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0020A

INTERIM REPORT
MECHANICAL & ELECTRICAL EQUIPMENT SEISMIC REEVALUATION PROGRAM HADDAM NECK NUCLEAR GENERATING STATION

For:

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1. INTRODUCTION

1.1 Background

This report presents the results of a seismic reevaluation of the electrical and mechanical equipment necessary for the safe shutdown of the Haddem Neck (Connecticut Yankee) Nuclear Power Station.

The plant is located on the Connecticut River in South-central Connecticut approximately twelve miles from Long Island Sound. The plant is a four loop, Pressurized Water Reactor (PWR) of 575 MWe capacity. The nuclear steam system was supplied by Westinghouse Electric, Inc. and Stone and Webster was the Architect-Engineer. Commercial operation started in January 1968.

Equipment at the Haddam Neck (Connecticut Yankee) Nuclear Power Station which were originally designated as "important to safety" (Seismic Category One) were designed to withstand a 0.17g earthquake. The ground response spectra used throughout the analyses were developed by Housner (Ref. 7.1) and are shown in Figure 1-1. Only a single level earthquake was specified with no provision for an OBE and SSE or equivalent.

It should be noted that on the basis of the U.S. Coast and Geodetic Survey (USGS) report originally performed (Ref. 7.2) it was decided that the maximum ground acceleration for an average earthquake would be 0.03g but that the previously mentioned 0.17g acceleration would be used in the design of "important to safety" systems (Seismic Category I). The 0.03g acceleration would be used in the design of all other structures and elements. Table 1-1 provides an overview of the original design criteria of the Haddam Neck safety equipment which is reevaluated in this report. In addition to the mechanical components identified in Table 1-1 supporting electrical equipment has also been selected for reevaluation.

1.2 Original Design

In the original design Seismic Category I equipment were designed so that steady state stresses or stresses resulting from hypothetical accident conditions when combined with seismic stresses resulting from a response spectrum analysis with a 0.17g ground acceleration did not exceed the yield strength of the material. Also, they would not suffer loss or impairment of function because of deflection or distortion.

All other structures and elements of the plant were designed to withstand seismic forces corresponding to a ground acceleration of 0.03g in addition to normal loads.

In applying the response spectrum to the design of systems or components, exclusive of the reactor internals and control rod drive system, an approximate design was established and the natural period determined. Using the damping factors listed in Table 1-2 and this natural period, the average acceleration response was then determined, using Figure 1-1, and the design reviewed to establish whether the stresses and deflections under this acceleration were within acceptable limits. This step was repeated as required until results were satisfactory. For minor systems

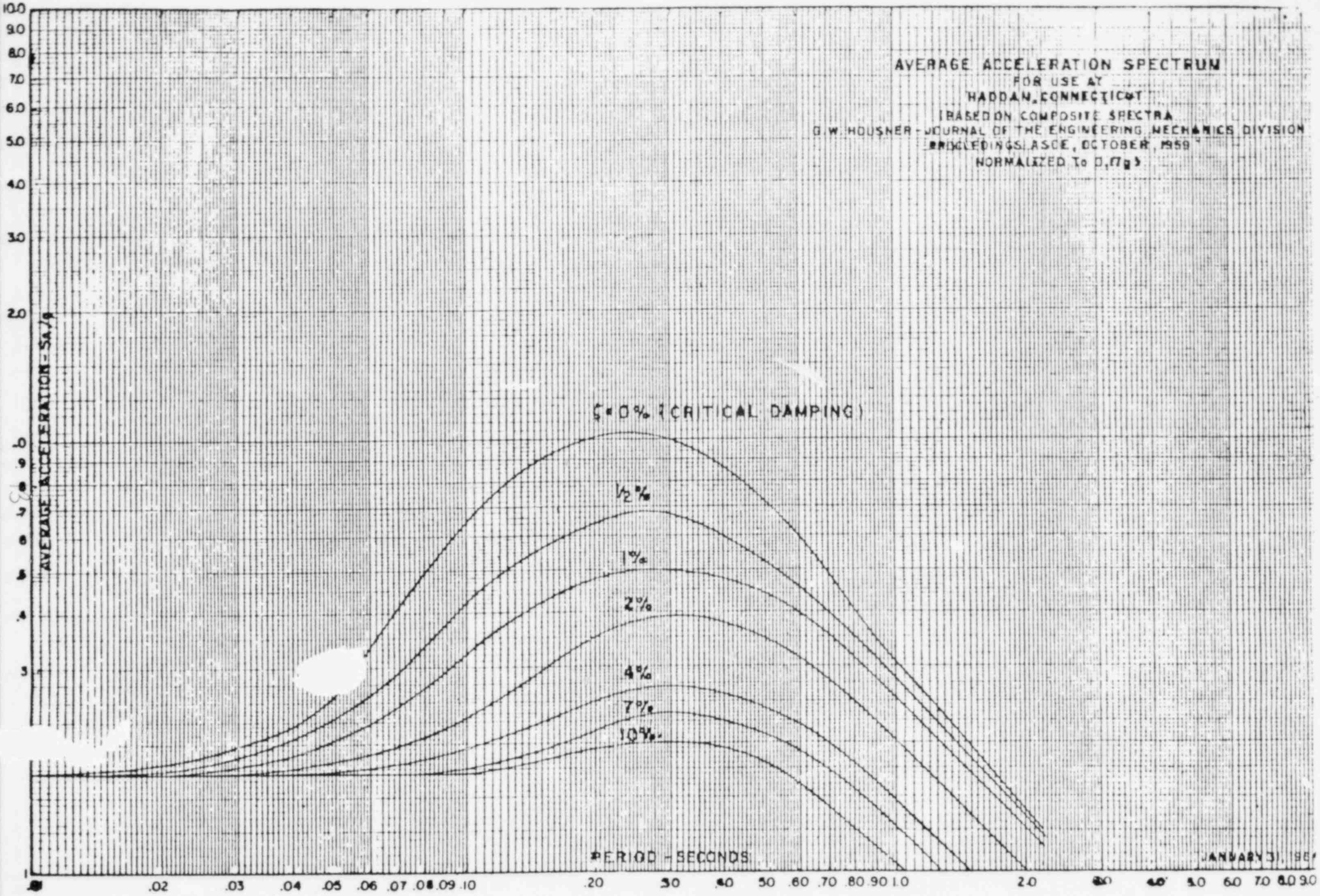

FIGURE 1-1. CONNECTICUT YANKEE GROUND RESPONSE SPECTRA

Table 1-1
Classification of Structures, Systems, and Components
Haddam Neck Nuclear Power Plant

Structures, Systems, and Components	Qualify Classification		Seismic Classification		Original Seismic Design
	Current Codes and Standards RG 1.26	Codes and Standards Used in Plant Design	RG 1.29	Used in Plant Design	
RESIDUAL HEAT REMOVAL SYSTEM					
Residual Heat Removal/ Low Pressure Safety Injection Pumps	ASME III	ASME VIII	Category I	Class I	.17g Horizontal .12g Vertical
Heat Exchanger - Tube Side	ASME III	ASME VIII (1962) and Code Case 1270N			.27 Horizontal .17g Vertical
SERVICE WATER SYSTEM					
Pumps	ASME III	Industry Standards SN 1.4-3.55	Category I	Class I	.4g @ C.G.
AUXILIARY FEED SYSTEM					
Pumps - Turbine Driven	ASME III	ASA B.31.1 1955	Category I	Class I	.17g
Demineralized Water Storage Tank	ASME III	ASA B96	Category I	Class I	.17g
Primary Water Storage Tank	ASME III		Category I		
CONTAINMENT COOLING SYSTEM					
Containment Fan Coolers (Fans and Cooling Coils)	ASME III		Category I	Class I	

Table 1-1 (Continued)
Classification of Structures, Systems, and Components
Haddam Neck Nuclear Power Plant

	Quality Classification		Seismic Classification		
Structures, Systems, and Components	Current Codes and Standards RG 1.26	Codes and Standards Used in Plant Design	RG 1.29	Used in Plant Design	Original Seismic Design
SAFETY INJECTION SYSTEM (EMERGENCY CORE COOLING SYSTEM)					
Refueling Water Storage Tank	ASME III		Category I	Class I	.5g Horizontal .17g Vertical
High Pressure Safety Injection Pumps	ASME III				.17g Horizontal .12g Vertical
CHEMICAL AND VOLUME CONTROL SYSTEM					
Regenerative Heat Exchanger	ASME III	ASME VIII (1962) and Code Cases 1270N, 1273N	Category I	Class I	
Volume Control Tank	ASME III	ASME VIII (1962) and Code Case 1270N	Category I	Class I	
Boric Acid Transfer Pumps	ASME III	No Code	Category I	Class I	.17 Horizontal .12g Vertical
Boric Acid Tank	ASME III	ASME VIII (1962)	Category I	Class I	

Table 1-1 (Continued)
Classification of Structures, Systems, and Components
Haddam Neck Nuclear Power Plant

Structures, Systems, and Components	Quality Classification		Seismic Classification		Orginal Seismic Design
	Current Codes and Standards RG 1.26	Codes and Standards Used in Plant Design	RG 1.29	Used in Plant Design	
Emergency Power System					
Emergency Diesel Generator	ASME III		Category I	Class I	.17g
5,000 Gal. V.G. Tank	ASME III		Category I		.03g
Clean Diesel Oil Day Tank	ASME III		Category I		
Diesel Air Start-Up Tanks	ASME III		Category I		
Diesel Exhaust Duct	ASME III		Category I		

Table 1-2 Original Earthquake Damping Factors At Connecticut Yankee Atomic Power Company

Component or Structure	Per Cent of Critical Damping
Reinforced concrete reactor containment, including foundation mat	7.0
Reinforced concrete framed structures	5.0
Steel framed structures, including supporting structures and foundations	
Bolted	2.5
Welded	1.0
Piping systems	
Carbon steel	0.5
Stainless steel	1.0
Reactor internals and control rod drive	
Welded assemblies	1.0
Bolted assemblies	2.0
Mechanical equipment including pumps and fans	2.0

or for particularly complex systems having a number of degrees of freedom, the peak of the appropriately damped acceleration curve was used to define inertia acceleration.

1.3 Current Seismic Requirements

Since the development of the seismic criteria used in the design of Connecticut Yankee, there have been significant changes to the seismic design criteria used to qualify mechanical and electrical equipment. These changes can be summarized as follows:

- Use of mean plus one sigma ground spectra rather than mean spectra
- rigorous use of floor response spectra as design input
- horizontal earthquake motion defined as two orthogonal components rather than a resultant
- changes in damping values considered in design
- changes in methods used to qualify electrical equipment operability by test

Because of the numerous changes which have taken place in the seismic qualification requirements for mechanical and electrical equipment since Connecticut Yankee was designed and built, and because documentation of the original seismic design adequacy cannot in all cases be located this qualification program has been developed to reevaluate the seismic design adequacy of the mechanical and electrical equipment in light of current requirements.

1.4 Organization of Report

This report has been divided into seven sections. The first section gives description of the overall report and provides the background associated with the seismic qualification of the Haddam Neck equipment. Section 2 defines the program scope. The third section details the approach and outlines the performance criteria used in the seismic reevaluation. Section 4 gives a general description of the methodology used in the reevaluation. The fifth section furnished a detailed explanation of the analysis performed for each individual component selected for review along with the results of the reevaluation. Section 6 includes both the conclusions reached regarding the equipment's seismic capability and any pertinent recommendations.

The seventh section is limited to listing the references mentioned in the report.

2. PROGRAM SCOPE

Within this section is a description of the seismic reevaluation program for the Connecticut Yankee Nuclear Power Station equipment which is being conducted as part of the Systematic Evaluation Program.

The equipment whose reevaluation is included in this report excludes all equipment and piping associated with the Reactor Coolant Pressure Boundary (reevaluation is being performed by others) and all other Seismic Category I piping (reviewed by NUSCO).

All the Seismic Category I equipment, devices, or components required for safe shutdown were identified and an "S-List" compiled. From this list and an in-situ inspection, at least one sample representative of each type of critical component was selected for a detailed reevaluation. Of course, the component selected and here reviewed is itself required for safe shutdown and, it is our belief, representative of the limiting conditions that actually exist.

The types of critical mechanical and electrical components and distribution systems evaluated are summarized as follows:

(a) Tanks & Vessels
(b) Heat Exchangers
(c) Pumps
(d) Electrical Equipment Racks
(e) Electrical Cabinets
(f) HVAC Equipment
(g) HVAC Duct

In Table 2-1 are listed the particular components which have been selected for reevaluation. The acceptance criteria use in the reevaluation of these components is listed in Table 3-1.

Evaluation of equipment shall include consideration of active as well as passive modes of failure where such evaluation can be performed by analysis.

Table 2-1 Equipment Selected For Reevaluation

Id No.	Component
M-1	ESW Pump
M-2	Diesel Exhaust Duct
M-3	Diesel Air Start-Up Tanks
M-4	CVCC Regenerative Heat Exchanger
M-5	Diesel Generator
M-6	Boric Acid Pump
M-7	High Pressure Safety Injection Pump
M-8	RHR Pump
M-9	RHR Heat Exchanger
M-10	Boric Acid Tank
M-11	Demineralized Water Storage Tank
M-12	Refueling Water Storage Tank
M-13	Steam Driven Aux. Feedwater Pump
M-14	Underground 5,000 Gal. Oil Tank
M-15	Clean Diesel Oil Day Tank
M-16	Volume Control Tank
M-17	Containment Fan Coolers
M-18	Primary Water Storage Tank
E-1	Battery Rack
E-2	MCC #1
E-3	Switch Gear (D.G. Room)
E-4	Control Panel (D.G. Room)
E-5	Engine Mounted Control Panel (on Diesel Gen)
E-6	4160-480 V Switchgear
E-7	Transformers (Switchgear Room)
E-8	MCC #5 & #6
E-9	Battery Charger
E-10	MCC #3
E-11	Main Control Board
E-12	Emergency Power Control Board
E-13	MCC #8

The criteria for selection of critical equipment to be evaluated are as follows:

(a) Critical equipment required for safe shutdown shall be identified based on functional importance.

(b) Based on walk-through and insitu inspection particular items of equipment shall be selected from the critical list which tend to exhibit a high sensitivity to seismic loading.

The equipment selected for reevaluation is listed in Table 2-1. Of the 31 components listed only 5 have been analyzed. These are:

1) Motor Control Center #1
2) Diesel Generator Exhaust Duct
3) Steam Driven Auxiliary Feedwater Pump
4) Boric Acid Tank
5) Refueling Cavity Water Storage Tank

Detailed methodology used, results obtained and conclusions reached are included for these five listed components only.

3. REEVALUATION CRITERIA

3.1 General

In terms of seismic design adequacy, nuclear power plant equipment and distribution systems fall into two main categories--active and passive. Typically found in the active category are pumps, valves, motors and associated motor control centers, and switchgear.

Seismic design adequacy of active components, which depends upon function as well as structural or leak tight integrity, may be demonstrated by either analysis or test. Testing is generally the preferred method but, because of size or weight restrictions of available test facilities or difficulty in monitoring function, many active components are seismically evaluated by analysis. To ensure active component function by analysis, with a relatively high degree of confidence, deformations must be limited and predictable. Therefore, unless rigorous non-linear analysis is performed, total stresses in such components should be limited to the elastic linear range, bounded by 1.0 times the specified minimum yield stress of the material, and in no case would the total stress in a component be allowed to exceed the actual "as built" yield stress. In evaluating the seismic design adequacy of active components, potential functional failure modes are identified, (ie. excessive bearing loads, impeller or rotor shaft deformation to cause binding, etc.) and factors of safety to failure determined.

Passive components considered in this report are of two types; P-1, those components that are required for safe shutdown and for which their only safety functions are to remain leak tight or maintain structural integrity during or following the postulated seismic hazard, P-2, these components do not need to move or change state during a seismic event but may be required to do so after the event and must retain structural and leak tight integrity.

Table 3-1 Allowable Stress Criteria & Damping Values

Northeast Utilities Service Company - Haddam Neck
SSE Level EQ. 0.17g ZPGA

	Component	A - Active; P-Passive 1 Or 2 (2)	Allowable Stress	Damping Values - %	Reference
M-1	ESW Pump	A	$S_{all} \leq .8\ S_y$	7	1,2,3
M-2	Diesel Exhaust Duct	P-1	$S_{all} \leq S_y$	4	1,2,3
M-3	Diesel Air Start-Up Tanks	P-1	$S_{all} \leq S_y$	7	1,2,3
M-4	CVCC Regenerative Heat Exchanger	P-1	$S_{all} \leq S_y$	4	1,2,3
M-5	Diesel Generator	A	$S_{all} \leq .8\ S_y$	7	1,2,3
M-6	Boric Acid Pump	A	$S_{all} \leq .8\ S_y$	7	1,2,3
M-7	High Pressure Safety Injection Pump	A	$S_{all} \leq .8\ S_y$	7	1,2,3
M-8	RHR Pump	P-2	$S_{all} \leq .9\ S_y$	7	1,2,3
M-9	RHR Heat Exchanger	P-1	$S_{all} \leq S_y$	4	1,2,3
M-10	Boric Acid Tank	P-1	$S_{all} \leq S_y$	7 - Impulsive 0.5 - Sloshing	1,2,3
M-11	Demineralized Water Storage Tank	P-1	$S_{all} \leq S_y$	7 - Impulsive 0.5 Sloshing	1,2,3
M-12	Refueling Water Storage Tank	P-1	$S_{all} \leq S_y$	7 - Impulsive 0.5 - Sloshing	1,2,3

Allowable Stress Criteria & Damping Values Continued

	Component	A - Active; P-Passive 1 Or 2 (2)	Allowable Stress	Damping Values - %	Reference
M-13	Steam Driven Aux. Feedwater Pump	P-2	$S_{all} \leq .9S_y$	7	1,2,3
M-14	Underground 5,000 Gal. Oil Tank	P-1	$S_{all} \leq S_y$	4 - Impulsive 0.5 - Sloshing	1,2,3
M-15	Clean Diesel Oil Day Tank	P-1	$S_{all} \leq S_y$	7 - Impulsive 0.5 - Sloshing	1,2,3
M-16	Volume Control Tank	P-1	$S_{all} \leq S_y$	4 - Impulsive 0.5 - Sloshing	1,2,3
M-17	Containment Fan Coolers	P-2	$S_{all} \leq .9S_y$	7	
M-18	Primary Water Storage Tank	P-2	$S_{all} \leq .9S_y$	7 % Impulsive	1,2,3

Allowable Stress Criteria & Damping Values Continued

	Component	A - Active; P-Passive 1 Or 2 (2)	Allowable Stress	Damping Values - %	Reference
E-1	Battery Rack	P-1	$S_{all} \leq S_y$	7	1,2,3
E-2	MCC #1	P-1	$S_{all} \leq S_y$	7	1,2,3
E-3	Switch Gear (D.G. Room)	P-1	$S_{all} \leq S_y$	7	1,2,3
E-4	Control Panel (D.G. Room)	P-1	$S_{all} \leq S_y$	7	1,2,3
E-5	Engine Mounted Control Panel (on Diesel Gen)	P-1	$S_{all} \leq S_y$	7	1,2,3
E-6	4160-480 V Switchgear	P-1	$S_{all} \leq S_y$	7	1,2,3
E-7	Transformers (Switchgear Room)	P-1	$S_{all} \leq S_y$	7	1,2,3
E-8	MCC #5 & #6	P-1	$S_{all} \leq S_y$	7	1,2,3
E-9	Battery Charger	P-1	$S_{all} \leq S_y$	7	1,2,3
E-10	MCC #3	P-1	$S_{all} \leq S_y$	7	1,2,3
E-11	Main Control Board	P-1	$S_{all} \leq S_y$	7	1,2,3
E-12	Emergency Power Control Board	P-1	$S_{all} \leq S_y$	7	1,2,3
E-13	MCC #8	P-1	$S_{all} \leq S_y$	7	1,2,3

Allowable Stress Criteria & Damping Values Continued

Component	A - Active; P-Passive 1 Or 2 (2)	Allowable Stress	Damping Values - %	Reference
Component Support Structures (4)	P-1	$S_{all} \le S_y$	As defined for supported component	1,2,3
Bolting				
-Embedded Bolting		$S_{all} \le 0.7\ S_u$		1,2,3
-Expansion Anchors		Ultimate Capacity /4		1,2,3
Welding		$S_{all} \le 0.6\ S_y$		

Notes:

1) The safety function of components is classified as follows"

   a) Passive Components I - Do not need to move or change state during or following a seismic event but must retain structural and leak tight integrity.
   b) Passive Components II - do not need to move or change state during a seismic event but may be required to do so after the event and must retain structural and leak right integrity.
   c) Active Components may be required to move or change state during or following a seismic event and must retain structural and leak tight integrity.

2) Allowable Buckling Load Equal to 2/3 Critical Buckling Load.

3) Detailed Stress Analysis of component in accordance with ASME Section III Class 2-1980 requirements.
4) Detailed Stress Analysis of component supports in acccordance with ASME Section III - NF-1980 and Appendix XVII.

References:

1) S & A Proposal
2) EDAC 175-130.01
3) USAEC Docket No. 50-213

Typically found in the passive category are pressure vessels, heat exchangers, tanks, piping and other fluid distribution systems, transformers, electrical distribution systems and their support structures and supports for active components where potential plastic deformation of the support does not result in plastic deformation in the component.

In determining seismic adequacy by analysis, the most important distinction between active and passive components is the stress level that the component is allowed to reach in response to the seismic excitation. For passive components, higher total stress limits, typically ranging from 1.2 times yield to 0.7 times the ultimate strength of the material, are permitted for SSE level seismic events by current design procedures and codes (Ref. 7-3).

It should be understood that where existing docket material more stringently limits the behavior of these components, that lower limit is reflected in the analyses included in this report.

3.2 Seismic Input Spectra

In this evaluation, input seismic motion to the component or system is based on an elastic ground or floor response spectra generated from a site specific ground response spectra which has a zero period ground acceleration (ZPGA) of 0.17g (See Fig. 3-1). Spectral curves were generated by Blume Associates.

3.3 Damping

Damping values used in the reanalysis of equipment are listed in Table 3-2. The damping values listed in Table 3-2 are generally not applicable until stress levels near yield or its equivalent are reached. However, their use is justified in the same manner single values of damping for the OBE level and SSE level earthquakes are justified in R.G.1.61. That is, if stress resultants obtained using the higher damping values in equipment are lower than those required to permit use of the higher equipment damping then the equipment stresses are lower than those allowed by the acceptance criteria. If equipment damping values are lower to reflect lower resultant stresses in the equipment, seismic loads and resultant stresses in the equipment will increase such that in the limit of allowable stress, the use of the higher damping values will be justified.

3.4 Load Combinations

The equipment seismic review shall be conducted for the following load combination:

$$U = 1.0\ W + 1.0\ P_D + 1.0\ SSE + 1.0\ NL$$

where:

U = Calculated load or stress on the equipment, device or component

W = dead load

P_D = Maximum operating pressure load

SSE = Inertial load due to Safe Shutdown Earthquake

NL = Nozzle Loads on the Component(1) (2)

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(1) Nozzle loads from nozzles 4 in. and smaller in components will generally not be considered in evaluation of support adequacy of the component.

(2) In cases where nozzle loads are not available, the limiting nozzle load is taken equal to one third of the ASME code allowable on the component in accordance with the recommendation of API 610.

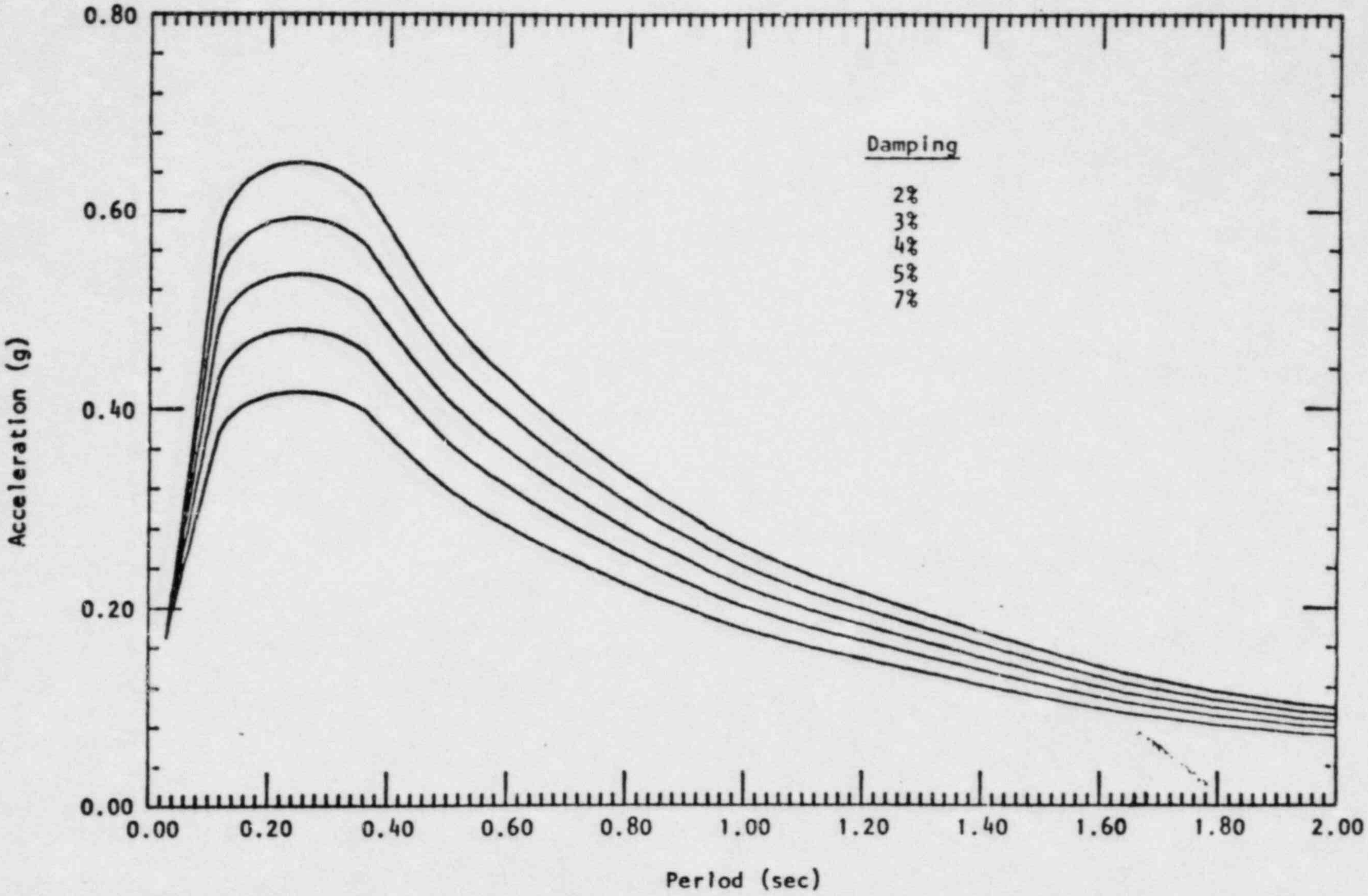

FIGURE 3-1 FREE-FIELD HORIZONTAL ACCELERATION RESPONSE SPECTRA
CONNECTICUT YANKEE ATOMIC POWER PLANT

Table 3-2 Damping Values - Percent Critical to be used in the Seismic Margins Review for Passive Components(4)

Structure or Component	Percent Critical Damping
Large diameter piping systems Pipe diameter ≥12 in.	4.0(1)
Small diameter piping systems Pipe diameter ≤ 12 in.	3.0(1)
Welded Steel Structures	4.0(3)
Bolted Steel Structures	7.0(3)
Welded Steel Components	4.0(2)
Bolted Steel Components	7.0(2)
Reinforced Concrete Structures	7.0(3)
Prestressed Concrete Structures	5.0(3)

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(1) These values are based on tests performed by Westinghouse Electric Co. and a review of existing test data. (Ref. 7-14)

(2) These damping values are consistent with damping values defined for welded and bolted structures defined in NUREG/CR 0098 and used in the SEP associated with Group 1 plants.

(3) R.G. 1.61 OBE damping levels shall be used as structural damping in generation of floor response spectra where total calculated stresses in the structure for the Site Specific Earthquake do not exceed 1/2 yield.

(4) Damping values used in evaluation of active components shall be reduced in the same proportion of OBE to SSE damping values as defined in Table 1 of R.G.1.61.

4. METHODOLOGY

4.1 General

Only elastic component and system analysis is used in accordance with the requirements of Appendix F of the ASME Code Section III.

All analysis has been/will be performed using one of three computer codes. These are:

a) DYNAFLEX - finite element program specifically tailored for analysis of piping and distribution systems.

b) SAP 4 general purpose finite element program.

c) ANSYS - general purpose finite element program.

Verification of the versions of these programs being used in this reevaluation are available as part of Stevenson and Associates general QA program. Verification was achieved by benchmarking each code used against other generally acceptable codes.

4.2 Tanks

In general, tanks are analyzed via one dimensional finite element multi-degree-of-freedom models using the sloshing model as developed in TID 7024 (Ref. 7.6) and the impulsive load model developed in the draft ASCE Standard on Seismic Design of Structure (Ref. 7.7). Soil-structure interaction is taken into account by representing the components anchorage as translational springs in one horizontal and one vertical direction together with one torsional spring about a horizontal axis. Generally soil structure interaction models are assumed for foundation media having shear wave velocities less than 4000 ft/sec otherwise a rigid base mode is assumed.

4.3 Mechanical Equipment

Mechanical equipment models are also represented as one dimensional finite element multi-degree-of-freedom models rigidly anchored or alternatively anchor bolt stiffness may be considered explicitly in the analysis. Modeling techniques used are as indicated by McDonald (Ref. 7-5).

4.4 Electrical Equipment

Electrical equipment of the panel and control board type are represented as finite element models composed of beam and plate elements.
In addition, several of the electrical components underwent an in-situ, low impedance test so as to determine:

- Natural Frequency
- Mode Shape
- Damping.

The results of the analyses are then compared to the experimental results so as to verify the accuracy of our mathematical model. Further, test results are being used to generate in-equipment response spectra so as to facilitate the functional qualification of all locally mounted electrical devices. The electrical components for which in-situ tests have been performed include:

- Generator Control Panel
- Local Engine Control Panel
- High Voltage Switchgear
- Battery Charger
- MCC #1
- MCC #3
- MCC #5
- MCC #6

5. EQUIPMENT EVALUATION

Following are detailed explanations of the seismic reanalysis performed for each of the 31 components listed in Table 2-1. In addition, a summary of the results obtained is also included for each of the components analyzed.

For this interim report, the results are given for only those five components that have been reviewed.

5.1 ESW PUMP

5.2 DIESEL EXHAUST DUCT

SYSTEM Mechanical - #2

COMPONENT NAME Diesel Exhaust Duct COMPONENT №

LOCATION Diesel Generator Room ELEVATION From EL. 35'-0" to EL. 47'-2"

COMPONENT SAFETY FUNCTION: ACTIVE ☐ PASSIVE 1 ☒ 2 ☐

S-LIST PAGE № 125

METHOD OF ANALYSIS: 3 Dimensional Pipe Element Model. Supports are then analyzed by applying forces obtained from the piping analysis

SPECTRAL CURVES USED: Response Spectra of Roof of the Diesel Generator Building - taken as 2x GRS

DAMPING VALUE ASSUMED: 4%

ACCEPTANCE BEHAVIOR CRITERIA USED: $S_{all} \le S_{yield}$, Equation 9 of Subsection NC-3652 of ASME Section III, Division I, ASME Appendix XVII & Appendix A of this report.

COMPUTER CODE USED: Dynaflex, Finite Element, Multi-Purpose Program

REMARKS:

Duct - o.k.

Tee section of the muffler supports requires stiffening

S&A

DESIGN REPORT COVER SHEET

NORTHEAST UTILITIES -

HADDAM NECK

REV.№	0		
BY	PNP		
DATE	4-1-82		
CHK'D	FAT		
DATE	4/15/82		
APPR.			
DATE	6/12/82		

5.2.1 INTRODUCTION

The Diesel Exhaust Duct is a 22" diameter pipe, fabricated from plate, that runs from the top of the Diesel Generator, through an expansion joint and a muffler, and then out to atmosphere through the roof of the diesel generator building.

A total of six supports exist on the line. Four are adjustable roll rod-type supports which provide vertical restraint in one direction. The other two supports are found on the muffler, one of which is an anchor, which provides full restraint against displacement and rotation in all three directions. The anchor consists of plates welded to the outside of the muffler and to a Tee section, which fits between the plates. The Tee section is then welded to support plates in the ceiling. The other muffler support provides restraint against displacement in the lateral and vertical direction and is similar to the anchor with the exception that a horizontal pin is placed in a slotted hole through the plates and the Tee section. Details of the supports are shown in Figures 5.2.1 through 5.2.6.

An isometric drawing is constructed which models the duct and its supports and is illustrated in Figure 5.2.7. The analysis of the duct was made using Dynaflex, a piping program with static and dynamic capabilities in accordance with the requirements of NC 3600 of the ASME BPVC Section III.

5.2.2 SUMMARY OF RESULTS

Based upon the results of the analysis, the web of the Tee section of the muffler supports must be stiffened to allow the duct system to maintain structural integrity during and after the defined seismic event.

The third computer run, as explained in Section 5.2.5 of this report, is used in the analysis and indicates a maximum primary stress of 11,485 psi, which occurs at an elbow, node point 11n, for the load combination of dead weight and seismic inertial forces. This value compares to the allowable stress limit of 36,000 psi, which is defined in Section 5.2.4 of this report.

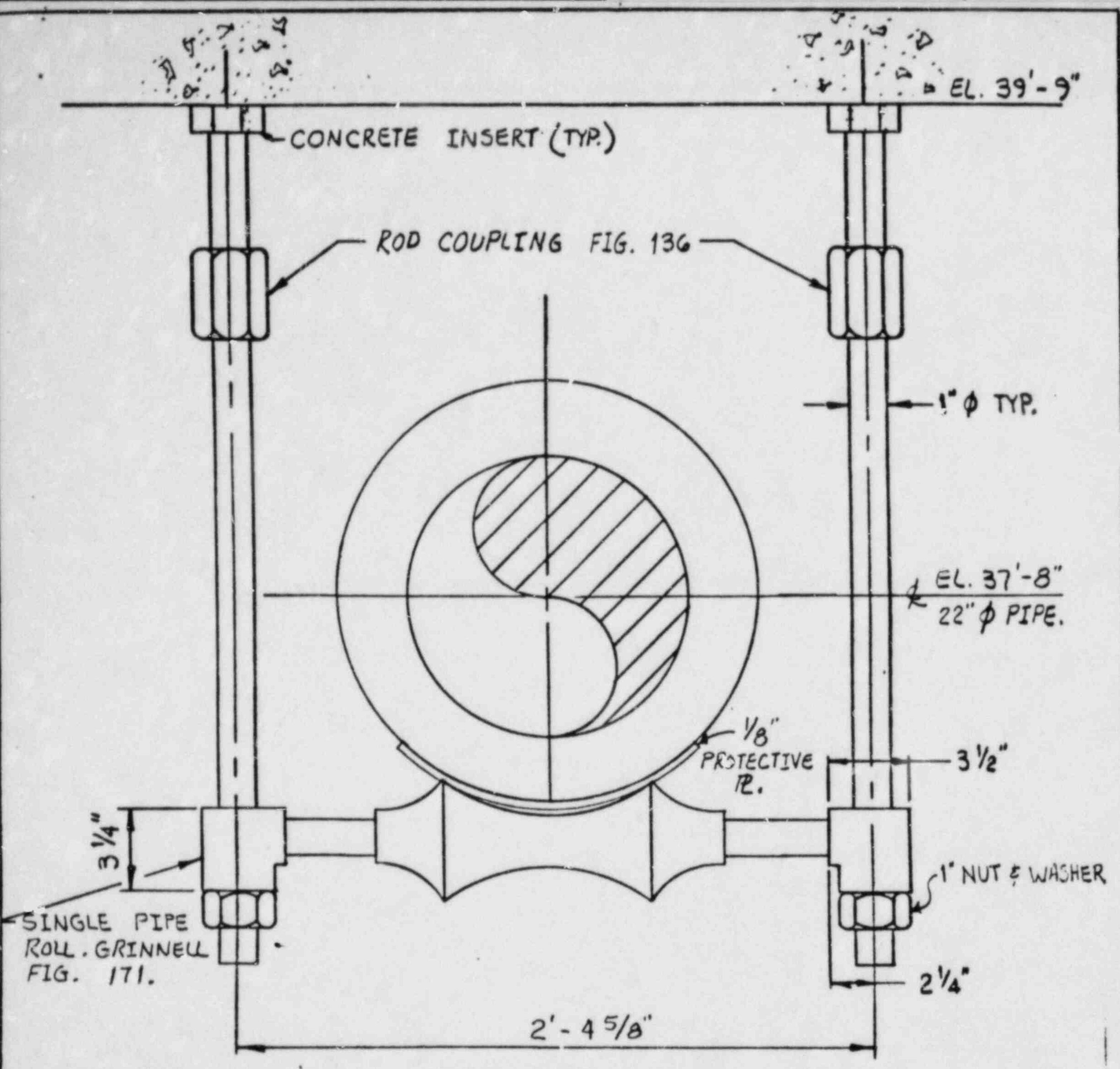
The pipe supports are then analyzed using the loads obtained in the computer run for the combined loadings of dead weight and seismic inertia. Considering only the limiting sections, the following results are obtained.

1) Adjustable Pipe Roll Support (Figures 5.2.1 and 5.2.2)

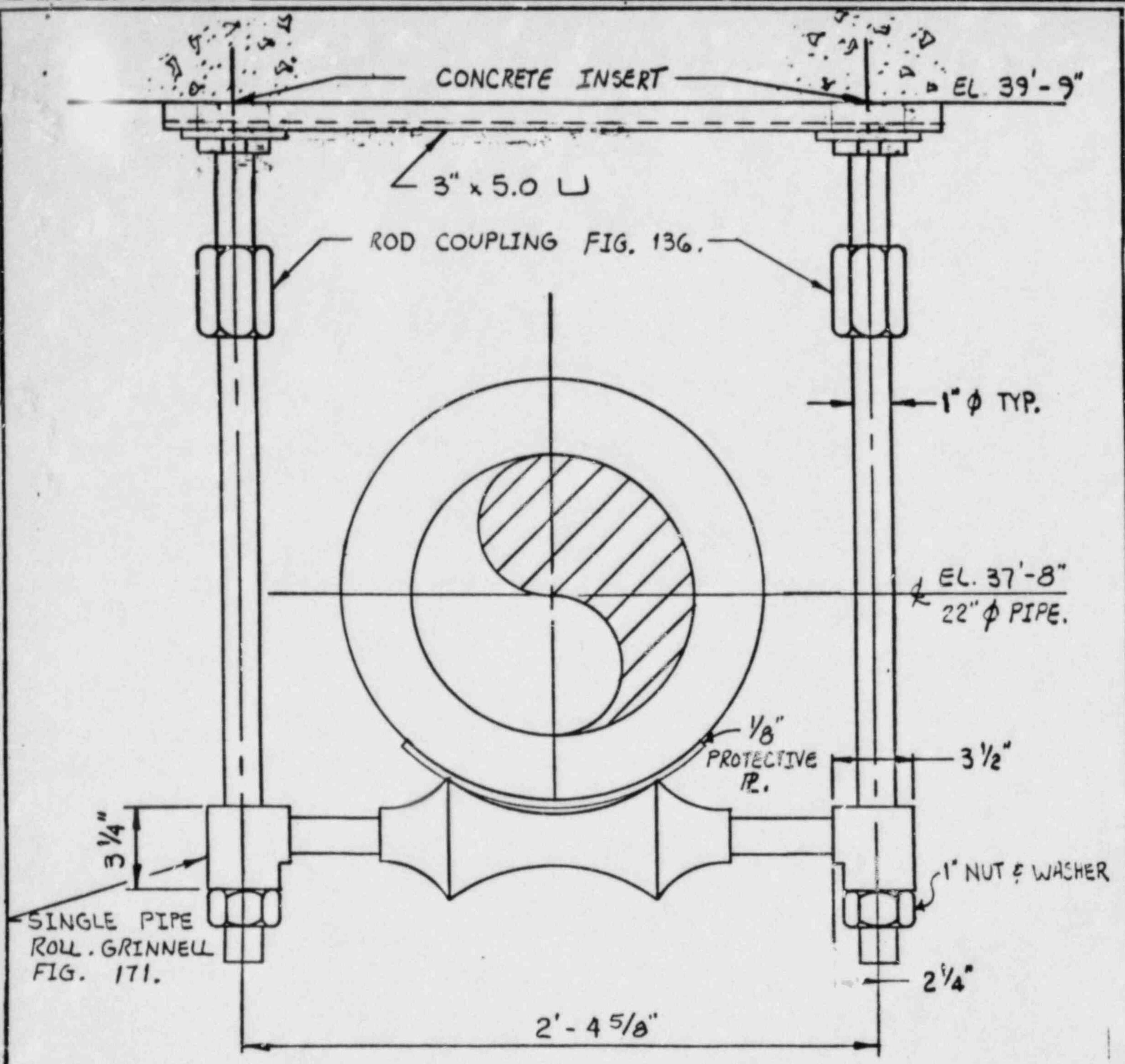
(a) 1" ϕ Rod

Tension: $f_t = 3713$ psi $F_t = 36{,}000$ psi

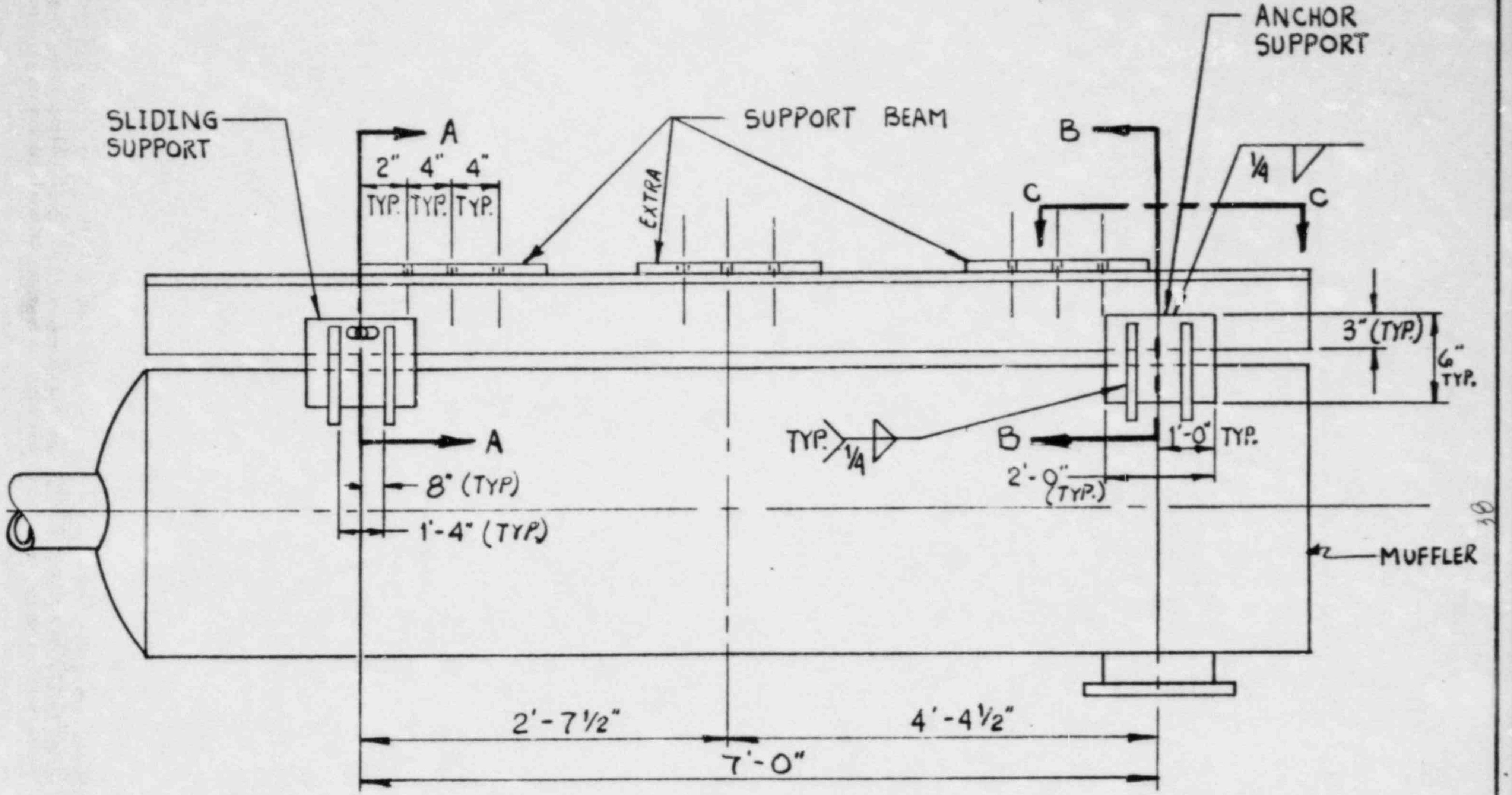
Safety Factor = 9.7



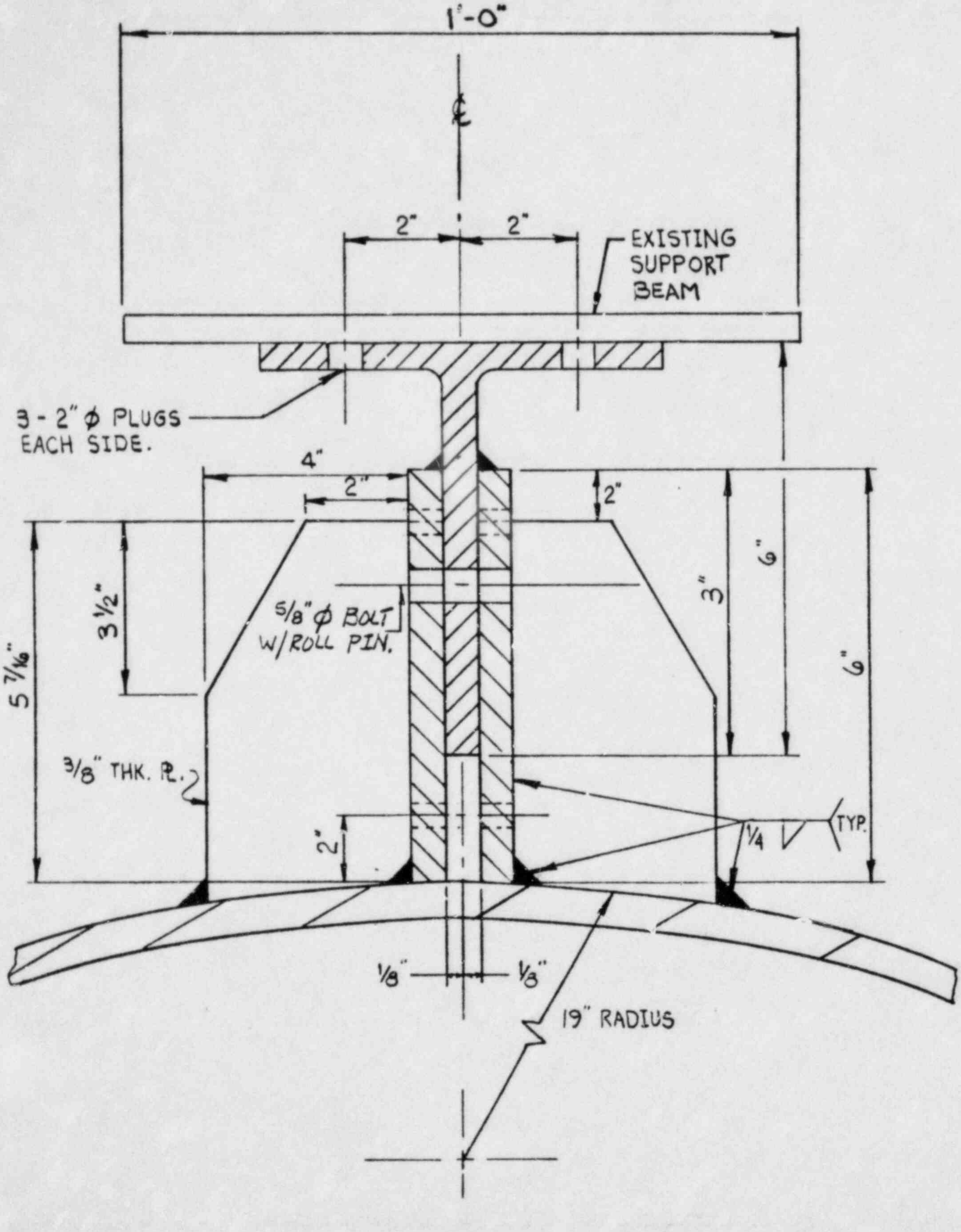
5.2-1 Adjustable Pipe Roll Support (RH 6, 7, and 8)



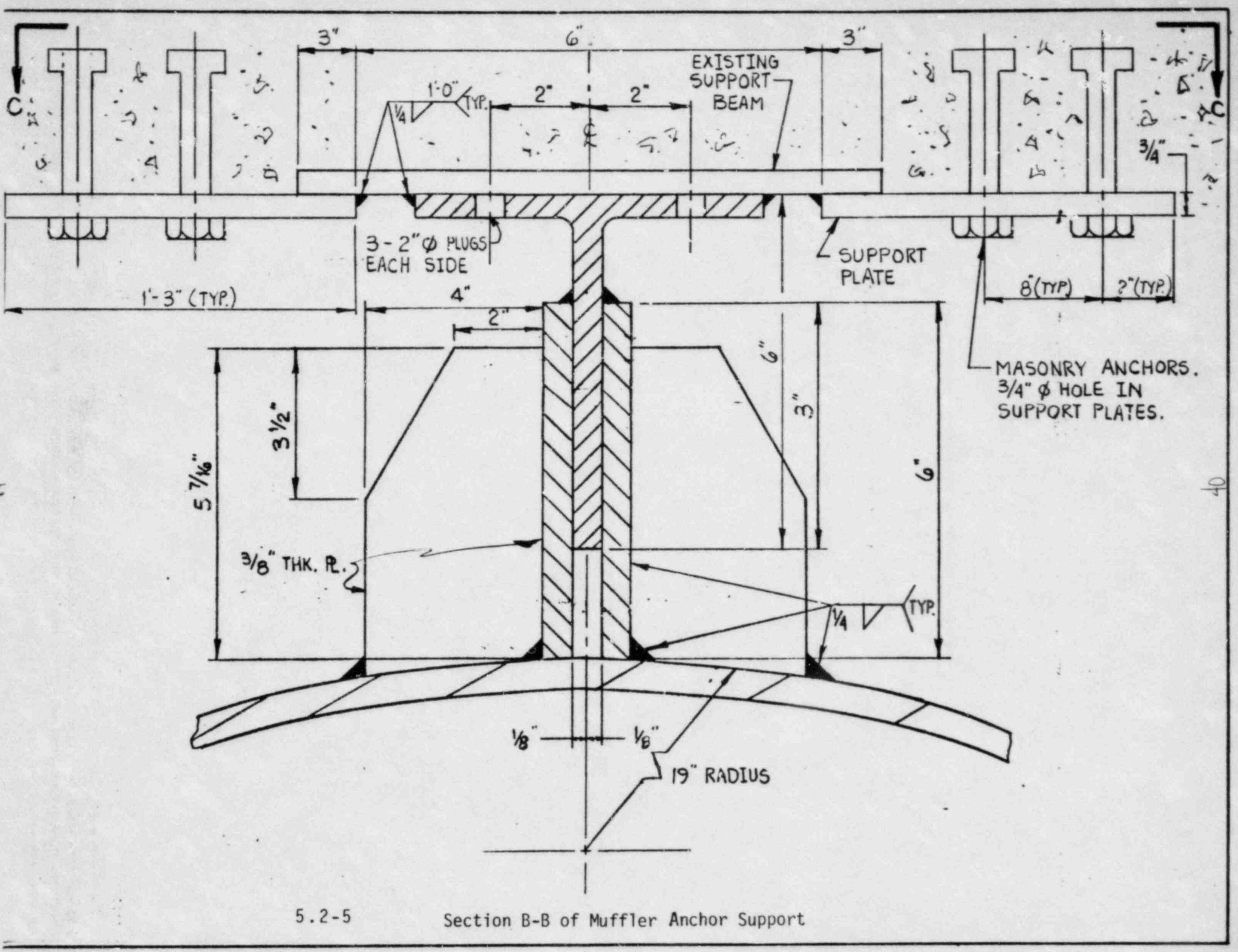
5.2-2 Adjustable Pipe Roll Support
(RH 9)



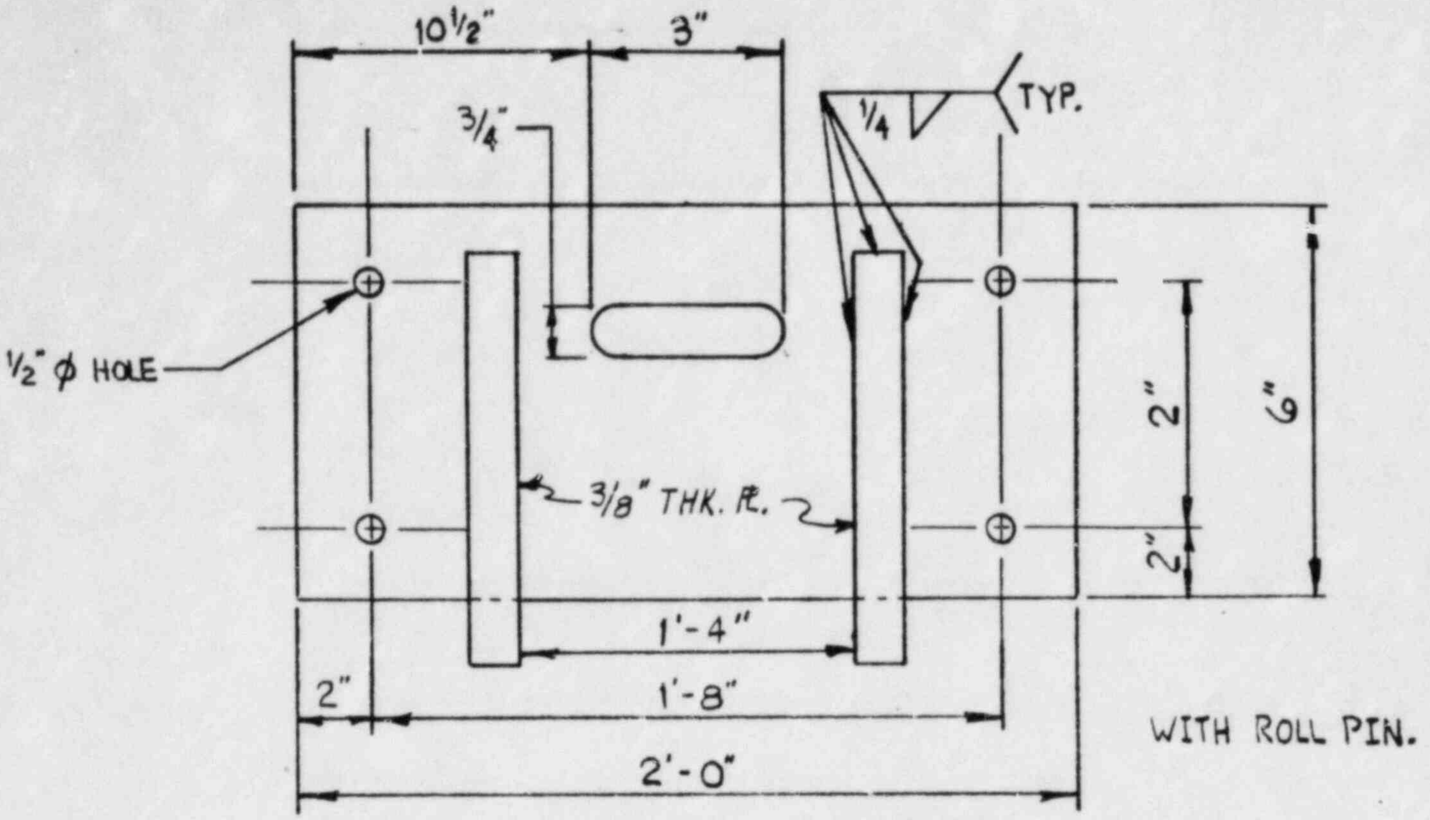
5.2-3 Muffler and Supports



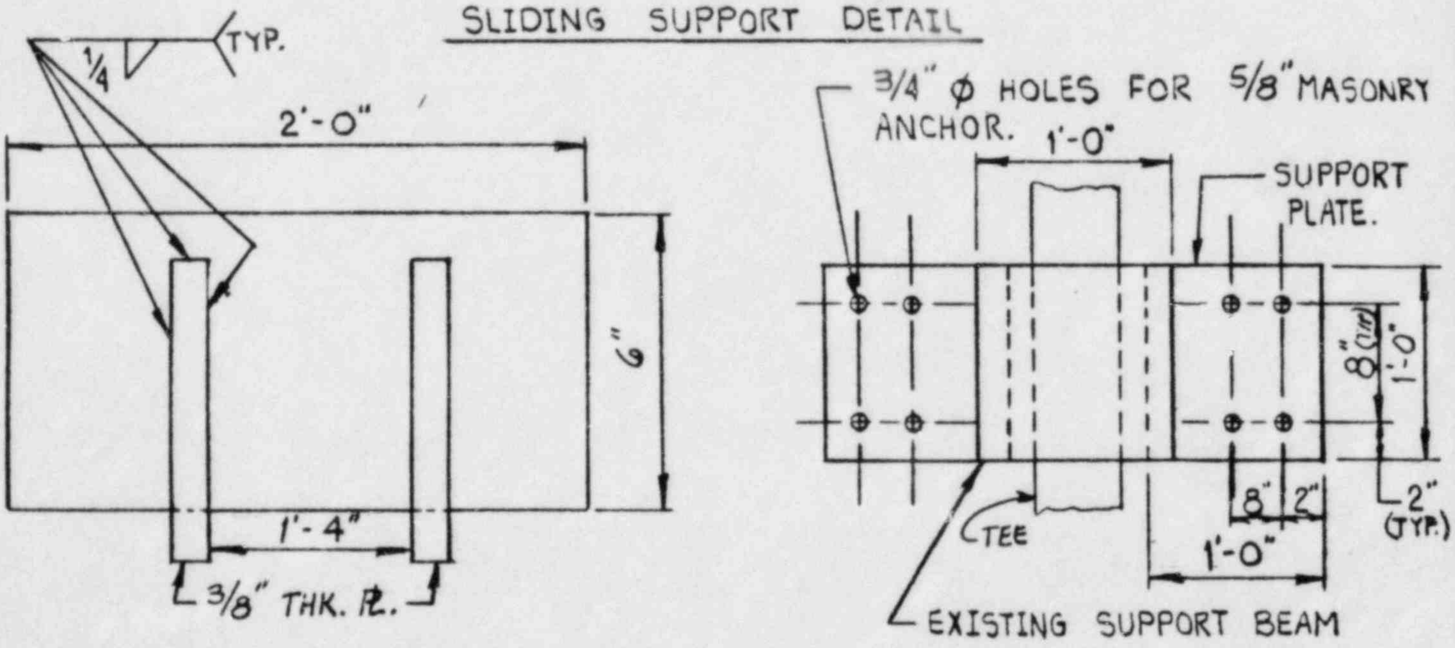
5.2-4 Section A-A of Muffler Sliding Support



5.2-5 Section B-B of Muffler Anchor Support



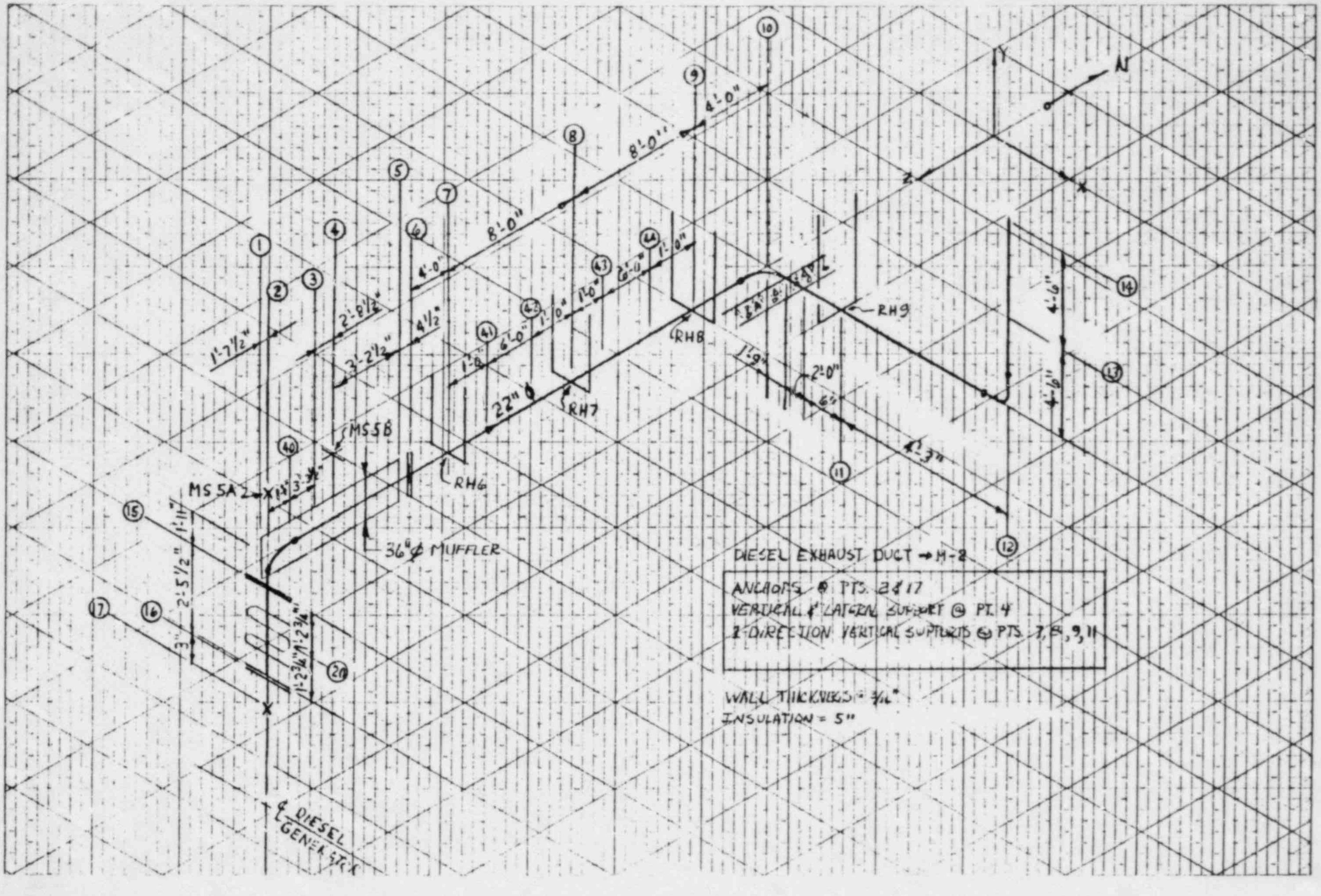
SLIDING SUPPORT DETAIL



ANCHOR SUPPORT DETAIL

SECTION C-C
(PLAN OF FIG. 5.)

5.2-6 Muffler Support Details



5.2-7 Isometric Drawing of Diesel Exhaust Duct System

(b) 1" ϕ Concrete Expansion Anchors (assuming a minimum embedment of 4 1/2" and 4,000 psi concrete)

Tension: t = 2250 lb   T = 16,000 lb

Interaction Equation = 0.84

Safety Factor = 1.19

2) Muffler supports (Figures 5.2.3 through 5.2.6)

(a) 1/4" fillet welds of the plates to the muffler

f_r = 5193 psi F_r = 33,600 psi

where;

f_r = resultant stress due to combined shear bending, and torsion.

F_r = allowable resultant stress taken as 1.6 x 0.3 F_y for E70XX electrodes.

Safety Factor = 6.47

(b) 1/4" thick web of the 6 x 9.5 T section (one support only)
Bending: f_b = 233,646 psi F_b = 36,000 psi

web is overstressed

(c) 5/8" ϕ pin (A 193 Gr. B7) on muffler sliding support

Double Shear: f_v = 3,012 psi F_v = 28,933 psi

Safety factor = 9.6

5.2.3 LOAD CRITERIA

The piping system and its supports are identified as passive, P-1 components. The load combination considered in the seismic design adequacy is;

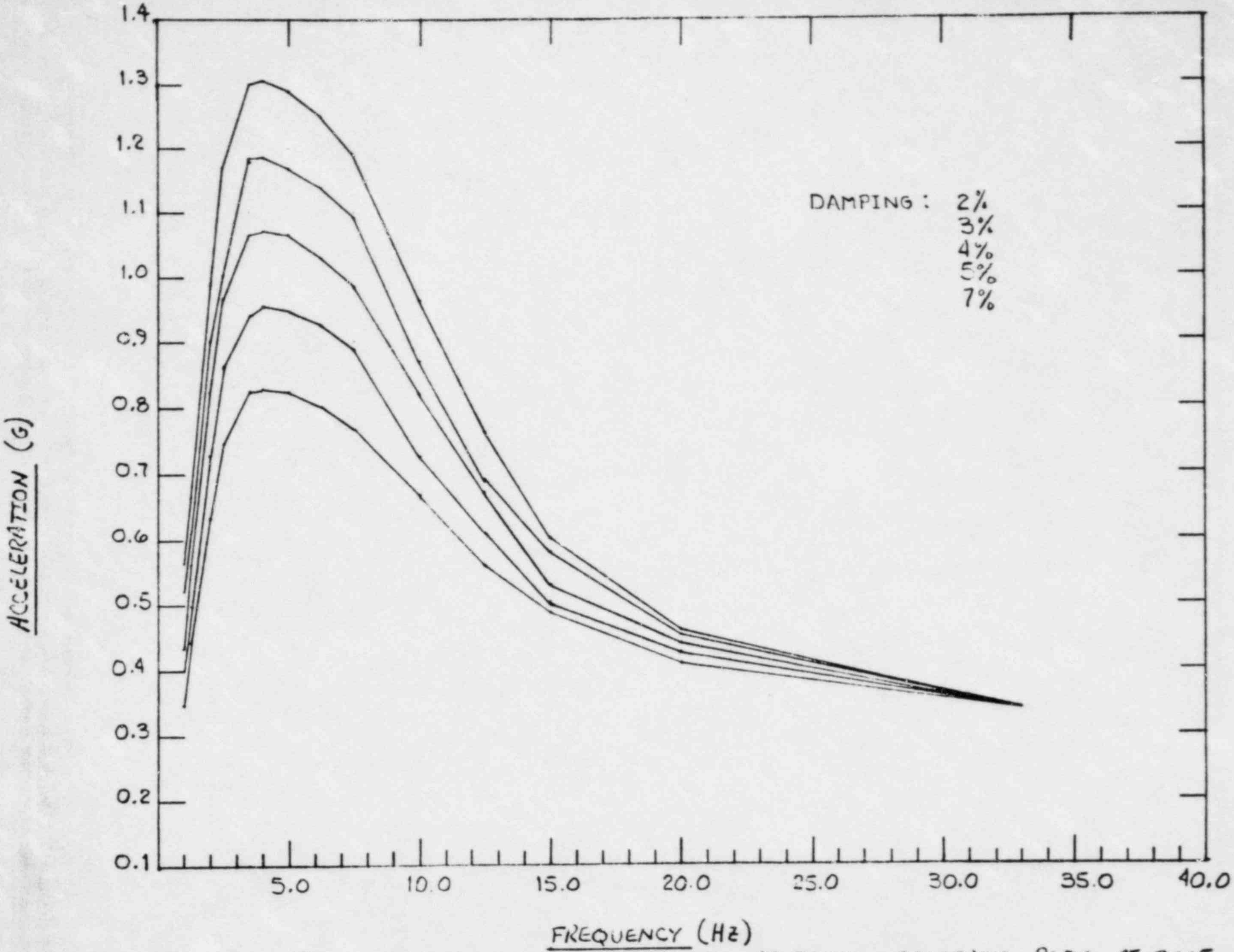
$$U = D + E$$

where:

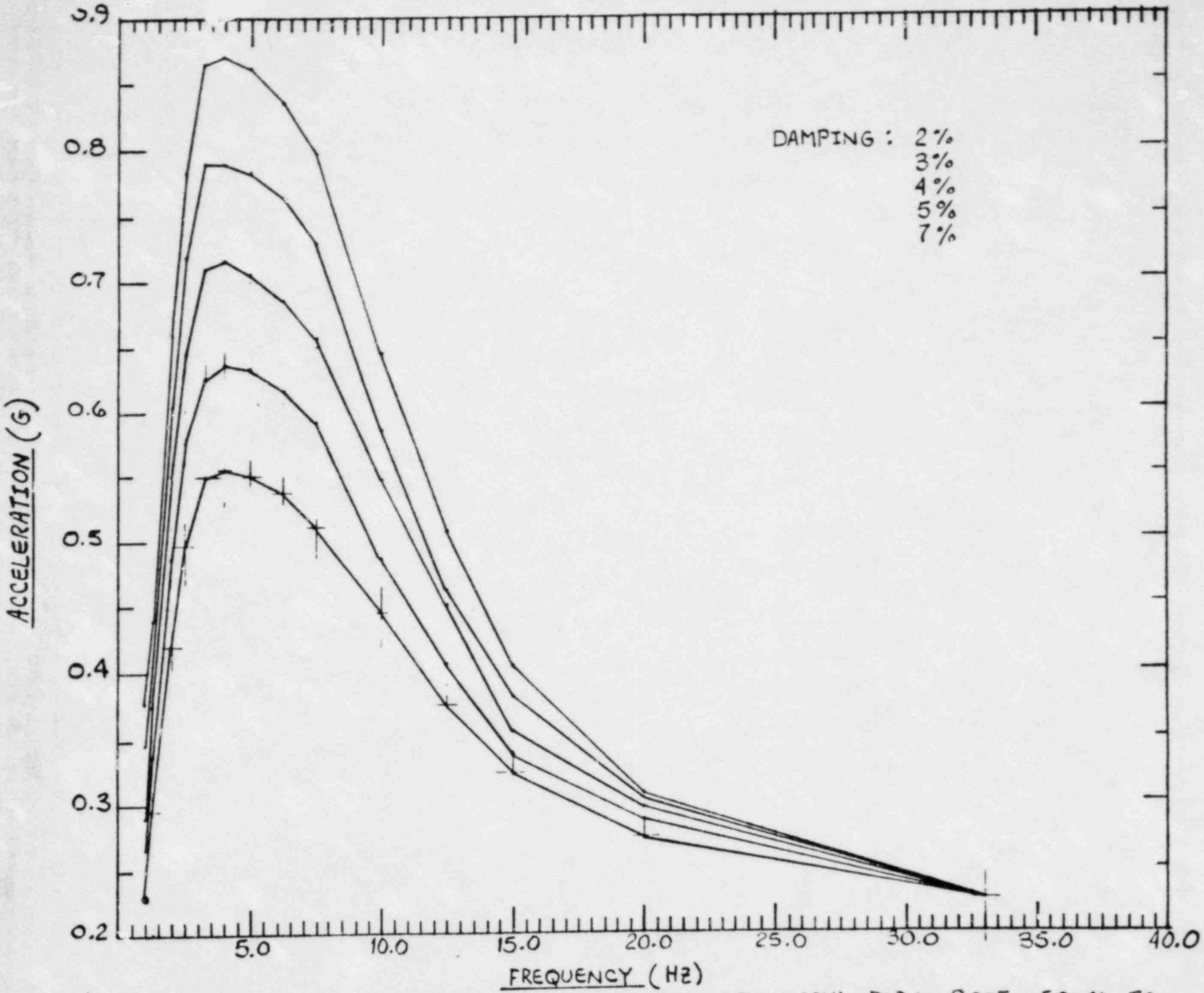
U = Load capacity of the component.

D = Dead load resulting from the pipe weight and insulation weight.

E = Load from the defined seismic event (as defined in Figures 5.2.8, and 5.2.9) which is considered representative of the roof of the diesel generator building for 4% damping.



5.2-8 HORIZONTAL RESPONSE SPECTRUM AT DIESEL GENERATOR BLDG. AT ROOF. EQUAL TO 2x GROUND RESPONSE AT ROCK SURFACE.



5.2-9 VERTICAL RESPONSE SPECTRUM AT DIESEL GENERATOR BLDG. ROOF. EQUAL TO 2x GROUND RESPONSE AT ROCK SURFACE.

5.2.4 STRESS DEFORMATION - STABILITY CRITERIA

The duct system, including its supports, is analyzed for the effect of the loads resulting from the earthquake. For service conditions the combined dead weight and seismic inertial loads must satisfy Equation 9 of Subsection NC-3652 for Class 2 pipe of ASME Section III, Division I, except that allowable stress is reduced from 2.4 S to 1.5 S or S_y

The allowable stress limit for the duct system is defined as

$$S_{all} \leq S_y;$$

which was determined from the "Allowable Stress Criteria for the Hadda Neck Plant" attached hereto.

5.2.5 METHODS OF ANALYSIS

Dynaflex was the computer program utilized in the analysis of the Diesel Exhaust Duct System. A static analysis was used for the dead weight load, and a dynamic analysis, using the response spectrum option, was performed for the seismic load. The resulting loads from the dead weight and the earthquake were then combined by absolute summation and applied to the supports to determine resultant forces and stresses. Stresses in the duct at the node points were calculated directly by the Dynaflex program.

The piping model was constructed with masses lumped at the specified node points according to the isometric drawing in Figure 5.2.7. At least two masses were lumped between supports, and the elbows were modelled with masses distributed by the program at the near, middle, and far points. The fundamental frequency of the duct system was equal to 2.89 hz. The second mode was at 4.65 hz andd the third through the seventh modes were 6.64 hz, 13.89 hz, 27.33 hz, 28.22 hz, and 32.76 hz, respectively.

Three computer runs were made for the duct system. In the first run the support at node point 9 had an upward force in the dead weight analysis. Hence, the support was not acting and another run was made to properly balance the loads between the adjacent supports in the dead weight analysis. In the second run the support at node point 9 was removed and the loads were redistributed in the dead weight analysis. However, under the earthquake load the duct at node points 7 and 8 had a net upward reaction force because the inertial force of the earthquake was greater than the dead weight force. Another computer run was made assuming the supports at node points 7, 8, and 9 were removed. The resultant loads were then redistributed during the earthquake with the adjacent vertical supports reflecting this change and with no other line segments lifting off the supports.

Several assumptions have been made to facilitate the development and implementation of the model for the analysis:

- The transition piece from the diesel generator to the duct will be included in the analysis of the diesel generator.
- The wall thickness of the duct is 3/16", which is the same thickness as indicated for the transition piece.
- The insulation thickness is 5" and weighs 53 lb/ft for the 22" duct.
- A Pathway expansion bellows with 10 convulutions and a 50 PSIG working pressure has been assumed in the analysis. The stiffness properties are 430 lb/in - axial, 5300 lb/in - lateral, and 500 in-lb/deg - rotational.
- The connection of the exhaust duct to the muffler was modelled as an unreinforced fabricated tee.

5.3 DIESEL AIR START-UP TANKS

5.4 CVCC REGENERATIVE HEAT EXCHANGER

5.5 DIESEL GENERATOR

5.6 BORIC ACID PUMP

5.7 HIGH PRESSURE SAFETY INJECTION PUMP

5.8 RHR PUMP

5.9 RHR HEAT EXCHANGER

5.10 BORIC ACID TANK

SYSTEM Mechanical - #10

COMPONENT NAME Boric Acid Tank COMPONENT № TK-2-1A

LOCATION Prim. Aux. Bldg. ELEVATION 35'-6"

COMPONENT SAFETY FUNCTION: ACTIVE ☐ PASSIVE 1 ☒ 2 ☐

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METHOD OF ANALYSIS: One Dimensional Finite Element (Beam) Multidegree of Freedom Dynamic Model. Sloshing effect done by TID-7024

SPECTRAL CURVES USED: FRS of PAB; Figures 19, 20, & 25

DAMPING VALUE ASSUMED: 7% (Impulsive), 0.5% (Sloshing)

ACCEPTANCE BEHAVIOR CRITERIA USED: $S_{all} \le S_{yield}$ or 2/3 critical buckling from Roark

COMPUTER CODE USED: SAP IV, Finite Element Multi-Purpose

REMARKS: STRUCTURE AND ANCHORAGE SATISFY ACCEPTANCE CRITERIA

S&A	DESIGN REPORT COVER SHEET	NORTHEAST UTILITIES - HADDAM NECK	REV.№	0		
			BY	JTW		
			DATE	4-1-80		
			CHK'D	FAT		
			DATE	6/17/80		
			APPR.	FAT		
			DATE	6/20		

5.10.1 INTRODUCTION

The Boric Acid Storage Tank is located at floor elevation 35'-6" in the Primary Auxiliary Building as shown in Figure 5.10-1. The tank is 17 feet in diameter and has a hemispherical bottom. The tank roof is flat and made out of lap welded stainless steel plate and structural framing which support an electric motor and mixer. Support for the tank is provided by a stiffened ring at about mid-height on the shell. This ring is bolted to the floor by 8-2 inch diameter anchor bolts. The tank shell and roof plates are 3/16" thick stainless plate and the hemispherical head is 3/8" stainless steel plate.

These calculations were made to evaluate the seismic design adequacy of the Boric Acid Storage Tank for the seismic floor response spectra input defined in Figures 5.10-2, 5.10-3 and 5.10-4.

5.10.2 RESULTS

The results of this analysis show that the Boric Acid Storage Tank will maintain its structural and leak tight integrity during the prescribed seismic disturbance defined by Figures 5.10-2, 5.10-3 and 5.10-4. Satisfactory performance for this component is defined, in the P-1 or passive one category of "Allowable Stress Criteria & Damping Values for NUSCO-Haddam Neck".

Three load cases for different assumptions of fluid level were investigated since the tank is supported near its midheight. Hence, it was not clear which condition controls design.

1. Tank full; no sloshing of fluid considered, entire mass acts impulsively.
2. Fluid height at base of support ring with all fluid and tank motion assumed impulsive.
3. Fluid height at base of support ring with all fluid assumed sloshing. Fluid and empty tank response are combined by SRSS.

The calculated natural frequencies for this tank are the following:

0.38	Hz	1st mode sloshing in half-full tank, load case 3.
6.26	Hz	Motor bouncing on tank roof. All load cases.
13.02	Hz	1st lateral shell mode, load case 1.
13.60	Hz	1st lateral shell mode, load case 2.
26.29	Hz	2nd mode of motor bouncing on roof, all load cases.
33.95	Hz	1st lateral shell mode, load case 3.

For each load case stresses were computed in the shell and anchor bolts of the tank as well as the support structure of the motor. Factors of safety were computed for combined bending and shear in the shell and combined shear and tension. These are shown in Table 5.10-1.

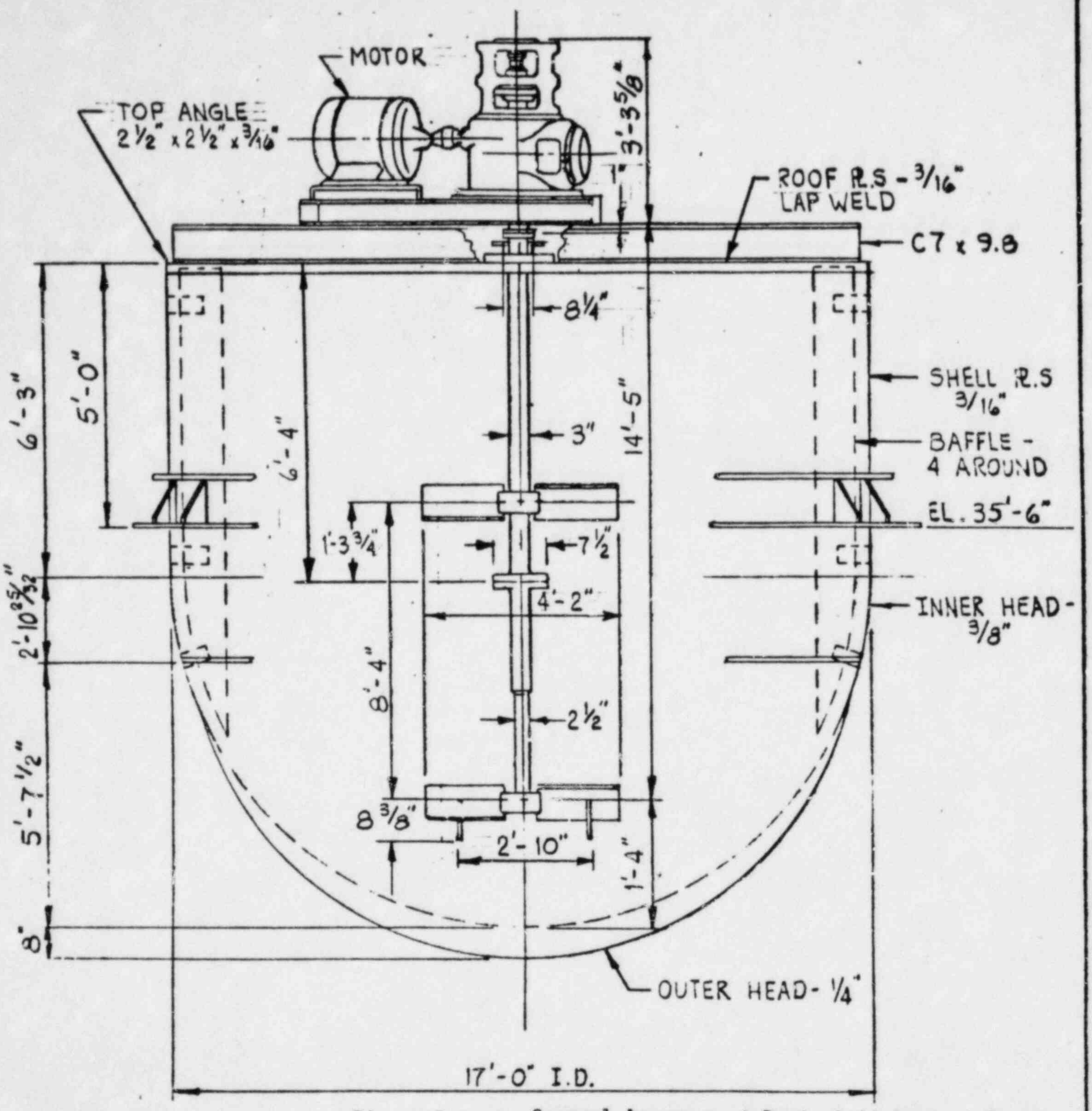


Figure 5.10-1 - General Arrangement Boric Acid Storage Tank

MATERIAL :

SHELL + HEADS - A 240 TYPE 304 SS
SUPPORT RING - A 283 C.
STEAM JACKET - A 285 C.
STRUCTURAL - A 36

MOTOR :

10 H.P. , 1800/900 RPM
WEIGHT - 265 lbs.
MIXER WT. (LESS MOTOR) - 1778 lbs.
IMPELLER RPM - 56/28
4 - BLADES

ALL MIXER PARTS IN CONTACT WITH TANK CONTENTS ARE 316 SS.

WT. FULL = 244267 lbs.
WT. EMPTY = 17307 lbs.

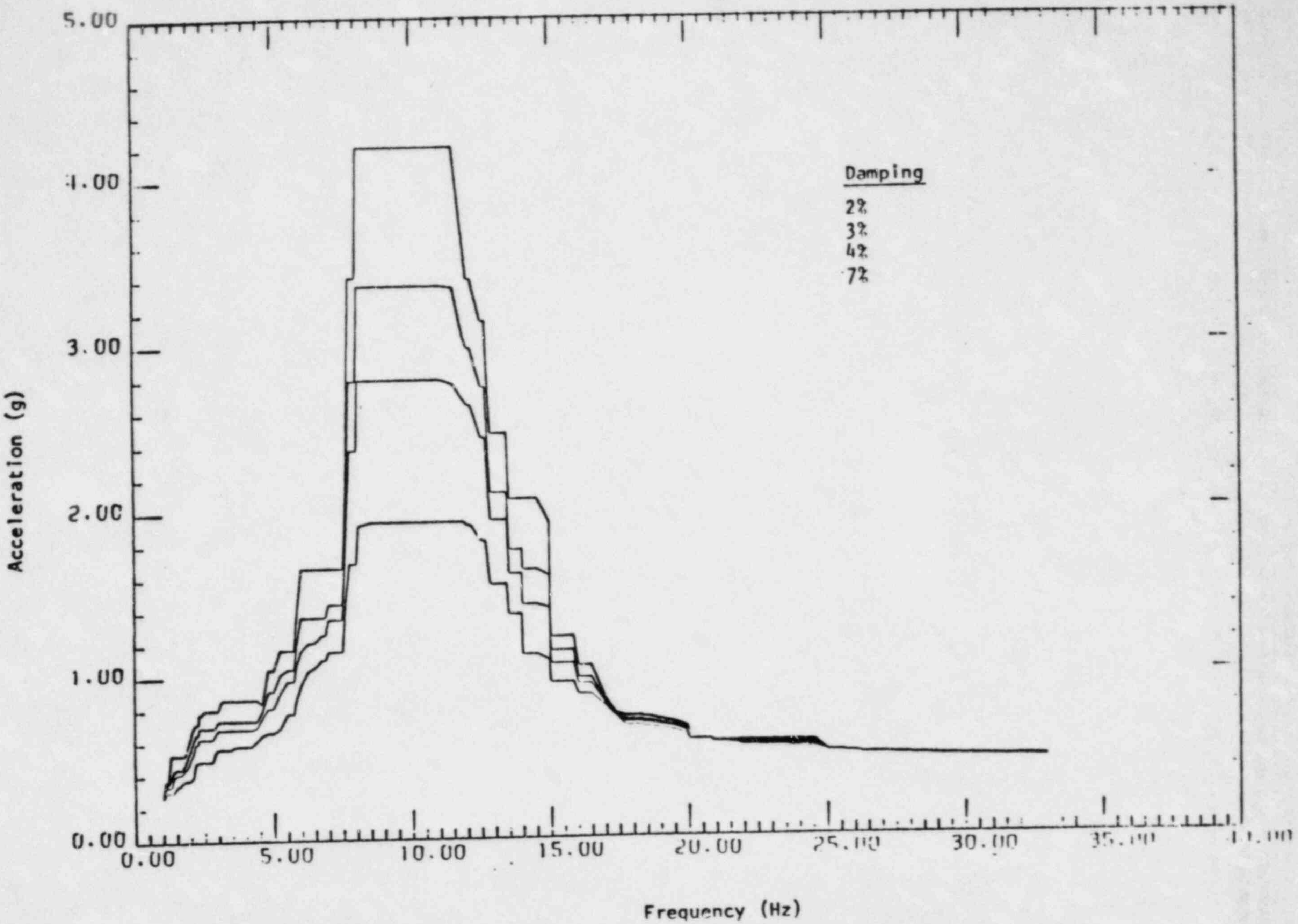


Figure 5.10-2 - North-South Floor Response Spectra For Elevation 35'-6", Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

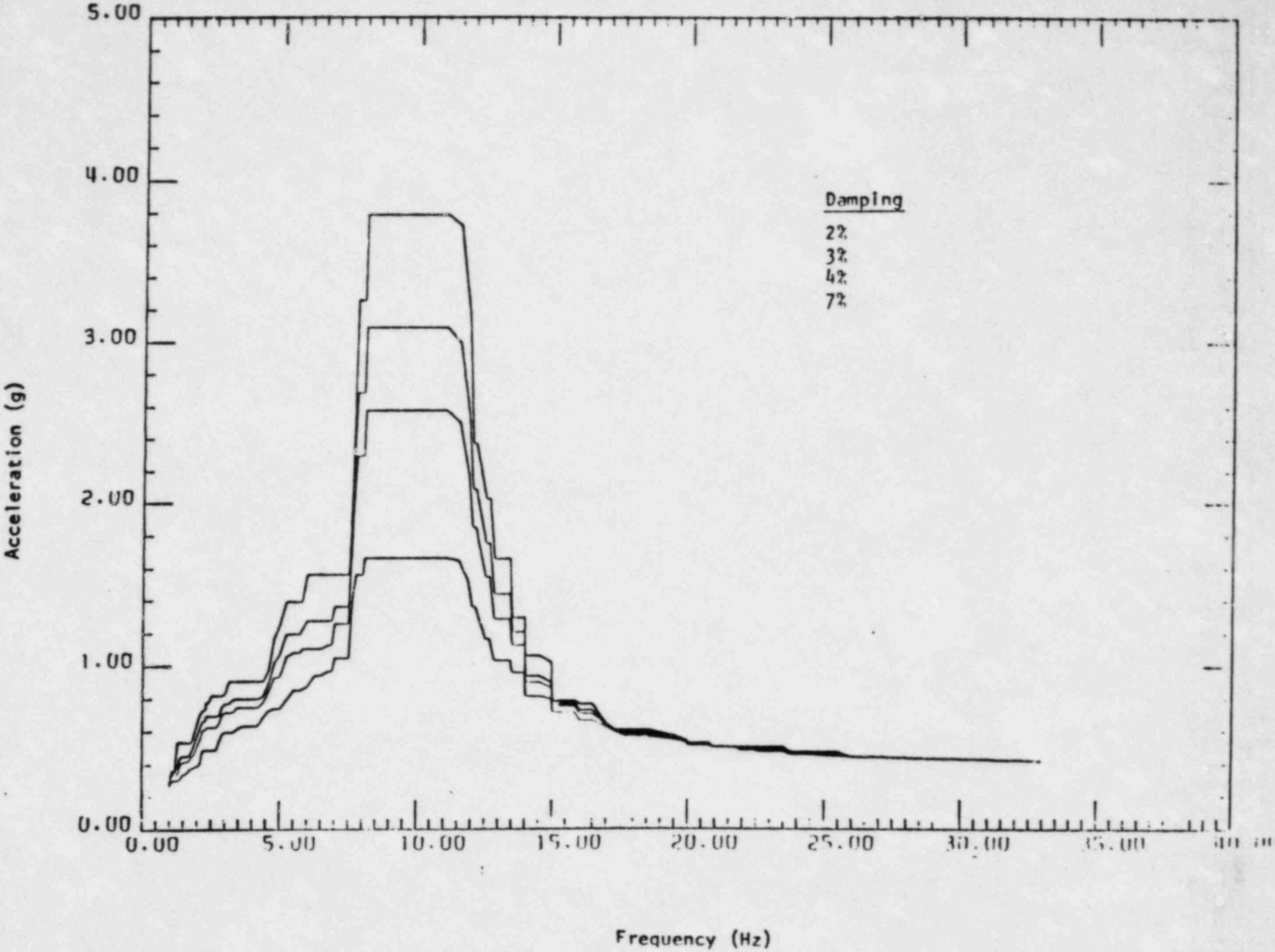


Figure 5.10-3 East-West Floor Response Spectra For Elevation 35'-6", Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

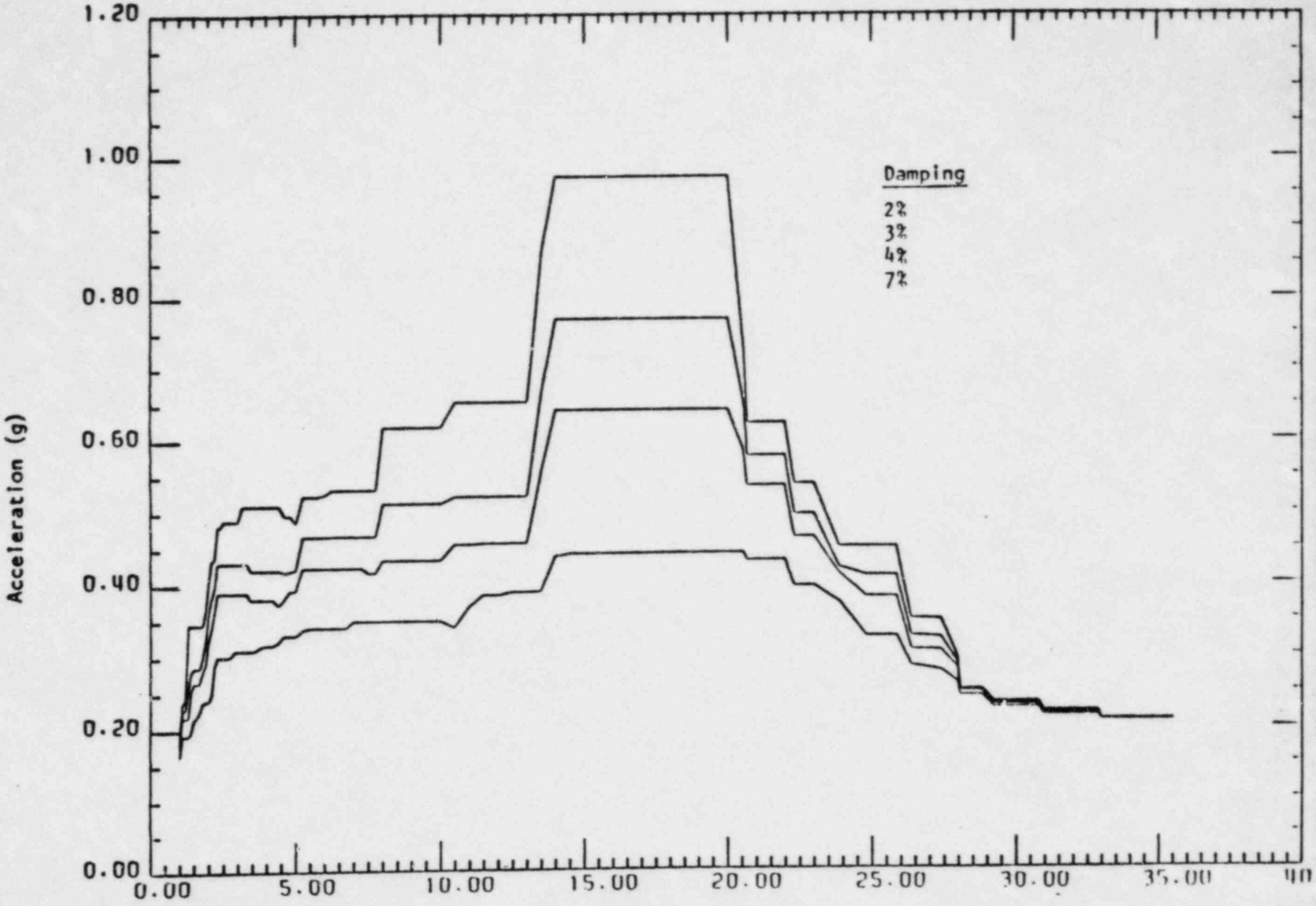


Figure 5.10-4 - Vertical Floor Response Spectra For Elevation 35'-6"
At Location 12'-6" West of Column Line J and 16'-9" South of Column Line 13½
Primary Auxiliary Bui 'ng, Connecticut Yankee Atomic Power Plant

Table 5.10-1 - Stress Resultants in Boric Acid Storage Tank Shell and Bolts

	Calculated Actual Stress				Allowable Stress[1]				Factors of Safety	
	Shell		Bolts		Shell		Bolts		Shell	Bolts
Load Case	Shear	Bending	Shear	Tension	Shear	Bending	Shear	Tension	Combined Bending & Shear	Combined Shear & Tension
1	1867	1162	4467	6822	7528	12924	16780	40600	6.67	10.08
2	2845	1783	6809	10472	7528	12924	16780	40600	3.64	4.34
3	843	555	2017	3256	7528	12924	16780	40600	17.53	47.7

All stresses in pounds/in^2

(1) Note: Allowable stresses are determined by buckling calculations

Safety factors for shear and bending in the shell plates were computed from the interaction equation:

$$F.S. = \frac{1}{\left(\frac{f_b}{F_b}\right) + \left(\frac{f_t}{F_t}\right)^2} \qquad \text{(Ref. 7.9)}$$

Safety factors for bolting followed from:

$$F.S. = \frac{1}{\left(\frac{f_v}{F_v}\right)^2 + \left(\frac{f_t}{F_t}\right)^2} \qquad \text{(Ref. 7.3)}$$

The factor of safety for lateral-torsional buckling of the main framing members on the roof is

$$F.S. = \frac{0.9F_y}{f_b} = \frac{32.4}{11.777} = 2.75$$

The maximum dynamic deflection of the roof is ± .09 inches.

5.10.3 LOAD CRITERIA AND FAILURE MODE ASSUMPTIONS

Because of the lack of available research information on tanks with hemispherical bottoms, the three load previously mentioned were considered.

Case 1. Tank full; no sloshing of fluid considered, entire mass acts impulsively.

This case was investigated because an overflow nozzle is located at 8" below the roof and the slosh height of the fluid is 21.8", which is greater than 2 times the freeboard. For this case the fluid must be treated impulsively.

Case 2. Fluid height at base of support ring with all fluid motion assumed impulsive, along with tank.

Since the fluid could be drained down at least to the base of the ring, this case was investigated because all fluid mass was on one side of the support ring. The fluid was treated as being impulsive because the exact motion of the fluid is not easily determined.

Case 3. Fluid height at base of support ring with all fluid assumed sloshing. Tank motion is impulsive and fluid and empty tank response are combined by SRSS.

In this case, the empty tank response was computed by machine and combined by hand with the sloshing fluid response by SRSS. The entire fluid mass is assumed to slosh because the exact motion of the fluid is not easily determined as stated in Case 2 also. It should be noted that the true response of the fluid is a combination of Cases 2 and 3 which at this time is difficult to determine.

No nozzle loads were considered on the Boric Acid Storage Tank. Nozzles less than 4" diameter do not normally produce loads of significant magnitude and no nozzle greater than 3" diameter enters the tank.

The response spectra used were provided by Northeast Utilities. The curves for 7% damping were used for all directions of input spectra associated with impulsive response. One-half percent damping was used for sloshing response mode. This amount of damping had negligible effect on forces produced from sloshing of the boric acid. Slight smoothing of the response spectra was done to reduce computer input time.

5.10.4 STRESS CRITERIA

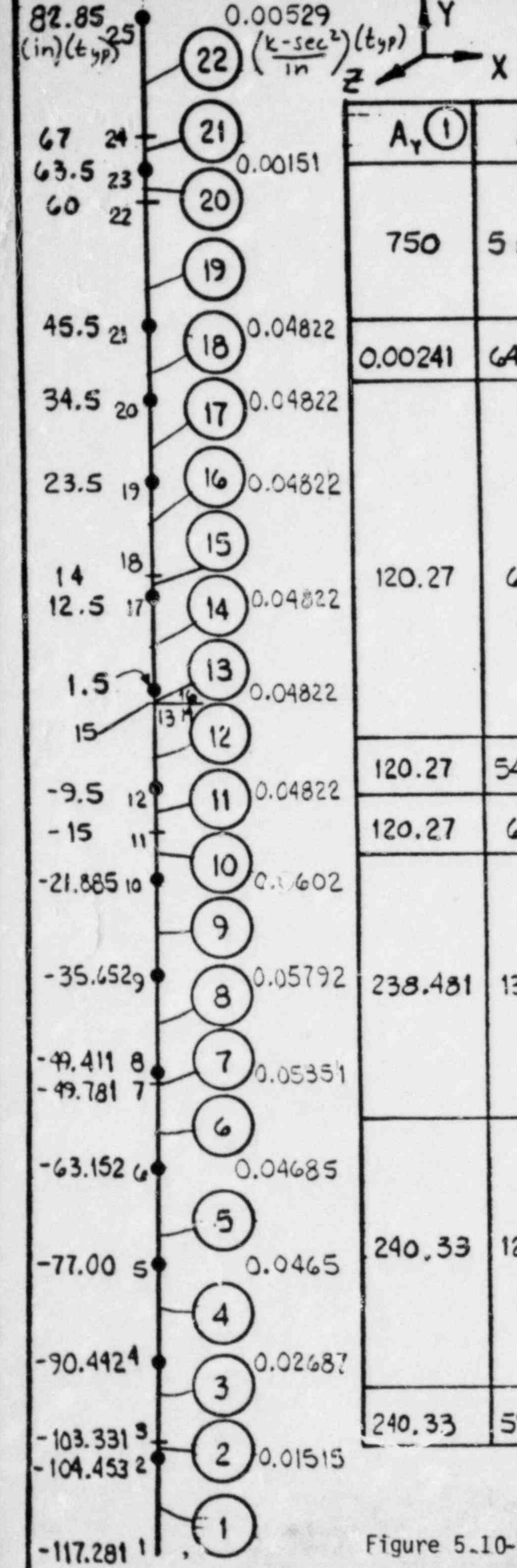
The stress criteria are as shown in Appendix A with these exceptions:

> Allowable stress in bending for the tank shell is governed by buckling criteria, S_{all} = 12.9 ksi.
>
> Allowable stress in shear for the tank shell is governed by buckling criteria, S_{all} = 7.528 ksi.

5.10.5 METHODS OF ANALYSIS

The SAP IV computer program was used for the analysis of the impulsive motion of tank and fluid and the sloshing motion was computed and combined with the computer analysis by hand. The boundary conditions for the computer model were input as being rigid.

The tank was modelled as a cylindrical beam stick with fluid mass lumped at points along the length. A sketch of the stick model for the tank analyzed is shown in Figure 5.10-5 along with the values for the section properties and lumped masses.



TANK NO. 10

A_Y ①	A_Z ②	A_X ③	I_Y ①	I_Z ②	I_X ③	ELEMENT NO.
750	5×10^5	5×10^5	200×10^6	100×10^6	100×10^6	㉒
0.00241	64.743	84.315	170×10^6	85×10^6	85×10^6	⑳ ㉑
120.27	60.14	60.14	1.25×10^6	6.27×10^5	6.27×10^5	⑲ ⑱ ⑰ ⑯
120.27	5410.27	5410.27	491×10^6	245×10^6	245×10^6	⑬ ⑭ ⑮
120.27	60.14	60.14	1.25×10^6	6.27×10^5	6.27×10^5	⑪ ⑫
238.481	137.963	137.963	2.487×10^6	1.24×10^6	1.24×10^6	⑩ ⑨ ⑧ ⑦
240.33	121.831	121.831	2.5×10^6	1.25×10^6	1.25×10^6	⑥ ⑤ ④ ③
240.33	59.471	59.471	2.5×10^6	1.25×10^6	1.25×10^6	① ②

Figure 5.10-5 Analytical Stick Model of Boric Acid Tank

5.11 DEMINERALIZED WATER STORAGE TANK

5.12 REFUELING WATER STORAGE TANK

SYSTEM MECHANICAL #12

COMPONENT NAME Refueling Water Str Tank **COMPONENT №** TK-4-1a

LOCATION YARD, NORTHEAST OF CNTNMT. **ELEVATION** 23'-0 5/16

COMPONENT SAFETY FUNCTION: ACTIVE ☐ **PASSIVE** 1 ☒ 2 ☐

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METHOD OF ANALYSIS: One dimensional finite element Multidegree of Freedom. Dynamic Model. Sloshing motion analyzed by TID-7024. Soil-Structure Interaction Represented by springs

SPECTRAL CURVES USED: GROUND RESPONSE AT ROCK LEVEL

DAMPING VALUE ASSUMED: 7% IMPULSIVE, .5% SLOSHING

NO SOIL DAMPING CONSIDERED EXPLICITLY

ACCEPTANCE BEHAVIOR CRITERIA USED:

$S_{allowable} \le S_{yield}$

COMPUTER CODE USED: SAP IV

REMARKS: ADDITIONAL ANCHORAGE IS NEEDED FOR THE TANK TO PASS THE ACCEPTANCE CRITERIA

S&A	DESIGN REPORT COVER SHEET	NORTHEAST UTILITIES – HADDAM NECK	REV. №	0		
			BY	JTW		
			DATE	7-17-8		
			CHK'D	EAT		
			DATE	7-17-8		
			APPR.	EAT		
			DATE	6/20		

5.12.1 INTRODUCTION

The Refueling Cavity Water Storage Tank TK4-1A, is located in the yard, northeast of the containment, at grade elevation. An elevation of the RCWS tank is shown in Figure 5.12-1. The tank is 35'-0 in diameter and 36'-0 from grade to top roof support angle. The tank is covered by a flat dome roof of 35'-0 radius. The plate material for this tank is 5052-F aluminum. Anchorage consists of 8-1" diameter A 36 threaded rods embedded in the slab foundation and bolted to aluminum anchor bolt chairs as shown in Figure 5.12-2. The maximum liquid height is 35'-0 from the bottom of the tank.

The foundation consists of a 4'-10" thick octagonal concrete slab which sits on a backfill of clean sand above rock. The distance from the bottom of slab to the rock surface is approximately 1.8 ft. However, the more conservative 6.0 ft dimension was used in the analysis.

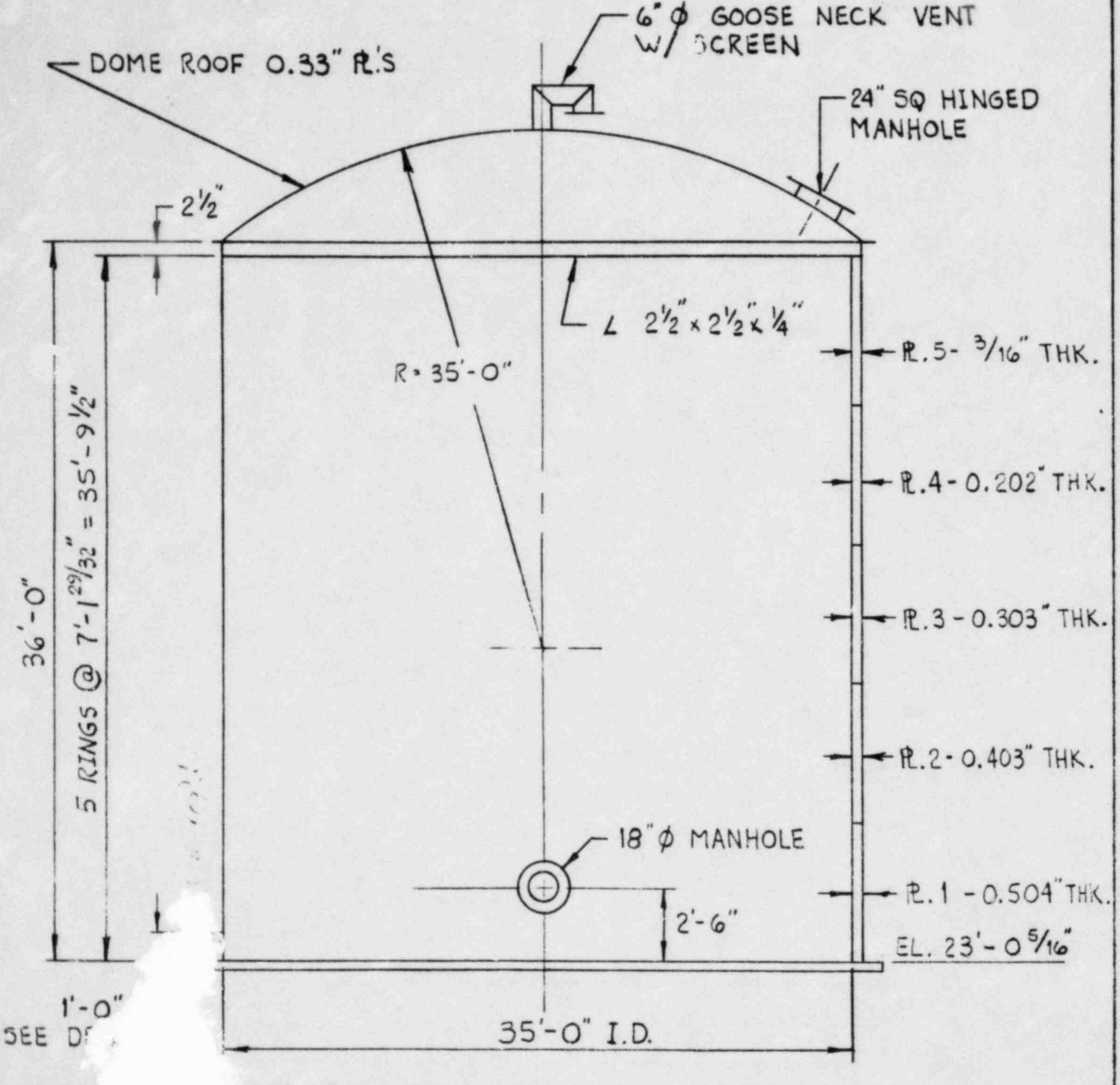
These calculations were made to evaluate the seismic design adequacy of the Refueling Cavity Water Storage Tank for the seismic response spectra input defined in Figures 5.12-3 and 5.12-4.

5.12.2 RESULTS

The results of this analysis show that the anchorage of the Refueling Cavity Water Storage Tank is not adequate to resist the forces and moments produced by the prescribed seismic disturbance defined by Figures 5.12.3 and 5.12.4. The anchor bolts will fail in tension from flexible tank and fluid overturning effects. It should be noted that failure of the anchorage does not necessarily mean that this tank will not maintain structural and leak tight integrity, as defined in category P-1 of "Allowable Stress Criteria & Damping Values for NUSCO Haddam Neck." However, the probability of the tanks leaking or the shell buckling is increased due to this anchorage failure. After anchor bolt failure the tank must be treated as being capable of uplift of the tank bottom from the overturning effects may occur. Seismic tests and observed tank response as the result of earthquakes have shown that greatly increased compressive stresses appear in the tank shell on the sides opposite to the uplift. In addition the welds between the tank bottom plate and wall cylinder would be required to transfer very large bending moments not considered in design as the result of lift off.

The potential for soil-strcucture interaction is also considered in evaluating the tank using the procedure defined in Ref. 7.10. However, since the tank bottom is within 6'-0" of the rock surface (1'-2") from bottom of concrete base the effect of soil-structure interaction is negligible as can be seen from the stiffness of the computed spring stiffness shown in 5.12-6b. The procedure used to determine soil-spring constants is given in Appendix 5.12A. Further, it should be noted that the dominate frequency for impulsive response of the tank is approximately 6 H_z which is very close to the peak of the applicable response spectra.

The tank was analyzed treating some of the fluid as impulsive and moving with the tank and some of the fluid as sloshing and oscillating at a much lower natural frequency.



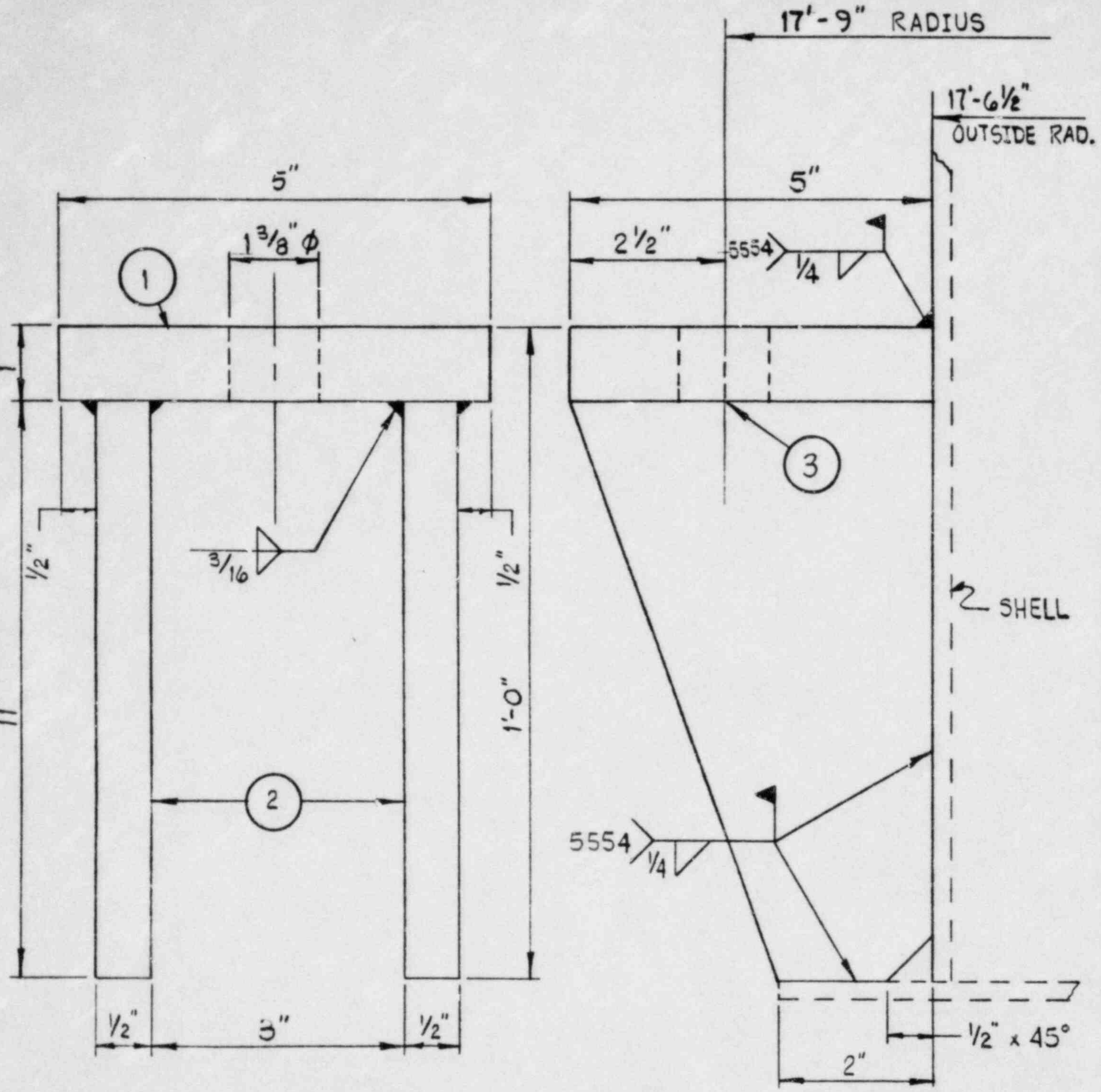
MATERIAL

PLATES - AL 5052 - F
STRUCTURAL - 6061 - T6

WEIGHT

EMPTY - 26,053 lbs.
FULL 2,127,307 lbs.

Figure 5.12-1 Refueling Water Tank Dimensional Properties



No.	DESCRIPTION & DIMENSION
1.	5083 ALUMINUM PL. 5"x5"x1". 8 REQ'D.
2.	5052 ALUMINUM PL. SK x ½". 16 REQ'D.
3.	1" ϕ ANCHOR BOLTS, NUTS & WASHERS.

Figure 5.12-2 Refueling Water Storage Tank Anchor Bolt Chair Assemblies

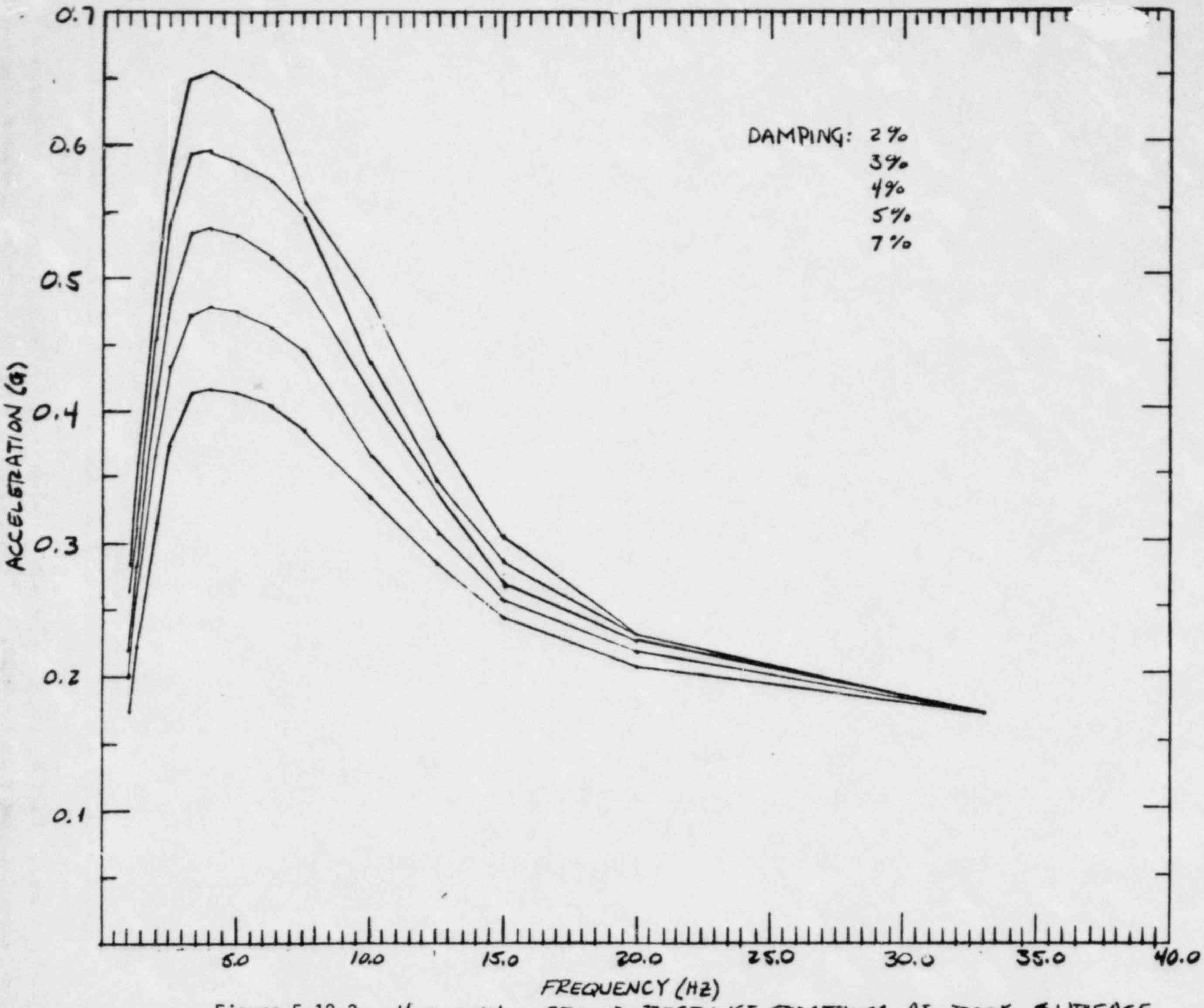


Figure 5.12-3 HORIZONTAL GROUND RESPONSE SPECTRUM AT ROCK SURFACE

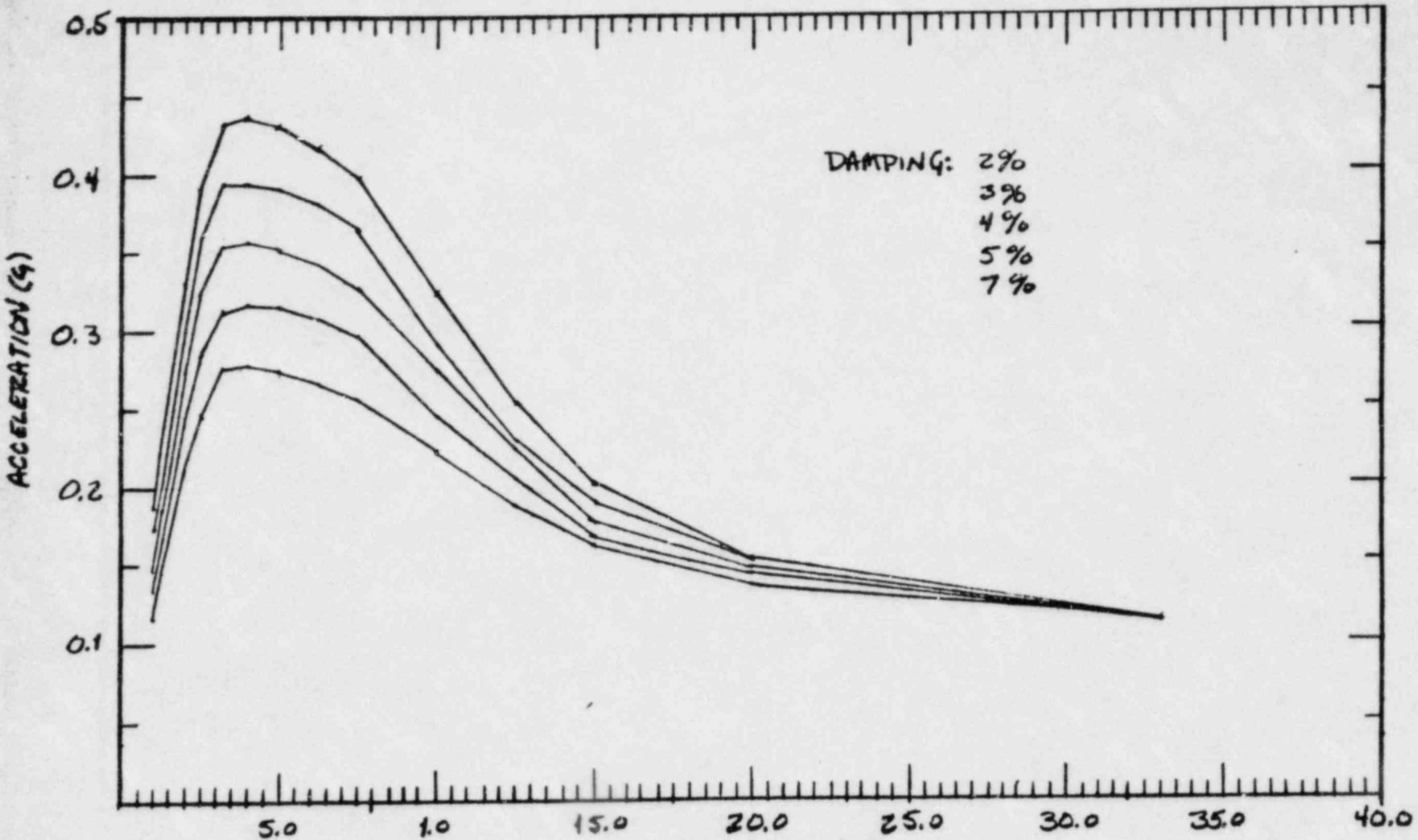


Figure 5.12-4 VERTICAL GROUND RESPONSE SPECTRUM AT ROCK SURFACE

The calculated natural frequencies for this tank are:

0.29	Hz	1st mode sloshing of fluid in tank.
5.88	Hz	1st mode of horizontal impulsive fluid and tank motion.
15.59	Hz	2nd mode of horizontal impulsive fluid and tank motion.
25.93	Hz	3rd mode of horizontal impulsive fluid and tank motion.
35.84	Hz	4th mode of horizontal impulsive fluid and tank motion.

Total overturning moment at base of tank

166767 k-in.

Total resisting moment at base of tank

395104 k-in.

No rigid body rotation of the tank will occur but there may be uplift after failure of the anchor bolts.

The tank shell was checked for buckling due to shear and bending with the following results given in Table 5.12-1.

Table 5.12-1 Summary of Tank Shell Stresses, psi

Computed Stress		Allowable Stress (1)		Facter of Safety
Shear	Bending	Shear	Bending	Combined Shear & Bending
1492	1663	3469	2099	1.72

The anchor bolts and anchor bolt chairs were checked for tension. The anchor bolts are assumed to be 36 ksi yield stress A 36 steel with an ultimate strength of 58 ksi material and the weld metal for the aluminum anchor bolt chairs is assumed to be 40 ksi yield stress. Stresses, shown in Table 5.12-2, are for the bolts and bolt chairs located in Figure 5.12-5. In this analysis it is assumed that the shear on the base of the tank is transferred to the foundation by friction between the tank bottom and foundation.

Table 5.12-2 Summary of Anchorage Stresses, psi

	Computed Stress		Allowable Stress		Factor of Safety	
	Bolt	Bolt Chair	Bolt	Bolt Chair	Bolt	Bolt Chair
A	160130	24500	40600	24000	FAILS	FAILS
B	145500	22266	40600	24000	FAILS	1.077
C	104310	15961	40600	24000	FAILS	1.504
D	44800	6856	40600	24000	FAILS	3.5

In calculating bolt loads the stabalizing effect of the weight of the fluid and the relative stiffness of the tank bottom plate has not been considered hence results shown are quite conservative. Any detailed redesign of the anchorage will consider all applicable loads and compatible structural response.

---

(1) allowable stresses are computed from buckling criteria in Ref. 7.9

$$FS = \frac{1}{\left(\frac{fb}{Fb}\right) + \left(\frac{fv}{Fv}\right)^2}$$

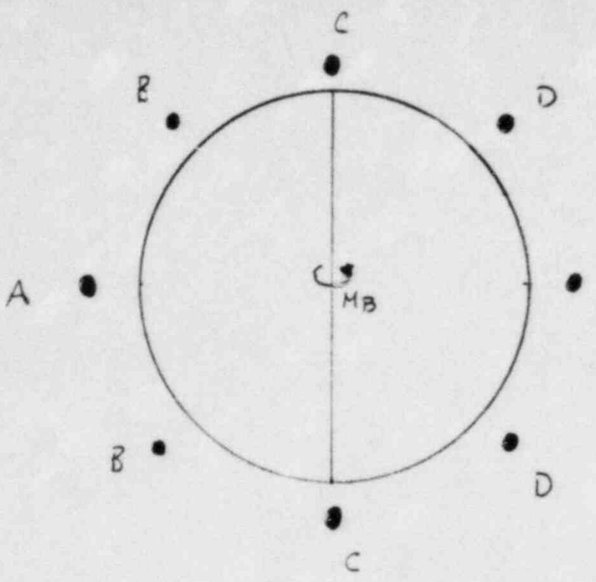


FIGURE 5.12-5 Anchorage Layout

The total hydrodynamic and hydrostatic pressure on the tank shell was computed and hoop stresses were calculated at the bottom of each tank shell ring as shown in Table 5.12-3.

TABLE 5.12-3 - Hydrodynamic and Hydrostatic Pressure and Resultant Stresses

	PLATE THICKNESS INCHES	DEPTH IN FLUID INCHES	P_T, PSI	HOOP STRESS KSI	ALLOWABLE STRESS KSI (Tension)
RING 5	.1875"	76.375"	.402	4.505	10
RING 4	.202"	162.28125"	7.275	7.563	10
RING 3	.303"	248.1875"	11.074	7.674	10
RING 2	.403"	334.09375"	15.24	7.941	10
RING 1	.504"	420"	19.17	7.988	10

$$\text{Hoop Stress} = \frac{P_T\, r}{t}$$

$$P_T = (P_1^2 + P_2^2 + P_V^2)^{1/2} + P_{STATIC} \quad \text{(Ref. 7.7)}$$

where:

P_1 = impulsive motion hydrodynamic pressure
P_2 = sloshing motion hydrodynamic pressure
P_V = vertical hydrodynamic pressure

5.12.3 LOAD CRITERIA AND FAILURE MODE ASSUMPTION

Failure of the tank anchorage was considered to be the area of greatest concern because of the analytical methods used in the original design.

The ground response spectra was used in the analysis of the tank. The basic ground response spectra is considered to be representative of accelerations at the rock surface, which is 6 feet below the foundation of the tank. The effect of intervening soil was included by the use of soil springs. Curves for seven percent damping were used with impulsive mode response. One-half percent damping was used for the sloshing mode of response.

Nozzle loads on the tank were assumed to produce limiting stresses in the nozzles of values equal to $S_{all}/3$. These loads were compared with the loads on the tank anchorage system from the earthquake. The largest nozzle induced load was .24% of the overturning movement on the base of the tank determined from the earthquake. Because of the small values determined, the effect of nozzle loads on tank supports were not combined with earthquake loads.

5.12.4 STRESS CRITERIA

The stress criteria are as shown in Appendix A with these exceptions:

Allowable stress in bending for the tank shell is governed by buckling criteria, S_{all} = 2.099 ksi.

Allowable stress in shear for the tank shell is governed by buckling criteria, S_{all} = 3.469 ksi.

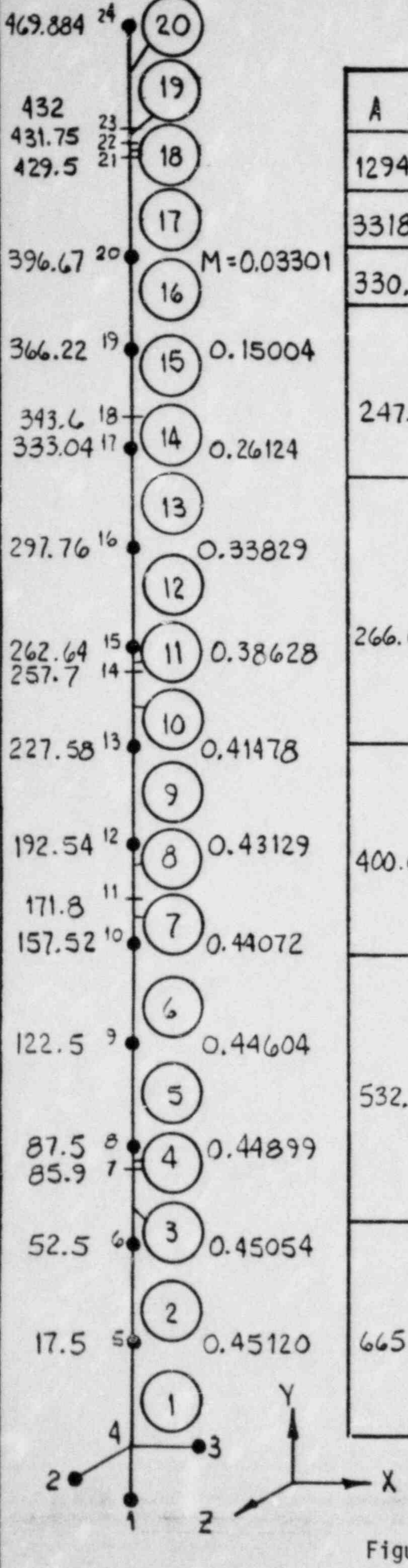
5.12.5 METHODS OF ANALYSIS

Since the tank is supported on compacted backfill, the effect of soil structure interaction was considered in the modeling of the tank. Equivalent soil springs were computed for vertical, lateral and rocking motions of the tank. A torsional spring about a vertical axis was considered very stiff and the largest value for spring constants in the computer program was used. The computed equivalent soil springs, because of the proximity of the rock to the surface, were also very stiff. These springs had a minimal effect on the tank's fundamental frequency of vibration compared to a rigid base modeling assumption. The fundamental frequency of the tank system model is approximately 6 Hz which is very near the peak of the response spectrum and produced the large loads on the anchorage. Any error in response frequency introduced by actual support conditions may tend to move the response away from the peak and, therefore, reduce resultant stresses and loads.

The SAP IV computer program was used for the analysis of the impulsive motion of the tank and fluid, and the sloshing motion was computed and combined with the computer analysis by hand calculations.

The boundary elements for the computer model were input with the approximate soil spring stiffness in each direction. The tank was modelled as a cylindrical beam stick with fluid mass lumped at points along the length. A sketch of the computer stick model is shown in Figure 5.12-6 along with the values for the section properties and lumped masses.

TANK Nº. 12 - DYNAMIC MODEL



A IN²	A_{vz}	A_{vx}	I_Y	I_z	I_x	ELEMENT Nº.
1294.83	1294.83	1294.83	56.86×10^6	28.42×10^6	28.42×10^6	⑳
3318.31	1659.15	1659.15	148.09×10^6	74.04×10^6	74.04×10^6	⑲
330.06	165.032	165.032	14.57×10^6	7.29×10^6	7.29×10^6	⑱
247.511	123.755	123.755	10.93×10^6	5.46×10^6	5.46×10^6	⑰ ⑯ ⑮
266.661	133.33	133.33	11.77×10^6	5.88×10^6	5.88×10^6	⑭ ⑬ ⑫ ⑪
400.088	200.044	200.044	17.67×10^6	8.83×10^6	8.83×10^6	⑩ ⑨ ⑧
532.256	266.128	266.128	23.52×10^6	11.76×10^6	11.76×10^6	⑦ ⑥ ⑤ ④
665.81	332.905	332.905	29.43×10^6	14.72×10^6	14.72×10^6	③ ② ①

Figure 5.12 -6a Refueling Water Tank Dynamic Model

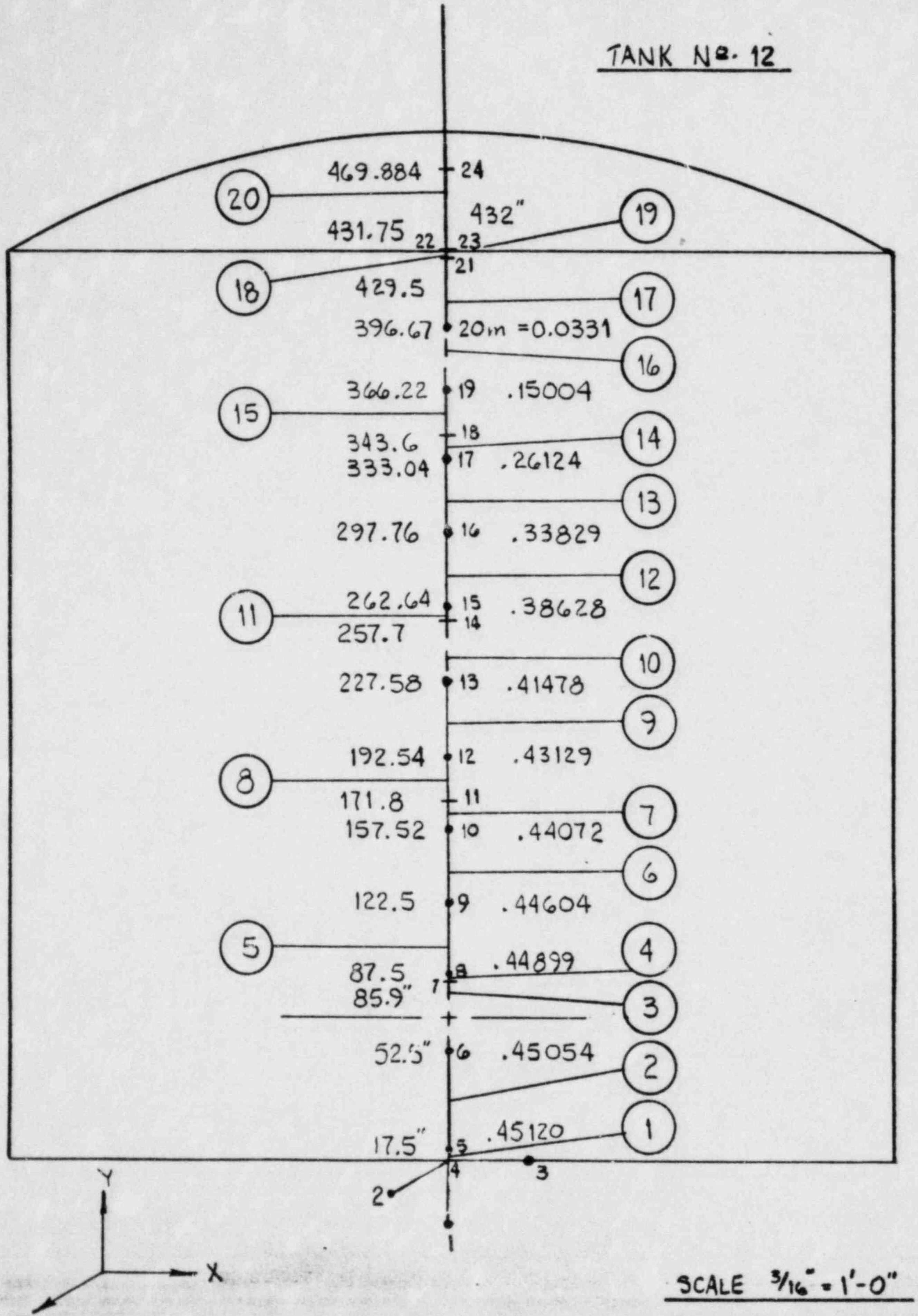


Figure 5.12-6b Refueling Water Tank Dynamic Model

5.12 A APPENDIX

Soil - Structure Interaction For Refueling Cavity Water Tank

5.12.A APPENDIX

Soil - Structure interaction was modelled for the tank by assuming a rigid circular disk for the foundation and applying equivalent soil springs to the boundary elements.

1. Soil Constants Supposition (Assume)

	V_s (ft/s)	p (lb/in^3)	E (psi)	v	G (psi)
Sand	2000 *1	0.061 *1		0.40 *1	9.09×10^4
Rock (granite)			11×10^6 *2	0.40 *2	3.93×10^6

*1 Assumption

*2; the average value of granite from Values for Rock computed from compressibility measurements by Brace (1966) at confining stress of 3-5 Kilobars, Lambe, Whitman Soil Mechanics, 1969

$$G_{sand} = \frac{pV_s^2}{g}$$

$$= \frac{0.061\ (200 \times 12)^2}{376.4}$$

$$= 9.09 \times 10^4 \text{ (psi)}$$

$$G_{rock} = \frac{E}{2(1+v)}$$

$$= \frac{11 \times 10^6}{2(1+0.40)} = 3.93 \times 10^6 \text{ (psi)}$$

where,

V_s = shear velocity

p = density

E = Young's Modulus

v = Poisson's Ratio

G = Shear Modulus

The following calculations are from Reference 7.10

2. Rotational Soil Spring

considered as semi-infinite elastic continuum.

In case of vertical loading the displacement w is

$$W = \frac{P}{4\pi G} \cdot \frac{1}{R} \left[\frac{Z^2}{R^2} + 2(1-\nu) \right]$$

and

$$\theta = \frac{\partial W}{\partial X} = \frac{P}{4\pi G} \left[\frac{3 \cdot X \cdot Z^2}{R^5} + \frac{X}{R^3} \cdot 2 \cdot (1-\nu) \right]$$

where

P = force on the surface
(x,y,z) = coordinates of the point
R = length between the point and loading point

$$R = (x^2 + y^2 + z^2)^{1/2}$$

in case of rotational loading applied the above formula

the stress $\sigma_z = M_y \cdot X / J_y$

$$\theta = \frac{1}{4\pi G} \cdot \frac{M_y}{J_y} \iint X \cdot \left[\frac{3 \cdot X \cdot Z^2}{R^5} + \frac{X}{R^3} \cdot 2 \cdot (1-\nu) \right] dX \cdot dY$$

$$= \frac{1}{4\pi G} \cdot \frac{M_y}{J_y} \int_0^{2\pi} \int_0^{a} \left[\frac{3 \cdot Z^2 \cdot r^2 \cdot \cos^2\theta}{(r^2 + Z^2)^{5/2}} + \frac{r^2 \cos^2\theta}{(r^2 + Z^2)^{3/2}} \cdot 2 \cdot (1-\nu) \right] \cdot r \cdot dr \cdot d\theta$$

$$= \frac{1}{4\pi G} \cdot \frac{M_y}{J_y} \cdot \pi \cdot \left[2Z - \frac{3 \cdot Z^2}{\sqrt{a^2 + Z^2}} + \frac{Z^4}{(a^2 + Z^2)^{3/2}} + 2 \cdot (1-\nu) \left\{ \frac{2Z^2 + a^2}{\sqrt{a^2 + Z^2}} - 2 \cdot Z \right\} \right]$$

$$\left(\begin{array}{l} \because \displaystyle\int_0^a \frac{r^3}{(r^2 + z^2)^{5/2}} dr = \frac{2}{3} \cdot \frac{1}{z} - \frac{1}{\sqrt{a^2 + z^2}} + \frac{1}{3} \frac{z^2}{(a^2 + z^2)^{3/2}} \\ \displaystyle\int_0^a \frac{r^3}{(r^2 + z^2)^{3/2}} dr = \frac{2z^2 + a^2}{\sqrt{a^2 + z^2}} - 2 \cdot z \end{array} \right)$$

a = radius
z = depth

on the surface (Z = 0)

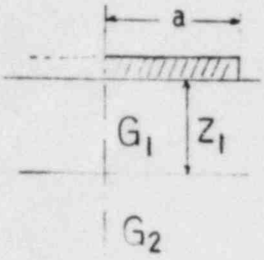
$$\theta_0 = \frac{1-\nu}{2G} \cdot \frac{M_Y}{J_Y} \cdot a$$

So, θ divided by θ_o, that is the amplification function F

$$F_R(Z/a) = \frac{1}{2a(1-\nu)} \cdot \left[2Z - \frac{3 \cdot Z^2}{\sqrt{a^2+Z^2}} + \frac{Z^4}{(a^2+Z^2)^{3/2}} + 2(1-\nu) \left\{ \frac{2 \cdot Z^2 + a^2}{\sqrt{a^2+Z^2}} - 2 \cdot Z \right\} \right]$$

$$= \frac{1}{2(1-\nu)} \cdot \left[2 \cdot \left(\frac{Z}{a}\right) - \frac{3 \cdot \left(\frac{Z}{a}\right)^2}{\sqrt{1+\left(\frac{Z}{a}\right)^2}} + \frac{\left(\frac{Z}{a}\right)^4}{\left\{1+\left(\frac{Z}{a}\right)^2\right\}^{3/2}} + 2(1-\nu) \left\{ \frac{2\left(\frac{Z}{a}\right)^2 + 1}{\sqrt{1+\left(\frac{Z}{a}\right)^2}} - 2 \cdot \left(\frac{Z}{a}\right) \right\} \right]$$

In this model



a = 210"
z_1 = 72" (Assume)

$$\theta = \theta_1 \cdot \left\{ 1 - F_R(Z_1/a) + (G_1/G_2) \cdot F_R(Z_1/a) \right\}$$

θ_1 = the disp. in the case of semi-infinite soil of G_1

$$F_R(Z_1/a) = F_R(72/210) = 0.779$$

$$\theta = \theta_1 \times \underbrace{0.239}_{\text{amplification factor}}$$

equivalent shear velocity, eV_S

$eV_S = 2000 / (0.239)^{1/2} = 4091$ ft/sec

equivalent shear modulus, eG

$eG = 9.09 \times 10^4 / 0.239 = 3.803 \times 10^5$ (psi)

Rotational Soil Spring K_θ

$$K_\theta = \frac{8\ eG\ a^3}{3(1-v)} \quad \text{(by Borowicka)}$$

v = 0.40 Poisson's Ratio

$= 1.57 \times 10^{13}$ lb in/rad

3. Horizontal Soil Spring

Horizontal displacement $\overline{U}_x$ surface force; $\tau_{xz}\,dx\,dy$

$$\overline{U}_x = \frac{\tau_{xz}\,dx\,dy}{4\pi G}\cdot\frac{1}{R}\left[\frac{X^2}{R^2}+1+\frac{(1-2\nu)\cdot R}{R+Z}\cdot\left(1-\frac{X^2}{R(R+Z)}\right)\right]$$

So, when force is uniform stress distribution for circular base

$$U_x = \int_0^{2\pi}\int_0^{a}\overline{U}_x\cdot r\cdot dr\cdot d\theta$$

$$= \frac{\tau_{xz}\cdot a}{2G}\cdot\left[\frac{1}{2}\,\frac{3+4\zeta^2}{\sqrt{1+\zeta^2}}-2\zeta+\frac{1-2\nu}{2}\left(\sqrt{1+\zeta^2}-\zeta\right)\right] \qquad \zeta=\frac{Z}{a}$$

On the surface ($Z=\zeta=0$)

$$U_{x0} = \frac{\tau_{xz}\cdot a}{2G}(2-\nu)$$

$$F_H(\zeta) = \frac{1}{(2-\nu)}\left[\frac{1}{2}\cdot\frac{3+4\zeta^2}{\sqrt{1+\zeta^2}}-2\zeta+\frac{1-2\cdot\nu}{2}\left(\sqrt{1+\zeta^2}-\zeta\right)\right]$$

$$U_x = U_{x1}\left\{1-F_H(Z_1/a)+(G_1/G_2)\cdot F_H(Z_1/a)\right\}$$

$$F_H(Z_1/a) = F_H(72/210) = 0.642$$

$$\therefore\ U_x = U_{x1}\times 0.373$$

$${}_eV_S = 2000/\sqrt{0.373} = 3275 \text{ ft/s}$$

$${}_eG_H = 9.09\times10^4/0.373 = 2.437\times10^5 \text{ psi}$$

Horizontal Soil Spring K_H

$$K_H = \frac{32\,(1-v)\,eG_Ha}{7-8v}$$

$$= 2.59\times10^8 \text{ lb/in}$$

4. Vertical Soil Spring

As mentioned in the paragraph 2., the vertical displacement is expressed

$$W = \frac{P}{4\pi G} \cdot \frac{1}{R} \cdot \left[\frac{Z^2}{R^2} + 2(1-\nu) \right]$$

As the results, F_V is expressed as follows,

$$F_V(z/a) = \sqrt{1+(z/a)^2} - \frac{1}{2(1-\nu)} \left\{ \frac{(z/a)^2}{\sqrt{1+(z/a)^2}} + (1-2\nu) \cdot (z/a) \right\}$$

$$F_V(72/210) = 0.343$$

$$\therefore \; W = W_1 \cdot \left\{ 1 - F_V(z_1/a) + (G_1/G_2) \cdot F_V(z_1/a) \right\}$$

$$= W_1 \cdot 0.114$$

equivalent shear velocity eV_s for the vertical

$eV_s = 2000/(.114)^{1/2} = 5923$ ft/sec

equivalent shear modulus eG for the vertical

$eG = 9.09 \times 10^4/0.114 = 7.974 \times 10^5$ (psi)

Vertical Soil Spring, K_V

$$K_V = \frac{4G \cdot a}{1-v}$$ (by Timochenko & Goodier 1951)

$= 1.12 \times 10^9$ lb/in

The rotational spring constant, K_θ, is larger than 10^{10} K-in/radian and not acceptable to SAP IV input. It was set equal to 10^{10} K-in/radian A similar value was selected for rotional stiffness about the vertical axis.

5.13 STEAM DRIVEN AUX. FEEDWATER PUMP

SYSTEM Mechanical # 13

COMPONENT NAME Stm. Driven Auxiliary Feedwater Pump COMPONENT № P-32-1A, 1B

LOCATION Outside ELEVATION 21'-0"

COMPONENT SAFETY FUNCTION: ACTIVE ☐ PASSIVE 1 ☐ 2 ☒

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METHOD OF ANALYSIS: Finite Element MDOF model beam model in 3 dimensional space

SPECTRAL CURVES USED: Ground Response Spectral Curves

DAMPING VALUE ASSUMED: 7 %

ACCEPTANCE BEHAVIOR CRITERIA USED: S all ≤ 0.9 S yield

COMPUTER CODE USED: SAP IV

REMARKS: Component is O.K for mechanical function following the earthquake as well as leak tight and structural integrity

S&A

DESIGN REPORT COVER SHEET

NORTHEAST UTILITIES-
HADDAM NECK

REV.№	0		
BY	[illegible]	CAM	
DATE	4-1-81	[illegible]	
CHK'D	FZT		
DATE	3/15/[illegible]		
APPR.	JFS		
DATE	04/12/82		

5.13.1 INTRODUCTION

The Horizontal Steam Driven Auxiliary Feedwater Pump is a single stage, Worthington model, centrifugal diffuser pump located at grade (elev. 21'-0") outside of containment. The pump is operated by a turbine driver and joined to the turbine by a mechanical coupling. The pump and turbine assembly are mounted on a common bedplate, consisting of channel sections fabricated from plates. Two additional end plates and another plate, which serves a a drainage collector and also provides stiffness, are part of the bedplate. Poured in place anchor bolts then fasten the bedplate to the concrete floor. Detailed information regarding the pump and turbine assembly are found in Figures 5.13 -1 through 5.13 -4.

The horizontal pump identified by equipment tag number, P-32, is classified as a passive component II. Pump Number P-32 is not required to move or change state during a seismic event, but, may be required to do so after the event, and must retain its structural and leak tight integrity. Thus, the intent of this report is to evaluate by analysis the anchorage system and the pump internals necessary for the pump P-32, to withstand a Safe Shutdown Earthquake Seismic Event and its ability to operate following the seismic event.

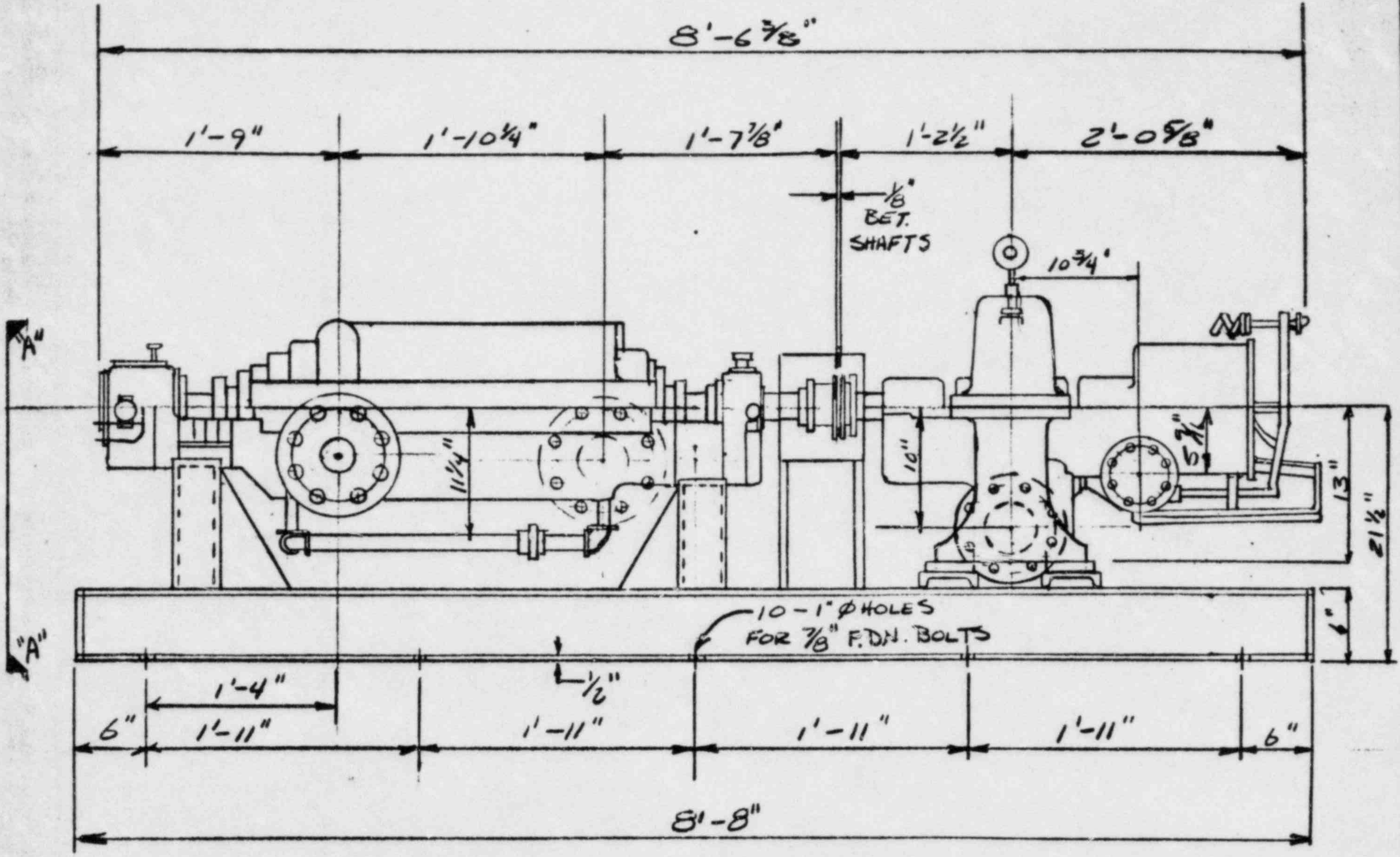
The analytical model of P-32 is illustrated in Figures 5.13 -5 and 5.13 -6, with SAP IV being the general purpose, finite element computer program used in the analyses.

5.13.2 SUMMARY OF RESULTS

Based on the results of the analyses, the Horizontal Steam Driven Auxiliary Feedwater Pump, P-32, will maintain its structural and leak tight integrity and be capable of operating following the defined seismic event.

A finite element model of the pump and turbine was prepared which considered the pump and turbine casings as rigid members. The shafts of the pump and turbine and bedplates were modelled with their respective structural stiffness properties. The bolts, in turn, were modelled as linear springs. This resulted in frequencies for the first eight modes to be less than 30 hz. They are as follows:

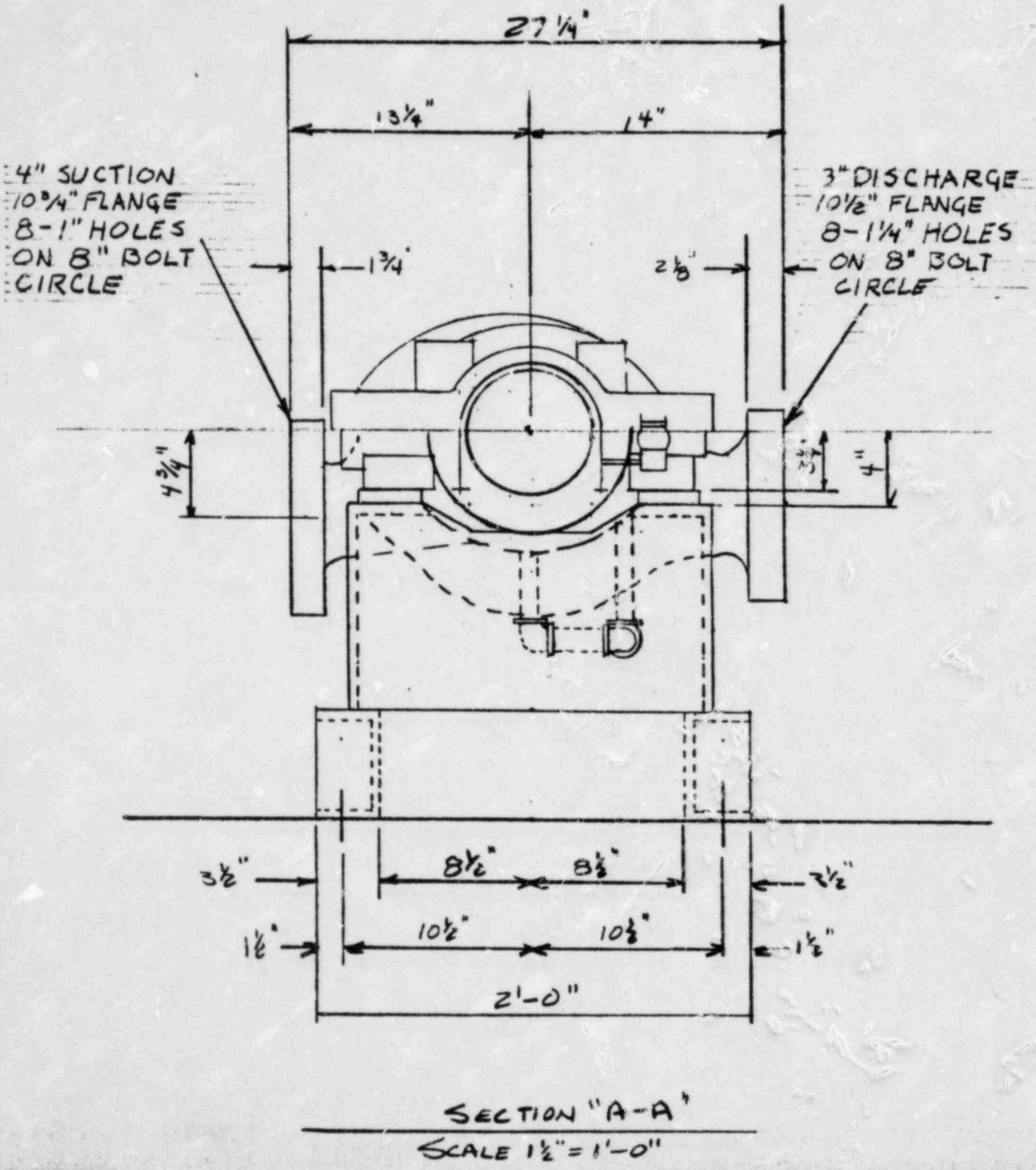
Mode Number	Frequency Hz	Dominant Direction
1	6.01	X
2	9.25	Y
3	11.40	Y
4	13.62	X
5	14.60	Z
6	22.47	Z
7	24.63	X
8	27.70	X
9	55.02	X



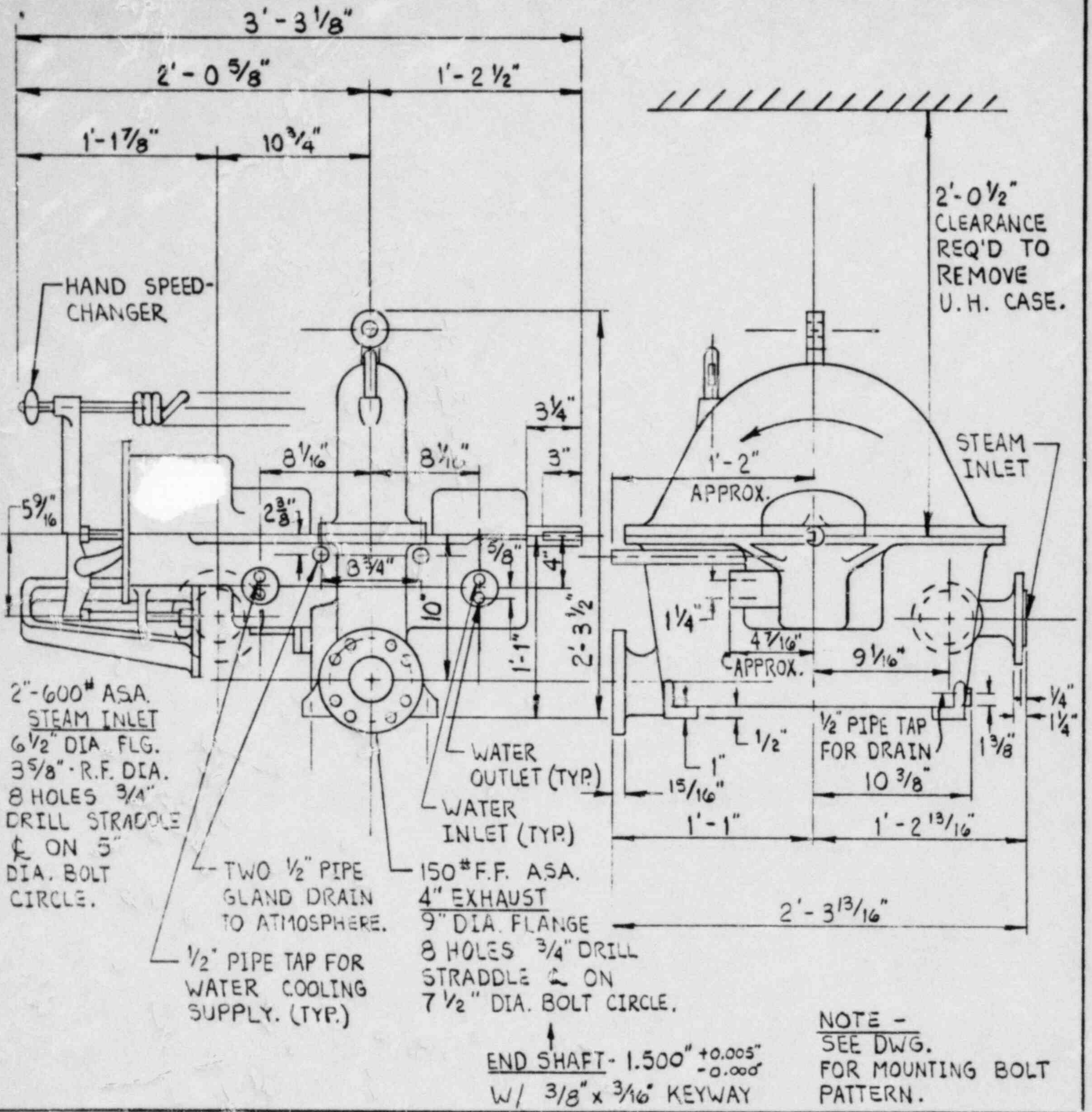
5.13-1 General Arrangement of Steam Driven Auxiliary Feedwater Pump

SCALE : 1" = 1'-0"

WEIGHTS	
PUMP & COMMON BASEPLATE	2900 Lbs
TURBINE DRIVER	700 Lbs
TOTAL	3600 Lbs

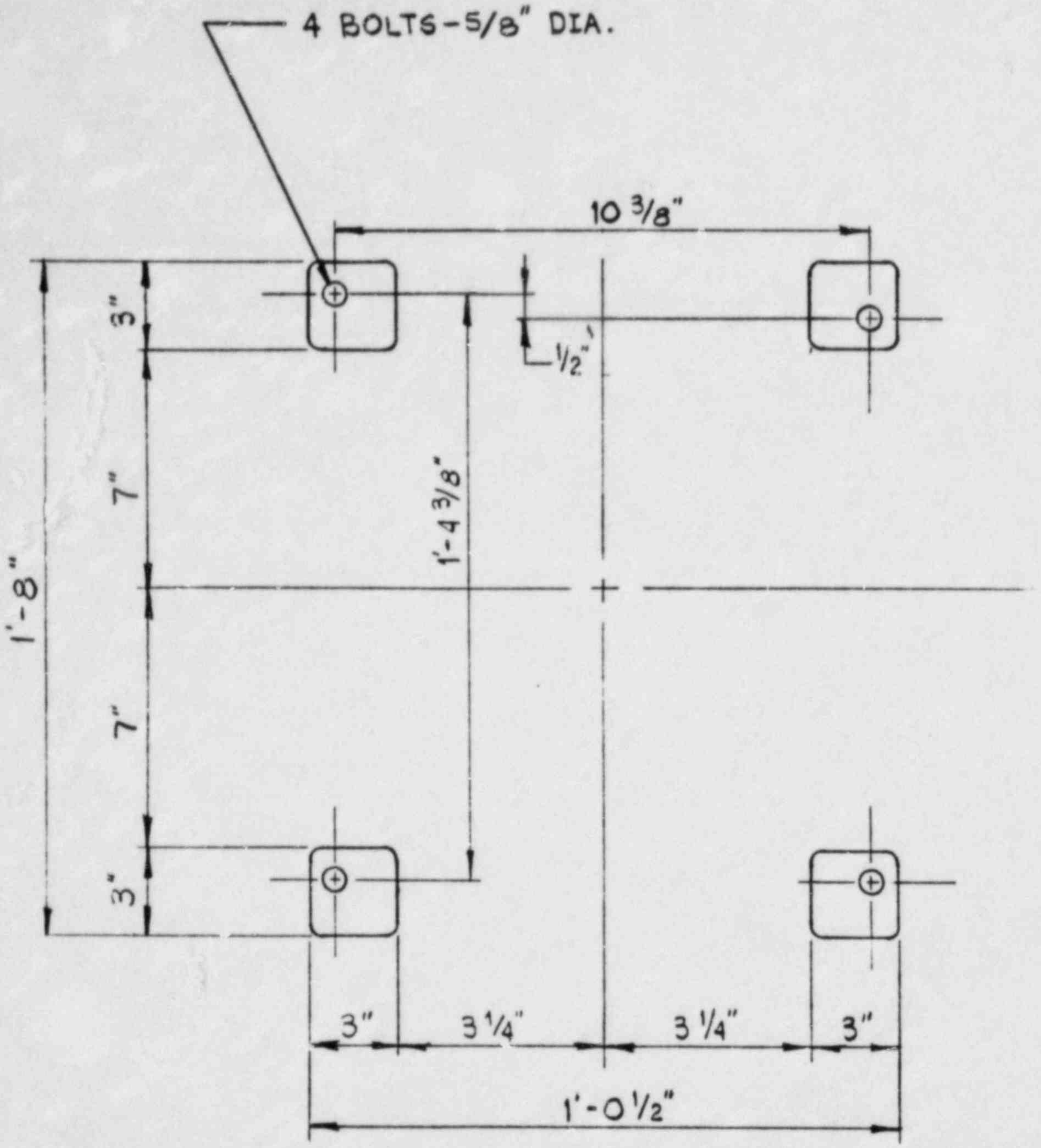


5.13-2 Steam Driven Auxiliary Feedwater Pump



NET WT. 700 lbs.
HEAVIEST PIECE 370 lbs.

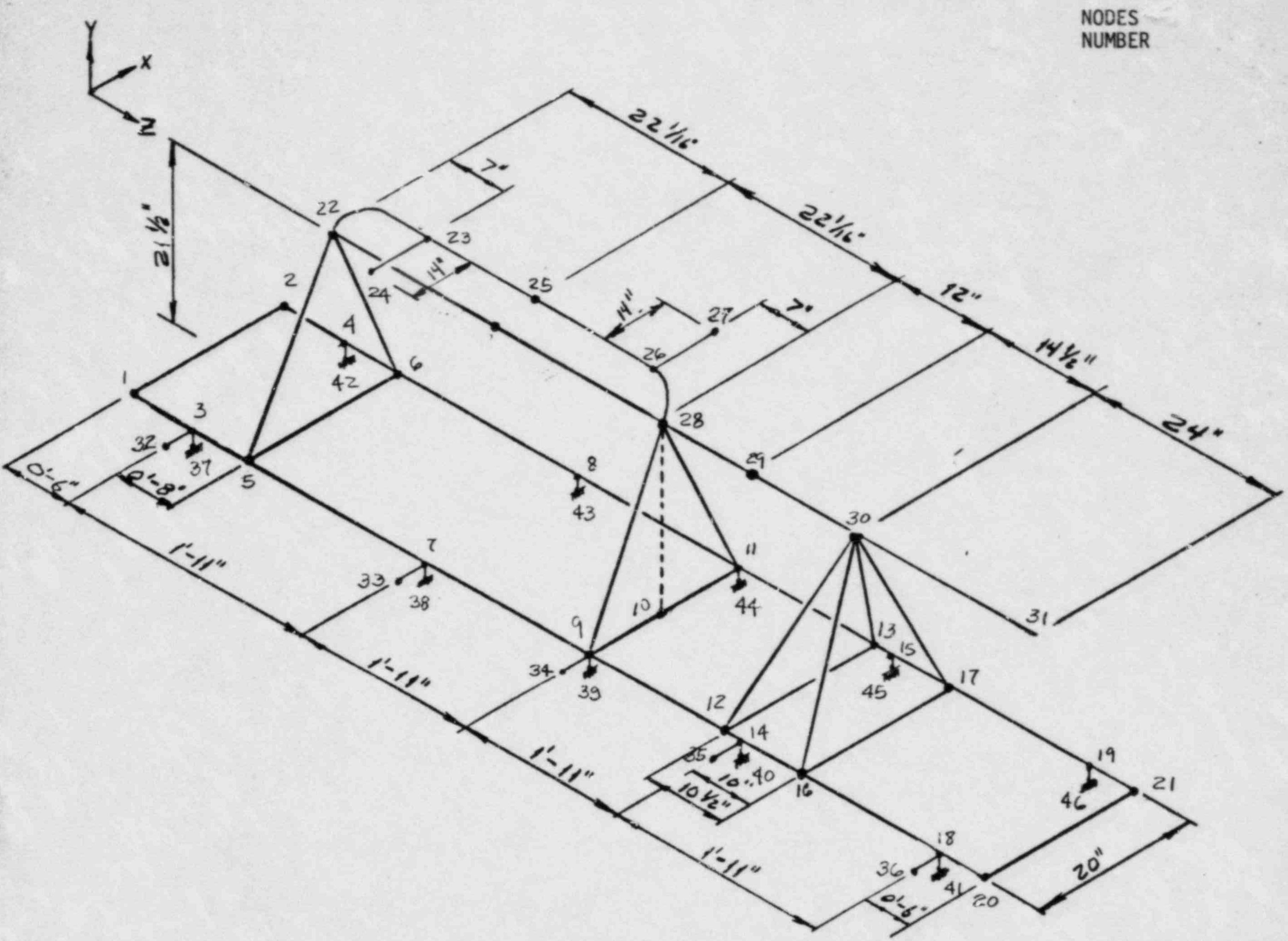
5.13-3 Detail Arrangement of Auxiliary Feedwater Pump



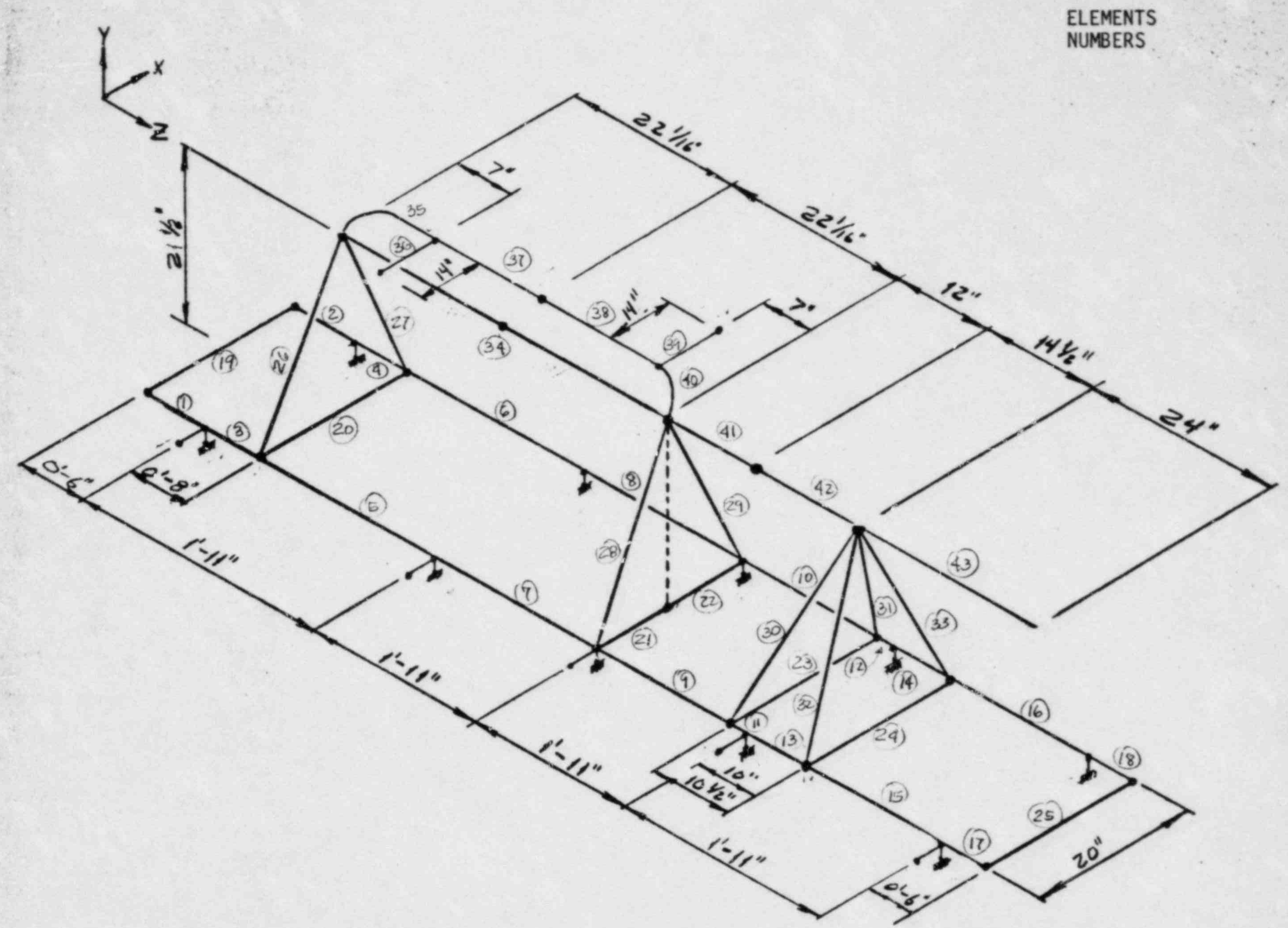
DIMENSIONS ARE APPROXIMATE.

PLAN VIEW OF FEET

5 13-4 Base Plate Layout of Auxiliary Feedwater Pump



5.13-5 Dynamic Model of Auxiliary Feedwater Pump - Node Arrangement



5.13-6 Dynamic Model of Auxiliary Feedwater Pump - Element Arrangement

The first mode obtained was a coupled sliding and rocking mode, with the sliding in the horizontal X-direction, and the rocking about the Z axis. The second and third modes were rocking modes about the horizontal X axis (transverse axis) with the shaft and pump housing participation in a vertial motion. The fourth mode was predomiently a coupled shaft and housing bending mode.

The allowable stress on the poured in place anchor bolts were as follows:

Tension $F_{tb} = \frac{S_u}{2} \times 1.4 = 40.6$ ksi

Shear $F_{vb} = \frac{0.62\, S_u}{3} \times 1.4 = 16.78$ ksi

Combined Shear and Tension $\frac{f_t^2}{F_{tb}^2} + \frac{f_v^2}{F_{vb}^2} \leq 1$

The bolts stresses and safety factors were as follows:

Tensile $f_t = 4.9$ ksi, S.F. = 8.3

Shear $f_v = 1.38$ ksi, S.F. = 12.2

Combined, using the interaction of Appendix XVII of the ASME Code.

$0.0214 \leqslant 1$ S.F. = 46.72

The shaft stresses

Tensile $f_t = 0.346$ ksi S.F. = 94

Shear $f_v = 0.114$ ksi S.F. = 284

Torsion $f_{tor} = 0.19$ ksi S.F. = 170

Bending $f_b = 13.98$ ksi S.F. = 2.3

The relative differential displacement between the shaft and pump casing was calculated and determined to be, 6×10^{-6} inches, which is well within the nominal allowable of 5×10^{-3} inches as recommended in Reference 7.14.

No evaluation of loads on the shaft bearings has been made but, given the low stresses and deformations in the shaft, bearing behavior should not be a limiting condition.

5.13.3 LOAD CRITERIA

The analysis was performed by considering both static and dynamic load cases, and then combining forces and moments by absolute summation method to determine stresses. The static load case was performed on a structural model of the pump, motor, and bedplate and applying the nozzle loads as external static loads. These external nozzle loads were obtained from API Standard 610 for horizontal centrifugal pumps, having 4" discharge nozzles or smaller. This allowed the external piping loads to be transmitted through the pump to the anchor bolts.

The dynamic load case considered the mass as well as stiffness characteristics of the pump. It incorporated the ground response spectra for two horizontal orthogonal directions oriented parrallel and perpendicular to the pump shaft as well as in the vertical direction. A damping value of 7% was used in the analysis. See Figures 5.13-7 and 5.13-8 for the response spectra curves used.

5.14.4 STRESS DEFORMATION - STABILITY CRITERIA

The allowable stress criteria for the passive, P-2, component is

$$S_{all} \leq 0.9\ S_y$$

and for tne poured in place bolting

$$S_{all} \leq 0.7\ S_u$$

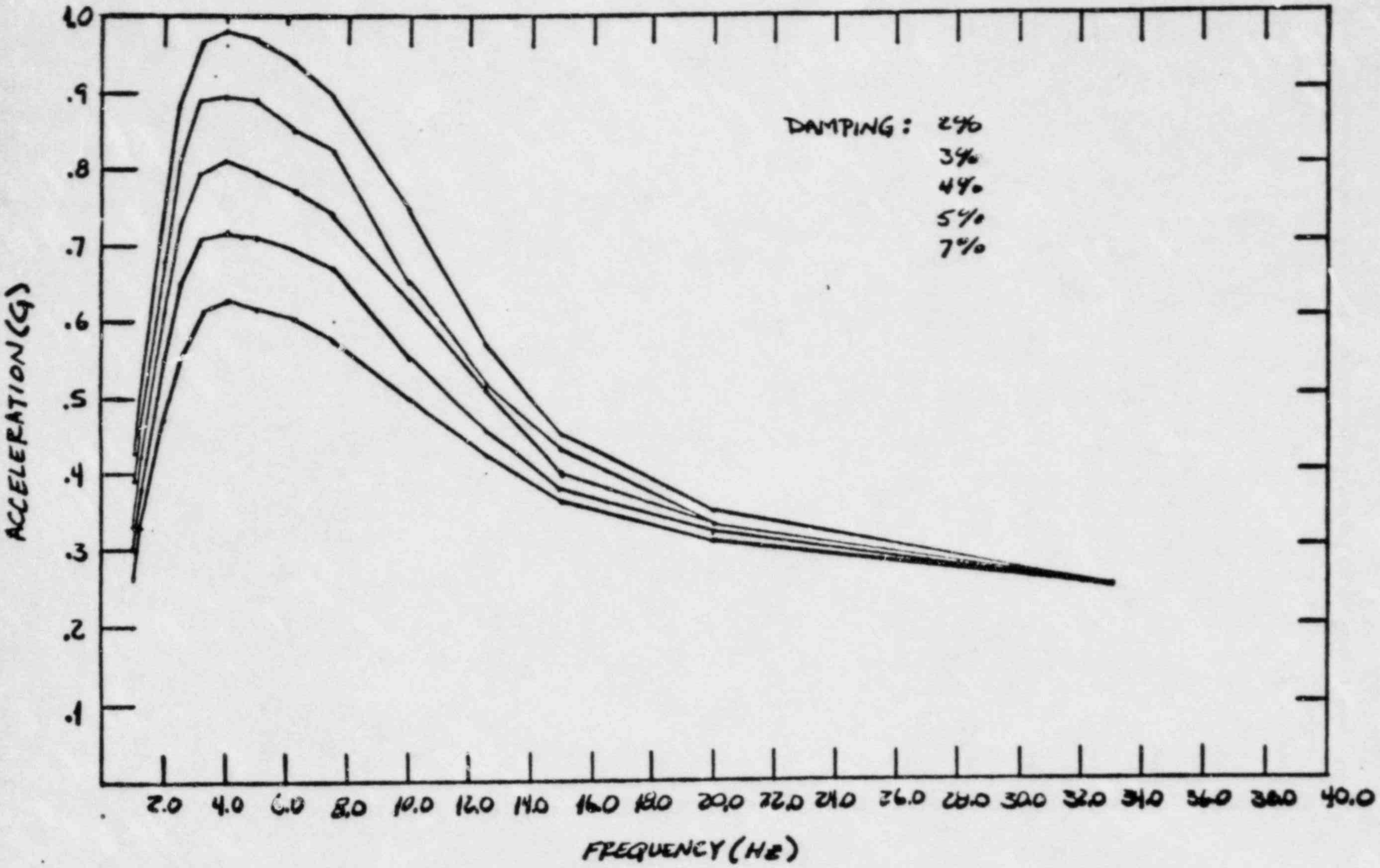
These stresses are established in the "Allowable Stress Criteria For the Haddem Neck Plant" and are attached to this report.

The requirement of API 610 for limiting the shaft deflection to a maximum of 0.005 inches has also been considered in this analysis.

5.13.5 METHOD OF ANAYLSIS

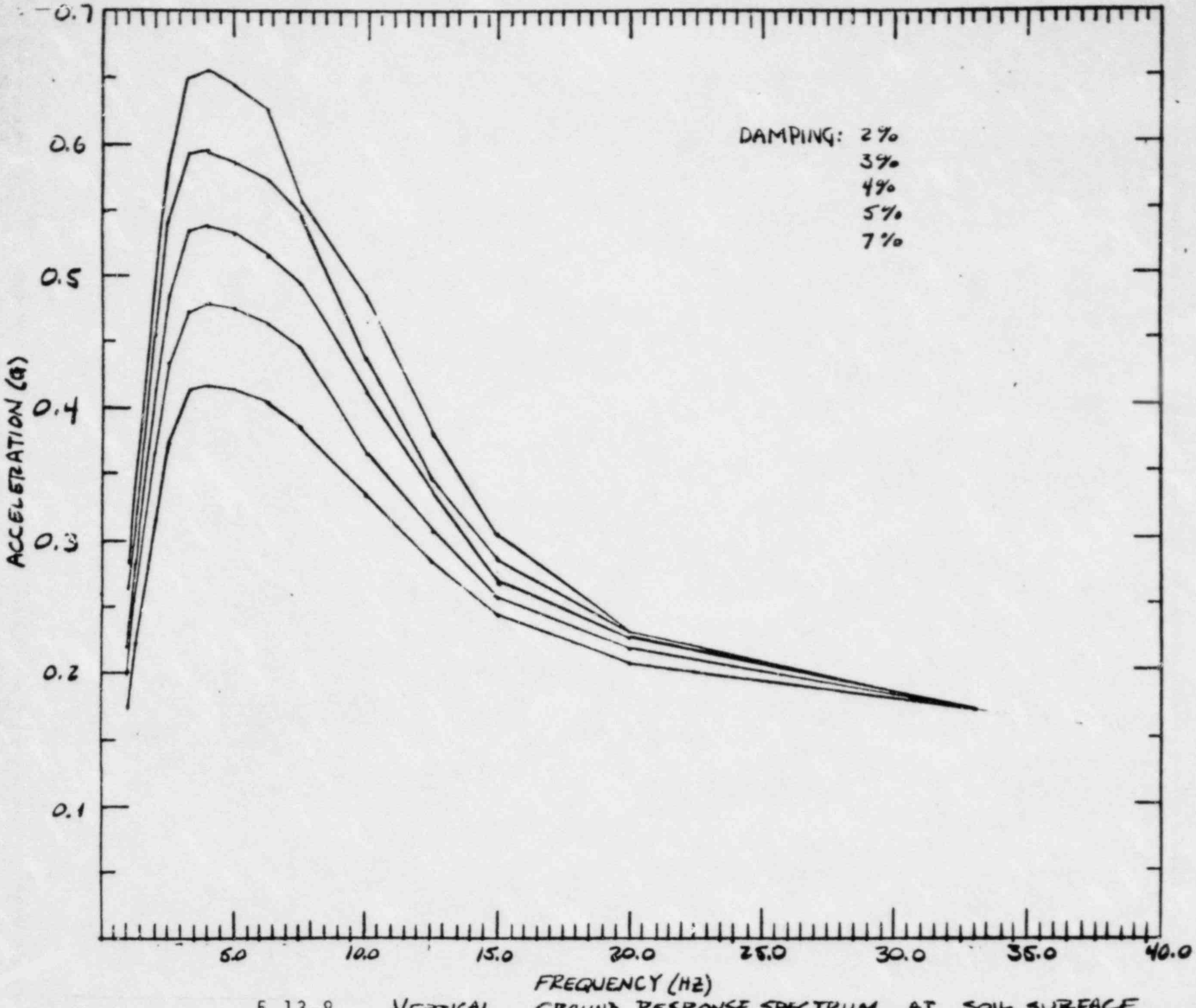
The steam driven auxiliary feedwater pump was analyzed by the response spectrum option of SAP IV, which is a general purpose, finite element program having both static and dynamic capabilities.

The analytical model, which will adequately predict the behavior of the mechanical component under seismic loads, was developed using the principles developed in a paper by C. K. McDonald entitled "Seismic Qualification by Analysis of Nuclear Power Plant Mechanical Components." Normally, McDonald recommends using ten lump masses to represent the pump assembly. These masses would then be located along the shaft centerline to represent the pump casing, impeller, coupling, motor casing, and motor rotor. Because the center of mass of the unit is several inches below the shaft centerline, applying the masses at the shaft centerline is then conservative with respect to the system overturning.



HORIZONTAL GROUND RESPONSE SPECTRA AT SOIL SURFACE

5.13-7



5.13-8 VERTICAL - GROUND RESPONSE SPECTRUM AT SOIL SURFACE

The bedplate is supported only at the anchor bolts. This is true for upward motion but for downward motion the bedplate is continuously supported. Therefore, the model will predict lower frequencies for the system than what actually exists. Since the predicted frequencies are usually higher than the resonance frequency where the pump is located, and the actual frequencies are even higher, the support assumptions are conservative.

Basically, two analyses were performed to predict the behavior of the piece of equipment.

1. A static analysis with nozzle loads
2. Dynamic analysis with a response spectra loading

Both resulting loads were then combined by direction summation and stresses were determined and shaft deflections were calculated.

5.14 UNDERGROUND 5,000 GAL. OIL TANK

5.15 CLEAN DIESEL OIL DAY TANK

5.16 VOLUME CONTROL TANK

5.17 CONTAINMENT FAN COOLERS

5.18 PRIMARY WATER STORAGE TANK

5.19 BATTERY RACK

5.20 MCC #1

SYSTEM Electrical - # 2

COMPONENT NAME Motor Control Center COMPONENT № MCC # 1

LOCATION Screenwell House ELEVATION 21'-6"

COMPONENT SAFETY FUNCTION: ACTIVE ☐ PASSIVE 1 ☒ 2 ☐

S-LIST PAGE № 150

METHOD OF ANALYSIS: A finite element model of beam and plate elements is constructed and a response spectra analysis is then done.

SPECTRAL CURVES USED: FRS of Screenwell House

DAMPING VALUE ASSUMED: 7%

ACCEPTANCE BEHAVIOR CRITERIA USED: ASME Appendix XVII & App. A, Sall & Syield for component and component support structure. Bolting and welding as indicated in "Allowable Stress Criteria"

COMPUTER CODE USED: SAP IV

REMARKS: Response Spectra Analysis is conducted after frequencies and mode shapes of the finite element model are in agreement with those determined by a low impedance, insitu test for MCC1

MCC1 structure and anchorage is O.K. for structural adequacy. No evaluation is made of Electrical Function.

S&A

DESIGN REPORT COVER SHEET

NORTHEAST UTILITIES -

HADDAM NECK

REV.№	0		
BY	PNP		
DATE	4-1-82		
CHK'D	FAT		
DATE	7/15/8		
APPR.	FAT		
DATE	6/20		

5.20.1 INTRODUCTION

Motor Control Center Number 1, MCC1, is located at floor elevation 21'-6" in the screenwell house. It is a box-shaped structure composed of three individual cabinets, bolted together, whose overall dimensions are 7'-0" x 1'-3" x 7'-6" high. These cabinets consist of unistrut framing members enclosed by metal plates, which are joined to the framing members by screw fasteners. Electrical devices contained within the cabinet are then bolted or screwed directly to the unistrut framing of the cabinet.

Revised seismic supports have already been added to MCC1. As shown in Figures 5.20-1 through 5.20-4, an angle is bolted at the top along the full length of the cabinet. Structural tubing is then used to connect the angle to an existing channel, which spans between building columns. On the bottom, two clip angles are attached to the cabinet by welded plates, and the angles are then expansion anchored to the floor.

The analytical model of MCC1, including the additional seismic supports, is illustrated in Figures 5.20-5 through 5.20-8. The analyses was made using the general purpose, finite element program, SAP IV, to determine the dynamic characteristics and structural adequacy of the MCC1 cabinet, its supports and anchorage system to withstand the defined seismic event.

5.20.2 SUMMARY OF RESULTS

Based upon the results of the analyses, the MCC1 will maintain structural integrity during and after the defined seismic event.

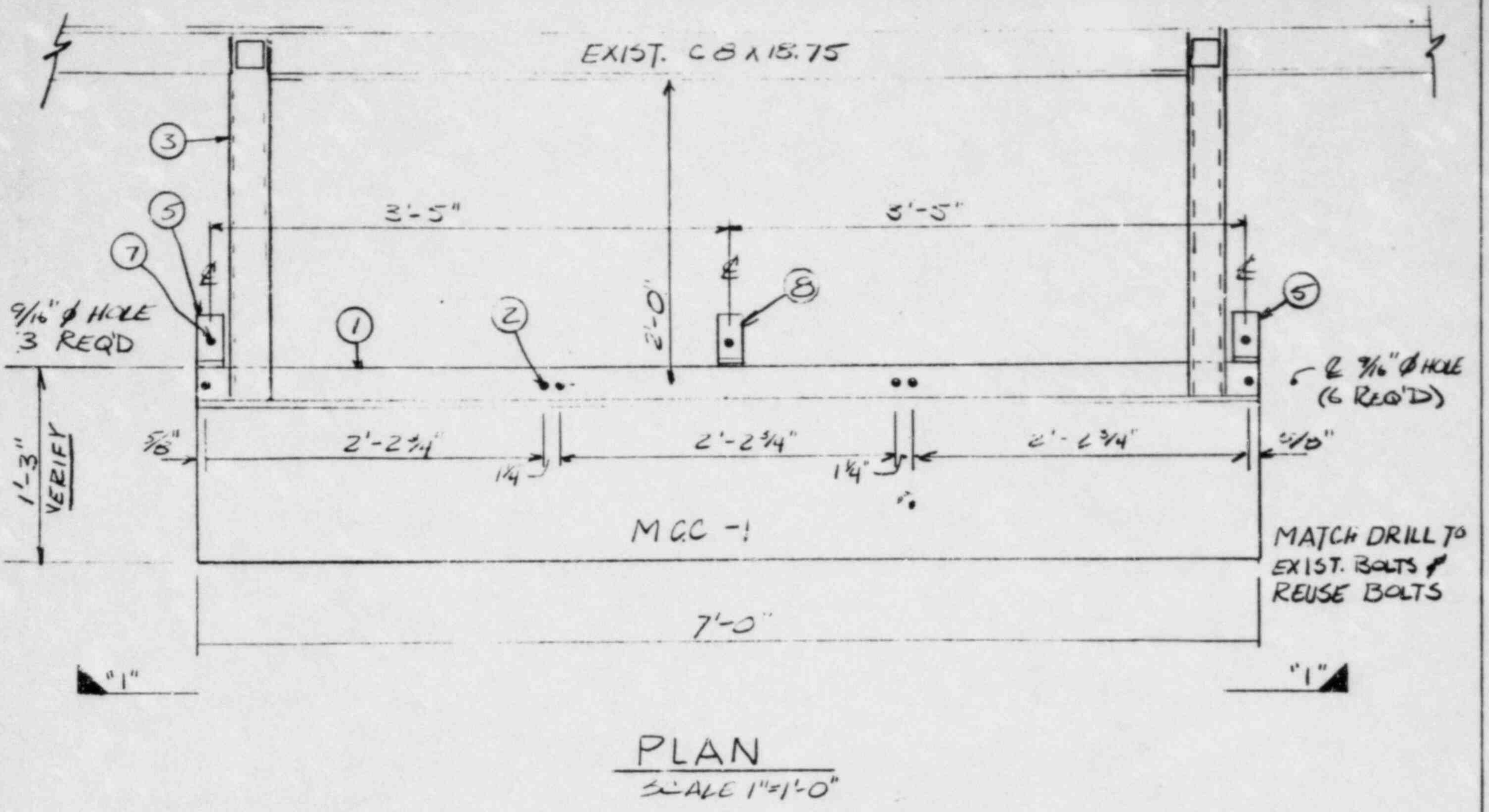
A finite element model was constructed of beam and plate elements and produced frequencies and mode shapes that were in close proximity to those that were previously established by a low impedance, insitu test of the MCC1. The frequencies for the first two modes are 17.2 hz and 23.6 hz for the test and 19.7 hz and 20.8 hz for the analytical model. Figures 5.20-9 and 5.20-10 for the test and Figures 5.20-11 and 5.20-12 for the model indicate the first two mode shapes of the cabinet with the relative displacements along the transverse (z) direction. Thus, the model was justified, and a response spectrum seismic analysis was then conducted.

All of the seismic supports exhibit calculated stresses that are well below the allowables for both the structural members and the connections. Considering only the limiting sections, the following results are obtained:

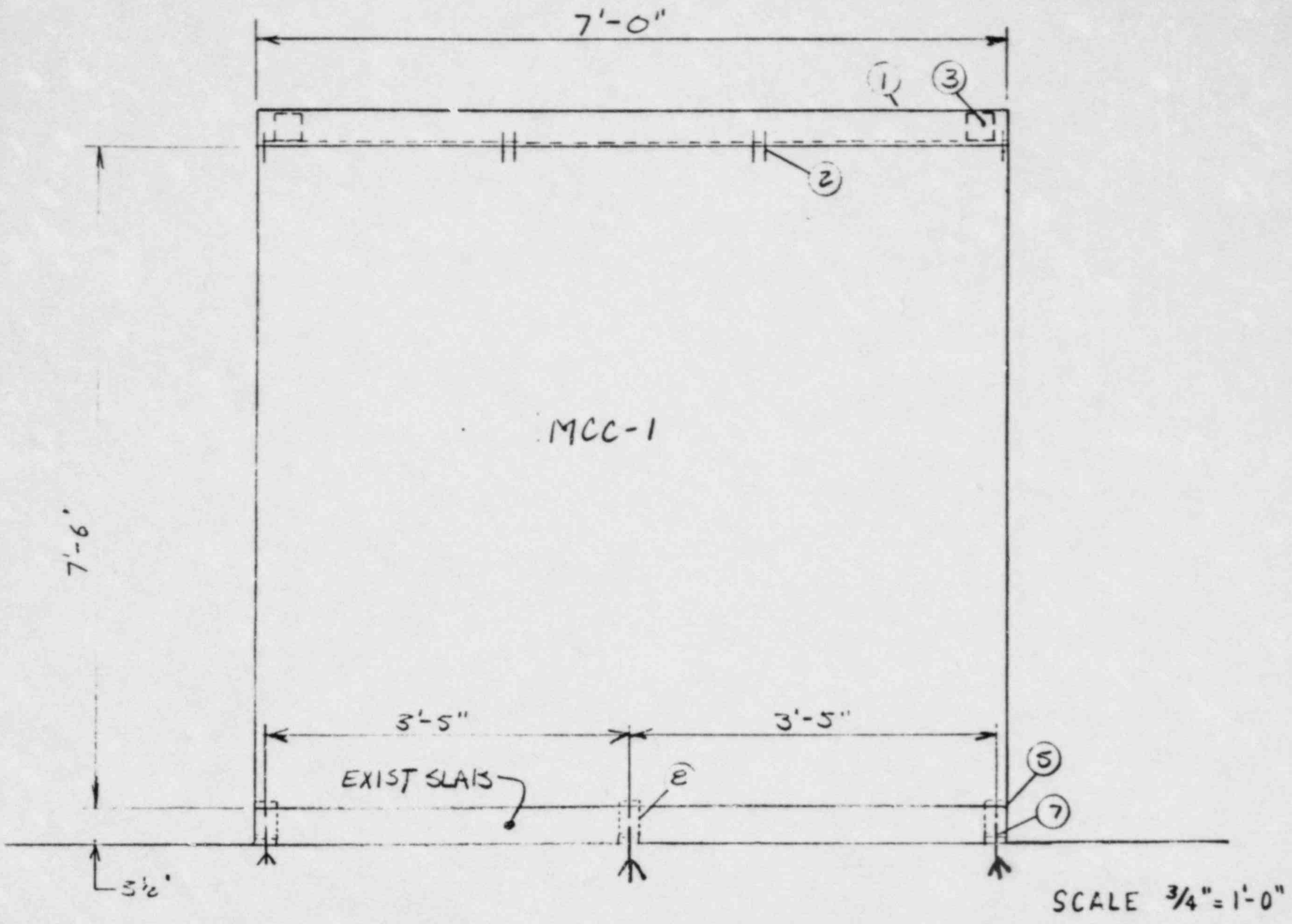
(1) Structural Member:

structural tubing (see item 3 on Fig. 5.20-1)

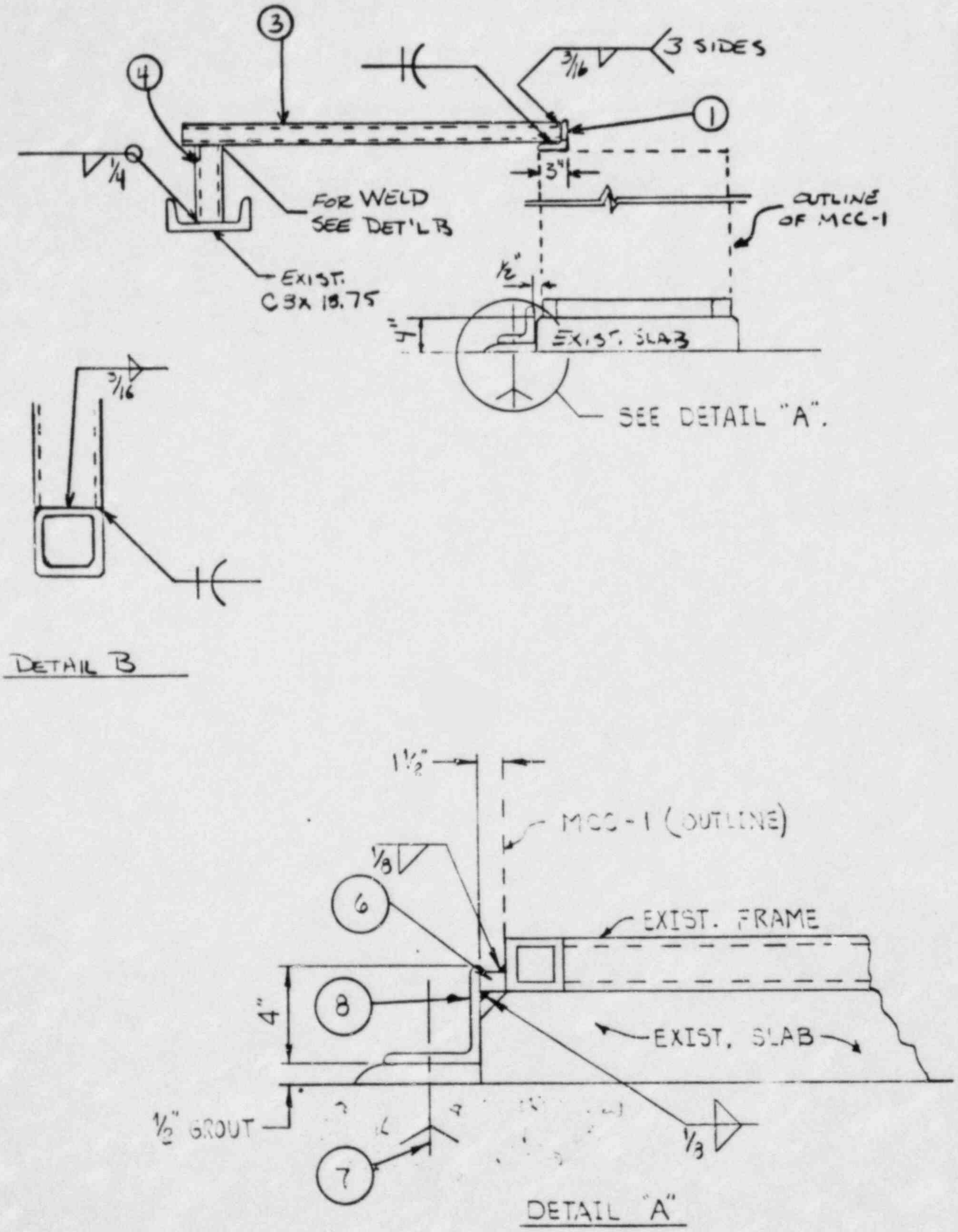
	Calculated:	Allowable:
TS 3 x 3 x 1/4	f_b = 1044 psi;	F_b = 24,000 psi
	f_a = 82 psi;	F_a = 19,940 psi



5.20-1 - Plan View of MCC1 and Revised Seismic Supports



5.20-2 Section "1-1" of MCC1

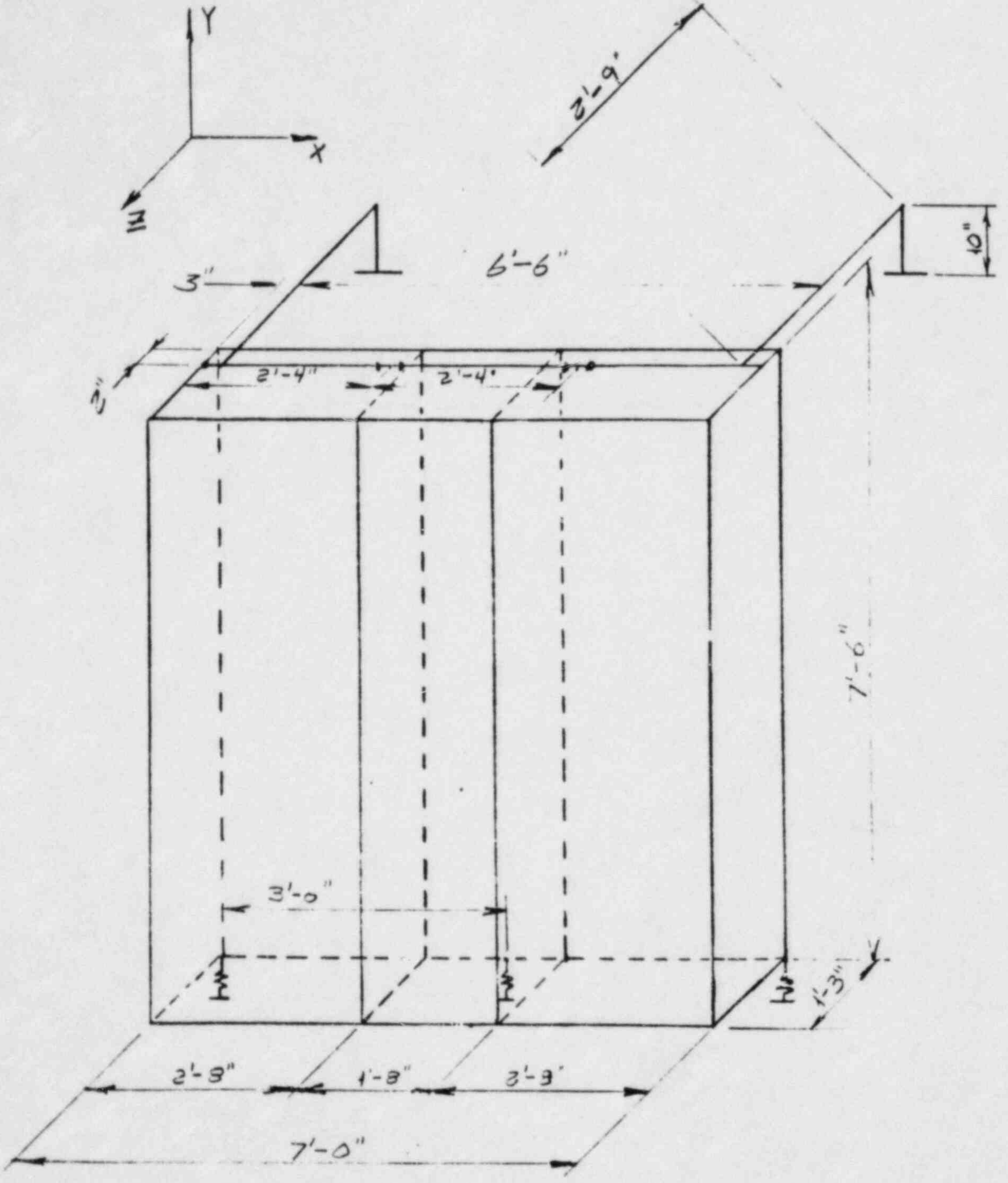


5.20-3 Details of Revised Seismic Supports for MCC1

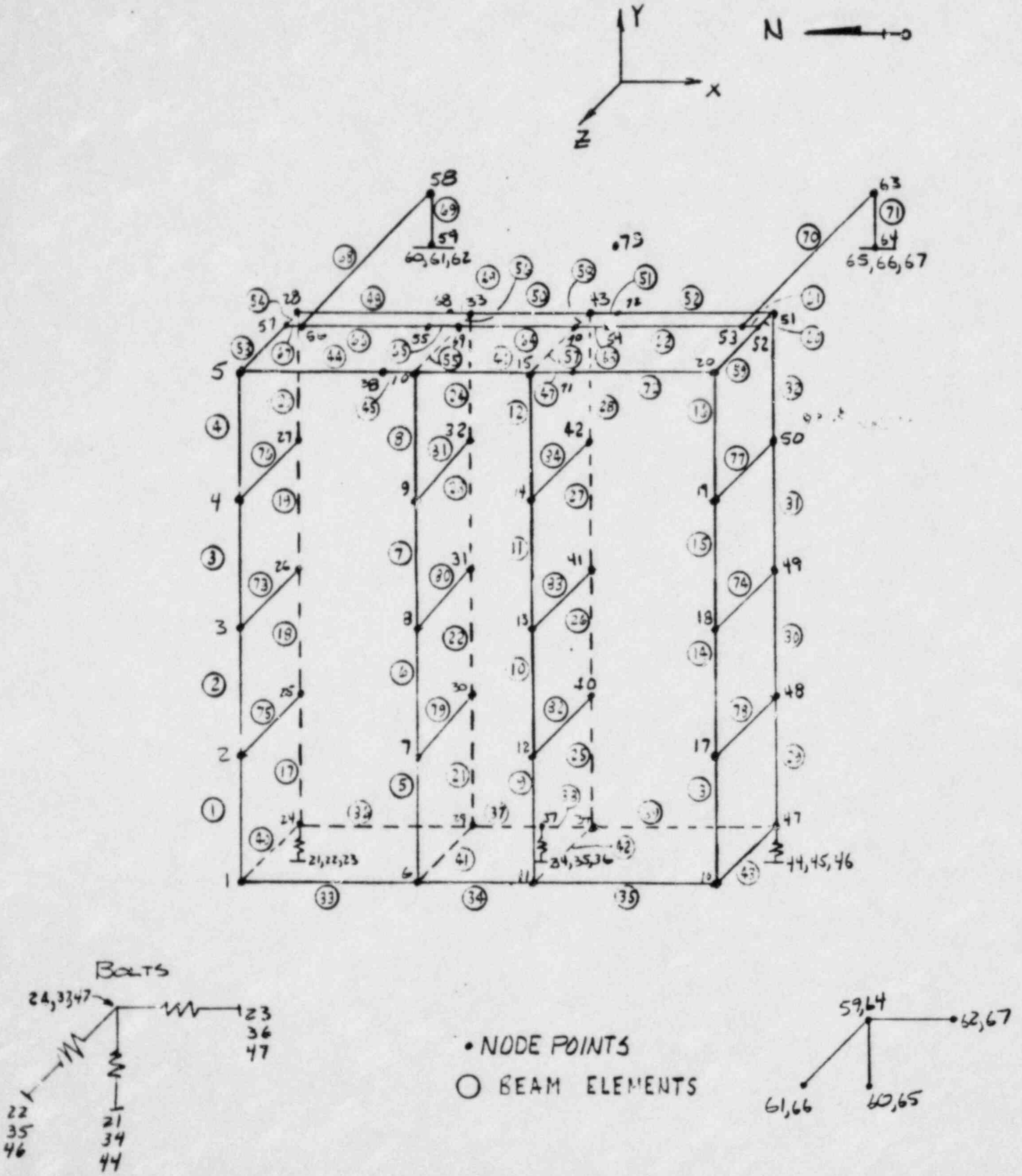
LIST OF MATERIALS - SEISMIC SUPPORT FOR MCC-1

ITEM №	№ REQ'D	DESCRIPTION	REMARKS
1	1	L 4 x 3 x 1/4 x 7'-0" LNG.	A-36
2	8	1/2" Ø x 2" LNG HEX HEAD BOLT w/NUTS	A-307
3	2	TS 3 x 3 x 1/4 x 2'-8 3/8" LNG	
4	2	TS 3 x 3 x 1/4 x 0'-10 1/2" LNG	
5	2	L 4 x 3 x 3/8 x 0'-2" LNG	A-36
6	3	BAR 3/8 x 1 1/2 x 2	A-36
7	3	1/2" Ø x 5 1/2' LNG HILTI KWIK-BOLT	2 1/4" MIN. EMB'NT
8	3	L 3 x 4 x 3/8 x 0'-2" LNG	A-36

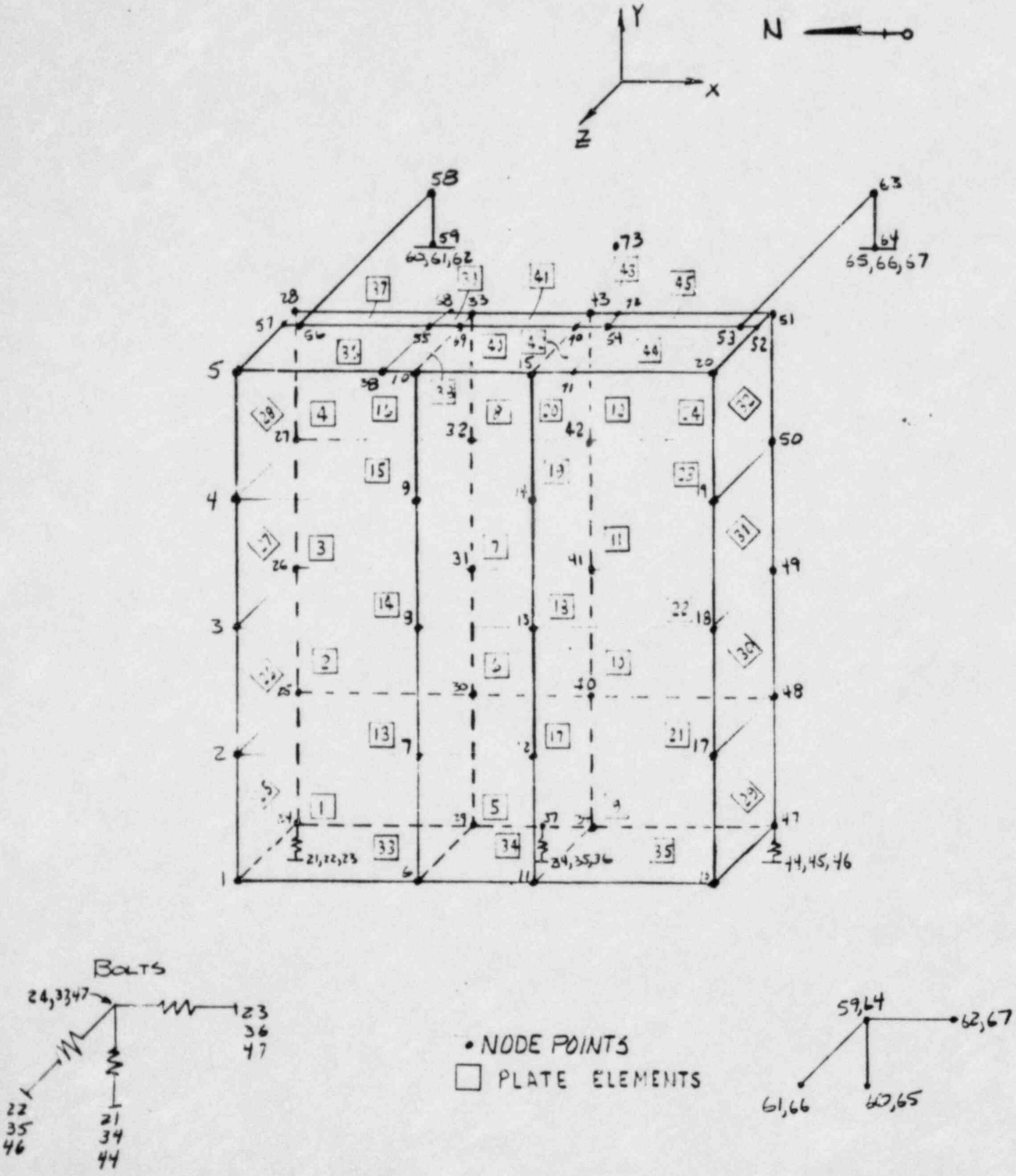
5.20-4 List of Materials for Revised Seismic Supports of MCC1



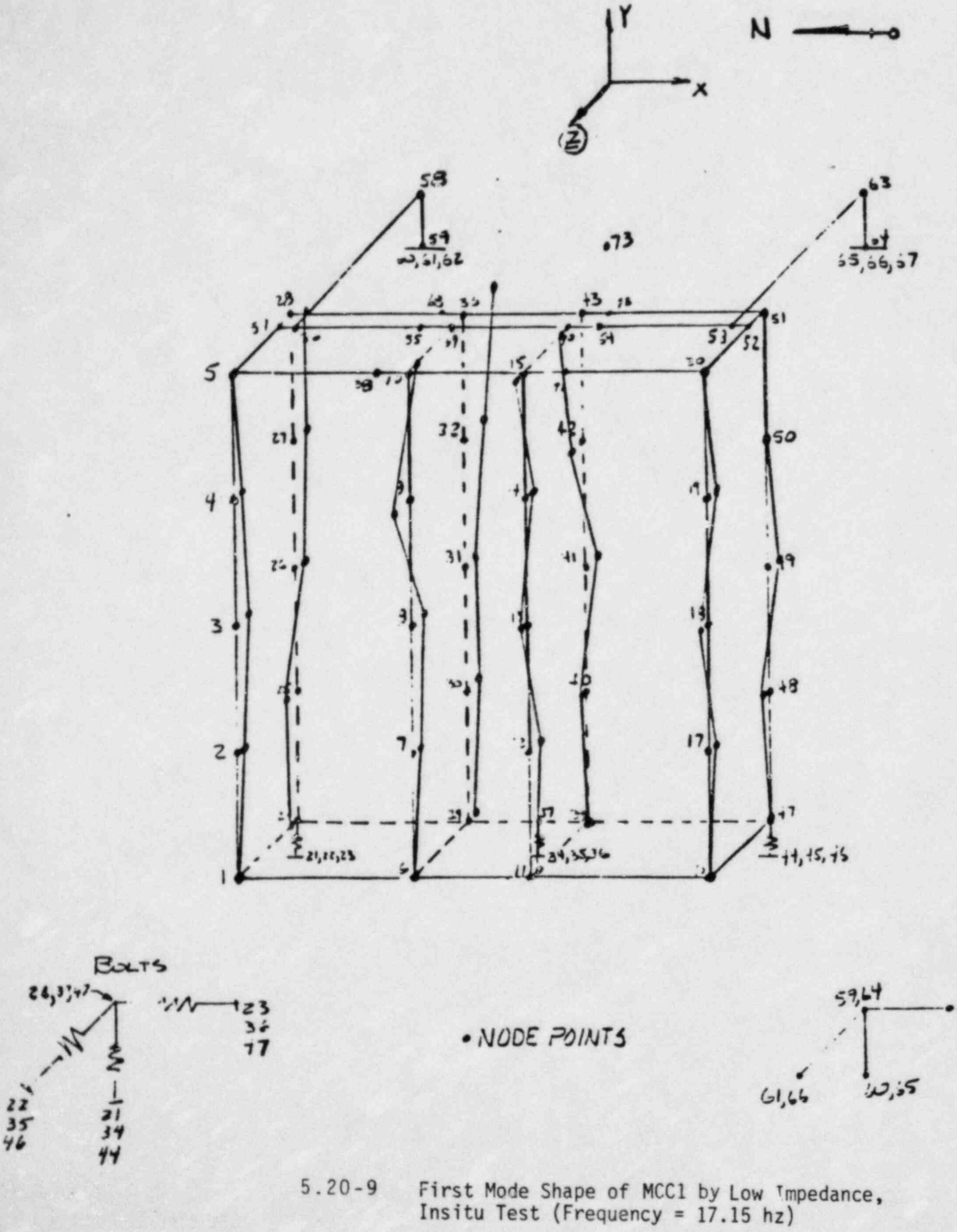
5.20-5 - Three Dimensional Model of MCC1



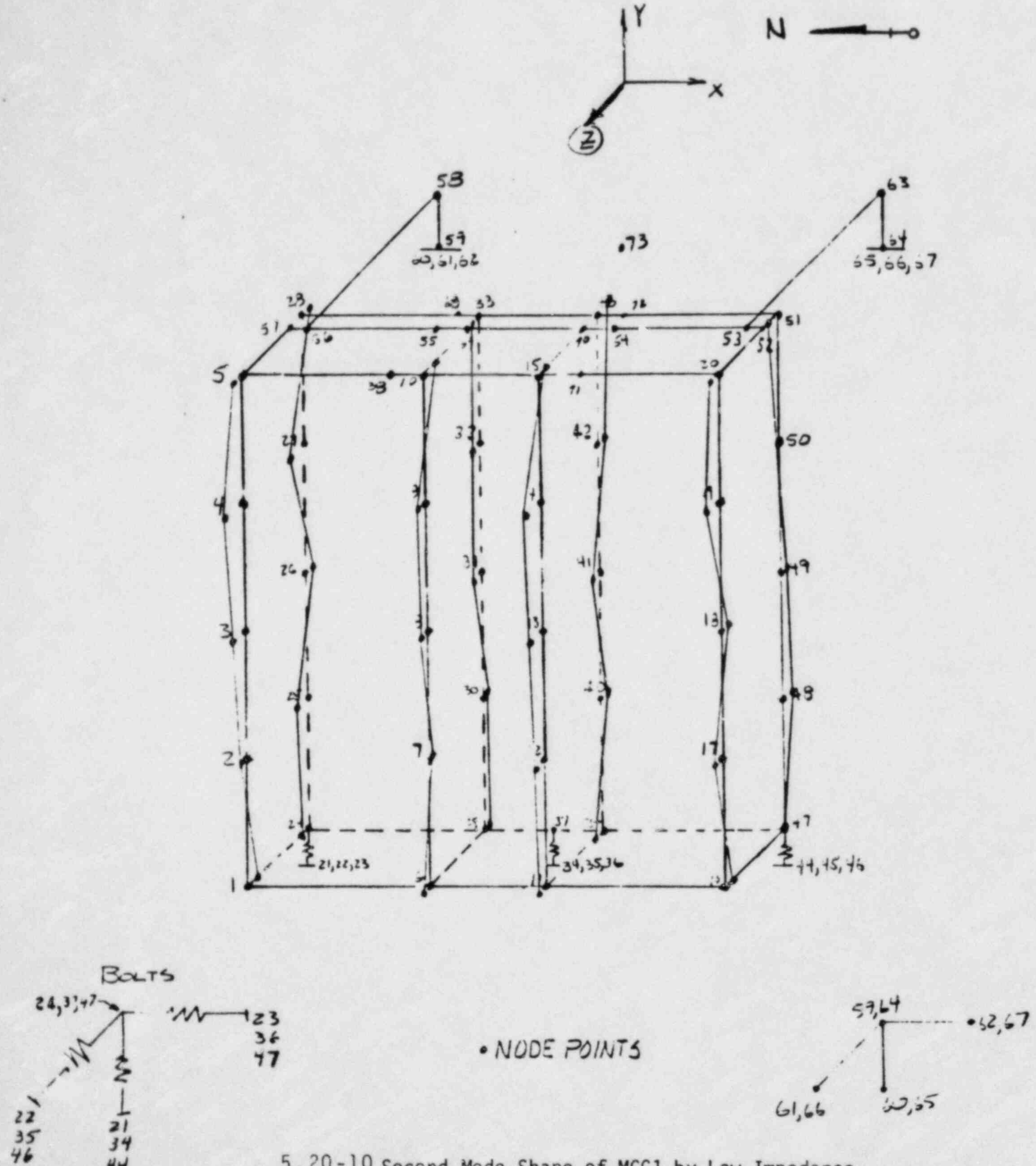
5.20-7 Three Deimensional Model Indicating Node Points and Beam Elements for MCC1



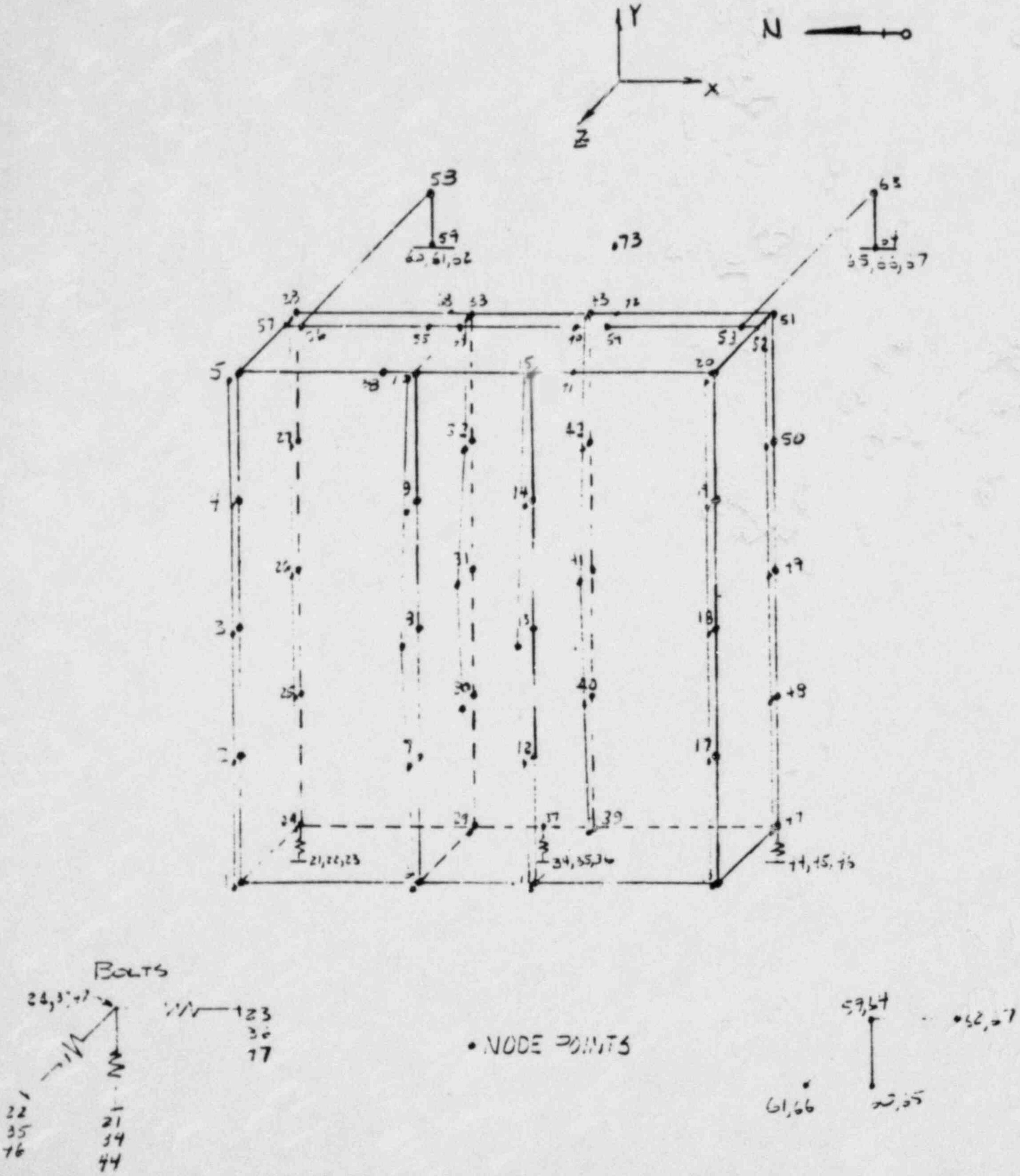
5.20-8 Three Dimensional Model Indicating Node Points and Plate Elements for MCC1



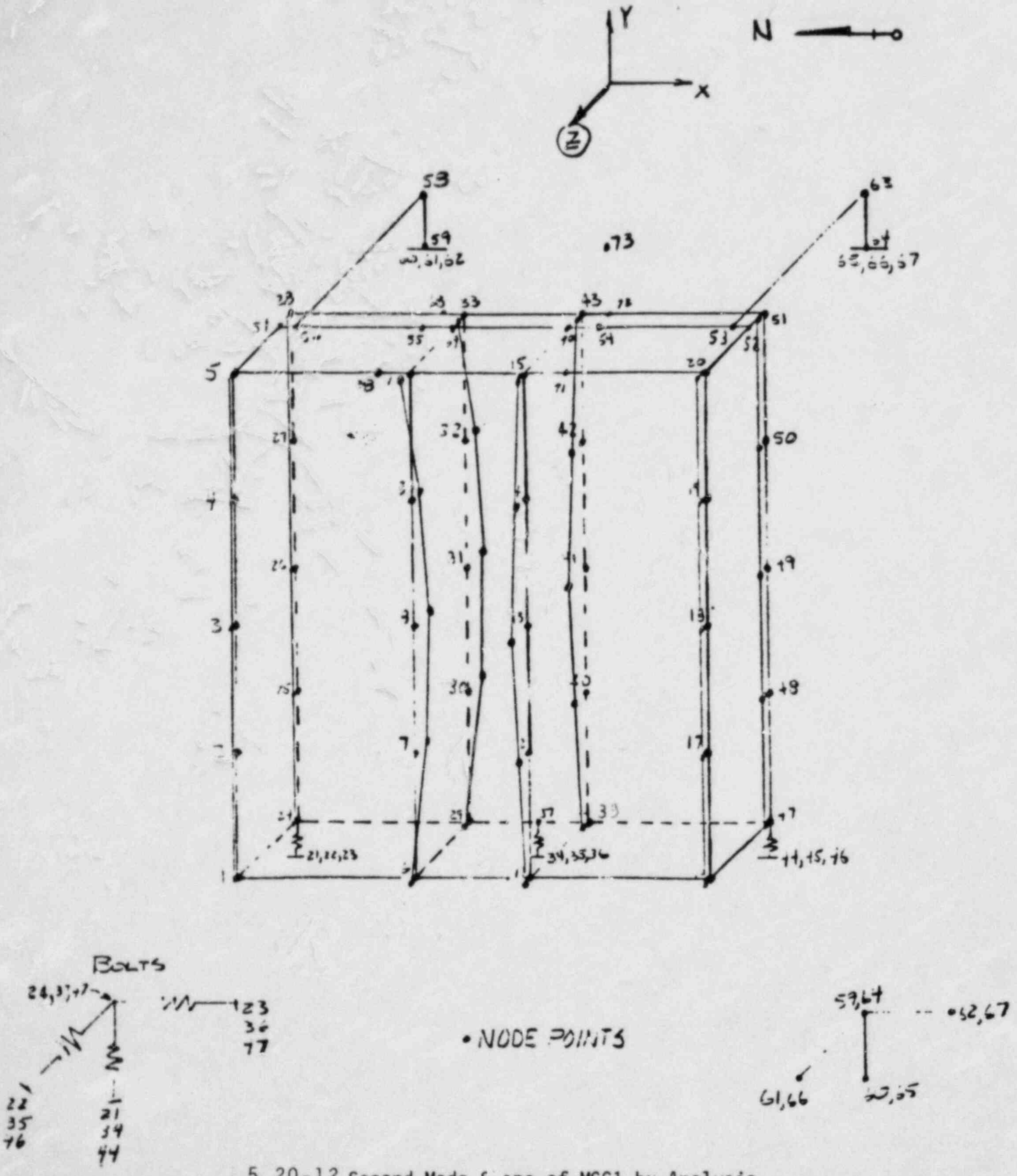
5.20-9 First Mode Shape of MCC1 by Low Impedance, Insitu Test (Frequency = 17.15 hz)



5.20-10 Second Mode Shape of MCC1 by Low Impedance, Insitu Test (Frequency = 23.6 hz)



5.20-11 First Mode Shape of MCC1 by Analysis
(Frequency = 19.75 hz)



5.20-12 Second Mode Shape of MCC1 by Analysis
(Frequency = 20.86 hz)

Applying the interaction formula from the ASME Code, Appendix XVII, for biaxial bending and axial compressive stresses yields 0.1, and the resultant safety factor exceeds 10.

(2) Welded Connection:

3/16" fillet weld between the horizontal TS 3 x 3 x 1/4 and the vertical TS 3 x 3 x 1/4

f_r = 989 psi; F_r = 33,600 psi

where:

f_r = resultant stress due to combined shear, bending, and torsion

F_r = allowable resultant stress taken as 1.6 x 0.3 F_y for E70xx electrodes

The safety factor is then equal to 34.

(3) Bolted Connection:

bolting of the ∠4 x 3 x 1/4 identified as item 1 in Figure 5.20-1, to MCC1

Shear f_v = 561 psi; $F_v = \frac{.62}{3} S_u \times 1.4$ = 16,780 psi

Tension f_t = 141 psi; $F_T = S_u/2 \times 1.4$ = 40,600 psi

Applying the interaction elipse formula found in Appendix XVII of the ASME Code yields $.001 \ll 1.0$ and the safety factor is approximately 1,000.

(4) Expansion Anchor:

1/2"ϕ Hilti expansion anchor bolts for 4,000 psi concrete with a minimum embedment of 2-1/4"

Shear	f_v = 106 lbs	F_v = 8316 lbs
Tension	f_t = 179 lbs	F_t = 5510 lbs

Using the interaction formula

$$IC = \left[\frac{f_t \times f_p}{F_t/4} + \frac{f_v}{F_v/4}\right] \times \frac{1}{f_r} \leq 1.0$$

where:

f_r = capacity reduction factor which accounts for bolt spacing and the distance to the free edge of concrete and is equal to 1.0

f_p = prying factor assumed equal to 1.5

IC = $0.25 \leq 1.0$

Thus, a safety factor of 4 is provided.

While seismic loads and structural capacity of the MCC1 cabinet are determined primarily by the adequacy of the unistrut frame, the cabinet side plate tends to act as a shear beam having a major stiffening effect on the frame. This significantly increases the frequency of the cabinet and, thereby, reduces the applied seismic inertial loads.

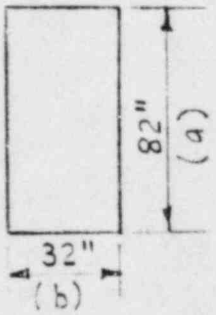
For this reason the potential for the side plates of the MCC1 cabinet to buckle was investigated. Should such buckling occur the fundamental frequency of the cabinet would be dramatically reduced and resultant seismic inertial loads increased substantially.

The following assumptions were made:

- The plates are fastened to the frame by screw fasteners located at the various node points
- The plates are 1/16" thick, rectangular, flat, and isotropic
- The plates are simply supported and subjected to loads in its plane

It was determined that the most limiting section was along the front and back portions of the cabinet because that was the longest unsupported length.

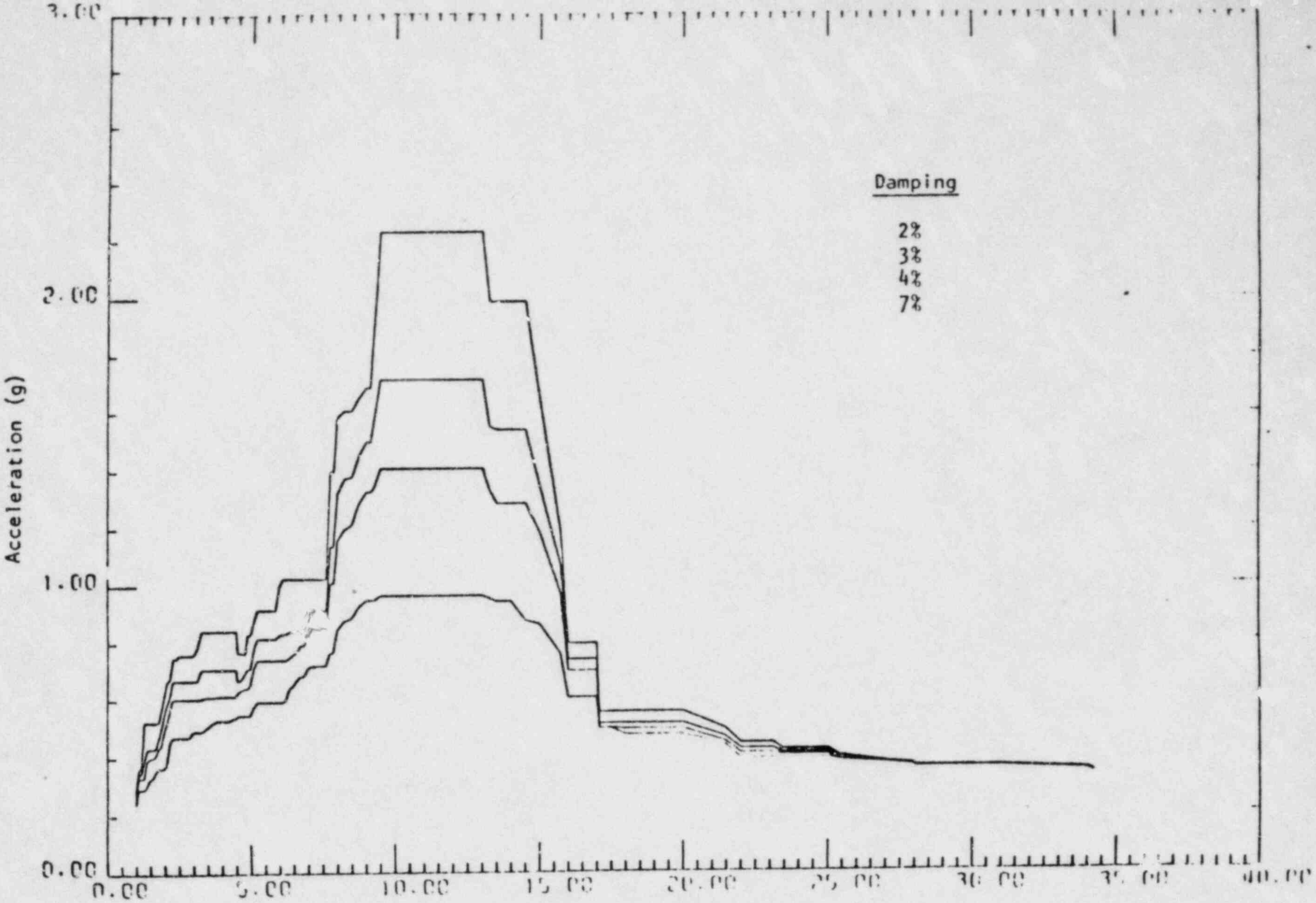
The USS Steel Design Manual, 1968 edition, was used to evaluate the elastic buckling stresses for the plate in the following manner:



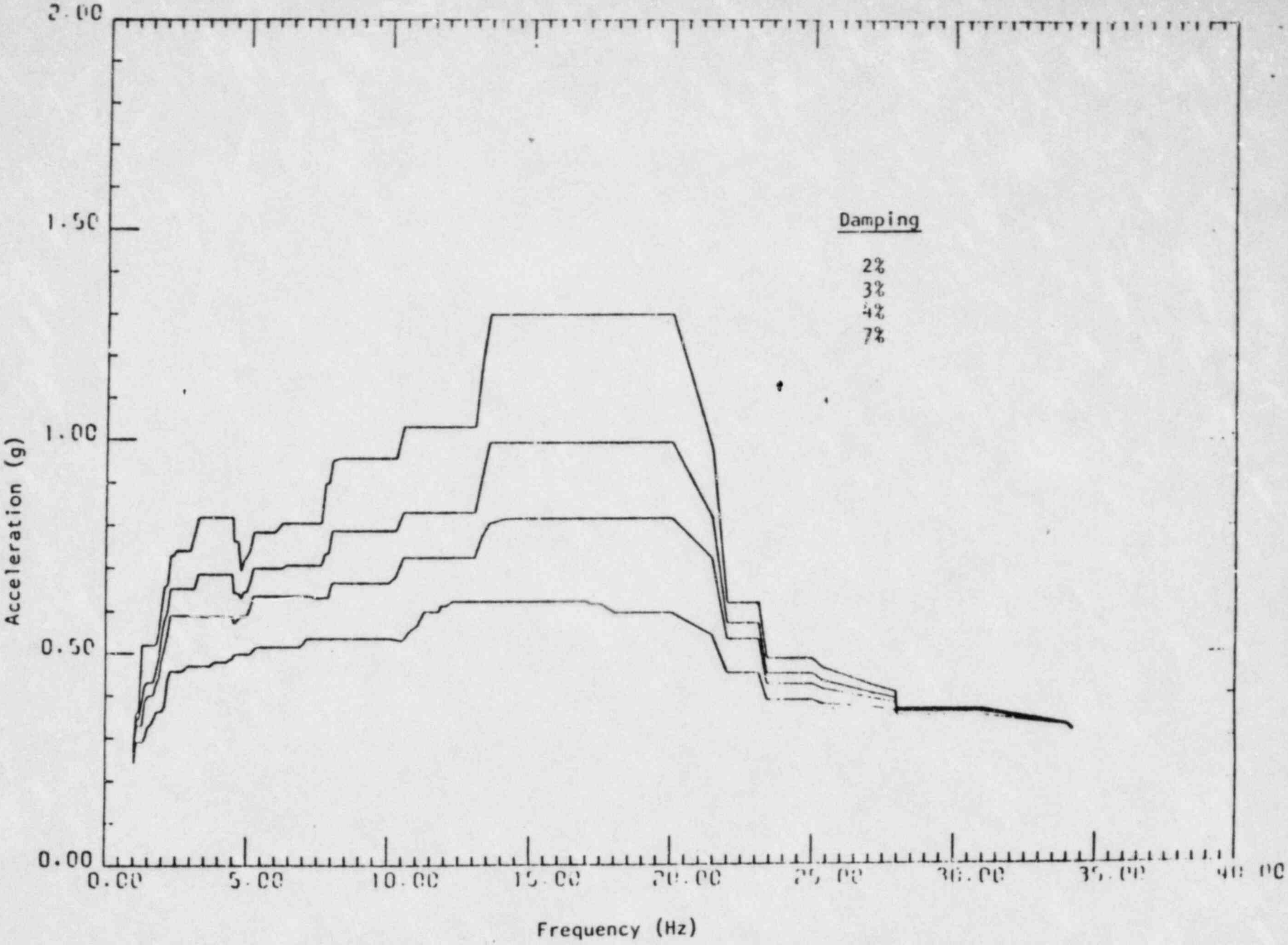
- For pure shear:

$$\tau_{CR} = \frac{k_s \pi^2 E}{12\,(1 - \mu^2)(b/t)^2}$$

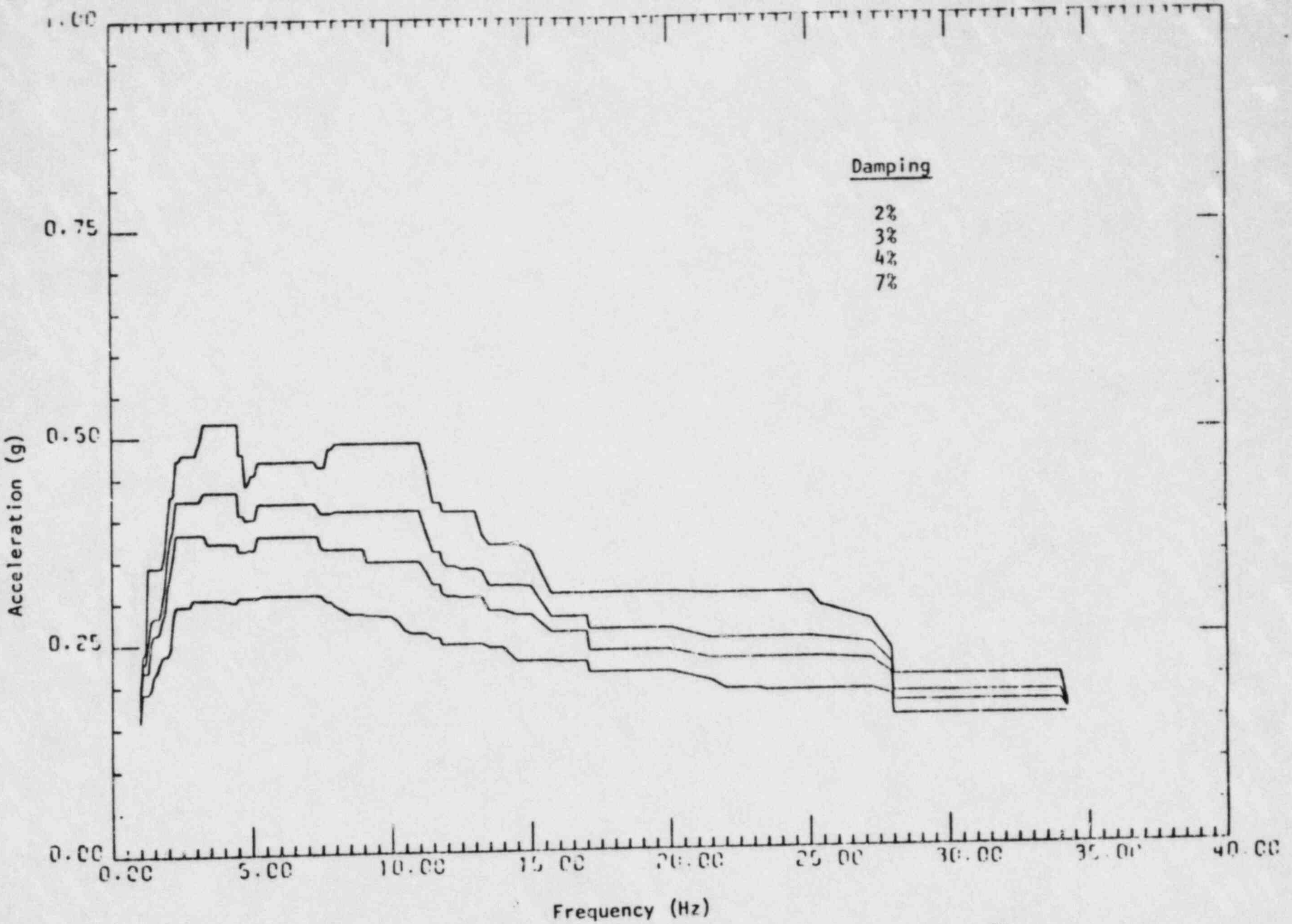
123



5.20-13 North-South Floor Response Spectra For Elevation 21'-6", Screenwell House, Connecticut Yankee Atomic Power Plant



5.20-14 East-West Floor Response Spectra For Elevation 21'-6"
Screenwell House, Connecticut Yankee Atomic Power Plant



5.20-15 Vertical Floor Response Spectra For Elevation 21'-6", Screenwell House, Connecticut Yankee Atomic Power Plant where:

k_s = a nondimensional plate buckling coefficient depending on the type of edge support (simply supported) and the length to width ($^a/b$) ratio.

From Figure 4.6, page 80, $k_s = 6$ and $\tau_{cr} = 614$ psi.

- For uniform compression;

$$\sigma_{cr} = \frac{k_c \pi^2 E}{12(1-\mu^2)(^b/t)^2}$$

where:

k_c = a nondimensional plate buckling coefficient which depends on the type of edge support (simply supported) and the $^a/b$ ratio.

From Figure 4.2, page 73, $k_c = 4$ and $\sigma_{cr} = 409$ psi.

The largest stresses for the individual plate elements, 32" x 22", found in this portion were then applied to the overall dimensions of the plate, 82" x 32". This was a conservative approach and resulted in:

$\tau_{max} = 25$ psi; $\tau_{all} = 2/3\left(\tau_{cr}\right) = 409$ psi

$\sigma_c = 24$ psi; $\sigma_{all} = 2/3\left(\sigma_{cr}\right) = 273$ psi

- For compression along one axis with shear

$$\frac{\tau}{\tau_{all}} = \frac{25}{409} = .06$$

$$\frac{\sigma_c}{\sigma_{all}} = \frac{24}{273} = .08$$

Referring to Figure 4.7, page 81, of the USS Design Manual the results are well within the interaction curve acceptance, hence, are acceptable.

5.20.3 LOAD CRITERIA

After the model was verified by comparing the frequencies and mode shapes to those that were established by the low impedance, insitu test, a response spectrum analysis was performed using the floor response spectral curves, shown in Figures 5.20-13, 5.20-14, and 5.20-15, located at elevation 21'-6" of the screenwell house for 7% damping.

5.20.4 STRESS DEFORMATION - STABILITY CRITERIA

The MCC1 is a passive, P-1, component whose stress limit is defined as

$$S_{all} \leq S_y;$$

which was determined from the "Allowable Stress Criteria for the Haddam Neck Plant" attached hereto as Appendix A.

5.20.5 METHODS OF ANALYSIS

The MCC1 was analyzed using the response spectrum option of SAP IV. SAP IV is a general purpose, finite element program that includes both static and dynamic capabilities for a broad range of elements and has been used extensively in the nuclear power industry for analyzing structures and mechanical component responses to dynamic loads.

Several assumptions have been made to facilitate the development and implementation of the model for analysis:

- The beam and plate members are modelled together with the plates joining the beam structure at every frame node and contribute to the stiffness of the structure.
- The additional mass of the cabinet has been lumped at the eight center frame nodes.
- The expansion anchor bolts are modelled as linear springs having both tension and compression capability as installed. This is an added conservatism as the floor and not just the bolts carry the compressive bearing loads. The effective anchor bolt length for the determination of bolt stiffness consists of the free length plus 5-bolt diameters depth into the floor.
- Where the structural tubing is welded to the existing channel section, it is considered to be a rigid connection.

6. CALCULATIONS

5.21 SWITCH GEAR (D.G. ROOM)

5.22 CONTROL PANEL (D.G. ROOM)

5.23 ENGINE MOUNTED CONTROL PANEL (ON DISEL GEN)

5.24 4160-480 V SWITCHGEAR

5.25 TRANSFORMERS (SWITCHGEAR ROOM)

5.26 MCC #5 & # 6

5.27 Battery Charger

5.28 MCC #3

5.29 MAIN CONTROL BOARD

5.30 EMERGENCY POWER CONTROL BOARD

5.31 MCC #8

6. CONCLUSIONS AND RECOMMENDATION

It is impossible, on the basis of the limited sample here available, to evaluate the overall seismic desing adequacy of all safety related equipment in the Connecticut Yankee Nuclear Power Station. However, the trend established here appears to indicate that little or no problems will be encountered with any components except grade mounted storage tanks. Whether or not this trend is valid will be examined in more detail as additional results of the equipment seismic verification become available.

In addition, Table 6-1, which indicates the status of each component along with the calculated seismic safety factors, will also be up-dated as additional results become available.

Table 6-1 Conclusions Regarding Equipment Reviewed

Item No.	Description	Safety Factor Comp.	Safety Factor Sup't.	Conclusions
M-1	ESW Pump			
M-3	Diesel Exhaust Duct	3.1	.15	O.K. for duct Duct structural integrity-WT anchor section requires stiffening
M-4	CVVC regenerative Heat Exchanger			
M-5	Diesel Generator			
M-6	Boric Acid Pump			
M-7	High Pressure Safety Injection Pump			
M-8	RHR Pump			
M-9	RHR Heat Exchanger			
M-10	Boric Acid Tank			
M-11	Demineralized Water Storage Tank			
M-12	Refueling Cavity Water Storaging Tank			
M-13	Steam Driven Aux. Feedwater Pump	2.3	8.3	O.K. for both leak tight integrity and functionality
M-14	Underground 5,000 Gal. Oil Tank			
M-15	Clean Disel Oil Day Tank			

Table 6-1 Conclusions Regarding Equipment Reviewed (Continued)

Item No.	Description	Safety Factor		Conclusions
		Comp.	Sup't.	
M-16	Volume Control Tank			
M-17	Containment Fan Coolers			
M-18	Primary Water Storage Tank			

Table 6-1 Conclusions Regarding Equipment Reviewed

Item No.	Description	Safety Factor		Conclusions
		Comp.	Sup't.	
E-1	Battery Rack			
E-2	MCC #1	12	4	Structural and Anchorage O.K. for structural integrity. No evaluation is made of electrical function.
E-3	Switchgear (Dies. Gen. Rm)			
E-4	Control Panel (Dies. Gen. Rr.)			
E-5	Engine Mounted Control Panel (on Dies. Gen.)			
E-6	4160-480 V Switchgear			
E-7	Transformers (Switchgear Rm.)			
E-8	MCC's #5 & #6			
E-9	Battery Charger			
E-10	MCC #3			
E-11	Main Control Board			
E-12	Emergency Power Conrol Board			
E-13	MCC #8			

7.0 REFERENCES

7.1 Housner, G.W., "Behavior of Structures during Earthquakes", Journal of the Engineering Mechanics Div. Proceedings of ASCE, October, 1959.

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7.3 ASME Section III, Division 1, "Nuclear Power Plant Components, Boiler and Pressure Vessel Code, "American Society of Mechanical Engineers, 1980.

7.4 Stevenson, J.D., "Structural Damping Values as a Function of Dynamic Response Stress and Deformation Levels," Paper K11/1 presented at 5th SMIRT Conf., Berlin, 14-20 August 1979.

7.5 C.K. McDonald-"Seismic Qualification by Analysis of Nuclear Power Plant Mechanical Components" Proceedings of the U.S. National Conference on Earthquake Engineering-Earthquake Engineering Research Institute, June, 1975.

7.6 "Nuclear Reactors and Earthquakes," TID-7024, U.S. A.E.C. Div. of Technical Information, August, 1963.

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7.9 "U.S. Steel Design Manual" R.L. Brockenbrough, U.S. Steel Corp., 2nd Printing, Nov. 1968.

7.10 "Interaction of Building and Ground, "H. Tajimi, Earthquake Engineering, Nov. 1968, Shokokusha Publishing Co. (In Japanese).

7.11 "Vibration of Soils and Foundations, "F.E. Richart, Jr., R.O. Woods, J.R. Hall, Prentice-Hall,, Inc. 1970.

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7.13 Task Group on Pipe Reactions on Pumps, minutes of the May 25, 1982 meeting, Pressure Vessel Research Committee Welding Research Council.

7.14 Ware, A.G. A Survey of Experimentally Determined Values in Nuclear Power Plant Piping Systems, EGG-2143, NUREG/CR-2406, Prepared by EGG Idaho, Inc. for U.S. Nuclear Regulatory Commission, Nov. 1981.

APPENDIX A

CONTAINMENT SHELL FLOOR RESPONSE SPECTRA

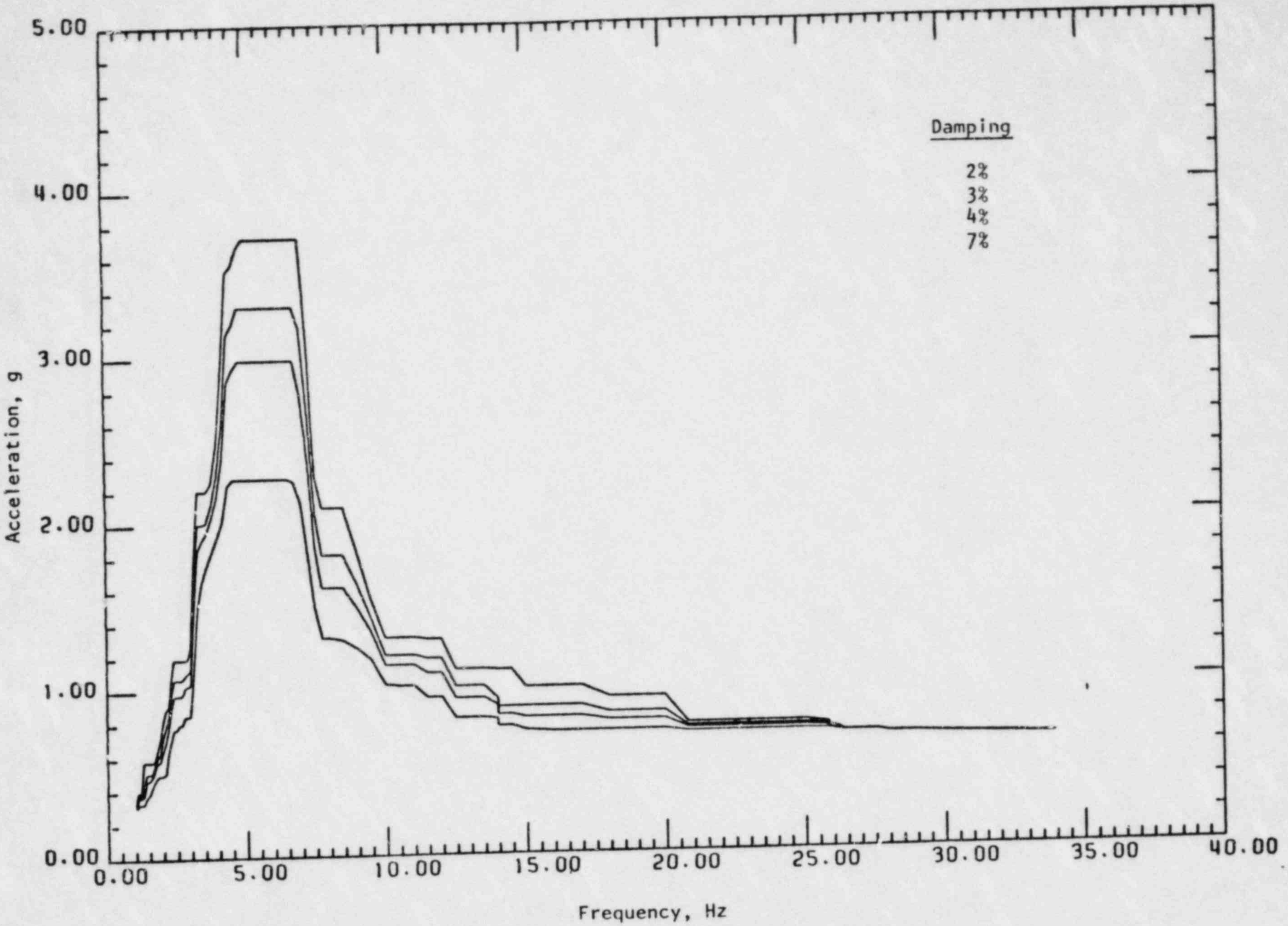

FIGURE 1A: HORIZONTAL (RADIAL DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 191 FT 0 IN

147

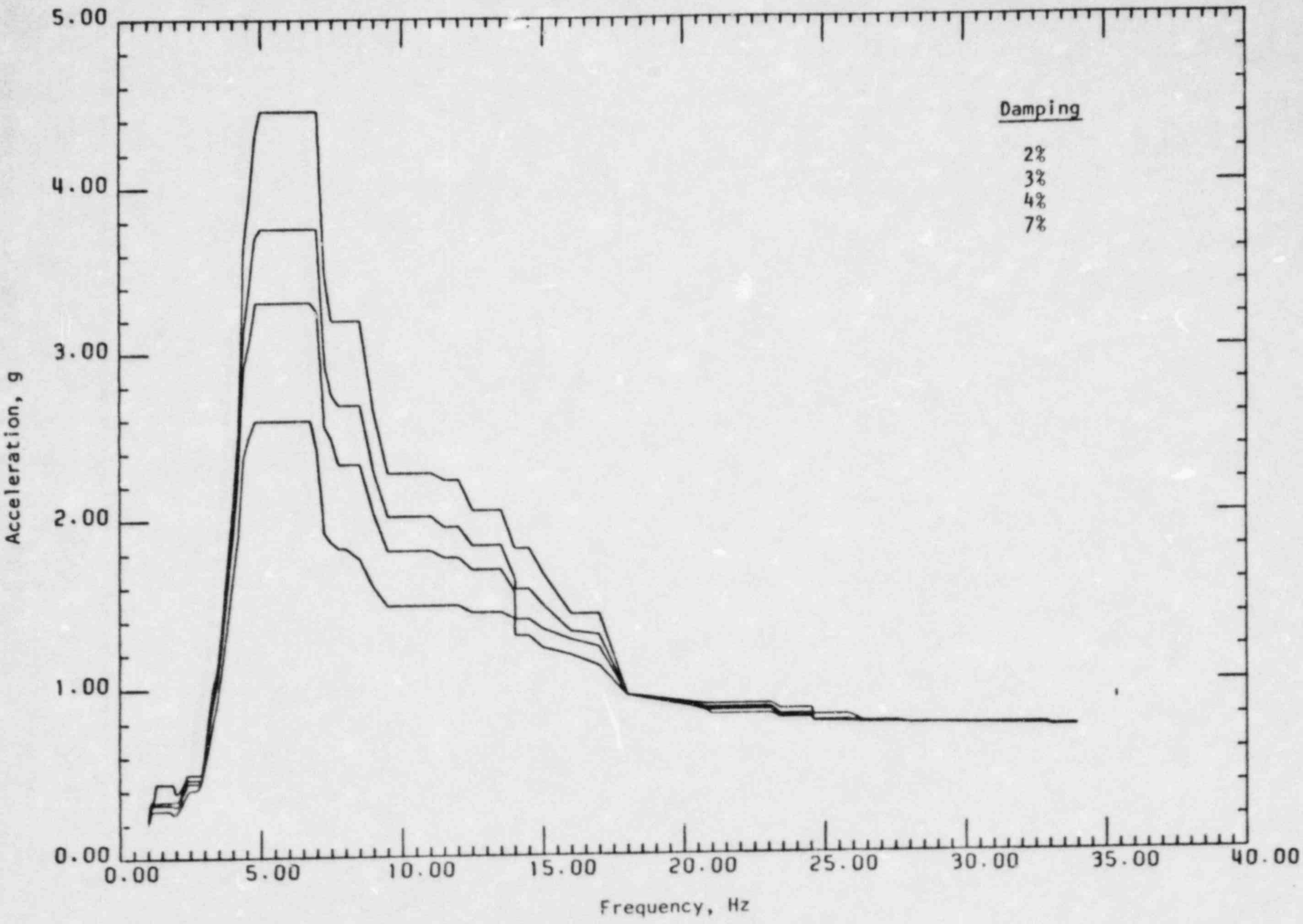

FIGURE 1B: HORIZONTAL (TANGENTIAL DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 191 FT 0 IN

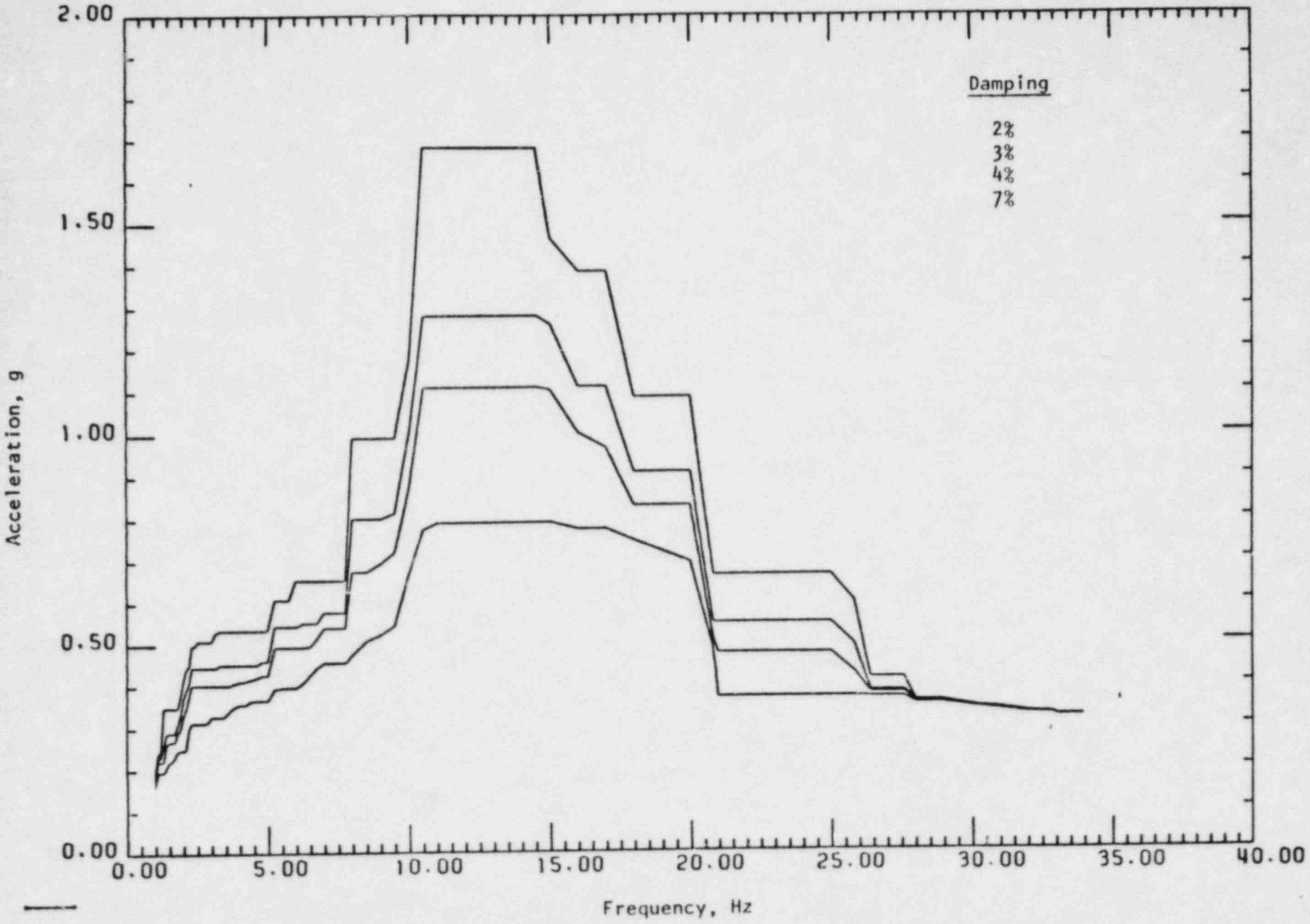

FIGURE 1C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 191 FT 0 IN

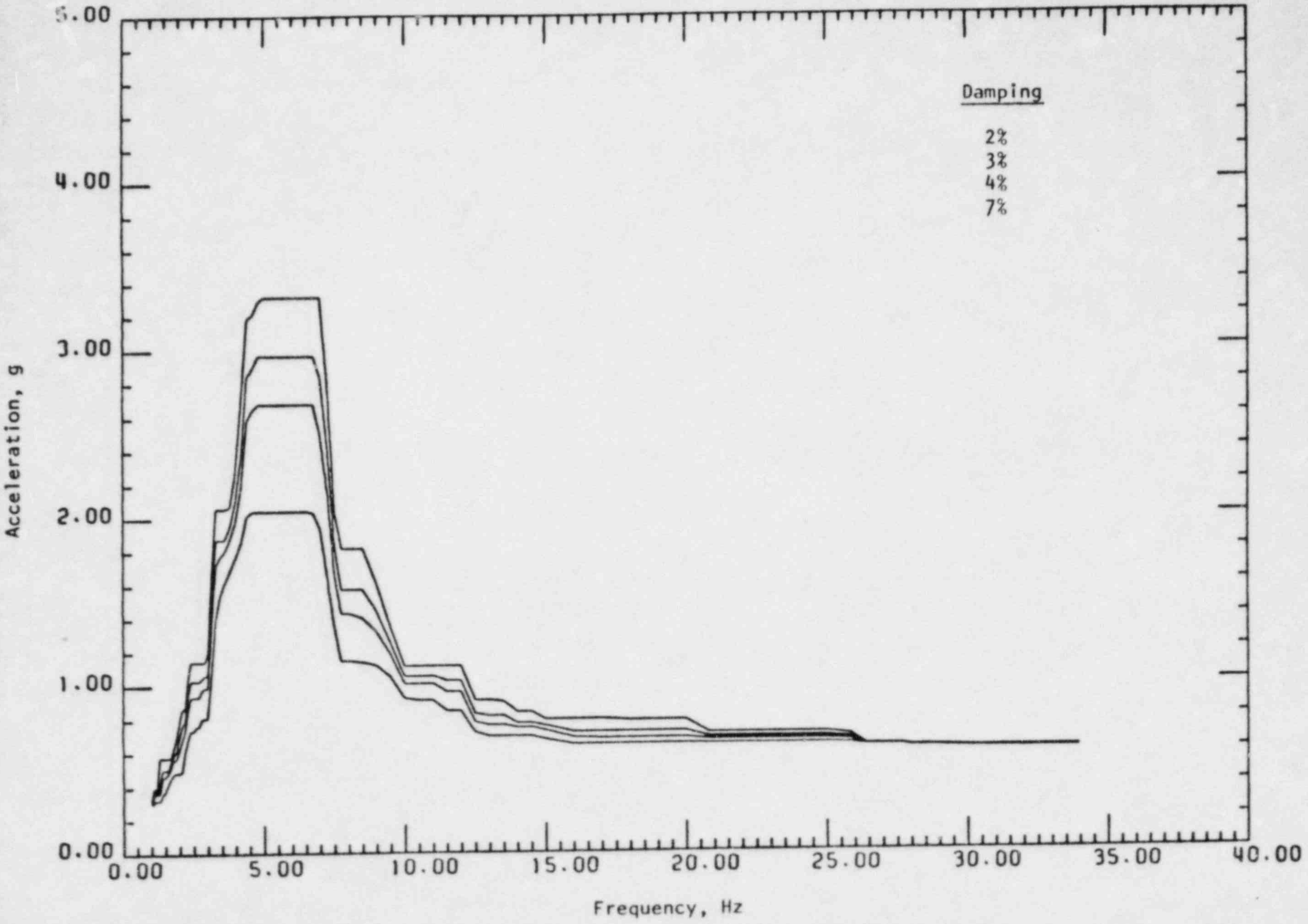

FIGURE 2A: HORIZONTAL (RADIAL DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 165 FT 0 IN

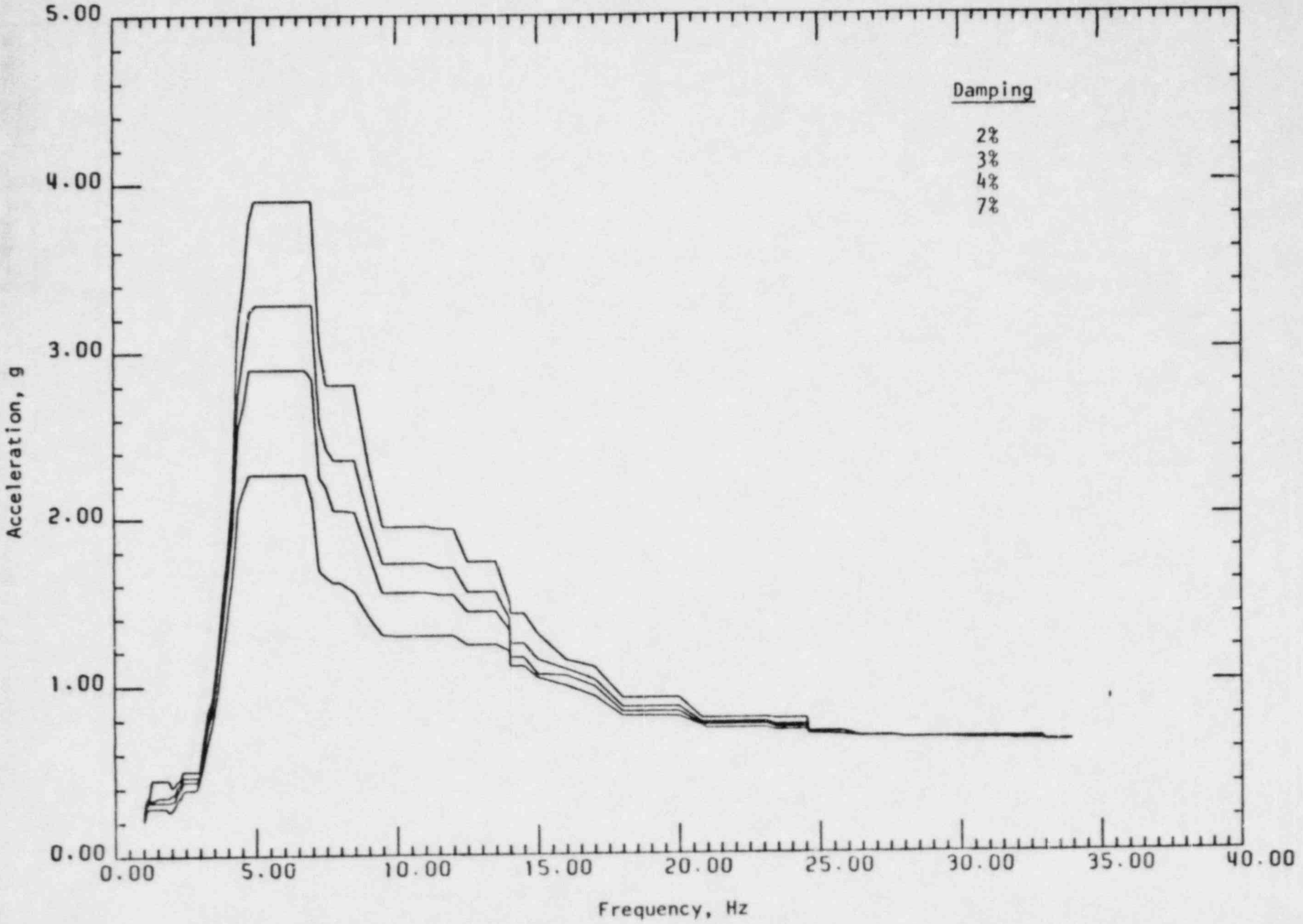

FIGURE 2B: HORIZONTAL (TANGENTIAL DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 165 FT 0 IN

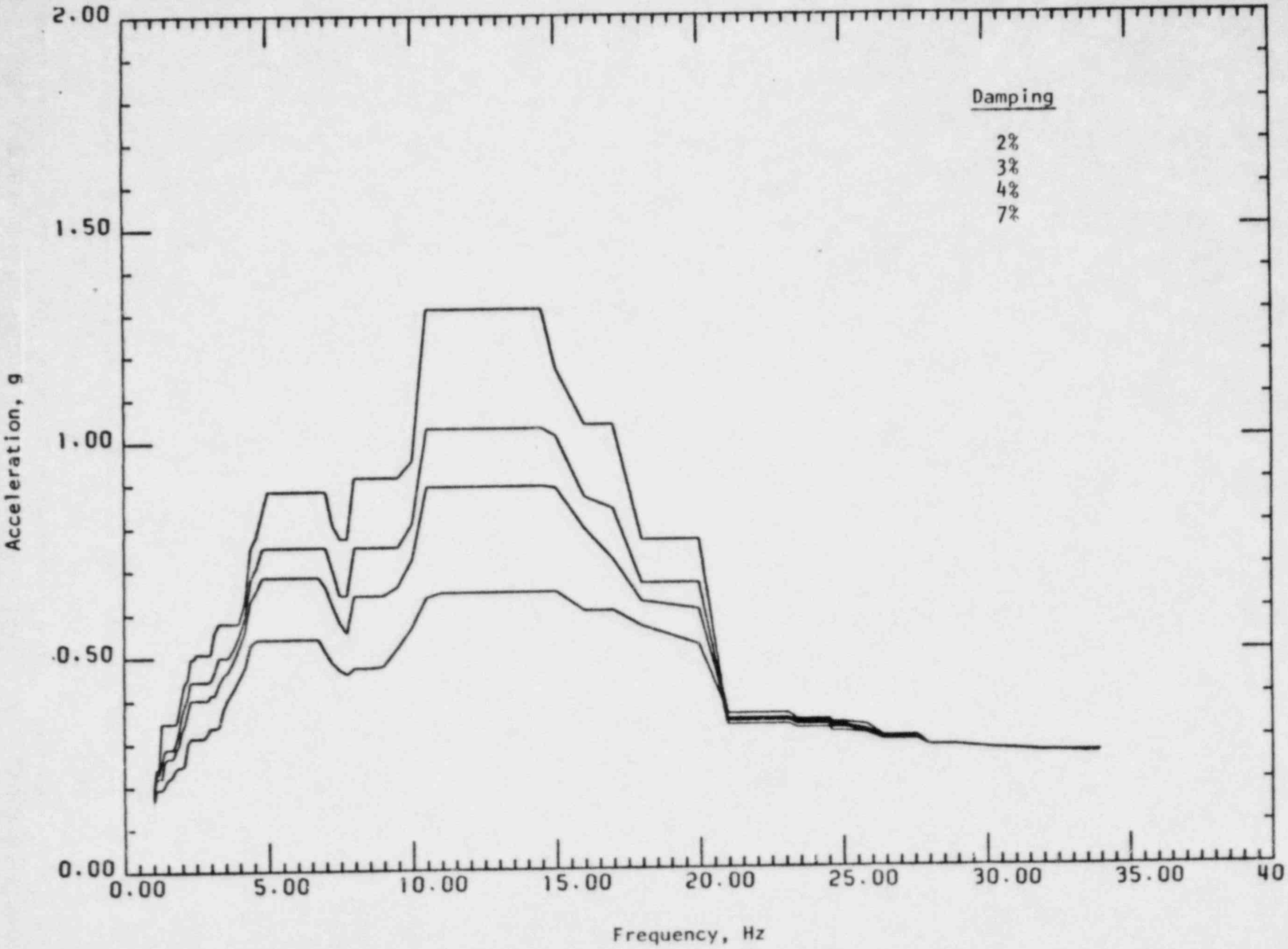

FIGURE 2C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA
FOR THE CONTAINMENT SHELL, ELEVATION 165 FT 0 IN

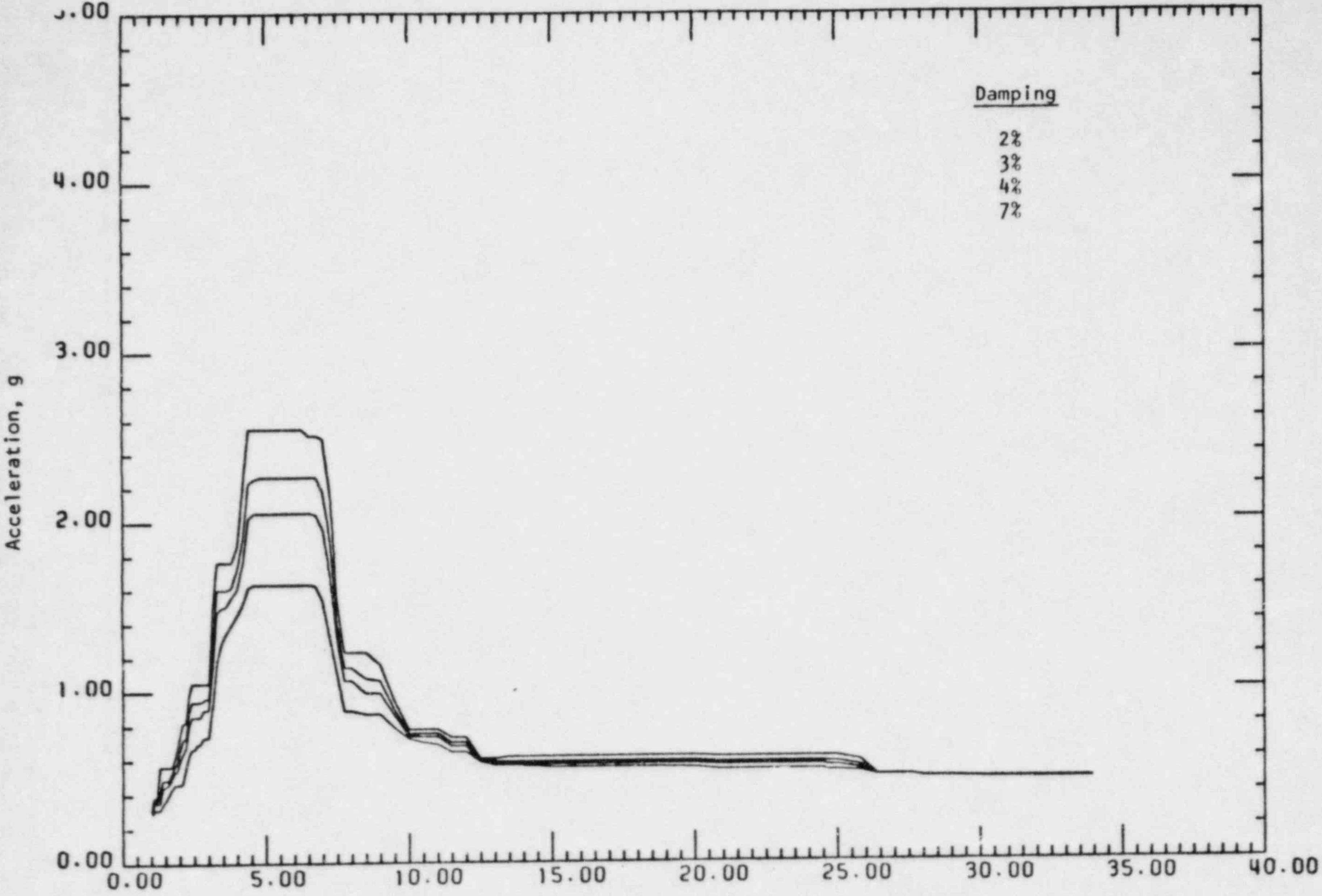

FIGURE 3A: HORIZONTAL (RADIAL DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 121 FT 0 IN

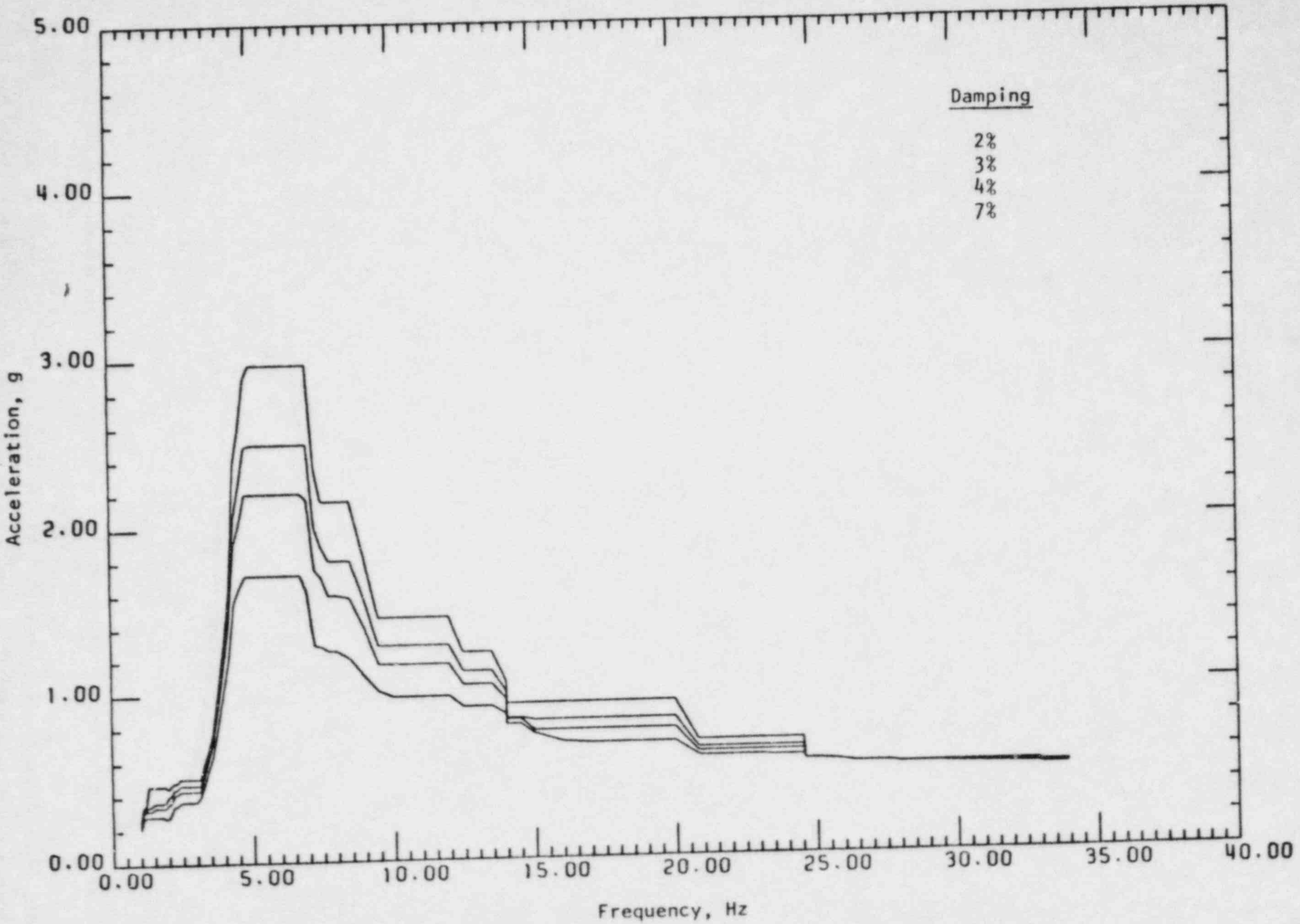

FIGURE 3B: HORIZONTAL (TANGENTIAL DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 121 FT 0 IN

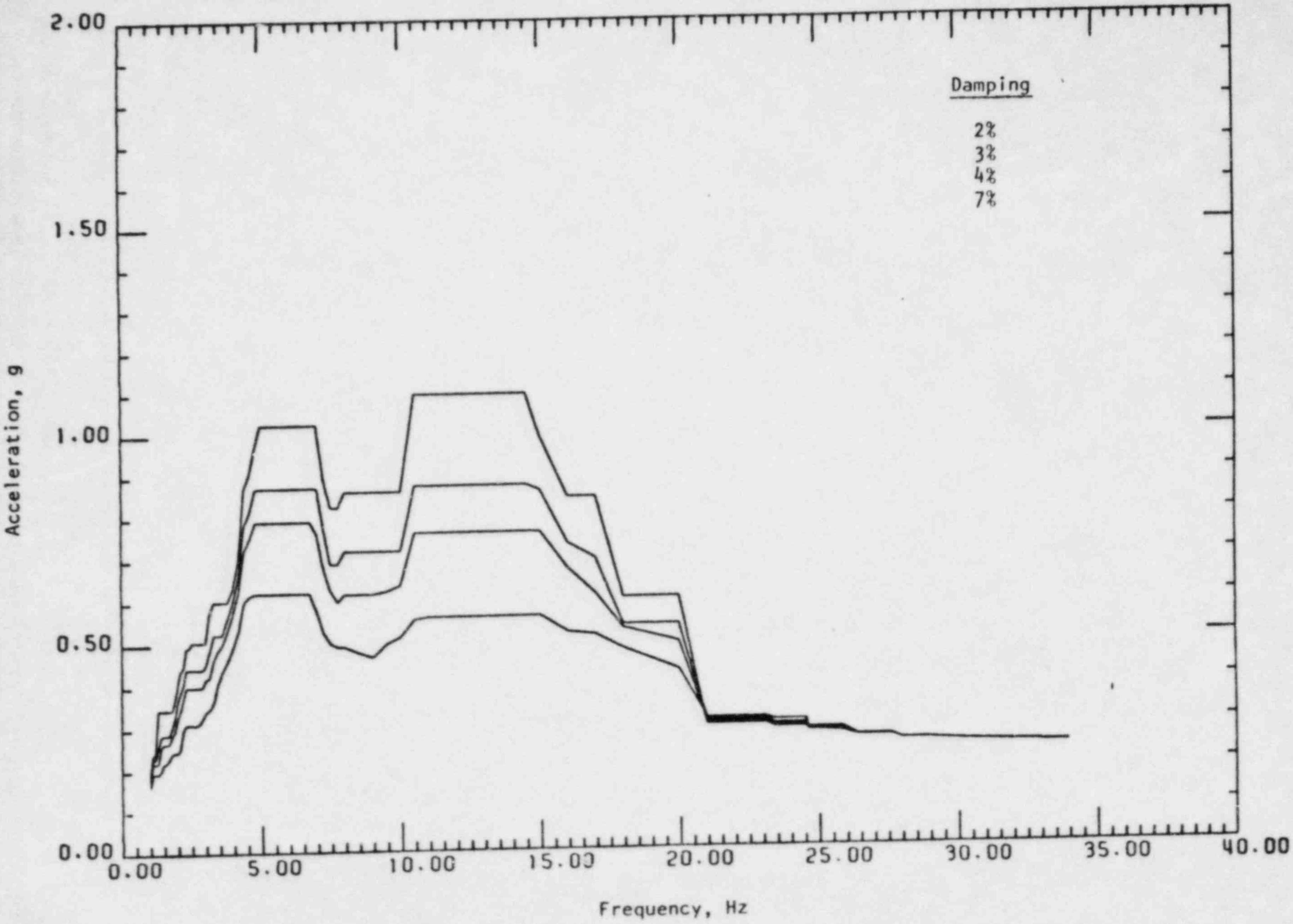

FIGURE 3C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 121 FT 0 IN

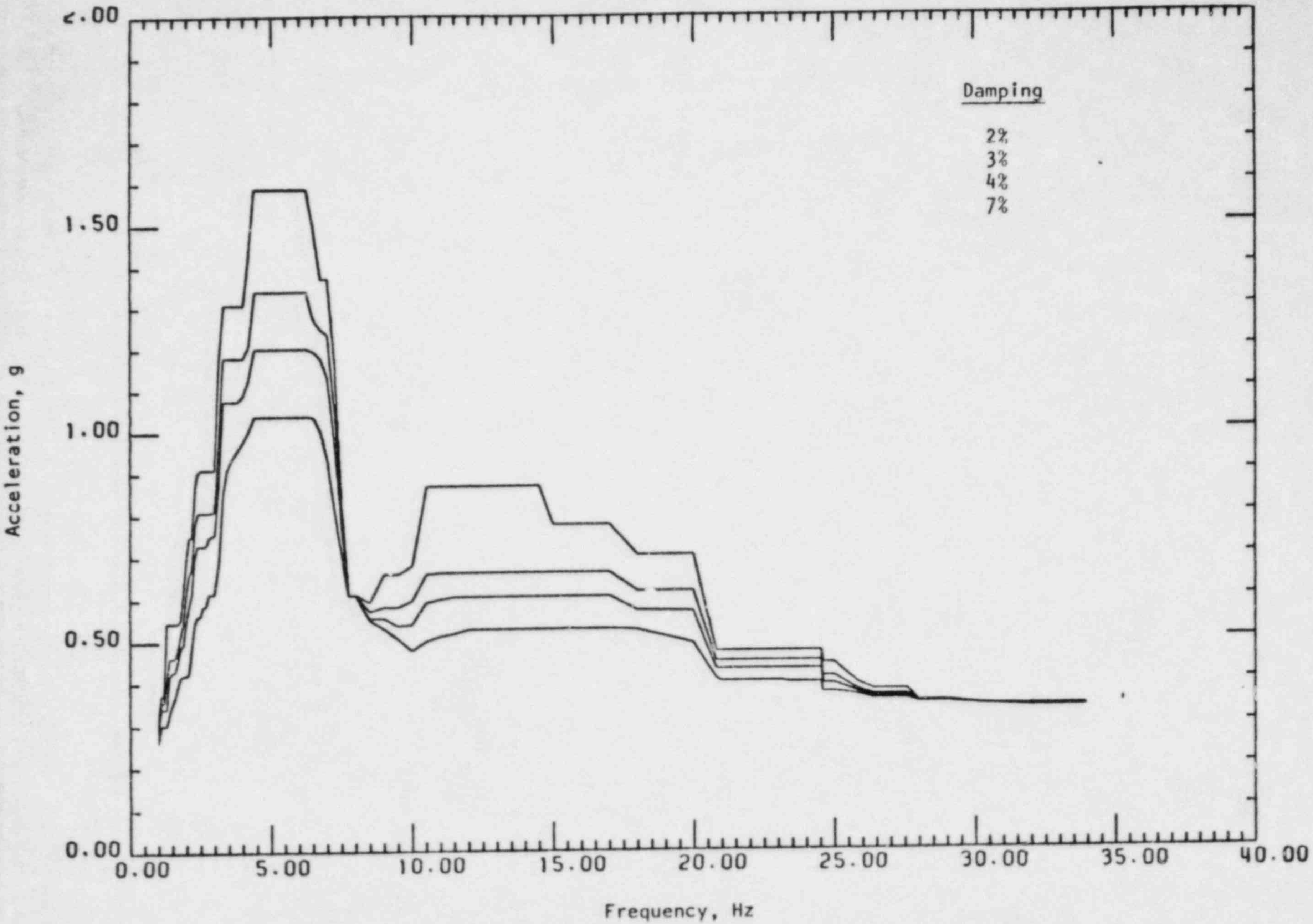

FIGURE 4A: HORIZONTAL (RADIAL DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 70 FT 0 IN

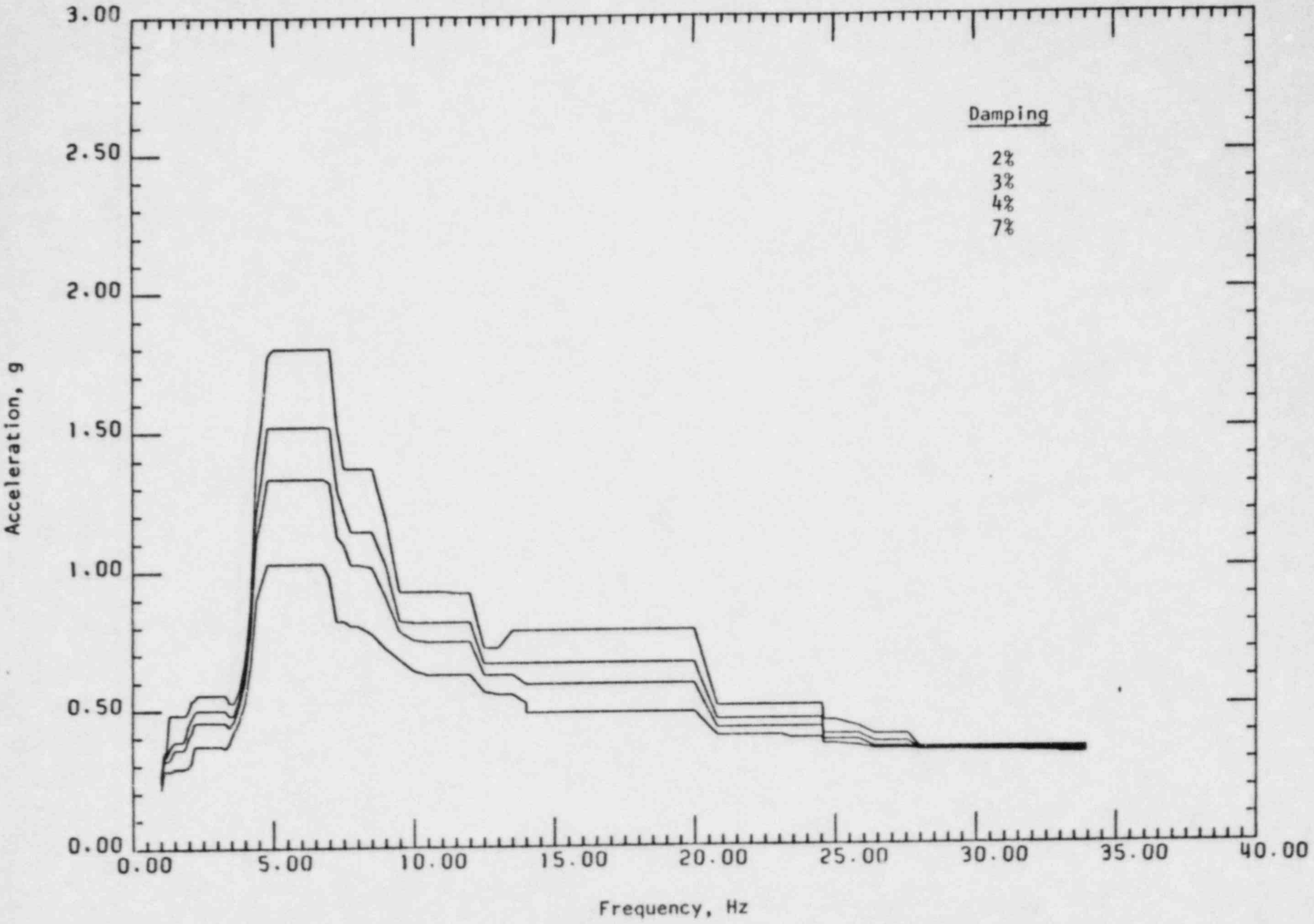

FIGURE 4B: HORIZONTAL (TANGENTIAL DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 70 FT 0 IN

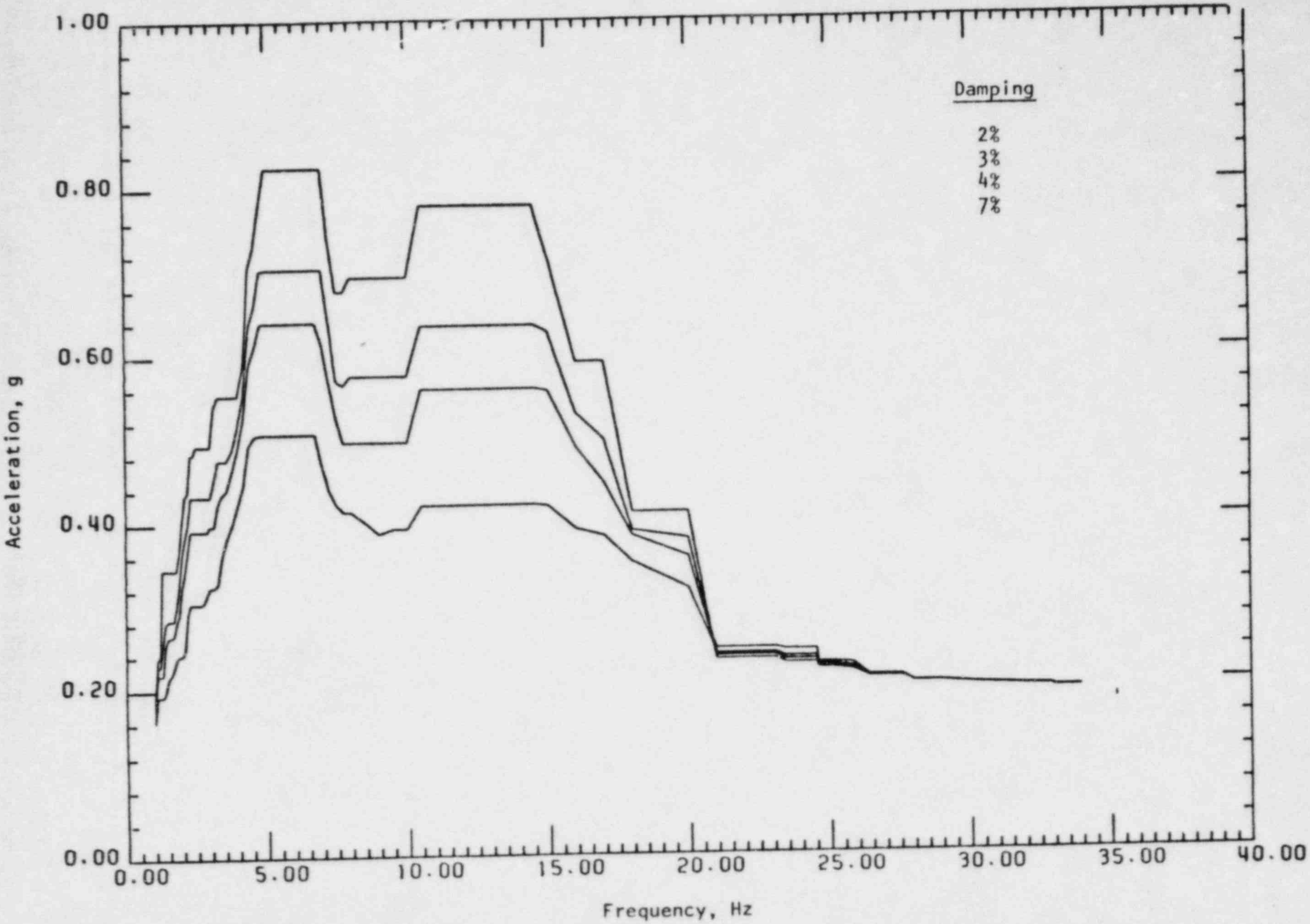

FIGURE 4C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA
FOR THE CONTAINMENT SHELL, ELEVATION 70 FT 0 IN

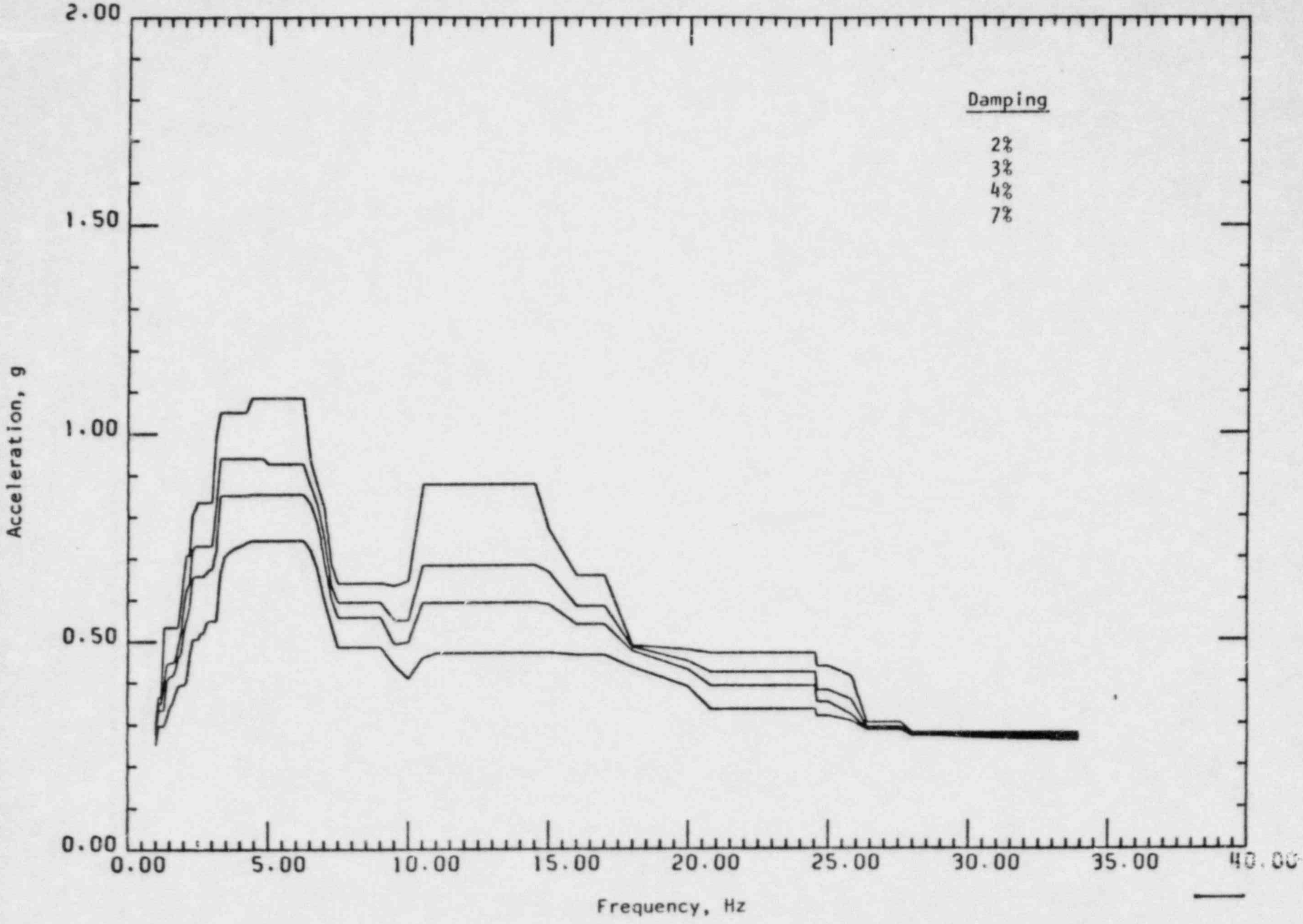

FIGURE 5A: HORIZONTAL (RADIAL DIRECTION) FLOOR ACCELERATION
RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 40 FT 0 IN

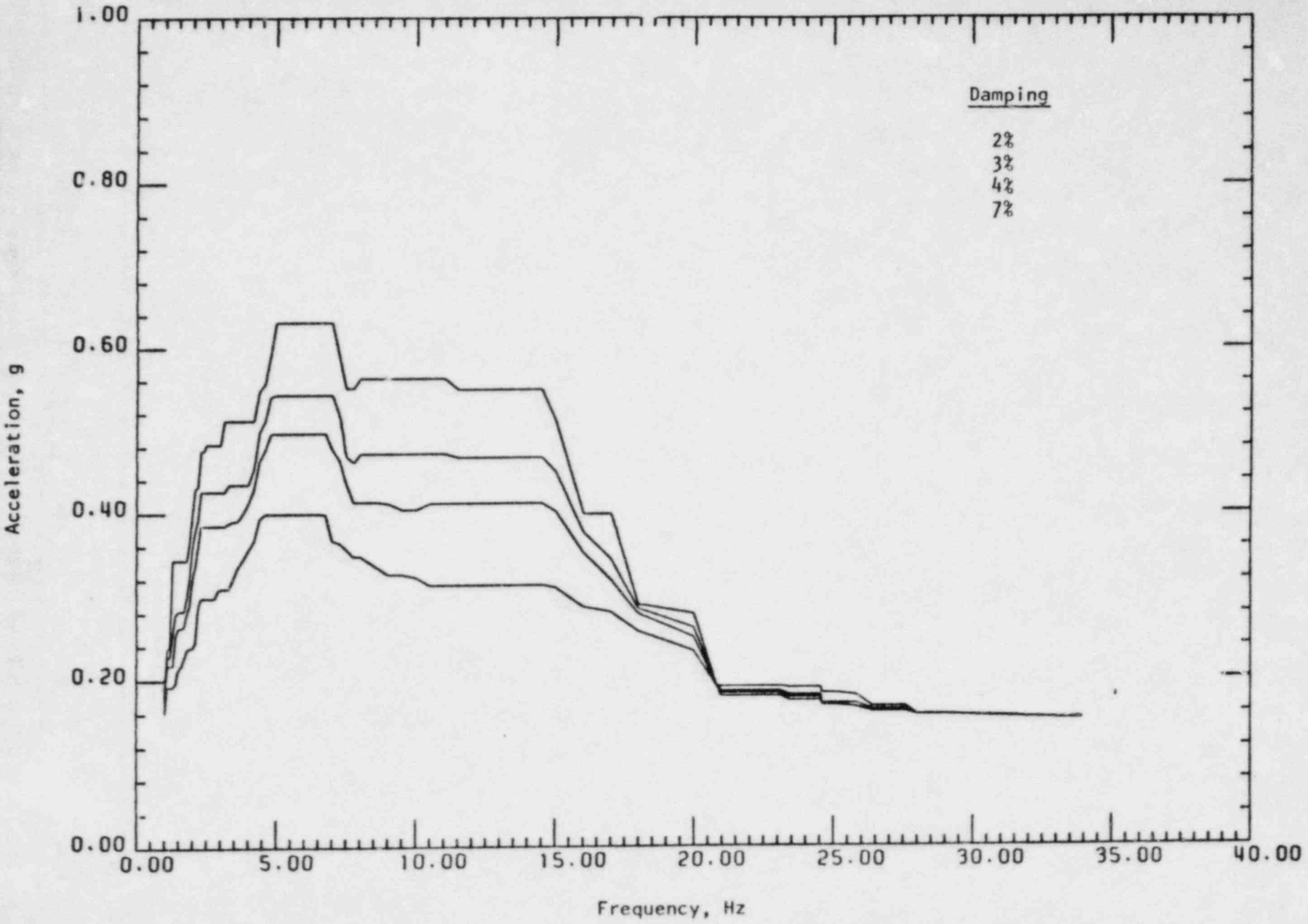

FIGURE 5B: HORIZONTAL (TANGENTIAL DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 40 FT 0 IN

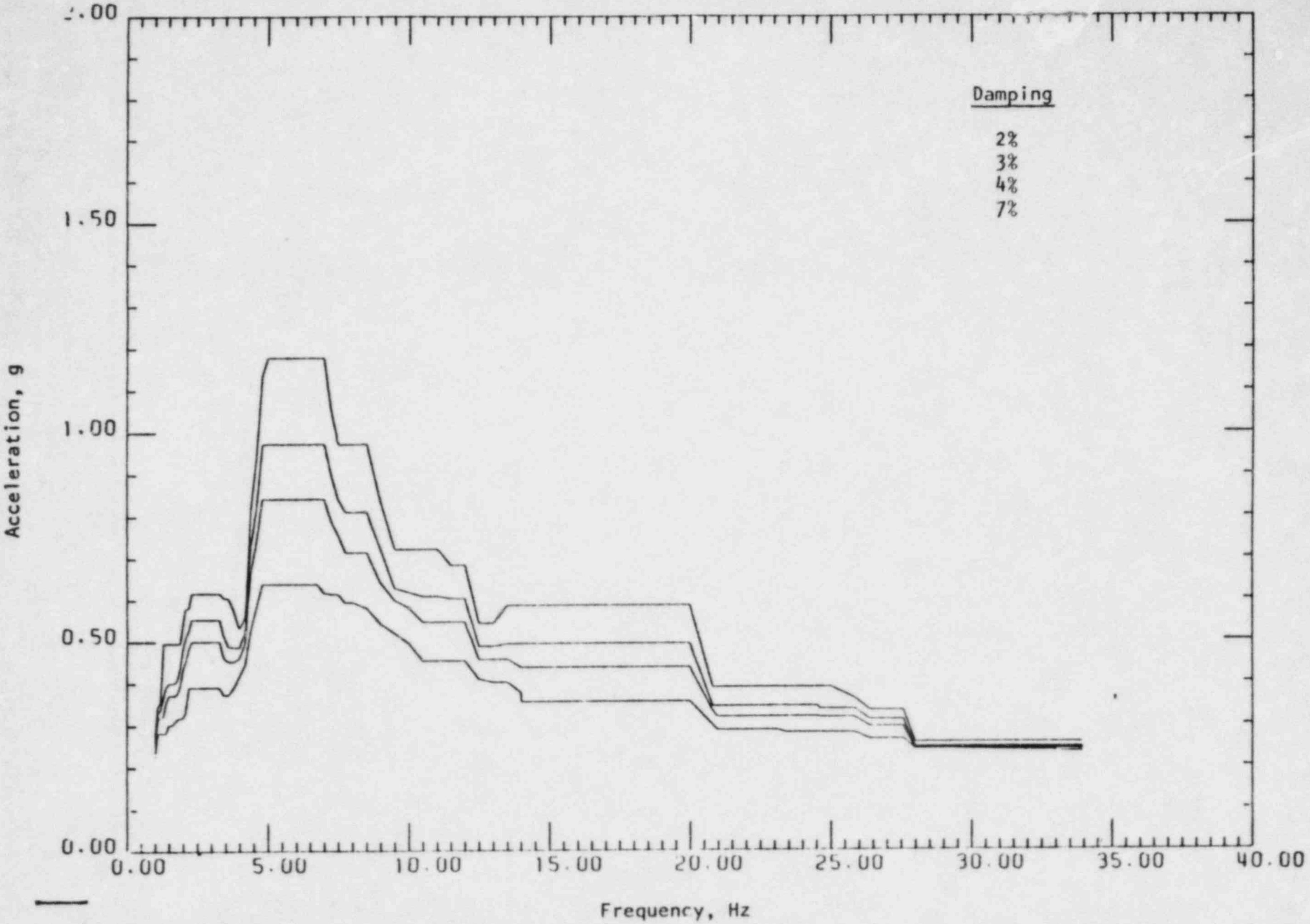

FIGURE 5C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 40 FT 0 IN

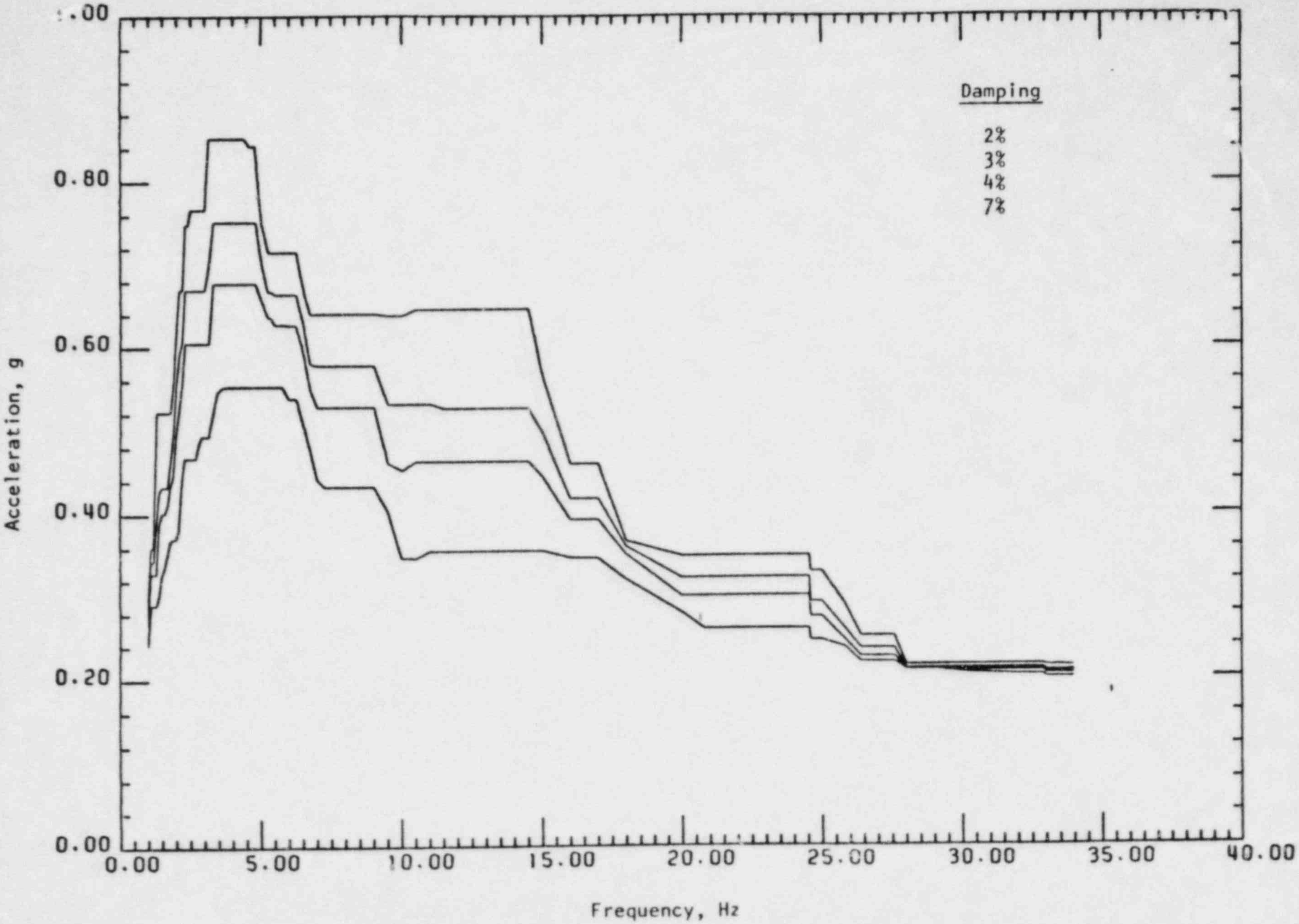

FIGURE 6A: HORIZONTAL (RADIAL DIRECTION) FLOOR ACCELERATION
RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 20 FT 0 IN

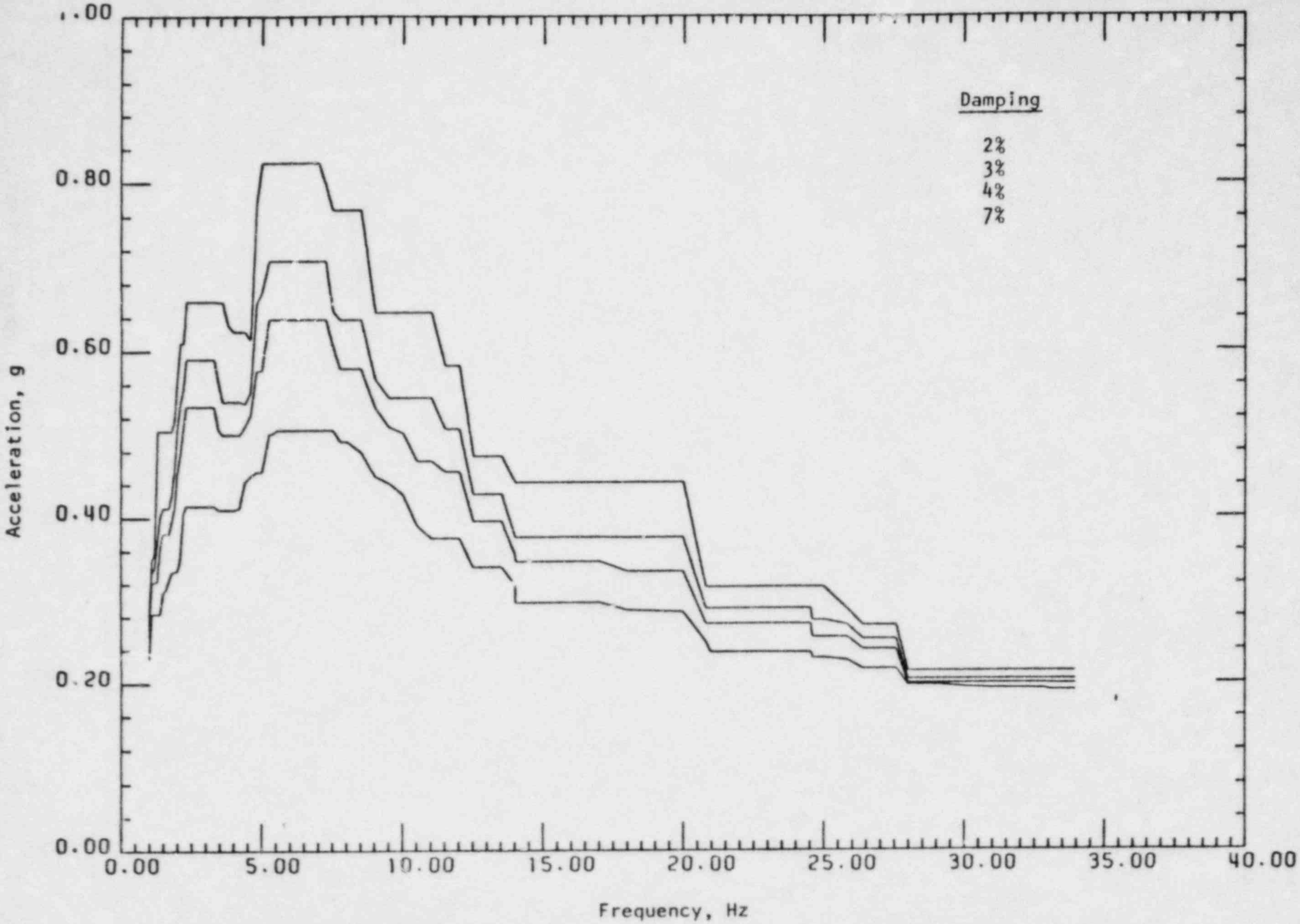

FIGURE 6B: HORIZONTAL (TANGENTIAL DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 20 FT 0 IN

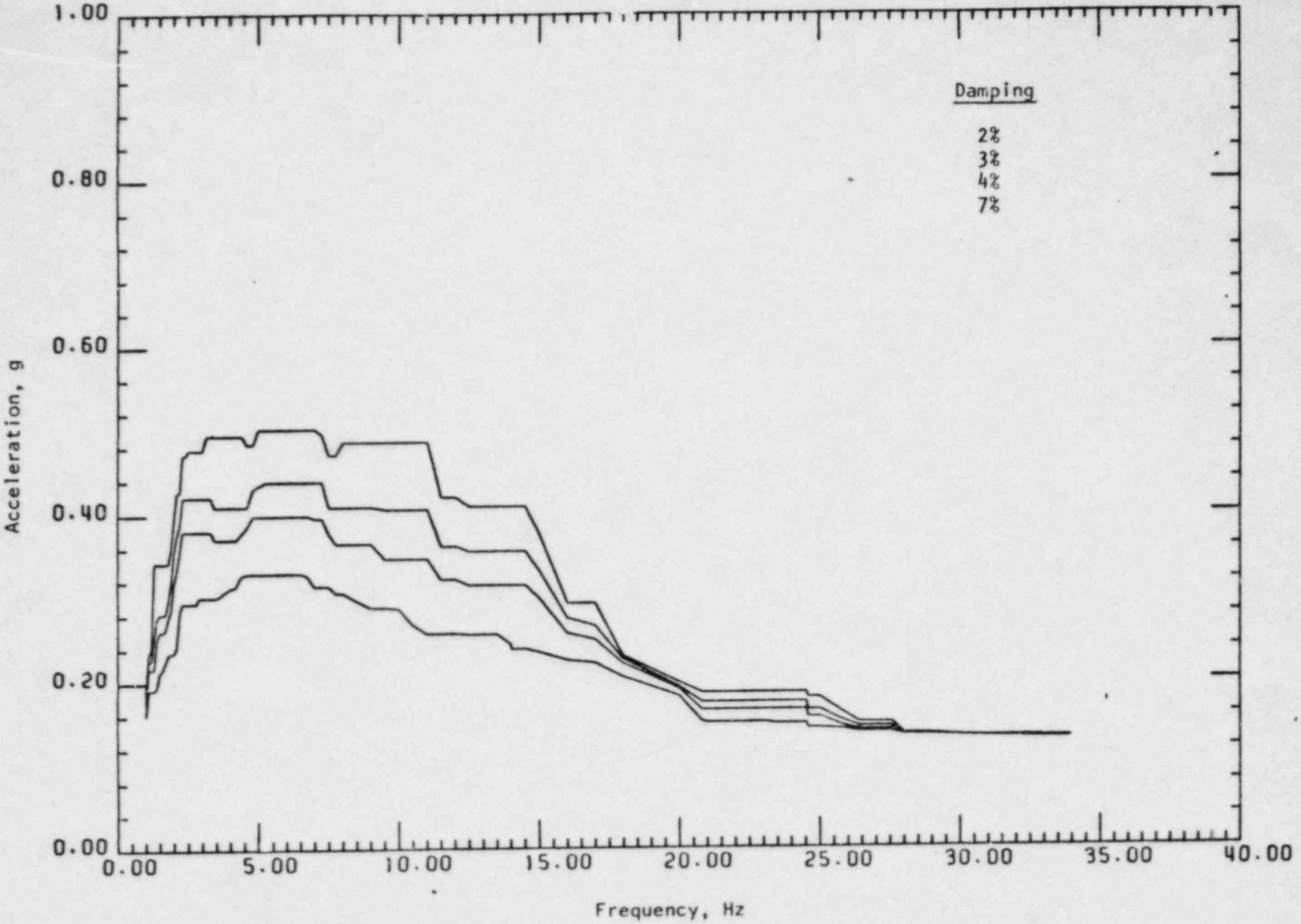

FIGURE 6C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA FOR THE CONTAINMENT SHELL, ELEVATION 20 FT 0 IN

APPENDIX B

SCREENWELL HOUSE AND PAB FLOOR RESPONSE SPECTRA

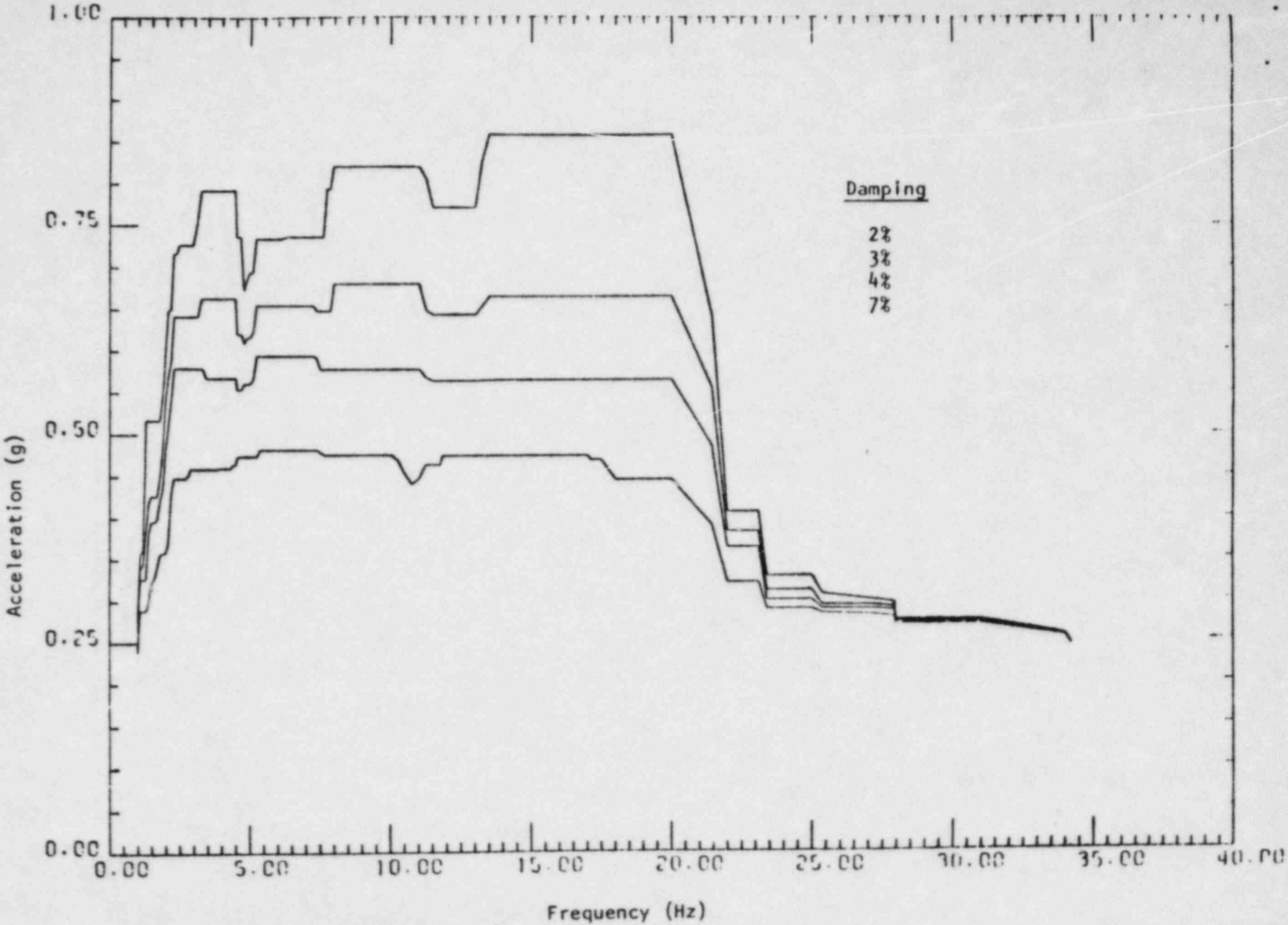

Figure 1: East-West Floor Response Spectra For Elevation 8'-0", Screenwell House, Connecticut Yankee Atomic Power Plant

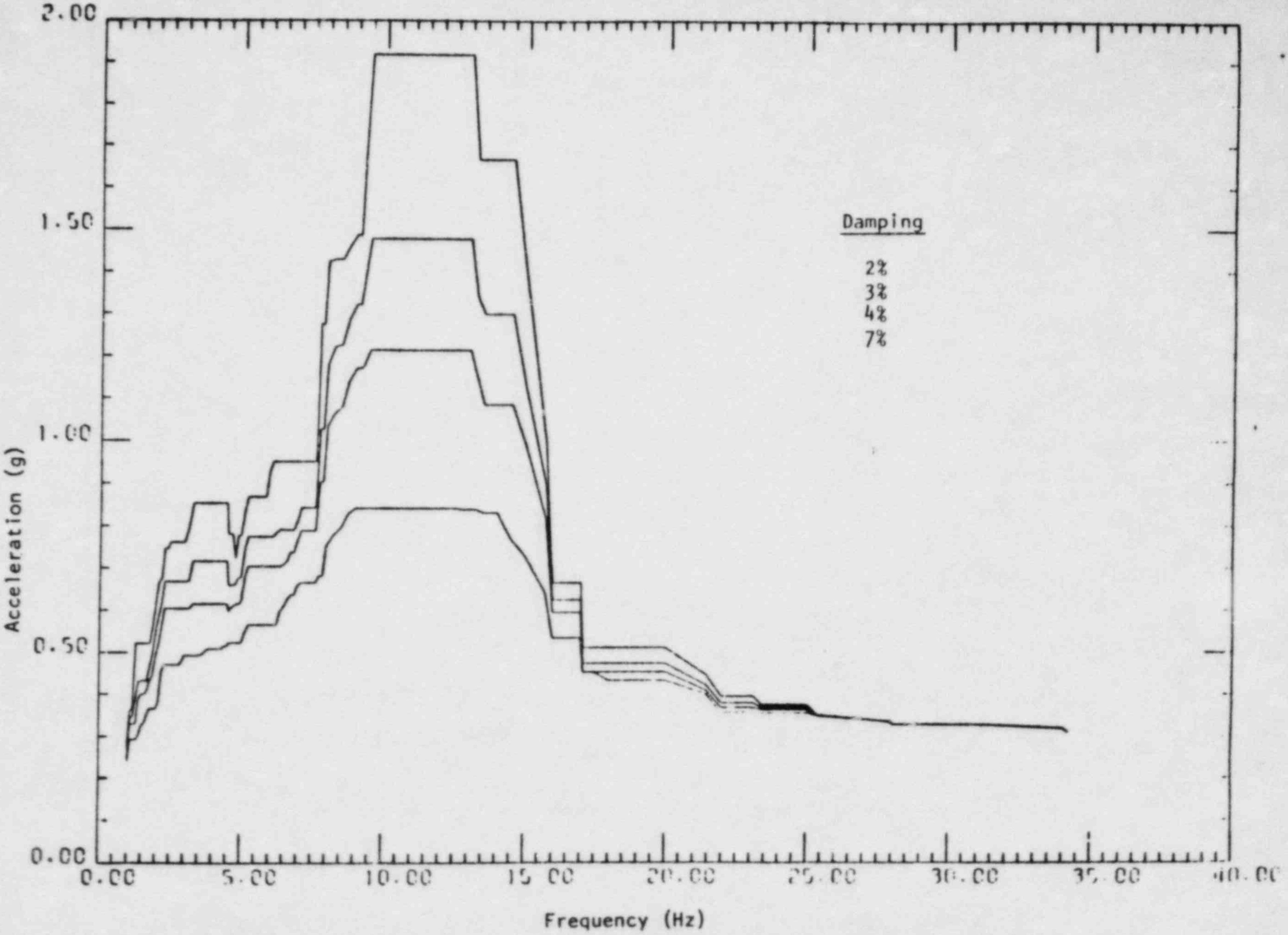

Figure 2: North-South Floor Response Spectra For Elevation 8'-0", Screenwell House, Connecticut Yankee Atomic Power Plant

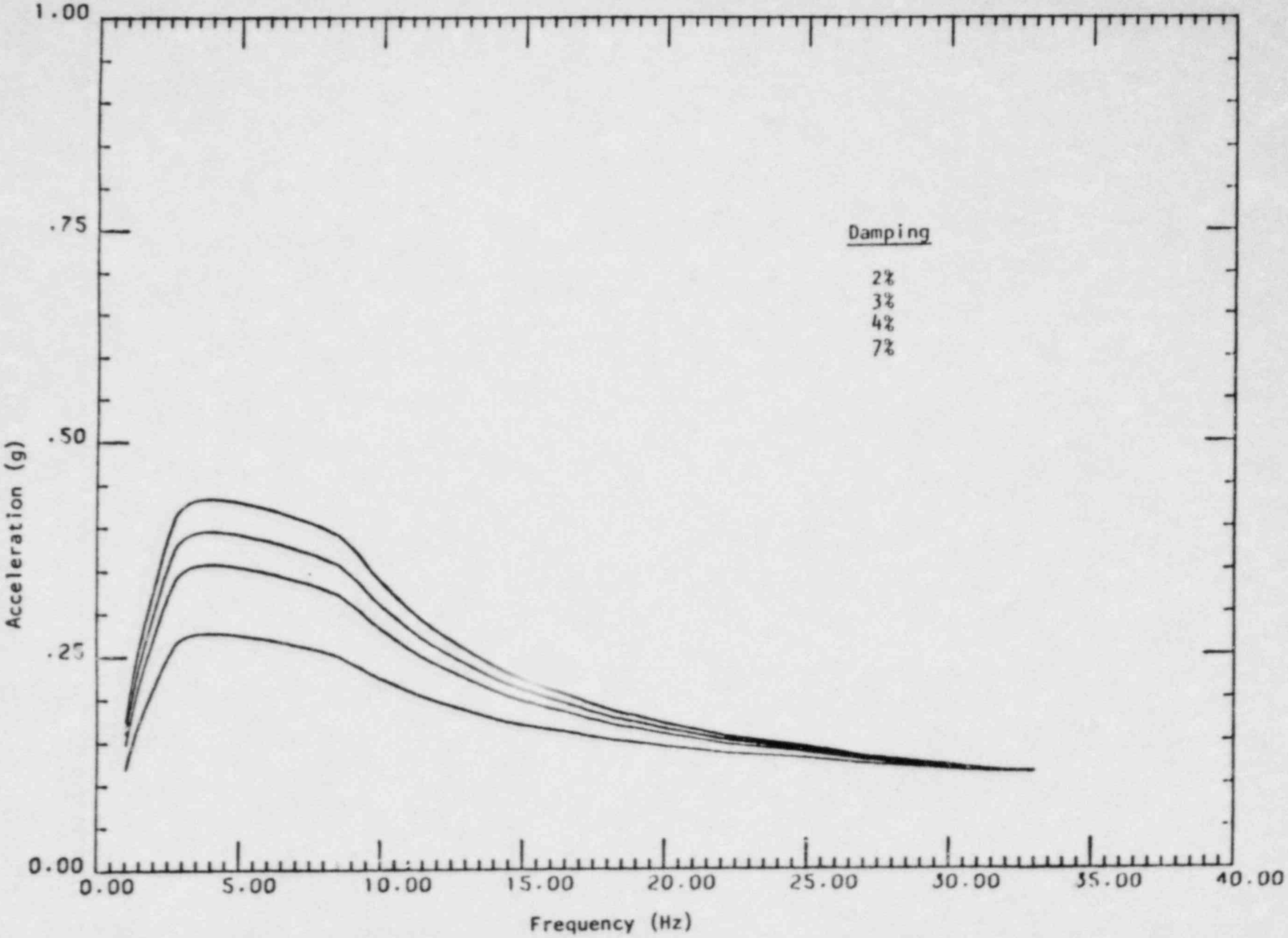

Figure 3: Vertical Floor Response Spectra For Elevation 8'-0"
Screenwell House, Connecticut Yankee Atomic Power Plant

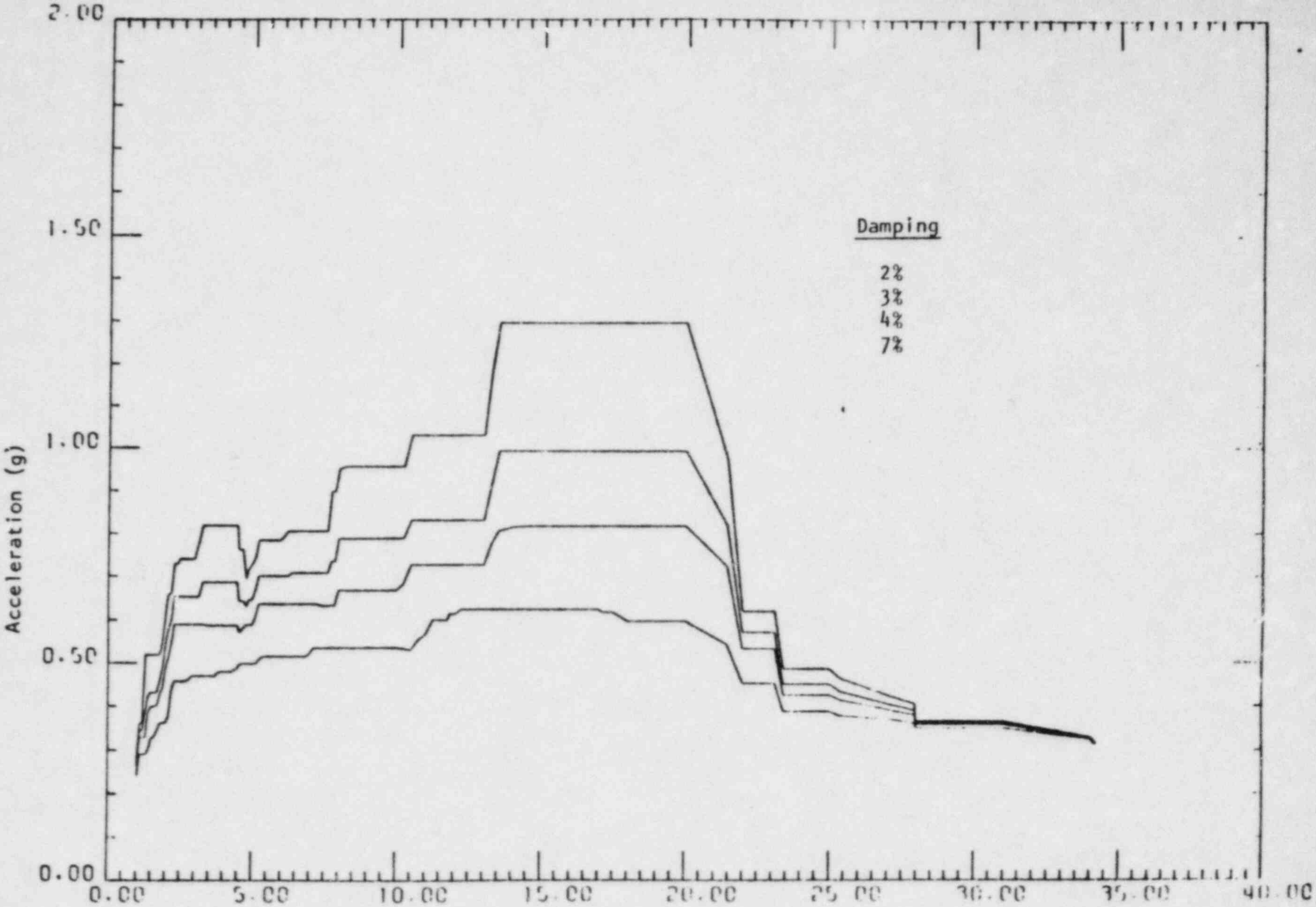

Figure 4: East-West Floor Response Spectra For Elevation 21'-6"
Screenwell House, Connecticut Yankee Atomic Power Plant

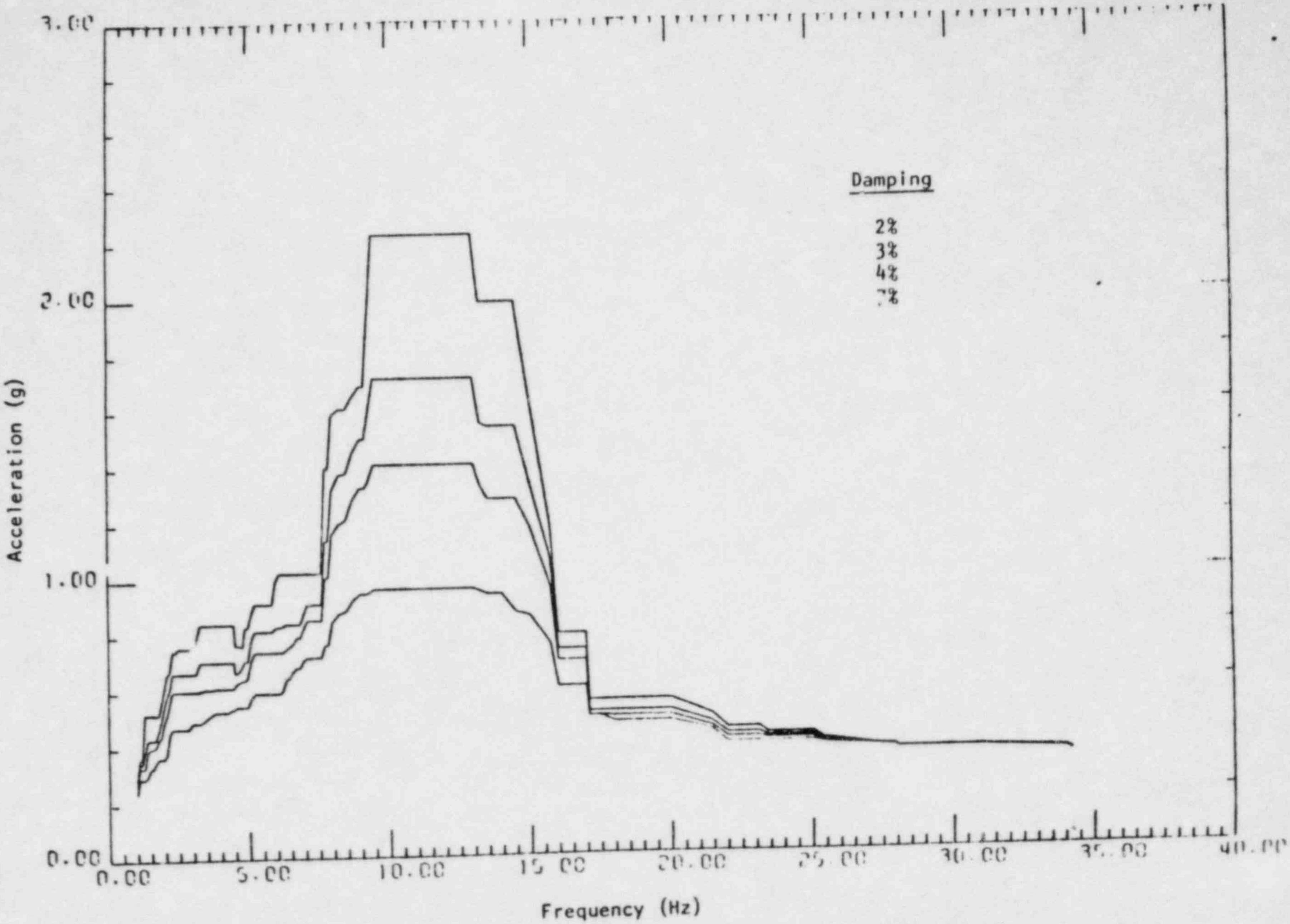

Figure 5: North-South Floor Response Spectra For Elevation 21'-6", Screenwell House, Connecticut Yankee Atomic Power Plant

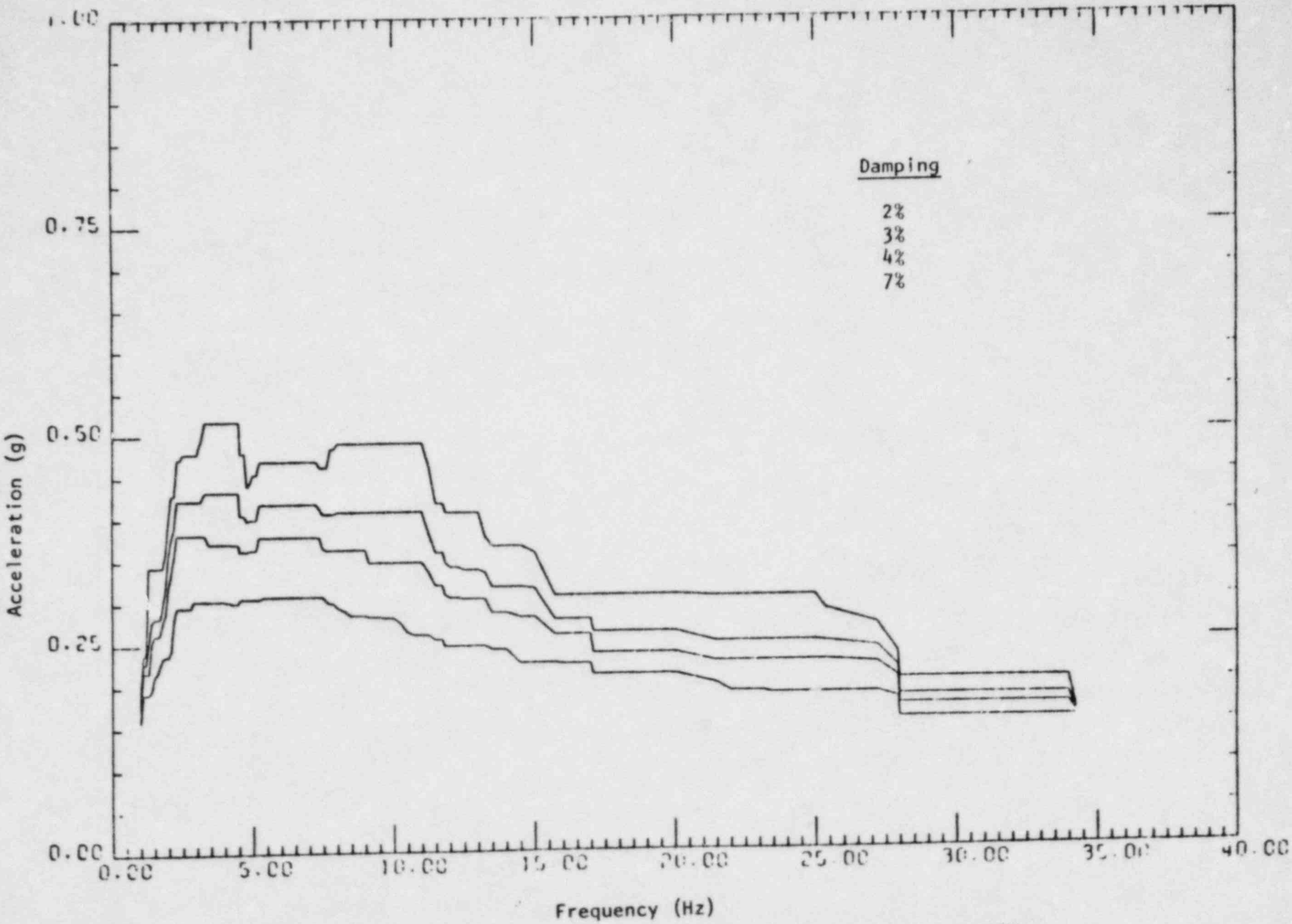

Figure 6: Vertical Floor Response Spectra For Elevation 21'-6", Screenwell House, Connecticut Yankee Atomic Power Plant

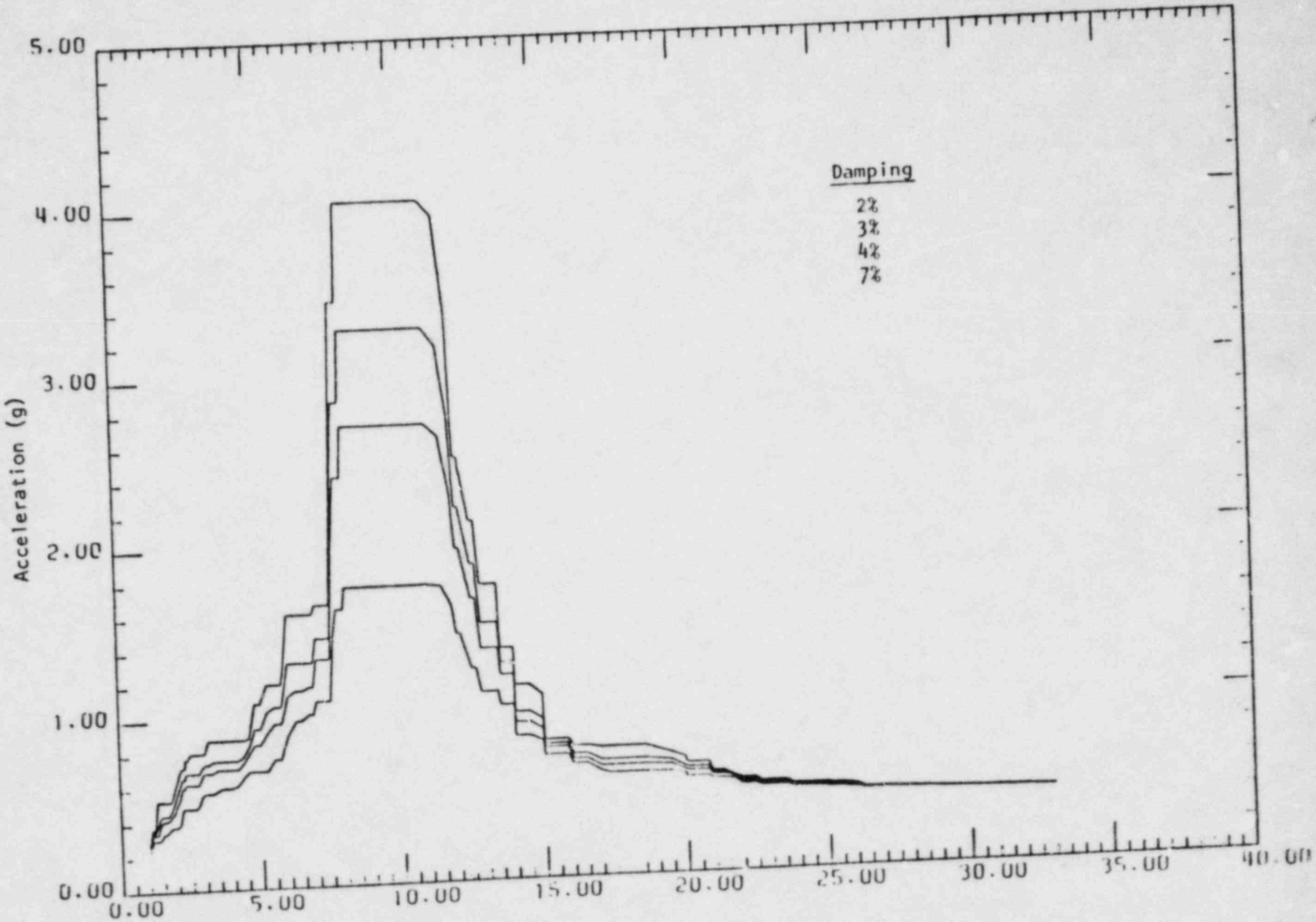

Figure 7: East-West Floor Response Spectra For Elevation 15'-6", Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

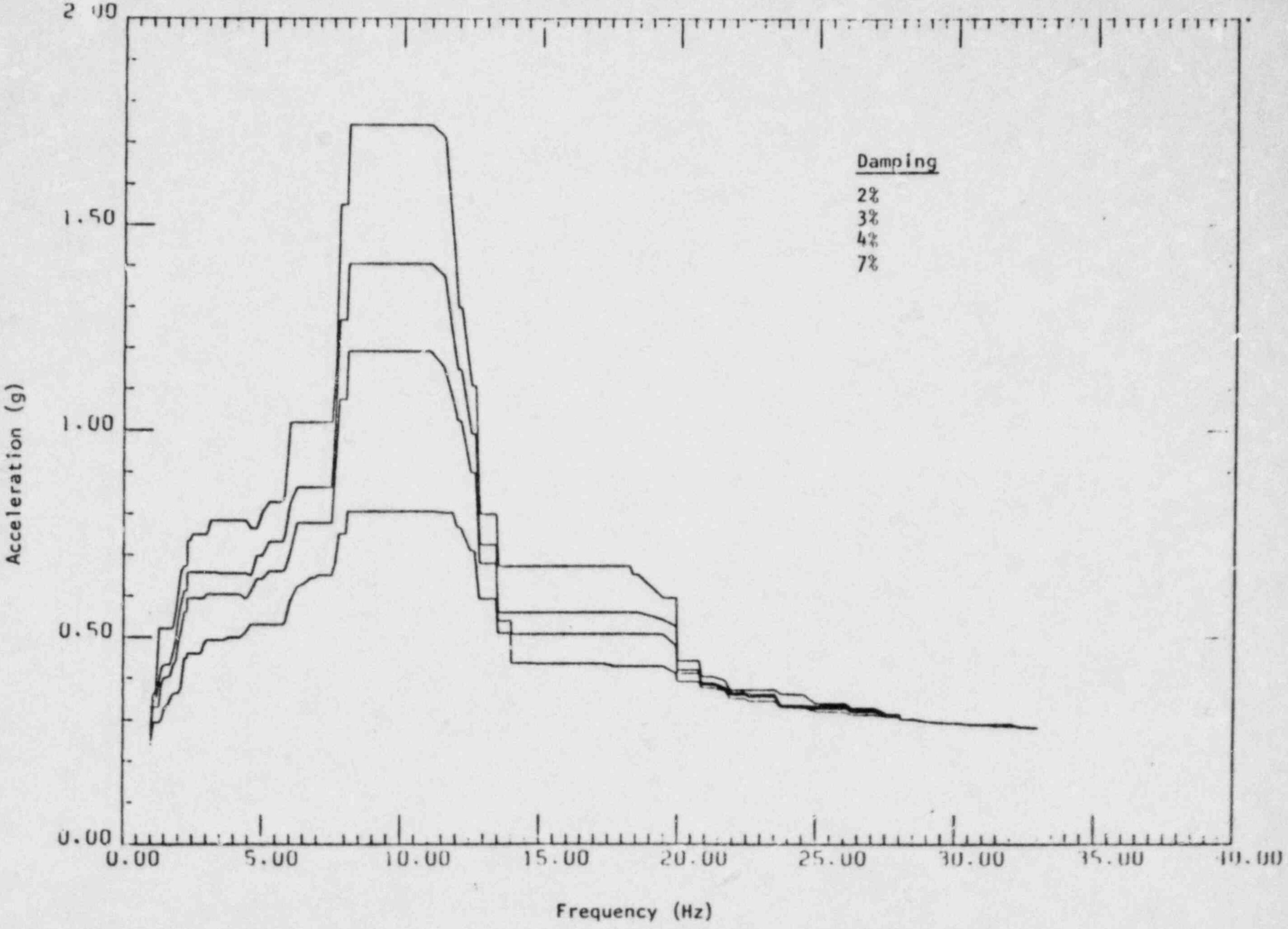

Figure 8: North-South Floor Response Spectra For Elevation 15'-6", Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

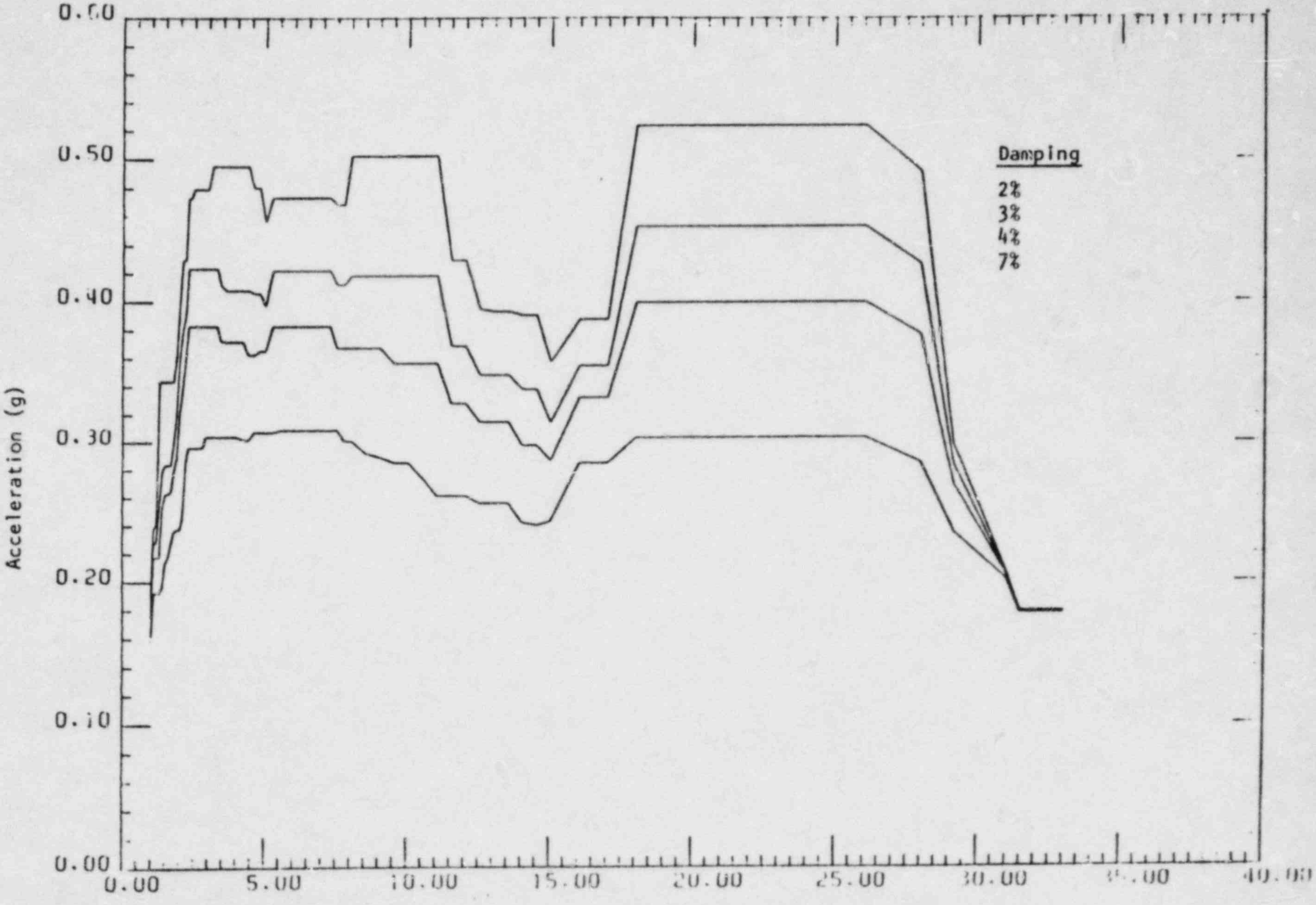

Figure 9: Vertical Floor Response Spectra For Elevation 15'-6"
At Floor Area Enclosed By Column Lines 10¼, 11¼, K and H,
Primary Auxi ry Building, Connecticut Yankee Atomic Power Pla

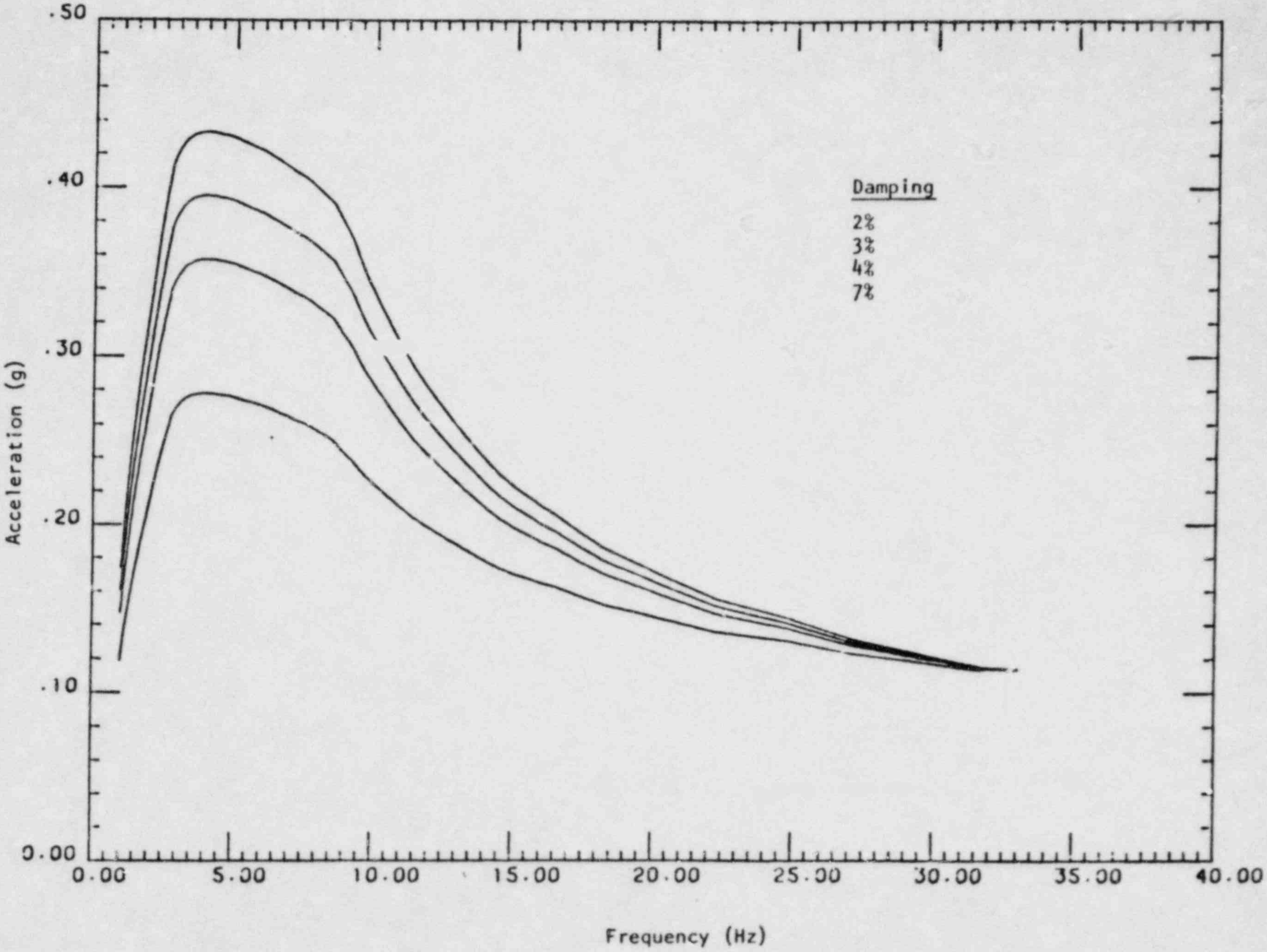

Figure 10: Vertical Floor Response Spectra For Elevation 15'-6", At Floor Area Enclosed By Column Lines 11 3/4, 13 1/4, L and G Primary Auxiliar uilding, Connecticut Yankee Atomic Power Plant

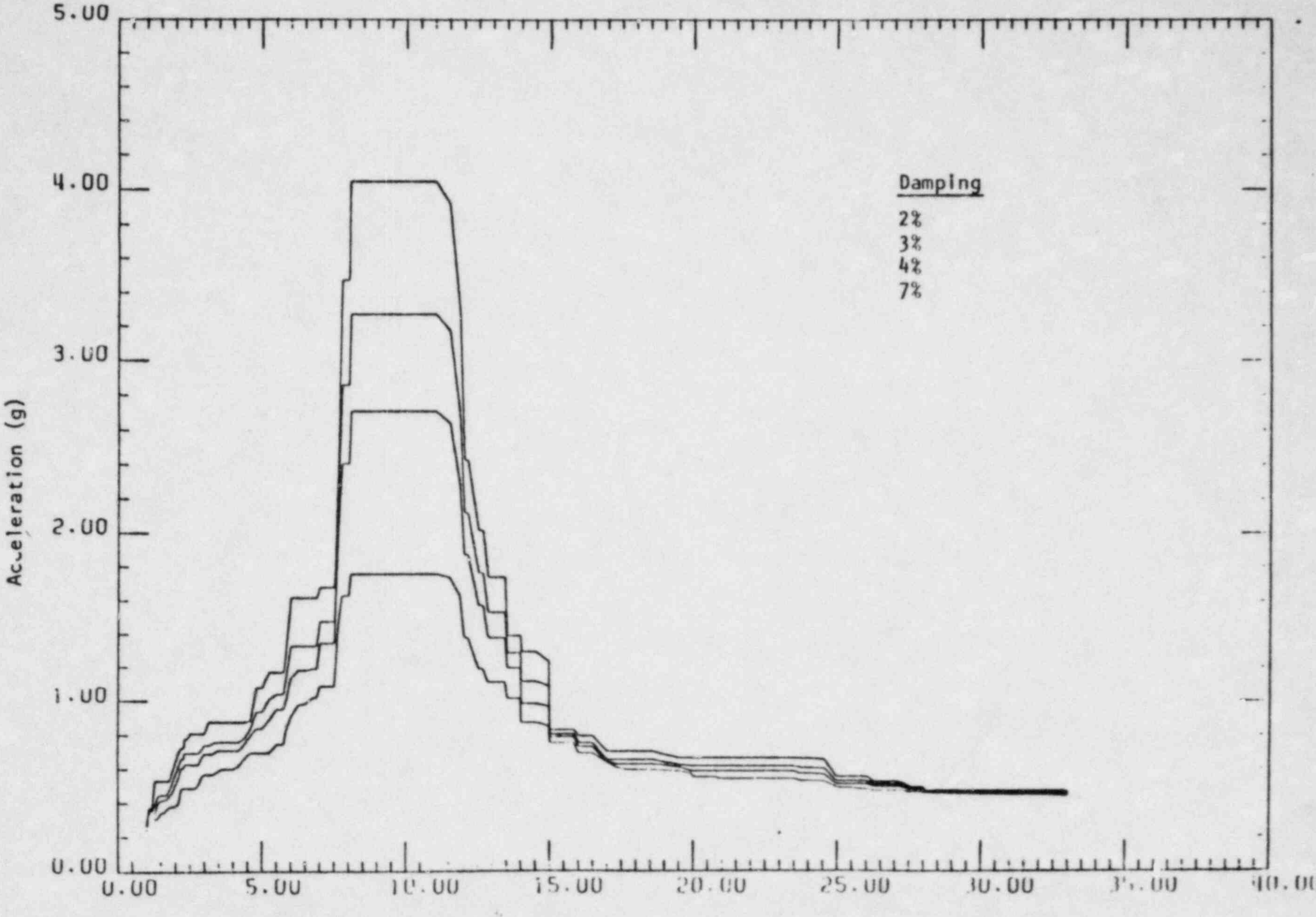

Figure 11: East-West Floor Response Spectra For Elevation 21'-6", Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

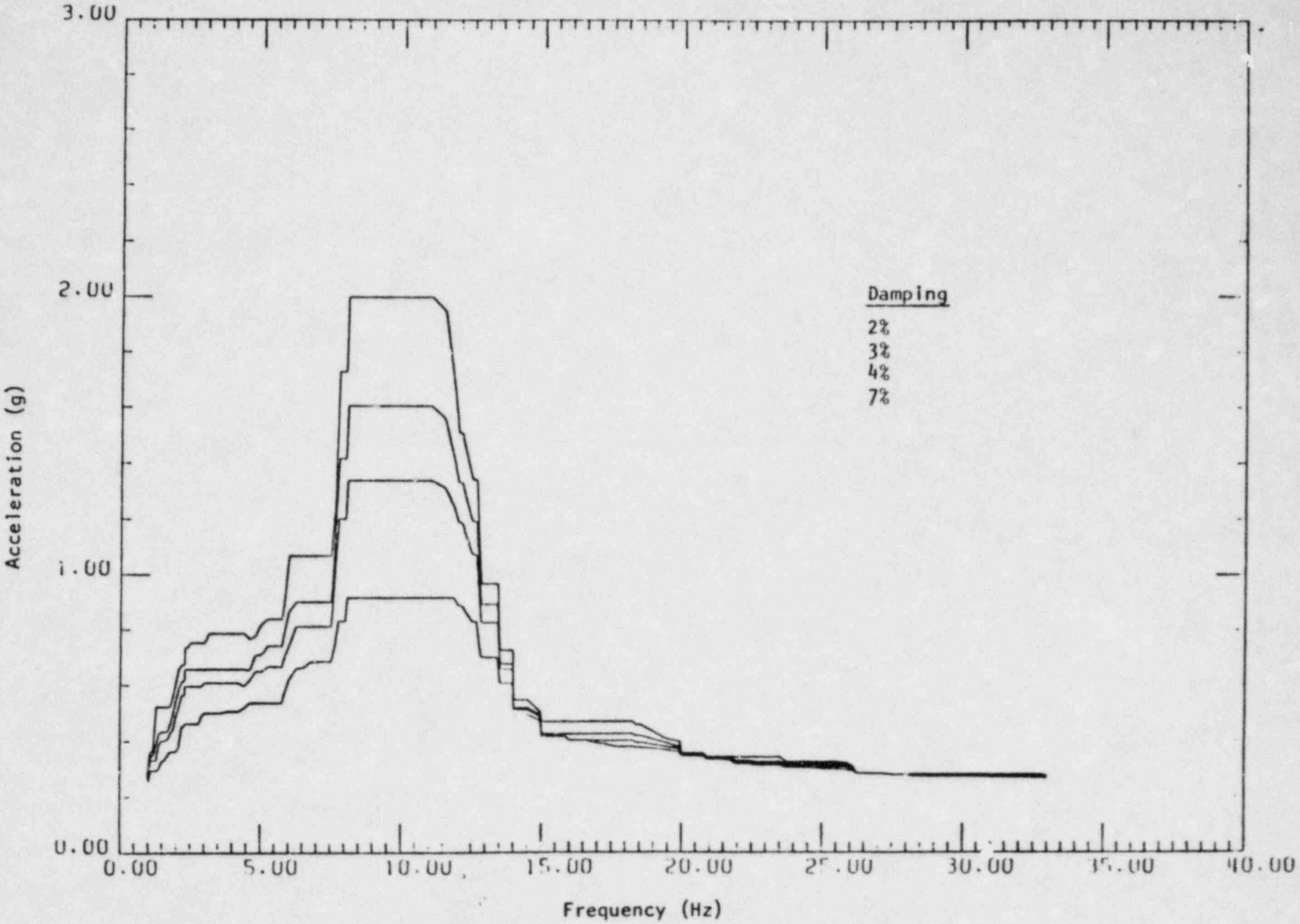

Figure 12: North-South Floor Response Spectra For Elevation 21'-6",
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

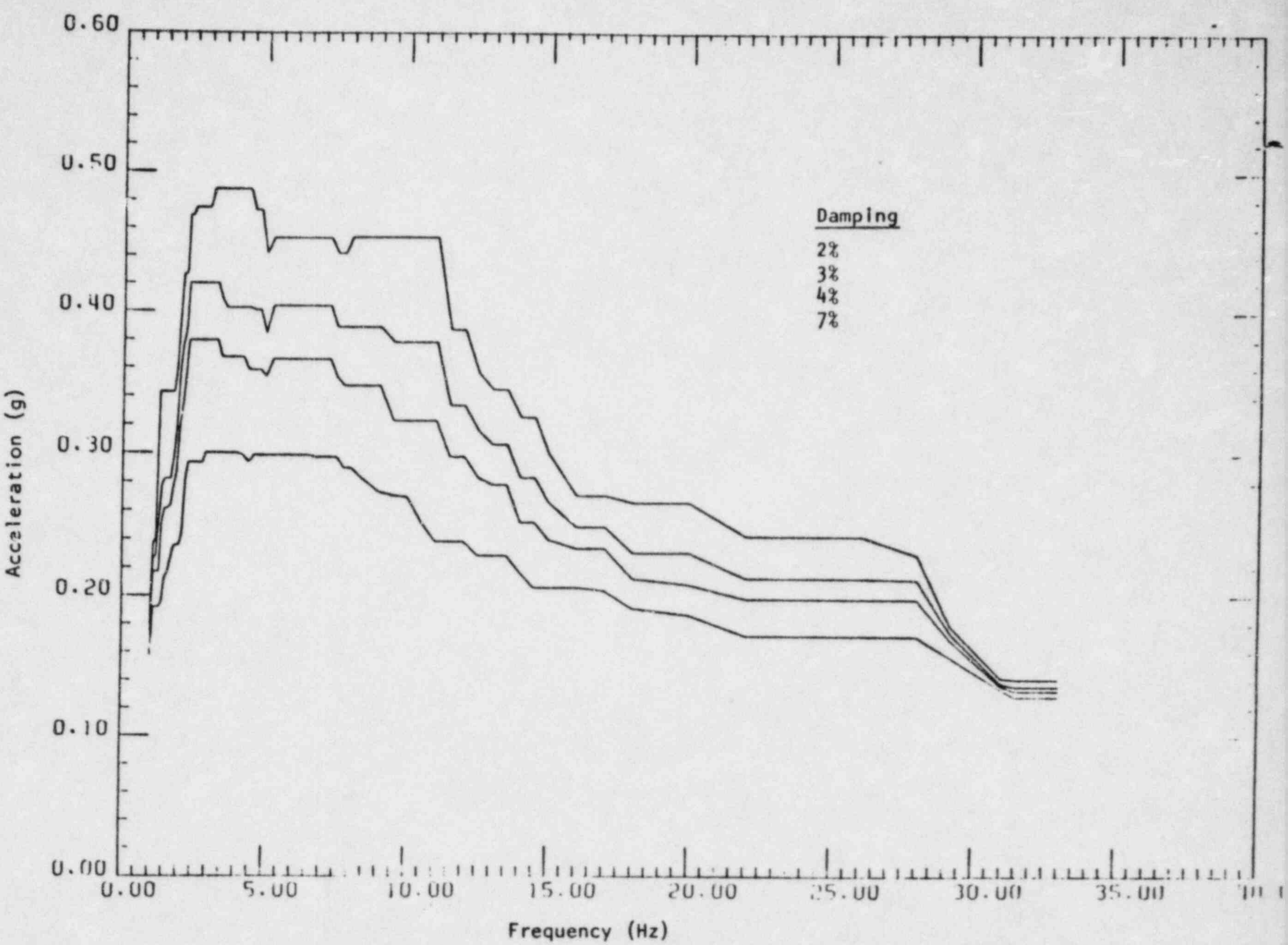

Figure 13: Vertical Floor Response Spectra For Elevation 21'-6",
At Location 1'-6" West of Column Line M and 5'-9" South of Column Line 11¼
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

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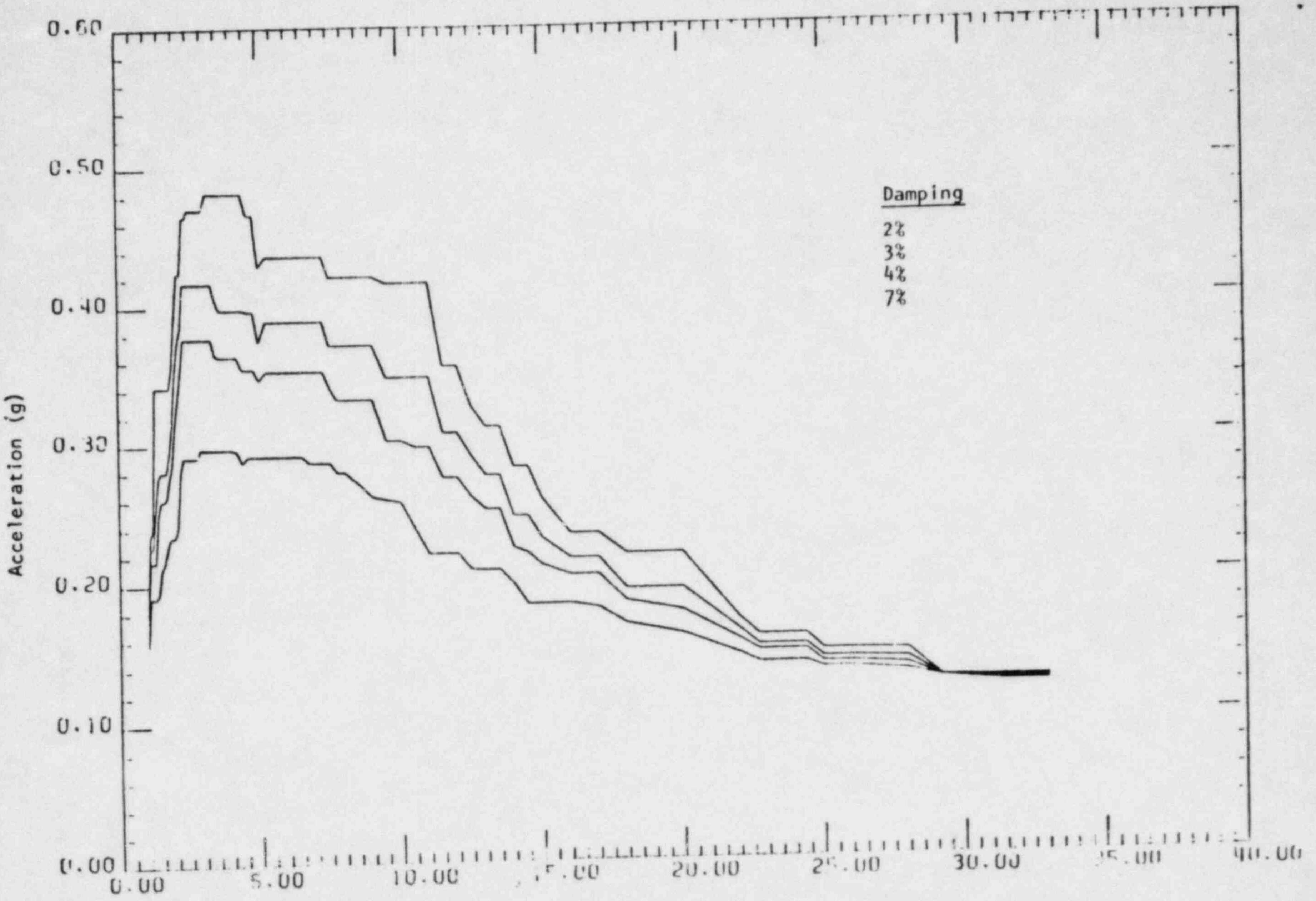


Figure 14: Vertical Floor Response Spectra For Elevation 21'-6" At Location 8'-0" West of Column Line L and 5'9" South of Column Line 11¼ Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

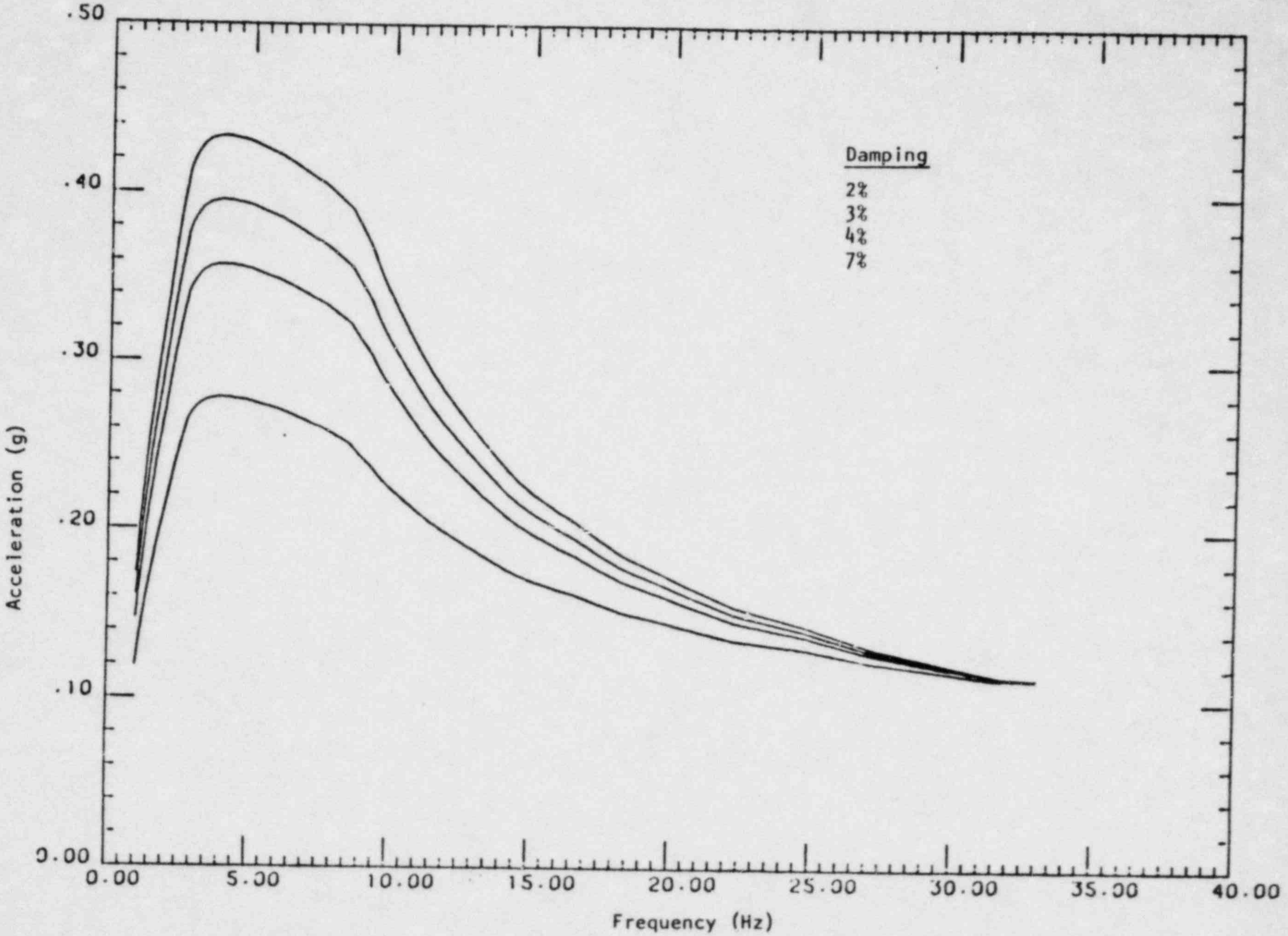


Figure 15: Vertical Floor Response Spectra For Elevation 21'-6"
At Floor Area Enclosed By Column Lines $11^3/4$, $13^1/4$, L and N
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

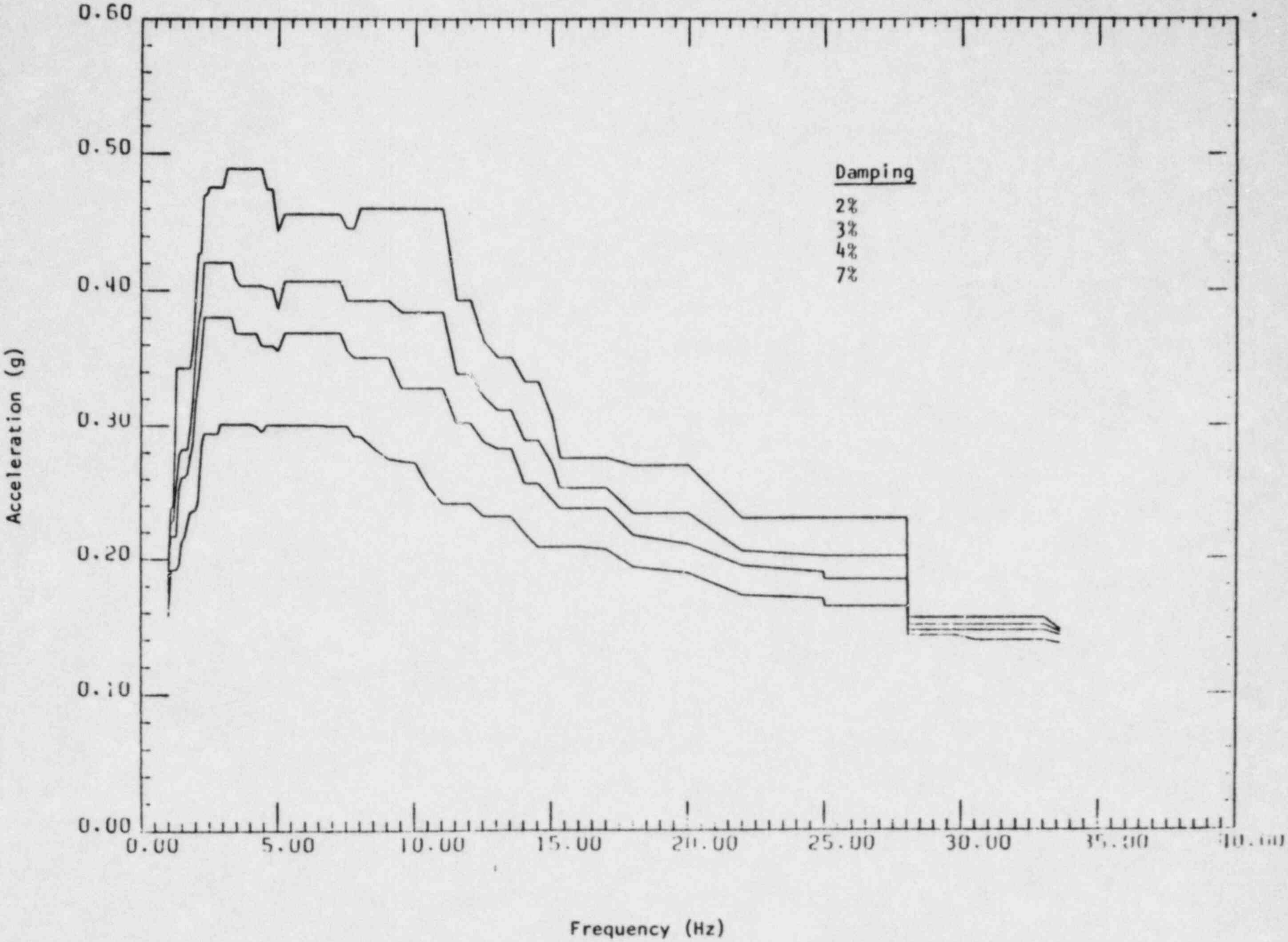


Figure 16: Vertical Floor Response Spectra For Elevation 21'-6"
At Floor Area Enclosed By Column Lines 10¼, 11¼, G and H
Primary Auxili Building, Connecticut Yankee Atomic Power Plant

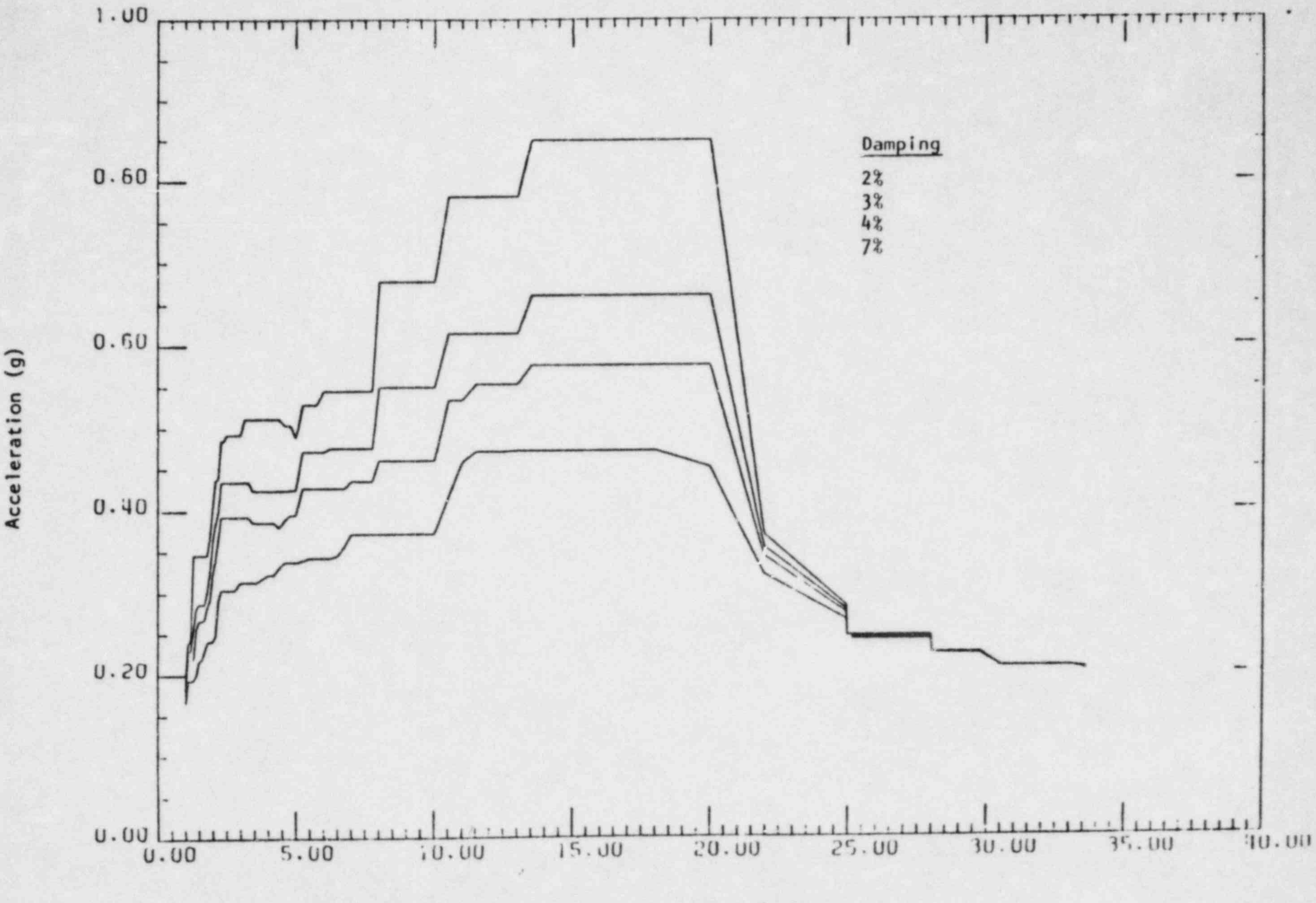


Figure 17: Vertical Floor Response Spectra For Elevation 21'-6" At Floor Area Enclosed By Column Lines $11^3/4$, $13^1/4$, F and G Primary Auxiliar Building, Connecticut Yankee Atomic Power Plant

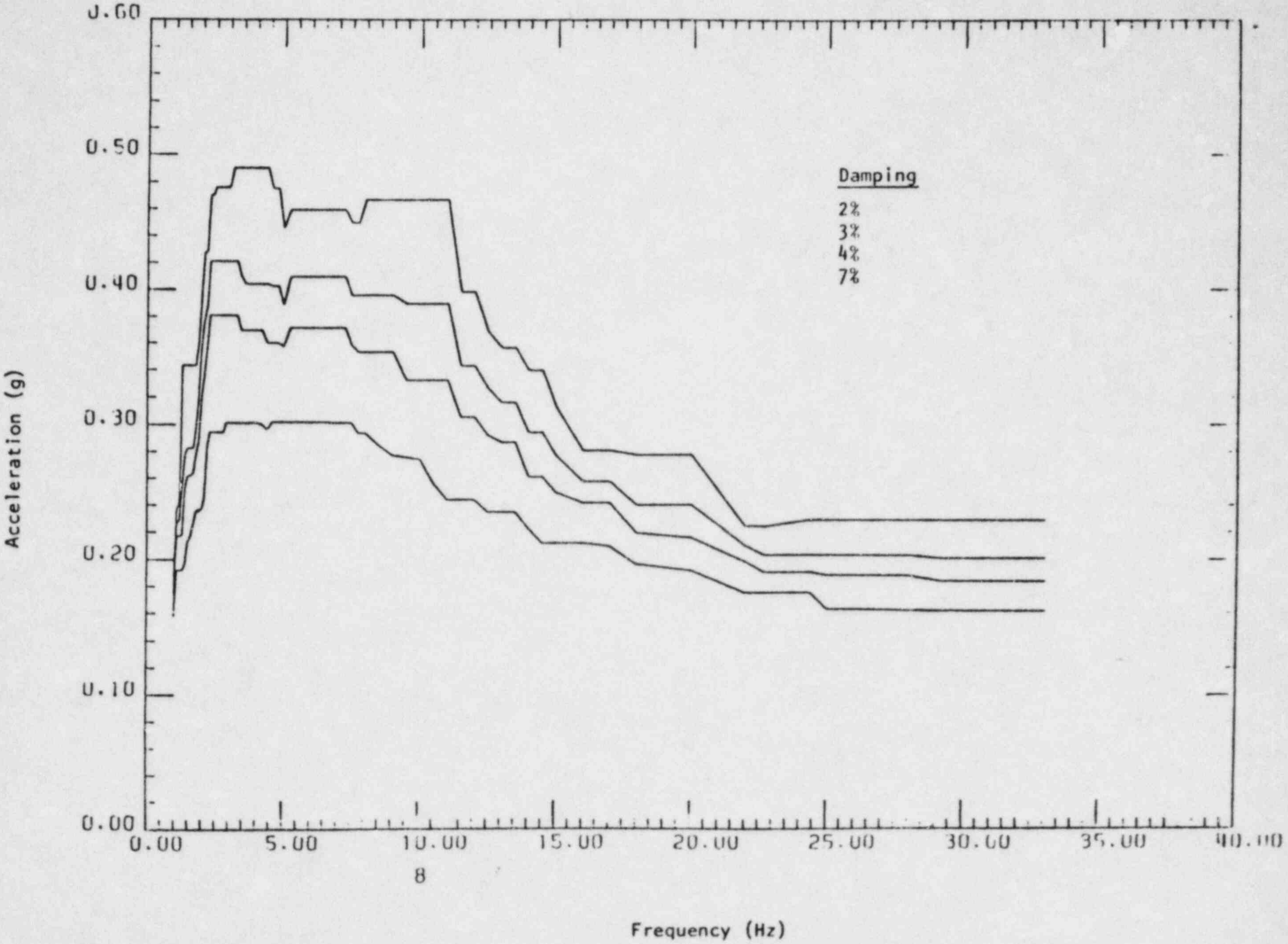


Figure 18: Vertical Floor Response Spectra For Elevation 21'-6"
At Location 8'-0" East of Column Line H and 13'-5" South of Column Line [illegible]
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

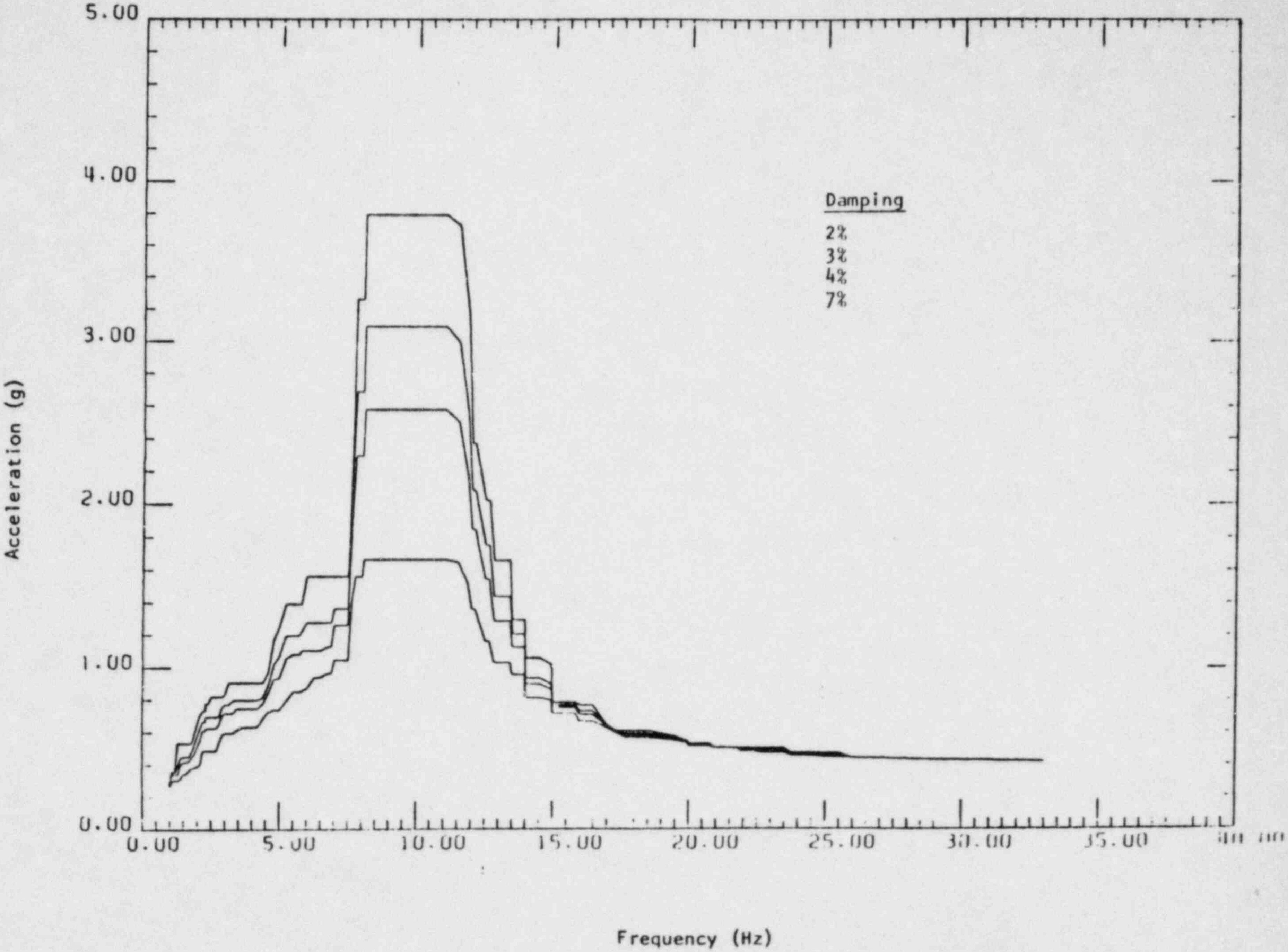


Figure 19: East-West Floor Response Spectra For Elevation 35'-6", Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

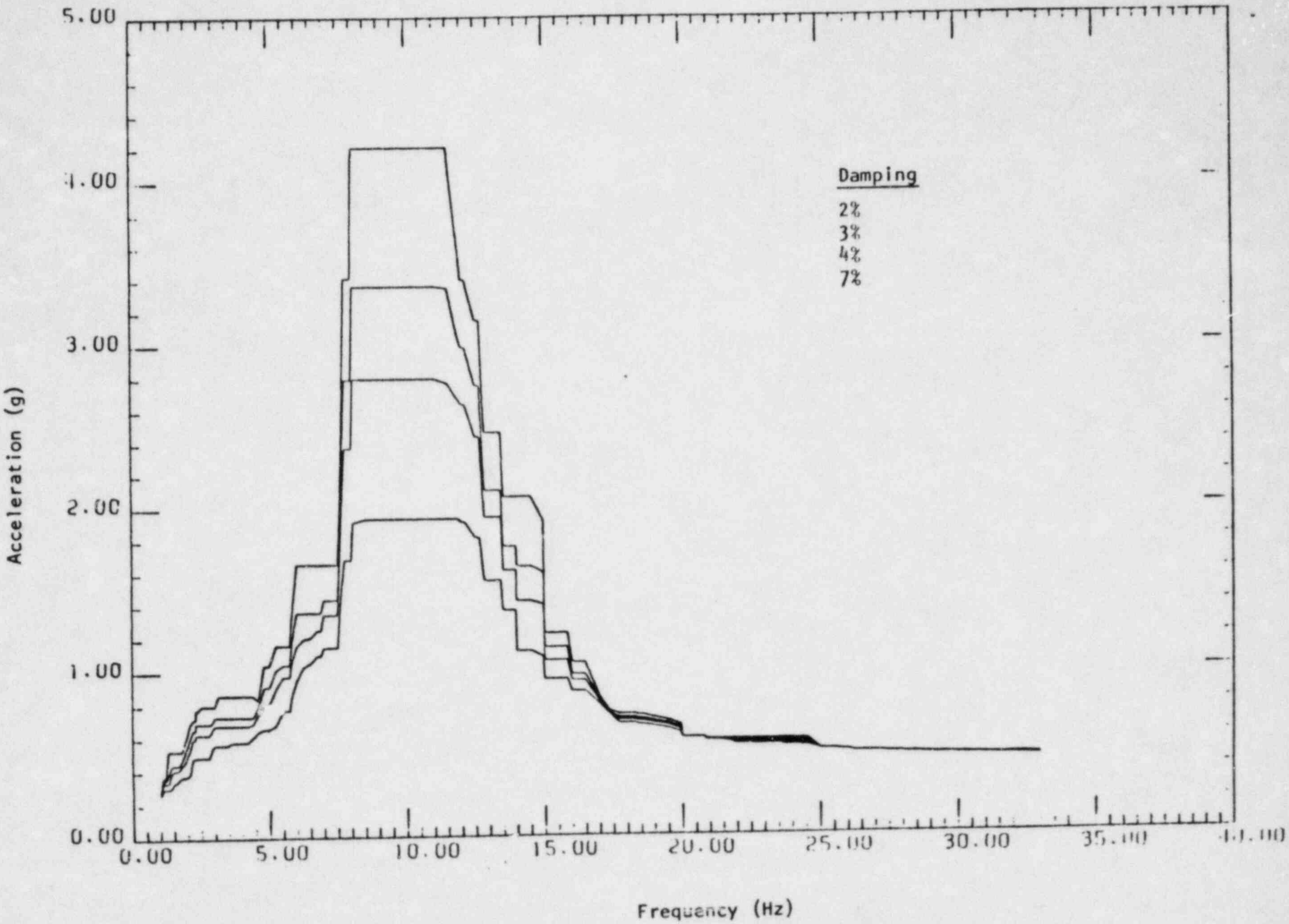


Figure 20: North-South Floor Response Spectra For Elevation 35'-6", Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

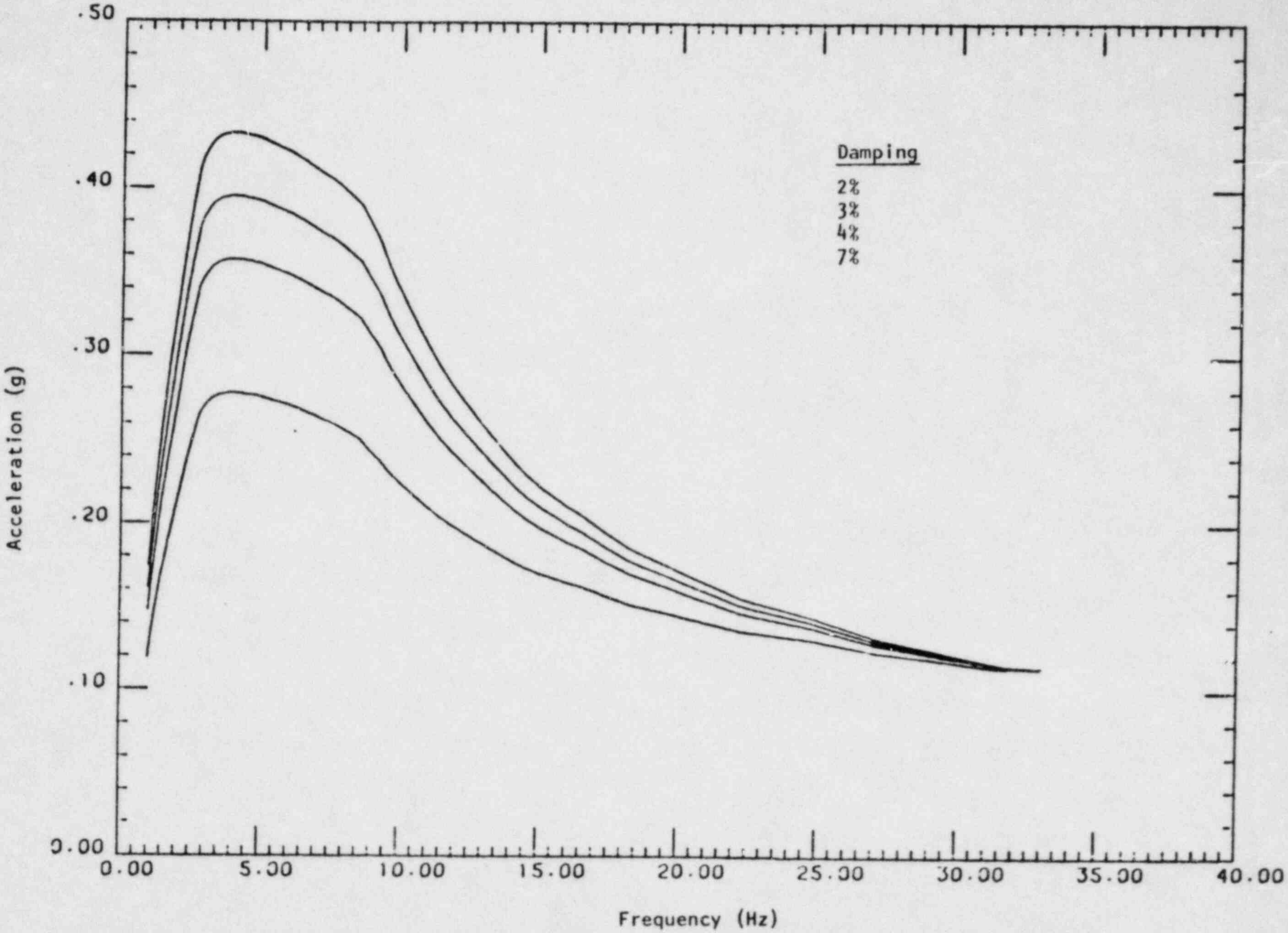


Figure 21: Vertical Floor Response Spectra For Elevation 35'-6"
At Location 9'-3" West of Column Line K and 6'-6" South of Column Line 13¼
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

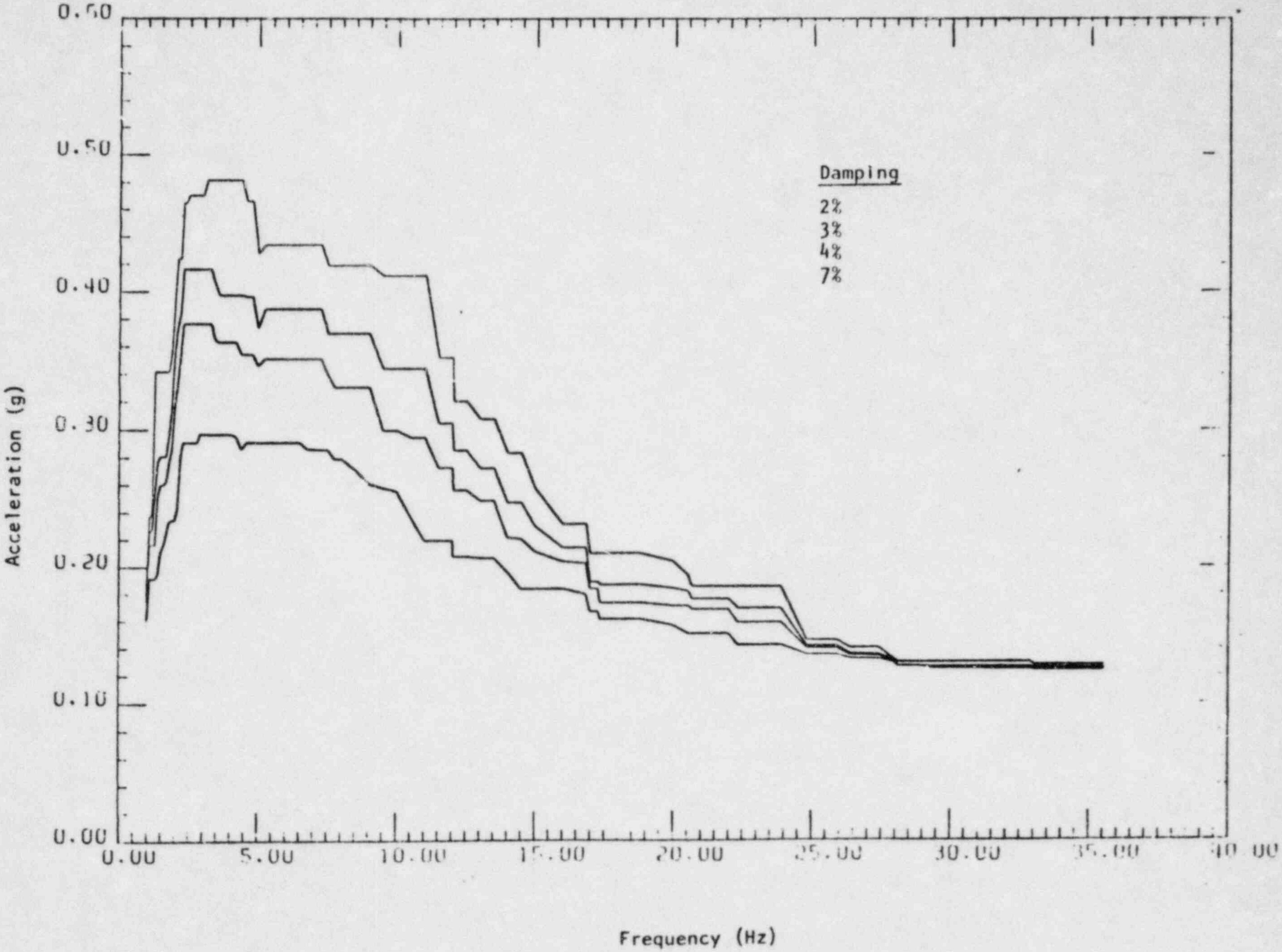


Figure 22: Vertical Floor Response Spectra For Elevation 35'-6"
At Location 9'-3" West of Column Line K and Column Line 11½
Primary Auxili Building, Connecticut Yankee Atomic Power Plant

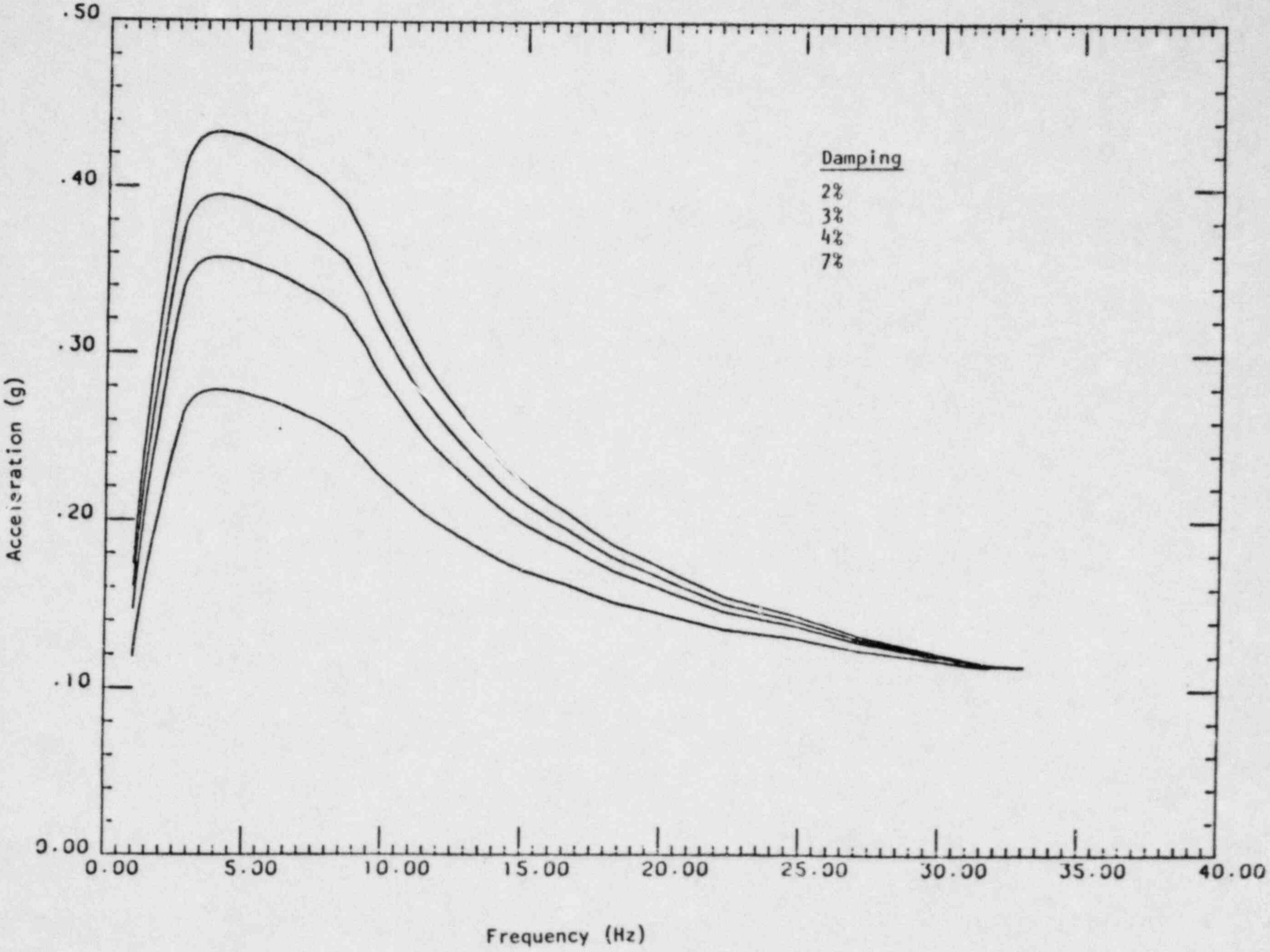
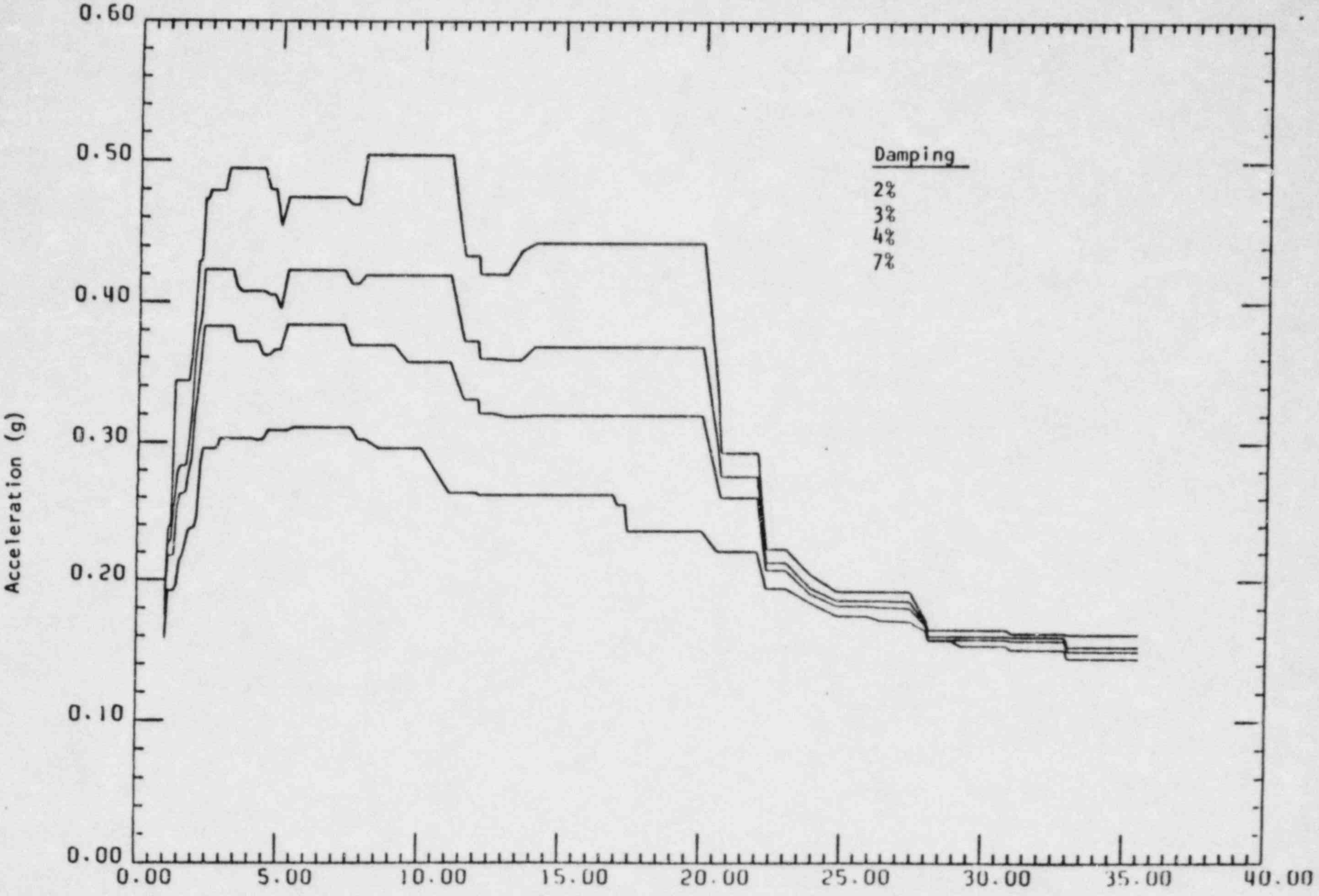


Figure 23: Vertical Floor Response Spectra For Elevation 35'-6"
At Location Column Li 1-1¼ and Column Line J
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant



Frequency (Hz)

Figure 24: Vertical Floor Response Spectra For Elevation 35'-6"
At Location 12'-6" West of Column Line J and 6'-6" South of Column Line 121
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

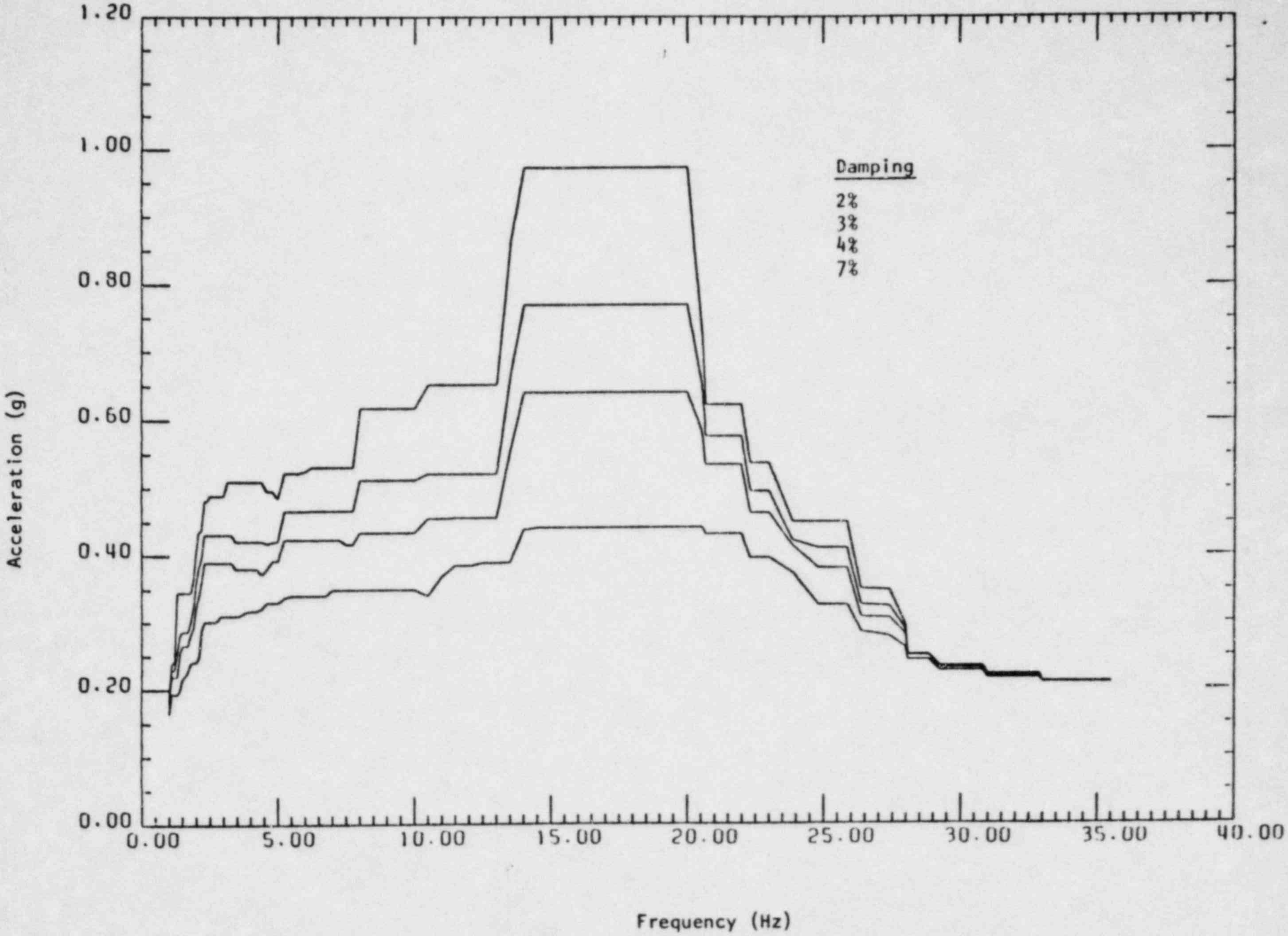


Figure 25: Vertical Floor Response Spectra For Elevation 35'-6"
At Location 12'-6" West of Column Line J and 16'-9" South of Column Line 13¼
Primary Auxiliary Bui 'ng, Connecticut Yankee Atomic Power Plant

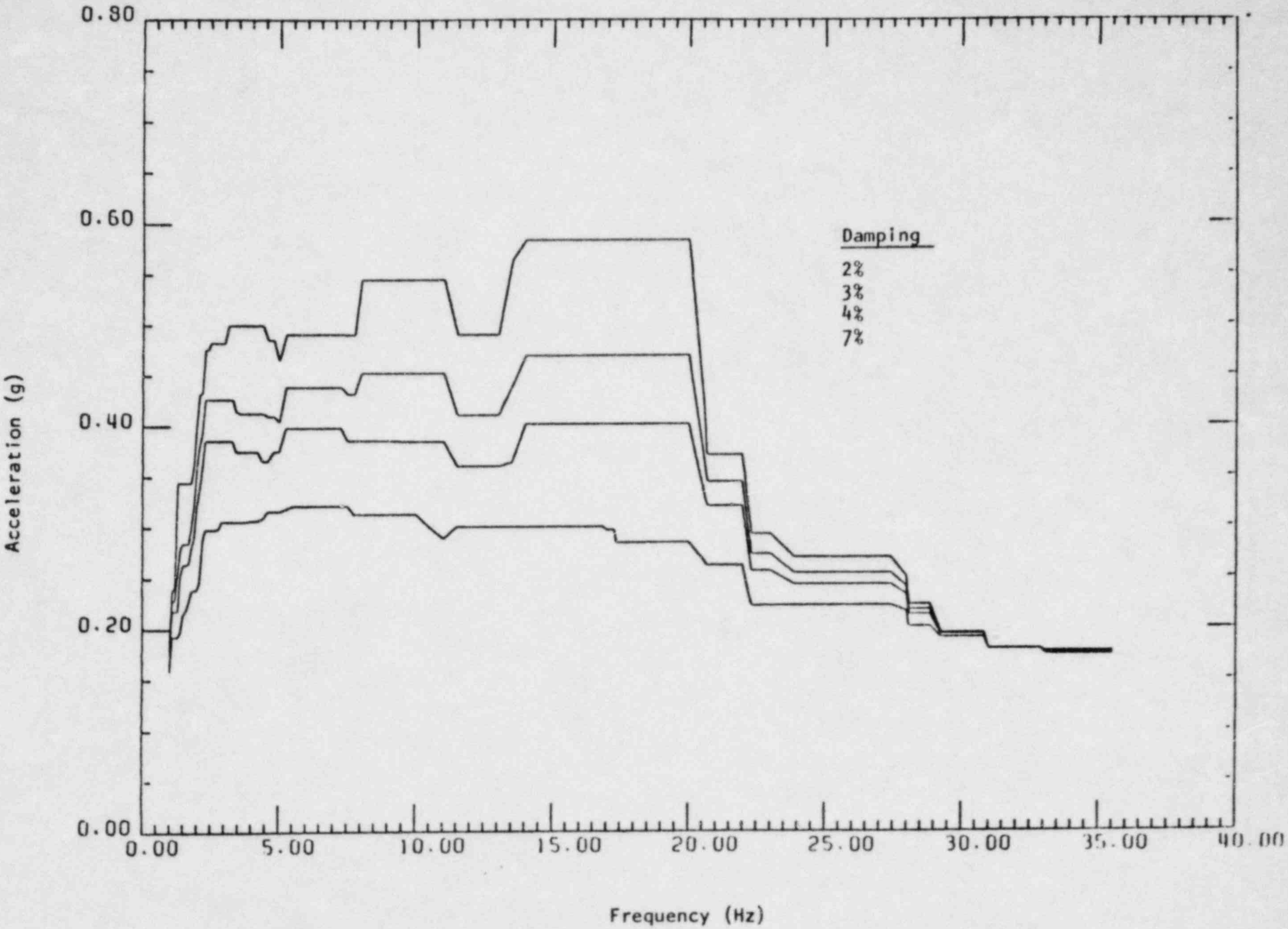


Figure 26: Vertical Floor Response Spectra For Elevation 35'-6"
At Location 12'-6" West of Column Line J and Column Line $11^3/_4$
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

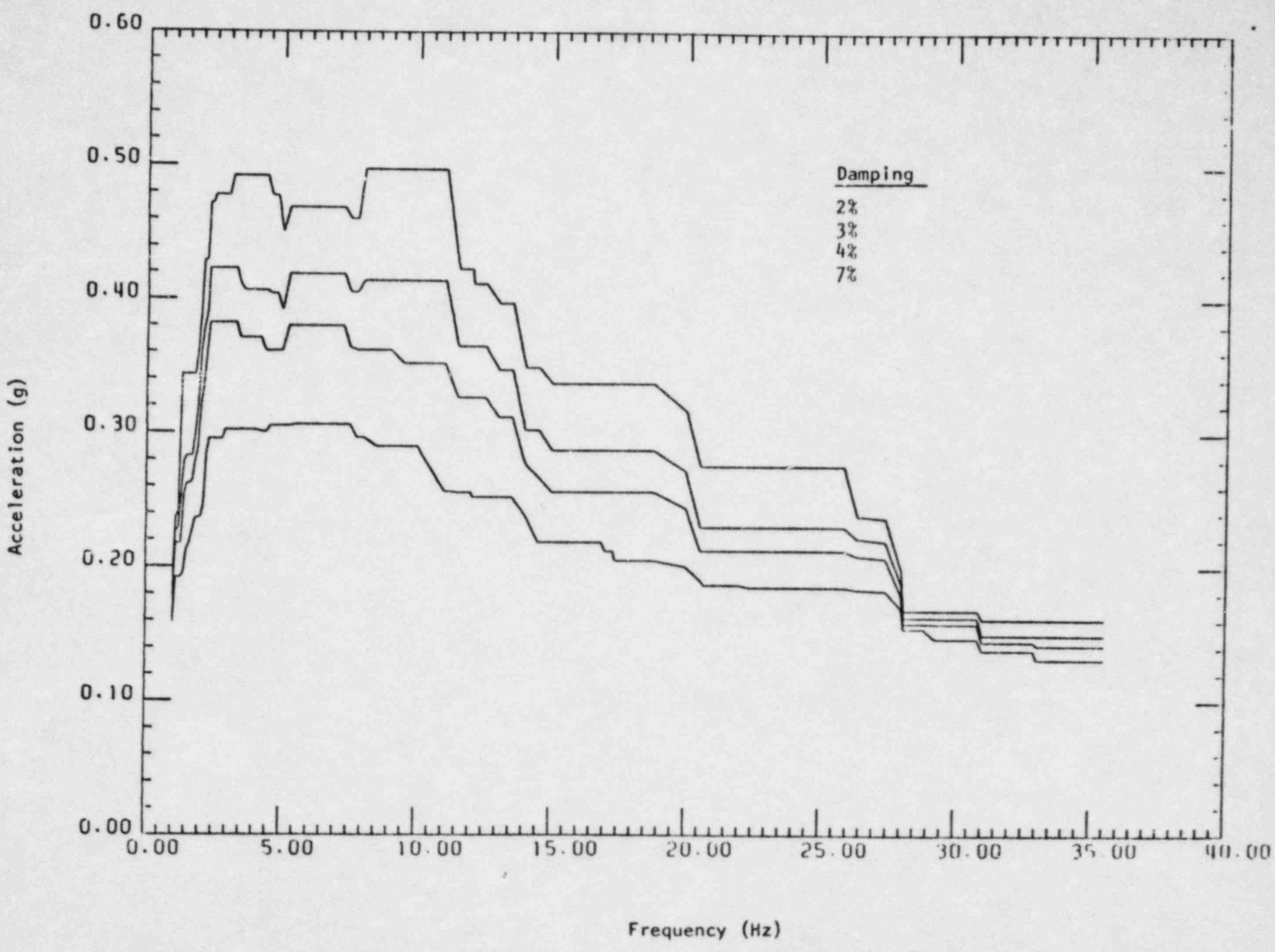


Figure 27: Vertical Floor Response Spectra For Elevation 35'-6"
At Location 12'-6" West of Column Line J and Column Line 11¼
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

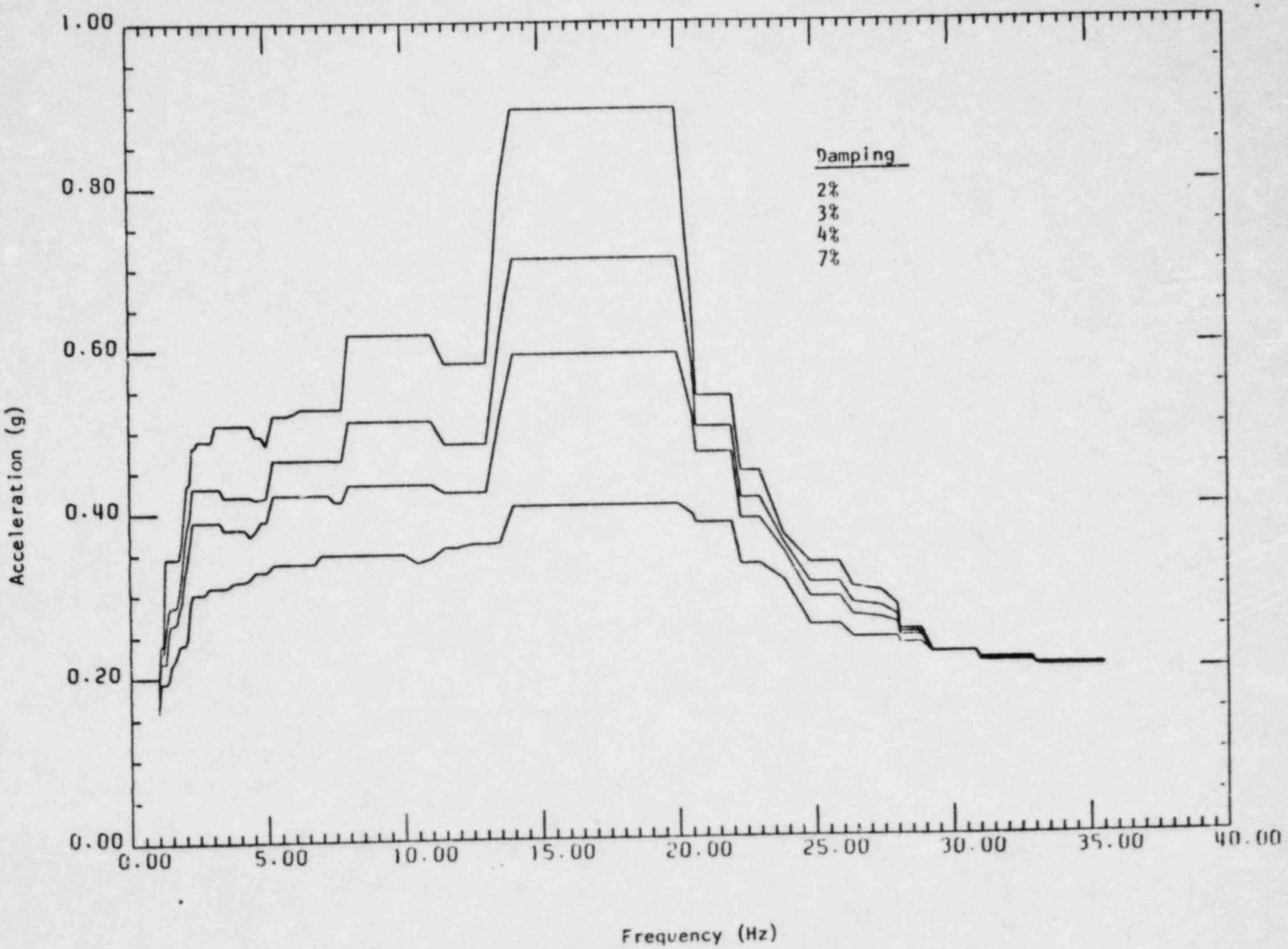


Figure 28: Vertical Floor Response Spectra For Elevation 35'-6"
At Location 12'-6" W · of Column Line J and 11'-6" South of Column Line 11½
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

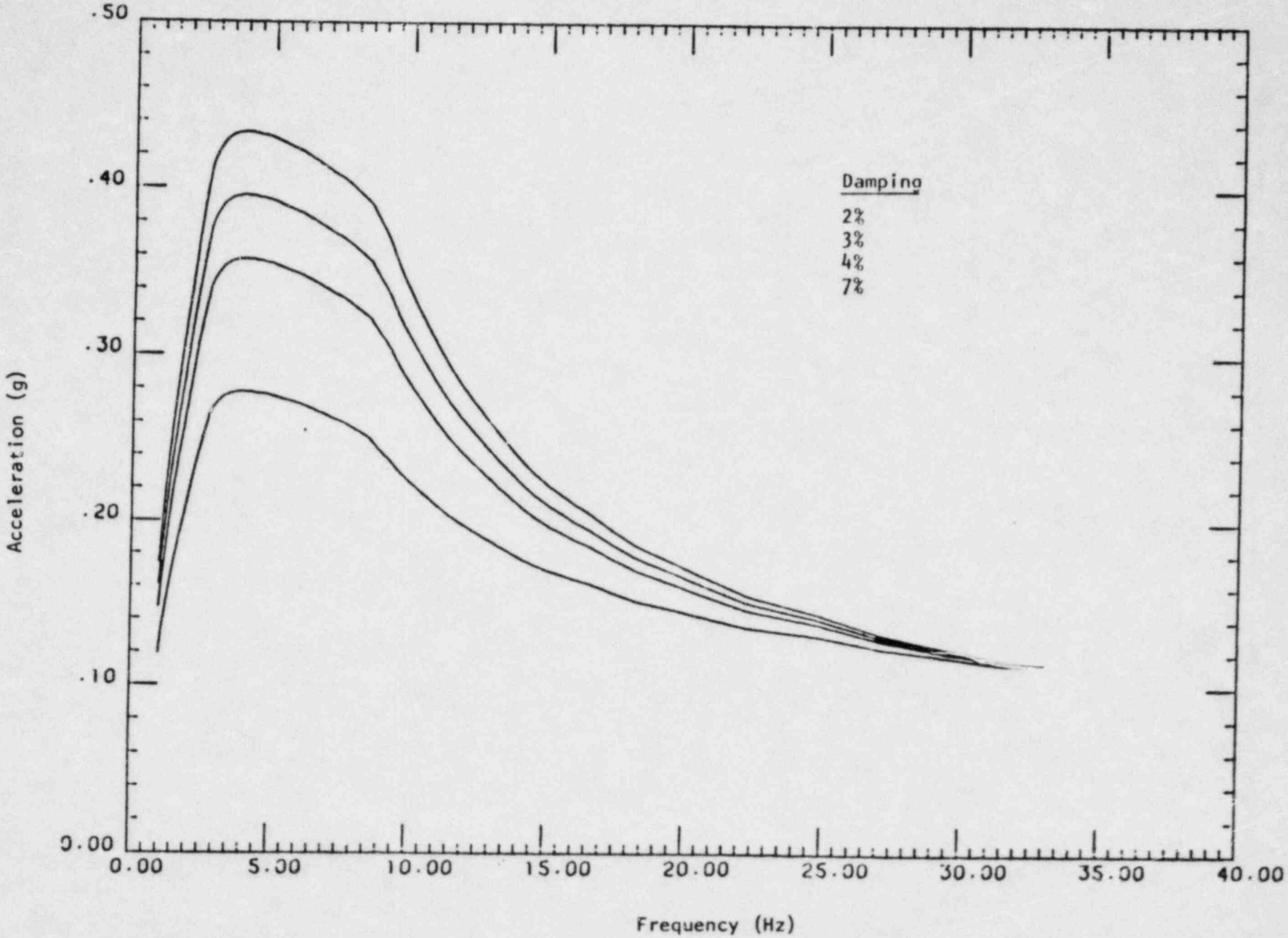


Figure 29: Vertical Floor Response Spectra For Elevation 35'-6"
At Location Column Line H and 6'-6" South of Column Line 13¼
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

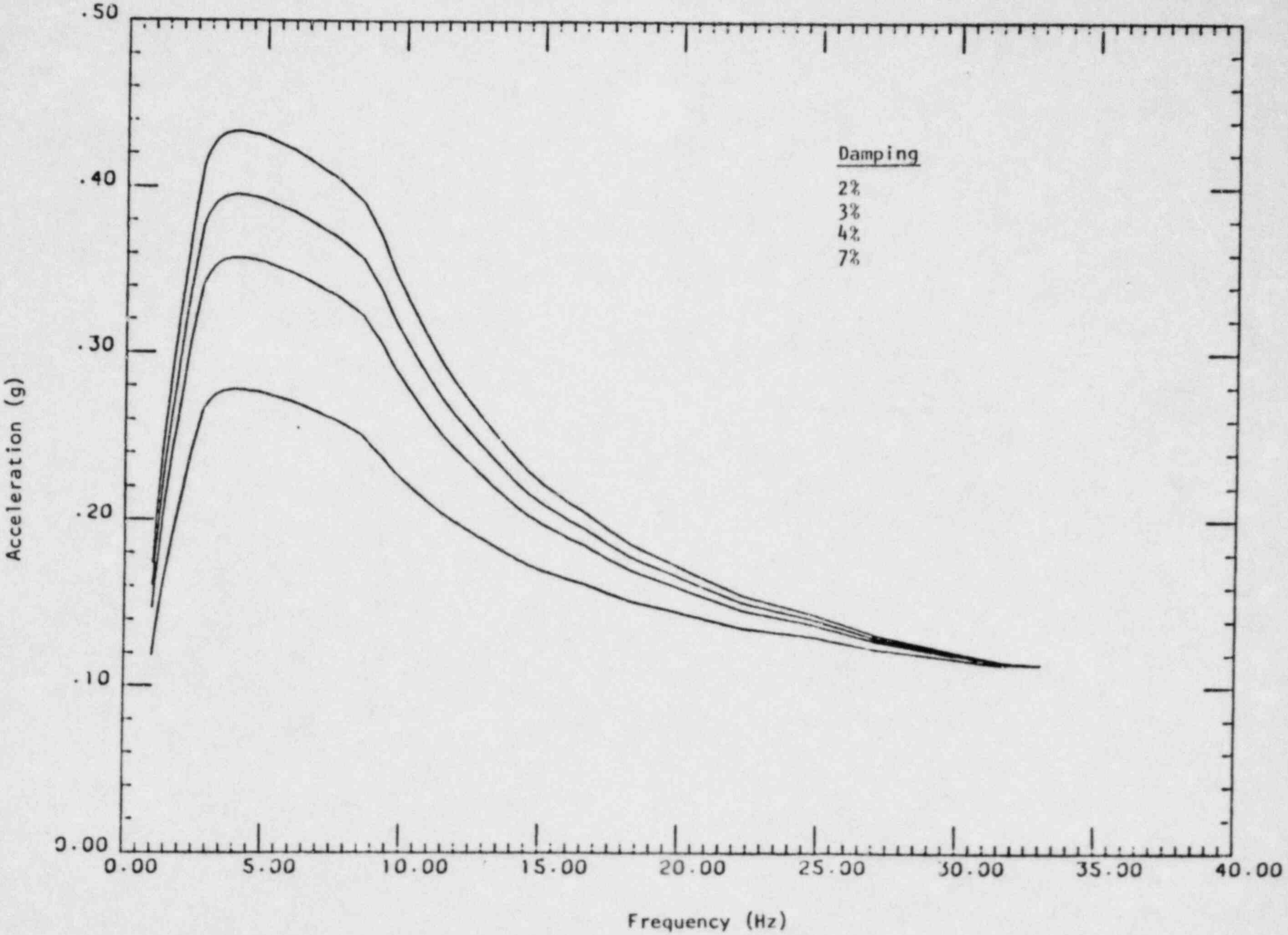


Figure 30: Vertical Floor Response Spectra For Elevation 35'-6"
At Location Column Line H and 16'-9" South of Column Line 13½
Primary Auxiliary Bu ing, Connecticut Yankee Atomic Power Plant

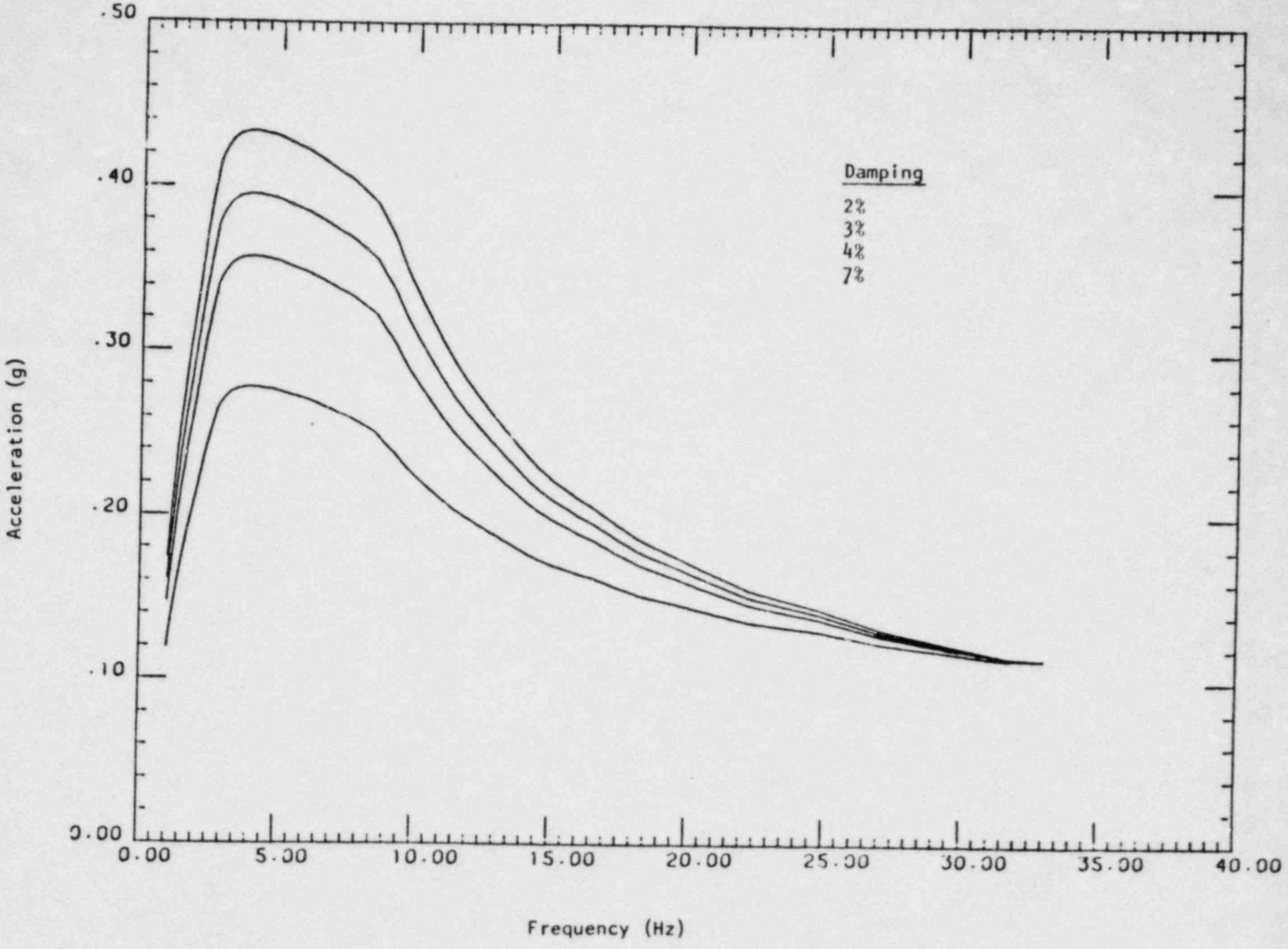


Figure 31: Vertical Floor Response Spectra For Elevation 35'-6"
At Location Column Line H and Column Line 11¼
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

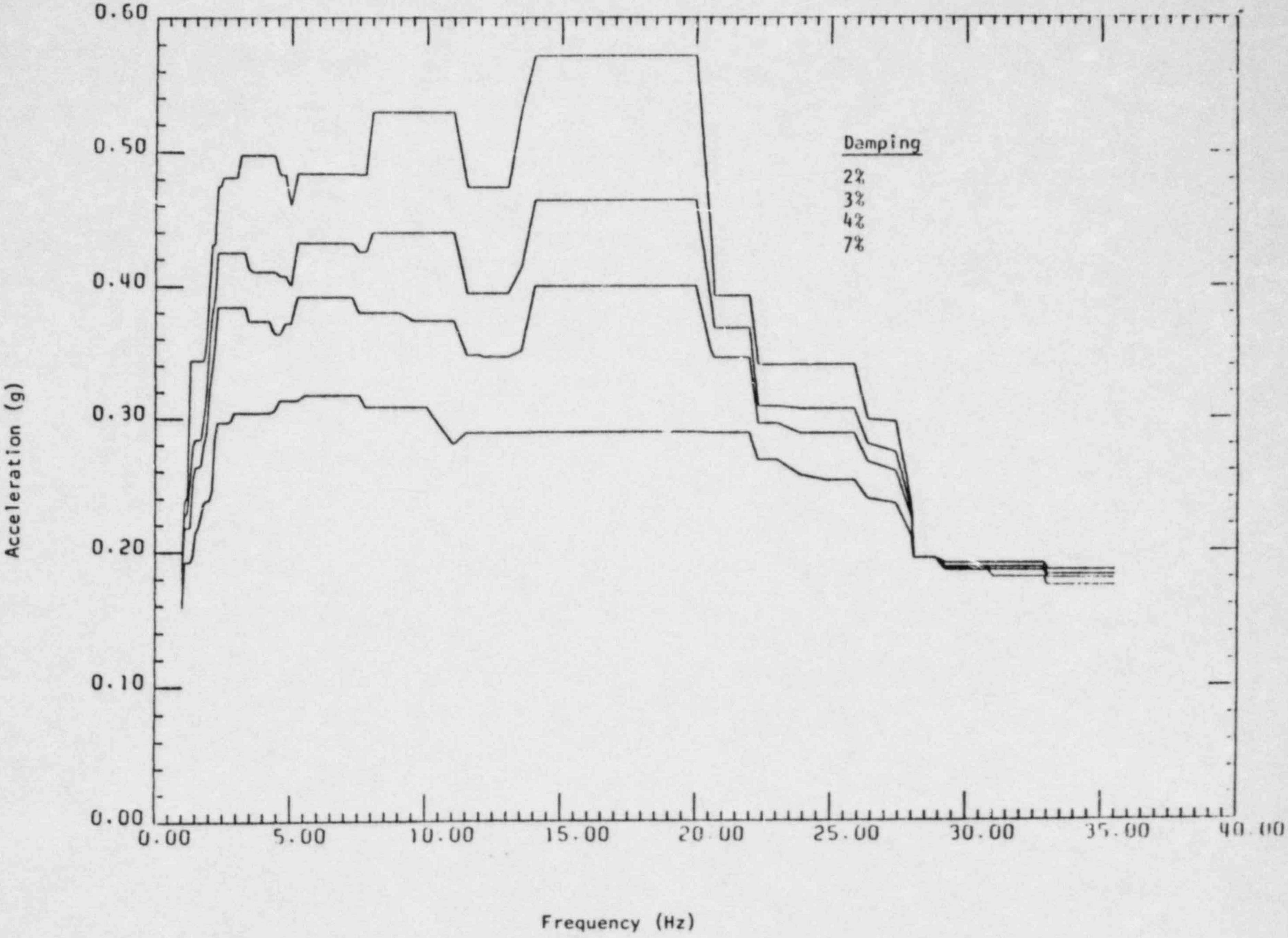


Figure 32: Vertical Floor Response Spectra For Elevation 35'6"
At Location Column Line H and 11'-6" South of Column Line 11½
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

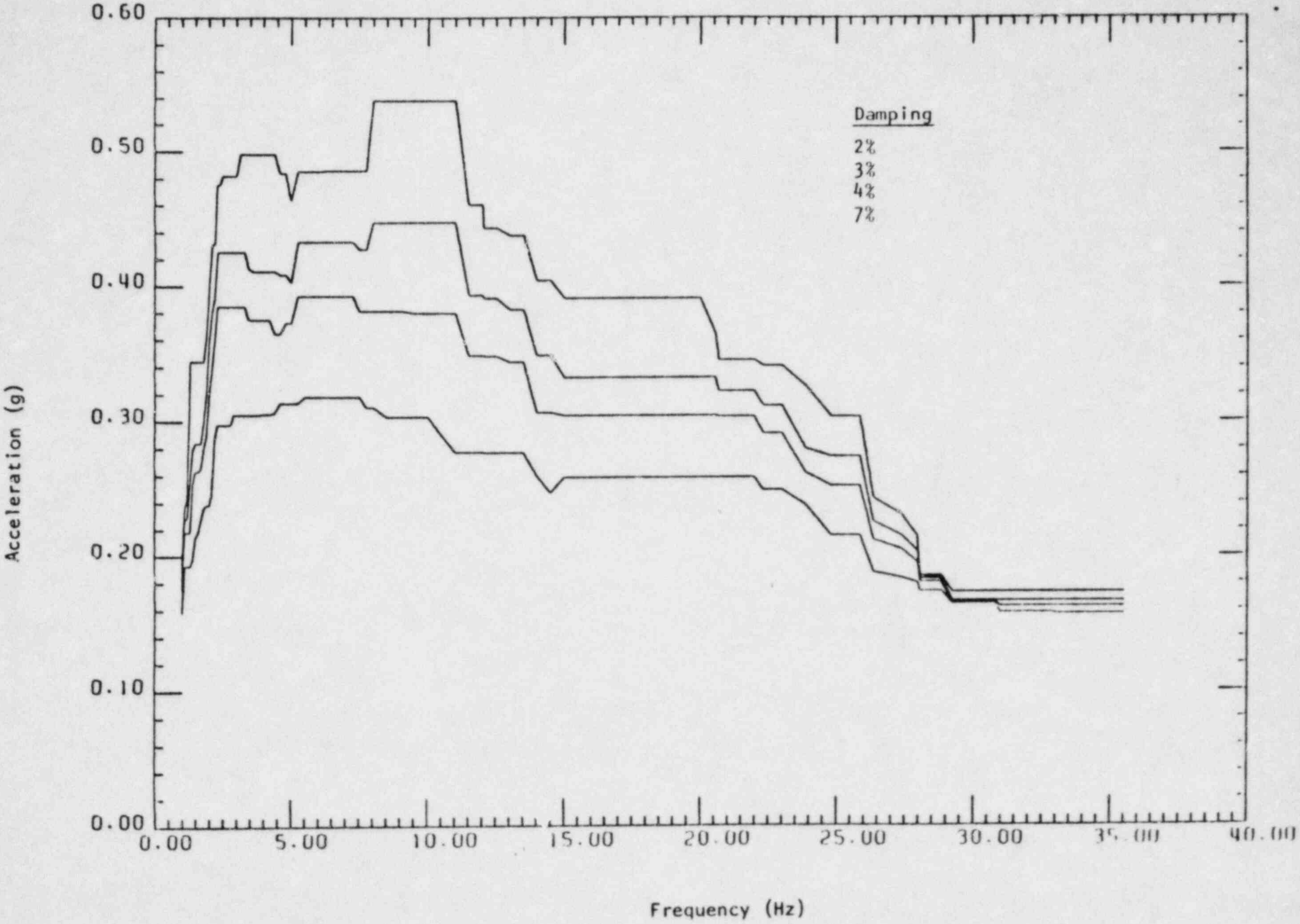


Figure 33: Vertical Floor Response Spectra For Elevation 35'-6"
At Location 11'-2" West of Column Line H and 16'-9" South of Column Line 13½
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

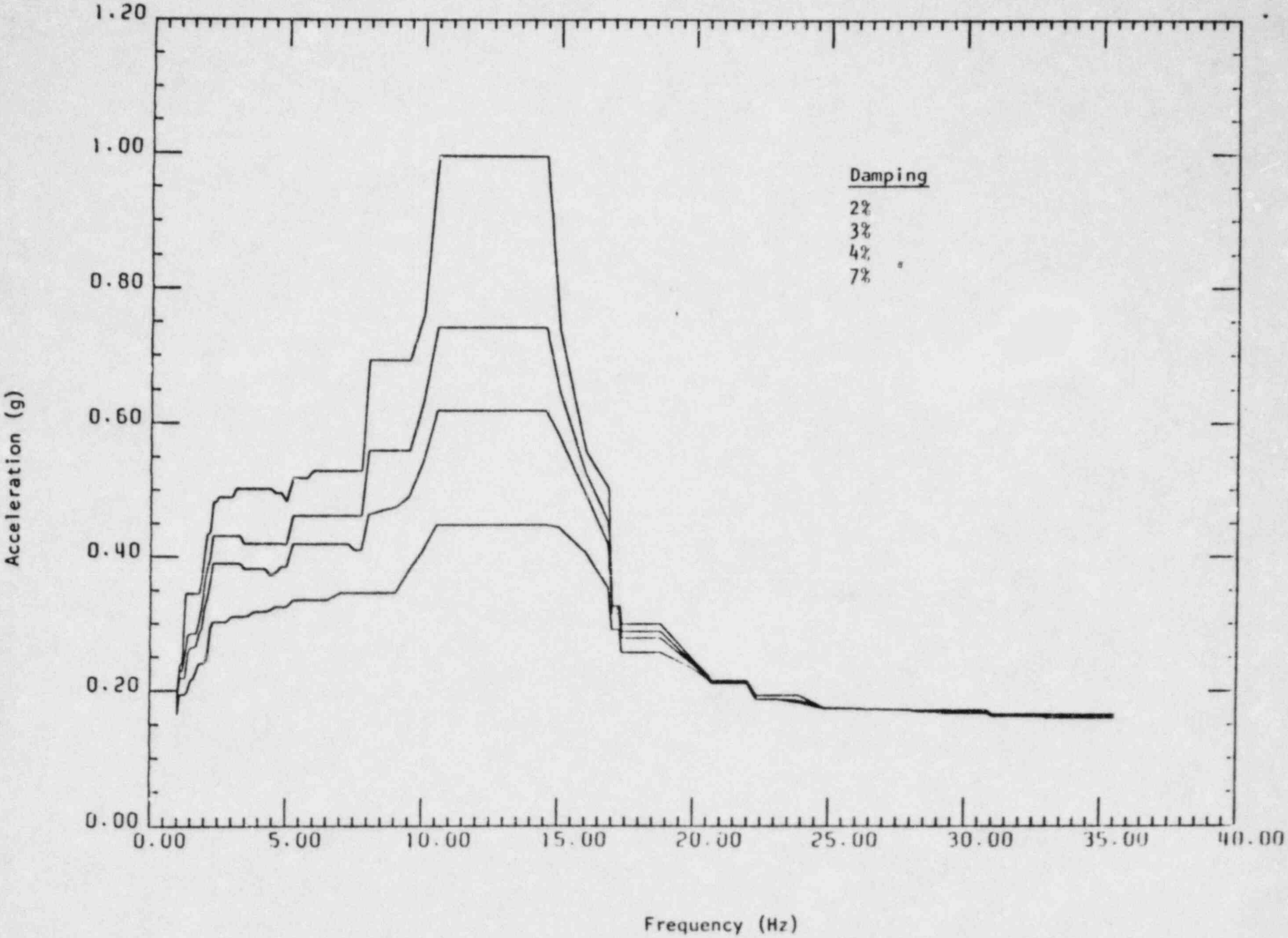


Figure 34: Vertical Floor Response Spectra For Elevation 35'-6"
At Location 8'-0" West of Column Line G and 8'-0" South of Column Line 1
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

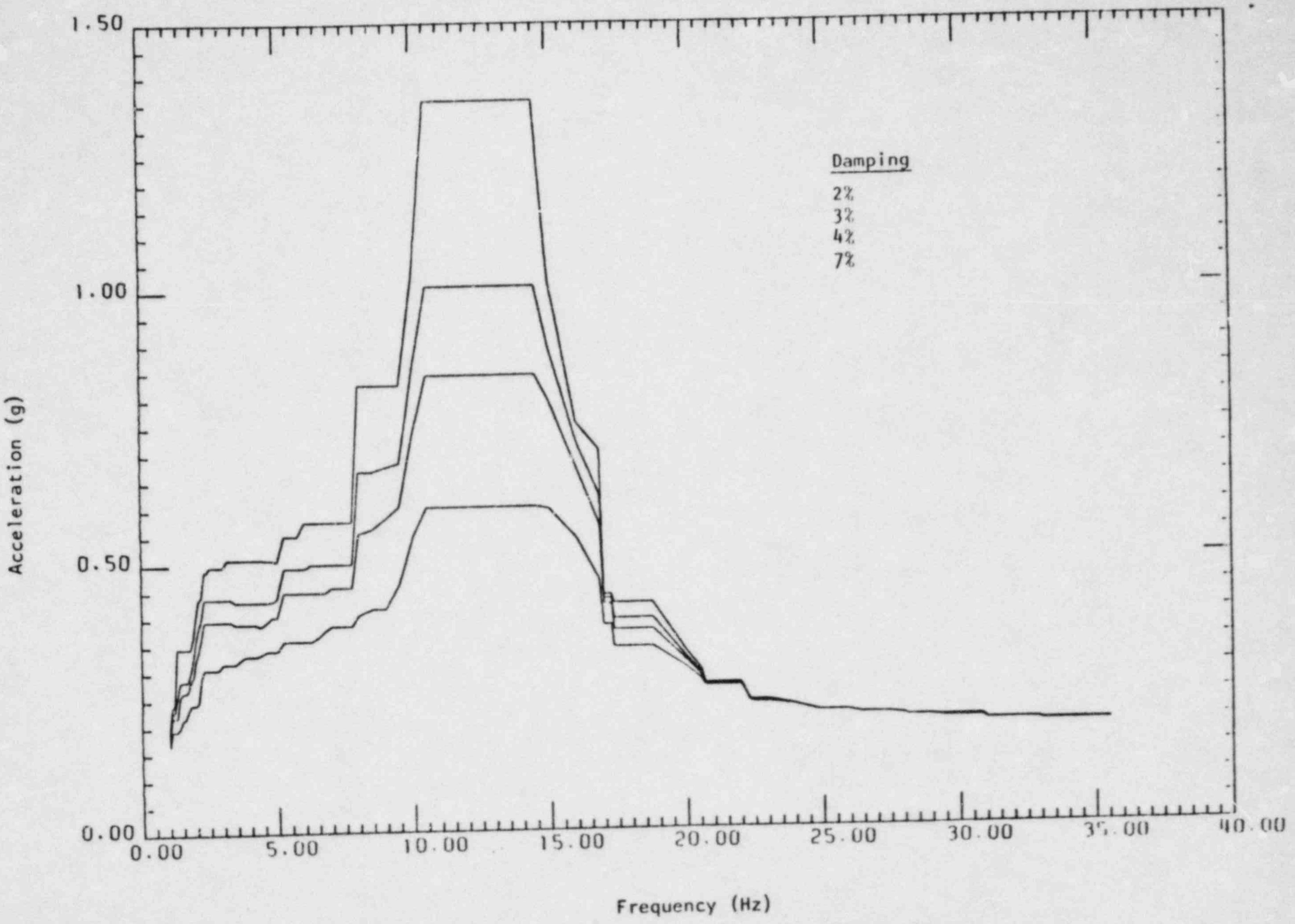


Figure 35: Vertical Floor Response Spectra For Elevation 35'-6"
At Location 8'-0" West of Column Line G and 15'-10" South of Column Line 13¼
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

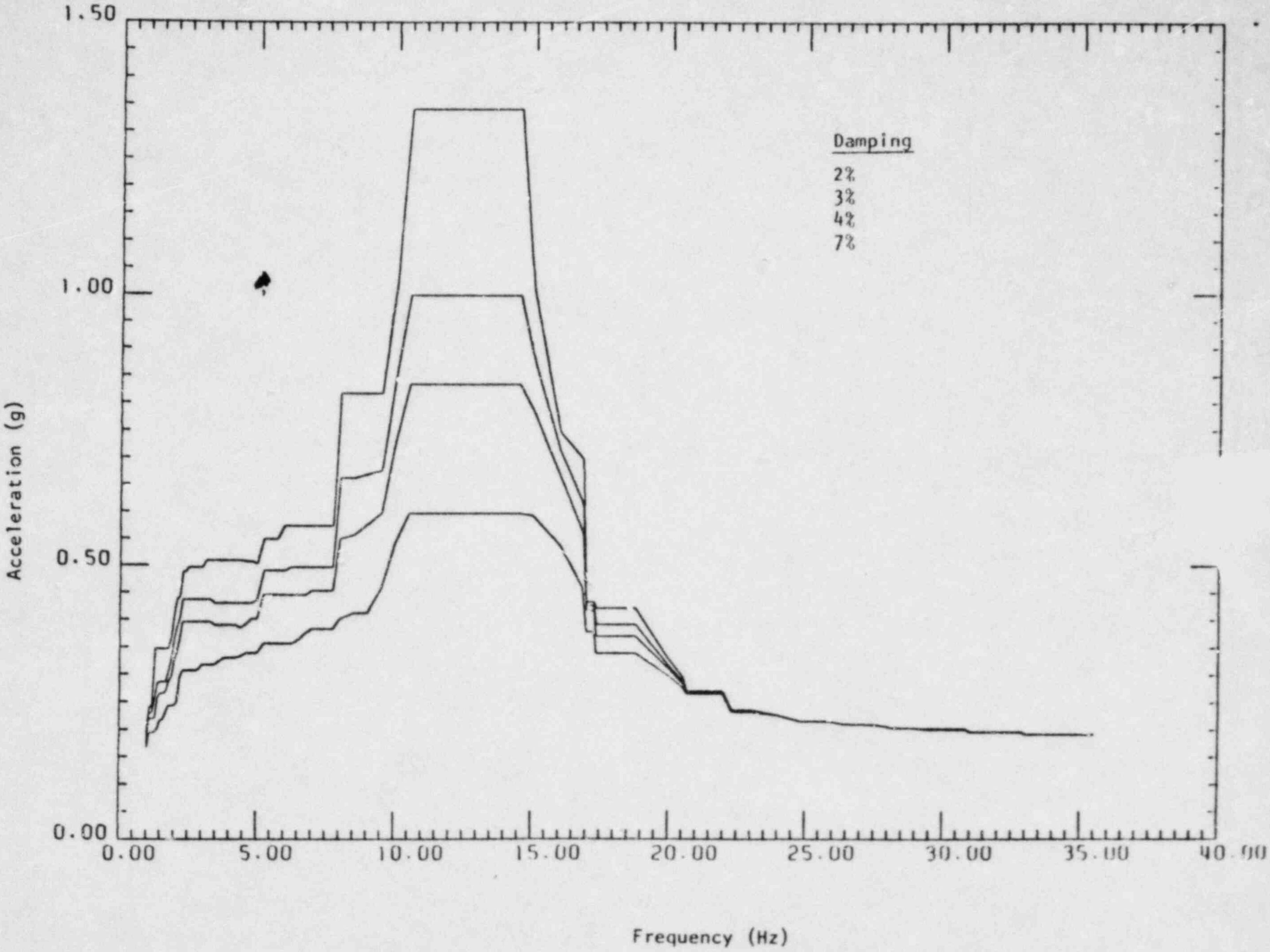


Figure 36: Vertical Floor Response Spectra For Elevation 35'-6"
At Location 15'6" West of Column Line G and 8'0" South of Column Line 1?'
Primary Auxiliary Bui ing, Connecticut Yankee Atomic Power Plant

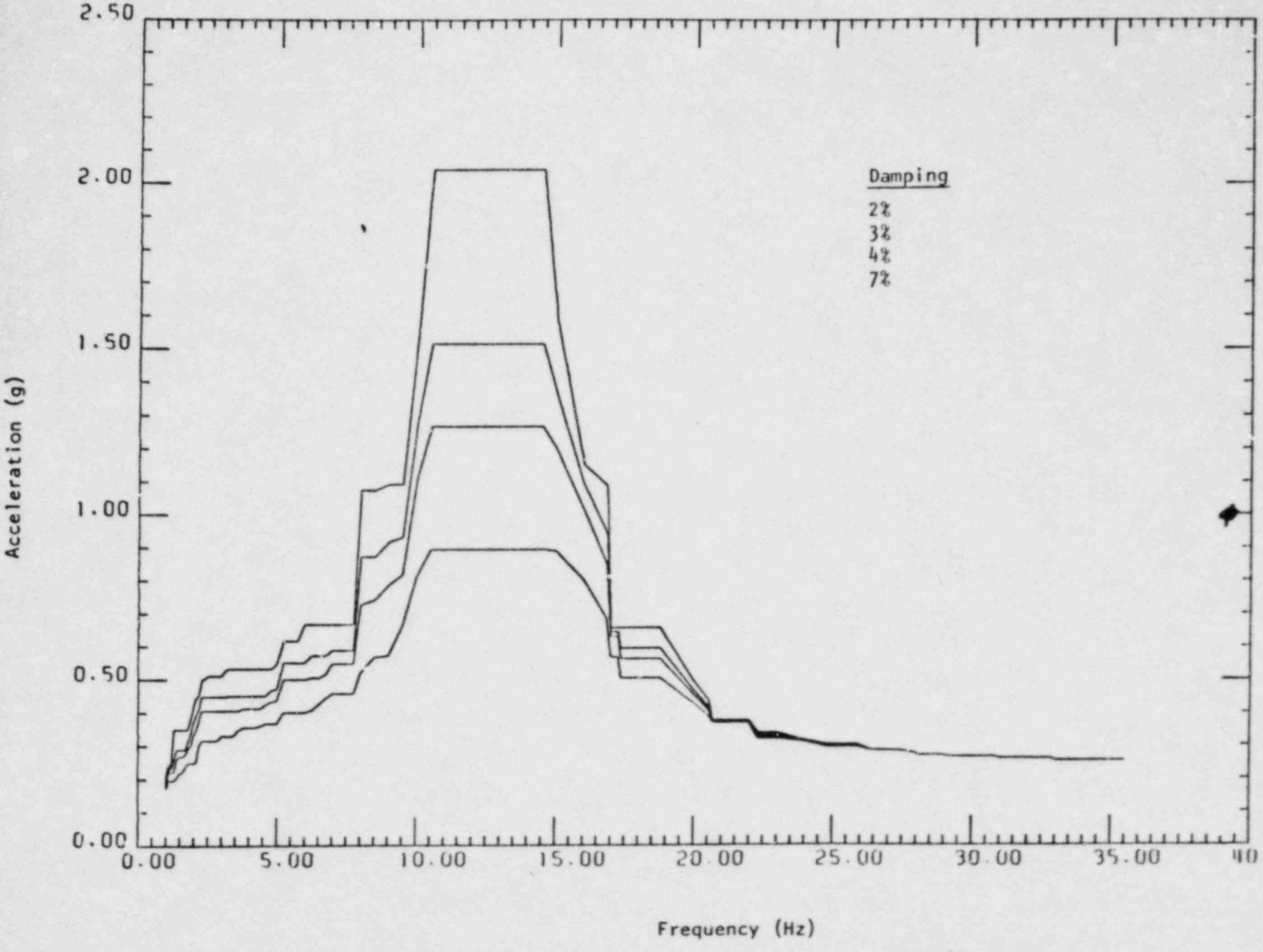


Figure 37: Vertical Floor Response Spectra For Elevation 35'-6"
At Location 15'-6" West of Column Line J and 15'-10" South of Column Line 13½
Primary Auxiliary Building, Connecticut Yankee Atomic Power Plant

APPENDIX C

CRANE WALL FLOOR RESPONSE SPECTRA

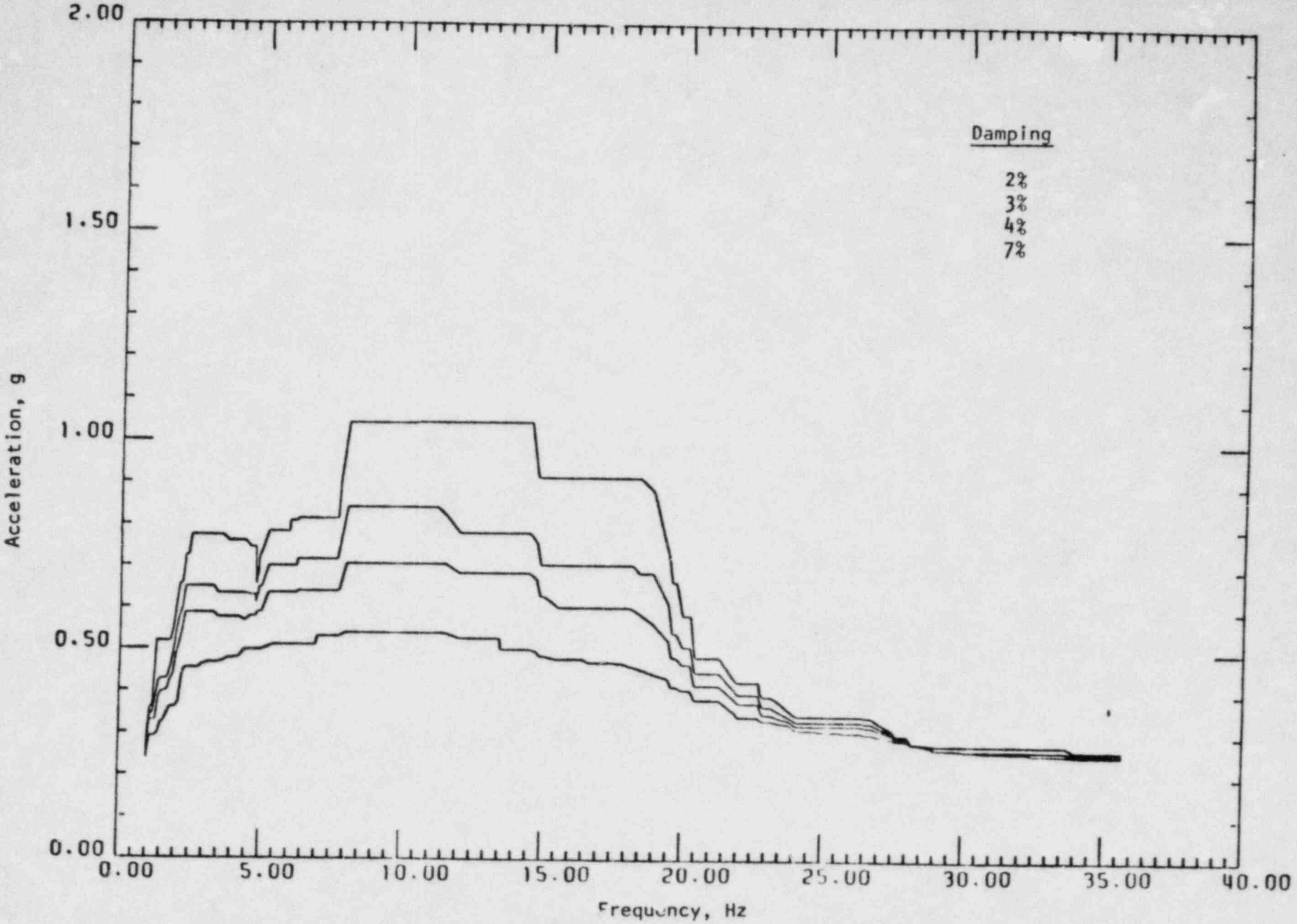


FIGURE 1A: HORIZONTAL (EAST-WEST DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 131, ELEVATION 48.5 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

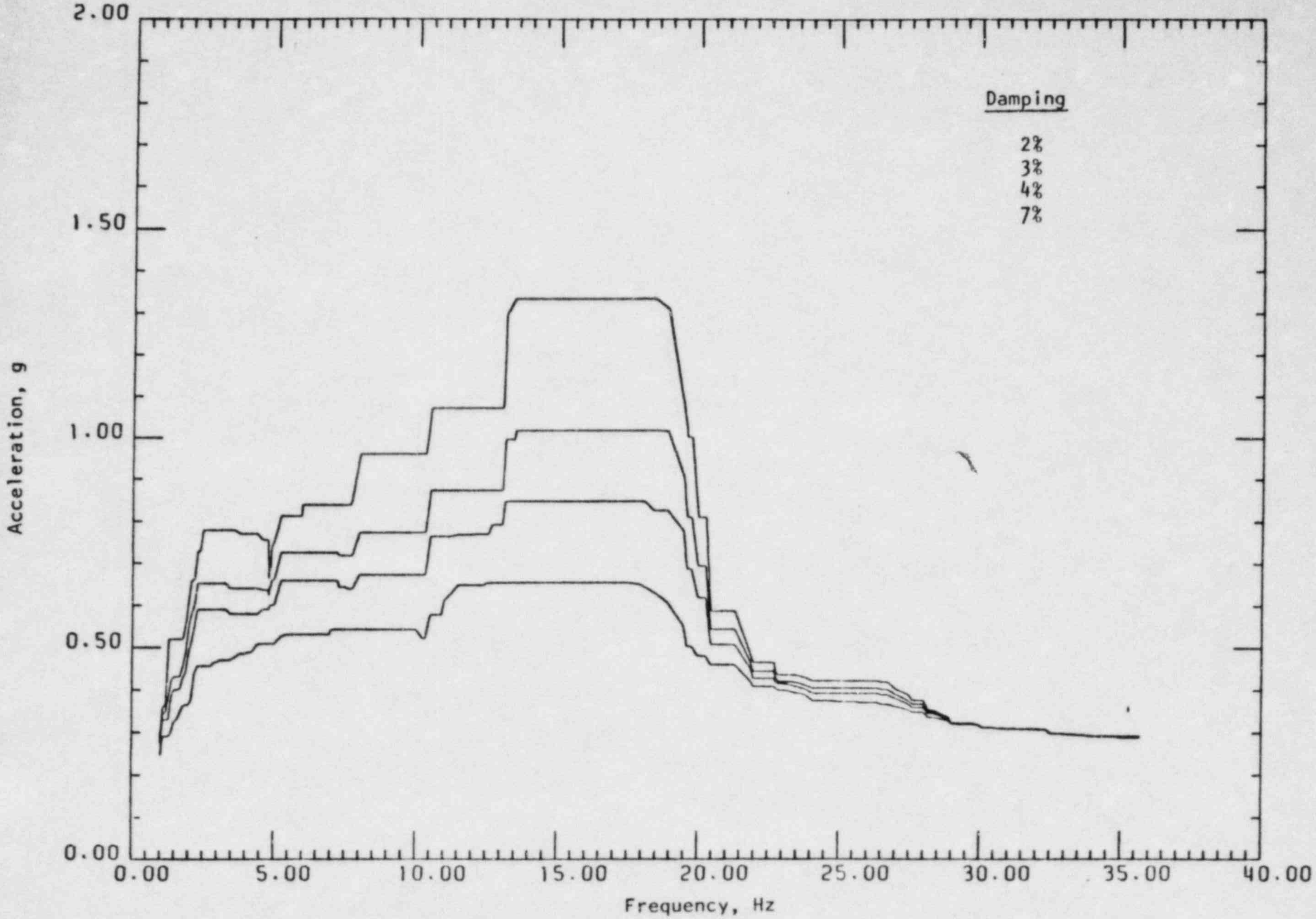


FIGURE 1B: HORIZONTAL (NORTH-SOUTH DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 131, ELEVATION 48.5 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

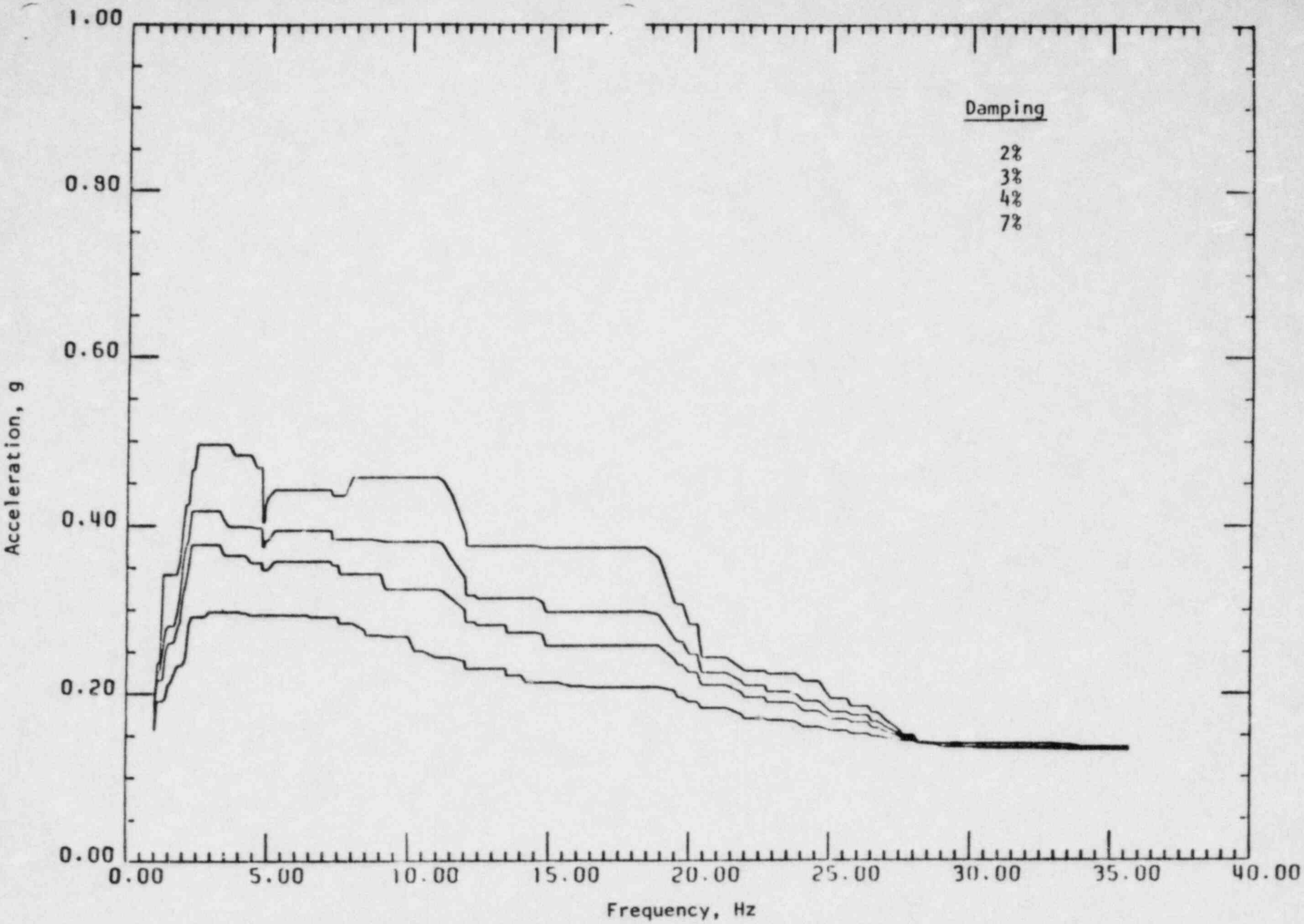


FIGURE 1C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 131, ELEVATION 48.5 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

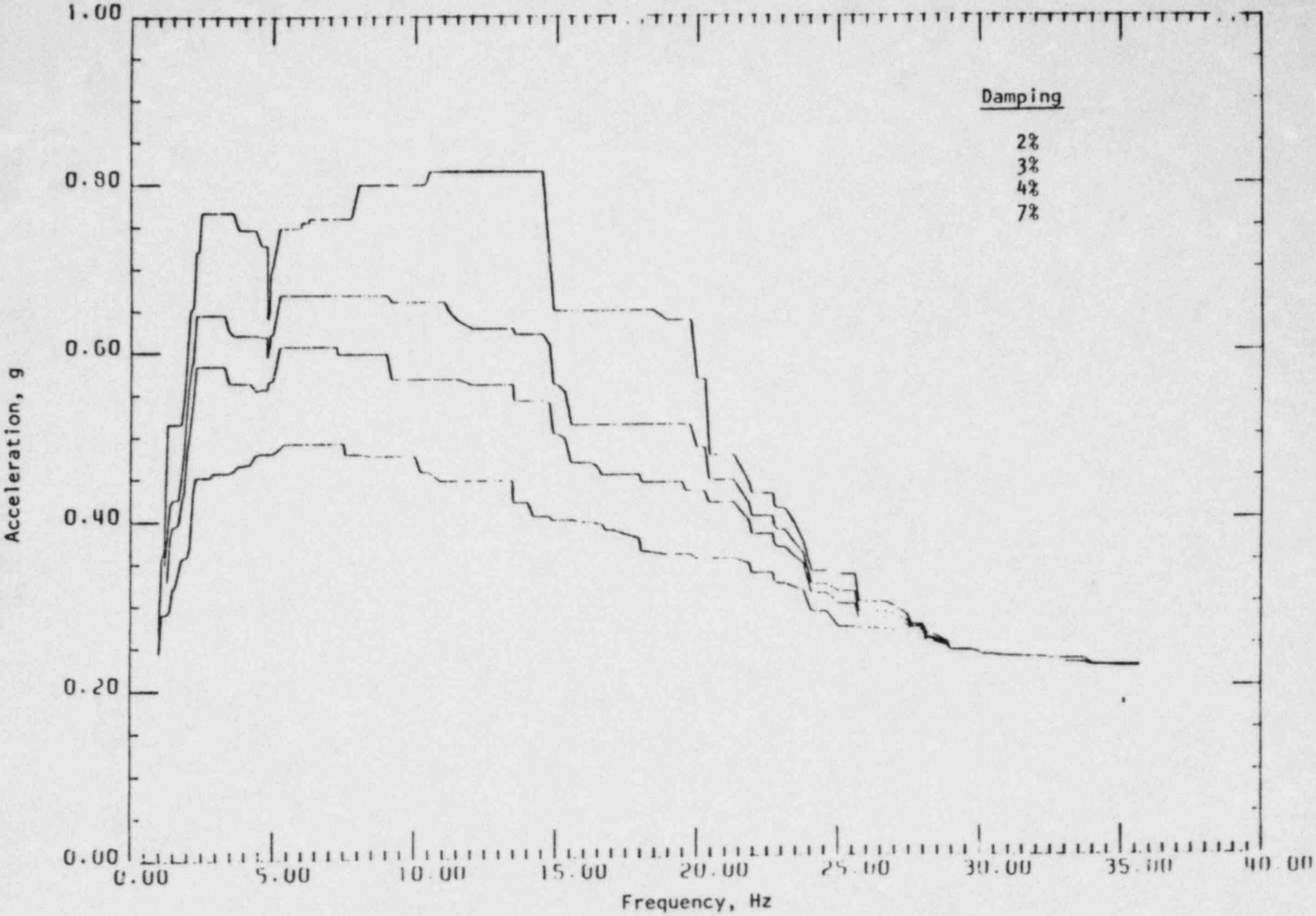


FIGURE 2A: HORIZONTAL (EAST-WEST DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 154, ELEVATION 48.5 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

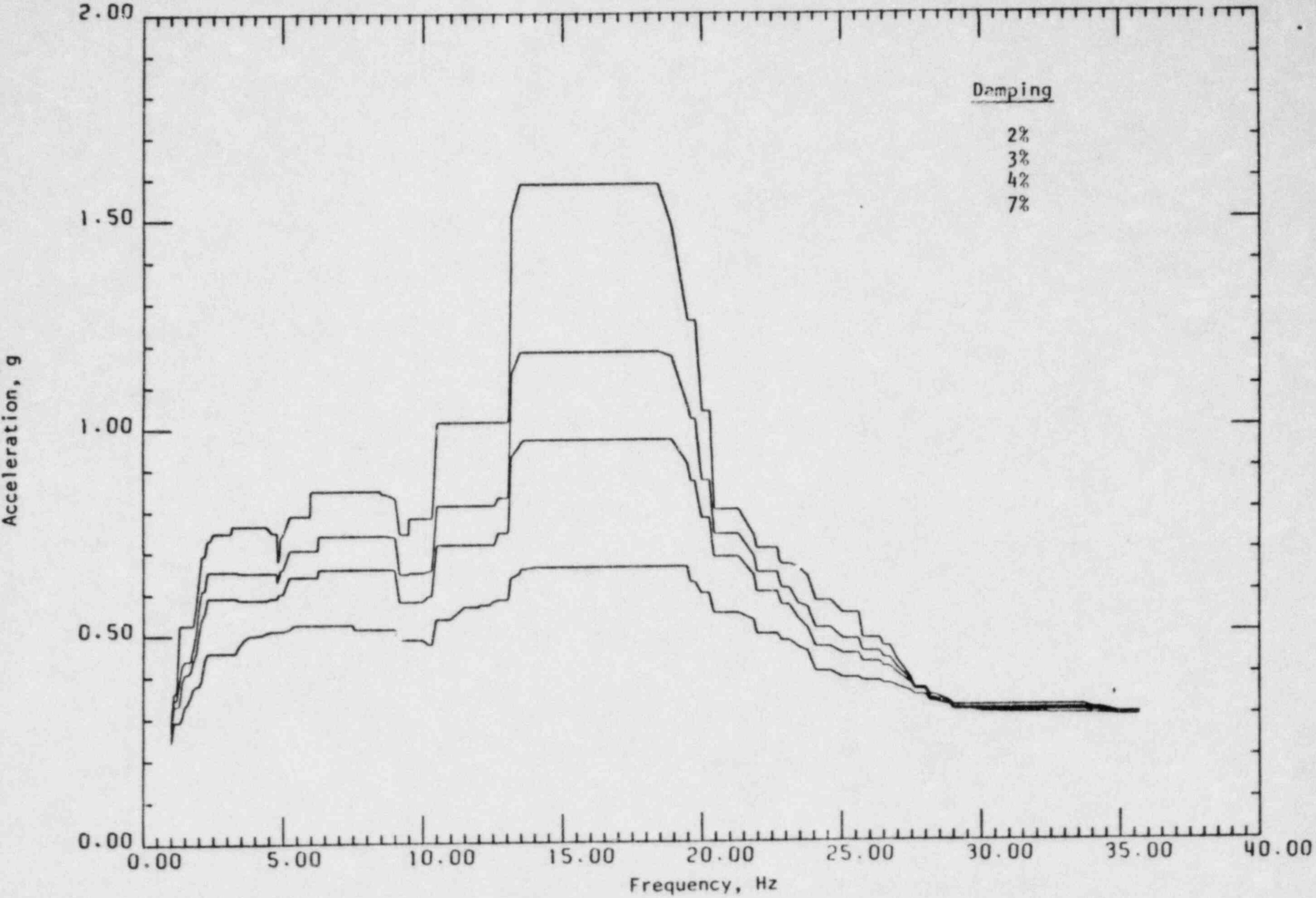


FIGURE 2B: HORIZONTAL (NORTH-SOUTH DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 154, ELEVATION 48.5 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

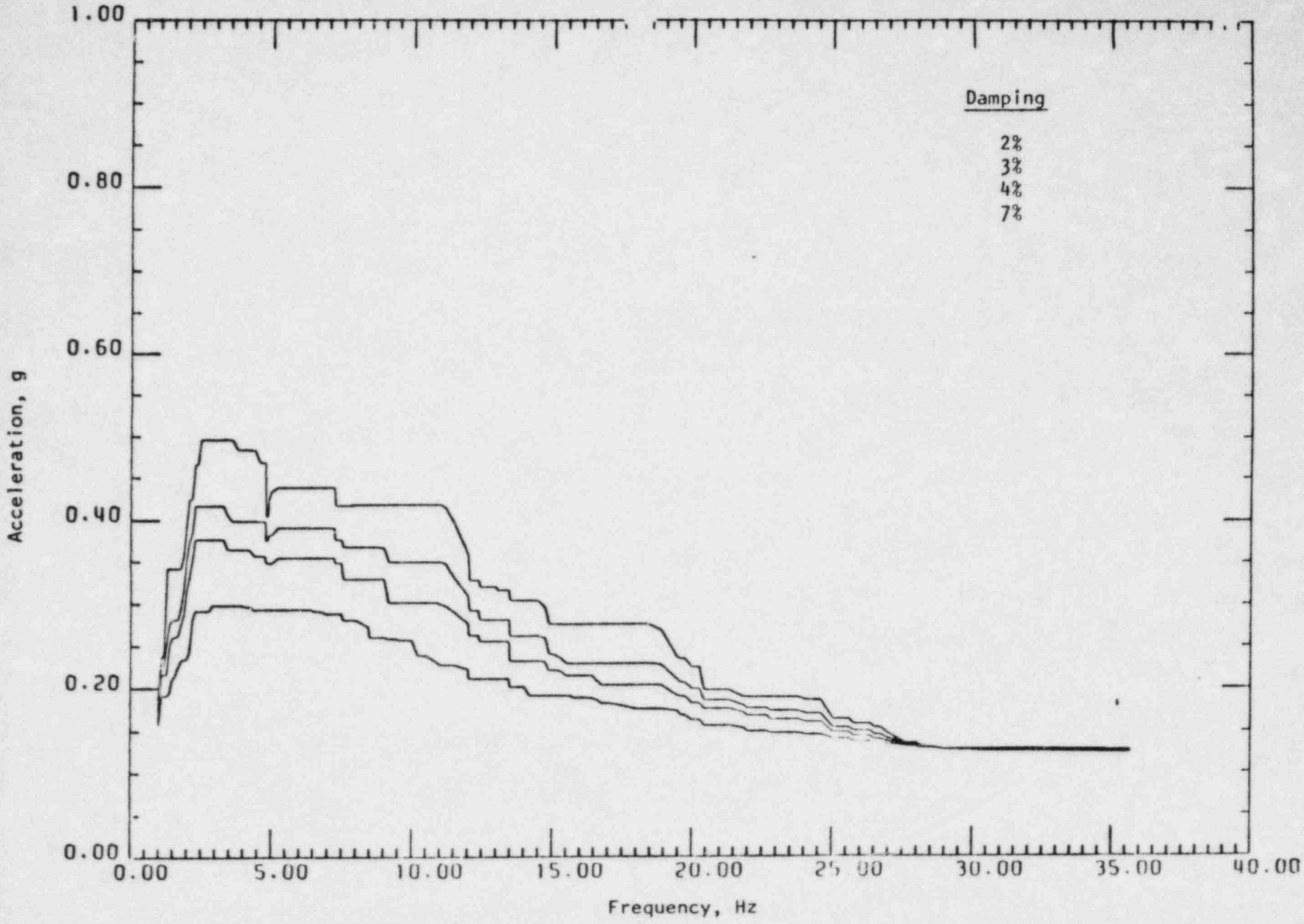


FIGURE 2C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 154, ELEVATION 48.5 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

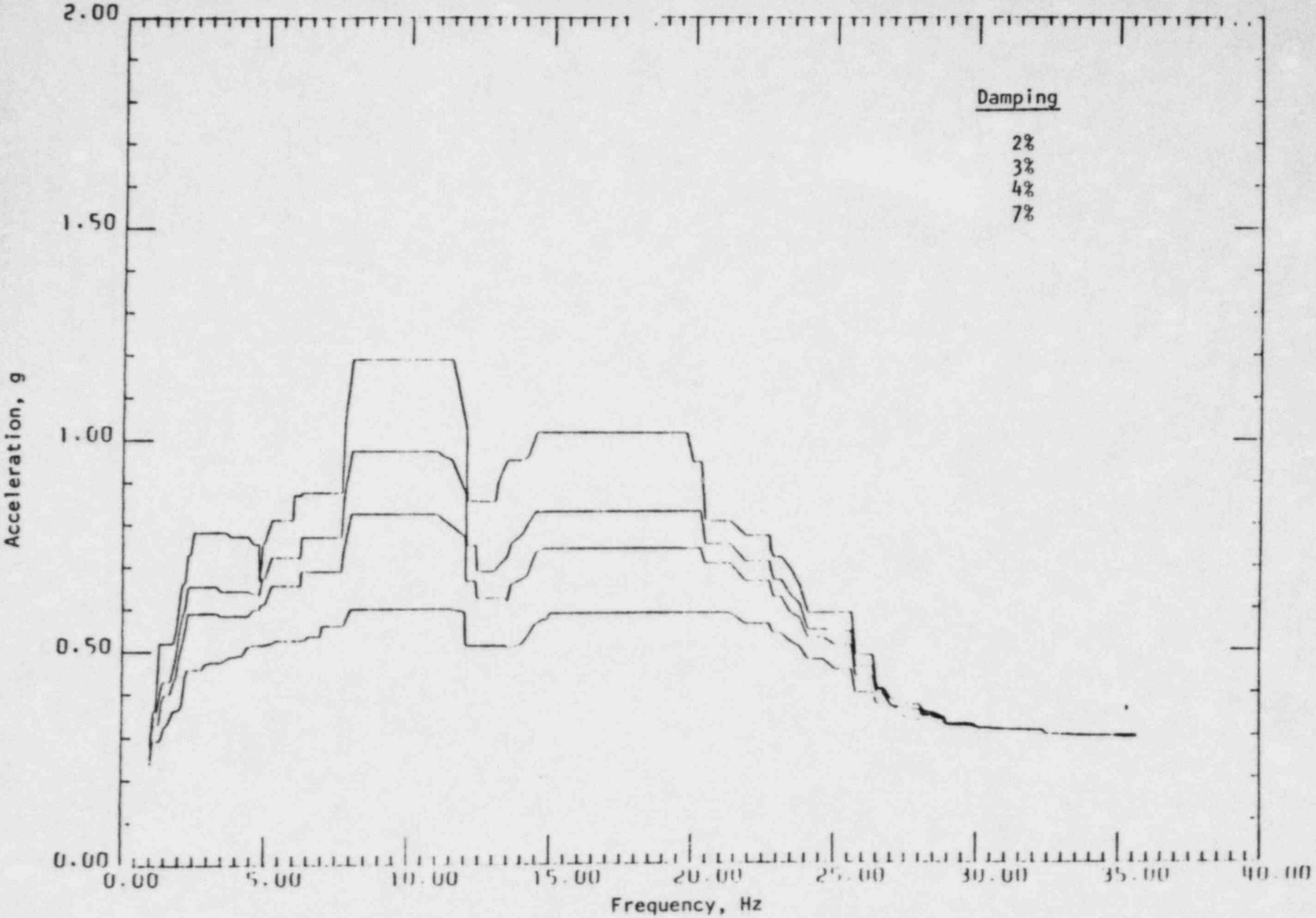


FIGURE 3A: HORIZONTAL (EAST-WEST DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 157, ELEVATION 48.5 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

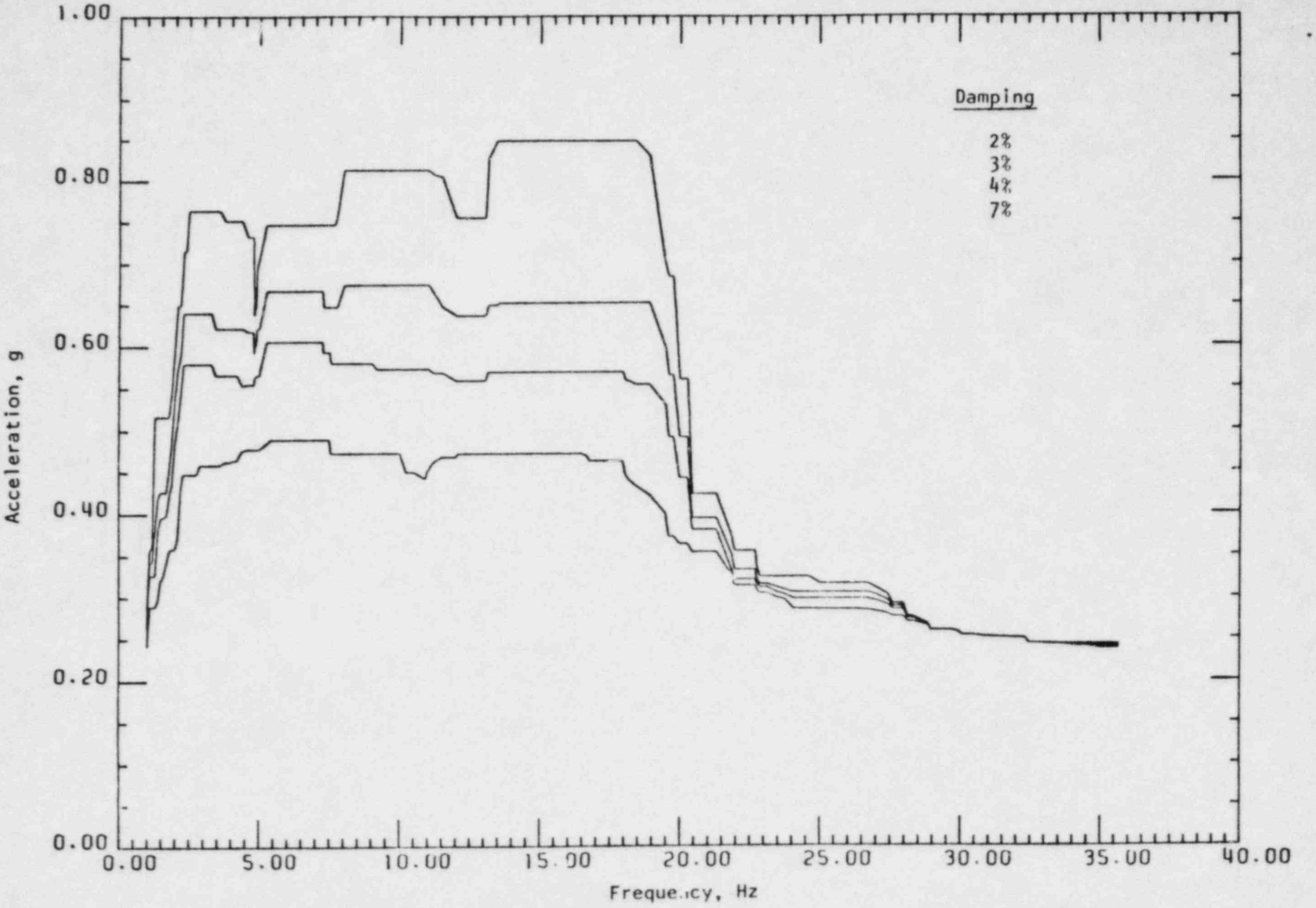


FIGURE 3B: HORIZONTAL (NORTH-SOUTH DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 157, ELEVATION 48.5 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

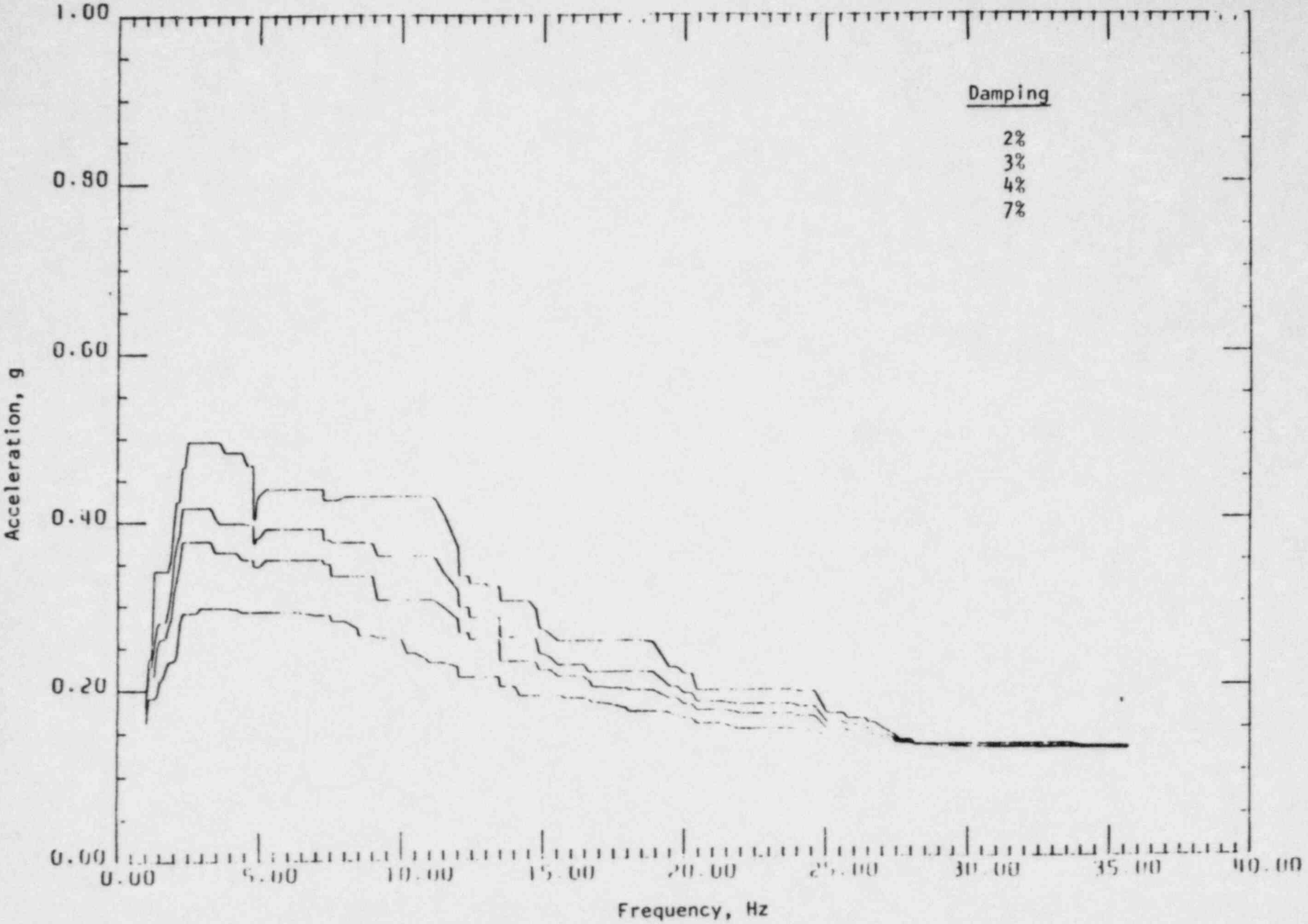


FIGURE 3C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 157, ELEVATION 48.5 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

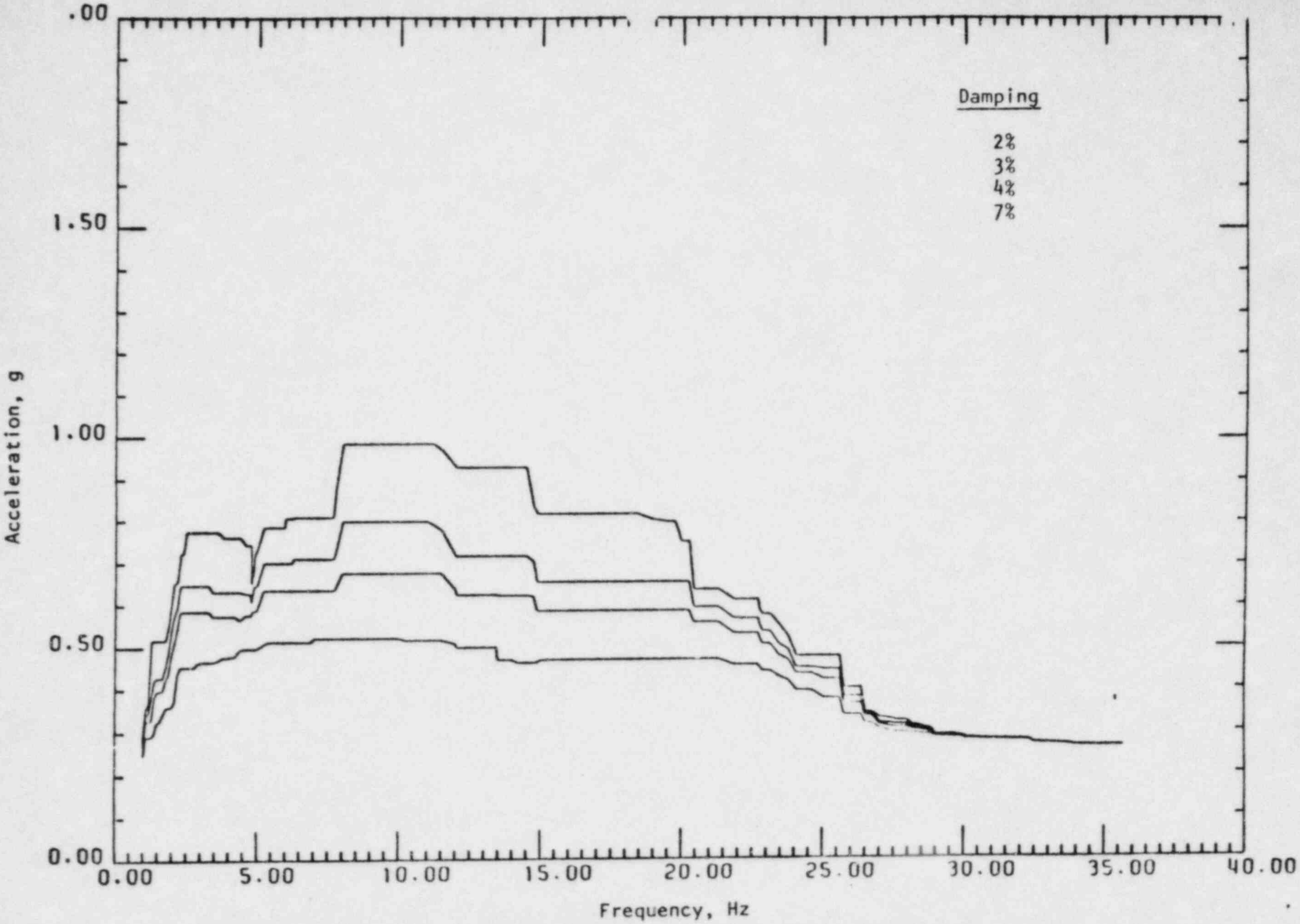


FIGURE 4A: HORIZONTAL (EAST-WEST DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 236, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

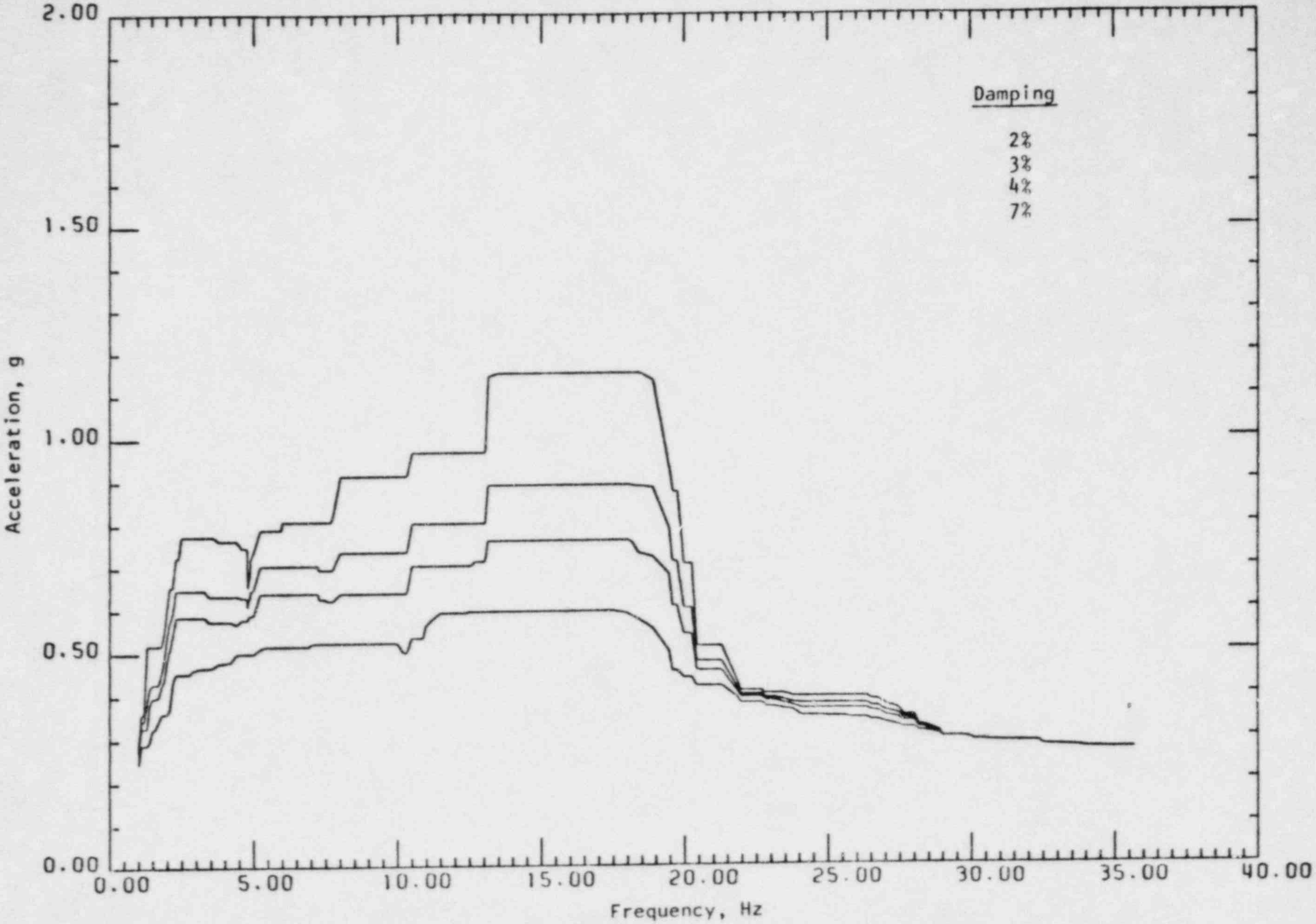


FIGURE 4B: HORIZONTAL (NORTH-SOUTH DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 236, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

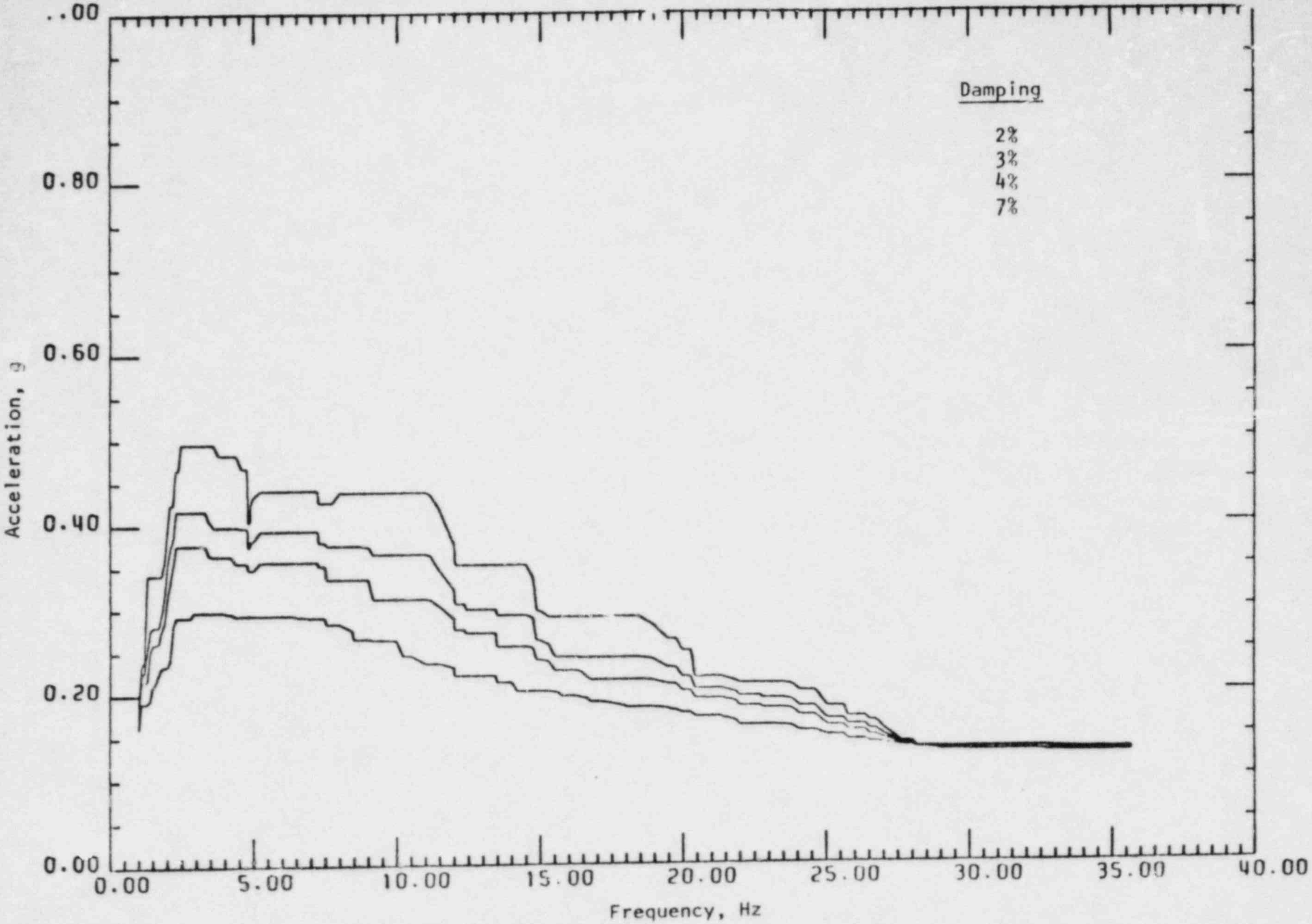


FIGURE 4C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 236, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

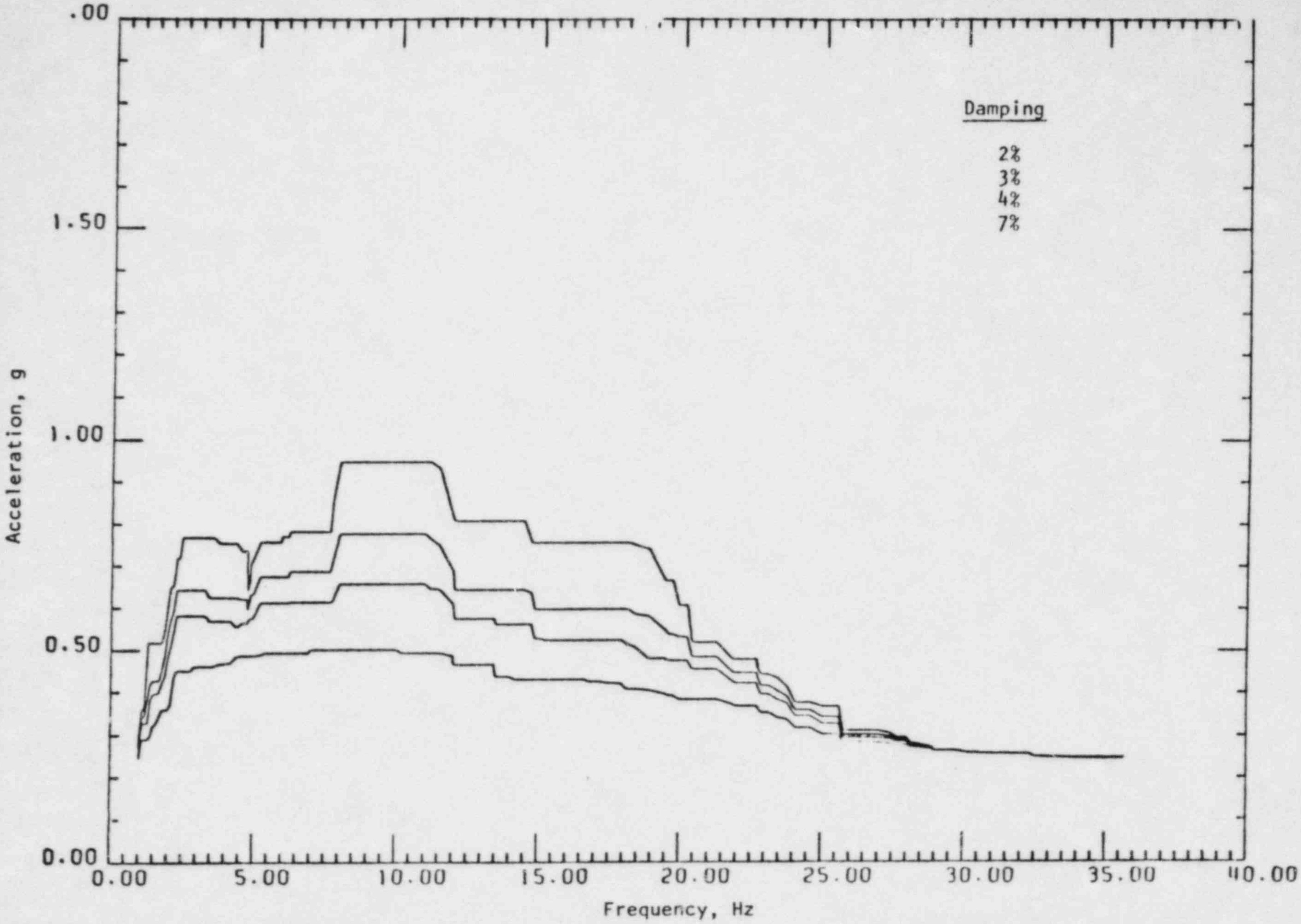


FIGURE 5A: HORIZONTAL (EAST-WEST DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 237, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

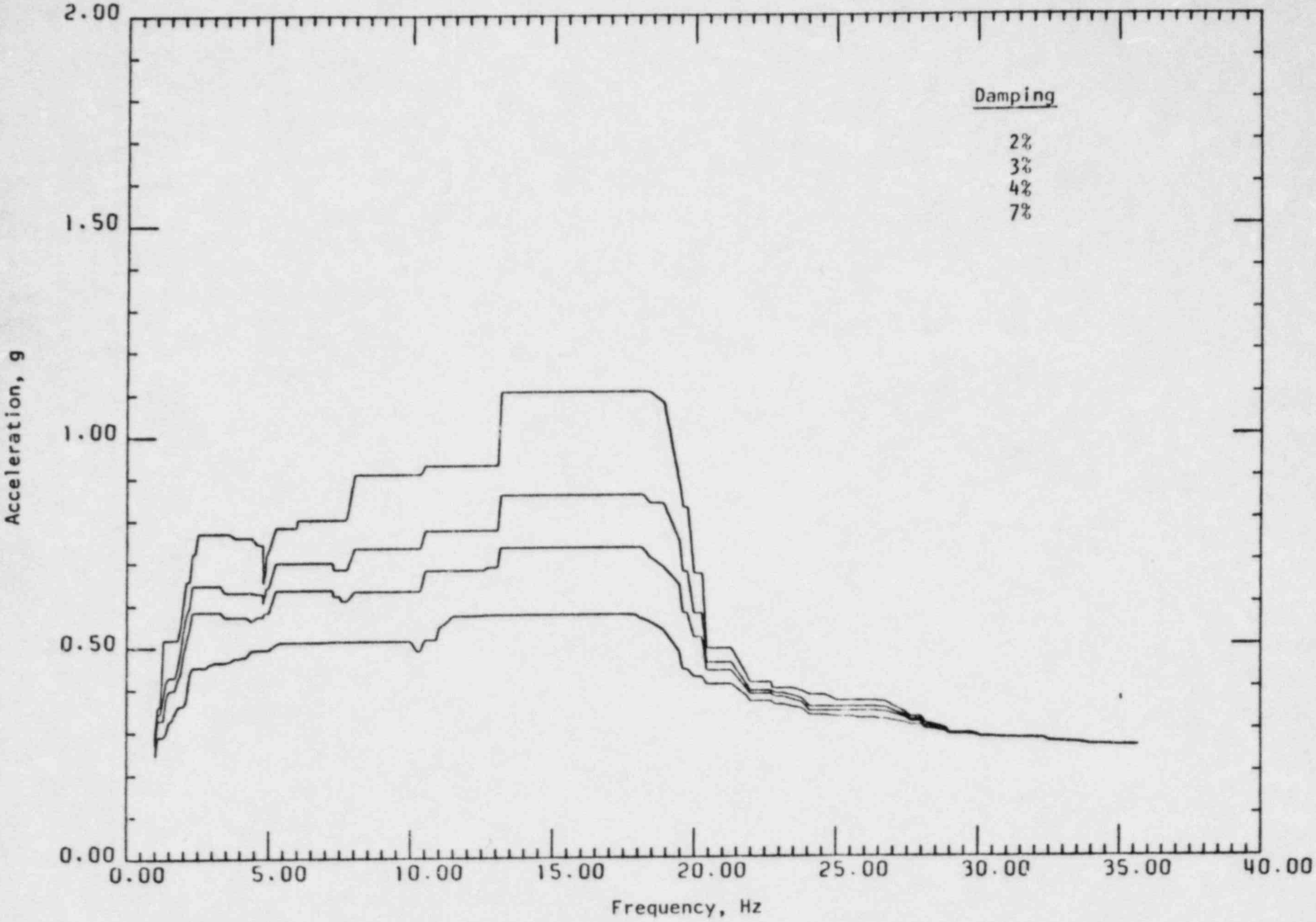


FIGURE 5B: HORIZONTAL (NORTH-SOUTH DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 237, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

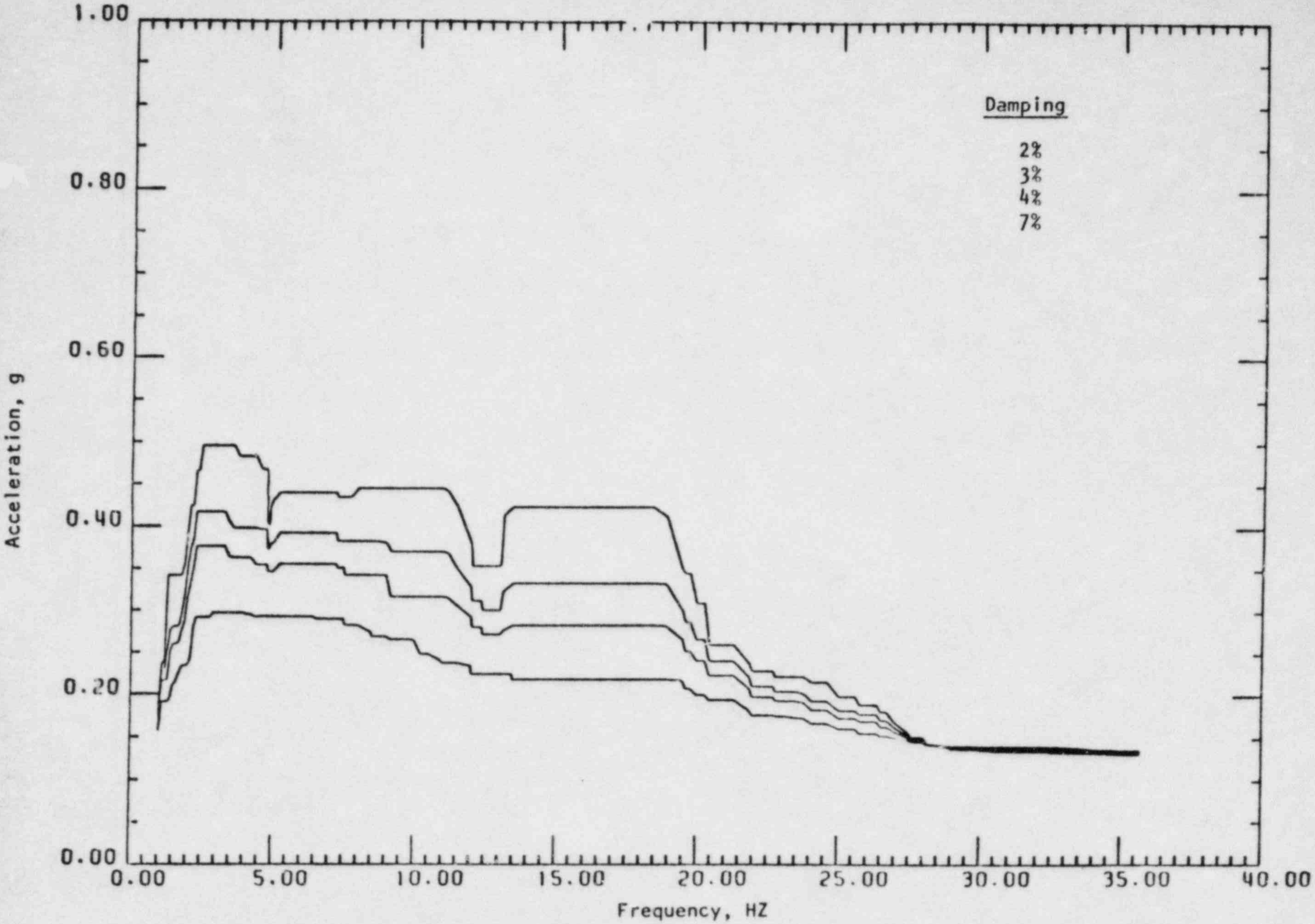


FIGURE 5C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 237, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

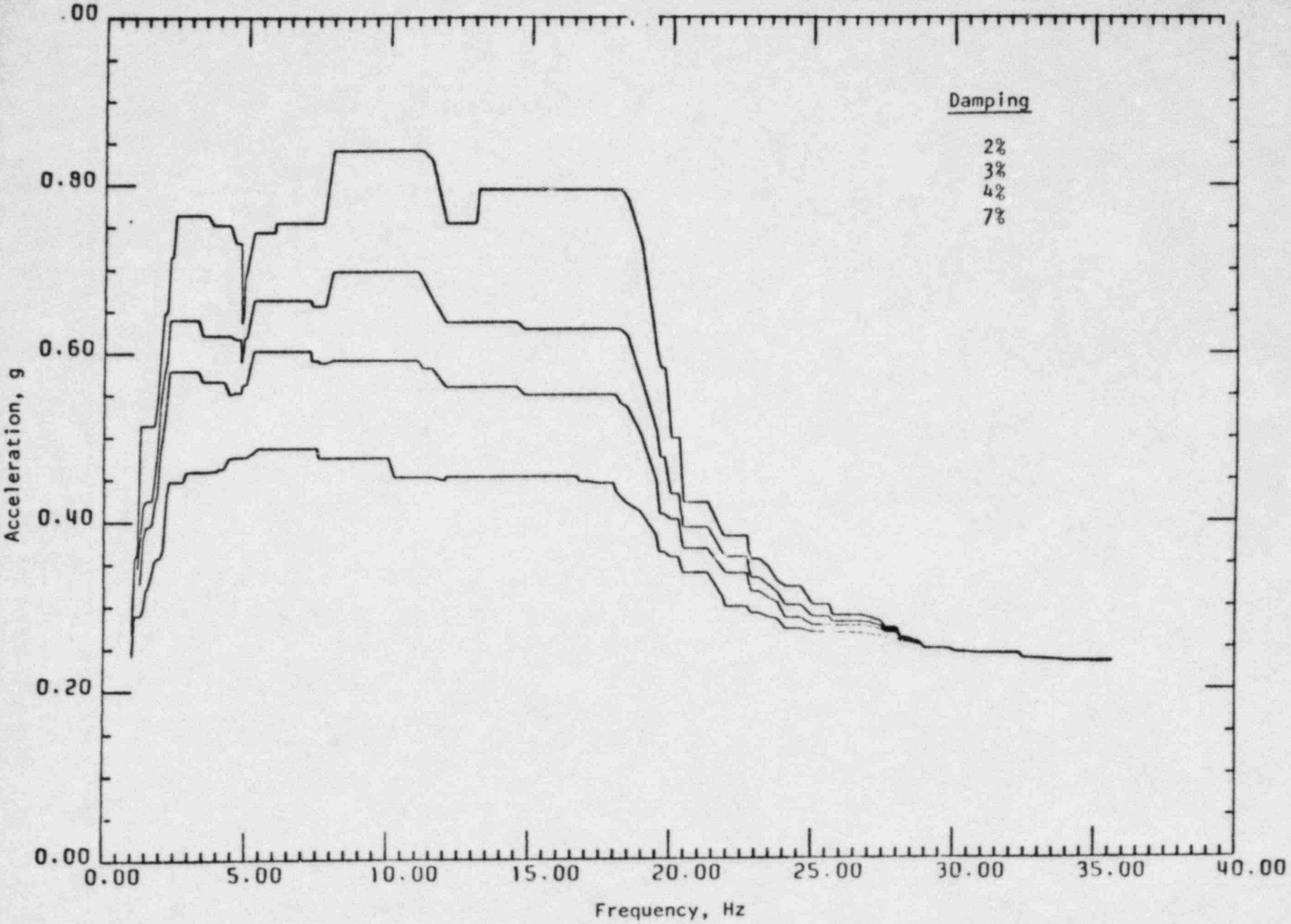


FIGURE 6A: HORIZONTAL (EAST-WEST DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 247, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

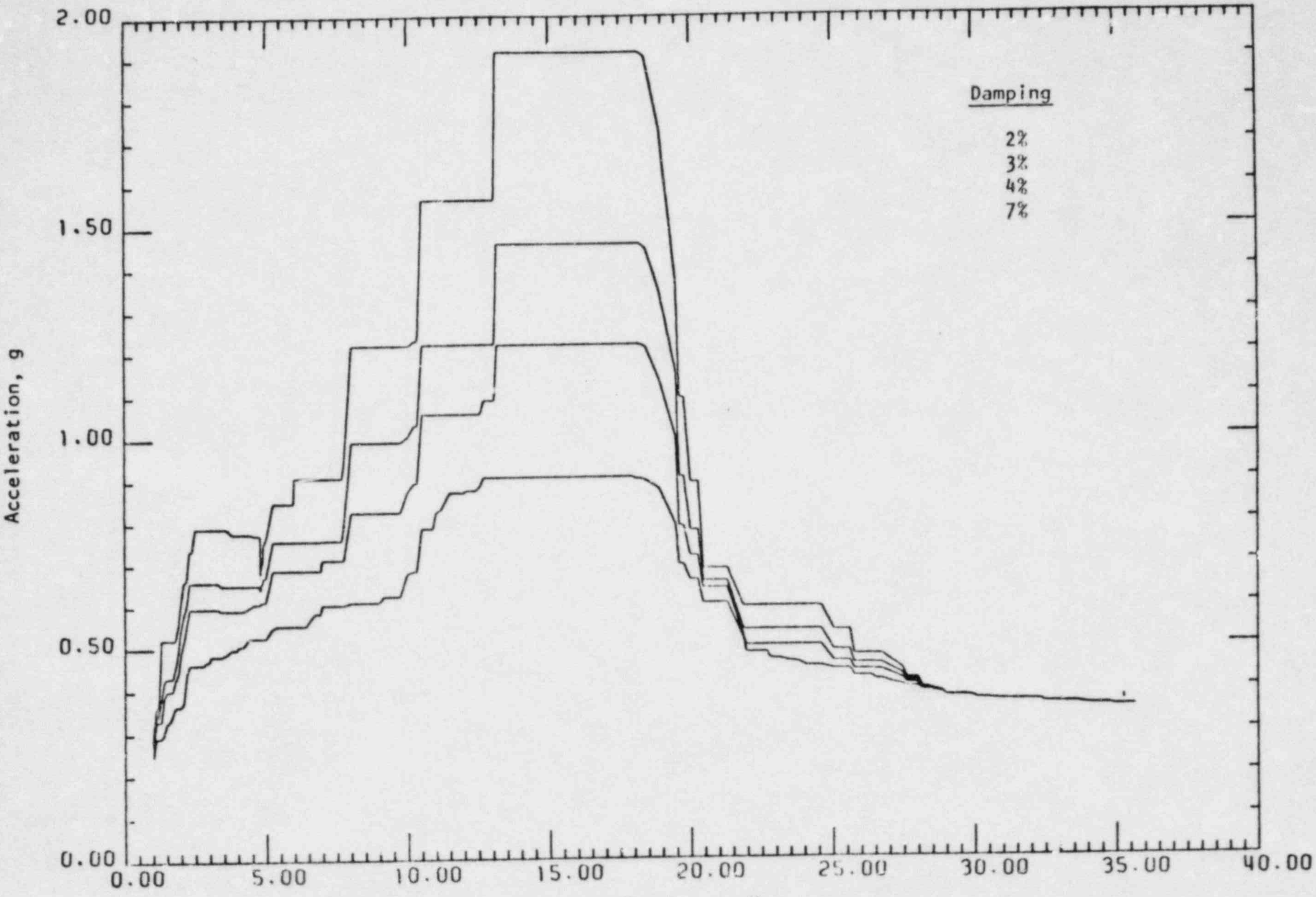


FIGURE 6B: HORIZONTAL (NORTH-SOUTH DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 247, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

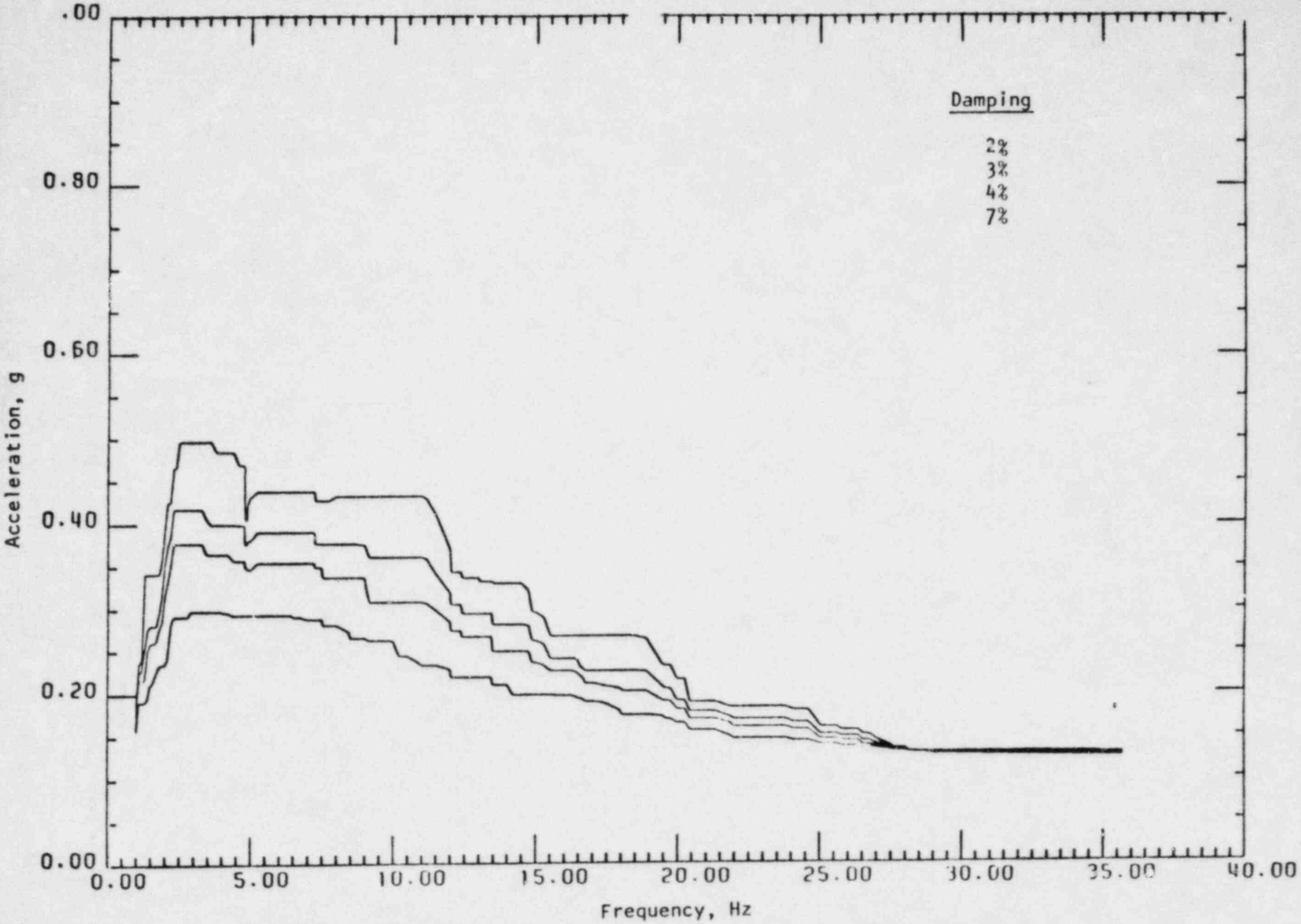


FIGURE 6C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 247, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

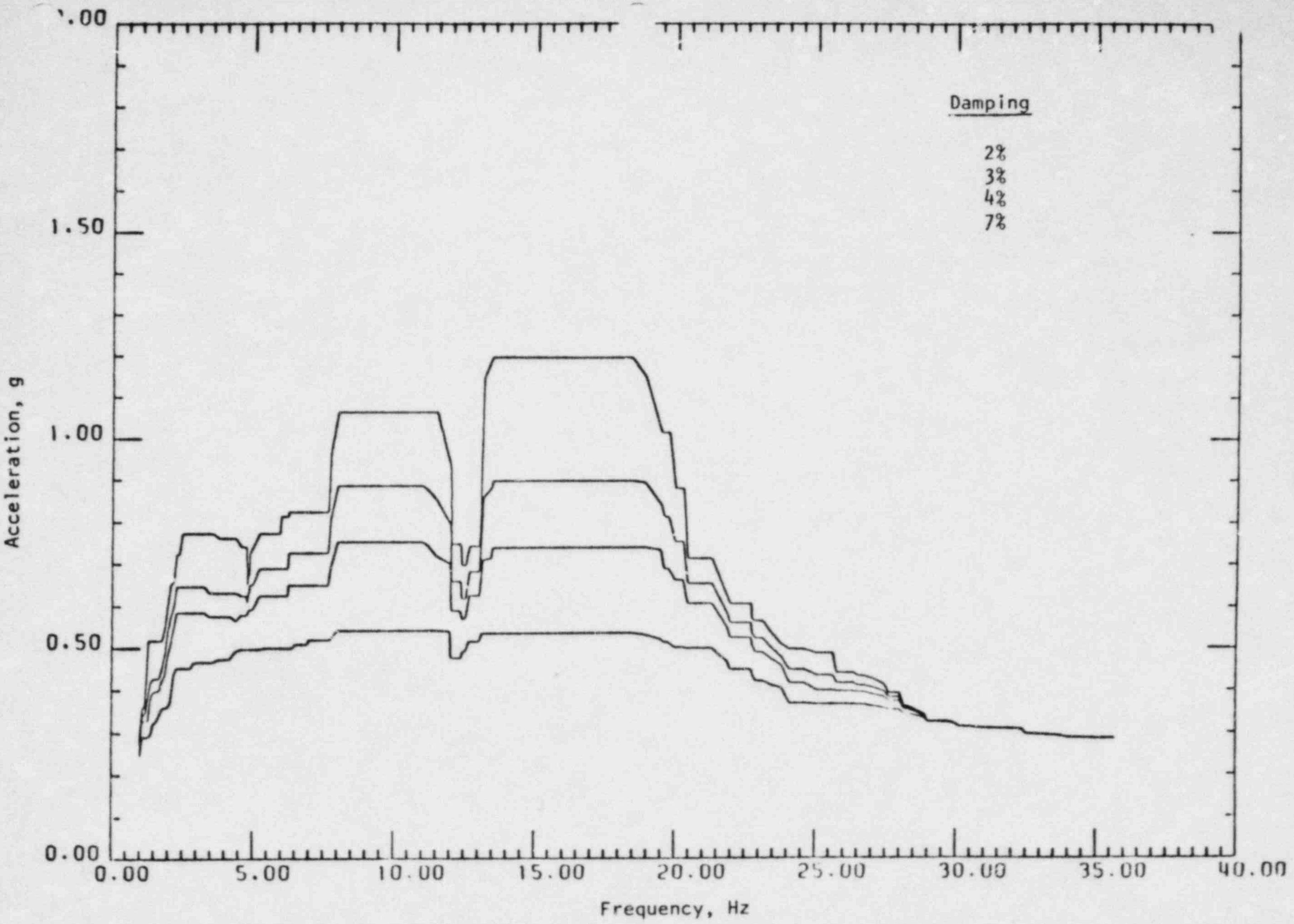


FIGURE 7A: HORIZONTAL (EAST-WEST DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 255, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

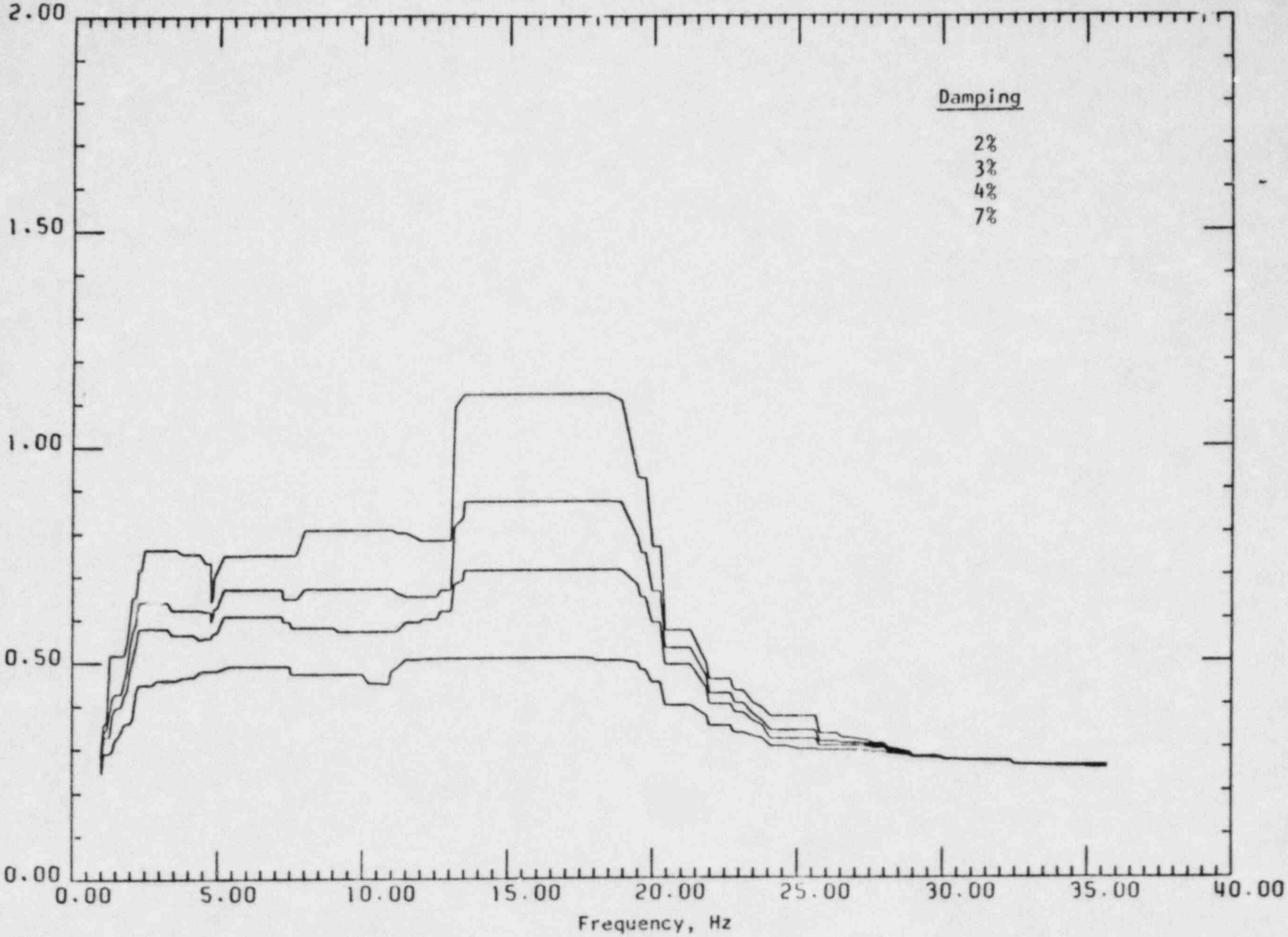


FIGURE 7B: HORIZONTAL (NORTH-SOUTH DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 255, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

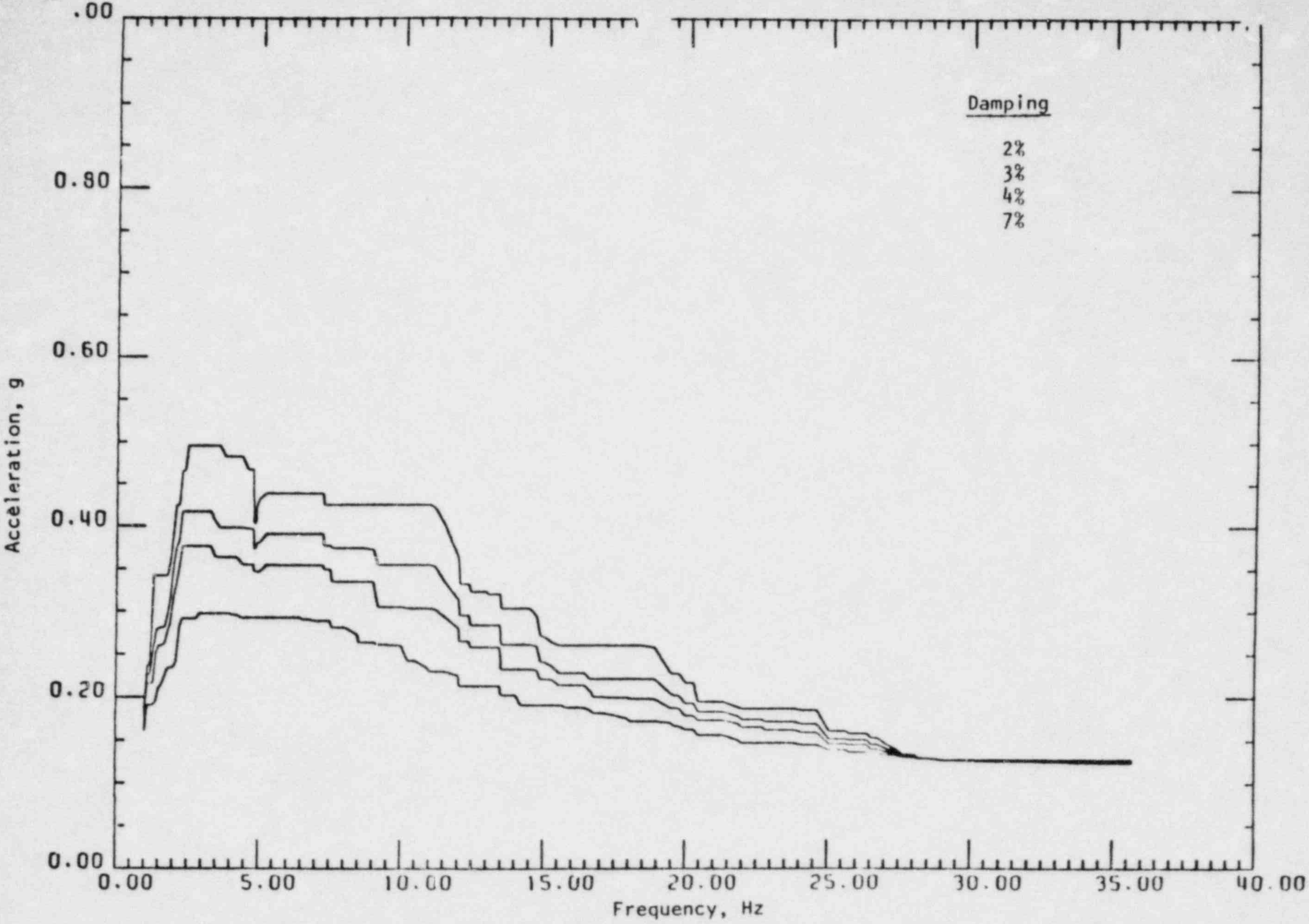


FIGURE 7C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 255, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

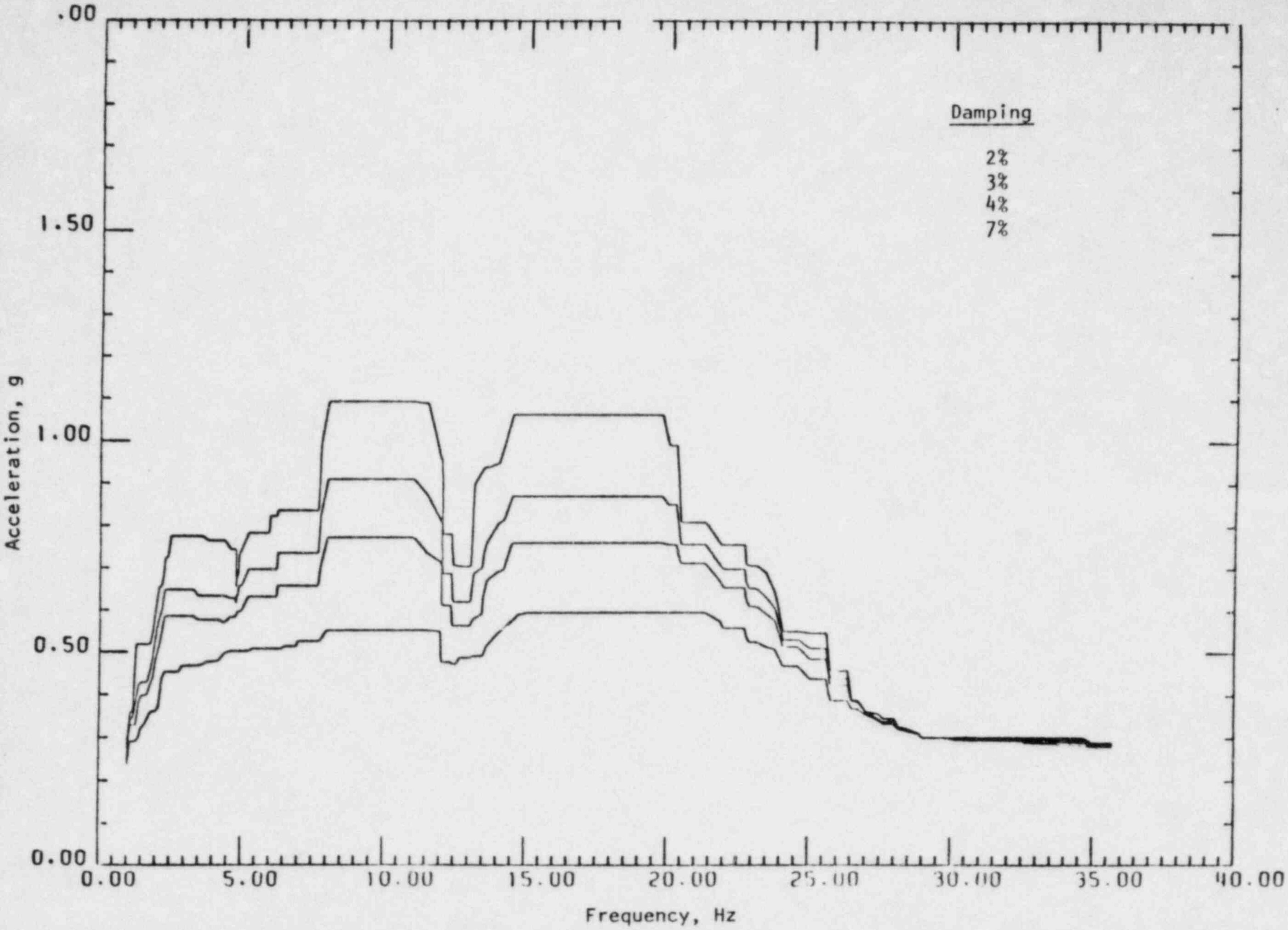


FIGURE 8A: HORIZONTAL (EAST-WEST DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 257, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

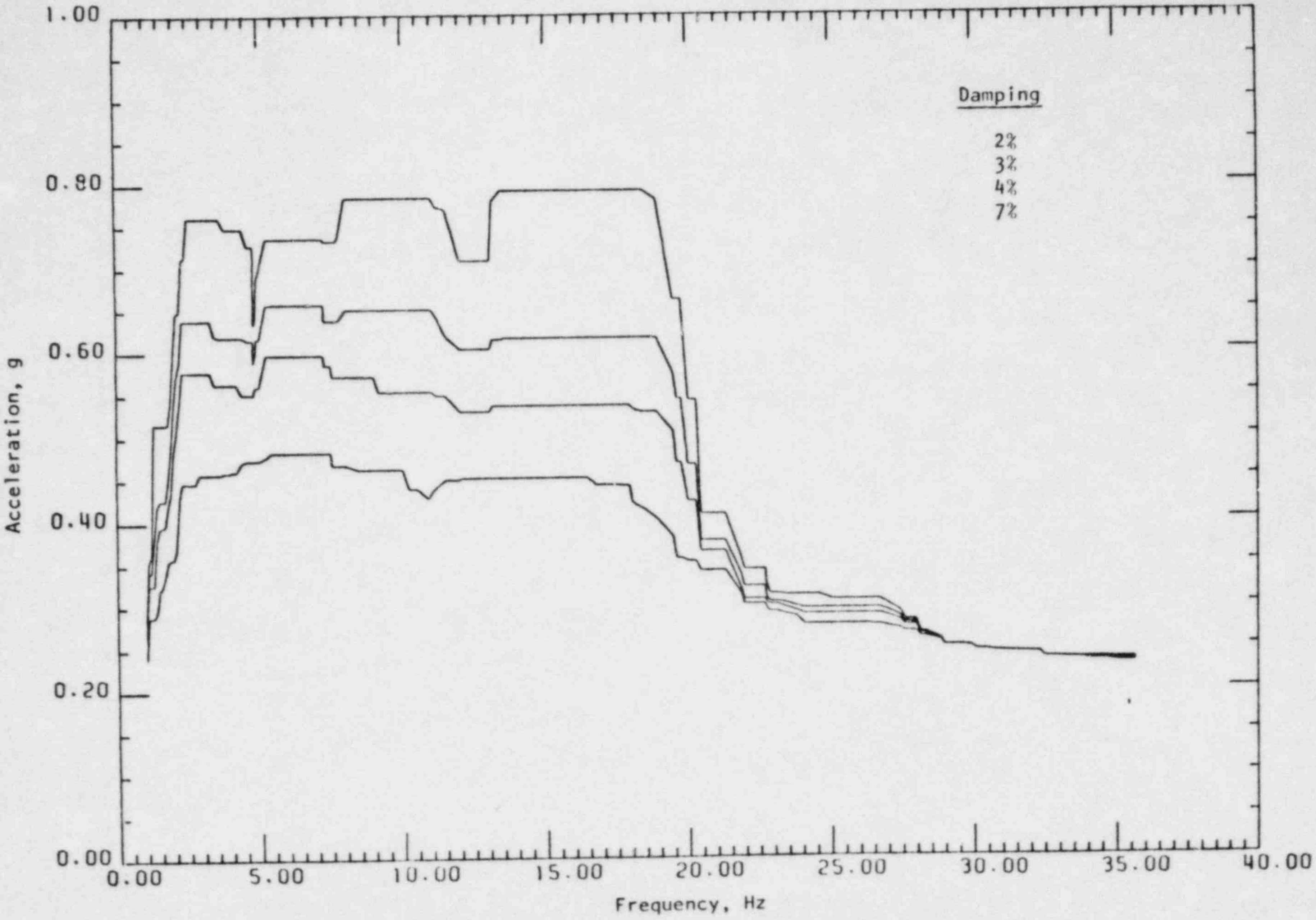


FIGURE 8B: HORIZONTAL (NORTH-SOUTH DIRECTION) FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 257, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE

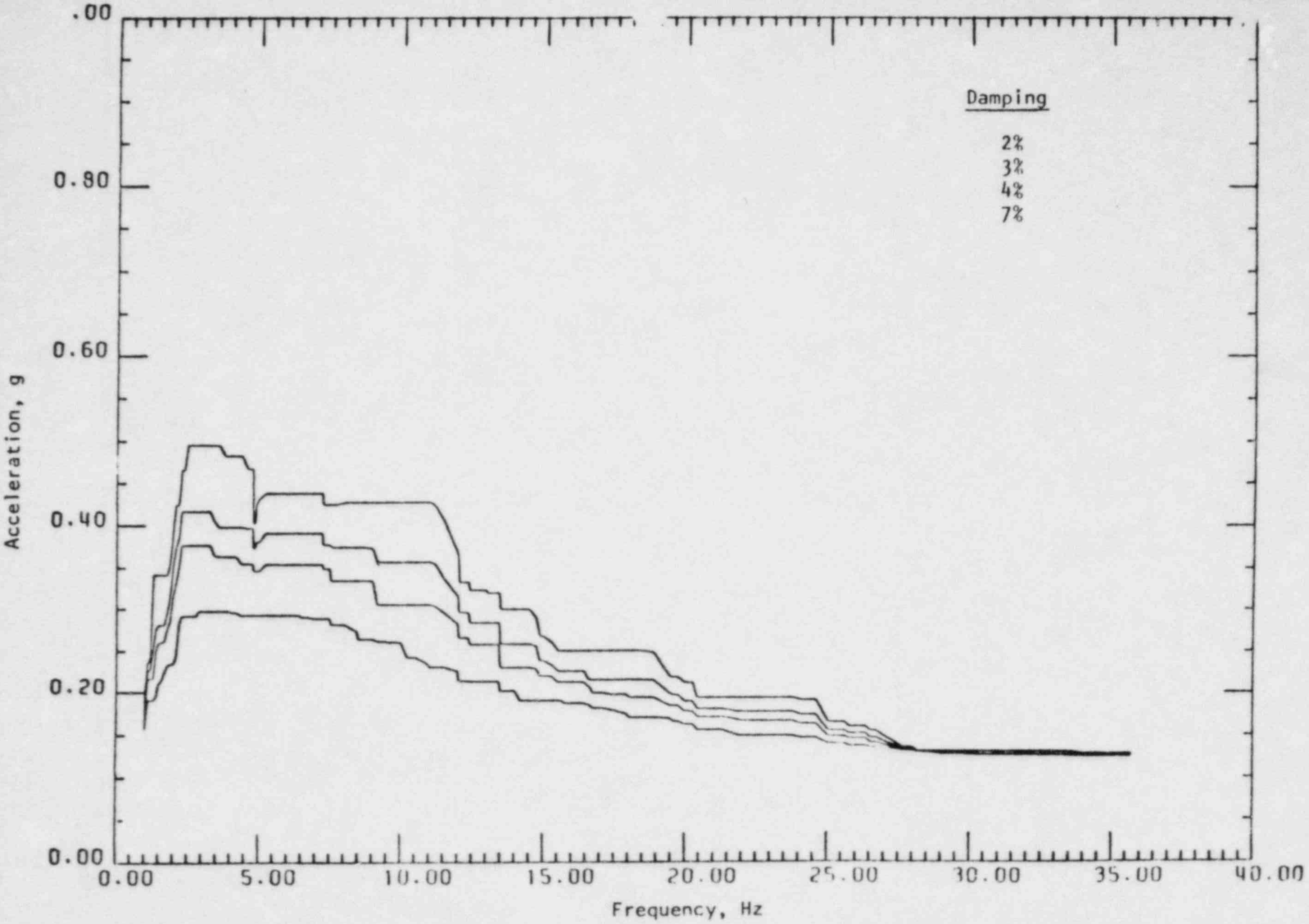


FIGURE 8C: VERTICAL FLOOR ACCELERATION RESPONSE SPECTRA FOR NODE 257, ELEVATION 38.125 FT, CRANE WALL OF CONTAINMENT INTERNAL STRUCTURE