UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D. C. 20555

AUG 1 0 1984

Docket No. 50-354

APPLICANT: Public Service Electric & Gas Company (PSE&G)

FACILITY: Hope Creek Generating Station

SUBJECT: SUMMARY OF STRUCTURAL AND GEOTECHNICAL ENGINEERING BRANCH (SGEB) MEETING

On August 1, 1984, a meeting was held in the Bethesda, Maryland offices of the NRC to discuss the remaining SGEB-Structural audit open items identified in the Draft SER. A list of meeting attendees is included as Enclosure 1 to this meeting summary.

Enclosure 2 is a listing of the open items. Those items for which the "status" column is blank were discussed at the meeting. Based on the meeting, all SGEB-Structural open items are considered closed with the following exceptions:

Open Item	Status/Action
39 & 51	PSE&G is to submit to the NRC staff: 1. information on the percentage of walls that have been evaluated, and 2. information on modeling (for SGEB-Geotechnical)
40 & 66	PSE&G will provide impedance values and stiffnesses (for SGEB-Geotechnical)
43	This item is currently under review by the Containment Systems Branch
44,45,46	PSE&G is currently revising response to Question 220.26 to indicate that ACI 349 deviations have no impact
47	Comparison with Regulatory Guide 1.60 curves will be provided

54 PSE&G to provide explanation of difference in February 17, 1984 and July 24, 1984 responses

The above listing is subject to revision resulting from further review.

Enclosure 3 contains draft responses to the open items identified for discussion at the meeting. PSE&G representatives indicated Enclosure 3 with the discussed revisions/actions incorporated within, will be submitted formally for staff review by August 10, 1984.

David H. Wagner

David H. Wagner, Project Manager
Licensing Branch No. 2
Division of Licensing

Enclosures: As stated

cc: See next page

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Enclosures: As stated

cc: See next page

SGEB/DE	DL:LB#2/PM	DL:LB#2/BC
LYang	DWagner:bdm	ASchwencer
8/10/84	8/10/84	8/10/84

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Enclosure 1

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OPEN ITEM NO.	DSER SECTION	RESP.*	SUBJECT	STRUCTURAL AUDIT ITEM NO.	AUDIT ITEM RESPONSE SENT TO NRC	STATUS
39	3.7.2.3	B	SSI Analysis for reactor & aux. bldgs. (220.21)	1/10/84, A.13	7/24/84	
40	3.7.2.3	B	SSI analysis for intake structure (220.21)	1/11/84, A.16	7/24/84	
41	3.8.2	B	Steel containment buckling analysis (220.11)	1/12/84, B.1	2/17/84	Closed
42	3.8.2	B	Ultimate capacity analysis (220.22)	1/12/84, B.2	2/17/84 7/24/84R	
43	3.8.2	N	SRV/LOCA pool dynamic loads	—	PUAR sent 2/10/84	
44	3.8.3	B	ACI 349 deviations for internal structure (220.24)	—	Incorp. in Amend. 4	
45	3.8.4	B	ACI 349 deviations for Cat. I structure (220.26)	—	Incorp. in Amend. 4	
46	3.8.5	B	ACI 349 deviations for foundations (220.26)	—	Incorp. in Amend. 4	
47	3.8.6	I	Base mat response spectra	1/10/84, A.3	2/17/84 7/24/84R	

* B - Bechtel
I - Impell
N - Nutech

Enclosure 2

DSER Structural Open Items

OPEN ITEM NO.	DSER SECTION	RESP.	SUBJECT	STRUCTURAL AUDIT ITEM NO.	AUDIT ITEM RESPONSE SENT TO NRC	STATUS
48	3.8.6	I	Rocking time histories	1/10/84, A.4	2/17/84 7/24/84R	
49	3.8.6	B/I	Gross concrete section	1/10/84, A.11	2/17/84 7/24/84R	
50	3.8.6	I	Vert. flr. flex. response spectra	1/10/84, A.12	2/17/84 7/24/84R	
51	3.8.6	B	Comp. of BPC independent verf. results (220.21)	1/10/84, A.13	7/24/84	
52	3.8.6	B	Ductility ratios due to pipe break (220.4)	1/10/84, A.16	2/17/84 7/24/84R	
53	3.8.6	B	Design of Cat. I tanks	1/12/84, A.4	4/24/84 7/24/84R	
54	3.8.6	I	Comb. of vert. responses	1/10/84; B.5	2/17/84 7/24/84R	
55	3.8.6	I	Torsional stiffness calc.	1/10/84, B.8	1/26/84	Closed

OPEN ITEM NO.	DSER SECTION	RESP.	SUBJECT	STRUCTURAL AUDIT ITEM NO.	AUDIT ITEM RESPONSE SENT TO NRC	STATUS
56	3.8.6	I	Drywell stick model development	1/10/84, B.9	1/26/84 7/24/84R	
57	3.8.6	I	Rotational time hist. inputs	1/10/84, B.10	2/17/84	Closed
58	3.8.6	I	"O" ref. pt. for aux. bldg. model	1/10/84, B.11	1/26/84	Closed
59	3.8.6	B	Overturning moment for RB foun. mat	1/11/84, A.7	1/26/84 7/24/84R	
60	3.8.6	B	BSAP element size limitations	1/11/84, A.8	2/17/84 7/24/84R	
61	3.8.6	B/I	Seismic modeling of drywell shield wall	1/11/84, A.9	2/17/84	Closed
62	3.8.6	B	Drywell shield wall bound. cond.	1/11/84, A.10	1/26/84	Closed
63	3.8.6	B	RB dome bound. cond.	1/11/84, A.11	1/26/84	Closed

OPEN ITEM NO.	DSER SECTION	RESP.	SUBJECT	STRUCTURAL AUDIT ITEM NO.	AUDIT ITEM RESPONSE SENT TO NRC	STATUS
64	3.8.6	I	SSI analysis - 12 Hz cutoff	1/11/84, A.12	2/17/84 7/24/84R	
65	3.8.6	B	Intk. struc. heavy load drop	1/11/84, A.15	1/26/84	Closed
66	3.8.6	B	Impedance anal. for intk. structure (220.21)	1/11/84, A.16	7/24/84	
67	3.8.6	B	Crit. load calc. for RB dome	1/11/84, A.17	1/26/84	Closed
68	3.8.6	B	RB found. mat contact press.	1/11/84, B.1	1/26/84	Closed
69	3.8.6	B	Sliding & overturning of drywell shield wall	1/11/84, B.2	1/26/84	Closed
70	3.8.6	B	Seismic shear force dist. in cyl. wall	1/11/84, B.3	1/26/84	Closed
71	3.8.6	B	Overturning of cyl. wall	1/11/84, B.4	1/26/84	Closed
72	3.8.6	B	Deep beam design of fuel pool walls	1/11/84, B.5	1/26/84	Closed
73	3.8.6	B	ASHSD dome model load inputs	1/11/84, B.6	1/26/84	Closed

OPEN ITEM NO.	DSER SECTION	RESP.	SUBJECT	STRUCTURAL AUDIT ITEM NO.	AUDIT ITEM RESPONSE SENT TO NRC	STATUS
74	3.8.6	B	Tornado depressurization	1/11/84, B.7	1/26/84	Closed
75	3.8.6	B	Aux. bldg. abnormal press	1/11/84, B.8	1/26/84	Closed
76	3.8.6	B	Tang. shear in drywell shield wall and cyl. wall	1/11/84, B.9	1/26/84	Closed
77	3.8.6	B	Overturning of intk. struc.	1/11/84, B.12	1/26/84 7/24/84R	
78	3.8.6	B	Dead load cal.	1/11/84, B.13	1/26/84	Closed
79	3.8.6	N	Torus post-mod seismic loads	1/12/84, A.1	1/26/84 7/24/84R	
80	3.8.6	N	Torus fluid struct. interactions	1/12/84, A.2	1/26/84	Closed
81	3.8.6	N	Seismic displ. of torus	1/12/84, A.3	1/26/84 7/24/84R	

DSER Structural Open Items

OPEN ITEM NO.	DSER SECTION	RESP.	SUBJECT	STRUCTURAL AUDIT ITEM NO.	AUDIT ITEM RESPONSE SENT TO NRC	STATUS
82	3.8.6	B	Cat. I tank design	1/12/84, A.4	4/24/84 7/24/84R	
83	3.8.6	B	Drywell buckling eval. (220.11)	1/12/84, B.1	2/17/84	Closed
84	3.8.6	B	Ultimate cap. of cont. (materials) (220.22)	1/12/84, B.2	2/17/84 7/24/84R	
85	3.8.6	B	Load comb consistency	1/12/84, B.3	2/17/84	Closed

Enclosure 3

Response to NRC Audit

Meeting Date: January 10, 1984

Question No.: A-3

Revised Response
Revision 1
6/30/84

DRAFT

QUESTION: Provide comparison between basemat response spectra and regenerated response spectra at basemat.

RESPONSE: Comparison of spectra for 2% damping was provided in the original response for both SSE and OBE cases.

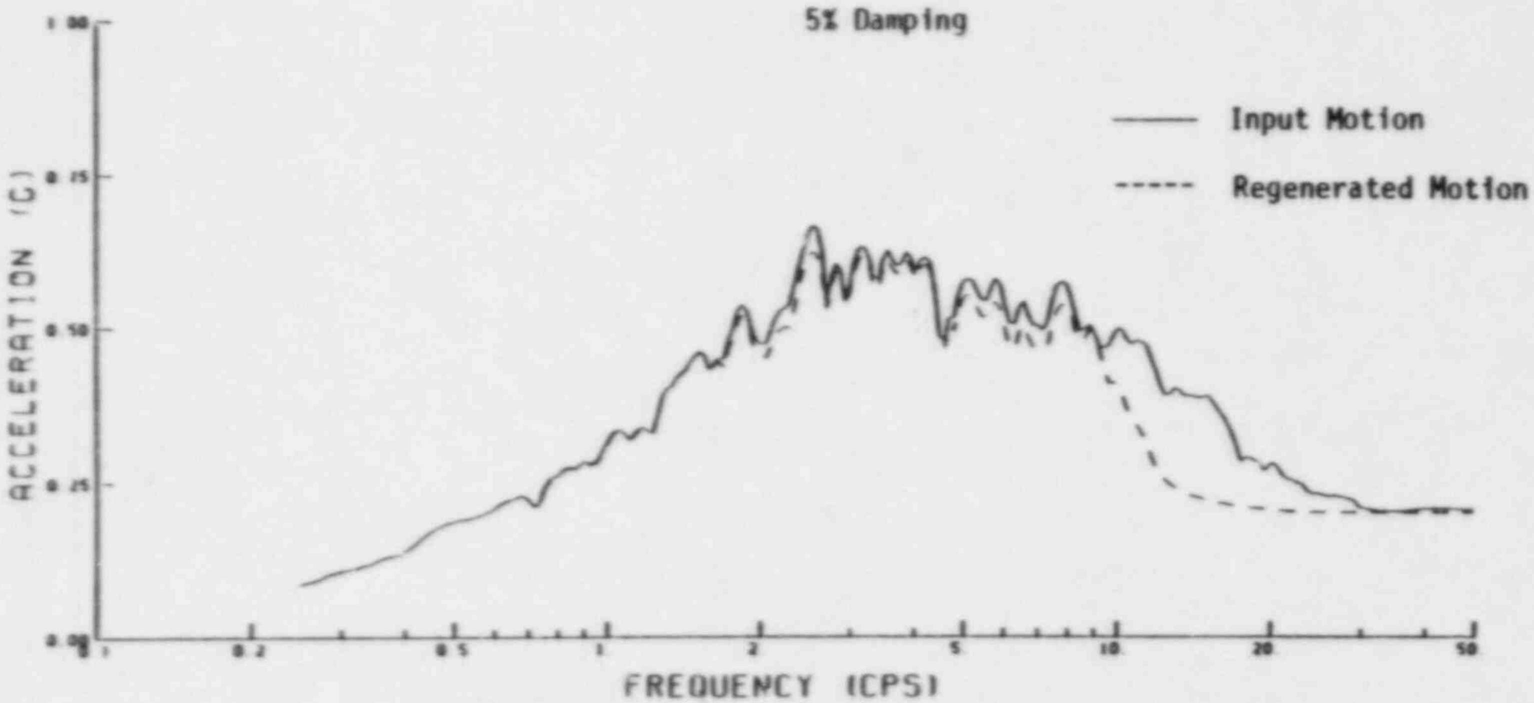
ADDITIONAL INFORMATION REQUESTED: Provide the same comparison for 5% damping value.

RESPONSE: The attached two figures provide the comparison between the response spectra of the defined input motion and regenerated response at the basemat elevation. These spectra were generated for 5% damping. Figures 1 and 2 show the comparison for the SSE and OBE events, respectively. The spectra for the input motion at the basemat level is obtained from section 3.7.1.2 of the Hope Creek FSAR.

Comparison of the response spectra for the input motion versus the R.G. 1.60 spectra is provided in response to NRC question 220.20.

A 12 Hz. cutoff frequency has been used in these analyses. As observed from the attached figures, the match between the two spectra are adequate below the 12 Hz. cutoff frequency. The adequacy of the 12 Hz. cutoff frequency is addressed in a separate response to Question A-12 from the audit meeting on January 11, 1984.

Revised Response
January 10/A-3

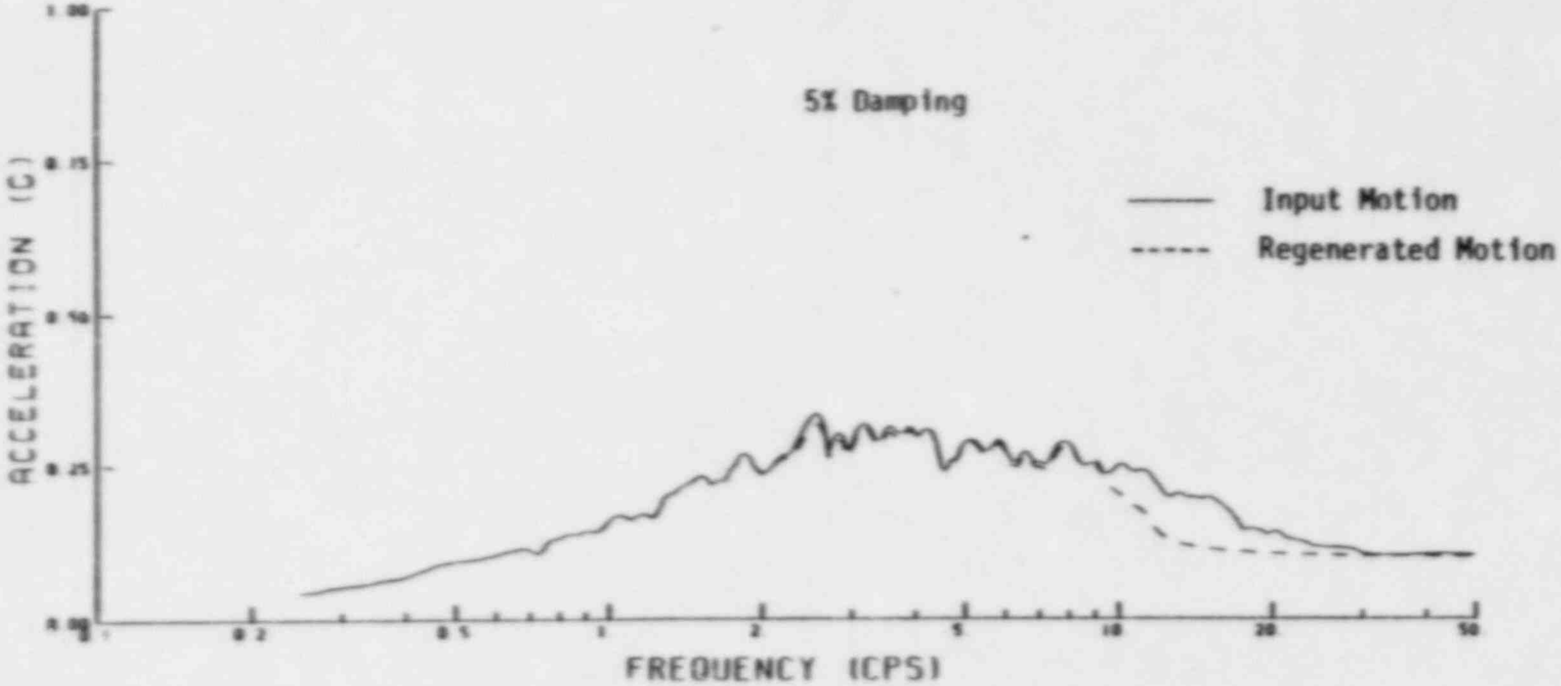


SSE CASE - AVERAGE SOIL PROPERTIES
COMPARISON OF RECOVERED VS. INPUT MOTIONS
ELEVATION 54.0 FT., FREE FIELD

HOPE CREEK
SOIL STRUCTURE INTERACTION

Figure 1

Revised Response
January 10/A-3



OBE CASE - AVERAGE SOIL PROPERTIES
COMPARISON OF RECOVERED VS. INPUT MOTIONS
ELEVATION 54.0 FT., FREE FIELD

HOPE CREEK
SOIL STRUCTURE INTERACTION

Figure 2

Response to NRC Audit Meeting

Date: January 10, 1984

Question no.: A-4

Revised Response
Revision 1
6/30/84

DRAFT

<u>QUESTION</u>: Describe method of establishing rocking time histories R(t).

<u>RESPONSE</u>: Method is described in original response to this question.

ADDITIONAL INFORMATION <u>REQUESTED</u>: Provide mass participation factors for the first few modes of the structure, including the dummy modes for rocking. Also provide a comparison of the response spectra for the input versus the response rocking time histories at the location of the dummy large rotational mass.

<u>RESPONSE</u>: Table 1 of the attachment to this response summarizes the mass participation factors for the first 10 modes of the Reactor Building, Unit 1. Note that as indicated in the original response, mode 1 (period = 200 sec.) corresponds to the dummy rocking mode about E-W axis and mode 2 (period = 150 sec.) corresponds to the dummy rocking mode about N-S axis.

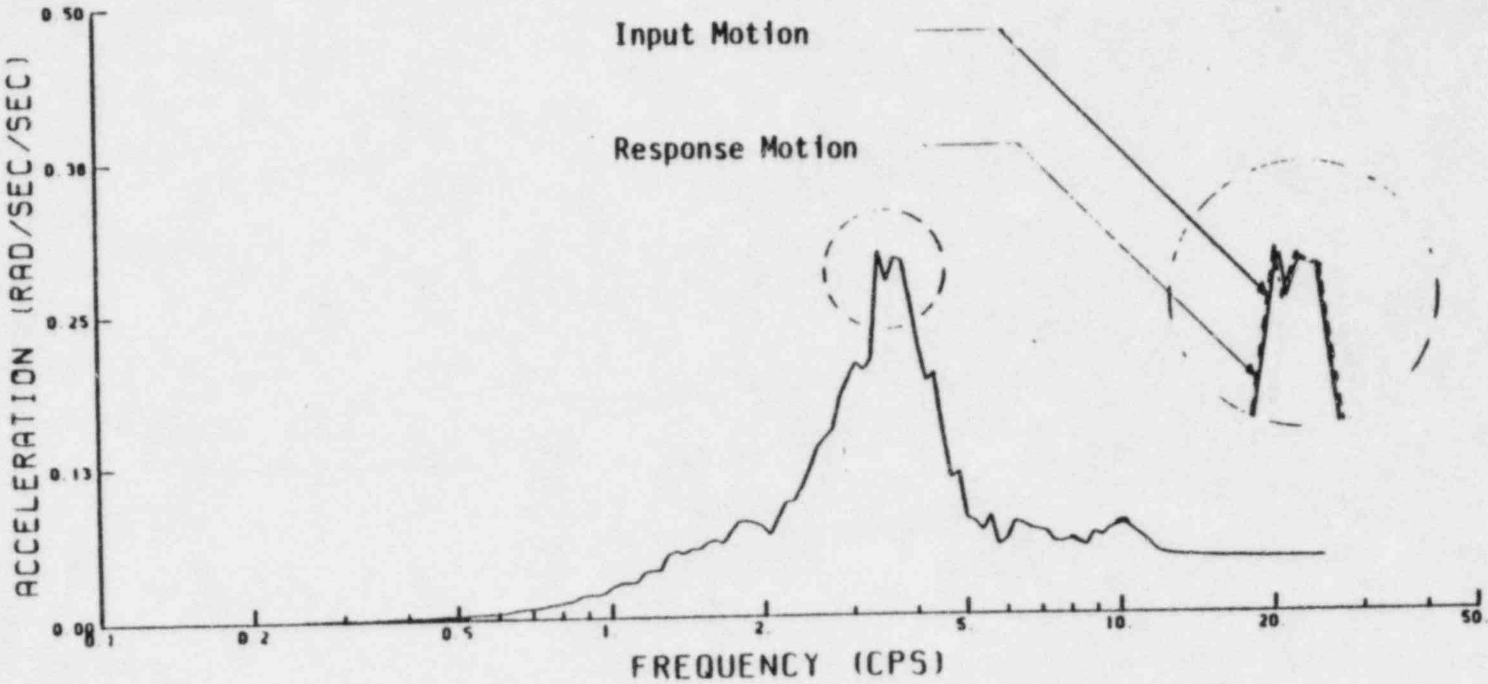
It is observed that for the x-direction earthquake (i.e., N-S translation and rocking about E-W axis) the mass participation of mode 1 is neglible compared to true structural modes (about an order of 10^{-5} lower). Similarly for the y-direction earthquake (i.e., E-W translation and rocking about N-S axis) the mass participation of mode 2 is considerably lower than other modes. Therefore, these dummy modes do not participate in the actual response of the structure.

Furthermore, to verify this point a comparison of the response spectra of input versus response time histories at the location of the dummy large rotational mass point is provided for rocking motion about the E-W axis. As the two response spectra are identical at all frequency points, it is concluded that the inclusion of these dummy modes has no influence on the response of the structure.

TABLE 1

Mode Number	Frequency (CPS)	Mass Participation Factor X	Y
1	.005	- 6.8925E-08	1.2559E-01
2	.007	2.2572E-01	- 1.8687E-07
3	2.731	- 6.7145E+00	- 1.7198E+01
4	4.100	6.5590E+01	1.3645E+01
5	4.226	- 2.2965E+00	4.4283E+01
6	4.346	- 1.9056E-01	2.2708E+00
7	4.414	1.8184E+00	- 5.3500E+01
8	7.092	- 8.7972E+00	- 2.7781E-01
9	7.103	9.2005E-01	6.3203E+00
10	8.821	1.8346E+01	- 3.2680E+01

Revised Response
January 10/A-4



BASEMAT ELEV. 54.0 FT.
ROCKING MOTION ABOUT EW AXIS
REACTOR BUILDING - SSE CASE

HOPE CREEK
SOIL STRUCTURE INTERACTION

FIGURE 1

Response to NRC Audit

Meeting Date: January 10, 1984

Revised Response
Revision 1
July 10, 1984

DRAFT

Question No.: A-11

QUESTION: Justify why it is acceptable to use gross concrete section.

RESPONSE:

In determination of the seismic response of Hope Creek Category I structures, gross (uncracked) concrete sections have been used when calculating the stiffnesses of the concrete structural elements. The use of gross section properties in this application is judged to be reasonable and appropriate based on the following:

1. ACI SP-60 Recommendations

   The use of gross concrete section properties is consistent with ACI recommendations. ACI publication SP-60 (Reference A-11-1), which addresses response of structures to vibratory loads, recommends neglecting cracking and basing section properties on the gross section, but neglecting the transformed area of reinforcing steel. This approach was used in the Hope Creek stiffness calculations.

2. Evaluation of Crack Potential under Seismic Loading

   Major Category I Buildings in the Hope Creek Plant are constructed with reinforced concrete shear walls. An evaluation was performed to assess whether gross cracking of Hope Creek shear walls could occur during the postulated seismic event. The lower elevations of the Reactor Building (approximate el. 54'), where the shear stresses are the maximum, were selected for evaluation. The following parameters were used in the evaluation when calculating the shear wall concrete strengths (Vc):

   - Concrete compressive strength determined in the 90 day cylinder tests was used.

   - ACI 349-76 code criteria (Equations 11.32 and 11.33) were used to establish the cracking concrete shear strength of the walls.

   The evaluation included calculation of the maximum seismic shear and flexural stresses in the shear walls. The cracking concrete shear strength was based on ACI code equation 11.33 which considers both flexural tension and shear and equation 11.32 which considers the diagonal tension shear cracking. The predicted seismically induced shear stresses were lower than the allowable concrete cracking shear strength specified by the ACI code. Thus, it is concluded that the shear walls which contribute most of the lateral stiffness are not expected to have gross cracking due to in-plane seismic loads.

3. Effects of Other Loads (such as LOCA)

Concrete cracking is likely to occur in elements subject to high tensile or shear stress. Typically, this may occur in reinforced concrete containment structures required to resist high LOCA pressures, and in localized compartments outside of containment, due to local abnormal loads such as pipe breaks.

Hope Creek uses a Mark I containment system which consists of a steel containment shell which is separated from the concrete shielding walls, and a steel torus suppression chamber. Since the steel containment is designed to resist the LOCA loads, the shielding walls will not be subjected to LOCA pressure loads.

Abnormal loads (due to pipe breaks) in local compartments may induce out-of-plane flexural stresses large enough to crack concrete. However, the cracking would be limited to the few walls and/or slabs in the vicinity of these loads. Furthermore, the localized cracking would not extend through the wall thickness. The associated reduction in stiffness in a few elements would not significantly affect the overall stiffness or response of the entire structure. Furthermore, the conservative response spectrum broadening criteria (FSAR Section 3.7.2.5) used by Hope Creek will accommodate any minor shift in frequency resulting from local concrete cracking.

4. Effect on Global Inertia Force

The seismic responses of Category I structures including soil-structure interaction effect have been calculated using the finite element method and independently verified by those obtained from the impedance approach (half-space) analysis. The fundamental soil-structure interaction frequencies of the Category I structures computed using the finite element method range from approximately 3.5 Hz to 6 Hz. These fundamental soil structure interaction frequencies are located within the "flat" portion of the NRC design spectra (Regulatory Guide 1.60) for horizontal (2.5 Hz to 9.0 Hz) and vertical (3.5 Hz to 9.0 Hz) earthquakes. If gross concrete cracking is postulated, the fundamental frequencies of the soil-structure system are likely to decrease even if the potential increase in structural damping due to cracking is not taken into consideration. Consequently, the global inertia forces will not change significantly. The same conclusion applies if the seismic responses of Category I structures are computed using the impedance approach. Therefore, it is concluded that the change in inertia force due to frequency shift as a result of concrete cracking will not adversely affect the structural design.

CONCLUSION

Based on the above, it is concluded that the use of gross concrete section properties in the Hope Creek seismic analysis is in conformance with industry codes and practices. A review of the plant structures found that seismically induced cracking would be minor and localized and would not be sufficient to affect the gross seismic response of the structures. Thus the use of gross concrete sections is reasonable and appropriate.

REFERENCE: A-11-1, "Vibration of Concrete Structures", Publication SP-60, American Concrete Institute, Paper SP 60-12.

Response to NRC Audit Meeting

Date: January 10, 1984

Question No.: A-12

Revised Response
Revision 1
6/30/84

DRAFT

QUESTION: Pick one particular floor to define why particular modes were selected for development of vertical floor flexibility response spectra.

RESPONSE: All significant modes below 25.0 Hz were selected from the finite element model of the floor slab for development of vertical floor flexibility response spectra. In general, three to five modes were included with each mode representing a particular region of the floor slab.

For the Reactor Building floor slab at elevation 145.0 feet, five modes were used. Based on the mode shapes of these modes, these modes were determined to represent five different regions of the floor slab, as shown in Figure 1. These five modes were represented by five single-degree-of-freedom beam elements in the vertical floor flexibility model shown in Figure 2.

ADDITIONAL INFORMATION REQUESTED: Provide the frequencies and mode shapes used to develop vertical floor flexibility spectra for elevation 145 feet of the Reactor Building.

RESPONSE: Figure 1 shows the finite element model of the Reactor Building floor at elevation 145 feet. Regions where the response of each mode is dominant is also shown in Figure 1.

The frequencies corresponding to these five modes are given in Table 1. The inclusion of the above modes in the vertical floor flexibility model were considered as a sufficient representation of the dynamic characteristics of the floor slab for this elevation. Examining the mass participation factors, among these 5 modes 92% of the total mass of the floor is represented.

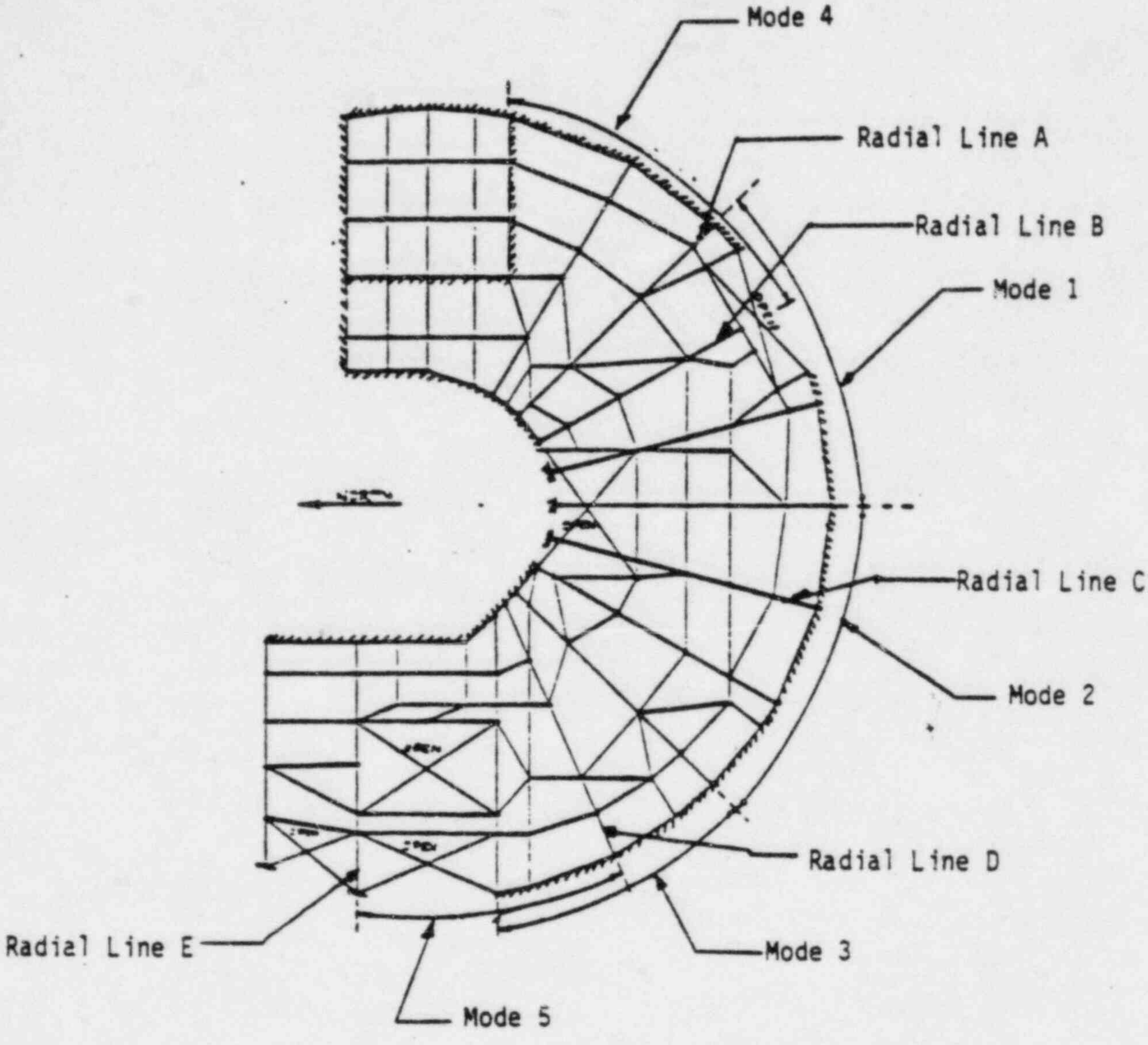
Plots of the five mode shapes along selected radial lines of the floor slab (see Figure 1) are given in Figures 3a, b, and c. Based on these mode shapes, regions 1 through 5 were identified on the floor slab.

TABLE 1

Floor Slab Frequencies and Mass Participation

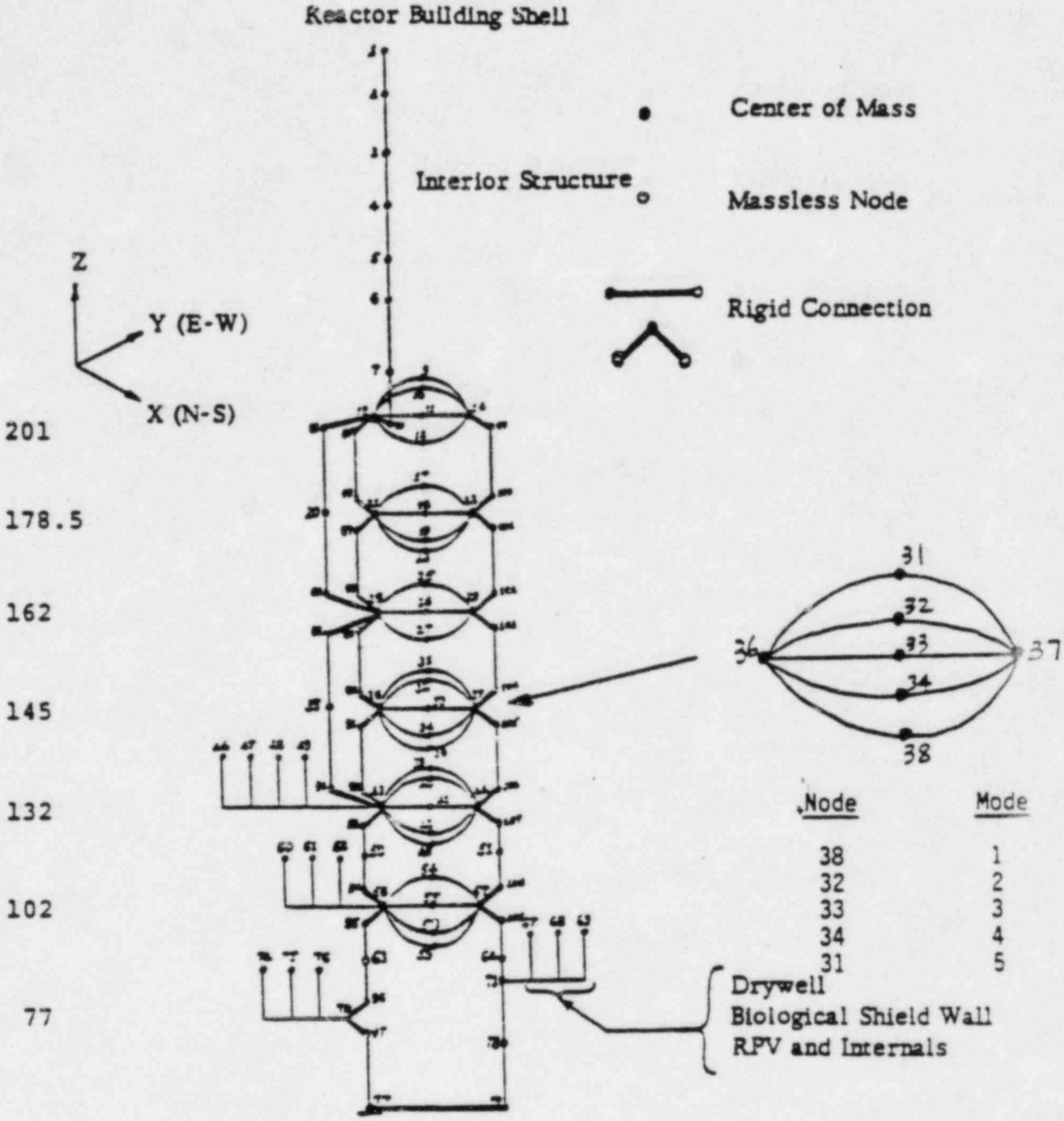
Mode No.	Frequency (Hz.)	Mass Participation Factor	% Total Mass*
1	6.31	6.98	33.9
2	7.65	7.34	37.5
3	8.53	4.01	11.2
4	11.58	2.86	5.7
5	13.27	2.42	4.1
		TOTAL	92.4%

* Total Mass = 143.68 kips $\frac{\text{-sec}^2}{\text{ft.}}$



REACTOR BUILDING VERTICAL FLEXIBILITY ANALYSIS
FINITE ELEMENT PLATE AND BEAM MODEL
FLOOR ELEVATION 145.0 FT.

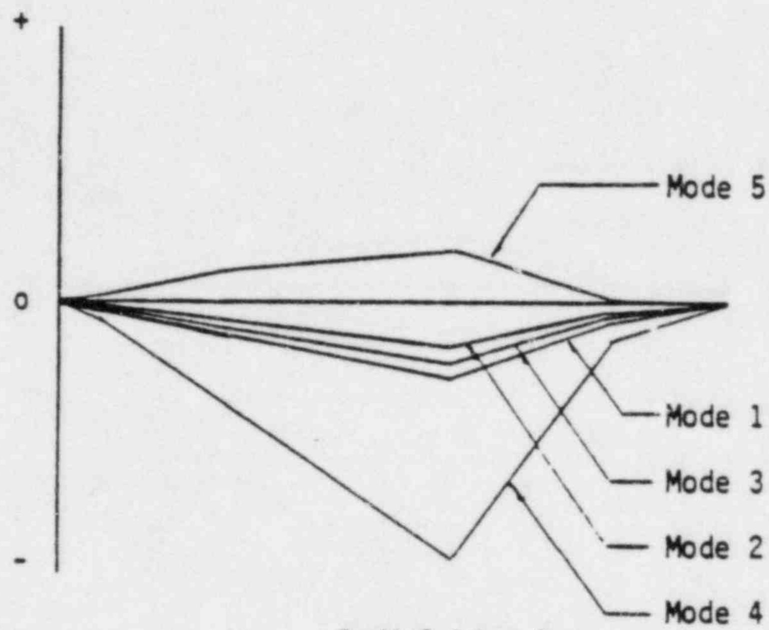
FIGURE 1



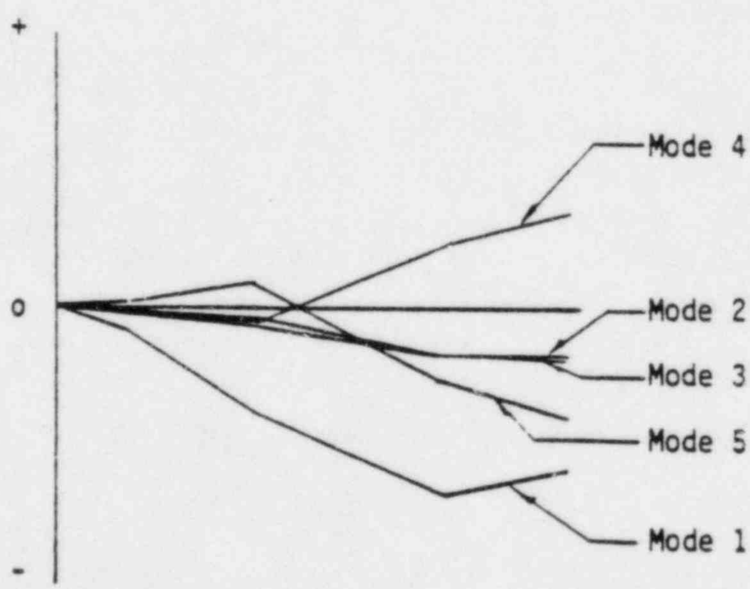
REACTOR BUILDING VERTICAL FLEXIBILITY ANALYSIS
MATHEMATICAL MODEL

FIGURE 2

Figure 3a

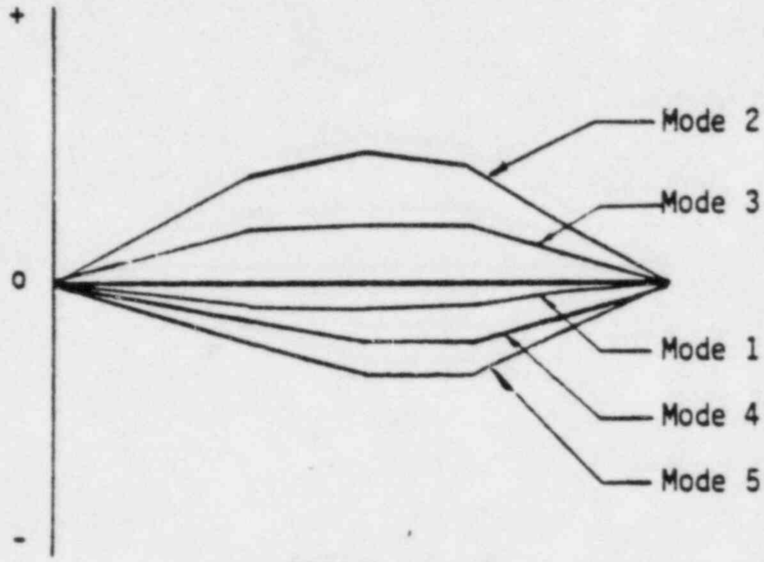


Radial Line A

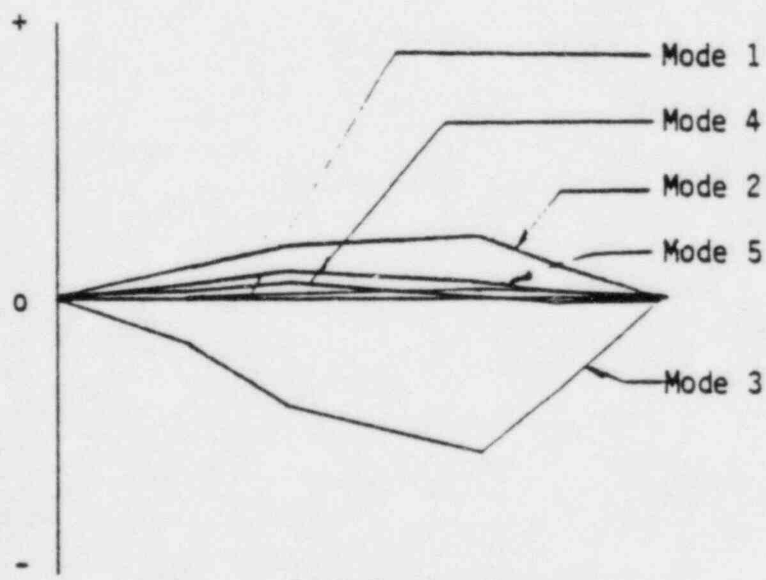


Radial Line B

Figure 3b

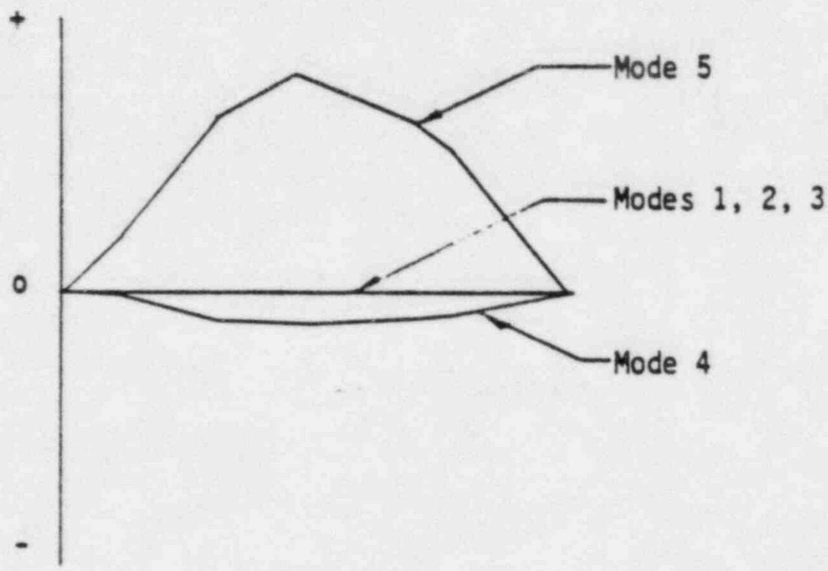


Radial Line C



Radial Line D

Figure 3c



Radial Line E

Meeting Date: January 10, 1984 **DRAFT** Revised Response Revision 1 7/2/84

Question No: A-13

Question: Provide comparison of Bechtel Independent Verification Results with the Design Basis Results.

Response:

As described in Amendment 1 of the FSAR (Section 3.7.2.4), three independent seismic soil-structure interaction analyses are performed for the major plant structures. The design basis analyses are performed using the finite element method by EDS Nuclear, Inc. (presently known as Impell Corporation). Independent finite element soil-structure interaction analyses are subsequently performed by Bechtel to verify the design basis analyses. In addition, in accordance with the requirements of the Standard Review Plan, Section 3.7.2 (NUREG 0800), impedance approach (the half-space) soil-structure interaction analyses are performed by Bechtel. The analytical method utilized for the impedance approach seismic soil-structure interaction analyses of power block structures and service water intake structure is given in FSAR Section 3.7.2.1. Figure A-13-1 summarizes the division of responsibilities for the seismic analyses.

Figures A-13-2 to A-13-37 show the comparison of the response spectra (2% damping) obtained from the above three seismic soil-structure interaction analyses. Discussions of these comparisons are as follows:

Power Block Structures

I. Comparison of Design basis and Independent Finite Element Verification Response Spectra

Bechtel's independent soil-structure interaction analyses are performed using the computer code FLUSH. The results of independent finite element analyses are in reasonable agreement with those of the design basis analyses. As can be seen from Figures A-13-2 through A-13-37, the horizontal response spectra obtained from the independent finite element analyses are generally enveloped by those obtained from the design basis analyses except for the frequency range lower than 2 Hz. The vertical response spectra showed some exceedances at the frequency range of 18 Hz. These exceedances are listed in Table A-13-1.

The effects of these exceedances are evaluated for the combined responses in three directions using the SRSS approach and compared with the design basis results. Table A-13-2 provides these comparisons. In all cases, these variations are judged to be minor and can be accommodated within the design margin. In areas where multimodal analysis is performed, the effects of these variations will be further reduced. It has been concluded that the variations between these two analyses are within the accuracy of analyses and can be accommodated within the design margin.

II. Comparison of Design Basis and Impedance Approach Response Spectra

The peak spectral accelerations obtained from the impedance approach analyses are generally lower than those obtained from the design basis analyses. However, these response spectra are not completely enveloped by those obtained from the design basis analysis, especially in the frequency range between 1.0 and 3.5 Hz. Also, there are some local exceedances in the higher frequency range, as shown in Figures A-13-2 through A-13-37.

As discussed during the NRC Structural Design Audit, dated January 10, 1984, sampling studies have been performed to confirm the adequacy of the plant design. Table A-13-3 describes the criteria used in selection of the samples for this study.

The results of sampling studies are as follows:

1. Structures

   All major reinforced concrete shear walls at the base of the reactor building have been evaluated for seismic forces and moments obtained from the impedance approach analyses. The actual shear stresses resulting from the impedance approach analyses were evaluated and found to be lower than the design basis stresses. Table A-13-4 provides the comparision of shear stresses at El. 54'-0. Tables A-13-5a and A-13-5b show the comparision of impedance approach and design basis moments for OBE and SSE cases respectively. The impedance approach moments exceeds the design basis moments at a few wall locations as indentified on Tables A-13-5a and A-13-5b. These walls were reevaluated and the resulting moments were found to be less than the allowables.

   The auxiliary building seismic forces and moments obtained from the impedance approach analysis are less than the design basis shears and moments. Therefore, no further evaluation of the auxiliary building structure is necessary.

   Based on the above, it is concluded that the as-built power block structures can accommodate the loads obtained from the impedance approach analysis.

2. Equipment

   The effects of the impedance approach response spectra was evaluated on 26 types of equipment. The selected items are located in the areas where the impedance approach

2. (Cont'd)

spectra were found to have higher spectral accelerations than those of the design basis response spectra. Each equipment was evaluated in accordance with the procedure described in Table A-13-3, and the results of the evaluation are summarized in Table A-13-6. In all cases, the as-built equipment designs were found acceptable.

3. Cable Tray and HVAC Supports

   a. Cable Tray Support

   Approximately 200 supports were evaluated. In all cases, the existing designs were determined to be acceptable.

   b. HVAC Supports

   Over 200 supports were evaluated. In all cases, it was found that the design basis spectral accelerations exceeded the impedance approach spectral accelerations for the support frequencies. Therefore, the HVAC supports were considered acceptable.

4. Piping and Pipe Supports

A total of 10 representative piping system calculations were selected out of 64 calculations affected by the impedance approach analysis results. The selection of these calculations was based on the criteria given in Table A-13-3.

The objective of performing detailed dynamic seismic analysis of the sample calculation was to demonstrate that although the design basis curve did not envelop the impedance curves in the low frequency range, such deviation do not have any affect on the adequacy of existing piping analysis and support design. In other words, the stresses and loads generated using the impedance response spectra curve as input are still within the ASME Section III code allowable for pipe and pipe support design.

The methodology used for evaluation was to subject the selected existing mathematical models of piping systems to the impedance approach response spectra and to compare the resulting pipe stresses with the ASME Section III code allowables for pipe and pipe support design. The reactions at equipment nozzles were compared with vendor's design allowables. All pipe supports were evaluated for adequacy under the revised loads.

In all cases, the pipe stresses were found to be within the code allowables as shown in Table A-13-7. Also, as illustrated in Table A-13-7, the equipment nozzle allowables were also met. The existing pipe support designs were also found adequate for the new loads and met the ASME Section III code Subsection NF allowables. This is illustrated in Table A-13-8.

Intake Structure

See responses to questions A-14 and A-16, meeting date January 11, 1984.

Table A-13-1

Comparison of Design basis and Independent Finite Element Verification Response Spectra

Building	Key Elevation	Design Earthquake	Earthquake Direction	Locations of Variations (Note 1)	Figure No.	Item No.	Spectral Acceleration (Note 2) Design Basis (g)	Bechtel FLUSH (g)
REACTOR	102	SSE	N-S	1.8 Hz	A-13-3	1	0.62	0.75
	201	SSE	N-S	1.8 Hz	A-13-4	2	1.00	1.22
	54	SSE	Vertical	18.5 Hz	A-13-8	3	1.50	1.75
	102	SSE	Vertical	22.0 Hz	A-13-9	4	1.35	1.68
	201	SSE	Vertical	18.0 Hz	A-13-10	5	2.15	2.45
AUXILIARY	54	SSE	N-S	3.6 Hz	A-13-11	6	1.34	1.56
	54	SSE	E-W	3.0 Hz	A-13-14	7	0.88	1.44
	102	SSE	E-W	3.0 Hz	A-13-15	8	1.10	1.68
	178	SSE	E-W	3.2 Hz	A-13-16	9	1.40	1.92
	102	SSE	Vertical	14.0 Hz	A-13-18	10	1.83	1.95
	178	SSE	Vertical	22.0 Hz	A-13-19	11	1.53	1.85

Table A-13-1 (Cont'd)

Comparison of Design Basis and Independent Finite Element Verification Response Spectra

Building	Key Elevation	Design Earthquake	Earthquake Direction	Locations of Variations (Note 1)	Figure No.	Item No.	Spectral Acceleration (Note 2)	
							Design Basis (g)	Bechtel FLUSH (g)
REACTOR	102	OBE	N-S	1.7 Hz	A-13-21	12	0.34	0.42
	54	OBE	E-W	4.3 Hz	A-13-23	13	0.50	0.67
	201	OBE	E-W	1.8 Hz	A-13-25	14	0.38	0.55
	102	OBE	Vertical	22.0 Hz	A-13-27	15	1.20	1.42
	201	OBE	Vertical	18.0 Hz	A-13-28	16	1.68	1.85
AUXILIARY	54	OBE	N-S	4.9 Hz	A-13-29	17	1.15	1.40
	54	OBE	E-W	4.4 Hz	A-13-32	18	0.75	0.85
	54	OBE	Vertical	22.0 Hz	A-13-35	19	1.17	1.26
	102	OBE	Vertical	18.0 Hz	A-13-37	20	1.47	1.54
	178	OBE	Vertical	18.0 Hz	A-13-37	21	1.80	1.95

NOTES:

1. This column identifies those locations where the results of the independent analysis exceed those of the design basis analysis.

2. For vertical earthquake direction, spectral acceleration includes the effect of gravity load (1.0 g).

Table A-13-2

SRSS Spectral Acceleration Comparison between Design Basis and Finite Element Verification Analysis

Item No.	SRSS Spectral Acceleration Comparison(g) (Note 1)		
	(A) Design Basis	(B) Bechtel-FLUSH	(B-A)/A Difference (%)
1	1.97	1.75	-11
2	2.24	2.20	-2
3	1.53	1.78	16
4	1.39	1.72	24
5	2.23	2.49	12
6	2.86	2.68	-6
7	2.34	2.32	-1
8	2.56	2.48	-3
9	4.27	3.44	-19
10	1.87	1.93	4
11	1.73	1.93	11
12	1.41	1.38	-2
13	2.02	1.66	-18
14	1.52	1.50	-1
15	1.21	1.43	18
16	1.71	1.86	9
17	2.24	2.07	-8
18	2.23	1.94	-13
19	1.19	1.27	7
20	1.86	1.99	7
21	1.51	1.56	3

NOTE: 1. The SRSS spectral acceleration values include the effect of gravity loads (1.0 g)

TABLE A-13-3
PROCEDURES FOR EVALUATION OF STRUCTURES, EQUIPMENT & COMPONENTS USING IMPEDANCE ANALYSIS RESULTS

INTRODUCTION

The results of the impedance analysis are used to assess the existing design of the HCGS structures, equipment and components. A sampling approach is used. The procedure for this evaluation is as follows:

A. STRUCTURES:

Since the maximum shear and axial forces and the maximum overturning moments occur at the base of the structures, and the design margins for the upper elevations are greater than those of the base, the effects of these loads at the base of each structure are evaluated.

B. EQUIPMENT:

The impedance analysis spectra in general are not completely enveloped by the design basis spectra in the following areas,

i) 1.0 to 3.5 Hz range throughout the reactor and auxiliary buildings

ii.) 6 to 15 Hz range in the reactor building at elevation 102 ft and below.

iii.) 6 to 15 Hz in the auxiliary building at elevation 54 ft.

Since typical equipment frequencies are not found in the range of 1.0 to 3.5 Hz, the item (i) above does not need any further evaluation. Items (ii) and (iii) are reconciled as follows:

- Review the significant frequencies of approximately 30% of all equipment selected at random and located in the areas where spectral variations were noted.

- If the significant equipment frequencies fall in the range where the difference in the spectra exist, additional evaluation is necessary. No further evaluation is necessary if the significant frequencies are outside the frequency range in question.

- The evaluation is performed either by comparing the test response spectra of the equipment with the impedance spectra (if the equipment is qualified by testing) or comparing the actual-to-allowable stress ratios with the spectrum exceedance ratios.

- If the above evaluation shows the equipment may not be qualified for the impedance spectra, detailed evaluation consisting of analysis and/or testing is performed.

- As a result of evaluation, if equipment requires modifications, the sample size for this evaluation is expanded as required.

C. CABLE TRAY AND HVAC SUPPORTS

Cable tray and HVAC supports do not have frequencies in the range of 1.0 to 3.5 Hz. Therefore any differences between the two spectra in this frequency range do not require any evaluation.

The effects of the spectrum exceedances at frequency range between 6 and 15 Hz are evaluated for approximately 200 cable tray and HVAC supports. These supports are selected at random but are located at the lower elevation (Reactor Building El. 54 to 102 ft., Auxiliary Building El. 54 ft.) where the spectrum differences exist. If the results of evaluation indicate need for modifications to any support, the sample size for this evaluation is expanded as required.

D. PIPING AND PIPE SUPPORTS

In general, impedance curves resulted in significant reductions in response spectrum peak accelerations as compared to those of the design basis curves. However, frequency shifts were observed in some curves, particularly in the low frequency ranges. To evaluate the effects of the frequency shift, a "biased" sample of affected piping systems is reanalyzed and reevaluated. The sample is selected as follows:

Individual impedance curves for various elevations and structures are superimposed on their corresponding design basis curves to identify those impedance curves which are not enveloped by design basis curves. Those impedance curves are then superimposed on the design basis "enveloped" response spectra used for various piping system design calculations. If the design basis enveloped response spectra curves affecting a calculation did not totally envelop all the corresponding impedance curves, that particular calculation is then identified as "affected" and a candidate for sampling.

A "biased" sample of the "affected" calculations was selected which emphasized the following important piping parameters:

1. Stress levels in the existing pipe stress calculations. Samples included systems with high stress levels.

2. Difference in "g" level (Δg) between impedance and design basis curves in the affected frequency zones. Sample selected to include curves showing significant differences.

3. High equipment nozzle loads in existing calculation.

4. Relative location of piping system in the plant in an attempt to include response of all structures in the sample selected.

The number of calculations included in the sample is:

Building	Total No. of Q-Calcs	No. of Calcs Reviewed	No. of Calcs affected	No. of Calcs in the sample
Drywell	32	32	23	3
Reactor	213	213	34	5
Auxiliary	124	124	7	2

Results of the analysis including support loads are compared against the design basis values for acceptability.

TABLE A-13-4

REACTOR BUILDING SHEAR STRESSES AT EL. 54'-0"

Wall Location	Design Basis Psi	Impedance Approach Psi	Allowable Psi
North Wall	323	207	630
South Wall	333	224	630
East Wall	298	261	630
West Wall	303	268	630
Cylindrical Shell	257	251	630
Pedestal	27	91	126

SOUTH RADWASTE SHEAR STRESSES AT EL. 54'-0"

Wall Location	Design Basis Psi	Impedance Approach Psi	Allowable Psi
North Wall	183	207	630
South Wall	216	224	630
East Wall	208	276	630
West Wall	458	257	630

Notes: 1. Concrete f'c = 4000 Psi

2. See FSAR Figures 1.2-2 for wall location.

TABLE A-13-5a

REACTOR/RADWASTE BUILDING - OBE SEISMIC MOMENTS AT EL. 54'0"

Wall Location	Design Basis Method (Kip-Ft)	Impedance Approach Method (Kip-Ft)
North-Reactor North-Radwaste	359,200	414,500
South-Reactor South-Radwaste	517,400	847,700
East-Radwaste	461,000	421,900
West-Radwaste	329,000	290,700
East-Reactor	434,500	276,900
West-Reactor	588,600	482,900
Cylindrical Shell	2,772,000 (N-S) 1,723,000 (E-W)	1,847,000 (N-S) 1,639,000 (E-W)

Note: See FSAR Figure 1.2-2 for wall location.

TABLE A-13-5b

REACTOR/RADWASTE BUILDING - SSE SEISMIC MOMENTS AT EL. 54'0"

Wall Location	Design Basis Method (Kip-Ft)	Impedance Approach Method (Kip-Ft)
North-Reactor North-Radwaste	912,100	699,100
South-Reactor South-Radwaste	1,344,000	1,429,000
East-Radwaste	675,000	732,300
West-Radwaste	654,000	504,500
East-Reactor	909,000	480,200
West-Reactor	1,320,000	837,400
Cylindrical Shell	4,471,000 (N-S) 3,054,000 (E-W)	3,092,000 (N-S) 2,668,000 (E-W)

Note: See FSAR Figure 1.2-2 for wall location.

TABLE A-13-6

POWER BLOCK SEISMIC CATEGORY I EQUIPMENT

Equipment or Component	Tag No.	Location Blgd./El.	Equipment Frequencies (Hz)	Method of Seismic Qualification	Applicable Note
HPCI Turbine	E41-C002	Reactor Bldg. El. 54	Horizontal- 10, 12 Vertical - 23	Testing	1
Residual Heat Removal Pump/ Motor	E11-C002	Reactor Bldg. El. 54	Horizontal- 8.7, 9.7 Vertical - >33	Analysis	3
Control Room Panels	H11-P617 H11-P618 H11-P640 H11-P641	Aux. Bldg. El. 102	Horizontal- 11.5, 16 Vertical - >33	Testing	1
Control Room Panels	H11-P620 through H11-P623 H11-P628 H22-P631	Aux. Bldg. El. 102	Horizontal- 21, 29 Vertical - >33	Testing	1
Control Room Panels	H11-P635 H11-P636	Aux. Bldg. El. 137	Horizontal- 19, 37 Vertical - >33	Testing	1
Control Room Panels	H11-608	Aux. Bldg. El. 137	Horizontal- 7, 12 Vertical - >33	Testing	1
Control Room Panels	H11-609 H11-611	Aux. Bldg. El. 137	Horizontal- 22, 37 Vertical - >33	Testing	1
RCIC Turbine	E51-C002	Reactor Bldg. El. 54	Horizontal- 16 Vertical - 18	Analysis & Testing	1, 2
LPCS Pump/ Motor	E21-C001	Reactor Bldg. El. 54	Horizontal- 11.5, 12.7 Vertical - >33	Analysis	2

TABLE A-13-6 (Cont'd)

POWER BLOCK SEISMIC CATEGORY I EQUIPMENT

Equipment or Component	Tag No.	Location Blgd./El.	Equipment Frequencies (Hz)	Method of Seismic Qualification	Applicabl Note
Chiller Water Tank	IAT, BT 410, 413	D. G.* El. 178	Horizontal - >33 Vertical - >33	Analysis	2
ECCS Jockey Pump	IAP, BP, CP, DP 228	Reactor Bldg. El. 54	Horizontal - >33 Vertical - >33	Analysis	2
SACS Expansion Tank	IAT, BT 205	Reactor Bldg. El. 201	Horizontal - 12.5 Vertical - >33	Analysis	2
5.0 Kv Switch-gear	IAN, BN, CN, DN 205	Reactor Bldg. El. 102	Horizontal - 8, 14 Vertical - 30	Testing	1
DC Switchgear & Control Center	IOD 251, 261	Reactor Bldg. EL. 54	Horizontal - 8, 35 Vertical - 20	Testing	1
Batteries Racks	IOD 421, 431	Aux. Bldg. El. 54	Horizontal - 14, 16 Vertical - 28	Testing	1
Inst.AC Power Panel	IYF 401-407 IYF 209	Aux. Bldg. El. 102	Horizontal - 17, 21 Vertical - 6	Testing	1
Control Panel	IAC, BC 201	Reactor Bldg. El. 102	Horizontal - 8, 17 Horizontal - >33	Analysis	2

Note: *D.G. - Diesel generator area of the auxiliary building.

TABLE A-13-6 (Cont'd)

POWER BLOCK SEISMIC CATEGORY I EQUIPMENT

Equipment or Component	Tag No.	Location Blgd./El.	Equipment Frequencies (Hz)	Method of Seismic Qualification	Applicable Note
Standby Diesel Generator Set	1(A-D)G 400	D. G. El. 102	Horizontal - >15 Vertical - >15	Analysis	2
SACS Heat Exchanger	1AIE, 1A2E201 1BIE, 1B2E201	Reactor Bldg. El. 54	Horizontal - 8, 10.4 Vertical - 21	Analysis	2
SACS Pumps	1(A-D)P210	Reactor Bldg. El. 201	Horizontal - >33 Vertical - >33	Analysis	2
Control Panel	ICC, DC201	Reactor Bldg. El. 102	Horizontal- 12.7, 17.6 Vertical - 29	Analysis	2
Accumulator Tank	1AT, BT412	D. G. EL. 54	Horizontal - 31, 33 Vertical - 35	Analysis	2
Air Handling Units A/C Units	1AVH407 1BVH407	D. G. El. 178	Horizontal - 16.6, 18 Vertical - 19	Analysis	2
Unit Cooler	1AVH208 1AVH209 1BVH208 1BVH209	Reactor Bldg. El. 102	Horizontal - 9.4, 21 Vertical - 26.4	Analysis	2
HVAC Control Panels	1AC, CC285 1AC, CC281 1AC, DC483	D. G. El. 178	Horizontal - 12.7, 16.4 Vertical - 16.9	Analysis	2
Centrifugal Water Chiller	1AK, BK403	D. G. El. 178	Horizontal - >30 Vertical - >30	Analysis	2

Notes: 1. TRS envelopes impedance approach spectra.

2. Impedance approach spectral acceleration is lower than that of the design-basis response spectra in the major equipment frequencies.

3. Although impedance approach spectral acceleration exceeds that of design basis response spectra in the equipment frequency range, a more detailed calculation showed that the equipment stresses are within the code allowables.

TABLE A-13-7

POWER BLOCK PIPE STRESS SUMMARY

Building	Calc. No.	Max. Seismic Stress Ratios / Max. Impedance Stress / Max. Design Basis Stress		ASME Code Equation Evaluation		Vendor Equip. Nozzle Allowables Met
				Eq. 9B* Code Allowable	Eq. 9D* Code Allowable	
		OBE	SSE	Upset	Faulted	
Auxiliary	C1549	0.51	0.76	0.29	0.66	YES
	C1581	0.64	0.86	0.40	0.28	YES
Drywell	C118	0.75	0.83	0.44	0.34	YES
	C1842	0.65	0.83	0.63	0.85	YES
	C120	0.30	0.52	0.49	0.39	YES
Reactor	C988	0.88	0.75	0.54	0.35	YES
	C911	0.88	0.94	0.84	0.63	YES
	C963	1.10	1.18	0.71	0.47	YES
	C918	0.29	0.39	0.33	0.21	YES
	C937	0.90	1.15	0.70	0.38	YES

*ASME Section III NC, ND-3652

TABLE A-13-8

POWER BLOCK PIPE SUPPORT LOAD SUMMARY

Building	Calc. No.	Total No. of Supports	No. of Supports with Load Increase	Average Percentage increase in Load		Support Design Adequate
				Upset	Faulted	
Auxiliary	C1549	5	0	N/A	N/A	YES
	C1581	16	6	11%	NONE	YES
Drywell	C118	8	1	2%	1%	YES
	C1842	34	0	N/A	N/A	YES
	C120	18	2	7%	NONE	YES
Reactor	C988	11	3	NONE	14%	YES
	C911	34	6	20%	17%	YES
	C963	7	4	27%	28%	YES
	C918	10	0	N/A	N/A	YES
	C937	17	5	17%	21%	YES

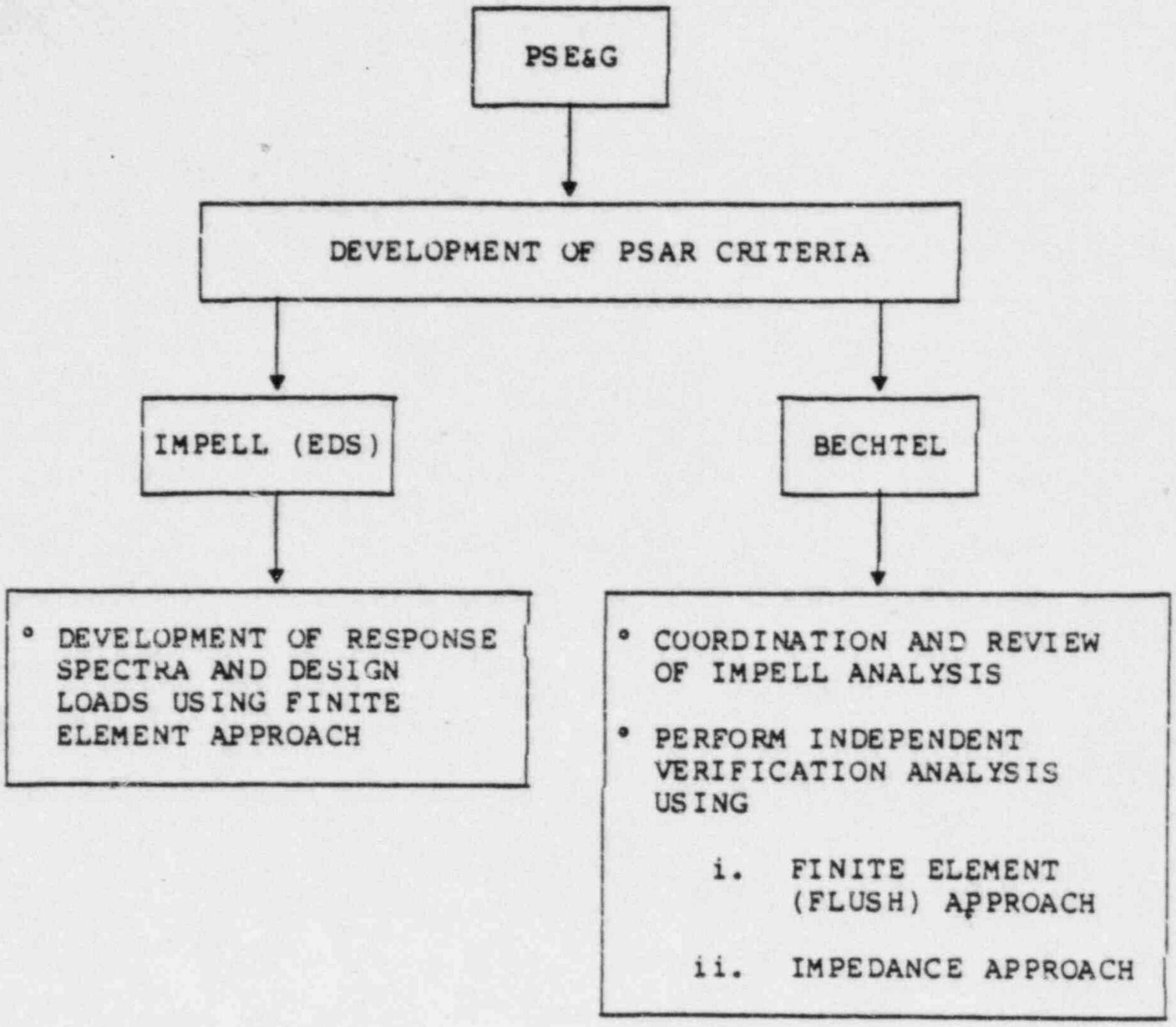


Figure A-13-1

Division of Responsibility

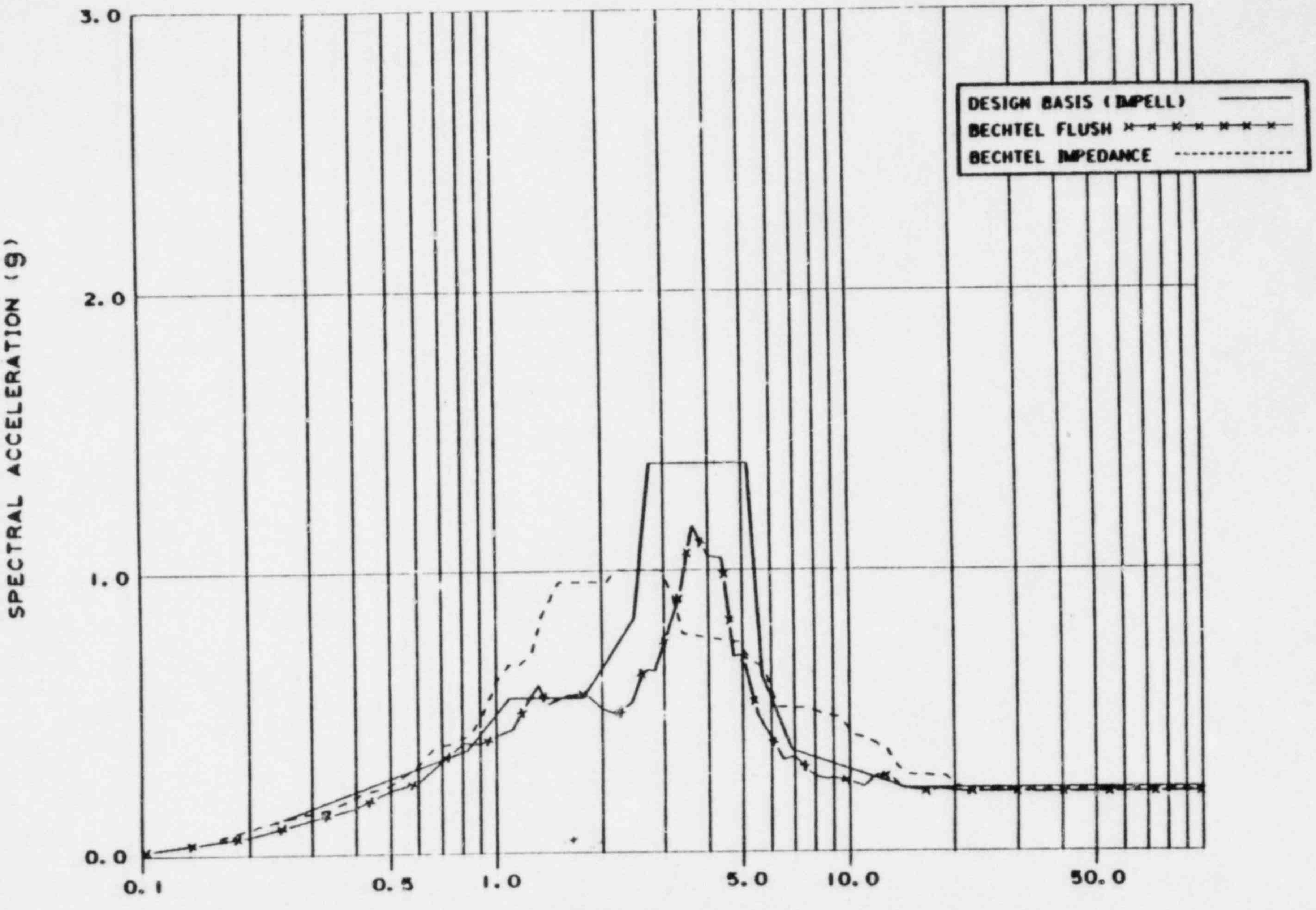


FIGURE A-13-2

RESPONSE SPECTRA COMPARISON,
UNIT I REACTOR BUILDING AT ELEV. 54'-0",
N-S SSE 2% DAMPING

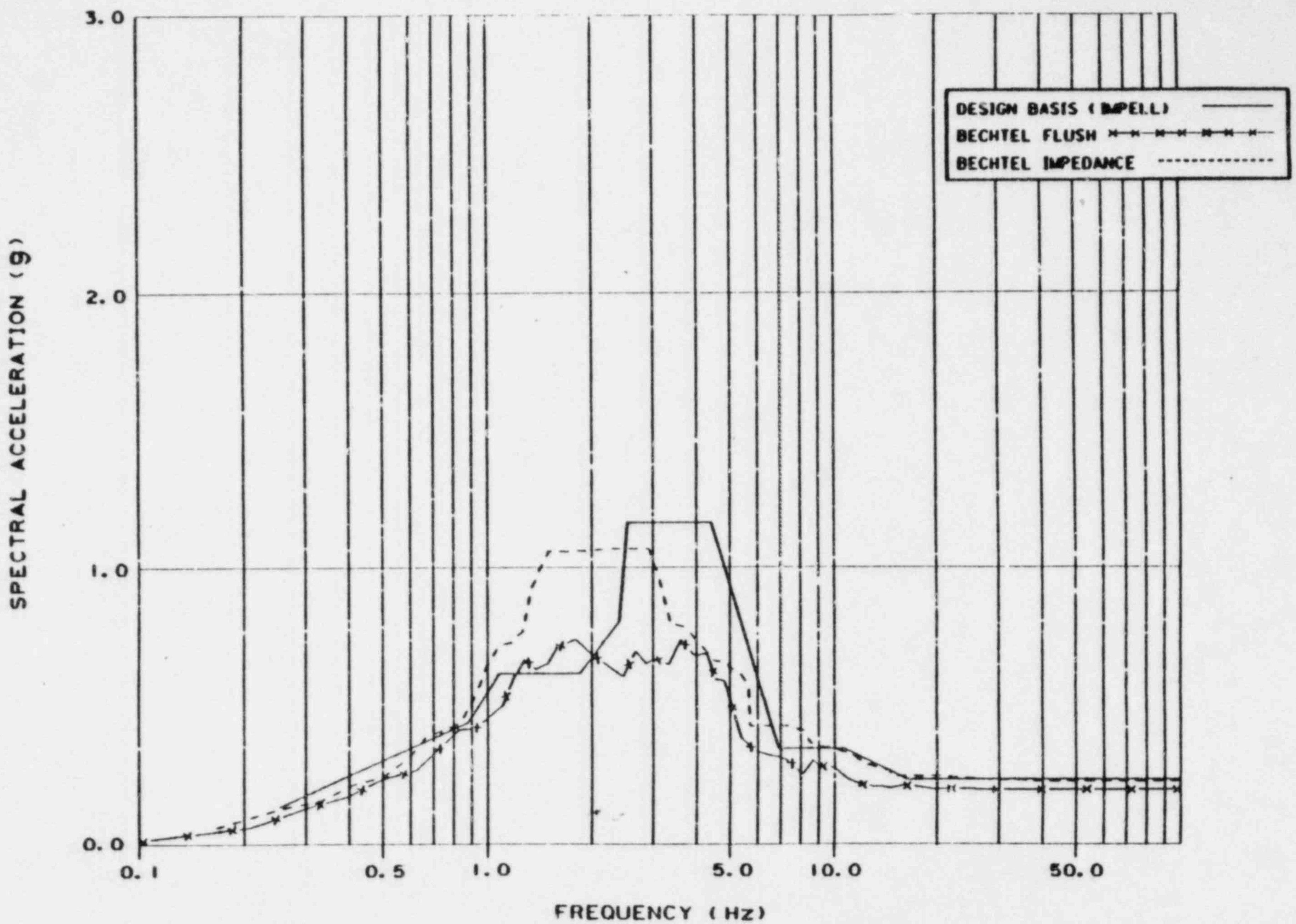


FIGURE A-13-3

RESPONSE SPECTRA COMPARISON,
UNIT 1 REACTOR BUILDING AT ELEV. 102' -0",

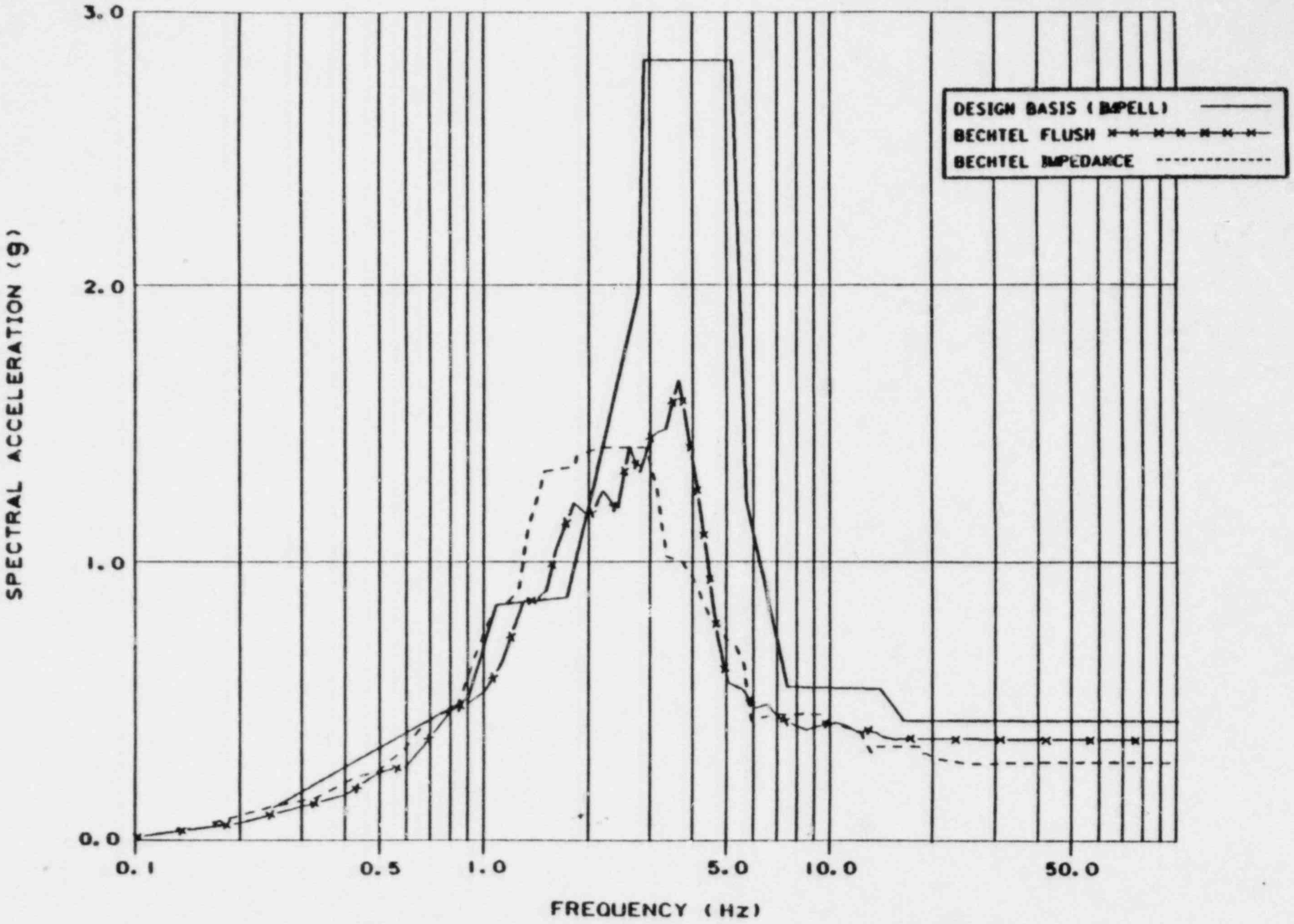


FIGURE A-13-4

RESPONSE SPECTRA COMPARISON,
UNIT I REACTOR BUILDING AT ELEV. 201'-0',

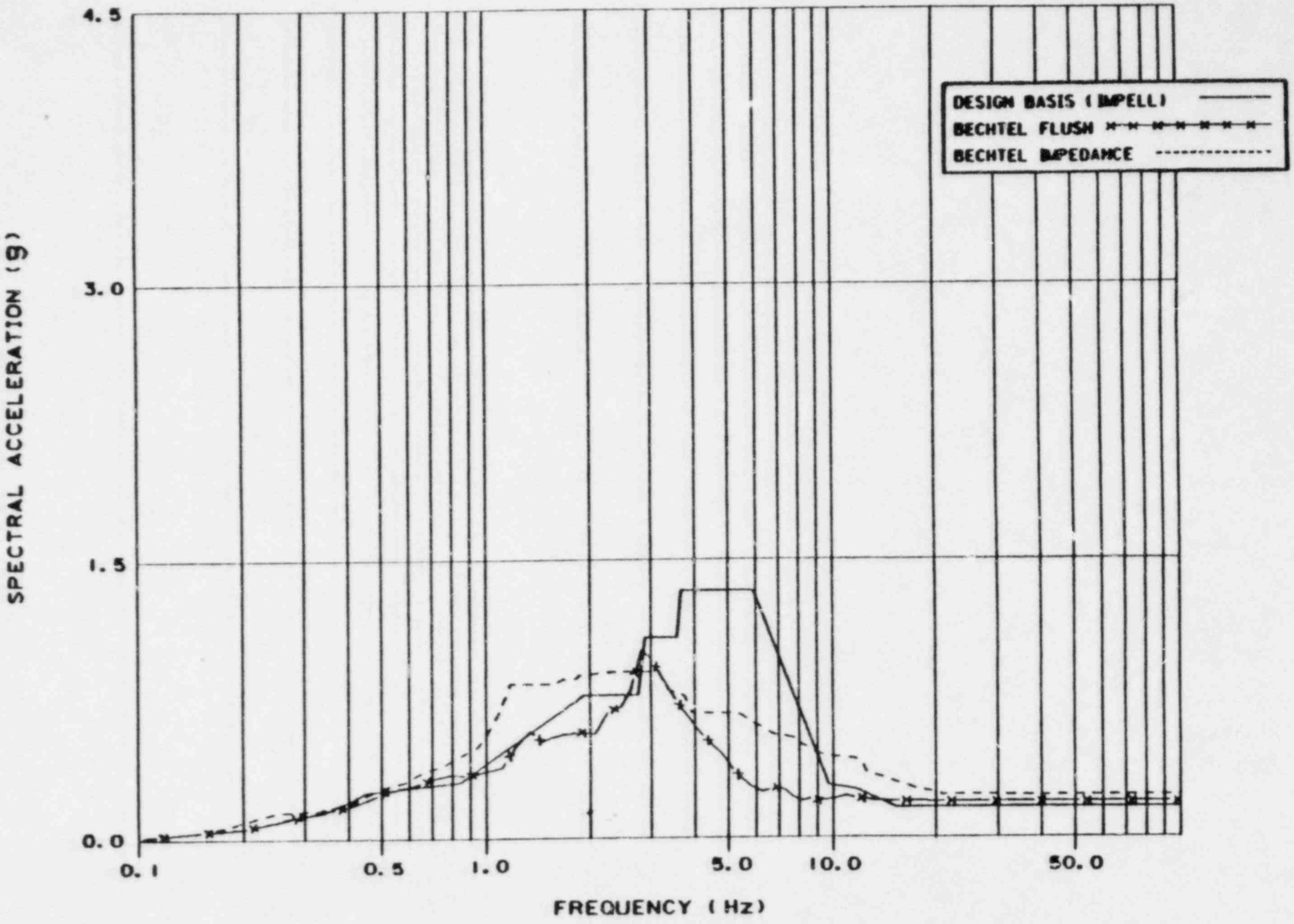


FIGURE I.-13-5

RESPONSE SPECTRA COMPARISON,
UNIT I REACTOR BUILDING AT ELEV. 54'-0".

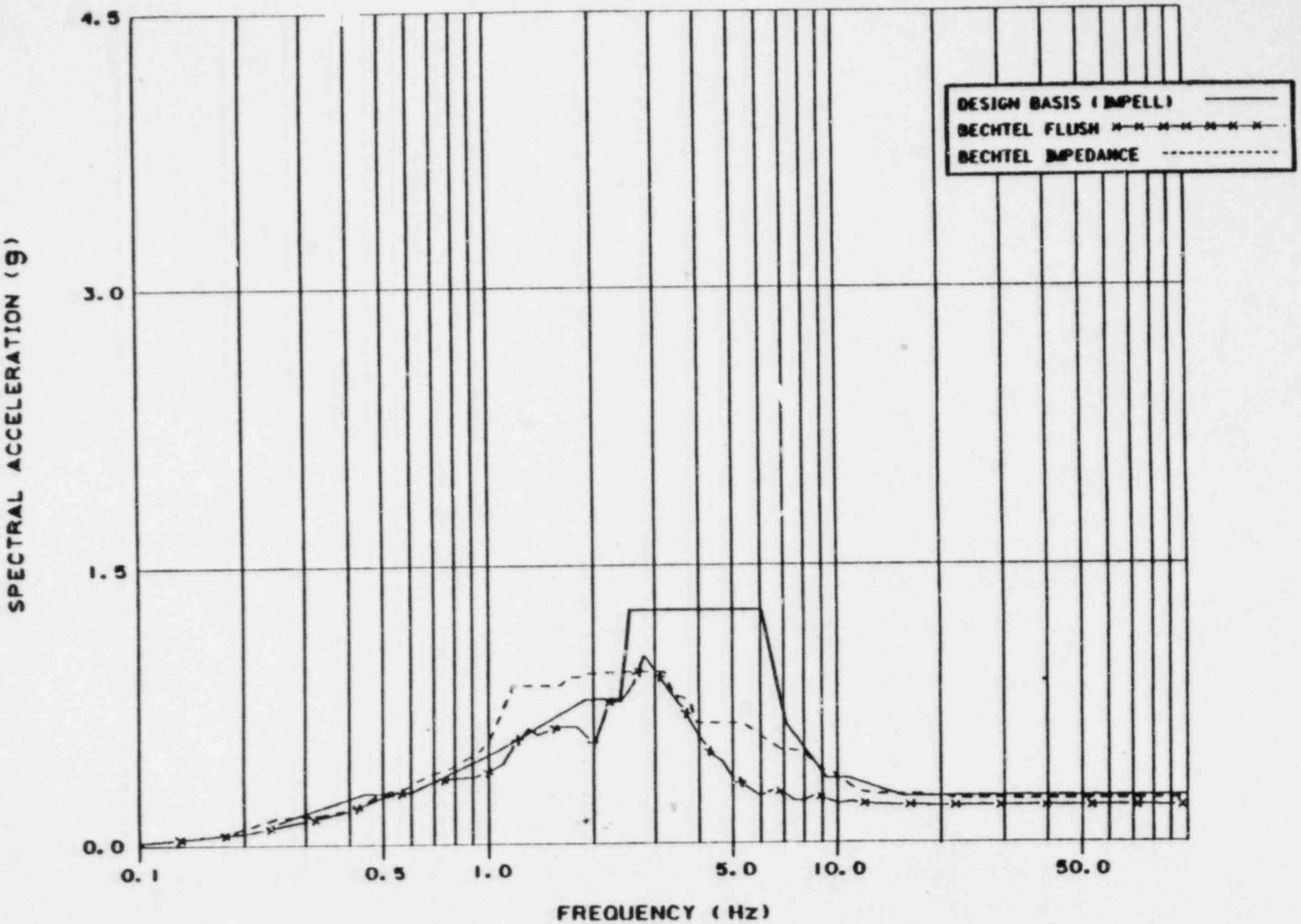


FIGURE A-13-6

RESPONSE SPECTRA COMPARISON,
UNIT I REACTOR BUILDING AT ELEV. 102'-0".

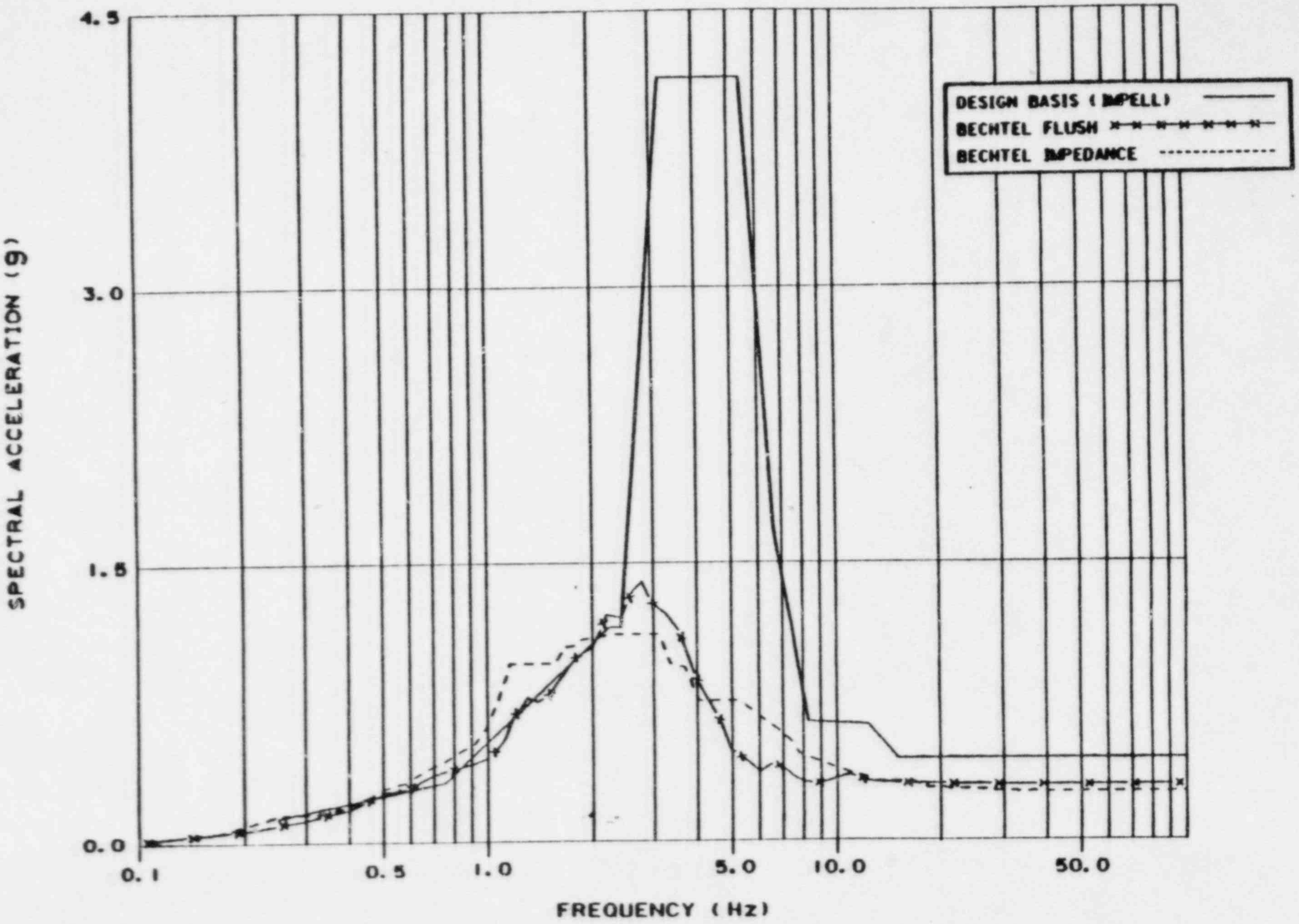


FIGURE A-13-7

RESPONSE SPECTRA COMPARISON,
UNIT I REACTOR BUILDING AT ELEV. 201'-0".

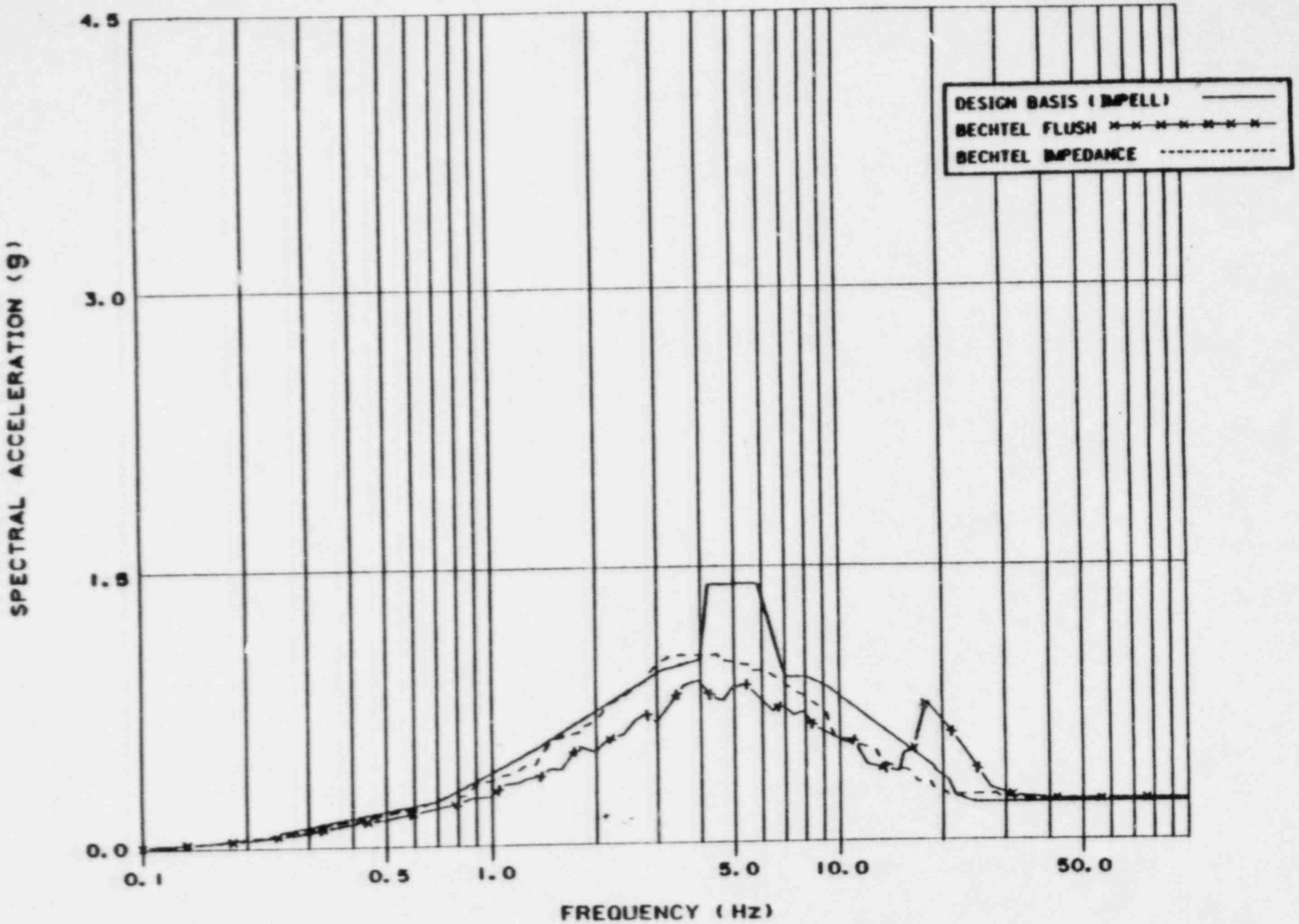


FIGURE A-13-8

RESPONSE SPECTRA COMPARISON,
UNIT 1 REACTOR BUILDING AT ELEV. 54' -0",

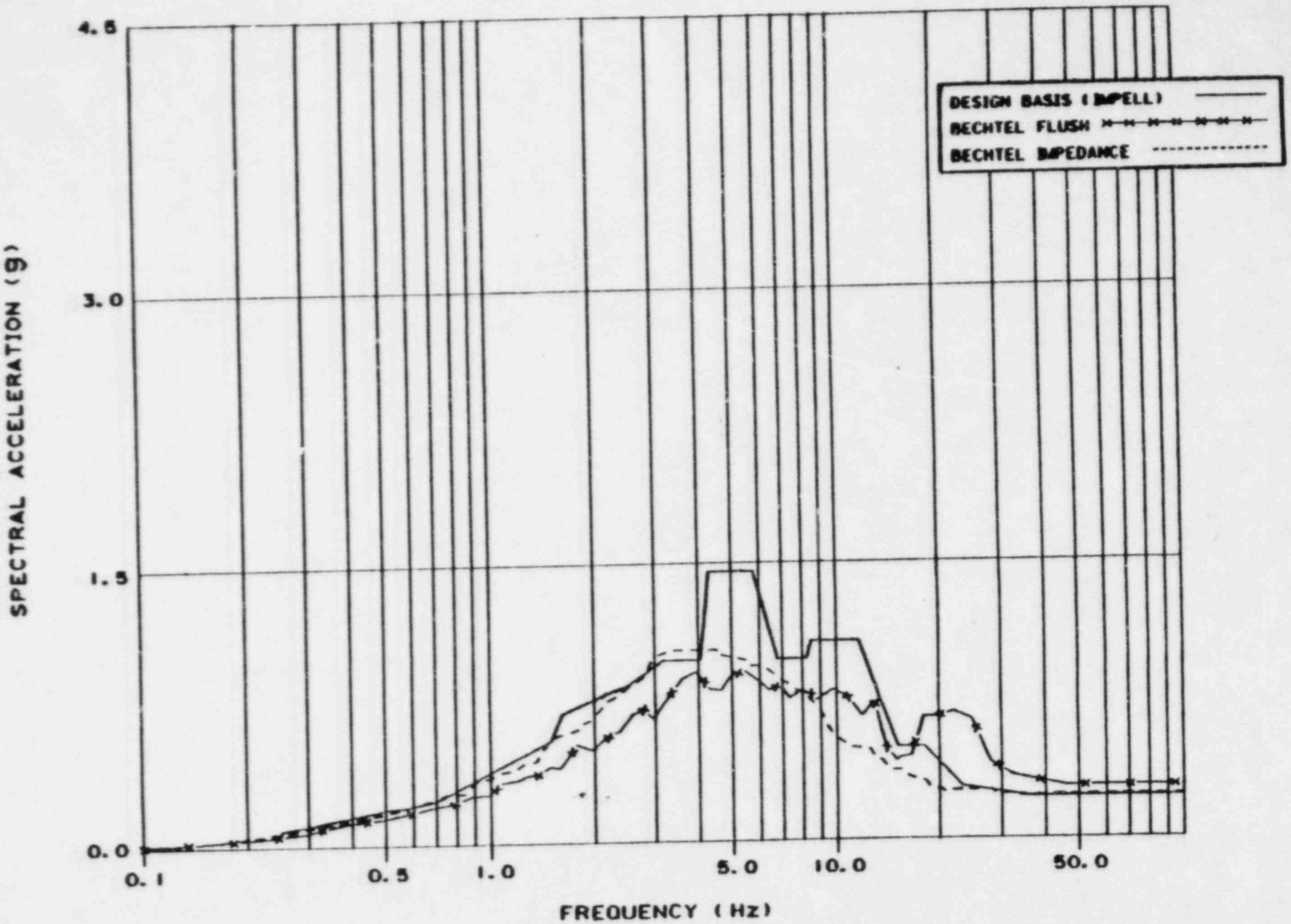


FIGURE A-13-9

RESPONSE SPECTRA COMPARISON,
[illegible] AT ELEV 102'-0".

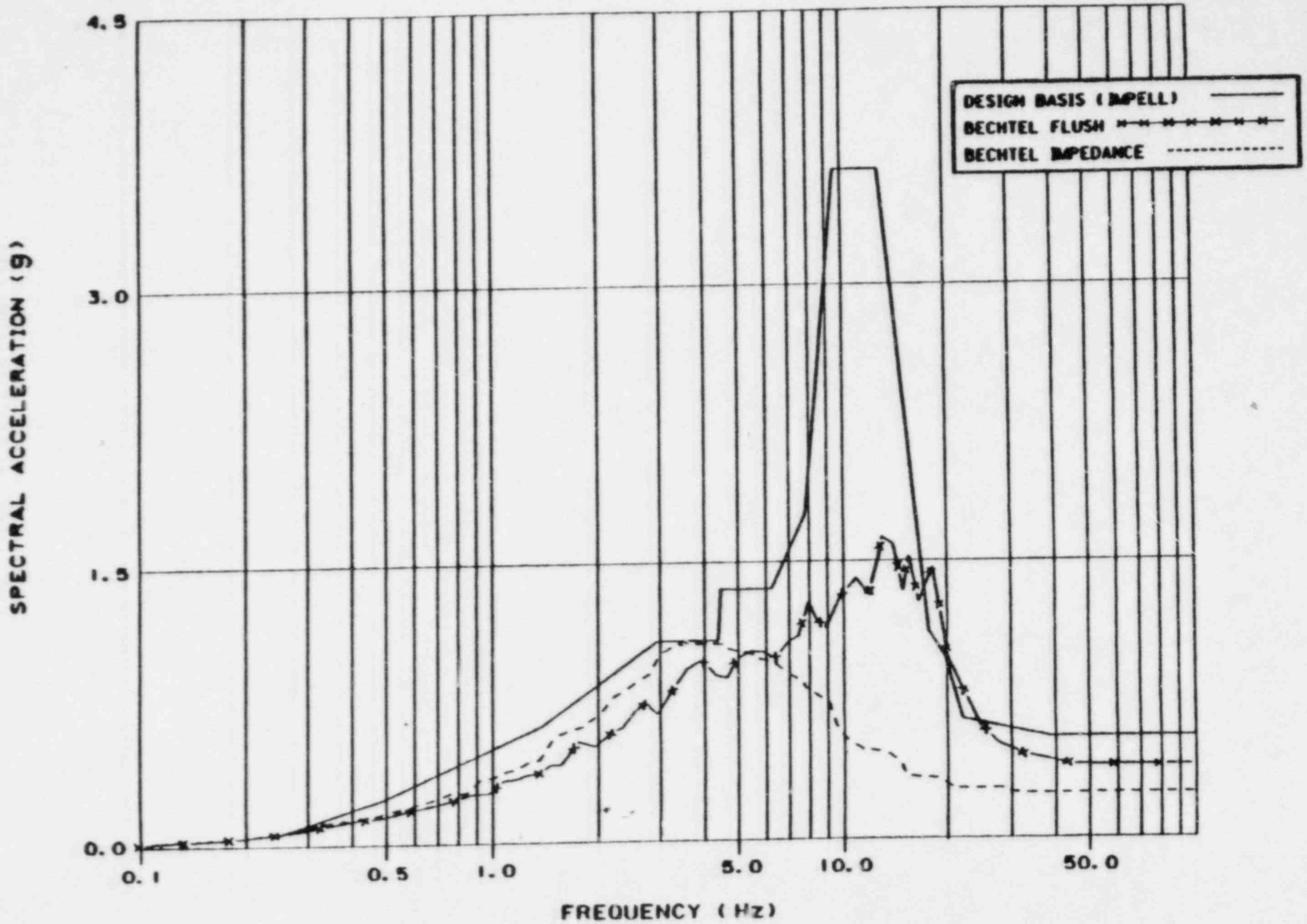


FIGURE A-13-10

RESPONSE SPECTRA COMPARISON,
UNIT 1 REACTOR BUILDING AT ELEV. 201'-0".

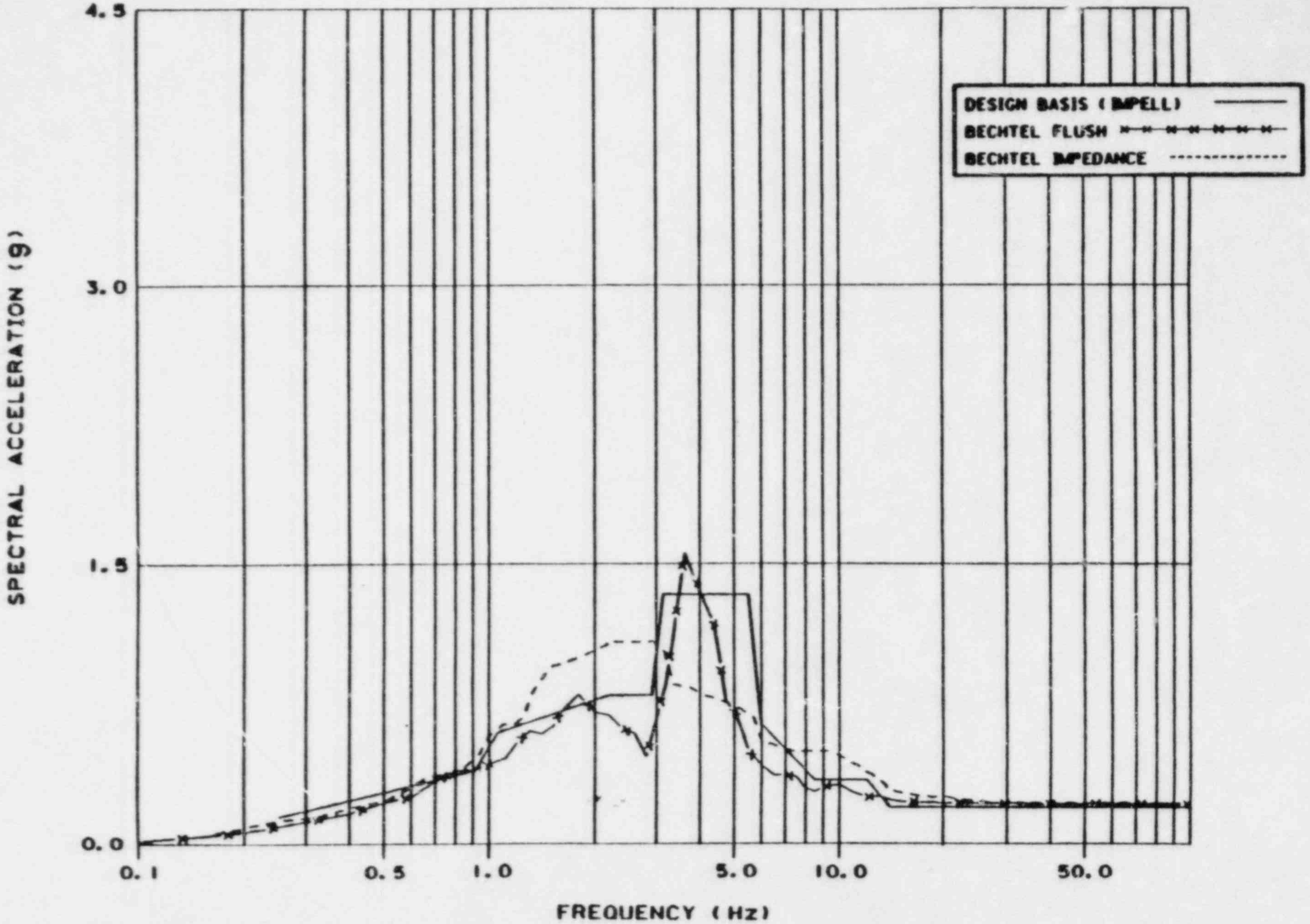


FIGURE A-13-11

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV 54'-0".

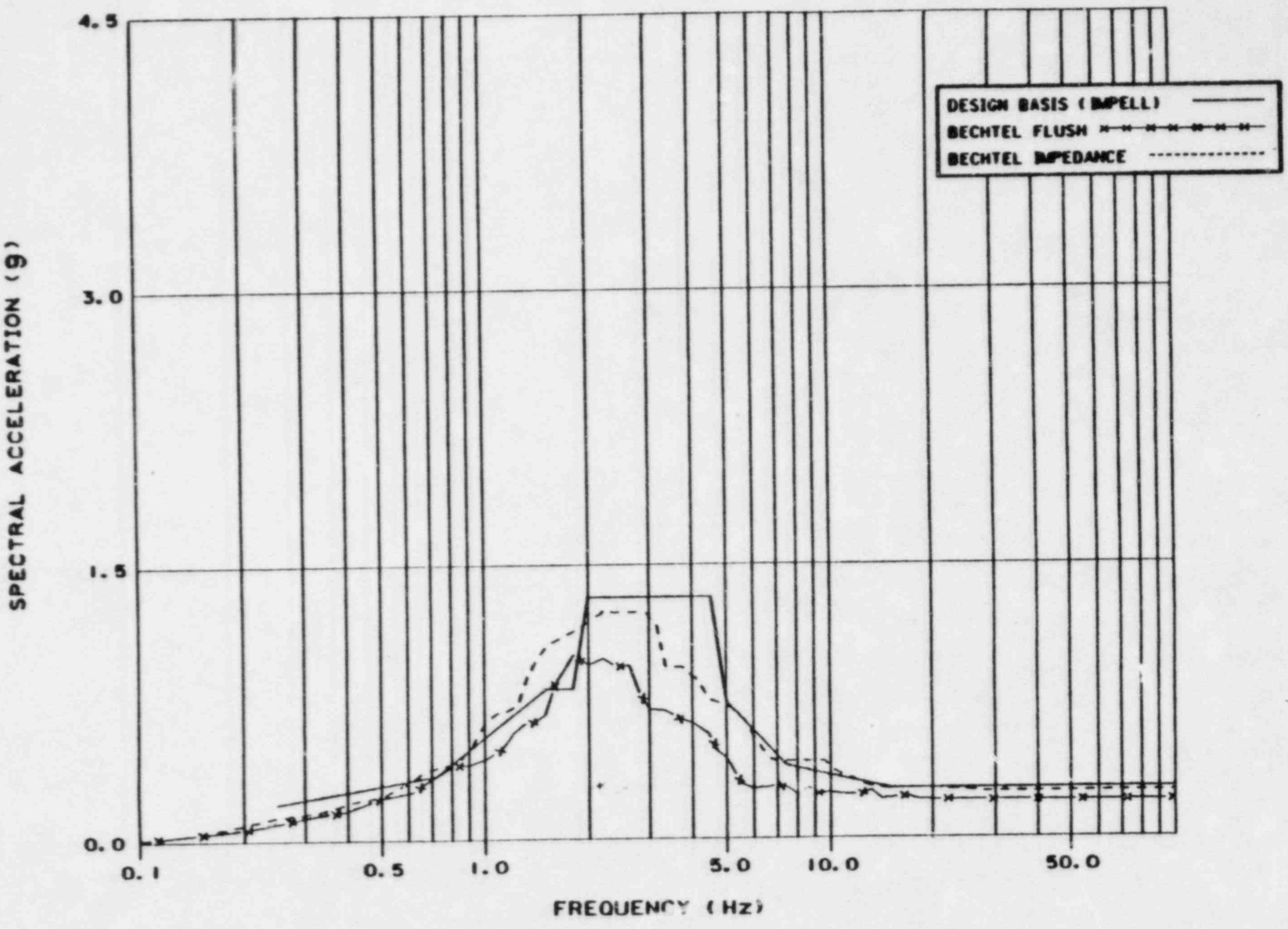


FIGURE A-13-12

RESPONSE SPECTRA COMPARISON,

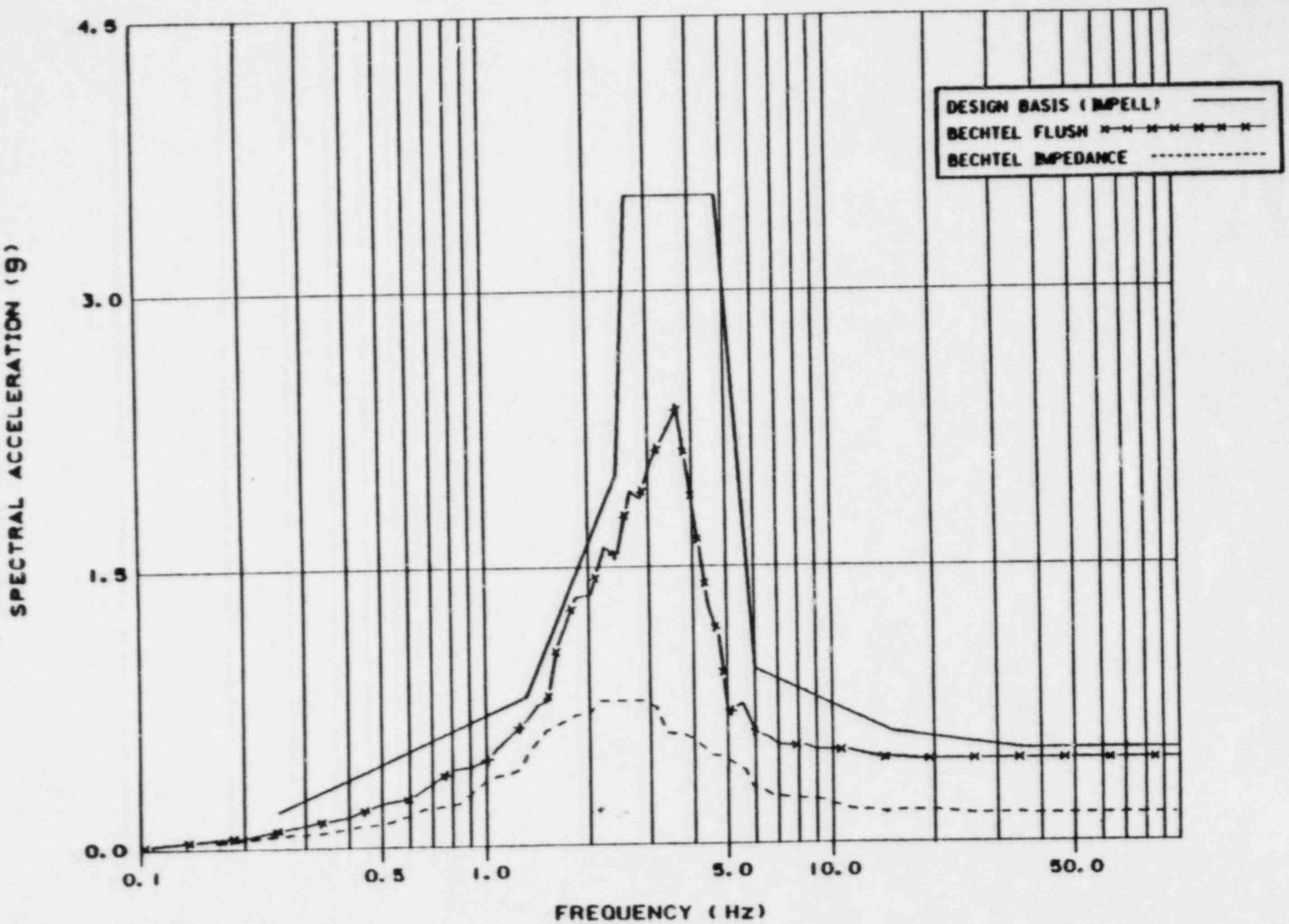


FIGURE A-13-13

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV. 178'-0".

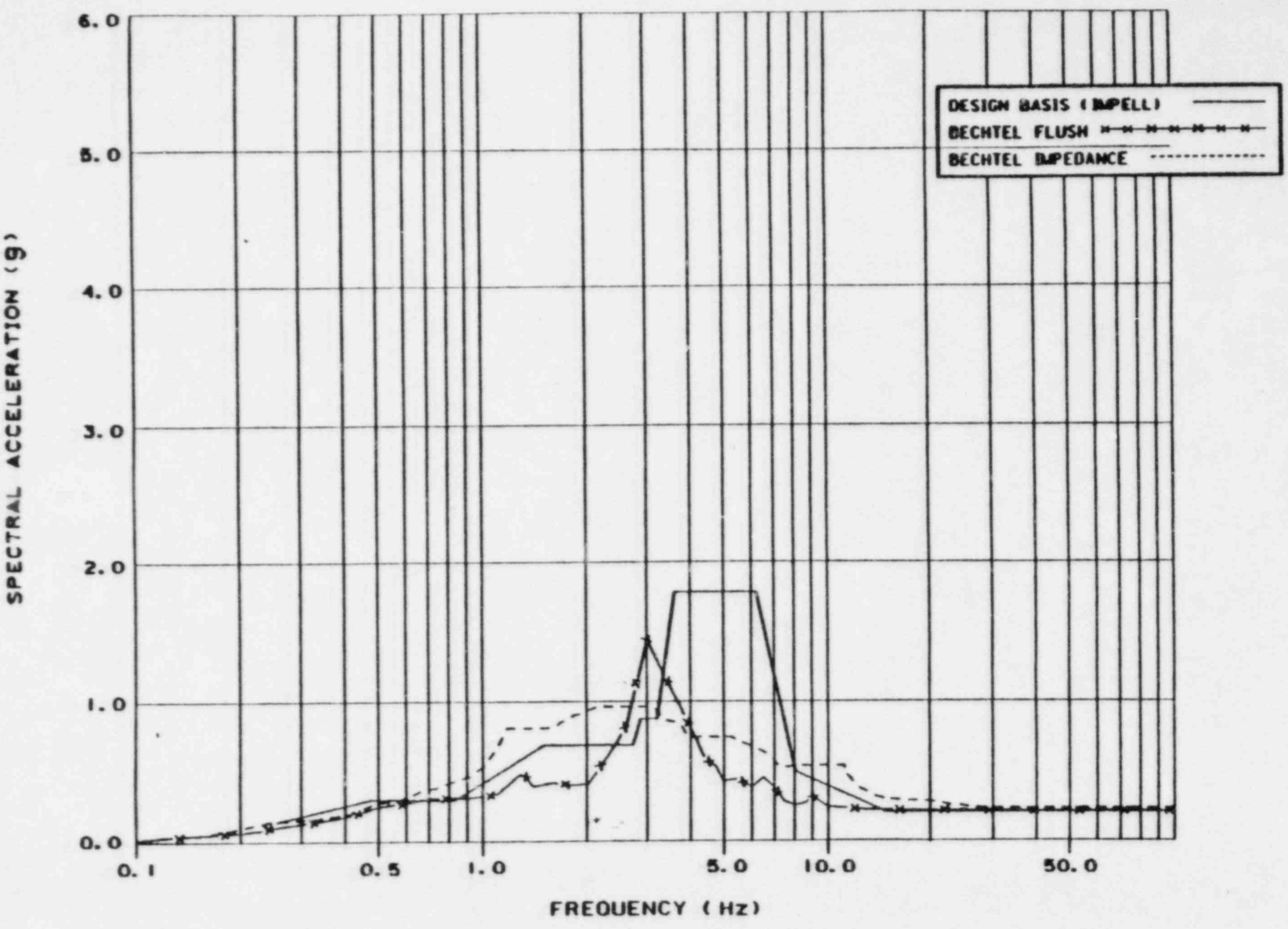


FIGURE A-13-14

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV. 54' -0",

HOPE CREEK UNIT 1

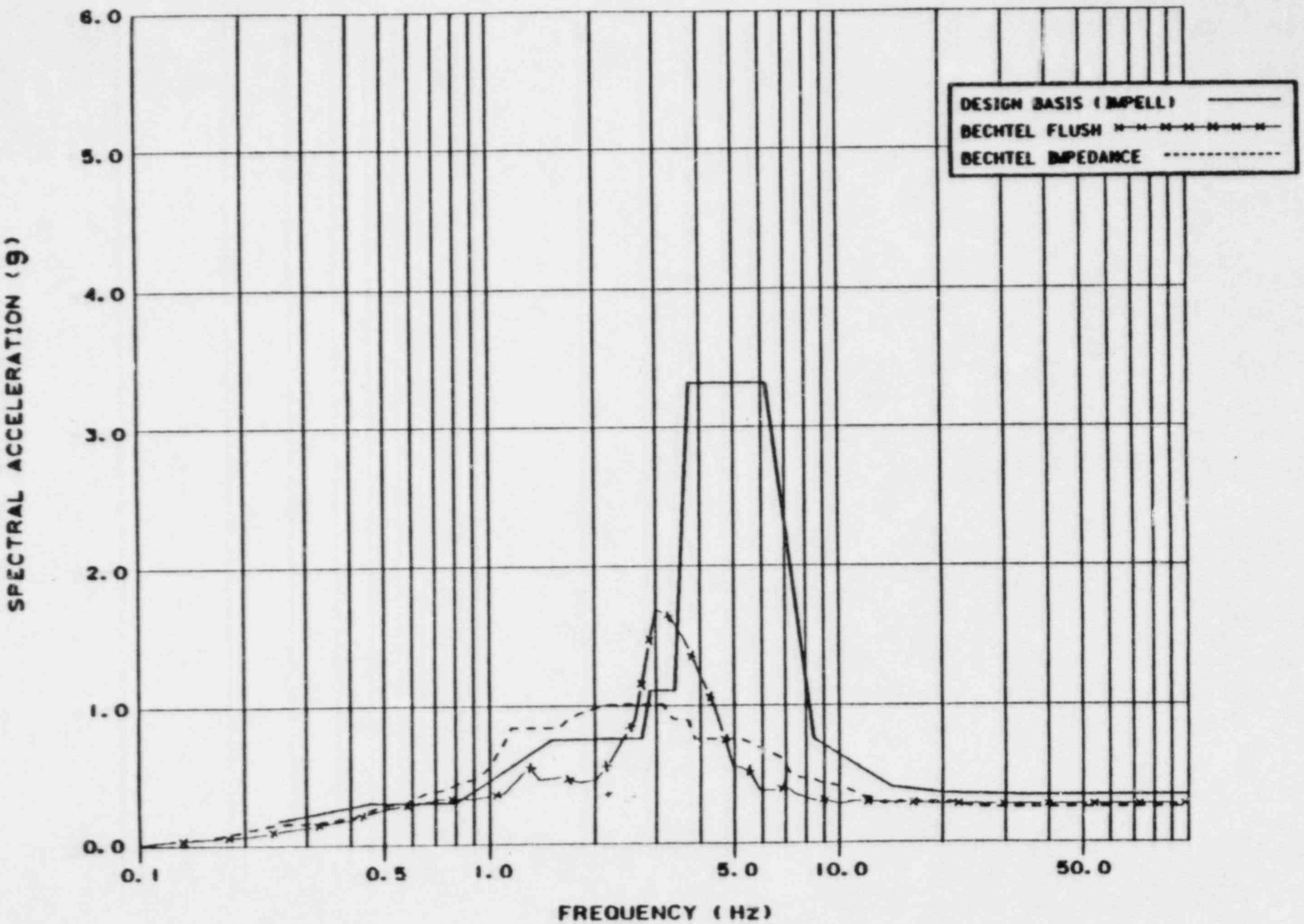


FIGURE A-13-15

RESPONSE SPECTRA COMPARISON,

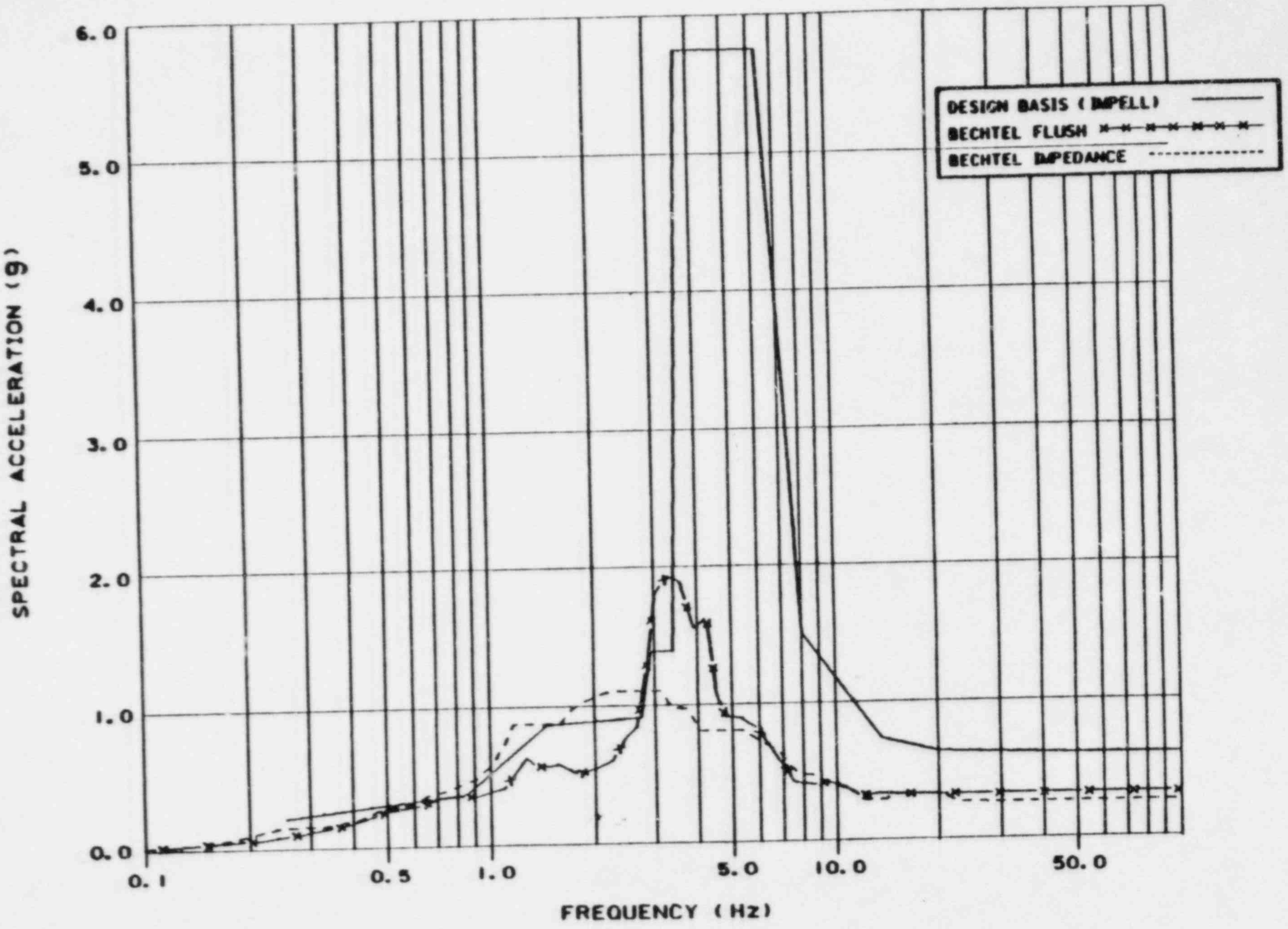


FIGURE A-13-16

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV. 178'-0".

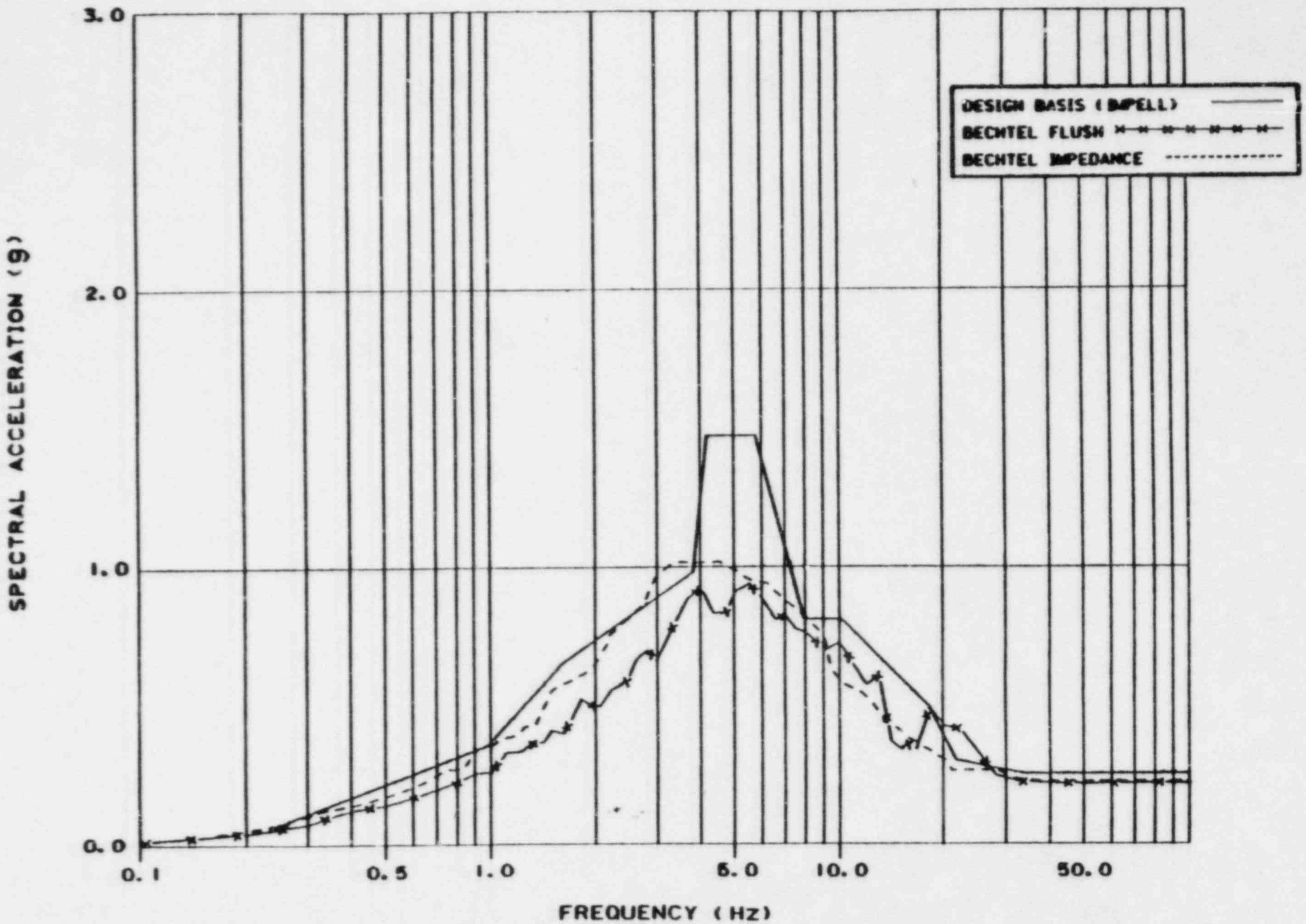


FIGURE A-13-17

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV. 54'-0".

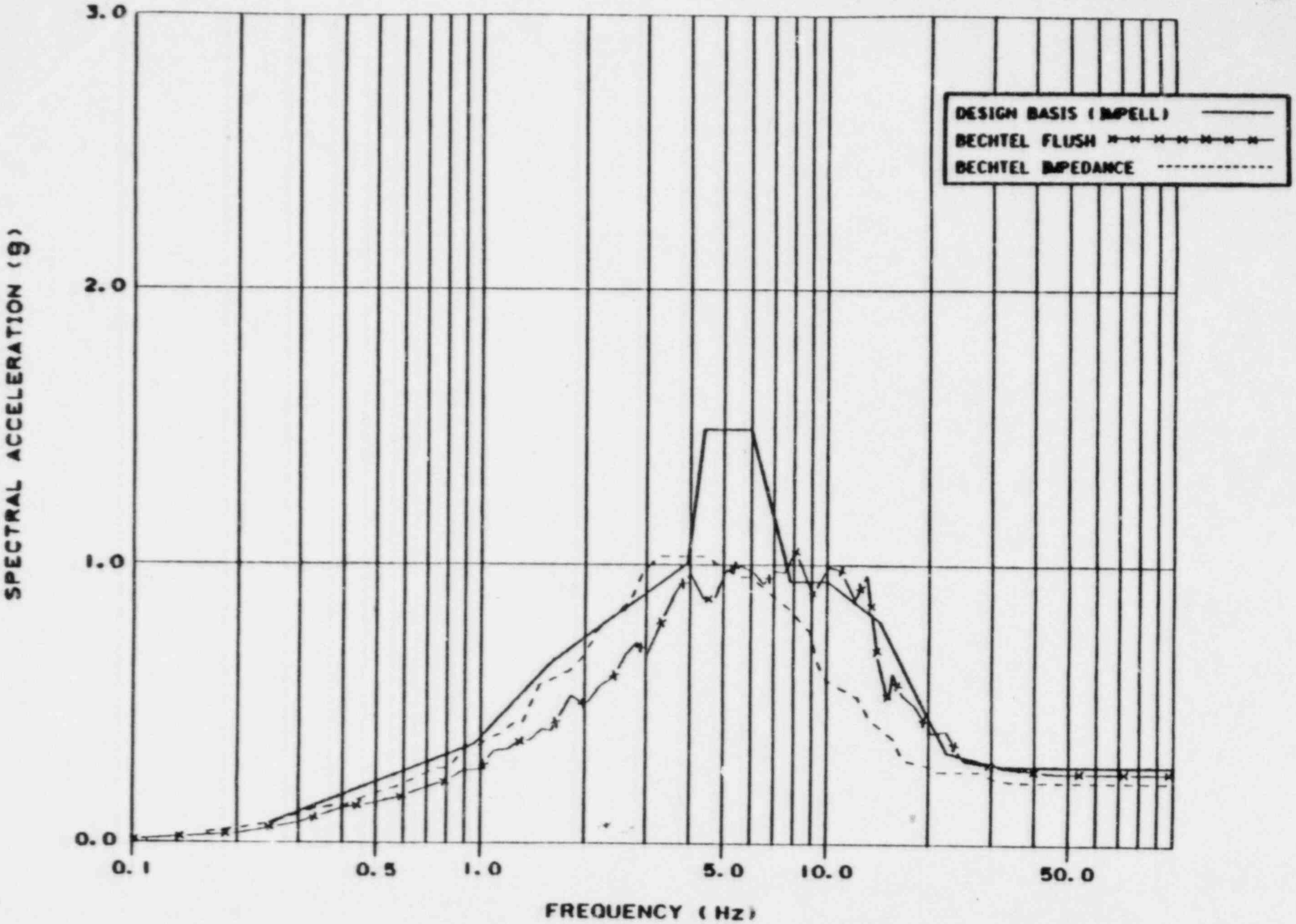


FIGURE A-13-18

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV. 102' -0",

HOPE CREEK UNIT 1

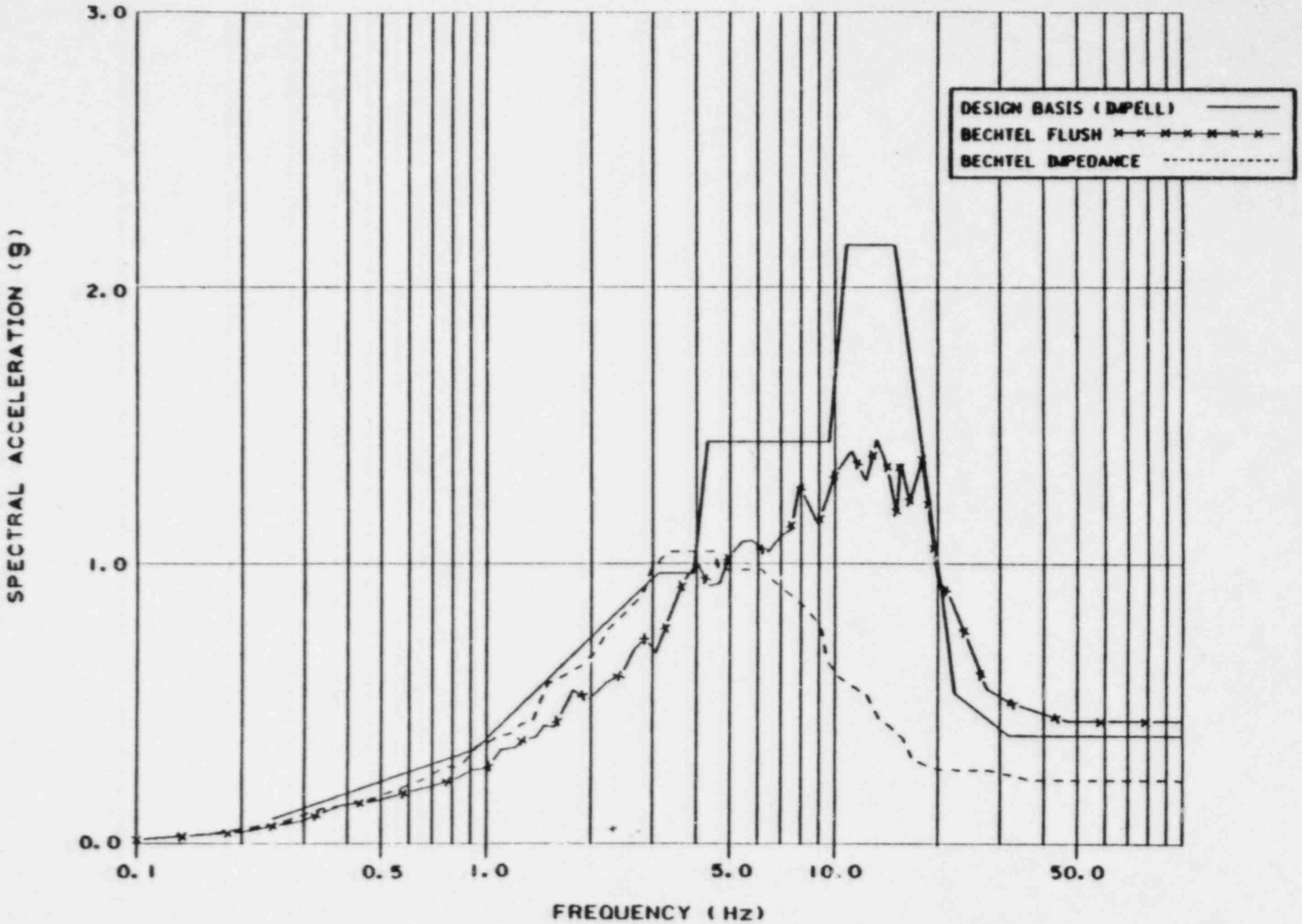


FIGURE A-13-19

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV. 178'-0",
VERTICAL SSE 2% DAMPING

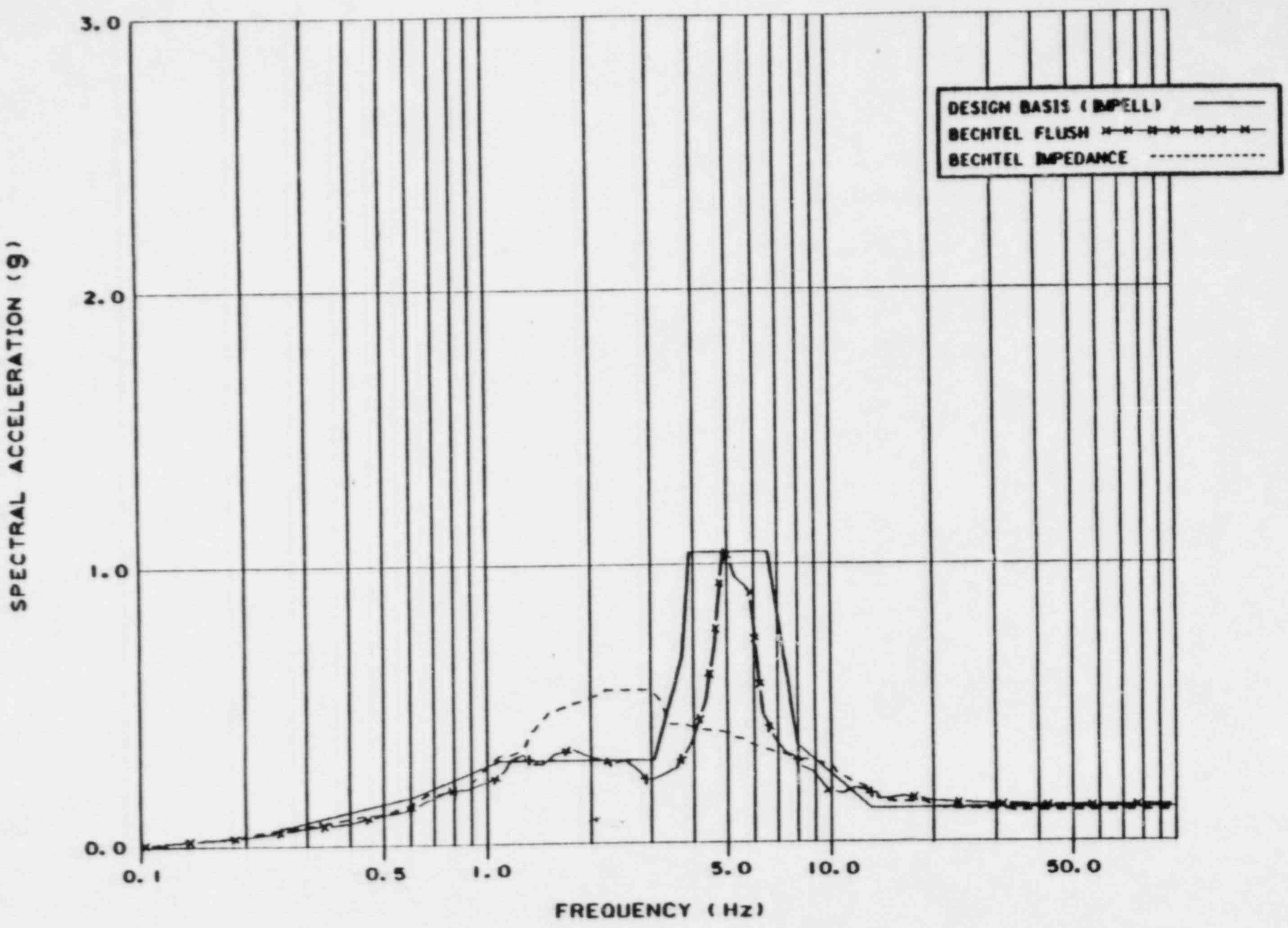


FIGURE A-13-20

RESPONSE SPECTRA COMPARISON,
UNIT 1 REACTOR BUILDING AT ELEV. 54'-0"

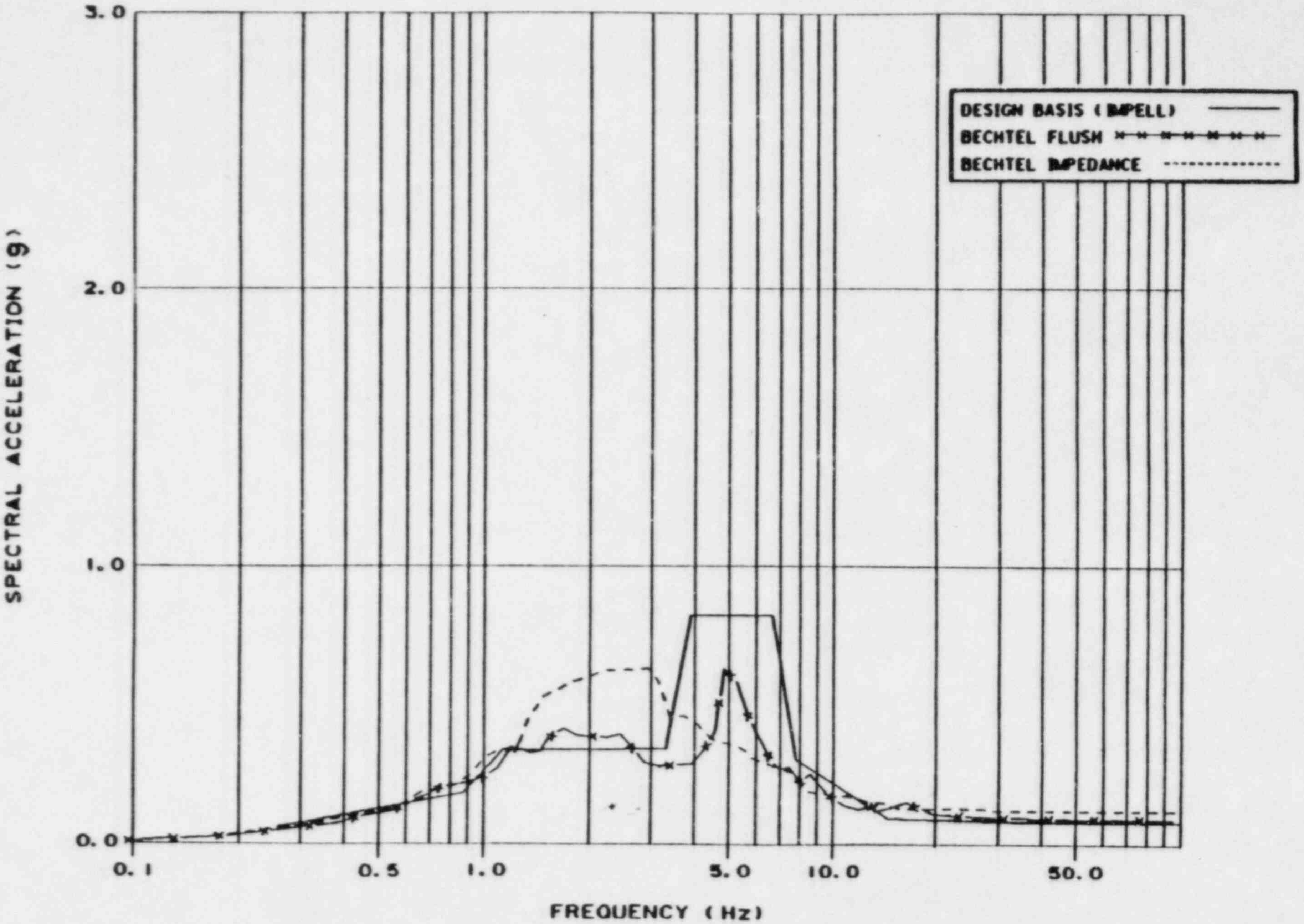


FIGURE A-13-21

RESPONSE SPECTRA COMPARISON,
UNIT 1 REACTOR BUILDING AT ELEV. 102'-0",

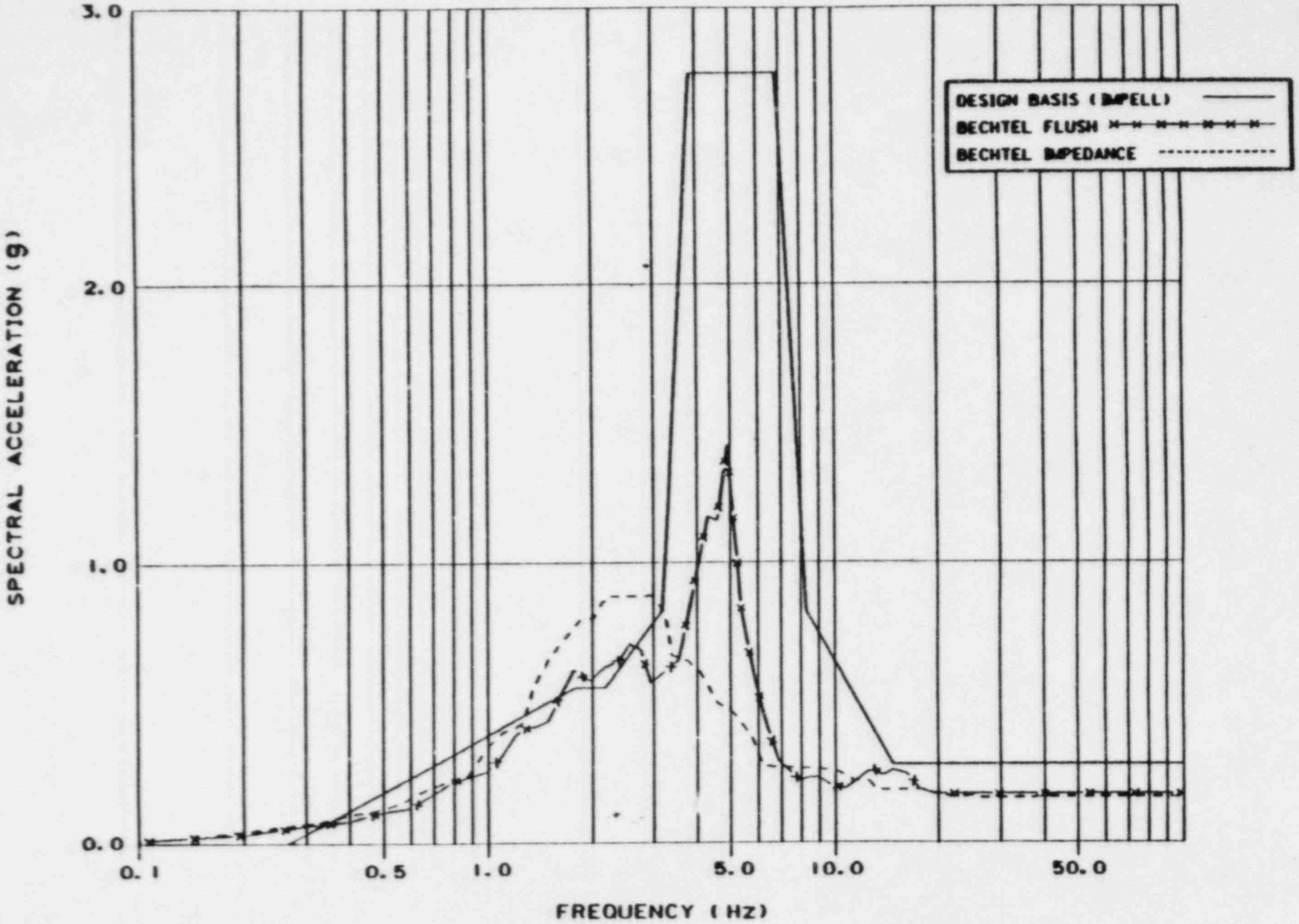


FIGURE A-13-22

RESPONSE SPECTRA COMPARISON, UNIT I REACTOR BUILDING AT ELEV. 201'-0".

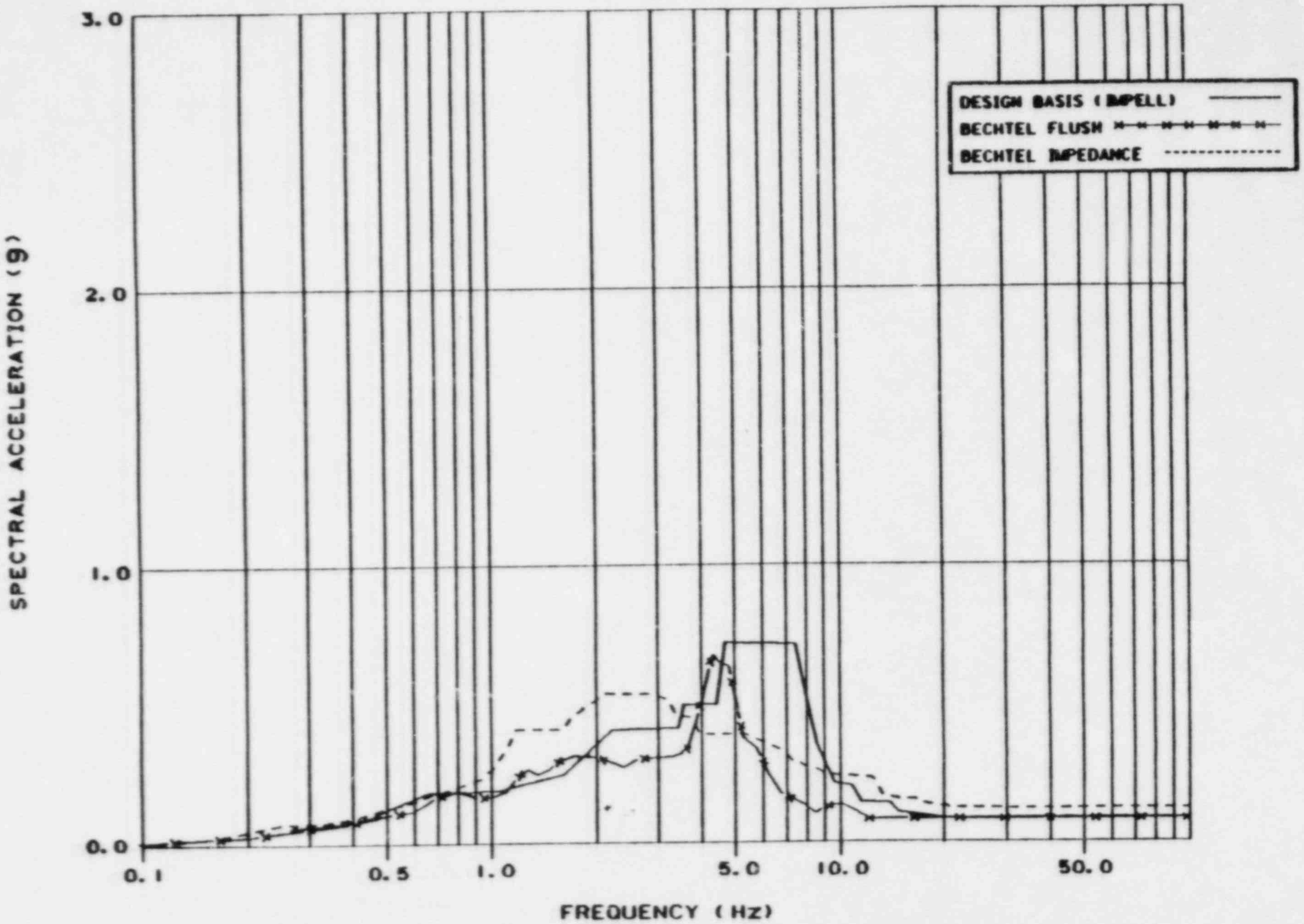


FIGURE A-13-23

RESPONSE SPECTRA COMPARISON,
UNIT 1 REACTOR BUILDING AT ELEV. 54'-0",

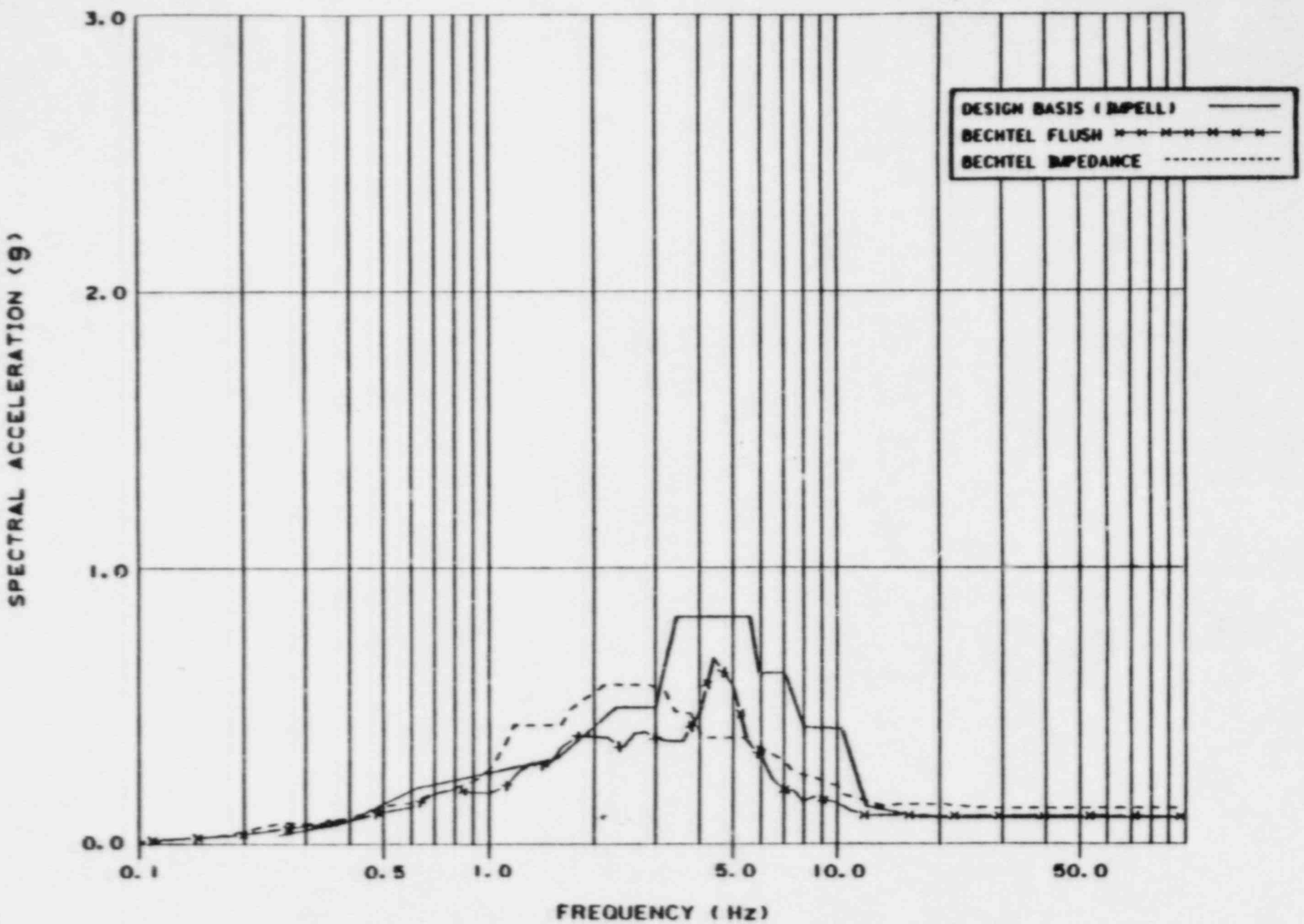


FIGURE A-13-24

RESPONSE SPECTRA COMPARISON,
UNIT 1 REACTOR BUILDING AT ELEV. 102' -0".

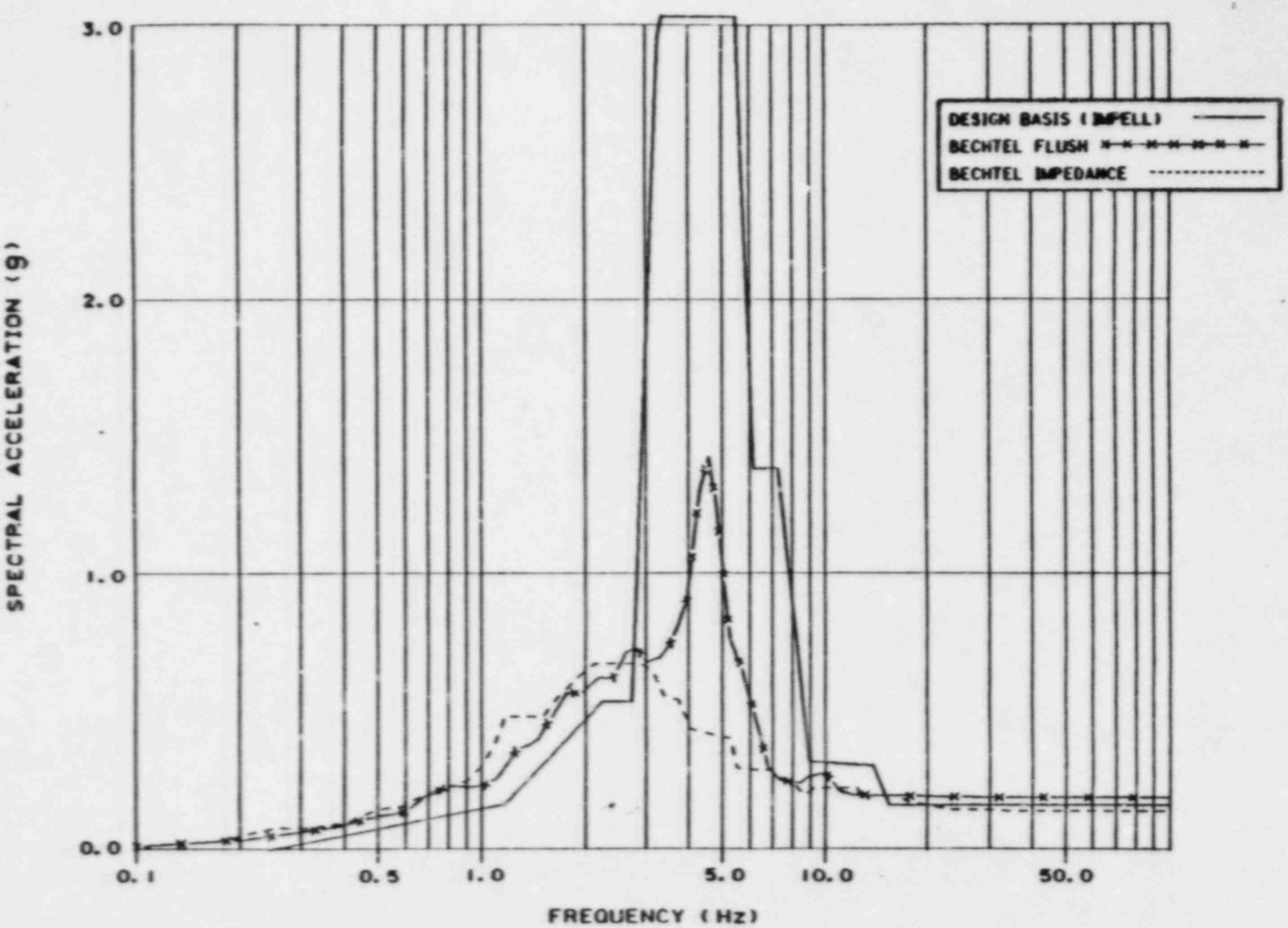


FIGURE A-13-25

RESPONSE SPECTRA COMPARISON,

UNIT 1 REACTOR BUILDING AT ELEV 201'-0"

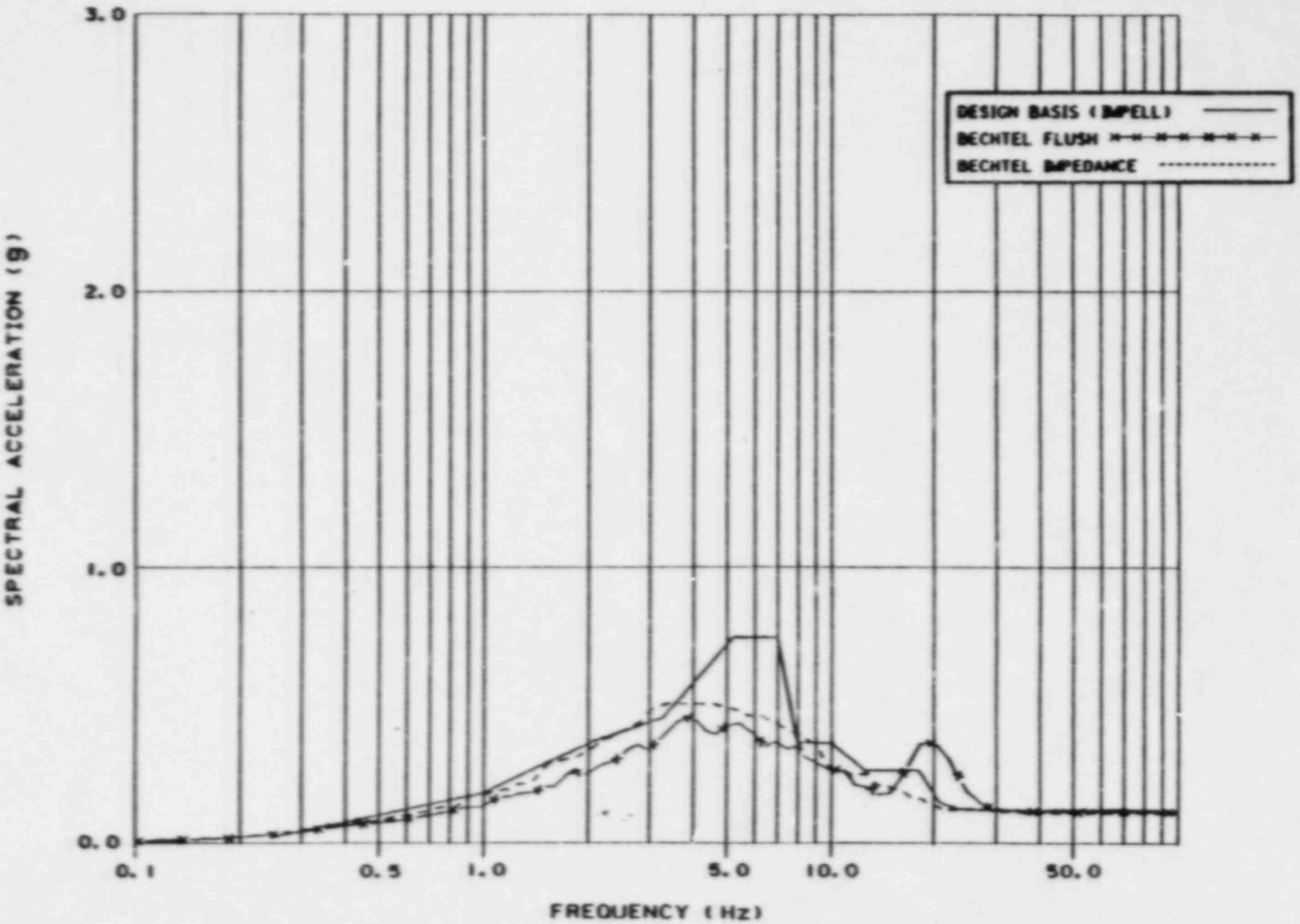


FIGURE A-13-26

RESPONSE SPECTRA COMPARISON,
UNIT 1 REACTOR BUILDING AT ELEV. 54'-0".

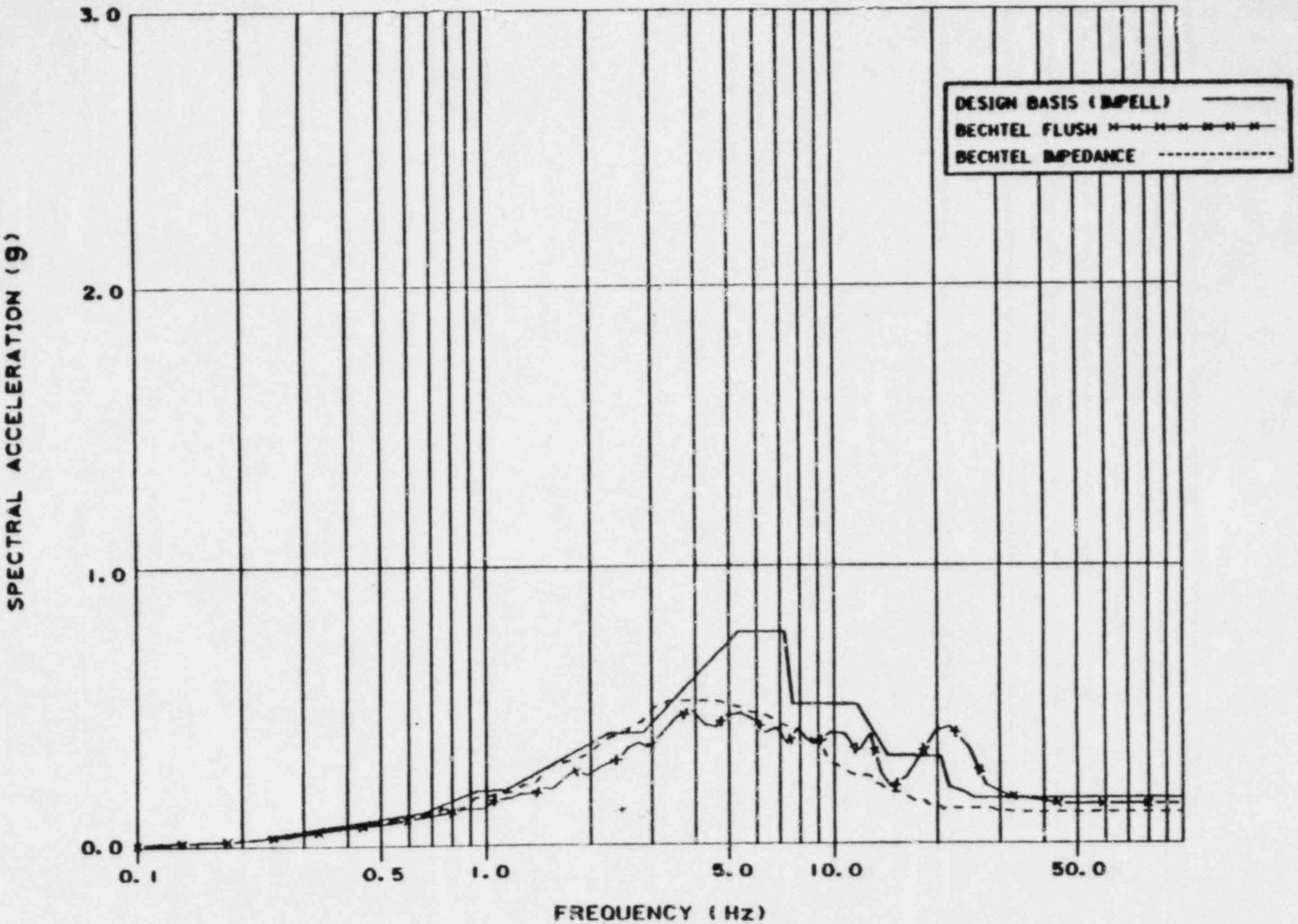
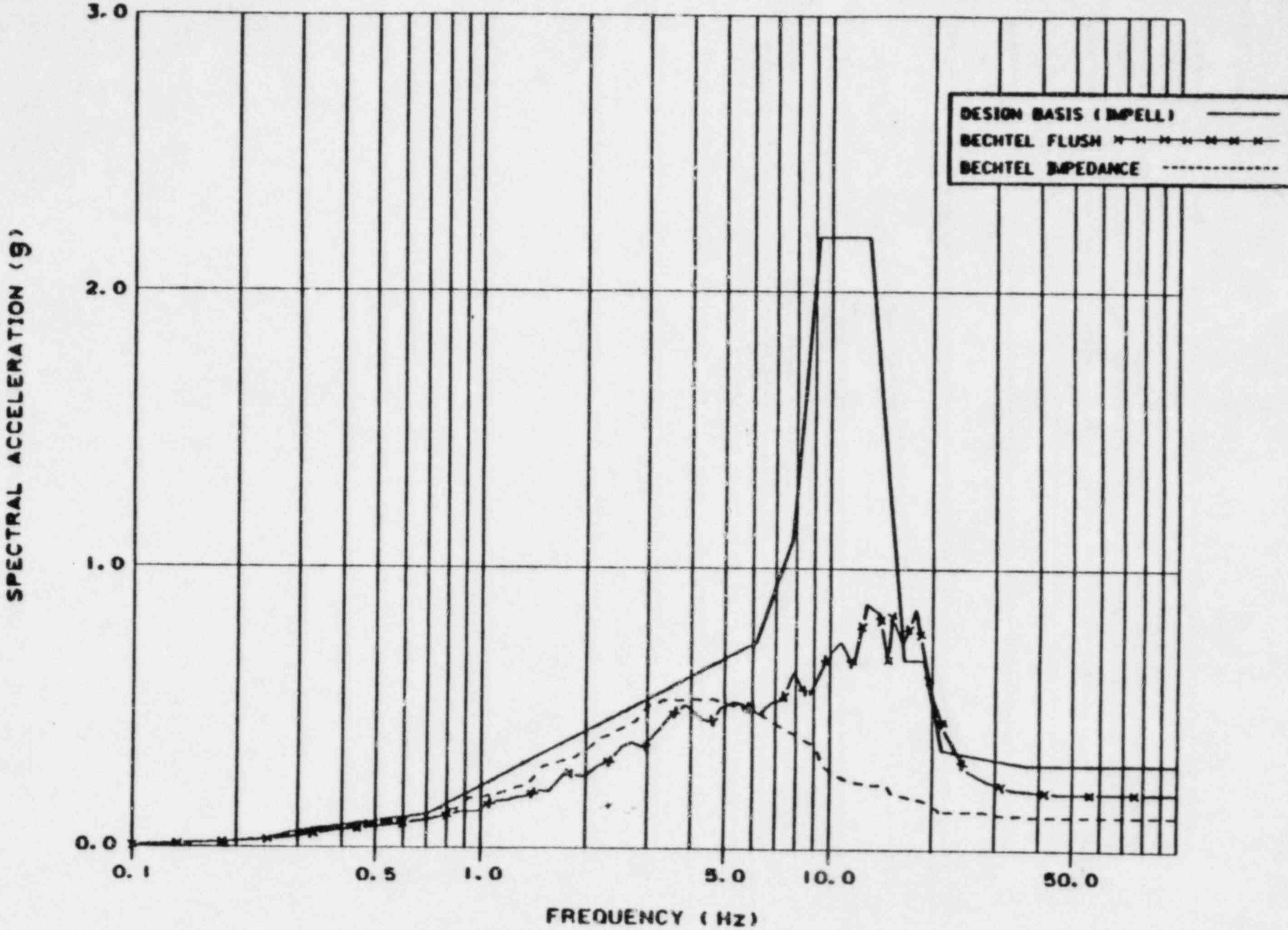


FIGURE A-13-27

RESPONSE SPECTRA COMPARISON,
UNIT 1 REACTOR BUILDING AT ELEV. 102'-0".



FIGUF_ A-13-20

RESPONSE SPECTRA COMPARISON,
UNIT 1 REACTOR BUILDING AT ELEV 201'-0"

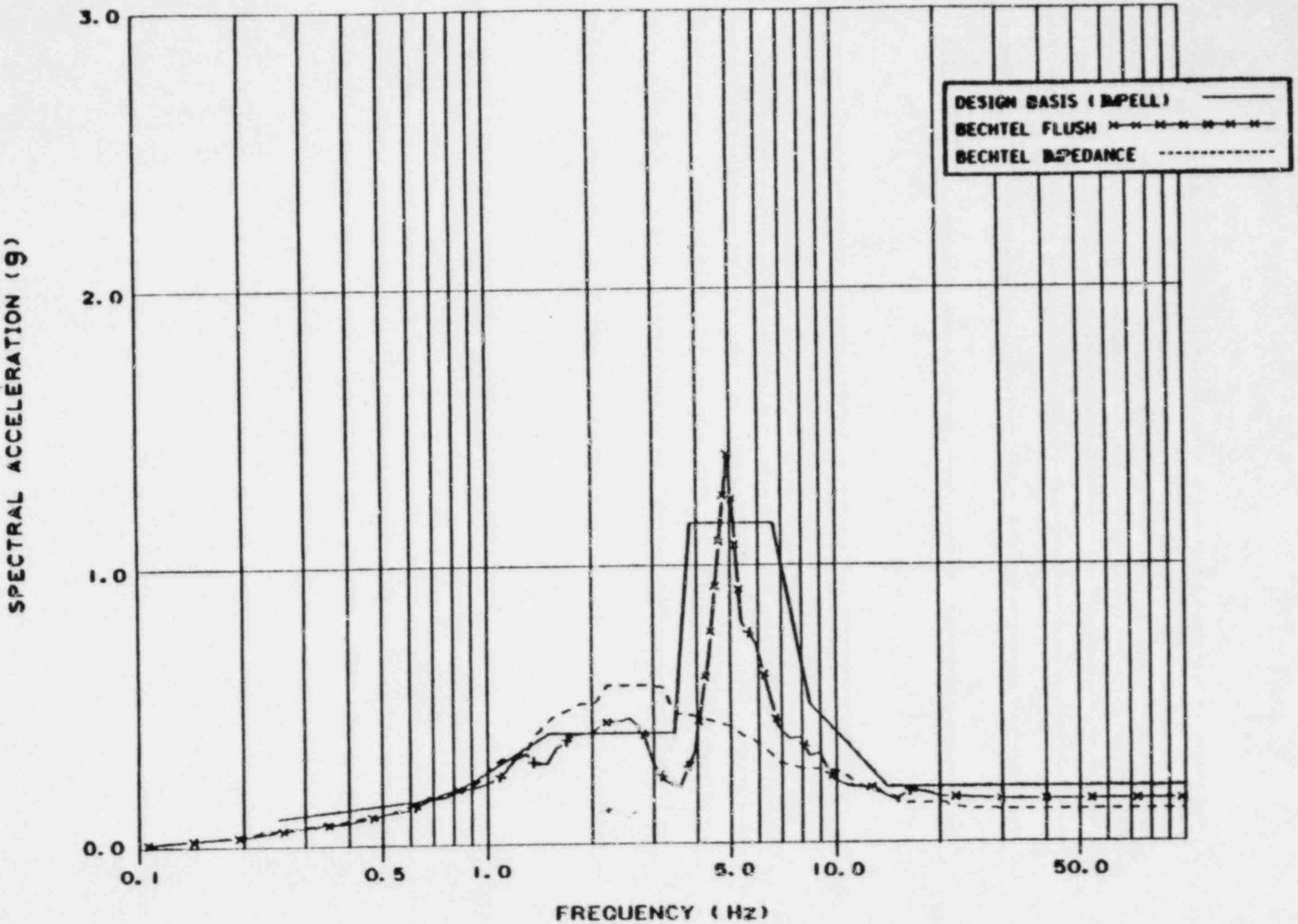


FIGURE A-13-29

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV. 54' -0",

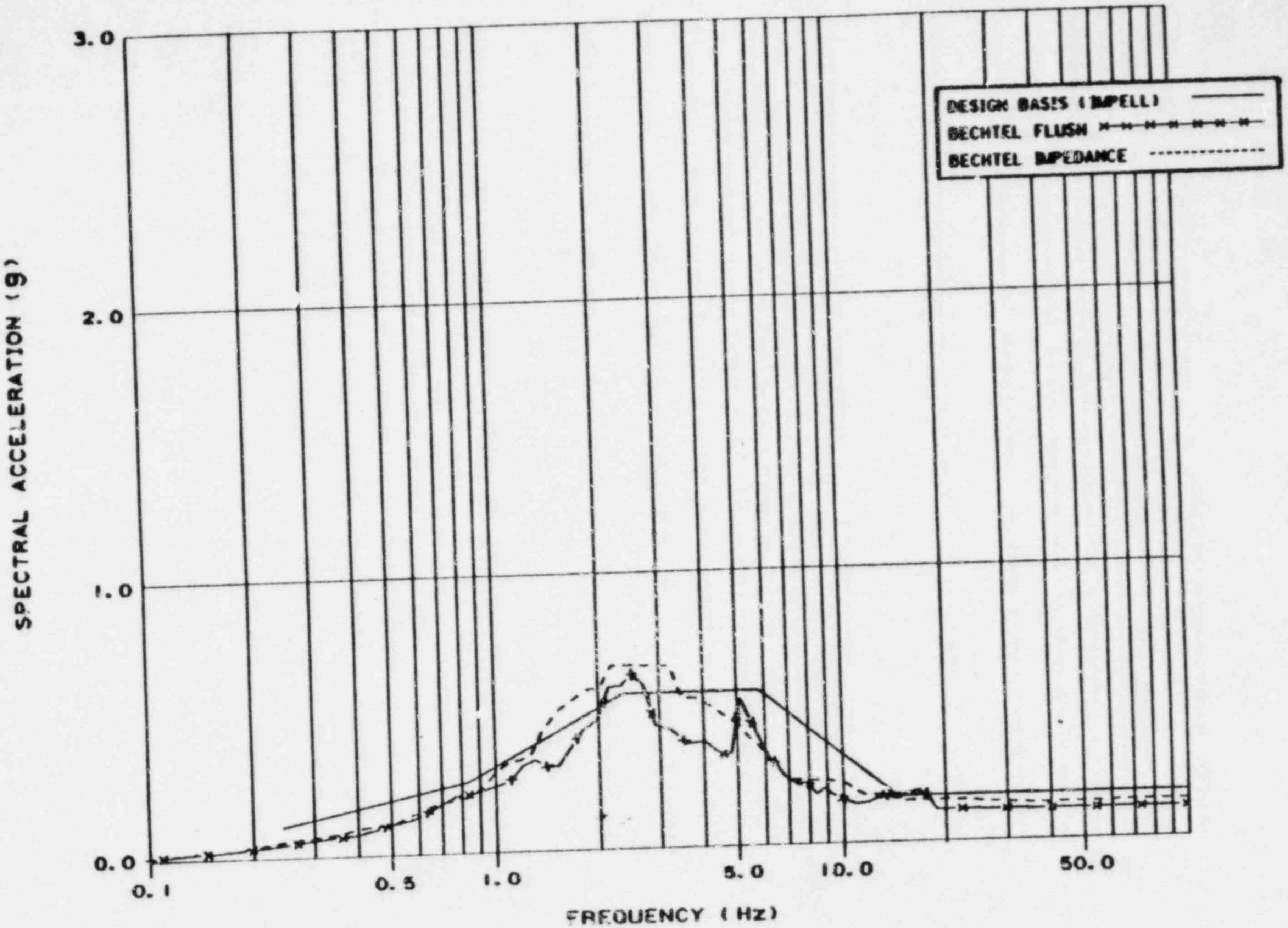


FIGURE A-13-30

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV. 102'-0".

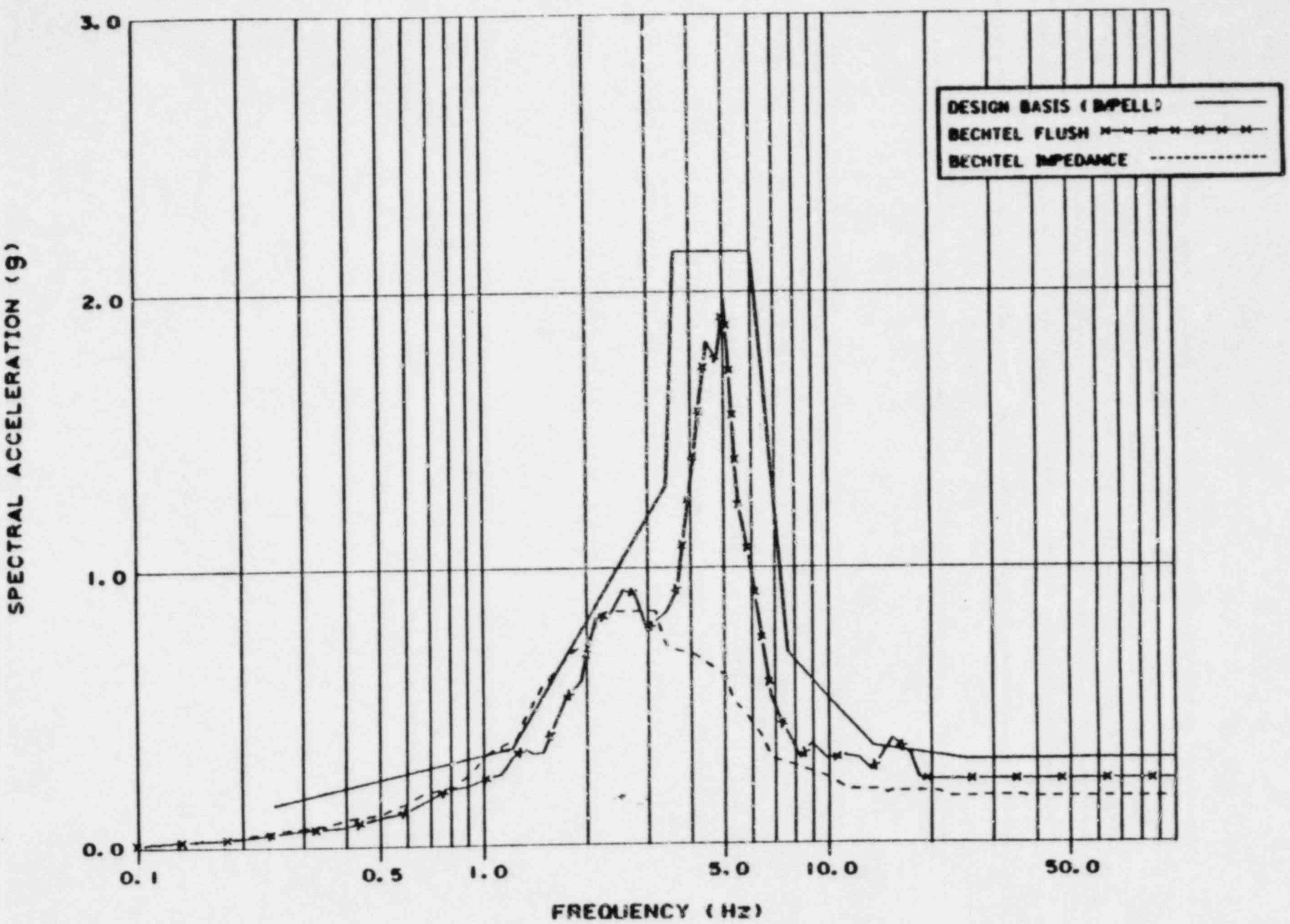


FIGURE A-13-31

RESPONSE SPECTRA COMPARISON,

...BUILDING AT ELEV 178'-0".

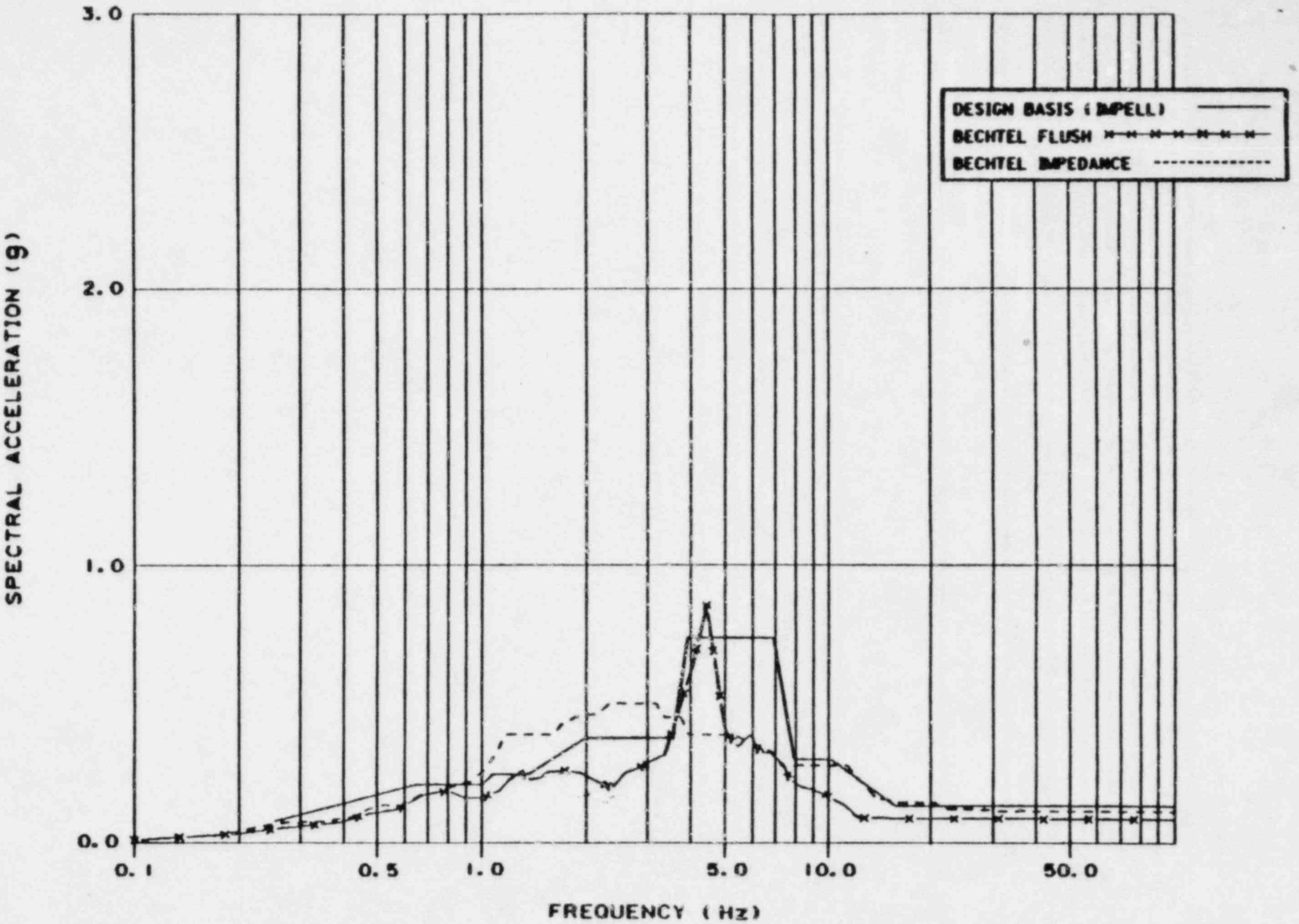
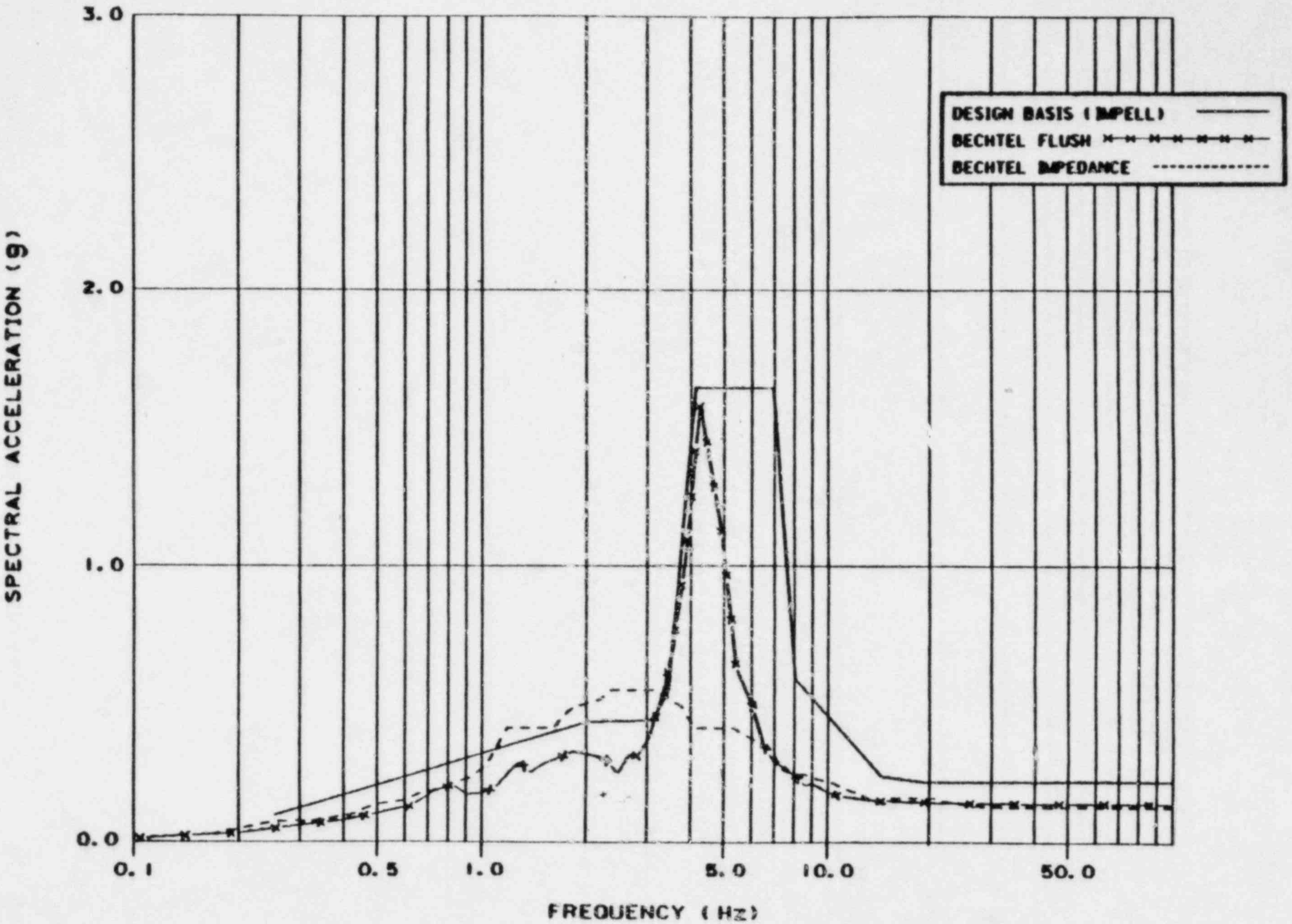


FIGURE A-13-32

RESPONSE SPECTRA COMPARISON,



<u>FIGURE A-13-33</u>

RESPONSE SPECTRA COMPARISON,

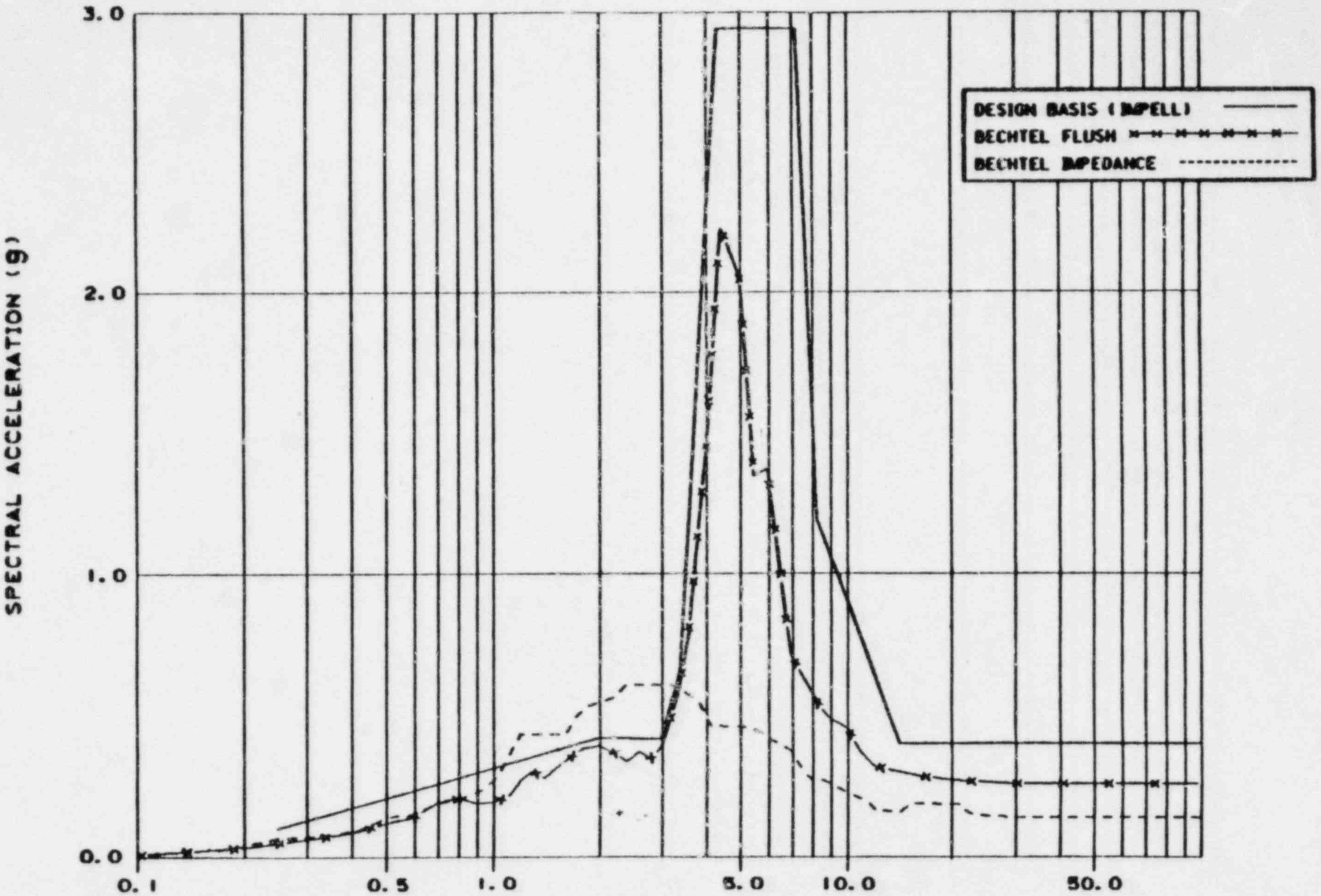


FIGURE A-13-34

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV. 178'-0"

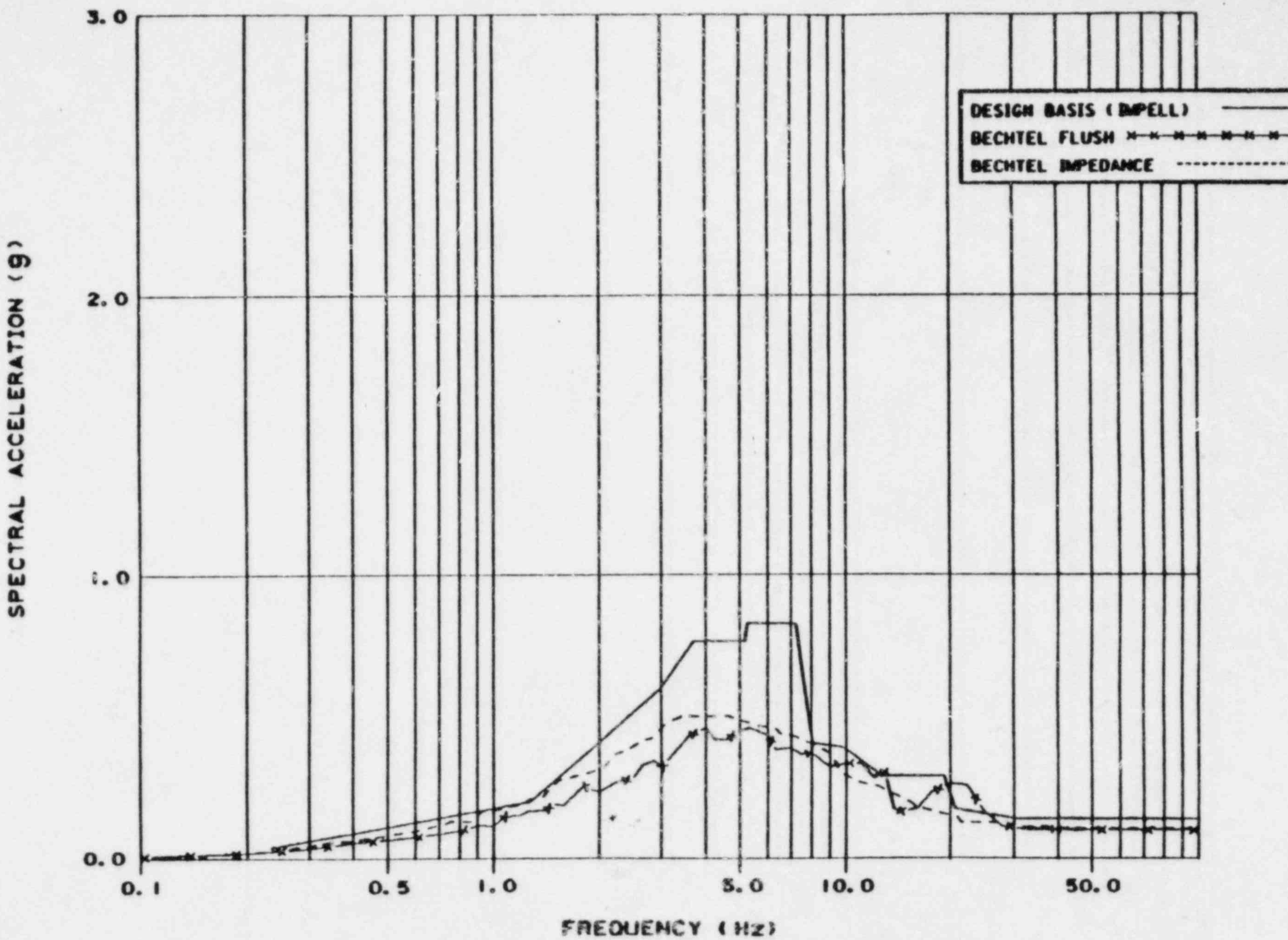


FIGURE A-13-35

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV. 54' -0".

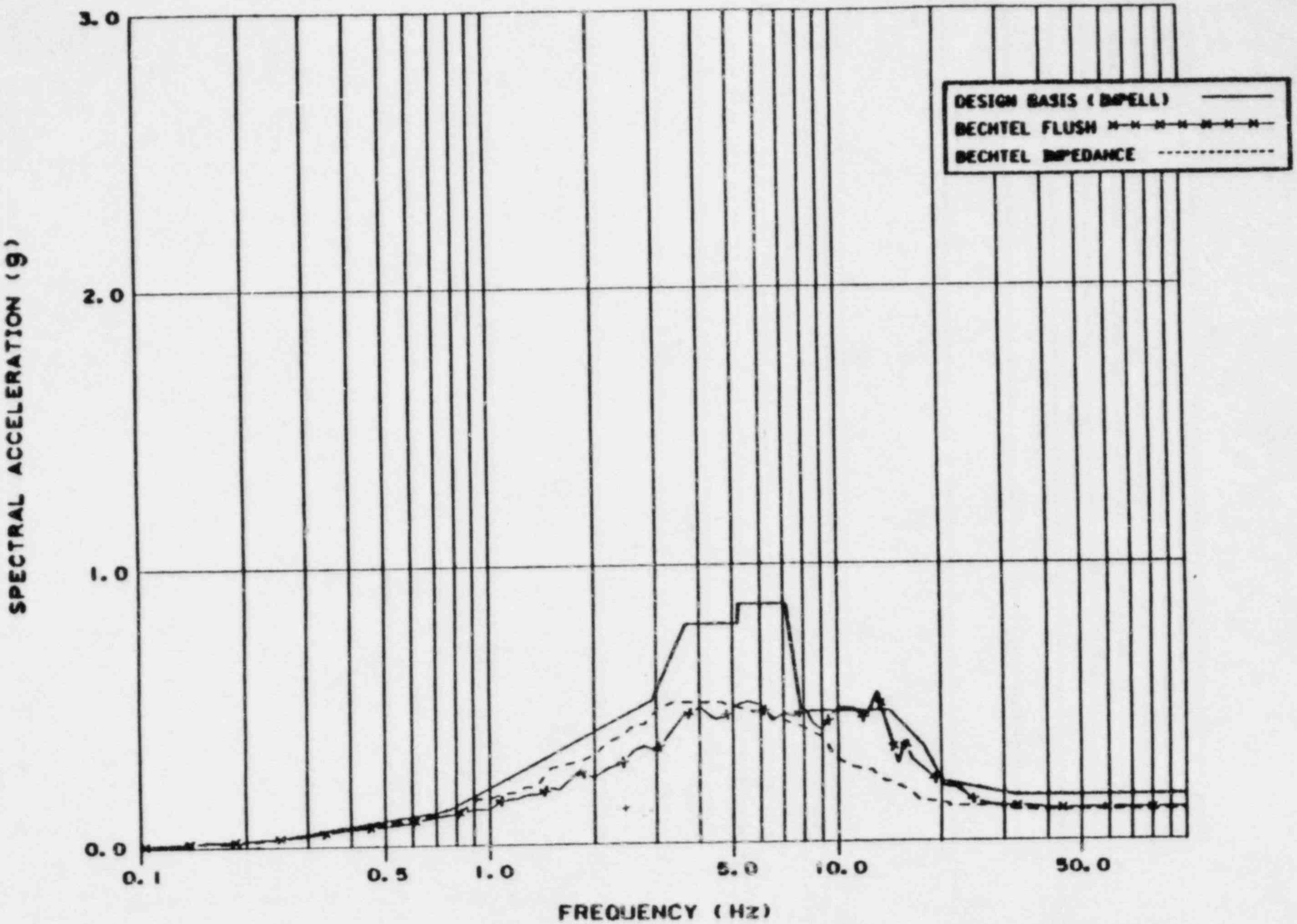


FIGURE A-13-36

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV. 102' -0".

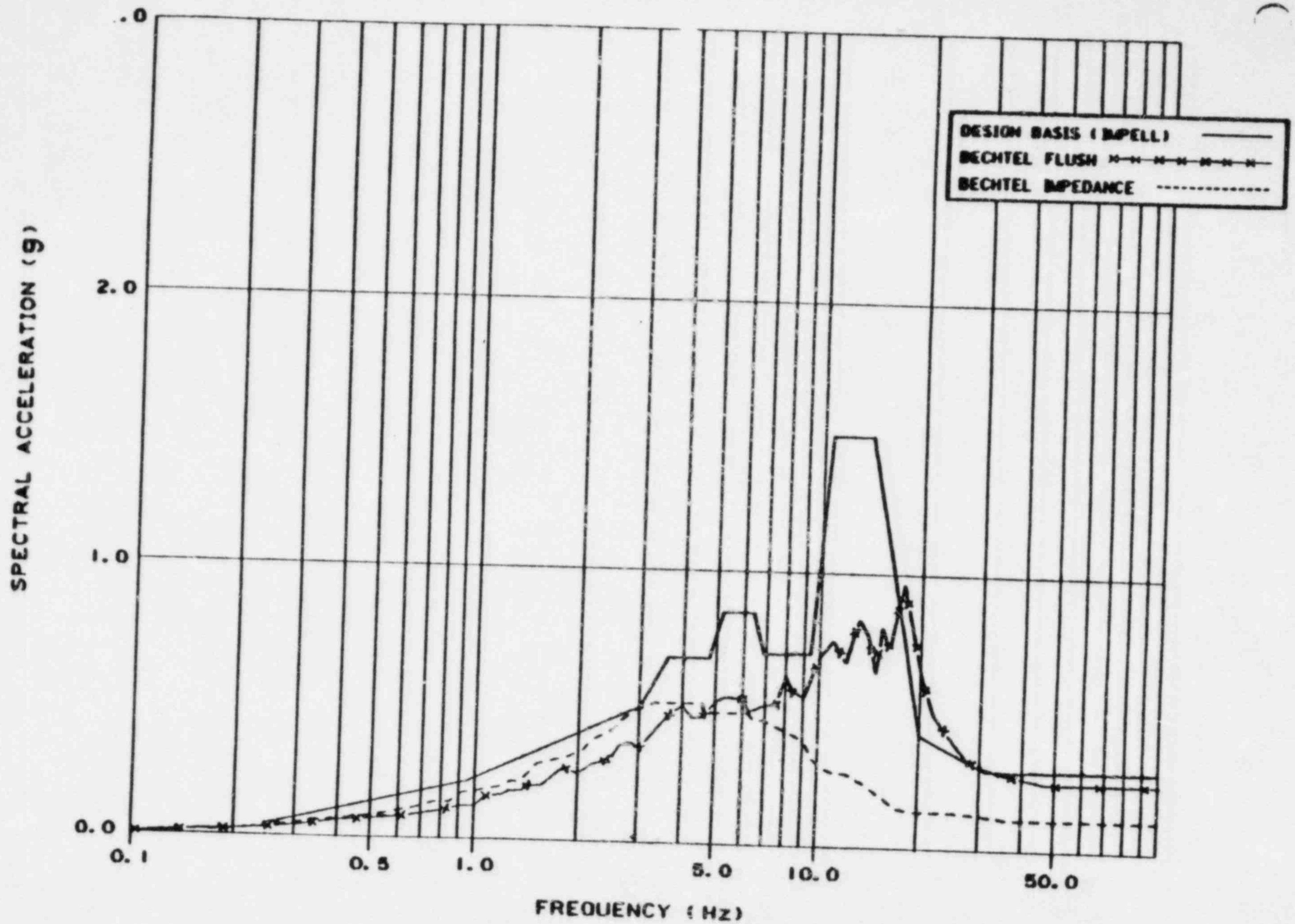


FIGURE A-13-37

RESPONSE SPECTRA COMPARISON,
AUXILIARY BUILDING AT ELEV. 178'-0",
VERTICAL, OBE, 2% DAMPING

OPE CREEK UNIT 1
AD. L36A, REV 1
4/20/84

DRAFT

Response to NRC Audit

Meeting Date: January 10, 1984

Revised Response
Revision 1
July 10, 1984

Question No.: A.16

<u>QUESTION</u>: Provide calculations of ductility ratios due to pipe break for key elements.

<u>RESPONSE</u>: FSAR Section 3.8.4.8.2 discusses the allowable ductility ratios used for the design of pipe whip restraints. For flexure in beams, an allowable ductility ratio of 20 is used.

As discussed with the NRC Staff, originally the majority of the pipe whip restraints had ductility ratios less than or equal to 10. However, the ductility ratios for approximately 25% of the pipe whip restraints exceeded 10 under the original design basis. These restraints have been reevaluated based on as-built conditions, final pipe break loads and actual hot gap requirements. This reevaluation revealed that all flexural members for pipe whip restraints have an actual ductility ratio of less than or equal to 10.

FSAR Section 3.8.4.8.2 will be revised to reflect compliance with SRP 3.5.3 for actual ductility ratios of flexural members.

Response to NRC Audit

Meeting Date: January 10, 1984

Question No.: B-5

Revised Response
Revision 1
July 10, 1984

DRAFT

QUESTION: Provide example calculation for combination of N-S, E-W, and vertical responses.

RESPONSE: Example calculation was provided in the original response to this question.

ADDITIONAL INFORMATION REQUESTED: Provide summary tables showing the contributions to the in-plane response due to out-of-plane excitation for 3 orthogonal directions. Two tables to be provided for both N-S and E-W responses.

RESPONSE: Tables 1 and 2 summarize the N-S and E-W response due to N-S, E-W, and vertical base motions for Reactor Building Unit 1, SSE case. Tables 3 and 4 provide similar information for the OBE case. Individual contributions and the resultant response maxima using the SRSS procedure are listed for selected elements in the Reactor Building mathematical model. As included in the original response, the out-of-plane response maxima (shear and moment) were found to have no significant contribution to the in-plane response maxima values.

TABLE 1

REACTOR BUILDING
OUT-OF-PLANE RESPONSE
SAFE SHUTDOWN EARTHQUAKE

Revised Response
January 10/B-5

Element Number	Variable	N-S Response: N-S Base Motion (A)	N-S Response: E-W Base Motion (B)	N-S Response: Vertical Base Motion (C)	SRSS (D)	Ratio (D)/(A)
1	Shear	9.139×10^2	1.746×10^1	2.053×10^2	9.368×10^2	1.03
	Moment	8.988×10^3	1.663×10^2	2.282×10^3	9.275×10^3	1.03
7	Shear	1.405×10^4	2.358×10^2	1.324×10^3	1.411×10^4	1.01
	Moment	9.796×10^5	1.646×10^4	1.856×10^5	9.972×10^5	1.02
11	Shear	2.180×10^4	3.490×10^2	1.714×10^3	2.187×10^4	1.01
	Moment	3.182×10^5	1.027×10^5	5.160×10^4	3.383×10^5	1.06
15	Shear	2.558×10^4	4.188×10^2	1.103×10^3	2.561×10^4	1.00
	Moment	2.653×10^6	4.347×10^4	1.070×10^5	2.656×10^6	1.00
19	Shear	4.502×10^4	2.635×10^3	2.949×10^3	4.519×10^4	1.00
	Moment	4.933×10^6	2.904×10^5	2.520×10^5	4.948×10^6	1.00
21	Shear	5.699×10^4	2.192×10^3	4.310×10^3	5.719×10^4	1.00
	Moment	6.775×10^6	2.881×10^5	3.830×10^5	6.792×10^6	1.00
33	Shear	1.523×10^3	5.135×10^1	5.328×10^2	1.614×10^3	1.06
	Moment	2.331×10^4	4.271×10^3	4.800×10^3	2.418×10^4	1.04
35	Shear	3.457×10^3	1.018×10^2	1.093×10^3	3.627×10^3	1.05
	Moment	8.488×10^4	1.374×10^4	1.830×10^4	8.791×10^4	1.04
37	Shear	9.890×10^3	2.031×10^2	1.022×10^3	9.945×10^3	1.01
	Moment	3.518×10^5	1.983×10^4	2.270×10^4	3.531×10^5	1.00
39	Shear	3.156×10^4	4.933×10^2	2.417×10^3	3.166×10^4	1.00
	Moment	1.181×10^6	1.984×10^4	7.040×10^4	1.183×10^6	1.00
42	Shear	1.280×10^4	1.790×10^3	1.153×10^3	1.298×10^4	1.01
	Moment	9.634×10^5	3.389×10^4	5.300×10^4	9.655×10^5	1.00
44	Shear	1.515×10^4	2.805×10^3	1.420×10^3	1.547×10^4	1.02
	Moment	1.471×10^6	5.650×10^4	1.020×10^5	1.476×10^6	1.00

Note: 1. Units: Kip, Ft.

TABLE 2

REACTOR BUILDING
OUT-OF-PLANE RESPONSE
SAFE SHUTDOWN EARTHQUAKE

Revised Response
January 10/B-5

Element Number	Variable	E-W Response: E-W Base Motion (A)	E-W Response: N-S Base Motion (B)	E-W Response: Vertical Base Motion (C)	SRSS (D)	Ratio (D)/(A)
1	Shear	8.829×10^2	1.164×10^1	8.628×10^1	8.872×10^2	1.01
	Moment	8.264×10^3	1.186×10^2	8.103×10^2	8.304×10^2	1.00
7	Shear	1.323×10^4	2.203×10^2	8.034×10^2	1.326×10^4	1.00
	Moment	8.504×10^5	1.267×10^4	6.400×10^4	8.529×10^5	1.00
11	Shear	1.698×10^4	4.092×10^2	3.796×10^2	1.699×10^4	1.00
	Moment	1.583×10^6	2.653×10^5	6.930×10^4	1.607×10^6	1.02
15	Shear	4.918×10^4	5.880×10^2	9.377×10^2	4.919×10^4	1.00
	Moment	8.257×10^5	1.138×10^4	1.230×10^5	8.349×10^5	1.01
19	Shear	6.499×10^4	6.400×10^2	1.204×10^3	6.500×10^4	1.00
	Moment	3.078×10^6	4.853×10^5	1.660×10^5	3.120×10^6	1.01
21	Shear	7.055×10^4	6.283×10^2	1.440×10^3	7.057×10^4	1.00
	Moment	5.337×10^6	1.837×10^5	1.990×10^5	5.344×10^6	1.00
33	Shear	1.601×10^3	5.216×10^1	3.740×10^2	1.645×10^3	1.03
	Moment	1.593×10^4	2.022×10^3	3.370×10^3	1.641×10^4	1.03
35	Shear	3.491×10^3	8.271×10^1	7.104×10^2	3.564×10^3	1.02
	Moment	6.442×10^4	4.509×10^3	1.240×10^4	6.576×10^4	1.02
37	Shear	5.981×10^3	1.188×10^2	7.497×10^2	6.029×10^3	1.01
	Moment	1.025×10^5	9.354×10^3	2.190×10^4	1.052×10^5	1.03
39	Shear	1.482×10^4	1.707×10^2	4.034×10^2	1.483×10^4	1.00
	Moment	3.107×10^5	1.200×10^4	3.630×10^4	3.130×10^5	1.01
42	Shear	8.162×10^3	1.084×10^2	1.621×10^2	8.164×10^3	1.00
	Moment	7.449×10^4	6.000×10^3	6.160×10^3	7.498×10^4	1.01
44	Shear	1.055×10^4	1.284×10^2	2.103×10^2	1.055×10^4	1.00
	Moment	2.138×10^5	6.323×10^3	1.350×10^4	2.143×10^5	1.00

Note: 1. Units: Kip, Ft.

TABLE 3

REACTOR BUILDING
OUT-OF-PLANE RESPONSE
OPERATING BASIS EARTHQUAKE

Revised Response
January 10/B-5

Element Number	Variable	N-S Response: N-S Base Motion (A)	N-S Response: E-W Base Motion (B)	N-S Response: Vertical Base Motion (C)	SRSS (D)	Ratio (D)/(A)
1	Shear	8.676×10^2	2.339×10^1	1.283×10^2	8.773×10^2	1.01
	Moment	8.247×10^3	2.239×10^2	1.426×10^3	8.372×10^3	1.02
7	Shear	1.175×10^4	3.322×10^2	8.275×10^2	1.178×10^4	1.00
	Moment	8.243×10^5	2.308×10^4	1.160×10^5	8.327×10^5	1.01
11	Shear	1.515×10^4	4.907×10^2	1.071×10^3	1.520×10^4	1.00
	Moment	2.549×10^5	6.064×10^4	3.225×10^4	2.640×10^5	1.04
15	Shear	1.306×10^4	5.542×10^2	6.894×10^2	1.309×10^4	1.00
	Moment	1.830×10^6	5.553×10^4	6.688×10^4	1.832×10^6	1.00
19	Shear	1.899×10^4	1.561×10^3	1.843×10^3	1.914×10^4	1.01
	Moment	2.873×10^6	1.798×10^5	1.575×10^5	2.883×10^6	1.00
21	Shear	2.406×10^4	1.578×10^3	2.694×10^3	2.426×10^4	1.01
	Moment	3.665×10^6	2.059×10^5	2.394×10^5	3.679×10^6	1.00
33	Shear	6.421×10^2	4.807×10^1	3.330×10^2	7.249×10^2	1.13 (2)
	Moment	7.924×10^3	1.967×10^3	3.000×10^3	8.698×10^3	1.10
35	Shear	1.468×10^3	1.040×10^2	6.831×10^2	1.622×10^3	1.10 (2)
	Moment	3.015×10^4	6.522×10^3	1.144×10^4	3.290×10^4	1.09
37	Shear	4.282×10^3	2.146×10^2	6.388×10^2	4.335×10^3	1.01
	Moment	2.260×10^5	1.090×10^4	1.419×10^4	2.267×10^5	1.00
39	Shear	1.455×10^4	6.914×10^2	1.511×10^3	1.464×10^4	1.01
	Moment	7.580×10^5	2.396×10^4	4.400×10^4	7.597×10^5	1.00
42	Shear	5.381×10^3	8.901×10^2	7.206×10^2	5.502×10^3	1.02
	Moment	5.879×10^5	2.707×10^4	3.313×10^4	5.895×10^5	1.00
44	Shear	6.382×10^3	1.315×10^3	8.875×10^2	6.576×10^3	1.03
	Moment	7.841×10^5	3.660×10^4	6.375×10^4	7.875×10^5	1.00

Notes: 1. Units: Kip, Ft.
2. This is considered insignificant because the shear and moment for this beam are very small.

TABLE 4

REACTOR BUILDING
OUT-OF-PLANE RESPONSE
OPERATING BASIS EARTHQUAKE

Revised Response
January 10/B-5

Element Number	Variable	E-W Response: E-W Base Motion (A)	E-W Response: N-S Base Motion (B)	E-W Response: Vertical Base Motion (C)	SRSS (D)	Ratio (D)/(A)
1	Shear	6.049×10^{2}	1.604×10^{1}	5.393×10^{1}	6.075×10^{2}	1.00
	Moment	5.660×10^{3}	1.571×10^{2}	5.064×10^{2}	5.685×10^{3}	1.00
7	Shear	8.723×10^{3}	2.178×10^{2}	5.021×10^{2}	8.740×10^{3}	1.00
	Moment	5.791×10^{5}	1.391×10^{4}	4.000×10^{4}	5.806×10^{5}	1.00
11	Shear	9.271×10^{3}	6.722×10^{2}	2.373×10^{2}	9.298×10^{3}	1.00
	Moment	9.943×10^{5}	1.798×10^{5}	4.331×10^{4}	1.011×10^{6}	1.02
15	Shear	2.437×10^{4}	6.431×10^{2}	5.861×10^{2}	2.439×10^{4}	1.00
	Moment	5.180×10^{5}	1.235×10^{4}	7.688×10^{4}	5.238×10^{5}	1.01
19	Shear	3.187×10^{4}	7.589×10^{2}	7.525×10^{2}	3.189×10^{4}	1.00
	Moment	1.517×10^{6}	3.060×10^{5}	1.038×10^{5}	1.551×10^{6}	1.02
21	Shear	3.431×10^{4}	8.172×10^{2}	9.000×10^{2}	3.433×10^{4}	1.00
	Moment	2.628×10^{6}	1.406×10^{5}	1.244×10^{5}	2.635×10^{6}	1.00
33	Shear	7.598×10^{2}	3.297×10^{1}	2.338×10^{2}	7.956×10^{2}	1.05
	Moment	7.889×10^{3}	9.159×10^{2}	2.106×10^{3}	8.217×10^{3}	1.04
35	Shear	1.679×10^{3}	5.966×10^{1}	4.440×10^{2}	1.738×10^{3}	1.04
	Moment	3.159×10^{4}	1.601×10^{3}	7.750×10^{3}	3.257×10^{4}	1.03
37	Shear	2.920×10^{3}	8.827×10^{1}	4.686×10^{2}	2.959×10^{3}	1.01
	Moment	6.786×10^{4}	1.183×10^{4}	1.369×10^{4}	7.023×10^{4}	1.04
39	Shear	7.333×10^{3}	1.910×10^{2}	2.521×10^{2}	7.340×10^{3}	1.00
	Moment	2.000×10^{5}	8.248×10^{3}	2.269×10^{4}	2.015×10^{5}	1.01
42	Shear	4.065×10^{3}	1.018×10^{2}	1.013×10^{2}	4.068×10^{3}	1.00
	Moment	3.724×10^{4}	3.039×10^{3}	3.850×10^{3}	3.756×10^{4}	1.01
44	Shear	5.111×10^{3}	1.129×10^{2}	1.314×10^{2}	5.117×10^{3}	1.00
	Moment	1.045×10^{5}	5.758×10^{3}	8.438×10^{3}	1.050×10^{5}	1.01

Note: 1. Units: Kip, Ft.

Response to NRC Audit Meeting

Date: January 10, 1984

Question No.: B-9

Revised Response
Revision 1
6/30/84

DRAFT

QUESTION: Provide calculation showing drywell stick model development (provide for one section only).

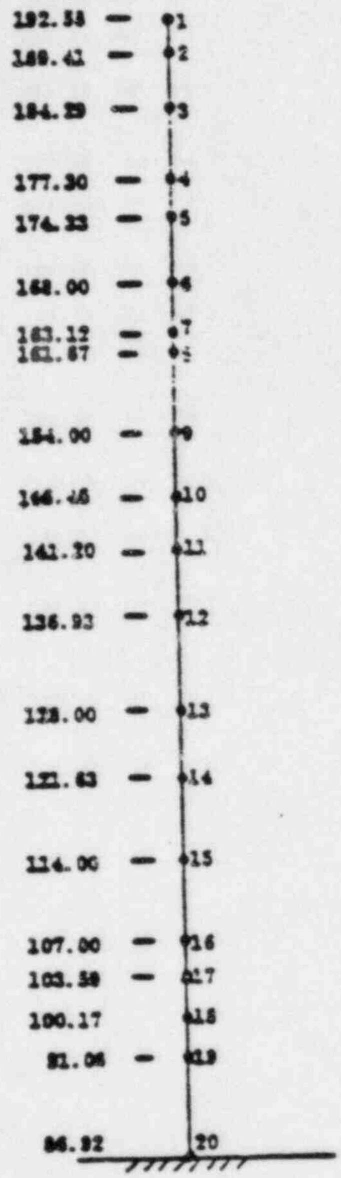
RESPONSE: Calculation is provided in the original response to this question.

ADDITIONAL INFORMATION REQUESTED: Provide clarification of the rotation value used in determining the equivalent beam properties.

RESPONSE: The rotational value ($\theta = 2.35 \times 10^{-7}$) determined in the sample calculation provided as Attachment I to this revised response is used to determine the property between elevations 86.94 and 91.06 of the equivalent beam model (Figure 1). This rotation is estimated based on the resultant displacements due to unit load applied at the shear lug location of the more detailed axisymmetric shell model of the drywell (Figure 2).

The best estimate of this rotation was arrived at based on examining the displacement pattern (Figure 3) of the shell model between node 82 (node 19 of beam model) and neighboring nodes.

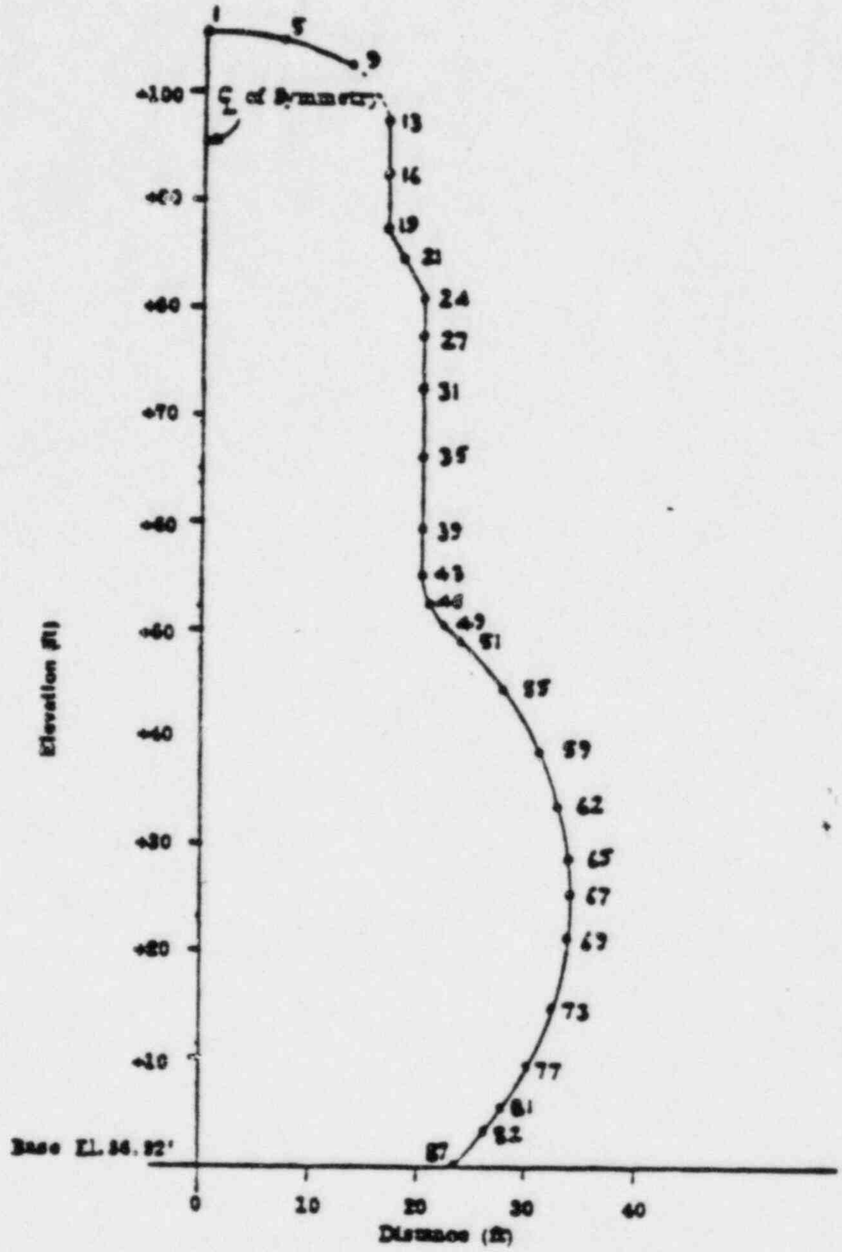
The rotational value of 2.39×10^{-7} is more representative of the rigid body rotation of the drywell stick between its base and approximately 50' above the base, as can be seen from Figure 3. However, a slightly lower value of rotation at node 82 is obtained based on calculations of rotation at this node and nodes 81 and 83 of the detailed shell model. The best estimate of this rotation is obtained as an average of these rotational values. This sample calculation is included as Attachment I to this revised response. Furthermore, the difference between the two values of rotation is less than 2% which is considered insignificant.



DRYWELL EQUIVALENT BEAM MODEL

TOTAL NODES = 20

FIGURE 1

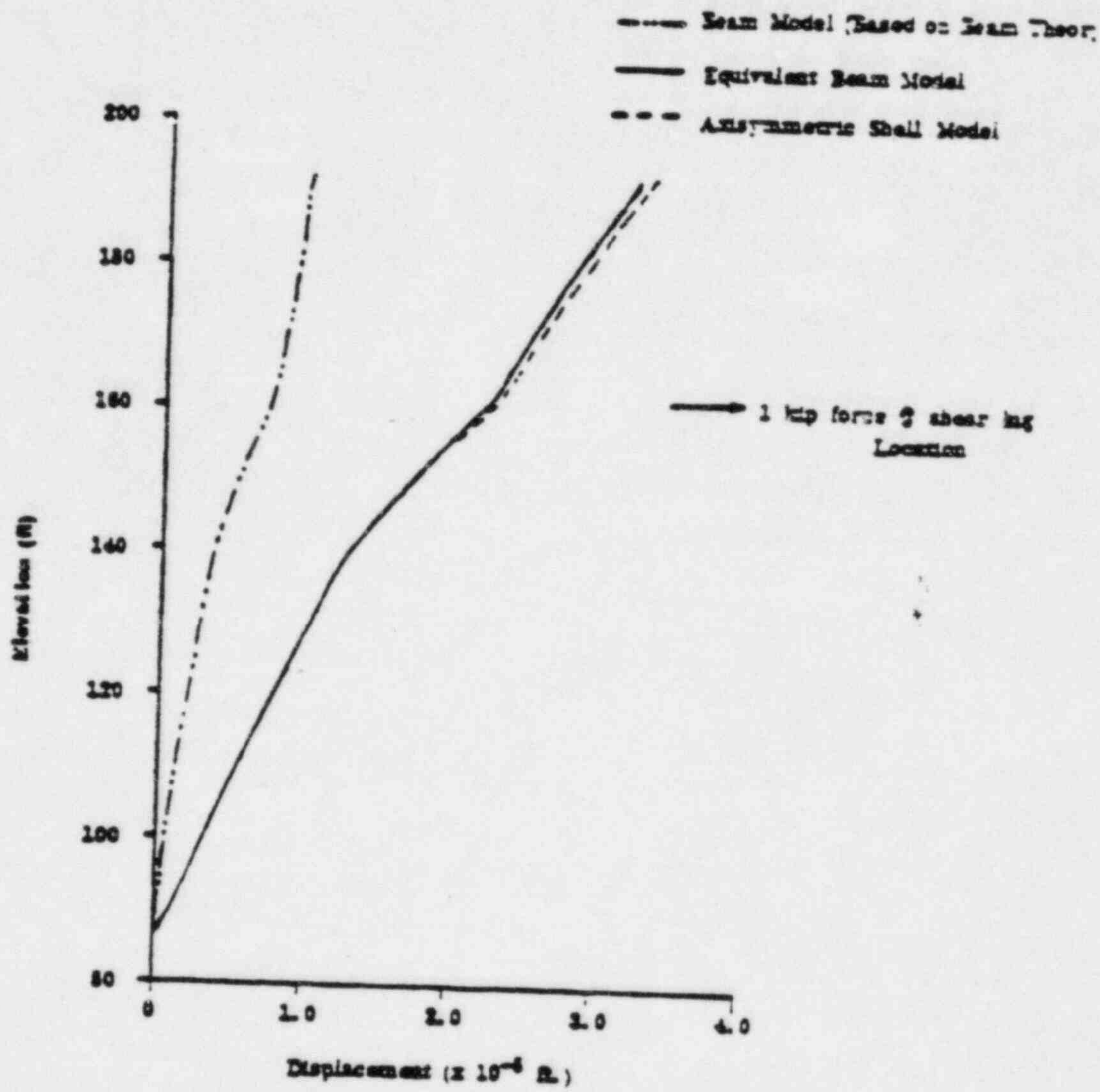


DRYWELL AXISYMMETRIC MODEL

TOTAL NODES = 87

FIGURE 2

STATIC DISPLACEMENT COMPARISON



DRYWELL (FREE STANDING)

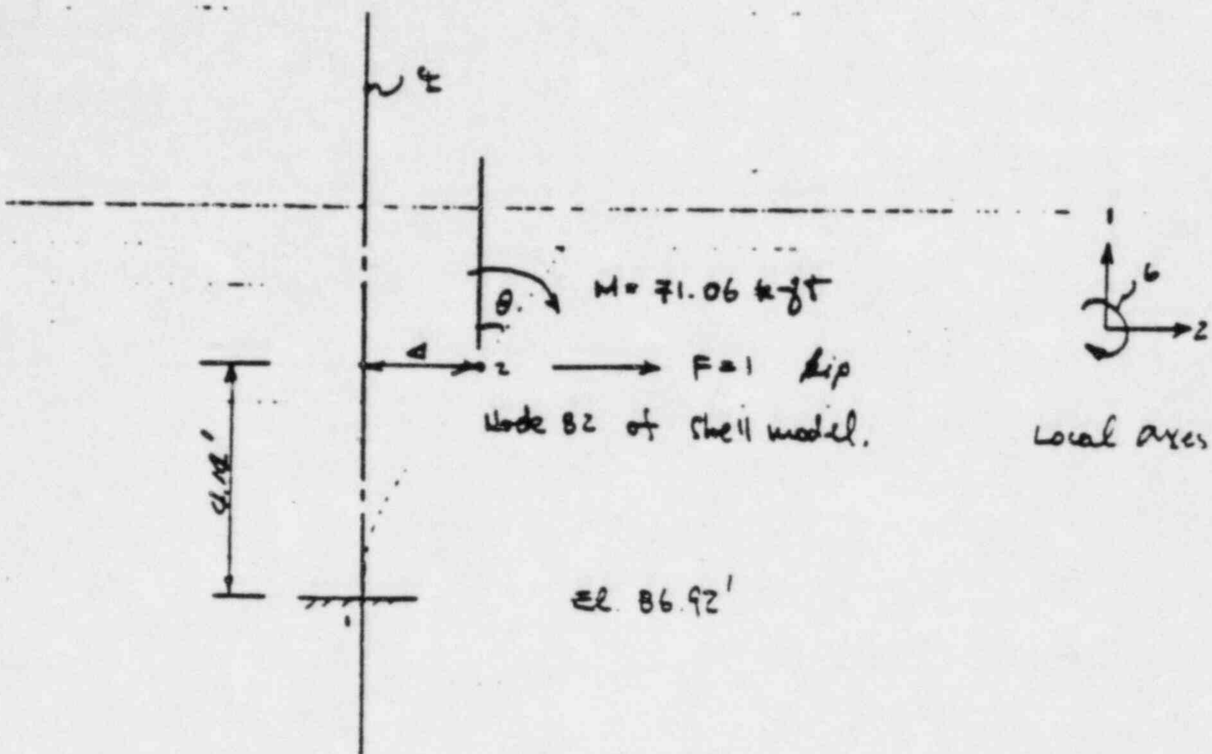
FIGURE 3

ATTACHMENT I

Development of Equivalent Beam Model Properties

Equivalent Beam Model

Change node 19 coordinate to (0, 0, 91.06).



Δ @ node 82 (90°) = 0.96009 × 10^{-6} ft (4.16' from drywell base)

Δ @ node 50 (90°) = 0.11921 × 10^{-4} ft (50.0' from drywell base)

$$\therefore \quad \theta = \frac{0.11921 \times 10^{-4} - 0.96009 \times 10^{-6}}{50 - 4.16} = 2.390 \times 10^{-7} \ *$$

✓

Ref: J.S. Przemieniecki, "Theory of Matrix Structural Analysis", PP79

$$\left.\begin{aligned} S_8 &= \frac{12EI}{\ell^3(1+\phi)}\Delta - \frac{6EI}{\ell^2(1+\phi)}\theta \\ S_{12} &= -\frac{6EI}{\ell^2(1+\phi)}\Delta + \frac{(4+\phi)EI}{\ell(1+\phi)}\theta \end{aligned}\right\} \quad (1)$$

* If we use 81 & 82, $\theta = \frac{0.11345 \times 10^{-5} - 0.96009 \times 10^{-6}}{5.08 - 4.16} = 2.387 \times 10^{-7}$ ✓

82 & 83 $\theta = 2.209 \times 10^{-7}$

ITEM	CLIENT
Drywell Beam Model	PG&E
	PROJECT Hatch? Cook?
	JOB NO. 140.0012
BY MH DATE 2/10/77 CKD GCH DATE 2/11/77	ITEM NO. SHT 3 OF 11

$(1)_1 \Rightarrow \qquad S_B = \frac{6EI}{\ell^2(1+\phi)} \left\{ \frac{2}{\ell}\Delta - \theta \right\}$

$$1+\phi = \frac{6EI}{S_B \ell^2} \left\{ \frac{2}{\ell}\Delta - \theta \right\}$$

or

$$\phi = \frac{6EI}{S_B \ell^2} \left(\frac{2}{\ell}\Delta - \theta \right) - 1$$

$(1)_2 \Rightarrow \qquad S_{12} = -\frac{6EI}{\ell^2(1+\phi)}\Delta + \frac{3EI}{\ell(1+\phi)}\theta + \frac{EI}{\ell}\theta$

$$= -\frac{\ell}{2} S_B + \frac{EI}{\ell}\theta$$

or

$$I = \frac{\ell}{E\theta} \left(S_{12} + \frac{\ell}{2} S_B \right)$$

✓

∴

$$\left.\begin{aligned} I &= \frac{\ell}{E\theta} \left(S_{12} + \frac{\ell}{2} S_B \right) \\ \phi &= \frac{6EI}{S_B \ell^2} \left(\frac{2}{\ell}\Delta - \theta \right) - 1 \end{aligned}\right\} \quad (2)$$

✓

ITEM: Drywell Beam Model	CLIENT: PSEG
	PROJECT: Hope Creek
	JOB NO: 140.0012
BY: HH / DATE: 2/10/77 / CHD: GCH / DATE: 2/11/77	ITEM NO: / SHT 4 OF 11

EDS NUCLEAR

Use $\Delta = 0.960 \times 10^{-6}$ $\theta = 2.35 \times 10^{-7}$

$S_B = 1.0$ $S_\theta = 71.0$

$E = 4.0 \times 10^{6}$ $\nu = 0.3$

Then,

$I = 321.8$

$\Phi = 102.1$

Now, $\Phi = \dfrac{12EI}{\ell^2 G A_s}$ $\therefore$ $A_s = \dfrac{24(1+\nu)I}{\ell^2 \phi}$ ✓

$\therefore$ $A_s = 5.74$

$\therefore$ <u>1st try</u>, $\left.\begin{array}{l} I = 322 \text{ ft}^4 \\ A_s = 5.74 \text{ ft}^2 \end{array}\right\}$

<u>Check</u>

$\Phi = \dfrac{12EI}{\ell^2 G A_s} = \dfrac{24(1+\nu)I}{\ell^2 A_s} = 102.12$ ✓

$\therefore$ $\left.\begin{array}{l} 0.528(E\Delta) - 1.053(E\theta) = 1 \\ -1.053(E\Delta) + 80.041(E\theta) = 71 \end{array}\right\}$

$\therefore E\Delta = \dfrac{80.041 + (1.053)(71)}{(0.528)(80.041) - (1.053)^2} = 3.839$

$E\theta = \dfrac{(0.528)(71) + 1.053}{(0.528)(80.041) - (1.053)^2} = 0.939$

$\therefore$ $\left.\begin{array}{l} \Delta = 9.597 \times 10^{-7} \\ \theta = 2.349 \times 10^{-7} \end{array}\right\}$ ✓

ITEM Drywell Beam Model	CLIENT PSE&G
	PROJECT Hope Creek
	JOB NO 140.0012
BY HM DATE 6/10/77 CKD GCH DATE 6/11/77	ITEM NO SHT 5 OF 11

EDS NUCLEAR: 220 MONTGOMERY ST. • SAN FRANCISCO, CALIFORNIA 9410

Meeting Date: January 11, 1984

Question No.: A.7

Revised Response
Revision 1
6/30/84

DRAFT

QUESTION : Provide a simplified calculation for overturning moment of reactor building foundation mat.

RESPONSE : Appendix 3H of FSAR presents the calculated factors of safety against floatation, sliding, and overturning. The factor of safety against overturning was computed using the energy balance method. This response examines the reactor building overturning stability using the conventional method.

The controlling load combination for the overturning stability check is $D + H + E_s$ as discussed in Appendix 3H of FSAR. Stability against overturning will be ensured by the dead weight of the structures and the passive soil pressure associated with the embedded portion of the structures. The buoyant force, which tends to increase the overturning potential of the structure, has been taken into account. Overturning for the reactor building, due to North-South earthquake, has been determined to be the most critical case.

Two factors of safety have been examined in this response which are as follows:

1. Factor of safety against global overturning.
2. Factor of safety against toe contact pressure failure.

The factor of safety against global overturning failure is defined as the ratio of the resisting moment under earthquake conditions (M_r) to the overturning moments (M_o) on the foundation:

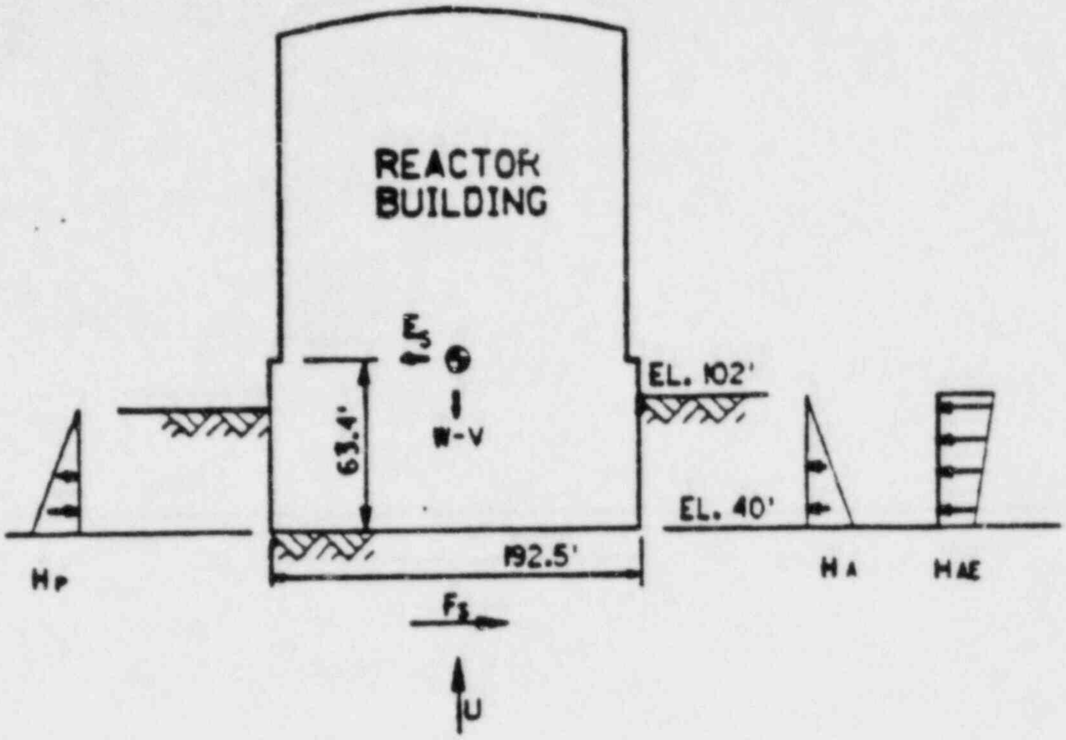
$$F.S. = M_r/M_o$$

As indicated in Figure A.7-1, the computed factor of safety against overturning is 1.61 which is greater than the required factor of safety of 1.1.

The factor of safety against toe failure is defined as the ratio of the allowable maximum soil bearing pressure to the calculated dynamic toe pressure.

$$F.S. = P\ allow/P\ actual$$

The computed factor of safety as indicated in the FSAR Appendix 3H (Section IV) is 3.60 which is greater than the required factor of safety of 1.1.



FORCE TYPE	RESISTING				DRIVING			
	NOTATION	ARM	FORCE	MOMENT	NOTATION	ARM	FORCE	MOMENT
DYNAMIC SOIL	—	NEGLECTED			H_{AE}	31.0	18,190	563,900
STATIC SOIL	H_P	20.7	210,000	4,347,000	H_A	20.7	7,675	158,900
RESULTANT VERTICAL LOAD	—	96.3	65,270	6,286,000	—	—	—	—
DYNAMIC INERTIA	—	—	—	—	E_S	63.4	92,980	5,895,000
FRICTION	F_S	0	52,870	0	—	—	—	—
TOTAL	—			10,633,000	—			6,618,000

$$(F.S.)_{OTM} = \frac{\text{RESISTING MOMENT}}{\text{DRIVING MOMENT}} = \frac{10,633,000}{6,618,000} = 1.61 > 1.1$$

NOTE:
W = DEAD WEIGHT = 464,900^K
V = VERTICAL DYNAMIC INERTIA = 37,130^K
U = UPLIFT DUE TO GROUNDWATER AND EXCESS PORE WATER PRESSURE = 362,500^K

ALL UNITS ARE GIVEN IN KIPS AND FEET

FIGURE - A.7-1

DRAFT

Revised Response

Revision 1
6/30/84

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: A.8

QUESTION:

Are BSAP element size limitations satisfied for the foundation mat model and the drywell shield wall model.

RESPONSE:

The BSAP program user's manual does not specify element size limitations. Element aspect ratio is a design parameter which varies with model configuration and analysis accuracy. Better accuracy is achieved with an aspect ratio close to 1.

The foundation mat and the drywell shield wall were originally analyzed using the BSAP computer program. During evaluation of Unit 2 cancellation, the drywell shield wall has been reanalyzed using the ASHSD computer program (ref.: FSAR, Appendix 3A). Brick elements were used for both the foundation mat and drywell shield wall models. For the foundation mat analysis, the element aspect ratios were approximately 5 in high stress areas. For the drywell shield wall analysis, the element aspect ratios were less than 2. Generic studies e.g., Desai, C.S., and Abel, J.F., Introduction to the Finite Element Method Van Nostrand Reinhold Co., N.Y., 1972, have shown that an aspect ratio of less than 5 or 8 gives errors of less than 10 percent or 15 percent, respectively, as compared to the bench mark solution. Sufficient design margin exists to justify this degree of error.

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: A-12

Revised Response
Revision 1
6/30/84

DRAFT

QUESTION: Justify the 12 Hz. cut-off frequency for SSI analysis.

RESPONSE: Two independent studies have been performed to justify the 12 Hz. cut-off frequency: a design base evaluation performed by Impell and a confirmatory evaluation by Bechtel. These studies are described separately below.

A. DESIGN BASE ANALYSIS

The selection of a cut-off frequency value was based on two primary considerations:

1. For the particular Hope Creek site, the evaluation of the highest shear wave frequency that can realistically be transmitted through the soil medium.

2. The contribution of the high frequency components of the input free-field (control) motion on the resultant structural response.

DECONVOLUTION ANALYSIS

Two cut-off frequency values were selected for consideration and study: 12 and 20 Hz. An operating basis earthquake was selected for the study, due to its lower peak acceleration level. Because of the nonlinear characteristics of the soil, the lower excitation level will result in stiffer soil properties than for the SSE level excitation, with the soil thus capable of transmitting higher frequency waves. This case will then be more critical for establishing a cut-off frequency value than the SSE.

A soil column, representing the Hope Creek free-field soil properties, was first constructed. The mesh refinement was selected such that a wave frequency of 20 Hz. could be transmitted without loss of numerical accuracy. A schematic representation of the soil column model is presented in Figure 1.

The free-field soil column is composed of a series of two-dimensional plane strain elements of unit width modeling the soil properties. The dimensions of the soil column extend between elevations 102.0 feet, corresponding to the elevation at finished grade for the Hope Creek site, down to elevation -300.0 feet, a depth found to be sufficiently deep to include all significant soil-structure interaction effects.

Using the above free-field soil column, a deconvolution analysis of the normalized OBE Regulatory Guide 1.60 synthetic time-history was performed for both 12 and 20 Hz. cut-off frequency values. This normalized Regulatory Guide control motion was input at elevation 40.0 feet, corresponding to the elevation of the bottom of the foundation base mats for the power block area. Deconvoluted time-history response was obtained at the base of the soil column model, corresponding to elevation -300.0 feet.

SOIL-STRUCTURE INTERACTION ANALYSIS

A simplified soil-structure model was developed for the cut-off frequency study. The model consists of a single soil colum, attached to a series of single-degree-of-freedom oscillators representing the Reactor Building structure. A sketch of the model, with the corresponding soil and structural properties, can be seen in Figures 2 and 3.

As in the case of the deconvolution analysis, the soil properties were modeled by a series of two-dimensional plane strain elements of unit width. The soil elements extend from elevation -300.0 feet to elevation 40.0 feet, corresponding to the elevation at the bottom of the foundation base mat. One additional plane strain element was placed between elevations 40.0 feet and 54.0 feet, to simulate the base mat properties. The Reactor Building dynamic proerties for the N-S modes with frequencies up to 20 Hz. were duplicated by a series of single-degree-of-freedom oscillators. The mass properties of these oscillators are drawn from the modal effective mass calculation of the detailed model.

A soil-structure interaction analysis was performed for both a 12 and 20 Hz. cut-off frequency value. The input motions obtained from the deconvolution analyses, were input at elevation -300.0 feet of the simplified interaction model. Using a system direct integration technique, a time-history analysis of the soil-structure system was performed, with time-histories of acceleration being obtained at the base mat level. An evaluation of the influence of the cut-off frequency was obtained by comparison of the derived base mat response spectra for each of the cut-off frequencies.

As demonstrated by Figure 4, the base mat response for the two cut-off frequencies are essentially identified for the frequency range below 12 Hz. For the frequency range of 12 to 20 Hz., however, the response at the base mat does diverge somewhat between the two cut-off frequencies. The 20 Hz. cut-off response exhibits a number of minor peaks, as a result of high frequency components of the bedrock motion. The effect of these minor peaks on structural response is insignicant as verified by the seismic structural analysis described in the following section. The 12 Hz. cut-off analysis, on the otherhand, exhibits a non-amplified response beyond 12 Hz., resulting in a constant spectral acceleration.

In order to verify the adequacy of the simplified model, a comparison with a detailed interaction model was made. A comparison of Figures 4 and 5 reveals that the motion at the base mat for the detailed and simplified models exhibit very similar trends, both with regard to the spectral peak and overall shape of the curves.

SEISMIC STRUCTURAL ANALYSIS

In order to identify the significance of the difference in basemat motion for the cut-off frequencies on the structural response of the Reactor Building, a seismic structural analysis of the Reactor Building was performed using the detailed three-dimensional model.

Using the basemat motions derived from the simplified interaction analyses for a 12 and 20 Hz. cut-off frequency, a modal time-history analysis of the Reactor Building was performed. Response spectra at selected elevations were computed from the resultant floor excitations, and a comparison of the results derived from the two cut-off frequencies was performed.

Figures 6 through 8 present response maxima for shear, moment and torque in the drywell of the Reactor Building, when subjected to each of the base mat excitations. The drywell was selected for comparison of cut-off frequency effects because it is a portion of the reactor pressure boundary, and the design of this structure is particularly critical. The comparison of results for the drywell is representative of other portions of the structure as well. As can be seen from these plots, the response maxima of the structure are virtually independent of the high frequency acceleration components of the base mat motion. Clearly the shear, moment and torque response values for the structure are essentially identical for the two different cut-off frequencies, indicating a dependance only on the low and mid frequency range of the base mat motions.

Response spectra plots at various elevations of the Reactor Building are presented in Figure 9 through 12. As can be seen from these figures, the spectral accelerations in the low and mid frequency range are essentially identical for the two cut-off frequencies. In the frequency range above 12 Hz., there are minor differences at the lower elevations of the Reactor Building, but almost no variation in the upper elevations. However, these minor differences are considered to have no significance on global structural response. For all elevations, the overall trend of the curves is identical, duplicating peak response values and shape of the spectral curves.

B. CONFIRMATORY ANALYSIS

Bechtel also performed a confirmatory independent analysis to evaluate the effect of cut-off frequency on the soil structure interaction analysis results. The North-South soil-structure model was analyzed for the SSE case. The soil model was discretized to have elements which are capable of transmitting frequencies of at least 18 Hz. Two soil-structure interaction analyses, with cut-off frequencies of 12 Hz and 18 Hz, were performed using computer code FLUSH. As shown in the response spectrum comparison plots (Figures 13 to 15), there is practically no effect in increasing the cut-off frequency from 12 Hz to 18 Hz on the response of the soil structure system.

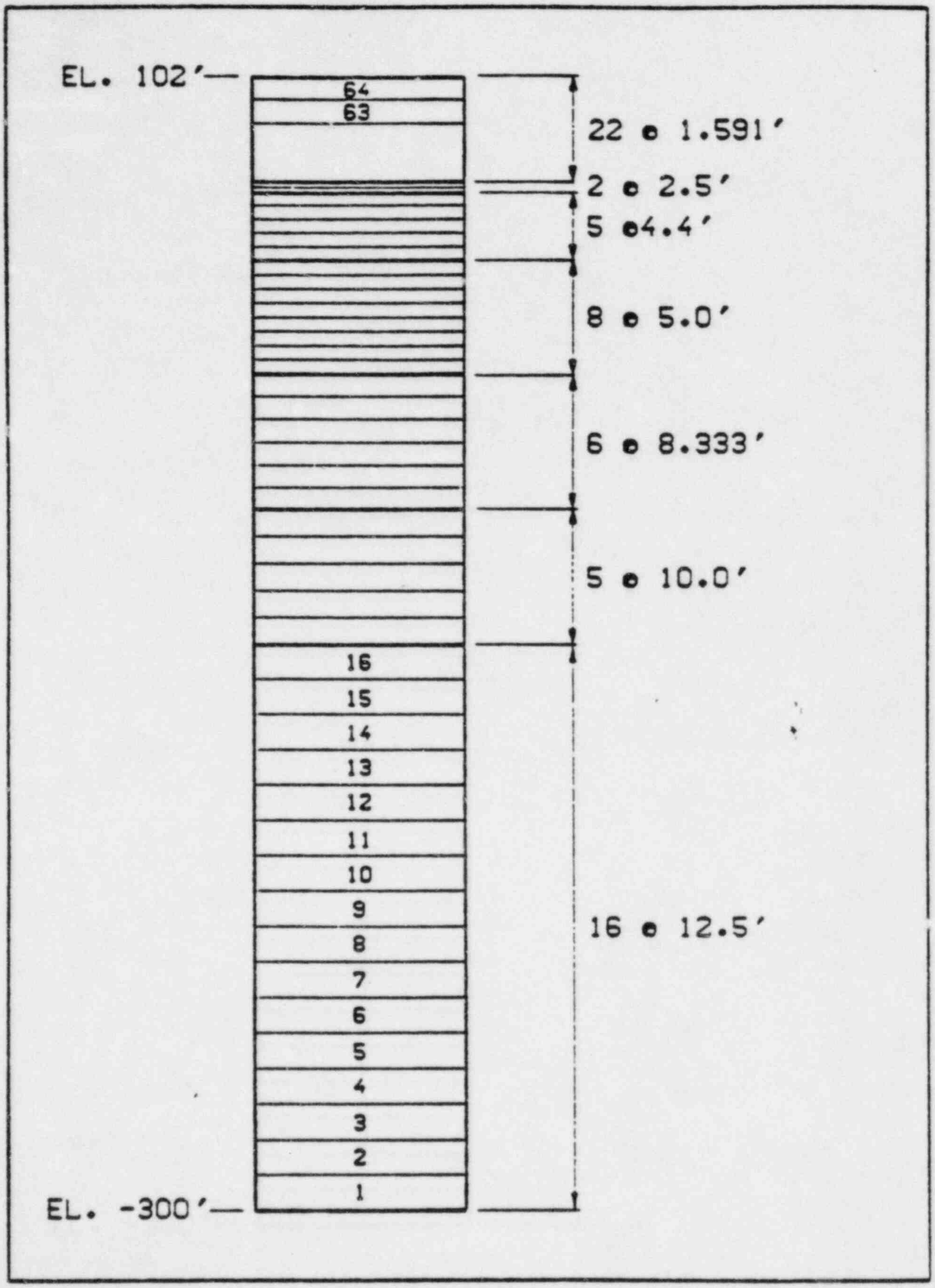
CONCLUSIONS

A study has been performed to evaluate the influence of the cut-off frequency value on the soil-structure interaction analysis subsequent seismic structural analysis for the Hope Creek site. A comparison of response results for a 12 and 20 Hz. cut-off frequency was made for all facets of the analysis.

Comparison of structural response results indicates only minor dependence on the high frequency acceleration components of the input motion, both in the generation of building response maxima and floor response spectra. It would thus be reasonable to assume that either an intermediate cut-off frequency between 12 and 20 Hz. or higher cut-off frequencies of up to 33 Hz. (cut-off for structural response evaluation) would produce only minor deviation from the response results of the 12 Hz. cut-off frequency analysis.

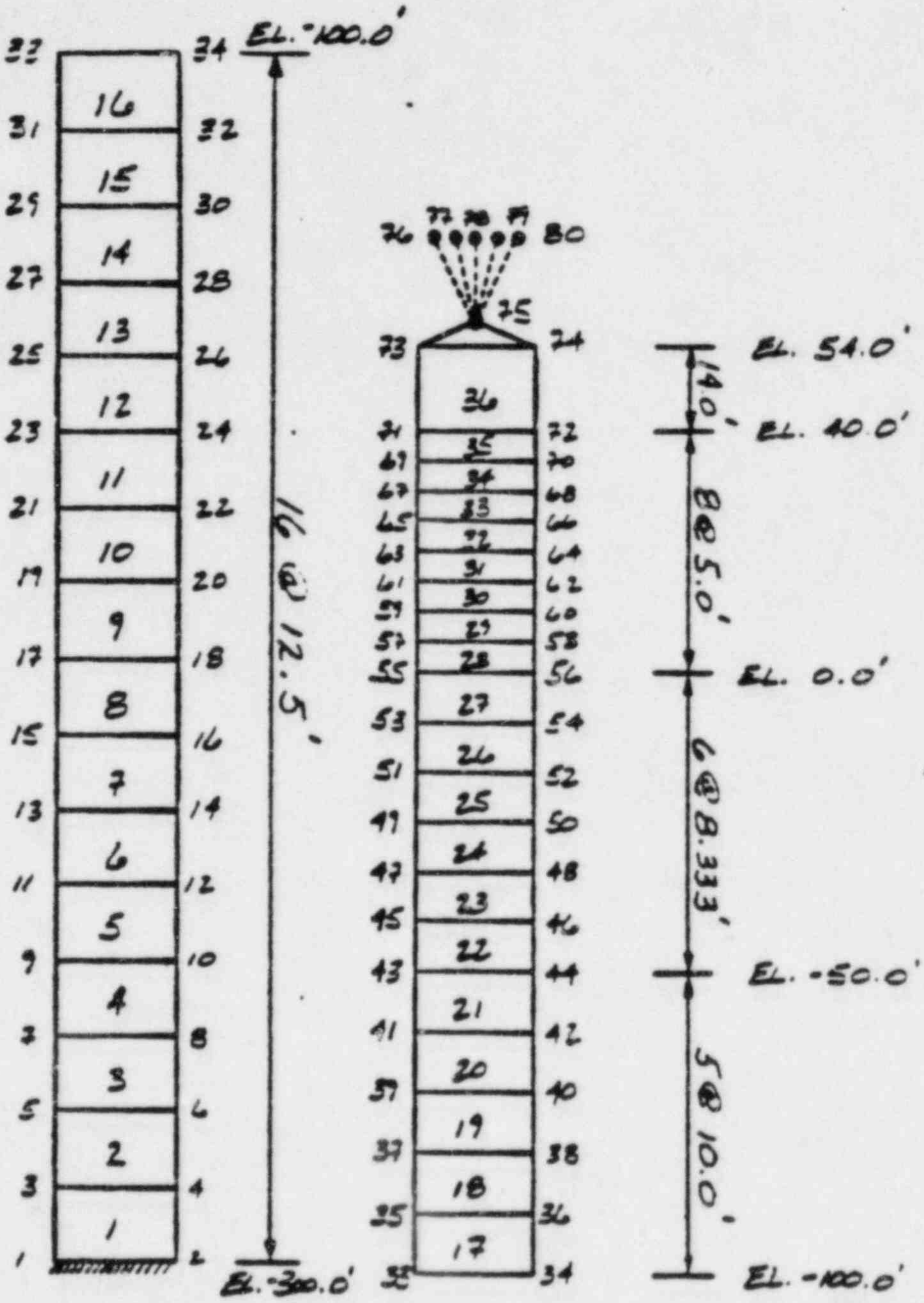
Based on the above considerations, it was found that a cut-off frequency of 12 Hz. for the soil-structure interaction analysis was both physically realistic for the Hope Creek site and, in addition, maintains adequate conservatism with regard to structural response. The use of a 12 Hz. cut-off frequency was thus selected for the Hope Creek analysis.

Furthermore, the adequacy of the 12 Hz. cut-off frequency for SSI analysis has been verified by an independent study performed by Bechtel.



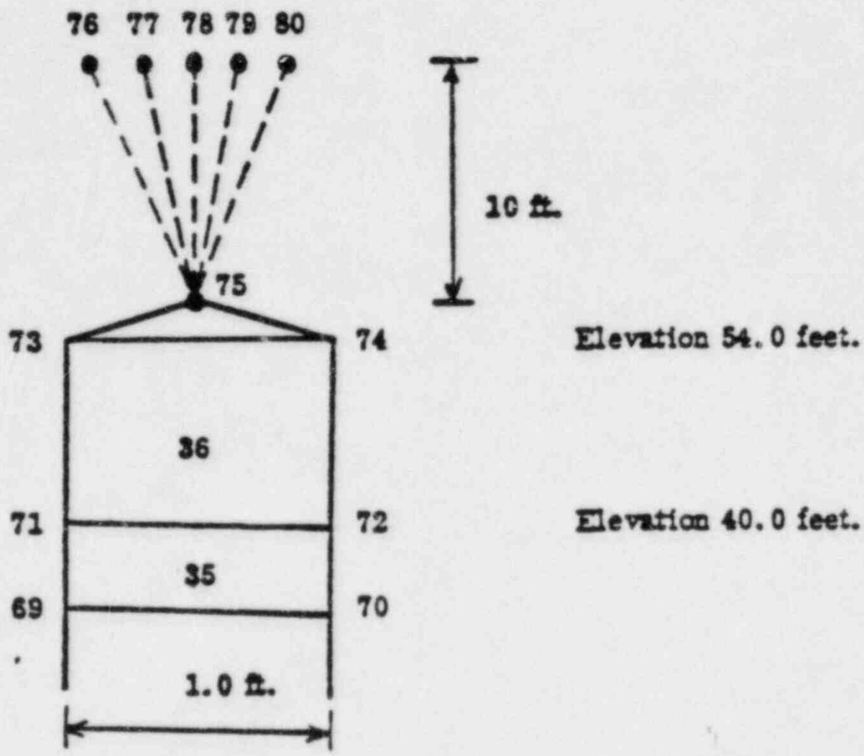
MATHEMATICAL MODEL OF FREE FIELD SOIL COLUMN
FOR DECONVOLUTION ANALYSIS

FIGURE 1



MATHEMATICAL MODEL FOR SOIL-STRUCTURE INTERACTION ANALYSIS

FIGURE 2



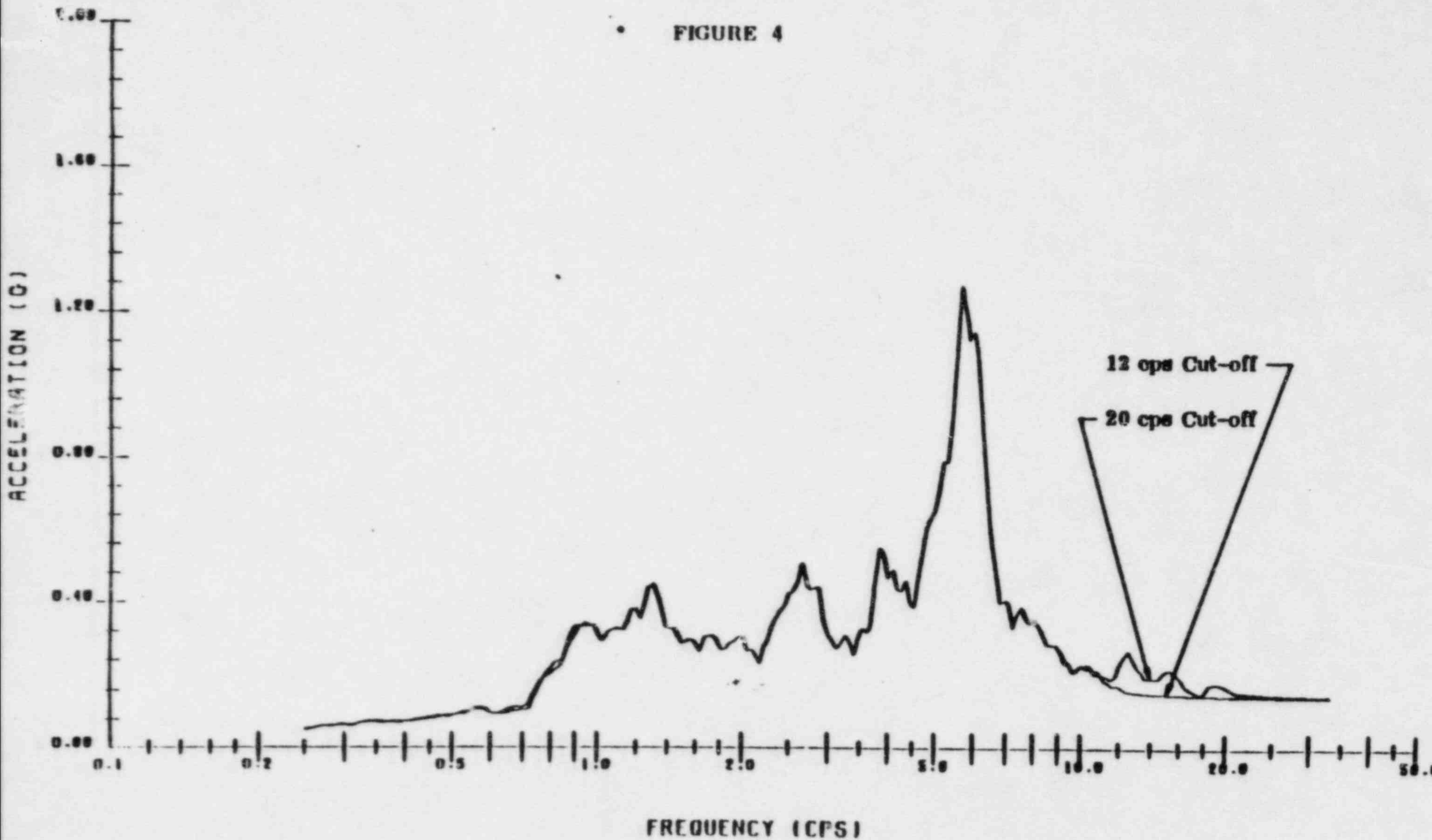
FREQUENCY	MASS PARTICIPATION FACTORS
f_1 = 4.11 Hz.	M_1 = 21.07
f_2 = 9.07 Hz.	M_2 = 10.43
f_3 = 12.08 Hz.	M_3 = 4.75
f_4 = 17.65 Hz.	M_4 = 0.65
f_5 = 19.76 Hz.	M_5 = 2.17

MATHEMATICAL REPRESENTATION OF REACTOR BUILDING
FOR SOIL-STRUCTURE INTERACTION ANALYSIS

FIGURE 3

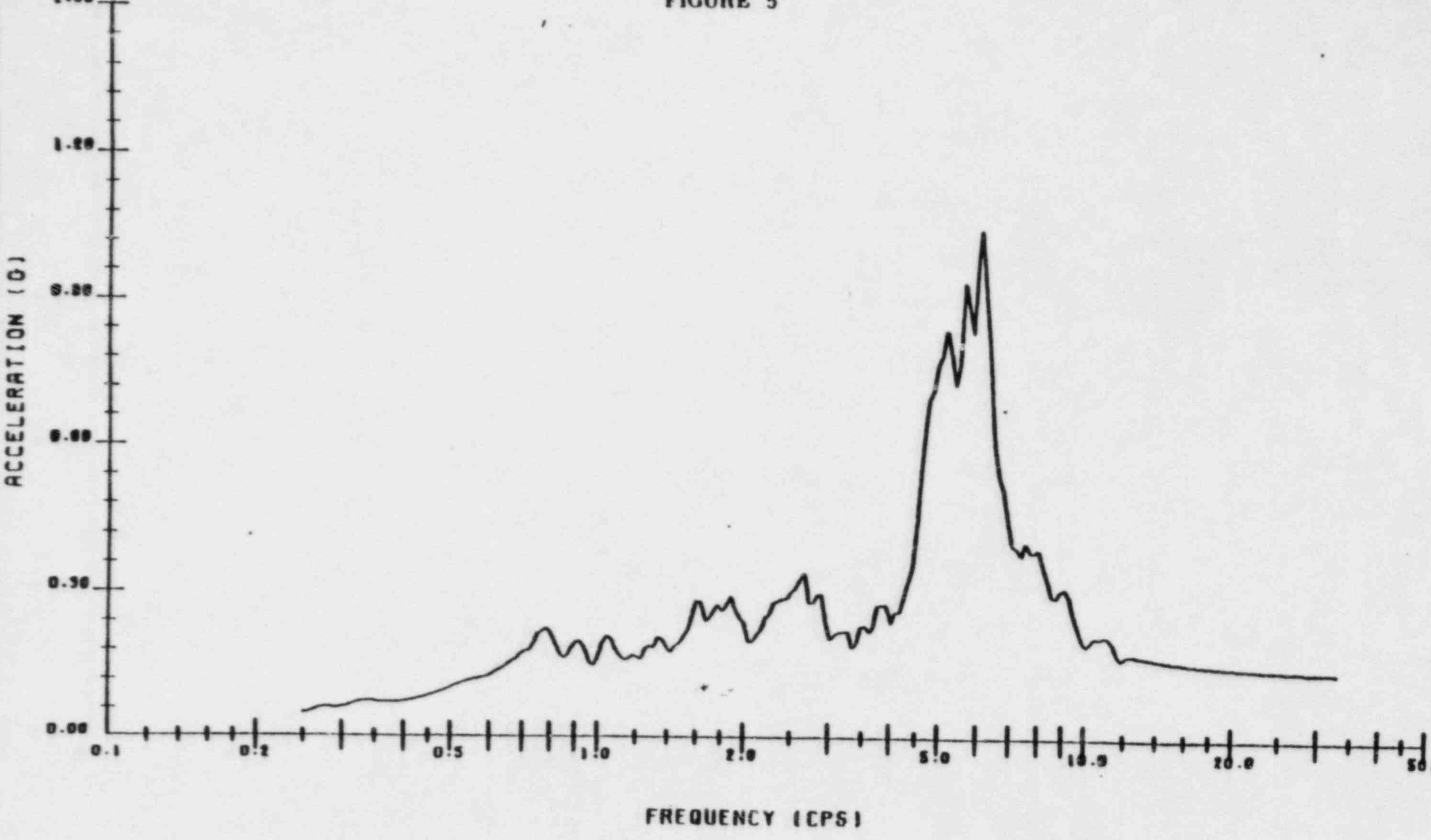
SIMPLIFIED INTERACTION ANAL.- N-6 OBE BASE MAT RESPONSE SPECTRA
(2% DAMPING)

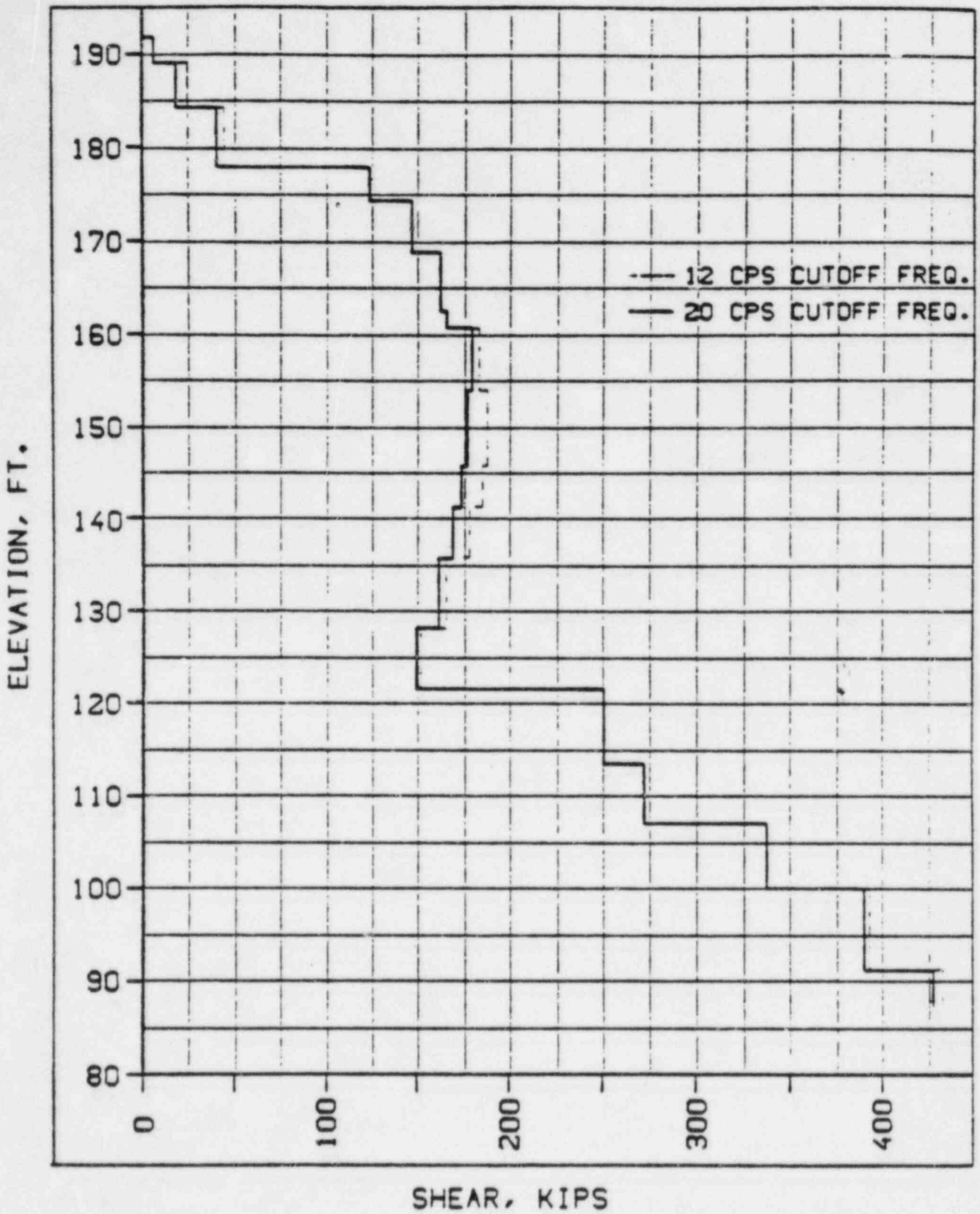
FIGURE 4



DETAILED INTERACTION ANALYSIS: N-S OBE BASE MAT RESPONSE SPECTRUM
(2% DAMPING)

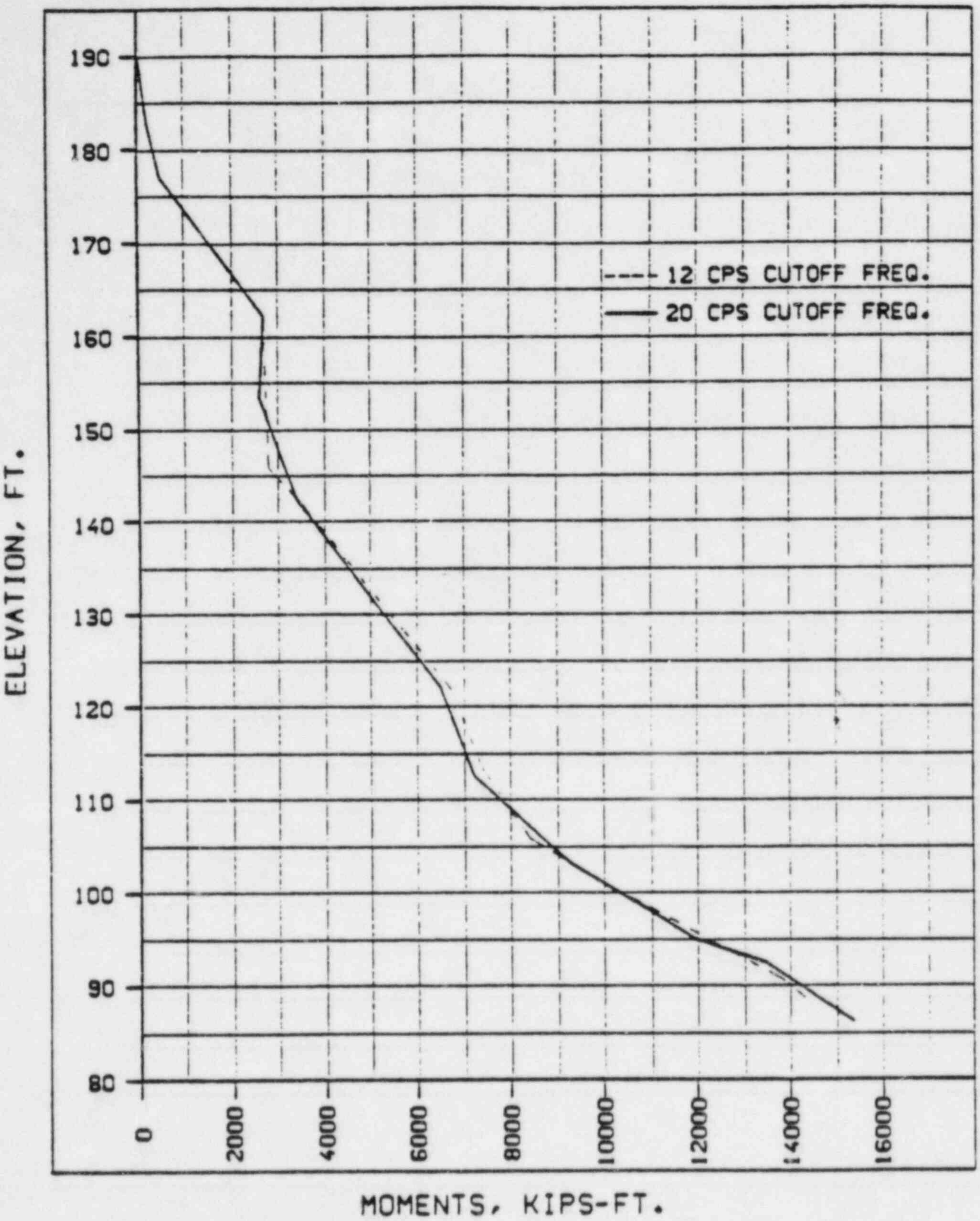
FIGURE 5





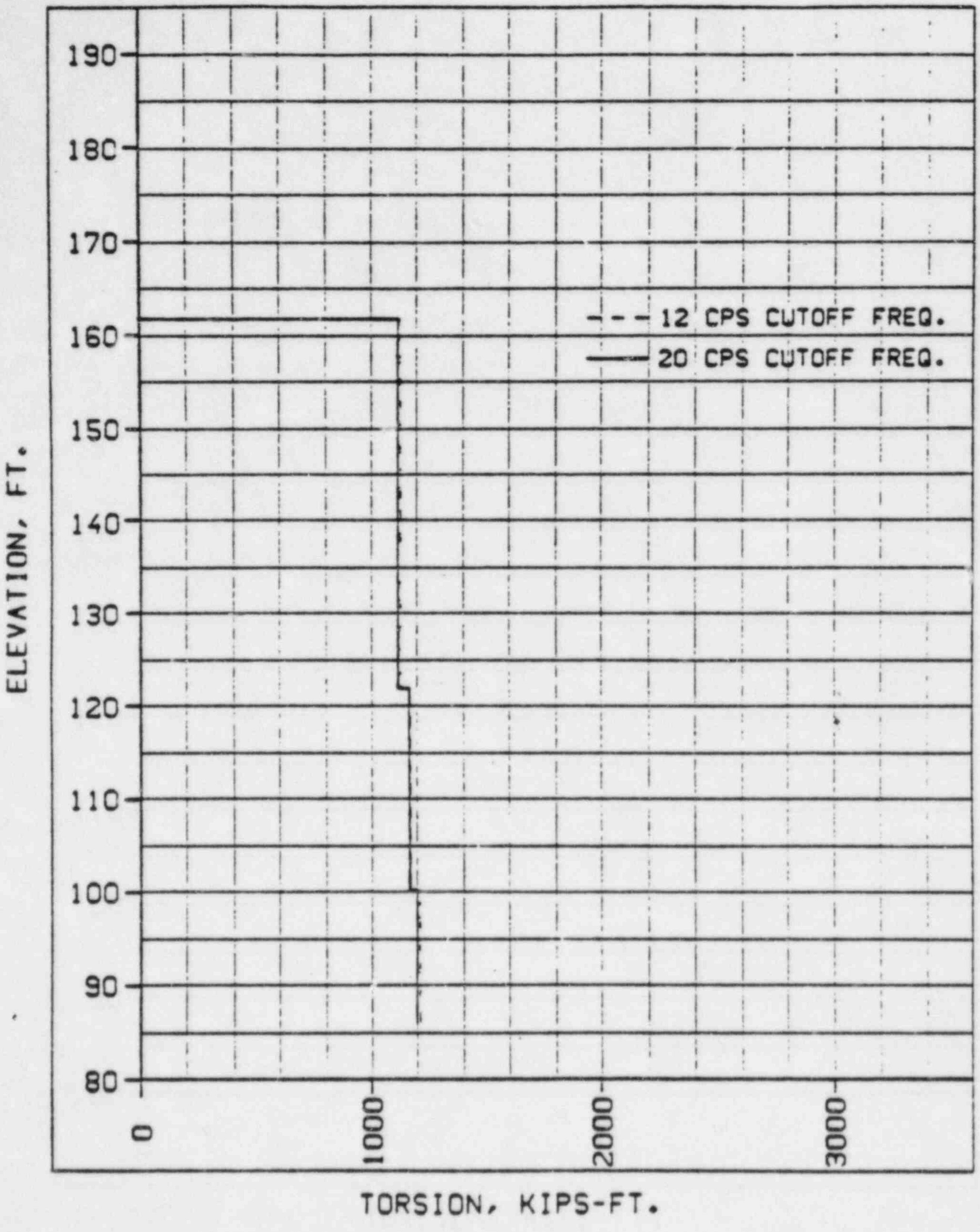
SHEAR RESPONSE MAXIMA FOR DRYWELL

FIGURE 6



MOMENT RESPONSE MAXIMA FOR DRYWELL

FIGURE 7

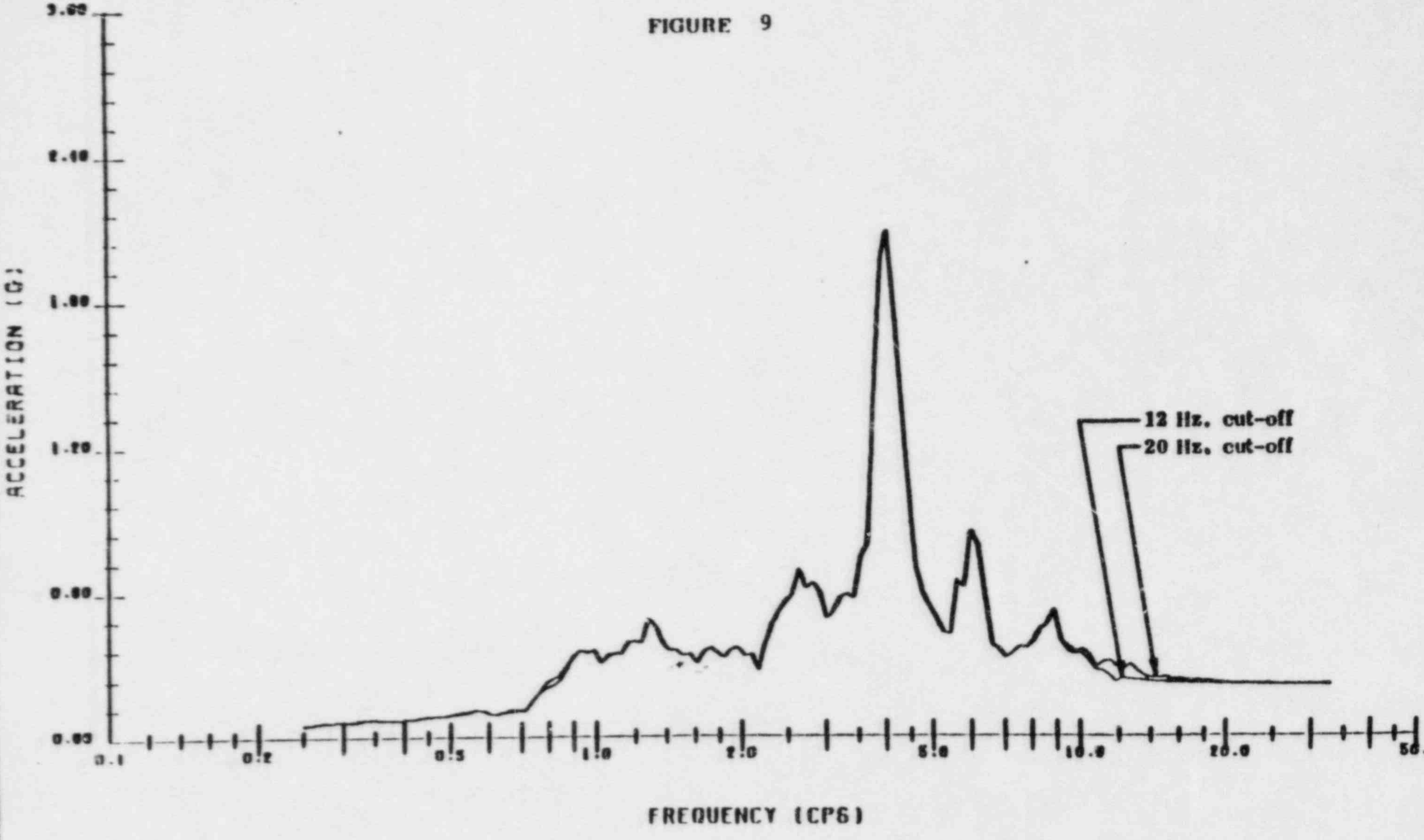


TORSION RESPONSE MAXIMA FOR DRYWELL

FIGURE 8

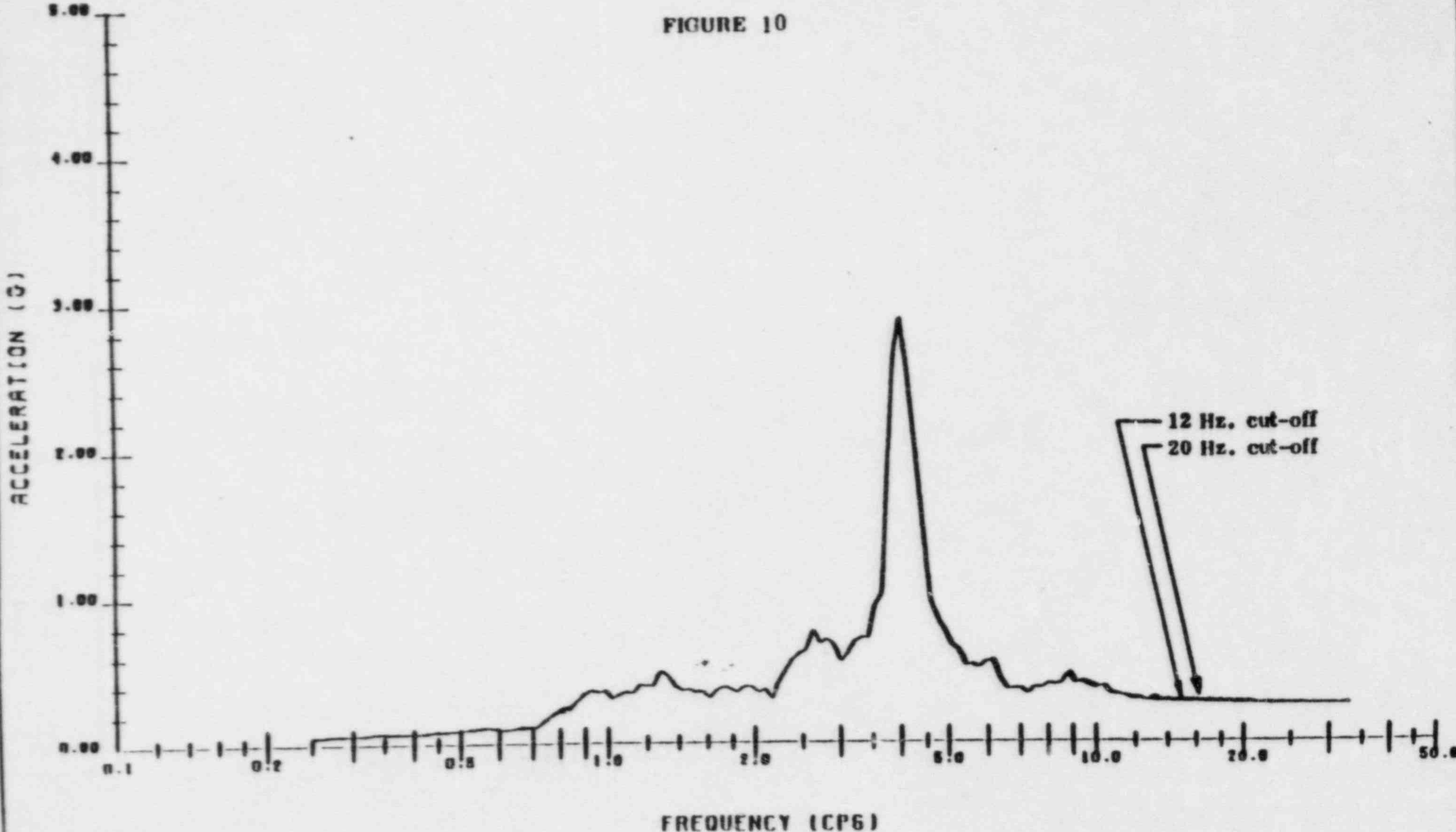
REACTOR BUILDING ELEV 145 N-6 OBE RESPONSE DUE TO SIMPLIED SSIA
(2% DAMPING)

FIGURE 9



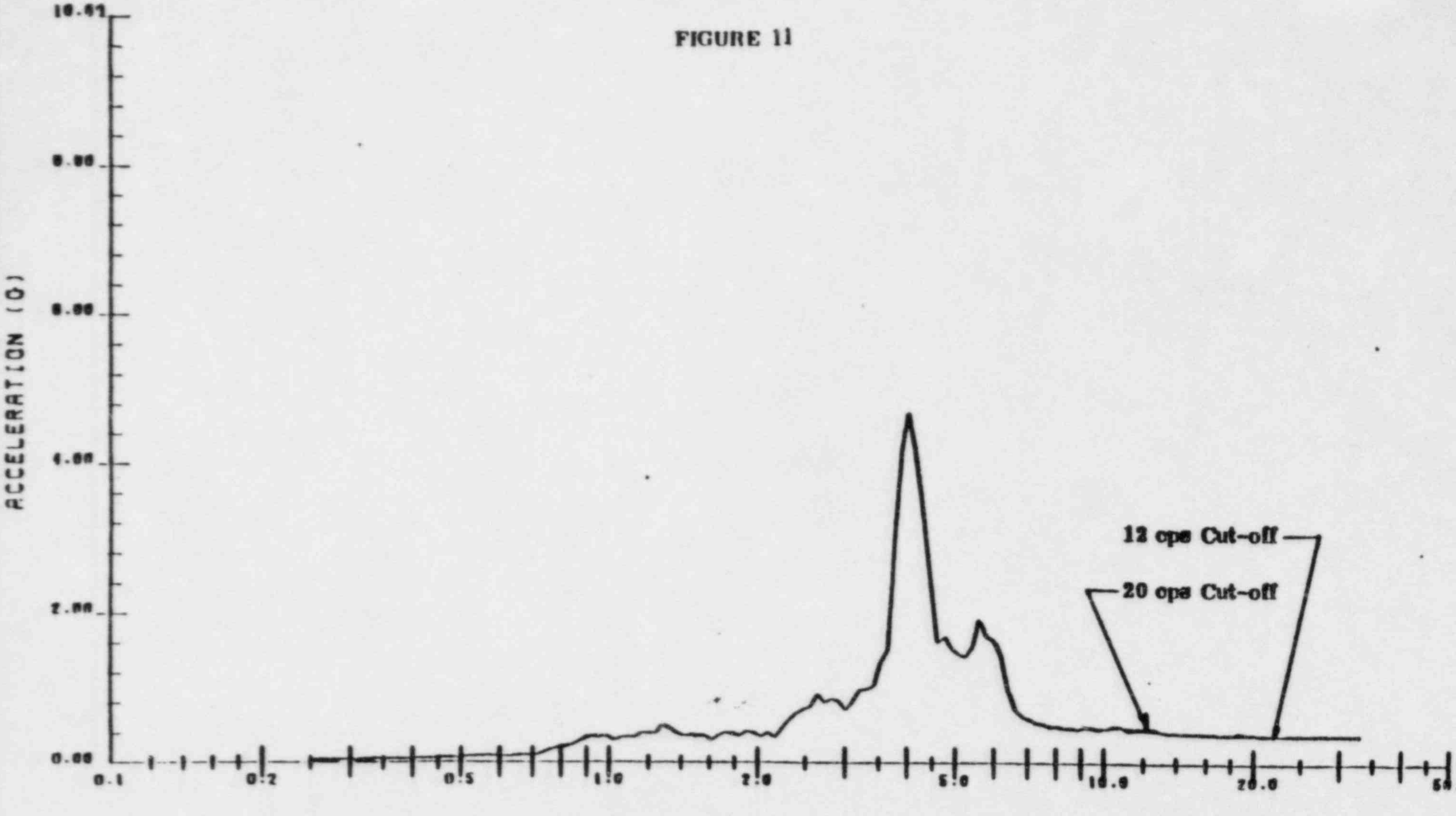
REACTOR BUILDING ELEV 162 N-S OBE RESPONSE DUE TO SIMPLIED SSIA
(2% DAMPING)

FIGURE 10



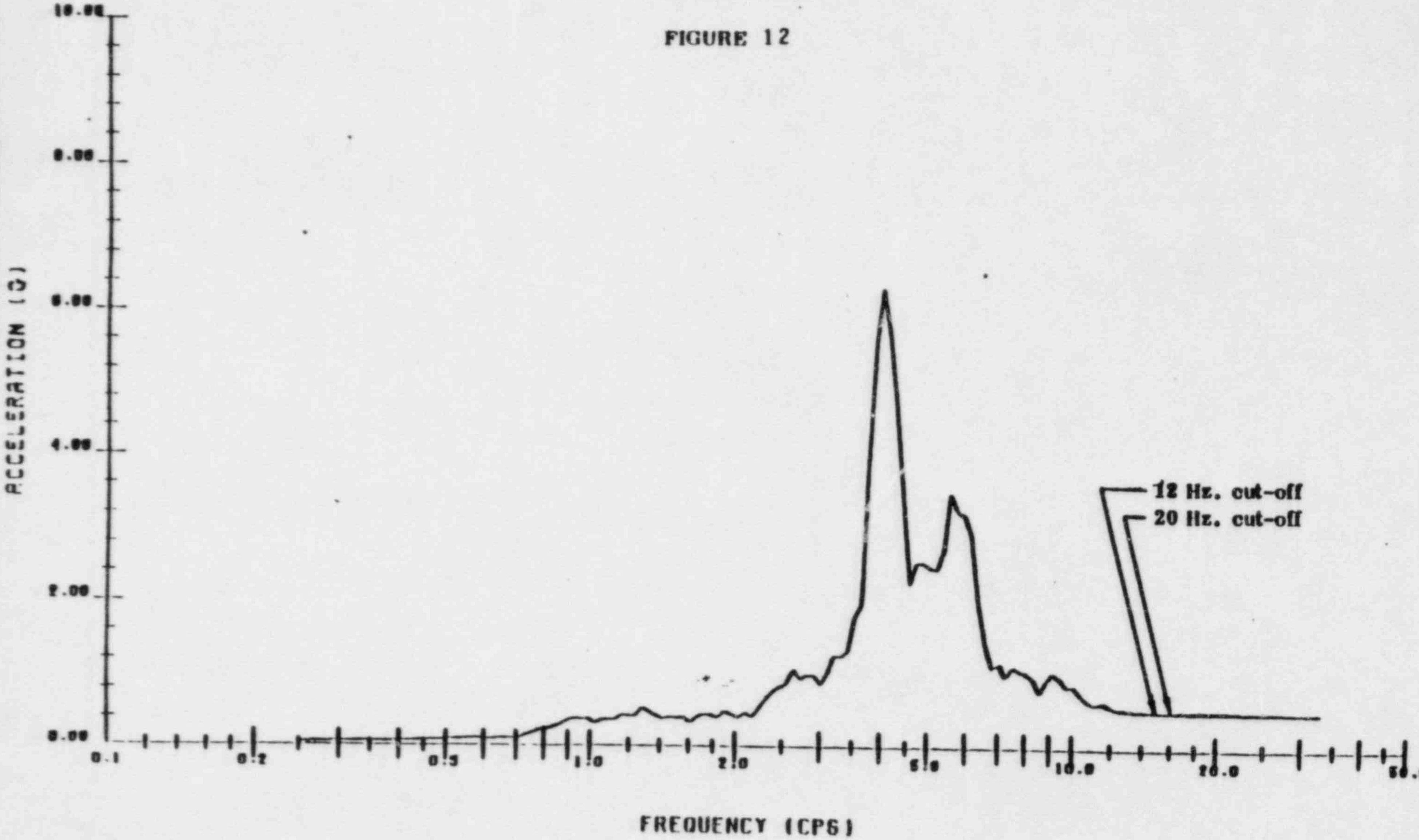
REACTOR BUILDING ELEV. 201 N-S OBE RESPONSE DUE TO SIMPLIFIED SSIA
(2% DAMPING)

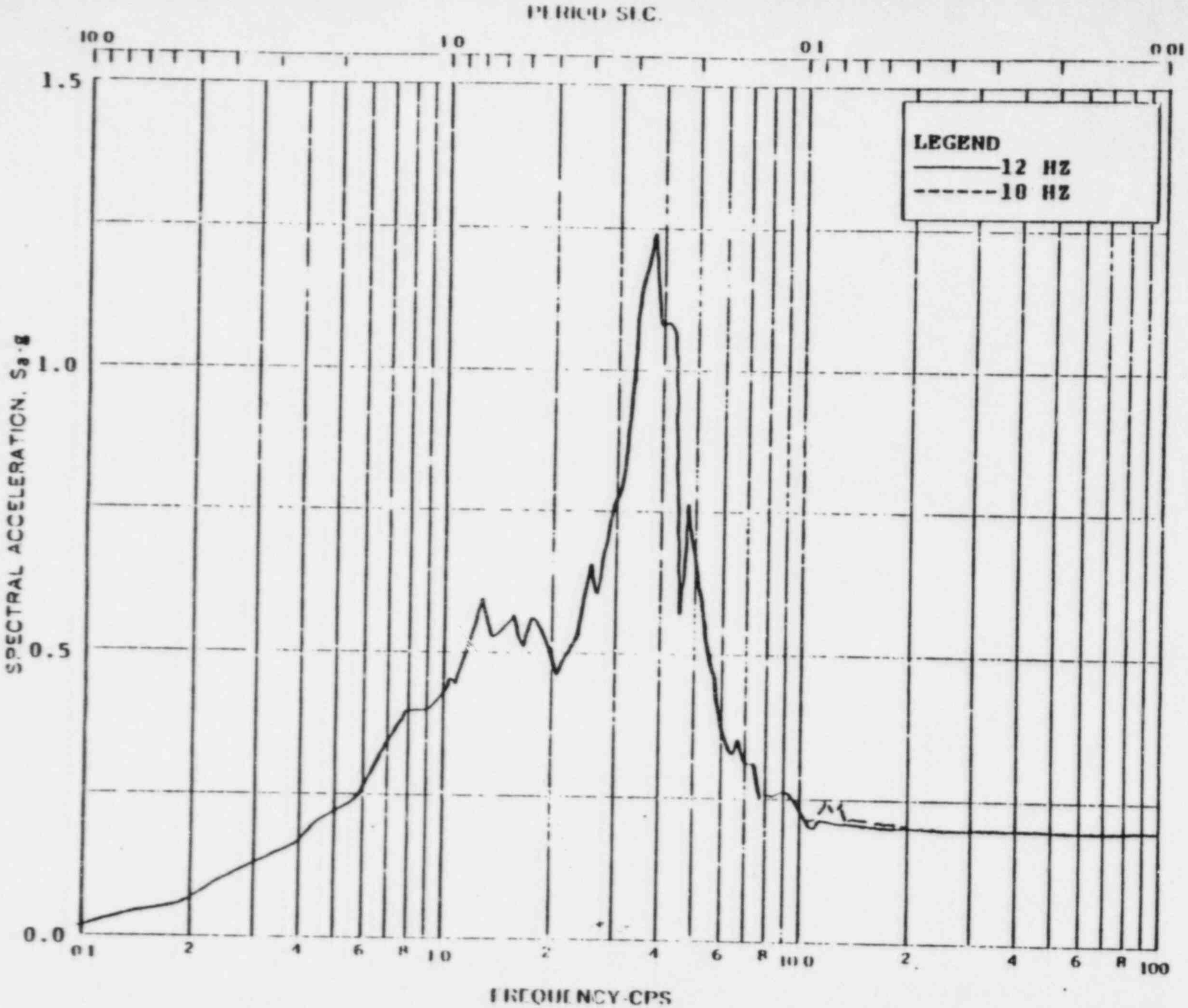
FIGURE 11



REACTOR BUILDING ELEV 252 N-S OBE RESPONSE DUE TO SIMPLIED SSIA
(2% DAMPING)

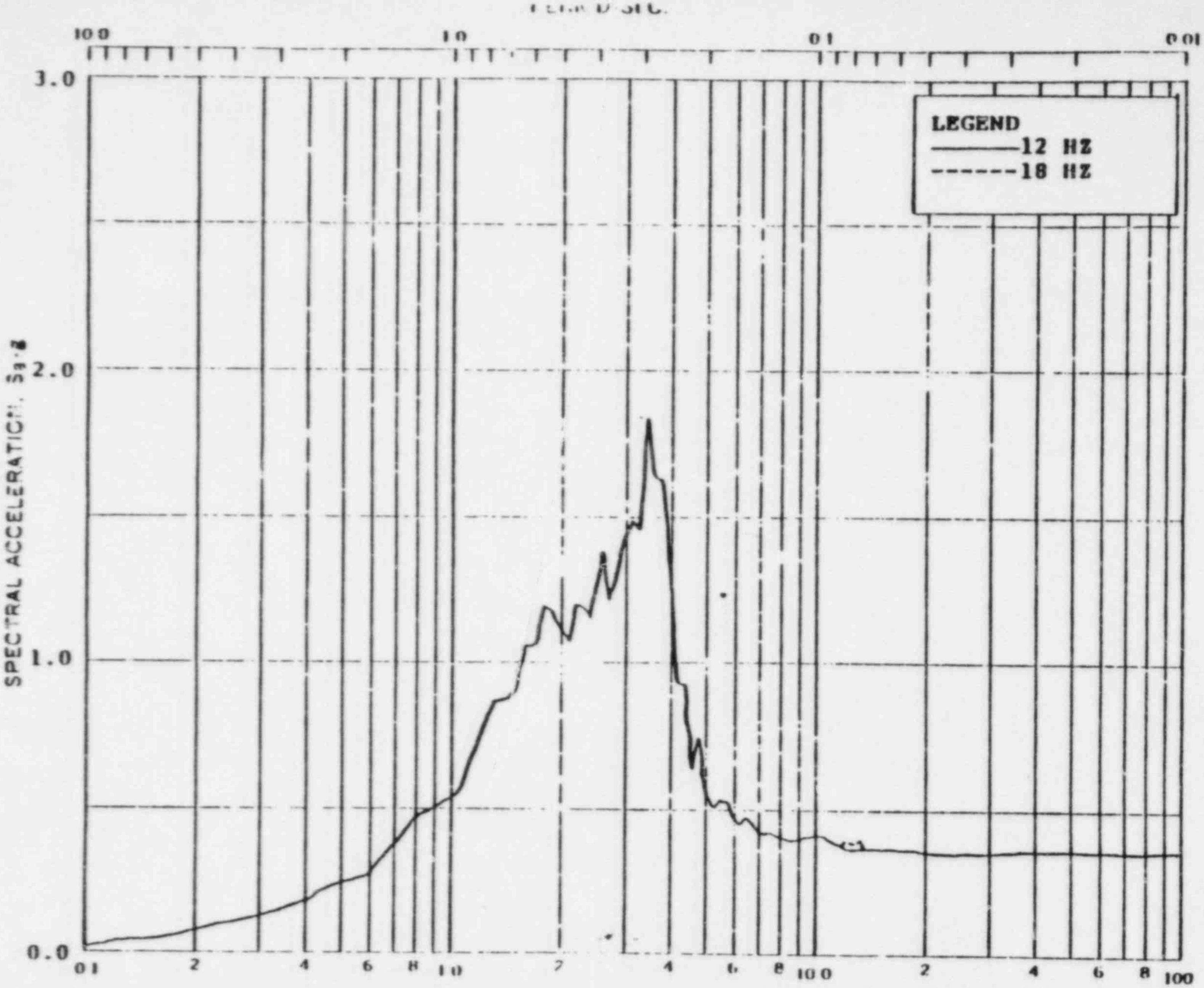
FIGURE 12





FREQUENCY CUT-OFF STUDY
12 HZ VS. 18 HZ
REACTOR BUILDING AT EL. 54'-0"
N-S, SSE, 2% DAMPING

FIGURE 13

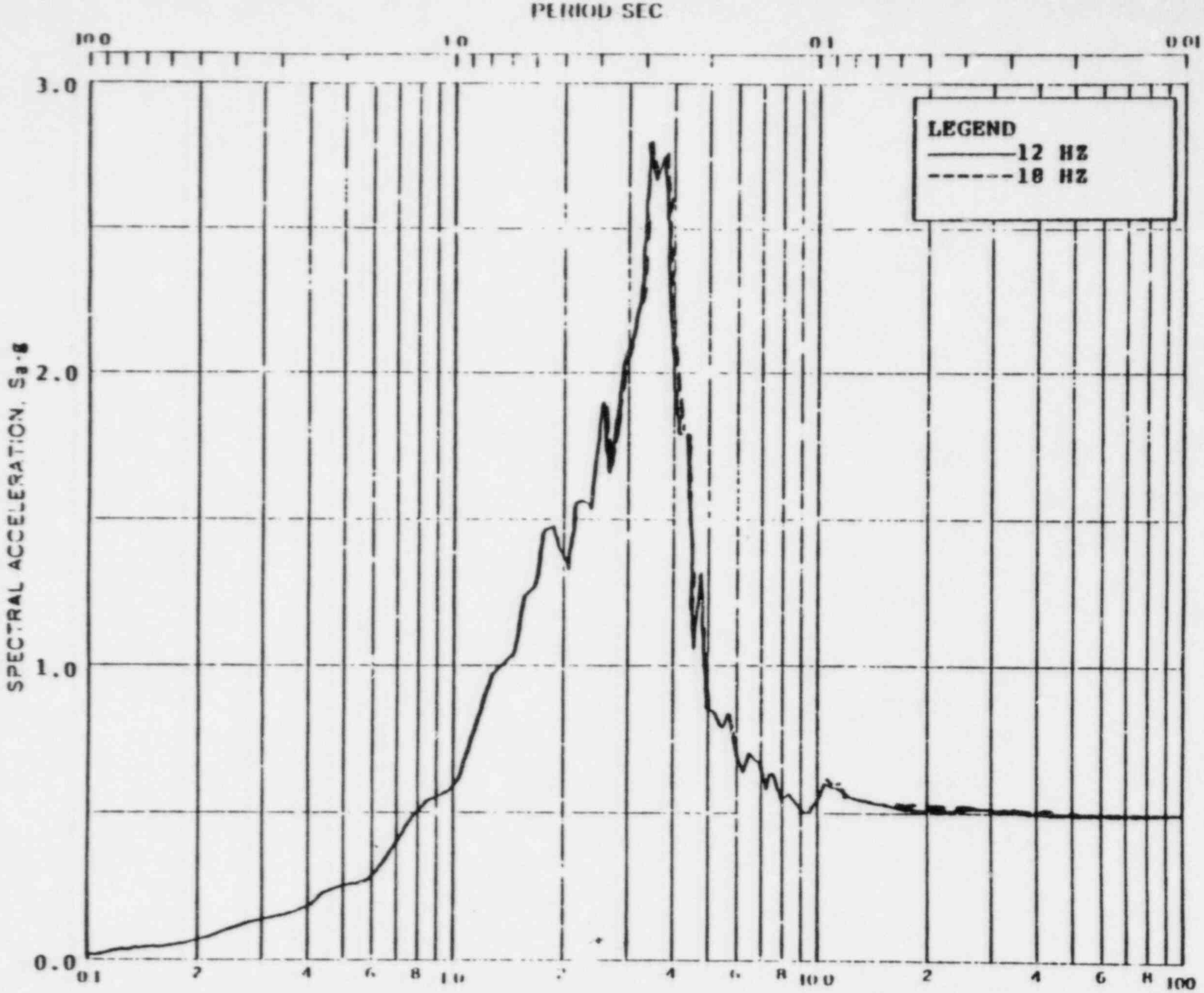


FREQUENCY CPS

FREQUENCY CUT-OFF STUDY
12 HZ VS. 18 HZ
REACTOR BUILDING AT EL. 201'-0"
N-S, SSE, 2% DAMPING

FIGURE 14

January 11/A-12



FREQUENCY CUT-OFF STUDY
12 HZ VS. 18 HZ
REACTOR BUILDING AT EL. 250'-0"
N-S, SSE, 2% DAMPING

January 11/A-12

FIGURE 15

Meeting Date: January 11, 1984

DRAFT

Question No.: A-16

Question: Perform an independent seismic verification analysis (impedance analysis) for the intake structure and compare the results with design basis results. Consider the effects of side boundaries, embedment and the presence of water masses in the analysis.

Response:

In accordance with the requirements of the Standard Review Plan, Section 3.7.2 (NUREG 0800), impedance approach (half-space) seismic soil-structure interaction verification analyses of the service water intake structure (SWIS) are performed by Bechtel. The analytical method used for the impedance approach seismic soil-structure interaction analyses of the SWIS is described in FSAR Section 3.7.2.1. The effects of side boundaries and embedment are considered using the method described in References A-16-1 to A-16-3. The effects of water masses are also accounted for by adding effective water mass to the related nodal points of the structural model in accordance with procedures described in Reference A-16-4.

Figures A-16-1 to A-16-18 show the comparison of the 2 percent damping response spectra obtained from the design basis finite element and the impedance approach seismic soil-structure interaction analyses. The impedance approach response spectra generally are enveloped by those obtained from the design basis analyses at elevation 114.0 feet of the SWIS. For other elevations, the impedance approach spectral accelerations exceed the design basis spectral accelerations in some frequency ranges. These ranges vary approximately between 1.5 and 10.0 Hz.

As discussed during the January 1984 NRC Structural Audit Meeting, sampling studies have been performed to confirm the adequacy of the SWIS design. The criteria used in selection of the samples for this study is given in Table A-16-1. The results of the sampling studies are as follows:

1. Structure

   All major reinforced concrete shear walls at the base of the intake structure have been evaluated for seismic forces and moments obtained from the impedance approach analyses. The shear stresses resulting from the impedance approach analyses were compared with those of the design basis analyses. Table A-16-2 shows comparison of shear stresses. In all cases these revised shear stresses were found to be within the allowables.

   The moments in the walls, obtained from the impedance analyses, were smaller than those of design basis analyses for both the East-West OBE and SSE cases, therefore, no further evaluation of these walls is required.

For North-South OBE and SSE cases, the moments obtained from impedance approach analyses exceeded the design basis moments. The increase in moments were mostly isolated to the eastern portion of the intake structure. This portion of the intake structure was reevaluated and the resulting moments were found to be less than the allowables.

Based on the above, it is concluded that the as-built SWIS can accommodate loads obtained from the impedance approach analyses.

2. Equipment

The effects of the impedance approach response spectra was evaluated on 8 types of seismic category I equipment located in the areas where the impedance approach spectra were found to have higher spectral accelerations than those of the design basis response spectra. The equipment evaluated represents over 30% of all equipment located in the intake structure.

Table A-16-3 summarizes the results of the above evaluation for equipment in the Intake Structure. It is concluded that all category I equipment can accommodate the response spectra obtained from the impedance analyses.

3. Cable Tray and HVAC Supports

All cable tray and HVAC supports were evaluated using the impedance analysis results. All supports were found to meet the impedance approach spectral response requirements.

4. Piping and Piping Supports

Piping and pipe supports were evaluated using the screening techniques discussed in Table A-16-1. The results are summarized in Tables A-16-4 and A-16-5. The analysis results show that piping stresses and nozzle loads are within allowable limits. There was no load increase found on existing supports.

It is therefore concluded that the existing design margins associated with the present project design basis seismic loading are not affected by the consideration of the loads generated from the impedance approach analyses as demonstrated by the SWIS piping systems.

References: A-16-1, Apsel, R.J., (1979) "Dynamic Green's Functions for Layered Media and Applications to Boundary Value Problems", Ph.D Thesis, University of California, San Diego.

Response to Question A-16 (cont'd)

References: (Cont'd)

A-16-2, Wong, H.L., and Luco, J.E., (1978) "Tables of Impedance Functions and Input Motions for Rectangular Foundations", Report No. CE78-15; University of California, San Diego.

A-16-3, Barneich, J.A., Johns, D.H., and McNeill, R.L., (1974) "Soil-Structure Interaction Parameters for Aseismic Design of Nuclear Power Stations", Preprint 2182, ASCE National Meeting on Water Resources Engineering, January 21-25.

A-16-4, Newmark, N. and Rosenblueth, E., "Fundamentals of Earthquake Engineering," Prentice-Hall, Englewood Cliffs, N.J. (1971)

TABLE A-16-1
PROCEDURES FOR EVALUATION OF
INTAKE STRUCTURES, EQUIPMENT & COMPONENTS
USING IMPEDANCE ANALYSIS RESULTS

INTRODUCTION

The results of the impedance analysis are used to assess the existing design of the HCGS intake structure, equipment and components. A sampling approach is used. The procedure for this evaluation is as follows:

A. STRUCTURES:

Since the maximum shear and axial forces and the maximum overturning moments occur at the base of the structure, and the design margins for the upper elevations are greater than those of the base, the effects of these loads at the base of the structure are evaluated.

B. EQUIPMENT:

The impedance analysis spectra in general are not completely enveloped by the design basis spectra in the 1.5 to 10.0 Hz and in the ZPA range throughout the intake structure.

The following procedure is selected for review:

- Review the significant frequencies of at least 30% of equipment located in the areas where the impedance approach spectra were found to have higher spectral accelerations than those of the design basis response spectra.
- If the significant equipment frequencies fall in the range where the difference in the spectra exist, additional evaluation is necessary. No further evaluation is necessary if the significant frequencies are outside the frequency range in question.
- The evaluation is performed either by comparing the test response spectra of the equipment with the impedance spectra (if the equipment is qualified by testing) or comparing the actual-to-allowable stress ratios with the spectrum exceedance ratios.
- If the above evaluation shows the equipment may not be qualified for the impedance spectra, detailed evaluation consisting of analysis and/or testing is performed.
- As a result of evaluation, if equipment requires modifications, the sample size for this evaluation is expanded as required.

C. CABLE TRAY AND HVAC SUPPORTS

All cable tray and HVAC supports are evaluated for impedance analysis results.

D. PIPING AND PIPE SUPPORTS

In general, impedance curves resulted in significant reductions in spectral accelerations as compared to those of the design basis curves. However, in some curves, the peak accelerations showed small increases. To evaluate the effects of the increase in peak accelerations a "biased" sample of affected piping systems is reanalyzed and reevaluated. The sample is selected as follows:

Individual impedance curves for various elevations and structures are superimposed on their corresponding design basis curves to identify those impedance curves which are not enveloped by design basis curves. Those impedance curves are then superimposed on the design basis "enveloped" response spectra used for various piping system design calculations. If the design basis enveloped response spectra curves affecting a calculation did not totally envelop all the corresponding impedance curves, that particular calculation is then identified as "affected" calculation and a candidate for sampling.

A "biased" sample of the "affected" calculations was selected which emphasized the following important piping parameters:

1. Stress levels in the existing pipe stress calculations. Samples included systems with high stress levels.

2. Difference in "g" level (Δg) between impedance and design basis curves in the affected frequency zones. Sample selected to include curves showing significant differences.

3. High equipment nozzle loads in existing calculation.

The number of calculations included in the sample is:

Building	Total No. of Q-Calcs	No. of Calcs Reviewed	No. of Calcs affected	No. of Calcs in the sample
Intake Structure	11	11	5	1

Results of the analysis including support loads are compared against the design basis values for acceptability.

Table A-16-2
Intake Structure Shear Stress at the Base

Base Elevation	Wall Location Column Line	Design Base (psi)	Impedance Approach (psi)	Allowable (psi)
79'-8"	Col. A (East Wall)	80	124	630
79'-8"	Col. Ac	66	98	630
79'-8"	Col. Ak	47	73	630
70'-0"	Col. C (West Wall)	47	77	126
79'-8"	Col. 5 (South Wall)	230	214	630
79'-8"	Col. 7	200	176	630
79'-8"	Col. 9 (North Wall)	230	214	630

Notes: 1. Concrete f'c = 4000 psi.
2. See FSAR Figures 1.2-40 and 1.2-41 for wall location.

Table A-16-3

Intake Structure - Seismic Category I Equipment

Equipment or Component	Tag No.	Elev.	Fundamental Frequencies (Hz)	Method of Seismic Qualification	Applicable Note
Travelling Water Screen (T.W.S.)	1(A-D)S501	70'-0"& 114'-0"	Horizontal - 7.4,14 Vertical - >33	Analysis	2
Control Panel (for T.W.S.)	1(A-D)C515	107'-0"	Horizontal - 21, 30 Vertical - >33	Testing	1
Service Water Pumps	1(A-D)P502	93'-0"	Horizontal - 28.4 Vertical - >33	Analysis	3
Supply Fans	0AV558 0BV558	128'-0"	Horizontal - >33 Vertical - >33	Analysis	2
Vane Axial Fans	1AV-DV503 1AV-DV504	122'-0"	Horizontal - >33 Vertical - >33	Analysis	3
HVAC Control Panel	1(A-D)C581	93'-0"	Horizontal - 15, 22 Vertical - >33	Analysis	2
Travelling Screen Spray Water Booster Pumps	1AP-DP507	79'-8"	Horizontal - >33 Vertical - >33	Analysis	2
Transformer Panel Board	10Y501-504	93'-0"	Horizontal - 29,31 Vertical - >33	Testing	1

Notes: 1. TRS envelops impedance approach spectra.

2. Impedance approach spectral acceleration is lower than that of the design basis response spectra in the major equipment frequencies.

3. Although impedance approach spectral acceleration exceeds that of design basis response spectra in the equipment frequency range, a more detailed calculation showed that the equipment stresses are within the code allowables.

Table A-16-4

Intake Structure Pipe Stress Summary

Calc. No.	Max. Seismic Stress Ratios: Max. Impedance Stress / Max. Design Basis Stress		ASME Code Equation Evaluation		Vendor Equipment Nozzle Allowables Met
			Eq. 9B* / Code Allow.	Eq. 9D* / Code Allow.	
	OBE	SSE	Upset	Faulted	
C2019	0.46	0.51	0.26	0.14	Yes

*ASME Section III NC, ND-3652

Table A-16-5

Intake Structure Pipe Support Load Summary

Calc. No.	Total No. of Supports	No. of Supports with load increase	Average Percentage increase in load		Support Design Adequate
			Upset	Faulted	
C2019	15	0	N/A	N/A	Yes

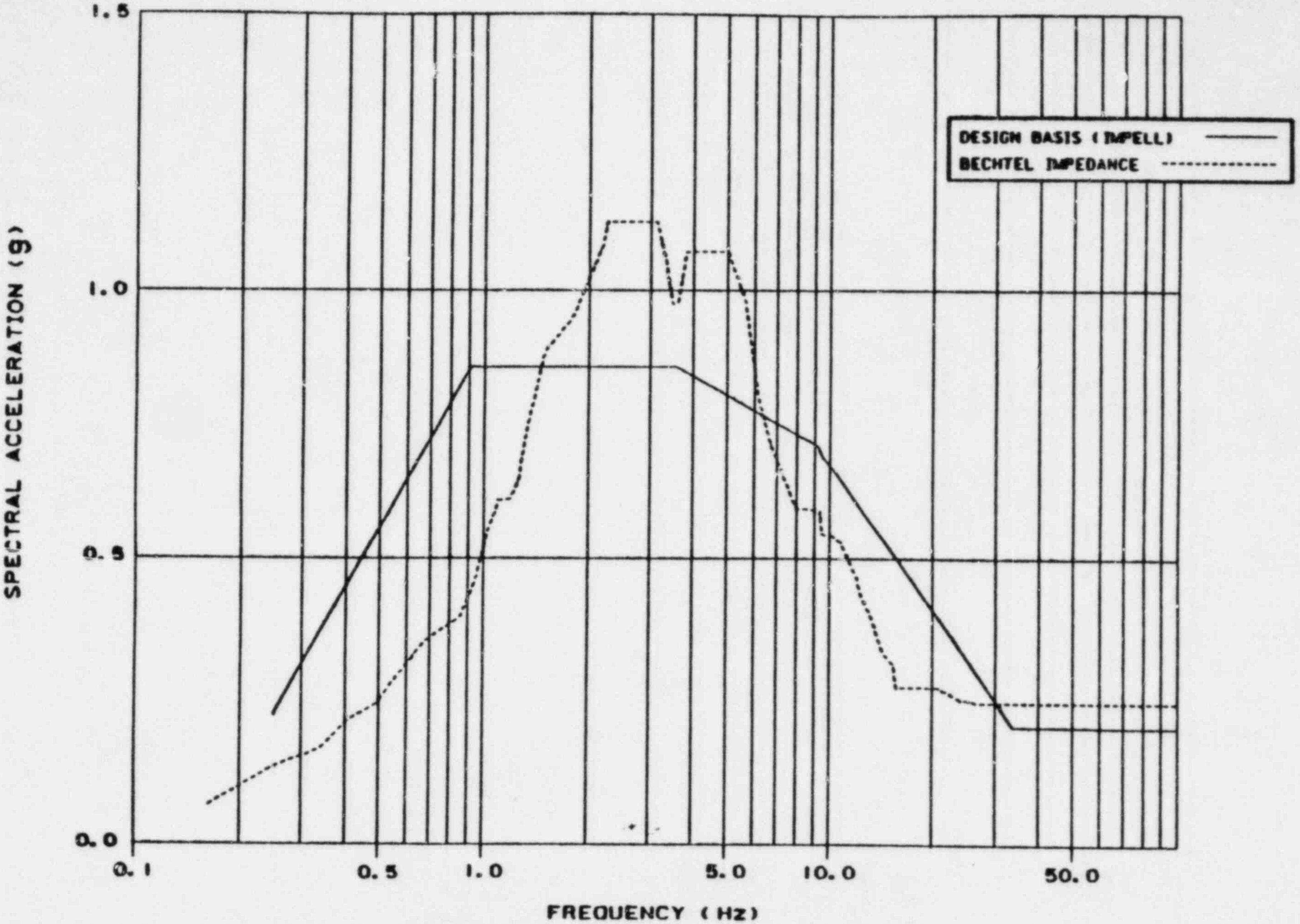


FIGURE A-16-1

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 93'-0",

HOPE CREEK UNIT 1

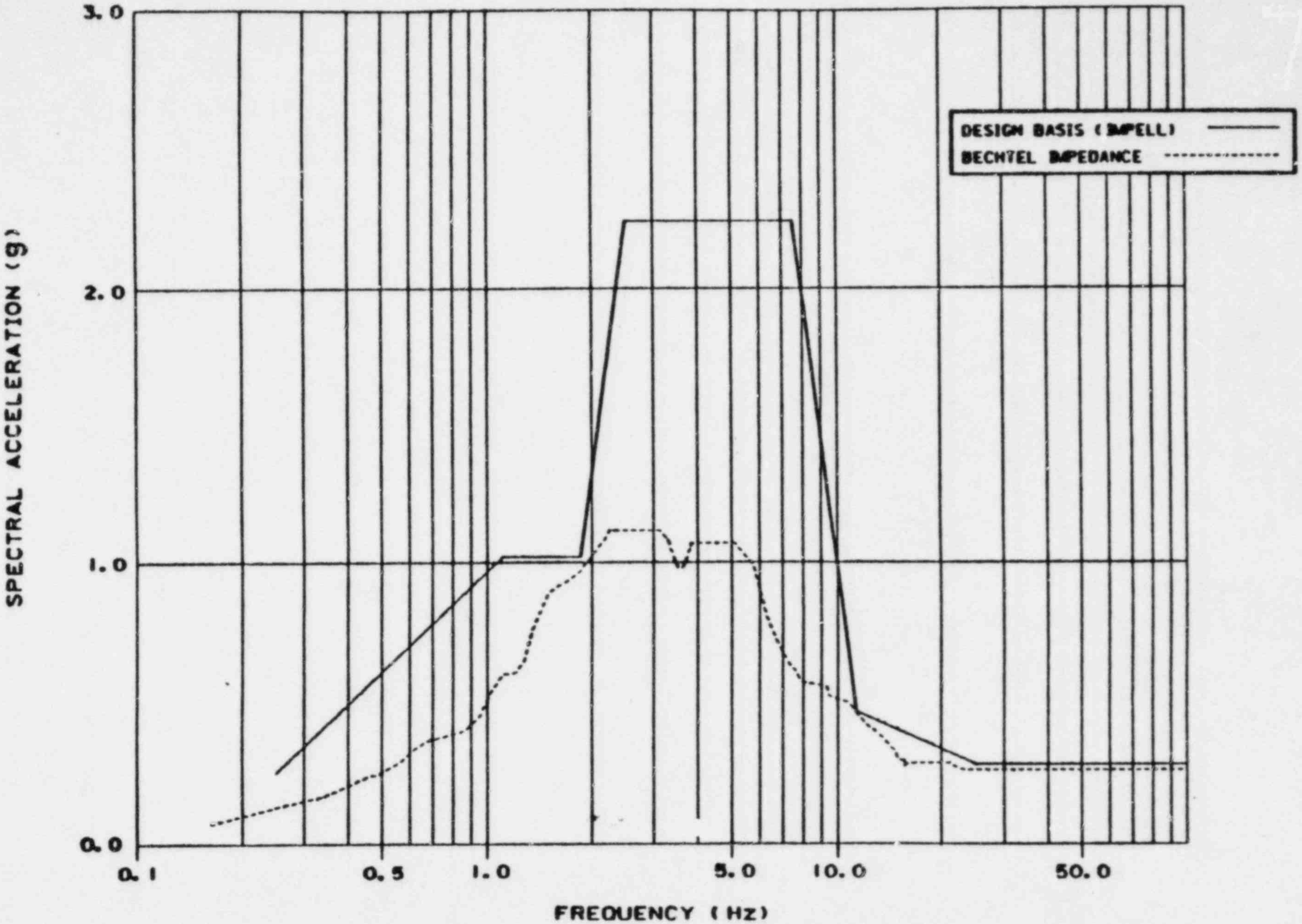


FIGURE A-16-2

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 114' -0",

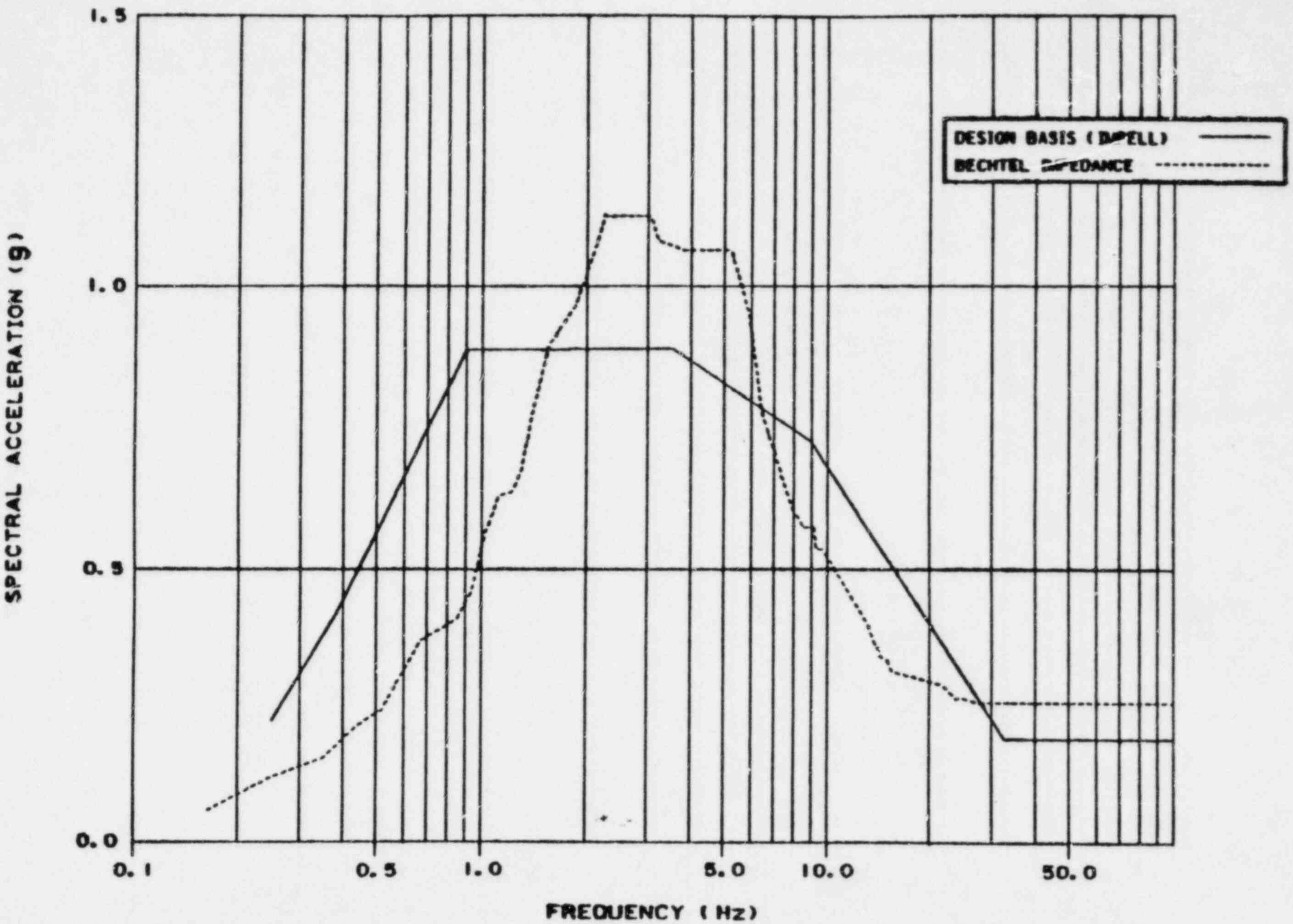


FIGURE A-16-3

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 135'-0".

HOPE CREEK UNIT 1

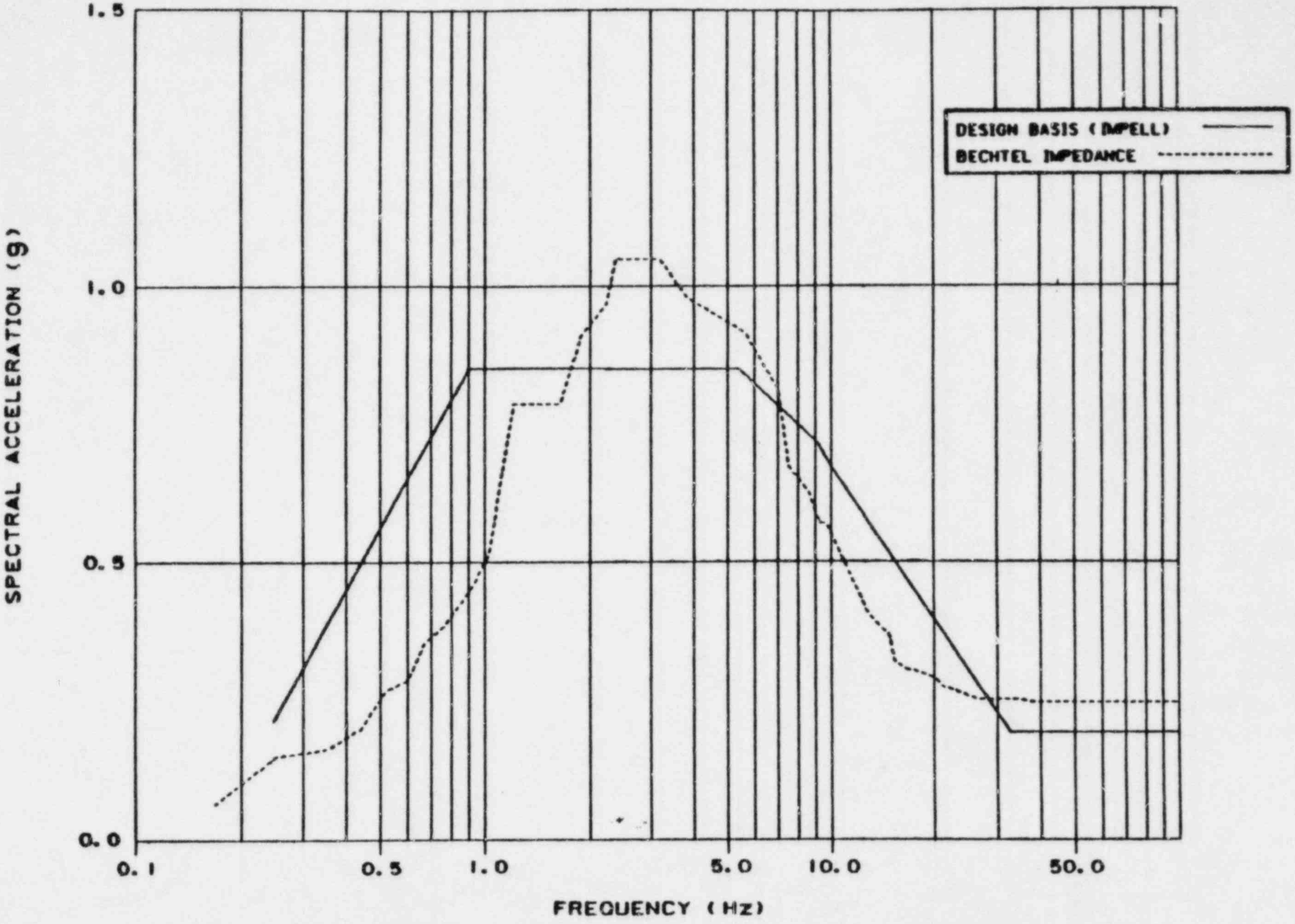


FIGURE A-16-4

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 93' -0".

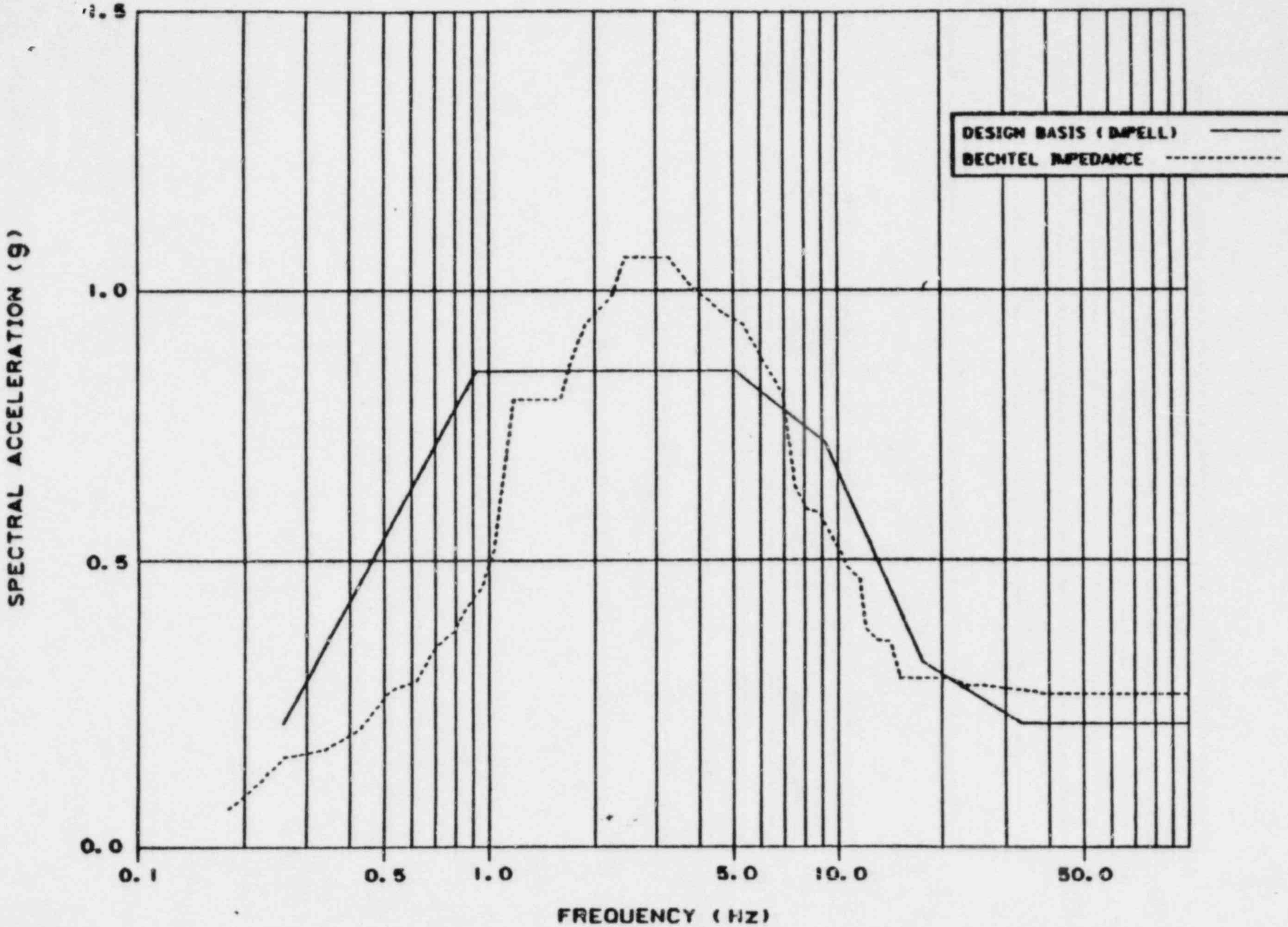


FIGURE A-16-5

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 114' -0".

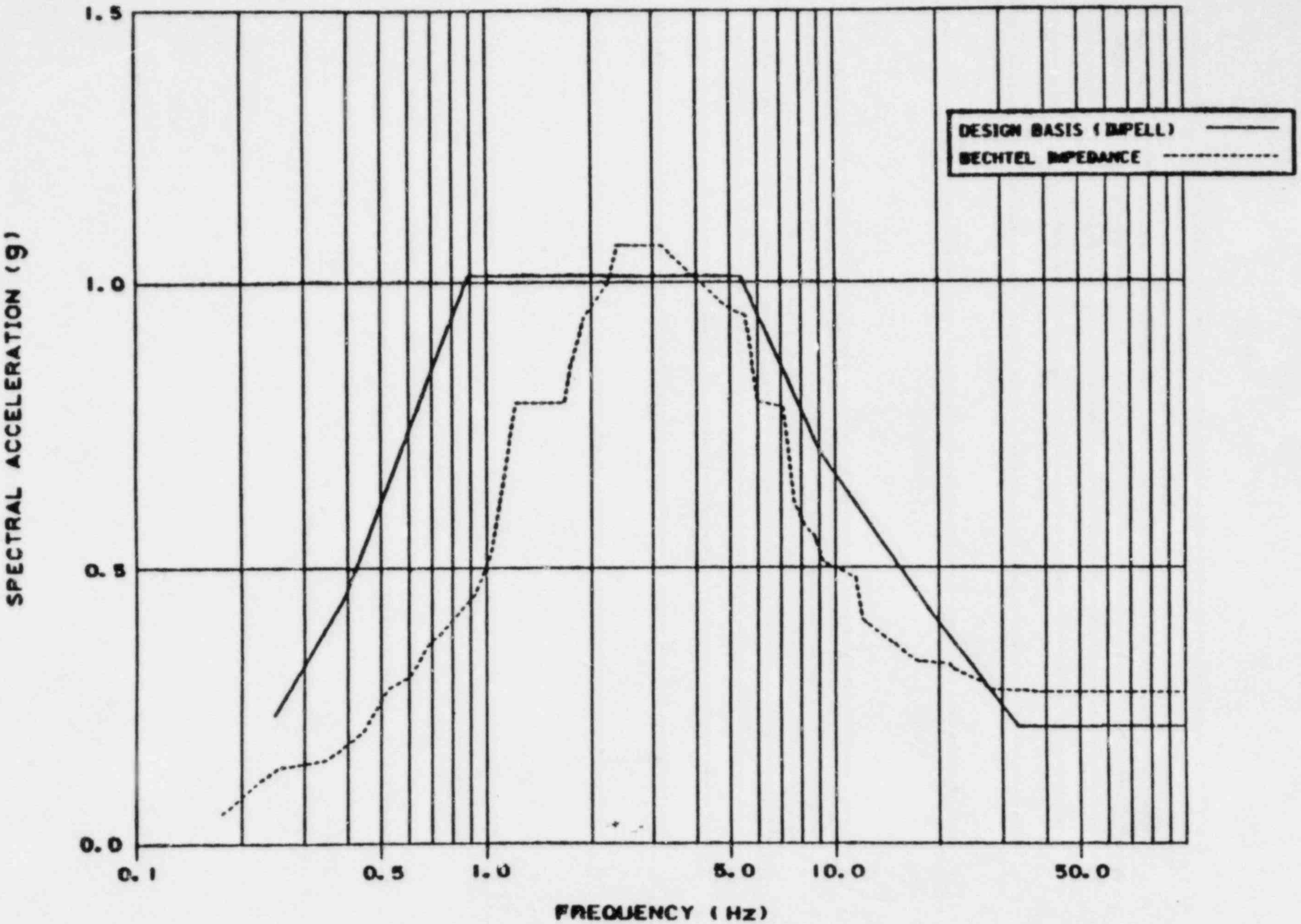


FIGURE A-16-6

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 135' -0",

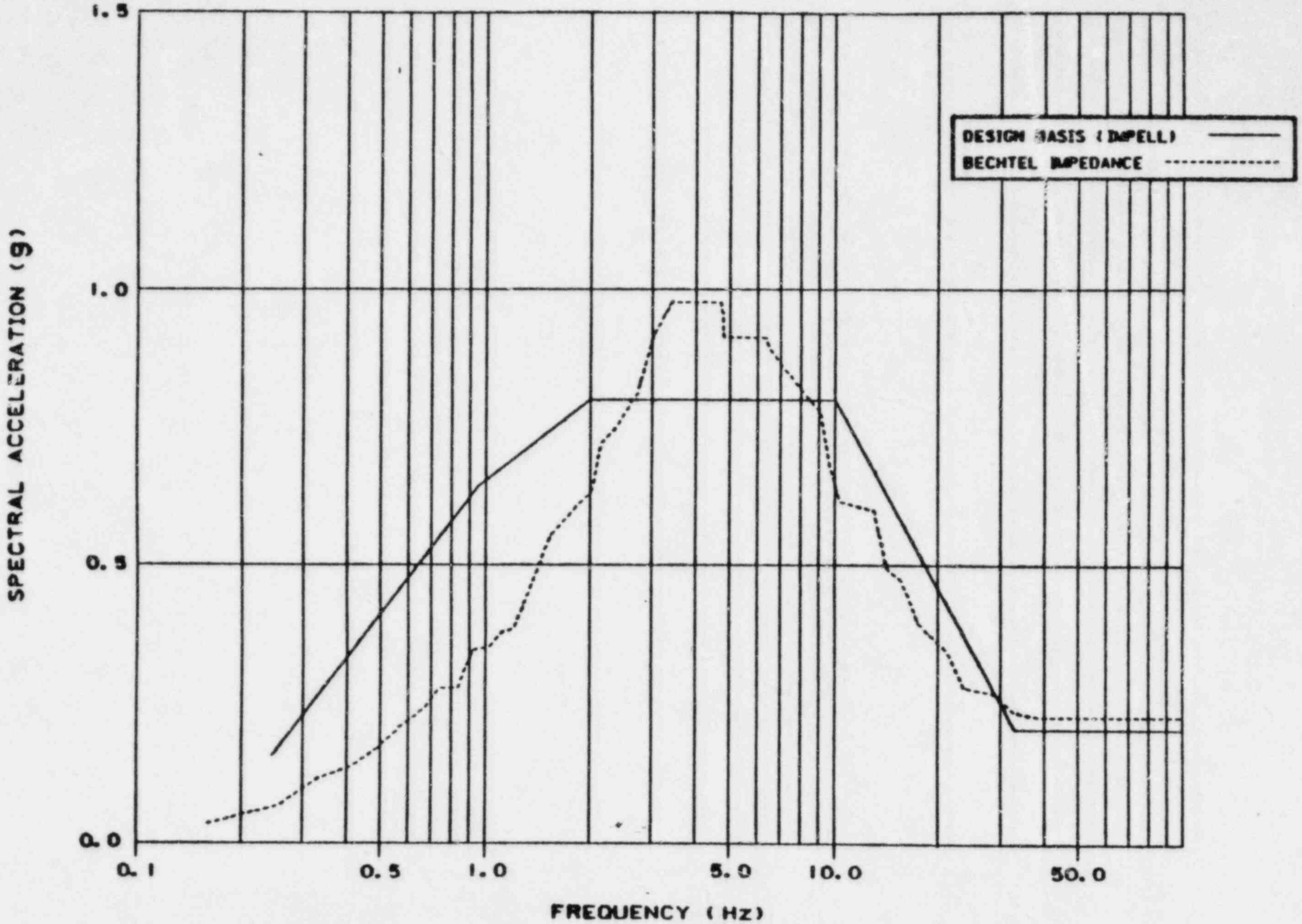


FIGURE A-16-7

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 93'-0",
VERTICAL - SSE - 2% DAMPING

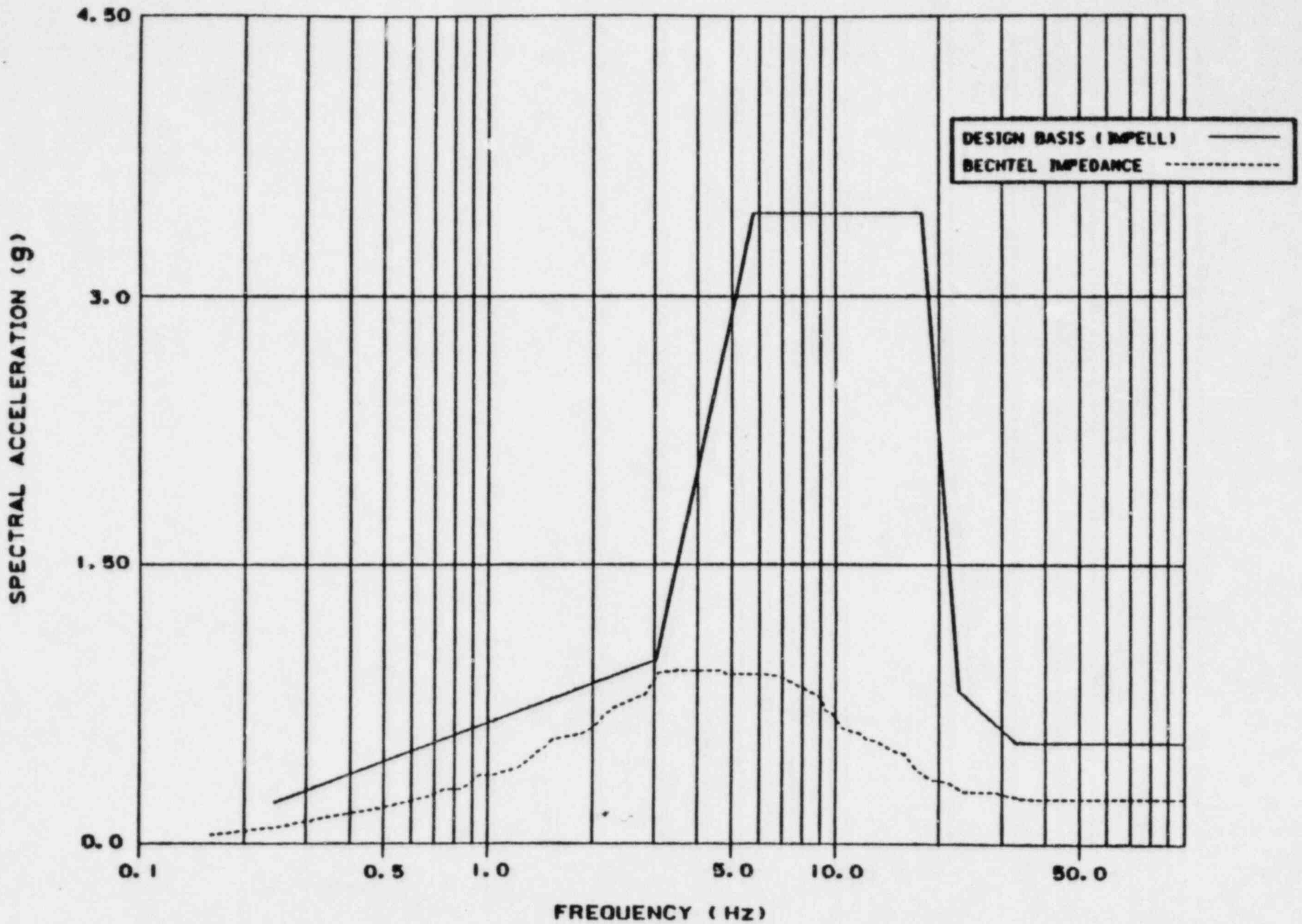


FIGURE A-16-8

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 114'-0",

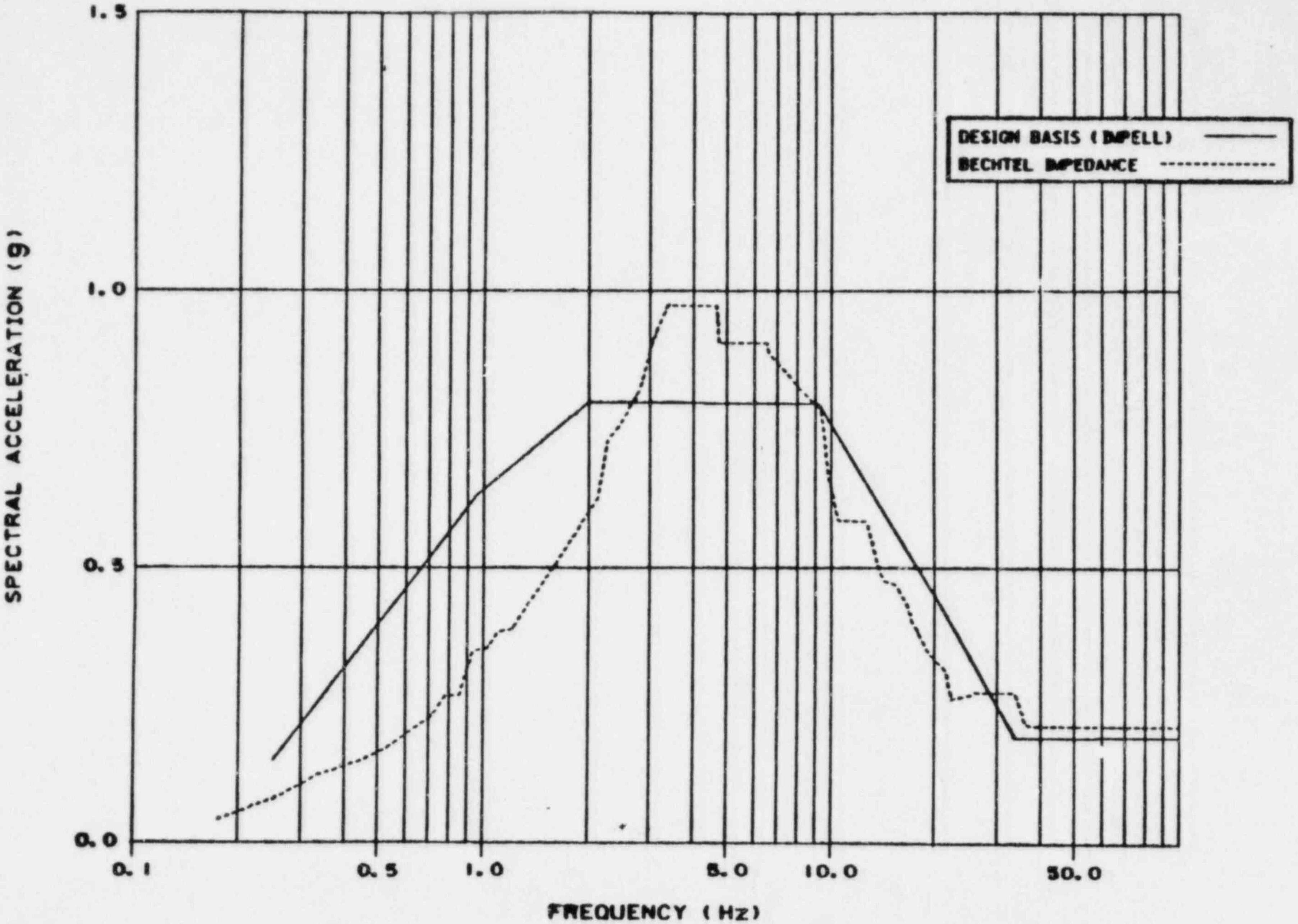


FIGURE A-16-9

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 135'-0",
VERTICAL – SSE – 2% DAMPING

HOPE CREEK UNIT 1
CAD 154 REV 0

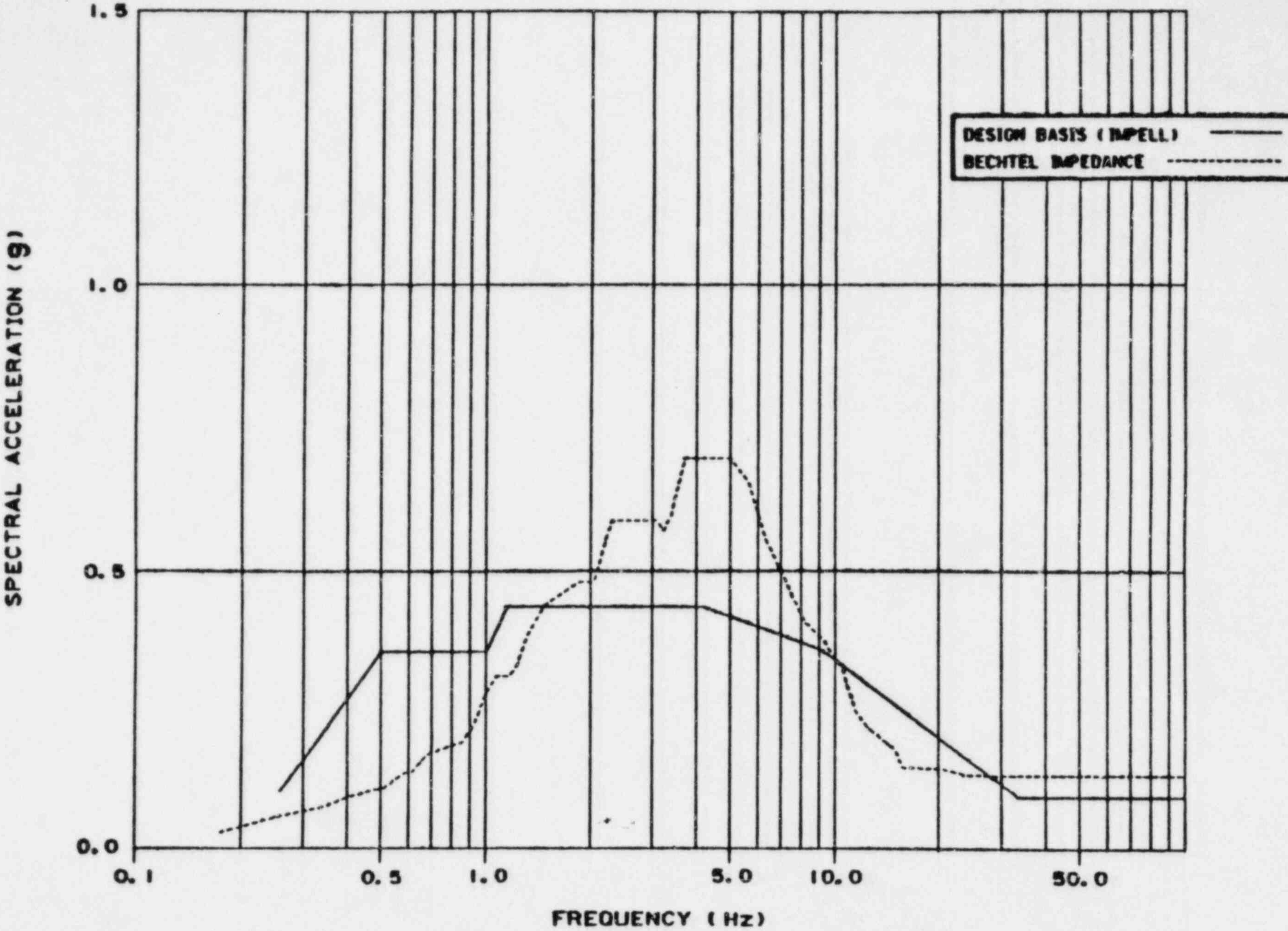


FIGURE A-16-10

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 93' -0".

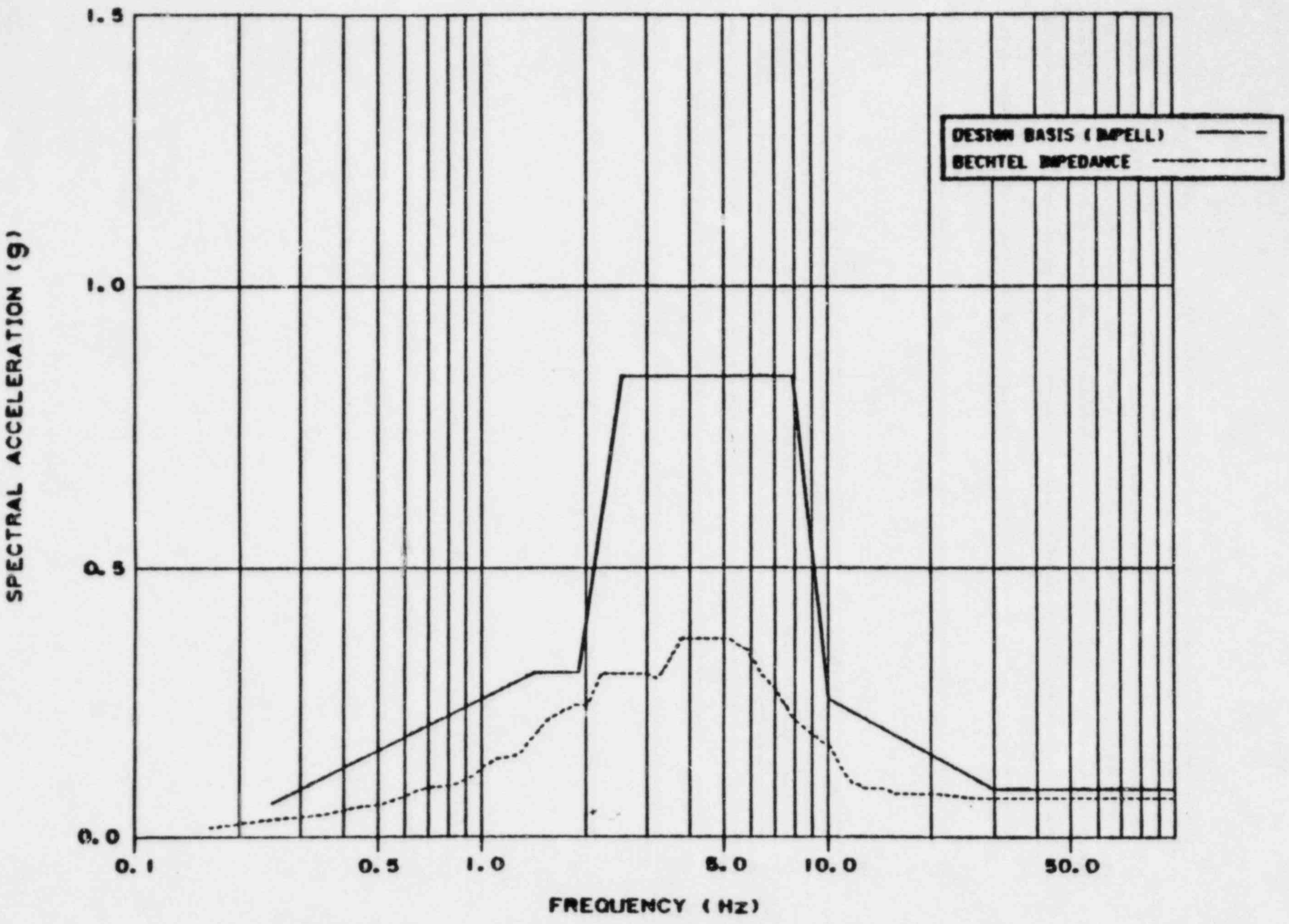


FIGURE A-16-11

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 114' -0",

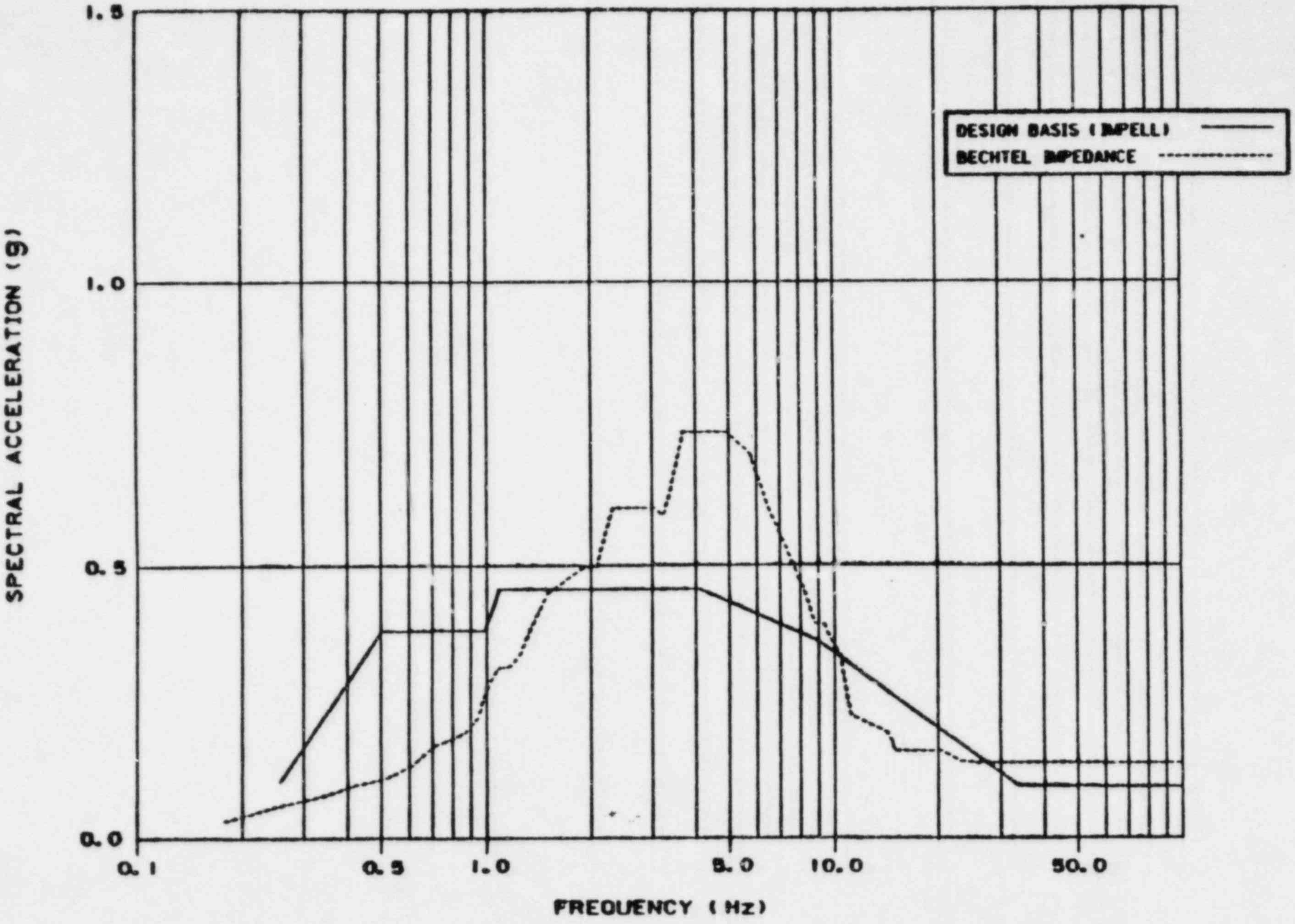


FIGURE A-16-12

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 135'-0".

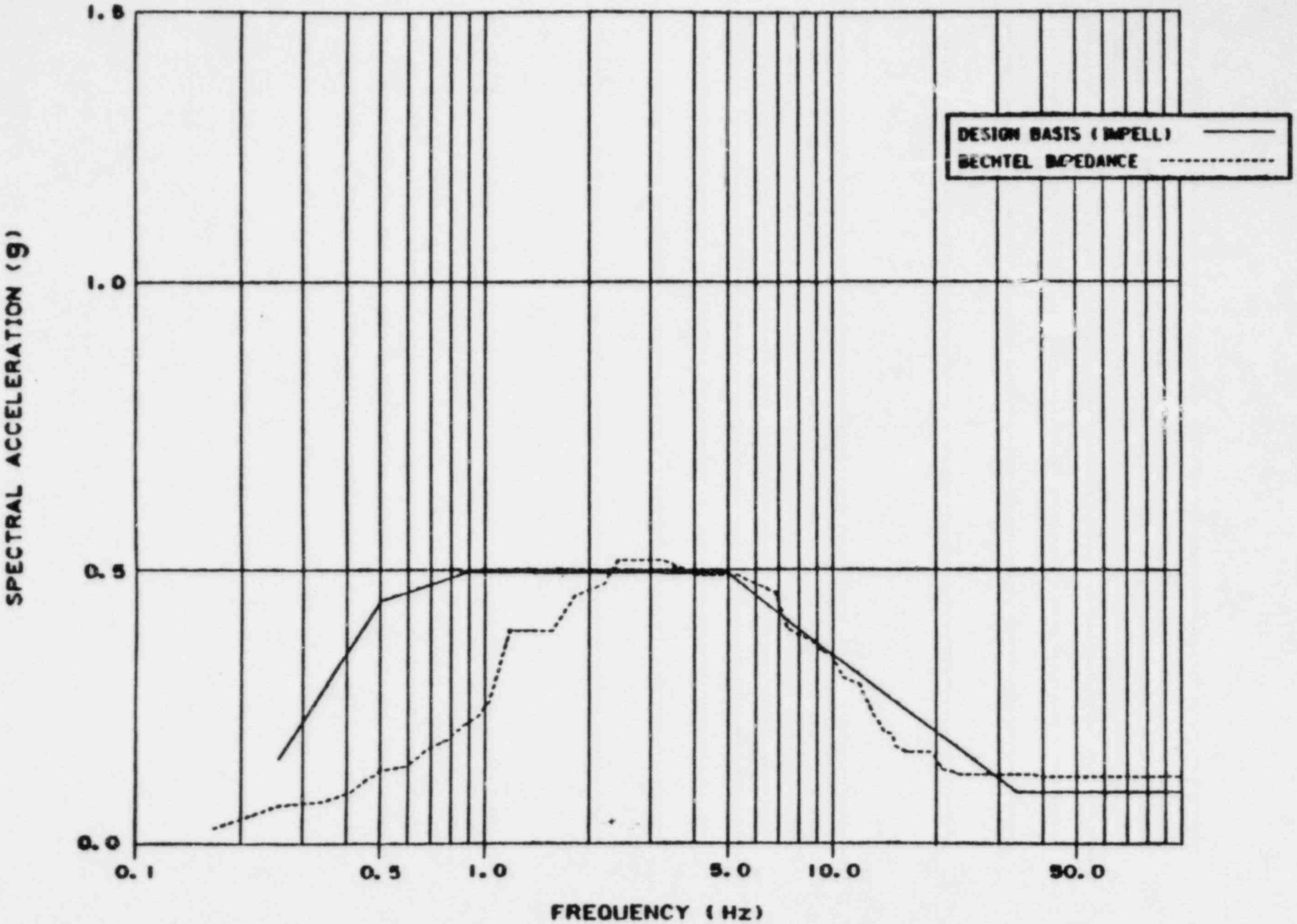


FIGURE A-16-13

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 93' -0",

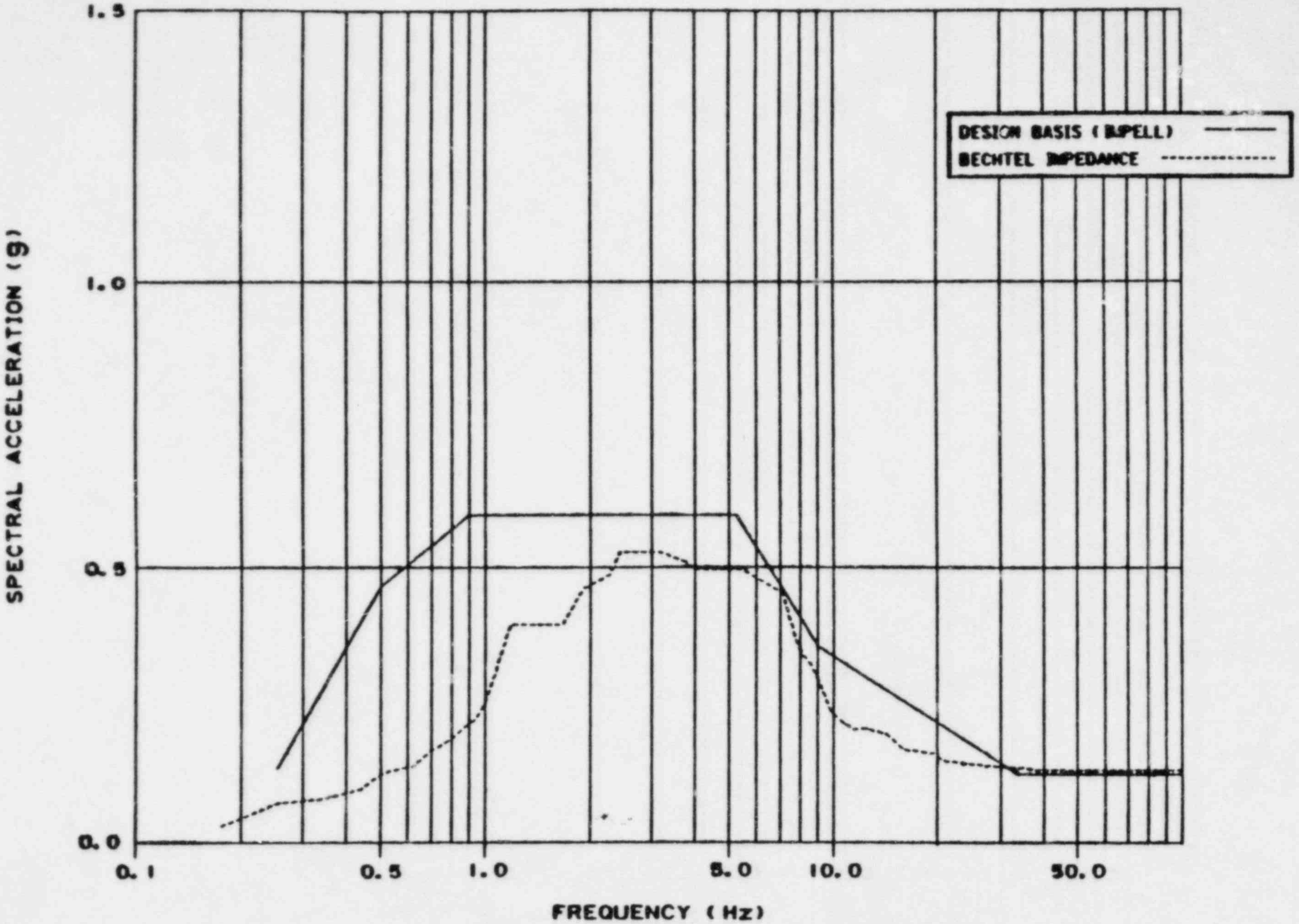


FIGURE A-16-14

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 114' -0",

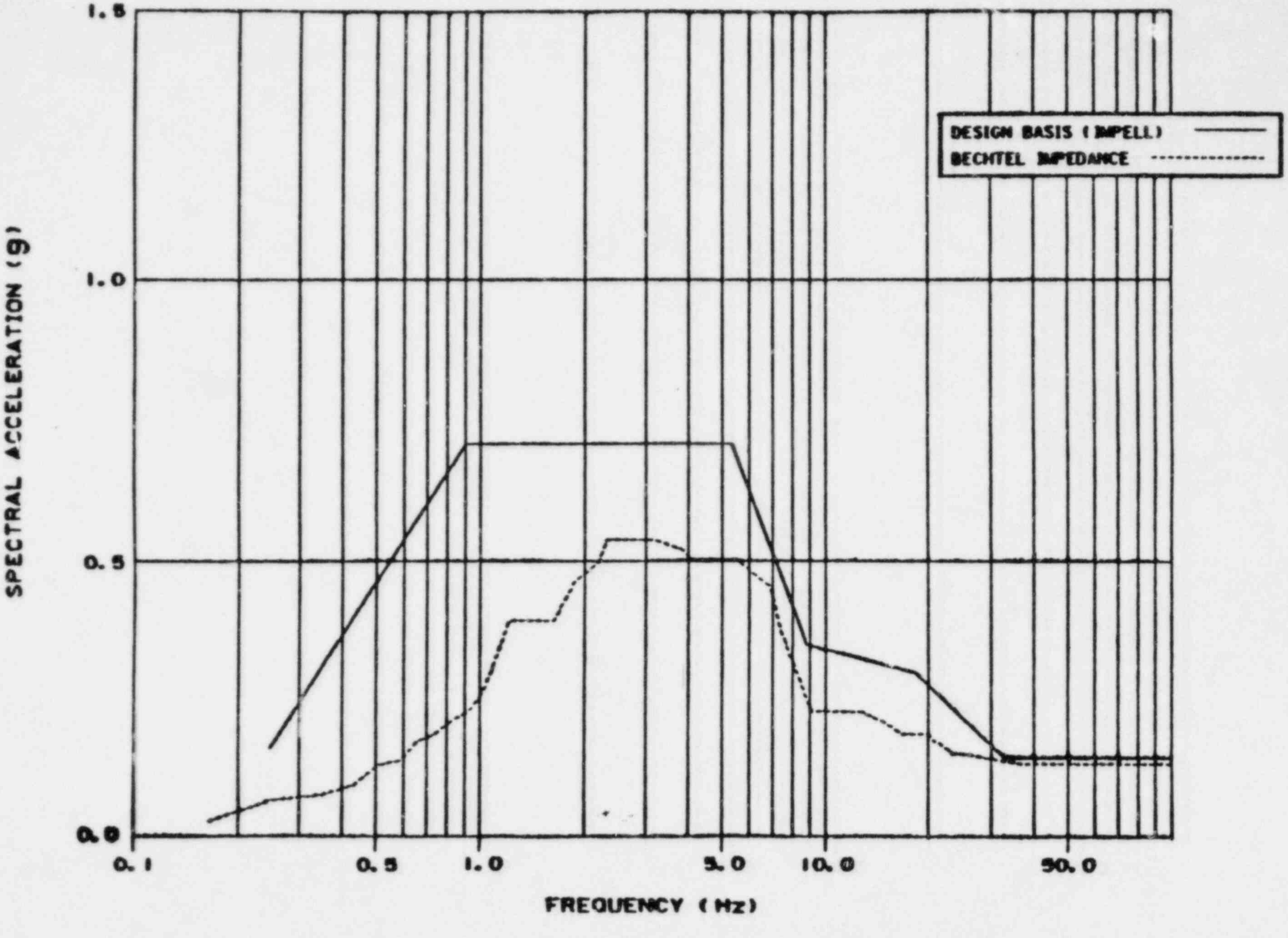


FIGURE A-16-15

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 135'-0",

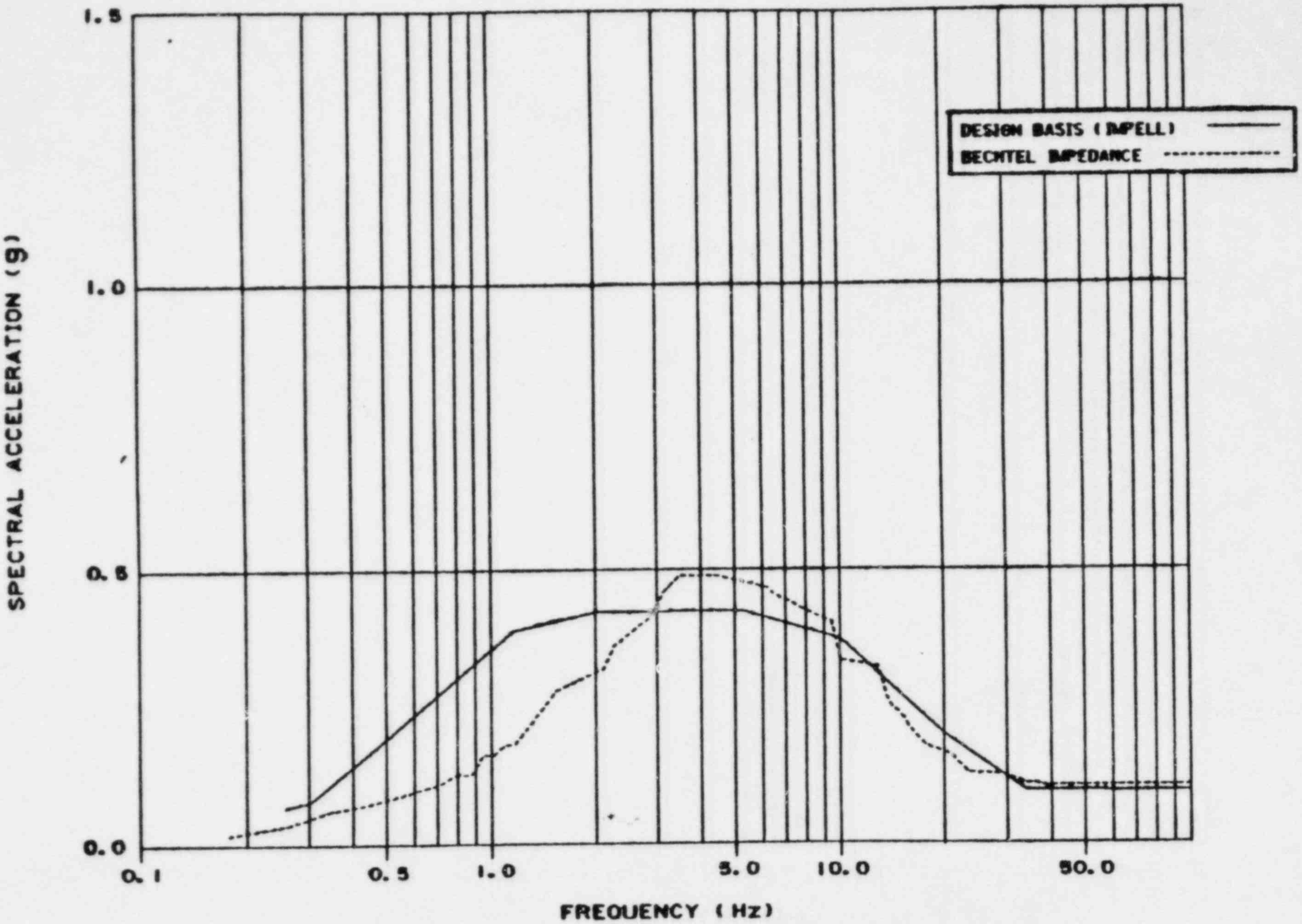


FIGURE A-16-16

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 93' -0".

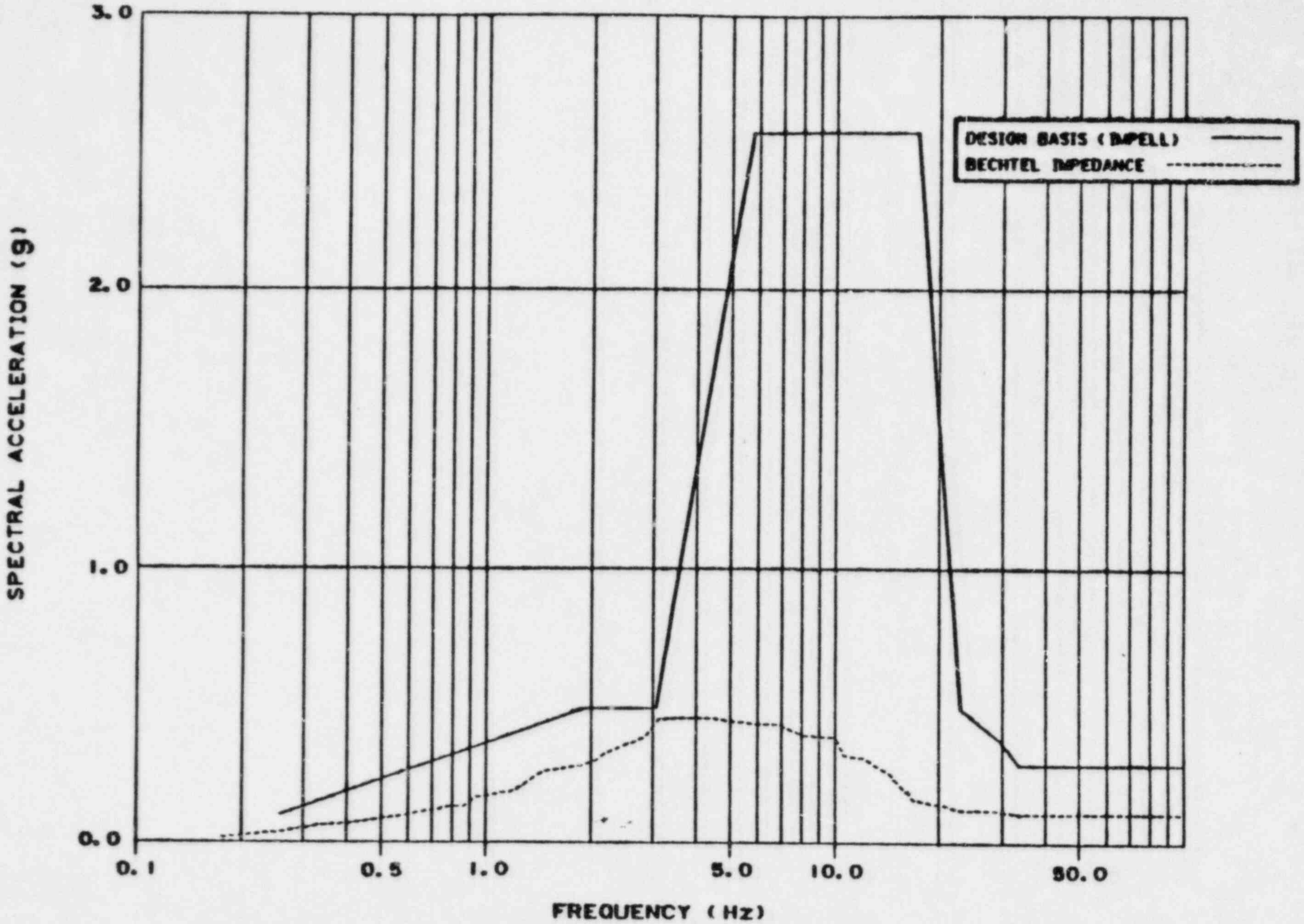


FIGURE A-16-17

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 114' -0".

JUL -3 '84 0267224

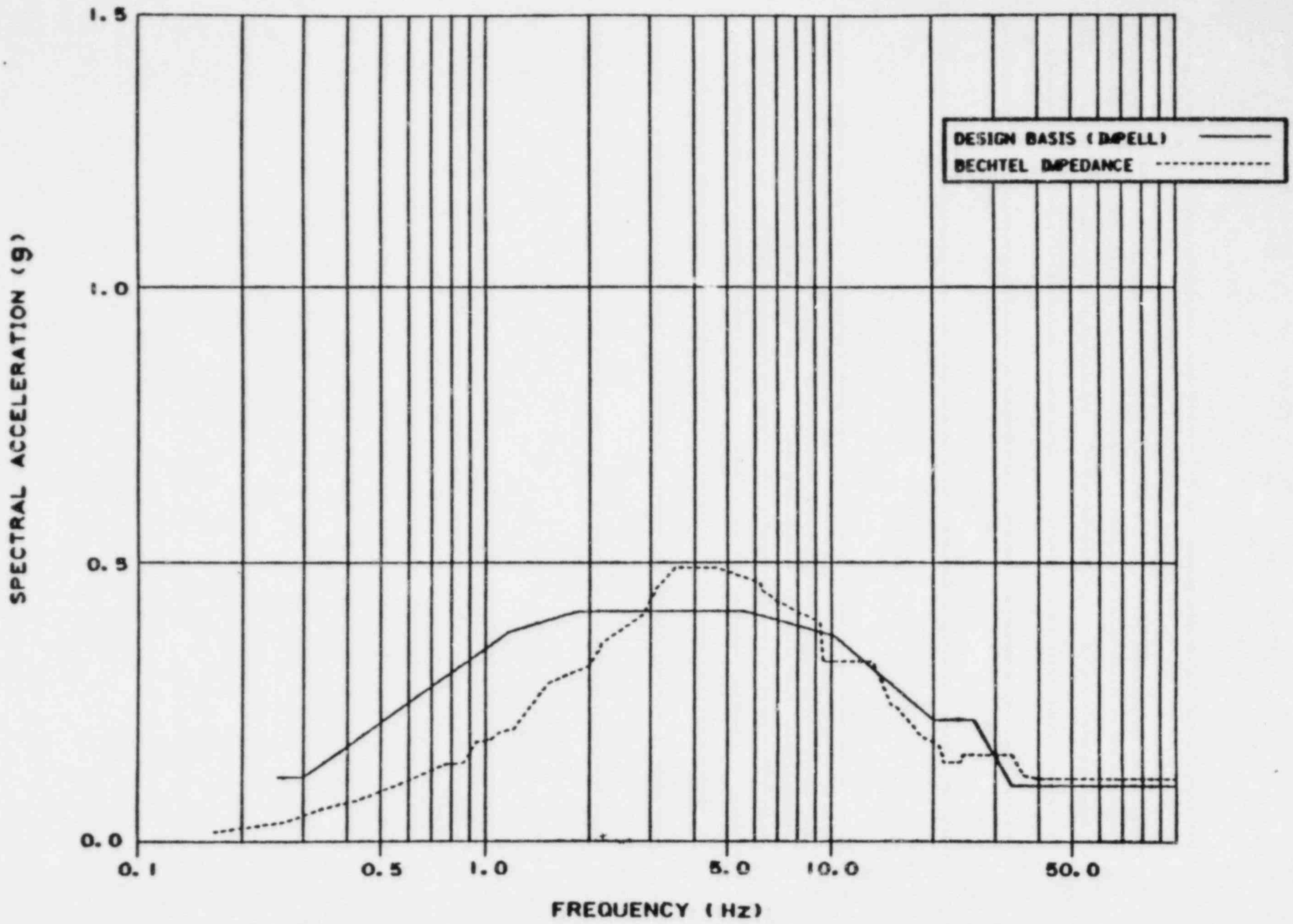


FIGURE A-16-18

RESPONSE SPECTRA COMPARISON,
INTAKE STRUCTURE AT ELEV. 135'-0",
VERTICAL, OBE, 2% DAMPING

HOPE CREEK UNIT 1
CAD. LT2. REV 0

Revised Response
Revision 1
6/30/84

Response to NRC Audit

Meeting Date: January 11, 1984

Question No.: B.12

DRAFT

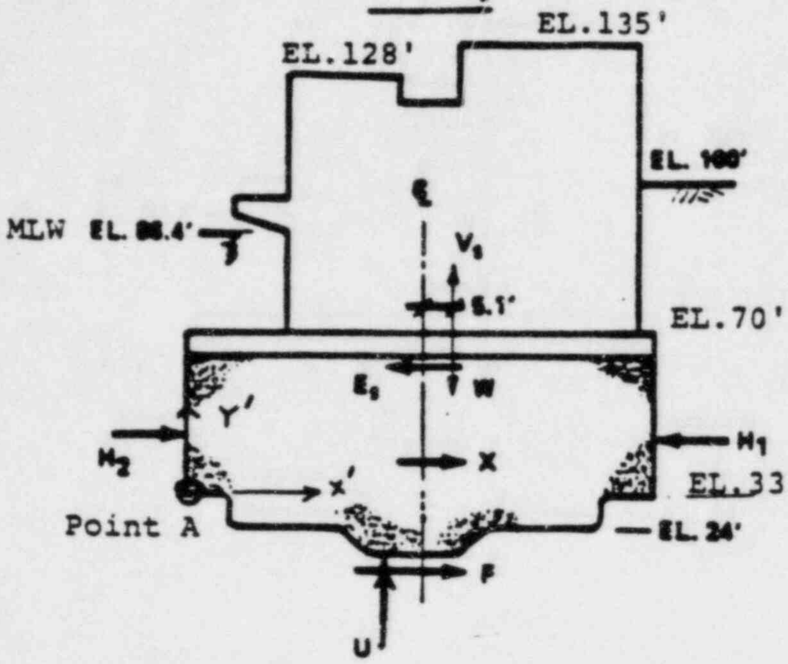
QUESTION: Provide static factor of safety against overturning for intake structure.

RESPONSE: The factor of safety against overturning for the intake structure as given in FSAR Appendix 3G was based on the energy method approach described in BC-TOP-4A (FSAR Reference 3.7-1). During discussions with the NRC, the NRC requested the factor of safety against overturning be calculated using conventional methods. The factor of safety against overturning using conventional methods is 1.12 which exceeds the minimum safety factor of 1.10 specified by SRP Section 3.8.5-II of NUREG-0800. FSAR Appendix 3G has been revised to indicate the factors of safety against overturning by both the energy method and conventional method.

Attached is a simplified calculation of the factor of safety against overturning using conventional methods.

Load Combination

$D + H + E_s$



OBJECTIVE :

Check Stability of Intake Structure against overturning by conventional method

			X'*	Y'*
FORCES	W	106,000K	57.1	-
	H_1	29,000	-	16.9
	H_2	22,760	-	5.7
	F	14,920	-	-6.0
	U	75,570	51.0	-
	E_s	21,210	-	28.5
	V_s	8,480	57.1	-
	X	15,860	-	4.5

E_s SAFE SHUTDOWN EARTHQUAKE LATERAL FORCE
F SUBGRADE FRICTION RESISTANCE
H_1 DRIVING EARTH PRESSURE
H_2 RESISTING EARTH PRESSURE
U UPLIFT DUE TO GROUNDWATER AND EXCESS PORE PRESSURE DURING SEISMIC EVENT, OR DESIGN BASIS FLOOD
V_s SAFE SHUTDOWN EARTHQUAKE VERTICAL FORCE
X TOTAL SIDE FRICTION
W DEAD LOADS

* X' and Y' are the coordinates of the load application point with respect to point A

OTM_A = OVERTURNING MOMENT AT 'A'

$= H_1y' + E_sy' + Ux' + V_sx'$

$= (29{,}000 \times 16.9) + (21{,}210 \times 28.5) + (75{,}570 \times 51.0) + (8{,}480 \times 57.1$

$= 5{,}432{,}900$ kft.

RM_A = RESTORING MOMENT AT 'A'

$= Wx' + Xy' + Fy' + H_2y'$

$= (106{,}000 \times 57.1) + (15{,}860 \times 4.5) - (14{,}920 \times 6.0) + (22{,}760 \times 5.7)$

$= 6{,}164{,}200$ kft.

F.S. = RM/OTM = 6,164,200/5.432,900 = 1.12 > 1.1 o.k.

Response to NRC Audit

Revised Response
Revision 1
6/30/84

Meeting Date: January 12, 1984

Question No: A.1

DRAFT

<u>Question</u>: Describe the procedures which assure that the post-modification seismic loads for the torus were examined and that the torus structure was found to be adequate to resist the post-modification seismic loads.

<u>Response</u>: The evaluation of post-modification seismic loads for the torus was separated into two parts: An evaluation for horizontal loads and an evaluation for vertical loads. The support design for the torus, i.e. pinned-pinned vertical columns and pinned lateral restraints, assures that horizontal and vertical behavior are uncoupled, thus allowing consideration of them separately. This was confirmed by the results of the seismic analysis of the unmodified structure, which also show that responses in each of the horizontal and vertical directions are dominated by one structural mode.

For horizontal loads, an evaluation was made of the effects of the torus modifications on the horizontal seismic analysis for the unmodified configuration. It was concluded that the effect of the torus modifications on the horizontal seismic response of the torus is negligible. The modifications added to the torus consist mainly of local column connection stiffening which does not significantly change the dominant horizontal torus frequency. The original analysis for horizontal loads is conservative, since the stiffening effect, though insignificant, would tend to increase the dominant frequency, resulting in lower accelerations applied to the torus because of the position of the frequency on the response spectrum curve. The evaluation described above was performed as part of the Hope Creek Plant Unique Analysis.

For vertical loads, a new analysis was performed using a finite element model of the modified torus. The results of this analysis are documented in the Hope Creek Plant Unique Analysis Report (PUAR).

The resulting vertical loads per support due to seismic loads are provided in PUAR Table 2-2.5-2. Combined column loads, which include seismic, hydrodynamic, and other loads, are reported in PUAR Table 2-2.5-4, and are compared to the allowable column loads. The maximum combined suppression chamber stresses, which include the effects of revised seismic loads and hydrodynamic loads, are reported in PUAR Table 2-2.5-3, and are compared to allowables therein. As can be seen by examining these tables, all column loads and component stresses are within allowable limits.

Response to NRC Audit

Meeting Date: January 12, 1984

Question No: A.3

Revised Response
Revision 1
6/30/84

DRAFT

<u>Question</u>: Describe how the effects of relative seismic displacements of the torus were considered for the stress evaluation of the vent system.

<u>Response</u>: The effects of relative seismic displacements of the torus on the vent system were considered by approximating the maximum relative support displacements using the floor response spectra. The maximum displacement for each support is predicted by $Sd = Sag/w^2$ where Sa is the spectral acceleration in g's at the high frequency end of the spectrum curve, g is the gravity constant, and w is the fundamental structural frequency of the torus in radians per second.

The resulting displacements from this calculation, 0.01 in. in the horizontal direction and 0.017 in. in the vertical direction, are small compared with those of other major vent system loadings such as SRV discharge and pool swell and have a neglible effect on the vent system. Therefore relative seismic displacements of the torus were not included in the stress evaluation of the vent system. To justify this assumption an evaluation of the effects of relative seismic displacements of the torus is provided as follows.

Imposing the torus seismic displacements on the vent system at the torus attachment points results in an increase in maximum primary membrane stress in the vent header of 0.54 ksi. The maximum increase in local primary membrane stress at the most highly stressed vent header - downcomer intersection is 0.01 ksi in the vent header and 0.01 ksi in the downcomer. Examining the maximum combined stresses shown in PUAR Table 3-2.5-3, it is apparent that these small increases in stress have a negligible effect on the adequacy of the containment.

DRAFT

Revised Response
Revision 1
July 10, 1984

Response to NRC Audit

Meeting Date: January 12, 1984

Question No.: A.4

QUESTION: Review the seismic design of all Seismic Category I tanks to determine whether the flexibility of the tank wall and the water mass within the tank were considered. For those tanks where these effects have not been considered, assess the impact of including these effects.

RESPONSE: All Seismic Category I tanks were reviewed to determine whether the flexibility of the tank wall and the water mass within the tank were considered. Review has indicated that in all tanks, except the diesel fuel oil storage tank, fluid mass and tank wall flexibility are addressed adequately and meet the guidelines of NUREG/CR-1161. The results of this review are shown in Table A.4-1.

In the case of the diesel fuel oil storage tank, an analysis to qualify the tank to include the effect of fluid mass and tank wall flexibility has been performed using the finite element method. The results of this revised analysis are compared with the original analysis results in Table A.4-2.

TABLE A.4-1

SUMMARY OF RESULTS FOR SEISMIC CATEGORY I TANKS
(EXCEPT DIESEL FUEL OIL STORAGE TANKS)

TANK DESCRIPTION	METHOD OF ANALYSIS	CRITICAL STRESS		
		LOCATION	STRESS (KSI)	
			CALCULATED	ALLOWABLE
FUEL OIL DAY TANKS Horizontal Length (L) = 31.5 in. Radius (R) = 18.0 in. Thickness = 3/8 in.	Finite element model was used. Fluid mass was considered in the analysis. Sloshing effects were also considered. Minimum frequency is 38 Hz. L/R = 7.3	Bolt, Tension Shear	5.1 1.7 (SSE)	20.2 8.2
JACKET WATER EXPANSION TANKS Vertical Height (H) = 56.0 in. Radius (R) = 12.0 in. Thickness = 3/8 in.	Tank was qualified by similarity to fuel oil day tanks and was found to be more rigid than day tanks. Fluid mass and sloshing effects were considered. H/R = 4.7	Bolt, Tension Shear	4.6 0.3 (SSE)	20.2 8.2
LUBE OIL MAKE-UP TANKS Vertical Height (H) = 87.5 in. Radius (R) = 15.25 in. Thickness = 3/8 in.	Tank was qualified by similarity to fuel oil day tanks and was found to be stiffer than day tanks. Fluid mass and sloshing effects were considered. H/R = 5.7	Bolt, Tension Shear	3.8 0.8 (SSE)	20.2 8.2

TANK DESCRIPTION	METHOD OF ANALYSIS	CRITICAL STRESS LOCATION	STRESS (KSI) CALCULATED	STRESS (KSI) ALLOWABLE
SACS EXPANSION TANKS Vertical Height (H) = 212.0 in. Radius (R) = 60.0 in. Thickness = 5/16 in.	Tank was analyzed by equivalent static method. Minimum frequency of 12.5 Hz was obtained using a beam model. Effect of water mas was considered. Sloshing effects were considered in the stress evaluation. H/R = 3.5	Bolt, Tension Shear	19.2 9.8 (SSE)	27.0 17.1
CONTROL AREA CHILLED WATER SYSTEM HEAD TANKS Horizontal Length (L) = 61.0 in. Radius (R) = 12.0 in. Thickness = 3/16 in.	Equivalent static analysis was performed. Fundamental frequency was found to be in the rigid range (>39Hz). Effect of water mass was considered. Sloshing effects were considered in the stress evaluation L/R = 5.0	Shell, at lugs Shell, at 2"Ø inlet Shell, at 2"Ø outlet Shell, at 1"Ø drain Mounting Tab Bolt, Tension Shear	24.9 15.0 12.8 12.4 1.1 5.8 6.7 (OBE)	25.9 15.7 15.7 15.7 17.8 20.0 10.0

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TANK DESCRIPTION	METHOD OF ANALYSIS	CRITICAL STRESS LOCATION	STRESS (KSI) CALCULATED	STRESS (KSI) ALLOWABLE
HEAD TANKS FOR SERVICE WATER PUMP LUBRICATION SYSTEM Horizontal Length (L) = 63.0 in. Radius (R) = 21.0 in. Thickness = 3/16 in.	Minimum frequency of 31.1 Hz was obtained using beam model, including the water mass. Sloshing effects were considered in the stress evaluation. Equivalent static analysis was used. L/R = 3.0	Shell, at support	25.7	37.7
		Shell, at 1" Ø inlet	8.1	37.7
		Shell, at 2" Ø outlet	13.1	37.7
		Bolt, Shear	5.8	13.3
		Tension	10.4 (SSE)	18.1
HYDROPNEUMATIC ACCUMULATOR TANKS Vertical Height (H) = 192.0 in. Radius (R) = 60.0 in. Thickness = 3/4 in.	Tank was analyzed by finite element analysis, including the water mass. Minimum frequency was 30.6 Hz. Sloshing effects were considered in the stress evaluation. H/R = 3.2	Shell, Circum-Tensile	16.8	17.5
		Inlet/outlet insert plate at shell	27.6	28.9
		Inlet/outlet nozzle to insert plate	27.3	28.9
		Bolt, Tension	39.2	52.5
		Shear	11.2 (SSE)	21.7

TABLE A.4-2

SUMMARY OF RESULTS FOR
DIESEL FUEL OIL STORAGE TANKS

METHOD OF ANALYSIS	CRITICAL STRESS			
	LOCATION	STRESS (ksi)		
		Calculated (Revised/Original)		OBE Allowable(1)
ORIGINAL Beam model was used. Minimum frequency >34 Hz.	Junction of cylindrical shell and saddle	(5.4/(2)) (6.5/(2))	OBE SSE	17.5
	Saddle Support	(11.8/2.0)(3) (13.8/2.6)	OBE SSE	20.6
REVISED Finite element qua[illegible] model was used. Fluid mass was uniformly distributed over the tank shell. Minimum frequency = 20 Hz.	Head	(1.9/2.0) (2.2/2.2)	OBE SSE	18.9
	Bolt, Shear (4)	(5.7/3.3) (9.0/4.8)	OBE SSE	10.0
HORIZONTAL	Saddle Stiffeners	(15.4/(2)) (17.4/(2))	OBE SSE	20.6
Length = 40 ft, Diameter = 11 ft. Shell thickness = 0.313 inch.	Base Plate	(7.7/ (2)) (12.4/(2))	OBE SSE	20.6

NOTES

1. SSE allowable stresses are not given since SSE calculated stresses are less than the OBE allowable stresses.

2. Original calculated stresses are not available.

3. Original analysis did not consider local stress evaluation.

4. Bolt does not experience any tension.

Revised Response
Revision 1
6/30/84

DRAFT

Response to NRC Audit

Meeting Date: January 12, 1984

Question No.: B.2

QUESTION: With respect to the ultimate capacity of the containment, expand the analysis to include the ultimate capacity of the materials and eliminate seismic considerations.

RESPONSE: The ultimate capacity analysis of the containment has been expanded to include the minimum specified tensile strengths of the materials and to eliminate seismic considerations. The resulting minimum ultimate internal pressure equals 190 psi. Therefore, the safety margin against the design pressure of 62 psi is 3.06.

Appendix 3I has been added to the FSAR to describe the ultimate capacity analysis of the containment.

The above response has also been given in response to Question 220.22.