BOSTON EDISON COMPANY
800 BOYLSTON STREET
BOSTON, MASSACHUSETTS 02199

WILLIAM D. HARRINGTON
SENIOR VICE PRESIDENT
NUCLEAR

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BECo 85-036

Mr. Domenic B. Vassallo, Chief
Operating Reactors Branch #2
Division of Licensing
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555

License DPR-35
Docket 50-293

Dear Sir:

By letter of January 10, 1985, the NRC requested clarification of several items associated with masonry wall design at Pilgrim Station. This request concerned information which had been presented to the NRC during a meeting between Boston Edison and the NRC conducted on July 19, 1984.

The attachment to this letter is provided to address that request. Should you wish further information concerning this submittal, please contact us.

Very truly yours,

WD Harrington

Attachment

PMK/kmc

A001
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A. Statistical Analysis of Boundary Strengths

1. With respect to the sampling technique used in the test verification program, please provide a technical assessment of the use of unequal exposed lengths for anchor verification in different walls. Also provide a technical assessment of the fact that the exposed length was not related to the length of the wall (i.e., total of 48 in of exposure was applied not only for a short wall, say an 8-ft. wall, but also for a long wall, say a 20-ft. wall or longer).

Response

Unequal Exposed Lengths

The statistical analysis of the test data has been based on the conversion of each sample point to a single number - "anchorage per unit length" - calculated as the number of anchors observed divided by the exposed length. Each data point is given equal weight-regardless of the length exposed. The sample statistics reported in Appendix A of the CES report 560-02. "Statistical Analysis of Boundary Strengths for Masonry Walls from Field Test Data," September 1983, were based on this method.

An alternative approach would be to consider each sample point as two measurements - length exposed and number of anchors found. The statistics of "anchorage per unit length" can then be calculated as derived quantities from a function of the two random variables (length, number of anchorages). This formulation explicitly includes the variation in exposed length from test to test.

For the three populations for which some anchors can be relied upon (side-steel, side-concrete, top-concrete) the sample statistics (mean and standard deviation) were recalculated on the above basis. The following are the results (per unit length of wall).

POPULATION / STATISTIC	SIDE STEEL	SIDE CONCRETE	TOP CONCRETE
No Length Variation (Previous Results)			
o Mean	0.0449	0.0354	0.0441
o Std. Dev.	0.0145	0.0183	0.0180
Length Variation			
o Mean	0.0451	0.0353	0.0439
o Std. Dev.	0.0116	0.0170	0.0176

It is observed that in all cases, the previous sample-means are accurate, and the previous sample standard deviations are slightly conservative (larger than those including length variation). It is therefore concluded that our previous analyses (not explicitly including the length variation from test to test) were both accurate and slightly conservative, and that the boundary strengths calculated previously satisfactorily account for any variations due to the use of unequal exposed lengths for different walls.

Exposed Length vs. Length of Wall

As noted in this question, the exposed length requirements in the test program were not related to the length of the wall. However, the data was treated on a "per unit length" basis, and thus the length of wall was not a direct consideration.

Tests were performed on walls of greatly varying length from very short (188.9-4'9", 184.8-5'3") to very long (several walls greater than 30' in length). Thus the sample is made up from data points corresponding to walls from a wide range of lengths, and results are equally applicable to long or short walls. Note that a length restriction applies, so that no anchors can be relied upon in very short walls unless they were explicitly located during the test program.

2. In a few cases, no anchors were found with a predetermined exposed length. The licensee should extend this exposed length to locate the anchors. The results will help to reinforce the adequacy of the statistical analysis method.

Response

The response to this question is divided into two parts. Part I provides an elaboration of the statistical meaning of the "zero data points." BECo believes that the presence of such points is both reasonable and predictable within the context of the overall acceptance criteria. Part II examines the individual walls in question and demonstrates why further investigations of embedded anchorage would be either impractical or unproductive.

Part I - Statistical Relevance of Zero Data Points

There are two boundary types which rely on anchors, yet have at least one "zero" data point in the sample. These are side boundary to steel (1 zero data point in 17 tests) and side boundary to concrete (3 zero data points in 20 tests).

In each case the zero data points are from tests with short exposed lengths (33", 30", 36", 32"), which are significantly less than the reliable spacing (48" and 80") for the two boundary types.

Side Boundary to Steel

There is a high correlation coefficient (82%) between exposed length and number of anchors found (i.e., little weight should be given to short lengths with no anchors). There are 6 tests with exposed lengths between 30" and 35", only one of which found no anchors. The reliable spacing for this population is 48".

Using a uniform 48" anchorage spacing, there is a 0.29 chance of finding no anchors in 33" of exposed length. Using this probability and the binomial distribution, the chance of finding at least one test (with a 33" exposed length) with zero anchors in six tests is 87%.

Considering this fact with the high correlation coefficient between exposed length and number of anchors, it is our conclusion that there is high confidence in the 48" spacing and no further exposure is justified.

Side Boundary to Concrete

There is a high correlation coefficient (70%) between exposed length and number of anchors found (i.e., little weight should be given to short lengths with no anchors). There are 12 tests with exposed lengths between 30" and 36" 3 of which found no anchors. The reliable spacing for this population is 80".

Using a uniform 80" anchorage spacing, there is a 0.54 chance of finding no anchors in 36" of exposed length. Using this probability and the binomial distribution, the chance of finding at least three tests (with a 36" exposed length) with zero anchors in twelve tests is 99%.

Considering this fact with the high correlation coefficient between exposed length and number of anchors, it is our conclusion that there is high confidence in the 80" spacing, and no further exposure is justified.

Part II - Specific Wall Conditions at Zero Data Points

It is BECo's position that the presence of zero data points is wholly consistant with the analytical approach adopted and does not warrant further field sampling. However, in the spirit of defense in depth, this part to the response examines the specific conditions which exist at each of the four zero data points which are at issue here.

Wall 194.20 - Side Boundary to Concrete
This is a side boundary to concrete condition. A field examination of the sample location showed an attachment which interrupted the sample. Boston Edison plans to modify wall 194.20 to resist seismic loads. In the analysis of the wall, the side boundary was considered a pin, however, the total load resisted by that side boundary was only 3 lb/in. Compared to our acceptance criteria of 93 lb/in this is about 3%. The predominant behavior is vertical spanning. An alternative calculation has been prepared to show that the wall will achieve qualification without reliance on the side boundary in question, once the modification is completed.

Wall 64.4 - Side Boundary to Concrete

This particular wall was qualified by reliance on modification. Boundary modifications completed during the 1981 - 1982 outage completely cover the area of the sample in question. Since this wall does not rely on any shear transfer through the boundary via internal reinforcing, the significance of the individual sample point is nil.

Wall 63.5 - Side Boundary to Concrete

This sample was interrupted by physical obstructions attached to and adjacent to the wall. The sample location is confined to a locked high-high radiation area presently reading 3R/hr. BECo requests an exclusion from performing any further sampling on this wall due to ALARA considerations. We will rely on the strengths of the technical arguments advanced in Part I to the response of this question. Note that this boundary is to a concrete wall, and that the statistical chance of finding at least 3 zero data points was 99%.

Wall 212.0 - Side Boundary to Steel

The actual sample location was confined by obstructions which prevented a full length sample. This particular wall was fully modified during the 1981 - 1982 outage and the qualification analysis takes no credit for internal boundary anchorage to the steel columns.

B. Orthotropic Plate Analysis

Based on the summary of finite element analysis and sample calculations of wall 63.4, 65.8, 64.4, and 188.10 given in the meeting on July 19, 1984, the following questions are presented:

1. CYGNA's methodology calls for two-way cracked analysis of block walls (level I and level II). There are no acceptable methods available in the literature for the bending analysis of block masonry walls in the post-cracking stage. This is primarily because of the complexity of the problem due to material anisotropy, the existence of planes of weakness which affect crack propagation, discontinuity due to partial grouting, and the uncertainty about the contribution of joint reinforcement in the lateral load resistance. In light of the above comments, justify two-way cracked analysis.

   Response

   This question addresses the applicability of two-way cracked analysis to the Pilgrim block walls. The Pilgrim walls are reinforced vertically at every other cell, with the cell fully grouted with type S grout. The walls are not reinforced horizontally, although joint wire reinforcement is provided. (A number of walls have horizontal bond beams with reinforcing steel. The following discussion does not apply to these walls.)

   The analyses employed finite element orthotropic plate bending models. Use of finite element models enabled proper spatial consideration of concentrated loads, openings, and boundary conditions. The orthotropic SAP IV element allows uncoupled properties in the two orthogonal directions to be input directly to the material constants matrix. It is important to consider that in the case of a uniformly loaded wall with zero stiffness in the horizontal direction, the Pilgrim finite element analysis yields the same results as a vertical strip equivalent beam calculation.

   At low levels of lateral load, all regions of a wall remain uncracked. Behavior is the same as if the wall were unreinforced. Since most of the stress using the linear strain assumption occurs in the face shell of the block and since these are globally isotropic, the wall behaves as an isotropic slab. The presence of vertical grouted cores does not significantly influence the response because they are close to the neutral axis. The moment of inertia based only on the face shells differs little from that including the grout cores. Likewise, the finite element analysis gives the same results as an isotropic closed form solution. Further, for a wall with an aspect ratio 2:1, the Pilgrim finite element analysis gives the same results as an equivalent strip analysis spanning in the short direction.

   As load increases, certain regions of the wall will reach the cracking stage. For the vertical direction, the grout cores passing through the bed joint reduce the effect of this plane of weakness [1, 2] and provide sufficient continuity to develop beam action of the type exhibited by reinforced concrete [3]. Because the cracking moment for masonry is taken conservatively low, using Branson's equation to account for the reduced rigidity in the reinforced direction yields conservatively high predicted displacements.

In the horizontal direction, if the moment exceeds the unreinforced allowable, based on the face shell area only, the section is assumed cracked and unable to transmit any load. In the finite element model, the element stiffness in the horizontal direction is set to zero. This conservatively neglects any contribution from grout cores or joint wire reinforcement, and Branson's equation is not used. Thus, the walls are assumed to exhibit two-way action <u>only</u> where uncracked in the horizontal direction.

An exception to this occurs where horizontal bond beams exist. The analyses of these walls used an average steel area over the wall height and two-way analysis. A comparative analysis of wall 64.4/65.8 showed this conservative with respect to considering the wall unreinforced horizontally except for a narrow beam strip containing the bond beam steel.

2. Equations developed for adequately reinforced concrete slabs have been used in the analysis to account for the orthotropic properties resulting from differing steel reinforcement details in the vertical and horizontal directions. The applicability of these equations to block masonry walls is questionable because of the notable differences between a reinforced concrete slab and a block masonry wall. First, concrete is a globally homogeneous material, whereas masonry is not. This is particularly true for partially grouted walls. Secondly, the percentage of reinforcement and detailing in the two directions are quite different in the two cases. Steel orthotropy for which these equations were developed is not applicable for the Pilgrim walls which have no horizontal steel. Thirdly, because masonry is a jointed medium, one expects crack patterns, and consequently the steel contribution, to be different from those in reinforced concrete. In light of these comments, justify the use of equations developed for the reinforced concrete slab to qualify the masonry walls.

Response

This question addresses the applicability of the principles of reinforced concrete analysis to the analysis of masonry walls. Schneider and Dickey state that "The precepts involved here in the analysis and design of reinforced masonry are those of the elastic theory of working stress design (WSD), long utilized in designing reinforced concrete elements. As a matter of fact, most of the design and/or analysis formulas are similar to those for reinforced concrete except that the ultimate strength of the masonry, f'm, and the allowable stresses are reduced to reflect the properties of masonry instead of concrete [3]."

Drysdale and Hamid [1] used a finite element program to analyze stress distributions in masonry assemblages (under direct stress conditions.) They recommend using a "macromechanics" rather than a "micromechanics" approach as the more practical method. They also recommend treating the masonry "as globally homogeneous, with the effects of the constituent materials averaged into the composite properties." This approach is the same as was used to analyze the Pilgrim block walls, and the assumption is used in any calculation of masonry wall natural frequency, even by simplified hand methods.

The Pilgrim approach recognized the difference in percentage reinforcement between the two directions. This is addressed using the method of Timoshenko [4]. The grout cores surrounding the reinforcing steel pass through the masonry joints, reducing the effect of this plane of weakness [1, 2] and providing sufficient continuity to develop beam action of the type exhibited by reinforced concrete [3]. For walls with no horizontal steel, as noted in the response to question 1, the stiffness of the cracked section is set to zero, and the problem reduces to one-way bending. Because the mortar joints initiate cracking either parallel or normal to the bed joints, these are the principal directions [1, 2] and the assumption of special orthotropy, which was used in the Pilgrim analysis, is appropriate. As described above, the stiffness horizontally (as reflected in Icx) is based on unreinforced properties (and is zero after cracking.) Since the principal directions align with the finite element z and y directions, the directional stiffnesses may be independently specified. An unusual feature of the computer program used to analyze the Pilgrim walls is the ability to input directly to the stress-strain constants matrix. This enables independent specification of the properties in the element x and y directions, and the unreinforced direction may be properly considered.

3. Modulus of elasticity of the walls is assumed to be equal in the two orthogonal directions. This is not true for masonry which is a composite material. Assessment of the accuracy of this assumptions and its impact on the outcome of the analysis needs to be investigated.

Response

Test data on modulus of elasticity is sparse, and there is considerable scatter even for just the vertical direction. For level II analysis, Pilgrim used the ACI-531 value recommended by the SGEB criteria. Certainly, for uniaxial direct stress tests, the presence of grouted cores will affect the composite modulus. However, for the case of lateral bending, the composite stiffness is primarily dependent of the stiffness of the face shell regions of the block (as discussed in the response to question 2.) Thus, for lateral bending the horizontal and vertical moduli should be quite close, even for partially grouted walls. For wall regions which are cracked normal to the horizontal stress, the horizontal moment of inertia is set to zero and the modulus of elasticity in the horizontal directions has no effect.

4. The Branson equation has been used in level II analysis to determine the effective moment of inertias of different elements. This empirical equation was originally developed for reinforced concrete members under uniazial bending. Its applicability to two-way bending of block masonry walls needs to be demonstrated. It must be noted that the Branson equation has been used to express effective moment of inertia in the horizontal direction where there is no reinforcing steel.

   Response

   While Branson's equations were originally developed for beams, Jofreit and McNeice [5] demonstrated their applicability to the analysis of one-way and two-way slabs. Additionally, as discussed in the response to question 1, the Pilgrim analysis did not use Branson's equation for the horizontal direction where there is no steel.

5. Higher damping values have been used in level II analysis. What is the basis for choosing higher damping values? Are these values changing with the level of loading?

   Response

   The level I analyses were performed prior to the issuance of the SGEB criteria and a very conservative value of damping was used. Level II analyses were performed after issuance of the SGEB criteria and the damping values were changed to agree with the SGEB recommendations. There was no guidance in the literature as to damping of reinforced masonry.

6. Review of calculations of wall 64.4/65.8 revealed a significant difference in element moments from level I and level II analyses. For example, moments in the critical element of the bond beam were reduce by 88% shifting from level I to level II analysis. How could this reduction be justified and what are the main reasons for such a large change?

   Response

   The level I analysis assumed "fully cracked" sections (ie, applied moment is equal to allowable moment) over the full height and width of the wall. Also, since it is an equivalent static analysis, response spectra accelerations were increased by 30%. The level I analysis of wall 64.4/65.8 used an applied acceleration of 2.83g's, the spectral peak at 5% damping times 1.3. The level II analysis used 1.0g, because the frequency was calculated to be 5.8Hz. (away from the resonant peak), damping was taken as 7%, and higher modes were included (which typically increase response less than 5%.) This implies a reduction in moment of about 65%.

   However, the moment reduction in the bond beam may be expected to be greater than 65%. This is because in the level II analysis the effective moment of inertia in the vertical direction is higher than for the level I analysis (because it is not "fully cracked".) Thus, proportionately more load is carried by the vertical steel than by the bond beam steel.

7. Wall 188.10 has an aspect ratio greater than 3, which calls for almost a single curvature with bending primarily in the shorter direction. The crack pattern (parallel to the shorter direction), which is predicted from the computer analysis, does not seem to be consistent with the one-way bending action of the wall. This inconsistency does not provide confidence in the capability of the proposed analytical model to predict actual behavior.

   Response

   For wall 188.10 the critical load case was determined to be PBOC. The response centers around this load case but the argument applies equally well to OBE and SSE loads. Attached are the initial (uncracked) and "converged" cracked states of the wall for the PBOC loads.

   The initial run (uncracked) indicates that the wall is spanning vertically as the cracking moments due to vertical bending are exceeded causing horizontal cracking as shown in Figure B.7-1 . When the cracked properties for these elements are included in a subsequent model, vertical cracks occur near the midheight of the wall as shown in the coverged crack pattern (Figure B.7-2). This load redistribution is to be expected after full length horizontal cracking has occurred. However, the wall is still carrying its out-of-plane loads to the top and bottom boundaries (i.e., it is spanning vertically) as indicated on sheet 24 of the original calculations. This sheet indicates that the top boundary takes a total load of 15,317 pounds and the bottom boundary takes 13,363 pounds for a total top/bottom load of 28,680 pounds. The total load applied over the wall is 72 psf x 10'-8" x 41'-3" = 31,185 pounds. This indicates - approximately 92% of the load is carried to the top and bottom boundaries, and the wall is in fact spanning vertically.

   This example does, then, demonstrate the ability of the analytical technique to predict actual behavior.

8. It is not clear how existing cracks (e.g, in wall 64.4) have been accounted for in the analysis.

Response

The reanalysis of 64.4/65.8 to include existing cracks considered the wall to be cracked over the entire height and width (rather than just where cracks are visible.) Thus, the horizontal stiffness was taken as zero except in the strip of elements containing the bond beam steel. For this strip and for the vertical direction, the effective moment of inertia was calculated using Branson's equation.

References:

1. Drysdale and Hamid, "Tension Failure Criteria for Plain Concrete Masonry," Journal of the Structural Division ASCE, Feb. 1984.

2. Drysdale, Hamid, and Heidebrecht, "Tensile Strength of Concrete Masonry," Journal of the Structural Division, ASCE, July 1979.

3. Schneider and Dickey, Reinforced Masonry Design Chapter 6, "Engineering Design."

4. Timonshanks and Woinowsky - Krieger, Theory of Plates and Shells, Chapter 11, "Bending of Anisotropic Plates."

5. Jofreit and McNeice, "Finite Element Analysis of Reinforced Concrete Slabs," Journal of the Structural Division, ASCE, March 1971.

FIGURE B.7-1

Wall 188.10 - crack Patterns - initial run

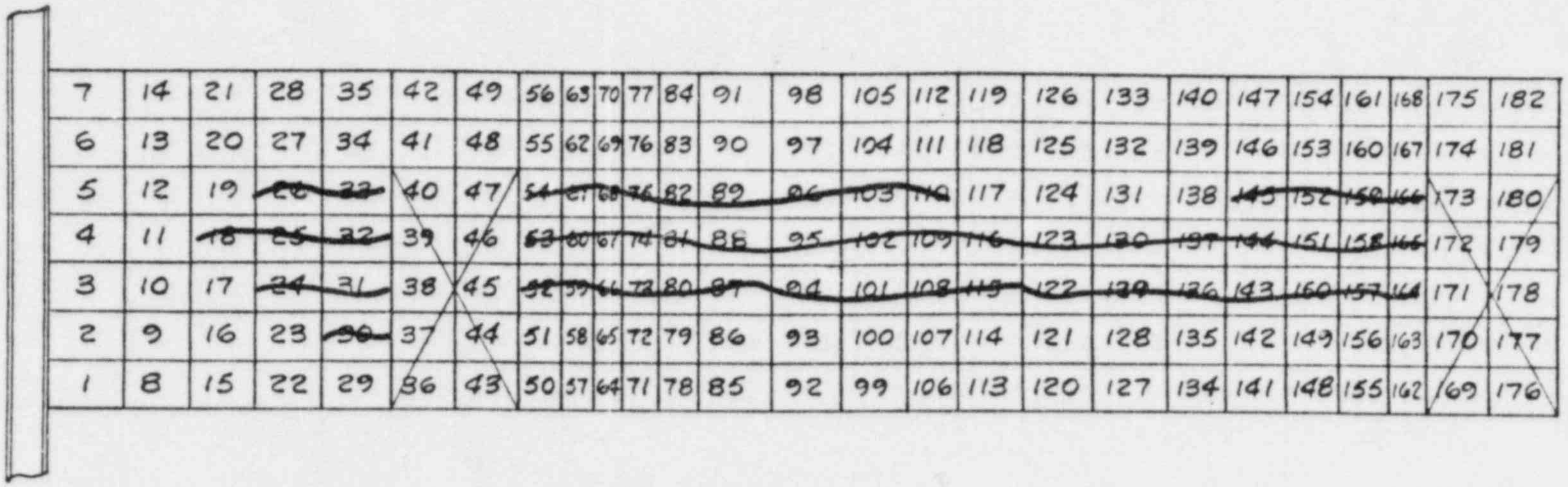

FIGURE B.7-2

Wall 188.10 - crack patterns - final run