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FLORIDA POWER & LIGHT COMPANY

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U.S. NUCLEAR REGULATORY COMMISSION

Mr. James P. O'Reilly, Director, Region II
Office of Inspection and Enforcement
U.S. Nuclear Regulatory Commission
101 Marietta Street, Suite 3100
Atlanta, Georgia 30303

Dear Mr. O'Reilly:

Re: Turkey Point Units 3 & 4
Docket Nos. 50-250 and 50-251
I&E Bulletin 80-11

Please find attached our final report in response to I&E Bulletin 80-11.

Very truly yours,

Robert E. Uhrig

Robert E. Uhrig
Vice President
Advanced Systems & Technology

REU/JEM/mbd

Attachment

cc: Mr. Harold F. Reis, Esquire

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FLORIDA POWER & LIGHT COMPANY

TURKEY POINT PLANT UNITS NO. 3 & 4

MASONRY WALL RE-EVALUATION
(NRC IE BULLETIN 80-11)

BECHTEL POWER CORPORATION

GAITHERSBURG, MARYLAND

MARCH, 1981

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FLORIDA POWER & LIGHT COMPANY
TURKEY POINT PLANT UNITS NO. 3 & 4

MASONRY WALL RE-EVALUATION
(NRC IE BULLETIN 80-11)

I. INTRODUCTION & CONCLUSION

This report is submitted to complete FPL's response to NRC IE Bulletin 80-11. It specifically addresses items "2b" and "3". Responses to items "1" and "2a" of the bulletin have been provided to the NRC via letter L-80-234, dated July 24, 1980.

The July 24, 1980 submittal indicated that 99 walls were to be re-evaluated. However, further review indicated that 97 walls are in proximity to or have attachments from safety-related piping or equipment. Also that submittal indicated the inaccessibility of the Unit 3 containment which precluded a walkdown. However, during a recent outage, a walkdown was completed by a survey team, and no concrete masonry walls were found in the Unit 3 containment.

All 97 walls have been re-evaluated to meet their intended function, under the appropriate loads and load combinations associated with these walls, based upon the methods and criteria provided in Section II of this submittal. Two of these walls were determined to be inadequate to withstand jet impingement loads only. Nuclear safety related function(s) associated with the remaining 95 walls are not adversely impacted. The two walls that could potentially fail have associated with them one reactor trip channel. Failure of the circuit will trip the channel, i.e., the circuit fails safe. Thus, the potential loss of either of the two walls is considered acceptable because the potential impact on the facility does not affect the ability to achieve and maintain a safe shutdown condition.

II. RESPONSES TO I.E. BULLETIN 80-11 ITEMS

A. Item 2b

Masonry walls at Turkey Point Plant Units 3 and 4 typically serve as partition walls, fire barriers, water barriers, flood barriers, shield walls, load bearing walls, or sound barriers. Documents indicating location and layout of masonry walls are available at the Turkey Point site for NRC review.

The types and strengths of materials for construction typically were as follows:

(1) Concrete block conforming to ASTM C-90-66-T Grade A, with linear shrinkage limited to 0.05 percent.

(2) Mortar conforming to ASTM C-270-64-T, using type "S" for unreinforced masonry in contact with earth, type "N" elsewhere, and type "M" for reinforced masonry.

(3) Grout for filling concrete block cells conforming to type "M" with maximum size of aggregate of 3/8 inch.

(4) Reinforcement: Bars conforming to ASTM A-15-66, intermediate grade, deformed.
Joint Reinforcement: Approved standard product (Dur-O-Wall) conforming to ASTM-A82-66.

Five multi-wythe masonry walls were identified. A total of 87 masonry walls were reinforced and a total of 10 masonry walls were unreinforced.

The typical masonry wall reinforcement detail shows #4 or #5 vertical reinforcing bars spaced at 16 inches on center and horizontal masonry reinforcement of Dur-O-Wall (or equal) spaced at either 16 inches or

32 inches on center. No masonry ties between the wythes are shown on design drawings for multi-wythe walls.

Typical construction practices associated with these walls follow:

(1) Concrete blocks were stacked under cover or protected from exposure to the weather or from contact with soil. Use of damaged blocks was not permitted.

(2) Mortar was prepared in batches of the volume that was to be used before initial set took place, and was placed within one hour after mixing.

(3) Grout was mixed in a clean mechanical mixer with only sufficient water added to produce a plastic mix which would flow readily into place without segregation.

(4) Vertical reinforcing steel typically consisted of rebar dowels in a base slab or footing lap spliced to vertical rebar in a grout filled block cell. The masonry blocks were placed in a staggered pattern over the dowels, vertical rebar lap spliced to the dowels and horizontal reinforcing placed at specified spacing. All cells including those containing vertical bars were grout filled with the reinforcing bars adequately anchored in place until the grout had set sufficiently to support the bars.

(5) Unreinforced masonry blocks were placed in a staggered pattern.

(6) Grout was poured in lifts not exceeding 16 inches. Each pour was rodded to insure compaction and bond to the proceeding pour and to prevent the development of voids.

The construction practices employed adequately prevent any significant voids or other weaknesses in the materials of construction.

All 97 masonry walls were re-evaluated for their intended function based upon appropriate Turkey Point Units 3 & 4 FSAR loads and load combinations using conventional analytical methods prescribed by the Working Stress Design method with a load factor of unity. The loads considered included those produced from safety and non-safety-related attachments, interstory drift (differential floor displacement), thermal effects, and the effects of potential cracking under dynamic loads. The FSAR does not specify the use of any particular code for the design of the masonry walls. For normal design loading conditions the re-evaluation was based on the American Concrete Institute "Building Code Requirements for Concrete Masonry Structures" (ACI 531-79). For loading conditions not directly covered by this code, supplemental allowable stresses and alternative design techniques were used as discussed hereinafter.

Calculated wall stresses were first compared against an allowable stress criteria. If allowable stresses were exceeded, then wall stability was checked using inelastic design approaches.

Design allowable stresses were used for loads normally encountered during plant operation or shutdown (such as normal operating thermal effects and pipe reactions) and infrequently encountered loads (such as operating basis earthquake (OBE) and wind loads). A 30% stress increase was permitted for load combinations containing normal operating thermal effects or displacement limited loads. The factor of safety against failure of the masonry for cases where the 30% stress increase is utilized reduces from 3.0 to 2.3, still well within the elastic range.

Allowable stresses utilized in the re-evaluation follow:

<u>Masonry</u>

Allowable working stresses as per Table 10.1 of ACI 531-79.

Cell Grout

Allowable tension stresses equal to 2.5 $\sqrt{f'c}$.

Reinforcing Steel

Stresses in steel reinforcement not to exceed the following limits: (ACI 531-79):

In Tension:

Grade 40 bars .... 20,000 psi

Joint wire reinforcement 50 percent of minimum ASTM specified yield strength, but not to exceed .... 30,000 psi

In Compression:

40 percent of ASTM specified yield strength, but not to exceed .... 24,000 psi

In-plane effects due to interstory drift were either determined by analysis or in-plane strains (Δ/H). They were limited to 0.00012, where Δ is the relative displacement between the top and bottom of the wall and H is the height of the wall. A totally confined wall was limited to a strain of 0.0008 for walls where (a) the structural shear resisting elements bounding each vertical side of the wall had a shear resisting capability larger than the wall and (b) the wall width to height ratio was at least 0.5.

In-plane strain allowables for interstory drift effects for non-shear walls were established below the level of strain required to initiate significant cracking. The allowable strain for a confined wall was based on the equivalent compression strut model discussed in Reference 1 and modified by a safety factor of 3.0 against crushing. Test data (References 1 through 7) associated with cracking strains for confined masonry walls subjected to in-plane displacements confirms the predicted strain as given by the equivalent strut model.

Design allowables were increased for loads which are highly improbable such as the safe shutdown earthquake (SSE). Code allowable stresses for masonry in tension, shear and bond were increased by a factor of 1.67 which provides a factor of safety against failure of 1.8. Masonry compression stresses were increased by a factor of 2.0 for axial stresses and 2.5 for flexural and bearing stresses which provides a safety factor against failure of 1.2. Allowable reinforcing steel stresses were 90% of minimum ASTM specified yield strength provided lap splice lengths and embedment (anchorage) could develop this stress level. Allowable bond stresses in determining splice and anchorage lengths were permitted to be increased by a factor of 1.67. In-plane strains due to interstory drift were limited to 1.67 times the values previously stated.

Damping for uncracked walls was set at 2% for OBE and SSE corresponding to stress levels ranging from approximately 0.3 to 0.6 of ultimate. Damping for reinforced walls which are expected to crack due to out-of-plane seismic inertia loading were set at 4% for OBE and 7% for SSE.

The modulus of rupture of concrete, grout and mortar was assumed to vary by 20%, therefore, a lower bound modulus of rupture was determined by applying a reduction factor of 0.8 to the theoretical concrete modulus of rupture of $7.5 \sqrt{f'c}$. For masonry, the modulus of rupture was approximated by increasing the code allowable flexural tensile stress by the factor of safety of 3 and then applying the 20% reduction to arrive at a lower bound value of 2.4 Ft, where Ft is the code allowable tensile stress.

Where the bending due to out-of-plane loading caused flexural stresses in the wall to exceed the previously stated design allowables, the wall was evaluated by alternate methods including the following:

Masonry walls (a) that were not relied upon to provide strength of the structure as a whole, and (b) that were subjected to out-of-plane seismic inertia loading causing flexural stresses in excess of design allowables were evaluated by means of the "energy balance technique" for reinforced walls.

Reinforced masonry walls evaluated by the "energy balance technique" (References 8 and 9) must have sufficient capability to preclude brittle failure and allow relatively large ductile flexural deformations. Tests (Reference 13) indicate that when flexure is the dominant action, ductilities are in excess of 25. Other tests (Reference 14) show that when compression failures occur, ductilities in excess of 5 can be achieved.

When reinforced masonry has adequate shear and compression capability, its behavior is expected to parallel that of reinforced concrete where allowable ductilities for predominately non-structural elements are normally set at 10. Thus, for out-of-plane seismic loading on non-shear walls constructed of masonry where brittle failures are precluded, a ductility of 5 was considered acceptable if the function of the safety system(s) associated with the wall are not jeopardized by wall deflection.

The deflection of a fully cracked reinforced wall subjected to seismic loading was determined by the "energy balance technique". If the predicted displacement exceeded three times the yield displacement, the resulting displacement was multiplied by a factor of 2. The resulting displacement was utilized to evaluate the potential impact on the function of safety related system(s) attached and/or adjacent to the wall.

In all cases the midspan displacement was limited to five times the yield displacement, and the masonry compression stresses were limited to 0.85f'm.

Arching Action

Masonry walls confined within a rigid frame or structure can develop substantial resistance to out-of-plane loadings after flexural cracking. These walls may be evaluated by use of the arching theory (References 10 through 12).

The resistance of the wall to out-of-plane forces were determined by assuming that a three-hinged arch is formed after flexural cracking. Due consideration was given to the rigidity of the supporting elements and their ability to restrict rotation of the wall about the supports. The effects of a gap at the supports were considered. The maximum allowable uniform load used was the lesser of:

a) One third of the predicted load based on a maximum masonry compression of 0.85f'm

b) Two thirds of the predicted load based on a maximum tension stress of $6\sqrt{f'm}$ along the 45° diagonal failure plane and one inch bearing width at 0.85f'm in the vicinity of the hinge.

The deflection at the interior hinge of the arch after full contact with the support was limited to 0.3 times the thickness of the wall.

A displacement of 2 times the calculated displacement was utilized to evaluate the potential impact on the function of safety related system(s) attached and/or adjacent to the wall.

The structural response of the masonry walls subjected to out-of-plane seismic inertia loads was based on a constant value of gross moment of inertia along the span of the wall for the elastic (uncracked) condition. If the wall was cracked, the moment of inertia was obtained by using the ACI-318 formula for effective moment of inertia used in calculating immediate deflections (Reference 15).

To determine the out-of-plane frequencies of masonry walls, the uncracked behavior and capacities of the walls and, if applicable, the cracked behavior and capacities of the walls were considered.

Uncracked Condition

The equivalent moment of inertia of an uncracked wall (I_t) was obtained from a transformed section consisting of the block, mortar, and cell grout. Alternatively, the cell grout, neglecting block and mortar on the tension side, was used.

Cracked Condition

If the applied moment due to all loads in a load combination exceeded the uncracked moment capacity, the wall was considered to be cracked. In this event, the equivalent moment of inertia was computed as follows:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_t + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

$$M_{cr} = f_r \left(\frac{I_t}{Y}\right)$$

where,

I_e = Equivalent moment of inertia
M_{cr} = Uncracked moment capacity
M_a = Applied maximum moment on the wall
I_t = Moment of inertia of transformed uncracked section
I_{cr} = Moment of inertia of the cracked section
f_r = Modulus of rupture
Y = Distance of neutral plane from tension face

If the use of an equivalent moment of inertia resulted in an applied moment less than the uncracked moment capacity, then the wall was verified for the uncracked capacity.

The effect of modes of vibration higher than the fundamental mode was considered. For this purpose, a modal analysis was performed or alternatively, the inertia load on the wall due to its own weight for the fundamental mode was considered as the uniform load in lieu of determining an effective mass. The corresponding bending moment and reaction account for the higher mode effects.

Uncertainties in structural frequencies of the masonry wall due to variations in structural properties and mass were taken into account. The lower bound frequency was utilized if it was on the higher frequency side of the peak response spectrum. If the lower bound frequency was on the lower frequency side of the peak, the peak acceleration was used. If the response spectra for a wall spanning between two floors were of the same shape, the average spectra was used. If they were different, the enveloping spectra was used corresponding to the wall's natural frequency.

Boundary conditions were determined considering one-way spans with hinged, fixed or free edges as appropriate. Two-way spans were not used. Load transfer at the boundary was evaluated based on boundary anchorage capacities. Fixed end conditions were justified for walls (a) built into thicker walls or continuous across walls and slabs, (b) that have the strength to resist the fixed end moment, and (c) that have sufficient support rigidity to prevent rotation. Otherwise, the wall edge was considered simply supported or free depending on the shear carrying capability of the wall and support.

Distribution of concentrated loads are affected by the bearing area under the load, horizontal and vertical wall stiffness, boundary conditions and proximity of load to wall supports. For predominantly one-way action, an effective beam width of 6 times the wall thickness for distribution of concentrated loads was conservatively used for the following conditions:

Concentrated load at midspan; simple supports:	$L > 9.6T$
Concentrated load on a cantilever:	$h > 2.4T$
Couple at midspan; simple supports:	$a > 4.8T$
Couple near a support, simple supports:	$a > 2.4T$

where: L = beam length

h = distance from the fixed end to the point of load application

a = distance between the concentrated loads producing a couple

T = thickness of the wall

Interstory drift values were derived from the dynamic analysis. Strain allowables depending on the degree of confinement were applied for in-plane drift effects on non-shear walls. They were set based on in-plane effects in a manner that ensures that a reasonable margin remains for out-of-plane loads. Out-of-plane drift effects were considered if some degree of fixity existed at the top and/or bottom of the wall.

Local loadings resulting from items such as piping and equipment support reactions were included. The evaluation included transfer of the loads into the wall by consideration of punching shear effects to ensure that failure due to local effects (i.e., block pullout) would not occur.

The re-evaluation of multi-wythe masonry walls was based on the absence of composite action between the two wythes.

B. <u>Item 3</u>

The methods and assumptions utilized in the masonry wall re-evaluation were based on conservative acceptance criteria and referenced literature which provide adequate support for the propriety of the evaluation methodology. Therefore, there is no need to conduct a masonry wall test program to support the re-evaluation conducted pursuant to NRC Bulletin 80-11.

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