TOLEDO EDISON

RICHARD P. CROUSE
Vice President
Nuclear
(419) 259-5221

Docket No. 50-346

License No. NPF-3

Serial No. 826

June 16, 1982

Mr. John F. Stolz, Director
Nuclear Reactor Regulation
Operating Reactors Branch No. 4
Division of Operating Reactors
United States Nuclear Regulatory Commission
Washington, D.C. 20555

Dear Mr. Stolz:

Your letter dated March 8, 1982 (Log No. 918) requested additional information concerning Masonry Wall Design; IE Bulletin 80-11, as it relates to Davis-Besse Nuclear Power Station Unit No. 1. Toledo Edison made a partial response in a May 3, 1982, submittal (Serial No. 814). Additional information and clarifications were provided in a meeting at Bechtel, Gaithersburg on May 27, 1982, between NRC staff reviewers, Bechtel and Toledo Edison personnel.

The attachment to this letter provides the same information provided in our May 3, 1982, submittal with additional information and clarifications added as indicated by change bars in the right margin. Part of the additional information was discussed in the May 27, 1982, meeting and is being provided in a formal manner at this time. This submittal completes our response to your March 8, 1982, request for additional information, except for Question 15. The information requested in Question 15 will be provided in early July 1982.

Very truly yours,

RPC:LDY:lab

attachment

cc: DB-1 NRC Resident Inspector

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RESPONSE TO NRC QUESTIONS
MASONRY WALL DESIGN
REVISION 1

1. Question

Table I (2) refers only to the edge conditions as indicated in the masonry wall drawings. Provide the boundary conditions assumed for the analysis.

Response

For CMU walls supported at the top and bottom edges, vertical strips have been analyzed assuming the top and bottom edges are pinned. This is conservative because the calculated natural frequency of the CMU wall, using this assumption, will be lower than the calculated natural frequency based on fixed or partially fixed connections. The various floor response spectra employed a constant response equal to the peak response on the low frequency side of the peak. This results in determining from the respective floor response curve, the upper bound seismic acceleration to be applied perpendicular to the plane of the CMU wall. In addition to upper bound seismic loads being applied to the CMU wall, the calculated moments used in stress calculations will be greater for the assumption of pinned edges versus the assumption of fixed or partially fixed edges, resulting in conservative stress calculations.

For CMU walls with short horizontal spans, supported on both vertical edges, horizontal strips have been analyzed assuming the vertical edges are pinned. This is conservative based on the above statements.

For CMU walls free at the top, the bottom connections are assumed fixed, with the stresses in the masonry and the connections analyzed accordingly.

2. Question

Indicate how the effects of higher modes were considered in cases where the analysis was based on the "block wall" program, which includes only three modes.

Response

The computer code "BLOCK WALLS" was verified by comparison with a solution considering nine modes. The comparison showed the computer code "BLOCK WALLS" produced a seismic moment due to flexure that was conservative by 1.1 percent versus the 9 mode solution.

3. Question

Indicate whether any of the walls was analyzed as a plate, with special reference to walls having cut-outs.

Response

Some CMU walls were analyzed using the computer code BSAP. Large openings, such as doors, were naturally included as part of the finite element mesh. In general openings larger that the selected mesh size were included in the model, openings

smaller were not, with the added consideration that the smaller opening sizes do not affect the structural integrity of the wall.

The mesh size selected was influenced by consideration of the height and length of the wall, location and number of applied loads, size and, therefore, subsequent cost of execution of the computer program created, and the spacing of both vertical and horizontal reinforcing. 1

For a summary of wall geometry and openings, see Table 1. Rectangular openings shown on the drawings at the time the walls were constructed are shown per the blockout schedules to have trim bars around the openings.

4. Question

Explain why the Table V (2) factors for operating basis earthquake (OBE) and wind load are 1.0, while the plant FSAR specifies a factor of 1.25 for these loads.

Response

Analysis of CMU walls used working stress criteria for which "SEB Interim Criteria for Safety-Related Masonry Wall Evaluation", July 1981, specifies a factor of 1.0 for both operating basis earthquake and wind. The factor of 1.25 in the FSAR is based on ultimate strength criteria.

5. Question

Indicate how equipment weight was considered in the analysis of the masonry walls. Standard Review Plan (SRP), Section 3.7.2 (7), suggests that the equipment weight should be multiplied by a factor of 1.5 times the peak floor acceleration.

Response

The equipment weight multiplied by a factor of 1.5 times the peak floor acceleration is an equivalent static load method of analysis which may be used in lieu of a dynamic method. All CMU walls which were analyzed used a dynamic method including actual dead weight of the wall plus the dead weight of the equipment.

6. Question

With reference to Section 7.1, Appendix E (2), use the envelope of the floor spectra or provide justification for using the average spectral acceleration.

Response

The evaluations herein demonstrate that the use of the average floor acceleration response spectra to calculate the response of the wall panel is appropriate.

For the purposes 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:

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:

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

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:

$$u(x,t) = (x/L)\ U_b + (1 - x/L)\ U_a + r(x,t) \qquad (2)$$

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) must satisfy the following simply-supported conditions:

$$r(o,t) = r\ (L,t) = 0 \qquad (3)$$

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

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

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

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

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

and the corresponding frequencies of vibration are:

$$\omega_n = n^2 \pi^2 \sqrt{\frac{EI}{mL^4}} \qquad n = 1, 2, 3, \ldots \tag{6}$$

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

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

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

$$\ddot{a}_n + \omega^2 n^a n = \Upsilon_n \left(\frac{\ddot{U}_a + \ddot{U}_b}{2} \right) \qquad n = 1, 3, 5, \ldots \tag{8a}$$

and

$$\ddot{a}_n + \omega^2 n^a n = \Upsilon_n \left(\frac{\ddot{U}_a - \ddot{U}_b}{2} \right) \qquad n = 2, 4, 6, \ldots \tag{8b}$$

where Υ_n = participation factor

$$= \frac{4}{n\pi} \tag{9}$$

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

$$\ddot{a}_n + 2\xi_n \omega_n \dot{a}_n + \omega^2 n^a n = \Upsilon_n \left(\frac{\ddot{U}_a + \ddot{U}_b}{2} \right) \qquad n = 1, 3, 5, \ldots \tag{10a}$$

and

$$\ddot{a}_n + 2\xi_n \omega_n \dot{a}_n + \omega^2 n^a n = \Upsilon_n \left(\frac{\ddot{U}_a - \ddot{U}_b}{2} \right) \qquad n = 2, 4, 6 \ldots \tag{10b}$$

Where ξ_{on} is the damping ratio of the nth 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 Equation 10a and 10b in terms of absolute acceleration response spectra gives:

$$|\partial_n|_{max} \leq |\Gamma_n| \left[\frac{S_a(\xi_{on}, \omega_n)}{2\omega^2 n} + \frac{S_b(\xi_{on}, \omega_n)}{2\omega^2 n} \right]$$

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

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

This illustrates that the use of the average of two floor acceleration response spectra to calculate the modal response of a wall panel is appropriate.

7. Question

With reference to Table II (2), indicate possible variations in the value E for masonry, and determine the actual value of E such that the spectral curve provides a conservative estimate for acceleration.

Response

A single value of E was used in the analysis of the CMU walls. Specifically, E was taken equal to 1000 f'_m per the Uniform Building Code, 1970 Edition, Volume 1, Table No. 24-H, page 170 (also refer to ACI 531-79, Table 10.1). The various floor response spectra employed a constant response equal to the peak response on the low frequency side of the peak. The value of E described above coupled with the modified frequency response spectrum results in conservative stress calculations.

8. Question

With reference to page 8 of Reference 2, provide sample calculations to illustrate that single wythe analysis of multiple wythe walls is conservative.

Response

All wythes in a multiple wythe CMU wall were assumed to respond as single wythe walls because of the difficulty in verifying the adequacy of the collar joint between the wythes. This was assumed to be conservative when using the re-

evaluation criteria and the objective of this response is to validate the degree of conservatism.

Two double wythe CMU walls (walls 1177 and 2107) were selected to compare the results obtained when computing the response as single wythe versus the response as double wythe using the re-evaluation criteria. In using the re-evaluation criteria, the two walls were assumed to have pinned supports at the top and bottom for the analysis of vertical spans. As a consequence, no forces are induced in the wall due to out-of-plane drift. Wall 1177 is 32 inches long, 87 inches high and consists of two wythes of eight inch wide units with 8" grout in between. Wall 2107 is 48 inches long, 217 inches high and consists of two wythes of 12" wide units with 12" grout in between.

Results

The results of the analyses performed for the two walls are given in Tables 2 and 3. Table 2 compares the frequencies of each wall acting either as a single or double wythe wall. Table 3 compares the maximum seismic moment and the tensile steel stress ratio for each wall acting either as a single or double wythe wall.

Discussion of Results

From the results presented in Table 2, it is clear that the single wythe assumption is conservative with respect to frequency shift. The fundamental or first mode frequency of double wythe wall 1177 is approximately five times that of the single wythe assumption. The fundamental or first mode frequency of double wythe wall 2107 is approximately eight times that of the single wythe assumption. 1

From the results presented in Table 3 it is clear that the single wythe assumption is conservative with respect to seismic moments and tensile steel stress ratios also. The seismic moments on single wythe walls are considerably greater than those for double wythe walls. Correspondingly, the tensile steel stress ratios for double wythe walls are much lower than for single wythe walls.

Conclusions

Two walls were selected to demonstrate that the use of the single wythe assumption for multiple wythe walls results in a conservative evaluation with respect to frequency shift and out of plane load considerations. The results indicate that the frequencies of the double wythe walls are greater than those of the single wythe walls. Therefore, from frequency shift considerations the use of the single wythe assumption is conservative. The results also indicate the seismic moments and the tensile steel stress ratios are much smaller for the double wythe walls compared to the single wythe walls.

The single wythe assumption applied to multiple wythe CMU walls is therefore conservative for the CMU wall re-evaluation criteria.

9. Question

It is the NRC's position that the energy balance technique and the arching theory should not be used in the absence of conclusive evidence of their validity as applied to masonry structures. With reference to Table I (2), explain the following points:

a. Provide sample calculations to show the procedure used to determine the ductility ratio of walls and explain the effect of wall boundary conditions on this ratio.

b. Explain why the ductility ratios for several walls are less than unity even though the working stresses have been exceeded.

c. Explain how a ductile mode of failure of the masonry walls can be guaranteed since it depends on several factors, such as the amount and distribution of reinforcements and the anchorage provided.

d. Explain how wall deflections are estimated for specific ductility ratios.

Response

a. The ductility of walls is calculated by the following equations:

$$\mu = 1/2 \left[1 + \left(\frac{Ma}{My}\right)^2 \right]$$

$$My = \frac{Fy\ Icr}{nd}$$

where: μ = ductility ratio

Ma = applied moment, including seismic

Fy = yield stress of reinforcing bar

Icr = cracked moment of inertia

n = modular ratio

d = distance from neutral axis to reinforcing bar

Example: Assume a 12" thick, totally grouted wall

$$My = \frac{40\ k/in\ \ x\ 275\ in\ /ft.}{20\ x\ 6.99\ in.}$$

$$= 78.68\ in\text{-}k/ft.$$

Ma = 109.69 in-k/ft. (Output from "BLOCK WALLS")

$$\mu = 1/2\ \ 1 + \frac{109.69^2}{78.68}$$

$$= 1.47$$

For effects of assumed wall boundary conditions on calculated moments and the resulting effect on the ductility ratio, see the response to Question #1.

b. A ductility ratio less than unity represents a stress in the reinforcing bar between the allowable and yield.

c. Reference 9 indicates that CMU walls with poured-in-place concrete columns and bond beams act as flexible structures, even when the design and construction is poor. This observation was based on actual structures located within 20 miles of the epicenter of an earthquake with a magnitude of 7.5 on the Richter scale. Reference 10 indicates considerable ductility where the reinforcement ratio is 0.15% or greater. All CMU walls analyzed for Davis-Besse Unit 1 have reinforcement ratios which exceed this value. Reference 11 indicates that a reinforced CMU wall is a ductile structure provided a flexural type of failure will occur with tensile yielding of the reinforcing steel. All CMU walls which were analyzed met this criteria for loads applied perpendicular to the plane of the wall.

d. Wall deflections are calculated by the following equation:

$$\Delta_{max.} = \Delta \times \frac{F_y}{f_s} \times \mu \times 2$$

where: Δ = deflection, from "BLOCK WALLS"

fs = stress of reinforcing bar, from "BLOCK WALLS"

2 = factor of safety

10. Question

With reference to Table IV (2), specify the allowable stresses for shear (shear walls and flexural members where reinforcement takes the shear), tension parallel to the bed joint, and tension normal to the bed joint.

Response

No CMU walls were provided to resist building shear or moments due to a seismic event. Therefore, no CMU walls have been evaluated as shear walls. In no case was it necessary to resist shear forces with reinforcement, because the masonry shear stress for each evaluated CMU wall was less than 0.02 f'_m. All CMU walls which were evaluated contain steel tension reinforcement to resist tension forces normal or parallel to the bed joint. Therefore, the assumed allowable masonry tensile stresses normal or parallel to the bed joint was zero for the evaluation performed in response to IE Bulletin 80-11.

The top edge detail for CMU walls fastened to the floor above contains a minimum of one inch thick expansion joint material. Therefore, the in-plane building shear will be minimal and the in-plane shear experienced by any CMU wall will be due to the weight of the wall plus attachments accelerated due to a seismic event. 1

11. Question

With reference to Table IV (2), justify the maximum value of 1200 psi specified for allowable stress in axial compression.

Response

The maximum allowable axial compressive stress of 1200 psi is given in the Uniform Building Code, 1970 Edition, Volume 1, Section 2418, page 158, and corresponds to a f'_m of 6000 psi. The values of f'_m established for Davis-Besse Unit 1 are 1500 psi for reinforced, completely grouted hollow or solid units and 1350 psi for reinforced, partially grouted hollow or ungrouted hollow units. The maximum allowable masonry compressive stresses for Davis-Besse Unit 1 are 300 psi and 270 psi, respectively.

12. Question

With reference to the proposed allowables for factored loads in Table IV (2), justify the increase factors of 3.17 for bearing, 1.5 for masonry shear, 1.67 for reinforcement shear, and 1.33 for bond. The SEB criteria (4) propose 2.5 for bearing, 1.3 for masonry shear, and 1.5 for reinforcement shear.

Response

The working stress allowables of Table IV are from the Uniform Building Code, 1970 Edition as referenced in the FSAR. The ultimate stress allowables are based on ACI 531-79, as recommended by the "SEB Interim Criteria for Safety-Related Masonry Wall Evaluation", July 1981 multiplied by an increase factor. The increase factors in the question result from comparing the ultimate stress allowables to the working stress allowables.

The increase factor based on ACI 531-79 for ultimate bearing stress is 2.5 (0.62/0.25) for bearing on full area and 2.5 (0.95/0.375) for bearing on one-third area or less. This is in agreement with the SEB criteria.

The increase factor of 1.5 for masonry shear, 1.67 for reinforcement shear and 1.33 for bond are intended for use with load combinations involving abnormal and/or extreme environmental conditions. Since ACI Code allowable stresses (reference ACI 531-79 Commentary, Chapter 10-1) are generally associated with a factor of safety of 3, the increase factors provide the following factors of safety against failure: 2.0 (3/1.5) for masonry shear, 1.8 (3/1.67) for reinforcement shear and 2.3 (3/1.33) for bond. It is our engineering judgement that these factors of safety are conservative and provide sufficient margin for abnormal and/or extreme conditions.

The only ultimate stress allowables used from Table IV were 0.85 f'_m for masonry compressive stress due to flexure which was checked when the energy balance technique was utilized and $0.9F_y$ for rebar tension stress which was used for load combinations containing safe shutdown earthquake. 1

13. Question

With reference to Table IV (2), justify the value for maximum allowable compression for reinforcement since it exceeds the ACI 531-79 maximum of 24,000 psi (6).

Response

The maximum allowable compressive stress for reinforcement is given in the Uniform Building Code, 1970 Edition, Volume 1, Section 2418, page 152. No CMU walls were evaluated utilizing reinforcement to resist compressive loads. This is accepted practice and is conservative.

14. Question

Provide details of proposed wall modifications with drawings, and indicate how these modifications will help to correct the wall deficiencies. Indicate how out-of-plane drift effects due to bracing are considered in the analysis.

Response

Details of the wall modifications were reviewed with the NRC in a meeting held on May 27, 1982.

Twelve CMU walls were modified by adding bracing members external to the walls, but the original wall boundary assumptions are not modified by introduction of these members. The boundary conditions were originally assumed as pinned (see response to Question 1), except for cantilever walls and, therefore, no forces are induced into the CMU walls due to out-of-plane drift. Differential displacement of the floors at the top and bottom of the walls may produce a stretching effect in the reinforcing bars located in the walls. For a typical wall, this has been calculated to produce a reinforcing bar stress of less than 30 psi for an OBE seismic event and less than 50 psi for a SSE seismic event. The magnitude of these stresses is insignificant. 1

15. Question

Provide a schedule for wall modifications.

Response

The schedule for wall modifications will be provided in early July, 1982. 1

REFERENCES

1. IE Bulletin 80-11
   "Masonry Wall Design"
   NRC, May 8, 1980

2. R. P. Crouse (Toledo Edison Company)
   Letter with attachments to J. G. Keppler (NRC)
   November 4, 1980

3. R. P. Crouse (Toledo Edison Company)
   Letter with attachments to J. G. Keppler (NRC)
   September 29, 1981

4. Standard Review Plan, Section 3.8.4, Appendix A
   "Interim Criteria for Safety-Related Masonry Wall Evaluation"
   NRC, July 1981

5. Uniform Building Code
   International Conference of Building Officials, 1979

6. ACI 531-79 and Commentary ACI 531R-79
   "Building Code Requrements for Concrete Masonry Structures"
   American Concrete Institute, 1979

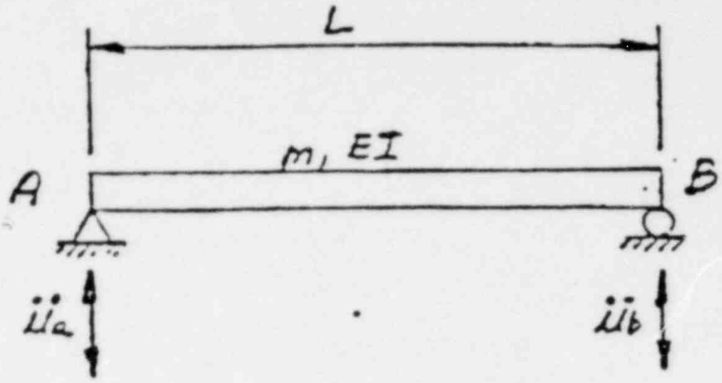
7. Standard Review Plan, Section 3.7.2
   "Seismic System Analysis"
   NRC, July 1981

8. Uniform Building Code, 1970 Edition, Volume 1

9. Structural Observations of the Kern Country Earthquake,
   ASCE Transactions, Paper No. 2777, August 1953,
   H. J. Degenkolb

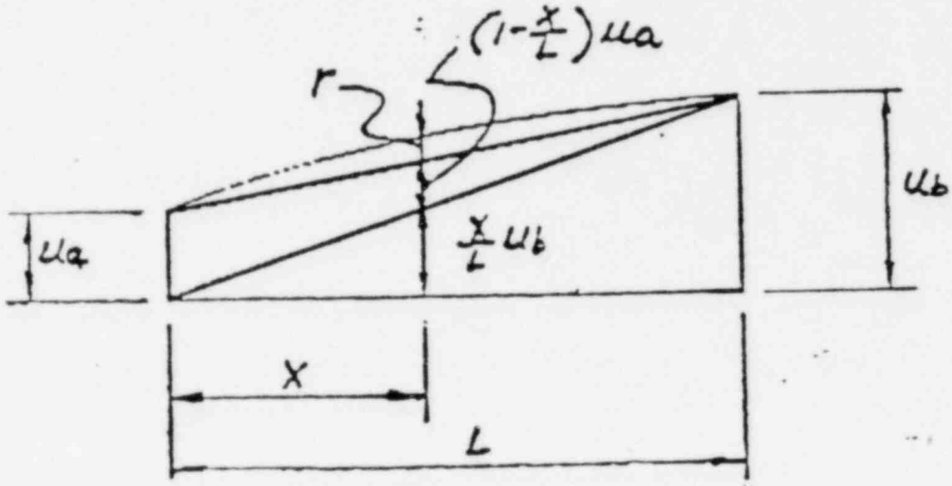
10. The Influence of Horizontally Placed Reinforcement on the
    Shear Strength and Ductility of Masonry Walls, 6th International World Conference on Earthquake Engineering, 1977,
    Sheppard, et al

11. Reinforced Masonry - Seismic Behavior and Design, Bulletin
    New Zealand Society for Earthquake Engineering, Vol. 5,
    No. 4, December 1972, J.C. Scrivner



IDEALIZED SIMPLY-SUPPORTED UNIFORM BEAM

FIGURE Nº 1



RELATION BETWEEN SEISMIC EXCITATION AND RELATIVE DISPLACEMENT

FIGURE Nº 2

TABLE 1

SUMMARY OF OPENINGS & MESH SIZES

FOR CMU WALLS ANALYZED BY BSAP

WALL NO.	LENGTH X HEIGHT OF THE WALL	LENGTH X HEIGHT OF THE RECTANGULAR OPENINGS INCLUDED	LENGTH X HEIGHT OF THE RECTANGULAR OPENINGS EXCLUDED	LARGEST ROUND OPENING EXCLUDED	LENGTH X HEIGHT OF MESH SIZE (MAXIMUM)	LENGTH X HEIGHT OF MESH SIZE (MINIMUM)
6097 (1)	37 1/4" x 196 1/2"	None	None	None	21 5/8" x 17 1/2"	21 5/8" x 14"
306D	143 1/4" x 156 7/8"	40" x 85"	10 1/4" x 17 5/8"	1/2" Ø	16" x 17"	7 1/2" x 13"
338D	108" x 282"	None	None	None	12" x 24"	12" x 21"
308D	116 1/2" x 282"	None	None	None		
309D	108 1/2" x 276 3/4"	None	None	None		
310D	107 3/8" x 278 1/4"	None	None	None		
3247	111 3/4" x 166 1/2"	12" x 22"	None	12" Ø	12 1/2" x 15 3/4"	12" x 11"
4026	137" x 222 1/2"	40 1/2" x 85 5/8" 18" x 12"	10" x 15 1/4"	1" Ø	18" x 22 1/8"	8 1/4" x 12"
2207	104" x 132"	20" x 12"	None	2 1/2" Ø	12" x 12"	12" x 10"
2217	120" x 132"	15" x 12"	None	Approx. 3" Ø	15" x 12"	9" x 12"
6037	103 1/2" x 196 1/2"	48" x 94"	None	4" Ø	12" x 17 1/2"	11 1/8" x 14"
4137	176 1/4" x 113 3/4"	33 1/4" x 41 1/4" 96 3/4" x 15"	None	None	25 1/2" x 20 5/8"	12 3/4" x 13 3/4"
5137	72 1/2" x 168"	17" x 30"	None	Approx. 1" Ø	17" x 18 1/2"	13" x 14 1/2"
4046	120" x 222 1/2"	40" x 86" 30" x 18"	None	Approx. 4" Ø	17 5/16" x 23 3/16"	10" x 18"
3357	274" x 176 5/8"	None	None	None	30" x 20"	14" x 16 5/8"
3016	153" x 198"	18" x 12"	2 3/4" x 4 1/2" (2)	Approx. 4" Ø	19 1/2" x 24"	18" x 12"
3026	74" x 198"	None	None	Approx. 3 1/2" Ø	18 1/2" x 24"	18 1/2" x 12"
3036	156" x 198"	78" x 108" 12" x 32" 18" x 12"	None	Approx. 5" Ø	26" x 20"	12" x 12"
3287	96" x 202"	40" x 86"	None	10" Ø	20" x 29"	14" x 21 1/2"
2297	216 3/4" x 176 3/8"	None	19" x 8 1/4" 21 5/8" x 15"	Approx. 4" Ø	18 3/4" x 18"	18" x 14 3/8"
4036	161 1/4" x 222 1/2"	15" x 34" 17 1/2" x 14 1/2" 27" x 34"	None	Approx. 4" Ø	17 1/2" x 17"	13 1/2" x 14 1/2"

TABLE 1 (Continued)

WALL NO.	LENGTH X HEIGHT OF THE WALL	LENGTH X HEIGHT OF THE RECTANGULAR OPENINGS INCLUDED	LENGTH X HEIGHT OF RECTANGULAR OPENINGS EXCLUDED	LARGEST ROUND OPENING EXCLUDED	LENGTH X HEIGHT OF MESH SIZE (MAXIMUM)	LENGTH X HEIGHT OF MESH SIZE (MINIMUM)
4786	83 1/4" x 223"	None	None	Approx. 5" Ø	18" x 18 3/4"	14 1/4" x 18"
4906 (3)		15" x 36"	None	None		
4796	155 3/4" x 222 1/4"	40" x 93 3/4"	6" x 27"	Approx. 5" Ø	19" x 16"	13" x 8 3/4"
4886 (3)		19" x 88 3/4"				
4896		13" x 31"				
5207	198" x 147 1/4"	22" x 14 1/4"	None	Approx. 1"Ø	22" x 24"	22" x 13"
311D	122 1/4" x 277 3/8"	None	None	None	12 1/4" x 24"	12 1/4" x 18"
3237	108" x 170"	60" x 102"	None	None	12" x 17"	12" x 17"
1068	125 1/2" x 96"	None	None	Approx. 2" Ø	16" x 16"	13 1/2" x 16"

Notes:

(1) Combined for Analysis with Wall 6037.
(2) 4" Deep Recess for Electrical Box.
(3) Combined for Analysis.

TABLE 2

FREQUENCY OF WALLS WITH SIMPLY SUPPORTED

BOUNDARY CONDITIONS

Wall No.	Thickness (Inches)	Wythes	Frequencies (HZ)
1177	8	1	21.165, 81.339, 186.786
	24	2	113.316, 448.576, 961.638
2107	12	1	3.349, 13.17, 28.258
	36	2	29.35, 115.782, 247.58

TABLE 3

MAXIMUM SEISMIC MOMENTS AND

TENSILE STEEL STRESS RATIOS OF

WALLS WITH SIMPLY SUPPORTED

BOUNDARY CONDITIONS.

Wall No.	Thickness (Inches)	Wythes	Seismic Moment (Inch Kips)	Tensile Steel Stress Ratio
1177	8	1	10.30	0.37*
	24	2	3.80	0.03
2107	12	1	139.60	1.66**
	36	2	49.0	0.11

**Wall acceptable by "Energy Balance Technique"

* Revised from value reported in " Masonry Wall Re-Evalaution, Response to NRC IE Bulletin No. 80-11, Davis-Besse Nuclear Power Station Unit 1, November 4, 1980