Enclosure 1

SUPPLEMENTAL REPORT

ON THE

EVALUATION OF MASONRY WALLS

FOR THE

ARIZONA NUCLEAR POWER PROJECT

PALO VERDE NUCLEAR GENERATING STATION

UNITS 1, 2 AND 3

SEPTEMBER 1986

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JOB NUMBER 10407
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NORWALK, CALIFORNIA

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1. INTRODUCTION

Subsequent to the submittal of the "Report on the Evaluation of Masonry Walls" on June 19, 1986 (Reference 1), ANPP has continued efforts to resolve the outstanding issues regarding the structural integrity of the PVNGS masonry walls. During the NRC/APS/Bechtel meeting of August 20, 1986, the NRC stated that the Control Building walls at Elevation 100'-0" were considered satisfactory and acceptable as is. The remaining NRC concerns pertained only to the Control Building walls at Elevation 74'-0" and included the frequency sensitivity of the walls, the effects of parameter variation on dynamic response, and the methods used to calculate wall stiffness. This report describes the studies and evaluations performed to address these concerns. Results and conclusions are presented to support the position that the walls at Elevation 74'-0" will perform their intended function under postulated seismic conditions.

2. BACKGROUND

On June 19, 1986, ANPP's report on the time history analyses of the PVNGS Control Building masonry walls at Elevations 74'-0" and 100'-0" was submitted to the NRC, (Reference 1). The report described analyses which demonstrated that, under seismic conditions, calculated masonry, reinforcement and bond stresses satisfied the acceptance criteria. The time history evaluation utilized an analytical model with beam elements representing the masonry walls coupled to the original Control Building lumped-mass model used to generate the published floor response spectra. This method of analysis verified that, under the postulated loading conditions, stress levels are within allowable limits accepted by the NRC and defined by Appendix A to SRP 3.8.4 (NUREG 0800, July 1981) (Reference 2) and ACI 531-79 (Reference 3). The report concluded that based on the computed stress levels the masonry walls will perform their intended function under 0.1g OBE and 0.2g SSE conditions and therefore the issue of wall integrity should be considered resolved.

In telephone conversations between the NRC, APS and Bechtel on July 17 and 30, 1986, the method used to calculate the stiffness of the walls, and the frequencies of the walls relative to the corresponding response accelerations were discussed. The NRC expressed concern that the use of a variable (3-stage) moment of inertia in the time history calculations resulted in frequencies higher than those computed using an "effective" moment of inertia as defined in the ACI 318 code. The NRC requested that additional justification be provided to substantiate the use of a variable moment of inertia.

ANPP met with the NRC on August 20, 1986 to discuss details of the time history analysis of the walls, provide justification for the assumptions used in the calculations (including variable moment of inertia), and present the results of an additional, more conservative, confirmatory analysis (Attachment 1). At this meeting the NRC stated that the issues regarding the Elevation 100'-0" walls were considered resolved and that the as-built walls at that location are acceptable. The NRC also stated that future efforts should be devoted to resolving the remaining concerns for the walls at Elevation 74'-0".

The methodology used in the time history analysis was described and shown to be in accordance with regulatory guidance and project licensing requirements. Assumptions used in modeling representative sections of the walls and establishing material properties were described as realistic (based on inspection) and conservative (based on code minimum values). It was explained that the variable moment of inertia (I 3-stage) was based on sound engineering principles and was intended to better approximate the behavior of a fully grouted masonry wall and its state of cracking under seismic load.

It was also stated that the "effective" moment of inertia from the ACI Code is intended for use in estimating deflections of reinforced concrete beams under dead loads and thus, is not directly applicable to the evaluation of PVNGS masonry walls.

Additional analyses had been performed to substantiate the time history analysis and provide confidence in the margin between calculated and allowable stresses. This confirmatory evaluation was performed for the walls at Elevation 74'-0" by response spectrum technique. Resulting stresses were all less than the allowable values except for the OBE masonry stress which exceeded the reduced allowable (33% below the full inspection allowable) value by only 20%. ANPP concluded that the time history analysis complied with regulatory requirements, that the response spectra analysis confirmed the margin of safety, and that the as-built walls have adequate margins of safety under OBE and SSE conditions.

After the explanations and clarifications provided by the presentation, the NRC indicated that the walls were not a serious safety concern, but they did not have sufficient information to accept the walls as-is.

On August 28, 1986, ANPP met with the NRC to further address frequency sensitivity to assumed parameters, moment of inertia calculations, and computed stress margins (Attachment 2). ANPP described how, based on a review of available published literature, a modulus of rupture higher than the minimum value recommended by the UBC (Reference 4) was justified and therefore could be utilized as part of the assessment of wall response. Based on tensile strength (modulus of rupture) tests of fully grouted masonry beams, a cracking moment for PVNGS was calculated. From the modulus of rupture obtained from the test data, it was determined that the walls at Elevation 74'-0" would remain uncracked even under SSE loading conditions.

This report, in conjunction with the June 19, 1986 submittal (Reference 1), documents the masonry wall evaluation, including the recent material presented at the August 20 and 28, 1986 meetings. The purpose is to make additional masonry wall information available to the NRC for their review and determination of the acceptability of the as-built walls at Elevation 74'-0" of the Control Building.

3. SUMMARY AND CONCLUSIONS

Several analyses have been performed to evaluate the adequacy of masonry walls in the PVNGS Control Building under seismic loads. Each analysis has concluded that the walls will withstand the seismic design loads and maintain their functional requirements. Additional evaluations and studies were performed only to resolve the remaining concerns for the walls at Elevation 74'-0", since the NRC had stated that the walls at Elevation 100'-0" are acceptable.

The remaining issues and their resolutions are as follows:

1) Use of a 3-stage moment of inertia:

The time-history analyses (Reference 1) and the confirmatory response spectrum analyses (Attachment 1) both utilized a 3-stage moment of inertia concept to better approximate the behavior of masonry walls under out-of-plane seismic loads. This approximation was considered conservative in that partial cracking (i.e., masonry faceshell only) was predicted under low seismic loads. An approach similar to 3-stage moment of inertia for approximating the effective moment of inertia for analytical purposes is discussed by R. G. Drysdale and A. A. Hamid in Reference 7.

Additional research (Attachment 2) concluded that the behavior of a masonry wall can be represented by a single cracking moment for the composite section, utilizing a realistic, yet conservative, modulus of rupture based on test results of fully grouted masonry walls. When the walls at Elevation 74'-0" were evaluated with this concept, it was shown that the walls will remain uncracked under seismic loads. Maximum calculated moments were similar to those values determined in the earlier time history and confirmatory response spectrum analyses.

2) The sensitivity of wall response to the calculated wall frequency:

The frequency of the uncracked walls at Elevation 74'-0" is about 7 Hz, based on a conservative modulus of elasticity. Referring to Figure 1, the wall accelerations will not change between 5 Hz and 7 Hz. Therefore, this results in a substantial (approximately 65%) margin in calculated stiffness (Attachment 2) before the frequency of the walls decreases to the point where calculated moments would start increasing.

3) Effect of variabilities and uncertainties on the calculated safety margin:

Uncertainties regarding analysis parameters (e.g., modulus of elasticity, modulus of rupture, and seismic input) involved were also studied and conservative assumptions were made. Uncertainties associated with modulus of elasticity were accounted for by using a factor of 0.75 times the expected value, since a lower value governs the design for PVNGS. For the modulus of rupture, a value equal to the expected mean minus 1.343 times the standard deviation based on the ACI code was used. Uncertainties in seismic input were incorporarred by performing confirmatory analysis using widened and smoothed floor response spectra.

Using the maximum moments, stresses in the walls were calculated disregarding the tensile strength contribution of the masonry. The masonry, rebar, and bond stresses from the time history analyses were all shown to be within SRP and ACI Code allowables. For the confirmatory response spectrum analyses, all stresses were within allowables except for the OBE masonry stress which exceeded the reduced allowables by only 20%.

Based on these evaluations, including a review of applicable literature, it is concluded that; 1) the methodology used is conservative and accurately predicts the masonry wall response, 2) the walls have capability to withstand seismic loads and will meet their functional requirements, and 3) the existing factors of safety are at least three for an OBE and two for an SSE.

Therefore, the as-built walls meet regulatory requirements, and no repairs or modifications are required.

4. RESOLUTION OF ISSUES

A. General

Since the submittal of the report that described the wall time history analyses (Reference 1), several issues have remained unresolved regarding the evaluation of the PVNGS masonry walls. These issues are; 1) the methods used to calculate the stiffness of the wall and the effect on the computed frequencies, 2) the dynamic response of the wall and its proximity to the amplified region of the response spectra curve, and 3) the effects of uncertainties on the available safety margin. The following sections address these issues and confirm the validity of the existing evaluations that establish the acceptability of the as-built walls.

B. Wall Stiffness, Frequency and Response

Wall stiffnesses and therefore frequencies are dependent on a combination of wall parameters. The geometric configuration of the wall such as the height, thickness and boundary conditions are defined by the as-built structure itself. Material properties such as modulus of elasticity, E_m, and modulus of rupture, f_r, have been quantified based on the data available from independent investigations and the ACI/UBC codes. The section properties of the walls are variable and are dependent on the cracking moment, M_{cr} (which is a function of f_r), and the maximum moment in the wall, M_a (which is dependent on the response spectra). Wall parameters reflecting as-built conditions were used in calculating the stiffness and frequency of the PVNGS masonry walls, while utilizing conservative code values for the strength evaluation of the walls. The determination of the modulus of elasticity is explained in Appendix A of this report.

The modulus of rupture value presented to the NRC in the August 28, 1986 meeting (Attachment 2) was 169 psi. In response to NRC's concern, additional conservatism, in accordance with the ACI 318 Code (Reference 5), has been incorporated. Using the ACI methodology, the modulus of rupture value was recalculated to be 157 psi (Based on Reference 7 test data as supported by References 8 and 9). This subject is further discussed in section 4.C.2 below and in Appendix B.

As the actual bending moment in the wall increases the corresponding moment of inertia decreases from that of an uncracked section, I_{gross}, to that of a fully cracked section, $I_{cracked}$, as a function of the cracking moment divided by the actual moment (M_{cr}/M_a). Utilizing the value of 157 psi for f_r, M_{cr} is calculated to be approximately 3.5 k-ft/ft. The maximum calculated moment from the SSE time history analysis at El. 74'-0" (Reference 1 and Attachment 1) was 2.9 k-ft/ft (based on a 3-stage moment of inertia) and 3.5 k-ft/ft for the subsequent response spectra analysis (Section 5.A of this report). Therefore, it is concluded that the walls will remain uncracked. Refer to Appendix C for additional discussion regarding calculation of the moment of inertia.

The frequency of the uncracked wall is about 7 Hz. Initial manual calculations, time-history analysis (Reference 1), and confirmatory response spectrum analyses (Attachment 1) resulted in lower frequencies because sections were calculated to be at least partially cracked based on the conservative 3-stage moment of inertia methodology. Analyses using a modulus of rupture based on test results preclude cracking and therefore the uncracked wall frequency of about 7 Hz is the most realistic value, although the frequencies may vary slightly during a seismic event.

C. Effects of Uncertainties

It is recognized that uncertainties associated with the construction of masonry walls affect parameters involved in analyses and therefore can affect the results. In the evaluation of PVNGS walls these uncertainties have been taken into account as follows:

1. Modulus of Elasticity: Masonry modulus of elasticity varies depending on materials and workmanship utilized in the construction. For PVNGS the lower value of E_m is the controlling factor because of the shape of the floor response spectra (FRS) and the fundamental frequency of the walls. For a discussion of the development of E_m used in stress and stiffness calculations, refer to Appendix A.

2. Modulus of Rupture: The uncertainties regarding f_r were accounted for by utilizing a value which is equal to expected mean minus 1.343 times the standard deviation based on ACI 318 provisions for concrete testing (Reference 5). This procedure assures that there is a probability of only one in 100 that the average of any three consecutive modulus of rupture tests will be less than the calculated value.

3. Floor Response Spectra: The artificial time-history utilized in developing the floor response spectra (FRS) envelopes the ground design spectra. Since it is impractical to develop a time-history record that matches the design spectra at all frequencies, the enveloping process introduces conservatism. After the response is calculated, additional conservatism is provided through the widening and smoothing process of the spectra. In addition, although the structure is embedded 30 feet, approximately 25% of its height, reduction of ground motion with depth (deconvolution) was not considered, thus introducing further conservatism into the calculated spectra.

D. Seismic Input

In the time-history analysis (Reference 1), the input motion for the walls is the in-structure response motion at the coupled model nodes located at the top and bottom of the wall. For the confirmatory analysis in-structure time-history records at these locations were utilized to obtain wall-specific FRS (with respect to both location and direction) at Elevation 74'-0".

Development of the wall-specific FRS is discussed in Appendix D. The new FRS are generated using procedures consistent with the project licensing commitments and provisions of the SRP (Reference 2) and RG-1.122 (Reference 6).

Figure 1 shows the 7 percent damping, 0.2g SSE FRS which is applicable to the Elevation 74'-0" wall. Also shown on this figure is the published FRS, scaled to 0.2g and averaged between elevations 74'-0" and 100'-0." In the development of the original published FRS, additional conservatism was incorporated through peak-widening at all spectral frequencies and a generous smoothing process. Comparison of the two curves illustrates the conservatism in the published spectra with respect to the wall specific spectra.

5. DISCUSSION OF RESULTS

A. General

In order to validate the time history analysis (Reference 1), a confirmatory analysis was performed utilizing the response spectrum techniques (Attachment 1). The floor response spectra (FRS) were developed as discussed in Appendix D. With these FRS as input several response spectrum analyses were performed using the 3-stage moment of inertia approach.

Both in the time-history analysis and confirmatory analysis using 3-stage moment of inertia, the analysis indicated that the section would be partially cracked. This was due to the fact that the modulus of rupture value utilized for initial cracking of the faceshell of masonry was assumed to be 97 psi versus 157 psi for the composite modulus of rupture derived from tests of fully grouted masonry specimens (References 7, 8 and 9).

Additional research has shown that behavior of a fully grouted masonry wall can be described by a single cracking moment (References 7, 8 and 9). Use of this realistic yet conservative cracking moment (Appendix C) indicates that the sections will remain uncracked. Therefore, additional response spectrum analyses have been performed using the effective moment of inertia approach in which I_{eff} is equal to the gross moment of inertia.

Resulting moments from all confirmatory analyses, corresponding stresses, and available margins are discussed below.

B. Calculated Moments:

Previous submittals have shown that calculated seismic moments will vary depending on the assumptions made regarding the behavior of the wall and floor response spectra.

Based on the test results for modulus of rupture (Appendix B) and the wall-specific FRS (Appendix D), Elevation 74 ft. walls will remain uncracked under seismic loads. Based on the confirmatory analysis, the calculated maximum moment at mid-height of the wall is 3.5 k-ft/ft (SSE) based on the enveloped FRS. Since this conservative moment value does not exceed the calculated cracking moment for the wall, the effective moment of inertia using the ACI 318 Code equation is the same as the uncracked, or gross, moment of inertia.

Even though the above value of 3.5 k-ft/ft is the maximum calculated moment for SSE, a higher moment value of 4.5 k-ft/ft was reported in Attachment 1. This value was based on a more conservative, widened, floor response spectra (Appendix D) and 3-stage I analysis. However when this analysis was repeated with the modulus of rupture based on test data (157 psi) and using I_{eff}, the resulting maximum moment was less than 3.5 k-ft/ft.

The higher moment value would imply some cracking of the wall and subsequent reduction in wall frequency from 7 to 6 Hz. This change in frequency does not, however, shift the wall response into the amplified region of the FRS. The higher moment values reported in Attachment 1, (4.5 k-ft/ft for SSE and 2.8 k-ft/ft for OBE) were utilized in the stress calculations discussed in the next section to provide additional margin in the analysis.

C. Calculated Stresses:

Once the maximum moments are determined, the corresponding stresses were calculated using standard principles of engineering mechanics.

The masonry, rebar, and bond stresses were calculated using the maximum moments of 4.5 k-ft/ft and 2.8 k-ft/ft for SSE and OBE, respectively as discussed above. Bond stresses were calculated using the minimum available lap length. The results are shown in Table 1. The allowable values are also listed in the same table.

The calculated stresses are all less than the allowable values except for the OBE masonry stress which exceeds the reduced allowable value by only 20%. Since the corresponding stress under the SSE condition is less than the allowable and since the exceedence of the OBE allowable will not impact serviceability of the structure (corresponding maximum displacement is less than 0.25 inch), all calculated stresses are considered acceptable.

D. Available Margins:

In the working stress design method design margins are provided mainly by the application of factors of safety to obtain allowable stresses. For masonry structures, the factor of safety utilized is about 3 under service loading conditions, including the OBE loads. The reduced masonry flexural allowable increases the available margin by a factor of 1.5 to account for the uncertainties in the construction of an uninspected structure. These higher margins are provided to account for the greater variability experienced with masonry structures. The allowable values shown in Table 1 reflect the traditional factors of safety for the OBE conditions. The allowables for SSE condition have been obtained from the criteria described in Appendix A to SRP 3.8.4 (Reference 2). Thus, the higher margins discussed above are inherent in the allowable values shown in Table 1.

Additional margins exist in the calculated stresses due to the methods utilized to account for the uncertainties in different parameters, as discussed in the previous section. These include:

1. Use of a conservative value for modulus of rupture and thus cracking moment which affects the moment of inertia and frequency.

2. Use of lower masonry compressive strength which increases the calculated response and reduces the allowable stresses.

3. Use of conservative floor response spectra which also increases the calculated response.

These margins in the calculated stresses, together with the margins inherent in the allowable stresses make up the overall safety margin of the walls. Review of the maximum stress values listed in Table 1 and consideration of the above points lead to the conclusion that the margin of safety for these walls is at least 3 for OBE loads and 2 for SSE loads. These values are higher than the margins normally utilized for steel or reinforced concrete structures and are consistent with the design margin of safety associated with masonry walls.

In summary, even though the walls have staggered splices and reduced lap lengths, the available margins assure that the PVNGS masonry walls will perform their intended safety function under PVNGS seismic conditions.

6. REFERENCES

1. "Report on the Evaluation of Masonry Walls" attached to the ANPP letter to the NRC, dated June 19, 1986 (ANPP-37062).

2. Standard Review Plan (SRP) NUREG-0800, July 1981.

3. Building Code Requirements for Concrete Masonry Structures (ACI 531-79), American Concrete Institute.

4. Uniform Building Code (UBC), International Conference of Building Officials, 1985 Edition.

5. Building Code Requirements for Reinforced Concrete (ACI 318), American Concrete Institute.

6. Regulatory Guide 1.122, Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components, U.S. Nuclear Regulatory Commission, Revision 1, February 1978.

7. "Effect of Grouting on the Flexural Tensile Strength of Concrete Block Masonry," R. G. Drysdale and A. A. Hamid, The Masonry Society (TMS)/Journal July-December 1984.

8. "Factors Influencing Deflections in Grouted Hollow Unit Concrete Masonry Walls", Atkinson-Noland & Associates, September 1986.

9. "Test Report on Slender Walls," ACI-SEASC Task Committee on Slender Walls, February 1980 - September 1982.

10. "Dynamic Stiffness of Concrete Beams," K. D. Johns and M. D. Belander, Title No. 78-18, ACI Journal/May-June 1981.

11. "Behavior of Concrete Block Masonry Under Axial Compression," R. G. Drysdale and A. A. Hamid, Title No. 76-32, ACI Journal/June 1979.

TABLE 1

Confirmatory Response Spectrum Analysis Maximum Stresses (psi)

(Elevation 74'-0")

ITEM	SSE (0.2g)		OBE (0.1g)	
	CALCULATED[a]	ALLOWABLE	CALCULATED[a]	ALLOWABLE
MASONRY	650	833	400[b]	333
REBAR	21,000	48,000	12,800	24,000
BOND	180	180	110	120

(a) Stresses are based on conservatively calculated moments of 4.5 k-ft/ft for SSE and 2.8 k-ft/ft for OBE.

(b) Exceedence of the reduced OBE allowable stress by 20% is acceptable since the corresponding SSE value is within the allowable and OBE deflections are small (less than 0.25 inch).

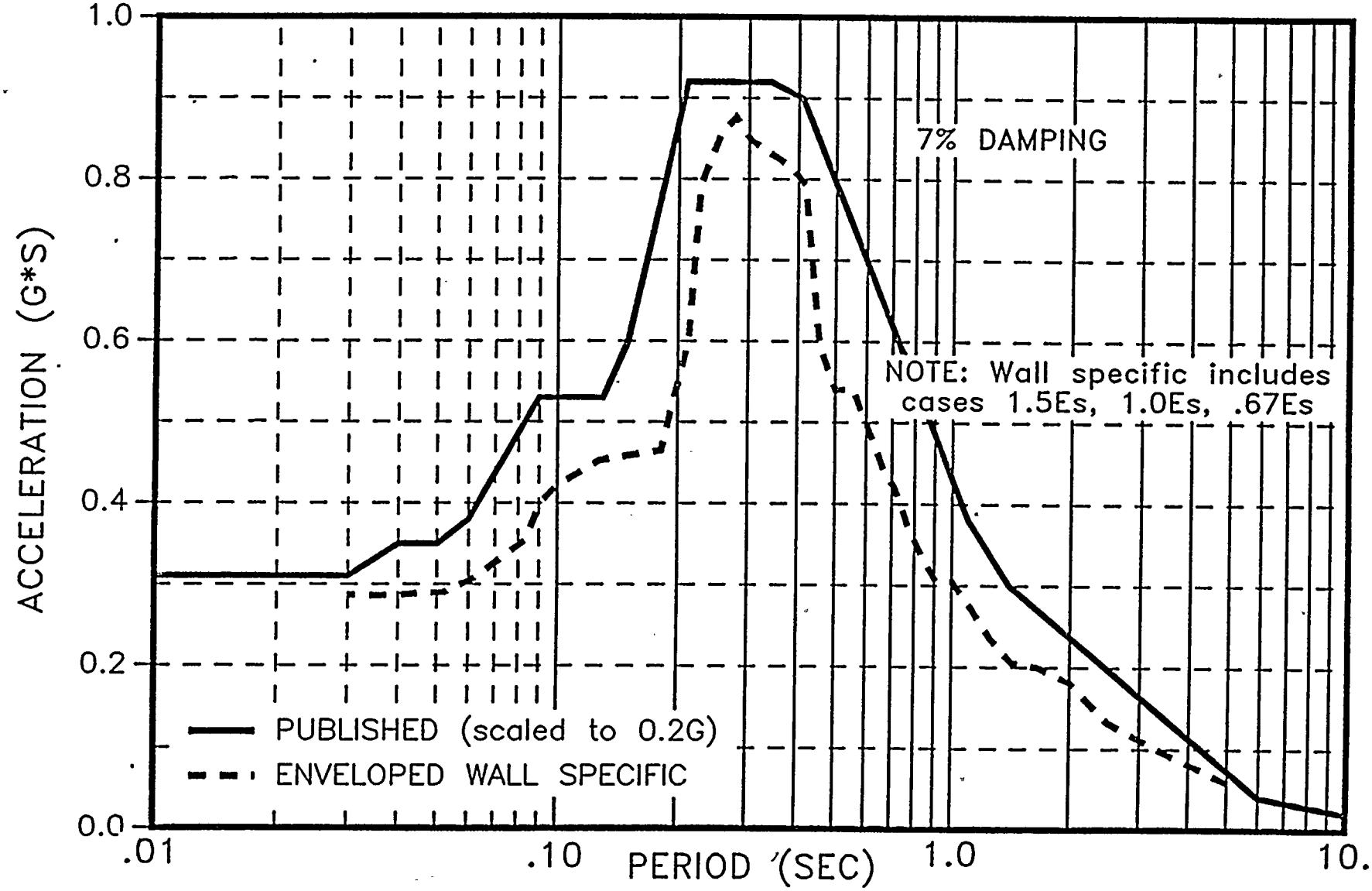


FIGURE 1: Comparison of Published & Enveloped Wall Specific SSE Spectra (Averaged between elevations 74' & 100')

APPENDIX A

DETERMINATION OF MODULUS OF ELASTICITY

In the evaluation of masonry walls, the modulus of elasticity is used to establish both wall stiffness and stresses. To assure an evaluation representative of the PVNGS walls, a E_m value based on expected compressive strength was utilized in stiffness/frequency calculations with due consideration of uncertainties.

Modulus of elasticity of grouted masonry walls, E_m depends on the type of masonry unit, mortar, and grout used. Traditionally, it has been expressed as a function of the masonry compressive strength, f'_m, as opposed to the concrete modulus of elasticity which is proportional to the square root of the concrete compressive strength, f'_c. The ACI 531 Code (Reference 3) identifies E_m as $1000xf'_m$. In the evaluation of PVNGS masonry wall stresses an E_m value of $1.5x10^6$ psi was utilized as a conservative value. As discussed in the August 20, 1986 meeting for PVNGS, because of the shape of the FRS and the fundamental frequency of the masonry walls, the lower value of E_m governs the design and therefore parametric studies using the upper value were not performed.

Review of literature (Reference 11) indicates that the expected average compressive strength of the in-situ walls will be at least 2000 psi. Using the code equation, the resulting modulus of elasticity used in stiffness calculations is $2.0x10^6$ psi. To address uncertainties, other nuclear plants have used lower and upper values of 0.8 E_m and 1.2 E_m respectively. To assure adequate margins for PVNGS, the lower value utilized was $0.75xE_m$ where E_m is the minimum expected average value.

Review of Reference 9 indicates that the modulus of elasticity of prisms that were cut from wall test specimens ranged between $1.5x10^6$ and $2.2x10^6$ psi. These values were for the wall thicknesses ranging from 6 to 10 inches. The PVNGS Control Building walls are 12-inch thick and thus are expected to yield even higher values since the effect of grout will be more dominant. It should be noted that the modulus of elasticity of grout alone, following the ACI 318 Code (Reference 5), is about $2.9x10^6$ psi.

Considering the above data, the modulus of elasticity used for PVNGS masonry walls is a conservative lower value for stiffness and thus for frequency calculations.

APPENDIX B

DETERMINATION OF MODULUS OF RUPTURE

The modulus of rupture for fully grouted masonry, f_r, depends on f_m', similar to concrete. The UBC 1985 Code (Reference 4) specifies a value of 2.5 f_m' which is equal to 97 psi, based on the UBC code value for f_m'. The coefficient of 2.5 in the UBC equation is a lower bound value derived solely from three 6-inch thick concrete masonry wall tests (Reference 8). This value is significantly lower than the actual test values for 8-inch and 10-inch walls. Therefore, it is not appropriate to use a value of 2.5 in stiffness/ frequency calculations of a 12-inch thick wall which will exhibit a greater modulus of rupture value (U 4.5-5.0).

A literature survey was performed to determine a more realistic f_r value for PVNGS for calculating the stiffness and thus frequency of the walls. Reference 8 concludes that the f_r value can be expressed as K f_m'. Based on the test results (Reference 9), it was shown that the K value for fully grouted concrete masonry walls increases with the thickness. Thus, the K value for a fully grouted 12-inch thick masonry wall was extrapolated to be about 5.3 (Reference 8). Considering the minimum PVNGS expected value of the compressive strength of 2000 psi, the resulting f_r for PVNGS is 237 psi. As shown in Section 5 of this report, maximum calculated tensile stress for the wall is 157 psi (i.e., minimum wall strength required to preclude cracking) which indicates that the walls remain uncracked. The data from Reference 8 is plotted in Figure B1, with the ordinate in psi for convenience. The projected PVNGS wall modulus of rupture and the minimum required strength are also indicated on that figure.

Reference 7 indicates an f_r value of 203 psi for 8-inch thick fully grouted masonry walls with similar properties to those of PVNGS walls. This value was obtained from seven tests, with a standard deviation of 34 psi. Table B1 shows a comparison of the parameters between the beam tests and PVNGS walls. Based on this comparison, the modulus of rupture value obtained from the beam tests are applicable to PVNGS.

Considering the lower value of 203 psi and the standard deviation associated with it, a conservative modulus of rupture can be determined following the methodology of the ACI 318 Code (Reference 5). The calculated value, with the probability of only 1 in 100 that the average of any three consecutive modulus of rupture tests will be less than that, is given by

$$f_r = f_r - 1.343$$

Based on the above equation and the test data from Reference 7, a modulus of rupture value for PVNGS of 157 psi was determined.

TABLE B1

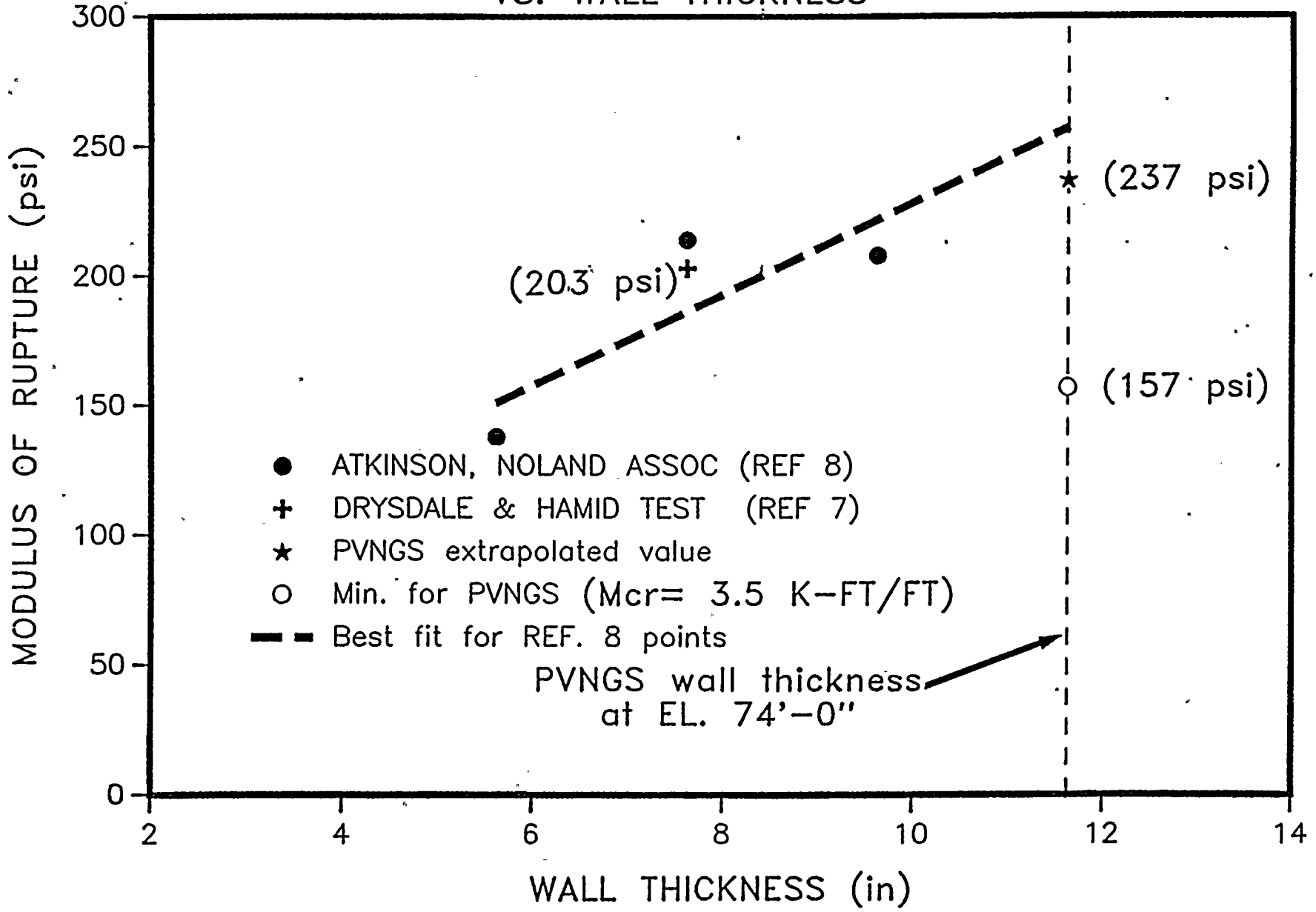
Comparison of Tested Specimens(1)

and PVNGS Masonry Walls

CHARACTERISTICS	TEST	PVNGS	REMARKS
GROUT STRENGTH	3060 psi	2600+ psi	DIFFERENCE IN GROUT STRENGTH DOES NOT SIGNIFICANTLY AFFECT f_r (Reference 7).
MORTAR STRENGTH	2520 psi	2200-2800 psi	PVNGS VALUES ARE SIMILAR TO TEST DATA.
MASONRY STRENGTH	3640 psi	2000 psi	DIFFERENCE IN MASONRY STRENGTH DOES NOT AFFECT f_r SINCE FAILURE OCCURS AT BED JOINTS.
REINFORCEMENT	NONE	REINFORCED	f_r IS INDEPENDENT OF REINFORCEMENT.
BLOCK SIZE	8"x8"x16"	4"x12"x16"	LOWER HEIGHT OF PVNGS UNITS REDUCES STRESS CONCENTRATIONS.
MEAN MODULUS OF RUPTURE ($\bar{f}_r$)	203 psi	203 psi(2)	DUE TO SIMILAR PROPERTIES.
STANDARD DEVIATION	34 psi	34 psi(2)	TO ACCOUNT FOR UNCERTAINTIES.
MODULUS OF RUPTURE ($\bar{f}_r - 1.343\sigma$)	157 psi	157 psi(2)	REALISTIC VALUE FOR PVNGS FREQUENCY ANALYSIS.

(1) From Reference 7
(2) Test data considered applicable for PVNGS use

FIGURE B1: MODULUS OF RUPTURE FROM WALL TESTS VS. WALL THICKNESS



APPENDIX C

DETERMINATION OF CRACKING MOMENT AND EFFECTIVE MOMENT OF INERTIA

The ACI 318 Code (Reference 5) gives an effective moment of inertia equation which is utilized to determine the long term deflections under dead loads. The equation is:

$$I_{eff} = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g$$

where M_{cr}=cracking moment, M_a=applied moment and I_g and I_{cr} are the gross and cracked moment of inertia, respectively. As can be seen, this equation is sensitive to the value of M_{cr}. Since the cracking moment is equal to S times f_r, where S is the uncracked or gross section modulus, substitution of low modulus of rupture value will result in low cracking moment and the effective moment of inertia will decrease rapidly as the applied moment increases beyond the cracking moment.

For PVNGS, the 3-stage moment of inertia methodology was utilized to more accurately predict the response of the masonry wall, because the ACI equation is intended for determining deflections for reinforced concrete elements under long term (dead load) loadings. Since the masonry walls are unstressed prior to a seismic event, the ACI equation does not predict actual masonry wall response due to seismic inertial forces.

Furthermore, Reference 10 shows that, based on beam tests, use of the ACI equation is inaccurate and gives a significantly lower moment of inertia than actually exists under dynamic loads. Reference 10 recommends an effective moment of inertia equation for dynamic analysis in which cracking moment/applied moment ratios are linear as opposed to the cubic ratios used in the ACI code equation. However the linear approach was not used in these analyses.

Due to the uncertainties involved in determining the effective moment of inertia, the time-history analysis reported in the June 19, 1986 submittal utilized a 3-stage moment of inertia to represent the actual cracking condition of the gross section. This method was intended to approximate the actual moment of inertia variation, taking into account the higher modulus of rupture value for the grout. It is standard practice to take into account the state of cracking in a seismic analysis of masonry walls. A similar approach to approximate the effective moment of inertia for analytical purposes is discussed by R. G. Drysdale and A. A. Hamid in Reference 7.

The moment that would cause the initial cracking was conservatively based on the code value of $2.5\sqrt{f_m'}$ and the moment that would cause full cracking was based on modulus of rupture of grout, $7.5\sqrt{f_g'}$. These moduli of rupture resulted in partial and full cracking moments of 2.2 k-ft/ft. and 5.4 k-ft/ft., respectively.

When the values of modulus of rupture discussed in Appendix B are utilized to determine the cracking moment, the results are:

Expected cracking moment: 4.6 k-ft/ft (based on 203 psi)

Minimum cracking moment: 3.5 k-ft/ft (based on 157 psi)

Since the calculated seismic moments range from 2.9 k-ft/ft for the time history analysis to 3.5 k-ft/ft for the response spectrum analysis (the latter value is the same for 3-stage I or uncracked I, considering the enveloped spectra), it may be concluded that the walls will remain uncracked under seismic loads.

It is recognized that the walls will not be in an ideal uncracked condition in all areas since minor cracking due to shrinkage or thermal effects will occur. Therefore, the effective moment of inertia may be slightly less than the uncracked (gross) moment of inertia. However, these pre-existing cracks will not affect the overall wall response because the resulting reduction in frequency due to minor changes in stiffness will be negligible.

APPENDIX D

DEVELOPMENT OF FLOOR RESPONSE SPECTRA

In order to substantiate the time-history analysis (Reference 1), a response spectrum analysis was performed. The in-structure time history records at the top and bottom of the walls were obtained from the overall structure time history analysis. Floor response spectra (FRS) were then developed using these time-history records.

Two approaches were taken in generating the FRS curves. The two approaches differ in the method utilized to account for variations in soil and structure parameters. In the first approach, the mean soil impedances were varied by a factor of 1.5 each way, thus performing the overall structure time-history analysis three times (with 0.67, 1.0, and 1.5 times E_s). The spectra were calculated from each time history analysis and then enveloped. This results in shifting soil-structure interaction modes by over 20% and is an acceptable analysis method per the SRP (Reference 2). The spectra thus obtained are called enveloped spectra.

In the second method, spectra were generated using the mean soil values. Then the response at each spectral frequency was widened by 15%, (i.e., broadening the spectral accelerations by $\pm$ 15%). These spectra are termed the broadened or the widened spectra. The widened spectra are generally more conservative since the curves are broadened at each spectral frequency, a calculational convenience, as opposed to broadening at only structural frequencies in accordance with the SRP (Reference 2) and RG-1.122 (Reference 6).

In both methods, the spectra at the top and bottom of the wall were averaged to obtain the wall-specific FRS. This is a reasonable approach since both the shape of the spectra and magnitude of the spectral accelerations are similar at the two elevations.

As reported in Attachment 1, the conservative widened spectra resulted in a maximum calculated moment of 4.5 k-ft/ft. However, when this analysis with the widened spectra was repeated with the modulus of rupture based on test data (157 psi) and using the effective moment of inertia approach, the resulting maximum moment was less than 3.5 k-ft/ft.

In the development of the enveloped spectra, the peak-widening is accomplished at the soil-structure interaction frequencies through the variation of soil impedances. Considering the fact that response is dominated by the soil-structure interaction modes, use of the enveloped spectra is justified.

To include additional conservatism, the maximum calculated moments (based on the widened response spectra and 3-stage I) were used in stress calculations.

ATTACHMENT 1

Presentation Slides from NRC/APS/Bechtel Meeting

of

August 20, 1986

(1)

PRESENTATION ON MASONRY WALLS
FOR THE
PALO VERDE NUCLEAR GENERATING STATION
UNITS 1, 2, AND 3

AUGUST 20 1986

AUGUST 20, 1986

AGENDA

I. CONCERN
II. DESCRIPTION OF EVALUATION
III. JUSTIFICATION OF ASSUMPTIONS
IV. RESULTS
V. CONFIRMATORY ANALYSIS
VI. MARGINS OF SAFETY
VII. CONCLUSIONS
VIII. RESPONSE TO NRC QUESTIONS

AUGUST 20, 1986

II. DESCRIPTION OF EVALUATION(1)

A. TIME-HISTORY (T-H) ANALYSIS ASSUMPTIONS:

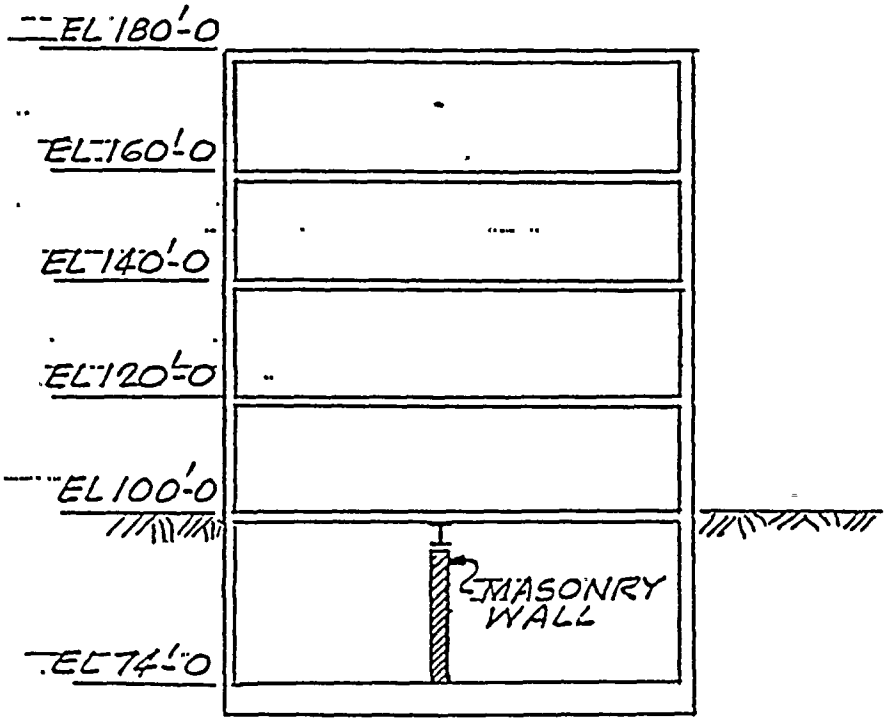
- COUPLED ANALYSIS WITH SOIL STRUCTURE INTERACTION (SSI)
- SINGLE T-H RECORD
- SINGLE DIRECTION TIME-HISTORY
- STRIP IDEALIZATION OF WALL

B. REALISTIC MATERIAL PROPERTIES AND METHODOLOGY

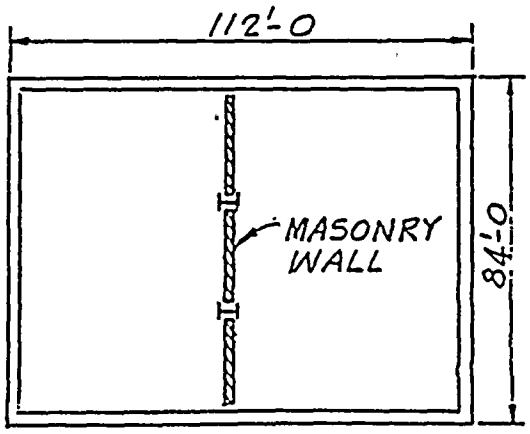
- WALL MODULUS OF ELASTICITY FOR STIFFNESS
- AVERAGE REBAR LOCATION ("d")
- GROUT STRENGTH BASED ON TEST
- 3-STAGE MOMENT OF INTERTIA - UNCRACKED, PARTIALLY CRACKED, AND FULLY CRACKED

(1) JUNE 19, 1986 SUBMITTAL

CONTROL BUILDING



ELEVATION LOOKING NORTH



PLAN @ ELEV 74'-0

AUGUST 20, 1986

III. JUSTIFICATION OF ASSUMPTIONS

A. TIME-HISTORY ANALYSIS

- COUPLED ANALYSIS WITH SSI
  - PER FSAR
  - PER NRC APPROVED BC-TOP-4A
  - ACCEPTED INDUSTRY PRACTICE
- SINGLE TIME-HISTORY RECORD
  - ENVELOPES RG-1.60 SPECTRA
  - PER NRC APPROVED BC-TOP-4A
  - VALID FOR LINEAR-ELASTIC ANALYSIS
- SINGLE DIRECTION TIME-HISTORY (REALISTIC APPLICATION)
  - TORSIONAL EFFECTS ARE MINIMAL
  - OUT-OF-PLANE (FLEXURAL) RESPONSE DOMINATES
- STRIP IDEALIZATION OF WALL
  - ONE WAY ACTION - CONSERVATIVE APPLICATION
  - RUNNING BOND CONFIGURATION ENSURES UNIFORM RESPONSE
  - PENETRATIONS AND OPENINGS ARE LOCALLY REINFORCED

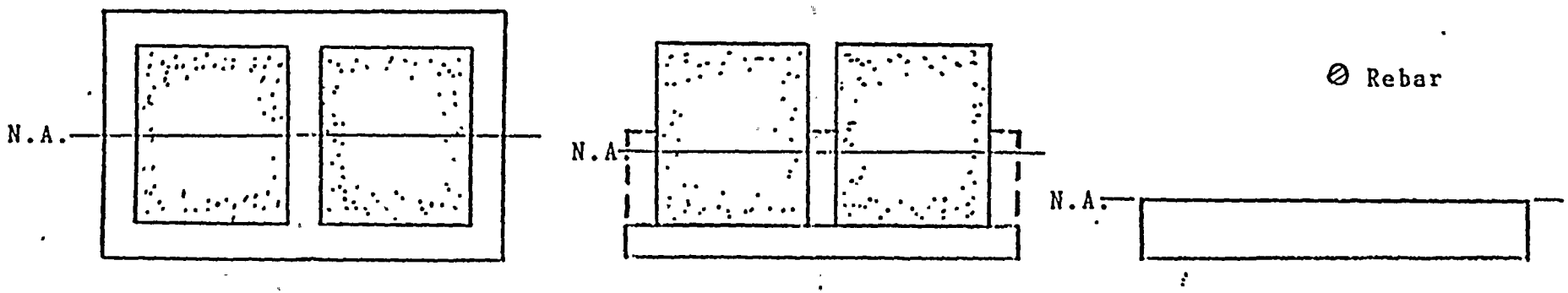
AUGUST 20, 1986

III. JUSTIFICATION OF ASSUMPTIONS (CONTINUED)

B. REALISTIC MATERIAL PROPERTIES AND METHODOLOGY

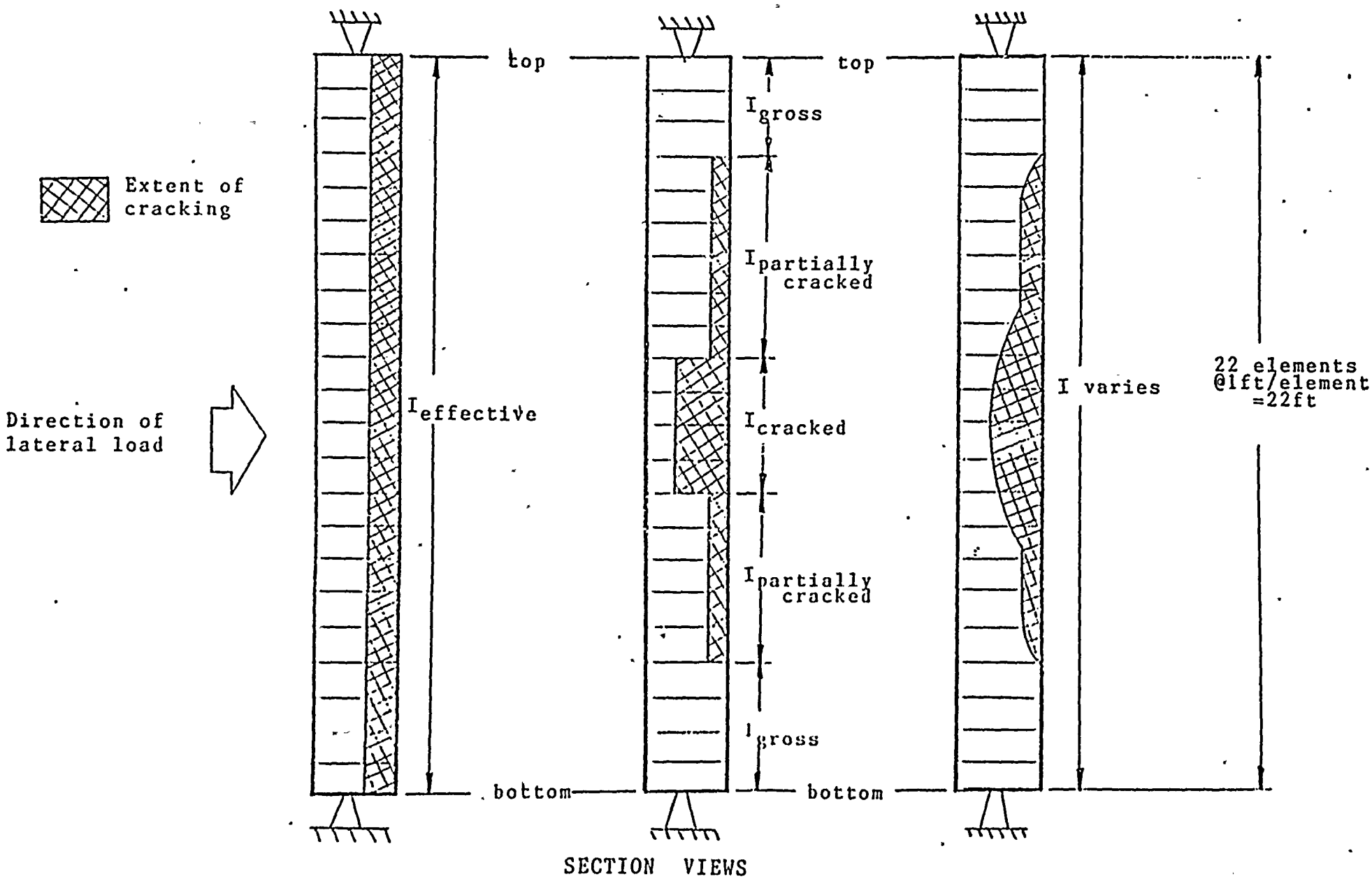
- WALL MODULUS OF ELASTICITY (E_W)
  - $E_W = 1.5 \times 10^6$ & 2.0×10^6 BASED ON CODE MINIMUM AND REALISTIC EXPECTED VALUES OF f'_m
- AVERAGE REBAR LOCATION ("d")
  - WALL ACTS AS A CONTINUUM
  - NRC CONCURRED
- CONSERVATIVE GROUT STRENGTH BASED ON PVNGS TEST
  - GROUT COMPRESSIVE STRENGTH USED LESS THAN TEST DATA (2300 PSI VERSUS 2600+ PSI)
- 3-STAGE MOMENT OF INERTIA - BASED ON SOUND ENGINEERING PRINCIPLES
  - FULLY GROUTED CELLS AT PVNGS
  - THEORY OF ELASTICITY (SRP 3.8.4 APPENDIX A)
    1) PLANE SECTIONS REMAIN PLANE
    2) SECTIONS REMAIN UNCRACKED UNTIL $(f_r)_m$ IS EXCEEDED
    3) SECTION REMAINS PARTIALLY CRACKED UNTIL $(f_r)_g$ IS EXCEEDED
  - REFINED GRID SIZE TO INCREASE ACCURACY OF ANALYSIS
  - SMALL BLOCK HEIGHT LIMITS STRESS CONCENTRATION
- WALL MODELING - VERIFIED BY INSPECTION
  - PINNED-PINNED END CONDITION
  - MASS INCLUDES ATTACHMENTS

DESCRIPTION OF $I_{3\text{-STAGE}}$



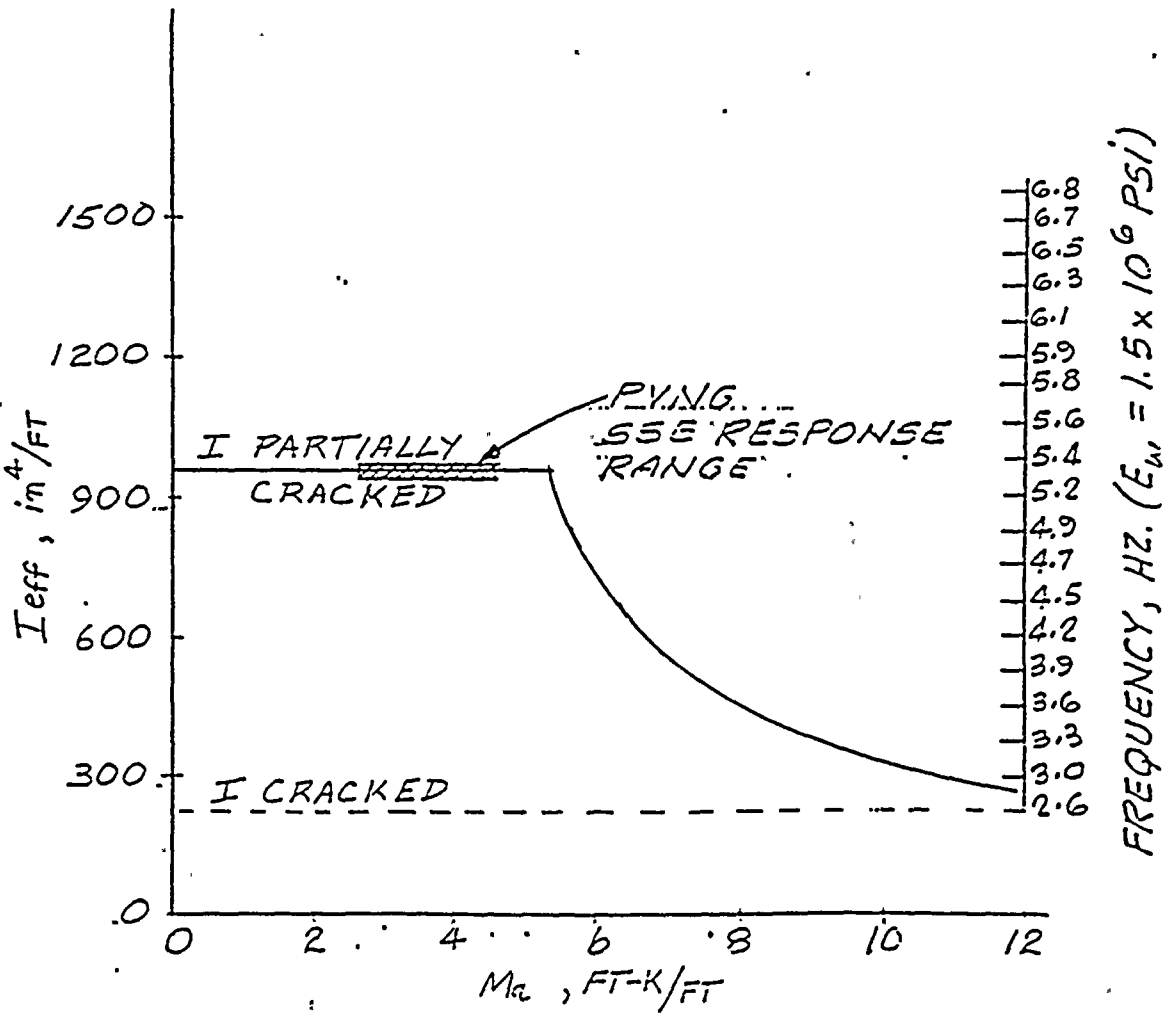
I_{GROSS} $I_{PARTIALLY\ CRACKED}$ $I_{CRACKED}$

DISTRIBUTION OF CRACKING AND STIFFNESS



$$I_{eff} = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr}$$

(PER ACI 318)



APPLIED MOMENT VERSUS EFFECTIVE MOMENT OF INERTIA NEGLECTING MASONRY FACESHELL, ELEVATION 74'-0"

AUGUST 20, 1986

PRELIMINARY

IV. RESULTS

TIME-HISTORY ANALYSES FREQUENCIES AND MOMENTS - ELEV. 74

ANALYSIS PARAMETERS	SSE (0.2G)		OBE (0.1G)	
	FINAL FREQUENCY	MA FT-K/FT	FINAL FREQUENCY	MA FT-K/FT
$E_W=1.5\times10^6$ $E'_S=0.67E_S$	4.9	2.7(1)	6.8	1.8(1)
$E_W=1.5\times10^6$ $E'_S=1.0E_S$	N/A(2)	N/A(2)	6.8	1.7(1)
$E_W=1.5\times10^6$ $E'_S=1.5E_S$	5.5	2.9	6.8	1.6(1)
$E_W=2.0\times10^6$ $E'_S=1.0E_S$	6.3	2.9	7.8	2.0(1)

(1) REPORTED IN JUNE 19, 1986 SUBMITTAL

(2) DATA NOT AVAILABLE

AUGUST 20, 1986

PRELIMINARY

IV. RESULTS CONTINUED

TIME-HISTORY ANALYSES MAXIMUM STRESSES (PSI) - ELEV. 74

ITEM	SSE (0.2G)		OBE (0.1G)	
	CALCULATED(1)	ALLOWABLE	CALCULATED	ALLOWABLE
MASONRY	420	833	290(2)	333
REBAR	13500	48000	9130(2)	24000
BOND	118	180	80(2)	120

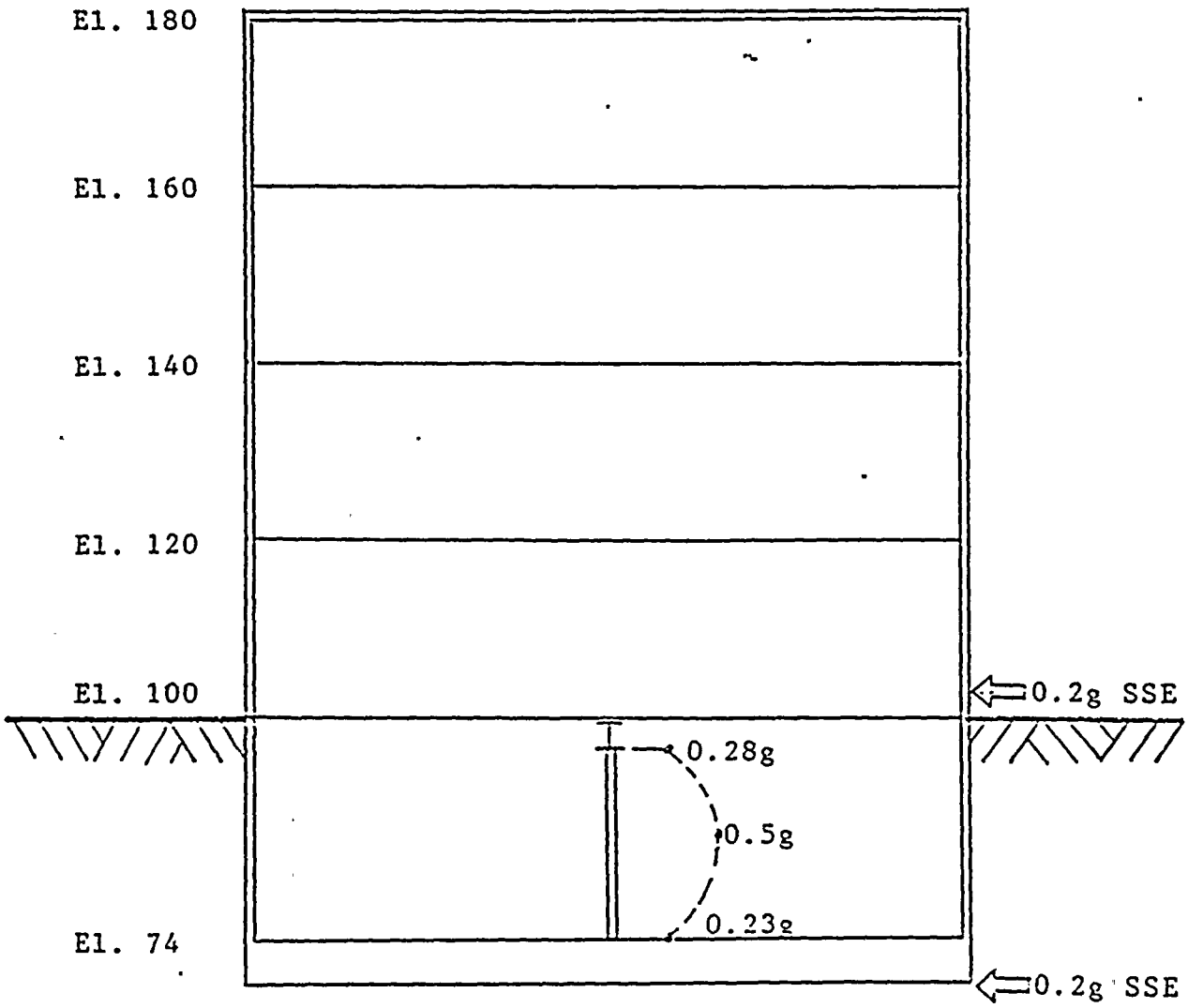
(1) CALCULATED STRESSES BASED ON UPDATED MAXIMUM MOMENT

(2) REPORTED IN JUNE 19, 1986 SUBMITTAL

GROUND ACCELERATION VERSUS WALL ACCELERATION

ELEVATION 74, 0.2g SSE

(TIME HISTORY ANALYSIS)



CONTROL BUILDING ELEVATION VIEW

AUGUST 20, 1986

V. CONFIRMATORY ANALYSIS

A. PURPOSE

- SUBSTANTIATE T-H ANALYSIS
- ADDRESS "DIP" IN T-H ANALYSIS
- ESTABLISH UPPER BOUNDS

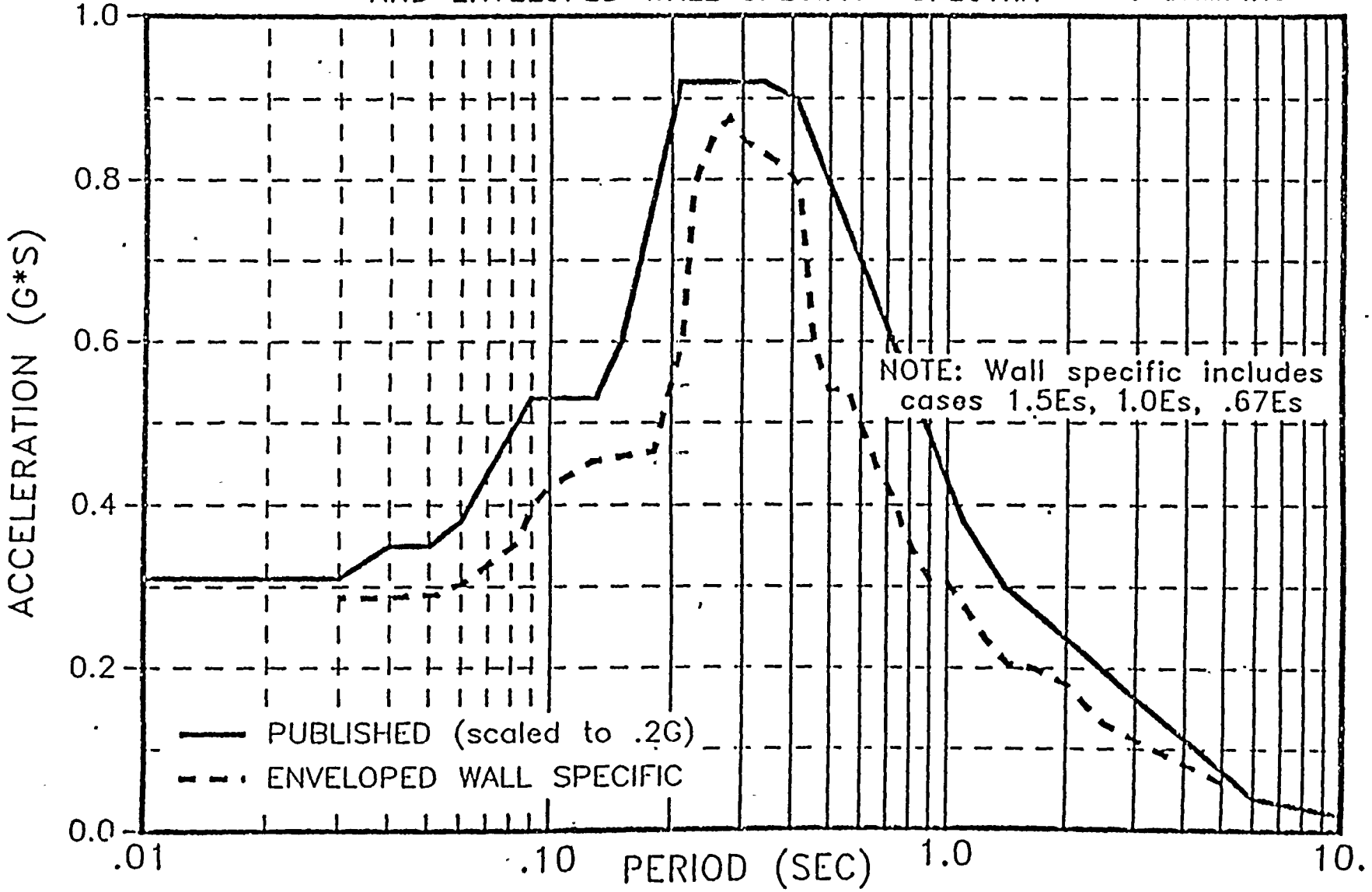
B. METHODOLOGY

- RSA
- ONE-DIRECTIONAL INPUT
- ENVELOPED SPECTRA/PEAK BROADENED SPECTRA
- 3-STAGE I
- UNCOUPLED STRIP MODEL

AVERAGE BETWEEN ELEVATIONS 74 & 100

COMPARISON OF PUBLISHED

AND ENVELOPED WALL SPECIFIC SPECTRA 7% DAMPING



AUGUST 20, 1986

PRELIMINARY

V. CONFIRMATORY ANALYSIS (CONTINUED)

C. RESULTS

RSA FREQUENCIES AND MOMENTS - ELEV. 74

PARAMETERS		SSE (0.2G)		OBE (0.1G)	
E_W(PSI)	TYPE OF SPECTRA	FREQ.	MA	FREQ.	MA
1.5×10^6	ENVELOPED	5.4	3.6	5.6	2.8
2.0×10^6	ENVELOPED	6.2	3.5	6.7	2.3
1.5×10^6	15% BROADENED	5.4	4.5	6.8	2.0
2.0×10^6	15% BROADENED	6.2	3.5	7.8	2.0
2.0×10^6	ENVELOPED(1)	6.1	3.5	6.1	2.3

RSA MAXIMUM STRESSES (PSI) - ELEV. 74

ITEM	SSE (0.2G)		OBE (0.1G)	
	CALCULATED	ALLOWABLE	CALCULATED	ALLOWABLE
MASONRY	650	833	400	333
REBAR	21,000	48,000	12,800	24,000
BOND	180	180	110	120

(1) BASED ON GROUT MOMENT OF INERTIA ONLY

AUGUST 20, 1986

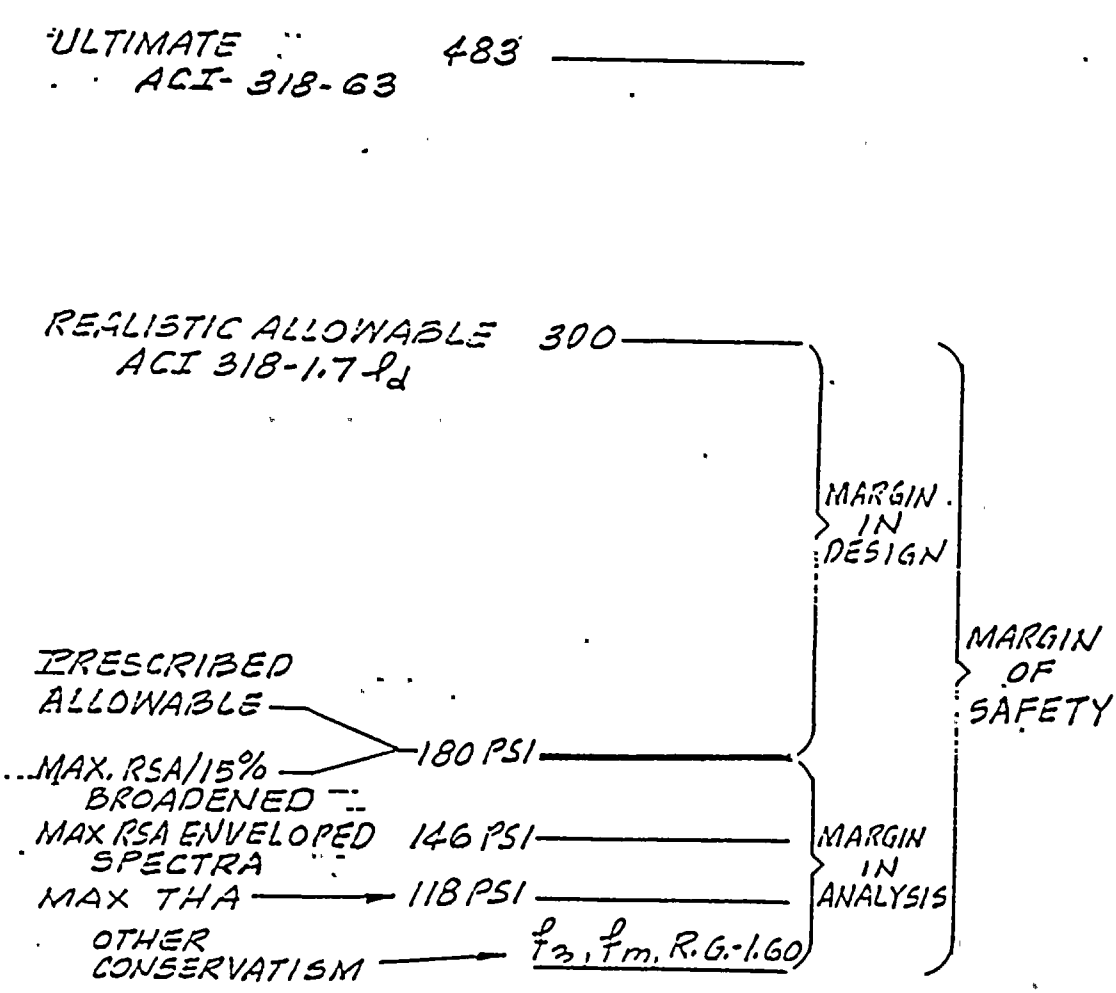
V. CONFIRMATORY ANALYSIS (CONTINUED)

D. CONCLUSIONS

- RSA RESULTS MORE CONSERVATIVE THAN T-H ANALYSIS RESULTS
- RSA ELIMINATES T-H "DIP"
- RSA CONFIRMS T-H ANALYSIS
- ALL SSE STRESSES ARE WITHIN PRESCRIBED ALLOWABLES
- OBE BOND AND REBAR STRESSES ARE WITHIN PRESCRIBED ALLOWABLES
- OBE MASONRY STRESS EXCEEDS THE REDUCED ALLOWABLES BY ONLY 20%
- WALLS ARE ADEQUATE AS-BUILT

VI MARGIN OF SAFETY

PICTORIAL REPRENTATION OF MARGINS OF SAFETY FOR BOND STRESS
ELEV 74'-0, SSE = 0.2g



NOTE:

OBE BOND STRESSES ARE NOT PRESENTED SINCE THEY ARE LESS CRITICAL

AUGUST 20, 1986

VII. CONCLUSIONS

- T-H ANALYSIS IS REALISTIC AND COMPLIES WITH SRP 3.8.4, APPENDIX A
  - ESTABLISHED PRINCIPLES OF ENGINEERING MECHANICS
  - SOUND ENGINEERING PRACTICES.
  - PROPER CONSIDERATION TO CRACKING OF SECTIONS AND BOUNDARY CONDITIONS
- RSA CONFIRMS MARGIN OF SAFETY
- WALLS AS CONSTRUCTED HAVE ADEQUATE MARGINS OF SAFETY FOR SSE AND OBE

ATTACHMENT 2

Presentation Slides from NRC/APS/Bechtel Meeting

of

August 28, 1986

PRESENTATION ON MASONRY WALLS
FOR THE
PALO VERDE NUCLEAR GENERATING STATION

UNITS 1, 2 AND 3

AUGUST 28, 1986

AGENDA

I. INTRODUCTION

II. CONCERNS

- A. NRC
- B. ANPP

III. RESOLUTION OF CONCERNS

IV. APPLICABILITY OF TEST DATA

V. RESULTS

VI. CONCLUSIONS

II. CONCERNS

A. NRC

- -FREQUENCY SENSITIVITY TO ASSUMED PARAMETERS
- -I 3-STAGE NOT SUPPORTED BY TESTS
- -INADEQUATE MARGINS
- -SCHEDULE

B. ANPP

- -WALLS ARE SAFE BUT UNACCEPTABLE TO NRC
- -APPLICATION OF CODE ALLOWABLES
- -AGGRESSIVE CONSTRUCTION ACTIVITY
- -CONSTRUCTION IMPACT ON SAFETY OF OPERATING PLANT

CONSTRUCTION IMPACT ON SAFE OPERATION

- MULTIPLE SAFETY RELATED COMPONENTS PENETRATING AND ATTACHED TO WALL, OR IN IMMEDIATE VICINITY.
- HOWEVER, MINIMAL SAFE SHUTDOWN RELATED COMPONENTS SUPPORTED FROM DIRECTLY OFF WALL OR PENETRATING WALL.
- STRINGENT WORKING CONDITIONS REQUIRED TO MINIMIZE THE RISK OF DAMAGE TO SAFETY RELATED COMPONENTS.
- CONSTRUCTION RISK IS ORDERS OF MAGNITUDE HIGHER THAN THE RISK ASSOCIATED WITH SSE.

III. RESOLUTION OF CONCERNS

-REVIEW AVAILABLE TEST DATA AND ITS APPLICABILITY

- A. HAMID'S PAPER
  - COMPARISON OF SPECIMENS TEST BY A. HAMID AND PVNGS WALLS
- ATKINSON, NOLAND & ASSOC. REPORT
  - MODULUS OF RUPTURE FROM WALL TESTS
- SONGS 1 DYNAMIC TESTING
  - COMPARISON OF PVNGS METHODOLOGY TO SONGS 1 TEST DATA

-REVIEW OF EXISTING ANALYSIS

- MAX. CALCULATED MOMENTS VERSUS CRACKING MOMENT
- WALL FREQUENCY

COMPARISON OF SPECIMENS TESTED BY A. HAMID AND PVNGS MASONRY WALLS

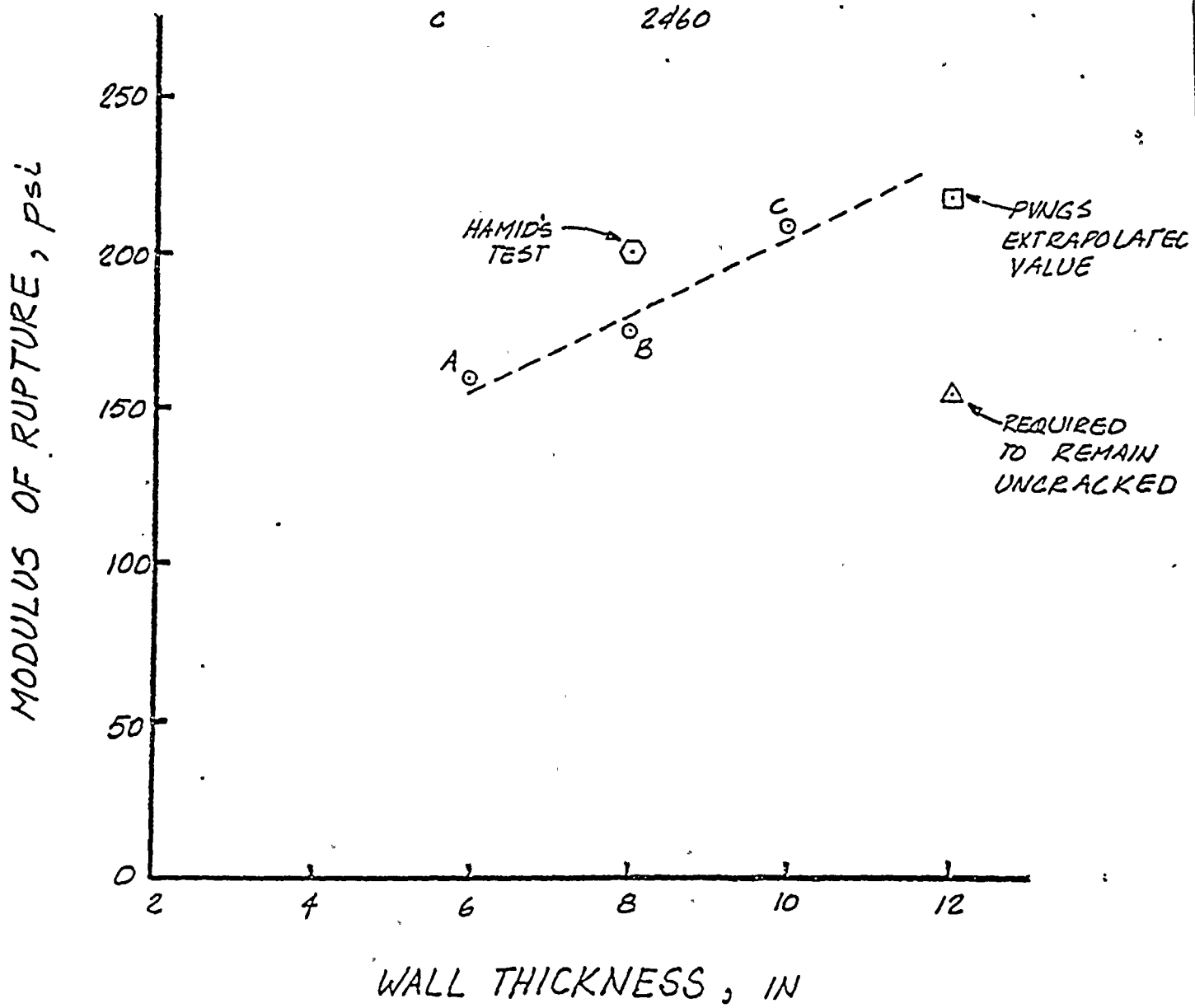
	TEST (1)	PVNGS	REMARKS
GROUT STRENGTH	3060 PSI	2600+ PSI	DIFFERENCE IN GROUT STRENGTH DOES NOT SIGNIFICANTLY AFFECT f_{rm} (TEST DATA).
MORTAR STRENGTH	2520 PSI	2200-2800 PSI	PVNGS VALUES ARE SIMILAR TO TEST DATA.
MASONRY STRENGTH	3640 PSI	2000 PSI	DIFFERENCE IN MASONRY STRENGTH DOES NOT AFFECT f_{rm} SINCE FAILURE OCCURS AT BED JOINTS.
REINFORCEMENT	NONE	REINFORCED	f_{rm} IS INDEPENDENT OF REINFORCEMENT.
BLOCK SIZE	8"X8"X16"	4"X12"X16"	LOWER HEIGHT OF PVNGS UNITS REDUCES STRESS CONCENTRATIONS.
MODULUS OF RUPTURE-MEAN (f_{rm})	203 PSI	APPLICABLE	FULLY GROUTED CELLS HAVE HIGHER f_{rm}.
STANDARD DEVIATION	34 PSI	APPLICABLE	ACCOUNTS FOR UNCERTAINTIES
MODULUS OF RUPTURE (FRM - 1.0σ)	169 PSI	APPLICABLE	REALISTIC VALUE FOR PNVGS ANALYSIS

REFERENCE: (1): EFFECT OF GROUTING ON THE FLEXURAL TENSILE STRENGTH OF CONCRETE BLOCK MASONRY BY ROBERT G. DRYSDALE AND AHMAD A. HAMID.

MODULUS OF RUPTURE FROM WALL TESTS VS. WALL THICKNESS

(BASED ON ATKINSON, NOLAND & ASSOCIATES REPORT)

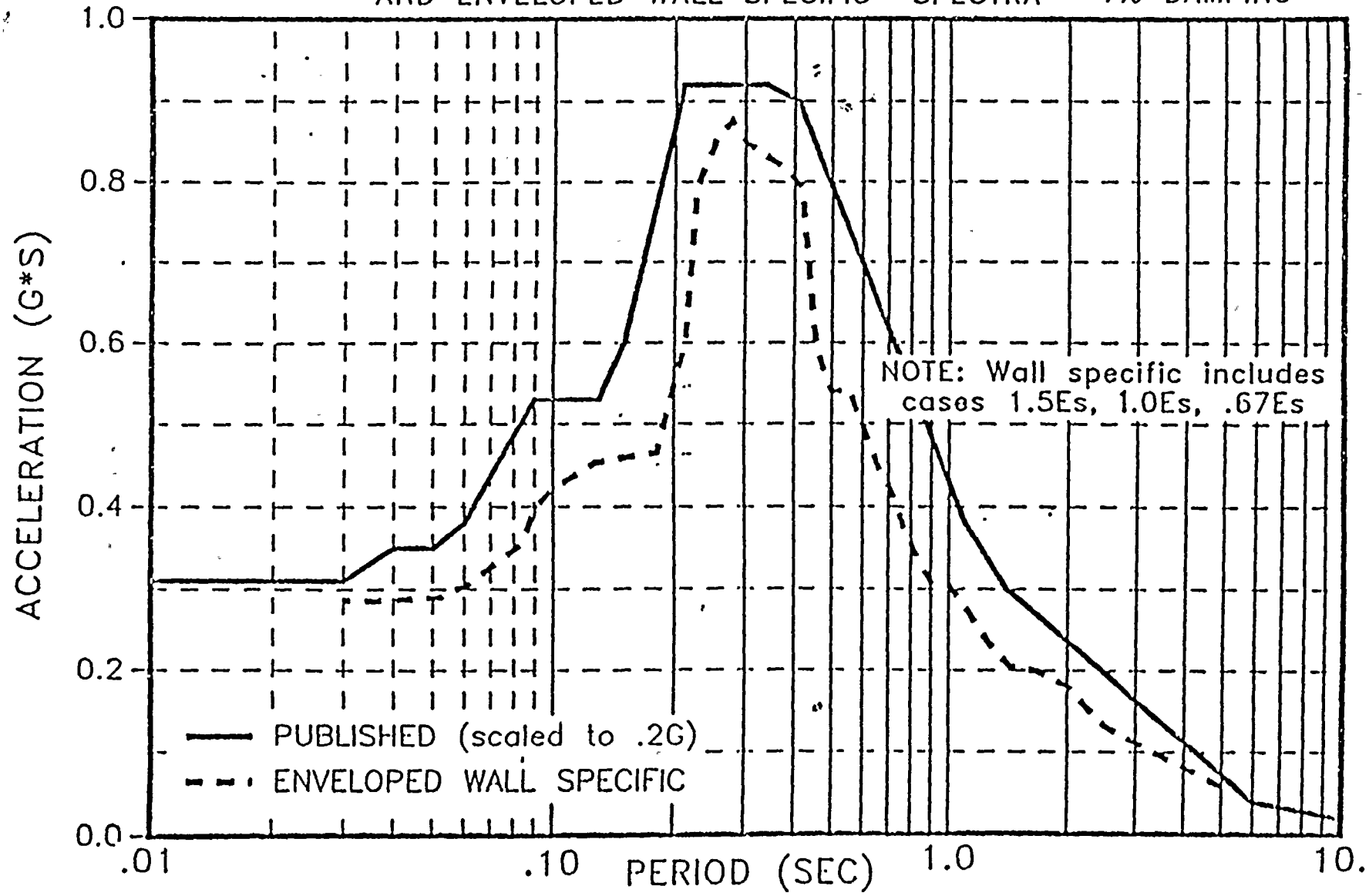
POINT	f'_m, psi
A	3185
B	2595
C	2460



AVERAGE BETWEEN ELEVATIONS 74 & 100

COMPARISON OF PUBLISHED

AND ENVELOPED WALL SPECIFIC SPECTRA 7% DAMPING



IV. RESULTS

- TEST RESULTS SUPPORT $f_{rm} > 2.5\sqrt{f'_m}$
  - A. HAMID'S PAPER: 203 PSI
  - ATKINSON, NOLAND & ASSOCIATES' REPORT: 219 PSI FOR f'_m = 2000 PSI

- BASED ON USE OF REALISTIC f_{rm}
  - $M_a < M_{cr}$
  - MAXIMUM TENSILE STRESS = 155 PSI

- MARGINS TO ACCOUNT FOR UNCERTAINTIES
  - MARGIN IN STIFFNESS IS 65% (F=6.8 VERSUS 5.3)
  - MARGIN BETWEEN M_a AND M_{cr} TO ACCOUNT FOR UNCERTAINTIES

V. CONCLUSIONS

-WALLS ARE ACCEPTABLE BASED ON JUNE ANALYSIS

- CODE ALLOWABLES MET
- ANALYSIS INCLUDES CONSERVATISM

-JUNE ANALYSIS SUPPORTED BY CONFIRMATORY ANALYSIS

- CODE ALLOWABLES MET
- ADDITIONAL CONSERVATISM IS PROVIDED

-REVIEW OF TEST DATA VALIDATES ANALYSES

- CONSISTENT INDEPENDENT TEST DATA
- ANALYSIS IS CONSERVATIVE - HIGHER f_{rm} THAN CODE VALUE

-NRC CONCERNS RESOLVED

- WALL REMAINS UNCRACKED (3-STAGE MODEL IS NOT USED)
- FREQUENCY AWAY FROM AMPLIFIED REGION
- ADDITIONAL MARGIN IN FREQUENCIES AND CALCULATED MOMENTS