ATTACHMENT 1

FRANKLIN RESEARCH CENTER

DIVISION OF ARVIN/CALSPAN

MASONRY WALL DESIGN

IOWA ELECTRIC LIGHT AND POWER COMPANY
DUANE ARNOLD ENERGY CENTER

TER-C5506-162

TECHNICAL REPORT

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TECHNICAL EVALUATION REPORT

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MASONRY WALL DESIGN

IOWA ELECTRIC LIGHT AND POWER COMPANY
DUANE ARNOLD ENERGY CENTER

TER-C5506-162

Prepared for

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FRANKLIN RESEARCH CENTER
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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.

1. INTRODUCTION

1.1 PURPOSE OF REVIEW

The purpose of this review is to provide a technical evaluation of the Licensee response to IE Bulletin 80-11 [1] with respect to compliance with the Nuclear Regulatory Commission (NRC) masonry wall criteria. In addition, if the Licensee plans repair work on masonry walls, the planned methods, procedures, and repair schedules are 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, Iowa Electric Light and Power Company provided the NRC with documents [2, 3] describing the status of masonry walls at Duane Arnold Energy Center. The information in these documents was reviewed, and a request for additional information was sent to the Licensee on December 29, 1981, to which the Licensee responded [4]. Additional questions were raised during a conference call between NRC and Iowa Electric representatives on May 17, 1982. The Licensee also responded to these questions [5]. This response prompted another set of questions which were sent to the Licensee

on February 22, 1984, and the Licensee provided its response [6]. The Licensee also provided its final response regarding the joint reinforcement issue [7].

According to Reference 5, the Licensee reviewed 445 masonry walls, out of which 295 were safety-related. The functions of these walls include partitioning, radiation shielding, fire protection, and missile protection. The walls also support piping, conduit, and instrumentation.

The walls under consideration at the Duane Arnold plant are typically reinforced horizontally and vertically. The cells containing reinforcing in non-shield walls are filled with grout, and all cells in shield walls are filled with grout. Both single- and multi-wythe construction are found at the plant. The following materials and material properties were used in construction:

- o masonry - ASTM C90, Grade PI, $f'm = 2000$ psi
- o mortar - ASTM C476, Type PL, $m_o = 2000$ psi
- o reinforcing steel - $fy = 60,000$ psi
- o joint reinforcing - Extra-Heavy Dur-O-Wal, $fy = 60,000$ psi

In its reevaluation of masonry walls, the Licensee found that no modifications to the walls were necessary since all safety-related walls were qualified under the Licensee's reevaluation criteria.

The Licensee has relied upon the energy balance technique to qualify five masonry walls.

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 [10]. In a subsequent meeting on January 20, 1983, consultants of utility companies presented their rebuttals [11] 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 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.

Joint reinforcement was used in resisting the tension in masonry walls. The Licensee indicated that the stresses in joint reinforcement of 57 walls exceeded 30 ksi (30 to 54 ksi).

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 [8], and ACI 531-79 [9].

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 9 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, 3] and subsequent responses [4, 5, 6, 7] 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 are based on ACI 531-79 [9].
- o The working stress design method and energy balance technique were used to qualify the walls. Of 295 safety-related walls, 5 have been qualified by the energy balance technique.
- o Joint reinforcement is used to qualify reinforced walls.
- o Damping for uncracked sections:

  2% for design basis earthquake (DBE)
- o Damping for cracked reinforced sections:

  7% for DBE
- 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 or two-way action assumption
  - obtain inertial loading from the floor response spectrum at either the bottom of the wall or the next higher elevation, whichever yields the highest response for the fundamental frequency. Apply this response over the entire area of the wall.
  - compute stress.
  - compare computed stresses with the allowable values.
- o A typical analytical procedure used in the energy balance technique is summarized below:
  - assume the wall to be a single degree of freedom system.

- assume the wall has an elastic-(perfectly)-plastic response.
- determine an equivalent elastic response to represent the inelastic response of the wall.
- determine wall maximum displacement.
- if displacement exceeds 3 times the yield displacement, multiply by 2 and check to see if it affects any safety-related items
- compare displacement with allowable of 5 times the yield displacement
- check compressive stress of masonry and compare with the allowable of 0.85 f'm.

The Licensee's criteria [5] and responses [4, 5, 6, 7] have been reviewed. With the exception of those items 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 a review of the Licensee's responses [4] to the NRC's original questions, as well as the responses [5, 6, 7] to all subsequent questions. Questions and responses covering the same topic have been combined.

Question 1

With regard to the material strength, identify the type of masonry and mortar used and justify their compressive strength (f'm = 2000 psi and m_o = 2000 psi [5]) by providing test results.

Response 1

The Licensee identified the masonry as ASTM C90, Grade PI hollow units, and the mortar as ASTM C476, Type PL. The Licensee also provided test results [6] that show that the masonry conforms to the specifications of ASTM C90 and support the values given in Reference 5 for f'm and m_o.

A summary of the test results is provided below:

o Concrete Block Unit Tests

The three compression tests for 12 in x 8 in x 16 in concrete blocks yielded the average compressive strength of 4705 psi. For 8 in x 8 in x 16 in concrete blocks, two compression tests were performed, and the average compressive strength was determined to be 4966 psi.

o Mortar Compression Tests

Thirty-four mortar compression tests were performed for samples taken from turbine building, reactor building, off-gas building, and control building block walls. These test results indicated that the minimum mortar compressive strength was 2220 psi and the maximum was 6070 psi.

In reviewing the Licensee's compression test results, the following conclusions were made:

- The mortar compressive strength of ASTM C476 Type PL used in the construction of reinforced masonry has the equivalent compressive strength of Type M or S mortar, and the specified mortar compressive strength of m_o = 2000 psi is judged to be adequate since the minimum mortar compressive strength was determined to be 2220 psi from the tests.

- The specified compressive strength of concrete masonry f'm = 2000 psi is judged to be adequate since ACI 531-79 Table 4.3 [9] allowed f'm to be 2000 psi for compressive test strength of masonry units of 4000 psi (compared to average strength from tests of 4705 and 4966 psi). Therefore, the Licensee's response is considered technically adequate and in compliance with the SGEB criteria.

Question 2

In Section 5.1.1 [5], the allowable shear or tension stresses at the concrete core/block wythe interface were stated to be 8 psi. Provide technical justification for this value.

Response 2

The allowable shear and tension value of 8 psi for core/block wythe interface referred to multi-wythe walls having a grouted core less than 3 inches thick. However, this value was not used to qualify any walls. Walls with collar joints between wythes were treated as if each wythe responds independently.

This response is adequate and in compliance with the SGEB criteria.

Question 3

With regard to shear and bond stresses for factored loads, a factor of 1.67 was introduced in Section 5.2.1 [5]. SGEB criteria (Appendix A) allow a factor of 1.3 for shear carried by masonry. Justify the use of a factor of 1.67.

Response 3

In this response, the Licensee indicated that code allowable stresses [8] were generally associated with a factor of safety of 3 and that a 1.67 increase in allowable would still provide a factor of safety of (3/1.67) = 1.8.

The Licensee stated that shear and bond stresses were not critical in masonry wall analysis, and since all walls have horizontal and vertical reinforcement to resist all tension forces in those directions, the discrepancy between the Licensee's factor and the SGEB factor is considered insignificant. The Licensee's response is satisfactory.

Question 4

> With regard to tension stress, a factor of 1.67 was introduced in Section 5.2.1 [5] for factored loads. Indicate if this factor is used for tension normal or parallel to the bed joint. SGEB criteria allow a factor of 1.3 for masonry tension perpendicular to the bed joint (for unreinforced masonry) and a factor of 1.5 for masonry tension parallel to the bed joint. In view of this, provide justification for the factor of 1.67.

Response 4

The Licensee responded that all tension normal or parallel to the bed joints was taken by reinforcing steel in all masonry walls at the plant. This means that the assumed tensile strength of mortar and masonry was zero and that the tension factor, 1.67, was not used in the analysis of masonry walls.

This response is adequate and acceptable.

Question 5

> In Section 5.2.1 [5], the Licensee discussed the stress values used for walls without inspection. Indicate if any walls at the Duane Arnold plant fall into this category.

Response 5

The Licensee responded that none of the walls at the Duane Arnold plant fall under the category of walls without inspection. The quality of the walls

was controlled by inspection during construction. Also, the specified compressive strengths for mortar and masonry concrete block were confirmed by tests performed by the Licensee as discussed in Response 1.

The concern regarding allowable stresses for walls without inspection has been satisfactorily resolved.

Question 6

With regard to the in-plane strain allowable for nonshear walls, provide the technical basis for the value used in Section 5.1.3 [5] for the unconfined walls.

Response 6

In this response, the Licensee indicated that the allowable in-plane strain, 0.0001, was based on work by Becica [13] and Fishburn [14] and that none of the masonry walls were required to resist building shear or moment.

This question has been resolved satisfactorily.

Question 7 [This question was raised in References 4, 5, and 6]

Identify the walls which were qualified with an inelastic analysis technique.

Response 7

In Reference 4, the Licensee indicated that the energy balance technique was the only alternate to the working stress technique used to qualify masonry walls at the Duane Arnold plant. In References 5 and 6, the Licensee stated that the energy balance technique was involved in the analysis of walls 200-7, 200-8, 417-25, 412-9, and 412-13. Walls 412-14, 412-17, and 412-18 have been reevaluated and qualified with the working stress design method. The response [5] to the conference call of May 17, 1982 also provided the following details about walls 200-7, 200-8, 417-25, 412-9, and 412-13:

- o applicable loading combinations
- o wall reinforcement and size of walls
- o safety systems and equipment on or near the walls

- o maximum deflections using energy balance technique
- o displacement ductility ratios.

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.

FRC and its consultants have issued their evaluation and assessment of the use of the energy balance technique for masonry walls [10, 12]. The Structural and Geotechnical Engineering Branch (SGEB) has issued a position statement regarding this subject which will be addressed in its Safety Evaluation Report.

Question 8

With regard to damping, Nuclear Regulatory Guide 1.61 [15] allows 4% for reinforced concrete subject to the operating basis earthquake (OBE). Justify the use of 5% damping for OBE, as stated in Section 5.3.1 [5].

Response 8

In this response, the Licensee stated that the safe shutdown earthquake (SSE) governed the analysis of all masonry walls. The 5% damping associated with OBE was not used in qualifying any walls since the OBE load was not a governing load.

This response is satisfactory.

Question 9 [This question was raised in References 4 and 5]

With respect to modes of vibration that are higher than the fundamental mode, indicate how higher mode effects are accounted for. Provide sample calculations to show that the first mode accounts for 99% of the total response.

Response 9

The Licensee stated that two dynamic analyses were carried out for one wall using the Bechtel Structural Analysis Program CE 800. One analysis was based on the fundamental mode only; the other was based on the first 10 modal

responses, combined by the square root of the sum of the squares method. The Licensee reported that the shears and moments from the two analyses were very close and that the shear and moment using only the first mode were about 99% of the 10-mode analysis.

According to the Licensee, the wall response not covered by the first mode is accounted for by the conservatisms in the wall analysis technique: an envelope of the floor response spectra at the top and bottom of the wall was used to determine inertial response of the wall.

It has been observed in other plants that the first mode normally contributes 95% or more of the total responses.

This response is adequate and satisfies the SGEB criteria.

Question 10

With regard to seismic analysis, indicate how the components of seismic load in various directions are accounted for.

Response 10

The Licensee responded that the maximum seismic acceleration response was always applied in a direction perpendicular to the wall, regardless of the orientation of the wall. Since none of the walls resists building shear or moment (see Response 6), the seismic components in the vertical and horizontal in-plane directions will be insignificant compared to the perpendicular component.

The Licensee's response is adequate and satisfies the intent of the SGEB criteria.

Question 11

Indicate how pipe and equipment loads are accounted for.

Response 11

In this response, the Licensee indicated that concentrated loads from wall attachments were determined by multiplying the weight of the attachment by the peak acceleration from the floor response envelope of the spectra at the bottom of the wall and the next higher elevation. In addition to these concentrated loads, the attachment weights were added to the uniform dead load of the wall.

This response is adequate and satisfies the SGEB criteria.

Question 12

With respect to the load combinations, the Licensee's submittal [3] did not provide any factor greater than 1.0 for components of the combinations. Explain and justify this deviation from the plant's FSAR.

Response 12

The Licensee indicated that the reinforced concrete design in the FSAR was based on the ultimate strength method which uses factored loads to maintain margins of safety. Masonry walls, however, were designed with the working stress method, which does not use factored loads, but maintains strength margins through the factors of safety included in the allowable stresses.

This response is satisfactory and consistent with the SGEB criteria.

Question 13

Discuss how the value of Young's modulus was selected for various calculations.

Response 13

The Licensee indicated that Young's modulus was taken from ACI 531-79, Table 10.1 [9] which recommended a value of 1000 f'm for masonry wall. The compressive strength used in the evaluation of masonry walls was chosen to be f'_m = 2000 psi, which was justified by the compression tests as discussed in Response 1. Therefore, the Licensee's response is satisfactory and in compliance with the SGEB criteria.

Question 14

With reference to the reinforcement in masonry walls, the ACI 531-79 Code [9] specifies that the minimum area of reinforcement in a wall in each direction, vertical or horizontal, shall be 0.0007 (0.07%) times the gross cross-sectional area of the wall and that the minimum total area of steel, combined vertical and horizontal, shall not be less than 0.002 (0.2%) times the gross cross-sectional area. Clarify whether the reinforced walls at this plant meet the above requirements. It should be noted that the horizontal reinforcement is installed to satisfy the minimum reinforcement requirement for a reinforced wall.

Response 14

In this response, the Licensee provided the percentages of reinforcement in the vertical and horizontal directions for masonry walls of various thicknesses. The smallest percentage of vertical reinforcement listed was 0.116%, and the smallest percentage of horizontal reinforcement listed was 0.081%. The smallest percentage of total reinforcement was 0.203%. These values are greater than the minimum limits specified in ACI 531-79; therefore, the Licensee's response satisfies the SGEB criteria.

Question 15 [6, 7]

a. If the joint reinforcement is used to resist tension in the walls meeting the above minimum requirements, it should follow the working stress design method which limits its allowable to 30 ksi. Please clarify whether this requirement has been satisfied. If this requirement is not satisfied, identify all affected walls along with the calculated stress value for each wall and indicate specific actions planned to correct this situation.

b. For those walls in which the calculated stress exceeded 40 ksi, please provide the following information for each wall: calculated stress, wall's dimensions and thickness, type and spacing of reinforcement, and connection details at the boundary. Also, provide sample calculations (with necessary explanation to make them understandable) illustrating the analytical procedures used in obtaining the stress values.

c. Provide the stress-strain relationship of the type of joint reinforcement used in the plant. If test data from the manufacturer are not available, it is recommended that some simple tests be conducted to obtain this relationship.

Response 15

a. The Licensee stated that joint reinforcement was used to resist tension in the analysis. Stress in the joint reinforcement was limited to 24 ksi under normal load conditions and was increased up to 0.9 fy for the severe extreme loading cases.

b. The Licensee provided the calculated stresses in the horizontal reinforcement along with the wall dimensions, thickness, the type and spacing of reinforcement, and the boundary conditions used for all walls having calculated stress higher than 40 ksi. Typical construction details are illustrated in Figures 1 and 2. Sample calculations were also provided for review.

c. The Licensee provided stress-strain relationships for plain and deformed wire obtained from tests (manufacturers' data [16, 17]). These curves are given in Figures 3 and 4. The curves shown in Figures 3 and 4 represent the joint reinforcement types found at the plant. Table 1 summarizes the data in these figures.

The current NRC staff acceptance criteria (SGEB criteria) do not approve the use of joint reinforcement as a tensile-resisting element in unreinforced masonry walls (walls without vertical rebars). For reinforced walls meeting the minimum reinforcement requirements of ACI 531-79 codes, the joint reinforcement can be used as a tensile-resisting element. However, the analysis should follow the working stress design method, and stresses in joint reinforcement should remain within 30 ksi.

The major concern assoicated with joint reinforcement is the lack of applicable test data to determine ductility, bond and anchorage capacity, strength degradation (due to cyclic, dynamic loadings) of joint reinforcement.

Based on the information provided by the Licensee, it was learned that there are a significant number of walls qualified using horizontal reinforcement as the tensile-resisting element. A total of 57 walls had stresses in excess of 30 ksi, of which 20 walls has stresses between 30 ksi and 40 ksi, 24 walls has stresses between 40 ksi and 50 ksi, and 3 walls had stresses higher than 50 ksi (one wall at 52 ksi and two walls at 54 ksi). All of these walls were constructed with bond beams at every 4 feet and the bond beam has four No. 4 rebars. In addition, joint reinforcement was installed at every course or every other course. The vertical reinforcement consists of

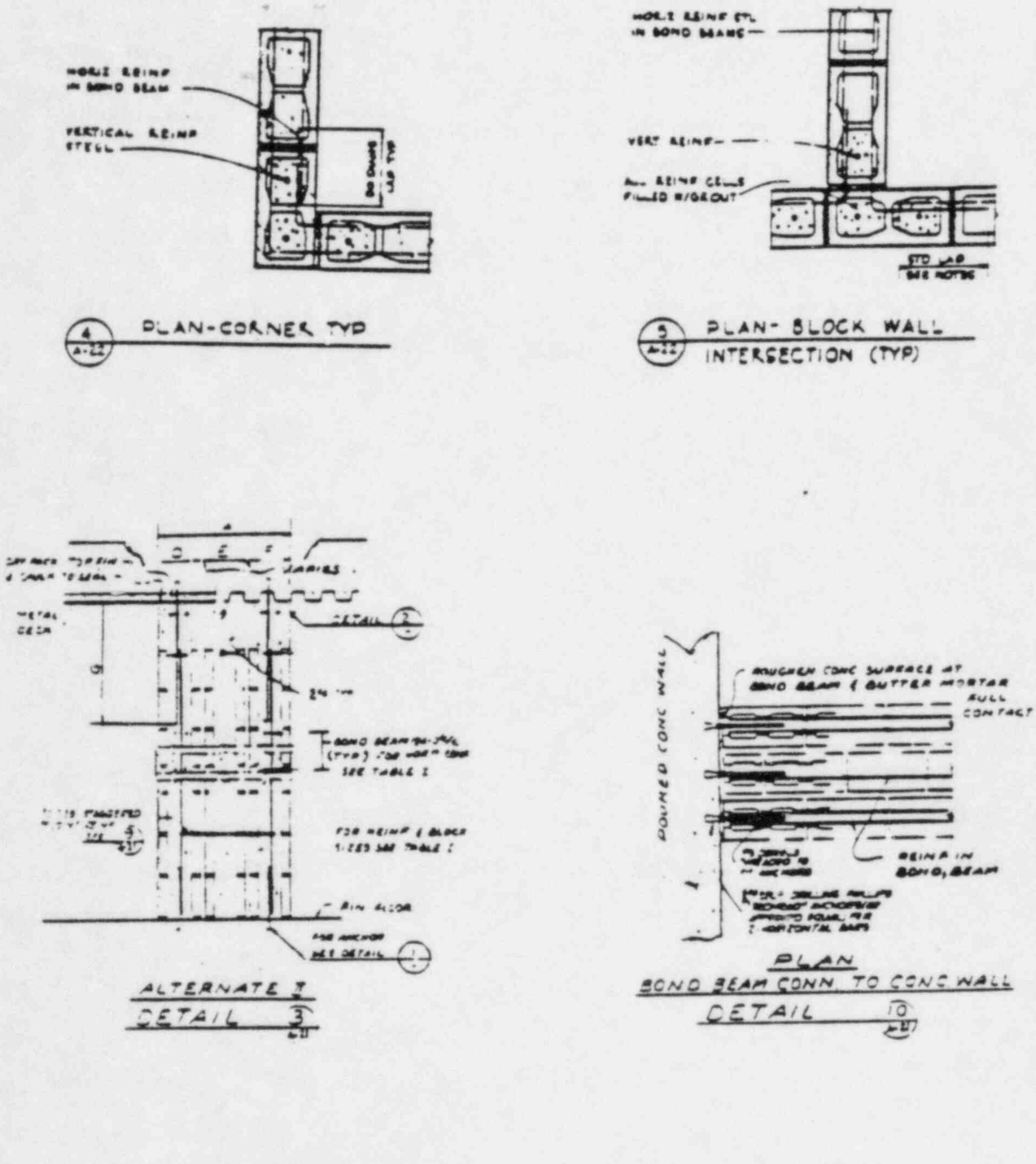

Figure 1. Typical Block Wall Support Details

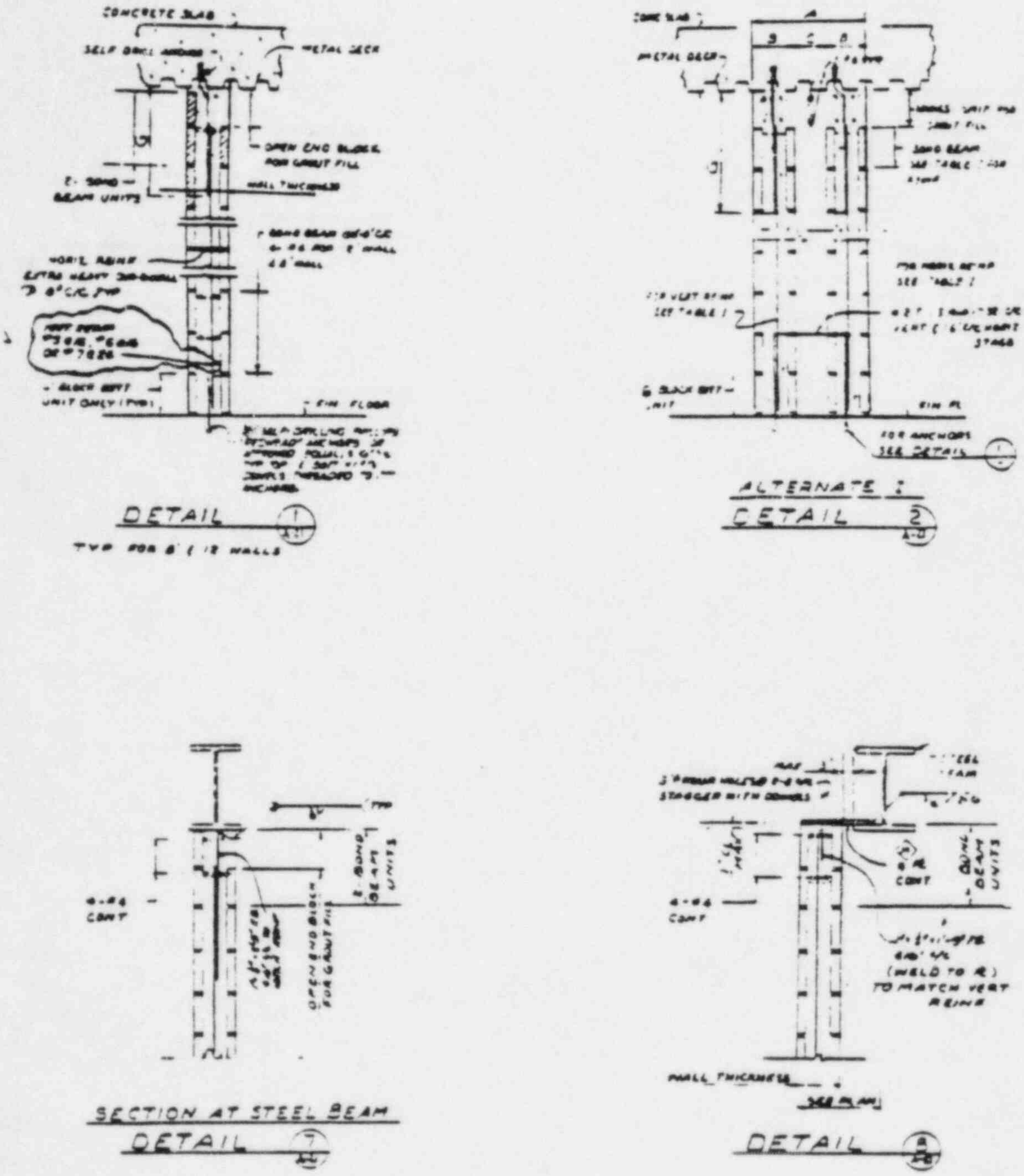

Figure 2. Typical Block Wall Support Details

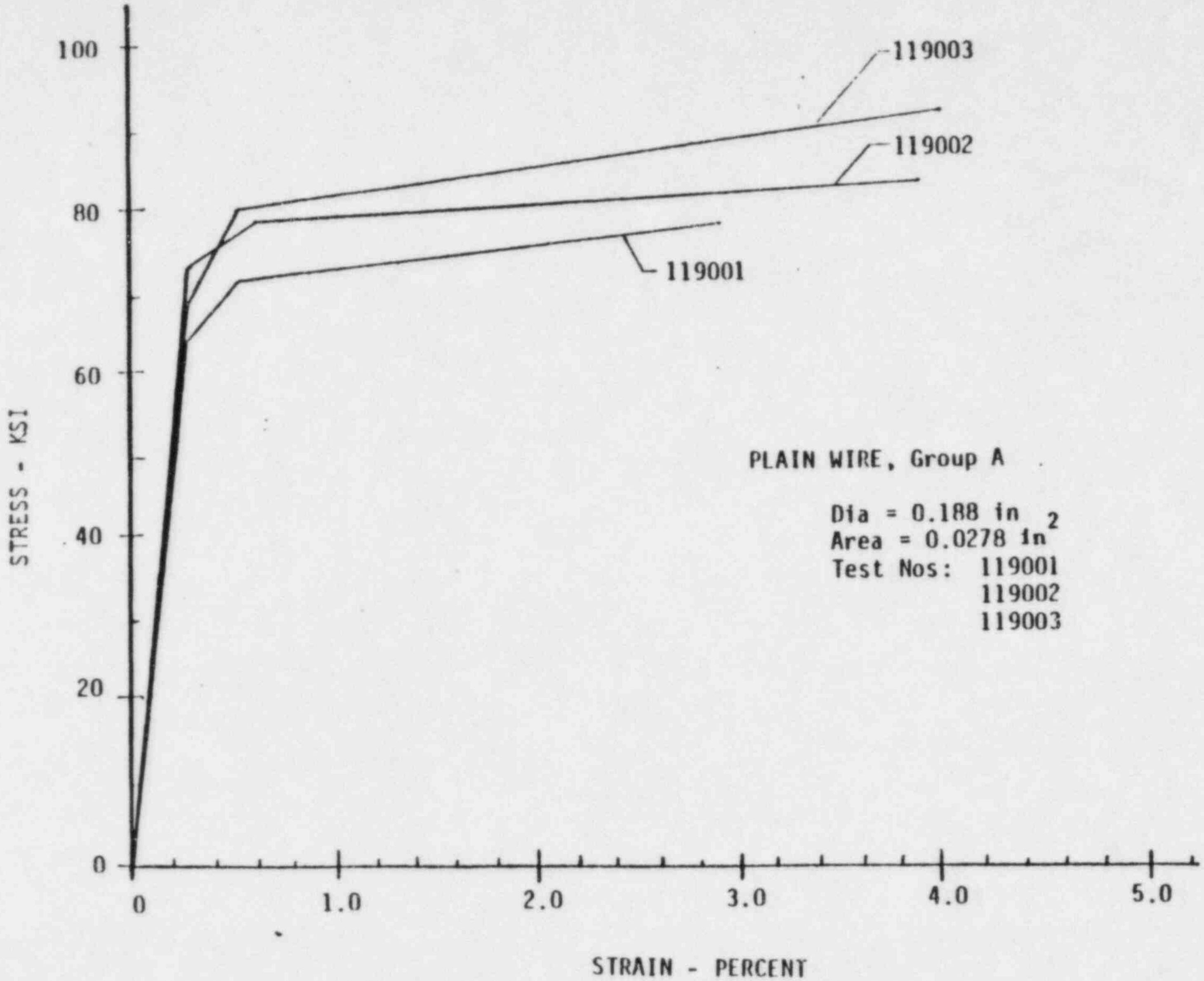

Figure 3. Typical Stress-Strain Curves for Plain Wire [16]

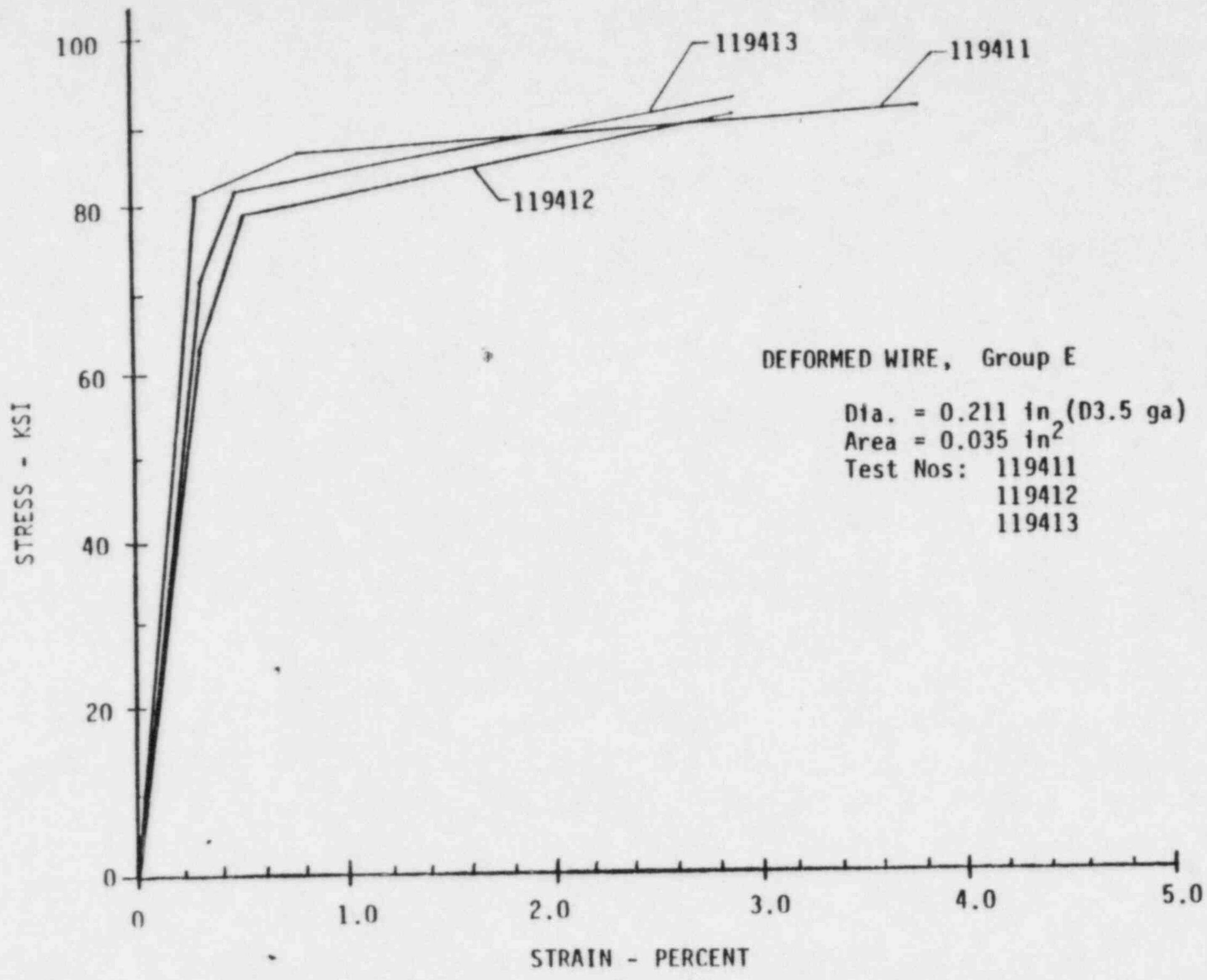

Figure 4. Typical Stress-Strain Curves for Deformed Wire [17]

Table 1

Comparison of Minimum Required Physical Properties for ASTM A 82 and ASTM A 496 Wire

	Plain Wire ASTM A 82	Deformed Wire ASTM A 496
Minimum Strength (ksi)		
General		
Yield	70	75
Ultimate	80	85
Welded Wire Fabric		
Yield	65(1)	70
Ultimate	75(1)	80
Bend Test Requirements		
Bend Angle	180 degrees	90 degrees
Pin diameter diameter(2)	One wire diameters(3)	Two wire

(1)Wire size W1.2 (0.124 inch diameter) and larger
(2)Wire size W7 (0.299 inch diameter) and smaller
(3)Wire size D-6 (0.276 inch diameter) and smaller

No. 5 rebar spaced at every 16 inches. The walls are either fully grouted or grouted every other core. Physical restraints exist all around the walls. Since the walls have bond beams along their horizontal direction, this special kind of construction will allow the walls to take the induced moment in the horizontal direction. The walls are considered to be well constructed and anchored.

Stress-strain relationships indicated that the steel strength has yielding in excess of 60 ksi. It is also noted that these walls are subject to load combinations, including seismic and other impact loads, and one sample calculation showed that seismic load contributed only a fraction (less than 10%) of wall responses. Therefore, strength degradation due to cyclic dynamic load, if it exists, may not be a significant factor.

It can be concluded that the use of joint reinforcement as a tensile-resisting element in Duane Arnold reinforced masonry walls meets the intent of the SGEB criteria and that the concerns associated with joint reinforcement have been resolved.

In another calculation, the wall has vertical and horizontal reinforcement and is restrained along all edges. However, the analysis conservatively assumed one-way action in the horizontal direction; no credit was taken from the vertical direction.

It can be concluded that the calculated stress levels for the horizontal reinforcement can be accepted in light of the above information.

3.2 EVALUATION OF LICENSEE'S APPROACH TO WALL MODIFICATIONS

The Licensee concluded that all safety-related masonry walls at the Duane Arnold Energy Center satisfy the reevaluation criteria. Therefore, no wall modifications have been proposed.

4. CONCLUSIONS

A detailed study was performed to provide a technical evaluation of the masonry walls at the Duane Arnold Energy Center. 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 factor of 1.67 for allowable stress in abnormal and extreme conditions was used for masonry shear, bond, and tension. The SGEB criteria permit a factor of 1.3 for shear and tension normal to the bed joint and a factor of 1.5 for tension parallel to the bed joint. However, since all walls are fully reinforced horizontally and vertically, with all tension taken by the reinforcing steel, the increase factor for masonry tension was not used. And, since tension governs the design of masonry walls, it was concluded that the difference between the Licensee's shear factor and the SGEB shear factor is inconsequential.

- o With regard to the energy balance technique, the following walls were affected: 200-7, 200-8, 417-25, 412-9, and 412-13. As stated in the review of Response 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.

- o With regard to the horizontal reinforcement, the Licensee stated that a number of walls were qualified based on joint reinforcement. As discussed in Response 15 of Section 3, a total of 57 walls have calculated stresses higher than 30 ksi (between 30 ksi and 54 ksi). All of these walls were constructed with bond beams (four No. 4 rebars) at every 4 feet and joint reinforcement at every course or every other course. Also, test results demonstrated that the yield strength of the joint reinforcement was in excess of 60 ksi. It was further learned through one sample calculation that the seismic load only contributed a fraction (less that 10%) to the total response. Hence, the strength degradation due to cylic, dynamic loads, if it exists, may not be a significant factor affecting walls responses. In another calculation, even though the wall is restrained along all edges, the analysis conservatively assumed one-way bending in the horizontal direction. It is evident that the stress levels in the horizontal reinforcement, although exceeding 30 ksi, are still within the yielding range predicted by tests and are acceptable. Therefore,

it is concluded that the use of joint reinforcement as a tensile-resisting element in Duane Arnold reinforced masonry walls meets the intent of the SGEB criteria and that the concerns associated with joint reinforcement have been resolved.

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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)

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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 considering 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.

ATTACHMENT 2

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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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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