REPORT ON THE

EVALUATION OF MASONRY WALLS

FOR THE

ARIZONA NUCLEAR POWER PROJECT

PALO VERDE NUCLEAR GENERATING STATION

UNITS 1, 2, AND 3

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1. INTRODUCTION

In accordance with ANPP letters to the NRC dated April 16, 1986 and April 18, 1986 (References 1 and 2), ANPP has continued to work to resolve outstanding issues regarding the integrity of some PVNGS masonry walls. The April 16, 1985 letter and its enclosure provided a summary of the significant attributes of the issues involved, a description of documentation reviews and field investigations, and a presentation of evaluation results available at that time. The April 18, 1986 letter stated that, to resolve the remaining issues, ANPP would provide an acceptable analysis, perform additional testing, or implement design modifications on the masonry walls, by December 22, 1986 for Units 1 and 2 and prior to initial criticality for Unit 3. The purpose of this report is to provide descriptions of subsequent evaluations and studies, and present the results and conclusions associated with the most recent investigations.

As discussed below, additional, more precise analyses show that under postulated seismic conditions the masonry compressive stresses, reinforcement tension stresses, and reinforcement bond stresses would be within allowable limits accepted by the NRC and defined by Appendix A to SRP 3.8.4 (NUREG 0800, July 1981) and the ACI 531-79 Code. The computed stress levels verify the adequacy of the walls to perform their intended function and ensure that the walls will not impair the performance of adjacent or attached Seismic Category I equipment.

2. SUMMARY

Detailed time history analyses* of the Control Building Masonry walls at Elevations 74'-0" and 100'-0" were performed to demonstrate that, under seismic conditions, masonry, reinforcement and bond stresses satisfy acceptance criteria. The analyses utilized an analytical model with beam elements representing the masonry walls coupled to the Control Building beam-stick model used to generate the existing floor response spectra. Investigations of the effects of soil and wall stiffness variation, and partial fixity of the base connection detail at Elevation 74'-0", were also performed.

The results of the time history analyses along with additional equivalent static analyses (for in-plane loads) were used to calculate the stresses in the various wall components. This more precise method of analysis verifies that, for both elevations, and under OBE and SSE loading conditions, the masonry, reinforcement, and bond stresses are within allowable limits defined by Appendix A to SRP 3.8.4 and ACI 531-79. Masonry compressive stresses have been shown to be below allowable limits that are conservatively reduced to compensate for the lack of full in-process

*These time-history analyses are dynamic response calculations utilizing the digitized PVNGS design acceleration time-history.

inspection. Reinforcement tensile stresses have been determined to be well below allowable limits and the bond stresses have been computed to be below allowable values at all lap splice locations. This verifies that the masonry units will respond under postulated loads within code requirements and that the development lengths provided by the existing lap splices (in the same or adjacent cells) satisfy the intent of the code.

3. CONCLUSION

Under seismic conditions, stress levels have been shown to be within allowable limits accepted by the NRC and defined by Appendix A to SRP 3.8.4 and ACI 531-79. The computed stresses verify the adequacy of the masonry walls to perform their intended function under 0.1g OBE and 0.2g SSE conditions (conditions consistent with PVNGS licensing commitments). The stress levels, along with calculated accelerations and deflections, also confirm that the walls will not impair the function of adjacent or attached Seismic Category I equipment. Because the walls have been shown to have ample additional stress margin beyond SSE requirements, it is demonstrated that the walls will remain elastic under operating and extreme environmental conditions. Therefore, it is concluded that the results of the most recent evaluations, along with information previously submitted to the NRC, respond to and resolve all of the outstanding NRC concerns regarding PVNGS masonry wall integrity.

4. DESCRIPTION OF ANALYSES AND EVALUATIONS

A. General

The walls being investigated are defined as those in the Control Building spanning from Elevations 74′-0" to 100′-0" and 100′-0" to 120′-0". These walls have been chosen because of their proximity to safety related equipment or because they have Seismic Category I items attached to them. The walls are non-shear wall, non-load bearing partitions subjected to vertical and lateral seismic inertial loads due to their own weight and the weight of light attachments (instrumentation tubing, conduit, junction boxes, etc.).

Previous seismic analyses of these masonry walls utilizing response spectrum analysis and equivalent static load analysis have yielded conservative results.

The analysis described below has incorporated time history analysis techniques resulting in a more precise and representative structural response. Stresses due to both in-plane and out-of-plane loading have been considered and are compared to limits specified by Appendix A to SRP 3.8.4 and the ACI 531-79 Code.

B. Time History Analysis

Previous equivalent static calculations and response spectra analyses have been performed for the subject walls, using conservative assumptions and utilizing decoupled models of the walls. In order to

provide a more precise evaluation, time history analyses were performed on coupled models that included representations of both the complete Control Building structure (including soil-structure interaction) and the masonry walls.

The lumped mass model of the Control Building that was used to develop the existing floor response spectra was used to represent the structure. Stick models representing one-foot strips of the masonry walls were formed using beam elements. A sketch of the coupled model used to analyze the wall at Elevation 74'-0" is shown in Figure 1. Single beams were used for each increment of wall height (10 to 12 inches). Section properties and mass of the wall were determined based on the one-foot width being analyzed. Wall mass included the weight of surface attachments, which was relatively small; approximately 1% of the wall mass at Elevation 74'-0" and 13% at Elevation 100'-0". The boundary conditions of the wall models were chosen to most closely reflect the properties of the as-built connection details as shown on the PVNGS Civil/Structural Detail Drawing 13-C-ZJS-172. The tops of the walls at both Elevations 74'-0" and 100'-0" were modeled as pinned. As described in Subsection 4.D of this report, the boundary condition of the base at Elevation 74'-0" was investigated in detail and concluded to be pinned. The base at Elevation 100'-0" is secured by dowels embedded in the wall and slab and, therefore, was considered as fixed. All of these boundary conditions are the same as those used in earlier analyses. The remaining five degrees of freedom associated with the boundary of the wall at each floor level were constrained to the Control Building beam-stick model degrees of freedom at Elevations 74'-0", 100'-0" and 120'-0".

Three different section properties were used to represent the masonry wall beam elements. The first section property was representative of the gross section. The second was representative of a partially cracked section (i.e., masonry unit is cracked to grout interface, and grout is uncracked). The third section property was used where the calculated moment equaled or exceeded that required to fully crack the element and represented a fully cracked section (i.e., cracked to neutral axis). Each analysis was initiated by assuming that all of the masonry elements had the properties of the gross section. If the resulting moments generated in any element were higher than those the wall section could tolerate without cracking, subsequent iterations were performed, gradually adjusting the section properties of those elements to reflect the actual conditions. Moments required to partially and fully crack a section were calculated based on the modulus of rupture value for masonry from UBC-85 and the modulus of rupture for grout from ACI 318-83, assuming $f'g = 2300$ psi. This iterative analysis was continued until the moments were compatible with the specified section properties. Once an element was either partially or fully cracked it was not restored.

The seismic time history analysis was performed using methodology and assumptions consistent with project licensing requirements. The evaluations, including modeling techniques, soil-structure interaction parameters and the free field acceleration time history input are consistent with project requirements described in PVNGS Final Safety Analysis Report (FSAR), Section 3.7. The walls at Elevation 74'-0" run in the north-south direction, and, therefore, only east-west time history analyses were performed to maximize out-of-plane moments. At Elevation 100'-0" the walls are oriented in north-south and east-west directions. The maximum span runs east-west and was considered critical. Therefore, only north-south analyses were performed. The effect of vertical inertial and dead loads were included by adding the resulting stresses to the out-of-plane inertial load stresses using absolute sum addition. Because the walls are non-load bearing, non-shear wall elements, stresses due to in-plane lateral inertial loading are negligible. The methodology and execution of the analysis are also in compliance with the applicable requirements of Bechtel Topical Report BC-TOP-4-A, Seismic Analysis of Structures and Equipment for Nuclear Power Plants, as referenced in FSAR Sections 1.6, 3.7, and 3.8. To evaluate the effects of possible variations in soil and wall stiffnesses, a parametric investigation was performed and is described in Section 4.C.

Figure 2 shows a comparison of OBE, 5% damping, horizontal floor response spectra for Control Building Elevation 74'-0". The smoothed and widened curves represent the published horizontal OBE, 5% damping, floor spectra for input scaled to 0.13g and 0.1g. The third curve is the actual or unwidened curve from the present analysis representing east-west horizontal OBE with 5% damping, and scaled to 0.1g for Elevation 74'-0". Five percent damping was chosen because a response curve exists for this value in the project design criteria, and it is the closest to 4% damping, which is used for OBE analysis and design of reinforced masonry. All curves were generated using identical soil-structure interaction parameters (i.e. soil stiffness). A comparison of the widened versus actual curves for the 0.1g input level both reflects and provides a level of confirmation that the structural model, soil-structure interaction parameters, and input time history are identical for both the original and present time history analyses. A comparison of the 0.13g widened curve to the 0.1g actual curve further shows the level of conservatism available between a simplified analysis using the existing published curves and a more precise time history analysis with input scaled to 0.1g.

An acceleration profile for the masonry wall between Elevations 74'-0" and 100'-0" is shown in Figure 3. The profile is for horizontal SSE excitation. The time history analysis representing the critical case of 2/3 of the soil stiffness and 1.0 times the calculated wall stiffness (see Section 4.C) was used to generate this curve. For comparison purposes, the uniform acceleration level used in previous, more conservative, calculations and compatible with a 0.2g free field spectra input level is also shown. The equivalent static acceleration level was obtained from averaging the

accelerations obtained from the floors at Elevations 74´-0" and 100´-0". The comparison shows the level of conservatism between the actual acceleration values obtained from a time history analysis and the assumed equivalent static acceleration level used in previous stress calculations.

C. Effects of Soil and Wall Stiffness Variation

In accordance with the intent of SRP 3.7.2, Sections II.4 and II.9, the effects of modifying the stiffnesses of the soil and masonry walls were also considered. In separate analyses, the soil stiffness parameters were, in one case, increased by a factor of 1.5 and, in another case, decreased by a factor of 1.5. and are used to evaluate the influence of possible variations in soil-structure interaction parameters otherwise considered in the $\pm$ 15% spectral widening. In addition, the effect of increasing the masonry wall stiffness by a factor 1.3 was also investigated. For comparison purposes, the results of these investigations for the time history input scaled to 0.1g, representing operating basis earthquake (OBE) conditions, are summarized in Table 1A. The soil/wall stiffness combinations yielding the highest out-of-plane moments are marked by an (*).

ELEVATION	SOIL STIFFNESS $\frac{E_s \text{ ASSUMED}}{E_s \text{ CALCULATED}}$	WALL STIFFNESS $\frac{E_w \text{ ASSUMED}}{E_w \text{ CALCULATED}}$	MAXIMUM OUT-OF-PLANE MOMENT KIP-FT/FT
74´-0"	0.67	1.0	1.8
	1.0	1.0	1.7
	1.5	1.0	1.6
	1.0	1.3	2.0*
100´-0"	0.67	1.0	0.80
	1.0	1.0	0.73
	1.5	1.0	0.86*
	1.0	1.3	0.72

* Combinations yielding highest moments

Table 1A: Effects of Variation of Soil and Wall Stiffnesses on 0.1g, OBE, Out-of-Plane Wall Moments

From the above table it can be seen, that for both elevations, the maximum difference in the calculated wall moments due to the parameter variations is 20%. For simplicity, it was assumed that the variation in SSE results for the same cases would be similar. Therefore, only two cases which maximize the final out-of-plane moments were chosen to be investigated. The soil/wall stiffness combinations resulting in the highest moments for the time history input scaled to 0.2g, representing SSE conditions, are shown in Table 1B.

ELEVATION	SOIL STIFFNESS $\frac{E_s \text{ ASSUMED}}{E_s \text{ CALCULATED}}$	WALL STIFFNESS $\frac{E_w \text{ ASSUMED}}{E_w \text{ CALCULATED}}$	MAXIMUM OUT-OF-PLANE MOMENT KIP-FT/FT
74′-0"	0.67	1.0	2.7
100′-0"	1.5	1.0	0.91

Table 1B: Soil and Wall Stiffnesses Combinations for 0.2g, SSE, Resulting in Maximum Out-of-Plane Moments

D. Investigations of the Boundary Conditions at Elevation 74′-0"

As explained in Section 4.B, the boundary conditions of the walls were modeled to represent the stiffnesses of the as-built connection details. Due to the unique configuration of the base connection at Elevation 74′-0", utilizing a channel-shaped steel plate and anchor bolts (see PVNGS Drawing 13-C-ZJS-172) it was necessary to perform a more detailed investigation.

To determine the approximate rotational stiffness at Elevation 74′-0", a two-dimensional finite element model representing the restraint conditions at the base was utilized. Membrane elements were used to model the steel plate, grout and masonry blocks. Spring elements were used to model the tension effects of anchor bolts and compression resistance of the basemat. A static moment approximately equal to that calculated from a previous fixed based evaluation was applied at the top of the model.

Tension springs and any element whose stress exceeded the allowable were reduced in stiffness to redistribute the stresses appropriately. After equilibrium was obtained, the rotation at the base was computed and a rotational spring constant value was calculated. Since the resulting spring constant was relatively low, compared to the stiffness of the adjacent wall element, the boundary condition of the base at Elevation 74′-0" was considered to be pinned for all of the time history evaluations discussed in this report.

E. Stress Calculations

The results of the time history analyses in conjunction with the results of some equivalent static analysis were used to calculate masonry, reinforcement and bond stresses in the subject walls. Applicable loading included out-of-plane (horizontal) and in-plane seismic (vertical) inertial loads, and in-plane dead load (weight of wall and attachments).

Stresses were calculated utilizing working stress design methods assuming fully cracked sections. Except where noted, reinforcement locations used in the calculations were based on average as-built "d" distance values obtained from PVNGS Non-Conformance Report CJ-5343. Bond stresses were calculated using the minimum as-built lap lengths.

Tables 2A, 2B, 3A and 3B summarize stress calculation results for out-of-plane bending and vertical in-plane loads for OBE and SSE conditions and the soil and wall stiffness combinations yielding the highest stresses. The tables include allowable stresses based on a minimum f'm = 1500 psi and the provisions of Appendix A to SRP 3.8.4 and ACI 531-79.

The soil and wall stiffness combinations that resulted in the highest calculated stresses were the same as those that provided highest out-of-plane wall moments (see Tables 1A and 1B) in all cases except one. At Elevation 100'-0", for OBE conditions, the highest moment was computed at the wall base for the combination of 1.5 the calculated soil modulus and full wall stiffness. However, the highest stresses occurred at approximate wall mid-height for the combination of 2/3 calculated soil stiffness and full wall stiffness. This occurred because, at wall mid-height, this as-built rebar location based on NCR CJ-5343 was used to calculate the masonry, reinforcement and bond stresses. Reinforcement at the base of the wall is defined by the dowel locations on Quality Class Q Drawing 13-C-ZJS-176. Because Quality Class Q dowel placement is inspected, the bars at the base are in the "as-designed" location. Therefore, although the maximum moment at the base is higher, the computed stresses using the "as-designed" reinforcement location yielded lower stresses than for a lower moment at mid-height.

FIGURE 1: TIME HISTORY ANALYSIS
FINITE ELEMENT MODEL OF CONTROL BUILDING
WITH BLOCK WALL AT EL. 74'-0

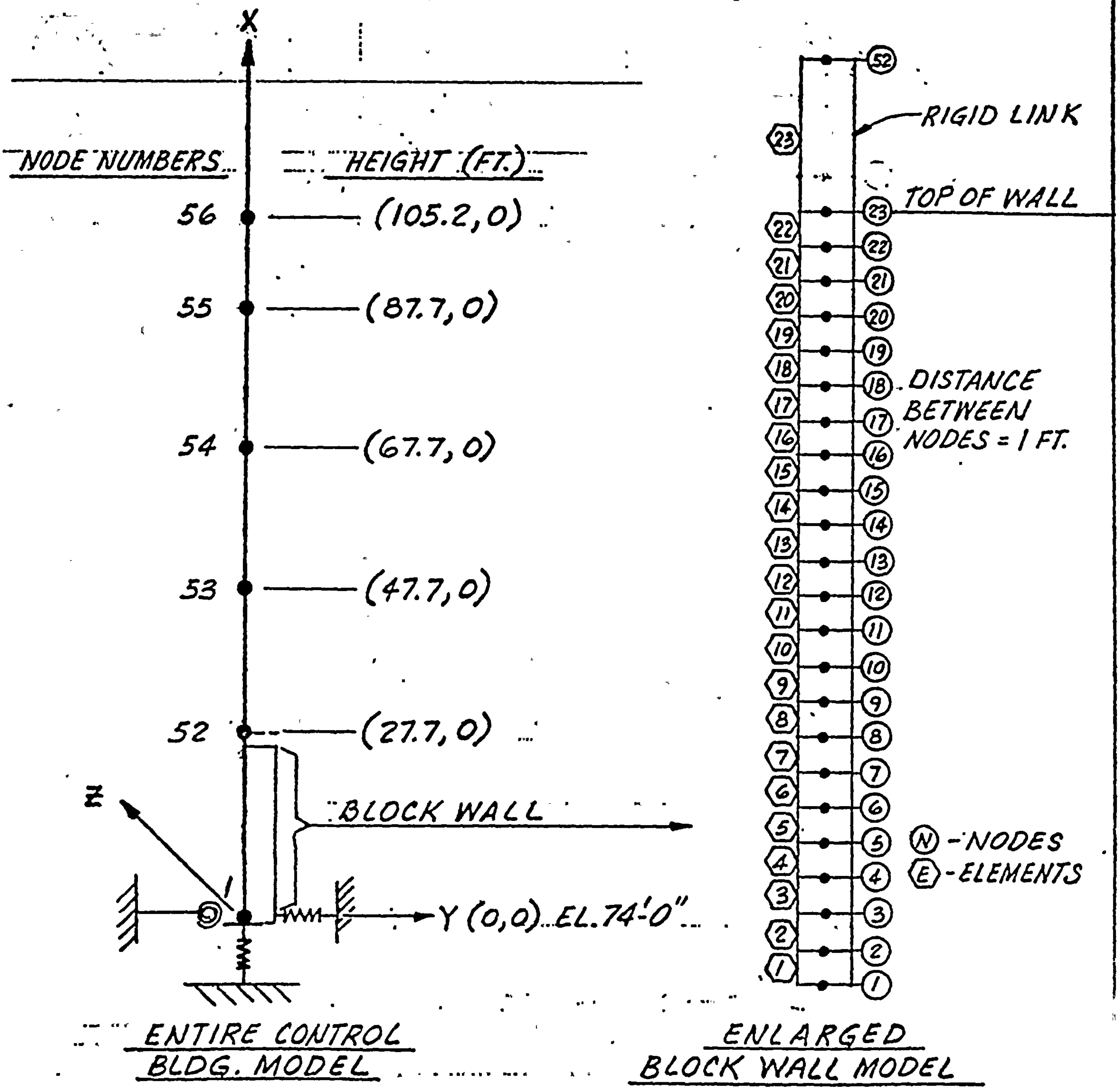


FIGURE 2

COMPARISON OF OBE HORIZONTAL

FLOOR SPECTRA AT ELEV. 74 (E-W) 5% DAMPING

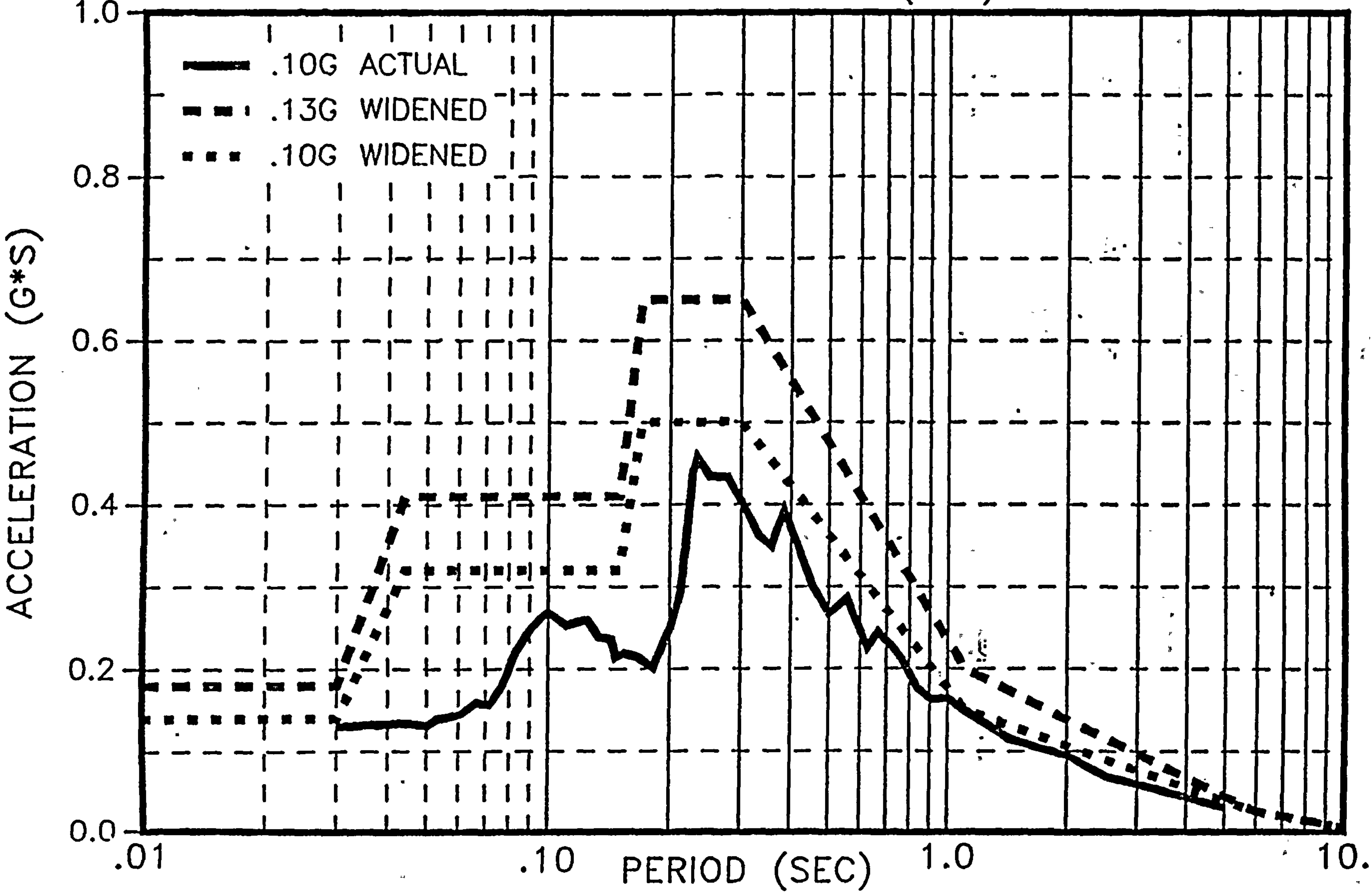
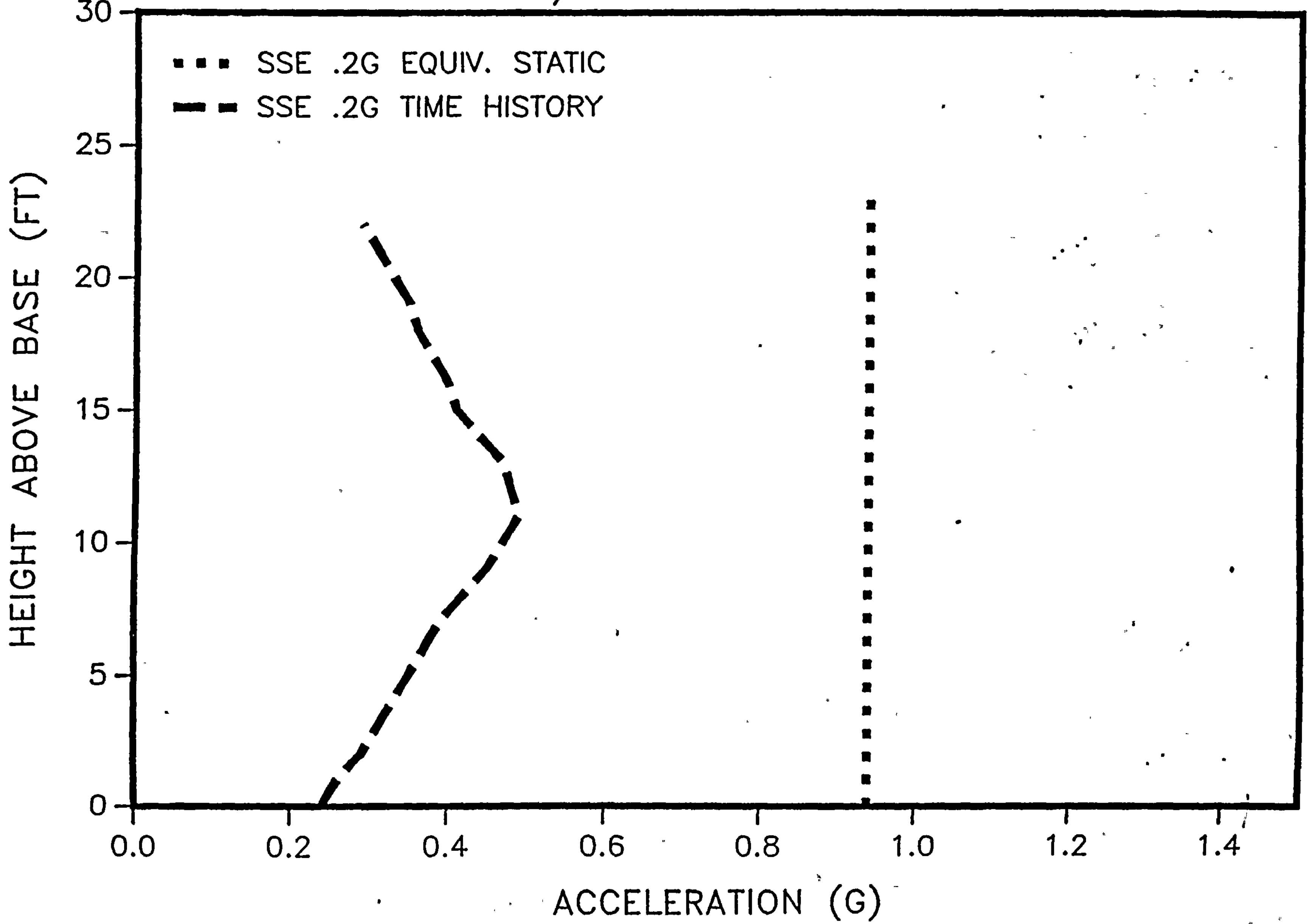


FIGURE 3

SSE HORIZ. ACC. PROFILE
BLOCK WALL AT ELEV. 74
USING 2/3 OF SOIL IMPEDANCES



ELEV.	STRESS[1]	CALCULATED STRESS	ALLOWABLE STRESS (ACI-531-79) FULL	REDUCED	STRENGTH YIELD/ULTIMATE	COMMENTS
74′-0″ [2]	f_m (max)	290	500	333	f'_m = 1500	f'_m = 1500 is the code minimum specified value.
	f_s (max)	9,130	24,000	24,000	F_y = 60,000 F_u = 90,000	
100′-0″ [2]	f_m (max)	260	500	333	f'_m = 1500	f'_m = 1500 is the code minimum specified value.
	f_s (max)	6,680	24,000	24,000	F_y = 60,000 F_u = 90,000	

NOTES: (1) f_m = compressive fiber stress in masonry; f_s = tensile stress in reinforcement.
(2) Calculated stresses are based on average "d" distance as determined from NCR. CJ-5343.

Table 2A: Masonry and Reinforcement Stresses (psi), OBE=0.1g

ELEV.	LOCATION	CALCULATED STRESS (1)	ALLOWABLE STRESS (ACI 531-79)	MIN. AS-BUILT LAP LENGTH
74'-0"	First Course Lap Splice EL. 80'-0"	80	120	16"
	Second Course Lap Splice EL. 84'-8"	70	120	24"
100'-0"	First Course Lap Splice EL. 106'-0"	50	120	16"
	Second Course Lap Splice EL. 110'-8"	40	120	24"

NOTES: (1) Calculated stresses are based on average "d" distance as determined from NCR CJ-5343.

Table 2B: Bond Stresses (psi), OBE = 0.1g

ELEV.	STRESS(1)	CALCULATED STRESS	ALLOWABLE STRESS (ACI-531-79) FULL	REDUCED	STRENGTH YIELD/ULTIMATE	COMMENTS
74′-0″ (2)	f_m (max)	390	1,250	833	f'_m = 1500	f'_m = 1500 is the code minimum specified value.
	f_s (max)	12,540	48,000	48,000	F_y = 60,000 F_u = 90,000	
100′-0″ (2)	f_m (max)	320	1,250	833	f'_m = 1500	f'_m = 1500 is the code minimum specified value.
	f_s (max)	8,370	48,000	48,000	F_y = 60,000 F_u = 90,000	

NOTES: (1) f_m = compressive fiber stress in masonry; f_s = tensile stress in reinforcement.
(2) Calculated stresses are based on average "d" distance as determined from NCR. CJ-5343.

Table 3A: Masonry and Reinforcement Stresses (psi), SSE=0.2g

ELEV.	LOCATION	CALCULATED STRESS (1)	ALLOWABLE STRESS (2) (ACI 531-79)	MINIMUM AS-BUILT LAP LENGTH
74′-0"	First Course Lap Splice EL. 80′-0"	110	180	16"
	Second Course Lap Splice EL. 84′-8"	100	180	24"
100′-0"	First Course Lap Splice EL. 106′-0"	60	180	16"
	Second Course Lap Splice EL. 110′-8"	60	180	24"

NOTES: (1) Calculated stresses are based on average "d" distance as determined from NCR CJ-5343.

(2) Allowable bond stress is in accordance with the value established by ACI 531-79, increased by a factor of 1.5, as recommended by the NRC.

Table 3B: Bond Stresses (psi), SSE = 0.2g

5. DISCUSSION OF RESULTS

Investigation of the variation of soil and wall stiffness parameters yielded different governing combinations for the individual locations and seismic events considered. This variation is explained by the observation that wall moments and stresses are greatest for stiffness combinations that cause the wall frequency to approach either the first or second modal frequency of the Control Building structure. When the wall is close to resonance with the total structure, acceleration amplification results in inertial loads that are higher than those for other soil and wall stiffness combinations.

As shown in Tables 2A, 2B, 3A and 3B, the masonry, reinforcement and bond stresses, for the critical soil and wall stiffness combinations, are within the allowable values accepted by the NRC and defined by Appendix A to SRP 3.8.4 and ACI 531-79. For comparison, full allowable stresses are presented with values that are reduced, per ACI 531-79 guidelines, to compensate for possible variations due to the absence of full in-process inspection.

The general trend is that the stresses at Elevation 100′-0" are lower than those at Elevation 74′-0". This occurs because the wall at Elevation 74′-0" is 22 feet high with a pinned base, compared to a height of 17 feet and a fixed base at Elevation 100′-0". In addition, the mass of the wall, including attachments, at Elevation 74′-0" is greater than that at Elevation 100′-0".

The calculated masonry stresses for the walls at both elevations and under both OBE and SSE loading conditions are below the reduced allowable stress values. The allowable bond stress value for OBE is in accordance with Section 8.7 of the commentary to ACI 531-79. The allowable bond stress value for SSE conditions is the same value as for OBE increased by a factor of 1.5, in accordance with the conclusion of the April 11, 1986 meeting with the NRC and consistent with the intent of Appendix A to SRP 3.8.4. For both OBE and SSE conditions, the calculated bond stresses at each lap splice location are shown to be below the lower (i.e., OBE) allowable stress limit.

Staggered reinforcement lap splices and splice lengths shorter than those specified by the code are acceptable provided bond stress levels are within allowable limits. In addition, the ACI 531-79 Code permits staggering of lap splices up to 8 inches. Masonry unit - grout interface strength is not critical, since, whether in the same or adjacent cells, mechanical locking of the reinforcement is provided by the tapered cell walls.

Lap splice length requirements in various codes are based on the average bond stress concept. Therefore, the actual bond stress was based on available lap length and the calculated reinforcement stress, and was compared to allowable limits.

In all cases, the calculated bond stresses are well below the established limits verifying acceptability of the existing lap splice lengths and locations.

As shown in Figure 3, the acceleration of the wall is highest at the middle and is greater than the zero period acceleration (ZPA) of the wall. However, this acceleration is less than 0.5g for OBE and 1.0g for SSE conditions, which are well below the qualification accelerations of the attached Seismic Category I equipment. Furthermore, analysis has shown that lateral wall deflections, due to OBE and SSE inertial loads will not exceed 0.5 inches. Because displacements of this magnitude are acceptable to the Seismic Category I items attached to the walls, it has been verified that the seismic response of the walls will not impair their function. In addition, the calculated stress levels establish that the walls will remain intact and not damage adjacent safety related equipment.

6. REFERENCES

1. Letter, E. Van Brunt, Jr. (ANPP) to G. Knighton (NRC), April 16, 1986, (ANPP-36153) with enclosure
2. Letter, E. Van Brunt, Jr. (ANPP) to G. Knighton (NRC), April 18, 1986, (ANPP-36301)