OCT - - 1984

Docket No. 50-336

OCT 24 1984

LICENSEE: Northeast Nuclear Energy Company

FACILITY: Millstone Nuclear Power Station, Unit 2

SUBJECT: SUMMARY OF MEETING WITH NORTHEAST NUCLEAR ENERGY COMPANY ON MASONRY WALL DESIGN

A meeting was held with Northeast Nuclear Energy Company (NNECO) on September 27, 1984. The purpose of the meeting was to clarify our request for additional information on masonry wall design contained in a February 24, 1984 letter to NNECO.

BACKGROUND

As a consequence of the staff's review of NNECO's submittal dated D=ember 3, 1982 on masonry wall design, a request for additional information was issued to NNECO on February 24, 1984. A partial response was given to that request by a NNECO letter of May 11, 1984.

Handouts provided at the meeting are enclosed, along with a listing of meeting attendees (enclosure 1). Also a NRC position paper on the energy balance technique provided to NNECO is enclosed (enclosure 2).

SUMMARY OF DISCUSSIONS

The handouts provide a summary of the four questions contained in our February 25, 1984 letter along with NNECO's preliminary response. As a result of the presentation, the staff will require the following:

Question No. 1: For the few walls which don't meet the combined vertical and horizontal minimum steel area requirements, identify those walls and provide justification for analyzing them as reinforced walls. Also discuss the applicability of partial reinforcement clause of UBC code to Millstone Unit 2 walls.

Question No. 2: Discuss and provide the analysis for applicable external (tornado generated) and internal missiles which will impact the safety related masonry walls.

Question No. 3: No additional explanation is needed.

Question No. 4: The assumptions used in evaluating the walls need to be examined in more detail.

Besides providing a formal submittal to the four questions contained in the February 24, 1984 letter by October 31, 1984. NNECO will give the total number of masonry walls and the total number which are safety related. They will also provide a summary of the type of QA/QC information that is available and, if applicable, provide an explanation for not repairing walls which exceed stress allowable limits.

Conclusion:

Depending on the depth of QA/QC information on file, the qualification of these walls to meet the projected loadings are apparently going to be adequately addressed.

Original signed by:

D. B. Osborne, Project Manager
Operating Reactors Branch No. 3
Division of Licensing

Enclosures:
As stated

cc w/enclosures:
See next page

ORB#3:DL	ORB#3:DL	ORB#3:DL
PKreutzer	DOsborne;ef	JMiller
10 /24/84	10 /24/84	10 /24/84

MEETING SUMMARY DISTRIBUTION

Licensee: Northeast Nuclear Energy Company

*Copies also sent to those people on service (cc) list for subject plant(s).

Docket File
NRC PDR
L PDR
ORB#3 Rdg
Project Manager -D. Osborne
JMiller
BGrimes (Emerg. Preparedness only)
OELD
NSIC
EJordan, IE
JNGrace, IE
ACRS-10

NRC Meeting Participants:

NChokshi
DJeng
DOsborne
CTrammel

Enclosure 1

List of Attendees

Nuclear Regulatory Commission

Dee Osborne	DL/ORB No. 3
Nilesh Chokshi	DE/SGEB
Dave Jeng	DE/SGEB

Franklin Research Center

Vu Con

Northeast Utilities

Michael Childers
Janice Mawson
Drina Beauregard
Walter Briggs

Cygna Energy Service

Paul Baughman

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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Ahmad A. Hamid (1)
Vu Con (2)

~~8410120139~~

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 Tas 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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9. Manos, G., Clough, R. and Mayes, R., "Shaking Table Study of Single Story Masonry Houses- Dynamic Performance under Three Component Seismic Input and Recommendations," Report No. UCEERC-83/11, Univ. of Calif., Berkeley, July 1983.

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NRC/NUSCO MEETING
SEPTEMBER 27, 1984
MILLSTONE UNIT NO. 2

MASONRY WALL BULLETIN 80-11 AGENDA

INTRODUCTION	W. J. BRIGGS
MINIMUM AREA OF REINFORCEMENT	J. L. MAWSON
TORNADO LOADS	J. L. MAWSON
SGEB INCREASE FACTORS	D. Z. BEAUREGARD
ARCHING ACTION TECHNIQUE	D. Z. BEAUREGARD

QUESTION 1

With reference to the reinforcement in masonry walls, the ACI 531-79 Code (1) specifies that the minimum area of reinforcement in a wall in each direction, vertical or horizontal, shall be 0.0007 (0.07 percent) 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 percent) 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.

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 (Code) 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.

Indicate if there are any walls that may have been qualified using the tensile resistance of the joint reinforcement but not satisfying the minimum steel requirements. It should be noted that the NRC, at present, does not approve the use of joint reinforcement to qualify this type of wall. (See attached staff position). In view of this, indicate all walls belonging to this category and your intended specific actions to bring these walls in compliance with the staff position.

MINIMUM AREA OF REINFORCEMENT

- ORIGINAL PLANT DESIGN WAS TO 1967 UBC

- ALL WALLS ARE PARTITION WALLS (RESISTING LOCAL LOADS) WHICH ARE CLASSIFIED AS PARTIALLY REINFORCED UNDER UBC

- ALL WALLS MEET UBC MINIMUM REINFORCEMENT REQUIREMENTS FOR PARTIALLY REINFORCED MASONRY

- HORIZONTAL JOINT REINFORCEMENT WAS PROVIDED BUT NOT USED FOR STRESS RESISTANCE

- UNDER 80-11 EVALUATION, NO WALLS WERE QUALIFIED USING THE TENSILE RESISTANCE OF JOINT REINFORCEMENT

QUESTION 2

With respect to tornado load (2), specify all walls subject to tornado load (if applicable) and provide a sample calculation (with any explanation necessary, to make it understandable. Also, indicate how the penetration depth, perforation, and spalling along with the overall structural behavior of the wall were evaluated for a tornado missile impact.

TORNADO WALLS

SITE GRADE: 14'-6"

WALL	ELEV.	LOCATION	PROTECTED UNPROTECTED
1.32	38'-6"	INTERIOR	PROTECTED
*6.1	54'-6"	EXTERIOR	UNPROTECTED
*6.2	54'-6"	EXTERIOR	UNPROTECTED
7.5	31'-6"	INTERIOR	PROTECTED
7.12	31'-6"	INTERIOR	PROTECTED
8.22	25'-6"	EXTERIOR	PROTECTED
8.29	25'-6"	EXTERIOR	PROTECTED
8.31	25'-6"	EXTERIOR	PROTECTED
10.5	45'-0"	INTERIOR	PROTECTED
10.12	45'-0"	INTERIOR	PROTECTED

*MORE THAN 30' ABOVE SITE GRADE.

TORNADO LOADS

- o DESIGN CRITERIA

  - 10 WALLS SUBJECT TO TORNADO
  - 360 MPH WIND LOAD
  - 3 PSI DEPRESSURIZATION LOAD
  - TORNADO MISSILES

- o REEVALUATION FOR 80-11

  - APPLIED PRESSURE LOADS
  - MISSILE LOADS NOT APPLICABLE BECAUSE OF PROTECTION OR HEIGHT

QUESTION 3

Regarding Responses 3 and 4 of Reference 2, identify walls that would not be qualified if the SGEB increase factors for allowable stresses (3) were to be used. It should be noted that for the OBE loading case, the SGEB criteria do not allow any increase factor, whereas the licensee used a factor of 1.33. Also, specify the percentage of exceedance for OBE, SSE, and other accident load cases. Explain all conservative measures (if any) used in the analysis to justify a higher increase factor.

QUESTION 3: INCREASE FACTORS

OPERATING BASIS EARTHQUAKE

- o ORIGINAL 80-11 EVALUATION USED 1.33 INCREASE FACTOR
- o REEVALUATED WALLS FOR 1.0 FACTOR
- o ALL WALLS STILL QUALIFY

QUESTION 3: INCREASE FACTORS

EXTREME ENVIRONMENTAL LOADS

o COMPARISON OF SGEB VS. ORIGINAL 80-11 FACTORS

TYPE OF STRESS	SGEB FACTOR	ORIGINAL 80-11 FACTOR
AXIAL OR FLEXURAL COMP.	2.5	2.5
BEARING	2.5	2.5
REINFORCEMENT STRESS EXCEPT SHEAR	2.0 NOT TO EXCEED 0.9 FY	*0.9 FY
SHEAR REINFORCEMENT AND/OR BOLTS	1.5	N/A
MASONRY TENSION PARALLEL TO BED JOINT	1.5	*1.67
SHEAR CARRIED BY MASONRY	1.3	*1.67
MASONRY TENSION PERPENDICULAR TO BED JOINT		
FOR REINFORCED MASONRY	0	0
FOR UNREINFORCED MASONRY	1.3	*1.67

* REEVALUATED WALLS FOR SGEB CRITERIA

QUESTION 3: INCREASE FACTORS

EXTREME ENVIRONMENTAL LOADS

- o REINFORCED WALLS
  - ALL STEEL STRESS LESS THAN 48 KSI (SGEB ALLOWABLE) EXCEPT WALL 10.3.
  - WALL 10.3 STRESS IS 48.6 KSI, STILL HAS ADEQUATE MARGIN
- o UNREINFORCED WALLS
  - ALL STRESSES LESS THAN SGEB ALLOWABLES

QUESTION 4

With regard to the nonlinear analysis technique (energy balance technique and arching action theory), please note the following and provide the information requested.

a. Arching Action: The NRC position on this issue states that the use of the arching action theory to qualify unreinforced masonry walls is not acceptable. These walls should be repaired so that they can be qualified based on the SGEB criteria (3). (The NRC position is attached.) In view of this, indicate your intended actions and schedule to bring the affected walls in compliance with the staff position.

b. Energy Balance Technique: The NRC is currently preparing a position statement regarding this technique, which will be forwarded to the licensee in the near future.

QUESTION 4: ARCHING ACTION

- o 18 WALLS QUALIFIED USING ARCHING ACTION
  - 7 NOT SAFETY RELATED
  - 11 REANALYZED TO SGEB CRITERIA
  - ALL ARE UNREINFORCED SOLID BLOCK
- o ORIGINAL 80-11 ANALYSIS
  - PERFORMED PRIOR TO ISSUANCE OF SGEB CRITERIA
  - SOME ASSUMPTIONS WERE OVERCONSERVATIVE
    - o DAMPING
    - o PLACEMENT OF LOADS
- o REEVALUATION
  - SGEB DAMPING CRITERIA
  - WORKING STRESS, ELASTIC ANALYSIS
  - CORRECT SPATIAL PLACEMENT OF LOADS

QUESTION 4: ARCHING ACTION

- o WALLS ARE ANALYZED BY MEANS OF A FINITE ELEMENT PROGRAM (SAP IV) AND ARE SUBJECTED TO STATIC AND DYNAMIC LOADING

- o MASONRY BEHAVIOR IS TREATED THE SAME AS CONCRETE, WITH APPROPRIATE ADJUSTMENTS FOR DIFFERENCES IN MATERIAL PROPERTIES

- o WALLS HAVE ASPECT RATIOS OF 2 TO 1 OR LESS, SO THE MODELS INCLUDE THE POSSIBILITY OF TWO WAY ACTION

- o CONCENTRATED LOADS AND OPENINGS ARE INCLUDED IN THE MODEL

- o THE FINITE ELEMENT FORMULATION ACCOUNTS FOR THE NON ISOTROPIC PROPERTIES OF MASONRY. THE PRINCIPAL DIRECTIONS ARE TAKEN AS NORMAL AND PARALLEL TO BED JOINTS

- o THE SAP IV COMPUTER PROGRAM'S ORTHOTROPIC PLATE BENDING ELEMENT IS USED

- o WHERE THE MOMENT AT A POINT EXCEEDS THE ALLOWABLE, LOCAL FAILURE IS ASSUMED AND THE WALL IS CHECKED FOR OVERALL EQUILIBRIUM

QUESTION 4: ARCHING ACTION

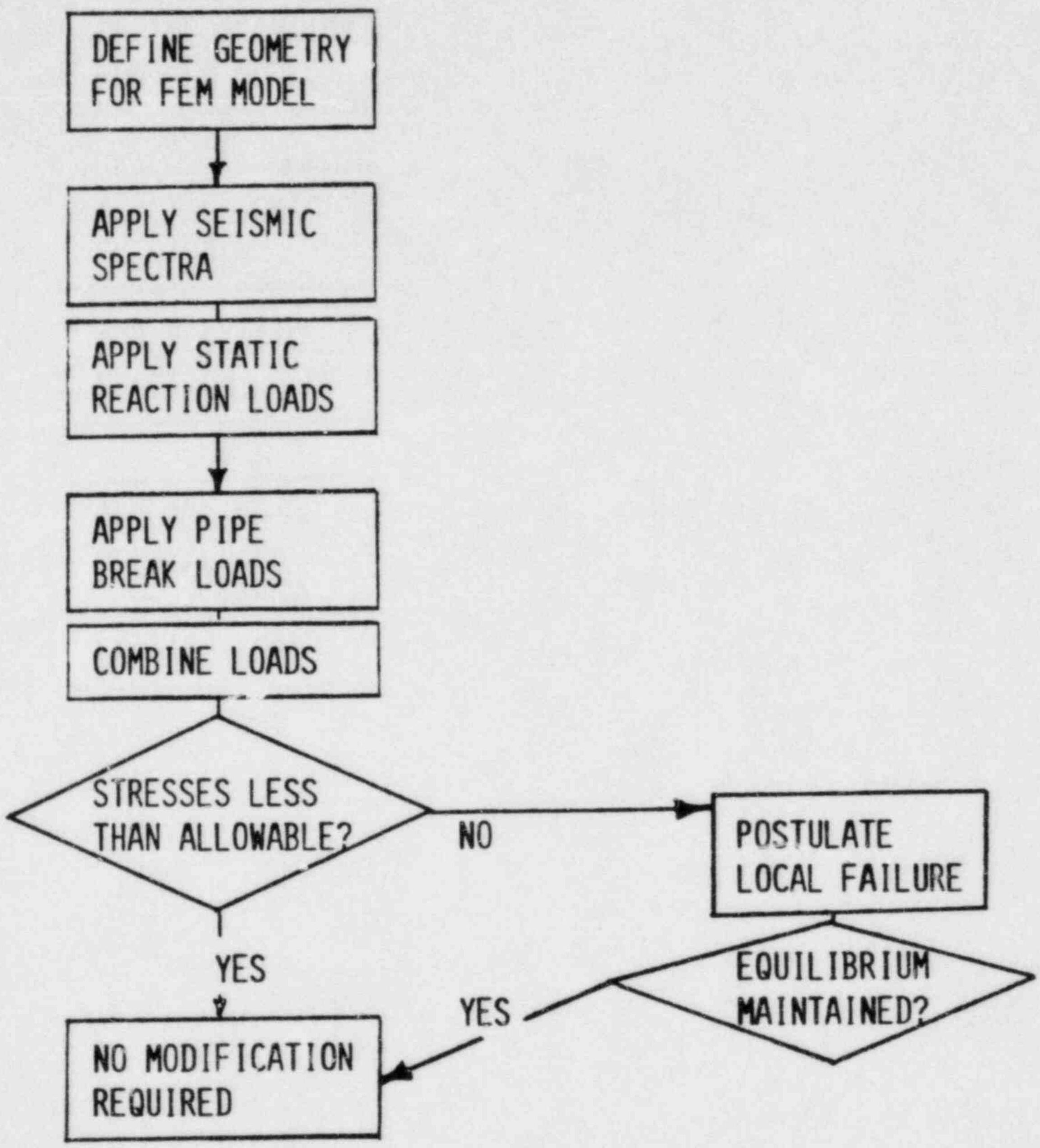
ARCHING ACTION WALLS

WALL	ELEVATION	DIMENSIONS	NUMBER OF WYTHES	FREQUENCY	GOVERNING LOADING
1.21	38'-6"	5'-8" x 13'-3"	4	87.41 CPS	D+L+Fp+E'
1.31	45'-6"	6'-10" x 5'-9"	4	42.37 CPS	D+L+Fp+E'
1.36	47'-6"	6'-2" x 5'-0"	2	52.28 CPS	D+L+Fp+E'
1.49	38'-6"	4'-0" x 4'-0"	2	102.5 CPS	D+L+Fp+E'
2.7	26'-11"	6'-0" x 8'-4"	3	50.94 CPS	D+L+E'
3.23	-5'-0"	4'-9" x 8'-10"	5	20.61 CPS	D+L+E'
3.30	-5'-0"	6'-10" x 17'-4"	7	9.11 CPS	D+L+E
3.31	-5'-0"	12'-4" x 17'-6"	5	5.53 CPS	D+L+E
4.21	-25'-0"	18'-0" x 18'-5"	4	5.95 CPS	D+L+E'
5.12	-45'-6"	8'-0" x 11'-9"	4	28.84 CPS	D+L+E'
5.13	-45'-6"	10'-6" x 9'-4"	4	19.49 CPS	D+L+E'

ALL ARE UNREINFORCED, SOLID BLOCK WALLS,
LOCATED IN THE AUXILIARY BUILDING

QUESTION 4: ARCHING ACTION

REEVALUATION PROCEDURE



QUESTION 4: ARCHING ACTION

ASSUMPTIONS

- o ALL COMPONENTS OTHER THAN PIPING SUPPORTED ON OR NEAR MASONRY WALLS ARE CONSIDERED RIGID AND THEREFORE DO NOT IMPOSE AMPLIFIED LOADS OR IMPACT LOADS ON THE WALL DUE TO SEISMIC DISPLACEMENT

- o PIPING REACTION LOADS ARE STATICALLY APPLIED AND ADDED TO INERTIAL LOADS. THE MASS OF THE ATTACHED PIPING IS ALSO INCLUDED IN THE INERTIAL CASE

- o SURFACE MOUNTED ATTACHMENTS WHICH PROJECT NO FURTHER FROM THE WALL SURFACE THAN THE WALL THICKNESS CONTRIBUTE ONLY IN-PLANE LOADS TO THE WALL

- o SUPPORT CONDITIONS FOR MASONRY WALLS ARE CONSIDERED PINNED WHEN SHEAR TRANSFER MECHANISMS ARE PRESENT; OTHERWISE, A FREE EDGE IS ASSUMED

- o MULTI-WYTHE WALLS ARE ANALYZED AS MULTIPLE SINGLE WYTHE WALLS TAKING NO CREDIT FOR COLLAR JOINT MORTAR SHEAR CAPACITY

- o A DYNAMIC LOAD FACTOR (DLF) OF 2 IS USED TO AMPLIFY THE JET IMPINGEMENT LOADS

QUESTION 4: ARCHING ACTION

SUMMARY OF RESULTS

WALL	MAXIMUM STRESS PARALLEL TO BED JOINTS (PSI)	ALLOWABLE (PSI)	MAXIMUM STRESS NORMAL TO BED JOINTS (PSI)	ALLOWABLE (PSI)
1.21	42.12	96	14.72	54.6
1.31	36.52	96	10.18	54.6
1.36	21.23	96	44.55	54.6
1.49	43.00	96	32.48	54.6
2.7	64.47	96	31.57	54.6
3.23	13.63	96	52.62	54.6
3.30	34.85	64	130.8	42
3.31	56.75	64	453.2	42
4.21	*		*	
5.12	17.33	96	5.43	54.6
5.13	4.20	96	27.83	54.6

*WALL 4.21 IS STILL BEING INVESTIGATED

QUESTION 4: ARCHING ACTION

WALLS 3.30 & 3.31

- o HIGH RADIATION AREA
  - OPERATING: 100 R/HR
  - SHUTDOWN: 25 R/HR
  - ESTIMATE DOSE FOR MODIFICATION: 5200 MAN-REM

- o COMPOSITE ACTION EVALUATED
  - MAXIMUM TENSILE STRESS IS 14.79 PSI
  - MAXIMUM COLLAR JOINT SHEAR IS 10.1 PSI
  - CONSTRUCTED UNDER QA PROGRAM REQUIREMENTS SATISFYING 10 CFR 50 APP. B

- o RECOMMENDATION:

  THESE WALLS SHOULD NOT BE MODIFIED BECAUSE STRESSES ARE NOT HIGH ENOUGH TO WARRANT SUCH A HIGH PERSONNEL EXPOSURE

QUESTION 4: ARCHING ACTION

CONCLUSIONS

- o ALL SAFETY RELATED WALLS EXCEPT 3.30 & 3.31 PREVIOUSLY QUALIFIED BY ARCHING ACTION HAVE BEEN SHOWN TO MEET SGEB CRITERIA

- o WALLS 3.30 & 3.31 SHOULD NOT BE MODIFIED BECAUSE OF LOW STRESS LEVELS AND HIGH RADIATION EXPOSURE