Appendix A

Technical Report
on

EVALUATION OF BLOCK MASONRY WALLS AT PALO VERDE NUCLEAR GENERATING STATION

submitted
to

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1- INTRODUCTION

Block masonry walls at palo Verde Nuclear generating Station (PVNGS) Units 1,2 and 3 are 12 in. block walls fully grouted and reinforced both vertically and horizontally. Vertical bars were constructed with lap splices which were not staggered and have lengths less than that specified in the ACI 531 masonry code (1) for reinforced masonry construction. The NRC staff and consultants have expressed their concerns regarding the bond stresses at the splices and the margins of safety under SSE and OBE earthquake loads for walls at 74 ft Elevation.

The NRC staff and consultants visited the plant on March 20,1986 and inspected the masonry walls. Several meetings were conducted at the NRC to discuss different aspects of the problem. Two reports dated April 16, 1986 (2) and June 19,1986 (3) were submitted by the licensee regarding masonry wall evaluation at PVNGS. A more recent and comprehensive report (4) was submitted to NRC on September 19, 1986 which contains information presented at the August 20 and 28, 1986 meetings.

This report presents a review of the September 19,1986 report regarding the technical evaluation of masonry walls at PVNGS.

2- ANALYTICAL METHODOLOGY

Time history analyses were performed by Bechtel on coupled models that included representations of both the control building structure and the masonry walls. The soil-structure interaction was considered in this study. A lumped mass model of the control building was used to develop the response spectra. A stick model of

1- ft. strip of the wall was used to analyze the masonry wall at Elevation 74 ft. A single direction T-H record was used to analyze the wall using the finite element method adopting a macro-analysis approach (i.e. ,mortar joints were not modelled). A number of assumptions was used in Betchel analysis presented in the June 19, 1986 report and subsequent meetings. An evaluation of these assumptions is presented below.

a) Single direction time history - This represents a realistic approach since the masonry walls in question are nonloadbearing elements for which the out-of-plane behavior dominates their response.

b) Strip idealization of the wall - The wall behavior is assumed to be one-way in the vertical direction which is a conservative and a realistic assumption because the side boundaries of the walls are free. Also, the wall pattern is a running bond and openings are adequately reinforced which assure continuity in the horizontal direction.

c) 3-stage moment of inertia - It is assumed that the wall undergoes three stages of cracking :1) uncracked, 2) partially cracked where only the faceshell is cracked (i.e. ,mortar debonding) when extreme fiber flexural stresses exceed 2.5 f'_m, and 3) fully cracked when the tensile stresses in the extreme fibers of the grout cores reach modulus of rupture of the grout which was assumed equals to 7.5 f'_m where f'_m is the grout compressive strength. Test results (5) do not support the Bechtel assumption of 3-stage cracking model. The tests indicate that cracking of the faceshell will occur simultaneously with cracking of the grout and that grouted masonry, as a composite material, has only one cracking moment. The Bechtel approach is neither realistic nor conservative in estimating wall stiffness.

d) Modulus of Elasticity - Wall modulus of elasticity is assumed to be equal to 1000 f'm, where f'm is the prism compressive strength. This formula, which is specified by current masonry codes (1,6) , highly overestimates the elastic modulus and would lead to nonconservative estimate of wall stiffness (7).

3- RESULTS

The time history analyses coupled with the 3-stage model revealed much lower bond stresses (110 psi for SSE and 80 psi for OBE at Elevation 74 ft) compared with those from previous simpified analysis presented in the April, 1986 report (2) .These stresses correspond to a wall frequency of 4.9 cps. The large difference is attributed to the fact that the wall dynamic response is very sensitive to calculated frequency because of the proximity to the amplified region of the response spectra curve, see Fig. 1. Bechtel concluded that walls at PVNGS are adequate because calculated bond stresses were below the code allowables (180 psi for SSE and 120 psi for OBE).

4- SENSITIVITY OF WALL RESPONSE

The NRC staff and consultants expressed during the meetings their concerns regarding the sensitivity of wall response to calculated frequencies and the uncertainities associated with estimating the material properties of PVNGS masonry walls.

Wall stiffnesses and therefore frequencies are dependent on a combination of parameters; mainly wall geometry boundary conditions, and material properties. Wall geometry and boundary conditions are well defined by the as built structure itself.

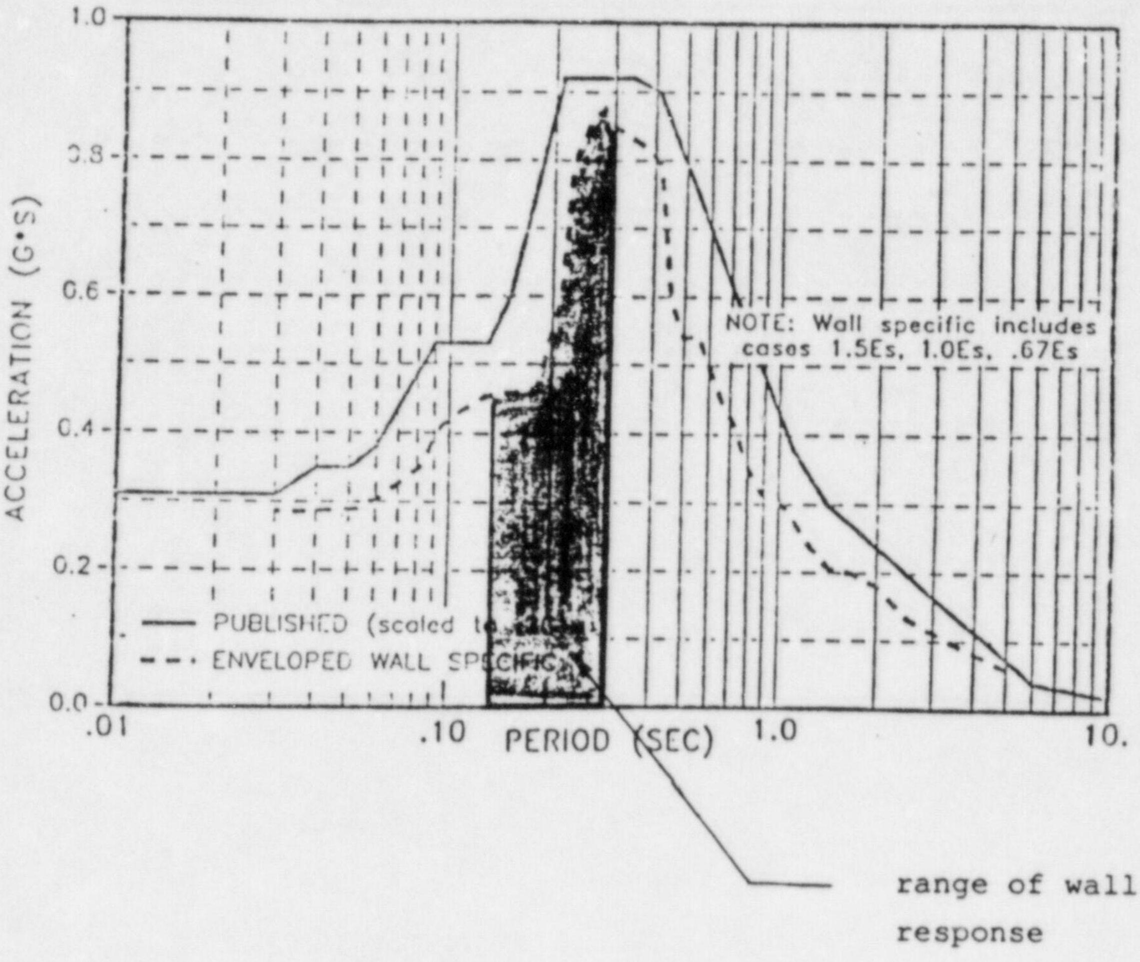

Fig. 1- Floor Spectra at Elevation 74'-0"

Material properties such as modulus of elasticity and modulus of rupture are not easily defined. Current codes provide guidelines based on available test data. It is also very difficult to base precise conclusions regarding the values of the modulus of elasticity and the modulus of rupture for a specific wall upon data in the literature simply because masonry is a highly variable material that depends not only on the components used but also on the workmanship and construction procedure.

In response to the concerns of the NRC Staff and consultants regarding the values of modulus of elasticity and modulus of rupture of PVNGS walls, Becthel conducted parameteric studies based on available test data. An evaluation of these studies is presented below:

a) Modulus of Elasticity

Modulus of elasticity affects wall stiffness and frequency. For PVNGS walls the minimum value of modulus of elasticity will be the critical one governing the design.In the evaluaion of PVNGS masonry wall stresses a value of 1.5×10^6 psi for modulus of elasticity was used. This is based on the code value of 1000 f'_m where f'_m is the prism compressive strength which was taken equal to 1500 psi. In the September 19, 1986 report it is stated that a more representative value of f'_m would be 2000 psi based on published experimental data (9) . These results were obtained from testing grouted masonry prisms that do not in any way represent PVNGS walls; this includes block and grout properties and construction procedure.

It is stated in the report (4) that the modulus of elasticity value of PVNGS walls would exceed 1.5 x 10^6 psi by extrapolating from data reported in Ref. 10 for concrete block masonry prisms which do not duplicate materials and construction procedures for PVNGS walls.

The modulus of elasticity as calculated from the stress-strain curve is highly sensitive to the method of testing, the shape of the prism, and the stress level at which it is calculated.As can be seen in Fig. 2 , high variation in the ratio of the elastic modulus to the compressive strength is reported (7).

The Atkinson and Noland report (8) and other studies (7) show that a more realistic ratio of the modulus of elasticity to compressive strength would be in the range of 500 to 700 .

Based on a conservative yet realistic ratio of modulus of elasticity to compressive strength of 500, the elastic modulus of PVNGS walls would be 500 (1500) = .75 x 10^6 psi. This value is 50 percent lower than the minimum value used in Bechtel calculation. This would result in a 33 percent reduction in frequency which will significantly increase the response of the wall due to its proximity to the amplified region of the response spectra.

b) Modulus of Rupture

In April and June, 1986 submittals Bechtel used a modulus of rupture equal to 2.5 f'_m where f'_m is prism compressive strength. This is according to UBC provisions (6) for grouted block masonry construction. The coefficient of 2.5 is based on test results of 6 in. concrete block masonry walls (10). Limited test data (10) shows a trend of increasing the modulus of rupture with wall

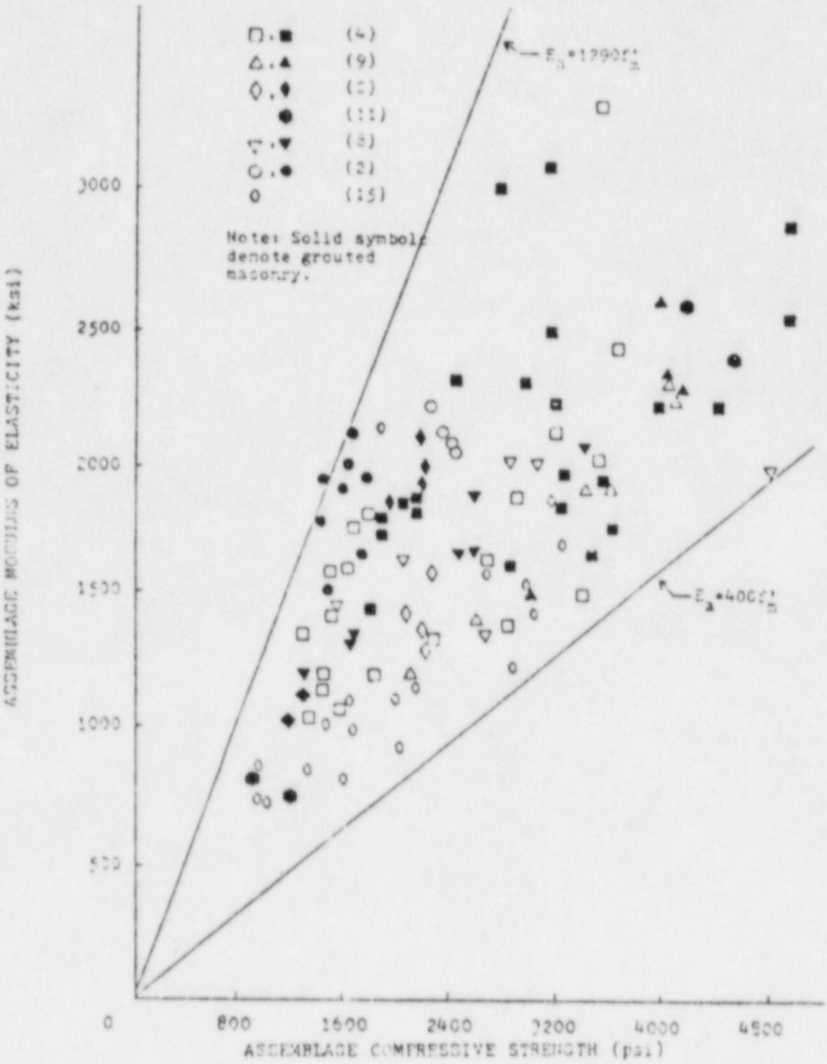

Fig. 2 - Test Data on the Relationship Between Modulus of Elasticity and Masonry Compressive Strength (7)

thickness. However, Atkinson and Noland (8) concluded that "it would be very difficult to base precise conclusions regarding the values of the modulus of rupture and the modulus of elasticity for a specific wall or walls upon data in the literature."

5- CONCLUSION

Based on the review of the information submitted in the September, 1986 report (4) and discussions of concerns presented above, it is concluded that Bechtel design methodology for PVNGS masonry walls regarding the calculation of wall stiffness is not justified nor conservative. It is not appropriate to base precise conclusions regarding the values of material properties of PVNGS walls upon extrapolating data in the literature. This approach could lead to nonconservative results for bond stresses in lap splices at Elevation 74 ft.

6- REFERENCES

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