TECHNICAL EVALUATION REPORT

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MASONRY WALL DESIGN

ROCHESTER GAS AND ELECTRIC CORPORATION

R. E. GINNA NUCLEAR POWER PLANT

TER-C5506-262

Prepared for

Nuclear Regulatory Commission
Washington, D.C. 20555

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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

In an effort to respond to IE Bulletin 80-11 [1]. Rochester Gas and Electric Corporation (RG&E), the Licensee of the R. E. Ginna Nuclear Power Plant, submitted its reevaluation of the masonry walls to the U.S. Nuclear Regulatory Commission (NRC) for review and evaluation. Franklin Research Center (FRC) was retained by the NRC to review and assess the Licensee's submittals. Dr. A. Hamid was retained by FRC to evaluate the nonlinear analysis methodology used by the Licensee in qualifying a number of masonry walls in the plant.

This report represents FRC evaluation and assessments based on the review of the Licensee's submittals, other published literature, and test data relating to this subject. The report also reflects the results of a number of meetings with the Licensee regarding safety issues of masonry walls in the plant.

In an attempt to qualify a number of walls in the control building, the Licensee relied on test data performed by Southern California Edison Company (San Onofre Nuclear Generating Station Unit 1) to validate a nonlinear analysis technique. Results of the test are considered proprietary information by the Licensee. Therefore, the information concerning the test data presented in this report is proprietary information.

2. PLANT-SPECIFIC BACKGROUND

In response to IE Bulletin 80-11, Rochester Gas and Electric Corporation (RG&E) provided NRC with letters and attachments dated July 7, 1980 [2], November 4, 1980 [3], and January 30, 1981 [4] describing the analysis of masonry walls at the R. E. Ginna Nuclear Power Plant. The information in these documents was reviewed and a request for additional information was sent to the Licensee on September 21, 1983. The Licensee responded in the submittals of July 13, 1984 [5] and October 19, 1984 [6]. As a result of a review of these submittals, more questions arose concerning the analysis method and acceptance criteria for the masonry walls at this plant; the questions were discussed in a meeting and site visit between the NRC, its consultants, and the Licensee on April 2 and 3, 1985.

The original seismic analysis of the masonry walls at the Ginna plant followed the FSAR and did not use floor response spectra. In its July 13, 1984 submittal, however, the Licensee indicated that the safety-related masonry walls had been reanalyzed using the site-specific SEP earthquake (0.17g ground acceleration) and floor response spectra.

RG&E reported a total of 37 safety-related masonry walls functioning as shield, partition, or fire walls. These walls were subdivided into 56 panels for engineering purposes.

Typically, the walls are hollow, single-wythe, and reinforced horizontally wit' Dur-O-Wall joint reinforcing every 8 or 16 inches. Twelve walls are also reinforced vertically. Most walls are 8 or 12 inches thick. The materials used in construction are as follows:

Hollow masonry units - ASTM C90-665, Grade 11
Interior partition units - Haydite block
Joint reinforcing - Dur-O-Wal
Mortar - ASTM C 270-64T, Type N.

A total of 29 panels satisfied the SGEB criteria after some form of modifications. The modifications consisted of the addition of structural steel supports at the edges to assure the assumed boundary conditions and the addition of intermediate steel supports across the wall face to reduce span length. An evaluation of the Licensee's approach to modifications is provided in Section 3.6.

Results of the original evaluation indicated that 27 panels did not meet the SGEB criteria. The failure of one panel (971-2M) was deemed acceptable as explained in Response 8 of Appendix B; the other 26 panels were under consideration for modifications and/or further refined analysis.

A total of 14 panels will be modified, and the remaining 12 panels in the control building were considered for further refined analysis (including nonlinear analysis). A meeting was held between NRC, FRC, and the Licensee on April 2 and 3, 1985 to discuss the nonlinear analysis method. A number of questions were raised in this meeting regarding the validity and applicability of this method to the walls in the Ginna plant. On September 24, 1985, another meeting was held to discuss and identify additional investigations regarding the validity of this methodology. As a result of this meeting and subsequent conference telephone conversations, the Licensee proposed, with NRC concurrence, the following course of actions:

- o Further investigation to indicate the number of panels that could be qualified relying on the SGEB criteria.
- o Additional work to validate the nonlinear analytical method before using it to qualify the panels in the control building.

The Licensee submitted additional information regarding the above actions in letters dated December 19, 1985 [7] and January 14, 1986 [8].

The status of the 56 panels is summarized below:

- o Twenty-nine panels satisfied the SGEB criteria after some modifications
- o Two panels satisfied the SGEB criteria after a refined analysis
- o Ten panels were qualified using a nonlinear analysis technique
- o One panel was reclassified as non safety-related
- o Fourteen panels are scheduled for modifications.

The Licensee confirmed in a letter dated March 25, 1986 that all modifications are expected to be completed by late 1988 to mid-1989. The

Licensee also stated that modifications will be provided to certain safety-related panels to protect them from tornado and missile effects. To install these modifications, the Licensee will take into account the possible damaging effects of seismic motion in light of the requirements of IE Bulletin 80-11.

3. TECHNICAL EVALUATION

3.1 LICENSEE'S CRITERIA

This evaluation is based on the Licensee's earlier responses (July 7, 1980; November 4, 1980; January 30, 1981) and subsequent responses [5-8] to the NRC's request for additional information. This evaluation is also a result of a number of meetings and a site visit. 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.

The criteria used by the Licensee are summarized below:

- For linear elastic analysis: the working stress design method was used. Allowable stresses are based on ACI 531-79 [9].
- Load combinations are taken from the FSAR.
- A damping value of 7% was used for the SEP (Systematic Evaluation Program) loading.
- A typical analytical procedure used in the working stress design method is summarized below:
  - determine wall boundary conditions (fixed, pinned, or free)
  - calculate the wall's fundamental frequency using a two-way plate action assumption (computer program SAP IV was used)
  - obtain inertial loading from the average of the response spectra occurring at the top and base of the wall of the walls is supported at the top; if the wall is free at the top, the spectrum at the base is used
  - compute stress
  - increase the computed stress 5% to account for higher modes of vibration
  - compare computed stresses with allowable values.

With regard to the linear elastic analysis, the Licensee's criteria are considered satisfactory and acceptable in light of the SGEB criteria. Further discussion on this is given in Appendix B.

The nonlinear analysis technique used to qualify a number of reinforced walls in the control building was developed for the inelastic transverse analysis of centrally reinforced masonry walls. This methodology allows wall deformation to extend beyond its elastic limits. As the wall deformation falls into the inelastic range, plastic hinges are formed; vertical rebars are allowed to yield while the mortar bed joints are in compression only (gap element).

The following criteria were introduced in the nonlinear analysis [10]:

Transverse Loads

- Maximum displacement of the wall is limited by a maximum steel strain to yield strain (ductility ratio) of 45
- Masonry face shell strain is limited to 0.003
- Wall stability is checked.

Load Combinations

Two load cases were used:

Load Case	Out-of-plane	In-plane	Vertical
1	100%	40%	40%
2	40%	100%	40%

3.2 PANELS IN THE CONTROL BUILDING

As previously stated, a total of 12 panels are in the Ginna control building and ten of these walls have been qualified relying on the nonlinear analytical procedures. The other two panels were qualified by a refined linear elastic analsysis. The layout of panels in the control building is provided in Figures 3-1, 3-2, and 3-3.

The panels in the control building have the following construction characteristics:

- Seven panels extending between two floors (spanning walls)
- Five cantilever panels extending partial story height
- The spanning panels are reinforced with one No. 3 bar at every 32 inches.
- The cantilever panels are reinforced with two No. 3 bars at every 16 inches.
- The joint reinforcement consists of either Dur-O-Wal standard truss type at every 8 inches or Dur-O-Wal extra heavy truss type at every 16 inches.

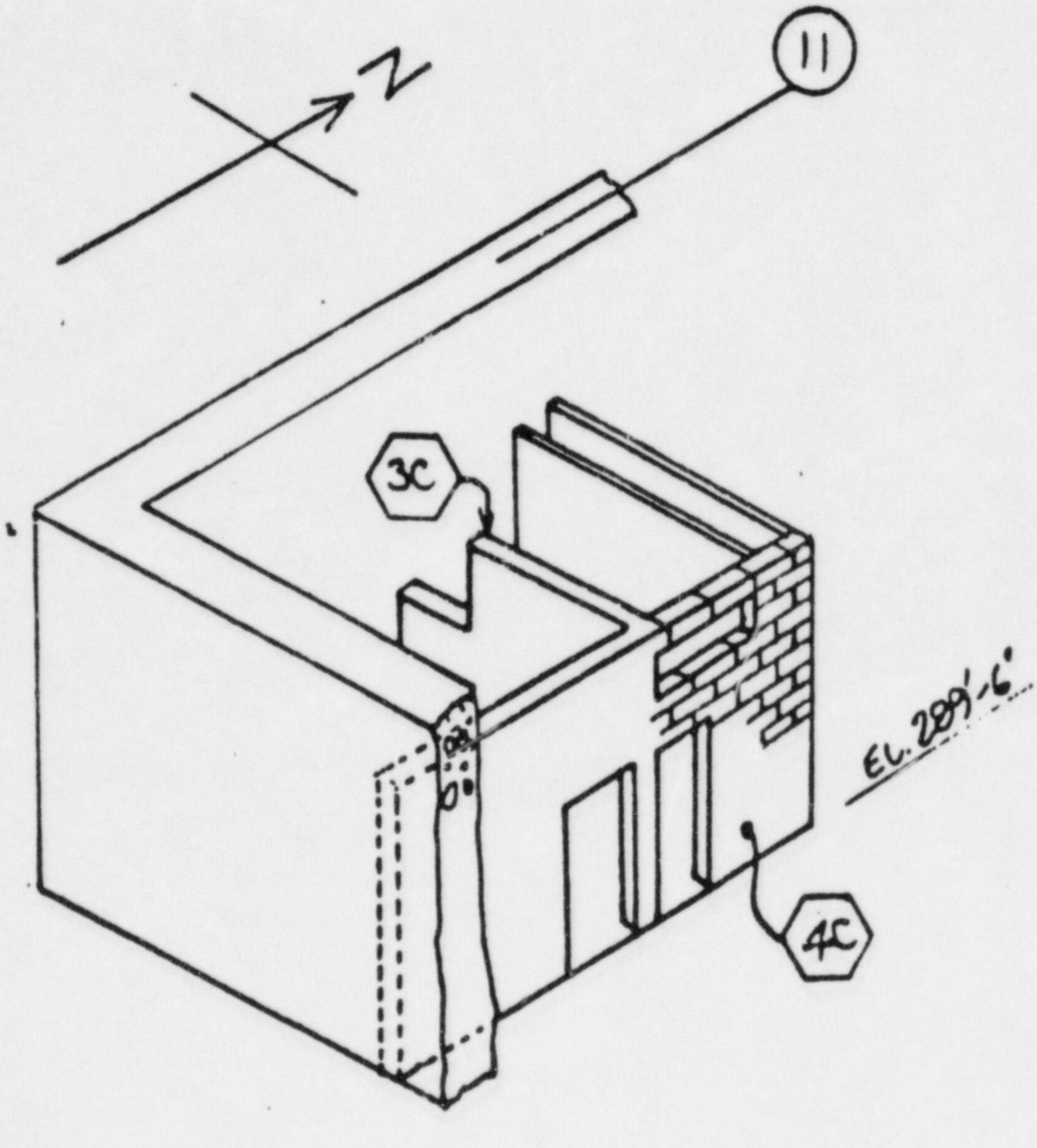

Figure 3-1. Control Building Walls Elevation 289 feet, 6 inches

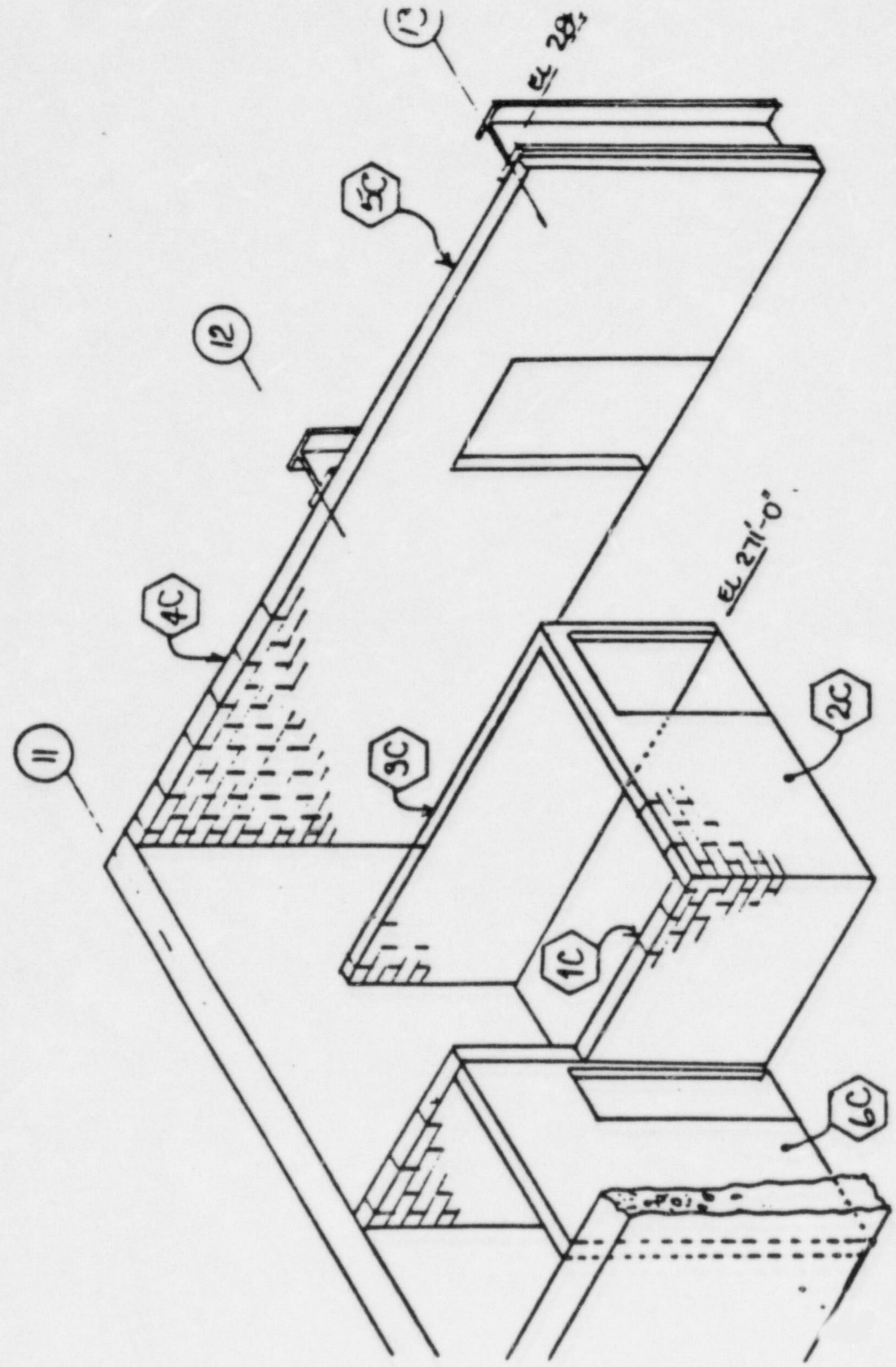

Figure 3-2. Control Building Walls Elevation 271 feet, 0 inches

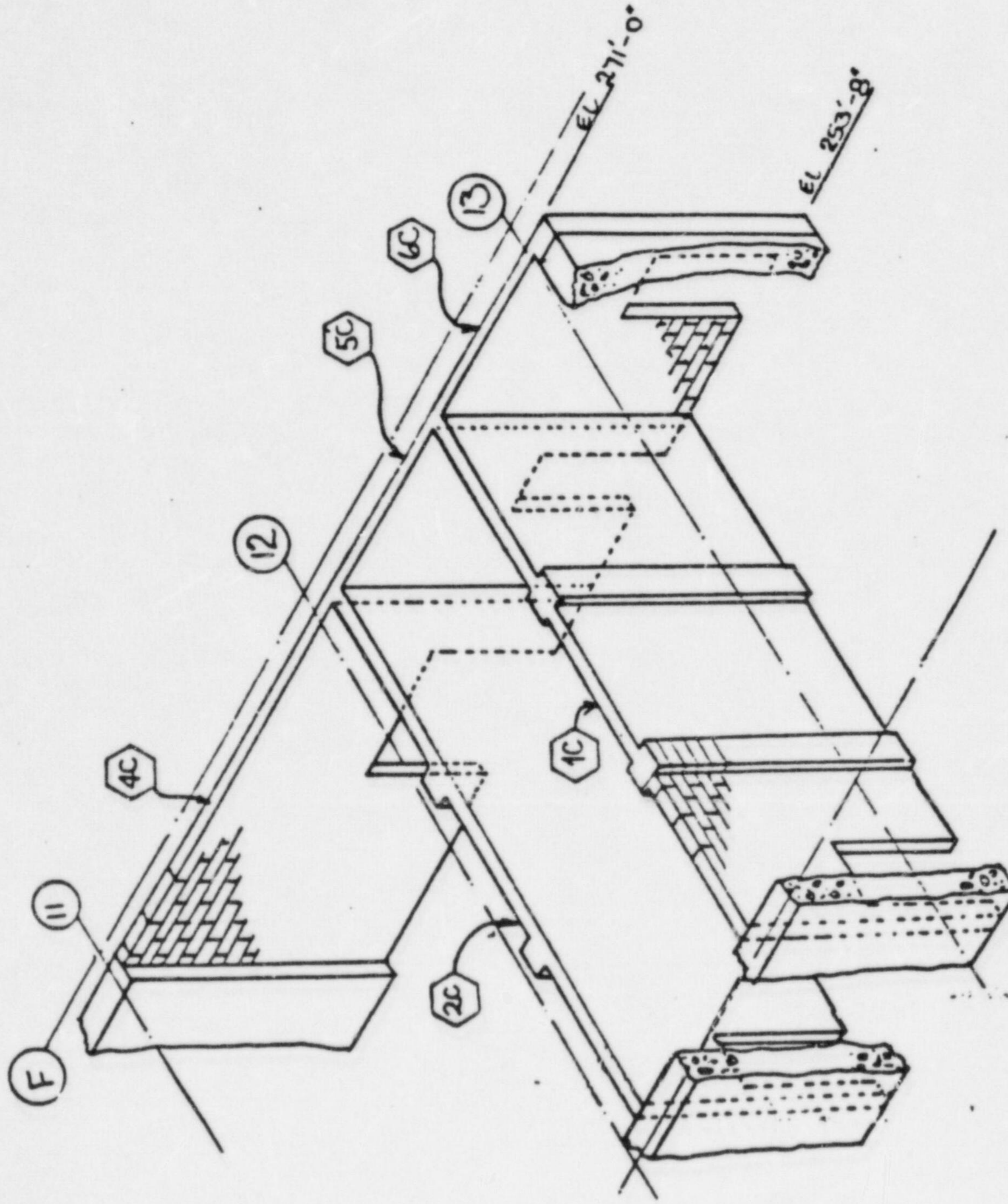

Figure 3-3. Control Building Walls Elevation 253 feet, 8 inches

- The panel height varies between 10 feet 3 inches to 18 feet and is 8 inches thick.

- The panels are partially grouted.

3.3 EVALUATION OF THE NONLINEAR ANALYTICAL METHOD

As discussed in the report prepared by FRC [11] regarding the nonlinear analysis method used by San Onofre Unit 1, a limited correlation was performed to validate the methodology. In general, the analytical procedures were able to capture the behavior of the test walls for the San Onofre case. However, due to differences in the input values of the test and the analysis, several parameters in the correlation study did not exhibit close agreement. It was judged that additional study should be conducted before using this methodology for walls in other plants [11].

Based on the results of several meetings with the Licensee, an additional correlation study of the San Onofre test and analytical data was performed by RG&E to validate the analytical procedures previously used in the San Onofre program so that they could be applied to the Ginna walls. This correlation study was carried out using the actual test data obtained from the San Onofre program.

As a result of this correlation study, the previously developed model has been modified and used to quality a total of 10 panels in the control building.

3.3.1 Analytical Procedures

The following information highlights the key features of the detailed model used in the analysis:

- A finite element model based on the DRAIN-20 and ANSR-II computer programs was used.

- Plastic hinges were included in the midsection and the base in the model.

- The plastic hinge was modeled as a truss bar which yields in tension.

- A minimum of two joints (mortar joints) on either side of the points of maximum moment were included in the model.

- At the mortar joint, the face shell was modeled as truss bar elastic in compression with no tension capacity.

- Blocks between mortar joint were modeled as plane stress elements.

Detailed discussions on the modeling techniques are summarized in Reference 11. The following steps outline the general approach used in the analysis:

- Development of a detailed model representing every block and mortar joint as well as the rebar and face shell of each joint. Figure 3-4 illustrates this model.
- Development of a force displacement relationship of the model by applying a monotonically increasing lateral load.
- Development of a substructure model including the force displacement characteristics obtained above. This substructure model was basically a simplified version of the detailed model in which the overall hysteresis loop was included as a part of a single yielding element.
- Dynamic analysis of the substructure model using the actual test time histories.
- Analysis of the detailed model using the maximum displacements obtained from the substructure model. The results of this analysis are the maximum masonry compressive strains, the maximum steel strain ratios, the length of yielding rebar, and the maximum gap openings.

The above steps are summarized in the flowchart shown in Figure 3-5.

3.3.2 Correlation Study

In order to validate the nonlinear analysis method that RG&E intended to apply to the masonry walls at the Ginna plant, RG&E examined the ability of the method to predict the behavior of test walls. To do this, RG&E performed a correlation study of actual test and analytical data from another plant, San Onofre, which had used the same nonlinear analysis method. If the correlation was good, then the analysis method could be considered valid and then could be applied to the walls at the Ginna plant.

To validate the methodology, the following items were examined in the correlation study (this study was carried out using the San Onofre test data):

- eccentricity of rebar
- masonry strains
- steel yield stress
- length of yielding rebar

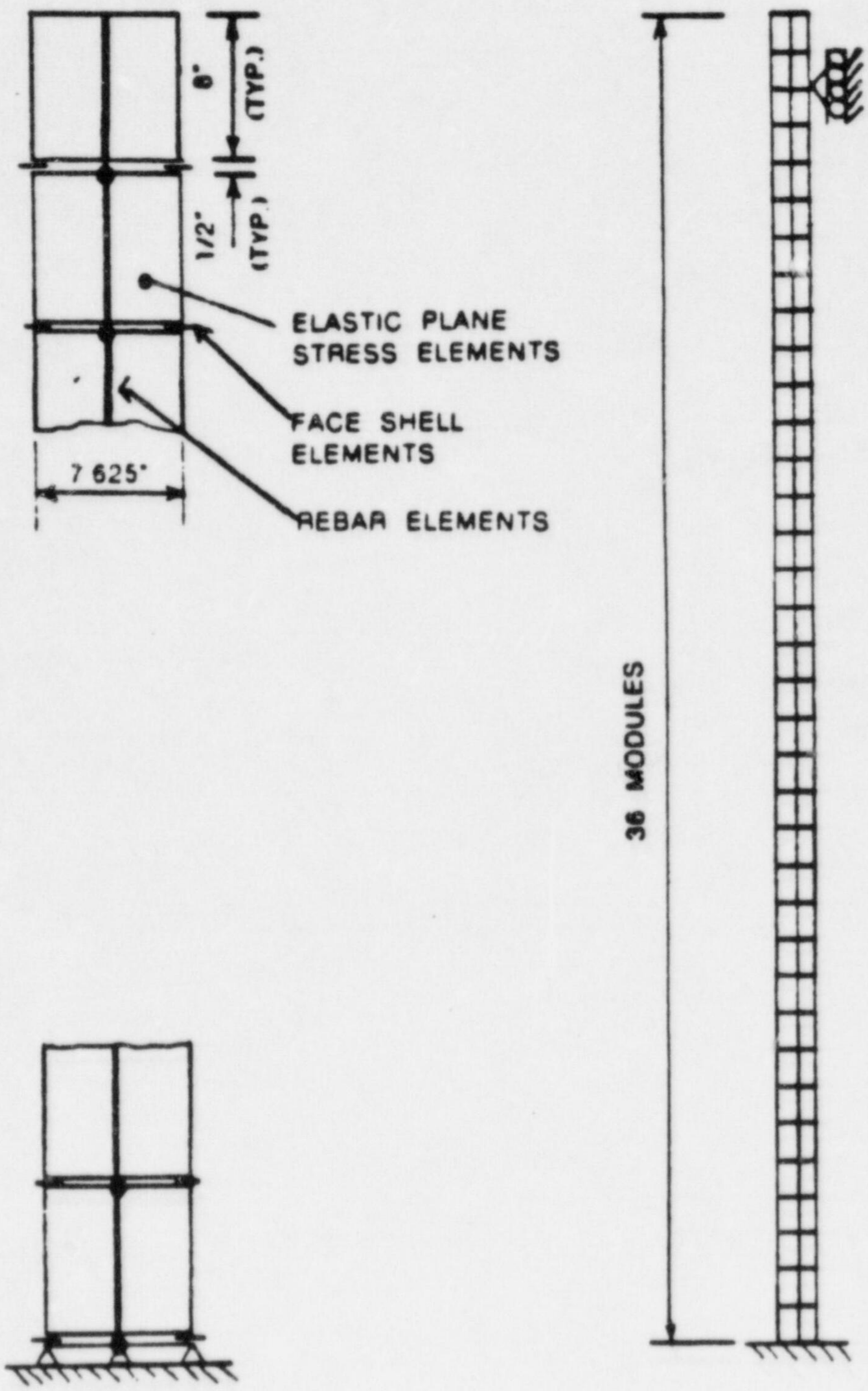

Figure 3-4. Detailed Wall Representation

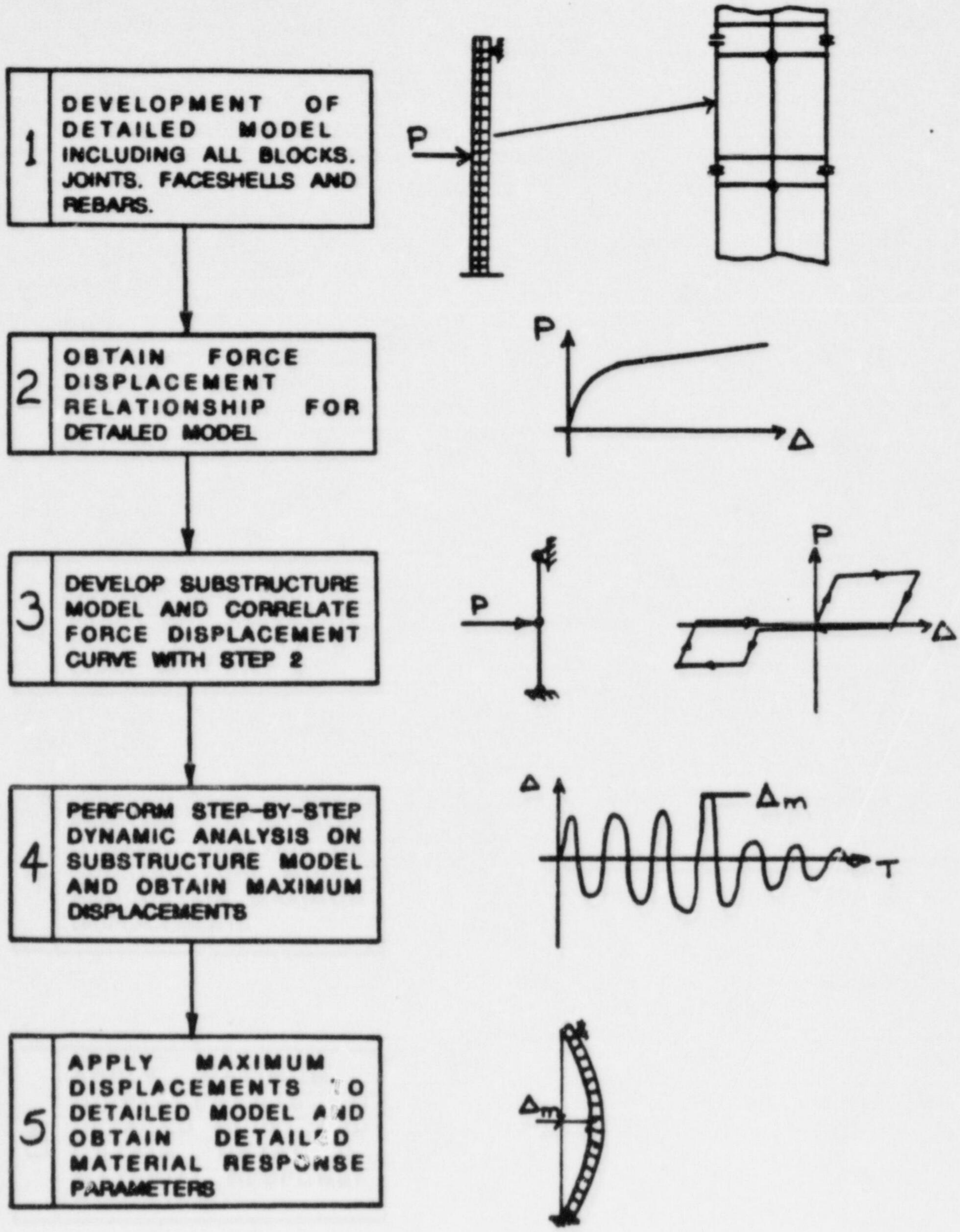

Figure 3-5. Flowchart for Evaluation Procedure

- o permanent set in the walls
- o differences in analytical and test input motions.

For the correlation, the analytical results of the most critical wall of the San Onofre plant (wall FB5 [11]) were compared with test results of three test panels of the San Onofre test program, namely wall types 1A, 1B, and 1C (1A, 1B, and 1C are three test samples representing the walls in the fuel storage building of the San Onofre plant).

The analysis was conducted using the actual input test motion and the actual tested yield strength for the rebar. In addition, the wall self-weight was included in the analysis. The length of yielding rebar was set so that it would vary according to displacement. The rebar eccentricity was also included in this analysis.

A comparison was obtained for the following parameters:

- o maximum mid-span deflections
- o maximum masonry compressive strains
- o maximum steel strain ratios
- o length of yielding rebar
- o maximum gap openings.

(Further discussions on these parameters are provided in Reference 11.)

3.3.3 Correlation Results

The results from the San Onofre plant are introduced in Table 3-1 and used in this correlation.

The results of the analysis of wall types 1A, 1B, and 1C are presented in Tables 3-2, 3-3, and 3-4. It is worth noting that the analysis was carried for a wall without eccentricity and a wall with eccentricities of 0.4 inch, 0.6 inch, and 0.8 inch.

3.3.4 Evaluation of the Correlation Results

As previously mentioned, the results of several parameters were obtained and are discussed below:

Table 3-1. Summary of Wall Test Results

PROPRIETARY INFORMATION
HAS BEEN REMOVED

Table 3-2. Results for Wall Type 1A

PROPRIETARY INFORMATION
HAS BEEN REMOVED

Table 3-3. Results for Wall Type 1B

PROPRIETARY INFORMATION
HAS BEEN REMOVED

Table 3-4. Results for Wall Type 1C

PROPRIETARY INFORMATION
HAS BEEN REMOVED

PROPRIETARY INFORMATION
HAS BEEN REMOVED

- o In general, the methodology is capable of capturing the overall wall behavior.

Further assessments of the correlation study are provided in Appendix C.

3.3.5 Analysis Results

As a part of the reevalution of 12 panels in the Ginna control building, an analysis was performed using the working stress method to determine whether any of these panels could satisfy the SGEB criteria by elastic analysis prior to employing the nonlinear analysis. One of the key features of the new elastic analysis was to consider the effects of the reinforcement, which was conservatively ignored in the original analysis.

The results of this analysis are tabulated in Table 3-5. It is obvious from this table that only two panels, 972-1C and 972-3C, were qualified by elastic analysis. The remaining ten panels have rebar stresses in excess of the allowable limits and therefore would require qualification by the inelastic analysis methodology.

Three critical panels were selected for nonlinear analysis: two spanning panels and one cantilever panel. Of the spanning panels, 971-1C and 971-6C were selected based on their highest and lowest steel stress ratio obtained from the elastic analysis. In this way, results for other spanning panels can be obtained by interpolating the results of these two panels. Of the three cantilever panels, panel 973-4C had a high stress ratio and low thickness (6 inches). This panel was selected for the analysis.

For the nonlinear analysis, the results of the correlation study described in Section 3.3.4 were factored in the nonlinear model, which include the following: the length of the yielding rebar was set as a variable in the model (as opposed to a preset fixed length in the original model) such that it would vary according to displacement; the eccentricity of the rebar was also included in the modified model. In addition, the effect of the wall self-weight was included in the modified model.

Panel 971-1C

Panel 971-1C is 16 feet high, 38 feet and 1 inch long. No. 3 rebars are at 32-inch spacing and Dur-O-Wal is in the horizontal direction.

Table 3-5. Ginna Control Building Walls: Elastic Analysis

WALL I.D.	MAXIMUM STRESSES				MAXIMUM STRESS RATIOS			
	HORIZONTAL		VERTICAL		HORIZONTAL		VERTICAL	
	Masonry	Steel	Masonry	Steel	Masonry	Steel	Masonry	Steel
SPANNING WALLS								
971-1C	82	19.0	677	78.3	0.09	0.30	0.67	2.18
971-2C	77	17.8	598	69.2	0.17	0.28	0.60	1.92
971-4C	61	14.0	677	55.3	0.06	0.22	0.67	1.54
971-5C	57	13.0	537	62.2	0.06	0.20	0.54	1.73
971-6C	62	14.3	389	45.1	0.06	0.23	0.39	1.25
972-4C/5C	107	24.6	558	64.5	0.11	0.39	0.55	1.79
972-6C	92	20.9	616	71.3	0.09	0.33	0.61	1.98
CANTILEVER WALLS								
972-1C	(1)							
972-2C	57	13.1	513	51.3	0.06	0.20	0.51	1.43
972-3C	84	6.9	337	33.7	0.09	0.11	0.33	0.94
973-3C	40	7.7	450	36.6	0.04	0.12	0.45	1.02
973-4C	92	17.8	579	47.3	0.09	0.28	0.58	1.31

NOTES:

1. Wall 972-1C did not crack.

2. Stresses are in psi for masonry and ksi for steel.

3. Maximum allowable stresses after cracking are assumed to be 0.85f'm for masonry and 0.9Fy for steel. The provides 1003 psi for masonry, 63 ksi for Dur-O-Wal and 36 ksi for vertical steel.

The time histories based on the El Centro, Olympia, and Taft time histories were used in the analysis. In addition, rebar eccentricities of 0.4 in, 0.6 in, and 0.8 in were evaluated using the El Centro based record. The results of the analysis are presented in Table 3-6.

Panel 971-6C

Panel 971-6C has construction details that are similar to those of panel 971-1C; the major difference is the amount of openings. The results of the analysis are shown in Table 3-7.

Panel 973-4C

Since panel 973-4C is a doubly reinforced panel, the hysteresis shape of the panel is not similar to those of the spanning panels; thus, the detailed model (instead of the substructure model) was employed for the analysis. To account for uncertainties in rebar placement, two eccentricities were considered:

- Each bar to be placed against the face shell rather than with a 0.5-in cover.
- One of two bars was assumed to be misplaced by 0.5 in towards the center of the grouted cell.

The results of the analysis are presented in Table 3-8.

It can be seen from Tables 3-6, 3-7, and 3-8 that the masonry and steel strain for the three selected panels are well below the Licensee's criteria. As discussed in Section 3.3.1, the new correlation study of the nonlinear analytical procedures using actual material properties and input motions resulted in a better correlation with test data. It is, therefore, concluded that the panels being analyzed will withstand the postulated seismic excitation.

3.4 EVALUATION OF LICENSEE'S APPROACH TO WALL MODIFICATIONS

Of the 56 safety-related panel panels at the Ginna plant, 29 meet the SGEB criteria after modifications.

Table 3-6. Results for Wall 971-1C

RESPONSE	EL CENTRO BASED RECORD			OLYMPIA BASED RECORD	TAFT BASED RECORD
	ECC = 0"	ECC = 0.4"	ECC = 0.8"		
CENTER DISPLACEMENT Maximum (inches)	1.82	1.88	2.38	1.72	1.98
MASONRY COMPRESSIVE STRAIN					
Mid-Height	0.0011	0.0011	0.0012	0.0011	0.0011
Base	0.0013	0.0013	0.0013	0.0013	0.0013
STEEL STRAIN RATIOS					
Mid-Height	2.2	2.5	3.7	1.9	2.3
Base	5.1	5.6	6.2	4.8	5.5
LENGTH OF YIELDING REBAR (inches)					
Mid-height	32	40	40	24	32
Base	8	8	816	8	8
MAXIMUM GAP OPENING					
Mid-Height (inches)	0.05	0.06	0.08	0.05	0.05
Base (inches)	0.13	0.12	0.12	0.12	0.14

Table 3-7. Results for Wall 971-6C

RESPONSE PARAMETER	EL CENTRO BASED RECORD	OLYMPIA BASED RECORD	TAFT BASED RECORD
CENTER DISPLACEMENT			
Maximum (inches)	1.26	1.89	2.20
MASONRY COMPRESSIVE STRAIN			
Mid-Height	0.0011	0.0011	0.0012
Base	0.0012	0.0013	0.0013
STEEL STRAIN RATIOS			
Mid-Height	-	2.5	2.9
Base	3.8	5.8	6.5
LENGTH OF YIELDING REBAR (inches)			
Mid-height	-	40	40
Base	8	8	8
MAXIMUM GAP OPENING			
Mid-Height (inches)	0.02	0.06	0.07
Base (inches)	0.09	0.14	0.16

Table 3-8. Results for Wall 973-4C

RESPONSE	EL CENTRO BASED RECORD			OLYMPIA BASED RECORD	TAFT BASED RECORD
	ECC = 0"	ECC = 0.375"	ECC = 0.50"		
TOP DISPLACEMENT Maximum (Inches)	2.95	3.23	3.22	3.50	3.59
MASONRY COMPRESSIVE STRAIN Base	0.0014	0.0017	0.0013	0.0016	0.0017
STEEL STRAIN RATIOS Base	2.2	2.5	1.3	3.0	4.2
LENGTH OF YIELDING REBAR (Inches) Base	8	8	8	8	8
MAXIMUM GAP OPENING Base (Inches)	0.03	0.03	0.02	0.04	0.05

Modifications to the 29 panels that meet the SGEB criteria consisted of the addition of structural steel members at the edges to assure that the boundary conditions (pinned) are as assumed in the analysis (Figure 3-6). In some cases, intermediate supports were added across the face of the panel to reduce the panel span length.

The Licensee's approach to panel modifications has been reviewed and found adequate.

Of the remaining 26 panels, 2 panels were qualified by a refined analysis using the working stress method, 10 panels were qualified by the nonlinear analysis method, and the other 14 panels will be modified. The Licensee has made a commitment to complete all modifications by late 1988 to mid-1989. These 14 panels will be modified in accordance with the table below:

Panel ID	Corrective Measure
973-17A (3)	Provide net to catch loose blocks
973-17A (4)	
973-17A (7)	
972-1I	Provide device to protect safety-related equipment from falling blocks
972-3I	
972-4I	
972-8I	
972-10I	
972-12I (5)	
972-12I (7)	
972-121 (8)	
973-1I (A)	
973-1I (C)	
973-1I (D)	

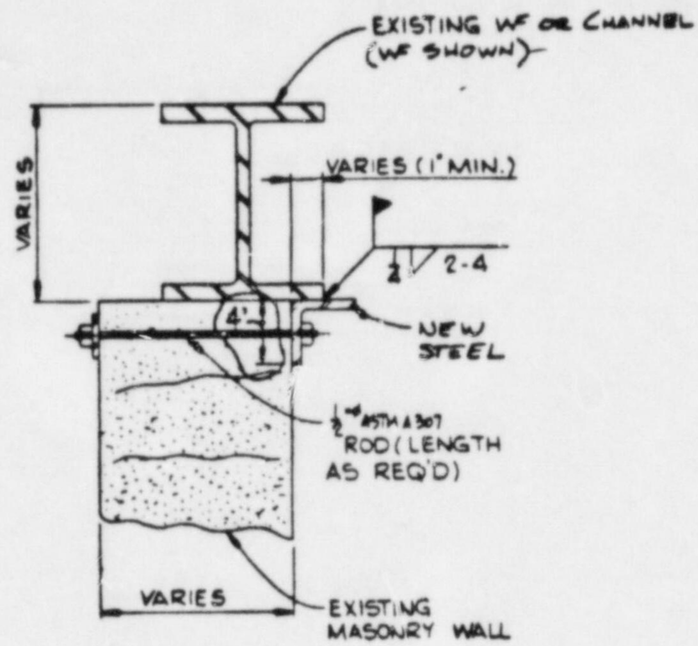

Figure 3-6. Typical Modification

4. CONCLUSIONS

A detailed study was performed to provide a technical evaluation of the masonry walls at the R. E. Ginna Nuclear Power Plant. 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 According to the NRC's Safety Evaluation Report (August 22, 1983), the loads for the safety-related structures at the Ginna plant were determined by the Systematic Evaluation Program and did not include the OBE loads. The reason for this was consideration of the SSE as the governing case and the major safety concern. Therefore, the evaluation of the masonry walls for only the SSE condition is adequate.

- o An increase factor of 1.5 for allowable tension stress normal to the bed joint was used instead of the factor of 1.3 specified by the SGEB criteria. However, even if the 1.3 factor were used, the factored allowable stress would be exceeded in only two cases, walls 3-17A-5 and Z-ZI, by 10% and 7%, respectively.

- o Of the 56 safety-related wall panels, 29 wall panels meet the SGEB criteria after structural modifications. These modifications include the addition of structural steel at the edges to ensure that the actual boundary conditions are as assumed in the analysis. In some cases, intermediate supports have been added across the wall face to reduce the span length of the wall. Twenty-seven walls do not meet the SGEB criteria, although one of these, 971-ZM, is a lead brick wall whose failure would not damage any safety-related equipment. Twelve panels were qualified by refined analyses. The remaining 14 panels will be modified as outlined in Section 3.4.

With regard to the nonlinear analytical procedures, the following conclusions are reached:

- o The new correlation study of the nonlinear analytical procedures using actual material properties and input motions resulted in better correlation with test data.

- o The results illustrated that ductility demand for the walls being analyzed is not high. Steel and masonry strains are well below the specified criteria.

- o The analytical procedures are judged to be adequate and the walls are expected to perform their intended functions during postulated seismic excitation.

Further assessments of the nonlinear analytical procedures are given in Appendix C.

The Licensee has made a commitment to modify the remaining 14 panels by late 1988 to mid-1989. The Licensee's approach to wall modifications has been reviewed and judged to be satisfactory.

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   Letter with attachments to G. E. Lear (NRC)
   Subject: Seismic Masonry Wall Analyses, R. E. Ginna Nuclear Power Plant
   Rochester Gas and Electric Corporation
   December 19, 1985

8. R. W. Kober
   Letter with attachments to G. E. Lear (NRC)
   Subject: Masonry Wall Design, R. E. Ginna Nuclear Power Plant
   Rochester Gas and Electric Corporation
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9. Building Code Requirements for Concrete Masonry Structures
   Detroit, American Concrete Institute, 1979
   ACI 531-79 and ACI 531-R-79

10. Dr. L. R. Jones
Presentation to NRC
Computech Engineering Services, Inc., September 24, 1985

11. "Masonry Wall Design, Southern California Edison Company San Onofre Unit 1," Technical Evaluation Report TER-C5506-405, Franklin Research Center, June 28, 1985

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 & RACE STREETS, PHILADELPHIA, PA 19103

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1. <u>General Requirements</u>

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. <u>Loads and Load Combinations</u>

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) <u>Service Load Conditions</u>

(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 (6) 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. References

(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

EVALUATION OF LICENSEE RESPONSES

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The following is a review of the Licensee's responses [July 13, 1984] to the NRC's request for additional information.

Question 1

> The SGEB criteria [1] do not allow an increase in allowable stresses for load combinations containing OBE or wind loads. Provide justification for the 1/3 increase in allowable stress [Reference 3, Section 3.4.2] used for load combinations (normal operating conditions). Identify the affected walls and include the calculated stresses for each wall. Also explain all conservative measures (if any) used in the analysis to justify the increase in allowable stresses.

Response 1

The Licensee indicated that, according to the NRC's Safety Evaluation Report (August 22, 1983), the loads for safety-related structures at the Ginna plant were defined by the Systematic Evaluation Program (SEP) and did not include OBE loads; the reason was that SSE was the more severe case and the major safety concern. The masonry walls at the Ginna plant, therefore, were not evaluated for OBE loads. Thus, the 1/3 increase in allowable stress was not used. Also, no increase in allowable stress was used for normal wind loadings.

This response is satisfactory and consistent with the SGEB criteria.

Question 2

> Justify the use of an allowable stress increase factor of 1.67 for load combinations containing accident pressures or SSE loads. This is in excess of several factors permitted by the SGEB criteria (1); they are listed below by type of stress:

masonry shear in flexural members	1.3
masonry shear in unreinforced shear walls	1.3
reinforcement takes entire shear	1.5
tension normal to bed joint	1.3
tension parallel to bed joint	1.5

> If any existing test data will be used to justify this increase factor, discuss the applicability of these tests to the walls at the Ginna plant with particular emphasis on the following:
>
> - boundary conditions
> - nature of loads

- size of test walls
- type of masonry construction (block or mortary type, grouted or ungrouted).

The Licensee is also requested to indicate the number of walls that would not be qualified if the SGEB criteria were to be used and to specify the percentages of exceedance. The Licensee is advised to explain all conservative measures (if any) used in the analysis to justify this increase factor.

Response 2

In this response, the Licensee indicated that the SGEB increase factors for allowable stress were used with the exception of the factor for tension normal to the bed joint, which was 1.5 instead of 1.3. However, even if the factor of 1.3 was used, the factored allowable stress would be exceeded in only two cases, walls 3-17A-5 and 2-2I, by 10% and 7%, respectively. The actual stresses correspond to increase factors of 1.43 and 1.38. These discrepancies are small, and walls 3-17A-5 and 2-2I are considered acceptable.

Question 3

When the response spectrum method of seismic analysis is used, the accelerations of walls on a particular floor should be based on the floor response spectrum for the floor elevation. However, as stated in Section 3.5.1 of Reference 2, the Licensee derives all wall accelerations from the ground response spectrum. Justify the use of the ground response spectrum instead of the floor response spectra.

Response 3

The Licensee stated that, although response spectra were not initially used in the evaluation of the walls, all walls have been reanlyzed using the appropriate floor response spectra and the site-specific earthquake (0.17 g) developed as part of the SEP program. Where the wall was supported at both the base and the top, the average response from the two elevations was used; if the top was free, the spectrum at the floor elevation on which the wall rests was used. Also, the OBE condition was not considered becvause the SSE condition was the governing case and the major safety concern (see Response 1).

This response is satisfactory.

Question 4

With reference to the reinforcement in masonry walls, the ACI 531-79 Code specifies that the minimum area of reinforcement in a wall in either direction, vertical or horizontal, shall be 0.007 (0.7%) times the gross cross-sectional area of the wall and that the minimum total area of steel, vertical and horizontal, shall not be less than 0.002 (0.2%) times the gross cross-sectional area. In view of this, clarify whether the reinforced walls at this plant meet the above requirements. The Licensee is also requested to provide the type and spacing of vertical reinforcement and the total number of vertically reinforced walls. It should be noted that the horizontal reinforcement is installed to satisfy the minimum reinforcement requirement for a reinforced walls.

With reference to the joint reinforcement, identify the number of walls qualified by the tensile strength of joint reinforcement and indicate the type and spacing of the joint reinforcement.

Based on the review of existing codes and published literature, the NRC does not, at present, approve the use of joint reinforcement, as a structural element. A staff position on this issue is being developed and will be provided to the licensee in the future.

Response 4

In this response, the Licensee indicated that 12 of the 37 safety-related walls contain vertical reinforcement: seven have one #3 bar on 32-inch centers and five have two #2 bars on 16-inch centers. Dur-O-Wal joint reinforcement is presented in all walls but was not used in the structural analysis. All 29 wall panels meeting the SGEB criteria (see Response 15) were qualified as unreinforced walls. Therefore, the amount of vertical or horizontal reinforcement in these walls is irrelevant.

For walls in the control building, the response is satisfactory.

Question 5

Indicate the boundary conditions used in the analysis and verify that they resemble the real physical conditions. Identify all of the mechanisms used to transfer shear and moment (if any) with particular emphasis for walls qualified by arching action. If any doubt exists (i.e., whether simply supported or fixed-end conditions should be assumed), verify that the assumed boundary conditions will produce conservative results.

Response 5

The Licensee indicated that the boundary conditions may be fixed, pinned, or free. The fixed condition is found only at wall bases where steel dowels tie the wall to the concrete slab. Pinned conditions occur where translation is restricted by a mortar joint or installed modifications, such as a structural steel member supporting the edge. Most of the analyzed walls have pinned boundary conditions.

This response is satisfactory.

Question 6

> Indicate how interstory drift effects, both in-plane and out-of-plane, were considered in the analysis. Also, indicate and justify by available test data the permissible strains used for both confined and unconfined walls.

Response 6

Regarding in-plane interstory drift effects, the Licensee indicated that no structural mechanism existed that would induce significant in-plane drift in the walls and that no walls at the Ginna plant served to resist building shear loads. With respect to out-of-plane interstory drift effects, the Licensee indicated that these were also insignificant because none of the masonry walls have fixed ends at both the base and ceiling; therefore, a small out-of-plane interstory displacement will not have a severe effect on the walls.

This response is adequate and complies with SGEB criteria.

Question 7

> Indicate whether concrete block walls are stacked or running bond. If any stack bond wall exists, provide sample calculations for stressses in a typical wall. Also identify the number of stacked bond walls and their appropriate allowable stresses.

Response 7

The Licensee stated that all safety-related masonry walls at the Ginna plant are running bond.

Question 8

Reference 2 indicated that some brick walls were constructed at the plant. Indicate that number of brick walls and specify the allowable stresses from appropriate codes used in the analysis. If any increase factor were used for SSE loading case, justification should be provided.

Response 8

In this response, the Licensee stated that one wall, 971-2M, was constructed of lead brick. It was also determined, upon further investigation, that the wall was adequately restrained in one direction by a framework of steel members and failure in the other direction will not affect any safety-related equipment. Wall 971-2M, therefore, is not dependent on any masonry allowable stresses but is nevertheless, adequate seismically.

This response is acceptable.

Question 9

With reference to Section 3.2.5 of Reference 2, the Licensee indicated that accident pressure and associated temperature loads are considered only inside containment when applicable. Provide a sample calculation (and any explanations necessary to make it understandable) illustrating the analysis procedure in this case.

Response 9

The Licensee stated that no safety-related masonry walls are subject to pressure or temperature differentials. Therefore, the analysis for these conditions is not applicable.

This response is adequate.

Question 10

In Section 3.5.1 of Reference 2, the Licensee indicated that the computed stresses are increased 5% to account for higher modes of vibration. Justify by sample calculation that 5% is an appropriate percentage for multimode effects.

Response 10

In this response, it was indicated that all of the masonry walls have been reanalyzed. In the latest analysis, all modes of vibration up to 33 Hz were combined to determine the response of the wall. Walls with natural frequencies greater than 33 Hz were considered rigid and the floor response acceleration for 33 Hz was used for analysis.

This response is adequate and complies with the SGEB criteria.

Question 11

Provide sample calculations (with explanations necessary to make the calculations understandable) for:

- a single-wythe wall analysis
- a multi-wythe wall analysis
- a brick wall analysis

Response 11

There are no multi-wythe masonry walls or brick walls at the Ginna plant. As for the single-wythe wall analysis, sample calculations were provided showing that the walls were analyzed as finite element plate models. The walls geometry, boundary conditions, material properties, and loadings were input into a SAM 4 computer program, which calculated the stresses. The computed stresses were then compared with allowables defined in ACI 531-79 [3].

This response is satisfactory and consistent with the SGEB criteria.

Question 12

According to Attachment 3 of Reference 2, only 84 walls were identified as safety-related; however, in a meeting at the NRC on January 20, 1983 with regard to the use of the nonlinear analysis technique (arching theory), the Licensee identified 101 walls qualified by arching theory. Explain this discrepancy.

Response 12

The Licensee responded that in its initial submittal [2] in response to IE Bulletin 80-11, 84 safety-related walls were reported; however, these walls correspond to 101 panels. The walls were subdivided into panels to facilitate engineering analysis.

Since the initial submittal, the number of walls and panels have been reduced due to work done under the SEP program. The current number of safety-related walls is 37, which corresponds to 56 panels. Arching action was not used to qualify any walls at the Ginna plant.

This response is adequate.

Question 13

Indicate how the uncertainties due to variations in mass, materials, and sections properties were accounted for in the analysis.

Response 13

In this response, the Licensee stated that the floor response spectra were broadened by 15% in the seismic analysis to account for uncertainties in the physical characteristics of the wall.

This response is adequate and consistent with SGEB criteria.

Question 14

Indicate whether collar joint strength has been used in the analysis. If so, provide and justify the allowable stresses of the collar joint.

Response 14

There are no multi-wythe safety-related masonry walls at the Ginna plant. The question of collar joint strength is, therefore, not applicable.

This response is adequate.

Question 15

Confirm whether all modifications have been completed and the modified walls are in compliance with the SGEB criteria.

Response 15

With the modifications currently installed, 29 out of 56 panels meet the SGEB criteria. Of the remaining 27 panels, one failure was deemed acceptable, wall 873-3M (see Response 8). Twenty-six panels require further modifications to meet the SGEB criteria. However, as indicated in Section 4 of this report, only 14 of 26 panels will be modified (the other 12 panels were qualified by refined analysis).

This response is adequate.

Question 16

> Explain how earthquake motions in three directions are treated in the analysis. Indicate whether any walls are subject to in-plane loading. If so, provide a sample calculation illustrating how the wall is qualified with respect to the SGEB criteria.

Response 16

The Licensee explained that the walls were evaluated for vertical plus out-of-plane earthquake directions and vertical plus in-plane earthquake directions. The vertical plus out-of-plane combination was the governing case. A sample calculation was also provided showing an analysis for in-plane loading. In this calculation, the wall was assumed to act as a vertical cantilever with vertical seismic reactions and horizontal seismic reactions and horizontal seismic reactions combined at the base.

This response is satisfactory and consistent with the SGEB criteria.

Question 17

> Explain and justify how cracked and uncracked moment of inertia was calculated.

Response 17

The Licensee indicated that no walls to date have been qualified using a cracked section analysis. Uncracked moments of inertia were calculated based on the geometry of the masonry units and conventional mechanics.

This response is adequate (nonlinear analysis).

Question 18

In Attachment 4 of Reference 2, the Licensee provided the test data for the compressive strength of concrete masonry walls. Provide the basis for selecting those five specimens for testing and indicate whether they represent the variety of material construction in all buildings. Since no detailed records relating to the masonry blocks walls construction were maintained, justify the strength of the mortar used in the analysis.

Response 18

The Licensee responded that two 12-inch hollow blocks and one 8-inch hollow block were removed from unreinforced walls in the intermediate building and that two 8-inch blocks were removed from the control building where the majority of reinforced walls are located. The selection was based on the representation of the greatest number of similar walls, according to thickness and wall construction type. Approximately 55% of the safety-related masonry walls have 12-inch-thick blocks and 45% have 8-inch-thick blocks. The mortar strength used for analysis was in accordance with ASTM C270-64T, Type N, which was specified in the construction specification, SP-5360.

This response is satisfactory.

REFERENCES

1. SGEB Criteria for Safety-Related Masonry Wall Evaluation
Developed by the Structural and Geotechnical Branch of the NRC, July 1981

2. J. E. Maier
Letter to B. H. Grier, NRC
Subject: IE Bulletin 80-11, Masonry Wall Design, R. E. Ginna Nuclear Power Plant
Rochester Gas and Electric Corp., 04-Nov.-80

3. Building Code Requirements for Concrete Masonry Structures
Detroit: American Concrete Institute, 1979 ACI 531-79 and ACI 531-R-79

APPENDIX C

"EVALUATION OF NONLINEAR ANALYSIS OF MASONRY WALLS IN THE CONTROL BUILDING OF R. E. GINNA NUCLEAR POWER PLANT," BY A. A. HAMID

Technical Report

on

EVALUATION OF NONLINEAR ANALYSIS OF MASONRY WALLS IN THE CONTROL BUILDING OF R.E. GINNA NUCLEAR POWER PLANT

submitted

to

Dr. Vu Con
Nuclear Engineering Department
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April 1986

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1. INTRODUCTION

Reinforced block masonry walls at R. E. Ginna Nuclear Power Plant have originally been analyzed by Gilbert Associates, Inc. using elastic analysis of one-way bending and SEB criteria for allowable stresses. A total of 12 walls in the Control Building were not qualified based on elastic analysis because maximum stresses exceeded the allowable stresses specified for working stress design of masonry. Steel stresses in some walls exceeded the yield stresses indicating an inelastic response under SSE loads.

The NRC and FRC staff and consultants discussed during the meeting of May 22, 1985 with R.E.&G. staff the need for more rigorous analysis to demonstrate the adequacy of Ginna walls to resist SSE input motion with adequate ductility. The licensee proposed to perform an inelastic one-way analysis of masonry walls at Ginna Plant.

Computech Engineering Services, (CES), conducted an inelastic dynamic analysis of the critical walls utilizing the nonlinear analytical methodology that was developed for SONGS-1 walls with necessary modification for Ginna walls. The NRC required, as a condition for accepting the use of the methodology at the Ginna Plant, that a more detailed correlation study of the Computech nonlinear analytical methodology be performed.

This report presents the writer's review and evaluation of CES anaytical methodology for Ginna walls as presented in Reports R 562-N3 (3) and R 562-N4 (2).

2. REVIEW OF CORRELATION OF SONGS-1 TEST DATA

2.1 Approach

Following completion of SONGS-1 test program a limited correlation study was performed. The NRC and FRC staff and consultants indicated that parametric study will help in providing more confidence in the proposed analytical methodology (1). Due to the fact that dynamic tests of walls representing actual construction at SONGS-1 demonstrated the adequacy of the walls to resist SSE load, further correlation was not deemed necessary for SONGS-1 walls. The staff in their evaluation of SONGS-1 data indicated that more detailed correlation study including actual parameters must be conducted prior to the application of the metnodology to the Ginna walls. These parameters include differences in analytical and test material properties and input motion, possible eccentricity of rebar, length of plastic hinge and permenant set (1).

Computech new correlation study (2) using actual material properties and input motions contains parametric study of the effect of rebar eccentricity and length of plastic hinge. The correlation study was performed by numerically comparing the match between the analytical and experimental results of walls 1A, 1B and 1C. It is noted that test walls were driven to the inelastic range as evident from the steel strain ratios

PROPRIETARY INFORMATION
HAS BEEN REMOVED

performed with rebar eccentricities of 0.0, 0.4, 0.6 and 0.8 inches. The results are listed in Tables 2.2, 2.3 and 2.4 of Ref. 2 for walls 1A, 1B and 1C, respectively. The resuts indicate the sensitivity of wall displacement to rebar eccentricity. Other parameters such as steel strain ratio and masonry strain are less sensitive to rebar eccentricity. It is interesting to notice that maximum response does not correspond to maximum eccentricity. It is not clear from the results that including bar eccentricity provides better correlation with test results. The effect of eccentricity on wall dynamic response is not well explained. However, the study showed a + 35% difference in response parameters due to the effect of eccentricity with the trend of higher response with larger eccentricities.

The analysis was not able to predict the permenant set experienced after wall tests. This parameter, however, should not be critical for Ginna walls due to the small displacement predicted. The new analysis with actual input data resulted in a better correlation with the test data for steel strain ratio and length of yielding compared to the analysis that have been previously conducted for SONGS-1 walls.

3. REVIEW OF ANALYSIS OF GINNA WALLS

3.1 Approach

Elastic two-way analysis was conducted on the 12 walls in the control building. Inelastic analysis was limited to the worst-case spanning wall and cantilever wall. The walls in the control building have the same construction details. Therefore, limiting the analysis to the worst-cases of the two types is justifiable.

3.2 Elastic Analysis

A finite element two-way analysis was performed for the 12 walls in the control building. If stresses exceeded the specified tension values then the wall was considered cracked and stresses in masonry and in vertical and horizontal steel were checked aganist the allowable stress criteria (0.85 f'_m for masonry and 0.9 f_y for steel). If the stresses are within these allowables the wall is considered qualified. If not, inelastic analysis, was considered.

Wall cracking was based on the bond strength of the mortar. In grouted masonry, grouting significantly increases the cracking load of the wall. Values for masonry tension used in the analysis highly underestimate the cracking capacity of the walls. They are, however, lower bound values and would lead to conservative results.

Results of the elastic analysis (Table 2.1, Ref. 1) revealed that stresses in the vertical steel exceeded the criteria limit (0.9 f_y) in all the seven spanning walls and in three of the five cantilever walls, and therefore inelastic analysis deemed necessary.

3.3 Inelastic Analysis

Spanning wall 971-1C has the highest steel ratio (actual/ 0.9 f_y) and therefore was selected for inelastic analysis. Wall 973-4C was also selected for the inelastic analysis because it had the highest steel ratio of the 6" thick cantilever walls. An additional spanning wall (971-6C) having lower steel ratio was also analyzed to provide a basis for interpolation of the remaining walls. The inelastic analysis of wall 971-1C was conducted for different rebar eccentricities and different input motions are presented in Table 4.1 of Ref. 1. The maximum wall displacement of 2.38" represents a low ratio of displacement to wall height which indicates no problem regarding wall instability. The results show the sensitivity of wall displacements to rebar eccentricity with the highest response corresponds to the largest eccentricity.

Results for wall 971-6C (Table 4.2, Ref. 1.) showed a much higher reponse for Taft and Olympia motions than for El Centro which was not the case for wall 971-1C. It is interesting to notice that maximum displacement of wall 971-6C for Taft motion was higher than that for wall 971-1C which is, according to elastic analysis, the most critical wall. This could be attributed to the sensitivity of wall response to the characteristics of input motion.

Displacement-time history plot for wall 971-1C presented in Fig. 4.3 of Ref. 1 shows unsymmetric response for Olympia based record which was not the case for other input motions and/or with other walls with the same motion.

In the inelastic analysis the contribution of joint reinforcement was ignored and the walls were analyzed for one-way bending in the vertical direction. This assumption is conservative and would lead to an upper bound for stresses in the vertical steel. The ductility demand, range from 1.1 to 2.4, was not high and steel and masonry strains were well below the criteria limits.

4. CONCLUSIONS

The new correlation study of the inelastic analysis for SONGS-1 test walls considering actual material properties and input motions resulted in a better correlation with test data. The analysis indicates the sensitivity of wall response to rebar eccentricity.

Despite the lack of explanation of the effect of some of the parameters on wall response and the inability of the model to predict permenant set, the methodology is reasonably capable of capturing the overall wall response. Assumptions used in the analysis of Ginna walls are conservative. The results indicate that ductility demand for the walls in the Control Building is not high. Steel and masonry strains are well below the criteria limits indicating satisfactory performance of Ginna walls under the SSE loading.

5. REFERENCES

1. Harris, H. and Hamid, A., "Technical Evaluation of The San Onofre Nuclear Generating Station (SONGS), Unit 1 Masonry Walls," Franklin Research Center, June 1985.

2. Computech Engineering Services, Inc., "Reinforced Masonry Wall Evaluation - Correlation of SONGS-1 Test Data," Report No. R 562-N3, December 1985.

3. Computech Engineering Services, Inc., "R.E. Ginna Power Plant-Evaluation of Control Building Reinforced Walls." Report No. R 562-N4, December 1985.