Attachment 2

TECHNICAL EVALUATION REPORT

MASONRY WALL DESIGN

TOLEDO EDISON COMPANY

DAVIS-BESSE NUCLEAR POWER STATION UNIT 1

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FOREWORD

This Technical Evaluation Report was prepared by Franklin Research Center under a contract with the U.S. Nuclear Regulatory Commission (Office of Nuclear Reactor Regulation, Division of Operating Reactors) for technical assistance in support of NRC operating reactor licensing actions. The technical evaluation was conducted in accordance with criteria established by the NRC.

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1. INTRODUCTION

1.1 PURPOSE OF REVIEW

The purpose of this review is to provide technical evaluations of licensee responses to IE Bulletin 80-11 [1]* with respect to compliance with the Nuclear Regulatory Commission (NRC) masonry wall criteria. In addition, if a licensee has planned repair work on masonry walls, the planned methods and procedures are to be reviewed for acceptability.

1.2 GENERIC ISSUE BACKGROUND

In the course of conducting inspections at the Trojan Nuclear Plant, Portland General Electric Company determined that some concrete masonry walls did not have adequate structural strength. Further investigation indicated that the problem resulted from errors in engineering judgment, a lack of established procedures and procedural details, and inadequate design criteria. Because of the implication of similar deficiencies at other operating plants, the NRC issued IE Bulletin 80-11 on May 8, 1980.

IE Bulletin 80-11 required licensees to identify plant masonry walls and their intended functions. Licensees were also required to present reevaluation criteria for the masonry walls with the analyses to justify those criteria. If modifications were proposed, licensees were to state the methods and schedules for the modifications.

1.3 PLANT-SPECIFIC BACKGROUND

In response to IE Bulletin 80-11, Toledo Edison Company provided the NRC with documents [2-6] describing the status of masonry walls at Davis-Besse Unit 1. The information in these documents was reviewed, and a request for additional information was sent to the Licensee on March 8, 1982, to which the Licensee responded [7, 9]. As the result of a meeting between Toledo Edison and members of the NRC staff on May 27, 1982 and the combined technical

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* Numbers in brackets indicate references, which are cited in Section 5.

meeting and site visit of June 21-23, 1983, additional questions were sent to the Licensee, to which it has responded [8, 13].

In Reference 6, the Licensee reported 169 safety-related masonry walls at Davis-Besse Unit 1. The main functions of these walls include fire and flood barriers, radiation shielding, and negative pressure boundaries. Also, the walls support minor platforms, piping, conduit, and instrumentation. No masonry walls were designed to act as shear walls.

Masonry walls at the plant are typically hollow unit construction (three cells), laid in a stack bond, partially or fully grouted, and vertically reinforced. The stack bond construction is not commonly used in nuclear plants, and exclusive use of stack bond construction is found at this plant. Both single- and double-wythe walls are found. The properties of the materials used in construction, according to Reference 4, are as follows:

reinforced masonry (fully grouted) - $f'm = 1500$ psi
reinforced masonry (partially grouted or hollow) - $f'm = 1350$ psi
vertical reinforcing - $fy = 40,000$ psi
horizontal reinforcing - Dur-O-Wal extra heavy truss type, $fy = 65,000$ psi

Reference 6 stated that 28 walls required structural modification involving the addition of steel braces and the reinforcing of boundary connections with angles and bolts.

The Licensee has relied upon the energy balance technique to qualify 75 masonry walls, 16 of which are among the modified walls mentioned above. NRC, FRC, and FRC's consultants (Drs. H. Harris and A. Hamid of Drexel University) have conducted an exhaustive review of this subject based on submittals provided by the Licensee and published literature and have concluded that the available data in the literature do not give enough insight for understanding the mechanics and performance of reinforced masonry walls under cyclic, fully reversed dynamic loading. As a result, a meeting with representatives of the affected plants was held at the NRC on November 3, 1982 so that the NRC and FRC's staff and consultants could explain why the applicability of the energy balance technique to masonry walls in nuclear power plants is questionable [13]. In a subsequent meeting on January 20, 1983, consultants of utility companies presented their rebuttals [14] and requested that they be treated on a plant-by-plant basis.

In accordance with the above request, NRC, FRC, and consultants visited several nuclear power plants, including Davis-Besse Unit 1 on June 21 through 23, 1983, to examine the field conditions of masonry walls in the plants and to gain first-hand knowledge of how the energy balance technique is applied to actual walls. Further discussion on this subject is provided in Section 3.1.

2. EVALUATION CRITERIA

The basic documents used for guidance in this review were the criteria developed by the Structural and Geotechnical Engineering Branch (SGEB) of the NRC (attached as Appendix A to this report), the Uniform Building Code [10], and ACI 531-79 [11].

The materials, testing, analysis, design, construction, and inspection of safety-related concrete masonry structure should conform to the SGEB criteria. For operating plants, the loads and load combinations for qualifying the masonry walls should conform to the appropriate specifications in the Final Safety Analysis Report (FSAR) for the plant. Allowable stresses are specified in Reference 11 and the appropriate increase factors for abnormal and extreme environmental loads are given in the SGEB criteria (Appendix A).

3. TECHNICAL EVALUATION

This evaluation is based on the Licensee's earlier responses [2-6] and subsequent responses [7, 8, 9, 12] to the NRC requests for additional information. The Licensee's criteria were evaluated with regard to design and analysis methods, loads and load combinations, allowable stresses, construction specifications, materials, and any relevant test data.

3.1 EVALUATION OF LICENSEE'S CRITERIA

The Licensee evaluated the masonry walls using the following criteria:

- o Allowable stresses for service loads are based on the Uniform Building Code of 1970.
- o The working stress design method and energy balance technique were used to qualify the walls. Of 169 safety-related walls, 75 have been qualified by the energy balance technique.
- o Damping for unreinforced walls:

  2% for operating basis earthquake (OBE) and safety shutdown earthquake (SSE)
- o Damping for reinforced walls:

  2% for OBE
  4% for SSE
- o When the natural frequencies of the walls are on the lower side of the peak floor response spectra, this peak value was used in obtaining the inertial loading.
- o A typical analytical procedure used in the working stress design method is summarized below.
  - determine wall boundary conditions (pinned or free)
  - calculate the wall's fundamental frequency using either a one-way vertical or one-way horizontal action assumption
  - obtain inertial loading from average floor response spectrum (If the fundamental frequency of the wall is on the lower side of the peak floor response spectra, the peak value was selected to obtain the inertial loading.)
  - compare computed stresses with the allowable values.
- o A typical analytical procedure used in the energy balance technique is summarized below:

- determine the applied moment
- determine the yielding moment
- obtain the ductility ratio (based on the formula given in Response 11 and compare it with the allowable ductility ratio of 5
- check the compressive stress of the masonry and compare with the allowable of 0.85
- obtain the deflection of the wall (based on the formula given in Response 11) and check if it affects any safety-related items.

The Licensee's criteria [4] and responses [7, 8, 9, 12] have been reviewed. Other than those areas identified in Section 4, the Licensee's criteria have been found to be adequate and in compliance with the SGEB criteria (Appendix A).

Following is an review of the Licensee's responses [7, 9] to the NRC's original questions as well as the responses [8, 12] to questions that arose from the meeting of May 27, 1982 and the combined meeting and site visit of June 21-23, 1983.

Question 1

Table I [4] refers only to the edge conditions as indicated in the masonry wall drawings. Provide the boundary conditions assumed for the analysis.

Response 1

The Licensee responded that, in walls that were analyzed as vertical strips, the top and bottom edges have been assumed pinned unless the wall is free at the top, in which case the bottom is fixed. (See Response No. 26 for further details.) In walls that were analyzed with horizontal spans, the sides were assumed pinned.

A typical base connection consists of Phillips Red Head Self Drilling Anchors in the concrete slab, with 24-in-long threaded bars in the anchors, at each vertical reinforcement. The method of analysis is judged to be adequate.

Question 2

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

Response 2

In this response, the Licensee stated that the computer code "BLOCK WALLS" was verified by comparison with a nine-mode solution and the difference was only 1.1% in the wall's responses. As has been seen in other plants, the first mode usually contributes 95% or higher to the response. For all practical purposes, the responses resulting from the first three modes should be adequate and in compliance with the SGEB criteria.

Question 3

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

Response 3

This response indicated that some masonry walls were analyzed using the finite element computer code BSAP. The size of the finite element mesh was influenced by the height and length of wall, location and number of applied loads, size and cost of the model created, and the spacing of reinforcing. Openings larger than the selected mesh size were included in the model, smaller ones were not.

The review of some calculations during the meeting on June 21 through 23, 1983, indicated that the mesh size used in the finite element model was reasonable adequate, and large openings were adequately accounted for in the model.

Question 4 (from meetings on June 21, 22, and 23, 1983)

Clarify and justify the use of I effective/t effective or I uncracked/t uncracked in the calculations of wall No. 2297, where:

I effective = effective moment of inertia
t effective = effective thickness of the section used in the analysis
I uncracked = uncracked moment of inertia
t uncracked = thickness of the uncracked section used in the analysis.

Response 4 [Reference 12]

Wall 2297 is a double-wythe wall consisting of two 8-in wythes, separated by 2 inches of concrete fill. The wall was originally analyzed as two independent 8-in walls (single-wythe) using the BLOCK WALLS computer program and a maximum ductility ratio of 2.07 (qualified based on energy balance technique). However, the steel floor beam on top of the wall was overstressed; therefore, a modification was required. For the purpose of designing the proper modification, the wall was reanalyzed assuming a complete thickness of the wall (composite section).

The BSAP dynamic run was performed to obtain the natural frequency of the wall using I effective (which is function of I cracked and I uncracked); assuming the applied moment is equal to the yield moment, this results in a minimum natural frequency and a maximum acceleration. This acceleration is then used as input for the BSAP static run to determine stresses and reactions. The stress of the steel floor beam with modification (addition of braces to the beam) was then checked.

The method of using I effective and the application of yield moment is reasonably adequate for designing the modification.

Question 5

> Explain why the Table V [4] 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 5 [Reference 7]

This response explained that the FSAR load factor of 1.25 for OBE and wind loads was based on the ultimate strength method of analysis, whereas the factor of 1.0 in Table V [4] is for the working stress method.

This response is satisfactory and is consistent with the SGEB criteria.

Question 6 (from the meetings on June 21, 22, and 23, 1983)

> Provide a summary of walls which have to be qualified by an increase in allowable stresses for OBE load combinations. Indicate actual calculated stresses and allowable stresses. Justify the increase in allowable stress.

Response 6 [Reference 12]

The Licensee responded that four walls were qualified on the basis of allowable stresses that had been increased by 1.25 for load combinations containing OBE loads. A table was provided, giving the actual steel and masonry stress in each of these four walls as well as the factored and unfactored allowable stresses. The four walls and the percentages by which the actual steel tensile stresses exceed the unfactored allowables are:

3016	12.5%
3198	13.0%
5137	23.0%
4026	14.0%

The SGEB criteria do not permit an increase in allowable stress for load combinations containing OBE loads. However, the Licensee made assumptions that compensate for the increase factor. The Licensee used a constant response floor response spectra, equal to the peak response for natural frequencies on the low frequency side of the peak and low damping (2% for OBE and 4% for SSE) in the analysis. Also, the four walls in question were analyzed using one-way simple spans (vertical for 3016 and 4026 and horizontal for 3198 and 5137). These assumptions will yield a conservative estimate of stress values. Because of this, the Licensee's response is acceptable with respect to the SGEB criteria.

Question 7

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

Response 7

This response states that the SRP factor of 1.5 refers to an equivalent static analysis, and since all masonry walls at Davis-Besse Unit 1 were analyzed with a dynamic method which includes the weight of the wall and equipment, a factor of 1.5 was not appropriate.

Since the equipment weight was included in the dynamic analysis, the concern about the factor of 1.5 is no longer applicable in this case.

Question 8

> With reference to Section 7.1, Appendix E [4], use the envelope of the floor spectra or provide justification for using the average spectral acceleration.

Response 8

The Licensee showed analytically that the use of the average response spectra is more appropriate than the conservative approach of using the envelope of the floor response spectra. Using an example of an undamped simply supported beam, it is shown that the absolute maximum modal dislacement response is less than or equal to the value obtained by using the average of spectral values corresponding to the two supports. Also, the use of the average response spectra is considered satisfactory because of the conservative method of determining response: a constant response, equal to the peak response, is used on the low frequency side of the peak. The Licensee's response is satisfactory.

Question 9

> With reference to Table II [4], 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 9

In this response, the Licensee stated that a value of E of 1000 f'm, based on the 1970 Uniform Building Code and ACI 531-79, was used. Since the floor response spectrum employed a constant response equal to the peak response for frequencies on the low frequency side of the peak, this value of E should be adequate.

Question 10

With reference to page 8 of Reference 4, provide sample calculations to illustrate that single-wythe analysis of multiple-wythe walls is conservative.

Response 10

In response to this question, the Licensee performed an analysis of two double-wythe walls using a single-wythe and double-wythe assumption for each. The results obtained from the single-wythe analysis were compared to the results obtained from the double-wythe analysis; this was done for each wall (1177 and 2107). The results of these analyses were shown in terms of frequencies, moments, and tensile steel stresses. The results indicate that the frequencies of the double-wythe wall are greater than those of the single-wythe walls and that the moments and tensile steel stress ratios are much smaller in the double-wythe wall. The single-wythe assumption applied to multiple-wythe masonry walls is considered conservative and in compliance with the SGEB criteria.

Question 11

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 [4], 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 the 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 11

The Licensee responded to the four points raised in this question as follows:

a. The Licensee provided the following equation used to determine the ductility ratio:

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

where Ma = applied moment
My = yield moment

This formula was developed based on the energy balance technique used in a single degree of freedom system where the material is assumed to have an elastic, perfectly plastic behavior.

b. It was stated that a ductility ratio less than unity simply means that the stress in the reinforcing bar falls below yield stress level.

c. The Licensee cited several references that support the assumption of a ductile mode of failure in masonry walls. Reference 16 observes the flexible behavior of masonry structures within 20 miles of the epicenter of an earthquake; Reference 17 indicates significant ductility in walls having a reinforcement ratio greater than 0.15% (all masonry walls at Davis-Besse Unit 1 have reinforcement ratios that exceed this value); Reference 18 indicates that a masonry wall is a ductile structure when a flexural type of failure will occur with tensile yielding in the reinforcing steel.

d. The equation by which masonry wall defections were calculated was given as follows:

$$\Delta_{max} = \Delta \times \frac{Fy}{fs} \times \mu \times 2$$

where Fy = yield stress of reinforcing bar
fs = calculated stress of reinforcing bar
Δ = calculated deflection
2 = factor of safety

NRC staff, FRC, and FRC's consultants have conducted an exhaustive review of available information on the energy balance technique and of the Licensee's responses to determine the technical adequacy of the methodology. In addition, a site visit and a meeting were held with Davis-Besse Unit 1 representatives on June 21 through 23, 1983 to examine the as-built conditions of the walls and to review the analytical procedures and specific calculations regarding this subject.

FRC and its consultants have issued their evaluation and assessment of the use of the energy balance technique for masonry walls [13, 15]. The Structural and Geotechnical Engineering Branch (SGEB) has issued a position statement regarding this subject which will be addressed in its Safety Evaluation Report.

The following walls have been qualified by the energy balance technique: 2057, 2067, 2147, 2167, 2427, 3417, 5017, 5147, 5187, 5197, 4867, 6107, 1237, 3357, 1348, 2107, 2267, 3297, 3367, 2077, 1038, 1157, 3277, 4917, 5127, 5157, 5277, 6087, 2247, 2317, 2447, 3267, 5367, 305D, 307D, 313D, 1197, 2018, 2227, 2257, 2277, 3257, 3347, 3397, 2371, 304D, 1087, 1147, 2237, 2367, 3227, 4046, 1227, 1267, 1337, 1428, 2087, 2177, 311D, 3036, 3307, 3167, 3177, 3187, 3287, 5107, 2337, 4016, 4036, 4796, 4886, 4896, 3407, and 4647.

Question 12 (from meeting on May 27, 1982)

> Provide additional information, as discussed in our meeting, to support the use of our energy balance technique for the masonry wall evaluation (schedule for submittal of this information will be provided by 6/4/82).

Response 12 [Reference 8]

To further substantiate the applicability of the energy balance technique to masonry walls, the Licensee referred to full scale tests performed by J. C. Scrivener and the SCACI-SEAOSC Task Committee on Slender Walls.

Scrivener tested an 8-ft 11-in high wall and partially grouted walls constructed of 4-in hollow bricks reinforced with a 1/2-in diameter bar at approximately 18 in on centers. The test results indicated that a ductility ratio in excess of 10 can be achieved.

The SCACI-SEAOSC Task Committee on Slender Walls tested 4-ft-wide by 24-ft-high walls constructed of 6-, 8-, and 10-in block reinforced with five

No. 4 bars. The minimum deflection of failure of any of the nine walls tested was 7.1 in, indicating a minimum ductility of 10.

As stated in the review of Response 11 above, FRC and its consultants have issued their evaluation of the use of the energy balance technique [13, 15], and the SGEB has issued a position statement which will be addressed in its Safety Evaluation Report.

Question 13

> With reference to Table IV [4], 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 13

The Licensee responded that no masonry walls were built to resist building shear, and in no case was reinforcement relied upon to resist shear, since the maximum masonry shear stress for every wall was under 0.02 f'm. Also, all tension parallel and normal to the bed joint was taken by reinforcement; so the allowable masonry tension parallel and normal to the bed joint was zero.

This response satisfies the SGEB criteria.

Question 14

> With reference to Table IV [4], justify the maximum value of 1200 psi specified for allowable stress in axial compression.

Response 14

The Licensee responded that the value of 1200 psi for allowable stress in axial compression is obtained from the 1970 Uniform Building Code and corresponds to an f'm of 6000 psi. However, this value was not used for Davis-Besse Unit 1. The values established for f'm at Davis-Besse Unit 1 are 1500 psi for reinforced-fully grouted or solid units and 1350 psi for reinforced-partially grouted or hollow units. These values result in maximum allowable masonry axial compression values of 300 psi and 270 psi, respectively, which are the allowables for Davis-Besse Unit 1.

The Licensee's maximum allowable values for masonry axial compression stress are in compliance with ACI 531-79 and are therefore acceptable.

Question 15

> With reference to the proposed allowables for factored loads in Table IV [4], 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 SGEB criteria [4] propose 2.5 for bearing, 1.3 for masonry shear, and 1.5 for reinforcement shear.

Response 15

In this response, the Licensee indicated that the factors used to obtain the stress allowables for bearing stress from the allowables in ACI 531-79 is 2.5. This value is in compliance with the SGEB criteria.

Regarding the factors for masonry shear and reinforcement shear, the Licensee stated that these increase factors were never used in the analysis (masonry shear was within the allowable for OBE case) and the only ultimate stress allowables that had to be used were 0.85 f'm for masonry compressive stress due to flexure and 0.9 Fy for tension in reinforcement. These values are in compliance with the SGEB criteria.

Question 16

> With reference to Table IV [4], justify the value for maximum allowable compression for reinforcement since it exceeds the ACI 531-79 maximum of 24,000 psi [11].

Response 16

This response stated that no masonry wall used reinforcement to resist compressive loads. Therefore, this question is not applicable.

Question 17

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 17

The Licensee provided details of wall modifications, which included the addition of structural steel braces and steel angles at the edges to reinforce the boundary conditions (see Appendix B for sketches of typical modifications). The original assumptions concerning boundary conditions (see Response 1) were not changed by the modifications, and since all walls, except cantilevered walls, were pinned at the boundaries, no significant out-of-plane drift effects are expected. Appendix B provides a summary of walls which required modifications.

Question 18 (from meeting of May 27, 1982)

Provide the number of modified walls which are not meeting working stress criteria.

Response 18 [Reference 8]

The Licensee stated that after the modifications were completed, 16 of the modified walls were qualified by the energy balance technique and not by the working stress criteria. The affected walls are: 311D, 3036, 3307, 3167, 3177, 3187, 3287, 5107, 2337, 4016, 4036, 4796, 4886, 4896, 3407, and 4647.

See the review of Response 11 for comments concerning the applicability of the energy balance technique to masonry walls.

Question 19 (from meeting of May 27, 1982)

Discuss the applicability of Branson's equation to calculate I_e, which was originally developed for reinforced concrete structures, to masonry walls. Applicability of this equation should be substantiated by comparison of results using this equation to the available results of a test program (i.e., recently completed SEAOCS tests).

Response 19 [Reference 8]

In this response, the Licensee substantiated the use of Branson's equation by comparing test results performed by J. C. Scrivener for wall deflections to calculated results.

Materials properties reported by J. C. Scrivener were used to calculate Ie (effective moments of inertia) using Branson's equation, which were in turn used to calculate deflections. A simply supported test wall was used for the comparison and the following table provides the results.

Test Load (lb/ft^2)	Test Deflection (in)	Calculated Deflection (in)
50	0.33	0.38
80	0.54	0.76
120	1.05	1.21
130	1.20	1.31

As can be seen from the above table, the deflections calculated using Branson's equation were consistently higher than the test deflections; therefore, the use of Branson's equation is conservative and appropriate.

Question 20 (from meeting on May 27, 1982)

Discuss the modulus of rupture values used as inputs to the "BLOCK WALLS" program. Indicate the types of analysis (i.e., horizontal or vertical strips) to which these values are applicable.

Response 20 [Reference 8]

For horizontal spans, the modulus of rupture was assumed to be zero. For frequency calculations of vertical spans, the concrete fill was the only material assigned a modulus of rupture. The modulus of rupture for fully grouted blocks was calculated as follows:

$f_r = 6\sqrt{f'c}$, where $f'_c = 2500$ psi (concrete fill)

$f_r = 300$ psi

This response is adequate and consistent with SGEB criteria.

Question 21 (from meeting on May 27, 1982)

There appears to be computation error in Appendix F of the November 1980 submittal [4] (the value of 1096.22 in^4 appears to be high for a cracked wall). Confirm that this is an illustration example only and does not represent any walls at your plant.

Response 21 [Reference 8]

The Licensee explained that the computer code "BLOCK WALLS" prints a statement if the wall cracks and does not print a statement if the wall does not crack; therefore, the value of 1096.22 in^4 for moment of inertia on page 9 of Appendix F [4] is correct, since the example wall does not crack. Also, there is a typographical error in Section 3.2 F of Appendix F: the input of the seismic moment in the equation for tension should be 24.79, not 26.79.

It was confirmed that the computation for the masonry wall in Appendix F [4] was an example and does not represent any walls at Davis-Besse Unit 1. This response is satisfactory.

Question 22 (from meeting on May 27, 1982)

Provide the details of the horizontal reinforcement in the walls and explain how it was treated in the analysis.

Response 22 [Reference 8]

The Licensee provided the following pattern of horizontal reinforcement:

8-inch thick walls

first 5 bed joints - 1 # 8 extra heavy Dur-O-Wal
sixth bed joint - 2 # 4 extra heavy Dur-O-Wal (placed side by side)

12-inch thick walls

first 5 bed joints - 1 # 12 extra heavy Dur-O-Wal
sixth bed joint - 2# 6 extra heavy Dur-O-Wal (placed side by side)

These patterns repeat for the full height of the wall.

Calculations made for comparison show that two Dur-O-Wals placed side by side result in larger section properties than a single Dur-O-Wal and, therefore, in higher natural frequencies and lower accelerations. For analyses using horizontal spans, only one Dur-O-Wal was assumed in each joint for the full height of the wall; this assumption will provide conservative results. The Licensee used a maximum allowable tensile stress of 30,000 psi for Dur-O-Wal.

This response is acceptable under the SGEB criteria.

Question 23 (from meeting on May 27, 1982)

Provide pre-modifications and post-modification calculations for wall No. 3167.

Response 23 [Reference 8]

The Licensee provided the requested calculations for wall 3167. Modifications to this wall consisted of steel braces which reduced the wall to a series of shorter spans (see Appendix C for further details). However, even with these modifications, the masonry reinforcement was overstressed in tension, under the working stress criteria. Therefore, the energy balance technique was used, with the modifications, to qualify wall 3167.

Question 24 (from meetings on June 21, 22, and 23, 1983)

Verify that for walls 3447, 3457, and 3467, the only safety system affected by wall failure would be the HVAC duct as assumed in the calculations.

Response 24 [Reference 12]

The Licensee indicated that further review verified the HVAC duct as the only wall attachment affected by walls 3447, 3457, and 3467. However, the HVAC duct is not nuclear safety-related; this is stated in the calculations for walls 3447, 3457, and 3467, but was misinterpreted during the meeting.

This response is satisfactory.

Question 25 (from meetings on June 21, 22, and 23, 1983)

Provide the basis for not performing a displacement/operability check for safety systems attached to wall qualified by the ductility values of less than 3.0.

Response 25 [Reference 12]

In this response, the Licensee stated that, if a deflection criterion of L/240 were used, a corresponding ductility ratio of 2.7 to 3.3 would be produced in masonry walls such as those found at Davis-Besse Unit 1. However, most walls at Davis-Besse Unit 1 that were analyzed by the energy balance technique were analyzed using the "BLOCK WALLS" progam which conservatively overestimates wall deflections. The Licensee felt that because of this conservatism a ductility criterion was more appropriate than a deflection criterion for walls accepted by the "BLOCK WALLS" program.

This response is satisfactory; however, see the review of Response 11 for comments concerning walls qualified by the energy balance technique.

Question 26 (from meetings on June 21, 22, and 23, 1983)

Justify the assumption of fixed boundary condition in some situations by examining a typical connection detail where this assumption is used. Assess the impact of joint flexibility on calculated results.

Response 26 [Reference 12]

The Licensee responded that, in a review of cantilevered walls (assumed fixed at the base) in which appropriate spring constants were used for the components of the boundary connections, it found that lower frequencies resulting from an increase in flexibility did not result in masonry or reinforcement stresses higher than the allowables. Also, the affected walls have one or more supported boundaries in addition to the fixed base that were not considered in the analysis.

A typical base connection consists of Phillips Red Head Self Drilling Anchors in the concrete slab, with 24-in-long threaded bars in the anchors, at each vertical reinforcement.

The Licensee concluded that an increase in flexibility would not affect the acceptance of cantilevered walls in which a fixed boundary was assumed.

Based on the information provided by the Licensee, this review concurs with the Licensee's conclusion: the assumption of a fixed boundary is reasonably appropriate.

3.2 EVALUATION OF LICENSEE'S APPROACH TO WALL MODIFICATIONS

According to Reference 6, 39 walls (including three walls mentioned in Response 24 of Section 3.1) did not meet the acceptance criteria. Eight of these walls will be removed and three were found acceptable upon further evaluation of the consequences of their failure (see Appendix B for further details). The remaining 28 walls were scheduled for structural modifications consisting primarily of the addition of external steel braces and the reinforcing of boundary connections with bolted angles. Reference 9 indicated that the modifications were scheduled for completion by November 1983.

Of the walls being modified, 16, were qualified by the energy balance technique after modifications. See Sections 3.1 and 4.0 for comments on the use of the energy balance technique to qualify masonry walls.

The Licensee provided sample calculations and details for wall modifications. The Licensee's approach to wall modifications has been reviewed and, with the exception of the use of the energy balance technique, has been found to be adequate and consistent with the SGEB criteria. See Appendix B for examples of wall modifications.

4. CONCLUSIONS

A detailed study was performed to provide a technical evaluation of the masonry walls at Davis-Besse Nuclear Power Station Unit 1. Review of the Licensee's criteria and additional information provided by the Licensee led to the conclusions given below.

The Licensee's criteria have been found technically adequate and in compliance with the SGEB criteria except for the following areas:

- o An increase of 25% in allowable stress was used for load combinations containing OBE loads. The following four walls required this increase in order to be qualified: 3016, 3198, 5137, and 4026. The SGEB criteria do not permit an increase in allowable stress where OBE loads are concerned; however, the Licensee employed several conservatisms in its computation of stress values that offset the increase factor for allowables. The Licensee used a constant response floor response spectra, equal to the peak response for natural frequencies on the low frequency side of the peak. Also, the walls in question were analyzed as one-way, simply supported beam strips. Because of the conservative stress values which these assumptions produce, it can be concluded that the Licensee's increase factor is acceptable and meets the intent of the SGEB criteria.

- o With regard to the energy balance technique, the following walls were affected: 2057, 2067, 2147, 2167, 2297, 2427, 3417, 5017, 5147, 5187, 5197, 4867, 6107, 1237, 3357, 1348, 2107, 2267, 3297, 3367, 2077, 1038, 1157, 3277, 4917, 5127, 5157, 5277, 6087, 2247, 2317, 2447, 3267, 5367, 305D, 307D, 313D, 1197, 2018, 2227, 2257, 2277, 3257, 3347, 3397, 2371, 304D, 1087, 1147, 2237, 2367, 3227, 4046, 1227, 1267, 1337, 1428, 2087, 2177, 311D, 3036, 3307, 3167, 3177, 3187, 3287, 5107, 2337, 4016, 4036, 4796, 4886, 4896, 3407, and 4647.

As stated in the review of Response 9 [7], FRC and its consultants have issued their assessment of the use of the energy balance technique in the analysis of masonry walls in nuclear power plants. The Structural and Geotechnical Engineering Branch (SGEB) has also issued a position statement on this subject, which will be addressed in its Safety Evaluation Report.

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APPENDIX A

SGEB CRITERIA FOR SAFETY-RELATED MASONRY WALL EVALUATION
(DEVELOPED BY THE STRUCTURAL AND GEOTECHNICAL ENGINEERING BRANCH
[SGEB] OF THE NRC)

FRANKLIN RESEARCH CENTER
DIVISION OF
ARVIN/CALSPAN
20th and Race Streets
Philadelphia, PA 19103

CONTENTS

1. General Requirements

The materials, testing, analysis, design, construction, and inspection related to the design and construction of safety-related concrete masonry walls should conform to the applicable requirements contained in Uniform Building Code - 1979, unless specified otherwise, by the provisions in this criteria.

The use of other standards or codes, such as ACI-531, ATC-3, or NCMA, is also acceptable. However, when the provisions of these codes are less conservative than the corresponding provisions of the criteria, their use should be justified on a case-by-case basis.

In new construction, no unreinforced masonry walls will be permitted. For operating plants, existing unreinforced walls will be evaluated by the provisions of these criteria. Plants which are applying for an operating license and which have already built unreinforced masonry walls will be evaluated on a case-by-case basis.

2. Loads and Load Combinations

The loads and load combinations shall include consideration of normal loads, severe environmental loads, extreme environmental loads, and abnormal loads. Specifically, for operating plants, the load combinations provided in the plant's FSAR shall govern. For operating license applications, the following load combinations shall apply (for definition of load terms, see SRP Section 3.8.4II-3).

(a) Service Load Conditions

(1) $D + L$

(2) $D + L + E$

(3) $D + L + W$

If thermal stresses due to T_o and R_o are present, they should be included in the above combinations as follows:

(1a) $D + L + T_o + R_o$

(2a) $D + L + T_o + R_o + E$

(3a) $D + L + T_o + R_o + W$

Check load combination for controlling condition for maximum 'L' and for no 'L'.

(b) Extreme Environmental, Abnormal, Abnormal/Severe Environmental, and Abnormal/Extreme Environmental Conditions

(4) $D + L + T_o + R_o + E$

(5) $D + L + T_o + R_o + W_t$

(6) $D + L + T_a + R_a + 1.5 P_a$

(7) $D + L + T_a + R_a + 1.25 P_a + 1.0 (Y_r + Y_j + Y_m) + 1.25 E$

(8) $D + L + T_a + R_a + 1.0 P_a + 1.0 (Y_r + Y_j + Y_m) + 1.0 E'$

In combinations (6), (7), and (8) the maximum values of P_a, T_a, R_a, Y_j, Y_r, and Y_m, including an appropriate dynamic load factor, should be used unless a time-history analysis is performed to justify otherwise. Combinations (5), (7), and (8) and the corresponding structural acceptance criteria should be satisfied first without the tornado missile load in (5) and without Y_r, Y_j, and Y_m in (7) and (8). When considereing these loads, local section strength capacities may be exceeded under these concentrated loads, provided there will be no loss of function of any safety-related system.

Both cases of L having its full value or being completely absent should be checked.

3. Allowable Stresses

Allowable stresses provided in ACI-531-79, as supplemented by the following modifications/exceptions, shall apply.

(a) When wind or seismic loads (OBE) are considered in the loading combinations, no increase in the allowable stresses is permitted.

(b) Use of allowable stresses corresponding to special inspection category shall be substantiated by demonstration of compliance with the inspection requirements of the SEB criteria.

(c) When tension perpendicular to bed joints is used in qualifying the unreinforced masonry walls, the allowable value will be justified by test program or other means pertinent to the plant and loading conditions. For reinforced masonry walls, all the tensile stresses will be resisted by reinforcement.

(d) For load conditions which represent extreme environmental, abnormal, abnormal/severe environmental, and abnormal/extreme environmental conditions, the allowable working stress may be multiplied by the factors shown in the following table:

Type of Stress	Factor
Axial or Flexural Compression[1]	2.5
Bearing	2.5
Reinforcement stress except shear	2.0 but not to exceed 0.9 fy
Shear reinforcement and/or bolts	1.5
Masonry tension parallel to bed joint	1.5
Shear carried by masonry	1.3
Masonry tension perpendicular to bed joint	
for reinforced masonry	0
for unreinforced masonry[2]	1.3

Notes

(1) When anchor bolts are used, design should prevent facial spalling of masonry unit.

(2) See 3(c).

4. Design and Analysis Considerations

(a) The analysis should follow established principles of engineering mechanics and take into account sound engineering practices.

(b) Assumptions and modeling techniques used shall give proper considerations to boundary conditions, cracking of sections, if any, and the dynamic behavior of masonry walls.

(c) Damping values to be used for dynamic analysis shall be those for reinforced concrete given in Regulatory Guide 1.61.

(d) In general, for operating plants, the seismic analysis and Category I structural requirements of FSAR shall apply. For other plants, corresponding SRP requirements shall apply. The seismic analysis shall account for the variations and uncertainties in mass, materials, and other pertinent parameters used.

(e) The analysis should consider both in-plane and out-of-plane loads.

(f) Interstory drift effects should be considered.

(g) In new construction, grout in concrete masonry walls, whenever used, shall be compacted by vibration.

(h) For masonry shear walls, the minimum reinforcement requirements of ACI-531 shall apply.

(i) Special constructions (e.g., multiwythe, composite) or other items not covered by the code shall be reviewed on a case-by-case basis for their acceptance.

(j) Licensees or applicants shall submit QA/QC information, if available, for staff's review.

In the event QA/QC information is not available, a field survey and a test program reviewed and approved by the staff shall be implemented to ascertain the conformance of masonry construction to design drawings and specifications (e.g., rebar and grouting).

(k) For masonry walls requiring protection from spalling and scabbing due to accident pipe reaction (Y_r), jet impingement (Y_j), and missile impact (Y_m), the requirements similar to those of SRP 3.5.3 shall apply. However, actual review will be conducted on a case-by-case basis.

5. <u>References</u>

(a) Uniform Building Code - 1979 Edition.

(b) Building Code Requirements for Concrete Masonry Structures ACI-531-79 and Commentary ACI-531R-79.

(c) Tentative Provisions for the Development of Seismic Regulations for Buildings - Applied Technology Council ATC 3-06.

(d) Specification for the Design and Construction of Load-Bearing Concrete Masonry - NCMA August, 1979.

(e) Trojan Nuclear Plant Concrete Masonry Design Criteria Safety Evaluation Report Supplement - November, 1980.

APPENDIX B

SUMMARY OF WALLS REQUIRING MODIFICATIONS

FRANKLIN RESEARCH CENTER
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Philadelphia, PA 19103

TABLE I - SUMMARY OF NON-ACCEPTABLE WALLS ANALYZED UNDER NRC BULLETIN NO. 80-11

Wall/Subwall	Load Causing Non-Acceptance	Wall Area or Item Causing Non-Acceptance	Summary of Corrective Action
311D	OBE	Upper half of east edge is not connected to any support.	Concrete pilaster will be added to provide support.
1068	SSE	Bottom connection of wall to floor.	Systems adjacent to this wall will not be affected due to postulated wall failure.
2047	SSE	Botton connection of wall to floor	Hangers 31-HCC-5-H5, 31-HCC-5-H6 and 31-HCC-5-H7 will be modified to remove attachment to wall.
2297	SSE	Top connection of wall (floor beam)	Floor beam will be braced.
2337	SSE	Bottom connection of wall to floor	Angles and expansion bolts will be used to reinforce connection.
3016(1)	OBE	Top connection to floor above	Fixing wall 3026 will also fix this wall.
3026(1)	OBE	Adjacent support walls (3016 & 3036) do not pass acceptance criteria	Top connection to floor above is missing and will be added.
3036(1)	OBE	Top connection to floor above	Fixing wall 3026 will also fix this wall.
3167	Compartment pressure due to a break in the main feedwater line	Masonry and top connection of wall (floor beam)	Steel bracing external to wall will be added.

TABLE I (Cont.)

Wall/Subwall	Load Causing Non-Acceptance	Wall Area or Item Causing Non-Acceptance	Summary of Corrective Action
3177	Compartment pressure due to a break in the main feedwater line	Masonry	Steel bracing external to wall will be added.
3187	Compartment pressure due to a break in the main feedwater line	Masonry and top connection of wall (floor beam)	Steel bracing external to wall will be added.
3237	Compartment pressure due to a crack in the main feedwater line	Top connection of wall (floor beam) and north edge connection to adjacent concrete wall	Floor beam will be braced and edge connection will be reinforced with angles and expansion bolts.
3287	Compartment pressure due to a crack in the main feedwater line	South edge connection to adjacent concrete wall	Connection will be reinforced with angles and expansion bolts.
3307	OBE	Top connection of wall (floor beam)	Floor beam will be braced.
3407	SSE	Top connection of wall (floor beam)	Floor beam will be braced.
3447(2)	OBE or compartment pressure caused by a crack in the main feedwater line	Masonry	Bracing external to wall will be added.
3457(2)	OBE or compartment pressure caused by a crack in the main feedwater line	Masonry	Bracing external to wall will be added.
3467(2)	OBE or compartment pressure caused by a crack in the main feedwater line	Masonry	Bracing external to wall will be added.

TABLE I (Cont.)

Wall/Subwall	Load Causing Non-Acceptance	Wall Area or Item Causing Non-Acceptance	Summary of Corrective Action
4016	OBE	Masonry, top connection to floor above and bottom connection to floor	Steel pilasters will be added.
4036	OBE	Masonry, top connection to floor above, bottom connection to floor, and east edge connection to adjacent masonry wall	Steel bracing external to wall will be added.
4107	Compartment pressure due to a crack in the main feedwater line	Masonry and all connections to adjacent supports	Steel bracing external to wall will be added.
4117	Compartment pressure due to a crack in the main feedwater line	Masonry and all connections to adjacent supports	Wall will be removed.
4127	Compartment pressure due to a crack in the main feedwater line	Masonry and all connections to adjacent supports	Wall will be removed.
4647	OBE	Seismic joint at south end of wall is missing	Required seismic joint will be added.
4786(3)	OBE	Masonry, top connection to floor above and bottom connection to floor	Steel bracing external to wall will be added.
4796(4)	OBE	Top connection to floor above	Steel bracing external to wall will be added.
4806	OBE	Seismic joint is missing	Wall will be removed.

TABLE I (Cont.)

Wall/Subwall	Load Causing Non-Acceptance	Wall Area or Item Causing Non-Acceptance	Summary of Corrective Action
4817	OBE	Adjacent support walls (4806 & 4826) do not pass acceptance criteria and masonry does not pass acceptance criteria	Wall will be removed.
4826	OBE	Seismic joint is missing	Wall will be removed.
4837	OBE	Masonry	Wall will be removed
4847	OBE	Masonry	Wall will be removed.
4857	OBE	Adjacent support wall (4847) does not pass acceptance criteria	Wall will be removed.
4886(4)	OBE	Top connection to floor above, bottom connection to floor and side connection to adjacent concrete wall	Steel bracing external to wall will be added.
4896(4)	OBE	Top connection to floor above	Steel bracing external to wall will be added.
4906(3)	OBE	Top connection to floor above	Steel bracing external to wall will be added.
5107	OBE	Top connection of wall (floor beam)	Floor beam will be braced.
5167	OBE	Masonry	Systems attached to wall have been examined for potential failure and failure is acceptable.
5207	SSE	Top connection of wall (floor beam)	Floor beam will be braced.

TABLE I (Cont.)

Wall/Subwall	Load Causing Non-Acceptance	Wall Area or Item Causing Non-Acceptance	Summary of Corrective Action
5257	OBE	Masonry	Systems attached to wall have been examined for potential failure and failure is acceptable.

Notes:

(1) Walls 3016, 3026 and 3036 have been analyzed as one wall unit.
(2) Walls 3447, 3457 and 3467 are one wall unit.
(3) Walls 4786 and 4906 are one wall unit.
(4) Walls 4796, 4886 and 4896 are one wall unit.

APPENDIX C

SKETCHES OF WALL MODIFICATIONS

FRANKLIN RESEARCH CENTER
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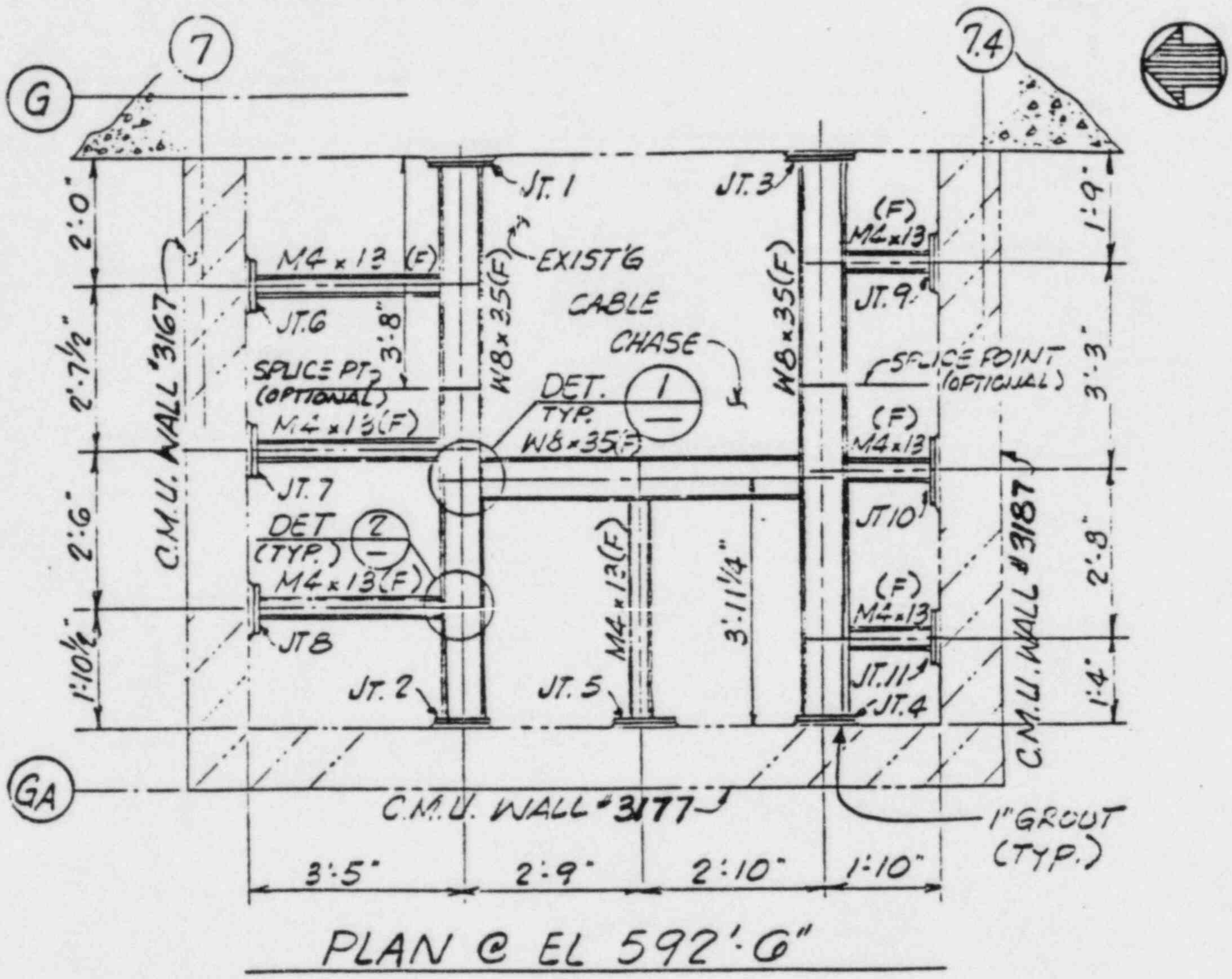
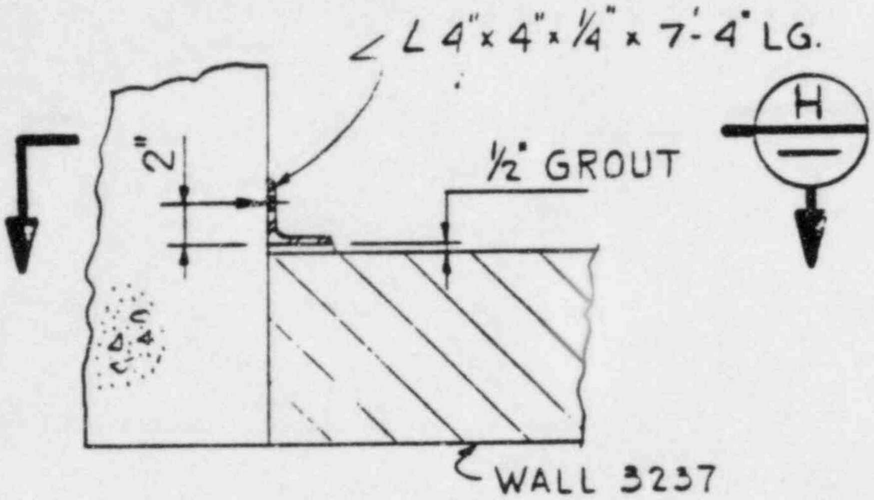
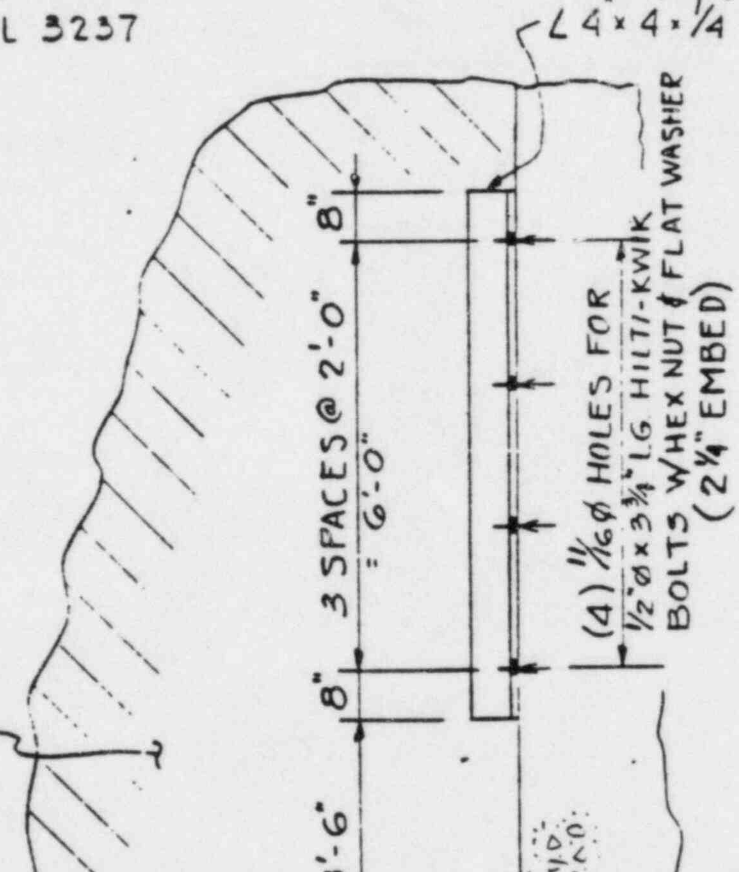


BRACES FOR WALLS 3167, 3177, and 3187



DETAIL 8



SECTION H

REINFORCEMENT OF BOUNDARY FOR WALL 3237

ATTACHMENT 3

SGEB STAFF POSITON ON USE OF ENERGY
BALANCE TECHNIQUE TO QUALIFY REINFORCED
MASONRY WALLS IN NUCLEAR POWER PLANTS

INTRODUCTION

Under seismic loads, strain energy transfer through elastic response is very small compared to the inelastic response for energy dissipation. Therefore, inelastic non-linear analysis of reinforced masonry walls is an attractive approach. Some of the licensees have relied on a non-linear analysis approach known as "energy-balance technique" to qualify some of the reinforced masonry walls in their plants.

The staff and their consultants have reviewed the basis provided by licensees to justify the use of energy-balance technique to qualify the reinforcd masonry walls. The staff met with a group of licesees representing approximately ten utilities on November 3, 1982 and January 20, 1983 to discuss this issue. Further, site visits and detailed review of design calculations were conducted by the staff and their consultants to gain first-hand knowledge of field conditions and the application of energy-balance technique in qualifying in-place masonry walls. Based on the information gained through the above activities, the staff has formulated the following position on the acceptability of the use of energy-balance technique to qualify reinforced masonry walls in operating nuclear power plants. The staff's technical basis for the position is discussed in the attached report.

POSITION

The use of energy-balance technique or any other non-linear analysis approach is not acceptable to the staff without further confirmation by an adequate test program. Therefore, the staff position consists of the following three options. Adoption of any one of the option and successful implementation will constitute a resolution of the issue regarding the qualification of reinforced masonry walls by energy balance technique or other non-linear techniques.

1. Reanalyse walls qualified by the energy-balance technique by linear elastic working stress approach as recommended in the staff acceptance criteria (SRP Section 3.8.4, Appendix A) and implement modifications to walls as needed.

2. Develop rigorous non-linear time-history analysis techniques capable of capturing the mechanism of the walls under cyclic loads. Different stages of behavior should be accurately modeled; elastic uncracked, elastic cracked and inelastic cracked with yielding of the central rebars. Then, a limited number of dynamic tests (realistic design earthquake motion inputs at top and bottom of the wall) should be conducted to demonstate the overall conservatism of the analysis results. In this case, "as built" walls should be constructed to duplicate the construction details of a specific plant.

3. For walls qualified by energy-balance technique, conduct a comprehensive test program to establish the basic non-linear behavioral characteristics of masonry walls (i.e. load-deflection hysteretic behavior, ductility ratios, energy absorption and post yield envelopes) for material properties and construction details pertaining to masonry walls in question. The

behavior revealed from tests should then be compared with that of elastic-perfectly-plastic materials for which the energy balance technique was originally developed. If there are significant differences, then the energy balance technique should be modified to reflect the actual wall behavior.

EVALUATION OF THE APPLICABILITY OF NONLINEAR ANALYSIS TECHNIQUES TO REINFORCED MASONRY WALLS IN NUCLEAR POWER PLANTS

Prepared by

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Ahmad A. Hamid (1)
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August 1984

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INTRODUCTION

In response to IE Bulletin 80-11, a total of 10 nuclear power plants have indicated that the energy balance technique has been employed to qualify some reinforced masonry walls in out-of-plane bending. Based on the review of submittals provided by the licensees and all available literature, the Franklin Research Center (FRC) staff and FRC consultants have concluded that the available data in the literature is not sufficient to warrant the use of nonlinear analysis techniques to predict the response of masonry walls under cyclic, fully reversed dynamic loading. As a result, a meeting with representatives of the affected plants was held at the NRC on November 3, 1982 so that the NRC, FRC staff and FRC consultants could explain their concern regarding the applicability of the energy balance technique to masonry walls in nuclear power plants [1]. In a subsequent meeting on January 20, 1983, consultants of utility companies presented their rebuttals [2] and requested that they should be treated on a plant-by-plant basis. In accordance with their requests, the NRC staff started the process of evaluating each plant on an individual basis. In this process, the NRC, FRC staff and consultants visited a few nuclear power plants to examine the field conditions of reinforced masonry walls in the plants and to gain first-hand knowledge of how the energy balance technique is applied to actual walls. Key calculations were reviewed with regard to the energy balance technique.

EVALUATION OF ENERGY BALANCE TECHNIQUE

Based on a review of the submittals provided by the licensees, specific plant visits, evaluation of typical design computations and review of all available literature, it is concluded that the concerns raised by the Franklin Research Center (FRC) staff and consultants pertaining to the use of energy balance technique have not been resolved. A summary of these concerns are listed below:

1. Only a few isolated tests have been reported on the lateral resistance of reinforced concrete block and brick masonry walls in out-of-plane bending. These tests can be summarized as follows:

(i) Tests have been conducted on 20' high reinforced concrete block walls 8" thick in running bond and stack bond configurations by Dickey and Mackintosh [3]. These tests, although limited, revealed that, under monotonically increasing load, some of the panels failed in a brittle mode prior to reaching yield and that the stack bond was less effective than the running bond.

(ii) More recent tests conducted by the ACI-SEASC Task Committee on Slender Walls [4] on face loaded 24' high reinforced masonry walls under monotonically increasing load showed relatively low ductility ratios in the 3 panels that attained failure. Two 6" nominal fully grouted concrete masonry walls attained ductility ratios of approximately 2 when they failed inadvertently in compression. One 6" hollow brick wall tested to failure also attained a ductility ratio of approximately 2. It has been noted that walls tested were fully grouted and have high steel percentages (0.22% to 0.37%).

(iii) Tests conducted by Scrivener [5,6] on face loaded reinforced masonry walls made of 4 1/4" reinforcing brick revealed high ductilities. The one cyclicallly loaded panel whose load-deflection results are reported [5] revealed very peculiar hysteretic behavior unlike the required elasto-plastic behavior needed for application of the energy balance technique.

(iv) Tests on small masonry structures resulting from an assembly of various components to form single story masonry homes have been carried out at the UC, Berkeley earthquake simulator [7]-[9]. The main objective was to provide design recommendations on the minimum reinforcement required for masonry housing in seismic zone 2. These are the only tests of reinforced masonry walls under realisitc earthquake loads. The reinforced walls tested under out-of-plane bending in this program did not yield under the applied loads. In addition, these walls did not have the boundary conditions of typical applications of masonry walls in nuclear power plants.

(v) Dynamic tests on slender reinforced block masonry walls have been conducted at the EERC, University of California, Berkeley for Bechtel Power Corporation. The program has been conducted to demonstrate the conservatism of the nonlinear dynamic analysis performed by Computech Engineering Services for the masonry walls in the San Onofre Nuclear Generating Station, Unit 1 (SONGS-1). The FRC staff and consultants witnessed one of the tests. It was shown that the wall was capable of resisting significant inelastic deformations when subjected to earthquake input motion. It has to be mentioned, however, that the few tests performed were plant specific and aimed at verifying the conservatism of the nonlinear dynamic analysis technique developed by Computech Engineering Services. Consequently, the parameters included in the program were limited to "as built" condition of the walls in SONGS-1. The program objective was not to verify the use of the energy balance technique.

The above tests that have been conducted on reinforced masonry walls and which are relevant to the evaluation of concrete masonry walls in nuclear power plants do not form a sufficient data base to warrant the use of the energy balance technique.

2. A Technical Coordinating Committee for Masonry Research (TCCMAR) has been formed under the auspises of the US-Japan Cooperative Research Program. It is a recognition of the urgent need for research in the area of seismic resistance of masonry. The committee met in Pasadena in February 1984 to assess the current state of knowledge and to outline an experimental program to provide the necessary data. It has been concluded that the current state-of-the-art of masonry has not progressed enough to

warrant inelastic analysis methodology of masonry structures [11]. A comprehensive test program was recommended. This significant undertaking is a clear indication of the lack of test data available for masonry. (Note: Dr. Hamid serves as a member of TCCMAR.)

3. A large number of variables exist in the construction of concrete block walls used in nuclear power plants. For example, the walls can be fully grouted, partially grouted, stack bond, running bond, single and multiple wythes with different block sizes ranging from 4" to 12" in width. No adequate test data exist in the literature to enable a clear understanding of the effects of these variables on the dynamic fully reversed cyclic behavior of masonry walls.

4. Effects of cut-outs and eccentric loads due to attachments on reinforced concrete masonry walls of the type used in nuclear power plants have not been evaluated experimentally. This type of information, when available, will help to substantiate the various assumptions made in the analysis of such safety related walls.

5. The limited tests that have been conducted and summarized in item 1 above have pointed out to the inability to preclude brittle type failures with low ductility ratios on face loaded panels under monotonically increasing load. A lack of knowledge exists on the maximum attainable compressive strains in the face shell of reinforced concrete masonry walls under out-of-plane bending. This is particularly true under cyclic dynamic loading.

6. In examining the available test data, it is also obvious that there is a significant lack of information about the post-yield envelope and established cyclic load characteristics for reinforced concrete masonry walls under out-of-plane bending which is essential to demonstrate the stable ductile behavior required for the applicability of the energy balance technique. This is attributed to the fact that most tests were not conducted to ultimate failure which is essential for the determination of the post-yield envelope. This deficiency exists for all of the types of masonry construction used in nuclear power plants [10].

7. Some walls are qualified based on one-way bending in the horizontal direction or two-way plate action. These walls are horizontally reinforced with joint reinforcement embedded in the mortar joints every course or every other course. This type of steel is a high tensile steel with a yield stress as high as 100,000 psi indicating a very limited ductility. Masonry codes are not specific about the usefullness of joint reinforcement, particularly in seismic areas [12,13]. If joint reinforcement is to be used to resist tensile stresses, the WSD method should be employed with an allowable steel stress limited to 30,000 psi. The only code [14] that addresses the use of joint reinforcement in seismic areas for categoriees C and D structures was developed by the Applied Technology Council. This code does not allow the use of joint reinforcement as a load carrying element for these two categories.. Safety-related masonry walls in nuclear power plants would fit into these categories. Information about the

cyclic behavior of joint reinforced masonry walls is not available in the masonry literature at the present time [12,13].

8. The energy balance technique has been originally developed as an approximate design tool to check the resistance of ductile concrete and steel frame buildings subjected to seismic loads. With the fast development in computers in recent years, more rigorous nonlinear dynamic analyses of ductile structures have also been made possible.

NONLINEAR ANALYSIS OF MASONRY WALLS

Under seismic loads, strain energy transfer through elastic reponse is very small compared to the inelastic response for energy dissipation. With regard to inelastic behavior, two methods have been used to investigate the dynamic response of concrete and steel structures to a strong motion earthquake. One of the methods requires the formulation of an inelastic model of the structure utilizing the finite element technique. The model is then subjected to time-history ground motion and the dynamic response is determined. The results of this approach, which is time consuming and costly, depends on how accurately the structure is represented by the inelasctic model and how well the material properties are defined. Therefore, a limited confirmatory dynamic test program should be conducted to check the conservatism of the assumptions used.

The other method, which is easier to apply in a design office, separates the properties of the structure from those of the earthquake. The earthquake is represented by a response spectrum which is then modified to accomodate the inelastic or ductile response of the wall [15]. This method which relies on the energy balance technique requires information about ductility and energy absorbtion capability of masonry walls which, as discussed previously, have not been demonstrated experimentally for general applications. A ductility factor of 1 or 1.5 is suggested [16] for damage-level earthquake intensities where as ductilities of 2 to 3 is recommended [16] for use with collapse-level response spectra. Because the energy balance technique is an approximate simplified method, an adequate and more comprehensive data base should be generated to check this design methodology.

TEST PROGRAM RELATED TO ENERGY BALANCE TECHNIQUE

If a confirmatory test program is elected to justify the use of the energy balance technique, it is expected that the test panels should represent the actual configuration, construction details and boundary conditions of masonry walls in nuclear power plants.

The test program should cover the different parameters that would affect wall performance such as steel percentage, bond type, partial grouting and block size.

The test objectives should be centered upon the following:

1. To demonstrate that the masonry walls would maintain their structural and functional integrity when subjected to SSE and other applied loads.

2. To demonstrate that a stable ductile behavior characterized by steel yielding is guaranteed and that any

brittle failure (e.g. crushing) is precluded.

3. To develop necessary information to verify the energy balance technique as a methodology for the qualification of reinforced masonry walls in nuclear power plants.

4. To demonstrate that adequate margins of safety exist for walls subjected to design lateral loads.

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

A review and evaluation of the available information on the nonlinear behavior of block masonry walls under out-of-plane loading has been presented. It is concluded that test data are needed to substantiate the use of nonlinear analysis techniques to qualify reinforced block walls in nuclear power plants.

To qualify masonry walls based on nonlinear analysis, two alternatives are recommended:

1- Develop rigorous nonlinear time-history analysis techniques capable of capturing the mechanism of the walls under cyclic loads. Different stages of behavior should be accurately modeled: elastic uncracked, elastic cracked and inelastic cracked with yielding of the central rebars. Then, a limited number of dynamic tests (realistic design earthquake motion inputs at top and bottom of the wall) should be conducted to demonstrate the overall conservatism of the analysis results. In this case, "as built" walls should be constructed to duplicate the construction details of a specific plant.

2- Conduct a comprehensive test program to establish the basic nonlinear behavioral characteristics of masonry walls (ie. load-deflection hysteretic behavior, ductility ratios, energy absorbtion and post-yield envelopes) for material properties and construction details pertaining to masonry walls in question. The behavior revealed from the tests should then be compared with that of elastic-perfectly-plastic materials for which the energy balance technique was originally developed. If there are significant differences, then the energy balance technique should be modified to reflect the actual wall behavior.

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