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May 26, 1982

Docket Nos. 50-277
50-278

IE Bulletin 80-11

Mr. John F. Stolz, Chief
Operating Reactors Branch #4
Division of Licensing
US Nuclear Regulatory Commission
Washington, DC 20555

Dear Mr. Stolz:

Your letter of March 10, 1982, requested additional information concerning our previous submittals on Bulletin 80-11, Masonry Wall Design, for Peach Bottom Atomic Power Station. The attached Appendix A provides a complete response to the items in your letter. In addition, in the course of our review we determined that several changes and additions should be made to our original submittal. These changes are described in Appendix A and revised pages are included in Appendix B.

If you have any questions or require additional information, please don't hesitate to call.

Very truly yours,

Jw Gallagher

Attachments

cc: US Nuclear Regulatory Commission
Office of Inspection & Enforcement
Division of Reactor Operation Inspection
Washington, DC 20555

C. J. Cowgill - Site Inspector

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APPENDIX A

Response to NRC Questions

on

IE Bulletin 80-11

for

Peach Bottom Atomic Power Station

Units 2 and 3

Docket Nos. 50-277 and 50-278

Question No. 1

With reference to Section 3, Appendix I, part 1 [4], justify the basis for load combinations 7, 8 and 9.

Response

Supplement No. 2 of the Peach Bottom FSAR was used in the selection of load combination 8 which considers design accident conditions for the masonry walls. The load combination specified in the Supplement was not meant for masonry walls; therefore, certain modifications to the load combination of the Supplement were made to arrive at load combination 8, which is now compatible with the nature of construction and function of the masonry walls. Load combination 8 ($1.05D+1.0P'+1.0P$) considers the combined effect of jet impingement (P) and pressurization (P') due to high energy line break. The term $1.0P$ was inadvertently omitted from the load combination which should, therefore read $1.05D+1.0P'+1.0P$. Section 3.0 and Tables 1 and 2 of Appendix I, Part 1 of the Report are revised to correct this error.

As an additional check, two more load combinations (i.e. combinations 7 and 9) were used under design accident conditions. In making the combinations judgement was used in following the event history that constitutes a design accident. Load combination 7 ($1.05D+1.25P$) recognizes the impulsive nature of the jet impingement load by using a 25% dynamic load factor. Since the peak pressure in a compartment due to high energy line break does not occur at the instant this impact takes place, the pressurization load is not included in this combination. Load combination 9 ($1.05D+1.0Ta$) considers the accident thermal stresses induced in the walls. Since these are long term loads - i.e. sufficient time must pass before the final temperature gradients are established across the thickness of the wall - they are not combined with loads of relatively short duration like those associated with jet impingement, pressurization due to high energy line break and seismic conditions.

Question No. 2

With reference to Table 1, Appendix I, part 1 [4], justify the proposed 30% increase in allowable stress for load combinations including OBE loads, for which no increase is allowed in the SEB criteria [5].

Response:

For the Peach Bottom masonry wall re-evaluation no stress increase has been allowed for the load combination that includes OBE effects alone. The 30% increase in allowable stress is allowed only when thermal effects are considered in addition to the OBE loads.

As mentioned in Sec. 5.1, Appendix I, Part 2 (Ref. 4), the 30%

stress increase for load combinations containing normal operating thermal effects or displacement limited loads has been typically accepted in the industry for reinforced concrete and is considered reasonable for masonry. The 30% increase when combining the OBE and thermal effects is also consistent with the structural acceptance criteria contained in Section 3.8.4 of Standard Review Plan.

Question No. 3

With reference to Table 1, Appendix I, part 1 (Ref. 4), justify the increase factors of 1.67 applied to allowable stresses in shear, bond, tension normal to the bed joint, and tension parallel to the bed joint. The SEB criteria [5] allow increase factors of only 1.5 for tension parallel to the bed joint and shear in the reinforcement and 1.3 for tension normal to the bed joint and masonry shear.

Response

All blockwalls at the Peach Bottom plant contain steel tension reinforcement which take all tension normal or parallel to the bed joint. Therefore the assumed allowable masonry tension stress normal or parallel to the bed joint was zero (psi) for the re-evaluation of blockwalls. Since the masonry tension strength at the bed joint was assumed to be zero the load factor stated in the question was not used in the reanalysis of the blockwalls. Further, the factor 1.67 has also not been used for shear or bond calculations for the masonry wall re-evaluation.

Code allowable stresses for masonry shear, bond and tension normal or parallel to the bed joint were increased by a factor of 1.67 for load combinations involving abnormal and/or extreme environmental conditions which are credible but highly improbable. Since the code allowable stresses (Reference 7, Chapter 10.1 of the commentary) are generally associated with a safety factor of 3, the 1.67 increase provides a factor of safety against failure of 1.8 (3 divided by 1.67). The factor of safety of 1.8 is conservative and allows sufficient margin for abnormal and/or extreme conditions.

Question No. 4

With reference to Section 5.2.4, Appendix I, Part 1 [4], justify the increase factor of 1.67 proposed for allowable in-plane strains.

Response

As mentioned in Section 5.1, Appendix I, Part 2 (Ref. 4), the allowable strain for a confined wall was based on the equivalent

compression strut model discussed in Ref. 9 and modified by a factor of safety of 3.0 against crushing. Using a similar justification as in the response to Question 3, the increase factor of 1.67 proposed for allowable in-plane strains provides a factor of safety against failure of 1.8 (3 divided by 1.67).

Question No. 5

Provide justification for the two approaches proposed in parts 1 and 2 of Section 5.4, Appendix I [4] for determining modulus of rupture.

Response

For the Peach Bottom masonry wall re-evaluation, only the first approach, mentioned in sub-subsection 'a' of Sec. 5.4, Appendix I, Part 1 (Ref. 4) has been used for determining the modulus of rupture.

As discussed in Sec. 5.4 of Appendix I, Part 2, the modulus of rupture of concrete, grout and mortar is assumed to vary by 20%. Hence, a lower bound modulus of rupture is determined by applying a reduction factor of 0.8 to the theoretical concrete modulus of rupture of $7.5 f'_c$ or to the modulus of rupture determined by testing samples taken from the as-built structure.

The other approach, mentioned in sub-section 'b' of Sec. 5.4, was not used in the masonry wall re-evaluation. However, as discussed in Sec. 5.4, Appendix I, Part 2 (Ref. 4) for masonry the modulus of rupture is approximated by increasing the code allowable flexural tensile stress by a factor of safety of 3 and then applying the 20% reduction to arrive at a lower bound value. ($0.8 \times 3 F_t = 2.4 F_t$, where F_t is the code allowable tensile stress).

Question No. 6:

Justify the use of alternative acceptance criteria of Section 6.0, Appendix I, part 1, since it is the NRC's position that energy balance techniques and the arching theory should not be used in the absence of conclusive evidence of their applicability to masonry structures in nuclear power plants.

Response:

As mentioned in the response to question No. 14, 5 walls out of a total of 86 were qualified under the alternative acceptance criteria using the energy balance technique: the arching theory was not used to qualify any of the walls. It should be noted that all category I masonry walls at Peach Bottom are non-bearing and are not part of the lateral load

resisting system. In addition, walls qualified under the energy balance technique were capable of developing ductile in-elastic flexural out-of-plane deformations without any shear or anchorage failure.

The justification for the alternative acceptance criteria, complete with the mathematical formulation and references, is contained in the next 7 pages. The response to question No. 14 contains more details on the application of alternative acceptance criteria to the walls.

Response to Question No. 6

Justification of the Energy Balance Technique

Reinforced masonry walls (a) that are not relied upon to provide strength of the structure as a whole, i.e., not acting as shear or bearing walls, and (b) that have sufficient capabilities to preclude brittle shear and anchorage failures, can undergo large ductile out-of-plane inelastic flexural deformations when subjected to the out-of-plane seismic inertia loading. Tests as reported in Reference 10 have indicated that regularly reinforced masonry walls are capable of developing ductile inelastic flexural deformations well beyond the cracking and yielding limits of the masonry wall subjected to out-of-plane uniform loadings. Tests in Reference 11 indicate that ductilities (defined as the ratio of maximum displacement of a single-degree-of-freedom elasto-plastic system to its yielding displacement) in excess of 25 are achievable when flexure is the dominant action. Other tests such as those reported in Reference 12 show that even when compression failures occur, ductilities in excess of 5 can be achieved. A more recent test program carried out by the Task Committee on Slender Walls of the Structural Engineer Association of Southern California (SEAOSC) in 1981 (Reference 13) has clearly indicated the capability of reinforced masonry walls in developing large ductile inelastic flexural deformations when subjected to out-of plane uniform loads.

It is, therefore, reasonable to allow inelastic flexural deformation with ductility of up to 5 for walls subjected to out-of-plane seismic loadings, as long as brittle failures are precluded and the safety systems on or adjacent to the wall are not jeopardized. For added conservatism, the maximum ductility allowed in the qualification of the five masonry walls at Peach Bottom was limited to only 3: the actual ductilities of the walls were well below this limit.

The "energy balance technique" utilized in analyzing the out-of-plane response of reinforced masonry walls subjected to seismic inertia load is, in essence, analogous to Newmark's inelastic seismic response spectrum technique (Reference 15) or Blume's reserve energy technique (Reference 14). This can be demonstrated by the seismic response of a single-degree-of-freedom (SDOF) system as shown in Figure 1(a). If this SDOF system is a linear elastic system, the maximum absolute acceleration response of the system subjected to a seismic input motion can be determined from the input motion acceleration response spectral value S_a at the system frequency ω and modal damping ratio β , i.e., $S_a(\omega, \beta)$. Since the damping ratio β is usually small, the maximum elastic displacement (Δ_e) and the maximum elastic force (F_e) induced by the seismic response can be determined by:

$$\Delta_e = \frac{m S_a(\omega, \beta)}{k} = \frac{S_a(\omega, \beta)}{\omega^2} \quad (1)$$

$$F_e = k \Delta_e \quad (2)$$

and the maximum elastic strain energy (E_e) stored in the system, as shown in Figure 1(b), is:

$$E_e = \frac{1}{2} F_e \Delta_e = \frac{1}{2} k \Delta_e^2 \quad (3)$$

If the elastic spring of the same SDOF system is an elasto-(perfectly)-plastic spring having a yielding force at F_y and yielding displacement at Δ_y the maximum response of the system subjected to the same seismic input motion can be determined approximately based on the "energy balance technique" which assumes that the maximum energy (E_p) attained in the elasto-plastic spring is equal to the maximum elastic energy attained as if the system is elastic, i.e., $E_p = E_e$. Referring to Figure 1(c), this energy balance can be expressed as:

$$E_e = \frac{1}{2} k \Delta_y^2 + k \Delta_y (\Delta - \Delta_y) = E_p ; \quad \Delta \geq \Delta_y \quad (4)$$

where Δ is the maximum displacement response of the elasto-plastic SDOF system, which can be solved from Equation (4).

Define the displacement ductility ratio μ to be:

$$\mu = \frac{\Delta}{\Delta_y} ; \quad \Delta = \mu \Delta_y \quad (5)$$

Using the Equation (5), Equation (4) can be rewritten as:

$$\Delta_e = \Delta_y \sqrt{2\mu-1} \quad , \quad \mu \geq 1 \quad (6)$$

or

$$\mu = \frac{1}{2} \left[\left(\frac{\Delta_e}{\Delta_y} \right)^2 + 1 \right] \quad , \quad \Delta_e \geq \Delta_y \quad (7)$$

The inelastic response acceleration can be defined as follows:

$$S_a^i = \frac{F_y}{m} = \frac{k\Delta_y}{m} \quad (8)$$

Using Equation (8), Equation (6), can be rewritten as:

$$\frac{S_a^i}{S_a} = \frac{1}{\sqrt{2\mu-1}} \quad ; \quad \mu \geq 1 \quad (9)$$

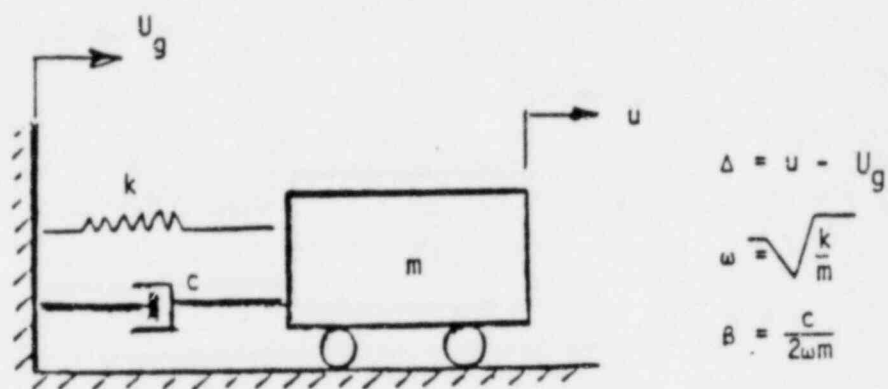
which is the expression of the Newmark inelastic acceleration response spectral value (Reference 15), and the ratio of inelastic to elastic displacement, from Equations (5) and (6),

$$\frac{\Delta}{\Delta_e} = \frac{\mu}{\sqrt{2\mu-1}} \quad ; \quad \mu \geq 1 \quad (10)$$

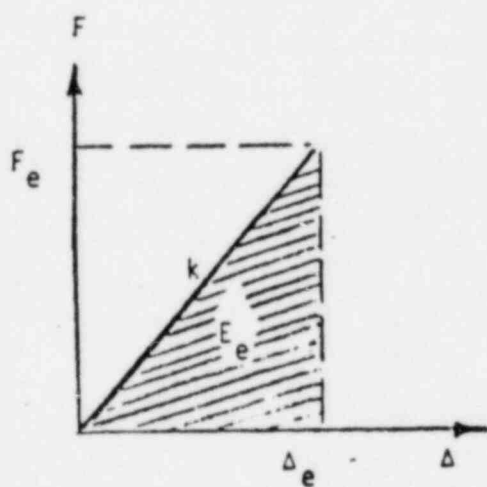
The application of the energy balance technique to the analysis of out-of-plane seismic response of reinforced masonry walls is illustrated in Figure 2. The application involves (a) the determination of the load (uniform out-of-plane seismic inertia load) vs. deflection (mid-span deflection) relationship for the

wall panel; (b) the calculation of the maximum out-of-plane inelastic displacement ductility of the wall based on the energy balance technique and the applicable floor response spectrum curve; (c) checking allowable ductilities, shear stresses, and anchorage; and (d) using a factor of 2 on the calculated inelastic displacement to check safety systems on or adjacent to the wall panel.

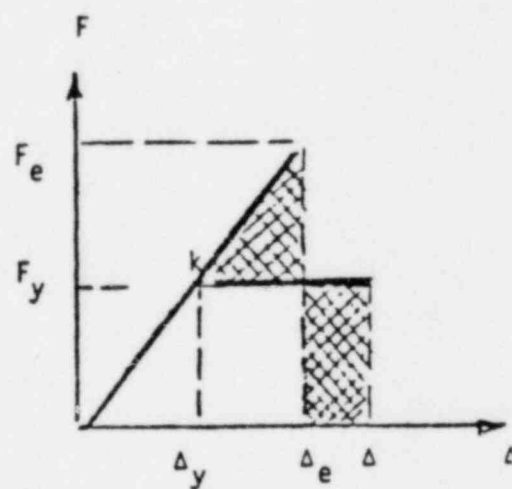
In summary, the energy balance technique for the analysis of out-of-plane seismic response of reinforced masonry walls is based upon the well-known inelastic analysis technique used in the industry for ductile structures. In application, careful check has been made to ensure that brittle failures are precluded and conservative allowable ductilities are used in the evaluation of the walls. Therefore, the application of the technique is justifiable.



(a) SDOF Linear Elastic System

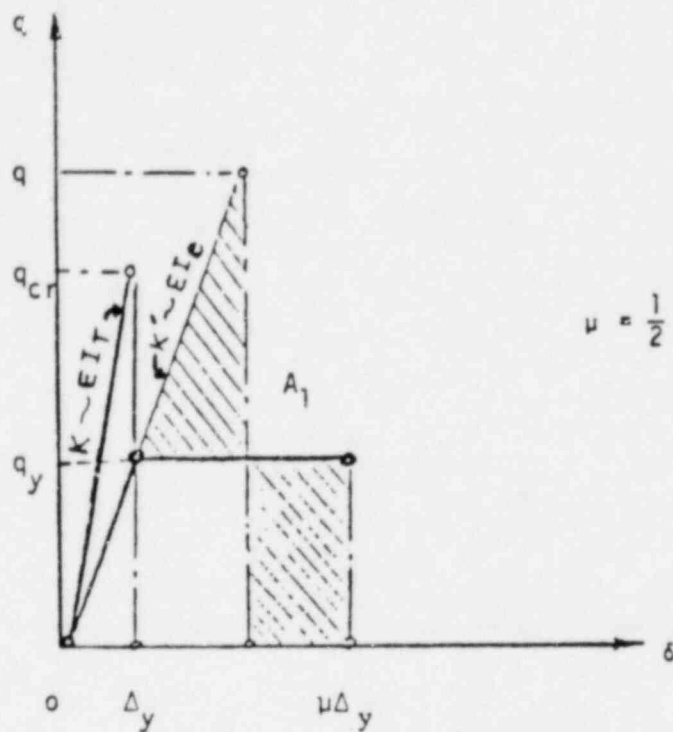


(b) Elastic Energy



(c) Energy Balance

Figure 1 Energy Balance Technique

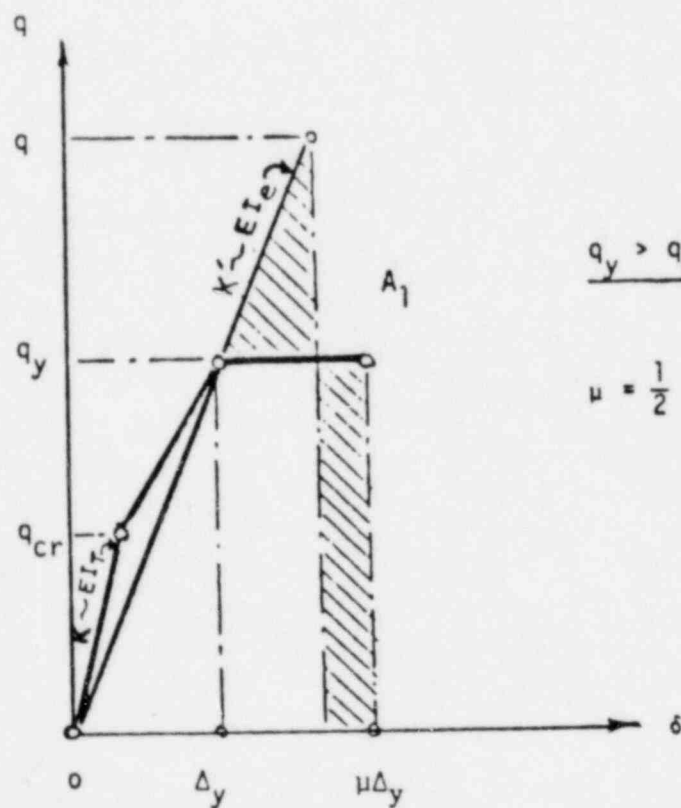
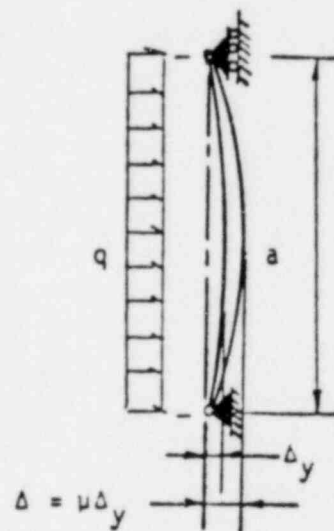


q_y = load at yielding of re-bar

q_{cr} = load at cracking of wall

$$\frac{q_y \leq q_{cr}}{}$$

$$\mu = \frac{1}{2} \left[1 + \left(\frac{q}{q_y} \right)^2 \right] ;$$



$$\frac{q_y > q_{cr}}{}$$

$$\mu = \frac{1}{2} \left[1 + \left(\frac{q}{q_y} \right)^2 \right] ;$$

Figure 2 Application of Energy Balance Technique to Seismic Response of Walls.

Question No. 7

With reference to Section 7.1.2, Appendix I, part 1 [4], provide sample calculations to indicate how the effects of higher modes of vibration are accounted for.

Question No. 8

With reference to Section 7.1.4, Appendix I, part 1 [4], justify the use of average floor acceleration instead of the envelope of the floor response spectra.

Response:

The combined response to Question No. 7 and 8, with sample calculations, is given in the next 11 sheets.

Response to Question No. 7 and 8:

The evaluations herein demonstrate that: (1) The use of the average floor acceleration response spectra instead of the envelope of the floor response spectra for calculating the response of the wall panel is appropriate (response to question no. 8), and (2) The effects of higher modes of vibration are accounted for when uniform inertia load, based on the averaged floor spectral acceleration for the fundamental mode, is used in calculating the maximum seismic response of the wall (response to question no. 7).

For the purpose of this evaluation, the seismic response of a simply-supported, uniform beam simulating a strip of the wall panel with unit width is considered, as shown in Figure 1.

(1) Use of Average Spectra

The equation of motion of an undamped, simply-supported beam can be written in terms of the total displacement with respect to some fixed reference axis as:

$$m \frac{\partial^2 u}{\partial t^2} + EI \frac{\partial^4 u}{\partial x^4} = 0 \quad (1)$$

Where M and EI are the mass density and flexural rigidity of the beam. Denote the seismic excitations at the ends of the beam as U_a and U_b . Then the total displacement $u(x, t)$ can be expressed in terms of the two seismic motions and the relative displacement

to the seismic motions as:

$$u(x,t) = (x/L) U_D + (1 - x/L) U_A + r(x,t) \quad (2)$$

where L is the length of the beam. The relation expressed by the above equation is shown in Figure 2. The relative displacement $r(x, t)$ needs to satisfy the following simply-supported conditions:

$$r(0,t) = r(L,t) = 0 \quad (3)$$

$$\left. \frac{\partial^2 r}{\partial x^2} \right|_{x=0} = \left. \frac{\partial^2 r}{\partial x^2} \right|_{x=L} = 0 \quad (4)$$

Substituting Equation 2 into Equation 1, the equation of motion in terms of relative displacement $r(x, t)$ can be expressed as:

$$m \frac{\partial^2 r}{\partial t^2} + EI \frac{\partial^4 r}{\partial x^4} = -m(x/L) \ddot{U}_D - m(1 - x/L) \ddot{U}_A \quad (5)$$

The eigen-function solutions for the homogeneous equation associated with Equation 5 that satisfy the boundary conditions specified by Equations 3 and 4 are:

$$\sin \frac{n\pi x}{L}, \quad n = 1, 2, 3, \dots,$$

and the corresponding frequencies of vibration are:

$$\omega_n = n^2 \pi^2 \sqrt{\frac{EI}{mL^4}} \quad n = 1, 2, 3, \dots \quad (6)$$

So, the solution of Equation 5 can be expressed as:

$$r(x,t) = \sum_{n=1}^{\infty} a_n(t) \sin \frac{n\pi x}{L} \quad (7)$$

Substitute Equation 7 into Equation 5, and multiply the latter by $\sin \frac{n\pi x}{L}$, and then integrate it with respect to x over the full length of the beam. The equation of motion can then be transformed into modal equations of motion as:

$$\ddot{a}_n + \omega_n^2 a_n = r_n \left(\frac{\ddot{u}_a + \ddot{u}_b}{2} \right) \quad n = 1, 3, 5, \dots \quad (8a)$$

and

$$\ddot{a}_n + \omega_n^2 a_n = r_n \left(\frac{\ddot{u}_a - \ddot{u}_b}{2} \right) \quad n = 2, 4, 6, \dots \quad (8b)$$

where r_n = participation factor

$$= \frac{4}{n\pi} \quad (9)$$

If damping in the form of modal damping ratio is included, Equations 8a and 8b become:

$$\ddot{a}_n + 2\xi_n \omega_n \dot{a}_n + \omega_n^2 a_n = r_n \left(\frac{\ddot{u}_a + \ddot{u}_b}{2} \right) \quad n = 1, 3, 5, \dots \quad (10a)$$

and

$$\ddot{a}_n + 2\xi_n \omega_n \dot{a}_n + \omega_n^2 a_n = r_n \left(\frac{\ddot{u}_a - \ddot{u}_b}{2} \right) \quad n = 2, 4, 6, \dots \quad (10b)$$

where ξ_n is the damping ratio of the n^{th} mode.

Equation 10a means that the odd-number modes which are symmetrical about the mid-span of the beam will be excited by the average of the two seismic excitations; while equation 10b means that the even-number modes which are antisymmetrical about the mid-span of the beam will be excited by half of the difference between the two seismic excitations.

Expressing the maximum modal displacement response in equations 10a and 10b in terms of absolute acceleration response spectra gives:

$$|a_n|_{\max} < |\Gamma_n| \left[\frac{S_a(\xi_n, \omega_n)}{\omega_n^2} + \frac{S_b(\xi_n, \omega_n)}{\omega_n^2} \right]$$

$$< \frac{4mL^4}{n^5 \pi^5 EI} \left[\frac{S_a(\xi_n, \omega_n) + S_b(\xi_n, \omega_n)}{2} \right] \quad (11)$$

$$n = 1, 2, 3, \dots$$

This illustrates that the use of the average of two floor acceleration response spectra, instead of the envelope of the floor response spectra, for calculating the modal response of a wall panel is appropriate.

(2) Contribution of Higher Modes

From Equation 11, the relative importance of modes can be evaluated by examining the frequency ratios, modal participation ratios, and maximum modal response ratios for constant acceleration which can be shown as:

$$\omega_1 : \omega_2 : \omega_3 : \dots = 1 : 4 : 9 : \dots \quad (12)$$

$$\Gamma_1 : \Gamma_2 : \Gamma_3 : \dots = 1 : -1/2 : 1/3 : \dots \quad (13)$$

$$\frac{\Gamma_1}{\omega_1^2} : \frac{\Gamma_2}{\omega_2^2} : \frac{\Gamma_3}{\omega_3^2} : \dots = 1 : -\frac{1}{32} : \frac{1}{243} : \dots \quad (14)$$

For an SRSS method of combining maximum response, the contribution of higher modes is clearly negligible.

If for example, the fundamental frequency ω_1 is above 8 Hz, the second frequency is above 32 Hz which is already in the rigid range, i.e., in the range of no amplification. Thus the S_a and S_b values associated with modes other than the fundamental will be the Zero-Period-Acceleration (ZPA) values of the two seismic motions U_a and U_b . Using the absolute sum (ABS) method of combining the modal maximum responses in this case, the contribution of higher modes is not more than 4% of the fundamental mode.

The relative importance of modes can also be evaluated by examining the moment and shear responses in the beam for each mode, as shown in the following:

The moment in the beam due to the n^{th} mode can be evaluated by:

$$M_n(X) = EI \frac{\partial^2}{\partial X^2} \left[a_n \sin \left(\frac{n\pi X}{L} \right) \right] \quad (15)$$

$$< \frac{4mL^2}{n^3 \pi^3} \left[\frac{S_a(\xi_n, \omega_n) + S_b(\xi_n, \omega_n)}{2} \right] \sin \left(\frac{n\pi X}{L} \right)$$

$$n = 1, 2, 3, \dots$$

The moment at the mid-span of the beam is contributed only by the symmetrical modes and can be expressed as follows:

$$M_n\left(\frac{L}{2}\right) < \frac{4mL^2}{n^3 \pi^3} \left[\frac{S_a(\xi_n, \omega_n) + S_b(\xi_n, \omega_n)}{2} \right] \quad (16)$$

$$n = 1, 3, 5, \dots$$

For a constant spectral acceleration, the contribution to the midspan moment of the beam from each mode can be expressed in the following ratio:

$$M_1\left(\frac{L}{2}\right) : M_3\left(\frac{L}{2}\right) : M_5\left(\frac{L}{2}\right) : \dots = 1 : \frac{1}{27} : \frac{1}{125} : \dots \quad (17)$$

Using the SRSS Method of combining modal responses, the contribution of the higher modes to the mid-span moment is less than 1% of that from the fundamental modes. Using the ABS method of combining

modal responses, the contribution of higher modes is less than about 5%.

The shear force in the beam due to the n^{th} mode can be evaluated as:

$$Q_n(X) = EI \frac{\partial^3}{\partial X^3} \left[a_n \sin\left(\frac{n\pi X}{L}\right) \right]$$

$$\leq \frac{4mL}{n^2\pi^2} \left[\frac{S_a(\xi_n, \omega_n) + S_b(\xi_n, \omega_n)}{2} \right] \cos\left(\frac{n\pi X}{L}\right) \quad (18)$$

$$n = 1, 2, 3, 4, \dots$$

The maximum shear occurs at the support of the beam and can be expressed as:

$$Q_n(0) \leq \frac{4mL}{n^2\pi^2} \left[\frac{S_a(\xi_n, \omega_n) + S_b(\xi_n, \omega_n)}{2} \right] \quad (19)$$

$$n = 1, 2, 3, 4, \dots$$

The contribution of the higher modes to the maximum shear at the beam support relative to that of the fundamental mode can be evaluated by comparing the modal effective masses (MEM) associated with the fundamental mode and the higher modes. The modal effective mass of the fundamental mode is

$$MEM_1 = \frac{8mL}{\pi^2} = 0.81 mL \quad (20a)$$

The modal effective mass associated with modes higher than the fundamental mode can be calculated as

$$MEM_1^i = (1 - 0.81)mL = 0.19 mL \quad (20b)$$

The ratio of MEM_1^i to MEM_1 is $0.19/0.81 = 23\%$. That is the contribution of higher modes to the maximum shear is at most 23% of the contribution due to the fundamental mode. This ratio does not take into account the ratio of the spectral acceleration for the fundamental mode to the ZPA value for the higher modes. When the difference in spectral accelerations is accounted for, the contribution of higher modes to the maximum shear would be less than 23%. For example, if the spectral acceleration for the fundamental mode is 1.5 ZPA, then the ratio of higher mode contribution would be $0.19/(0.81 \times 1.5) = 16\%$.

(3) Uniform Inertia Load Approximation

Using the modal responses, the maximum moment and shear of the beam can be calculated. This moment and shear can then be compared to the moment and shear based on a uniform inertia load using the average of the two floor spectral accelerations at the fundamental mode of the beam.

The maximum moment occuring at the mid-span of the beam induced by a uniform load with the following magintude:

$$f(X) = m \left[\frac{S_a (\xi_1, \omega_1) + S_b (\xi_1, \omega_1)}{2} \right] \quad (21)$$

can be expressed as:

$$M^* \left(\frac{L}{2} \right) = \frac{mL^2}{8} \left[\frac{S_a (\xi_1, \omega_1) + S_b (\xi_1, \omega_1)}{2} \right] \quad (22)$$

From Equation 16, the moment at the mid-span of the beam due to the fundamental mode is:

$$M_1 \left(\frac{L}{2} \right) < \frac{4mL^2}{\pi^3} \left[\frac{S_a (\xi_1, \omega_1) + S_b (\xi_1, \omega_1)}{2} \right] \quad (23)$$

The maximum difference between the moments from Equations 22 and 23 is about 3%.

The maximum shear occurring at the support of the beam induced by the uniform load expressed in Equation 21, can be written as:

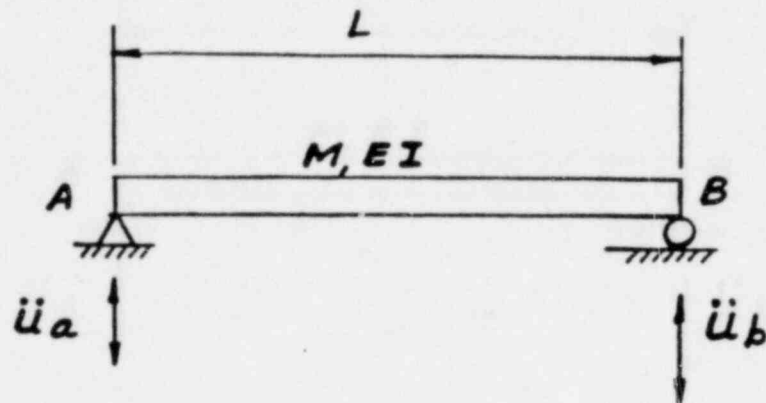
$$Q^*(0) = \frac{mL}{2} \left[\frac{S_a (\xi_1, \omega_1) + S_b (\xi_1, \omega_1)}{2} \right] \quad (24)$$

From Equation 19, the shear at the support of the beam due to the fundamental mode is:

$$Q_1(0) < \frac{4mL}{\pi^2} \left[\frac{S_a(\xi_1, \omega_1) + S_b(\xi_1, \omega_1)}{2} \right] \quad (25)$$

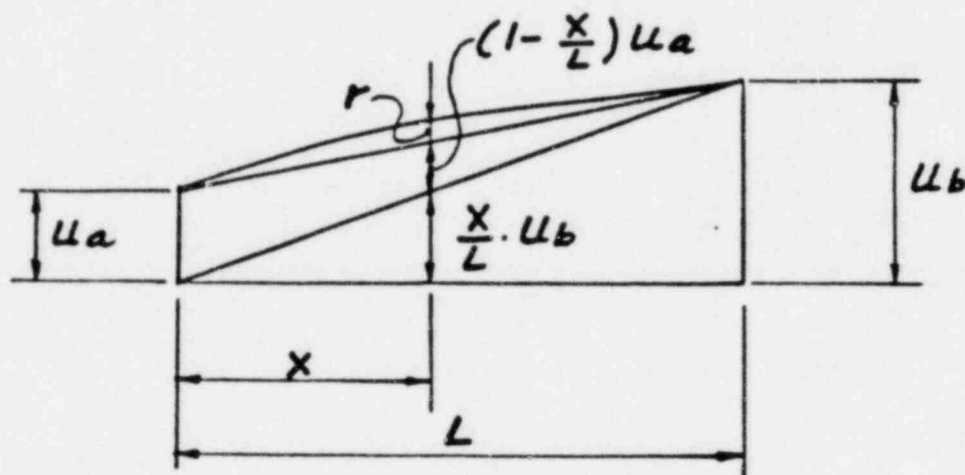
The shear from Equation 24 is greater than the shear from Equation 25 by about 25%. This margin can well cover the contribution to the shear due to the higher mode effect, as discussed previously.

From this comparison, it can be concluded that the effects of higher modes of vibration are accounted for when uniform inertia load based on the averaged floor spectral acceleration for the fundamental mode, is used in calculating the maximum seismic response of the wall.



IDEALIZED SIMPLY-SUPPORTED UNIFORM BEAM

FIGURE No 1



RELATION BETWEEN SEISMIC EXCITATION
AND RELATIVE DISPLACEMENT

FIGURE No 2

Question No. 9:

With reference to Section 7.2, Appendix I, part 2 [4], provide justification and references for the formulae used to determine moments. Indicate how the walls with openings were analyzed.

Response:

The formula referred to in the question give upperbound values for moments in a plate due to concentrated loads. These formulae were not used in the re-evaluation of masonry walls at Peach Bottom. Instead, as mentioned in the response to Question No. 10, computer programs were used to analyze the walls accurately when concentrated loads were present and/or the walls had significant openings.

In any case, the justifications for these formulae are as follows:

a) Justification for $M = 0.4P$

- (1) From Ref. 19 the expressions for moments in a simply supported rectangular plate with a concentrated load are:

$$M_x = \frac{P}{4} \pi \left[\alpha (1 + \nu) + 1 + \gamma_1 \right]$$

$$M_y = \frac{P}{4} \pi \left[\alpha (1 + \nu) + \nu + \gamma_2 \right]$$

Where:

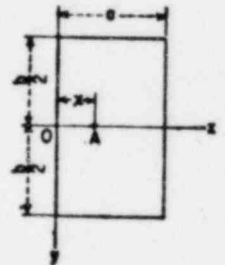
M_x = Moment per unit length in the plate on sections parallel to the y-axis.

M_y = Moment per unit length in the plate on sections parallel to the x-axis.

P = Concentrated load acting on a circular area with diameter $2c$ equal to the thickness of the wall

$$\alpha = \ln \frac{2 \sin \left(\frac{\pi x}{a} \right)}{\pi \frac{c}{a}}, \text{ where 'x' is the distance}$$

of the concentrated load from the simply supported edge and 'a' is the plate dimension in the same direction.



ν = Poissons's ratio, .17

γ_1, γ_2 = numerical factors the magnitude of which depends on the aspect ratio and the position of the load on the x-axis.

Limiting the height-to-thickness ratio to 24 the maximum height for an 8 in. wall is 16 feet. Then, at mid-height:

$$\alpha = \ln \frac{2 \sin \frac{\pi(0.5)}{16 \times 12}}{\frac{\pi 4}{16 \times 12}} = 3.42$$

$$M_x = \frac{P}{4} \pi [3.42 \times 1.17 + 1] \quad \text{taking } \gamma_1 \text{ equal to zero for an infinitely long plate which is conservative for } M_x.$$

$$M_x = \frac{5}{4} (P)$$

$$= .398P < 0.4P \quad (1)$$

The maximum value of γ_2 is .135 for b/a equal to 1.0.

Thus,

$$M_y = \frac{P}{4} [3.42 \times 1.17 + .17 + .135] \\ = .343P < 0.4P \quad (2)$$

- (2) From Ref. [20] the maximum moment in a slab panel at midspan when the concentrated load P acts on the center line is given by:

$$M = \frac{1.16P}{3 + 10 \frac{c}{b}}$$

Taking 'c' equal to zero for conservatism

$$M = \frac{1.16P}{3} = .387P < 0.4P \quad (3)$$

- (3) Sheet No.27 is taken from Ref. 21 and it shows that as β approaches zero, i.e. in the case of plates, for all values of γ , $M/P = 0.4$ or,

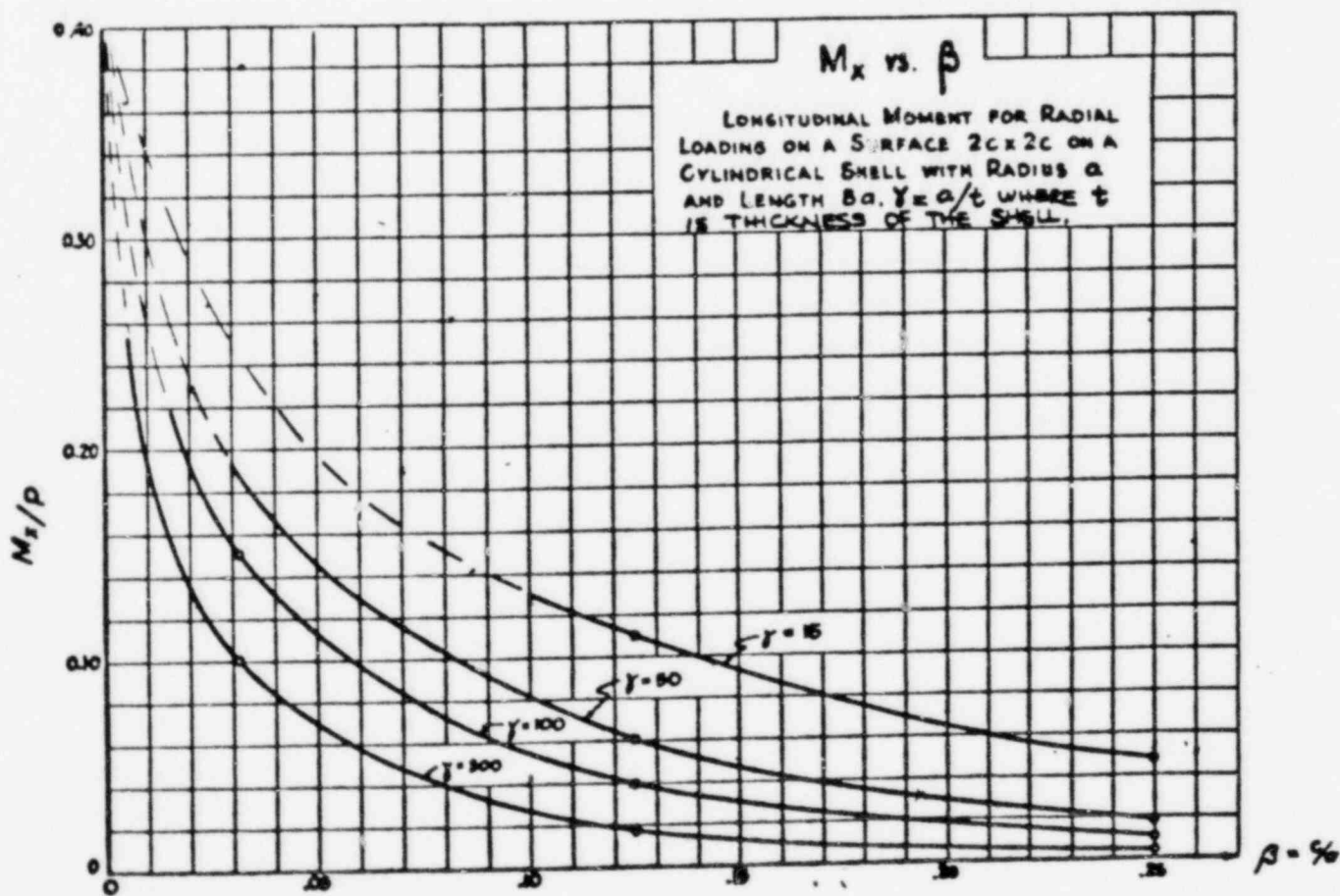
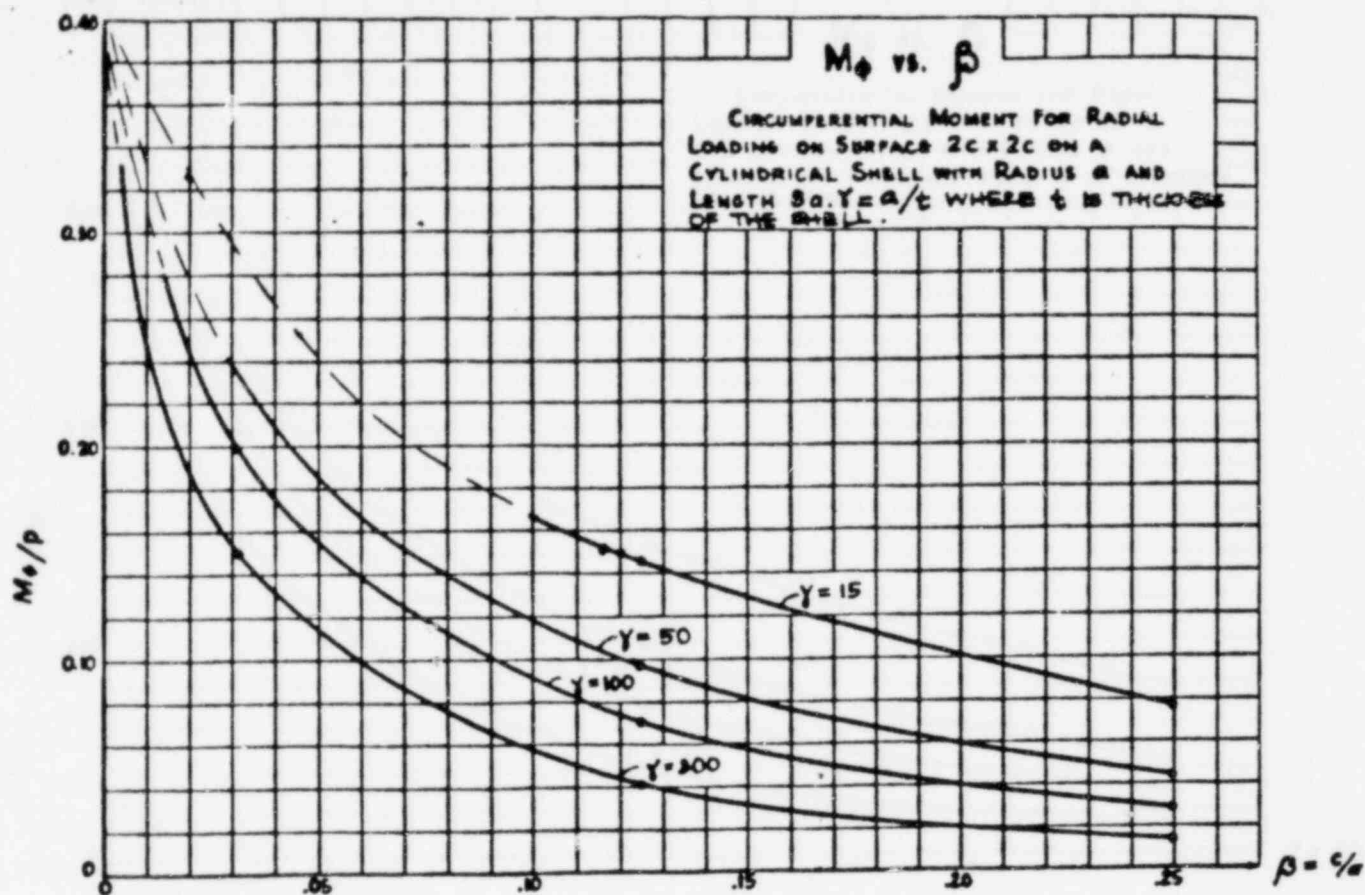
$$M = 0.4P \quad (4)$$

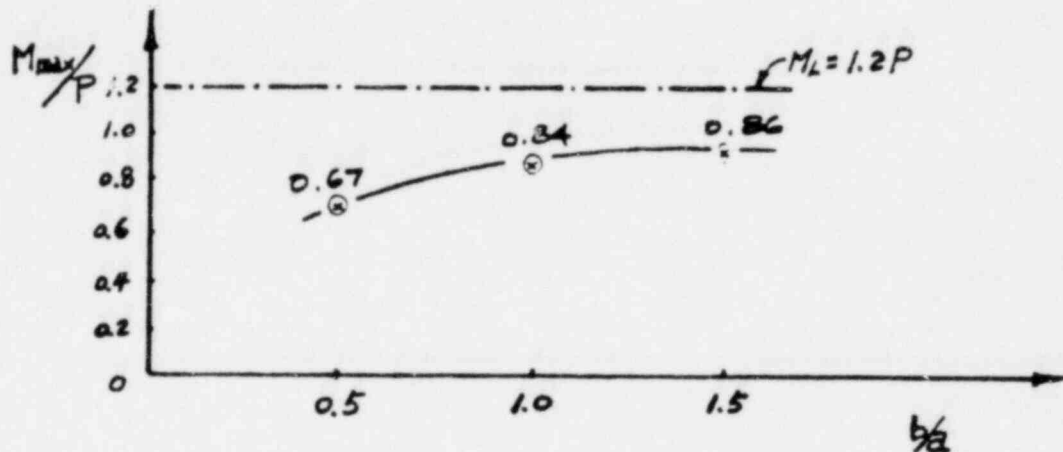
Equations (1) through (4) illustrate that a conservative estimate of the maximum localized moment per unit length for plates supported on all sides and subjected to a concentrated load P can be taken as:

$$M = 0.4P$$

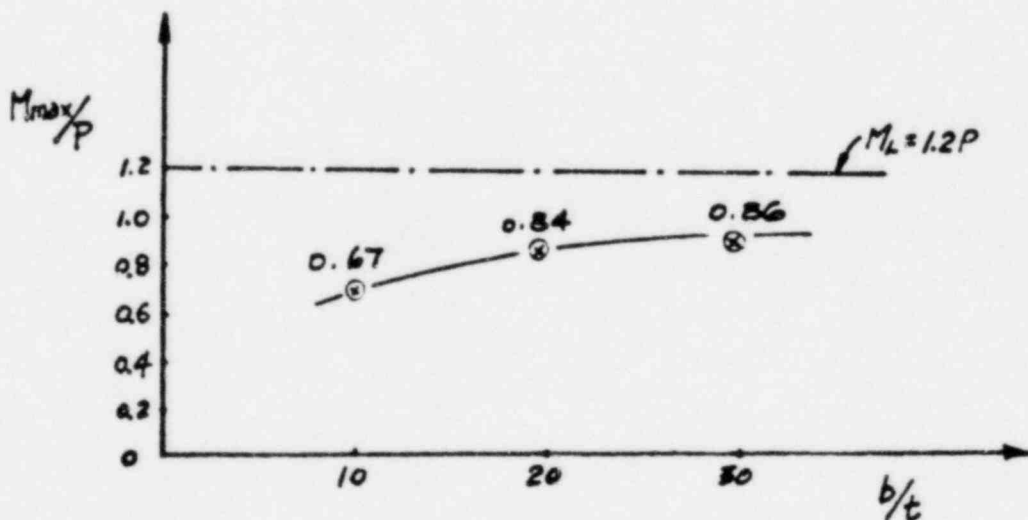
b) Justification for $M = 1.2P$

The moments in a plate with three sides simply supported and one side free, subjected to a concentrated load at the center of the free side, were computed using the finite element technique. Three plates of the same thickness with aspect ratios 0.5, 1.0 and 1.5, were considered for this analysis. The maximum moment occurred near the free side at the location of the load. This moment is plotted for various aspect ratios and side-to-thickness ratio on Sh.28. It is clear from the curves that in no case will the maximum moment close to the free side exceed $1.2P$ which is the upper limit value specified in Sec. 7.2, Appendix I, Part 2 (Ref. 4).





MOMENT - ASPECT RATIO CURVE



MOMENT - WIDTH TO THICKNESS RATIO CURVE

Question No. 10:

Provide brief descriptions for the analytical approaches used for single wythe and multiple wythe walls.

Response:

The single wythe concrete masonry walls were evaluated first by a simple procedure considering one-way action (beam analysis) under one or two most critical loading conditions. If the wall was found to be inadequate, then a more refined analysis considering plate action of the wall under all loading combinations was carried out. Whenever possible, hand calculations using available plate theories were used in the wall analysis. However, if the wall boundary conditions and/or loading were not covered in the available information, or if the walls had significant openings, computer programs using finite element model were used to analyze the walls accurately.

Consideration was given to cracking of walls for frequency determinations, and, to account for any uncertainties in material properties and effective mass, computed frequencies were varied to arrive at conservative inertia loads. Damping values, mentioned in Sec. 5.3 of Appendix I, Part 1 (Ref. 4), were used in the seismic analysis of the walls. In the presence of significant out-of-plane concentrated loads, local effects (block pullout), in addition to global effects on the wall, were also investigated.

The collar joint shear strength in multiple wythe walls was neglected, and, therefore, they were treated as an assembly of single wythe walls and analyzed accordingly. For seismic analysis of the multiple wythe walls, additional inertia due to the un-reinforced interior wythes was imposed on the reinforced exterior wythes.

Using the results of the analysis, various stresses were determined and checked against the allowables of Tables 1 and 2, Appendix I, Part 1 (Ref. 4). The alternative acceptance criteria was used to demonstrate the functional integrity of the walls that had stresses in excess of these allowables. The response to question no. 14 contains a detailed discussion of the walls qualified under the alternative acceptance criteria.

Question No. 11:

With regards to seismic analysis, indicate how the equipment loads were accounted for and how the earthquake forces in horizontal and vertical directions were considered.

Response:

Weights of the equipment and pipes attached to walls were generally considered as uniform loads distributed over the whole wall. However, the seismic reactions of attached Class I pipes were considered as concentrated loads on the wall.

For horizontal seismic analysis in the out-of-plane direction the uniform loads from attachments were added to the dead load of the wall and, simultaneously, the concentrated seismic loads from Class I pipes were also applied.

Since the masonry walls are not load bearing the effect of vertical seismic was generally insignificant. However, where pipes or other equipment were supported from a bracket connected to the wall, their vertical seismic reaction caused local moment on the wall due to the eccentricity. These local moment were combined with those due to the out-of-plane inertia loading using the absolute sum method. In addition local effects (i.e. block pullout) due to the concentrated moments were checked where necessary.

The effects of earthquake component along the in-plane horizontal direction were insignificant since the masonry walls at the Peach Bottom Plant are non-bearing and are not part of the lateral shear resisting system. Further, the pipe supports for all seismic Class I pipes attached to the walls allow movement of pipes along the in-plane direction of the wall; therefore no seismic force due to attached Class I pipes is exerted on the walls in the in-plane direction.

Question No. 12:

Provide details of proposed wall modifications and indicate, using sample calculations, how these modifications will correct the wall deficiencies.

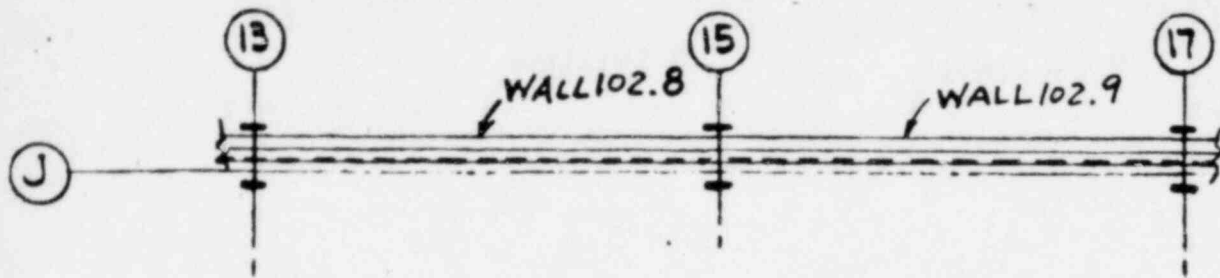
Response:

The details of the wall modifications for walls 102.8, 102.9 418.10 and 418.11 are shown on sh. 52 and 53.
32 33.

These walls have a simple support at the bottom and their vertical sides are free, except at the corners where 6x4, continuous along the top edge of each wall, is welded to the column webs. Prior to the modification, 6x4 was anchored in the 4 in. wythe by means of 1/2 in. diameter expansion bolts and had no connection with the 8 in. wythe. In the re-evaluation of masonry walls a conservative assumption was made that collar joints had zero shear strength. Theoretically, this meant that, except under bearing conditions, the

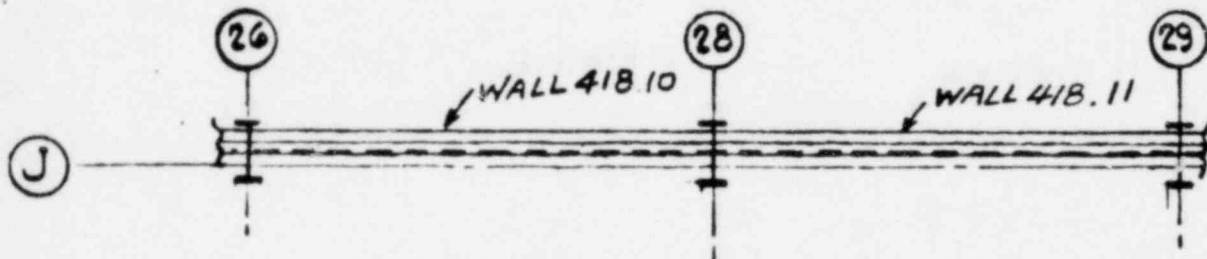
angle on the 4 in. wythe provided no support to the 8 in. wythe. To perform a meaningful analysis, consistent with the assumption of zero collar joint strength, it was necessary to make the angle support at the top effective for the 8 in. wythe. Modifications to the wall were, therefore, made to improve the support conditions at the top edges.

As shown in the modification details, the 1/2 in. thick plate is anchored in the 8 in. wythe with 5/8 in. diameter expansion bolts. Since this plate is also welded to $\angle 6 \times 4$, the two wythes are now connected at the top and the angle on the 4 in. wythe provides the necessary support for the 8 in. wythe. An analysis performed on the modified wall indicated that the stresses and deflections, both in the wall as well as the angle, were well within allowable limits under all applicable loading conditions.



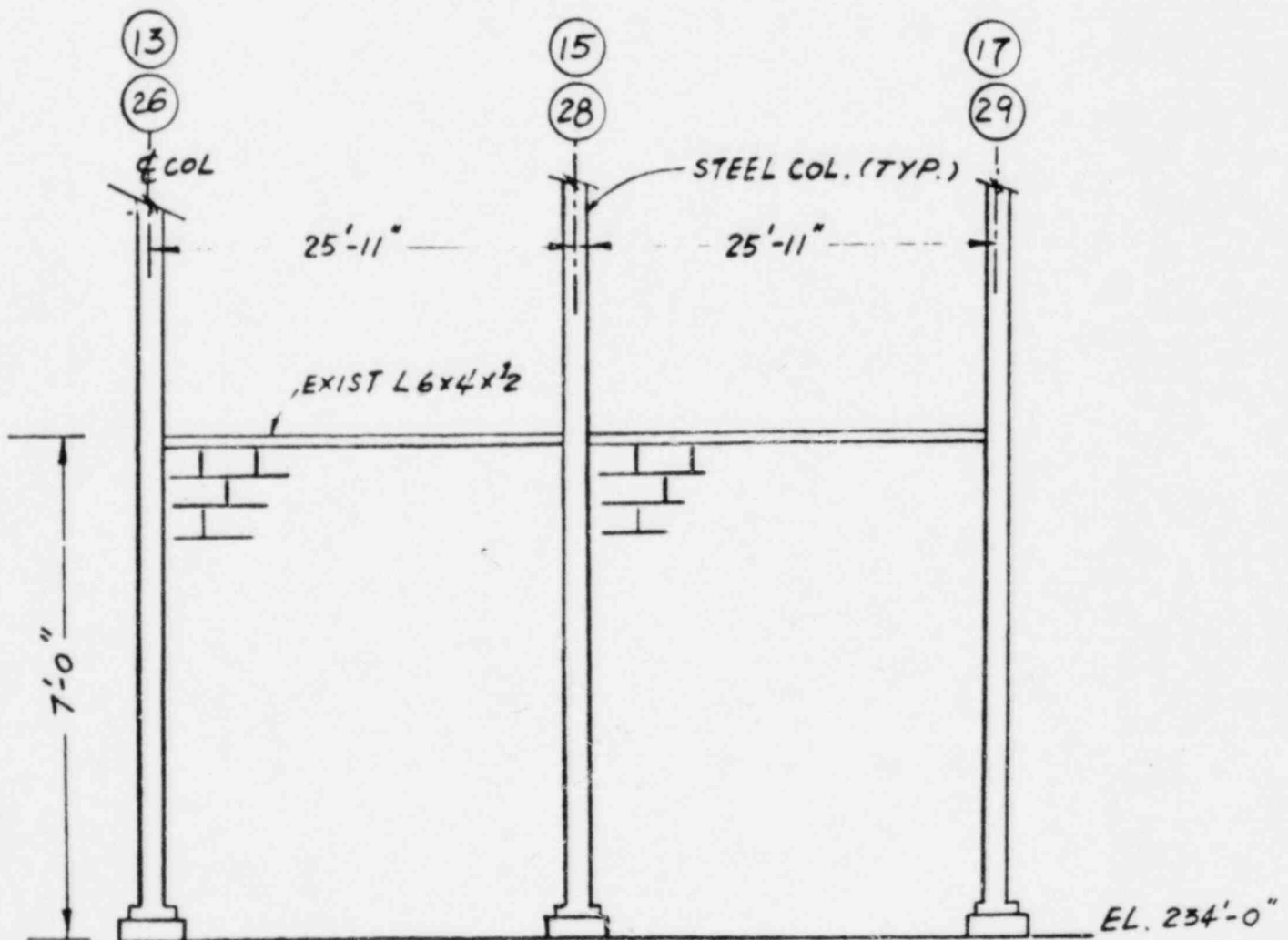
→ N

FOR UNIT 2
(SEE DWG.:S-102)

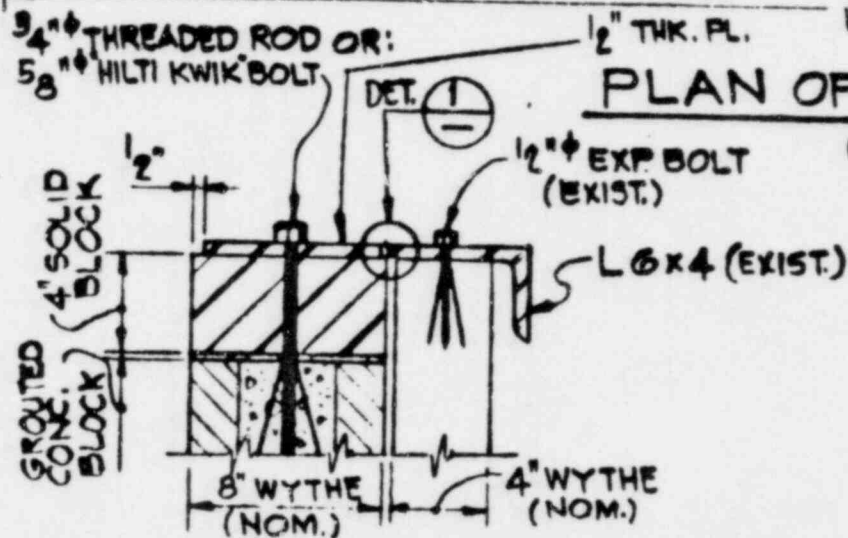
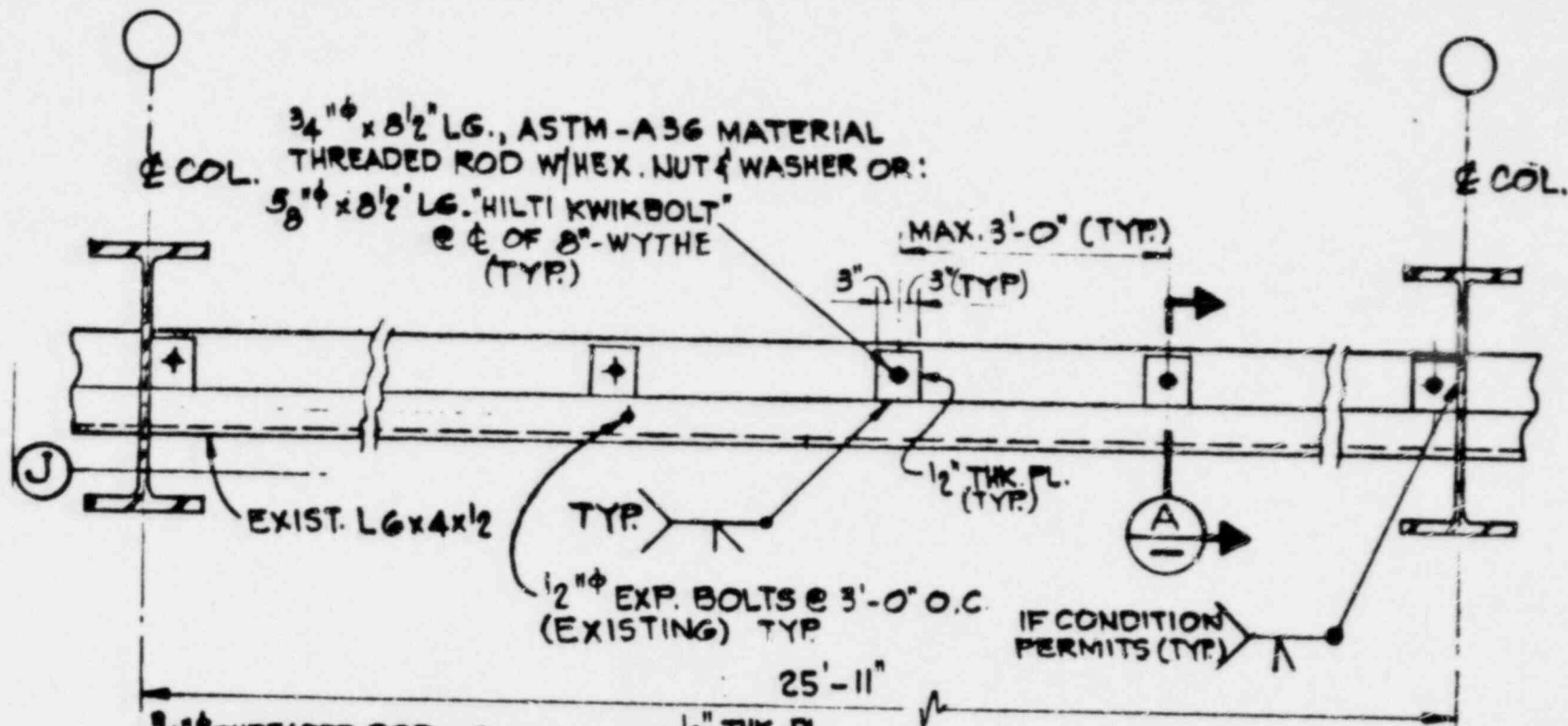


FOR UNIT 3
(SEE DWG.:S-418)

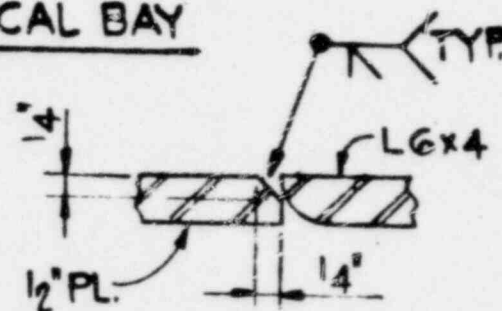
PLAN OF WALLS REQUIRING SUPPORT
MODIFICATION (4 WALLS)
(N.T.S.)



ELEVATION OF WALLS
(N.T.S.)



SECTION A
N.T.S.



DETAIL 1
N.T.S.



Question No. 13:

Provide a status report of the proposed wall modifications.

Response:

All wall modifications have been completed.

Question No. 14:

Provide the results of the wall analysis indicating the walls that do not qualify under the working stress criteria.

Response:

Out of a total of 86 concrete masonry walls 5 walls were qualified using the alternative acceptance criteria. (The other 4 walls previously qualified by alternative acceptance criteria have been re-evaluated. With pressure load reduction and reevaluation of stresses, the 4 walls have been qualified by elastic analysis methods. Section 6.2 of the Report is revised to delete the walls #45.1, #45.2, #406.1 and #406.2 from the listing to reflect this reevaluation). For all of these 5 walls the induced stresses due to the out-of-plane loads exceeded the working stress allowables of Tables 1 and 2 of Appendix I part 1 (Ref. 4)

Energy balance technique was used to qualify all of these walls (i.e. wall no. 68.2, 68.3, 532.1, 532.2 and 532.3). For these walls seismic loads were the governing loads. These walls were well anchored and supported such that the brittle mode of failure was precluded. In addition, the walls were reinforced such that the governing failure mode was flexural yielding of reinforcement, which meant that they were capable of undergoing large ductile inelastic out-of-plane deformations. The analysis indicated that for all of these five walls the maximum displacement ductilities were well within acceptable limits as mentioned in the response to question No. 6.

Arching theory was not used to qualify any of the walls. For all five walls qualified under the alternative acceptance criteria, it was ensured that a displacement of 2 times the calculated displacement would not adversely impact the required function of safety-related system attached and/or adjacent to the walls.

Question No. 15:

Indicate whether the door modifications and vent installations recommended by Bechtel (4) pertain to walls 68.1 to 68.4; if they do not, provide a complete description of the problem, including wall identification and proposed modification.

Response:

The door modifications and vent installations pertain to walls 68.1 to 68.4.

REFERENCES

1. Masonry Wall Design
USNRC, 06-May-80
IE Bulletin 80-11
2. S. L. Daltroff
Letter to B. H. Grier, NRC.
Subject: Response to IE Bulletin 80-11
Peach Bottom Atomic Power Station
Philadelphia Electric Co., 02-Jul-80
3. S. L. Daltroff
Letter to B. H. Grier, NRC.
Subject: Interim Response to Item 2b of IE Bulletin 80-11
Peach Bottom Atomic Power Station
Philadelphia Electric Co., 03-Nov-80
4. S. L. Daltroff
Letter with attachment to B. H. Grier, NRC.
Subject: IE Bulletin 80-11/ Reevaluation of Concrete Masonry
Walls for Peach Bottom Atomic Power Station
Philadelphia Electric Co., 04-May-81
5. Interim Criteria for Safety-Related Masonry Wall Evaluation
USNRC, 00-July-81
SRP 3.8.4, Appendix A
6. Uniform Building Code
International Conference of Building Officials, 1979
7. Building Code Requirements for Concrete Masonry Structures
Detroit: American Concrete Institute, 1979
ACI 531-79 and ACI 531-R-79
8. M. J. Cooney
Letter to B. H. Grier, NRC
Subject: Licensee Event Report
Narrative Description and Transmittal of LER
Philadelphia Electric Co., 04-Jun-81
LER 2-81-32/IT-0
9. Klinger, R. E. and Bertero, V. V.,
"Infilled Frames in Earthquake Resistant Construction",
Report No. EERC 76-32, Earthquake Engineering Research Center,
University of California, Berkely, CA December 1976
10. Dickey, W.L. and A. Mackintosh, "Results of Variation of
"b" or Effective Width in Flexural Concrete Block Panels ",
Masonry Institute of America, Los Angeles, 1971.

11. Scrivener, J.C., "Reinforced Masonry-Seismic Behaviour and Design," Bulletin of New Zealand Society for Earthquake Engineering, Vol. 5, No. 4, December 1972.
12. Scrivener, J.C., "Face Load Tests on Reinforced Hollow-brick Non-loadbearing Walls," New Zealand Engineering, July 15, 1969.
13. Simpson, W.M., "The Slender Walls Test Program" and Foth, V.A. and Johnson, J.R., "Results of Slender Wall Tests," Proceeding of Structural Engineer Association of Southern California, July 1981.
14. Blume, J.A., N.M. Newmark, and L.H. Corning, "Design of Multi-Story Reinforced Concrete Buildings for Earthquake Motions," Portland Cement Association, Ill. 1961.
15. Newmark, N.M., "Current Trends in the Seismic Analysis and Design of High-Rise Structures," Chapter 16, Earthquake Engineering, Edited by R.L. Wiegel, McGraw Hill, 1970.
16. Gabrielson, B.L. and K. Kaplan, "Arching in Masonry Walls Subjected to Out-of-Plane Forces," Earthquake Resistance of Masonry Construction, National Work Shop, NBS 016, 1978, pp. 283-313.
17. McDowell, E.L., K.E. McKee, and E. Savin, "Arching Action Theory of Masonry Walls," Journal of the Structural Division, ASCE, Vol. 82, No. ST2, March 1956, Paper No. 915.
18. McKee, K.E. and E. Sevin, "Design of Masonry Walls for Blast Loading," Journal of the Structural Division, ASCE Transactions, Proceeding paper 1511 January 1958.
19. Timoshenko, S and Woinowsky-Krieger, S., "Theory of Plates and Shells", 2nd Edition, McGraw-Hill Book Co., 1959.
20. Newmark, N.M., "Design of I-Beam Bridges", Highway Bridge Floors-A Symposium, Transaction ASCE Vol. 114, Paper No. 2381, p. 979-1072, 1949 (Moiseff Award, 1950).
21. Bijlaard, P.P., "Stresses from Radial Loads in Cylindrical Pressure Vessels", Welding Journal Research Supplement, 1954, p. 567-575.

APPENDIX B

Revisions to the
"Report on the Re-Evaluation
Of Concrete Masonry Walls"

For
Peach Bottom Atomic Power Station
Units 2 and 3
Docket Nos. 50-277 and 50-278

iii) Block Pullouts
and Load Transfer
Mechanism

Block pullouts were investigated by considering punching of the masonry walls by bearing plate or by shear pullout of the block at the mortar joint. In all cases, the existing capacity of the block joint was more than the applied forces. The mechanism for load transfer into the masonry walls consists of either through-bolts or expansion anchors, depending on the magnitude of applied loads.

iv) Interstory Drift

Since there are no rigid connections between the top slab (or girder) and the masonry walls, the interstory drift effects are not considered significant. The masonry walls are not part of the lateral load resisting system of the building in which these are located and as such are not affected by the interstory drift.

v) Thermal Effects

Studies were made to determine the moments and forces caused by thermal loading. It was determined that the temperature gradient across the thickness of the wall was not severe enough to induce any significant stresses in the wall.

REV. | A total of 77 walls out of 86 walls were qualified in accordance with conventional working stress criteria. Additionally, 5 walls were qualified by the alternative acceptance criteria of Section 6.0 of Appendix I, part 1, to demonstrate their acceptability functionally and structurally. For the walls qualified by the alternative acceptance criteria, attached safety related systems and the systems in proximity, were checked and found not to be adversely affected by up to twice the calculated deflection of the wall.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 All walls evaluated under this program, with the exception of those mentioned in para. 6.3, meet the requirements as set forth in the re-evaluation criteria.

- 6.2 Following walls were qualified by the alternative acceptance criteria of Section 6.0 of Appendix I, Part 1:

	<u>Serial</u>		
	<u>No.</u>	<u>Group No.</u>	<u>Wall Nos.</u>
REV.	1	3	68.2
	2	3	68.3
	3	12	532.1
	4	12	532.2
	5	12	532.3

- 6.3 In order to fully meet the requirements of the re-evaluation criteria, following walls require some minor fix. This fix is required because of our conservative assumption that collar joint tension and shear strengths are zero (Ref. Sec. 5.0 of this report).

<u>Serial</u>				
<u>No.</u>		<u>Group No.</u>	<u>Wall Nos.</u>	<u>Remark/Recommendation</u>
1	11	418.10	Modify support condition	
2	11	418.11	at top of the walls.	
3	11	102.8		
4	11	102.9		

The recommended modifications have been completed for walls 102.8 and 102.9 in Unit-2, and will be completed during the current Unit-3 refueling outage for walls 418.10 and 418.11.

3. $D+W'$
 4. $D+E+T_O$
 5. $D+E'+T_O$
 6. $D+F$
 7. $1.05D+1.25P$
 8. $1.05D+1.0P'+1.0P$
 9. $1.05D+1.0T_a$

Where:

D: Dead load of structure and equipment plus any other permanent loads and normal operating live load at 50 psf if applicable.
 E: OBE loads
 E': SSE loads
 W': Tornado loads
 T_O : Operating temperature loads
 T_a : Accident temperature loads
 F: Flood loads
 P: jet impingement load
 P': Pressurization load due to high energy line break

Load combinations 1 through 6 are obtained directly from the project FSAR. Load combinations 7, 8 and 9 have been arrived at by following the project design criteria, the PBAPS FSAR supplement No. 2, and, the recommendations of the Mechanical/Nuclear group and the Civil Staff.

4.0 MATERIALS

The materials used in the masonry wall construction are as follows:

Reinforced Masonry:

Hollow concrete units of ASTM C90-66 Grade U-1 (equivalent of Grade A of UBC-67 and Grade N of UBC-79) and grouted solid are used with ultimate compressive stress f'_m as indicated in Tables 1 and 2.

Reinforcing Steel

Rebar is ASTM A 615 Grade 40 or 60.

Mortar:

Mortar is ASTM C270, Type N with average compressive strength m_o of 750 psi at 28 days.

TABLE 1 (Cont'd)


Load Combination	Materials & Stress Description	Allowable Stress: ACI 531-79 (psi)
D + E	<u>Reinforcing Steel:</u>	
	Tension	20,000 for 40 grade steel (#3 thru #7) 24,000 for 60 grade steel (larger than #7)
	Compression	40% of ASTM specified yield - 16,000 for 40 grade steel 24,000 for 60 grade steel
D + E + T _o	Stresses for All Materials	Increase the allowables for load combination D + E by a factor of 1.3
D & E' D + W' D + E' + T _o D + F 1.05D+1.25P  1.05D+1.0P' +1.0P 1.05D+1.0T _a	<u>Masonry:</u> The allowable masonry stresses for load combination D + E shall be increased as follows:	
		<u>Increase Factor</u>
	Compression axial:	2.0
	flexural:	2.5
	Bearing:	2.5
	Shear and Bond:	1.67
	Tension	
	No tension rebar tension normal to bed joints:	1.67
	tension parallel to the bed joints; in running bond:	1.67

TABLE 1 (Cont'd)

Load Combination	Materials & Stress Description	Allowable Stress: ACI 531-79 (psi)
D & E'	<u>Reinforcing Steel:</u>	
D + W'		
D + E' + T _O	Tension & Compression	0.9 F _y , provided lap splice lengths and embedment (anchorage) develop this stress level. Allowable bond stresses may be increased by a factor of 1.67 in determining splice and anchorage lengths.
D + F		
1.05D+1.25P		
1.05D+1.0P'		
+1.0P		
1.05D+1.0T _a		

Notes:

1. The core concrete (or cell grout) has average compressive strength f'_c of 3800 psi.
2. From Table 4.3 of ACI 531-79, f'_m is interpolated as 1175 psi. This corresponds to compressive test strength of 3270 psi for the masonry units, based on the net cross-sectional area.

TABLE 2 (Cont'd)

Load Combination	Materials & Stress Description	Allowable Stress: UBC-67 (psi)
D + E'	<u>Reinforcing Steel:</u>	
	Tension & Compression	20,000 for grade 40 steel (#3 thru #7) 24,000 for grade 60 steel (larger than #7)
D + E + T _o	Stress for all materials	Increase the allowables for load combination D + E by a factor of 1.3
D + E'	<u>Masonry:</u> The allowable masonry stresses for load combination D + E shall be increased as follows:	
D + W'		
D + E' + T _o	Compression axial:	<u>Increase Factor</u> 2.0
D + F	flexural	2.5
1.05D+1.25P	Bearing:	2.5
② 1.05D+1.0P' +1.0P	Shear and Bond:	1.67
1.05D+1.0T _a	Tension	1.67

TABLE 2 (Cont'd)

Load Combination	Materials & Stress Description	Allowable Stress: UBC-67 (psi)
D & E'	<u>Reinforcing Steel:</u>	
D + W'		
D + E' + T _o	Tension & Compression	0.9 F _y , provided lap splice lengths and embedment (anchorage) develop this stress level. Allowable bond stresses may be increased by a factor of 1.67 in determining splice and anchorage lengths.
D + F		
1.05D+1.25P		
1.05D+1.0P' + 1.0P		
1.05D+1.0T _a		

Notes:

1. The core concrete (or cell grout) has average compressive strength f'_c of 3800 psi.
2. UBC allowable stresses for masonry are based on $f'_m = 1500$ psi (Ref. sec. 2404 UBC-1967)
3. Web reinforcement shall be provided to carry the entire shear in excess of 20 psi whenever there is required negative reinforcement and for a distance of one-sixteenth the clear span beyond the point of inflection.