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DENTON,H.R. Office of Nuclear Reactor Regulation, Director

SUBJECT: Forwards addl info re IE Bulletin 80-11, "Masonry Wall Design," in response to NRC 811229 request.Technical justification provided for allowable shear of tension stresses at concrete core/block wythe interface.

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Iowa Electric Light and Power Company
March 5, 1982
LDR-82-065

LARRY D. ROOT
ASSISTANT VICE PRESIDENT
NUCLEAR GENERATION

Mr. Harold Denton, Director
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Dear Mr. Denton:

The attachment to this letter provides the additional information to our IE Bulletin 80-11 response requested in Mr. Thomas Ippolito's letter to Mr. Duane Arnold, dated December 29, 1981.

Should you have any questions, please contact this office.

Very truly yours,

Larry D. Root

Larry D. Root
Assistant Vice President
Nuclear Generation

LDR/YB/dmh*
Attachment

cc: Y. Balas
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RESPONSE TO NRC QUESTIONS

1. Question

With regard to the material strength, identify the type of masonry and mortar used and justify their compressive strengths as given in Attachment 3, Section 4 [3].

Response

The type of masonry used at DAEC was ASTM C90, Grade PI. The compressive strength over the average net area, as given by Table II of ASTM C90-66T and as given in Reference 1, Attachment 3, Section 4 is 2000 psi. Results of laboratory testing, which are on file, confirm that this value is conservative.

The type of mortar used at DAEC was ASTM C476, Type PL. The compressive strength as called for in contract specification and as given in Reference 1, Attachment 3, Section 4 is 2000 psi. Results of laboratory testing, which are on file, confirm that this value is conservative.

2. Question

In Section 5.1.1 [3], the allowable shear or tension stresses at the concrete core/block wythe interface was stated to be 8 psi. Provide technical justification for this value.

Response

For the analysis in response to Bulletin 80-11, multiple wythe walls were classified into the following three major categories:

1. Walls with mortar joints (collar joints) between masonry wythes.

2. Walls with a grouted core less than three inches thick between the masonry wythes.

3. Walls with a placed concrete or grouted core three inches or greater in thickness between the masonry wythes.

In the first category, the allowable shear and tensile stresses due to bond between the mortar and the masonry were assumed to be zero. For these walls, either the wythes were shown to be capable of resisting the loading independently or a check was made to ensure that the reinforcing ties connecting the wythes would not be overstressed.

For the second category, the very low basic allowable stress for shear and tension of 8 psi was established as an acceptable limit at the beginning of the NRC IE Bulletin 80-11 analysis work. This limit was not subsequently used to qualify any of the walls in this category. All walls in this second category were qualified in the same manner as those in the first category. Since the 8 psi was not used, further discussion for justification of this value is not pertinent.

For discussion of allowable core/wythe interface shear and tensile stresses for the third category walls, see response to question 13.

3. Question

With regard to shear and bond stresses for factored loads, a factor of 1.67 was introduced. SEB criteria [4] allow a factor of 1.3 for shear carried by masonry. Justify the use of a factor of 1.67.

Response

The factor of 1.67 for shear and bond stresses was chosen in May 1980 (Reference 1, Attachment 3) prior to the SEB criteria being published in July 1981 (Reference 3). Code allowable stresses for masonry, shear, and bond were increased by a factor of 1.67 for load combinations involving abnormal and/or extreme environmental conditions which are credible but highly improbable. Since code allowable stresses (Reference 2, Chapter 10.1 of the commentary) are generally associated with a factor of safety of 3, the 1.67 increase provides a factor of safety against failure of 1.8 (3 ÷ 1.67). It is our engineering judgement that a factor of safety of 1.8 is conservative and allows sufficient margin for abnormal and/or extreme conditions.

4. Question

With regard to tension stress, a factor of 1.67 was introduced for factored loads. Indicate if this factor is used for tension normal or parallel to the bed joint. SEB criteria [4] allows a factor of 1.3 for masonry tension perpendicular to the bed joint (for unreinforced masonry) and a factor of 1.5 for masonry tension parallel to the bed joint. In view of this, provide justification for the factor of 1.67.

Response

The factor of 1.67 for tension stress was chosen in May 1980 (Reference 1, Attachment 3), prior to the SEB criteria being published in July 1981 (Reference 3). Code allowable stresses for masonry tension normal or parallel to the bed joint were increased by a factor of 1.67 for load combinations involving abnormal and/or extreme environmental conditions which are credible but highly improbable. Since the code allowable stresses (Reference 2, Chapter 10.1 of the commentary) are generally associated with a safety factor of 3, the 1.67 increase provides a factor of safety against failure of 1.8 (3 ÷ 1.67). It is our engineering judgement that a factor of safety of 1.8 is conservative and allows sufficient margin for abnormal and/or extreme conditions.

All blockwalls at the DAEC plant contain steel tension reinforcement which take all tension normal or parallel to the bed joint (Reference 1, Response to Item 2b). Therefore the assumed allowable masonry tensile stress normal or parallel to the bed joint was zero (psi) for the analysis in response to Bulletin 80-11. Since the masonry tensile strength at the bed joint was assumed to be zero the strength increase factor stated in the above question was not used in the analysis of the blockwalls.

5. Question

In Section 5.2.1 [3], for factored loads, a factor of 1.5 was given for the shear and tension of the collar and core/wythe interface. Justify this factor.

Response

For extreme and/or abnormal load conditions, a factor of 1.5 was used for shear and tension of the collar joint and core/wythe interface. Since the code allowable stresses for masonry shear and tension are generally associated with a safety factor of 3 (Reference 2, Chapter 10.1 of the commentary), the 1.5 increase provides a factor of safety against failure of 2.0 (3 ÷ 1.5). It is our engineering judgement that a factor of 2.0 is conservative and allows sufficient margin for abnormal and/or extreme conditions.

6. Question

In Section 5.2.1 [3], the Licensee discussed the stress values used for walls without inspection. Indicate if any walls at the Duane Arnold plant fall into this category.

Response

None of the walls at the Duane Arnold Energy Center fall into the category of walls without inspection.

7. Question

With regard to the in-plane strain allowable for nonshear walls, provide the technical basis for the value used for the unconfined wall.

Response

The allowable in-plane strain was .0001 for an unconfined wall (Reference 1, Attachment 3). The technical basis for using this value is based on the fact that none of the masonry walls are required to carry any of the building's story shear or moment. Also, work by Fishburn (Reference 5) and Becica (Reference 4), yielded an allowable shear strain for an unconfined masonry wall of .0001.

8. Question

The Licensee introduced (a) the method of nonlinear analysis, (b) the energy balance technique, and (c) the arching theory. It is the NRC position that these techniques should not be used in the absence of conclusive evidence of their validity as applied to masonry structures.

Response

The energy balance technique was the only alternative method used in the analysis of masonry walls. Justification for using this alternative method of analysis is given in Reference 1, Attachment 8, Subsection 6.0, Alternative Acceptance Criteria.

The "energy balance technique" was used to show that the overstressing will not cause wall failure. In addition, the overstressing does not occur near safety-related items, nor are there any safety-related systems attached to the wall which would be affected by the calculated deflection (Reference 1, Response to Item 2).

9. Question

With regard to damping, the Nuclear Regulatory Guide [4] allows 4% for reinforced concrete subject to the safe shutdown earthquake. Justify the use of 5%.

Response

Damping values specified for the analysis in response to Bulletin 80-11 at DAEC were 5% for Operating Basis Earthquake (OBE) and 7% for Safe Shutdown Earthquake (SSE). The recommended values given for reinforced concrete in the U.S. Atomic Energy Commission Regulatory Guide 1.61 are 4% for OBE and 7% for SSE. The damping value for masonry blockwalls is expected to exceed the damping value for a comparable concrete wall. Therefore the 5% damping value used for OBE is considered adequate. The 7% damping value for SSE is the same as that specified in the Regulatory Guide for reinforced concrete and is therefore considered conservative.

The governing seismic design load for the analysis of all masonry blockwalls at DAEC was the safe shutdown earthquake (SSE). Therefore, the 5% damping value specified for OBE was not used in the analysis since OBE loading was not a governing load in the analysis of the masonry blockwalls.

10. Question

With respect to modes of vibration that are higher than the fundamental mode, indicate how the higher mode effects are accounted for.

Response

In lieu of a more detailed modal analysis, conservative procedures were used in the analysis in response to Bulletin 80-11 which were sufficient to account for higher mode effects (Reference 1, Attachment 3). The analysis considered indirectly the higher modes by using the seismic acceleration associated with the floor spectra either at the bottom of the wall or the floor spectra at the next higher elevation, whichever yielded the maximum acceleration response for the first mode frequency. This maximum acceleration response for the first mode frequency was then applied over the entire area of the wall rather than varying the acceleration based on the deformed shape of the wall. The increased bending moments and shears due to using these procedures will account for the higher mode effects, since the first mode response typically accounts for over 99% of the total square root sum of squares (SRSS) response of a wall.

11. Question

With regard to seismic analysis, indicate how the components of seismic load in various directions are accounted for.

Response

The components of seismic load in various directions were considered by using the maximum acceleration response in a direction perpendicular to the plane of the wall. If a wall was at an angle to the principle directions of response, the response curve which gave the maximum response was used. In either case, the maximum acceleration response will act in a direction perpendicular to the plane of the wall.

12. Question

Indicate how pipe and equipment loads are accounted for.

Response

The attachment load on a wall was determined by approximating the weight of the attachment and multiplying it by the peak acceleration (Reference 1, Attachment 3). The peak acceleration was based on the floor spectra at the bottom of the wall or the floor spectra at the next higher elevation, whichever yielded the maximum response. In a few cases, the original seismic design load from the piping seismic analysis was used when the load information was available. Along with applying a concentrated load to the wall, the total weight of all attachment loads were added to the uniform dead load of the wall.

13. Question

With regard to the composite behavior of multiple wythe walls, the Licensee limited the shear and tension wythe interface to 22.4 psi for normal loading cases and 37.3 psi for extreme loading cases. Provide the technical basis for these values.

Response

The specified shear and tension stress for the interface between masonry block and a concrete or grout core (greater than 3" thick) was 22.4 psi (Reference 1, Response to Item 2b). This value is based upon the relationship $.5\sqrt{m_o'}$ given in ACI 531-79 which is the lowest allowable stress specified by the Code (Tension normal to a bed joint in hollow unit masonry). Since the core is greater than 3" thick, sufficient vibration of the grout infill was performed to provide a bond between the grout core and the masonry units. The bond between the concrete core and the block wythe is considered to be stronger than that between the block and the mortar in the bed joint. Therefore, this low allowable stress value is considered to be very conservative. The 37.3 psi, allowable for extreme loads given in the report is a typographical error which should have been 33.6 psi which is 1.5 times the 22.4 psi normal load allowable value.

14. Question

With respect to the load combinations, the Licensee's submittal [3] did not provide any factor greater than 1.0 for components of the combinations. Explain and justify this deviation from the plant's FSAR.

Response

The analysis used in the reevaluation of masonry walls in response to Bulletin 80-11 was based on the working stress method (Reference 1, Attachment 3). The working stress method does not use factored loads, as compared to the ultimate strength methods used for reinforced concrete design referenced in the FSAR. The strength margins for working stress method are maintained by the safety factors inherent in the allowable stresses.

15. Question

Discuss how the value of Young's modulus was selected for various calculations.

Response

The value of Young's modulus was taken from ACI 531-79, Reference 2, Table 10.1.

References

1. L.D. Root (Iowa Electric Light and Power Company) Letter with Enclosures to J.G. Keppler (NRC) dated November 10, 1980

2. ACI 531-79 and Commentary ACI 531-R-79 "Building Code Requirements for Concrete Masonry Structures," American Concrete Institute, 1979.

3. Standard Review Plan, Section 3.8.4, Appendix A, "Interim Criteria for Safety-Related Masonry Wall Evaluation," NRC, July 1981.

4. Becica, I.J., and H.G. Harris, "Evaluation of Techniques in the Direct Modeling of Concrete Masonry Structures," Drexel University Structural Models Laboratory Report No. M77-1, June 1977.

5. Fishburn, C.C., "Effect of Mortar Properties on Strength of Masonry," National Bureau of Standards Monograph 36 U.S. Government Printing Office, Nov. 1961.