HELPING BUILD ARKANSAS

ARKANSAS POWER & LIGHT COMPANY

P.O. BOX 551 • LITTLE ROCK, ARKANSAS 72203 • (501) 371-4191

April 30, 1982

DONALD A. RUETER
DIRECTOR
TECHNICAL AND
ENVIRONMENTAL SERVICES

ØCANØ48211

Director of Nuclear Reactor Regulation
ATTN: Mr. Robert A. Clark, Chief
Operating Reactors Branch #3
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Director of Nuclear Reactor Regulation
ATTN: Mr. J. F. Stolz, Chief
Operating Reactors Branch #4
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

SUBJECT: Arkansas Nuclear One - Units 1 & 2
Docket Nos. 50-313 and 50-368
License Nos. DPR-51 and NPF-6
Final Response to NRC's Questions
on AP&L's Submittal on IE Bulletin
80-11, "Masonry Wall Design"
(File: 1510.1, 2-1510.1)

Gentlemen:

Your letter dated January 15, 1982, indicated that NRC has reviewed AP&L's letter dated January 29, 1981, concerning our response to IE Bulletin 80-11, "Masonry Wall Design", and the conclusion was reached that additional information was needed. An attached Franklin Research Center Technical Evaluation Report identified the requested information. Our initial response to your January 15, 1982, letter was submitted on March 15, 1982. It is the purpose of this letter to provide you with our final response to each request for information.

Attachment A provides responses to the questions raised by Franklin Research Center for Unit 1. Attachment B provides responses for Unit 2 questions.

A001
S1/1

This completes AP&L's commitment to NRC in response to IEB 80-11. The actual cost to our customers to provide the attached responses totaled approximately twenty-five thousand dollars ($25,000.00).

Sincerely,

Donald A. Rueter

Donald A. Rueter

RWH:DET:sc

Attachments

STATE OF ARKANSAS)
) SS
COUNTY OF PULASKI)

I, Donald A. Rueter, being duly sworn, subscribe to and say that I am Director, Technical & Environmental Services for Arkansas Power & Light Company; that I have full authority to execute this oath; that I have read the document numbered ØCANØ48211 and know the contents thereof; and that to the best of my knowledge, information and belief the statements in it are true.

Donald A. Rueter

Donald A. Rueter

SUBSCRIBED AND SWORN TO before me, a Notary Public in and for the County and State above named, this 30 day of April, 1982.

Sharon Kaye Hendrix

Notary Public

My Commission Expires:

9-19-89

Attachment A

RESPONSE TO NRC QUESTIONS ON ARKANSAS NUCLEAR ONE - UNIT 1

IE BULLETIN 80-11 REPORT

QUESTION 1:

Explain why the frequencies of some of the walls presented in Tables 4, 5 and 6 of Reference 2 are widely different from the frequencies of other walls of comparable dimensions. Also explain why some of the frequencies in these tables are indicated as OBE or DBE.

Response:

The frequency of concrete masonry walls depends not only on wall dimensions; it is also affected by the boundary conditions and section moment of inertia considered in the analysis. The following paragraphs describe the effects on the frequency due to the variation of these parameters.

1. Boundary Conditions

   a. One-Way vs. Two-Way Behavior

      Wall frequencies calculated based on one-way behavior were always lower than those calculated from a two-way plate analysis even though the thickness and height are comparable. One-way action was generally considered when boundary conditions and/or the width to height aspect ratio of the wall

can substantiate this simplified technique. Two-way action was considered for other cases, as appropriate, that required a more refined analysis.

b. Hinged Support vs. Fixed Support

Wall frequencies calculated based on hinged supports were always lower than those with fixed supports. Hinged support condition was generally considered unless the use of fixed support condition can be substantiated.

2. Effective Moment of Inertia Due to Cracking

Frequencies obtained based on the fully cracked moment of inertia were always lower than those based on the partially cracked effective moment of inertia. As stated in the criteria for the reevaluation of concrete masonry walls, the calculation of the wall frequency based on the partially cracked behavior was necessary for conservatism when the frequency for the fully cracked section was on the low frequency side of the response spectral peak.

The indication of OBE (Operating Basis Earthquake) or DBE (Design Basis Earthquake) for some of the frequencies was for clarification purposes since the frequency is affected by the effective moment of inertia, which in turn is affected by the seismic load level. This is further explained

in the response to Question No. 5b. When the frequency in the table was indicated as OBE, it implies that the load case involving OBE was a controlling load case.

QUESTION 2:

With reference to Section 6.1.4, Appendix A (2), justify using the average acceleration rather than the envelope of the response spectra for walls supported by two floors.

RESPONSE:

The evaluation herein demonstrates that the use of the average floor acceleration response spectra to calculate the response of the wall panel is appropriate. For the purpose of this evaluation, the seismic response of a simply-supported, uniform beam simulating a strip of the wall panel with unit width is considered, as shown in Figure 1.

The equation of motion of an undamped, simply-supported beam can be written in terms of the total displacement with respect to some fixed reference axis as:

$$m \frac{\partial^2 u}{\partial t^2} + EI \frac{\partial^4 u}{\partial x^4} = 0 \qquad (1)$$

Where m and EI are the mass density and flexural rigidity of the beam. Denote the seismic excitations at the ends of the beam as U_a and U_b. Then the total displacement u(x,t) can be expressed in terms of the two seismic motions and the relative displacement to the seismic motions as:

$$u(x,t) = (x/L)\, U_b + (1 - x/L)\, U_a + r(x,t) \qquad (2)$$

Where L is the length of the beam. The relation expressed by the above equation is shown in Figure 2. The relative displacement r(x,t) needs to satisfy the following simply-supported conditions:

$$r(o,t) = r(L,t) = 0 \tag{3}$$

$$\left.\frac{\partial^2 r}{\partial x^2}\right|_{x=o} = \left.\frac{\partial^2 r}{\partial x^2}\right|_{x=L} = 0 \tag{4}$$

Substitute Equation 2 into Equation 1, the equation of motion in terms of relative displacement $r(x,t)$ can be expressed as:

$$m\frac{\partial^2 r}{\partial t^2} + EI\frac{\partial^4 r}{\partial x^4} = -m(x/L)\ddot{U}_b - m(1 - x/L)\ddot{U}_a \tag{5}$$

The eigen-function solutions for the homogeneous equation associated with Equation 5 that satisfy the boundary conditions specified by Equations 3 and 4 are:

$$\sin\frac{n\pi x}{L}, \; n = 1, 2, 3, \ldots,$$

and the corresponding frequencies of vibration are:

$$\omega_n = n^2\pi^2 \left[\frac{EI}{mL^4}\right]^{\frac{1}{2}} \qquad n = 1, 2, 3, \ldots \tag{6}$$

So, the solution of Equation 5 can be expressed as:

$$r(x,t) = \sum_{n=1}^{\infty} a_n(t)\sin\frac{n\pi x}{L} \tag{7}$$

Substitute Equation 7 into Equation 5, and multiply the latter by sin $\frac{n\pi x}{L}$, and then integrate it with respect to x over the full length of the beam, the equation of motion can be transformed into modal equations of motion as:

$$\ddot{a}_n + \omega^2{}_n a_n = \Gamma_n \frac{(\ddot{U}_a + \ddot{U}_b)}{2} \qquad n = 1, 3, 5, \ldots \qquad (8a)$$

and

$$\ddot{a}_n + \omega^2{}_n a_n = \Gamma_n \frac{(\ddot{U}_a - \ddot{U}_b)}{2} \qquad n = 2, 4, 6, \ldots \qquad (8b)$$

where Γ_n = participation factor

$$= \frac{4}{n\pi} \qquad (9)$$

If damping in the form of modal damping ratio is included, Equations 8a and 8b becomes:

$$\ddot{a}_n + 2\xi_n\omega_n\dot{a}_n + \omega^2{}_n a_n = \Gamma_n \frac{(\ddot{U}_a + \ddot{U}_b)}{2} \qquad n = 1, 3, 5, \ldots \qquad (10a)$$

and

$$\ddot{a}_n + 2\xi_n\omega_n\dot{a}_n + \omega^2{}_n a_n = \Gamma_n \frac{(\ddot{U}_a - \ddot{U}_b)}{2} \qquad n = 2, 4, 6, \ldots \qquad (10b)$$

Where ξ_n is the damping ratio of the n^{th} mode.

Equation 10a means that the odd-number modes which are symmetrical about the mid-span of the beam will be excited by the average of the two seismic excitations; while equation 10b means that the even-number modes which are antisymmetrical about the mid-span of the beam will be excited by half of the difference between the two seismic excitations.

Expressing the maximum modal displacement response in equations 10a and 10b in terms of absolute acceleration response spectra gives:

$$|a_n|_{max} \leq |\Gamma n| \left[\frac{S_a(\xi_n, \omega_n)}{2\omega^2_n} + \frac{S_b(\xi_n, \omega_n)}{2\omega^2_n} \right]$$

$$\leq \frac{4mL^4}{n^5 \pi^5 EI} \left[\frac{S_a(\xi_n, \omega_n) + S_b(\xi_n, \omega_n)}{2} \right] \qquad (11)$$

$$n = 1, 2, 3, \ldots$$

This illustrates that the use of the average of two floor acceleration response spectra to calculate the modal response of a wall panel is appropriate.

QUESTION 3:

With reference to Section 5.8 (2), justify neglecting out-of-plane interstory drift in the analysis and explain whether the predicted in-plane interstory drift of 0.0006 inch/ft of height applies to confined walls.

Response:

As stated in the report (Reference 1), out-of-plane stresses due to interstory displacements are insignificant due to the flexibility of the wall in the out-of-plane direction. This is particularly true if the rotational stiffness of the floor-wall connection is taken into consideration. In order to demonstrate that the above statement is valid, a parametric study was performed using the same maximum interstory drift of 0.0006 inch/ft of height as for the in-plane drift study. In this study, the top and bottoms of the wall were conservatively assumed to be fixed, even though a 100% fixity cannot be actually attained. Listed below is the result of the analysis which shows that the resulting stresses are negligible compared with the capacity of the masonry walls.

H (height = 19'-0"

t(thickness) inches	fs (steel stress) psi	fm (masonry compressive stress) psi	fv (shear stress) psi
8	111.7	3.2	0
12	185.3	4.07	0
18	507.0	6.7	0
24	751.7	8.1	0.13
30	918.1	9.0	0.15
36	1168.6	10.2	0.2

The predicted interstory drift of 0.0006 inch/ft of height was applied to confined walls in the analysis.

QUESTION 4:

With reference to Section 6.1.2, Appendix A(2), provide sample calculations to show that analysis using only the fundamental mode is adequate and is comparable to a multimode analysis.

Response:

Descriptions and results of the analysis of two sample walls are provided herein to show that the analysis using the uniform inertia load associated with only the fundamental mode is adequate and is comparable to a multimode analysis.

The first sample wall is a multiwythe cantilever wall of 1'-6" thick and 8'-0" high. This wall was analyzed as two cantilever beams tied by springs in one-way action. A typical mathematical model is shown in Figure 3. A dynamic analysis as described in the response to Question 5C was performed to obtain moments and shears of the wall due to seismic loads in the out-of-plane direction.

The second sample wall is a rectangular wall of 1'-6" thick supported at four sides with a dimension of 15'-0' by 22'-0". This wall was modeled as a plate with four simply-supported edges. The mathematical model is shown in Figure 4. A dynamic analysis using Computer Program STARDYNE was performed. A modal analysis was first performed to determine mode shapes, participation factors, and natural frequencies of the wall, which were then used for a spectral response analysis to determine the resulting moments and shears in the wall.

Resulting moments and shears based on modal responses of the above two walls for the fundamental mode as well as the multimode combination using SRSS procedures are listed on the first two lines in the following table. Another set of moments and shears for these two walls based on the uniform inertia load associated with only the fundamental mode is also listed on Line 4 in the same table for comparison.

		Cantilever Wall (at El. 335'-0")		Four Side Simply-Supported Wall (At El. 386'-0")	
		Moment (at base) in-lb/ft	Shear (at base) lb/ft	Moment (at center in-lb/ft	Shear (at base) lb/ft
(1)	Fundamental Mode (use modal responses)	24,128	344	38,188	1061
(2)	SRSS Combination (use modal resposes)	24,165	351	38,246	1066
(3)	Difference	0.15%	1.97%	0.15%	0.47%
(4)	Fundamental Mode (use uniform inertia load)	27,042	563	39,477	1165

As shown in the table, the resulting moments and shears (Line 1) from the fundamental mode are only 0.15% and 1.97%, respectively, less than those (Line 2) from the SRSS combination. It can also be seen that the values

on Line 2 are even smaller than those (Line 4) from the fundamental mode using the uniform inertia load. The comparison demonstrates that the analysis using the uniform inertia load associated with only the fundamental mode is adequate and conservative.

QUESTION 5:

With reference to Table 5 (2), briefly describe the techniques used for (a) verification by curves, (b) effective inertia analysis, and (c) dynamic analysis. Also clarify whether pipe reactions due to thermal expansion are considered in the analysis.

Response:

The techniques used for the blockwall reevaluation as requested are as follows:

a. Verification by Curves

The purpose of the design curve was to verify the adequacy of the seismic Category 1 blockwalls by categorizing them into several groups with similar parameters such as boundary conditions, floor elevations, wall thickness and height. These curves were developed using the same methods and criteria as described in Reference 1. The following type of curves have been generated to account for various applicable parameters.

1. Vertically Spanned Beam

The curves in this category are applicable for walls acting as a simply supported beam spanned vertically. One set of curves for each different wall thickness has been generated. Each set consists of several curves for walls located on different floor elevations. The curves were plotted for various wall heights versus reinforcing steel and masonry stresses. A typical curve

is shown in Figure 5. Walls with a proper aspect ratio, i.e., the length is larger than the height, were entered into an appropriate curve for evaluation.

2. Horizontally Spanned Beam

The curves in this category are applicable for walls acting as a simply supported beam spanned horizontally. Similar sets of curves as for Item 1 above have been generated. Walls with a proper aspect ratio, i.e., the wall height is larger than the length, were entered into an appropriate curve for evaluation.

3. Plate

The curves in this category are for walls with the boundary conditions, i.e., a fixed base, a free top edge and two hinged sides, for which the curves for Items 1 and 2 above were not appropriate. Similar sets of curves as for Item 1 above have been generated. Walls with the above mentioned boundary conditions were entered into an appropriate curve for evaluation.

The wall was considered to be adequate if it falls within the allowable range as shown on the appropriate curves. Otherwise, a more refined manual or computer method would be used to evaluate the adequacy of the wall.

b. Effective Inertia Analysis

In the structural analysis of concrete masonry walls, cracked section properties were generally considered in computing the moment of inertia of walls. For some walls, the effective moment of inertia was used to account for the partially cracked condition.

The method of computing the effective moment of inertia (Ie) for blockwalls was described in detail in Appendix A (Reference 1) which is in essence the same technique as set forth in Sections 9.5.2, ACI 318 (Reference 2). It should be noted that Ie is a function of the applied moment (Ma) which is mainly due to the inertia force of the wall in this case. Since the inertia force is a frequency dependent parameter, and the frequency largely depends on the value of Ie, an iterative technique was employed to determine the correct Ie for the walls in which the partially cracked condition is a controlling factor. The iterative technique is outlined as follows:

1. Use an assumed moment of inertia Ia (lies between the values of Ig and Icr) to compute the wall frequency and the applied moment Ma.

2. Compute Ie with the calculated Ma from Step 1.

3. Compare Ie with Ia used in Step 1. If they match closely, Ia would be used as a correct Ie for the evaluation of the particular blockwall. If they do not match, another iterative step would be performed until the assumed Ia matches closely with the corresponding Ie.

c. Dynamic Analysis

As described in Section 5.6 (Reference 1), a dynamic analysis was used to verify those walls which could not be shown to be adequate by a simplified manual analysis nor by a static computer analysis. The dynamic analysis described herein was primarily for multiwythe cantilever walls.

Since the shear strength of mortar and grout at the collar joint was conservatively assumed to be zero for all multiwythe walls for the ANO-1 plant, a non-composite section was considered in the analysis. Two exterior wythes of the wall were modeled as two separate flexural members tied together by tie bars. These tie bars were modeled as springs connected to the lumped masses of each flexural member. A typical mathematical model for the multiwythe cantilever wall is shown in Figure 3.

A modal analysis was performed to determine mode shapes, participation factors and natural frequencies for the wall. A spectral response analysis considering multimode responses was then performed to determine the resulting moments and shears in the wall

based on the appropriate floor response spectrum as input. The modal responses were combined using the SRSS procedure (the square root of the sum of the squares). The resulting moments and shears were used for the evaluation of the adequacy of the wall.

Significant pipe thermal reactions have been considered in the blockwall analysis.

QUESTION 6:

Provide more information on seismic analysis in different directions and explain how the equipment weights and pipe weights were accounted for.

Response:

One horizontal and one vertical seismic inertia load was considered simultaneously in the analysis of concrete blockwalls. Since all concrete blockwalls in ANO plants were not load bearing walls nor shear walls, stresses in the in-plane direction due to seismic load were insignificant. Therefore, seismic load due to the horizontal earthquake component in the out-of-plane direction combined with the vertical seismic load using the absolute sum method was considered in the evaluation.

Weights of the equipment and pipes attached to walls were generally considered as equivalent uniform loads distributed over the whole wall except for seismic Class 1 pipes. The wall weight was generally increased by 10% to represent the weight of all these attachments in determining the inertia force in the out-of-plane direction during a seismic event. Reactions of seismic Class 1 pipe supports were considered as concentrated loads applied to the wall in addition to the 10% allowance. Calculations on several sample walls indicated that the 10% allowance for attachment weights was adequate. For a few walls, the actual weights of the attachments were used in lieu of the 10% increase.

QUESTION 7:

With reference to Section 5.0, Appendix A (2), provide the values for allowable stresses in axial compression, bearing, tension normal to the bed joint, and tension parallel to the bed joint.

Response:

The values for allowable stresses as requested are listed as follows:

1. Allowable axial compression stress = 0.22 f'm = 330 psi (This value was to be multiplied by $[1- (\frac{h}{40t})^3]$ if the wall had a significant vertical load.) where,

   f'm = Masonry ultimate compressive strength = 1500 psi

   h = height of wall

   t = thickness of wall

2. Allowable bearing stress = 0.25 f'm = 375 psi (on full area)

   = 0.375 f'm = 563 psi (on 1/3 area or less)

3. Allowable tension stress normal to the bed joint = 1.0 $(m_o)^{\frac{1}{2}}$ (40 psi max.) = 40 psi where, m_o = Mortar compressive strength = 2000 psi

4. Allowable tension stress parallel to the bed joint = 1.5 $(m_o)^{\frac{1}{2}}$ (80 psi max.) = 67 psi

It should be noted that the values for tension normal to the bed joint and tension parallel to the bed joint as listed above are for unreinforced masonry only. As stated in Reference 1, all Seismic

Category 1 masonry blockwalls for the ANO-1 plant are reinforced masonry. These values were only used to indicate if the wall is cracked or uncracked at the bed joint. The walls indicated as uncracked in Table 6 (Reference 1) have been further evaluated using the cracked section moment of inertia. The resulting reinforcing steel tensile stresses and masonry compressive stresses were found to be within the allowable limits.

QUESTION 8:

With reference to Section 5.2.1 of Appendix A (2), justify the proposed increase factor of 1.67 for shear, bond, tension normal to the bed joint, and tension parellel to the bed joint. The SEB criteria (3) suggest an increase factor of 1.3 for masonry shear, 1.5 for masonry tension parallel to the bed joint, and 1.3 for unreinforced masonry tension normal to the bed joint.

Response:

The factor of 1.67 for shear bond stresses was chosen in January 1981 (Reference 1) prior to the SEB criteria being published in July 1981 (Reference 3). Code allowable stresses for masonry, shear, bond and tension normal or parallel to the bed joint were increased by a factor of 1.67 for load combinations involving abnormal and/or extreme environmental conditions which are credible but highly improbable. Since ACI-531 code allowable stresses (Reference 4, Chapter 10.1 of commentary) are generally associated with a factor of safety of 3, the 1.67 increase still provides a factor of safety against failure of 1.8 (3/1.67).

QUESTION 9:

Indicate the present status of walls which were inaccessible and hence excluded from the original field survey.

Respon.a:

During the initial field surveys, the follwing walls in proximity to safety related systems were determined to be inaccessible (Reference 1).

Wall No.	Floor El.	Wall Thick	Wall Height	Wall Type
4-B-66	335'-0"	1'-0"	16'-3"	I
4-B-169	372'-0"	1'-9"	12'-0"	I
6-B-42	335'-0"	3'-6"	14'-9"	I
6-B-44	335'-0"	3'-6"	14'-9"	I

Walls 6-B-44 and 6-B-42 will be surveyed in accordance with "Procedure for Field Survey to Determine Seismic Category I Pipe Supports with Concrete Expansion Bolts on Block Walls" as submitted in Reference 1. This survey will be accomplished during the present unit outage. Drawing reviews indicate no substantial items attached to these walls. Should the field survey determine anything which would require further attention, NRC will be notified.

Wall 4-B-66 is in a high radiation area (3 REM minimum) caused by spent resin holding tank. Based on a drawing review and previous analysis performed on this wall AP&L has determined that the potential radiation exposure does not warrant a survey of this wall.

Wall 4-B-169 is inside a closed pipe chase and will never be accessible without significant removal of existing plant structure. Therefore no survey is planned for this wall.

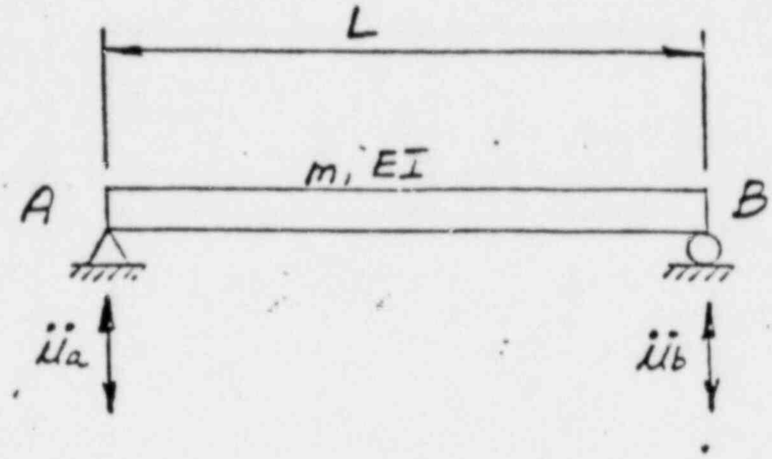
REFERENCES

1. AP&L's Letter dated January 29, 1981 "Response to IE Bulletin 80-11 Masonry Wall Design".

2. ACI Standard 318-77, Building Code for Reinforced Concrete (ACI 318-77).

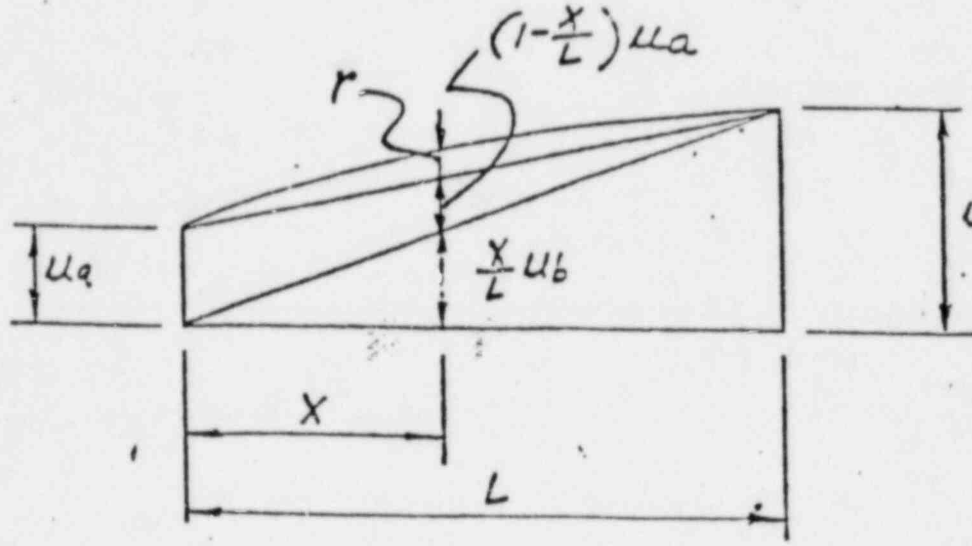
3. Standard Review Plan, Section 3.8.4, Appendix A, "Interim Criteria for Safety-Related Masonry Wall Evaluation", NRC, July 1981.

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



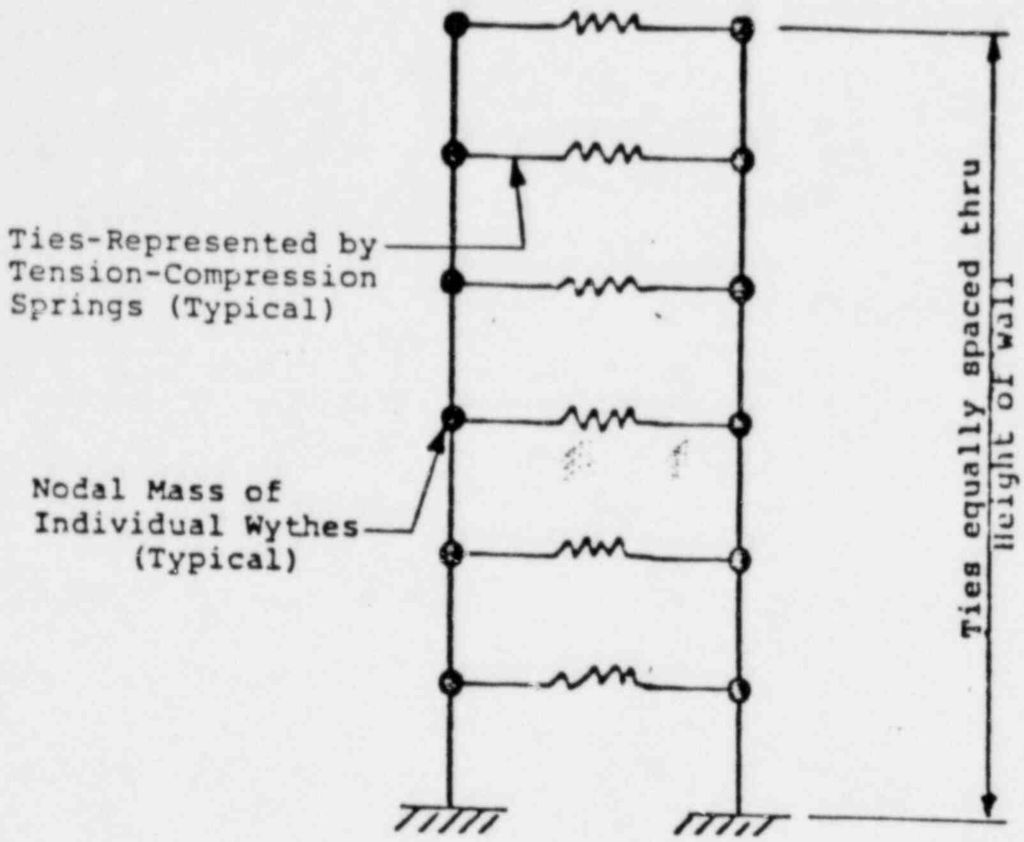
IDEALIZED SIMPLY-SUPPORTED UNIFORM BEAM

FIGURE 1



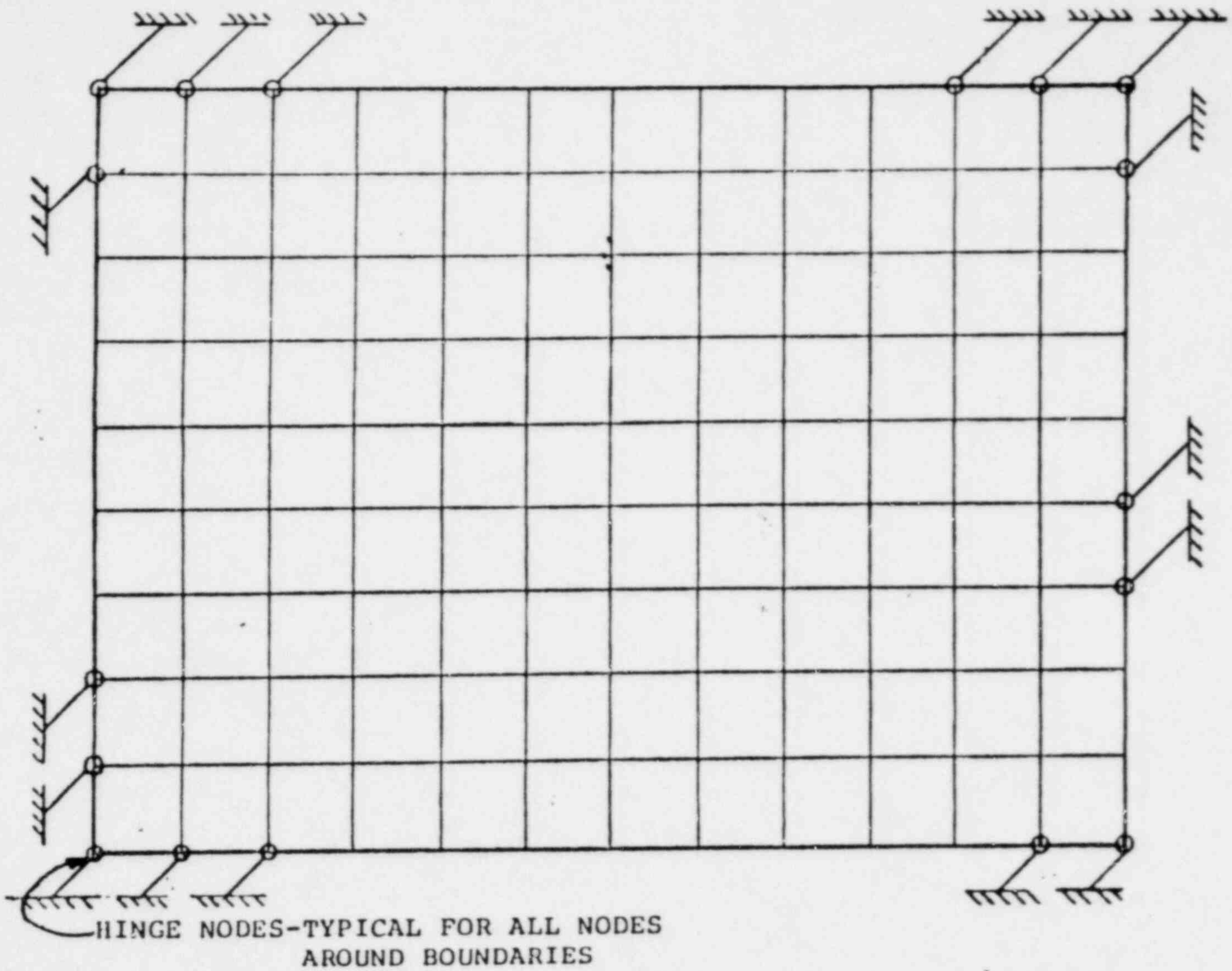
RELATION BETWEEN SEISMIC EXCITATION
AND RELATIVE DISPLACEMENT

FIGURE 2



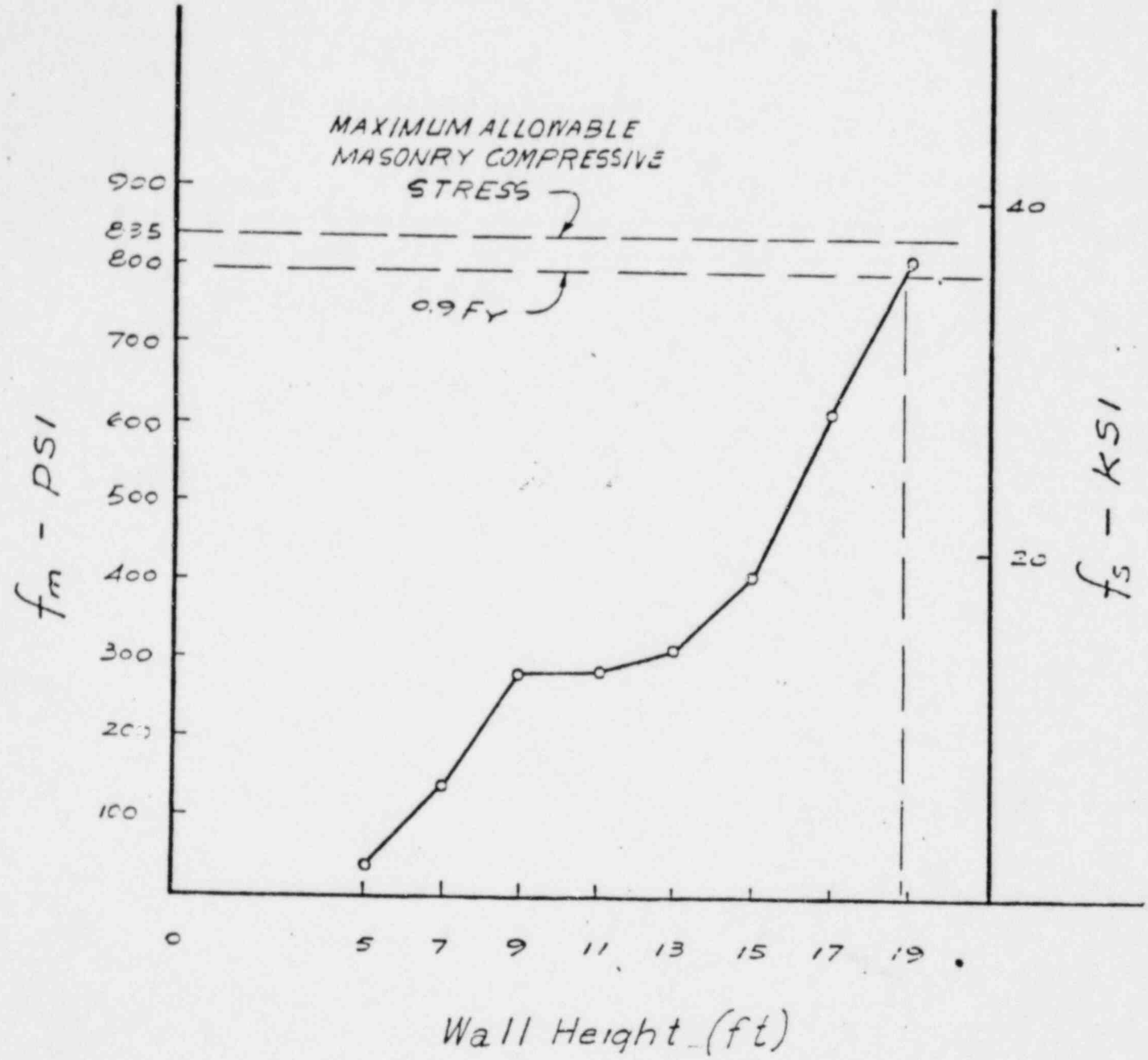
TYPICAL MATHEMATICAL MODEL
DYNAMIC ANALYSIS (MULTI-WYTHE CANTILEVER WALL)

FIGURE 3



MATHEMATICAL MODEL
DYNAMIC ANALYSIS (PLATE)

FIGURE 4



TYPICAL CURVE
VERTICALLY SPANNED BEAM

FIGURE 5

ATTACHMENT B

RESPONSE TO NRC QUESTIONS ON ARKANSAS NUCLEAR - UNIT 2

IE BULLETIN 80-11 REPORT

QUESTION 1:

Explain why the frequencies of some of the walls presented in Tables 4, 5 and 6 of Reference 2 are widely different from the frequencies of other walls of comparable dimensions. Also explain why some of the frequencies in these tables are indicated as OBE or DBE.

Response:

The frequency of concrete masonry walls depends not only on wall dimensions; it is also affected by the boundary conditions and section moment of inertia considered in the analysis. The following paragraphs describe the effects on the frequency due to the variation of these parameters.

1. Boundary Conditions

   a. One-Way vs. Two-Way Behavior

      Wall frequencies calculated based on one-way behavior were always lower than those calculated from a two-way plate analysis even though the thickness and height are comparable. One-way action was generally considered when boundary conditions and/or the width to height aspect ratio of the wall

can substantiate this simplified technique. Two-way action was considered for other cases, as appropriate, that required a more refined analysis.

b. Hinged Support vs. Fixed Support

Wall frequencies calculated based on hinged supports were always lower than those with fixed supports. Hinged support condition was generally considered unless the use of fixed support condition can be substantiated.

2. Effective Moment of Inertia Due to Cracking

Frequencies obtained based on the fully cracked moment of inertia were always lower than those based on the partially cracked effective moment of inertia. As stated in the criteria for the reevaluation of concrete masonry walls, the calculation of the wall frequency based on the partially cracked behavior was necessary for conservatism when the frequency for the fully cracked section was on the low frequency side of the response spectral peak.

The indication of OBE (Operating Basis Earthquake) or DBE (Design Basis Earthquake) for some of the frequencies was for clarification purposes since the frequency is affected by the effective moment of inertia, which in turn is affected by the seismic load level. This is further explained

in the response to Question No. 6b. When the frequency in the table was indicated as OBE, it implies that the load case involving OBE was a controlling load case.

QUESTION 2:

With reference to Section 6.1.4, Appendix A (2), justify using the average acceleration rather than the envelope of the response spectra for walls supported by two floors.

Response:

The evaluation herein demonstrates that the use of the average floor acceleration response spectra to calculate the response of the wall panel is appropriate. For the purpose of this evaluation, the seismic response of a simply-supported, uniform beam simulating a strip of the wall panel with unit width is considered, as shown in Figure 1.

The equation of motion of an undamped, simply-supported beam can be written in terms of the total displacement with respect to some fixed reference axis as:

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Where m and EI are the mass density and flexural rigidity of the beam. Denote the seismic excitations at the ends of the beam as U_a and U_b. Then the total displacement u(x,t) can be expressed in terms of the two seismic motions and the relative displacement to the seismic motions as:

$$u(x,t) = (x/L)\, U_b + (1 - x/L)\, U_a + r(x,t) \qquad (2)$$

Where L is the length of the beam. The relation expressed by the above equation is shown in Figure 2. The relative displacement r(x,t) needs to satisfy the following simply-supported conditions:

$$r(o,t) = r(L,t) = 0 \tag{3}$$

$$\left.\frac{\partial^2 r}{\partial x^2}\right|_{x=o} = \left.\frac{\partial^2 r}{\partial x^2}\right|_{x=L} = 0 \tag{4}$$

Substitute Equation 2 into Equation 1, the equation of motion in terms of relative displacement r(x,t) can be expressed as:

$$m \frac{\partial^2 r}{\partial t^2} + EI \frac{\partial^4 r}{\partial x^4} = -m(x/L)\ddot{U}_b - m(1 - x/L)\ddot{U}_a \tag{5}$$

The eigen-function solutions for the homogeneous equation associated with Equation 5 that satisfy the boundary conditions specified by Equation 3 and 4 are:

$$\sin \frac{n\pi x}{L}, \; n = 1, 2, 3, \ldots,$$

and the corresponding frequencies of vibration are:

$$\omega_n = n^2\pi^2 \left[\frac{EI}{mL^4} \right]^{\frac{1}{2}} \quad n = 1, 2, 3, \ldots \tag{6}$$

So, the solution of Equation 5 can be expressed as:

$$r(x,t) = \sum_{n=1}^{\infty} a_n(t) \sin \frac{n\pi x}{L} \tag{7}$$

Substitute Equation 7 into Equation 5, and multiply the latter by sin $\frac{n\pi x}{L}$, and then integrate it with respect to x over the full length of the beam, the equation of motion can be transformed into modal equations of motion as:

$$\ddot{a}_n + \omega^2{}_n a_n = \Gamma_n \frac{(\ddot{U}_a + \ddot{U}_b)}{2} \qquad n = 1, 3, 5, \ldots \qquad (8a)$$

and

$$\ddot{a}_n + \omega^2{}_n a_n = \Gamma_n \frac{(\ddot{U}_a - \ddot{U}_b)}{2} \qquad n = 2, 4, 6, \ldots \qquad (8b)$$

where Γ_n = participation factor

$$= \frac{4}{n\pi} \qquad (9)$$

If damping in the form of modal damping ratio is included, Equations 8a and 8b becomes:

$$\ddot{a}_n + 2\xi_n\omega_n\dot{a}_n + \omega^2{}_n a_n = \Gamma_n \frac{(\ddot{U}_a + \ddot{U}_b)}{2} \qquad n = 1, 3, 5, \ldots \qquad (10a)$$

and

$$\ddot{a}_n + 2\xi_n\omega_n\dot{a}_n + \omega^2{}_n a_n = \Gamma_n \frac{(\ddot{U}_a - \ddot{U}_b)}{2} \qquad n = 2, 4, 6, \ldots \qquad (10b)$$

Where ξ_n is the damping ratio of the n^{th} mode.

Equation 10a means that the odd-number modes which are symmetrical about the mid-span of the beam wil! be excited by the average of the two seismic excitations; while equation 10b means that the even-number modes which are antisymmetrical about the mid-span of the beam will be excited by half of the difference between the two seismic excitations..

Expressing the maximum modal displacement respnse in equations 10a and 10b in terms of absolute acceleration response spectra gives:

$$|a_n|_{max} \leq |\Gamma n| \quad \left[\frac{S_a(\xi_n, \omega_n)}{2\omega^2_n} + \frac{S_b(\xi_n, \omega_n)}{2\omega^2_n} \right]$$

$$\leq \frac{4mL^4}{n^5\pi^5 EI} \quad \left[\frac{S_a(\xi_n, \omega_n) + S_b(\xi_n, \omega_n)}{2} \right] \qquad (11)$$

$$n = 1, 2, 3, \ldots$$

This illustrates that the use of the average of two floor acceleration response spectra to calculate the modal response of a wall panel is appropriate.

QUESTION 3:

With reference to Section 5.8 (2), justify neglecting out-of-plane interstory drift in the analysis and explain whether the predicted in-plane interstory drift of 0.0006 inch/ft of height applies to confined or unconfined walls.

Response:

As stated in the report (Reference 1), out-of-plane stresses due to interstory displacements are insignificant due to the flexibility of the wall in the out-of-plane direction. This is particularly true if the rotational stiffness of the floor-wall connection is taken into consideration. In order to demonstrate that the above statement is valid, a parametric study was performed using the same maximum interstory draft of 0.0006 inch/ft of height as for the in-plane drift study. In this study, the top and bottoms of the wall were conservatively assumed to be fixed, even though a 100% fixity cannot be actually attained. Listed below is the result of the analysis which shows that the resulting stresses are negligible compared with the capacity of the masonry walls.

H (height = 19'-0"

t(thickness) inches	fs (steel stress) psi	fm (masonry compressive stress) psi	fv (shear stress) psi
8	111.7	3.2	0
12	185.3	4.07	0
18	507.0	6.7	0
24	751.7	8.1	0.13
30	918.1	9.0	0.15
36	1168.6	10.2	0.2

The predicted interstory drift of 0.0006 inch/ft of height was applied to confined walls in the analysis.

QUESTION 4:

With reference to Section 6.1.2, Appendix(2), provide sample calculations to show that analysis using only the fundamental mode is adequate and is comparable to a multimode analysis.

Response:

Descriptions and results of the analyses of two sample walls are provided herein to show that the analysis using the uniform inertia load associated with only the fundamental mode is adequate and is comparable to a multimode analysis.

The first sample wall is a multiwythe cantilever wall of 1'-6" thick and 8'-0" high. This wall was analyzed as two cantilever beams tied by springs in one-way action. A typical mathematical model is shown in Figure 5, A dynamic analysis as described in the response to Question 6c was performed to obtain moments and shears of the wall due to seismic loads in the out-of-plane direction.

The second sample wall is a rectangular wall of 1'-6" thick supported at four sides with dimension 15'-0" by 22'-0". This wall was modeled as a plate with four simply-supported edges. The mathematical model is shown in Figure 6. A dynamic analysis using Computer Program STARDYNE was performed. A modal analysis was first performed to determine mode shapes, participation factors, and natural frequencies of the wall, which were then used for a spectral response analysis to determine the resulting moments and shears in the wall.

Resulting moments and shears based on modal responses of the above two walls for the fundamental mode as well as the multimode combination using SRSS procedures are listed on the first two lines in the following table. Another set of moments and shears for these two walls based on the uniform inertia load associated with only the fundamental mode is also listed on Line 4 in the same table for comparison.

		Cantilever Wall (at El. 335'-0")		Four Side Simply-Supported Wall (At El. 386'-0")	
		Moment (at base) in-lb/ft	Shear (at base) lb/ft	Moment (at center in-lb/ft	Shear (at base) lb/ft
(1)	Fundamental Mode (use modal responses)	24,128	344	38,188	1061
(2)	SRSS Combination (use modal resposes)	24,165	351	38,246	1066
(3)	Difference	0.15%	1.97%	0.15%	0.47%
(4)	Fundamental Mode (use uniform inertia load)	27,042	563	39,477	1165

As shown in the table, the resulting moments and shears (Line 1) from the fundamental mode are only 0.15% and 1.97%, respectively, less than those (Line 2) from the SRSS combination. It can also be seen that the values

on Line 2 are even smaller than those (Line 4) from the fundamental mode using the uniform inertia load. The comparison demonstrates that the analysis using the uniform inertia load associated with only the fundamental mode is adequate and conservative.

QUESTION 5:

With reference to the cover letter and Table 5 of the attachment (2), provide a description of the bracing system installed for two Unit 2 cantilever walls and indicate whether out-of-plane drift effects were included in the analysis.

Response:

The bracing systems installed for two Unit 2 cantilever blockwalls are essentially a set of structural steel angles with steel bearing plates at both ends. These angle members serve as lateral supports to resist the out-of-plane inertia force generated by the cantilever blockwalls during a seismic event. One end of the angle member attaches to the upper portion of the blockwall by means of through-bolts. The other end is supported by an adjacent reinforced concrete wall using concrete expansion anchors. This reinforced concrete wall was evaluated and found to be capable of resisting the additional loads transmitted by the bracing system. Locations and configuration of these bracing systems are shown in Figures 3 and 4.

As explained in the responses to Question 3, the out-of-plane drift effects are insignificant, and therefore, have not been included in the analysis.

QUESTION 6:

With reference to Table 5 (2), briefly describe the techniques used for (a) verification by curves, (b) effective inertia analysis, and (c) dynamic analysis. Also clarify whether pipe reactions due to thermal expansion are considered in the analysis.

Response:

The techniques used for the blockwall reevaluation as requested are as follows:

a. Verification by Curves

The purpose of the design curve was to verify the adequacy of the seismic Category 1 blockwalls by categorizing them into several groups with similar parameters such as boundary conditions, floor elevations, wall thickness and height. These curves were developed using the same methods and criteria as described in Reference 1. The following type of curves have been generated to account for various applicable parameters.

1. Vertically Spanned Beam

The curves in this category are applicable for walls acting as a simply supported beam spanned vertically. One set of curves for each different wall thickness has been generated. Each set consists of several curves for walls located on different floor elevations. The curves were plotted for various wall heights versus reinforcing steel and masonry stresses. A typical curve

is shown in Figure 7. Walls with a proper aspect ratio, i.e., the length is larger than the height, were entered into an appropriate curve for evaluation.

2. Horizontally Spanned Beam

The curves in this category are applicable for walls acting as a simply supported beam spanned horizontally. Similar sets of curves as for Item 1 above have been generated. Walls with a proper aspect ratio, i.e., the wall height is larger than the length, were entered into an appropriate curve for evaluation.

3. Plate

The curves in this category are for walls with the boundary conditions, i.e., a fixed base, a free top edge and two hinged sides, for which the curves for Items 1 and 2 above were not appropriate. Similar sets of curves as for Item 1 above have been generated. Walls with the above mentioned boundary conditions were entered into an appropriate curve for evaluation.

The wall was considered to be adequate if it falls within the allowable range as shown on the appropriate curves. Otherwise, a more refined manual or computer method would be used to evaluate the adequacy of the wall.

b. Effective Inertia Analysis

In the structural analysis of concrete masonry walls, cracked section properties were generally considered in computing the moment of inertia of walls. For some walls, the effective moment of inertia was used to account for the partially cracked condition.

The method of computing the effective moment of inertia (Ie) for blockwalls was described in detail in Appendix A (Reference 1) which is in essence the same technique as set forth in Sections 9.5.2, ACI 318 (Reference 2). It should be noted that Ie is a function of the appliced moment (Ma) which is mainly due to the inertia force of the wall in this case. Since the inertia force is a frequency dependent parameter, and the frequency largely depends on the value of Ie, an iterative technique was employed to determine the correct Ie for the walls in which the partially cracked condition is a controlling factor. The iterative technique is outlined as follows:

1. Use an assumed moment of inertia Ia (lies between the values of Ig and Icr) to compute the wall frequency and the applied moment Ma.

2. Compute Ie with the calculated Ma from Step 1.

3. Compare Ie with Ia used in Step 1. If they match closely, Ia could be used as a correct Ie for the evaluation of the particular blockwall. If they do not match, another iterative step would be performed until the assumed Ia matches closely with the corresponding Ie.

c. Dynamic Analysis

As described in Section 5.6 (Reference 1), a dynamic analysis was used to verify those walls which could not be shown to be adequate by a simplified manual analysis nor by a static computer analysis. The dynamic analysis described herein was primarily for multiwythe cantilever walls.

Since the shear strength of mortar and grout at the collar joint was conservatively assumed to be zero for all multiwythe walls for the ANO-2 plant, a non-composite section was considered in the analysis. Two exterior wythes of the wall were modeled as two separate flexural members tied together by tie bars.

These tie bars were modeled as springs connected to the lumped masses of each flexural member. A typical mathematical model for the multiwythe cantilever wall is shown in Figure 5.

A modal analysis was performed to determine mode shapes, participation factors and natural frequencies for the wall. A spectral response analysis considering multimode responses was then

preformed to determine the resulting moments and shears in the wall based on the appropriate floor response spectrum as input. The modal responses were combined using the SRSS procedure (the square root of the sum of the squares). The resulting moments and shears were used for the evaluation of the adequacy of the wall.

Significant pipe thermal reactions have been considered in the blockwall analysis.

QUESTION 7:

Provide more information on seismic analysis in different directions and explain how the equipment weights and pipe weights were accounted for.

Response:

One horizontal and one vertical seismic inertia load was considered simulatenously in the analysis of concrete blockwalls. Since all concrete blockwalls in ANO plants were not load bearing walls nor shear walls, stresses in the in-plane direction due to seismic load were insignificant. Therefore, seismic load due to the horizontal earthquake component in the out-of-plane direction combined with the vertical seismic load using the absolute sum method was considered in the evaluation.

Weights of the equipment and pipes attached to walls were generally considered as equivalent uniform loads distributed over the whole wall except for seismic Class 1 pipes. The wall weight was generally increased by 10% to represent the weight of all these attachments in determining the inertia force in the out-of-plane direction during a seismic event. Reactions of seismic Class 1 pipe supports were considered as concentrated loads applied to the wall in addition to the 10% allowance. Calculations on several sample walls indicated that the 10% allowance for attachment weights was adequate. For a few walls, the actual weights of the attachments were used in lieu of the 10% increase.

QUESTION 8:

With reference to Section 5.0, Appendix A (2), provide the values for allowable stresses in axial compression, bearing, tension normal to the bed joint, and tension parallel to the bed joint.

Response:

The values for allowable stresses as requested are listed as follows:

1. Allowable axial compression stress = 0.22 f'm = 330 psi (This value was to be multiplied by $[1- (\frac{h}{40t})^3]$ if the wall had a significant vertical load.) where,

   f'm = Masonry ultimate compressive strength = 1500 psi

   h = height of wall

   t = thickness of wall

2. Allowable hearing stress = 0.25 f'm = 375 psi (on full area)

   = 0.375 f'm = 563 psi (on 1/3 area or less)

3. Allowable tension stress normal to the bed joint = 1.0 $(m_o)^{\frac{1}{2}}$ (40 psi max.) = 40 psi where,

   m_o = Mortar compressive strength = 2000 psi

4. Allowable tension stress parallel to the bed joint = 1.5 $(m_o)^{\frac{1}{2}}$ (80 psi maximum = 67 psi

It should be noted that the values for tension normal to the bed joint and tension parallel to the bed joint as listed above are for unreinforced masonry only. As stated in Referenced 1, all Seismic Category 1 masonry blockwalls for the ANO-2 plant are reinforced masonry. These values were only used to indicate if the wall is cracked in Table 6 (Reference 1) have been further evaluated using the cracked section moment of inertia. The resulting reinforcing steel tensile stresses and masonry compressive stresses were found to be within the allowable limits.

QUESTION 9:

With reference to Section 5.2.1 of Appendix A (2), justify the proposed increase factor of 1.67 for shear, bond, tension normal to the bed joint, and tension parallel to the bed joint. The SEB criteria (3) suggest an increase factor of 1.3 for masonry shear, 1.5 for masonry tension parallel to the bed joint, and 1.3 for unreinforced masonry tension normal to the bed joint.

Response:

The factor of 1.67 for shear bond stresses was chosen in January 1981 (Reference 1) prior to the SEB criteria being published in July 1981 (Reference 3). Code allowable stresses for masonry, shear, bond and tension normal or parallel to the bed joint were increased by a factor of 1.67 for load combinations involving abnormal and/or extreme environmental conditions which are credible but highly improbable. Since ACI-531 code allowable stresses (Reference 4, Chapter 10.1 of commentary) are generally associated with a factor of safety of 3, the 1.67 increase still provides a factor of safety against failure of 1.8 (3/1.67).

QUESTION 10:

Indicate the present status of walls which were inaccessible and hence excluded from the original field survey.

Response:

Inaccessible walls listed in Reference 1 will be field surveyed were practical during the next refueling outage. ALARA reviews will be performed to insure that radiation exposures are minimized and extremely high radiation areas will not be entered only for this survey. Drawing surveys will be substituted for these walls.

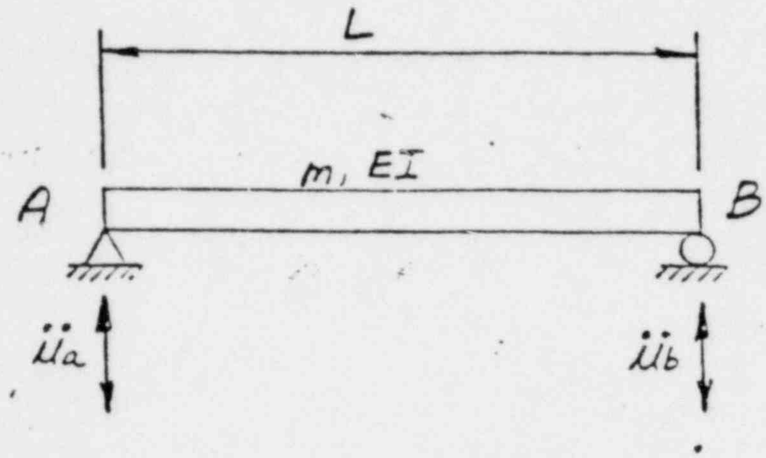
References

1. AP&L letter dated January 29, 1981 "Response to IE Bulletin 80-11 "Masonry Wall Design".

2. ACI Standard 318-77, Building Code for Reinforced Concrete (ACI 318-77).

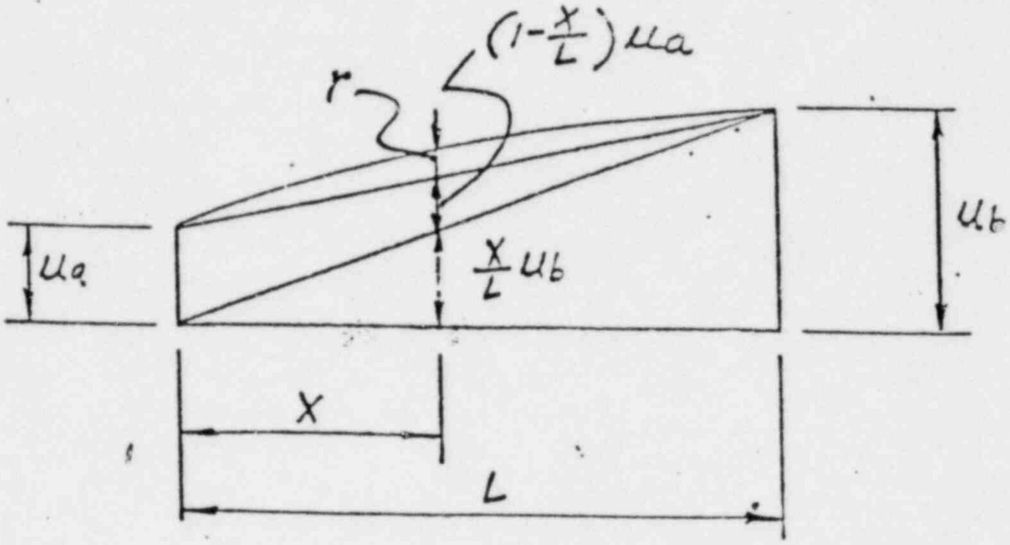
3. Standard Review Plan, Section 3.8.4, Appendix A, "Interim Criteria for Safety-Related Masonry Wall Evaluation", NRC, July 1981.

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



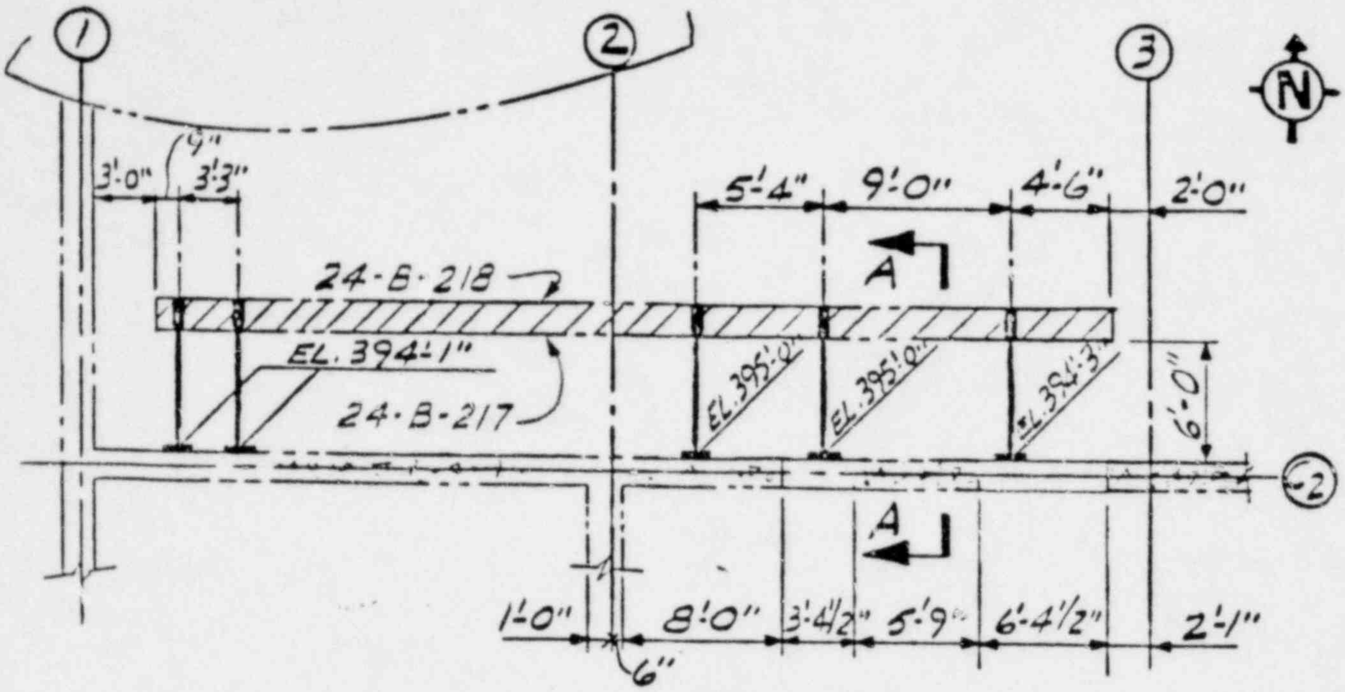
IDEALIZED SIMPLY-SUPPORTED UNIFORM BEAM

FIGURE 1

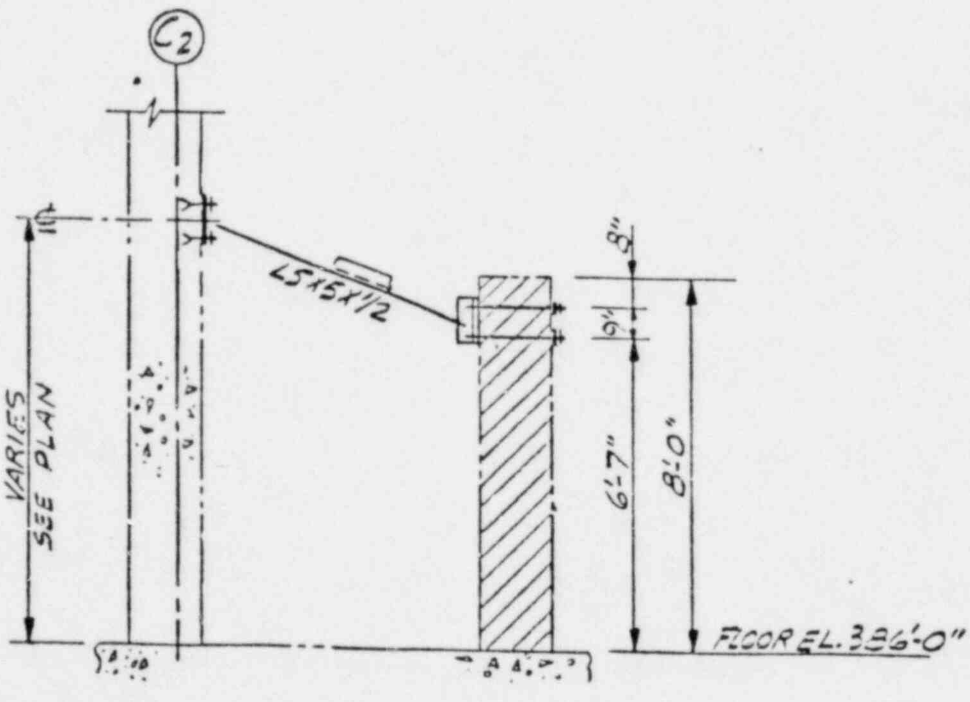


RELATION BETWEEN SEISMIC EXCITATION
AND RELATIVE DISPLACEMENT

FIGURE 2

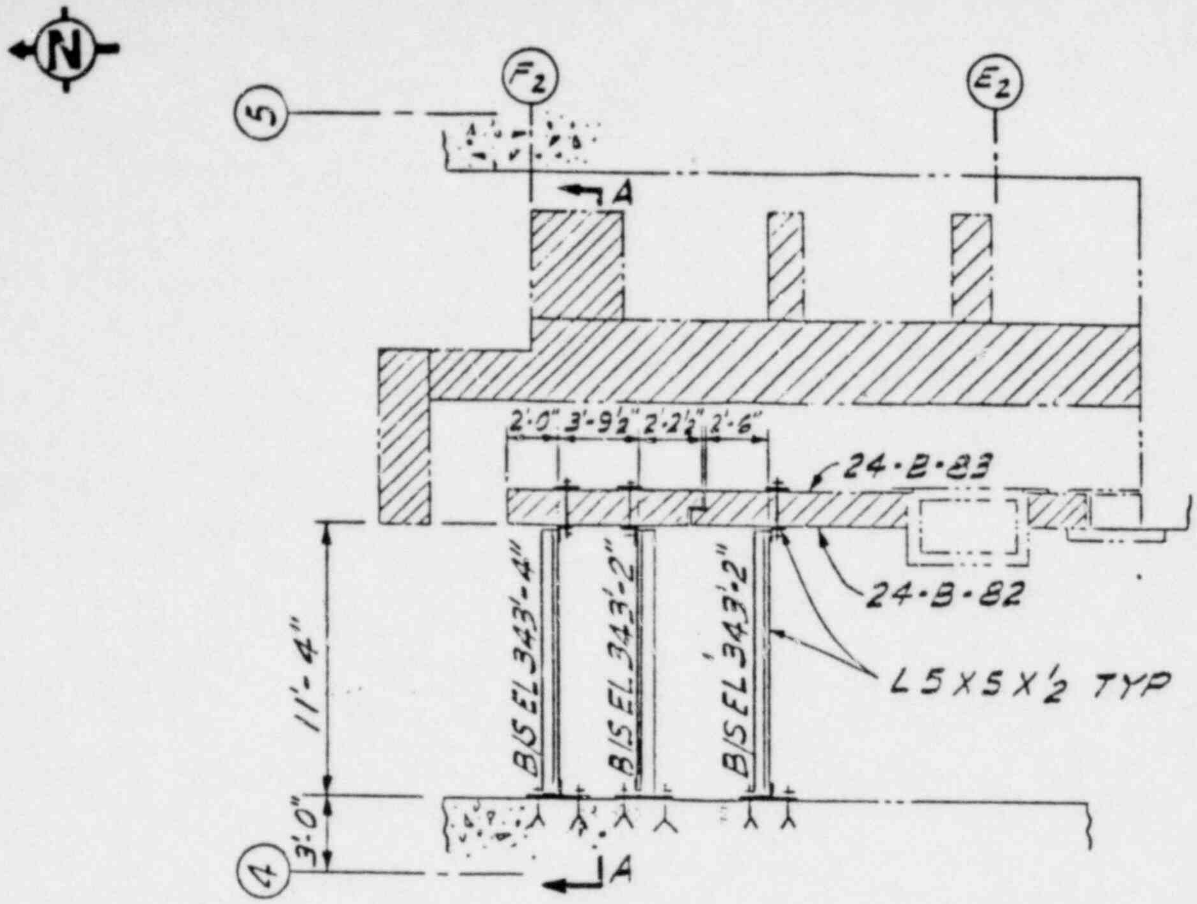


PLAN @ EL. 386'-0"

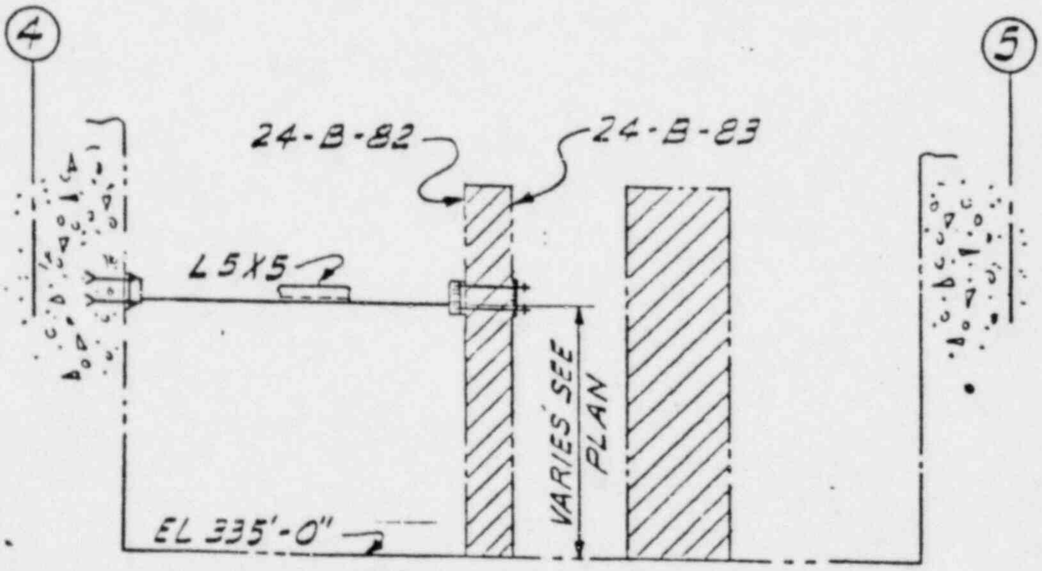


SECTION A-A

FIGURE 3

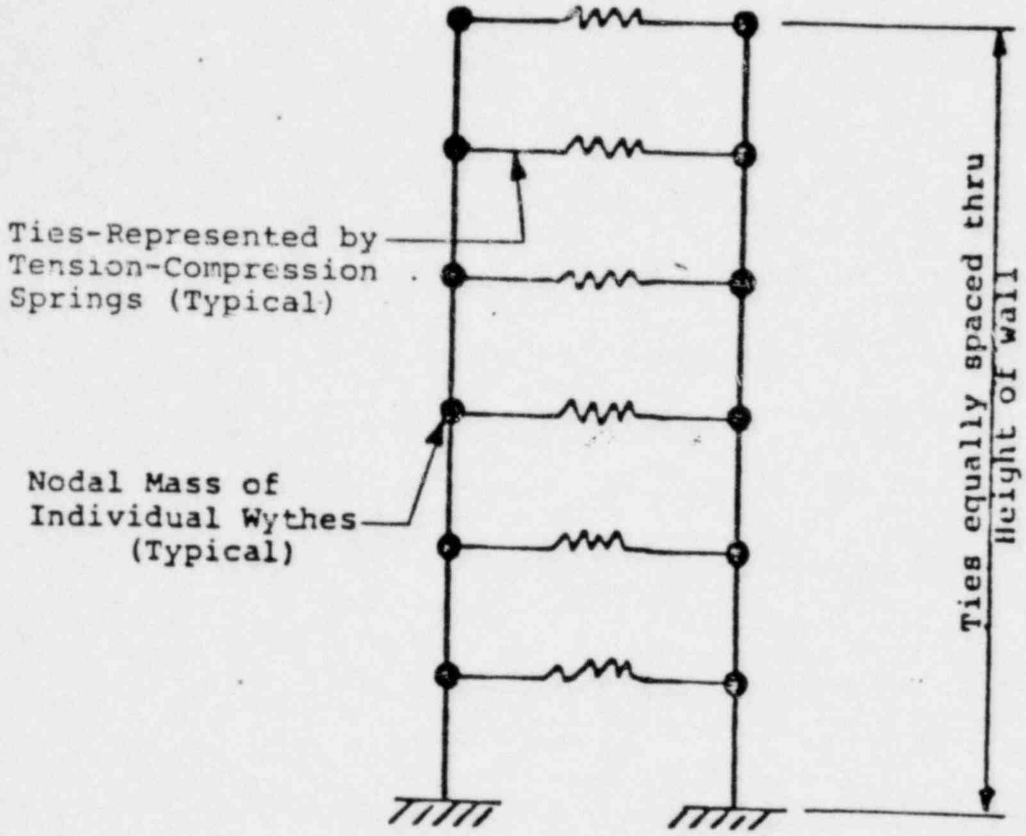


PLAN @ EL 335'-0"



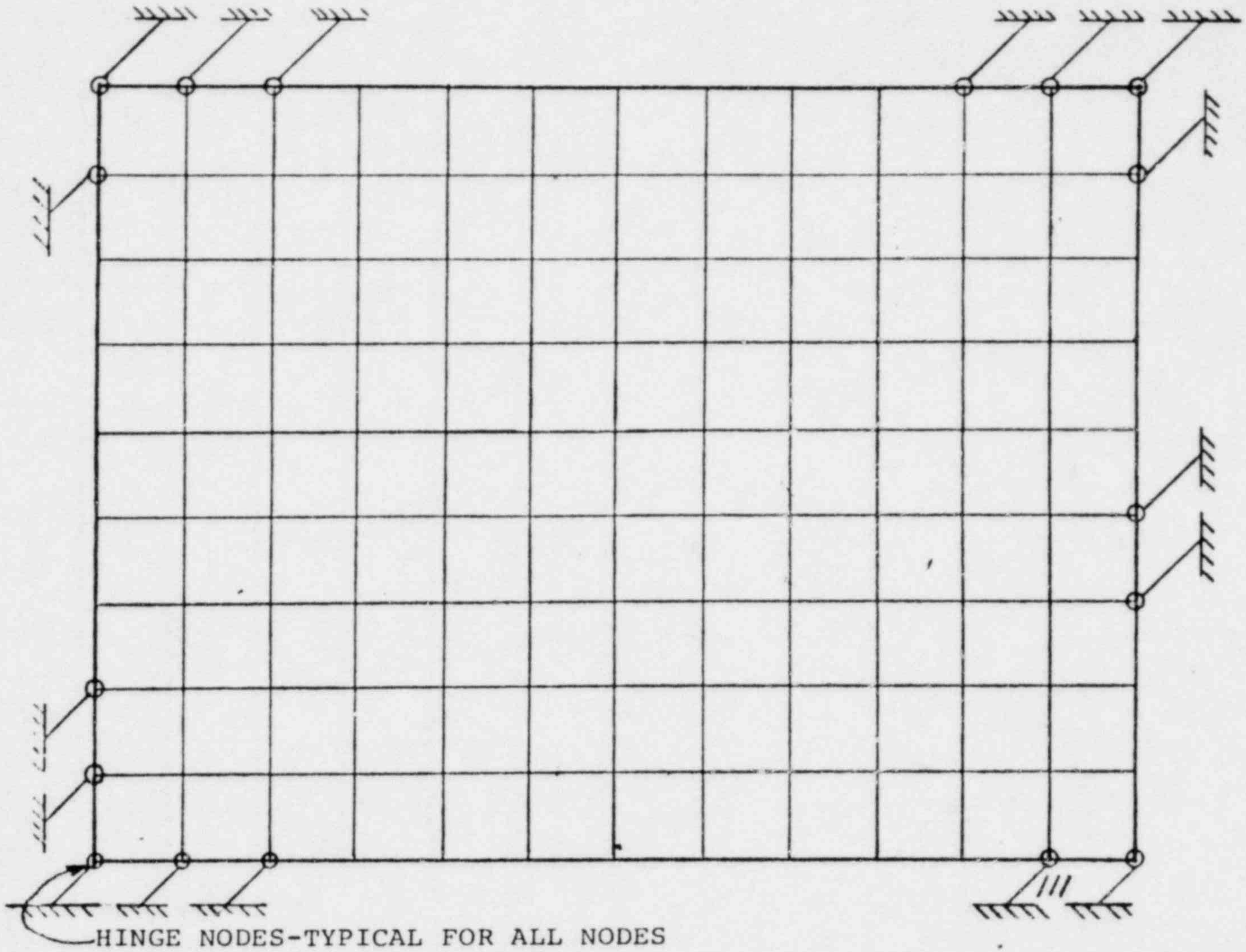
SECTION A-A

FIGURE 4



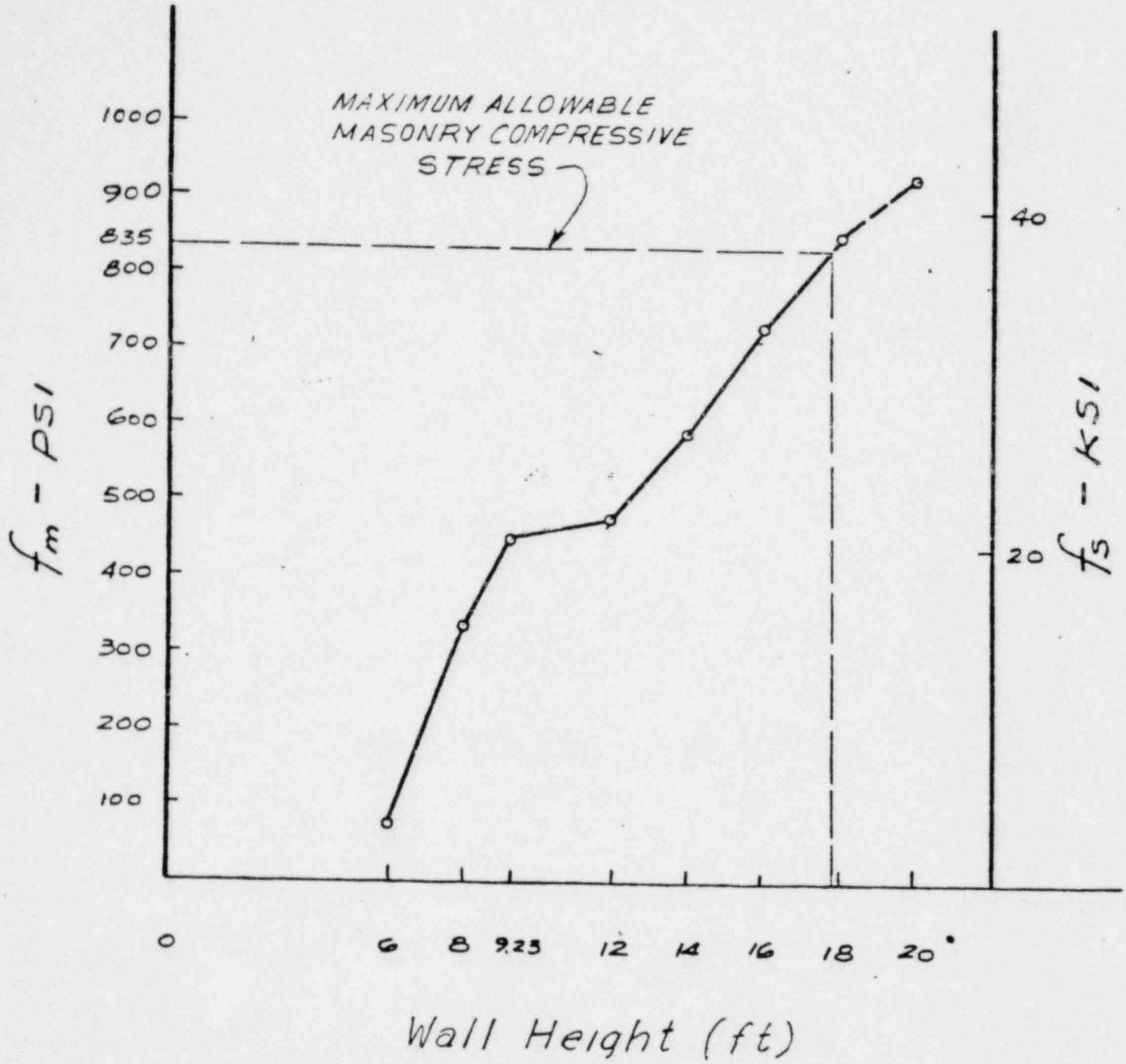
TYPICAL MATHEMATICAL MODEL
DYNAMIC ANALYSIS (MULTI-WYTHE CANTILEVER WALL)

FIGURE 5



MATHEMATICAL MODEL
DYNAMIC ANALYSIS (PLATE)

FIGURE 6



TYPICAL CURVE
VERTICALLY SPANNED BEAM

FIGURE 7