff Exh.6, p.13; Tr.1569, 1571, 1757).

37. The effects of the estimated interstory displacements on safety related equipment, components, piping and cables were examined in detail by both the Licensee (PGE Exh.11) and the Staff (Staff Exh.7). The Licensee conducted a survey of the facility which demonstrated that displacements as high as one inch between floor and ceiling in any given part of the Control Building would not adversely affect pipes and electrical cables, all of which was found to be very flexible and insensitive to the postulated displacements (Tr.803-805, 949-50, PGE Exh.11; PGE Exh.10, p.31). The Staff conducted its own evaluation which shows that safety-related piping and cables in the Control Building can easily accommodate the estimated interstory displacements without adverse effects (Staff Exh.7, pp.3-5).

24/ Through an independent calculation based on consideration of reduced stiffnesses, this displacement was conservatively estimated to be about one inch (PGE Exh.12, pp.8-9).

38. Considering the estimated maximum displacements of the Control Building in the North-South and East-West directions and calculated Turbine Building and Containment deflections, the gaps between the Control and Turbine Buildings were shown to be reduced by a maximum of 2.4 and 2.49 inches in the North-South and East-West directions, respectively. Since there is a three-inch gap between these buildings at the place of maximum displacement, no building contact will occur (Staff Exh.6, p.5; PGE Exh.12, p.12). Similarly, the minimum separation between the Control Building and Containment, considering maximum building deflections, is 0.76 inches (Staff Exh.6, p.5). Moreover, cables and cable trays running between buildings will not be affected by building displacements since there is ample slack in the cables to accommodate the displacements and the cable trays are not rigidly connected between buildings but are separately attached to each building with a four inch gap between the trays (Staff Exh.7, pp.2-3). One pipe which provides service water to room coolers in the switchgear room is rigidly attached to both the Turbine Building and the Control Building such that relative motion between the buildings could cause the pipe to break. However, the equipment served by this pipe has a redundant counterpart which would be unaffected by rupture of this pipe. Moreover, the rise in temperature in the switchgear room if this pipe ruptured would be sufficiently slow that several hours would be available in which to effect cooling of the room by placing fans in the doorway. Water from the ruptured pipe would be collected in drains and carried away. Thus, rupture of this pipe would have no affect on safe shutdown of the plant (Staff Exh.7, p.4; Tr.2206-10).

39. Based on the foregoing, we find that displacements of the as-built Control Building under seismic loadings up to and including the SSE will not affect the ability to safely shut down the facility and maintain it in a shutdown condition.

(vi) Effects of Spalling

40. The potential for concrete spalling[25/] was investigated. The evidence indicates that even for a 0.25g SSE, spalling of walls and ceilings in the as-built Control Building should not occur (Tr.554-55, 558-59, 565-66, 628, 834, 874-75, 2106-07, 2138, 2169). Should spalling occur in any event, it will be minor in nature, will result in pieces of blocks falling very close to the wall from which they are dislodged, and will not adversely affect safety related equipment (Tr.555-58, 565, 835-36, 2106; PGE Exh.11, p.2).

(vii) Floor Response Spectra and Seismic Qualification of Equipment

[This section to be supplied later]

D. Additional Considerations

(i) Licensing Board Questions on Seismic Instrumentation, Procedures Following a Seismic Event, and Impact of Control Building Modifications on Interim Operation

41. During a Special Prehearing Conference on August 14, 1978, we requested that the Licensees and the NRC Staff address several matters related to the Control Building design deficiencies and the safety of

25/ Spalling occurs when, as a result of an impact on a wall, pieces of concrete are dislodged from the wall (Tr.564).

interim operation (Tr.413-417, transcript pagination corrected). Most of these questions have been addressed in the previous discussions and will not be elaborated upon further. The remaining questions are addressed below.

42. The Licensees were requested to provide information with regard to the facility's seismic instrumentation and operator actions following an indication of an OBE. In this vein, evidence was presented showing that three separate instrumentation systems are installed and operating at the facility. The first is a triaxial multi-element response spectrum recorder with peak shock annunciator (PGE Exh.15, p.1). The response spectrum recorder for this system is attached to the Containment base slab (PGE Exh.15, p.2; Tr.882) and is designed to visually alarm in the Control Room for any earthquake having peak horizontal ground accelerations equal to or greater than the OBE (PGE Exh.15, p.2). This instrumentation, which can be set to alarm in the Control Room at OBE levels of 0.08g (Tr.2044) and is currently set at 0.11g (PGE Exh.16, p.1), measures peak acceleration as a function of frequency in the North-South, East-West and vertical directions[26/] (PGE Exh.15, p.2; Tr.1867-68). The second system consists of five triaxial time-history recording accelerographs which measure and record frequency, amplitude and mode shapes of the seismic response of Seismic Category I structures. These instruments, which are located in the Category I structures, are

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26/ This instrument will detect approximately the maximum peak accelerations, regardless of earthquake direction (Tr.945-47, 1869). Since the Control Building structural response is influenced most by the North-South and East-West components of an earthquake and has been evaluated for maximum SSE and OBE accelerations in these directions, the earthquake acceleration components in these directions, which are measured by this instrumentation, are the components of importance (Tr.1771-73).

not direction dependent (PGE Exh.15, p.2; Tr.1869). The third system consists of seven triaxial peak recording accelerographs which are located at various places throughout the facility and provide additional data which may be used to evaluate the effects of a seismic event. This system also is not direction-dependent (PGE Exh.15, p.3; Tr.1869). The recorders for these systems activate at a peak horizontal ground acceleration of 0.01g (PGE Exh.15, p.3; Tr.2048-49) with the activation mechanism located in the most motion-sensitive part of the facility (Tr.1960). All seismic instrumentation is subject to Technical Specifications which require monthly testing and surveillance (Tr.1683-84), and continuous operability (PGE Exh.15, p.1).

43. If an earthquake occurs which meets or exceeds the OBE level, the peak shock annunciator of the first system described above would cause at least one of 72 frequency-dependent alarm lights (36 for OBE accelerations and 36 for SSE accelerations) in the Control Room to light (Tr.2043) and the time-history recorder in the Control Room would activate (Tr.2053-54; Tr.1686). With the annunciator light indication and either the activation of the time-history recorder or "felt" seismic motion in the Control Room, the shutdown procedure would begin (Tr.2051, 2053-54). The operators would first assess plant status and take any necessary actions to deal with abnormal conditions, and then proceed immediately to cold shutdown as required by the Operating License (PGE Exh.16, pp.1-3; Tr.2051-54). It would take approximately eight hours to bring the plant to cold shutdown in an orderly manner (Tr.2059), although the

facility would be shut down immediately and put in a hot, stable shutdown condition in less than one hour (Tr.2057-58). While the plant was being brought to a cold shutdown condition, an overall inspection of the facility would be conducted to verify the integrity of equipment and components (PGE Exh.16, p.2). Within one to two hours following a seismic event requiring shutdown, the NRC would be notified pursuant to the Technical Specifications[27/] (Tr.1873-77). All of these actions following a seismic event would be in accordance with the Licensees' Emergency Procedure (PGE Exh.16, p.1).

44. Subsequent to achieving cold shutdown, the Licensees would undertake a detailed investigation and evaluation using seismic instrument data to analytically evaluate structures and equipment and, based on such evaluations, would conduct additional inspections to determine the effects of the earthquake on the facility (PGE Exh.16, pp.2-3; PGE Exh.15, p.4). In this vein, we also inquired as to the actions of the NRC following a seismic event requiring shutdown. The evidence shows that immediately upon notification, the NRC's Office of Inspection and Enforcement would activate headquarters and regional response teams dependent upon the severity of the event (Staff Exh.1, pp.11). Inspectors would be dispatched to the plant who, in combination with the NRC's resident inspector, would verify the status of the plant and determine the nature of radiological releases, if any (Staff Exh.1, pp.13-14). A detailed visual inspection would be conducted of joints between structures, steel frames,

27/ Additional assurance of prompt NRC notification would be provided by the fact that the facility has an NRC resident inspector (Tr.1723).

beams and columns, wall penetrations, ductwork, piping, cable trays, pipe restraints, walls, floors, hatches and doors for shifting deformation, breakage, spalling, cracking, system leaks and the like to determine if any damage was caused by the earthquake (Staff Exh.1, pp.13-15). Based on this inspection and an evaluation of the Licensees' analyses, a determination would be made as to whether resumption of operation should be authorized by the NRC or whether additional inspections, testing or analysis of structures, systems and components would be required (Staff Exh.1, p.15).

45. In view of the evidence thus described, we find that the facility's seismic instrumentation is adequate to provide both an immediate indication of an earthquake of a 0.08g OBE level or above so that shutdown procedures may be initiated. We also find that this instrumentation is adequate to provide the seismic data necessary to conduct a detailed analytical evaluation of the effects of the earthquake on structures, systems and components.

46. Further, we find that the facility procedures for actions to be taken during and subsequent to a seismic event requiring shutdown are appropriate and adequate to assure that the facility will be brought safely to the cold shutdown condition and that the NRC will be notified promptly so that any necessary NRC actions can be initiated in a timely manner.

47. Finally, the evidence demonstrates that the NRC Staff procedures for inspection of the facility following an earthquake requiring shutdown

and for evaluation of Licensees' actions are adequate to assure that safe conditions are maintained at the facility and that resumption of operation will not be authorized until all inspections, tests and analyses necessary to demonstrate the safety of resumed operation have been performed.

48. Finally, we inquired as to the effects on interim operation of plant modifications that might be undertaken during the period of interim operation. In this vein, the Licensees provided a description of the procedures related to plant modifications. These procedures require that all proposed modifications be internally reviewed to assess the potential impact on plant safety and particularly on the strength of the as-built Control Building shear walls. If this review, which is documented and kept on file, indicates that a proposed modification could reduce the strength of the shear walls, the modification will not be made without prior NRC approval (PGE Exh.13, pp.3, 4 and Attach. 1). Pursuant to the NRC Staff's request, the Licensees have also undertaken to provide to the Licensing Board and parties in this proceeding, including the NRC Staff, an identification and description of all proposed work which might potentially affect the Control Building shear walls (PGE Exh.13, Attach. 1, p.2; PGE Exhs. 2, 3 and 4). Based on our review of this evidence and on the fact that prior to the time that major modifications to substantially restore the desired and intended margins to the as-built Control Building are undertaken, such modifications and their effects on interim operation will be fully assessed (Staff Exh.6,

p.17), we are satisfied that adequate procedures are in-place to assure that the safety of interim operation will not be adversely affected by plant modifications. Moreover, in view of the fact that the matter of timing of modifications will be considered fully in the second phase of this proceeding, at which point information will be available as to the scope of the modifications and the time required to implement them, we will defer a decision as to the appropriate time limit for interim operation until the modification phase of this proceeding.

(ii) Intervenors' Concerns

49. In view of the narrow issue of interim operation to be addressed in the initial phase of this proceeding, the Licensing Board did not require that intervenors formulate and submit contentions bearing upon this issue.[28/] The intervenors in this proceeding did not present direct evidence on the issue of interim operation but contributed to the compilation of a full and complete record through cross-examination. During that cross-examination and in opening statements (Tr.467-68), intervenors raised the issue of the need for an overall safety audit of the Trojan facility in view of the admitted design deficiencies with regard to the Trojan Control Building.

50. Our jurisdiction in the initial phase of the proceeding is a limited one - to determine if interim operation of the as-built Trojan Control Building and the equipment therein can be authorized with reasonable assurance that such operation will not endanger the public health

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28/ Order of August 25, 1978, pp.8-9.

and safety in view of the identified design deficiencies. We do not have the authority to examine every matter which, of necessity, must be and was explored at the construction permit and operating license stages for this facility nor can we expand the issues in this proceeding beyond those related to the design deficiencies that resulted in the notice of hearing which describes the issues which we are authorized to consider. Houston Lighting & Power Co. et al. (South Texas Project, Units 1&2), ALAB-381, 5 NRC 582, 592-93 (1977); Public Service Co. of Indiana, Inc. (Marble Hill Nuclear Generating Station, Units 1&2), ALAB-316, 3 NRC 167, 170-71 (1976); Vermont Yankee Nuclear Power Corp. (Vermont Yankee Nuclear Power Station), ALAB-245, 8 AEC 873 (1974). A safety audit of the entire Trojan facility is thus beyond our power to order. A "safety audit" of the specific effects of the identified Control Building design deficiencies is, on the other hand, precisely what has been undertaken in this proceeding. In this regard, all parties were given wide latitude to explore all areas related to the safety aspects of the design deficiencies and interim operation. Nothing in the record indicates that additional matters need be explored to reach a determination as to the safety of interim operation with the Control Building in its as-built condition. Thus, intervenors demand for a general safety audit of the Trojan Nuclear Plant must be rejected.

IV. SUMMARY AND CONCLUSIONS REGARDING STRUCTURAL ADEQUACY OF CONTROL BUILDING COMPLEX FOR INTERIM OPERATION

51. The uncontroverted evidence clearly establishes that, based on all of the detailed evaluations performed, the Trojan Control Building

in its as-built condition and the interconnecting structures comprising the Control/Auxiliary/Fuel Building complex[29/] have adequate structural capacity and strength to safely withstand the licensed SSE during a period of interim operation prior to modifications[30/] (Tr.659-62; Tr. 860-61; Tr.1035; PGE Exh.12, p.6; Staff Exh.5, pp.27-28; Staff Exh.6, pp.11, 16; Or. Exh.1, pp.9, 12). Gross failure or collapse of the as-built Control Building or the shear walls therein is not a credible consequence of earthquakes up to and including the SSE and such failure cannot occur for this level of seismic event (Tr.668; Tr.687-88; Tr.1469; Tr.1471-72; Tr.1527-28; Tr.1548-50; Tr.1756; PGE Exh.12, pp.10, 13). There is general agreement among the qualified structural experts who testified in this proceeding that the as-built Control Building can safely withstand an earthquake at least 50% higher than the licensed SSE (Tr.663-64; Tr.1474-76; Tr.2105, 2110; Tr.2291; PGE Exh.10, p.28).

52. By the same token, the OBE capacity of the Control Building has been reduced as a result of the design deficiencies. Because the OBE capacity has been reduced and because the potential exists that the structure's ability to withstand an SSE may be adversely affected by

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29/ The Control Building design deficiencies have no effect on the seismic capacity of the Fuel Building. The ratio of capacity to load in that structure remains on the order of 6 to 10 (Tr.1753-54) and forces on the hold-up tank enclosure and the spent fuel pool are well below the capacity of these structures (PGE Exh.12, pp.11, 12).

30/ Since tornado loadings on the as-built Control Building are only one-quarter of the loadings imposed by a 0.25g SSE, such loadings pose no threat to the structural integrity of the building (PGE Exh.10, Ref.2, p.6-1; Tr.730).

prior earthquakes less than the SSE, as we discussed previously, we find that the "OBE" level of 0.08g recommended by the Staff is appropriate for interim operation.

53. While we have found that the as-built Control Building retains its ability to withstand the SSE from the standpoint of inherent building strength, we note that the Order for Modification of License of May 26, 1978 would impose a condition on interim operation prohibiting any modification that would reduce the strength of the existing shear walls without prior NRC authorization. The Licensees have in fact made a commitment to comply with this condition (PGE Exh.13, Attach.1, p.2; Tr.2045-46) and we see no basis for not imposing it.

54. The uncontroverted evidence also demonstrates that structural displacements of the as-built Control Building and spalling of Control Building walls will not adversely affect safety-related equipment, components, piping or cables or impair the ability to safely withstand the licensed SSE and to safely bring the facility to the cold shutdown condition in the event of an SSE.

55. [Conclusion on floor response spectra and equipment qualification - to be added].

56. Based on the foregoing, we find that the Trojan facility can safely operate for at least a one year period until modifications to substantially restore the desired and intended margins to the as-built Control Building are approved and completed, provided that the conditions

discussed in Paragraphs 52 and 53 above are imposed and complied with.[31/] We, therefore, conclude that such interim operation should be authorized and that the operating license should be amended accordingly.

V.
ENVIRONMENTAL CONSIDERATIONS

57. In view of our findings that the Control Building, with its design deficiencies, will safely withstand the SSE without gross failure or collapse of the structure or walls therein, and that the facility can safely be brought to the cold shutdown condition after the occurrence of any earthquake up to and including the SSE, it is clear that authorization of interim operation will not result in environmental effects or impacts that differ in any way from those previously evaluated for this facility at the operating license stage (Staff Exh.8, p.1). There is no evidence that would indicate that interim operation would involve environmental impacts other than those previously considered and evaluated.[32/]

31/ In so finding, we intimate no conclusions with regard to the question of need for modifications to the facility. The matter of modifications will be dealt with fully in the second phase of this proceeding.

32/ In a written limited appearance statement submitted by Doreen L. Nepom and dated October 25, 1978, the argument was made that under the Commission's regulations, any license amendment permitting interim operation must, per se be accompanied by an environmental impact statement as would be required for the initial issuance of an operating license. This argument misapprehends the requirements of the National Environmental Policy Act and the Commission's regulations. In an amendment proceeding, a Licensing Board may not

> ...embark broadly upon a fresh assessment of the environmental issues which have already been thoroughly considered and which were decided in the initial decision. Rather, the Board's role in the environmental sphere will be limited to assuring itself that the ultimate NEPA conclusions

(FOOTNOTE CONTINUED ON NEXT PAGE)

Consequently, we find that authorization of interim operation does not require the preparation and issuance of either an environmental impact statement or an environmental impact appraisal and negative declaration pursuant to 10 CFR §51.5(b) and (c) (Staff Exh.8, p.1).

VI.
CONCLUSIONS OF LAW

The scope of this proceeding is limited to the issue of whether interim operation of the Trojan Nuclear Plant with identified design deficiencies in the Control Building should be permitted prior to modifications required by the NRC's Order for Modification of License of May 26, 1978. We have thoroughly reviewed all of the evidence submitted by all parties with respect to this issue. We have also considered all of the proposed findings of fact and conclusions of law submitted by the parties. Those proposed findings not adopted in this Initial Decision are herewith rejected. Based upon our evaluation of the entire record, including all exhibits admitted into evidence as well as the answers elicited from witnesses in response to questions of the Board and the parties, we conclude that:

(1) Interim operation of the Trojan Nuclear Plant should be permitted in accordance with the amendment to the Operating License set forth in the Order below and subject to the terms and conditions therein;

32/ (FOOTNOTE CONTINUED FROM PREVIOUS PAGE)

reached in the initial decision are not affected by such new developments...

Georgia Power Co. (Alvin W. Vogtle Nuclear Plant, Units 1&2), ALAB-291, 2 NRC 404, 415 (1975); The Detroit Edison Co. (Enrico Fermi Atomic Power Plant, Unit 2), LBP-78-11, 7 NRC 381, 393 (1978). See also Northern States Power Co. (Prairie Island Nuclear Generating Plant, Units 1&2), ALAB-455, 7 NRC 41, 46 at n.4 (1978). Ms. Nepom's arguments are thus without merit.

(2) There is reasonable assurance that the activities authorized by the operating license, as thus amended and including the terms and conditions set forth in the Order below, can be conducted without endangering the health and safety of the public;

(3) There is reasonable assurance that the activities authorized by the operating license, as thus amended and including the terms and conditions set forth in the Order below, will be conducted in compliance with the Commission's regulations;

(4) The issuance of this operating license amendment set forth in the Order below will not be inimical to the common defense and security or to the health and safety of the public; and

(5) The issuance of this operating license amendment is not a major federal action significantly affecting the quality of the human environment and does not require the preparation of an environmental impact statement under the National Environmental Policy Act of 1969, as amended, 42 U.S.C. §4321, et seq., and Part 51 of the Commission's regulations, 10 CFR Part 51, or the preparation of an environmental impact appraisal and negative declaration under Part 51 of the Commission's regulations.

VII. ORDER

Wherefore, it is ORDERED, in accordance with the Atomic Energy Act of 1954, as amended, and the regulations of the Nuclear Regulatory Commission, and based on the findings and conclusions set forth herein, that the Director of Nuclear Reactor Regulation is authorized to make appropriate findings in accordance with the Commission's regulations and to issue the appropriate license amendment to Facility Operating License No. NPF-1 authorizing interim operation of the Trojan Nuclear Plant. The aforementioned license amendment shall contain the following provisions and conditions:

(1) Upon the effective date of this amendment to Facility Operating License No. NPF-1 and until further Order of the Atomic Safety and Licensing Board issued in conjunction with the decision on the scope and timeliness of modifications required by the Order for Modification of License of May 26, 1978, Facility Operating License No. NPF-1 is modified by waiver of those portions of Technical Specification 5.7.1 and the FSAR criteria referenced therein which are not complied with because of the identified design deficiencies in the Control Building shear walls, including:

(a) the requirement that the Control Building meet an OBE capacity of 0.15g using 2% damping as required by FSAR Table 3.7.1;

(b) the requirement that the Control Building meet an OBE capability of 0.15g and an SSE capability of 0.25g using a yield strength for reinforcing steel of 40,000 psi in accordance with ASTM minimum values as required by FSAR Section 3.8.1.3.3; and

(c) the requirement that the masonry portions of the Control Building walls meet Uniform Building Code requirements for reinforced grouted masonry as specified in FSAR Section 3.8.1.4.

(2) During the term of this amendment, the facility shall be operated in accordance with the following conditions:

(a) no modification which may reduce the strength of the existing shear walls shall be made without prior NRC approval; and

(b) in the event that an earthquake occurs that exceeds the facility criteria for a 0.08g peak ground acceleration at the plant site, the facility shall be brought to a cold shutdown condition and be inspected to determine the effects, if any, of the earthquake. Operation cannot resume under these circumstances without prior NRC approval.

It is further ORDERED, in accordance with 10 CFR §§2.760, 2.762, 2.764, 2.785 and 2.786, that this Initial Decision shall be effective immediately and shall constitute the final action of the Commission forty-five (45) days after the issuance thereof, subject to any review pursuant to the above-cited Rules of Practice. Exceptions to this Initial Decision may be filed within ten (10) days after service of this Initial Decision. A brief in support of any such exceptions must be filed within thirty (30) days thereafter (forty (40) days in the case of the NRC Staff). Within thirty (30) days of the filing and service of the brief of the Appellant (forty (40) days in the case of the NRC Staff), any other party may file a brief in support of, or in opposition to, the exceptions.

IT IS SO ORDERED.

THE ATOMIC SAFETY AND LICENSING BOARD

Marshall E. Miller, Esq., Chairman

Dr. Kenneth A. McCollom, Member

Dr. Hugh C. Paxton, Member

DATED:

Respectfully submitted,

Joseph R. Gray
Counsel for NRC Staff

Dated at Bethesda, Maryland
this 20th day of November, 1978

APPENDIX A

LIST OF EXHIBITS ADMITTED IN EVIDENCE

Licensees' (PGE) Exhibits

No.		Id.	Evid.
1	Protective Agreement on Bechtel Contract	956	956
2	August 14 Report on Work to be Done	956	990
3	October 10 Report on Work to be Done	958	990
4	October 23 Report on Work to be Done	958	990
5	No Exhibit with this number		
6	Licensee's letter dated May 5, 1978 transmitting Licensee Event Report (LER) 78-13 and attachments	531	532
7	Licensee's letter dated May 24, 1978 and attached "Supplemental Information to LER 78-13"	531	532
8	"Trojan Control Building Supplemental Structural Evaluation, September 19, 1978"	531	532
9	Responses to NRC Staff Questions	531	532

A. Letter, August 19, 1978 and attached responses to Staff Questions

B. Letter, August 21, 1978 and attached responses to NRC Staff Questions

C. Letter, September 12, 1978 and attached correction to supplementary information transmitted by letter August 21, 1978

D. Letter, September 20, 1978 and attached responses to NRC Staff Questions "Response to Questions from the Nuclear Regulatory Commission dated August 30, 1978"

E. Letter, October 10, 1978 and attached responses to NRC Staff Questions

Licensees' (PGE) Exhibits

No.		Id.	Evid.
	F. Letter, October 13, 1978 and attached responses to NRC Staff Questions		
	G. Letter, October 17, 1978 and attached responses to NRC Staff Questions		
10	Testimony, references and qualifications of Bechtel Panel	549	549
11	"Flexibility Survey Response," Trojan Nuclear Power Plant, Survey to Determine Capability of Equipment to Withstand Building Displacements - Control Building and Surrounding Structures, September 18, 1978	949	953
12	Testimony, references and qualifications of Myles J. Holley, Jr. and Boris Bresler	1029	1030
13	Testimony and qualifications of Donald J. Broehl	1808	1808
14	Testimony and qualifications of John L. Frewing	1810	1810
15	Testimony and qualifications of S.R. Christensen	1814	1814
16	Testimony and qualifications of Bart D. Withers	1816	1816
17	FSAR Figures 3.7-1 and 3.7-2 (Design Response Spectra for OBE, SSE)	1840	2060
18	Letter, April 26, 1978, Williams to Miller	1068	2115

Staff Exhibits

No.			
1	Testimony and qualifications of Robert T. Dodds	1624	1624
2	Qualifications of Kenneth S. Herring	2117	2128
3	Qualifications of James E. Knight	2118	2128
4	Qualifications of Charles M. Trammell	2119	2128
5	Testimony of Herring re: structural adequacy of Trojan control building for interim operation	2122	2123

Staff Exhibits

No.		Id.	Evid.
6	Testimony of Herring re: supplemental Stardyne analysis and effect on structural capacity of Trojan control building	2124	2128
7	Testimony of James E. Knight	2125	2128
8	Memo, August 15, 1978, Grotenhuis to Schwencer	2127	2128

Oregon Exhibits

No.			
1	Testimony and qualifications of H.I. Laursen	1068	2069

Consolidated Intervenor Exhibits

No.			
1	Bechtel-PGE c[illegible]act	683	Later admitted as Board Exhibit 1 (Tr. 961)
2	Miller Press Release on DBA Sequencers dated September 20, 1977	2004	Withdrawn
3	Letter, April 13, 1978, Miller to Williams	2018	2018

Board Exhibits

No.			
1	Bechtel-PGE contract	961	961

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	
	)	
PORTLAND GENERAL ELECTRIC COMPANY,	)	Docket No. 50-344
ET AL.	)	(Control Building)
	)	
(Trojan Nuclear Plant)	)	

CERTIFICATE OF SERVICE

I hereby certify that copies of "NRC STAFF'S PARTIAL PROPOSED FINDINGS OF FACT ON ISSUE OF INTERIM OPERATION OF TROJAN FACILITY PRIOR TO MODIFICATIONS TO CONTROL BUILDING" in the above-captioned proceeding have been served on the following by deposit in the United States mail, first class, or, as indicated by an asterisk, through deposit in the Nuclear Regulatory Commission's internal mail system, this 20th day of November, 1978:

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