Portland General Electric Company

Bart D. Withers Vice President

October 23, 1980

Trojan Nuclear Plant
Docket 50-344
License NPF-1

Director of Nuclear Reactor Regulation
ATTN: Mr. Robert A. Clark, Chief
Operating Reactors Branch No. 3
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Dear Mr. Clark:

Attached is Portland General Electric Company's response to your letter dated July 3, 1980 in which you requested certain information related to IE Bulletin 80-11. Attachment 1 hereto contains the evaluation criteria for single and double wythe and composite masonry walls for consideration of in-plane and out-of-plane loads. Attachment 2 provides the requested description and justification of tornado loads for these walls. Attachment 3 provides Licensee's basis for concluding that a long-term confirmatory testing program is unnecessary at this time. Specifically addressed in that attachment are the items which you suggested should be considered in such a test program.

Please contact us if you have any questions concerning this response.

Sincerely,

BDWithers

Bart D. Withers
Vice President
Nuclear

BDW/DJB/mg/4sa4A2
Attachments

c. Mr. R. H. Engelken, Director
U. S. Nuclear Regulatory Commission
Region V

Mr. Lynn Frank, Director
State of Oregon
Department of Energy

Office of Inspection and Enforcement
Division of Reactor Operations Inspection
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

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ATTACHMENT 1

Evaluation Criteria for Single and Double Wythe
and Composite Masonry Walls
for Consideration of In-Plane and Out-of-Plane Loads

1. GENERAL

This document is prepared in response to the letter from Mr. R. Clark to Mr. C. Goodwin, July 3, 1980, and pursuant to closeout of NRC requested documentation relative to the design criteria for single and double wythe and composite walls having safety significance for consideration of all in-plane and out-of-plane loads and the interaction of these loads. Definition of walls having safety significance is given in Attachment 2 to the letter from Mr. Broehl to Mr. Engelken, dated June 28, 1980. Justification is given for the acceptability of the design parameters both in regard to stiffness and capacity of the walls, the bases of which are either the applicable code allowables or test results. The interaction of in-plane and out-of-plane loads is discussed as it affects both the stiffness and capacity of the walls.

The discussions herein are on walls located in the Control, Auxiliary and Fuel Buildings. The same criteria and methodologies, where applicable, are used in evaluation of masonry walls in other locations in the plant.

2. DESCRIPTION OF MASONRY WALLS

Masonry walls at Trojan are located in the Control-Auxiliary-Fuel Building Complex (collectively the "Complex"), in the Turbine Building and in the Containment.

The Complex is composed of a structural steel framing system with steel beams and columns supporting reinforced concrete floor slabs, and with shear walls designed to resist horizontal loading. The major shear walls are located around the perimeter of the building and are generally composite walls. Composite construction is also utilized in areas requiring heavy radiation shielding from equipment such as filters and demineralizers. Composite walls consist of concrete core, either reinforced or unreinforced, placed between two wythes of reinforced concrete blocks. The masonry portions are composed of either standard weight or heavy weight concrete block. The composite walls generally sandwich the structural steel frame; thus, while the reinforcing steel in the masonry blocks is continuous or lapped with vertical dowels embedded in floor slabs, the core reinforcing steel, where present, is interrupted by the embedded structural steel framing in the majority of walls.

The mortared double wythe walls generally serve as divider or partition walls for areas requiring: a) light radiation shielding (eg, from piping); missile barriers (eg, equipment or tornado missiles); and/or train separation for common mode events (eg, flooding and fire). They are made of either standard weight or heavy weight concrete blocks, are reinforced and are fully grouted. These walls are nominally 14-inch or 16-inch thick and have reinforcing steel both in the horizontal and vertical directions. The collar joint between the wythes is mortared and the wythes are further connected by #3 tie bars spaced at 4 feet centers both horizontally and vertically.

The single wythe masonry walls serve as partition walls or fire barriers primarily where missiles and radiation are not a concern. They are made of either standard or heavy weight concrete blocks, are fully grouted and reinforced both horizontally and vertically, and are 8 inches and 12 inches thick.

Except for the 12-inch thick single wythe walls where closed cells are used, all masonry walls are constructed with A-type concrete blocks with one end open.

3.0 CONSTRUCTION OF WALLS

The steel frame and concrete floor slabs of the Complex were constructed first. Later the concrete block shear walls were constructed with concrete cores placed between the two wythes of reinforced concrete block masonry.

The Complex is designed to have the steel frame carry most of the vertical load and the walls carry the loads caused by earthquakes.

3.1 Erection

Masonry was laid using standard construction procedures. Each course was solidly bedded in mortar. Joints were approximately 3/8 inch high and extended full depth of face shells. Anchors, wall plugs, accessories, and other items required to be built-in were placed as the masonry wo k progressed. Spaces around built-in items were solidly filled with grout or mortar. Clean-out openings were provided for the concrete block cells at the bottom of each grout lift. The collar joint between wythes in double wythe walls was mortared.

Concrete block walls which extend from floor slab to floor slab were connected to the respective floor slabs or foundation grade beams by dowels. Grout was placed in lifts not exceeding 8 feet. Each placement was thoroughly vibrated to insure consolidation and bonding to the preceding placement.

Code specified standard practice was followed. At the top of each lift, laitance was removed and the existing fill was dampened and coated with neat cement when work was stopped for a period of 45 minutes or longer before additional fill was placed.

3.2 Tests, Inspection, and Quality Assurance

For structures designated as Category I, the Contractor established and maintained a Quality Assurance Program in accordance with Appendix B to 10CFR Part 50. A special masonry inspector was employed to perform quality control inspection of the masonry wall construction.

4.0 APPLICABLE CODES, STANDARDS, AND REFERENCE DOCUMENTS

Unless otherwise stated, evaluations of all masonry walls are performed in accordance with applicable portions of the following codes and reference documents.

4.1 Codes and Standards

a. International Conference of Building Officials, "Uniform Building Code" (UBC), 1967 Edition, Volume 1, Chapter 24.

b. Building Code Requirements for Reinforced Concrete ACI 318-71, American Concrete Institute.

4.2 Reference Documents

a. Trojan Final Safety Analysis Report (FSAR)

b. Report on Design Modifications for the Trojan Control Building (PGE-1020), Revision 4, February 12, 1980.

5.0 LOADS AND LOAD COMBINATIONS

This section describes the governing loads and load combinations for which the masonry walls are evaluated. Other load combinations given in FSAR Section 3.8 do not govern the design and, therefore, are not listed here.

$U = 1.25(D + L + H_o + E) + 1.0\ T_o$

$U = 1.4(D + L + E) + 1.0\ T_o + 1.25\ H_o$ *

$U = 1.0D + 1.0L + 1.0E' + 1.0T_o + 1.25\ H_o$

$U = 1.0D + 1.0L + 1.0W_t + 1.0T_o + 1.25H_o$

*For shear wall in-plane seismic loads only

where,

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

L = Live load.

T_o = Thermal loads due to temperature gradient through wall during operating conditions.

H_o = Force on structure due to thermal expansion of pipes during operating conditions.

E = Operating Basis Earthquake (OBE) resulting from ground surface acceleration of 0.15g.

E' = Safe Shutdown Earthquake (SSE) resulting from ground surface acceleration of 0.25g.

W_t = Tornado loads (tornado wind loads and missile effects are discussed in FSAR Section 3.3.2).

6.0 REEVALUATION PROCEDURE

This section describes the procedures for reevaluation of masonry walls with respect to in-plane loads, out-of-plane loads and the interaction of in-plane and out-of-plane loads. The evaluation criteria are outlined for each of the loading conditions and justifications for their acceptability are provided.

6.1 In-Plane Loading Condition

In order to evaluate the masonry walls, the in-plane shear force in each horizontal load carrying wall element corresponding to the governing loading condition is first determined. The wall capacity is then determined in accordance with the governing behavioral mode and compared with the shear force.

6.1.1 Analytical Method for Shear Force Determination

The governing loading conditions for the in-plane loading are the ones involving earthquake loading, since the tornado produces loads which are much smaller. Therefore, the discussion on the methodology for shear force determination is limited to seismic loading.

6.1.1.1 Seismic Input Criteria

The seismic input criteria define the input to the seismic analysis in terms of peak ground accelerations for the OBE and the SSE, along with the associated ground response spectra. The OBE criteria are more conservative than the SSE criteria and, hence, only the OBE criteria are used in the evaluation. The OBE input criteria that are applied with respect to peak ground acceleration, ground response spectra and associated damping values are those specified in FSAR Section 3.7.

6.1.1.2 Analytical Model

In order to perform seismic analyses for determination of loads, displacements and floor response spectra, an analytical model of the structure which characterizes its behavior in an earthquake is developed. Analytical models for Seismic Category I structures are described in FSAR Section 3.7. For the Complex, a more detailed analytical model is also constructed. This model is a linear elastic three-dimensional finite element model (the "STARDYNE" model) which provides a more accurate representation of mass distribution and stiffness characteristics of the Complex. Mass distribution in the model is based on knowledge of the distribution of mass in the Complex, and the requirements of FSAR Section 3.7 with respect to lumping masses are met. The stiffness of the structural elements in the model is based on material properties and boundary conditions of these elements. The analytical model is based on the conventional assumption of linear elastic behavior and perfect connectivity of the wall panels. However, a more sophisticated approach is used considering the potential nonlinear behavior which results in a reduction of stiffness of the structural elements.

A reduction in the stiffness of an entire structure will change its natural frequency and could potentially result in an increase or decrease in the seismic loads imposed on the overall structure. However, as explained in Section 6.1.1.4 of this document, the calculated base shear in the Complex is at its maximum value since the predominant frequencies of the Complex correspond to the peak of the ground response spectrum. Any changes in structural frequency will, therefore, not increase the base shear force. Since walls with greater relative stiffness tend to attract more load than less stiff elements, a change in stiffnesses due to nonlinear behavior could alter the distribution of the seismic loads within the structure. A reduction in stiffness will affect building frequencies and floor response spectra. As explained in the following sections, the impacts of such a reduction in stiffness have been accounted for by making iterative STARDYNE analyses and processing the responses predicted by the linear elastic model to account for nonlinear effects.

6.1.1.3 Seismic Analysis

With application of the seismic input criteria and the use of the STARDYNE model, the Complex is seismically analyzed using a response spectrum method in conformance with FSAR Section 3.7. Other applicable criteria specified in that section, including provisions regarding consideration of vertical response, torsional modes of vibration and calculation of overturning moments, are applied in conformance with that section. In addition, although a separate analysis was not required for the original design of the Complex, the potential effects of earthquake cycles are considered in the reevaluation of the Complex.

A potential source of nonlinear behavior of the Complex results from cracking that may develop in the concrete block wall panels under seismic loading conditions. Another potential source of nonlinearity is the possible relative vertical movement between adjacent panels of a wall which are partially separated by embedded steel columns.

The specimens in the testing program described in Section 6.1.3 of this document exhibited nonlinear behavior under certain conditions and, therefore, data from such tests are used to predict potential reductions in initial stiffness as a function of the percentage of vertical reinforcement, level of normal stress, shear stress and the number of cycles of stress on the wall. Since the reduction in stiffness is a function of the level of shear stress, several iterative STARDYNE analyses were made until the shear stress levels and reduced stiffnesses were mutually consistent.

In determining the amount of normal stress contributing to wall stiffness, the dead load of the portions of the wall above the elevation under consideration, reduced for the effect of vertical earthquake, is taken into account. Consideration is given to the possible reduction in effective dead load due to the potential effects of creep and shrinkage, stiffening of beams due to encasement in concrete, and the effect of changes in mean wall temperature for exterior walls. Quantification of the potential effects of such factors on the magnitude of dead load is included in the analysis. It was also shown on the basis of the testing program that vertical growth which would occur in the wall panels during an earthquake due to the development of flexural cracking would more than compensate for the potential reduction in dead load due to these factors when the panels are subjected to numbers of stress cycles (see Ref. 1).

Seismic loads may also create a nonlinear condition in that overall bending tends to increase compressive load on one end of a wall which is parallel to the component of the earthquake being considered and to decrease the compressive load on the other end of that wall. Such changes in vertical loads would result in an increase and a decrease respectively in wall stiffness in the local wall areas. (This behavior is referred to as "gross bending effect)". The STARDYNE analysis does not account for this local behavior. However, since the increase in stiffness in some areas of a wall due to gross bending is simultaneously associated with a decrease in stiffness in other areas, the overall stiffness would not change substantially.

6.1.1.4 Load Determinations

The STARDYNE linear elastic analysis predicts the magnitude of the seismic loads to be resisted by the Complex and the distribution of such loads.

An overall reduction in the stiffness of the Complex due to potential nonlinear behavior would not result in any significant change in the total inertia forces to be resisted by the structure, since the natural frequency of the Complex approximates the frequency which corresponds to the peak of the ground response spectra.

Reductions in stiffness due to potential dead load reductions would not cause a substantial change in the relative stiffness of the walls in the Complex because the capacities of the major shear walls are sufficiently similar and relative changes in the stiffness of such walls would be approximately the same. Thus, the relative distribution of loads among the major shear walls would not be substantially altered by consideration of such potential effects.

Reductions in stiffness due to potential effects of gross bending could result in a portion of the load being shifted from panels on the tension side of the shear walls to panels on the compression side. However, such changes would be offset by corresponding changes in shear capacity.

6.1.2 Evaluation of Capacity

For evaluating in-plane capacities of the walls of the Complex, the governing capacities are determined from consideration of three distinct behavioral modes:

(a) flexural mode
(b) sliding mode
(c) diagonal tension (shear) mode

Equations for each of these behavioral modes are described below, with appropriate references. Capacities calculated by application of these equations are in substantial agreement with results obtained from the testing program described in Ref. 2 and discussed in Section 6.1.3 of this document.

Notations

A_B = Area of masonry portion of wall section, inches^2
A_g = Area of concrete including cell grout and core, inches^2
A_w = $A_B + A_g$ = Total area of section, inches^2
A_{sc} = Cross sectional area of steel column
f'_c = Specified compressive strength of concrete, psi
f'_g = Computed compressive strength of composite section, psi
f'_b = Compressive strength of block, psi
f_y = Specified yield strength of reinforcing steel, psi
f_{ys} = Specified yield strength of column steel, psi
h = Height of wall panel, inches
l_w = Width of wall panel, inches
N_c = Total number of columns crossing the shear plane
V_b = Shear force resistance of the frictional component of beam to column connection
V_s = Shear force resistance developed by shear-friction of reinforcing steel on vertical edges of wall panel
V_1 = $V_b + V_s$
v_f = Nominal shear stress capacity of wall panel, psi
t = Thickness of wall
u = Coefficient of shear friction
ρ_v = Vertical reinforcing steel ratio
ρ_h = Horizontal reinforcing steel ratio
σ_o = Wall compressive stress due to normal load, psi

6.1.2.1 Flexural Mode

(i) Double Curvature

The procedure for determination of capacity of a wall behaving in the double curvature mode is described in Section 3.4.2.2 of Ref. 2, and the equation developed therein is as follows:

$$v_f = \frac{lw}{h} (0.93 \rho_v f_y + 0.94 \sigma_o)$$

(ii) Single Curvature

For behavior in the single curvature mode, the top edge is considered free and the bottom edge is considered restrained. The magnitude of this moment restraint depends on the vertical reinforcing steel, the effective panel normal load at the top edge, and the vertical shears as limited by the beam-column connection and the horizontal reinforcing steel across the two vertical edges. The equation for the single curvature shear capacity is:

$$v_f = \frac{lw}{h} (0.456 \rho_v f_y + 0.47 \sigma_o) + \frac{V_l}{ht}$$

6.1.2.2 Sliding Mode

The method for determining sliding resistance at a wall slab interface of the Complex walls is presented in Ref. 3. The total sliding resistance is developed as the summation of the resistance offered by the embedded steel columns, with the shear strength of the steel taken as $f_y/\sqrt{3}$, and the shear-friction developed by the normal force and the vertical reinforcing steel crossing the shear plane. The equation for the sliding resistance is:

$$v_f = u\sigma_o + 1.4\ \rho_v f_y + \frac{A_{sc}}{l_w t}\ \frac{f_{ys}}{\sqrt{3}}\ \frac{(N_c-1.5)}{(N_c-1)}$$

where the coefficient of friction, u, for the normal force is taken as the weighted average based on the relative areas of bearing composed of mortar bed joints and concrete in the block cell and the wall core. In the absence of any code-specified value, the coefficient of friction for the mortar bed joints is taken as 0.75 which is a lower bound value used in Ref. 4. The numerator in the multiplying factor for the column resistance, $(N_c - 1.5)$, represents the number of columns available to resist sliding in the entire wall section and is equal to the total number of columns crossing the shear plane minus one end column and half the other end column. The term $(N_c - 1)$ in the denominator represents the number of panels; therefore,

$$v_f = \left(0.75\ \frac{A_B}{A_w} + 1.4\ \frac{A_g}{A_w}\right)\sigma_o + 1.4\ \rho_v f_y + \frac{A_{sc}}{l_w t}\ \frac{f_{ys}}{\sqrt{3}}\ \frac{(N_c-1.5)}{(N_c-1)}$$

6.1.2.3 Diagonal Tension (Shear) Mode

For evaluating the diagonal-tension related shear capacity of a wall panel, the ACI deep beam equation is used which gives the shear stress v_c as

$$v_c = (3.5 - 2.5 \frac{M}{l_w V})\ 2\sqrt{f'_g}$$

where M and V are the moment and shear force at the section. For a cantilever shear panel, M = Vh.

$$v_c = (3.5 - 2.5 \frac{h}{l_w})\ 2\sqrt{f'_g}$$

The compressive strength of the grouted masonry composite wall section is taken as a weighted average of masonry and concrete.

$$\text{or, } \sqrt{f'_g} = \sqrt{f'_b}\ (\frac{A_B}{A_w}) + \sqrt{f'_c}\ (\frac{A_g}{A_w})$$

Accounting for the normal stress, σ_o, on the wall section the resulting shear capacity, v_c*, is

$$v_c^* = \sqrt{(v_c + \frac{\sigma_o}{2})^2 - (\frac{\sigma_o}{2})^2}$$

The horizontal and vertical reinforcing steel provide additional contributions to the diagonal tension strength. Test results from specimens with height-to-width ratio of 0.5 (Ref. 5) and height-to-width ration of 1.17 (though titled as 1.0 in Ref. 6) indicated that the cracking plane engages both the horizontal and vertical reinforcing steel and, therefore, may be considered as equally effective in providing the shear resistance. Thus the final equation for the diagonal tension mode is:

$$v_f = v_c^* + 0.5 \left(\rho_v + \rho_h \frac{h}{l_w}\right) f_y$$

6.1.3 Criteria Justification

Section 3.8 of the FSAR indicates that the "Concrete Block Walls" in Category I structures, including the Complex, are designed to the UBC requirements for masonry. As noted earlier, the major shear walls of the Complex are constructed of a high strength concrete core, either reinforced or unreinforced (nonmasonry structural element), sandwiched between two wythes of reinforced grouted concrete blocks (masonry units). Since the provisions of the UBC applicable to masonry construction do not address a combination of masonry and nonmasonry units, the shear walls of the Complex are not addressed directly by the UBC. However, Sections 106 and 107 of the UBC allow departures from certain detailed code formulae and quantifications where such departures are supported by substantiation through testing. In addition, testing is the basis upon which most code criteria have been established. Accordingly, application of test results in the determination of wall capacities is appropriate.

Such a testing program, conducted during the period September 1978 through February 1979, is described in Appendix B to Ref. 2. The testing utilized 23 specimens which were designed to simulate parameters of the walls in the Complex. The materials used in the construction of these specimens as well as their aspect ratios and thicknesses were similar to the wall panels in the Complex. The testing program was extensive for its purposes and was typical of testing performed to substantiate code compliance.

The test results indicated that the hybrid unit of steel column-composite wall will have a lower bound diagonal tension capacity of 300 psi for the large percentage of composite walls which have an aspect ratio of approximately 0.5. However, this behavior relies upon bond between the embedded steel frame and the surrounding concrete. In the development of the capacity criteria, this important structural aspect is conservatively neglected.

The capacity values obtained in the tests are not factored directly into the computation of the capacities of the Complex walls. Rather, the characteristics of the composite walls demonstrated by the test specimens formed the basis for development of theoretical equations which predicted the shear capacities of the individual wall panels as functions of percentage of vertical and horizontal reinforcing steel, the embedded steel columns, and the vertical load bearing on the wall panels from the dead load of the wall above.

The formulae given in Section 6.1.2 of this document, based upon understanding of behavior gained from the testing program, reflect at least the same level of conservatism as code equations.

6.2 Out-of-Plane Loading Conditions

Out-of-plane loads on the masonry walls are the loads which occur in a direction perpendicular to the wall panel. Examples of out-of-plane loads are wind loads, seismic loads associated with the mass of the wall and the supported equipment, piping or equipment restraint reactions, seismic induced relative floor-to-floor displacements, and differential temperature effects across the wall thickness (exterior walls only). Since the out-of-plane inertial loading of a wall is a function of its natural frequency of vibration and is consequently dependent upon the wall's stiffness, any potential reduction of stiffness due to tensile stresses in the materials comprising the wall must be taken into account.

The out-of-plane loads on the masonry walls are resisted by the wall acting as a flexural member. In order for both the wythes of a double wythe wall or the wythe and the concrete core in a composite wall to act together as a unit, the respective interfaces must be capable of transmitting a small shear stress. The allowable values of collar joint shear stress for double wythe walls and interface stress for composite walls are listed in Tables 2, 3 and 5.

The loading combinations described in Section 5.0 of this document are also applicable for out-of-plane loadings, except that the load factor associated with the OBE is taken as 1.25. This is justified because in accordance with the FSAR, the 1.4 load factor is applicable only to those modes of behavior which contribute to the stability of the structure, ie, in-plane behavior of shear walls (walls for which in-plane shear behavior is relied upon globally). For nonshear wall concrete structures the applicable loading combinations contain an OBE load factor of 1.25.

The global stiffness and strength of the Complex are controlled by the in-plane stiffnesses and strengths of the walls. Neither the out-of-plane stiffnesses nor the out-of-plane capacities are relied upon to provide lateral load resistance of the overall system. Also, as discussed in Section 6.3 of this document, the reduction in a wall's out-of-plane stiffness due to cracking will not significantly impair its capacity to resist the in-plane load demand. Recognizing this important wall characteristic along with the implicit FSAR criteria of a wall not being subjected to the effects of maximum in-plane and out-of-plane load demands simultaneously, it is reasonable to have a lower load factor for the wall out-of-plane behavior.

The various parameters used in the analysis of wall out-of-plane behavior and determination of wall capacity are summarized in Tables 6 and 7, respectively. These tables also show the values of the parameters, their bases, and the reference document. The following sections discuss the procedure for determining the out-of-plane loads, capacities and the criteria for allowable stresses.

6.2.1 Structural Response of Masonry Walls

6.2.1.1 Equivalent Moment of Inertia (I_e)

To determine out-of-plane frequencies of masonry walls, the uncracked behavior and capacities of the walls (Step 1) and, if applicable, the cracked behavior and capacities of the walls (Step 2) are considered.

Step 1 - Uncracked Condition

The equivalent moment of inertia of an uncracked wall (I_t) is obtained from a transformed section consisting of the block, mortar, cell grout and core concrete, neglecting block and mortar on the tension side.

Step 2 - Cracked Condition

If the applied moment (M_a) due to all loads in a load combination exceeds the uncracked moment capacity (M_{cr}), the wall is considered to be cracked. The equivalent moment of inertia (I_e) is then computed as follows:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_t + \left(1 - \left(\frac{M_{cr}}{M_a}\right)^3\right) I_{cr}$$

$$M_{cr} = f_r \frac{I_t}{y}$$

where,

M_{cr} = Uncracked moment capacity
M_a = Applied maximum moment on the wall
I_t = Moment of inertia of transformed section
I_{cr} = Moment of inertia of the cracked section
f_r = Modulus of rupture (as specified in Table 2)
y = Distance of neutral plane from tension face

If the use of I_e results in an applied moment M_a which is less than M_{cr}, then the wall capacity is verified for M_{cr}.

6.2.1.2 Frequency Variations

Variations in structural frequencies of a masonry wall due to variations in structural properties and mass are taken into account. Variations include mass, boundary conditions, extent of cracking, in-plane and out-of-plane loads, two-way action, and composite action of multi-wythe walls.

When plate action is utilized, the values of uncracked tensile capacity for masonry horizontal spans are based on the modulus of rupture of cell concrete (head joint) specified in Table 1. When the lowest frequency of a vertical strip model is on the high frequency side of the response spectrum peak, the lower bound rupture modulus values for the horizontal span are used. When the lowest frequency of a vertical strip model is on the low frequency side of the response spectrum peak, the higher bound rupture modulus values for the horizontal span are used.

6.2.1.3 Damping

FSAR Table 3.7-1 lists the damping values for structures made of reinforced concrete as 2% when the stresses are at working stress level and 5% when the stresses reach yield point. This criterion has been conservatively interpreted as implying that the ground response spectra for 0.15g OBE analysis be used with 2% damping and for 0.25g SSE with 5% damping, although the load demands predicted by them are approximately the same.

The out-of-plane wall loading is determined by considering the wall as a decoupled structure and treating it similarly to an item of equipment placed on the floor. The floor response spectra utilized for determination of wall responses are generated from global analyses of the structure using damping ratios of 2% and 5% for OBE and SSE inputs, respectively, and include conservative spectra broadening to account for potential changes in overall building stiffness. The out-of-plane wall response is evaluated by considering the state of stress in the wall and, therefore, for both OBE and SSE responses, structural damping for uncracked ($M_a < M_{cr}$) and cracked ($M_a > Mcr$) walls is taken to be 2% and 5%, respectively.

6.2.1.4 Accelerations

For a wall spanning between two floors, the effective acceleration is taken to be the average of the accelerations as given by the bounding floor response spectra corresponding to the natural frequency and assumed damping of the walls (Ref. 7).

6.2.1.5 Interstory Drift Effects

To d- -ine the forces associate with interstory displacemer e following procedure i. d:

(1) The capacities at the boundaries, prior to cracking, are calculated considering both the upper and lower bound values of the modulus of rupture specified in Table 1. The moment of inertia of the wall section is determined based on 6.2.1.1, Step 1.

(2) Following cracking, the moment capacities at the boundaries are determined consistent with the displacement profile for the structures and based on a value of yield strength of the reinforcing steel of 50 ksi.

(3) Amplified interstory displacements accounting for possible nonlinearities in the north-south and the east-west directions in the Complex are taken from the results of finite element analyses and post-processing which account for potential changes in stiffnesses of the structural elements.

6.2.1.6 Concentrated Loads

(i) Two-Way Action

Where two-way bending is present in the wall, the localized moments per unit width under a concentrated load are determined using appropriate analytical procedures for plates.

(ii) One-Way Action

For one-way bending, local moments are determined using beam theory and an effective width of six times the wall thicknesses.

6.2.1.7 Thermal Effects

Thermal effects, applicable to exterior walls only, are considered for partially fixed end conditions, and an analysis is performed based on cracked section.

6.2.1.8 Tornado Loading

The exterior walls and slabs of the Complex and other structures in the Plant housing safety-related systems are evaluated for their resistance to tornado wind pressure and tornado-induced pressure differential prescribed in FSAR Section 3.3. These resisting elements are also evaluated to withstand penetration by the potential tornado missiles specified in the FSAR. The evaluation is made by applying the modified Petry Formula in accordance with FSAR Section 3.5.

6.2.1.9 Stress Calculations

All stress calculations are performed by the conventional working stress design method. The collar joint shear stresses in double wythe walls and the grout-masonry interface shear stresses in composite walls are determined by the relationship VQ/Ib. The effects of cracking are appropriately considered in determining section properties.

6.2.2 Evaluation of Capacities

The allowable stress limits for the capacity evaluation of masonry walls are shown in Tables 1 through 5. The uncracked section moment capacities for both horizontal and vertical spans are calculated based on the transformed sections in the respective directions by considering the strain compatibility of block, mortar, cell grout and core concrete. The block and mortar on the tension face is, however, neglected. The capacity at the wall boundaries is calculated taking into account the modulus of rupture of the materials at those locations and considering, as appropriate, both the upper and lower bound values.

6.2.3 Criteria Justification

The allowable tension, compression, shear, bond and bearing stresses in masonry and the allowable tension, or compression stresses in reinforcing steel are as given and provided for in the governing codes. There is no clear definition, however, of the permissible shear stresses in the collar joint of double wythe walls, or the tension and shear stresses at the grout-block interface in composite walls. The following tests were, therefore, performed to establish the acceptable stress limits.

6.2.3.1 Collar Joint Shear in Double Wythe Walls

In-situ direct shear tests were performed on existing representative double wythe masonry walls in the Complex. The direct shear was taken as a measure of the collar joint mortar capacity in double wythe masonry walls. Details of the testing program and the test results are provided in Ref. 8. By applying conservative factors of safety on the failure shear stresses, permissible ultimate stresses were obtained for both standard weight and heavy weight block walls. These values are shown in Tables 2 and 3.

6.2.3.2 Interface Principal Stress in Composite Walls

Tests were performed on existing composite walls in the Complex to obtain a measure of the tensile bond existing at the concrete core-block interface. The details of the testing program and test results are provided in Ref. 9. By a conservative application of the strength reduction factor, the failure tensile stress was reduced to obtain a measure of the permissible ultimate principal stress at the interface. This limiting value is shown in Table 5.

6.2.3.3 Material Properties of Concrete Grout in Masonry Block Walls

In addition to the two sets of tests described above, tests were also run to determine the compressive strength, modulus of elasticity and splitting tensile strength of the in-place concrete grout in the double wythe masonry walls in the Complex. The tests were performed on core samples of the concrete grout and are discussed in Ref. 10. The test results formed the basis of the parameters used for stiffness and strength calculations for double wythe walls and are summarized in Table 1.

6.3 Interaction of In-Plane and Out-of-Plane Loads

The interaction of in-plane and out-of-plane loads could potentially affect the stiffness and the capacity of the walls. As indicated below, this interaction has only a minor influence on both quantities.

The global (overall) stiffness of the Complex is controlled by in-plane stiffnesses of the walls. The majority of the in-plane stiffness comes from composite walls in the Control and Auxiliary Buildings and reinforced concrete walls of the Fuel Building. Double wythe walls contribute less than 10% of the stiffness at any elevation. When subjected to out-of-plane loads, composite walls will crack only at the top and bottom edges of the wall panels, and this crack will only be open on one side of the wall (the tension side). The compression side will be tightly closed resulting in a very small net vertical strain across the thickness. If the wall panels do not crack due to the out-of-plane loading, there is no effect on the in-plane stiffness since the net vertical strain across the thickness is zero. The stiffness used in the finite element seismic analysis was derived from test specimens that had a horizontal crack through the thickness at the bottom for all specimens and additional cracking in some specimens. For these cracks also, the out-of-plane loads would tend to close the crack on the compression side of the wall and open the crack on the tension side. Again, this will produce a very small net vertical strain resulting in a negligible effect on the in-plane stiffness.

The out-of-plane inertia loads are controlled by the frequency and damping of the wall. The frequency is in turn controlled by out-of-plane flexural stiffness. The primary effect of the in-plane shear force is to cause a crack at the top and bottom of the wall panel. This causes a reduction in the rotational resistance at these locations which results in a reduction in the out-of-plane stiffness of the wall panel. This effect has been considered in determining the wall panel frequency. Any further cracking of the panel will be due to the out-of-plane loading which at this stage of deformation is independent of the in-plane loading.

The loads that develop due to interstory drift are governed by the moment capacity at the top and bottom edges of the wall and the interstory deflection. The interstory deflection depends on global stiffness of the structure which is comprised of the in-plane stiffnesses of the shear walls and not on their out-of-plane stiffnesses. As indicated above, the out-of-plane loads have only a minor influence on the in-plane stiffnesses. The moment capacity at the top and bottom edges depends on the modulus of rupture and the tension stresses at these locations. Since the stresses due to the in-plane and out-of-plane behavior are additive in some fashion, although both do not reach their respective maximums simultaneously, a crack will develop at these locations. This interaction has been included and considered in the evaluation of the wall.

The ability of the total structure to resist the global effects of the earthquake is determined by the in-plane shear resistance of the walls. The out-of-plane capacities of the walls are not relied upon to resist the global effects since they are negligible by comparison.

The in-plane shear capacity is controlled by the vertical reinforcing steel. If the reinforcing steel is stressed less than yield, the out-of-plane effects will tend to increase the stress on one side of the wall and decrease the stress on the opposite side, resulting in only a minor net vertical strain across the thickness. Since the loading from an earthquake is dynamic in nature and not a sustained loading, the energy absorbing characteristics of the wall can limit the additional straining beyond yield. The out-of-plane effects will simply cause some additional straining on one side and a reduction in the strain on the opposite side. If seismic loads were able to develop to the extent predicted by the linear elastic analysis for a 0.15g OBE or a 0.25g SSE, the double curvature model for capacity evaluation would predict the maximum strain in the reinforcing steel, which occurs in the corners of the wall panels, to be at or above yield in these small local zones to the modest level as shown in Ref. 11.

The capacity of the wall panels with respect to out-of-plane inertia loads is controlled either by the modulus of rupture of the concrete grout, or the stress in the reinforcing steel which are both maximum at the panel mid-height and decrease toward the edge. The interaction of the in-plane and out-of-plane inertia loads does not create maximum stress conditions in the reinforcing steel at the same location, and consequently, the criteria capacity limits corresponding to these two loading conditions are not mutually restrictive.

References:

1. Licensee's April 14, 1980 Response to NRC Staff Questions of April 3, 1980 and April 10, 1980, NRC question No. 7.

2. PGE-1020, Report on Design Modifications for the Trojan Control Building, Rev. 4, February 12, 1980.

3. PGE letter to A. Schwencer dated June 29, 1979, response to NRC Staff Question 11 (a) dated May 18, 1979.

4. Hatzinikolas, M., Longworth, J., and Warwaruk, J., "Evaluation of Tensile Bond and Shear Bond of Masonry by Means of Centrifugal Force", Alberta Masonry Institute, Edmonton, Alberta.

5. Hidalgo, P. A., Mayes, R. L., McNiven, H. D., and Clough, R. W., "Cyclic Loading Tests of Masonry Single Piers - Volume 3 - Height to Width Ratio of 0.5", Report No. UCB/EERC-79/12, College of Engineering, University of California, Berkeley, California, May 1979.

6. Chen, S. J., Hidalgo, P. A., Mayes, R. L., Clough, R. W., and McNiven, H. D., "Cyclic Loading Tests of Masonry Single Piers - Volume 2 - Height to Width Ratio of 1", Report No. UCB/EERC-78/28, College of Engineering, University of California, Berkeley, California, December 1978.

7. Letter from Broehl to Engelken dated June 10, 1980, Attachment 6, "Justification of Using Approximation Method to Determine Maximum Wall Panel Responses to Seismic Motion".

8. Report on Tests of Shear Strength of Collar Joint Mortar in Double Wythe Masonry Walls, April 14, 1980.

9. Report on Testing of Composite Masonry Walls November, 1979.

10. Summary of Meeting held on May 1-2, 1980 to Discuss Resolution of the Trojan "Wall Problem", Enclosure 2, dated May 16, 1980.

11. Letter from Broehl to Schwencer dated December 22, 1979, response to NRC question 9 dated September 14, 1979.

TABLE 1

MATERIAL PROPERTIES FOR CAPACITY AND STIFFNESS EVALUATION

MATERIALS	f'_c (psi)	f_r (psi)	f'_m (psi)	f_y (ksi)	E (psi)	BASIS
HEAVY BLOCK	4,100	215-500	---	---	1.75×10^6*	See 1 & 2 below
STD. BLOCK	2,700	175-400	---	---	1.0×10^6*	See 1 & 2 below
MORTAR (BED JOINT)	3,700	50-125	---	---	---	See 2 below
MORTAR (HEAD JOINT)	---	0-40	---	---	---	UBC
CELL CONCRETE (CONT.)	5,000	450-1000	---	---	4.0×10^6	See 1 & 2 below
CELL CONCRETE (COLD JT.)	---	200-450	---	---	---	See 2 below
CELL CONCRETE (HEAD JT.)	---	60-250	---	---	---	See 2 below
DRY PACK		10-50				Eng. Judgment
PRISM (HEAVY BLOCK)	---	---	4,000	---	---	See 3 below
PRISM (STD. BLOCK)	---	---	2,000	---	---	See 3 below
REINFORCING STEEL	---	---	---	40 (Min) 50 (Max)	30.0×10^6	See 4 below

Where:

f'_c = Design compressive strength of applicable material, derived from test results

f_r = Design modulus of rupture or tensile bond strength

f'_m = Design compressive strength of masonry, derived from test results

f_y = Design yield strength of reinforcing steel

E = Design modulus of elasticity.

* Composite modulus consisting of block and mortar

Notes:

1. Lower bound based on $6.7\sqrt{f'_c}$ with factor of safety equal to 2.0; upper bound based on 7 to 8 $\sqrt{f'_c}$ (Holm, T. A., Proceedings of the North American Masonry Conference; "Structural Properties of Block Concrete", August 1978, Boulder, Colorado)

2. Letter from Broehl to Schwencer, dated April 1, 1980

3. Test results - refer to NRC/PGE/Bechtel meeting dated May 1-2, 1980 (Ref. 9)

TABLE 2

ALLOWABLE STRESSES FOR HEAVY WEIGHT DOUBLE WYTHE MASONRY WALLS

TYPE OF STRESS	ALLOWABLE STRESS	BASIS
A. Masonry		
1. Membrane Compression	1.50 S	See Note 1
2. Flexural Compression	1,200 psi	$f'_c/3$ **
3. Flexural Shear	1.50 S	See Note 1
4. Collar Joint Shear	10 psi	$0.45\ (\bar{X} - 2\sigma)$
5. Bearing	1,100 psi	$0.3\ f'_c$ ***
B. Reinforcing Steel		
1. Tension/Compression	$0.9f_y$	ACI

S = Allowable working stress based on Table No. 24H of UBC-1967

f_y = Design yield strength of reinforcing steel (40,000 psi)

** Flexural compression stress is derived from the lesser of the block and mortar strength divided by a factor of 3.0

*** Based on 0.3 times the compressive strength of the weakest material

Note:

1. Adjustment of UBC allowable working stress values for ultimate strength conditions

TABLE 3

ALLOWABLE STRESSES FOR STANDARD WEIGHT DOUBLE WYTHE MASONRY WALLS

TYPE OF STRESS	ALLOWABLE STRESS	BASIS
<u>A. Masonry</u>		
1. Membrane Compression	1.50 S	See Note 1
2. Flexural Compression	900 psi	$f'_c/3$
3. Flexural Shear	1.50 S	See Note 1
4. Collar Joint Shear	20 psi	$0.45 (\bar{X} - 2\sigma)$
5. Bearing	800 psi	$0.3 f'_c$
<u>B. Reinforcing Steel</u>		
1. Tension/Compression	$0.9f_y$	ACI

For definition of terms see Tables 1 and 2.

Note:

1. Adjustment of UBC allowable working stress values for ultimate strength conditions

TABLE 4

ALLOWABLE STRESSES FOR SINGLE WYTHE MASONRY WALLS

TYPE OF STRESS	ALLOWABLE STRESS		BASIS
	HEAVY WEIGHT	STANDARD WEIGHT	
A. Masonry			
1. Membrane Compression	1.50 S	1.50 S	See Note 1
2. Flexural Compression	1200 psi	900 psi	$f'_c/3$
3. Flexural Shear	1.50 S	1.50 S	See Note 1
4. Bearing	1100 psi	800 psi	$0.3\ f'_c$
B. Reinforcing Steel			
1. Tension/Compression	$0.9f_y$	$0.9f_y$	ACI

For definition of terms see Tables 1 and 2.

Note:

1. Adjustment of UBC allowable working stress values for ultimate strength conditions

TABLE 5

ALLOWABLE STRESSES FOR COMPOSITE WALL

TYPE OF STRESS	ALLOWABLE STRESS		BASIS
	HEAVY WT. BLOCK	STD. WT. BLOCK	
<u>A. Masonry</u>			
1. Membrane Compression	1200 psi	900 psi	$f'_c/3$
2. Flexural Compression	1200 psi	900 psi	$f'_c/3$
3. Interface Principal Stress	60 psi	60 psi	See Note 1
4. Bearing	1100 psi	800 psi	$0.3\ f'_c$
<u>B. Reinforcing Steel</u>			
1. Tension/Compression	$0.9f_y$	$0.9f_y$	ACI

For definition of terms see Tables 1 and 2.

Note:

1. The allowable stress is based on test data obtained from the composite wall block/concrete tensile bond test, Ref. 9.

Table 6

Analysis Assumptions Affecting Load Demand of Out-of-Plane Bending

Parameter	Value Used	Basis	Documentation
A. Frequency			
1. Moment of Inertia			
I_t	transformed section	Principle of Mech., planes remain plane	Supplement No. 4 to LER 79-15
I_e	---	ACI 318-71, Sect. 9 for reinforced concrete beams	
f_r	---	See Table 1	
2. Boundary conditions		Inplane and out-of-plane effects can cause cracks along top and bottom edges reducing the rotational resistance, hence, moment free condition is a reasonable approximation	Supplement No. 4 to LER 79-15
Beam strip:	pin-pin		
Plate:	pin-pin at top and bottom edges		
3. Modulus of Elasticity	See Table 1	Test results	Letter from Broehl to Engelken dated June 28, 1980
4. Mass		Actual mass of structure and attached equipment, components and piping	PGE 1020
B. Damping			
Global (In-plane)			
OBE	2%	FSAR	
SSE	5%	FSAR	
Local (Out-of-plane)			
Uncracked Cond.	2%	Interpretation of FSAR	
Cracked Cond.	5%	Interpretation of FSAR	

Table 6 (continued)

Analysis Assumptions Affecting Load Demand of Out-of-Plane Bending

Parameter	Value Used	Basis	Documentation
C. Seismic Loading	Average of floor spectra at top and bottom	Analysis	Supplement No. 4 to LER 79-15
D. Moment and Shears			
Bending moment	Values obtained from beam or plate theory	Principle of Mechanics	N.A.
Transverse shear	Values obtained from beam or plate theory and interstory drift	Principle of Mechanics	N.A.
E. Interstory Drift			
1) Interstory Displacement	STARDYNE analyses with post processing	Amplified interstory displacements to account for potential linear behavior were obtained by analyses	See Section 6.2.1.5 of this document
2) Steel stress up to yield	fy = 50 ksi	Upper bound yield stress	Letter from Broehl to Engelken dated June 28, 1980
3) High range of material properties at top and bottom of walls	See Table 1	Test results, engineering judgement	Letter from Broehl to Engelken dated June 28, 1980
F. Horizontal Floor Response Spectra Generation	Raw spectra widened 10% on high frequency side and 41% on low frequency side	Extensive widening on low side to conservatively account for potential nonlinear structural response	PGE-1020

Table 7

Assumptions Affecting Evaluation of Capacity for Out-of-Plane Bending

Parameter	Value Used	Basis	Documentation
A. Bending			
1. Uncracked capacity (based on concrete)	See Table 1	Test results, engineering judgement	Supplement No. 4 to LER 79-15
2. Cracked capacity (based on steel)	fy = 40 ksi	Lower bound yield stress	Supplement No. 4 to LER 79-15
B. Shear			
1. Collar Joint Shear	See Tables 2, 3	Test results	Supplement No. 4 to LER 79-15
2. Principal Stress	See Table 5	Test results	Letter from Broehl to Engelken dated June 28, 1980
C. Membrane Compression & Bearing	See Tables 2, 3, 4 and 5	UBC	Letter from Broehl to Engelken dated June 28, 1980

ATTACHMENT 2

Description and Justification of Tornado Load Criteria for the Trojan Nuclear Plant

1. Design Basis

Tornado load criteria as now included in the NRC Standard Review Plan and parameters specified for design guidance in NRC Regulatory Guide 1.76 had not yet been established during the period in which Trojan plant structures were designed. For the Trojan plant, criteria to evaluate safety-related structures for potential effects of tornadoes were developed pursuant to AEC General Design Criterion 2. FSAR Section 2.3 documented the greatest historical wind storms which have been classified as tornadoes, occurring within a 60-mile radius of the plant site, and describes the very low probability of a tornado striking a particular area in any year within this 60-mile radius. Although the probability of occurrence of a major tornado in the vicinity of Trojan was considered to be extremely low, plant safety-related structures were evaluated to determine their threshold capacity to withstand tornado loads, and at appropriate locations new shield walls were added and existing walls were strengthened during construction of the plant.

As described in FSAR Section 3.3, all plant structures containing systems needed to achieve and maintain safe shutdown were determined to be capable of resisting 200-mph tornado loads, and many of these structures were evaluated to be further capable of resisting 300-mph tornado loads. We consider the 200-mph and 300-mph tornado maximum wind speed criteria to be conservative with respect to the wind speed associated with the maximum tornado that could reasonably be hypothesized for the site, particularly considering the low probability of tornado wind generation in the Pacific Northwest coupled with the inherent protection afforded by the local rugged terrain.

The bases upon which plant protection was provided for the potential effects of tornados was reviewed and accepted by the AEC prior to issuance of the Trojan operating license.

2. Tornado Missiles and Masonry Shield Wall Design

Postulated tornado generated missiles which were considered in the evaluation of Trojan plant safety-related structures, as described in FSAR Section 3.3, are:

Missile	Impact Velocity (mph) 200 mph Tornado	300 mph Tornado
◦ Wood plank 4" x 12" x 12', wt = 108 lbs	200	300
◦ 3" Ø Sch 40 pipe 10', wt = 76 lbs	75	100
◦ Car, 4000 lbs	40	50

Estimated missile penetration depths were calculated by use of the conventional modified Petry formula as given in FSAR Section 3.5. Masonry walls designed as missile shield walls were considered to be equivalent concrete sections for the purpose of estimating the missile penetration coefficient, K, used in the modified Petry formula. A value of K corresponding to a compressive strength for masonry of 1500 psi was used, although the compressive strength of masonry is at least 2700 psi and that of the cell fill grout, which constitutes approximately 50% of the total volume, is at least 5000 psi. Based on FSAR missile penetration equation 3.5-9 which is derived using the modified Petry equation, the thickest section predicted to be just penetrated due to the governing missile impact for the majority of the Complex (4" x 12" plank at 200 mph) is about 6-1/2 inches for an equivalent concrete compressive strength of 2700 psi, and about 4-1/2 inches for a concrete compressive strength of 5000 psi. The minimum missile shield wall provided to protect those areas in the plant where redundant trains are located in the same vicinity is a 16-inch thick double wythe masonry wall, except as described in Section 3 of this attachment. Such a wall could be considered as providing 16 inches of masonry of equivalent concrete compressive strength of at least 2700 psi, or two columns of cell fill concrete each about 5-1/2 inches thick with a compressive strength of at least 5000 psi. In either case, adequate missile penetration protection is provided based on FSAR equation 3.5-9.

Use of the modified Petry formula, which is based on empirical relationships developed from concrete wall tests, was justifiable since the masonry walls are constructed of concrete blocks with fully grouted cells, and reinforcing steel ratios similar to what would be used in reinforced concrete walls of equal thicknesses. No data specific to masonry were available to further refine estimates of such coefficients.

3. Tornado Missile Protection for Plant Safety-Related Systems

A review was recently conducted of tornado missile barriers which exist to protect plant safety-related systems needed to achieve and maintain safe shutdown of the plant. All such systems are protected from direct exposure to tornado missiles and, in general, are diverse such that postulated tornado missiles could not simultaneously affect redundant trains even if missile penetration of existing barriers were hypothesized.

Isolated areas where redundant trains of safe shutdown equipment are located in the same proximity and are protected by essentially the same missile barriers were specifically evaluated. Each of these areas is protected by a minimum of a 16-inch thick double-wythe masonry wall missile barrier (or the equivalent) with two exceptions. One exception is a small portion of the east wall of the Fuel Building near column line 61 between elevations 45 feet and 61 feet where tornado missile protection less than described above is provided for Component Cooling Water System (CCW) makeup pumps, two 4-inch Service Water System (SWS) pipes (emergency water supply to the spent fuel pool and CCW makeup pumps), and small diameter piping. The SWS pipes are located about 30 feet from the east wall of the Fuel Building and the CCW makeup pumps are located about 37 feet from this wall. A small window about 4 feet wide by 12 feet high exists where a 6-inch thick precast concrete panel on the exterior of the east wall of the Fuel Building provides the only barrier to tornado missiles.

The other exception is in the same area of the Fuel Building near column line 61 between elevations 61 feet and 77 feet. At this location, protection for the two 4-inch service water system pipes (described above) is provided by a 76-1/2 inch deep structural steel girder which has a 3/4-inch thick web, and the 14-inch thick double wythe masonry walls on the north and east sides of the spent fuel pool heat exhanger room.

The joint probability of the occurrence of a large tornado, the generation of high velocity missiles and a strike in the most penetrating orientation at these isolated areas and penetration of the existing barriers is believed to be extremely remote.

ATTACHMENT 3

Discussion of NRC Suggested Items

for Consideration in a Testing Program

Item 1

Wall frequency calculation, dynamic behavior, damping, stiffness, etc.

Response:

The overall concern is the out-of-plane response of the walls to earthquake induced loads. In evaluating this behavior, the response must be within certain stress limits. The walls are considered adequate if these stress criteria are met. Frequency, damping characteristics and floor motion are required for the evaluation of a wall's out-of-plane response. The stiffness enters only indirectly through the frequency calculation, or in a limited way, through the influence of floor-to-floor displacements.

Wall frequency depends on spans (vertical and horizontal), boundary conditions at the four edges, mass, and flexural rigidity. Spans and mass are known without any uncertainties. Boundary conditions can be assumed at their upper and lower bound values. Upper and lower bounds of flexural rigidity can also be defined with a high level of confidence. The transition from an uncracked to a cracked condition is uncertain, but the uncertainty is fully accounted for by the assumption that cracking is associated with a modulus of rupture with the widely spaced upper and lower limits of 1000 psi and 450 psi.

We conclude that our evaluations have included assumptions which conservatively bound possible frequencies. With respect to this parameter, tests are not warranted.

Dynamic behavior depends on assumed input motions, frequencies and assumed damping. The input motions involve the response of the entire structure, a response which has been conservatively evaluated. The input motions are nearly independent of the out-of-plane movement of the wall and there is nothing that can be tested which will provide any information on the input motion to the wall other than a full scale test of the Complex. As noted above, the frequencies have been conservatively bounded and no further conservatism is to be achieved through tests. The assumed wall damping ratios (2% uncracked and 5% cracked) are conservative and consistent with the reasoning behind the choice of these same factors for the desired overall response of structures to OBE and SSE conditions.

Large scale wall models subjected to dynamic loading would be required to demonstrate the conservatism in the described evaluation of the dynamic behavior. In the absence of assurance that the results of such tests could be used to relax the present design conservatisms, the tests are not justified.

Item 2

Anchor bolts in composite, double and single wythe masonry under in-plane and out-of-plane loading.

Response:

The application of expansion anchor bolts at Trojan is similar to that at other nuclear plants. The criteria used in the design of supports utilizing expansion anchor bolts, given in PGE's response to IE Bulletin 79-02, are based on the manufacturer's ultimate capacity for installation in concrete having a compressive strength of 3500 psi, divided by a safety factor of 5 and multiplied by a reduction factor of 0.60 for a masonry application. In addition, many of the supports formerly anchor-bolted to masonry walls have either been through-bolted or removed from the wall.

There are currently test programs being run by various organizations to determine the detailed characteristics of expansion anchor bolts installed in masonry. These tests are being followed closely, and any important implications will be addressed. In view of the criteria set forth in PGE's response to IE Bulletin 79-02, and the through-bolting that has been done, it seems reasonable to await the results of testing programs currently underway before deciding if additional tests are warranted.

Item 3

Local load capacity, eg, tornado missiles, block pullout from bolted connection, including anchor and through-bolt configurations, etc.

Response:

This response is directed to all parts of item 3 except tornado missiles which are addressed in Attachment 2.

In the modifications made to date to meet criteria pursuant to the resolution of LER 79-15, the major loads on the block walls have either been through-bolted or the pipe supports have been removed from the wall. Therefore, the remaining loads do not have the capability to cause a local failure. To quantify this, the possiblity of a local failure was first considered at the bolt location. Since the allowable capacities of expansion anchors have a minimum safety factor of 8.33 (5/0.6) for block walls on the manufacturer's ultimate load corresponding to 3500 psi concrete, a local failure is very unlikely.

Moving out from the zone immediately around the bolt, the next possibility of local failure was considered at the boundary of the block. To quantify this, four 3/4" diameter expansion anchor bolts are considered attached to one block. Since, in actuality, the bolts are spaced at 10 diameters, having four bolts in one block is unlikely, but possible. In virtually all supports, the anchor bolts are provided to resist a combination of shear, tension and moments. The most critical loading to cause a failure at the boundary of the block is a direct tension on all bolts. Even though this is a highly unlikely combined loading condition for a support requiring four 3/4" expansion anchors, this loading condition was considered as a bounding case example. This loading would put direct tension on the block of 7800 pounds. If this load was resisted by only shear in the concrete (f'_c = 5000 psi) in the vertical cells of the block, the shear stress in an 8-inch wythe would be only 7800/128 = 60.9 psi. The ACI code recommends an ultimate shear of $2\phi\sqrt{f'_c}$ = 120 psi for unreinforced concrete. This indicates that even for the conservative loading case considered and neglecting the shear capacity of the bed joint mortar and the shear capacity of the head joint grout on the area of the open end of the A-type block, a local failure at the block boundary is very unlikely. If a more realistic loading condition is considered, which would be some combination of shear, tension and moments, the load resistance capacity would be even larger.

The local capacity of a masonry wall at a through-bolted support could be governed by bearing under the base plate, punching shear due to a tensile or compressive load, or bearing against the bolt due to a shear load. At through-bolted supports, the average bearing stress is limited to 0.3 f'_c which is a conservative value. The punching shear is evaluated using a shear stress limit of $2\phi\sqrt{f'_c}$ on the concrete in the vertical cells and 60 psi as the shear stress on the area at the open end of the A-type blocks. The shear stress in the mortar is neglected. With the allowable bearing stresses at such a conservative value, the base plates are of such a size that a punching shear limit will not control. The allowable stress capacity of the wall will be controlled by bending, collar joint shear or some other global parameter. The shear capacities of the through-bolts are evaluated according to the UBC code. This, therefore, is a conservative design procedure which will preclude local failure of the through-bolted configuration.

Relative to the criteria described above, there is considerable excess capacity with respect to local block pullout which implies that some other mode of behavior controls the capacity of the walls. Therefore, testing the local block pullout mode of behavior is unwarranted.

Item 4

Confirmation that smearing local loads over 6t is justified and reasonably conservative.

Response:

The response to Item 3 demonstrates that a local failure is very unlikely. If a local failure does not occur, the next level of investigation is to consider the bending capacity of the wall.

The bending in the wall depends on the supporting conditions of the wall, aspect ratio, location of the load and the area over which the load is distributed. As a simplification in the bending analysis of plates and slabs (eg, walls and floors) subjected to concentrated loads, it is common practice to use an equivalent beam of width 6t and span equal to the shortest span of the wall.

The range of parameters over which this analysis procedure is conservative when considering only elastic responses can be determined by considering the bending moment under a concentrated load on a flat slab or elastic plate. To obtain a limiting case, an elastic plate simply supported on two opposite sides and extending to infinity in the other two directions is considered. For a concentrated load spread over a circular area, centered at midspan, the diameter of which is 5% of the span, the maximum bending moment is 0.381P for a Poisson's ratio of 0.17 and a concentrated load P. By equating the moment per foot of beam width to maximum moment in the plate, the span for which the beam and plate bending models are equivalent can be determined and are as follows:

Thickness, inches	4	6	8	12	14	16
Span, feet	3.1	4.6	6.1	9.1	10.7	12.2

If the span of the actual wall is greater than that shown in the above table, the beam model produces a conservative bending moment, ie, greater than the moment from the plate solution. If the concentrated load is located at the quarter point of the span of the plate, the following results are obtained:

Thickness, inches	4	6	8	12	14	16
Span, feet	3.7	5.6	7.5	11.2	13.7	14.9

The bending in the wall depends on the supporting conditions of the wall, aspect ratio, location of the load and the area over which the load is distributed. As a simplification in the bending analysis of plates and slabs (eg, walls and floors) subjected to concentrated loads, it is common practice to use an equivalent beam of width 6t and span equal to the shortest span of the wall.

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Thickness, inches	4	6	8	12	14	16
Span, feet	3.7	5.6	7.5	11.2	13.7	14.9

For the majority of the walls in the plant, the effective span is approximately 14 feet.

In addition to this justification of 6t for the block walls, there are additional items which make the approach conservative. The solution has been based on a condition which has simply supported edges, ie, moment resistance at the supports is considered to be zero. In the actual case, there is some rotational resistance and, therefore, some bending moment at the supports which tends to reduce the spans shown above. The two-way action resulting from the presence of any cross walls has also been neglected. In all cases, except for cantilevers, the walls are supported on at least three sides and most walls are supported on all four sides. This will also cause a reduction in the spans shown above or increase the degree of conservatism in the beam strip analysis procedure. The solution also depends on the base plate covering a circular area, the diameter of which is 5% of the shortest span. Actual base plates are typically larger. For the major concentrated loads on the wall, the base plates cover a larger area which adds to the conservatism. In all cases, however, the moment per unit length is not taken less than that for two-way plate action including cantilever walls.

Beyond these conservatisms is the concept of using peak elastic stresses for the design of a ductile system. Reinforced concrete slabs just do not behave in this manner. As the load increases, there is some redistribution which takes place and produces a condition less severe than predicted by an elastic solution. The redistribution of the bending moment under the concentrated load is possible because the moment from the elastic solution reduces so rapidly. For example, if the concentrated load covers a circular area equal to 5% of the span of the plate described above, the average bending moment over the center 20% of the span is only 0.253P compared to a maximum moment of 0.381P. Since the walls have low reinforcing steel ratios, which results in a ductile system, the redistribution capability provides additional conservatism.

The elastic analysis shows that using a beam of width 6t is conservative for single and double wythe walls which are the only walls that are affected significantly by concentrated loads. If actual tests to failure were conducted, the ductile behavior of walls will be activated resulting in a more favorable distribution of moment in the vicinity of the load, thereby showing the 6t as more conservative than shown by the elastic solution. Therefore, testing is unwarranted.

Item 5

Local bearing stresses for bearing normal to wall.

Response:

The nominal bearing stress under a base plate has been limited 0.30 f_c' not to exceed 1200 psi, which is consistent with the intent of UBC. The UBC does not specify whether this allowable bearing stress is for a load normal to the plane of the wall or normal to the edge of the wall, such as a vertical load on the top edge of a wall. It is our judgement that this allowable stress is applicable in either direction. Since the confining effect of the surrounding material is better when the load is applied normal to the plane of the wall than when the load is applied on the edge of the wall, the local bearing capacity of the wall should be greater for the former loading conditions than for the latter. It is virtually impossible for the wall to fail in bearing due to loads normal to the plane of the wall. Some other mode of behavior will control.

The walls in the plant have been subjected to loads normal to the plane of the wall during some of the wall modifications. Some walls were strengthened to resist shear stress due to out-of-plane loads by through-bolting. These through-bolts were tensioned until the nominal bearing stress under the base plates was approximately 600 psi. During the tensioning process, Bechtel engineers observed the behavior of block, looking for signs of distress. None were observed. Instrumented tests to failure under loads normal to the wall could be done to determine the ultimate bearing capacity, but such tests seem unnecessary considering the performance of the base plates to date and considering that bearing stress is not a controlling parameter in the response of the wall to seismic, thermal or other nonprestress loads.