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VARGA,S.A. Operating Reactors Branch 1

SUBJECT: Forwards addl info re masonry wall design,in response to NRC Dec 1983 request per IE Bulletin 80-11 covering arching methodology & applicability to facilities.

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FPL

FLORIDA POWER & LIGHT COMPANY

February 1, 1984
L-84-24

Office of Nuclear Reactor Regulation
Attention: Mr. Steven A. Varga, Chief
Operating Reactor Branch #1
Division of Licensing
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Dear Mr. Varga:

Re: Turkey Point Units 3 & 4
Docket Nos. 50-250 & 50-251
IE Bulletin 80-11
Masonry Wall Design
Request for Additional Information

In December 1983, via telecopy, the NRC requested that FPL provide additional information regarding masonry wall design. The questions were centered around the use of the arching methodology and its application at Turkey Point. Attached please find the response to these concerns.

If you have additional questions, please contact us.

Very truly yours,

J. W. Williams
Vice President
Nuclear Energy

JWW/GJK/mp
Attachment

cc: J.P. O'Reilly, Region II
Harold F. Reis, Esquire

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RESPONSE

TO

NRC REQUEST FOR

ADDITIONAL INFORMATION

ON DECEMBER 1, 1983

MASONRY WALL RE-EVALUATION

(NRC IE BULLETIN 80-11)

FLORIDA POWER & LIGHT COMPANY

TURKEY POINT PLANT UNITS NO. 3 & 4

BECHTEL POWER CORPORATION

GAITHERSBURG, MARYLAND

JANUARY, 1984

PROJECT SUMMARY

In August 1983, FPL hosted a site visit for the NRC staff (NRR) to observe and examine the Turkey Point masonry walls in light of the construction deficiencies noted in reportable occurrence No. 250-83-06. Following the site visit, NRR conducted a review of the calculations and analyses used for masonry wall qualification in Bechtel's Gaithersburg office.

During the meetings with the NRC representatives additional information came to light which has resulted in some change to the strategy for resolution of masonry wall deficiencies. Specifically, FPL was informed of the following NRC positions: 1) use of the arching methodology for the response of masonry walls to a seismic event is unacceptable without testing back-up (a branch position is forthcoming), and 2) the use of tension in masonry is acceptable for elastic analyses of the walls.

As a result of this information, FPL decided to justify all walls by elastic analyses as-constructed or to repair them to meet elastic stress requirements. Although all calculations have not been completed, this new approach appears to be feasible in all cases except for the Steam Generator Feed Pump (SGFP) enclosures.

The proposed approach for analyzing the SGFP enclosure walls is to meet elastic criteria for all load combinations except for a postulated break in the feedwater pump discharge line. Since this load is not cyclic, one-way arching action will be used for this situation.

In October, 1983 FPL informed the NRC of this new approach. Additional questions posed by the NRC in light of the new approach are discussed in the following pages. These questions were transmitted to FPL via telecopy.

APPLICABILITY OF ARCHING THEORY TO UNREINFORCED MASONRY WALLS UNDER PRESSURE LOADS

The following is provided in response to questions forwarded to Florida Power & Light by the NRC on December 1, 1983. Note that the pressure load being resisted by arching action in the Steam Generator Feed Pump (SGFP) enclosures is due to a postulated break in the feedwater pump discharge line, and does not result from a postulated tornado. This is the basis of the responses provided below.

Question 1

Provide the technical basis and justification for treating tornado differential pressure loads as an equivalent static load.

Response 1

The peak pipe break pressure is 1.14 psig, with a rise time of 1.33 seconds. The minimum natural frequency of the walls in question is 30.47 cps. This frequency is conservatively based on a hollow vertical wall strip that is assumed to be simply supported at top and bottom. According to reference (1), the dynamic load factor (DLF) for a constant force with finite rise time (such as the subject pipe break) is determined from the following expression:

$$DLF = 1 + \frac{1}{wtr} \left\{ \sin w(t - tr) - \sin wt \right\}$$

Where tr = rise time = 1.33 sec.
w = natural frequency = 30.47 (2) (3.14) = 191.35 rad/sec.
t = time

The maximum possible value of the term $\{\sin w(t - tr) - \sin wt\}$ is 2. Substituting this value in the above expression yields a maximum possible DLF of

$1 + 2/(191.35)\ (1.33) = 1.0079$

This maximum possible value is approximately 0.8 percent greater than the value corresponding to the static condition, and therefore justifies the assumption of an equivalent static pressure load.

Question 2

Identify test data which could be used to verify arching action and, based on these tests, discuss the following items:

a) Applicability of blast loading (type, magnitude, rate and duration) to tornado differential pressure. Discuss the technical basis and justification for the assumption used.

b) Applicability of test data to actual walls in the plant with regard to: boundary conditions, materials, geometry, openings and attachments.

c) The statistical significance of available test data (number of relevant tests, level of confidence) in view of the fact that masonry is a composite material with high variability.

d) The correlation between the test data and the proposed analytical procedure with regard to ultimate load and maximum mid-height deflection.

e) Discuss the safety factors, if any, used in the analytical procedure and how they are reflected from test data. Based on URS test data, determine the minimum factor of safety against collapse for walls in question.

f) Discuss the effect of joint cracking (particularly at top of walls), if any, on the behavior and resistance of the walls under tornado loads.

g) Calculation of the maximum deflection of a typical wall under tornado loads utilizing the proposed technique and how it is compared to URS test data. Discuss the impact of maximum displacement on the functionality of the wall attachments.

Response 2

Test data which can be used to verify arching action in the walls enclosing the SGFP enclosure is contained in reference (2). Specifically, the results of Tests No. 77 and 78 apply to the walls in question.

a) The blast load used in the referenced test is similar to the pipe break design load in that both are relatively uniform pressure loads. The maximum reflected pressure from a surviving test wall is 7 psi compared with a peak pipe break pressure of 1.14 psi. The rise time of the experimental load was 10 milliseconds or less, compared with a 1.33 second rise time for the pipe break. The blast load had a duration of approximately 100 milliseconds, which is several orders of magnitude less than the duration of the design pressure.

   The extremely rapid rise time of the experimental load would imply a much more severe dynamic loading condition than the design pipe break load.

b) Test walls 77 and 78 are very similar to the actual SGFP enclosure walls. Both experimental and actual walls are constructed of 8 inch hollow block in running bond. The boundary conditions for both sets of walls allow rigid arching to take place. The SGFP walls are all 8'-0" high while the test walls are 8'-6". The SGFP enclosure walls have no window or door openings; neither do test walls 77 and 78. Some of the SGFP enclosure walls do have pipe penetrations; however, these penetrations represent less than 3 percent of the total wall surface. There are few items attached to the walls. All but one of these attachments weigh less than 100 lbs. The maximum attachment to the walls is a 12 foot run of cable tray weighing approximately 40 lbs/ft. Neither of the test walls had attachments.

c) The fact that only two test results are available for comparison implies negligible statistical significance. However, the level of pressure at failure of the test walls when compared to the design pressure provides a high level of confidence in the SGFP enclosure walls.

d) The test data and the values calculated by the arching method for the SGFP enclosure walls are compared in Table 1.

TABLE 1

	Test	Calculated (Max. Allowable)	Calculated (Postulated)
Ultimate Load	7-9 psi	3.33 psi	1.14 psi
Max. Deflection at Mid-height	0.4 in	0.120 in	0.043 in

e) The primary safety factor employed in the analysis is a reduction in the calculated maximum allowable arching force by a factor of (1/1.5). The factor of safety against ultimate for the SGFP enclosure walls based on the minimum URS test pressure is

$$\frac{7 \text{ psi}}{1.14 \text{ psi}} = 6.14$$

f) Test wall 77 was cracked in a number of locations after a blast loading of 7 psi. Even in this cracked condition, it still withstood a second blast load of 4 psi which "caused little if any additional damage" (reference 2). Based on these test results and the relatively low design pressure of 1.14 psi, the effect of joint cracking (if any) will be minor.

g) The calculated maximum allowable, calculated postulated, and the maximum tested wall deflections are listed in Table 1 under Item (d) above. Note that the factor of safety based on calculated postulated deflection versus tested maximum deflection is

$$\frac{0.4 \text{ inches}}{0.043 \text{ inches}} = 9.30$$

The calculated maximum postulated deflection, which is less than 1/16th inch, has no impact on the functionality of attachments to the walls.

References

1) Biggs, J. M., Introduction to Structural Dynamics, McGraw Hill, 1964.

2) URS Research Co., Shock Tunnel Tests of Preloaded and Arched Wall Panels, June 1973.