UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

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

TESTIMONY OF KENNETH S. HERRING, OFFICE OF NUCLEAR REACTOR REGULATION, ON STRUCTURAL ADEQUACY OF THE TROJAN CONTROL BUILDING FOR INTERIM OPERATION

I. INTRODUCTION

On April 28, 1978, it was reported to the NRC that the shear walls in the Trojan Control Building, a seismic Category I structure, were not in conformance with the overall design criteria stated in the Safety Analysis Report for the facility. The Trojan Nuclear Plant Control Building reinforced concrete and grouted masonry block shear walls provide the resistance to the entire Control Building and a portion of the Auxiliary Building lateral (horizontal) loadings. These lateral loadings arise from the occurrence of earthquake, wind or tornado events (See FSAR Section 3.8.1.5.1). Of these postulated design lateral loadings, the seismic loading led to the Control Building not conforming with FSAR structural criteria.

The shear walls encase a steel frame which was designed to carry the vertical loads. The cross section of these walls consists of an 8-inch thick layer of reinforced grouted masonry in contact with each face of an inner reinforced concrete core which varies in thickness for different walls. The masonry block cells, which have a cross sectional area equal

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to approximately one-half of the total block cross sectional area, were fully grouted with grout having a compressive strength similar to that of the concrete core. Therefore, the original design concept was based on considering a composite wall of this type as a reinforced concrete wall with an equivalent thickness equal to the thickness of the reinforced concrete plus one-half the total masonry block thickness. The reinforced concrete Ultimate Strength Design formulae of ACI 318-63,[1/] with a design compressive strength (f'_c) of 5000 psi and the equivalent wall thickness, were utilized to determine the capability of the walls to resist forces calculated from a linear elastic structural analysis.

II. SUMMARY OF DESIGN ERRORS

Based on discussions with representatives of the Portland General Electric Company (PGE) and the Bechtel Corporation on May 1 and 19, 1978, and on a review of PGE's written submissions of May 5 and May 24, 1978, the following design errors were found to exist with respect to the Control Building shear walls:

(1) Both the horizontal and the vertical steel reinforcement in the reinforced concrete core of the walls was found to be generally discontinuous (not properly anchored) rather than continuous as assumed in design. Also, there was a limited amount of horizontal discontinuous steel in some of the inner masonry block layers of the walls. Therefore, there was actually less reinforcing steel in the walls than the amount determined in the original design calculations that

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1/ "Building Code Requirements for Reinforced Concrete", ACI 818-63.

could be considered fully effective in resisting applied lateral loadings.

According to Section 3.8.1.5 of the Trojan FSAR, the requirements which governed the design of these walls to resist the lateral loads were in accordance with those from the ACI 318-63 Code for reinforced concrete (Ultimate Strength Method) and the 1967 edition of the Uniform Building Code (UBC-67) for reinforced grouted masonry. Each of these codes requires that the steel reinforcement be adequately anchored by bond, hooks, or mechanical anchors (see Sections 917 and 918 of ACI 318-63, and Section 2417 of UBC-67).

For the composite Control Building walls, the discontinuity of the steel reinforcing bars resulted from interruption of the reinforcement by the steel frame members embedded in the concrete. The construction drawing details which were used to place the steel in the walls during construction did not show the proper anchorage required at all of the areas where the steel frame intersects the steel reinforcement.

(2) Misapplication of ACI 318-63 shear design formulae, and the applicable limiting Operating Basis Earthquake (OBE) seismic loading combination[2/] resulted in lower than the required amounts of

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[2/] The OBE loading condition limited the actual design rather than Safe Shutdown Earthquake (SSE) loading condition, since the spectral accelerations for the structural modes of vibration which contribute the majority of the earthquake loading are essentially the same for both the OBE and the SSE with peak ground accelerations of 0.15g (FOOTNOTE CONTINUED ON NEXT PAGE) reinforcing steel to resist the original design loadings being placed in the shear walls.

These design errors were:

(a) Rather than assuming an allowable concrete shear stress (V_c) of $2\phi\sqrt{f_c'}$ in calculating the shear resistance of the composite walls, as required by Section 1701 of ACI 318-63, the designer inappropriately utilized the maximum value permitted by ACI 318-63, Section 1701(d) which is $3.5\phi\sqrt{f_c'}$.

(b) Section 1702 of ACI-63 requires that adequate steel web reinforcement (the horizontal steel) be provided to resist any nominal applied shear stresses which are in excess of the allowable concrete shear stresses. Section 1703(a) of ACI 318-63 provides the formula for computing the appropriate amounts of web reinforcement. The load equation from FSAR Section 3.8.1.3.2 leads to the requirement for these walls that their ultimate capacities as defined by ACI 318-63 requirements must be greater than or equal to the resultant loadings from the governing load combination of 1.4 ($D + E_o$), where D is the dead load and E_o is the OBE earthquake loading. This requirement can be summarized for the shear walls by writing:

$$1.4V_u \leq V_c + V_s$$

where V_u = Applied shear force per unit length of wall calculated over the effective depth of the section.

2/ (FOOTNOTE CONTINUED FROM PREVIOUS PAGE)
and 0.25g, respectively, due to the additional three (3) percent structural damping allowed for the SSE above that for the OBE. Therefore, multiplying the OBE and SSE loads by the factors of 1.4 and 1.0, respectively (as defined in the FSAR), and requiring both of these factored loads to be less than the calculated ultimate strength capacity of the composite wall system led to the OBE loads governing the design.

V_c = ACI 318-63 defined concrete shear force capacity per unit length.

= $2\phi\sqrt{f_c'}\,t$ (t is the effective wall thickness)

V_s = ACI 318-63 defined reinforcement shear force capacity per unit length = $(\frac{Av}{S})\ \phi f_y = A_s \phi f_y$

A_s = The area of steel required per unit height of the wall = $\frac{Av}{S}$.

ϕ = ACI 318-63 defined capacity reduction factor = 0.85 for shear and 0.90 for flexure.

f_y = The specified yield strength of the steel reinforcement.

Therefore, $V_s \geq 1.4V_u - V_c$

The original designer inadvertently used:

$$V_s = A_s\ \phi\ f_y \geq 1.4\ (V_u - V_c)$$

which led to the use of:

$$A_s \geq \frac{1.4(V_u - V_c)}{\phi f_y}$$

for the calculation of the area of steel shear reinforcement required per foot of the wall height.

The proper formula to use was:

$$A_s \geq \frac{1.4V_u - V_c}{\phi f_y}$$

The combined effects of the calculational errors delineated in (a) and (b) above led to the use of the equation:

$$A_s \geq \frac{1.4\ V_u - 1.4\ (3.5)\ \phi\sqrt{f'_c}\ t}{\phi f_y}$$

or,

$$(1) \quad A_s \geq \frac{1.4\ V_u - 4.9\ \phi\sqrt{f'_c}\ t}{\phi f_y}$$

rather than the appropriate equation:

$$(2) \quad A_s \geq \frac{1.4\ V_u - 2.0\ \phi\sqrt{f'_c}\ t}{\phi f_y}$$

for the calculation of the required reinforcing steel area. It can be readily noted from Equations (1) and (2) above that concrete contribution assumed in the original calculations, $4.9\ \phi\sqrt{f'_c}\,t$, was 2.45 times that which should have been assumed, $2.0\ \phi\sqrt{f'_c}\ t$.

The result of these calculational errors was that too little reinforcing steel was calculated as being required in the walls. Therefore, too little steel shear reinforcement was placed in the walls to resist the original design loadings under the original assumptions.

III. LICENSEE REEVALUATION

As a result of the discovery of these design errors, a detailed reevaluation of the Control Building in its existing configuration has been performed by the Licensee to assess the present capability of the structure to withstand the Operating Basis Earthquake (OBE) and the Safe Shutdown Earthquake (SSE), which produce the limiting lateral loadings on the shear walls.

The following criteria were relied upon in the reevaluation of the lateral seismic resistance of the Control Building to determine more realistic seismic loadings, and to calculate the shear capacities of the individual walls:

1. The original seismic analysis derived elastic member stiffnesses by considering the walls as having the properties of uncracked concrete with no consideration of the reinforcement. The previously delineated errors would not affect this assumption. Therefore, the results of the original seismic dynamic analysis were utilized with the following modifications:

   a. The original seismic loadings were conservatively derived by combining modal responses by the more conservative absolute sum technique, although the original seismic analysis criteria allowed modal response combination by either this method or the more realistic Square Root of the Sum of the Squares (SRSS) technique. Utilization of the SRSS technique for the combination of modal responses resulted in loads which are 80 percent of those computed by the absolute sum technique.

b. The masses in the original seismic analysis at the various elevations were conservatively computed by considering the dead load combined with 50 percent of the live load since at the time of the analysis the final weights for the structure and its contents were not defined precisely. Consideration of the as-built weight information resulted in loads which are 87 percent of the original design loads.

2. Rather than the 40,000 psi yield strength assumed for the steel reinforcement in the original analysis, a value of 45,000 psi was utilized in the reevaluation based upon the minimum value obtained from the mill certificates for the steel utilized in the wall reinforcement. At the time of the design, the assumed yield strength was based on the specifications which were applicable to the type of reinforcement called for in the design. These specifications merely specify minimum properties. Once the materials were delivered, their mill certificates provided the properties indicative of those for the actual batches of reinforcement which were placed in the walls.

3. Rather than the design compressive strengths of 5000 psi for the concrete and the grout, the 90-day compressive strengths for the actual concrete and the 28-day compressive strengths for the actual grout indicate that the as-built compressive strengths of these materials are in excess of 6000 psi. Therefore, the higher strength was utilized for both materials in the reevaluation. The original design was based on the specification of concrete and grout mixes which would insure the development of the appropriate design

strengths within the specified periods. The strengths in actuality exceed the minimum requirements.

4. The shear capacity of the composite reinforced concrete and grouted masonry block was computed by considering an equivalent wall thickness consisting of the thickness of the concrete core plus one-half the thickness of the grouted masonry block. The ACI 318-71 shear formulae were applied with a conservative permissible concrete shear stress of $2\sqrt{f'_c}$. Only one-half of the grouted masonry block was considered since this is the approximate area infilled by the grout which has a higher compressive strength than the block itself. Also, only the continuous and the adequately embedded reinforcing steel was utilized for the capacity determination.

5. The moment resisting capacity of the shear wall piers was based upon considering the equivalent wall thickness described above, and limiting the concrete strain to 0.002 in./in. and the strain in the outer reinforcing steel to twice the yield strain (well below the ultimate strain for the steel).

6. The original design assumed that the Control and Fuel Buildings provided almost the entire lateral support for the Auxiliary Building. The only Auxiliary Building walls considered as structural members in the original design were the North-South wall between column lines L and K, the southern North-South wall between column lines N and L, and the East-West walls on column lines 46 and 55. (Refer to the Trojan Control Building Plan at El.45').

These walls extend between Elevations 45 and 61 of the Auxiliary Building. However, there are some other walls in the Auxiliary Building which are now being 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 Auxiliary Building walls was considered to carry loads in dowel action. Dowel action was considered to limit the wall capacity since it was not readily determined that the construction joint at the top of the wall was sufficient to develop the full capacity of the wall.

Individual wall capacities were calculated for each wall at each elevation of the Control and Auxiliary Buildings utilizing the criteria delineated above. For the wall system between each elevation of the buildings, these capacities were summed for each of the walls parallel to a given direction of earthquake input (either North-South or East-West) to determine the total lateral load resistance of the wall systems in that direction. The capacities were then compared to the appropriate seismic shear forces which must be transmitted through these wall systems to the foundation. The shear forces were determined from the results of the original seismic dynamic analysis, as modified by the above criteria. The wall systems between Els. 45-59 and 61-75 were the most critical. Therefore, detailed results have been presented which focus on these wall systems. Additionally, the walls perpendicular to a given direction of loading were found to adequately resist the gross bending moments. The vertical shear which must be developed at the ends of the sidewalls can be developed by the dowel action of the steel in the block and the beam to column connections.

The Licensee has found that application of this procedure leads to the calculation of a structural capability to withstand the prescribed SSE with a 0.25g peak ground acceleration utilizing the original FSAR structural damping of 5 percent. In addition, considering the original FSAR structural damping of 2 percent and the 1.4 increase in forces in the appropriate FSAR OBE load combination, application of this procedure leads to the calculation of a structural capability to withstand a 0.11g earthquake.

This procedure of summing individual member (wall) capacities to determine the resistance of a given wall system to the lateral loads parallel to their direction is somewhat different from the procedure normally followed in the reinforced concrete shear wall design process. In a normal design process, a static elastic analysis of the wall system would be performed in which uncracked concrete properties would be assigned to each wall. The total loads resulting from the linear elastic seismic dynamic analysis would be proportioned to each wall according to its relative stiffness. Each wall would then be designed to have the ACI 318 Code Ultimate Strength capacities to resist these proportioned loads. While proportioning loads to concrete members according to their relative stiffnesses does not guarantee that each wall will reach its capacity at the same deflection, this procedure has been found to be conservative. Coupled with the ACI 318 Code design requirements, this procedure results in the concrete cracking, and therefore the inelastic behavior, being limited to smaller amounts than would be expected utilizing the criteria for the Control Building reevaluation.

The procedure followed in the reevaluation of the Trojan Control Building, which utilizes the loads derived from the linear elastic seismic dynamic analysis, does not proportion loads to the walls according to their relative stiffnesses. The total resistance was determined assuming all walls reached their calculated capacities at the same stage of loading. The shear wall systems contain several doorways which create several shear piers with various stiffnesses. While not necessarily implying large degrees of overall inelastic structural behavior, in reality certain walls will reach and exceed their calculated capacities before others. Therefore, the licensee has assessed the degree to which the walls of a given wall system would exceed their calculated capacities. Empirical information was provided to substantiate that the estimated nonlinear behavior of the walls, considering the cyclic nature of the earthquake loadings, would not reach unacceptable levels such that the load carrying capability of major walls would deteriorate with successive cycles of loading. Typical load vs. deflection curves for the N-S wall system between Elevations 45 and 61, and the E-W wall system between Elevations 61 and 77 were derived from a static elastic-pseudo-plastic analysis of the wall systems by applying loads to the wall system models in increments. These increments were defined by the loading levels at which walls within the given wall system reached their capacity. When a wall had reached its capacity, its stiffness was no longer relied upon and the wall was only assumed to resist a load equivalent to its calculated shear capacity (i.e. each wall was assumed to have an elasto-plastic resistance function with the plastic limit being the calculated wall capacity). The load on the wall system was increased by these increments until the last member reached its calculated capacity. No credit was taken for the higher strengths for the shear walls (with the height to length ratio distributions indicated by the Licensee) implied by the empirical data attached to the NRC August 3, 1978 meeting summary[3/] and the empirical data generated by the Portland Cement Association concerning the behavior of reinforced concrete shear panels.

The information presented by the Licensee illustrates that the more significant walls (i.e. those with relatively large load carrying capability) do not reach their calculated capacities until the final stages of deformation. This information indicates that the lower limit for all wall systems at which the major walls reach their capacity would be approximately at a load level of 70 to 75% of the total capacity for a given wall system. (The total capacity is defined as the point at which the last wall in a given system has reached its calculated capacity. The point at which the more significant walls begin to reach their capacities is the point at which the change in slope of the load vs. deflection curves first becomes significant. This point is at 86% and 88% of the total capacity illustrated for the N-S walls between Elevations 45 and 61 and the E-W between 61 and 77, respectively.)

Both of these curves do not take into account the contribution of the additional Auxiliary Building walls considered in the reevaluation.

3/ "Summary of the Site Visit and Meeting Held on July 6, 1978, At the Trojan Site to Discuss the Control Building".

Also, the curve between Elevations 45 and 61 in the N-S direction considers a concrete compressive strength of 6000 psi and a steel yield strength of 45,000 psi, and the curve between Elevations 61 and 77 in the E-W direction considers a concrete compressive strength of 5000 psi and a steel yield strength of 40,000 psi. However, the conclusions drawn about the integrity of the Control Building considering these curves would be essentially the same if both curves were derived considering the additional Auxiliary Building walls, a concrete compressive strength of 6000 psi, and a steel yield strength of 45,000 psi (which were considered in calculating the capacities of the elevations in the reevaluation) since the proportions of the curves would be essentially the same.

An upper bound for the reinforcing steel strains utilizing the assumed criteria was determined to be approximately six to eight times the yield strain.

The Licensee has also performed an evaluation of the capability of the structure to transmit the seismic shear forces from the modified seismic dynamic analysis across assumed cracks in the reinforced concrete core and grouted masonry. The cracks were assumed to extend horizontally through the structure at various elevations between elevations 45 through 75 for both the North-South and East-West directions. It was further assumed that the entire horizontal shear force was carried in dowel action by only the continuous and the adequately embedded steel reinforcement, and only the columns fully embedded in the shear walls. For the steel reinforcement, the ultimate capacity is defined as the point at which the rebar area reached 70 percent of the ultimate stress, as determined from the mill certificates. The ability for these stress levels to be developed in the rebar was substantiated by test data. For the columns, the ultimate capacity is the lesser of (1) the load at which either the web or flange area, depending upon the direction of loading, reached 67 percent of the ultimate stress, with the material properties being taken as the ASTM specified minimums for the column material; or (2) the load as limited by the capacity of the concrete against which the columns bear to resist column shear force. Shear friction contributions to the resistance from the concrete and grouted masonry (i.e., forces, in addition to the dowel forces, resulting from friction between the wall sections above and below the crack) were conservatively neglected. This analysis was based on the previous discussion which concluded that the walls themselves would be expected to have greater shear strengths than those predicted by assuming the shear strength to be limited to $2\sqrt{f_c'}$ for the equivalent wall thickness, and the premise that cracks would be most likely to form at the construction joints in the walls from El. 45 through 75.

The results of this analysis demonstrated that at its defined ultimate capacity, the structure had approximately 1.4 times the required SSE resistance at the most critical elevation. Also, from the load deflection curves for the shear stud test data, the area under the load deflection curve to 70 percent of ultimate is approximately 12 percent of the total area under the curve to ultimate (the area under the load vs. deflection curve provides a measure of energy absorption capability). Therefore, even at its defined ultimate capacity, there is adequate assurance that there is some reserve energy-absorbing capacity still remaining in the structure. Limitations of the dowel capacity by the calculated member capacities was not considered.

Additionally, displacements were estimated from this procedure. The maximum between floor elevation displacement was estimated to be between about 0.1 and 0.15 inches and was found by the Licensee to be acceptable. This is between about 3 to 5 times that predicted from the derived load vs. displacement curve, and therefore seems to be a reasonable upper limit estimate. Also, the Licensee has determined that there is adequate assurance that the effects of the reduced mass and the possible nonlinear structural behavior on the floor response spectra for the structure would not have any detrimental effects on the safety of the plant.

IV. NRC EVALUATION

The NRC Staff has reviewed the previously summarized analyses, methodologies and results, and the supporting information submitted by the Licensee. The evaluation and conclusions are presented in the following discussions.

The utilization of the original seismic dynamic analysis, modified as set forth previously, is acceptable since the use of the SRSS combination of modal responses was acceptable in the original criteria and is still acceptable by current criteria. Also, the inclusion of the as-built weight criteria is reasonable since this information would not be expected to be accurately available at the time of the building design.

It should be noted that since the natural frequencies of the building are proportional to the square root of the ratio of stiffness to mass, consideration of the as-built mass and original stiffness of the structure in the seismic dynamic analysis would tend to raise the natural frequencies of the structure by about 7% as an upper bound since this neglects the interaction of the other structures (the Auxiliary and Fuel Buildings) through the floor diaphragms. This in turn would lead to some reduction in the seismic loading since the original natural frequencies lie approximately at the peak of the response spectra, such that an increase in natural frequency would lead to the building being subjected to lower accelerations.

It is acceptable to rely upon the capacity of other capable shear walls in the Auxiliary Building to carry part of its lateral loading; and acceptable to consider only the steel reinforcement to resist the load considering dowel action as limiting the strength of the walls to a value lower than their calculated capacities.

Utilization of the current, as-built material properties for the concrete, the grout and the rebar is justifiable at this time since the properties are derived from tests of the actual material utilized in the structure. These actual properties would not normally be available at the time of the design of the structure. In addition, the structure has aged a number of years and the compressive strength of concrete is known to increase with time. This increase would be expected to be as much as about 10 to 20 percent. However, any compressive strength increase for the concrete due to this effect was conservatively neglected.

Utilizing the ACI 318-71 formulae to calculate a concrete and grouted masonry block combined wall shear capacity with the equivalent thickness of the masonry and an allowable concrete shear capacity of $2\sqrt{f'_c}$ is a reasonable method of determining its capacity. The grout, with strength comparable to the concrete, infills approximately one-half the block area. The concrete core, although it contains rebar which cannot be relied upon since it is generally discontinuous at the wall-floor slab and wall-column junctions, is confined by the concrete block and the steel frame, and the discontinuous steel will contribute somewhat to the restraint of the concrete. Also, a concrete shear strength of $2\sqrt{f'_c}$ is a conservative approximation of the strength of deep concrete sections such as the shear walls, as will be explained in detail further on. Considering only that rebar which is continuous for calculating the reinforcement contribution to the shear resistance is acceptable.

The criteria utilized to determine the moment-resisting capacity for the shear piers provides a reasonable estimate of their individiual capacity since the strain in the concrete is limited to somewhat below the crushing strain, and all of the uniformly distributed rebar in a wall would not be at the yield tensile strain at a capacity calculated by limiting the outer rebar strain to twice the yield strain. Therefore, the capacity considered is actually somewhat below the ultimate moment-controlled strength of the wall.

Although the walls do not meet all of the applicable portions of the ACI 318-63, ACI 318-71, or Uniform Building Codes, the additional information considered in the analysis substantiates that the methods utilized to calculate the individual wall capacities are acceptable, and that the summation of these individual wall capacities to determine the total lateral load capacity of a wall system is an adequate method for the determination of their capability to resist the applied lateral loads. This summation of individual wall capacities will be rationalized in detail below for the Trojan Control Building.

It should be pointed out that members which have their behavior controlled by bending moment are more ductile and therefore have a more desirable behavior under cyclic loading than do members controlled by shear behavior. For members controlled by shear behavior, as compared to members controlled by bending moment behavior, after the load level at which the load deflection curve for a member begins to change slope substantially, failure is more sudden and cyclic degradation of the load carrying capacity is more severe. The degree of severity is a function of the amount of steel reinforcement in the wall. (This is illustrated by the test data referenced herein).

Empirical data developed by the Portland Cement Association for reinforced concrete shear panels[4/] demonstrates that the nominal shear stress (that shear stress computed by dividing the applied shear force by the quantity consisting of the wall thickness multiplied by its effective depth (length)) carried by the concrete in walls is a function of the height to length ratio (H/L) of the walls. The tests were conducted on shear panel specimens which had a top slab representing the floor through

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4/ "Shear Strength of Low-Rise Walls With Boundary Elements" by Felix Barda, John M. Hanson and W. Gene Corley.

which load is applied and which were constrained on their sides by two steel reinforced flanges, reinforced so as to assure shear failures in the specimens. The base of the specimen was fixed but the top where the load was applied was not constrained externally. For this case the test data indicated that at the formation of the first shear crack the nominal shear stress, based on effective depths which considered the reinforcement in the flanges and varied between about 1.01 and 1.09 times the wall length between the flanges, was between 4.9 $\sqrt{f'_c}$ and 6.5 $\sqrt{f'_c}$ with an average of about 5.5 $\sqrt{f'_c}$ for H/L's of 0.5 and less. For an H/L of 1.0, the strength dropped to 3.5 $\sqrt{f'_c}$. Assuming a linear variation of first shear crack stress with H/L between the average at an H/L of 0.5 and an H/L of 1, a first crack shear stress of 4.2 $\sqrt{f'_c}$ is calculated for an H/L of 0.75. These tests also showed that the first crack shear stress was essentially independent of the amount of vertical and horizontal reinforcement in the wall panels with an H/L of 0.5 and less (this parameter was not studied for H/L greater than 0.5). Also, the first shear crack shear stress level was well below the point at which substantial nonlinearities developed in the load deflection curves and, therefore, below the point at which the tests indicated substantial cyclic degradation.

Many of the Trojan shear piers are rectangular and do not have flanges. Since these would be expected to have different cross-sectional shear stress distributions than the flanged specimens tested, an assessment must be made of the difference between the test cross section and a rectangular cross section. Because there was no substantial cracking up to the formation of the first shear crack (which is based on tensile failure of the concrete), it is reasonable to assume that the wall remains linear elastic to this point. For an assumed beam cross sectional transverse shear stress distribution in the test specimen, the actual maximum shear stress is approximately 11% greater than a nominal shear stress computed on the basis of wall length or depth of the beam. The amount of shear resisted by the flanges would be only about 4% of the total. For a rectangular beam cross section with no flanges, the maximum shear stress is 1.5 times the nominal shear stress computed using the thickness and the total length of the wall. For the Trojan walls, only 80% of the length was utilized as the effective depth of the members. If all these factors are considered for a rectangular section, the first shear crack strengths for the concrete (which are less than the ultimate strengths), neglecting the additional conservatisms that the Trojan wall capacities are even further reduced by the capacity reduction factor of 0.85 and that some of the walls are constrained by cross walls (flanges), would be expected to be reduced to between about 4.7 $\sqrt{f'_c}$ and 5.8 $\sqrt{f'_c}$, with an average of about 5.1 $\sqrt{f'_c}$, for H/L's of 0.5 and less. For an H/L of 1, a first crack strength of about 3.3 $\sqrt{f'_c}$ is determined. Assuming a linear variation of first shear crack shear stress with H/L between the average at an H/L of 0.5 and an H/L of 1.0 implies a first crack stress of about 4.2 $\sqrt{f'_c}$ for an H/L of 0.75. A concrete capacity of 2 $\sqrt{f'_c}$ was used for determining the capacities of the Trojan walls. The majority of the capacity from the walls between

Elevations 45 - 61 in the N-S direction have height to length ratios which are less than 0.75. Between El. 45-61 in the E-W direction, and in both directions between El. 61-77, more than 90% of the walls have H/L's less than 0.5. Between Elevations 45-61 in the N-S direction, which is representative of a lower bound, about 80% of the total capacity comes from walls with an H/L less than 0.75. The test data, as modified to consider rectangular shear piers, indicates a first shear crack shear stress (which would be below the ultimate shear stress) for the reinforced concrete core which is 2.1 times the $2\sqrt{f'_c}$ assumed for the Trojan walls. This factor would be expected to be even greater considering that 48% of this capacity comes from walls with an H/L less than or equal to 0.5.

The test results for rectangular masonry shear piers with the bottom fixed and the rotation of the top where the load is applied restrained by external steel columns (see the NRC August 3, 1978 "Summary of the Site Visit and Meeting Held on July 6, 1978, at the Trojan Site to Discuss the Control Building") indicate that the masonry portions of the Control Building walls are also capable of sustaining loadings to higher levels than those being relied upon.

In the calculation of wall capacities, only one-half the block thickness was considered to contribute to the wall with a shear strength of $2\sqrt{f'_c}$ over this thickness and an effective depth equal to 80% of the wall length. The $2\sqrt{f'_c}$ translates into an allowable nominal shear stress of 155 psi for a compressive strength of 6000 psi. This stress level on one-half of the masonry block area is analogous to a stress level of 77.5 psi on the gross masonry area.

Grouted hollow concrete block masonry piers with masonry which should be of equal or less overall strength than the Trojan masonry were tested. The test data for a wall containing no horizontal and no vertical steel reinforcement, and that for a wall containing a small amount of vertical and no horizontal steel reinforcement, both fully grouted with an H/L of 1.0, indicate that these test specimens developed stress levels higher than those assumed for the Trojan walls. If the test data for these two specimens is converted to nominal shear stresses based upon an area equal to the product of their thickness and 80% of their total length, instead of the area considered in the report which is the product of the thickness and 100% of the total length, and neglecting the additional conservatism that the Trojan capacities are reduced by a capacity reduction factor of 0.85, the results for these two specimens indicate stress levels of approximately 130 psi as being the point at which severe non-linearities developed in the hysteresis envelopes (i.e. beyond this point, cyclic degradation would begin to become significant) for the load deflection curves. (The ultimate strength on the same basis, was approximately 140 psi.) This is approximately 70% greater than the capacity being relied upon for the Trojan walls.

The August 3, 1978 NRC meeting summary also gives "Schneiders Empirical Shear Values for Concrete Block Piers." This curve indicates that the stress level given above for the piers with H/L of 1.0 would be increased further for the majority of the Trojan walls since the H/L for the majority of the walls is less than 0.75. This curve would imply a 30% increase, which is consistent with the 27% taken for the reinforced concrete data discussed above, on the above stresses for a pier with an H/L of 0.75 above those stresses for a pier with an H/L of 1. Therefore, the 130 psi would become about 170 psi for piers with an H/L of 0.75 which is 2.2 times the capacity relied upon at Trojan and is approximately the same as 2.1 previously estimated for the reinforced concrete core based on the reinforced concrete panel test results.

For elevation 45-61 in the N-S direction, the steel contributes about 39 percent of the total capacity, and about 61% comes from the reinforced concrete and masonry block. Approximately 80% of the total capacity comes from walls with H/L less than or equal to 0.75. Also a lower bound for the point at which the more significant members of a wall system begin reaching their calculated capacities is at about 70% of the total wall system capacity. Therefore, since on the average the walls which contribute about 80% of the total capacity actually will have about 60% of their capacities greater than that assumed, on the average a member would exceed the calculated concrete and masonry capacity at most by about 60% (.3 ÷ .8 ÷ .6). Considering the additional 110% greater capacity (lower bound from the concrete core) which has been estimated to be available for the concrete and masonry portions of the walls, a margin of approximately 50% to the point where nonlinear behavior (and therefore cyclic degradation) would become significant exists. Therefore, given the margin indicated by the above simplified analysis, it is unlikely that the load-deflection curves presented by the Licensee, derived under the assumption that a wall has an elasto-perfectly plastic resistance function, would ever develop. Further, unless bending moment behavior began to govern the behavior of some walls before the higher shear stress levels indicated by the above discussion were reached, the load-deflection curves would actually remain essentially linear up to the required capacity. However, since there are a number of walls for which the capacity may begin to be governed by moment behavior before these higher shear levels are reached, the curves presented should be an upper bound for the amount of nonlinear behavior which would develop. Additionally, the members which have a given calculated moment capacity, based upon the assumption that only steel and not the concrete carries tension, may sustain a higher moment if it is realized that concrete can resist some tension. However, this tensile capacity is not consistent and is not relied upon in design.

If it is assumed that the load deflection curves between the floors were to be as presented by the Licensee, an equivalent ductility for the added energy dissipation can be estimated. The dynamic analysis was performed assuming a linear variation in the load-deflection curve between zero load and the required load capacity with a slope equal to that of the initial portion of the curve and the predominant modes of vibration lying in the energy conservation region of the response spectra. Therefore, the area under the calculated nonlinear curves can be equated to the area under an elasto-plastic bilinear resistance function with the linear portion from the origin to the plateau having the same slope as the initial portion of the nonlinear load deflection curves and the plateau being at the required capacity. If this is done for the two curves presented by the Licensee between Elevations 45-61 and 61-77, equivalent ductility ratios of about 1.50 and 1.25, respectively, are calculated for these typical wall systems. Since this would be typical for all wall systems, it can be assumed typical for the structure. Even the development of a small ductility of 10% (i.e., ductility ratio of 1.1) implies approximately a 9% decrease in the overall seismic loadings from the predominant modes of vibration. A ductility ratio of 1.25 would imply approximately an 18% decrease in these forces.

Additionally, if these amounts of nonlinear behavior were to occur, an upper bound for the percentage shift in the natural frequencies of the structure can be estimated by calculating the percentage difference in slope between the initial linear portion and that of a line which passes through the origin and the point on the calculated load-deflection curves at the total capacity. This would yield an estimate of the stiffness reduction. For Elevations 45-61 and 61-77, this implies stiffness reductions of about 65% and 89%, respectively. Taking the natural frequencies to be proportional to the square root of the quantity consisting of the stiffness divided by the mass leads to the calculation of upper bound frequency reductions of about 20% and 5.5% corresponding to the 65% and 89% stiffness reductions, respectively. Due to the interconnection of the various structures by the floor diaphragms, and the fact this upper bound would not be expected to develop due to the previous reasoning, a more reasonable estimate would be on the order of a 10% reduction, which when offset by the 7% increase due to the reduced mass, would not be expected to be significant.

It should be noted further that although the steel reinforcement which is discontinuous and is not considered in the reevaluation cannot be fully relied upon, it will contribute to crack control in the walls and will contribute somewhat to the overall shear strength. However, the amount cannot be quantified.

Further confidence in the capability of the structure to resist the seismic loadings is provided by the calculations done considering dowel action of the rebar and columns. These calculations neglected the shear friction contribution of the concrete and the masonry block. Not all walls at the typical elevation for which dowel capacities of the individual walls were given had dowel capacities which were greater than the lower of their shear or moment controlled capacity. However, other walls which have greater expected shear capacities than those calculated by the reevaluation criteria had dowel capacities which would compensate for the walls with lower dowel capacities. Also, the percentage by which the dowel capacities were exceeded was a maximum of 12.4%, and differences of this order should be compensated for by shear friction.

This dow capacity evaluation demonstrated that the structure had a minimum dow capacity approximately 1.4 times the required SSE capacity across a given elevation. If it is assumed that the load-deflection curve for the structure would be similar to that of a single dowel, the area under the structure's load-deflection curve at the assumed ultimate would be approximately 12 percent of the total area under the inelastic load-deflection curve to actual ultimate. Through equivalent energy assumptions, if this percentage area is considered to be equal to the area under the initial linear portion of the curve for a bilinear resistance function with a constant plateau at the assumed capacity, an equivalent ductility ratio to actual failure can be considered to be approximately 3.7. The seismic loading is conservatively based upon a linear elastic seismic analysis. If inelastic response is considered, utilizing a ductility ratio of only 1.5, which is less than one-half of the above estimated limit for ductility, the elastically calculated predominant mode loads would be reduced by approximately 30 percent. Considering the limitations on the individual member capacities and the possibility that all dowels may not reach their ultimate capacity, the ductility ratio of 1.5 should be a reasonable upper limit for acceptable ductilities. However, if this full ductility were utilized, the elastically calculated loads would be reduced approximately 60 percent. In summary, inelastic structural response would limit the seismic forces, and therefore shear stresses, to lower levels than would be calculated from a linear elastic dynamic seismic analysis.

Utilizing the previously discussed criteria, the Licensee has demonstrated an SSE peak ground acceleration level capacity of 0.25g, and an OBE peak ground acceleration level capacity of 0.11g, although the original OBE acceleration level approved for the Trojan Nuclear Plant site was 0.15g. There is adequate assurance that the structure has the required strength to resist an earthquake up to and including the SSE.

However, since the results are based upon a total capacity determination and the walls do not meet the appropriate code requirements, cyclic degradation becomes an important consideration. The Trojan FSAR Section 3.8.1.3.3 states that "The Category I structures are proportioned to maintain elastic behavior when subjected to various combinations of dead loads, thermal loads, seismic and accident loads." The upper limit to elastic behavior (or yield point, which is the terminology utilized in the FSAR) for concrete is taken to be the "ultimate resisting capacity as calculated from the 'Ultimate Strength Design' portion of the ACI-318-Code." The Trojan FSAR Section 3.7.2.18 states that fatigue from an earthquake was not a concern since the calculated stresses were below the yield point, as defined above for concrete. This criterion is no longer satisfied for all members in the Trojan Control Building.

The estimated upper bound for steel strains was approximately 6 to 8 times the yield strain. For strain levels of this magnitude, Figure XIV-1221.3(c)-1 of Appendix XIV to Section III of the ASME Boiler and Pressure Vessel Code indicates that the number of design cycles that can be sustained at these strain levels is about 180 cycles (corresponding to 8 times the yield strain). Therefore cyclic effects from earthquake loadings should not be of concern for the steel reinforcement.

Additionally, based upon the previous evaluation and information, there is adequate assurance that the structural behavior will not adversely impact the safety related components, equipment and piping by producing significant changes in the original floor response spectra or intolerable displacement levels.

From the previously referenced test data, it can be seen that cyclic degradation of the concrete and masonry portions of a wall should not be substantial until that wall has been loaded to the point where it has reached the point on its load deflection curve where there is a severe change of slope. As stated previously, this is not expected to occur for the significant Control Building walls. Therefore, the Control Building is expected to withstand an earthquake up to and including the SSE. However, since the 0.15g OBE stresses are approximately equal to the 0.25g SSE stresses (due to the difference in damping as previously discussed) the criteria utilized for the revaluation does not preclude stressing some members above their calculated capacities with earthquakes up to but not exceeding the OBE of 0.15g. In fact, at the determined OBE capacity of 0.11g, this nonlinear behavior is not necessarily preluded by the criteria.

Given that (1) the damping for the OBE (2% of critical) is low (2) there is a factor of 1.4 which is placed on the resulting forces, and (3) a lower bound at which the more substantial members were determined to reach their calculated capacities was about 70% of the total required capacity, there is reasonable assurance that the occurrence of one earthquake at just below the 0.11g level (of which at most only one earthquake in this range or greater would be likely to occur in the approximate one year period it takes to repair the building) should not significantly effect the structure's ability to withstand a subsequent SSE. However, its occurrence could have some effect on the strength of the structure. Over the remaining life of the plant, although unlikely, it is not inconceivable that the plant could experience one or more earthquakes near the previously approved OBE level for the site, followed by an SSE. The structure's ability to resist this subsequent SSE may be impaired by the occurrence of the previous earthquake(s) near the OBE level. The structure does not meet the facility criteria for the design OBE which has a 0.15g peak ground acceleration. Utilizing the reevaluation criteria, the licensee has demonstrated an OBE capacity for the structure of only 0.11g peak ground acceleration. The present 0.11g capability is not in accord with Appendix A to 10 CFR Part 100 which requires that the structure be able to withstand earthquake accelerations for the OBE of at least one-half of those associated with the SSE.

Therefore, given the reduced margins of safety, the possibility of successive degradation over the plant life, and the fact that the test data which substantiate the higher strengths of the walls is limited in both the number of specimens and the number of cycles to which the specimens were subjected, it has been determined that the necessary modifications to substantially restore the originally intended safety margins and durability to the structure be performed in a timely manner. Addititionally, it has been determined that the plant be shut down and inspected in the event that an earthquake occurs before the modifications are completed which exceeds the facility criteria for a 0.11g effective peak ground acceleration (EPGA, the acceleration level at which the site ground response spectra are anchored) earthquake.

It should also be noted that the occurrence at the plant site of even one significant earthquake within the time period until the necessary modifications are completed is improbable, and that the probability of an earthquake occurring which reaches earthquake levels of the order of the OBE, and especially the SSE, is more remote. The Modified Mercalli Intensities (MM) for the OBE and the SSE at the plant site are MM VII and MM VIII, respectively, as stated in Section 2.5.2.2 of the Trojan FSAR. The SSE EPGA of 0.25g corresponds to a MM VIII, and the OBE EPGA of 0.15g is slightly higher than that which corresponds to a MM VII. The acceleration level of 0.11g corresponds to an intensitity slightly below MM VII. The MM intensity scale gives for these intensity levels:[5/]

MM VII - "Difficult to stand. Noticed by drivers of motor cars. Hanging objects quiver. Furniture broken. Damage to masonry D including cracks. Weak chimneys broken at roof line. Fall of plaster, loose bricks, stones, tiles, cornices, unbraced parapets, and architectural ornaments. Some cracks in masonry C. Waves on ponds; water turbid with mud. Small slides and caving in along sand or gravel banks. Large bells ring. Concrete irrigation ditches damaged;"

MM VIII -"Steering of motor cars affected. Damage to masonry C; partial collapse. Some damage to masonry B; none

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5/ Taken from "Fundamentals of Earthquake Engineering" by Nathan M. Newmark and Emilio Rosenbleuth (Copyright 1971).

to masonry A. Fall of stucco and some masonry walls. Twisting, fall of chimneys, factory stacks, monuments, towers, elevated tanks. Frame houses moved on foundations if not bolted down; loose panel walls thrown out. Decayed piling broken off. Branches broken from trees. Changes in flow or temperature of springs and wells. Cracks in wet ground and on steep slopes;"

where Masonry A, B, C, and D definitions are:

"Masonry A, B, C, D. To avoid ambiguity of language, the quality of masonry, brick or otherwise, is specified by the following lettering (which has no connection with the conventional Class A, B, C construction).

Masonry A. Good workmanship, mortar, and design; reinforced, especially laterally, and bound together by using steel, concrete, etc.; designed to resist lateral forces.

Masonry B. Good workmanship and mortar; reinforced, but not designed in detail to resist lateral forces.

Masonry C. Ordinary workmanship and mortar; no extreme weaknesses like failing to tie in at corners, but neither reinforced nor designed against horizontal forces.

Masonry D. Weak materials, such as adobe; poor mortar; low standards of workmanship; weak horizontally."

In addition, the above reference describes an MM VI earthquake, which would correspond to an EPGA of approximately one-half of that for the MM VII earthquake, as:

> "Felt by all. Many frightened and run outdoors. Persons walk unsteadily. Windows, dishes, glassware broken. Knickknacks, books, and so on, off shelves. Pictures off walls. Furniture moved or overturned. Weak plaster and masonry D cracked. Small bells ring (church, school). Trees, bushes shaken visibly, or heard to rustle."

Conclusion:

In summary on the basis of the information which has been discussed herein, the NRC staff has concluded that there is reasonable assurance that the facility as presently constructed will withstand the SSE, as well as the less severe OBE. However, we have additionally concluded that the originally intended margins of safety have been reduced and that the previously stated applicable codes are not satisfied. We have thus concluded that interim operation for the approximate one year period necessary to effect the repairs and improvements is appropriate, however, the original structural safety margins should be restored to the extent practicable in order to ensure adequate protection of the health and safety of the public during the long term operation of the facility.

PROFESSIONAL QUALIFICATIONS
OF
KENNETH S. HERRING

I am an Applied Mechanics Engineer in the Engineering Branch, Division of Operating Reactors, Office of Nuclear Reactor Regulation. My duties and responsibilities involve the review, analysis, and evaluation of structural and mechanical aspects related to safety issues for reactor facilities licensed for power operation, and test reactor facilities, including the formulation of regulations and safety criteria. I am also responsible for coordinating various outside technical assistance programs related to structural and mechanical applications for nuclear power plants.

I have a M.S. in Civil Engineering from the University of Illinois (1974) and a Bachelor of Engineering from the State University of New York at Stony Brook (1973).

Prior to my present appointment, I was associated with Stone and Webster Engineering Corporation as an engineer in the Engineering Mechanics Group. My duties and responsibilities included the analysis and design of safety related nuclear power plant structures, with an emphasis on seismic and other dynamic analysis techniques and applications.