ST LUCIE PLANT - UNIT NO. 1
NRC IE BULLETIN 80-11
FINAL REPORT

PREPARED BY
EBASCO SERVICES INCORPORATED
FOR
FLORIDA POWER & LIGHT COMPANY

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I. Introduction

On May 8, 1980 the NRC issued IE Bulletin 80-11 on the subject of design of masonry walls. Florida Power & Light Company in response, through their architect-engineer, Ebasco Services Incorporated, instituted a field inspection program and design re-evaluation program to verify the adequacy of the existing masonry wall design as requested by the bulletin.

The field inspection program was completed in October, 1980. By then the re-evaluation criteria had been finalized and the analysis portion of the program was underway. The re-evaluation of the walls was essentially completed by the end of December 1980. There remained only the final analysis of a few walls and the investigation of the ceiling attachments for the full-height walls. This portion of the program was completed in January 1981.

FPL in its initial response to Bulletin 80-11 of July 24, 1980 (Ltr L-80-233) addressed items 1, 2a, and 3 of the bulletin, describing the planned inspection and re-evaluation phases of the program. A second interim response of November 4, 1980 (Ltr L-80-374) reported the completion of the field inspection and the development of the re-evaluation criteria and requested an extension to February 9, 1981 for submittal of the final report to the NRC.

The following report presents in detail the information requested in item 2.b of Bulletin 80-11. The procedures for the inspection and verification programs are discussed, as well as the results of those programs and corrective actions taken.

II. Description of Masonry Walls

204 masonry walls were constructed in the Reactor Auxiliary Building and Fuel Handling Building for St Lucie Unit 1. All the safety-related masonry walls are located in the Reactor Auxiliary Building. The functions of the walls include pressure retention (primarily for maintaining HVAC flow balancing), security, personnel control and shielding for radiation exposure reduction. Wall construction included both stacked and running bond types. Where multiple thicknesses were provided for shielding purposes, the joints were staggered.

Of the 203 walls in the RAB, 101 were originally designed for seismic loading. The remaining 102 walls in the RAB and one wall in the FHB were not designed for seismic loading. The seismically designed walls were provided with vertical reinforcement consisting of eight #4 reinforcing bars, four in each cell, and the cells were filled with mortar. These reinforced units are spaced 4' - 0 on centers. "Dur-O-Wal" horizontal truss reinforcement was placed at every mortar joint during erection of the reinforced masonry walls.

The walls not designed for seismic loading were provided with reinforcement consisting of "Dur-O-Wal" every third course. These are described as "unreinforced walls" elsewhere in this report.

No masonry ties between the wythes were provided for multi-wythe walls.

The materials of construction used were as follows:

Masonry Units - ASTM C90 Grade N
Mortar - ASTM C270 Type S
Reinforcing Steel - ASTM A615 Grade 40
Structural Steel (supporting angles, embedments) - ASTM A36

Masonry walls which are in proximity to or have attachments from safety-related piping or equipment such that wall failure could affect a safety-related system are designated as safety-related walls.

The inspection program identified 90 walls falling into this category. Of these, 65 were reinforced and 25 were unreinforced. All safety-related walls required design verification to establish structural adequacy to carry postulated design loads. The design verification program is described in Section V of this report.

The remaining 114 walls were designated "not safety-related;" no further evaluation of these walls was required.

III. Construction Practices

A - Reinforced Block Walls

The block walls are supported by a 2' - 0 high concrete starter wall which is doweled into the floor slab with number 6 reinforcing bars on 12 inch centers on each face. The reinforced block wall units, spaced at every 4 feet, received 8 number 4 reinforcing bars. The positioning of the reinforcing bars was accomplished by building up the block wall to four courses high then filling the cells with type "S" mortar, rodding the mortar to achieve homogeneity. Then the 3' - 0 long reinforcing bars were inserted into the mortar-filled cells leaving a projection of four inches beyond the blocks.

Eight splice bars were introduced adjacent to the original bars, projecting three feet and splicing 1' - 0 with the initial rebar. Next the blocks were inserted over the projecting reinforcing and built up to four additional courses, continually filling the cells with mortar and rodding the mortar. The sequence was repeated until the wall was built up to within four courses of the ceiling.

In order to position the last four courses in such a way that the reinforcing was continuous to the ceiling, the sidewall of the block was chipped out and the rebar inserted horizontally. The cell was then filled with mortar and the chipped out sidewall of the block repaired with the same mortar.

Horizontal reinforcing (Dur-O-Wal) was placed at every mortar joint during the erection of the block wall.

Horizontal reinforcement splices were staggered vertically so that no splices in two adjoining courses are less than 8 inches horizontally apart. Where a multiple thickness wall was required due to shielding considerations the vertical joints were staggered.

B - Unreinforced Block Walls

A 3/8" mortar bedding is provided on top of the slab and the block is layed on this bedding, every third course receives "Dur-O-Wal" horizontal reinforcing.

For partial height walls that receive a precast slab over them, the last course is a precast bond beam with two number 5 reinforcing bars (continuous). The bond beam is connected to the roof slab by number 4 bars at 3' - 4 O.C. The cell is then filled with mortar.

Full height walls spanning from floor to ceiling received Dur-O-Wal at every third course and a bond beam at approximately mid-height and at the top of the wall, with similar reinforcing and grouting to that provided for the partial height walls.

IV. Inspection of Masonry Walls

Pursuant to the requirements of NRC IE Bulletin 80-11 dated May 8, 1980, a field inspection program was developed to "identify all masonry walls in your facility which are in proximity to or have attachments from safety-related piping or equipment such that wall failure could affect a safety-related system."

In accordance with the Procedure for Inspection of Concrete Masonry Walls, FLO-128-4.800, Rev 3, the field inspection program consisted of two phases. Phase I inspection included a review of the site general arrangement and concrete masonry wall drawings, to determine the extent of the inspection. A master set of reference drawings showing all masonry walls was assembled and each wall was assigned a unique identification number. These master reference drawings are marked-up reproducible copies of general arrange-

ment drawings, showing wall location and identification and issued under Backfit Change Sketch numbers BCS 128-4.300 thru .304.

An inspection data sheet was prepared (FLO 128-4.800 Att #1) for each wall and the Phase I portion of the sheet completed. This portion included the wall unique identification number and orientation of the wall as shown on the design drawings. Reference was made to the master reference drawing (BCS) showing the wall, and a description of the plant location, including building, floor elevation and reference to major equipment in the area. In addition, the design function of the wall was listed at this time. These included pressure retaining or primarily for maintaining HVAC flow balancing, security or partition walls for personnel control and shield walls for radiation exposure reduction. Phase I work also included a review of the wall design construction drawings and a listing of the thickness and composition of the wall, including reinforcement details and multiple wythe construction.

Upon completion of the Phase I portion of the inspection data sheets, the field inspection program or Phase II portion was implemented.

Inspectors performing Phase II field inspections were Ebasco design or engineering personnel, familiar with power plant operation and safety-related equipment identification. The inspectors were briefed, documented, certified, in accordance with Inspection Procedure FLO 128-4.800.

The Phase II field inspection consisted of locating each wall identified on the master reference drawings. After location of each wall, an inspection was made with a James Electronics R-Meter, to verify the existence of vertical reinforcing rebar and/or horizontal reinforcing Dur-O-Wal, if called for in the design construction drawings. This inspection was a sampling verification to determine the existence of at least one vertical reinforced rebar column and two horizontal reinforced Dur-O-Wal joints. After further field verification of wall thickness and composition (where verifiable), the appropriate section of the inspection data sheet was checked.

Following this, an inspection of each wall was made to determine if there was any safety-related equipment mounted or in close proximity of the wall. This equipment included, but was not limited to, safety-related piping and supports, conduit, cables, electrical boxes, pumps, heat exchangers and instrumentation. For the purpose of this inspection, "close proximity" was defined as: i) a distance equal to approximately five (5) feet for reinforced and un-reinforced full height walls or, ii) a distance equal to the height plus one (1) foot for cantilevered reinforced walls. This distance was measured as a perpendicular distance from the wall to the safety-related equipment.

Upon the determination that no safety-related equipment was attached, or in close proximity to a wall, the inspection data sheet was marked "not Safety-Related" and signed off, with no further verification required. If any safety-related equipment was identified, a sketch of the wall was made, locating all safety-related and significant non-safety related equipment loads on the wall. "Significant," as used in this inspection, was defined as any equipment which in the very conservative judgement of the engineers performing the inspection contributed a load to the masonry wall greater than twenty-five (25) pounds per square foot wall surface. In addition, all wall penetrations for HVAC, electrical cable trays, ductwork and grill penetrations were shown on the wall sketch. All loads having a center of gravity greater than one foot from the wall surface were noted, as well as any general observations by the inspector concerning the "as-built" condition of the wall.

Since the intent of Bulletin 80-11 is to identify equipment affected should a wall failure occur, a detailed list of safety-related equipment in proximity to a wall was developed for walls for which an engineering evaluation might show failure under certain postulated load conditions.

Upon completion of the inspection data sheets, copies were transmitted to the Ebasco NYO Lead Civil Engineer for review, in accordance with "Verification of Concrete Masonry Wall Design" Procedure FLO 128-4.802. During this review, and at the request of the lead Civil Engineer, an additional inspection was made of twenty-four (24) of the masonry walls. For this inspection, two (2) one-half inch holes were drilled into the cells of the selected block walls to verify

the existence of the grout or mortar fill.

In addition to the field inspections performed at the site to verify the "as-built"condition of concrete masonry walls, several Quality Assurance audits were performed in accordance with the "Quality Assurance Procedure for Compliance with NRC Bulletin 80-11" FLO 128-4.801. These audits of the Phase I and Phase II portions of the inspection were performed jointly by FPL and Ebasco and included verification of inspector training, data collection and documentation of findings.

V. Design Verification

Masonry walls identified by the field inspection program as safety-related, i.e. having safety-related equipment mounted on the wall or located in the vicinity of the wall such that it could be damaged by possible wall failure, required a design re-evaluation to demonstrate their capacity to withstand postulated design loads. A design verification program for this purpose was conducted in accordance with Procedure 128-4.802, "Procedure for Verification of Concrete Masonry Wall Design."

The masonry walls are not shear resistant elements in the building structure system, nor load bearing walls. They primarily function as shielding and partition walls except in one case as a pressure boundary. Therefore, the primary concern of the masonry wall re-evaluation was focused on the behavior of the masonry walls in the event of the safe shutdown earthquake.

A. Loads and Load Combinations

The loads that are imposed on the masonry walls are:

1) Dead Load (D) - This includes the weight of the wall and structures or equipment supported by the wall. The attachment loads are conduits, small pipes, junction boxes, switches and transformers.

2) Seismic Loads

a) Feqo - This is the load generated by the operating basis earthquake (OBE) specified for the site of the plant and developed for the wall by the seismic analyses of the building. The seismic accelerations are applied to the mass of the wall and all attached equipment. In-plane and out-of-plane loadings and the effects of interstory drift of walls are considered.

b) Feqs - This is the load generated by the safe shutdown earthquake (SSE) specified for the site of the plant, and developed as described above for OBE.

3) Pressure Load (Pa) - This is the pressure equivalent static load within the masonry wall compartment caused by failure of equipment. The load includes an appropriate dynamic load factor determined by analysis.

Since all walls are located indoors, there are no wind or tornado loads. The walls are not subjected to pipe rupture reaction loads.

There are three possible load combinations when combining the above four (4) different individual loads:

			Allowable Stresses
1)	Severe Environmental Condition	= D + OBE	S (Table 1)
2)	Extreme Environmental Condition	= D + SSE	U (Table 2)
3)	Abnormal Extreme Environmental Condition	= D + SSE + Pa	U (Table 2)

Since the allowable stresses used for combination (2) are in general only 1.67 times those used for combination (1), while the SSE loading is twice the OBE loading, load combination (1) is not governing.

The postulated load combinations are consistent with the FSAR commitments. Since the FSAR does not specify allowable stresses to be used for design of masonry walls, the allowable stresses listed in Tables 1 and 2 are based on ACE 531-79, "Building Code Requirements for Concrete Masonry Structures."

Table 1: Allowable Stresses in Unreinforced Masonry

Description	S		U	
	Allowable (psi)	Maximum (psi)	Allowable (psi)	Maximum (psi)
Compressive				
Axial (1)	$0.22f'_m$	1000	$0.44f'_m$	2000
Flexural	$0.33f'_m$	1200	$0.85f'_m$	3000
Bearing				
On Full area	$0.25f'_m$	900	$0.62f'_m$	2250
On one-third area or less	$0.375f'_m$	1200	$0.95f'_m$	3000
Shear				
Flexural members (2, 3)	$1.1\sqrt{f'_m}$	50	$1.7\sqrt{f'_m}$	75
Shear Walls (2)	$0.9\sqrt{f'_m}$	34	$1.35\sqrt{f'_m}$	51
Tension				
Normal to bed joints				
Hollow units	$0.5\sqrt{m_o}$	25	$0.83\sqrt{m_o}$	62
Solid or grouted	$1.0\sqrt{m_o}$	40	$1.67\sqrt{m_o}$	67
Parallel to bed joints (4)				
Hollow units	$1.0\sqrt{m_o}$	50	$1.67\sqrt{m_o}$	84
Solid or grouted	$1.5\sqrt{m_o}$	80	$2.5\sqrt{m_o}$	134
Grout Core	$2.5\sqrt{f'_c}$		$4.2\sqrt{f'_c}$	
Collar joints				
Shear		8		12
Tension		8		12

Notes to Table 1:

(1) These values should be multiplied by $(1 - (\frac{h}{40t})^3)$ if the wall has a significant vertical load

(2) Use net bedded area with these stresses.

(3) For stacked bond construction use two-thirds of the values specified.

(4) For stacked bond construction use two-thirds of the values specified for tension normal to the bed joints in the head joints of stacked bond construction.

(5) Note: For St Lucie Unit #1 m_o = 1800 psi

f'_m = 900 psi

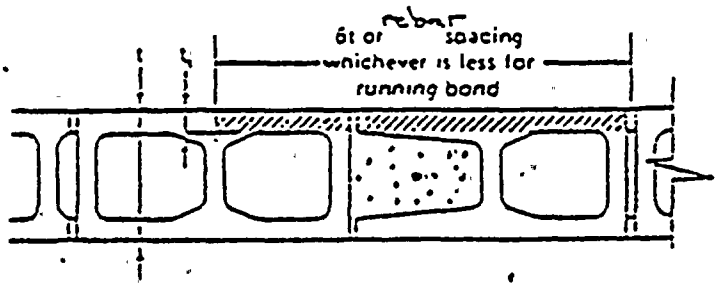
Table 2: Allowable Stresses in Reinforced Masonry

Description	S		II	
	Allowable (psi)	Maximum (psi)	Allowable (psi)	Maximum (psi)
Compressive				
Axial[1]	$0.22f'_m$	1000	$0.44f'_m$	2000
Flexural	$0.33f'_m$	1200	$0.85f'_m$	2400
Bearing				
On full area	$0.25f'_m$	900	$0.62f'_m$	1800
On one-third area or less	$0.375f'_m$	1200	$0.95f'_m$	2400
Shear				
Flexural members[2]	$1.1\sqrt{f'_m}$	50	$1.7\sqrt{f'_m}$	75
Shear Walls[3, 4]				
Masonry Takes Shear				
$M/Vd \geq 1$	$0.9\sqrt{f'_m}$	34	$1.5\sqrt{f'_m}$	56
$M/Vd = 0$	$2.0\sqrt{f'_m}$	74	$3.4\sqrt{f'_m}$	123
Reinforcement Takes Shear				
$M/Vd \geq 1$	$1.5\sqrt{f'_m}$	75	$2.5\sqrt{f'_m}$	125
$M/Vd = 0$	$2.0\sqrt{f'_m}$	120	$3.4\sqrt{f'_m}$	180
Reinforcement				
Bond				
Plain Bars		60		80
Deformed Bars		140		186
Tension				
Grade 40		20,000		$0.9F_y$
Grade 60		24,000		$0.9F_y$
Joint Wire		$.5F_y$ or 30,000		$0.9F_y$
Compression		$0.4F_y$		$0.9F_y$

Notes to Table 2:

(1) These values should be multiplied by $(1- (\frac{h}{40t})^3)$ if the wall has a significant vertical load.

(2) This stress should be evaluated using the effective area shown in figure below except as noted in (6).



Area assumed effective in flexural compression, force normal to face

(3) Net bedded area shall be used with these stresses.

(4) For M/Vd values between 0 and 1 interpolate between the values given for 0 and 1.

(5) Note: For St Lucie Unit #1 m_o = 1800 psi
f'_m = 900 psi

(6) If Dur-O-Wal reinforcement is provided for stack bond walls the effective width of the reinforced units can be increased to the same amount as that used for running bond walls.

B. Analytical Model

All masonry walls were transformed into equivalent homogeneous plate elements spanning vertically to resist out-of-plane bending loads. For reinforced masonry walls, the top support was assumed to be simply supported, since the walls are restrained by two clip angles on both sides. The bottom support was assumed to be fixed because dowels inside the walls can transfer bending moments to the starter walls. The effective width of each reinforced unit is a little less than the spacing of the reinforcing units for the stack bond walls according to ACI-531-79. However, DUR-O-WAL reinforcement was provided for every course, and cement mortar is filled in the cell cores of all blocks so that the entire width of the wall was considered effective for the model.

For unreinforced walls, the simply supported condition was assumed for both top and bottom supports. Rigid arching was also assumed when arching analysis was performed, since no gap was detected at the top support during inspection. (See figure 1 and 2)

Finite element models were used to represent the masonry walls. All large openings were included in the model. The weights of attachments were input as mass for frequency analysis. All of the attachments except transformers are rigidly connected to the walls with the center of mass less than a foot from the wall surface. Also the maximum weight of the attachment is less than 1% of the total weight of the masonry wall itself. Therefore, dynamic amplification of the attachments was not considered, except in the case of the transformers, where an independent dynamic analysis was performed. The ANSYS computer program was used for all the analyses.

Tha analysis of multi-wythe walls does not assume composite action between the wythes.

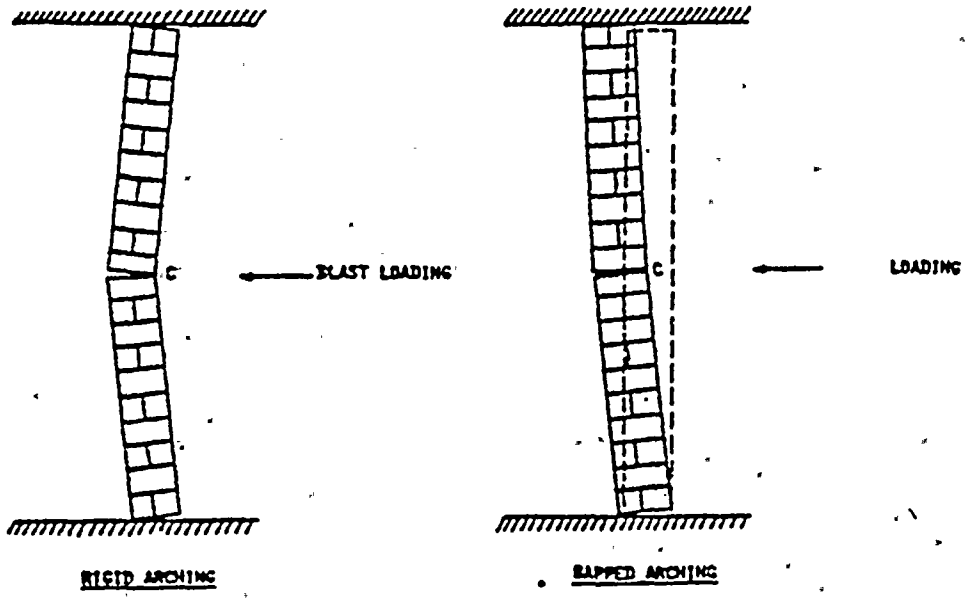


Fig. 1. Sketch Illustrating the Differences in Motion Between Rigid and Gapped Arching.

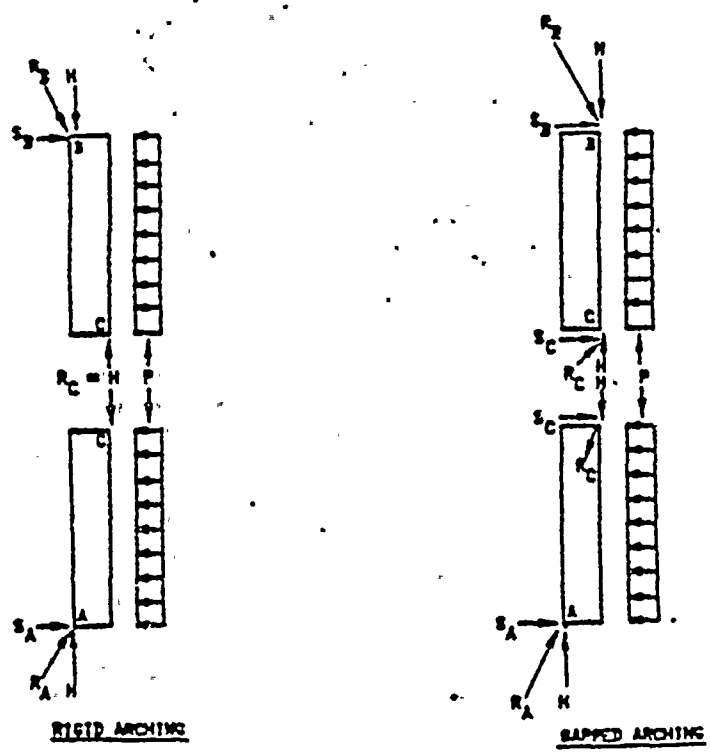


Fig. 2. Free Body Diagrams Showing Forces in Rigid and Gapped Arching.

C. Dynamic and Static Analyses

The frequency analysis of the masonry wall is the same as for ordinary plate elements except that both uncracked and cracked sections are considered for the reinforced masonry walls. The critical damping values used in generation of floor response spectra for SSE were 7% for reinforced cracked walls and 2% for uncracked walls (both reinforced and unreinforced). Seismic acceleration values were selected from the floor response spectrum at the bottom of the wall or the floor response spectrum at the higher elevation, whichever yielded the maximum response at the predetermined frequency. A 25% variation of the frequency range was also considered due to variations of masonry material and other factors.

All the loadings including dead weight of the masonry wall, attachment loads and seismic loads (horizontal and vertical) were input into the computer. The output stresses were compared with the allowable stresses listed in Tables 1 and 2.

D. Special Analyses

Since the re-evaluation of the masonry walls is based on the assurance of "no collapse" of the walls for the most critical load combination, if the flexural stresses exceeded the design allowables at one section of the wall, the wall is still not necessarily considered to fail. Two special analysis techniques were used to evaluate this type of situation. The "Yield-Line Theory" or "Plastic Design" was used for the reinforced masonry walls; while the "Arching Analysis" was utilized for the unreinforced masonry walls.

1) Yield Line Theory - As stated above, in the analytical model the reinforced masonry wall was assumed fixed at the bottom and simply supported at the top. This is considered an indeterminate structure. Therefore, a line of hinges was inserted in the model where the flexural stresses at that location exceeded the allowable. The bending moment was then redistributed by re-analysis of the wall. If the resulting bending stresses

D. Special Analyses (Cont'd)

were found to be acceptable at all other locations and the displacements were not excessive; the wall was considered to be qualified.

2) Arching Analysis - The behavior of a cracked unreinforced wall may be considered as that of a 3-hinged arch with hinges formed at midspan, top and bottom supports. If a gap exists at the top of the wall, gapped arching action should be assumed; otherwise, rigid arching action is assumed as illustrated in Figure 1. The reactions for the 3-hinged arch can be solved by means of the free-body diagrams shown in Figure 2.

E. Interstory Drift

Although the St Lucie Unit 1 concrete masonry walls are not intended to carry a significant part of the building story shear, in-plane shear may be imposed on them by the relative displacement between floors during seismic events. The strain acceptance criteria was used for evaluation of in-plane interstory drift. The relative displacement between floors includes two types of displacements. One is due to bending deformation of the structural shear walls; the other is due to shear deformation. The bending deformation of the structural shear walls will only cause the masonry walls to elongate or shorten on the sides. It is the shearing deformation of the structural shear walls between floors which will cause the masonry walls to have in-plane shearing strain effects. It is these strains which are evaluated as described below:

The gross shear strain is defined to be:

$$= \frac{\Delta}{H}$$

Where γ = strain

Δ = relative displacement between top and bottom of wall

H = height of wall

The permissible in-plane shearing strains are:

E. Interstory Drift (Cont'd)

γ_u = 0.0001 for unconfined walls *

γ_c = 0.001 for confined walls *

The above values were used for normal and severe environmental load combinations. For other load combinations, the allowable strains were multiplied by 1.67.

* An unconfined wall is attached on one vertical boundary and its base. A confined wall is attached in one of the following ways: (a) on all four sides; (b) on the top and bottom of the wall; (c) on the top, bottom and one vertical side of the wall; (d) on the bottom and two vertical sides of the wall.

The out-of-plane interstory drift of the wall due to differential displacements between the two floors will not be significant due to the following reasons:

1) If the wall is simply supported at the top and bottom, the same bending moment capacity will remain after considering the out-of-plane drift effects.
2) If the wall is fixed at the bottom and simply supported at the top, the out-of-plane drift effects will cause at most the fixed end support to yield. The maximum bending moment capacity of the wall will remain the same.

Therefore, the out-of-plane drift effects were not input into the computer analysis.

Several Quality Assurance audits of the design verification work were performed in accordance with Procedure FLO 128-4.801, "Quality Assurance Procedure for Compliance with NRC Bulletin 80-11." The audits were performed by Ebasco and FPL Quality Assurance personnel.

VI. Results of Inspection and Design Verification

A - Inspection

The initial Phase I and Phase II inspection program (described in Section IV of this report) was completed in early October 1980. The inspection included 204 masonry walls, of which 90 were identified as safety-related.

In the course of this inspection, 14 safety-related walls were found to be missing the top support angles called for in the original design. It was decided to immediately install the angles to bring the walls to the original design configuration without first performing an analysis to determine whether in fact wall failure would occur without the top support. Subsequently it was determined that the structural integrity of the walls during a seismic event could not be demonstrated without assuming the presence of the angles. One wall side (inaccessible during plant operation), will be repaired during the 1981 refueling outage. This wall cannot fail in such a way as to affect safety related equipment. A supplementary inspection was conducted for those safety-related walls where the complexity of the attachment configurations indicated the need for a more precise definition of loading application on the wall in order to obtain a more representative analytical model. This portion of the inspection program was completed in late October 1980.

Many masonry walls shown on design drawings as unreinforced nevertheless had cells filled with mortar or grout for radiation shielding purposes. During the course of the design re-evaluation, it became necessary to determine whether some unreinforced walls did in fact have filled cells. An additional inspection of 24 walls was conducted in mid-December 1980 to verify the presence of cell mortar, as described in Section IV of this report. 13 of the walls were found to contain mortar in their cells and this fact was incorporated into the analysis. All the unreinforced walls were ultimately qualified by analysis except wall 114, which required modification as described below.

B - Design Verification

The analytical effort to verify the design adequacy of the 90 safety-related masonry walls began in October 1980 with the issuance of the re-evaluation criteria. By this time, most of the field inspection data had been received. Evaluation of the masonry wall designs was completed in December 1980; and the evaluation of the top supports and anchorages was completed in January 1981.

At the end of December 1980, 6 safety-related walls remained which appeared to not satisfy the established re-evaluation criteria. A final inspection of these walls was conducted on January 6 and 7, 1981 to explore feasible methods of strengthening the walls. It was established that apparent cracks in two of the walls were facial only and not stress related. Repair was accomplished by enlarging the cracks and filling in with mortar. The presence of reinforcing in a third wall was confirmed and a new analysis resulted in qualification of that wall.

The remaining 3 walls were deemed to require modification as described below:

1. Wall 114 - The analysis of this unreinforced wall resulted in unacceptably large compressive and shear stresses. The height of the wall was reduced to 60 percent of its originally assumed value by the introduction of a supporting structural member acting in concert with a slab framing into the wall at the same elevation. A new analysis resulted in qualification of the wall as modified.

2. Wall 159 - An excessive number of large opening in this reinforced wall prevent the vertical reinforcing bars from running all the way through. The analytical model was revised with the addition of supporting angles to the vertical edges of the lower half of the wall, fastened to the adjacent reinforced concrete column or wall. The additional supports resulted in the qualification of the wall.

3. Wall 203 - This wall had the same problems Wall 159. In addition, part of the top supporting angle was missing due to blocked access from ductwork. A fix was developed similar to that provided for Wall 159, to be applied along one vertical edge of the wall. A supporting channel pair was added along the top edge where required, to be bolted into the ceiling where access permitted. A stiffening channel pair extending from the top channels down to a more substantial area of the wall provides support for the remainder of the top edge. A new analysis resulted in qualification of the wall as modified.

All wall attachments were analyzed locally for block pullout, as well as being integrated into the overall analytical model for the wall. No stress problems arose in this area. One bracket-type support for a transformer required an independent analysis to determine the dynamic amplification imposed on the wall.

As the stress analyses for the walls were completed, the reactions at the tops of the walls were used to evaluate the adequacy of the supporting angles and anchorages where these were provided. A review of the design details indicated that, for 20 walls, the supporting angle attachment to the building structure had to be modified to accommodate the calculated reactions.

The following table explains the modifications to the top supports:

Wall Number	Description of Modification
8A	Addition of clip angles expansion anchored to ceiling
11A	Addition of clip angle expansion anchored to ceiling and plate welded to ceiling embedment
34, 74, 125 165, 200, 201 202, 205	Addition of expansion anchors for clip angle support

Wall Number	Description of Modification
62A	Addition of clip angles, filler plate welded to ceiling embedded plate
80, 123, 124	Thru-bolting clip angles on either side of wall
81, 82, 163, 174	Thru-bolting clip angles on either side of wall Addition of expansion anchors for clip angle support
160	Addition of expansion anchors for clip angle support Addition of mortar to enable block wall to bear on adjacent concrete beam
166	Addition of fillet weld between clip angle and filler plate

VII. Summary

The inspection and design verification of masonry walls for St Lucie Unit 1 conducted between September 1980 and January 1981 established the following:

Number of walls inspected - 204

Number of walls classified as safety-related - 90

Number of walls classified as not safety-related - 114

Number of safety-related walls where missing clip angles were replaced - 14*

Number of safety-related walls re-evaluated - 90

Number of walls requiring field modification - 3

Number of walls requiring reinforcement of top edge support - 20

Documentation, to include inspection procedures and design verification details will be available at the St. Lucie site for inspection and review and has not been attached to this report.

*Further clip angle work is required for 1 wall to bring it to original design.

STATE OF FLORIDA)
) ss.
COUNTY OF DADE)

Robert E. Uhrig, being first duly sworn, deposes and says:

That he is a Vice President of Florida Power & Light Company, the Licensee herein;

That he has executed the foregoing document; that the statements made in this said document are true and correct to the best of his knowledge, information, and belief, and that he is authorized to execute the document on behalf of said Licensee.

Robert E. Uhrig

Robert E. Uhrig

Subscribed and sworn to before me this

11 day of February, 1981

Cheryl F. Fredrick

NOTARY PUBLIC in and for the county of Dade, State of Florida

My commission expires: Notary Public, State of Florida at Large My Commission Expires October 30, 1983 Bonded thru Maynard Bonding Agency