SAFETY EVALUATION BY THE OFFICE OF NUCLEAR REACTOR REGULATION
PALO VERDE NUCLEAR GENERATION STATION, UNITS, 1, 2, AND 3
ENGINEERING BRANCH
DIVISION OF PWR LICENSING-B

PALO VERDE MASONRY WALL EVALUATION

The insufficient length of vertical reinforcing steel lap splices in the masonry walls due to construction errors has been an issue with the Palo Verde Nuclear Generating Station (PVNGS), Units 1, 2, and 3. Splicing of vertical reinforcing steel bars in PVNGS walls was due to construction convenience. In a lap splice, the stress in one bar must be taken out and put back to the other bar and this stress transfer must be accomplished within the length of the splice through bonding mechanism of concrete. This bonding mechanism creates bond stress in concrete. With a sufficient length of lap splice, the bars at the splice can reach yield stress before the concrete fails in bond. If the length of a lap splice is too short, the bond stress becomes too high at high load and will eventually fail the concrete before the bars at the splice can develop their yield stresses. The relationship among the length of a splice, tensile or compressive stress in the bars at the splice, and bond stress in the concrete at the splice have been determined through laboratory experiments. Thus, with any two variables known the third can be calculated. Building codes have specified allowable bond stresses for calculation use. The two most significant codes for masonry structures in the United States are the "Building Code Requirements for Concrete Masonry Structures" (ACI 351-79) and "Uniform Building Code." The 1979 edition of these two codes has been referenced by the Standard Review Plan. An allowable bond stress in concrete for lap splice of 120 psi has been specified in the ACI 351-79 code. Allowable bond stresses of 100 psi associated with no special inspection and 140 psi with special inspection have been specified in the 1979 Uniform Building Code.

An evaluation report of PVNGS masonry walls was telecopied to NRC from the licensee on April 4, 1986 (Reference 1). In the report the calculated bond stresses were 296 psi for the wall at elevation 74 feet and 140 psi for the

wall at 100 feet during SSE, with a peak ground acceleration of 0.2g. The report listed the allowable bond stress as 300 psi, which was calculated by using the allowable bond stress of 120 psi in the ACI 351-79 code multiplied by a factor of 2.5. The report rationalized that the factor of 2.5, which was specified in the Standard Review Plan for flexural and axial compression stress conditions for SSE, could also be applicable for bond stress. Since all the calculated bond stresses were below the allowable bond stress, the report concluded that the PVNGS walls were adequate. No information was contained in this report for the OBE condition.

A meeting was held on April 11, 1986 between the licensee and its consultant, Bechtel, and NRC at Bethesda, Maryland to discuss the April 4, 1986 report. During the meeting the licensee submitted additional analysis results for the OBE condition which were not available in the April 4, 1986 report. The analysis results indicated that the calculated bond stresses were 224 psi for the wall at elevation 74 feet and 50 psi for the wall at elevation 100 feet during OBE with a peak ground acceleration of 0.1g. On the information sheet the allowable bond stress was listed as 120 psi for comparison purposes. The analysis results also indicated that the calculated tensile stresses in the bars at the splices were 25,800 psi for the wall at elevation 74 feet and 12,800 psi for the wall at elevation 100 feet, compared with the ACI-531-79 code allowable stress of 24,000 psi. During the discussion the NRC staff pointed out that the allowable bond stress of 300 psi for the SSE condition listed in the April 11, 1986 report was incorrectly calculated because a factor of 1.5 instead of 2.5 should have been used for bond stress and, thus, the allowable should have been 180 psi instead of 300 psi. Since the calculated bond stresses have exceeded their allowables substantially (296 psi vs. 180 psi for SSE and 224 psi vs. 120 psi for OBE), the NRC staff told the licensee that the wall at 74 feet appeared to be inadequate. The NRC staff also told the licensee that the wall at 100 feet appeared to be adequate because the calculated bond stresses were below the allowables for both the SSE and OBE conditions.

The licensee formally submitted its evaluation report of PVNGS masonry walls on April 16, 1986 (Reference 2). In this report, the calculated bond stresses for both SSE and OBE remained the same as they had shown in the April 4, 1986 report and presented at the April 11, 1986 meeting, but the allowable bond stresses were revised as 387 psi for SSE and 230 psi for OBE for the wall at elevation 74 feet. Due to this revision, the licensee concluded that the PVNGS walls were adequate because the calculated bond stresses were below the allowables for both SSE and OBE conditions. The staff notified the licensee on April 17, 1986 that revised allowable bond stresses of 387 psi for SSE and 230 psi for OBE in the April 16, 1986 report were in error and the correct values should have been 180 psi for SSE and 120 psi for OBE and, thus, the walls at elevation 74 feet were inadequate.

The licensee submitted a revised evaluation report of masonry walls dated June 19, 1986 (Reference 3). In this report the allowable bond stresses were correctly stated as 180 psi for SSE and 120 psi for OBE. However, the calculated bond stresses were reduced to 110 psi for SSE and 80 psi for OBE for the walls at elevation 74 feet. The licensee then concluded that the walls were still adequate because the calculated bond stresses were below the allowable bond stresses.

The ratios of calculated bond stresses between the two reports (dated April 16, 1986 and June 19, 1986) are 2.69 for SSE and 2.80 for OBE. The licensee has not provided the reasons for the differences nor discussed the adequacy of its numerical values in view of the substantial differences. The staff has performed its evaluation and the evaluation results are discussed below.

The main cause of the differences in calculated bond stresses between the two reports is due to different ways of calculating the effective moment of inertia, I_e. The April 16, 1986 report had used the ACI code method to calculate I_e, whereas the June 19, 1986 report used another method to calculate I_e (this method is referred to hereafter as the Bechtel method). The I_e is a measure of

the stiffness of a masonry wall at a particular earthquake level, such as SSE or OBE. The stiffness determines the frequency or period (the inverse of frequency) of a wall, and the frequency then determines the seismic forces the wall will experience during an earthquake.

The process from an estimated I_e to stresses in a wall during an earthquake can be best illustrated by using the response spectra, as shown in Fig. 2 of the June 19, 1986 report, which is attached at the end of the text. When I_e was calculated using the ACI code method, the frequency of the wall at elevation 74 feet subjected to SSE became 3.03 cps (Reference 4), which corresponds to a period of 0.33 second, and at this period the wall would experience the near maximum acceleration and stress, as the response spectra represented by the solid line indicated in Fig. 2. However, if I_e was calcluated using the Bechtel method, the frequency of the same wall subjected to the same SSE became 4.88 cps (Reference 5), which corresponds to a period of 0.2 second, and at this period the wall would experience the near minimum acceleration and stress, as the response spectra indicated in Fig. 2.

Therefore, the ACI code method would yield a conservative value of I_e for the wall at elevation 74 feet even if the actual I_e of the wall were substantially different from the calculated I_e. However, the Bechtel method would yield a nonconservative value of I_e should the actual I_e be different from the calculated I_e.

The adequacy of the Bechtel method has not been verified, but the adequacy of the ACI code method has. The ACI code method was first introduced in 1971 and has been retained in the ACI Building Code since then, although other sections of the code have gone through several revisions. ACI Committee 435 reported in 1972 that by using the ACI code method of estimating I_e for deflection calculation for simply supported beams under controlled laboratory conditions, "there is approximately a 90% chance that the deflection of a particular beam will be within the range of 20% less than to 30% more than the calculated value" (Reference 6). Therefore, the ACI code method was used to

benchmark the adequacy of the Bechtel method. Since the frequency of the wall at elevation 74 feet during SSE was calculated to be 3.03 cps by using the ACI code method and 4.88 cps by using the Bechtel method, the ratio in frequencies between the two methods is 1.61. This is equivalent to a ratio of 2.59 in the values of I_e between the two methods, because I_e is a square function of frequency. The Bechtel method of establishing I_e overestimates the wall stiffness by a factor of 2.59 in comparison with the ACI code value. It must be pointed out that the mathematical model of estimating I_e in the Bechtel method is not an established method and has not been benchmarked by laboratory tests. Had Bechtel compared its computed deflections using its method of estimating I_e with the measured deflections of those controlled laboratory test data used by ACI Committee 435, Bechtel would have found that its computed deflections had been substantially lower than measured deflections.

The previous paragraph has discussed the inadequacy of the Bechtel method in estimating I_e. This paragraph discusses an evaluation consideration that is necessary but has not been considered in the June 19, 1986 report. ACI Committee 435 also reported in Reference 6 that "...a variety of factors may be known or estimated in the laboratory that are unknown in the design situation. Because of these uncertainties, the variability of deflection between actual field and calculated deflections can be expected to be greater than the variability of deflection between actual laboratory and calculated deflection. Therefore, the results of the analysis reported in this report must be viewed as minima rather than average or expected values. Greater variability would most likely be obtained if field data were used instead of laboratory data." As previously discussed, the main reason that Bechtel could reduce the calculated bond stresses substantially in the June 19, 1986 report from those in the previous report was due to the fact that the period of the wall at elevation 74 feet had been tuned to 0.2 second by using the Bechtel method in estimating I_e instead of 0.33 second by using

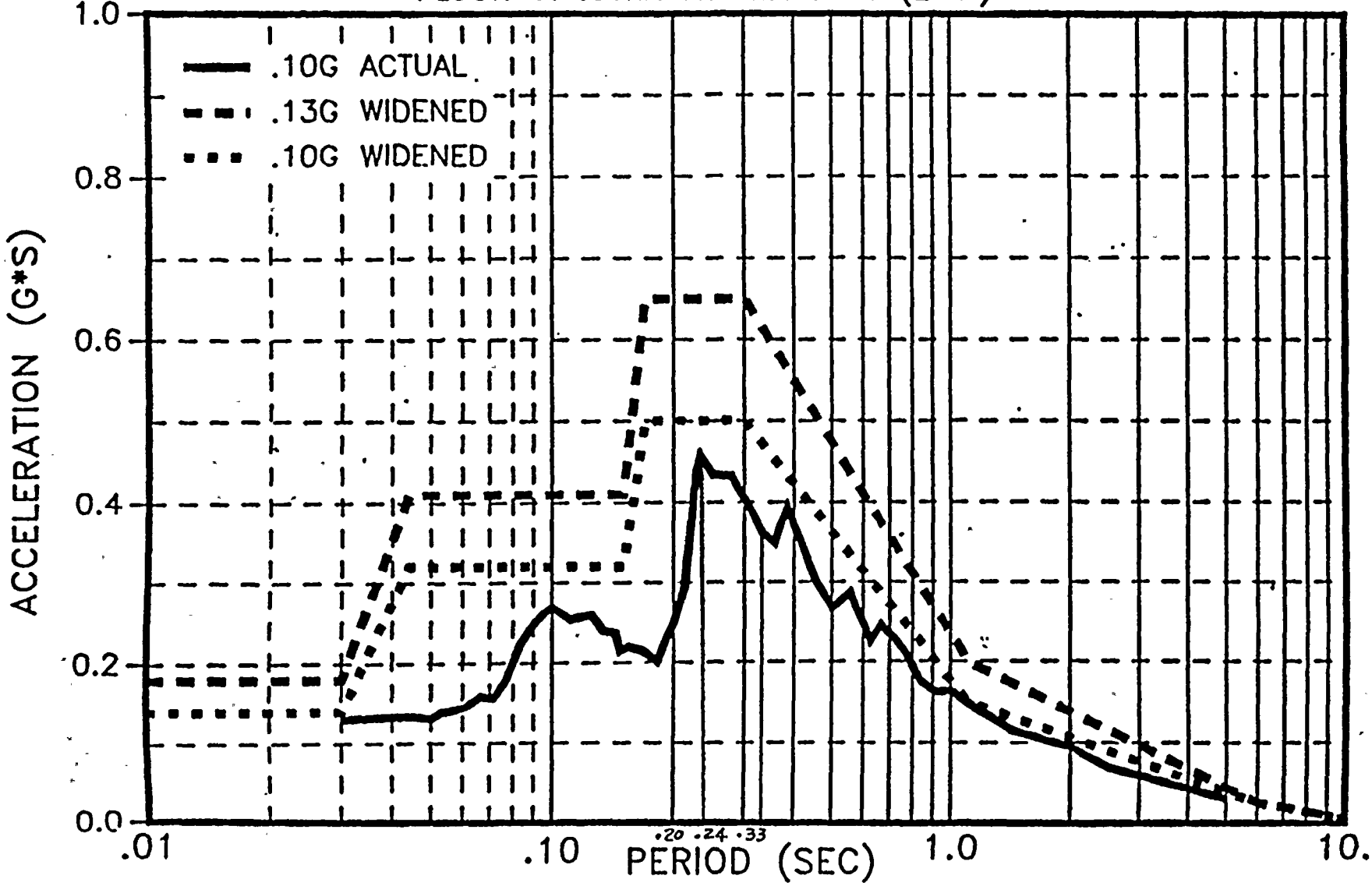
the ACI code method. In view of the range of variability of the actual field I_e against the assumed I_e in design calculation as stated by the ACI Committee 435, some variation of I_e must be considered in the analysis. If a variability in I_e of 40% is considered, the period of 0.2 second of the wall at elevation 74 feet would become 0.24 second. It can be seen from the response spectra in Fig. 2 that the wall could experience from the near minimum acceleration and stresses at the period of 0.2 second to the maximum acceleration and stresses at the period of 0.24 second within a variability in I_e of 40%. This was not considered in the June 19, 1986 report.

The review results performed by A. Hamid of Drexel University on the applicant's June 19, 1986 submittal are presented in Appendix A. Dr. Hamid has concluded that the Bechtel evaluation methodology regarding the calculation of wall stiffness is not justified for the following three reasons: (1) the modulus of elasticity of walls has been overestimated, which led to a non-conservative estimate of wall stiffness, (2) the assumption of using a 3-stage moment of inertia is neither realistic nor conservative in estimating wall stiffness, and (3) the sensitivity of stress change in response to wall frequency shift due to the variability of calculational accuracy and of inherent material nature has not been considered. Appendix A was prepared independently by Dr. Hamid and its conclusion is in agreement with the staff's evaluation.

Based on the review of the information submitted by the licensee and the reasons as stated and discussed above, the staff has concluded that strengthening the walls at elevation 74 feet is required.

FIGURE 2

COMPARISON OF OBE HORIZONTAL
FLOOR SPECTRA AT ELEV. 74 (E-W) 5% DAMPING



References

1. "Evaluation of PVNGS Masonry Walls," from Bill Quinn, Arizona Nuclear Power Project, to E. Licitra, NRC, dated April 3, 1986.

2. "Evaluation of PVNGS Masonry Walls," from E. E. Van Brunt, Jr., Arizona Nuclear Power Project, to G. W. Knighton, NRC, dated April 16, 1986.

3. "PVNGS Masonry Walls," from E. E. Van Brunt, Jr., Arizona Nuclear Power Project, to G. W. Knighton, NRC, dated June 19, 1986.

4. Bechtel Calculation No. 13-CC-25-120, Sheet No. 7.

5. "Response to NRC Requests for Information," telecopied to NRC from Bechtel, dated July 30, 1986.

6. "Variability of Deflections of Simply Supported Reinforced Concrete Beams." Report by ACI Committee 435, ACI Journal, Proceedings 69, January 1972, 29-35.