BOSTON EDISON COMPANY
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March 22, 1982

BECo. Ltr. #82-90

Mr. Domenic B. Vassallo, Chief
Operating Reactors Branch #2
Division of Licensing
Office of Nuclear Reactor Regulation
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555

Docket Number 50-293
License No. DPR-35

Ref. (a): T. A. Ippolito to A. V. Morisi 1/7/82

Additional Information on Block Walls

Dear Sir:

Enclosed please find our response to questions concerning block walls requested by your letter of January 7, 1982 (Reference (a)).

NRC Bulletin 80-11 requires the licensee to submit a written report upon completion of the re-evaluation program. This report must include the following:

1. description of the masonry walls,
2. description of the construction practices employed in the construction of the walls,
3. re-evaluation criteria and a detailed justification.

Items 2 and 3 are addressed in response to Questions 6 and 4, respectively, of Reference (a). A description of the masonry walls and associated system as well as the results of the re-evaluation (required in the 60 day response) program will be provided under separate cover.

We trust that this response sufficiently answers your concerns. If you need further information or clarification do not hesitate to contact us.

Very truly yours,

W H Deacon

W. H. Deacon
Acting Manager
Nuclear Operations Support

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1/1

ATTACHMENT

<u>References</u>:

R1. "Recommended Guidelines for the Reassessment of Safety Related Concrete Masonry Walls", Prepared by Owners and Engineering Firms Informal Group on Concrete Masonry Walls, October 6, 1980.

R2. Final Safety Analysis Report, Unit 1, Pilgrim Station No. 600.

R3. Schneider, R.R., "Shear in Concrete Masonry Piers", California State Polytechnic College, Pomona, California, 1959.

R4. Englekirk, R. E., and G. C. Hart, "Seismic Design of Concrete Masonry Shear Walls", ASCE National Convention, Hollywood, Florida, October 1980.

R5. ACI Standard, "Building Code Requirements for Concrete Masonry Structures", (ACI 531-79).

R6. ACI Committee 531, "Concrete Masonry Structural - Design and Construction," <u>Journal of the American Concrete Institute</u>, Proceedings Vol. 67, No. 5, May 1970, pages 380-403.

R7. "Specification for Furnishing, Delivery and Installation for of Concrete Unit Masonry for Unit No. 1, Pilgrim Station No. 600, Boston Edison Company", Rev. 1, Bechtel Corporation, San Francisco, California, 1972.

R8. "Work Instruction for Testing of Masonry Walls", Pilgrim Nuclear Power Station, Boston Edison Company, WI-6, Rev. 2, November 25, 1981.

R9. Construction Drawings, Unit No. 1, Pilgrim Station No. 600, California, Edison Company, Bechtel Corporation, San Francisco

R10. Omote, Y., Mayes, R. L., Chen, S. W. J., R. W. Clough, "A Literature Survey-Transverse Strength of Masonry Walls", EERC-77/07, March 1977.

R11. "Cygna Generic Calculation Set No. G18000".

R12. Boston Edison Letter to Mr. Boyce H. Grier, NRC (BECo. Ltr. #81-58) March 18, 1981

1. With respect to the equivalent static analysis, the Licensee should provide justification for the use of the amplification factor of 1.3 to account for multi-mode effects.

This question refers to the level 1 analysis described in Section 4.2.1 of the revised design criteria. It was felt that the factor of 1.3 is conservative for the geometric characteristics of concrete block walls. A factor of 1.05 was recommended by the Informal Owners Group on Concrete Masonry Walls (R1). The pertinent section of that report is appended to this answer.

Table 1 provides a summary of multi-mode effects for Pilgrim walls evaluated by level 2 analysis as described in Section 4.2.2 of the revised criteria. The table compares the moments from the fundamental mode with those from the 12 mode SRSS combination. The walls considered represent a variety of geometries and boundary conditions. The range of the critical parameters are:

Aspect ratio:	$0.56 < H/L < 1.65$
Thickness:	$8'' < t < 42''$
Openings	0 < No. Openings < 5
Fund. Frequency	$7.2 < f_1 < 28.6$ Hz

The maximum change in moment due to higher modes occurs for the x-direction moment in wall 195.6. The amplification factor is 1.05.

TABLE 1

Wall	(1) Aspect Ratio	Thick	Opens	(2) Bound L-T-R-B	OBE or SSE	f_1	M_{xx} Mode 1	M_{xx} 12 f	M_{yy} Mode 1	M_{yy} 12 f	(3) % X	(3) % Y
62.3	1.33	30	0	PFFP	OBE	19.7	237	238	295	295	99.6	100
64.7A	1.20	42	0	PPPP	OBE	15.2	1639	1639	1281	1281	100	100
					SSE	15.2	----	2453	----	1918	----	----
64.7A	1.20	42	0	PFPP	OBE	10.2	3652	3658	731	747	99.8	97.9
					SSE	10.2	5342	5351	1069	1101	99.8	97.1
198.0	1.61	8	1	FFPP	SSE	16.6	62	62	65	65	100	100
65.4	0.99	42	2	PPPP	OBE	11.6	1509	1510	511	522	99.9	97.9
					SSE	11.6	2208	2208	747	765	100	97.6
195.6	1.65	18	0	PPFP	OBE	19.7	70	73	277	277	95.9	100
					SSE	19.7	114	120	452	452	95.0	100
				FPFP	SSE	8.0	361	361	1967	1967	100	100
195.16	0.95	8	1	PPPP	OBE	28.6	42	42	70	70	100	100

(1) Aspect Ratio - Height/Length

(2) Boundary Condtion on Left, Top, Right and Bottom of Wall. F = Free, P = Pinned

(3) % = ($M_{mode\ 1}/M_{12f}$) x 100.

TABLE 1 (con't.)

Wall	(1) Aspect Ratio	Thick	Opens	(2) Bound L-T-R-B	OBE or SSE	f_1	M_{xx} Mode 1	M_{xx} 12 f	M_{yy} Mode 1	M_{yy} 12 f	(3) % X	(3) % Y
191.50	0.59	18	0	PPFP	OBE	17.7	80	82	307	307	97.6	100
185.1 (Sec. I)	1.22	8	3	PPPP	OBE	19.1	100	100	165	165	100	100
					SSE	19.1	157	157	260	260	100	100
185.1 (Sec. I)	1.22	8	3	PPFP	OBE	18.6	130	130	132	132	100	100
					SSE	18.6	205	205	208	208	100	100
185.1 (Sec.II)	1.22	24	1	PPPP	OBE	19.1	1092	1093	341	341	99.9	100
					SSE	19.1	1721	1721	537	537	100	100
185.1 (Sec.II)	1.22	24	1	PPFP	OBE	18.6	1135	1135	333	333	100	100
					SSE	18.6	1793	1793	526	526	100	100
191.25	0.60	18	2	PPPP	OBE	6.9	435	441	1790	1793	98.6	99.8
					SSE	7.8	1324	1328	1604	1605	99.7	99.9
209.3	1.17	12	5	PPPP	OBE	9.7	1340	1335	1255	1319	100.4	95.1

(1) Aspect Ratio - Height/Length

(2) Boundary Condtion on Left, Top, Right and Bottom of Wall. F = Free, P = Pinned

(3) % = ($M_{mode\ 1}/M_{12f}$) x 100.

TABLE 1 (con't.)

Wall	(1) Aspect Ratio	Thick	Opens	(2) Bound L-T-R-B	OBE or SSE	f_1	M_{xx} Mode 1	M_{xx} 12 f	M_{yy} Mode 1	M_{yy} 12 f	(3) % X	(3) % Y
191.53	0.56	18	0	PPPP	OBE	19.2	134	136	280	281	98.5	99.6
					SSE	19.2	212	215	442	443	98.6	99.8
191.53	0.56	18	0	FPFP	OBE	14.5	83	83	484	485	100	99.8
					SSE	14.5	125	126	729	729	99.2	100
191.53	0.56	18	0	PFPP	OBE	7.2	1643	1643	441	445	100	99.1
					SSE	7.2	2084	2085	559	568	100	98.4

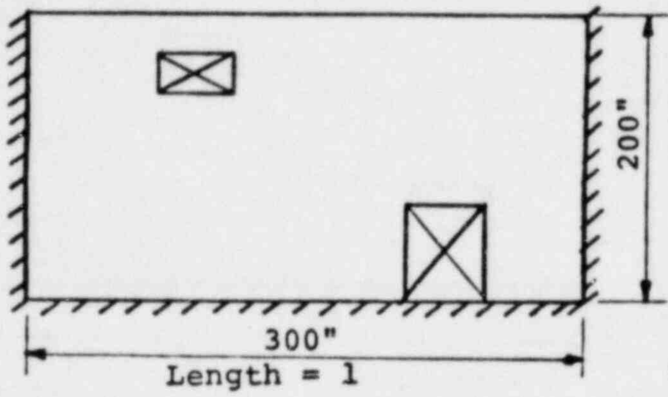
(1) Aspect Ratio - Height/Length

(2) Boundary Condtion on Left, Top, Right and Bottom of Wall. F = Free, P = Pinned

(3) % = $(M_{mode\ 1}/M_{12f}) \times 100$.

TABLE 1 (con't)

The example below is provided to clarify the column headings.



Heading	Example
Aspect Ratio = H/L Opens = # of openings	AR = 200/300 = 0.667 Opens = 2
Boundary = Boundary Condition on each edge clockwise starting at left P = Pinned, F = Free	Bound = PFPP

f_1 = Fundamental Frequency

%X, %Y = Ratio of Mode 1 Moment To 12f Moment (as a percent)

= M(mode 1)/M(12mode) x 100

ATTACHMENT 1

(To Question #1)

Below is an excerpt from Reference R1 regarding the use of a 1.05 factor to represent the multi-mode effects.

"Modal Participation

It is recommended that the seismic acceleration for both reinforced and non-reinforced concrete masonry walls with rigid supports be increased by a factor equal to 1.05 to account for the participation of higher modes for out-of-plane flexural calculations. When the lowest fundamental frequency determined in Item B is greater than 33 Hertz, this factor is not required. A modal analysis may be used to justify a lower value between 1.0 and 1.05."

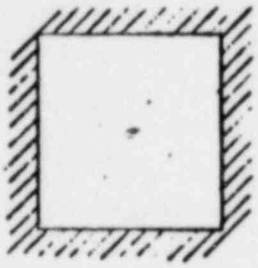
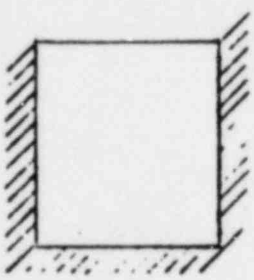
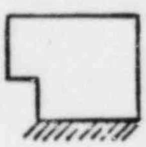
The same reference also supplies the following supporting commentary.

"Modal Participation

The committee conducted a parametric study to determine the effect of the participation of higher modes. For each of the three boundary conditions referenced in Exhibit 3.1, and for a cantilevered wall with an aspect ratio equal to 1.0, the modal displacements for the first eight modes were compared to the SRSS displacement for wall panels with the full E_mI (D) and a reduced E_mI (D) within the third segment of the panels. The results are tabulated in Exhibit 3.3 and indicate that the 99% of the displacement is contributed by the first mode. In addition to considering displacement, the committee reviewed the effect of modal participation on moments for the wall panels indicated in Exhibit 3.4. It was again demonstrated that the first mode moment contributed more than 99% of the SRSS moment for an eight mode analysis. For both the displacement and the moment study, an acceleration of 1.0g applied over the entire frequency range was used as the input spectra.

The finite element program used did not provide shear values within the elements nor at the supports. It is the Committee's opinion, however, that the first mode would, likewise, contribute at least 99% of the shear in the wall and at the supports."

Exhibit 3.4 from Reference R1 has been included on the following pages.

Support Case	h/L	Modes	Contribution from First Mode Moment for SRSS of All Mode Moments	
	1.0	1 thru 8	99.30%	
	.67			
	.54	1 thru 8	99.86%	
	1.0	1 thru 5	99.82%	
	0.67	1 thru 5	99.78%	
	0.50	1 thru 5	99.70%	
 FIXED	1.0	1 thru 8	99.75%	R1

COMPARISON OF FIRST MODE MOMENT TO SRSS MOMENT

2. With regard to the availability of the amplified response spectra, the Licensee stated that "if no ARS is available, earthquake loads may be evaluated based on a rigid range acceleration value as determined by the original building seismic analysis. In this case the wall must be shown to have a fundamental frequency greater than the rigid range cutoff frequency value." The Licensee should indicate how the rigid range acceleration and cutoff frequency could be determined.

Amplified response spectra are available for all masonry walls evaluated at Pilgrim. The revised criteria does not contain the statement quoted in question 2.

3. For the permissible strain of non-shear walls, the Licensee should provide the technical basis for using 0.1% and 0.01% for confined and unconfined walls, respectively.

This question refers to Section 4.2.4 of the revised design criteria. The Pilgrim FSAR (R2) requires that one horizontal seismic excitation be evaluated in conjunction with vertical excitation (2-D earthquake criteria). For non-shear walls, the critical horizontal direction is out-of-plane. Thus, the in-plane criteria was developed to limit structure imposed distortions to prevent excessive cracking and degradation which might adversely impact the ability to carry the non-concurrent out-of-plane inertia loading.

For confined walls, a series of shear tests on concrete masonry piers were performed under the direction of Professor Robert Schneider at California State Polytechnic College at Pomona, California for Masonry Research of Los Angeles (R3). These tests were performed on masonry piers constructed using typical methods and materials: 8 inch ASTM type C-90 block, type S mortar. The piers were built of varying size and detail so that the effect on ultimate panel strength of parameters such as steel reinforcement, significant axial stress, type of panel restraint, and grouted vs. non-grouted block could be evaluated.

It should be kept in mind that the intent of these tests was to measure the ultimate capacity of these panels under in-plane shear and not elastic or early inelastic behavior. However, since the ultimate capacity of the piers was measured against the load carrying capability at first visible cracking, ample data at first cracking is available to form a basis for the criteria utilized.

The test program results were tabulated calculating the average shear stress, the shear distortion, and the wall height to width ratio.

The average shear stress, V/A, is determined from the first crack as tabulated in Table 5 of the test report. In some cases, the first crack may be a "moment crack" rather than a "shear crack". Generally the shear stress in the wall increases after the first moment crack.*

The test results of Δ/h vs. h/d were plotted in Figure 1 for four types of walls:

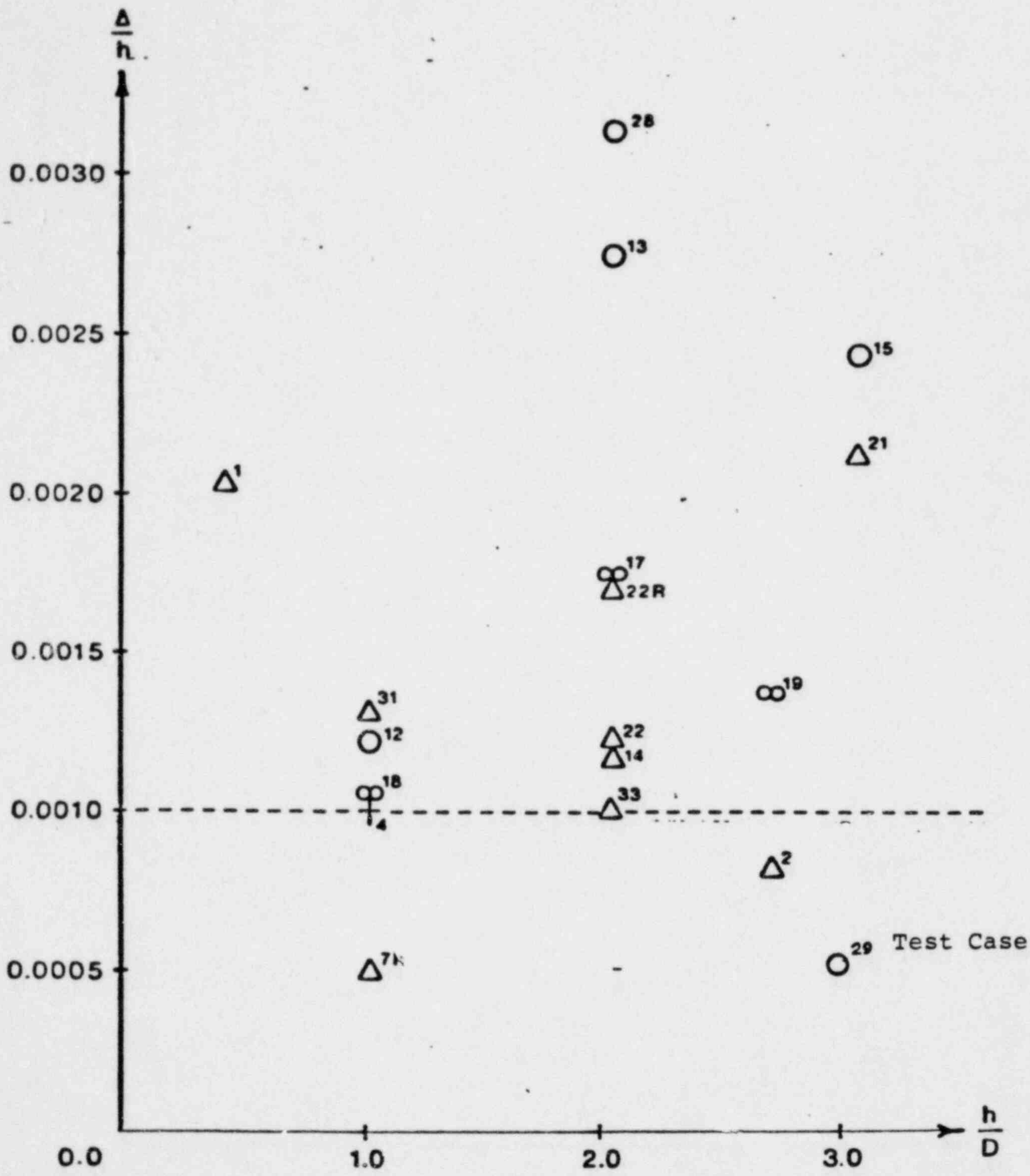
1) $\triangle$ - restrained, grouted

2) $\bigcirc$ - restrained, grouted, horizontal ties

3) $+$ - restrained, partially grouted

4) ∞ - restrained, grouted, axially loaded.

*The first crack in test case 29 is a "moment crack".



IN-PLANE DISTORTION $\left(\frac{\Delta}{h}\right)$

VS.

WALL HEIGHT TO WIDTH RATIO $\left(\frac{h}{D}\right)$

Figure 1

Although first cracking occurs at values less than $\Delta/h = 0.001$ in some instances, the majority of the cases do fall above this value.

The criteria recommended by the Informal Owners Group on Concrete Masonry Walls (R 1) for in-plane distortion of confined walls is:

$$\Delta/h \leq .001$$

where: Δ = relative displacement between top and bottom of wall

h = height of wall.

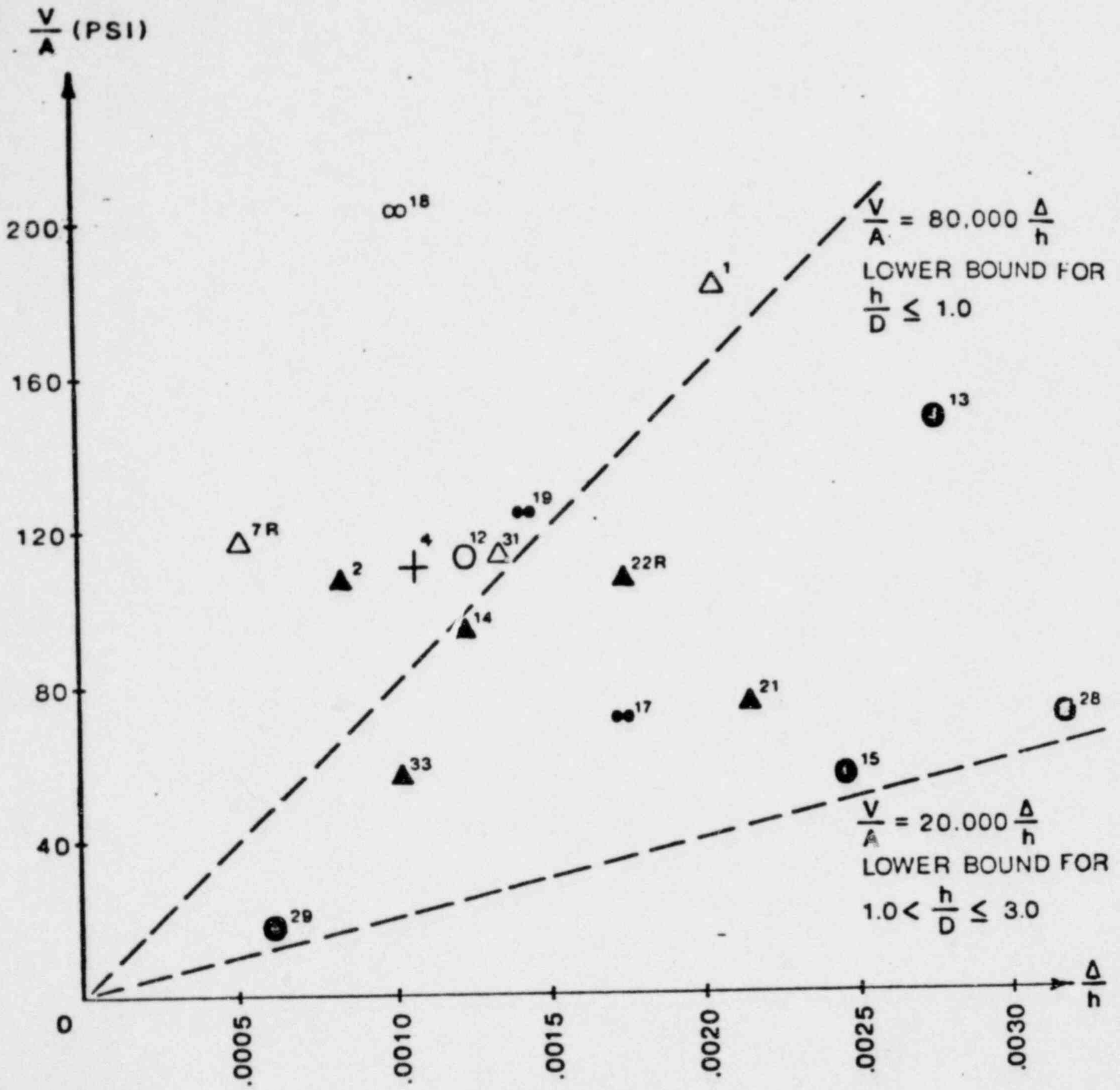
The preliminary study used for their criteria is appended to this answer.

The next step in developing the criteria was to consider the effect of an in-plane force. The graph in Figure 2 shows average shear stress (V/A) vs. shear distortion (Δ/h). A minimum bound on this data is sought so as to maximize the calculated displacement due to the in-plane force. This displacement, Δf, will be added to the imposed displacement due to story drift, Δd, and the combined value compared to the allowable.

The shear stresses, V/A, plotted are those at first cracking, be it a 'moment' or 'shear' type crack. The deformations, Δ/h, are those at first shear crack with test case 29 representing the only moment crack. Test case 29 results in conservatism since shear stress capacity still increases after the first moment crack and even after the first shear crack.

A plot of the data reveals that the height to width (h/D) ratio of the wall greatly influences its shear stress capacity versus deflection. Consequently, two relationships are developed one for h/D ratios ≤ 1.0 (long walls) and one for $1.0 < h/D \leq 3.0$ (short walls). The mode of deformation of short walls is different than the long walls under the same average shear stress conditions.

The previous data presented was from laboratory tests. There is also data presented in Reference R 4 from an actual concrete masonry structure subjected to a major earthquake ground motion.



AVERAGE SHEAR STRESS $\left(\frac{V}{A}\right)$

VS.

IN-PLANE DISTORTION $\left(\frac{\Delta}{h}\right)$

Figure 2

The darkened symbols are for h/D > 1.0.

The structure is North Hall located on the campus of the University of California at Santa Barbara. In the structure, there is a 33 ft. high and 13 ft. wide concrete block shear wall. On the wall, strong motion earthquake accelerometers were positioned to measure the lateral acceleration parallel to the wall. One was located at the base and the other at an elevation 24 ft above the base.

The maximum relative displacement obtained from an analysis of these records was 1.12 cm over the 24 ft. distance. The walls showed no distress and performed well within the failure limits. The relative movement corresponds to an in-plane strain, /h, of 0.15 percent, which exceeds the design criteria used to evaluate the masonry walls at Pilgrim.

Based on the foregoing data, the following criteria combining in-plane forces and in-plane displacements was used for the qualification of confined walls.

$$\frac{\Delta f}{h} + \frac{\Delta d}{H} < .001 \qquad \frac{h}{D} < 3.0$$

$$\text{and} \quad \frac{V}{A} < v_m$$

$$\text{for} \quad \frac{h}{D} < 1.0, \quad \frac{\Delta f}{h} = \frac{V}{80{,}000\ A}$$

$$\text{for } 1.0 < \frac{h}{d} < 3.0, \quad \frac{\Delta f}{h} = \frac{F}{20{,}000\ A}$$

where:

Δd = imposed story displacement

Δf = displacement due to imposed forces

H = story height

h = wall height

A = effective area of wall under in-plane loading

v_m = allowable in-plane shear stress

V = applied in-plane shear force.

For unconfined walls, the study by the Owners Group was utilized. The preliminary study report, as mentioned previously, is appended to this answer.

Owners Group Study of In-Plane Effects

The objective here is to begin to define in-plane strain or displacement criteria for non-structural concrete block masonry walls. For this purpose a non-structural wall is defined as follows:

1. It does not carry a significant part of the story shear or moment.
2. It does not significantly modify the behavior of adjacent structural elements.

In other words, the behavior of the structure must be substantially the same whether such walls are present or not.

Preliminary review of the available literature indicates conservatively that unconfined concrete block masonry can withstand in-plane strains of 0.0001, based primarily on the work of Becica and Fishburn. For story heights of 15 to 20 feet, the corresponding allowable story drift is about 0.02 inches.

For confined masonry, the equivalent strut approach appears promising. Klingner has obtained excellent correspondence between experimental and analytical results for masonry infilled frames loaded cyclicly to failure. His analysis assumes that first degradation of the infill corresponds to failure of the equivalent strut in compression. A survey of the published work on the equivalent strut approach indicates that this "compression" failure is generally coincident with the formation of principal diagonal cracking. This approach to the problem indicates an acceptable story drift an order greater that that for unconfined walls.

The remainder of the list of references provides background on the equivalent

strut method, the effects of openings, and dynamic characteristics. Care must be taken in reviewing the literature since none of the investigators has had the requirements of IE 80-11 in mind. Benjamin, for example, presents a great deal of load displacement data for clay brick infilled frames in which the first crack is clearly identified. However, a closer examination of the text reveals that the crack identified is a boundary crack between infill and frame - a relatively benign effect. In addition, one suspects that the cracking displacement corresponds to the paint in the experiment that the investigator could clearly see a boundary crack that had formed at an earlier stage.

REFERENCES

Becica, I.J. and H.G. Harris, "Evaluation of Techniques in the Direct Modeling of Concrete Masonry Structures," Drexel University Structural Models Laboratory Report No. M77-1, June 1977.

Fishburn, C.C. "Effect of Mortar Properties on Strength of Masonry," National Bureau of Standards Monograph 36 U.S. Government Printing Office, Nov. 1961.

Klingner, R.E. and V.V. Bertero, "Earthquake Resistance of Infilled Frames," Journal of the Structural Division, ASCE, June 1978.

Benjamin, J.R. and H.A. Williams, "The Behavior of One-Story Reinforced Concrete Shear Walls," Journal of the Structural Division, ASCE, Proceedings, Paper 1254, Vol. 83, No. ST3, May, 1957, pp. 1254.1-1254.39.

Benjamin, J.R. and H.A. Williams; "The Behavior of One-Story Brick Shear Walls," Journal of the Structural Division, ASCE, Proceedings, Paper 1723, Vol. 84, ST4, July, 1958, pp. 1723.1 - 1723.30.

Benjamin, J.R. and H.A. Williams, "Behavior of One-Story Reinforced Concrete SHear Walls Containing Openings," Journal of the American Concrete Institute, Proceedings, Vol. 30, No. 5, November, 1958, pp. 605-618.

Holmes, M., "Steel Frames with Brickwork and Concrete Infilling," Proceedings of the Institution of Civil Engineers, Vol. 19, August, 1961, pp. 473-478.

Holmes, M., "Combined Loading on Infilled Frames," Proceedings of the Institution of Civil Engineers, Vol. 25, May, 1963, pp. 31-38.

Liauw, T.C., "Elastic Behavior of Infilled Frames," Proceedings of the Institution of Civil Engineers, Vol. 46, July, 1970, pp. 343-349.

Mallick, D.V. and R.T. Svern, "The Behavior of Infilled Frames Under Static Loading," Proceedings of the Institution of Civil Engineers, Vol. 39, February, 1968, pp. 261-287.

Smith, B.S., "Lateral Stiffness of Infilled Frames," Journal of the Structural Division, ASCE, Vol. 88, No. ST6, December, 1962, pp. 183-199.

Smith, B.S., "Behavior of Square Infilled Frames," Journal of the Structural Division, ASCE, Vol. 91, No. ST1, February, 1966, pp. 381-403.

Smith, B.S., "Model Test Results of Vertical and Horizontal Loading in Infilled Frames," Journal of the American Concrete Institute, Proceedings, Vol. 65, No. 8, August, 1968, pp. 618-623.

Smith, B.S. and C. Carter, "A Method of Analysis for Infilled Frames," Proceedings of the Institution of Civil Engineers, Vol. 44, September, 1969, pp. 31-48.

4. Reference R12 provided the design criteria for reinforced masonry walls for Pilgrim Unit 1. The Licensee should also provide the design criteria for unreinforced masonry walls.

Revision 1 of the Pilgrim Unit 1 design criteria is enclosed. This document includes and supercedes the contents of the two previously submitted criteria documents (Generic and Plant Specific). Attachment A of the revised criteria provides this information.

Job No. 80034
DC - 1 Rev. 1
September 4, 1981

PILGRIM NUCLEAR POWER STATION

DESIGN CRITERIA

FOR

RE-EVALUATION OF MASONRY WALLS

BOSTON EDISON COMPANY

BOSTON, MASSACHUSETTS

Prepared Carolyn C Fiorelli 9/4/81
Group Leader Date

Approved [signature] 9/8/81
Independent Reviewer Date

Approved Paul D B[signature] 9/9/81
Project Engineer Date

Earthquake Engineering Systems, Inc.
600 Atlantic Avenue
Boston, Massachusetts 02210

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Attachments A and B

1.0 GENERAL

This Design Criteria provides the technical basis for the re-evaluation of reinforced masonry walls and the design of modifications at Unit 1, Pilgrim Nuclear Power Station. To develop this information, EES technical personnel have performed an extensive survey of the several codes and standards applicable to the subject and the appropriate literature concerned with research and experience in masonry construction. Further, because the design bases which have developed over the years for nuclear plant structures are very specialized in their application, it has been necessary to reformulate existing criteria for buildings to match the unique nature of the design loading conditions.

The criteria also differs from ordinary building code criteria in that the scope of the Bulletin 80-11 project is to assess the probability and consequences of failure under hypothetical load conditions rather than provide a basis for construction of common residential or industrial buildings. Using the literature as a data base, the degrees of conservatism inherent in the building code requirements have been identified and adjustments made to reflect the qualities of materials and construction in nuclear plant structures and the intent of the various plant design bases.

2.0 REFERENCES

2.1 U. S. Nuclear Regulatory Commission, Office of Inspection and Enforcement, I&E Bulletin No. 80-11, dated May 8, 1980.

2.2 Final Safety Analysis Report, Unit 1, Pilgrim Station No. 600.

2.3 Specification for the Design, Fabrication, & Erection of Structural Steel for Buildings, American Institute of Steel Construction, New York, New York, dated November 1, 1978.

2.4 Reinforced Masonry Design, Robert R. Schneider and Walter L. Dickey, Prentice-Hall, Inc., Englewood Cliffs, N. J., 1980.

2.5 "Specification for Furnishing, Delivery and Installation of Concrete Unit Masonry" for Unit No. 1, Pilgrim Station No. 600, Boston Edison Company, Spec. No. 6498-A- , Revision 1, dated February 1, 1972.

2.6 Theory of Plates and Shells, S. Timoshenko and S. Woinowsky-Krieger, second edition, McGraw-Hill Book Company, 1959, Chapter 11.

2.7 American Society for Testing and Materials, Philadelphia, Pa. Specifications:

C90- 70	Hollow Load Bearing Concrete Masonry Units
C145-71	Solid Load-Bearing Concrete Masonry Units
C476-71	Mortar and Grout for Reinforced Masonry
A82-70	Cold-Drawn Steel Wire for Concrete Reinforcement
A615-68 (formerly A15)	Deformed Billet Steel for Concrete Masonry
A36-67	Structural Steel

2.8 "Civil and Structural Design Criteria for Unit No. 1, Pilgrim Station No. 600, Boston Edison Company", Bechtel Corporation, Job No. 6498, Rev. 3, January 30, 1970.

2.9 Boston Edison Company--Pilgrim Station No. 600 Reactor Building Seismic Analysis, Bechtel Engineering Corp., August 1969.

2.10 Boston Edison Company--Pilgrim Station No. 600 Turbine Building Seismic Analysis, Bechtel Engineering Corp., September 1969.

2.11 Boston Edison Company--Pilgrim Station No. 600 Radwaste Building Seismic Analysis, Bechtel Engineering Corp., September 1969.

2.12 "Analysis of the Consequences of High Energy Piping Failures Outside the Primary Containment," Final Safety Analysis Report, Amendment No. 34, Pilgrim Nuclear Power Station.

2.13 "Damping Values for Seismic Design of Nuclear Power Plants", Regulatory Guide 1.61, U.S. Atomic Energy Commission, October 1973. 1

2.14 Introduction to Structural Dynamics, John M. Biggs, McGraw-Hill Book Company, 1964.

2.15 Response of Arching Walls and Debris from Interior Walls caused by Blast Loading, URS Research Company, Feb. 1975.

3.0 ASSUMPTIONS

3.1 All components other than piping supported on or near masonry walls are rigid for the purposes of this evaluation, and therefore do not impose amplified loads or impact loads on the wall due to seismic displacements.

3.2 Masonry walls that are not part of the structural load resisting system do not carry significant seismic shears or vertical seismic loads due to building inertia forces. However, the effect of imposed displacements due to story drift will be evaluated. 1

3.3 Surface mounted attachments which project no further from the wall surface than the wall thickness contribute only in-plane loads to the wall.

3.4 Linear elastic stress-strain behavior in the compression zone is assumed for masonry.

3.5 Assumptions used in Reference 2.2, as supplemented by references 2.9, 2.10 and 2.11, to model the dynamic response of buildings containing masonry walls are incorporated into this Design Criteria.

3.6 Damping values for reinforced concrete from reference 2.13 may be used for reinforced masonry. Higher values may be used if verified by in-situ tests.

4.0 ANALYSIS AND DESIGN

4.1 General

4.1.1 Stresses in reinforced walls shall be calculated using the working stress method of analysis as described in Chapter 6 of Ref. 2.4.

4.1.2 Support conditions for reinforced walls shall be considered pinned if the reinforcing bars pass through the support interface and are anchored in the adjacent structure or if other shear transfer mechanisms exist. Interlocking edges of intersecting block walls shall also be considered pinned. Edges of block walls that abut adjacent structures shall be considered as free edges. Control joints shall also be considered as free edges.

4.1.3 Reinforced walls shall be analyzed considering one-way or two-way behavior, whichever is appropriate for the boundary conditions, wall dimensions, and reinforcement configuration. Finite element methods may be employed in the analysis.

4.1.4 Reinforced multi-wythe walls shall be analyzed as multiple single-wythe walls connected by cross-ties. No credit is taken for collar joint mortar shear capacity unless allowable values are verified by in-situ tests.

4.2 Seismic Loads 1

Seismic loadings on reinforced walls may be analyzed at three levels as described below. The results at each level shall be compared to the acceptance criteria before proceeding to the next level.

4.2.1 Level 1 Analysis

The natural frequency of the masonry wall shall be determined assuming fully cracked section properties throughout the wall. Orthotropic properties resulting from differing steel reinforcement details in the horizontal and vertical direction shall be taken into account in the analysis as follows:

$$C_{xx} = \frac{E_c}{(1-v^2)} \cdot \frac{I_{tx}}{I_o}$$

$$C_{yy} = \frac{E_c}{(1-v^2)} \cdot \frac{I_{ty}}{I_o}$$

$$C_{xy} = \frac{vE_c}{(1-v^2)} \cdot \frac{(I_{tx} \cdot I_{ty})^{½}}{I_o}$$

$$G_{xy} = \frac{E_c}{2(1+v)} \cdot \frac{(I_{tx} \cdot I_{ty})^{½}}{I_o}$$

Where:

C_{xx}, C_{yy}, C_{xy}, and G_{xy} are the elastic properties of an orthotropic, reinforced masonry wall for use in computer programs that compute stiffnesses based on a solid section.

I_o = Uncracked moment of inertia of solid section per unit length of wall ($t^3/12$ where t = wall thickness.)

I_{tx} = Cracked, transformed moment of inertia per unit length of wall in the x direction.

I_{ty} = Cracked, transformed moment of inertia per unit length of wall in the y direction.

v = Poisson's ratio = 0.2

E_c = Lower bound value of Modulus of Elasticity of masonry = 600 f'm (810,000 psi)

An acceleration corresponding to the fundamental frequency shall be selected from the appropriate response spectrum. The fundamental frequency of the wall per level 1 analysis is based on lower bound values of moments of inertia and Modulus of Elasticity, and as such is underestimated. Therefore, the peak spectral acceleration shall be used if the frequency of the wall is less than the frequency of the peak.

The response spectrum at the building elevation corresponding to the top of the wall shall be used. Conservatively low dampings of 2% (Design Earthquake) and 5% (Maximum Earthquake) of critical damping shall be used in the analysis. The acceleration value selected shall be increased by 1.3 to account for multimode effects.

The spectral acceleration shall then be applied to the mass of the wall and attached components and a static analysis performed.

4.2.2 Level 2 Analysis

If the level 1 analysis fails to meet the acceptance criteria, level 2 analysis may be performed. An iterative procedure employing finite element methods shall be used in the analysis.

Level 2 analysis takes into account the actual cracking pattern in the wall based on the variation of applied moments. Also, a more respresentative value of the Modulus of Elasticity of masonry of 1000 f'm (1,350,000 psi) is used to obtain transformed moments of inertia and orthotropic section properties. 1

The wall shall first be analyzed using uncracked section properties for the masonry in the x and y directions. A response spectrum analysis shall be performed using curves for 4% (Design Earthquake) and 7% (Maximum Earthquake) of critical damping. 1

If the moments in the wall exceed the uncracked moment capacity, the moments in each element shall be used to calculate the effective moment of inertia in each element in the x and y direction as follows:

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^4 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^4\right] I_t$$

$$M_{cr} = F_t \cdot \frac{I_g}{y}$$

Where:

M_{cr} = Uncracked moment capacity.

M_a = Moment at each element.

I_g = Moment of inertia of uncracked section.

I_t = Moment of inertia of the cracked, transformed section.

F_t = Modulus of Rupture of masonry (twice the allowable tensile stress for factored loading conditions, M')

y = Distance of neutral plane from tension face.

The wall shall then be reanalyzed using the effective moments of inertia in the x and y directions. A response spectrum analysis again shall be performed. The resulting moments shall be used to determine the next set of I_e's. When significant moments (moments which are greater than 20% of the maximum moment in the same direction) from two successive iterations are within 10%, and the wall frequency within 5%, the solution is considered to have converged. The procedure may be terminated prior to convergence if a conservative bound on the results has been established. 1

Finally, a response spectrum analysis for 12 modes is performed to account for the contribution from higher modes. 1

4.2.3 Level 3 Analysis

Level 3 analysis may be performed to resolve local overstresses. For this analysis, the wall shall be synthesized as an equivalent single degree of freedom (SDOF) system. A non-linear resistance function shall be calculated by applying successive static loads to the wall. 1

The displaced shape under each load is calculated using the procedure of Section 4.2.2. However, convergence is achieved when the maximum displacement does not vary by more than one percent. The equivalent stiffness corresponding to each static load is calculated by the method of Reference 2.15.

If the bending moment in any element exceeds the allowable moment, the element effective moment of inertia from Section 4.2.2 is modified in one of two ways depending on whether the allowable moment is controlled by compression in the masonry or tension in the reinforcing steel.

If the masonry compression controls the allowable moment, a crushing failure is postulated with a resultant degradation of stiffness. Therefore, the element effective moment of inertia is reduced such that the moment in the element is less than 10% of the allowable moment. 1

If reinforcing steel tension controls, the reinforcement is assumed to yield and the moment remains constant during subsequent distortion. In this case, the element effective moment of inertia is reduced until the element moment is within 10% of the allowable moment.

After the resistance function of the equivalent SDOF system has been computed, a non-linear dynamic analysis is performed using as input the seismic time history at the elevation of the point of maximum displacement in the wall. This time history may be obtained by linearly interpolating between time histories calculated at elevations above and below the wall.

After the maximum displacement of the wall has been computed, the distribution of stresses may be obtained from the corresponding static load case. An overstress is considered acceptable if less than 20% of the surface area of the wall experiences inelastic behavior resulting from the conservative assumptions above.

For walls qualified using Level 3 analysis, equipment mounted on the wall must be evaluated for the effects of out-of-plane displacements on system operability.

In lieu of the above procedure, walls may be qualified using a response spectrum specified at the mid-height of the wall. This response spectrum is obtained by linearly interpolating between response spectra at elevations above and below the wall. In this case, all moments must be within the allowables using the procedure of Section 4.2.2. 1

4.2.4 Other Effects 1

The effect of in-plane loading on reinforced partition walls shall be evaluated in accordance with the criteria specified in Note 12 of Attachment A. The effect of inertial reactions from adjoining walls shall be considered in the evaluation. Building shear walls shall satisfy the allowable stresses given in Attachment A.

The effects of boundary structure flexibility, wall group interaction, and wall openings shall be evaluated.

Out of plane wall displacements due to transverse loadings shall be evaluated for their effect on operability of attached equipment.

Masonry block pullout due to concentrated inertial loadings imposed by attached components shall be evaluated.

4.3 Transient Pressure Loads

Transient pressure loadings on walls may be analyzed at three levels as described below. The results at each level shall be compared to the acceptance criteria before proceeding to the next level. 1

4.3.1 Level 1 Analysis

The natural frequency of the wall shall be determined using the procedure of Section 4.2.1.

The shape of the pressure transient shall be conservatively approximated by a simple geometric shape such as a rectangle, triangle, or sinusoid. A dynamic load factor (DLF) shall be obtained using charts for elastic systems such as in Reference 2.14. Where a higher natural frequency might result in a higher DLF, the higher DLF shall be used.

A static analysis shall be performed using the peak pressure multiplied by the DLF. In lieu of the above procedure, a DLF of 2.0 may be used.

4.3.2 Level 2 Analysis

If the level 1 analysis fails to qualify the wall, a level 2 analysis may be performed using an iterative procedure to determine the DLF.

A response spectrum analysis is performed using the procedure of Section 4.2.2 to calculate the fundamental mode. The response spectrum is obtained by multiplying the peak pressure by the DLF versus frequency curve from Section 4.3.1. 1

4.3.3 Level 3 Analysis

Level 3 analysis may be performed to resolve local overstresses. The analysis shall follow the procedure of Section 4.2.3 using the pressure time history as input.

The wall stiffness under increasing static load may include the additional resistance due to arching action as computed by the method of Ref. 2.15 provided the following restrictions are met: the pressure transient consists of a single pulse acting in one direction; the wall is grouted solid; there is no gap at the top of the wall; the top of the wall is anchored against lateral movement; there are no openings greater than 24 inches; there are no concentrated loads greater than 100 pounds; the stiffness of the surrounding structure resisting the axial thrust load is at least 10 times the wall axial stiffness computed using the modulus of elasticity in line loading.

When arching effects are included, the displacement of the wall must not exceed 0.15 times the

wall thickness. The resultant line load must not exceed one-half the limit calculated by Reference 2.15. 1

4.4 Modification Design

When a masonry wall is not qualified using the procedure of Sections 4.2 and 4.3, modifications shall be designed which enable the wall to satisfy the acceptance criteria. Design calculations shall use the methods of Sections 4.2.2 and 4.3.2.

Modifications shall be designed in accordance with Reference 2.3 subject to the limitations stated in Section 6.0, herein. When structural steel is anchored to solid grouted masonry using drilled in anchors, allowable shears shall be based on 2000 psi concrete. Drilled in anchors shall not be used for tension connections to masonry unless allowable pull-out loads are verified by in-situ testing. 1

5.0 MATERIAL SPECIFICATIONS & PROPERTIES

5.1 Concrete Block (Ref. 2.5)

Hollow Block	ASTM C90 (Ref. 2.7) Grade U-1 Heavyweight
Solid Block	ASTM C145 (Ref. 2.7) Grade U-1 Heavyweight

5.2 Masonry Reinforcement (Ref. 2.5)

Bars	ASTM A615 (Ref. 2.7) Grade 40
"DUR-O-WAL"	ASTM A82 (Ref. 2.7) Heavyweight, truss type

5.3	Mortar (Ref. 2.5)	ASTM C476 (Ref. 2.7) Type PL Compressive Strength @ 28 days = 2000 psi
5.4	Grout (Ref. 2.5)	ASTM C476 (Ref. 2.7) Coarse Compressive Strength @ 28 days = 2000 psi

5.5 Concrete

Reactor Building:
f'_c = 4000 psi
All other poured in place concrete (unless shown otherwise on the drawings):
f'_c = 3000 psi
(Ref. 2.8)

6.0 LOADS AND LOAD COMBINATIONS

The loads and load combinations in this section are based on the structural loading criteria given in Appendix C of the Pilgrim FSAR (Ref. 2.2)

6.1 Loads to be considered in evaluating the masonry walls are described below.

D	Dead load of the structure and related equipment plus any other permanent loads contributing stress, such as soil or hydrostatic loads; live loads expected to be present when the station is operating; and the loads due to thermal expansion under normal operating conditions.
R	Loads resulting from jet forces and pressure and temperature transients associated with rupture of a single pipe within the primary containment.
R'	Loads resulting from jet forces and pressure and temperature transients associated with rupture of single pipe outside the primary containment.
E (E_v, E_h)	Loads due to the design earthquake. (E_v and E_h are vertical and horizontal components of the design earthquake loads, respectively.)
E' (E'_v, E'_h)	Loads due to the maximum earthquake. (E'_v and E'_h are vertical and horizontal components of the maximum earthquake loads, respectively.)
$T^{(1)}$	Loads due to the effects of a tornado.

6.2 Masonry walls shall be evaluated for the following load combinations:

$$
\begin{array}{llll}
EQ(1)\ D + E_v + E_h(2) & < & S^{(3)}, & M^{(4)} \\
EQ(2)\ D + R + E_v + E_h(2) & < & S, & M \\
EQ(3)\ D + T & < 1.5 & S^{(5)}, & M'^{(4)} \\
EQ(4)\ D + R + E'_v + E'_h(2) & < 1.5 & S^{(5)}, & M' \\
EQ(5)\ D + R' & < 1.5 & S^{(5)}, & M'
\end{array}
$$

Notes:

(1) The effects of tornado winds to be considered for class 1 structures are given in Section 12.2.3.3 and Appendix H of the Pilgrim FSAR (Ref. 2.2).

(2) The effects of one horizontal component and the vertical component of earthquake loading shall be combined in all loading combinations which include earthquake loads. 1

(3) S is the normal allowable stress in structural steel sections, bolts, and welds permitted by Ref. 2.3 (excluding the provisions of Section 1.5.6 therein).

(4) M and M' are the allowable stresses for evaluating existing masonry walls (including reinforcing steel) as given in Attachment A.

(5) 1.5 S not to exceed the material yield stress.

7.0 ACCEPTANCE CRITERIA

7.1 Reinforced Walls

Allowable stresses and in-plane distortions for reinforced masonry are tabulated in Attachment A. 1

The basis for the allowable stresses is discussed in Attachment B.

7.2 Modifications

Stresses in structural steel sections, bolts, and welds shall conform to the requirements of Part 1 of the A.I.S.C. Code (Ref. 2.3), excluding the provisions of Section 1.5.6 therein. Allowable stresses may be increased by 1.5 as indicated by the loading combinations in Section 6.0 of this design criteria. 1

ATTACHMENT A

ALLOWABLE STRESSES

FOR

EVALUATION OF REINFORCED MASONRY WALLS

AT

PILGRIM NUCLEAR POWER STATION, UNIT 1

Description		Allowable Stresses (Masonry) (psi) (11) (f'm and m_o are the masonry strength and mortar strength, respectively, in pounds per square inch.)			
		M (1)		M' (1)	
		$f'm/m_o$	$f'm=1350$ $m_o=2000$	$f'm/m_o$	$f'm=1350$ $m_o=2000$
COMPRESSION					
Axial (2) (3)	F_a	0.22 f'm	297	0.44 f'm	594
flexural (4)	F_m	0.33 f'm	446	0.66 f'm	891
BEARING (5)	F_a	0.25 f'm	338	0.50 f'm	675
SHEAR					
No special shear reinforcement					
a. Beams (6)	v_m	$1.1\sqrt{f'm}$	40	$1.65\sqrt{f'm}$	61
b. Walls					
Out of plane (6)	v_m	$1.5\sqrt{f'm}$	55	$2.25\sqrt{f'm}$	83
In plane (12) (7)					
$M/Vd_v \geq 1$	v_m	$0.9\sqrt{f'm}$	33	$1.35\sqrt{f'm}$	50
$M/Vd_v = 0$ (8)	v_m	$2.0\sqrt{f'm}$	73	$3.0\sqrt{f'm}$	110
Reinforcement taking shear					
a. Beams (6)	v_m	$3.0\sqrt{f'm}$	110	$4.5\sqrt{f'm}$	165
b. Walls					
Out of plane (6)	v_m	$1.5\sqrt{f'm}$	55	$2.25\sqrt{f'm}$	83
In plane (12) (7)					
$M/Vd_v \geq 1$	v_m	$1.5\sqrt{f'm}$	55	$2.25\sqrt{f'm}$	83
$M/Vd_v = 0$ (8)	v_m	$2.0\sqrt{f'm}$	73	$3.0\sqrt{f'm}$	110

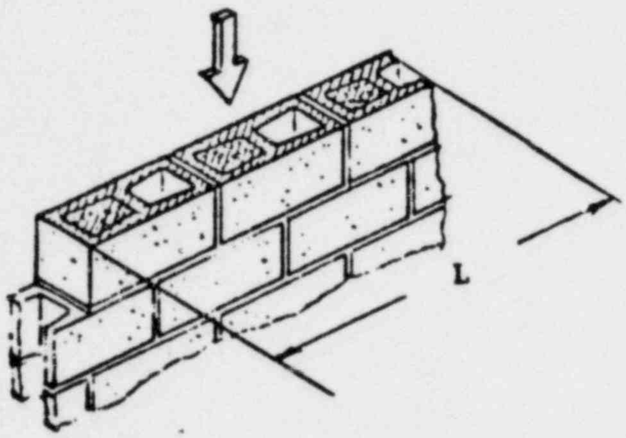
Description		Allowable Stresses (Masonry) (psi) (11) ($f'm$ and m_o are the masonry strength and mortar strength, respectively, in pounds per square inch.)			
		M (1)		M' (1)	
		$f'm/m_o$	$f'm=1350$ $m_o=2000$	$f'm/m_o$	$f'm=1350$ $m_o=2000$
TENSION (13)					
Normal to bed joints (14)					
a. Hollow	F_t	$0.5\ \sqrt{m_o}$	22	$0.75\ \sqrt{m_o}$	34
b. Solid/Grouted	F_t	$1.0\ \sqrt{m_o}$	45	$1.50\ \sqrt{m_o}$	67
Parallel to bed joints in running bond					
a. Hollow	F_t	$1.0\ \sqrt{m_o}$	45	$1.50\ \sqrt{m_o}$	67
b. Solid/Grouted	F_t	$1.5\ \sqrt{m_o}$	67	$2.25\ \sqrt{m_o}$	101

		Allowable Stresses (Reinforcement) (psi)	
		M	M'
BOND			
Deformed bars	u	140	140
TENSION & COMPRESSION			
Reinforcing steel	F_s		
Grade 40 bars		20,000	$0.9\ f_y$ (9)
Joint wire reinforcement		$0.5\ f_y$ (10)	$0.9\ f_y$

Notes:

(1) M and M' shall be used for evaluating stress in accordance with Section 6.2.

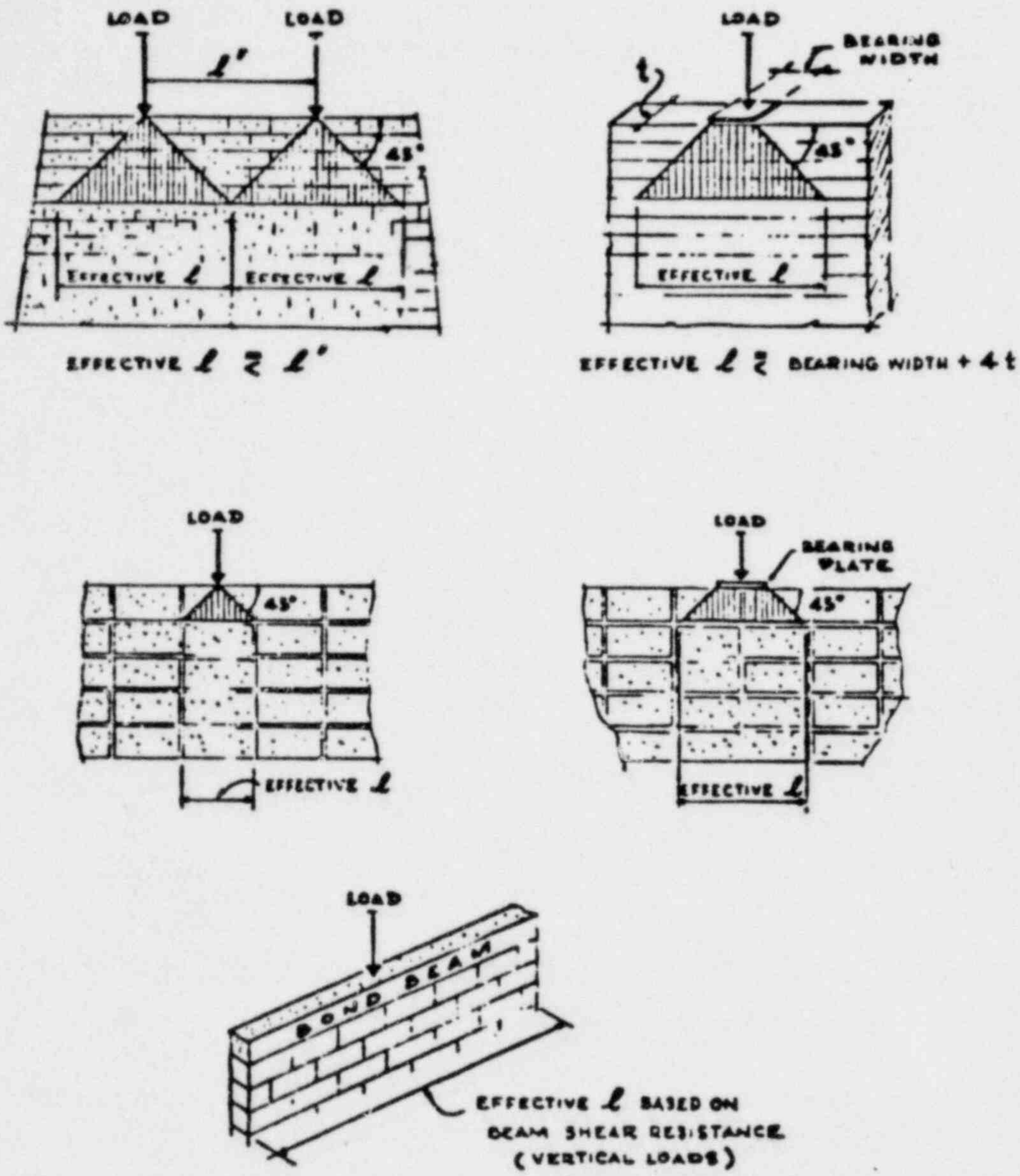
(2) The effective area to be used for evaluating axial compressive stress is shown below:



Effective length L depends on type of bond and loading (see next page).

///// Effective area for axial compressive stress calculations on net section of masonry units plus grouted cores.

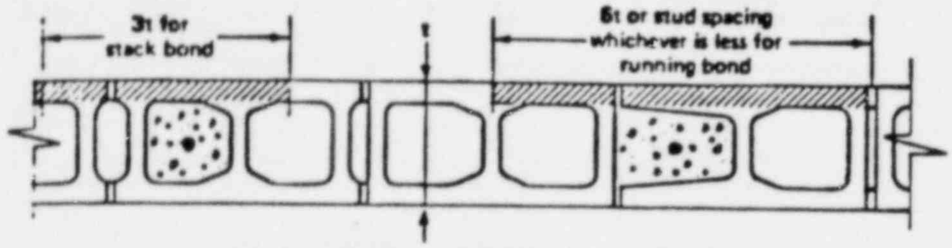
(2) (cont'd)



Effective lengths for axial compressive stress evaluation.

(3) Multiply these values by $(1-(h/40t)^3)$ if the wall has significant vertical load at the top edge.

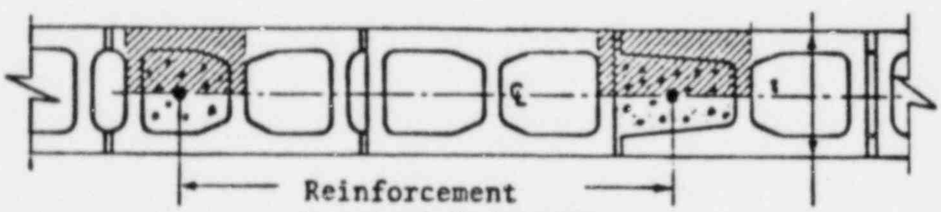
(4) The effective area to be used for evaluating flexural compressive stress is shown below.



Area assumed effective in flexural compression, force normal to face

(5) Allowable bearing stress may be increased to 0.375 f'm for M and 0.75 f'm for M' if load is applied on one-third of the compression area or less.

(6) The effective area for evaluating shear stress for walls in flexure is shown below.



Area assumed effective in shear, force normal to face

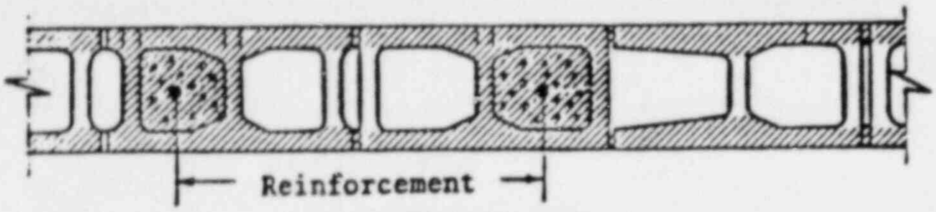
The effective area for evaluating shear stress in rectangular masonry beams is b · d, where:

d=distance from extreme compression fiber to centroid of tension reinforcement.

b=width of compression face of member.

The area of ungrouted cells, and webs between ungrouted cells should be neglected in shear area computations.

(7) The effective area for evaluating shear stress for shear walls is shown below.



Area assumed effective in shear, force parallel to face

(8) M is the maximum bending moment ocurring simultaneously with the shear load V at the section under consideration. d_v is the length of the wall in the direction of shear. Interpolate by straight line for M/Vd_v values between 0 and 1.

(9) f_y is the specified yield strength of the reinforcement. (f_y = 40 KSI for grade 40 reinforcement)

(10) 0.5 f_y not to exceed 30,000 psi.

(11) Values for the Modulus of Elasticity and Poisson's ratio to be used in the analysis are as follows:

Modulus of Elasticity E_c = 600 f'm (for level 1 analysis)
= 1000 f'm (for level 2/3 analysis)

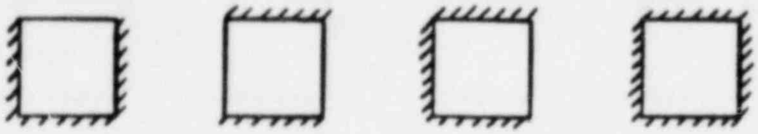
Poisson's Ratio ν = 0.2

(12) For non-shear walls which are confined in the structure and subjected to shear distortion due to relative floor displacements, the allowable relative displacement (Δ) is 0.1% of the height of the wall (h).

For non-shear walls shich are subjected to shear distortions due to relative floor displacement but cannot be classified as confined walls, the allowable relative displacement is 0.01% of the height of the wall.

(12) (cont'd)

Confined walls are bounded by adjacent steel or concrete primary structures. As a minimum, confined walls are bounded top and bottom or bounded on three sides. Examples of confined walls are shown schematically below.



Confined walls: $e_v = \Delta/h \leq 0.001$

Confined walls that are subjected to in-plane forces as well as displacements, but are not building shear walls, shall satisfy the following:

$$\frac{\Delta_f}{h} + \frac{\Delta_d}{H} \leq 0.001$$

$$\text{and } V/A \leq v_m$$

$$\text{for } h/D \leq 1.0, \quad \Delta_f/h = V/80{,}000A$$

$$\text{for } 1.0 < \frac{h}{D} \leq 3.0, \quad \Delta_f/h = V/20{,}000A$$

Where:

Δ_d = imposed story displacement

Δ_f = displacement due to imposed forces

H = story height

h = wall height

D = wall length

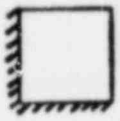
A = area of wall under in-plane loading

v_m = allowable shear stress under in-plane loading

V = applied in-plane load

(12) (con't)

Unconfined walls are not bounded by adjacent steel or concrete primary structures sufficiently to create a confining effect. An example of an unconfined wall is shown schematically below.



Unconfined Wall: $e_v = \Delta/h \leq 0.0001$

(13) The modulus of rupture used to define the masonry cracking moment (M_{cr}) for level 2 and 3 analysis shall be twice the allowable stress for factored loading conditions (M').

(14) For reinforced sections, the allowable moment is computed based on tension in the steel reinforcement only.

ATTACHMENT B

BASIS
FOR THE
ALLOWABLE STRESSES
FOR
REINFORCED MASONRY WALL EVALUATION

INTRODUCTION

The acceptance criteria have been expressed in terms applicable to nuclear plant design, and similar to those used for concrete evaluation. It is therefore important to differentiate between normal load conditions and factored load conditions.

Normal, or unfactored, loads are loads encountered during normal operation of nuclear plants. Included in this category are those severe environmental loads which may be anticipated during the life of the facility, such as the operational basis earthquake. The loads in this category considered in the evaluation of the masonry walls at Pilgrim 1 include the Design Earthquake (analogous to the OBE), dead load on the structure and other permanent loads, and live loads expected to be present during normal operation of the unit. For concrete structures, these loads are evaluated by ultimate strength methods using appropriate load factors.

Factored loads, on the other hand, are those hypothetical loads which have a very low probability of occurrence over the life of the facility but which are evaluated because of safety considerations. These loads include extreme environmental and abnormal loads, such as the safe shutdown earthquake. The loads in this category considered in the evaluation of the masonry walls at Pilgrim 1 include the Maximum Earthquake (analogous to the SSE), building depressurization loads due to a tornado, and the loads due to a high energy pipe rupture outside containment. The ultimate acceptance criteria for these load conditions is that operability of critical plant systems not be impaired. For concrete structures, these loads are generally evaluated by ultimate strength methods using load factors of unity.

It is difficult to use building code values to develop criteria for factored load evaluation because masonry design is based on working stress methods rather than ultimate strength techniques. At present, the state of the art has not progressed sufficiently to embrace the more sophisticated precepts of ultimate strength design, principally because of the lack of knowledge of many of the fundamental material properties (e.g., ultimate strain of the masonry assemblage), the performance characteristics of reinforced masonry systems, and the wide scatter of variable values reflected in much of the test data.

Therefore, EES has reviewed the literature (references are listed at the end of this commentary) relative to the various stress values, determined reasonable lower bounds on ultimate loads, reduced them by appropriate amounts, and applied them to working stress design methods. For evaluation of factored loads, allowable stresses have generally been taken as one half the lower bound ultimate while a factor of four is generally used for normal load allowables. Thus there are three levels of conservatism inherent in the evaluation criteria: use of lower bound ultimate values, capacity reduction factors of two and four, and use of working stress design methods.

In cases where not enough test data is available to determine a lower bound ultimate, the building codes have been used for guidance in selecting values for normal loads. Stress limits for factored loads have been determined by applying increases consistent with those for similar conditions.

COMPRESSION

Allowable stresses which relate to the masonry compressive strength are expressed in terms of f'm, the ultimate compressive strength of the masonry assemblage. This strength may be determined by test or may be conservatively estimated using the table below.

Compressive test strength of masonry units, psi, on the net cross-sectional area	Compressive strength of masonry f_m', psi	
	Type M and S mortar	Type N mortar
6000 or more	2400	1350
4000	2000	1250
2500	1550	1100
2000	1350	1000
1500	1150	875
1000	900	700

Values of f'm for Masonry

The building code values for axial compression fall around 0.22f'm. This is consistent with a factor of four under ultimate (assuming a lower bound of about 0.9f'm) for normal loads. For factored loads, a value of 0.44f'm gives a factor of two under the lower bound ultimate.

For compression due to bending, the peak stress is computed on an elastic basis by working stress methods and assumes a triangular stress distribution. In reality, the stress distribution is more uniform, especially at high stress levels. The building codes recognize this by allowing a 50% increase in the allowable for peak compression under bending. Since there is no test data contradicting this well established practice, a value of 0.33f'm is used for normal loads. Applying an increase consistent with that for uniform compression gives a value of 0.66f'm for factored loads.

For walls which support significant vertical loads, the effects of slenderness should be considered. There is a good deal of test data on this subject, and the capacity reduction factor given in note 3 of Attachment A is well supported. This should apply to all the allowable compressive values including those for factored loads. In evaluating vertical loading, consideration of bending due to load eccentricity is required.

BEARING

The value for allowable bearing stress is taken from the building codes for normal loads and is the same as for concrete under ACI 318-63. It gives a factor of four on ultimate. Increasing this value the same as for other compressive stresses gives 0.50f'm for factored loads. Actually, this value is rather conserative, as concentrated loads will either bear on a block or on mortar, so that use of the composite strength is not really appropriate. It would be more correct to use the block or mortar strength for bearing calculations and use the composite strength when evaluating compressive stress over the effective tributary length.

When the bearing surface is less than the total surface, confinement effects will permit higher bearing loads. The codes allow a 50% increase if the bearing area is less than one-third the total area. This increase is permitted only

when the least distance between the edges of the loaded and unloaded areas is a minimum of one-fourth of the parallel side dimension of the loaded area. The allowable bearing stress on a reasonably concentric area greater than one-third, but less than the full area, may be interpolated between the values given.

FLEXURAL SHEAR (Beams and Walls)

The allowable flexural shear values developed for this criteria vary for beams and walls and depend on whether the stress is carried by shear reinforcement.

For beams, the values for allowable flexural shear stress ($1.1 \sqrt{f'm}$ for no special shear reinforcement; $3.0 \sqrt{f'm}$ for reinforcement taking shear) are taken from ACI 531-79 and are the same as the working stress concrete values given in ACI 318-63. Most of the testing done to develop flexural shear stress allowables has been performed on masonry beams, and the data show that a factored load increase of 50% leaves at least a factor of safety of two against lower bound ultimate.

For walls, the allowable flexural shear stress is analogous to the peripheral shear value used in the evaluation of concrete slabs. Since no code value for peripheral shear exists for masonry, it is reasonable to base one on that for plain concrete; i.e., $2 \sqrt{f'c}$ from ACI 318-63. The ACI 318-63 allowable flexural compressive stress is $0.45\sqrt{f'c}$ versus a normal load allowable of 0.33 f'm for masonry. Using the same ratio for peripheral shear gives a masonry allowable of $1.5 \sqrt{f'm}$.

As for beams, an increase factor of 1.5 is used for factored loadings to provide at least as large a margin of safety against ultimate. For reinforced walls, the shear is based on the depth to reinforcement. For unreinforced walls, the full cross-section is used since the masonry is uncracked. Higher allowables for shear reinforcement do not apply to walls since there is not sufficient depth in general to develop steel reinforcement.

IN-PLANE SHEAR (Walls)

The allowable in-plane shear stress/distortion criteria depends on whether the wall is a load bearing shear wall or a non-load bearing partition wall, and whether the wall has special shear reinforcement.

If the purpose of a masonry wall is to resist structural shear forces, then allowable stresses based on the shear force divided by the effective shear area must be satisfied.

The in-plane shear stress values for shear walls for normal loads are taken from the building codes. The values for factored loads provide a factor of at least two against lower bound ultimate when compared to test data for reinforced walls.

The allowable values for walls with special shear reinforcement for normal loads are again taken from the building codes, with the same increase for factored loads as for the case of no special reinforcement. From the test data, it appears that for horizontlly reinforced walls with low height to length ratios, allowables can be higher than $3.0 \sqrt{f'm}$ and still provide a factor of two on ultimate. However, this would have to be evaluated on a case by case basis.

Walls which are not part of the main structural system (partition walls) need not satisfy the allowable stress criteria for in-plane shear. However, they must be checked to make sure that drift imposed distortions do not cause significant cracking which will impair ability to carry out-of-plane loads. Test data from Reference 64 gives values of shear displacement at first cracking for confined walls of various height to length ratios. The allowable displacement criteria are based on a lower bound from the data which applies to both normal and factored loads.

TENSION

For unreinforced walls analyzed on an elastic basis, the resistive capacity is evaluated on the basis of an allowable computed tensile stress. For vertical tensile stresses, the critical section is through the mortar bed joints. However, for horizontal stress in running bond, the actual load path is not tension through the mortar but rather shear transfer up and down along adjacent courses.

For vertical tension normal to the bed joints, test results indicate a factor of safety of four for the value of $0.5\sqrt{m_o}$, where m_o is the mortar compressive strength, for service loads. This is about one-third the allowable value for plain concrete under ACI 318-63 and one-twentieth the value based on the formula for modulus of rupture in concrete. However, some dynamic tests on unreinforced, vertically spanning

walls showed initiation of cracking at stresses close to $0.5\sqrt{m_o}$, although only after several load cycles. The ultimate capacity of the walls were quite a bit greater, though not quantified, than the cracking strength. Hence, the allowable value for factored loads is not increased more than 50% over that for normal loads. Moreover, the use of tensile capacity normal to the bed joints is limited to cases where horizontal spanning, as in two-way action, or arching capacity can provide an assurance that local failure in the bed joint will not cause collapse of the wall.

For horizontal tension, on the other hand, the resistive capacity is not a function of the mortar tensile strength but of the interlocking effect of the running bond pattern. Test results show a capacity for horizontally spanning walls of twice that and more compared to vertically spanning walls. For this reason, the service load allowable of $1.0\sqrt{m_o}$, is quite conservative. However, the increase for factored loads is kept at 50% to be consistent with the shear allowables, insofar as the interlocking effect is achieved by shear transfer in the bed joints. The testing reported in the literature clearly shows that a higher allowable could be derived using a safety factor of two criterion. However, there is not much dynamic data, and it is prudent to be more conservative in this area.

MODULUS OF ELASTICITY

The value of 600 f'm specified for level 1 analysis is a lower bound value of the modulus of elasticity. As such, it is appropriate for the modulus, and is the same as the value specified in the building codes. Variations in this value are offset by the peak spread of the response spectra and by other conservatisms built into the analysis and the allowable stresses.

The value of 1000 f'm specified for level 2 and 3 analyses represents the most expected value of the modulus, and is the same as the value specified in the building codes. Variations in this value are offset by the peak spread of the response spectra and by other conservatisms built into the analysis and the allowable stresses.

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Additional to Pilgrim 80-11 Design Criteria

Page 3: Add the following references

2.16 Deflection of Two Way Reinforced Concrete Floor Systems: State-of-the-Art Report (ACI 435.6R-74).

2.17 Deflections of Reinforced Concrete Flexural Members (ACI 435.2R-66)

Page 4: Change line 7 of Section 4.2.1 to read

"follow (Ref. 2.6):"

Page 6: Change line 5, paragraph 4, Section 4.2.2 to read

"tion as follows (Ref. 2.16, 2.17):

change I_e formula to read

"$I_e = \ldots \leq I_g$"

Page 7: Insert these paragraphs at the end of Section 4.2.2:

"A special case exists for walls that are primarily reinforced in one direction. At the conclusion of the level 2 analysis, the stresses in the strong direction may meet the acceptance criteria while those in the weak direction exceed the acceptance criteria. These walls are acceptable if all stresses are within allowable after setting I_t (weak) = 0 for all elements overstressed in the weak direction and repeating the level 2 analysis.

"In repeating the level 2 analysis with reduced stiffness in some elements, the wall frequency will shift to a lower value. If both the original and new frequency are below the resonant range of the ARS, or the original is above and the new below, the reduced stiffness analysis may be unconservative. To assure conservatism, the highest spectral acceleration in the range between the original level 2 analysis frequency and the reduced stiffness frequency shall be applied to the reduced stiffness model."

5. With respect to the non-linear behavior analysis of masonry structures, it would be advisable not to resort to this approach, if possible. However, if the Licensee chooses to adopt this approach, conclusive evidence must be submitted.

This question refers to the level 3 time history analysis of Sections 4.2.3 and 4.3.3 of the revised criteria. This technique was intended for the resolution of local overstress and is quite conservative in its representation of post-allowable behavior. However, no masonry walls have been qualified using the level 3 time history analysis.

6. With respect to the special inspection category of stress values, the Licensee should indicate if the construction practice for the masonry structures at Pilgrim Unit 1 was in conformance with the provisions specified for the special inspection category in ACI 531-79.

The inspection requirements of ACI 531-79 (R5) are given below:

4.5 - Inspection

4.5.1 When specified, masonry construction shall be inspected during the various stages by the Engineer/Architect, or by a person acceptable.

4.5.2 Inspection shall include checking for compliance with project drawings and specifications and keeping of records which cover the following:

4.5.2.1 Quality and testing of masonry units and materials for mortar, grout, and making of prisms when required.

4.5.2.2 Proportioning, mixing, and consistency of mortar and grout.

4.5.2.3 Laying, mortaring, and grouting of masonry units and elements.

4.5.2.4 Condition, grade, size, spacing, and placement of reinforcement.

4.5.2.5 Any significant or unusual construction loads on masonry structural elements.

4.5.2.6 General progress of work.

4.5.2.7 When ambient temperature falls below 40 F or rises above 100 F, a complete record of weather conditions and of preconditioning and protection given to masonry materials, and protection and curing of completed work, shall be maintained.

4.5.3 Inspection records shall be available to Building Official, Owner, and Architect/Engineer during progress of work and for two (2) years thereafter.

At the time of construction of Pilgrim 1, ACI 531-79 did not exist. The previous document, ACI 531-70 (R6), does not require retention of inspection records. Bechtel Power Corporation was both Architect/Engineer and Constructor for Pilgrim and provided continuous surveillance during construction. The construction specification (R7) is appended to this answer.

Construction of Pilgrim was completed in 1972. Records still existing in files cover the following:

Core samples: 10 walls

Field Inspection Reports: Off Gas Retention Building
Turbine Building

Test Reports: Compression Strength
Absorption
Weight
Dimensions
Material Certificates

Prior to starting the Bulletin 80-11 reevaluation, a plant walk-down was performed by consultants experienced in masonry design and construction. They concluded, to the extent possible from external visual examination that the concrete block walls at Pilgrim appeared to have been erected using proper construction techniques. Additionally, a test program (R8) was carried out to verify that internal construction details were in accordance with the construction drawing (R9). Details to be verified were reinforcement, grouted cells, and anchorage. The method of verification was removal of blocks and visual inspection. Where details varied from the construction drawing, the as-built conditions were used for analysis.

Job No. 80034
WI-6, Rev. 2
November 25, 1981

PILGRIM NUCLEAR POWER STATION

WORK INSTRUCTION

FOR

TESTING OF MASONRY WALLS

BOSTON EDISON COMPANY

BOSTON, MASSACHUSETTS

Approved John D. McWilliams 11/25/81
Project Engineer Date

Louis J Barbieri for SC White 11/25/81
Quality Assurance Date

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1.0 PURPOSE

The purpose of this testing program is to verify that the masonry block walls at Pilgrim I Nuclear Power Station were built in accordance with original design drawings and specifications. As-built wall details will be used as a basis for the structural evaluation of masonry walls for response to NRC IE Bulletin 80-11.

2.0 SCOPE

The scope of work shall include testing to verify the existence of vertical reinforcement, dur-o-wall horizontal reinforcement, grouted cells in the masonry, positive anchorage to adjacent structures, and lack of voids in the collar joint for multi-wythe/composite walls where called for on the design drawings and specifications. A random sample of masonry walls shall be selected to statistically ensure a 95% confidence level. The specific scope of work shall be specified in a separate document.

3.0 References

3.1 "Method of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete", ASTM C42, CSA A23.2.16

3.2 American Society for Nondestructive Testing (1975) - Recommended Practice ASNT-TC-1A

3.3 Kapur, K.C. and Lamberson, L.R., Reliability in Engineering Design, John Wiley and Sons, New York, 1977

3.4 "Specification for Furnishing, Delivery, and Installation of Concrete Unit Masonry for Unit No. 1, Pilgrim Station No. 600" - Bechtel Corp. Specification No. 6498.A-1, Rev. 1

4.0 TEST METHODS

4.1 Core Drilling - A core sample shall be removed from the masonry wall. Cores shall be 2" in diameter and 15" in length. This test may be done to verify the existence of grout or mortar in the collar joint of multi-wythe/composite walls. All coring shall be done in accordance with ASTM C42, CSA A23.2.16.

4.2 Cutting and Chipping - The face of a masonry block shall be removed by chipping and/or saw cutting. A maximum of one half the block thickness shall be removed. This test may be done to verify the existence of grout in cells, vertical reinforcement, dur-o-wall horizontal reinforcement, and anchorage to adjacent structures.

4.3 Ultrasonic Examination - Ultrasonic pulses shall be transmitted through the masonry wall and recorded on the far side. This test may be done to verify the existence of vertical and/or horizontal reinforcement. Ultrasonic examination shall be done in accordance with ASNT-TC-1A.

4.4 Borascope Examination - A pilot hole (approximately 3/4" in diameter) shall be drilled to a depth of 15 inches. An optical instrument, such as a borascope (or fiberscope) shall be inserted into the hole to visually examine the interior of the masonry wall. This test may be done to verify the existence of grout in single or multi-wythe/composite walls.

4.5 Radiographic Examination - Radiographic inspection of walls may be done to verify the existence of horizontal and/or vertical reinforcement in single wythe walls. This method should only be used when both faces of the wall are accessible. Personnel, procedures, and equipment shall be qualified by the requirements of ASNT-TC-1A.

5.0 WALL SAMPLING

The scope of walls to be tested shall be as set forth by the BECo 80-11 Project Manager. For the minimum wall sampling requirements, see Section 7.0.

6.0 PROCEDURE

Any wall examined may be so tested to verify the following:

6.1 Dur-o-wall Horizontal Reinforcement - For walls which call for dur-o-wall horizontal reinforcement on the design drawings (i.e., all 8" single wythe walls), bed joints on each wall tested shall be randomly examined at three different elevations. This shall be done preferably by cutting and chipping out the masonry block block and mortar around the bed joint to verify the existence of the dur-o-wall. After examination, all block and mortar areas removed shall be regrouted as outlined in Section 6.7. Should this test method prove impractical for any wall, ultrasonic or radiographic examination may be used, but only if both sides of the wall are readily accessible. Also, radiographic examination should be limited to single wythe walls only. Document the results of the examination as described in Section 8.0.

6.2 Vertical Reinforcement - Each selected masonry wall shall be randomly examined at a minimum of three different horizontal locations. The preferable method of inspection shall be by cutting and chipping away the face of one block at each location, to a depth of one half the thickness. Vertical reinforcement shall be examined and documented as required in Section 8.0. Cut away sections of block shall then be regrouted as outlined in Section 6.7.

Alternatively, ultrasonic or radiographic examination may be used. If either method is selected, the following procedure should be used: Randomly select masonry block units at three different horizontal locations on the wall. The exact location should preferably be at the mid height of the block to avoid distortion of results due to the horizontal reinforcement at the bed joints. Also, blocks should be selected that are generally distant from any metal attachments, such as piping or conduit. Further limitations include walls that are accessible on both sides, and for radiographic examination, wall samples should be limited to single wythe. Document the results of all tests, according to Section 8.0.

6.3 Anchorage to Adjacent Structures - This shall be examined by cutting and chipping away the face of certain blocks along top and side supported edges. Three consecutive blocks shall be cut away along the top edge and four consecutive blocks shall be cut away along the side edges where applicable. The cut into the block shall be to a minimum depth of one half the block thickness (if anchorage is found), and to a maximum depth of the entire depth of grout in the block cells, with care being taken not to break through the far face of the block.

Inspect all blocks for the existence of dowels. Document the size, number and location of all dowels observed. Also document any unusual findings, such as poor quality of grout in the block or poor quality of welds between dowels and adjacent steel.

All results shall be documented as outlined in Section 8.0. Replace all cutaway block sections with grout, as outlined in Section 6.7

6.4 Grouted Cells - This shall be examined by cutting and chipping away the face of masonry blocks. Three blocks shall be chosen at random, and the block face shall be chipped away to a depth of 2" to 3". For walls

designated as "Partition Walls" inspect the exposed blocks for grout fill at least every 16 inches horizontally. For walls designated as "Shield Walls" inspect the exposed blocks for grout fill in all of the cells. Document results as described in Section 8.0. Replace removed block sections with grout, as outlined in Section 6.7.

6.5 Multi-wythe/Composite Construction - Testing of these walls shall be done preferably by core drilling. Three core samples from each tested wall shall be randomly selected. The core samples shall be generally taken after the block facing has been chipped away to locate the vertical rebar. The sample shall be taken at a location 4" to the right or left of the vertical rebar, so as not to damage any reinforcement in the wall. The samples should be 2" in diameter and 15" long. The cores shall be inspected for voids between adjacent wythes or between the outer wythe and grouted center. Document results according to Section 8.0. Fill all cored holes with grout as outlined in Section 6.7.

Alternatively, borascope testing may be used. Pilot holes at least 3/4" in diameter and 15" long shall be drilled in blocks at 3 locations, similar to those required for core drilling described above. An optical instrument, such as a borascope, shall be inserted into the hole to inspect for voids between adjacent wythes or between the outer wythe and grouted center. Document results.

6.6 Combination of Testing - Where practical, any of the above tests may be combined. For example, any block that has its face cut and chipped away may be inspected for vertical reinforcement, horizontal reinforcement, grouted cells, and if applicable, anchorage to adjacent structures. Efforts should be made to minimize the extent of wall that must be disturbed for testing.

6.7 Regrouting - All portions of block walls disturbed or damaged by testing procedures shall be restored with grout. This shall apply to all block sections that have been cut and chipped away, and to all holes that have been core drilled. The grouting procedure shall be as set forth in Attachment 1.

6.8 Additional testing may be initiated if in the opinion of the inspecting engineer, it is necessary to accurately report the as-built condition of the wall. However, in no instance will the wall be chipped further than the depth of the block cell.

7.0 ACCEPTANCE CRITERIA

The following criteria are established to statistically ensure within a 95% confidence level that the masonry walls were built in accordance with the design drawings and specifications. The criteria are based upon the acceptance and rejection levels and sampling sizes set forth in Reference 3.3. These criteria shall be the bases for determining whether or not the details shown on as-built drawings should be assumed correct.

Any of the following items shall be acceptable for all walls if the first thirty walls tested for that item are all acceptable. Conversely, any item shall be unacceptable for all walls if the first six walls tested for that item prove unacceptable. For test results falling between these bounds, additional testing may be required. (See Section 7.6)

7.1 Horizontal Reinforcement - Detail as shown is acceptable for that wall if horizontal reinforcement is observed in all of the bed joints examined.

7.2 Vertical Reinforcement - Detail as shown is acceptable for that wall if vertical reinforcement is observed in all of the blocks examined.

7.3 Anchorage to Adjacent Structures - Detail as shown is acceptable for that wall if dowels are observed in each block examined along the top and in at least one half of the blocks examined along the sides.

7.4 Grouted Cells - Detail as shown is acceptable for that wall if grout is observed in all the cells examined for shield walls, and in at least one of the cells examined for partition walls.

7.5 Multi-wythe/Composite Construction - Detail as shown is acceptable for that wall if no voids greater than 1/4" are observed in any of the samples taken.

7.6 Unacceptable Results - In the event that one or more of the design details being verified proves to be unacceptable or is unable to be verified, based upon the criteria above, the detail shall be deemed "UNVERIFIED" and therefore unacceptable. The Project Engineer shall review the results to determine if additional testing would be advantageous. Additional testing and acceptance criteria shall be based upon the acceptance and rejection levels shown in Attachment 2. The Project Engineer shall notify BECo in writing of his recommendation. No additional testing shall be done prior to BECo's approval.

8.0 DOCUMENTATION

All pertinent information shall be documented on the "Pilgrim I Masonry Wall Testing - Results Log" (Exhibit A). This information shall include the following:

8.1 Wall Number - the masonry wall identification number.

8.2 Test Method - the type of testing being used, such as cutting and chipping, core drilling, etc.

8.3 Test Number - for each test method on a particular wall, each test shall have a sequential number (i.e. If four core drilled samples are to be taken on a wall, they shall be numbered 1, 2, 3, and 4.).

8.4 Results

Document as follows any item being investigated:

If observed, indicate "Yes" in the approprite column.

If not observed, indicate "No" in the appropriate column.

For all remaining items that are not being investigated by that particular test, indicate "NA" (not applicable) in the appropriate column.

The following items shall be documented:

8.4.1 Vertical Reinforcement

8.4.2 Dur-o-wall horizontal reinforcement in the bed joints

8.4.3 Anchorage to adjacent structures

8.4.4 Grouting within cells of masonry units

8.4.5 Multi-wythe or Composite Construction - Indicate whether the internal sections of the wall are masonry units or grout fill. If masonry units are found, document as "MW" (multi-wythe). If grout is detected, document as "CC" (composite construction). Also, from the wall sample taken, document the largest void area detected.

8.5 Any unusual observations shall be so noted under the "remarks" column.

8.6 Additionally, provide a stretch of the wall showing where testing has been done along with all observations. An example of this sketch is shown in Exhibit A - Results Log Sheet 2 of 2.

8.7 Sign-offs

The log shall be filled in by the engineer inspecting each test. The inspecting engineer shall then initial and date under the "Inspected By" column on the log.

A second engineer shall then check the test results. If he concurs, he shall initial and date under the "Checked By" column.

A Quality Control Engineer shall also verify all test results. He shall initial and date the log under the "QC Verified" column.

8.8 Originals of the "Results Log" shall be maintained by the Project Secretary in Boston. Copies of the log shall be forwarded to Boston Edison.

Attachment 1

Requirements for Grouting Masonry Wall

Sections Removed for Testing

All masonry block wall sections that have been cut or chipped away, and all holes that have been core drilled shall be restored with grout, as described below.

1. Material Requirements

   a. Five Star Grout (standard nonshrink cement-based grout; 5,000 psi compressive strength).

   b. Certificates of Compliance required from U.S. Grout Corporation

   c. Compressive strength requirements set forth by ASTM-C109.

2. Surface Preparation

   a. All concrete surfaces in contact with grout shall be free of all oil, grease, etc. Concrete surfaces should be roughened to assure a good bond of grout to existing concrete.

   b. All metal surfaces including anchor bolts and rebar must be cleaned of all oil, grease, and foreign substances.

3. Storage

   a. Grout shall be stored in a dry, weatherproof area, away from the ground, and within the temperature range of 40 F to 90 F.

   b. Any grout which becomes damp is defective and is to be disposed of.

   c. The total shelf life of nonshrink grout shall be limited to ten (10) months.

4. Preparation

   a. Prepare grout in accordance with the manufacturer's specifications.

5. Placing of Grout

   a. The grout shall be hand packed into all areas that are cut or chipped away.

   b. Core drilled holes shall be completely filled with grout and finished.

6. Curing

   a. The grout shall remain undisturbed for 48 hours.

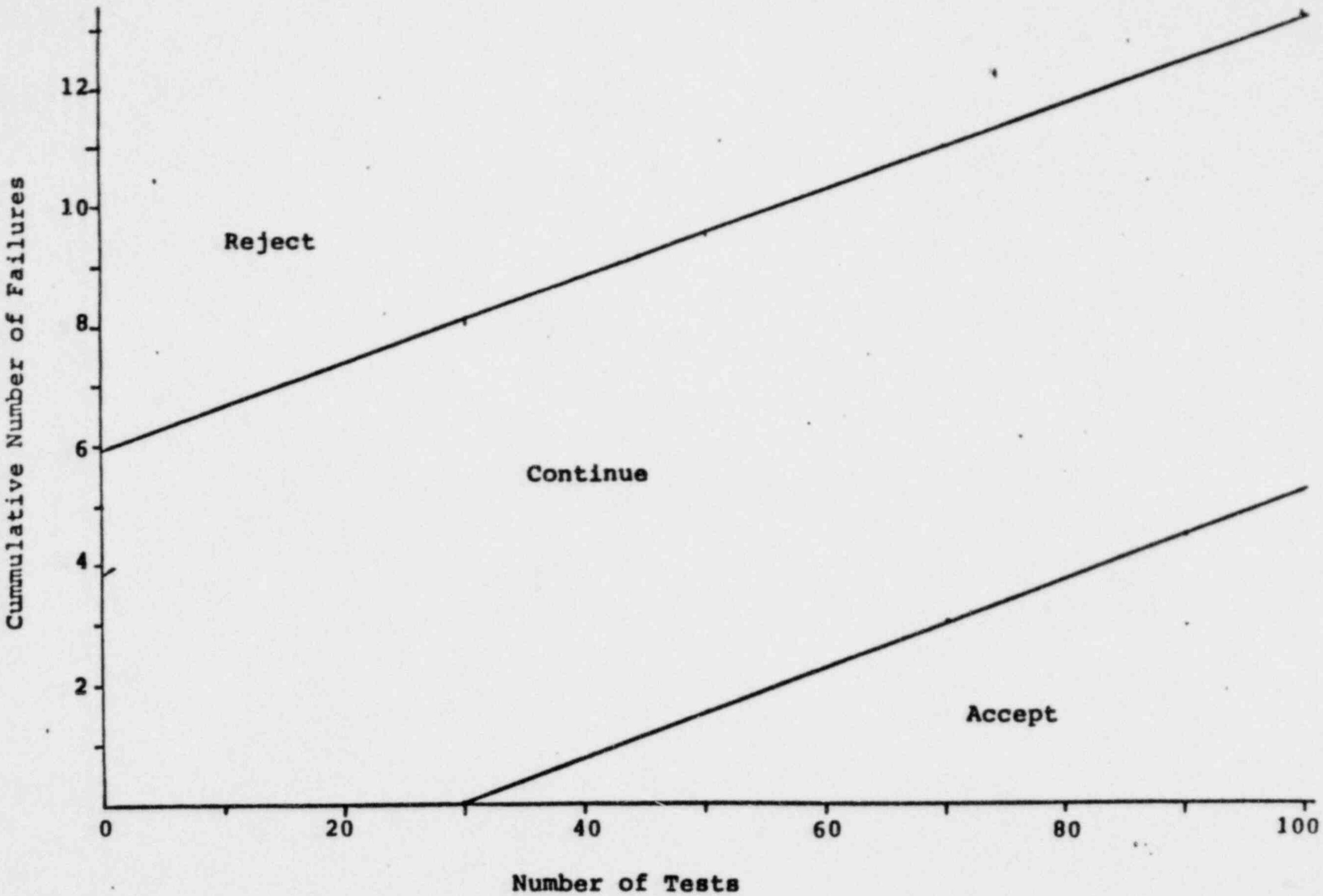
7. Inspection of Repairs

   a. Perform and document grout repair inspection per applicable BECo QC procedure to verify satisfactory completion of items 1 through 6 above.

Att [illegible]nt 2

Pilgrim I Masonry Wall Analysis

Sequential Testing Procedure



Based on Reference 3.3

Pilgrim I Masonry Wall Testing Program

Results Log

Wall No	Testing Method	Test No	Results						Remarks	Inspected by (date)	Checked by (date)	QC Verifi (date)
			Single or Multi-wythe				Multi-wythe/ composite					
			Vertical Rebar Observed	Dur-o-wall Observed	Anchorage Observed	Grouted Cell Observed	Type	Max Void Size				

Exhibit A
WI - 6
Rev 2

Job No. 80034
Sh. 2 of 2

RESULTS LOG
(Example)

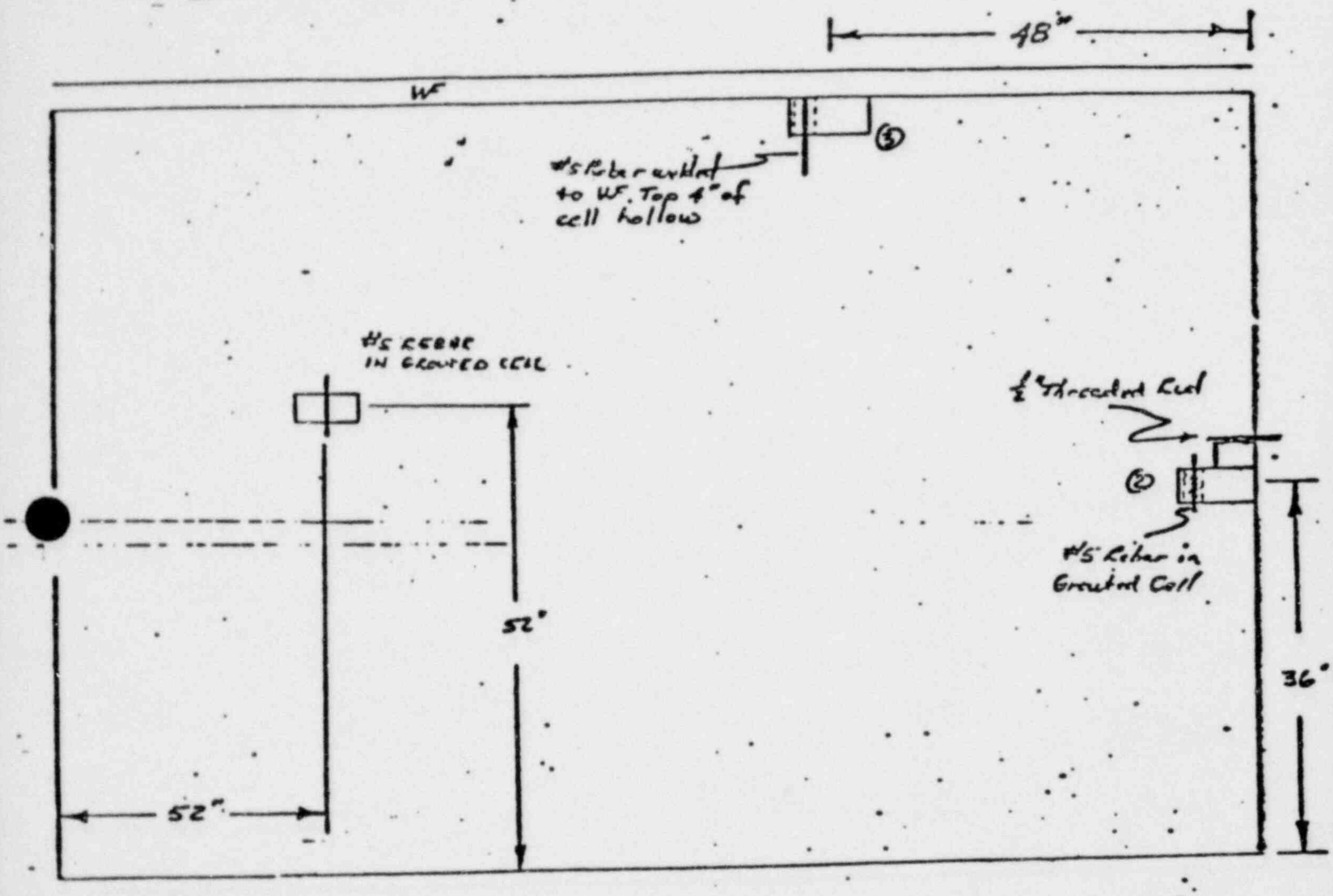
WALL NUMBER 65.

IN REPT. NO. I-81-

BUILDING Reactor El 51'-0"

LOCATION (3) LINE EAST FACE

WALL TYPE 8" SINGLE WYTHE



WALL ELEVATION

Exhibit A
WI - 6
Rev. 2

Specification No. 6498-A-1
Rev. 1

SPECIFICATION
FOR
FURNISHING, DELIVERY AND INSTALLATION
OF
CONCRETE UNIT MASONRY
FOR
UNIT NO. 1
PILGRIM STATION NO. 600
BOSTON EDISON COMPANY

CONTENTS:

SUBCONTRACT FORM
SUPPLEMENTARY GENERAL CONDITIONS
GENERAL TERMS AND CONDITIONS
INSTRUCTIONS AND INFORMATION TO BIDDERS
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SUMMARY OF PROPOSAL

		Revision 1
Prepared by	mp, elm	[illegible]
Date	7-28-69	2-1-72
Approved by	[illegible]	[illegible]
Date	7-28-69	2-1-72

Bechtel Corporation
San Francisco, California

SPECIFIC CONDITIONS

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SPECIFICATION
FOR
FURNISHING, DELIVERY AND INSTALLATION
OF
CONCRETE UNIT MASONRY
FOR
UNIT NO. 1
PILGRIM STATION NO. 600
BOSTON EDISON COMPANY

SPECIFIC CONDITIONS

1.0 GENERAL

The Work is subject to the "Instructions and Information to Bidders" preceding these Specific Conditions, the "General Terms and Conditions" and the Supplementary General Conditions of the Subcontract of which this Specification is a part. The Subcontractor shall be responsible for meeting and be governed by all of the requirements therein.

2.0 WORK INCLUDED

The Work includes the furnishing of all labor, supervision, materials, tools and equipment and the performance of all operations necessary for the furnishing, delivery, unloading and erection of masonry work, as specified herein and as shown on the referenced drawings. The Work includes, but is not limited to, the following:

2.1 Furnishing and installing of concrete unit masonry.

2.2 Furnishing and installing of all reinforcing required in concrete unit walls.

2.3 Furnishing of test specimen as may be required.

3.0 WORK NOT INCLUDED

The following items of work are not included:

3.1 Brick masonry.

3.2 Furnishing and installing of door frames, anchors, sleeves, inserts, etc.

4.0 DESIGN DRAWINGS

The nature and extent of the work is shown on the drawings listed in the Requisition.

5.0 TIME REQUIREMENTS

The work shall be performed in accordance with the requirements stated in the Requisition.

6.0 CO-OPERATION WITH OTHERS AT THE SITE

The Contractor and other Subcontractors will be performing work at the site concurrently with this Subcontractor's operations. The Subcontractor shall so conduct his work as not to interfere unduly with the work of others.

7.0 APPLICABLE PUBLICATIONS

The latest editions of the publications listed below, but referred to hereafter by basic designation only, form part of this specification to the extent indicated by the references thereto:

7.1 United States of America Standards Institute:

1972

ASTM A82:	Specification for Cold-Drawn Steel Wire for Concrete Reinforcement
ASTM A15:	Specification for Deformed Billet Steel for Concrete Reinforcement
ASTM C90:	Specification for Hollow Load-Bearing Concrete Masonry Units
ASTM C140:	Sampling and Testing Concrete Masonry Units
ASTM C144:	Specification for Aggregate for Masonry Mortar
ASTM C150:	Specification for Portland Cement
ASTM C143:	Slump of Portland Cement Concrete
ASTM C145:	Solid Load Bearing Concrete Masonry Units
ASTM C305:	Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement
ASTM C33:	Specification for Normal Weight Aggregates for Concrete Masonry Units
ASTM C331:	Specification for Lightweight Aggregates for Concrete Masonry Units
ASTM C207:	Specification for Hydrated Lime for Masonry Purposes
ASTM C404:	Specification for Aggregates for Masonry Grout
ASTM C476:	Specification for Mortar and Grout for Reinforced Masonry

8.0 SAMPLES AND CERTIFICATES

8.1 Samples

Before delivery of any concrete units to the site the Subcontractor shall submit to the Contractor for approval at the site two units of each size he proposes to use. No work shall be started until such approval in writing has been received by the Subcontractor.

8.2 Certificate

The Subcontractor shall furnish certificates in triplicate accompanying each shipment verifying the following:

8.2.1 Name of Contractor

8.2.2 Project location

8.2.3 Quantity of each type, dimension, and average dry unit weight

8.2.4 Method of shipment

8.2.5 Date or dates of shipment or delivery to which certificate applies

8.2.6 Linear shrinkage

8.2.7 Moisture content

8.2.8 Curing method, duration, temperature, date started and completed

8.2.9 Type of aggregate

8.2.10 Concrete mix, material and ratio

9.0 CONCRETE MASONRY UNITS

9.1 Hollow Heavyweight Concrete Masonry Units

Shall be Grade U-I units conforming to ASTM C90. Linear shrinkage shall be limited to 0.05 per cent. Oven dry concrete weight for units shall be not less than 120 pounds per cubic foot. Sizes and location as shown on drawings.

9.2 Solid Heavyweight Concrete Masonry Units

Shall be Grade U-I units conforming to ASTM C145. Linear shrinkage shall be limited to 0.05 per cent. Oven dry concrete weight for units shall be not less than 120 pounds per cubic foot. Sizes and location as shown on drawings.

9.3 Hollow Lightweight Concrete Masonry Units

Shall be Grade U-I units conforming to ASTM C90. Linear shrinkage shall be limited to 0.05 per cent. Oven dry concrete weight for units shall be not less than 105 pounds per cubic foot. Sizes and location as shown on drawings.

10.0 MATERIALS

10.1 Portland cement shall conform to ASTM C150, Type I or II.

10.2 Hydrated lime shall conform to ASTM C207, Type S.

10.3 Aggregate for Heavyweight concrete units shall conform to ASTM C33.

10.4 Aggregate for Lightweight concrete units shall conform to ASTM C331.

10.5 Aggregate for mortar shall conform to ASTM C144, either natural or manufactured.

10.6 Aggregate for grout shall consist of natural or manufactured sand used in combination with natural coarse aggregate in accordance with ASTM C404.

10.7 Grout admixture shall be a type which reduces early water loss to the masonry units, reduces initial shrinkage and promotes bonding of the grout to all interior surfaces of the masonry units. Admixture shall have the written approval of the Contractor.

10.8 Water shall be free of injurious amounts of oil, acid, alkali, organic matter or other deleterious substances and shall be potable.

10.9 Reinforcement

10.9.1 Bars shall conform to ASTM A15, Grade 40.

10.9.2 Horizontal Joint Reinforcement shall be "Dur-O-Wal" heavyweight truss type or approved equal and shall conform to ASTM A82 or as shown on the Design Drawings.

10.10 Control joints shall be "Dur-O-Wal" wide flange rapid control joint.

11.0 HANDLING AND STORAGE

11.1 Concrete blocks shall be stacked under a cover or otherwise protected from exposure to the weather and from contact with the soil immediately upon delivery to the site. Care shall be exercised in handling these items to avoid chipping and breakage, and to protect them from damage by construction operations.

11.2 Cementitious materials, immediately upon delivery to the site, shall be stored in weatherproof sheds, or upon platforms raised free from the ground and effectively protected from weather or moisture until used.

12.0 PROPORTIONING AND MIXING OF MORTAR AND GROUT

Mortar and grout for solid unit masonry and reinforced hollow unit masonry construction shall conform to ASTM C476 and as specified in this paragraph as follows:

12.1 Mortar Proportioning

Mortar shall be type PL proportioned within the limits of ASTM C476, Table I as follows: one cubic foot of Portland Cement; one-half cubic foot of Hydrated Lime; and not less than 3-1/2 and not more than 4-1/2 cubic feet of fine aggregate.

Mortar shall attain a minimum compressive strength of 2000 pounds per square inch at 28 days.

Mortar shall be mixed and maintained on the boards to a slump of approximately 2-3/4 inches using a truncated cone 4 inches to 2 inches, 6 inches high.

12.2 Grout Proportioning

Grout type shall be Coarse Grout proportioned within the limits of ASTM C476, Table II as follows: one part Portland Cement to which may be added not more than one-tenth part Hydrated Lime; three parts fine aggregate; two parts coarse aggregate. All measurements shall be by volume.

Grout dry weight in place shall be not less than 150 pounds per cubic foot.

Grout shall have a slump of approximately 4-1/2 inches using a truncated cone 4 inches to 2 inches, 6 inches high.

Grout shall attain a minimum compressive strength of 2000 pounds per square inch at 28 days.

12.3 Mortar Mixing

Mortar shall be prepared in batches of the volume that will be used before the initial set takes place and shall be placed within two hours after mixing. Mortar which has stiffened shall be retempered to restore its workability and water shall be added as needed during the maximum time interval specified above. As much mixing water as is practicable without impairing the workability of the mortar shall be used.

12.4 Grout Mixing

Sufficient water shall be added to make a workable mix that will flow into all joints of the masonry without separation or segregation. Grout shall be used within 45 minutes after mixing. Grout shall not be used after the cement has begun its final set.

13.0 MASONRY CONSTRUCTION

13.1 Masonry Units

13.1.1 Masonry units shall be sound, dry, clean and free of cracks or broken corners when placed in the structure.

13.1.2 The proper masonry unit shape shall be used in bond beams, lintels, pilasters and so forth with a minimum of cutting. Cuts in masonry units shall be neat and true.

13.2 Laying Masonry Units

13.2.1 All block work shall be plumb, level and true to line and all corners and angles shall be square unless otherwise indicated on the drawings.

13.2.2 Line blocks shall be used wherever possible. When it is absolutely necessary to use a line

pin, the hole in the joint shall be filled with mortar immediately after the pin is withdrawn.

13.2.3 All masonry units shall be laid with full head and bed joints. Blocks shall have full mortar coverage on horizontal and vertical faces.

13.2.4 Mortar joints shall be three-eighths inch (3/8") in thickness unless otherwise indicated on the drawings.

13.2.5 Mortar for all bed joints shall be spread or buttered on to the face shell and cross webs of the unit below. In walls to be grouted solid, all cross webs shall be spread or buttered. Ends of bed joint mortar that protrude into the vertically aligned cell shall be avoided. If they occur, however, and do not exceed the 3/8 inch thickness of the bed joint, they shall be left in place. If they exceed the 3/8 inch thickness, they shall be knocked off and removed through the cleanouts.

13.2.6 If it is necessary to move a unit, either for alignment or any other reason after it has once set in place, the unit shall be removed from the wall, cleaned and reset in fresh mortar.

13.2.7 All unfinished work exposed to the weather shall be protected from rain by covering the top of the wall to prevent the entrance of water into the open wall. Exposed joints in exterior wall surfaces shall be tooled and joints in interior wall surfaces shall be finished in accordance with the details shown.

13.2.8 When it is necessary to stop off a longitudinal run of masonry, it shall be stopped off only by racking back one-half unit length in each course. Toothing will not be permitted except as shown or as authorized.

13.3 Bonding

13.3.1 For bonding the masonry to the foundation the top surface of the concrete foundation shall be thoroughly roughened and cleaned with laitance removed before starting the masonry construction.

13.3.2 The starting joint on foundations shall be laid with full mortar coverage on the bed joint except that the area where grout occurs shall be kept free from mortar so that the grout will contact the foundation.

13.3.3 Where no bond pattern is shown the wall shall be laid up in straight uniform courses with regular running bond. No toothing shall be allowed.

13.3.4 Intersecting masonry walls and partitions shall be bonded by the use of steel ties at 24 inches o.c. minimum unless noted otherwise. Corners shall have a standard masonry bond by overlapping units and shall be grouted solid.

13.3.5 Anchors, wall plugs, accessories and other embedded items shall be fully and solidly grouted in place.

13.4 Reinforcing

13.4.1 When a foundation dowel does not line up with a vertical core it shall not be sloped more than one horizontal in six vertical. Dowels shall be grouted into a core in vertical alignment even though it is in an adjacent cell to the vertical wall reinforcing.

13.4.2 Reinforcement shall be placed as shown on the drawings. Reinforcing bars shall be accurately placed and adequately anchored in place until the grout has hardened sufficiently to support the bars.

13.5 Grouting

13.5.1 All cells of hollow unit masonry walls for radiation shielding shall be grouted solid.

13.5.2 Grout may be placed in the hollow unit masonry wall after the units have been set for twenty-four hours.

13.5.3 Grout may be placed by grout pump, concrete hopper, or bucket.

13.5.4 Grout spaces shall not be wetted down prior to pouring grout.

13.5.5 Grout shall be poured in lifts not to exceed four feet. The first lift shall be consolidated with a three-quarter inch (3/4") flexible cable vibrator immediately after placement. Consolidation of the second grout lift and reconsolidation of the second grout lift may be done in the same operation. Reconsolidation of the last lift of the pour is required as a special and individual operation.

13.5.6 When work is to be stopped for a period of 45 minutes or longer, the pour shall be stopped approximately 1-1/2 inches below the top of the last course and the surface of the grout shall be thoroughly roughened. When work is resumed, the laitance shall be removed and the existing grout shall be dampened and coated with neat cement before additional grout is poured.

13.6 Jointing

All mortar joints shall be flush, clean and uniform in thickness and appearance, unless otherwise noted on the drawings. Defective joints shall be cut out and repointed.

13.7 Pointing and Cleaning

13.7.1 Mortar and grout stains on the face of the wall shall be removed immediately.

13.7.2 At the completion of the Work all holes and defective mortar joints in the exposed masonry shall be repointed. The Subcontractor shall clean all masonry, remove scaffolding and equipment used in the Work, remove all debris and refuse pertaining to the Work ans shall remove all surplus masonry materials.

14.0 WEATHER PRECAUTIONS

No exterior masonry shall be erected when the temperature is below 40°F. Interior masonry may be erected when the temperature outside is below 40°F, provided that an ambient temperature above 40°F is maintained while erection is in progress and for a period of not less than 48 hours after erection has stopped.

15.0 TEST SPECIMENS

The Contractor will sample and test concrete masonry units for compressive strength, absorption, weight, moisture content and dimensions in accordance with ASTM C140. Samples for test will be taken at the place of manufacture from the lots ready for delivery.

The Contractor will random sample concrete masonry units at the time of delivery to the job site for weight tests. The tests will be recorded.

The Subcontractor shall provide samples of concrete masonry units, mortar, or grout for tests when and as directed.

16.0 INSPECTION

The Contractor will inspect the Work during construction and upon completion of construction of each wall.

Method of inspection may be by taking sample cores, removing face panels of blocks at bottom of wall or by radiology.

Any wall or portion of wall which is found to have voids in the grout which the Contractor decides would affect the structural requirements or the radiation shielding effectiveness shall be replaced by the Subcontractor in a manner which will be determined and approved by the Contractor. The Subcontractor shall replace all face panels defaced by inspection. The cost of replacement shall be by Contractor where grout consolidation is satisfactory.

17.0 SAFETY

The Subcontractor's work shall comply with all local, state and Federal safety requirements. This compliance is mandatory for all phases of work, personnel, equipment and materials.

18.0 CLEAN-UP

The Subcontractor shall clean up all debris daily and shall remove all unnecessary scaffolding, equipment and surplus materials. When the work is complete, the premises shall be left in a neat and clean condition in respect to the Subcontractor's work.

19.0 EXCEPTION TO SPECIFICATION

If the Bidder takes exception to portions of these "Specific Conditions" or any attachments thereto, he is required to attach to his proposal a statement identifying the exception in detail.

7. For tension normal to bed joints, an increase of 50% over the allowable value for normal loads was assigned to factored loads. However, the SEB criteria (2) allow only up to 30%. The Licensee should identify literature references or test data to support the use of a factor of 1.5.

The basis for the allowable tension stress values are provided in Appendix B of the revised criteria. It should be noted that the masonry walls at Pilgrim are vertically reinforced and allowable moments were computed based on tension in the reinforcing steel only.

The Owner's Group (R1) reported 14 tests of hollow unit construction with type M or S mortar (see Table 1). The mean value and standard deviation of the modulus of rupture from these tests are 88.4 and 14.3 psi respectively. Taking a conservatively low value of modulus of rupture of one standard deviation below mean gives 74.1 psi, which is approximatley a factor of 2 greater than the Pilgrim factored load allowable of 34 psi.

Additionally, Omote, et.al. (R10) reported tests for hollow brick masonry showing variation of modulus of rupture versus mortar compressive strength. This graph is reproduced in Figure 1 along with a curve showing twice the Pilgrim allowable stress for hollow unit masonry. The curve is conservative with respect to the data.

FLEXURAL STRENGTH—SINGLE WYTHE WALLS OF HOLLOW UNITS—
UNIFORM LOAD—VERTICAL SPAN

Mortar Type Proportion ASTM C 270	Modulus of Rupture psi, Net Area	Reference
M	110	10
M	108	NCMA
M	102	10
M	97	10
M	95	NCMA
S	94	NCMA
M	91	NCMA
M	89	NCMA
N	88	4
S	84	10
S	83	NCMA
S	81	10
S	75	NCMA
S	69	NCMA
N	67	4
N	62	4
S	60	10
N	58	4
N	45	4
O	60	10
O	41	4
O	36	4
O	36	4
O	33	4
O	32	4
O	30	10
O	27	4

TABLE 1

(From Owner's Group)

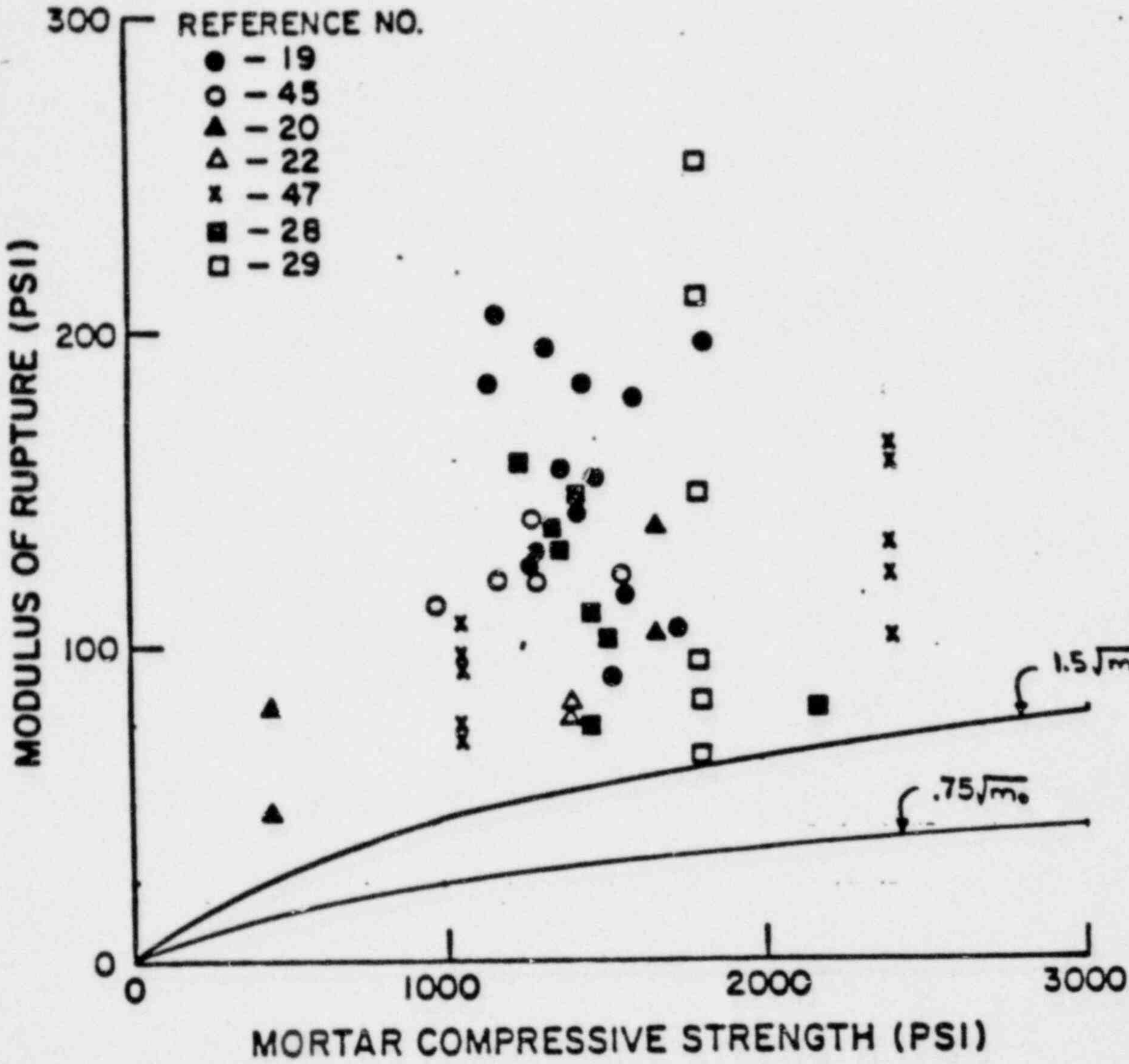


FIG. 5.9 EFFECT OF MORTAR COMPRESSIVE STRENGTH ON MODULUS OF RUPTURE OF BRICK WALLS

FIGURE 1

8. In Exhibit G of Attachment 3 (1), for factored loads, a factor of 1.5 was introduced for allowable shear with no special reinforcement (both in-plane and out-of-plane). SEB criteria (2) suggest a factor 1.3. The Licensee should cite literature reference or test data to support the use of a factor of 1.5.

The basis for the allowable shear stress values are provided in Appendix B of the revised criteria (R1). The ACI 531 and other masonry codes address shear stress for in-plane loads only. The Pilgrim criteria differentiates between in-plane shear and out-of-plane shear, which is analogous to peripheral shear in slabs.

The in-plane shear factored load allowables were derived from the work by Schneider (R4). A plot of his data and the Pilgrim factored load allowable is shown below. Pilgrim walls are reinforced but do not have special shear reinforcement. Materials used in the Schneider report are similar to those used at Pilgrim. The ultimate strength curve for fixed and grouted piers is reproduced below with the Pilgrim factored load allowable drawn in. The Pilgrim allowable maintains a factor of at least 2.0 below the comparable ultimate strength curve.

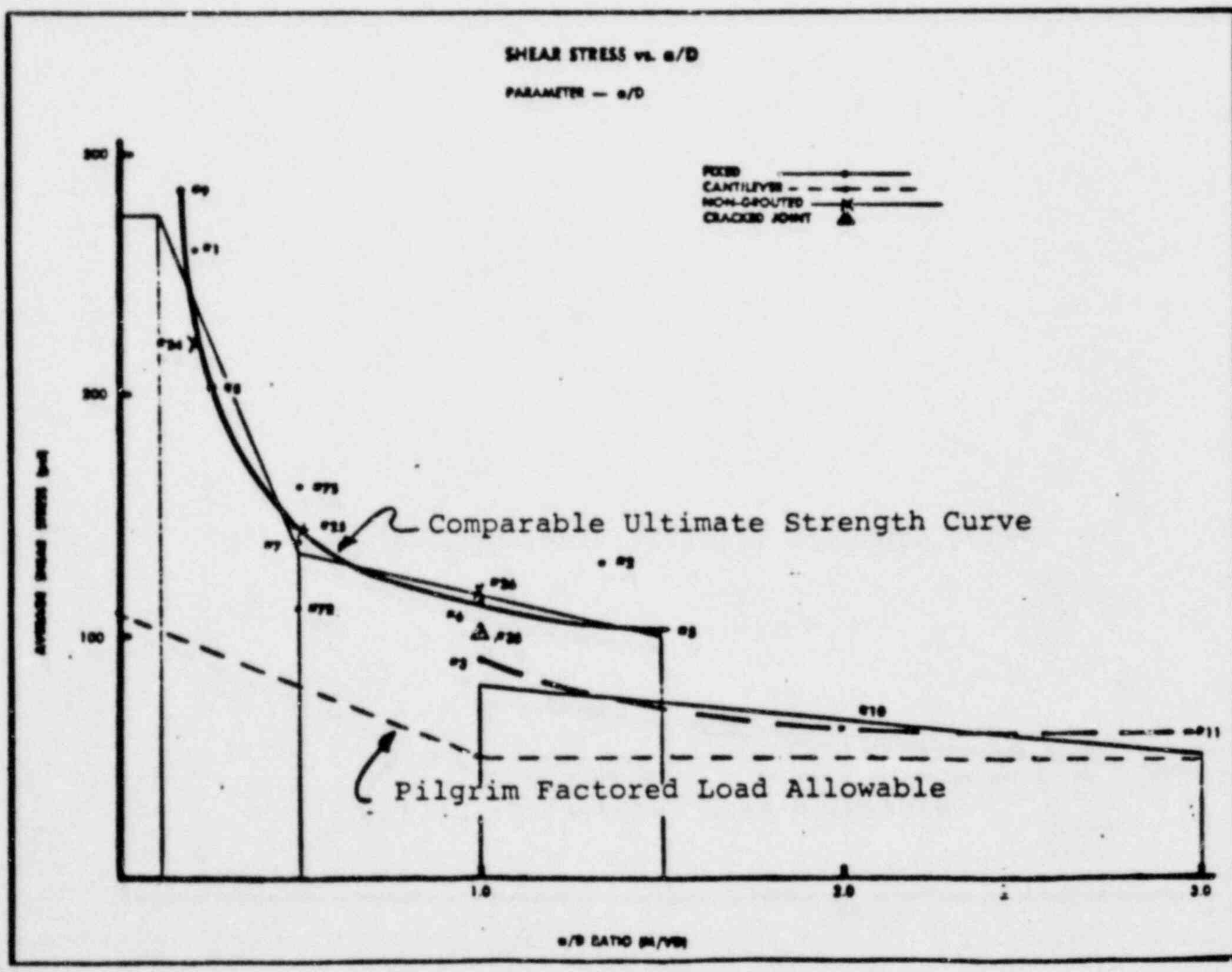


FIGURE 15

Additionally, a study was undertaken by Englekirk and Hart (R^4) to develop a basis for limit state design of masonry shear walls. The experimental work consisted of utilizing existing data from load/deflection tests. The most complete testing program was done at the Earthquake Engineering Research Center, University of California, Berkeley, California. A series of tests were conducted for wall height to width values of 0.5, 1.0 and 2.0, corresponding to M/Vd ratios of 0.25, 0.50 and 1.0. Since the ultimate compressive masonry stress, f'm, varied among tests, the ultimate shear stress is expressed in terms of f'm. The ultimate test shear stresses, using the average ultimate shear stresses for the minimum reinforcement level per UBC (1979 Edition) are:

$(M/Vd) = 0.25,\ V = 6.2\ \sqrt{f'm}$ (Ref. EERC 79/12, Test HCBL 12-3)

$(M/Vd) = 0.50,\ V = 3.3\ \sqrt{f'm}$ (Ref. EERC 78/28, Test HCBL 11-4)

$(M/Vd) = 1.00,\ V = 2.0\ \sqrt{f'm}$ (Ref. EERC 76/8, Test 6)

The value 2.0 f'm for M/Vd = 1.00 is a conservative value since the test specimen did not have horizontal steel.

From the test data, a limit state design standard was proposed. The values proposed for masonry without special reinforcement are shown below in comparison to the Pilgrim factored load allowables.

	Pilgrim Factored Load Allowable	Ultimate Strength Proposed Allowable
$M/Vd \geq 1.0$	$1.35\ \sqrt{f'm}$	$1.7\ \sqrt{f'm}$
$M/VD = 0.0$	$3.0\ \sqrt{f'm}$	$4.0\ \sqrt{f'm}$

9. <u>Provide appendices C, H, I, and J of Attachment 2 to the design criteria.</u>

The appendices to the design criteria contained data necessary to perform the concrete block wall evaluations. Since they were subject to change during the course of the project, they were eliminated from the revised criteria and re-issued as CYGNA project memoranda. The project memoranda corresponding to the original appendices are listed below and are included for your review as part of this response.

Appendix	Title	Project Memo No.
A	Response Spectra for Level 1 Masonry Wall Seismic Analysis	10
B	Response Spectra for Level 1 Masonry Wall Seismic Analysis	10
C	Differential Floor Displacement Values	7
D	Masonry Wall Section Properties	9
E	Attached Component/Equipment Weights for Dead Load Calculations	8
F	Loads due to Pipe Breaks Outside Containment	4
G	Allowable Stresses in Reinforced Masonry Walls	*
H	Support Conditions for Reinforced Masonry Walls	**
I	Allowable Block Pullout Loads	24
J	Tornado Loads	5

*See Attachment A of the revised criteria contained in Question #4.

**See Section 4.1.2 of the revised criteria, (DC-1, Rev. 1).

Memorandum

Project Memo #10
Revision #1

To: Project Personnel

Date: October 26, 1981

From: J. D. McWilliam

Job No: 80034

Subject: ARS for Level 1 & Level 2 Analysis

Copies: P. Baughman
C. DiNunzio
S. White
Project File
Central File

Attached are the amplified response spectra (ARS) that are to be used in the renalysis of masonry walls at Pilgrim I.

Attachment 1 includes all ARS for 2% damping (Design Earthquake) and 5% damping (Maximum Earthquake) for use in Level 1 analysis. Attachment 2 includes all ARS for 4% damping (Design Earthquake) and 7% damping (Maximum Earthquake) for use in Level 2 analysis. Refer to DC-1, Design Criteria for Re-evaluation of Masonry Walls for further instruction in the use of these curves.

J. D. McWilliam

J. D. McWilliam
Project Engineer

JDM/jp
attachment

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 1A

EARTHQUAKE: Design

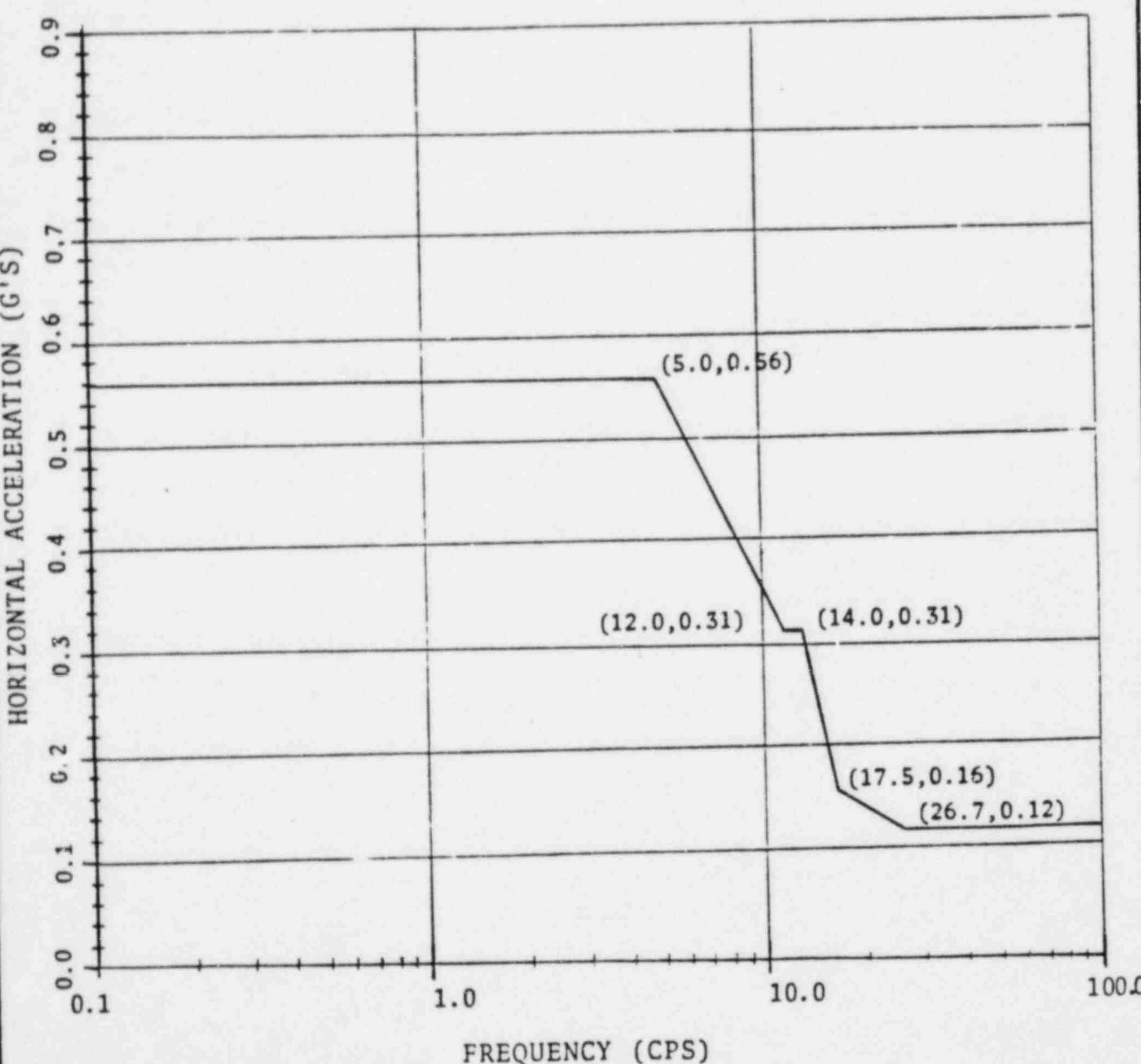
BUILDING: Reactor

DAMPING: 2%

ELEVATION: -17'-6"

MASS POINT: 1

Ref: 2.9



EES

PROJECT MEMO #10
80034
PILGRIM NUCLEAR POWER STATION
BOSTON EDISON COMPANY

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 1B

EARTHQUAKE: Maximum

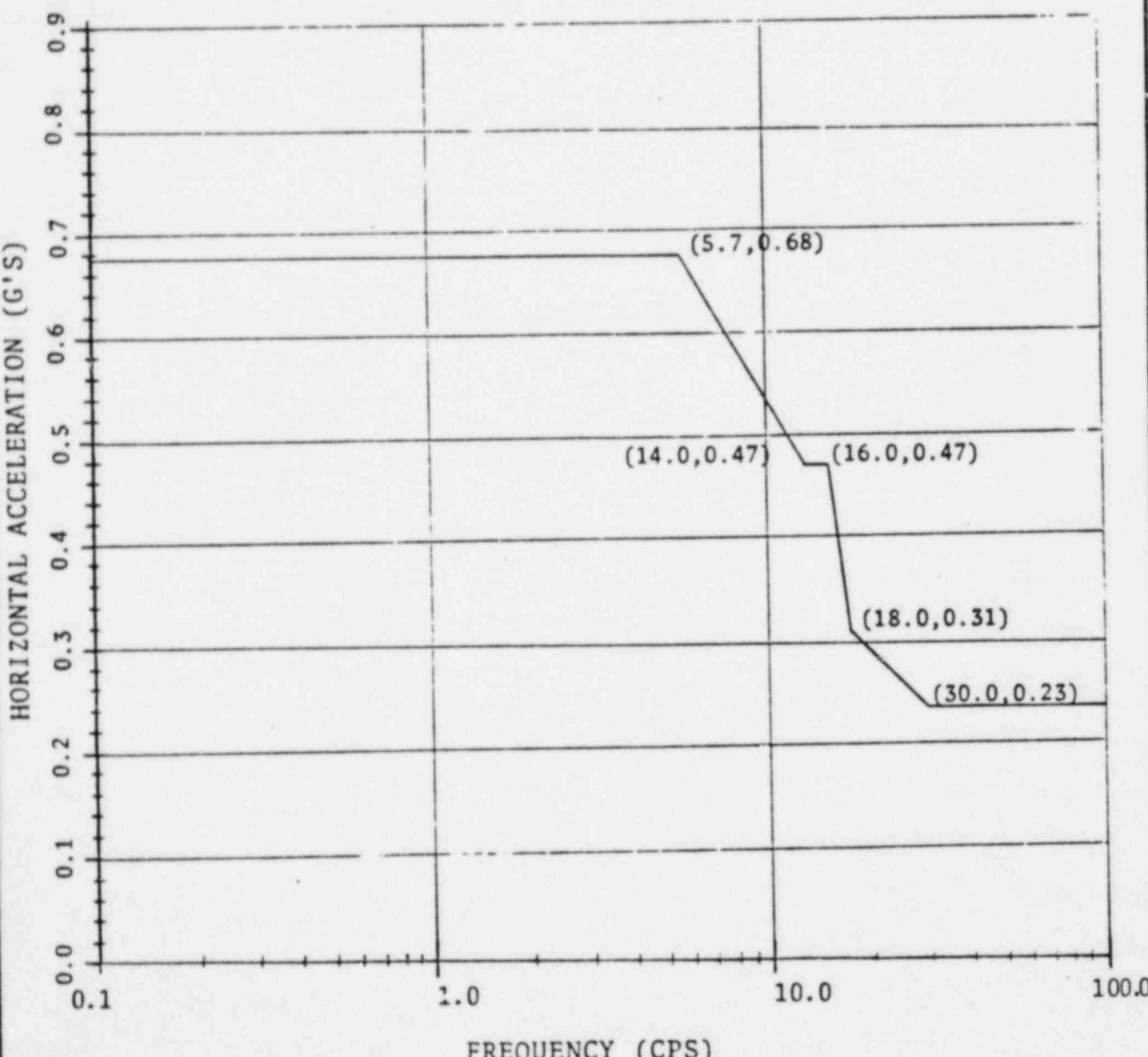
BUILDING: Reactor

DAMPING: 5%

ELEVATION: -17'-6"

MASS POINT: 1

Ref: 2.9



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 2A

EARTHQUAKE: Design

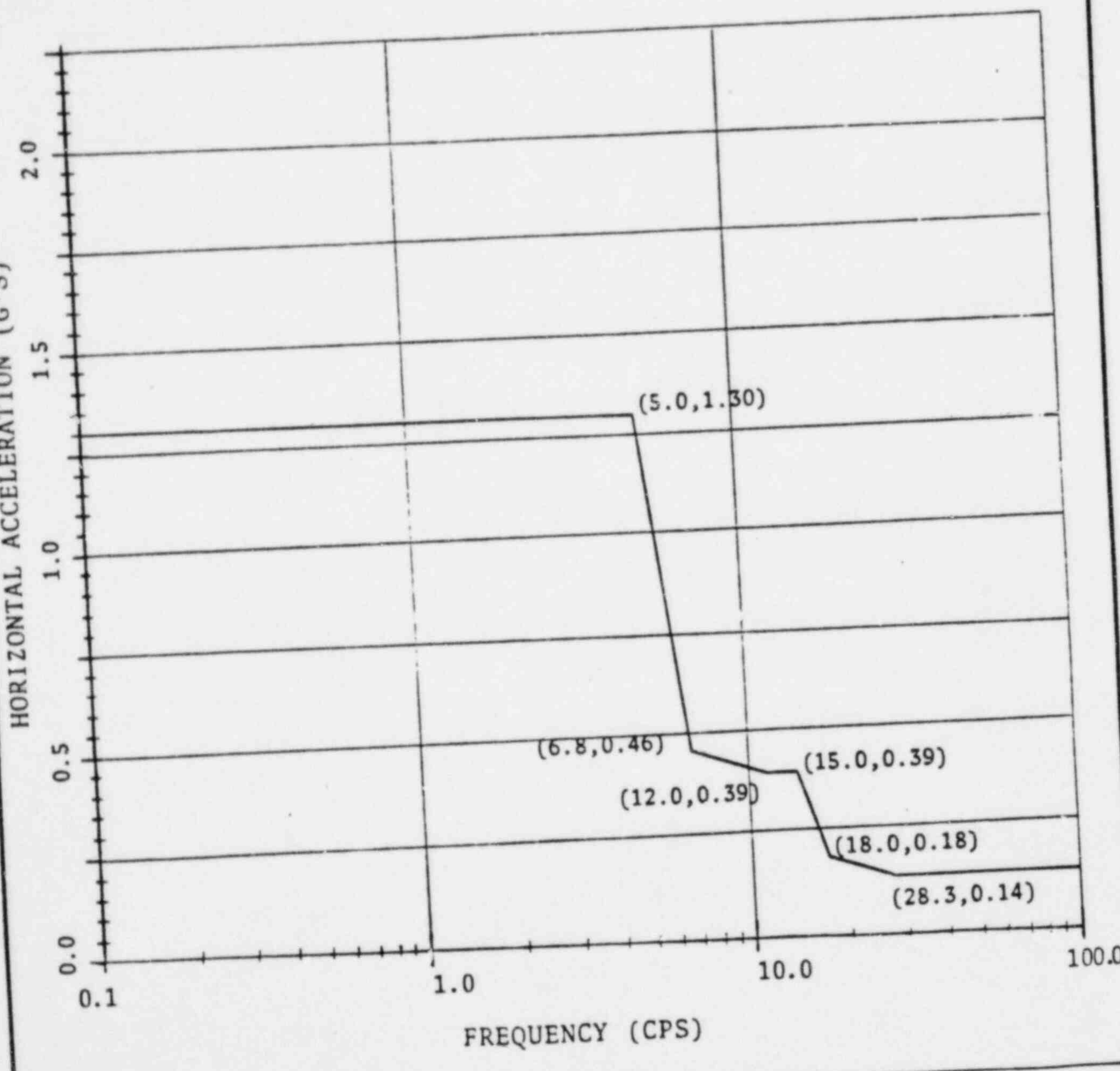
BUILDING: Reactor

DAMPING: 2%

ELEVATION: 23'-0"

MASS POINT: 2

Ref: 2.9



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 2B

EARTHQUAKE: Maximum

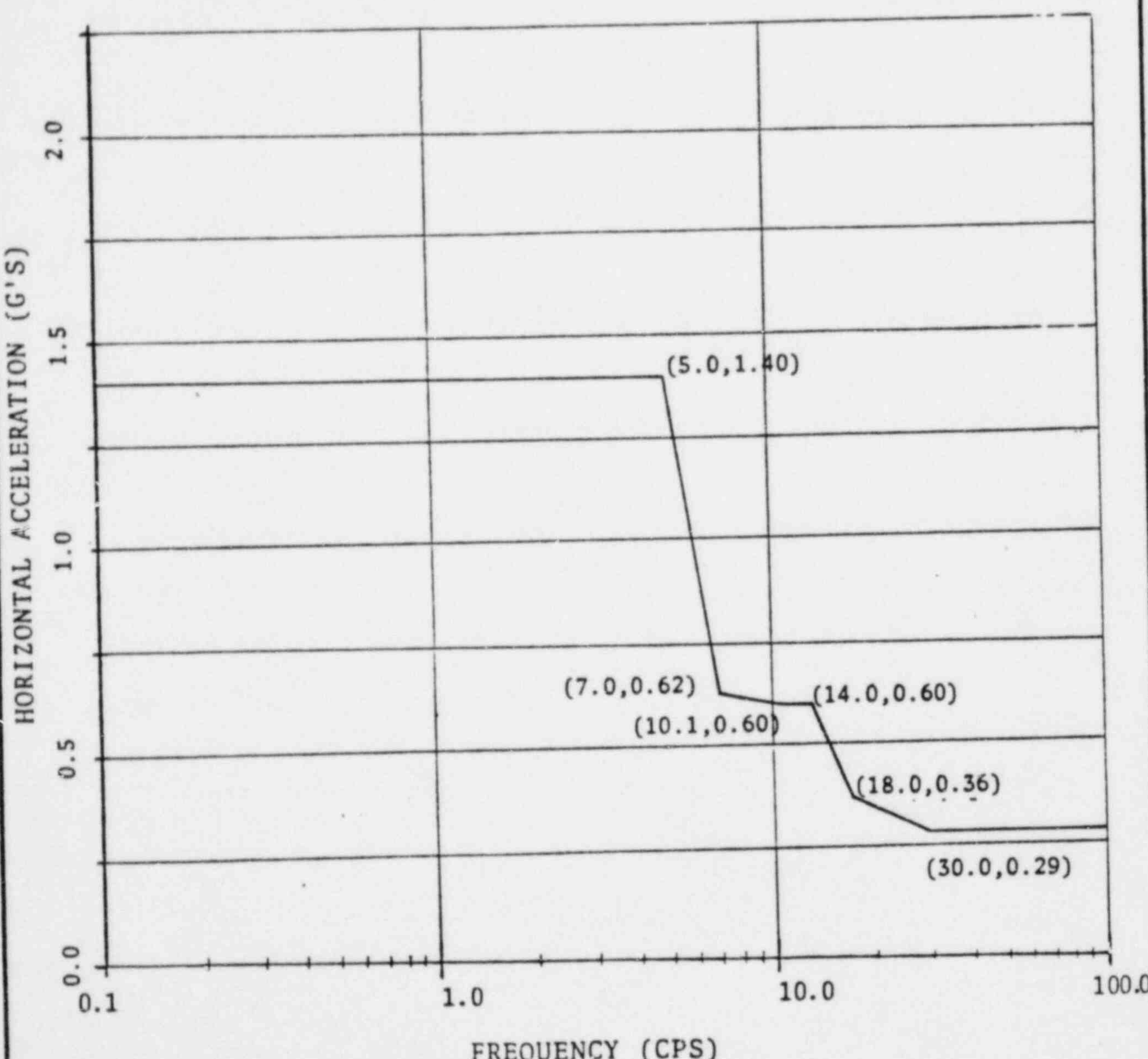
BUILDING: Reactor

DAMPING: 5%

ELEVATION: 23'-0"

MASS POINT: 2

Ref: 2.9



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 3A

EARTHQUAKE: Design

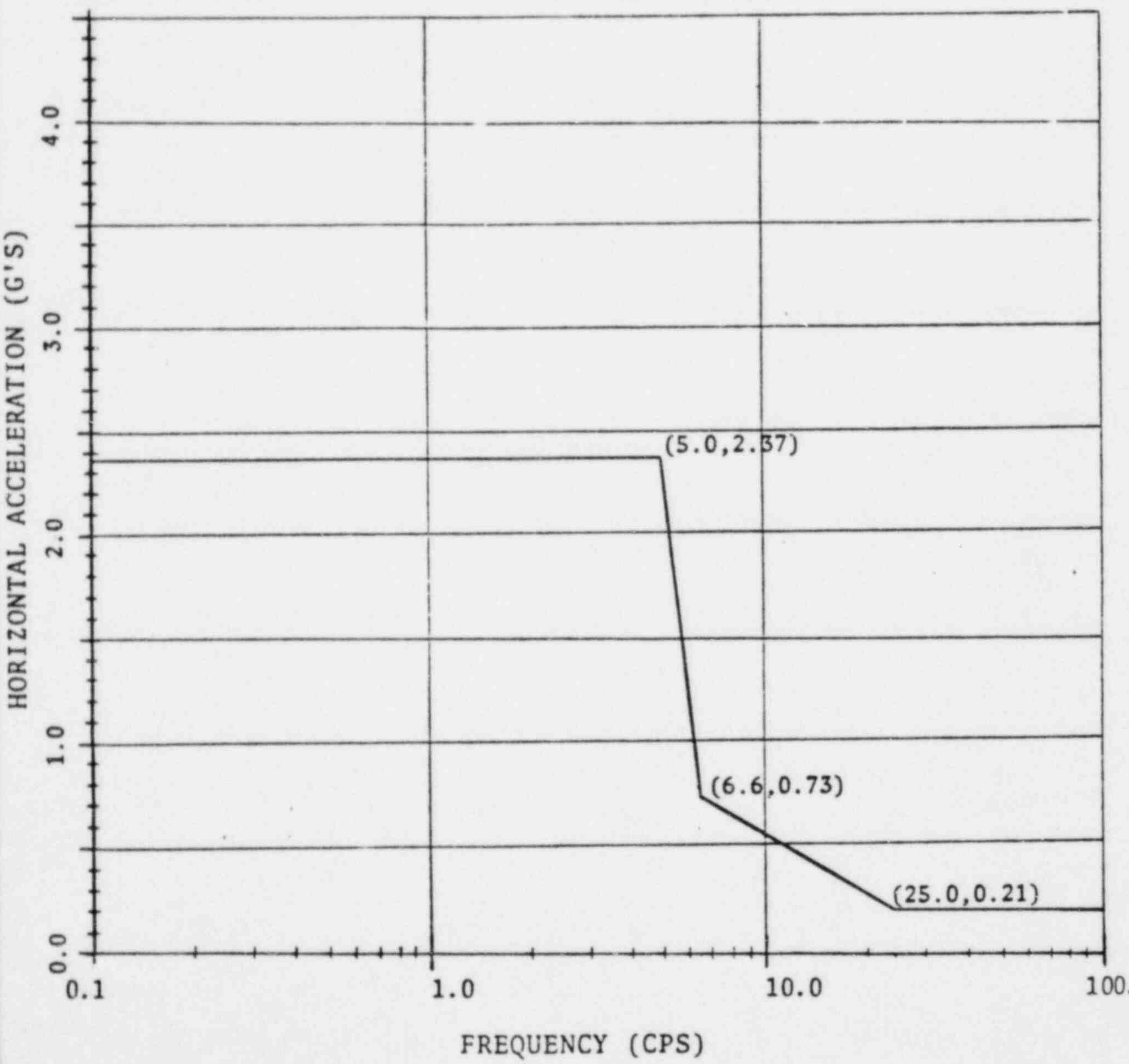
BUILDING: Reactor

DAMPING: 2%

ELEVATION: 51'-0"

MASS POINT: 3

Ref: 2.9



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 3B

EARTHQUAKE: Maximum

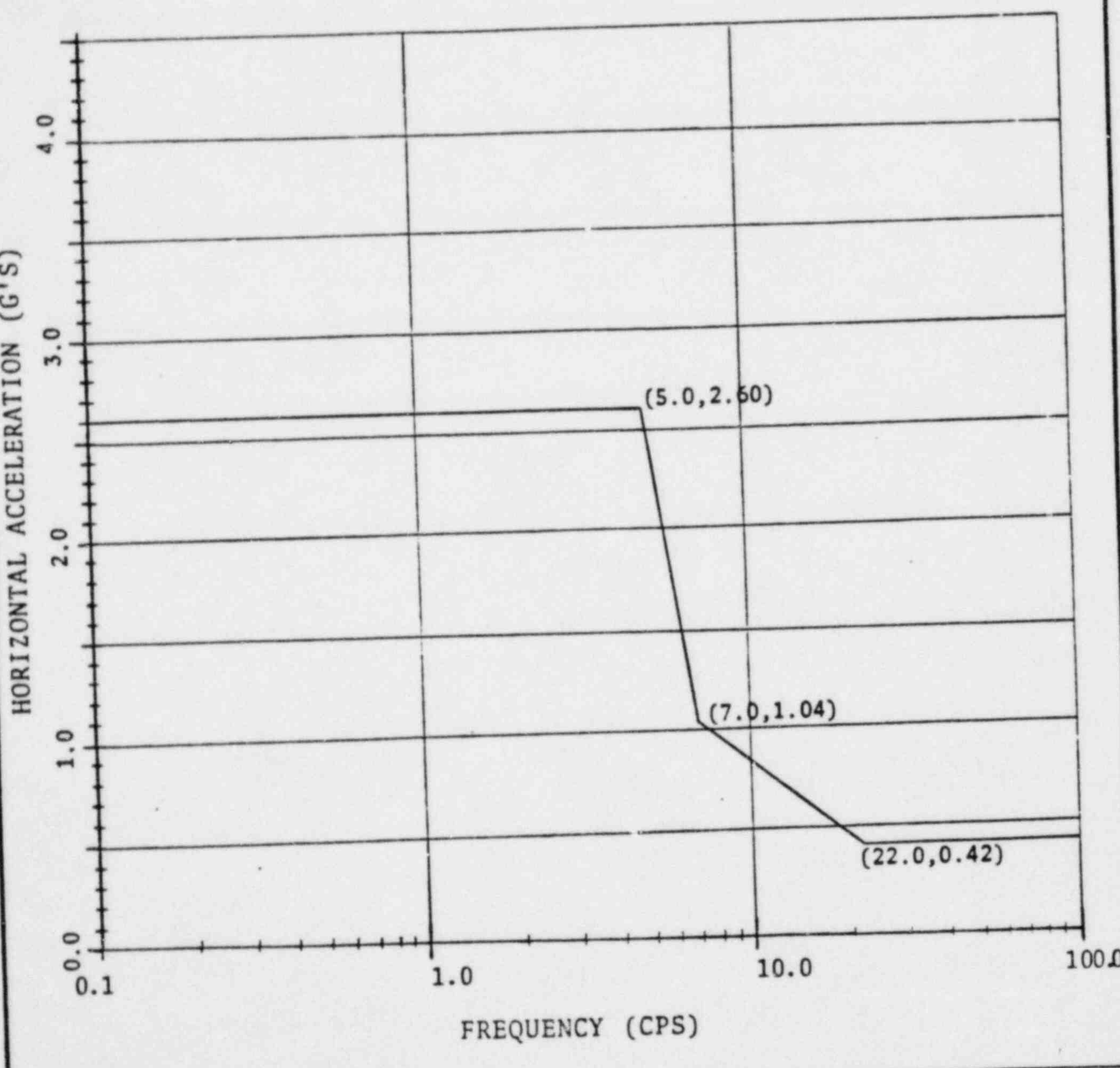
BUILDING: Reactor

DAMPING: 5%

ELEVATION: 51'-0"

MASS POINT: 3

Ref: 2.9



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80034
PILGRIM NUCLEAR POWER STATION
BOSTON EDISON COMPANY

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 4A

BUILDING: Reactor

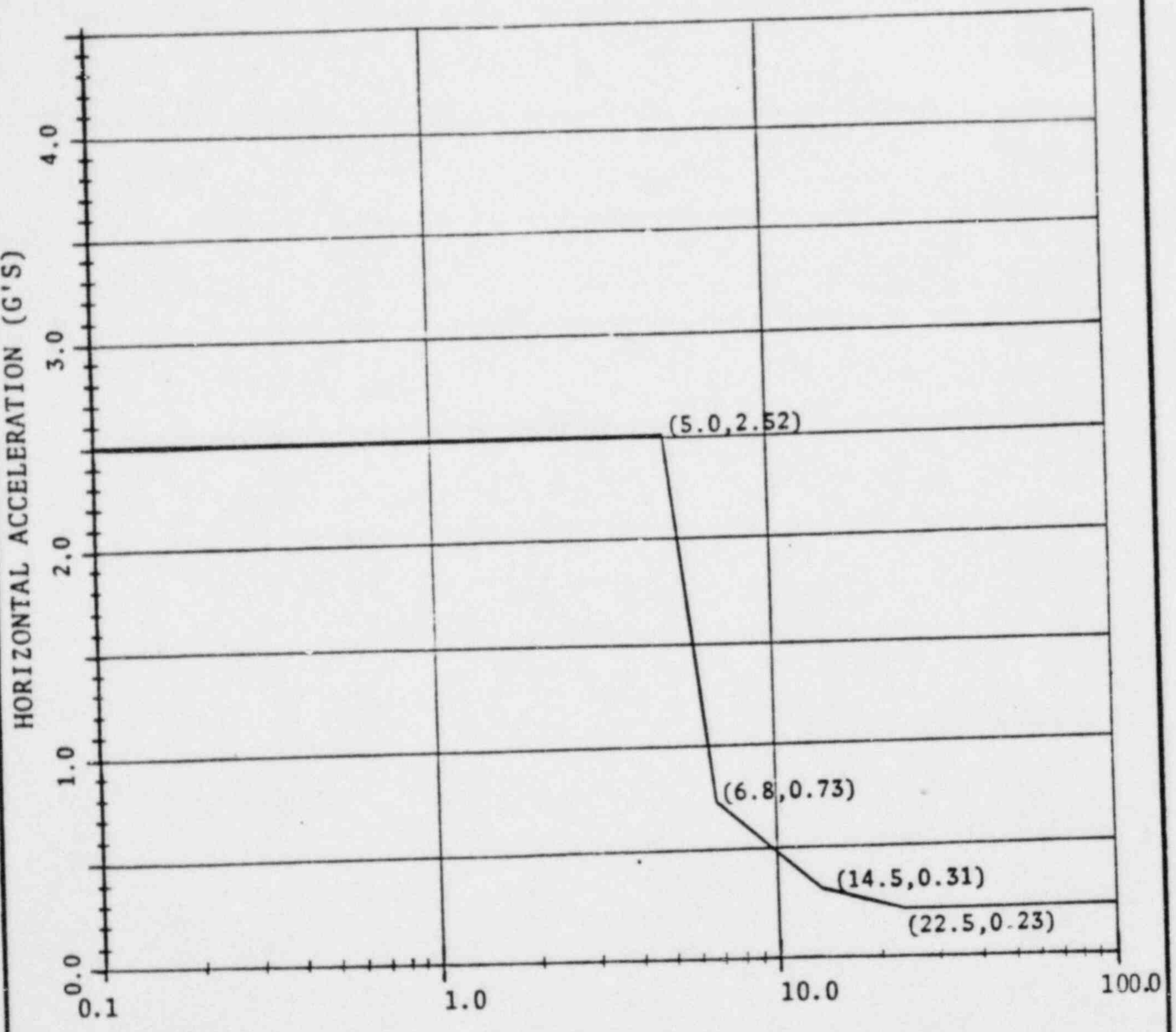
ELEVATION: 74'-3"

Ref: 2.9

EARTHQUAKE: Design

DAMPING: 2%

MASS POINT: 4



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 4B

EARTHQUAKE: Maximum

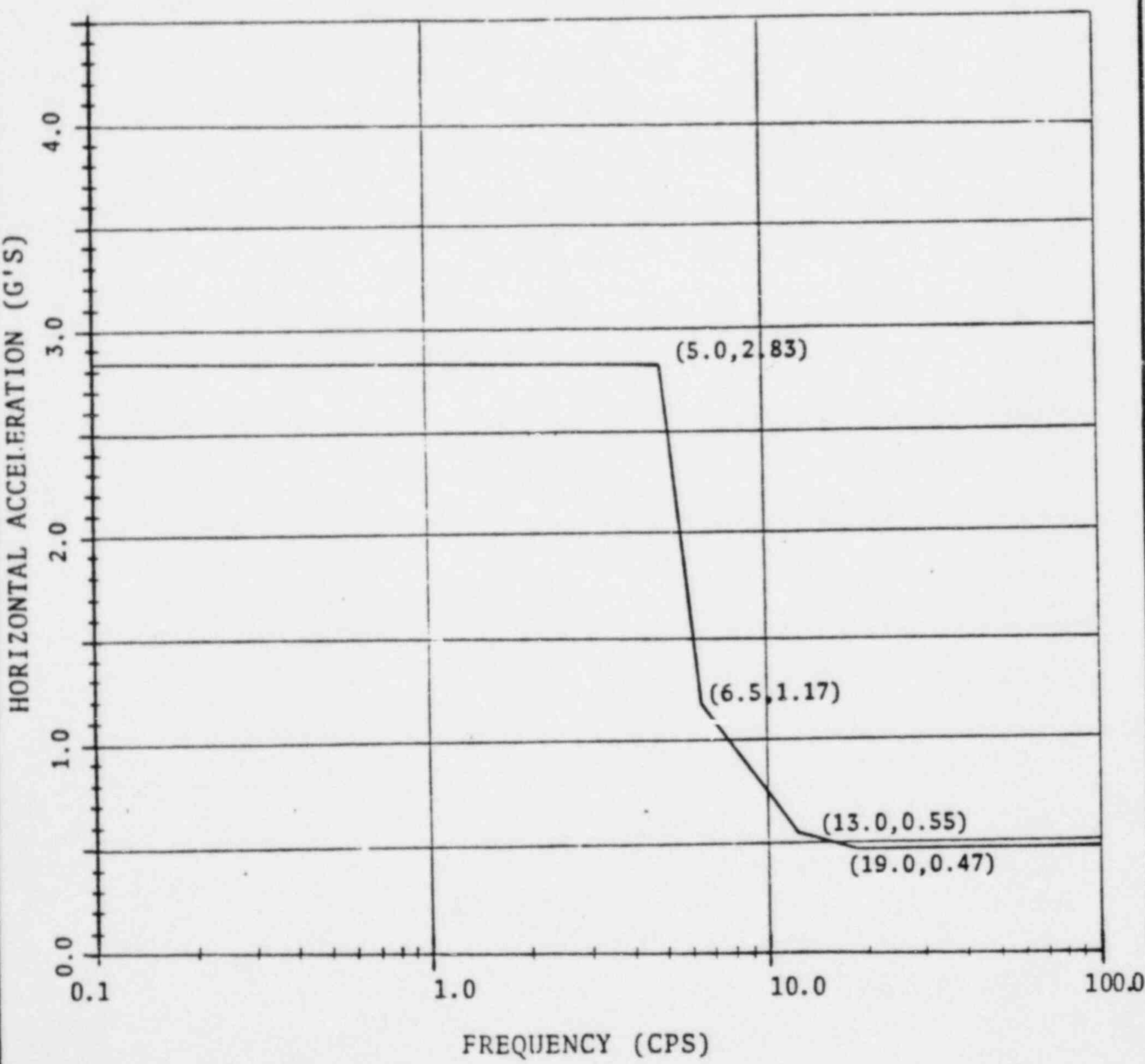
BUILDING: Reactor

DAMPING: 5%

ELEVATION: 74'-3"

MASS POINT: 4

Ref: 2.9



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 5A

BUILDING: Reactor

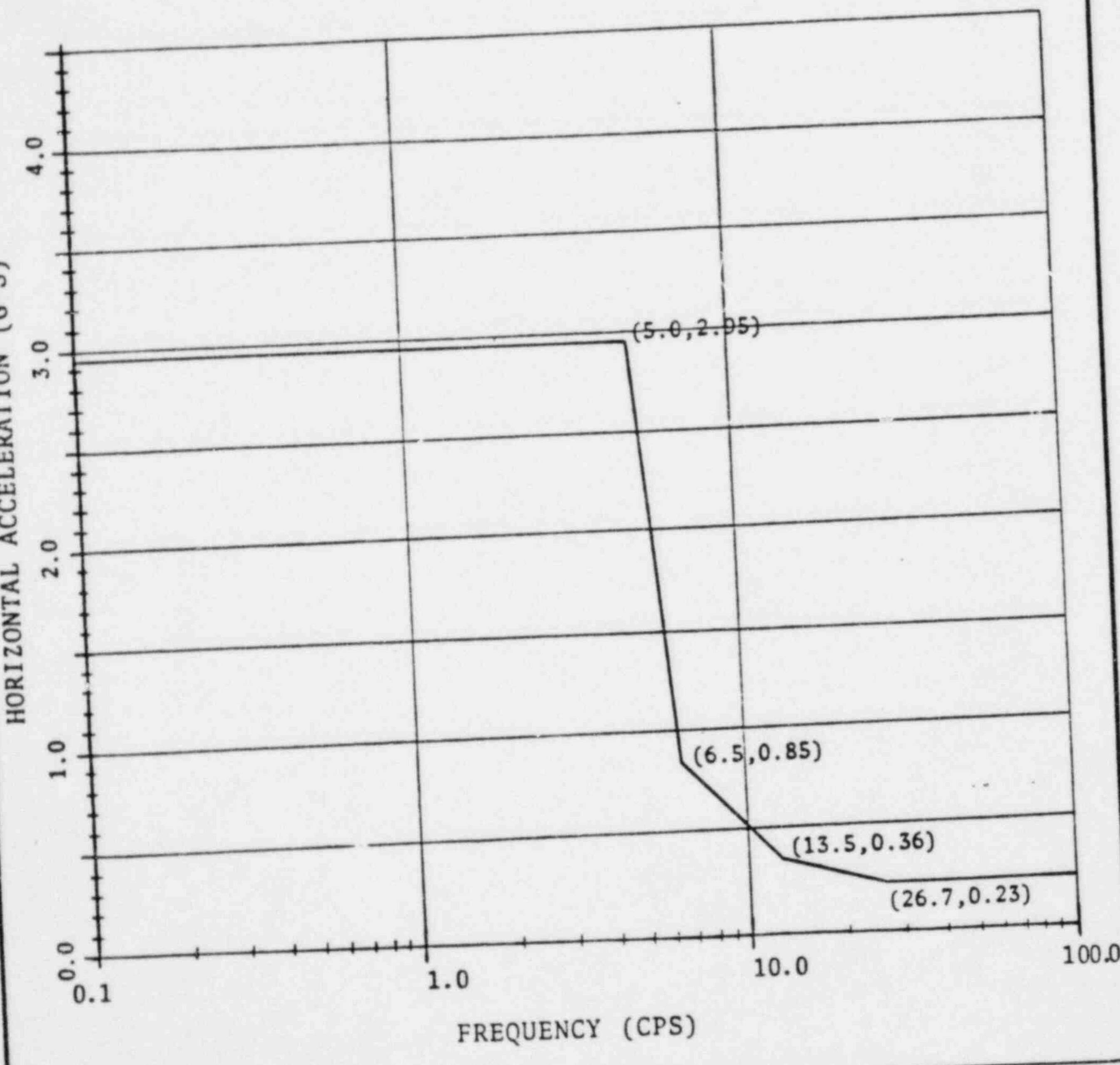
ELEVATION: 91'-3"

Ref: 2.9

EARTHQUAKE: Design

DAMPING: 2%

MASS POINT: 5



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BOSTON EDISON COMPANY

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 5B

BUILDING: Reactor

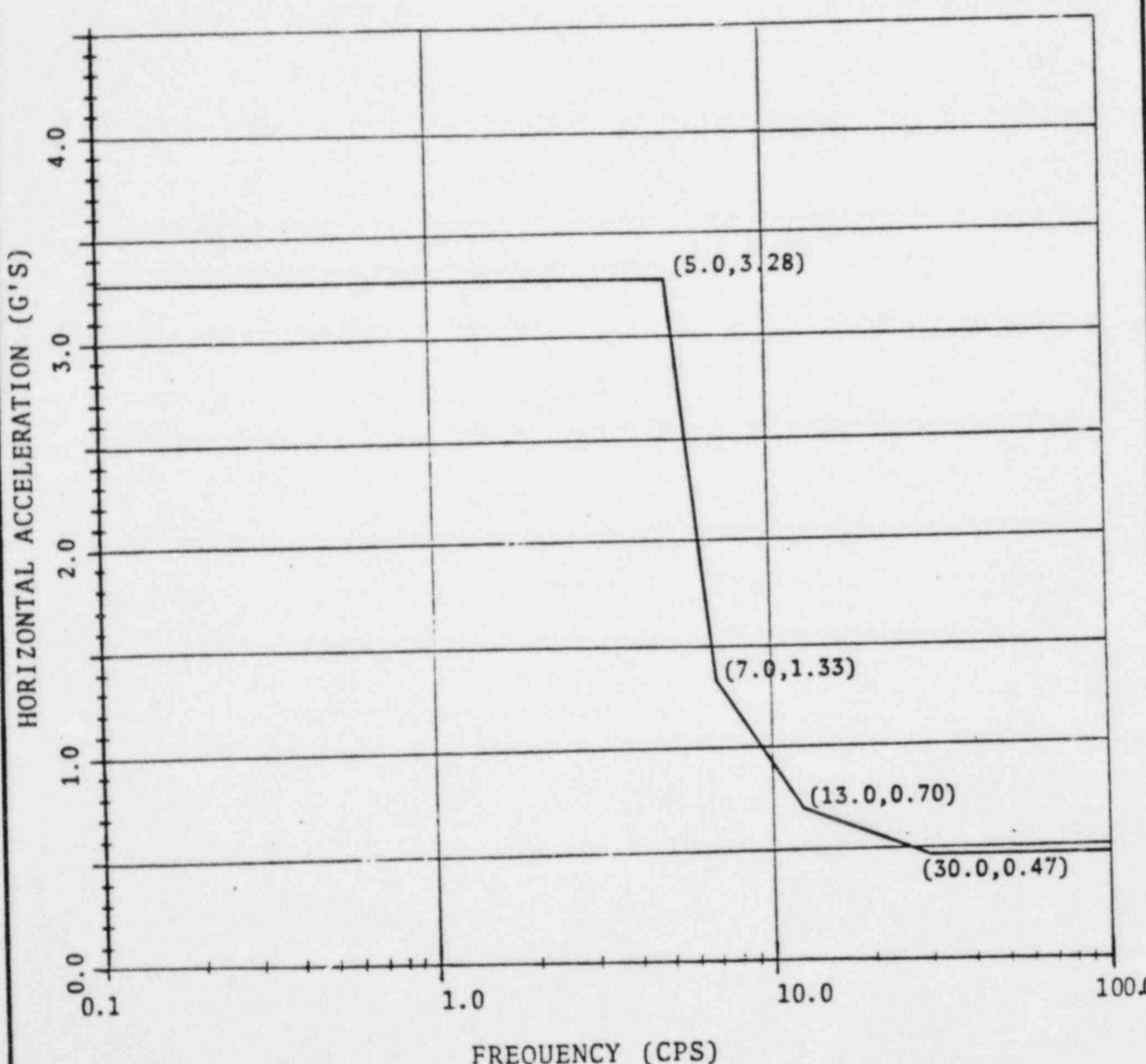
ELEVATION: 91'-3"

Ref: 2.9

EARTHQUAKE: Maximum

DAMPING: 5%

MASS POINT: 5



EES	PROJECT MEMO #10 80034 PILGRIM NUCLEAR POWER STATION BOSTON EDISON COMPANY	Page 10 of 44

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 6A

EARTHQUAKE: Design

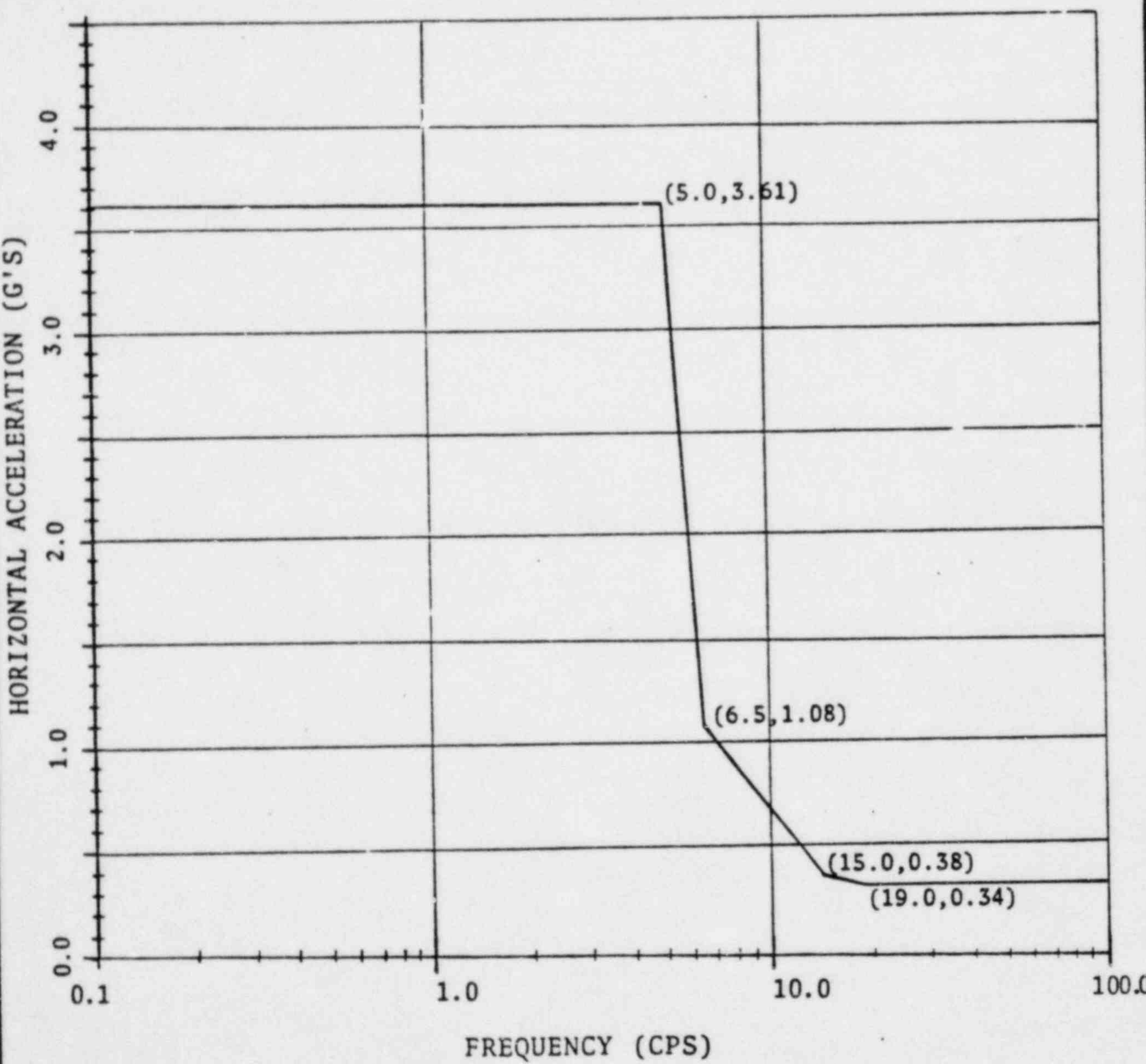
BUILDING: Reactor

DAMPING: 2%

ELEVATION: 117'-0"

MASS POINT: 6

Ref: 2.9



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 6B

EARTHQUAKE: Maximum

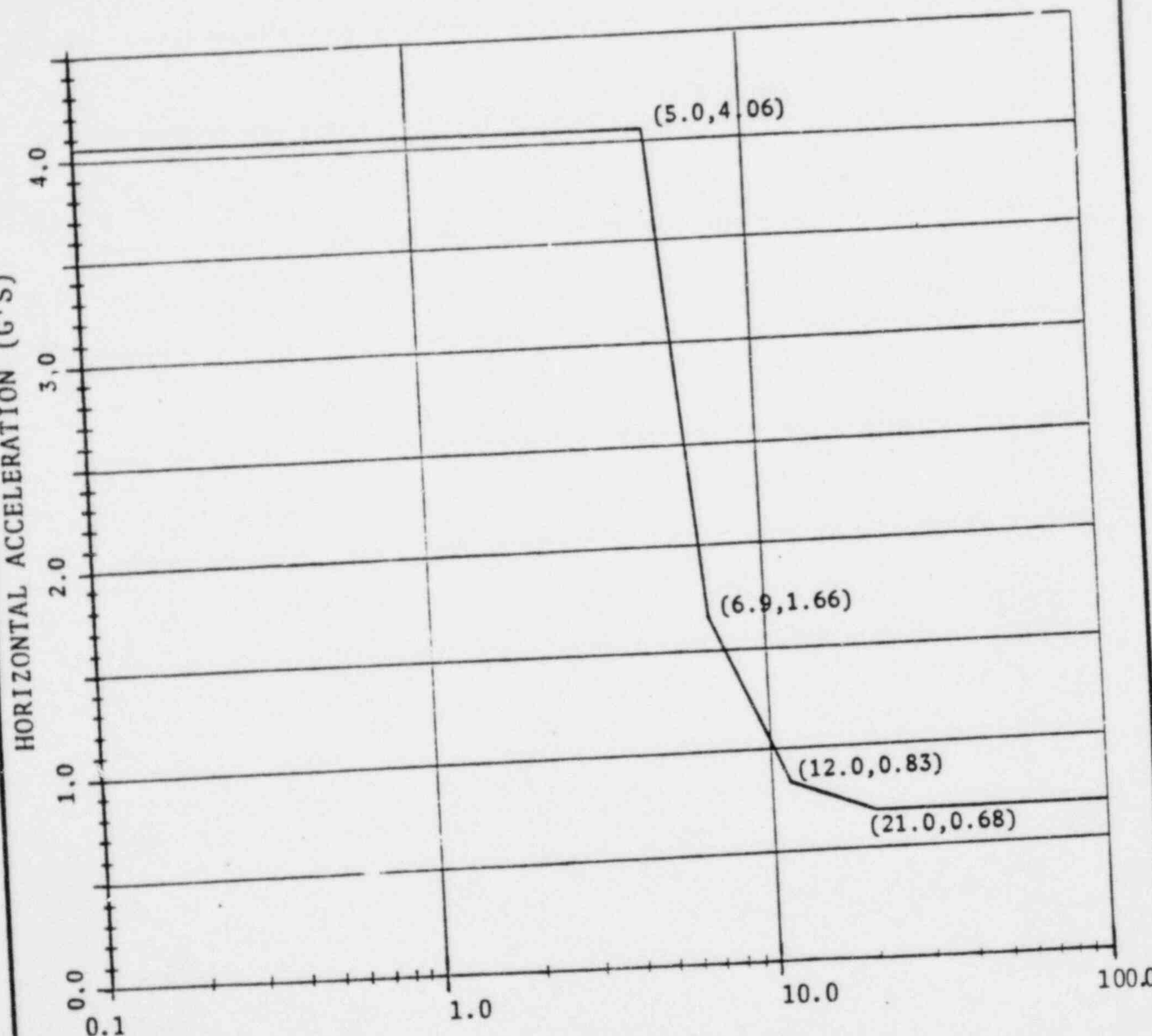
BUILDING: Reactor

DAMPING: 5%

ELEVATION: 117'-0"

MASS POINT: 6

Ref: 2.9



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PROJECT MEMO #10

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PILGRIM NUCLEAR POWER STATION

BOSTON EDISON COMPANY

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 7A

EARTHQUAKE: Design

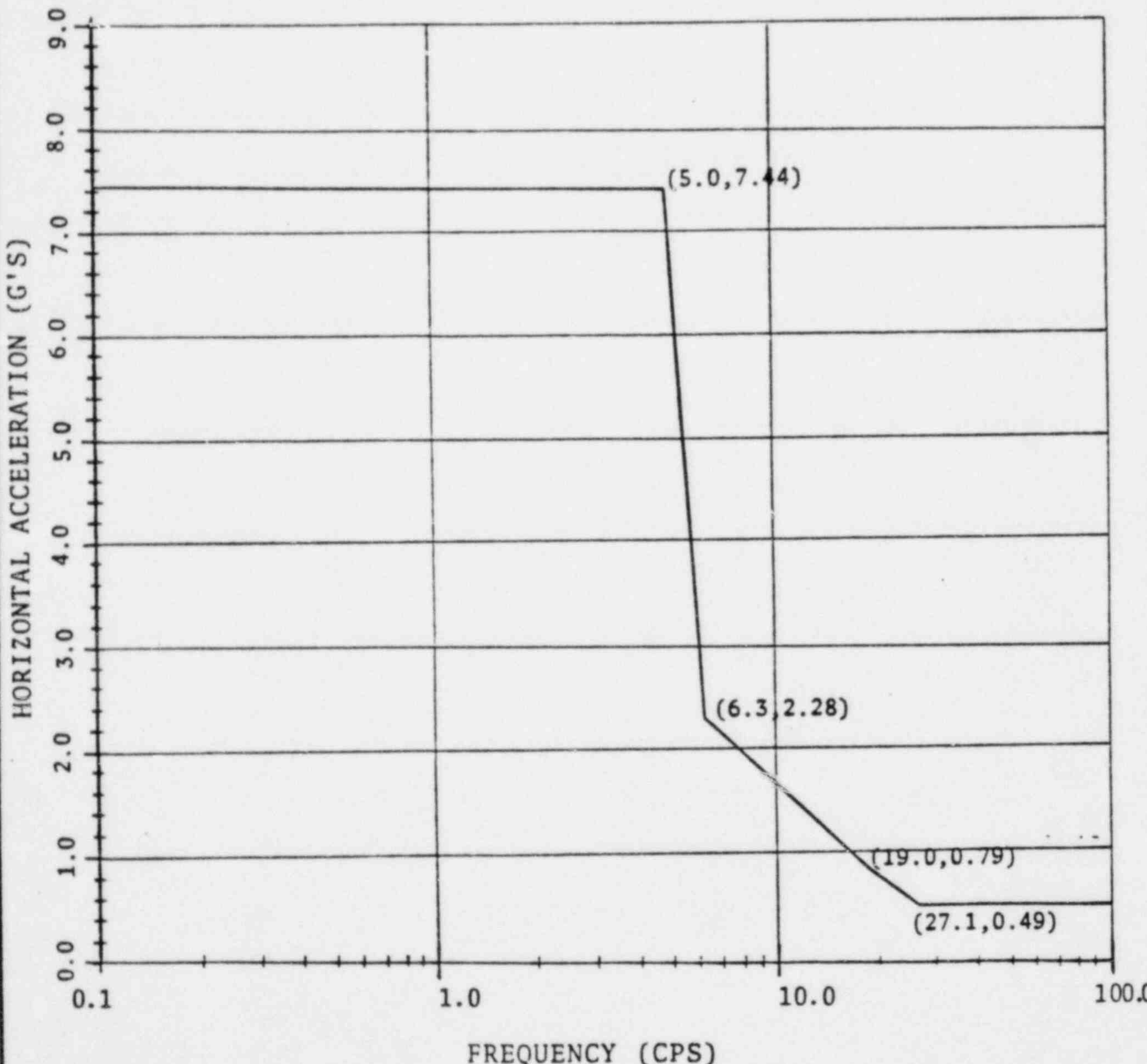
BUILDING: Reactor

DAMPING: 2%

ELEVATION: 138'-0"

MASS POINT: 7

Ref: 2.9



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BOSTON EDISON COMPANY

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 7B

EARTHQUAKE: Maximum

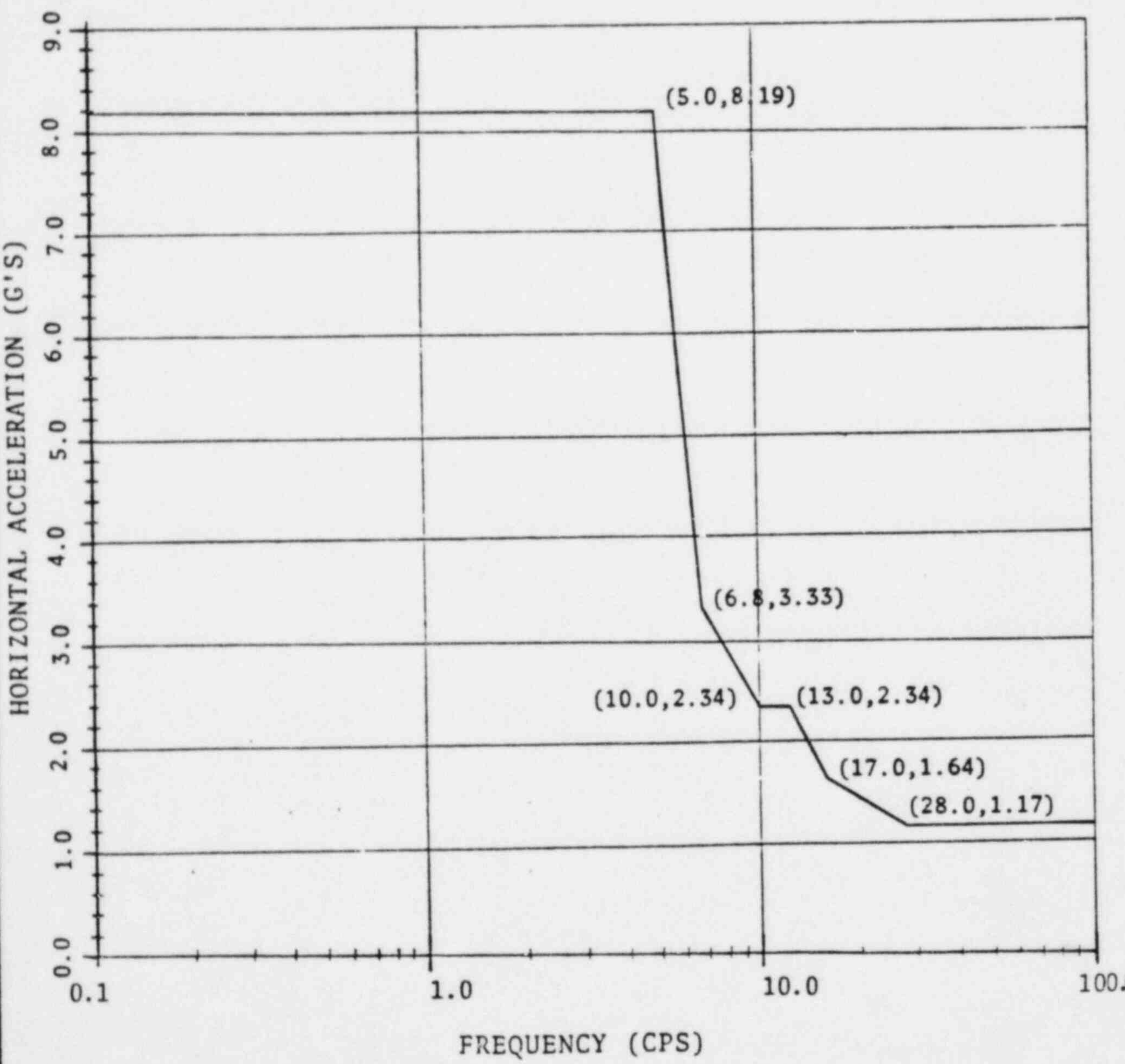
BUILDING: Reactor

DAMPING: 5%

ELEVATION: 138'-0"

MASS POINT: 7

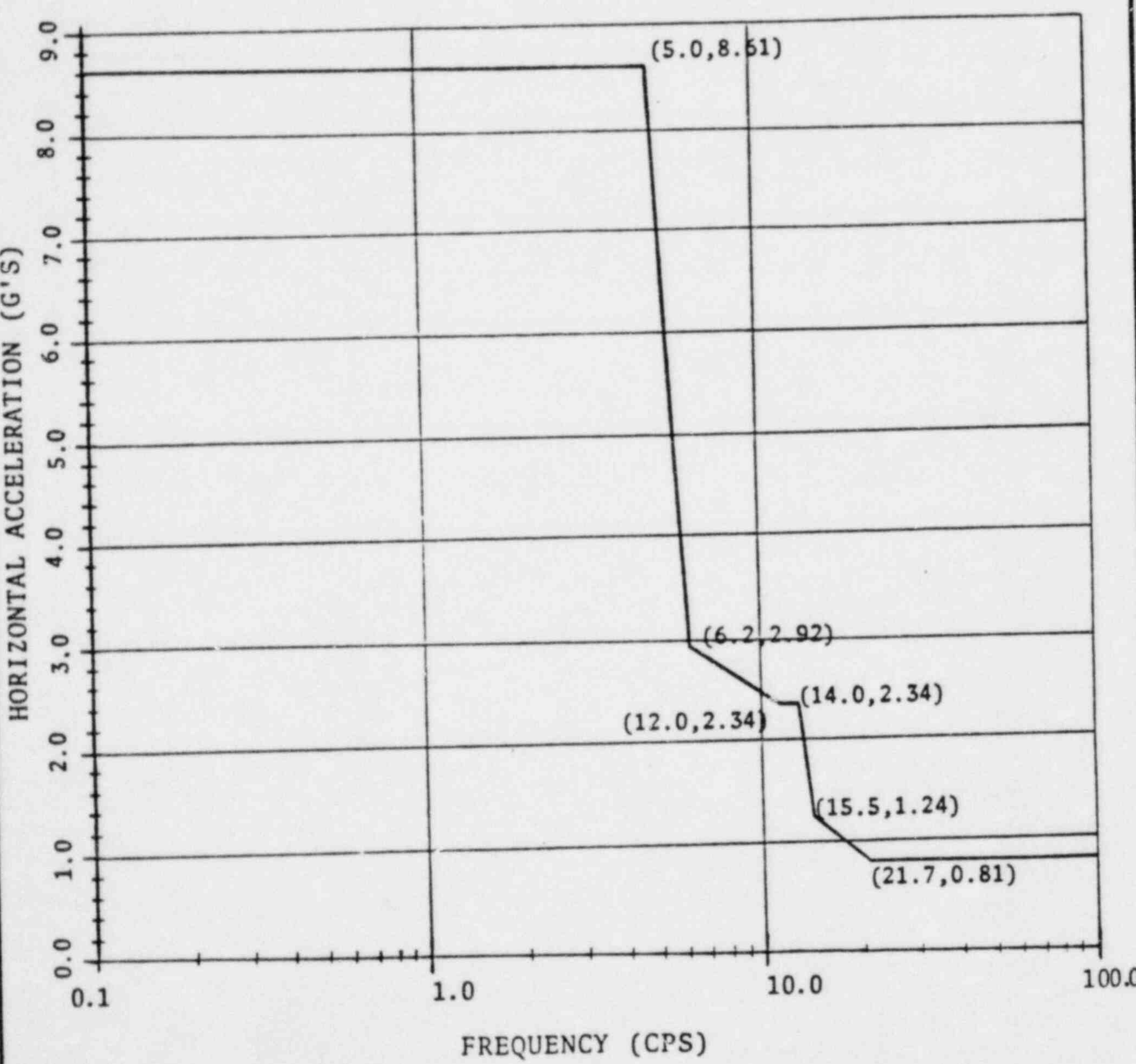
Ref: 2.9



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 8A	EARTHQUAKE: Design
BUILDING: Reactor	DAMPING: 2%
ELEVATION: 164'-6"	MASS POINT: 8

Ref: 2.9



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: R - 1 - 8B

EARTHQUAKE: Maximum

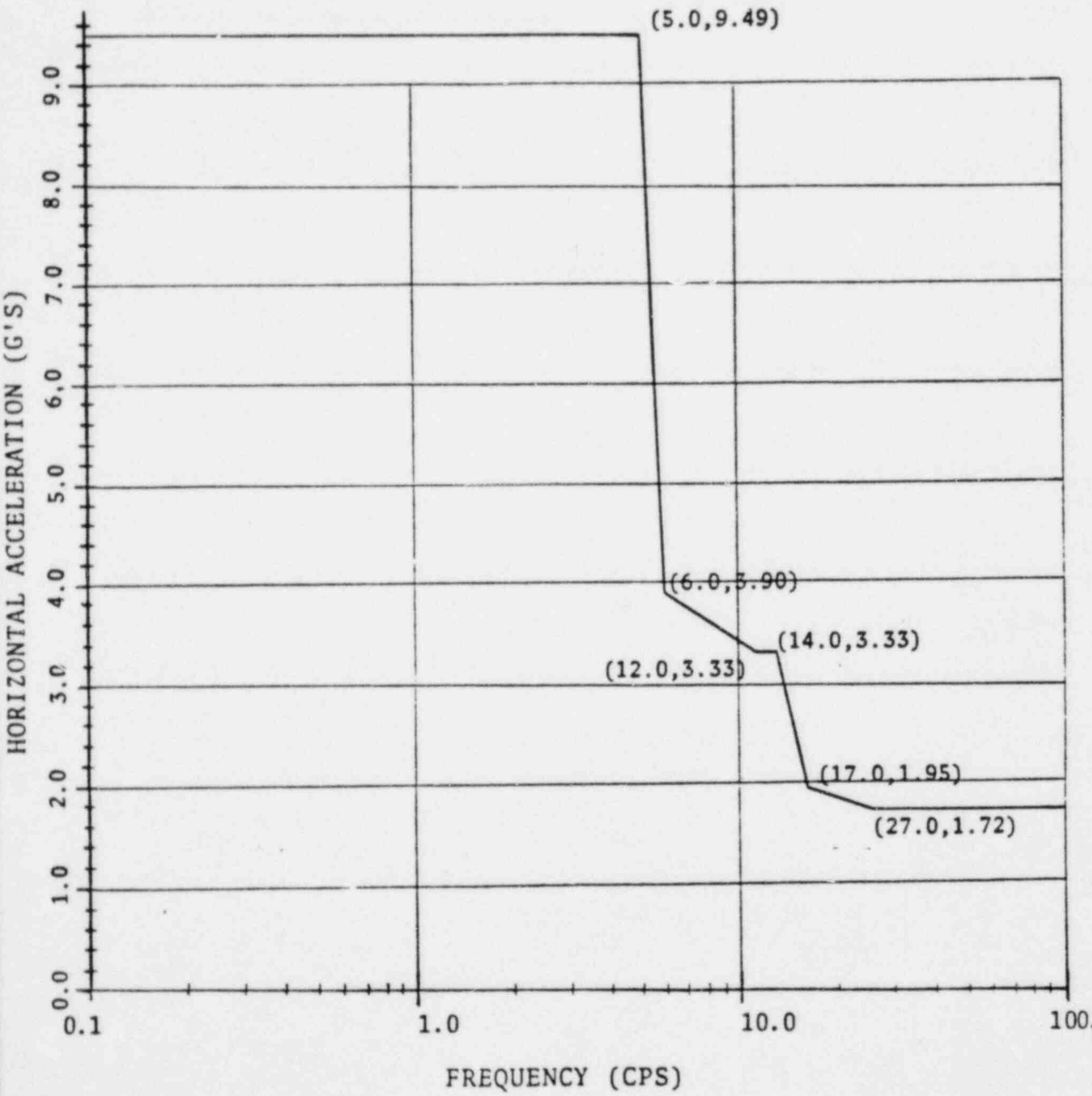
BUILDING: Reactor

DAMPING: 5%

ELEVATION: 164'-6"

MASS POINT: 8

Ref: 2.9



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: T - 1 - 2A

EARTHQUAKE: Design

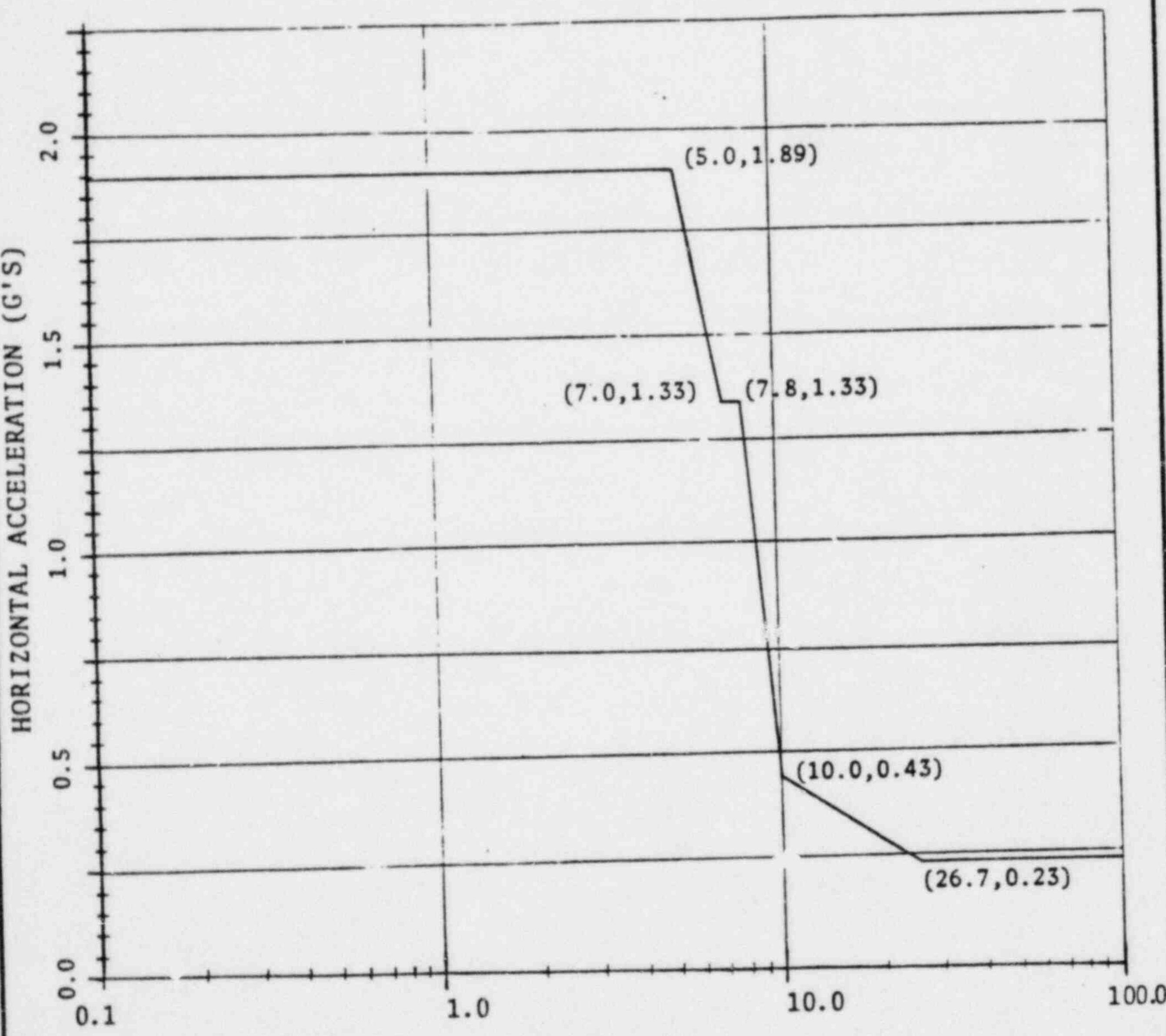
BUILDING: Turbine

DAMPING: 2%

ELEVATION: 23'-0"

MASS POINT: 2

Ref: 2.10



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BOSTON EDISON COMPANY

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: T - 1 - 2B

EARTHQUAKE: Maximum

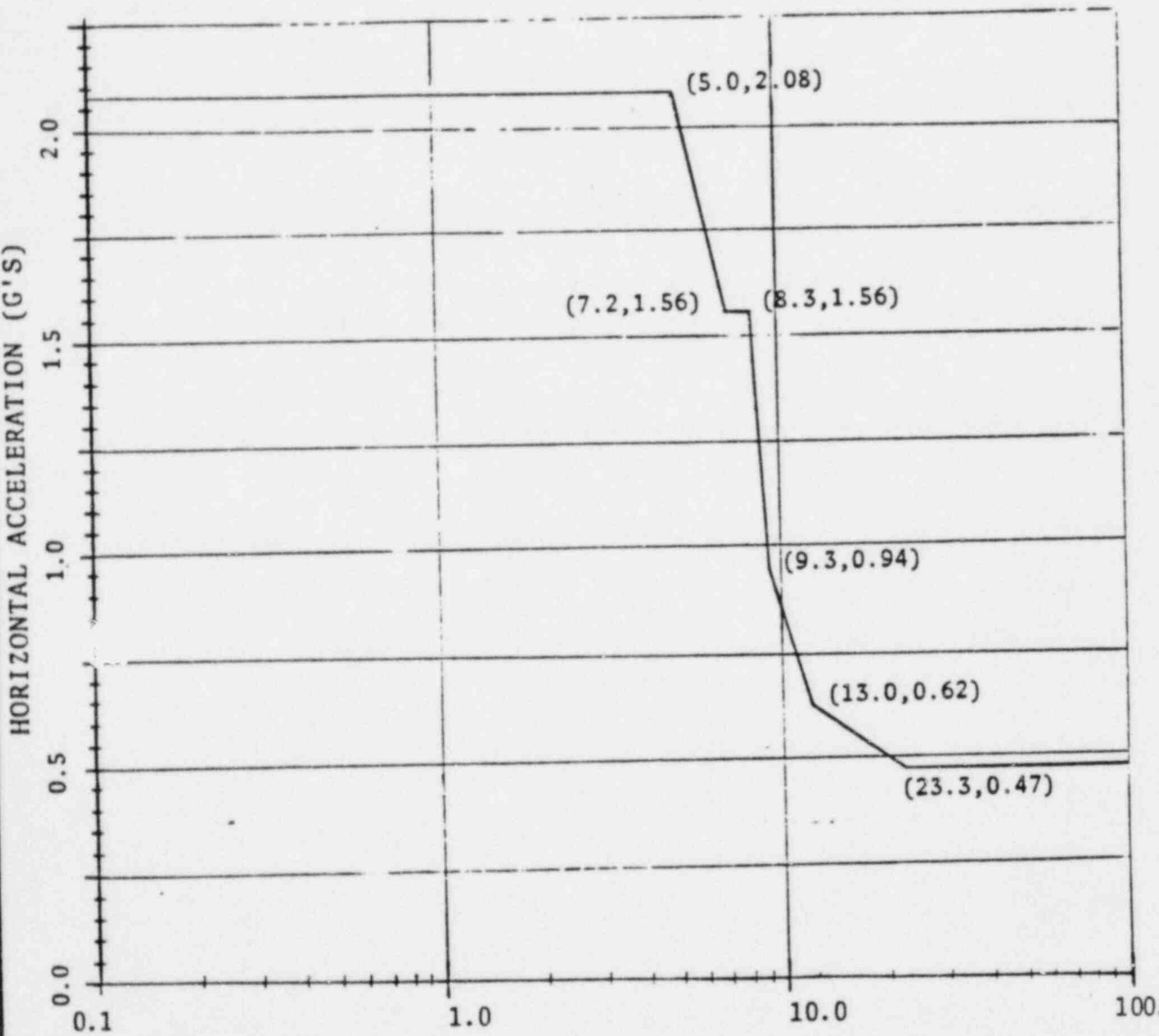
BUILDING: Turbine

DAMPING: 5%

ELEVATION: 23'-0"

MASS POINT: 2

Ref: 2.10



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PROJECT MEMO #10
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BOSTON EDISON COMPANY

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: T - 1 - 3A

EARTHQUAKE: Design

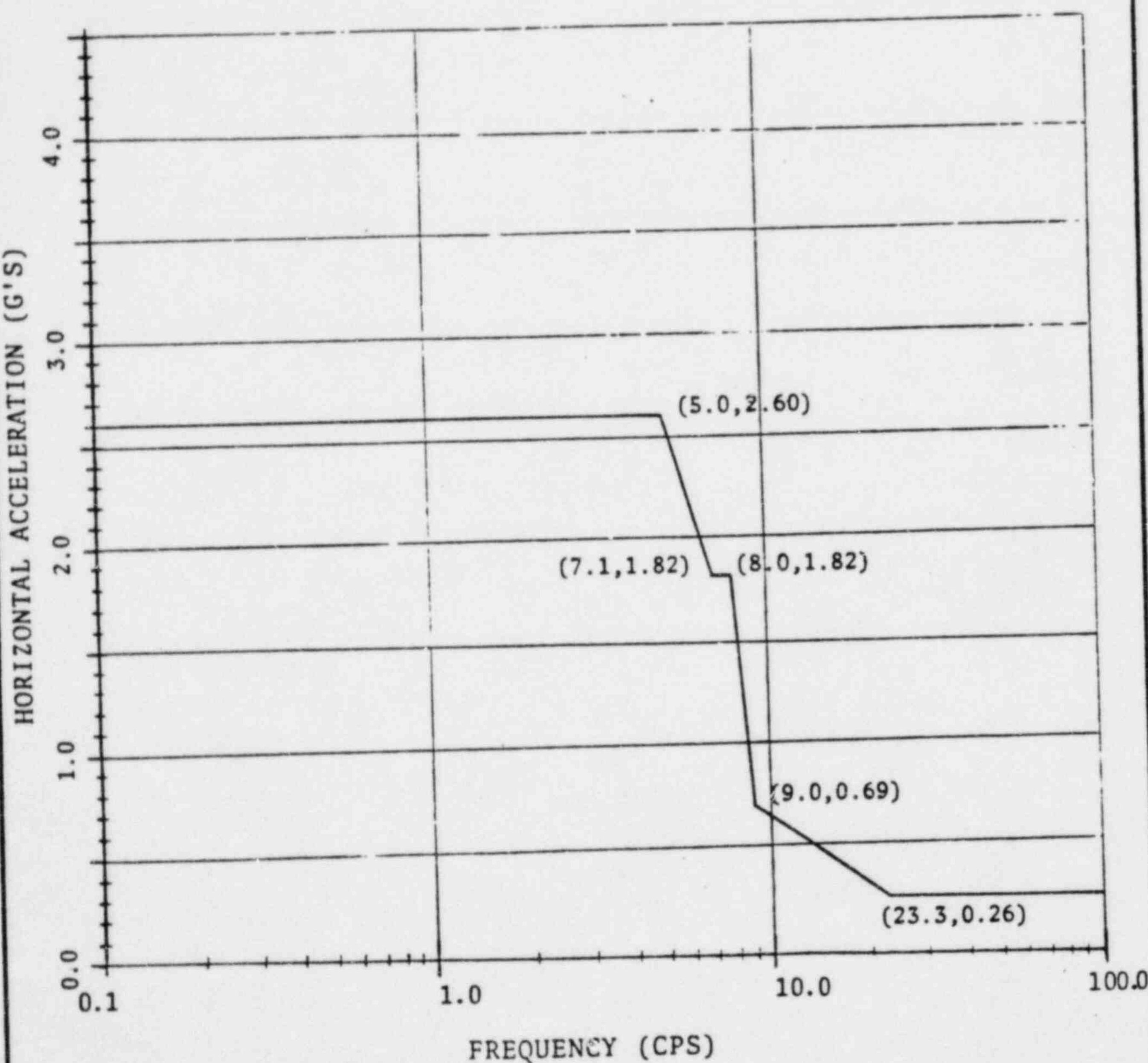
BUILDING: Turbine

DAMPING: 2%

ELEVATION: 37'-0"

MASS POINT: 3

Ref: 2.10



EES	PROJECT MEMO #10 80034 PILGRIM NUCLEAR POWER STATION BOSTON EDISON COMPANY	Page 19 of 44

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: T - 1 - 3B

EARTHQUAKE: Maximum

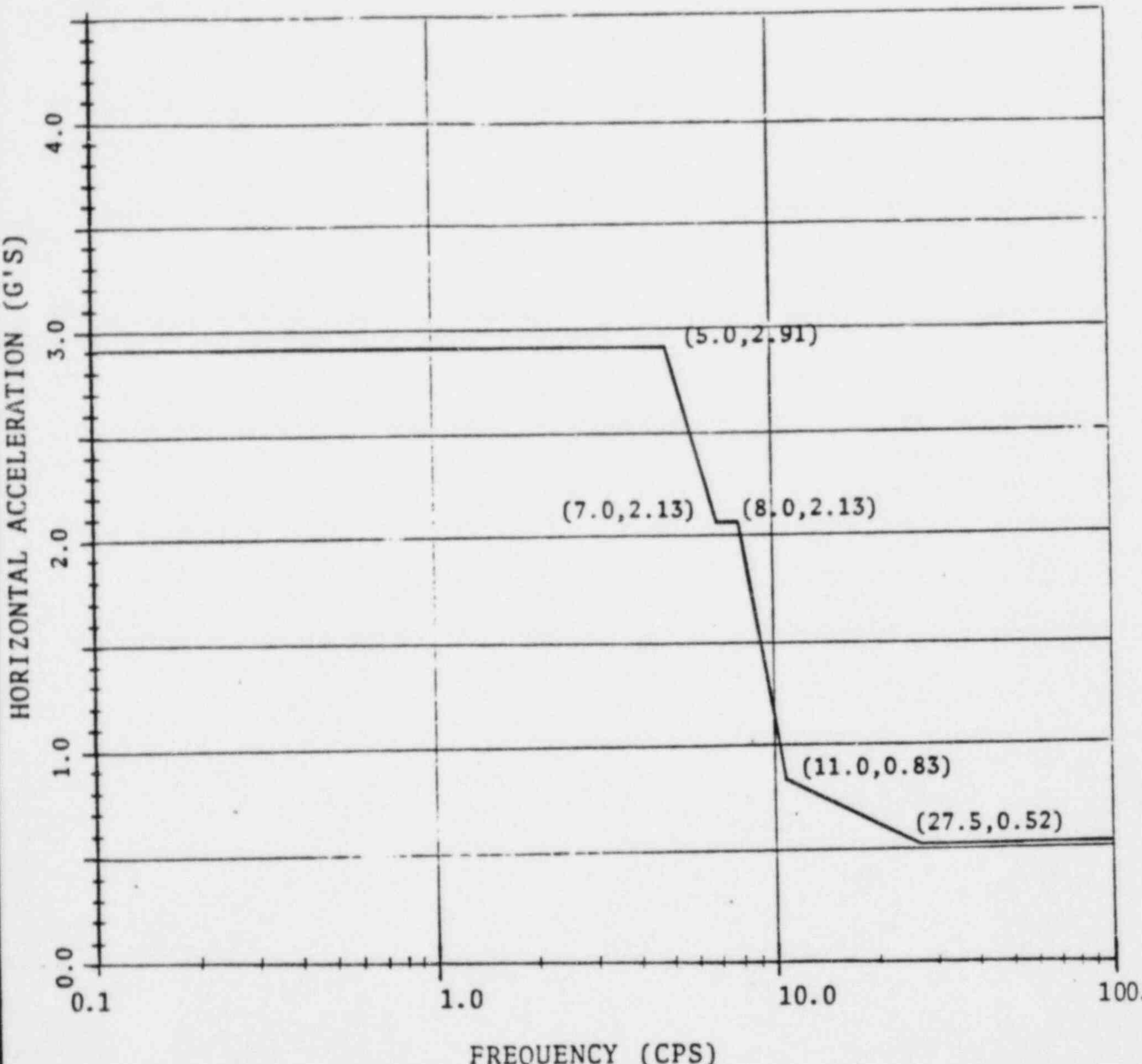
BUILDING: Turbine

DAMPING: 5%

ELEVATION: 37'-0"

MASS POINT: 3

Ref: 2.10



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: T - 1 - 4A

EARTHQUAKE: Design

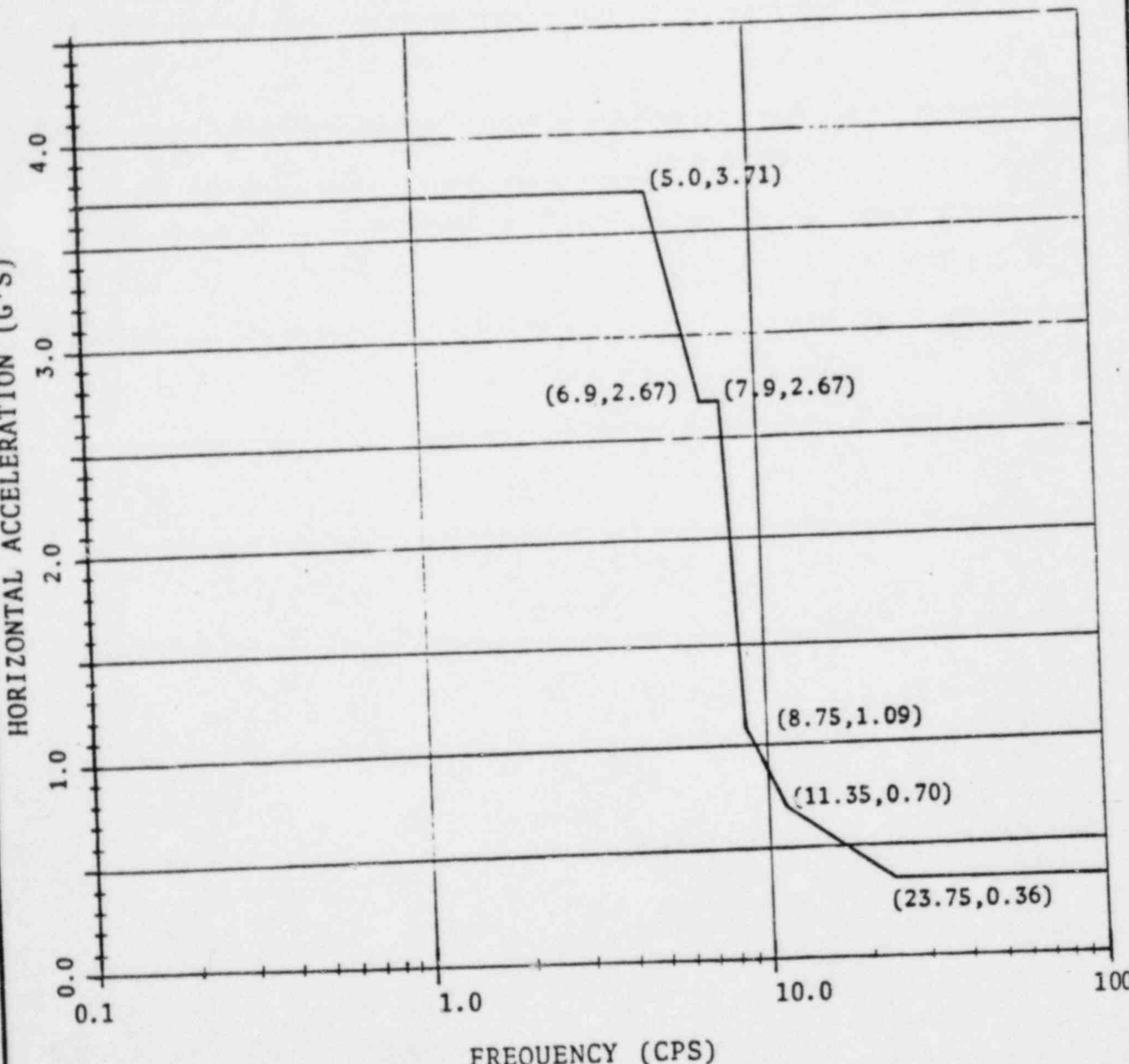
BUILDING: Turbine

DAMPING: 2%

ELEVATION: 51'-0"

MASS POINT: 4

Ref: 2.10



EES	PROJECT MEMO #10 80034 PILGRIM NUCLEAR POWER STATION BOSTON EDISON COMPANY	Page 21 of 44

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: T - 1 - 4B

EARTHQUAKE: Maximum

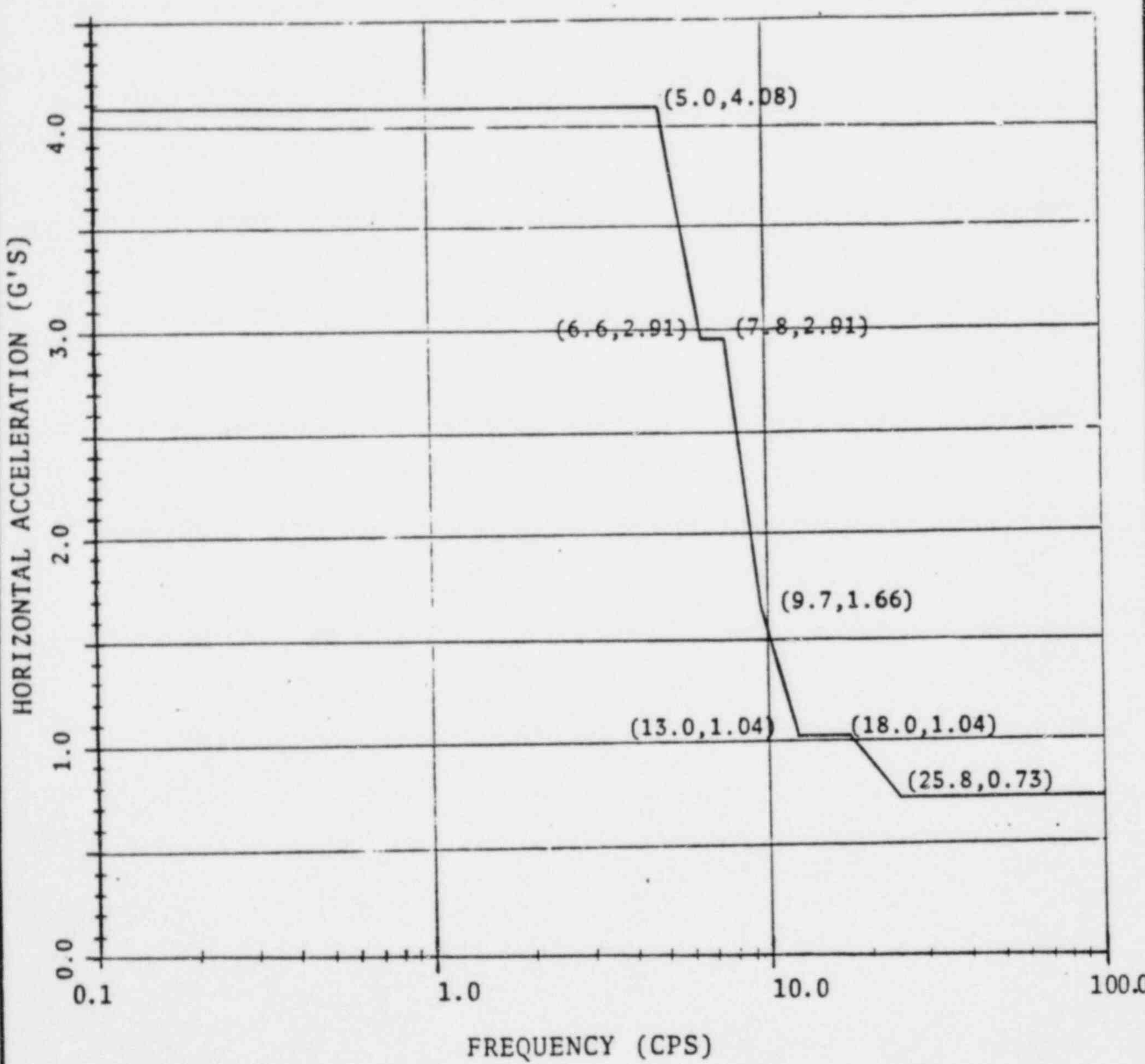
BUILDING: Turbine

DAMPING: 5%

ELEVATION: 51'-0"

MASS POINT: 4

Ref: 2.10



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PROJECT MEMO #10
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PILGRIM NUCLEAR POWER STATION
BOSTON EDISON COMPANY

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: T - 1 - 7A

EARTHQUAKE: Design

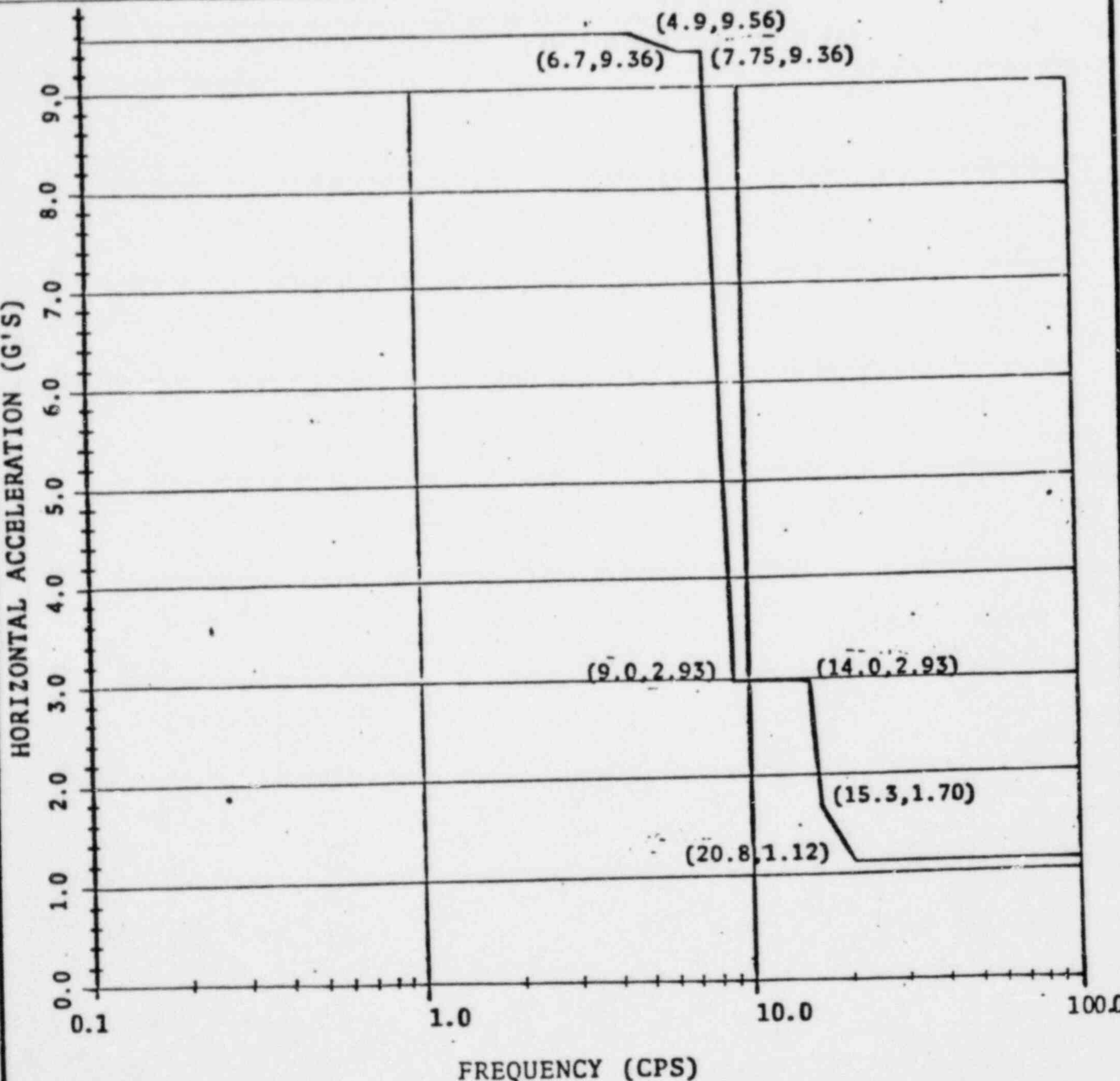
BUILDING: Turbine

DAMPING: 2%

ELEVATION: 105'-6"

MASS POINT: 7

Ref: 2.10



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: T - 1 - 7B

EARTHQUAKE: Maximum

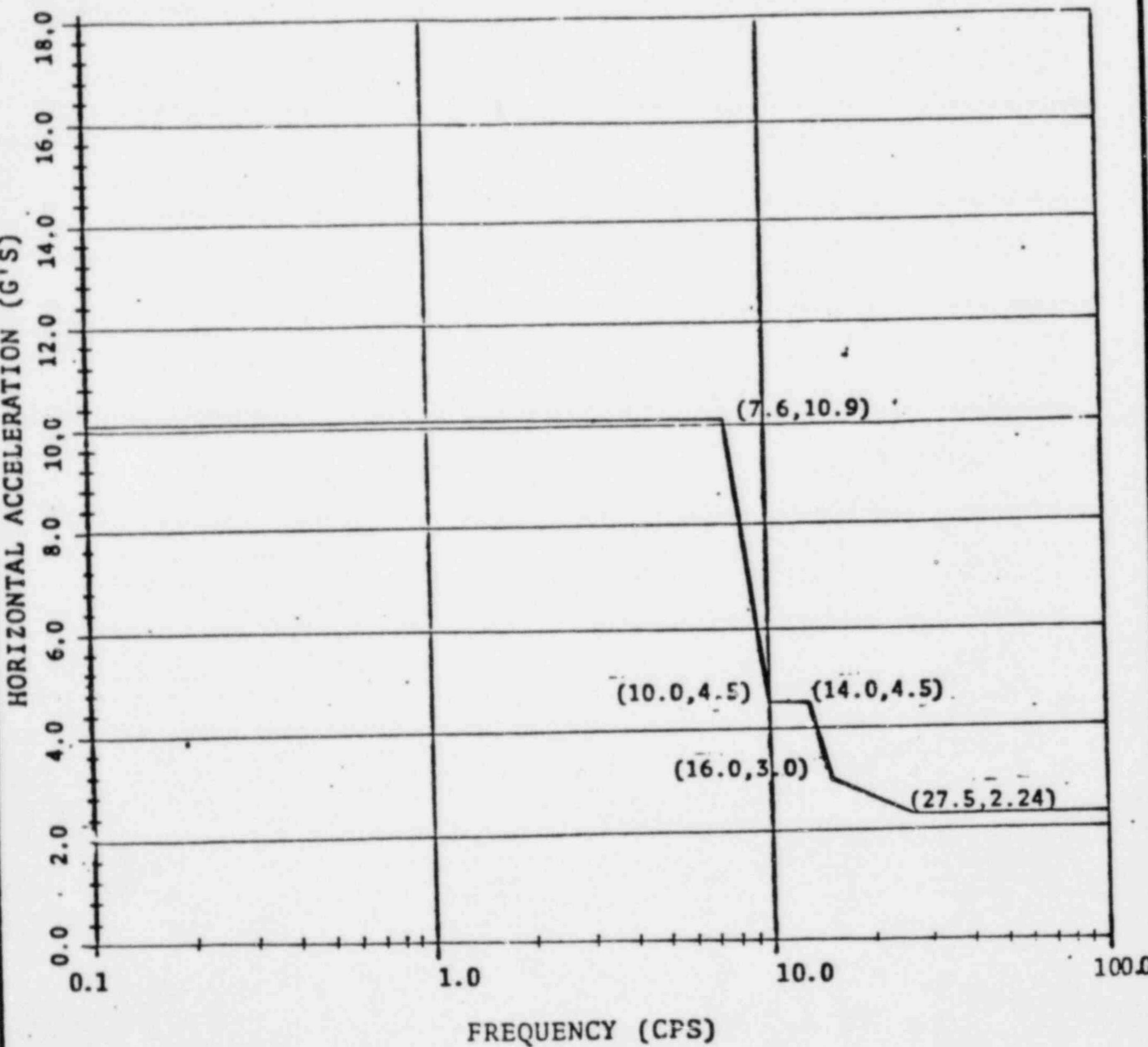
BUILDING: Turbine

DAMPING: 5%

ELEVATION: 105'-6"

MASS POINT: 7

Ref: 2.10



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PROJECT MEMO #10

80034

PILGRIM NUCLEAR POWER STATION

BOSTON EDISON COMPANY

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: RW - 1 - 3A

EARTHQUAKE: Design

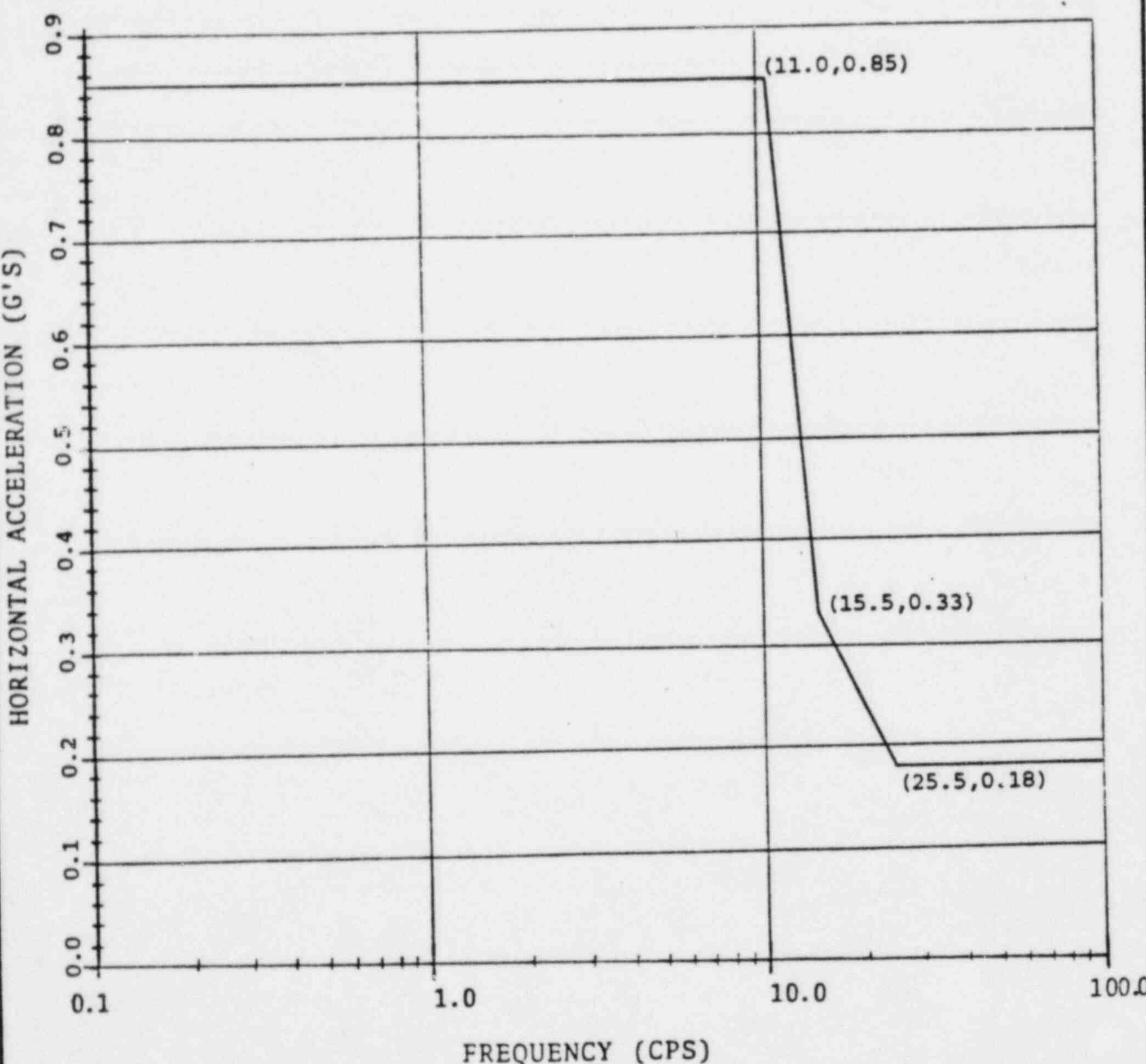
BUILDING: Radwaste

DAMPING: 2%

ELEVATION: 23'-0"

MASS POINT: 3

Ref: 2.11



EES	PROJECT MEMO #10 80034 PILGRIM NUCLEAR POWER STATION BOSTON EDISON COMPANY	Page 25 of 44

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: RW - 1 - 3B

EARTHQUAKE: Maximum

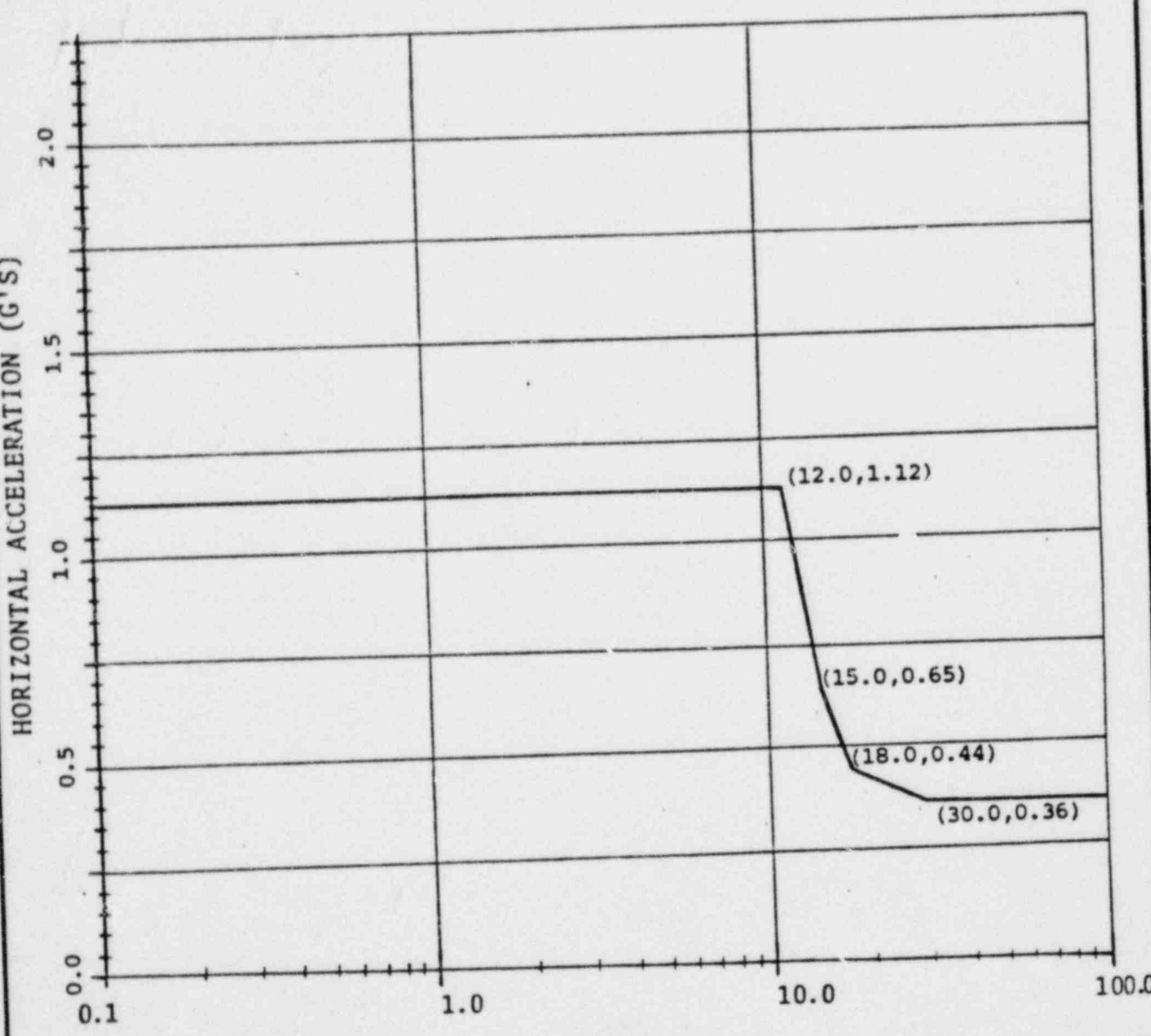
BUILDING: Radwaste

DAMPING: 5%

ELEVATION: 23'-0"

MASS POINT: 3

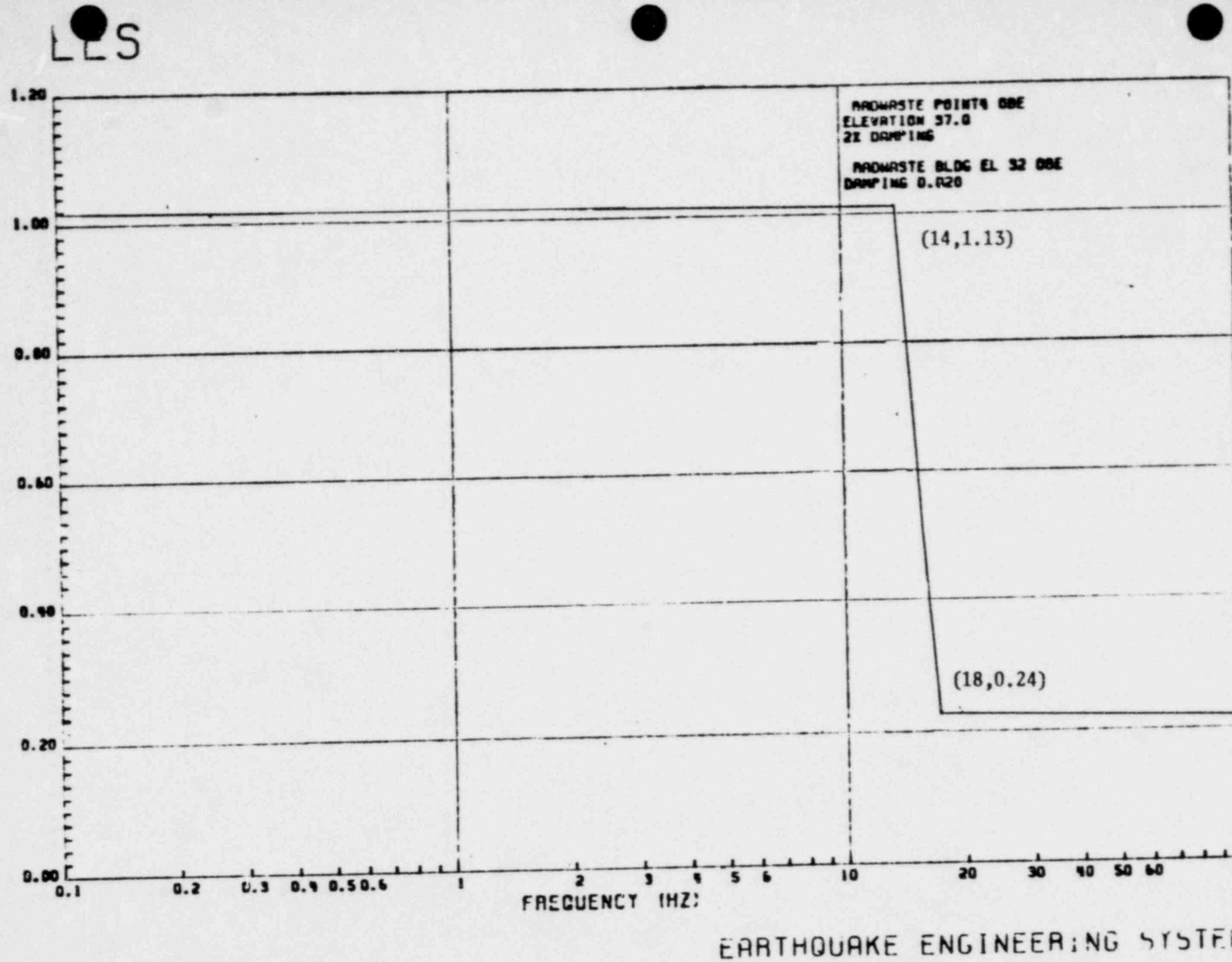
Ref: 2.11



EES	PROJECT MEMO #10 80034 PILGRIM NUCLEAR POWER STATION BOSTON EDISON COMPANY	Page 26 of 44

Attachment 1

Response Spectra for Level 1 Masonry Wall Seismic Analysis



Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: RW - 1 - 4B

EARTHQUAKE: Maximum

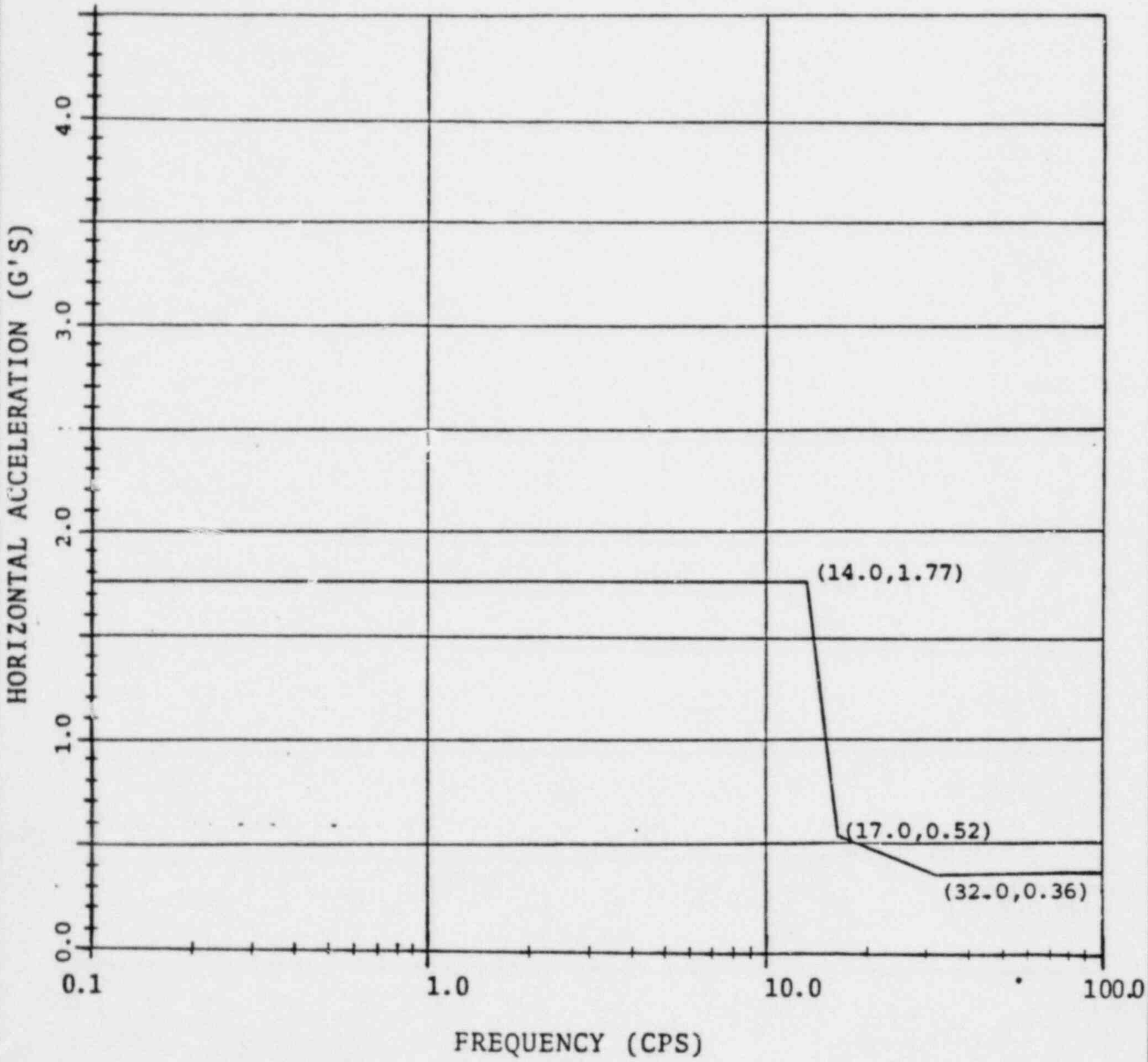
BUILDING: Radwaste

DAMPING: 5%

ELEVATION: 37'-0"

MASS POINT: 4

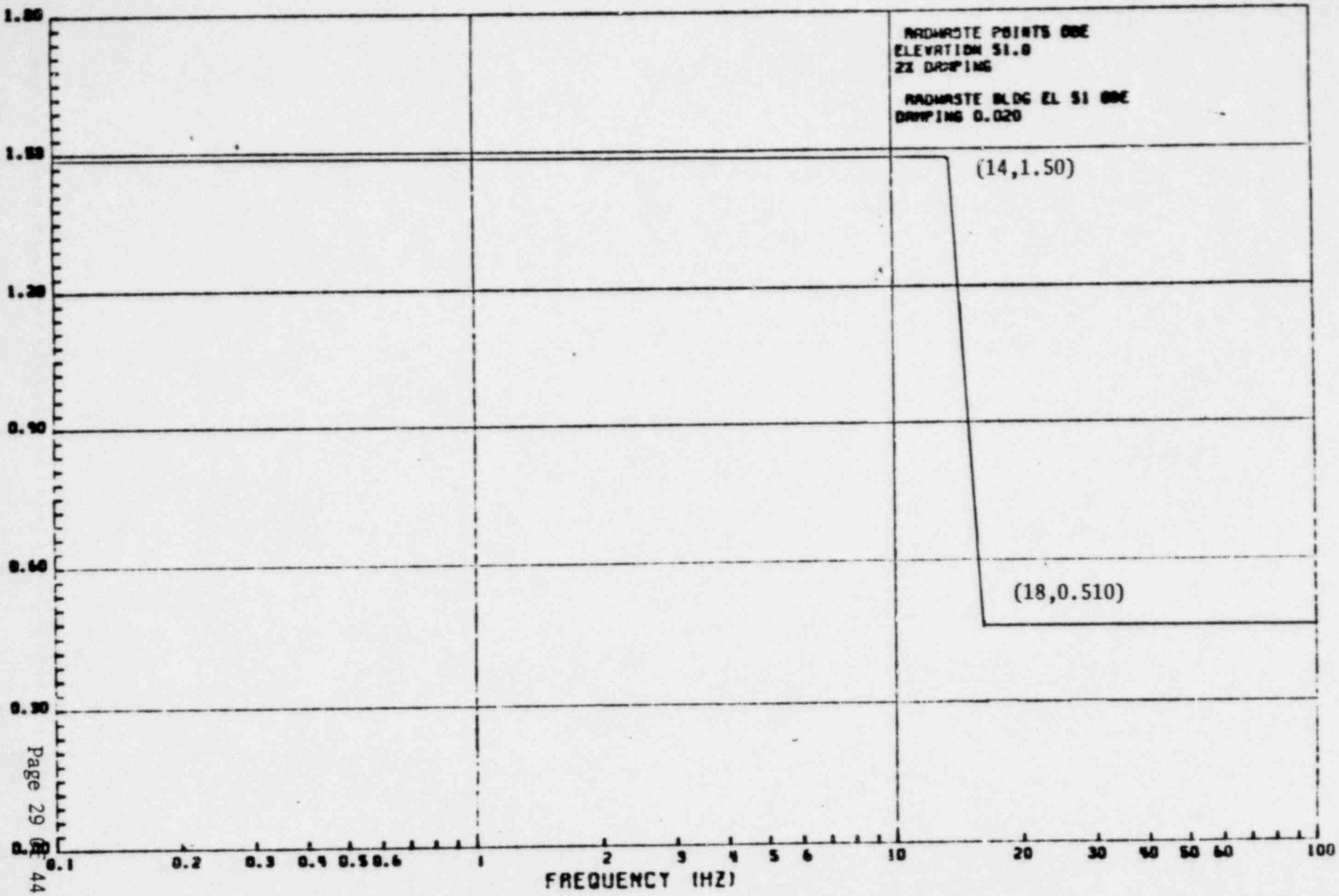
Ref: 2.11



EES

PROJECT MEMO #10
80034
PILGRIM NUCLEAR POWER STATION
BOSTON EDISON COMPANY

EES



EARTHQUAKE ENGINEERING SYSTEMS

Attachment 1

Response Spectra for Level 1 Masonry Wall Seismic Analysis

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: RW - 1 - 5B

EARTHQUAKE: Maximum

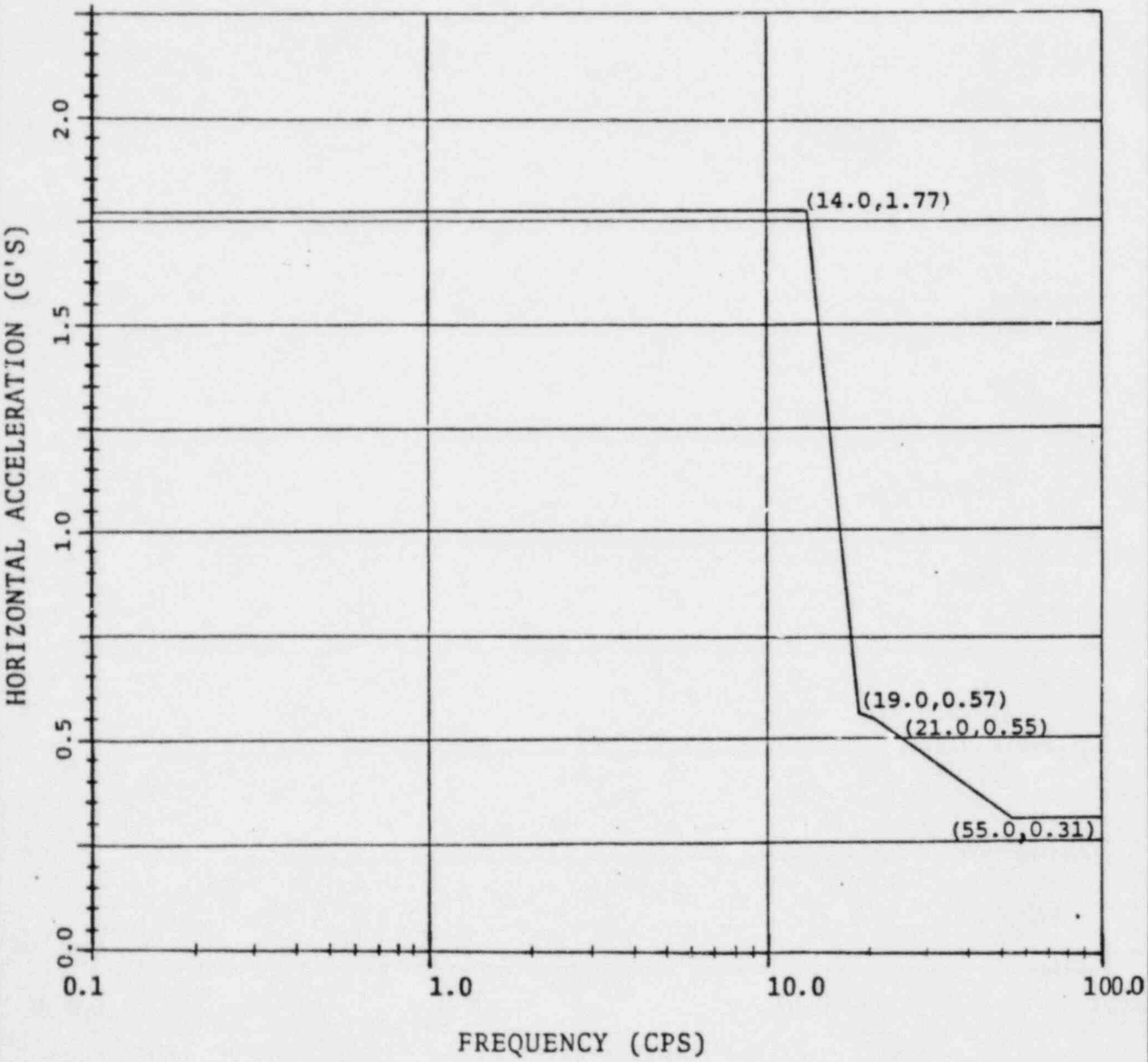
BUILDING: Radwaste

DAMPING: 5%

ELEVATION: 51'-0"

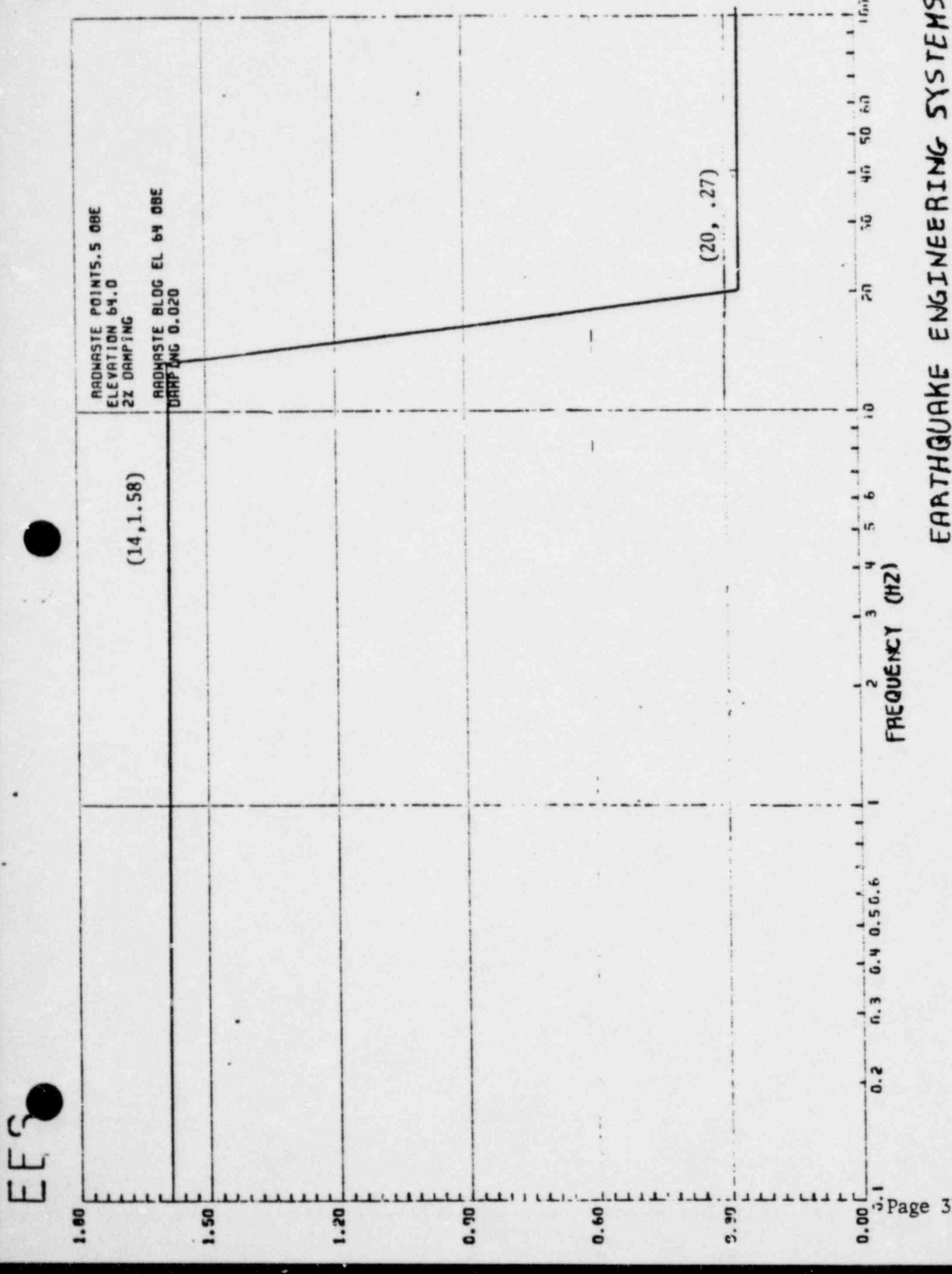
MASS POINT: 5

Ref: 2.11



Attachment 1

Response Spectra for Level 1 Masonry Wall Seismic Analysis



Attachment 1: Response Spectra for Level 2 Masonry Wall Seismic Analysis

SPECTRUM NO: RW-1-5.5B

EARTHQUAKE: Maximum

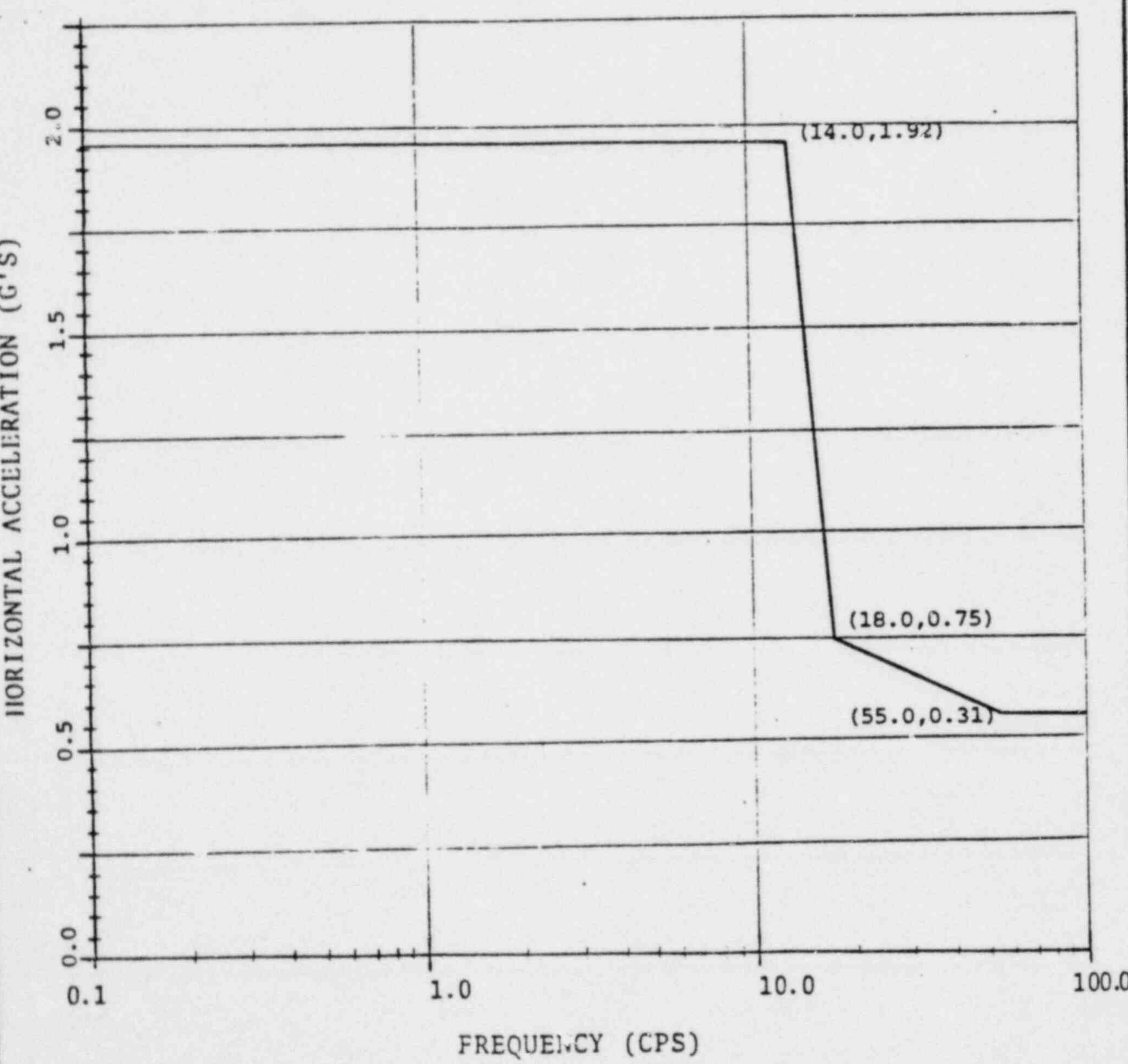
BUILDING: Radwaste

DAMPING: 5%

ELEVATION: 64'-0"

MASS POINT: 5.5

Ref: 2.11



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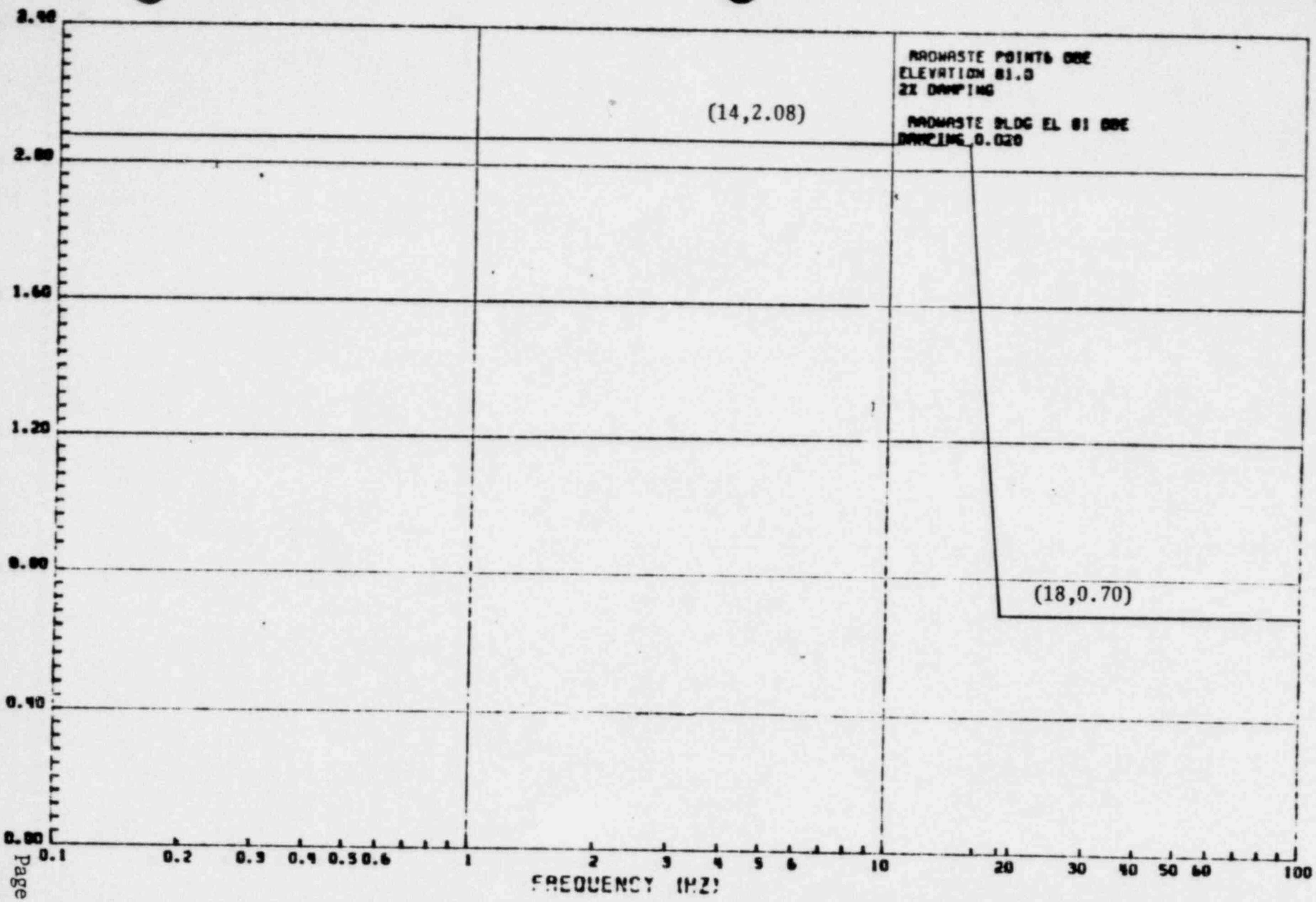


EARTHQUAKE ENGINEERING SYSTEMS

Attachment 1

Response Spectra for Level 1 Masonry Wall Seismic Analysis

Attachment 1: Response Spectra for Level 1 Masonry Wall Seismic Analysis

SPECTRUM NO: RW - 1 - 6B

EARTHQUAKE: Maximum

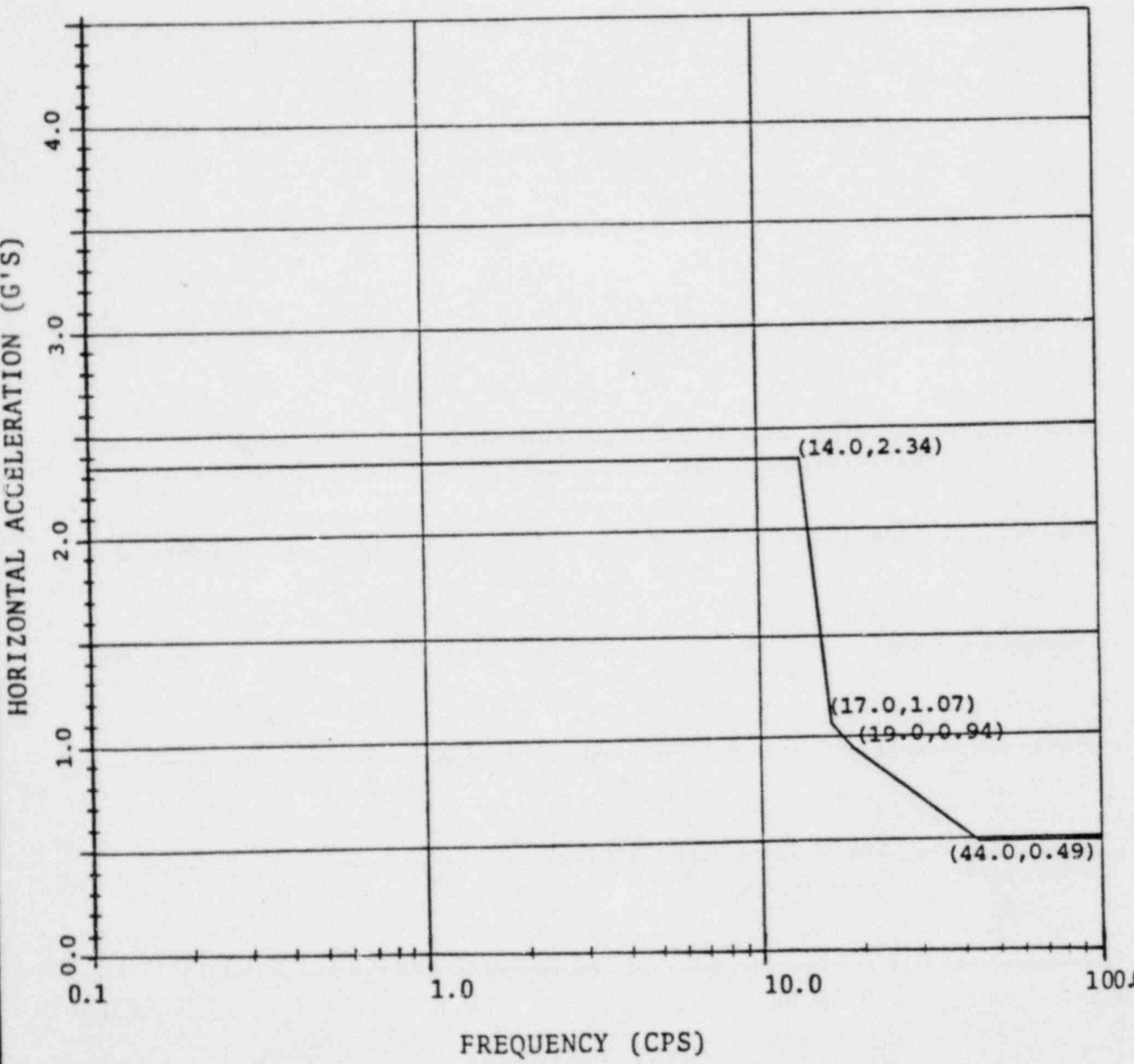
BUILDING: Radwaste

DAMPING: 5%

ELEVATION: 81'-0"

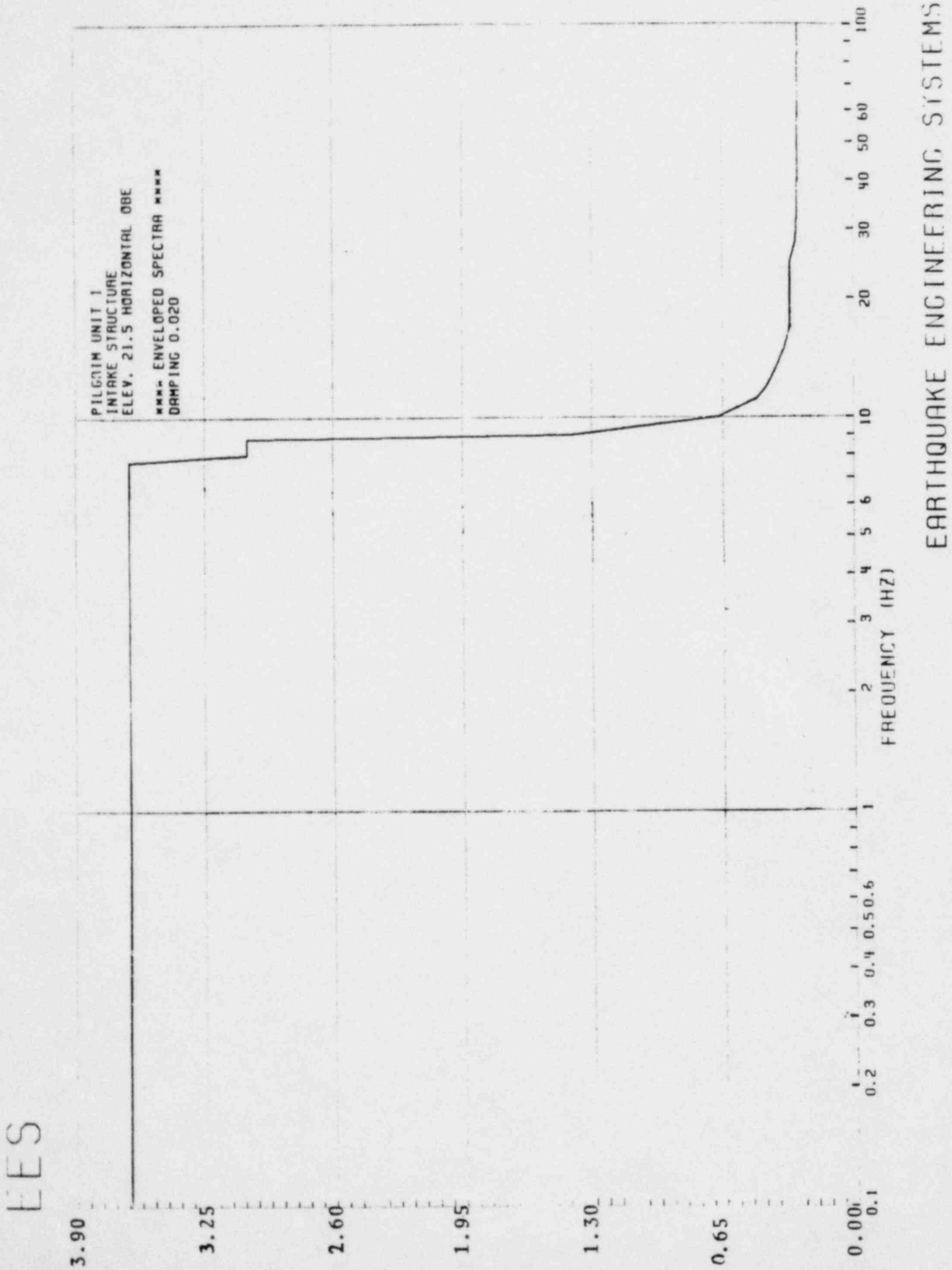
MASS POINT: 6

Ref: 2.11



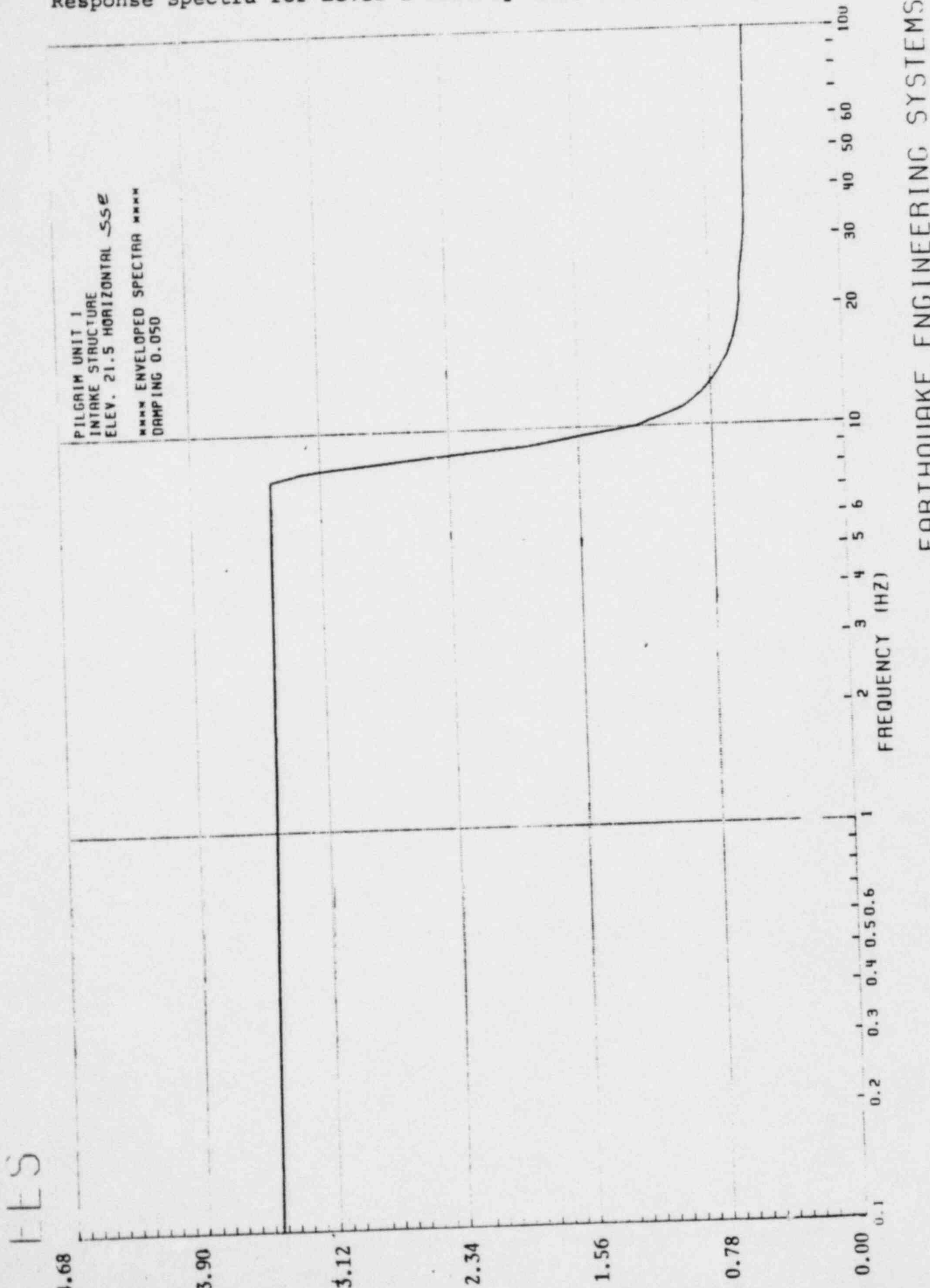
ATTACHMENT 1

Response Spectra for Level I Masonry Wall Seismic Analysis



<u>ATTACHMENT 1</u>

Response Spectra for Level I Masonry Wall Seismic Analysis



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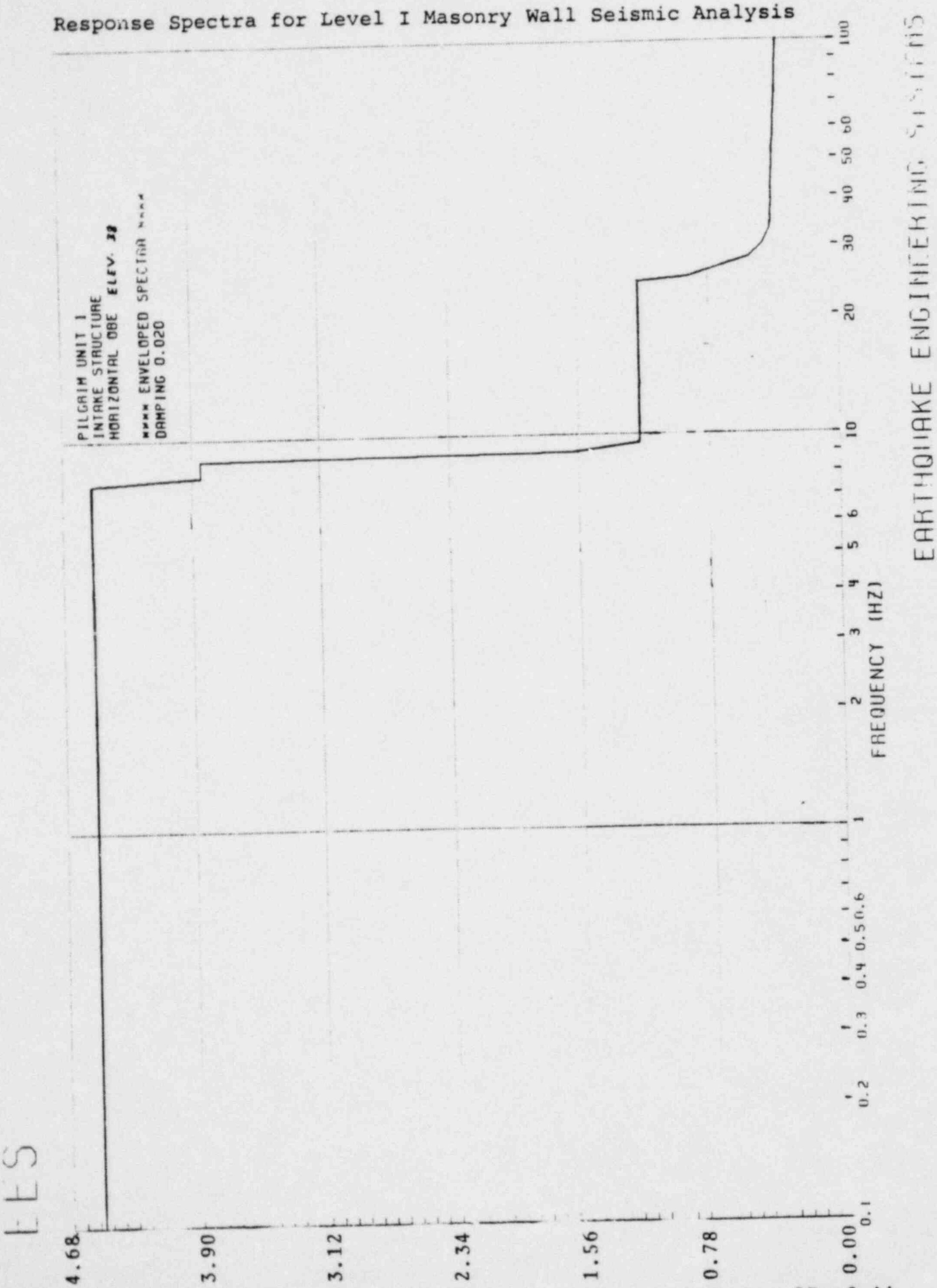
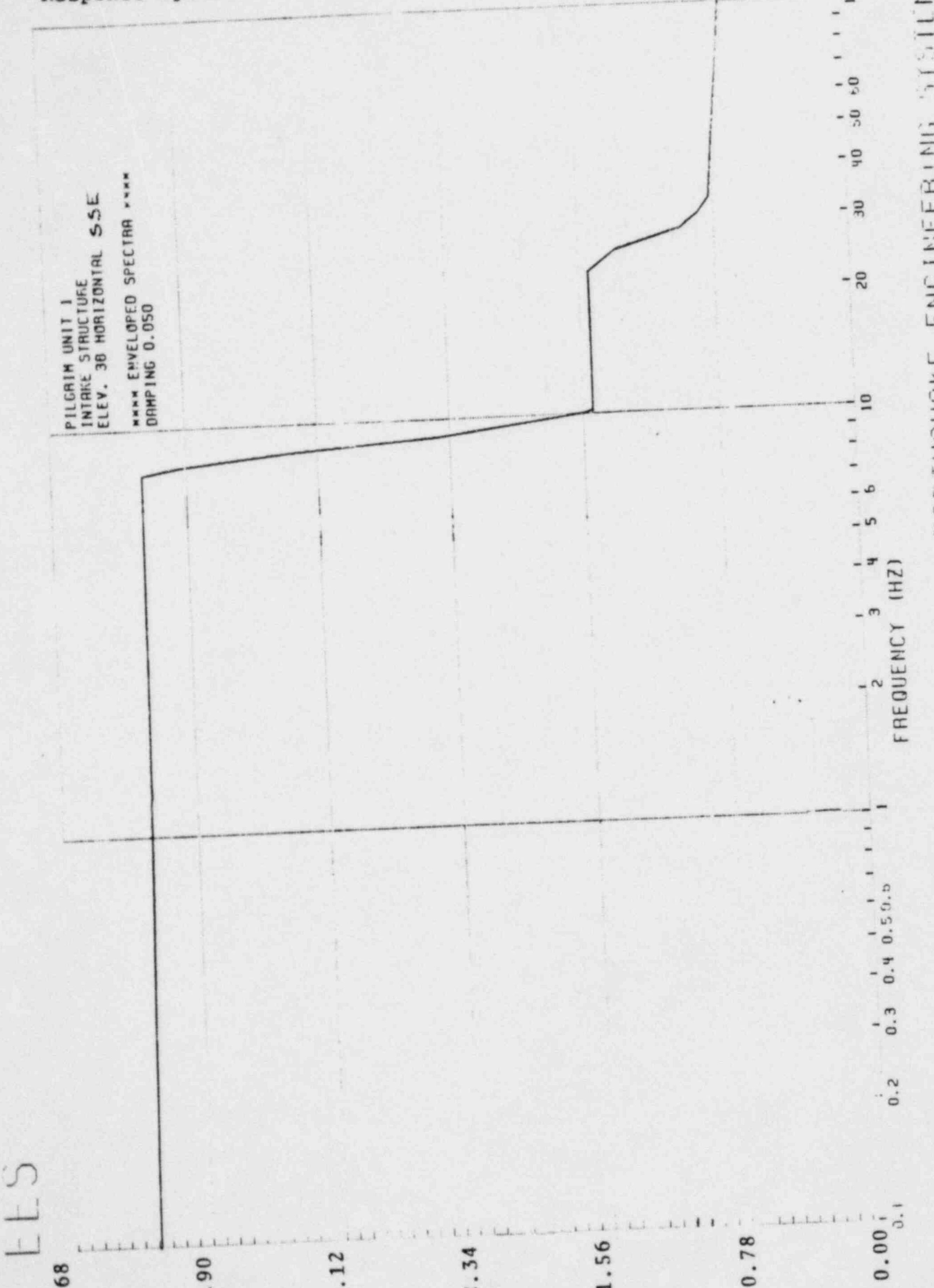
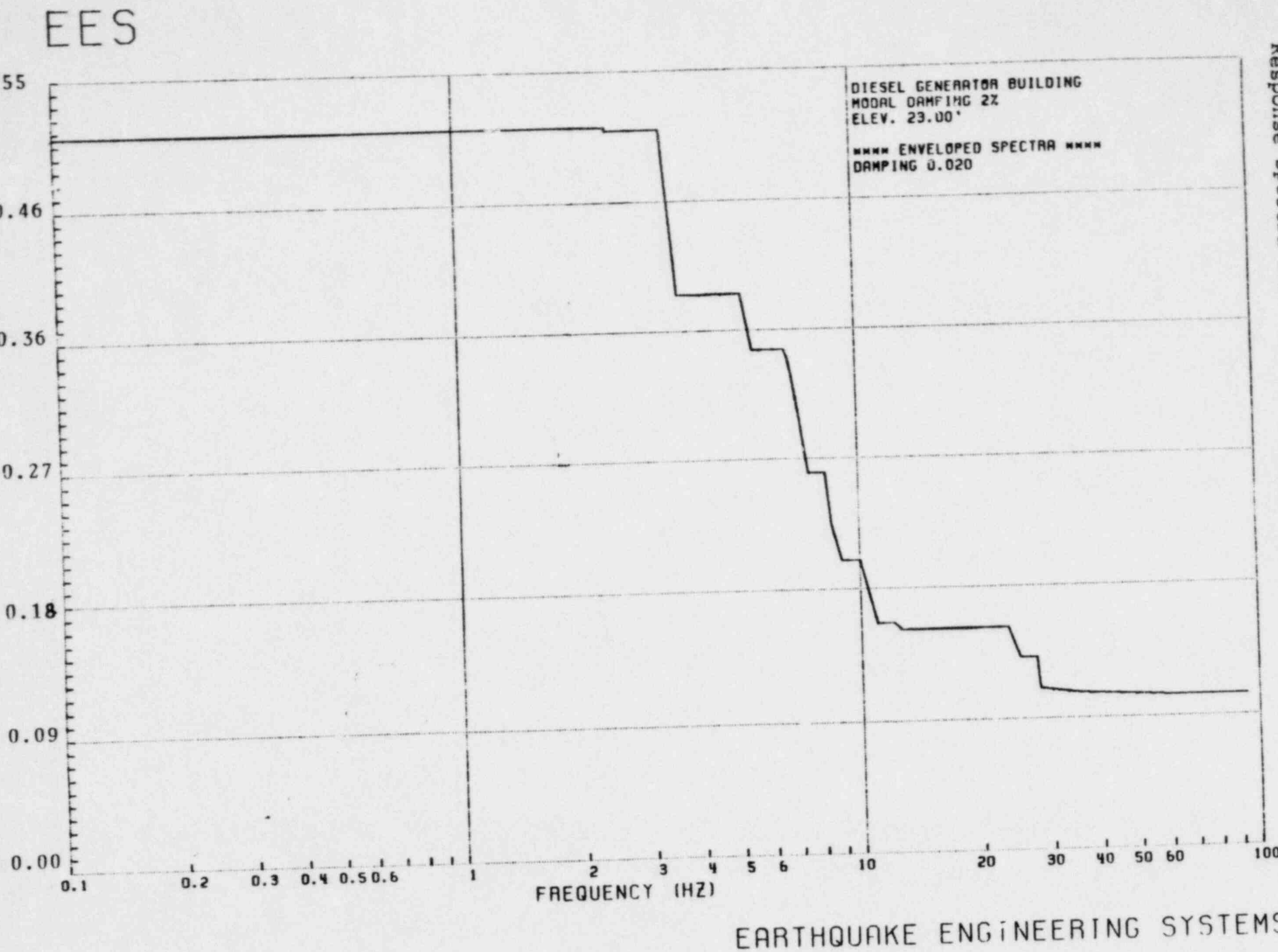
Response Spectra for Level I Masonry Wall Seismic Analysis



<u>ATTACHMENT 1</u>

Response Spectra for Level I Masonry Wall Seismic Analysis

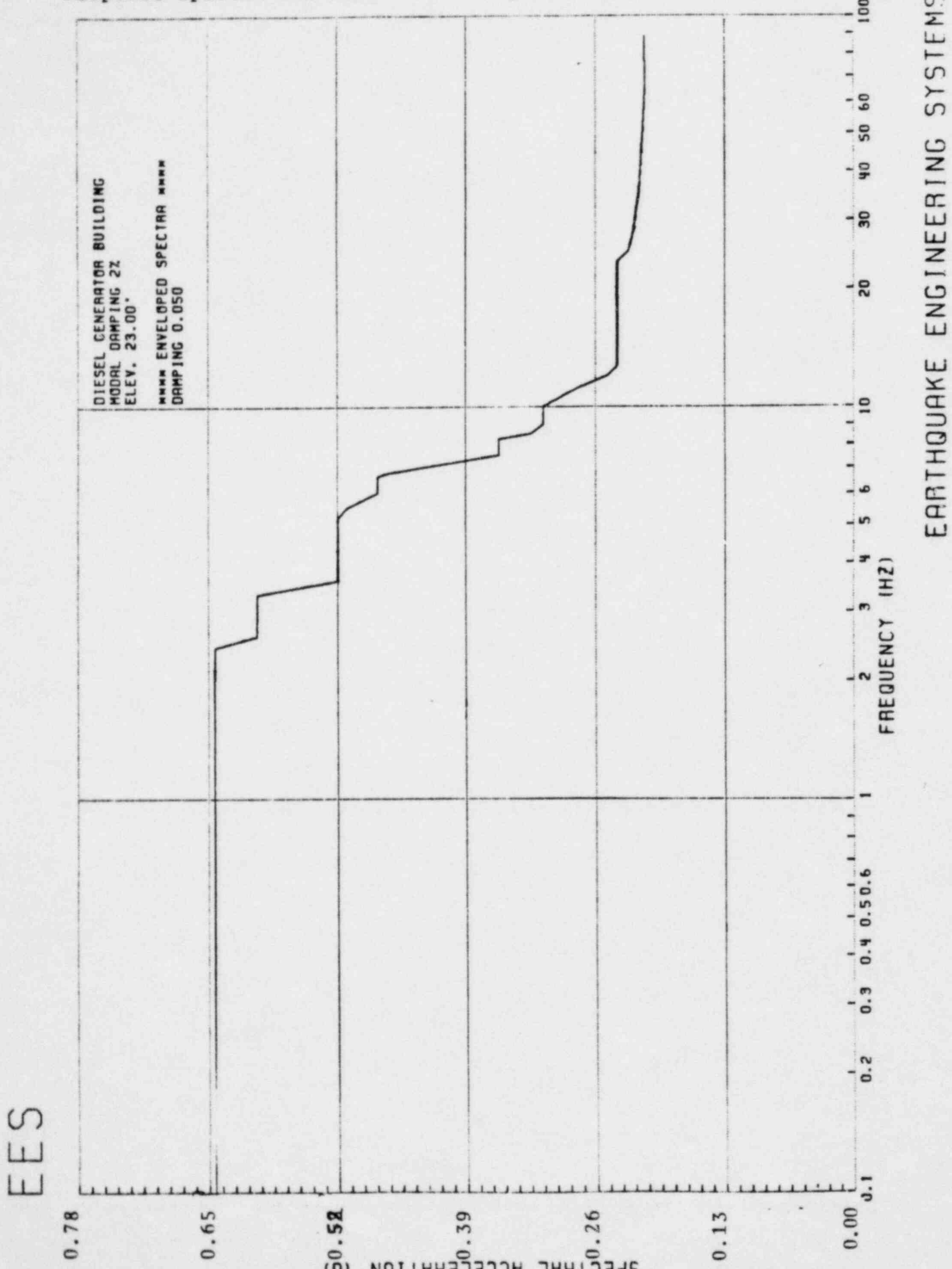




ATTACHMENT 1

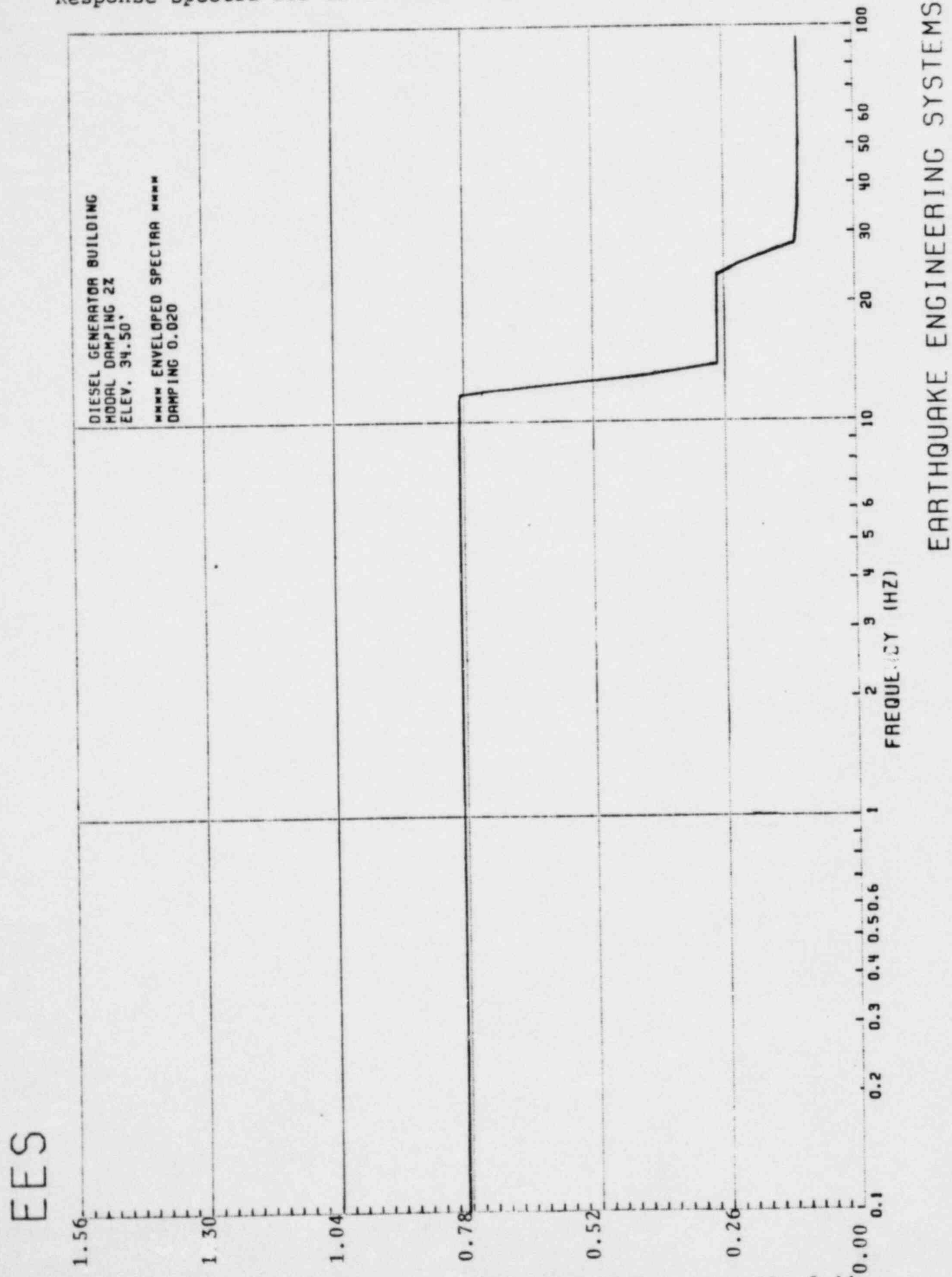
Response Spectra for Level I Masonry Wall Seismic Analysis

Response Spectra for Level I Masonry Wall Seismic Analysis



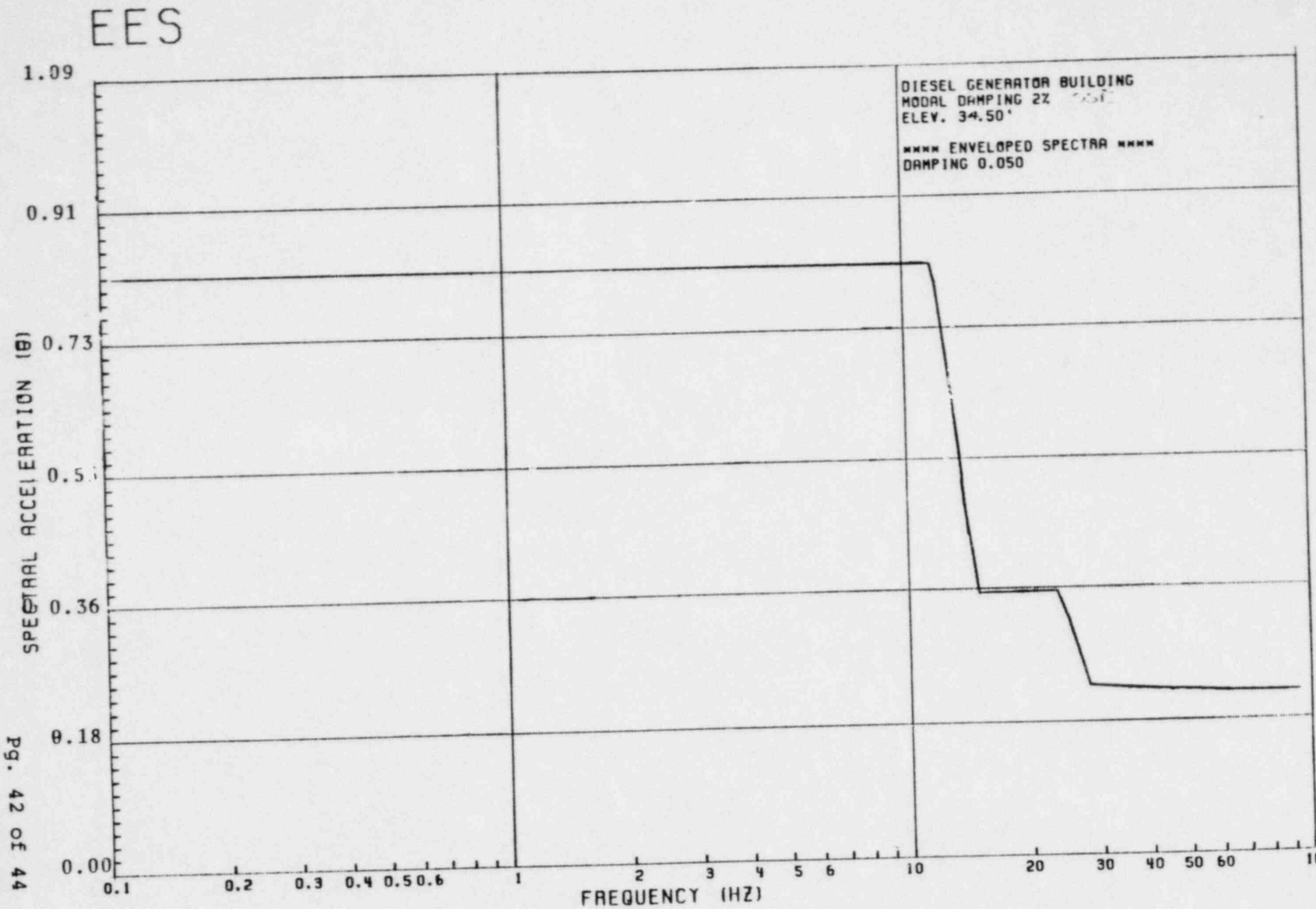
ATTACHMENT 1

Response Spectra for Level I Masonry Wall Seismic Analysis



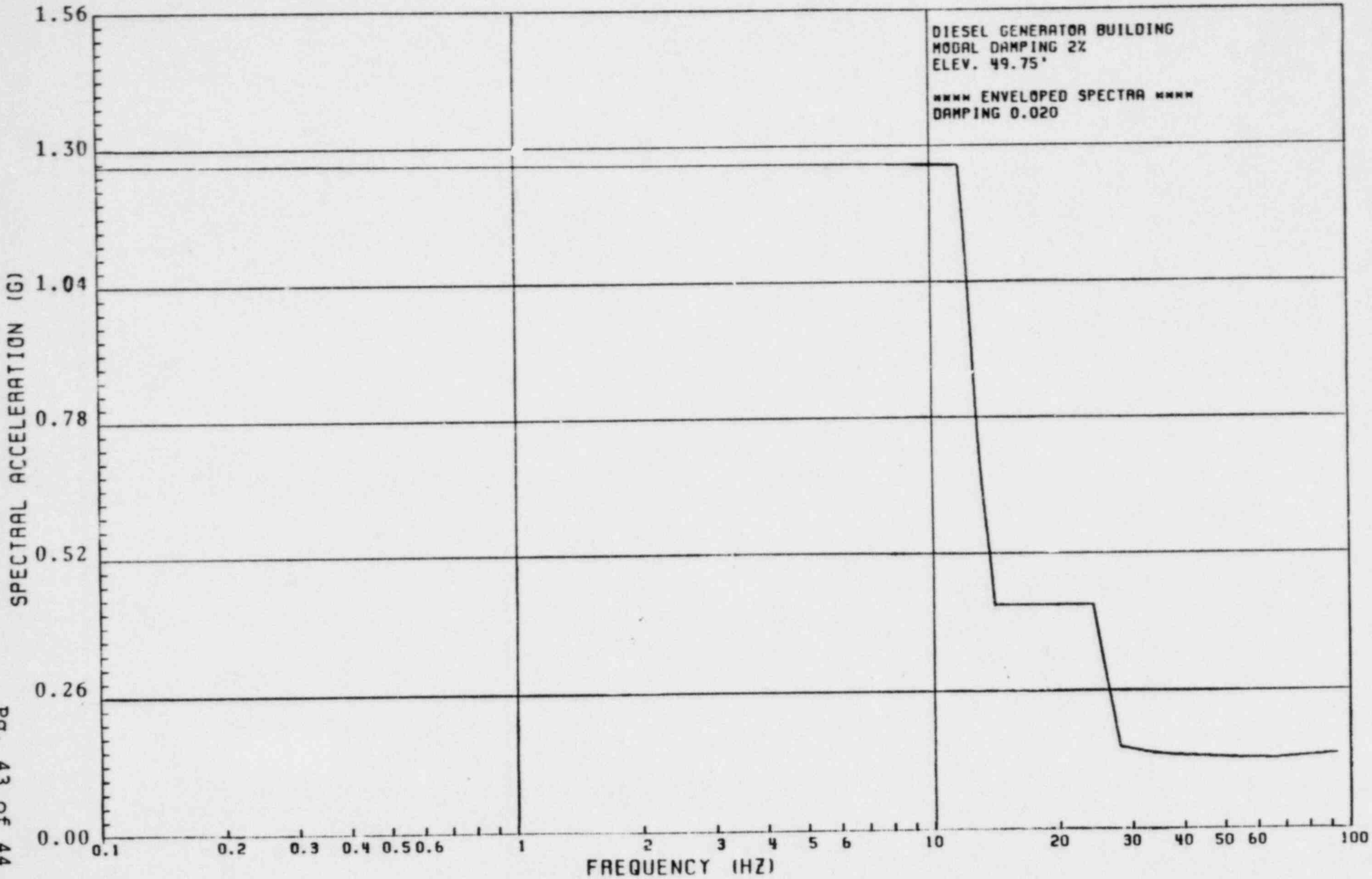
ATTACHMENT 1

Response Spectra for Level I Masonry Wall Seismic Analysis



EARTHQUAKE ENGINEERING SYSTEMS

EES



EARTHQUAKE ENGINEERING SYSTEMS

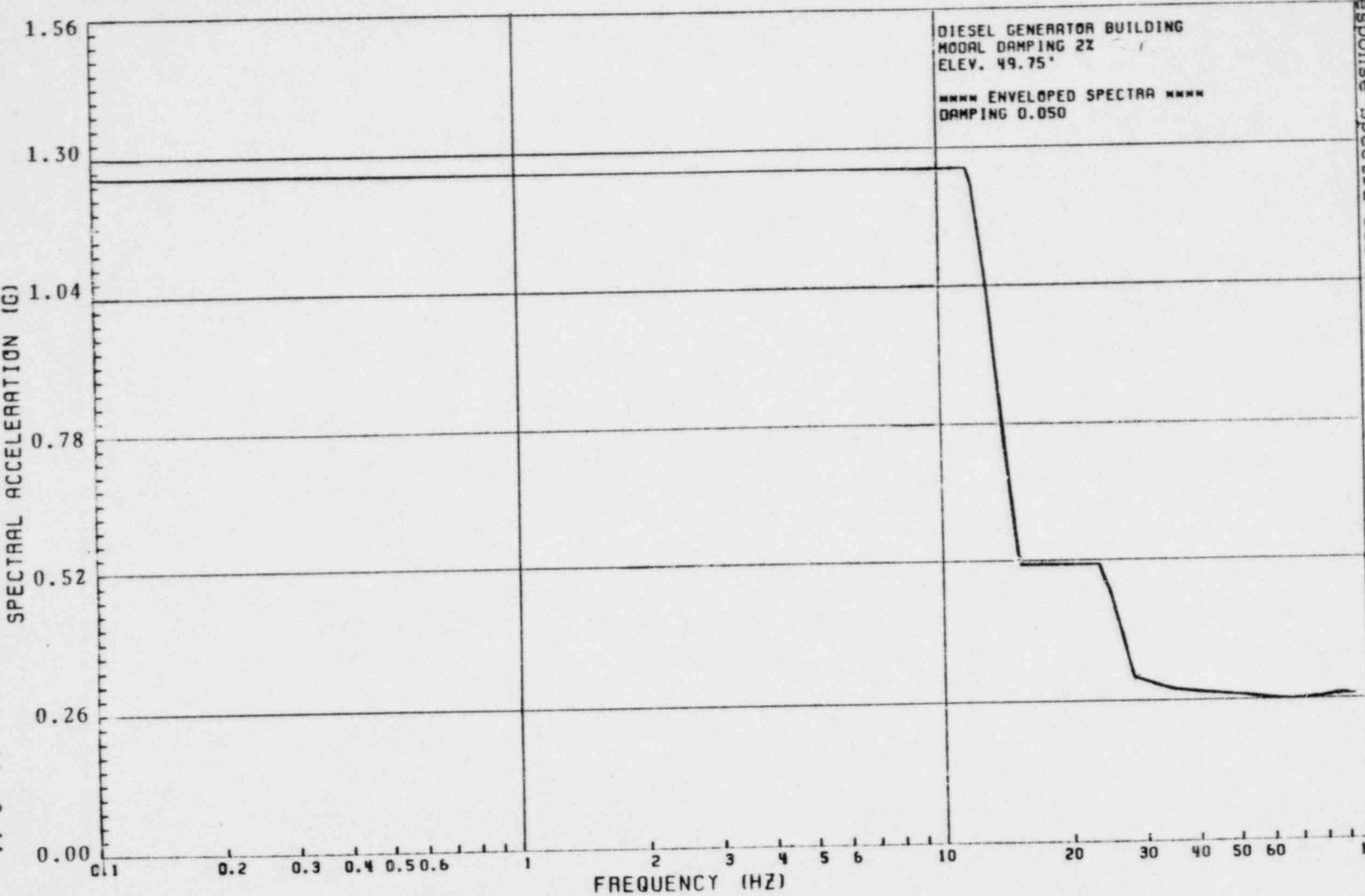
Response Spectra for Level I Masonry Wall Seismic Analysis

ATTACHMENT 1

ATTACHMENT 1

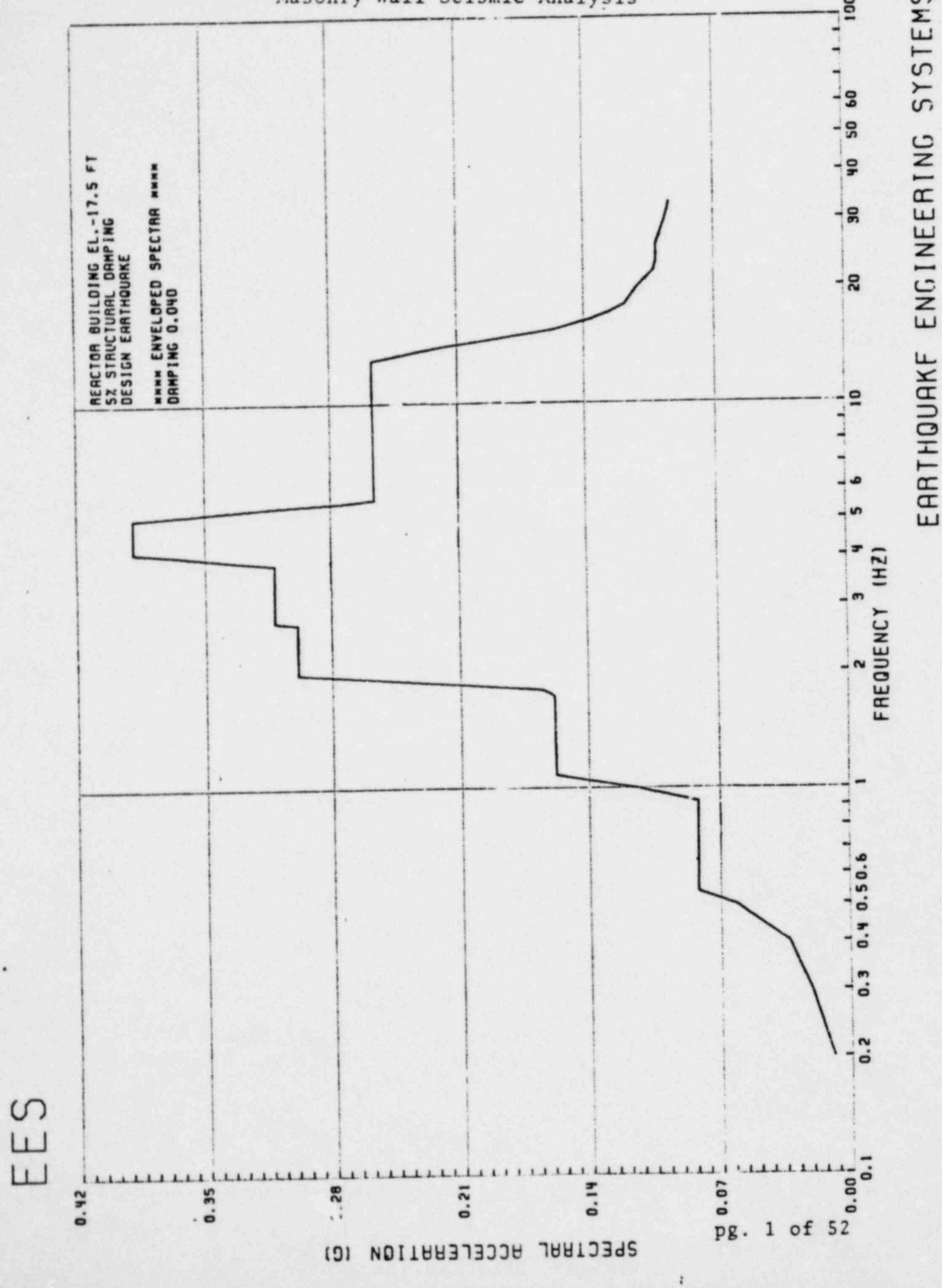
Response Spectra for Level I Masonry Wall Seismic Analysis

EES



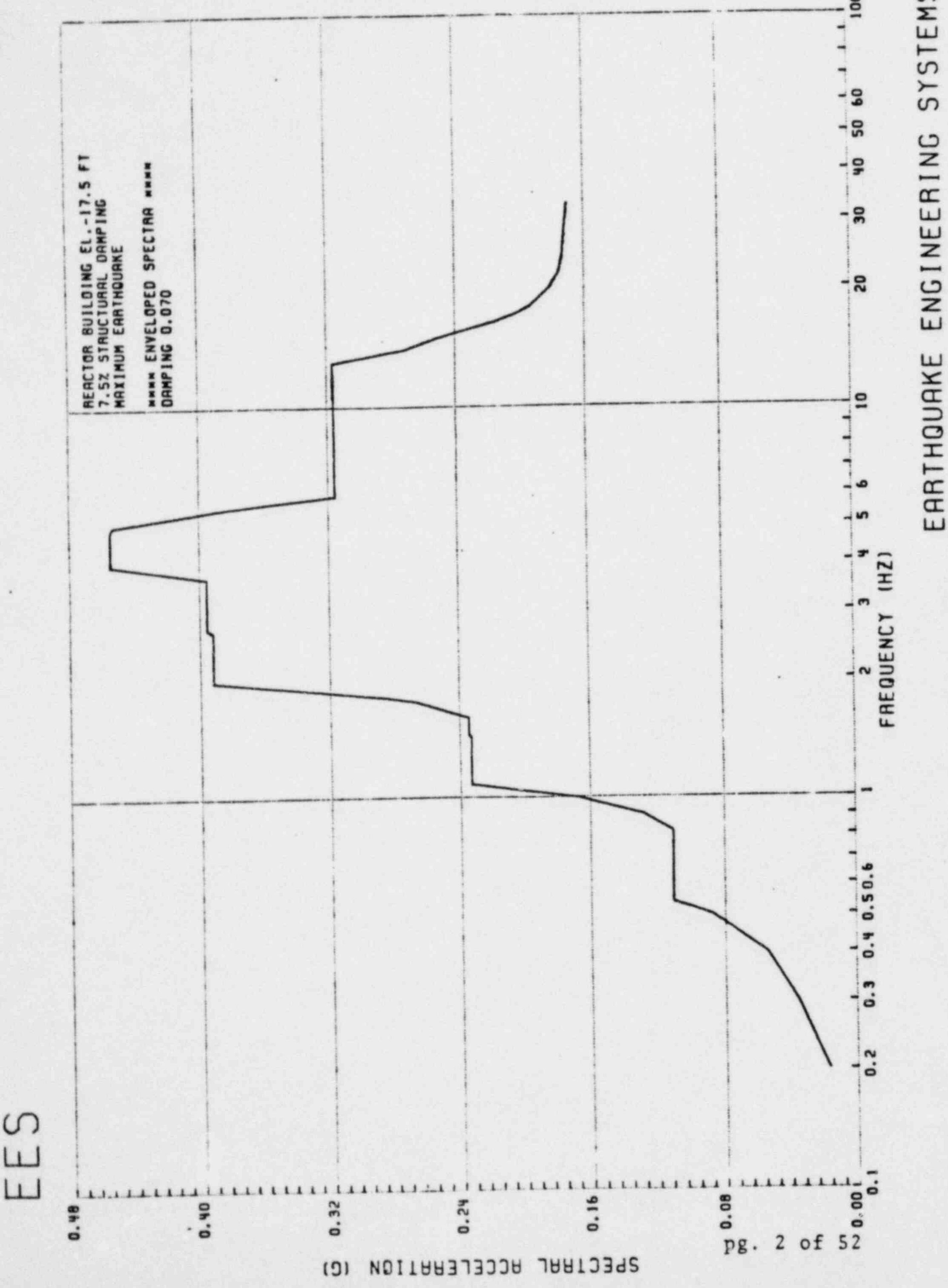
EARTHQUAKE ENGINEERING SYSTEMS

Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

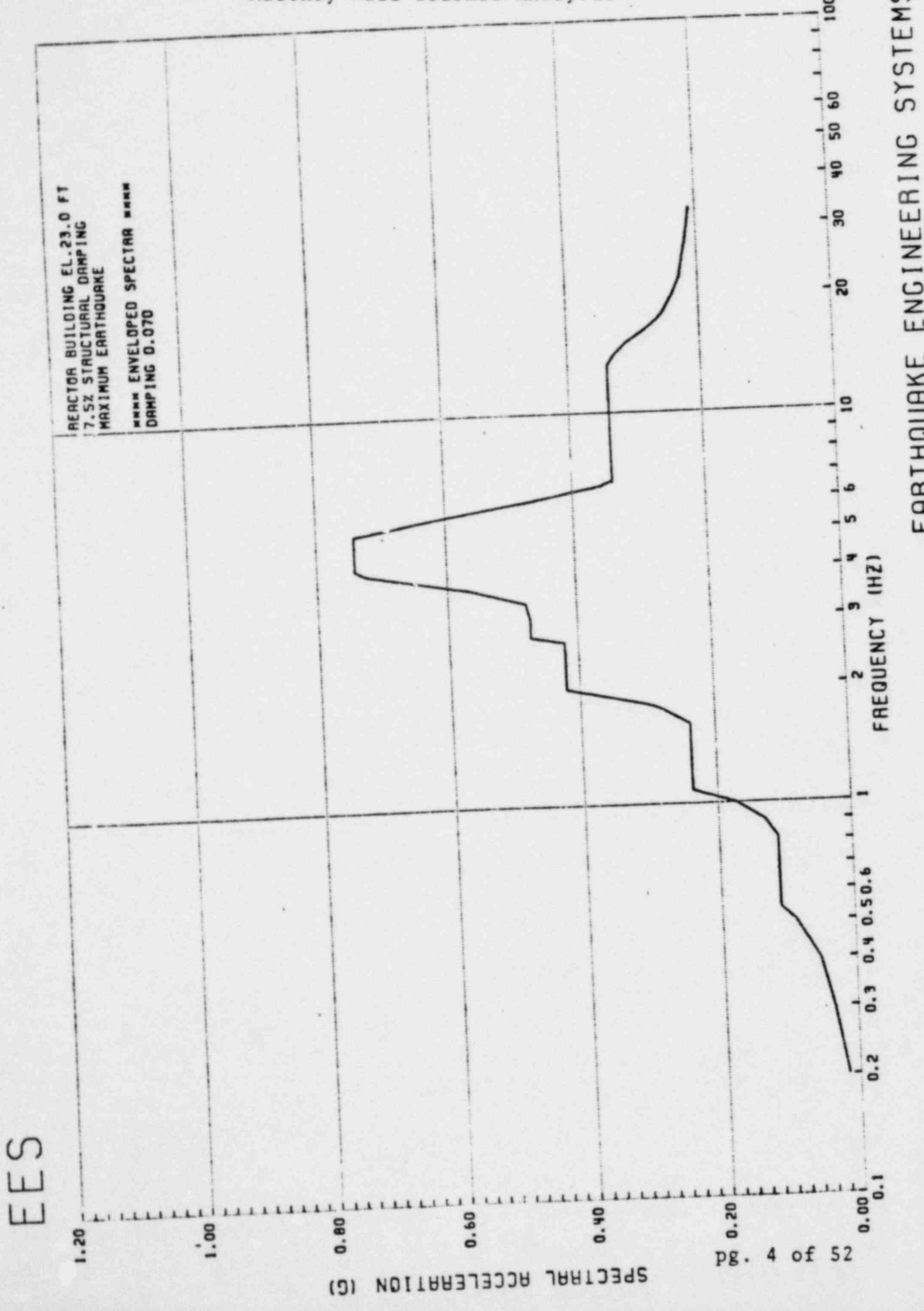


Attachment 2

Response Spectra for Level 2
Masonry Wall Seismic Analysis



Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis



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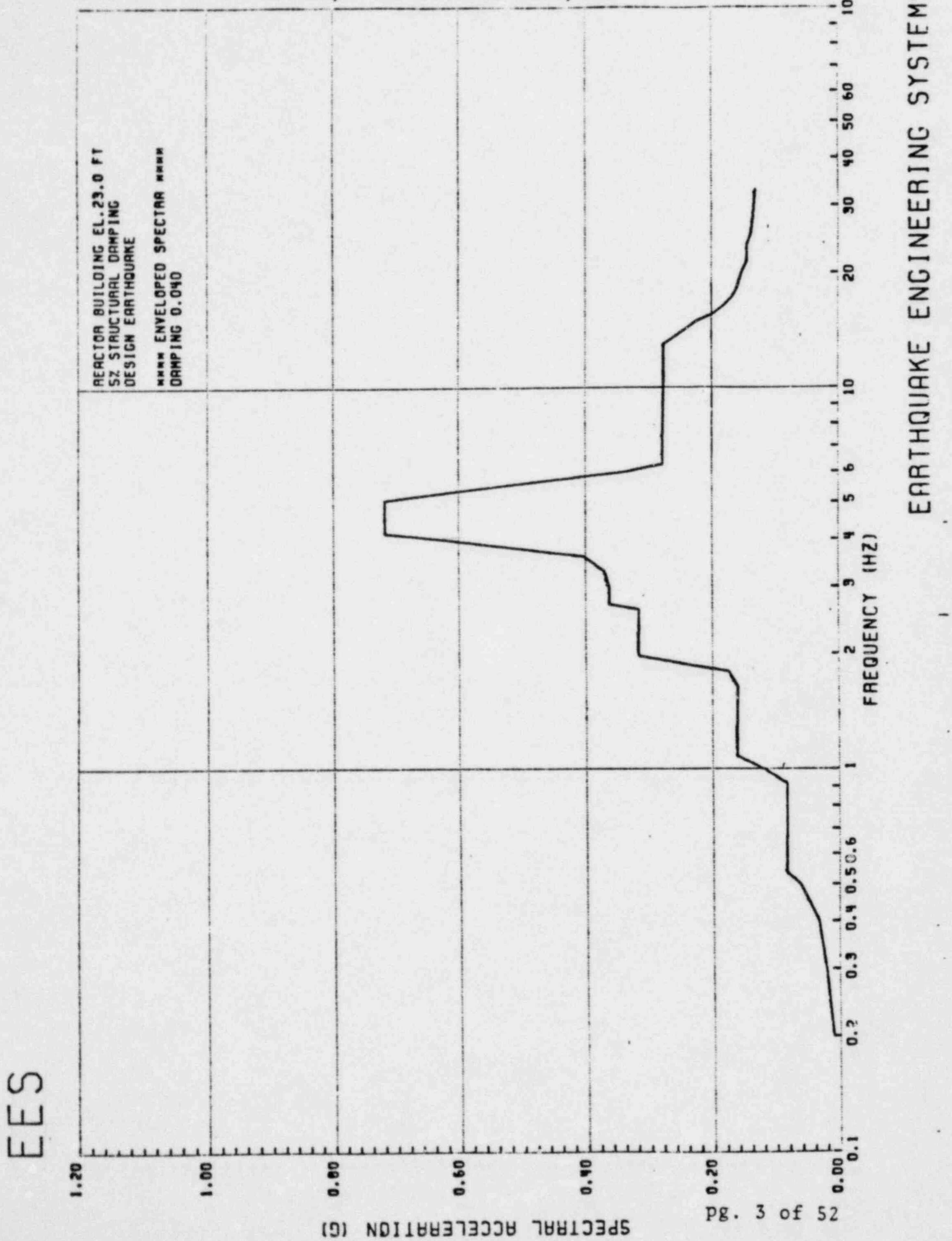
Response Spectra for Level 2
Masonry Wall Seismic Analysis



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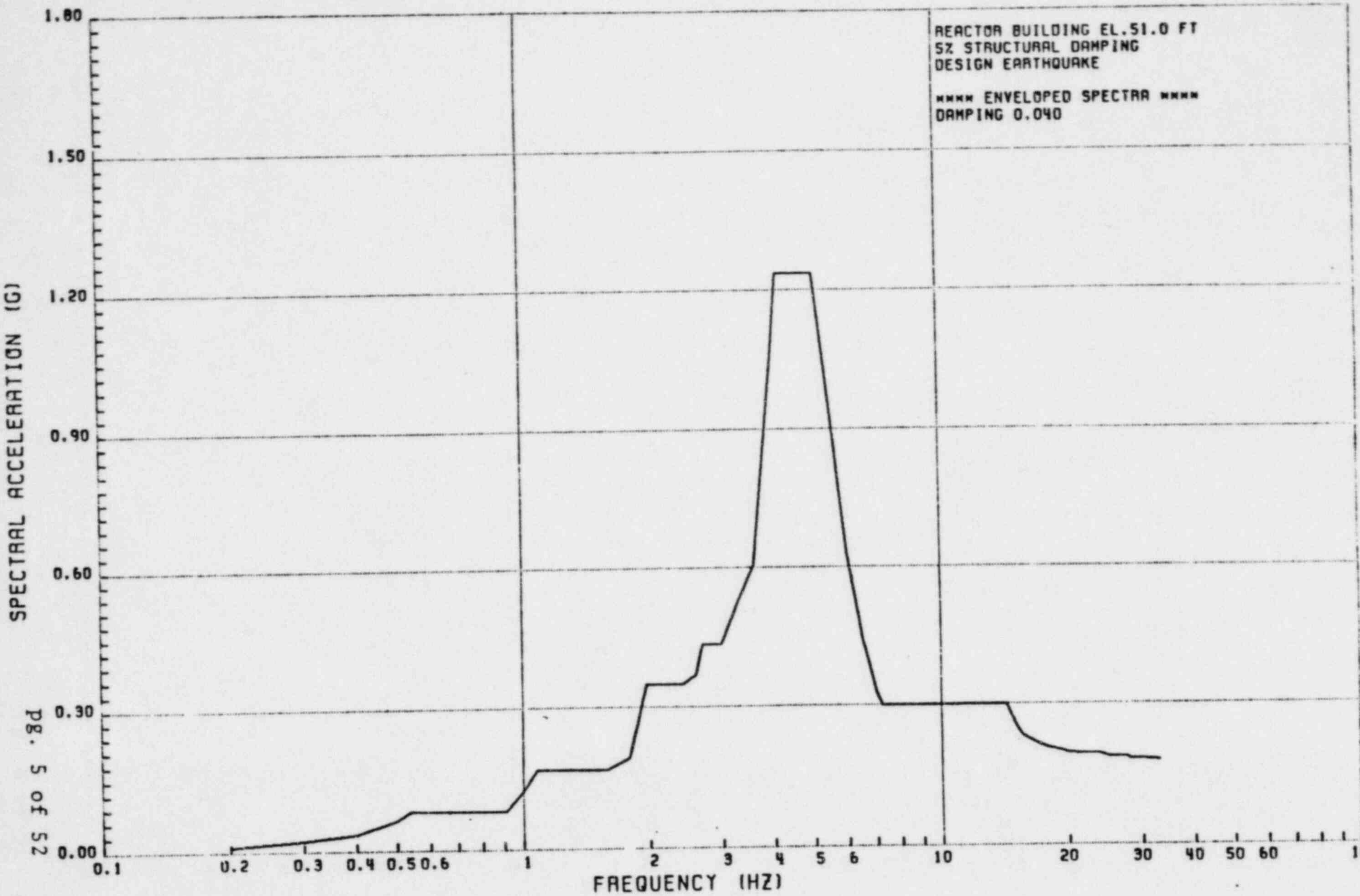
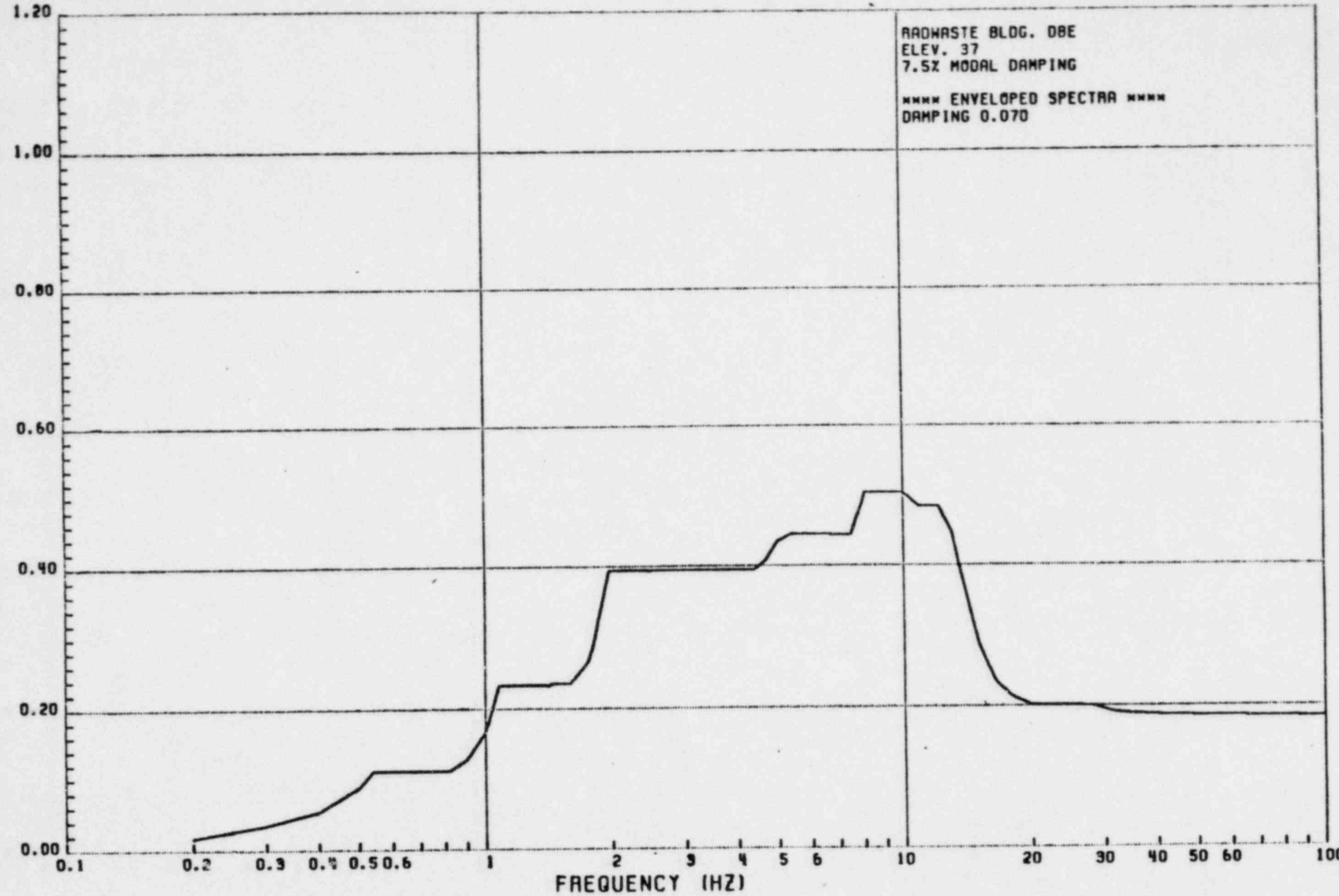


Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

EARTHQUAKE ENGINEERING SYSTEMS

EES

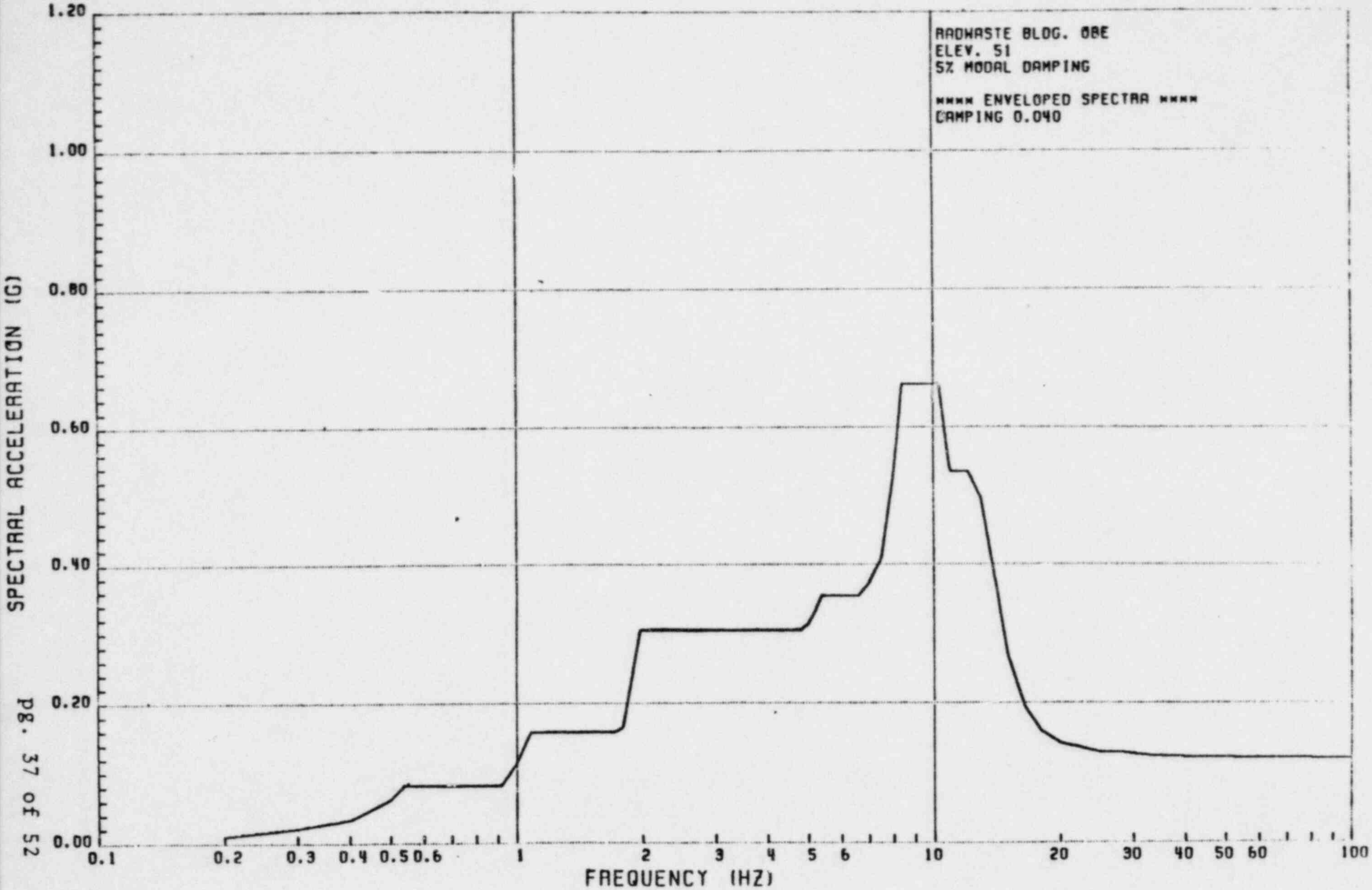


Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

pg. 36 of 52

EARTHQUAKE ENGINEERING SYSTEMS

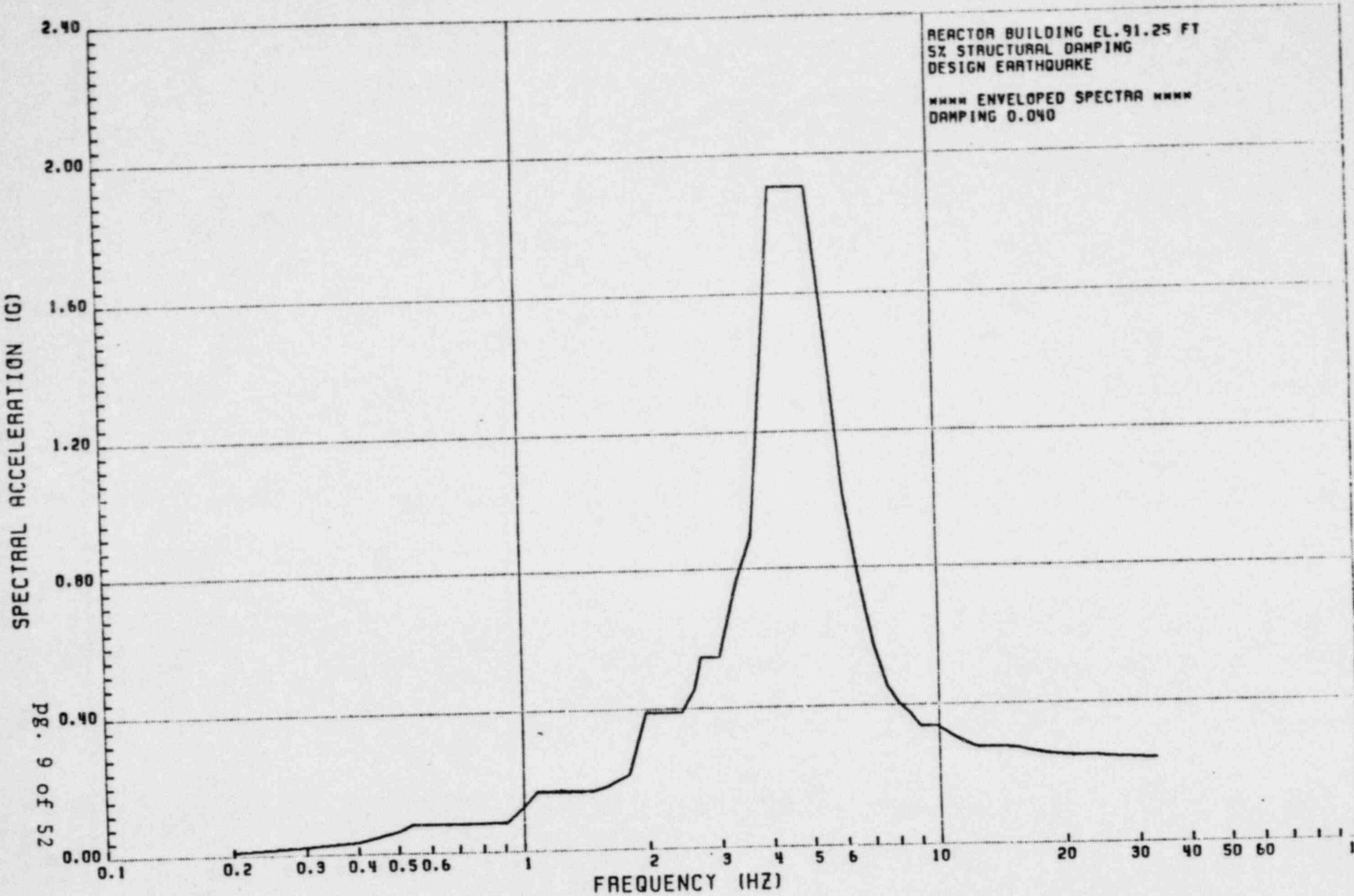
EES



EARTHQUAKE ENGINEERING SYSTEMS

Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

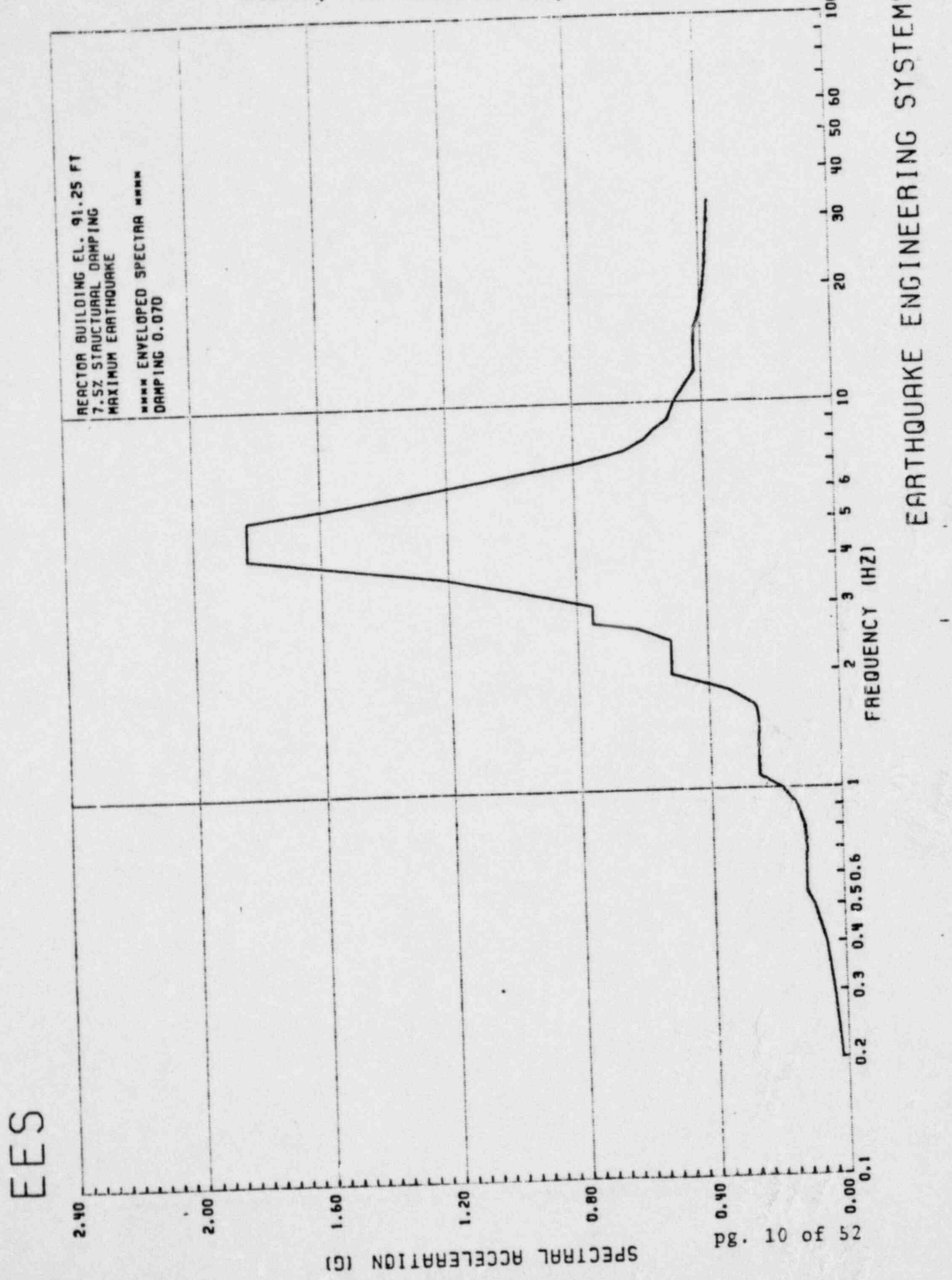
EES



EARTHQUAKE ENGINEERING SYSTEMS

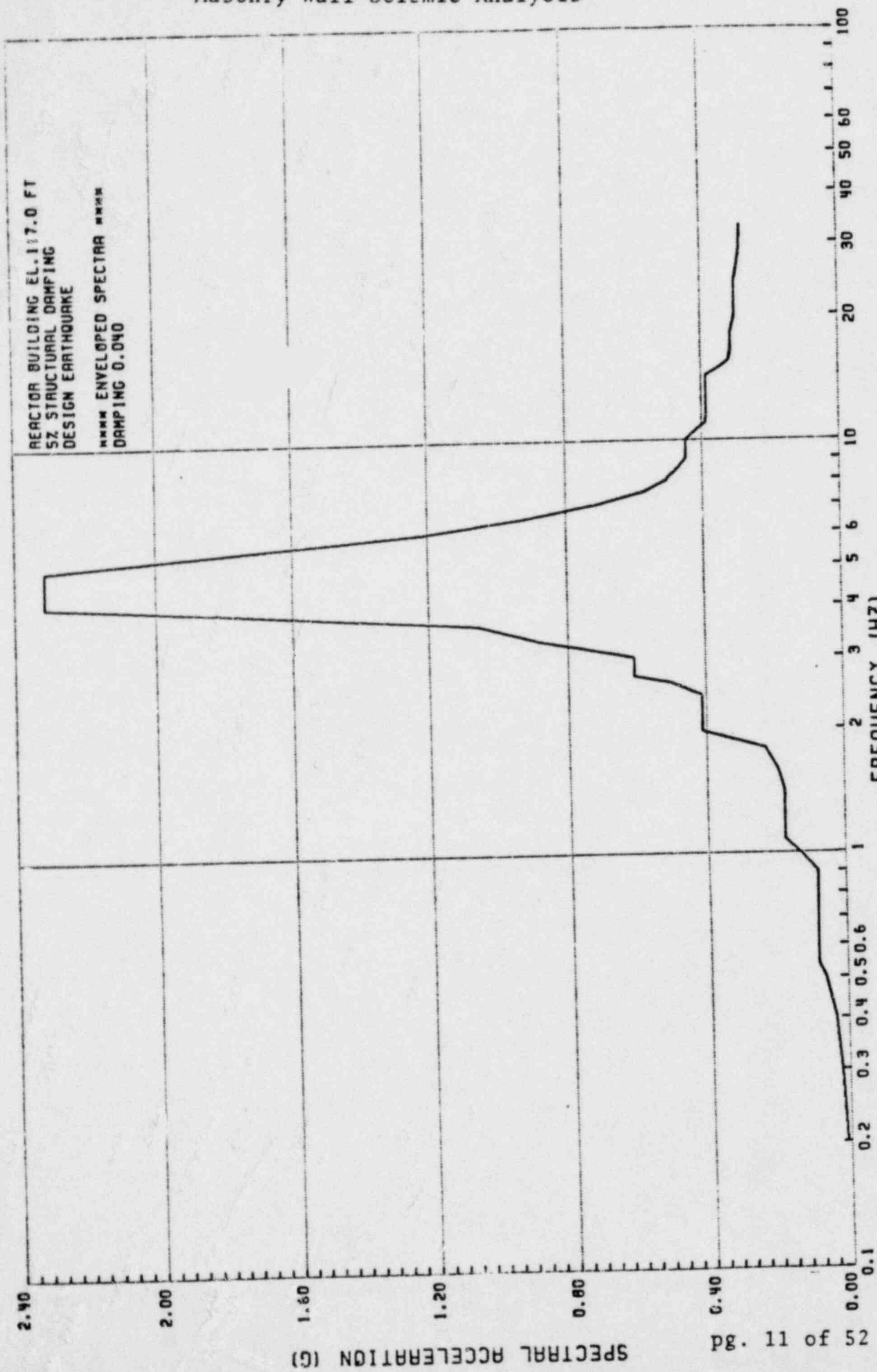
Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis



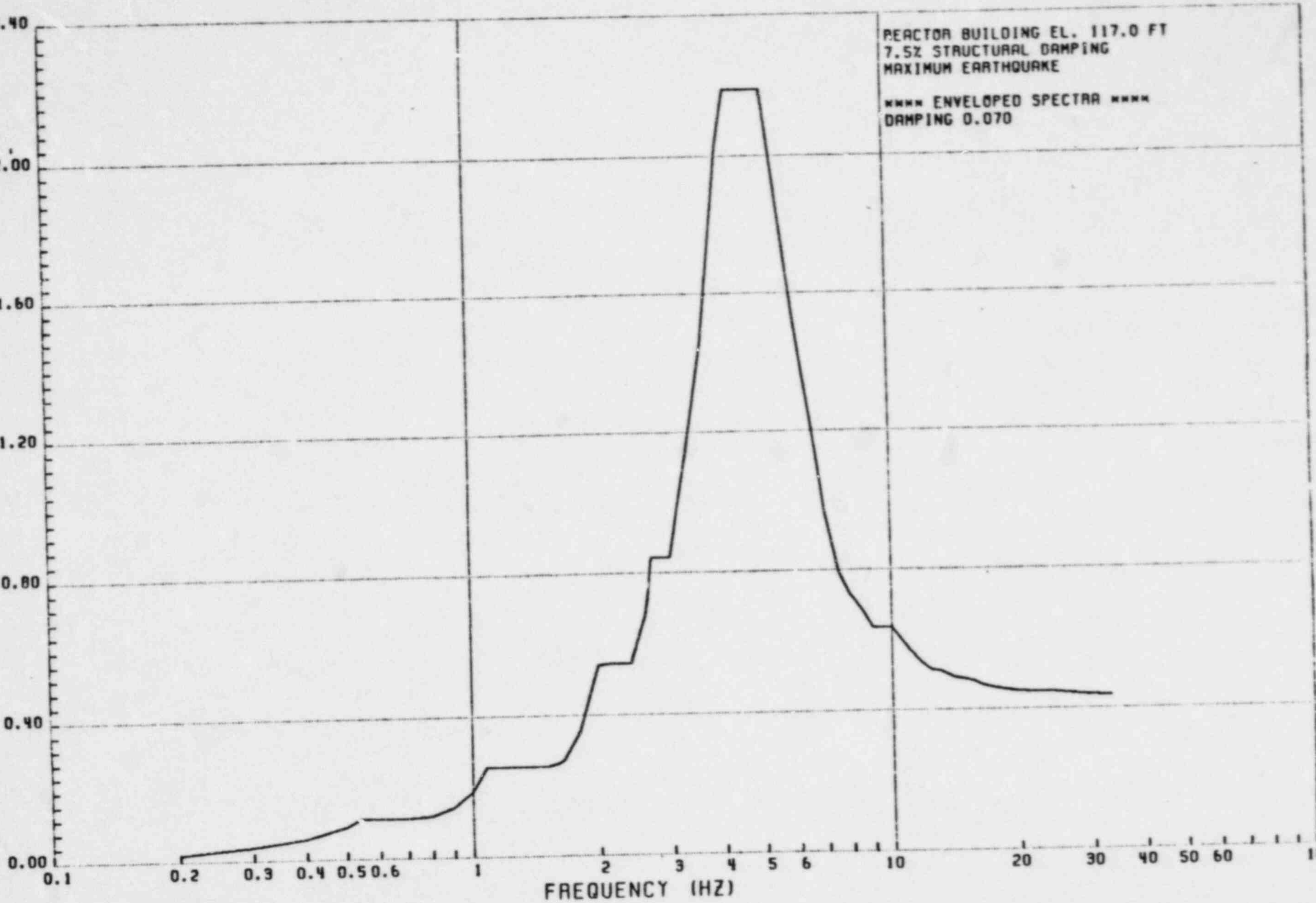
Response Spectra for Level 2
Masonry Wall Seismic Analysis

EES



EARTHQUAKE ENGINEERING SYSTEMS

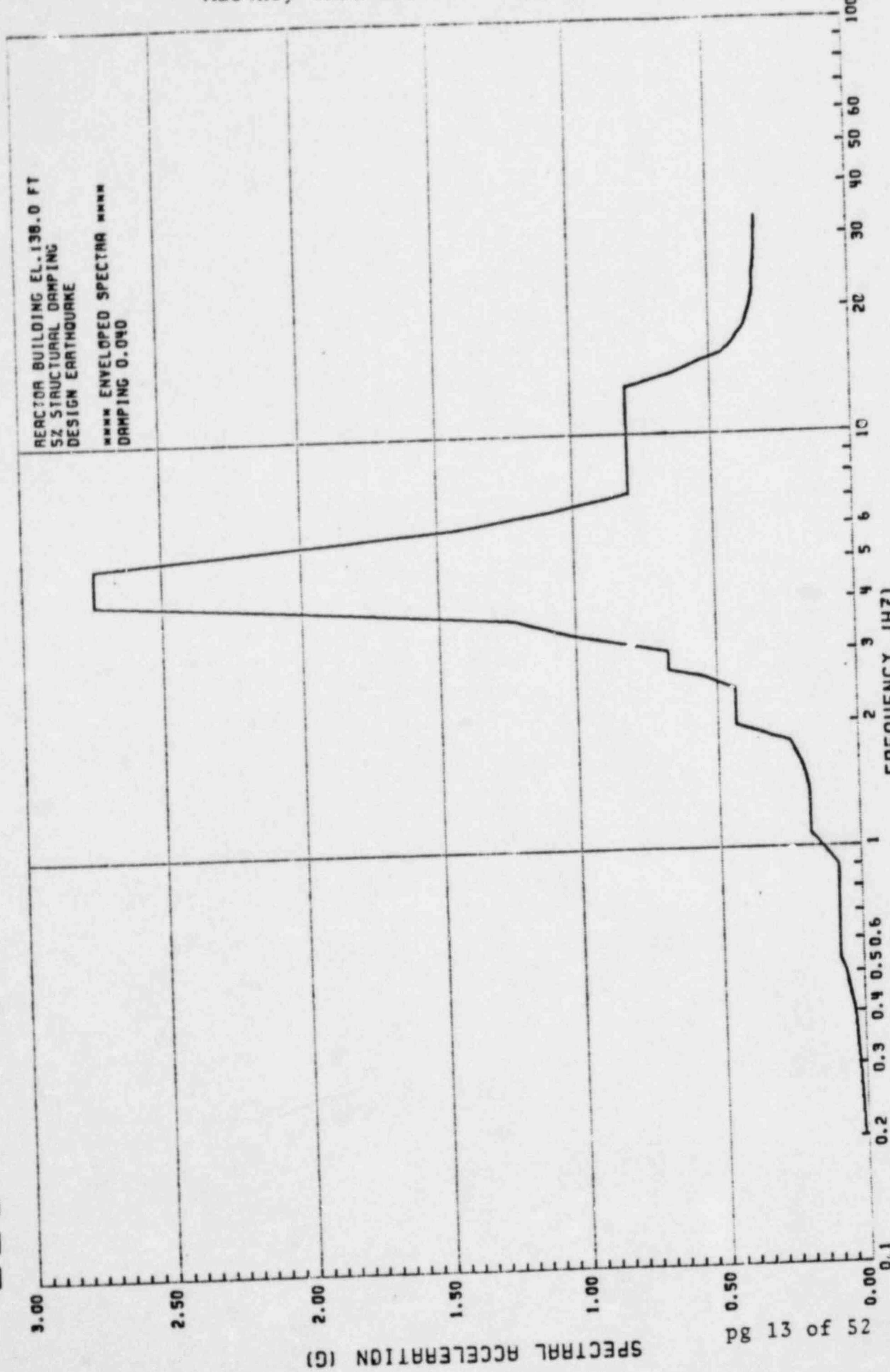
EES



EARTHQUAKE ENGINEERING SYSTEMS

Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

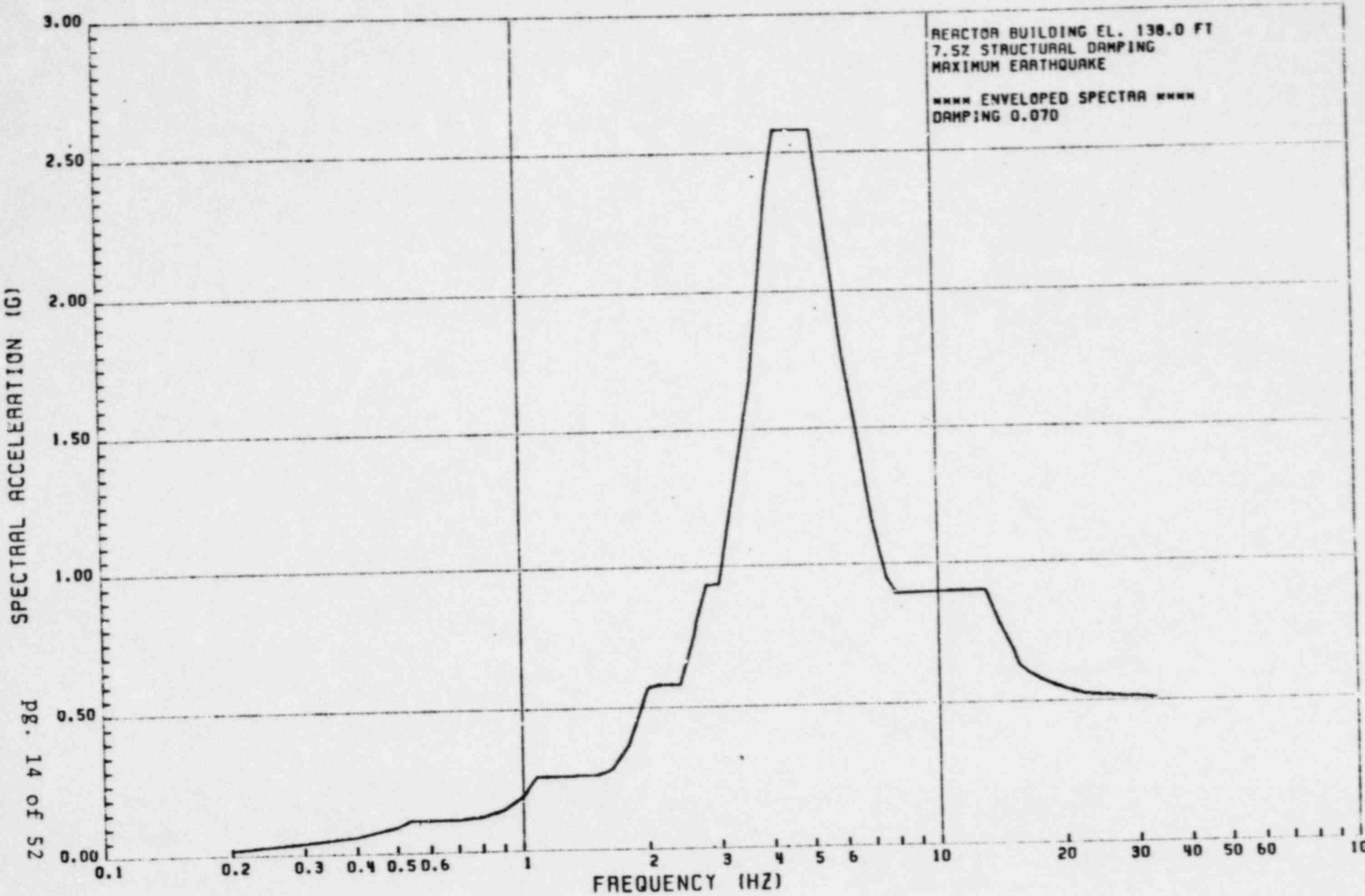
Attachment2
Response Spectra for Level 2
Masonry Wall Seismic Analysis



EES

EARTHQUAKE ENGINEERING SYSTEMS

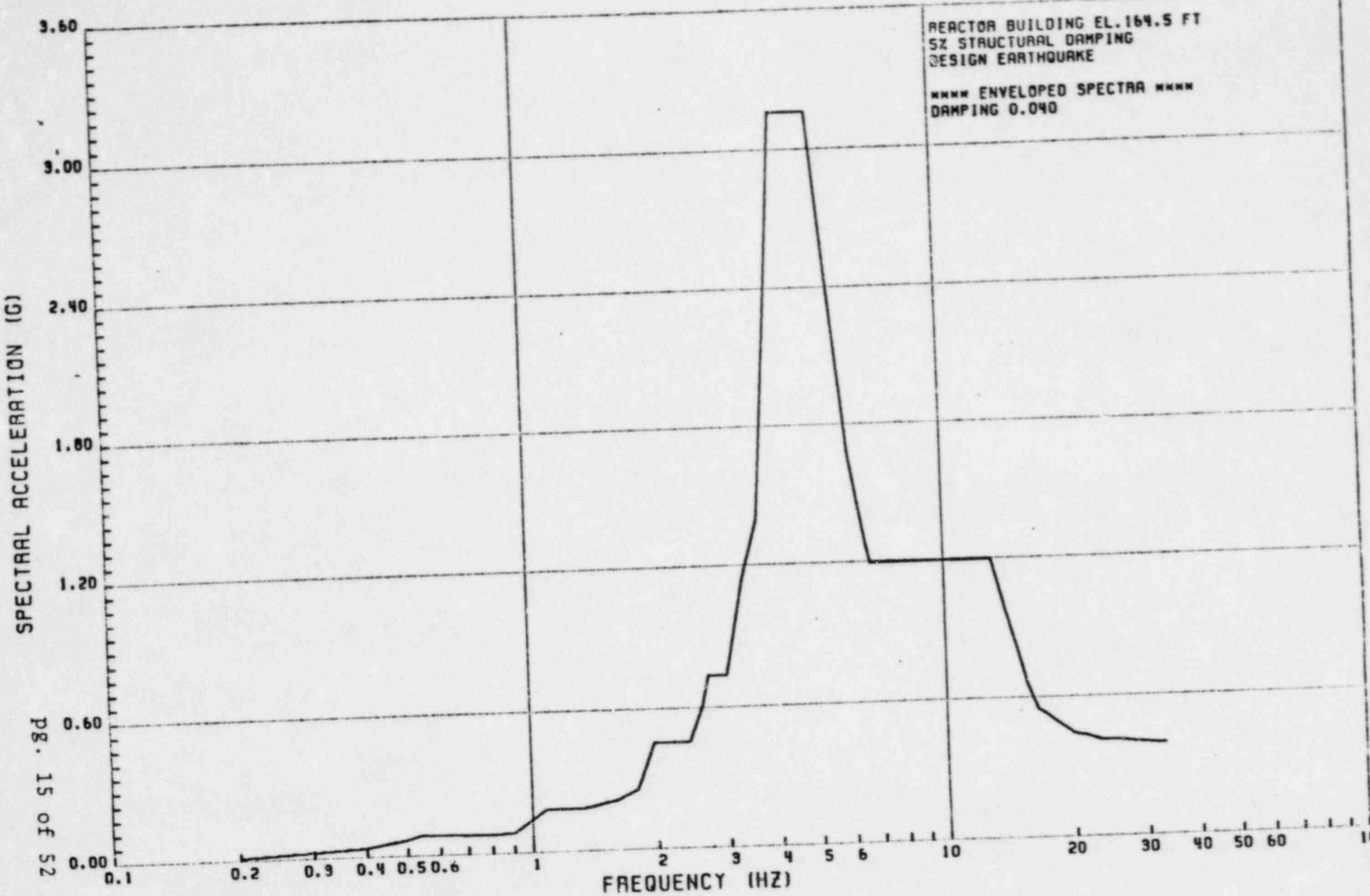
EES



EARTHQUAKE ENGINEERING SYSTEMS

Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

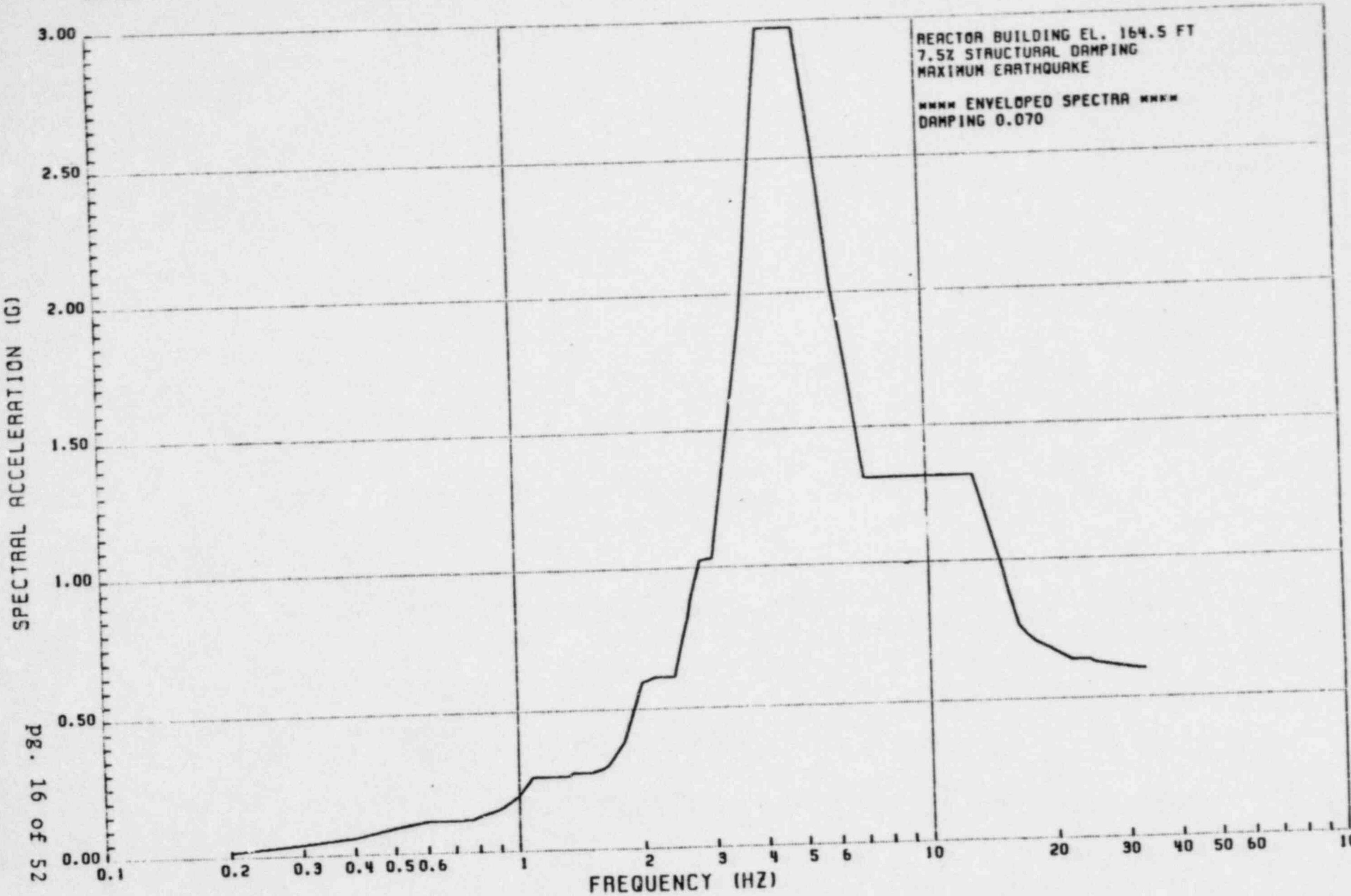
EES



EARTHQUAKE ENGINEERING SYSTEMS

Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

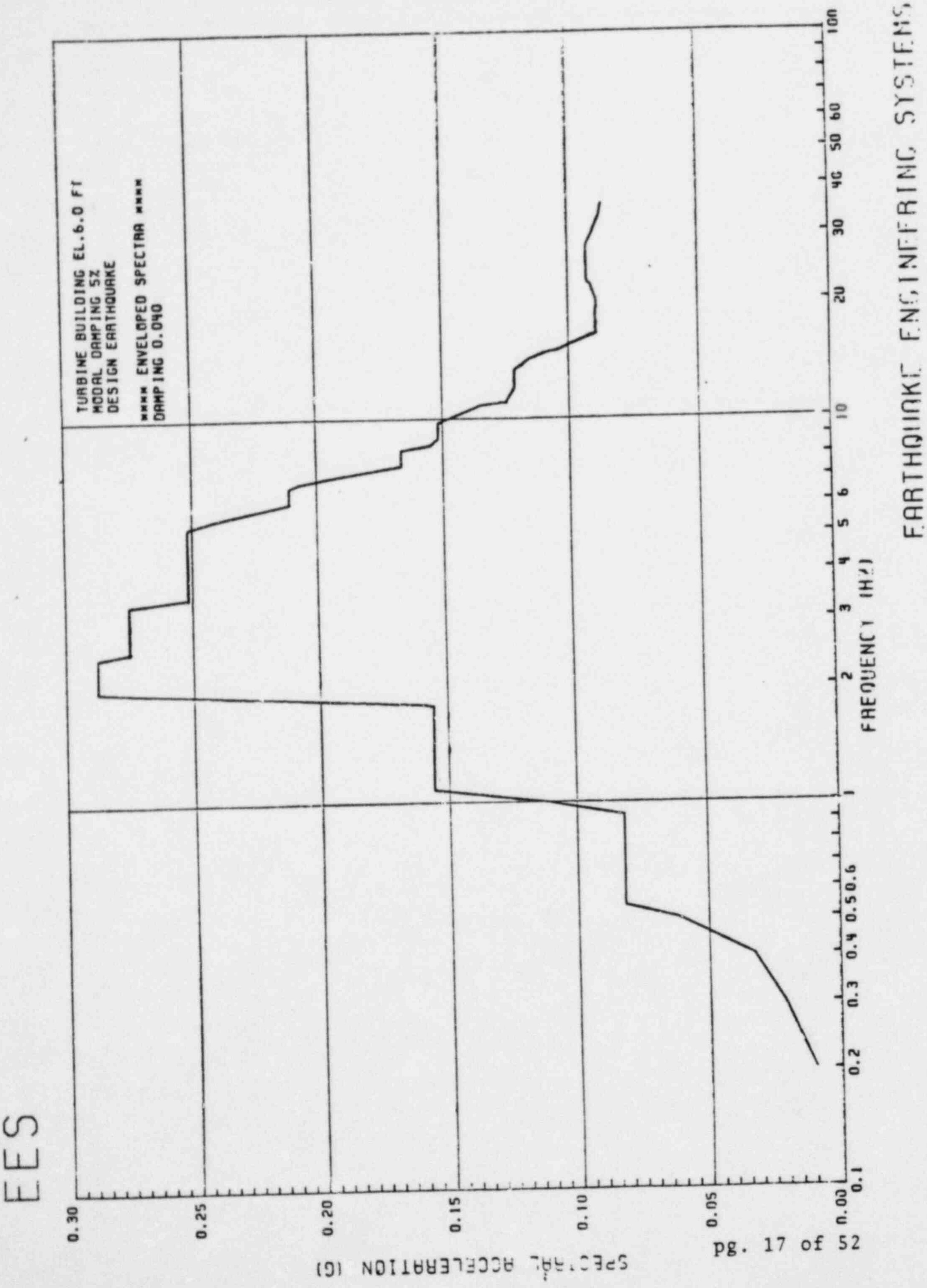
EES



EARTHQUAKE ENGINEERING SYSTEMS

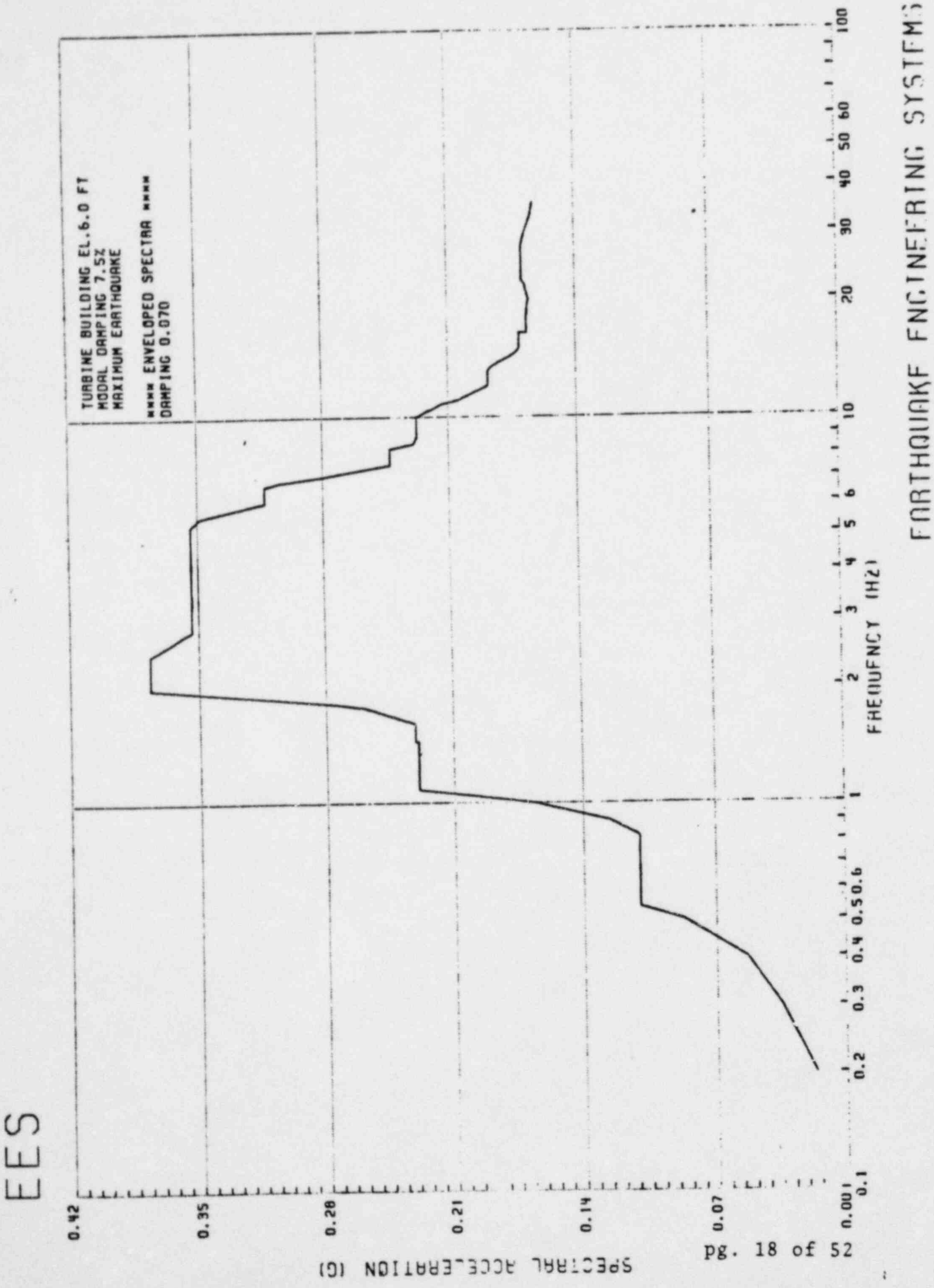
Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

<u>Attachment 2</u>
Response Spectra for Level 2
Masonry Wall Seismic Analysis

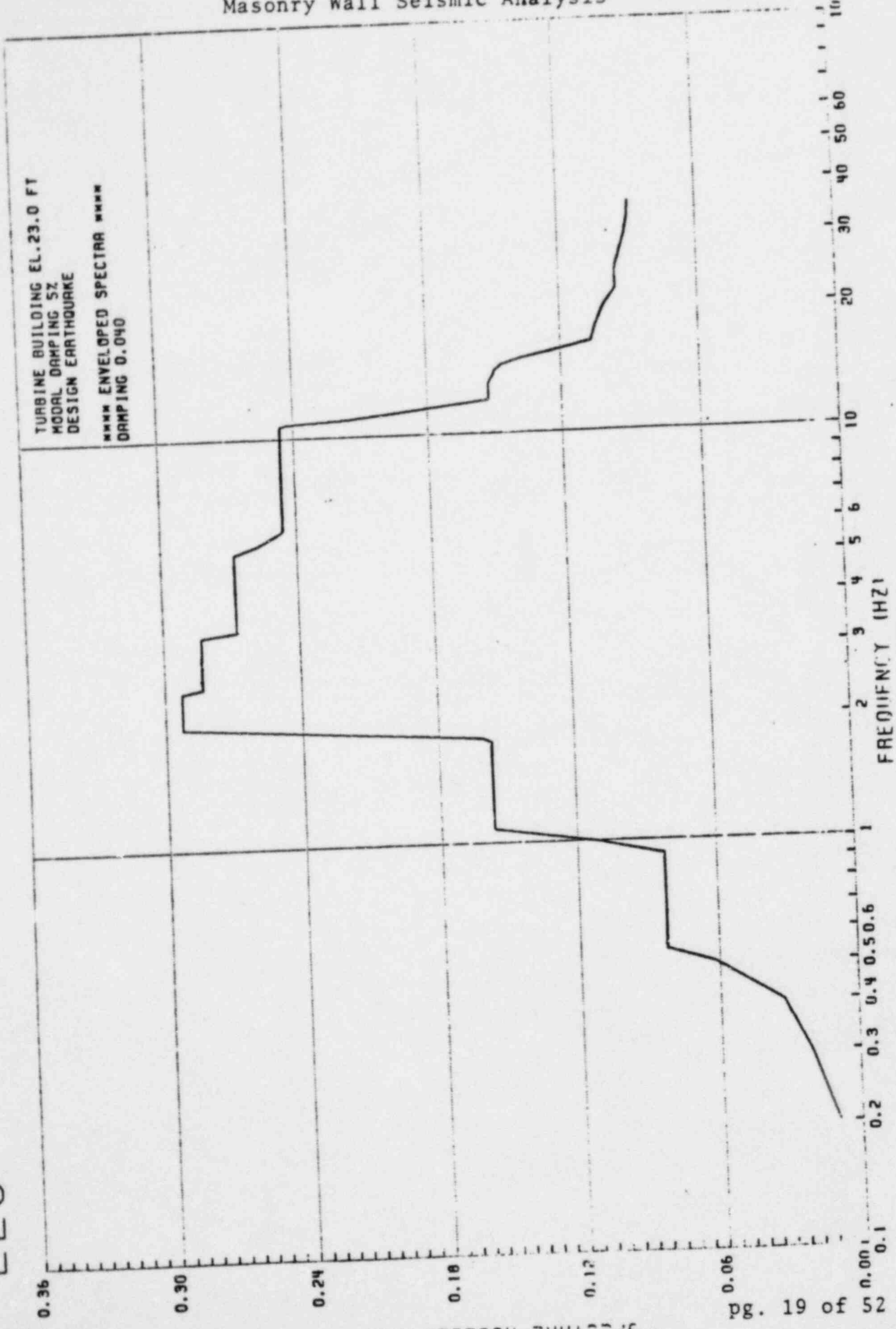


Attachment2

Response Spectra for Level 2
Masonry Wall Seismic Analysis



Attachment 2
Response Spectra of Level 2
Masonry Wall Seismic Analysis

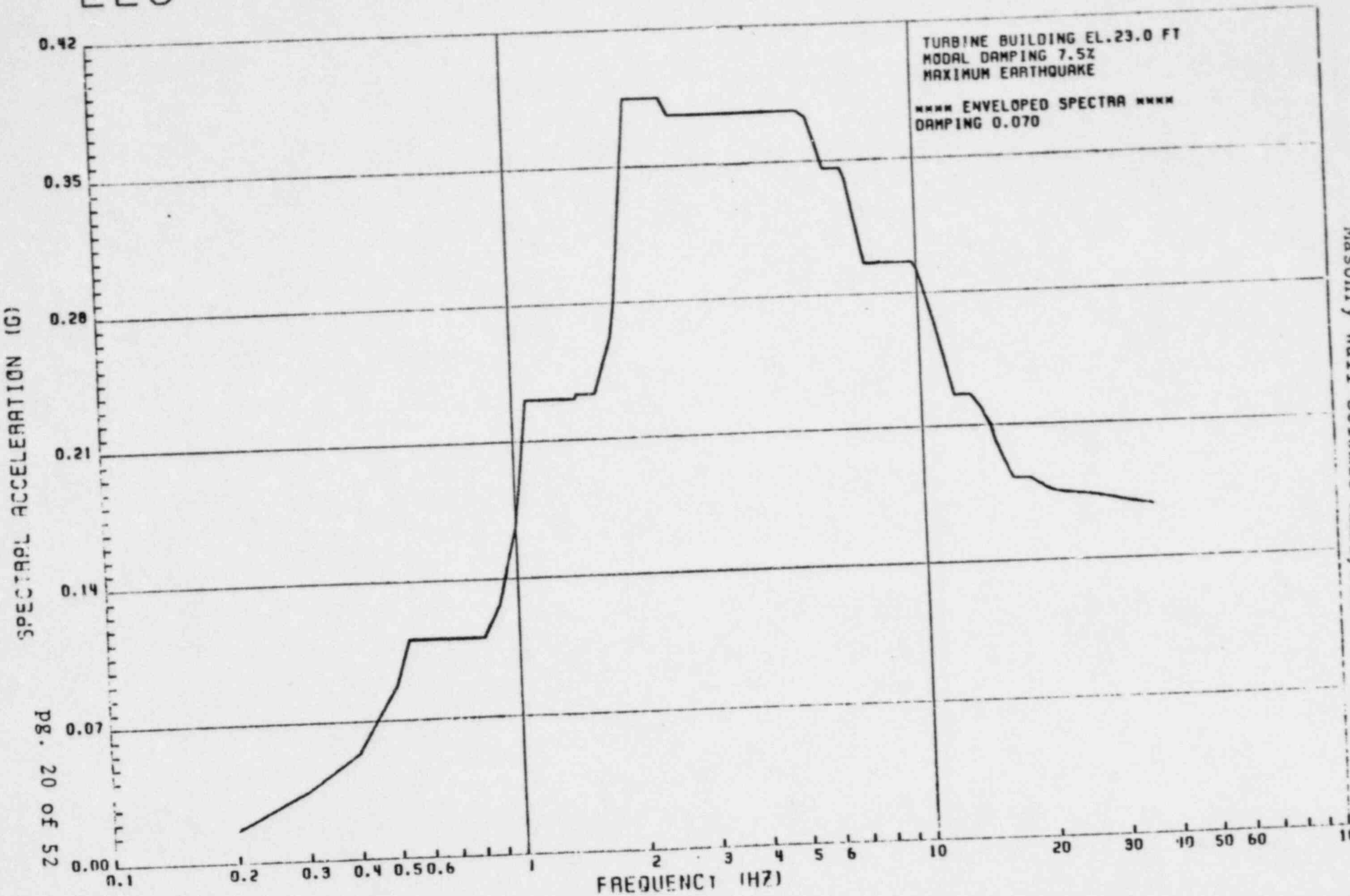


EES

EARTHQUAKE ENGINEERING SYSTEMS

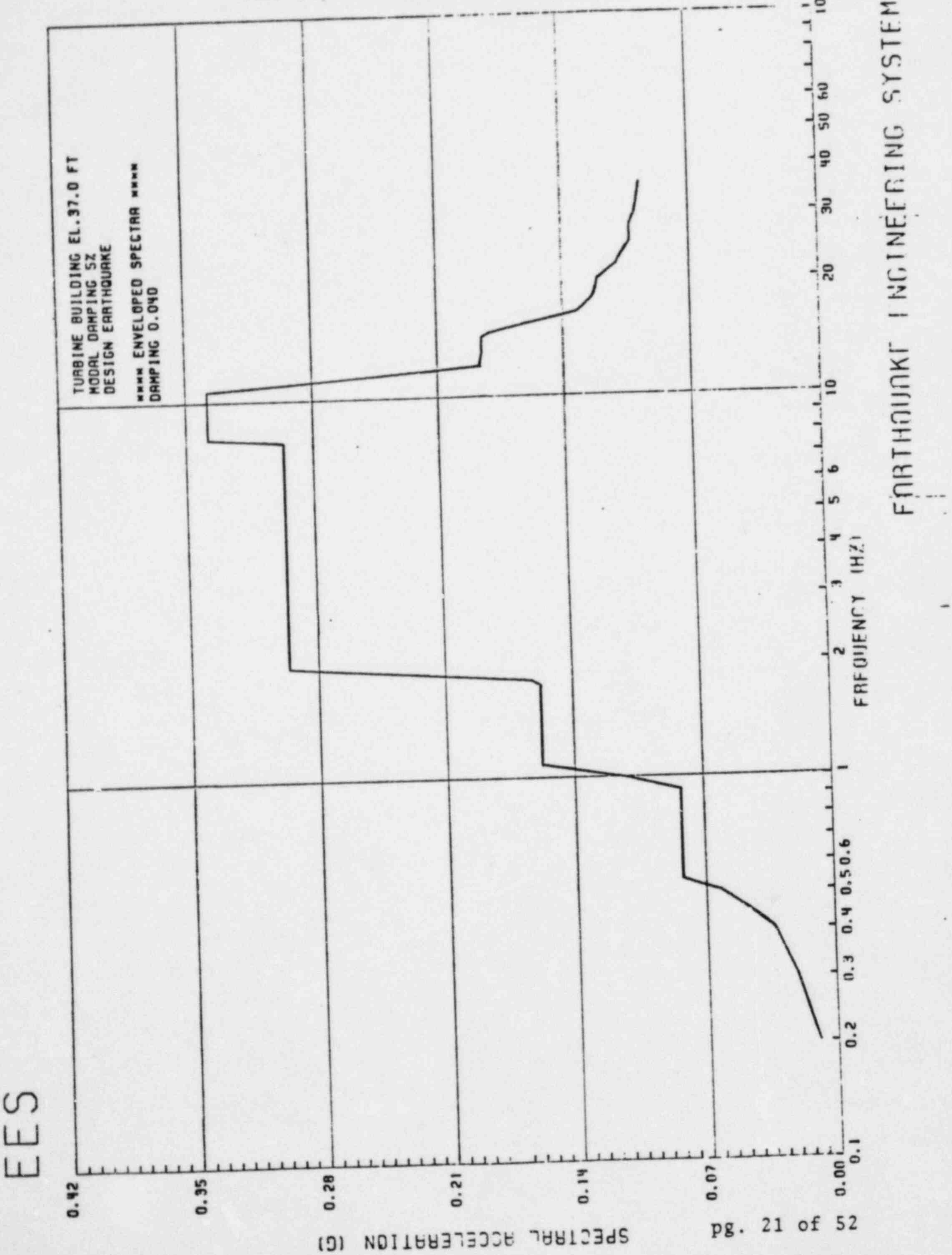
EES

Attachment 2
Response Spectra of Level 2
Masonry Wall Seismic Analysis

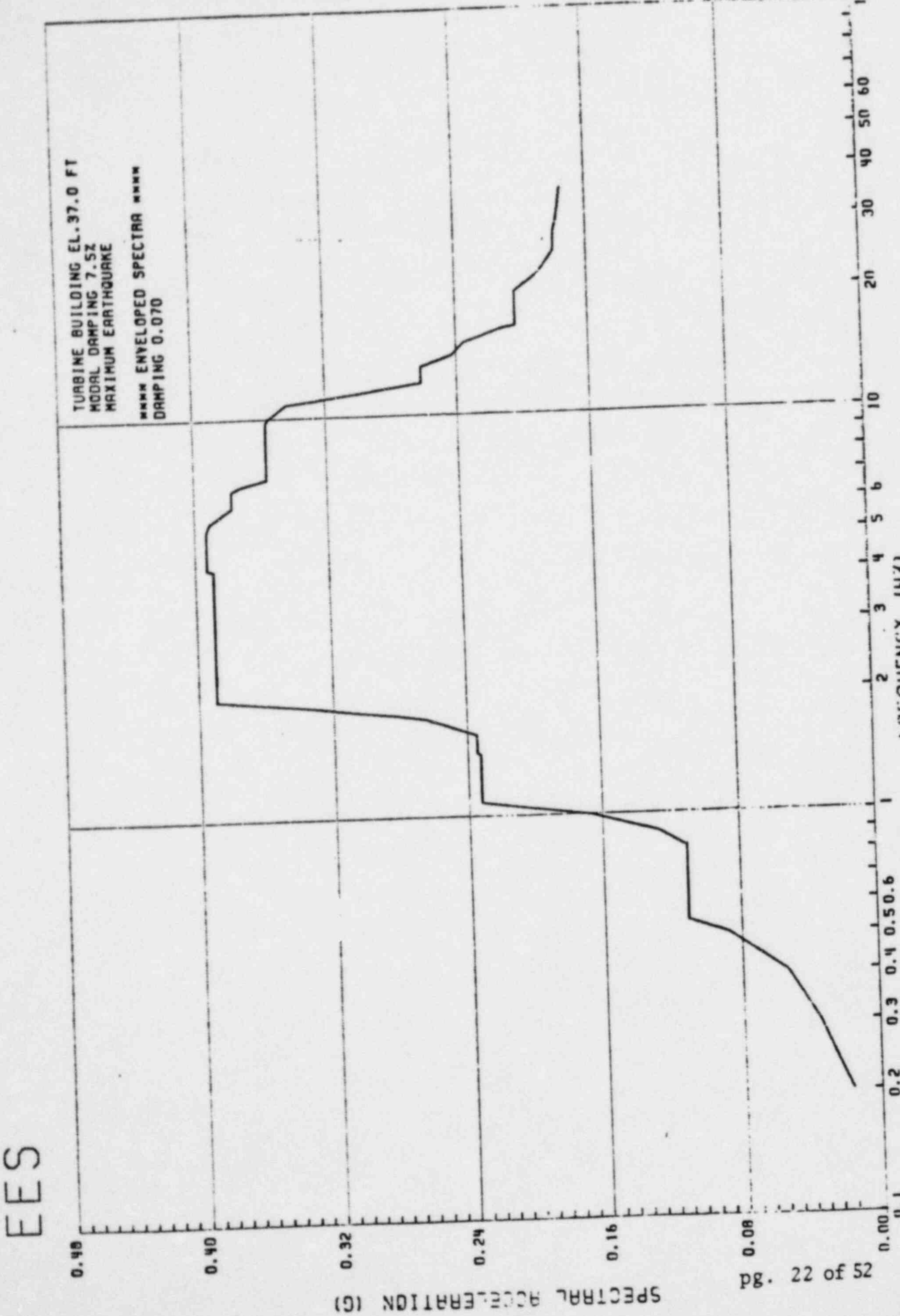


EARTHQUAKE ENGINEERING SYSTEMS

Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis



<u>Attachment 2</u>
Response Spectra for Level 2
Masonry Wall Seismic Analysis



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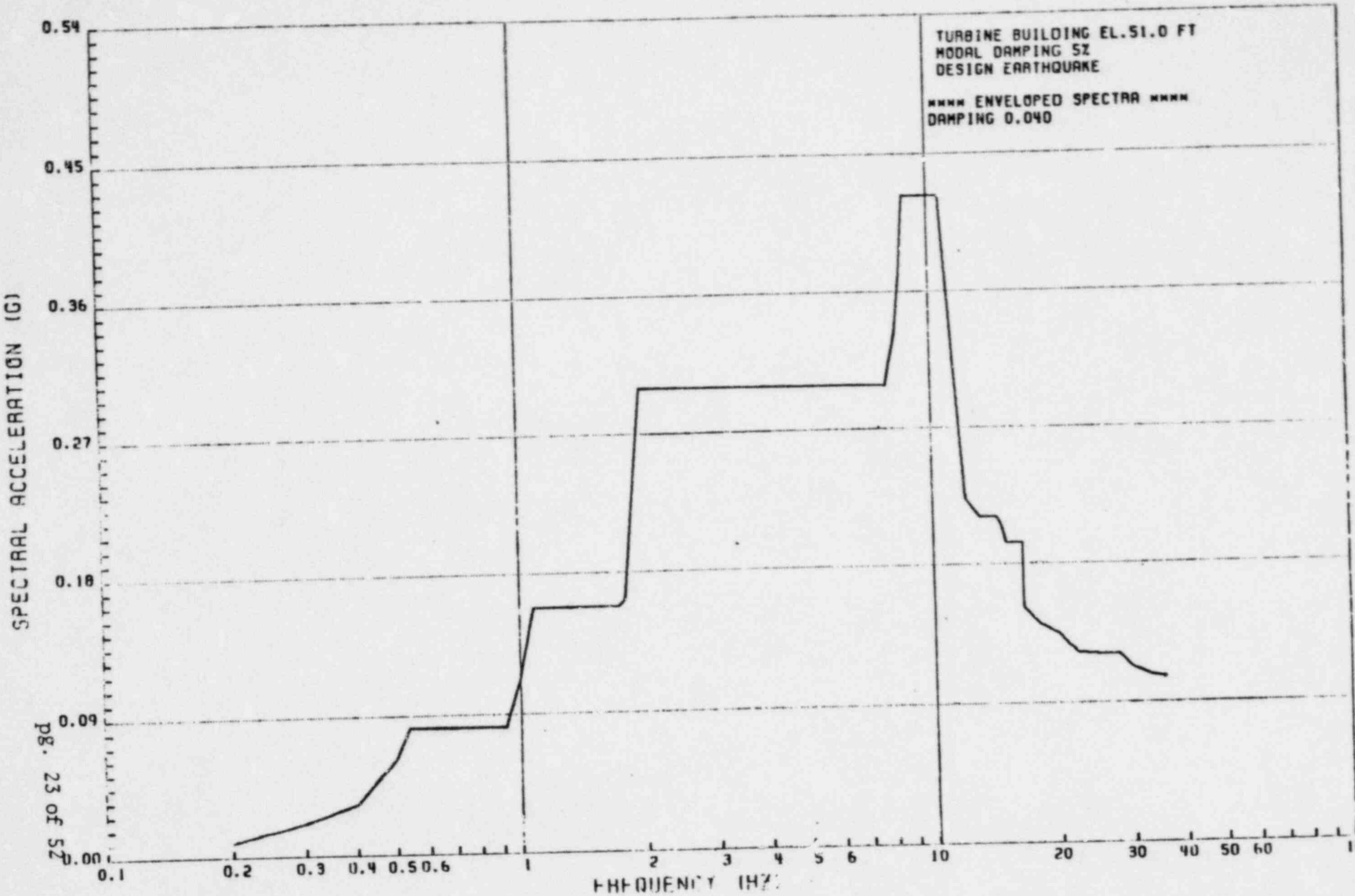
EARTHQUAKE ENGINEERING SYSTEMS

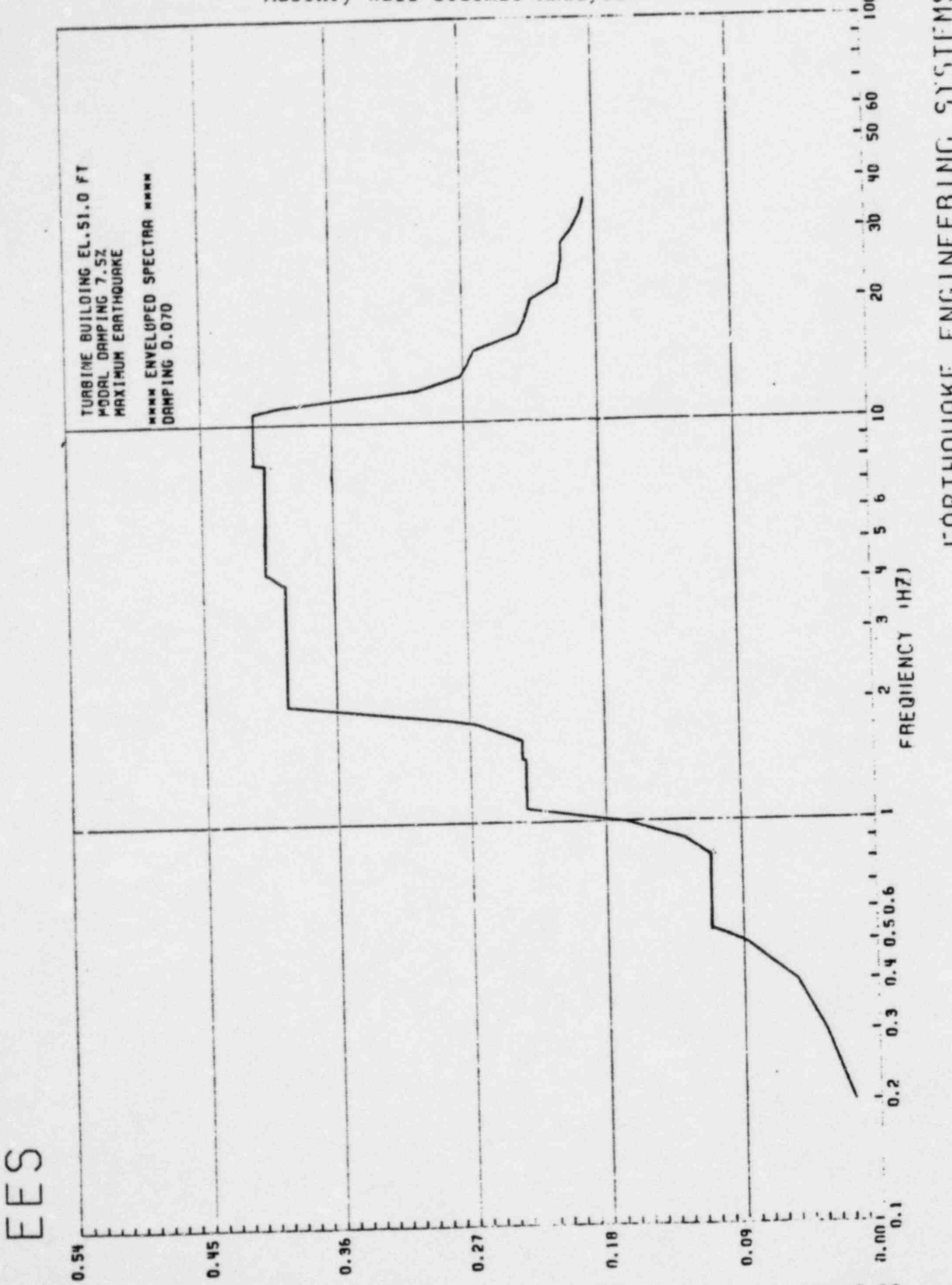
EES



Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

EARTHQUAKE ENGINEERING SYSTEMS

Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis



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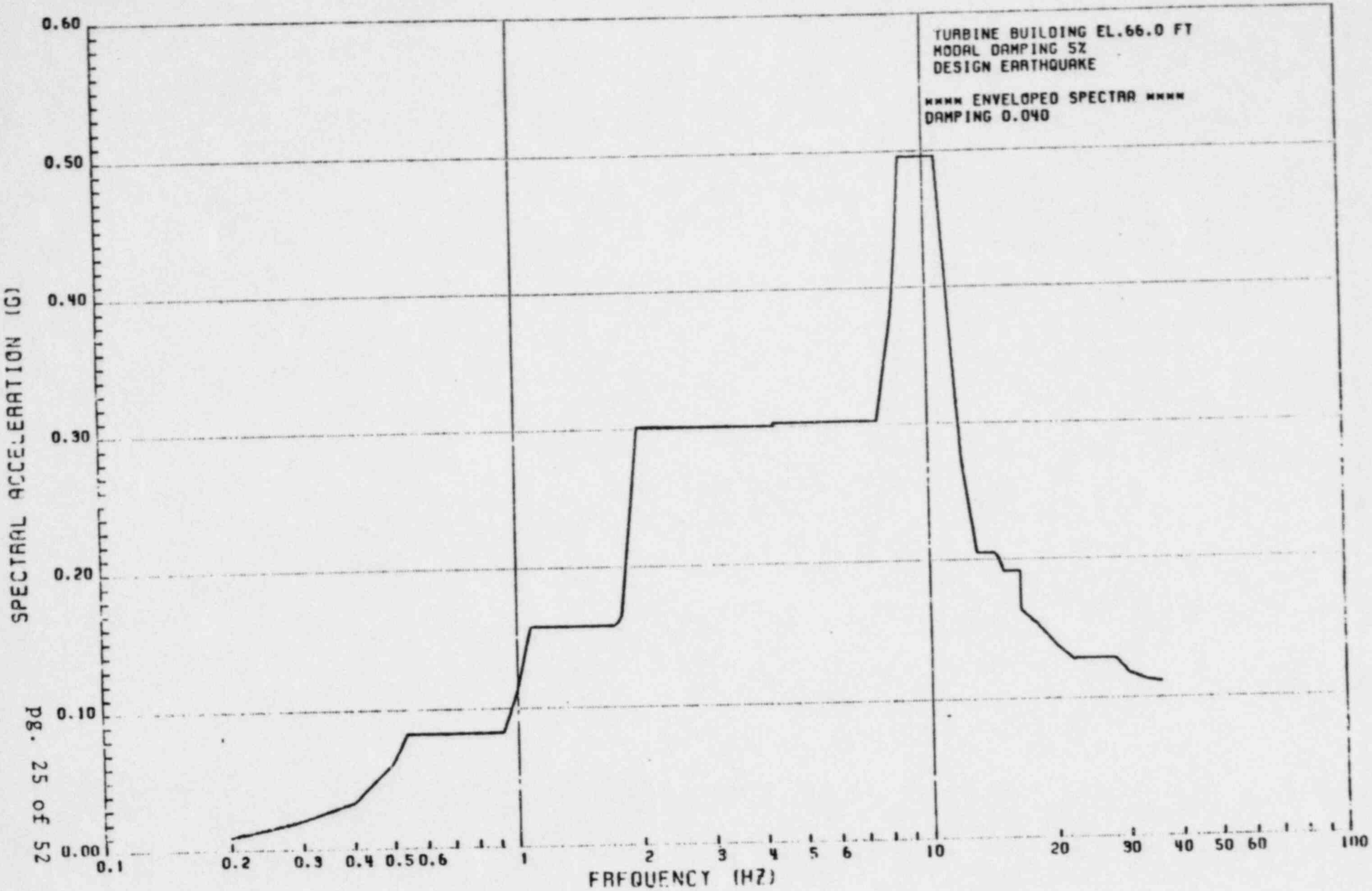

Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

EARTHQUAKE ENGINEERING SYSTEMS

Attachment 2

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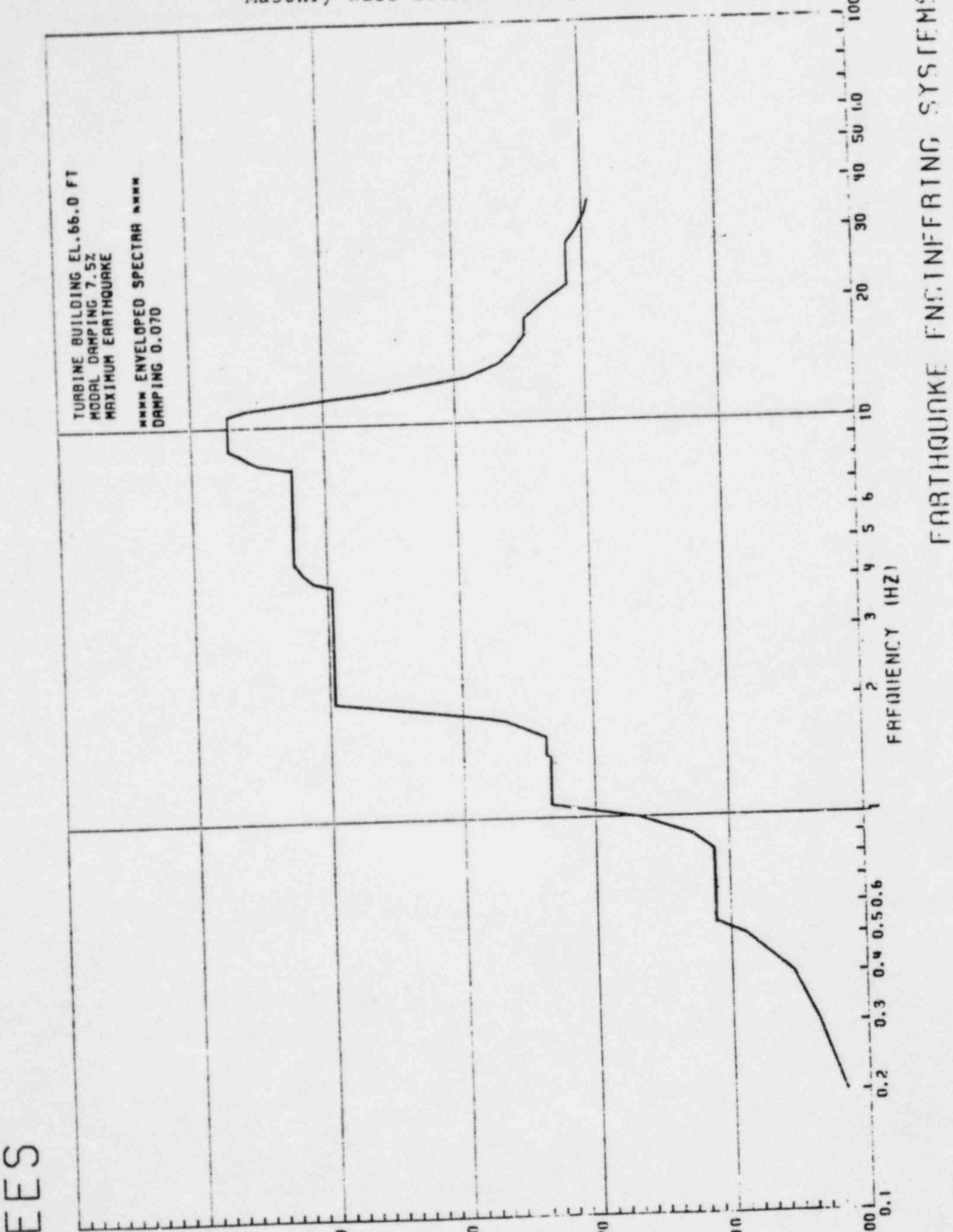
Masonry Wall Seismic Analysis



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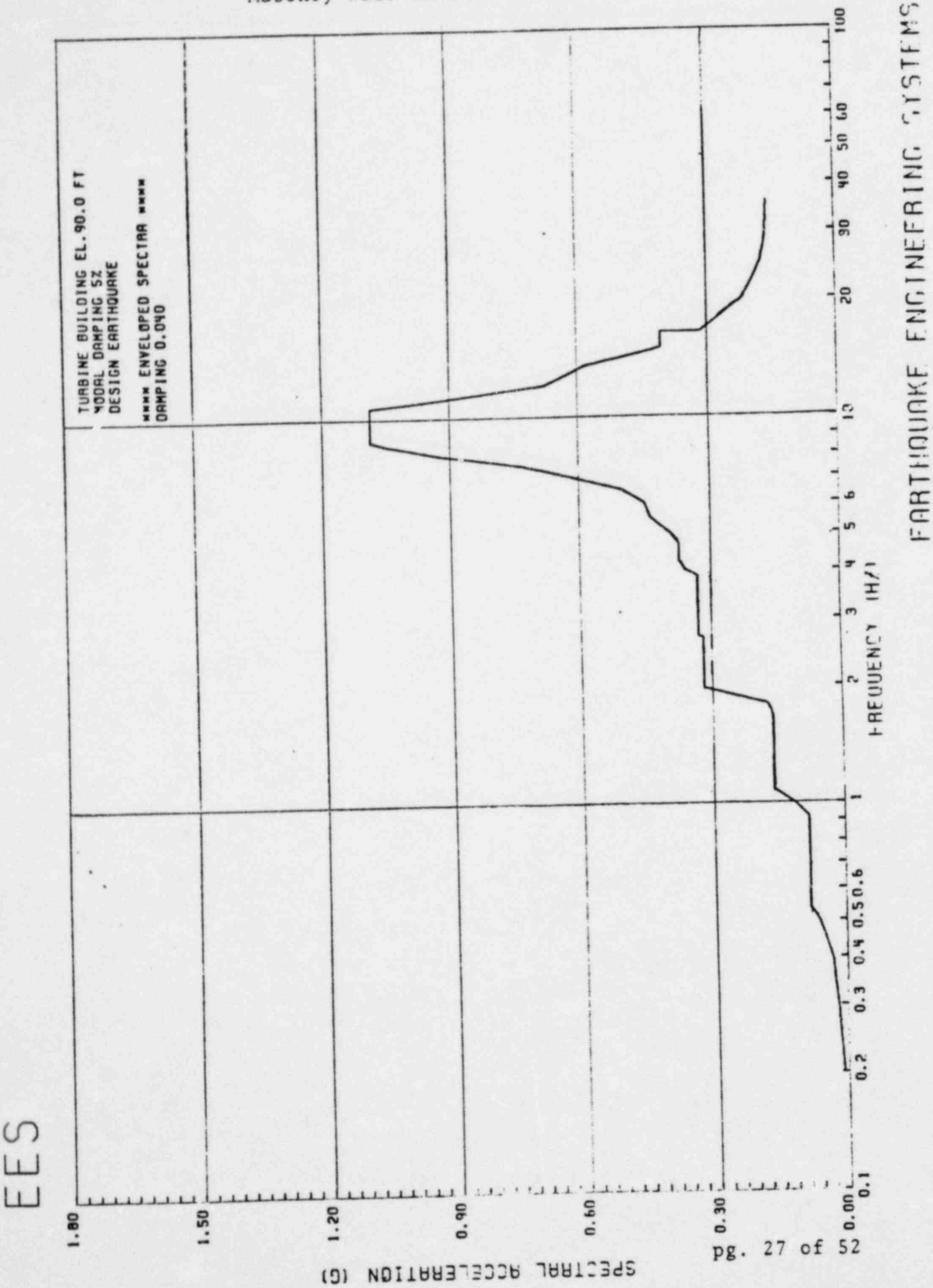
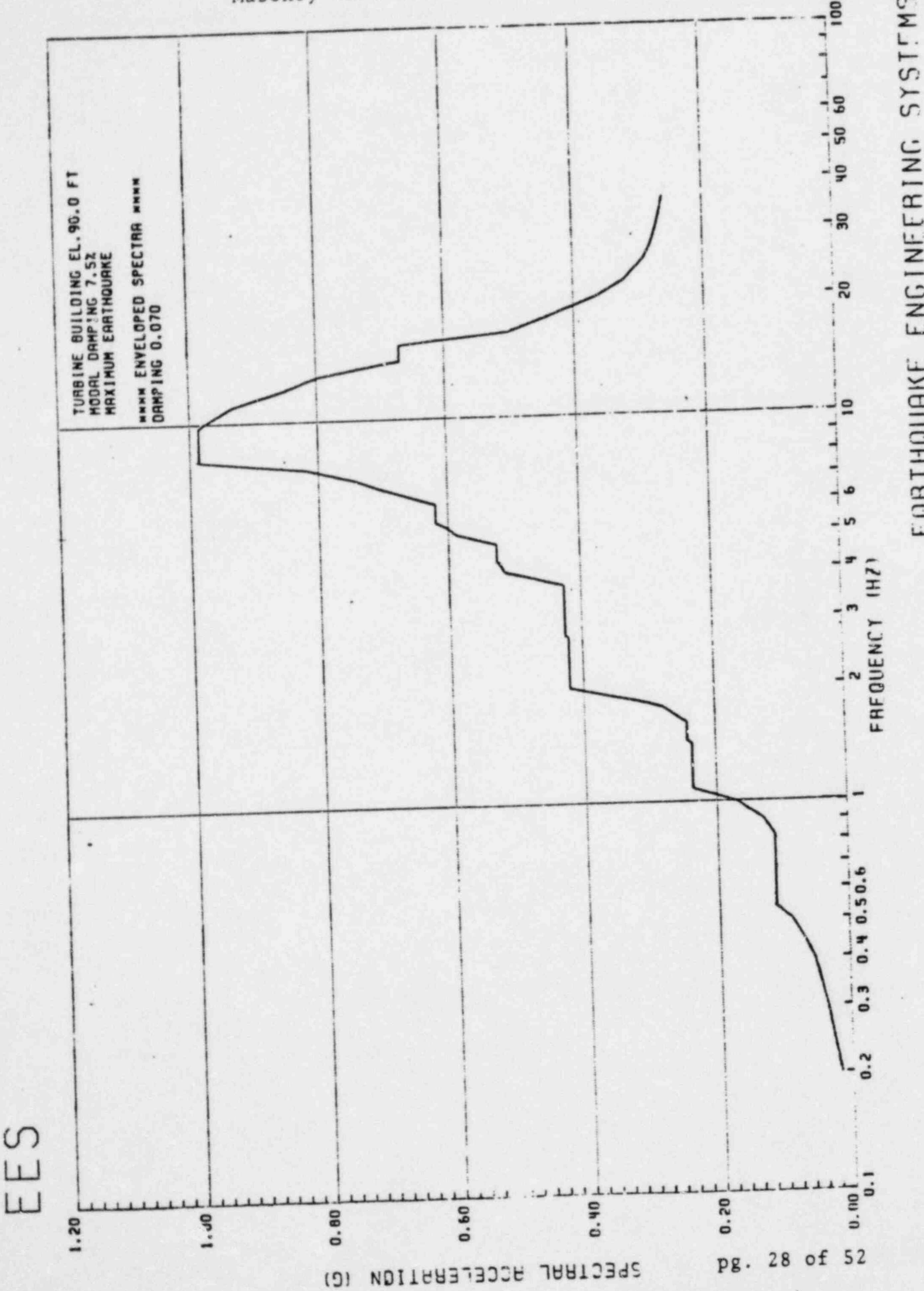
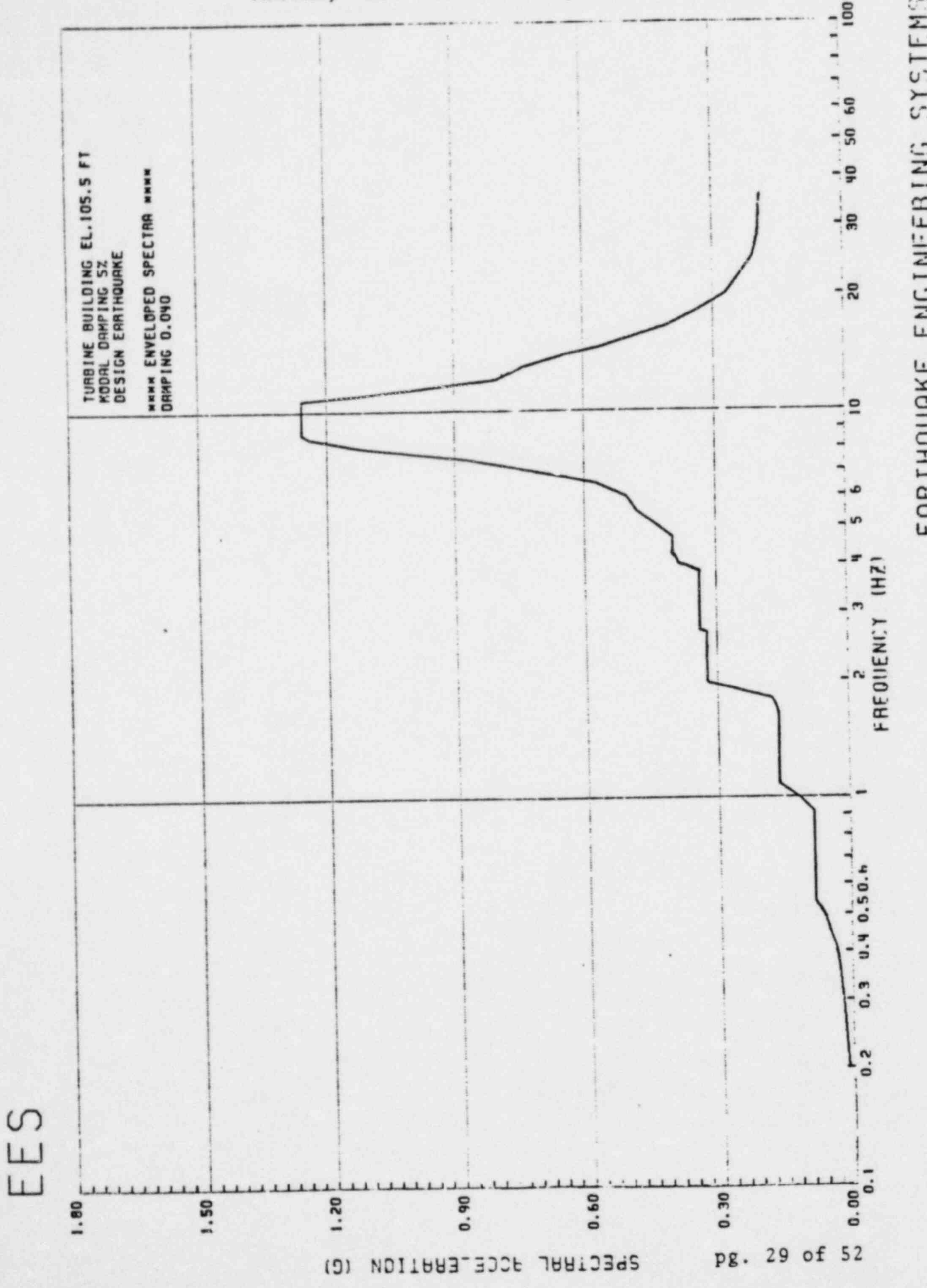
Response Spectra for Level 2
Masonry Wall Seismic Analysis



Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis



Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis



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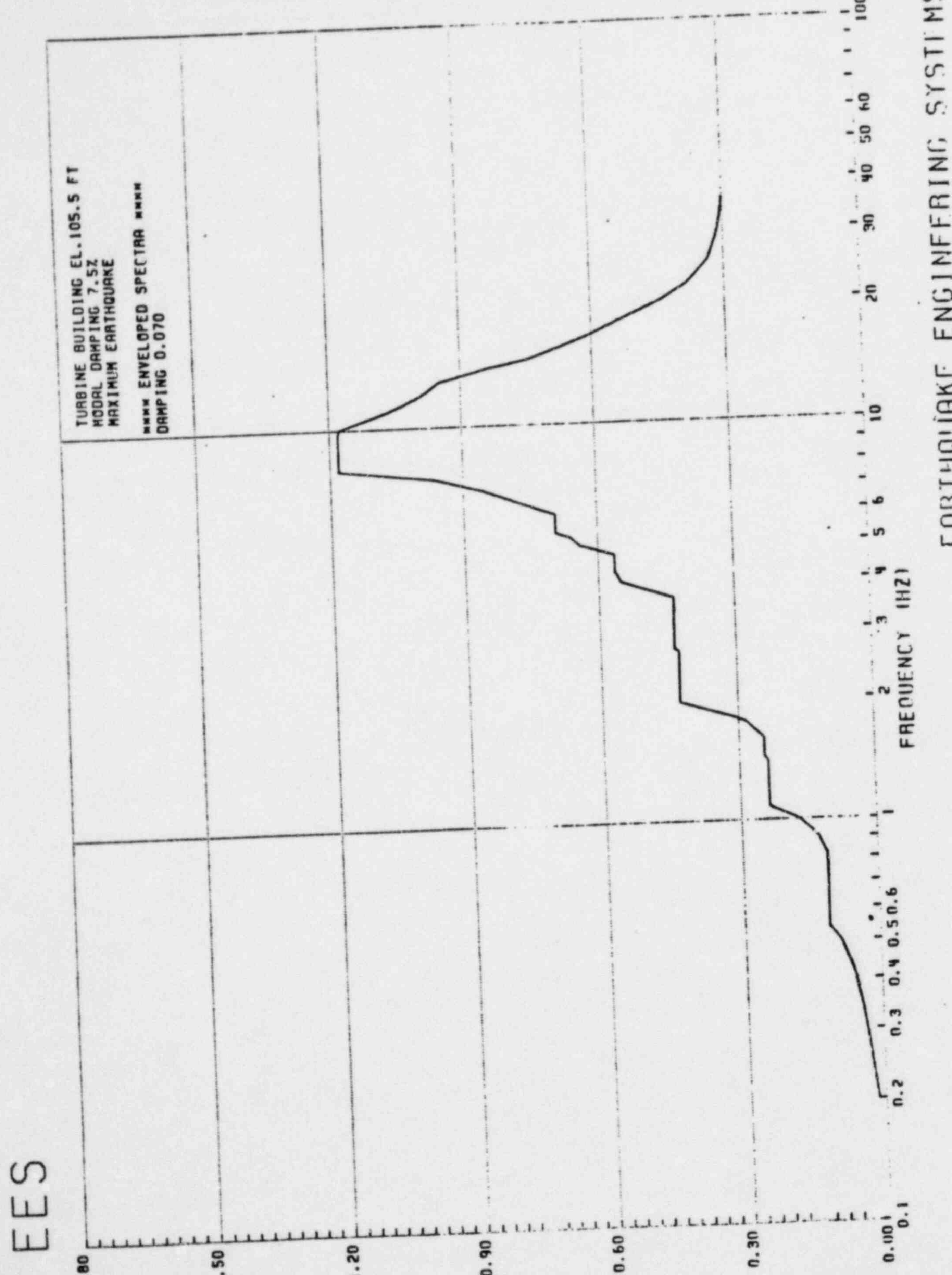
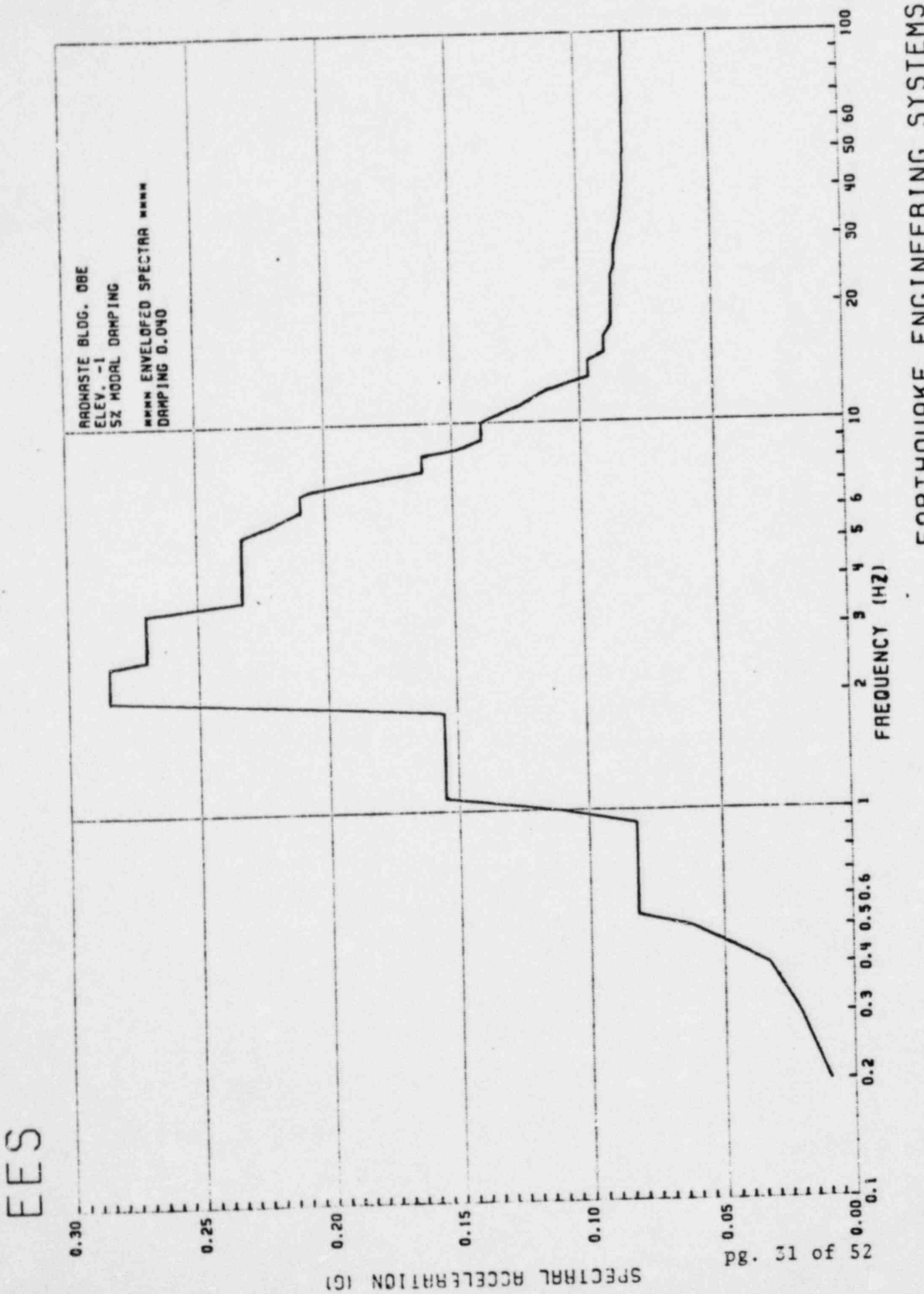
Response Spectra for Level 2
Masonry Wall Seismic Analysis



Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis



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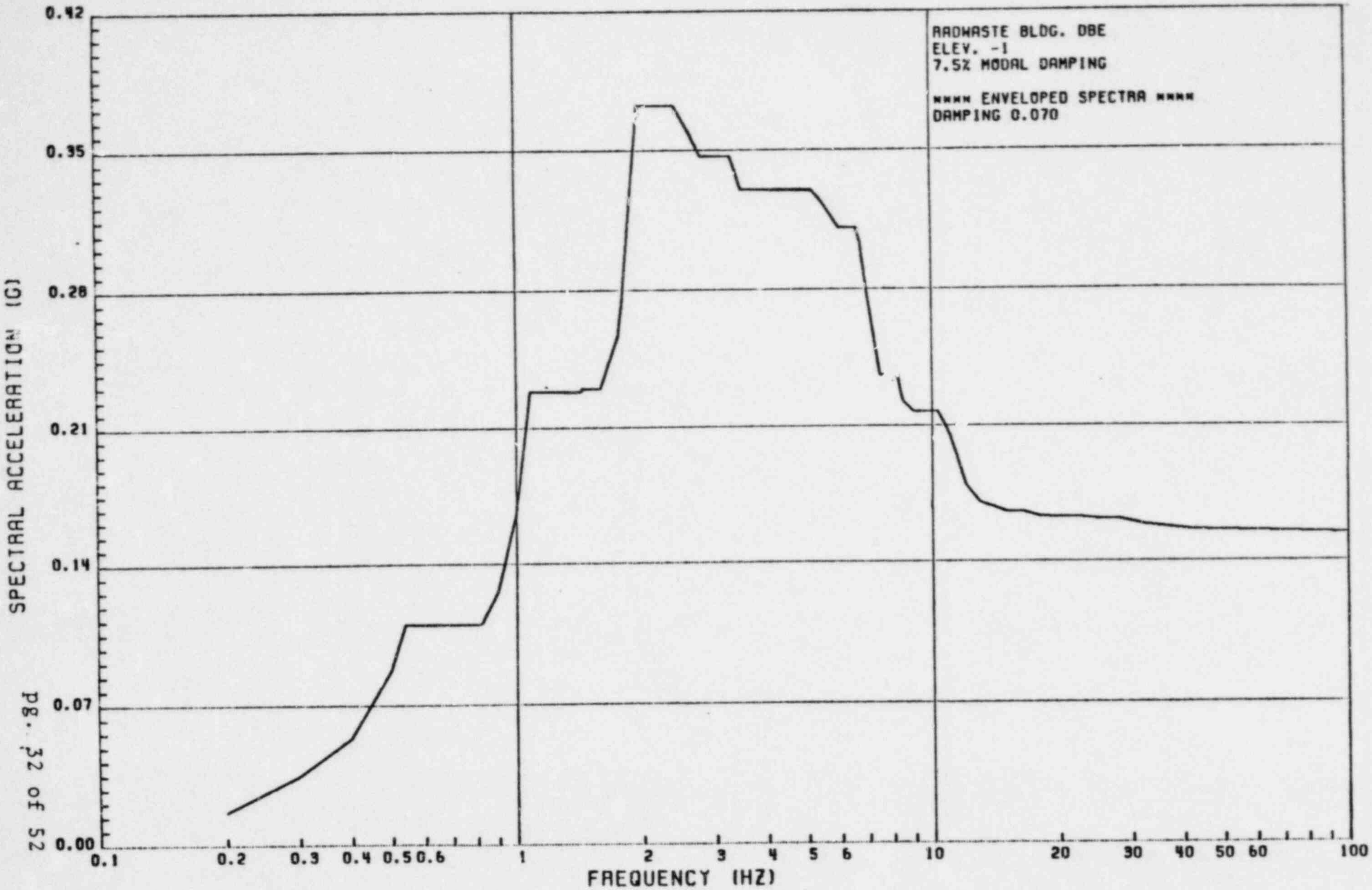
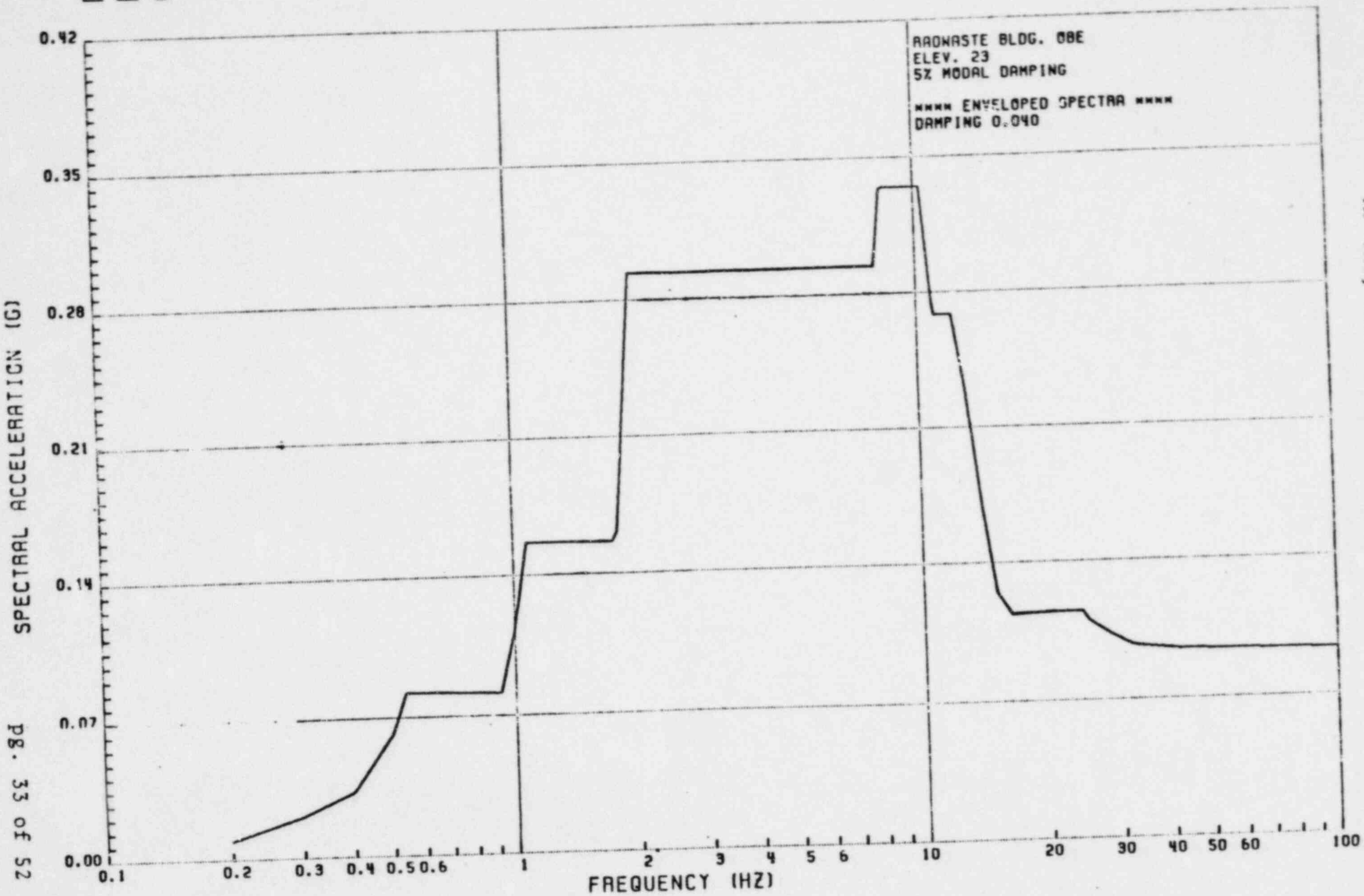


EARTHQUAKE ENGINEERING SYSTEMS

Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

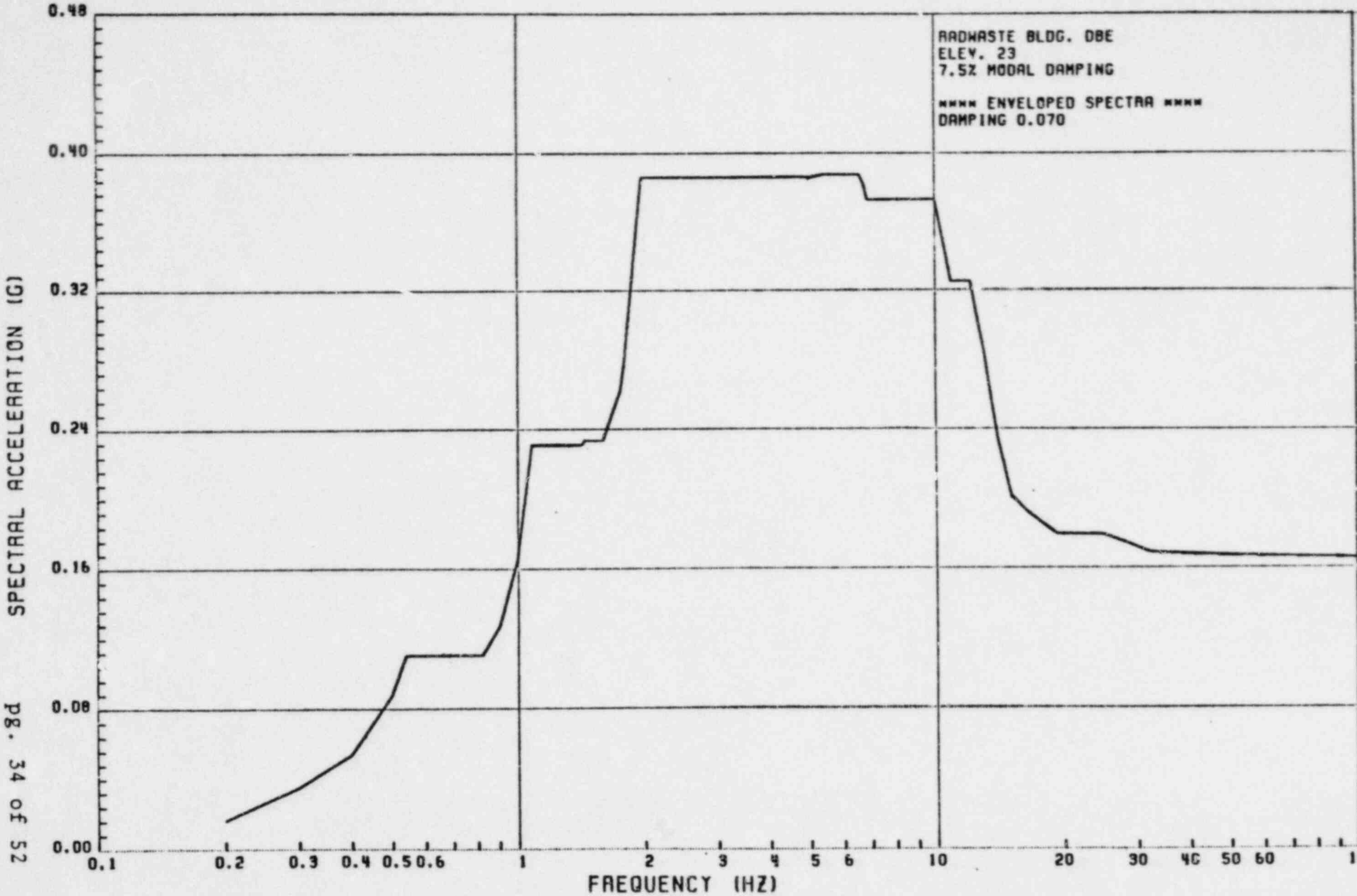
EES



EARTHQUAKE ENGINEERING SYSTEMS

Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

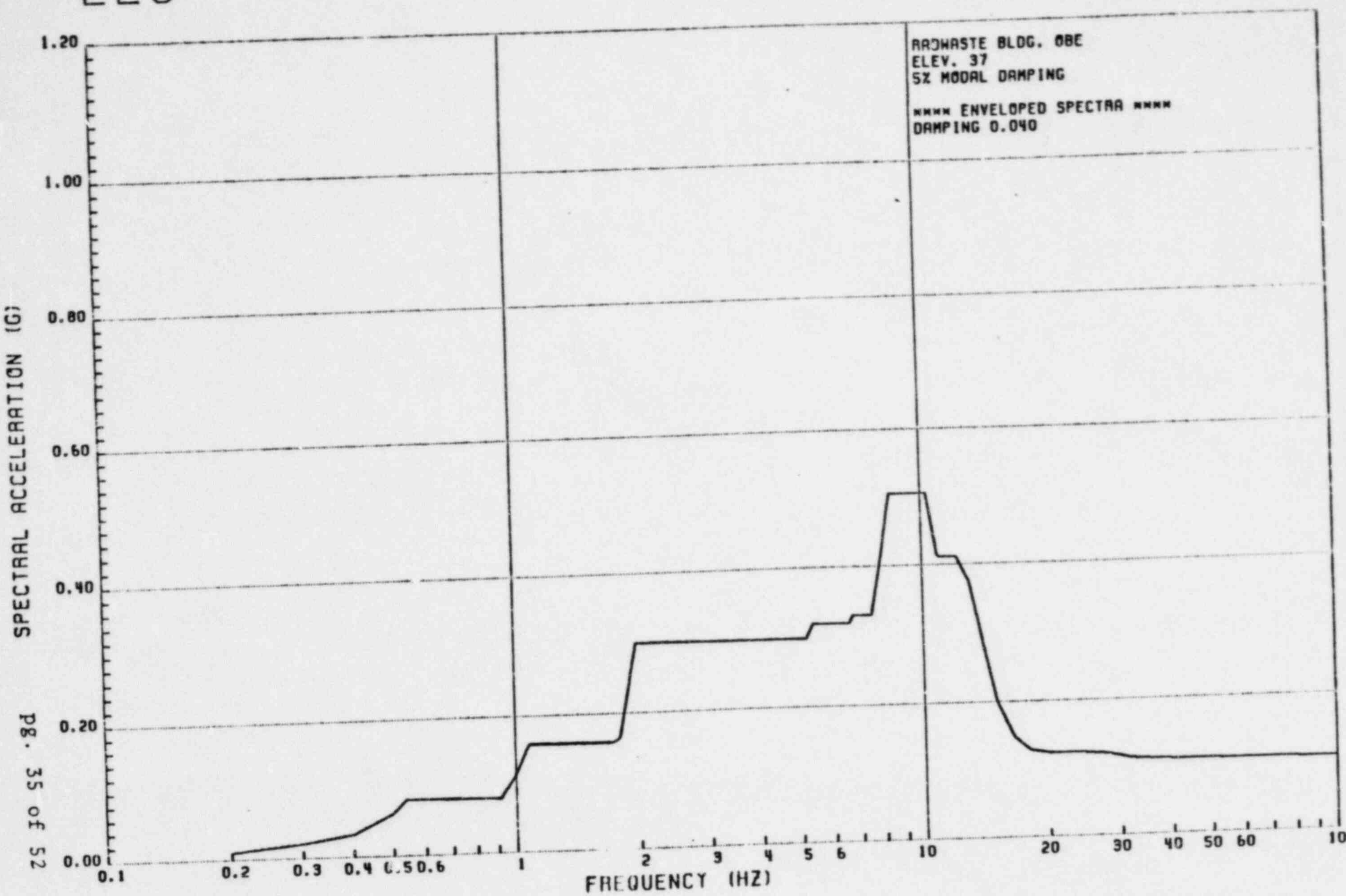
EES



EARTHQUAKE ENGINEERING SYSTEMS

Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

EES



Attachment 2
Response Spectra for Level 2
Masonry Wall Seismic Analysis

EARTHQUAKE ENGINEERING SYSTEMS

memorandum

to: Project Personnel
from: J. D. McWilliam
subject: Differential Floor Displacements
date: September 1, 1981
job no: 80034
copies: P. Baughman
C. DiNunzio
Project File
Central File

Attached are the differential floor displacements for all applicable buildings at Pilgrim I. These displacements shall be used in evaluating masonry walls for the effects of relative interstory drift.

Please refer to DC-1 and WI-2 for further instruction.

J. D. McWilliam
J. D. McWilliam
Project Engineer

JDM/jp
attachment

EES
600 ATLANTIC AVENUE • BOSTON, MASSACHUSETTS 02210 • (617) 973-9671

Reactor Building

Story Displacements

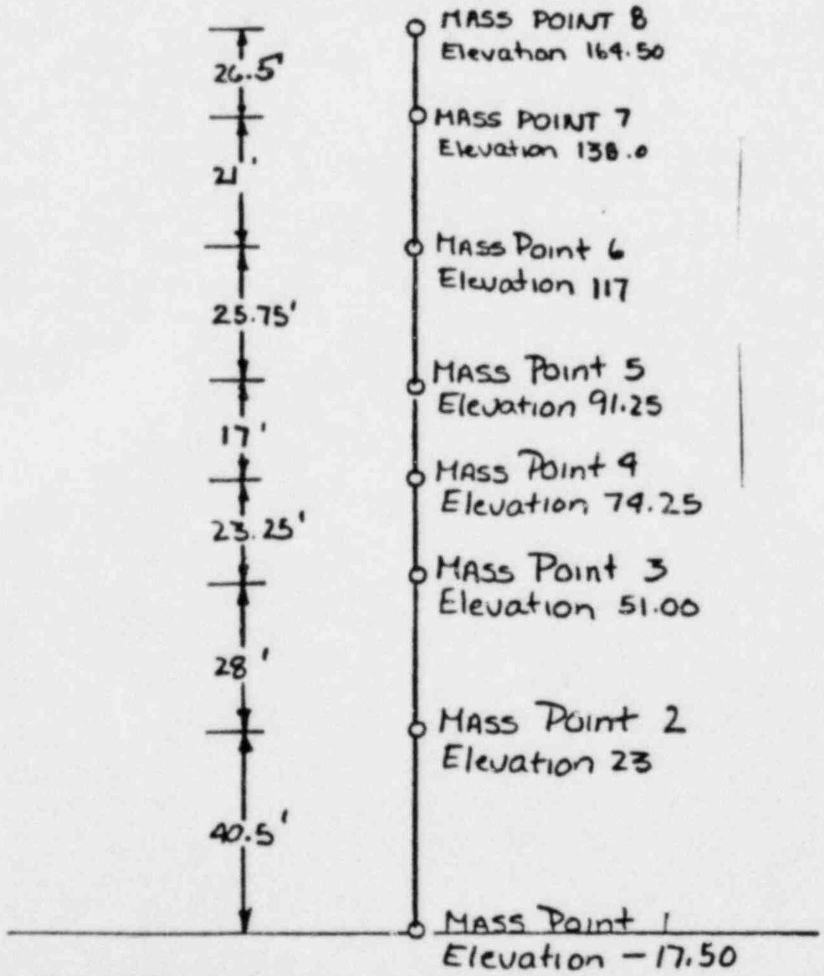
Earthquake Direction	Floor El. (ft)	Story Hgt. (H - in)	Design Earthquake (E): Floor Disp. (in)	Design Earthquake (E): Story Disp (Δ_d - in)	Design Earthquake (E): $\frac{\Delta_d}{H}$	Maximum Earthquake (E'): Floor Disp. (in)	Maximum Earthquake (E'): Story Disp. (Δ_d - in)	Maximum Earthquake (E'): $\frac{\Delta_d}{H}$
North-South	-17.5		.0134			.0242		
		486.		.0145	3.0×10^{-5}		.0258	5.3×10^{-5}
	23.0		.0279			.0500		
		336.		.0192	5.7×10^{-5}		.0339	$10. \times 10^{-5}$
	51.0		.0471			.0839		
		279.		.0172	6.2×10^{-5}		.0302	$11. \times 10^{-5}$
	74.25		.0643			.1141		
		204.		.0092	4.5×10^{-5}		.0161	7.9×10^{-5}
	91.25		.0735			.1302		
		309.		.0192	6.2×10^{-5}		.0344	$11. \times 10^{-5}$
	117.0		.0927			.1646		
		252.		.0253	$10. \times 10^{-6}$		.0459	$18. \times 10^{-6}$
	138.0		.118			.2105		
		318.		.032	$10. \times 10^{-6}$		.0472	$15. \times 10^{-5}$
	164.5		.15			.2577		
East-West	-17.5		.0133			.0183		
		486.		.0173	3.6×10^{-5}		.0248	5.1×10^{-5}
	23.0		.0306			.0431		
		336.		.0163	4.9×10^{-5}		.0367	$11. \times 10^{-5}$
	51.0		.0469			.0798		
		279.		.0143	5.1×10^{-5}		.0294	$11. \times 10^{-5}$
	74.25		.0612			.1092		
		204.		.0092	4.5×10^{-5}		.0174	8.5×10^{-5}
	91.25		.0704			.1266		
		309.		.0151	4.9×10^{-5}		.0236	7.6×10^{-5}
	117.0		.0855			.1502		
		252.		.0168	6.7×10^{-5}		.0226	9.0×10^{-6}
	138.0		.1023			.1728		
		318.		.0185	6.8×10^{-5}		.0279	8.8×10^{-5}
	164.5		.1208			.2007		

EES

PROJECT MEMO NO. 7
80034
PILGRIM NUCLEAR POWER STATION
BOSTON EDISON COMPANY

REACTOR BUILDING
N-S/E-W HORIZONTAL STORY DISPLACEMENT ENVELOPE

POINT #	DESIGN EARTHQUAKE RELATIVE FLOOR DISPL (Δ)	Δ/H $\left(\frac{IN}{FT}\right)$*	MAXIMUM EARTHQUAKE RELATIVE FLOOR DISP (Δ)	Δ/H $\left(\frac{IN}{FT}\right)$*
1-2	.0173	4.27×10^{-4}	.0258	6.37×10^{-4}
2-3	.0192	6.86×10^{-4}	.0367	1.31×10^{-3}
3-4	.0172	$7.4 \; 10^{-4}$	.0302	1.30×10^{-3}
4-5	.0092	5.41×10^{-4}	.0179	1.02×10^{-3}
5-6	.0192	7.46×10^{-4}	.0344	1.34×10^{-3}
6-7	.0253	1.2×10^{-3}	.0459	2.19×10^{-3}
7-8	.032	1.19×10^{-3}	.0472	1.76×10^{-3}



TURBINE BLDG

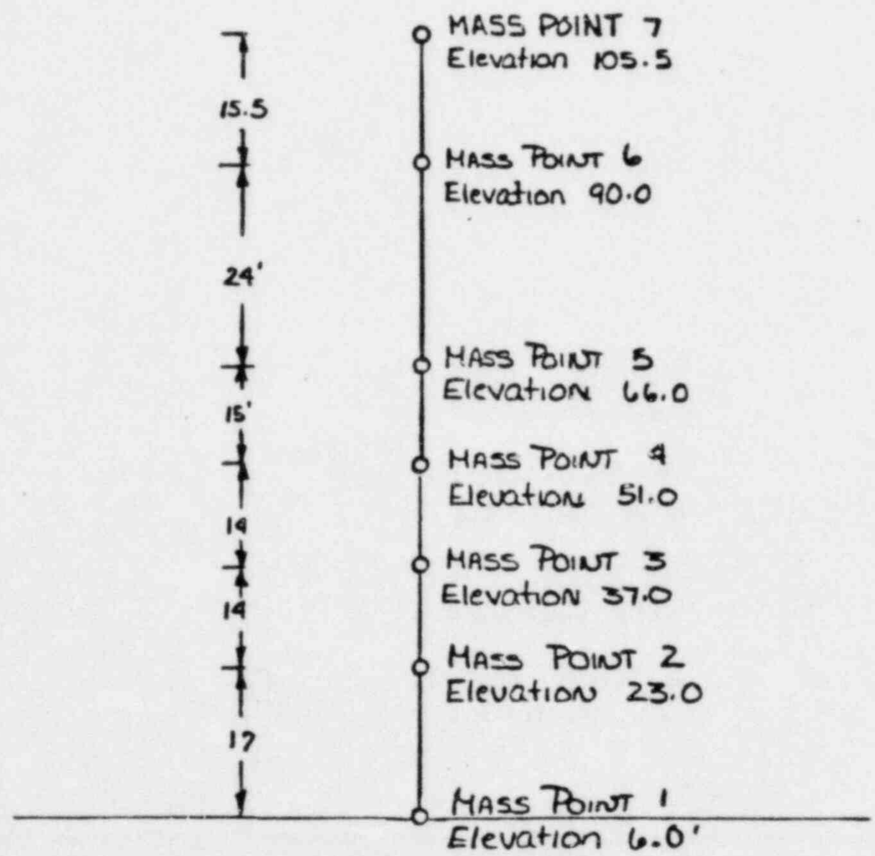
STORY DISPLACEMENTS

		Design Earthquake			Maximum Earthquake		
	Point #	Displ.	Relative Floor Disp (Δ_j)	Δ_d/H	Disp.	Relative Floor Disp. (Δ_j)	Δ/H
North South	1	.0005	.0021	1.24×10^{-4}	.00092	.00386	2.27×10^{-4}
	2	.0026	.0018	1.29×10^{-4}	.00478	.00321	2.29×10^{-4}
	3	.0044	.0019	1.36×10^{-4}	.00799	.00331	2.36×10^{-4}
	4	.0063	.0021	1.4×10^{-4}	.0113	.0037	2.47×10^{-4}
	5	.0084	.0065	2.7×10^{-4}	.0150	.0116	4.83×10^{-4}
	6	.0149	.0030	1.94×10^{-4}	.0266	.0055	3.55×10^{-4}
	7	.0179			.0321		
East West	1	.0005	.0030	1.76×10^{-4}	.00093	.00559	3.29×10^{-4}
	2	.0035	.0024	1.71×10^{-4}	.00652	.00458	3.27×10^{-4}
	3	.0059	.0025	1.79×10^{-4}	.0111	.0048	3.43×10^{-4}
	4	.0089	.0026	1.73×10^{-4}	.0159	.0048	3.2×10^{-4}
	5	.0110	.0054	2.25×10^{-4}	.0207	.0102	4.25×10^{-4}
	6	.0164	.0031	2.0×10^{-4}	.0309	.0057	3.68×10^{-4}
	7	.0195			.0366		

TURBINE BLDG

E-W/N-S HORIZONTAL STORY DISPLACEMENT ENVELOPE

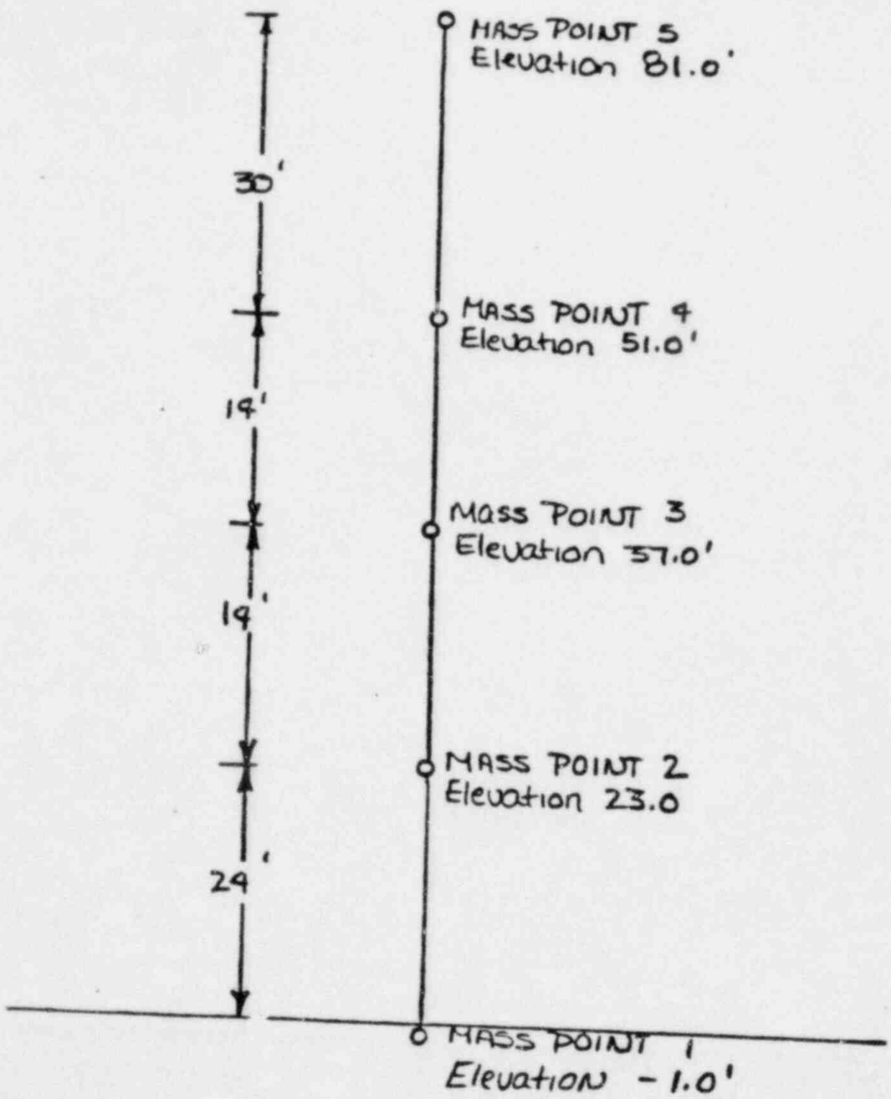
	DESIGN EARTHQUAKE		MAXIMUM EARTHQUAKE	
POINT #	RELATIVE FLOOR DISP Δ	Δ/H	RELATIVE FLOOR DISP Δ	Δ/H
1-2	.0030	1.76×10^{-4}	.00559	3.29×10^{-4}
2-3	.0024	1.71×10^{-4}	.00458	3.27×10^{-4}
3-4	.0025	1.79×10^{-4}	.0048	3.43×10^{-4}
4-5	.0026	1.72×10^{-4}	.0048	3.2×10^{-4}
5-6	.0065	2.7×10^{-4}	.0116	4.83×10^{-4}
6-7	.0031	2.0×10^{-4}	.0057	3.68×10^{-4}



RADWASTE BLDG

E-W / N-S HORIZONTAL STORY DISPLACEMENT ENVELOPE

POINT #	DESIGN EARTHQUAKE: RELATIVE FLOOR DISP (Δ_d)	DESIGN EARTHQUAKE: Δ_d/H	MAXIMUM EARTHQUAKE: RELATIVE FLOOR DISP (Δ_d)	MAXIMUM EARTHQUAKE: Δ_d/H
1-2	.000467	1.95×10^{-5}	.000849	3.52×10^{-5}
2-3	.000270	1.93×10^{-5}	.00098	3.93×10^{-5}
3-4	.000253	1.81×10^{-5}	.00045	3.21×10^{-4}
4-5	.00059	1.97×10^{-5}	.00107	3.57×10^{-5}



EES

PROJECT MEMO NO. 7
80034
PILGRIM NUCLEAR POWER STATION
BOSTON EDISON COMPANY

RADWASTE BUILDING
STORY DISPLACEMENTS

		DESIGN EARTHQUAKE			MAXIMUM EARTHQUAKE		
	POINT #	DISPL	RELATIVE FLOOR DISPL (Δ_d)	Δ_d/H	DISPL.	RELATIVE FLOOR DISPL (Δ)	Δ_d/H
NORTH SOUTH	1	.000144	.000273	1.14×10^{-5}	.000264	.000495	2.1×10^{-5}
	2	.000417	.0002	1.43×10^{-5}	.000759	.000361	2.58×10^{-5}
	3	.000617	.000166	1.19×10^{-5}	.00112	.0003	2.19×10^{-5}
	4	.000783	.000467	1.56×10^{-5}	.00142	.00081	2.7×10^{-5}
	5	.00125			.00223		
EAST WEST	1	.000130	.000467	1.95×10^{-5}	.000236	.000849	3.52×10^{-5}
	2	.000597	.000270	1.93×10^{-5}	.00108	.00048	3.43×10^{-5}
	3	.000867	.000253	1.81×10^{-5}	.00156	.00045	3.21×10^{-5}
	4	.00112	.00059	1.97×10^{-6}	.00201	.00107	3.57×10^{-5}
	5	.00171			.00308		

CYGNA

Memorandum

Project Memo #9
Revision #2

To: Project Personnel

Date: October 9, 1981

From: W. J. Duffy

Job No: 80034

Subject: Masonry Wall Section Properties

Copies: P. Baughman
C. DiNunzio
B. Gang (SF)
H. Reeser (SD)
P. DiDonato (SF)
M. DeGuzman (SF)
Project File
Central File

Attached are the masonry wall section properties, cracking moments, and allowable moments that are to be used in Level 1 and Level 2 analysis of masonry walls. Attachments 1 and 2 describe the properties and moments for Level 1 analysis, while Attachments 3 and 4 cover Level 2 analysis.

This revision is issued to bring this memo into compliance with DC-1, Rev. 1.

Refer to DC-1 and WI-1, WI-2, or WI-3 for further instructions in the use of these tables.

W. J. Duffy

W. J. Duffy
Assistant Project Engineer

JDM/jp
attachments

ATTACHMENT 1

Single Wythe Masonry Wall Section Properties

Level 1 Analysis

1a: Definition of Reinforcement Cases

Vertical

Case A: Shield Wall with 1#5 bar
Case B: Shield Wall with 2#5 bars
Case C: Partition Wall with 1#5 bar
Case D: Partition Wall with 2#5 bars

Horizontal

Case E*: Sheild Walls with no reinforcement
Case E**: Partition Wall with no reinforcement
Case F*: Shield Wall with Bond Beam
Case F**: Partition Wall with Bond Beam
Case E*F*: Shield Wall
Case E**F**: Partition Wall

1b: Wall Mass Density and Equivalent Static Pressure

Reinforcement Case	Wall Thickness (in)	Actual Wall Thickness (in)	Equivalent Static Pressure+ (lb/in^2)	Mass Density ($lb\text{-}sec^2/in^4$)
A	8	7.625	.579	1.872×10^{-4}
	12	11.625	.868	1.872×10^{-4}
B	8	7.625	.579	1.872×10^{-4}
C	8	7.625	.428	1.383×10^{-4}
	12	11.625	.567	1.222×10^{-4}
D	8	7.625	.428	1.383×10^{-4}

+ Due to a 1.0 g horizontal acceleration

1c: Material Constants

E (masonry) = 600 f'm = 810,000 psi
E (masonry) = 337,500 psi

1d: Moments of Inertia

Cracked Section

Reinforcement Case	Thickness (in)	I_{tx} (in^4/in)	I_{ty} (in^4/in)
A	8	-	4.72
	12	-	12.6
B	8	-	10.3
C	8	-	4.72
	12	-	12.5
D	8	-	10.1
E*	8	36.94	-
E**	8	25.73	-
F*	12	8.68	-
F**	12	8.63	-
E*F*	8	36.94	-
E**F**	8	25.73	-

Uncracked Section

Reinforcement Case	Thickness (in)	I_{gx} (in^4/in)	I_{gy} (in^4/in)	I_o (in^4/in)
A	8		36.94	36.94
	12		130.93	130.93
B	8		36.94	36.94
C	8		31.57	36.94
	12		105.23	130.93
D	8		31.57	36.94
E*	8	36.94		36.94
F*	12	130.93		130.93
E*F*	8	36.94		36.94
E**	8	25.73		36.94
F**	12	77.45		130.93
E**F**	8	25.73		36.94

1e: Orthotropic Section Properties

Cracked Section

Reinforcement Case Vert.	Reinforcement Case Horiz.	Wall Thick. (in)	C_{xx} (psi)	C_{xy} (psi)	C_{yy} (psi)	G_{xy} (psi)
A	E*	8	844000	60400	108000	121000
A	F*	12	55700	13500	81000	27000
B	E* F*	8	LATER	LATER	LATER	LATER
C	E**	8	588000	50300	108000	101000
C	F**	12	55700	13300	81000	26700
D	E**F**	8	LATER	LATER	LATER	LATER

Uncracked Section

Reinforcement Case Vert.	Reinforcement Case Horiz.	Wall Thick. (in)	C_{xx} (psi)	C_{xy} (psi)	C_{yy} (psi)	G_{xy} (psi)
A	E*	8	844000	169000	844000	338000
A	F*	12	844000	169000	844000	338000
B	E* F*	8	844000	169000	844000	338000
C	E**	8	588000	130000	721000	260000
C	F**	12	499000	116000	675000	232000
D	E**F**	8	588000	130000	721000	260000

1f: Allowable Moments (1)(2)

Vertical Reinforcement Case	Wall Thickness (in)	Allowable Moment (in-lb/in) Design Earthquake M_{yy}	Maximum Earthquake M_{yy}'
A	8	1230	2260
	12	1963	3534
B	8	1814	3266
C	8	1214	2277
	12	1982	3568
D	8	1826	3286

Horizontal Reinforcement Case	Wall Thickness (in)	Allowable Moment (in-lb/in) Design Earthquake M_{xx}	Maximum Earthquake M_{xx}'
E*	8	649	979
E**	8	304	452
F*	12	1317	2371
F**	12	1406	2403
E*F*, E**F**	8	Later	Later

(1) Applicable to load combinations (1), (2), and (4) if 'D' imposes only bending on the wall.

(2) Maximum earthquake limits are applicable to load combination (3) if both 'D' and 'T' impose only bending on the wall.

ATTACHMENT 2

Multi-Wythe Masonry Wall Section Properties

Level 1 Analysis

2a: Wall Mass Density and Equivalent Static Pressure

Wall Thickness (in)	Actual Wall Thickness (in)	Equivalent Static Pressure+ (lb/in^2)	Mass Density ($lb\text{-}sec^2/in^4$)
16	15.25	1.25	2.122×10^{-4}
18	17.25	1.414	2.122×10^{-4}
24	23.25	1.906	2.122×10^{-4}
30	28.875	2.367	2.122×10^{-4}
36	34.875	2.859	2.122×10^{-4}
42	40.50	3.32	2.122×10^{-4}

+ Due to a 1.0 g horizontal acceleration

2b: Orthotropic Section Properties

Wall thickness (in)	I_{tx} (in^4/in)	I_{ty} (in^4/in)	C_{xx} (psi)	C_{xy} (psi)	C_{yy} (psi)	G_{xy} (psi)
16	73.88	9.45	211000	15100	27000	30200
18	73.88	9.45	146000	10400	18600	20900
24	17.37	25.18	14000	3370	20300	6740
30	17.37	25.18	7300	1760	10600	3520
36	44.52	32.02	10600	1800	7640	3600
42	62.10	32.02	9470	1360	4880	2720

2c: Material Constants

E (masonry) = 600 f'm = 810,000 psi
G (masonry = 337,500 psi

2d: Allowable Moments (1)(2)

Wall Thickness (in)	Allowable Moment (in-lb/in) Design Earthquake M_{xx}	M_{yy}	Maximum Earthquake M_{xx}'	M_{yy}'
16	1298	2460	1958	4520
18	1298	2460	1958	4520
24	2634	3926	4742	7068
30	2634	3926	4742	7068
36	3287	5466	5916	9838
42	4814	5466	8664	9838

(1) Applicable to load combinations (1), (2), and (4) if 'D' imposes only bending on the wall.

(2) Maximum earthquake limits are applicable to load combination (3) if both 'D' and 'T' impose only bending on the wall.

ATTACHMENT 3

Single Wythe Masonry Wall Section Properties

Level 2 Analysis

3a: Definition of Reinforcement Cases

Vertical

Case A: Shield Wall with 1#5 bar
Case B: Shield Wall with 2#5 bars
Case C: Partition Wall with 1#5 bar
Case D: Partition Wall with 2#5 bars

Horizontal

Case E*: Sheild Walls with no reinforcement
Case E**: Partition Wall with no reinforcement
Case F*: Shield Wall with Bond Beam
Case F**: Partition Wall with Bond Beam
Case E*F*: Shield Wall
Case E**F**: Partition Wall

3b: Wall Mass Density and Equivalent Static Pressure

Reinforcement Case	Wall Thickness (in)	Actual Wall Thickness (in)	Equivalent Static Pressure+ (lb/in^2)	Mass Density ($lb\text{-}sec^2/in^4$)
A	8	7.625	.579	1.872×10^{-4}
	12	11.625	.868	1.872×10^{-4}
B	8	7.625	.579	1.872×10^{-4}
C	8	7.625	.428	1.383×10^{-4}
	12	11.625	.567	1.222×10^{-4}
D	8	7.625	.428	1.383×10^{-4}

+ Due to a 1.0 g horizontal acceleration

3c: Material Constants

E (masonry) = 1000 f'm = 1,350,000 psi
E (masonry) = 562,500 psi

ATTACHMENT 3 (con't)

3d: Moments of Inertia

Cracked Section

Reinforcement Case	Thickness (in)	I_{tx} (in^4/in)	I_{ty} (in^4/in)
A	8	-	3.33
	12	-	8.64
B	8	-	7.19
C	8	-	3.34
	12	-	8.58
D	8	-	7.16
E*	8	0.00	-
E**	8	0.00	-
F*	12	5.92	-
F**	12	5.92	-
E*F*	8	(Later)	-
E**F**	8	(Later)	-

Uncracked Section

Reinforcement Case	Thickness (in)	I_{gx} (in^4/in)	I_{gy} (in^4/in)	I_o (in^4/in)
A	8		36.94	36.94
	12		130.93	130.93
B	8		36.94	36.94
C	8		31.57	36.94
	12		105.23	130.93
D	8		31.57	36.94
E*	8	36.94		36.94
F*	12	130.93		130.93
E*F*	8	36.94		36.94
E**	8	25.73		36.94
F**	12	77.45		130.93
E**F**	8	25.73		36.94

ATTACHMENT 3 (con't)

3e: Cracking Moments

Reinforcement Case	Wall Thickness (in)	M_{crx} (in-lb/in)	M_{cry} (in-lb/in)
A	8	-	659
	12	-	1531
B	8	-	659
C	8	-	563
	12	-	1231
D	8	-	563
E*	8	1299	-
E**	8	904	-
F*	12	3018	-
F**	12	1785	-

3f: Allowable Moments (1)(2)

Vertical Reinforcement Case	Wall Thickness (in)	Allowable Moment (in.-lb/in) Design Earthquake M_{yy}	Maximum Earthquake M_{yy}'
A	8	1051	2103
	12	2011	3620
B	8	1760	3453
C	8	1056	2112
	12	2002	3603
D	8	1724	3448

Horizontal Reinforcement Case	Wall Thickness (in)	Allowable Moment (in-lb/in) Design Earthquake M_{xx}	Maximum Earthquake M_{xx}'
E*	8	649	979
E**	8	304	452
F*	12	1350	2430
F**	12	1359	2447
E*F*, E**F**	8	Later	Later

(1) Applicable to load combinations (1), (2), and (4) if 'D' imposes only bending on the wall.

(2) Maximum earthquake limits are applicable to load combination (3) if both 'D' and 'T' impose only bending on the wall.

ATTACHMENT 4

Multi-Wythe Masonry Wall Section Properties

Level 2 Analysis

4a: Wall Mass Density and Equivalent Static Pressure

Wall Thickness (in)	Actual Wall Thickness (in)	Equivalent Static Pressure+ (lb/in^2)	Mass Density ($lb\text{-}sec^2/in^4$)
16	15.25	1.25	2.122×10^{-4}
18	17.25	1.414	2.122×10^{-4}
24	23.25	1.906	2.122×10^{-4}
30	28.875	2.367	2.122×10^{-4}
36	34.875	2.859	2.122×10^{-4}
42	40.50	3.32	2.122x10-4

+ Due to a 1.0 g horizontal acceleration

4b: Moments Of Inertia

Wall Thickness (in)	Uncracked Section I_{Gx} (in^4/in)	Uncracked Section I_{Gy} (in^4/in)	Cracked Section I_{Tx} (in^4/in)	Cracked Section I_{Ty} (in^4/in)
16	73.88	73.88	0.00	6.66
18	73.88	73.88	0.00	6.66
24	261.86	261.86	11.68	17.28
30	261.86	261.86	11.68	17.28
36	261.86	261.86	27.65	22.47
42	261.86	261.86	38.83	22.47

4c: Material Constants

E (masonry) = 1000 f'm = 1,350,000 psi
G (masonry) = 562,500 psi

4d: Cracking Moments

Wall Thickness (in)	M_{crx} (in-lb/in)	M_{cry} (in-lb/in)
16	2598	1318
18	2598	1318
24	6036	3062
30	6036	3062
36	6036	3062
42	6036	3062

4e: Allowable Moments (1)(2)

Wall Thickness (in)	Allowable Moment (in-lb/in) Design Earthquake M_{xx}	Design Earthquake M_{yy}	Maximum Earthquake M_{xx}'	Maximum Earthquake M_{yy}
16	1298	2102	1958	4206
18	1298	2102	1958	4206
24	2700	4022	4800	7240
30	2700	4022	4800	7240
36	3322	4783	5980	9555
42	4878	4783	8782	9555

(1) Applicable to load combinations (1), (2), and (4) if 'D' imposes only bending on the wall.

(2) Maximum earthquake limits are applicable to load combination (3) if both 'D' and 'T' impose only bending on the wall.

memorandum

to: Project Personnel

from: J. D. McWilliam

subject: Weights of Equipment on Masonry Walls

date: September 1, 1981

job no: 80034

copies: P. Baughman
C. DiNunzio
Project File
Central File

Attached are the weights of attached components that shall be used in the analysis of masonry walls. Attachment 1 lists the piping weights, Attachment 2 lists the conduit weights, and Attachment 3 lists the weights of miscellaneous equipment.

Refer to DC-1 and WI-1, WI-2 and WI-3 for further instruction in applying these loads.

J. D. McWilliam
Project Engineer

JDM/jp
attachments

EES

600 ATLANTIC AVENUE • BOSTON, MASSACHUSETTS 02210 • (617) 973-9671

ATTACHMENT 1

PIPING

ATTACHED COMPONENT/EQUIPMENT WEIGHT

PIPE NOM. SIZE (O.D., IN.)	STEEL FILLED WITH WATER (LBS/FT)	COPPER FILLED WITH WATER (LBS/FT)
1/4	0.57	0.49
3/8	0.8	0.7
1/2	1.2	1.1
3/4	1.7	1.5
1.0	2.5	2.1
1 1/4	3.6	3.3
1 1/2	4.4	4.0
2.0	6.3	5.6
2 1/2	9.5	8.0
3.0	13.1	11.7
3 1/2	16.4	15.3
4.0	20.0	18.0
5.0	28.7	24.1
6.0	39.9	30.7
80.0	63.2	51.4
10.0	95.4	77.3
12.0	132.5	100.5

ATTACHMENT 2

CONDUIT NOM. SIZE (IN.)	CONDUIT PLUS STEEL (LBS/FT)	CONDUCTOR WEIGHT ALUMINUM (LBS/FT)
3/4	1.4	0.7
1	2.1	1.0
1 1/4	3.0	1.4
1 1/2	3.6	1.8
2	5.0	2.5
2 1/2	7.9	4.1
3	11.0	6.0
4	16.5	9.5
5	24.0	14.0
6	32.5	19.5

ATTACHMENT 3

EQUIPMENT*		WEIGHT (LBS)
JUNCTION BOXES	SURFACE AREA x 5.0 LBS/FT^2	
SWITCHES		5
EMERGENCY LIGHTS		50
STEEL PLATES	PER AISC	
RECEPTACLES		15
DOOR		100
SPEAKER		25
HOSE REEL		75
FIRE PROTECTION PANELS	SURFACE AREA x 16 LBS/FT^2	
FIRE EXTINGUISHER		30
GAITRONICS BOX		20
PRESSURE GAGE		10
FUSE BOX		30
LADDER	8 LBS/FT	
UNISTRUTS	3.8 LBS/FT	
GRATING	13 LBS/FT	
LIGHTS		5
AIR TANK		10
HOT WATER HEATER	TANK WT AND WATER WT	
WIREWAYS	PER AISC	
REMOTE VALVE OPERATOR SUPPORT PLATES		50
UNIT HEATER		400
CABLE TRAYS	50 LBS/FT	

*Note: For equipment weight not specified above consult group leader

CYGNA

Memorandum

Project Memo #4
Revision #1

To: Project Personnel

Date: October 2, 1981

From: J. D. McWilliam

Job No: 80034

Subject: PBOC Loads

Copies: P. Baughman H. Reeser
C. DiNunzio B. Gang
Project File P. DiDonato
Central File M. DeGuzman

Attached are the pipe break outside containment (PBOC) loads that shall be used in the analysis of masonry walls. Please refer to DC-1 and WI-2 and WI-3 for further instruction in the application of these loads.

J. D. McWilliam
J. D. McWilliam
Project Engineer

JDM/jp
attachments

ATTACHMENT 1

DIFFERENTIAL PRESSURE LOADINGS ON SAFETY-RELATED MASONRY WALLS SUBJECT TO PIPE BREAK OUTSIDE CONTAINMENT LOADING (R')

Wall No.	Location	Peak Differential Pressure (PSI)	Equivalent Static Pressure (PSI)
62.0	R.B. El 23'-0"	0.51	0.56
62.1	R.B. El 23'-0"	0.51	0.56
62.2	R.B. El 23'-0"	13.6	20.4
62.4	R.B. El 23'-0"	0.51	0.56
62.5	R.B. El 23'-0"	0.51	0.56
62.9	R.B. El 23'-0"	0.51	0.56
62.10	R.B. El 23'-0"	0.51	0.56
62.11	R.B. El 23'-0"	0.51	0.56
62.12	R.B. El 23'-0"	13.6	20.4
62.13	R.B. El 23'-0"	0.51	0.56
63.1	R.B. El 23'-0"	0.25	0.28
63.4	R.B. El 23'-0"	2.70	2.97
63.5	R.B. El 23'-0"	3.50	3.85
63.7	R.B. El 23'-0"	3.50	3.85
63.8	R.B. El 23'-0"	0.15	0.17
63.9	R.B. El 23'-0"	0.15	0.17
63.10	R.B. El 23'-0"	0.51	0.56
63.11	R.B. El 23'-0"	0.51	0.56
63.12	R.B. El 23'-0"	0.51	0.56
64.4	R.B. El 51'-0"	0.52	0.57

DIFFERENTIAL PRESSURE LOADINGS ON SAFETY-RELATED MASONRY WALLS SUBJECT TO PIPE BREAK OUTSIDE CONTAINMENT LOADING (R')

Wall No.	Location	Peak Differential Pressure (PSI)	Equivalent Static Pressure (PSI)
64.5 (1)	R.B. El 51'-0"	9.66	9.66
64.5 (2)	R.B. El 51'-0"	3.25	3.58
64.6	R.B. El 51'-0"	0.20	0.22
64.7	R.B. El 51'-0"	0.20	0.22
64.8 (1)	R.B. El 51'-0"	9.56	9.56
64.8 (2)	R.B. El 51'-0"	3.25	3.58
64.13	R.B. El 51'-0"	0.52	0.57
65.0	R.B. El 51'-0"	0.20	0.22
65.1 (1)	R.B. El 51'-0"	9.56	9.56
65.1 (2)	R.B. El 51'-0"	3.22	3.54
65.2 (1)	R.B. El 51'-0"	9.56	9.56
65.2 (2)	R.B. El 51'-0"	3.22	3.54
65.4 (1)	R.B. El 51'-0"	0.10	0.11
65.4 (2)	R.B. El 51'-0"	2.70	2.97
65.5 (1)	R.B. El 51'-0"	0.10	0.11
65.5 (2)	R.B. El 51'-0"	1.38	1.52
65.6 (1)	R.B. El 51'-0"	0.10	0.11
65.6 (2)	R.B. El 51'-0"	1.38	1.52
65.7 (1)	R.B. El 51'-0"	0.10	0.11
65.7 (2)	R.B. El 51'-0"	1.32	1.45

ATTACHMENT 1 (con't)

DIFFERENTIAL PRESSURE LOADINGS ON SAFETY-RELATED MASONRY WALLS SUBJECT TO PIPE BREAK OUTSIDE CONTAINMENT LOADING (R')

Wall No.	Location	Peak Differential Pressure (PSI)	Equivalent Static Pressure (PSI)
65.8	R.B. El 51'-0"	0.52	0.57
65.9	R.B. El 51'-0"	0.52	0.57
65.10	R.B. El 51'-0"	0.52	0.57
65.18	R.B. El 51'-0"	0.76	0.91
65.19	R.B. El 51'-0"	0.76	0.91
65.21	R.B. El 51'-0"	0.52	0.57
65.22	R.B. El 51'-0"	0.52	0.57
66.0	R.B. EL 74'-3"	0.51	0.56
66.1	R.B. El 74'-3"	0.51	0.56
66.2	R.B. El 74'-3"	0.51	0.56
66.3	R.B. El 74'-3"	0.51	0.56
66.4	R.B. El 74'-3"	0.51	0.56
66.5	R.B. El 74'-3"	0.51	0.56
66.6	R.B. El 74'-3"	0.51	0.56
66.11	R.B. El 74'-3"	0.51	0.56
66.12	R.B. El 74'-3"	0.51	0.56
66.18	R.B. El 74'-3"	0.51	0.56
66.21	R.B. El 74'-3"	0.51	0.56
66.22	R.B. El 74'-3"	0.51	0.56
66.23	R.B. El 74'-3"	0.51	0.56

DIFFERENTIAL PRESSURE LOADINGS ON SAFETY-RELATED MASONRY WALLS SUBJECT TO PIPE BREAK OUTSIDE CONTAINMENT LOADING (R')

Wall No.	Location	Peak Differential Pressure (PSI)	Equivalent Static Pressure (PSI)
66.12	R.B. El 74'-3"	0.51	0.56
66.18	R.B. El 74'-3"	0.51	0.56
66.21	R.B. El 74'-3"	0.51	0.56
66.22	R.B. El 74'-3"	0.51	0.56
66.23	R.B. El 74'-3"	0.51	0.56
66.24	R.B. El 74'-3"	0.51	0.56
67.1	R.B. El 91'-3"	0.43	0.49
67.2	R.B. El 91'-3"	0.43	0.49
68.0	R.B. El 74'-3"	0.51	0.56
68.1	R.B. El 74'-3"	0.51	0.56
68.2	R.B. El 74'-3"	0.51	0.56
68.4	R.B. El 74'-3"	0.58	0.64
70.0	R.B. El 117'-0"	1.5	1.7
70.1	R.B. El 117'-0"	0.50	0.55
77.0	R.B. El 17'-6"	1.13	1.22
77.1	R.B. El 17'-6"	1.13	1.22
95.0	R.B. El 51'-0"	0.59	0.65
95.1	R.B. El 51'-0"	0.59	0.65
111.0	R.B. El 51'-0"	0.0	0.0

ATTACHMENT 1 (con't)

DIFFERENTIAL PRESSURE LOADINGS ON SAFETY-RELATED MASONRY WALLS SUBJECT TO PIPE BREAK OUTSIDE CONTAINMENT LOADING (R')

Wall No.	Location	Peak Differential Pressure (PSI)	Equivalent Static Pressure (PSI)
111.1	R.B. El 51'-0"	0.0	0.0
111.2	R.B. El 51'-0"	0.0	0.0
111.3 (1)	R.B. El 51'-0"	9.66	9.66
111.3 (2)	R.B. El 51'-0"	3.41	5.97
111.4 (1)	R.B. El 51'-0"	9.66	9.66
111.4 (2)	R.B. El 51'-0"	3.41	3.75
111.5 (1)	R.B. El 51'-0"	9.66	9.66
111.5 (2)	R.B. El 51'-0"	3.41	3.75
111.6 (1)	R.B. El 51'-0"	9.66	9.66
111.6 (2)	R.B. El 51'-0"	3.41	5.97
111.7 (1)	R.B. El 51'-0"	9.66	9.66
111.7 (2)	R.B. El 51'-0"	3.41	3.75
111.8 (1)	R.B. El 51'-0"	9.66	9.66
111.8 (2)	R.B. El 51'-0"	3.41	3.75
111.9 (1)	R.B. El 51'-0"	9.66	9.66
111.9 (2)	R.B. El 51'-0"	3.41	3.75
111.13	R.B. El 74'-3"	0.51	0.56
111.14	R.B. El 74'-3"	0.51	0.56
111.15	R.B. El 74'-3"	0.51	0.56
111.16	R.B. El 51'-0"	0.52	0.57

ATTACHMENT 1 (con't)

DIFFERENTIAL PRESSURE LOADINGS ON SAFETY-RELATED MASONRY WALLS SUBJECT TO PIPE BREAK OUTSIDE CONTAINMENT LOADING (R')

Wall No.	Location	Peak Differential Pressure (PSI)	Equivalent Static Pressure (PSI)
111.17	R.B. El 51'-0"	0.52	0.57
111.18	R.B. El 51'-0"	0.52	0.57
184.0	R.B. Aux Bay El 37'-0"	0.50	0.55
184.9	R.B. Aux Bay	0.76	0.91
185.1	R.B. Aux Bay El. 3'-0"	0.56	0.62
185.2	R.B. Aux Bay El. 3'-0"	0.56	0.62
185.7	R.B. Aux Bay El. -17'-6"	0.62	0.68
185.9	R.B. Aux Bay El. -17'-6"	0.62	0.68
185.10	R.B. Aux Bay El. -17'-6"	0.62	0.68
188.7	R.B. Aux Bay El. 23'-0"	0.50	0.55
188.8	R.B. Aux Bay El. 23'-0"	0.50	0.55
188.9	R.B. Aux Bay El 23'-0"	0.51	0.56
188.10	R.B. Aux Bay El. 23'-0"	0.50	0.55
188.12	R.B. Aux Bay El. 23'-0"	0.50	0.55

ATTACHMENT 1 (con't)

DIFFERENTIAL PRESSURE LOADINGS ON SAFETY-RELATED MASONRY WALLS SUBJECT TO PIPE BREAK OUTSIDE CONTAINMENT LOADING (R')

Wall No.	Location	Peak Differential Pressure (PSI)	Equivalent Static Pressure (PSI)
188.13	R.B. Aux Bay El. 23'-0"	0.50	0.55
196.0	Radwaste Bldg. El. 51'-0"	0.76	0.91
209.0	T.B. El 23'-0"	0.68	0.82
209.1	T.B. El 23'-0"	0.70	0.84
209.2	T.B. El 23'-0"	0.33	0.36
209.3	T.B. El 23'-0"	0.33	0.36
209.6	T.B. El 23'-0"	0.33	0.47
209.7	T.B. El 23'-0"	0.33	0.36
209.8	T.B. El 23'-0"	1.98	2.24
210.0	T.B. El 37'-0"	0.65	0.75
210.1	T.B. El 37'-0"	0.33	0.36
210.2	T.B. El 37'-0"	0.33	0.36
210.3	T.B. El 37'-0"	0.33	0.36
212.1	T.B. El 51'-0"	0.76	0.91
212.2	T.B. El 51'-0"	0.76	0.91

(1) Values for 30 ft^2 pipe being closed, PBOC break 2A
(2) Values for 30 ft^2 pipe being open, PBOC break 2T

CYGNA

Memorandum

Project Memo #24

To: Project Personnel

Date: February 2, 1982

From: J. D. McWilliam
MJD for JDM

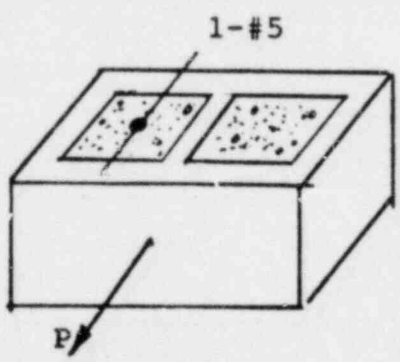
Job No: 80034

Subject: Allowable Block Pullout Loads

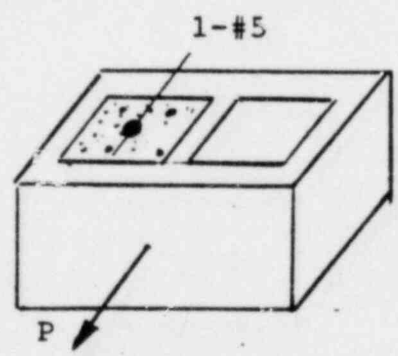
Copies: P. Baughman, B. Gang, J. Spitulnik, H. Reeser, P. DiDonato, M. DeGuzman, Project File, Central File

Reinforced Masonry Wall Cases	Wall Thickness (in)	P = Allowable Pullout Load (lb)	
		M	M'
A & B	8"	9735	14691
	12"	14850	22410
C & D	8"	4895	7387
	12"	6545	9877

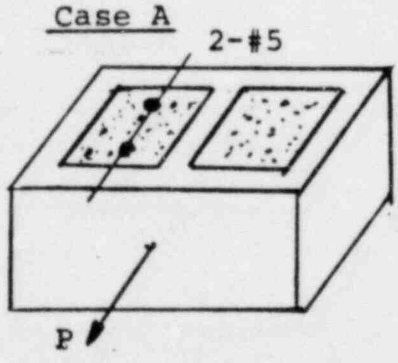
*Design values are based on one block unit, for size less than one unit, allowable loads shall be reduced.



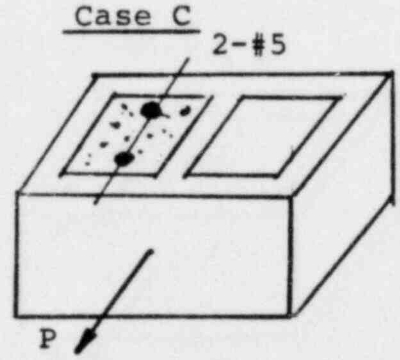
Case A



Case C



Case B



Case D

EES	CALCULATION COVER SHEET	Job No. 80034 File No. ZF Calc. Set No. G 8000 No. of Sheets 6

PROJECT PILGRIM UNIT #1

CLIENT BOSTON EDISON

SUBJECT

MASONRY WALL ANALYSIS FOR NRC IE BULLETIN 80-11

STATEMENT OF PROBLEM

· PROVIDE THE ALLOWABLE PULLOUT LOADS FOR EACH BLOCK UNIT OF THE SINGLE WYTHE REINF. MASONRY WALL DUE TO CONCENTRATED INERTIA LOADINGS IMPOSED BY ATTACHED COMPONENTS.

SOURCES OF DATA

① EES PNPS #1 DESIGN CRITERIA - 1, REV. 0

② REINFORCED MASONRY DESIGN BY ROBERT R. SCHNEIDER & WALTER L. DICKEY

③ EES PNPS CAL'S SET NO. G1000, PG 8 & 8A

SOURCES OF FORMULAE & REFERENCES

① EES PNPS #1 DESIGN CRITERIA - 1, REV. 0

② REINFORCED MASONRY DESIGN BY ROBERT R. SCHNEIDER & WALTER L. DICKEY

REMARKS

SEE PAGE 6 FOR ALLOWABLE PULLOUT LOADS

ORIGINATORS	CHECKERS	DISTRIBUTION	REVISION NO. 0
C.T. LIN 6/17/81	M. WEINER 6/18/81		SUPERSEDES CALCULATION
			SET NO. N/A
			APPROVED BY: C Fiorelli
			DATE: 6/18/81

EB2-0480-Rev.0

EES	CALCULATION SHEET	PREPARED BY C. T. LIN	DATE 6/17/81
	PROJECT PILGRIM UNIT #1	CHECKED BY M. WEINER	DATE 6/18/81
	SUBJECT MASONRY WALL ANALYSIS	JOB NO. 80034	FILE NO. 2F
	SYSTEM ALLOW. BLOCK PULLOUT LOADS		
	ANALYSIS NO. 8000 REV. NO. 0	SHEET NO. G8000-1	

REF

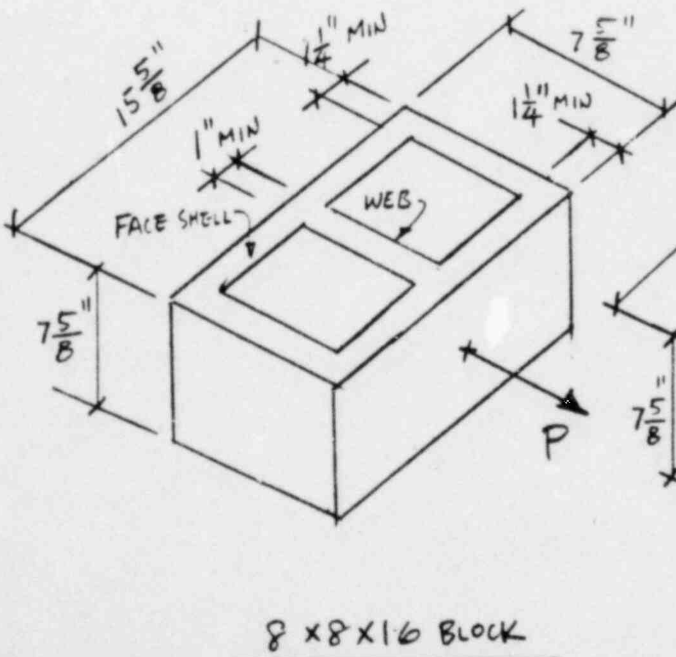
INTRODUCTION

THE FOLLOWING CAL'S ARE USED TO ESTABLISH ALLOWABLE BLOCK PULLOUT LOADS FOR THE DESIGN CRITERIA OF PILGRIM UNIT #1 MASONRY WALL ANALYSIS.

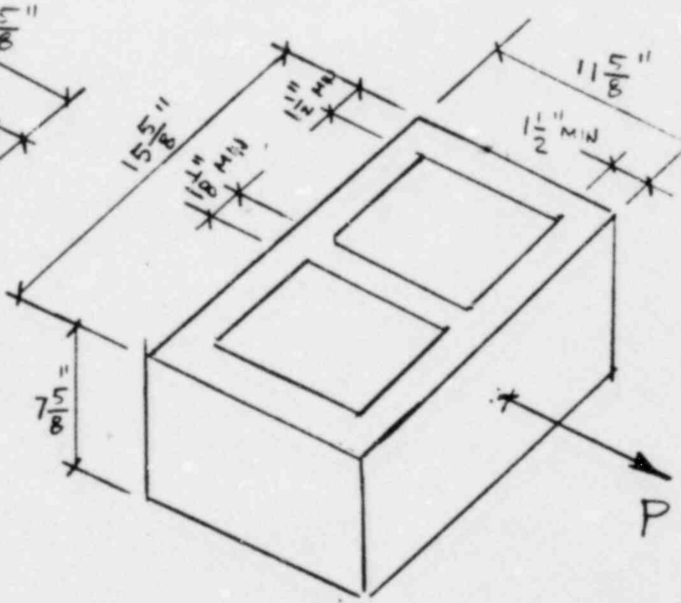
4 CASES OF REINFORCEMENT MASONRY WALL ARE STUDIED AS DEFINED BY CASE A, B, C & D FROM PG 1/5 OF EXHIBIT D, DESIGN CRITERIA 1. AND, FOR EACH CASE, TWO DIFFERENT WALL THICKNESSES 8" & 12" ARE CONSIDERED FOR THE SINGLE WYTHE MASONRY WALL.

BLOCK DIMENSIONS

PAGE 68 & 70 REINFORCED MASONRY DESIGN BY ROBERT R. SCHNEIDER & WALTER L. DICKEY



8 X 8 X 16 BLOCK



8 X 12 X 16 BLOCK

CALCULATION SHEET	PREPARED BY C. T. LIN	DATE 6/17/81
	CHECKED BY M. WEINER	DATE 6/18/81

EES

PROJECT PILGRIM UNIT #1
SUBJECT MASONRY WALL ANALYSIS
SYSTEM ALLOW. BLOCK PULLOUT LOADS
ANALYSIS NO. 8000 REV. NO. 0

JOB NO. 80034 FILE NO. 2F
SHEET NO. G-8000-2

ASSUMPTIONS:

①. EFFECTIVE SHEAR AREAS FOR PULLOUT ARE BASED ON CRACKED WALL SECTION AREA INSTEAD OF FULL AREA. SEE PAGE 3 + 4.

② ALLOWABLE SHEAR STRESSES OF GROUT, MOTAR AND MASONRY ARE ASSUMED THE SAME AND GIVEN BY DESIGN CRITERIA #1, EXHIBIT G, PAGE 1/7

③ ASSUME PULLOUT LOAD ACTING ON THE CENTER OF BLOCK UNIT SURFACE AND IGNORE THE ADDITIONAL MOMENT DUE TO ECCENTRICITY EFFECT.

④ ALLOWABLE PULLOUT LOADS ARE BASED ON ONE SINGLE UNIT OF 8×8×16 OR 8×12×16 BLOCK, FOR BLOCK SIZE LESS THAN ONE UNIT, ALLOWABLE PULLOUT SHALL BE REDUCED

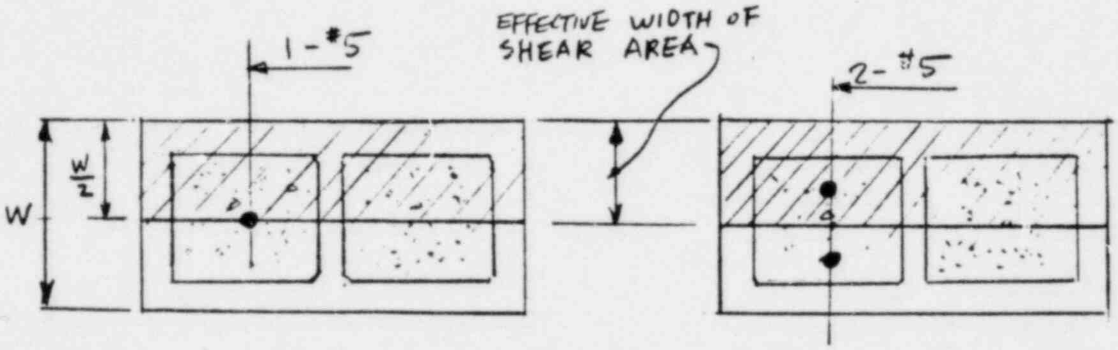
EES	CALCULATION SHEET	PREPARED BY C. T. LIN	DATE 6/17/81
	PROJECT PILGRIM UNIT #1	CHECKED BY M. WEINER	DATE 6/18/81
	SUBJECT MASONRY WALL ANALYSIS	JOB NO. 8024	FILE NO. 2F
	SYSTEM ALLOW. BLOCK PULLOUT LOAD		
	ANALYSIS NO. 8000 REV. NO. 0	SHEET NO. 8000-3	

REF.

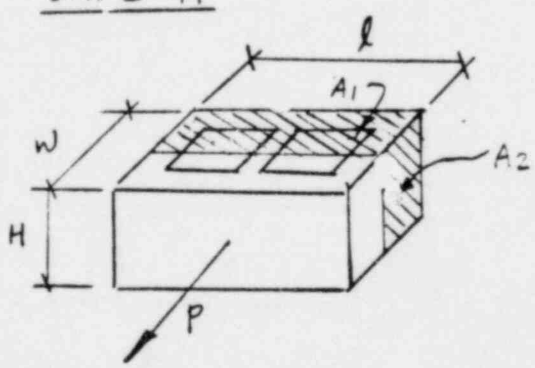
CASE A & B (EFFECTIVE SHEAR AREA)

CASE A = SHIELD WALL, FULLY GROUTED CELLS WITH 1-#5 VERTICAL REBARS FOR EACH BLOCK UNIT

CASE B = SHIELD WALL, FULLY GROUTED CELLS WITH 2-#5 VERTICAL REBARS FOR EACH BLOCK UNIT



CASE A CASE B



TOTAL EFFECTIVE SHEAR AREA FOR PULLOUT

$$A = (A_1 + A_2) \times 2 = (l \times \frac{W}{2} + H \times \frac{W}{2}) \times 2 = W(l+H)$$

8×8×16 BLOCK $A = 7.625 \times (15.625 + 7.625) = 177 \ IN^2$

8×12×16 BLOCK $A = 11.625\,(15.625 + 7.625) = 270 \ IN^2$

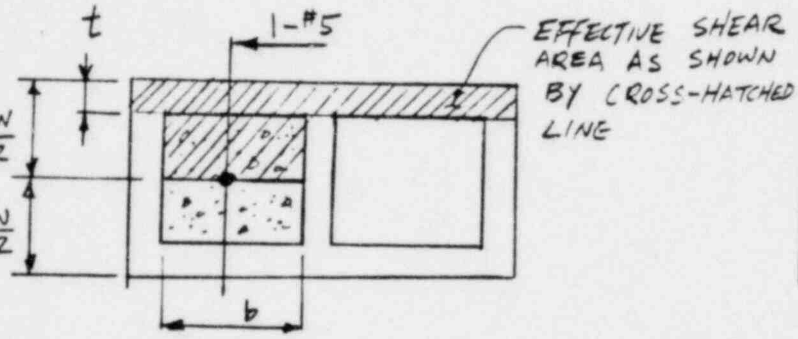
EES — **CALCULATION SHEET**

PROJECT: PILGRIM UNIT #1	PREPARED BY: C. T. LIN	DATE: 6/17/8	
SUBJECT: MASONRY WALL ANALYSIS	CHECKED BY: M. WEINER	DATE: 6/18/81	
SYSTEM: ALLOW. BLOCK PULLOUT LOAD	JOB NO.: 80034	FILE NO.: 2F	
ANALYSIS NO. 8000 REV. NO. 0	SHEET NO. G8000-4		

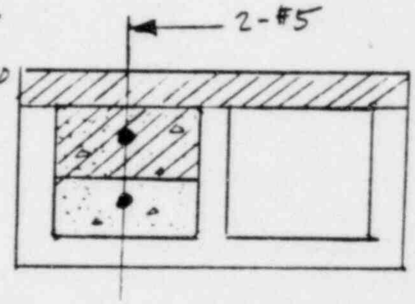
CASE C & D · (EFFECTIVE SHEAR AREA)

CASE C : PARTITION WALL, GROUTED IN ALTERNATE CELLS WITH 1 - #5 VERTICAL REBARS FOR EACH BLOCK UNIT

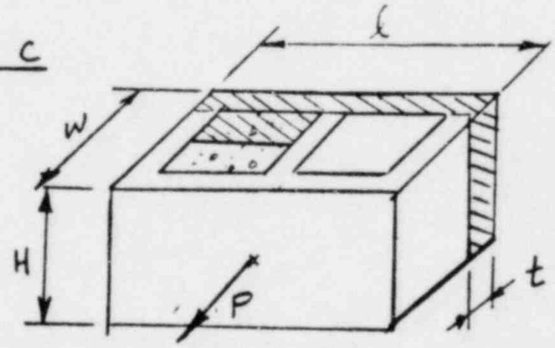
CASE D : PARTITION WALL, GROUTED IN ALTERNATE CELLS WITH 2 - #5 VERTICAL REBARS FOR EACH BLOCK UNIT



CASE C



CASE D



TOTAL EFFECTIVE SHEAR AREA FOR PULLOUT

$$A = (A1 + A2 + A3)\times 2 = [l\times t + (\tfrac{W}{2} - t)\times b + H\times t]\times 2$$
$$= 2t(l - b + H) + Wb$$

8X8X16 BLOCK, $t = 1.25''$, $b = 6.06''$, $W = 7.625''$, $H = 7.625''$, $l = 15.625''$

$$A = 2\times 1.25\,(15.625 - 6.06 + 7.625) + 7.625\times 6.06$$
$$= 89 \text{ IN}^2$$

8X12X16 BLOCK $t = 1.5''$, $b = 5.75''$, $W = 11.625''$, $H = 7.625''$, $l = 15.625''$

$$A = 2\times 1.5\,(15.625 - 5.75 + 7.625) + 11.625\times 5.75$$
$$= 119 \text{ IN}^2$$

EES

CALCULATION SHEET

PROJECT: PILGRIM UNIT #1	PREPARED BY: C. T. LIN — DATE: 6/17/81
SUBJECT: MASONRY WALL ANALYSIS	CHECKED BY: M. WENGER — DATE: 6/18/81
SYSTEM: ALLOWABLE BLOCK PULLOUT LOADS	JOB NO.: 80034 — FILE NO.: 2F
ANALYSIS NO.: 8000 — REV. NO.: 0	SHEET NO. G8000-5

ASSUME ALLOWABLE SHEAR STRESS $= 1.5\sqrt{f'_m}$ FOR M (OBE)

" " " $= 2.25\sqrt{f'_m}$ FOR M' (SSE)

(REFER TO SHEAR (OUT OF PLANE) AS SHOWN ON EXHIBIT G PAGE 1 OF 7, DC-1)

$\therefore \quad V_m = 55$ psi ... M

$V_m = 83$ psi .. M'

CASE A & B

8" WALL $P_M = 55 \times 177 = 9735\#$ } M

12" WALL $P_M = 55 \times 270 = 14850\#$ } M

8" WALL $P_{M'} = 83 \times 177 = 14691\#$ } M'

12" WALL $P_{M'} = 83 \times 270 = 22410\#$ } M'

CASE C & D

8" WALL $P_M = 55 \times 89 = 4895\#$ } M

12" WALL $P_M = 55 \times 119 = 6545\#$ } M

8" WALL $P_{M'} = 83 \times 89 = 7387\#$ } M'

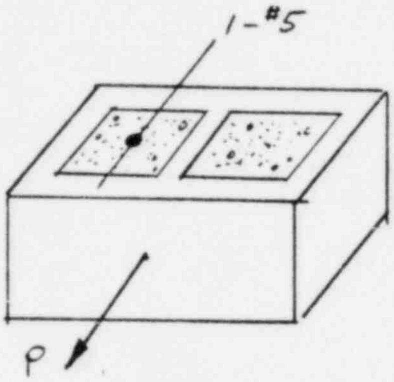
12" WALL $P_{M'} = 83 \times 119 = 9877\#$ } M'

EES

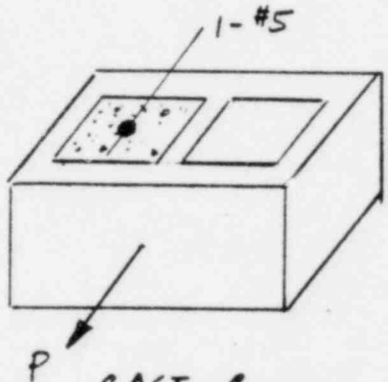
CALCULATION SHEET

PROJECT: PILGRIM UNIT #1	PREPARED BY: C. T. LIN	DATE: 6/17/81	
SUBJECT: MASONRY WALL ANALYSIS	CHECKED BY: M. WEINER	DATE: 6/18/81	
SYSTEM: ALLOWABLE BLOCK PULLOUT LOADS	JOB NO.: 80034	FILE NO.: 2F	
ANALYSIS NO.: 8000 REV. NO.: 0	SHEET NO.: G8000-6		

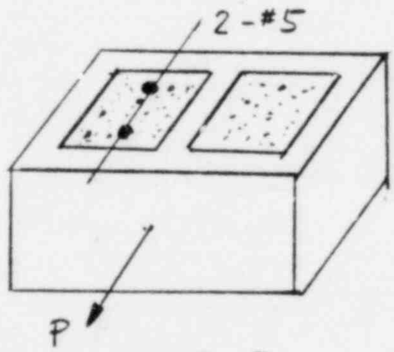
EXHIBIT I: ALLOWABLE BLOCK PULLOUT LOADS



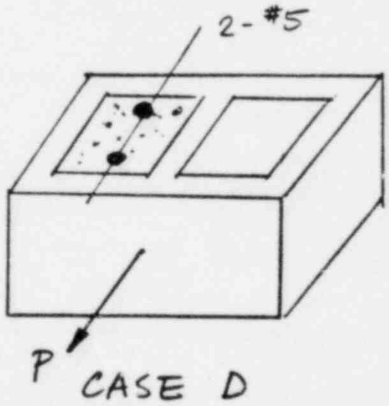
CASE A



CASE C



CASE B



CASE D

REINF. MASONRY WALL CASES	WALL THICKNESS (IN)	P = ALLOWABLE PULLOUT LOAD (lb)	
		M	M'
A & B	8"	9735	14691
	12"	14850	22410
C & D	8"	4895	7387
	12"	6545	9877

* DESIGN VALUES ARE BASED ON ONE BLOCK UNIT. FOR SIZE LESS THAN ONE UNIT, ALLOWABLE LOADS SHALL BE REDUCED.

CYGNA

Memorandum

Project Memo #5
Revision #1

To: Project Personnel

Date: October 13, 1981

From: W. J. Duffy

Job No: 80034

Subject: Tornado Depressurization Loads

Copies: P. Baughman B. Gang
C. DiNunzio H. Reeser
Project File P. DiDonato
Central File M. DeGuzman

Attached are the tornado depressurizatrion loads that shall be used in the analysis of masonry walls at Pilgrim I.

Please refer to DC-1 and WI-2 and WI-3 for further instruction in the application of these loads.

W. J. Duffy

W. J. Duffy
Assistant Project Engineer

JDM/jp
attachment

ATTACHMENT 1

Maximum Tornado Pressures

Building: Diesel Generator Building

Wall No.	Max. Ptorn (psi)	DLF*	Max Ptorn x DLF (psi)
198.0	0.859	1.0	0.859
198.1	0.859	1.0	0.859
198.2	0.859	1.0	0.859
198.3	0.859	1.0	0.859
198.4	0.0004	1.095	0.0005

Building: Radwaste Building

Wall No.	Max. Ptorn (psi)	DLF*	Max Ptorn x DLF (psi)
196.0	0.412	1.95	0.804
196.1	1.29	1.0	1.29
196.1	0.869	1.0	0.869
196.3	0.869	1.1	0.955
196.4	0.673	1.0	0.673
196.4	1.29	1.0	1.29
196.6	0.084	1.21	0.102
196.7	0.084	1.21	0.102
196.8	0.863	1.0	0.863
196.11	0.023	1.90	.043
196.14	1.47	1.0	1.47
196.15	1.47	1.0	1.47
196.16	1.47	1.0	1.47
196.17	1.47	1.0	1.47
196.18	1.47	1.0	1.47
196.19	0.00035	1.21	0.00042
195.4	1.55	1.095	1.69
195.4	1.54	1.10	1.69
195.10	1.56	1.0	1.56
195.10	0.00	0.0	0.0
195.10	0.869	1.12	0.973
195.14	0.924	1.12	1.03
195.17	1.32	1.0	1.32
195.18	0.194	1.12	0.218
195.19	0.259	1.0	0.259
195.20	0.806	1.21	0.975
195.23	0.43	1.0	0.43

Building: Radwaste Building (con't)

Wall No.	Max. Ptorn (psi)	DLF*	Max Ptorn x DLF (psi)
195.23	0.19	1.0	0.19
195.26	0.182	1.99	0.362
194.20	1.47	1.21	1.78
194.21	1.02	1.12	1.15
194.21	1.49	1.21	1.81
194.21	1.06	1.21	1.28
194.22	1.49	1.095	1.63
194.22	0.697	1.095	0.763
194.23	0.724	1.0	0.724
194.24	0.0413	1.21	0.05
191.24	1.50	1.10	1.65
191.24	1.54	1.0	1.54
191.25	1.098	1.095	1.2
191.25	0.107	1.6	0.171
191.26	0.03	1.0	0.03
191.26	0.08	1.15	0.0915
191.28	0.814	1.0	0.814
191.29	1.47	1.0	1.47
191.29	1.46	1.10	1.60
191.29	0.68	1.10	0.75
191.34	0.814	1.0	0.814
191.35	0.166	1.5	0.249
191.36	0.166	1.12	0.187
191.37	1.47	1.0	1.47
191.39	0.065	1.9	0.124
191.40	0.065	1.21	0.079
191.41	0.083	1.21	0.101
191.42	0.083	1.88	0.157
191.43	0.083	1.21	0.101
191.44	0.095	1.2	0.114
191.45	0.066	1.65	0.109
191.46	0.10	1.0	0.10
191.46	0.13	1.0	0.13
191.46	0.18	1.0	0.18
191.46	0.255	1.21	0.308
191.46	0.16	1.0	0.16
191.48	0.681	1.0	0.681
191.49	0.826	1.0	0.826
191.50	0.824	1.0	0.824
191.51	1.56	1.21	1.89
191.51	0.10	1.0	0.10

Building: Radwaste Building (con't)

Wall No.	Max. Ptorn (psi)	DLF*	Max Ptorn x DLF (psi)
191.53	1.56	1.0	1.56
191.54	1.56	1.0	1.56
191.57	0.096	1.2	0.115
191.55	0.04	1.87	0.069
191.56	0.04	1.9	0.075
191.59	0.0959	1.95	0.187

Building: Reactor Auxiliary Bay Building

Wall No.	Max. Ptorn (psi)	DLF*	Max Ptorn x DLF (psi)
184.0	0.361	1.0	0.405
184.2	0.644	1.0	0.644
184.3	0.644	1.095	0.706
184.4	0.63	1.0	0.63
184.7	0.631	1.0	0.631
184.8	0.631	1.0	0.631
184.9	0.415	1.8	0.747
188.1	0.68	1.1	0.75
188.2	0.68	1.1	0.75
188.3	0.644	1.095	0.706
188.4	0.644	1.0	0.644
188.6	1.57	1.0	1.57
188.7	1.06	1.0	1.06
188.8	0.361	1.12	0.405
188.9	0.51	1.99	1.02
188.10	0.20	1.16	0.232
188.10	0.20	1.21	0.243
188.11	0.00035	1.21	0.00042
188.12	0.154	1.12	0.186
188.13	0.171	1.21	0.207
185.1	0.644	1.0	0.644
185.2	0.788	1.12	0.883
185.3	0.616	1.0	0.616
185.4	0.616	1.0	0.616
185.5	1.0	1.0	1.0
185.6	1.0	1.0	1.0
185.7	1.03	1.0	1.03
185.9	1.03	1.0	1.03

ATTACHMENT 1 (con't)

Building: Reactor Auxiliary Bay Building (con't)

Wall No.	Max. Ptorn (psi)	DLF*	Max Ptorn x DLF (psi)
185.10	1.03	1.0	1.03
185.11	2.16	1.0	2.16
185.12	1.61	1.0	1.61
185.13	1.61	1.0	1.61
185.14	1.61	1.0	1.61

Building: Reactor

Wall No.	Max. Ptorn (psi)	DLF*	Max Ptorn x DLF (psi)
62.2	0.069	1.00	0.069
62.4	0.313	1.12	0.351
62.5	0.313	1.21	0.379
62.7	0.706	1.12	0.791
62.8	0.706	1.0	0.706
62.9	0.005	1.21	0.006
62.10	1.170	1.0	1.170
62.10	0.005	1.0	0.005
62.11	1.29	1.0	1.29
62.13	1.170	1.0	1.170
63.1	0.049	1.21	0.059
63.4	0.055	1.0	0.055
63.5	0.204	1.0	0.204
63.7	0.204	1.0	0.204
63.8	0.342	1.0	0.342
63.9	0.342	1.0	0.342
63.10	0.218	1.0	0.218
63.11	0.218	1.0	0.218
63.12	0.218	1.0	0.218
64.4	1.18	1.10	1.30
64.4	1.210	1.7	2.057
64.5	1.98	1.095	0.217
64.5	0.10	1.0	0.10
64.6	0.037	1.21	0.045
64.7	0.037	1.12	0.041
64.8	0.098	1.0	0.098
64.13	2.0	1.0	2.0
65.0	0.037	1.12	0.041
65.1	0.099	1.0	0.099

ATTACHMENT 1 (con't)

Building: Reactor (con't)

Wall No.	Max. Ptorn (psi)	DLF*	Max Ptorn x DLF (psi)
65.2	0.085	1.12	0.0975
65.2	0.00	1.0	0.00
65.4	0.073	1.21	0.088
65.5	0.061	1.12	0.068
65.6	0.061	1.21	0.074
65.7	0.096	1.0	0.096
65.8	1.24	1.7	2.11
65.8	1.14	1.10	1.25
65.9	2.0	1.0	2.0
65.10	2.0	1.0	2.0
65.18	0.903	1.21	1.09
65.19	0.903	1.12	1.01
65.19	0.59	1.0	0.59
65.19	0.756	1.12	0.847
65.19	0.0	0.0	0.0
65.17	1.56	1.0	1.56
65.13	1.56	1.10	1.72
65.13	1.56	1.0	1.56
65.12	1.56	1.0	1.56
65.14	1.51	1.0	1.51
65.20	0.631	1.0	0.631
65.21	1.14	1.09	1.24
66.0	0.57	1.10	0.63
66.0	0.00	1.0	0.00
66.0	1.54	1.0	1.54
66.1	1.33	1.0	1.33
66.2	0.57	1.10	0.63
66.2	1.33	1.0	1.33
66.3	1.33	1.0	1.33
66.4	0.00	1.0	0.00
66.5	0.67	1.10	0.74
66.5	1.29	1.0	1.29
66.6	1.55	1.0	1.55
66.6	0.674	1.0	0.674
66.7	0.017	1.88	0.032
66.10	1.20	1.21	0.46
66.10	0.07	1.0	0.07
66.11	0.68	1.0	0.68
66.11	0.68	1.10	0.75
66.12	0.693	1.0	0.693
66.18	0.693	1.0	0.693

ATTACHMENT 1 (con't)

Building: Reactor (con't)

Wall No.	Max. Ptorn (psi)	DLF*	Max Ptorn x DLF (psi)
66.21	0.674	1.0	0.674
66.22	3.0	1.0	3.0
66.23	3.0	1.0	3.0
66.24	3.0	1.0	3.0
67.1	0.705	1.095	0.771
67.2	0.705	1.12	0.789
68.0	1.58	1.0	1.58
68.1	1.56	1.0	1.56
68.2	1.92	1.0	1.92
68.3	2.97	1.0	2.97
68.4	1.45	1.0	1.45
68.5	0.074	1.21	0.09
68.8	0.00	1.0	0.00
68.9	0.00	1.0	0.00
68.10	0.28	1.05	0.297
70.0	0.837	1.0	0.837
70.1	0.837	1.35	1.13
77.0	0.514	1.0	0.514
77.1	0.514	1.0	0.514
111.0	0.00	1.0	0.00
111.1	0.00	1.1	0.00
111.2	0.00	1.0	0.00
111.3	2.95	1.12	3.3
111.4	2.95	1.0	2.95
111.5	2.95	1.0	2.95
111.6	2.95	1.12	3.3
111.7	2.95	1.0	2.95
111.8	2.95	1.0	2.95
111.9	2.95	1.0	2.95
111.11	2.97	1.12	3.3
111.12	2.97	1.12	3.3
111.13	2.97	1.0	2.97
111.14	2.97	1.0	2.97
111.15	2.98	1.0	2.98
111.16	2.96	1.0	2.96
111.17	2.96	1.0	2.96
111.18	2.96	1.0	2.96

ATTACHMENT 1

Building: Turbine Building

Wall No.	Max. Ptorn (psi)	DLF*	Max Ptorn x DLF (psi)
212.1	0.748	1.09	0.816
212.2	0.748	1.0	0.748
210.0	0.25	1.12	0.28
210.1	0.954	1.0	0.954
210.2	0.954	1.0	0.954
210.3	1.00	1.0	1.00
209.0	0.248	1.21	0.30
209.1	0.586	1.12	0.66
209.2	0.586	1.095	0.642
209.3	0.586	1.21	0.709
209.6	0.586	1.75	1.026
209.7	0.602	1.0	0.602
209.8	0.534	1.0	0.534

NOTE: For walls with more than one tornado pressure, check to see which compartments the wall separates.

10. <u>Provide results of analysis for tornado, pipe break outside containment (PBOC) loadings, and thermal effects.</u>

Differential pressure loads on safety-related concrete block walls are tabulated in CYGNA project memoranda 4 and 5 for PBOC and tornado depressurization respectively. These loads have been derived from the pressure curves and flow models provided by Bechtel. The wall analyses have not yet been finalized. A summary of these results will be provided in the final report.

The thermal loadings on the safety-related concrete block walls at Pilgrim result from the PBOC loading condition. The thermal loads are transient in nature and must be derived from the pressure and temperature curves calculated by the pipe break analysis. Two time regimes are of interest: (1) the short term thermal effects which must be combined with the pressure load; and (2) the long term effects after the pressure load has passed.

An evaluation of the short term effects (R11) has shown that wall moments due to heat transfer from the pipe break environment are very small compared to allowables and may be neglected in the pressure analysis. For the long term effects, since there is no applied pressure load, any overstress is self-relieving (secondary) and need not be evaluated. Also, since the Pilgrim walls are reinforced, tension cracking from thermal moments will not affect the load carrying capacity.

11. <u>Provide a report of the final re-evaluation</u>.

NRC Bulletin 80-11 requires the licensee to submit a written report upon completion of the re-evaluation program. This report must include the following:

1. description of the masonry walls,
2. description of the construction practices employed in the construction of the walls,
3. re-evaluation criteria and a detailed justification.

Items 2 and 3 are addressed in response to questions 6 and 4, respectively, of this submittal. A description of the masonry walls and associated system as well as the results of the re-evaluation (required in the 60 day response) program will be provided under separate cover. Although the re-evaluation is essentially complete, the calculations require finalizing to reflect as-built conditions. A schedule for close-out activities is being developed.

12. Provide information on both the method and schedule of any planned modifications.

Upon completion of any masonry wall re-evaluation, the need for modification to restore appropriate design safety factors was determined by the design limits given in the CYGNA Design Criteria, Revision 1 (See Question #1).

Fifteen (15) modifications were required by the re-evaluation program. A list of these modifications is attached. All construction except that required for Wall 64.4 has been completed. This wall is slightly overstressed but has been determined to result in deflections less than those considered unstable in a design base event. The schedule will be determined as part of an integrated replanning of all major modifications now in progress for Pilgrim Station.

Note that some modifications are labeled as structural and others as venting. If the wall was overstressed due to pipe break or tornado depressurization pressures, the feasibility of relieving the pressure was investigated. By testing the possible vent options in the building flow models for the particular event, potential pressure relief pathways were identified. The vents were designed using a diaphram arrangement, door latch, or door closing system which allow pressures to flow as analyzed in the event required, thus alleviating the need to strengthen the wall.

DATE 2/17/82
REV. 9

THIS REPORT IS A PRIORITIZED LIST OF BLOCK WALLS FOR THE 80-11 PROJECT. WALLS ARE LISTED BY ORDER OF WALL NUMBER.
THIS REPORT WILL BE UPDATED PERIODICALLY, AND USERS ARE CAUTIONED TO BE SURE THEY ARE USING THE MOST RECENT REVISION. COMMENTS AND QUESTIONS SHOULD BE DIRECTED TO NELSON MCLEAN, PRUDENTIAL, X-2608, OR TO NANCY WILLIAMS, PRUDENTIAL, X-2763

NOTES:

1. ACCESS CODES ARE AS FOLLOWS:
   - A - ACCESSIBLE DURING PLANT OPERATION
   - B - INACCESSIBLE DURING PLANT OPERATION
   - C - NEVER ACCESSIBLE - PHYSICALLY
   - D - NEVER ACCESSIBLE - RADIOLOGICALLY
   - E - ACCESSIBILITY VARIES WITH DAILY OPERATION
   - F - INDUSTRIAL HAZARD AREA

2. TYPE OF CONSTRUCTION IS SINGLE WYTHE(SW), MULTIPLE WYTHE (MW),OR COMPOSIT (COMP); DIFFERENT PORTIONS OF A WALL MAY BE OF DIFFERENT CONSTRUCTION, E.G., SWMW.

3. PDCR NUMBERS ARE SHOWN WITHOUT THE PARENT PDCR PREFIX. THE FULL NUMBER IS OF THE FORM: #81-53.AA.N.

4. A SAFETY RATING WITH AN 'X' (E.G., SX2), INDICATES THAT THE RATING HAS BEEN REVISED FROM ITS ORIGINAL VALUE.

5. A PDCR # WITH AN'X' INDICATES THAT THE PDCR HAS BEEN CANCELLED.

6. S/C = SECONDARY CONTAINMENT

STRUCTURAL MODIFICATIONS
- OUTAGE RELATED -

CONTRACTOR	WALL NUMBER	WALL FACE NUMBER	SFTY RTNG	WALK-DN SKETCH	ACCESS CODE	WALL LOC (1)	WALL LOC (2)	WALL LOC (3)	WALL LOC (4)	WALL FUNC (1)	WALL FUNC (2)	WALL FUNC (3)	WALL FUNC (4)	TYPE OF CONST	STRUCT PDCR #	ANCHOR PDCR #	
	111.3	E	SR1	7	B	REACT	65-0	K.2/13.3	K.4/13.3	SHIELD	BLK-OUT			MW	RF.5		STRUCT.
		W	-	-	C	REACT	65-0	K.2/13.3	K.4/13.3	SHIELD	BLK-OUT			MW	RF.5		
	111.6	E	SR1	-	B	REACT	61-9	L/13.8	L.3/13.8	BLK-OUT	SHIELD			MW	RF.6		STRUCT.
		W	-	-	C	REACT	61-9	L/13.8	L.3/13.8	BLK-OUT	SHIELD			MW	RF.6		
	184.3	E	SR2	469	A	AUX	37-0	H.3/4.4	H.6/4.4	PART	FIRE			SW	AD.1		VENT
		W	SR2	470	A	AUX	37-0	H.3/4.4	H.6/4.4	PART	FIRE			SW	AD.1		
RCI	188.8	N	SR1	460	A	AUX	23-0	J.5/2.5	J.5/4.3	SHIELD	FIRE			SW	AC.4		STRUCT.
RCI		S	SR1	461	A	AUX	23-0	J.5/2.5	J.5/4.3	SHIELD	FIRE			SW	AC.4		
RCI	184.0		SR1	460	A	AUX	37-0	J.5/2.4	J.5/4.5	PART	FIRE			SW	AC.4		
RCI			SR1	461	A	AUX	37-0	J.5/2.4	J.5/4.5	PART	FIRE			SW	AC.4		
RCI	194.21	E	SR1	574,575	A	RADWASTE	23-0	E.6/19.8	G.9/19.8	PART	FIRE			SW	WC.5	WC.5	VENT
RCI		J	SR1	589,590	A	RADWASTE	23-0	E.6/19.8	G.9/19.8	PART	FIRE			SW	WC.5	WC.5	
RCI	196.0	N	SR1	550THRU553	A	RADWASTE	51-0	G.9/18	G.9/20.1	PART	FIRE			SW	WE.1		STRUCT.
RCI		S	SR1	554THRU557	A	RADWASTE	51-0	G.9/18	G.9/20.1	PART	FIRE			SW	WE.1		
RCI	209.1	N	SR1	358	A	TURBINE	23-0	C.7/16.1	C.7/17.8	PART	FIRE			SW	TB.4		STRUCT.
RCI		S	HSR	359	A	TURBIHE	23-0	C.7/16.1	C.7/17.8	PART	FIRE			SW	TB.4		
RCI	209.2	E	SX2	368	A	TURBINE	23-0	A.8/17.7	B.4/17.7	PART	FIRE			SW	TB.2		STRUCT.
RCI		W	SX2	368	A	TURBIHE	23-0	A.8/17.7	B.4/17.7	PART	FIRE			SW	TB.2		
CROUSE	212.1	N	HSR	373,374	A	TURBINE	51-0	G/4.5	G/15.2	SHIELD				SWMW	TD.1		STRUCT.
CROUSE		S	SR1	365,378	A	TURBIHE	51-0	G/4.5	G/15.2	SHIELD				SWMW	TD.1		
CRSE/NOD	65.1	N	SR1	3,4,5	B	REACT	51-0	K.3/13.6	L/15	SHEAR	SHIELD			MW	RB.10	RB.10	VENT
CRSE/NOD		S	SR1	7,12,13,15	B,A	REACT	51-0	K.3/13.6	L/15	SHEAR	SHIELD			MW	RB.10	RB.10	
RCI	65.19	N	SR1	197THRU200	A	REACT	51-0	G.9/4.8	G.9/15.7	SHIELD	FIRE			SW	RB.1,2,3		STRUCT.
RCI		S	SR1	193THRU196	A	REACT	51-0	G.9/4.8	G.9/15.7	SHIELD	FIRE			SW	RB.1,2,3		
CROUSE	65.4	N	SR1	20	A	REACT	51-0	K.1/14.6	K.3/15.8	SHIELD	BLK-OUT			MW	RB.5		STRUCT.
CROUSE		S	-	-	D	REACT	51-0	K.1/14.6	K.3/15.8	SHIELD	BLK-OUT			MW	RB.5		
RCI	67.1	N	SR1	126	A	REACT	91-3	N/14.9	N/17	PART				MW	RD.2		STRUCT.
RCI		S	SR1	127	A	REACT	91-3	N/14.9	N/17	PART				MW	RD.2		
MERC	67.2	E	SR2	129	A	REACT	91-3	L.6/15	N/15	PART				MW	RD.1		STRUCT.
MERC		W	SR2	128	A	REACT	91-3	L.6/15	N/15	PART				MW	RD.1		

PLANNING FOR STRUCTURAL MODIFICATIONS
- POST OUTAGE -

CONTRACTOR	WALL NUMBER	WALL FACE NUMBER	SFTY RTNG	WALK-DN SKETCH	ACCESS CODE	WALL LOC (1)	WALL LOC (2)	WALL LOC (3)	WALL LOC (4)	WALL FUNC (1)	WALL FUNC (2)	WALL FUNC (3)	WALL FUNC (4)	TYPE OF CONST	STRUCT PDCR €	ANCHOR PDCR #	
RCI	64.4	E	SR1	179-181	A	REACT	51-0	L/7	P/7	SHIELD	FIRE			SW	RB.8	RB.8	VENT
RCI		W	SR1	182-184	A	REACT	51-0	L/7	P/7	SHIELD	FIRE			SW	RB.8	RB.8	S/C