PHILADELPHIA ELECTRIC COMPANY
2301 MARKET STREET
P.O. BOX 8699
PHILADELPHIA, PA. 19101
(215) 841-4000

EDWARD G. BAUER, JR.
VICE PRESIDENT AND GENERAL COUNSEL
EUGENE J. BRADLEY
ASSOCIATE GENERAL COUNSEL
DONALD BLANKEN
RUDOLPH A. CHILLEMI
E. C. KIRK HALL
T. H. MAHER CORNELL
PAUL AUERBACH
ASSISTANT GENERAL COUNSEL
EDWARD J. CULLEN, JR.
THOMAS H. MILLER, JR.
IRENE A. McKENNA
ASSISTANT COUNSEL

June 10, 1983

Docket Nos. 50-352
50-353

Mr. A. Schwencer, Chief
Licensing Branch No. 2
Division of Licensing
U.S. Nuclear Regulatory Commission

Subject: Limerick Generating Station (LGS)-Units 1&2 Open Items from NRC Draft Safety Evaluation Report (DSER) - Structural Engineering Branch (SEB)

References: Letter from A. Schwencer to E. G. Bauer, Jr., dated March 11, 1983

Dear Mr. Schwencer:

Attached please find our response to SEB-DSER open item numbers 1, 4, 8, and 9. These open items were transmitted to us via the referenced letter. With these responses, all SEB-DSER open items addressed in the referenced letter have been closed-out.

Response item numbers 1, 4, and 8 represent our formal response while item 9 indicates the FSAR page changes which will be made in Revision 21 (June 1983).

Very truly yours,

E J Bradley

Attachments

Copy to: See Service List

B001
1/1

cc:	Judge Lawrence Brenner	(w/ enclosure)
	Judge Richard F. Cole	"
	Judge Peter A. Morris	"
	Troy B. Conner, Jr., Esq.	"
	Ann P. Hodgdon, Esq.	"
	Mr. Frank R. Romano	"
	Mr. Robert L. Anthony	"
	Mr. Marvin I. Lewis	"
	Judith A. Dorsey, Esq.	"
	Jacqueline I. Ruttenberg	"
	Thomas Y. Au, Esq.	"
	Mr. Thomas Gerusky	"
	Director, Pennsylvania Emergency Management Agency	"
	Steven P. Hershey	"
	Charles W. Elliott, Esq.	"
	Donald S. Bronstein, Esq.	"
	Mr. Joseph H. White, III	"
	David Wersan, Esq.	"
	Robert J. Sugarman, Esq.	"
	Martha W. Bush, Esq.	"
	Atomic Safety and Licensing Appeal Board	"
	Atomic Safety and Licensing Board Panel	"
	Docket and Service Section	"

ENCLOSURE 1

Response to LGS DSER-SEB Open Item Numbers 1, 4, 8, and 9

SEB (220)

DSER #(1)

Response Spectra Frequencies (3.7.1)

In the design of plant structures, systems and components, an operating basis earthquake (OBE) of .075g horizontal and a safe shutdown earthquake (SSE) of 0.15g horizontal were used. The values for the vertical component of the design response spectra are 2/3 of the horizontal values described above. The response spectra are based on data developed from records of previous earthquake activity and represent an envelope of motion expected at a sound rock site from a nearby earthquake.

Comparison between the LGS design response spectra and R.G. 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants", indicated that for frequencies between 1 hz and 5 hz, R.G. 1.60 exceeds the LGS design spectra. The applicant should discuss the significance of these exceedances on structures, piping, equipment and systems essential for the safe shutdown of the plant. The staff considers this to be a confirmatory item.

Response:

The LGS design response spectra (Figure 3.7-1), in conjunction with the damping values shown in Tables 3.7-1 and 3.7-2, forms the design basis for LGS seismic analysis. For the seismic analyses of Category 1 structures (e.g. containment structures, reactor & control enclosures, diesel generator building, and the spray pond pump house) which contain safety related piping, equipment, and systems, damping values equal to 2% and 5% of critical are used for OBE and SSE, respectively. In accordance with RG 1.61, damping values equal to 4% and 7% of critical are acceptable for these reinforced concrete structures.

For comparison, the horizontal LGS design spectra are plotted with the RG 1.60 horizontal spectra in figures SER-1-1 and SER-1-2. These plots show that both the OBE and SSE horizontal LGS design spectra envelop the RG 1.60 spectra for frequencies greater than 1.0 hertz.

The vertical LGS design spectra are equal to two-thirds of the horizontal, per FSAR section 3.7.1.1. For comparison, the vertical LGS design spectra are plotted with the RG 1.60 vertical spectra on figures SER-1-3 and SER-1-4. These plots show that the RG 1.60 vertical spectra are generally higher than the LGS design spectra.

In the WASH-1255 report (Reference 1) Newmark has shown, based upon fourteen strong seismic motion records, that the ratio of vertical to the horizontal ground acceleration is two-thirds on the average. Although only 3 of 14 earthquakes considered were on rock, the ratio of the vertical to the horizontal acceleration is less in rock than in alluvium. Later, in the NUREG/CR-0098 report (Reference 2) Newmark recommended that the vertical design motion be taken as two-thirds of the horizontal across the entire frequency range. In addition, from the research based on the analysis of thirty vertical recordings made on "hard" or rock sites, the authors Rizzo, Shaw, and Snyder report (Reference 3) that the ratio of two-thirds between the vertical and horizontal accelerations is conservative. Their study included sites located in the Eastern United States, such as Blue Mountain Lake and New York. It is stated in their report that the RG 1.60 spectra envelop both rock and soil sites, and it is shown that the RG 1.60 provisions are overly conservative for "hard" or rock sites.

Since all principal Category 1 structures (containment structures, reactor & control enclosures, diesel generator building, and the spray pond pump house) of the project are founded on competent rock, we have concluded, based on the above discussion, that the difference between the LGS design vertical spectra and the RG 1.60 vertical spectra has no impact on safety-related structures, piping, equipment, and systems. The LGS design spectra, used in conjunction with the more conservative damping values, assures safe operation of the plant during a seismic event.

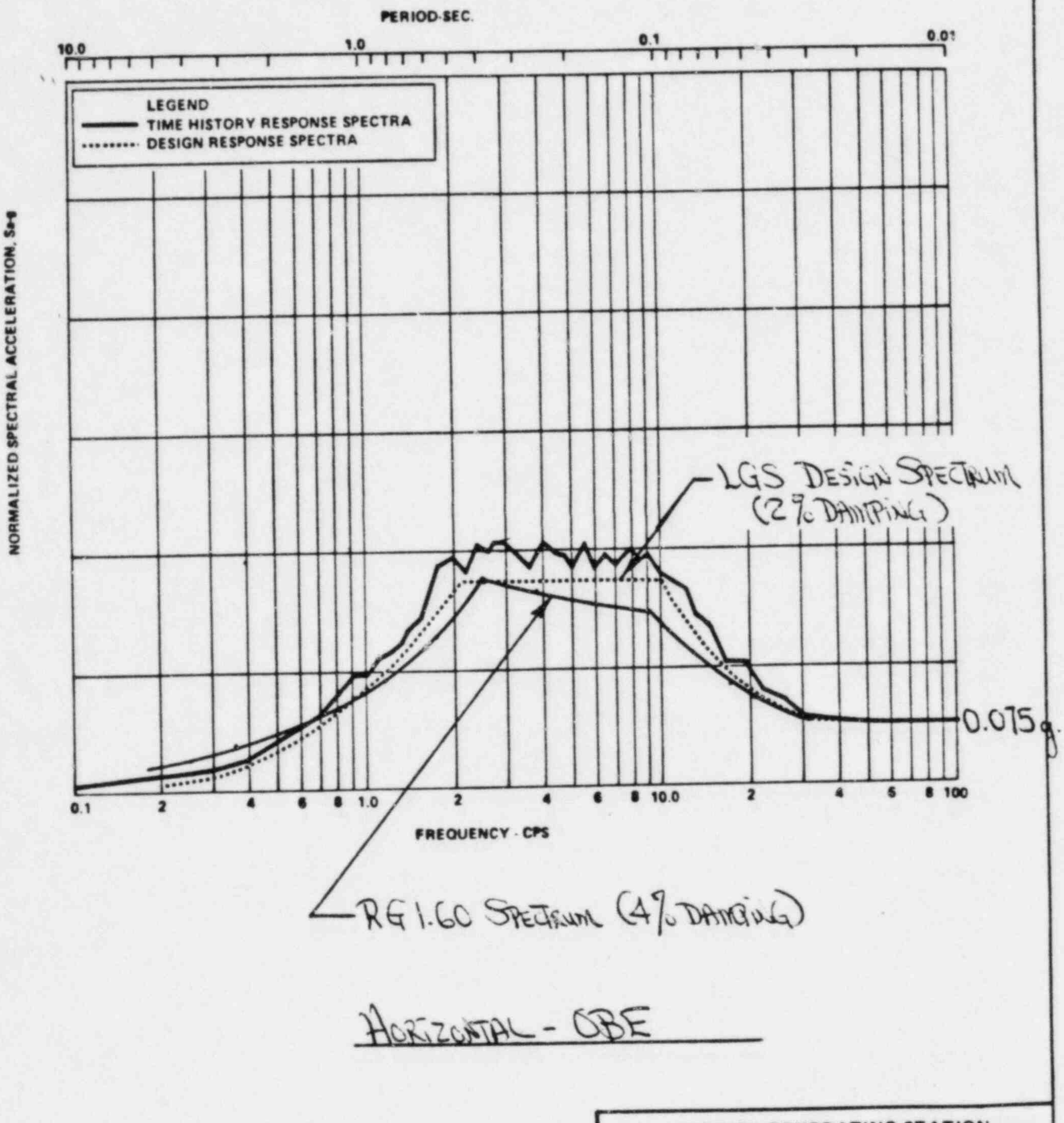
TGS/dmc 15/1

References for Enclosure 1

1) "A Study of Vertical and Horizontal Earthquake Spectra", USAEC Contract No. AT (49-5)-2667, WASH-1255. N. M. Newmark Consulting Engineering Services, Urbana, Illinois, April 1973.

2) "Development of Criteria for Seismic Review of Selected Nuclear Power Plants", USNRC Contract No. AT (49-24)-0116, NUREG/CR-0098, N. M. Newark Consulting Engineering Services, Urbana, Illinois, May 1978.

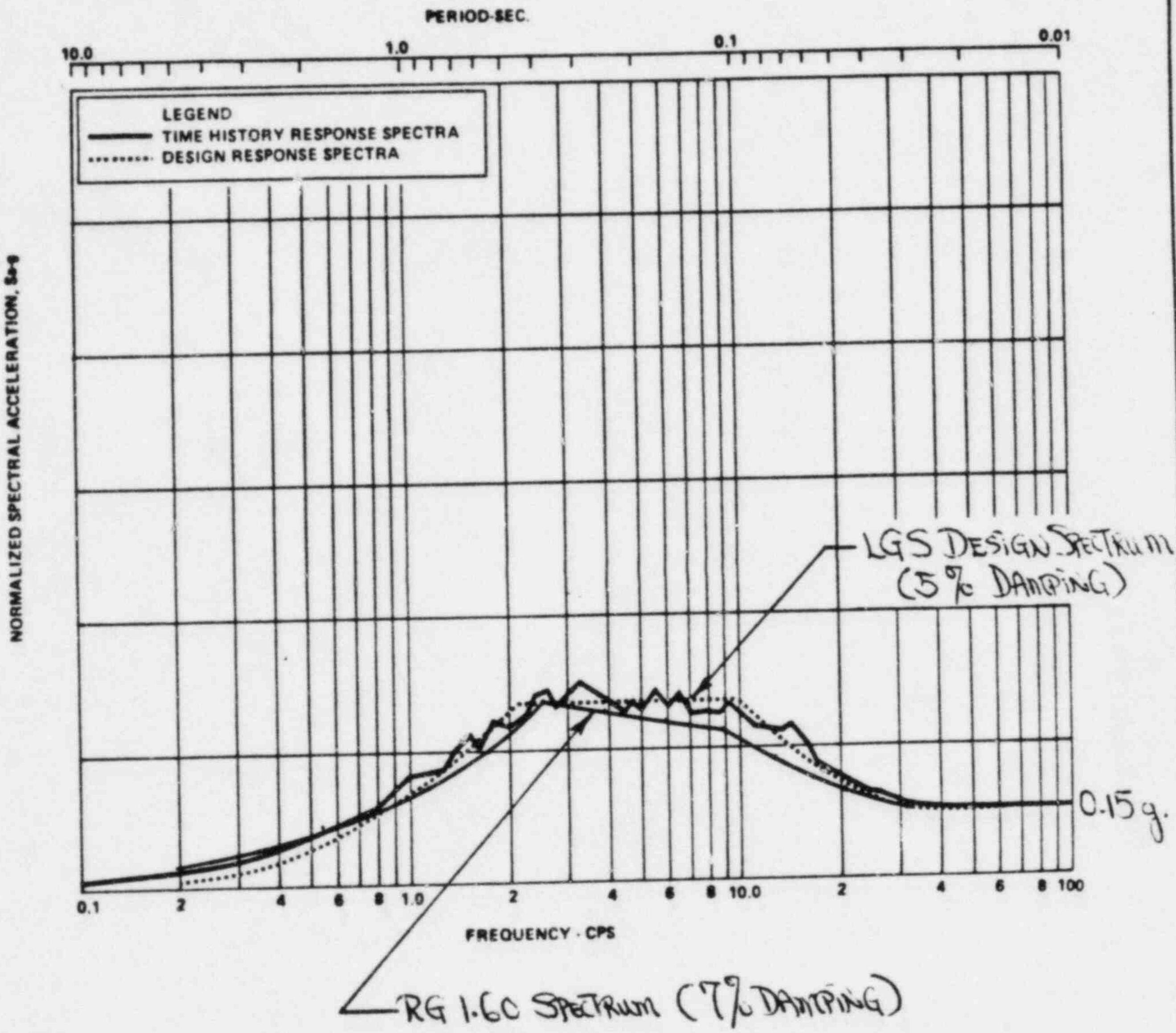
3) "Vertical Seismic Response Spectra", P. C. Rizzo, D. E. Shaw, and M. D. Snyder, Journal of the Power Division, ASCE, January 1976.



LIMERICK GENERATING STATION
UNITS 1 AND 2
FINAL SAFETY ANALYSIS REPORT

COMPARISON OF TIME HISTORY RESPONSE SPECTRA AND DESIGN RESPONSE SPECTRA (2% DAMPING)

FIGURE (SER-1-1)

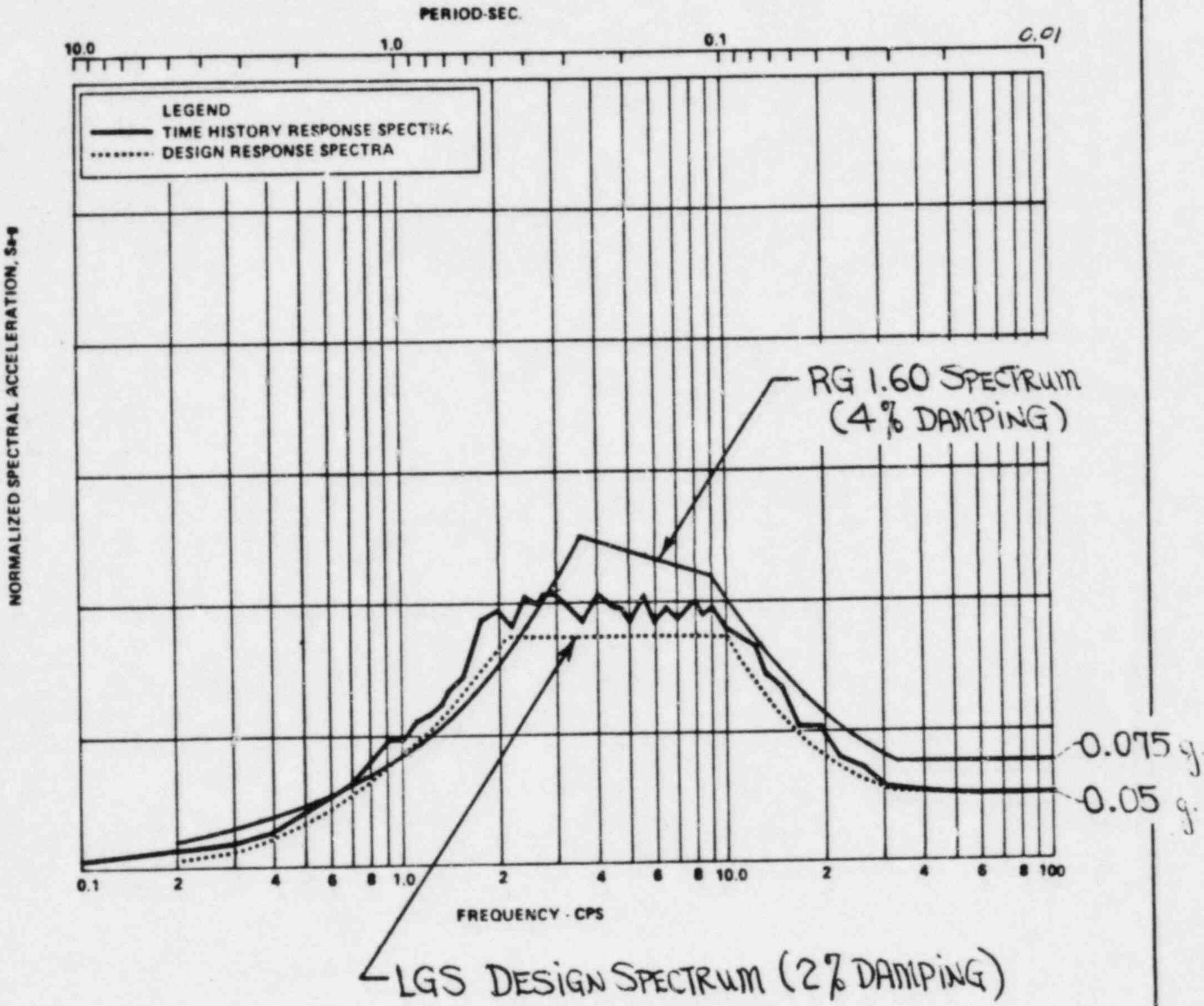


HORIZONTAL - SSE

LIMERICK GENERATING STATION
UNITS 1 AND 2
FINAL SAFETY ANALYSIS REPORT

COMPARISON OF TIME HISTORY RESPONSE SPECTRA AND DESIGN RESPONSE SPECTRA (5% DAMPING)

FIGURE (SER-1-2)

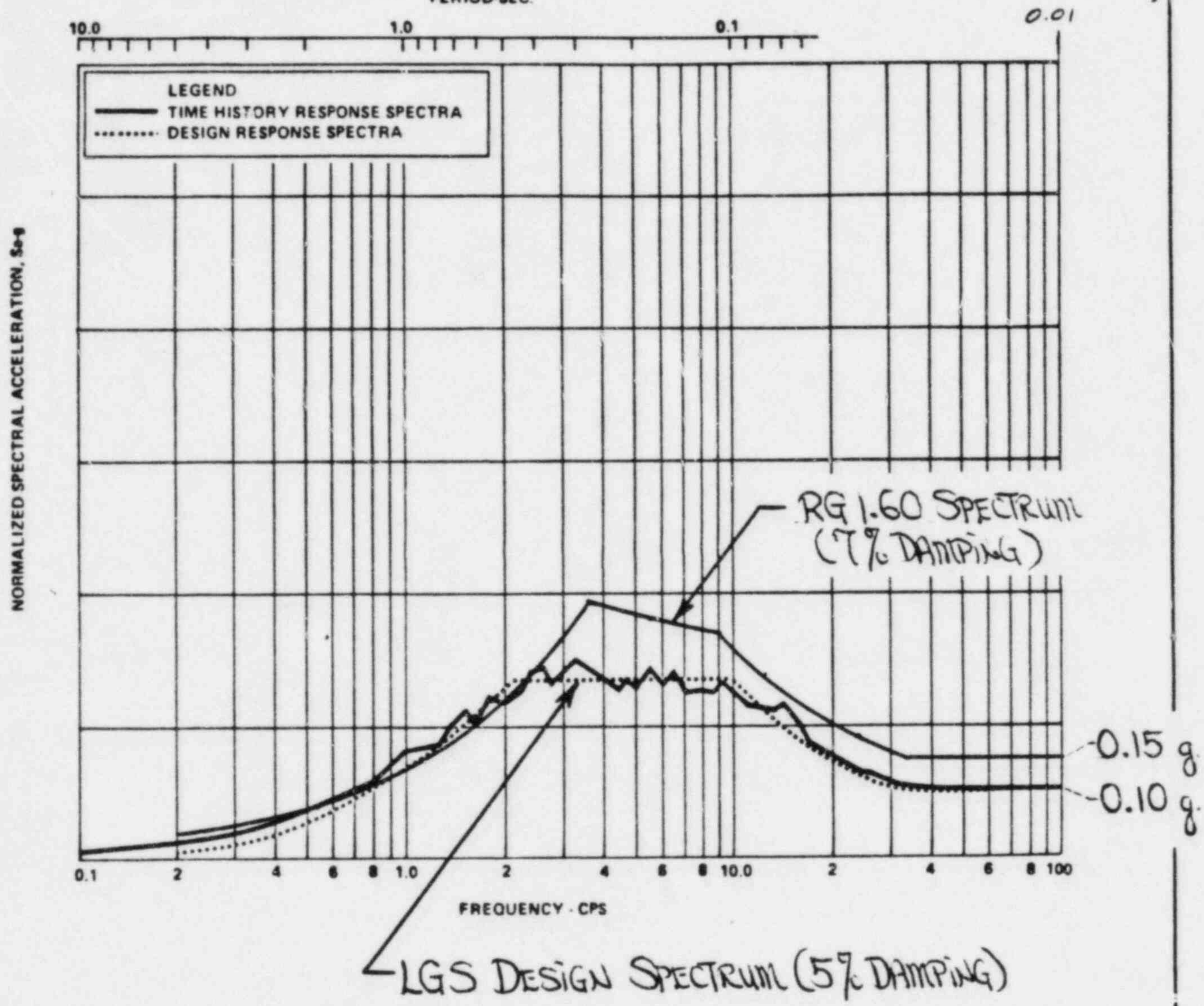


VERTICAL - OBE

LIMERICK GENERATING STATION
UNITS 1 AND 2
FINAL SAFETY ANALYSIS REPORT

COMPARISON OF TIME HISTORY RESPONSE SPECTRA AND DESIGN RESPONSE SPECTRA (2% DAMPING)

FIGURE (SER-1-3)



VERTICAL - SSE

LIMERICK GENERATING STATION
UNITS 1 AND 2
FINAL SAFETY ANALYSIS REPORT

COMPARISON OF TIME HISTORY RESPONSE SPECTRA AND DESIGN RESPONSE SPECTRA (5% DAMPING)

FIGURE (SER-1-4)

DSER #4 - Containment Dynamic Model (3.7.2)

In additicn to the problems with the rock parameters mentioned in (3) above, a structural design audit performed by the staff revealed problems with the containment dynamic model. These problems consist of errors in calculating the containment diaphragm slab spring stiffness and mass distribution of the containment building floors. The staff considers these problems to be open issues.

Response:

(A) Spring Stiffness of Diaphragm Slab

The diaphragm slab is rigidly connected to the containment wall and the reactor pressure vessel (RPV) pedestal wall. Hence, the equivalent spring representing the diaphragm slab in the vertical containment model is calculated using a fixed-fixed boundary condition. The error* in calculating the equivalent spring stiffness has been corrected.

The containment vertical uncracked seismic model has been reanalyzed with the new corrected spring stiffness for the SSE event. Comparisons of modal frequencies, modal participation factors, and selected acceleration response spectra between this revised spring model and the design model are presented in attachment 1. From these comparisons, it is concluded that:

a. Adjustment of the spring stiffness of the diaphragm slab induced negligible variation in the structural response for the containment structure, and

b. The current seismic analysis of the containment structure is adequate.

(B) Mass Distribution of Platforms within the Drywell of the Containment Structure

All platforms spanning between the containment wall and the RPV pedestal wall in the drywell are supported in the horizontal direction only at one end and are free to slide at the other end. They are supported at both ends in the vertical direction. Current horizontal and vertical seismic models of the containment structure assumed a platform mass distribution of 80% to the containment wall and 20% to the pedestal wall. A study has been performed using the uncracked containment models to determine the effects of different distribution of this platform mass.

The horizontal study model considers 100% of the platform masses lumped to the restrained ends and zero mass to the unrestrained ends. The vertical study model redistributes the platform masses to the containment wall stick and to the RPV pedestal stick of the model according to tributary area (approximately 60% to the containment wall and 40% to the pedestal wall).

* Discovered during the NRC-SEB audit of Bechtel in October of 1982.

The study models were analyzed for the OBE event and the results compared to the design values. These comparisons are shown in attachment 2 for the horizontal model and in attachment 3 for the vertical model.

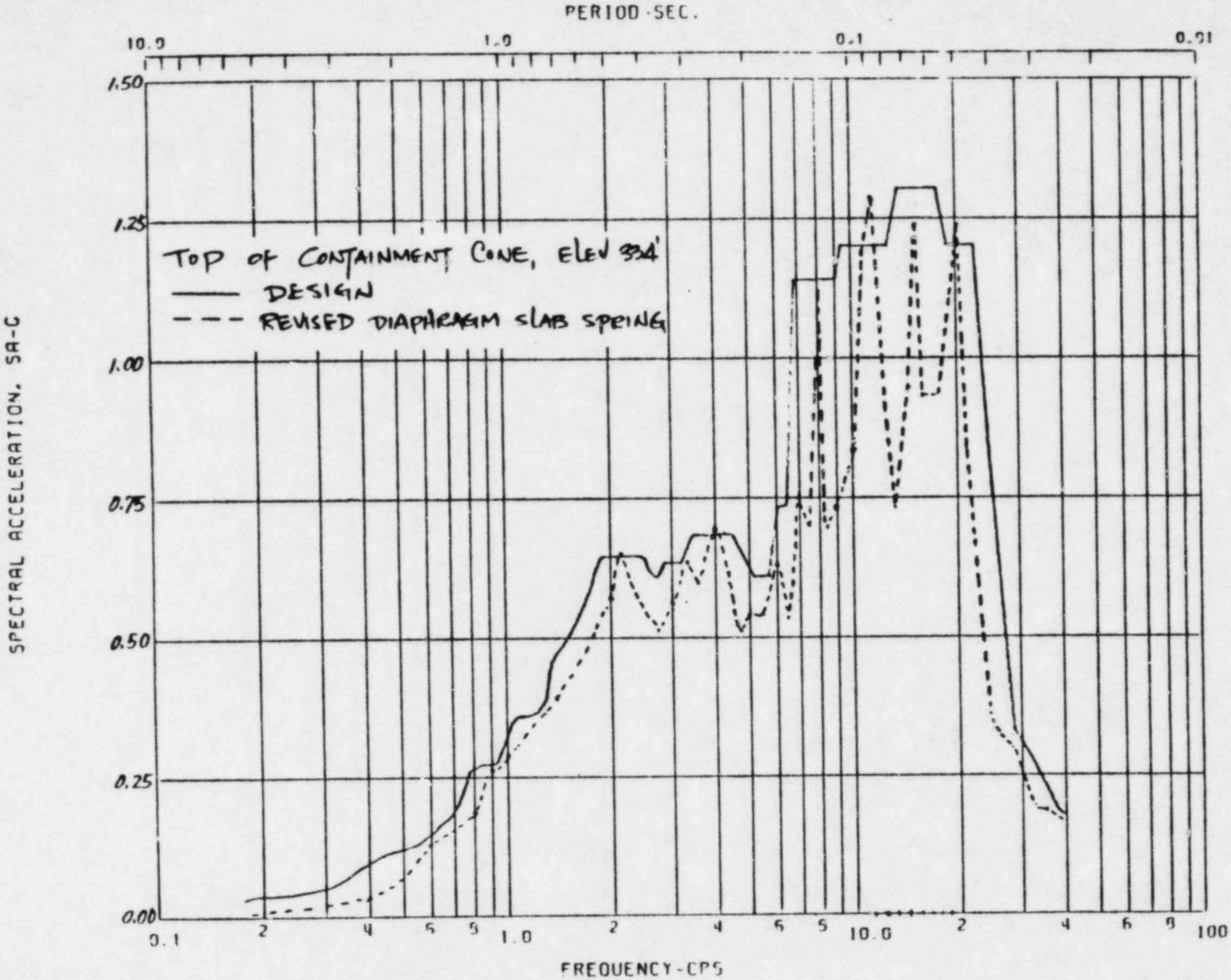
These comparisons show that the distribution of platform masses have negligible effect on the structural response of the containment structure. Therefore, the current seismic analysis of the containment structure is adequate.

TGS/dmc 15/2

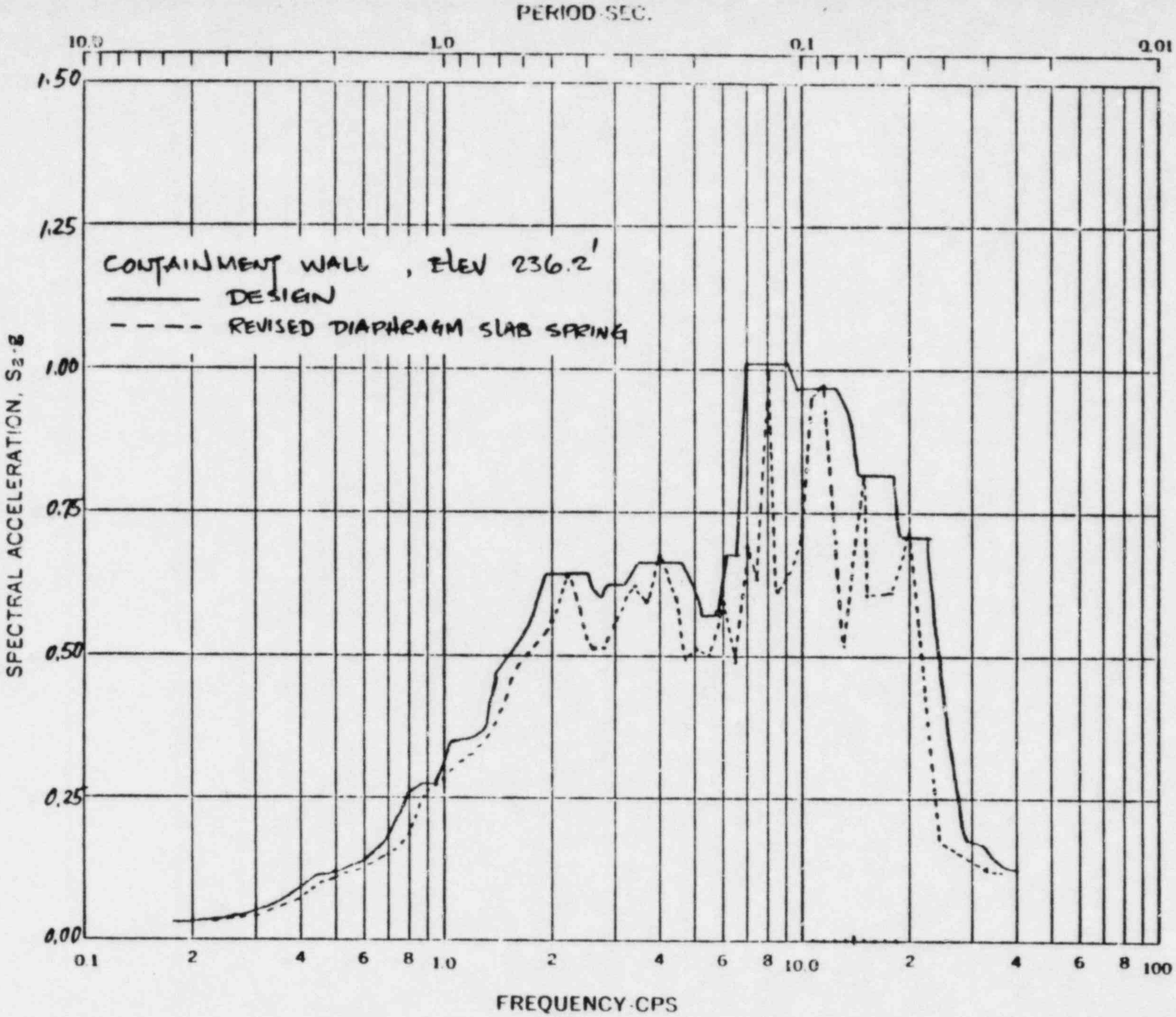
MODE	FREQUENCY (HZ)			PARTICIPATION FACTOR		
NO.	DESIGN	REVISED SPRING	% CHANGE	DESIGN	REVISED SPRING	% CHANGE
1	15.80	15.89	1	28.9	29.5	2
2	19.92	20.02	0	26.8	26.2	-2
3	35.26	35.36	0	3.9	3.9	0
4	55.61	55.60	0	15.3	15.3	0

COMPARISON OF MODAL PROPERTIES

CONTAINMENT MODEL - VARIATION IN DIAPHRAGM SLAB SPRING STIFFNESS

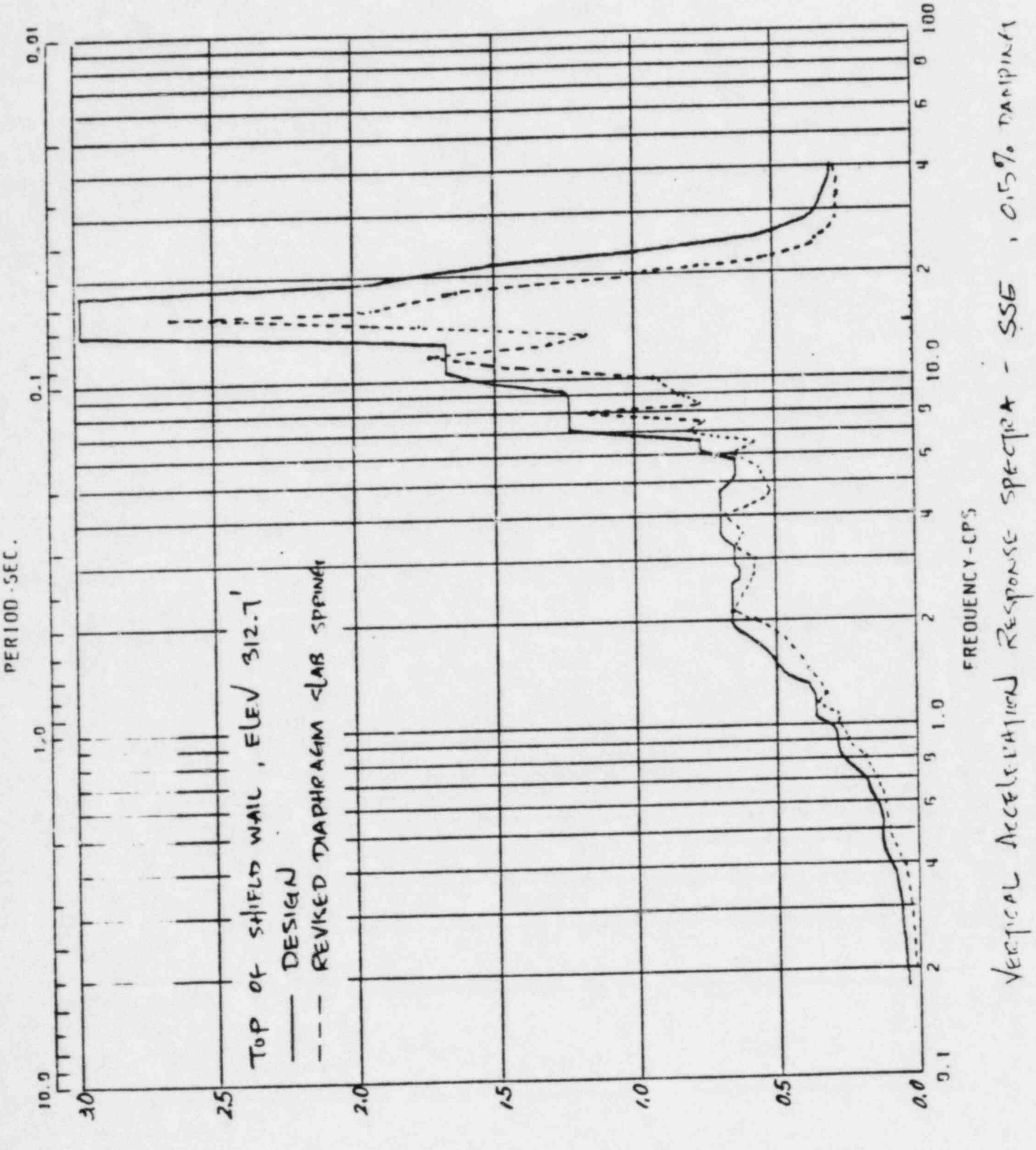


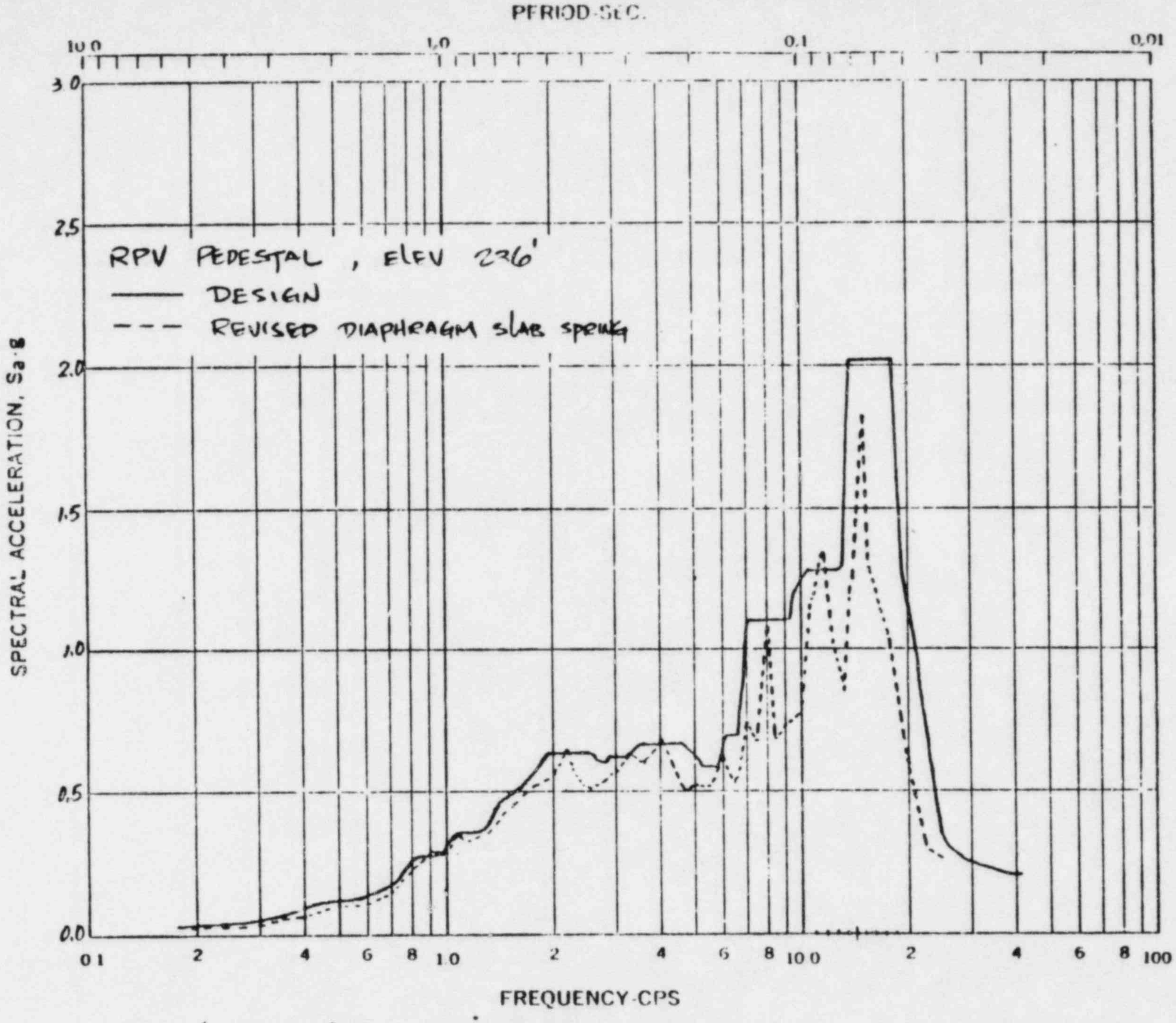
VERTICAL ACCELERATION RESPONSE SPECTRA - SSE, 0.5% DAMPING



VERTICAL ACCELERATION RESPONSE SPECTRA - SSE , 0.5% DAMPING

ATTACHMENT 1, PAGE 3 OF 5


PERIOD-SEC.
FREQUENCY-CPS
SPECTRAL ACCELERATION, SA-G
TOP OF SHIELD WALL, ELEV 312.7'
— DESIGN
--- REVISED DIAPHRAGM SLAB SPRING
VERTICAL ACCELERATION RESPONSE SPECTRA - SSE, 0.5% DAMPING

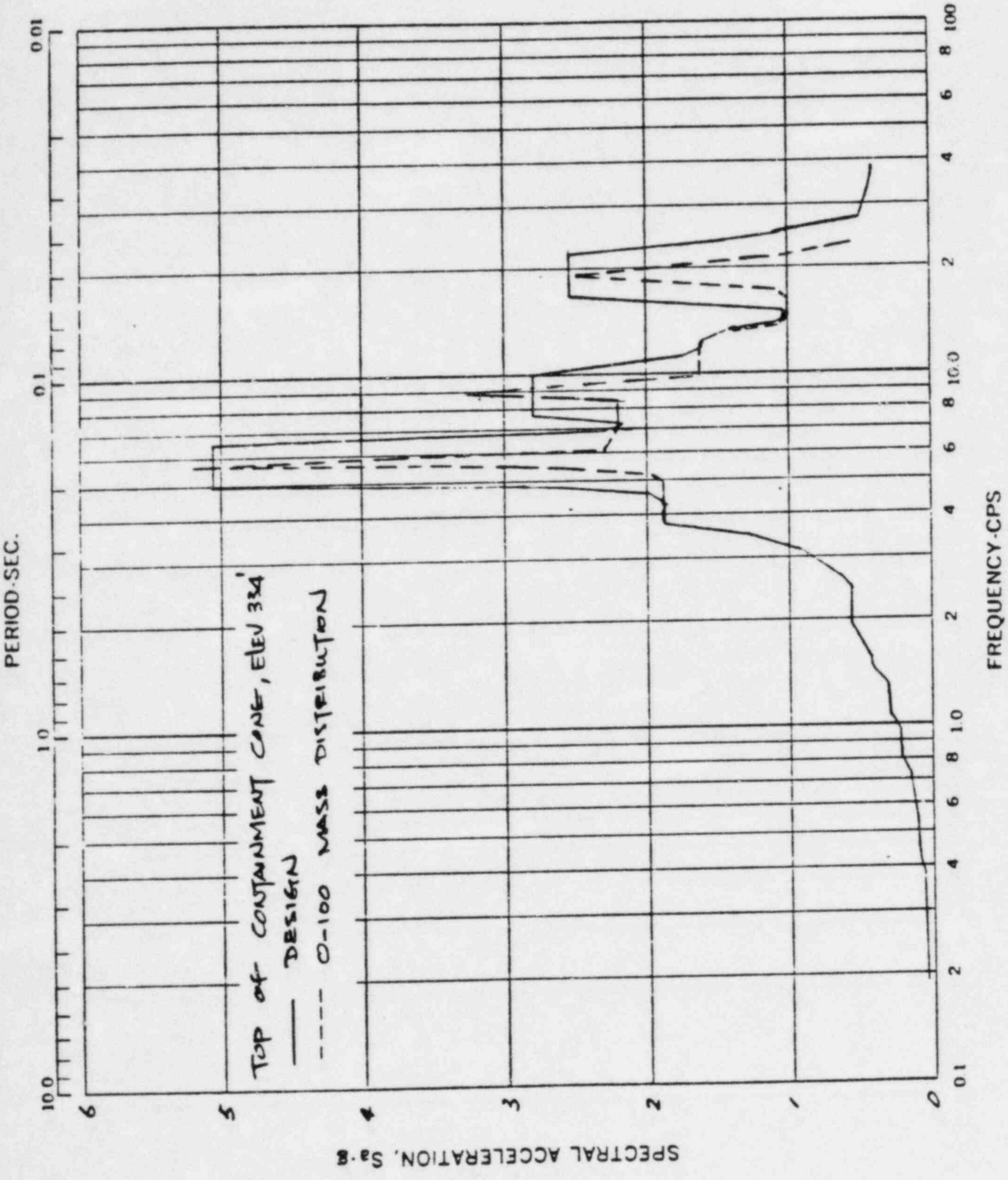


VERTICAL ACCELERATION RESPONSE SPECTRA - SSE , 0.5% DAMPING

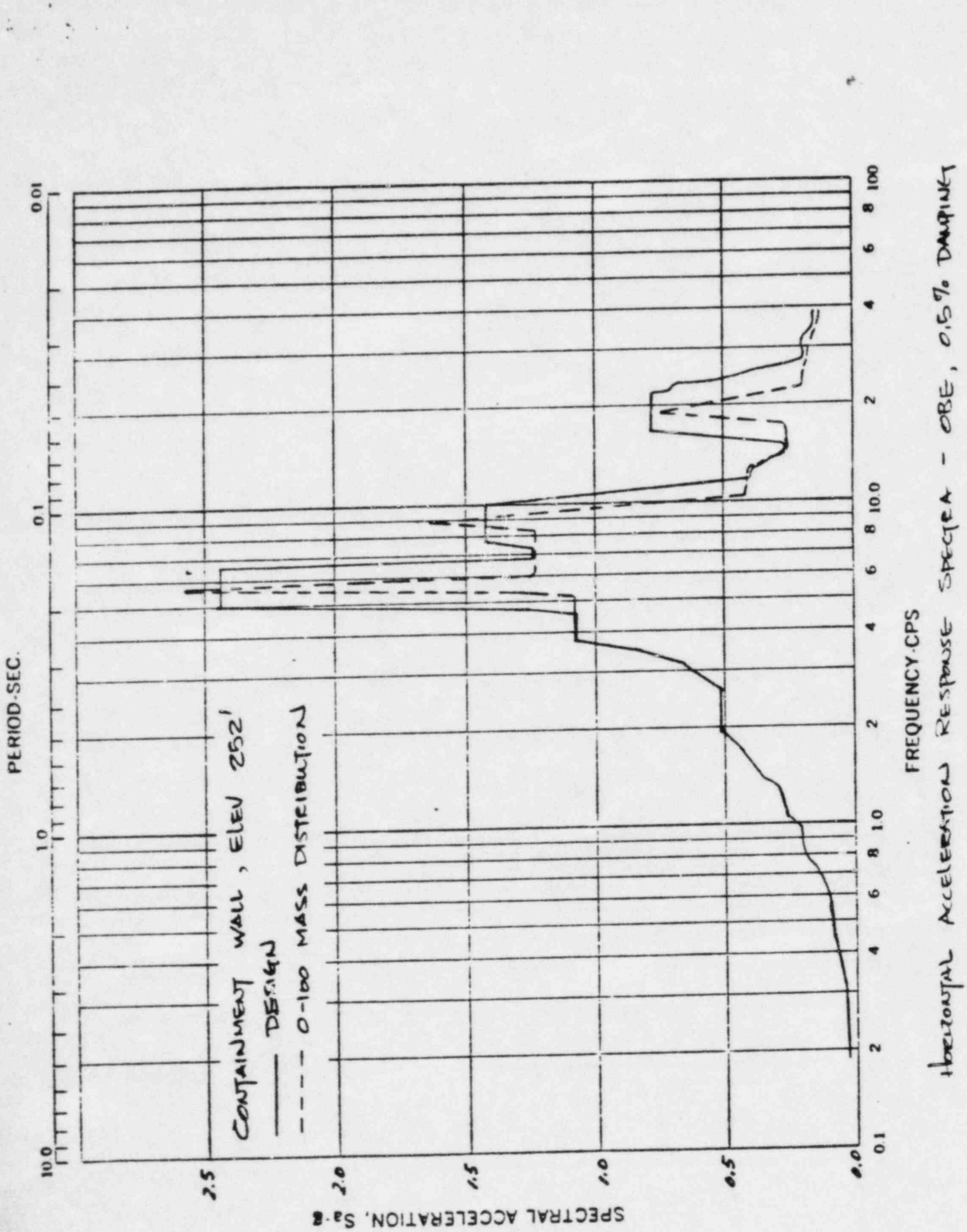
MODE No.	FREQUENCY (HZ)			PARTICIPATION FACTOR		
	DESIGN	REDISTRIBUTE MASS	CHANGE (%)	DESIGN	REDISTRIBUTE MASS	CHANGE (%)
1	4.34	4.33	0	13.02	13.05	0
2	5.80	5.80	0	22.72	22.73	0
3	7.16	7.16	0	14.02	13.99	0
4	8.92	8.93	0	18.33	18.33	0
5	10.57	10.57	0	0.95	0.96	1
6	17.87	17.82	0	4.27	4.42	4
7	19.82	19.79	0	15.11	14.70	-3
8	20.92	20.90	0	11.33	11.78	4
9	31.66	31.63	0	3.21	3.29	2
10	34.44	34.48	0	8.26	8.18	-1

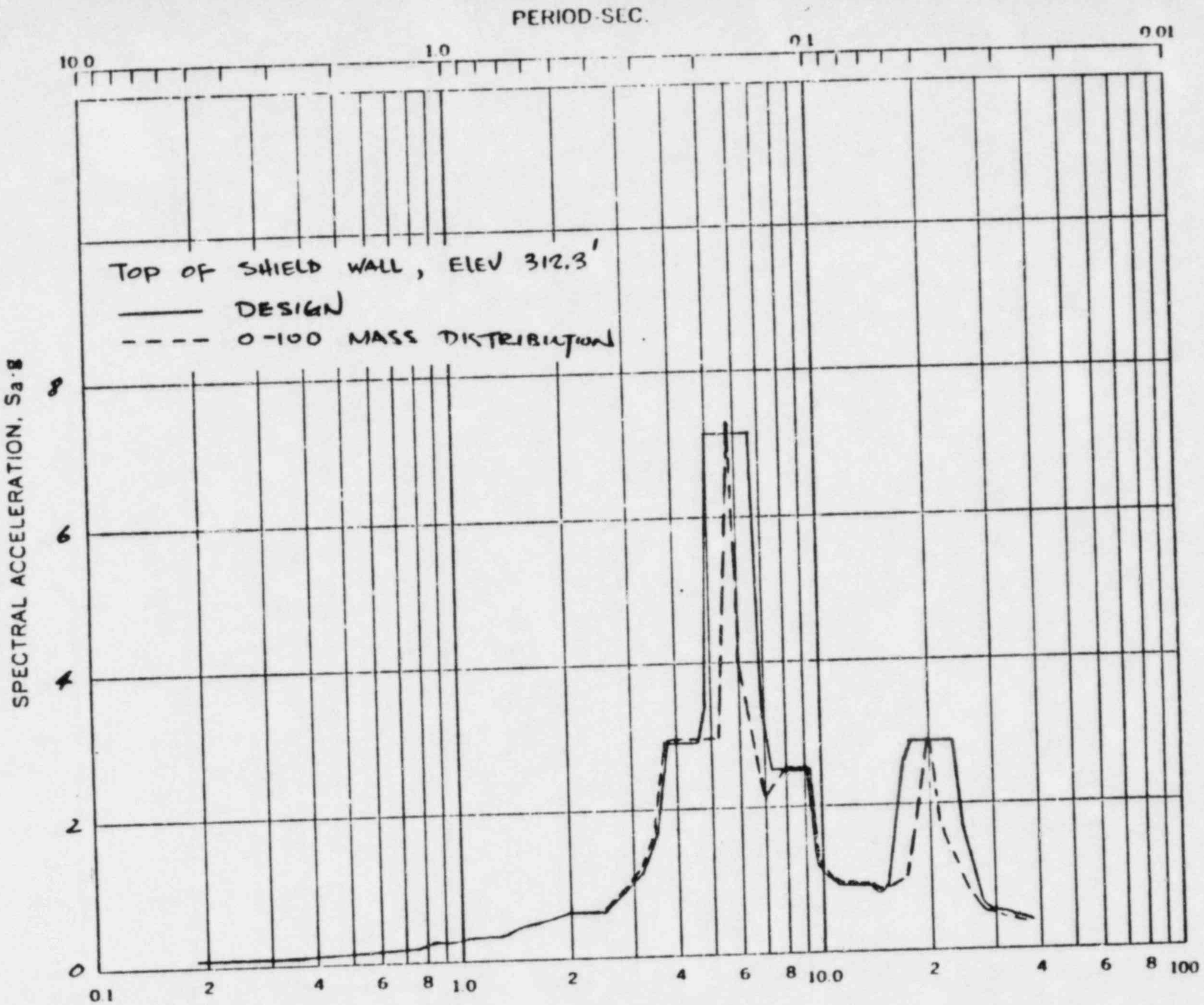
COMPARISON OF MODAL FREQUENCIES AND PARTICIPATION FACTORS

CONTAINMENT MODEL – REDISTRIBUTION OF HORIZONTAL PLATFORM MASS

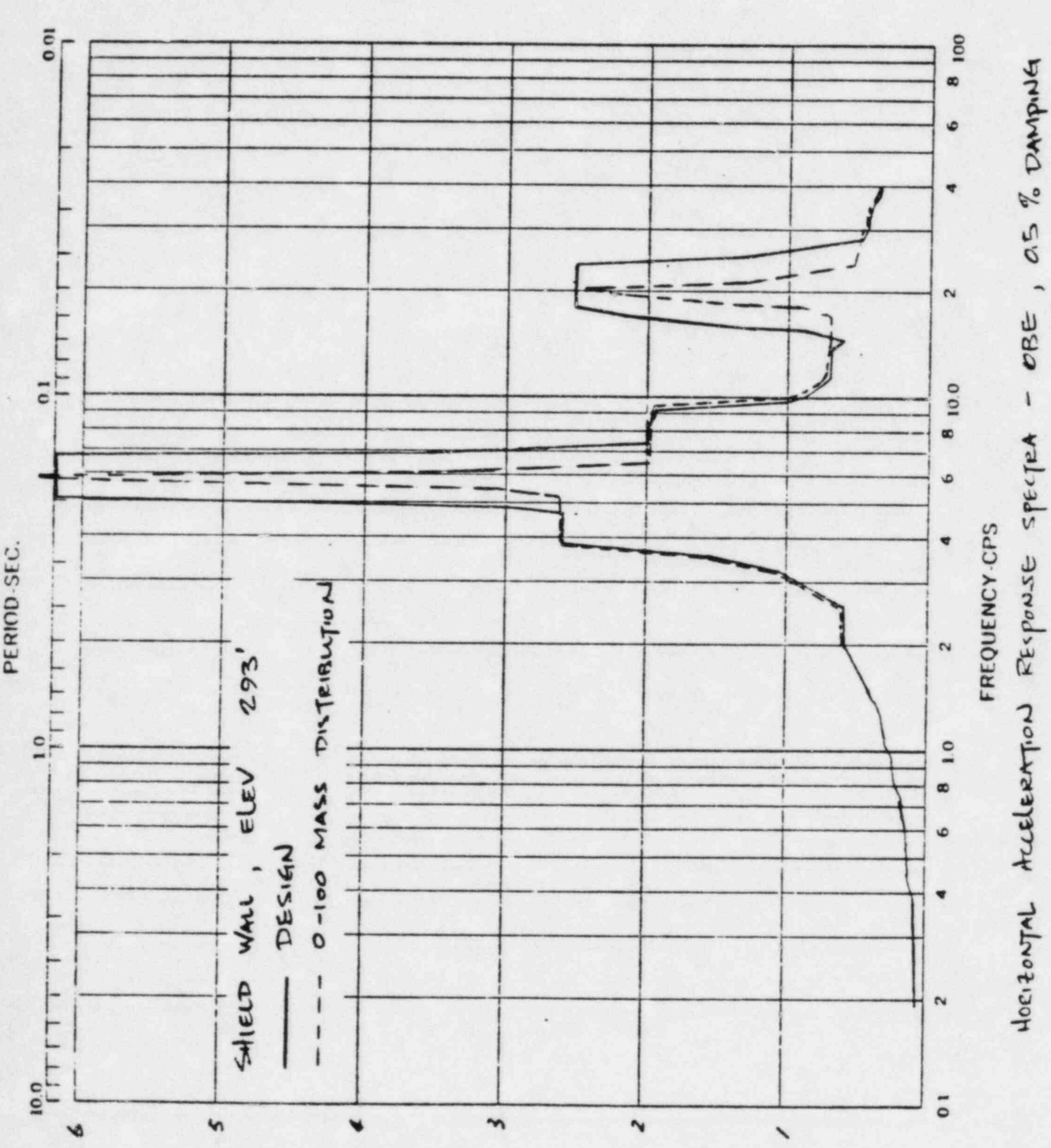


HORIZONTAL ACCELERATION RESPONSE SPECTRA - OBE, 0.5% DAMPING


PERIOD-SEC.
10.0
1.0
0.1
0.01
SPECTRAL ACCELERATION, Sa-g
2.5
2.0
1.5
1.0
0.5
0.0
FREQUENCY-CPS
0.1
2
4
6
8
1.0
2
4
6
8
10.0
2
4
6
8
100
CONTAINMENT WALL, ELEV 252'
DESIGN
0-100 MASS DISTRIBUTION
HORIZONTAL ACCELERATION RESPONSE SPECTRA - OBE, 0.5% DAMPING



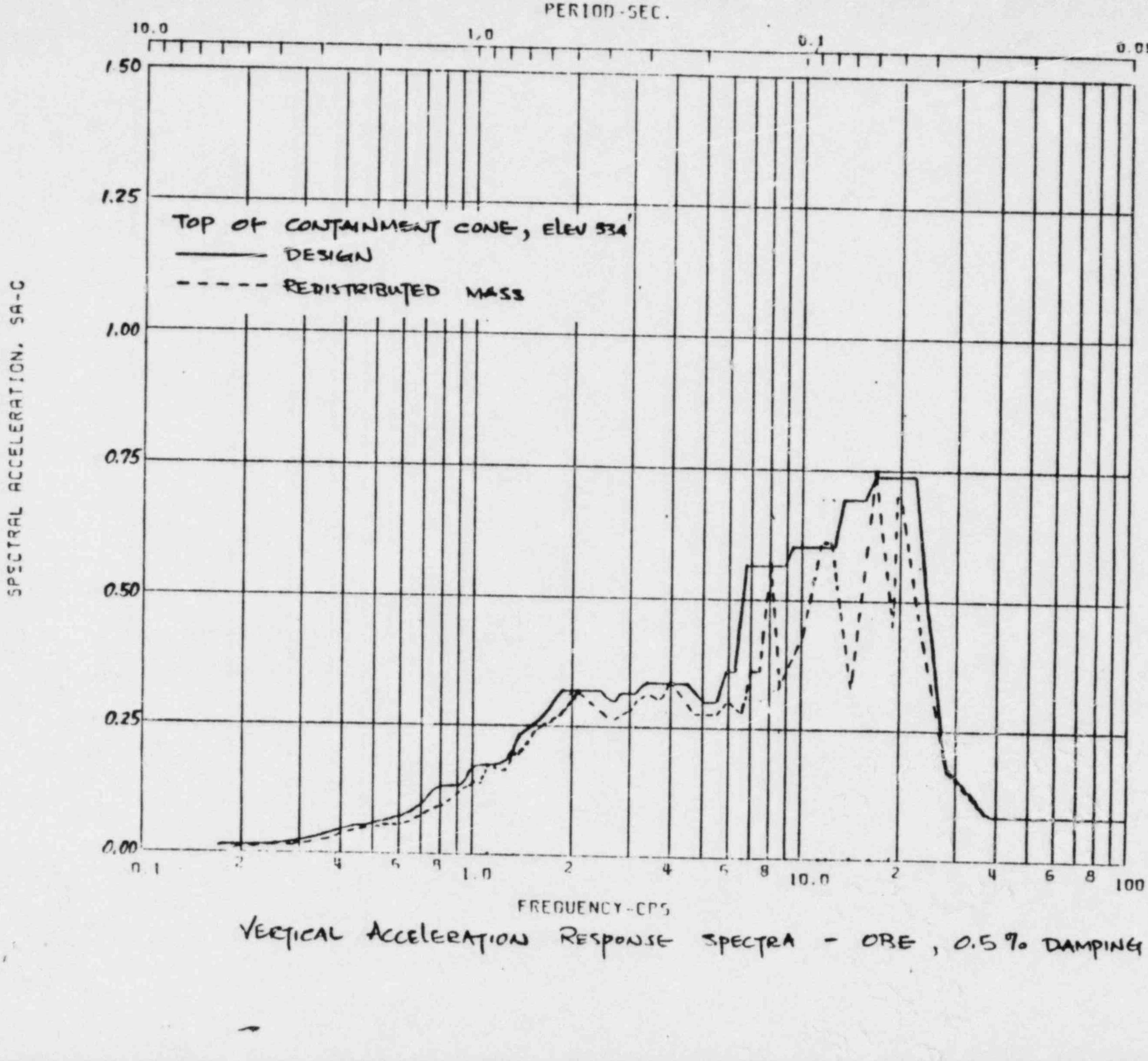
HORIZONTAL ACCELERATION RESPONSE SPECTRA - OBE, 0.5% DAMPING


PERIOD-SEC.
FREQUENCY-CPS
SPECTRAL ACCELERATION, Sa-g
SHIELD WALL, ELEV 293'
DESIGN
0-100 MASS DISTRIBUTION
HORIZONTAL ACCELERATION RESPONSE SPECTRA - OBE, 0.5 % DAMPING

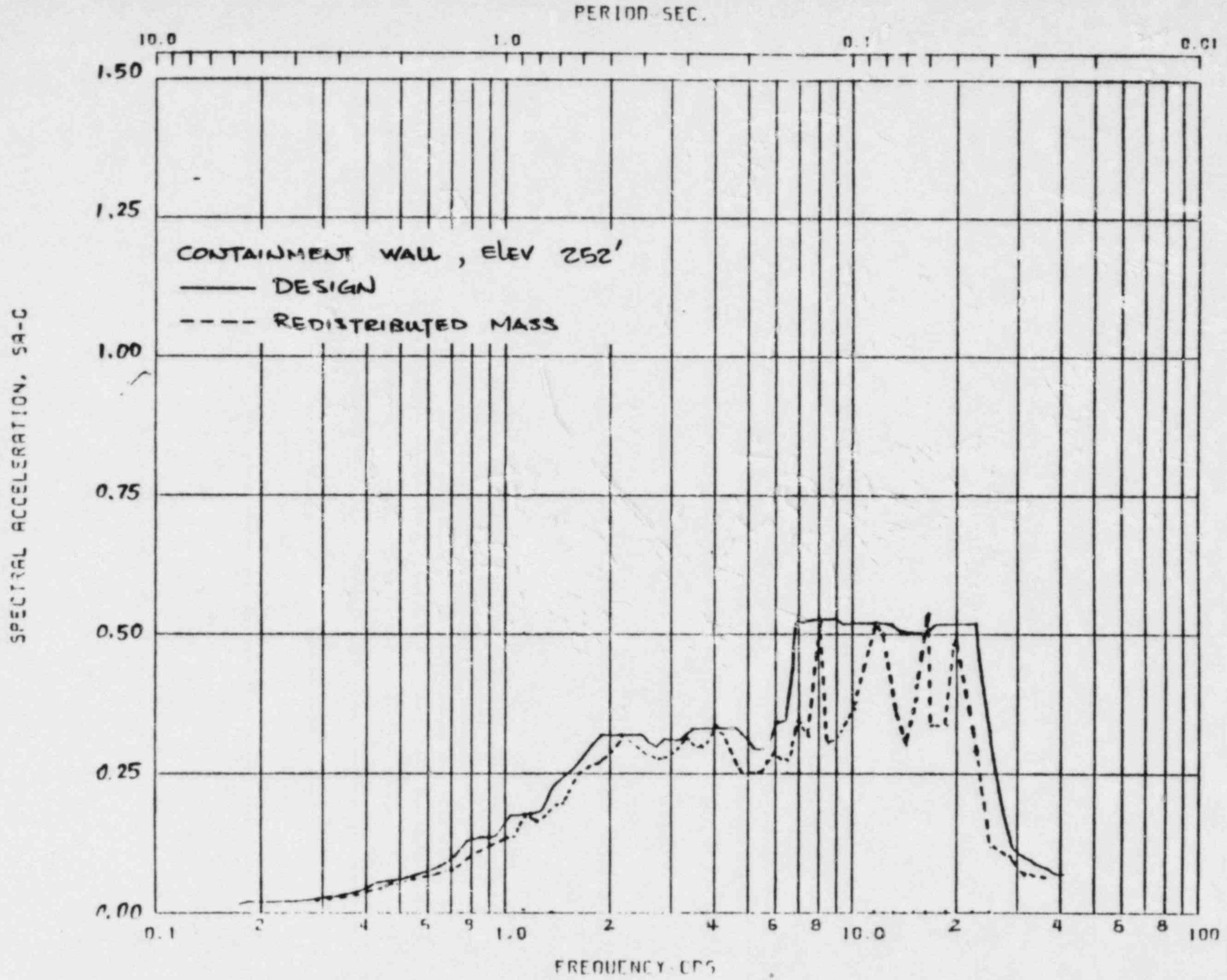
MODE NO.	FREQUENCY (HZ)			PARTICIPATION FACTOR		
	DESIGN	REDISTRIBUTE MASS	% CHANGE	DESIGN	REDISTRIBUTE MASS	% CHANGE
1	15.80	15.73	0	28.94	28.77	-1
2	19.92	19.95	0	26.84	27.04	1
3	35.26	35.10	0	3.92	3.88	-1

COMPARISON OF MODAL PROPERTIES

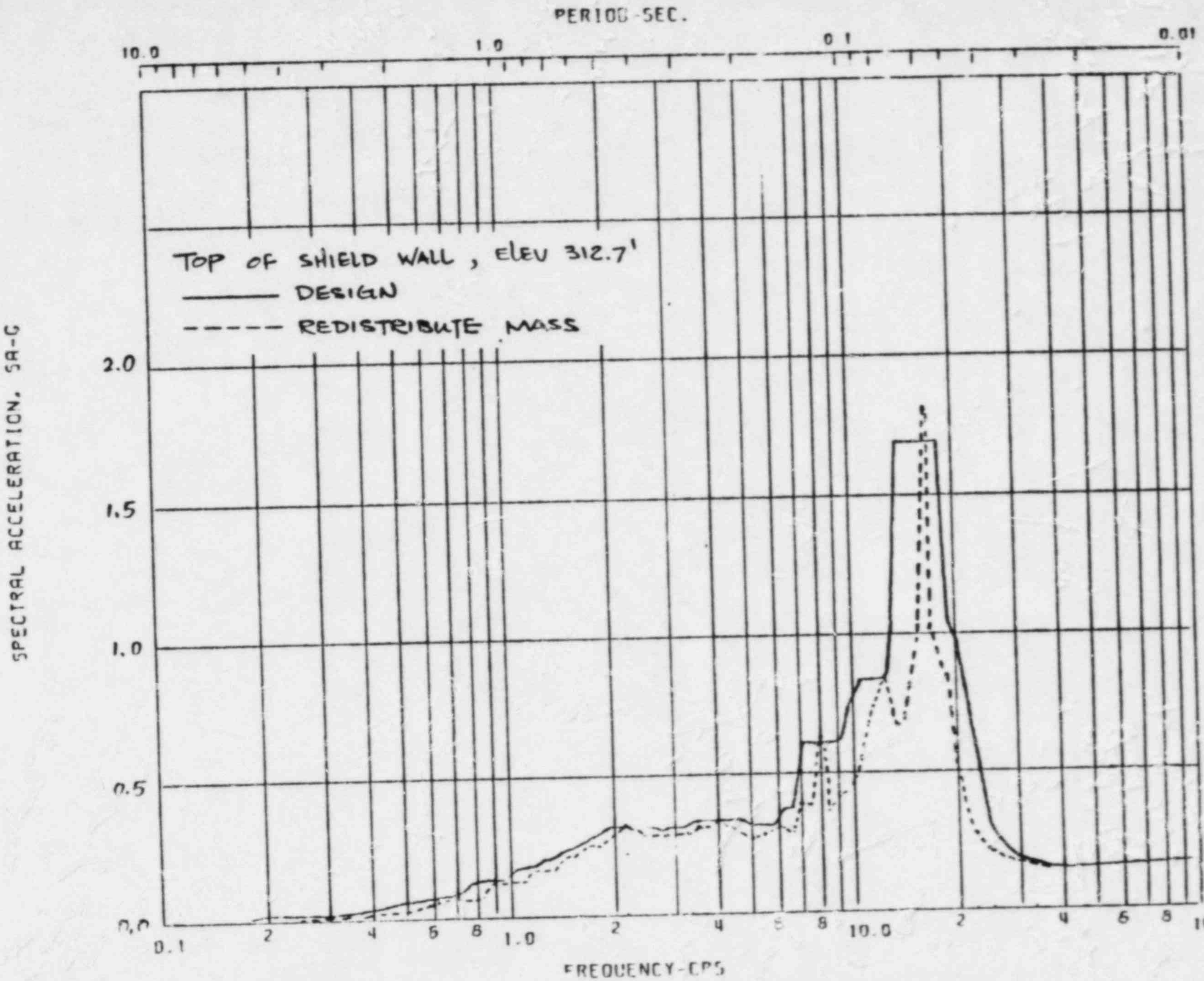
CONTAINMENT MODEL - VARIATION OF VERTICAL PLATFORM MASS



VERTICAL ACCELERATION RESPONSE SPECTRA - OBE, 0.5% DAMPING

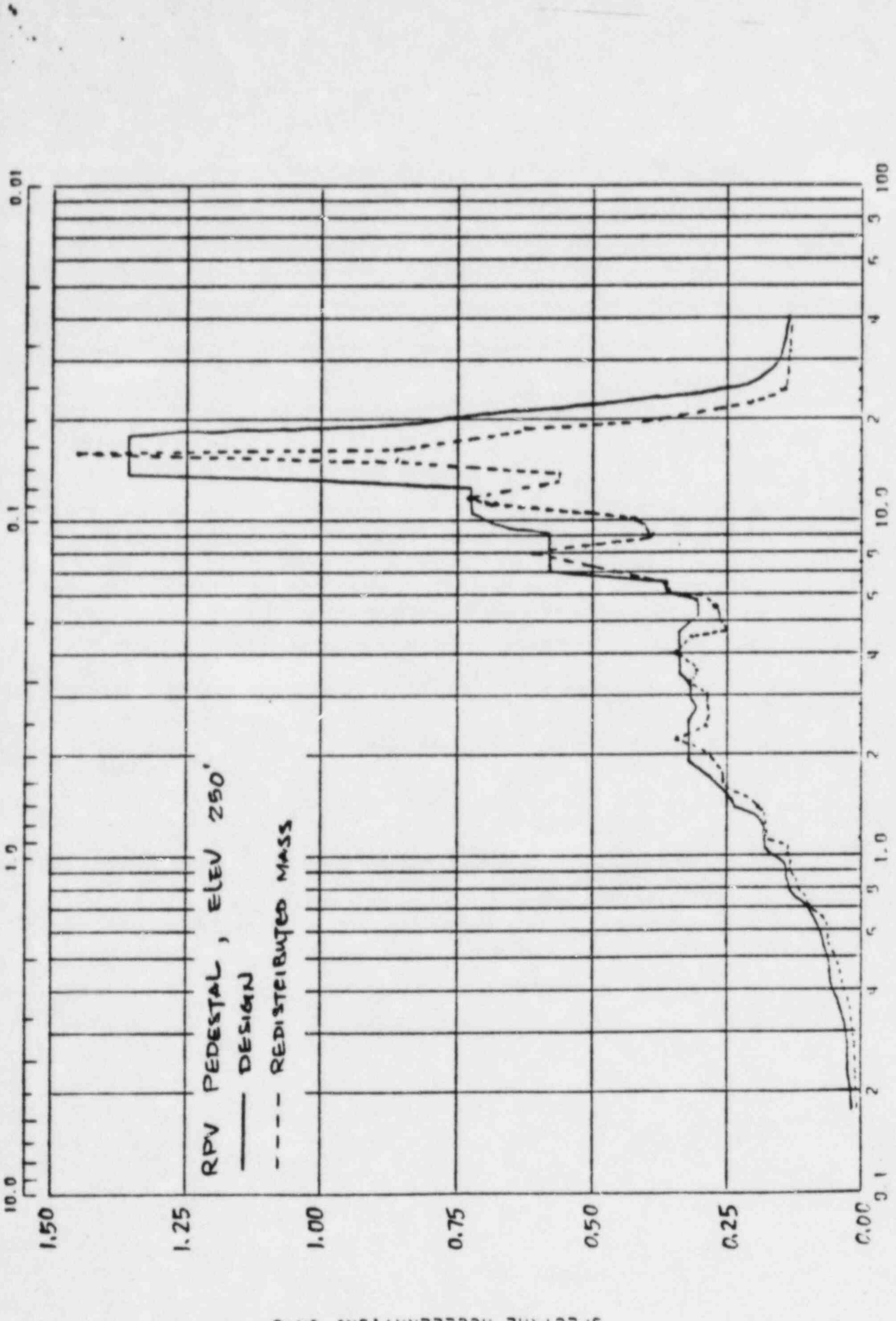


VERTICAL ACCELERATION RESPONSE SPECTRA - OBE, 0.5% DAMPING



VERTICAL ACCELERATION RESPONSE SPECTRA - OBE, 0.5% DAMPING

ATTACHMENT 3



VERTICAL ACCELERATION RESPONSE SPECTRA - OBE, 0.5% DAMPING

ATTACHMENT 3 PAGE 5 OF 5

DSER #8 - Reactor/Control Building Load Path Assumptions (3.8.3)

Seismic Category I structures other than the containment and its interior structures include the reactor/control building, diesel generator building, spray pond pump structure and the spray pond. All of the aforementioned structures are of concrete and structural steel.

During a structural design audit of the reactor/control building calculations by the staff, it was learned that the horizontal load transfer assumptions between the roof/floor diaphragms, the shear walls and the foundation were not clear. To verify the logic of the load path assumptions relied upon, the applicant should provide a description of the assumptions made and verification of agreement between the calculations and the assumptions. The staff considers this to be a confirmatory item.

Response:

In the design of shear walls, the following assumptions are made:

1) All exterior and interior bearing reinforced concrete walls (parallel to the direction of the earthquake) are considered to be subjected to earthquake forces and are accounted for in the stress analysis. Perpendicular walls are included in the analysis for horizontal torsion.

2) A floor is considered to be a rigid diaphragm if the entire floor has a reinforced concrete slab and the total horizontal forces at any level are assumed to be distributed to the vertical resisting elements (shear walls) in proportion to their relative rigidities.

3) Where a shear wall has openings, each pier between the openings is assumed to be tied at the top and the bottom of the openings by a stiff spandrel beam or by a foundation at the bottom. The point of inflection is assumed to be at the mid height of the pier. The total lateral force on the wall is distributed to the piers in proportion to their relative rigidities.

4) If the openings are close together in a row, the piers between them are assumed to be incapable of transfering lateral seismic forces.

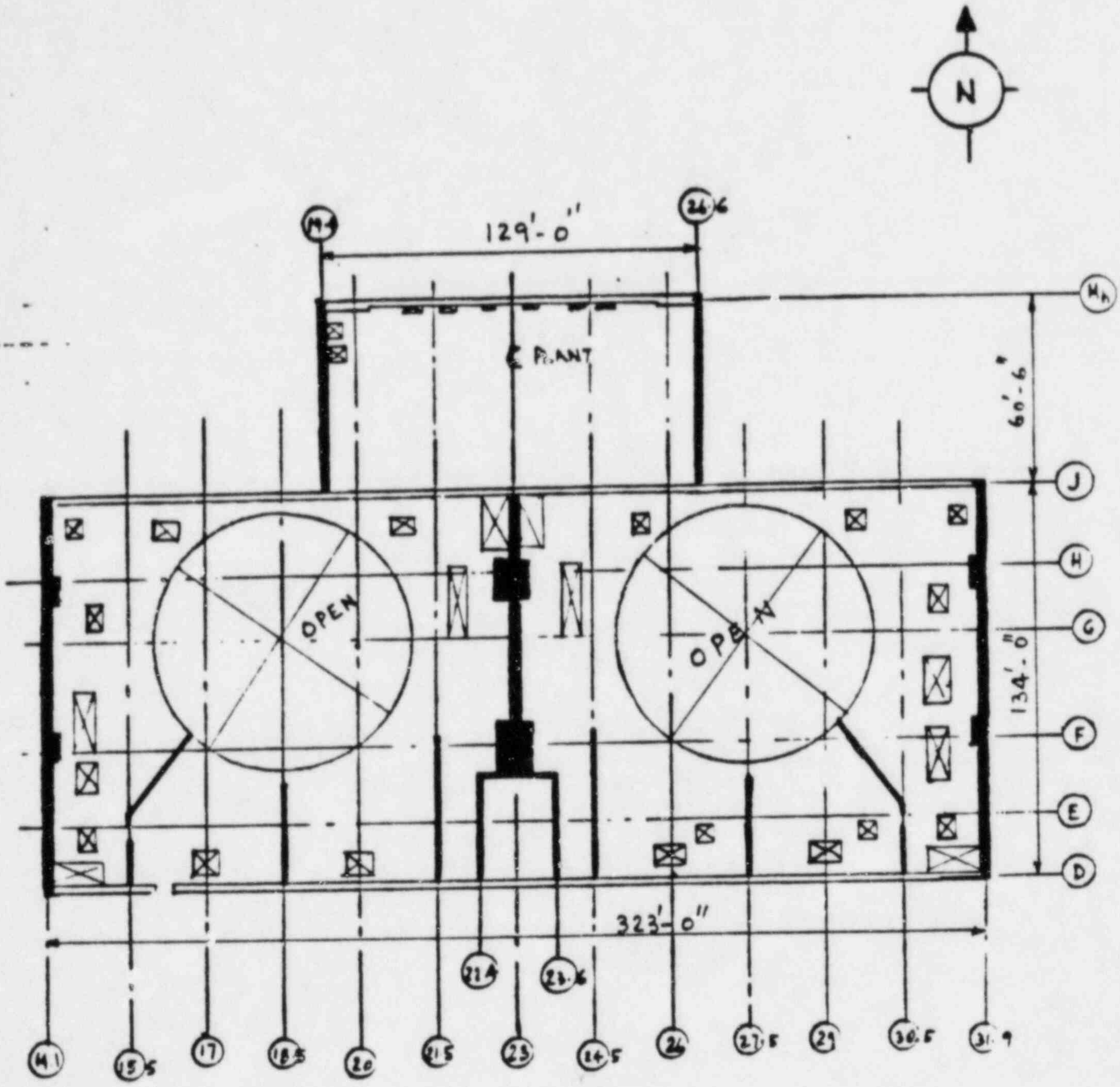
5) Where the floor slab is interrupted by large openings, such as the containment structures inside the Reactor Enclosure, a partial diaphragm configuration is considered. Only the shear walls connected to the partial diaphragm are considered for the lateral seismic force distribution. Grating floors are not considered to be acting as a diaphragm.

6) At each main floor level, each shear resisting element in a shear wall is checked for the assigned forces for shear and bending. For the Reactor/Control Enclosures the entire structure is considered as an integral multi-celled box structure and is checked for overall overturning moment due to lateral seismic forces in each (N-S or E-W) direction combined with the vertical seismic forces. The calculated stresses (tension and compression) from this procedure are then combined with stresses due to local bending in each of the shear resisting elements, including the piers, for the checking of concrete bending and shear stresses and designing of reinforcing steel requirements in accordance with the allowable values in ACI-318-71 code.

7) An illustrative example of the above assumptions and procedures is presented in the attachment. Walls on Col. line 19.4 and 26.6 between elev. 200/201' and 217' at the Reactor/Control Enclosures have been analyzed for the N-S earthquake.

TGS/dmc 15/3

ATTACHMENT
Sheet 1/7



PLAN AT EL 217'-0"

FOR REACTOR/CONTROL BLDG.

(REFER DWG. C-412 & C-701 FOR WALLS)

(

SHEAR DISTRIBUTION

V= 120,400 K

EARTHQUAKE N – S DIRECTION

* M_z = 1,965,000'K

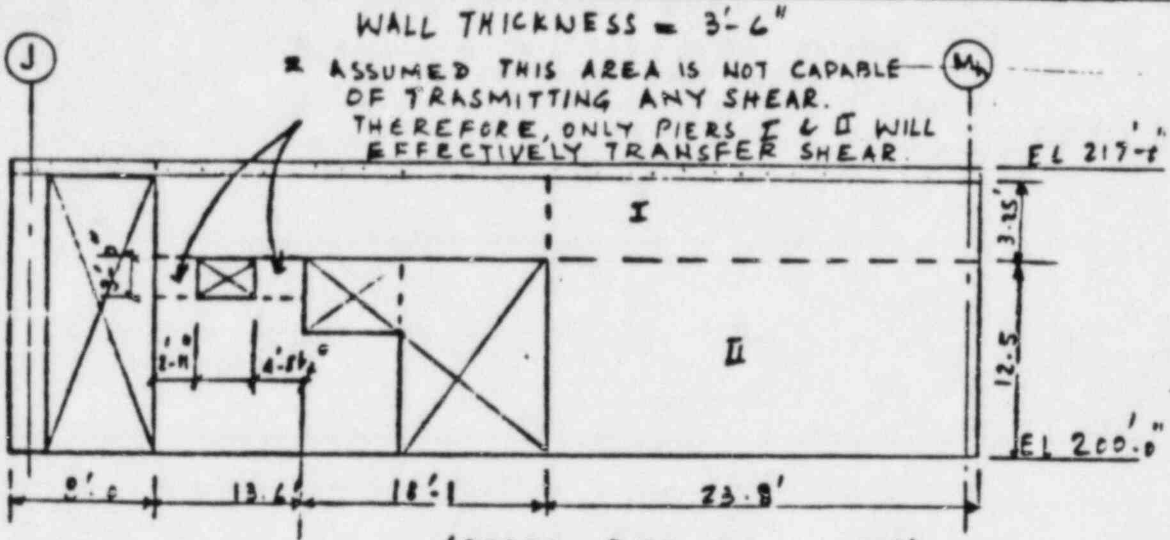
(BETWEEN EL. 200'-0"/201'-0" & 217'-0")

	WALL NO.	for, P=1000k δ (IN.)	R RIGIDITY	R_{REL}	TRANSLATIONAL SHEAR, $V_p = R_{REL} \times V$ (KIPS)	D (FT)	① $D \times R_{REL}$	② $D^2 \times R_{REL}$	SHEAR DUE TO ACCIDENTAL TORSION, $V_m = M_z \times \frac{①}{\Sigma ②}$ (KIPS)	COMBINED SHEAR, $V = V_p + V_m$ (KIPS)
N-S WALLS	14.1	0.03088	32.4	0.194	23,358	162	31.43	5,091	± 3,343	26,701
USED AS ILLUSTRATED EXAMPLES	19.4	0.1650	6.1	0.037	4,455	64.5	2.39	154	± 254	4,709
USED AS ILLUSTRATED EXAMPLES	** 23	0.01375	72.71	0.435	52,374	-	-	-	-	52,374
USED AS ILLUSTRATED EXAMPLES	26.6	0.3816	2.6	0.016	1,926	64.5	1.03	67	± 110	2,036
N-S WALLS	31.9	0.03088	32.4	0.194	23,358	162	31.43	5,091	± 3,343	26,701
N-S WALLS	18.5	0.1621	6.2	0.037	4,455	81	3.0	243	± 319	4,474
N-S WALLS	27.5	0.1621	6.2	0.037	4,455	81	3.0	243	± 319	4,474
N-S WALLS	15.5	0.2367	4.2	0.025	3,010	135	3.38	456	± 359	3,369
N-S WALLS	30.5	0.2367	4.2	0.025	3,010	135	3.38	456	± 359	3,369
	Σ		167.0	1.00	120,401			11,801		
E-W WALLS	D	-	-	0.445	-	87.7	39	3,420	± 4,148	4,148
E-W WALLS	J	-	-	0.333	-	46.3	15	695	± 1,595	1,595
E-W WALLS	M_h	-	-	0.222	-	106.8	24	2,560	± 2,553	2,553
								6,675		
							ΣN-S + EW	18,476		

* MOMENT DUE TO FLOOR ECCENTRICITY

** WALL SYSTEM 23 INCLUDES WALLS AT COLUMN LINE 22.4, 23.6, 21.5, 24.5 AND 23.0.

WALL ON LINE 19.4 (BETWEEN EL 200'-0" & 217'-0") LOOKING WEST



(REFER DWG. NO. C-409)

TOTAL SHEAR TO WALL @ EL. 217'-0", V = 4,709 K

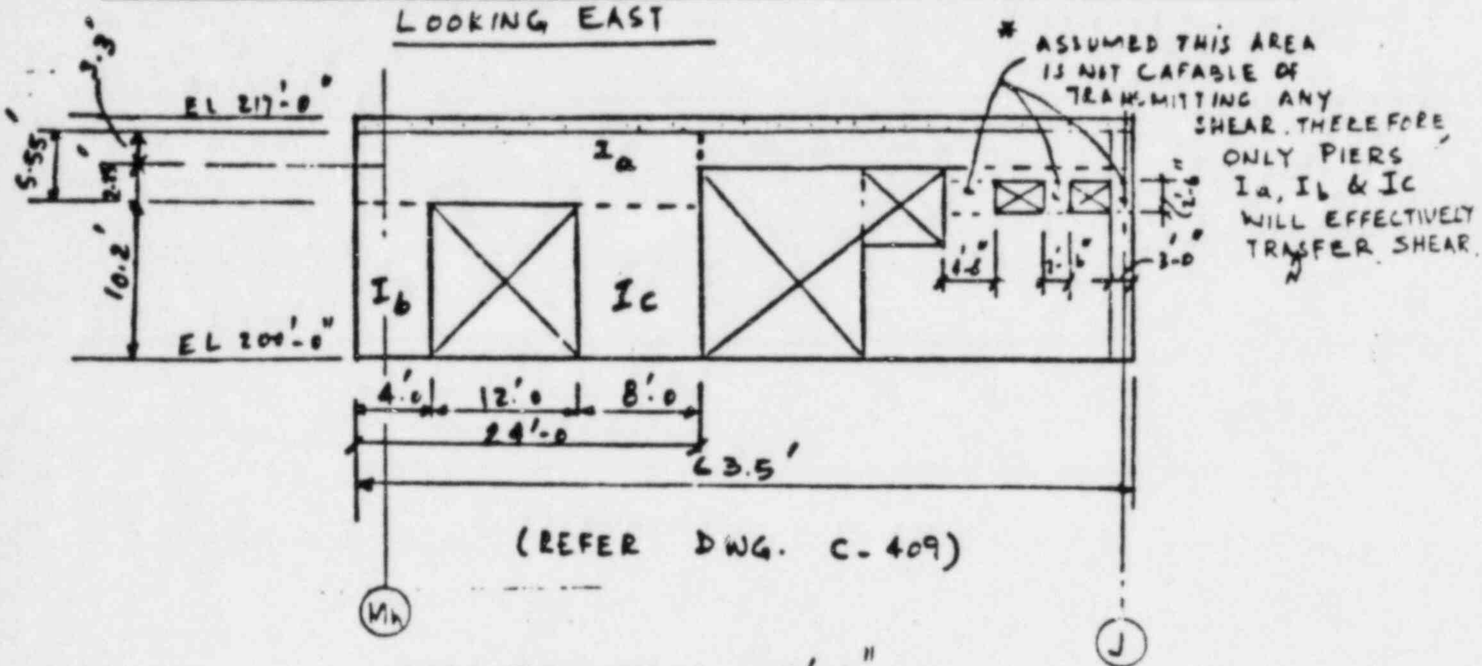
PIER NO.	FOR, P=1000K DEFLEC-TION, δ (IN)	RIGIDITY (R)	RELATIVE RIGIDITY (R_{REL})	HORIZONTAL SHEAR	HORIZONTAL SHEAR STRESS v_u (psi)	MAX. ALLOW. SHEAR STRESS ($10\sqrt{f'_c}$)	$v_c = 2\sqrt{f'_c}$	$(v_u - v_c)$ (psi)
I	0.0286	34.9	1.0	4,709	404	632 psi	127 psi	277
II	0.1366	7.32	1.0	4,709	462	632 psi	127 psi	335

CARRIED BY SHEAR REINFORCEMENT

* REINFORCED FOR CONFINING CONCRETE.

WALL ON LINE 26.6. (BETWEEN EL 200'-0" & 217'-0")

LOOKING EAST



WALL THICKNESS = 4'-0"

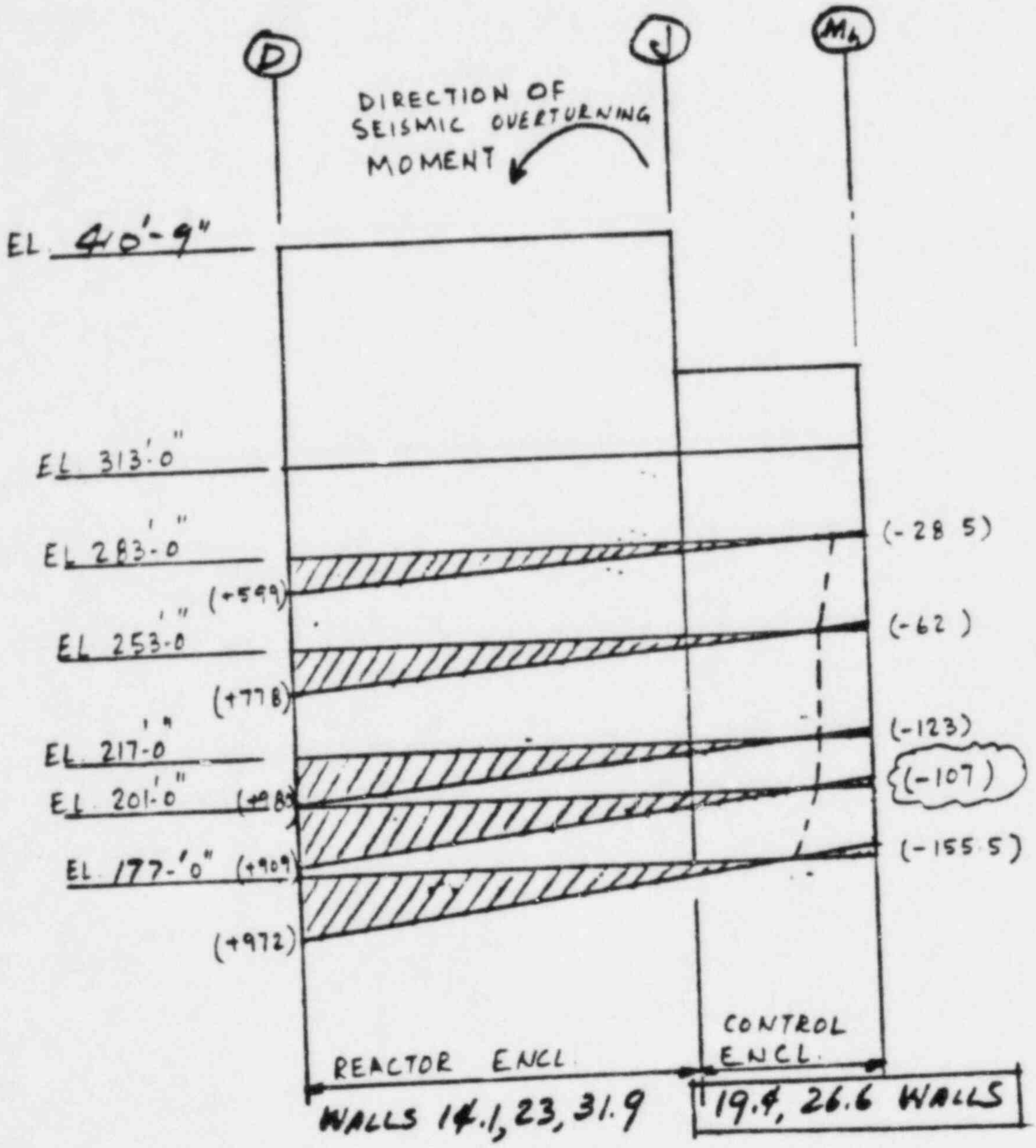
TOTAL SHEAR TO WALL @ EL 217'-0", $V = 2{,}036\,k$

PIER NO.	DEFLECTION (δ)	RIGIDITY (R)	RELATIVE RIGIDITY R_{REL}	HORIZONTAL SHEAR V_P	HORIZONTAL SHEAR STRESS v_u (psi)	MAX ALLOW. SHEAR STRESS ($10\sqrt{f'_c}$)	$v_c = 2\sqrt{f'_c}$ psi	$(v_u - v_c)$ psi
I_a	0.0516	19.4	1.0	2,036	173	632 psi	127	46
I_b	1.6845	0.59	0.195	396	202	632 psi	127	75
I_c	0.4095	2.44	0.805	1,640	419	632 psi	127	292

CARRIED BY SHEAR REINFORCEMENT

& REINFORCED FOR CONFINING CONCRETE.

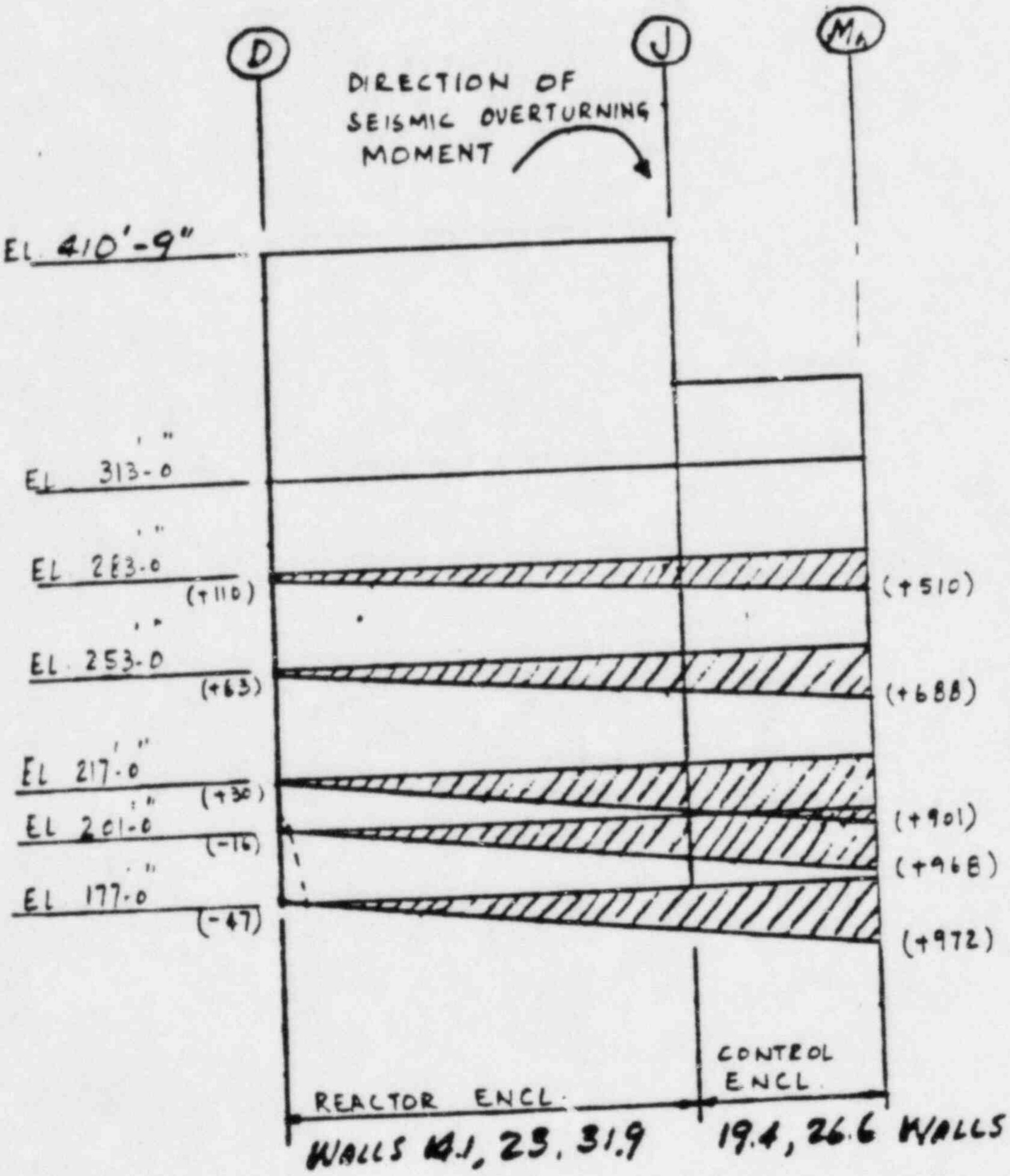
COMBINED TENSION / COMPRESSION STRESSES IN WALLS DUE TO DEAD LOAD, LIVE LOAD AND SSE (OVERALL OVERTURNING)



(ALL STRESSES ARE SHOWN IN PSI)

MAX. TENSILE STRESS @ EL 201'-0", = 107 psi

COMBINED TENSION/COMPRESSION STRESSES IN WALLS
DUE TO DEAD LOAD, LIVE LOAD AND SSE (OVERALL OVERTURNING)



(ALL STRESSES ARE SHOWN IN PSI)

MAX. COMPRESSIVE STRESS = 968 psi $< 0.7 \times 0.85 f'_c$
(@ EL. 201'-0")

FOR f'_c = 4000 psi, ALLOWABLE BEARING = 2,380 psi
f'_c = 5000 psi, " " = 2,980 psi

FOR WALL @ 19.4

MAX SHEAR STRESS, PIER II = 462 psi

VERT REINF FOR LOCAL BENDING = $\frac{M}{\phi f_y (d - a/2)}$

$= \frac{4709 \times 10\ 83}{60\ 5 \times 54 \times 23\ 8} = 0.66\ in^2/ft$

DUE TO TENSILE FORCE = $\frac{107 \times 42 \times 12}{0.9 \times 60000} = 1.0\ in^2/ft$

TOTAL REINF. REQD. = 0.66+1.0 = 1.66 in^2/ft <

PROVIDED #9 @ 6" EF 4.0 in^2/ft

O.K.

FOR WALL @ 26.6

MAX SHEAR STRESS, Ic = 419 psi

VERT REINF. FOR LOCAL BENDING = $\frac{1640 \times 10\ 2}{7.5 \times 54 \times 2 \times 8} = 2.58$

DUE TO TENSILE FORCE = $\frac{107 \times 48 \times 12}{0.9 \times 60000} = 1.14\ in^2/ft$

TOTAL REINF REQD. = 2.58+1.14

= 3.72 in^2/ft <

#9 @ 6" EF. 4 in^2/ft

OK

DSER #9 - Spent Fuel Pool (3.8.3)

In the original spent fuel pool (SFP) design, the applicant assumed low density storage racks would be used and the SFP floor was designed accordingly. Revision 10, of the Limerick FSAR mentions the use of high density free standing storage racks. The applicant should provide verification that the SFP floor retains sufficient structural capacity to withstand all loads, load cases, and load combinations relied upon in the original calculations. The staff considers this to be a confirmatory item.

Response:

The LGS FSAR will be changed in the June 1983 revision to read as shown in Attachment A.

energy. These impact loads have been verified by full-size tests on an actual top grid casting.

For condition 3, an unimpeded fuel assembly drop through an empty cavity, an equivalent static load was determined to shear out the bottom fuel support. The following presents the equivalent static loads for the three drop conditions.

Condition	Description	Load
1	36 inch drop, corner of rack	82,500 lb
2	36 inch drop, middle of rack	64,300 lb
3	Drop through empty cavity of rack	24,600 lb

Conditions 1 and 2 are the loads due to vertical impact. The subsequent roll over impact load was shown to be less than the above stated vertical impact values. Equivalent static loads for different dropped fuel bundle cases were applied at proper locations to the ANSYS finite element model of the rack and combined with the dead weight vertical load (rack full of fuel). Stresses for each member and plate were tabulated and were less than the factored allowables of Equation 4, Table 9.1-20.

9.1.2.3.2.4 Pool Interface Loads

The spent fuel rack load acting on the spent fuel pool is determined using the results from the ANSYS seismic model described in Section 9.1.2.3.2.2.a and shown in Figure 9.1-36. The force response at the rack and pool interface elements of the two-rack model are computed for each time step of the dynamic analysis. These force responses which include SSE loading are used to determine the maximum concentrated local forces and the equivalent uniform global rack load acting on the spent fuel pool slab. The maximum pool slab bearing and punching shear stresses are computed from the maximum spring forces at nodes 101 and 102 including the impact effect. The equivalent uniform rack load is conservatively computed using the maximum reaction force acting at either nodes 201, 202, 203, or 204.

The allowable stresses for bearing and punching shear of the spent fuel pool slab are in accordance with sections 10.14 and 11.10, respectively, of the building code requirements for reinforced concrete (ACI 318-71). the allowable uniform rack load on the pool slab is calculated to be 2815 psf, using a finite element model at the spent fuel pool. The controlling load combinations are normal operating and severe environmental as given in Table 3.8-9. The assessment results given below include the additional conservatism of substituting SSE values for the OBE loads.

Condition	Allowable	Fuel pool slab interface load (including SSE)
Bearing on Concrete-Floor (Local Area)	4760 psi	4748 psi
Punching Shear-Floor	253 psi	132 psi
Uniform Load-Floor Slab (Seismic)	2815 psi	2811 psf

9.1.2.3.2.5 Summary and Conclusions

A 5.50-inch minimum clearance is maintained between the rack and any pool wall or obstruction to avoid rack impact during a seismic event.

All member and plate stresses satisfy the stress combination limits and factored allowable stresses of Equations 1 through 6, Table 9.1-20, for the seismic and dropped fuel conditions. The stresses on the concrete floor for both seismic and dropped fuel conditions are acceptable in accordance with the allowable limits.

9.1.2.3.3 Installation of New High Density Racks

Racks are lifted individually from the refueling floor laydown area and lowered into position in the pools using the 125-ton reactor enclosure crane and a remotely actuated pneumatic lifting device provided by the rack vendor.

Each rack is aligned and leveled as it is placed in its proper pool. Each rack is a freestanding unit that rests on the pool floor using four bearing pads attached to corner leveling screws.

TGS/dmc 15/4