UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

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

NRC STAFF REVISED TESTIMONY OF KENNETH S. HERRING AND DREW PERSINKO ON THE STRUCTURAL ADEQUACY OF THE PROPOSED MODIFICATIONS TO THE TROJAN CONTROL BUILDING

Q.1 Please state your name and position with the NRC.

A.1 My name is Kenneth S. Herring. I am a Senior Structural Engineer in the Engineering Branch of the Division of Operating Reactors, Office of Nuclear Reactor Regulation.

A.1 My name is Drew Persinko. I am a Structural Engineer in the Engineering Branch of the Division of Operating Reactors, Office of Nuclear Reactor Regulation.

Q.2 Have you prepared statements of professional qualifications?

A.2 Yes. Copies of our statements are attached to "NRC Staff Revised Testimony of Kenneth S. Herring and Drew Persinko on CFSP Contention 20, 12/13 and 16 and on Structural Aspects of the Modification Work Itself."

Q.3 What are your responsibilities with regard to the NRC Staff's review of the proposed modifications to the Trojan Control Building?

A.3 (Mr. Herring) As a Senior Structural Engineer, I have prime responsibility for the Staff's structural and mechanical review and evaluation

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of the proposed modifications. This includes a review to determine what structural effects actual modification work itself might have on existing structures as well as a review of the modifications to determine whether they will substantially restore seismic margins to the Control Building Complex and bring that Complex into substantial compliance with the requirements of the Trojan license. It also includes assuring that any effects of the modifications on the response of safety related systems, piping, equipment and components are adequately accounted for.

A.3 (Mr. Persinko) As a Structural Engineer in the Engineering Branch, I am responsible for assisting Mr. Herring in the structural review and evaluation of the Control Building modifications described by Mr. Herring.

Q.4 What is the purpose of this testimony?

A.4 The purpose of this testimony is to present the Staff's basis for requiring modifications to the Trojan Control, Auxiliary and Fuel Building Complex (Complex), the Staff's position on the time dependence of interim operation, and the Staff's assessment of the structural adequacy of the proposed modifications to the Complex to restore the seismic margins to the Complex and to bring the Complex into substantial compliance with the requirements of the Trojan license. In so doing, we will describe the unresolved items identified in the Staff's Safety Evaluation Report (SER) on the proposed modifications and indicate the status of resolution of those items.

This testimony is also intended to provide responses to the structural questions set forth by the Licensing Board at the Prehearing Conference on March 11, 1980, and to address the structural aspects of Coalition for Safe Power (CFSP) Contention 22.

I. REASONS FOR REQUIRING MODIFICATIONS

Q.5 Are the modifications required by the May 26, 1978 Order for Modification of License considered applicable to the SSE as well as the OBE?

A.5 (Mr. Herring) Yes. The Order required substantial restoration of original design margins for the SSE as well as the OBE.

Q.6 Why is restoration of margins necessary?

A.6 (Mr. Herring) These margins are necessary to account for uncertainties in analysis, design and construction procedures, and in addition, are relied upon by the NRC in assessing the designs of older plants in light of current-day seismic design requirements.

Q.7 How have seismic design requirements changed since the time Trojan was licensed?

A.7 (Mr. Herring) A chronology of basic seismic design requirements, including those from around the time Trojan was designed to the present, is set forth below.

The basic seismic design requirements have undergone many changes over approximately the past 25 years. Prior to 1960, there were no specific requirements other than those contained in local building codes. Since that time, the development of the basic seismic design practices can be generally summarized as follows:

PRIOR TO 1960		Uniform Building Code Requirements
	-	Static seismic coefficient applied to structures
1960 - 1964	-	Ground motion described by Housner's averaged ground response spectra.
	-	Single degree of freedom systems were used for the evaluation of seismic responses.
	-	Horizontal and vertical earthquake responses were not combined.
1965 - 1967	-	Ground motion described by Housner's averaged ground responses spectra (in some cases Housner made revisions from the previous spectra).
	-	Multi-modal two dimensional models were used for the evaluation of seismic responses. The response spectrum approach was used most often. Time history was used occasionally.
	-	Damping values were taken as 0.5% for piping. 1% - 2-1/2% for steel structures, and 4% - 7-1/2% for concrete structures.
	-	Compliance (flexibility) for plant foundation medium was considered.
	-	Sum of the absolute value of the responses arising from the largest horizontal and the vertical earthquake was generally used for response determination.

1967 - 1971	-	Ground motion described by Housner's averaged ground response spectra modified, especially in short periods, using Newmark criteria (known as modified Newmark spectra, 1967 - 1969).
	-	Soil structure interaction effects were considered using discrete soil springs and in some cases assuming material damping.
	-	Floor response spectra generated and used in the evaluation of equipment and piping.
1971 - 1973	-	Modal damping values for the soil-structure system to represent contributions from both material and radiation damping limited to 10% of critical damping.
1973 - 1977	-	Reg. Guides 1.60 and 1.61 were introduced to define ground response spectra, and damping values (for structures, piping, equipment and components), respectively.
	-	Damping for small and large piping was raised to 2% and 3%, respectively.
	-	Soil damping determinations were required to account for the nonlinear stress - strain relationships for the foundation medium.
	-	Finite element procedures were required in the calculation of soil-structure interaction for deeply embedded structures.
	-	Three components of earthquake motion were required to be considered by taking the SRSS of the responses to each component (Reg. Guide 1.92).
	-	Floor response spectra generated per Reg. Guide 1.122.
AFTER 1977	-	Layered soils accounted for in an elastic half space soil-structure interaction analyses.
	-	The limit of 10% of critical damping on modal damping values in soil-structure interaction analyses was removed.

- Comparison of elastic half-space and finite element soil-structure interaction analyses results.

Q.8 How do current design requirements compare to those used for plants such . jan?

A.8 (Mr. Herring) In many respects they are more stringent, especially with regard to the definition of seismic loads.

Q.9 Why are older plants such as Trojan not required to be backfitted to meet current requirements?

A.9 (Mr. Herring) There are conservatisms in the design of the structures of older plants, such as Trojan, which are relied upon in determining that backfitting to meet current requirements is not necessary. In general, these conservatisms can be summarized as follows.

Conservatisms associated with the methodologies for seismic analysis and design.

a. Conservatisms for structures, systems, and components.

   1. Dynamic analysis.

      Elastic dynamic analysis are performed using low damping values and time-history or response spectrum analysis methods. In modal response spectrum analysis, closely spaced modes are combined by absolute summation.

2. Three input components.

   Three input components of an earthquake (2 horizontal and 1 vertical) are considered. Both horizontal earthquake components are assumed to be equal.

3. Loading combinations.

   Loading combinations consider other loadings (e.g., dead weight, live loads, pressure loads, etc.) in addition to the seismic loadings. Seismic loading is only a part of the total loading and in fact, other loadings besides seismic may in cases govern design.

b. Effect of inelastic behavior.

   In reality, well engineered structures, components and systems are capable of sustaining loads which are beyond those which would bring them to their elastic limit without sustaining damage. For small excursions into the inelastic range, seismic inertial loads are reduced as a function of the amount of inelastic action in comparison with those calculated elastically. This phenomenon can be considered by the use of a ductility factor which is equal to unity for purely elastic behavior and increases with increasing inelastic behavior. For example, a ductility of 1.5 would have the effect of reducing accelerations of elastically calculated response spectra by as much as 1/3. Here ductility is defined as the ratio of displacement level in the nonlinear range to the displacement associated with the yield point for an elastic/perfectly plastic resistance vs. displacement function.

Conservatisms in the structural and mechanical resistance.

a. Allowable stress limits.

   Engineering codes specify "code minimum strength" for materials. These code minimum strengths are in turn specified by the applicant when the materials are ordered; any material found to be under that strength is rejected. The result is that the material supplier provides material of higher strength. Also, margins exist between allowable stresses and ultimate strengths.

b. 28-day concrete strength (structural only).

   Designs are usually based upon the 28-day design strength of concrete. Concrete continues to gain strength with increasing time beyond 28 days. Additionally, the strength at 28 days often exceeds the called-for design strength.

c. Static strength vs. dynamic resistance.

Code material strengths are based upon static load tests. Since dynamic loads contain a limited amount of energy and are applied at a faster rate, the margin between stress limits and failure for dynamic loads is greater than that for static loads.

d. Standard size structural members and pipes.

The design of the structural elements is such that their capacities usually exceed the requirements called for by the analyses. Much of the actual structural design is controlled by the availability of standard structural members such as beams and piping sections, so that larger sizes than are needed are often used.

e. Redundancy in indeterminite structures and components allows for redistribution of loads.

From the standpoint of function, major structures and components can tolerate much deformation, and typically failure of numerous structural members. This deformation and loss of structural members can be sustained because of redundancy, (i.e., more than one path available to carry loads) which allows for redistribution of loads formerly carried by failed members.

f. Ductility to failure.

In deforming to failure, beyond the elastic limit, the inelastic behavior of well engineered concrete and steel structures, components and systems provides for energy absorption normally counted on in design.

g. Minor attachments absorb energy.

Nonstructural elements which are not considered to carry any loads in design, do absorb energy through inelastic behavior or collapse during a seismic event.

Q.10 Does damping increase with increased nonlinear behavior of the structure?

A.10 (Mr. Herring) Yes, in the sense that "damping" is used to refer to increased energy absorbtion of the structure with increased nonlinear behavior.

Q.11 Can that increased "damping" be relied upon in determining conformance with the Trojan design criteria?

A.11 (Mr. Herring) No. This increase in "damping" (or energy absorption) is one of the items relied upon by the NRC in determining that it is not necessary to backfit the older plants to current seismic design requirements which have become more stringent with the evolution of NRC requirements. Also, it must be recognized that, on the basis of the results of the test program carried out by PGE in support of the proposed modifications, the proposed modifications will result in less conservatism inherent in the modified Complex than that which would have been present had there been no design deficiencies.

Q.12 Why are design rather than as-built material strengths now being used for capacity determinations for the modified Complex?

A.12 (Mr. Herring) These too are some of the conservatisms previously described as being relied upon by the NRC in determining that backfitting to current design requirements is not necessary.

Q.13 Your testimony for interim operation indicated that the structure was capable of resisting earthquakes in excess of 0.25g and, in fact, as high as 0.35g. What was the basis for your judgment in this regard?

A.13 (Mr. Herring) That judgment was based upon a 0.35g earthquake as defined by the Trojan FSAR seismic input criteria and an assessment of capacity based upon extrapolition of test data in existance at the time which was not directly applicable to Trojan. It was also based upon allowing for energy absorption through inelastic behavior of the structure. Only the structure was considered. While the structures and systems within the Complex are felt to be capable of resisting earthquakes in excess of 0.25g as defined by the Trojan criteria, at some level below 0.35g there may be local failures of piping and equipment supports which were not factored into this consideration, and the type and extent of these potential failures were not analyzed.

Q.14 Does the Complex in its present configuration have appropriate margins to substantially meet the FSAR commitments?

A.14 (Mr. Herring) No.

Q.15 Are the margins which are present adequate to provide for operation of the facility for the remaining duration of its operating license?

A.15 (Mr. Herring) No.

Q.16 Explain why they are not.

A.16 (Mr. Herring) As I discussed at the December 28, 1979 hearing session, it was determined to be acceptable for the facility to operate at reduced margin until appropriate modifications could be made to substantially restore the margins suggested by the initial design criteria. The time period necessary to implement the modifications, and that during which the margins would be reduced, is substantially shorter than the time remaining until expiration of the Trojan operating license. The concept of overall risk, as is ingrained in load combinations, provides the basis for this.

Q.17 What is the time for which interim operation should be allowed?

A.17 (Mr. Herring) As I discussed at the December 28, 1979 hearing session, there is no explicit time limit although the length of interim operation is time dependent. Operation for a period on the order of about 3 to 4 years or so from the issuance of the May 26, 1978 Order would be appropriate.

II. STRUCTURAL ADEQUACY OF THE PROPOSED MODIFICATIONS

Unresolved Items in The Staff's SER of February 14, 1980

Q.18 Please identify and describe the significance of the unresolved items with regard to structural adequacy of the proposed modifications that are listed in the SER.

A.18 (Mr. Herring) The items identified in the SER as unresolved which have a bearing on determining the structural adequacy of the proposed modifications are:

(1) Method of accounting for the encased steel frame in deriving stiffnesses (SER §5.1.1.1, p. 63). In this regard, the effect of double curvature behavior had not been accounted for and it had not been shown that double curvature behavior would not occur. Moreover, the licensee's assumption that slip in the beam-column

connections would be sufficient to develop twice the AISC allowables was not demonstrated as valid. The significance of this is that the stiffness derivation for the modified Complex had not been shown to be adequate insofar as stiffness is dependent upon proper treatment of the encased steel frame.

(2) Dead Load Determination (SER §5.1.1.3, p.65). Stiffness of the structural elements is proportional to the normal forces on the elements and the normal forces are dependent upon the dead load and the vertical earthquake components. In this regard:

(a) The effect on dead load of creep and shrinkage had not been adequately quantified;

(b) The assumed value of shrinkage strain had not been adequately considered;

(c) Stiffening of beams due to encasement in concrete and the effect of this on dead load had not been properly considered; and

(d) The effect of a 50°F change in mean temperature on dead load reduction for exterior walls had not been addressed.

Each of these matters will affect the dead load in walls. Without properly determined dead load, normal forces and, therefore, stiffnesses cannot be correctly determined.

(3) Gross Bending Moment Effects on Stiffness (SER §5.1.1.3, pp. 66, 68). Gross bending moments from an earthquake will cause shifting in wall normal forces and, therefore, shifting in stiffnesses. Any tension induced in walls from this gross bending moment effect must be shown not to be detrimental to stiffness over a number of cycles (the licensee's test program accounted only for compression, not for tension effects). The effect on stiffness of tension and cycles of tension from the gross bending moments must be quantified before stiffness of the modified Complex can be adequately known.

(4) Single vs. Double Curvature Mode of Failure (SER §5.1.1.4, p.68). The licensee's test program did not demonstrate the actual behavior of walls in the Complex and whether the single or double curvature mode of failure would occur. Stiffness is dependent upon whether walls behave in the single or double curvature mode. Consequently, because neither mode of behavior has been fully demonstrated to be applicable to the Complex exclusive of the other, both modes of behavior and their effects on stiffness should be accounted for.

(5) Capacities of new structural elements (walls and plate) - slippage and the coefficient of friction between steel and concrete (SER §5.2.1, p.69). Stiffness in the structure will decrease due to overturning moments and single curvature behavior. The new structural elements must be capable of withstanding these effects and this, in turn, is dependent upon slippage and frictional resistance. The use of a 0.7 coefficient of friction between the steel plate and concrete, relied upon to transmit seismic forces to the plate, required justification. Similarly, the resistance to sliding between columns and footings needed to be justified. The capacities of the new structural elements, and therefore of the modified Complex, is dependent upon a demonstration that the slippage and friction assumptions made are justified.

(6) Capacities relied upon can be developed (SER §5.2.2.1, p.71). Each wall panel in the Complex must be capable of carrying the forces calculated and relied upon to be withstood for the flexure, sliding and diagonal tension (shear) modes of failure. This must be verified for each element of the modified Complex before a conclusion on the capacity of the modified Complex may be reached.

(7) Flexure Mode of Failure and Flexural Capacities (SER §5.2.2.1, pp. 72-73). For a proper determination of flexural loads and capacities, the following items had to be resolved:

(a) Dead load contribution to normal forces will affect flexure loads and capacities. Consequently, those unresolved items delineated in items (2) and (3) as affecting dead load had to be resolved in order that the proper dead load contribution to normal forces and the effects on flexure loads and capacities can be determined;

(b) Dead load effect on flexure capacity of individual wall panels. While the dead load contributes only 6% of overall flexure capacity for the entire structure and, thus, the unresolved items with regard to dead load should have little effect on the overall flexure capacity of the structure, the effects on flexure capacity of individual wall panels could be more significant. Such effects on individual panels had to be examined;

(c) If single curvature behavior is assumed, certain displacements must take place in order to develop the necessary resistance to flexure failure. It must be shown that these displacements can take place and that they are compatible with the deformations of the structure. In addition, if the required displacements do occur, the resulting vertical shear forces at some places on the R and N walls may exceed capacities. The acceptability of this had to be demonstrated.

(8) Sliding Mode of failure and sliding capacities (SER §5.2.2.1, p.73). Shear friction contributes to the resistance to sliding failure. The licensee's formulation of the shear friction resistance to sliding failure was inadequate and gave too large a resistance for the Trojan walls. An appropriate relationship for the shear friction resistance to sliding had to be used before a correct determination of capacity against sliding could be made. Also, this resistance mechanism is affected by the matters discussed in items (2) and (3) above.

(9) Displacements as affected by stiffness, frictional resistance to sliding and gross overturning moment effects (SER §5.2.3, p.74). The elastic displacements determined from the STARDYNE model may be increased by stiffness degradation, frictional resistance to sliding and gross overturning moment effects. Thus, the following unresolved matters were identified as affecting final calculated displacements:

(a) Stiffness effects. Displacement depends on stiffness but unresolved matters remained with regard to stiffness derivations as indicated under items (1), (2), (3) and (4).

(b) Shear friction. Shear friction resistance to sliding will affect displacements and the inadequacy in the shear friction resistance formulation described in item (8) had to be corrected.

(c) Gross overturning moments. Gross overturning moments affect displacements but such effects had not been addressed.

(10) Floor response spectra as affected by stiffness (SER §5.3, pp. 74, 75). The floor response spectra for the modified Complex is dependent upon stiffness and stiffness degradation. Before final floor response spectra could be properly derived, the unresolved matters regarding stiffness as described in items (1), (2), (3) and (4) had to be satisfactorily resolved.

(11) Cyclic effects (SER §5.5). The cyclic effects of the occurrence of multiple earthquakes will degrade the stiffness of the structure. The unresolved items with regard to stiffness described in items (1), (2), (3) and (4) had to be resolved and accounted for before the ability of the modified Complex to withstand multiple earthquakes could be finally determined.

(12) Shrinkage values considered, the method of consideration and the increase in shrinkage with decreasing wall thickness (SER §5.6, p. 76). Shrinkage increases as wall thickness decreases. This phenomenon should be addressed to assure that shrinkage, which is important because of the encased steel frame and which has substantial effects on dead load, is adequately accounted for.

(13) Final capacity to force ratios and wall degradation (SER §5.12, p.83). The unresolved matters described in items (1)-(8) and (12) will affect the determination of capacities and forces for walls in the Complex. Final capacity to force ratios are needed before the potential effects of wall degradation for walls with capacity to force ratios less than one, and the effects of such degradation on equipment in the vicinity of those walls can be finally determined.

Q.19 As to unresolved item 2(a), provide the bases for your determination, as expressed in the SER, that up to 140×10^{-6} in./in. is an appropriate value of restrained shrinkage strain to be use in calculating the maximum dead load reductions to be expected for the existing walls.

A.19 Shrinkage is a volume change of concrete and is an inelastic deformation that is caused by a loss of water as curing progresses. It is a complicated phenomenon that is independent of externally applied loads and temperature imposed changes. The American Concrete Institute (ACI) suggests a method of calculating unrestrained shrinkage strain in its publication No. SP 27-13. Here, the unrestrained shrinkage strain is a function of ultimate shrinkage strain, time, humidity, member thickness, slump, cement content, percent fines and air content. This unrestrained value will be modified by any restraints in the actual situation, such as rebars. A method for performing this calculation is presented by Park and Paulay in their book "Reinforced Concrete Structures" where restrained shrinkage strain is calculated for a section of concrete restrained by rebars. Both of the above references deal with only reinforced concrete walls. The licensee has utilized the approach suggested by the above references. However, complications arise due to the introduction of the masonry wythes which sandwich the concrete core. The above references do not address specifically this situation and therefore judgments must be made.

In the licensee's calculation of restrained shrinkage, the masonry wythes were counted in the overall wall thickness similar to concrete and also as restraint similar to rebar. The thicker a wall is, the lower the shrinkage strain will be at a given time. It appears appropriate to count the masonry in obtaining wall thickness because it will obstruct the flow of moisture to the atmosphere which will be at the face of the masonry and thus lessen shrinkage. However, the pre-shrunk masonry blocks will expand as they contact the fresh concrete in the core and the water begins to flow through the masonry to the air surface. Shrinkage is a reversible phenomenon. The masonry block will then shrink again as moisture leaves the wall system at the block - air interface. Thus, the block behaves differently than the rebar and any restraint it offers is difficult to estimate considering long term effects.

Taking the 70×10^{-6} in./in. as calculated by the licensee and counting the masonry in overall wall thickness but not as restraint, one obtains 123×10^{-6} in./in. Taking a two dimensional effect into account through Poissons ratio of $\mu = .15$ yields a restrained strain of as much as 141×10^{-6} in./in. This value is for a 30" thick wall and will increase as wall thickness decreases. In addition, a Poisson's ratio of 0.21 was previously indicated by PGE as being appropriate for the in-situ walls.

Thus a value of restrained shrinkage strain of 140×10^{-6} in./in. appears to be a reasonably conservative value. Additionally, although even discontinuous core steel will provide restraint to shrinkage above that relied on above, substantial reliance cannot be placed on the existence of core steel in the composite walls throughout the Complex to resist shrinkage, although some may be discontinuous. The March 20, 1980 letter from PGE to the NRC regarding reinforcing steel in the Complex shear walls indicates that no composite wall panels in the Fuel Building contain any (vertical or horizontal) reinforcing steel in the concrete core, few (about 15%) of the composite wall panels in the Auxiliary Building contain any (vertical or horizontal) reinforcing steel in the concrete core, and only about 60% of the panels above Elevation 93' and about 94% of the panels below Elevation 93 in the Control Building contain any (vertical or horizontal) core reinforcing steel based on calculations performed by the Staff.

Q.19A What reliance can be placed on bond between concrete in a wall and the steel columns at Trojan?

A.19A Although there may be some bond between the columns and the concrete, for the reasons set forth below, the bond may not exist and therefore it should not be relied upon and should be looked at both assuming bond and assuming no bond. It is stated in a number of references that bond between concrete and steel is negligible. For example, "Reinforced Concrete Structures" by Park and Paulay states "The bond resistance of plain bars is often thought of as chemical adhesion between mortar paste and bar surface. However, even low stresses will cause sufficient slip to break the adhesion between the concrete and the steel." Winter and Nilson in "Design of Concrete Strctures" state in reference to plain bars "initial bond strength was provided only by the relatively weak chemical adhesion and mechanical friction between steel and concrete." In a report by the Missouri State Highway Department entitled, "Pushout Tests with High Strength Bolt Shear Connectors," rolled steel sections were investigated for the effects of bond. Pushout tests were performed in which a W8 x 48 steel beam with sandblasted surfaces was connected to concrete slabs by means of bolts or studs and was in contact with the slab. The concrete was cast around the bolts and was in contact with the W8 x 48 as it cured. One test was to determine the effect of natural bond. The test indicated that little or no natural bond was present and that "natural composite behavior observed in the non-composite concrete-steel members is due to friction resulting from the weight of the slab rather than natural bond." A conclusion of the report is that "little or no bond exists between concrete slabs and steel beams." Pushout tests to determine bond between concrete and rolled steel were performed by Bryson and Mathey for different surface preparations. Here, a portion of a rolled shape was completely embedded in concrete as opposed to slabs on flanges. The results indicate freshly sandblasted specimens exhibit around 455 psi bond stress and normal rust and mill scale specimens around 317 psi.

At Trojan, the existing columns would have a surface similar to normal mill scale. Due to the way in which the surfaces are being prepared, the exposed columns would be expected to be somewhere between sandblasting and normal mill scale before concrete is replaced.

Bresler states that this type of bond is reliable, however, where concrete slabs are supported on I-beams, the bond is considered unreliable. It would appear that shrinkage of concrete placed uniformly around a completely embedded specimen would increase bond by forming a clamping mechanism. The effect is absent in T-beams. Shrinkage of slabs on T-beams would tend to shear and somewhat pull the concrete away from the steel.

At Trojan, for the in-situ walls, cracks in the masonry along columns indicate shrinkage of the wall in the in-plane direction. This would tend to pull the concrete away from the steel column similar to slabs on I-beams and reduce or eliminate any natural bond. The shrinkage of the new concrete walls being added would tend to cause a similar effect. It was stated by PGE that test specimens L1 and L2 exhibited higher strengths due to bond. The specimens failed in single curvature by yielding of the embedded columns in tension thus indicating higher strength due to additional steel similar to rebars -- not due to bond. The columns would have been effective with or without bond under these circumstances. A similar behavior was demonstrated by the specimen containing the unbonded struts. Also, thermal effects on exterior walls would create stresses at the wall/steel column interfaces due to restraint of the intersecting interior walls and the slightly different coefficients of thermal expansion for concrete and steel, .00055 and .00065 in./in./100°F, respectively. This too would effect bond.

For these reasons, although there may be bond between steel columns and concrete at Trojan, it is not a reliable mechanism either for strength or as a source of conservatism.

Q.19B Did the Bechtel test specimen in themselves demonstrate that the in-situ walls are capable of withstanding without delamination the simultaneous application of in-plane (horizontal and vertical) and lateral seismic loading, in appropriate combination with other loads?

A.19B No.

Q.19C Why not?

A.19C The test specimens were tested such that both the inner and outer masonry wythes and the inner concrete cores were subject to the same displacement and were more uniformly loaded in-plane than would be the case for the in-situ walls. Such out-of-plane lateral loads would induce stresses which would tend to induce delamination. For the in-situ walls where the beam-column connection is relied upon to resist single curvature failure, the load path for this resistance mechanism has significant differences from that in the test specimen. First of all, the force from the beam-column connection for the in-situ walls must be transmitted through the core to the outer wythes, unless the beam is in contact with all wythes which is not the case for most major composite walls.

This will induce shear stresses of the wythe interfaces. Secondly, the force from shear friction of the horizontal steel in the outer block wythes must be transmitted to the concrete core, thereby inducing shear stresses at the wythe interfaces. These were not represented in the test specimens. No test specimens had a beam embedded in the core nor horizontal steel continuous past the columns to an adjacent panel. Only two specimens had embedded columns in the massive upper and lower loading beams. (The effect of an encased beam is addressed in the 2/13/80 PGE submittal, PGE Exhibit 25 Q.) For the in-situ walls, the encased steel beams provide different mechanisms for transfer of shear force at the combined steel/concrete, concrete/concrete and masonry/concrete interfaces which were not modeled by using the upper and lower concrete beams for loading of the test specimen. The test specimen did show that large amounts of equal displacement of all panels could be sustained without inducing delamination. In the test program, no lateral loads were applied which would induce delamination. It is recognized that the purpose of the testing program was to test only in-plane capacity. To conclude from the test program that delamination will not occur in the in-situ walls would ignore these additional, important factors.

The adequacy of the walls to resist out-of-plane loads without delamination is addressed in the Staff's SER on the "Wall Problem".

Q.19D Comment on the adequacy of the analysis submitted in the 2/13/80 PGE submittal (PGe Exhibit 25 Q) regarding wythe interface shear stresses in composite walls resulting from resistance to in-plane loads.

A.19D The referenced PGE response relates to the ability of the multi-wythe wall system to transfer shear stress at the junction of the concrete core and masonry wall. The case where the steel beam is completely encased in the concrete core and the flanges do not lie in the masonry is judged to be most critical and is analyzed. It is important to remember that what we actually have at Trojan in many walls is a concrete core in which is embedded steel columns and beams. The core and steel frame is sandwiched by masonry walls with the steel framing interrupting only the core concrete. This is as opposed to a complete in-fill panel where the steel framing would interupt all three wythes and possibly exhibit different behavior.

It must be shown that there is adequate ability to transfer shear between wythes since the two resistance mechanisms (beam-column connection and shear friction) exist in different wythes and the only connection between the wythes is the shear transfer ability at the interface. If this shear transfer ability does not adequately exist so that the wythes do not act together, it is possible to have the masonry wythes behave independently of the concrete core. It would be expected that the major effect of total delamination would be degradation of the out-of-plane resistance over the in-plane resistance.

The test program included two tests where there were embedded columns but in no tests were there embedded beams in addition to columns to simulate the in-situ walls. In the test, load was applied to all three wythes simultaneously. Because no test specimen existed to verify whether this delamination failure will occur, analytic methods were employed in the answer to the above referenced question.

In the answer, a small, local zone under the beam-column connection has been identified where failure will occur. Failure was defined as the area where interface shear stresses exceed 150 pounds per square inch. This shear value, although in the range of tensile values obtained from pull-testing performed at the plant, has yet to be confirmed. Also, it was stated that the stress at the interface has the ability of stress redistribution once failure has occurred in the local area. This would cause a larger area to fail progressively until a large enough area is exposed at the interface to arrest this failure. The reinforcing ties would not provide for a large resistance through shear friction due to the fact that there are only #3 ties (reinforcing bars with a nominal 3/8 inch diameter) spaced both horizontally and vertically at 4 foot intervals. As noted in the response, the in-situ cases where the steel beam is completely encased by concrete and the concrete extends over the top of the masonry wythes would lessen this interface stress.

The 150 psi failure value is a reasonable value and failure between wythes and core should remain in a local area. The confirmatory long range testing program will address the concerns raised above, namely, delamination due to embedded beams and columns, failure interface shear stress value, and combination of in-plane and out-of-plane loads. The adequacy of the walls to resist out-of-plane loads without delamination is addressed in the Staff's SER on the "Wall Problem."

Q.19E Do you agree with the use of friction coefficients against initial slip of 0.7 for the steel plate against grout and 1.0 for the new concrete placed against the existing walls?

A.19E No. This is discussed in detail in Section 5.2.1 of Staff Exhibit 13 A (SER).

Q.19F What is currently being used by PGE?

A.19F The values quoted above were used initially and a factor of safety of two (2) was applied, thereby resulting in the effective use of 0.35 and 0.5 for the coefficients of friction between steel/concrete and new concrete/old concrete, respectively, for the unfactored OBE loads. To resist a factored OBE load, with a capacity reduction factor of 0.85 for shear, the coefficients of friction needed are 0.58 and 0.82, rather than the 0.7 and 1.0, respectively. Since the steel plate on the R wall is being roughened and the existing concrete block face on the N line wall is being bush hammered to increase frictional resistance, we feel that these reduced coefficients (0.58 and 0.82) of friction for initial slip are reasonable.

Q.19G What is the effect of using these lower coefficients of friction?

A.19G The effects of the use of these reduced coefficients of friction are to reduce capacity to force ratios to 1.4 for the unfactored OBE, rather than 2, not considering the effects of gross bending, dead load reduction and the neglect of the encased steel frame. Consideration of these effects would reduce the 1.4 to something less, as given in the April 14, 1980 PGE submittal; however, the capacity against initial slip remains greater than the demand under an unfactored OBE. The ratio is greater if the effect of dead load reduction is neglected. On the N line wall they are greater than 1.4 but somewhat less than 1.4 on the R line wall. Furthermore, these friction coefficients would increase to those initially assumed if some slip takes place. Therefore, the proposed bolting arrangements are adequate.

Q.19H Has the licensee provided final structural displacements for the factored OBE loads?

A.19H No. This condition was addressed in PGE responses to May 1979 Staff questions. However, in the March 17, 1980 licensee submittal (PGE Exhibit 25 U), PGE states that the "prediction of structural displacements for factored OBE load is neither an explicit nor an implicit criteria of the FSAR. Furthermore, additional consideration of all the postulated events causing structural nonlinearity should not be the basis to address the factored OBE loading as regards the displacement." While this may not be a clearly explicit criteria, it is an implicit criteria since it must be assured that the displacement level at which the required resistance is reached is attainable or else the required resistance will not be developed. This is especially important for the Complex, given the large displacement required for the test specimens to attain ultimate capacities. If the displacement required to attain the resistance exceeds the gap between adajcent structures given proper consideration of the displacements of these structures, then the structures would impact and the extra capacity could not be shown to develop with the current analyses which assume no building impact.

Q.19I Has this been satisfactorily addressed by PGE?

A.19I Yes. PGE Exhibit 32 for identification and the April 14, 1980 submittal indicated that adequate margins (a factor of at least 9.52 on the elastic STARDYNE displacements) exist to preclude building contact given the loading of the Complex with the factored OBE loads.

Q.20 What is the status of resolution of the unresolved matters that you have identified above in response to question 18?

A.20 (Mr. Herring) Based on a review of additional analyses, evaluations and testimony and of the analyses and evaluations submitted by the licensee in its March 17, 1980 responses to Staff questions of March 7, 1980 (PGE Exhibit 25 U), PGE Exhibit 32 for identification, and the April 14, 1980 submittal in response to additional Staff questions, we have determined that all of the matters previously identified as unresolved are now resolved. Specifically:

(1) As to the method for accounting for the encased steel frame (beam-column connection), the concern raised was that more credit was taken for the beam-column connections than should have been. This has been resolved since PGE Exhibit 25 U, PGE Exhibit 32 for identification and the licensee's April 14, 1980 submittal demonstrate that the modifications are adequate even neglecting the contribution of the beam-column connections (or encased steel frame).

(2) and (3) Dead load determination and gross bending moment effects on stiffness. In the licensee's analyses referenced above, the licensee accounted for the effects of 140×10^{-6}

in./in. shrinkage strain resulting in reduced dead load. Furthermore, the local effects of a 50°F mean temperature change for exterior walls was considered.

In addition, the licensee demonstrated in its April 14, 1980 submittal that, based on the results of its test program, the majority of walls will most likely experience axial growth which would more than offset any postulated shortening due to creep and shrinkage. The licensee's analyses referenced above demonstrate that, although there may be some local reductions in capacity to force ratios below 1.4 for the unfactored OBE, the overall structural integrity will be maintained to the required level under the combination of dead load reduction and gross bending effects and neglecting the encased steel frame. It has been shown that any required load redistributions are capable of taking place. If the dead load reduction is not as great as assumed in these analyses, the local effects are less significant and the degree of redistribution less.

(4) As to single and double curvature, both of these modes of behavior have been accounted for in the determination of capacities, frequency shifts in response spectra and displacements, and the effects have been shown to be acceptable.

(5) Capacities of new structural elements. Revised capacity to force ratios for the new structural elements (new walls and

the steel plate) were determined and these elements themselves were shown to have capacity to force ratios for the unfactored OBE of greater than 1.4. The margin with respect to the initial slip between the steel plate and wall on the R line and between the new wall and the existing wall on the N line could be less than 1.4 for the unfactored OBE. However, the acceptability of this is discussed in response to the previous questions 19E, 19F and 19G.

(6) Capacities relied upon can be developed. The licensee has demonstrated that no panel will have a capacity to force ratio for an unfactored OBE of less than 1.1 to 1.2. Moreover, no major wall has a capacity to force ratio for the entire wall of less than 1.4 for the unfactored OBE. Even for the factored OBE, any necessary redistribution of forces will take place. Thus, this matter is resolved.

(7) and (8) Flexural and Sliding Modes and Capacities. The resolution of items (1), (2), (3), (4) and (5) has resulted in the resolution of these items. In addition, the matters raised by items (7) and (8) were assessed in light of the capacity determinations for all 3 modes of behavior (flexure, sliding and diagonal tension) as set forth in response 1 (a and e) of PGE Exhibit 25 U with appropriate capacity reduction factors. The equations presented have been applied to all walls in the Complex and ultimate shear stress was limited to 300 psi for composite walls and 150 psi for double block walls.

(9) Displacem   n as affected by stiffness, frictional resistance to sliding and gross overturning moment effects. PGE Exhibits 25 U and Exhibit 32 for identification as well as the criteria set forth in other PGE submittals in the Phase II proceeding demonstrate that, considering all effects raised by this open item, the elastic displacements increase by a factor of 2.1. The Control Building can displace 9.52 times the elastic displacements without building contact at any point. Therefore, this additional margin is sufficient to provide reasonable assurance that building contact will be precluded even under a factored OBE. As set forth below, the stiffness derivation adequately accounted for the degradation due to 5 OBEs and 1 SSE. Displacements are directly proportional to stiffness. The displacement determinations discussed here have also accounted for the stiffness degradation that would occur for 5 OBEs and 1 SSE. This is discussed in item (11) of this response.

(10) Floor response spectra as affected by stiffness. The elastically calculated frequencies based upon consideration of wall degradation per PGE-1020, Appendix B criteria will be broadened 41% on the low side and 10% on the high side. This is adequate to account for the effects on stiffness and stiffness degradation identified in this unresolved item.

(11) Cyclic Effects. Standard Review Plan (SRP) Section 3.7.3.II.2 states "During the plant life at least one safe shutdown earthquake (SSE) and five operating basis earthquakes (OBE) should be assumed, the number of cycles per earthquake should be obtained from the synthetic time history (with a minimum duration of 10 seconds) used for the system analysis, or a minimum of 10 maximum stress cycles per earthquake may be assumed." Even though these are explicitly applicable to piping, equipment and components, as discussed in the SER, it was felt to be reasonable and somewhat conservative to apply these to the structure, considering the cyclic degradation indicated by the test data and the corresponding effects on frequency shifts of the floor response spectra. PGE only analyzed the effects of five (5) OBEs without additionally considering the effects of one (1) SSE which is required by this SRP Section, all having ten (10) effective full stress cycles. However, the effects of 60 full stress cycles (five OBEs and one SSE) on the "growing" of the walls under lateral loads was appropriately addressed in the April 14, 1980 PGE submittal.

There is no effect on the peak broadening of considering the one (1) SSE in addition to the five (5) OBEs. Presently, the peak is being broadened 16.6% to the low side to account for 50 full stress cycles due to multiple OBEs as discussed in the response to question 2 of PGE Exhibit 25 U. Using the methodology presented in the licensee's 12/21/79 response to the Staff's 10/2/79 question #21 which is appropriate, the Staff has calculated the peak broadening for 60 full stress cycles with a graph felt to be more representative and conservative. Using both 2 and 3 cycles as a base, peak broadenings to the low side of 16.5% and 15.2%, respectively, have been calculated by the Staff for 60 full stress cycles. Thus, the broadening of 16.6% being used by the licensee encompasses values the Staff believes to be appropriate for 60 full stress cycles. The 31% peak broadening to the low side, in addition to the initial 10%, for a total of 41% is acceptable. Consideration of the lower percentages calculated above would only indicate about one half of one percent reduction in the amount already considered, which is negligible.

(12) Shrinkage increase with decreasing wall thickness. This matter was adequately accounted for through demonstration that the consequences of 140×10^{-6} in./in. shrinkage, were it to occur, were acceptable and that the growth of the walls from cracking will more than offset the shrinkage effects. The 140×10^{-6} in./in., which is appropriate for a 30-inch wall, is also representative of the entire structure which consists mostly of 30-inch walls and some walls of

greater thickness (for which shrinkage would be less than 140×10^{-6} in./in.) and some walls of lesser thickness (for which shrinkage would be greater than 140×10^{-6} in./in.).

(13) Final capacity to force ratios. It is impossible to state precise capacity to force ratios for the walls given that exact behavior of the structure cannot be predicted. The potential effects on capacity to force ratios due to the matters referred to have been properly accounted for and considered in PGE Exhibit 25 U, PGE Exhibit 32 for identification and the licensee's April 14, 1980 submittal. We find the results of these additional studies acceptable and that the intent of the Order of May 26, 1978 is met. Furthermore, the maximum rebar strains, per the April 14, 1980 PGE submittal, in any panel would be about 3 times the yield strain which are not excessive and would not result in detrimental cracking of the wall. This matter is resolved.

LICENSING BOARD QUESTIONS

Q.21 At the Prehearing Conference held in this proceeding on March 11, 1980, the Licensing Board set forth a number of questions bearing on the structural adequacy of the modified Complex. With regard to the criteria for determining whether the proposed modifications will substantially restore the seismic margins and bring the Control

Building into substantial compliance with the Trojan license, the Board asked:

(1) What are the criteria that we should use to assure that the Control Building is brought into substantial compliance and the intended margins met? (Tr. 3531).

(2) On what basis will it be determined that the modified structure will have increased seismic capacity to safely resist the 0.15g OBE forces with the margins inherent in the original design criteria? (Tr. 3531-32).

(3) How do you assure yourself that you have met the original design criteria and are in substantial compliance with that criteria as set out in the technical specifications? (Tr. 3532).

Please respond to these Licensing Board questions.

A.21 The basic seismic design requirements for the Complex have been set forth in Section 3 of the Staff's SER. This Section references the appropriate portions of the Trojan FSAR, as referenced by Trojan Technical Specification 5.7.1, and discusses the degree to which they are met, as determined by the NRC Staff review. Rather than demonstrating substantial literal compliance with all appropriate design requirements, the results of a testing program were implemented by the licensee to demonstrate the capability of the in-situ walls. Given the required seismic input definition, it must be demonstrated that sufficient margin exists in the modified Complex, including the new and existing walls, to resist the loads resulting from the use of these inputs, and that uncertainties over the actual behavior of the structure when extrapolating the results of the testing program to the behavior of the in-situ walls are adequately accounted for. These uncertainties arise from the effects of the encased frame on lateral load resistance, the higher stress levels being present in the walls than would have resulted from appropriate design of the structure initially, and the sensitivity of the stiffnesses and capacities to the parameters contributing to them, as indicated by the testing program. It should be noted that FSAR Section 3.8.1-5.1 specifically states that the structural steel framing for the Control, Auxiliary and Fuel Buildings was initially intended to carry only vertical loads, while the lateral loads due to earthquake, wind, and tornado are resisted by reinforced concrete and concrete block shear walls. However, the encased steel frame is now being relied upon to supply lateral load resistance. Since this element of conservatism which was present in the original design but was not relied upon is now being relied upon, it requires a careful assessment of the structural response and the capacity to force ratios for the walls.

Q.21A At the hearing sessions in this proceeding during the week of March 31, 1980, the Licensing Board inquired of the licensee's structural witnesses whether the licensee's statement of the criteria by which it will be judged whether the proposed modifications will substantially restore the desired and intended seismic margins and bring substantial compliance with the Trojan license (PGE Exhibit 30, Response to Dr. McCollom's questions

1, 2 and 3) differs from the Staff's statement of criteria in this regard. Please summarize what you believe the criteria to be and indicate whether this differs from that set forth by the licensee.

A.21A In essence, it is the Staff's view that proposed modifications will substantially restore the intended seismic margins and bring substantial compliance with the Trojan license requirements if it is demonstated that such proposed modifications bring substantial compliance with the seismic design requirements of the Trojan FSAR as referenced by Technical Specification 5.7.1 or, where substantial literal compliance with those requirements is not or cannot be met, that acceptable compensatory equivalent requirements are met with all uncertainties associated with meeting these alternate requirements adequately accounted for.

In our view, there is no major difference between this standard and that set forth by the licensee. Any difference between the Staff and the licensee really arises from a difference in interpretation as to how this standard may be met; specifically:

(1) There is a difference of opinion as to how much credence can be placed in STARDYNE to justify departures from Code criteria. It is the Staff's view that, while STARDYNE is a sophisticated analytical tool that can be used to accurately predict the elastically determined forces in the Complex, the use of STARDYNE alone does not justify departures from Code criteria. Rather, STARDYNE itself has limitations which must be taken into account.

(2) There has been a difference of opinion between the Staff and the licensee as to the degree to which uncertainties in structural behavior and uncertainties in extrapolating results from the licensee's test program must be accounted for.

However, these "differences of opinion" are not issues or matters which would preclude a finding that the proposed modifications are adequate because, in evaluating the adequacy of the modifications, the licensee has properly accounted for the limitations in STARDYNE and for the uncertainties in structural behavior and in applying the test program results, despite the licensee's view that, in many respects, these limitations and uncertainties need not be considered. The licensee's analyses have shown that while some capacity to force ratios for individual wall panels for the unfactored OBE will be less than 1.4 under the worst possible combinations of dead load reduction, gross bending and single and double curvature behavior, redistribution of forces in the wall will occur so that the capacity to force ratio for the entire wall will not fall below 1.4. Thus, the walls will maintain substantial margins in capacity even when uncertainties in structural behavior and application of test results are considered by analyzing for the worst combinations of loading and structural behavior dictated by the uncertainties. From the standpoint of frequency calculations, floor response spectra and equipment qualification, the uncertainties have been accounted for by adequate peak broadening. The Staff has concluded that the proposed modifications will substantially restore the intended seismic margins and bring substantial compliance with license requirements based on the licensee's STARDYNE analysis in conjunction with the test program and the additional analyses to account for the uncertainties discussed above. Thus, while there are differences of opinion as to the degree to which uncertainties had to be accounted for, the licensee nevertheless accounted for those uncertainties to the degree thought necessary by the Staff and demonstrated the adequacy of the modifications in that regard.

Q. 22 With regard to the original design of the Complex, the Licensing Board asked:

(9) How was the construction of the composite walls taken into consideration in meeting the building code requirements for the original specifications and construction when there was no Uniform Building Code requirement appropriate for that kind of construction? (Tr. 3533).

Please respond to this Licensing Board question.

A.22 As indicated in the PGE submittals regarding the proposed Control Building modifications, there were no explicit Code (ACI or UBC) requirements for the composite (major shear walls) in the Complex. The initial design concept for these composite walls for in-plane loads neglected the area of the block and relied upon only the equivalent wall thickness, determined from considering the cell grout and concrete core, subject to the Ultimate Strength Design requirements of ACI 318-63. Out-of-plane wall capacities were based on ACI 318-63 Ultimate Strength Design concepts. The single wythe and mortared double wythe masonry block walls were designed using the load combinations for reinforced concrete, including the load factors, and the allowable stress was defined as an increase factor times the UBC allowable stresses. The details of the criteria are summarized in the response to NRC question 2 in PGE's December 31, 1979 submittal regarding the "wall problem".

Q.23 With regard to demonstrating the adequacy of the modified Complex under the building codes, the Licensing Board stated:

(4) We should know just how the building codes permit this kind of test results to be used in meeting the code specifications. (Tr. 3532)

(12) Show whether the Uniform Building Code provides for the use of, and allows accounting for, the more sophisticated analyses of the seismic forces provided by the STARDYNE analysis. (Tr. 3533)

Please provide your responses.

A.23 Sections 106 and 107 of UBC 1967 allow for the determination of structural strengths based upon testing. This provision is also included in Section 104 of ACI 318-63. Additionally, much of the design criteria within the codes are established through the evaluation of the results of testing programs. Guidance is not given in either the UBC or the ACI Codes regarding acceptable methods of performing tests to establish design criteria for loads such as earthquake. Only static load tests of actual structures are explicitly addressed (see Section 24 of UBC 1967 and Chapter 2 of ACI 318-63).

While the ACI Code does allow for departure from certain design rules or formula on the basis of analysis, it is not felt that the STARDYNE analyses which have been performed for the determination of the loads in the various wall elements in the Complex is sufficient to qualify for reduction in allowable stresses for the thus determined loads. The Codes do require that the loads in structural elements be determined using sound principals of engineering mechanics. Appropriate consideration must be given to the complexity of the structure when choosing a particular analytical technique for load determination. Limitations on the chosen analytical technique should be recognized and adequately considered in determining the final load on a particular element for design purposes. Appropriate Code provisions should then be used to establish the design of the structural element. It is felt that the STARDYNE analyses can give appropriate load definition for the walls in the Complex, given that appropriate stiffnesses for the structural elements are used and that any uncertainties which cannot be incorporated into the STARDYNE analyses are adequately accounted for in the evaluation and designs of the structural elements.

Q.24 The Licensing Board made inquiries with regard to the testing program and results which are being used in the analysis of the modified Complex. In this regard, the Board asked:

(10) Show how a 32-inch type A wall with a vertical load of 105 psi. would have a unit shear capacity 18 percent greater than a similar 24-inch wall. (Tr. 3533).

Please respond.

A.24 The referenced example of an increase in unit shear capacity with increasing wall thickness which is the subject of this question was set forth in the initial version of PGE-1020, was deleted in Revision 1 to PGE-1020 and has not been reinstated in subsequent revisions to that document.

The current criteria for the walls involves an investigation of three modes of failure, namely flexural (both double & single curvature), sliding, and diagonal tension (shear) modes of failure. The unit shear stress capacities for the flexural modes are independent of wall thickness. Therefore, there would be no difference in this capacity for the thicker walls.

The sliding mode capacity is governed by vertical reinforcement ratio, embedded column capacity, and a contribution from the normal force which is a function of the ratio of the area of concrete to the area of mortared block. The latter contribution to capacity would increase with increasing core (wall) thickness. The exact percentage increase depends on the magnitude of the other contributing factors (i.e. vertical reinforcement ratio, embedded column capacity).

The diagonal tension capacity is a function of horizontal and vertical reinforcement ratios, the compressive stress on the wall, and the ratio of areas of block to areas of concrete cores. This capacity would increase with increasing core (wall) thickness, the exact percentage increase depending on the magnitude of the other contributing factors.

Each of these criteria is investigated and relied upon for each in-situ wall (see March 17, 1980 PGE submittal in response to Staff's March 7, 1980 requests for additional information). Therefore, this additional element of conservatism alluded to in the initial version of PGE-1020 is no longer present.

Q.25 (20) Determine if there is any reason to believe that there might be a scaling factor for the ability of walls to withstand seismic forces with the same aspect ratio but much larger as occurs in the Control Building compared to the test models. (Tr. 3534-35).

A.25 The test specimens were of sufficient size such that when it is considered that the walls of the Complex meet the height to thickness ratios of the UBC, it is felt that the criteria that have been developed based on the results of these test specimens are adequate for the in-situ walls. No scaling factor is necessary.

Q.26 Also with regard to the test program:

(21) Determine if there is a different effect on the stress versus the displacement in a wall if the frequencies of cycling were high as in an earthquake compared to the test frequencies used for the wall evaluations. (Tr. 3535)

A.26 The effects of higher frequency cyclic loading should reduce the stiffness degradation relative to that demonstrated by the licensee's test program in which specimens were tested in a psuedo-static manner. However, the effects of higher frequency loading cannot be quantified as they were not the subject of the licensee's test program.

Q.27 The Licensing Board made a number of inquiries with regard to the seismic analyses for the modified Complex and input to those analyses. Specifically

(5) How do you conclude that the earthquake ground response spectra has a vertical ground acceleration that's two-thirds of the horizontal acceleration? (Tr. 3532).

A.27 As stated in Section 3.2.1.1.1 of the Staff's SER, the design response spectra and peak ground acceleration are specified in Section 3.7.1.1 of the Trojan FSAR. As stated in the Staff's SER, these have been incorporated into the seismic analyses of the modified Complex. The referenced FSAR section defines the SSE and OBE response spectra as well as the peak ground accelerations to be assumed for the OBE and SSE in the vertical direction, namely 0.1g vertically for the OBE (or 2/3 of the 0.15g horizontal acceleration), and 0.17g vertically for the SSE (or 2/3 of the 0.25g horizontal acceleration).

Accordingly, the vertical ground accelerations are two-thirds of the horizontal accelerations under the licensed criteria for the plant.

Q.28 (7) The existing wall capacities are determined based on the testing results using the total dead load on the wall reduced by 20 percent to account for the vertical earthquake effect. How and why was that done? (Tr. 3532).

A.28 The dead load is being reduced by 13%, not 20%. The basis for this under OBE conditions is that the vertical rigid range OBE acceleration is 2/3 (.15g) = .1g, amplified by 30% to account for vertical building response. Thus, 1.3 x .1g = .13g or 13%g. This vertical motion fluctuates between ± 13%g, and is thus the basis for a 13% dead load reduction to account for vertical earthquake motion.

If an amplification of 16% (claimed by the licensee to be more representative such that the use of 13% dead load reduction is "conservative" relative to this more representative value) is used, this is reduced to about 0.12g (1.16 x 0.1g) or 12%g, thus indicating that the degree of "conservatism" is not substantial (i.e. 1%g).

Q.29 (11) Show how the structural steel columns in the shear walls will be used in determining the failure limitations of the Control Building walls, if any, or if it is used as a safety factor. (Tr. 3533).

A.29 The encased steel frame, which includes the steel columns and beams, is relied upon to provide lateral resistance in the determination of capacities for single curvature flexure behavior and sliding. In addition, the beam-column connections are relied upon to resist gross vertical shears along the column lines. Therefore, the columns and the beams are not being taken as added factors of conservatism. The encased framing, while contributing to the resistance against modes of failure other than double curvature, does not contribute to the double curvature capacity, nor to double curvature stiffness.

Q.30 (16) Provide an evaluation of the temperature coefficient of expansion effects that might take place between the steel plate and the concrete wall to which it is bound and tensioned once the concrete wall has reached full strength. (Tr. 3534).

A.30 The effects of temperature changes and thermal expansion have been factored into the design of the steel plate. The effects of temperature changes and thermal expansion have also been accounted for in the calculation of bolt tension losses, and the analyses of the effect of the plate on wall capacity.

Q.31 (17) Show whether there are any tension effects in the bolts that influence wall strength. (Tr. 3534).

A.31 In the analysis of the effect of the steel plate on strength of the Control Building west wall reliance is placed on 75% of the initial bolt preload to resist shear forces in the building. The 25% allowance was based upon consideration of the losses in bolt tension which will occur over time due to bolt relaxation, concrete creep and shrinkage and temperature effects. Accordingly, it is necessary to assure that, on the average, 75% of bolt preload is available and that this level of preload is uniform.

To provide such assurance an in-service inspection program should be required. Various in-service inspection programs for the bolts have been proposed by the licensee and commented upon by the Staff. The program proposed by the licensee in its April 14, 1980 submittal to the NRC has resulted. We have reviewed that proposed program and have concluded that it will provide assurance that the required bolt tension and the level of uniformity of preload will be maintained throughout the remainder of the plant's operating life. It is the Staff's view that the referenced Technical Specification and Bases for the inservice inspection and test program for bolts should be incorporated into the facility Technical Specifications so that it is made a license requirement.

Q.32 With regard to the rail stop to be installed in the Turbine Building railroad bay adjacent to the newly installed west wall of the Control Building, the Licensing Board stated:

> (18) Determine if there is any foreseeable possibility of an impact on the rail stop in the Turbine Building that might result in a force against the face of the Control Building if the stop were to fail and what this would do to the construction of the wall where it would impact the Control Building. (Tr. 3534).

Please respond.

A.32 An impact of a train on the rail stop cannot be totally ruled out. The consequences of hitting the rail stop is a function of both the weight and the speed of the train. Depending upon the weight of the train, the rail stop will prevent an impact on the Control Building west wall for a train traveling at very low speeds. At higher speeds and weights, the rail stop alone will not be adequate to stop a train and penetration of the west wall of the Control Building could occur. We have not performed an analysis to determine the precise combination of train speeds and weights that could result in a wall penetration nor have the consequences of such penetration been evaluated. Rather, as set forth in Section 3.7 of the Staff's SER and in "NRC Staff Testimony of Charles M. Trammell, III on Questions Regarding Relocation of the Railroad Spur and Reduction in Size of An Equipment Hatch Under the Proposed Modifications," filed on March 17, 1980, administrative controls should provide assurance that movement of trains on-site will be adequately controlled so as to preclude impacts that will result in damage to the Control Building.

CFSP CONTENTION 22

Q.33 CFPS Contention 22 states

> The effect of the steel plate on displacement in the Complex has not been completely analyzed.

What displacements can occur during an earthquake?

A.33 An earthquake will cause displacements of structures and components. These diplacements are time dependent and at any particular time during an earthquake, different parts of structures may undergo different displacements in different directions. From the standpoint of the effects of the steel plate, the displacements of concern are interstructure displacements - that is, relative displacements between two adjacent structures, in this case, between the Control Building and the Turbine Building.

Q.34 Why are such relative displacements a concern?

A.34 Depending on the magnitude and the direction of the displacements for each building, building contact could occur. For interim operation, an analysis was performed which demonstrated that the maximum relative displacements of structures caused by an earthquate were less than the gaps between structures, with margin, at all elevations, thereby assuring that building contact would not occur. The concern with installation of the steel plate on the west wall of the Control

Building under the modifications is that the three-inch thick plate will reduce the gap available between the Control and Turbine Buildings at elevations 93' and 69' where Turbine Building girders and floor slabs are located.

Q.35 What measures will be taken to compensate for the reduced gaps where the steel plate will be installed?

A.35 At elevation 93', three inches will be removed from the flange of the steel girder in order to increase the gaps between the installed steel plate and the slab and girder. At elevation 69', 18 inches of the overhanging part of a concrete slab will be removed. These modifications will provide a gap of about four inches for displacements in the north-south (N-S) direction (parallel to the steel plate) and gaps in the east-west (E-W) direction (perpendicular to the plane of the steel plate) of two inches at elevation 93' and 2.5 inches at elevation 69'.

Q.36 What effect will the steel plate and the added walls from the modifications have on displacement of structures?

A.36 The steel plate and added walls should have no effect at all on displacement of the Turbine Building. The plate and added walls will stiffen the Control Building Complex in the N-S direction and will thus cause displacements in that direction to be less for the modified Complex than for the as-built Complex. The addition of walls and the plate will not significantly stiffen the Complex in the E-W direction, however, and, therefore, displacements in that direction will not be significantly reduced relative to those for the unmodified structures. Also, there would not be a significant reduction in the E-W displacements of the Complex due to the proposed "structural improvements" to the walls running parallel to the E-W direction.

Q.37 What assurance is there that the material removal that you described will prevent contact between the Control and Turbine Buildings?

A.37 It was shown in Phase I of this proceeding that for N-S displacements, a gap of three inches between buildings is sufficient to preclude contact of the as-built buildings during an earthquake. Since displacement of the Control Building in the N-S direction will be reduced by the addition of the three walls and the steel plate, a gap of four inches after the modifications should be adequate. It was shown in Phase I of this proceeding that for E-W displacements, a gap of two inches at elevation 99' was adequate to maintain clearance between the buildings with margin. While displacements in the E-W direction will not be significantly reduced by the modifications, gaps of two inches at elevation 93' and 2.5 inches at elevation 69' after the modifications should be adequate to prevent building contact. Although we feel that the proposed modifications are adequate (per the April 14, 1980 PGE submittal) due to all uncertainties, the margins against building contact have not been precisely quantified.

Conclusion on Structural Adequacy of the Proposed Modifications

Q.38 Based on the analyses and evaluations that have been performed and on the status of the unresolved matters previously identified, what is your conclusion with regard to the adequacy of the proposed modifications to substantially restore the seismic margins and bring the Control Building into substantial compliance with the requirements of the Trojan license?

A.38 There are no unresolved items regarding structural adequacy of the modifications and we agree with the conclusion of the licensee that the proposed modifications will substantially restore the seismic margins and bring the Control Building into substantial compliance with the requirements of the Trojan license.

While the Staff and the licensee disagree somewhat as to the need to account for uncertainties (as indicated by statements made by the licensee in testimony) the licensee has gone further and has done additional analyses demonstrating the adequacy of the modified structure when those uncertainties are properly accounted for. Accordingly, the Staff has been able to conclude that the proposed modifications satisfy the requirements of the

May 26, 1978 Order for Modification of License and should be authorized. This conclusion is based not only on an evaluation of structural adequacy alone, as was the case in the evaluations performed by Professors Holly and Bresler and Dr. Laursen, but also on an evaluation of the effects of the behavior of the modified Complex on equipment, components and piping in the Complex.

Q.39 If the proposed modifications are approved and implemented, is any further license requirement needed, in your view?

A.39 Yes. Technical Specifications 5.7.2.1 and 5.7.2.2, as proposed by the licensee in its April 14, 1980 submittal should be imposed. These will assure that, subsequent to the approved modifications, the structural adequacy of the Complex will be maintained at the level required by the Order of May 26, 1978, while at the same time providing a small allowance for changes to structures which are below any threshold requiring any further NRC review.

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

February 14, 1980

Docket No. 50-344

Mr. Charles Goodwin, Jr.
Assistant Vice President
Portland General Electric Company
121 S.W. Salmon Street
Portland, Oregon 97204

Dear Mr. Goodwin:

Enclosed for your information is a copy of the NRC Staff's Safety Evaluation relating to the proposed design modifications to the Trojan Control Building, as described in PGE-1020, as supplemented and amended.

As you will see, the Safety Evaluation contains a number of unresolved items, but is being published at this time to conform with the Licensing Board's desire to issue such a report by this date. Since important information having a bearing on the adequacy of the proposed modifications was received from PGE as late as February 13, it was necessary to establish a review cut-off date in advance of this date in order to assure that the SER would be issued today as scheduled. It is therefore possible that further review could resolve some of the currently unresolved items. On the other hand, considering the lengthy Staff review to date, it is also possible that these unresolved items will remain due to differing professional opinions between the NRC Staff and the PGE/Bechtel engineers.

Sincerely,

A. Schwencer, Chief
Operating Reactors Branch #1
Division of Operating Reactors

Enclosure:
Safety Evaluation Report

cc: w/enclosure
See next page

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UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

SAFETY EVALUATION BY THE OFFICE OF NUCLEAR REACTOR REGULATION

RELATING TO DESIGN MODIFICATIONS TO THE CONTROL BUILDING

TROJAN NUCLEAR PLANT

DOCKET NO. 50-344

1.0 Background and Summary

On April 13, 1978, Portland General Electric Company (PGE or licensee), operator of Trojan Nuclear Plant, orally informed the NRC of potential design errors related to the shear walls of the Control Building at the facility. PGE and Bechtel Power Corporation (architect-engineer for the facility) investigated the matter and confirmed, in a letter dated April 28, 1978, that design errors did, in fact, exist and that the Control Building walls did not conform to the design criteria as set forth in the Final Safety Analysis Report (FSAR) for the facility. Additional details and the licensee's assessment of the impact of these design errors were furnished in a letter dated May 5, 1978, which also forwarded a Licensee Event Report (No. 78-13) on the matter as required by the Trojan Technical Specifications.

While the plant was shut down for refueling in April 1978, Bechtel Power Corporation studied, at PGE's request, the feasibility of cutting an opening and installing a security window in a wall of the Control Building. It was during this design review that the non-conformances with the FSAR criteria were identified.

The Control Building is composed of a structural steel framing system with steel beams and columns supporting reinforced concrete floor slabs, with shear walls designed to resist lateral seismic forces. The major shear walls are located around the perimeter of the building, and generally consist of a reinforced concrete core placed between two layers of reinforced grouted masonry block. The two-block layers generally sandwich the structural steel frame so that the reinforced concrete core is partially or completely interrupted by the steel frame members.

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A detailed NRC staff review of PGE's investigation and analysis of May 5, 1973 revealed the following design errors:

1. The steel reinforcement in the reinforced concrete core of the walls was generally discontinuous and, therefore, the concrete core could not be relied upon to resist shear to the extent assumed in the approved design.

2. The shear capacity of the reinforced concrete and grouted masonry block was computed incorrectly. This resulted in the amount of steel reinforcement needed to resist shear beyond the capacity of the concrete and grouted masonry block being computed incorrectly. Therefore, less steel than required for the design was used in the structure.

As a result of these identified design errors, the NRC staff concluded that the Control Building did not comply with the requirements of the Trojan license in that the shear walls do not have the intended design margin to the extent required by the FSAR to resist Trojan's Operating Basis Earthquake (OBE) of 0.15g nor the Safe Shutdown Earthquake (SSE) of 0.25g.

As a result of the identification of the non-conformances, a detailed reevaluation of the Control Building in its existing configuration was performed by PGE to assess the as-built capability of the structure to withstand the Operating Basis Earthquake and the Safe Shutdown Earthquake. The NRC staff determined that there had been a reduction in conservatism and design margins, with respect to the Control Building seismic capability, below the level intended and desired for the 33 years remaining in the expected plant life. Because this reduction in margin was significant, the NRC staff concluded that the appropriate margins should be restored by modifications to the Control Building. PGE indicated its intent to make such modifications.

The NRC staff also determined that, based on data supplied by PGE, there was adequate assurance of safety until Control Building modifications could be implemented, since the Trojan Plant had the capability to withstand an SSE of the magnitude established for that facility and could be brought to a safe shutdown condition. In addition, the NRC staff determined that the facility could be operated in the interim without endangering the health and safety of the public, provided that no modifications to the Control Building were made that would reduce the strength of the existing shear walls. Also, since the NRC staff had concluded on the best available information that the OBE capability for the Control Building had been reduced to 0.11g (0.15g was established for the facility), actions that would otherwise be required for a 0.15g earthquake would have to be taken in the event that a 0.11g peak ground acceleration earthquake were to occur at the plant site.

Having made these determinations, the Acting Director of the Office of Nuclear Reactor Regulation on May 26, 1978, issued an Order dealing with this matter. The Order, which offered an opportunity for hearing, was to be effective June 26, 1978, or on a date specified in an Order made following a hearing, if one were held in connection with the Order.

The May 26, 1978 Order called for:

*Design modifications to restore the seismic design margins originally intended to the Control Building with the Control Building brought into substantial compliance by June 1, 1979.

*An implementation schedule, to be reviewed and approved by the NRC, by July 1, 1978.

*Detailed design information by September 1, 1978, for NRC staff review and approval, together with supporting analyses and application for license amendments as necessary to implement these modifications.

*Conditional license waiver of the areas of non-conformance noted above until the Control Building has been brought into substantial compliance in these areas. The conditions called out were that no modifications affecting the strength of the Control Building shear walls were to be made without NRC approval and the facility should be brought to cold shutdown in the event that an earthquake reaching 0.11g ground acceleration should occur at the site and that subsequent restart would require prior NRC approval. The Order noted that since the facility - shut down at the time - did not conform with existing license requirements, it could not be operated without violating the license.

In offering an opportunity for a hearing, the Order defined the issues that could be raised: (1) whether interim operation prior to the modifications required by the Order should be permitted, and (2) whether the scope and timeliness of the modifications required by the Order to bring the facility into substantial compliance with the license are adequate from a safety standpoint.

Numerous requests for a hearing were received, and a hearing was ordered. The hearing was divided into two phases. Phase I of the hearing addressed the question of interim operation, and was originally scheduled to begin September 6, 1978. However, on August 22, PGE advised NRC of new information resulting from a new finite-element analysis which differed in several respects from information previously provided. Accordingly, the hearing on interim operation was postponed, and subsequently held October 23 to November 3, and December 11 to 14, at which time the new information was considered.

On December 21, 1978, the Atomic Safety and Licensing Board issued a Partial Initial Decision relating to interim operation (Phase I) of the facility. That decision authorized interim operation, with conditions, pending further hearings on the nature of modifications to the Control Building to bring it into substantial compliance with the requirements of the operating license (Phase II). The conditions prohibited any modifications that would reduce the strength of existing shear walls; required plant shutdown in the event an earthquake exceeding 0.08g should occur at the site; and required modifications to some pipe supports and restraints prior to operation in order to ensure qualification of related piping systems to earthquake levels up to the Safe Shutdown Earthquake (0.25g).

A conforming amendment was issued on December 22, 1978, and plant operation resumed on December 30, 1978.

On January 17, 1979, PGE filed the proposed modifications to the Control Building. A summary of the proposed modifications is provided in Section 2.0 of this report.

This Safety Evaluation Report addresses those issues encompassed by Phase II of the hearing - whether the scope and timeliness of the modifications required by the Order of May 26, 1978 to bring the facility into substantial compliance with the license are adequate from a safety standpoint.

The Trojan facility is currently in an operational status, governed by the interim restrictions imposed by the Atomic Safety and Licensing Board in its Partial Initial Decision (Interim Operation) described above.

2.0 Description of Modifications

As shown on the attached Figures 1 and 2, the Control Building modifications consist of the addition of three new reinforced concrete shear walls across the existing railroad bay in the Control Building. Part of the new west wall would extend south of column line 46 into the existing locker room at elevation 45'. In addition, part of the west wall would be further strengthened by the addition of a three-inch thick steel plate bolted to the outside of the wall between elevations 59'-3" and 97'-3" with high-strength steel bolts.

To facilitate handling, fabrication and erection, the steel plate will be installed sequentially in a total of eight pieces and welded together. The steel plate has four cut-outs for the passage of electrical cables and associated cable trays. The steel plate pieces have been designed so that, when assembled, the openings are formed around the cables, making it unnecessary to remove them for plate installation.

The new east wall would extend above the railroad bay to elevation 95'-6" and be fastened to the existing east wall with reinforcing steel and high-strength steel bolts.

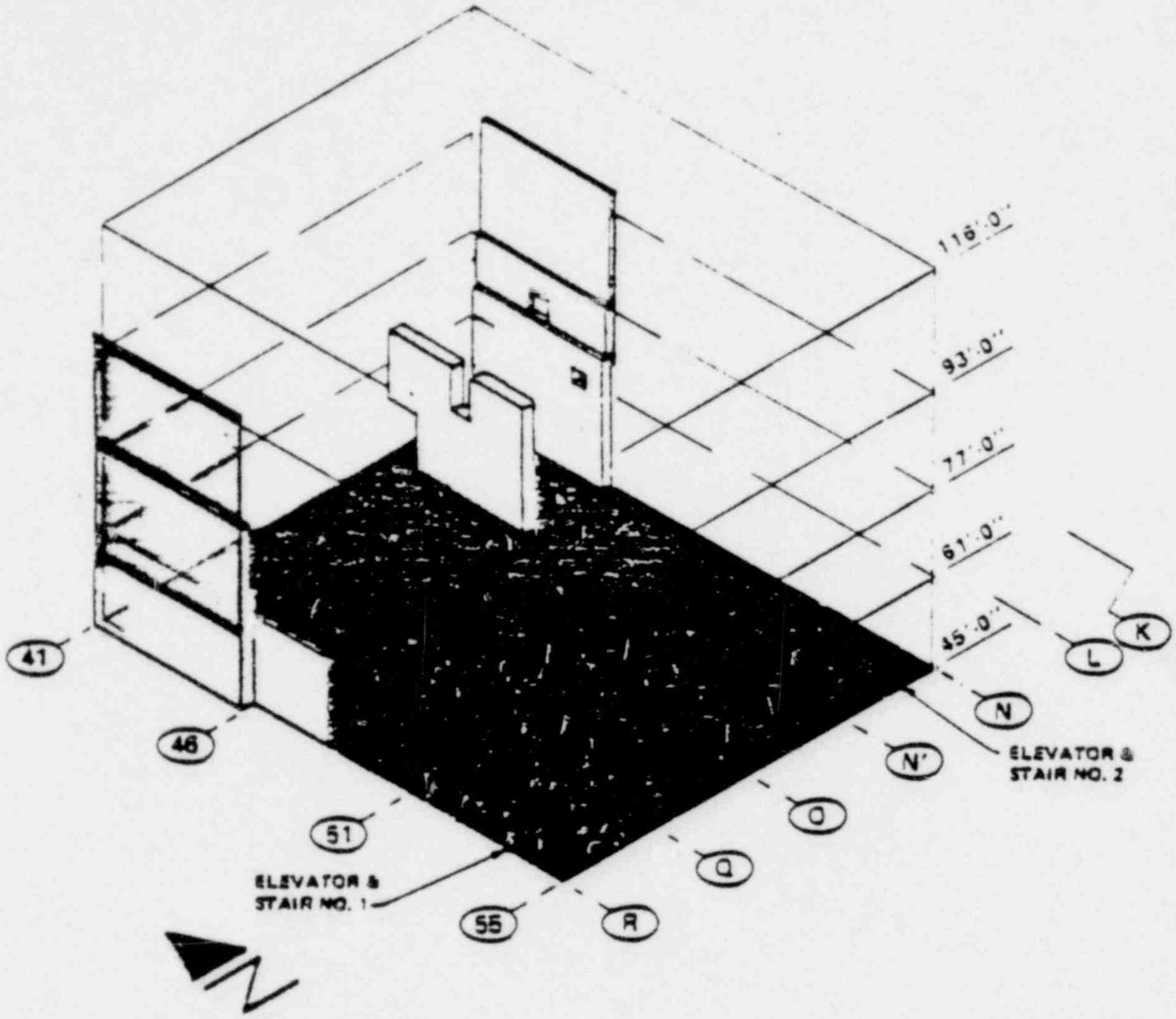
The existing equipment access opening in the east wall at elevation 65' would be reduced in size to 4 feet square.

The existing diesel generator combustion air path consists of the open railroad bay of the Control Building. Because this path would be closed off by the new walls, PGE would install a new diesel generator combustion air intake in the north wall of the Turbine Building located to the west of the Control Building west wall. To provide an unrestricted path, a roll-up door would be relocated to the west of the louvered diesel air intake.

The railroad spur, which presently runs through both the Turbine and Control Buildings to the Fuel Building, would be terminated in the Turbine Building. A railroad bumping post would be placed at the end of the line.

A second railroad spur would be added to maintain railway service to the Fuel Building.

The modifications are described in more detail in Section 3.2 of PGE-1020, "Report on Design Modifications for the Trojan Control Building."



116'-0"
93'-0"
77'-0"
61'-0"
45'-0"
K
L
N
N'
O
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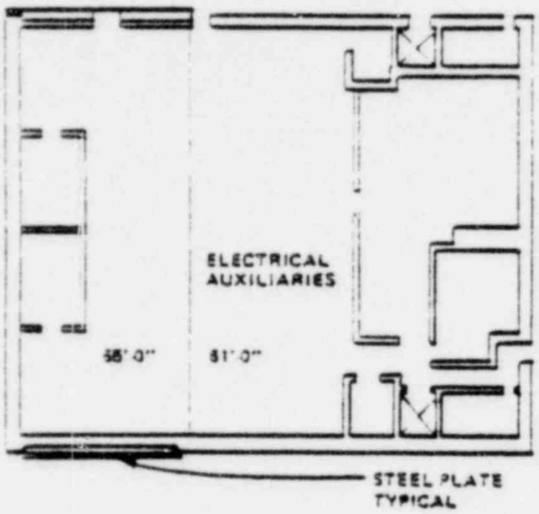
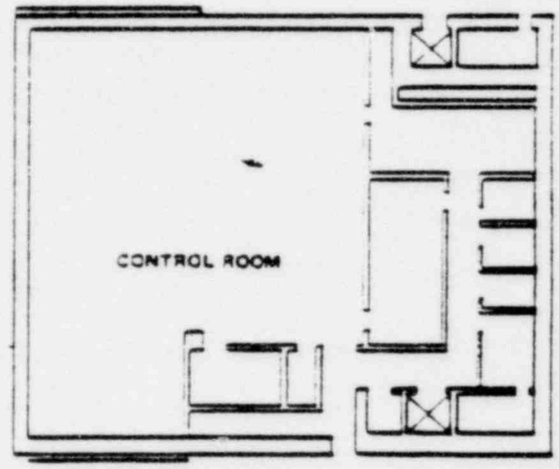
R
41
46
51
55
ELEVATOR & STAIR NO. 2
ELEVATOR & STAIR NO. 1
N

FIGURE 1

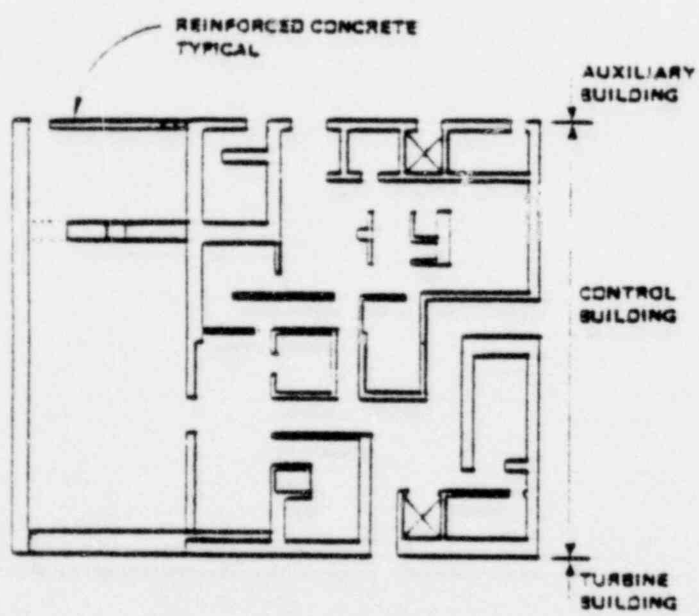
N



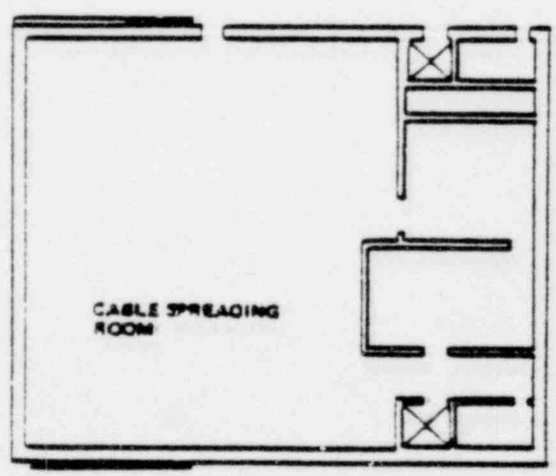
FLOOR PLAN EL 67'-0" & 65'-0"



FLOOR PLAN EL 93'-0"



FLOOR PLAN EL 45'-0"



FLOOR PLAN EL 77'-0"

FIGURE 2

3.0 SEISMIC STRUCTURAL CRITERIA

3.1 Original Design Criteria

3.1.1 Original Seismic Analysis Criteria

The seismic analysis criteria for the Trojan Nuclear Plant are delineated in detail in Section 3.7 of the Trojan Nuclear Plant FSAR. In general, this section, which is incorporated into the Trojan operating license in Technical Specification 5.7.1, defines the seismic input criteria, and the seismic analysis techniques which were incorporated in the design of the facility and accepted by the NRC.

Section 3.7.1 of the Trojan FSAR defines the seismic input criteria applicable to the Control/Auxiliary/Fuel Building Complex. These criteria define the input in terms of peak ground accelerations for the Operating Basis Earthquake (OBE) and the Safe Shutdown Earthquake (SSE), along with the associated ground response spectra and damping values. The ground response spectra are defined in Figures 3.7-1 and 3.7-2. The damping values are defined in Table 3.7-1. The peak ground accelerations are 0.15g and 0.25g for the OBE and SSE, respectively. The following areas are specifically addressed:

FSAR Sections

3.7.1 Input Criteria
3.7.1.1 Design Response Spectra
3.7.1.2 Derived Spectra
3.7.1.3 Percentage of Critical Damping
3.7.1.4 Site Dependent Analysis
3.7.1.5 Depths of Bedrock
3.7.1.6 Soil Interaction

The requirements for the seismic analysis of the Complex are contained in Section 3.7.2 of the Trojan FSAR. This section describes the seismic analysis requirements which were applicable for the Complex and the bases for these requirements. Specifically, the items addressed in this section are:

FSAR Sections

3.7.2 Seismic System Analysis
3.7.2.1 Seismic Analysis of Category I Structures
3.7.2.2 Criteria for Lumping Masses
3.7.2.3 Validity of Fixed-Base Models
3.7.2.4 Finite Element Analysis
3.7.2.5 Response Spectra Multi-Mass Method
3.7.2.6 Effects of Expected Variations of Structural Properties

FSAR Sections (Continued)

3.7.2.7	Vertical Response Loads
3.7.2.8	Torsional Modes of Vibration
3.7.2.9	Comparison Between Response Spectrum and Time History Methods
3.7.2.10	Seismic Analysis of Dams
3.7.2.11	Design Control
3.7.2.12	Overturning Moments
3.7.2.13	Simplified Seismic Analysis Methods
3.7.2.14	Criteria for Composite Damping
3.7.2.15	Criteria for Modal Period Variation
3.7.2.16	Damping Factors
3.7.2.17	Seismic Analysis of Category II Structures
3.7.2.18	Earthquake Cycles

3.1.2 Original Seismic Design Criteria

The Design Criteria applicable for the original design of the Control/Auxiliary/Fuel Building Complex are delineated in Section 3.8.1 of the Trojan FSAR. In general, this section defines the appropriate load combinations and corresponding acceptance criteria, design documents, materials and their specifications, and the preoperational testing. Specifically contained in this section are:

FSAR Sections

3.8.1	Structures Other than Containment
3.8.1.1	Design Bases and Structure Description
3.8.1.2	Design Documents
3.8.1.3	Load Combinations
3.8.1.4	Analytical Methods
3.8.1.5	Design Methods
3.8.1.6	Identification of Construction Materials
3.8.1.7	Structural Preoperational Testing Procedures

The construction practices and the construction materials and their specification used for the construction of the Complex are described in Section 3.8.3 of the Trojan FSAR.

3.2 Conformance of the Modified Complex with the Original Design Criteria

The structural analysis, evaluation and design of the modified Complex are described in detail in PGE-1020 and the associated PGE submittals. They are evaluated in detail in Section 5 of this Safety Evaluation Report. This section generally compares the analysis and design criteria for the modified Complex to that which was committed to in Sections 3.7 and 3.8 of the Trojan FSAR, as referenced by and incorporated into the Trojan Technical Specifications.

3.2.1 Seismic Analysis Criteria

3.2.1.1 Input Criteria (FSAR Section 3.7.1)

3.2.1.1.1 Design Response Spectra (FSAR Section 3.7.1.1)

This section of the Trojan FSAR defines the peak ground accelerations in the horizontal and vertical directions, along with the associated ground response spectra, for the OBE and SSE. These ground response spectra are those which were incorporated into the seismic analyses of the modified Complex. Therefore, this FSAR commitment is being met.

3.2.1.1.2 Derived Spectra (FSAR Section 3.7.1.2)

Floor response spectra were derived initially using a time history analysis of the structure. Floor response spectra are being derived using the techniques described in Appendix B of PGE-1020 in a time history analyses. Though the artificial time history and frequency intervals for calculating the floor response spectra from the time history analysis of the structure are different, this FSAR commitment is being met. The new artificial time history better characterizes the motion described by the ground response spectra and is the same time history which has been previously found acceptable under Phase I of the Control Building Proceeding. The frequency intervals now being used for calculating floor response spectra are those previously accepted under Phase I of the Control Building Proceeding and are in conformance with current practice as delineated in Regulatory Guide 1.122.

3.2.1.1.3 Percentage of Critical Damping (FSAR Section 3.7.1.3)

The percentage of critical damping being assumed for the analysis of the structure were previously defined as 1 percent of critical for low stress levels, 2 percent of critical for working stress levels, and 5 percent of critical at the yield point. Current analyses of the structure assume 2 percent of critical damping for the OBE analysis and 5 percent of critical damping for the SSE analysis, although that results in computed OBE and SSE stresses in the structure which are essentially the same. This assumption of working stress level damping for the OBE and yield point level damping for the SSE is in conformance with current criteria which are delineated in Section 3.7 of the U.S.N.R.C. Standard Review Plan. The numerical values of damping are in conformance with Section 3.7.1.3 of the Trojan FSAR. Therefore, this FSAR commitment is being met.

3.2.1.1.4 Site Dependent Analysis (FSAR Section 3.7.1.4)

The initial ground response spectra are being used in the analysis of the modified Complex. No new site dependent spectra were generated. Therefore, this FSAR commitment continues to be met.

3.2.1.1.5 Depths to Bedrock (FSAR Section 3.7.1.5)

This is unaltered by the proposed modifications to the Complex.

3.2.1.1.6 Soil Interaction (FSAR Section 3.7.1.6)

A fixed base model was used in the initial analysis of the complex as well as in the analysis of the modified Complex. Therefore, this FSAR section remains unaltered.

3.2.1.2 Seismic System Analysis (FSAR Section 3.7.2)

3.2.1.2.1 Seismic Analysis of Category I Structures (FSAR Section 3.7.2.1)

The dynamic loads for the Complex were initially determined by response spectrum analysis using the appropriate natural periods, mode shapes and damping factors. This procedure has been adhered to in the seismic analysis of the modified Complex, with modal responses being combined by the SRSS method with appropriate consideration of closely-spaced modes. However, the analytical models, and referenced allowable stresses described in this FSAR Section have been superceded by those described in

PGE-1020 and the associated PGE submittals. The acceptability of these is discussed in Sections 3 and 5 of this Safety Evaluation Report. The seismic load combinations used in the analysis of the modified Complex are the same as those referenced in this section of the FSAR.

3.2.1.2.2 Criteria for Lumping Masses (FSAR Section 3.7.2.2)

The initial 3-D fixed-base beam-stick model of the Complex consisted of essentially four sticks representing the Control Building, the Auxiliary Building, the Fuel Building Hold-up Tank Enclosure, and the Fuel Building Spent Fuel Pool. The masses of the structures were lumped at each floor level for each stick. Beam elements representing the structural elements supporting the floors connected the masses. The sticks were tied together laterally through beam elements representing the common floor slabs.

The STARDYNE model of the modified C plex, as discussed in PGE-1020, contained the same basic floor elev ons as the initial model; however, all shear walls and floor . abs were modeled using finite elements. This representation provided a more accurate representation of the mass magnitudes and their distribution for the modified Complex than did the initial model. Floor response spectra are generated for both the OBE and SSE conditions for the modified Complex using this STARDYNE model. The QA/QC procedures discussed in this section of the FSAR have been superceded by those described in PGE-1020 which are procedures currently accepted by the NRC.

3.2.1.2.3 Validity of Fixed-Base Models (FSAR Section 3.7.2.3)

This FSAR section demonstrated that fixed base models were valid for the analyses of structures. The validity of a fixed base model of the Complex was further addressed in Phase I of the Control Building Proceeding. A fixed base model was used for the seismic analysis of the modified Complex. Additionally, for the properties of the supporting rock at the Trojan site (shear wave velocity of 4500-5000 fps), current criteria as delineated in Section 3.7.2 of the NRC Standard Review Plan would sanction the use of a fixed-base model. Therefore, this FSAR commitment continues to be met.

3.2.1.2.4 Finite Element Analysis (FSAR Section 3.7.2.4)

This FSAR section stated that the finite element analysis technique was not used. However, this technique has been used for the seismic analysis of the modified Complex.

3.2.1.2.5 Response Spectra Multi-Mass Method (FSAR Section 3.7.2.5)

This method was not used to generate floor response spectra in the initial seismic analysis and has not been used for generation of the floor response spectra for the modified Complex. Therefore, this FSAR commitment continues to be met.

3.2.1.2.6 Effects of Expected Variations of Structural Properties (FSAR Section 3.7.2.6)

This section referenced BC-TOP-4, "Seismic Analysis of Structures and Equipment for Nuclear Power Plants," Bechtel Corporation (April 30, 1971) for the method used in accounting for variations of structural properties and damping. This area is discussed in PGE-1020 and the associated PGE submittals, and Section 5 of this Safety Evaluation Report. No variation in damping was considered in the analysis. The variations in floor response spectra peaks are being accounted for by widening the spectral peaks per BC-TOP-4A criteria. This Bechtel topical report has the same title as BC-TOP-4 but is the NRC approved topical version and is dated November 1974. Further widening of the peaks is performed to account for potential effects of the assumed occurrence of five earthquakes of the OBE level.

3.2.1.2.7 Vertical Response Loads (FSAR Section 3.7.2.7)

This section requires that the analyses for the horizontal and the vertical directions be performed using the ground response spectrum curves. The forces, moments and resulting stresses were combined assuming a simultaneous occurrence of the vertical and horizontal motions. This is being met since the structural forces, moments and resulting stresses are obtained by combining the vertical earthquake response with each of the horizontal responses absolutely and taking the greatest of the two resultants as the resultant force, moment or resulting stress that must be resisted.

Additionally, the vertical members were initially considered vertically rigid, while horizontal members were further investigated for vertical response. As discussed in PGE-1020 and the associated PGE submittals, and Section 5 of this SER, appropriate vertical amplification of the ground response has been considered in the calculation of building response. The initial vertical floor response spectra remain unchanged due to the modification of the Complex. Therefore, these FSAR commitments are met.

3.2.1.2.8 Torsional Modes of Vibration (FSAR Section 3.7.2.8)

The initial analysis and the analysis of the modified Complex consider torsional response. Therefore, this FSAR commitment is met.

3.2.1.2.9 Comparison Between Response Spectrum and Time History Methods (FSAR Section 3.7.2.7)

This FSAR section provided a comparison of the results obtained for the acceleration of the Containment based on the response spectrum and time history methods, using the artificial time history defined for the initial analyses, and indicated good agreement. Since a new artificial earthquake time history was used for the analysis of the modified Complex, this FSAR commitment is met by the fact that this comparison was provided for the unmodified Complex, using the new artifical time history, in the October 27, 1978 response to the NRC questions of October 16, 1978. This comparison indicated good agreement between the response spectrum and the time history analysis results. A similar comparison was not provided for the modified Complex; however, this is not deemed necessary. The analysis of the modified Complex would provide similar good agreement since the modifications will not significantly alter the response of the structure. Therefore, this FSAR commitment is met.

3.2.1.2.10 Seismic Analysis of Dams (FSAR Section 3.7.2.10)

This matter was not applicable to the Trojan Plant. This remains unaltered by the proposed modification to the Complex.

3.2.1.2.11 Design Control (FSAR Section 3.7.2.11)

The appropriate design controls for the modifications to the Complex are covered by PGE's QA/QC program which is approved by the NRC. The QA/QC procedures are described in PGE-1020. Therefore, this FSAR commitment is met.

3.2.1.2.12 Overturning Moments (FSAR Section 3.7.2.12)

This FSAR section required that overturning moments for the Complex be calculated using response spectrum analyses and that the stability of the structures be checked by combining the overturning moment with the dead load of the structure and the vertical earthquake affects. As discussed in PGE's 2/13/80 submittal and Section 5 of this SER, this has been done.

3.2.1.2.13 Simplified Seismic Analysis Methods (FSAR Section 3.7.2.13)

As for the initial criteria, the analysis of the modified Complex is in conformance with the criteria described in FSAR Section 3.7.2.1, which is discussed in Section 3.2.1.2.1 of this report. Therefore, this FSAR commitment is met.

3.2.1.2.14 Criteria for Composite Damping (FSAR Section 3.7.2.14)

The damping values for the OBE and SSE are the same for all elements of the modified Complex in all modes of vibration. Therefore, it is not necessary to account for composite damping.

3.2.1.2.15 Criteria for Modal Period Variation (FSAR Section 3.7.2.15)

In the predominant range of modal frequencies for the modified Complex, the ground response spectra values for acceleration are essentially constant. Therefore, the frequency variation does not have a significant effect on the structural forces, displacements and accelerations. However, the modal frequency variations are accounted for in broadening of the floor response spectra. This is further discussed in Section 3.2.1.2.6 of this SER.

3.2.1.2.16 Damping Factors (FSAR Section 3.7.2.16)

The damping values used in the analysis of the modified Complex are in accordance with those referenced in this FSAR section. This is further discussed in Section 3.2.1.2.3 of this SER. Therefore, this FSAR commitment is met.

3.2.1.2.17 Seismic Analysis of Category II Structures (FSAR Section 3.7.2.17)

The Complex is considered as Seismic Category I. Therefore, this section is not applicable to the modified Complex.

3.2.1.2.18 Earthquake Cycles (FSAR Section 3.7.2.18)

As stated in this FSAR section, the original design of the Complex was such that it was not necessary to consider cyclic loading in the design. The modified Complex, as substantiated by the testing program on the sample walls, may experience cyclic degradation given the occurrence of OBE level earthquakes. These potential degradations are being accounted for by considering the occurrence of five (5) OBE's, each resulting in the structure experiencing ten (10) effective full stress cycles. This is discussed further in PGE-1020 and the therin referenced December 21, 1979 PGE submittal, and Section 5 of this SER. The choice of five (5) OBE's, each producing ten (10) effective full cycles of stress, is based upon the current requirements of the U.S.N.R.C. Standard Review Plan Section 3.7.3.II.2. Although this criteria is specifically applicable to subsystems (piping and equipment) and components, it is felt that this criteria is reasonable and conservative to apply to the modified Complex due to higher damping present in the structure than in the subsystems and components resulting in the structure experiencing somewhat fewer total cycles than the subsystems and components. However, the degree of conservatism cannot be reasonably quantified. In summary, a different approach is being taken than that initially taken to meet the intent of this section; namely, that the cyclic effects of earthquakes be considered.

3.2.2 Seismic Design Criteria

The seismic design criteria for the initial design of the Complex to resist the loads resulting from the seismic analyses performed per criteria described in Trojan FSAR Section 3.7, are described in Section 3.8 of the Trojan FSAR. The FSAR Section 3.8 design requirements are incorporated in FSAR Section 3.7 by reference. The structural design criteria which were applicable to the Complex initially are given in Trojan FSAR Section 3.8.1.

3.2.2.1 Structures Other Than Containment (FSAR Section 3.8.1)

3.2.2.1.1 Design Bases and Structure Description (FSAR Section 3.8.1.1)

This FSAR section provides the design bases for the structures comprising the Complex; namely, the Control Building, the Auxiliary Building and the Fuel Building. FSAR Table 3.8-1 itemizes these requirements. The loads which cause a lateral loading of the Complex and are thereby required to be reevaluated due to the Control Building design deficiencies are the seismic and wind, including tornado, loadings since protection of the Complex against these events is required. As was determined under the evaluations performed under Phase I of the Control Building Proceeding, wind and tornado criteria were not the limiting loads on the Complex due to the Control Building design deficiencies which are the subject of this proceeding. The seismic loads are the limiting loads and it is in this area that determinations regarding the adequacy of the modified Comlex are being focused. The adequacy of the modified Complex to withstand seismic loadings, in conjunction with other appropriate loads, is discussed in detail in Section 5 of this SER. The basic descriptions of the buildings comprising the Complex remains unchanged by the proposed modifications. The buildings are described in Section 3.3.3 of PGE-1020. The clearances between the structures remain unchanged except for the gap between the Turbine Building and Control Building at the Control Building R line wall at elevations 93' and 69' where the gap is reduced to 2.0" and 2.5" after modification, and at Turbine Building column S41 where the gap is 4 inches. The adequacy of the clearances for the modified Complex is discussed in Section 5 of this SER.

3.2.2.1.2 Design Documents (FSAR Section 3.8.1.2)

This section states the design documents applicable to the initial design of the Complex. This section is superceded for the modified Complex by PGE-1020, PGE's associated submittals, and the references incorporated into these. The acceptability of these documents to provide for adequate design and evaluation of the modified Complex is discussed in Section 5 of this SER.

3.2.2.1.3 Load Combinations (FSAR Section 3.8.1.3)

This FSAR section provides the loads, and load combinations and corresponding acceptance criteria which were incorporated in the initial design of the Complex. As stated in Section 3.2.3 of PGE-1020, both the existing Complex structural elements and the new structural elements for the Control Building modifications are designed for these loads and load combinations. The acceptance criteria for the new and existing structural elements is discussed in PGE-1020, PGE's associated submittals and Section 5 of this SER. Actual floor and equipment loadings are being substituted for the design floor live loads specified in Table 3.8-2 of the FSAR which is reasonable since the actual loads are not normally known accurately in the initial design phases but are now known and may appropriately be used. The pertinent governing load combinations which are being addressed in the analyses of the modified Complex existing walls and proposed modifications to resist gross lateral loading of the modified Complex are:

(1) $1.4\ (D+L+E)=U$
(2) $1.0\ (D+L+E')=U$

where D = Dead load of structure and equipment plus other permanent loads contributing stresses, such as soil or hydrostaic loads.

L = Live load

E = OBE load resulting from ground surface acceleration of 0.15g

E'= SSE load resulting from ground surface acceleration of 0.25g

U = Required ultimate load capacity

3.2.2.1.4 Analytical Methods (FSAR Section 3.8.1.4)

This FSAR section essentially commits that classical theory, empirical equations and numerical methods were used as necessary in the initial analysis of the Complex. Further, it commits that loads and load combinations as delineated in FSAR Section 3.8.1.2 were considered. The techniques now being incorporated are discussed in PGE-1020 and the associated PGE submittals. Their adequacy is addressed in Section 5 of this SER.

3.2.2.1.5 Design Methods (FSAR Section 3.8.1.5)

The design methods appropriate for the modified Complex are discussed in PGE-1020 and the associated PGE submittals, and in Section 5 of the SER.

3.2.2.1.6 Identification of Construction Materials (FSAR Section 3.8.1.6)

This FSAR section describes the materials used in the construction of the facility. The materials to be used for the modifications to the Complex are described in PGE-1020 and the associated PGE submittals. The materials comprising the walls and slabs in the existing elements of the Complex have been described in the documents submitted under Phase I of the Control Building Proceeding. They are somewhat different than those listed in this FSAR section in that the FSAR did not give the block strengths and grout strengths and did not list where 5000 psi concrete was used rather than the 3000 psi concrete specified therein.

Section 3.8.3 of the Trojan FSAR is referenced as fully describing the quality control procedures, construction practices and materials. Those for the proposed modifications to the Complex are described in PGE-1020, the associated PGE submittals, and Section 5 of this SER. The QA/QC procedures are governed by PGE's NRC approved QA program.

3.2.2.1.7 Structural Preoperational Testing Procedures (FSAR Section 3.8.1.7)

No structural preoperational testing of the Complex was initially performed. None is required for the modified Complex. However, the FSAR commitment to periodically visually inspect the structures during the plant life for apparent structural deterioration such as large cracks and excessive deflection is still required for the modified Complex. Further, inservice inspection of the pre-tensioned bolts connecting the new structural elements to those existing in the Control Building is being required as discussed in Section 6 of this SER.

4.0 Modification Work and Effects on Plant Operation and on Safety-Related Equipment

The proposed modification work has been evaluated in detail to determine how damage to safety-related equipment could possibly occur from performance of this work during operation of the plant and to ascertain the adequacy of the protection provided against such damage. Based on this evaluation, we have determined that the areas of concern are those aspects of the modification work involving:

1. drilling holes through walls,
2. bolt assembly installation,
3. welding and cutting operations (addressed in section on fire protection),
4. modifications to safety systems required for performance of the Control Building modifications,
5. effects on operator actions,
6. steel plate installation,
7. construction generated dust and dirt,
8. noise and vibration due to modification work,
9. work sequence,
10. effects of modifications (structural assessment), and
11. compliance with Technical Specifications during modification work.

Other than these areas, there are no aspects of the proposed modifications that will have an impact on safety-related equipment or the safety of plant operation.

Each of the areas identified above are addressed in this section of the SER. Also discussed in this section are the QA/QC requirements for the modification work and the inspections of the work that will be performed by the NRC.

4.1 Hole Drilling

To provide for bolting steel plates to the west wall of the Control Building as well as to provide for bolting the new wall at column line N to the existing structure, it will be necessary to drill holes through the west and east walls between column lines 41 and 47 and in the Electrical Auxiliaries Room to insert rebar for the attachment of the new wall at column line N'.

Such drilling could potentially affect safety-related cables, cable trays, equipment and/or conduit at the following locations:

1. the west wall of the Control Building/east wall of the Turbine Building between elevations 77' and 93' where four banks of cable trays and conduits pass through the walls from the Control Building to the Turbine Building,

2. the west wall of the Electrical Auxiliaries Room, between elevations 65' and 77' where safety-related cables in trays and conduits are located near the inside of the wall,

3. the west wall of the Cable Spreading Room between elevations 77' and 93' where safety-related cables in trays and conduits are located near the inside of the wall,

4. the Control Room west wall between elevations 93' and 97'3" where several safety-related conduits are located near the inside of the wall,

5. the east wall of the Control Room between elevations 93' and 95'6" between column lines 41 and 46 (no safety-related equipment or cables are at this location),

6. the east wall of the Cable Spreading Room between elevations 77' and 93' between column lines 41 and 46 where safety-related cable trays are located near the wall,

7. the east wall of the Electrical Auxiliaries Room between elevations 65' and 77' (no cable trays near the wall, one electrical cabinet near the wall), and

8. the floor of the Electrical Auxiliaries Room at column N' (no safety-related equipment or cables are located at these drilling sites).

For each of these areas, the licensee will determine the location of each hole to be drilled by surveying. The survey methods to be used provide a precise location of the holes by means of tape measurements from existing column lines and by means of a survey transit where it is necessary to establish additional points of reference. This method should allow the location of holes with sufficient accuracy to prevent damage to all but the imbedded conduits discussed separately below.

Once the location of the hole is established on the side of the wall from which drilling is to be done, the location of the hole on the opposite side of that wall will be accurately established using the same survey methods described above and marked prior to the commencement of drilling. The holes must be located where there are no obstructions and where there is sufficient space between the wall and any cables, cable trays or conduit near the wall to allow installation of the washer and the nut on the bolt. This requires a horizontal distance from the wall to the nearest cable, cable tray or conduit at the hole location to be at least five inches to accommodate the large washer (approximately 2 3/8 inches thick), a small washer (approximately 1/4 inches thick), the nut (approximately 1 3/4 inches thick), about 1 inch of grout, plus excess bolt length beyond the tightned nut. A radial distance from the hole of at least nine inches is required to accommodate the washer against the wall. Consequently, prior to drilling a particular hole, the surveying operations will be performed to assure that the hole location is such as to allow at least these minimum distances between the wall hole and the nearest cable or conduit where the drill bit comes through the wall. Therefore, for any hole, the drill would have to penetrate through the wall and then continue to travel at least an additional five inches before any contact with cable, conduits or cable trays could occur.

A positive drill control will be provided. This will prevent contact with cable trays, cables or conduits by limiting the extent to which the drill can travel past the surface of the wall once the drill has penetrated the wall. The drill advance through the wall is controlled manually; that is, a rack and pinion gear must be rotated by hand before the drill can advance. This provides for a slow positive control over the drill preventing any punch-through once the wall is penetrated and thereby preventing any damage to cable trays, conduits or equipment on the opposite side of the wall. For each hole to be drilled, a painted stripe or tape will be provided on the drill so the operator knows what depth has been penetrated and thereby be aware when the drill has penetrated the wall depth.

In addition, personnel will be stationed at the opposite side of the wall from the driller to monitor the hole location and drill penetration. Positive communications between the monitor and driller will be provided by means of portable radio communications or by sound or battery powered telephones to assure that the drilling is terminated as soon as the drill penetrates the wall.

Each of the areas identified in items 1 through 3 above were closely examined by members of the NRC staff during a site visit on June 13-14, 1979. For the areas described in item 1, drilling will be parallel to the four banks of cable trays and conduits which pass through the walls from the Control Building to the Turbine Building. Sufficient space must be provided between the location of holes to be drilled and the existing cable trays/conduit which pass through the walls to accommodate the drilling equipment as well as the width of the washer (9 inch radius) and nut which will ultimately be installed on the bolt. This space is sufficient to assure that drilling in this area will not result in contact between the drill and the cable trays/conduit which pass through the wall For the areas described in items 2 through 4, 6 and 7 above, cables, cable trays and conduits are located only on the side of the wall opposite the side from which drilling will take place. For these areas, the five inch minimum required spacing between the surface of the wall and the nearest cable tray/conduit at the hole location, along with the positive drill control and the communications between the driller and personnel monitoring the drilling, will assure that the drill will not contact cable trays, cables or conduit. For items 5 and 8, there are no safety-related equipment, cables and conduit on the opposite side of the wall and floor where drilling is to be accomplished. Based on this evaluation and on our examination of the areas where drilling will take place, we conclude that the measures discussed above can be practically implemented and that they will prevent any damage to safety-related cables, cable trays and conduits from drilling.

One exception to the above-described method for accurately predetermining hole location to prevent damage to conduits containing safety-related cables is at the west wall of the Control Building near column line 46 on column line R where two conduits, each containing two cables, are embedded in the concrete.

The location of these conduits can be determined approximately by finding where the conduits enter and leave the wall; then, marking their approximate location by drawing straight lines between these points. It is, therefore, unlikely that one of the conduits would be contacted. If one of the conduits were struck, the rigid steel conduit would provide

Therefore, it is our conclusion that adequate measures are being taken by the installation of passive means (steel cable tray covers and scaffold timber planking) and direct supervision of the work to protect cables against damage during the bolt assembly installation while the plant is operating.

4.3 Fire Potential and Protection During Modifications

The proposed modification work may potentially affect fire protection at the facility because of (1) the possible introduction of ignition sources from the modification work; (2) the possible accumulation of combustible materials from the modification work; (3) the potential impairment of fire brigade access to areas of the facility due to storage of construction equipment and/or materials, work-in-progress or other aspects of the modifications; and (4) impairment of fire barriers.

Ignition sources that will be introduced by the modifications are Cadwelding, electric arc welding and cutting by electric arc or oxyacetylene torch. Such sources raise a possible concern with regard to the potential for ignition of combustible materials.

Areas where Cadwelding, welding or cutting will be performed which may contain combustible materials such as electrical cable insulation, wood and plastics, etc. are in the Turbine Building above elevation 69' along the R line wall (division A cables here), and the Electrical Auxiliaries Room at elevations 65' to 77' near column 41 R. Such work may also be required in the Cable Spreading Room and the Control Room.

Administrative controls required under the facility's NRC approved fire protection plan provide that it is the joint responsibility of the Cognizant Supervisor, the Safety Supervisor and the Shift Supervisor to review and evaluate proposed work activities to identify potential transient fire loads and other potential sources of danger. Prior to the initiation of any Cadwelding, welding or cutting work in the plant, a welding or cutting permit must be obtained by the responsible supervisor. This permit requires, among other things, that combustible materials in the area where welding or cutting is to take place be removed or that potentially combustible materials such as the insulation on installed electrical cables be protected from welding and cutting operations. The permit also requires that a trained fire watch (one or more plant personnel trained in fire-fighting) equipped with a fire extinguisher or hose be present in the work area. The fire watch must remain in the area for 30 minutes after completion of the welding or cutting to assure that no fire or smoldering has occurred. With regard to physical fire protection for the cables, the licensee has committed to using Claremont Weld Shield 800-24 or FabriCote 1584-white fire blankets over all cables in the area where welding and/or cutting will be done. These are designed to stop welding and cutting slag and sparks from penetrating the blanket and reaching items protected by the blankets. Based on our evaluation of the qualification test procedure and the test results and report for these blanket materials, we have concluded that the fire blanket material provides adequate protection of combustibles from welding, Cadwelding and cutting operations. In addition, the licensee has committed to remove wood form materials prior to commencement of any welding or cutting operations and to store other necessary flammable material such as rags in self-closing containers which will be removed from the area at the end of each work day. Both the use of protective blankets and the stationing of fire watches in areas where welding and cutting operations are being performed have been found acceptable by the staff as adequate protection in preventing fires or detecting and extinguishing fires from the performance of such work during plant operation. These protective measures will assure adequate fire protection from all welding, Cadwelding and cutting activities performed during plant operation due to the proposed modifications.

Smoke will be generated by welding, Cadwelding and cutting operations as will some dust from concrete removal. To protect the equipment in the Electrical Auxiliaries Room from the work at column 41 R, an enclosure of fire retardant plastic and wood will be constructed around the work area with fans provided to direct any smoke and dust outside. If such work is necessary in the Control Room or Cable Spreading Room, a similar enclosure will be provided there as well. Plastic used with wood to construct enclosures will remain for the duration of work in an area. Other plastic sheeting will be removed from any safety-related area at the end of the work shift.

Because the modifications require construction equipment, materials and tools to be brought into areas where modification work is to be performed, this will result in the temporary introduction and use, while work is actually being performed, of minor combustibles such as wood, rags, rubber tires on dollies, and the like. However, aside from wood used for concrete forms and wood and plastic used for dust enclosures, this will not result in the accumulation of combustibles and an increase in the potential for fires in safety-related areas because:

1. Plant procedures require safety-related areas to be free of permanent material storage at all times and equipment, materials, tools and combustibles will not be stored near safety-related equipment, components, cables or piping either during work periods or non-work periods.

2. While equipment, materials and tools for construction may be brought into safety-related areas during work periods, they are required to be removed at the end of each work shift; consequently, safety-related areas will not be used for storage of combustible materials or equipment for any unattended period of time.

For these reasons, the proposed modification will not result in accumulation of miscellaneous combustible materials that could increase the potential for fires that could affect safety-related equipment, components, piping or cables. There are, however, areas where wood framing, planking, concrete forms and plastic will be necessarily in place for the period of time required to complete a particular aspect of work such as curing of concrete before forms are removed. The licensee has identified the following safety-related areas where wood will be used:

1. Control Room;

2. Cable Spreading Room;

3. Electrical Auxiliaries Room;

4. East wall of the Turbine Building between column lines 41 and 46.

The plastic and wood will be fire retardant with the exception of the wood used as concrete form material. This will not be treated with fire retardant chemicals to avoid any possible deleterious affects the chemicals may have on the new concrete. Inasmuch as these materials represent a significant increase in the quantity of combustible material for the period while the material is in the area, the licensee has committed to establish a fire watch whose sole responsibility will be to tour at hourly intervals the areas where these materials are used. This fire watch will be established whether the wood and plastic are fire retardant or not. This fire watch will not be necessary during the times when a continuous watch has been established in an area for other purposes. This fire watch will be instituted when the material enters an area and continue until it is removed.

This added measure of protection (fire watch patrol) will assure adequate protection for these areas where wood may, of necessity, have to remain during offwork periods.

As to modification work inhibiting access by the fire brigade to areas where a fire has been initiated, the proposed modifications and activities related thereto could inhibit access by:

1. the storage of construction equipment, materials or tools blocking ingress and egress to and from areas of the Control Building complex,

2. workers and actual work-in-progress blocking ingress and egress to and from areas of the Control Building complex,

3. the removal of access ways (e.g., stairwells, ladders) for performance of modification work.

As previously indicated, construction equipment, materials and tools will not be stored near safety-related equipment or cables. Consequently, access to such areas will be unaffected from the standpoint of storage of construction items. While construction equipment, materials and tools may be present temporarily in safety-related areas during any particular work shift, such items must necessarily be of a portable nature (since they must be removed at the end of each work shift) and will be stored so they do not block access required for operation or for firefighting.

As to workers and actual work-in-progress blocking access to any areas in the Control Building complex, it is to be noted that all modification work will be closely supervised; in the event that workers are blocking access, they can simply be directed to move. Also, work in the Cable Spreading Room, the Electrical Auxiliary Rooms and the Control Room will not restrict access to equipment either within these areas or in adjacent areas because the location of the work is against the walls out of normal paths to equipment and the tools and equipment to be used to perform modification work in these areas is small in size (e.g., wrenches, nuts, washers). In the case of the work at column N' in the Electrical Auxiliaries Room, the work will consist of cutting some concrete at floor level and therefore will not block access to any safety-related equipment. The plastic enclosures to be constructed in the Electrical Auxiliaries Room and which may also be constructed in the Control Room and Cable Spreading Room to prevent dust and smoke from entering these areas will be against walls and hence will not impede access to any safety-related equipment.

During our site visit on June 12-13, 1979, all areas of the Control Building complex were examined for possible operator/fire brigade access problems. Based on that examination and a consideration of the modification work to be performed, the only area where work-in-progress will affect access is at elevation 45 feet in the Turbine Building between columns 41 and 47. In this area, access through this area from the railroad bay to the Control Building will be impaired temporarily while plates 1 through 6 are lifted into place. This impairment of access will last only for the period of time in which the plates will be lifted from elevation 45 feet to their point of installation. This will not restrict access to any safety-related area where access is necessary either to operate equipment or to fight fires because safety-related equipment is not located in the areas to which access would be impaired. In addition, other access ways into these areas are available. These alternate routes provide access from the outside or the inside around the work area which is temporarily inhibited while the plates are being lifted into place. Consequently, we conclude that modification work-in-progress will have essentially no effect on operator or fire brigade access.

The proposed modifications were also examined to determine whether there were any other aspects of the modification work that could restrict access to various parts of the Control Building complex. To perform certain parts of the modification work, the steel stairway leading from the Turbine Building operating floor at elevation 93 feet to the Control Room viewing gallery will be removed. This stairway provides access only to the viewing gallery and is not an access way to any safety-related equipment, components, piping or cables since no safety-related items are in the viewing gallery area. An alternative access way is available into the viewing area should access be required to fight a fire in this area. In view of this, the impairment of access to the viewing gallery by temporary removal of the stairway will have no safety significance with regard to ingress and egress to and from the area by operators or the fire brigade.

Also, pursuant to the modification work, a ladder and steel platform leading to the Turbine Building roof and the crane cab at elevation 93' in the Turbine Building will be temporarily removed. However, alternate access to the roof will be provided by means of a temporary ladder. In view of this, the temporary removal of the ladder and steel platform will have no safety significance with regard to access to the Turbine

Building roof by operators or the fire brigade. Access to the crane cab is not necessary for safety-related purposes and therefore a determination of such access is not required.

The final concern related to fire protection is the possible impairment of fire barriers because of the modification work. Based on our review of all aspects of the proposed modification work, areas where work may affect a fire barrier are the areas where holes will be drilled through existing walls as identified in Section 4.1 above, the floor in the Electrical Auxiliaries Room at column line N' where holes will be drilled for anchoring a new wall, at column 41 R and possibly at 46 R, 46 N and 41 N at various elevations where concrete removal for rebar installation will result in penetration of the wall. Penetrations resulting from the drilling and concrete removal operations on these walls will provide a path through which fire could potentially pass. The Trojan technical specifications require a fire watch patrol to inspect, on an hourly basis, the areas where a fire barrier is nonfunctional. The licensee has committed to providing an hourly fire watch whenever wood is in any of these areas. This commitment is adequate to accomplish both purposes and a second fire watch need not be established during periods when such inspection is necessary due to wood being in the area. In addition, where holes have been drilled for bolt installation, the licensee has committed to temporarily seal the holes, until the bolts are installed, with the same material used to seal the plant's electrical penetrations of fire barriers. This is in compliance with Trojan Technical Specification 3/4 7.9, "Penetration Fire Barriers," which covers the requirements for periods when a fire barrier is impaired or is nonfunctional.

It is our conclusion that, with the fire watch proposed by the licensee, adequate fire protection will be provided to compensate for the disabling of fire barriers and for situations in which there is an increased potential for fire due to the introduction of ignition sources and combustibles and that there is reasonable assurance that the health and safety of the public will not be affected by the proposed modifications insofar as fire protection at the facility is concerned.

4.4 Modifications to Safety Systems Required for Performance of the Control Building Modifications

We have examined the proposed modifications with a view toward determining the need for intentionally taking equipment out of service during performance of part or all of the modification work. We initially determined that the only safety-related equipment which might be temporarily taken out of service was the Battery Room exhaust. This exhaust system provides ventilation air flow for the Battery Room and serves to remove hydrogen which may be generated in small quantities when the batteries are recharged.

The Battery Room exhaust system is an auxiliary system and is not required to bring the plant to, or maintain the plant in, cold shutdown, to mitigate the consequences of accidents, or to process or contain radioactive materials and limit their release to the environment. Consequently, the temporary removal of this system from service would not affect the ability to shut down the plant, mitigate accidents or limit releases of radioactivity.

On the other hand, the Battery Room exhaust system does serve to assure that any hydrogen generated during battery recharging is removed from the room and, for this reason, some means of ventilation is desirable. Accordingly, the licensee had committed to providing an alternate means of Battery Room ventilation during the period when the normal system might be out of service.

The licensee has since stated that the Battery Room exhaust duct will not be disabled during the modification. We have reviewed and determined that the exhaust duct need not be disabled to accomplish any modification work. Therefore, this matter is of no further concern.

4.5 Effect of Modification Work on Operator Actions

The potential concern with regard to modification work affecting operator actions is the possibility that the storage of construction equipment, materials and tools, actual work-in-progress, or the removal of accessways during the modification work could impair the access of operators to safety-related equipment and thus impair or prevent manual operator actions necessary for normal operation or to cope with emergencies. This matter is discussed in some detail with regard to fire protection and fire brigade access.

As indicated in the section on fire protection, construction equipment, materials and tools will not be stored near safety-related equipment or cables or in safety-related areas. Consequently, operator access to such areas will be unaffected from the standpoint of storage of construction items.

Similarly, except for work involving movement of plates at elevation 45 feet in the Turbine Building, work-in-progress will not impair operator access since workers and the tools and materials they are using will simply be moved out of the way if access is required. Moreover, areas at elevation 45 feet in the Turbine Building where access will be impaired while plates are moved are not safety-related areas containing any equipment for which access is necessary.

Finally, while the modification work will require the temporary removal of both a stairway leading from the Turbine Building operating floor (elevation 93 feet) to the Control Room viewing gallery and a ladder and platform leading to the crane cab and Turbine Building roof, no safety-related items requiring operator access are contained in these areas. Accordingly, temporary removal of these access ways will not impair operator access to any equipment, cables, piping or components which are necessary for plant operation or which serve a safety-related function.

4.6 Handling of Heavy Loads

The Control Building modifications will require the handling of some heavy steel plates not anticipated or accounted for in the Trojan licensing review at the operating license stage. To upgrade the Control Building horizontal shear strength, both ends of the Control Building railroad bay will be sealed with reinforced concrete. To further improve the horizontal shear strength of the Control Building west wall adjacent to the Turbine Building, a three inch thick steel plate will be attached to the outer surface of the west wall. The plate will extend from elevation 59'-3" to 97'-3" between columns 41 and 47, a distance of approximately 33'-6" at its widest point. It will be assembled from eight individual plates that will be lifted sequentially from a transporter in the Tubine Building railroad bay, and transported to their mounting positions with a combination of temporary and permanent plant devices. After being properly positioned, the plates will be anchored to the existing wall by bolting. All mating edges of individual plates will be joined by welding to form one continuous plate. The individual plates will range in weight from 2,700 pounds to 47,000 pounds. The upper plates (identified as plates 7 and 8) will be of such shape as to permit them to be lowered into position without disturbing the cables contained in four groups of cable trays passing between the Control Building Cable Spreading Room (elevation 77') and the Turbine Building.

Starting with the lowest plate, the first six plates (identified as plates 1-6), ranging in weight from 7,000 pounds to 24,000 pounds, will be sequentially lifted from the transporter and placed on a dolly at grade level (elevation 45'). The dolly will move the plates directly below the point where they will be lifted into position by the 16 ton capacity chain hoists mounted to the Turbine Building crane rail support beams. Aside from the support provided the chain hoists by the Turbine Building crane rail support beam, the rigging will be such that each chain hoist, sling and attachment point on the plate will be independent of the others. For the first six plates, two chain hoists will carry the load. A third chain hoist, positioned over each plate's center of gravity, will be provided to serve as a backup in the event of a single failure of either of the two load carrying hoists.

The safety margins of the load carrying hoists for the first six plates, with respect to their static load rating, will range between 2.6 and 9.1. Assuming a failure in one of the two load carrying hoists, the safety margin for the backup hoist, with respect to its static load rating, will range from 1.3 to 4.5. It should be noted that the static load rating of these chain hoists is 20 percent of their ultimate design load capability. Therefore, the safety margins, with respect to the ultimate load carrying capability of the hoists, are five times the above. This, we believe, is adequate because it provides substantial margin to failure of the hoists.

The essential systems that are located below grade and could be affected by dropping plates 1 through 6 are Train A and B fluid lines and electrical cables. With the exception of the Train B electrical conduit, the systems run below grade in the east-west direction parallel to the existing railroad tracks, and are located between columns 41 and 46. The fluid lines are the service water lines to both Emergency Diesel Generators, the service water suction lines for both the turbine and diesel engine driven Auxiliary Feedwater Pumps and fuel oil supply lines for both A and B Emergency Diesel Generators. The centerline of the water lines is located 2'-0" below the railroad bay floor (elevation 45') and the fuel oil lines are 3'-6" below the railroad bay floor. The concrete conduit containing Train A electrical cables supplies power to the Service Water, Component Cooling, Centrifugal Charging, RHR, Containment Spray, and Safety Injection Pumps. These cables range in depth below grade from 5'-9" to 8'-10". The other concrete conduit contains Train B electrical cables supplying power and/or control for the following Train B systems:

Component Cooling and Service Water Valves, Engineered Safety Feature Initiation, Diesel Generator subsystems such as Fuel Oil Transfer Pump, Fuel Oil Valve, and the Train B diesel air compressor. This conduit is located between columns 46 and 51 and also is in the area of the building modifications. It runs in the east-west direction. It is located directly below the southern end of the added Control Building west reinforced concrete wall. The below-grade depth of the cables ranges from 4'-8-1/2" to 8'-6".

To provide assurance that the essential equipment located below grade will not be damaged from impact loads in the unlikely event one of the plates (1-6) should be dropped, a temporary energy absorber will be provided at elevation 45'-0". It will be conservatively sized to accommodate the maximum kinetic energy capable of being developed by a plate during the handling operations without exceeding the allowable compressive stresses on the underlying concrete and rock foundation. It will consist of a crushable corrugated aluminum structure, two three-inch thick steel plates and an appropriate guide to ensure t the impact load will be distributed over the energy absorbing material.

From our evaluation of the proposed energy absorber, we conclude that it has been conservatively designed so as to limit the impact loading such that damage to the pipes and conduits below grade will be precluded. From this we conclude that the essential systems located below grade (elevation 45 feet) have been adequately protected in the unlikely event that any one of the plates numbered 1 through 6 should be dropped during the handling operations.

The last two plates, numbered 7 and 8, weigh 2,700 pounds and 47,000 pounds, respectively. After installation of plates 1 through 6, plates 7 and 8 are to be sequentially lowered into position above the four groups of cable trays which run between the Control and Turbine Buildings at elevations ranging from 77'-0" to 88'-3". To accomplish this, they will be lifted from the transporter in the Turbine Building railroad bay (elevation 45 feet) with the 25 ton capacity Turbine Building Crane auxiliary hoist to the Turbine Building operating floor (elevation 93 feet) and then transported laterally over the Turbine Building operating floor to a location within the reach of the chain hoists. Three of the 16 ton chain hoists will be attached to plate 7 (2,700 pounds) and then it will be lowered into position. Z bars mounted on the installed and anchored plates 5 and 6 will provide lateral guidance and restraint to plate 7 as it is being lowered into position. To reduce the impact load on the previously installed plates in the very unlikely event that plate 7 should be dropped, a corrugated aluminum HEXCEL stabilized and pre-crushed energy absorber will be mounted on plate 4 below. The acceptability of the drop of plate 7 is discussed elsewhere in Section 4. From this we conclude that the electrical cables in adjacent cable trays have been adequately protected should plate 7 drop while being lowered into position and that handling and installation of plate 7 while the plant is operating is, therefore, acceptable.

After being lifted to the Turbine Building operating floor, plate 8 will be placed on and attached to a temporarily constructed plate support frame and roller. Two A-shaped frames will be attached to plate 8 (one on each side) prior to the time that it is transported across the Turbine Building operating floor. The purpose of these A-shaped frames is to provide vertical stability and thereby prevent a flat plate drop of plate 8 onto the operating floor. To attain additional control over plate 8 during further movement, one end of the plate will be attached to one of the 16 ton chain hoists. By means of light rigging equipment, the plate will be positioned below the Turbine Building crane rail and the row of 16 ton capacity chain hoists. Three additional hoists will then be attached to independent lifting points provided on the plate. Two of the four hoists will carry the load and the remaining two attached hoists will act as backup or redundant lifting devices in the event either of the two load carrying hoists should fail. With respect to their rated static capacity, the safety margin of each of the two hoists supporting the load as well as each of the two backup chain hoists is 1.3. With respect to the ultimate design capacity of the chain hoist, the static capacity safety margin is 6.5 for each of the lifting and backup hoists.

To provide additional protection in the unlikely event that plate 8 should drop while being lowered into position, timber cribbing and stabilized, precrushed HEXCEL pads will be placed and supported by the top edges of plates 4, 6, and 7, prior to handling plate 8. The cribbing will be removed in 4 inch increments as the plate is lowered in a fashion to assure that the maximum free drop height will not exceed 4 inches in the unlikely event plate 8 is dropped.

The licensee has analyzed the dropping of plate 8 when it is being lowered into position to verify that the previously installed plates, bolts and associated structures could absorb the kinetic energy. The analysis shows that the dynamic load in the Hexcel pad will not exceed 374,000 pounds assuming a 30% increase in strength due to the dynamic load. The impact velocity is insufficient to make this assumption; therefore a factor of 1.1 is more appropriate. Use of this factor will result in an 18% increase in the deformation of the pad. However, the licensee assumed a factor of 1.0 in its calculation which is conservative. Therefore, the 1-inch Hexcel pad is acceptable. Section 4 addresses the ability of the structure to withstand this postulated plate drop.

To assure that plate 8 is properly guided in the unlikely event of a load drop, three one-inch thick plates 2 feet wide and one one-inch thick plate of lesser width will be securely bolted to previously installed plates. The upper end of these one-inch thick plates will also be held in the proper position by the Turbine Building floor slab curb at elevation 93'-0". Notwithstanding the above precautionary measures, the licensee has made the conservative assumption that all cables in the upper three penetrations will be severed if plate 8 is dropped. Should the plant be at power, the licensee stated that the plant could be brought to a cold shutdown condition by the plant personnel performing certain manual operations.

The load handling operations will be supervised and carried out by experienced and qualified Bechtel personnel plus journeymen skilled in their respective crafts. Prior to plate movements, procedures will be written, reviewed and approved by qualified Bechtel personnel. The craftsmen will be familiar with the equipment and procedures. All hoists will be tested prior to commencing the lift operation. Load cells will be incorporated in the load line of the backup or redundant chain hoists in order to reliably monitor the load lines to assure that they are taut but have not assumed an inordinate amount of the load.

Based on our evaluation of the heavy load handling operations (i.e., those operations involving the eight 3" thick steel plates that will be attached to the west wall of the Control Building), we believe that all reasonable and practical precautions will be taken to minimize the possibility of uncontrolled movement or dropping of plates. As recounted above, those precautions include the development, review, approval and implementation of written step-by-step procedures for plate handling operations, the use of personnel experienced in their crafts and familiar with the equipment, testing of lifting equipment prior to use, provision of adequate margins of safety in the lifting devices, and the temporary installation of an energy absorbing device in the railroad bay (elevation 45'-0") prior to the lifting and installation of plates 1 through 6.

In addition, plate guidance structures will be provided during the lowering of plates 7 and 8 and a crushable type energy absorption device will be provided to limit the impact loads to an acceptable level should plates 7 or 8 be dropped. With all of these precautionary measures implemented the licensee believes it would be acceptable to install all eight plates while the reactor is at power.

It is the staff's view that the handling of the heavy three-inch thick plates would be acceptable only if (1) it is demonstrated that the probability of uncontrolled load movements or load drops leading to unacceptable consequences is acceptably low, (2) or it is demonstrated that, even should an uncontrolled load movement or load drop occur, unacceptable consequences are precluded. Because of the difficulty of predicting the probability of an uncontrolled load movement or load drop, the licensee has taken the following approach: (1) implementing all reasonable measures to preclude an uncontrolled load movement and/or load drop when handling plates 1 through 8, (2) for plates 1 through 6, installing a temporary energy absorbing device at elevation 45'-0" to absorb the resulting kinetic energy of a plate drop and thereby protect the essential systems located below grade and precluding unacceptable consequences, (3) in regard to plate 7, providing a temporary plate guidance structure and impact limiter in the form of crushable energy absorber, and (4) in regard to plate 8, utilizing A-shaped frames to preclude a flat drop of plate 8 onto the Turbine Building operating floor and utilizing temporary timber cribbing and a crushable energy absorber to limit the drop height and the resulting dynamic loads to acceptable levels for a drop of plate 8 during installation between the Control and Turbine Building walls. Nevertheless, the licensee has made the conservative assumption that all electrical cables in the upper three cable tray penetrations would be severed by a drop of plate 8. With this assumption it is stated the plant can be brought to a cold shutdown condition with certain manual operations. However, to avoid the potential of unacceptable consequences due to any plant personnel's inability to assess the status of the plant and take the required corrective actions, the licensee has agreed to place the plant in a cold shutdown condition prior to the handling and installation of plate 8 and thereby obviate the need for the manual operations required to achieve a cold shutdown.

If, despite all precautions noted, plate 8 should fall during its installation between the Control and Turbine Building walls, it is possible that it could contact and damage some cables if, for example, the plate did not drop evenly at both ends. In this event, only train "A" cables would be affected. The redundant train "B" equipment needed for cold shutdown would be unaffected. Therefore, the maintenance of a cold shutdown condition would be unaffected since only one train of this equipment is necessary to maintain cold shutdown. Since the plant would have already been placed in the cold shutdown condition prior to the lowering of plate 8, the position of the valves for residual heat removal would already have necessarily been established. The severing of the "A" cables would not result in the changing of the position of these valves. The power cables for the train "A" pumps necessary to maintain cold shutdown are buried in the ground below the concrete floor and would not be damaged. These pumps could therefore be placed in operation by manual actuation of the breakers from the switchgear room.

It is therefore our conclusion that, although the drop of plate 8 and its subsequent damage to train "A" cables is a very unlikely event, the plant would still be maintained in a safe cold shutdown condition should such an event occur.

Consequently, based on our review and evaluation of the proposed plate handling and installation operations we conclude the following:

1. While the probability of uncontrolled movement or dropping of plates 1 through 8 has not been quantified, we believe that all practical measures will be taken to reduce such an eventuality. Further, the reliability and safety provided by the proposed lifting equipment are comparable to that required in NUREG-0554, "Single-Failure-Proof Cranes for Nuclear Power Plants." Thus, the probability of dropping any plate will be low, although it has not been shown to be so low that we can conclude that a plate drop will not occur.

2. For plates 1 through 6, an energy absorbing device will be provided to adequately protect the essential systems located below grade in the unlikely event of a plate drop accident. Therefore, it is acceptable for the plant to be at power during the handling and installation of these plates since the energy absorbing device will preclude damage to the pipes and conduits located below grade.

3. For plate 7, the previously installed plates 1 through 6 and the installed Z bars and energy absorbing device will limit the resulting dynamic load to an acceptable level and properly guide plate 7 so

that damage will not occur to the adjacent electrical cables in the unlikely event of a plate drop accident. Therefore, it is acceptable for the plant to be at power during the handling and installation of plate 7 since no damage affecting the safety of the plant will occur if plate 7 is dropped.

4. For plate 8, the analysis shows that the previously installed plates 1 through 7 plus the guide plates, crushable energy absorber and timber cribbing will limit the drop height such that the dynamic load will be kept within acceptable levels in the unlikely event that plate 8 should drop during installation between the Control and Turbine Building walls. However, some uncontrolled motions may occur during a drop that could place the electrical cables in the upper three cable penetrations at risk. Due to the uncertainties associated with this event and the fact that, should plate 8 damage these cables, substantial manual (and as yet undefined) operator actions might be necessary to bring the plant to cold shutdown, we believe that the plant should be placed and held in a cold shutdown condition from the time plate 8 is lifted by the Turbine Building crane from the transporter at elevation 45 feet until plate 8 is secured to the Control Building west wall by all through bolts.

No modifications are being proposed to the Emergency Diesel Generator exhaust system, but we have reviewed the potential consequences in the unlikely event of uncontrolled movement of the 47,000 pound plate (plate 8) while it is being moved into position at the east end of the Turbine Building wall at elevation 93 feet. During this movement, the potential exists for the 47,000 pound plate to fall against and disable the two B train emergency diesel generator exhaust systems. The A train emergency diesel generator would be unaffected by this event because its exhaust systems are located near the opposite (west) Turbine Building wall. In addition because the plant will be in cold shutdown condition prior to the plate 8 handling operations, damage to Train B diesel generator exhausts should have no affect on the ability to bring the plant to or maintain it in a cold shutdown condition provided, of course, that the Train A emergency diesel generator is operable. Prior to handling plate 8, we believe that it should be verified that the Train A emergency diesel generator is functionally capable of responding to any demand for emergency power. For the above reasons, in the unlikely event that the 47,000 pound plate should fall against the B train exhaust system, we find the consequences acceptable.

As previously mentioned, two A-shaped frames will be attached to plate 8 to preclude a flat plate drop onto the Turbine Building operating floor while the plate is being transported across the floor. As indicated in Section 4.11.1 of this SER, the structural adequacy of those A-shaped frames to preclude a flat plate drop has not been determined by the Staff because the licensee has not finalized the design and connection details for these frames. Nevertheless, assuming that the final design of the A-shaped frames is shown to be adequate to preclude a flat plate drop, we believe that the A-shaped frames should be installed on plate 8 and utilized throughout the transport of that plate over the Turbine Building operating floor. More specifically, the A-frames should remain attached to the plate as it is brought into position for insertion into the the slot between the Control and Turbine Building walls. As plate 8 is brought into this position, the A-frame on the side of the plate nearest to the Control Building should be removed only when such removal is necessary to prevent contact between the A-frame and the Control Building. The A-frame on the opposite side (or west side) of the plate should be removed only after all four chain hoists have been secured to the plate and made taut and the plate is partially inserted into the slot between the Control and Turbine Building walls. If this procedure for removal of the A-shaped frames is followed, it will provide maximum coverage for assuring that a flat plate drop onto the Turbine Building operating floor will not occur (assuming that the A-frames, once installed, are adequate to prevent a flat plate drop) since, once the plate is partially inserted into the slot between the Control and Turbine Building walls, there is reasonable assurance that a flat plate drop onto the Turbine Building operating floor will not occur.

4.7 Construction Generated Dust and Dirt

The control, monitoring and powering of essential systems* is largely accomplished by electrical means which in turn require electrical contacts. When dust, dirt and/or grit is deposited on electrical contacts, the likelihood of making an acceptable electrical contact is very significantly reduced. Since the Control Building modifications involve the removal and replacement of concrete, building blocks and dirt as well as the drilling of many holes in the concrete, these operations have the potential of creating significant amounts of these airborne particulates that

* Essential systems for the purpose of this review includes those systems needed to bring the plant to cold shutdown conditions and maintain it in a cold shutdown condition and those systems required to mitigate the consequences of accidents.

were not anticipated or accounted for in the Trojan operating license review. Therefore, the staff has reviewed the proposed Control Building modifications to ensure that adequate measures have been taken to preclude essential system malfunction due to dust, dirt and grit.

The areas housing essential systems or the essential systems themselves that potentially could be impaired by construction dust and dirt are:

(a) Diesel Generator Systems, located in the Turbine Building at elevation 45 feet.

(b) Electrical Auxiliary Room, located in the Control Building at elevations 61 and 65 feet.

(c) Engineered Safety Feature Switchgear Room, located in the Turbine Building at elevation 69 feet, and

(d) Control Room located in the Control Building at elevation 93 feet.

4.7.1 Diesel Generator Systems

The potential adverse effects of the Control Building modifications on reliable operation of the on-site emergency diesel generators was reviewed from the standpoint of the quality and quantity of ventilating and combustion air available during and following the building modifications.

Currently, the air intake system for the emergency diesel generators relies on an opening to the outside through the railroad bay in the Control Building. Before the Control Building railroad bay is sealed off at column line R (Control Building west wall), an alternate air intake system will be provided in the north wall of the Turbine Building railroad bay. Therefore, an adequate supply of ventilating and combustion air will be available throughout the modifications.

The licensee addressed the unlikely event of the outside louver for the relocated air intake being impaired by earthquakes, wind, tornadoes, rain, ice and snow. The new Turbine Building air intake louver has been sized (182 square feet) to allow the blockage of 50 percent of the intake area by snow, ice or debris. The Turbine Building air intake louver has not been and need not be, designed to withstand abnormal loads, since collapse of the louver would not preclude a sufficient air supply to the diesel generators. If the Turbine Building railroad bay air intake louver were to be damaged or were to become clogged, sufficient air could be temporarily drawn from the Turbine Building ventilation system through the Turbine Building equipment hatch at least until the louver is repaired or the obstruction is removed from the louver or the Turbine Building railroad bay door is opened. The total air requirements for both A and B train Emergency Diesel Generators would constitute approximately 10 percent of the Turbine Building's air flow rate.

Further, should debris pass through the Turbine Building air intake louver, the larger pieces of debris would tend to settle out, because of the lower velocity of the air in the Turbine Building railroad bay, before the air is drawn into the individual diesel generator compartment air intake systems.

The creation of dust and dirt will be controlled during the building modifications by the use of water sprays and the performance of regular cleanups. Should it prove necessary, evacuation fans and ducts will be provided in the respective work areas. Upon completion of the work, normal housekeeping will preclude the presence of residual dust. All construction work will be suspended during the periodic diesel generator tests and following all automatic starts of the units. Consequently, construction generated dust and dirt should have no effect on the quality of the combustion air for the diesel generators. Further, it should be noted that all electrical relays for the engines, located within the diesel generator compartments, are housed within water-tight and dust-tight enclosures. Consequently, dust generated by the modification work should not affect electrical relays and contacts for the diesel generators.

Based on our review of the proposed modifications, we conclude that the diesel generators will not be adversely affected due to insufficient air supply, dust or dirt during and following the Control Building modifications and, therefore, the proposed modifications are acceptable in this regard.

4.7.2 Electrical Auxiliary Room

The Electrical Auxiliaries Room is located in the Control Building between elevations 61'-0" and 77'-0" and columns 41 and 51. This room, which is normally closed and locked, contains safety and non-safety related equipment. The licensee has provided preliminary details illustrating the concrete modifications to be carried out on the floor and walls. The following Control Building modifications have been identified as potential sources of adverse conditions such as dust, dirt, debris and water that have the potential for degrading the safety-related equipment housed within the Electrical Auxiliaries room.

(a) At the junction of column lines R and 41, all concrete in the corner will be removed, between elevations 65'-0" and 77'-0", in order to join the additional rebars of the strengthened R line wall to the existng rebars by Cadwelding.

(b) A series of 2 inch deep slots (approximately 2'-0" long) and holes will be cut in the floor along column line N' from columns 41 to 46 for anchoring the rebar to the top of the N' wall at elevation 65'.

(c) A number of holes will be drilled in the R and N line walls in order to tie the added walls to the existing structure.

The following safety-related equipment has been identified as being within the area of influence of the column 41R modifications.

(1) 120 volt preferred instrument AC panels - Y11 and Y13
(2) Plant Static Inverters - Y15 and Y17
(3) Solatron Line Voltage Regulator - Q35 and Q37
(4) Battery Charger - D21
(5) Train A Cable Tray ABA 288 - contains cables associated with the following train A safety related equipment:

a - Power for Inverter - Y15
b - Power for Battery Charger - D21
c - Power for Inverter - Y17
d - Power for Battery Charger - D23
e - Control for CCW and Service Water Valves - CV 3715A, SV 3303, SV 3287, and SV 3725
f - Control for Pressurizer Relief Valves PCV 455A, PCV 469 and CV 8021
g - Control for RCS Valves
h - Control for Safety Injection Valves
i - Main Steam Line Valves, SIS Test Line Valve
j - Control for Diesel Generator A - G101
k - Control for AFW and CVCS valves MO 3071 and CV 8149 A, B, C
l - Control for Turbine Trip - Train A
m - Control for ES 480V LC - B01/B03
n - Control for ES 4 KV switchgear A1
o - Control for CVCS valves CV 8149 A, B, C
p - Control for Reactor Trip Breakers
q - Power for NIS cab A Control C 31A
r - Power for NIS cab A Instr. C31A
s - Power for Process Protection Set - I C36A1
t - Power for Process Control Group I C36A6
u - Power for Diesel Generator - G101
v - Power for Radiation Monitoring Panel - C41A
w - Power for SS Protection Input Rack - C46A
x - Power for SS Protection Input Rack - C47A
y - Power for Containment Pressure Instrumentation
z - Power for ESF DBA timers, AFW Pump Auto Start Circuit
aa - Power for C.R. Vent Indication - C254
ab - Power for Safety Injection Status Lamps - C29A
ac - Power for Steam Generator Blowdown Sample Valves - SV 2809A, 11A, 80A, 14A

Removal of concrete in the wall corner at 41 R will commence with cutting 1/2" deep vertical grooves at the boundary of the material to be removed. A 60 pound jack hammer, a chipping hammer, and wire brush will be utilized in breaking up and removing concrete and exposing the existing rebar. The existing rebar will be cut and joined by Cadwelding, to the new rebar. Prior to commencing this modification the work area will be isolated from the equipment in the Electrical Auxiliaries room by erecting a dust-tight flame retardant enclosure from the floor (elevation 65'-0") to the ceiling (i.e., underside of the Cable Spreading Room floor-elevation 77'-0"). Therefore, the dust, dirt, debris and smoke generated during this modification work will not degrade the equipment within the Electrical Auxiliaries Room.

As a result of our review of the additional information submitted in the December 17, 1979 submittal (response #7 to staff questions of 9/28/79) regarding the building modifications at 41R, the staff believes that the licensee has failed to demonstrate that the proposed measures will adequately protect the above safety-related systems from missiles. The above safety-related equipment may be damaged by missiles generated by the modification activities or by missiles generated by other outside conditions because the current missile barriers (existing wall) will be removed, thereby exposing the equipment.

Based on our review of the described modifications and protective measures to be taken at column 41R we conclude that (a) the proposed measures to keep dust, dirt, debris and water within acceptable limits are acceptable provided the work is periodically monitored and continued work is contingent upon approval by the NRC Resident Inspector, (b) the plant should be placed and held in the cold shutdown condition throughout all phases of the modification work where essential equipment is vulnerable to damage from natural as well as building modification-work-generated missiles.

The licensee's December 17, 1979 response to staff questions indicates that the design details of the Electrical Auxiliaries room modifications are not yet finalized. It is possible, as the details are finalized, that work similar to that described at column 41R may be required at columns 46R, 46N and 41N at various elevations. In the event similar type work is required at these other locations, the licensee has committed to providing the same protective measures as those described for 41R.

Considering the present lack of detail regarding the work required at 46R, 46N and 41N and the specific equipment at risk at each of these locations, we can only conclude that the positions taken in regard to the work at 41R would be equally applicable to work at 46R, 46N and 41N. Consequently, it is the Staff's position that the plant should be placed and held in the cold shutdown condition throughout all phases of the modification work where safety-related cables and equipment are vulnerable to damage from natural as well as building modification-work-generated missiles related to concrete removal work at column lines R and 41, R and 46, N and 46, and N and 41.

A diamond tipped concrete saw and a hand held chipping hammer will be utilized in making the series of 2" deep slots in the Electrical Auxiliaries room floor along column line N' between columns 41 to 46. Further, a diamond tipped core drill will be used to drill the two holes, in the bottom of each slot, through the remaining concrete topping and the precast floor panels for the placement of the U shaped rebar to be used to anchor the top of the N' wall to the floor slab. Water will be sprayed on the diamond tipped cutting tools. It acts as a lubricant and a cooling medium and also prevents the dust from becoming airborne. A small hand held chipping hammer will remove the material between the saw cuts. It will not generate vibratory motion sufficient to affect any safety-related equipment. The dust, dirt, debris and excess water will be continuously cleaned up using mops, brooms, dust pans and a shop vacuum.

We conclude the described measures for work done at 41R and column line N', if properly implemented, will adequately assure that this modification work will not adversely affect the safety-related equipment in the vicinity of the work from the standpoint of dust and dirt generation. Such measures should also be implemented if similar work is required at 46R, 46N and 41N.

To provide assurance that the adverse affects of the modification work activities are held to acceptable limits (e.g., dust, dirt and debris), the resident NRC inspector will periodically monitor the work. If in his judgment the measures taken to protect the essential equipment are inadequate, the work shall be stopped until adequate measures have been implemented.

Anchoring of the three-inch thick steel plates to the Control Building R line wall will require that holes be drilled into the west wall of the Electrical Auxiliaries room. Further, anchoring a new wall at column line N will require drilling into the east wall of the Electrical Auxiliary Room portion of the Control Building between columns 41 and 46. For the drilling of these holes, a positive feed control drill will be utilized. To suppress the generation of airborne dust, water will be directed on the drill. To limit the accumulation of construction dust and dirt, regular cleanups will be performed.

The licensee has indicated that (1) diamond tipped core drills and water spray will be used (drilling from the outside), and (2) the workman, stationed at the breakout area on the inside wall, in addition to monitoring when the positive feed drill penetrates the walls, will hold a small enclosure against the wall to collect and contain any generated dust, dirt, debris and water when the breakthrough occurs. In addition to the previously described measures taken to contain and collect any dust or debris that may be released in the room, the licensee has committed to employ alternate equipment to reduce dust level in the unlikely event that the dust level should become excessive (that is, reach a level at which it could potentially affect equipment performance). Filtered ventilating air is supplied to this room through an air intake located on the Control Building roof at elevation 116', far above the level at which dust and dirt will be generated from construction work. Routine surveillance will be made to verify that no dust enters the room around the door seals as a result of other construction activities being carried on outside the room.

Based on our review of the submittals and compliance with the requirements stated herein, we concude that adequate measures will be employed to preclude dust, dirt and debris levels which could adversely affect the operation of the equipment within the Control Building Electrical Auxiliary Room. The proposed modifications are, therefore, acceptable with regard to the control of construction-related dust, dirt and debris and water in the Electrical Auxiliary Room.

4.7.3 Engineered Safety Feature Room

The Engineered Safety Feature Room, located in the Turbine Building at elevation 69 feet, has no outside air intake system. Ventilation and cooling are provided by a recirculation system. The room is closed and locked. During the modifications, periodic routine surveillance of the room will be performed to verify that no dust or dirt has entered around the door seals. No modifications will be made to the room's existing floor, walls, ceiling or penetrations and, as such, the modification work will not generate dust or dirt within the room itself.

Based on the measures to be taken to control the generation and accumulation of dust at the respective work areas and the periodic surveillance for the entry of dust into the Engineered Safety Feature Room, we conclude that sufficient measures will be taken to preclude any adverse effects on the equipment within the Engineered Safety Feature Room from dust and dirt generated during the building modifications.

4.7.4 Control Room

The Control Building modifications will require the drilling of a number of holes in the walls of the Control Room. On the west wall, the holes are for anchoring the three-inch thick plates to the building's outer surface and on the east wall for bonding of the added reinforced concrete wall. The drilling of the holes has raised concern about the possible entry and adverse effects of construction-generated dust and dirt on the operation of the equipment in the Control Room. The concern in this regard is that dust could settle on electrical contact points thereby impairing adequate electrical contact.

The drilling operations will be conducted from the outside wall surface with drilling equipment using a positive feed control, diamond tipped core drills and wet drilling techniques. The wet drilling technique will significantly reduce the generation of dust. In addition, the workman stationed at the inside wall, to visually observe and alert the driller when the wall has been penetrated, will also hold a small enclosure over the area to collect and contain any resulting dust, dirt, debris and water when breakthrough occurs. Further, regular periodic cleanups will limit the accumulation of dust and dirt. The licensee has indicated that if, in spite of the above measures, excessive levels of dust or dirt result from the drilling operations, the licensee will use lighter equipment. In addition, all Control Room electrical equipment housing electrical contacts are located in fully enclosed cabinets. The Control Room may be required, under certain conditions, to be maintained at a slight positive pressure relative to the outside. In order to maintain the capability of developing and maintaining a slight positive pressure differential in the Control Room, should it be required, each hole will be temporarily plugged after being drilled. Further, during installation of the through bolts to support the steel plate, the sequence of steps will be such that not more than one hole will be open to the atmosphere at one time.

We believe that the above precautionary measures taken during drilling will limit the amount of dust entering the Control Room to a level where it will have no adverse effect on the operability of equipment and instrumentation.

4.8 Noise and Vibration

Certain activities of the proposed modification work will generate noise and vibration at the specific site of the work. In order to determine whether the noise will have any adverse affect on Control Room operator actions and whether vibration should have any effect on equipment or components, we have examined the locations of the work that could generate noise and vibration relative to the location of safety-related equipment and the Control Room as well as the measures proposed to mitigate the effects of noise and vibration.

The following proposed modifications will be the potential source of some noise and vibration:

1. Numerous holes will be drilled in the west wall of the Control Building between columns 41 and 46 from elevations 59'-3" to 97'-3" for the installation of the steel plates.

   There will also be through-wall holes drilled in the east wall of the Control Building between columns 41 and 47 from elevations 65'-0" to 95'-6" for structurally connecting the new wall to the existing wall. The new east and west Control Building walls enclosing the railroad bay between columns 41 and 47 and elevations 45'-0" and 65'-0" will be structurally attached to the existing walls using reinforced steel grouted into holes drilled into the existing structure. The Control Building interior wall along column N' will be strengthened by the addition of a new reinforced concrete wall and footing in the railroad bay. Its elevation will be from below elevation 45'-0" to the underside of the floor at elevation 65'-0". It too will be structurally connected to the wall and existing ceiling utilizing drilled holes and embeds.

2. The concrete in the Control Building corner area at the junction of columns 41 and R line will be removed in order to expose, partially remove, and re-install additional reinforcing steel in order to properly join the existing east-west wall along column 41 to the newly modified R line wall from elevation 65'-0" to 77'-0". Concrete removal may similarly occur at columns 46R, 46N and 41N at various elevations.

3. Removal of Control Building concrete blocks will be required. For example, a new opening will be made in the block wall at column line 46 between column lines R and Q at elevation 45'-0". A portion of the Turbine Building floor slabs at elevations 93'-0" and 69'-0" along column line S between columns 41 and 46 will be removed to provide clearance and eliminate interference with the new Control Building west wall. The floor slab and railroad spur foundation concrete will be removed along column line R between columns 41 and 46.

4. A two-inch wide strip will be removed from the top and bottom flanges of the existing Turbine Building steel girder at elevation 93'-0" in order to obtain the required clearance between the new Control Building west wall and the east wall of the Turbine Building.

5. The relocated railroad spur shown in Figure 3.1-1 of PGE-1020 which is outside of the Control Building will involve the removal of approximately 350 cubic yards of rock and fill of a natural depression approximately 250 feet from the point of excavation.

For the performance of all this work, the licensee has committed to the following:

a. Explosives will not be used for the removal of structural material or earthwork;

b. The tools employed to accomplish the work will have the minimum noise level consistent with the task;

c. The removal of the earthwork associated with the new railroad spur will use hydraulic or air operated hammers, small front end loaders and dump trucks;

d. Removal of conventional back fill material around the building will be accomplished using light hand tools such as shovels. Should rock be encountered, light power tools will be used.

As to vibration, it is expected that the most severe vibration-producing work is that involving the use of air or hydraulic operated hammers and small front-end loaders for excavation and fill for the relocated railroad spur. This excavation and fill work will occur outside existing buildings at grade level. Similar work has been performed at the site in the past and has not resulted in excessive vibration in the structures.

Also, hydraulic operated hammers and light power tools will be used for the removal of fill material around buildings and for the removal of concrete and block at columns 41R (and possibly at 46R, 46N and 41N). While all of this work as well as the drilling operations may create some low level of vibration, we believe that vibration resulting from performance of the modification work should have no effect on the equipment which must be qualified to withstand the severe vibratory motions from earthquakes up to and including the SSE.

As to noise, the concern in this regard is that construction-generated noise will interefere with operations in the Control Room, either by drowning out the annunciators or by interfering with operators' voice communications. As can be seen from the previous listing of work to be performed, with the exception of the drilling of holes in walls and the removal and replacement of concrete, construction will be carried out at an appreciable distance from the Control Room which is located at elevation 93'. The distance from construction activities mentioned above should serve to substantially reduce the level of noise reaching the Control Room from the site of the construction activities. Moreover, the existing Control Room walls and floor slabs will serve further to muffle noise coming from construction activities. Although drilling into the Control Room east and west walls will occur, the drilling operation will involve the use of diamond tipped core drills. This will materially reduce both noise and vibration generated by drilling because of the large number of tool cutting edges which provide clean smooth cutting. A water spay on the drill will also minimize noise and vibration. In addition, the positive feed of the drill allows noise and vibration to be minimized through low speed feeding. Finally, drilling will take place from the outside surface of the Control Building walls. Consequently, the noise from drilling will be attenuated by the wall itself.

Because of the various factors set forth above, we believe that noise from construction work will not adversely affect operators' communications or the ability of operators to hear annunciators in the Control Room. Moreover, the licensee has committed to maintain construction noise to levels so as not to interfere with normal voice communication in the Control Room. Should the Control Room operator determine that construction noise interferes with normal voice communications, lighter weight tools or other means of material removal will be employed to reduce noise. We believe that in order to give added assurance that construction noise will be maintained sufficiently low in the Control Room, the NRC Resident Inspector should be empowered, as a condition of the proposed modifications, to require the use of lighter weight tools or the employment of other means for material removal to reduce noise, in the event that he determines that construction noise is interfering with normal voice communications in the Control Room. Thus, voice communications among the operators is assured, and, since the sound level of Control Room annunciators is significantly above that of normal voice communication, the ability to hear annunciators is also assured. Consequently, we conclude that the proposed modifications are acceptable from the standpoint of the noise they will generate.

4.9 Inspection of the Control Building Modification Work

The Control Building modification work will be examined by a construction inspector from the Region V Office of Inspection and Enforcement and by the Resident Inspector. The inspections conducted will include examination of quality assurance implementing procedures, construction procedures, specifications, drawings and quality records. The actual work-in-progress will be examined, on an audit basis, for conformance to the codes and standards referenced in document PGE-1020, "Report on Design Modifications for the Trojan Control Building," Revision 4 Section 3.2.2 Applicable Codes, Standards, Guides and Specifications. The work expected to be examined will include material receipt, material storage, drilling and grouting of dowels, reinforcing steel and concrete placement, concrete testing activities, concrete curing, welding, filler material control, nondestructive examination of welds, stud welding, and high strength bolting.

The inspections will be conducted to confirm that the modification work has been and is being performed in accordance with the approved design and procedures and that the Quality Assurance program addressed in the PGE-1020 report has been implemented and followed. The quality of the materials used will also be assured through the inspections conducted. Confirmation of completion of the modification work will be assessed by the above-described inspections and review of as-built drawings, quality records and QA reports on completed work.

It is expected that approximately 150 inspector manhours will be required for this inspection program.

4.10 Work Sequence

The implementation of the previously described modifications will be as discussed herein. Generally, the installation sequence of the new shear walls will be to work simultaneously on all three walls below elevation 65', to drill bolt holes in the R line wall above elevation 65' and to complete the entire N' wall. These items will be done concurrently. Upon completion of the N' wall and the concrete reaching its design strength, construction will resume on the R line wall (excluding plate 8 installation) and the N line wall above elevation 65'. Total time estimated to complete each wall is 2 months for N', 8 months for R, and 6 months for N.

More specifically, excavation of parts of the existing railroad spur foundation and exposing existing column foundations at columns N'-41 and N'-46 will commence, excluding the use of explosives, to begin installation of the new shear wall along N'. This foundation will be formed and poured. Wall core reinforcing steel will be made continuous along column line 41. The N' wall will then be formed and poured. Below elevation 65', other work to be done concurrently includes the removal of a portion of a non-shear wall between columns Q and O along line 46, a short section of a non-shear wall north of column 46 along R, installation of R and N line walls up to elevation 65', and drilling of bolt holes above elevation 65' in the R line wall.

Steel plates are to be installed on the west side of the R line wall. Preparation to install the steel plates will include the removal of edges of floor slabs from column line 41 to beyond column line 46, and temporary removal of platforms, stairs and other interfering facilities above elevation 93' in the Turbine Building. Concrete in the R line wall will be poured to El. 59'-3" and when the concrete in the wall reaches a minimum strength of 2000 psi, steel plate #1 will be raised into place by chain hoists, through-bolts will be installed, and more concrete will be poured to about 3" from the top of the plate using the plate as a form. This concrete will also raise the wall from the previous elevation 59'-3" to just beneath the elevation 65' slab. Upon the N' wall concrete reaching its design strength and the installation of the R line wall up to elevation 65' (concrete between elevations 59'-3" and 65' need not be at design strength) work will progress above elevation 65'. Plate #2 will then be lifted into position above plate #1, welded to plate #1 and the same procedure for installation of the concrete will be followed. This procedure will be repeated for all plates (excluding plate #8) with grout to be used behind plates above elevation 77'. Bolts will then be tightened to the prescribed torque value and the items temporarily moved will be replaced.

Concurrently with the R wall, the N line wall will be erected using common construction methods for installation of rebar, tie-ins to the existing structure, and forming and pouring of concrete. The wall will be poured to elevation 65' before any work above this level begins. After the wall has been poured to elevation 65', (concrete design strength not required), the work will progress to elevation 77' and similarly until the wall is completed.

Work other than shear wall erection that will be performed includes the installation of a louvered section in column line 41, west of column S in the Turbine Building and a new railroad spur to the Fuel Building. The louvers provide an air supply route to the diesel generators and will be installed before sealing off either end of the railroad bay with the new shear walls. The railroad spur will replace the existing one through the Control Building which is being dismantled. Again, no explosives will be used for excavation, including excavation of rock. The railroad spur construction is independent of the other modifications and can be performed when desired. Lastly, the newly enclosed railroad bay will be turned into office and work spaces which will require installation of a lightweight structural floor system at elevation 55'.

4.11 Effects of Modification Work - Structural Evaluation

4.11.1 Plate Drops

As discussed in PGE's responses to NRC Question 5 (9/20/79) and Question 6 (9/14/79), effects of a postulated plate drop on the plates previously installed and on the existing floors, walls and supporting crane girder were considered. The crane girder was evaluated f‹ postulated plate drop effects, including dynamic effects, and found to be adequate.

Additionally, the floor at elevation 93' has been examined for a postulated drop of plate #8 since it must be transported along the floor before the hoists are attached. The plate is to be transported on 2 A-shaped frames to preclude a flat plate drop. During transport of plate 8 on the A frames, it will be no higher than 1 inch above the floor. As the plate is being maneuvered to seat it onto the plate support frame, it will be about 2 feet above the floor. During this operation, a jack will be situated under one end to preclude one end from falling. All of the floor supports were evaluated and found to be adequate. As for effects on previously installed plates below and gross wall effects during installation of plates #7 and #8, both plates #7 and #8 will have precrushed Hexcel pads placed on the plates below to absorb the energy of a postulated drop and therefore limit the force transmitted to the plates below to an acceptable level. Additionally, for plate #8, 4" X 4" wood planks will be stacked vertically on the plates below and held by guide plates so that the maximum distance plate #8 can fall during installation is 4". The wall was evaluated for this postulated drop and found to be adequate. Additionally, capacities of numerous elements that are subject to a postulated plate drop have been investigated and found to be adequate. In addressing possible plate drops, the licensee has considered the consequences of such an occurrence and has instituted numerous safety measures discussed previously to preclude such an occurrence. We have evaluated these safety measures in detail. Our evaluation of the safety measures to be used during the installation of plate 8 has been based on the assumption that the various safety precautions will be adequately implemented and that the shutdown for plate 8 installation will not result in an expeditious scheudle that could induce carelessness in plate handling.

The staff concludes that should a drop of plate #8 occur while it is being lowered into place along the wall behind the guide plates or a drop occur on a corner or edge as the plate is being transported along the Turbine Building operating floor to the wall, the effects would not create safety hazard.* The resisting mechanisms to falls of these types have been examined and shown to be adequate. Regarding plate #7, it too will have a Hexcel pad energy absorbing

* Has not been analyzed for the simultaneous occurrence of impact and earthquake loads.

system installed in the slot between the Control ar i Turbine Buildings. The loads induced by a drop of plate #7 have been calculated and found to be less than those for plate #8 thus making plate #8 the more critical. Therefore, a drop of plate #7 will be adequately resisted and not create a safety hazard. The above is contingent upon the following concerns being resolved resolved before plate installation: (1) possible "kicking out" of the jack from beneath the edge of plate 8 as the plate is being lifted onto the plate support frame, and (2) showing that the A-shaped frames attached to the plate as it is being transported along the Turbine Building operating floor will preclude a flat plate drop. In addition, the staff will require that: (1) the concrete behind plates #1-6 reach design strength before plates #7 and #8 are installed because the resisting mechanisms to a drop of #7 or #8 rely on concrete strength, (2) the bolts in plates below #7 as #7 is being installed and below #8 as #8 is being installed be tightened to the value needed to produce the friction force relied upon in resisting a drop of plate #7 or #8, and (3) should a drop of #7 or #8 occur, plates #1-6 be removed, the wall examined by an NRC inspector for possible damage and, if necessary, repaired.

4.11.2 Excavations

During excavation, as stated, no explosives will be used. The location of underground lines have been identified and hand tools will be used when excavating in proximity to those and to existing facilities to insure that they are not damaged. The staff concludes that these measures will give reasonable assurance that no damage will be done to existing facilities and underground lines from excavation work.

4.11.3 Removal of Parts of Existing Structures

During the installation of the new shear walls, holes will be drilled into existing structural elements so that the units may be tied together. It is also necessary to drill through-holes in walls in some locations. Checking of as-built drawings and use of metal detectors to locate the existing rebars will minimize damage to them. In response to NRC Question #1 (10/2/79), it has been shown that a drill operator can detect hitting a rebar so that drilling can be stopped and minimal damage inflicted on the rebar. PGE concludes that such damage will have inconsequential effects on the wall capacity even considering hitting multiple rebars. It is concluded by PGE that the reduction in wall area due to the bolt holes is less than 4% in the horizontal plane, 6% in the vertical plane and 5% in any diagonal plane and these reductions in shear areas have been considered in evaluating shear capacities of the existing walls. The staff believes that due precaution is being taken by the Licensee to locate rebars before drilling commences and that should striking the rebar by the drill occur, no significant degradation to the Complex will occur. PGE has demonstrated that the maximum indentation that will occur in a rebar should it be struck by a drill will be about 1/8" (Response to NRC Question 1 of 10/2/79 submittal). If one bar is struck, location of the other bars can be better established and result in a lower likelihood of other bars being struck. The staff believes that the amount of material removed from the wall is small and will not significantly degrade the Complex. Additionally, this removal has been accounted for in evaluating capacities.

Face masonry, and in some instances, core concrete will be removed at various columns when tie-ins of the new walls are made. This will be done along N' at columns 41, 46; along column line R at columns 41, 46; and along N at columns 41, 46. As discussed in PGE-1020, Rev. 4, Section 5.3.2.1, the steel columns will be exposed only after certain modifications have been installed. The Licensee states this will result in the capacity of the shear walls in the Complex to remaining above the capacity required to resist the .25g SSE using the seismic criteria in PGE-1020.

For the currently proposed work sequence, it is possible to expose columns N-41, N-46, R-41, R-46, N'-41, N'-46 simultaneously below elevation 65'. The licensee has indicated that no credit was taken for the steel columns or masonry reinforcing at wall panel vertical boundaries when calculating the flexural capacity of the walls. This is true but both single and double curvature modes of flexure must still be accounted for when calculating flexural capacity of panels adjacent to these columns. As is currently proposed above elevation 65', no columns will be exposed until the new N' wall is installed and the concrete has reached design strength. It will then be possible to expose columns R-41, R-46, N-41, N-46 simultaneously since work on column lines R and N is scheduled to proceed concurrently. Exposing the columns affects the ability of a wall to resist a single curvature flexure mode of failure. In calculating the capacity to resist this mode of failure, reliance is placed on the shear-friction concept which utilizes rebars anchored in concrete and crossing a crack plane with the concrete in contact. If the column is exposed, the shear friction concept no longer holds. Additionally, at N-41 and R-41, exposing the columns would affect shear transfer around the corners. R-41 and R-46 will be exposed above elevation 65' to elevation 77' only after the the concrete wall along R has been installed to elevation 65'. At this point, the concrete in this wall will be at design strength below El. 59'3" but need not be at design strength between El. 59'-3" and El. 65' (the wall is to be erected in two pours). Columns N-41, N-46 will be exposed above El. 65' to El. 77' only after the new N line wall is installed to elevation 65'; however, it too need not be at design strength. The N wall would then be poured to El. 77' and columns between El. 77'-93' exposed although the wall beneath need not be at design strength. This process would be followed to roof level. The licensee believes that the vertical shear at these locations would be reduced sufficiently due to the completion of the N' wall and the new concrete in the new R wall or N wall being placed to elevation 65', although still below design strength. It is unclear to the staff how this vertical and horizontal shear will be sufficiently reduced along the columns above El. 65 given that the concrete in the new N wall below El. 65' and the R wall between El.59'-3"-65' need not have attained design strength. We believe that walls below the level where the columns are to be exposed in R or N should achieve design strength before the columns are exposed. The number of columns above El. 65' that can be exposed simultaneously on both R and N walls must still be determined. Also, panels adjacent to columns below El.65 must have been evaluated for both single and double curvature flexure. Capacity criteria are considered in Section 5 of this SER.

The licensee has considered temporary additional loads on the structure (such as form work, resteel, newly poured concrete, etc.) during construction as discussed in response to NRC Questions 3 of 9/6/79 and determined that the load will not be sufficient to significantly degrade the structure. The staff believes that the Complex will remain seismically qualified with the temporary additional loads on the structure because these loads should be small relative to the weight of the structure.

The licensee has also considered the removal of parts of the existing structure. Portions of wall are being removed along column R, north of 46 and along 46 near Q. Portions of a steel beam along N near El. 77' must be exposed. The staff believes that the Complex will remain seismically qualified due to these removals, because the wall portions being removed are from non-shear walls and because no credit was taken for the concrete along the beam in the evaluation of the existing Complex.

4.12 QA/QC Requirements

PGE will be responsible for the administration and control of the total quality assurance program and has delegated to Bechtel the responsibility for quality assurance of engineering, procurement, and construction activities. The modification work is being performed in accordance with previously NRC approved Quality Assurance/Quality Control procedures.

The plates to be installed on the west wall of the Control Building will be fabricated per ASTM Standard A-36-77 and welded together per AWS Standard D1.1. Completed welds will be non-destructively tested by the magnetic particle method.

4.13 Equipment Qualification During Modification Work

The proposed modifications to the Control/Auxiliary/Fuel Building Complex will result in a slight frequency increase in floor response. PGE has committed that equipment, components, and piping required for safe shutdown, ECCS, or to mitigate the consequences of accidents which could result in 10 CFR 100 releases will remain seismically qualified for earthquakes up to and including the SSE throughout all structural modification work. Any changes needed to insure this will be performed before the structural modification necessitating the change is made. The floor response spectra for the modified Complex have been submited by PGE and are discussed in Section 5 of this Safety Evaluation Report. The seismic acceptance criteria of the modified Complex are per PGE-1020 and associated submittals.

Regarding the seismic qualifications of safety-related mechanical equipment, the effect of the response spectra for the modified complex will be evaluated to determine whether there will be an increase in loading. If an increase occurs, the new load will be evaluated against allowables and if an overstress occurs, the element will be strengthened.

Safety-related piping will be analyzed using the response spectra for the modified complex to assure adequate restraint. Additional restraints will be installed as required.

Also, electrical equipment, cable trays and control equipment will be reviewed and modified as needed to insure seismic qualification under the final response spectra for the modified complex.

In view of the fact that the systems described above are seismically qualified for interim operation of the as-built Control Building Complex SSE response spectra and will remain so during the modification work and that they will be seismically qualified for the SSE response spectra for the modified Complex prior to commencement of any work affecting the response spectra, it follows that the safety systems described above will be seismically qualified while modifications to the Complex are being performed. This conclusion is contingent upon the completion of the review of safety-related equipment and instrumentation as discussed in PGE's 7/6/79 response to NRC Question #29.

4.14 Compliance with Technical Specifications During Modification Work

Prior to undertaking any particular phase of the modification work, Bechtel will prepare detailed work plans for the work to be accomplished. These work plans will be reviewed by the PGE on-site QA staff to ensure that the planned activity is consistent with the Technical Specifications and the plant's administrative procedures. If any inconsistencies are found, necessary changes will be made before embarking on the activity to assure that Technical Specification requirements are met. This review should be adequate to provide assurance that no Technical Specification violations related to the modification work will occur while the modifications are in progress. The Technical Specifications are only peripherally related to the modification work, inasmuch as the Technical Specifications primarily involve equipment operability, operating limits, etc., whereas the modification work primarily involves the construction of new shear walls and therefore has little direct bearing on equipment and operating limitations. Therefore the on-site QA staff review for Technical Specification compliance during the modification work should be straight-forward.

The NRC staff has conducted a review of the Trojan Technical Specifications as they relate to the modification work. There are several activities to be performed which relate to Technical Specification requirements--fire protection, fire barrier integrity, and control room ventilation.

With respect to fire protection, it is possible that some fire hose stations may have to be temporarily disabled during the modification work. In this event, Technical Specification 3.7.8.3 requires that another equivalent-capacity fire hose be routed to the unprotected area.

The modification work will also involve the drilling of holes through walls and removal of portions of walls which serve as fire barriers. When this is done, Technical Specification 3.7.9 requires compensatory action. If the fire barrier protects areas with a combustible loading of more than one lb./sq. ft. of equivalent wood, a continuous fire watch must be posted; or a fire detector installed, a temporary fire barrier (1 hr. rating) erected, and a fire watch patrol established.

If the fire barrier protects areas with a combustible loading of less than or equal to one lb./sq. ft. of equivalent wood, a fire watch patrol to inspect the area at least once per hour must be established and either a fire detector or a temporary fire barrier (1 hr. rating) must be installed.

The modification work will also involve the drilling of holes in the Control Room east and west walls. This could affect the ability of the control room emergency ventilation system to maintain a positive pressure in the Control Room under emergency conditions (chlorine gas release or radiation) and thereby affect Control Room habitability. Technical Specification 4.7.6.1.d.3 requires that the ability of the Control Room emergency ventilation system to maintain at least 1/8" water column (W.C.) be demonstrated at least once every 18 months. The Technical Specifications also do not allow the licensee to alter a system such that a test would be unsuccessful should it fall due.

The licensee is aware of these requirements, and will control the hole drilling activity such that only one 3" hole will be open at one time. Further, the hole will be plugged with a preformed fireproof plug immediately after the hole is drilled. Also, during the installation of plate B, each hole will be unplugged one at a time and a seal installed immediately following the installation of each bolt to preserve the leakage integrity of the wall. The preformed plugs will be tapered and inserted from the interior so that any positive pressure in the Control Room would tend to seat the plug and thereby form a tighter seal.

Based on the above, we conclude that the ability to maintain a positive pressure in the Control Room will be preserved during the modification work.

Based on discussions with the licensee and a review of responses to staff questions, it is clear that the licensee is aware of the Technical Specification requirements regarding fire barriers and Control Room emergency ventilation, and is providing measures to assure that these requirements are met. The Technical Specification regarding fire hose stations has not been previously discussed since whether or not a fire hose station need be disabled will depend on detailed work plans yet to be developed. Also, there may be other Technical Specifications that could be impacted by the modifications, depending again, on the exact content of the work plans. The PGE QA staff review should be an adequate administrative means to assure compliance with all Technical Specification requirements during the modification work. The licensee has not identified any Technical Specifications which would, of necessity, be violated by the modification program, and no relief from any Technical Specification has been required or granted.

We conclude that there is reasonable assurance that Technical Specifications will be adhered to during the modification work.

5.0 Evaluation of the Modified Complex and the Adequacy Thereof

As discussed in Section 3.2.1.1 of this SER, entitled, "Input Criteria," the evaluation of the modified Complex is being performed with seismic input criteria which are basically in accordance with that specified in the Trojan FSAR Section 3.7.1, as referenced by Technical Specification 5.7.1. The acceptability of the deviations from the initial seismic input criteria has been discussed in Section 3.2.1.1 of this SER. Therefore, the use of this seismic input criteria provides an adequate basis for the determination of the adequacy of the modified Complex. As discussed in Section 3.2.2.1.3, it is the seismic load combinations which govern for the determinations as to the adequacy of the proposed modifications to the Complex.

Section 3.2.1.3 of this SER, entitled, "Seismic System Analysis," discusses the conformance of the seismic analysis of the modified Complex to the criteria delineated in Section 3.7.2 of the Trojan FSAR, as referenced by Technical Specification 5.7.1. Those Sections of this SER which referred to this Section for further discussion regarding the acceptability of the analysis of the modified Complex were:

(1) 3.2.1.2.1 regarding the acceptability of the analytical models and the allowable stresses for the modified Complex.

(2) 3.2.1.2.6 regarding the acceptability of the variations in structural properties considered in the analysis of the modified Complex and the adequacy thereof.

(3) 3.2.1.2.7 regarding the acceptability of the vertical amplification factors assumed for determination of structural forces.

(4) 3.7.1.2.12 regarding the acceptability of the resistance of the modified Complex to earthquake induced overturning moments.

(5) 3.7.1.2.18 regarding adequacy of the considerations of the effects of the assumed occurrence of five (5) OBE's of 0.15 g with ten (10) effective full stress cycles on the strength of the modified Complex and the induced frequency shifts in the derived floor response spectra.

Section 3.2.2.1 of this SER addresses the design bases for the modified Complex. As stated in that Section, this SER Section will discuss the details and acceptability of the design bases for the modified Complex excluding subsection 3.2.2.1.7 - Structural Preoperational Testing, which has been covered sufficiently in Section 3.

5.1 Analytical Model

The mathematical model of the modified Complex is a linear elastic 3-D finite element model. The specific details of this model are discussed in PGE-1020 and the associated PGE submittals. While this model gives an accurate representation of stiffness and mass distributions for the modified Complex, the implicit assumptions of linear elastic behavior and total connectivity between the boundaries of the adjacent elements preclude the analyses using this model from accounting for the potential nonlinear behavior of the structure directly. The nonlinear structural response is due to the design deficiencies in the Complex necessitating the development of design criteria for the in-situ walls through testing. The testing which was done indicates certain degrees of nonlinear behavior. However, it was not extensive enough to ascertain the behavior of the modified Complex. It is necessary for the analytical model to accurately capture the behavior which would be expected for the modified Complex in order for it to predict meaningful structural responses, namely forces in structural elements, displacements and floor response spectra.

In an attempt to account for the stiffnesses which would be exhibited by the walls in the modified Complex, the stiffnesses of the individual elements were modified per the criteria presented in PGE-1020 and the associated PGE submittals. The adequacy of these is discussed later.

The acceptability of the techniques for performing the response spectrum and time history analyses of the modified Complex, given the adequacy of the structural model, has been discussed in Section 3 of this SER.

5.1.1 Adequacy of Assumed Stiffnesses

The assumed stiffnesses are derived from the shear wall test program which is described in Appendix A of PGE-1020. The elastically calculated stiffnesses are reduced by a stiffness reduction factor using the methods as described in PGE-1020 and the associated PGE submittals. As stated therein, the stiffness reduction factors for the in-situ walls are a function of the amount of steel reinforcement present in the wall, the shear stress and the normal stress acting on the wall, and the number of cycles of stress to which the walls are subjected (this is addressed in a later subsection of this SER). The stiffness reduction factors are derived from comparisons of the secant modulii of the test specimens to the initial elastic modulus considering the above variables. These factors are then applied to the elastic modulii of the elements representing in-situ walls. The elastic modulii of the in-situ walls, which are a function of the strengths of the masonry and concrete, are determined on the basis of the as-built material properties, rather than design values for these properties. This provides a more representative estimate of structural response.

5.1.1.1 Adequacy of the Assumed Wall Reinforcement Ratios for Stiffness Derivations

The reinforcement ratios which were assumed for the in-situ walls considered the encased steel frame to contribute to the reinforcement ratio for the continuous vertical steel, as discussed in the 12/17/79 PGE response to the 10/2/79 NRC question 8, the 7/6/79 PGE response to question 45 and the 7/10/79 PGE response to question 46. These responses and Appendix B of PGE-1020 describe how the columns in the test specimen were considered in deriving the stiffness degradation factors considering this effect.

The method for accounting for the encased frame for the in-situ walls was explicitly described in the latter of the referenced questions. The methods by which the stiffness degradation factors were derived from the test specimens, including the way in which the columns in the test specimen were factored into this derivation, is appropriate However, neither the results of the test specimen, nor the additional information presented by PGE validates the approach for the consideration of the encased frame for the in-situ walls. The method used to account for the encased frame has implicit in it:

(1) That the individual wall panels will behave in single curvature. If the individual wall panels behaved in double curvature,

which is one of the cases assumed for capacity determination, this contribution would not be developed.

(2) That the slip that will develop in the beam-column connection will be sufficient to develop twice the AISC allowable capacity. The first assumption has not been substantiated since the mode of behavior of the walls in the modified Complex has not been defined. However, it may be single curvature, double curvature, or somewhere in between. The testing program was not extensive enough to better define the behavior of the walls in this regard. If behavior is not quantifiable, then behavior for all reasonably postulated modes of behavior must be evaluated and found acceptable.

The second assumption seems to be invalidated by the 12/22/79 PGE response to NRC question 6 of 10/2/79 and the 2/13/80 PGE submittal. The 2/13/80 submittal indicates that slip at the connections is about .01" or less, assuming even single curvature. The 12/22/79 PGE response indicates that at this level of slip, only the frictional connection resistance is mobilized which is not obvious as being twice the AISC connection capacities for the Complex. Additionally, the 12/21/79 PGE response to NRC question 2 of 10/2/79 indicates 4 connections in the Complex with ultimate capacities which are less than this value of 2.

The above factors must be resolved in order to substantiate that the effect of the encased frame is appropriately considered.

5.1.1.2 <u>Adequacy of Assumed Shear Stresses for Stiffness Derivations</u>

Given known normal forces and reinforcement ratios for the system of walls modeled in STARDYNE for the modified Complex, the chosen procedure for iterating using the STARDYNE model as described in Section 2.2.1.3 of Appendix B to PGE-1020 to determine the final shear stresses and corresponding stiffnesses for the various walls is appropriate, using the relationships developed from the test data for stiffness reduction factors as a function of shear stress, normal stress and reinforcement ratio. However, as discussed in Section 5.1.1.1 and 5.1.1.3 of this SER, the reinforcement ratios for the panels and the normal forces acting on the panels in light of the nonlinear dependence of stiffness degradation factor on this parameter, must be appropriately considered.

5.1.1.3 Adequacy of Assumed Normal Forces for Stiffness Determinations

The stiffnesses are proportional to the amount of normal force acting on the wall. This normal force is affected by the amount of dead load acting on the wall, the vertical earthquake component, and the shifting normal forces during the course of an earthquake due to gross overturning.

The dead load assumed to be acting on the walls was the direct dead load as discussed in PGE-1020 and the associated PGE submittals. However, the encased steel frame will carry a portion of this dead load. The amount of dead load which is carried by the frame is influenced by the construction sequence of the walls, and the ensuing creep and shrinkage of the walls. In the 9/5/79 PGE response to NRC question 27, an analysis was submitted demonstrating that for the conditions assumed in that example, a reduction in dead load of 10% due to creep and shrinkage was predicted. Arguments presented in PGE-1020 and the 12/22/79 PGE response to the 10/2/79 NRC question 23 indicate that this would be compensated for by an increase in dead load on the walls during an earthquake. However, this does not seem viable considering the encased nature of the steel framing and is therefore inappropriate. In fact, if the stiffness of the walls were to degrade, the opposite effect would tend to occur. Furthermore, using this method for accounting for the reduction as in the September PGE response, calculations made by the Staff indicate that this percentage reduction can be substantially higher and vary throughout the Complex. This is due to the dependence of this percentage on column size, beam size, wall dimensions, wall material properties, and level of dead load stress. In addition, the assumed value for shrinkage strain was not considered adequately. A better approximation for this phenomenon, given all the uncertainties inherent in the calculation of a numerical value, is twice the value assumed by PGE in their determinations. This would further increase the percentage of the direct dead load being carried by the encased frame. Also, the stiffening of the beam by the encasement in concrete was not addressed. Further, the effect of the 50°F change in mean temperature of the wall on dead load reduction for exterior walls, discussed in the 12/21/79 response to the 10/2/79 NRC question 10, has not been addressed. The strain due to this effect would be about 50(.00055)(.01)=275 u in/in (even neglecting the biaxial nature of this affect in the wall), which is even more significant than the shrinkage strain of 140 u in/in. Even consideration of a 25°F mean wall temperature drop would result in strains greater than those considered for shrinkage. Increasing the stiffness of the beam would cause additional load to be transferred into the frame. The method presented in the PGE September response (i.e., using the methodology for a beam on an elastic foundation) seems to be reasonable in light of the in-situ conditions of frame encasement and continuity of reinforcement past the beam. However, the appropriate conditions for each of the in-situ wall panels must be considered.

In an attempt to account for shrinkage strain twice that initially assumed in their 2/13/80 response, PGE presented an analysis using a different approach than the one described above. Different beams were considered also. This analysis concluded that a dead load reduction of 30% was possible. However, the analytical model assumed that the wall was rigid and that the beam and wall were not constrained from separating. This is not realistic considering the in-situ walls and is not necessarily conservative as compared to the method considering a beam on an elastic foundation. The table presented below shows the differences between the two methods. A percentage greater than 100% would imply that some amount of tension may exist in a wall. Additionally, this response addressed the effect of a 30% dead load reduction on capacities but not on the assumed stiffnesses.

The effects of the vertical earthquake component have been considered to reduce the direct dead load on the walls by 13% based on an assumed conservative vertical amplification factor of 30%. This is based upon consideration of the OBE with a peak horizontal ground acceleration of 0.15g. Applying this amplification factor to the SSE would result in a dead load reduction of 1.3 (2/3 (.25))100=22%. If the actual amplification factor of 1.16 is considered, this percentage of dead load reduction becomes 19%. This would result in a further reduction in dead load under the SSE condition of (1-(1-.19)/(1-.13))=7% beyond that considered for the OBE. This further adds to uncertainty but should not be important if all other factors were adequately considered in a reasonably conservative manner.

The gross bending moments due to an earthquake would cause a shift of normal force present on the wall from side to side in the Complex, with stiffnesses of walls increasing with increasing normal forces

TABLE: Comparison of Dead Load Reductions

Method 1 - Beam on Elastic Foundation
Method 2 - Beam on Rigid Foundation ("Strength of Materials" II - Timoshenko Ch III #6)

CREEP AND SHRINKAGE:

Ct = .86
in all cases, restrained shrinkage strain over 40 yrs = $140 \times 10^{-6} \frac{in}{in}$

	% load transferred from wall to columns	
	Method 1	Method 2
CASE 1 W24 x 68 beam t wall = 26" span = 19.25' W14 x 142 col. sigma n = 100 psi	(18.6%) 14.%*	18%*
CASE 2 W24 x 68 beam t wall = 26" span = 19.25' W14 x 142 col. sigma n = 50 psi	(32.5%) 27.9%*	26%*
CASE 3 W24 x 68 beam t wall = 35" span = 31' W14 x 142 col. sigma n = 10 psi	(73%) 71%*	25%*
CASE 4 W36 x 280 beam t wall = 28" span = 30' W14 x 142 col. sigma n = 100 psi	(13.7%) 10.3%*	---
CASE 5 W36 x 280 beam t wall = 28" span = 30' W14 x 142 col. sigma n = 50 psi	(24%) 20.6%*	---
CASE 6 W36 x 280 beam t wall = 28" span = 30' W14 x 142 col. sigma n = 10 psi	(106.5%) 103.2%*	---

* indicates due to shrinkage only
( ) indicates due to creep and shrinkage

Licensee proposes using 30% total for creep and shrinkage under Method #2.

and vice versa. The stiffnesses are nonlinear functions of the normal stress. The 2/13/80 PGE submittal addressed this effect in a simplified manner. This effect was estimated to reduce the overall stiffness of the Complex by about 18%. Additionally, it must be substantiated that any tensions induced in the walls are not detrimental to the stiffness of the panel over a number of cycles since all specimens tested in the test program had only compression induced in them.

5.1.1.4 Single Vs. Double Curvature Behavior

The test program was not sufficient to establish the behavior of an ensemblage of shear walls. Only single shear piers were tested, with double curvature behavior being investigated for most specimens. Use of stiffness degradation factors if adequately derived from the test specimen in the STARDYNE model seems reasonable if the connectivity between the elements assumed in the analysis was maintained in reality. However, the precise mode of failure has not been established. In order to preclude a single curvature failure of the elements of the structure, shear friction mechanisms are relied upon for resistance. For these mechanisms to be invoked, relative slippage must take place between elements. These required relative displacements must be shown to be compatible with the overall displacement behavior of the Complex. It has not been demonstrated that this slippage will or will not occur. In other words, the slippage may or may not take place. Therefore, both cases must be considered as a possible mode of behavior. In the 2/13/80 PGE submittal, it was estimated that an additional 30% increase beyond the calculated STARDYNE displacements is necessary to invoke this mechanism under the calculated earthquake forces. This would indicate a reduction in stiffness of (1-1/1.3)=23%. In addition, if temperature (where present) and shrinkage effects were adequately described by the model for dead load reduction presented by PGE in this response, then such effects would have to be overcome before the connection would act and would require additional displacement beyond that needed to develop shear friction if this separation was greater than needed for shear friction.

5.2 Adequacy of Structural Responses - Forces and Displacements

The parameters which influence the resultant forces and displacements determined from the analytical model were discussed in Section 5.1 of this SER. The variations in parameters discussed therein must be adequately accounted for in order to assure that the new and existing elements have been evaluated and designed for the forces and displacements which could potentially occur, accounting for the uncertainties in the behavior of the modified Complex. The adequacy to resist these must be determined.

5.2.1 Capacities of New Structural Elements

The allowable stresses (loads) for the new structural elements are being determined per the criteria discussed in PGE-1020 and associated submittals. The use of these acceptance criteria in the referenced FSAR load combinations would be acceptable, if it can be substantiated that forces were determined considering the previously discussed variabilities. Additionally, the 2/13/80 PGE submittal indicates that their analyses to date show that stiffnesses will decrease (causing increased displacements) due to overturning moments and single curvature behavior, yet did not demonstrate that the new structural elements (walls and plates) were capable of withstanding these effects. No slippage has been accounted for in the model. It must be considered since it cannot be precluded.

Coefficient of Friction Between Steel and Concrete

The licensee proposes using a coefficient of friction (u) of 0.7 between steel and concrete when calculating frictional resistance. The frictional resistance is relied upon to transmit seismic forces into the steel plate and to resist sliding between the columns and spread footings. The value of 0.7 is obtained from the ACI 318-77 code and tests by Mattock.

The staff believes that use of u=.7 requires further justification. In the ACI 318-77 code, u=.7 is suggested for use in calculating shear friction of steel to concrete using the shear friction method which utilizes dowels attached to the steel and embedded in the concrete crossing the crack perpendicular to the crack plane. Per ACI 318-77 commentary, the applied shear is resisted by friction, shearing off of protrusions on the faces, and dowel action of bars crossing the crack plane. Stated in the ACI 318-77 commentary, "in the shear friction method of calculation, it is assumed that all the shear resistance is due to friction between the crack faces. It is therefore necessary to use artifically high values of the coefficient of friction u in the shear friction equation in order that the calculated shear strengths shall be in reasonably close agreement with test results." This indicates that the code believes 0.7 is high to account for friction alone. The referenced paper by Mattock has shear studs crossing the plane so that value obtained is also not for friction alone. Since the plates at Trojan are assumed not to slip, the other mechanisms employed by the code to obtain an equivalent u cannot be employed. Dowel action would only be valid after slip has occurred. In a paper by the Portland Cement Association*, tests were performed to obtain u for steel-grout and steel-concrete interfaces with no steel studs or bars crossing the crack plane. Compression forces were applied externally. It was found that for the concrete-steel face, peak u with dry faces and normal stress equal to 60 psi was .69. For the same arrangement but with wet faces, peak u was .68, approximately the same. However, if normal stress on the wet faces increased to 100 psi, u decreased to .64. A conclusion of the paper was that u decreases as normal force increases. At Trojan, it is estimated by PGE that stress immediately under the bolt is 1120 psi and at 6-1/2" away is 600 psi. Because the plate is assumed not to slip and the stresses at Trojan are high, 0.7 requires further justification. It appears that perhaps the ACI shear friction approach was used elsewhere where no slip has occurred to develop dowel action. This may be the case at the N line wall where concrete is placed against hardened concrete (concrete block in this case) and ACI recommends u=1. This problem would be alleviated by the installation of shear keys in the existing wall.

5.2.2 Capacities of Existing Structural Elements

For the existing elements in the modified Complex, excluding the composite and masonry shear walls encasing the steel frame, PGE-1020 Section 3.2.2 indicates that the codes, standards, guides and specifications remain as described in the FSAR.

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* Rabbat, B. G., Russell, H. G., "Tests to Evaluate the Coefficient of Static Friction Between Steel and Concrete," Construction Technologies Laboratories, Division of the Portland Cement Association, February 1979.

One of the requirements when the modifications were ordered was that the existing shear walls be brought back into conformance with the Uniform Building Code - 1967 version, as referenced in the Trojan Operating License. To accomplish this, PGE conducted a testing program to determine the strengths of the in-situ walls, considering the encasement of the steel frame. The shortcomings of this testing program were mentioned previously. An additional shortcoming was that the specimens with embedded columns were embedded so that their full yield strength could be developed instead of imposing limitations on their strength to represent the in-situ beam-columns connections. Sections 106 and 107 of the UBC-1967 provide for testing as an alternative to substantiate Code requirements. Testing is appropriate. However, it must be sufficient to substantiate all assumptions regarding strength and behavior. Any significant uncertainties remaining as a result of the testing should be either accounted for or dismissed by additional testing. As discussed in Section 5.1 of this SER, it has not been determined that the uncertainties in the analyses have been sufficiently addressed. The effects of these uncertainties on rebar strain must also be considered and the impacts on those already reported assessed.

5.2.2.1 Capacity Determinations

For the walls in the modified Complex, three distinct failure modes are possible, namely:

(a) flexure failure
(b) sliding failure
(c) diagonal tension (shear) failure

Each of these mechanisms must be investigated for its resistance capacity, with the mode with the lowest capacity being the controlling capacity. Additionally, as discussed later, the consideration of the individual wall panels for capacity determination is considered appropriate. Each panel must be able to develop its required resistance in order for capacities to be consistent with the mass and stiffness distributions assumed in the analytical model. Each element should be capable of carrying the forces calculated and relied upon to be withstood, namely shear, tension and moment. It is not obvious that this is met for each panel in the Complex and that must be verified by PGE.

Flexural Failure

The vertical cracking at the column lines which is observed in the as-built structure indicates that examining a single segment of the wall bounded by two adjacent columns and two adjacent floor levels is a reasonable model for estimating the flexural capacity of the wall system. The procedure for considering only continuous vertical reinforcing steel in the calculation of flexural capacity when double curvature behavior is assumed, and of considering the contribution from continuous vertical steel, any horizontal steel continuous past the columns, and the encased beam-column connection in the calculation of flexural capacity when single curvature behavior is assumed is reasonable. The lower of the capacities from either of these assumptions should be taken as controlling. However, the dead weight contribution must consider all the variabilities in normal stress as discussed in Section 5.1 of this SER. The adequacy of the PGE consideration of the dead load has not been established. Therefore, the consideration of the effects of overturning moments, as discussed in the PGE 2/13/80 submittal, cannot be determined to be adequate.

The single curvature mode of behavior requires substantiation that the displacements required to develop the necessary resistance mechanisms along element interfaces are compatible with the overall behavior of the structure. This was addressed in the 2/13/80 PGE submittal. An estimate was made of the increased displacements required to develop the resistance mechanisms considering single curvature behavior of the Complex. The comparison of vertical shears along the column lines for the present model of the Complex for the factored OBE to the capacities indicates that the capacities at elevation 93' on wall R and down to elevation 61' along column line 46 on wall N are exceeded. The acceptability of these has not been substantiated. Additionally, these comparisons did not consider that the vertical shear resistance would be reduced somewhat if a capacity reduction factor of .9 was considered to reduce the column connection contribution. The basis for this reduction is that the bending capacity of the overall Complex is being evaluated and the Complex can be considered essentially in the same manner as a reinforced concrete beam for which a capacity reduction factor of 0.9 is appropriate per the ACI and UBC Codes.

Horizontal shear capacities were investigated per the method presented in the PGE 7/6/79 response to question 43. Provided that the in-situ conditions where openings are encountered were properly considered, this response indicates that there is adequate capacity to resist the factored OBE.

Dead load was found to contribute only 6% or less to the overall shear capacity for this mode of behavior. However, it is necessary to ensure that this effect does not adversely impact individual wall panel capacities.

Sliding Failure

As indicated by the test data, sufficient resistance to sliding must be developed such that the wall is able to develop required shear resistance. The resistance to sliding is developed by the vertical reinforcement in the wall, the normal stress acting on the wall and any columns which are fully embedded in the wall. It is reasonable to assumed that for vertical reinforcement embedded in the core and the grout a shear friction coefficient of 1.4 will develop if it is constructed such that any joints can be considered to be cast monolithically. This would not be appropriate for any walls where drypack was used at the top. The effect of normal stress (force) can be considered to have a shear friction only on the area of the wall composed of grout and concrete which can be considered to be cast monolithically. The mortared area cannot be relied upon for this same resistance contribution due to its lack of aggregate. (This is also the basis for the inappropriateness of the 1.4 shear friction coefficient for the rebar where drypack has been used.) This conclusion is substantiated by examining the figure attached to the 6/29/79 PGE response to question 41. At higher values of the quantity $1.4(\rho fy+N)$ for the composite specimen, the shear resistance to sliding was less than this term where a large contribution to this term was coming from the applied normal force. Also, specimen D1 (no core concrete) failed below this value for a lower value of this term which is felt to be due to the higher ratio of mortar area to grout area (which is assumed to be equivalent to core concrete) than for the composite specimen. Work by Hatzinikolas also indicates that the shear friction coefficient for mortar is less than 1.4, the lower bound of his test data being .75. Therefore, the relationship $1.4\ (\rho fy+N)=V$ as proposed by PGE to consider this effect is not considered appropriate. An appropriate relationship must be developed and its impact must be shown to not adversely impact the walls. This must also consider the appropriate applied normal force considering the possible variations discussed previously. The consideration of columns which are fully embedded in a wall panel by the method presented by PGE is appropriate; however, it should be looked at on a panel-by-panel basis.

Diagonal Tension (Shear) Failure

Given that the flexural and sliding modes of failure are prevented, the limitation of the shear stress on a double block panel or pier to 150 psi, and on a composite panel or pier to 300 psi, is reasonable since the reinforcement ratios in the in-situ walls are greater than or equal to those in the A, B and D specimens.

5.2.3 Adequacy of Considered Displacements

The elastic displacements calculated from the STARDYNE model may be increased due to:

(a) degradation of the stiffness of the walls,
(b) development of shear friction along the column lines if this mechanism were invoked,
(c) the effects of gross overturning moments,
(d) the development of the frictional resistance to sliding of the wall panels.

Although (a) and (d) above were considered in the analytical model by deriving the stiffnesses for the various wall elements in the analytical model, they are subject to the items discussed in Section 5.1 of this SER.

In the 2/13/80 PGE submittal, the effects of item (b) on resulting displacements of the current analytical model were addressed; however item (c) was not.

5.3 Adequacy of the Derivation of Floor Response Spectra

Concerns over the adequacy of the parameters influencing the stiffness incorporated into the analytical model have been addressed in Section 5.1 of this SER. Therefore, the calculated center frequencies of the floor response spectra for the modified Complex cannot be determined. The variation in these frequencies of $\pm 10\%$ as discussed in Appendix B to PGE-1020 seems reasonable, given the items addressed. (The adequacy of the additional 20% reduction in frequency to account for the assumed occurrence of 5 OBE's of 0.15 g is discussed in a later subsection of this SER.) However, as discussed elsewhere in Section 5, additional uncertainties exist. In the 2/13/80 PGE submittal, estimates are made of additional potential frequency shifts due to gross bending and the development of shear friction to resist gross bending of the Complex. It was estimated in this response that these could cause an additional 16.6% frequency reduction but proposed that these would be covered by the 20% assumed for cyclic effects. This is not acceptable to account for these effects since stiffness degrades as a function of the logarithm of the number of cycles. Figure 21-1 of the 12/21/79 PGE response to NRC question 21 of 10/2/79 indicates that the majority of the cyclic degradation takes place within the first 10 full cycles of stress.

5.6 Creep and Shrinkage Values

The licensee originally used an unrestrained shrinkage strain of 100×10^{-6} in/in and a restrained shrinkage strain of 70×10^{-6} in/in for the existing composite walls when calculating the distribution of wall dead load to the embedded steel frame. For calculation of bolt tension losses in the existing walls, a restrained shrinkage value of 200×10^{-6} in/in is used. For the new walls, an unrestrained shrinkage value of 174×10^{-6} in/in was calculated for use in determination of bolt losses but 355×10^{-6} in/in is used. 280×10^{-6} in/in is used for wall shrinkage when calculating dead load distribution. The creep coefficient (Ct) used in the new and existing walls is .86 for dead load distribution effects and 1.6 for bolt losses. The shrinkage values differ because they apply to different circumstances and the licensee has used different margins depending on the circumstances. After discussing with the staff the resistance supplied by the block in obtaining restrained shrinkage, the licensee now proposes using 140×10^{-6} in/in instead of the original 70×10^{-6} in/in.

The staff concludes that with the inclusion of this new restrained shrinkage value, the above numbers seem reasonable even considering possible variations with the exception listed below. An important parameter used in calculating unrestrained shrinkage values is the ultimate shrinkage value which is a function of various parameters such as aggregate type, volumetric content of the aggregate and water-cement ratio. If nothing is known about the concrete, the ACI suggested value is 800×10^{-6} in/in. In calculating the ultimate unrestrained shrinkage, the licensee has used 990×10^{-6} in/in for the existing walls, which is a conservative value, and for the new concrete 500×10^{-6} in/in, because the licensee proposes using a low-shrink aggregate. This value, although not conservative, is compensated for by the value the licensee assumed for the minimum thickness coefficient in the calculations. In the case of creep, the creep coefficient is dependent upon the ultimate creep value which the licensee has assumed to be 1.5 for both new and existing walls. The ACI recommends using Cu=2.35 if nothing is known about the concrete. Even if the low shrink aggregate does not reduce Cu to 1.5, this non-conservatism would be compensated for because the licensee has assumed a value twice as large as calculated when considering bolt losses. In the calculation of dead load transfer from the wall to steel frame, changing Cu from 1.5 to the suggested value of 2.35 has little significance when using the beam on elastic foundation approach.

The shrinkage calculation example given for the existing wall is for a wall 30" thick. Shrinkage will increase as wall thickness decreases (unrestrained shrinkage for 30" wall is about 128×10^{-6} in/in but for 24" wall is about 200×10^{-6} in/in). This matter needs to be addressed.

It should be noted that the phenomena of creep and shrinkage is highly complex and there exists wide variations in measured values. It becomes more complex when one considers the non-homogeniety of the actual walls. It becomes an important parameter due to the encased steel frame and the substantial effects of dead load.

5.7 Adequacy of the Bumping Post

We have reviewed the PGE analysis of the bumping post and do not feel that inelastic deformation of the bumping post can be relied upon for energy dissipation. This is based upon PGE considering that buckling of the compression members occurs based upon the actual curve established by the Column Research Council (CRC). (This is what is used when the factor of safety is taken out of the AISC Code equation). The effective length assumed for these members is not certain. The predicted stresses are only within 11 1/2% of the curve derived value. Review of the test data for these type members used to establish the CRC curve in the intermediate effective-length range indicates that the test specimen always failed below the curve in the range of the calculated stresses. Therefore, there is not reasonable assurance that the member will not buckle. Furthermore, recent testing by Popov indicates that strength drops off rapidly after buckling with increased deformation in the intermediate effective length range such that the elasto-plastic behavior assumed in PGE's analysis is not valid. This conclusion was reinforced in conversations with the manufacturer of the bumping post. It is concluded that once the AISC allowables are exceeded, the limits of behavior cannot be relied upon with reasonable assurance. Given the low velocities at which a flatcar was indicated to be stopped at the Code allowable levels, the possibility for heavier loads on the flatcar, and that the locomotive weight (on the order of 300 to 400 kips) was neglected, the administrative controls imposed on the train movement must be relied upon to prevent impact of a train with the wall as discussed in PGE-1020 and the associated PGE submittals.

The bumping post would be located about 33 feet from the normal spotting position of the railroad car in the railroad bay of the Turbine Building. Considering the slow speed of the train, this distance should be adequate to allow the train to stop before impacting the post, and provide an adequate allowance for any movement inaccuracies.

The flatcar would be in full view of the train engineer, so the position of the car will be observable, and any approach to either the bumping post or the end of the Turbine Building railroad bay obvious. In addition, the rail bed from the derailer at the security fence to the Turbine Building railroad bay is level, which would simplify train maneuvering.

The accidental approach of a train to the railroad bay from the main track is prevented by 2 derailers located both outside and inside the security fence.

Based on the above we have concluded that administrative controls will be adequate to prevent a train impacting the bumping post or the Control Building west wall.

5.8 Building Displacement - Effects on Equipment

For interim operation, the effects of Control Building story-to-story displacement and the relative displacement between the Turbine and Control Buildings and between the Control Building and the Containment were evaluated. In that evaluation, the effects of such deflections due to earthquakes on safety-related cables and piping within the Control Building and between the Control Building and the other buildings were assessed and found to be acceptable. Interstory displacements of 0.5" were evaluated. The differential displacement between the Control and Turbine Building was taken to be 2.5" at all elevations above grade level. The relative displacement between the Control Building and Containment Building was 0.76". Cable runs were found to have adequate slack to accommodate these displacements. The review of piping systems disclosed that one piping run between the Control and Turbine Building (service water to the switchgear room coolers) might be affected, but that the resultant water flow from a broken or leaking pipe would not cause flooding problems, and that the switchgear room could be adequately cooled, if needed, by alternate means. Therefore, the effects of both interstory and relative building displacements were found acceptable. It was also found that substantially higher displacements could be safely accommodated. Inasmuch as these same displacements would be less after the modifications to the Trojan plant than for interim operation (discussed in Section 5), the conclusions reached for interim operation would be equally valid following the modifications.

5.9 Gaseous Waste System

We have reviewed the licensee's submittal with regard to reevaluation of equipment, components and piping in the radioactive gaseous waste system previously seismically qualified in accordance with the FSAR. The license's position is that the waste gas compressor, waste gas surge tank, gas collection header exhaust filter and decay tank piping downstream of the first isolation valve and piping associated with the above equipment need not be seismically reevaluated with response spectra for the modified complex.

Regulatory Guide 1.143 and Branch Technical Position ETSB 11-1 require seismic design of those portions of the gaseous radioactive waste system that are intended to store or delay the releases of gaseous radioactive waste. The waste gas compressor, waste gas surge tank, gas collection header exhaust filter, and decay tank piping downstream of the first isolation valve and piping associated with the above equipment are not designed to store or delay the gaseous radioactive waste, and therefore, do not require seismic requalification.

We find the licensee's submittal concerning the subject system acceptable, and, therefore, these items do not have to be seismically requalified.

5.10 Railroad Spur After Modifications

The existing railroad spur enters the Turbine Building at the west end of the building between column lines 41 and 46 and proceeds east through the Turbine Building, through the Control Building (entering at column line R and exiting at column line N), parallel to the Auxiliary Building, and into the Fuel Building (entering at column line D) where it terminates. The proposed modifications involve closing of the existing railroad bay openings in the Control Building by the addition of walls at the existing railroad bay openings at column lines R and N between column lines 41 and 46. Consequently, the existing railroad spur will terminate in the Turbine Building, with a rail stop installed in that building just west of the west wall of the Control Building. The existing track in the Control Building railroad bay will be removed and the existing railroad bay will be converted to office space. An additional railroad spur will be added running outside the Control and Turbine Buildings and this spur will join the existing spur at a point next to the Auxiliary Building where the existing spur runs between the Control Building and the Fuel Building.

The existing railroad spur was taken through the Control Building as a matter of convenience and efficiency. Since it was necessary that railroad cars enter the Turbine Building (to facilitate the movement of large pieces of equipment onto the Turbine Building operating floor through the Turbine Building railroad bay) and also the Fuel Building (for movement of fuel casks), the most convenient and efficient path for the railroad spur was through the Turbine and Control Buildings and into the Fuel Building. The 45' elevation of the Control Building merely provides a path for the track between the Turbine and Fuel Buildings but the capability of moving railroad cars through the Control Building is not needed for any purpose, and in point of fact, there are no existing provisions (e.g., cranes) or need for loading or unloading railroad cars in the Control Building railroad bay. Consequently, the removal of the track through the Control Building will have no safety-related impact with regard to removing the capability of having railroad cars pass through the Control Building since there is no need for railroad cars to pass through the Control Building and since the capability to move railroad cars into both the Turbine Building and the Fuel Building where they may be needed will be retained with the relocated and modified railroad spurs.

5.11 Sliding Equipment Hatch in Control Building

Pursuant to the proposed modifications, an existing equipment hatch at elevation 65' on the east wall (column line N) of the Control Building approximately midway between column lines 41 and 46 will be reduced in size from 8 feet high by 7 feet wide to 4 feet high by 4 feet wide. This equipment hatch was provided in the original design of the Control Building to allow movement of equipment into and from the Mechanical and Electrical Auxiliary Rooms at elevation 61' and the Battery Rooms at elevation 65'. The large accessway provided by the existing hatch allows such equipment as motor-generator sets, transformers, switchgear cabinets, cooling units and battery chargers to be brought into and removed from this elevation of the Control Building easily without the need for disassembly of the equipment.

Although the equipment hatch is being reduced in size, it is not being eliminated and will still be useable for transferring some equipment into and out of the Control Building at this elevation. Nevertheless, after the modifications, it may be necessary to disassemble some equipment to a greater degree than was previously necessary in order to fit it through the smaller equipment hatch or to move equipment to or from this area by use of the Control Building elevator or Auxiliary Building access ways. While this would be more inconvenient than moving equipment through the existing hatch without disassembly, we can identify no safety significance from this. The existing hatch is not useable for fast, emergency transfers of equipment into and out of the Control Building since the existing hatch is 20 feet above grade and use of the hatch for transferring equipment requires special preparation and handling procedures. Nor is the existing hatch used for personnel access to the Control Building since the hatch is 20 feet above grade and is closed with a steel door and a heavy missile shield. Consequently, the proposed reduction in the size of the equipment hatch will have no safety significance with regard to personnel or equipment access to the Control Building.

5.12 Impact of Wall Failures on Safety-Related Equipment

The Staff has not yet been able to conclude that the wall capacities calculated and reported by PGE are appropriate. Until we are able to conclude that wall capacities have been adequately determined, we believe that capacity to force ratios reported by the licensee are subject to change and possibly to a reduction. Nevertheless, based on reported capacity to force ratios, there are two walls within the modified Control Building, identified as walls 6 and 8, which have capacity to force ratios in the event of an SSE of less than one. As to these walls, the question arises at this time as to whether wall degradation might occur during an earthquake resulting in the impact of debris from the wall on safety-related equipment.

During the staff site visit of June 12 - 13, 1979 these walls were examined to determine the likelihood of safety-related equipment being damaged by debris falling from these walls. Based on this examination, safety-related equipment was found to be (1) located sufficiently far away from the walls that it would not be impacted by falling debris or (2) located sufficiently high on the wall itself that any debris would either not impact the equipment or would have such a short distance to fall before impact that it would not pose a potential for damage to the equipment.

5.13 Effects of Completed Modifications on Fire Protection

Each of the areas in which modification work is to be performed was evaluated to determine if the modifications, once completed, would have any detrimental effect on fire protection for these areas. The completed modifications do not result in an increase in combustible materials in any of these areas since the only additional material which will remain after the modifications is either concrete or steel which are not combustibles.

The modifications will not result in decreased accessibility for the fire brigade to fight fires in any safety-related area nor will they increase the difficulty in reaching or fighting fires in such areas. The modifications could potentially affect access only in the following areas:

1. At the east wall of the Control Building, along column line N, above elevation 65', where the equipment hatch is reduced in size from 8' high by 7' wide to 4' high by 4' feet wide.

2. At elevation 45', column line R between column lines 41 and 47 where a concrete wall will be erected.

Reduction in the size of the equipment hatch will have no effect on access for fire fighting or emergency operator actions since such access never has been provided by the hatch. This opening goes to the outside of the Control Building and is at elevation 65', 20' above grade. Moreover, the hatch is covered by a closed steel door with a metal missile shield bolted to the inside. The hatch was not intended as a fire brigade access way and has not been used or relied upon for that purpose.

Because of the addition of a wall at column line R as described in 2 above, access by this means between the Turbine Building and the Control Building through the railroad bay area will be blocked off. Access to the Turbine Building from the Control Building through the railroad bay is not required since other access ways which would normally be used in any event are available. A new access door into the Control Building from the Control Building section of the railroad bay will be provided. Access from outside the buildings into each of these areas will still be available after the modification. Access through these areas to safety-related equipment or for fire brigade members is not required. Thus, access to safety-related areas has not been affected by the new wall at column line R.

Devices for detecting or extinguishing fires will not be blocked or in any way impaired by the modifications. The fire barrier between the Turbine Building and the Control Building formed by the Control and Turbine Building walls in that area will remain intact as a fire barrier upon completion of the modifications.

Based on the foregoing, we conclude that the level of fire protection for the facility will not be diminished as a result of the completed modifications.

6.0 License and Technical Specification Changes

6.1 Technical Specifications

6.1.1 As discussed in Section 4.10, no Technical Specification changes are required during the modification work itself.

6.1.2 Because of the reliance placed upon the bolts to provide shear transfer between the new and existing structural elements in the west wall of the Control Building, the staff has concluded that an inservice inspection program should be implemented to provide assurance that bolt tension will be maintained throughout the life of the plant. Accordingly, the staff requested that an appropriate program be proposed by PGE. We have reviewed the licensee's response and find the proposed program acceptable with modifications as described below (PGE letter of 12-17-79, Q.7 NRC Questions 9-14-79). We therefore, plan to incorporate this program (in Technical Specification format) into the Technical Specifications with any amendment authorizing PGE to proceed with the modifications. The modifications to the proposed program are:

1. At each inspection, 5 bolts should be removed and inspected. The condition of the tape should be noted, as well as the condition of the bolt with tape removed to ensure that the tape and bolt will continue to perform their functions with the design safety margins present in the bolt.

2. Beginning with the third year inspection, a time history of bolt preload vs. time should be developed to assure that the existing bolt preload is not predicted to drop below the initial bolt preload ($\bar{x}-2\sigma \geq .75Xo$) during the balance of the next inspection interval ($\overline{2}$ years or 5 years thereafter).

3. If the sample average preload drops below .75Xo, the circumstances should be reported pursuant to Technical Specification 6.9.1.8.i.

6.1.3 In the Design Features section of the Technical Specifications, Technical Specification 5.7.1 should be modified following completion of the modifications to both accurately describe the revised seismic design of the Control/Auxiliary/Fuel Building complex and require that it be maintained. This is consistent with the requirements for other seismic category I structures and the requirements for the Control Building design prior to the identified design deficiencies. At the present time, this Technical Specification, as it relates to the Control Building shear walls, is waived by License Condition 2.C.(10)(a) pursuant to the Licensing Board's Partial Initial Decision of December 21, 1978, and Amendment No. 35 issued on December 22, 1978.

The staff proposes that Technical Specification 5.7.1 be revised to read as follows, to be effective upon completion of the modifications:

"5.7.1 Those structures, systems, and components identified as Category I items in Section 3.2.1 of the FSAR shall be designed and maintained to the original design provisions contained in Section 3.7 of the FSAR (except for the Control/Auxiliary/Fuel Building Complex) with allowance for normal degradation pursuant to the applicable Surveillance Requirements. The Control/Auxiliary/Fuel Building Complex shall be designed and maintained to the design provisions contained in PGE-1020, as revised through Rev. 4*, with allowance for normal degradation pursuant to the applicable Surveillance Requirements".

6.2 License Conditions

6.2.1 In the event that the proposed modifications are authorized, the staff recommends the imposition of a license condition requiring that all authorized modification work be performed in accordance with the descriptions, procedures, and commitments set forth in PGE-1020 as revised through the latest revision and as supplemented by licensee's letters submitted in response to staff questions and certain interrogatories identified below. Any deviations or changes from these documents should be made only in accordance with 10 CFR 50.59.

In addition, the staff recommends that approval in the form described above be subject to certain additional conditions. These conditions are necessary because the determination of adequacy of certain activities related to the modification work is dependent on them. In some cases, these recommended conditions are at variance with, or

* To become effective upon completion of the modifications to the Control Building.

supplementary to, actions commited to by the licensee, and therefore these further license conditions would make clear that these requirements supersede any conflicting licensee statements.

6.2.2 In the event that the proposed modifications are authorized, the staff recommends that the following be added to Facility Operating License No. NPF-1:

"Control Building Modifications
The licensee is authorized to and shall proceed with modifications to the Control Building in order to substantially restore the originally intended design margins. The modification program shall be accomplished in accordance with PGE-1020, "Report on Design Modifications for the Trojan Control Building", as revised through Revision No. 4, and as supplemented by licensee letters dated February 28, March 28, June 22, June 29, July 5, 6 and 10, August 13, September 5 and 26, November 21, December 17, 21 and 22, 1979; January 28, and February 13, 1980, and as further supplemented by "Licensee's Responses to Interrogatories Dated August 27, 1979 From the State of Oregon" dated September 17, 1979. Any deviations or changes from the foregoing documents shall be accomplished in accordance with the provisions of 10 CFR 50.59.

The modification program shall be further subject to the following:

(a) The modification program shall be completed by not later than twelve months from the date of this amendment. When complete, license condition 2.C.(10), relating to interim operation pending completion of modifications, is cancelled.

(b) For the installation of steel plate No. 8, the plant shall be in the cold shutdown condition (Modes 5 or 6) from the time that the plate is lifted from the Turbine Building railroad bay at elevation 45' until the plate has been set in place with all bolts made snug. Prior to this evolution, diesel generator A shall be started and proper operability verified.

(c) Solid steel cable tray covers shall be installed over cable trays in work areas where cable damage is possible from accidental dropping of steel plate washers during their installation.

(d) A fire watch patrol shall be established whose sole responsibility shall be to make at least hourly inspections of all safety-related areas where combustible materials (e.g., wood framing, planking, plastic, etc.) related to the modification work must remain in the work area (not required for areas in which a continuous fire watch is present).

(e) Scaffolding and timber planking shall be installed against the R-line wall in the Cable Spreading Room during the installation of the steel plate washers at each location where washers are to be installed. The planking shall be placed and constructed to limit the maximum height of a dropped washer to three feet or less.

(f) Any construction work in the diesel generator combustion/ventilation air pathway which could potentially generate dust, dirt, or debris shall be temporarily halted when any diesel-generator is in operation.

(g) In the event that either the Shift Supervisor or NRC Resident Inspector determines that construction noise is resulting in noise levels in the Control Room of such magnitude as to interfere with normal communications, the construction activity shall be halted until alternate means are devised (e.g., lighter weight tools, other means of concrete/block removal, etc.) to proceed with the work with acceptably reduced Control Room noise levels.

(h) In the event that the NRC Resident Inspector determines that the construction activity in the Electrical Auxiliaries Room or Control Room is generating excessive dust, dirt, or debris or the use of water is being improperly controlled, construction work shall be halted until appropriate corrective measures have been taken.

(i) The plant shall be placed in the cold shutdown condition (Modes 5 or 6) during periods when, due to open walls (e.g., concrete block removal at 41R, elevation 65', or equipment access hatch modification at column line N, elevation 65') in the Control Building at or above elevation 65', safety-related equipment is vulnerable to either external missiles or missiles from construction equipment (e.g., jackhammers).

(j) During hole drilling in the east and west walls of the Control Building, personnel shall be stationed on the opposite side of the wall from the driller to monitor the drill penetration. Continuous voice communications shall be maintained between the drill operator and the monitor.

(k) Fire blankets (Claremont Weld Shield 800-24 or FabriCote 1584-white) shall be used over all cables in areas where Cadwelding, welding or cutting will be performed.

(l) The Battery Room exhaust duct shall not be disabled unless an alternate, equivalent means of Battery Room ventilation is first provided.

(m) Prior to the installation of plates 1 through 6, a temporary energy absorber shall be installed to preclude exceeding the allowable compressive strength of the underlying concrete in the event of an accidental plate drop.

(n) An energy absorber shall be placed on plate 4 prior to the installation of plate 7.

(o) A 1-inch-thick precrushed, stabilized HEXCEL pad and timber cribbing shall be used for energy absorption during the installation of plate 8.

(p) The work area at 41R (elevation 65') shall be protected by a dust-tight flame-retardant enclosure. Similar protective measures shall be applied at any other locations in the Electrical Auxiliaries Room or Control Room where wall removal is necessary.

(q) Piping systems within the Control/Auxiliary/Fuel Building Complex required for safe shutdown or to maintain off-site doses from accidents to within 10 CFR 100 guideline values shall remain seismically qualified for earthquakes up to and including the SSE throughout all structural modification work. Any changes to piping systems necessary to ensure that this condition is met shall be performed before the structural modifications are made.

(r) The licensee shall perform three grout tests for each size and orientation of reinforcing steel (rebar) to be grouted into the existing walls and hole size (considering both depth and radius) in which they are to be grouted prior to proceeding with actual construction (grouting of rebar). These tests shall be designed to demonstrate that the yield strength of the rebar can be developed by the grout. If any test result is unsuccessful the NRC shall be notified.

7.0 Schedule for Modification Work

As indicated in Figure 4-1 of PGE-1020, PGE estimates that the modification work will require 8 1/2 months plus an additional 5 weeks for the installation of plate 8. The total time for completion of the modification program is therefore slightly less than 10 months. The schedule shown in Figure 4-1 does not establish a definite time in the work sequence for the installation of plate 8. Rather, PGE proposes to install this plate during the first cold shutdown following completion of all work which must be done in advance of this activity.

The staff has reviewed this modification work schedule and believes it to be a reasonable estimate of the time that will be needed to complete the modification work. However, the staff is not in favor of leaving the installation of plate 8 (the last construction activity needed to finally complete all modifications) for the first cold shutdown after all necessary preparations have been made because the occurrence of this event is too indefinite.

If, for example, the plant is refueled in April 1980 as planned, the next refueling would most likely fall in April 1981. If the plate were ready for installation, it would be installed then. However, should it not be ready for installation at that time, the next certain cold shutdown would probably occur in April 1982, another refueling outage. Almost a year could elapse before plate 8 is installed. This is not consistent with the staff's desire that the seismic design margins for the complex be restored as soon as is reasonably possible.

The staff believes, therefore, that the license should be conditioned to require that the modifications be completed by a fixed date tied to the date of authorization, and not left to be completed at some indefinite time in the future.

The staff has therefore concluded that it would be reasonable to require that the modification program be complete not more than 12 months from the date that authorization to commence work is obtained by PGE. This fixed period of time is about 2 months longer than the period of time established by PGE to complete all work, and would allow some margin for contingencies and schedular flexibility while at the same time providing a fixed date to be finished.

8.0 Environmental Consideration

The proposed modifications were evaluated to determine whether they would entail environmental impacts of any sort. Specifically, the modification work was examined with regard to the potential for socioeconomic impacts, environmental effects from the work itself, commitment of resources, and the generation of effluents and wastes.

With regard to socioeconomic effects (i.e., impacts on local and area-wide services and service facilities), the number of additional workers over and above those normally onsite was considered. The licensee has indicated that a maximum of 25 additional workers will be onsite at any one time for a period of about six months for performance of the modification work. The type of work involved will require general construction personnel and laborers, riggers, welders, cement finishers, etc. Except for the Bechtel personnel involved in the modification work, the type of trades required should be available from the local labor pool. That being the case, there should be no need for large numbers of workers moving into the area. Moreover, the total number of additional workers required for the modifications is rather small, especially when compared to the normal compliment of employees onsite (150) and the number of workers occasionally onsite for refueling (300 or more). Consequently, the affect of the additional workers for the modification on local services should be nil. Similarly, the additional workers should have a negligible effect on traffic in the vicinity of the plant because of their small numbers. While some construction materials will be delivered to the site for the modification work, this also should have no noticeable affect on traffic in the area since it is very similar to the type of traffic which normally is present on roads in the site vicinity.

A substantial portion of the modification work itself will be performed inside existing buildings and thus will have no noticeable effect on the outside environs. Certain limited excavation work will take place outside, at the east side of the Control Building (N line wall), as will the relocation of the railroad spur. All of this outside work will be performed within the perimeter fence in areas which have been previously disturbed by construction and which are overlain by gravel. Thus, none of the modification work will involve disturbance of trees, vegetation or animal habitats. Rock and other material removed for the railroad spur relocation will be used to fill an existing material depression or for a proposed embankment, all within the perimeter fence controlled by the licensee. All of the outside work is located at least 500 feet from the nearest body of water and in level areas overlain with gravel so that runoff from the rain should not result in suspended solids problems. Activity in the plant area from construction equipment and construction work outside will result in the generation of some dust and noise. However, this is not appreciably different in nature from heavy equipment movement and construction activities which continually take place at the plant site for normal maintenance and operation. Because of this and of the fact that virtually all of the vehicle traffic and construction activity will take place in existing disturbed areas within the perimeter fence, additional environmental impacts from dust and noise due to the modifications should be negligible.

The proposed modifications will require the use of about 350 cubic yards of concrete, 38 tons of reinforcing steel and 73 tons of steel plate. All of these materials will be permanently committed to plant structures in which they are being installed. None of these materials are scarce, all are readily available in abundant supply and the amounts of these materials required for the proposed modifications are extremely small fractions of the quantities of these resources that are consumed annually in the United States. Consequently, the amounts of materials required for the proposed modifications are insignificant and do not represent a significant irreversible commitment of material resources.

Some minor solid wastes such as scrap building materials may be generated because of the proposed modifications. Since the proposed modifications are not of major proportions, the amount of solid waste generated should be small and readily accommodated by the existing collection and disposal procedures for uncontaminated waste generated by normal plant operation and maintenance. Sanitary facilities for the additional workers will be provided by temporary connections to the existing plant water and sanitary sewer systems. Thus, no additional plant water or sewer systems will be required for the modifications. In addition, the limits on types and amounts of effluent releases set forth in the facility's Technical Specifications will not be changed or affected by the proposed modifications.

Based on our evaluation, as summarized above, we have concluded that the proposed modifications will not result in significant environmental impacts and that the impacts, if any, will be negligible. In view of this and of the fact that authorization of the proposed modifications is not encompassed within any of the actions set forth in 10 CFR §51.5(a) as requiring preparation of an Environmental Impact Statement (EIS) or within any of the actions set forth in 10 CFR §51.5(b) as requiring preparation of an Environmental Impact Appraisal (EIA) and publication of a Negative Declaration (ND), we have concluded that this action falls within 10 CFR §51.5(d)(4) and that neither an EIS nor an EIA/ND need be prepared in conjunction with authorization of the proposed modifications.

9.0 Conclusion

Based on the foregoing, the staff is unable to conclude at this time that the Control Building modifications proposed by PGE meet the intent of the Order for Modification of License of May 26, 1978, which required that modifications be made to bring the facility into substantial compliance with the operating license.

Whereas quite a few aspects of the proposed modification and activities associated with the modification have been reviewed and found acceptable, questions, nevertheless, remain regarding the determination of forces, displacements, floor response spectra, capacity determination and work sequence. Resolution of these items are critical to reaching the central question of structural adequacy of the proposed modifications. At this point, the staff cannot determine that the above factors have been adequately accounted for. Therefore, uncertainty remains regarding the structural behavior of the proposed modified structure and therefore its structural adequacy.

Date: February 14, 1980

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	FILE LOCATION		

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Chairman Ahearne requests staff prepare for the Commission & serve on the Licensing Board and parties in the proceeding a summary of staff position on Trojan Control Building Modification	PRIORITY

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