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VAN BRUNT,E.E. Arizona Public Service Co.
RECIP.NAME RECIPIENT AFFILIATION
KERRIGAN,J. NRC - No Detailed Affiliation Given

SUBJECT: Forwards responses to NRC structural audit questions asked at 810811-13 meeting.Meeting agenda,list of open items & completed audit forms encl. See Reports for Encl's III [illegible]

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ARIZONA NUCLEAR POWER PROJECT
Post Office Box 216[illegible]hoenix, Arizona 85036

August 19, 1981
ANPP-18697 - JMA/WFQ

Ms. Janis Kerrigan
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Subject: Palo Verde Nuclear Generating Station
(PVNGS) Units 1, 2 and 3
Docket Nos. STN-50-528/529/530
File: 81-056-026; G.1.10

Dear Janis:

A meeting with the NRC/NRR's Structural Engineering Branch (SEB) was conducted from August 11 to August 13, 1981. Prior to the meeting, we were requested to respond to 10 NRC questions and to complete typical SEB audit forms for our Category I structures.

In this regard, the following information is enclosed:

Enclosure 1 . . . . Meeting Agenda
Enclosure 2 . . . . List of Meeting Open Items
Enclosure 3 . . . . Completed Audit Forms
Enclosure 4 . . . . Responses to NRC Questions

Very truly yours,

E. E. Van Brunt

E. E. Van Brunt, Jr.
APS Vice President
Nuclear Projects
ANPP Project Director

EEVBJr/WFQ/pc

Enclosures

cc: S. Chan (NRC) w/encl.
P. Hourihan w/encl.
A. C. Gehr w/encl.

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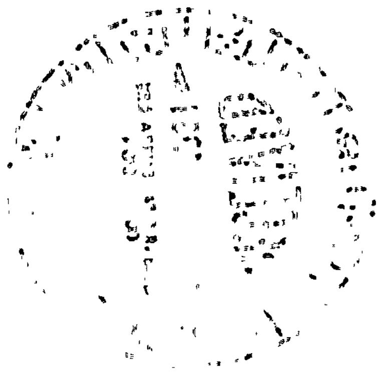

STATE OF ARIZONA)
) ss.
COUNTY OF MARICOPA)

I, Edwin E. Van Brunt, Jr., represent that I am Vice President Nuclear Projects of Arizona Public Service Company, that the foregoing document has been signed by me on behalf of Arizona Public Service Company with full authority so to do, that I have read such document and know its contents, and that to the best of my knowledge and belief, the statements made therein are true.

Edwin E. Van Brunt

Edwin E. Van Brunt, Jr.

Sworn to before me this 21st day of AUGUST, 1981.

Connie Lou Armstrong

Notary Public

My Commission expires:

June 24, 1983

Enclosure 1

AGENDA

NRC SER AUDIT MEETING
STRUCTURAL ENGINEERING BRANCH

Week of August 10, 1981

Monday: Tour of Jobsite in Phoenix, Arizona

Tuesday:

I. INTRODUCTION

II. REVIEW OF STRUCTURAL AUDIT WORKSHEETS

A. Brief Overview of Entire Plant.

B. Containment Building

Wednesday

C. Main Steam Support Structure

D. Auxiliary Building

E. Fuel Building

F. Control Building

Thursday

G. Diesel Generator Building

H. Category I Tanks

I. Spray Ponds

J. Category I Structures

III. REVIEW OF RESPONSES TO NRC QUESTIONS

IV. SUMMARY

[illegible]

Enclosure 2

DATE OF MEETING: August 11, 12, 13, 1981

LOCATION: Downey, California

ATTENDEES:

APS	Bechtel		NRC/NRR/SEB
W. Quinn	D. Gibson	G. Kopchinski	D. Jeng
W. Hurst	* R. Kosiba	* A. Hadjian	R. Lipinski
	O. Gurbuz	* S. Jain	S. Chan
	R. Peters	* G. Guevara	
	P. Wong	* W. Brandes	
	K. Schechter	* R. Senczyszyn	
	D. Keith	* R. Platoni	
	B. Bitner	* D. Niehoff	
	* B. Linderman	* W. Au	

* Part time.

SUBJECT: SEB DRAFT SER

PURPOSE: Working meeting to resolve questions and provide requested information.

I. OPEN ITEMS FROM STRUCTURAL AUDIT WORKSHEETS.

	Date Information To Be Furnished
1. Provide justification for the use of 2 OBE seismic events in lieu of 5 OBE events with a minimum of 10 cycles each. Show that the intent of SRP 3.7.3.2 is met.	8/24/81
2. Provide a discussion of transferring both damping (material damping) and shear modulus values shown on Figure 3A-2 into actual determination of compliance functions used for seismic analysis of Category I structures.	9/21/81

	Date Information To Be Furnished
3. Provide live load sketches for Category I structures, noting that the absence of live load is considered to obtain the greatest loading.	8/24/81
4. NRC requests Bechtel provide an ultimate capacity assessment of the Containment Structure against internal pressure, accounting for the specific parameters in Attachment 1. (A sophisticated computer analysis is not required.)	Not Applicable
5. Present a calculation showing that the liner system design satisfies all the loading requirements resulting from a negative pressure of 4 psig.	8/24/81
6. Bechtel will provide technical justification for the vertical analysis of the polar crane. This justification will account for the upward motion of the suspended mass, including the flexibility of the cable.	9/21/81
7. Provide technical justification, in the form of calculations, demonstrating that the accidental torsion applicable to the Containment Building (i.e., 5% of the size dimension multiplied by the maximum lateral force at the base) is properly accounted for in the containment shell and interior structure design.	9/21/81

	Date Information To Be Furnished
8. Bechtel will provide technical basis for deciding boundary and extent of the soil included in the 'FINEL' analysis.	8/24/81
9. Bechtel will provide: the angle of rotation for the Containment Building basemat, the actual calculation determining the factor of safety (FS) against overturning, and an additional calculation determining the FS against overturning using conventional methods.	8/24/81
10. Bechtel will elaborate upon the answer given on page 26, Item 5-B Containment Building; specifically, provide discussion on the procedure used by the applicant to meet the requirements of SRP 3.7.II.8. (Regarding non-Category I over Category I criteria)	8/24/81
11. Provide typical calculation (a paragraph) determining the fictitious temperature drop accounting for the effect of prestressing tendon (both orthogonal and draped tendons).	8/24/81
12. Provide a copy of the equipment hatch calculation (Sub-Section NE). [This will serve as the typical calculation procedure for the personnel hatch as well.]	8/24/81

		Date Information To Be Furnished
13.	Provide a calculation of the junction between the containment wall and mat according to the computer analysis results. Also, explain the method in which seismic forces are combined with nonseismic forces.	9/21/81
14.	Provide a calculation for radial reinforcement to account for the delamination effect in the dome.	8/24/81
15.	Provide calculations for the critical section of the primary shield around the hot and cold leg penetrations.	9/21/81
16.	Bechtel will provide SEB with the results of the calculations checking the crane runway girder stability against D.L. + L.L. + SSE. Also provide calculations for the polar crane bracket.	10/6/81
17.	Provide the calculations for the steam generator upper support lever arm and snubbers (Bechtel scope of work).	8/24/81
18.	Bechtel will provide a typical calculation for the development of story stiffness parameters for MSSS.	8/24/81
19.	Provide justification that the corridor building has been designed to meet SRP section 3.7.II.8:	12/11/81

		Date Information To Be Furnished
20.	Provide technical justification for one critical wall of the Auxiliary Building similar to that requested in Action Item #8.	9/21/81
21.	Provide the mechanical properties of Rodofoam II and EVERLASTIC Micro II.	8/24/81
22.	Provide a conventional stability analysis against overturning of the Auxiliary Building.	8/24/81
23.	Provide the design calculations for the concrete column at elevation 88'-0" in the Auxiliary Building. The column has an axial force of 1981 k and a moment of 203 ft-k.	8/24/81
24.	Provide the calculation developing the hydrodynamic aspects of the Fuel Building seismic analysis model. (Lumped mass model)	8/24/81
25.	Provide an accidental torsional effect analysis (i.e., 5% of the size dimension multiplied by the maximum lateral force at the base) for one critical wall of the Fuel Building.	9/21/81
26.	Provide the calculations for the west wall of the spent fuel pool to withstand the impact from a fuel cask drop.	8/24/81

		Date Information To Be Furnished
27.	Provide an accidental torsional effect analysis (i.e., 5% of the size dimension multiplied by the maximum lateral force at the base) for one critical wall of the Control Building.	9/21/81
28.	Provide an approximate sum of the participation factors of 3 modes considered in the Diesel Generator Building seismic analysis as a percent of total response	8/24/81
29.	Provide an accidental torsional effect analysis (i.e., 5% of the size dimension multiplied by the maximum lateral force at the base) for one critical wall of the Diesel Generator Building.	9/21/81
30.	Provide calculations determining the soil pressure on the Diesel Generator Building foundation due to three directional seismic load.	9/21/81
31.	Provide an impact assessment on the seismic response of the refueling water tank accounting for partial cracking of the concrete shell.	9/21/81
32.	Provide calculations for considering tornado missile overall response for the roof and one wall of the Auxiliary Building.	9/21/81

		Date Information To Be Furnished
33.	Provide justification for not considering the soil structure interaction effect in the seismic analysis of the spray pond pump house.	8/24/81
34.	Provide a calculation determining the loads used in the design of both the spray pond walls and pump house walls.	8/24/81
35.	Provide an accidental torsional effect analysis (i.e., 5% of the size dimension multiplied by the maximum lateral force at the base) for one critical wall of the spray pond pump house.	9/21/81
36.	Provide a calculation showing that the natural frequency of the floor/structure support is at least twice the frequency of the pump/motor unit.	8/24/81
37.	Provide portions of calculation for Category I tanks, 13-CC-CT-015 (pages 1-53; Appendix pages F1-F18).	8/24/81
38.	Provide justification on the procedure used in calculating the partial embedment versus total embedment effect (per BC TOP 4A - Rev. 3) on the compliance functions for the seismic analysis of the tanks.	9/21/81

Date Information To Be Furnished

39. Provide a discussion on how major cable tray test results were used in arriving at the 20% modal damping. The discussion should assure consistency of observed data and calculations used. 12/11/81

40. Why was cable tray test input loading applied at a 45° angle instead of simultaneous horizontal and vertical load input? What are the implications of this testing method upon the validity of the recommended 20% damping (e.g., with respect to statistical independency requirements of different directional inputs)? 12/11/81

41. Will sprayed-on fireproofing affect cable friction and thus the damping ratios? 12/11/81

42. The cable tray test conditions do not reflect the actual physical site situation. Provide the rationale for extending the test results to the actual design which is different from the test configuration. 12/11/81

43. Specify different conditions under which different modal damping ratios ranging from 7-20% are used. (cable tray) 12/11/81

		Date Information To Be Furnished
44.	It appears that the scope of the cable tray test and the number of tests may not support direct extension to APS cable tray design. Justify that the scope of test conducted is adequate for direct design application.	12/11/81
45.	It appears that response to NRC Question 220.12 contains several deviations from draft Appendix A to Standard Review Plan Section 3.5.3. Indicate compliance with the above position or describe in quantitative terms the impact of any deviations from the above position.	9/21/81
46.	Provide calculations for the design of electric duct banks, Category I manholes, and Category I buried piping.	9/21/81
47.	CONDAM computer program will be added to the FSAR appendix 3B.	9/21/81

4. The space within the opening is sufficiently occupied by piping and pipe supports to preclude missile penetration.

The roof of the Main Steam Support Structure is elevated from the top of the walls to allow the escape of steam in the event of a major pipe break. The roof is cantilevered beyond the wall to provide the necessary missile protection.

Question 3A.14 (NRC Question 450.2) (3.5.1.4)

Describe the protection of the control room air intakes and diesel generator exhaust pipes from tornado-generated missiles.

RESPONSE: 4

- The control room air intakes are enclosed within a box structure located within the Control Building (See figure 3A-7). The wall sections exposed to tornado-generated missiles are designed to withstand such impact without adverse effect upon the system.
- The diesel generator exhaust pipes are enclosed within a 1'9" thick vertical, concrete chimney which is designed to withstand tornado-generated missile impact. A thick, steel pipe sleeve, also capable of withstanding tornado-generated missile impact, provides protection for the exhaust piping at the vent opening at the top of the chimney.

Question 3A.15 (NRC Question 220.11) (3.5.3)

The allowable displacement of reinforced concrete flexural members stated as Formula (3-2) in Appendix 3C to FSAR is inconsistent with the ACI 349-76 Code. Please correct this error. 6

RESPONSE: The response is contained in section 3C.3.1.2.

Question 3A.16 (NRC Question 220.12) (3.5.3)

The ductility ratios listed in Table 3C.3-4 and used for design of missile barriers are not acceptable. It is the staff position that the permissible ductility ratio under impactive and impulsive loads be not greater than 10 unless it is justified by submitting applicable experimental evidence in the plant SAR. A copy of the draft Appendix A to Standard Review Plan Section (SRP) 3.5.3 is attached herewith for your reference (Attachment 1). Please also note that the ductility ratio for pressurization, a condition that BC-TOP-9A does not cover, remains 1. Express your intention to comply with this staff position.

RESPONSE: PVNGS complies with the staff position. ~~A maximum ductility ratio of one has been used for pressurization.~~ Further information is contained in amended table 3C.3-4.

6 Question 3A.17 (NRC Question 220.13) (3.7.4)

There are some deviations in seismic instrumentation from the requirements of Regulatory Guide (R.G.) 1.12 as stated in Sections 1.8 and 3.7.4 of the FSAR. Give reasons for such deviations and state whether the suggested alternatives would exceed the requirements or the intended functions in R.G. 1.12. Also, explain how and when the control room operator be notified of the occurrence of an OBE which is defined by the design response spectra of Figures 3.7-3 and 3.7-4 of the FSAR.

RESPONSE: Deviations from Regulatory Guide 1.12 are defined and explained in section 1.8. The seismic instrumentation provided meets or exceeds the intended functions of Regulatory Guide 1.12. The control room operator is notified of an safe shutdown earthquake (SSE) or an operating base earthquake (OBE) by audible and visual annunciation as defined by amended section 3.7.4.3.

220.12

Table 3C.3-4

DUCTILITY RATIOS

Member Type and Load Condition	Maximum Allowable Value of μ
Reinforced Concrete	
Flexure:	
Beams and one-way slabs	$\frac{0.10}{p-p'} \leq 10$
6 Slabs with two-way reinforcing	$\frac{0.10}{p-p'} \leq 10$
Axial compression:	
Walls and columns[30]	1.3
Shear, concrete beams and slabs in region controlled by shear:	
Shear carried by concrete only	1.3
Shear carried by concrete and stirrups	1.6
Shear carried completely by stirrups	2.0
Shear carried by bent-up bars	3.0
Structural Steel	
6 Columns and beams with uniform moment[31]	$\mu \leq \frac{14 \times 10^4}{F_y\left(\frac{KL}{r}\right)^2} + 1/2 \leq 10$
Beams with moment gradient	10
Shear	10
Axial tension and steel plates membrane tension[32]	$0.5 \frac{e_u}{e_y}$

Notes:

1. Based on information contained in references 9, 12, and 14 through 25.
2. p and p' are the positive and negative reinforcing steel ratios.
3. See figure 3C.3-2 for allowable ductility ratios where there is a beam-column action.
4. KL/r is the member slenderness ratio. F_y is the yield stress (ksi).
5. e_u and e_y are the ultimate and yield strains. e_u shall be taken as the ASTM specified minimum.

3.5.1.2.2 Pressurized Component Failure Missiles

A. Reactor Coolant System Pressure Boundary (RCPB)

The selection of potential missiles is based on the application of single-failure criteria to the normal retention features of plant equipment for which there is a source of energy capable of creating a missile in the event of the postulated removal of the normal retention features. Where redundancy is provided by the normal retention features, such that sufficient retention capability remains to prevent creation of a missile in the event of a postulated failure of a single retention feature, no potential missile is postulated. Table 3.5-4 presents the potential missiles postulated to originate from RCPB equipment, ~~and~~ summarizes their characteristics, AND LISTS PROVIDED ~~The provided~~ missile protection (INCLUDING MISSILES FROM EQUIPMENT WITHIN THE C-E SCOPE OF SUPPLY). ~~is presented in table 3.5-4.~~ 6

B. Non-RCPB Systems

A tabulation of missiles generated from failures of pressurized components, their sources and characteristics, and provided missile protection, is given in table 3.5-4. The bases for selection are identical to those described in section 3.5.1.1.2.

3.5.1.3 <u>Turbine Missiles</u>

3.5.1.3.1 Turbine Placement and Orientation

The placement and orientation of the turbine generators is shown in figure 3.5-1.

3.5.1.3.2 Missile Identification and Characteristics

Analysis has indicated that high-pressure turbine missiles and generator missiles would be retained by their respective

Question 3A.18 (NRC Question 220.14) (3.7.4)

Is instrumentation needed for measuring torsional input provided? If yes, describe type, numbers, and layout of the torsional instrumentation.

RESPONSE: Instrumentation for measuring torsional input has not been provided.

Question 3A.19 (NRC Question 220.15) (3.8.1)

Provide the manufacturer's name or the name of the post-tensioned prestressing system used in the Palo Verde containment structures. Has this system been accepted by the NRC staff as indicated in R. G. 1.103?

RESPONSE: The PVNGS containment structures use a BBRV post-tensioned prestressing system. This ~~method~~ system is accepted by Regulatory Guide 1.103.

Question 3A.20 (NRC Question 220.16) (3.8.1) 6

Has buckling been considered in design of containment building? Would it cause any problem in design?

RESPONSE: Buckling has not been considered as part of the design of the containment exterior concrete wall. The 4-foot-thick wall and geometry preclude buckling as a design factor in the containment design. Buckling has been considered in the liner plate design since the liner plate acts as the inside form for the containment shell. (Refer to BC-TOP1A, Rev 1, 1972.)

Question 3A.22 (NRC Question 220.19) (3.8.4)

Are there any (a) intake structures, submerged pipes or tunnels, (b) structure-pile-soil medium systems, (c) spent or new fuel pool structures, used in Palo Verde plants? If yes, describe key dimensions, structure modeling, static and seismic analysis criteria, assumptions and computer codes used. A copy of "Minimum Requirements for Design of Spent Fuel Pool Racks" is enclosed for your reference (Attachment 3).

RESPONSE:

(a) Intake Structures: The Circulatory Intake Structure is a non-Category I structure.

Submerged Pipe: There are submerged pipes in the essential spray ponds and the spent fuel pool. In the spray ponds the pipes vary in diameter with the maximum dimension being 24 in. The pipes in the fuel pool are 16 in. in diameter. Both types of pipe are designed in accordance with sections 3.6 and 3.9.

Tunnels: There are two tunnels: the condensate tunnel and the essential pipe density tunnel. These tunnels are approximately 135 feet long. The condensate tunnel is 15 feet high and 15 feet wide with the top approximately at grade level. The essential pipe density tunnel is 15 feet high and 30 feet wide with the top approximately at grade level. Both are designed as Category I.

(b) There are no structure-pile-soil medium systems.

(c) The spent fuel pool and new fuel pit structures are discussed in section 3.8.4.1.2. The spent fuel racks are free-standing structures designed and supplied by Combustion Engineering, Inc. The design complies with NRC General Design Criteria 62 and Regulatory Guide 1.13.

Question 3A.18 (NRC Question 220.14) (3.7.4)

Is instrumentation needed for measuring torsional input provided? If yes, describe type, numbers, and layout of the torsional instrumentation.

RESPONSE: Instrumentation for measuring torsional input has not been provided.

Question 3A.19 (NRC Question 220.15) (3.8.1)

Provide the manufacturer's name or the name of the post-tensioned prestressing system used in the Palo Verde containment structures. Has this system been accepted by the NRC staff as indicated in R. G. 1.103?

RESPONSE: The PVNGS containment structures use a BBRV post-tensioned prestressing system. This ~~method~~ system is accepted by Regulatory Guide 1.103.

Question 3A.20 (NRC Question 220.16) (3.8.1) 6

Has buckling been considered in design of containment building? Would it cause any problem in design?

RESPONSE: Buckling has not been considered as part of the design of the containment exterior concrete wall. The 4-foot-thick wall and geometry preclude buckling as a design factor in the containment design. Buckling has been considered in the liner plate design since the liner plate acts as the inside form for the containment shell. (Refer to BC-TOP1A, Rev 1, 1972.)

Question 3A.21 (NRC Question 220.17) (3.8.1)

Provide an ultimate capacity analysis of the containment building in regard to hydrogen burning. The guideline and the staff position on this subject is enclosed (Attachment 2).

RESPONSE: The ultimate capacity of the containment is at least 90 psig.

Question 3A.22 (NRC Question 220.19) (3.8.4)

Are there any (a) intake structures, submerged pipes or tunnels, (b) structure-pile-soil medium systems, (c) spent or new fuel pool structures, used in Palo Verde plants? If yes, describe key dimensions, structure modeling, static and seismic analysis criteria, assumptions and computer codes used. A copy of "Minimum Requirements for Design of Spent Fuel Pool Racks" is enclosed for your reference (Attachment 3).

RESPONSE:

(a) Intake Structures: The Circulatory Intake Structure is a non-Category I structure.

Submerged Pipe: There are submerged pipes in the essential spray ponds and the spent fuel pool. In the spray ponds the pipes vary in diameter with the maximum dimension being 24 in. The pipes in the fuel pool are 16 in. in diameter. Both types of pipe are designed in accordance with sections 3.6 and 3.9.

Tunnels: There are two tunnels: the condensate tunnel and the essential pipe density tunnel. These tunnels are approximately 135 feet long. The condensate tunnel is 15 feet high and 15 feet wide with the top approximately at grade level. The essential pipe density tunnel is 15 feet high and 30 feet wide with the top approximately at grade level. Both are designed as Category I.

(b) There are no structure-pile-soil medium systems.

(c) The spent fuel pool and new fuel pit structures are discussed in section 3.8.4.1.2. The spent fuel racks are free-standing structures designed and supplied by Combustion Engineering, Inc. The design complies with NRC General Design Criteria 62 and Regulatory Guide 1.13.

Question 3A.23 (NRC Question 220.20) (3.8.4)

Enclosed is a copy of staff interim criteria on safety-related masonry wall evaluation (Attachment 4). Identify any difference in requirements of materials, testing, analysis, design, construction and inspection related to safety-related concrete masonry walls between the Palo Verde design and the staff position. Modify your analysis and design, if necessary, to agree with this interim position. State and discuss your hardship, if any, of compliance.

RESPONSE: The design of masonry walls has been addressed in the response to Question 3A.12. The design of these non- 6
structural concrete masonry walls meets the requirements of NRC SEB Interim Criteria for Safety Related Masonry Wall Evaluation.

Question 3A.24 (NRC Question 220.22) (3.8.4)

Prepare for the structural design audit scheduled for the week of August 10, 1981. A copy of requirements and guidelines for implementation of structural design audits is enclosed (Attachment 6).

RESPONSE: Preparations have been completed as required for the structural design audit scheduled for August 10, 1981.

Close up space

Question 3A.25 (NRC Question 220.23) (3.5.3)

BC-TOP-9A does not cover design of barriers against turbine missiles. Barriers against turbine missiles should be designed in accordance with the requirements and criteria of R. G. 1.115. Address your commitment to comply with R. G. 1.115 or identify and justify all deviations and discrepancies that exist in your design. It is also the staff's position that for turbine
6 missile barriers, penetration and scabbing predictions should be based on empirical equations such as the modified NDRC formula or the results of a valid test program. Other formulas which are supported by test data are acceptable and will be reviewed on a case-by-case basis.

RESPONSE: The response is contained in amended section 3.5.1.3.

3.7.4.2.3 Seismic Switches

One triaxial seismic switch with dual setpoint is installed adjacent to the SMA in the containment base slab. It is a backup device which actuates a visual and audible annunciator in the control room if either the SSE or OBE has been exceeded at the seismic switch location. The setpoints for the switch are:

OBE Horizontal = 0.18g	SSE Horizontal = 0.31g
OBE Vertical = 0.17g	SSE Vertical = 0.34g

3.7.4.2.4 Response Spectrum Analyzer

The response spectrum analyzer consists of a microprocessor-based computational unit and a printer unit. The computational unit is operated in conjunction with a playback system. The analyzer computes the response spectrum from the recorded data. The printer unit prints response acceleration versus frequency in a hard copy form.

3.7.4.2.5 System Control Panel

A panel located in the control room houses the recording, playback, and calibration units which are used in conjunction with the SMA sensors to produce a time-history record of the earthquake. It also contains signal conditioning and display equipment associated with the response spectrum analyzer, audible and visual annunciators associated with the seismic switch, audible and visual annunciators wired to display initiation of the SMA recorder, and the power supply components for the equipment contained within the panel.

220.13 3.7.4.3 Control Room Operator Notification

Activation of the seismic triggers causes an audible and visual annunciation in the control room to alert the plant

operator that an earthquake has occurred. This annunciation is initially set to occur at 0.01g horizontal and/or vertical acceleration on the containment tendon gallery or at 0.02g horizontal and/or vertical acceleration in the containment operating floor. These levels cause initiation of the SMA recording system at horizontal or vertical acceleration levels slightly higher than the expected background level, including induced vibrations from sources such as traffic, elevators, people, and machinery. These initial setpoints are based on experience in existing plants and may be changed once significant plant operating data have been obtained which indicate that a different setpoint would provide better SMA system operation. Audible and visual annunciators are provided in the control room to indicate if the SSE or OBE
6 floor accelerations have been exceeded for the seismic switch location, as defined by the design response spectra of figures 3.7-3 and 3.7-4.

The peak acceleration level experienced on the containment tendon gallery is available immediately following the earthquake. This is obtained by playing back the recorded SMA data from this location and reading the peak value for this data from the printer unit.

Significant response spectra from the containment tendon gallery are available in the control room immediately following an earthquake on readout equipment suitable for comparing the measured response spectra with the OBE and SSE response spectra.

3.7.4.4 Comparison of Measured and Predicted Responses

Initial determination of the earthquake level is performed immediately after the earthquake by comparing the measured response spectra from the containment tendon gallery with the OBE and SSE response spectra for the corresponding location.

Enclosure 3

PALO VERDE NUCLEAR STATION UNITS 1, 2, 3
DESIGN ADEQUACY AUDIT

STRUCTURAL AUDIT OF CONTAINMENT BUILDING
Part I - General Analysis

I. BASIC DESIGN CRITERIA

(1) 'g' value - free field

	Seismic level based on construction permit license	Seismic level used in design of structures and equipment
SSE	0.20g	0.25g
OBE	0.10g	0.13g

Reference: FSAR, Section 3.7

(2) Spectra (attach figs. for all damping values, ductilities)

A. zero period acceleration

SSE 0.25g
OBE 0.13g

Reference: FSAR, Figures 3.7-1-- 3.7-4 (Pages 89- 92) and Section 3.7.1.1
This is consistent with Reg. Guide 1.60

Frequency (or period) interval

Refer to BC-TOP-4A, Section 2.5.1 (c)

(3) Damping

Refer to FSAR, Section 3.7.1.3
This consistent with Reg. Guide 1.61
Refer to FSAR Figures 3.7-5 and 3.7-6 (Pages 92A and 92B)

(4) Artificial time history and corresponding spectra (attach figures)

A. original time history and its composition, i.e., rising time, strong motion and tail end.

Refer to FSAR, Section 3.7.1.2 and BC-TOP-4A, Section 2.5

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B. base line correction, check the integrated velocity and displacement time histories

The velocity and displacement time histories were checked and found to be satisfactory. In addition, only the acceleration time history was used as input in analysis.

C. time interval - compatible with the highest frequency considered in the spectral calculation

0.005 seconds

(5) Motion duration

24 seconds

Refer to FSAR, Section 3.7.1.2

(6) components of motion including their relative motion amplitudes

Analysis was performed for the three principle directions with equal amplitudes.

(7) Dead and live loads for various operating floors and base slab

~~Refer to Project Design Criteria Manual, Part II, Sections 3.5 and Part III, Section 1.4.1.~~2. Dead load - includes all structures, major equipment load and 50 psf equivalent for small equipment. Live load - See action item 3.

(8) Internal pressure

Containment Shell: 60 psig design pressure

Reactor Shield Wall and Reactor Cavity: 110 psid design pressure

Steam Generator Compartment: 30 psid design pressure

Z

(9) Ground water level

The groundwater design level is at plant El 70′-0". The actual water level is at approximately plant EL. 60′-0".

Reference: FSAR, Sections 2.4.13.2.4 and 2.4.13.5.

(10) Backfill earth pressure, wind, overpressure due to postulated external explosions (as applicable)

1) The backfill is 23′ (The top of the containment basemat is located at elevation 77′-3" and plant grade is elevation 100′-0".). This occurs over about a 120° segment in plan.

2) The wind load is not a governing load for the containment structure.

3) The containment structure is designed for an external pressure of 4 psig.

Reference: FSAR, Section 6.2.1.1.3.6

(11) Other considerations

None

II. ANALYSIS METHOD

(1) Seismic Analysis

A. Mathematical model-general description with sketch.

Three models were used for the seismic analysis:

(1) Two planar lumped parameter models were used for a time history analysis to generate in-structure response spectra and determine seismic shear and bending moments for the containment shell design. Refer to FSAR, Figure 3.7-10 and Attachment A for sketches of the models
(Page 96) (Page 64)

(2) A 3 dimensional lumped parameter model that was coupled with CE's 3-D NSSS model to be used in their 3-dimensional analysis.

(3) A finite element model that was used in a modal response spectrum analysis to analyze and design the internal structures. See Attachment B for a sketch of the model.
(Pages 65-70)

(a) parameters used

(i) concrete modulus

E_c = 6.00 x 10^6 psi for f'c = 6000 ~~K~~PSI

E_c = 5.50 x 10^6 psi for f'c = 5000 ~~K~~PSI

Reference: Project Design Criteria

(ii) rebar modulus and yield strength

E = 29 x 10^3 KSI Fy = 60 KSI

(iii) Poisson's ratio

ν = 0.24 for concrete

(iv) damping

Refer to FSAR, Section 3.7.1.3 and Table 3.7-1. (Page 88A)
This is consistent with Reg. Guide 1.61

(v) structural steel modulus and yield strength

E_s = 29 x 10^3 KSI

F_Y = 36 KSI

[illegible]

(vi) properties of foundation materials.

- Shear modulus

  Refer to FSAR, Figures 3.7-7, 3.7-8, 3.7-9 (Pages 93-95)

- Subgrade reactions

  ~~Refer to Project Design Criteria, Part II, Section 3.4.5.4 for coefficient of subgrade reaction.~~
  Coefficient of subgrade reation = 40-60 KIPS/FT²/FT

- Bearing capabilities

  Refer to FSAR, Tables 2.5-15 and 2.5-16 (Pages 87-88)

(vii) other parameters

Refer to FSAR, Figure 3.7-7 (Page 93), for mass density and Poisson's ratio for soil.

b. stiffness calculations

(i) concrete shell method of incorporating different layers of materials (concrete, rebars, and slip surface). State the method used to account for containment shell cracking due to preoperational pressure tests.

Refer to FSAR, Section 3.7.2.3.2.

(ii) internals

Refer to FSAR, Section 3.7.2.3.2.

B. Method of Analysis

(a) Method of analysis used (Time history, response spectrum methods, etc.) and consideration of torsional and translational response

Two planar lumped parameter models were used to generate in-structure response spectra from a time-history analysis.

(i) general description

Refer to FSAR, Section 3.7.2.3.2.

The soil structure interaction analysis method was used and compared against current NRC review position and this meets the intent of NRC soil structure interaction position

(ii) findings and comments

(b) selection of number of masses and degrees of freedom

(i) general description

Data for the planar models are:

N-S Model: 42 DDOF 42 masses (horizontal)

E-W Model: 42 DDOF 42 masses (horizontal)

Vertical: 21 DDOF, 21 masses.

(2) findings and comments

c. number of modes considered

	SSE	OBE
Horizontal (E-W)	10 modes	10 modes
Horizontal (N-S)	12 modes	12 modes
Vertical	6 modes	6 modes

The frequencies of the highest modes considered were all greater than 33 cps.

(1) general description

Refer to FSAR, Appendix 3A, Question 3A.6.

(2) findings and comments

d. combining modal responses

This is not applicable for the time history analysis.

(1) actual procedures used

Refer to FSAR, Section 3.7.2.7

(2) general findings

e. consideration of three components of motion

(i) actual procedures used

Refer to FSAR, Section 3.7.2.6
It is consistent with Reg. Guide 1.92

(ii) general findings

f. consideration of soil-structure interaction and interaction among adjacent buldings

Refer to FSAR, Section 3.7.2.4 for consideration of soil-structure interaction. The containment is separated from the surrounding buldings by means of a 6 inch gap above grade. There is Rodofoam below grade which is a compressible material.

structure - structure interaction is negligible.

(i) general description

See FSAR Section 3.7.2.4 for soil impedance calculation.

<u>Soil Springs:</u>

	$K_X = 1.334 \times 10^6$	K/FT	$K_{XX} = 6.800 \times 10^9$	K-FT/RAD
<u>SSE</u>	$K_Y = 1.911 \times 10^6$	K/FT	$K_{YY} = 9.160 \times 10^9$	K-FT/RAD
	$K_Z = 1.334 \times 10^6$	K/FT	$K_{ZZ} = 6.800 \times 10^9$	K-FT/RAD
	$K_X = 1.720 \times 10^6$	K/FT	$K_{XX} = 8.700 \times 10^9$	K-FT/RAD
<u>OBE</u>	$K_Y = 1.911 \times 10^6$	K/FT	$K_{YY} = 1.178 \times 10^{10}$	K-FT/RAD
	$K_Z = 1.720 \times 10^6$	K/FT	$K_{ZZ} = 8.700 \times 10^9$	K-FT/RAD

X: East - West direction
Y: Vertical direction
Z: North - South direction

(ii) findings and comments

g. decoupling criteria for subsystems

(i) general procedure

The mass of a subsystem may be lumped into the supporting structure mass if its mass is less than one-tenth that of the supporting mass. Otherwise, the subsystem must be modeled into the structural model.

Reference: BC-TOP-4A, Rev. 3

(ii) key examples

A lumped parameter model of the NSSS that provided an adequate representation of the mass and stiffness of this subsystem was furnished by Combustion Engineering. The NSSS was then coupled to the structural lumped parameter model. Mass of other major equipment was included in the analysis.

(iii) The other criteria pertaining to frequency ratio as defined in SRP 3.7.3.II.3.b are also met

iv
(~~iii~~) general findings and comments

3. development of in-structure response spectra

Refer to FSAR, Section 3.7.2.5

a. general procedures

The "SPECTRA" program was used to compute the response spectra. Refer to FSAR Appendix 3B for a description and applications of this program.

(i) smoothing (describe specific smoothing method used)

The smoothened response spectra represent an envelope of the maximum peaks.

(ii) peak widening

± 15%

Refer to FSAR, Section 3.7.2.9

b. typical results (attach figures)

(i) basemat spectra

Refer to FSAR, Figures 3D-23, -25x (Pages 149 and 150).

(ii) reactor supports spectra

Refer to FSAR, Figures 3D-13, -15, -17x (Pages 144-146).

(iii) steam generator supports

Refer to FSAR, Figure 3D-21x (Page 148).

(iv) Steam generator upper supports

Refer to FSAR, Figures 3D-7, -19x (Pages 141 and 147)

(v) reactor coolant pump support:

Refer to FSAR, Figures 3D-9, -11x (Pages 142 - 143)

(vi) pressurizer supports

Refer to Pages 151, 152 and 158 for ~~Project Design Criteria Part IV, pp. IV.2-25, IV.2-27, IV.2-79~~

applicable response spectra

(vii) operating floor and crane support.

Refer to Pages 156 - 157 for applicable ~~Project Design Criteria Part IV, pp. IV.2-67 and IV.2-75.~~ response spectra

(viii) top of steam generator

Refer to FSAR, Figure 3D-19 (Page 147)

(ix) base of fuel pool

Refer to Pages 153 - 154 for applicable ~~Project Design Criteria Part IV, pp. IV.2-39 and IV.2-63.~~ response spectra

(x) interior floors key floors with floor elevations identified)

Pages 153-156 for applicable
Refer to ~~Project Design Criteria Part IV, pp. IV.2-39, IV.2-63, IV.2-65, IV.2-67.~~
response spectra.

D. Vertical Dynamic Analysis

a. Mathematical Model - general description with sketch

The vertical dynamic analyses utilized similar models and methods as used for the horizontal dynamic analyses.

For sketches refer to FSAR, Figure $\underbrace{\text{3.7-10}}^{\text{(Page 96)}}$ and Attachment A (Page 64).

b. Development of stiffnesses, including floor stiffness, as applicable.

See answer to part (a) above.

c. Method of Analysis

See answer to part (a) above.

Description of method used as well as each subitems considered in the analysis

See answer to part (a) above.

E. Seismic Analysis for Polar Crane

a. Mathematical Model - general description with sketch

A lumped mass model was used in conjunction with the response spectrum analysis.

b. Stiffness calculations

c. Inputs

d. Key analysis results

F. Seismic Analysis for Buried Piping and/or Electrical conduits

a. Method of Analysis

Refer to Amended FSAR, Section 3.7.3.12 (see Attachment C for buried piping), Pages 70A-70GG, Appendix 3G of Amended FSAR

b. Stiffness calculations

c. Inputs

d. Key analysis results.

2. Stress Analysis

A. containment shell

a. Mathematical model - general description w/sketch

Refer to FSAR, Section 3.8.1.4 and ~~Final Analysis Design Report~~, Figures A.1-1, A.1-2, A.1-3 (Pages 159-161)

b. Method of analysis--incorporation of torsion

The overall analysis of the containment for the application of axisymmetric loads were performed by Bechtel's nonlinear FINEL finite-element computer program.

For the seismic induced loads (non-axisymmetric loads), the analysis was performed by Bechtel's linear elastic ASHSD finite-element computer program. The torsional loads on the axisymmetric containment structure shell are negligible.

Refer to FSAR, Section 3.8.1.4.2.

B. Containment Internals

a. mathematical model - general description w/sketch

A three dimensional finite element BSAP model was developed for the analysis and design of the major concrete walls of the internal structures. Concrete slabs and major structural steel members were included in the model ~~for~~ mass and stiffness ~~only.~~ determination. The structural steel, concrete slabs and other minor structures were analyzed and designed manually. See Attachment B for a sketch of the finite element model.

b. Method of analysis - incorporation of torsion

A three dimensional finite element model of the containment internal structure was developed to obtain the stress distributions within the interior structure. For seismic induced loads, the response spectra technique was employed. Torsional effects are accounted for by modeling the structure three-dimensionally, inherently incorporating the actual eccentricity of the structure in the analysis. Accidental torsional effects will be addressed in the design.

c. Load combinations

Refer to FSAR, Section 3.8.3.3. We are in compliance with the SRP

d. key results (figures, etc.)

Refer to amended FSAR, Table 3.8-4A. (See Attachment K) (not included)

C. Foundation mat including reactor pit

a. mathematical model - description of boundary conditions

FINEL
In both the ASHSD and ~~Finel~~ finite-element analyses, elements were extended into the soil to account for the elastic nature of the soil material and its effect on the behaviour of the basemat.

Refer to FSAR, Section 3.8.1.4.2 and ~~Final Analysis Design Report,~~ Figures A.1-1, 4 1-2, A.1-3, and Attachment Dx (Page 71).
A (Pages 159 - 161)

b. Method of analysis

The basemat and reactor pit were analyzed using the same methods as for the containment shell. Refer to page 19, part b.

c. load combinations

Refer to FSAR, Table 3.8-1Ax (Pages 97 and 98) We are in compliance with ASME, III, Div. 2, Subsection CC 3000 and BCTOP 5A.

d. key results (figures, etc.)

Refer to FSAR, Table 3.8-1Bx (Pages 99 - 107)

(3) computer programs used in analysis

BSAP, SAP 1.9, ASHSD, OPTCON, FINEL, FOSIN, LUCON, TENDON, SUPER SMIS, CECAP, STICK, STRUDL-DYNAL, SPECTRA, CONDAM.

A. Assumptions and limitations

See FSAR, Appendix 3B.

B. applicability

Refer to FSAR, Appendix 3B and FSAR, Sections 3.8.1.4.2, 3.7.2.3, 3.7.2.

C. Verification

Sensitivity study in case of numerical solutions (e.g., finite element analysis)

Refer to FSAR, Appendix 3B

D. load input (include all cases)

o Dead and live loads are inputted as gravity loads.

o Prestressing loads are inputted as either equivalent pressure loads, external loads, or using pseudo thermal loads.

o Pressure loads are inputted as surface pressure loads.

o Seismic loads are inputted from the free-field design spectra or from floor response spectra.

o Thermal loads are inputted as actual temperature differences or as thermal gradients.

o Equipment reaction loads (including LOCA and seismic loads) are inputted as concentrated loads.

E. Output (include all cases)

See FSAR Appendix 3B.

F. Other discussions

(4) overall stability

A. forces and moments from seismic analysis

ELASTIC FORCES & MOMENTS

	Horizontal Force	Overturning Moment
OBE	24,143 K	2.60×10^6 K-FT
SSE	39,600 K	4.27×10^6 K-FT

B. Various cases considered

Seismic event loading combinations considered SSE or OBE applied in the North-South, East-West, and vertical directions simultaneously.

C. bearing pressure versus bearing capacity and safety factor against bearing failure

Refer to FSAR, Section 2.5.4.10 and Tables 2.5-15 and 2.5-16x (Pages 87 and 88),

D. factors of safety

Refer to FSAR, Section 3.8.5.5 and Table 3.8-5x (Page 111)

a. sliding

Factor of Safety - 1.2 (SSE)

b. overturning

Factor of Safety - 1200 (SSE)

(5) interaction of non-category I structures

A) identification of pertinent non-category I structures

None. There are no non-category I structures adjacent to the Containment Building.

B) consideration given to potential failure of non-category I systems on Category I systems

Yes, During a walkdown, those items whose failure will not affect any safety related equipment are left as they are. If they are judged that they might affect category I systems, they are designed to maintain their structural integrity under an SSE.

C) general findings and comments

III. Conformance to Acceptable Criteria

(1) Identification of deviations, if any

None.

(2) justification of deviations and disposition of the deviations

(3) Comparison of reevaluation results with the original design bases and discussions.

(4) general comments

PART II - AUDIT OF KEY DESIGNS

For each key design area audited, the design calculations should be reviewed together with applicable drawings, sketches, etc. Also, key details and/or sections, as appropriate, in this audit report should be included.

(1) Containment liner design

conformance with Div. 2-Article CC-3000

The design of the liner plate system conforms to the provisions of "Containment Building Liner Plate Design Report," BC-TOP-1, Revision 1, Bechtel Power Corporation, San Francisco, CA, and "Additional Information Requested by the Atomic Energy Commission on BC-TOP-1, Revision 1, Containment Building Liner Plate Design Report," dated September 1973. We are in compliance with ASME, III, Div. 2, Article CC-3000.

specific check of key liner locations

A. cylinder-base mat junction

(a) sketch

See Attachment E (Page 72)

(b) forces and displacement obtained from computer analysis

Refer to FSAR, Tables 3.8-1A and 3.8-1B (Pages 97 - 107).

(c) controlling stress, strain from analysis considering various load combinations

Refer to FSAR, Tables 3.8-1A and 3.8-1B (Pages 97-107).

B. Anchorage between the liner and interior concrete slab

Refer to FSAR, Figures 3.8-12 and 3.8-13 (Pages 119-120), and Attachment F (Page 73)

C. liner anchor design (model, analysis, procedure, assumption)

See response on page 28, item (1)

D. other embedment design

Polar Crane Bracket

Refer to FSAR, Figure 3.8-20 (Page 124)

E. key penetration design

Refer to FSAR, Figures 3.8-17, 3.8-18 and 3.8-19 (Pages 121-123).

For design, refer to BC-TOP-1 and BC-TOP-5A

F. preliminary audit findings

(2) containment hatch design

A. design requirements and assumptions

The design requirements for the concrete portion of the containment equipment hatch are given in the FSAR, Section 3.8.1. The containment equipment hatch analysis used as input the resultant forces and moments obtained from the overall containment analysis..

The steel parts of the equipment hatch not backed by concrete were designed by W. J. Woolley Co. in accordance with the ASME B & PV Code, Section III, Division 1, Subsection NE.

B. model

The equipment hatch area is analyzed using a detailed finite element model. the model incorporates conditions, to account for Symmetrical and Non-Symmetrical Loading. The model has 1750 nodes, 1216 brick elements and 525 truss elements. Brick elements were used to model the concrete and truss elements were used to model the Post-Tensioning System.

(Page 74)

See Attachment G for a sketch of the model

C. analysis procedure and results

A three dimensional finite element model of the equipment hatch was prepared and analyzed by the use of the BSAP computer program. Forces and Moments from BSAP computer programs are used as input for the OPTCON computer program to optimize reinforcing steel.

The steel parts of the equipment hatch were designed manually by W. J. Woolley.

D. key controlling loads including appropriate load combinations

The key controlling loads are:

1. $D + F + E_o + T_o$
2. $D + F + 1.25Pa + 1.25E_o + Ta$
3. $D + F\ 1.5Pa + Ta$

D = Dead Load	E_o = OBE
F = Prestress	T_o = Operating Temperature
Pa = Accident Pressure	Ta = Accident Temperature

E. key stresses and strains for section designs

F. conformance to CC-3000

concrete portion of the ASME III, DIV. 2, subsection

The hatch design is in compliance with CC-3000.
Reference: FSAR, Section 3.8.1.4

The steel hatch design is in compliance with ASME III, DIV. 1, subsection NE.

G. general comments and preliminary audit findings

(3) foundation slab design

A. design requirements and model

The design requirements for the Containment Basemat are given in the FSAR, Section 3.8.1. Sketches of the finite-element computer models are shown in ~~the Containment Final Analysis Design Report,~~ Figures A.1-1, A.1-2 and A.1-3 (Pages 159-161)

B. design loads (from general analysis)

Refer to FSAR, Section 3.8.1.3

C. forces and moments at key sections

Refer to FSAR, Tables 3.8-1A and 3.8-1B (Pages 97-107).

D. elastic deformation curve of the slab

See Attachment H (Page 75)

E. detailed design of rebar placement at key section:

Refer to FSAR, Figures 3.8-1 and 3.8-2 (Pages 112-113).

F. conformance to CC-3000

ASME III, DIV. 2, Subsection

The design of the basemat is in compliance with CC-3000.
Reference: FSAR, Section 3.8.1.4

G. general comments and preliminary audit findings

(4) containment wall-base mat junction design

A. design requirements and model

See reply on page 34 for foundation slab design.

B. design loads (from general analysis)

Refer to FSAR, Section 3.8.1.3

C. forces and moments at key sections

Refer to FSAR, Tables 3.8-1A and 3.8-1B (Pages 97-107).

D. detailed design of rebar

Refer to FSAR, Figure 3.8-4 (Page 114).

E. Any waterstop membranes at the joint, their design considerations and installations.

Refer to FSAR Figure 3.8-2 (Page 113). The waterstops are made of styrene - butadienc synthetic rubber. The water stops are used at construction joints below the design groundwater level.

F. Conformance to CC - 3000

The wall - basemat junction is in compliance with ASME III, DIV 2, Subsection CC-3000.
Reference: FSAR, Section 3.8.1.4

G. general comments and preliminary audit findings

(5) general design for membrane shear forces

A. design requirements and model

Refer to FSAR Section 3.8.1.5.3.

B. design loads (from general analysis)

Refer to FSAR, Section 3.8.1.3

C. forces and moments at key sections

Appendix 3F of Amended FSAR, 75A

Refer to ~~Appendix J, of the Containment Final Analysis Design Report.~~

(ATTACHMENT I, Pages 75A-75K)

D. detailed design of rebar

Refer to FSAR, Figure 3.8-4 (Page 114).

E. conformance to CC-3000

The ASME Section III, Division 2 Code presently has no tangential shear requirements. Refer to Amended FSAR, Section 3.8.1.5.3 (See Attachment I, Pages 75A-75K, Appendix 3F of Amended FSAR).

F. general comments and preliminary audit findings

(6) dome-to-cylinder junction design

A. design requirements and model

Refer to answer on page 34, item A.

B. design loads (from general analysis)

Refer to FSAR, Section 3.8.1.3

C. forces and moments at key sections

Refer to FSAR, Tables 3.8-1A and 3.8-1B (Pages 97-107).

D. detailed design of rebar placement at key sections

Refer to FSAR, Figure 3.8-6 (Pages 116 & 117)

E. conformance to CC-3000

The dome-to-cylinder junction design is in compliance with ASME III, Div.2, subsection CC-3000.

F. general comments and preliminary audit findings

(7) primary shield wall-base mat junction

A. design requirements and model

The primary shield is intended to be the biological shield for the reactor vessel. It also serves as the support for the vessel under all loading conditions. A three dimensional finite element model was used for the analysis and design. See Atachment J K (Pages 76-77).

B. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

C. forces and moments at key sections

Refer to Amended FSAR, Table 3.8-4A (See attachment K) (not included)

D. detailed design of rebar placement at key sections

Refer to FSAR, Figures 3.8-30, 3.8-31, 3.8-32, and 3.8-33 (Pages 131-134)

E. code jurisdiction boundary definition and anchor treatment at interface.

The boundary between the primary shield and the containment basemat is the containment leaktight boundary. The primary shield is anchored into the reactor pit basemat by means of B-Series Cadwelds welded to the top and bottom of the thickened liner plate. Reinforcing steel is then attached to the B-Series Cadwelds in the reactor pit.

F. conformance to SRP requirements

The design of the primary shield is in compliance with SRP requirements.

G. general comments and preliminary audit findings

(8) operating floor design

A. design requirements and model

The design requirements for the operating floor are given in FSAR, Section 3.8.3. The design of the operating floor was based on manual calculations. A plan of the operating floor is shown in Figures 3.8-35 and 3.8-36 of the FSAR (Pages 137-138)

B. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

C. forces and moments at key sections

Refer to Amended FSAR, Table 3.8-4A (See Attachment K) (not included)

(9) crane support design

A. design requirements and model

Refer to FSAR Section 3.8.3.4 for design requirements.
The design of the supports were done manually. A sketch of the polar crane support girders is shown on Figure 3.8-38 of the FSAR (Page 140).

B. design loads (from general analysis)

Load Combination	Vertical Downward Load	Horizontal Load
DL+LL+Impact	*563k	31k
DL+OBE	359	147k
DL+SSE	416k	240k

These loads are for each corner of the polar crane. Each corner has two trucks and each truck has two wheels.

C. forces and moments at key sections

For operating conditions
Maximum moment at mid-span of support girder: 5867 in-k

Maximum load onto a single polar crane bracket: 305 kips

D. detailed design of rebar placement at key sections

Refer to FSAR, Figure 3.8-3, 3.8-4 (Pages 113A and 114)

E. Interface with containment shell, if applicable

For a sketch of the polar crane bracket and its interface with the containment shell, refer to FSAR, Figures 3.8-37 and 3.8-20~~x~~ (Pages 124 and 139)

F. conformance with SRP requirements and reevaluation criteria

The design of the crane supports is in compliance with SRP requirements.

G. general comments and preliminary audit findings

(10) reactor vessel support design

A. design requirements and model

The design requirements for all NSSS supports were furnished by Combustion Engineering. Bechtel manually designed the NSSS supports.

For a sketch of the reactor vessel supports, refer to FSAR Figures 3.8-23 and 3.8-24x (Pages 125-126)

B. design loads (from general analysis)

The design loads were furnished by Combustion Engineering.

C. forces and moments at key sections

See Attachment L (Pages 78-79)

D. detailed design of rebar

Refer to FSAR, Figures 3.8-30, -31, -32, -33 (Pages 131-134)

E. conformance with SRP requirements

The reactor vessel support design is in compliance with SRP requirements.

F. general comments and preliminary audit findings

(11) steam generator support design

A. design requirements and model

The design requirements for all NSSS supports were furnished by Combustion Engineering. Bechtel manually designed the NSSS Supports.

For a sketch of the steam generator support, refer to FSAR, figures 3.8-25 and 3.8-26x (Pages 127-128)

B. design loads (from general analysis)

The design loads were furnished by Combustion Engineering.

C. forces and moments at key sections

See Attachment M (Pages 80-82)

D. detailed design of rebar placement at key sections

See Attachment N (Pages 83-84)

E. conformance with SRP requirements

The steam generator support design is in compliance with SRP requirements.

F. general comments and preliminary audit findings

(12) coolant pump support design

A. design requirements and model

The design requirements for all NSSS Supports were furnished by Combustion Engineering. Bechtel manually designed the NSSS supports.

For a sketch of the reactor coolant pump supports, refer to FSAR, Figure 3.8-27 and 3.8-28x (Pages 129-130)

B. design loads (from general analysis)

The design loads were furnished by Combustion Engineering.

C. forces and moments at key sections

MAXIMUM TENSION IN ANCHOR BOLTS

	DESIGN	ALLOWABLE
Lower Supports	280^k	600^k
Lower Lateral Supports	242^k	600^k
Upper Lateral Support	256^k	600^k
Snubber Supports	204^k	600^k

Reference: FSAR, Figure 3.8-27, and 3.8-28 (Pages 129-130)

D. detailed design of rebar placement at key section

See Attachment O (Page 85)

E. conformance with SRP requirements

The reactor coolant pump support design is in compliance with SRP requirements.

F. general comments and preliminary audit findings

(13) secondary shield walls

A. design requirements and model

The design requirements for the secondary shield walls are given in FSAR, Section 3.8.3. See Attachment B for sketches of the finite element model used in the analysis and design.

(Pages 65-70)

B. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3

C. forces and moments at key sections

Refer to Amended FSAR, Table 3.8-4A (See Attachment K) (not included)

D. detailed design of rebar placement at key section

Refer to FSAR, Figure 3.8-34 (Page 136)

E. conformance with SRP criteria

The design of the secondary shield walls are in compliance with SRP requirements.

F. general comments and preliminary audit findings

(13) other steel structures

A. design requirements and model

The design requirements for structural steel within the containment structure are given in the FSAR Section 3.8.3. The steel structures were designed by conventional hand calculations.

B. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3

C. forces and moments at key sections

Refer to FSAR, Table 3.8-4B (Pages 108-110)

D. detailed design

See Attachment P (Page 86)

E. conformance with SRP requirements

The steel design is in compliance with SRP requirements.

F. general comments and preliminary audit findings

(15) Post-Tensioning System and Anchorage

A. Post-Tensioning System

1. Tendon System used

The post-tensioning system used is the BBRV system. The tendons consists of 186-1/4 inch diameter, high strength wire in conformance with ASTM A421 Type BA. The wires are anchored by means of button heads. Reference: FSAR, Figure 3.8-7 (Page 118)

2. Prestressing force at transfer

Cylinder Hoop Tendons - 1495 kips per tendon (163.8 ksi)
Dome Hoop Tendons - 1496 kips per tendon (163.9 ksi)
Vertical ∩ shaped tendons at top of dome - 1299 kips per tendon (142.3 ksi).

3. Tendon load under LOCA

Cylinder Hoop Tendons - 1237 kips per tendon (135.5 ksi)
Dome Hoop Tendons - 1255 kips per tendon (137.5 ksi)
Vertical ∩ shaped tendons, at top of dome - 1055 kips per tendon (115.5 ksi). The forces and stresses given above are after all losses at the end of plant life.

4. Method used to calculate transfer losses:

a) Friction Calculated from the equation $f_x = f_o e^{-(KL + \mu\alpha)}$ given in ACI-318-71, Chapter 18.

b) creep

Based on creep strain of 500×10^{-6} in/in at the end of plant life.

c) Concrete shrinkage.

Based on shrinkage strain of 100×10^{-6} in/in at the end of plant life.

5. Buttress Design

a) Maximum bursting stress in concrete

Refer to BC-TOP-7, "Full Scale Buttress Test for Prestressed Nuclear Containment Structures"

b) Reinforcing provided to resist bursting stresses - -

(Page 115)

For a sketch of the reinforcement in the buttress, refer to the FSAR, Figure 3.8-5. The design of the tendon anchorage zones is based on two test programs conducted by Bechtel to demonstrate the adequacy of several reinforcing patterns for use in anchorage-zone concrete in the basemat and buttresses. The test results demonstrate satisfactory performance of the test anchorages. The design of the tendon anchorage zones is based on the results and recommendations of these tests. Refer to the FSAR, Section 3.8.1.4.4 for a description of the test programs.

c) Stress under the anchor plate

3.5 ksi

d) Allowable stresses

4.2 ksi

e) Stress under the " ∩ " tendons anchorage

3.5 ksi

f) Method of calculation of stresses

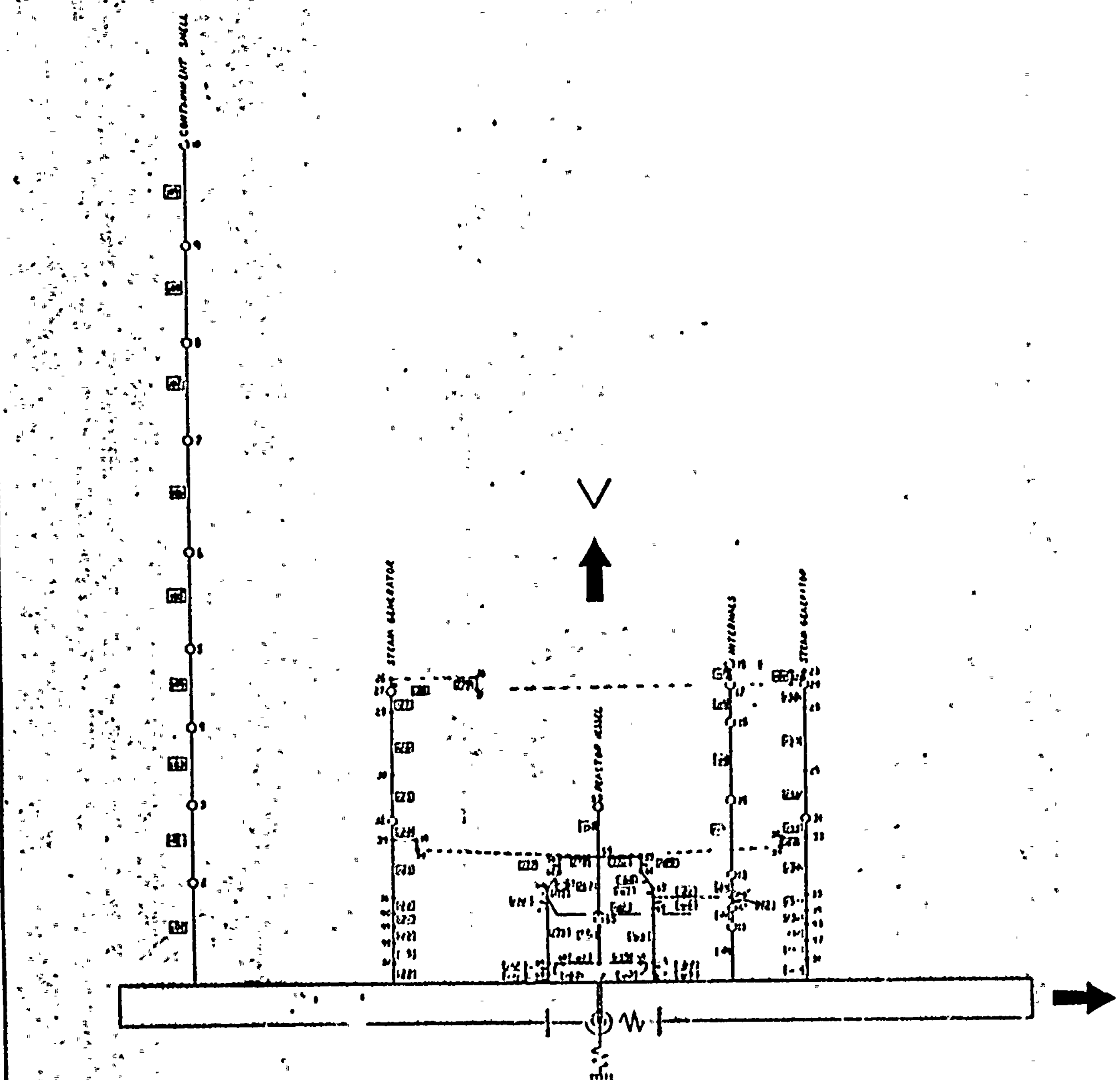
The stresses were determined by a manual calculation

g) Allowable stresses

4.2 ksi

3.6

	HORIZONTAL			VERTICAL	
DYNAMIC DEGREE OF FREEDOM	NODE DISPLACEMENT	DYNAMIC DEGREE OF FREEDOM	NODE DISPLACEMENT	DYNAMIC DEGREE OF FREEDOM	NODE DISPLACEMENT
1	Y_1	21	Y_{11}	1	V_1
2	θ_1	22	θ_{11}	2	V_2
3	Y_2	23	Y_{12}	3	V_3
4	θ_2	24	θ_{12}	4	V_4
5	Y_3	25	Y_{13}	5	V_5
6	θ_3	26	θ_{13}	6	V_6
7	Y_4	27	Y_{14}	7	V_7
8	θ_4	28	θ_{14}	8	V_8
9	Y_5	29	Y_{15}	9	V_9
10	θ_5	30	θ_{15}	10	V_{10}
11	Y_6	31	Y_{16}	11	V_{11}
12	θ_6	32	θ_{16}	12	V_{12}
13	Y_7	33	Y_{17}	13	V_{13}
14	θ_7	34	θ_{17}	14	V_{14}
15	Y_8	35	Y_{18}	15	V_{15}
16	θ_8	36	θ_{18}	16	V_{16}
17	Y_9	37	Y_{19}	17	V_{17}
18	θ_9	38	θ_{19}	18	V_{18}
19	Y_{10}	39	Y_{20}	19	V_{19}
20	θ_{10}	40	θ_{20}	20	V_{20}
		41	Y_{21}	21	V_{21}
		42	θ_{21}		



MODEL FOR N-S DIRECTION (PERPENDICULAR TO THE REACTOR HOT LEG)

LEGEND

- RIGID MEMBER
- MASS POINT
- NODAL POINT
- TRANSLATIONAL SPRING
- ROTATIONAL SPRING
- MEMBER NUMBER

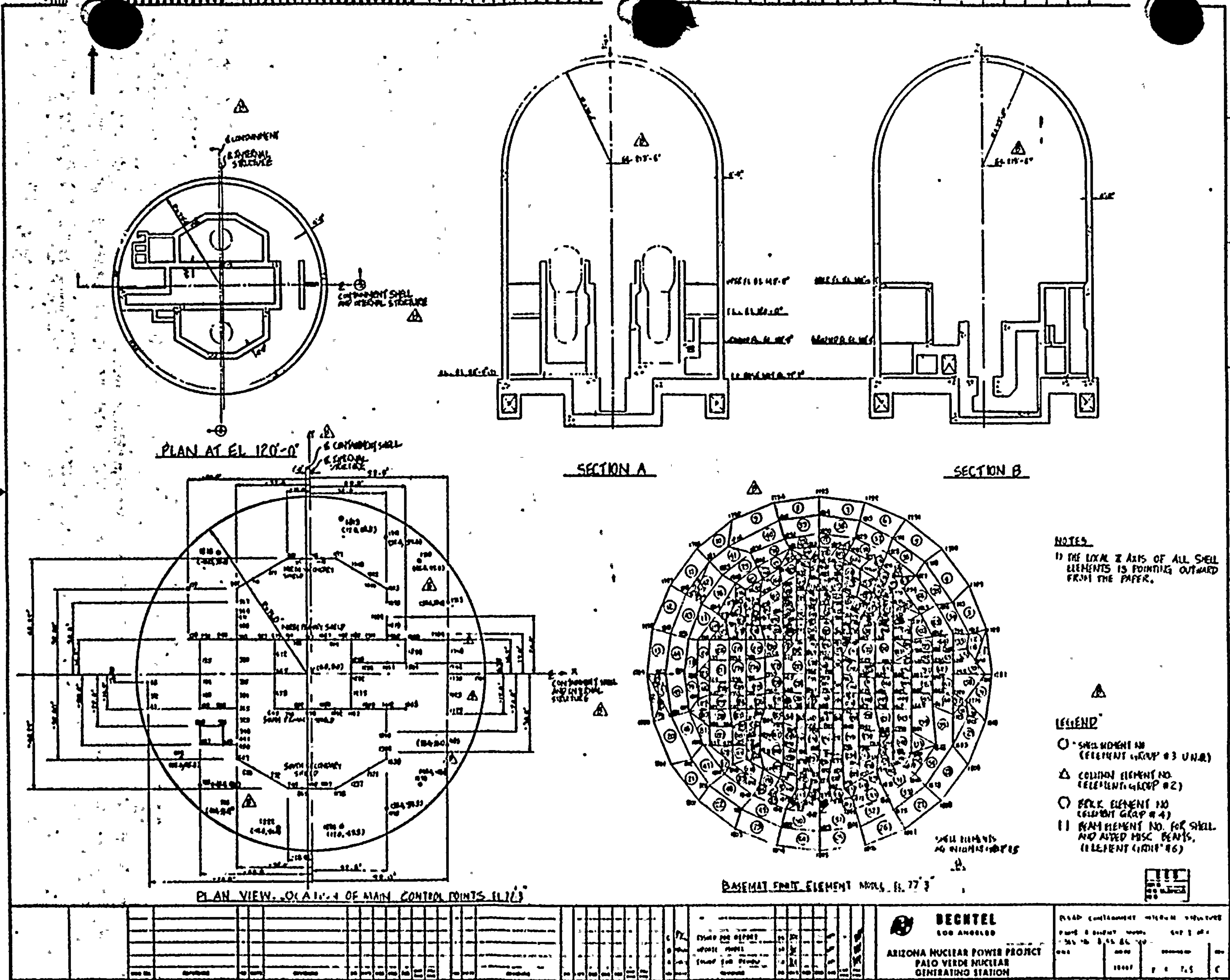
ATTACHMENT A

BECHTEL
Los Angeles

ARIZONA NUCLEAR POWER PROJECT
PALO VERDE NUCLEAR
GENERATING STATION

CONTAINMENT SEISMIC ANALYSIS
CONTAINMENT - INTERNAL NSSS
MATHEMATICAL MODEL (N-S)

SK C 275 | 4



ATTACHMENT B

1 OF 6

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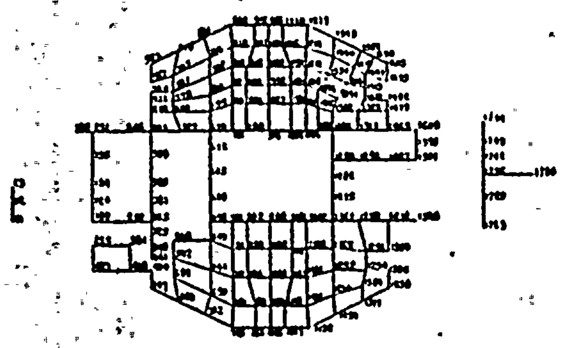
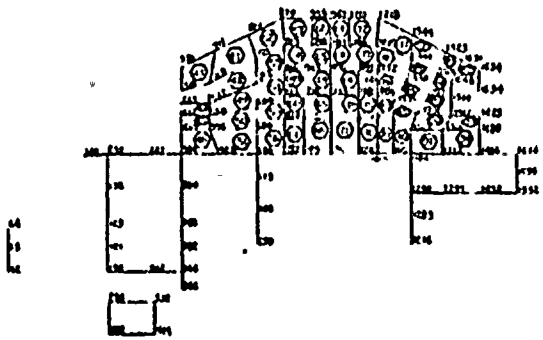
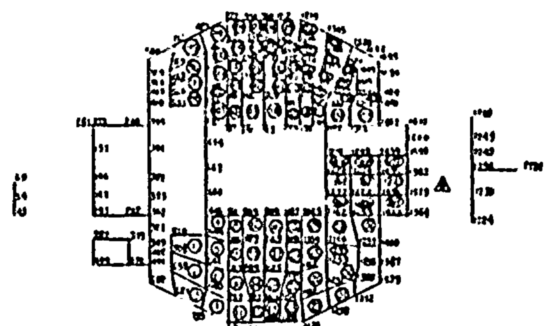


PLAN VIEW - EL 77.25'

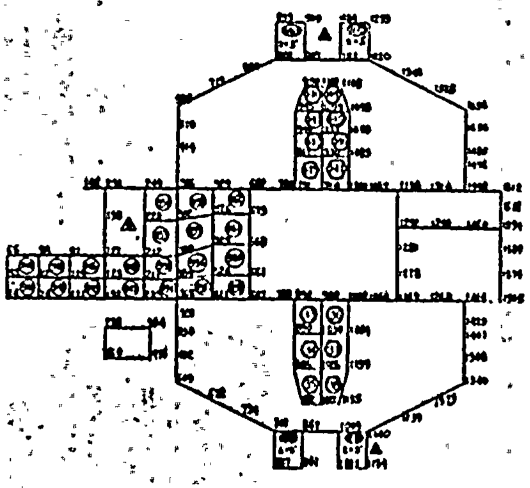


PLAN VIEW EL 82.0'



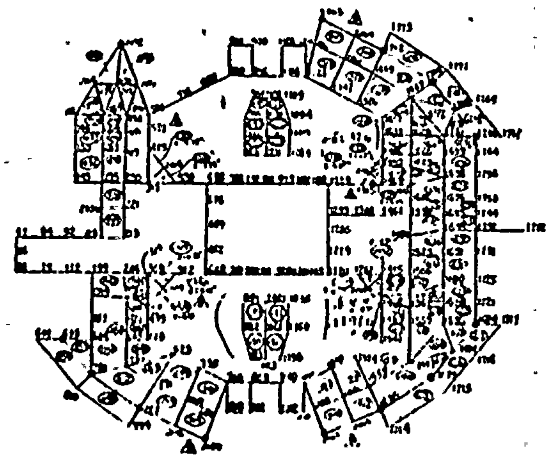
FOR SHELL ELEMENTS t = 5.0'

PLAN VIEW EL 87.0'



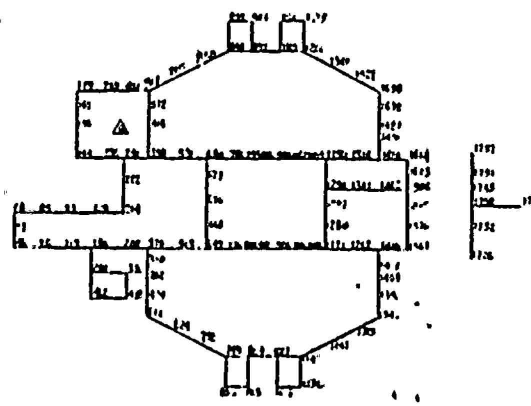
FOR SHELL ELEMENTS t = 6.5' U.N.O.

PLAN VIEW - EL 94.0'



FOR SHELL ELEMENTS t = 2.5 UNO

PLAN VIEW - EL 9[illegible]



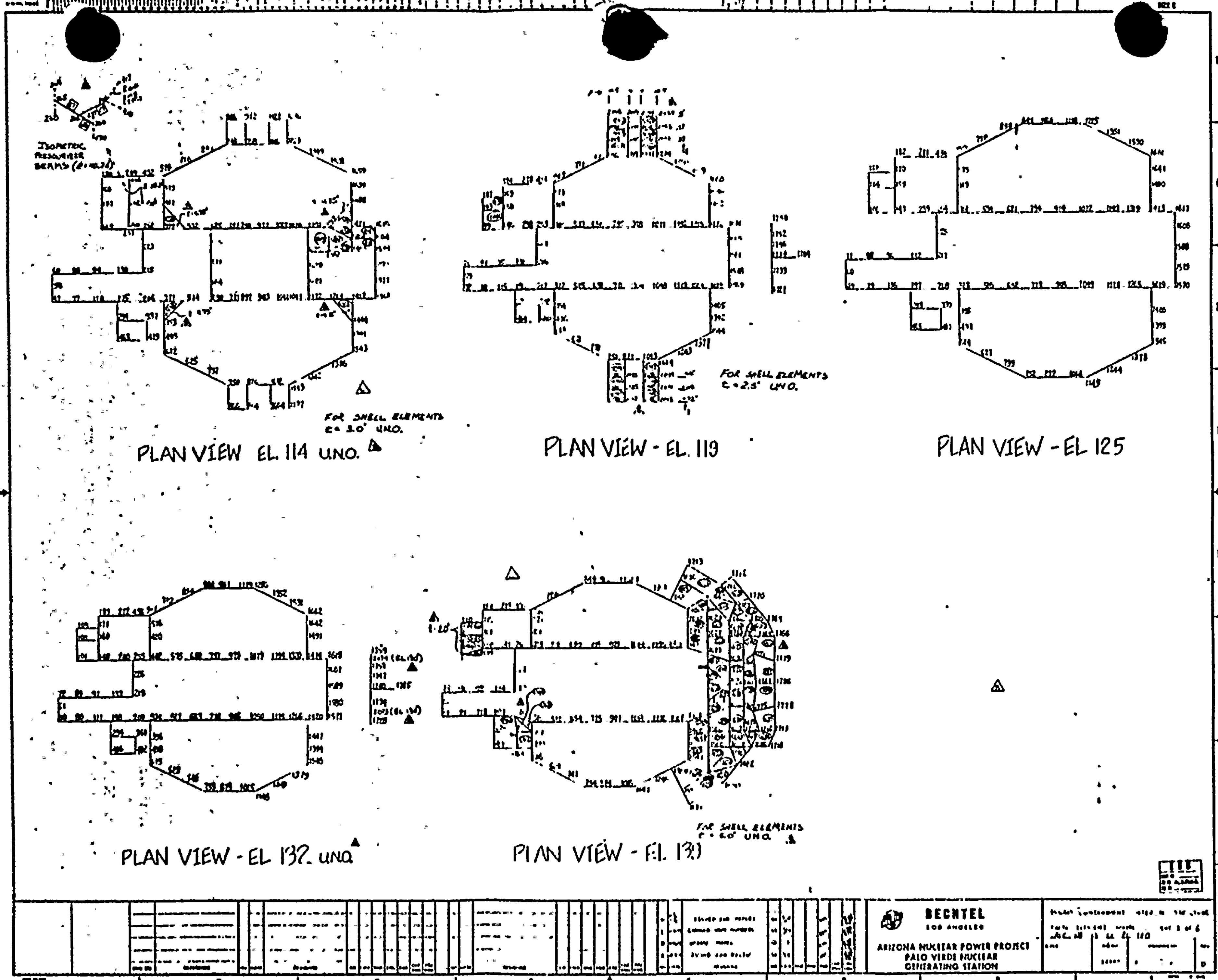
PLAN VIEW EL 106.5'

2 OF 6

ATTACHMENT B

BECHTEL
LOS ANGELES

ARIZONA NUCLEAR POWER PROJECT
PALO VERDE NUCLEAR
GENERATING STATION



ATTACHMENT B 3 OF 6

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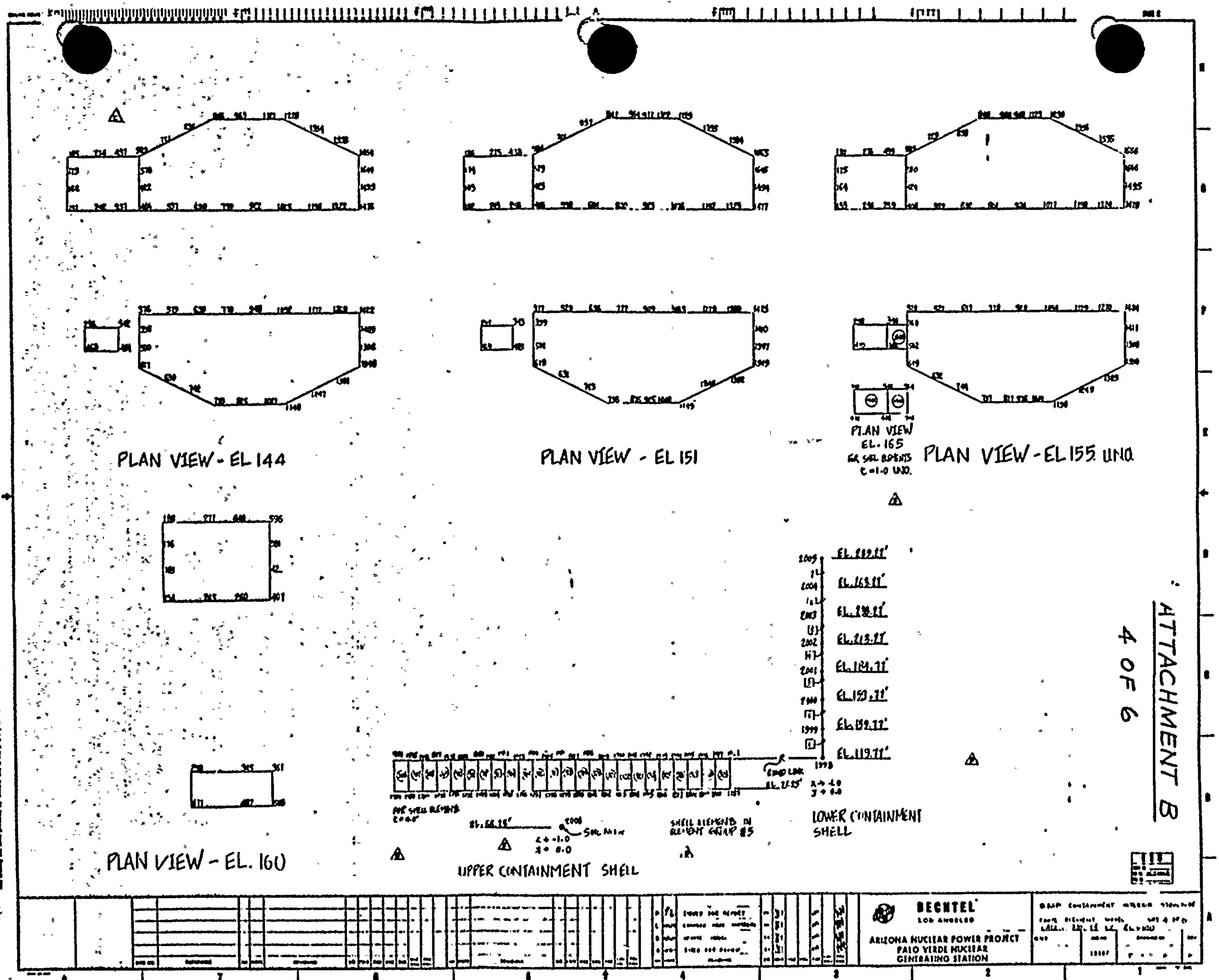
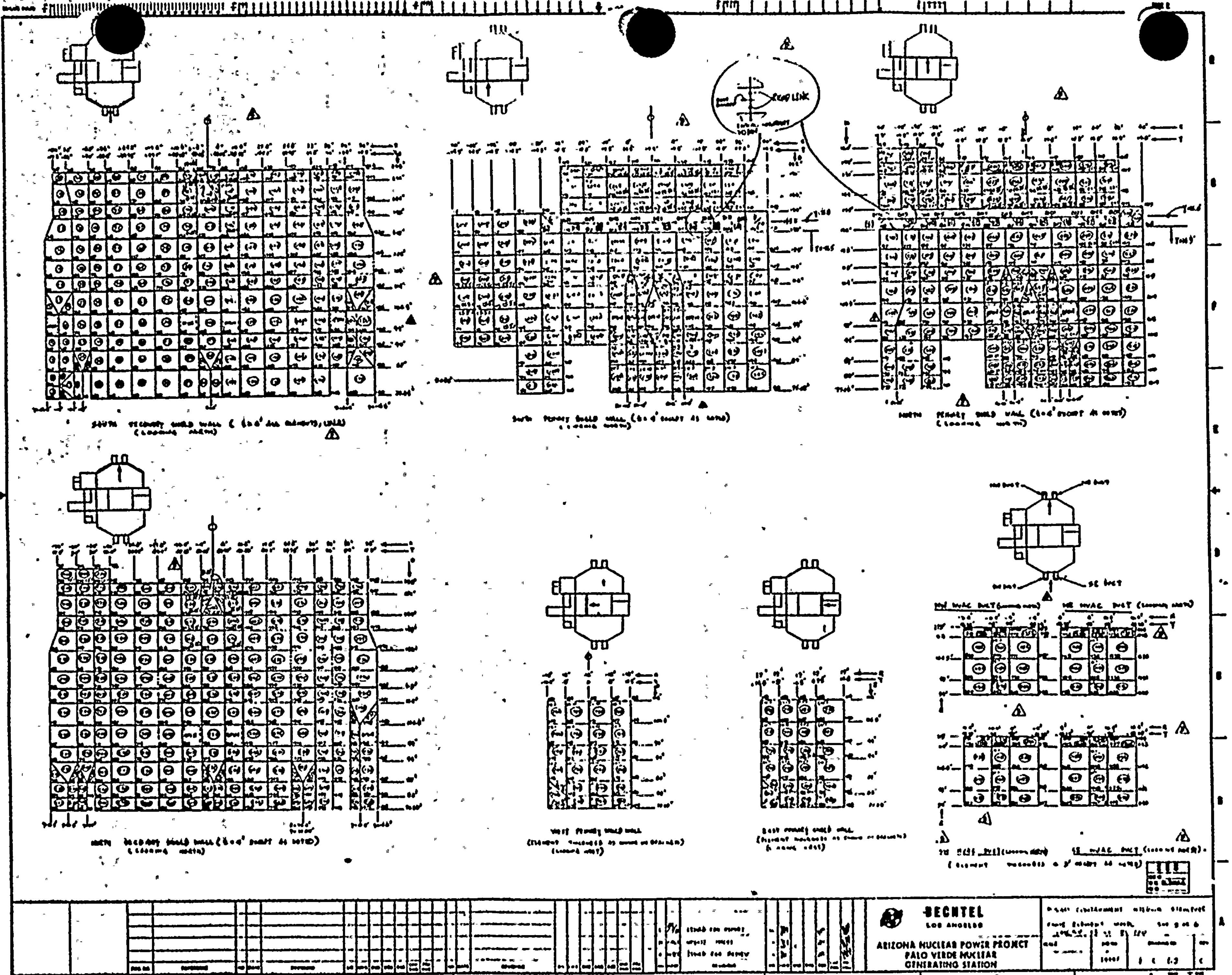


PLAN VIEW - EL 144
PLAN VIEW - EL 151
PLAN VIEW EL. 165
PLAN VIEW - EL 155 UNO
PLAN VIEW - EL. 160
UPPER CONTAINMENT SHELL
LOWER CONTAINMENT SHELL
ATTACHMENT B
4 OF 6
BECHTEL
LOS ANGELES
ARIZONA NUCLEAR POWER PROJECT
PALO VERDE NUCLEAR GENERATING STATION



ATTACHMENT B

5 OF 6

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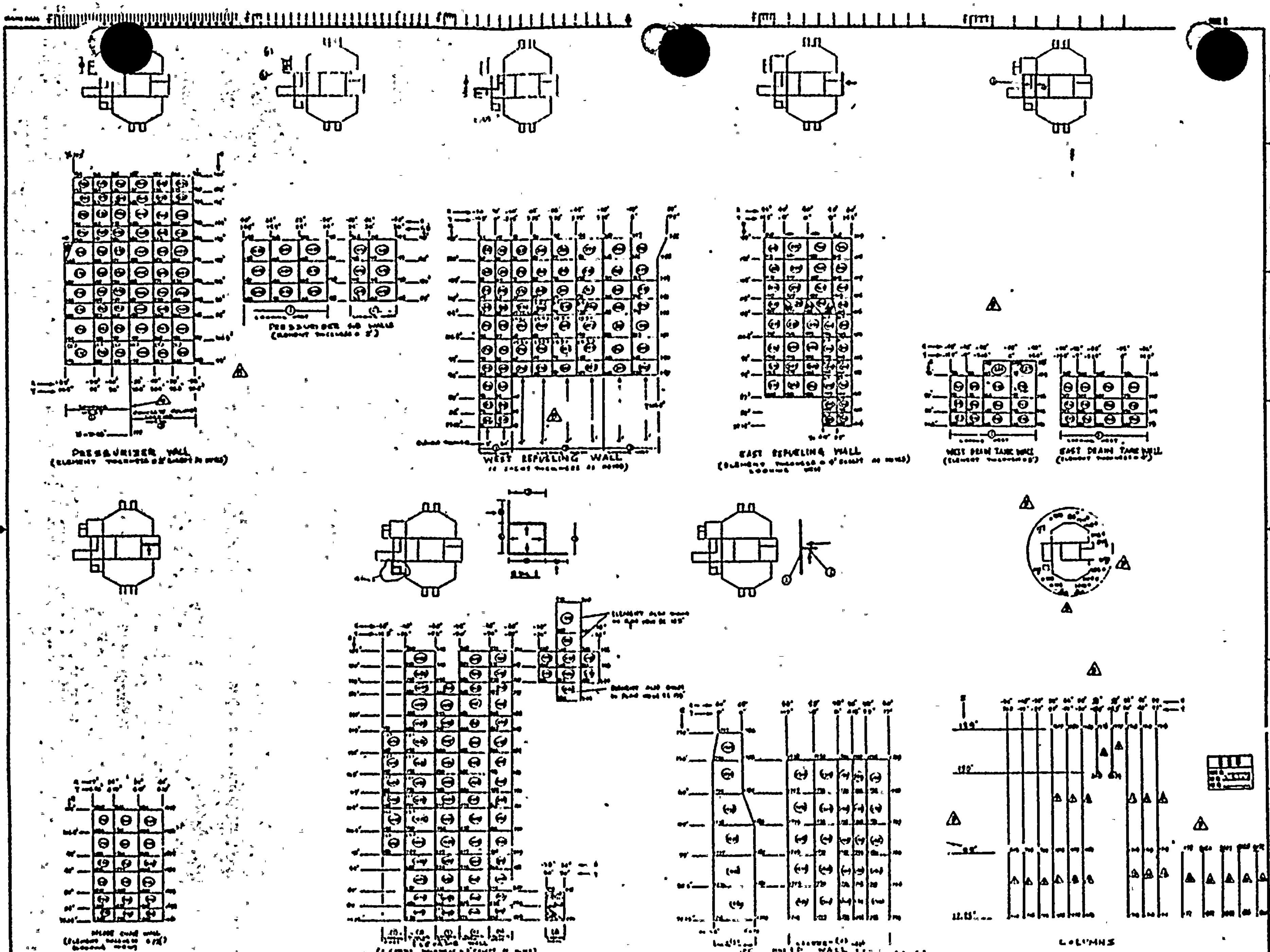

BECHTEL
LOS ANGELES

ARIZONA NUCLEAR POWER PROJECT
PALO VERDE NUCLEAR
GENERATING STATION

ATTACHMENT B
6 OF 6

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ATTACHMENT C

33 SHEETS

APPENDIX 3G

SEISMIC STRESSES IN

UNDERGROUND STRUCTURES

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REVIEW

APPENDIX 3G

SEISMIC STRESSES IN UNDERGROUND STRUCTURES

3G.1 SUMMARY

This section describes methods used for seismic analysis of buried structures such as conduits, tunnels, and well casings. The effects of earthquakes on buried structures may be broadly grouped into two classes: faulting and shaking. Faulting includes the direct, primary shearing displacement of bedrock which may carry through the overburden to the ground surface. Such direct shearing of the rock or soil is generally limited to relatively narrow zones of seismically active faults which may be identified by geological and seismological surveys. From a structural viewpoint, landsliding, ground fissuring, and consolidation of backfill soil have similar effects on buried structures. In general, it is not desirable to design structures to directly sustain such major soil displacements. However, design measures can be taken to mitigate the effects of the displacements and to identify and avoid areas prone to such displacements. 6

The effects of earthquake ground motion on underground conduits, in the absence of direct fault displacement or unstable soil conditions such as liquefaction, are:

1. Axial tension and compression due to traveling seismic wave
2. Shear and bending due to traveling seismic wave
3. Strain caused by dynamic differential movement at connections

Analytical procedures for evaluating these effects are described in the following sections. For very long structures, and procedures are based on the assumption that there is no relative motion between the flexible structure and the

ground. Seismic stresses in the conduit are estimated from the calculated strains and curvature in the surrounding soil due to the passage of seismic waves. For short structures, slippage may occur between the conduit and the soil and the calculated axial stresses are proportionately less than those assuming the conduit strain equal to the soil strain. The effects of bends and differential displacement at connections to buildings are evaluated using procedures based on equations for beams on elastic foundations. The calculated seismic stresses must be combined with stresses from other loading conditions, including pressure and surcharge loading, for final design.

The interaction of stresses and strains due to seismic wave propagation and boundary displacements, both at bends and at structures, is a complicated problem. The conservative assumption can be made that the strains due to the several sources are additive and, hence, an SRSS combination may be used. In case these resultant stresses and strains are unacceptable, the problem can be circumvented by designing discontinuities to be flexible to allow for the resultant displacements.

3G.2 STRESSES IN STRAIGHT SECTIONS

3G.2.1 GENERAL EQUATIONS FOR AXIAL AND BENDING STRAIN

The portions of a long, buried structure far from the ends and free of any external support other than the surrounding soil are assumed to be flexible and to follow essentially the displacements and deformations of the soil during seismic ground motion. Soil displacements due to the passage of shear, compression, and surface waves are calculated based on wave propagation velocities and the maximum ground particle acceleration and velocity due to the design earthquake.

Stresses in the structure are calculated using the resulting strain, curvature, and modulus of elasticity of the structural material.

The assumption that relative motion between the buried structure and the surrounding soil is negligible has been shown by O'Rourke and Wang (1978) to be a valid assumption for most practical cases. For special situations where the relative motion is not negligible, and analysis techniques described by Hindy and Novak (1978) and O'Rourke and Wang (1978) can be used. Internal walls which may not follow the motion of the surrounding soil can be treated as simple oscillators subject to the design ground motion at the depth of burial.

The basic relations for calculating maximum longitudinal strain and curvature induced in a flexible, buried structure have been presented by Hall and Newmark (1978). For a compression wave propagating along the longitudinal axis of the buried structure 6

$$\varepsilon_m = \pm \frac{v_{mp}}{C_p} \qquad (3G\text{-}1)$$

and for a shear wave propagating along the longitudinal axis

$$\varepsilon_m = \pm \frac{v_{ms}}{2C_s} \qquad (3G\text{-}2)$$

$$K_m = \frac{a_{ms}}{C_s^2} \qquad (3G\text{-}3)$$

where

ε_m = maximum longitudinal strain
K_m = maximum curvature
v_{mp} = maximum compression wave particle velocity
v_{ms} = maximum shear wave particle velocity
a_{ms} = maximum shear wave particle acceleration

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C_p = compression wave propagation velocity
C_s = shear wave propagation velocity

The maximum strain as given by Eqs. 3G-1 and 3G-2 is an upper bound since it is limited by the pipe-soil interface friction. Slippage would occur if the computed axial force, ε_m AE, exceeds the frictional resistance as given by Eq. 3G-21.

The appropriate particle acceleration (a_m) for calculating maximum soil strain is the maximum ground acceleration. The maximum particle velocity should be selected for the corresponding wave type. For example, the maximum ground velocity for the compression wave portion of ground motion prior to arrival of the surface wave component is typically less than the maximum ground velocity associated with the surface wave component. Therefore, it may be unnecessarily conservative to
6 take the maximum ground velocity in the entire ground motion when calculating maximum soil strain due to a compression wave.

The value of wave propagation velocity to be used when calculating maximum soil strain surrounding a buried structure is the effective velocity of the ground motion disturbance past the structure. For rock or very stiff and dense soils, the effective propagation velocity is equal to the in-situ wave propagation velocity as measured by field or laboratory tests. If the structure is embedded in a softer layer or at a shallow depth in uniform soils, the effective propagation velocity should be taken as the propagation velocity of the underlying competent soil or rock (Hall and Newmark, 1978). For example, the effective shear wave propagation velocity should not be taken as less than the shear wave velocity at a depth of 400 to 500 feet or, in any case, never less than about 2000 fps.

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3G.2.2 MAXIMUM AXIAL AND BENDING STRESSES

Equations for calculating maximum axial and bending stresses as a function of angle of incidence of the various wave-types have been presented by Yeh (1974). For an oblique compression wave of amplitude A_p (Fig. 3G-1a).

$$\sigma_a = \pm \frac{Ev_{mp}}{c_p} \cos^2\theta \qquad (3G\text{-}4)$$

$$\sigma_b = \pm \frac{ERa_{mp}}{c_p^{\,2}} \sin\theta \cos^2\theta \qquad (3G\text{-}5)$$

where

σ_a = maximum axial stress
σ_b = maximum bending stress
E = modulus of elasticity for the structure
v_{mp} = maximum compression wave particle velocity 6
a_{mp} = maximum compression wave particle acceleration
R = distance from the cross-sectional neutral axis of the structure to the extreme fiber
θ = angle of incidence of propagating wave from the structural axis

The maximum possible values of the axial and bending stresses due to an oblique compression wave are

$$\sigma_a = \pm \frac{Ev_{mp}}{c_p} \quad \text{for } \theta = 0° \qquad (3G\text{-}6)$$

$$\sigma_b = \pm 0.385 \frac{ERa_{mp}}{c_p^{\,2}} \quad \text{for } \theta = 35°16' \qquad (3G\text{-}7)$$

For an oblique shear wave of amplitude A_s (Fig. 3G-1b)

$$\sigma_a = \pm \frac{Ev_{ms}}{c_s} \sin\theta \cos\theta \qquad (3G\text{-}8)$$

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$$\sigma_b = \pm \frac{Era_{ms}}{C_s^2} \cos^3\theta \qquad (3G\text{-}9)$$

where

v_{ms} = maximum shear wave particle velocity
a_{ms} = maximum shear wave particle acceleration

The maximum possible values of the axial and bending stresses due to an oblique shear wave are

$$\sigma_a = \pm \frac{Ev_{ms}}{2C_s} \quad \text{for } \theta = 45° \qquad (3G\text{-}10)$$

$$\sigma_b = \pm \frac{ERa_{ms}}{C_s^2} \quad \text{for } \theta = 0° \qquad (3G\text{-}11)$$

For an <u>incident surface wave</u> of amplitude A_R, the motion is equivalent to the combination of a compression wave of amplitude A_{Rp} and a shear wave of amplitude A_{Rs} (Fig. 3G-1c) and

$$\sigma_a = \pm \frac{Ev_{mr}}{C_R} \cos^2\theta \qquad (3G\text{-}12)$$

$$\sigma_b = \pm \frac{ERa_{mr}}{C_R^2} \sin\theta \cos^2\theta \qquad (3G\text{-}13)$$

for the compressional component

$$\sigma_b = \pm \frac{ERa_{mr}}{C_R^2} \cos^2\theta \qquad (3G\text{-}14)$$

for the shear component

where

v_{mr} = maximum surface wave particle velocity
a_{mr} = maximum surface wave particle acceleration
C_R = surface wave propagation velocity

The maximum possible values of the axial and bending stresses due to an incident surface wave are

$$\sigma_a = \pm \frac{Ev_{mr}}{C_R} \qquad \text{for } \theta = 0° \qquad (3G\text{-}15)$$

$$\sigma_b = \pm\, 0.385 \frac{ERa_{mr}}{C_R{}^2} \quad \text{at } \theta = 36°16' \qquad (3G\text{-}16)$$

for the compressional component

$$\sigma_b = \pm \frac{ERa_{mr}}{C_R^2} \qquad \text{at } \theta = 0° \qquad (3G\text{-}17)$$

for the shear component

3G.2.3 WAVE TYPES AND COMBINATION OF STRESSES

The maximum ground velocity and acceleration for an earthquake motion contain contributions from compressional, shear, and surface waves. The choice of wave type to be used for design depends on the location and orientation of the structure to the earthquake source, as well as on the nature of the source and local geologic conditions along the travel path. 6

It is not presently possible, in general, to determine the relative contributions to the total motion of each of the various wave types. The axial and bending stresses should be maximized separately according to wave type and angle of incidence, and the resulting maximimums for axial and bending stress should be combined by the SRSS method since the maximum values are unlikely to occur simultaneously.

The calculated axial and bending stresses are combined to provide the total seismic design stress. The combined stress is maximized for an incident angle between 0° and 45? for each wave type using the equations provided in section 3G.2.2. This combined stress for each wave type will always be less than the sum of the maximum possible values of axial and

bending stress which are based on different angles of incidence and, therefore, do not occur simultaneously. The maximized values of axial and bending stress for each wave type are then combined using the SRSS method to give the total seismic design stress ($\sigma_a + \sigma_b$) as follows:

$$\sigma_a = \pm \left[(\sigma_{ap})^2 + (\sigma_{as})^2 + (\sigma_{ar})^2\right]^{1/2} \qquad (3G\text{-}18)$$

$$\sigma_b = \pm \left[(\sigma_{bp})^2 + (\sigma_{bs})^2 + (\sigma_{br})^2\right]^{1/2} \qquad (3G\text{-}19)$$

where the subscripts p, s, and r identify the maximum axial and bending stresses due to a compressional, shear, and surface wave, respectively.

For buried piping of relatively small diameter (less than about 48 inches), the bending stresses are small compared to the calculated normal stresses. In this case, the maximum possible values of axial and bending stress for each wave type can be added directly without performing the maximizing procedure prior to combining. For buried structures of much greater dimensions, such as tunnels, and bending stress will be significant compared to the axial component and the maximizing procedure should be carried out.

If the calculated stresses exceed the allowable stresses, increasing the cross-sectional area of the structure is of no value since the stresses are due to an imposed strain. In this case, the solution may be to either articulate the structure to make it more flexible or to isolate the structure partially or completely from the surrounding soil.

3G.2.4 SHORT SECTIONS

In case of a straight structural element embedded in soil, the transfer of soil strain as axial strain into the element depends on the end bearing of the element against the soil

and the frictional resistance between the element surface and the soil. At the ends of a long, straight element, frictional resistance will develop for some length (ℓ) along which the element will displace relative to the surrounding soil due to strain incompatibility between the soil and the element (Fig. 3G-2a). Neglecting end bearing, the minimum length of structure (L) required to develop full friction has been shown by Shah and Chu (1974) to be twice the maximum slippage length (ℓ_m) which is calculated as follows:

$$\ell_m = \frac{\varepsilon_m AE}{f} \qquad (3G\text{-}20)$$

where

ε_m = maximum soil strain
A = structure cross-sectional area
E = structure modulus of elasticity
f = friction force per unit length 6

For buried structures where $L < 2\ell_m$, the calculated axial stresses will be proportionately less than those calculated assuming no relative slippage between the structure and the soil (Fig. 3G-2b).

The frictional force (f) per unit length of a pipeline structure is given by

$$f = \pi D p_r \mu \qquad (3G\text{-}21)$$

where

D = pipe diameter
p_r = average radial soil pressure on pipe
μ = coefficient of friction

The average radial soil pressure on the pipe (p_r) is approximated by

$$p_r = \left(\frac{1 + K_o}{2}\right) \gamma_{soil} H_d \qquad (3G\text{-}22)$$

where

K_o = coefficient of lateral stress at rest
γ_{soil} = soil unit weight
H_d = burial depth at pipe centerline

The parameters (μ) and (K_o) are evaluated based on the type of structural material and soil conditions for a specific project. The coefficient of friction (μ) is typically in the range of 0.3 to 0.5 for a smooth pipe embedded in soil. The lateral stress coefficient (K_o) typically ranges from 0.5 to 1.0.

3G.2.5 AXIAL DISPLACEMENT OF FREE END RELATIVE TO THE SOIL

Neglecting the effect of end bearing and considering the maximum soil strain to remain constant over the length of the structure, Shah and Chu (1974) give the longitudinal displacement of the ends of a structure relative to the soil as follows:

$$\Delta = \varepsilon_m \ell_e - \frac{f \ell_e^2}{2AE} \qquad (3G\text{-}23)$$

where

Δ = Δ (soil) - Δ(structure)
ℓ_e = effective slippage length (Fig. 3G-2b)

In the case of a short structure where $L < 2\ell_m$, the effective slippage length equals one-half the total length ($\ell_e = L/2$) and

$$\Delta = \frac{\varepsilon_m L}{2} - \frac{f L^2}{8AE} \qquad (3G\text{-}24)$$

For a long structure, $\ell_e = \ell_m$ and

$$\Delta = \frac{\varepsilon_m \ell_m}{2} \quad \frac{\varepsilon_m^2 AE}{2f} \qquad (3G\text{-}25)$$

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Provisions should be made in the design to accommodate this displacement at the intersection of long elements with massive embedded structures.

3G.2.6 SHEAR FORCE DUE TO AN AXIAL SHEAR WAVE

The basic relations for maximum longitudinal strain and curvature presented by Hall and Newmark (1978) can be extended to provide the rate of change of curvature of a buried structure due to a propagating shear wave. For a shear wave propagating with wave velocity (C_s) along the x-axis, the particle displacement in the transverse (y) direction is

$$y = f(x - C_s t) \tag{3G-26}$$

The third derivative of equation (3G-26) with respect to x and t gives the following relation for the rate of change of curvature 6

$$\frac{\partial^3 y}{\partial x^3} = f'''(x-C_s t) = -\frac{1}{C_s{}^3}\left(\frac{\partial^3 y}{\partial t^3}\right) \tag{3G-27}$$

Defining (h) as the maximum derivative of the ground acceleration

$$h = \frac{\partial^3 y}{\partial t^3} \tag{3G-28}$$

and using the elementary beam relationship between the change in curvature and the shearing force (Q)

$$Q = -EI\left(\frac{\partial^3 y}{\partial x^3}\right) \tag{3G-29}$$

The shearing force in the buried structure is

$$Q = EIh/C_s{}^3 \tag{3G-30}$$

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The quantity (h) can be evaluated using the relationships between the maximum values of ground acceleration, displacement, and velocity where $a_m d_m/v_m^2 = 6$ (Hall and Newmark, 1978, Table 1) and $hv_m/a_m^2 = \beta a_m d_m/v_m^2$ (Newmark and Rosenblueth, 1971, p. 492). The coefficient (β) accounts for uncertainties in the relationship between the various ground motion parameters, with a reasonable level of conservatism obtained by taking $\beta = 1.5$. Based on these assumptions

$$h = 9a_m^2/v_m \tag{3G-31}$$

Combining equations (3G-30) and (3G-31) yields the following expression for the maximum shear force in the structure

$$Q = \frac{9EIa_m^2}{C_s^3 v_m} \tag{3G-32}$$

3G.2.7 CURVATURE

The maximum curvature (K_m) at a point can be calculated using Eq. (3G-3). If the calculated curvature is equal to or less than the allowable value of M/EI, the structure can be assumed to follow the ground motion without overstress and no articulation is necessary. However, some rotational capability may be required in sections where the calculated curvature exceeds the allowable value of M/EI and in the vicinity of connections to structures. The angular distortion for a given length of structure (L) can be calculated using the relation

$$\phi = K_m L \tag{3G-33}$$

If sections of an underground structure are effectively isolated from the surrounding component soil, the angular distortion is a function of the relative motion of the support points. The maximum relative motion in the transverse

direction between two points a distance (L) apart during an earthquake can be calculated according to Yeh (1974):

$$\Delta = \frac{V_{ms} L}{C_s} \qquad (3G\text{-}34)$$

The angular distortion is then

$$\phi = \arcsin \frac{\Delta}{L} \qquad (3G\text{-}35)$$

Sufficient rotational capability should be provided at joints and connections to permit the calculated angular distortion (ϕ) from the appropriate equation above.

3G.3 STRESSES AT BENDS

3G.3.1 GENERAL PROCEDURE

The analysis of buried structures with bends or restrained ends is based on the equations for beams on elastic foundations derived by Hetenyi (1946). In the case of a bend, the transverse leg is assumed to deform as a beam on an elastic foundation due to the axial force in the longitudinal leg (Fig. 3G-3). The displacement (Δ) at the bend is defined by the overall spring constant at the bend (K) where

$$K = \frac{P}{\Delta} \qquad (3G\text{-}36)$$

The spring constant at the bend depends on the stiffness of the longitudinal and transverse legs as well as the degree of fixity at the bend and at the far ends of the legs. The approximate deformed shapes for a number of typical combinations of leg stiffness and end condition are shown in Fig. 3G-5. The stiffness of the leg is classified according to Hetenyi (1946) as rigid ($\lambda L < \pi/4$), intermediate ($\pi/4 < \lambda L < \pi$), and flexible ($\lambda L > \pi$) where

λ = $\sqrt[4]{k/4EI}$ system characteristic

L = length of the leg

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k_s = modulus of subgrade reaction for structure of width (B)

k = k_s(B) where B is width of the structure

Solutions for the bend spring constant (K) for some typical configurations (cases A through E) are shown in table 3G-1. Solutions for other configurations can be derived using the appropriate equations for beams on elastic foundations.

3G.3.2 EQUATIONS FOR STRUCTURE WITH RESTRAINED END

The configuration and deformed shape of a buried structure with a bend are shown in Fig. 3G-4. According to Shah and Chu (1974), the maximum axial force is

$$F_{max} = Q + f\ell_e \tag{3G-37}$$

and

$$\Delta = \frac{Q}{K} \tag{3G-38}$$

Establishing displacement compatibility at the bend leads to the following expression:

$$\frac{F\ell_e^2}{2AE} + \ell_e \left(\frac{f}{K} - \frac{F_{max}}{AE} + \varepsilon_m\right) - \frac{F_{max}}{K} = 0 \tag{3G-39}$$

If the structure is long (L_1 or $L_2 > \ell_e + \ell_m$), $F_{max} = \varepsilon_m AE$ and equation (3G-39) reduces to

$$\ell_e = \frac{AE}{K}\left(\sqrt{1.0 + \frac{2\varepsilon_m K}{f}} - 1.0\right) \tag{3G-40}$$

In the case of a short structure (L_1 or $L_2 < \ell_e + \ell_m$), F_{max} = $f(L - \ell_e)$ and equation (3G-39) can be written in the form

$$\frac{f\ell_e^2}{2AE} + \ell_e \left[\frac{f}{K} - \frac{f(L - \ell_e)}{AE} + \varepsilon_m\right] - \frac{f(L - \ell_e)}{K} = 0 \tag{3G-41}$$

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Equation (3G-41) can be solved by trial and error for the effective slippage length (ℓ_e). Having the effective slippage length, the displacement (Δ) at the bend can then be calculated. With the displacement (Δ), the shear (Q) and moment (M) in the transverse leg can then be calculated for the appropriate configuration (cases A through E) in Table 3G-1. More complicated cases can be handled by discretizing the structure as described by Hindy and Novak (1978).

3G.4 STRESSES AT CONNECTIONS TO BUILDINGS

3G.4.1 AXIAL MOVEMENT

Stresses are induced in buried structures at penetrations to buildings due to relative movement between the building and the soil. In the case of relative movement in the axial direction of an underground structure with the far end unrestrained, the maximum axial force (P) in a long structure ($L > \ell_e$) is given by Yeh (1974):

$$P = \sqrt{2EAf\Delta_x} \qquad (3G\text{-}42)$$

where

Δ_x = relative movement between the building and soil in the axial direction.

ℓ_e = P/f effective slippage length

For a short structure ($L<\ell_e$), the maximum axial force is limited to

$$P = fL \qquad (3G\text{-}43)$$

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Table 3G-1
BEND CHARACTERISTICS

Case	Spring Constant At Bend (K)	Shear In Transverse Leg (Q)	Moment In Transverse Leg (M)	Remarks
(A)	$K = \frac{k}{2\lambda}$	$Q = \frac{k\Delta}{2\lambda}$	$M = \frac{0.1662 k\Delta}{\lambda^2}$ @$X = \frac{\pi}{4\lambda}$	
(B)	$K = \frac{3k}{4\lambda}$	$Q = \frac{3k\Delta}{4\lambda}$	$M = \frac{k\Delta}{4\lambda^2}$	Equal moment of inertia (I) in longitudinal and transverse legs
(C)	$K = \frac{k}{\lambda}$	$Q = \frac{k\Delta}{\lambda}$	$M = \frac{k\Delta}{2\lambda^2}$	
(D)	$K = \frac{k}{\lambda C_1}$	$Q = \frac{k\Delta}{\lambda C_1}$	$M = \frac{k\Delta C_2}{\lambda^2 C_1}$	$C_1 = \frac{\sinh(2\lambda L) - \sin(2\lambda L)}{\cosh^2(\lambda L) + \cos^2(\lambda L)}$ $C_2 = \frac{\sinh(\lambda L)\cos(\lambda L) + \cosh(\lambda L)\sin(\lambda L)}{\cosh^2(\lambda L) + \cos^2(\lambda L)}$
(E)	$K = \frac{k}{\lambda\left(2C_1 - \frac{C_2^2}{C_3}\right)}$	$Q = \frac{k\Delta}{\lambda\left(2C_1 - \frac{C_2^2}{C_3}\right)}$	$M = \frac{k\Delta C_2}{2\lambda^2\left(2C_1C_3 - C_2^2\right)}$	$C_1 = \frac{\sinh(\lambda L)\cosh(\lambda L) - \sin(\lambda L)\cos(\lambda L)}{\sinh^2(\lambda L) - \sin^2(\lambda L)}$ $C_2 = \frac{\sinh^2(\lambda L) + \sin^2(\lambda L)}{\sinh^2(\lambda L) - \sin^2(\lambda L)}$ $C_3 = \frac{\sinh(\lambda L)\cosh(\lambda L) + \sin(\lambda L)\cos(\lambda L)}{\sinh^2(\lambda L) - \sin^2(\lambda L)}$

Note: See Fig. 3G-4 for definition of cases.

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If a bend in the underground structure is located near the penetration, the connection to the building will be influenced by this restraint. In this case, the following expression can be solved for the maximum axial force (F_{max}):

$$\Delta_x = \frac{F_{max}L}{AE} - \frac{fL^2}{2AE} + \frac{F_{max}}{K} - \frac{fL}{K} \qquad (3G\text{-}44)$$

where K is evaluated for the appropriate configuration (Fig. 3G-4).

3G.4.2 LATERAL MOVEMENT

In the case of relative movement between the building and soil in the direction transverse to the buried structure, stresses are determined assuming the structure to be a semi-infinite beam supported on an elastic foundation with a fixed or hinged end at the connection to the building (Yeh, 1974). For a fixed connection to the building:

$$\sigma_b = \pm \frac{KR}{2\lambda^2 I} (\Delta_Y) \qquad (3G\text{-}45)$$

$$\tau = \frac{\alpha K}{\lambda A} (\Delta_y) \qquad (3G\text{-}46)$$

where

σb = maximum bending stress at the connection

τ = maximum shear stress at the connection

Δ_y = relative movement between the building and soil in the transverse direction

α = shape factor for the structural cross section and is equal to 2 for a thin circular section

For a hinged connection to the building:

$$\sigma_b = \pm\ 0.161\ \frac{kR}{\lambda^2 I}\ (\Delta_y) \qquad (3G\text{-}47)$$

$$\tau = \frac{\alpha k}{2\lambda A}\ (\Delta_y) \qquad (3G\text{-}48)$$

where

σ_b = maximum bending stress located at a distance $\pi/4\lambda$ from the connection

τ = maximum shear stress at the connection

3G.5 DESIGN EXAMPLE

Given An underground steel pipeline connecting two buildings as shown in plan view in Fig. 3G-6. The properties of the pipe and supporting soil, and the earthquake motion are as follows:

Pipe	Soil	Earthquake
30-inch I.D.	γ_{soil} = 118 pcf	a_m = 120 in./sec^2
t = 3/8-inch	C_p = 7500 fps	v_{mp} = 5 in./sec
I = 4130 in.^{4}	$C_s = C_R$ = 3000 fps	$v_{ms} = v_{mr}$ = 14 in./sec
A = 35.8 in.^{2}	K_o = 0.7	
E = 30x10^6 psi	H_d = 6.0 ft.	
L_1 = 500 ft.	μ = 0.4	
L_2 = 100 ft.	k_s = 98 lb/in.^{3}	

Find

A. Seismic design stresses in the straight sections of the pipeline away from the bend and connections to the buildings.

B. Design condition at the bend, including

- stresses in the pipeline if restrained at the bend
- maximum axial displacement of the ends of the pipeline if unrestrained at the bend
- maximum angular distortion at the bend.

C. Design condition at the building connection, including

- stresses in the pipeline assuming a hinged or fixed connection and 0.5 in. relative movement in the axial or lateral direction
- maximum axial displacement of the ends of the pipeline at the connections assuming no restraint
- maximum angular distortion at the connections

Solution

Since the pipeline is of relatively small diameter, the maximum values of axial and bending stress for each wave type will be added directly without maximizing for angle of incidence as discussed in section 3G.2.3. The maximum axial and bending stresses due to individual compression, shear, and surface waves for a pipeline following the ground motion are determined from the appropriate equations of section 3G.2.2:

Compression wave

$$\sigma_a = \pm \frac{Ev_{mp}}{C_p} = \pm\ 1667\ \text{psi}$$

$$\sigma_b = \pm \frac{0.385\ ERa_{mp}}{C_p^{\ 2}} = \pm\ 2.6\ \text{psi}$$

Shear wave

$$\sigma_a = \pm \frac{EV_{ms}}{2C_s} = \pm\ 5833\ \text{psi}$$

$$\sigma_b = \pm \frac{ERa_{ms}}{C_s^{\ 2}} = \pm\ 42.7\ \text{psi}$$

Surface wave

$$\sigma_a = \pm \frac{Ev_{mr}}{C_R} = \pm\ 11{,}667\ \text{psi}$$

$$\sigma_b = \pm \frac{ERa_{mr}}{C_R^{\ 2}} = \pm\ 42.7\ \text{psi}$$

(a) Seismic design stresses in the long, straight section (L_1) are obtained from stresses for the individual wave types using the SRSS method:

$$\sigma_a = \pm \left[(1{,}667)^2 + (5{,}833)^2 + (11{,}667)^2 \right]^{1/2}$$

$$= \pm\ 13{,}100\ \text{psi}$$

$$\sigma_b = \pm \left[(2.6)^2 + (42.7)^2 + (42.7)^2 \right]^{1/2} = \pm\ 60\ \text{psi}$$

Design stresses in the shorter section (L_2) can be reduced to account for slippage between the pipeline and the soil as discussed in section 3G.2.4:

$$\varepsilon_m = \frac{\sigma_a + \sigma_b}{E} \quad \frac{13{,}096 + 60}{30 \times 10^6} = 0.438 \times 10^{-3}$$

$$p_r = \left(\frac{1 + K_o}{2}\right) \gamma_{soil} \ (H_d) = 602 \text{ psf}$$

$$f = \pi D p_r \mu = 1935 \text{ lb/ft}$$

$$\ell_m = \frac{\varepsilon_m AE}{f} = 243 \text{ ft.}$$

For the shorter section, $L_2 < 2\ell_m$ and the seismic design stresses can be reduced in accordance with Fig. 3G-2b:

$$\sigma_a = \frac{f}{A}\left(\frac{L_2}{2}\right) = 2{,}700 \text{ psi}$$

(b) If the pipeline is restrained at the bend by a rigid elbow or other structure, shear and bending stresses will be induced in the pipeline in addition to an axial stress as discussed in section 3G.3. For displacement at the bend in the east-west direction (axial force in L_1):

$$\lambda = \left[\frac{k_s B}{4EI}\right]^{1/4} = 8.83 \times 10^{-3}$$

Both pipeline sections (L_1 and L_2) can be considered as infinitely long for purposes of calculating the spring constant at the bend since both λL_1 and $\lambda L_2 > \pi$.

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The appropriate spring constant at the bend is case (B) of Table 3G-1:

$$K = \frac{3k}{4\lambda} = 2.56 \times 10^5 \text{ lb/in.}$$

The effective slippage length along section L_1 is calculated from Eq. (3G-40):

$$\ell_e = \frac{AE}{K}\left(\sqrt{1.0 + \frac{2\varepsilon_m K}{f}} - 1.0\right) = 2291 \text{ in.}$$

The shear, moment, and transverse displacement induced in section L_2 at the bend are:

$$Q_2 = F_{max} - f\ell_e = \varepsilon_m AE - f\ell_e = 101{,}000 \text{ lb}$$

$$\Delta_2 = \frac{Q_2}{K} = 0.39 \text{ in.}$$

$$M_2 = \frac{k\,\Delta_2}{4\lambda^2} = 3.77 \times 10^6 \text{ in. lb}$$

For displacement at the bend in the north-south direction (axial force in L_2), one-half the section length (600 in.) is less than the effective slippage length (ℓ_e = 2291 in.) calculated using Eq. (3G-40). In this case, Eq. (3G-41) must be solved for the effective slippage length by trial and error:

$$\ell_e = 470 \text{ in.}$$

The shear, moment, and transverse displacement induced in section L_1 at the bend are:

$$Q_1 = F_{max} - f\ell_e = f(L_2-\ell_e) - f\ell_e = 41,860 \text{ lb}$$

$$\Delta_1 = \frac{Q_1}{K} = 0.16 \text{ in.}$$

$$M_1 = \frac{k\Delta_1}{4\lambda^2} = 1.55 \times 10^6 \text{ in. lb}$$

If the pipeline is not restrained at the bend, the longitudinal displacement of the ends relative to the soil can be calculated by Eqs. (3G-23) and (3G-25). The displacement of the end of the long section (L_1) is

$$\Delta_1 = \frac{\varepsilon_m{}^2 AE}{2f} = 0.64 \text{ in.}$$

The displacement of the end of the short section (L_2) is

$$\Delta_2 = \frac{\varepsilon_m L_2}{2} - \frac{fL_2^2}{8AE} = 0.24 \text{ in.}$$

The angular distortion of the pipeline can be calculated by Eq. (3G-33):

$$\phi = K_m (L) = \frac{a_m (L)}{C_s{}^2}$$

For the long section (L_1), $\phi_1 = 0.03$ deg.; for the shorter section (L_2), $\phi_2 = 0.01$ deg.

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(c) Connection at Point (A)

The axial force induced in the longer section (L_1) due to the design axial relative movement (Δ_x) of 0.5 in. is:

$$P_1 = \sqrt{2EAf\Delta_x} = 415{,}800 \text{ lb.}$$

for a long structure where L_1 = 500 ft. > $\frac{P}{f}$ = 215 ft.

Assuming a fixed connection, the maximum bending and shear stresses in the pipeline at the connection due to the design lateral relative movement (Δ_x) of 0.5 in. are:

$$\sigma_b = \pm \frac{kR}{2\lambda^2 I} (\Delta_y) = \pm 35{,}100 \text{ psi}$$

$$\tau = \frac{\alpha k}{\lambda A} (\Delta_y) = 9{,}500 \text{ psi}$$

Assuming a hinged connection, the maximum bending and shear stresses are:

$$\sigma_b = \pm \frac{0.161kR}{\lambda^2 I} (\Delta_y) = \pm 11{,}300 \text{ psi at a}$$

distance $\frac{\pi}{4\lambda}$ = 89 in. from the connection and

$$\tau = \frac{\alpha k}{2\lambda A} (\Delta_y) = 4{,}800 \text{ psi}$$

Connection at Point (C)

For the shorter section, L_2 = 100 ft. < $\frac{P}{f}$ = 215 ft. and Eq. (6-44) can be solved to obtain the maximum axial force due to the design axial relative displacement (Δ_x) of 0.5 in.:

$$F_{max} = 271{,}000 \text{ lb.}$$

The maximum bending and shear stresses in the pipeline at the connection due to the design lateral relative movement (Δ_y) of 0.5 in. are the same as for the connection at point (A).

Angular Distortion

The design angular distortion at the connections is the same as for the bend:

$$\phi_1 = 0.03 \text{ deg.}$$

$$\phi_2 = 0.01 \text{ deg.}$$

The calculated seismic stresses and displacements at various locations along the pipeline must be combined with stresses due to all other loading conditions to obtain total design stresses (Goodling, 1978). If the total calculated stresses exceed the allowable stresses, the overstressed section can be made more flexible or isolated partially or completely from the surrounding soil. In the vicinity of the connections to the buildings, for example, a fixed connection would result in very high bending stresses which could be greatly reduced by use of a hinged connection.

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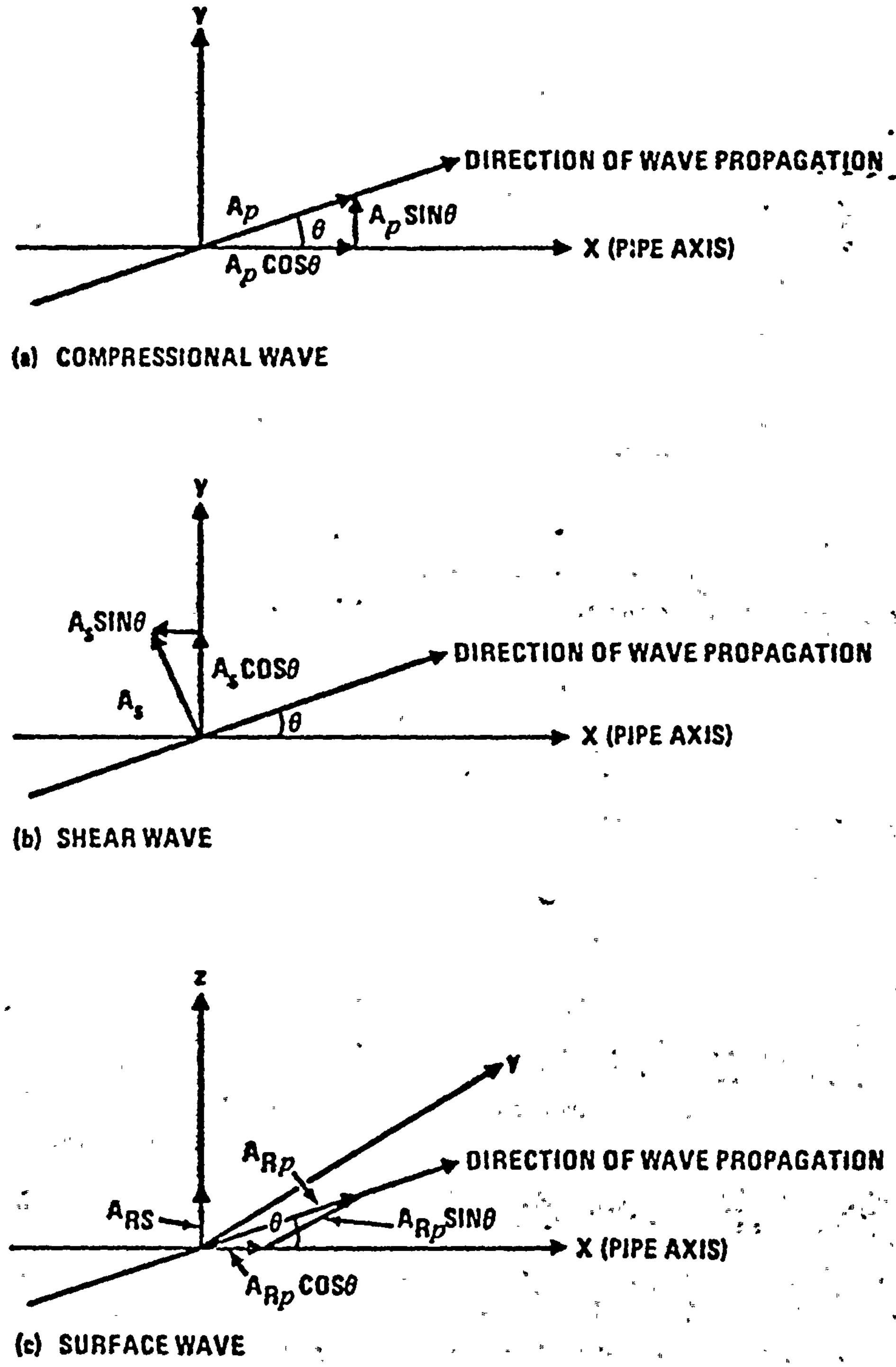

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AMPLITUDES OF COMPRESSIONAL, SHEAR AND SURFACE WAVES
Figure 3G-1

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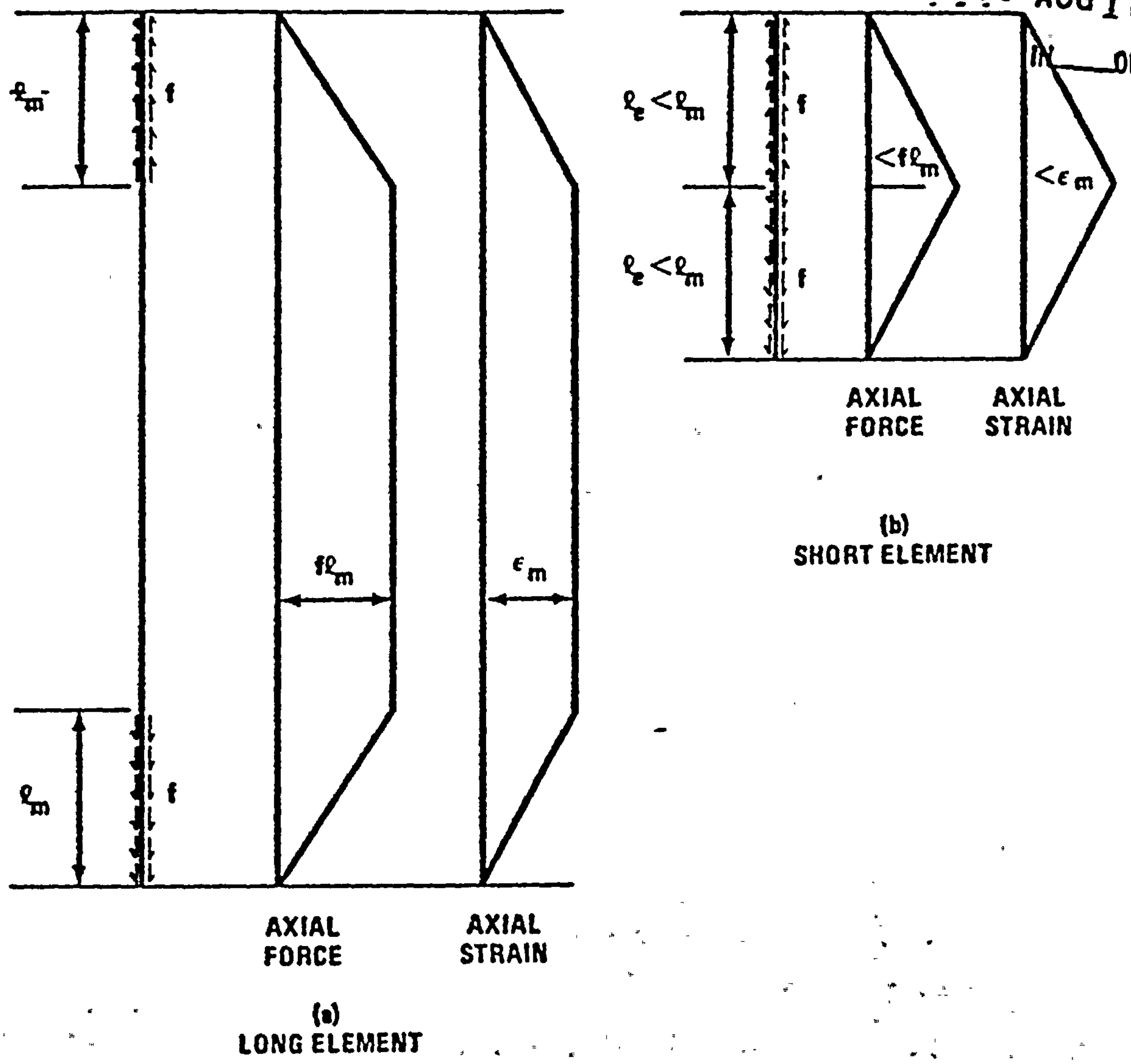

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AXIAL FORCE AND STRAIN IN
IN STRAIGHT ELEMENT
Figure 3G-2

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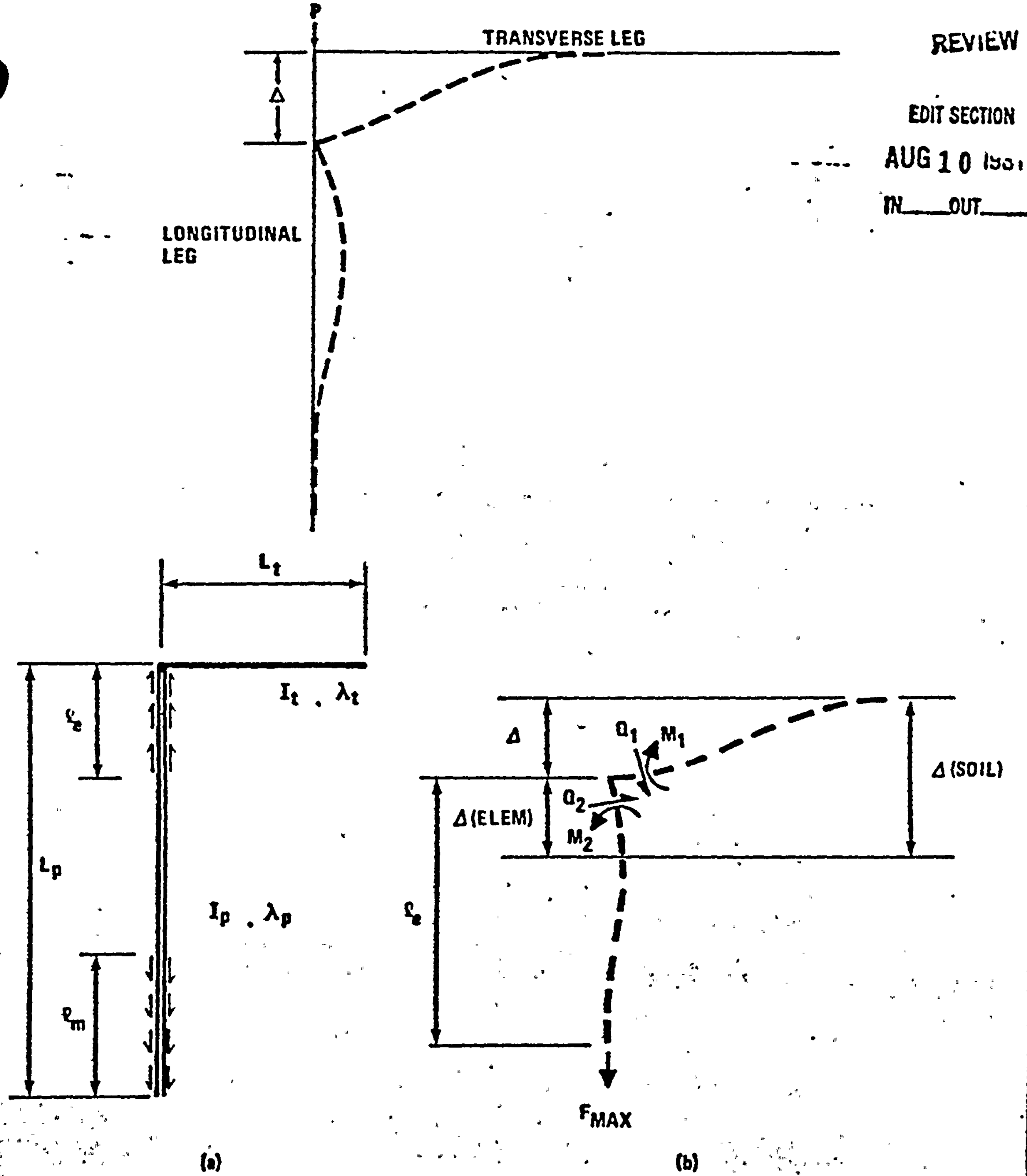

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ELEMENT WITH BEND AND DEFORMATIONS AT A BEND
Figure 3G-3

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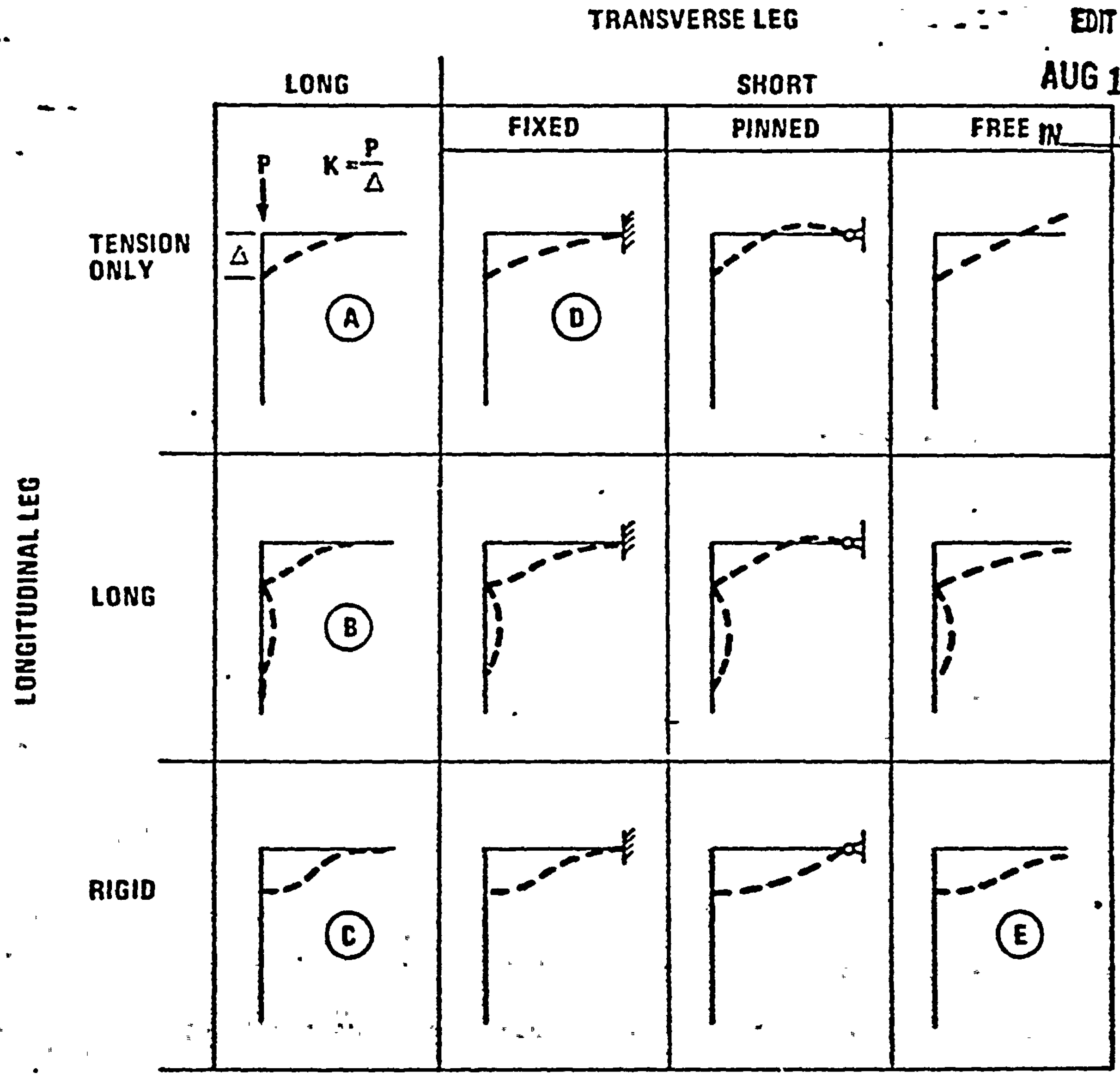

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BEND CONFIGURATIONS
Figure 3G-4

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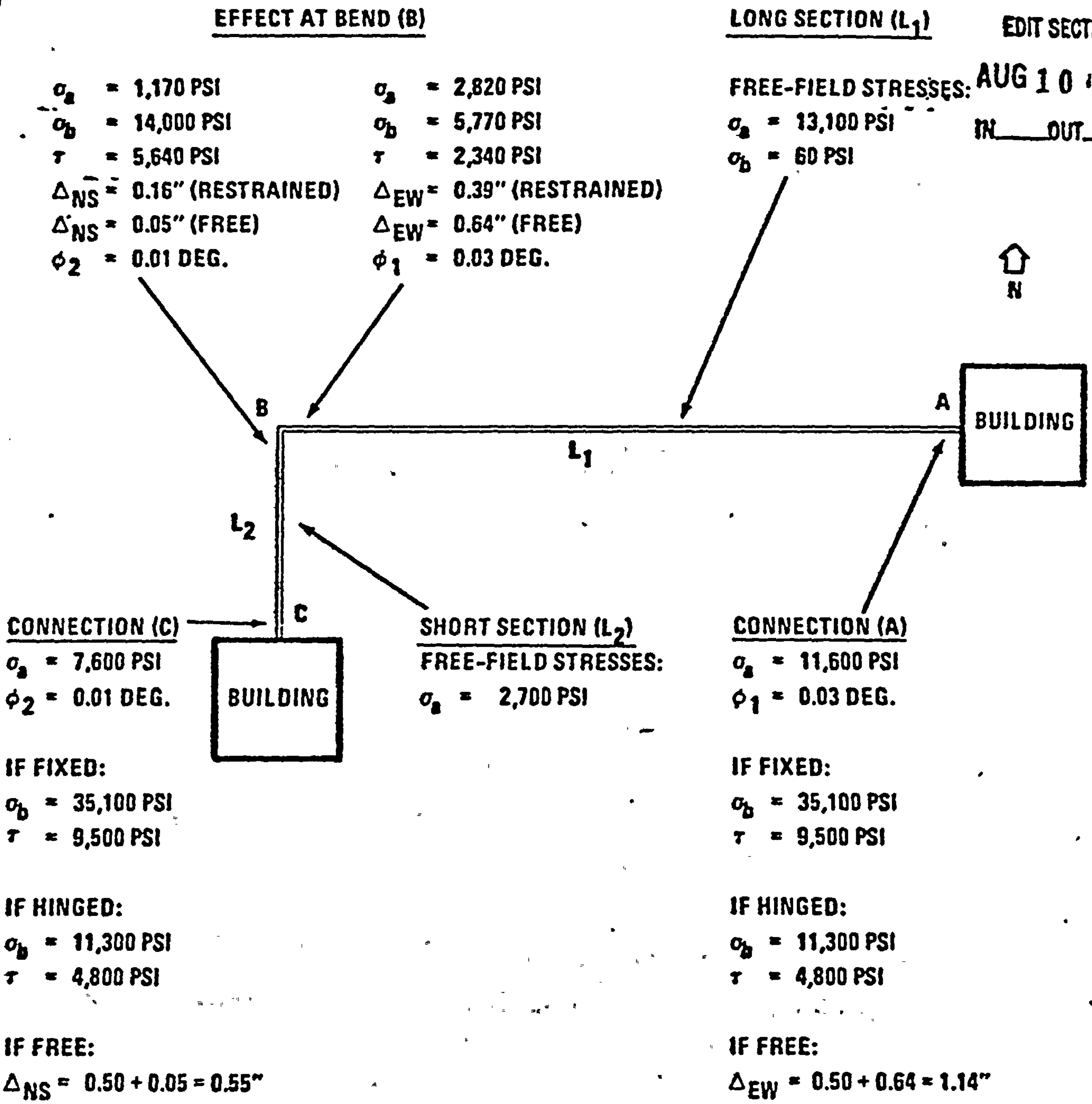

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DESIGN EXAMPLE
Figure 3G-5

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ATTACHMENT D

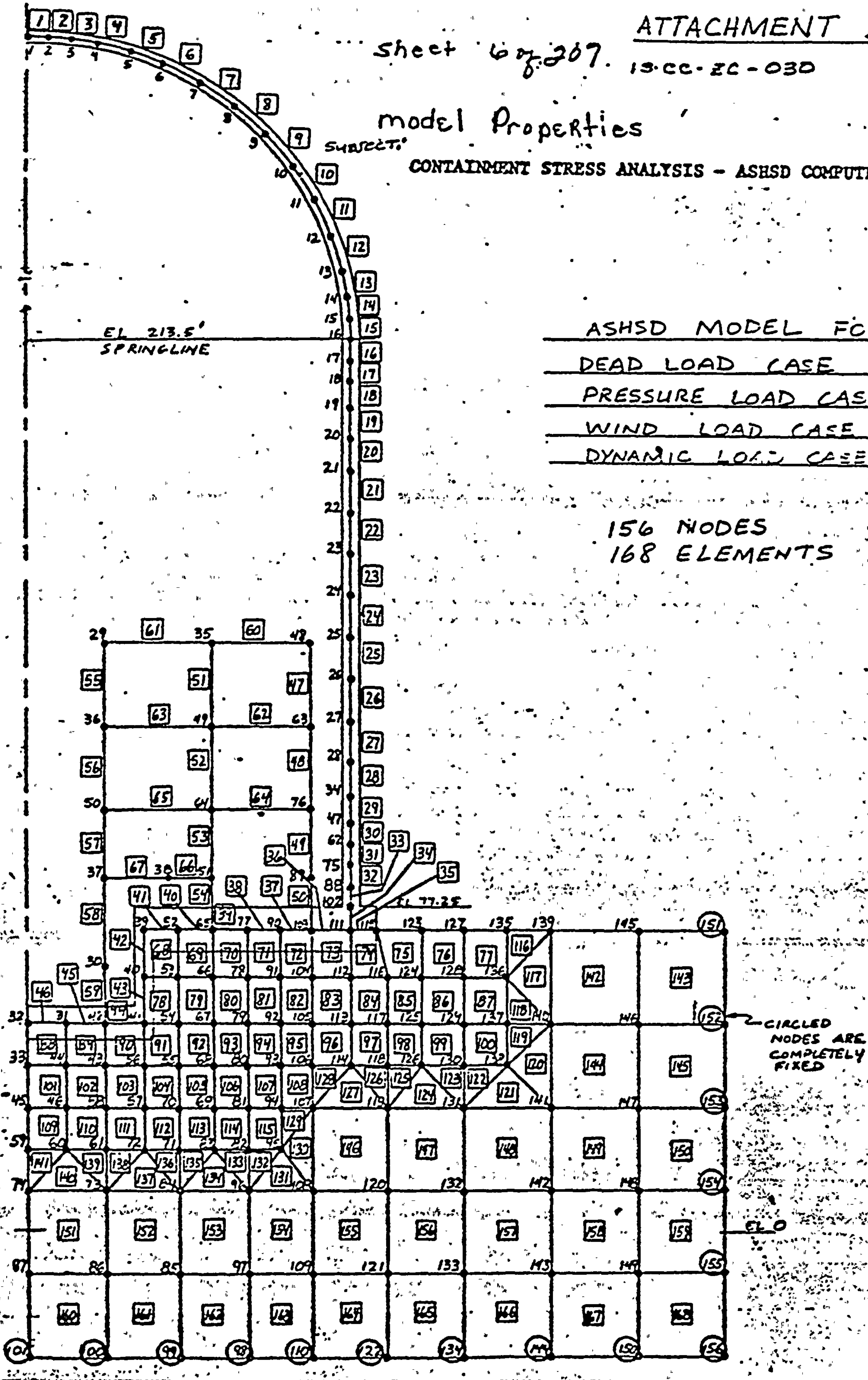
sheet 6 of 207. 13-CC-ZC-030

model Properties

SUBJECT: CONTAINMENT STRESS ANALYSIS - ASHSD COMPUTER ANALY

ASHSD MODEL FOR
DEAD LOAD CASE
PRESSURE LOAD CASE
WIND LOAD CASE
DYNAMIC LOAD CASE

156 NODES
168 ELEMENTS



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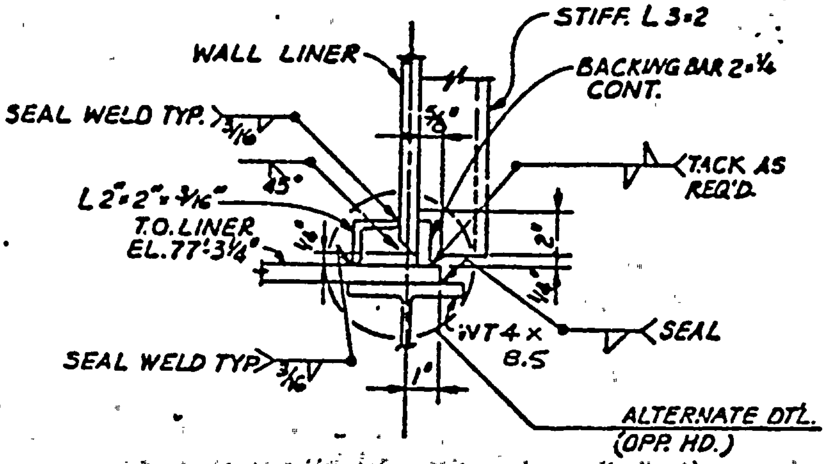
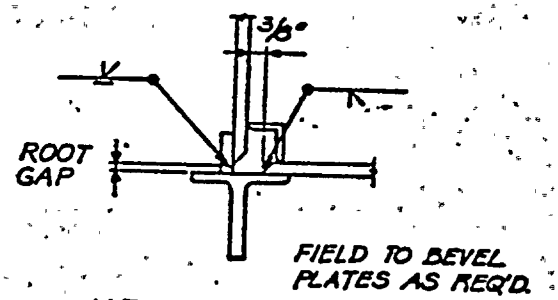
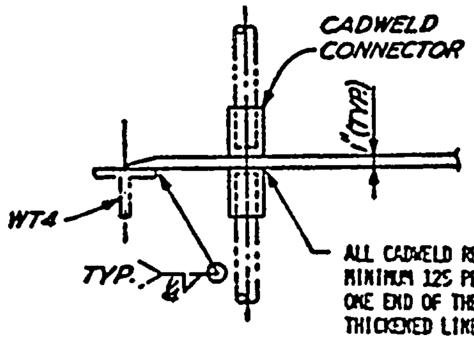




ALTERNATE
DETAIL

LINER PLATE CYLINDER - BASE MAT JUNCTION

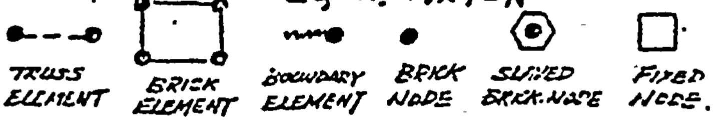


ALL CADWELD REBAR SPLICING SHALL BE "B" SERIES WITH MINIMUM 125 PERCENT YIELD STRENGTH OF THE REBAR SPLICED. ONE END OF THE SLEEVE SHALL HAVE J-GROOVE TO WELD WITH THICKENED LINER PLATE. ORIENTATION OF TAP HOLE ON CADWELD SLEEVES SHALL BE SUBMITTED BY FABRICATOR FOR ENGINEER'S APPROVAL.

ANCHORAGE BETWEEN LINER AND INTERIOR CONCRETE

SIG. M. Heu DATE 9/1/77 CHECKED SFJ[illegible] DATE [illegible]
PROJ. PVNGS JOB NO [illegible]
SUBJ. STRESS ANALYSIS AND REINF. DESIGN AROUND EQUIP. HATCH SHEET 9a OF 238
CALC NO. 13-CC-ZC-040

SYMB	TYPE OF NODE	DOF	NO. OF NODES	TOTAL DOF
●	BRICK	3	1640	4920
⬡	BRICK-BRICK INTERFACE COUPLING	0	30	60
□	BRICK FIXED	0	80	0



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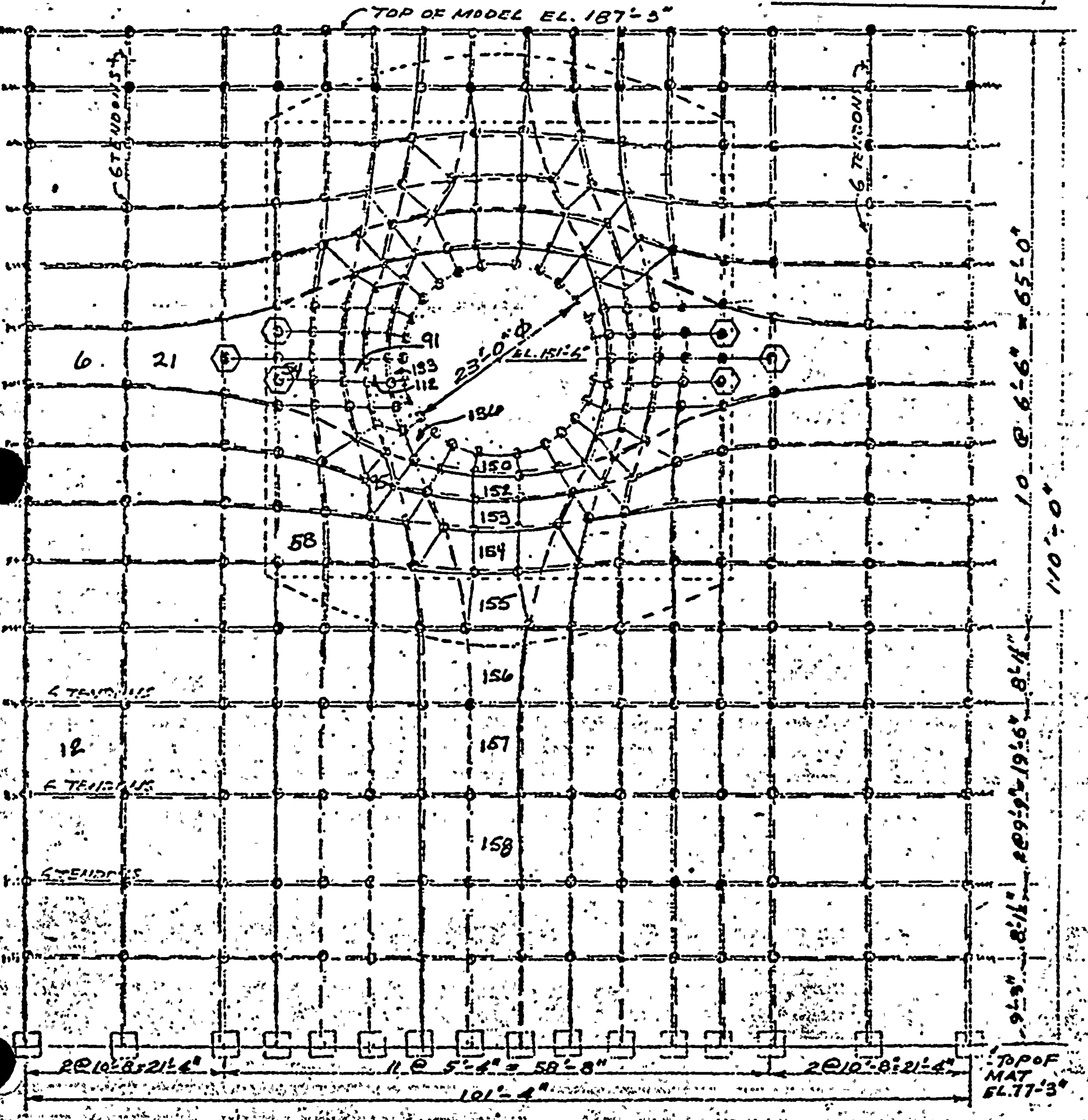
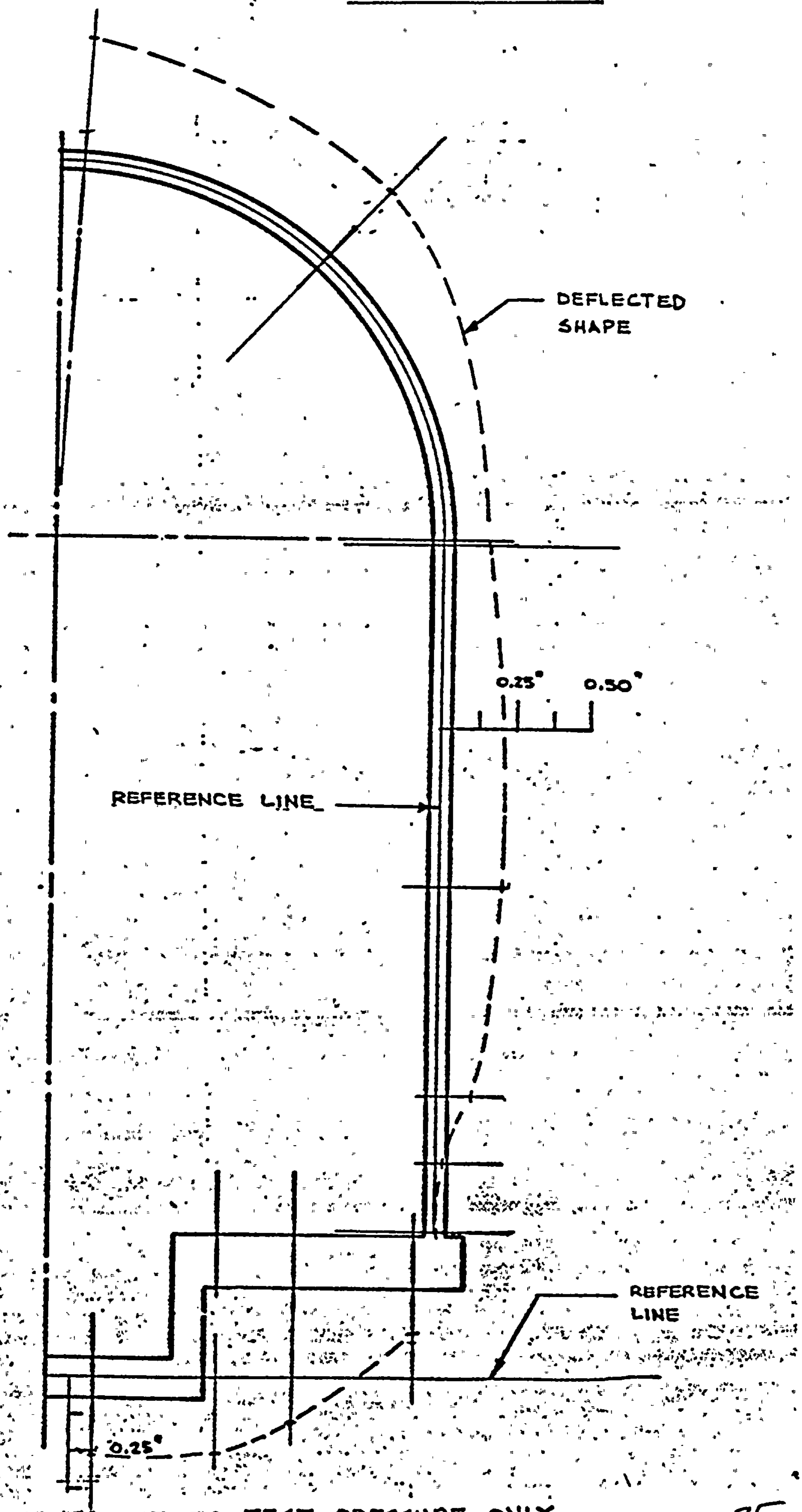


FIG. 2 DEVELOPED ELEVATION OF MATHEMATICAL MODEL OF EQUIPMENT HATCH OPENING

1. DEVELOPED AT OUTSIDE RADIUS OF CONTAINMENT R=77'-0"
2. EACH HORIZONTAL TRUSS ELEMENT REPRESENTS 4 HORIZONTAL TENDONS UNLESS NOTED OTHERWISE
3. EACH VERTICAL TRUSS ELEMENT REPRESENTS 2 VERTICAL TENDONS UNLESS NOTED OTHERWISE

ATTACHMENT H



PREDICTED DISPLACEMENTS DUE TO TEST PRESSURE ONLY

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APPENDIX 3F 6

TANGENTIAL AND RADIAL SHEAR

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TANGENTIAL AND RADIAL SHEAR

3F.1 GENERAL

In this appendix tangential shear stresses are evaluated in detail. In the following subsections the critical loading combinations included in this Appendix are listed below:

RLC #1: $D + F_i + P_t + T_t$

RLC #2: $D + F_i + T_t$

RLC #3: $D + F + P_v + T_o$

RLC #4: $D + F + E_o + T_o$

RLC #11: $D + F + E_{ss} + T_o$

RLC #15: $D + F + 1.5\ P_a + T_a$

RLC #18: $D + F + 1.25\ P_a + 1.25\ E_o + T_a$

RLC #24: $D + F + P_a + E_{ss} + T_a$ 6

3F.2 TANGENTIAL SHEAR

There are no criteria in the Code for tangential shear in prestressed concrete containments. In the following paragraphs the tangential shear is evaluated using Bechtel's criteria.

A. Containment sections and governing loading combinations:

The only loading that incuces significant tangential shear (in-plane shear) in the structure is seismic loading. Also, tangential shear may be significant only in the shell. Furthermore, the effect of tangential shear is more significant when it occurs simultaneously with the internal pressure (postulated LOCA) because the shear capacity of a section decreases with reduced membrane compression, and internal pressure tends to reduce membrane compression due to prestressing and dead load.

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For these reasons it will be sufficient to consider only three reference loading combinations. These combinations are: RLC Nos. 11, 18, and 24.

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B. Section resultants:

Horizontal and vertical membrane forces (N_h and N_v) due to all loads other than earthquake are obtained from the FINEL analysis and are shown in table 3F.2-1.

Horizontal and vertical membrane forces (N_{he} and N_{ve}) and tangential shear (V_u) due to earthquake loads are also shown in table 3F.2-1.

C. Maximum applied shear:

Maximum applied tangential shear must not exceed $v_u \leq 8.5bt \sqrt{f_c'} \cong 379$ k/ft. In the above equation b = width (12 inches). t = thickness (48 inches), f_c' = concrete strength (6000 psi). This limit is based on the ACI 318-77 code.

Tangential shear is less than the maximum

Table 3F.2-1 shows that the maximum allowable for each section.

D. Shear carried by concrete:

If the section is under biaxial compression, the concrete is allowed to resist the following shear.

$$V_c = \left[(N_h + N_{he}) (N_v + N_{ve})\right]^{1/2}$$

Assuming, conservatively, that the maximum membrane forces and tangential shear due to seismic loads occur at the same point, the concrete allowable shear force, V_c, is calculated using the above equation. These values are also shown in table 3F.2-1 for the given loading combinations.

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E. Evaluation of results:

Table 3F.2-1 shows that V_u exceeds V_c in only three cases. In accordance with the criteria 3F.1, whenever $V_u > V_c$, it is assumed that the shear carried by concrete is equal to zero. Thus, these three cases need further analysis as shown in the following paragraphs.

F. Further analysis of section with $V_c = 0$:

In this case a total equivalent membrane force is defined as follows:

$$N_{ht} = N_h + \left[N_{he}^2 + V_u^2\right]^{1/2}$$

$$N_{vt} = N_v + \left[N_{ve}^2 + V_u^2\right]^{1/2}$$

The section is then analyzed using these equivalent membrane forces and corresponding bending moments (table 3F.2-2). Results of these analyses are given in table 3F.2-3.

G. Final results:

Table 3F.2-3 shows that, in the three cases where concrete shear capacity may, conservatively, be assumed to be zero, the resulting concrete and reinforcement stresses are within the allowable limits.

Thus, it is shown that, considering the effects of tangential shear, all the sections are adequate. 6

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Table 3F.2-1

MEMBRANE FORCES AND TANGENTIAL SHEAR IN THE SHELL
(Sheet 1 of 2)

Section	Ref. Loading Comb.	Hoop		Meridional				Remarks
		N_h k/ft	N_{he} k/ft	N_v k/ft	N_{ve} k/ft	V_c k/ft	V_u k/ft	
	11	-469	22	-585	11	507	17	
7	18	-88	16	-200	7	118	10	
	24	-164	22	-277	11	194	17	
	11	-703	40	-578	35	600	56	
16	18	-4	30	-194	25	86	34	
	24	-200	40	-271	35	194	56	
	11	-765	17	-604	120	602	101	
18	18	24	13	-220	89	0	61	See note 7
	24	-134	17	-297	120	144	101	
	11	-516	15	-630	200	464	122	
20	18	-76	10	-246	148	80	74	
	24	-154	15	-323	200	131	122	
	11	-259	26	-636	217	312	121	
21	18	-149	17	-253	161	110	73	
	24	-149	26	-329	217	117	121	See note 7
	11	-221	36	-639	226	276	118	
22	18	-162	26	-255	167	109	72	
	24	-135	36	-332	226	102	118	See note 7

<u>Notes</u>:

1. Notation:

   N_h, N_v = hoop and meridional membrane forces due to other loads

   N_{he}, N_{ve} = hoop and meridional membrane forces due to seismic loads

   V_c = shear carried by concrete alone

   V_u = applied tangential shear

2. V_c is zero if either or both total membrane forces are positive (tension)

3. N_h, N_v are from FINEL analysis

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Table 3F.2-1

MEMBRANE FORCES AND TANGENTIAL SHEAR IN THE SHELL
(Sheet 2 of 2)

4. N_{he}, N_{ve}, V_u are from ASHSD analysis.

5. Whenever $V_c > V_u$, concrete shear capacity alone is adequate to carry tangential shear. 6

6. In all cases the calculated tangential shear force, V_u, is less than the total allowable shear on the section, 369 k/ft.

7. In these cases $V_u > V_c$ and therefore further analysis is required considering tangential shear contribution to seismic membrane forces.

Table 3F.2-2[a]

AXIAL FORCE - MOMENT SETS WITH DUE CONSIDERATION TO TANGENTIAL SHEAR

Loading Combination	Section	Primary				Primary + Secondary			
		Meridional		Hoop		Meridional		Hoop	
		Axial	Moment	Axial	Moment	Axial	Moment	Axial	Moment
$D + F + 1.25\ P_a + 1.25\ E_o + T_a$	18	-112	-3	86	7	-112	430	91	346
$D + F + P_a + E_{ss} + T_a$	21	-81	-133	-25	-20	-81	424	-187	629
$D + F + P_a + E_{ss} + T_a$	22	-77	-398	-12	-67	-77	65	-224	618

Notes:

(a) Sign conventions are:

Axial forces (kips) . . . . . (+) tension . . . . (-) compression

Moments (ft-kips) . . . . . . (+) tension on outside face . . . (-) compression on outside face

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Table 3F.2-3

STRESS ANALYSIS RESULTS

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BLE 3F.2-3. STRESS ANALYSIS RESULTS

		Concrete Stresses								Reinforcement Stresses									
		Meridional				Hoop				Meridional				Hoop				Liner Strains[b]	
		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary			
ference oading bination	Sec-tion	MEM psi	MEM & BEN psi	MEM psi	MEM & BEN psi	MEM psi	MEM & BEN psi	MEM psi	MEM & BEN psi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Meridional x 10^{-6} in/in	Hoop x 10^{-6} in/in
llowable	Shell	-3600	-4500	-4500	-5100	-3600	-4500	-4500	-510(	±54	±54	±54	±54	±54	±54	±54	±54	±10000	±10000
18[c]	18	-194	-226	-194	-2696	(d)	(d)	(d)	-1519	-1.0	-1.2	-1.6	39.3	26.0	10.3	5.4	42.5	-519	-326
24[c]	21	-141	-710	-141	-3138	-43	-86	-325	-273:	3.3	-2.7	-1.8	54.0[e]	-.1	-.4	-4.8	28.6	-932	-640
24[c]	22	-134	-2685	-134	-303	-21	-373	-389	-267:	46.4	-4.9	-1.3	-.1	4.4	-1.1	-5.7	24.3	2317	-623

tes:

.) Sign convention:
Stress and strains . . . . (+) tensile . . . . (-) compressive
) Allowable liner strains shown are based on the lowest values from the ASME Code, Section III, division 2.
:) The stresses were obtained from OPTCON computer output.
l) A completely cracked section
) Reinforcement is assumed to yield at 54 ksi, the calculated strain is .00208 in./in.

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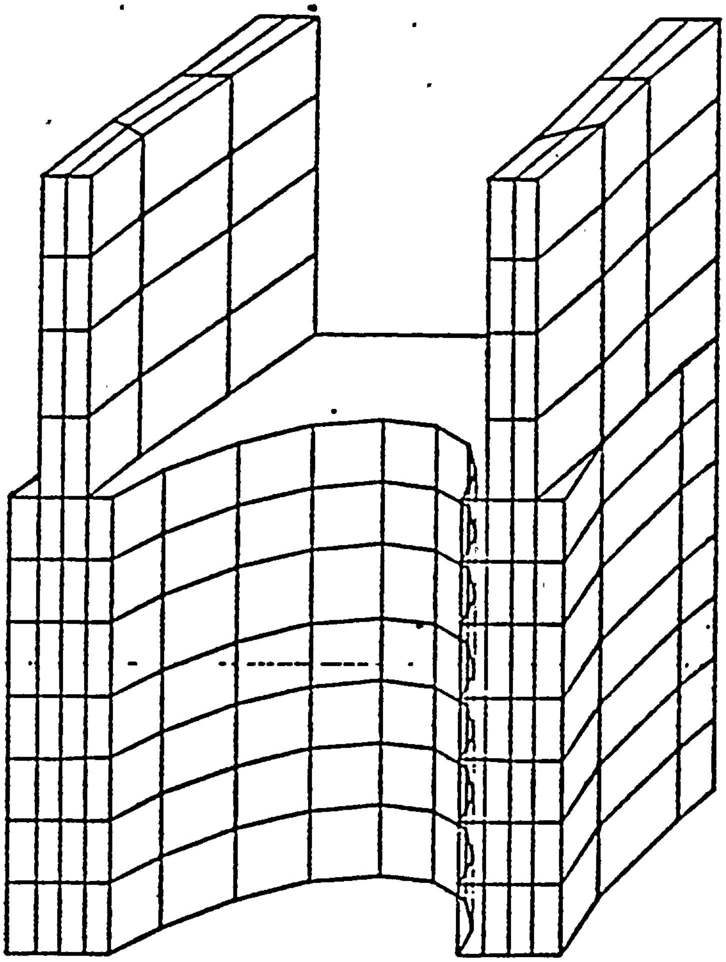
REFERENCES

1. "ASME Boiler and Pressure Vessel Code, Section III--Rules for Construction of Nuclear Power Plant Components; Division 2--Code for Concrete Reactor Vessels and Containments, "ASME Boiler and Pressure Vessel Committee, Subcommittee, Subcommittee on Nuclear Power, and ACI-ASME Joint Technical Committee, 1975 Edition.

2. BC TOP 5A, Revision 3, February, 1975, Prestressed Concrete Nuclear Reactor Containment Structures.

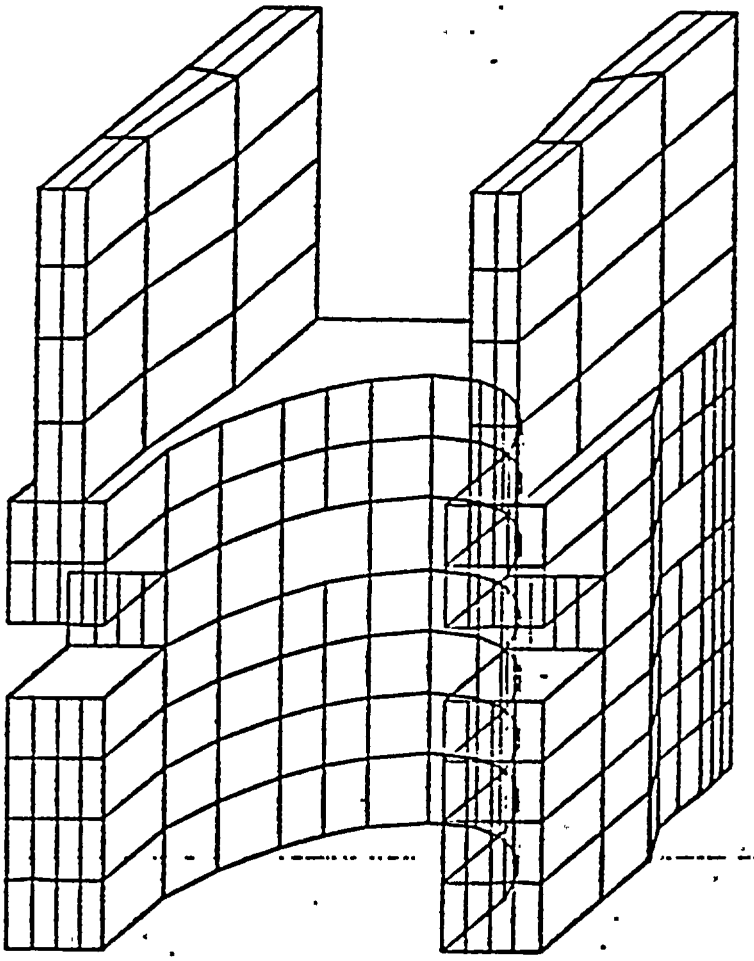
6

ANPP REACTOR CAVITY FINITE ELEMENT MODEL
EAST WALL FROM NORTH-SOUTH CENTERLINE



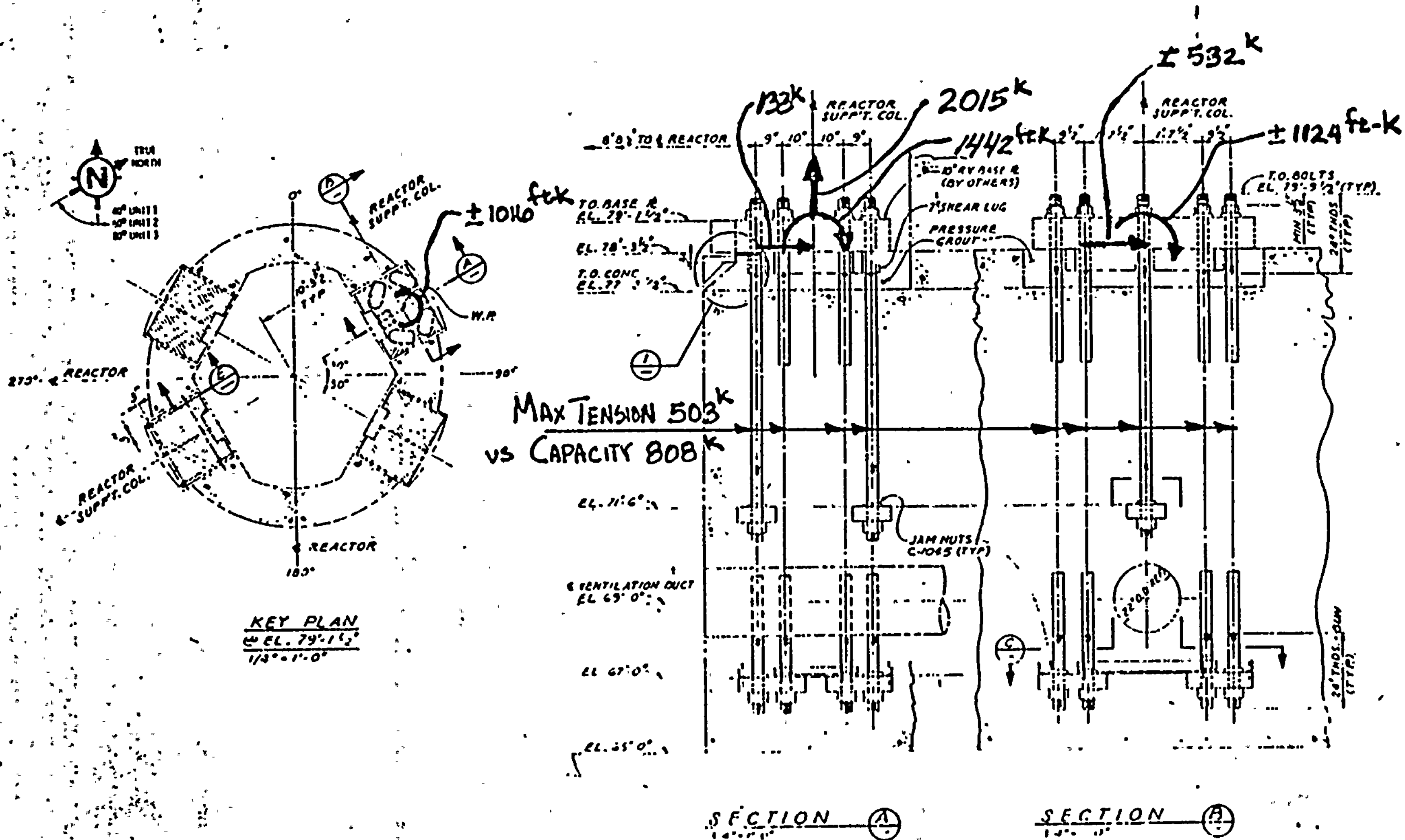
SAP - PVNGS REACTOR CAVITY PRIMARY SHIEL
D

ANPP REACTOR CAVITY FINITE ELEMENT MODEL
WEST WALL FROM NORTH-SOUTH CENTERLINE



SAP-PVNGS REACTOR CAVITY PRIMARY SHIELD

FORCES & M[illegible]ENTS AT KEY SECTIONS
FOR REACTOR VESSEL LOWER SUPPORTS



ATTACHMENT L 1 OF 2

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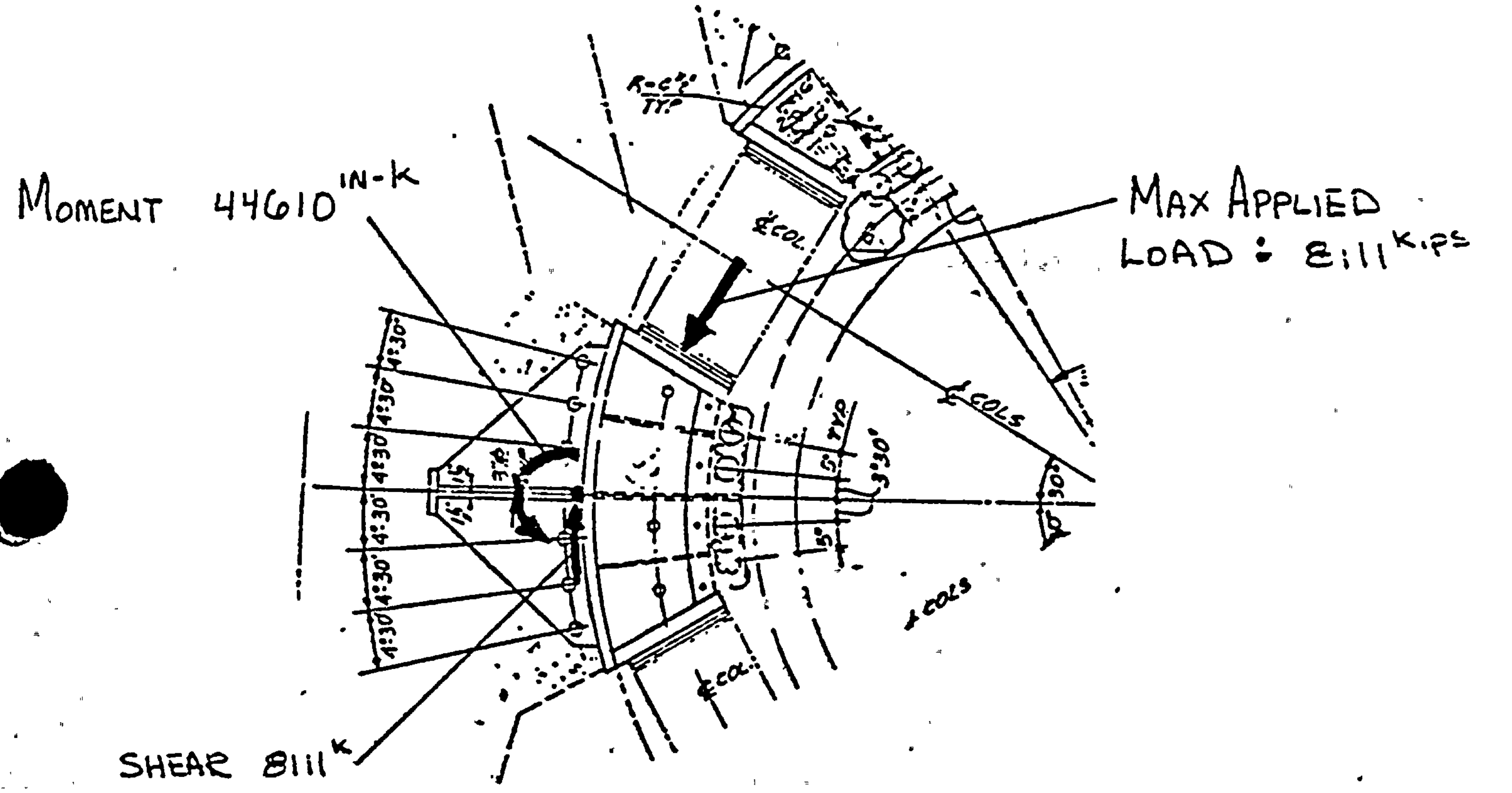
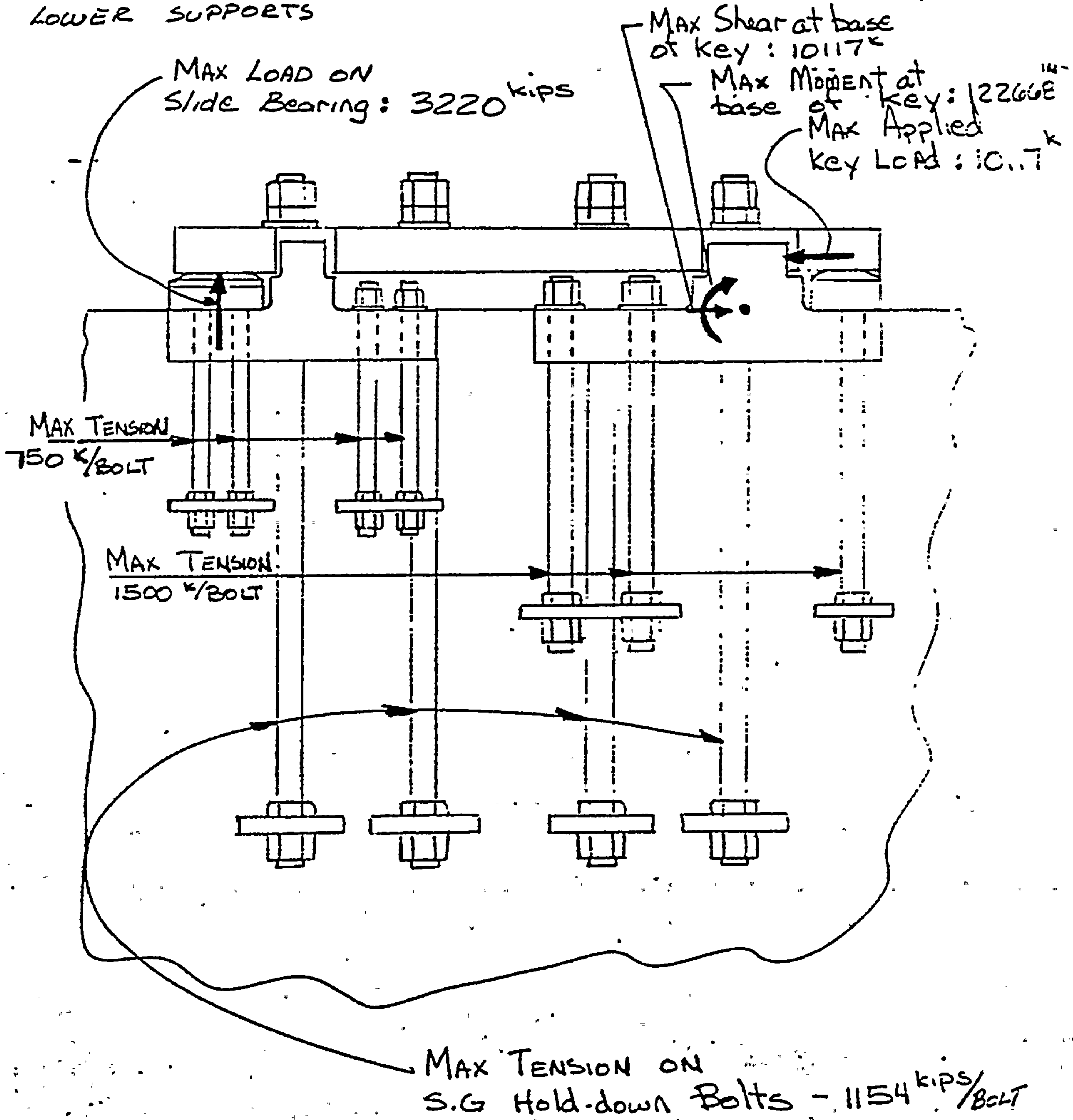
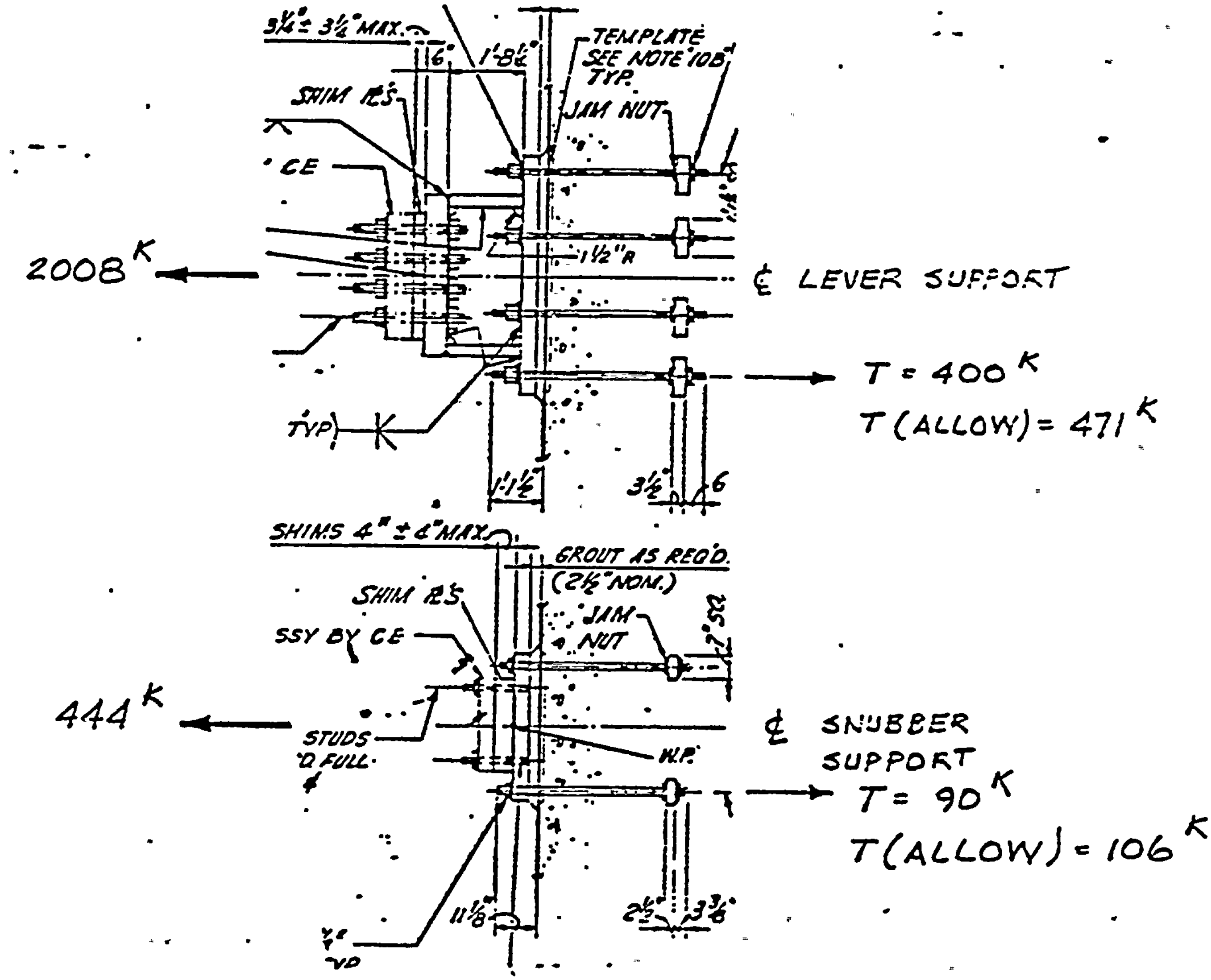
FORCES & MOMENTS AT KEY SECTIONS of REACTOR VESSEL UPPER LATERAL SUPPORTS



FORCES & MOMENT AT KEY SECTIONS of STEAM GENERATOR LOWER SUPPORTS





STEAM GENERATOR LEVER AND SNUBBER SUPPORTS

FOR ONE BOLT

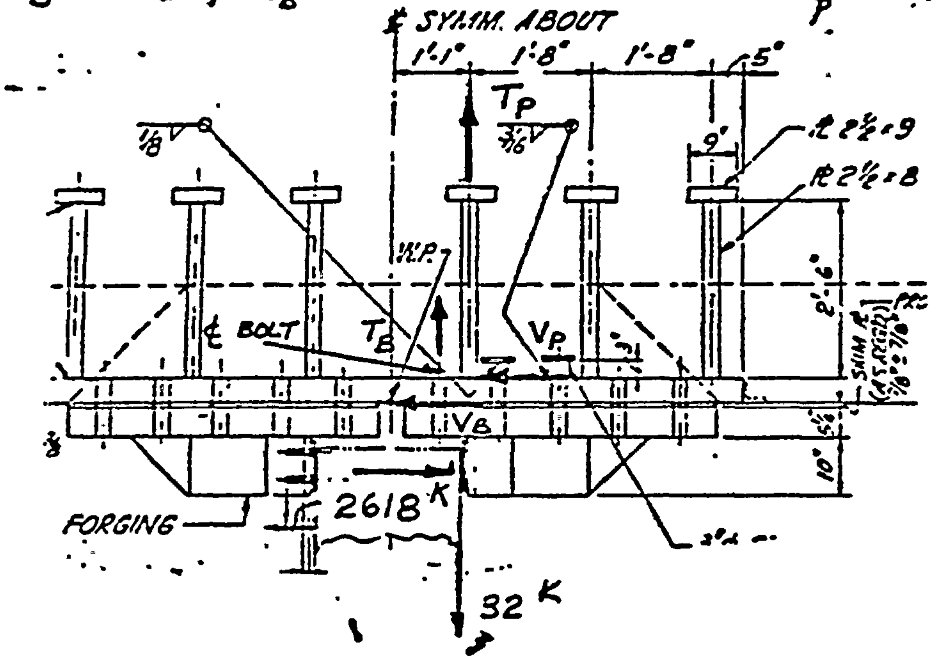
$T_B = 274^K$, $T_B(ALLOW) = 304^K$

$V_B = 226^K$, $V_B(ALLOW) = 296^K$

FOR ONE PLATE.

$T_P = 598^K$

$V_P = 218^K$

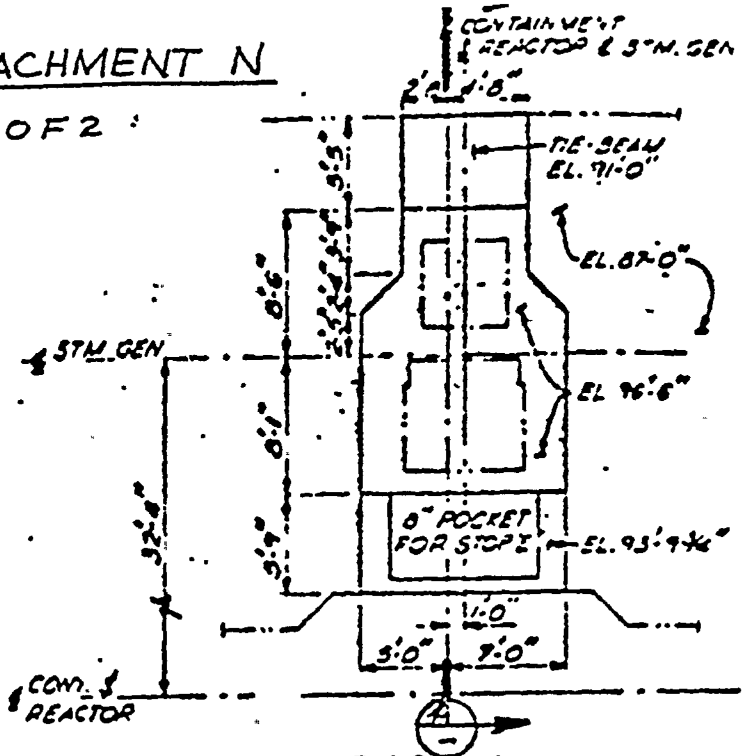


STEAM GENERATOR UPPER KEY SUPPORT

ATTACHMENT N

1 OF 2

REBAR PLACEMENT FOR STEAM GENERATOR BASE SUPPORTS



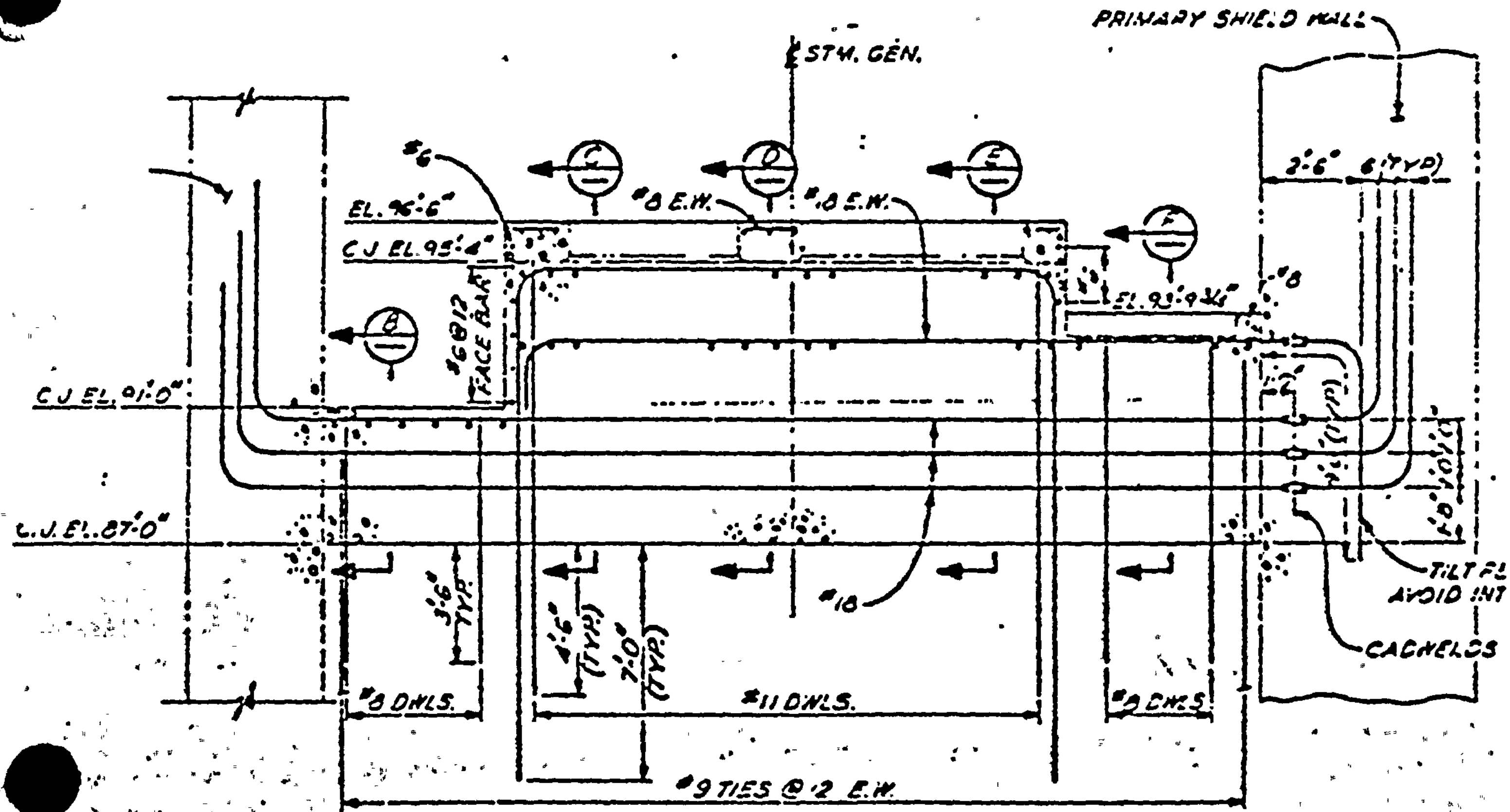
KEY PLAN

STEAM GENERATOR FOUNDATION

STM GEN. NO 1-AS SHOWN

STM. GEN. NO 2-OPP HAND

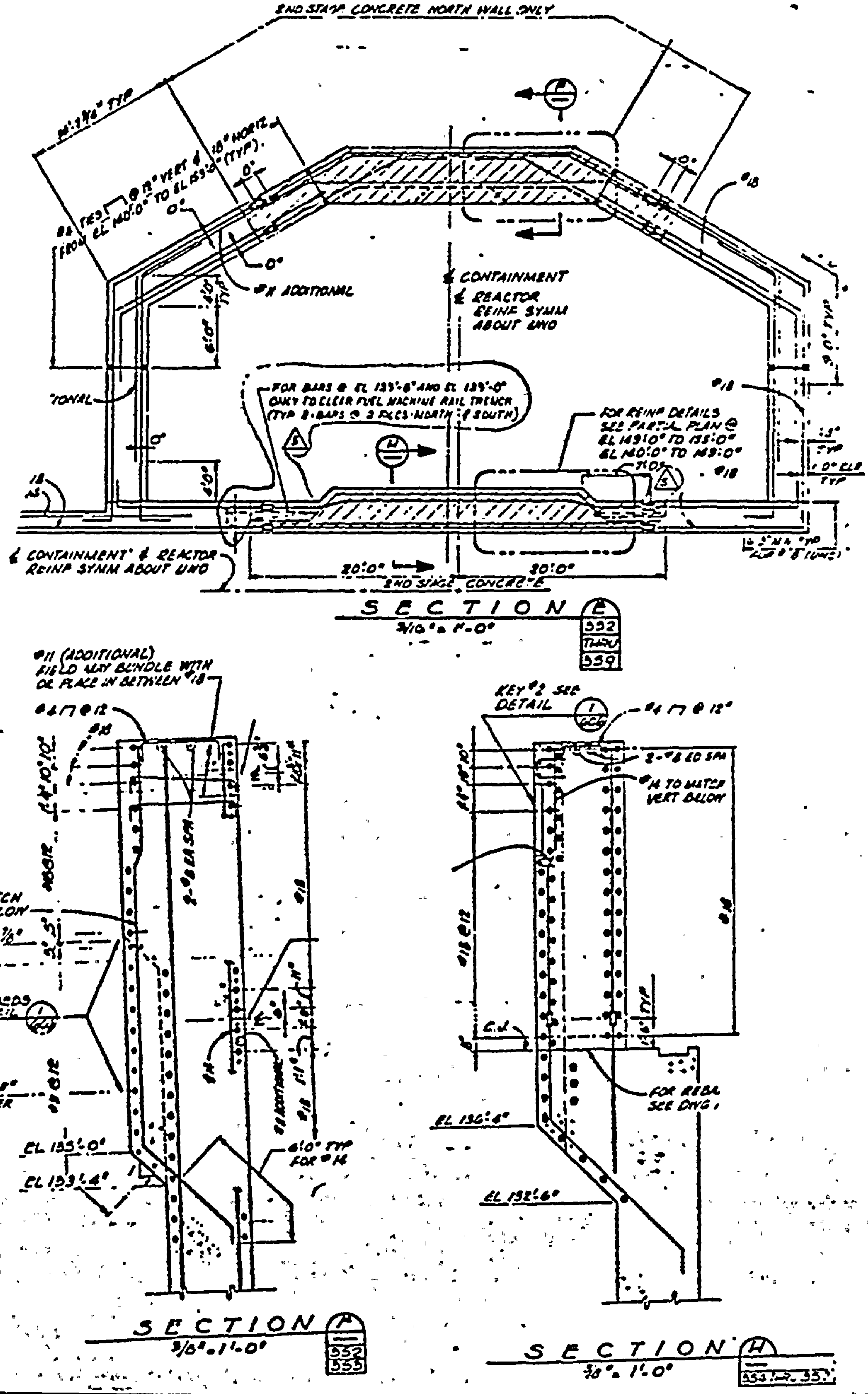
3/16"=1'-0"



NOTE: FOR OTHER REINF. BELOW EL. 87'-0" NOT SHOWN HERE SEE DWG. 13-C-ZCS-319, 320, & 321.

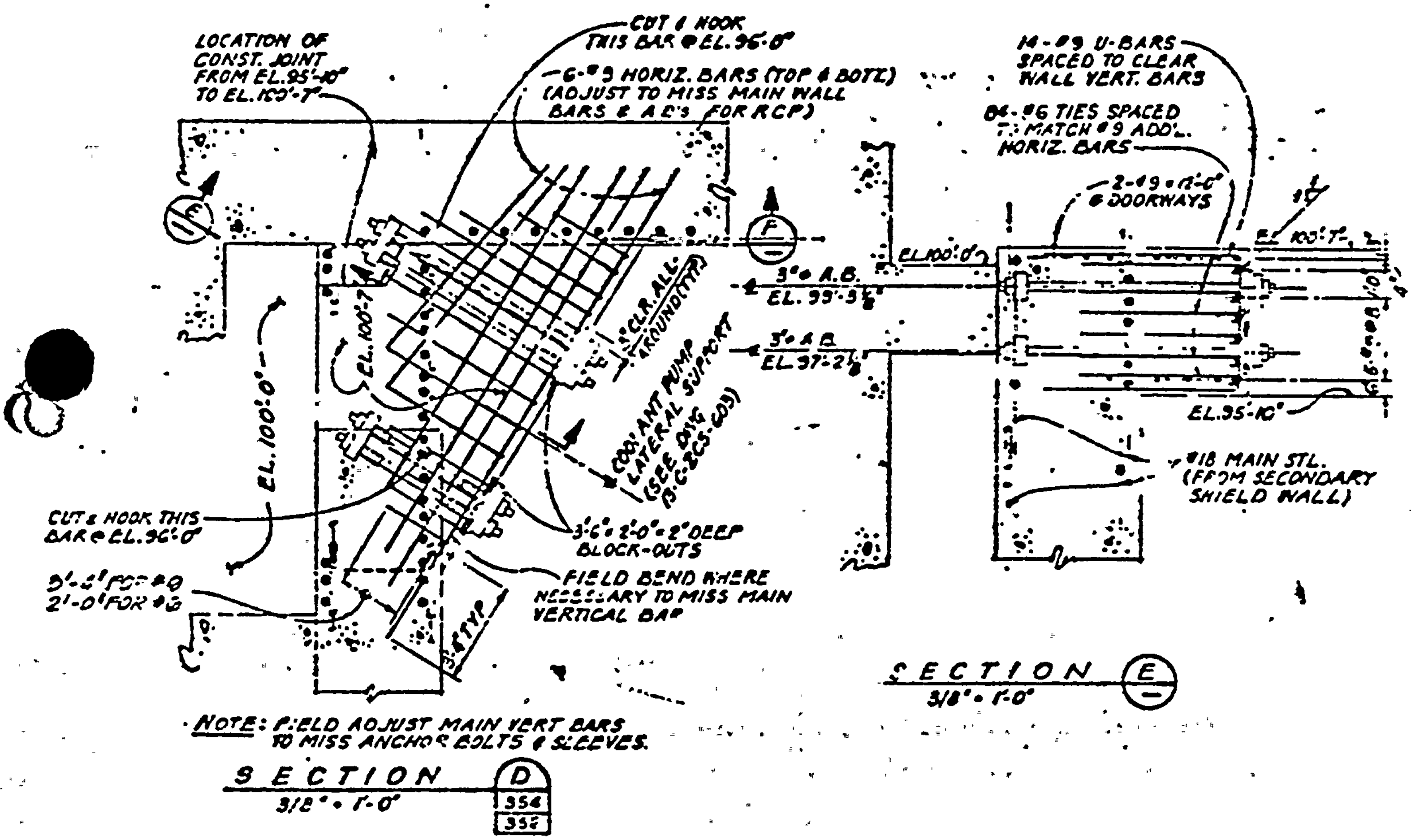
SECTION A

REBAR PLACEMENT FOR
STEAM GENERATOR UPPER SUPPORTS



ATTACHMENT O.

REBAR PLACEMENT FOR REACTOR COOLANT PUMP LATERAL SUPPORTS

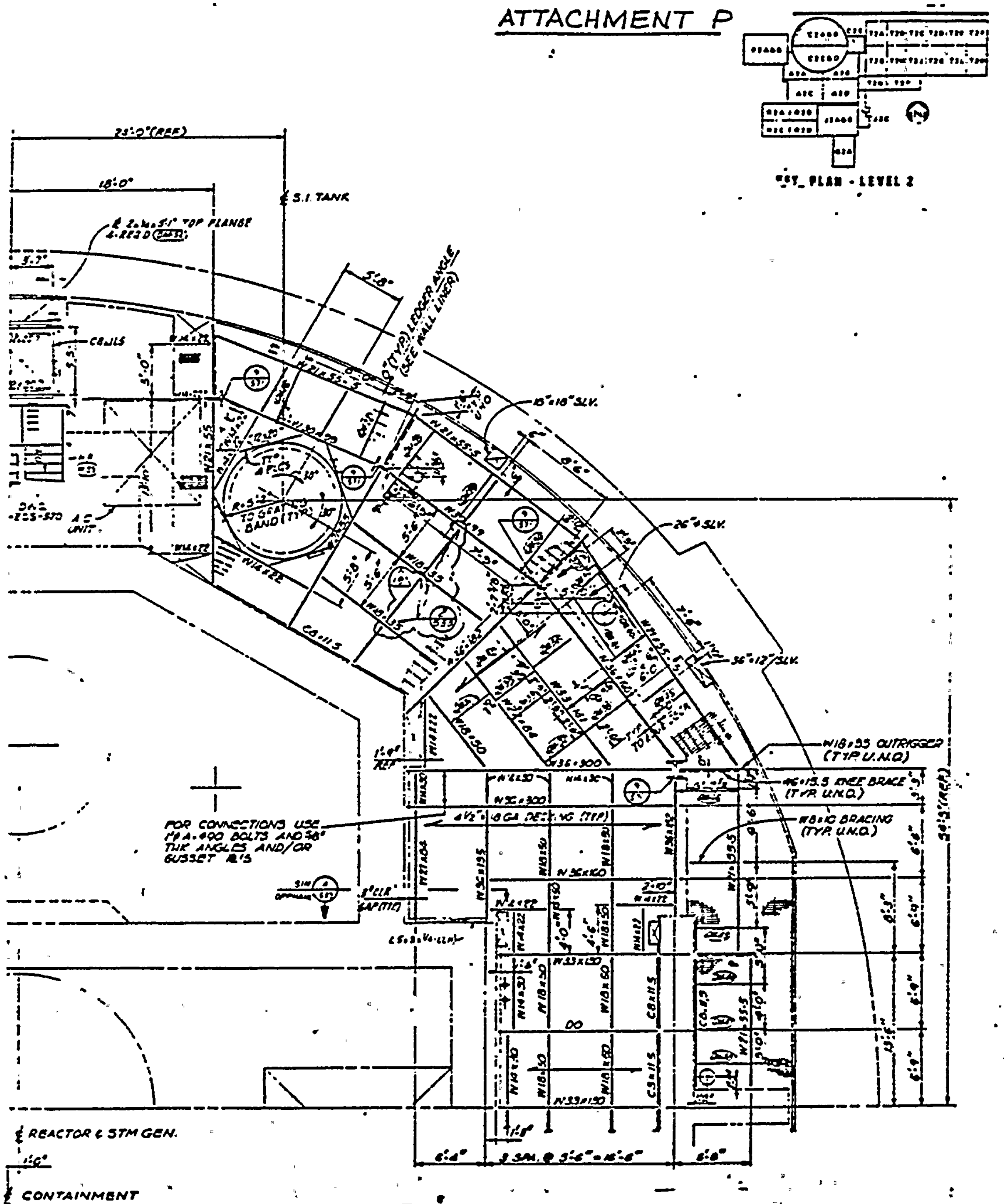


SECTION E
3/8" = 1'-0"

NOTE: FIELD ADJUST MAIN VERT BARS TO MISS ANCHOR BOLTS & SLEEVES.

SECTION D 354/352
3/8" = 1'-0"

ATTACHMENT P



T.O. GRATING EL. 120'-0" (U.N.O.)

T.O. STRUCT. STL. (SUPPORTING GRATING) EL. 119'-10 3/4" (U.N.O.)

T.O. STRUCT. STL. (SUPPORTING CONC. SLAB) EL. 118'-1 1/2" (U.N.O.)

Table 2.5-15

STATIC BEARING CAPACITY OF CATEGORY I STRUCTURES

Structure	Average Static Design Load $q_s (k/ft^2)$	Ultimate Bearing Capacity $q_o (k/ft^2)$	Factor of Safety (q_o/q_s)
Containment Building	7.9	35.7	4.5
Auxiliary Building (deep section)	6.2	34.9	5.6
Main Steam Support Structure	7.1	64.8	9.1
Control Building	3.3	45.3	13.7
Fuel Building	5.3	54.9	10.4
Diesel Generator Building	3.1	79.5	25.6
Refueling Water Tank	4.4	90.4	20.5
Condensate Storage Tank	3.5	112.4	32.1

Table 2.5-16

DYNAMIC BEARING CAPACITY OF CATEGORY I STRUCTURES[a]

Structure	Equivalent Uniform Vertical Stress q_d (k/ft^2)	Ultimate Bearing Capacity q_o (k/ft^2)	Factor of Safety (q_o/q_d)
Containment Building	16.1	32.2	2.0
Auxiliary Building (deep section)	10.3	25.8	2.5
Main Steam Support Structure	25.3	60.6	2.4
Control Building	9.8	39.8	4.1
Fuel Building	19.1	50.3	2.6
Diesel Generator Building	5.6	75.5	13.5
Refueling Water Tank	13.2	58.7	4.4
Condensate Storage Tank[b]	13.2	30.2	2.3

a. Based upon maximum dynamic loads derived from analyses described in section 3.7.

b. Condensate storage tank loads were conservatively chosen to be equal to the dynamic design load for the refueling water tank. Actual loads will be less.

1

Table 3.7-1
DAMPING VALUES
(PERCENT OF CRITICAL DAMPING)

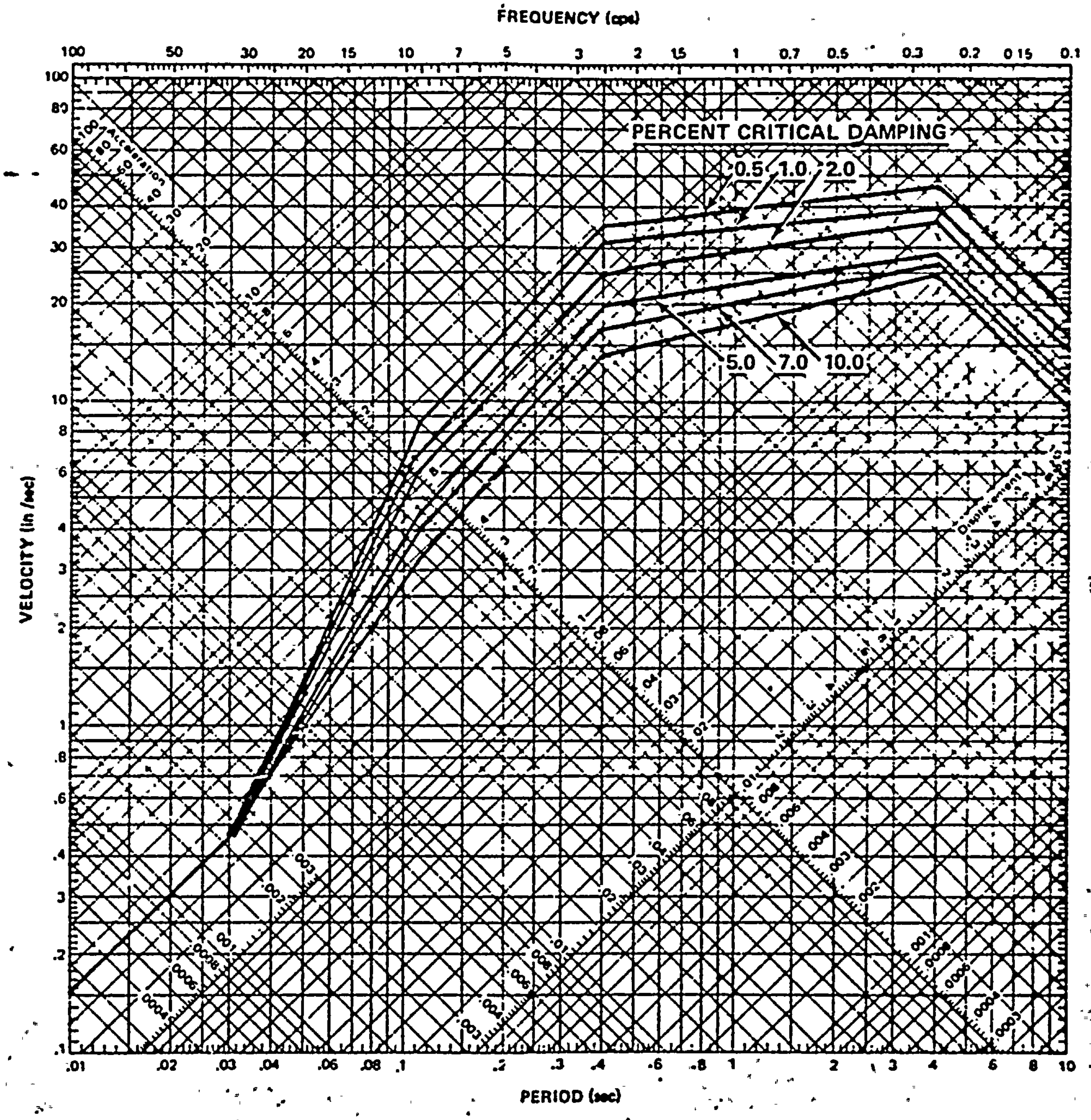
Structure or Component	Operating Basis Earthquake	Safe Shutdown Earthquake
Equipment and large-diameter piping systems, pipe diameter greater than 12 in.	2	3
Small-diameter piping systems, diameter equal to or less than 12 in.	1	2
Welded steel structures	2	4
Bolted steel structures	4	7
Prestressed concrete structures	2	5
Reinforced concrete structures	4	7

The applicable allowable design levels are given in section 3.8 for the various loading combinations which include seismic loadings.

3.7.1.4 Supporting Media for Seismic Category I Structures

For purposes of the seismic analysis, the site is assumed to be a multi-layer system consisting of soil over bedrock. The approximate depth of soil deposit over bedrock for each unit at the site is as follows:

	Unit 1	Unit 2	Unit 3
Depth of Soil, ft	330	350	295



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR SSE 0.25 g

Figure 3.7-1

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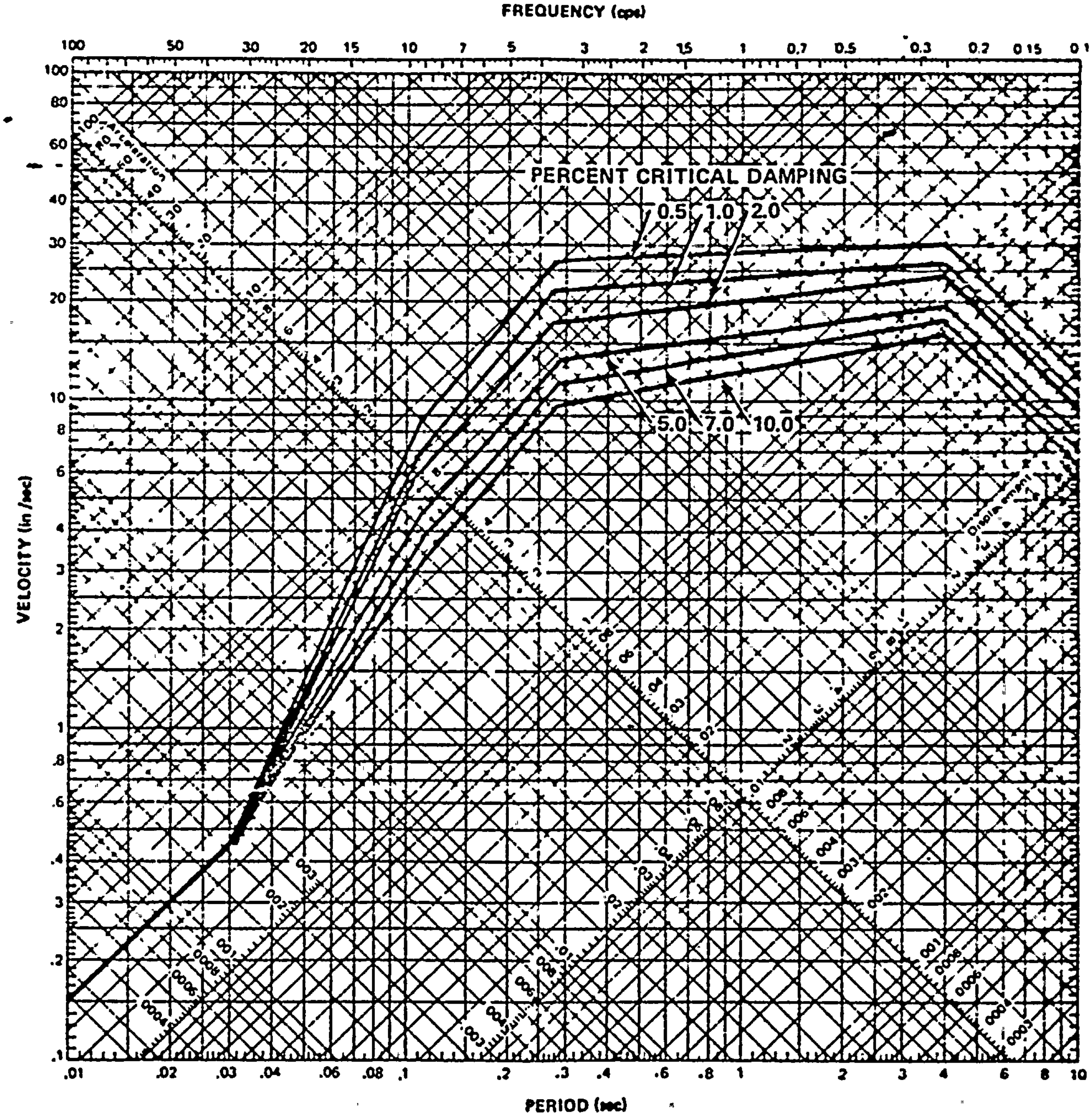
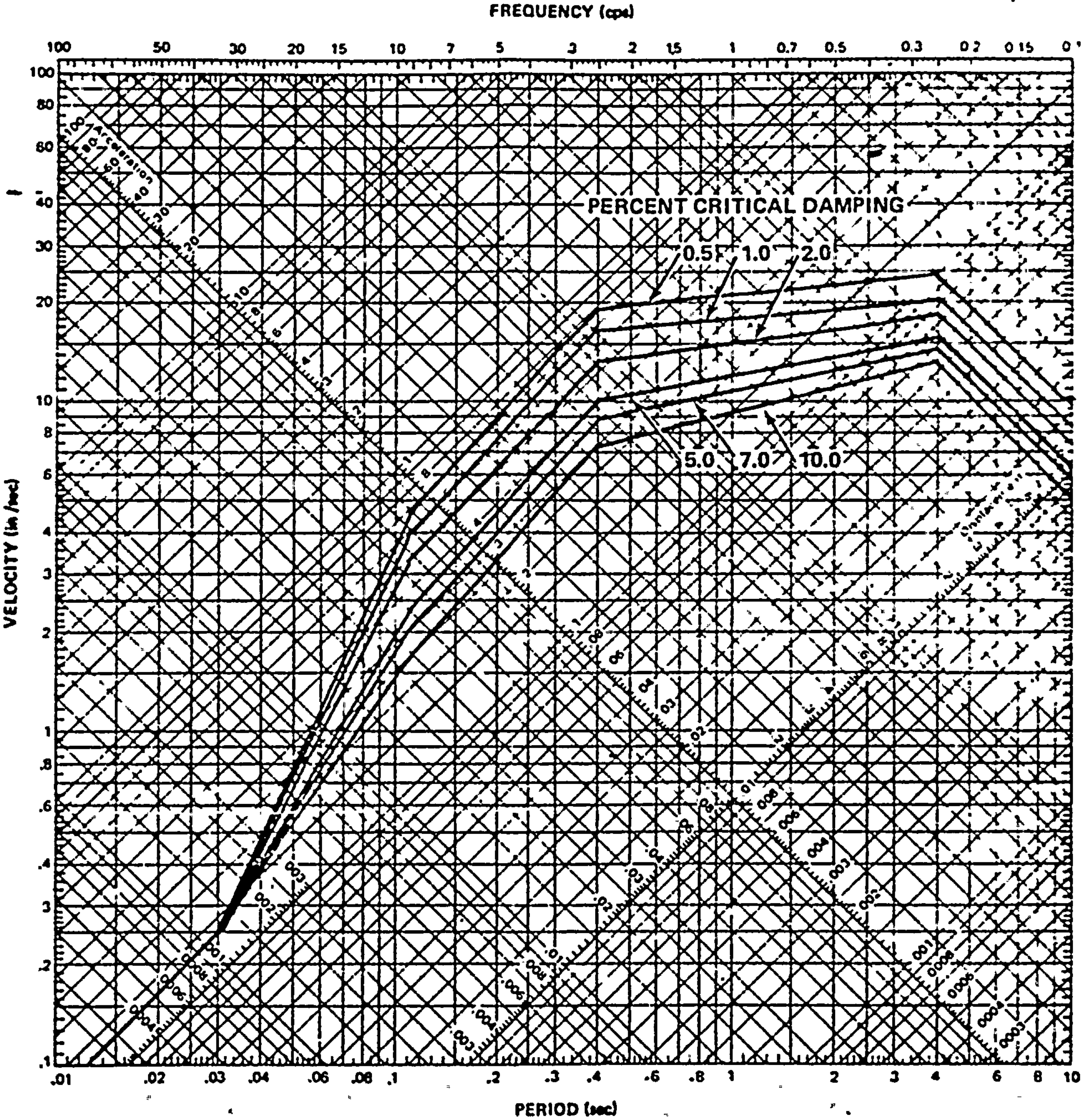


Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR SSE 0.25 g

Figure 3.7-2



Palo Verde Nuclear Generating Station
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HORIZONTAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-3

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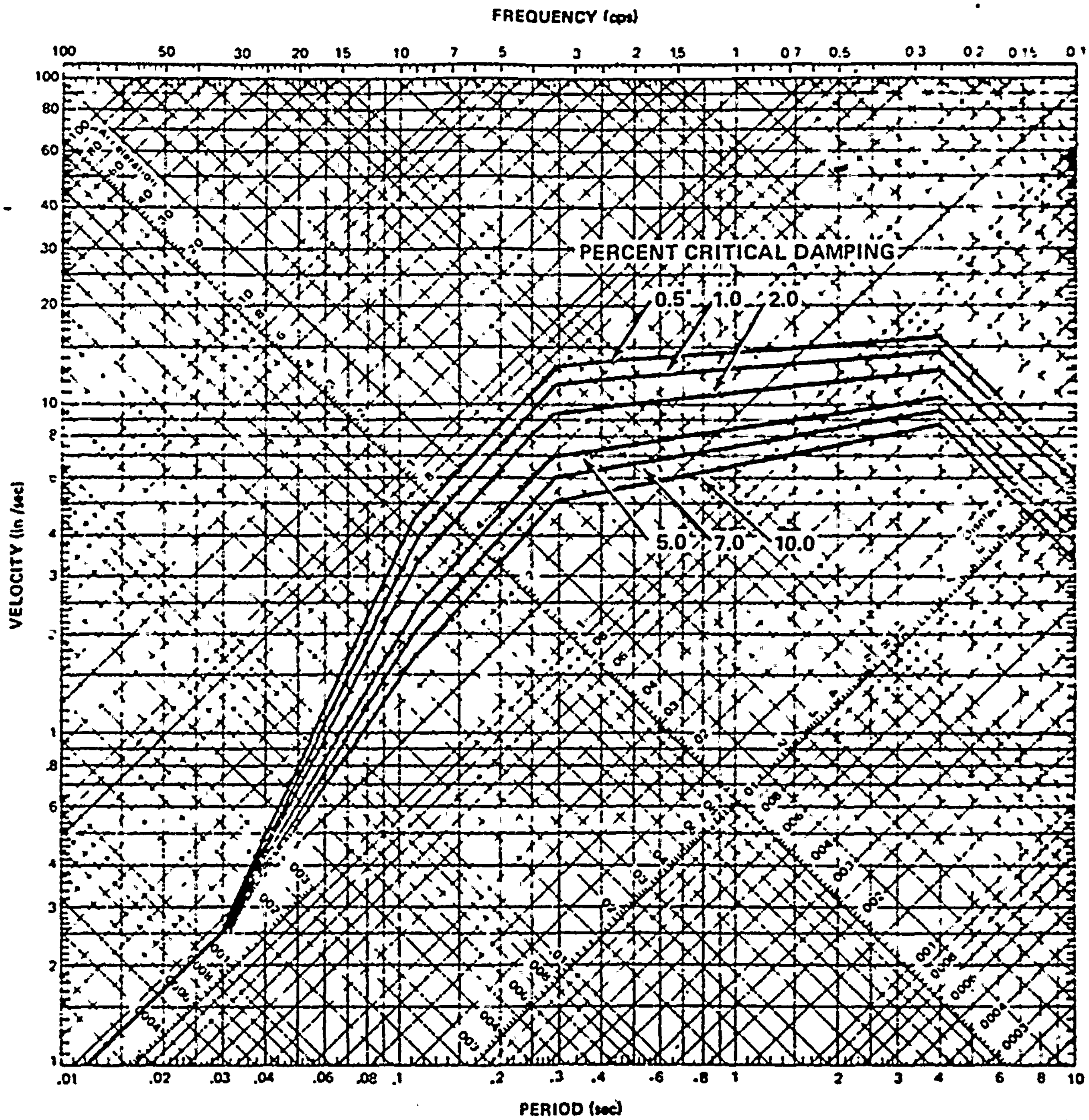

Palo Verde Nuclear Generating Station
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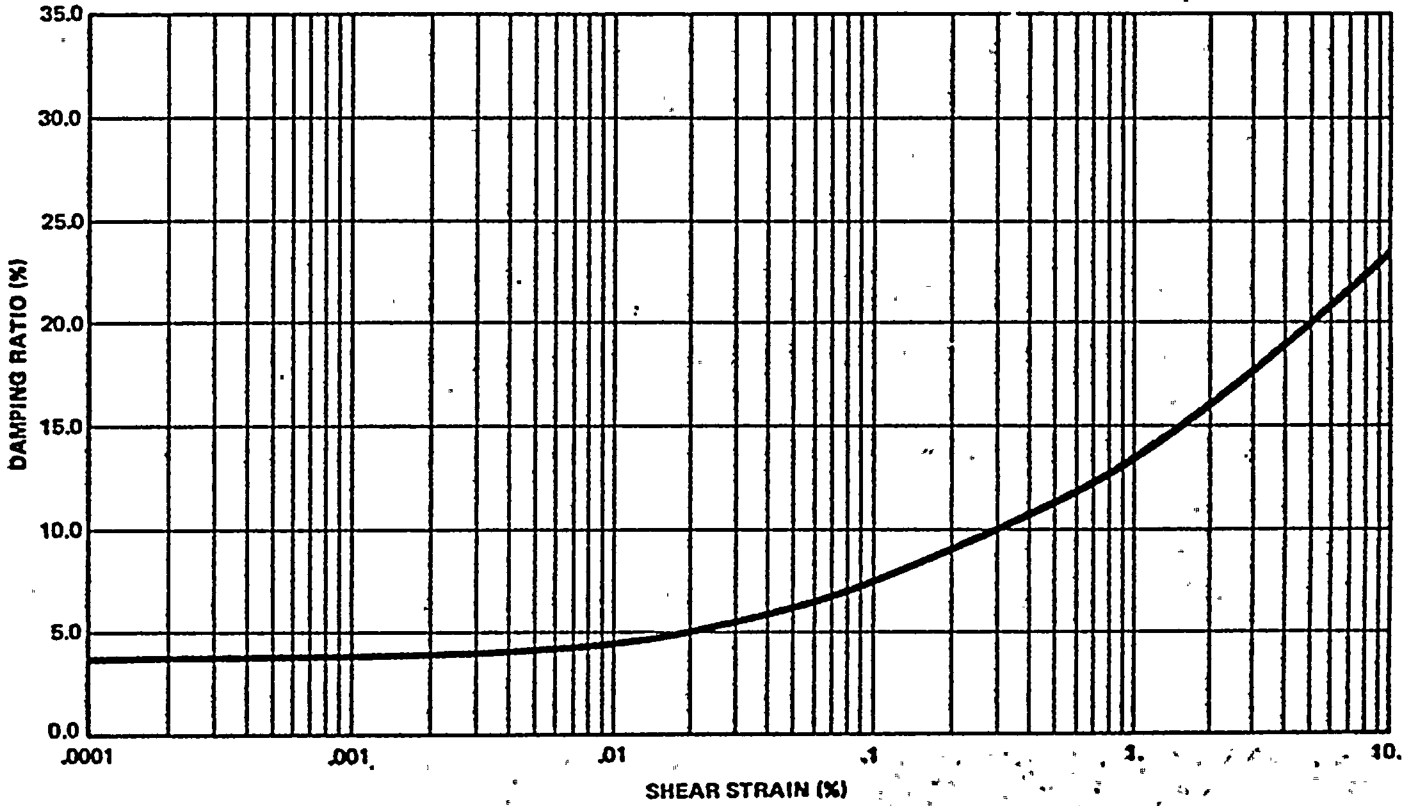
VERTICAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-4

92



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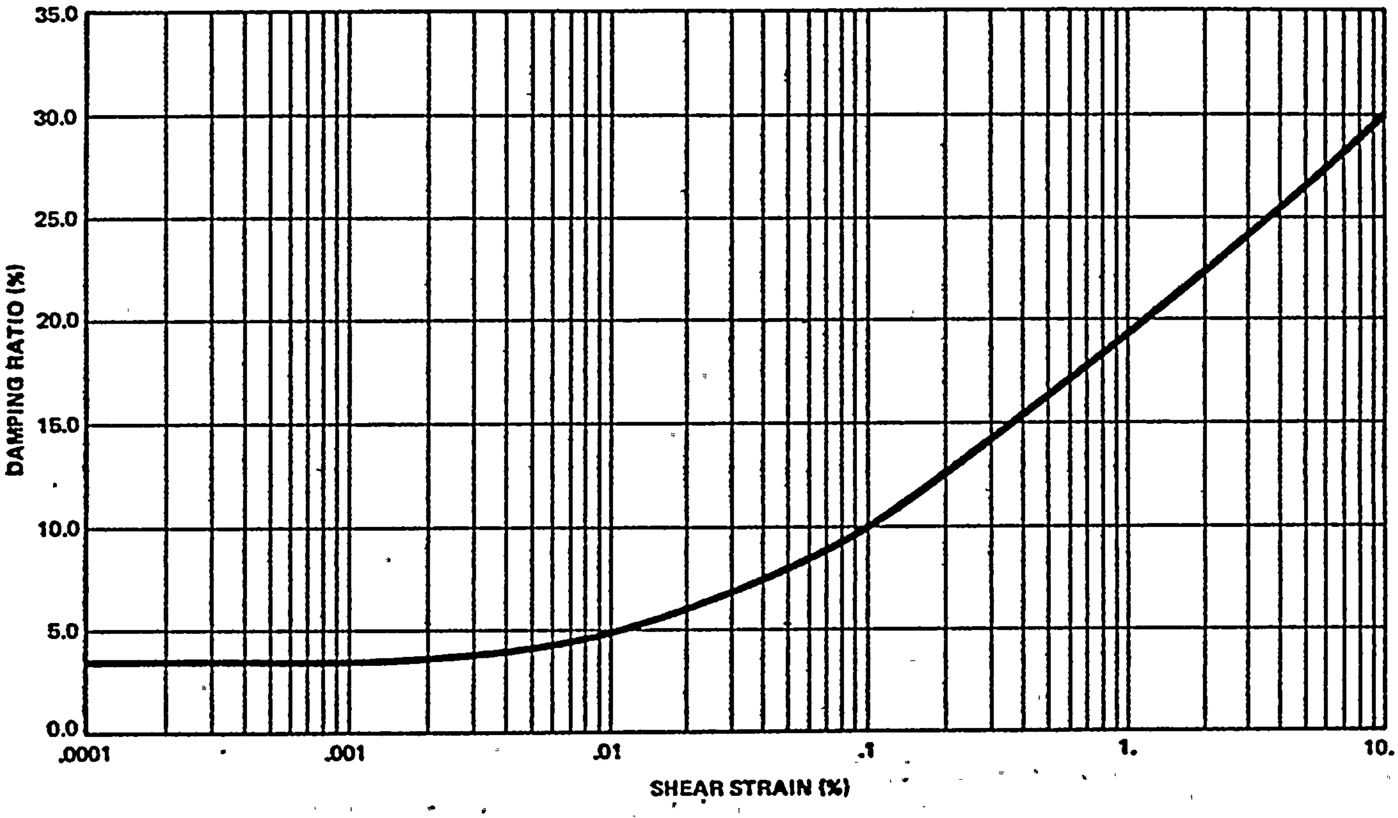
DAMPING VS. STRAIN - CLAY

FIGURE 3.7-5



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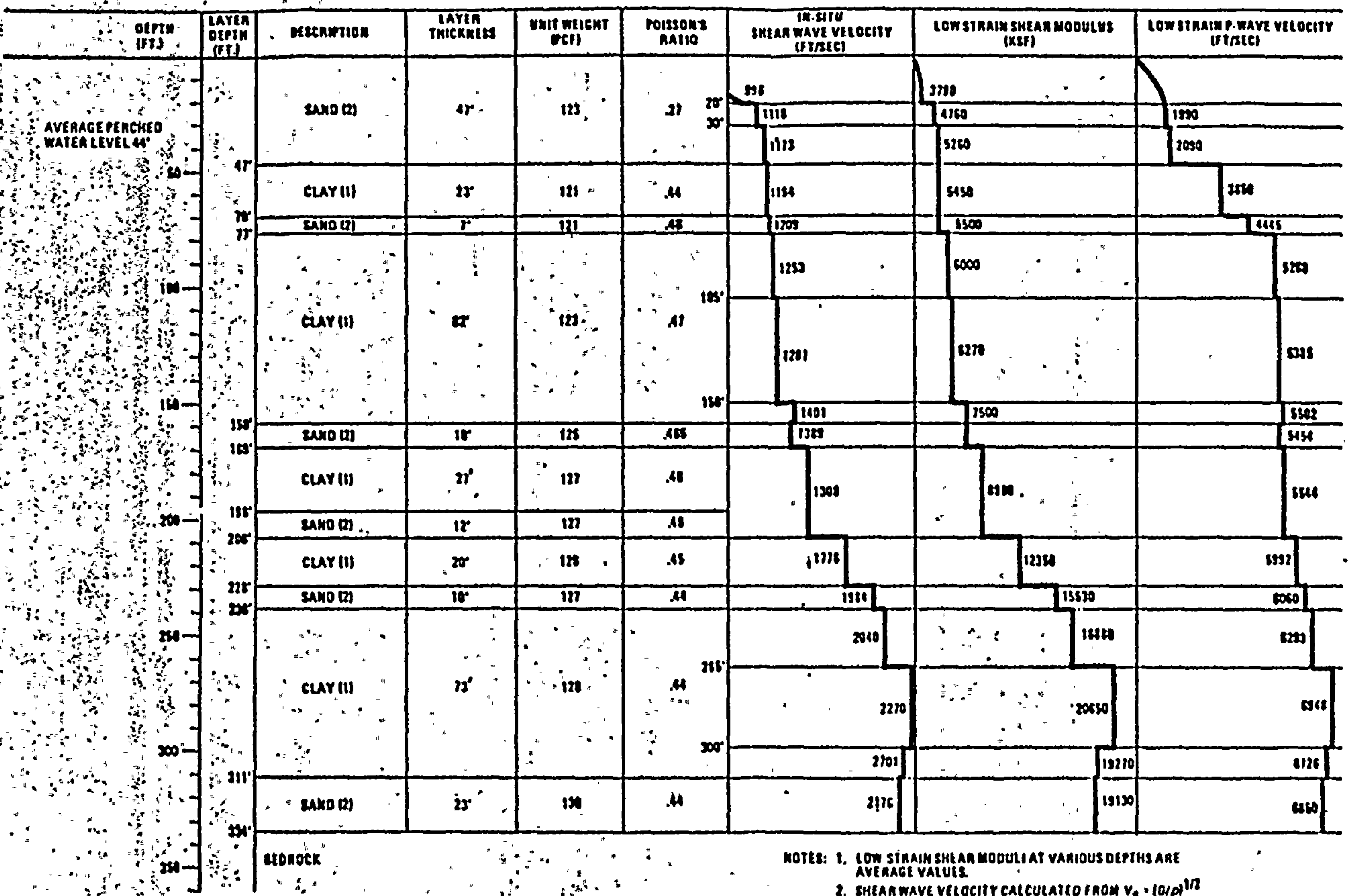
DAMPING VS. STRAIN - SAND

FIGURE 3.7-6



NOTES: 1. LOW STRAIN SHEAR MODULI AT VARIOUS DEPTHS ARE AVERAGE VALUES.
2. SHEAR WAVE VELOCITY CALCULATED FROM $V_s = (G/\rho)^{1/2}$
3. P-WAVE VELOCITY CALCULATED FROM $V_p = V_s [(2 - 2\nu)/(1 - 2\nu)]^{1/2}$

Palo Verde Nuclear Generating Station FSAR

SOIL PROFILE

FIGURE 3.7-7

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CHANGE

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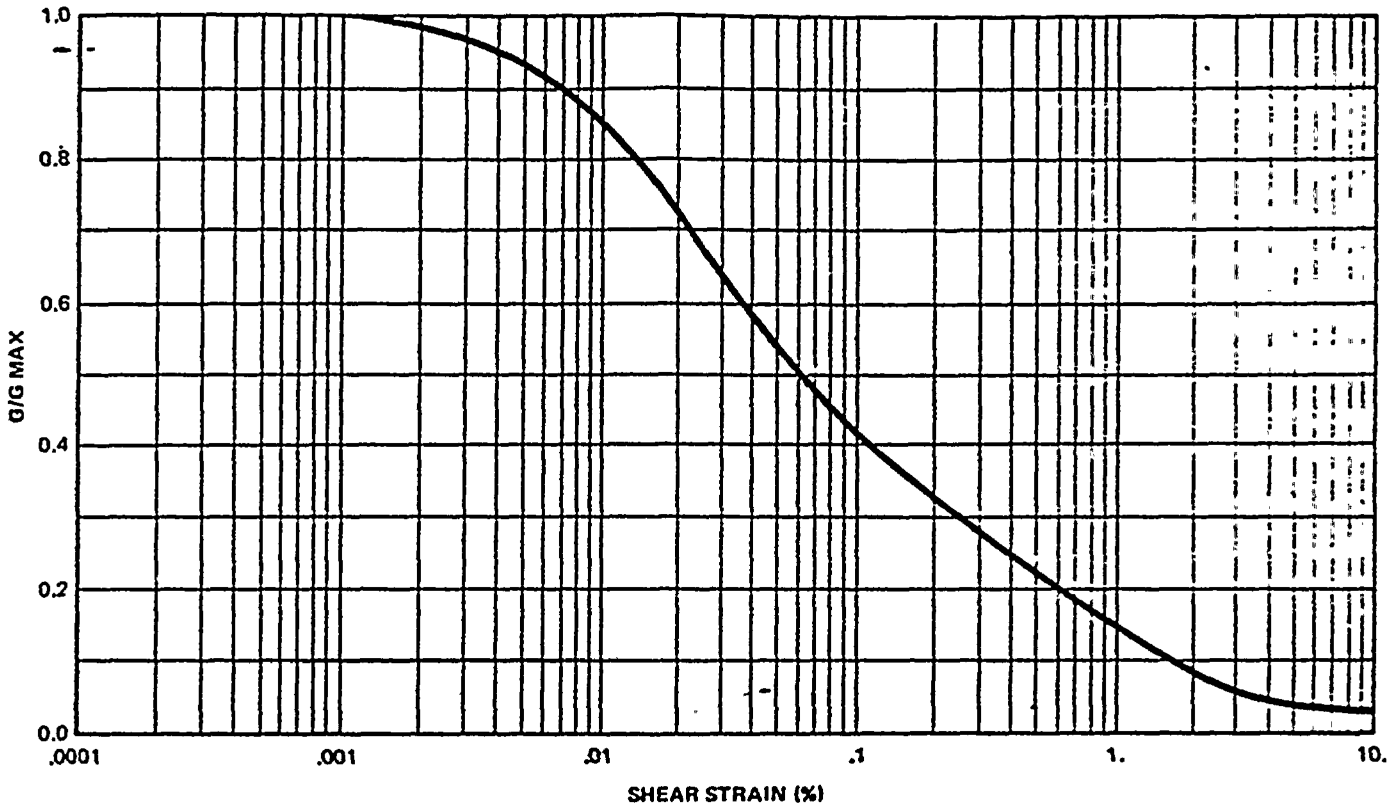

Palo Verde Nuclear Generating Station
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SHEAR MODULUS VS. STRAIN - CLAY

FIGURE 3.7-8

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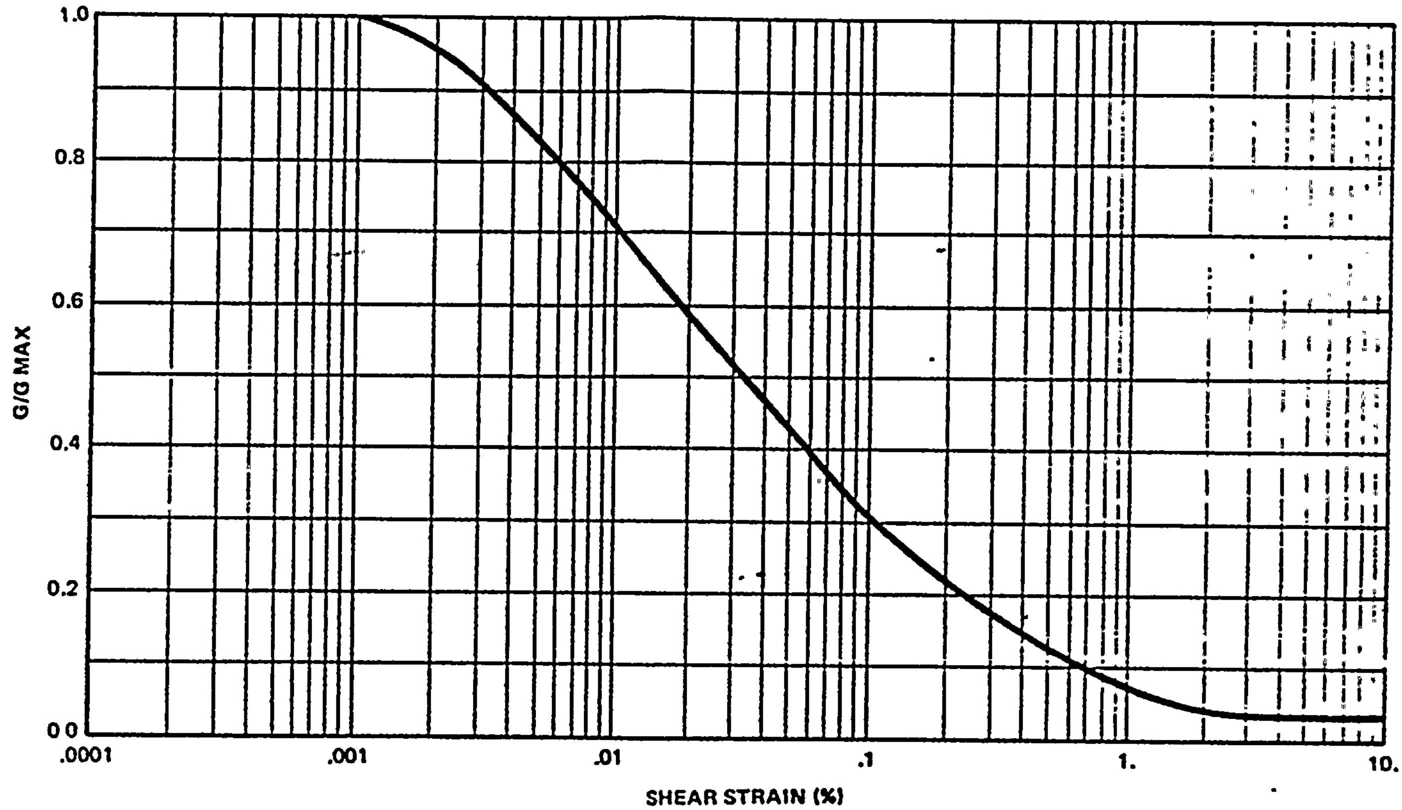

Palo Verde Nuclear Generating Station
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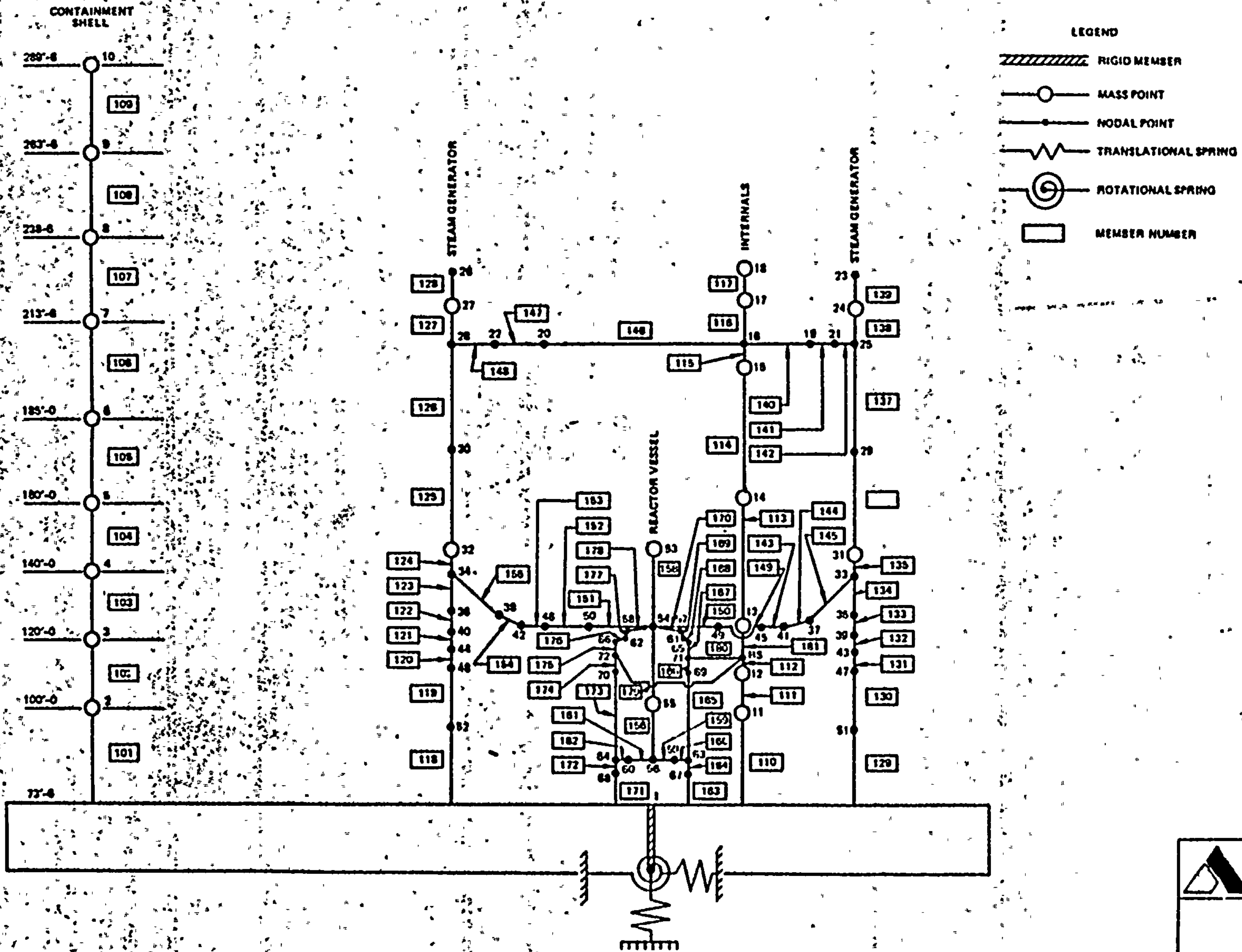
SHEAR MODULUS VS. STRAIN - SAND

FIGURE 3.7-9



Palo Verde Nuclear Generating Station
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CONTAINMENT BUILDING
LUMPED MASS MODEL

FIGURE 3.7-10

Table 3.8-1A

LOADING COMBINATION FOR DESIGN AND FINAL ANALYSIS OF CONTAINMENT SHELL (Sheet 1 of 2)

Reference Loading (RLC)	PVNGS Project Criteria: Category	PVNGS Project Criteria: Loading Combination	ASME Sect III, Div. 2	BC-TOP-5A R-3	Final Analysis Performed	Remarks
1	Test	$D + L + P_i + P_t + T_t$	Same	Same	Yes	T_t is considered same as T_o; initial prestress is more critical
2	Construction	$D + L + P_i + T_o$	Same	Same	Yes	Initial prestress case is more critical
3	Normal Operating Loads	$D + L + F + T_o + R_o + P_v$	Same	Same	Yes	R_o is a local load
4	Severe Environment	$D + L + F + T_o + E_o + R_o + P_v$	Same	Same	Yes	R_o is a local load, P_a is omitted conservatively
5	Severe Environment	$D + L + F + T_o + W + R_o$	None	Same	No	Less severe than loading Combination #4
6	Severe Environment	$D + L + F + T_o + W + R_o + P_v$	Same	None	No	Less severe than loading Combination #5
7	Severe Environment	$D + 1.3L + F + T_o + 1.5E_o + R_o$	None	Same	No	Less severe than loading Combination #11
8	Severe Environment	$D + 1.3L + F + T_o + 1.5W + R_o$	None	Same	No	Less Severe than loading Combination #7
9	Severe Environment	$D + 1.3L + F + T_o + 1.5E_o + R_o + P_v$	Same	None	No	Less severe than loading Combination #7
10	Severe Environment	$D + 1.3L + F + T_o + 1.5W + R_o + P_v$	Same	None	No	Less severe than loading Combination #8
11	Extreme Environment	$D + L + F + T_o + E_{ss} + R_o + P_v$	None	Same	Yes	R_o is a local load; P_v is omitted conservatively
12	Extreme Environment	$D + L + F + T_o + W_t + R_o + P_v$	None	Same	No	Less severe than loading Combination #11

Notation

- D = Dead load
- L = Live Load
- P_i = Initial prestress
- F = Final prestress
- T_o = Normal operating temperature
- P_a = Design accident pressure
- W = Wind Load
- R_o = Pipe reactions during normal operating or shutdown conditions
- R_a = Pipe reactions due to postulated break (including R_o)
- T_a = Design accident temperature
- P_t = Test pressure (=1.15 P_a)
- T_t = Test temperature (assumed equal to T_o)
- P_v = Design external pressure (vacuum)
- E_o = Operating basis earthquake
- E_{ss} = Safe shutdown earthquake
- W_t = Tornado Loads (including differential pressure and tornado missiles)
- R_r = Local effects of containment due to postulated pipe breaks

3

Table 3.8-1A

LOADING COMBINATION FOR DESIGN AND FINAL ANALYSIS OF CONTAINMENT SHELL (Sheet 2 of 2)

Reference Loading (RLC)	PVNGS Project Criteria: Category	PVNGS Project Criteria: Loading Combination	ASME Sect III, Div. 2	BC-TOP-5A R-1	Final Analysis Performed	Remarks
13	Extreme Environment	$D + L + F + T_o + E_{ss} + P_v$	Same	None	No	Less severe than loading Combination #11
14	Extreme Environment	$D + L + F + T_o + W_t + P_v$	Same	None	No	Less Severe than loading Combination #12
15	Abnormal	$D + L + F + 1.5P_a + T_a + R_a$	Same	Same	Yes	R_a is a local load
16	Abnormal	$D + L + F + P_a + 1.25R_a$	None	Same	No	Less critical than loading Combination #17 in local analysis
17	Abnormal	$D + L + F + P_a + T_a + 1.25R_a$	Same	None	No	R_a is a local load, less severe than loading Combination #15
18	Abnormal with Severe Environment	$D + L + F + 1.25P_a + T_a + 1.25E_o + R_a + R_r$	None	Same	Yes	R_a and R_r are local loads
19	Abnormal with Severe Environment	$D + L + F + 1.25P_a + T_a + 1.25W + R_a + R_r$	None	Same	No	Less severe than loading Combination #18
20	Abnormal with Severe Environment	$D + L + F + 1.25P_a + T_a + 1.25E_o + R_a$	Same	None	No	Same as loading Combination #18 without local loads
21	Abnormal with Severe Environment	$D + L + F + 1.25P_a + T_a + 1.25W + R_a$	Same	None	No	Less severe than loading Combination #19
22	Abnormal with Severe Environment	$D + L + F + T_o + E_o$	Same	None	No	Less severe than loading Combination #13
23	Abnormal with Severe Environment	$D + L + F + T_a + W$	Same	None	No	Less Severe than loading Combination #14
24	Abnormal with Extreme Environment	$D + L + F + P_a + T_a + E_{ss} + R_a + R_r$	Same	Same	Yes	R_a and R_r are local loads

Notation

- D = Dead load
- L = Live Load
- F_i = Initial prestress
- F = Final prestress
- T_o = Normal operating temperature
- P_a = Design accident pressure
- W = Wind Load
- R_o = Pipe reactions during normal operating or shutdown conditions
- R_a = Pipe reactions due to postulated break (including R_o)
- T_a = Design accident temperature
- P_t = Test pressure(=1.15 P_a)
- T_t = Test temperature (assumed equal to T_o)
- P_v = Design external pressure (vacuum)
- E_o = Operating basis earthquake
- E_{ss} = Safe shutdown earthquake
- W_t = Tornado Loads (including differential pressure and tornado missiles)
- R_r = Local effects of containment due to postulated pipe breaks

Note: Local loads are not considered in the overall analysis but are taken into account in local design. Also the live load has a negligible effect on the pressure boundary and thus is not included in the final analysis.

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Table 3.8-1B[a]

STRESS ANALYSIS RESULTS (Sheet 1 of 9)

Reference Loading Combination: $D + F_i + P_t\ T_t$ (81 from Table 3.8-1A)

Portion	Section (Shown in Figure 3.8-22A)	Concrete Stresses								Reinforcement Stresses								Liner Strains[b]		Deflection[c] (Primary Loads)
		Meridional				Hoop				Meridional				Hoop						
		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary				
		MEM (psi)	MEM & BEN (psi)	MEM (psi)	MEM & BEN (psi)	MEM (psi)	MEM & BEN (psi)	MEM. (psi)	MEM & BEN (psi)	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Meridional $x\ 10^{-6}$ in/in	Hoop $x\ 10^{-6}$ in/in	(in)
Allowable	Shell	-1800	-2700	-2700	-3600	-1800	-2700	-2700	-3600	±30	±30	±40	±40	±30	±30	±40	±40	±4000	±4000	-
	Basemat	-1500	-2250	-2250	-3000	-1500	-2250	-2250	-3000											
Dome	2	-338	-327	-315	-916	-328	-330	-320	-906	-1.5	-1.7	-2.7	6.3	-1.5	-1.7	-2.6	6.3	-376	-366	-0.24
	7	-592	-627	-585	-1360	-365	-403	-384	-1085	-2.4	-1.9	-3.7	2.8	-1.6	-1.5	-3.0	6.6	-284	-428	-0.16
Wall	16	-549	-554	-549	-1401	-467	-424	-451	-1211	-2.9	-2.6	-5.2	4.4	-1.9	-1.8	-3.8	5.5	-440	-449	-0.06
	18	-594	-564	-594	-1408	-342	-326	-338	-1064	-3.2	-3.2	-5.6	3.7	-1.3	-1.2	-2.9	6.4	-429	-448	-0.04
	20	-639	-680	-639	-1315	-387	-362	-357	-1048	-3.1	-4.0	-5.7	1.6	-1.5	-1.4	-2.9	6.4	-358	-452	-0.05
	21	-650	-1049	-650	-1069	-303	-358	-196	-814	-2.2	-6.4	-5.2	-0.3	-1.0	-1.0	-1.7	7.5	-238	-450	-0.03
	22[e]	-676	-2214	-858	-941	-218	-527	-135	-560	0.1	-13.0	-4.1	-5.3	-0.5	-0.3	-1.6	6.0	636	-368	-0.01
Basemat Slab	23	27	-244	-37	-333	-6	-152	-29	-352	12.6	-0.5	2.0	0.5	-0.9	3.9	-3.4	8.3	516	-462	-0.01
	25	-40	-769	-129	-1212	-16	-336	-58	-605	-3.8	11.1	-7.6	14.5	-1.0	3.7	-3.2	8.3	-869	-450	-0.37
	26	-11	-380	-73	-705	16	-159	-29	-355	-2.4	7.5	-5.6	11.0	-0.5	1.0	-2.5	6.3	-650	-388	-0.48
Reactor Cavity	27	-177	-195	-98	-132	5	-13	79	(d)	-1.0	-1.3	-1.1	-1.0	0.3	0.2	2.7	4.9	13	-90	0.00
	28	92	-83	67	-104	52	91	107	(d)	0.6	0.7	2.8	10.4	0.6	0.6	0.8	9.6	-320	-372	-0.48

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Table 3.8-1B

STRESS ANALYSIS RESULTS (Sheet 2 of 9)

Footnotes:

(a) Sign Conventions are:

Stresses and strains . . . . (+) tensile . . . . (-) compressive

Deflections . . . . (+) outward . . . . (-) inward

(b) Allowable liner strains shown are based on the lowest values from the ASME Code, Section III, division 2.

(c) All deflections shown are normal to the given surface.

(d) Completely cracked sections; partially cracked sections are not indicated.

(e) The stresses for section 22 were determined from a more detailed analysis, in addition to the FINEL analysis.

(f) The stresses were obtained from OPTCON computer output.

(g) Membrane stress is greater than 200 psi and thus the section is assumed cracked.

(h) Reinforcement is assumed to yield at 54 ksi, the calculated strain is 0.00200 in/in.

Table 3.8-1B[a]

STRESS ANALYSIS RESULTS (Sheet 3 of 9)

Reference Loading Combination: $D + P_i + T_o$ (#2 from Table 3.8-1A)

Portion	Section (Shown in Figure 3.8-22A)	Concrete Stresses								Reinforcement Stresses								Liner Strains[b]		Deflection[c] (Primary Loads)
		Meridional				Hoop				Meridional				Hoop						
		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary				
		MEM (psi)	MEM & BEN (psi)	MEM (psi)	MEM & BEN (psi)	MEM (psi)	MEM & BEN (psi)	MEM (psi)	MEM & BEN (psi)	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Meridional x 10^{-6} in/in	Hoop x 10^{-6} in/in	(in)
Allowable	Shell	-1800	-2700	-2700	-3600	-1800	-2700	-2700	-3600	±30	±30	±40	±40	±30	±30	±40	±40	±4000	±4000	-
	Basemat	-1500	-2250	-2250	-3000	-1500	-2250	-2250	-3000											
Dome	2	-1062	-990	-1050	-1831	-1040	-1005	-1039	-1830	-5.1	-5.1	-7.4	1.7	-5.2	-5.2	-7.5	1.6	-382	-373	-0.67
	7	-1234	-1259	-1232	-2154	-1000	-1053	-996	-1946	-4.7	-4.2	-6.5	.5	-5.1	-4.8	-7.6	2.0	-309	-430	-0.49
Wall	16	-1161	-1128	-1161	-2092	-1469	-1276	-1464	-2265	-5.6	-5.3	-8.6	.9	-6.8	-6.5	-9.8	-.3	-432	-462	-0.21
	18	-1207	-1150	-1207	-2130	-1604	-1459	-1598	-2446	-5.6	-5.6	-8.6	.8	-7.9	-7.6	-11.0	-1.4	-429	-463	-0.24
	20	-1252	-1365	-1252	-1795	-1054	-960	-895	-1704	-5.8	-7.7	-8.6	-2.4	-4.8	-4.5	-6.6	3.9	-299	-460	-0.16
	21	-1264	-1535	-1263	-2261	-497	-560	-332	-1152	-8.1	-4.4	-11.4	4.7	-1.6	-1.5	-1.9	7.4	-778	-451	-0.05
	22[e]	-1247	-1978	-1237	-3356	-401	-592	-348	-1298	-6.3	-5.0	-8.6	11.7	-.6	-.6	-1.5	6.1	-981	-366	-0.02
Basemat Slab	23	-23	-365	-116	-468	-57	-134	-121	-444	-.9	3.8	-3.6	7.5	-.5	.4	-3.5	4.2	-434	-300	-0.16
	25	-28	-225	-155	-777	-28	-111	-90	-464	-1.1	.7	-5.5	4.3	-.4	.2	-3.1	4.1	-383	-283	-0.21
	26	-19	-93	-129	-542	-21	-56	-40	-328	-.6	.5	-4.5	4.1	-.3	.1	-2.7	4.2	-339	-271	-0.22
Reactor Cavity	27	-80	-84	40	-42	-19	-22	63	(d)	-.5	-.5	-.2	.5	-.1	-.1	2.0	4.2	-30	-88	0.00
	28	13	-48	58	-87	11	13	31	27	.1	.2	-.4	9.2	.1	.1	-.9	7.2	-401	-360	-0.22

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Table 3.8-1B[a]

STRESS ANALYSIS RESULTS (Sheet 4 of 9)

Reference Loading Combination: $D + F + T_o + P_v$ (8) from Table 3.8-1A)

Portion	Section (Shown in Figure 3.8-22A)	Concrete Stresses – Meridional – Primary – MEM (psi)	Concrete Stresses – Meridional – Primary – MEM & BEM (psi)	Concrete Stresses – Meridional – Primary and Secondary – MEM (psi)	Concrete Stresses – Meridional – Primary and Secondary – MEM & BEM (psi)	Concrete Stresses – Hoop – Primary – MEM (psi)	Concrete Stresses – Hoop – Primary – MEM & BEM (psi)	Concrete Stresses – Hoop – Primary and Secondary – MEM (psi)	Concrete Stresses – Hoop – Primary and Secondary – MEM & BEM (psi)	Reinforcement Stresses – Meridional – Primary – Inside ksi	Reinforcement Stresses – Meridional – Primary – Outside ksi	Reinforcement Stresses – Meridional – Primary and Secondary – Inside ksi	Reinforcement Stresses – Meridional – Primary and Secondary – Outside ksi	Reinforcement Stresses – Hoop – Primary – Inside ksi	Reinforcement Stresses – Hoop – Primary – Outside ksi	Reinforcement Stresses – Hoop – Primary and Secondary – Inside ksi	Reinforcement Stresses – Hoop – Primary and Secondary – Outside ksi	Liner Strains[b] – Meridional $\times 10^{-6}$ in/in	Liner Strains[b] – Hoop $\times 10^{-6}$ in/in	Deflection[c] (Primary Loads) (in)
Allowable	Shell	-1800	-2700	-2700	-3600	-1800	-2700	-2700	-3600	±30	±30	±40	±40	±30	±30	±40	±40	±4000	±4000	-
	Basemat	-1500	-2250	-2250	-3000	-1500	-2250	-2250	-3000											
Dome	2	-947	-983	-916	-1777	-926	-921	-888	-1762	-4.7	-4.7	-7.0	2.6	-4.8	-4.8	-7.1	2.0	-403	-375	-0.63
	7	-1088	-1145	-1085	-2086	-877	-949	-899	-1875	-4.3	-3.8	-6.3	0.6	-4.6	-4.3	-7.2	2.3	-651	-429	-0.46
Wall	16	-1030	-1027	-1030	-2030	-1264	-1127	-1259	-2153	-5.2	-4.8	-8.3	1.1	-5.9	-5.7	-9.2	0.3	-433	-460	-0.18
	18	-1075	-1048	-1075	-2065	-1383	-1291	-1375	-2317	-5.2	-5.2	-8.4	1.0	-7.0	-6.7	-10.2	-0.7	-429	-462	-0.21
	20	-1120	-1241	-1120	-1769	-921	-860	-797	-1667	-5.3	-7.0	-8.3	-1.7	-4.2	-4.0	-6.4	3.0	-321	-460	-0.14
	21	-1131	-1432	-1131	-2636	-451	-518	-377	-1210	-7.6	-4.1	-11.1	5.0	-1.5	-1.4	-2.3	7.0	-784	-452	-0.05
	22[e]	-1136	-2147	-1136	-3660	-388	-601	-399	-1427	-5.9	-4.6	-8.5	12.3	-0.6	-0.6	-2.1	5.4	-1005	-367	-0.02
Basemat Slab	23	-15	-329	-88	-442	-67	-134	-191	-534	-0.6	1.3	-3.3	6.1	-0.5	0.2	-4.2	3.0	-372	-286	-0.16
	25	-6	-57	-70	-377	-55	-112	-176	-604	-0.1	0.2	-2.9	2.0	-0.7	0.1	-4.9	3.2	-193	-316	-0.20
	26	6	-46	-34	-316	-73	-142	-200	-712	-0.1	0.6	-2.3	4.8	-1.0	-0.1	-5.8	3.0	-279	-383	-0.22
Reactor Cavity	27	-57	-117	-68	-217	-23	-29	96	189	-0.6	-0.1	-1.9	0.5	-0.1	-0.1	0.5	2.9	-97	-99	0.00
	28	20	-54	-75	-261	17	31	-57	-331	-0.1	0.4	-3.1	5.3	0.1	0.2	-2.9	2.7	-355	-232	-0.24

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Table 3.8-1B[a]

STRESS ANALYSIS RESULTS (Sheet 5 of 9)[f]

Reference Loading Combinations: $D + F + T_o + E_o$ (#4 from Table 3.8-1A)

Portion	Section (Shown in Figure 3.8-22A)	Concrete Stresses								Reinforcement Stresses								Liner Strains[b]		Deflection[c] (Primary Loads)
		Meridional				Hoop				Meridional				Hoop						
		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary				
		MEM (psi)	MEM & BEM (psi)	MEM (psi)	MEM & BEM (psi)	MEM (psi)	MEM & BEM (psi)	MEM (psi)	MEM & BEM (psi)	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Meridional $\times 10^{-6}$ in/in	Hoop $\times 10^{-6}$ in/in	(in)
Allowable	Shell	-1800	-2700	-2700	-3600	-1800	-2700	-2700	-3600	±30	±30	±40	±40	±30	±30	±40	±40	±4000	±4000	-
	Basemat	-1500	-2250	-2250	-3000	-1500	-2250	-2250	-3000											
Dome	2	-907	-930	-869	-1859	-887	-880	-869	-1868	-5.1	-5.5	-9.6	.6	-5.1	-5.2	-9.9	.3	-425	-419	-
	7	-1049	-1065	-1045	-2054	-826	-858	-844	-1814	-6.3	-6.0	-10.5	-.6	-5.0	-4.6	-8.6	.6	-433	-416	-
Wall	16	-969	-986	-969	-2167	-1179	-1091	-1174	-2274	-5.8	-5.6	-10.6	-.1	-6.5	-6.4	-10.7	-.5	-480	-496	-
	18	-925	-942	-925	-2107	-1309	-1279	-1302	-2448	-5.4	-5.6	-10.2	.2	-7.7	-7.5	-12.0	-1.7	-473	-500	-
	20	-899	-1062	-889	-1694	-882	-870	-757	-1808	-4.6	-6.0	-8.0	-1.6	-5.0	-5.2	-7.7	1.3	-312	-439	-
	21	-880	-1211	-880	-2839	-425	-489	-307	-1439	-6.3	-3.6	-11.1	9.5	-2.6	-1.9	-4.0	8.0	-995	-580	-
	22[e]	-877	-1346	-877	-3617	-347	-465	-354	-1526	-6.3	-3.0	-12.2	23.7	-2.3	-1.3	-4.5	7.6	-1740	-587	-
Basemat Slab	23	9	-295	-69	-723	-49	-233	-177	-782	-1.1	5.8	-3.6	7.8	-1.2	1.5	-4.4	4.6	-448	-351	-
	25	24	-329	-48	-824	-38	-254	-161	-908	-1.2	6.7	-4.0	10.0	-1.3	1.8	-5.1	5.8	-548	-426	-
	26	31	-337	-14	-673	-57	-272	-191	-1085	-1.2	7.4	-3.0	9.6	-1.5	1.3	-6.1	7.1	-495	-518	-
Reactor Cavity	27	-34	-249	-44	-448	-1	-25	105	(d)	-1.1	2.5	-1.8	5.6	-.1	.4	6.5	4.9	-302	63	-
	28	48	-174	-36	-687	61	(d)	-39	-526	.9	10.6	-1.8	12.1	4.5	8.1	-1.6	8.2	-584	-410	-

Table 3.8-1B[(a)]

STRESS ANALYSIS RESULTS (Sheet 6 of 9)[(f)]

Reference Loading Combination: $D + F + T_o + E_{ss}$ (all from Table 3.8-1A)

Portion	Section (Shown in Figure 3.8-22A)	Concrete Stresses								Reinforcement Stresses								Liner Strains[(b)]		Deflection[(c)] (Primary Loads)
		Meridional				Hoop				Meridional				Hoop						
		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary				
		MEM (psi)	MEM & BEN (psi)	MEM (psi)	MEM & BEN (psi)	MEM (psi)	MEM & BEN (psi)	MEM (psi)	MEM & BEN (psi)	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Meridional x 10^{-6} in/in	Hoop x 10^{-6} in/in	(in)
Allowable	Shell	-3600	-4500	-4500	-5100	-3600	-4500	-4500	-5100	±54	±54	±54	±54	±54	±54	±54	±54	±10000	±10000	-
	Basemat	-3000	-3750	-3750	-4250	-3000	-3750	-3750	-4250											
Dome	2	-901	-924	-863	-1855	-879	-873	-861	-1862	-5.1	-5.5	-9.5	.6	-5.1	-5.2	-9.9	.4	-426	-420	-
	7	-1040	-1056	-1036	-2046	-810	-841	-828	-1803	-6.3	-5.9	-10.4	-.5	-4.9	-4.5	-8.6	.8	-434	-418	-
Wall	16	-943	-965	-943	-2158	-1151	-1065	-1146	-2234	-5.7	-5.5	-10.4	.2	-6.4	-6.2	-10.6	-.3	-489	-497	-
	18	-840	-859	-840	-2068	-1299	-1269	-1292	-2430	-4.8	-5.1	-9.7	1.2	-7.7	-7.5	-12.8	-1.6	-499	-499	-
	20	-747	-1010	-747	-1475	-870	-856	-745	-1807	-3.4	-5.5	-6.8	-1.1	-4.9	-5.1	-7.6	1.5	-276	-445	-
	21	-727	-1064	-727	-3225	-405	-474	-286	-1460	-5.3	-2.7	-9.5	23.6	-2.5	-1.8	-3.8	9.0	-1600	-621	-
	22[(e)]	-717	-1192	-717	-3950	-321	-442	-328	-1544	-5.7	-2.1	-10.1	40.0	-2.2	-1.1	-4.3	8.7	-2430	-636	-
Basemat Slab	23	19	-267	-59	-706	-44	-281	-171	-827	-.9	6.3	-3.4	8.2	-1.4	2.7	-4.5	5.6	-456	-399	-
	25	38	-488	-33	-977	-32	-313	-155	-966	-1.7	10.1	-4.6	13.3	-1.6	3.8	-5.3	6.9	-703	-478	-
	26	63	-631	-2	-770	-82	-328	-187	-1137	-1.5	9.7	-3.4	11.9	-1.7	2.2	-6.4	8.0	-608	-565	-
Reactor Cavity	27	-12	-342	-22	-538	2	-38	109	(d)	-1.1	5.9	-1.8	9.8	-.1	.7	6.5	5.2	-441	53	-
	28	63	-258	-22	-784	86	(d)	-14	-555	1.1	14.4	-1.0	15.8	6.6	11.2	-1.2	11.2	-738	-321	-

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Table 3.8-1B[a]

STRESS ANALYSIS RESULTS (Sheet 7 of 9)

Reference Loading Combination: $D + F + 1.5P_a + T_a$ (815 from Table 3.8-1A)

Portion	Section (Shown in Figure 3.8-22A)	Concrete Stresses								Reinforcement Stresses								Liner Strains[b]		Deflection[c] (Primary Loads)
		Meridional				Hoop				Meridional				Hoop						
		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary				
		MEM (psi)	MEM & BEM (psi)	MEM (psi)	MEM & BEM (psi)	MEM (psi)	MEM & BEM (psi)	MEM (psi)	MEM & BEM (psi)	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Meridional x 10^{-6} in/in	Hoop x 10^{-6} in/in	(in)
Allowable	Shell	-3600	-4500	-4500	-5100	-3600	-4500	-4500	-5100	±54	±54	±54	±54	±54	±54	±54	±54	±10000	±10000	-
	Basemat	-3000	-3750	-3750	-4250	-3000	-3750	-3750	-4250											
Dome	2	29	-88	378	-120	31	60	120	82	.3	.1	15.0	20.5	.3	.1	19.5	31.6	-232	-458	0.96
	7	-222	-277	-218	-1519	-22	-49	-99	-778	-1.1	-.6	5.3	11.1	.2	.3	14.3	23.7	-255	-422	0.71
Wall	16	-204	-249	-204	-1572	123	(d)	-281	-1055	-1.0	-1.6	8.3	16.4	4.1	4.0	12.2	21.0	-370	-425	0.12
	18	-250	-359	-250	-1731	(g)	(d)	(g)	-408	-1.8	-.8	6.8	17.3	18.9	16.4	16.7	25.3	-481	-418	0.52
	20	-295	-319	-295	-2879	77	(d)	-138	-1768	-1.9	-1.5	7.2	20.9	4.2	4.1	8.3	17.2	-661	-435	0.15
	21	-305	-1301	-305	-987	-226	-186	-439	-2702	9.7	-7.9	8.5	-3.0	-.9	.9	.7	10.0	852	-453	-0.02
	22[e]	-318	-3410	-318	-2678	-434	-2019	-559	-2835	42.4	-10.9	35.6	-13.9	-2.8	-1.4	-1.6	6.8	3828	-385	-0.86
Basemat Slab	23	22	-578	5	-579	-51	-403	-72	-484	29.5	.7	26.4	4.6	-2.7	10.6	-6.6	12.0	1131	-681	0.08
	25	11	-817	-9	-869	-119	-913	-142	-1108	-4.4	18.2	-5.6	18.3	-5.2	9.6	-7.7	12.5	940	-790	-0.16
	26	18	-832	4	-883	-319	-946	-270	-1313	-4.4	18.2	-5.5	18.4	-5.8	5.9	-8.4	9.0	-939	-718	-0.63
Reactor Cavity	27	-323	-558	-389	-885	49	(d)	107	(d)	-3.8	6.9	-4.5	5.6	2.4	2.9	4.5	7.1	-412	-285	0.02
	28	114	-118	72	-140	112	114	71	138	.7	.9	-.1	0.0	.7	.6	-.1	7.0	-343	-333	-0.69

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Table 3.8-1B[a]

STRESS ANALYSIS RESULTS (Sheet 8 of 9)[f]

Reference Loading Combination: $D + F + 1.25P_a + T_a + 1.25E_o$ (#18 from Table 3.8-1A)

Portion	Section (Shown in Figure 3.8-22A)	Concrete Stresses								Reinforcement Stresses								Liner Strains[b]		Deflection[c] (Primary Loads)
		Meridional				Hoop				Meridional				Hoop						
		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary				
		MEM (psi)	MEM & BEM (psi)	MEM (psi)	MEM & BEM (psi)	MEM (psi)	MEM & BEM (psi)	MEM (psi)	MEM & BEM (psi)	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Meridional $\times 10^{-6}$ in/in	Hoop $\times 10^{-6}$ in/in	(in)
Allowable	Shell	-3600	-4500	-4500	-5100	-3600	-4500	-4500	-5100	±54	±54	±54	±54	±54	±54	±54	±54	±10000	±10000	-
	Basemat	-3000	-3750	-3750	-4250	-3000	-3750	-3750	-4250											
Dome	2	-119	-145	18	-1334	-113	-133	-52	-1162	-.6	-.8	2.4	50.8	-.5	-.7	-1.4	28.6	-2034	-1227	-
	7	-350	-404	-344	-2173	-130	-166	-203	-1642	-2.2	-1.7	-5.0	24.6	-.9	-.6	-2.5	23.3	-1332	-1155	-
Wall	16	-293	-345	-293	-2762	-76	-97	80	-1722	-1.9	-1.5	-1.2	44.1	-.5	-.3	.4	28.2	-2167	-1345	-
	18	-227	-258	-227	-2639	64	(d)	73	-1611	-1.2	-1.4	-2.4	35.5	10.3	6.9	3.1	35.7	-1734	-1578	-
	20	-170	-238	-170	-3144	-115	-135	-260	-2434	-1.2	-.7	-.6	52.3	-.7	-.5	-2.3	31.1	-2566	-1627	-
	21	-160	-1331	-160	-1748	-229	-296	-464	-2527	12.5	-3.9	1.3	31.6	-1.0	-1.6	-6.5	18.5	-1467	-1217	-
	22[e]	-153	-3264	-153	-1397	-236	-429	-517	-2486	59.8	-8.1	14.3	-3.9	-.7	-2.2	-7.1	15.7	2854	-1110	-
Basemat Slab	23	34	-450	33	-354	-19	-532	-55	-825	17.3	1.9	12.4	1.2	-1.9	11.2	-3.4	14.7	606	-711	-
	25	38	-1088	11	-1261	-84	-892	-112	-1270	-4.6	19.6	-5.7	20.8	-4.6	9.0	-6.6	13.6	-1041	-794	-
	26	46	-1002	22	-1194	-172	-854	-208	-1351	-4.5	20.2	-5.2	20.5	-4.8	4.6	-7.6	10.3	-1011	-703	-
Reactor Cavity	27	-228	-512	-223	-686	145	(d)	91	(d)	-3.0	.3	-3.0	2.0	7.3	8.3	3.8	6.1	-239	-94	-
	28	117	(d)	77	-275	135	(d)	84	(d)	5.1	19.2	1.8	15.4	11.3	16.8	4.5	13.0	-590	-156	-

3

Table 3.8-1B[a]

STRESS ANALYSIS RESULTS (Sheet 9 of 9)[f]

Reference Loading Combination: $D + F + P_a + T_a + E_{ss}$ (#24 from Table 3.8-1A)

Portion	Section (Shown in Figure 3.8-22A)	Concrete Stresses								Reinforcement Stresses								Liner Strains[b]		Deflection[c] (Primary Loads)
		Meridional				Hoop				Meridional				Hoop						
		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary		Primary		Primary and Secondary				
		MEM (psi)	MEM & BEM (psi)	MEM (psi)	MEM & BEM (psi)	MEM (psi)	MEM & BEM (psi)	MEM (psi)	MEM & BEM (psi)	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Inside ksi	Outside ksi	Meridional $x 10^{-6}$ in/in	Hoop $x 10^{-6}$ in/in	(in)
Allowable	Shell	-3600	-4500	-4500	-5120	-3600	-4500	-4500	-5100	±54	±54	±54	±54	±54	±54	±54	±54	±10000	±10000	-
	Basemat	-3000	-3750	-3750	-4250	-3000	-3750	-3750	-4250											
Dome	2	-272	-290	-153	-1977	-262	-279	-210	-1735	-1.4	-1.7	-3.1	40.3	-1.4	-1.6	-5.3	24.5	-1026	-1219	-
	7	-482	-328	-476	-2476	-257	-292	-324	-2066	-3.0	-2.5	-8.1	22.6	-1.6	-1.3	-4.7	23.7	-1244	-1275	-
Wall	16	-410	-459	-410	-3160	-278	-278	-153	-2195	-2.5	-2.1	-5.0	44.2	-1.5	-1.4	-3.3	24.1	-2292	-1320	-
	18	-307	-337	-307	-3165	-203	-214	-196	-2304	-1.6	-1.9	-3.8	41.9	-1.2	-1.0	-2.6	20.5	-2094	-1500	-
	20	-214	-294	-214	-3460	-241	-247	-377	-2713	-1.0	-1.6	-1.2	54.0[h]	-1.4	-1.3	-4.4	29.4	-2676	-1647	-
	21	-194	-617	-194	-2904	-214	-242	-495	-2577	.7	-2.7	-.1	50.7	-1.0	-1.3	-7.0	19.7	-2460	-1392	-
	22[e]	-184	-2610	-184	-347	-172	-315	-540	-2612	30.9	-6.0	-1.5	-.4	-.9	-1.6	-7.5	16.9	2174	-3186	-
Basemat Slab	23	57	-439	22	-219	-23	-471	-87	-797	17.5	2.0	9.1	1.1	-1.0	9.1	-3.7	10.0	609	-569	-
	25	46	-834	18	-1135	-38	-693	-101	-1125	-3.3	18.3	-5.0	19.2	-3.3	8.4	-5.8	11.8	-950	-693	-
	26	45	-830	35	-1109	-89	-848	-160	-1230	-3.3	16.1	-4.7	19.9	-4.4	8.1	-6.7	10.7	-960	-688	-
Reactor Cavity	27	-163	-430	-156	-764	63	(d)	78	(d)	-2.4	.7	-3.0	5.4	2.7	4.1	3.1	5.3	-376	-92	-
	28	111	-230	86	-434	135	(d)	100	(d)	3.0	20.5	.6	16.9	10.4	17.5	5.3	17.1	-739	-496	-

3

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Table 3.8-4B

CONTAINMENT INTERNAL STRUCTURES SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS (Sheet 1 of 3)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number	Combined Stress Ratio (<1.0)
W 18 X 35 Beam	El. 100'-0" @ Column #1	2(a)	0.94
W 21 X 55 Beam	El. 100'-0" between Columns #2 and #3	2(a)	0.85
W 33 X 130 Beam	El. 100'-0" between Columns #3 and #4	2(a)	0.38
W 24 X 84 Beam	El. 100'-0" between Columns #4 and #5	2(a)	0.99
W 21 X 55 Beam	El. 100'-0" @ Column #4	2(a)	0.78
W 30 X 108 Beam	El. 100'-0" @ Column #6	2(a)	0.69
W 30 X 108 Beam	El. 100'-0" between Columns #6 and #7	2(a)	0.58
W 24 X 84 Beam	El. 100'-0" @ Column #10	2(a)	0.31
W 30 X 172 Beam	El. 100'-0" between Columns #14 and #15	2(a)	0.41
W 30 X 99 Beam	El. 100'-0" @ Column #16	2(a)	0.68
W 36 X 135 Beam	El. 120'-0"	4(b)	0.5
W 33 X 130 Beam	El. 120'-0"	4(b)	0.41
W 36 X 300 Beam	El. 120'-0" @ Column #8	4(b)	0.32

a. Refer to section 3.8.3.3.3.A(1) for description of load combination number.

b. Refer to section 3.8.3.3.3.B(1) for description of load combination number.

3

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Table 3.8-4B

CONTAINMENT INTERNAL STRUCTURES SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS (Sheet 2 of 3)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number	Combined Stress Ratio (<1.0)
W 36 X 182 Beam	El. 120'-0" @ Column #8	4[b]	0.28
W 36 X 182 Beam	El. 120'-0" @ Column #7	4[b]	0.41
W 30 X 99 Beam	El. 120'-0" between Columns #6 and #7	2[a]	0.42
W 21 X 55 Beam	El. 120'-0" @ Column #5	2[a]	0.87
W 14 X 43 Beam	El. 120'-0" @ Equipment Hatch	2[a]	0.80
W 33 X 130 Beam	El. 120'-0" between Columns #4 and #5	2[a]	0.34
W 21 X 55 Beam	El. 120'-0" between Columns #2 and #3	2[a]	0.88
W 18 X 35 Beam	El. 120'-0" between Columns #1 and #2	2[a]	0.78
W 21 X 55 Beam	El. 120'-0" between Columns #1 and #2	2[a]	0.86
W 30 X 99 Beam	El. 120'-0" @ Column #1	2[a]	0.71
W 36 X 300 Beam	El. 140'-0" between Columns #8 and #9	2[a]	0.49
W 36 X 245 Beam	El. 140'-0" between Columns #9 and #10	4[b]	0.21
W 36 X 300 Beam	El. 140'-0" between Columns #7 and #8	2[a]	0.67
W 24 X 84 Beam	El. 140'-0" @ Column #8	2[a]	0.96
W 24 X 55 Beam	El. 140'-0" @ Column #6	2[a]	0.80
W 30 X 210 Beam	El. 140'-0" between Columns #14 and #15	2[a]	0.26
W 24 X 68 Beam	El. 140'-0" between Columns #12 and #13	2[a]	0.48
W 30 X 108 Beam	El. 140'-0" @ Column #17	2[a]	0.61

Table 3.8-4B

CONTAINMENT INTERNAL STRUCTURES SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS (Sheet 3 of 3)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number	Combined Stress Ratio (<1.0)
W 24 X 31 Beam	El. 140'-0" between Columns #15 and #16	2[(a)]	0.73
W 24 X 84 Beam	El. 140'-0" @ Column #16	2[(a)]	0.78
W 24 X 55 Beam	El. 140'-0" between Columns #17 and #18	2[(a)]	0.81
W 14 X 150 Column	Column #1 between El. 100'-0" and 120'-0"	2[(a)]	0.64
W 14 X 150 Column	Column #2 between El. 96'-6" and 120'-0"	2[(a)]	0.79

3

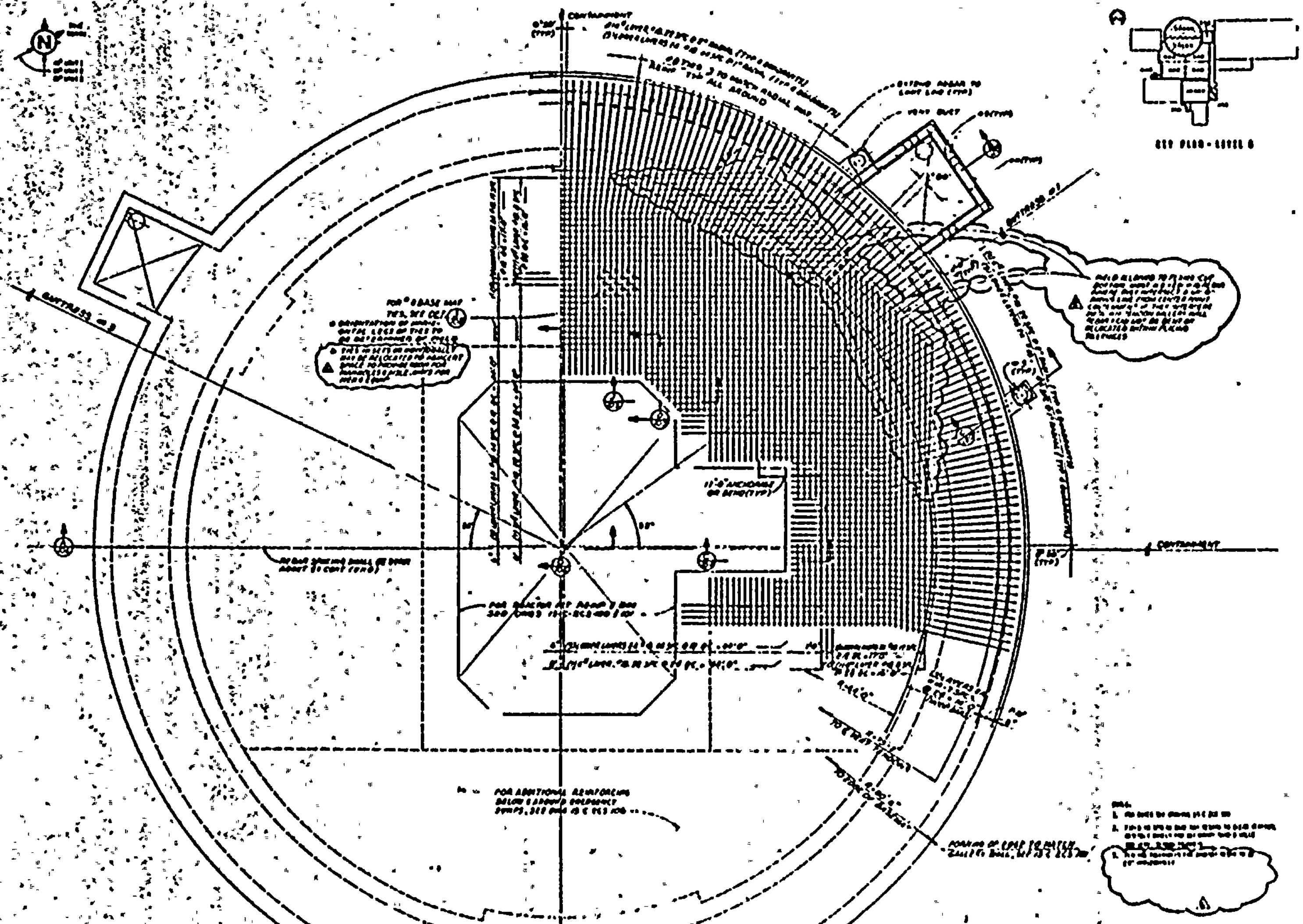
3.8.5.5 Structural Acceptance Criteria

The foundations of Seismic Category I buildings are designed to meet the same structural acceptance criteria as the buildings themselves. These criteria are discussed in sections 3.8.1.5, 3.8.3.5, and 3.8.4.5. The limiting conditions for the foundation medium, together with a comparison of actual capacity and estimated structure loads, are found
in sections 2.5.4.10 and 2.5.4.11. Computed factors of 1
safety against overturning, sliding, and flotation for
Category I structures are given in table 3.8-5. 1

Table 3.8-5
COMPUTED FACTORS OF SAFETY

Structure	Overturning		Sliding		Flotation
	OBE	SSE	OBE	SSE	
Auxiliary	3200	830	2.2	1.3	4.7
Containment	3400	1200	1.7	1.2	4.5
Control	1500	420	1.6	1.2	4.8
Diesel Generator	1200	400	2.2	1.1	NA[a]
Fuel	1600	400	1.9	1.1	NA
Main Steam Support	340	91	1.6	1.1	NA
Condensate Storage and Refueling Water Tanks	500	150	1.7	1.4	NA

a. Not applicable



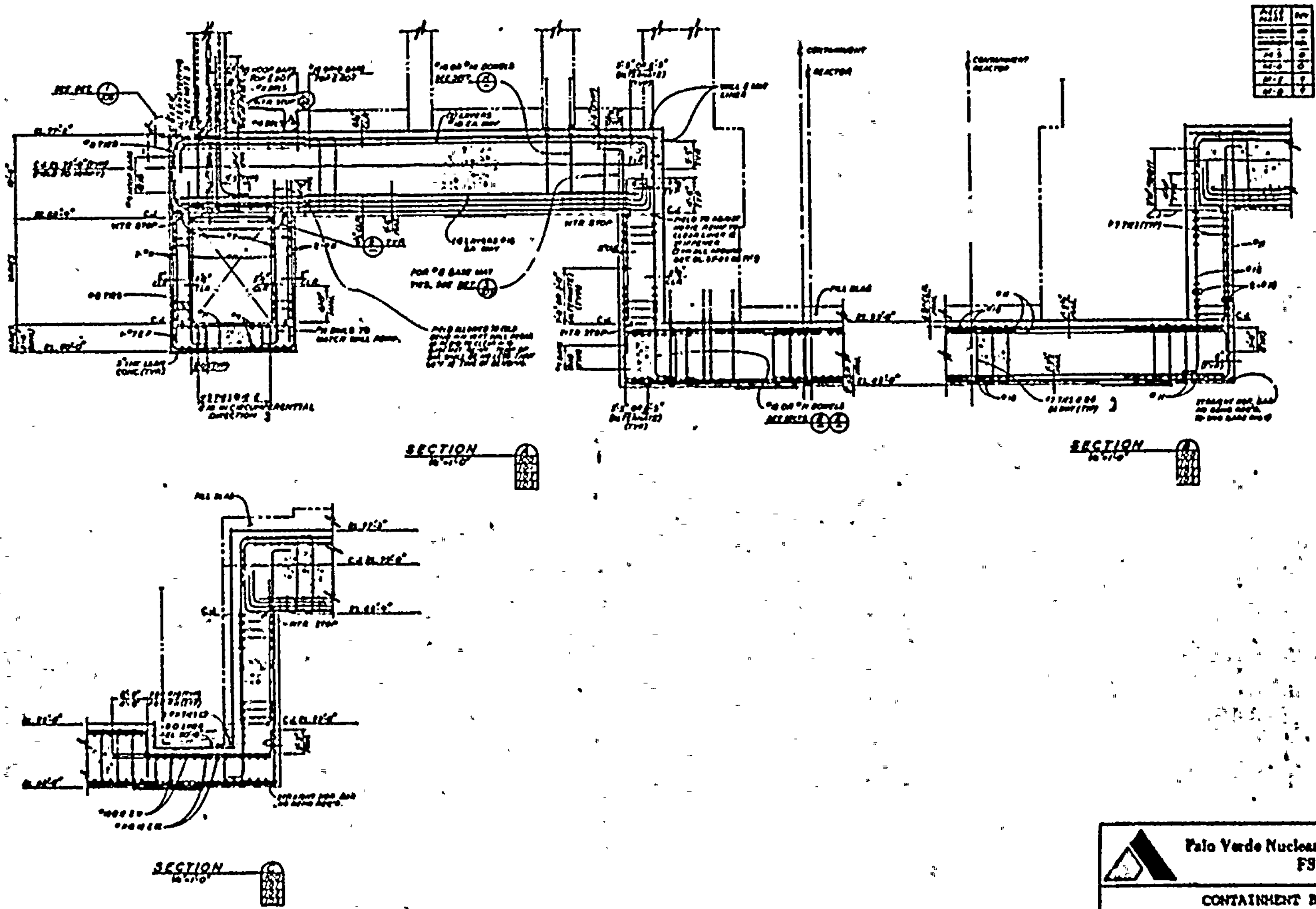
13-C-ZCS-102 REV.6

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING
BASE MAT REINF PLAN-BOTTOM LAYERS
AREAS CAA, CAB, CAC & CAD

Figure 3.8-1

112

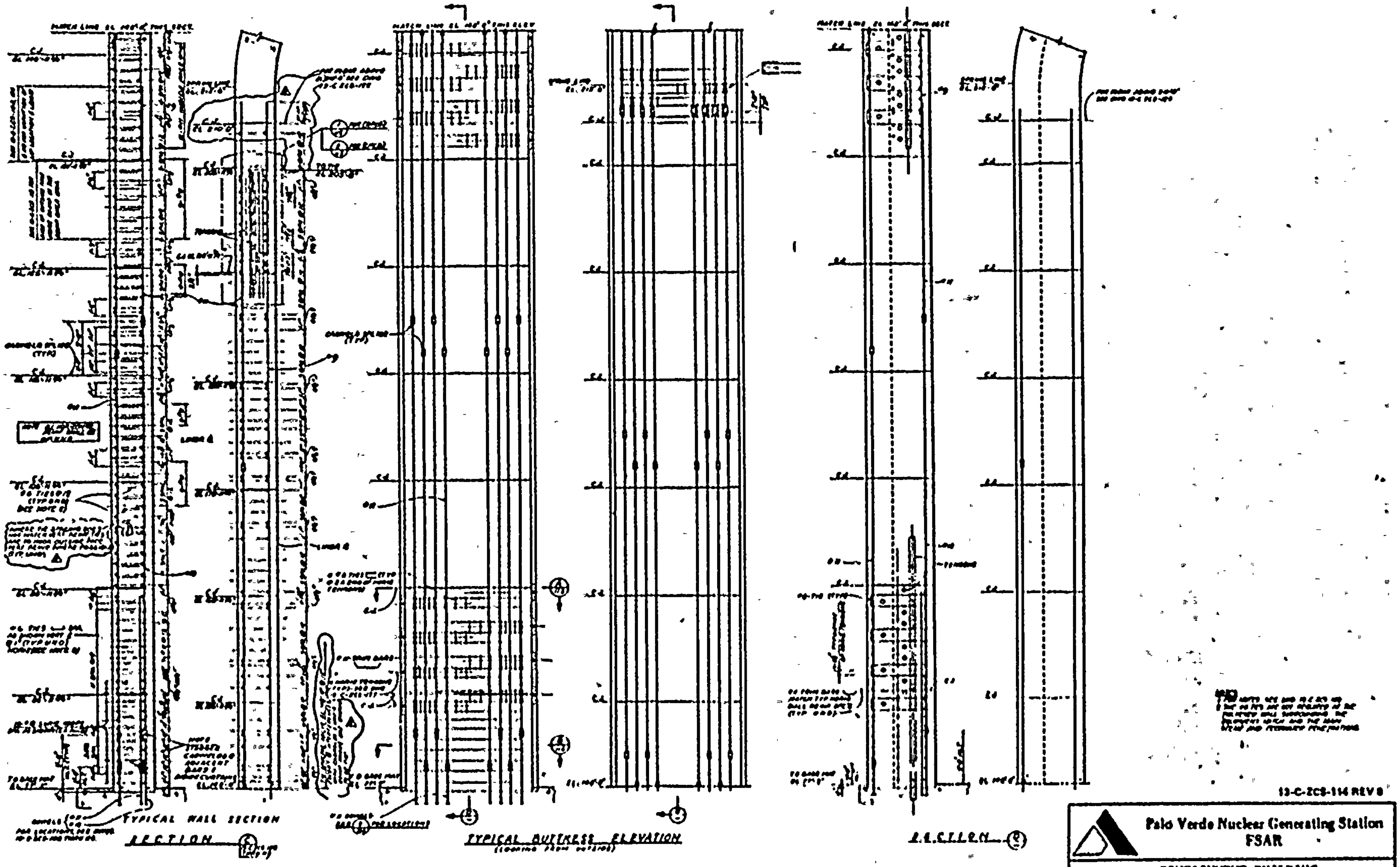


13-C-ZCS-104 REV 13

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING
BASE MAT REINFORCING
SECTIONS AND DETAILS

Figure 3.8-2



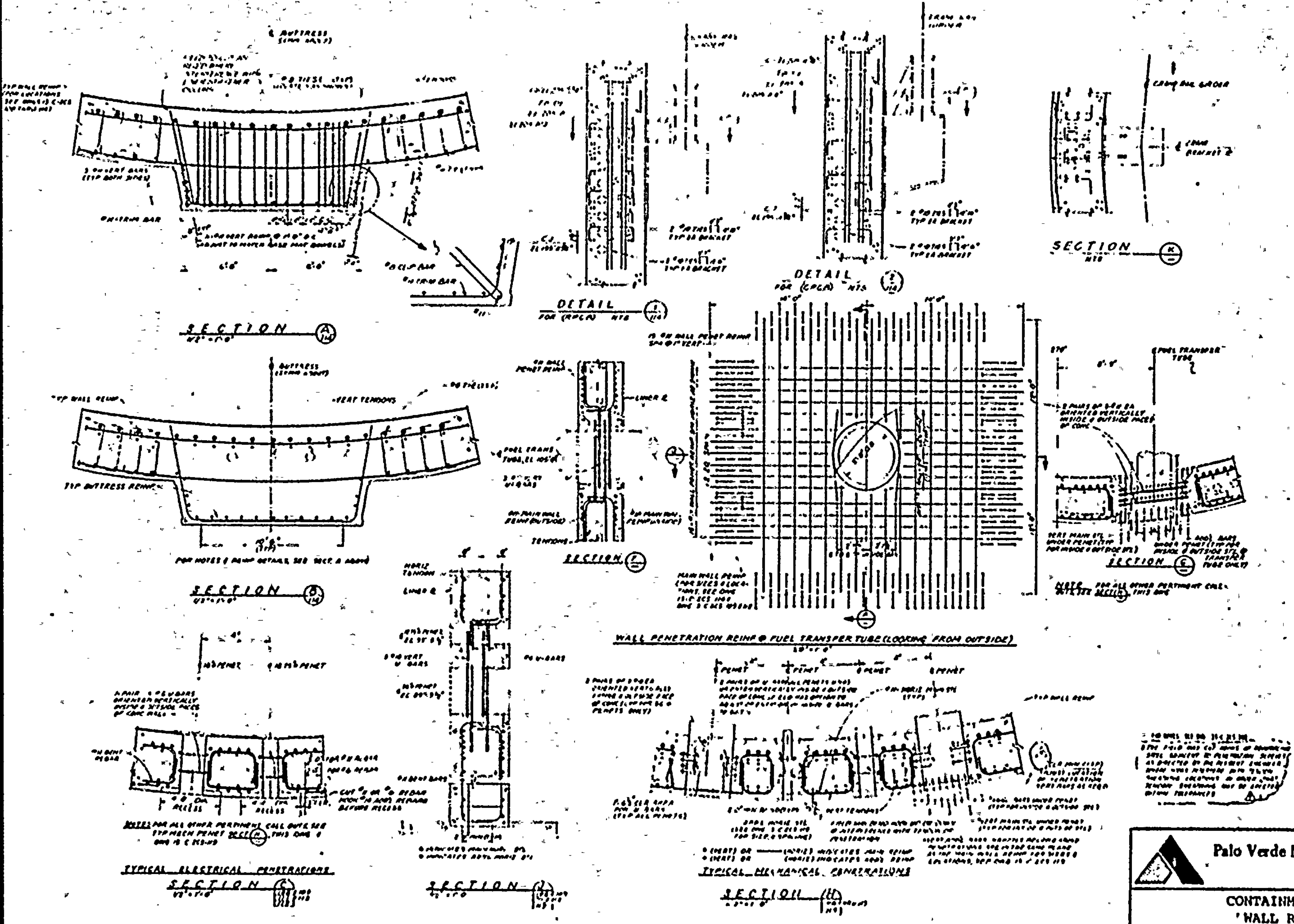
13-C-ZCS-114 REV 8

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING
WALL REINFORCING
SECTIONS AND DETAILS

Figure 3.8-4

114

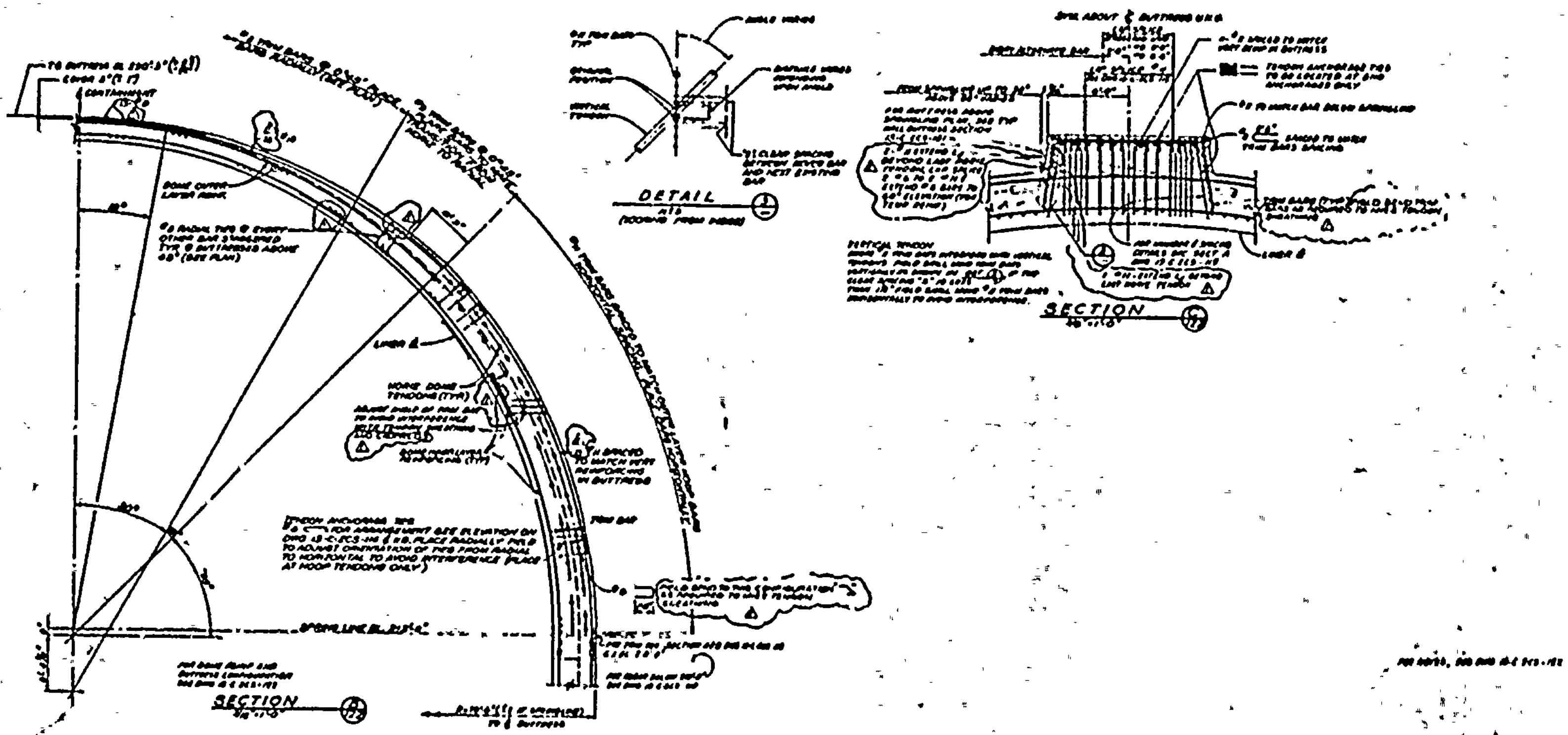


13-C-ZCS-118 F

Palo Verde Nuclear Generating Stati
FSAR

CONTAINMENT BUILDING
WALL REINFORCING
SECTIONS AND DETAILS

Figure 3.8-5

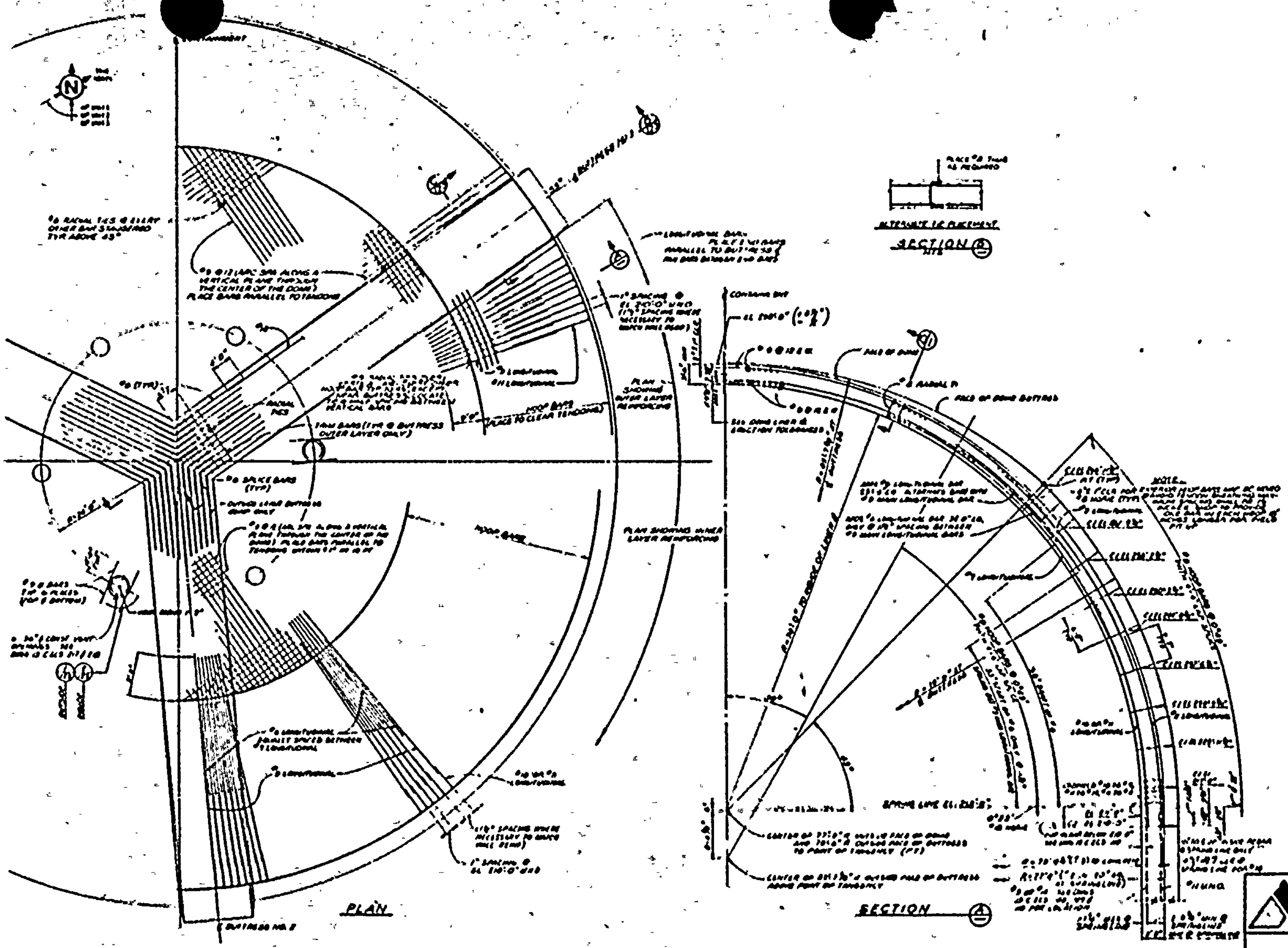


13-C-ZCS-123 REV 1

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING
DOME REINFORCING
(Sheet 1 of 2)

Figure 3.8-6

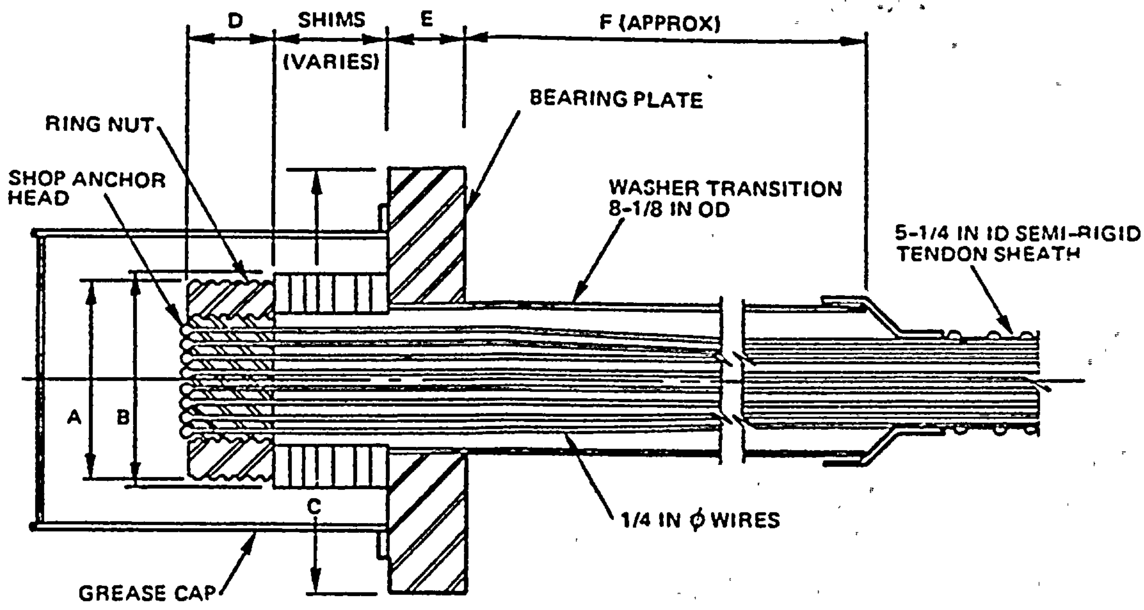


13-C-ZCS-122 REV 2

Palo Verde Nuclear Generating Station FSAR

CONTAINMENT BUILDING
DOME REINFORCING
(Sheet 2 of 2)

Figure 3.8-6



186 WIRE SYSTEM

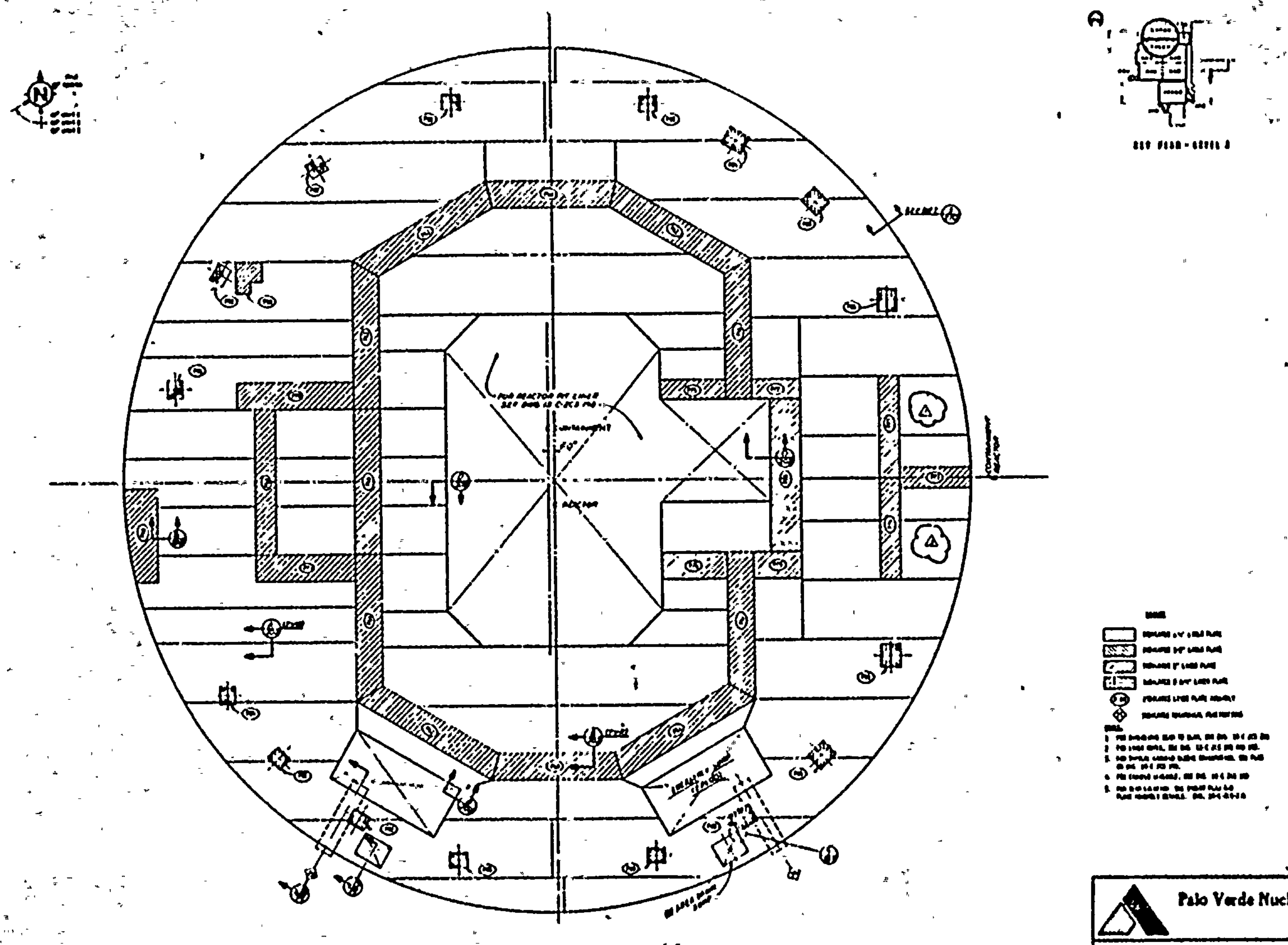
TYPE	A	B	C	D	E	F
186 WIRE	10-1/2 IN DIA	11 IN SQ	24 IN	4 IN	4-1/4 IN	5 FT 7-3/4 IN

Palo Verde Nuclear Generating Station
FSAR

TENDON END ANCHORS

Figure 3.8-7

118

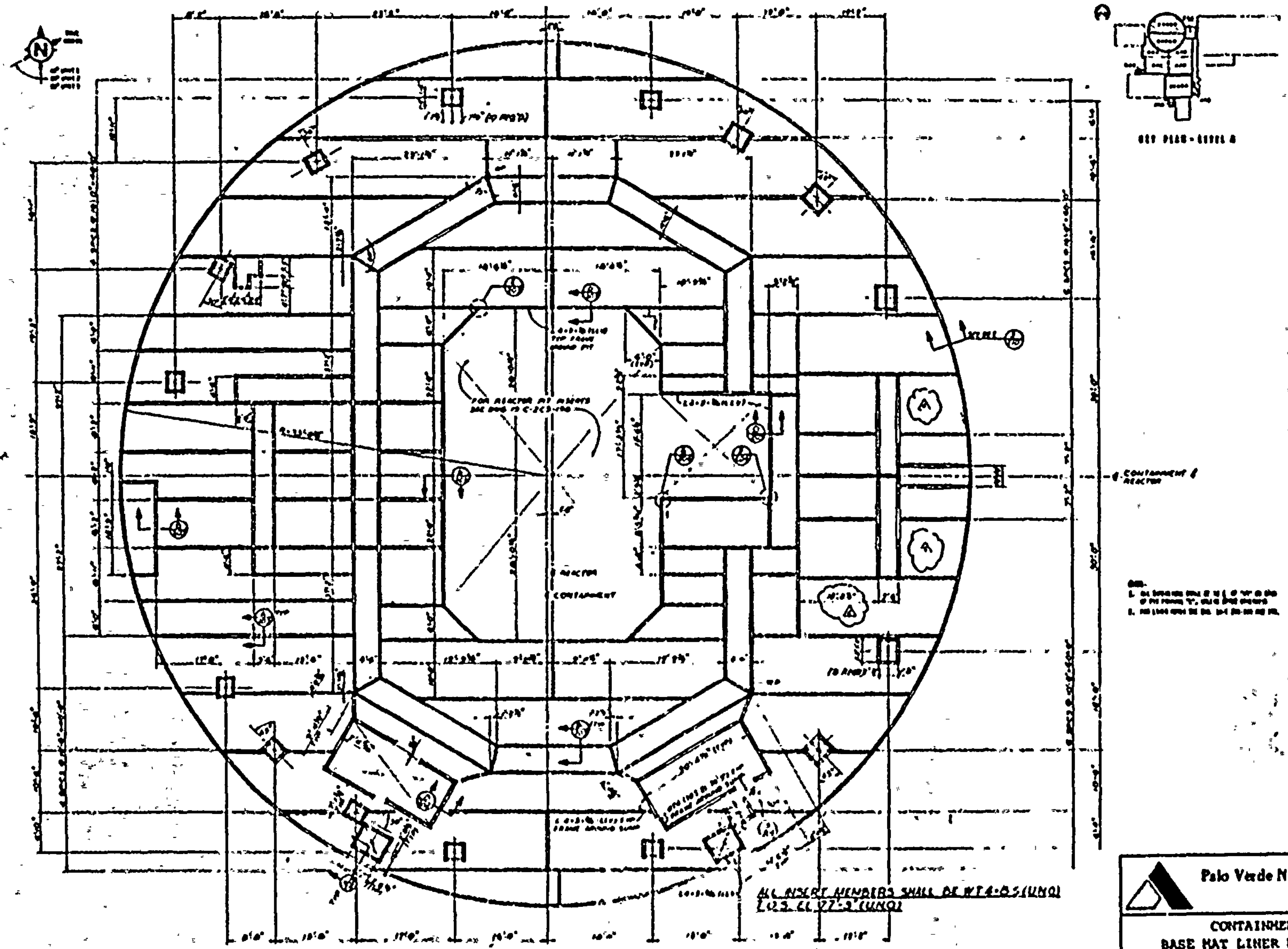


13-C-ZCS-200 REV 3

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING
BASE MAT LINER PLATE-FLOOR PLAN
AREAS CAA, CAB, CAC & CAD

Figure 3.8-12

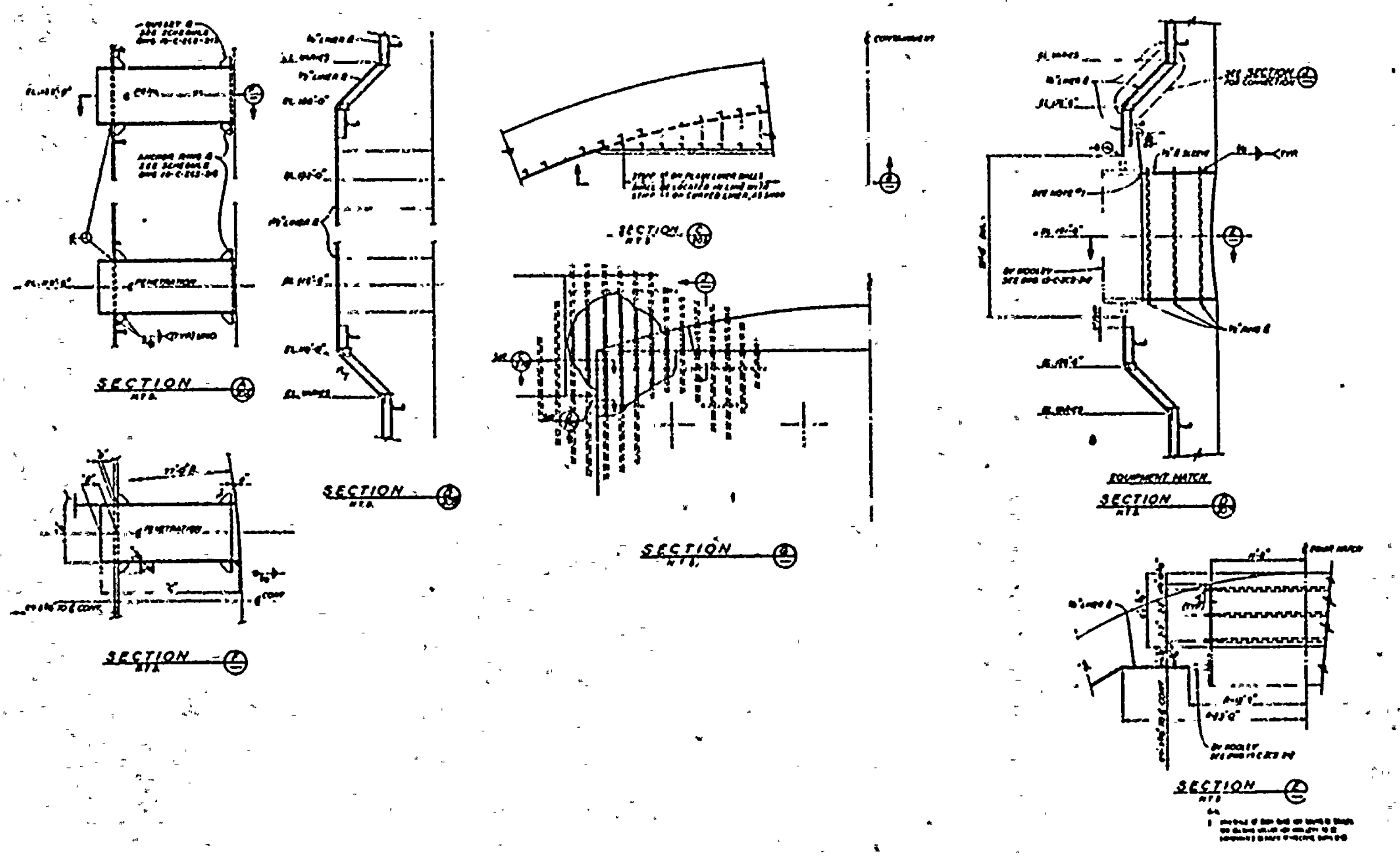


13-C-ZCS-301 REV 4

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING
BASE MAT LINER PLATE-INSERTS PLAN
AREAS CAA, CAB, CAC & CAD

Figure 3.8-13



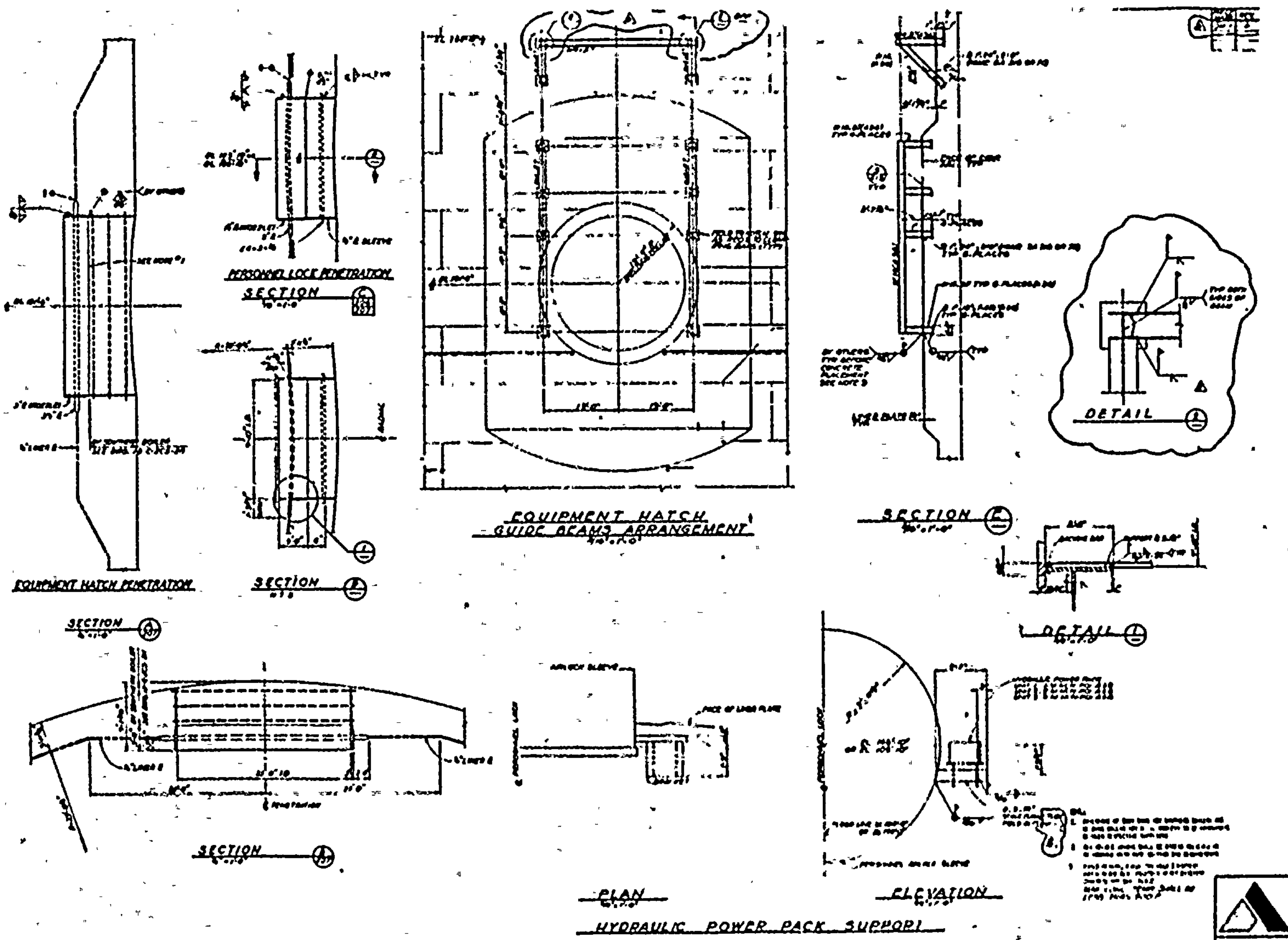
13-C-ZCS-211 REV 8

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING
WALL LINER PLATE
SECTIONS AND DETAILS

Figure 3.8-17

121



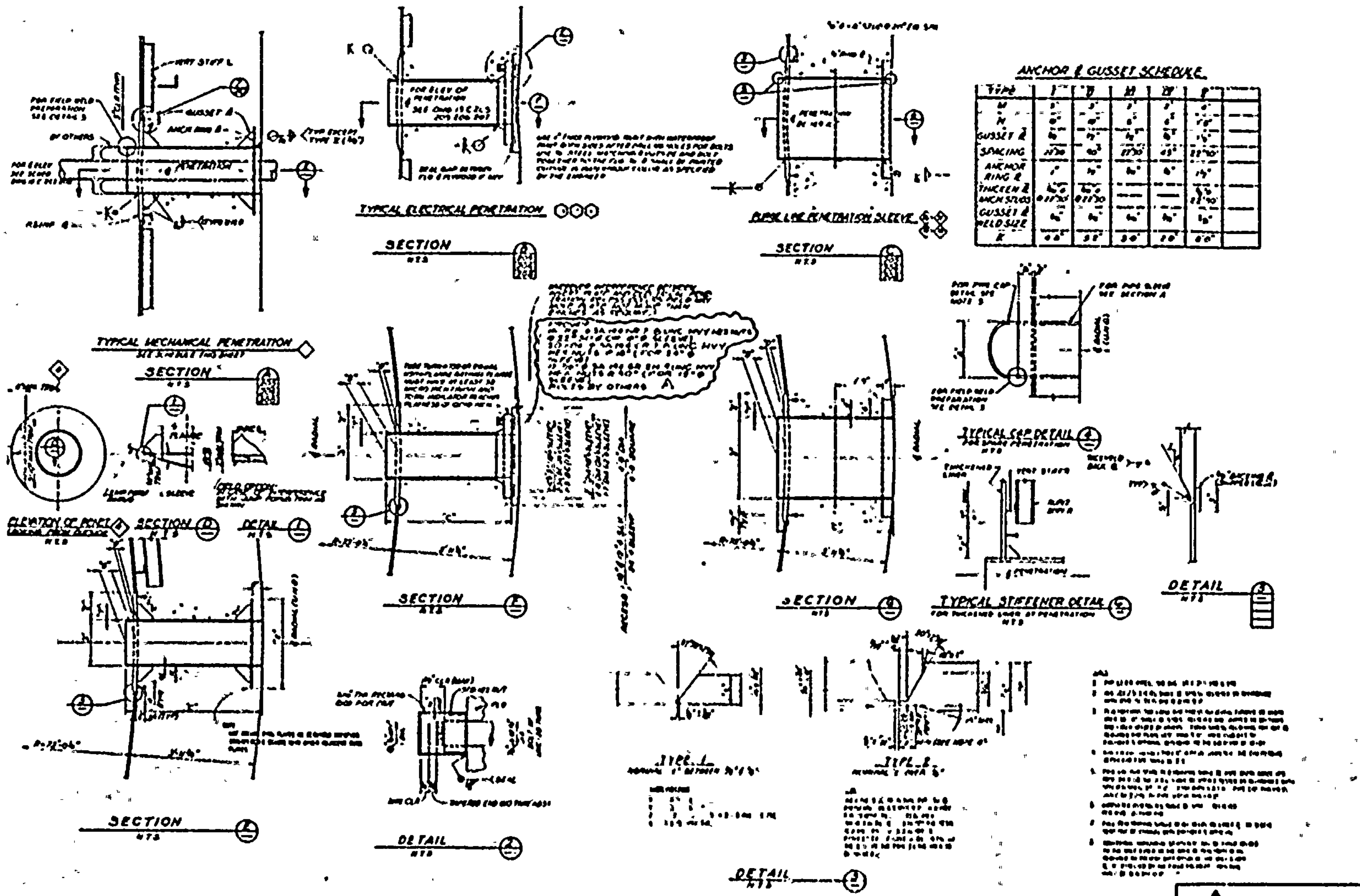
13-C-ZCS-212 REV 6

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING
WALL LINER PLATE
SECTIONS AND DETAILS

Figure 3.8-18

122

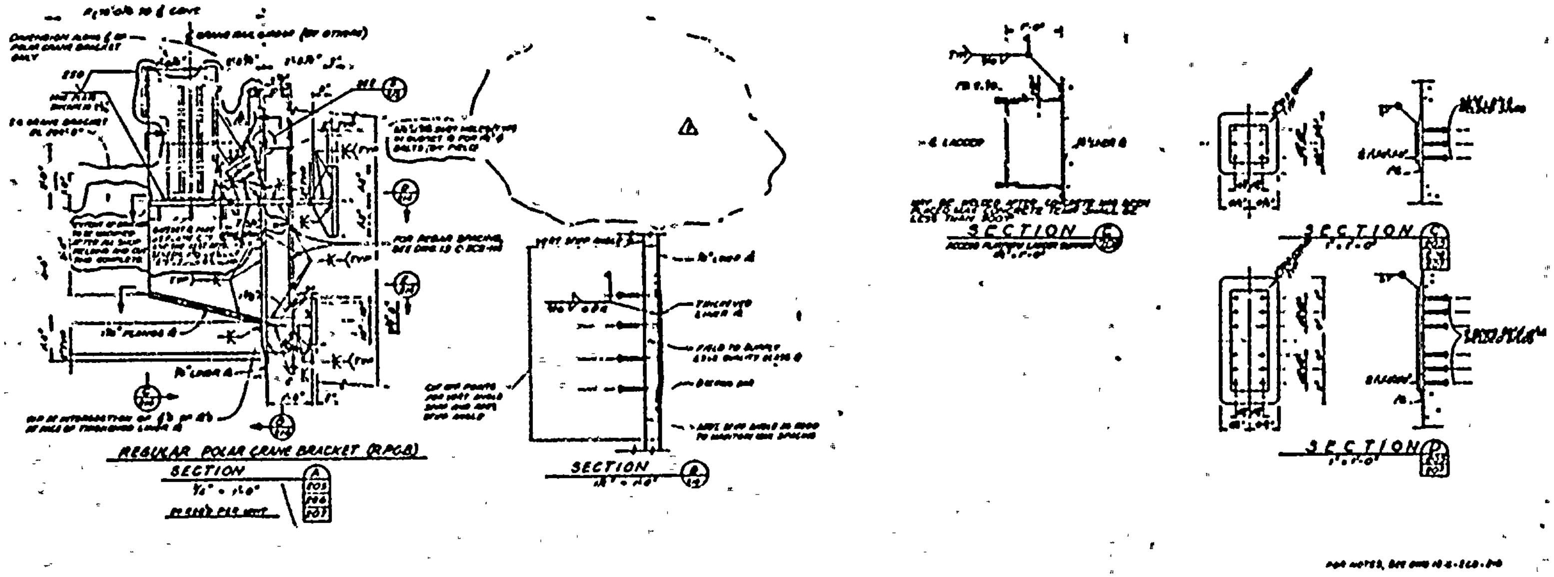


13 C-2CS-213 REV 10

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING
WALL LINER PLATE
SECTIONS AND DETAILS

Figure 3.8-19

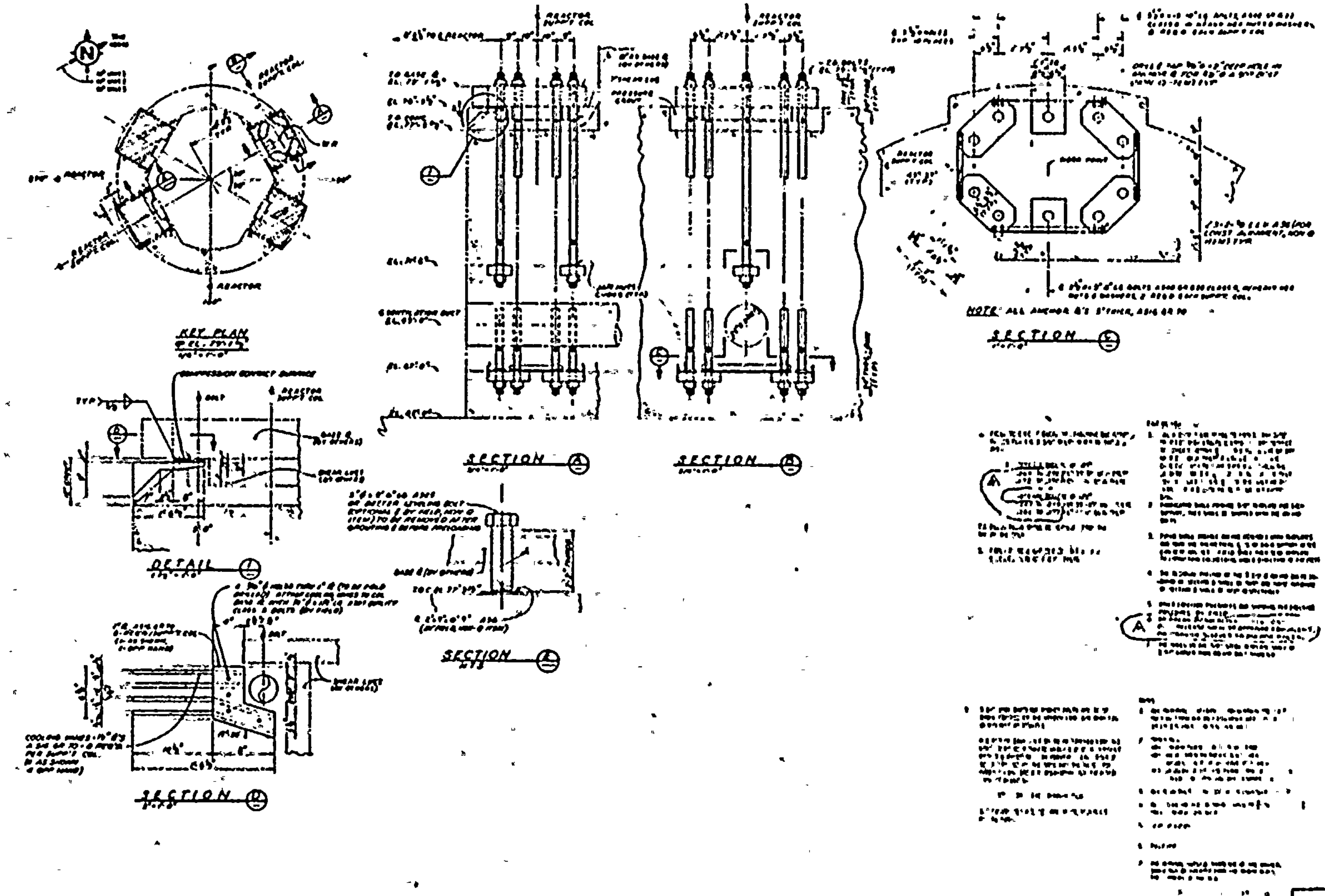


13-C-ZCS-219 REV 3

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING
WALL LINER PLATE
SECTIONS & DETAILS

Figure 3.8-20

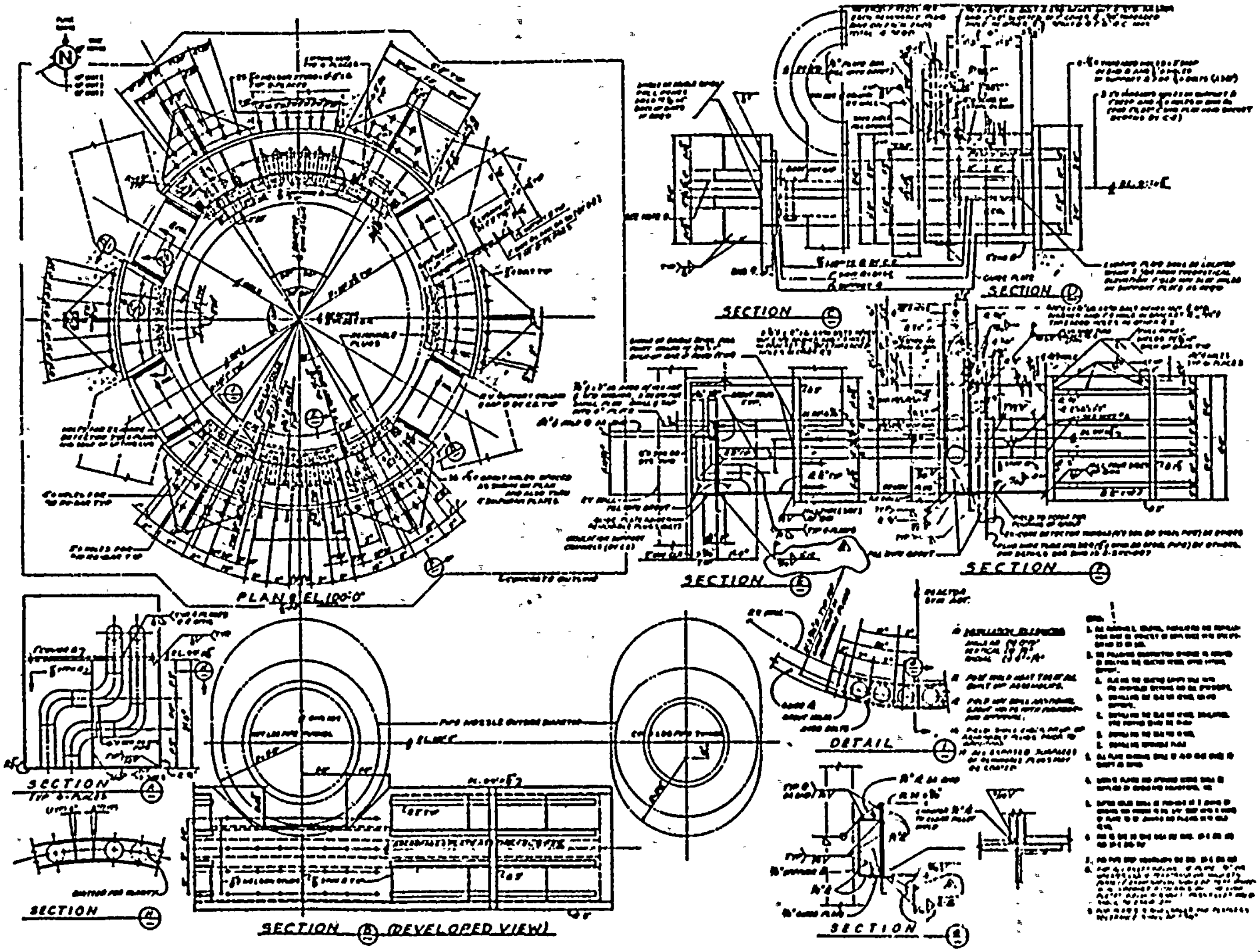


13-C ZCS 600 REV 6

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
REACTOR VESSEL SUPPORTS

Figure 3.8-23

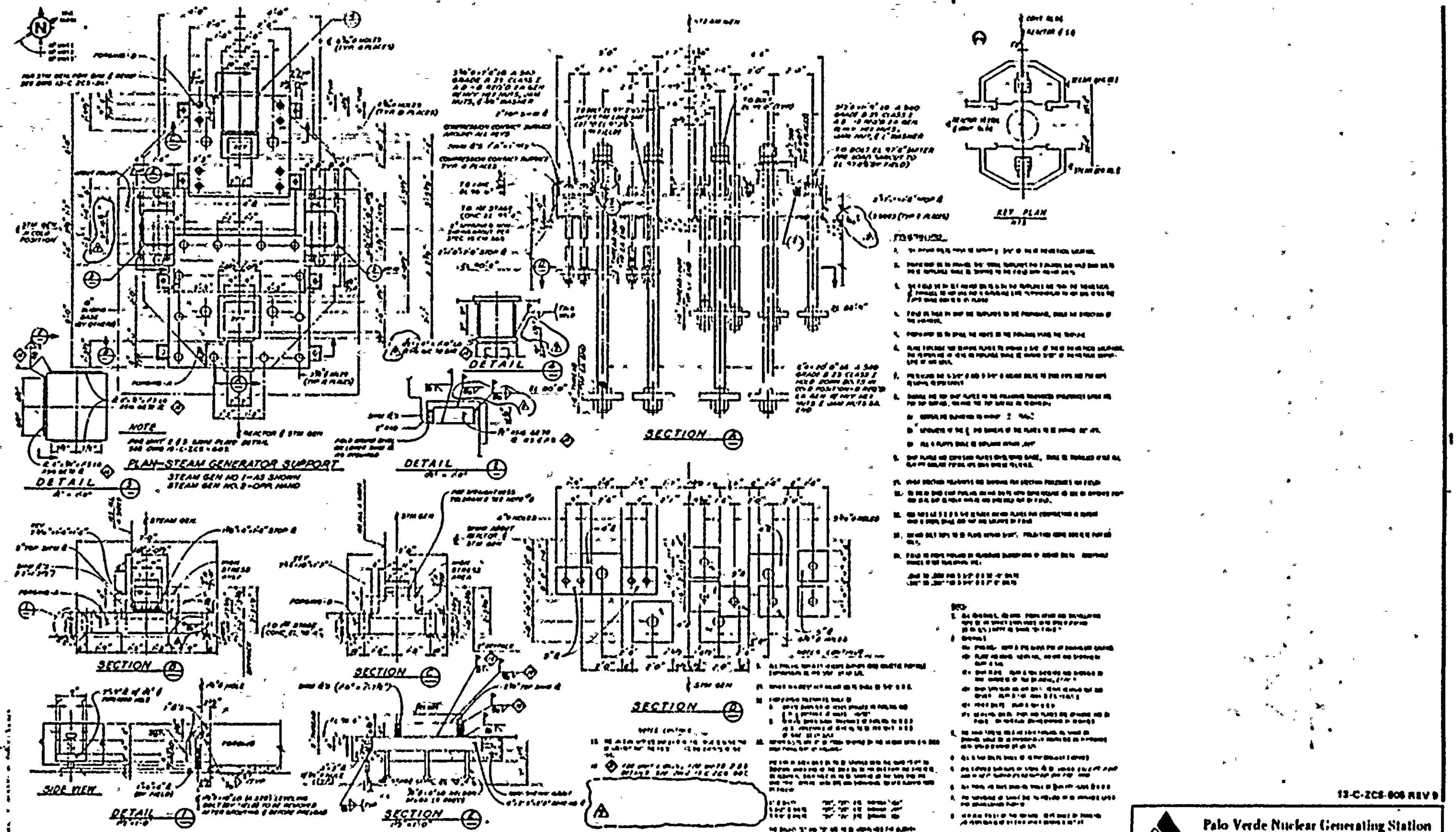


13-C-ZCS-001 REV 8

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
REACTOR VESSEL SUPPORTS

Figure 3.8-24

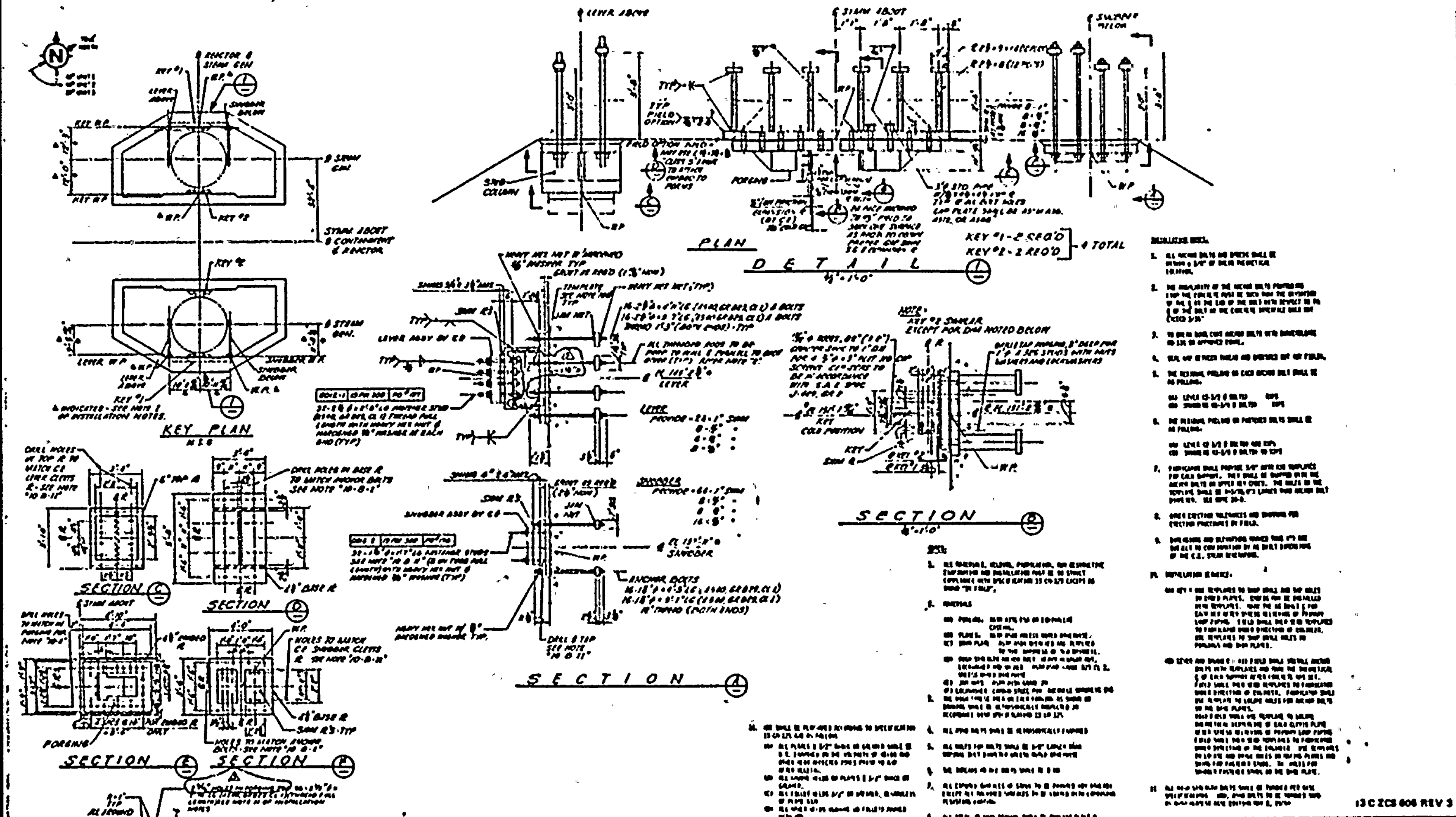


Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
STEAM GENERATOR LOWER SUPPORTS
SECTIONS AND DETAILS

Figure 3.8-25

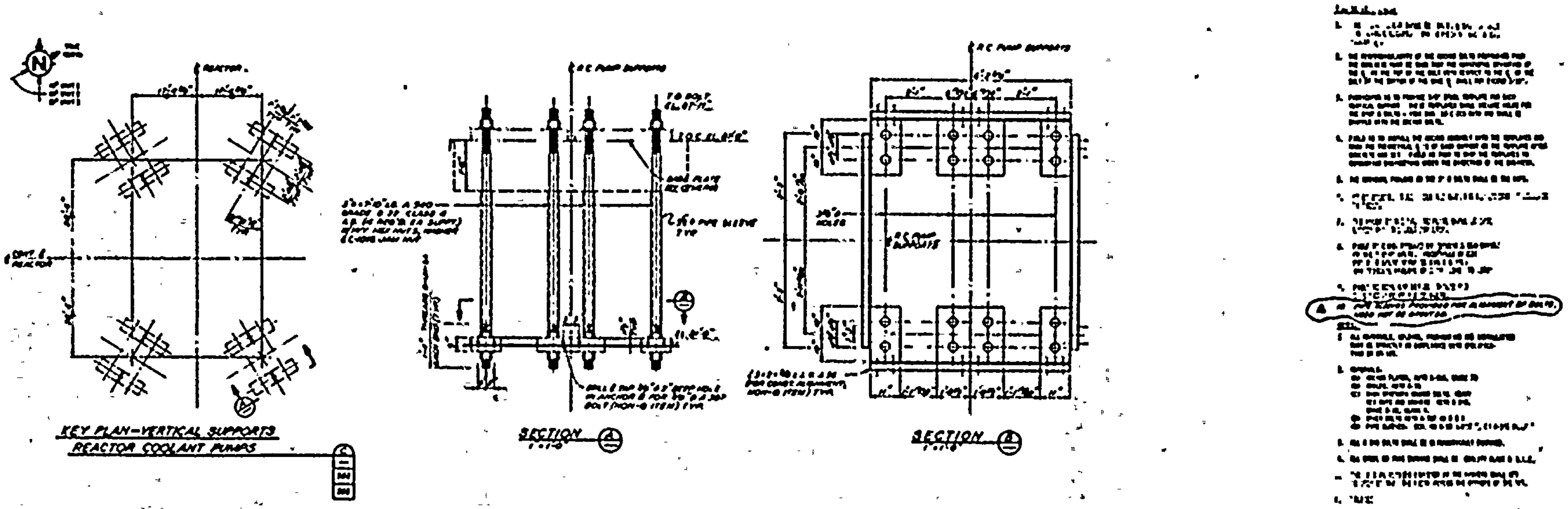
March 1980 Amendment 1



Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
STEAM GENERATOR UPPER SUPPORTS
SECTIONS AND DETAILS

Figure 3.8-26

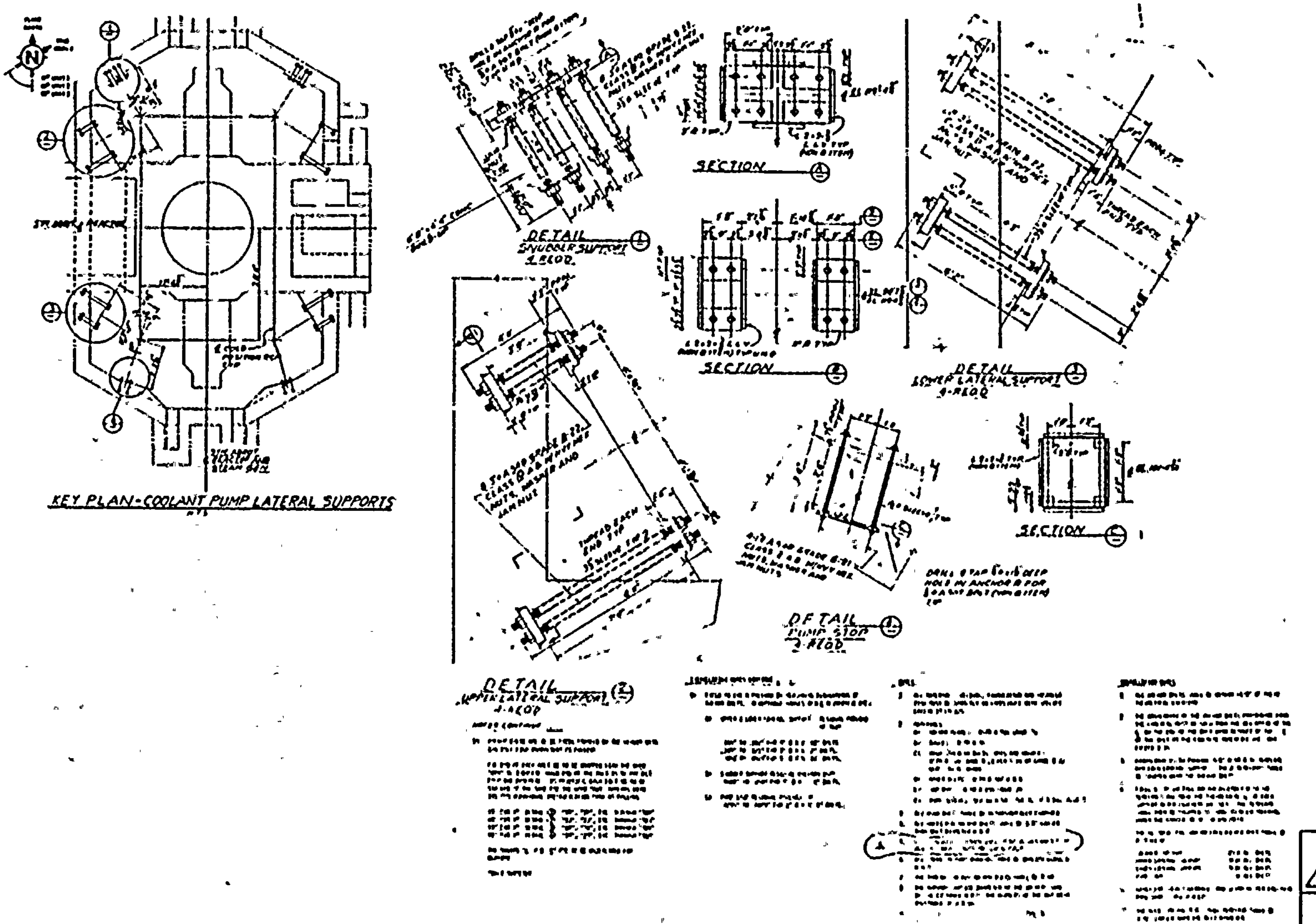


13-C-ZCS-002 REV 6

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
COOLANT PUMP SUPPORTS

Figure 3.8-27



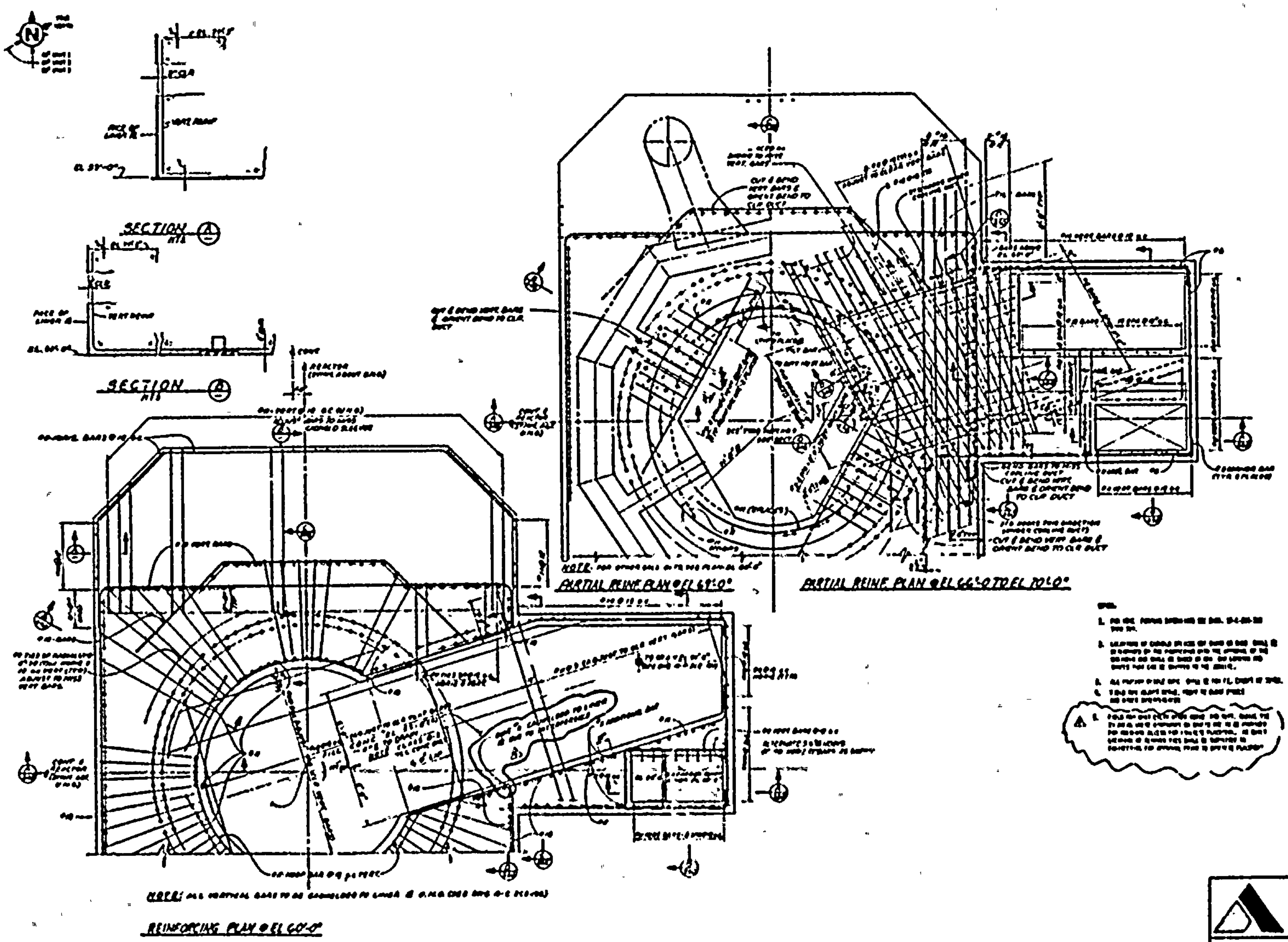
13 C ZCS 603 REV 3

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
COOLANT PUMP SUPPORTS

Figure 3.8-28

130

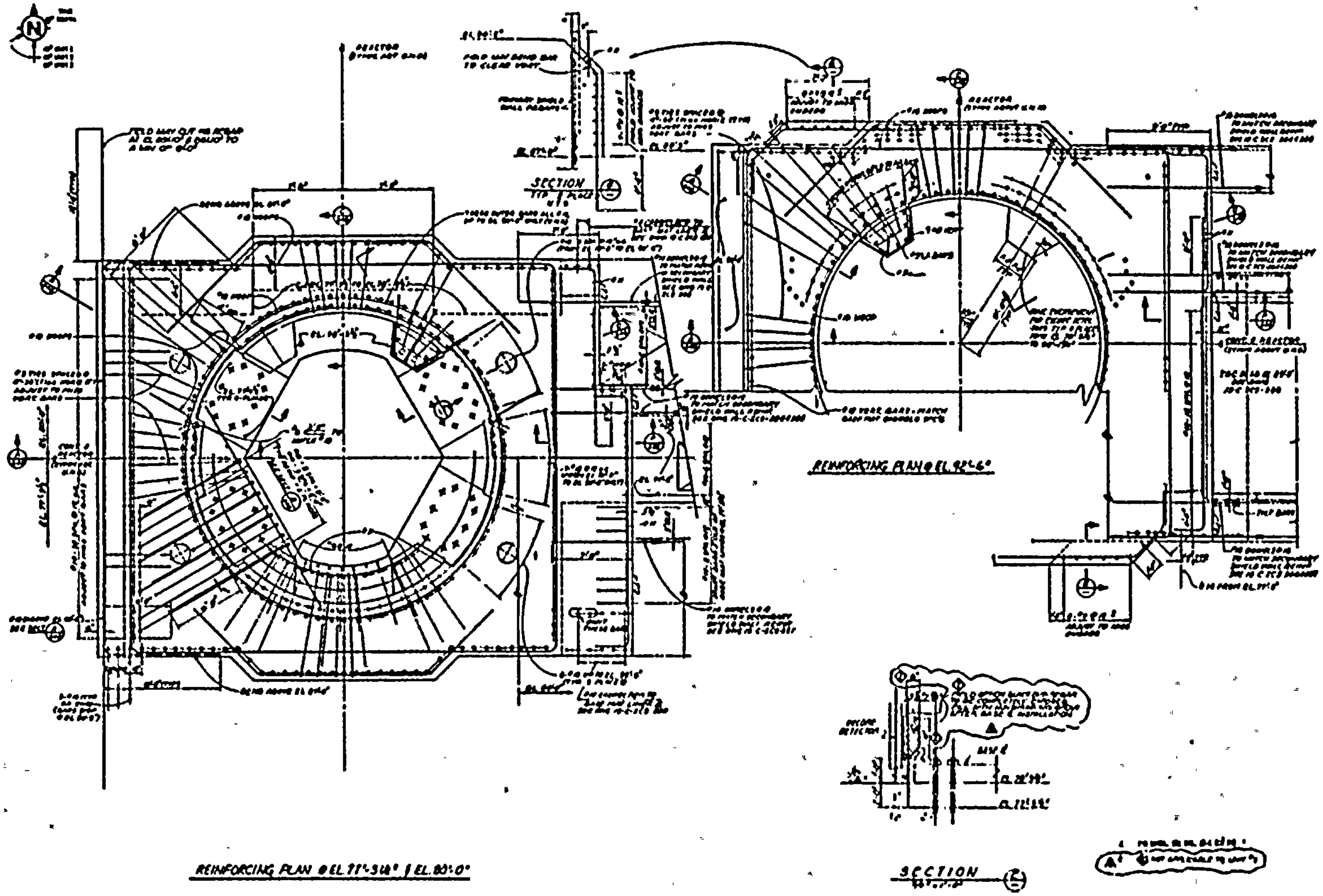


13-C-ZCS-348 REV 4

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
REINFORCED CONCRETE
PRIMARY SHIELD

Figure 3.8-30

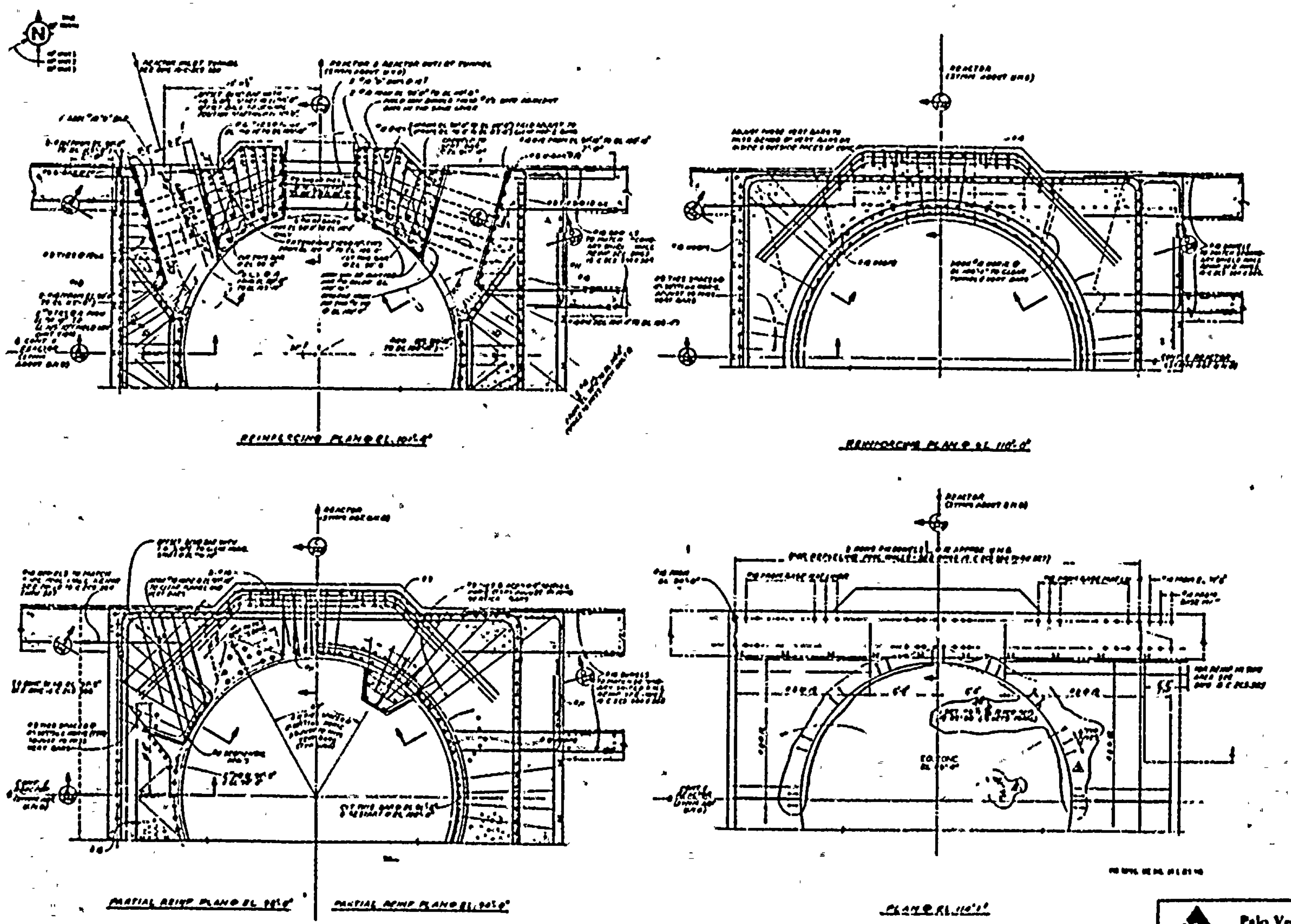


13-C-ZCS-340 REV 8

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
REINFORCED CONCRETE
PRIMARY SHIELD

Figure 3.8-31

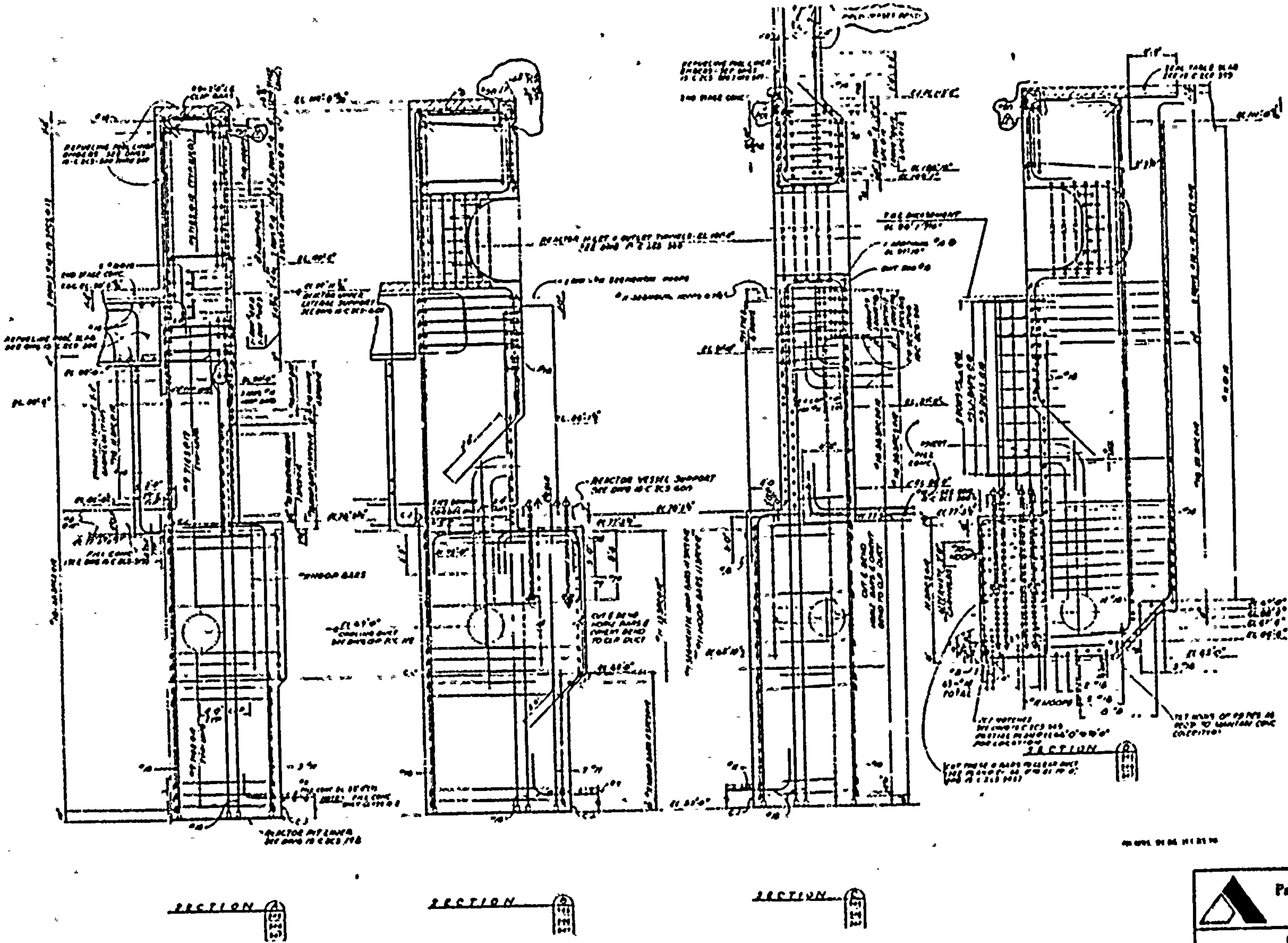


13-C-ZCS-347 REV 8

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
REINFORCED CONCRETE
PRIMARY SHIELD

Figure 3.8-32

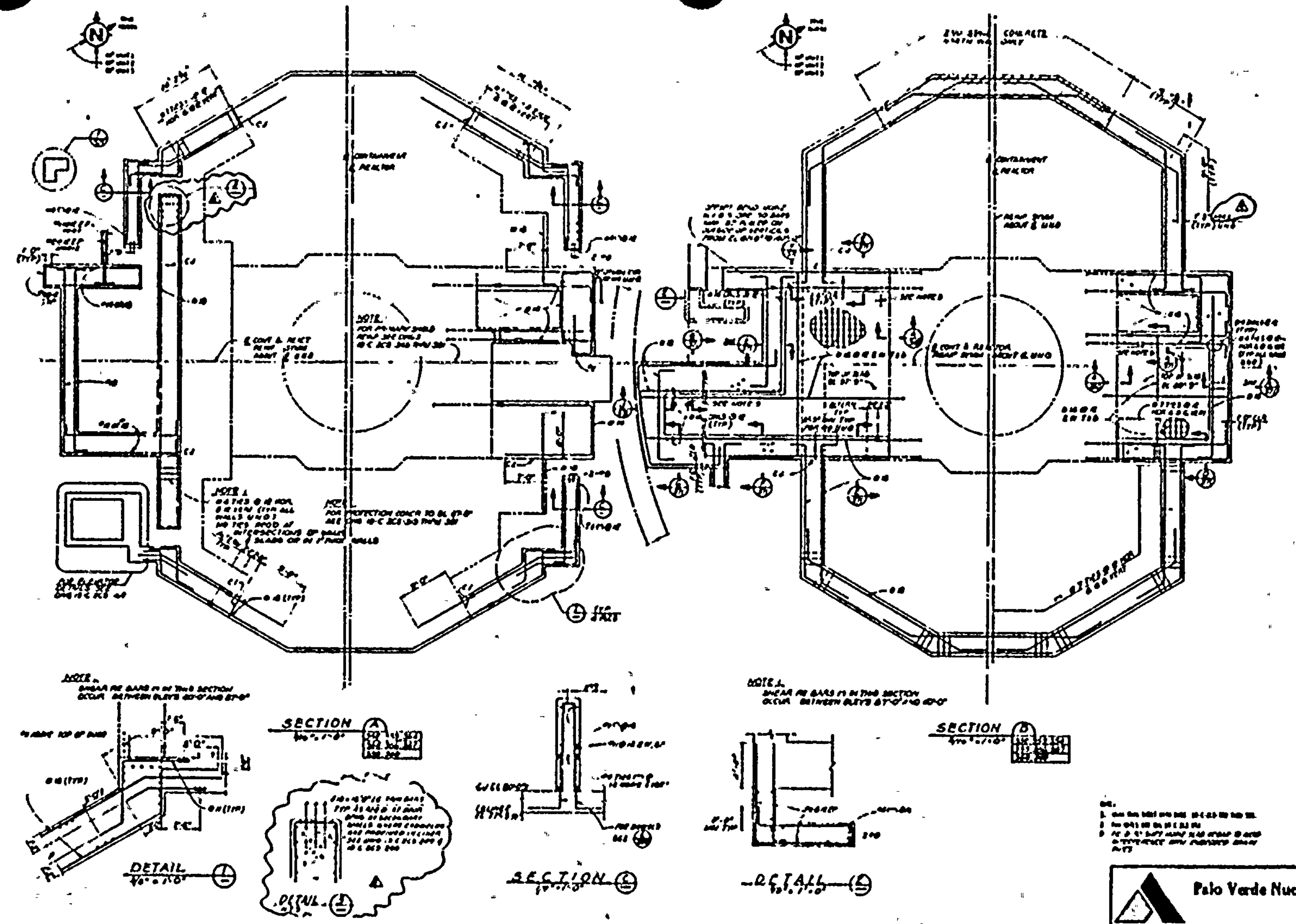


13 C ZC0 348 REV 7

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
REINFORCED CONCRETE
PRIMARY SHIELD

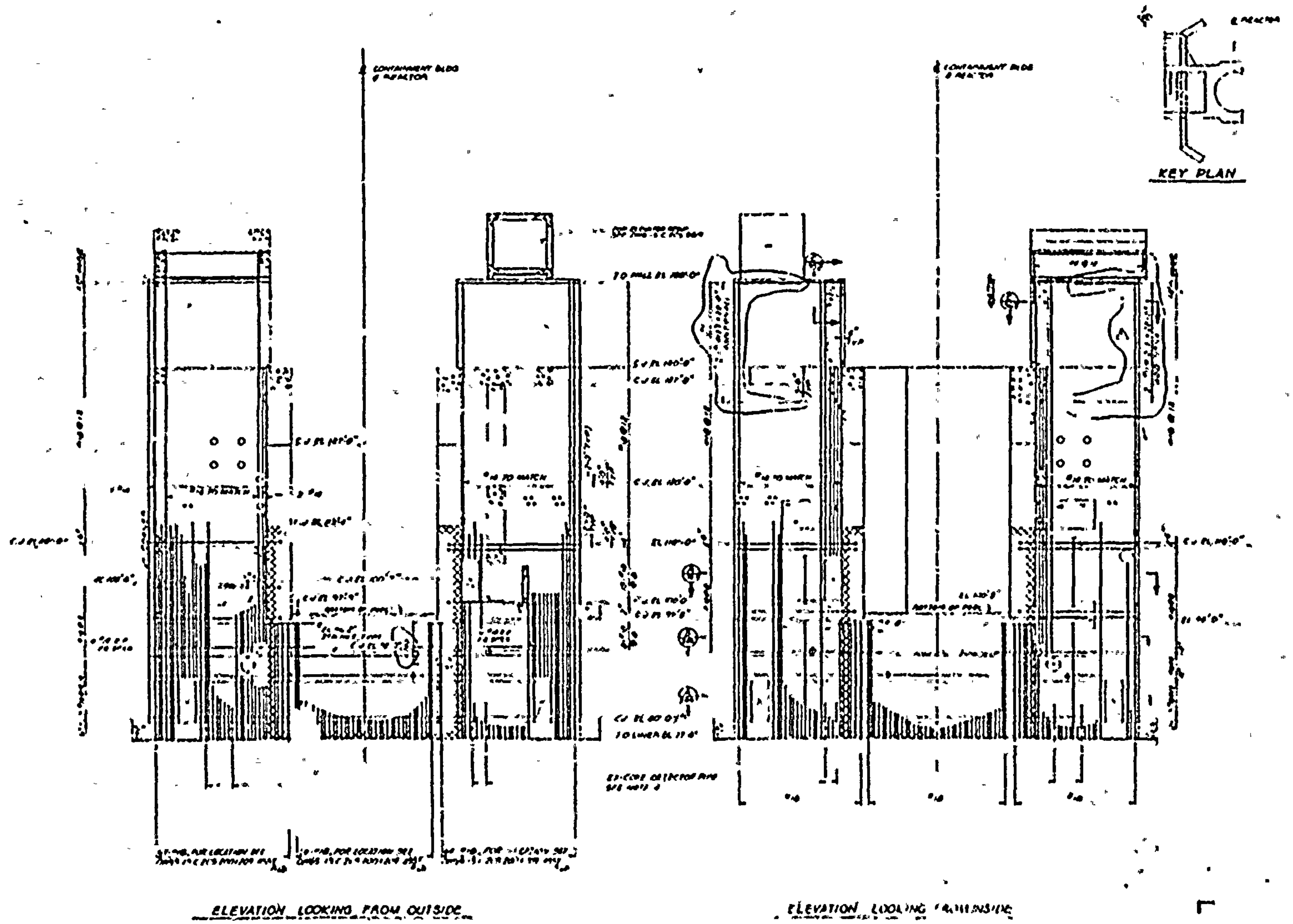
Figure 3.8-33



13-C-ZCS-364 REV 4

Palo Verde Nuclear Generating Station
FSAR

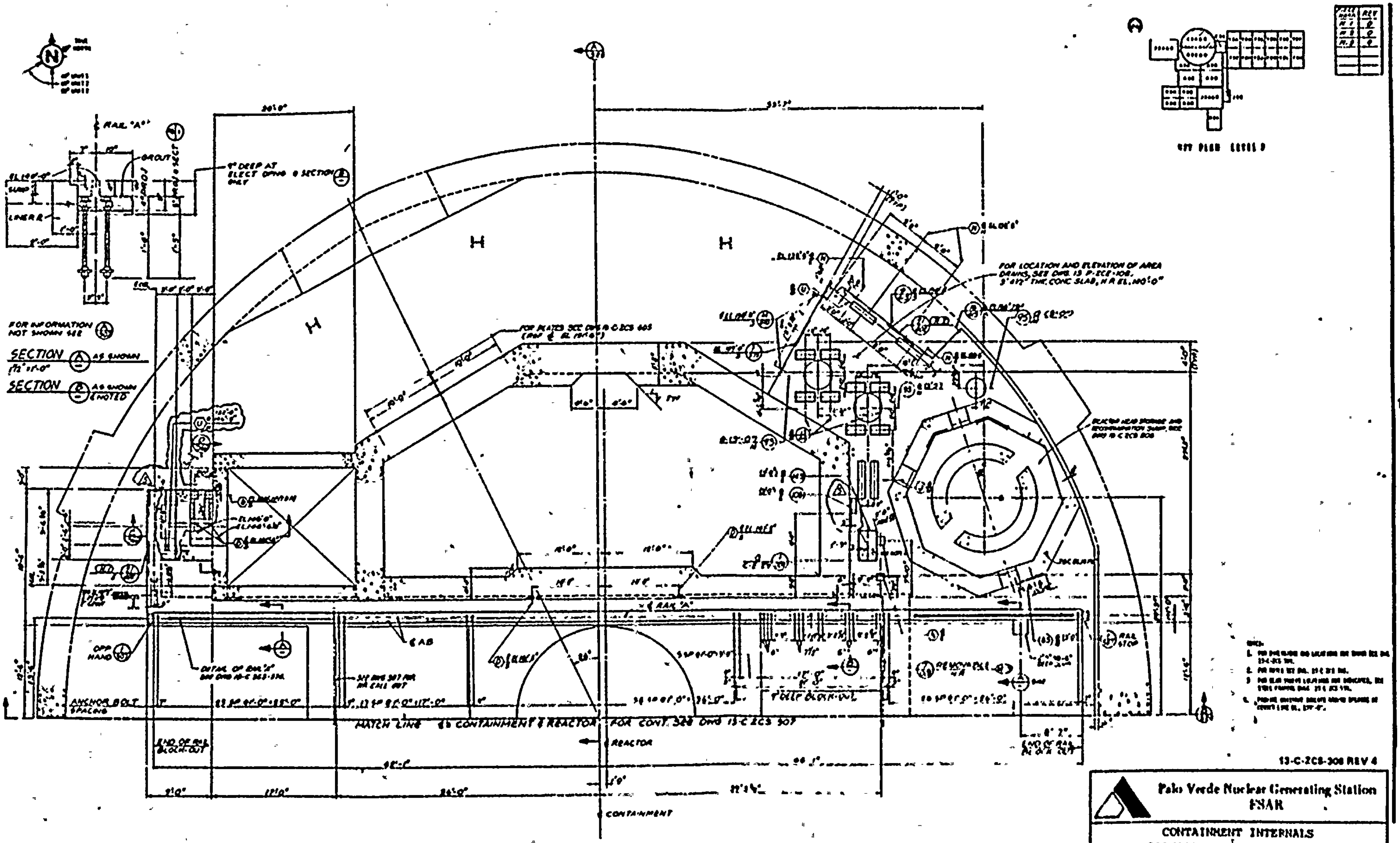
CONTAINMENT INTERNALS
REINF. CONC. - SHIELD WALLS
SECTIONS AND DETAILS
(Sheet 1 of 2)
Figure 3.8-34



13 C-ZCS-350 REV 5

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
REINFORCING CONCRETE
WEST SECONDARY SHIELD WALLS
(Sheet 2 of 2)
Figure 3.8-34

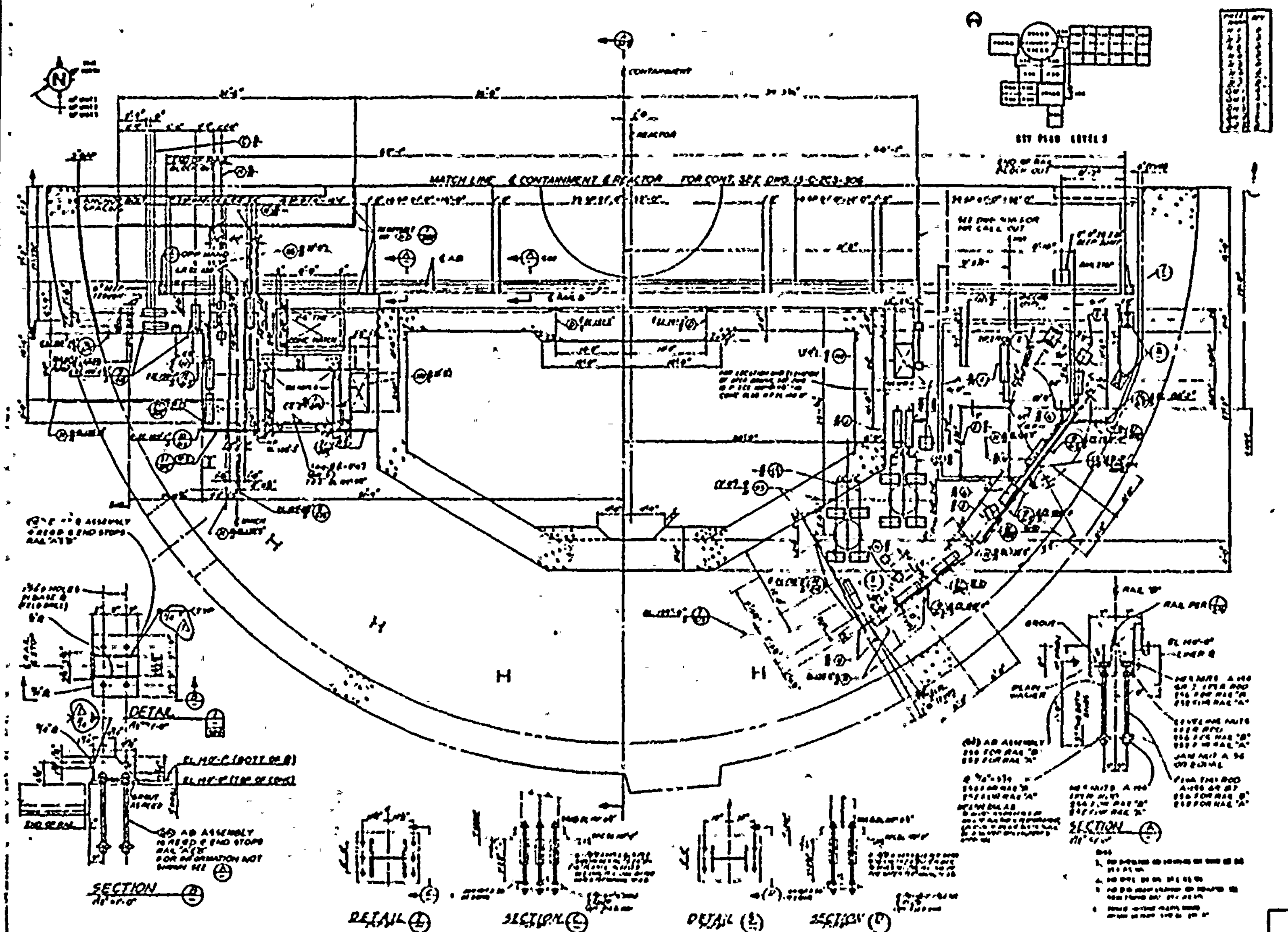


13-C-ZCS-308 REV 4

Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
PARTIAL CONCRETE PLAN AT
EL 140'-0" AREAS C3A & C3B

Figure 3.8-35



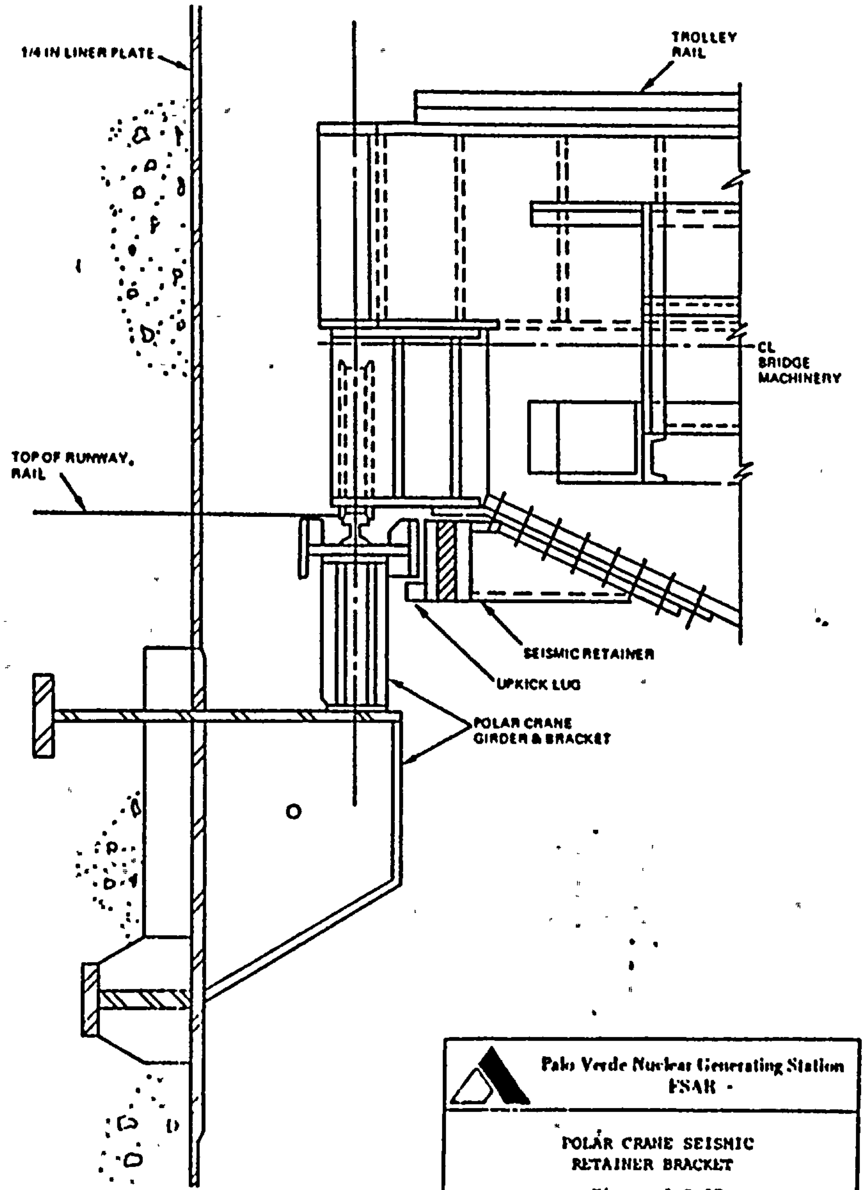
Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
PARTIAL CONCRETE PLAN AT
EL. 140'-0" AREAS C1C & C3D

Figure 3.8-36

March 1980

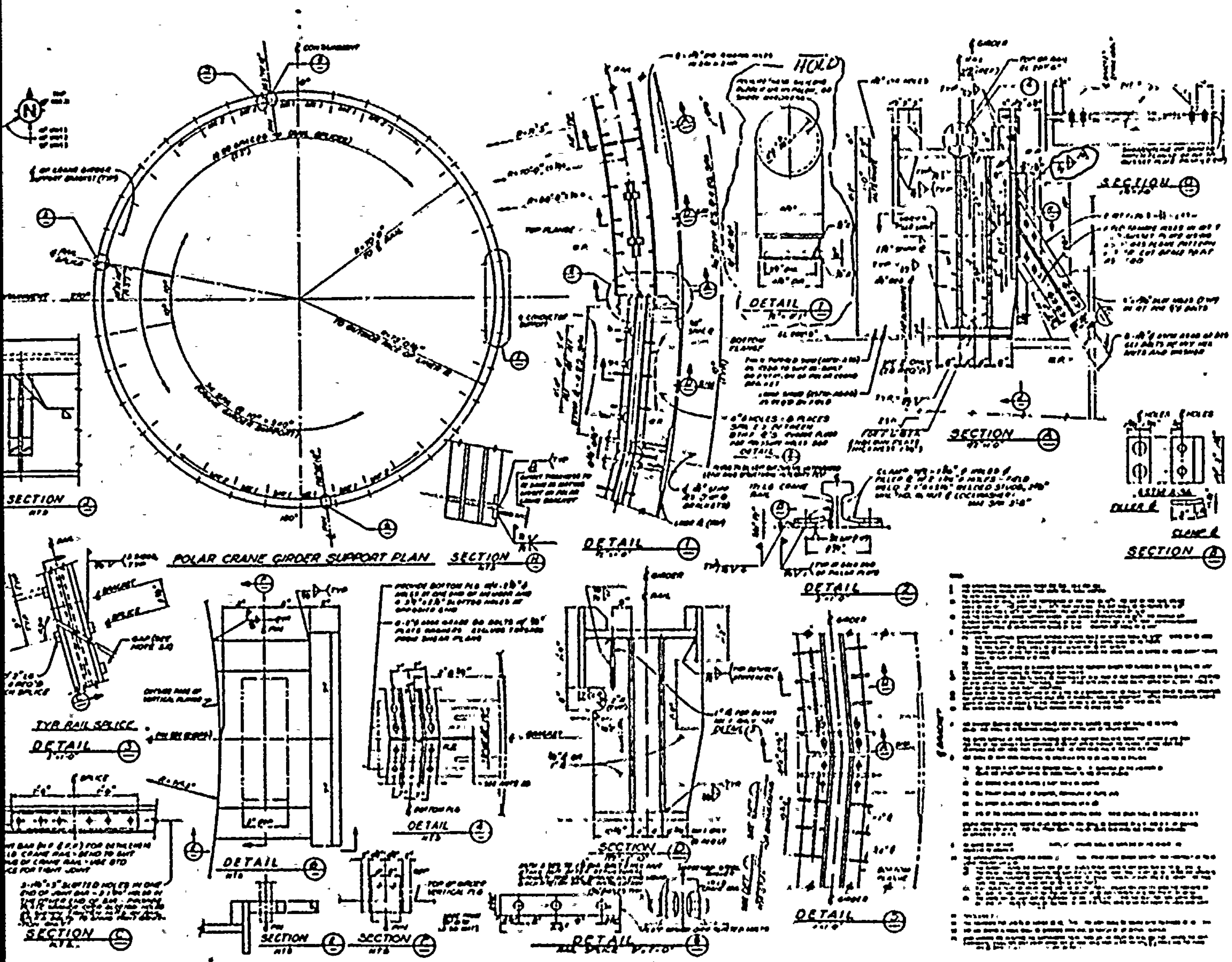
Amendment 1



Palo Verde Nuclear Generating Station
FSAR

POLAR CRANE SEISMIC RETAINER BRACKET

Figure 3.8-37

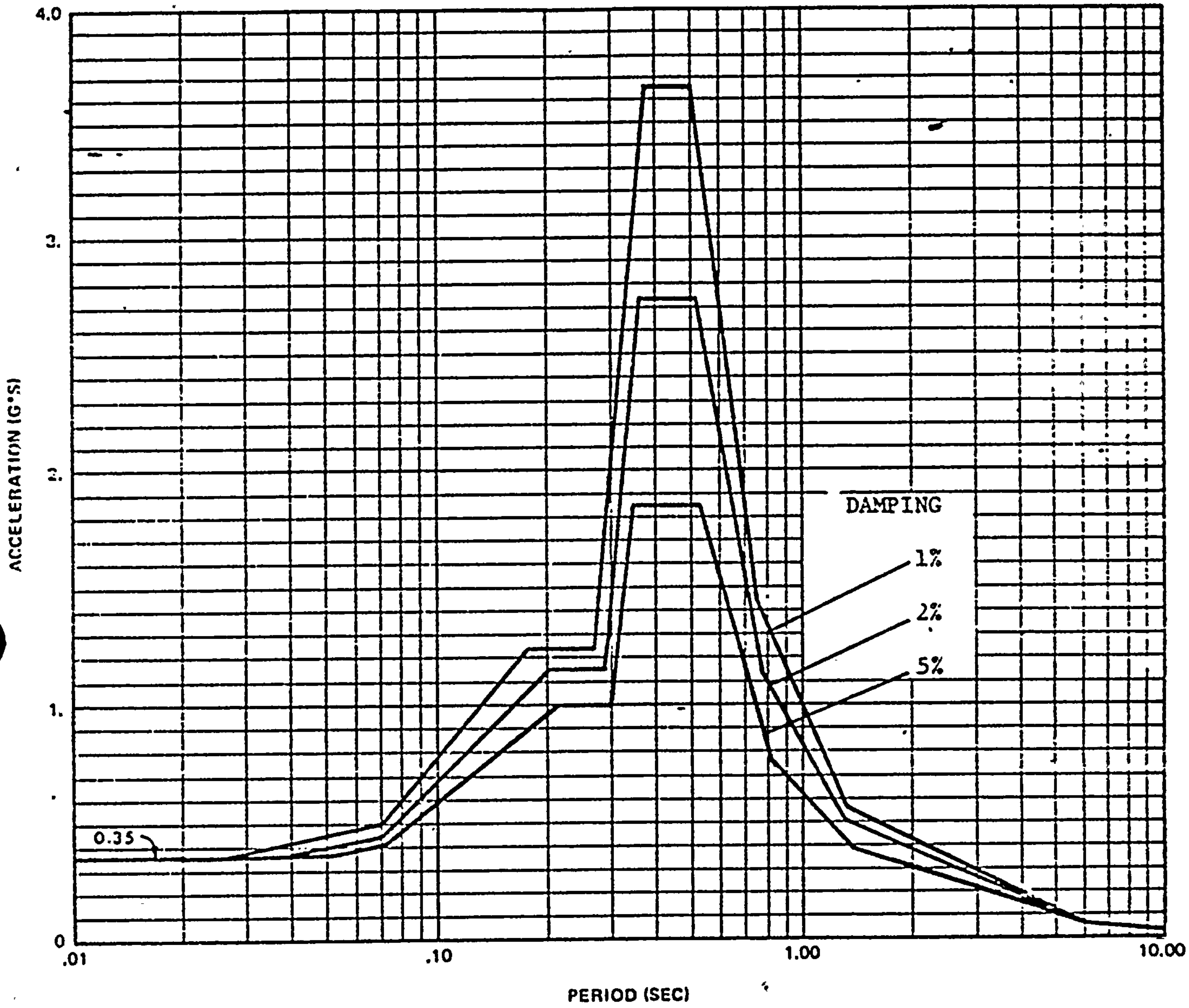


Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT INTERNALS
POLAR CRANE SUPPORT GIRDER
SECTIONS AND DETAILS

Figure 3.8-38

March 1980 Amendment 1



Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING
SSE HORIZ (E-W) ACC. RESPONSE SPECTRA
EL. 143.5 FT, STEAM GENERATOR SNUBBERS

Figure 3D-7

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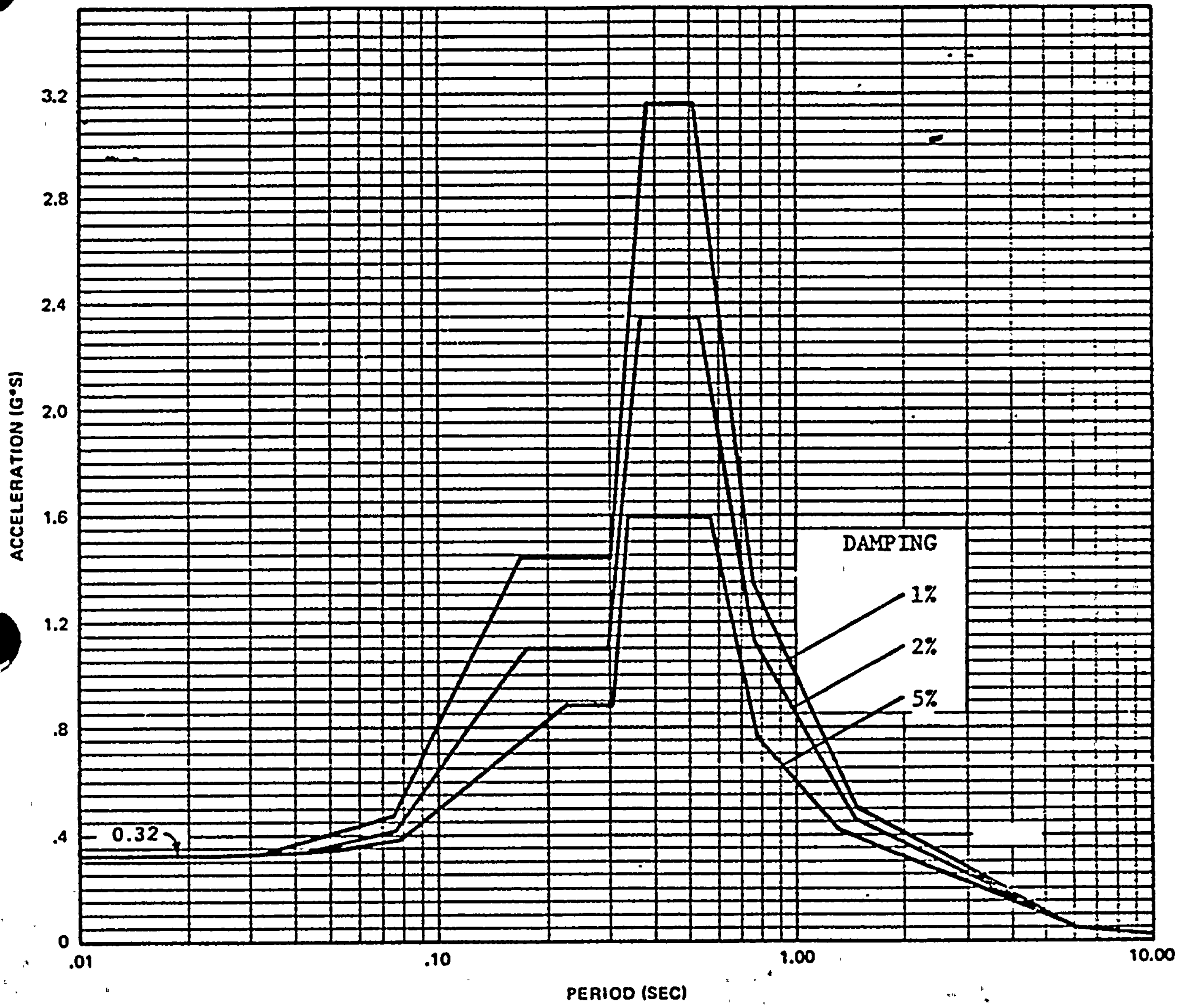
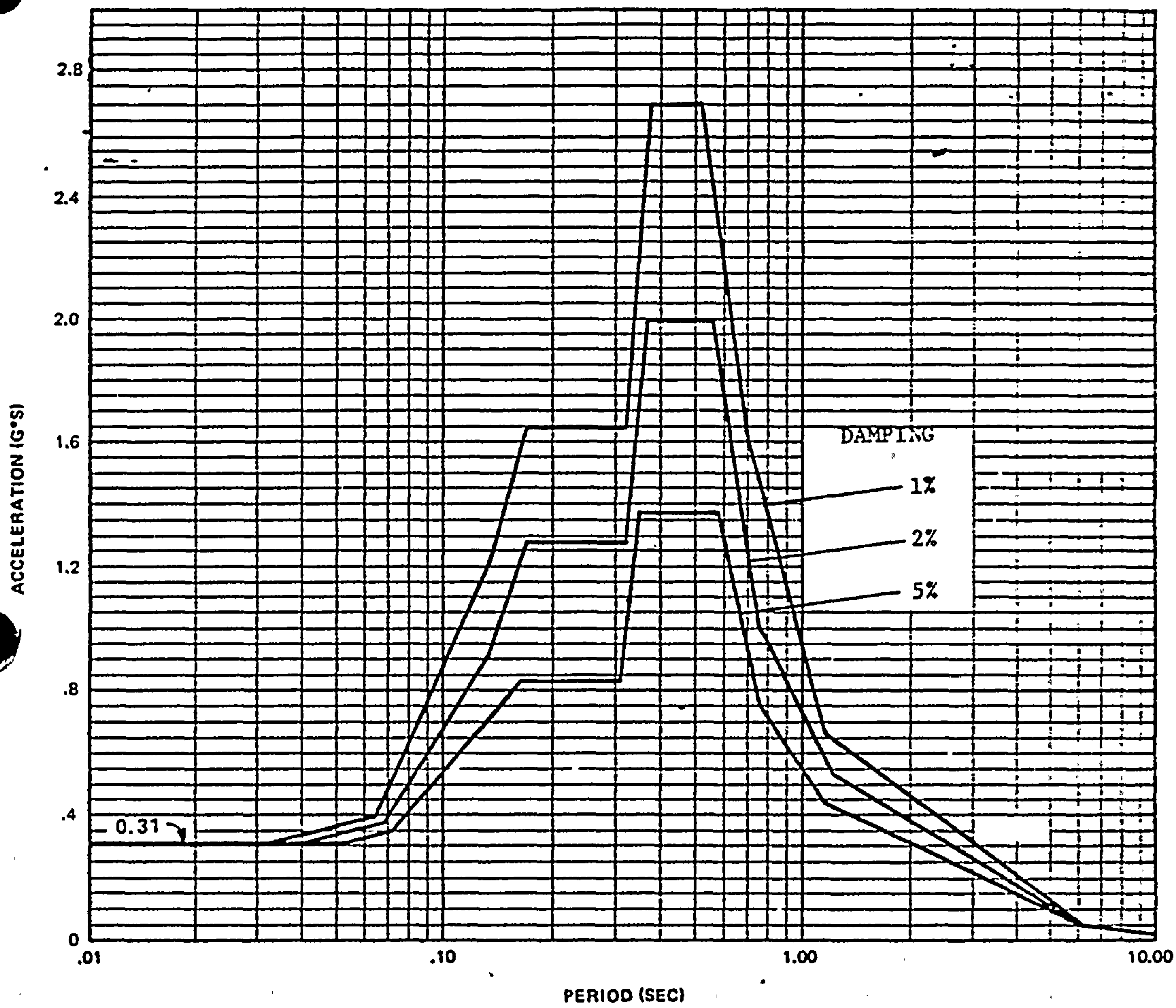


Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING SSE HORIZ (E-W)
ACC. RESPONSE SPECTRA EL. 119.5 FT,
R.C. PUMP UPPER HORIZ SUPPORTS

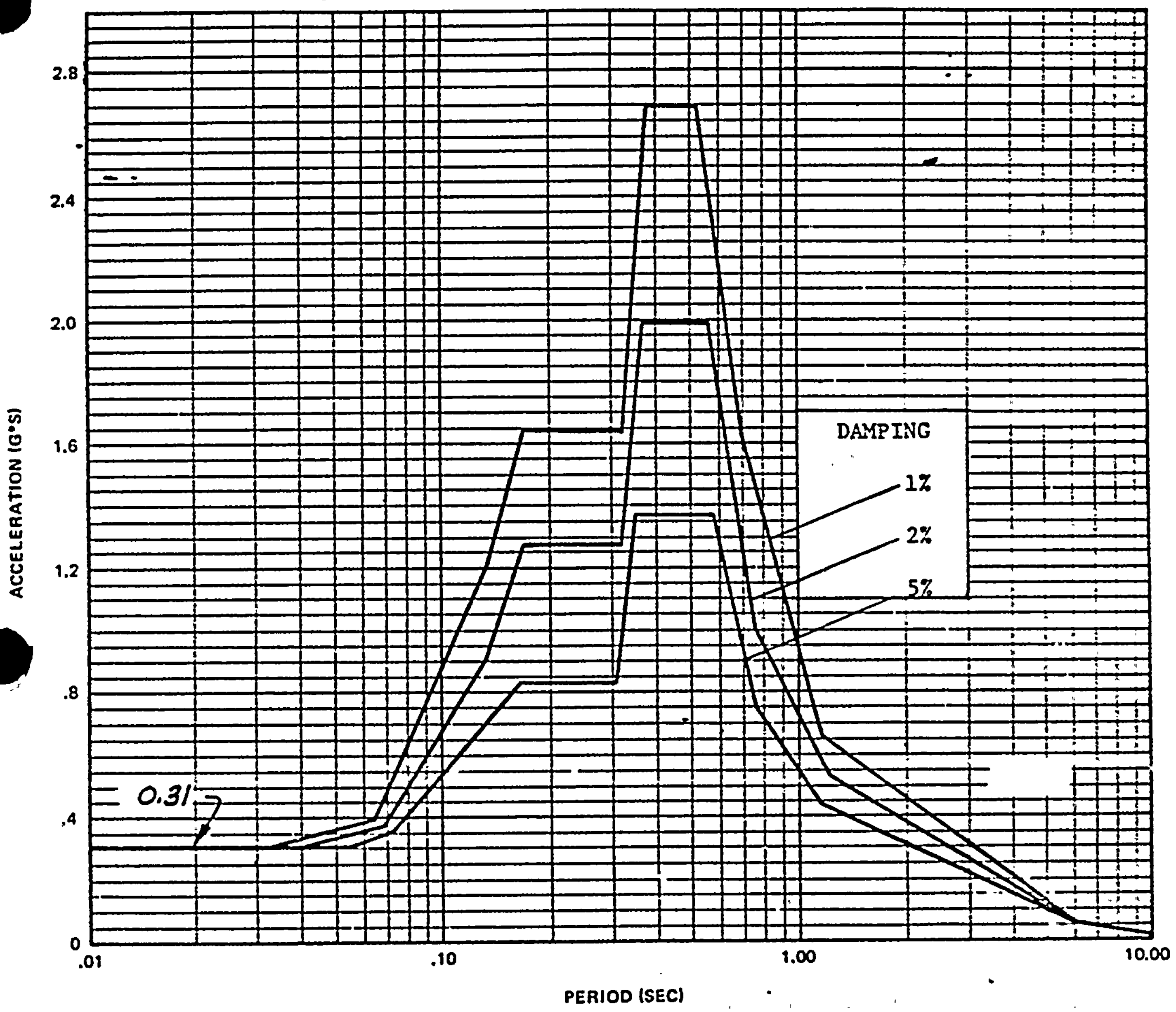
Figure 3D-9



Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING SSE HORIZ (E-W)
ACC. RESPONSE SPECTRA EL. 97.7 FT,
R.C. PUMP LOWER HORIZ SUPPORTS

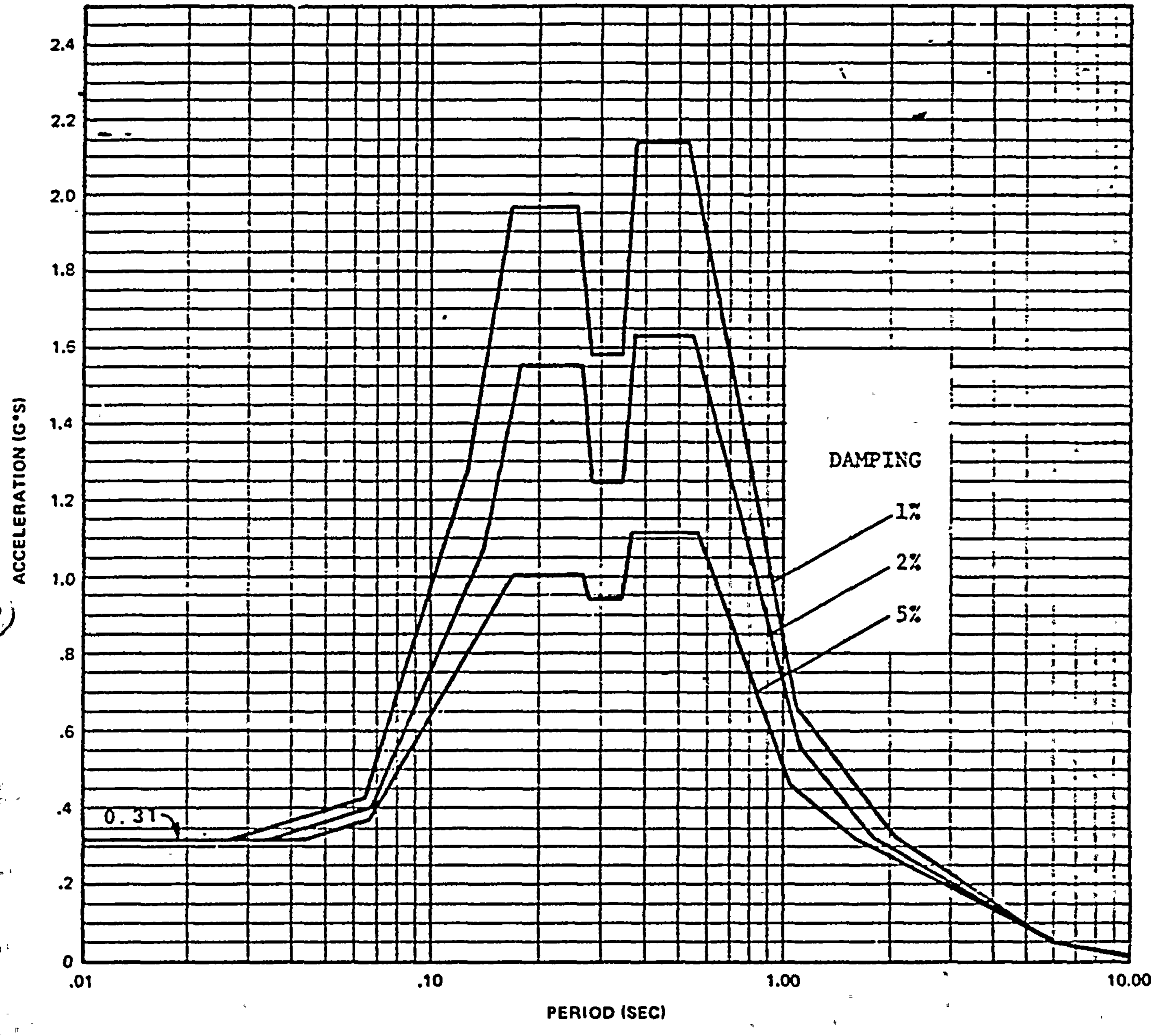
Figure 3D-11



Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING SSE HORIZ (E-W)
ACC. RESPONSE SPECTRA EL. 97.6 FT.
R.V. COL. UPPER HORIZ GUIDES

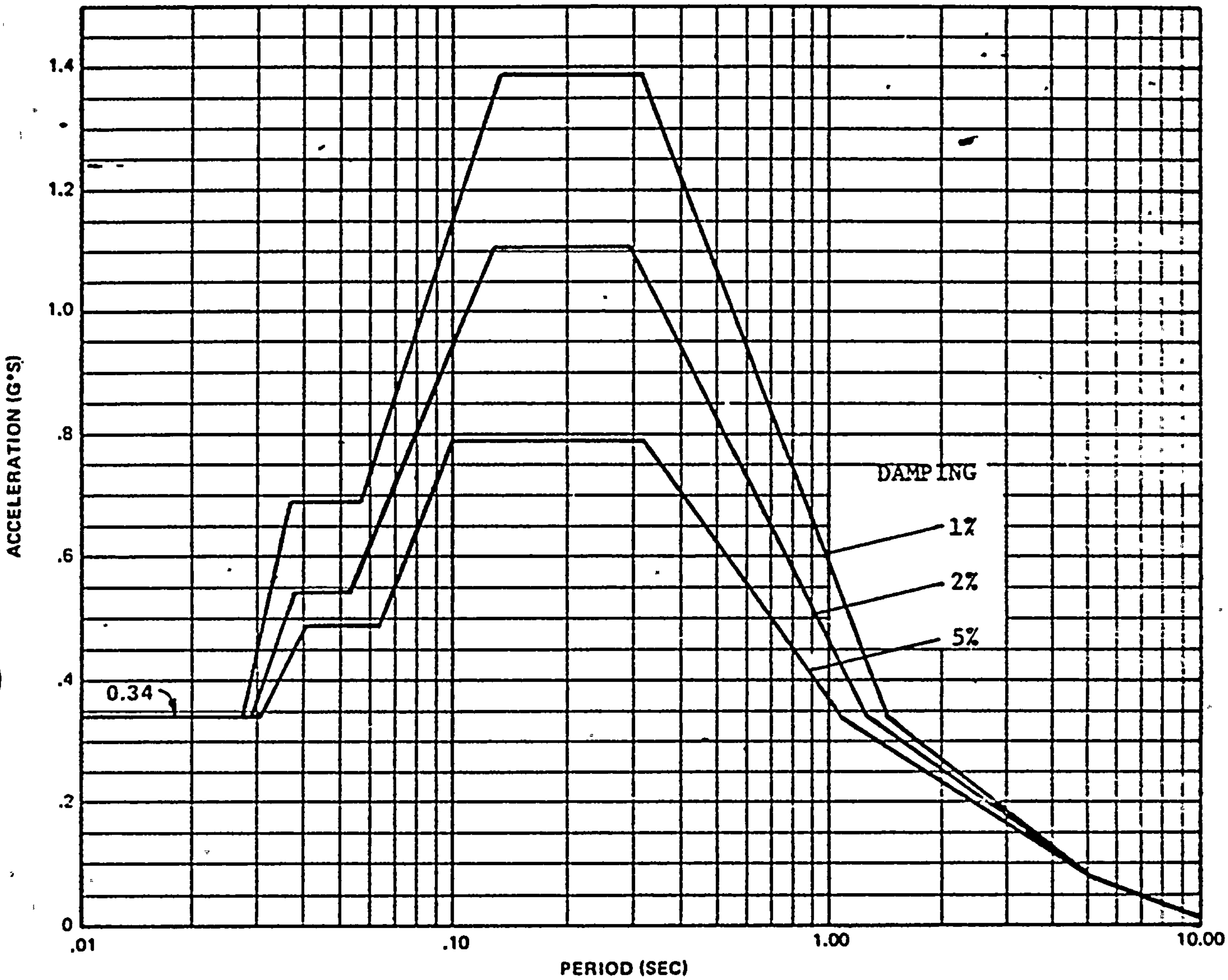
Figure 3D-13



Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING SSE HORIZ (E-W)
ACC. RESPONSE SPECTRA EL. 78.0 FT,
R.V. COL BASES & LOWER KEYS

Figure 3D-15



Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING SSE VERTICAL ACC.
RESPONSE SPECTRA
EL. 78.0 FT, R.V. COLUMN BASES

Figure 3D-17

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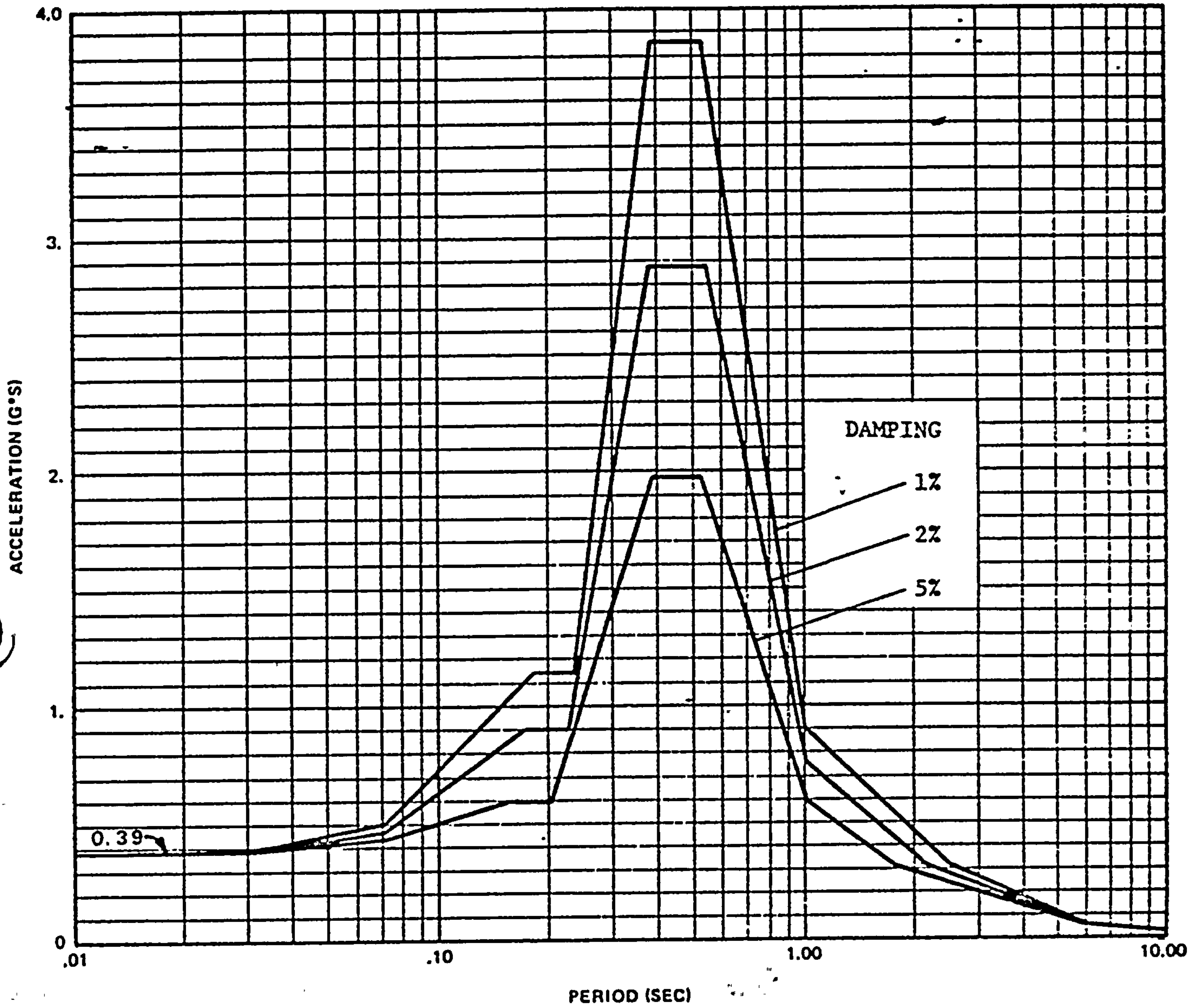
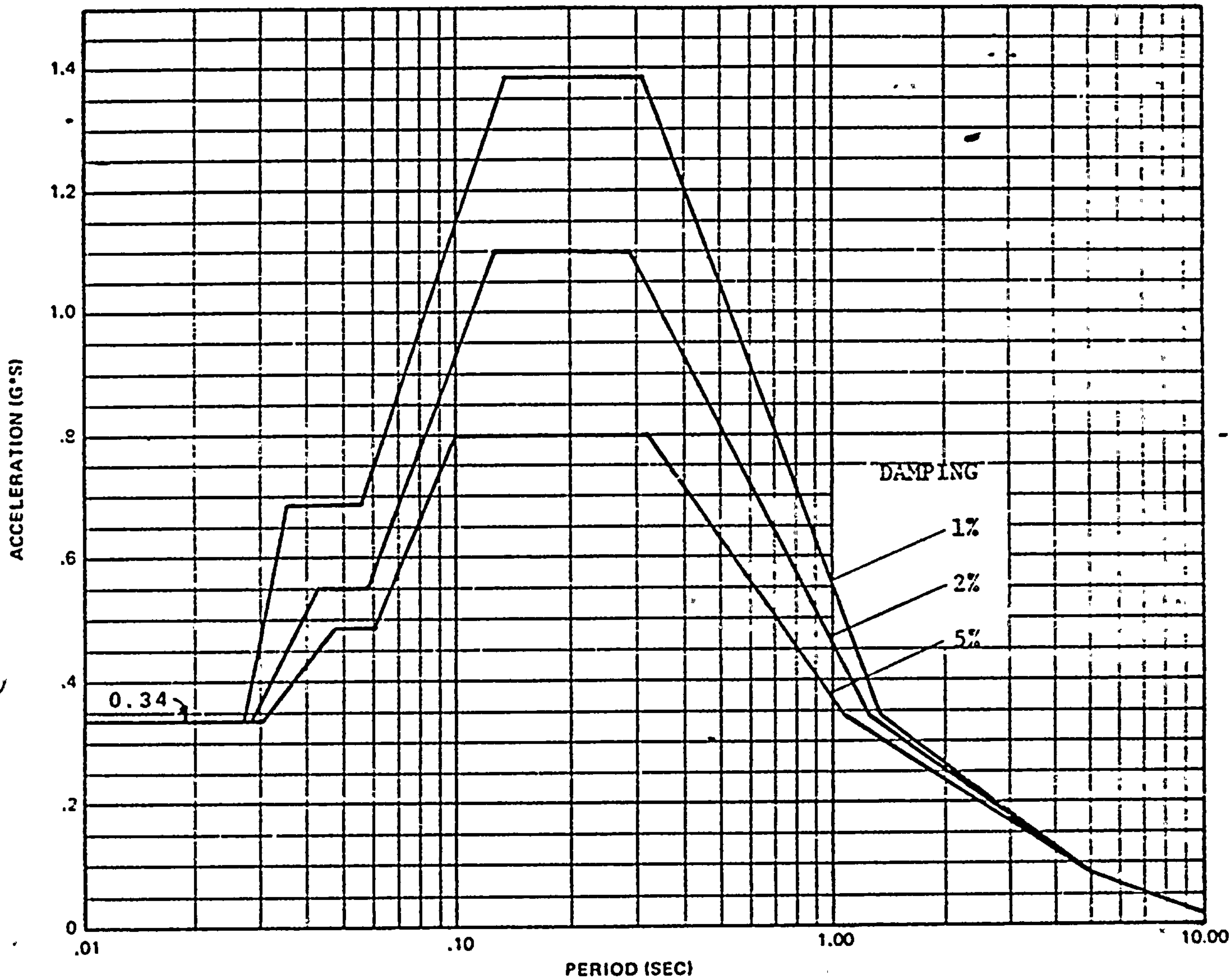


Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING SSE HORIZ (N-S)
ACC. RESPONSE SPECTRA EL. 150.9 FT,
STEAM GENERATOR UPPER KEYS

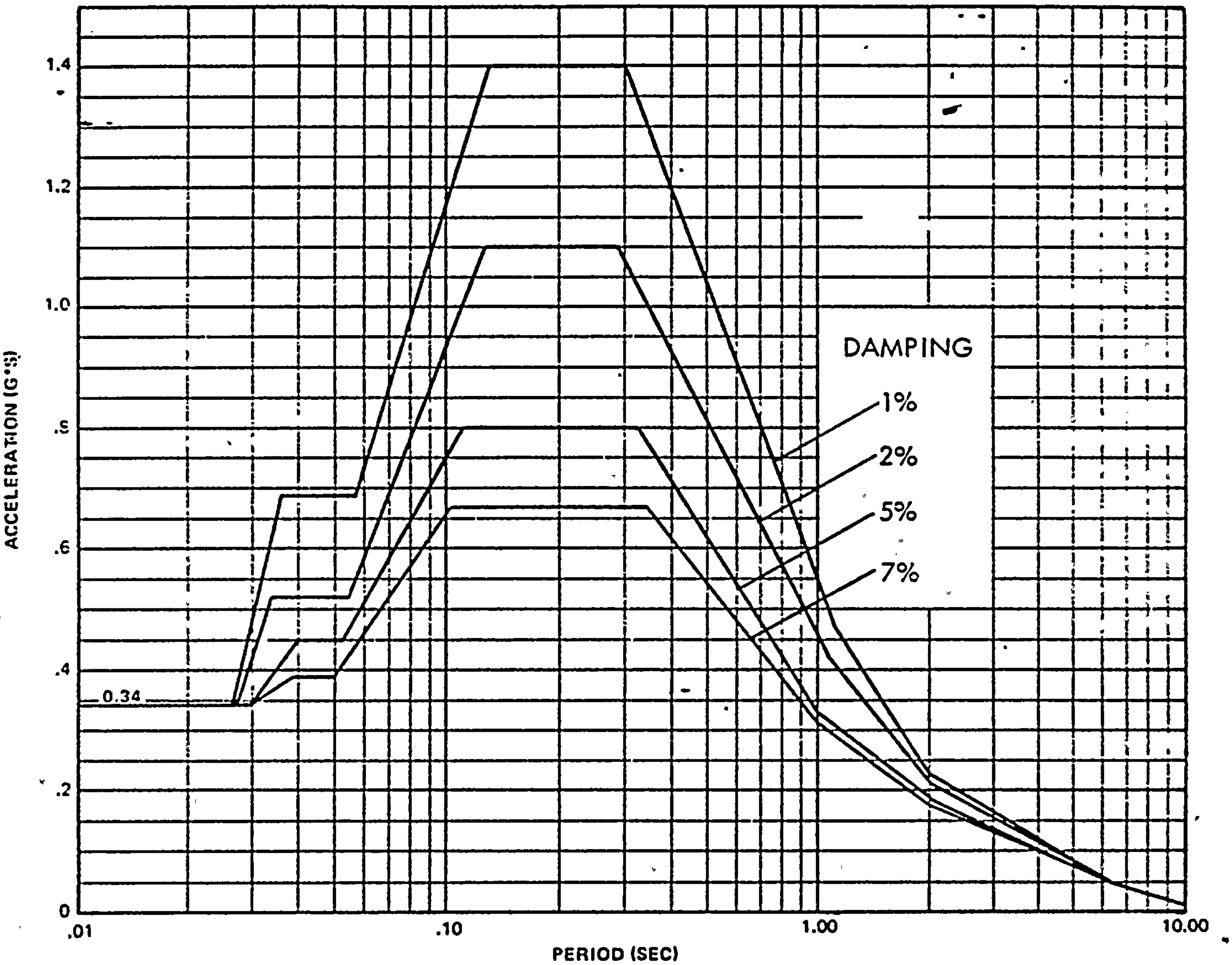
Figure 3D-19



Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING SSE VERTICAL
ACC. RESPONSE SPECTRA
EL. 96.7 FT, STEAM GENERATOR BASES

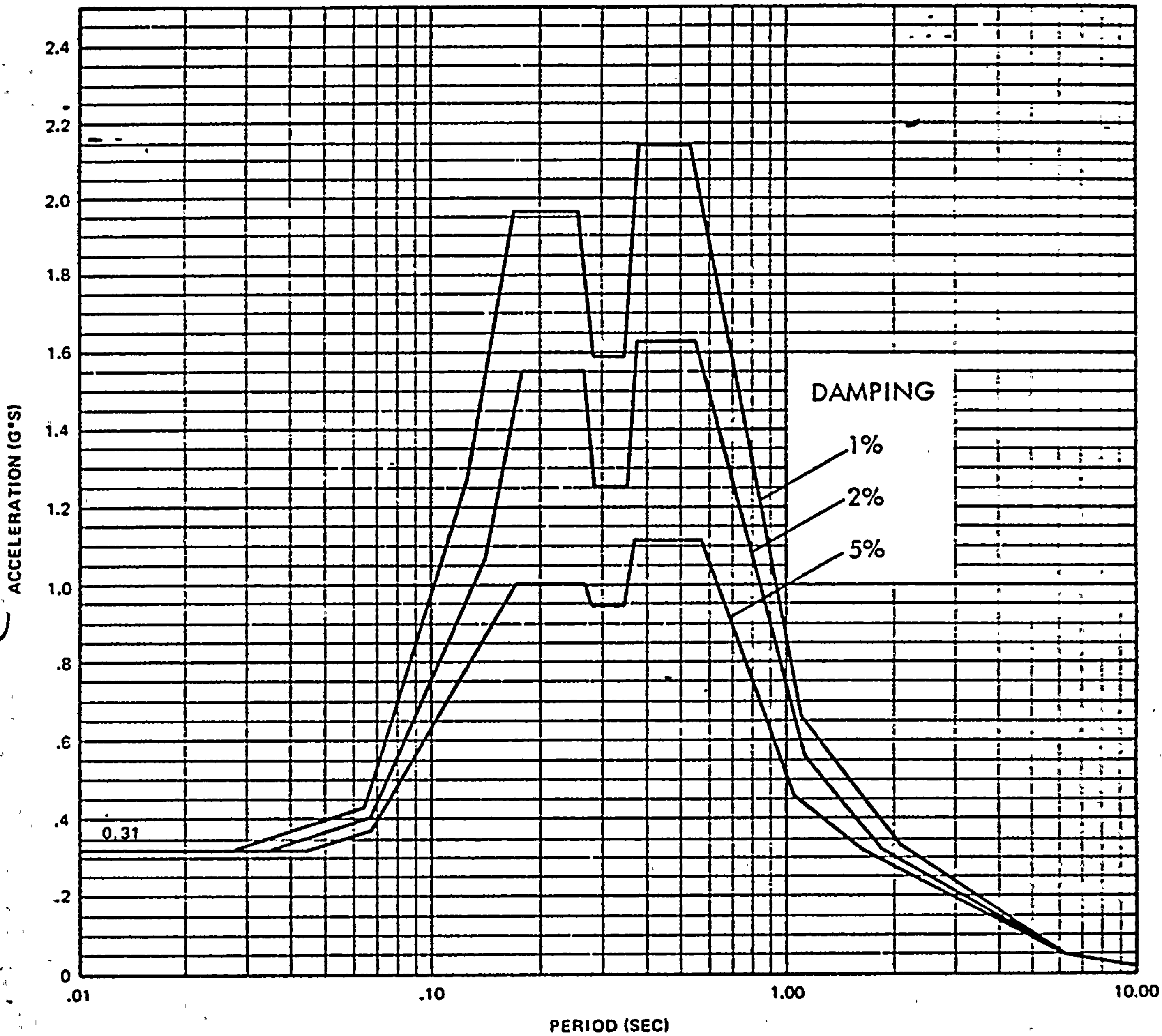
Figure 3D-21



Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING SSE VERTICAL ACC. RESPONSE SPECTRA, CONTAINMENT SHELL AND INTERIOR STRUCTURE

Figure 3D-23

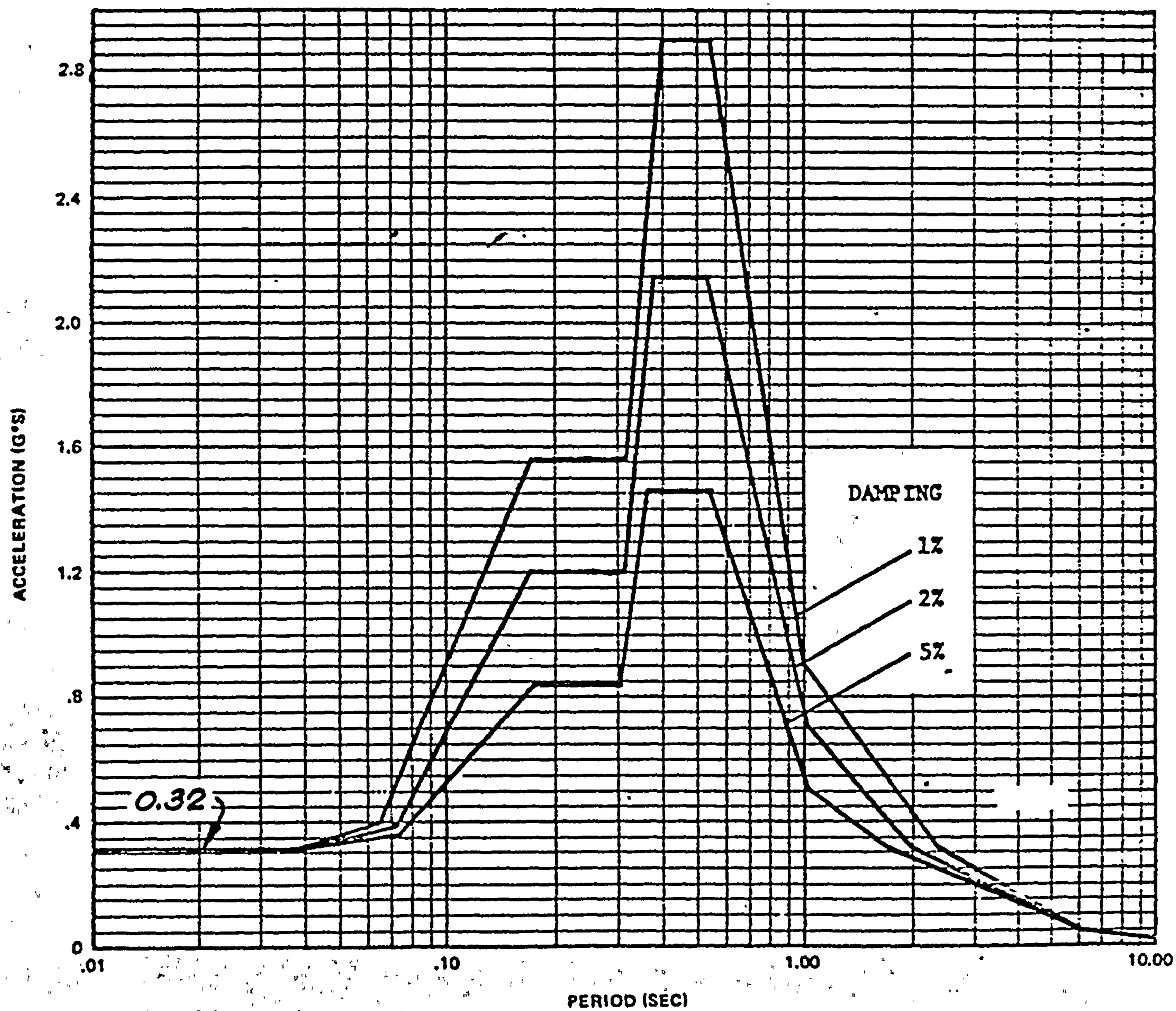


Palo Verde Nuclear Generating Station
FSAR

CONTAINMENT BUILDING SSE
HORIZONTAL ACC. RESPONSE SPECTRA
EL. 73.5 FT, BASE MAT

Figure 3D-25

	ARIZONA NUCLEAR POWER PROJECT PALO VERDE NUCLEAR GENERATING STATION UNITS 1, 2, AND 3		
CONTAINMENT BUILDING SSE HORIZ. (E-W) ACC. RESPONSE SPECTRA EL. 108.3 FT., PRESSURIZER BASE			
PREPARED: JP/mc/R.C.	REVIEWED: RDF JFO'S	APPROVED: AH WGB	
JOB NO. 10407	PAGE NUMBER: IV.2-25	REV. A	DATE 1-1/2-1-C



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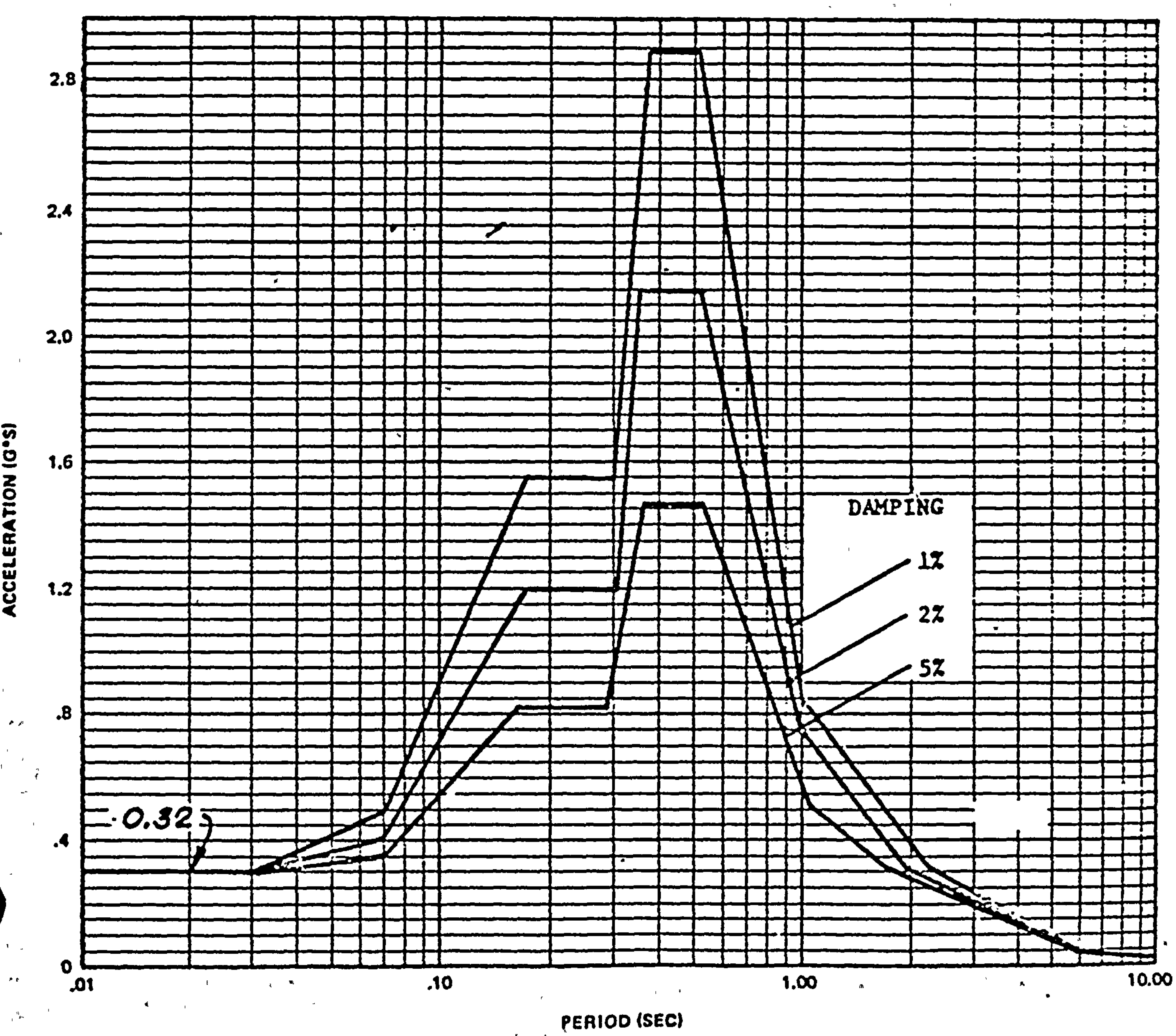
ARIZONA NUCLEAR POWER PROJECT
PALO VERDE NUCLEAR
GENERATING STATION
UNITS 1, 2, AND 3

CONTAINMENT BUILDING
SSE HORIZ. (N-S) ACC. RESPONSE SPECTRA
EL. 108.3 FT., PRESSURIZER BASE

PREPARED:	REVIEWED:	APPROVED	
JP/mc/R.C	RBF JFO'S	WSB	
JOB NO. 10407	PAGE NUMBER: IV.2-27	REV. A	DATE 11/27/7[illegible]



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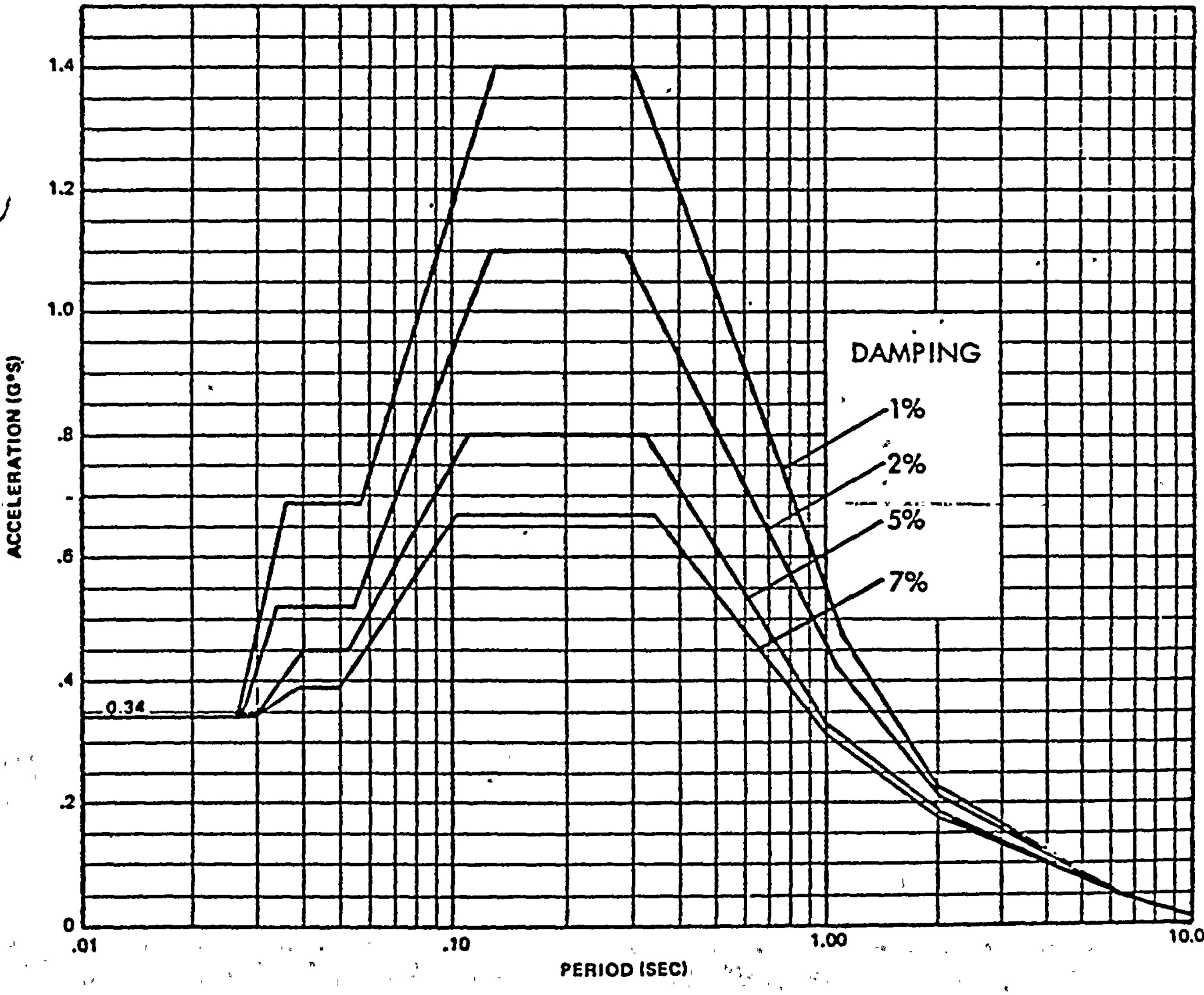
BECHTEL	ARIZONA NUCLEAR POWER PROJECT PALO VERDE NUCLEAR GENERATING STATION UNITS 1, 2, AND 3

CONTAINMENT BUILDING
SSE VERTICAL ACC. RESPONSE SPECTRA
CONTAINMENT SHELL AND INTERIOR STRUCTURE

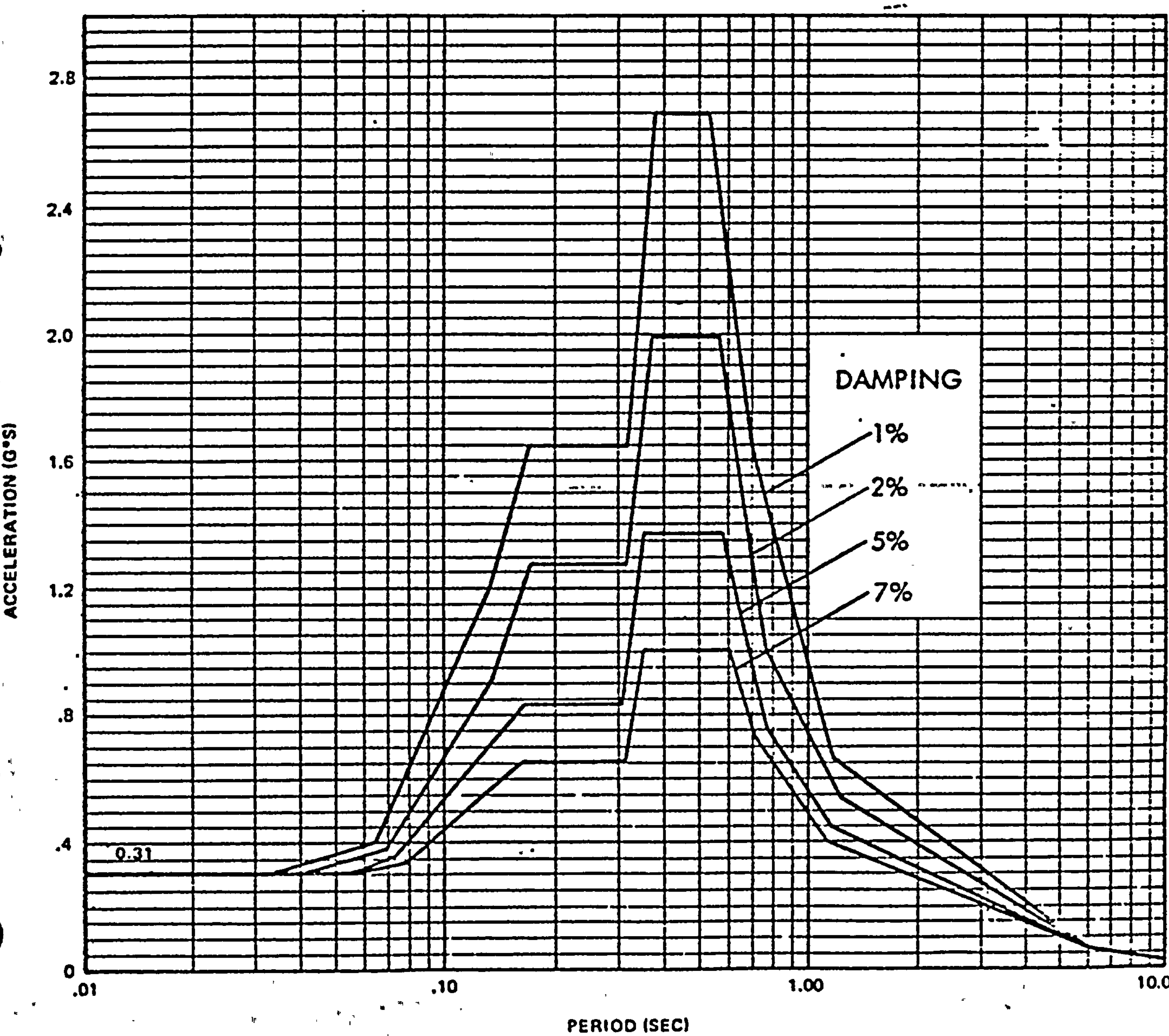
PREPARED:	REVIEWED:	APPROVED	
J.P./M.C./KC	RBF JFO'S	HH WGB	
JOB NO. 10407	PAGE NUMBER: IV.2-39	REV. B	DATE 11-15-77



ARIZONA NUCLEAR POWER PROJECT
PALO VERDE NUCLEAR
GENERATING STATION
UNITS 1, 2, AND 3

CONTAINMENT BUILDING
SSE HORIZONTAL ACC. RESPONSE SPECTRA
FLOOR SLAB AT ELEVATION 100.0 FT.

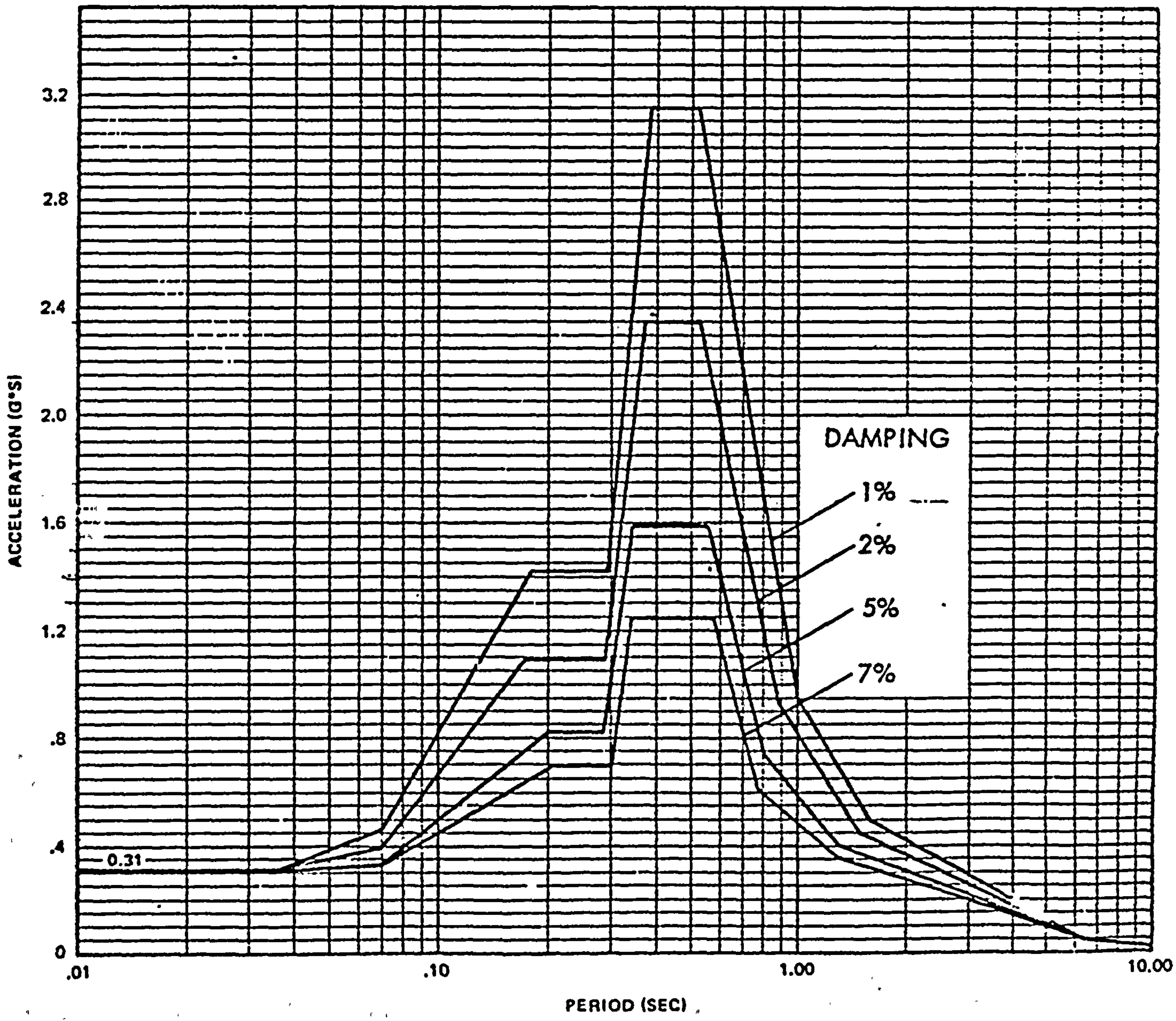
PREPARED:	REVIEWED:	APPROVED:	
JP/mc/RC	RBF JFO's	WGB	
JOB NO. 10407	PAGE NUMBER: IV.2-63	REV. B	DATE 11-15-77



ARIZONA NUCLEAR POWER PROJECT
PALO VERDE NUCLEAR
GENERATING STATION
UNITS 1, 2, AND 3

CONTAINMENT BUILDING
SSE HORIZONTAL ACC. RESPONSE SPECTRA
FLOOR SLAB AT ELEVATION 120.0 FT.

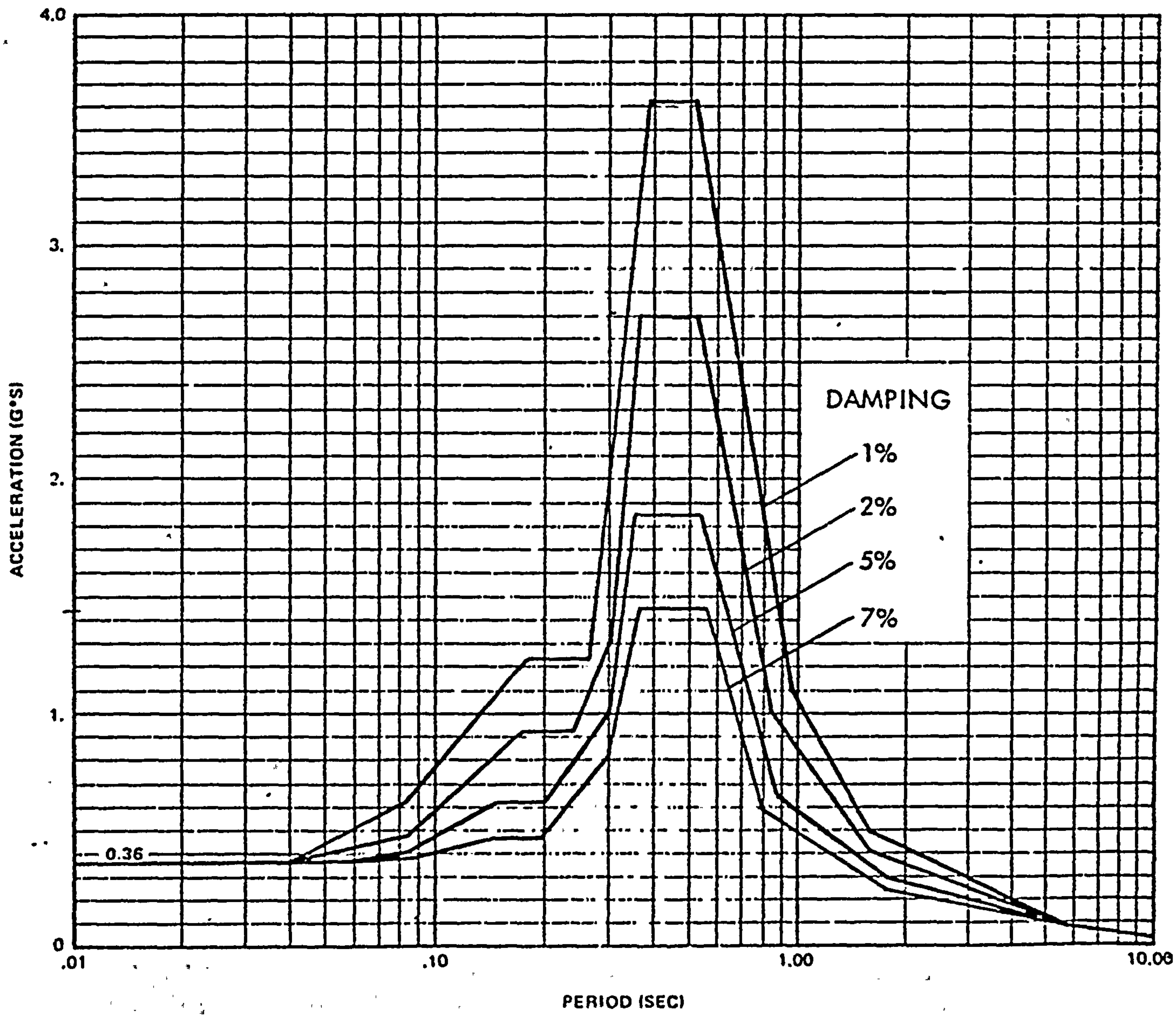
PREPARED:	REVIEWED:	APPROVED.	
J.P./m.c./RC	RBF. JFO'S	WGB	
JOB NO. 10407	PAGE NUMBER: IV.2-65	REV. B	DATE. 11-15-77



BECHTEL	ARIZONA NUCLEAR POWER PROJECT PALO VERDE NUCLEAR GENERATING STATION UNITS 1, 2, AND 3

CONTAINMENT BUILDING
SSE HORIZONTAL ACC. RESPONSE SPECTRA
FLOOR SLAB AT ELEVATION 140.0 FT.

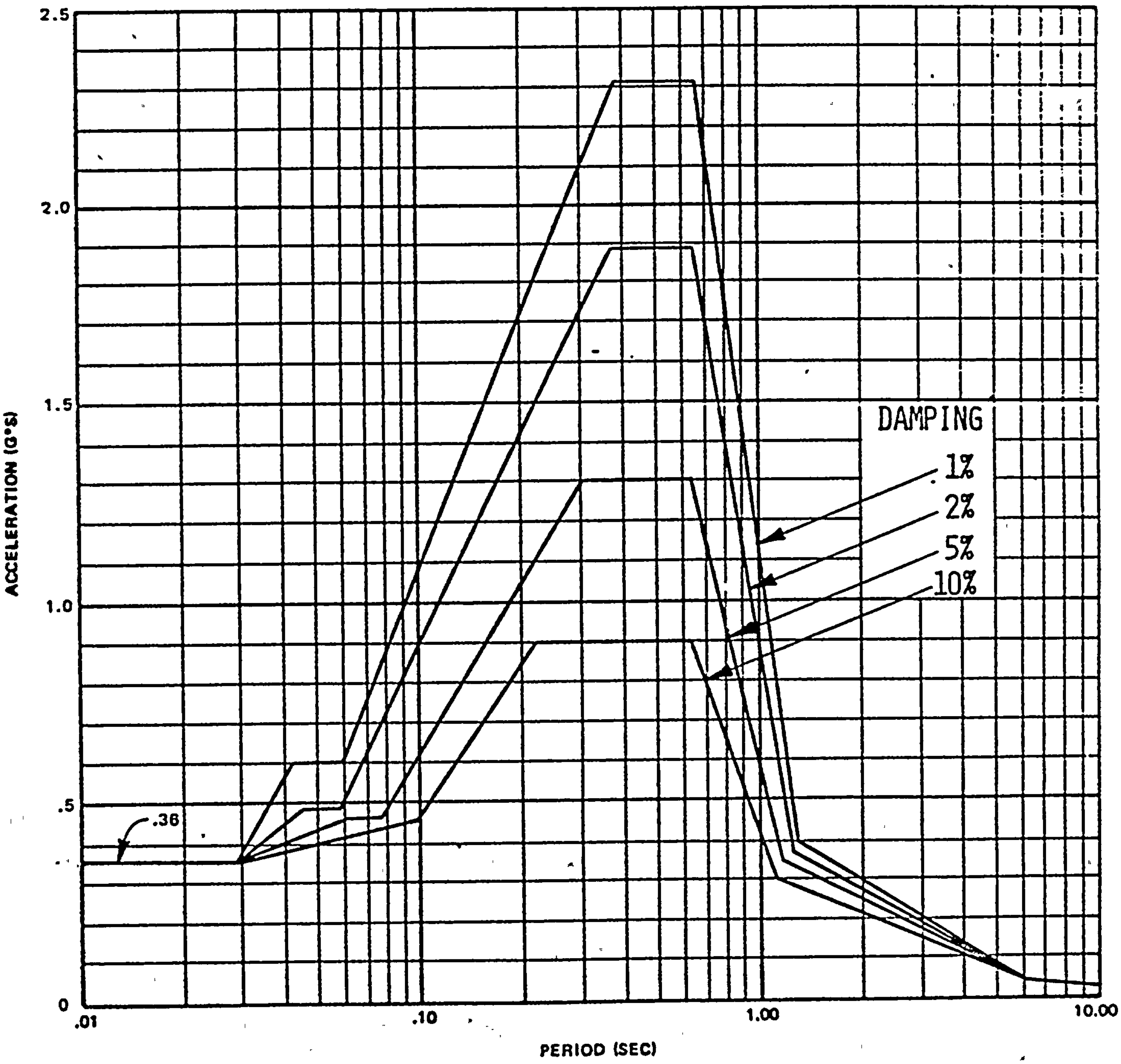
PREPARED:	REVIEWED,	APPROVED	
JP/mc./RC	RBF JFO'S	WGB	
JOB NO. 10407	PAGE NUMBER: IV.2-67	REV. B	DATE 11-15-77



BECHTEL	ARIZONA NUCLEAR POWER PROJECT PALO VERDE NUCLEAR GENERATING STATION UNITS 1, 2, AND 3	C

CONTAINMENT BUILDING
SSE VERTICAL ACC. RESPONSE SPECTRA
EL. 213.5 FT., POLAR CRANE

PREPARED:	REVIEWED:	APPROVED.	
JP/MC/R.C.		WGB JB	
JOB NO. 10407	DRAWING NO.: IV.2-75	REV. A	DATE. 7/1/76

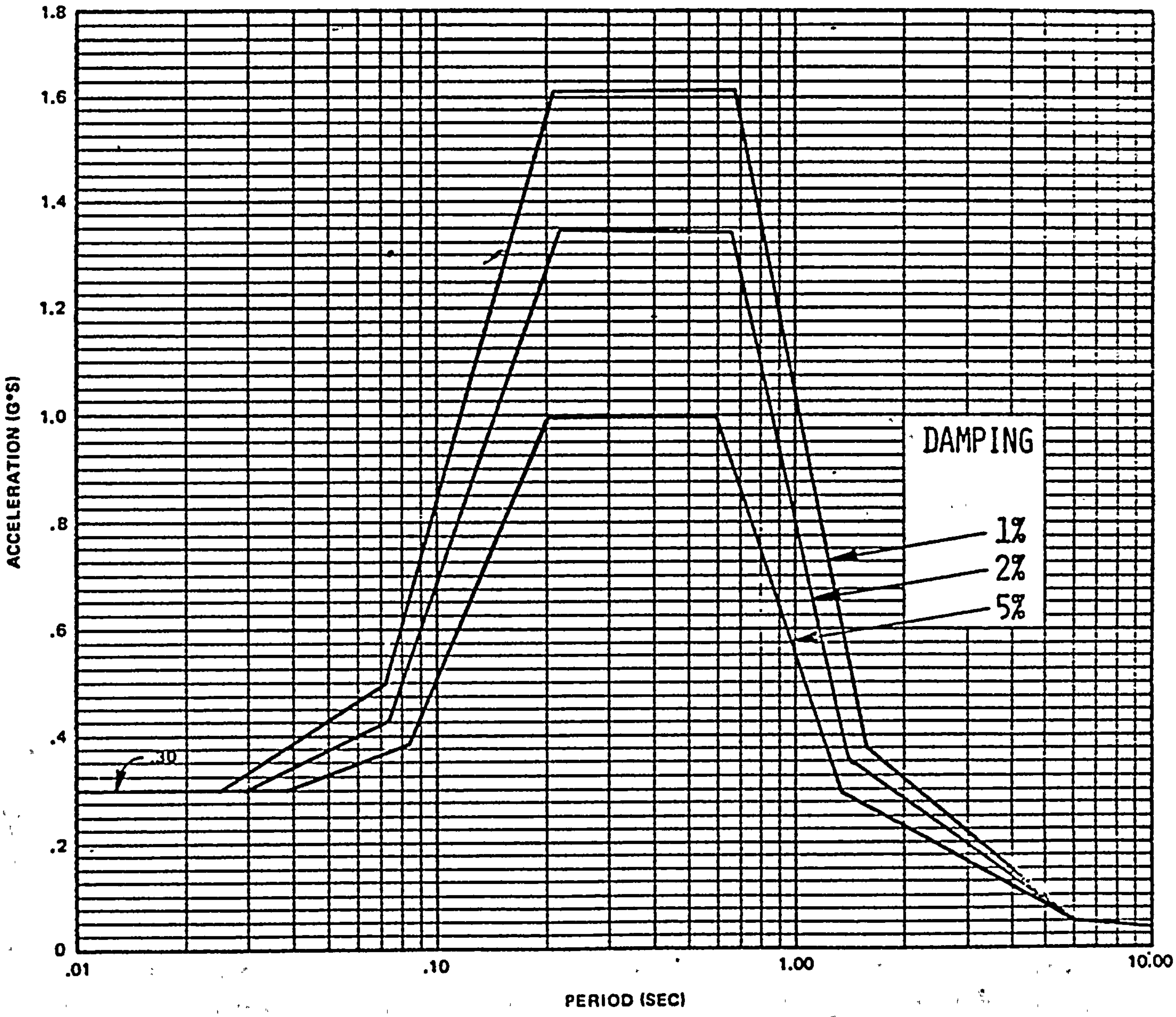


ARIZONA NUCLEAR POWER PROJECT
PALO VERDE NUCLEAR
GENERATING STATION
UNITS 1, 2, AND 3

C

CONTAINMENT BUILDING
SSE VERTICAL ACC. RESPONSE SPECTRA
EL. 110.0 FT., PRESSURIZER BASE

PREPARED:	REVIEWED:	APPROVED:	
JP/mc/R.C.		HPP	
JOB NO. 10407	DRAWING NO.: IV.2-79	REV. A	DATE 7/27/76



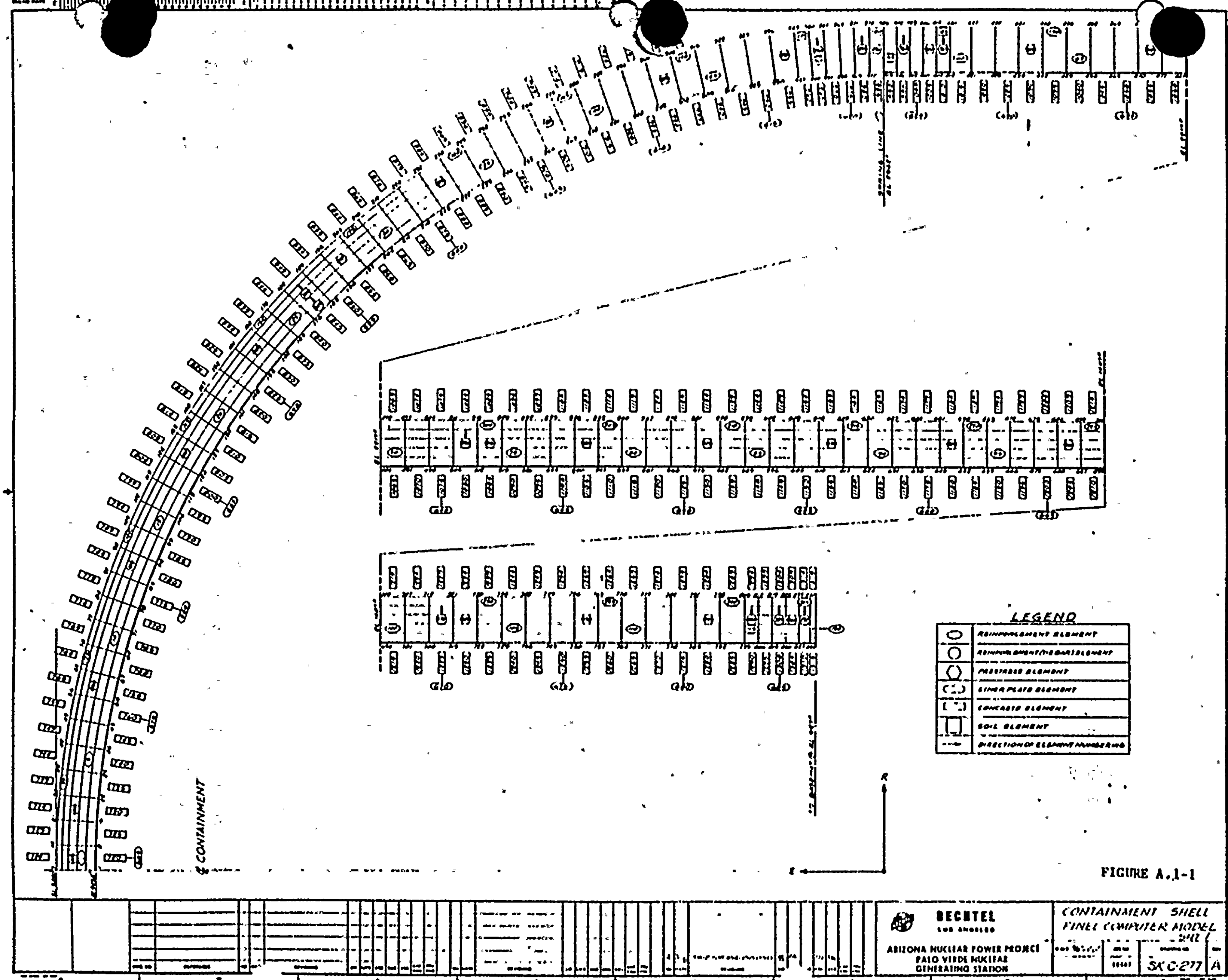
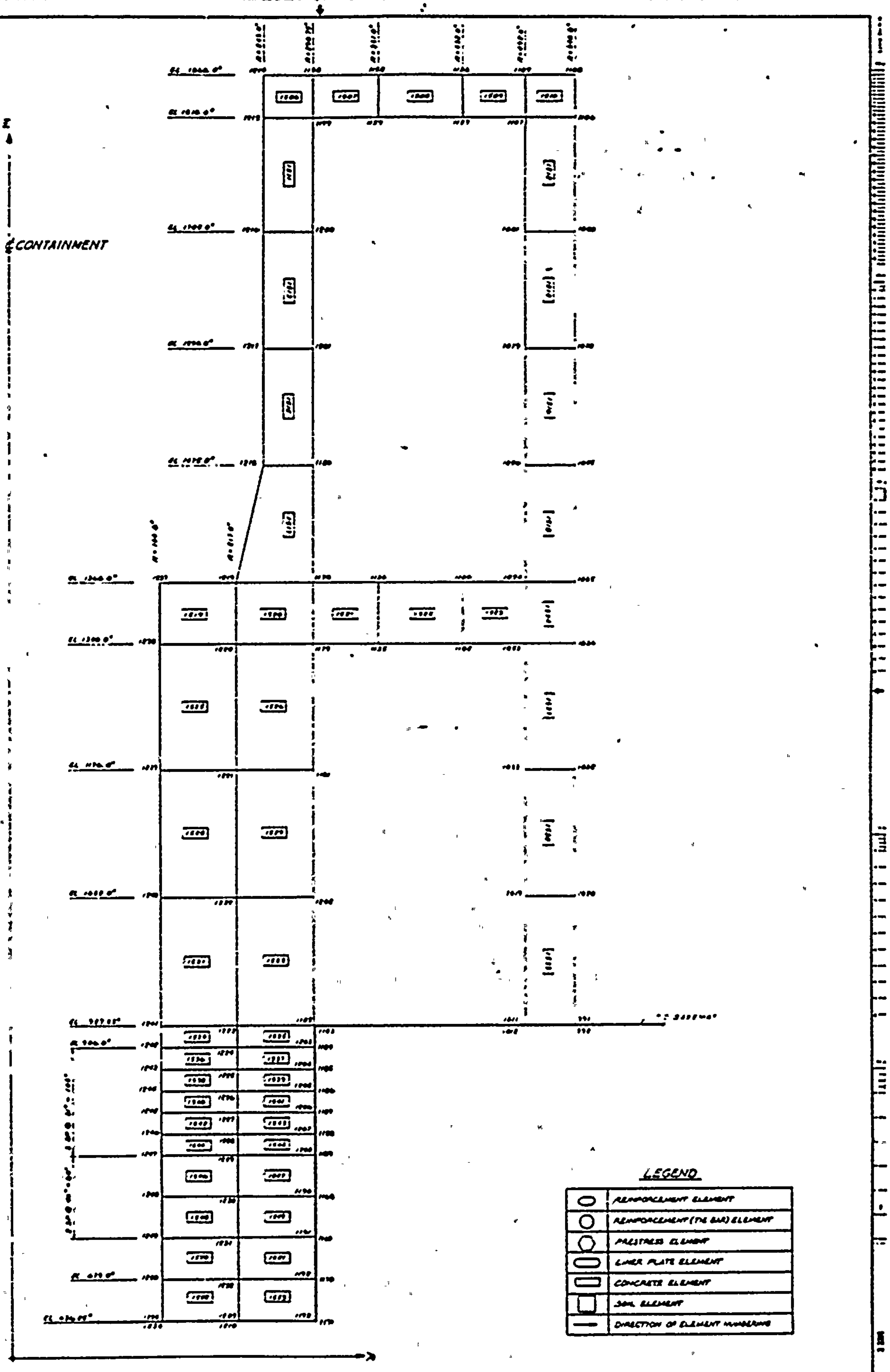

LEGEND
REINFORCEMENT ELEMENT
REINFORCEMENT (REBAR) ELEMENT
PRESTRESS ELEMENT
LINER PLATE ELEMENT
CONCRETE ELEMENT
SOIL ELEMENT
DIRECTION OF ELEMENT NUMBERING
℄ CONTAINMENT
R
BECHTEL
LOS ANGELES
ARIZONA NUCLEAR POWER PROJECT
PALO VERDE NUCLEAR
GENERATING STATION
CONTAINMENT SHELL
FINEL COMPUTER MODEL
SK C-277
A

FIGURE A.1-1



BECHTEL
LOS ANGELES
ARIZONA NUCLEAR POWER PROJECT
PALO VERDE NUCLEAR GENERATING STATION

CONTAINMENT SHELL
FINEL CONTAINER MODEL
SHT 5

A-C-27A

FIGURE A.1-3

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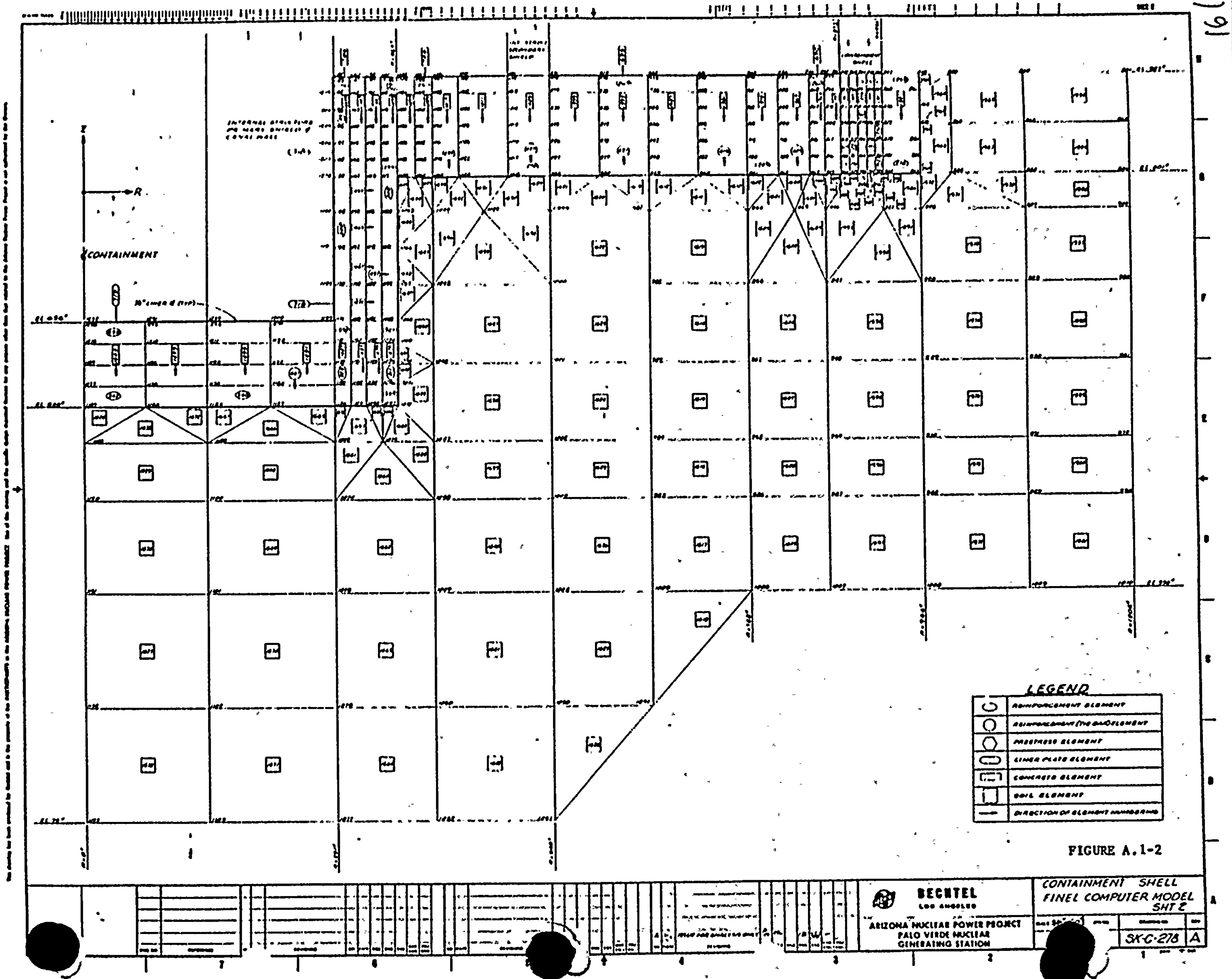


CONTAINMENT
LEGEND
REINFORCEMENT ELEMENT
PRESTRESS ELEMENT
LINER PLATE ELEMENT
CONCRETE ELEMENT
SOIL ELEMENT
DIRECTION OF ELEMENT NUMBERING
BECHTEL
LOS ANGELES
ARIZONA NUCLEAR POWER PROJECT
PALO VERDE NUCLEAR GENERATING STATION
CONTAINMENT SHELL
FINEL COMPUTER MODEL
SHT 2
SK-C-278
A

FIGURE A.1-2

PALO VERDE NUCLEAR STATION UNITS 1, 2, 3
DESIGN ADEQUACY AUDIT

STRUCTURAL AUDIT OF MAIN STEAM SUPPORT STRUCTURE
Part I - General Analysis

I. BASIC DESIGN CRITERIA

A. 'g' value - free field

	Seismic level based on construction permit license	Seismic level used in design of structures and equipment
SSE	0.20g	0.25g
OBE	0.10g	0.13g

Reference: FSAR, Section 3.7

B. Spectra (attach figs. for all damping values, ductilities)

1. zero period acceleration

SSE 0.25g
OBE 0.13g

Reference: FSAR, Figures 3.7-1 -- 3.7-4 (Pages 213-216), and Section 3.7.1.1
This is consistent with Reg. Guide 1.60

Frequency (or period) interval

Refer to BC-TOP-4A, Section 2.5.1 (c)

C. Damping

Refer to FSAR, Section 3.7.1.3
This is consistent with Reg. Guide 1.61
Refer to FSAR Figures 3.7-5 and 3.7-6 (Pages 216A and 216B)

D. Artificial time history and corresponding spectra (attach figures)

1. original time history and its composition, i.e., rising time, strong motion and tail end.

Refer to FSAR, Section 3.7.1.2 and BC-TOP-4A, Section 2.5

2. base line correction, check the integrated velocity and displacement time histories

   The velocity and displacement time histories were checked and found to be satisfactory. In addition, only the acceleration time history was used as input in analysis.

3. time interval - compatible with the highest frequency considered in the spectral calculation

   0.005 seconds

E. Motion duration

   24 seconds

   Refer to FSAR, Section 3.7.1.2

F. components of motion including their relative motion amplitudes

   Analysis was performed for the three principle directions with equal amplitudes.

G. Dead and live loads for various operating floors and base slab

   ~~Refer to Project Design Criteria Part II, Section 3.0 and Part III, Section 4.0~~

   Dead load - includes all structures, major equipment load and 50 psf equivalent for small equipment.

   Live load - see action item 3

H. Ground water level

The groundwater design level is at plant El 70′-0". The actual plant level is at approximately plant EL. 60′-0".

Reference: FSAR, Sections 2.4.13.2.4 and 2.4.13.5.

I. Backfill earth pressure, wind, overpressure due to postulated external explosions (as applicable)

~~Refer to Project Design Criteria Part II, Sections 3.4.5.3 and 3.5.5 for lateral earth pressure, wind, and tornado loads.~~

For lateral earth pressure see Pages 208A – 208D.
Wind does not govern.

J. Other considerations

All penetrations in roof and exterior walls that expose safety related equipment have tornado missile protection. Concrete covers or steel plate shields to prevent missile perforation are provided.

II. ANALYSIS METHOD

A. Seismic Analysis

1. Mathematical model-general description with sketch.

Two planer lumped parameter models were used.

Refer to FSAR Section 3.7.2.3.3 and attachment AX (Page 207)

a. (1) concrete modulus

$E_c = 3.83 \times 10^6$ psi for $f'_c = 4000$ psi

$E_c = 4.03 \times 10^6$ psi for $f'_c = 5000$ psi

(2) rebar modulus and yield strength

$E = 29 \times 10^3$ KSI $F_y = 60$ KSI

(3) Poisson's ratio

$\nu = 0.20$ for concrete

(4) damping

(Page 212)

See FSAR, Section 3.7.1.3 and Table 3.7-1 and response to NRC Question 220.2 (Q 3A-4).

This is consistent with Reg. Guide 1.61

(5) properties of foundation materials

- shear modulus

  Refer to FSAR, Figures 3.7-7, -8, -9 (Pages 217-219)

- subgrade reactions

  ~~Refer to Design Criteria Manual Part II, Section 3.4.5.4 for coefficient of subgrade reaction.~~
  Coefficient of subgrade reaction = 40-60 KIPS/FT²/FT

- bearing capabilities

  Refer to Tables 2.5-15 and 2.5-16 (Pages 209-210) of the FSAR.

(6) other parameters

b. stiffness calculations

(1) exterior walls

Stiffness calculations were performed manually using standard engineering methods.

(2) interior walls

Same as exterior walls.

2. method of Analysis

a. method of analysis used (time history, response spectrum methods, etc.) and consideration of torsional and translational response

Time history analyses were performed to derive in-structure response spectra.

(1) general description

Two planar, lumped parameter models were used in the analysis. Soil structure interaction was incorporated into the model by adding to the fixed-base system discrete soil springs based on elastic half-space theory.

The soil structure interaction analysis method was used and compared against current NRC review position and this meets the intent of NRC soil structure interaction position

(2) findings and comments

b. selection of number of masses and degrees of freedom

(1) general description

For horizontal direction the model consisted of 6 nodes and 12 degrees of freedom.

For vertical direction, the model consisted of 6 nodes and 6 degrees of freedom.

(2) findings and comments

c. number of modes considered

	SSE	OBE
Horizontal (E-W)	4 Modes f = 26.9 cps	4 Modes f = 26.9 cps
Horizontal (N-S)	5 Modes f = 30.2 cps	5 Modes f = 30.3 cps
Vertical	1 Mode f = 5.85 cps	1 Mode f = 6.26 cps

Frequencies shown above are for the highest mode considered.

(1) general description

~~See Attachment.~~

Refer to FSAR, Appendix 3A, Question 3A.6.

(2) findings and comments

d. combining modal responses

(1) actual procedures used

Refer to FSAR, Section 3.7.2.7

(2) general findings

e. consideration of three components of motion

(1) actual procedures used

Refer to FSAR, Section 3.7.2.6
It is consistent with Reg. Guide 1.92

(2) general findings

f. consideration of soil-structure interaction

Soil Springs

	$Kx = 4.67 \times 10^5$ K/FT	$Kxx = 2.89 \times 10^8$ K-FT/RAD
<u>SSE</u>	$Ky = 6.57 \times 10^5$	
	$Kz = 4.67 \times 10^5$	$Kzz = 2.89 \times 10^8$
	$Kx = 6.07 \times 10^5$ K/FT	$Kxx = 3.75 \times 10^8$ K-FT/RAD
<u>OBE</u>	$Ky = 7.78 \times 10^5$	
	$Kz = 6.07 \times 10^5$	$Kzz = 3.75 \times 10^8$

x: North-south direction
y: Vertical direction
z: East-West direction

(1) general description

Refer to FSAR, Section 3.7.2.4.

(2) findings and comments

g. decoupling criteria for subsystems

(1) general procedure

Refer to BC-TOP-4A, Section 3.2
The other criteria pertaining to frequency ratio as defined in SRP 3.7.3.II.3.b are also met.

(2) key examples

(3) general findings and comments

h. modeling of hydrodynamic effects in spent fuel pool

Not applicable.

i. modeling of spent fuel pool w~~e~~alls and interior floor slabs and equipment thereof

Not applicable.

3. development of in-structure response spectra

Refer to FSAR, Section 3.7.2.5

a. general procedures

The "SPECTRA" program was used to compute the response spectra. Refer to FSAR Appendix 3B for a description and applications of this program.

(1) smoothing (describe specific smoothing method used)

The smoothened floor response spectra represent an envelope of the maximum peaks.

(2) peak widening

± 15%

Reference: FSAR, Section 3.7.2.9

b. typical results (attach figures)

Refer to FSAR, Figures 3D-37, -38, -39 (Pages 223 - 225)

B. Stress Analysis

1. shear walls and floors

a. mathematical model - general description w/sketch

Shear wall and floor stresses were computed by performing standard manual calculations. Vertical loads were distributed in the structure by conventional methods. Lateral loads were calculated by multiplying lumped mass and seismic acceleration obtained from floor spectra at each level. These lateral loads were then distributed among the shear walls according to their relative stiffness and location.

b. method of analysis--incorporation of torsion

The minimum torsion moment was taken as the total seismic force times 5% of the long dimension (47′) of the building. Geometric eccentricity is insignificant.

c. load combinations

Refer to FSAR, Section 3.8.3.3.

1. foundation mat

a. mathematical model - description of boundary conditions

The foundation mat was designed as a one-way slab. The slab had fixed boundaries at the junction of the walls with the basemat.

b. method of analysis

The analysis was done manually.

c. load combinations

Refer to FSAR, Section 3.8.3.3.
This is in compliance with SRP

d. key results (figures, etc.)

M_u = 597 Ft-K/Ft
V_u = 210 K/Ft

3. Material to protect against structure - structure interaction

Below grade (EL 100′-0") Rodofoam II (WR Grace and Co.) or EVERLASTIC Micro II (Williams Products, Inc.) may be left in place in seismic joints between structures. Above grade gaps between walls were left open, typically 6" between Category I structures.

a. mechanical properties

b. additional pressure on walls

c. findings and comments

4. vertical dynamic analysis

   a. mathematical model - general description with sketch

      The planer lumped parameter model consisted of one beam founded on a rigid mat, representing the exterior and interior walls.

   b. development of stiffnesses, including floor stiffness, as applicable

      Stiffnesses were calculated manually using standard engineering methods.

   c. method of analysis

      The model described above was used for acceleration time-history analysis using Fosin.

C. Computer Programs Used in Analysis

LUCON, SMIS, FOSIN, SPECTRA

1. assumptions and limitations

   Refer to FSAR, Appendix 3B.

2. applicability

   Refer to FSAR, Appendix 3B.

3. verification

   - sensitivity study in case of numerical solutions (e.g., finite element analysis)

     Refer to FSAR, Appendix 3B.

4. load input (include all cases)

PROGRAM	INPUT
SMIS	Mass matrix, stiffness matrix, damping values.
SPECTRA	In-structure time histories, frequency or periods and damping values.
FOSIN	Free-field time history, damping values, frequency.
LUCON	Shear modules, damping values.

5. output (include all cases)

   Refer to FSAR, Appendix 3B.

6. other discussions

D. Overall Stability

1. forces and moments from seismic analysis

Elastic forces and moments

Horizontal Force		Overturning Moment	
OBE	SSE	OBE	SSE
4270^K	6160^K	241,900 K-FT	348,800 K-FT

2. various cases considered

Seismic loading combinations considered an SSE or OBE applied in the North-South, East-West and vertical directions simultaneously.

3. bearing pressure versus bearing capability and safety factor against bearing failure

Refer to FSAR, Section 2.5.4.10 and Tables 2.5-15, -16 (Pages 209-210)

4. factors of safety

Refer to FSAR, Section 3.8.5.5 and Table 3.8-5 (Page 222)

a. sliding

Factor of Safety = 1.1 (SSE)

b. overturning

Factor of Safety = 91 (SSE)

E. Interaction of Non-category I Structures with the Structure Considered

1. identification of pertinent non-Category I structures

   The Corridor and Turbine Buildings are adjacent to the Main Steam Support Structure.

2. consideration given to potential failure of non-Category I systems on Category I systems

   The ~~Corridor and~~ Turbine Building~~s~~ is ~~are~~ designed to preclude structural failure of building or parts thereof that could damage the Main Steam Support Structure and its Category I systems. Within the Main Steam Support Structure, non-Category I systems which potentially could affect Category I systems, are designed for structural integrity under SSE equivalent loads. During a walkdown, those items whose failure will not affect any safety *

   Reference: FSAR, Section 3.8.4.4

3. general findings and comments

* related equipment are left as they are. If they are judged that they might affect category I systems, they are designed to maintain their structural integrity under SSE.

F. Design Consideration for Tornado Missiles

1. design requirements

Refer to FSAR, Sections 3.5.1.4 and 3.5.3, and Table 3.5-8x (Page 2U)

2. models for

a. local damage

Refer to FSAR, Section 3.5.3.

b. overall response

Refer to FSAR, Section 3.5.3.

3. load combinations

Refer to FSAR, Section 3.8.3.3.

4. forces

The governing structural members are of sufficient thickness to preclude perforation by postulated missiles and maintain structural integrity.

6. general comments and preliminary audit findings

III. CONFORMANCE TO ACCEPTABLE CRITERIA

A. Identification of deviations, if any

None.

B. Justification of deviations and disposition of the deviations

D. general comments

<u>Part II Audit of Key Designs</u>

A. Exterior Shear Walls

1. design requirements

   The walls were designed to satisfy structural functions as bearing walls, shear walls, resistance to external and internal pressures, and protection against tornado missiles.

2. design loads (from general analysts)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4L (Page 220) This table is being amended.

4. detailed design of rebar placement at key sections

   See Attachment B (Pages 202-208)

5. general comments and preliminary audit findings

B. Interior Shear Walls

1. design requirements

   Same as exterior walls.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4L* (Page 220)
   This table is being amended

4. detailed design of rebar placement at key sections

   See Attachment B* (Pages 202-208)

5. general comments and preliminary audit findings

C. Main Floors and Roofs

1. design requirements

The main floor and roof were primarily designed for vertical dead, live and seismic loads.~~as defined in the Project Design Criteria~~. The roof were also designed to satisfy minimum thickness to preclude perforation by tornado generated missiles.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4L~~x~~ (Page 220)

This table is being amended.

4. detailed design of rebar placement at key sections

See Attachment B~~b~~ (Pages 202-208)

5. general comments and preliminary audit findings

D. Steel Structural Bracing Systems (if any)

1. design requirements

The main steam support structure roof is elevated above the top of the walls by a structural steel frame. The frame was designed for dead, live, seismic and accident pressure. ~~per the Project Design Criteria.~~

2. design loads

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4M ~~K~~ (Page 221)

6. general comments and preliminary audit findings

E. Foundation Mat

1. design requirements

   Refer to FSAR, Section 3.8.5.4.2 and 3.8.5.5

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   M_u = 597 Ft-K/Ft

   $V\cancel{X}_u$ = 210 K/Ft

4. detailed design of rebar placement at key sections

   See Attachment B* (Pages 202-208).

5. general comments and preliminary audit findings

F. Main Frame Concrete Column Design (Key Columns)

1. design requirements

   Not applicable. There are no concrete columns in the main steam support structure.

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design of rebar placement at key sections

5. general comments and preliminary audit findings

G. Secondary Floors

1. design requirements

The Main Steam Support Structure has steel grating platforms spanning between the concrete walls. The structural steel is designed for dead, live, seismic and pipe support loads.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4M (Page 221)

4. detailed design of rebar placement at key sections

See Attachment B (Pages 202-208)

5. general comments and preliminary audit findings

H. Detailing at Floor-Wall Joints

1. design requirements

   As per ACI 318-71 Code, Chapters 6 and 17.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Moment = 103.5 K-FT/FT (positive & negative)

   Shear = 191.3 psi/FT stirrups are provided

4. detailed design of rebar placement at key sections

   See Attachment B X (Pages 202 -208)

5. general comments and preliminary audit findings

I. Dynamic Effects Applied to Floors and Walls by Machinery

1. design requirements

   Dynamic effects from machinery are negligible.

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design

5. general comments and preliminary audit findings

J. Crane & Support

1. design of bents (columns and roof trusses)

Not applicable. There are no cranes in the MSSS.

a. design requirements

b. design loads (from general analysis)

c. forces and moments at key sections

d. detailed design

e. general comments and preliminary audit findings

2. design of girders supporting crane rails

   Not applicable.

   a. design requirements

   b. design loads (from general analysis)

   c. forces and moments at key sections

   d. detailed design

e. general comments and preliminary audit findings

3. design of spent fuel bridge

Not applicable.

a. design requirements

b. design loads (from general analysis)

c. forces and moments at key sections

d. detailed design

e. general comments and preliminary audit findings

K. Fuel Pool Liner Design

Not applicable.

1. stresses and strain controls

2. conformance to code requirements

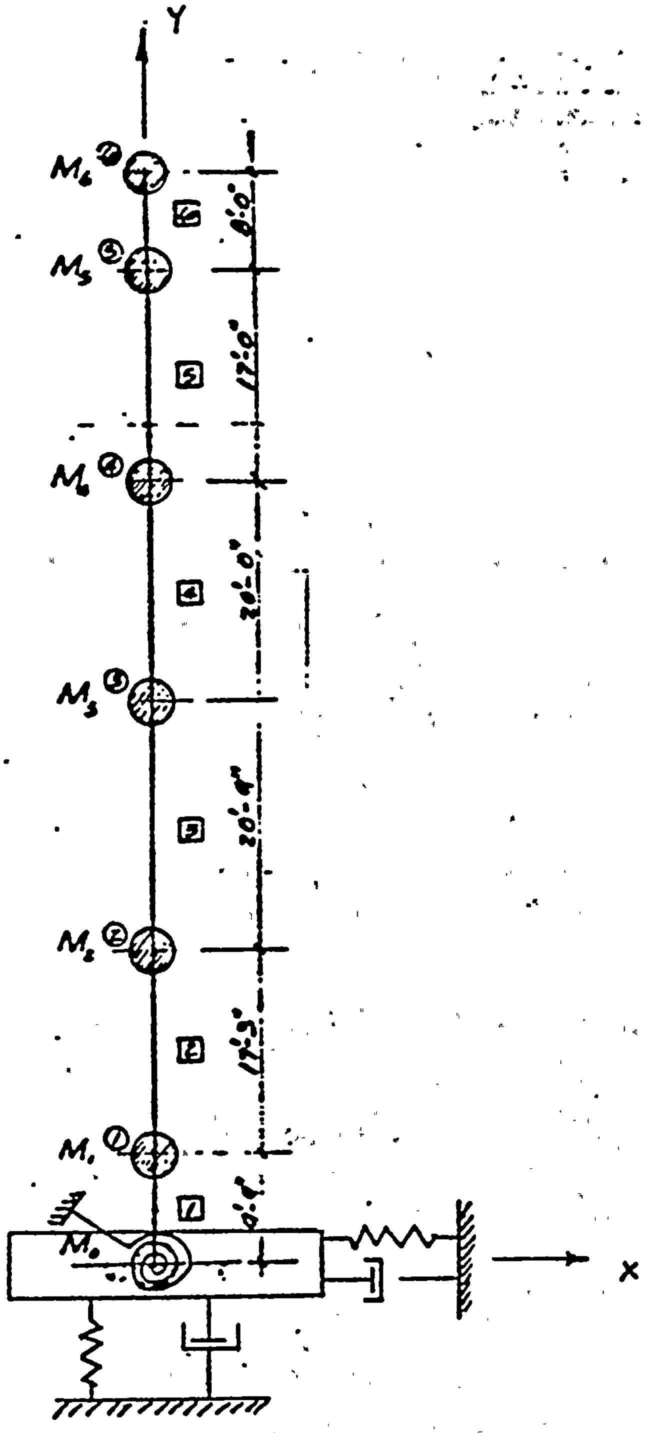
3. analysis procedure and results

4. consideration of accidental drop of crane loads

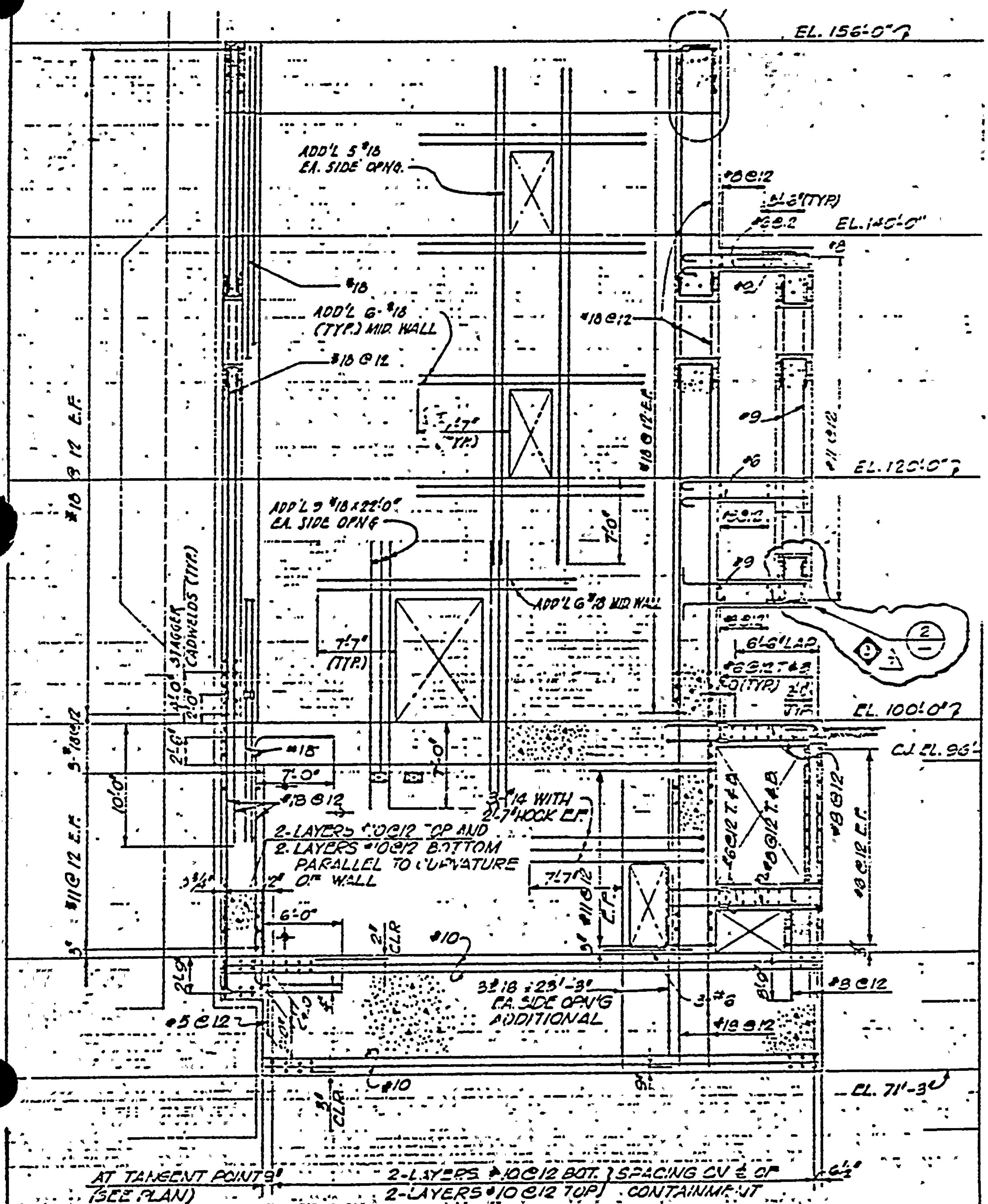
5. corrosion effects (e.g., pitting) on liner integrity

6. preliminary findings of audit results

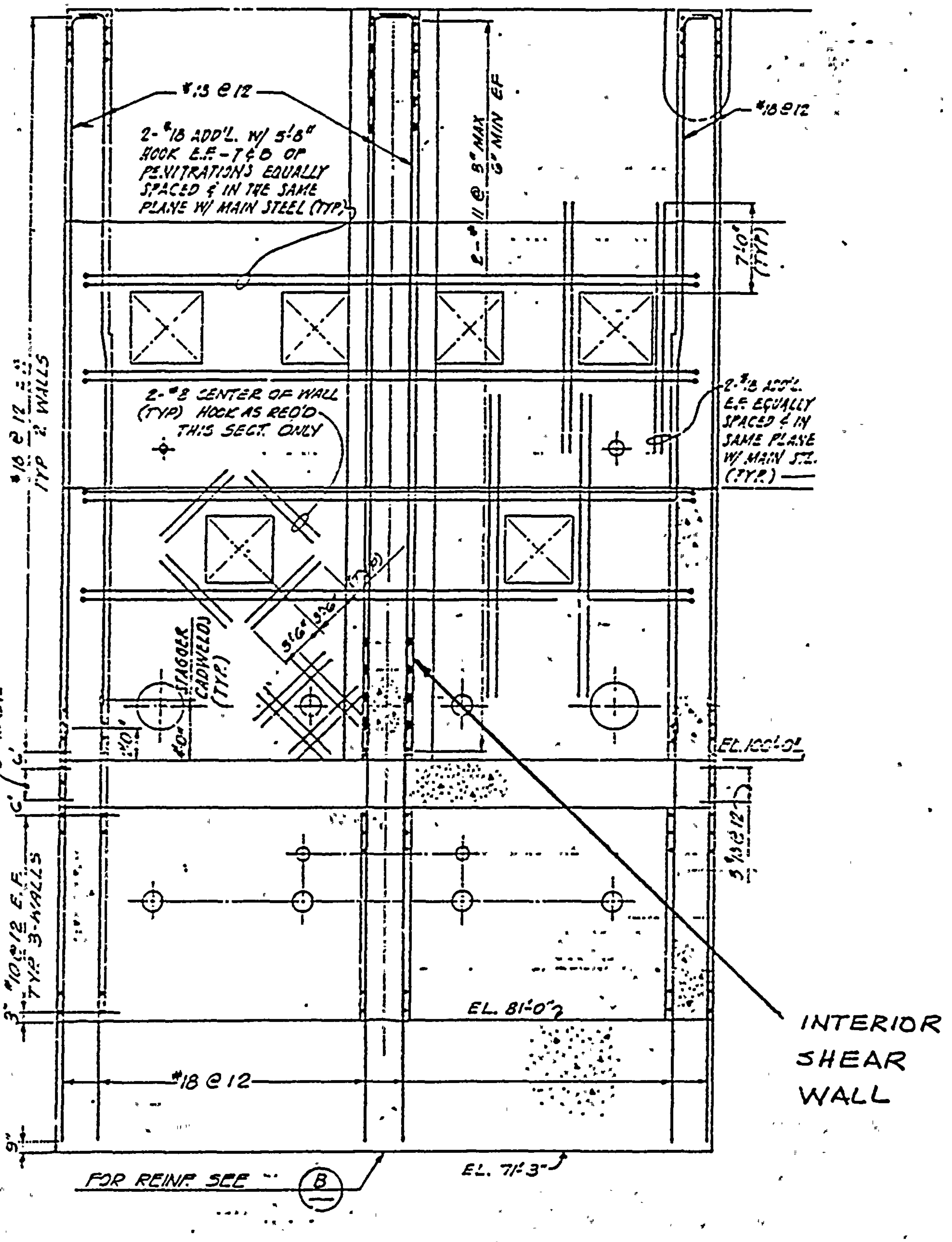
ATTACHMENT A



MAIN STEAM SUPPORT STRUCTURE.

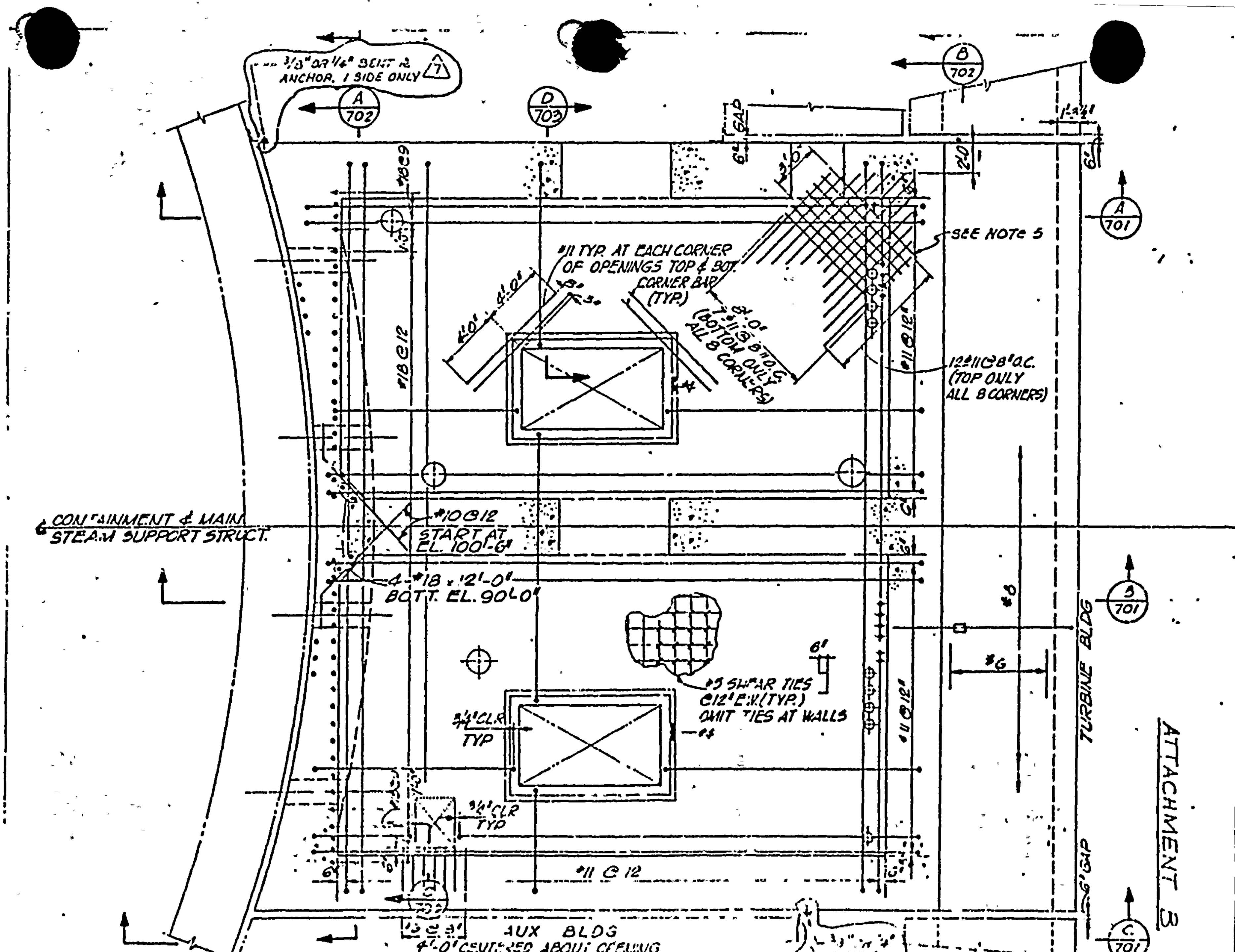


MAIN STEAM SUPPORT STRUCTURE EXTERIOR SHEAR WALL



MAIN STEAM SUPPORT STRUCTURE ... INTERIOR SHEAR WALL

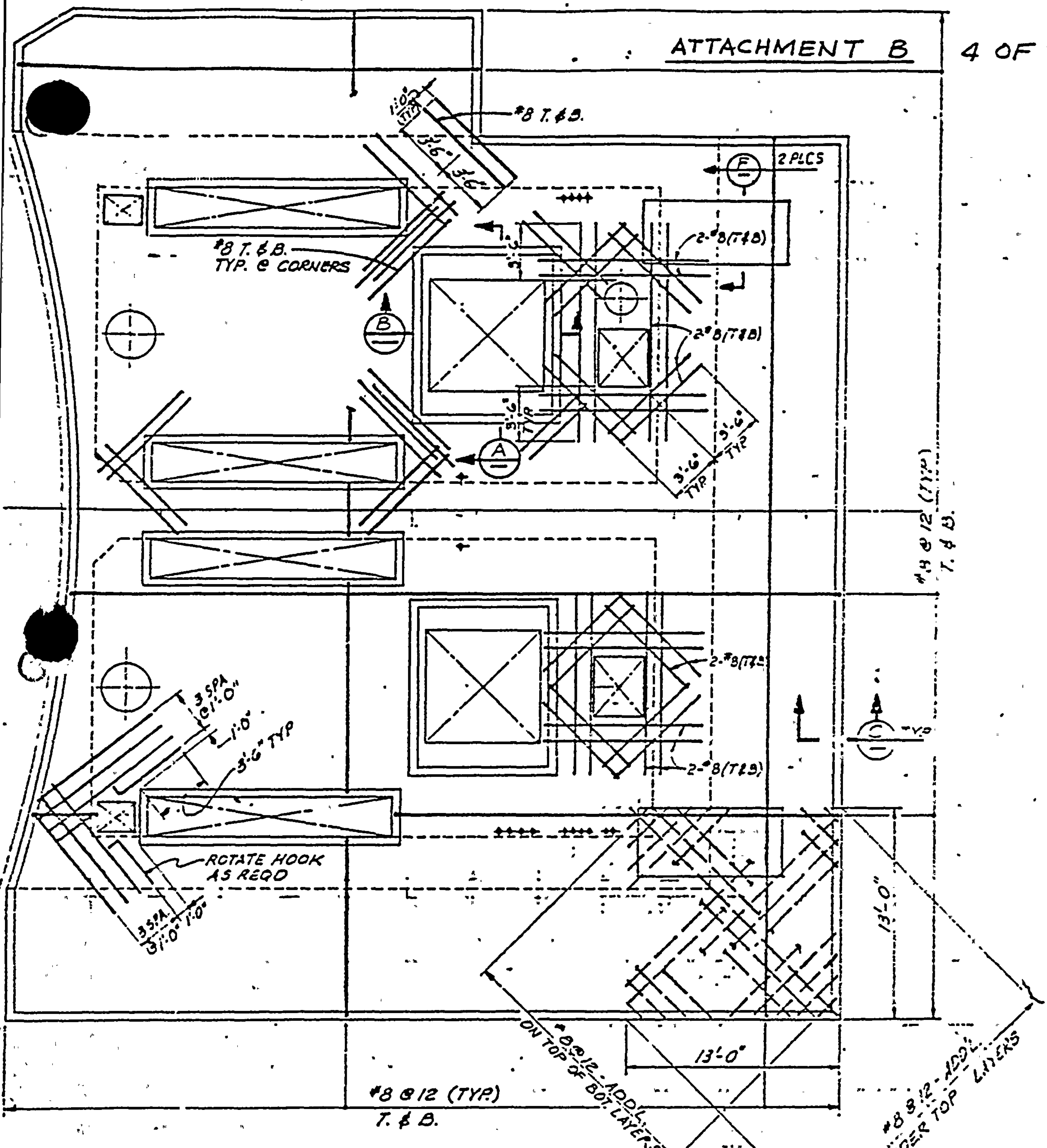
SECTION A/700



FLOOR PLAN T.O.C. EL 100'-0"

MAIN STEAM SUPPORT STRUCTURE PRIMARY FLOOR

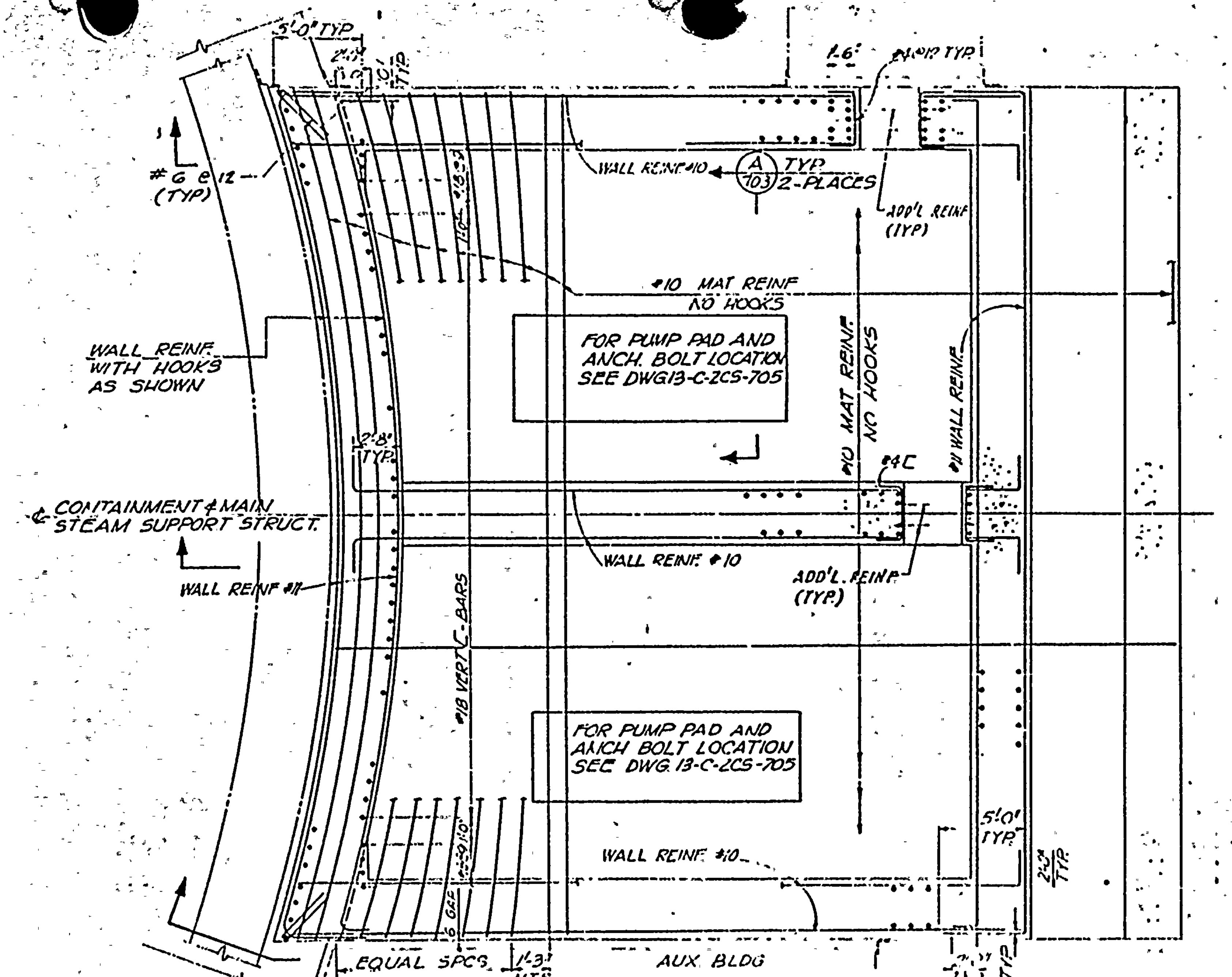
ATTACHMENT 3



REINFORCING PLAN @ EL. 166'-11"
1/4" = 1'-0"

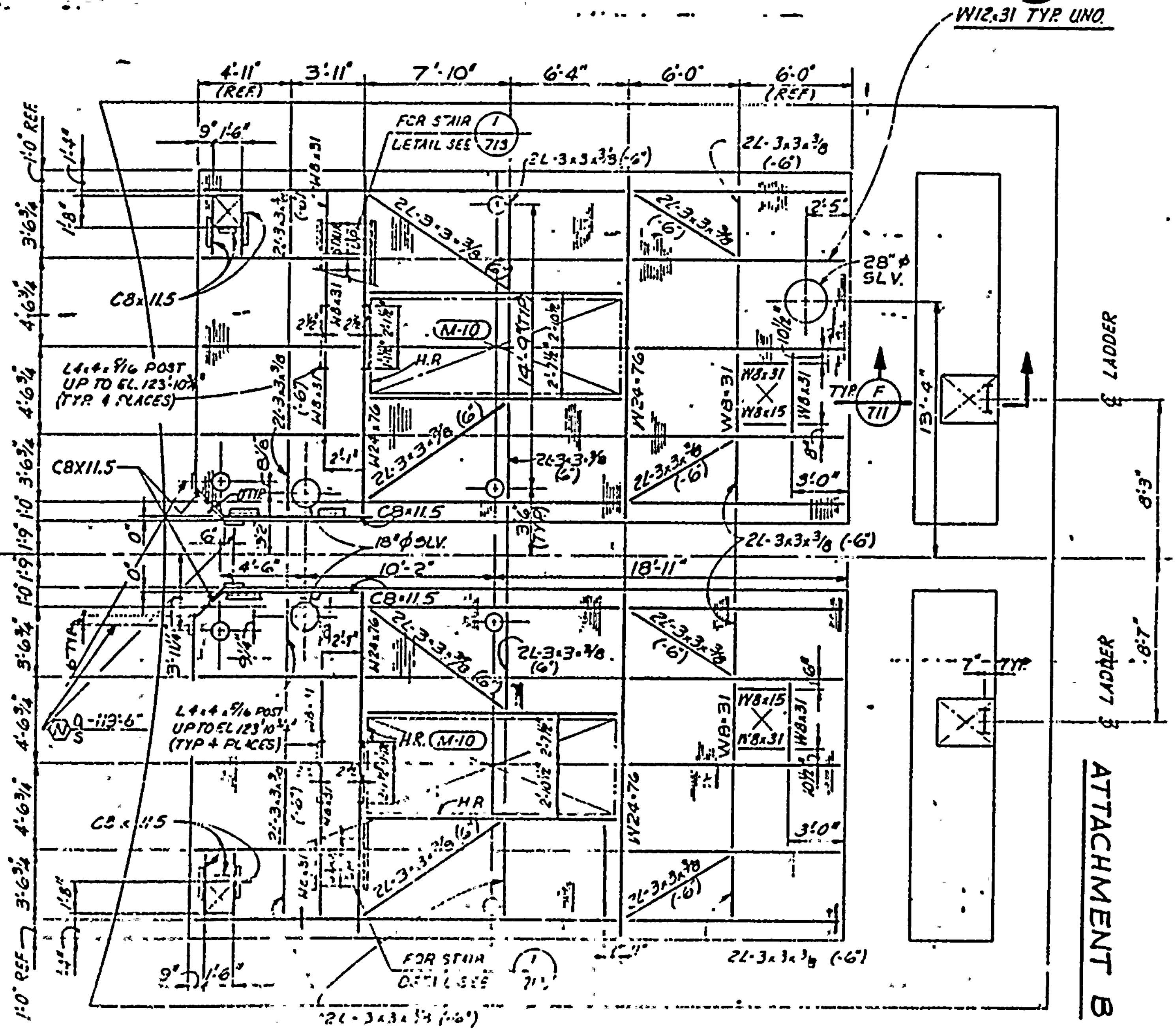
PLAN OF ROOF

MAIN STEAM SUPPORT STRUCTURE



FOUNDATION PLAN T.O.C. EL. 81'-0"

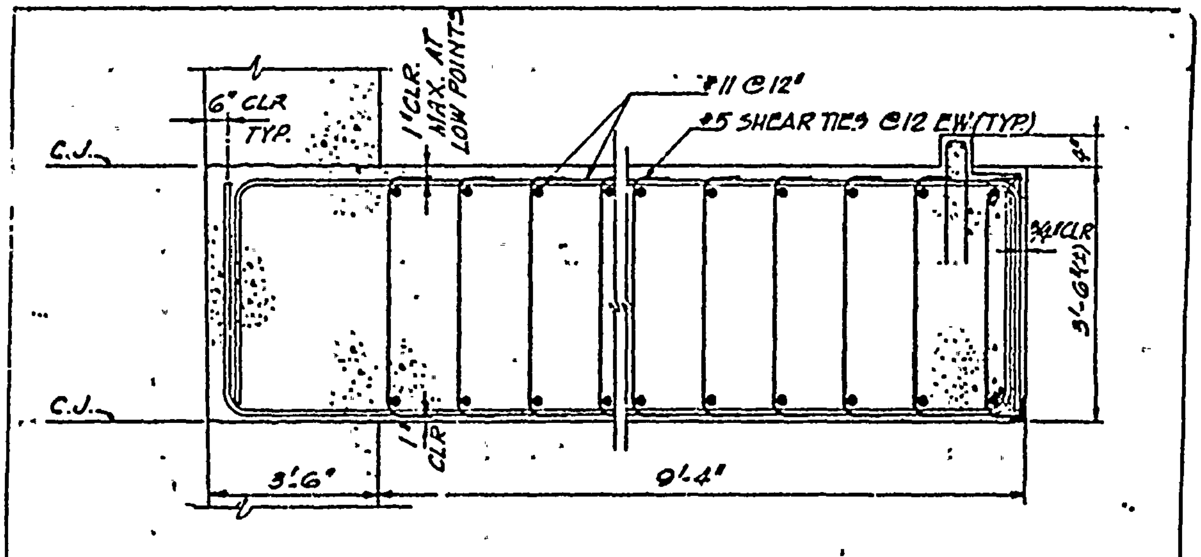
MAIN STEAM SUPPORT STRUCTURE



T.O.G. EL. 120'-0" T.O.S EL. (-1 1/2") U.N.O.

MAIN STEAM SUPPORT STRUCTURE SECONDARY FLOOR

ATTACHMENT B



SECTION D/700
3/4" = 1'-0"

MAIN STEAM SUPPORT STRUCTURE

TYPICAL WALL - FLOOR JUNCTION

Lateral Earth Pressure

The total lateral earth pressure on foundation and retaining walls shall be based on the sum of the appropriate static and dynamic lateral forces. Static forces shall be based either on the active case (P_A) for the case of a retaining wall free to rotate and translate, or on the compacted backfill case (P_B) for the case of a rigid foundation wall. Dynamic forces shall be based on the dynamic increment for a level backfill condition (P_{AE}) plus the surcharge effect (P_{AES}), if applicable.

A. Static conditions:

	Equivalent Fluid Unit Weight (lb/ft^3) (Horizontal Backfill)	
Case	Above Water Table	Below Water Table
Active (P_A)	36	19
Passive (P_P)	228	118
Backfill (P_B)	90	47

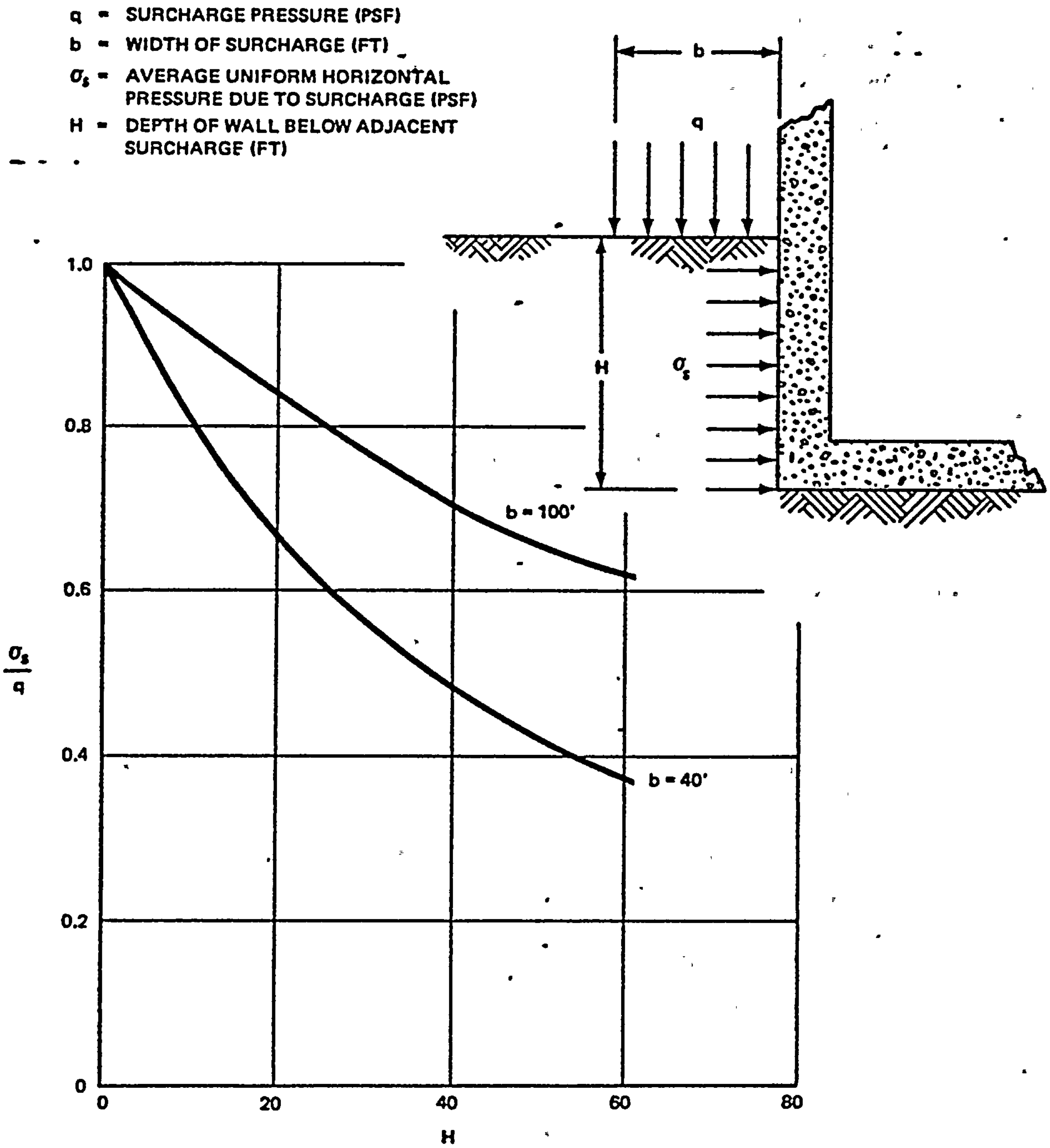
The increment of lateral pressure due to adjacent surcharge for the case of a rigid foundation wall shall be computed using Figure 1.

The increment of lateral pressure due to adjacent surcharge for the case of a retaining wall free to rotate and translate shall be computed using Figure 2.

B. Dynamic conditions:

The dynamic lateral force increment due to seismic effects shall be computed in accordance with Figure 3.

The total dynamic lateral force increment, P_{AE} or $P_{AE} + P_{AES}$, shall be added to the lateral force calculated for either the active case or the compacted backfill case. The lateral force is calculated in the usual manner and includes hydrostatic pressure if a water table is present. The dynamic lateral force increments are independent of the water table.



AVERAGE LATERAL PRESSURE DUE TO ADJACENT SURCHARGE FOR RIGID RETAINING WALL

Figure. 1

σ_s = AVERAGE UNIFORM HORIZONTAL PRESSURE DUE TO SURCHARGE (PSF)

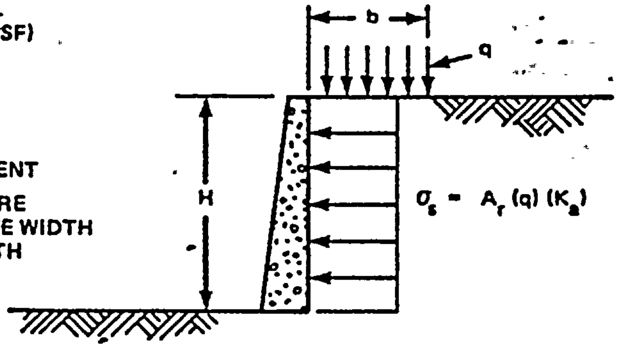
q = SURCHARGE PRESSURE (PSF)

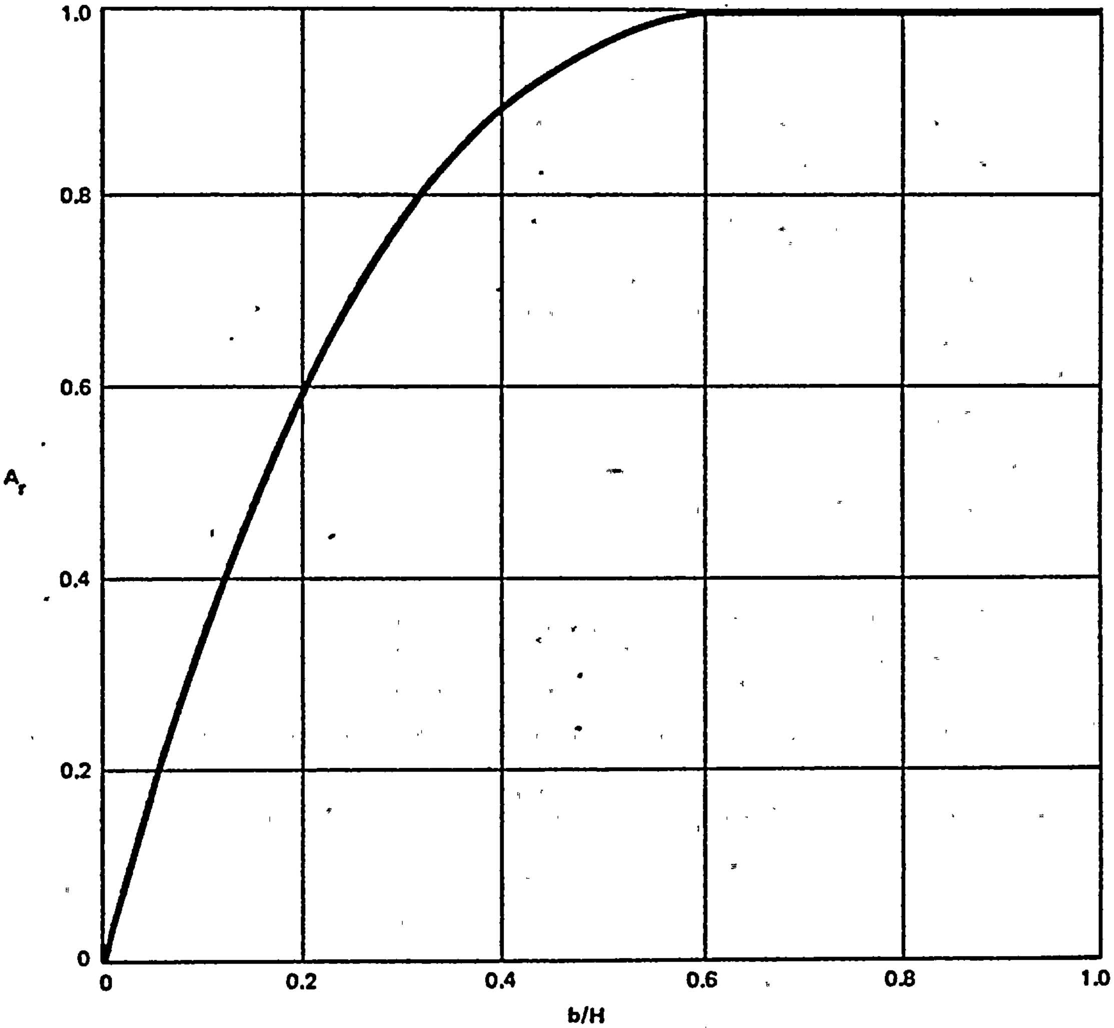
b = WIDTH OF SURCHARGE (FT)

H = HEIGHT OF WALL (FT)

K_a = 0.28, ACTIVE PRESSURE COEFFICIENT

A_r = RATIO OF ACTIVE EARTH PRESSURE CAUSED BY SURCHARGE OF FINITE WIDTH TO SURCHARGE OF INFINITE WIDTH

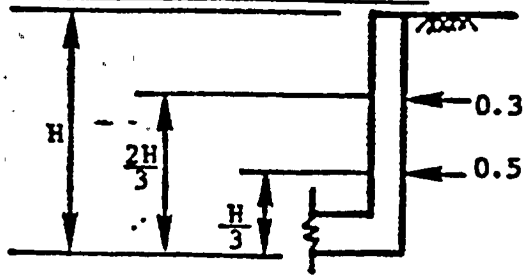




AVERAGE LATERAL PRESSURE DUE TO ADJACENT SURCHARGE FOR RETAINING WALL FREE TO ROTATE

Figure 2

Level Backfill Condition



$$P_{AE} = 0.5\ k_v K_A \gamma_t H^2 + 0.3(1 + k_v) k_h \gamma_t H^2$$

where: P_{AE} = dynamic lateral force increment due to seismic effects (lb)

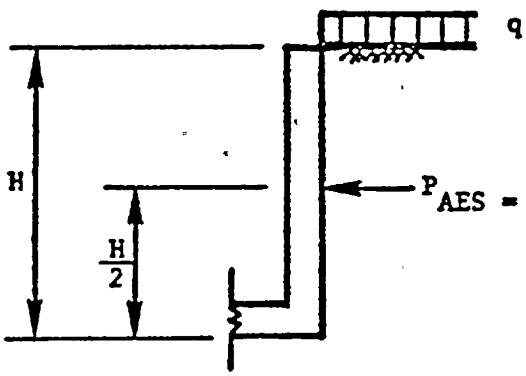
K_A = 0.28, coefficient of active lateral earth pressure.

k_h = k_v = 0.25 under SSE conditions, and 0.13 under OBE conditions

H = wall height (ft)

γ_t = total unit weight (129 lb/ft^3)

Surcharge Effect



where: P_{AES} = dynamic lateral force increment due to adjacent surcharge loading (lb)

q = Surcharge pressure (lb/ft^3)

H = Depth of wall below adjacent surcharge (ft)

K_A = 0.28

k_h = k_v = 0.25 under SSE conditions, and 0.13 under OBE conditions

DYNAMIC LATERAL FORCE INCREMENT DUE TO SEISMIC EFFECTS

Figure 3

Table 2.5-15

STATIC BEARING CAPACITY OF CATEGORY I STRUCTURES

Structure	Average Static Design Load q_s (k/ft^2)	Ultimate Bearing Capacity q_o (k/ft^2)	Factor of Safety (q_o/q_s)
Containment Building	7.9	35.7	4.5
Auxiliary Building (deep section)	6.2	34.9	5.6
Main Steam Support Structure	7.1	64.8	9.1
Control Building	3.3	45.3	13.7
Fuel Building	5.3	54.9	10.4
Diesel Generator Building	3.1	79.5	25.6
Refueling Water Tank	4.4	90.4	20.5
Condensate Storage Tank	3.5	112.4	32.1

Table 2.5-16

DYNAMIC BEARING CAPACITY OF CATEGORY I STRUCTURES[a]

Structure	Equivalent Uniform Vertical Stress q_d (k/ft^2)	Ultimate Bearing Capacity q_o (k/ft^2)	Factor of Safety (q_o/q_d)
Containment Building	16.1	32.2	2.0
Auxiliary Building (deep section)	10.3	25.8	2.5
Main Steam Support Structure	25.3	60.6	2.4
Control Building	9.8	39.8	4.1
Fuel Building	19.1	50.3	2.6
Diesel Generator Building	5.6	75.5	13.5
Refueling Water Tank	13.2	58.7	4.4
Condensate Storage Tank[b]	13.2	30.2	2.3

a. Based upon maximum dynamic loads derived from analyses described in section 3.7.

b. Condensate storage tank loads were conservatively chosen to be equal to the dynamic design load for the refueling water tank. Actual loads will be less.

1

240

Table 3.5-8

TORNADO-GENERATED MISSILES CONSIDERED IN DESIGN OF SAFE SHUTDOWN STRUCTURES

Description	Weight (lbs)	Impact Area (ft^2)	Maximum Velocity (ft/s)	Kinetic Energy (ft-lbs)
(A) A 12-foot wood plank, 4 x 12 inches in cross-section, traveling end on at a speed of 240 mi/h.	200	0.333	352	3.85×10^5
(B) A steel pipe, Schedule 40, 3 inches in diameter by 10 feet long, traveling end on at 120 mi/h.	78	0.063	176	3.75×10^4
(C) A steel rod, 1 inch in diameter, 3 feet long, traveling end on at 180 mi/h.	8	0.005	264	8.66×10^3
(D) A steel pipe, Schedule 40, 6 inches in diameter by 15 feet long, traveling end on at 120 mi/h.	285	0.24	176	1.37×10^5
(E) A steel pipe, Schedule 40, 12 inches in diameter by 15 feet long, traveling end on at 120 mi/h.	743	0.886	176	3.57×10^5
(F) A utility pole, 13-1/2 inches in diameter, 35 feet long, traveling end on at 120 mi/h.	1490	0.994	176	7.17×10^5
(G) An automobile of 4,000 pounds weight, striking the structure at 60 mi/h.	4000	20.0	88	4.81×10^5

Table 3.7-1
DAMPING VALUES
(PERCENT OF CRITICAL DAMPING)

Structure or Component	Operating Basis Earthquake	Safe Shutdown Earthquake
Equipment and large-diameter piping systems, pipe diameter greater than 12 in.	2	3
Small-diameter piping systems, diameter equal to or less than 12 in.	1	2
Welded steel structures	2	4
Bolted steel structures	4	7
Prestressed concrete structures	2	5
Reinforced concrete structures	4	7

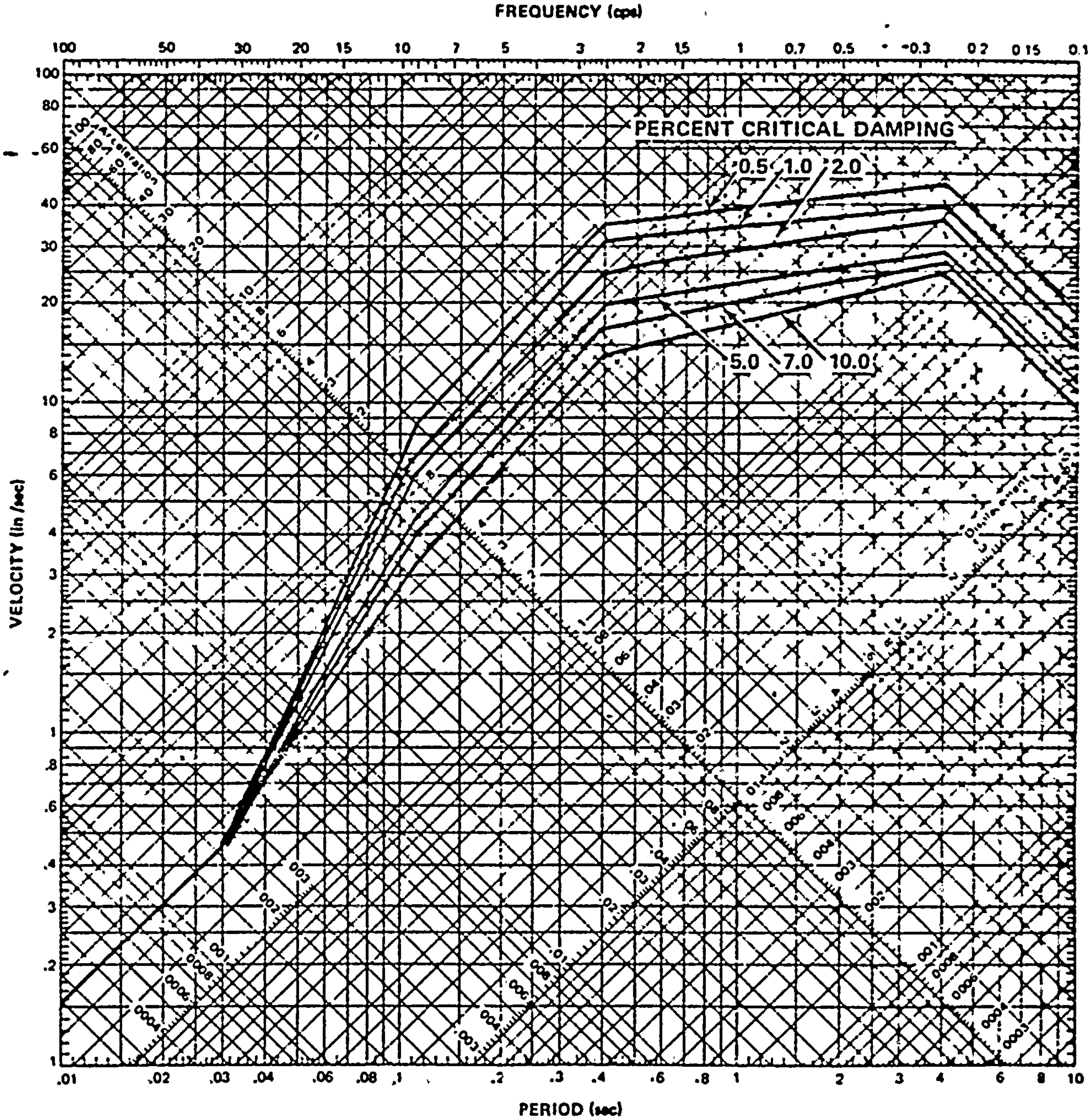
The applicable allowable design levels are given in section 3.8 for the various loading combinations which include seismic loadings.

3.7.1.4 Supporting Media for Seismic Category I Structures

For purposes of the seismic analysis, the site is assumed to be a multi-layer system consisting of soil over bedrock. The approximate depth of soil deposit over bedrock for each unit at the site is as follows:

	Unit 1	Unit 2	Unit 3
Depth of Soil, ft	330	350	295

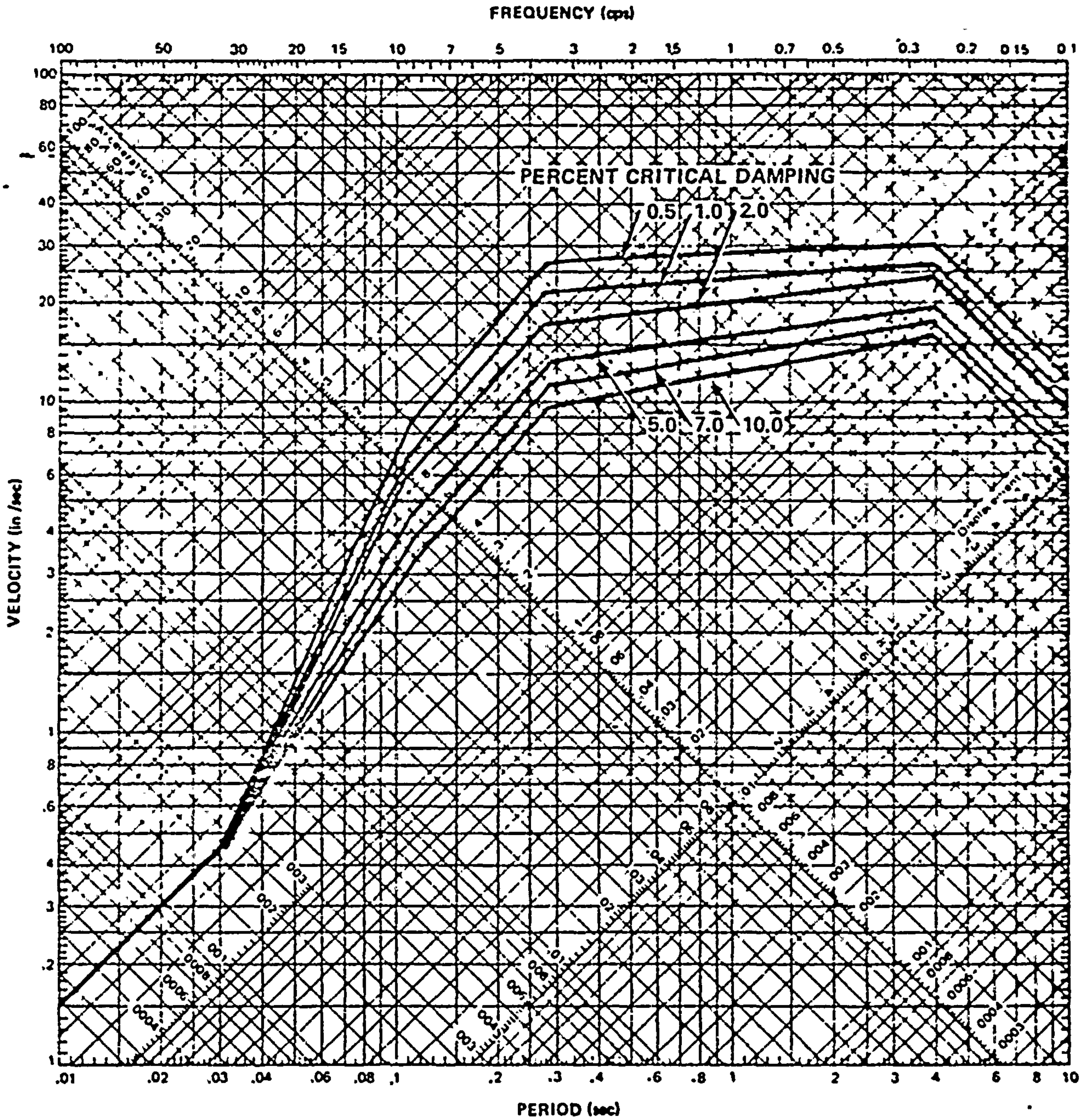
2/28



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR SSE 0.25 g

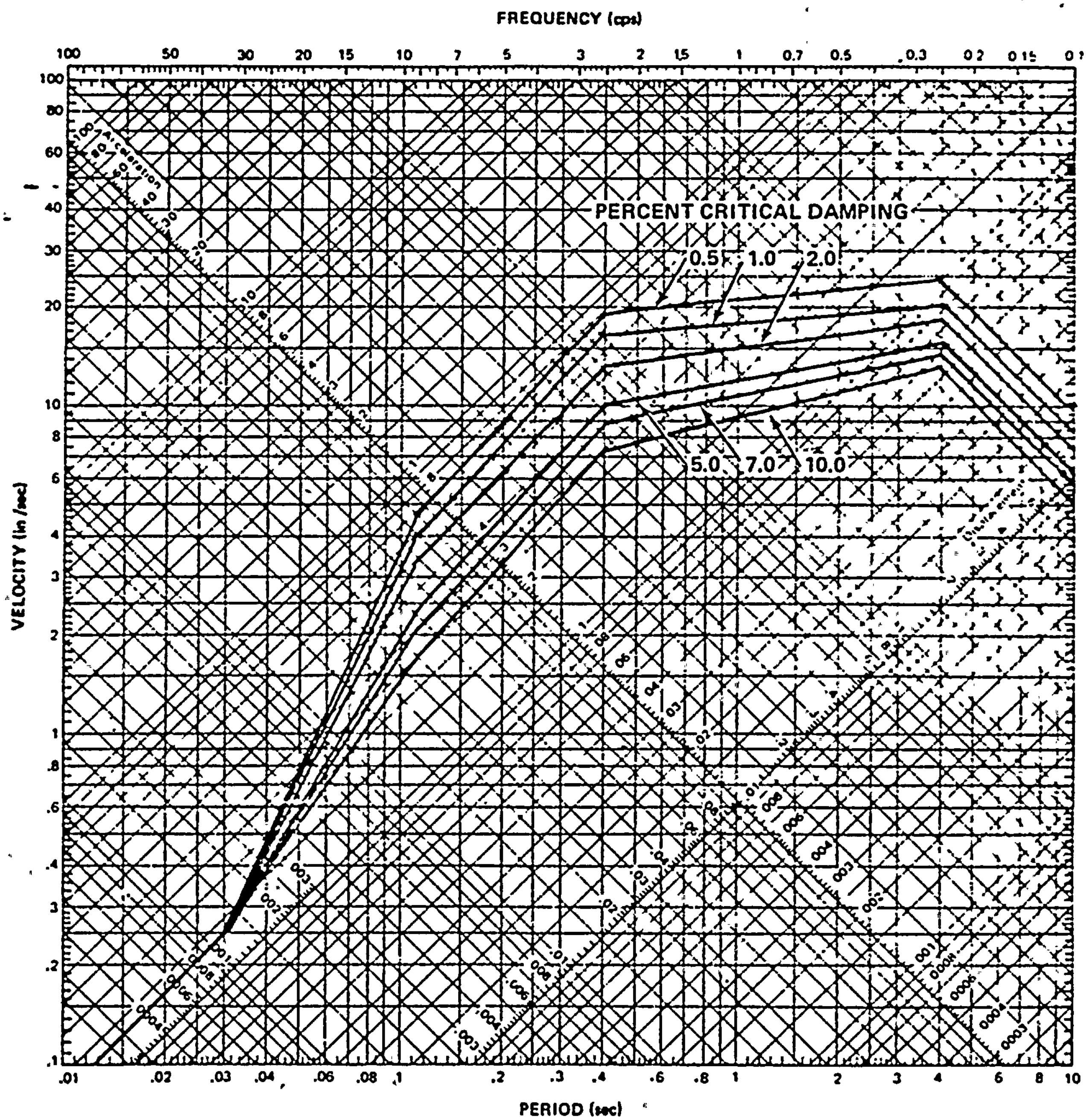
Figure 3.7-1



Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR SSE 0.25 g

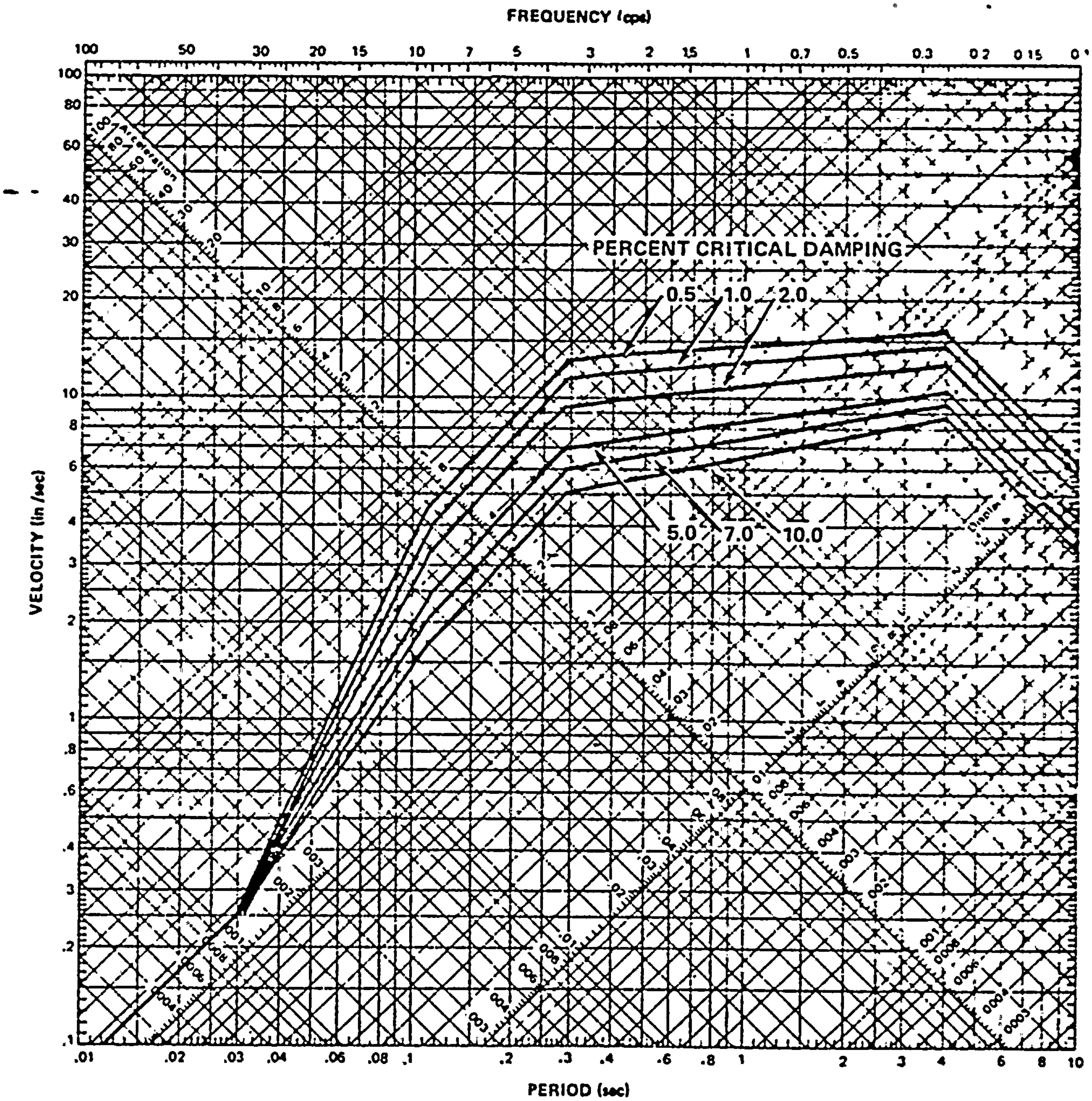
Figure 3.7-2



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-3

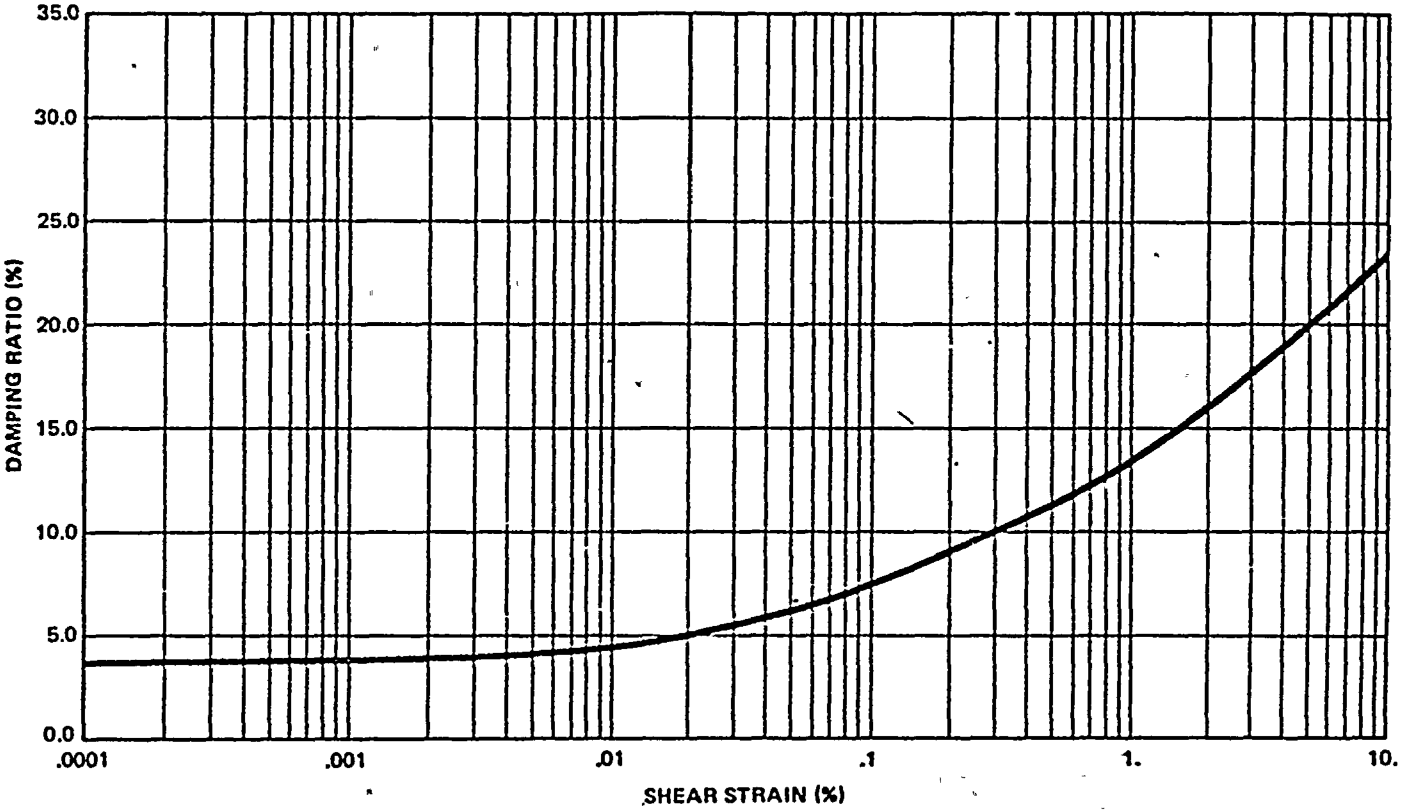


Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-4

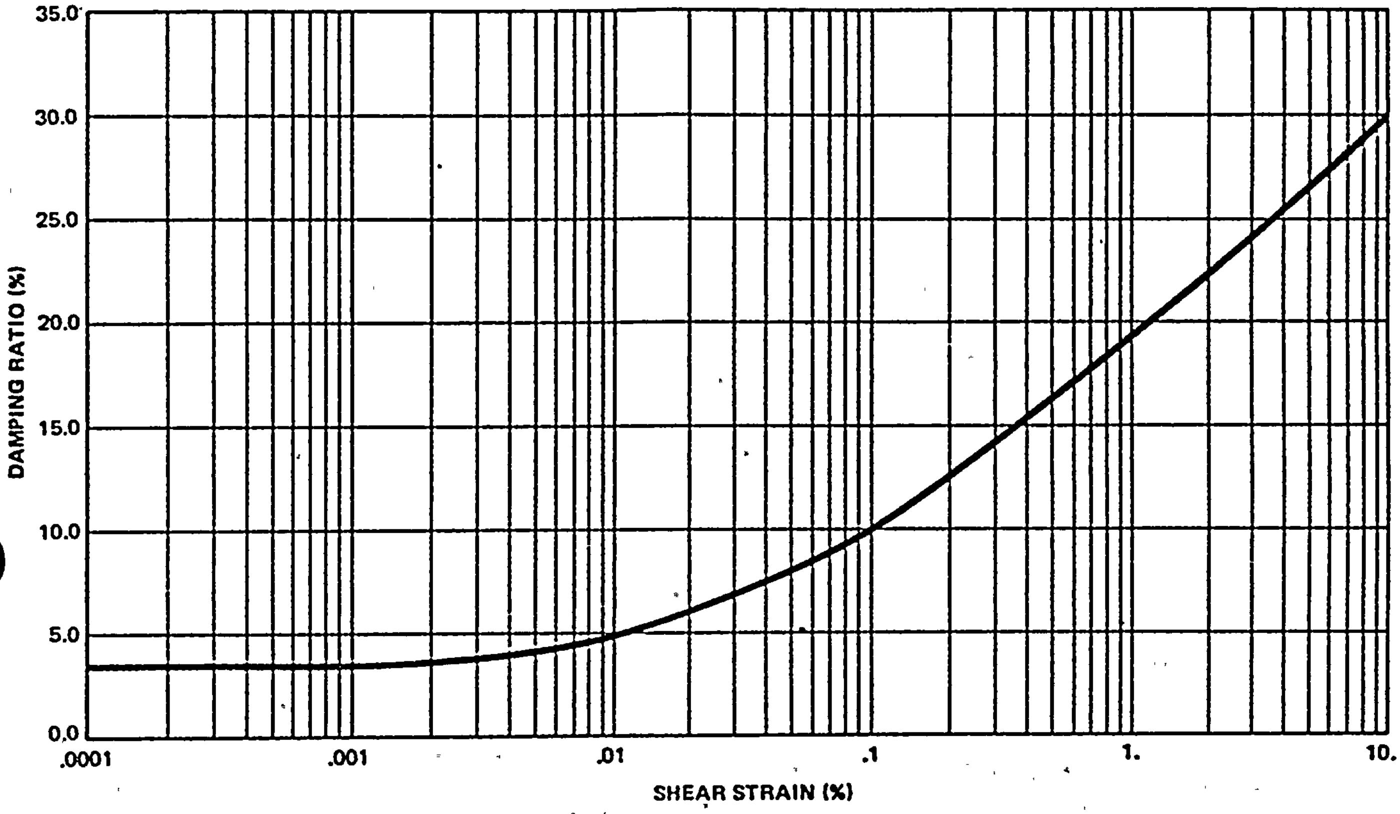
571



Palo Verde Nuclear Generating Station
FSAR

DAMPING VS. STRAIN - CLAY

FIGURE 3.7-5



Palo Verde Nuclear Generating Station
FSAR

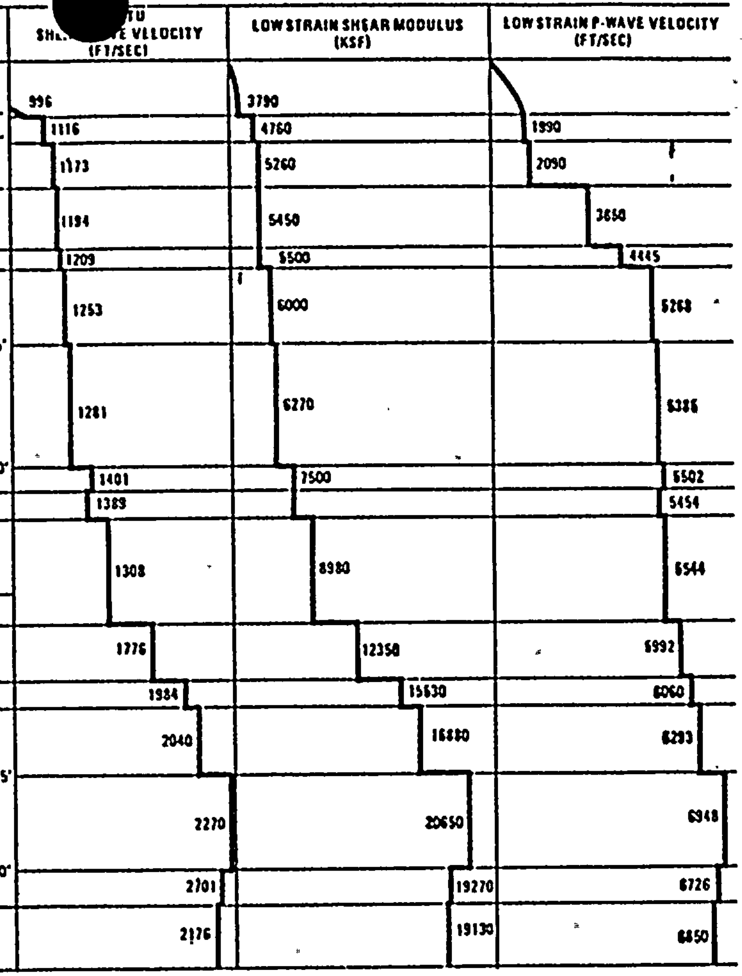
DAMPING VS. STRAIN - SAND

FIGURE 3.7-6

DEPTH (FT.)	LAYER DEPTH (FT.)	DESCRIPTION	LAYER THICKNESS	UNIT WEIGHT (PCF)	POISSON'S RATIO
	0'–47'	SAND (2)	47'	123	.27
	47'–70'	CLAY (1)	23'	121	.44
	70'–77'	SAND (2)	7'	121	.46
	77'–159'	CLAY (1)	82'	123	.47
	159'–169'	SAND (2)	10'	125	.465
	169'–196'	CLAY (1)	27'	127	.46
	196'–206'	SAND (2)	12'	127	.46
	206'–228'	CLAY (1)	20'	126	.45
	228'–238'	SAND (2)	10'	127	.44
	238'–311'	CLAY (1)	73'	128	.44
	311'–334'	SAND (2)	23'	130	.44
	334'	BEDROCK			

AVERAGE PERCHED WATER LEVEL 44'

Depth scale (FT.): 50, 100, 150, 200, 250, 300, 350



NOTES: 1. LOW STRAIN SHEAR MODULI AT VARIOUS DEPTHS ARE AVERAGE VALUES.
2. SHEAR WAVE VELOCITY CALCULATED FROM $V_S = (G/\rho)^{1/2}$
3. P-WAVE VELOCITY CALCULATED FROM $V_P = V_S\,[(2-2\nu)/(1-2\nu)]^{1/2}$

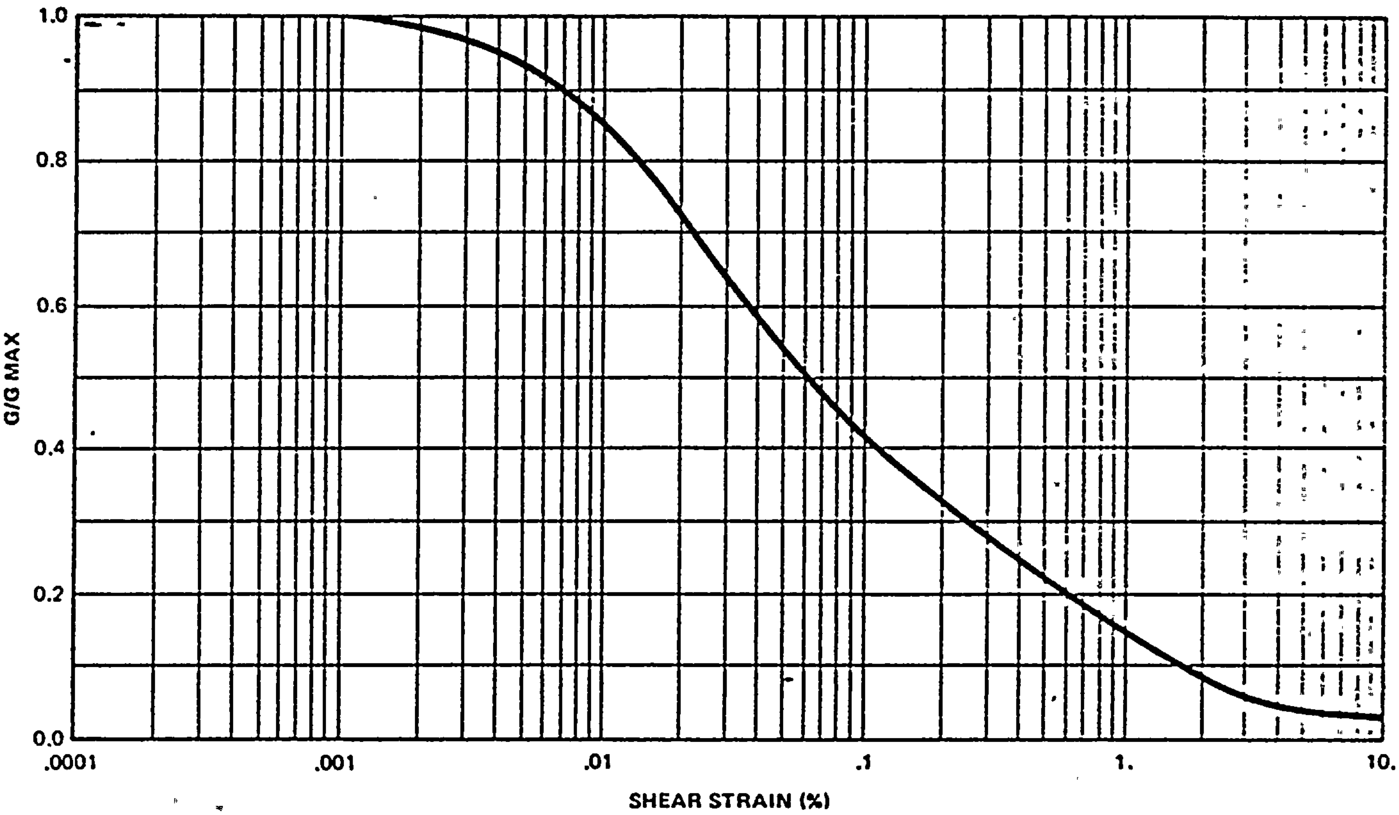
Palo Verde Nuclear Generating Station
FSAR

SOIL PROFILE

FIGURE 3.7-7

September 1980 Amendment 2

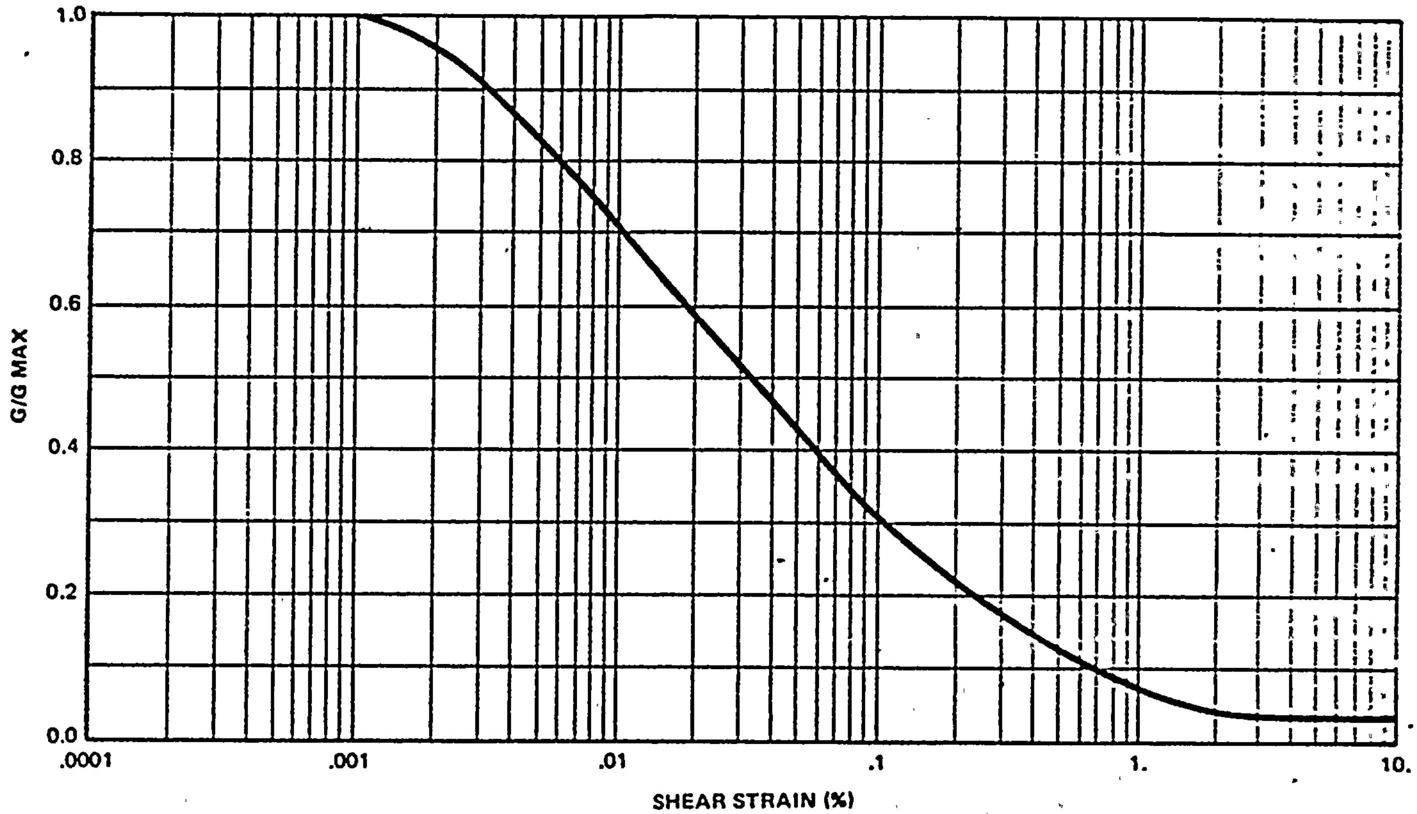
CHANGE



Palo Verde Nuclear Generating Station
FSAR

SHEAR MODULUS VS. STRAIN - CLAY

FIGURE 3.7-8



Palo Verde Nuclear Generating Station
FSAR

SHEAR MODULUS VS. STRAIN - SAND

FIGURE 3.7-9

Table 3.8-4L

MAIN STEAM SUPPORT STRUCTURE SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS

Description of Principal Member	Location of Principal Member	Governing Load Combination Number (a)	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u) (b) (c)	Calculated Shear Load (V_u) (b)	Maximum Shear Capacity (V_u) (b)
			P_u (b)	M_u (c)			
3'-6" thick wall - vertical reinforcement	Exterior north wall El. 81'-0" to 100'-0"	5	+600	97,480	2.76×10^6	2,433	6,384
3'-6" thick wall - vertical reinforcement	Interior wall El. 81'-0" to 100'-0"	3	+600	144	637	37.2	48.1
3'-6" thick wall - vertical reinforcement	Exterior south wall El. 100'-0" to 156'-0"	5	+1,200	67,300	2.76×10^6	2,037	15,900
Variable wall thickness vertical reinforcement	Exterior west wall El. 100'-0" to 156'-0"	5	+545	93,460	3.08×10^6	3,536	25,400
3'-6" thick slab	El. 100'-0"	3	0	103.5	264	94.3	100.9
2'-5-1/4" thick slab	El. 166'-7"	3	0	34	100	34.7	41.4

3

a. Refer to Section 3.8.3.3.2.b for description of load combination number.
b. P_u and V_u are in kips; sign convention for P_u: Compression (-), Tension (+).
c. M_u is in ft-k/ft for slabs and ft-k for walls.

Table 3.8-4M

MAIN STEAM SUPPORT STRUCTURE SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS

Description of Principal Members	Location of Principal Members	Governing Load Combination Number	Combined Stress Ratio ($\leq$1.0)
C 10 X 30 Beam	El. 88'-11-1/2"	2[a]	1.0
W 16 X 15.5 Column	El. 88'-11-1/2"	1[a]	1.0
W 24 X 76 Beam	El. 120'-0"	2[a]	0.94
W 12 X 31 Beam	El. 120'-0"	2[a]	0.96
W 12 X 40 Beam	El. 132'-0"	2[a]	0.89
W 18 X 77 Beam	El. 129'-8"	2[a]	0.60
W 18 X 106 Beam	El. 129'-8"	2[a]	0.42
W 18 X 97 Beam	El. 140'-0"	2[a]	0.96
W 14 X 176 Beam	El. 140'-0"	2[a]	1.0
W 12 X 40 Beam	El. 140'-0"	2[a]	0.51
W 8 X 40 Beam	El. 149'-0"	2[a]	0.87
W 16 X 77 Beam	El. 164'-6-1/4"	5[b]	0.96
W 30 X 132 Beam	El. 164'-6-1/4"	5[b]	0.96
W 16 X 40 Beam	El. 164'-6-1/4"	5[b]	0.97
W 14 X 120 Column	El. 164'-6-1/4"	5[b]	1.0

a. Refer to section 3.8.3.3.3.A(1) for description of load combination number.

b. Refer to section 3.8.3.3.3.B(1) for description of load combination number.

3.8.5.5 Structural Acceptance Criteria

The foundations of Seismic Category I buildings are designed to meet the same structural acceptance criteria as the buildings themselves. These criteria are discussed in sections 3.8.1.5, 3.8.3.5, and 3.8.4.5. The limiting conditions for the foundation medium, together with a comparison of actual capacity and estimated structure loads, are found in sections 2.5.4.10 and 2.5.4.11. Computed factors of safety against overturning, sliding, and flotation for Category I structures are given in table 3.8-5. 1

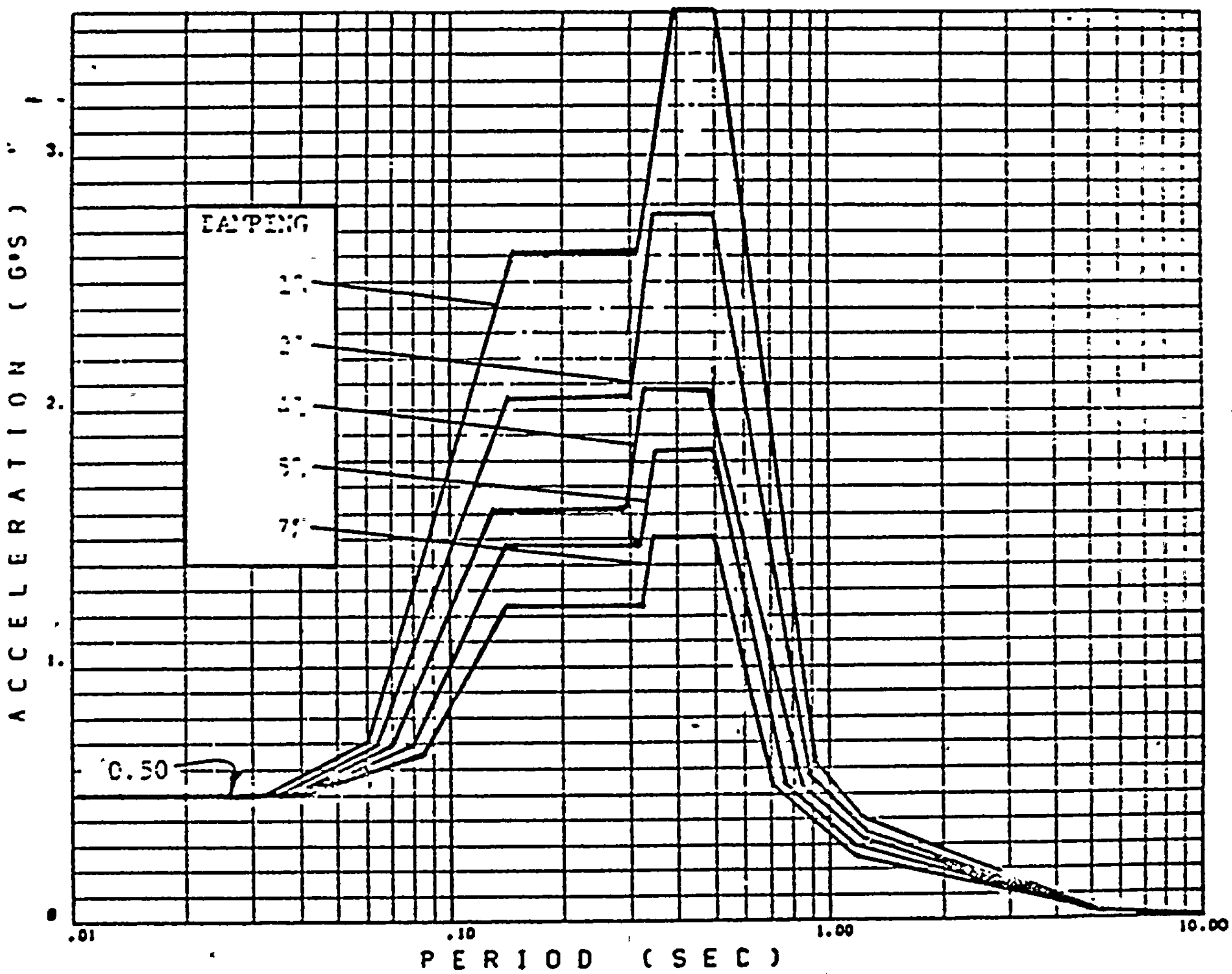
Table 3.8-5
COMPUTED FACTORS OF SAFETY

Structure	Overturning		Sliding		Flotation
	OBE	SSE	OBE	SSE	
Auxiliary	3200	830	2.2	1.3	4.7
Containment	3400	1200	1.7	1.2	4.5
Control	1500	420	1.6	1.2	4.8
Diesel Generator	1200	400	2.2	1.1	NA[a]
Fuel	1600	400	1.9	1.1	NA
Main Steam Support	340	91	1.6	1.1	NA
Condensate Storage and Refueling Water Tanks	500	150	1.7	1.4	NA

a. Not applicable

1

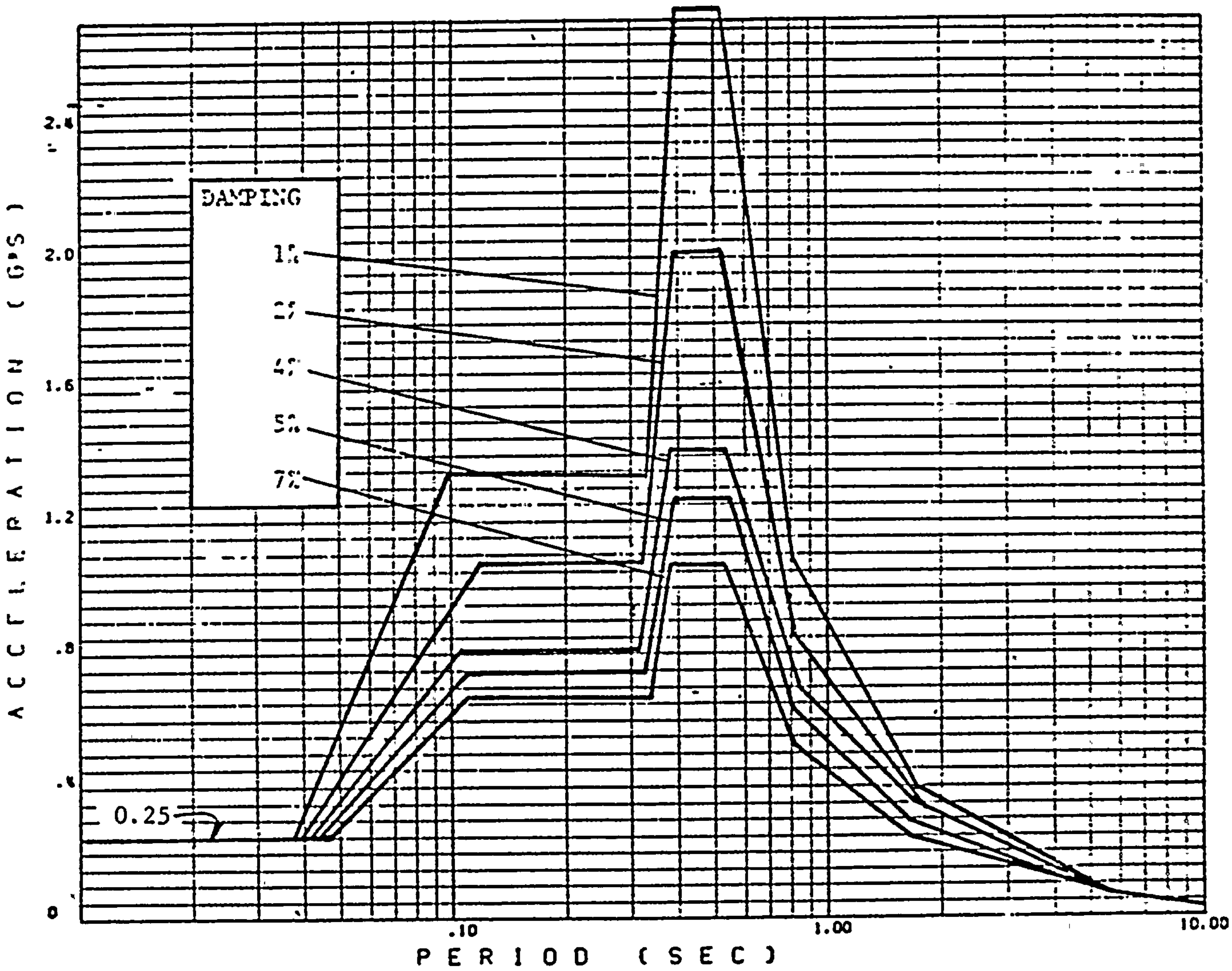
222



Palo Verde Nuclear Generating Station
FSAR

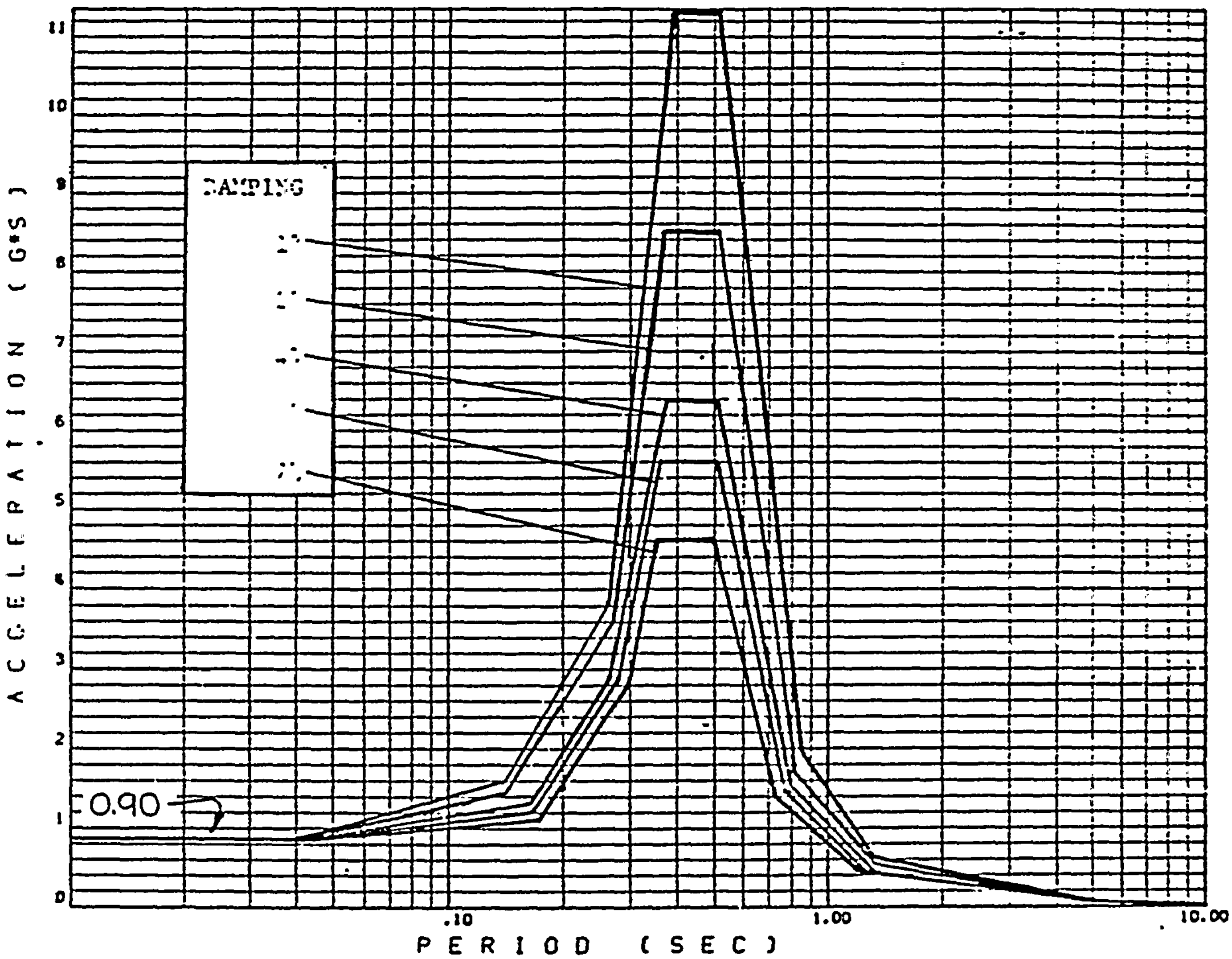
MAIN STEAM SUPPORT STRUCTURE
SSE VERTICAL
ACCELERATION RESPONSE SPECTRA

Figure 3D-37



Palo Verde Nuclear Generating Station
FSAR

MAIN STEAM SUPPORT STRUCTURE
SSE HORIZONTAL (E-W)
ACCELERATION RESPONSE SPECTRA
EL 81'-0", BASE MAT
Figure 3D-38



Palo Verde Nuclear Generating Station
FSAR

MAIN STEAM SUPPORT STRUCTURE
SSE HORIZONTAL (E-W)
ACCELERATION RESPONSE SPECTRA
EL 164'-0", ROOF
Figure 3D-39

PALO VERDE NUCLEAR STATION UNITS 1, 2, 3
DESIGN ADEQUACY AUDIT

STRUCTURAL AUDIT OF AUXILIARY BUILDING
Part I - General Analysis

I. BASIC DESIGN CRITERIA

A. 'g' value - free field

	Seismic level based on construction permit license	Seismic level used in design of structures and equipment
SSE	0.20g	0.25g
OBE	0.10g	0.13g

Reference: FSAR, Section 3.7

B. Spectra (attach figs. for all damping values, ductilities)

1. zero period acceleration

SSE 0.25g
OBE 0.13g

Reference: FSAR, Figures 3.7-1 - 3.7-4 and Section 3.7.1.1
(Pgs. 270, 1, 2, 3)
This is consistent with Reg. Guide 1.60.

Frequency (or period) interval

Refer to BC-TOP-4A, Section 2.5.1 (c)

C. Damping

Refer to FSAR, Section 3.7.1.3
This is consistent with Reg. Guide 1.61
Refer to FSAR Figures 3.7-5 and 3.7-6 (Pages 92A, 92B)

D. Artificial time history and corresponding spectra (attach figures)

1. original time history and its composition, i.e., rising time, strong motion and tail end.

Refer to FSAR, Section 3.7.1.2 and BC-TOP-4A, Section 2.5

2. base line correction, check the integrated velocity and displacement time histories

   The velocity and displacement time histories were checked and found to be satisfactory. In addition, only the acceleration time history was used as input in analysis.

3. time interval - compatible with the highest frequency considered in the spectral calculation

   0.005 seconds

E. Motion duration

24 seconds

Refer to FSAR, Section 3.7.1.2

F. components of motion including their relative motion amplitudes

Analysis was performed for the three principle directions with equal amplitudes.

G. Dead and live loads for various operating floors and base slab

~~Refer to Project Design Criteria Part II, Sections 3.5.1 and 3.5.2, and Part III, Section 1.4.1.~~

Dead Load - includes all structures, major equipment load and 50 psf equivalent for small equipment

Live load - see action item 3

H. Ground water level

The groundwater design level is at plant El 70'-0". The actual water level is at approximately plant El. 60'-0". A portion of the building extends down to El. 40'-0". Buoyancy effects have been considered in the design.

Reference: FSAR, Sections 2.4.13.2.4 and 2.4.13.5.

I. Backfill earth pressure, wind, overpressure due to postulated external explosions (as applicable)

~~Refer to Project Design Criteria Part II, Sections 3.4.5.3 and 3.5.5 for lateral earth pressure, wind, and tornado loads.~~

For lateral earth pressure see pages 208A & 208B, 208C, 208D
Wind does not govern

J. Other considerations

All penetrations in roof and exterior walls that expose safety related equipment have tornado missile protection. Concrete covers or steel plate shields to prevent missile perforation are provided.

II. ANALYSIS METHOD

A. Seismic Analysis

1. Mathematical model-general description with sketch.

   Two planar lumped parameter models were used.

   Refer to FSAR, Figure 3.7-11 and Section 3.7.2.3.

   a. (1) concrete modulus

   $E = 3.64 \times 10^6$ psi   For F'c = 4000 psi
   $E = 4.07 \times 10^6$ psi   For F'c = 5000 psi

   (2) rebar modulus and yield strength

   $E = 29 \times 10^3$ KSI

   $F_y = 60$ KSI

   (3) Poisson's ratio

   $\gamma = .24$ for concrete

   ~~Reference: Project Design Criteria, Part II, Section 3.5.7~~

   (4) damping

   Refer to FSAR Section 3.7.1.3 ~~and responce to NRC Question 220.2 (Q3A - 4).~~ and Table 3.7-1 (Page 88A)
   This is consistent with Reg. Guide 1.61.

(5) properties of foundation materials

- shear modulus

  Refer to FSAR, Figures 3.7-7, -8, -9. (Pages 274,5,6)

- subgrade reactions

  ~~Refer to Project Design Criteria, Part II, Section 3.4.5.4 for coefficient of subgrade reaction.~~
  Coefficient of subgrade reaction = 40-60 kips/ft^2/ft

- bearing capabilities

  Refer to Tables 2.5-15 and 2.5-16 of the FSAR. (Pages 266 & 267)

(6) other parameters

b. stiffness calculations

(1) exterior walls

Exterior walls and interior walls stiffness calculations were performed manually using standard engineering methods.

(2) interior walls

Same as exterior walls except for concrete masonry walls (fire barrier walls) and plaster walls which were neglected.

2. method of Analysis

a. method of analysis used (time history, response spectrum methods, etc.) and consideration of torsional and translational response

The models described above were used for acceleration time history and modal response spectrum analyses. The time history analysis was performed using FOSIN. The modal response spectrum analysis was performed using SAP 1.9.

(1) general description

The Auxiliary Building analysis used two planar lumped parameter models. Soil structure interaction was incorporated into the model by adding to the fixed-base system discrete soil springs based on elastic half-space theory.
The Soil Structure interaction analysis method was used and compared against current NRC review position and this meets the intent of NRC Soil structure interaction position

(2) findings and comments

b. selection of number of masses and degrees of freedom

(1) general description

For horizontal direction earthquake, the model consisted of 6 nodes and 12 degrees of freedom.

For vertical direction earthquake, the model consisted of 6 nodes and 6 degrees of freedom.

(2) findings and comments

c. number of modes considered

	SSE	OBE
Horizontal (N-S)	5 modes 37.9 cps	5 modes (37.6 cps)
Horizontal (E-W)	5 modes 40.0 cps	5 modes (39.7 cps)
Vertical	3 modes 69.1 cps	3 modes (68.7 cps)

Frequency shown above are for the highest mode considered.

(1) general description

Refer to FSAR, Table 3.7-4 for modal frequencies and participation factors. (Page 279)

(2) findings and comments

d. combining modal responses

(1) actual procedures used

Refer to FSAR, Section 3.7.2.7.

(2) general findings

e. consideration of three components of motion

(1) actual procedures used

Refer to FSAR, Section 3.7.2.6.
This is consistent with Reg Guide 1.92

(2) general findings

f. consideration of soil-structure interaction

Soil Springs

OBE

North-South	$Kx = 2.87 \times 10^6$ K/ft	$Kxx = 1.158 \times 10^{10}$ K-ft/Rad
East-West	$Ky = 2.755 \times 10^6$ K/ft	$Kyy = 2.975 \times 10^{10}$ K-ft/Rad
Vertical	$Kz = 3.905 \times 10^6$ K/ft	

SSE

North-South	$Kx = 2.206 \times 10^6$ K/ft	$Kxx = 8.902 \times 10^9$ K-ft/Rad
East-West	$Ky = 2.117 \times 10^6$ K/ft	$Kyy = 2.286 \times 10^{10}$ K-ft/Rad
Vertical	$Kz = 3.00 \times 10^6$ K/ft	

(1) general description

Refer to FSAR, Section 3.7.2.4.

(2) findings and comments

g. decoupling criteria for subsystems

(1) general procedure

Refer to BC-TOP-4A, Section 3.2.

(2) key examples

(3) The other criteria pertaining to frequency ratio as defined in SRP 3.7.3.II.3.b are also met.

(3) general findings and comments

h. modeling of hydrodynamic effects in spent fuel pool

Not applicable. The Spent Fuel Pool is located in the Fuel Handling Building.

i. modeling of spent fuel pool wells and interior floor slabs and equipment thereof

Not applicable.

3. development of in-structure response spectra

Refer to FSAR, Section 3.7.2.5

a. general procedures

The "SPECTRA" program was used to compute the response spectra. Refer to FSAR, Appendix 3B for a description and applications of this program.

(1) smoothing (describe specific smoothing method used)

The smoothened floor response spectra represent an envelope of the maximum peaks.

(2) peak widening

± 15%

Reference: FSAR, Section 3.7.2.9

b. typical results (attach figures)

Refer to FSAR, Figures 3D-1, -2, -3. (Pgs. 283, 4, 5)

B. Stress Analysis

1. shear walls and floors

a. mathematical model - general description w/sketch

Shear wall and floor stresses were computed by performing standard manual calculations. Vertical loads were distributed in the structure by conventional methods. Lateral loads were calculated by multiplying lumped mass and seismic accelerations obtained from floor spectra at each level. These lateral loads were then distributed among the shear walls according to their relative stiffness and location.

b. method of analysis--incorporation of torsion

The torsional moment was determined by resolving the couple due to eccentricity between the center of mass and the center of rigidity at each floor. Shear derived from this torsional moment was added directly to the forces considered for the individual shear walls.

c. load combinations

Refer to FSAR, Section 3.8.3.3.

1. foundation mat

   a. mathematical model - description of boundary conditions

      Calculation was performed manually.

   b. method of analysis

      Pressure loading is obtained from equivalent static analysis considering total dead load, live load and its eccentricity and three directional earthquake forces. The basemat at El. 40′ and 70′ were designed as rigid mats, using soil pressure as loading and walls as supports. ~~Due to lack of rigid walls in the penetration area at El. 77′-0" this mat was designed as a plate on elastic foundation.~~

   c. load combinations

      Refer to FSAR, Section 3.8.3.3.
      We are in compliance with the SRP.

   d. key results (figures, etc.)

      Refer to FSAR, Table 3.8-4D. (Page 279)

3. Material to protect against structure - structure interaction

Below grade (EL 100'-0") Rodofoam II (WR Grace and Co.) or EVERLASTIC Micro II (Williams Products, Inc.) may be left in place in seismic joints between structures. Above grade gaps between walls were left open, typically 6" between Category I structures.

a. mechanical properties

b. additional pressure on walls

c. findings and comments

4. vertical dynamic analysis

a. mathematical model - general description with sketch

The planar lumped parameter model consisted of one vertical beam on a rigid mat which represents the exterior and interior walls.

Reference: FSAR, Figure 3.7-11. (Page 277)

b. development of stiffnesses, including floor stiffness, as applicable

The model consists of lumped nodal masses representing floor mass tributary to the mass point and of axial elements representing vertical stiffnesses of walls with vertical degrees of freedom only. Stiffnesses of the walls were calculated manually.

c. method of analysis

The models described above were used for acceleration time history and modal, response spectrum analyses was performed by using SAP 1.9 computer program. The time history analysis was performed using FOSIN. The modal response spectrum analysis was performed using SAP 1.9.

C. Computer Programs Used in Analysis

SAP 1.9, SPECTRA, FOSIN, LUCON

1. assumptions and limitations

   Refer to FSAR, Appendix 3B.

2. applicability

   Refer to FSAR, Appendix 3B.

3. verification

   - sensitivity study in case of numerical solutions (e.g., finite element analysis)

     Refer to FSAR, Appendix 3B.

4. load input (include all cases)

PROGRAM	INPUT
SAP 1.9	Finite element model (modes and elements), loading (pressure, nodal loads), response spectra.
SPECTRA	In-structure time histories, frequency or periods and damping values.
FOSIN	Free-field time history, damping values, frequency.
LUCON	Shear modulus, damping values

5. output (include all cases)

   Refer to FSAR, Appendix 3B.

6. other discussions

D. Overall Stability

1. forces and moments from seismic analysis

   Refer to FSAR, Figure 3.7-16. (Page 278)

2. various cases considered

   Seismic event loading combinations considering SSE or OBE applied in North-South, East-West and vertical directions simultaneously.

3. bearing pressure versus bearing capability and safety factor against bearing failure

   Refer to FSAR, Section 2.5.4.10 and Tables 2.5-15, -16. (Page 266, 7)

4. factors of safety

   Refer to FSAR, Section 3.8.5.5 and Table 3.8-5. (Page 282)

   a. sliding

      Factor of Safety = 1.3 (SSE).

   b. overturning

      Factor of Safety = 830 (SSE).

E. Interaction of Non-category I Structures with the Structure Considered

1. identification of pertinent non-Category I structures

The Radwaste Building and Corridor Building are adjacent to the Auxiliary Building.

2. consideration given to potential failure of non-Category I systems on Category I systems

The Radwaste ~~and Corridor~~ Buildings was ~~were~~ designed to preclude structural failure of building or parts thereof that could damage the Auxiliary Building and its Category I systems. Within the Auxiliary Building, non-Category I systems which potentially could affect Category I systems, were designed for structural integrity under SSE loads.

Reference: FSAR, Section 3.8.4.4.

3. general findings and comments

During a walk down those items, whose failure will not affect any safety related equipment are left as they are. If they are judged that they might affect category I systems, they are designed to maintain their structoral integrity under an SSE.

F. Design Consideration for Tornado Missiles

1. design requirements

   Refer to FSAR, Table 3.5-8. (Pg. 268)

2. models for

   a. local damage

      Refer to FSAR, Section 3.5.3.

   b. overall response

      Refer to FSAR, Section 3.5.3.

3. load combinations

   Refer to FSAR, Section 3.8.3.3.

4. forces

   The governing structural members are of sufficient thickness to preclude perforation by postulated missiles and maintain structural integrity.

6. general comments and preliminary audit findings

III. CONFORMANCE TO ACCEPTABLE CRITERIA

A. Identification of deviations, if any

None.

B. Justification of deviations and disposition of the deviations

D. general comments

Part II Audit of Key Designs

A. Exterior Shear Walls

1. design requirements

   The exterior walls were designed to satisfy structural function as bearing walls, shear walls, and protection against tornado missiles.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4D. (Pg. 279)

4. detailed design of rebar placement at key sections

   See Attachment A. (Pg. 265)

5. general comments and preliminary audit findings

B. Interior Shear Walls

1. design requirements

The interior walls were designed to satisfy structural function as bearing walls and shear walls.

2. design loads (from general analysis)

Same as exterior walls except that plaster and concrete masonry walls (fire barrier walls) were neglected for the strength of the building but were designed to resist the loads due to their own weight (Dead Load plus seismic) in order to preclude damage to safety related equipment.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4D. (Pg. 279)

4. detailed design of rebar placement at key sections

See Attachment A. (Pg. 265)

5. general comments and preliminary audit findings

C. Main Floors and Roofs

1. design requirements

Main floors and roof slabs were primarily designed for vertical dead, live and seismic loads. ~~as defined in the Project Design Criteria.~~ Roofs were also designed to satisfy minimum thickness to preclude perforation by tornado generated missiles.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4D. (Pg. 279)

4. detailed design of rebar placement at key sections

See Attachment A. (Pg. 265)

5. general comments and preliminary audit findings

D. Steel Structural Bracing Systems (if any)

1. design requirements

In general, steel beams are designed for two functions:

a) To support dead load of wet concrete and construction loads

b) To support piping and cable trays and other miscellaneous equipment loads.

For penetration area, floors are supported on structural steel beams, girders and columns which are designed for dead, live and seismic loads.

2. design loads

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4E. (Pg. 280)

6. general comments and preliminary audit findings

E. Foundation Mat

1. design requirements

   Refer to FSAR, Sections 3.8.5.4.2 and 3.8.5.5.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4D. (Pg. 279)

4. detailed design of rebar placement at key sections

   See Attachment A. (Pg. 265)

5. general comments and preliminary audit findings

F. Main Frame Concrete Column Design (Key Columns)

1. design requirements

The concrete columns were primarily designed for vertical loading due to dead, live and seismic loads. The columns were not used as lateral-load-resisting elements.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

	Axial Force	Moment
1) Column at El. 40'0"	1022 K	282 Ft-K
2 ~~1~~) Column at El. 88'0"	1981 K	203 Ft-K
3 ~~1~~) Column at El. 120'0"	999 K	460 Ft-K

4. detailed design of rebar placement at key sections

See Attachment A. (Pg. 265)

5. general comments and preliminary audit findings

G. Secondary Floors

1. design requirements

   Same as main floors. Refer to page 26.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4D. (Pg. 279)

4. detailed design of rebar placement at key sections

   See Attachment A. (Pg. 265)

5. general comments and preliminary audit findings

H. Detailing at Floor-Wall Joints

1. design requirements

   As per ACI 318-71 Code, Chapter 6 and 17.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4D. (Pg. 279)

4. detailed design of rebar placement at key sections

   See Attachment A. (Pg. 265)

5. general comments and preliminary audit findings

I. Dynamic Effects Applied to Floors and Walls by Machinery

1. design requirements

   Dynamic effects from machinery are negligible. Major pumps are located on the basemat. HVAC units are mounted with isolation pads.

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design

5. general comments and preliminary audit findings

J. Crane & Support

1. design of bents (columns, and roof trusses)

Not applicable. There are no cranes located in this building.

a. design requirements

b. design loads (from general analysis)

c. forces and moments at key sections

d. detailed design

e. general comments and preliminary audit findings

2. design of girders supporting crane rails

   Not applicable.

   a. design requirements

   b. design loads (from general analysis).

   c. forces and moments at key sections

   d. detailed design

e. general comments and preliminary audit findings

3. design of spent fuel bridge

Not applicable.

a. design requirements

b. design loads (from general analysis)

c. forces and moments at key sections

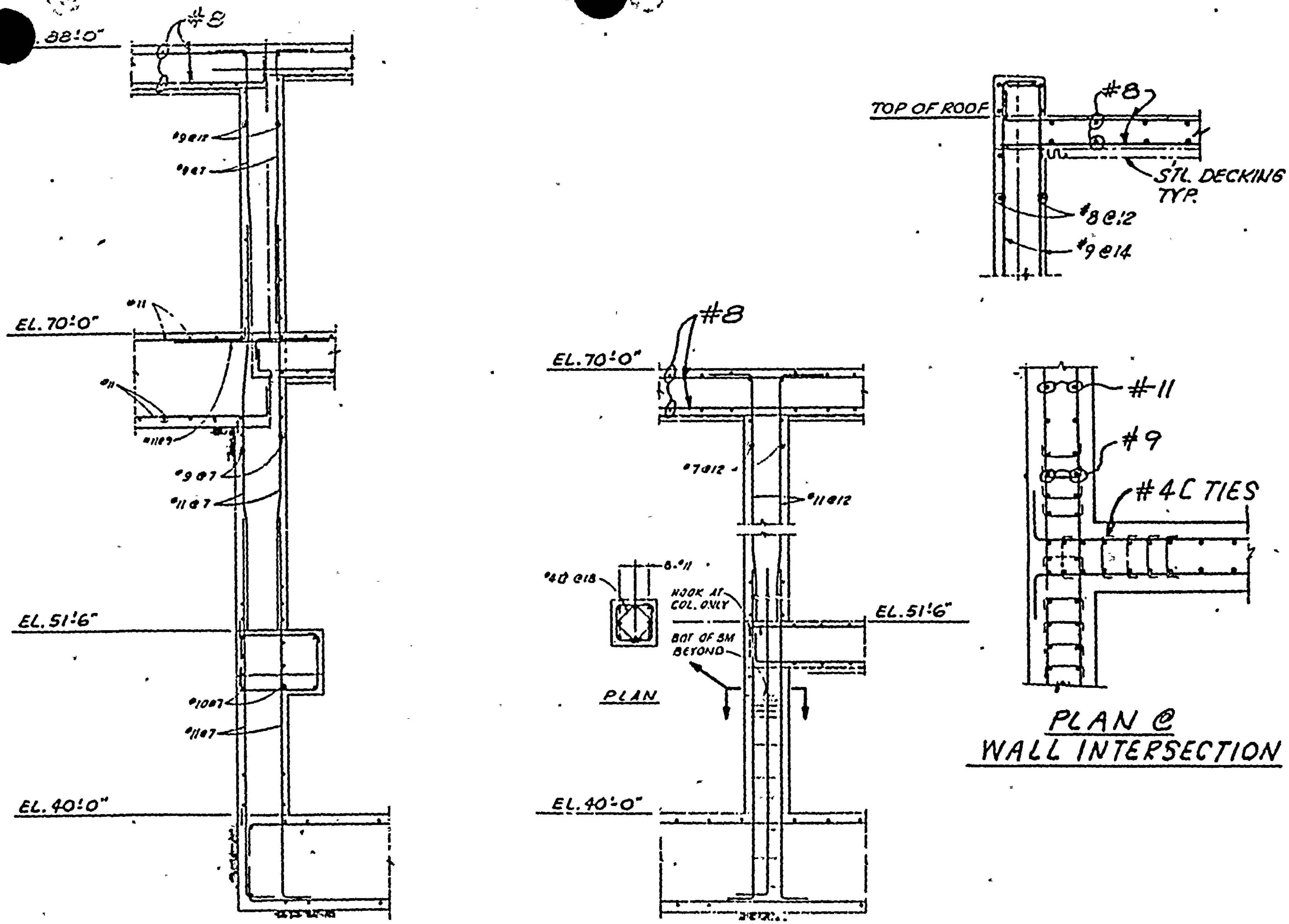
d. detailed design

e. general comments and preliminary audit findings

K. Fuel Pool Liner Design

Not applicable.

1. stresses and strain controls
2. conformance to code requirements
3. analysis procedure and results
4. consideration of accidental drop of crane loads
5. corrosion effects (e.g., pitting) on liner integrity
6. preliminary findings of audit results



AUXILARY BUILDING
TYP. WALL & COLUMN SECTS. & DETS.

ATTACHMENT "A"

Table 2.5-15

STATIC BEARING CAPACITY OF CATEGORY I STRUCTURES

Structure	Average Static Design Load q_s (k/ft^2)	Ultimate Bearing Capacity q_o (k/ft^2)	Factor of Safety (q_o/q_s)
Containment Building	7.9	35.7	4.5
Auxiliary Building (deep section)	6.2	34.9	5.6
Main Steam Support Structure	7.1	64.8	9.1
Control Building	3.3	45.3	13.7
Fuel Building	5.3	54.9	10.4
Diesel Generator Building	3.1	79.5	25.6
Refueling Water Tank	4.4	90.4	20.5
Condensate Storage Tank	3.5	112.4	32.1

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Table 2.5-16

DYNAMIC BEARING CAPACITY OF CATEGORY I STRUCTURES[a]

Structure	Equivalent Uniform Vertical Stress q_d (k/ft^2)	Ultimate Bearing Capacity q_o (k/ft^2)	Factor of Safety (q_o/q_d)
Containment Building	16.1	32.2	2.0
Auxiliary Building (deep section)	10.3	25.8	2.5
Main Steam Support Structure	25.3	60.6	2.4
Control Building	9.8	39.8	4.1
Fuel Building	19.1	50.3	2.6
Diesel Generator Building	5.6	75.5	13.5
Refueling Water Tank	13.2	58.7	4.4
Condensate Storage Tank[b]	13.2	30.2	2.3

a. Based upon maximum dynamic loads derived from analyses described in section 3.7.

b. Condensate storage tank loads were conservatively chosen to be equal to the dynamic design load for the refueling water tank. Actual loads will be less.

1

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Table 3.5-8

TORNADO-GENERATED MISSILES CONSIDERED IN DESIGN OF SAFE SHUTDOWN STRUCTURES

Description	Weight (lbs)	Impact Area (ft^2)	Maximum Velocity (ft/s)	Kinetic Energy (ft-lbs)
(A) A 12-foot wood plank, 4 x 12 inches in cross-section, traveling end on at a speed of 240 mi/h.	200	0.333	352	3.85×10^5
(B) A steel pipe, Schedule 40, 3 inches in diameter by 10 feet long, traveling end on at 120 mi/h.	78	0.063	176	3.75×10^4
(C) A steel rod, 1 inch in diameter, 3 feet long, traveling end on at 180 mi/h.	8	0.005	264	8.66×10^3
(D) A steel pipe, Schedule 40, 6 inches in diameter by 15 feet long, traveling end on at 120 mi/h.	285	0.24	176	1.37×10^5
(E) A steel pipe, Schedule 40, 12 inches in diameter by 15 feet long, traveling end on at 120 mi/h.	743	0.886	176	3.57×10^5
(F) A utility pole, 13-1/2 inches in diameter, 35 feet long, traveling end on at 120 mi/h.	1490	0.994	176	7.17×10^5
(G) An automobile of 4,000 pounds weight, striking the structure at 60 mi/h.	4000	20.0	88	4.81×10^5

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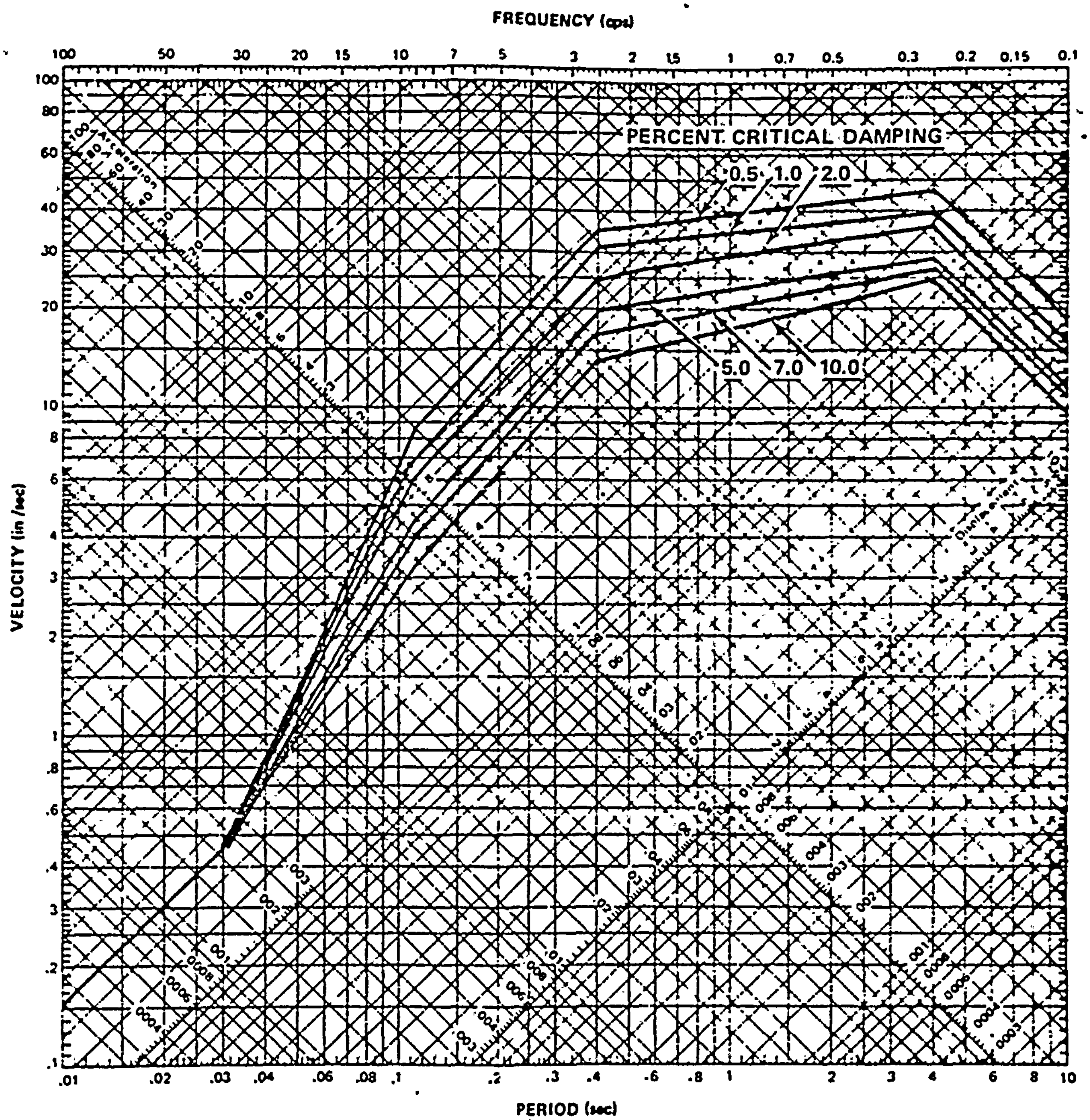
Table 3.7-4

AUXILIARY BUILDING NATURAL FREQUENCIES[a] 4

		Mode	Frequency (Hz)
Horizontal (N-S)	OBE	1	3.6
		2	7.9
		3	18.8
		4	26.1
		5	37.6
	SSE	1	3.2
		2	7.0
		3	18.4
		4	26.3
		5	37.9
Horizontal (E-W)	OBE	1	3.9
		2	7.9
		3	16.1
		4	28.0
		5	39.7
	SSE	1	3.5
		2	7.0
		3	15.9
		4	28.1
		5	40.0
Vertical	OBE	1	5.2
		2	33.5
	SSE	1	4.6
		2	33.5
		3	[illegible].1

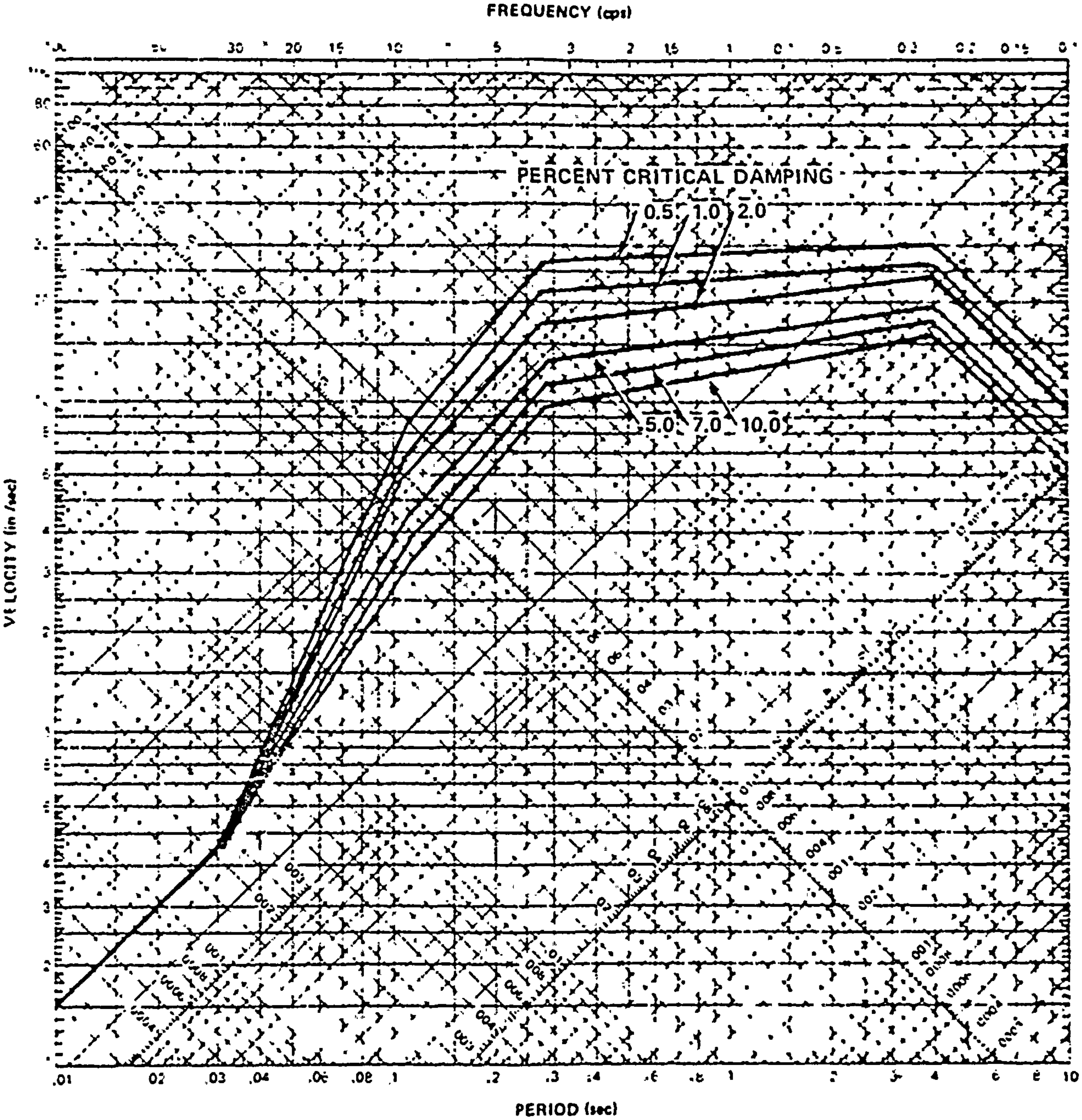
a. See figure 3.7-21 for mode shapes and participation factors. 4



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR SSE 0.25 g

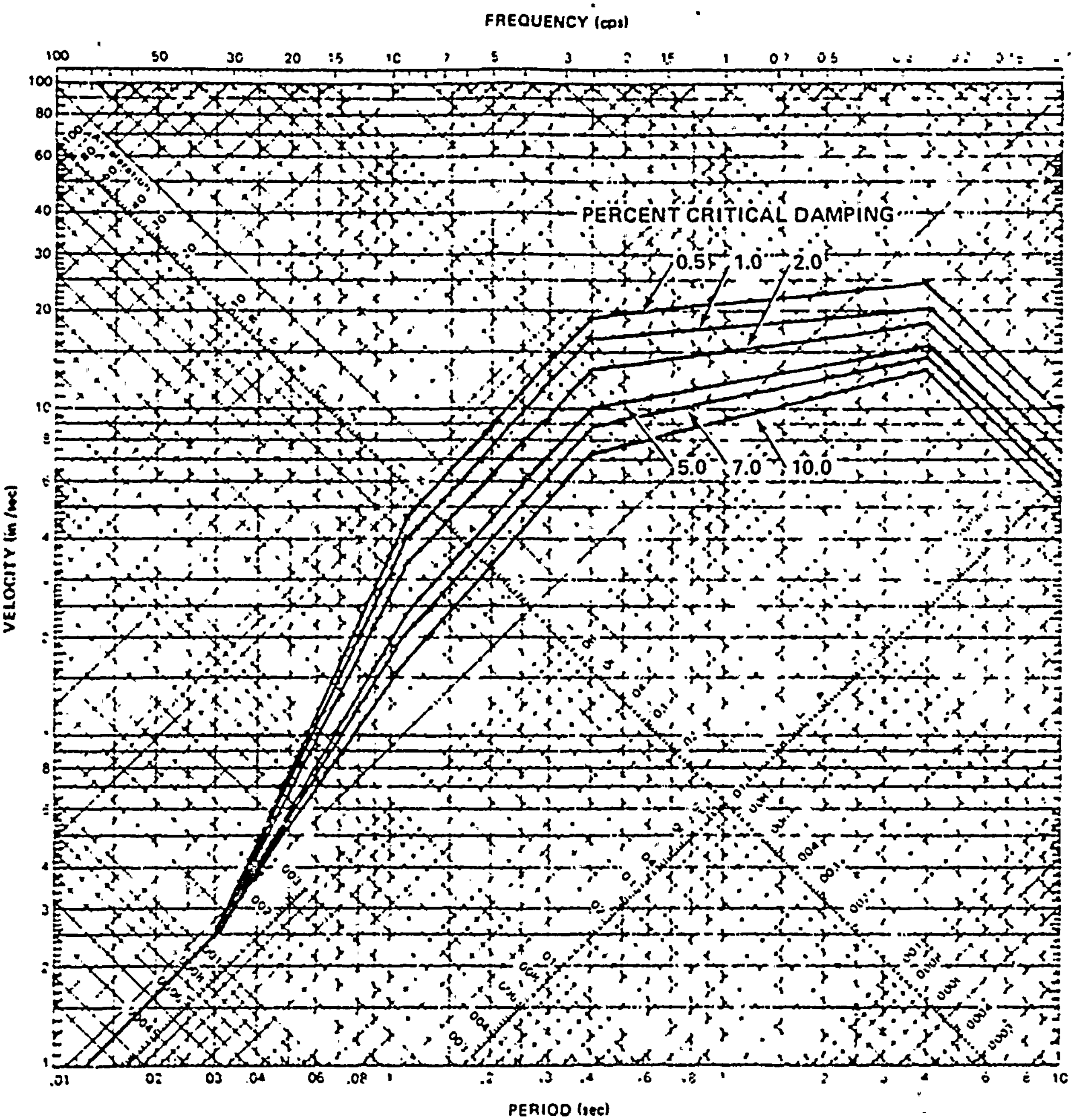
Figure 3.7-1



Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR SSE 0.25 g

Figure 3.7-2



Palo Verde Nuclear Generating Station
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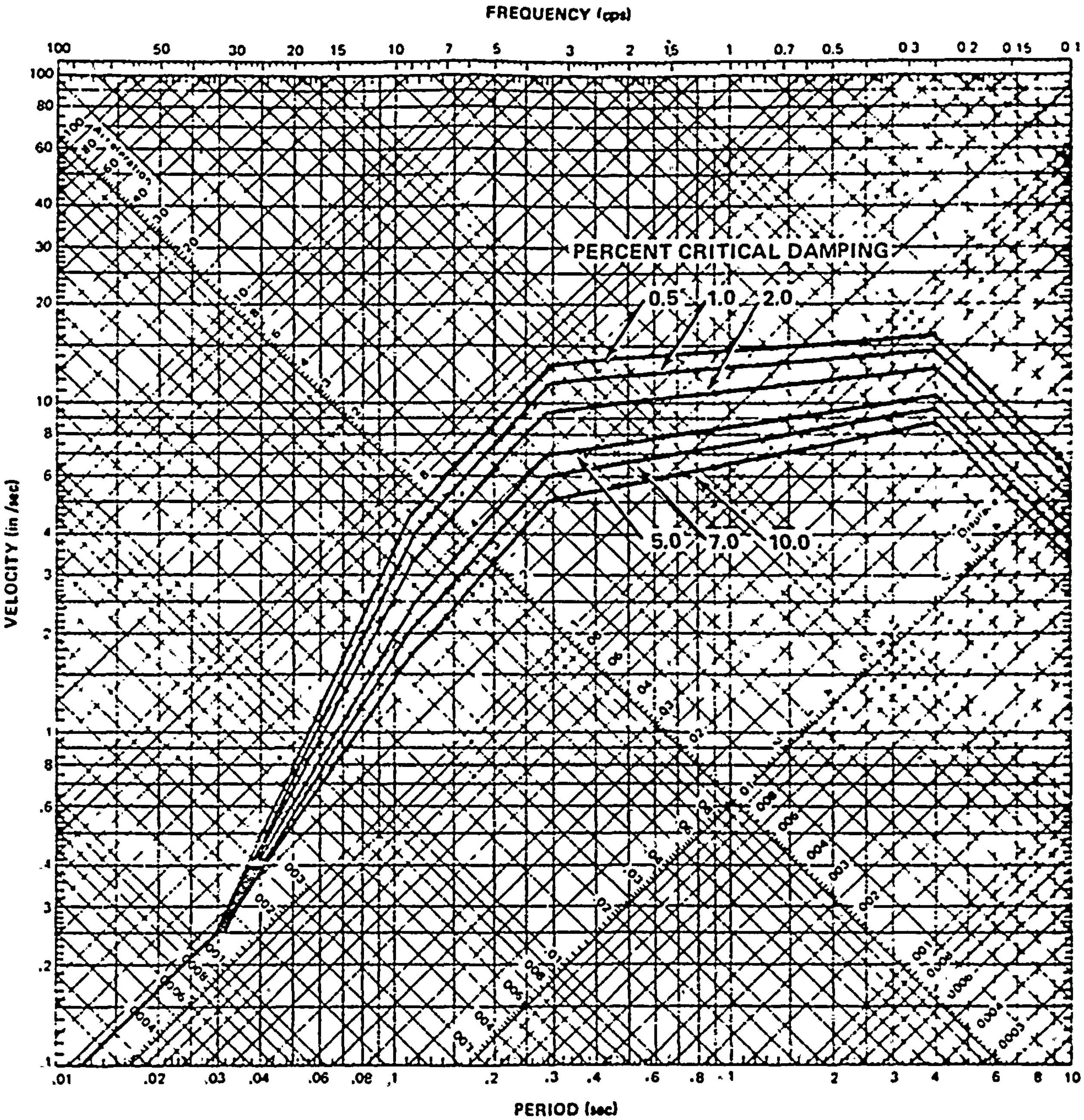
HORIZONTAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-3



Palo Verde Nuclear Generating Station
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VERTICAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-4

DEPTH (FT.)	LAYER DEPTH (FT.)	DESCRIPTION	LAYER THICKNESS	UNIT WEIGHT (PCF)	POISSON'S RATIO	IN SITU SHEAR WAVE VELOCITY (FT/SEC)	LOW STRAIN SHEAR MODULUS (KSF)	LOW STRAIN P-WAVE VELOCITY (FT/SEC)
AVERAGE PERCHED WATER LEVEL 44'		SAND (2)	47'	123	.27	996	3790	
20'						1116	4160	1990
30'						1173	5260	2090
50	47'	CLAY (1)	23'	121	.44	1194	5450	3850
	70'	SAND (2)	7'	121	.48	1209	5500	4445
100	77'	CLAY (1)	82'	123	.47	1253	6000	5288
105'						1281	6270	5388
150'						1401	7500	5502
150	153'	SAND (2)	13'	125	.465	1389		5454
	163'	CLAY (1)	27'	127	.46	1308	8980	5544
200	196'	SAND (2)	12'	127	.46			
	206'	CLAY (1)	20'	126	.45	1776	12358	5592
	228'	SAND (2)	10'	127	.44	1984	15530	6060
250	238'	CLAY (1)	73'	128	.44	2040	18880	6293
265'						2270	20650	6348
300								
300'						2701	19270	6726
	311'	SAND (2)	23'	130	.44	2176	19130	6850
350	334'	BEDROCK						

NOTES: 1. LOW STRAIN SHEAR MODULI AT VARIOUS DEPTHS ARE AVERAGE VALUES.
2 SHEAR WAVE VELOCITY CALCULATED FROM $V_S = (G/\rho)^{1/2}$
3 P WAVE VELOCITY CALCULATED FROM $V_P = V_S\,[(2-2\nu)/(1-2\nu)]^{1/2}$

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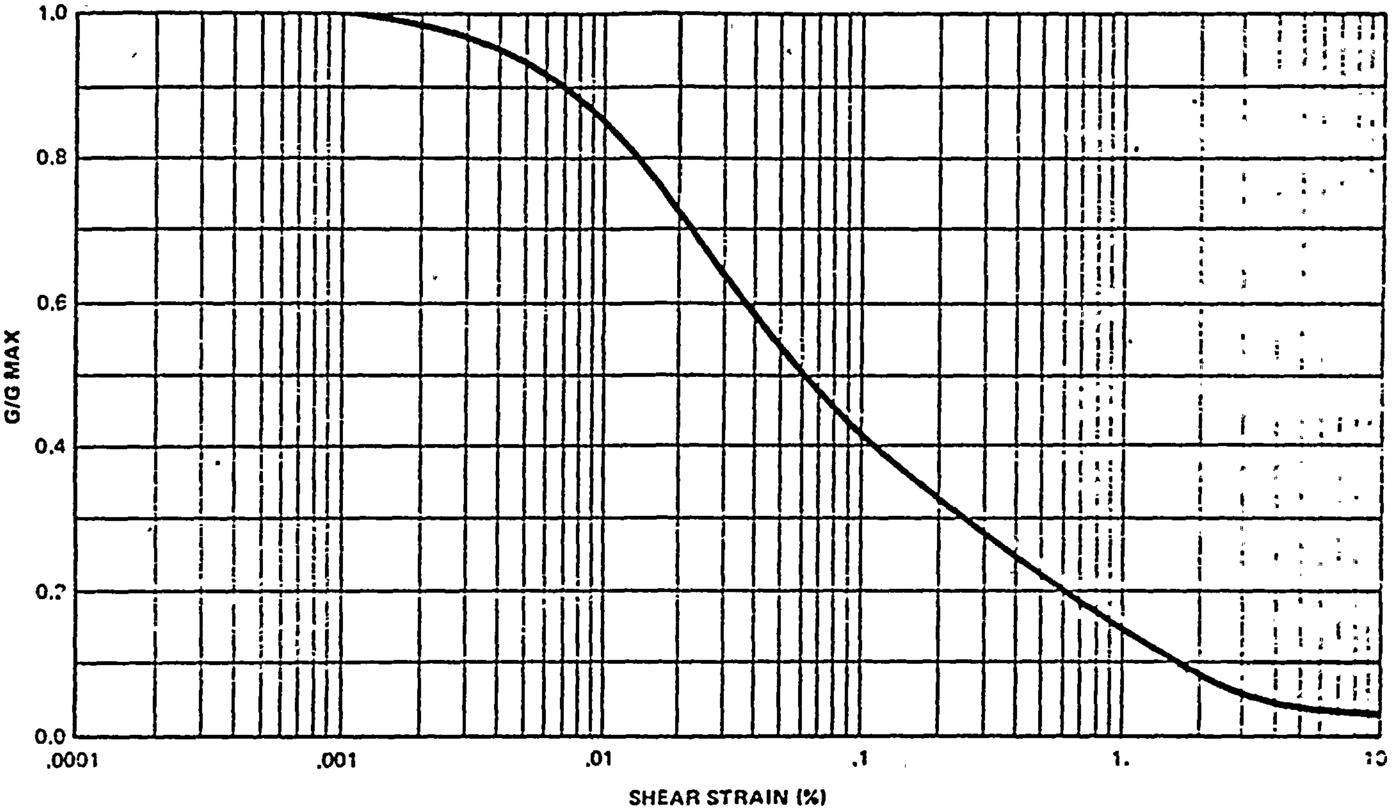
CHANGE

Palo Verde Nuclear Generating Station
FSAR

SOIL PROFILE

FIGURE 3.7-7

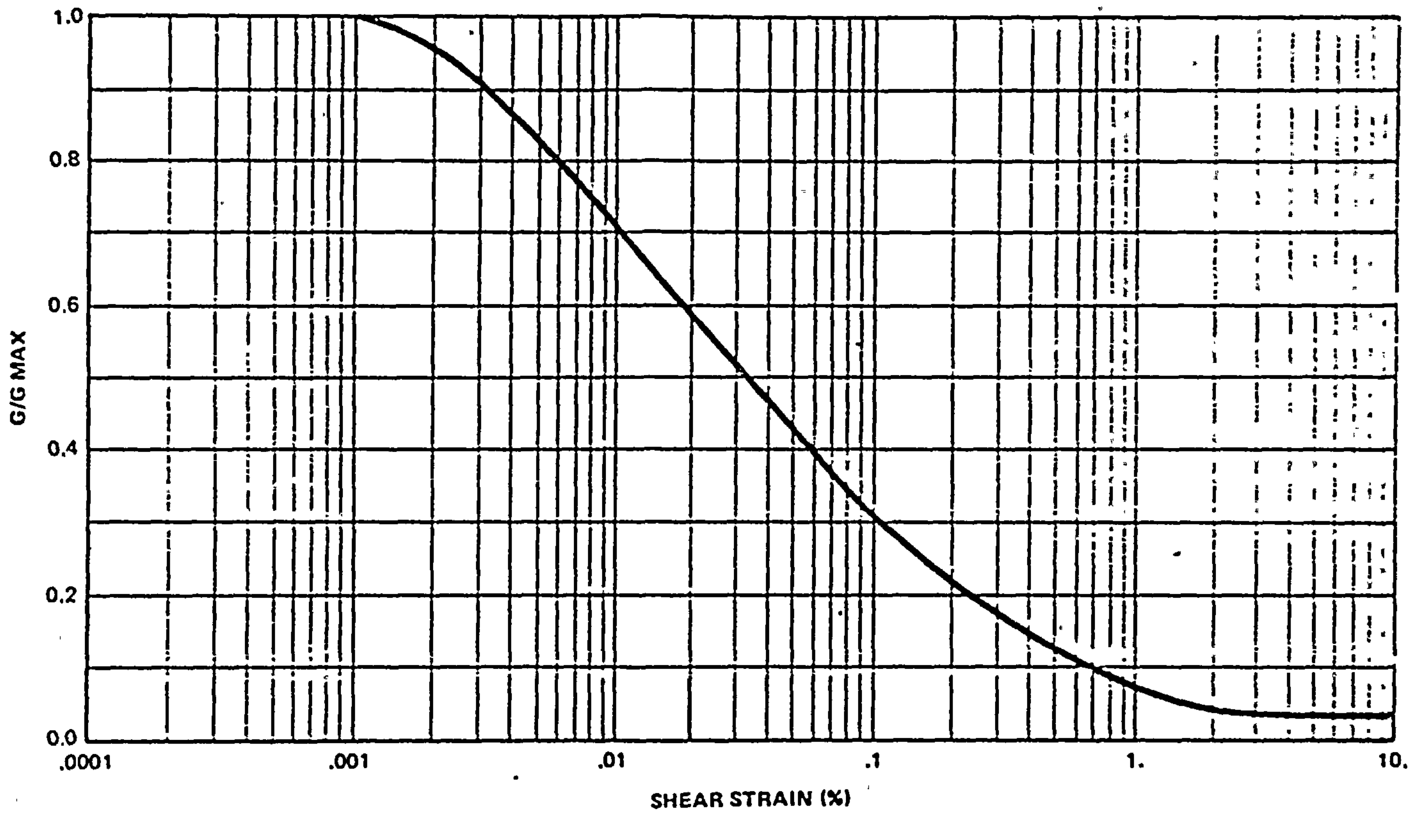
September [illegible] Amendment



Palo Verde Nuclear Generating Station
FSAR

SHEAR MODULUS VS. STRAIN - CLAY

FIGURE 3.7-8



Palo Verde Nuclear Generating Station
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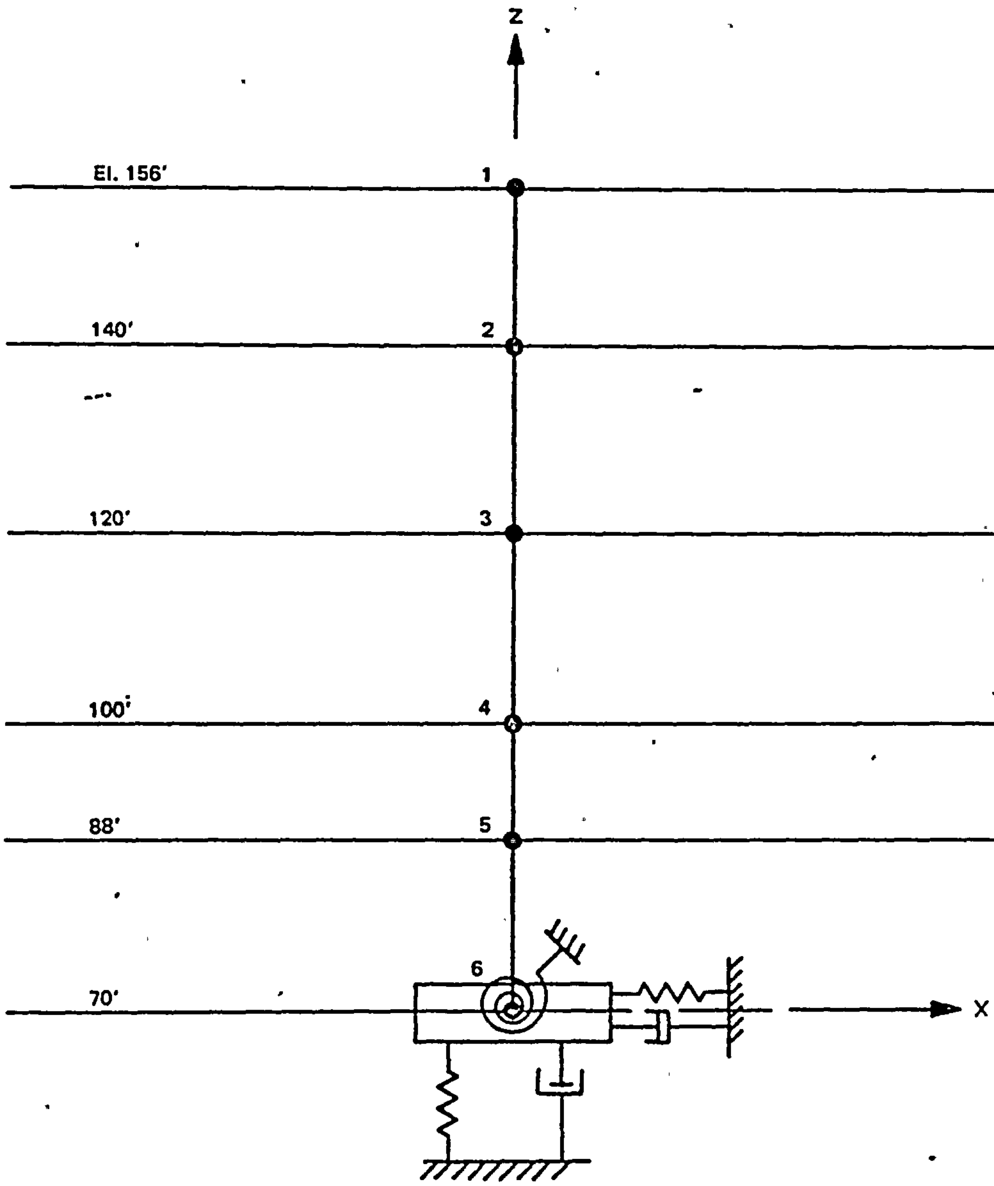
SHEAR MODULUS VS. STRAIN - SAND

FIGURE 3.7-9



Palo Verde Nuclear Generating Station
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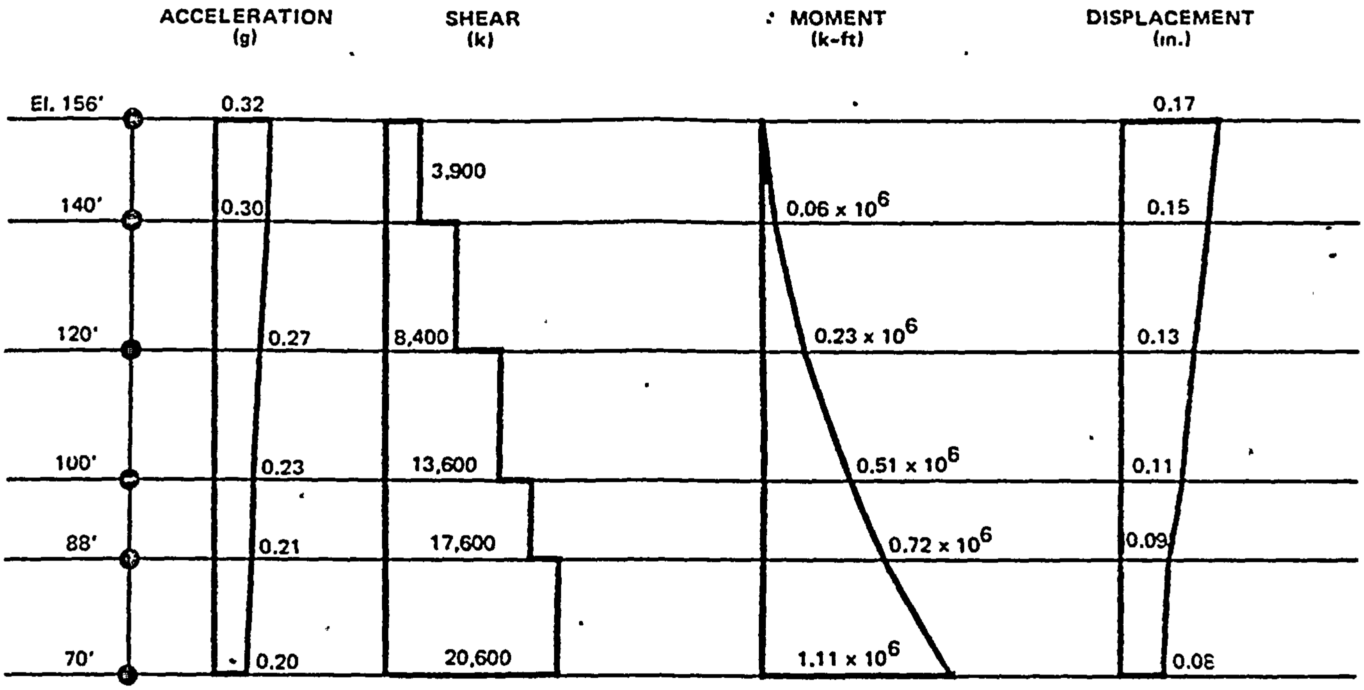
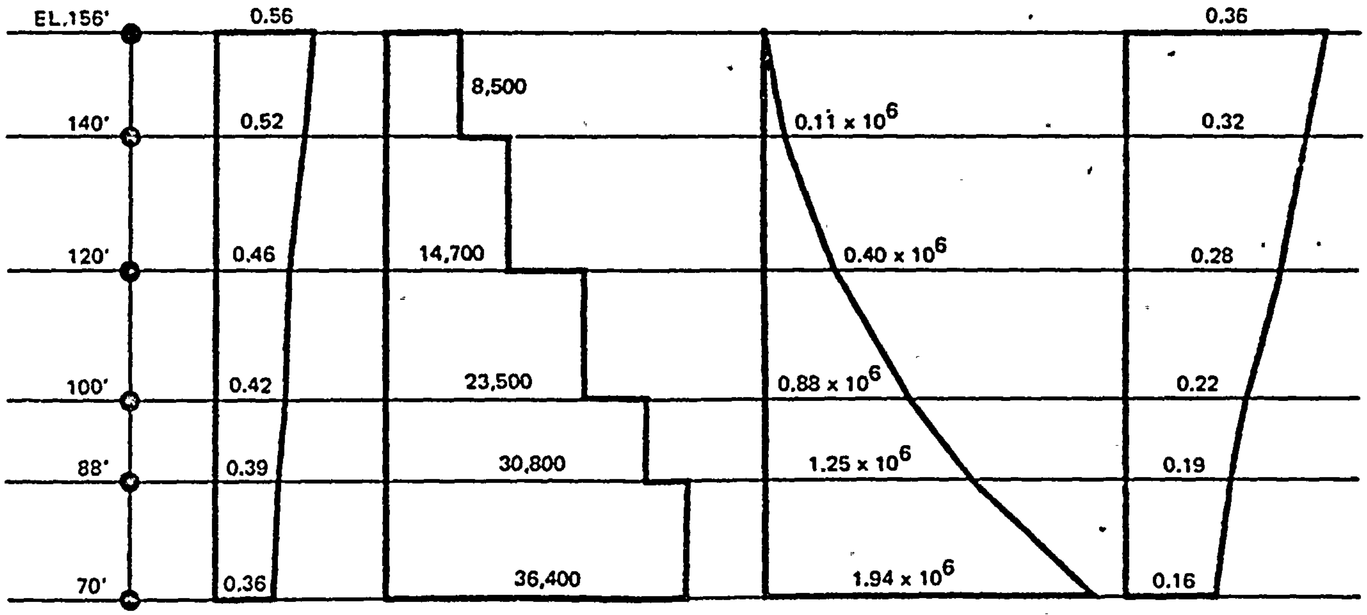
AUXILIARY BUILDING
LUMPED MASS MODEL

FIGURE 3.7-11



HORIZONTAL (OBE)



HORIZONTAL (SSE)

Palo Verde Nuclear Generating Station
FSAR

AUXILIARY BUILDING
DESIGN RESPONSE

FIGURE 3.7-16

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Table 3.8-4D

AUXILIARY BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 2 of 2)

Description of Principal Member	Location of Principal Member	Governing Load Combination Number[a]	Calculated Axial Load (P_u) and Flexural Load (M_u) P_u[b]	M_u[c]	Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u)[b][c]	Calculated Shear Load (V_u)[b]	Maximum Shear Capacity (V_u)[b]
2' - 2-1/2" x 19'-4" wall - vertical and horizontal reinforcement	Interior wall at El. 88'-0"	2	-559	52,153	115,314	1,537	2,093
3'-0" x 22'-3" wall - vertical and horizontal reinforcement	Interior wall at El. 100'-0"	2	-1,544	65,440	94,008	1,706	2,417
3'-0" x 6'-9" wall - vertical and horizontal reinforcement	Exterior wall at El. 100'-0"	2	-301	7,258	7,702	247	686
2'-9" thick slab - N-S or E-W reinforcement	El. 70'-0"	2	-[d]	128	181	-[d]	~~-[d]~~ d
1'-6" thick slab - E-W reinforcement	El. 88'-0"	2	-	43	51	-	-
2'-0" thick slab - N-S reinforcement	El. 88'-0"	2	-	65	73	-	-
2'-9" thick slab - N-S or E-W reinforcement	El. 88'-0"	2	-	112	131	-	-
2'-9" thick slab - N-S reinforcement	El. 100'-0"	2	-	84	104	-	-
1'-3" thick slab - N-S or E-W reinforcement	El. 120'-0"	2	-	21	26	-	-
6'-0" thick basemat - E-W reinforcement	El. 40'-0"	2	-	326	544	-	-
6'-0" thick basemat - E-W reinforcement	El. 70'-0"	2	-	933	1,031	-	-

3

Table 3.8-4E

AUXILIARY BUILDING SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS (Sheet 1 of 2)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[(a)]	Combined Stress Ratio ($\leq$1.0)
W 16 X 36	Floor Beam @ El. 51'-6"	1	0.88
C 10 X 25	Floor Beam @ El. 51'-6"	2	0.82
W 14 X 78	Floor Beam @ El. 51'-6"	2	0.82
W 21 X 49	Floor Beam @ El. 70'-0"	1	0.85
W 18 X 60	Floor Beam @ El. 88'-0"	1	0.94
W 12 X 27	Top Chord of Truss @ El. 88'-0"	2	0.96
W 14 X 184	Column @ El. 77'-3"	2	0.73
W 14 X 158	Column @ El. 120'-0"	2	0.66
W 14 X 84	Bottom Chord of Truss @ El. 88'-0"	2	0.83
W 27 X 177	Main Girder @ El. 100'-0"	2	0.80
W 21 X 73	Main Girder @ El. 120'-0"	2	0.94
W 16 X 64	Floor Beam @ El. 120'-0"	2	0.93
W 27 X 94	Floor Beam @ El. 140'-0"	2	0.99
W 27 X 177	Main Girder @ El. 140'-0"	2	0.95
W 27 X 114	Main Girder @ El. 140'-0"	2	0.95
W 21 X 55	Floor Beam @ El. 156'-4"	2	0.81

a. Refer to section 3.8.3.3.3.A(1) for description of load combination number.

3

Table 3.8-4E

AUXILIARY BUILDING SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS (Sheet 2 of 2)

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[(a)]	Combined Stress Ratio ($\leq$1.0)
C 10 X 15.3	Staircase Stringer (typ.)	2	0.71
C 10 X 15.3	Platform Channel @ El. 43'-6"	2	0.95
W 8 X 28	Platform Beam @ El. 110'-0"	2	0.98

3.8.5.5 Structural Acceptance Criteria

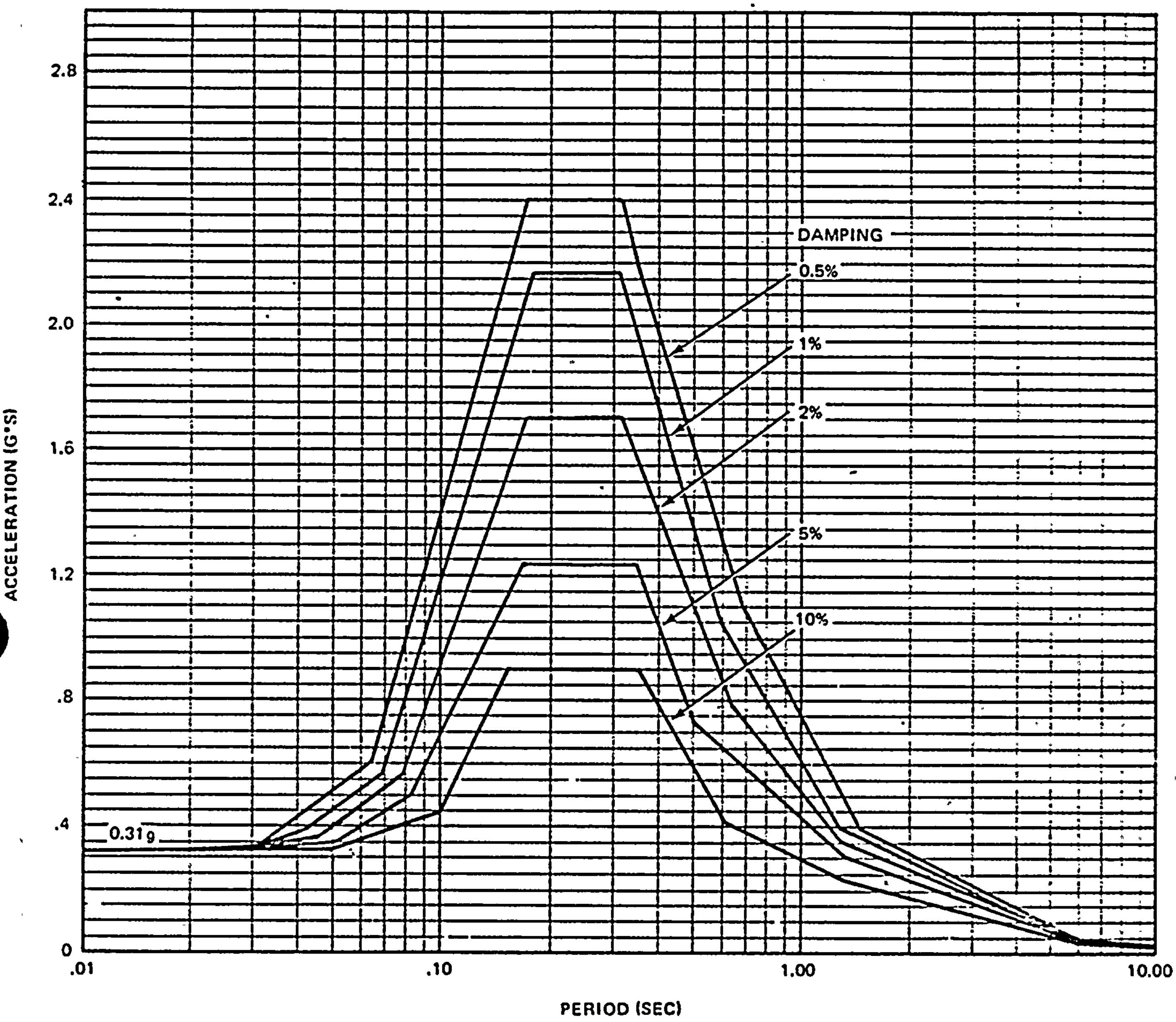
The foundations of Seismic Category I buildings are designed to meet the same structural acceptance criteria as the buildings themselves. These criteria are discussed in sections 3.8.1.5, 3.8.3.5, and 3.8.4.5. The limiting conditions for the foundation medium, together with a comparison of actual capacity and estimated structure loads, are found in sections 2.5.4.10 and 2.5.4.11. Computed factors of 1 safety against overturning, sliding, and flotation for Category I structures are given in table 3.8-5. 1

Table 3.8-5
COMPUTED FACTORS OF SAFETY

Structure	Overturning		Sliding		Flotation
	OBE	SSE	OBE	SSE	
Auxiliary	3200	830	2.2	1.3	4.7
Containment	3400	1200	1.7	1.2	4.5
Control	1500	420	1.6	1.2	4.8
Diesel Generator	1200	400	2.2	1.1	NA[a]
Fuel	1600	400	1.9	1.1	NA
Main Steam Support	340	91	1.6	1.1	NA
Condensate Storage and Refueling Water Tanks	500	150	1.7	1.4	NA

a. Not applicable

1



Palo Verde Nuclear Generating Station
FSAR

AUXILIARY BUILDING
SSE VERTICAL ACCELERATION
RESPONSE SPECTRA

Figure 3D-1

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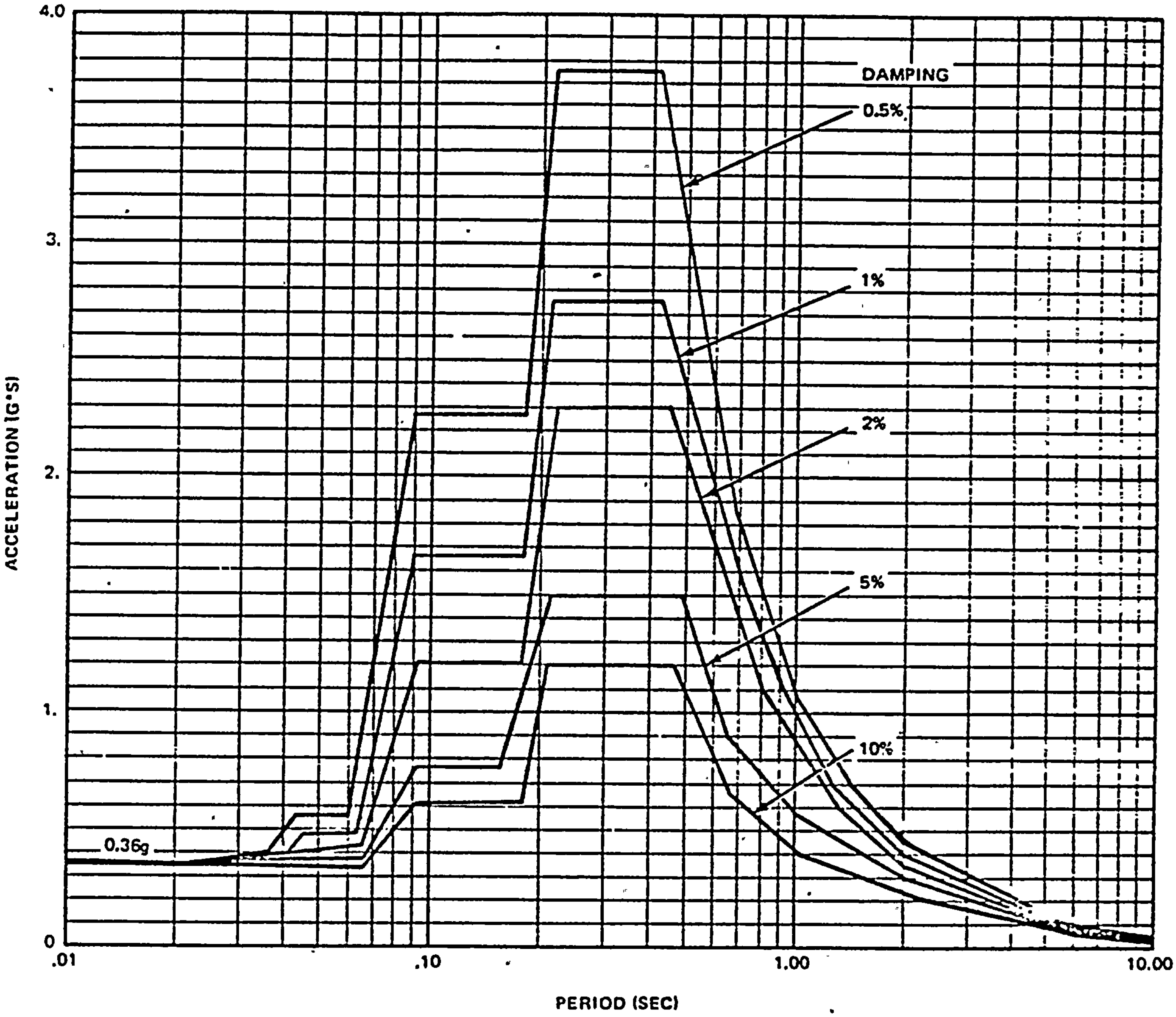

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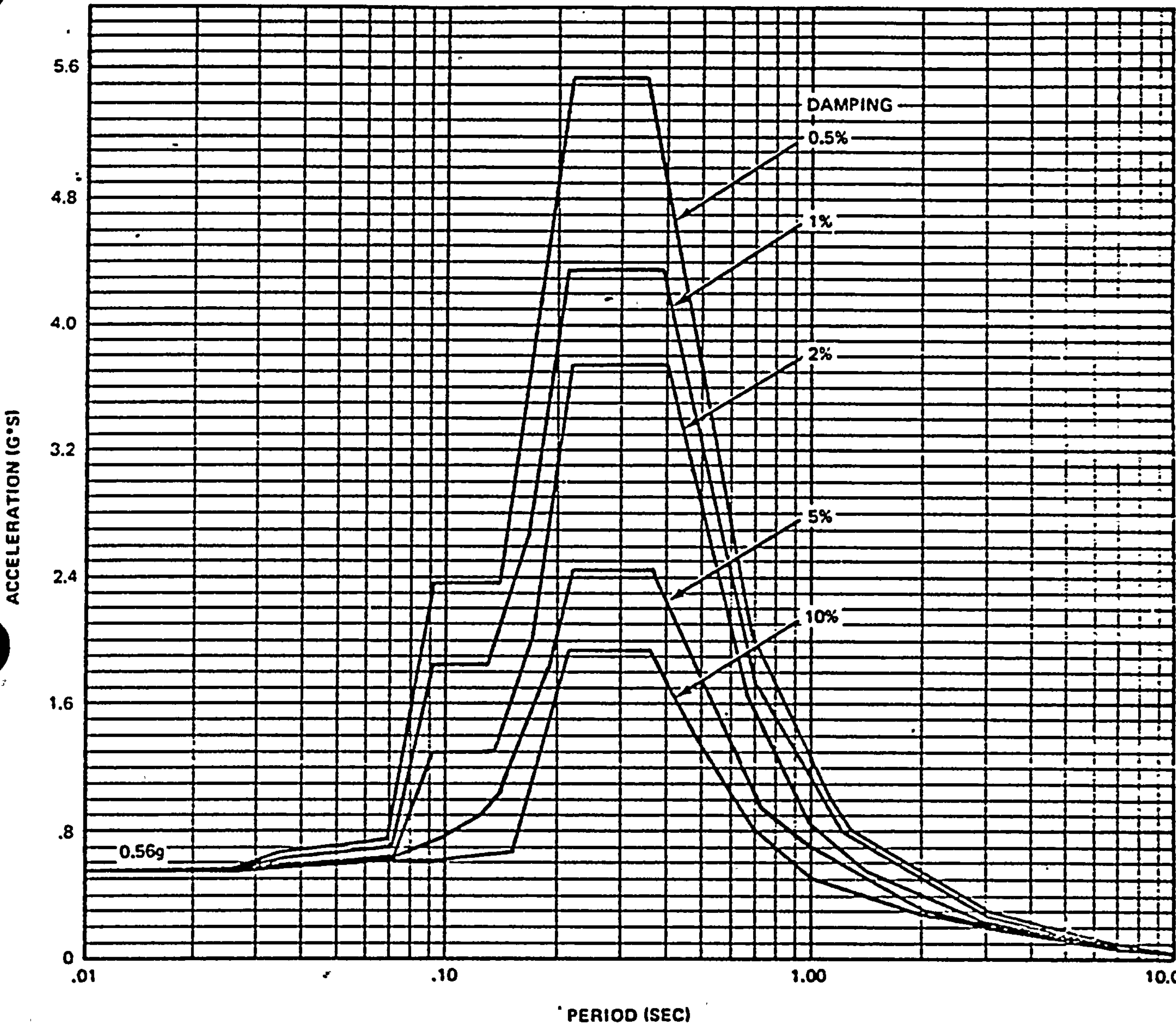
AUXILIARY BUILDING
SSE HORIZONTAL ACCELERATION
RESPONSE SPECTRA EL 40'-0"

Figure 3D-2

284.



Palo Verde Nuclear Generating Station
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AUXILIARY BUILDING
SSE HORIZONTAL ACCELERATION
RESPONSE SPECTRA EL 156'-0", ROOF

Figure 3D-3

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PALO VERDE NUCLEAR STATION UNITS 1, 2, 3
DESIGN ADEQUACY AUDIT

STRUCTURAL AUDIT OF FUEL BUILDING
Part I - General Analysis

I. BASIC DESIGN CRITERIA

A. 'g' value - free field

	Seismic level based on construction permit license	Seismic level used in design of structures and equipment
SSE	0.20g	0.25g
OBE	0.10g	0.13g

Reference: FSAR, Section 3.7

B. Spectra (attach figs. for all damping values, ductilities)

1. zero period acceleration

SSE 0.25g
OBE 0.13g

Reference: FSAR, Figures 3.7-1 - 3.7-4 (Pages 332, 3, 4, 5) and Section 3.7.1.1
This is consistent with Reg. Guide 1.60

Frequency (or period) interval

Refer to BC-TOP-4A, Section 2.5.1 (c)

C. Damping

Refer to FSAR, Section 3.7.1.3
This is consistent with Reg. Guide 1.61
Refer to FSAR figures 3.7-5 and 3.7-6 (Pages 92A, 92B)

D. Artificial time history and corresponding spectra (attach figures)

1. original time history and its composition, i.e., rising time, strong motion and tail end.

Refer to FSAR, Section 3.7.1.2 and BC-TOP-4A, Section 2.5

2. base line correction, check the integrated velocity and displacement time histories

   The velocity and displacement time histories were checked and found to be satisfactory. In addition, only the acceleration time history was used as input in analysis.

3. time interval - compatible with the highest frequency considered in the spectral calculation

   0.005 seconds

E. Motion duration

   24 seconds

   Refer to FSAR, Section 3.7.1.2

F. components of motion including their relative motion amplitudes

   Analysis was performed for the three principle directions with equal amplitudes.

G. Dead and live loads for various operating floors and base slab

   ~~Refer to Project Design Criteria Part II, Section 3.0 and Part III, Section 4.0~~

   Dead Load - includes all structures, major equipment load and 50 psf equivalent for small equipment.

   Live Load - See action item 3

H. Ground water level

The groundwater design level is at plant El 70′-0". The actual water level is at approximately plant El. 60′-0".

Reference: FSAR, Sections 2.4.13.2.4 and 2.4.13.5.

I. Backfill earth pressure, wind, overpressure due to postulated external explosions (as applicable)

~~Refer to Project Design Criteria Part II, Sections 3.4.5.3 and 3.5.5 for lateral earth pressure, wind and tornado loads.~~

For lateral earth pressure see pages 208 A & 208 B., 208C, 208D

Wind does not govern.

J. Other considerations

All penetrations in roof and exterior walls that expose safety related equipment have tornado missile protection. Concrete covers or steel plate shields to prevent missile perforation are provided.

II. ANALYSIS METHOD

A. Seismic Analysis

1. Mathematical model-general description with sketch.

Two planar lumped parameter models were used.

Refer to FSAR, Sections 3.7.2.3.3 and Figure 3.7-13.

A three dimensional finite element model was used for final verification of design.

a. (1) concrete modulus

$E_c = 3.64 \times 10^6$ psi for F'c = 4000 psi

$E_c = 4.07 \times 10^6$ psi for F'c = 5000 psi

(2) rebar modulus and yield strength

$E = 29 \times 10^3$ Ksi

$F_y = 60$ Ksi

(3) Poisson's ratio

$\nu = 0.17$ for concrete.

Reference: "Theory of Plates and Shells" 2nd edition by Timoshenko, Woinowsky, Kreiger p. 97.

(4) damping

Refer to FSAR, Section 3.7.1.3 ~~and responce to NRC Question 220.2 (Q3A-4)~~.

This is consistent with Reg. Guide 1.61.

(5) properties of foundation materials

- shear modulus

  Refer to FSAR, Figure 3.7-7, -8, -9.

- subgrade reactions

  ~~Refer to Project Design Criteria Part II, Section 3.4.5.4 for coefficient of subgrade reaction.~~
  Coefficient of subgrade reaction = 40-60 kips/ft²/ft

- bearing capabilities

  Refer to Tables 2.5-15 and 2.5-16 of the FSAR.

(6) other parameters

b. stiffness calculations

(1) exterior walls

Exterior and interior walls stiffness calculations were performed manually using standard engineering methods.

(2) interior walls

Same as exterior walls.

2. method of Analysis

a. method of analysis used (time history, response spectrum methods, etc.) and consideration of torsional and translational response

Two planar lumped parameter mathematical models were used for an acceleration time history analyses to generate in-structure response spectra and to perform a response spectrum analysis.

(1) general description

The Fuel Handling Building analysis used two planar lumped parameter models. Soil structure interaction was incorporated into the model by adding to the fixed base system discrete soil springs based on elastic half space theory. The three dimensional finite element model was used for a response spectrum analysis to determine seismic stresses within the structure to verify the design.
Based on the results of 4 cat. I structures by the comparison of lumped model & finite element model, the NRC SSI intent is met.

(2) findings and comments

b. selection of number of masses and degrees of freedom

(1) general description

Data for the planar lumped parameter model are as follows:

For horizontal direction earthquake, the model consisted of 12 nodes and 37 degrees of freedom.

For vertical direction earthquake, the model consisted of 5 nodes and 5 degrees of freedom.

Refer to FSAR, Figure 3.7-13. (Pg. 339)

(2) findings and comments

c. number of modes considered

The time history analyses performed for the lumped parameter model considered all modes since the direct integration method was utilized. The response spectrum analysis used in the finite element model verification of design considered 66 modes.

(1) general description

Refer to FSAR, Table 3.7-6 for modal frequencies and participation factors. (Pg. 331)

(2) findings and comments

d. combining modal responses

(1) actual procedures used

Refer to FSAR, Section 3.7.2.7.

(2) general findings

e. consideration of three components of motion

(1) actual procedures used

Refer to FSAR, Section 3.7.2.6.
This is consistent with Reg Guide 1.92

(2) general findings

f. consideration of soil-structure interaction

OBE	East-West	$Kx = \cancel{4.507}\ 5.6 \times 10^5$ K/ft	$Kxx = 2.363 \times 10^9$ K-Ft/Rad
	North-South	$Ky = 6.222 \times 10^5$ K/ft	$Kyy = 1.490 \times 10^9$ K-Ft/Rad
	Vertical	$Kz = 7.046 \times 10^5$ K/ft	
SSE	East-West	$Kx = \cancel{5.6}\ 4.507 \times 10^5$ K/ft	$Kxx = 1.902 \times 10^9$ K-Ft/Rad
	North-South	$Ky = 5.008 \times 10^5$ K/ft	$Kyy = 1.199 \times 10^9$ K-Ft/Rad
	Vertical	$Kz = 5.672 \times 10^5$ K/ft	

(1) general description

(2) findings and comments

g. decoupling criteria for subsystems

(1) general procedure

Refer to BC-TOP-4A, Section 3.2.

(2) key examples

(3) the other criteria pertaining to frequency ratio as defined in SRP 3.7.3.II.3.b are also met.

(3) general findings and comments

h. modeling of hydrodynamic effects in spent fuel pool

The spent fuel pool water was coupled into the model using the concepts of US AEC TID 7024, Nuclear Reactors and Earthquakes.

i. modeling of spent fuel pool wells and interior floor slabs and equipment thereof

3. development of in-structure response spectra

Refer to FSAR, Section 3.7.2.5

a. general procedures

The "SPECTRA" program was used to compute the response spectra. Refer to FSAR, Appendix 3B for a description and applications of this program.

(1) smoothing (describe specific smoothing method used)

The smoothened floor response spectra represent an envelope of the maximum peaks.

(2) peak widening

± 15%

Reference : FSAR, Section 3.7.2.9

b. typical results (attach figures)

Refer to FSAR, Figures 3D-31, -32, -33. (Pages 343, 4, 5)

B. Stress Analysis

1. shear walls and floors

a. mathematical model - general description w/sketch

Shear wall and floor stresses were computed by performing standard manual calculations. Vertical loads were distributed in the structure by conventional methods. Lateral loads were calculated by multiplying lumped mass and seismic accelerations obtained from floor spectra at each level. These lateral loads were then distributed among the shear walls according to their relative stiffness and location.

A three dimensional finite element model was used for verification of the design.

b. method of analysis--incorporation of torsion

The torsional moment was determined by resolving the couple due to eccentricity between the center of mass and the center of rigidity at each floor. Shear derived from this torsional moment was added directly to the forces considered for the individual shear walls.

The three dimensional finite element model was used for stress analysis considering all loads including the response spectrum analysis for seismic loads.

c. load combinations

Refer to FSAR, Section 3.8.3.3.

1. foundation mat

a. mathematical model - description of boundary conditions

The original design was based on manual calculations. The three dimensional finite element model was used for verification.

b. method of analysis

c. load combinations

Refer to FSAR, Section 3.8.3.3.
This is in compliance with the SRP

d. key results (figures, etc.)

Refer to FSAR, Table 3.8-4F. (Pg. 340)

3. Material to protect against structure - structure interaction

Below grade (EL 100′-0″) Rodofoam II (WR Grace and Co.) or EVERLASTIC Micro II (Williams Products, Inc.) may be left in place in seismic joints between structures. Above grade gaps between walls were left open, typically 6″ between Category I structures.

a. mechanical properties

b. additional pressure on walls

c. findings and comments

4. vertical dynamic analysis

   a. mathematical model - general description with sketch

      The planar lumped parameter model consisted of one vertical beam on a rigid mat which represents the exterior and interior walls.

   b. development of stiffnesses, including floor stiffness, as applicable

      Stiffnesses of the walls were calculated manually.

   c. method of analysis

      The model described above was used for acceleration time history and modal response spectrum analyses. The time history analysis was performed using super-SMIS. The modal response spectrum analysis was performed using super-SMIS and SPECTRA.

      The three dimensional finite element model was also used to perform vertical dynamic analysis.

C. Computer Programs Used in Analysis

1. assumptions and limitations

   SAP1.9, SUPER SMIS, SPECTRA, OPTCON, LUCON, BSAP

2. applicability

   Refer to FSAR, Appendix 3B.

3. verification

   - sensitivity study in case of numerical solutions (e.g., finite element analysis)

     Refer to FSAR, Appendix 3B.

4. load input (include all cases)

PROGRAM	INPUT
SAP1.9	Finite Element Model (Modes and Elements), Loading (Pressure, Nodal loads), Response Spectra, Time History.
SPECTRA	In-structure Time Histories, Frequency of Periods and Damping Values.
LUCON	Shear Modulus, Damping Values
SUPER SMIS	Mass Matrix, Stiffness Matrix, Response Spectra, Time History, Damping Matrix
OPTCON	Design Loads and Stresses

5. output (include all cases)

   Refer to FSAR, Appendix 3B.

6. other discussions

D. Overall Stability

1. forces and moments from seismic analysis

SSE

~~P_x 1.80×10^4 K~~

~~P_y 1.59×10^4 K~~

~~P_z 1.35×10^4 K~~

~~M_x 7.93×10^5 FT-K~~

~~M_y 5.82×10^5 FT-K~~

~~x: North-South~~
~~y: East-West~~
~~z: Vertical~~

EAST - WEST

$P = 1.59 * 10^4$ K

$M = 5.82 * 10^5$ FT-K

NORTH - SOUTH

$P = 1.80 * 10^4$ K

$M = 7.93 * 10^5$ FT-K

VERTICAL

$P = 1.35 * 10^4$ K

2. various cases considered

Seismic event loading combination considered OBE or SSE applied in the East-West, North-South and vertical directions simultaneously.

3. bearing pressure versus bearing capability and safety factor against bearing failure

   Refer to FSAR, Section 2.5.4.10 and Tables 2.5-15, -16. (Pages 328, 9)

4. factors of safety

   Refer to FSAR, Section 3.8.5.5 and Table 3.8-5. (Pg. 342)

   a. sliding

      Factor of Safety = 1.1 (SSE)

   b. overturning

      Factor of Safety = 400 (SSE)

E. Interaction of Non-category I Structures with the Structure Considered

1. identification of pertinent non-Category I structures

None: There are no non-Category I structures adjacent to the Fuel Building

2. consideration given to potential failure of non-Category I systems on Category I systems

Within the Fuel Building, non-Category I Systems which potentially could affect Category I Systems were designed for structural integrity under SSE loads.

3. general findings and comments

During a walk-down those items whose failure will not affect any safety related equipment are left as they are. If they are judged that they might affect Category I systems, they are designed to maintain their structural integrity under an SSE.

F. Design Consideration for Tornado Missiles

1. design requirements

   Refer to FSAR, Table 3.5-8. (Pg. 330)

2. models for

   a. local damage

      Refer to FSAR, Section 3.5.3.

   b. overall response

      Refer to FSAR, Section 3.5.3.

3. load combinations

   Refer to FSAR, Section 3.8.3.3.

4. forces

   The governing structural members are of sufficient thickness to preclude perforation by postulated missiles and maintain structural integrity.

6. general comments and preliminary audit findings

III. CONFORMANCE TO ACCEPTABLE CRITERIA

A. Identification of deviations, if any

None.

B. Justification of deviations and disposition of the deviations

D. general comments

Part II Audit of Key Designs

A. Exterior Shear Walls

1. design requirements

   The exterior walls are designed to satisfy structural function as bearing walls, shear walls, and protection against tornado missiles.

2. design loads (from general analysts)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4F. (Pg. 340)

4. detailed design of rebar placement at key sections

   See Attachment A. (Pg. 325)

5. general comments and preliminary audit findings

B. Interior Shear Walls

1. design requirements

The interior walls are designed to satisfy structural function as bearing walls and shear walls.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4F. (Pg. 340)

4. detailed design of rebar placement at key sections

See Attachment A. (Pg. 325)

5. general comments and preliminary audit findings

C. Main Floors and Roofs

1. design requirements

Main floors and roofs are primarily designed for dead, live and seismic loads. ~~as defined in the Project Design Criteria.~~ Roofs are also designed to satisfy minimum thickness to preclude perforation by tornado generated missiles.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4F. (Pg. 340)

4. detailed design of rebar placement at key sections

See Attachment A. (Pg. 325)

5. general comments and preliminary audit findings

D. Steel Structural Bracing Systems (if any)

1. design requirements

The Fuel Building floors at elevation 120′-0", 140′-0", and roof are supported on structural steel beams, girders and columns. Beams in general are designed for construction loads. Girders and columns are designed for dead, live and seismic loads.

2. design loads

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4G. (Pg. 341)

6. general comments and preliminary audit findings

E. Foundation Mat

1. design requirements

   Refer to FSAR, Sections 3.8.5.4.2 and 3.8.5.5.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4F. (Pg. 340)

4. detailed design of rebar placement at key sections

   See Attachment A. (Pg. 325)

5. general comments and preliminary audit findings

F. Main Frame Concrete Column Design (Key Columns)

Not applicable. There are no concrete columns in this building.

1. design requirements

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design of rebar placement at key sections

5. general comments and preliminary audit findings

G. Secondary Floors

Not applicable.

1. design requirements

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design of rebar placement at key sections

5. general comments and preliminary audit findings

H. Detailing at Floor-Wall Joints

1. design requirements

   As per ACI 318-71 Code, Chapters 6 and 17.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4F. (Pg. 340)

4. detailed design of rebar placement at key sections

   See Attachment A. (Pg. 325)

5. general comments and preliminary audit findings

I. Dynamic Effects Applied to Floors and Walls by Machinery

Dynamic effects from machinery are negligible. Major pumps are located on the basemat. HVAC units are mounted with isolation pads.

1. design requirements

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design

5. general comments and preliminary audit findings

J. Crane & Support

1. design of bents (columns and roof trusses)

The crane support system consists of corbels and pilasters.

a. design requirements

Corbels and pilasters, located along the north and south walls, support the crane, rail and crane support girders. They are designed for dead, live, impact and seismic loads. ~~per the Project Design Criteria.~~

b. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

c. forces and moments at key sections

Maximum forces in the pilasters at El. 167′-0" subjected to crane lateral seismic loads are:

M_u = 4442 K-Ft

P_u = 171.6 K

d. detailed design

See Attachment A. (Pg. 325)

e. general comments and preliminary audit findings

2. design of girders supporting crane rails

a. design requirements

1) 150 T Crane

2) 10 T Crane

b. design loads (from general analysis)

The crane support girders are designed for dead, live, impact, and seismic loads. ~~per the Project Design Criteria.~~

b. design loads (from general analysis)

~~Refer to Project Design Criteria Part II, Section 3.5.4 and Part III, Section 1.4.1.3.~~
Dead load includes girder and all the associated equipment. Live load is the load lifted by crane. Capacities of cranes are 150 T & 10 T.

c. forces and moments at key sections

150 T Crane support Girder: Moment = 3646 K-Ft
Shear = 600 K

10 T Crane support Girder: Moment = 486 K-Ft
Shear = 133 K

d. detailed design

See Attachment B. (Pg. 326)

e. general comments and preliminary audit findings

3. design of spent fuel bridge

The spent fuel bridge is designed and furnished by Combustion Engineering.

a. design requirements

b. design loads (from general analysis)

c. forces and moments at key sections

d. detailed design

e. general comments and preliminary audit findings

K. Fuel Pool Liner Design

1. stresses and strain controls

   The liner is not designed as a load carrying member. It acts as a water tight membrane for the pool.

2. conformance to code requirements

   Design is in accordance with AISC requirements.

3. analysis procedure and results

   See Item 1.

4. consideration of accidental drop of crane loads

   The west wall of the spent fuel pool was designed to withstand the impact of a fuel cask drop.

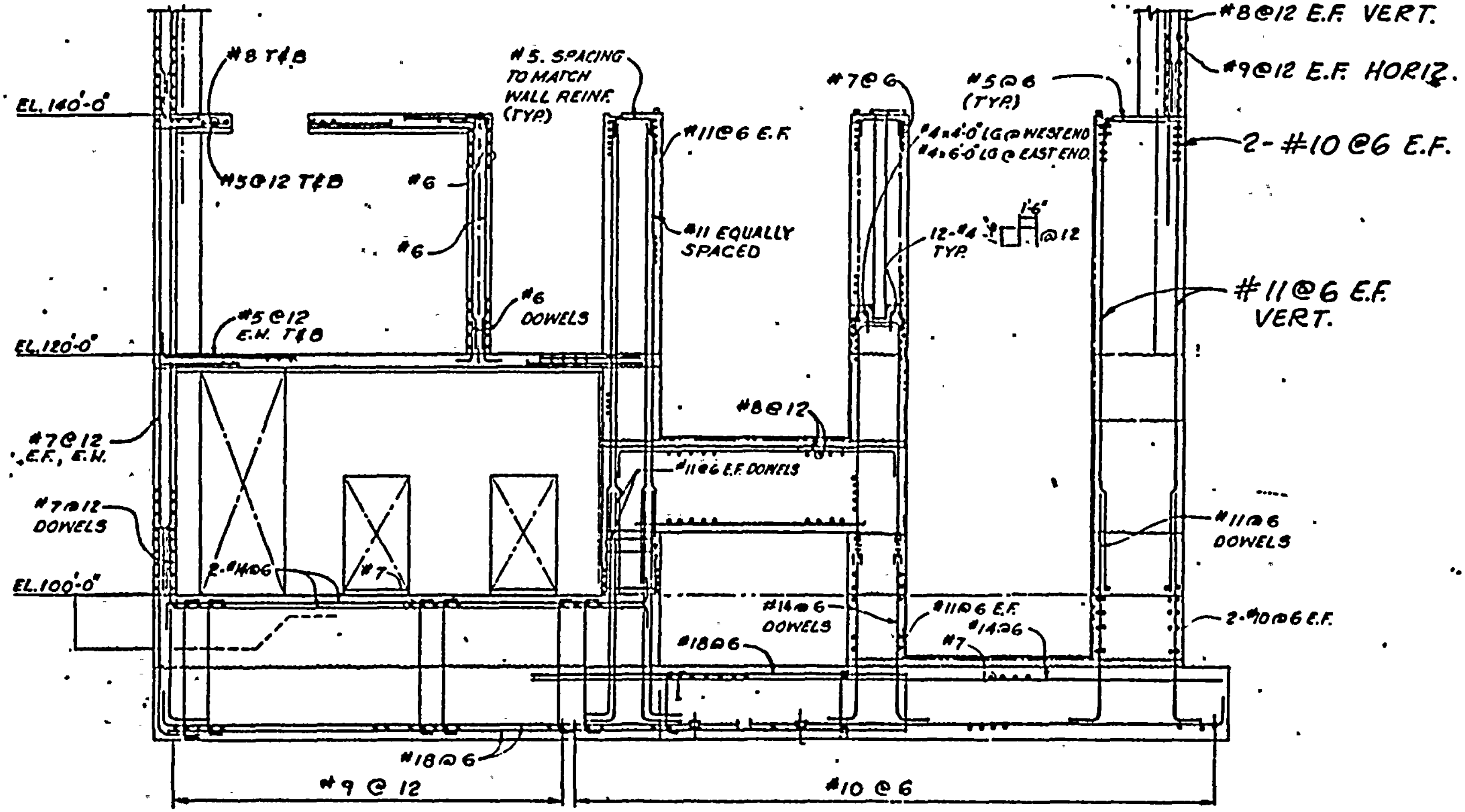
   Refer to FSAR, amended Section 3.8.4.1.2. (See Attachment C.)

5. corrosion effects (e.g., pitting) on liner integrity

   Stainless steel liner plate has been used.

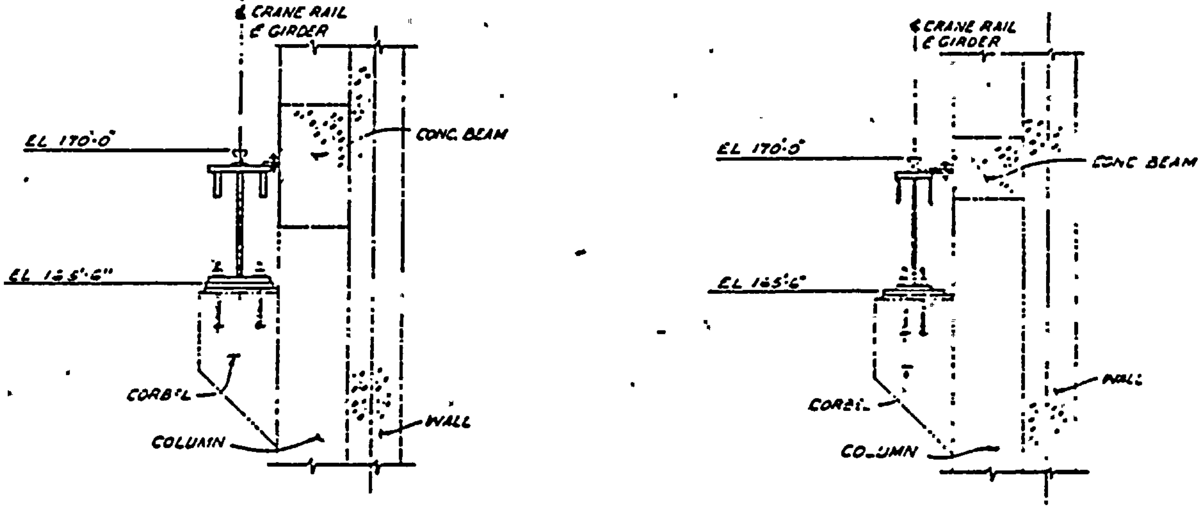
6. preliminary findings of audit results

ATTACHMENT "A"

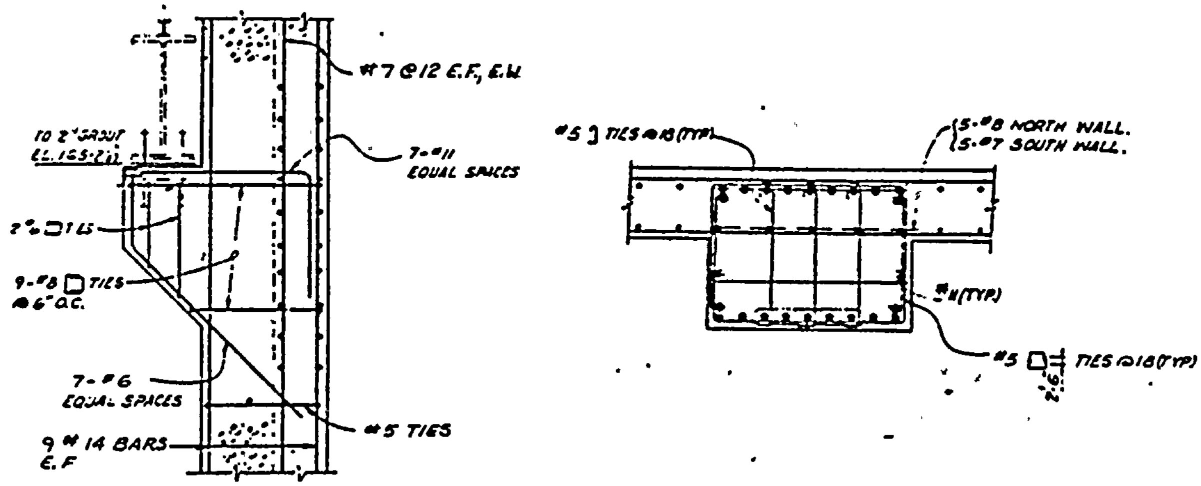


FUEL BUILDING
TYP. WALL SECTION

FUEL BUILDING



TYP. CRANE GIRDER DETAILS



TYP. WALL SECTIONS

ATTACHMENT "B"

ATTACHMENT C

3.8.4.1.2 Fuel Building

The fuel building is 88 x 124 feet in plan and is a reinforced concrete structure whose roof is 94 feet above grade. It is physically separated from adjoining structures and has an independent foundation. The building contains the new fuel storage area and spent fuel pool. The walls and the floor of the spent fuel pool are lined with stainless steel plates for leaktightness.

The new and spent fuel storage is described in section 9.1.

The fuel building has an overhead crane capable of handling such heavy loads as the fuel cask. Travel of this crane over the spent fuel pool is prevented by design. Interlocks are provided to prevent the crane from moving over the new fuel area during cask handling operations. A new fuel handling crane, running on rails mounted over the operating floor, is provided to handle the new fuel assemblies.

A spent fuel handling machine, running on rails mounted on the operating floor, is provided to handle spent fuel assemblies.

An aluminum honeycomb energy absorption pad mounted on the wall in cask decontamination pit, is provided to prevent any damage to west wall of the spent fuel pool from the fuel cask drop.

Building plans and elevations are shown in figures 1.2-4 through 1.2-13.

Table 2.5-15

STATIC BEARING CAPACITY OF CATEGORY I STRUCTURES

Structure	Average Static Design Load $q_s (k/ft^2)$	Ultimate Bearing Capacity $q_o (k/ft^2)$	Factor of Safety (q_o/q_s)
Containment Building	7.9	35.7	4.5
Auxiliary Building (deep section)	6.2	34.9	5.6
Main Steam Support Structure	7.1	64.8	9.1
Control Building	3.3	45.3	13.7
Fuel Building	5.3	54.9	10.4
Diesel Generator Building	3.1	79.5	25.6
Refueling Water Tank	4.4	90.4	20.5
Condensate Storage Tank	3.5	112.4	32.1

Table 2.5-16

DYNAMIC BEARING CAPACITY OF CATEGORY I STRUCTURES[a]

Structure	Equivalent Uniform Vertical Stress q_d (k/ft^2)	Ultimate Bearing Capacity q_o (k/ft^2)	Factor of Safety (q_o/q_d)
Containment Building	16.1	32.2	2.0
Auxiliary Building (deep section)	10.3	25.8	2.5
Main Steam Support Structure	25.3	60.6	2.4
Control Building	9.8	39.8	4.1
Fuel Building	19.1	50.3	2.6
Diesel Generator Building	5.6	75.5	13.5
Refueling Water Tank	13.2	58.7	4.4
Condensate Storage Tank[b]	13.2	30.2	2.3

a. Based upon maximum dynamic loads derived from analyses described in section 3.7.

b. Condensate storage tank loads were conservatively chosen to be equal to the dynamic design load for the refueling water tank. Actual loads will be less.

1

Table 3.5-8

TORNADO-GENERATED MISSILES CONSIDERED IN DESIGN OF SAFE SHUTDOWN STRUCTURES

Description	Weight (lbs)	Impact Area (ft^2)	Maximum Velocity (ft/s)	Kinetic Energy (ft-lbs)
(A) A 12-foot wood plank, 4 x 12 inches in cross-section, traveling end on at a speed of 240 mi/h.	200	0.333	352	3.85×10^5
(B) A steel pipe, Schedule 40, 3 inches in diameter by 10 feet long, traveling end on at 120 mi/h.	78	0.063	176	3.75×10^4
(C) A steel rod, 1 inch in diameter, 3 feet long, traveling end on at 180 mi/h.	8	0.005	264	8.66×10^3
(D) A steel pipe, Schedule 40, 6 inches in diameter by 15 feet long, traveling end on at 120 mi/h.	285	0.24	176	1.37×10^5
(E) A steel pipe, Schedule 40, 12 inches in diameter by 15 feet long, traveling end on at 120 mi/h.	743	0.886	176	3.57×10^5
(F) A utility pole, 13-1/2 inches in diameter, 35 feet long, traveling end on at 120 mi/h.	1490	0.994	176	7.17×10^5
(G) An automobile of 4,000 pounds weight, striking the structure at 60 mi/h.	4000	20.0	88	4.81×10^5

Table 3.7-6

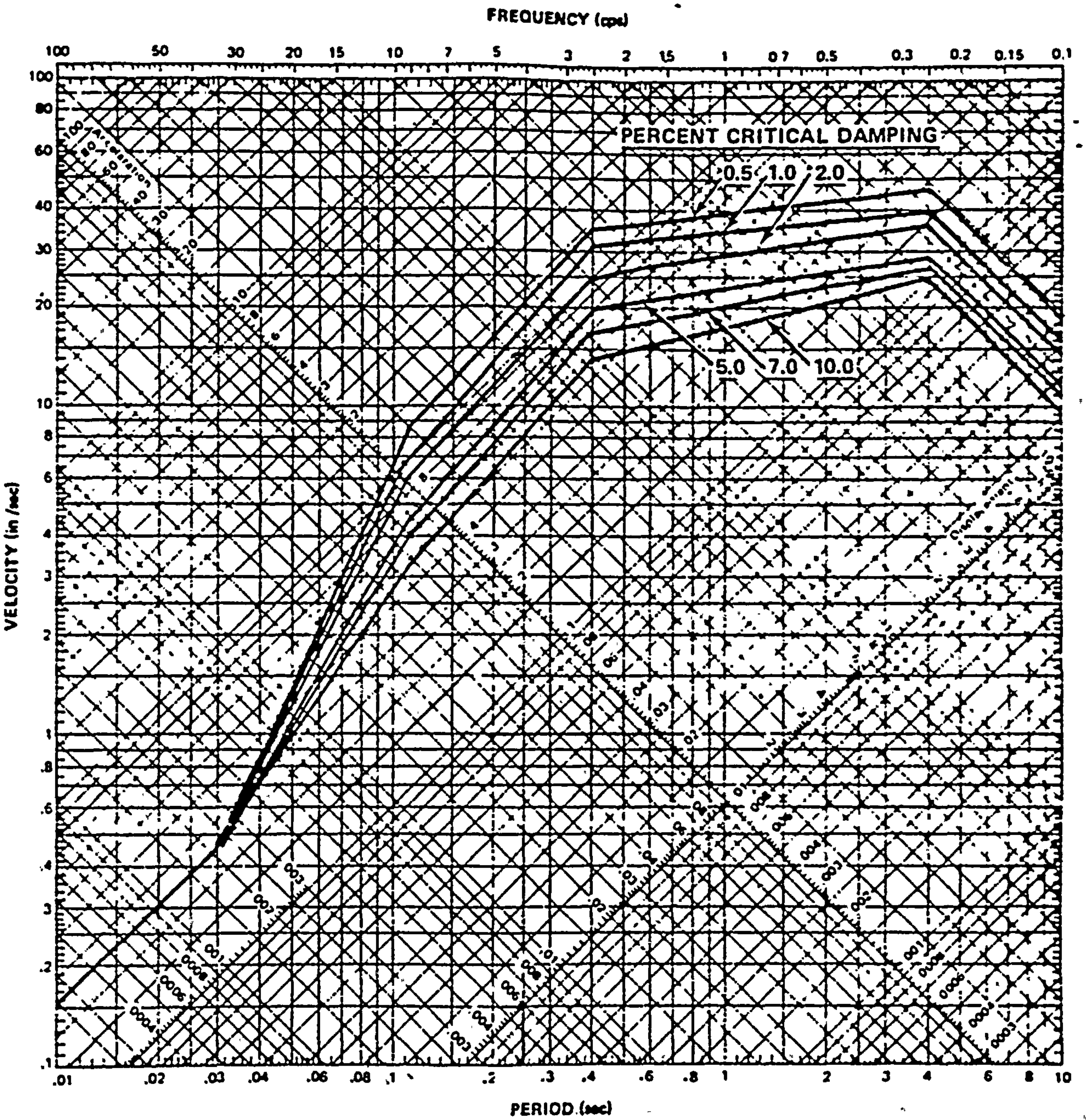
FUEL BUILDING

NATURAL FREQUENCIES (b) 4

		Mode	Frequency (Hz)
Horizontal (N-S)	OBE	1	0.26 (a)
		2	2.77
		3	4.81
		4	5.42
		5	5.77
		6	6.34
		7	17.26
		8	29.92
		9	34.03
	SSE	1	0.26 (a)
		2	2.52
		3	4.36
		4	5.38
		5	5.51
		6	5.99
		7	16.95
		8	29.83
		9	33.91
Horizontal (E-W)	OBE	1	0.26 (a)
		2	2.73
		3	3.46
		4	5.47
		5	5.60
		6	6.13
		7	7.81
		8	14.66
		9	20.84
	SSE	1	0.26 (a)
		2	2.59
		3	3.36
		4	4.97
		5	5.40
		6	5.76
		7	7.77
		8	14.66
		9	20.63
Vertical	OBE	1	4.61
		2	36.77
	SSE	1	3.87
		2	36.73

a. Fluid oscillation mode

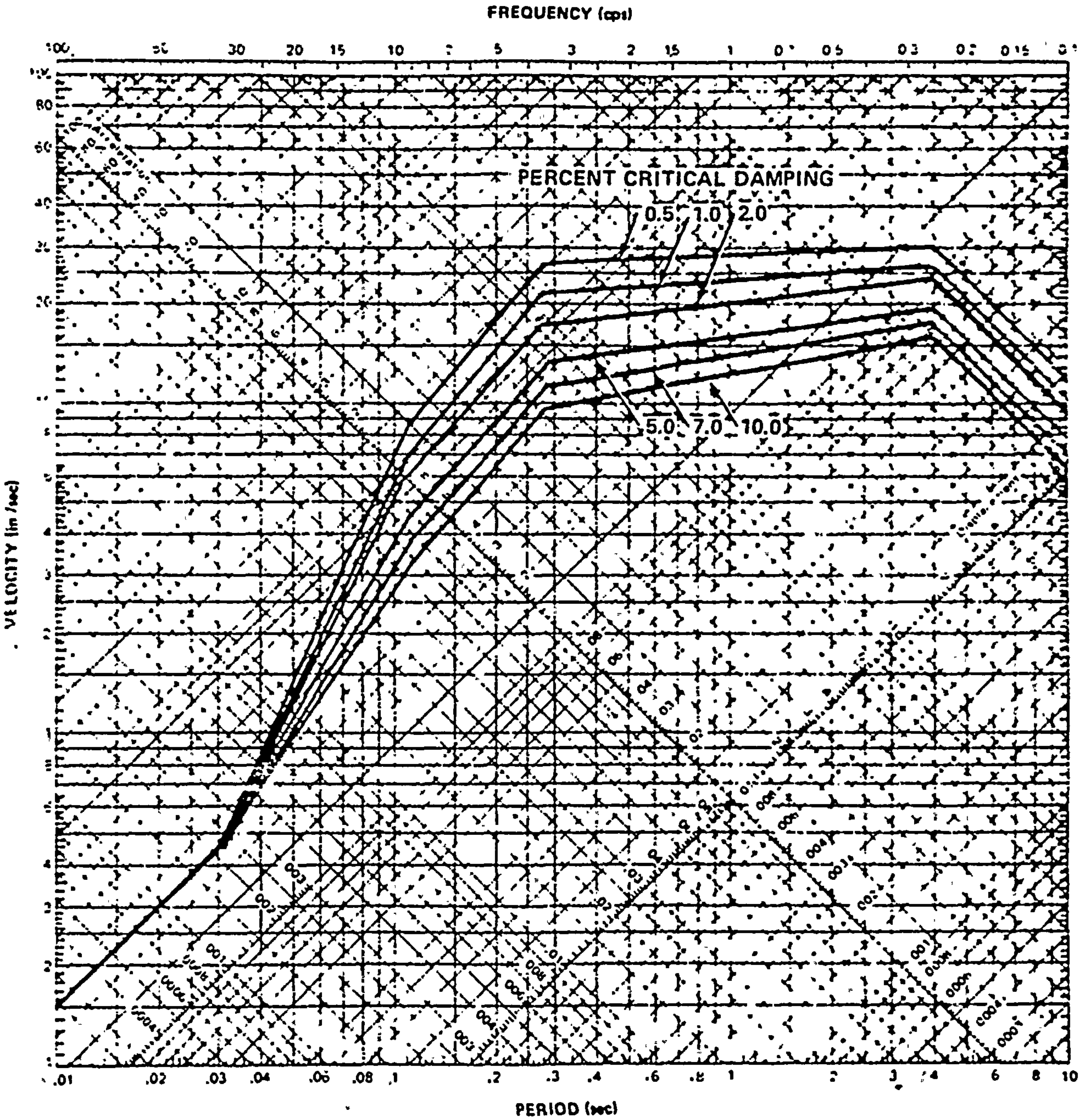
b. See figure 3.7-23 for mode shapes and participation factors. 4



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR SSE 0.25 g

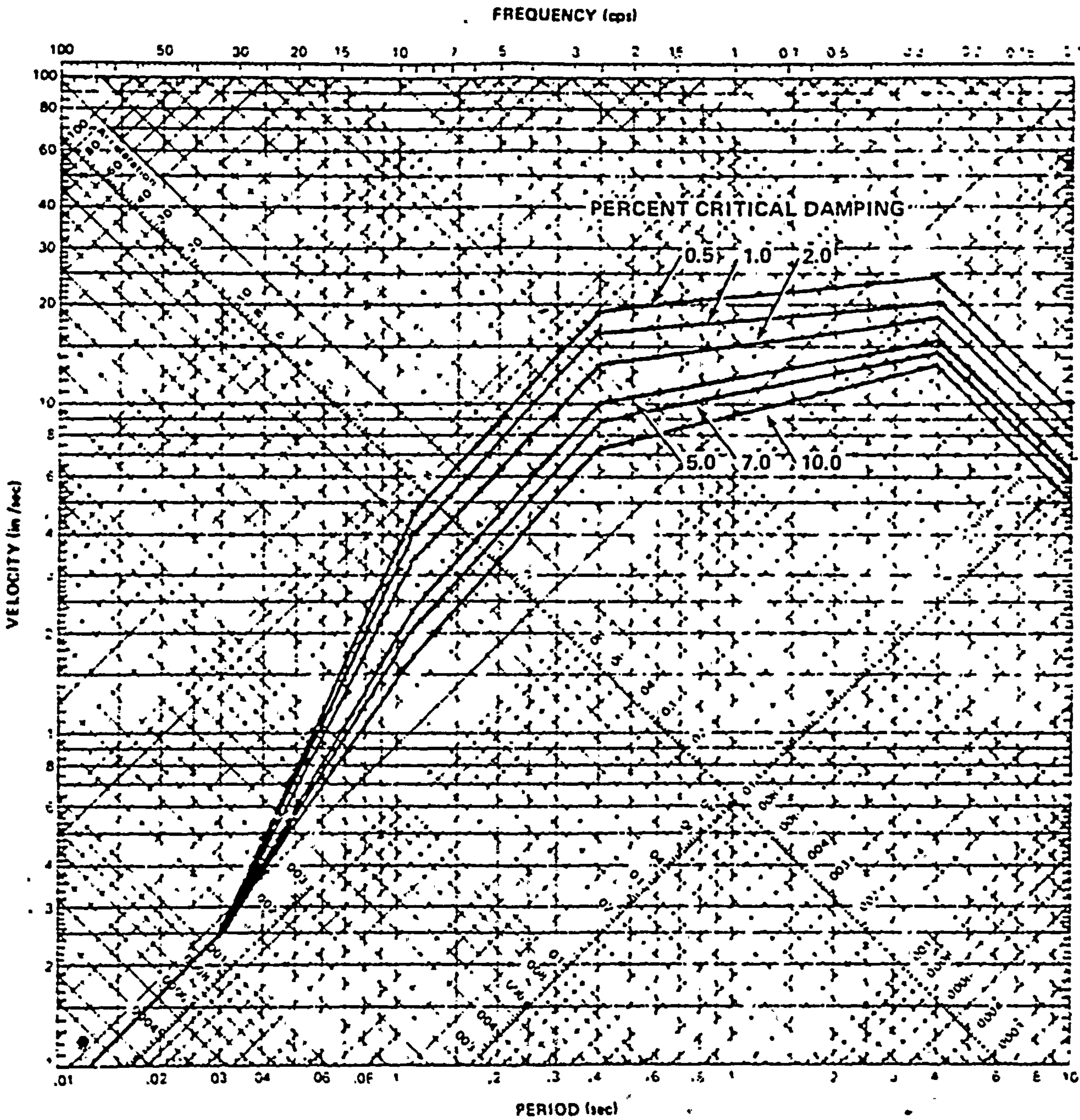
Figure 3.7-1



Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR SSE 0.25 g

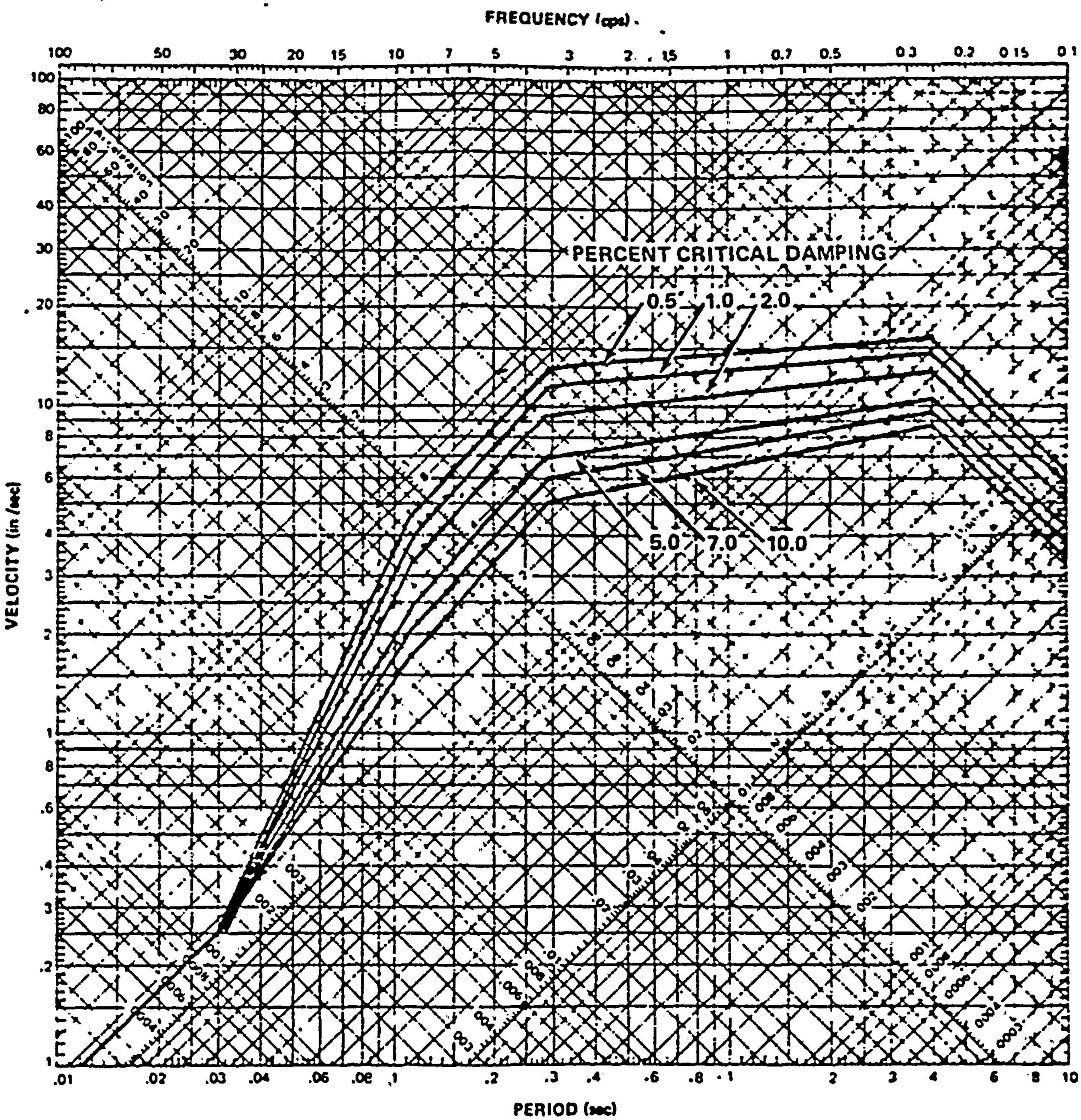
Figure 3.7-2



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR OBE 0.13 g

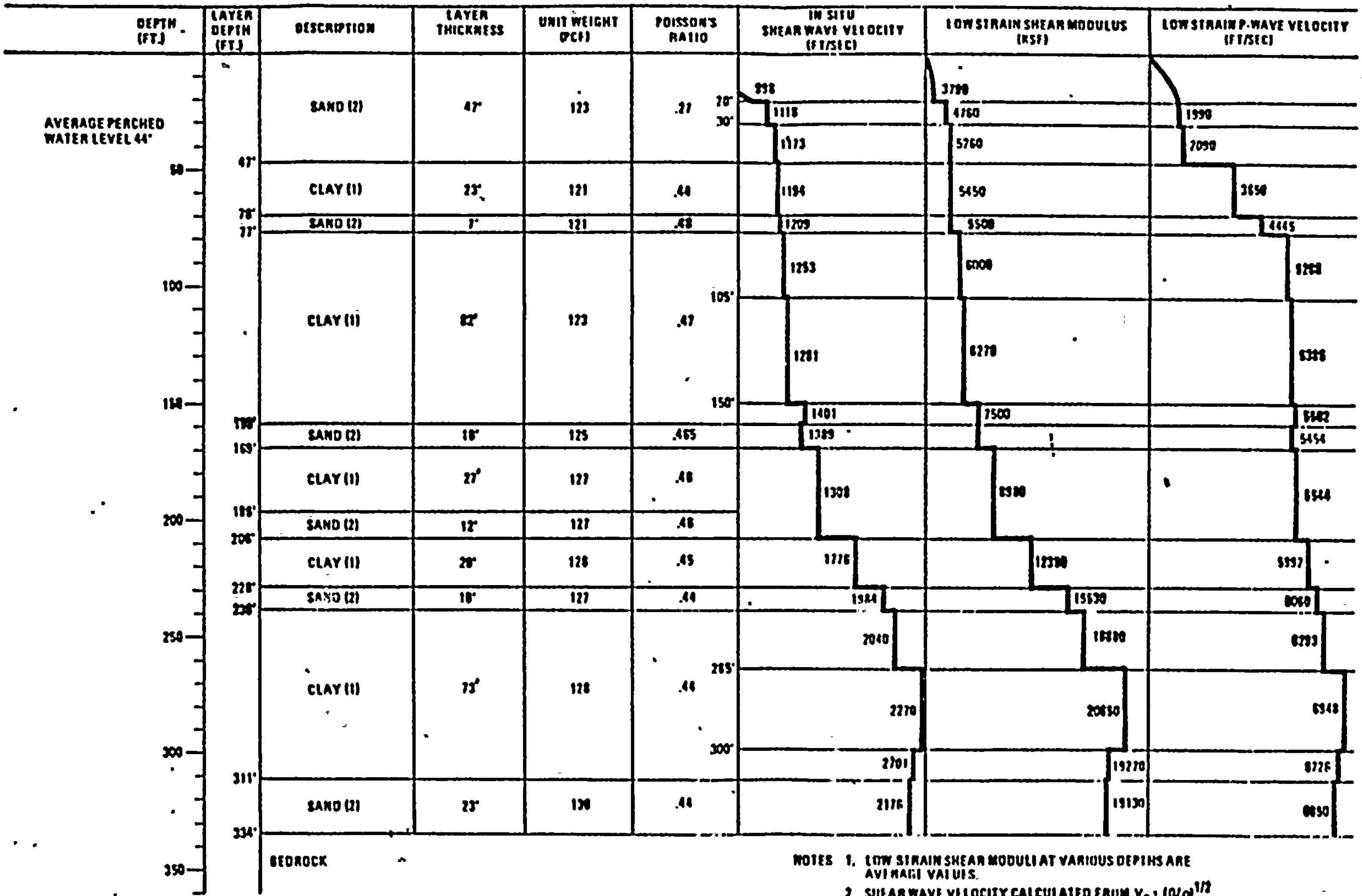
Figure 3.7-3



Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-4



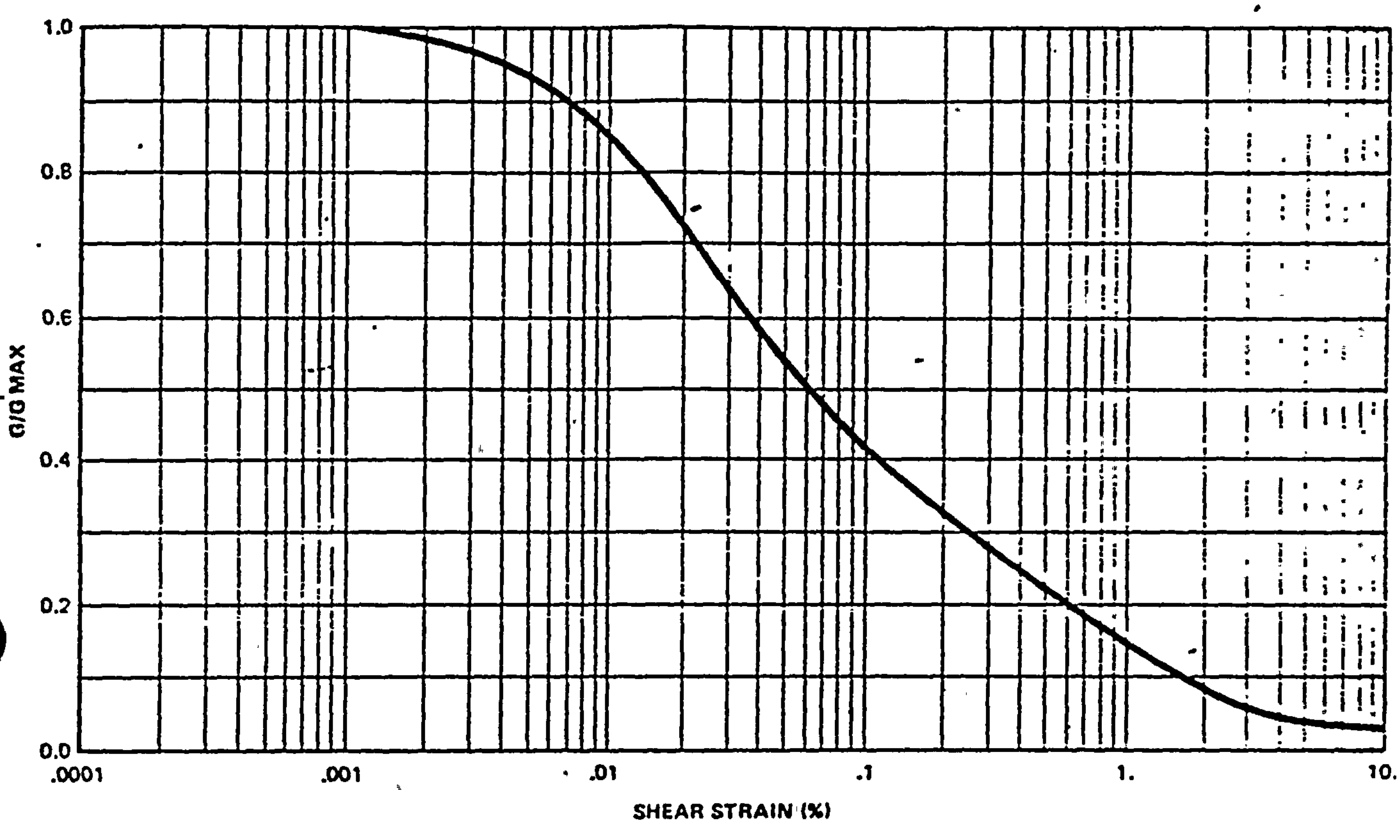
NOTES 1. LOW STRAIN SHEAR MODULI AT VARIOUS DEPTHS ARE AVERAGE VALUES.
2. SHEAR WAVE VELOCITY CALCULATED FROM $V_S = (G/\rho)^{1/2}$
3. P WAVE VELOCITY CALCULATED FROM $V_P = V_S\,[(2 - 2\nu)/(1 - 2\nu)]^{1/2}$

CHANGE

Palo Verde Nuclear Generating Station
FSAR

SOIL PROFILE
FIGURE 2.7-7

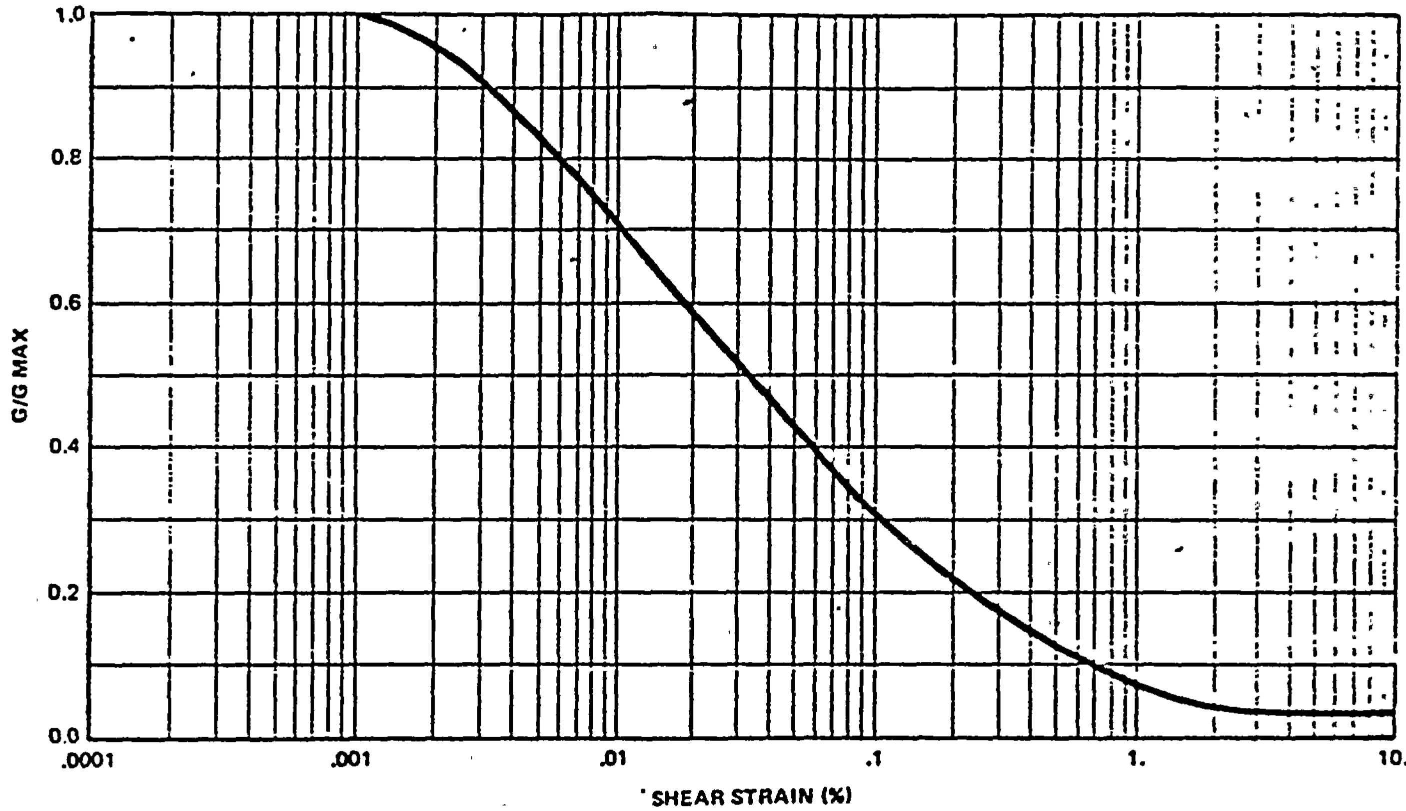
September 1980 Amendment 2



Palo Verde Nuclear Generating Station
FSAR

SHEAR MODULUS VS. STRAIN - CLAY

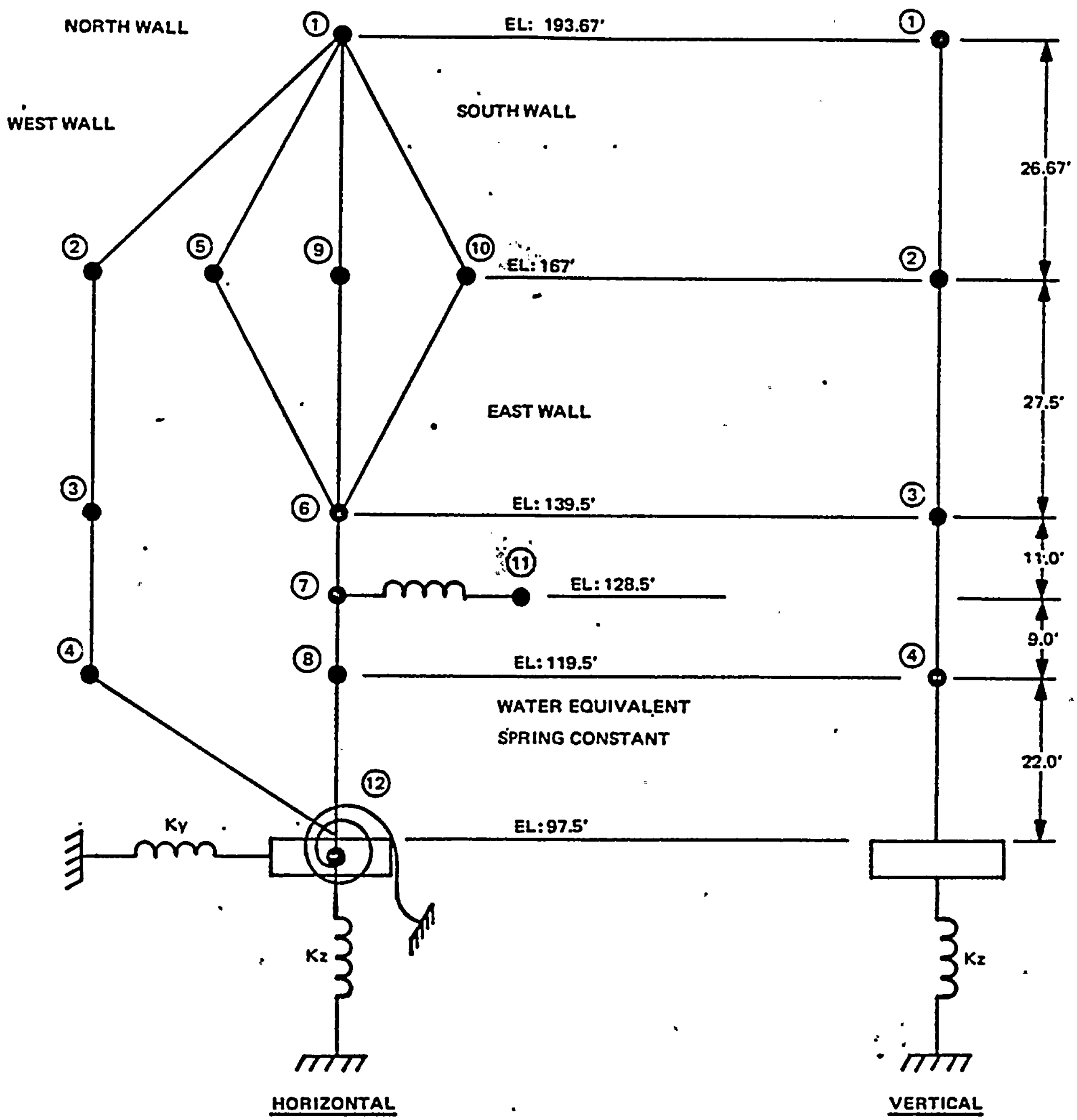
FIGURE 3.7-8



Palo Verde Nuclear Generating Station
FSAR

SHEAR MODULUS VS. STRAIN - SAND

FIGURE 3.7-9



CHANGE

Palo Verde Nuclear Generating Station
FSAR

FUEL BUILDING
LUMPED MASS MODEL

FIGURE 3.7-13

Table 3.8-4F

FUEL BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 1 of 2)

Description of Principal Member	Location of Principal Member	Governing Load Combination Number[a]	Calculated Axial Load (P_u) and Flexural Load (M_u): P_u[b]	Calculated Axial Load (P_u) and Flexural Load (M_u): M_u[c]	Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u)[b][c]	Calculated Shear Load (V_u)[b]	Maximum Shear Capacity (V_u)[b]
Basemat surrounding sump	West side of base mat El. 100'-0"	2	39	-252	-603	-[d]	-[d]
Grade beam 4'-6" thick	N-W extremity of basemat	2	125	-216	-1073	-	-
10'-6" thick area of basemat	North peripheral external strip	2	65	-46	-621	-	-
6'-2" thick area of basemat slab	Decontamination pit floor	2	66	55	132	-	-
7'-0" thick area of basemat slab	Cask loading pit floor	2	54	-3	-11	-	-
12'-0" thick area of basemat slab	Equipment area (E) floor)	2	44	52	86	-	-
1'-0" thick slab	El. 120'-0"	2	11	-2	-11	-	-
1'-0" thick slab	El. 140'-0"	2	25	3	14	-	-
1'-0" thick slab	El. 140'-0"	2	19	-3	-7	-	-
5'-0" thick wall	Exterior east wall at El. 123'-0"	2	35	89	1222	-	-

a. Refer to section 3.8.3.3.2.A(2) for description of load combination number.
b. P_u and V_u are in kips; Sign convention for P_u: Compression (-), Tension (+).
c. M_u is in ft-k/ft.
d. Negligible.

Table 3.8-4F

FUEL BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS (Sheet 2 of 2)

Description of Principal Member	Location of Principal Member	Governing Load Combination Number[a]	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u)[b][c]	Calculated Shear Load (V_u)[b]	Maximum Shear Capacity (V_u)[b]
			P_u[b]	M_u[c]			
1'-9" thick wall	Exterior south wall at El. 145'-0"	2	18	3	27	-	-
8'-0" thick wall	Exterior north wall at El. 129'-6"	2	27	318	1957	-	-
5'-0" thick wall	West wall of spent fuel pool at El. 104'-6"	2	151	3	818	-	-
5'-0" thick wall	South wall of transfer tube canal at El. 129'-6"	2a	38	76	818	-	-
2'-0" thick wall	Wall above equipment area between El. 120' and 140'-0"	2a	0.2	0.5	44	-	-

3

Table 3.8-4G

FUEL BUILDING SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[a]	Combined Stress Ratio (≤ 1.0)
W 14 X 202	Column @ FD - F2.4	2	0.97
W 18 X 35	Floor Beam @ El. 120'-0"	2	0.89
W 30 X 108	Main Girder @ El. 120'-0"	2	0.90
W 30 X 190	Floor Beam @ El. 140'-0"	2	0.89
W 36 X 300	Main Girder @ El. 140'-0"	2	0.97
W 14 X 342	Top Chord Roof Truss	2	0.94
W 14 X 311	Bottom Chord Roof Truss	2	1.00
W 12 X 136	Compression Member Roof Truss	2	0.86
W 12 X 136	Tension Member Roof Truss	2	0.91

a. Refer to section 3.8.3.3.3.A(1) for description of load combination number.

3

3 341

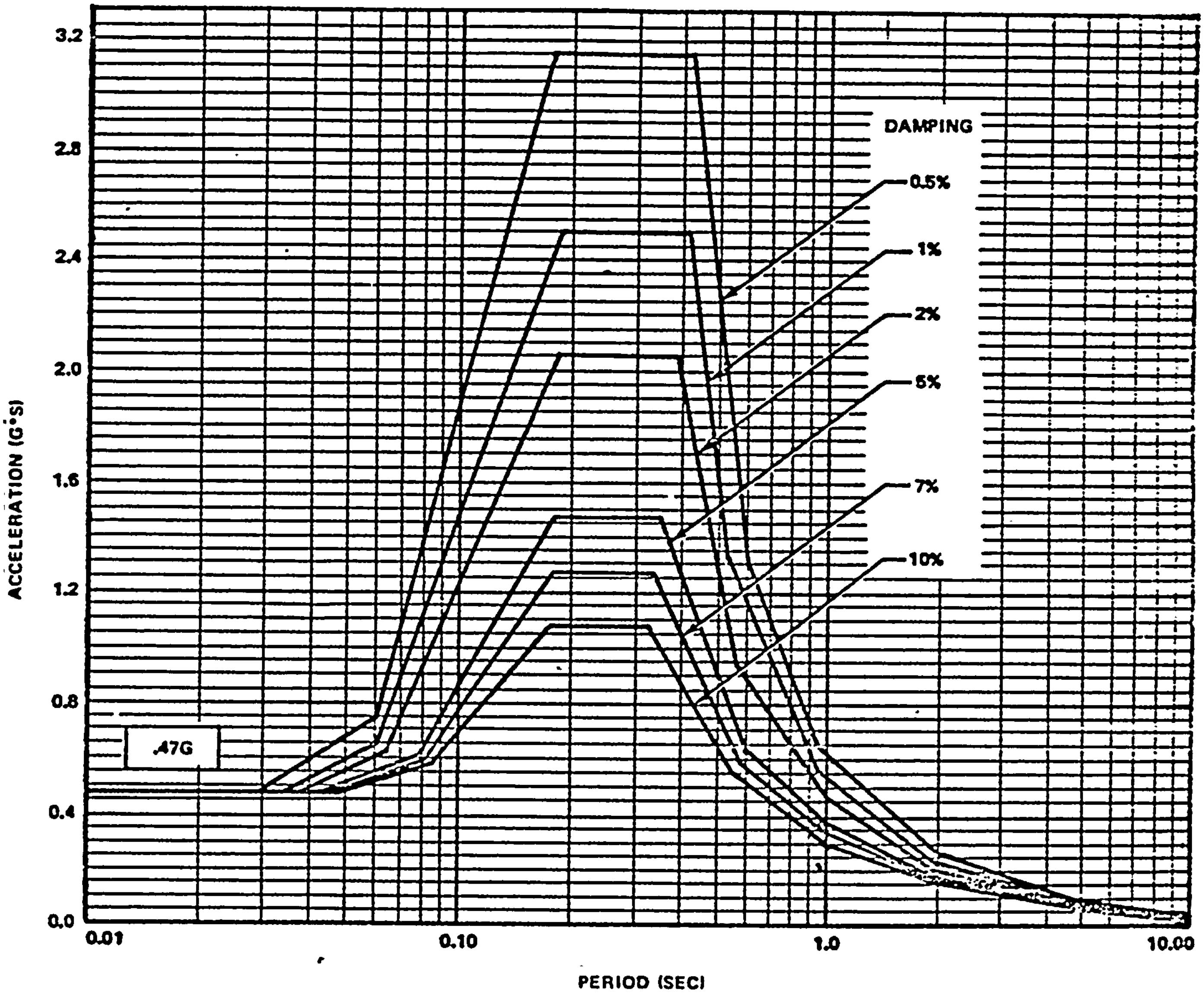
3.8.5.5 Structural Acceptance Criteria

The foundations of Seismic Category I buildings are designed to meet the same structural acceptance criteria as the buildings themselves. These criteria are discussed in sections 3.8.1.5, 3.8.3.5, and 3.8.4.5. The limiting conditions for the foundation medium, together with a comparison of actual capacity and estimated structure loads, are found in sections 2.5.4.10 and 2.5.4.11. Computed factors of safety against overturning, sliding, and flotation for Category I structures are given in table 3.8-5. 1

Table 3.8-5
COMPUTED FACTORS OF SAFETY

Structure	Overturning		Sliding		Flotation
	OBE	SSE	OBE	SSE	
Auxiliary	3200	830	2.2	1.3	4.7
Containment	3400	1200	1.7	1.2	4.5
Control	1500	420	1.6	1.2	4.8
Diesel Generator	1200	400	2.2	1.1	NA[a]
Fuel	1600	400	1.9	1.1	NA
Main Steam Support	340	91	1.6	1.1	NA
Condensate Storage and Refueling Water Tanks	500	150	1.7	1.4	NA

a. Not applicable

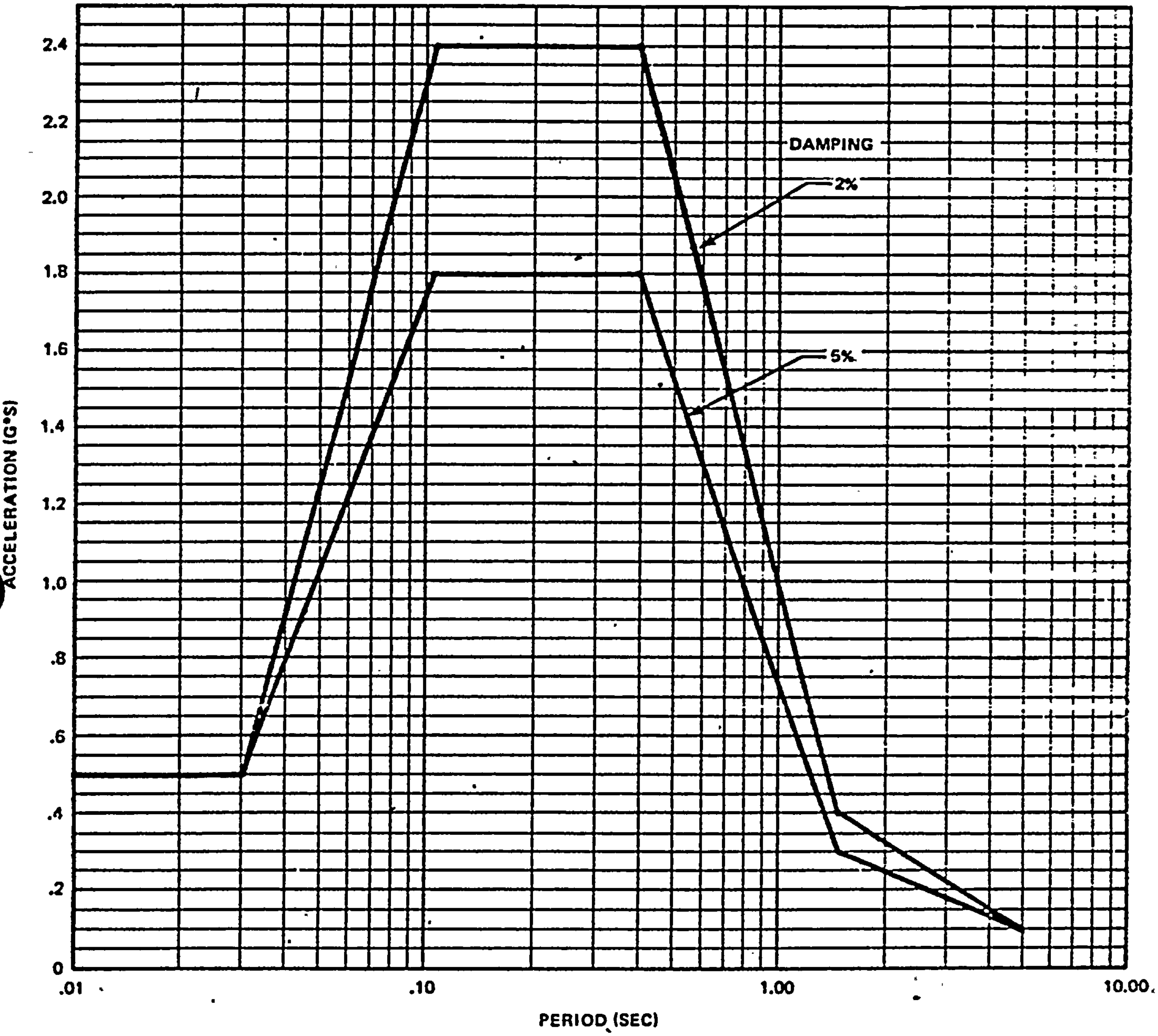


Palo Verde Nuclear Generating Station
FSAR

FUEL BUILDING SSE VERTICAL
ACCELERATION RESPONSE SPECTRA

Figure 3D-31

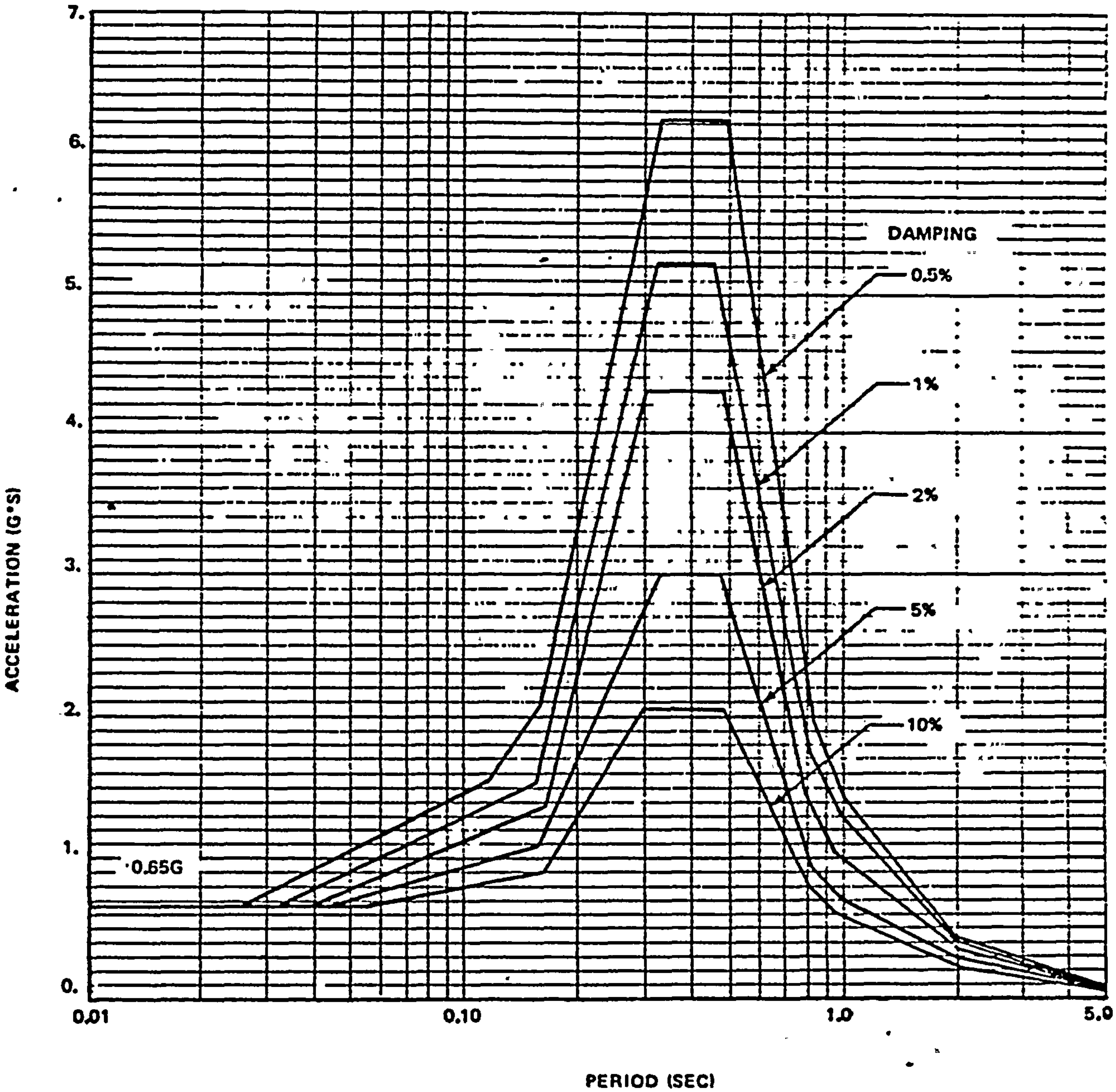
243



Palo Verde Nuclear Generating Station
FSAR

FUEL BUILDING SSE HORIZONTAL
ACCELERATION RESPONSE SPECTRA
EL 105'-0", FUEL TRANSFER TUBE

Figure 3D-32



Palo Verde Nuclear Generating Station
FSAR

FUEL BUILDING SSE HORIZONTAL
(N-S) ACCELERATION RESPONSE SPECTRA
EL 193.67 FT

Figure 3D-33

345

PALO VERDE NUCLEAR STATION UNITS 1, 2, 3
DESIGN ADEQUACY AUDIT

STRUCTURAL AUDIT OF CONTROL BUILDING
Part I - General Analysis

I. BASIC DESIGN CRITERIA

A. 'g' value - free field

	Seismic level based on construction permit license	Seismic level used in design of structures and equipment
SSE	0.20g	0.25g
OBE	0.10g	0.13g

Reference: FSAR, Section 3.7

B. Spectra (attach figs. for all damping values, ductilities)

1. zero period acceleration

SSE 0.25g
OBE 0.13g

Reference: FSAR, Figures 3.7-1 - 3.7-4 (Pages 389, 390, 391, 392) and Section 3.7.1.1
This is consistent with Reg. Guide 1.60

Frequency (or period) interval

Refer to BC-TOP-4A, Section 2.5.1 (c)

C. Damping

Refer to FSAR, Section 3.7.1.3
This is consistent with Reg. Guide 1.61
Refer to FSAR figures 3.7-5 and 3.7-6 (Pages 92A, 92B)

D. Artificial time history and corresponding spectra (attach figures)

1. original time history and its composition, i.e., rising time, strong motion and tail end.

Refer to FSAR, Section 3.7.1.2 and BC-TOP-4A, Section 2.5

2. base line correction, check the integrated velocity and displacement time histories

   The velocity and displacement time histories were checked and found to be satisfactory. In addition, only the acceleration time history was used as input in analysis.

3. time interval - compatible with the highest frequency considered in the spectral calculation

   0.005 seconds

E. Motion duration

24 seconds

Refer to FSAR, Section 3.7.1.2

F. components of motion including their relative motion amplitudes

Analysis was performed for the three principle directions with equal amplitudes.

G. Dead and live loads for various operating floors and base slab

~~Refer to Project Design Criteria Part II, Section 3.0 and Part III, Section 4.0~~

Dead load - includes all structures, major equipment load and 50 psf equivalent for small equipment.

Live load - see action item 3.

H. Ground water level

The groundwater design level is at plant El 70′-0". The actual water level is at approximately plant El. 60′-0".

Reference: FSAR, Sections 2.4.13.2.4 and 2.4.13.5.

I. Backfill earth pressure, wind, overpressure due to postulated external explosions (as applicable)

~~Refer to Project Design Criteria Part II, Sections 3.4.5.3 and 3.5.5 for lateral earth pressure, wind and tornado loads.~~
For lateral earthpressure see pages 208 A & 208 B, 208C, 208D
Wind does not govern.

J. Other considerations

All penetrations in roof and exterior walls that expose safety related equipment have tornado missile protection. Concrete covers or steel plate shields to prevent missile perforation are provided.

II. ANALYSIS METHOD

A. Seismic Analysis

1. Mathematical model-general description with sketch.

   Two planar lumped parameter models were used.
   Refer to FSAR Section 3.7.2.3.3 and Figure 3.7-12

   a. (1) concrete modulus

   $E_c = 3.64 \times 10^6$ psi for f'c = 4000 psi

   $E_c = 4.07 \times 10^6$ psi for f'c = 5000 psi

   Reference: ACI 318-71

   (2) rebar modulus and yield strength

   $E = 29 \times 10^3$ ksi

   Fy = 60 ksi

   (3) Poisson's ratio

   $\gamma = 0.17$ for concrete

   Reference: "Theory of Plates and Shells" second edition by Timoshenko, Woinowsky-Kreiger page 97.

   (4) damping

   Refer to FSAR Section 3.7.1.3 ~~and response to NRC Question 220.2 (Q3A-4)~~.
   this is consistent with Reg. Guide 1.61

(5) properties of foundation materials

- shear modulus

  Refer to FSAR, Figures 3.7-7, -8, -9.

- subgrade reactions

  ~~Refer to Project Design Criteria Part II, Section 3.4.5.4 for coefficient of subgrade reaction.~~
  coefficient of subgrade reaction = 40-60 kips/ft²/ft

- bearing capabilities

  Refer to Tables 2.5-15 and 2.5-16 of the FSAR.

(6) other parameters

b. stiffness calculations

(1) exterior walls and interior walls

Stiffness calculations were performed manually using standard engineering methods.

(2) interior walls

Same as exterior walls except for concrete masonry walls (fire barrier walls) and plaster walls which were neglected.

2. method of Analysis

a. method of analysis used (time history, response spectrum methods, etc.) and consideration of torsional and translational response

Time history analyses were performed to generate in-structure response spectra using SAP 1.9. Modal response spectrum analyses were performed to obtain the seismic loads for design of structural elements using SAP 1.9.

(1) general description

The Control Building analysis used two planar lumped parameter models. For a horizontal earthquake the model consisted of one vertical cantilever beam, having 6 nodes, representing each floor level. For a vertical earthquake the model consisted of two vertical branches on a common rigid mat. The first branch, representing the exterior walls at each floor level, has 6 nodes. The second branch, representing the interior stuctural steel columns between each level, also has 6 nodes. Attached to each of these 6 nodes was a single degree of freedom subsystem, which represents the floor structural steel framing system at each level.

Reference: FSAR, Figure 3.7-12 (Pg.396)

The soil structure Interaction analysis method was used and compared against current NRC review position and this meets the intent of NRC soil structure interaction position

(2) findings and comments

b. selection of number of masses and degrees of freedom

(1) general description

For horizontal direction earthquake, the model consists of 6 nodes with 12 degrees of freedom.

For vertical direction earthquake, the model consists of 22 nodes with 16 degrees of freedom.

(2) findings and comments

c. number of modes considered

		SSE	OBE
Horizontal	E-W	3 modes f = 23.1 cps	3 modes f = 23.6 cps
Horizontal	N-S	3 modes f = 20.6 cps	3 modes f = 21.1 cps
Vertical		3 modes f = 11.8 cps	3 modes f = 11.7 cps

Frequencies shown above are for highest mode considered.

(1) general description

Refer to FSAR, Table 3.7-5 for modal frequencies and participation factors. (Pg. 388)

(2) findings and comments

d. combining modal responses

(1) actual procedures used

Refer to FSAR, Section 3.7.2.7

(2) general findings

e. consideration of three components of motion

(1) actual procedures used

Refer to FSAR, Section 3.7.2.6.
this is consistent with Reg. Guide 1.92

(2) general findings

f. consideration of soil-structure interaction

Soil Springs

<u>OBE</u>	$Kx = 2.01 \times 10^6$ K/ft (East-West)	$Kxx = 5.32 \times 10^9$ K-ft/rad
	$Ky = 2.63 \times 10^6$ K/ft (Vertical)	
	$Kz = 2.01 \times 10^6$ K/ft (North-South)	$Kzz = 7.06 \times 10^9$ K-ft/rad
<u>SSE</u>	$Kx = 1.54 \times 10^6$ K/ft (East-West)	$Kxx = 4.09 \times 10^9$ K-ft/rad
	$Ky = 2.02 \times 10^6$ K/ft (Vertical)	
	$Kz = 1.54 \times 10^6$ K/ft (North-South)	$Kzz = 5.42 \times 10^9$ K-ft/rad

(1) general description

Refer to FSAR, Section 3.7.2.4

(2) findings and comments

g. decoupling criteria for subsystems

(1) general procedure

Refer to BC-TOP-4A, Section 3.2.

(2) key examples

(3) The other criteria pertaining to frequency ratio as defined in S.R.P. 3.7.3.II.3.b are also met.

(3) general findings and comments

h. modeling of hydrodynamic effects in spent fuel pool

Not applicable. The spent fuel pool is in Fuel Handling Building.

i. modeling of spent fuel pool wells and interior floor slabs and equipment thereof

Not applicable.

3. development of in-structure response spectra

Refer to FSAR, Section 3.7.2.5

a. general procedures

The "SPECTRA" program was used to compute the response spectra. Refer to FSAR, Appendix 3B for a description and applications of this program.

(1) smoothing (describe specific smoothing method used)

The smoothened response spectra represent an envelope of the maximum peaks.

(2) peak widening

± 15%

Reference: FSAR, Section 3.7.2.9.

b. typical results (attach figures)

Refer to FSAR, Figures 3D-4, -5, -6. (Pages 402,3,4)

B. Stress Analysis

1. shear walls and floors

a. mathematical model - general description w/sketch

Shear wall and floor stresses were computed by performing standard manual calculations. Vertical loads were distributed in the structure by conventional methods. Lateral loads were calculated by multiplying lumped mass and seismic acceleration obtained from floor spectra at each level. These lateral loads were then distributed among the shear walls according to their relative stiffness and location.

b. method of analysis--incorporation of torsion

The center of mass and center of rigidity of the Control Building coincided. Therefore, torsion was not considered.

c. load combinations

Refer to FSAR, Section 3.8.3.3

1. foundation mat

· Finite element model (SAP 1.9)

a. mathematical model - description of boundary conditions

A three-dimensional finite element model was used in the basemat analysis. Equivalent soil springs were attached to each node.

b. method of analysis

Dead and live loads on the exterior walls and columns were calculated based on tributary floor areas. Seismic loads were obtained from the response spectrum analysis. (Three-dimensional earthquake was considered by using the component factor method). These loads were then applied as nodal point loads to obtain the moment and shear forces in the foundation mat.

c. load combinations

Refer to FSAR Section 3.8.3.3.

this is in compliance with the SRP

d. key results (figures, etc.)

Refer to FSAR Table 3.8-4H. (Pg. 399)

3. Material to protect against structure - structure interaction

Below grade (EL 100'-0") Rodofoam II (WR Grace and Co.) or EVERLASTIC Micro II (Williams Products, Inc.) may be left in place in seismic joints between structures. Above grade gaps between walls were left open, typically 6" between Category I structures.

a. mechanical properties

b. additional pressure on walls

c. findings and comments

4. vertical dynamic analysis

a. mathematical model - general description with sketch

The planar lumped parameter model consisted of two vertical branches founded on a common rigid mat. The first of the two branches represents the exterior walls. The second branch represents the steel columns. A single degree of freedom subsystem was attached at each of the column node points. It represents the local effects created by vibrating floor beams framing into the steel columns.

Refer to FSAR, Figure 3.7-12.

b. development of stiffnesses, including floor stiffness, as applicable

Stiffnesses were calculated manually for walls, columns and floors.

c. method of analysis

The model described above was used for acceleration time-history and modal response spectrum analyses. The above analyses were performed by using the SAP 1.9 computer program.

C. Computer Programs Used in Analysis

SAP 1.9, SPECTRA,
Refer to FSAR, Appendix 3B.

1. assumptions and limitations

Refer to FSAR, Appendix 3B.

2. applicability

Refer to FSAR, Appendix 3B.

3. verification

- sensitivity study in case of numerical solutions (e.g., finite element analysis)

Refer to FSAR, Appendix 3B.

4. load input (include all cases)

Program	Input
SAP 1.9	Finite element model (nodes and elements) loading (pressure, nodal loads), response spectra, time history.
SPECTRA	In-Structure Time histories, frequency or periods and damping value.

5. output (include all cases)

   Refer to FSAR, Appendix 3B.

6. other discussions

D. Overall Stability

1. forces and moments from seismic analysis

   Refer to FSAR, Figure 3.7-17, -18. (Pages 397, 8)

2. various cases considered

   Seismic event loading combinations considered SSE or OBE applied in the North-South, East-West, and vertical directions simultaneously.

3. bearing pressure versus bearing capability and safety factor against bearing failure

   Refer to FSAR, Section 2.5.4.10 and Tables 2.5-15, -16. (Pages 385, 6)

4. factors of safety

   Refer to FSAR, Section 3.8.5.5 and Table 3.8-5. (pg. 401)

   a. sliding

      Factor of Safety = 1.2 (SSE)

   b. overturning

      Factor of Safety = 420 (SSE)

E. Interaction of Non-category I Structures with the Structure Considered

1. identification of pertinent non-Category I structures

The Corridor Building and Radwaste Building are adjacent to the Control Building.

2. consideration given to potential failure of non-Category I systems on Category I systems

The ~~Corridor and~~ Radwaste Buildings ~~were~~ was designed to preclude structural failure of building or parts thereof that could damage the Control Building and its Category I systems. Within the Control Building non-Category I systems which potentially could affect Category I systems were designed for structural integrity under SSE loads.

Reference: FSAR, Section 3.8.4.4.

3. general findings and comments

During a walk-down those items whose failure will not affect any safety related equipment are left as they are. If they are judged that they might affect Category I systoms, they are designed to maintain their structural integrity under an SSE.

F. Design Consideration for Tornado Missiles

1. design requirements

   Refer to FSAR, Table 3.5-8. (Pg. 387)

2. models for

   a. local damage

      Refer to FSAR, Section 3.5.3.

   b. overall response

      Refer to FSAR, Section 3.5.3.

3. load combinations

   Refer to FSAR, Section 3.8.3.3.

4. forces

   The governing structural members are of sufficient thickness to preclude perforation by postulated missiles and maintain structural integrity.

6. general comments and preliminary audit findings

III. CONFORMANCE TO ACCEPTABLE CRITERIA

A. Identification of deviations, if any

None

B. Justification of deviations and disposition of the deviations

D. general comments

Part II Audit of Key Designs

A. Exterior Shear Walls

1. design requirements

The exterior walls were designed to satisfy structural function as bearing walls, shear walls, and protection against tornado missiles.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4H. (Pg. 399)

4. detailed design of rebar placement at key sections

See Attachment A. (Pg. 384)

5. general comments and preliminary audit findings

B. Interior Shear Walls

Not Applicable

1. design requirements

~~The interior walls were designed to satisfy structural function as bearing walls and shear walls.~~

2. design loads (from general analysis)

~~Same as exterior walls except that plaster and concrete masonry walls (fire barrier walls) were neglected for the strength of the building but were designed to resist the loads due to their own weight (Dead Load plus Seismic) in order to preclude damage to safety related equipment.~~

3. forces and moments at key sections

~~Refer to FSAR, Table 3.8-4H.~~

4. detailed design of rebar placement at key sections

~~See Attachment A.~~

5. general comments and preliminary audit findings

C. Main Floors and Roofs

1. design requirements

Main floors and roofs were primarily designed for vertical dead, live and seismic loads. ~~as defined in the Project Design Criteria.~~ Roofs were also designed to satisfy minimum thickness to preclude perforation by tornado generated missiles.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4H. (Pg. 399)

4. detailed design of rebar placement at key sections

See Attachment A. (Pg. 384)

5. general comments and preliminary audit findings

D. Steel Structural Bracing Systems (if any)

1. design requirements

The Control Building floors and roof slab are supported on structural steel beams, girders, and columns which primarily are designed for dead, live, and seismic loads. ~~per the Project Design Criteria.~~

2. design loads

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-41. (Pg. 400)

6. general comments and preliminary audit findings

E. Foundation Mat

1. design requirements

   Refer to FSAR, Sections 3.8.5.4.2 and 3.8.5.5.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4H (Pg. 399)

4. detailed design of rebar placement at key sections

   See Attachment A. (Pg. 384)

5. general comments and preliminary audit findings

F. Main Frame Concrete Column Design (Key Columns)

1. design requirements

   Not applicable. There are no concrete columns in this building.

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design of rebar placement at key sections

5. general comments and preliminary audit findings

G. Secondary Floors

1. design requirements

   Not applicable.

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design of rebar placement at key sections

5. general comments and preliminary audit findings

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H. Detailing at Floor-Wall Joints

1. design requirements

   As per ACI 318-71 Code, Chapter 6 and 17.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4H. (Pg. 399)

4. detailed design of rebar placement at key sections

   See Attachment A. (Pg. 384)

5. general comments and preliminary audit findings

I. Dynamic Effects Applied to Floors and Walls by Machinery

Dynamic effects from machinery are negligible. Major pumps are located on the basemat. HVAC units are mounted with isolation pads.

1. design requirements

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design

5. general comments and preliminary audit findings

J. Crane & Support

Not applicable. There are no cranes located in this building.

1. design of bents (columns and roof trusses)

   a. design requirements

   b. design loads (from general analysis)

   c. forces and moments at key sections

   d. detailed design

e. general comments and preliminary audit findings

2. design of girders supporting crane rails

   Not applicable.

   a. design requirements

   b. design loads (from general analysis)

   c. forces and moments at key sections

   d. detailed design

e. general comments and preliminary audit findings

3. design of spent fuel bridge

   Not applicable.

   a. design requirements

   b. design loads (from general analysis)

   c. forces and moments at key sections

   d. detailed design

   e. general comments and preliminary audit findings

K. Fuel Pool Liner Design

Not applicable. The spent fuel liner is located in the Fuel Handling Building.

1. stresses and strain controls

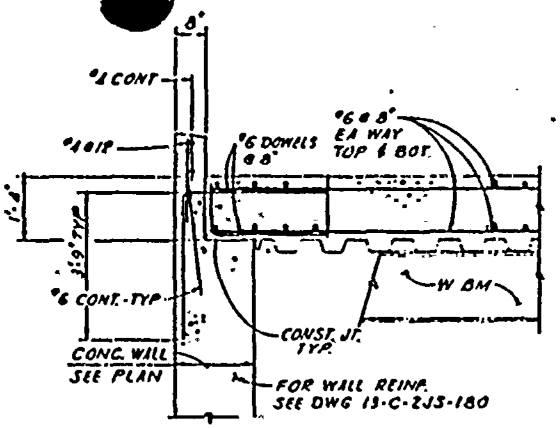
2. conformance to code requirements

3. analysis procedure and results

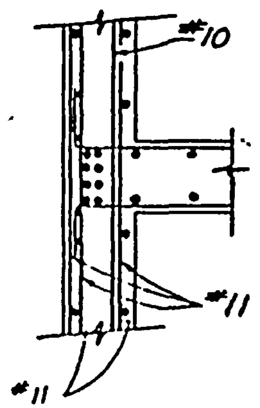
4. consideration of accidental drop of crane loads

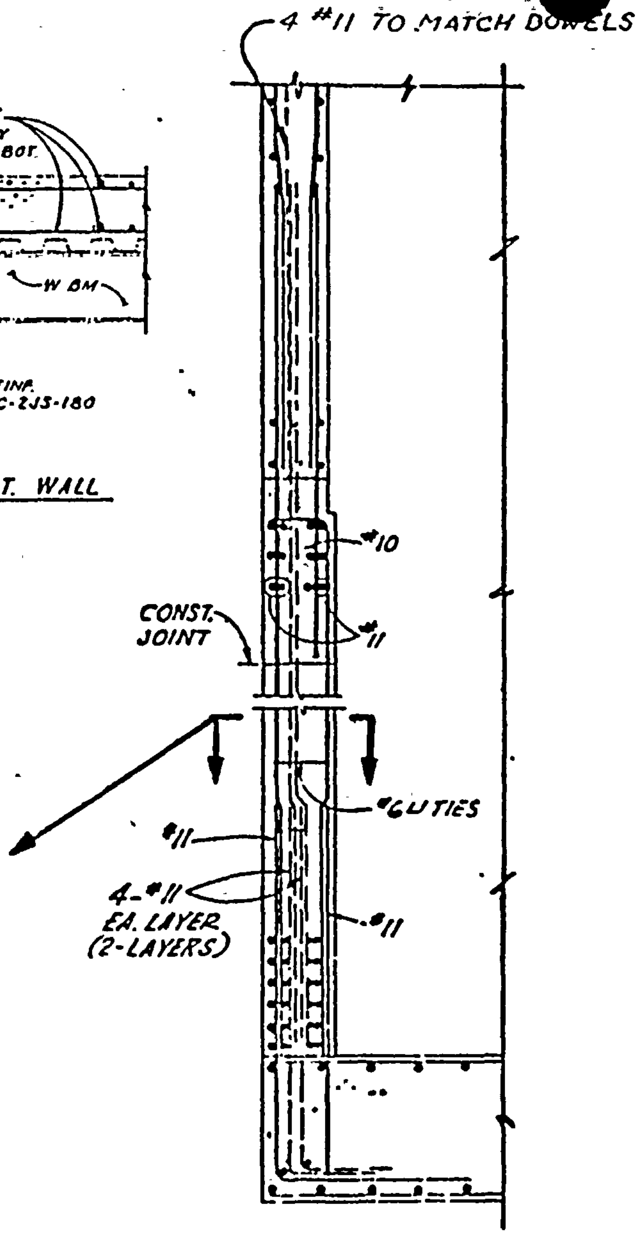
5. corrosion effects (e.g., pitting) on liner integrity

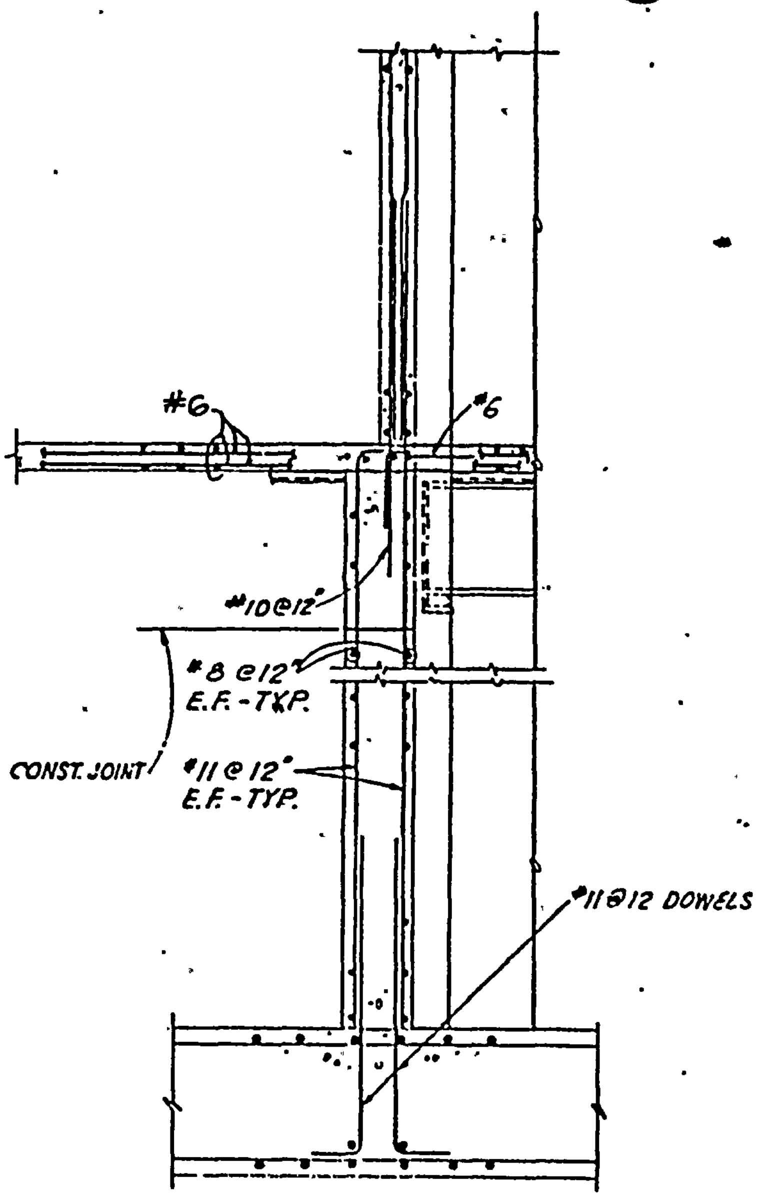
6. preliminary findings of audit results



SLAB PARALLEL TO EXT. WALL







ATTACHMENT "A"

CONTROL BUILDING
TYP. WALL SECTIONS AND DETAILS
(NO SCALE)

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Table 2.5-15

STATIC BEARING CAPACITY OF CATEGORY I STRUCTURES

Structure	Average Static Design Load q_s (k/ft^2)	Ultimate Bearing Capacity q_o (k/ft^2)	Factor of Safety (q_o/q_s)
Containment Building	7.9	35.7	4.5
Auxiliary Building (deep section)	6.2	34.9	5.6
Main Steam Support Structure	7.1	64.8	9.1
Control Building	3.3	45.3	13.7
Fuel Building	5.3	54.9	10.4
Diesel Generator Building	3.1	79.5	25.6
Refueling Water Tank	4.4	90.4	20.5
Condensate Storage Tank	3.5	112.4	32.1

Table 2.5-16

DYNAMIC BEARING CAPACITY OF CATEGORY I STRUCTURES[a]

Structure	Equivalent Uniform Vertical Stress q_d (k/ft^2)	Ultimate Bearing Capacity q_o (k/ft^2)	Factor of Safety (q_o/q_d)
Containment Building	16.1	32.2	2.0
Auxiliary Building (deep section)	10.3	25.8	2.5
Main Steam Support Structure	25.3	60.6	2.4
Control Building	9.8	39.8	4.1
Fuel Building	19.1	50.3	2.6
Diesel Generator Building	5.6	75.5	13.5
Refueling Water Tank	13.2	58.7	4.4
Condensate Storage Tank[b]	13.2	30.2	2.3

a. Based upon maximum dynamic loads derived from analyses described in section 3.7.

b. Condensate storage tank loads were conservatively chosen to be equal to the dynamic design load for the refueling water tank. Actual loads will be less.

1

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Table 3.5-8

TORNADO-GENERATED MISSILES CONSIDERED IN DESIGN OF SAFE SHUTDOWN STRUCTURES

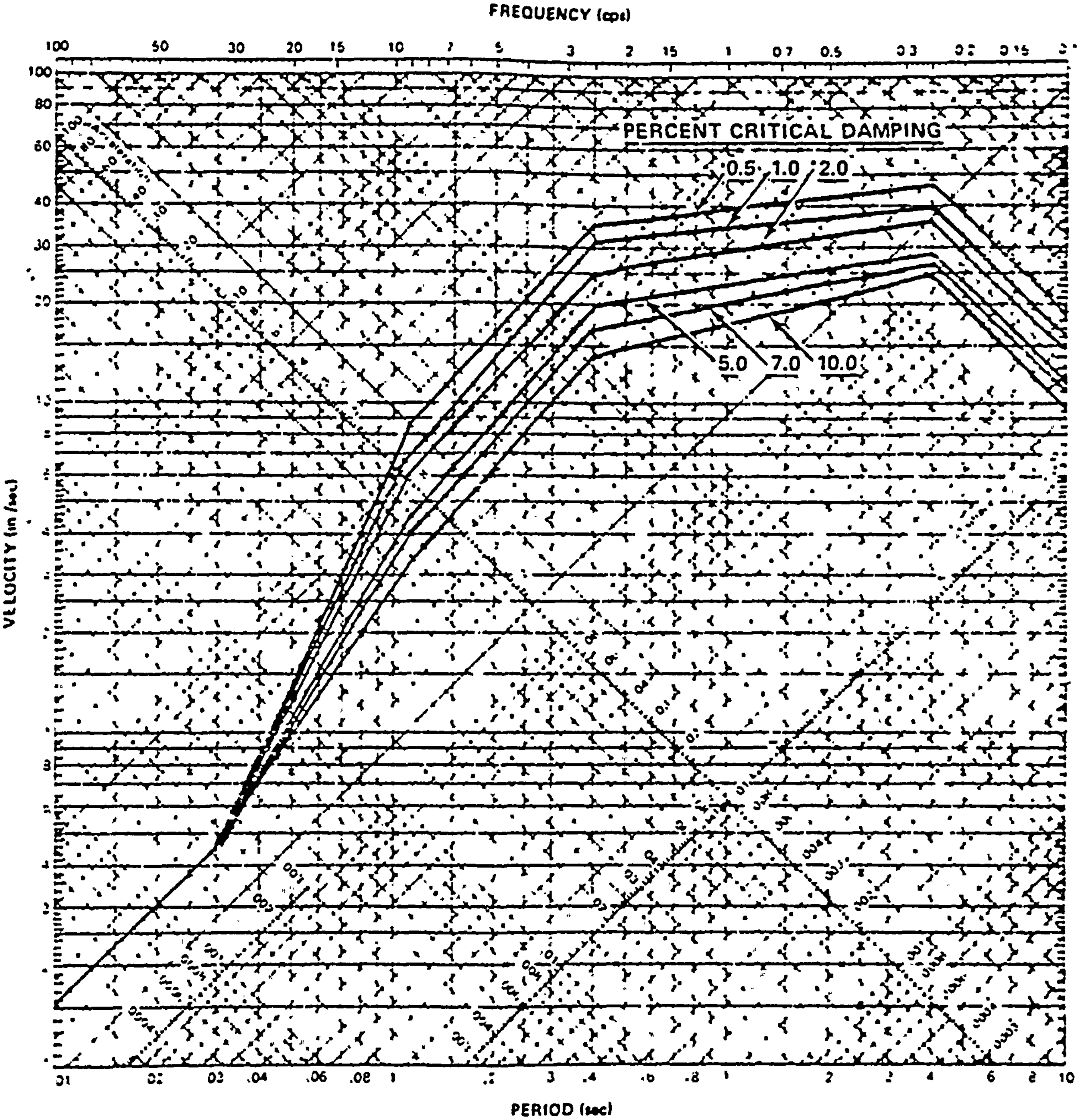
Description	Weight (lbs)	Impact Area (ft^2)	Maximum Velocity (ft/s)	Kinetic Energy (ft-lbs)
(A) A 12-foot wood plank, 4 x 12 inches in cross-section, traveling end on at a speed of 240 mi/h.	200	0.333	352	3.85×10^5
(B) A steel pipe, Schedule 40, 3 inches in diameter by 10 feet long, traveling end on at 120 mi/h.	78	0.063	176	3.75×10^4
(C) A steel rod, 1 inch in diameter, 3 feet long, traveling end on at 180 mi/h.	8	0.005	264	8.66×10^3
(D) A steel pipe, Schedule 40, 6 inches in diameter by 15 feet long, traveling end on at 120 mi/h.	285	0.24	176	1.37×10^5
(E) A steel pipe, Schedule 40, 12 inches in diameter by 15 feet long, traveling end on at 120 mi/h.	743	0.886	176	3.57×10^5
(F) A utility pole, 13-1/2 inches in diameter, 35 feet long, traveling end on at 120 mi/h.	1490	0.994	176	7.17×10^5
(G) An automobile of 4,000 pounds weight, striking the structure at 60 mi/h.	4000	20.0	88	4.81×10^5

Table 3.7-5

CONTROL BUILDING NATURAL FREQUENCIES[a]

		Mode	Frequency (Hz)
Horizontal (N-S)	OBE	1	4.0
		2	11.6
		3	21.1
	SSE	1	3.7
		2	10.4
		3	20.6
Horizontal (E-W)	OBE	1	4.4
		2	12.2
		3	23.6
	SSE	1	4.0
		2	10.9
		3	23.1
Vertical	OBE	1	7.1
		2	10.3
		3	11.7
	SSE	1	6.5
		2	10.0
		3	11.8

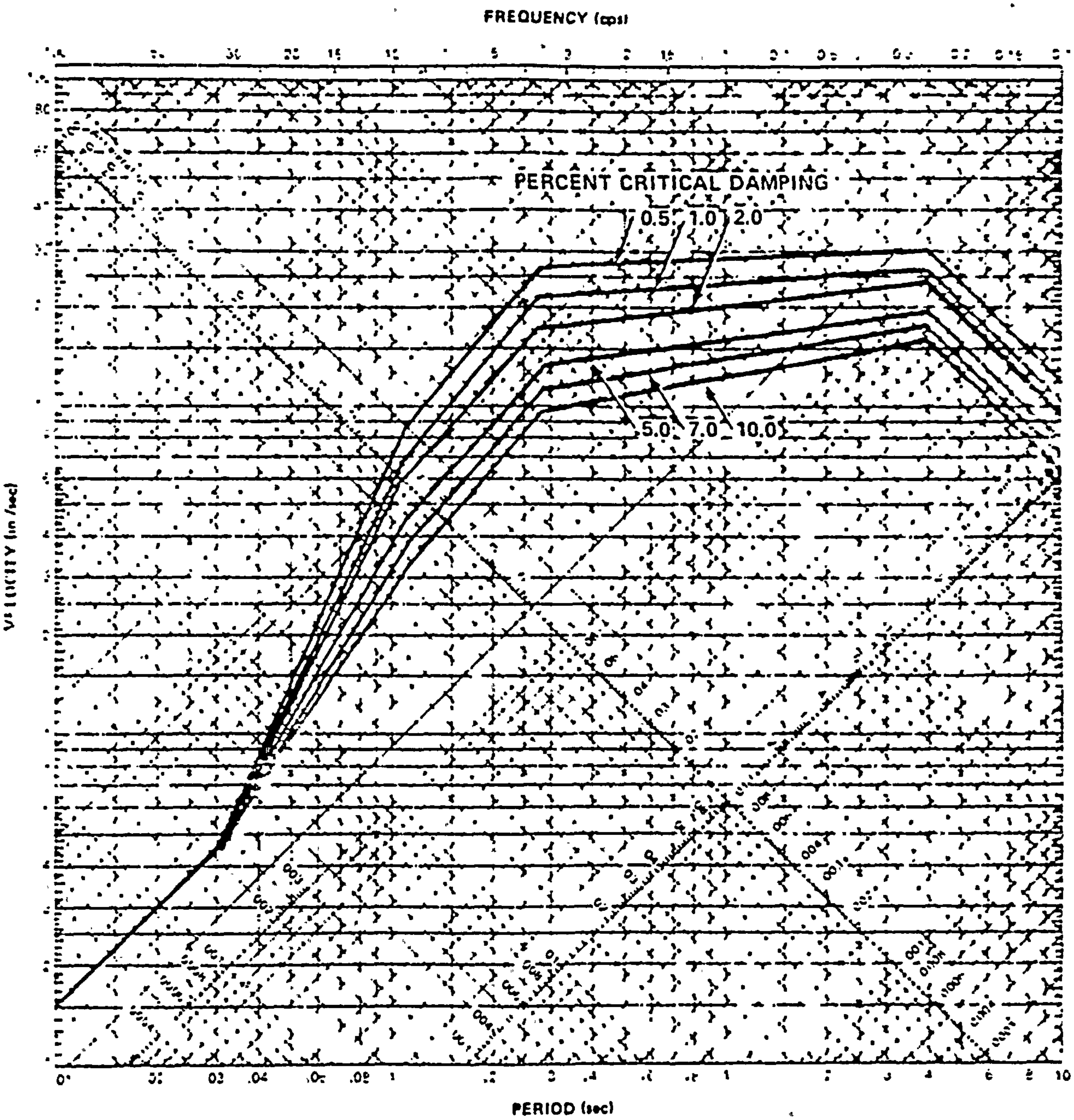
4 a. See figure 3.7-22 for mode shapes and participation factors.



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR SSE 0.25 g

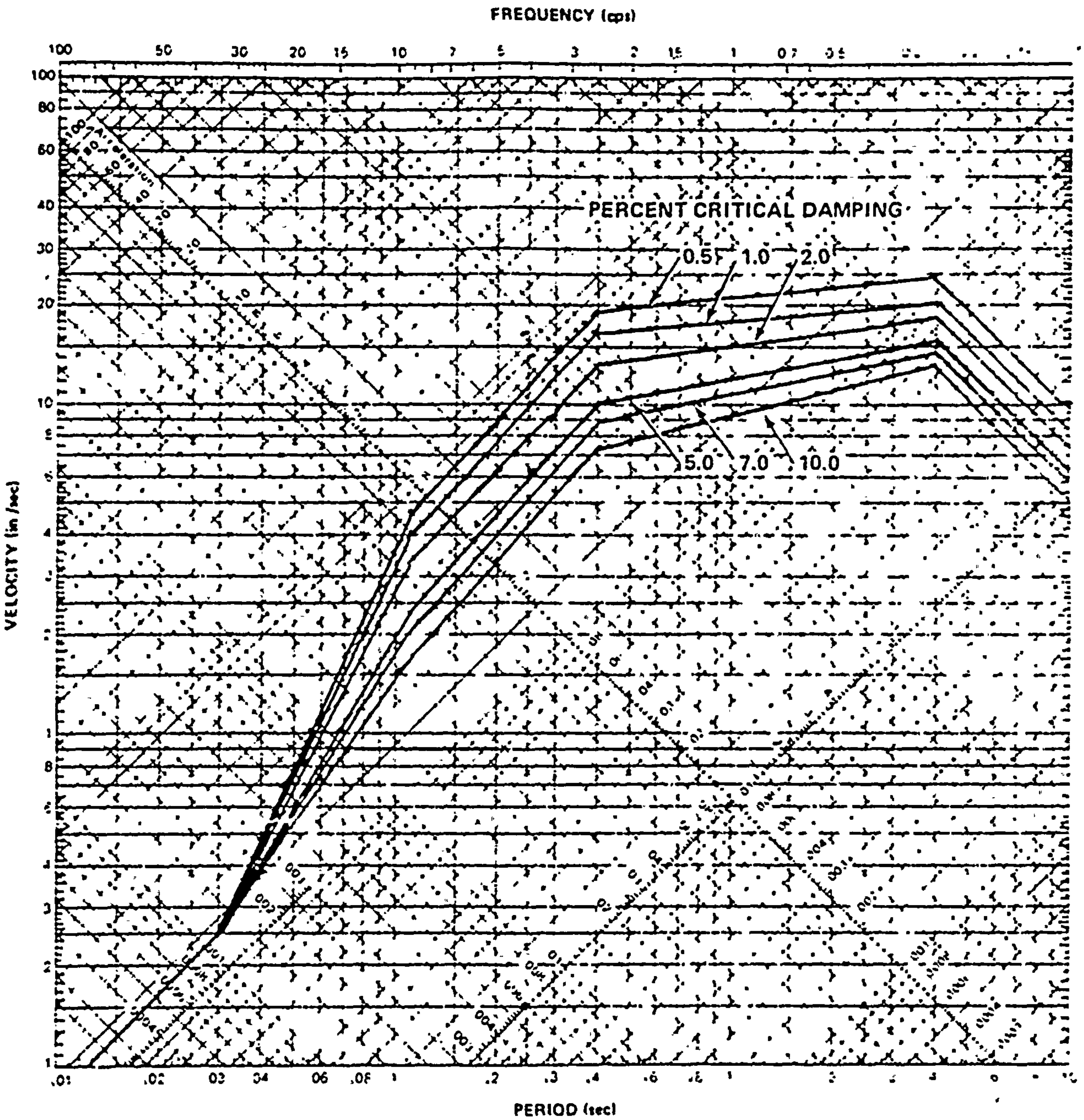
Figure 3.7-1



Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR SSE 0.25 g

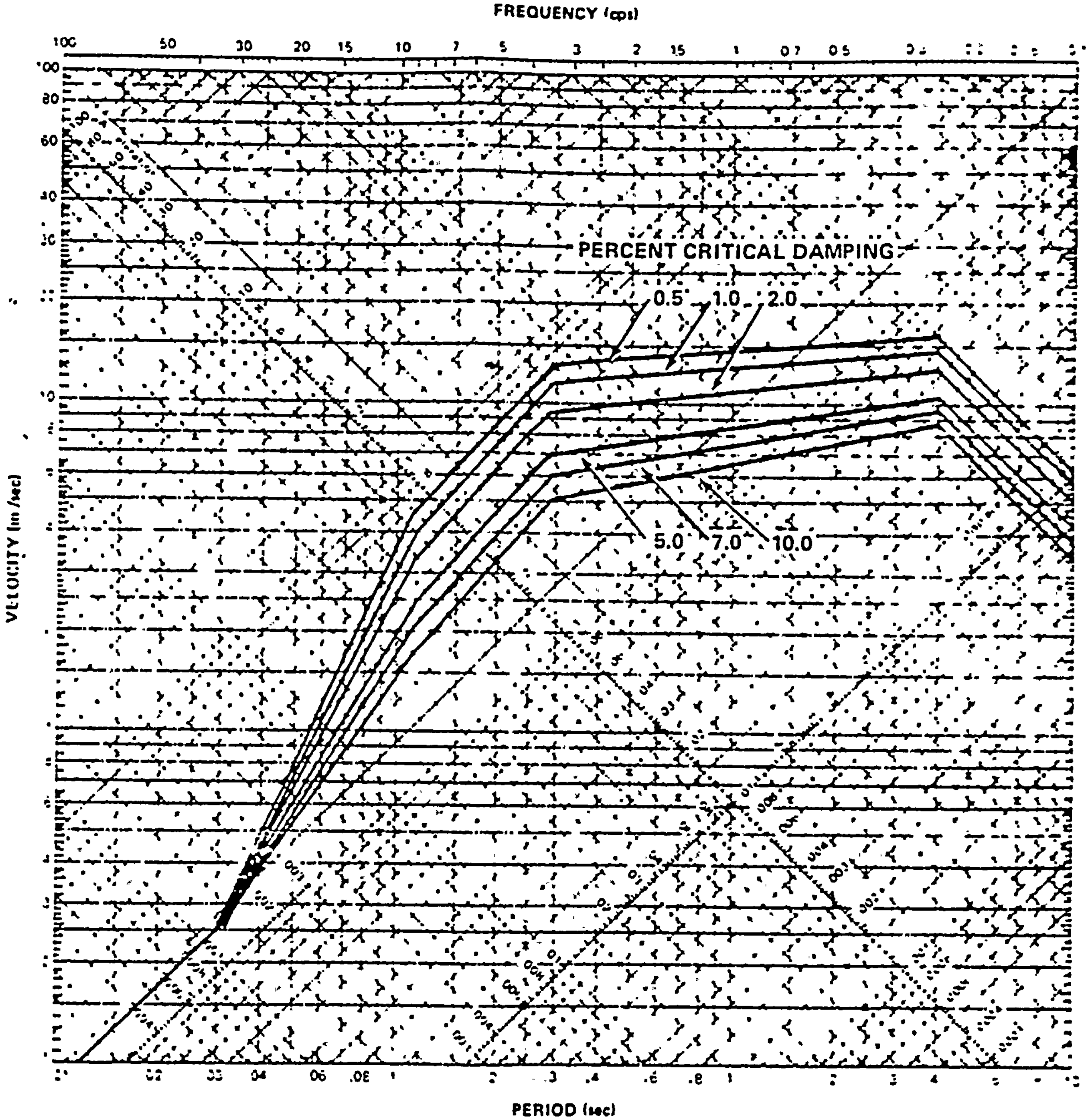
Figure 3.7-2



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR OBE 0.13 g

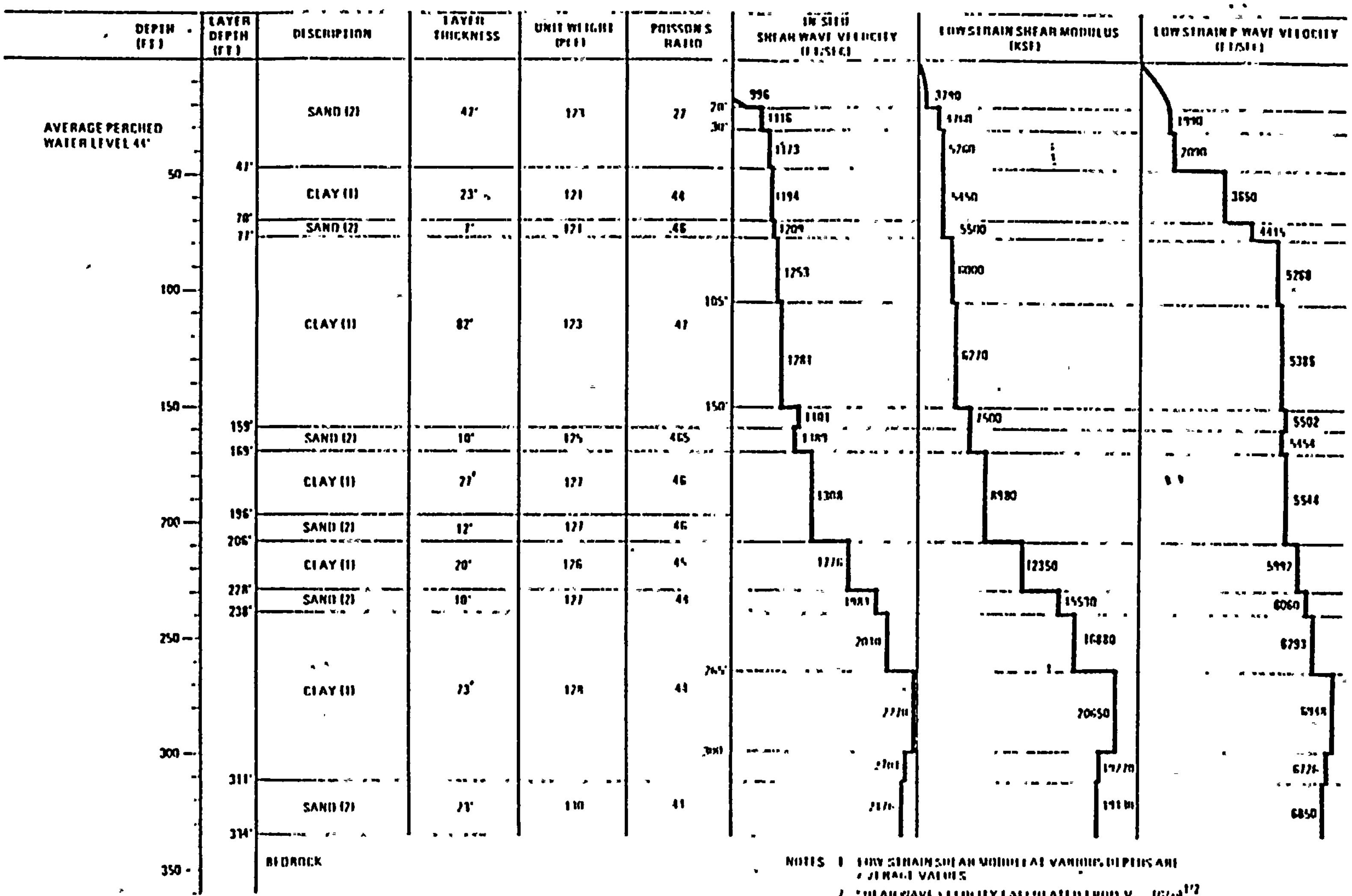
Figure 3.7-3.



Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-4



NOTES 1. LOW STRAIN SHEAR MODULI AT VARIOUS DEPTHS ARE AVERAGE VALUES

2. SHEAR WAVE VELOCITY CALCULATED FROM V_s ...

3. P WAVE VELOCITY CALCULATED FROM ...

Palo Verde Nuclear Generating Station FSAR

SOIL PROFILE

FIGURE [illegible]

CHANGE

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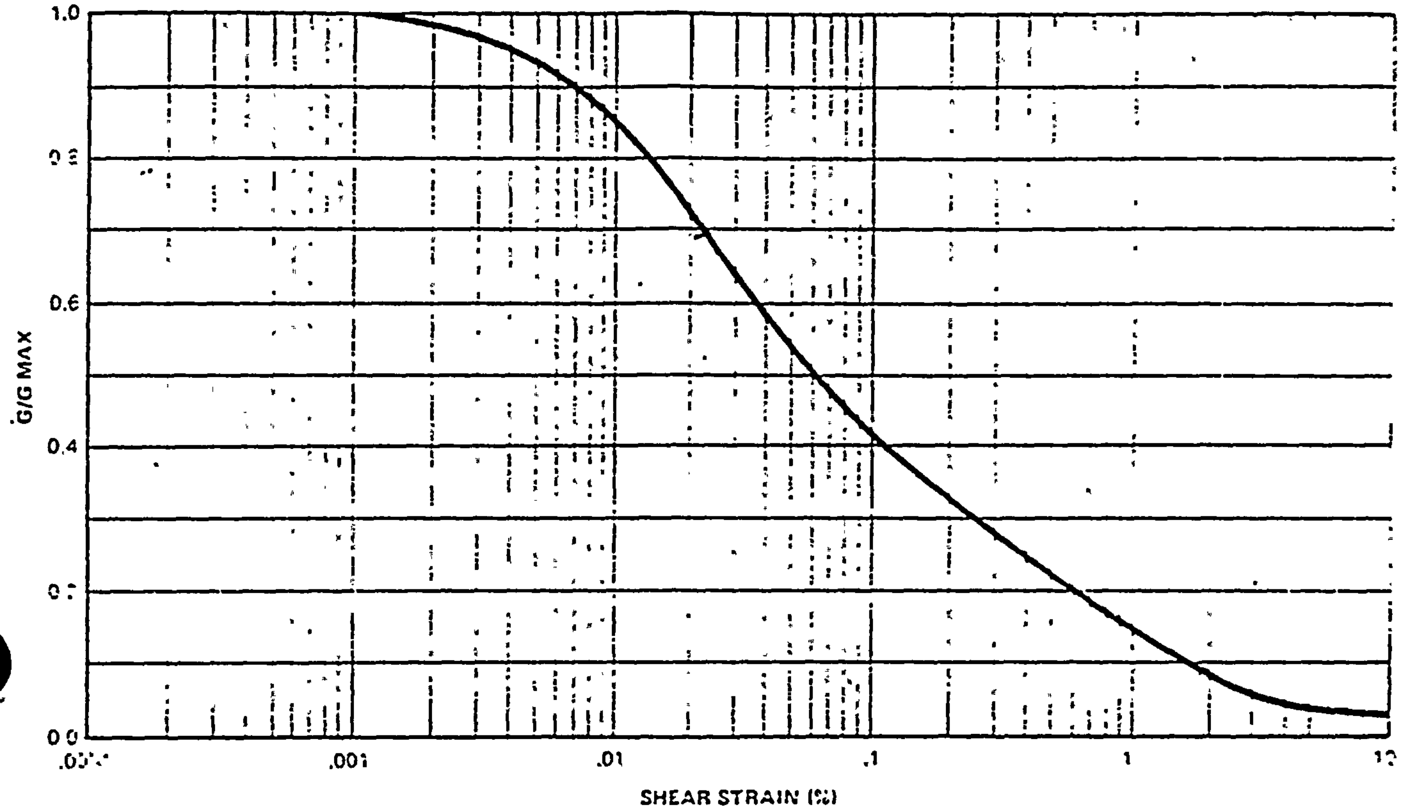
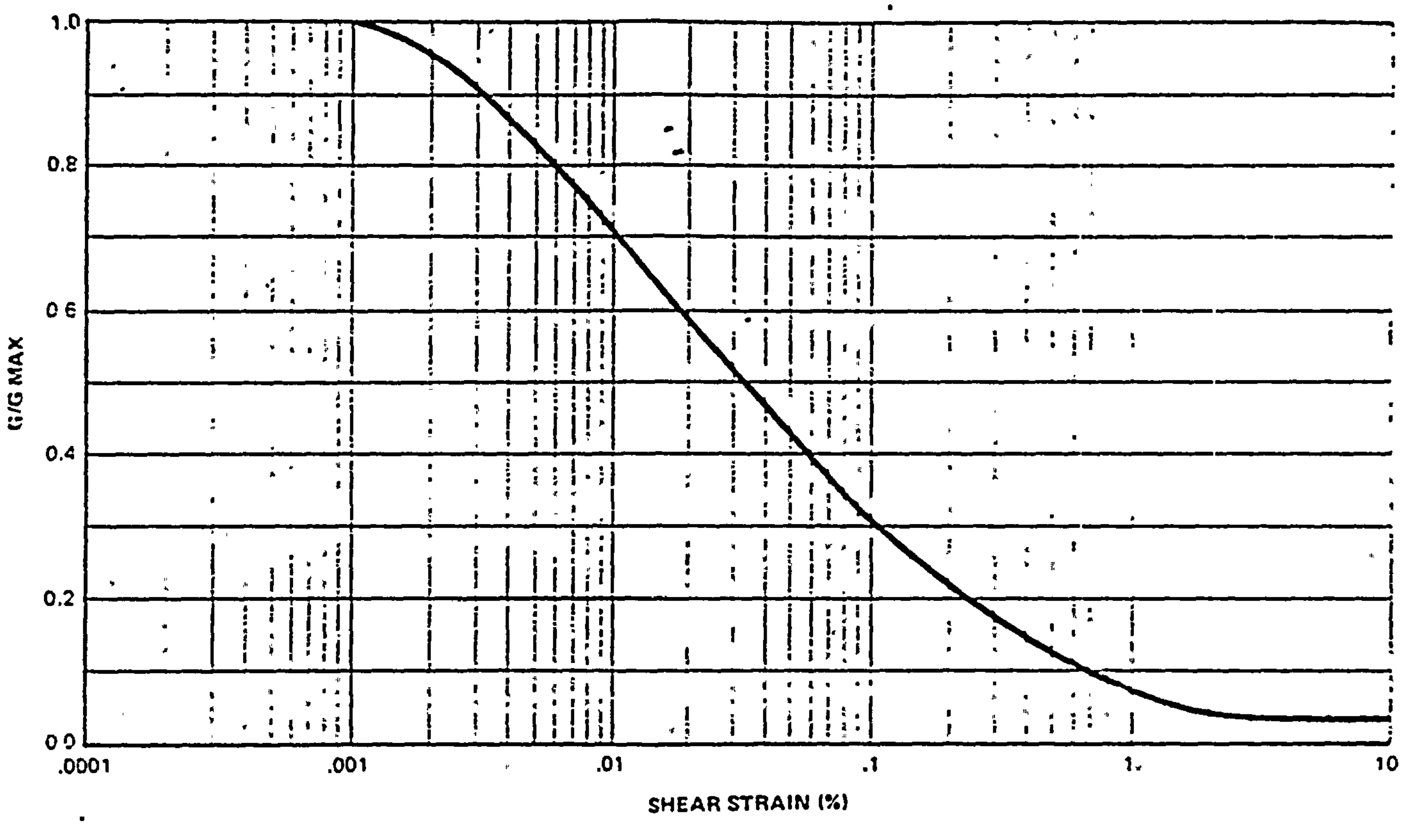


Palo Verde Nuclear Generating Station
FSAR

SHEAR MODULUS VS. STRAIN - CLAY

FIGURE 3.7-8



Palo Verde Nuclear Generating Station
FSAR

SHEAR MODULUS VS. STRAIN - SAND

FIGURE 3.7-9

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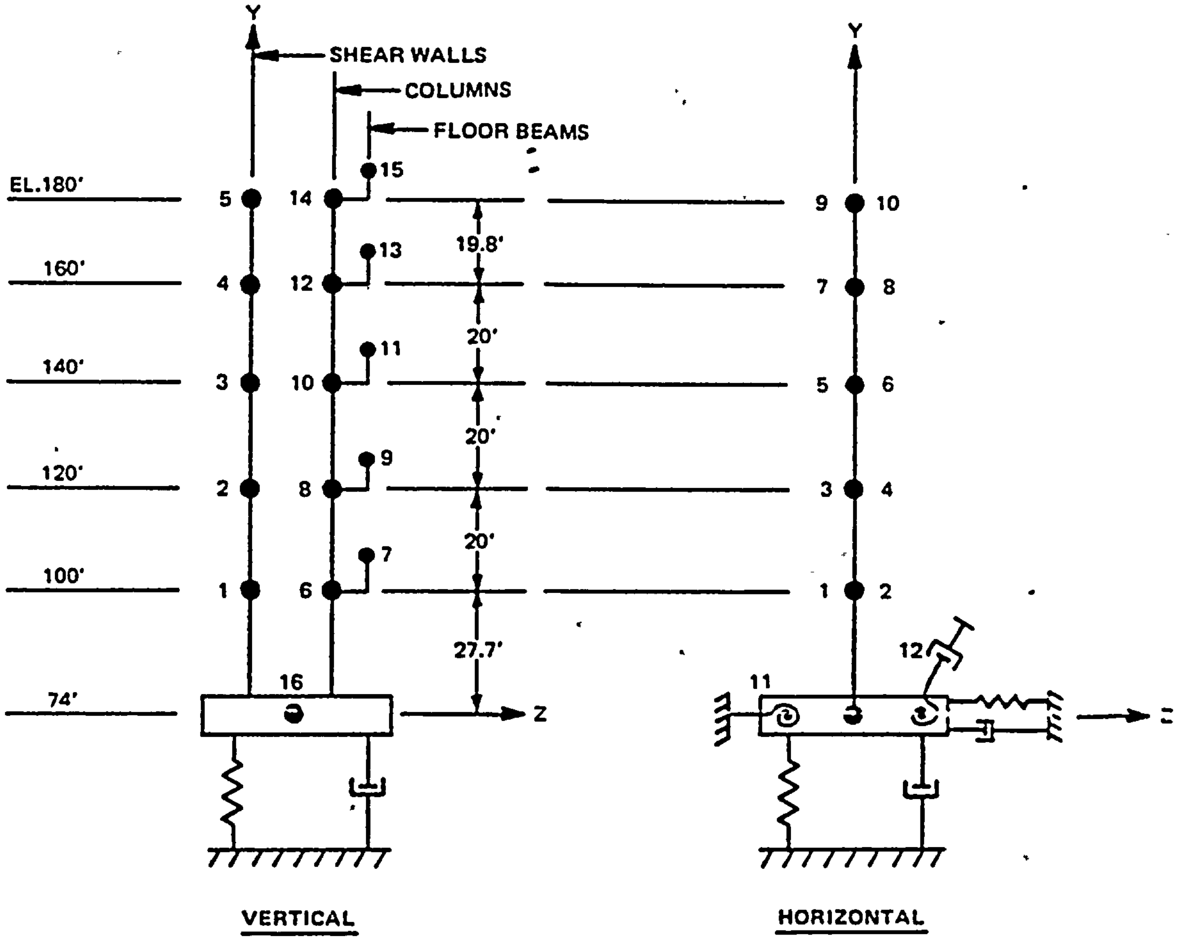

Palo Verde Nuclear Generating Station
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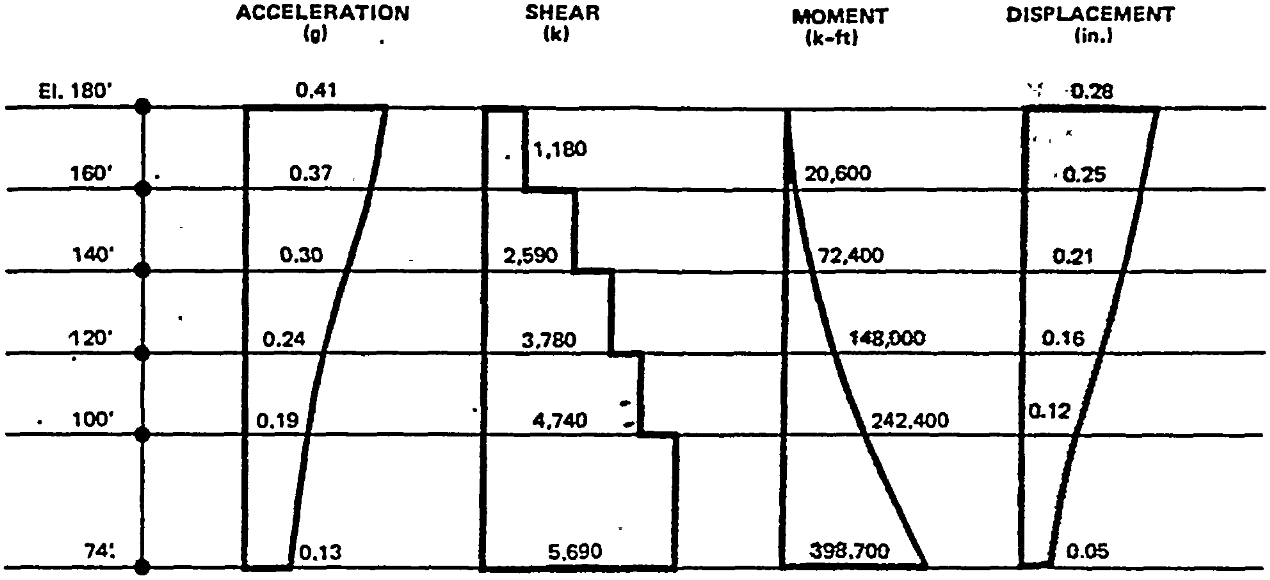
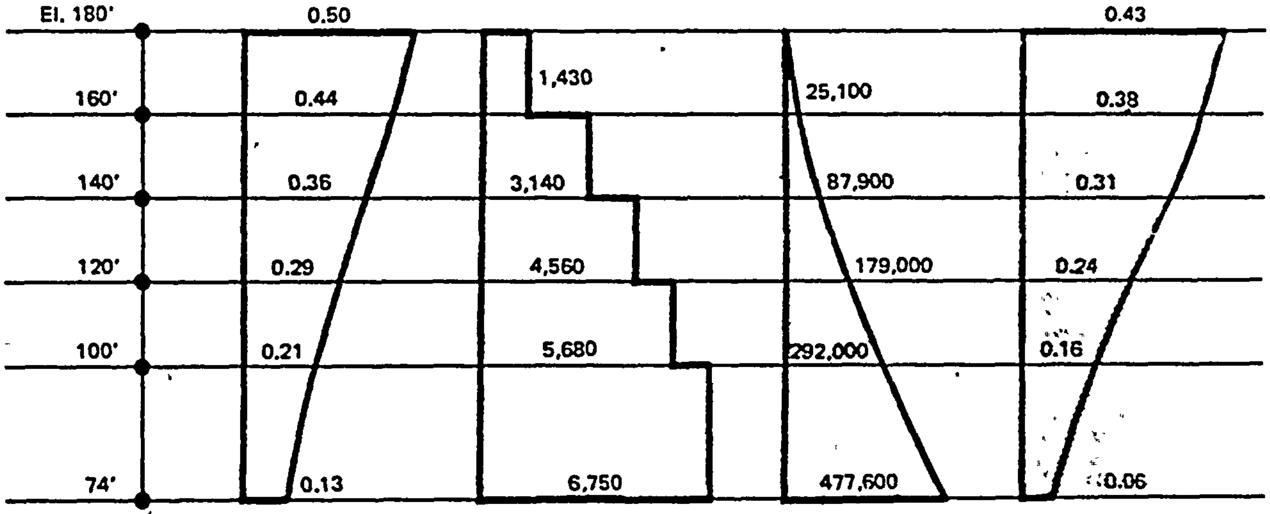
CONTROL BUILDING
LUMPED MASS MODEL

FIGURE 3.7-12



HORIZONTAL E-W



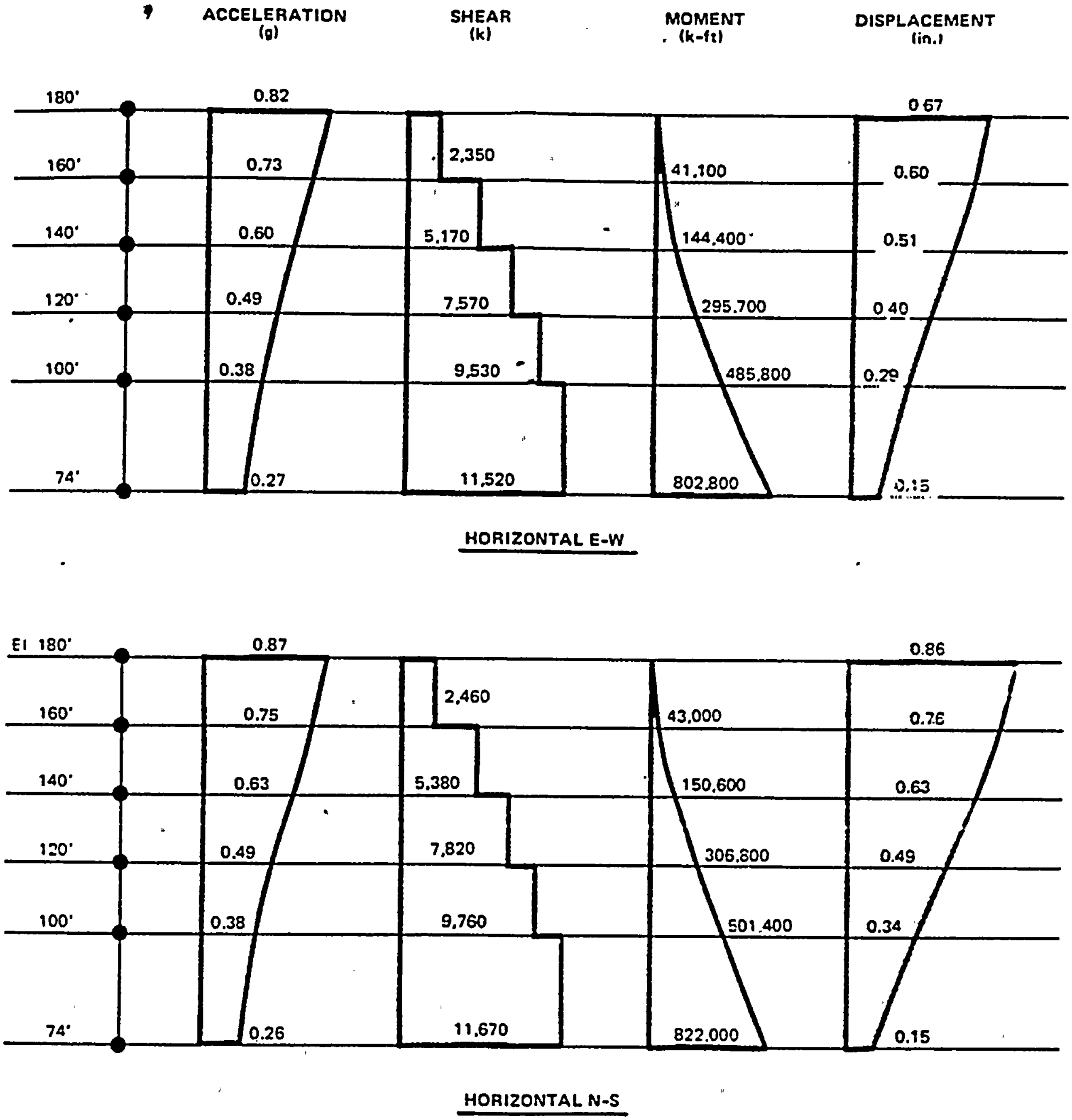
HORIZONTAL N-S

CHANGE

Palo Verde Nuclear Generating Station
FSAR

CONTROL BUILDING
DESIGN RESPONSE (OBE)

FIGURE 3.7-17.



Palo Verde Nuclear Generating Station
FSAR

CONTROL BUILDING
DESIGN RESPONSE (SSE)

FIGURE 3.7-18

CHANGE

Table 3.8-41

CONTROL BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS

Description of Principal Member	Location of Principal Member	Governing Load Combination Number[a]	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u)[b][c]	Calculated Shear Load (V_u)[b]	Maximum Shear Capacity (V_u)[b]
			P_u[b]	M_u[c]			
2'-0" thick wall - vertical reinforcement	Exterior west wall @ El. 74'-0"	2	-73	415	437	-[d]	-[d]
1'-0" thick wall - vertical reinforcement	Interior wall @ El. 74'-0"	2	+118.5	37	45.4	-	-
1'-9" thick wall - vertical reinforcement	Exterior west wall @ El. 100'-0"	2	+71.2	233	284	-	-
1'-9" thick wall - vertical reinforcement	Exterior south wall @ El. 120'-0"	2	+53	61	118	-	-
1'-0" thick slab - E-W reinforcement	El. 100'-0"	2	-[d]	74	90	-	-
8" thick slab - E-W reinforcement	El. 120'-0"	2	-	9	11	-	-
4'-0" thick basemat	El. 74'-0"	2	-	1,189	1,395	-	-
4'-0" thick basemat	El. 74'-0"	2	-	899	1,132	-	-

a. Refer to Section 3.8.3.3.2.A.(2) for description of load combination number.
b. P_u and V_u are in kips; Sign convention for P_u: Compression (-), Tension (+).
c. M_u is in ft-k/ft.
d. Negligible.

3

Table 3.8-4I

CONTROL BUILDING SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[a]	Combined Stress Ratio ($\le$1.0)
W 27 X 84	Floor Beam @ El. 100'-0"	2	0.85
W 36 X 300	Main Girder @ El. 100'-0"	2	0.52
W 24 X 55	Floor Beam @ El. 100'-0"	2	0.82
W 14 X 550	Column @ El. 74'-0"	2	0.55
W 27 X 84	Floor Beam @ El. 120'-0"	2	0.91
W 36 X 300	Main Girder @ El. 120'-0"	2	0.59
W 14 X 314	Column @ El. 140'-0"	2	0.34
W 12 X 35	Staircase Beam	2	0.76

a. Refer to section 3.8.3.3.3.A(1) for description of load combination number. 3

400

3.8.5.5 Structural Acceptance Criteria

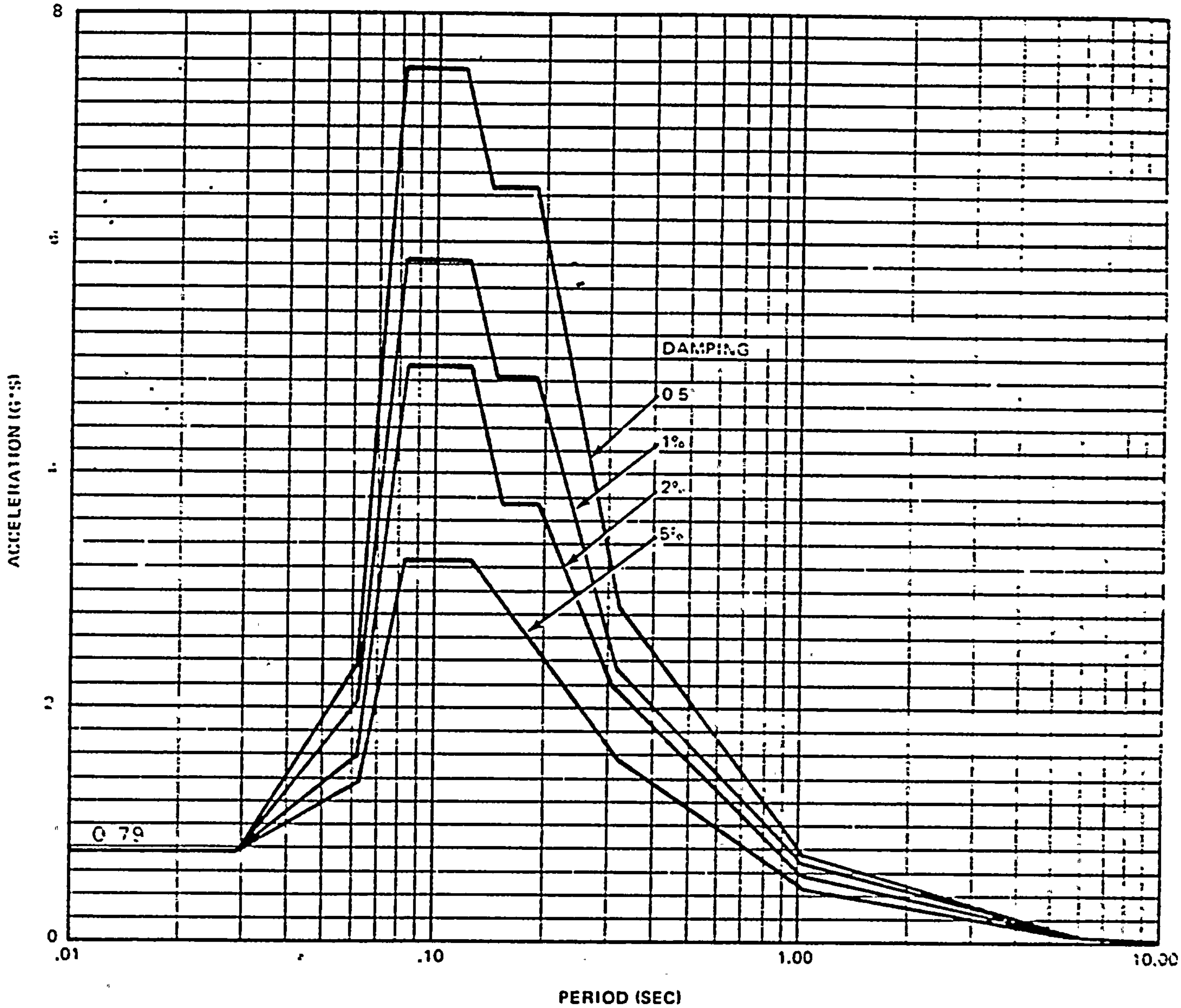
The foundations of Seismic Category I buildings are designed to meet the same structural acceptance criteria as the buildings themselves. These criteria are discussed in sections 3.8.1.5, 3.8.3.5, and 3.8.4.5. The limiting conditions for the foundation medium, together with a comparison of actual capacity and estimated structure loads, are found in sections 2.5.4.10 and 2.5.4.11. Computed factors of safety against overturning, sliding, and flotation for Category I structures are given in table 3.8-5.

Table 3.8-5
COMPUTED FACTORS OF SAFETY

Structure	Overturning		Sliding		Flotation
	OBE	SSE	OBE	SSE	
Auxiliary	3200	830	2.2	1.3	4.7
Containment	3400	1200	1.7	1.2	4.5
Control	1500	420	1.6	1.2	4.3
Diesel Generator	1200	400	2.2	1.1	NA[a]
Fuel	1600	400	1.9	1.1	NA
Main Steam Support	340	91	1.6	1.1	NA
Condensate Storage and Refueling Water Tanks	500	150	1.7	1.4	NA

a. Not applicable

401

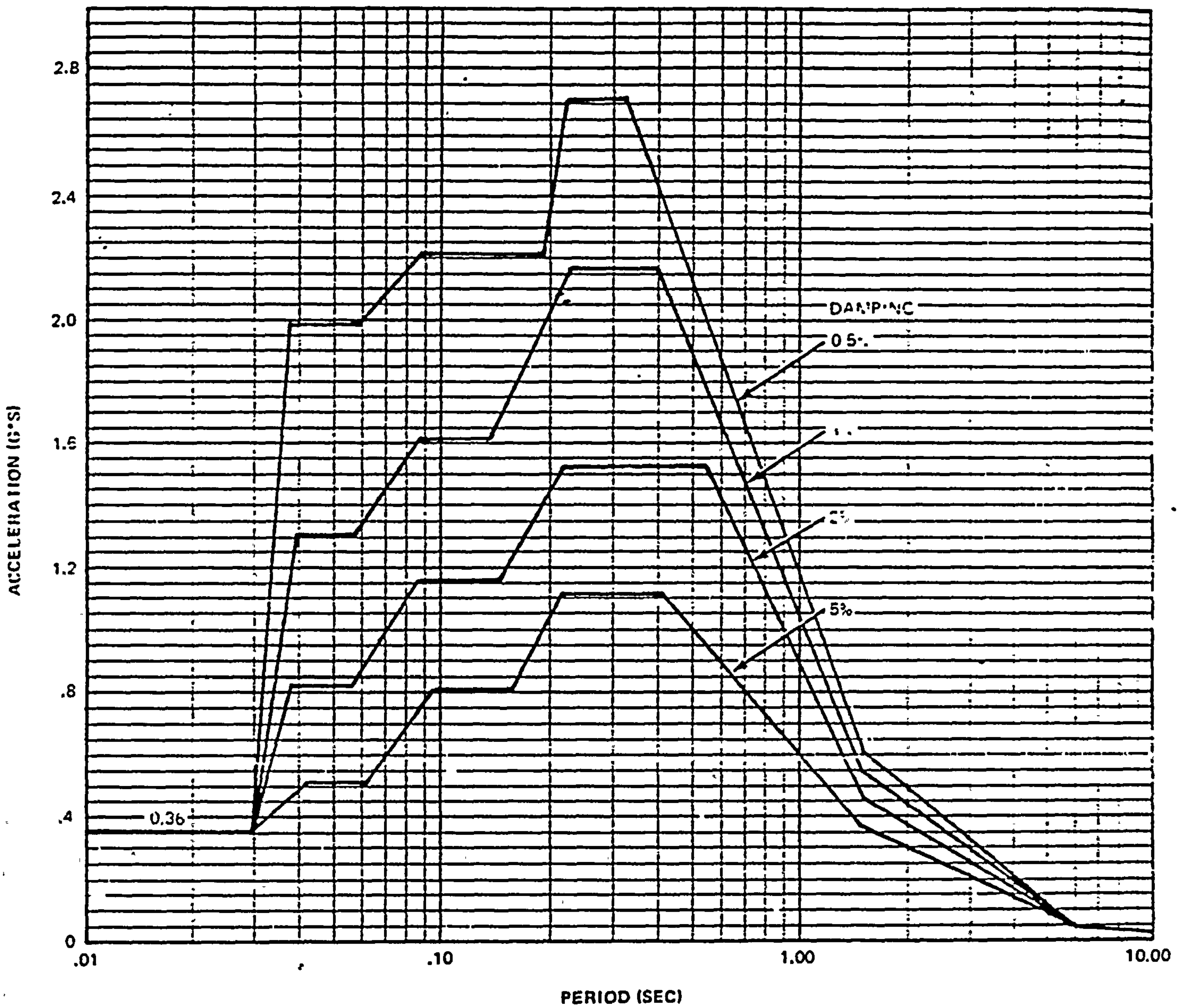


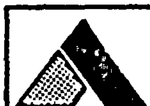
Palo Verde Nuclear Generating Station
FSAR

CONTROL BUILDING SSE VERTICAL
ACCELERATION RESPONSE SPECTRA
EL 140'-0"

Figure 3D-4

402

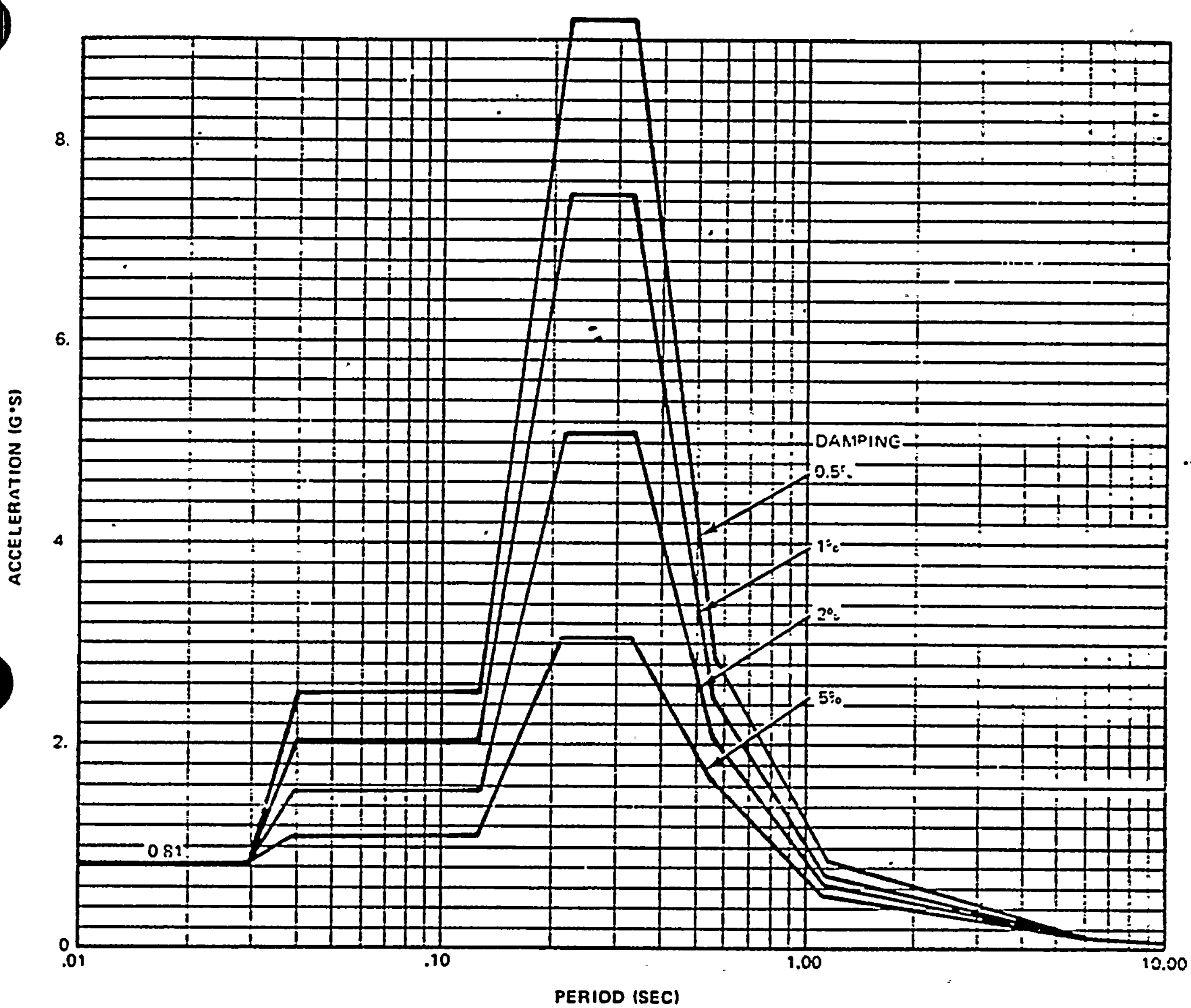




Palo Verde Nuclear Generating Station
FSAR

CONTROL BUILDING SSE HORIZONTAL
ACCELERATION RESPONSE SPECTRA
EL 74'-0", BASE MAT

Figure 3D-5



Palo Verde Nuclear Generating Station
FSAR

CONTROL BUILDING SSE HORIZONTAL
ACCELERATION RESPONSE SPECTRA
EL 180'-0", ROOF

Figure 3D-6

PALO VERDE NUCLEAR STATION UNITS 1, 2, 3
DESIGN ADEQUACY AUDIT

STRUCTURAL AUDIT OF DIESEL GENERATOR BUILDING
Part I - General Analysis

I. BASIC DESIGN CRITERIA

A. 'g' value - free field

	Seismic level based on construction permit license	Seismic level used in design of structures and equipment
SSE	0.20g	0.25g
OBE	0.10g	0.13g

Reference: FSAR, Section 3.7

B. Spectra (attach figs. for all damping values, ductilities)

1. zero period acceleration

SSE 0.25g
OBE 0.13g

Reference: FSAR, Figures 3.7-1 - 3.7-4 (Pages 451, 2, 3, 4) and Section 3.7.1.1
This is consistent with Reg Guide 1.60

Frequency (or period) interval

Refer to BC-TOP-4A, Section 2.5.1 (c)

C. Damping

Refer to FSAR, Section 3.7.1.3
This is consistent with Reg. Guide 1.61
Refer to FSAR figures 3.7-5 and 3.7-6 (Pages 92A, 92B)

D. Artificial time history and corresponding spectra (attach figures)

1. original time history and its composition, i.e., rising time, strong motion and tail end.

Refer to FSAR, Section 3.7.1.2 and BC-TOP-4A, Section 2.5

2. base-line correction, check the integrated velocity and displacement time histories

   The velocity and displacement time histories were checked and found to be satisfactory. In addition, only the acceleration time history was used as input in analysis.

3. time interval - compatible with the highest frequency considered in the spectral calculation

   0.005 seconds

E. Motion duration

24 seconds

Refer to FSAR, Section 3.7.1.2

F. components of motion including their relative motion amplitudes

Analysis was performed for the three principle directions with equal amplitudes.

G. Dead and live loads for various operating floors and base slab

~~Refer to the Project Design Criteria Part II, Sections 3.5.1 and 3.5.2, and Part III Sections 1.4.1.1 and 1.4.1.2.~~

Dead load - includes all structures, major equipment load and 50 psf equivalent for small equipment.

Live load - see action item 3.

H. Ground water level

The groundwater design level is at plant El 70′-0". The actual water level is at approximately plant EL 60′-0".

Reference: FSAR, Sections 2.4.13.2.4 and 2.4.13.5.

I. Backfill earth pressure, wind, overpressure due to postulated external explosions (as applicable)

~~Refer to Project Design Criteria Part II Sections 3.4.5.2 and 3.7.1 for lateral earth pressure, wind, and tornado loads.~~
For lateral earth pressure see pages 208 A & 208 B, 208 C, 208 D.
Wind does not govern.

J. Other considerations

All the penetrations in the roof and the exterior walls have tornado missile protection. Concrete slabs, hatches, and panels are provided to prevent missile penetration.

II. ANALYSIS METHOD

A. Seismic Analysis

1. Mathematical model-general description with sketch.

Two planar lumped parameter models were used.

See Attachment A.

a. (1) concrete modulus

$E = 3.64 \times 10^6$ psi for $f'c = 4000$ psi

$E = 4.07 \times 10^6$ psi for $f'c = 5000$ psi

Reference: ACI 318-71

(2) rebar modulus and yield strength

$E = 29 \times 10^3$ KSI

$Fy = 60$ KSI

(3) Poisson's ratio

$\gamma = 0.24$

~~Reference: Project Design Criteria, Part II, Section 3.5.7.~~

(4) damping

Refer to FSAR, Section 3.7.1.3 ~~and response to NRC Question 220.2 (Q3A-4).~~

This is consistent with Reg Guide 1.61

(5) properties of foundation materials

- shear modulus

  Refer to FSAR, Figures 3.7-7, -8, -9

- subgrade reactions

  ~~Refer to Project Design Criteria Part II, Section 3.4.5.4 for coefficient of Subgrade reaction.~~
  coefficient of subgrade reaction = 40-60 kips/ft²/ft

- bearing capabilities

  Refer to Tables 2.5-15 and 2.5-16 of the FSAR. (Pages 448,9)

(6) other parameters

b. stiffness calculations

(1) exterior walls

Stiffness calculations were performed manually using standard engineering methods.

(2) interior walls

Same as exterior walls.

2. method of Analysis

a. method of analysis used (time history, response spectrum methods, etc.) and consideration of torsional and translational response

Time history analyses were performed to generate the in-structure response spectra using Fosin. Modal response spectrum analyses were performed to obtain the seismic loads for design of structural elements using SAP 1.9.

(1) general description

The Diesel Generator Building analysis used two planar lumped parameter models. Soil structure interaction was incorporated into the model by adding to the fixed base system descrete soil springs based on elastic half-space theory.

~~The SSI analysis method was~~ Based on the results of 4 category I structures by the comparison of lumped model & finite element model, the NRC SSI intent is met.

(2) findings and comments

b. selection of number of masses and degrees of freedom

(1) general description

For horizontal direction earthquake, the model consisted of 3 nodes with 3 degrees of freedom.

For vertical direction earthquake, the model consisted of 3 nodes with 3 degrees of freedom.

(2) findings and comments

c. number of modes considered

	SSE	OBE
Horizontal (E-W)	3 modes f = 43.7 cps	3 modes f = 43.3 cps
Horizontal (N-S)	3 modes f = 47.7 cps	3 modes f = 47.8 cps
Vertical	3 modes f = 126 cps	3 modes f = 127 cps

f = Frequency of highest mode considered.

(1) general description

See Attachment B for modal frequencies and participation factors. (Pg. 445)

(2) findings and comments

d. combining modal responses

(1) actual procedures used

Refer to FSAR, Section 3.7.2.7.

(2) general findings

e. consideration of three components of motion

(1) actual procedures used

Refer to FSAR, Section 3.7.2.6.
this is consistent with Reg. Guide 1.92

(2) general findings

f. consideration of soil-structure interaction

Soil Springs

	East-West	$K_x = 1.38 \times 10^6$ K/ft	$K_{xx} = 2.67 \times 10^9$ K-ft/rad
OB E	Vertical	$K_y = 1.90 \times 10^6$ K/ft	
	North-South	$K_z = 1.52 \times 10^6$ K/ft	$K_{zz} = 1.76 \times 10^9$ K-ft/rad
	East-West	$K_x = 1.06 \times 10^6$ K/ft	$K_{xx} = 1.91 \times 10^9$ K-ft/rad
S SE	Vertical	$K_y = 1.46 \times 10^6$ K/ft	
	North-South	$K_z = 1.06 \times 10^6$ K/ft	$K_{zz} = 1.35 \times 10^9$ K-ft/rad

(1) general description

Refer to FSAR, Section 3.7.2.4.

(2) findings and comments

g. decoupling criteria for subsystems

(1) general procedure

Refer to BC-TOP-4A, Section 3.2

(2) key examples

(3) The other criteria pertaining to frequency ratio as defined in S.R.P 3.3.3.II.3.b are also met.

(3) general findings and comments

h. modeling of hydrodynamic effects in spent fuel pool

Not applicable.

i. modeling of spent fuel pool wells and interior floor slabs and equipment thereof

Not applicable.

3. development of in-structure response spectra

Refer to FSAR, Section 3.7.2.5

a. general procedures

The "SPECTRA" program was used to compute the response spectra. Refer to FSAR, Appendix 3B for a description and applications of this program.

(1) smoothing (describe specific smoothing method used)

The smoothened floor response spectra represent an envelope of the maximum peaks.

(2) peak widening

± 15%

Reference: FSAR, Section 3.7.2.9

b. typical results (attach figures)

Refer to FSAR, Figures 3D-34, -35, -36 (Pages 461, 461A, 462)

B. Stress Analysis

1. shear walls and floors

a. mathematical model - general description w/sketch

Shear wall and floor stresses were computed by performing standard manual calculations. Vertical loads were distributed in the structure by conventional methods. Lateral loads were calculated by multiplying lumped mass and seismic accelerations obtained from floor spectra at each floor level. These lateral loads were then distributed among the shear walls according to their relative stiffness and location.

b. method of analysis--incorporation of torsion

The torsional moment was determined by resolving the couple due to eccentricity between the center of mass and the center of rigidity at each floor. Shear derived from this torsional moment is added directly to the forces considered for the individual shear walls.

c. load combinations

Refer to FSAR, Section 3.8.3.3.

1. foundation mat

a. mathematical model - description of boundary conditions

Basemat calculations were performed manually.

b. method of analysis

Loading on the basemat was obtained from an equivalent static analysis considering total dead and live load and its eccentricity, and three directional earthquake forces. Soil pressures were calculated directly from these forces using manual procedures.

c. load combinations

Refer to FSAR, Section 3.8.3.3.
this is in compliance with the SRP

d. key results (figures, etc.)

Refer to FSAR, Table 3.8-4J. (Pg. 458)

3. Material to protect against structure - structure interaction

Below grade (EL 100'-0") Rodofoam II (WR Grace and Co.) or EVERLASTIC Micro II (Williams Products, Inc.) may be left in place in seismic joints between structures. Above grade gaps between walls were left open, typically 6" between Category I structures.

a. mechanical properties

b. additional pressure on walls

c. findings and comments

4. vertical dynamic analysis

a. mathematical model - general description with sketch

The planar lumped parameter model consisted of one vertical beam on a rigid mat. Three nodes, representing the floor slabs at each elevation in the building, were used for the modal response spectrum model. The addition of a fourth node was used in the time history model to obtain the in-structure seismic response of the crane girder located 17.7 feet above the building's foundation.

See Attachment A. (Pg. 444)

b. development of stiffnesses, including floor stiffness, as applicable

Stiffness calculations were performed manually using standard engineering methods.

c. method of analysis

The models described above were used for time history and modal response spectrum analysis. The time-history analysis was performed using FOSIN. The Modal response spectrum analysis was performed using BSAP.

C. Computer Programs Used in Analysis

BSAP - SMIS - LUCON - FOSIN - SPECTRA

1. assumptions and limitations

See FSAR, Appendix 3B.

2. applicability

See FSAR, Appendix 3B.

3. verification

- sensitivity study in case of numerical solutions (e.g., finite element analysis)

See FSAR, Appendix 3B.

4. load input (include all cases)

PROGRAM	INPUT
BSAP	Finite element model (modes and elements), scaled response spectra, soil stiffness.
SMIS	Mass matrix, stiffness matrix, damping values.
SPECTRA	In-structure time history, frequency or period, and damping values.
FOSIN	Free-field time history, damping values, frequency.
LUCON	Shear modulus, damping values

5. output (include all cases)

See FSAR Appendix 3B.

6. other discussions

D. Overall Stability

1. forces and moments from seismic analysis

Seismic responses at the basemat, center of resistance:

SSE

$P_x = 0.555 \times 10^4$ K

$P_y = 0.488 \times 10^4$ K

$P_z = 0.555 \times 10^4$ K

$M_x = 1.08 \times 10^5$ Ft-K

$M_z = 1.08 \times 10^5$ Ft-K

~~x = East-West~~
~~y = Vertical~~
~~z = North-South~~

EAST-WEST

$P = 0.555 * 10^4$ K

$M = 1.08 \times 10^5$ ft-K

NORTH-SOUTH

$P = 0.555 * 10^4$ K

$M = 1.08 * 10^5$ ftK

VERTICAL

$P = 0.488 * 10^4$ K

2. various cases considered

Seismic event loading combinations considered SSE or OBE applied in the North-South, East-West, and vertical directions simultaneously.

3. bearing pressure versus bearing capability and safety factor against bearing failure

   Refer to FSAR Section 2.5.4.10 and Table 2.5-15, -16 (Pages 448, 9)

4. factors of safety

   Refer to FSAR, Section 3.8.5.5 and Table 3.8-5. (Pg. 460)

   a. sliding

      Factor of Safety = 1.1 (SSE)

   b. overturning

      Factor of Safety = 400 (SSE)

E. Interaction of Non-category I Structures with the Structure Considered

1. identification of pertinent non-Category I structures

   None. There are no non-Category I structures adjacent to the Diesel Generator Building.

2. consideration given to potential failure of non-Category I systems on Category I systems

   Non-Category I systems which could affect Category I systems were designed for structural integrity under SSE loads.

   Reference: FSAR, Section 3.8.4.4.

3. general findings and comments

During a walk-down those items whose failure will not affect any safety-related equipment are left as they are. If they are judged that they might affect Category I systems, they are designed to maintain their structural integrity under an SSE.

F. Design Consideration for Tornado Missiles

1. design requirements

   Refer to FSAR, Table 3.5-8. (Pg. 450)

2. models for

   a. local damage

      Refer to FSAR, Section 3.5.3.

   b. overall response

      Refer to FSAR, Section 3.5.3.

3. load combinations

   Refer to FSAR Section 3.8.3.3.

4. forces

   The governing structural members are of sufficient thickness to preclude perforation by postulated missiles and to maintain structural integrity.

6. general comments and preliminary audit findings

III. CONFORMANCE TO ACCEPTABLE CRITERIA

A. Identification of deviations, if any

None.

B. Justification of deviations and disposition of the deviations

D. general comments

Part II Audit of Key Designs

A. Exterior Shear Walls

1. design requirements

The exterior walls were designed to satisfy structural function as bearing walls, shear walls, and protection against tornado missiles.

2. design loads (from general analysts)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4J. (Pg. 458)

4. detailed design of rebar placement at key sections

See Attachment C (Pg. 446)

5. general comments and preliminary audit findings

B. Interior Shear Walls

1. design requirements

The requirements for the interior shear walls are the same as those for the exterior shear walls with the additional requirement that the wall separating the two diesel generators must withstand the effects of an internally-generated missile resulting from a crank case explosion.

Refer to FSAR, Section 9A - NRC Question 430.7.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4J. (Pg. 458)

4. detailed design of rebar placement at key sections

See Attachment C. (Pg. 446)

5. general comments and preliminary audit findings

C. Main Floors and Roofs

1. design requirements

Main floors and roofs are primarily designed for vertical dead, live, and seismic loads. ~~as defined in the Project Design Criteria.~~ Roofs are also designed to satisfy minimum thickness to prevent perforation by tornado generated missiles.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4J. (Pg. 458)

4. detailed design of rebar placement at key sections

See Attachment C. (Pg. 446)

5. general comments and preliminary audit findings

D. Steel Structural Bracing Systems (if any).

1. design requirements

The Diesel Generator Building floors and roof slabs are supported by steel beams spanning between concrete walls. The beams in general are designed for construction loads. Girders are designed for dead, live and seismic loads.

2. design loads

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

Refer to FSAR, Table 3.8-4K (Pg. 459)

6. general comments and preliminary audit findings

E. Foundation Mat

1. design requirements

   Refer to FSAR, Sections 3.8.5.4.2 and 3.8.5.5.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4J. (Pg. 458)

4. detailed design of rebar placement at key sections

   See Attachment C. (Pg. 446)

5. general comments and preliminary audit findings

F. Main Frame Concrete Column Design (Key Columns)

Not applicable. There are no concrete columns in this building.

1. design requirements

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design of rebar placement at key sections

5. general comments and preliminary audit findings

G. Secondary Floors

1. design requirements

   Secondary floors were designed in the same manner as main floors and roofs.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4J. (Pg. 459)

4. detailed design of rebar placement at key sections

   See Attachment C. (Pg. 446)

5. general comments and preliminary audit findings

H. Detailing at Floor-Wall Joints

1. design requirements

   As per ACI 318-17 Code, Chapters 6 and 17.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Refer to FSAR, Table 3.8-4J. (Pg. 459)

4. detailed design of rebar placement at key sections

   See Attachment C. (Pg. 446)

5. general comments and preliminary audit findings

I. Dynamic Effects Applied to Floors and Walls by Machinery

Due to the nature of the equipment foundations, the dynamic motion of the diesel generators will have no effect on the floors and the walls of the structure. A two inch gap filled with an expansion material completely isolates the diesel generator pads from the foundation of the building. These equipment pads are designed to satiisfy all of the manufacturer's requirements and will transmit all dynamic vibrations from the diesel generators directly into the soil.

1. design requirements

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design

5. general comments and preliminary audit findings

J. Crane & Support

1. design of bents (columns and roof trusses)

   Not applicable.

   a. design requirements

   b. design loads (from general analysis)

   c. forces and moments at key sections

   d. detailed design

e. general comments and preliminary audit findings

2. design of girders supporting crane rails

a. design requirements

The crane rails are supported by a built-up steel member. This built-up member is in turn supported by a cantilevered wide flange bracket connected to an insert plate located in the shear walls.

b. design loads (from general analysis)

~~Refer to Project Design Criteria Part II, Section 3.5.~~

Dead load ~~and live loads~~, includes girder and all the associated equipment.
Live load is the load lifted by crane. (Capacity - 5T)

c. forces and moments at key sections

	DESIGN MOMENT		DESIGN SHEAR
	Mx	My	
Built-up crane rail support girder	41.4 ft-K	31.3 ft-K	60 K
Cantilevered Bracket	60 ft-K		60 K

d. detailed design

See Attachment D. (Pg. 447)

e. general comments and preliminary audit findings

3. design of spent fuel bridge

   Not applicable.

   a. design requirements

   b. design loads (from general analysis)

   c. forces and moments at key sections

   d. detailed design

   e. general comments and preliminary audit findings

K. Fuel Pool Liner Design

Not applicable.

1. stresses and strain controls

2. conformance to code requirements

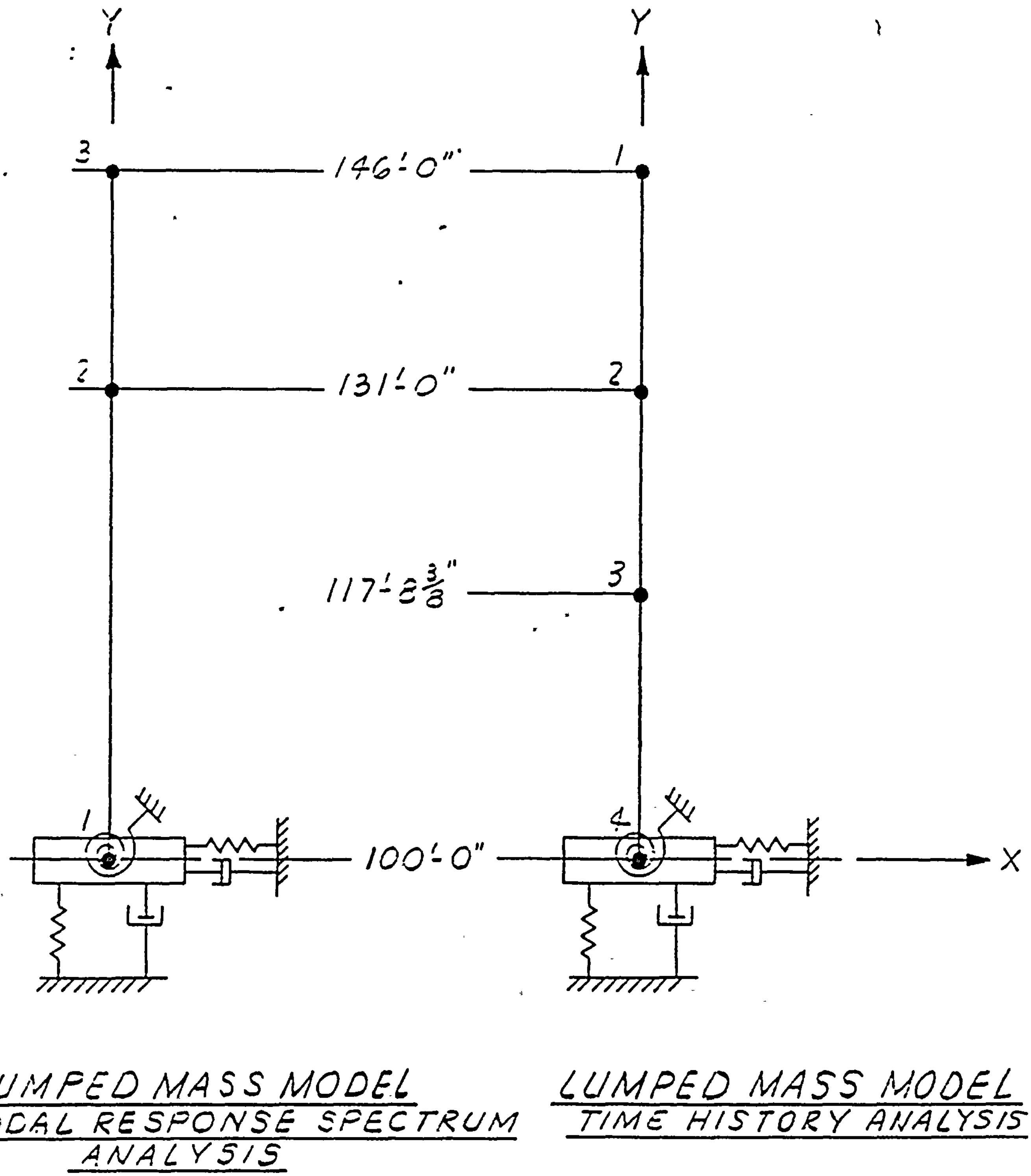
3. analysis procedure and results

4. consideration of accidental drop of crane loads

5. corrosion effects (e.g., pitting) on liner integrity

6. preliminary findings of audit results

DIESEL GENERATOR BLDG.

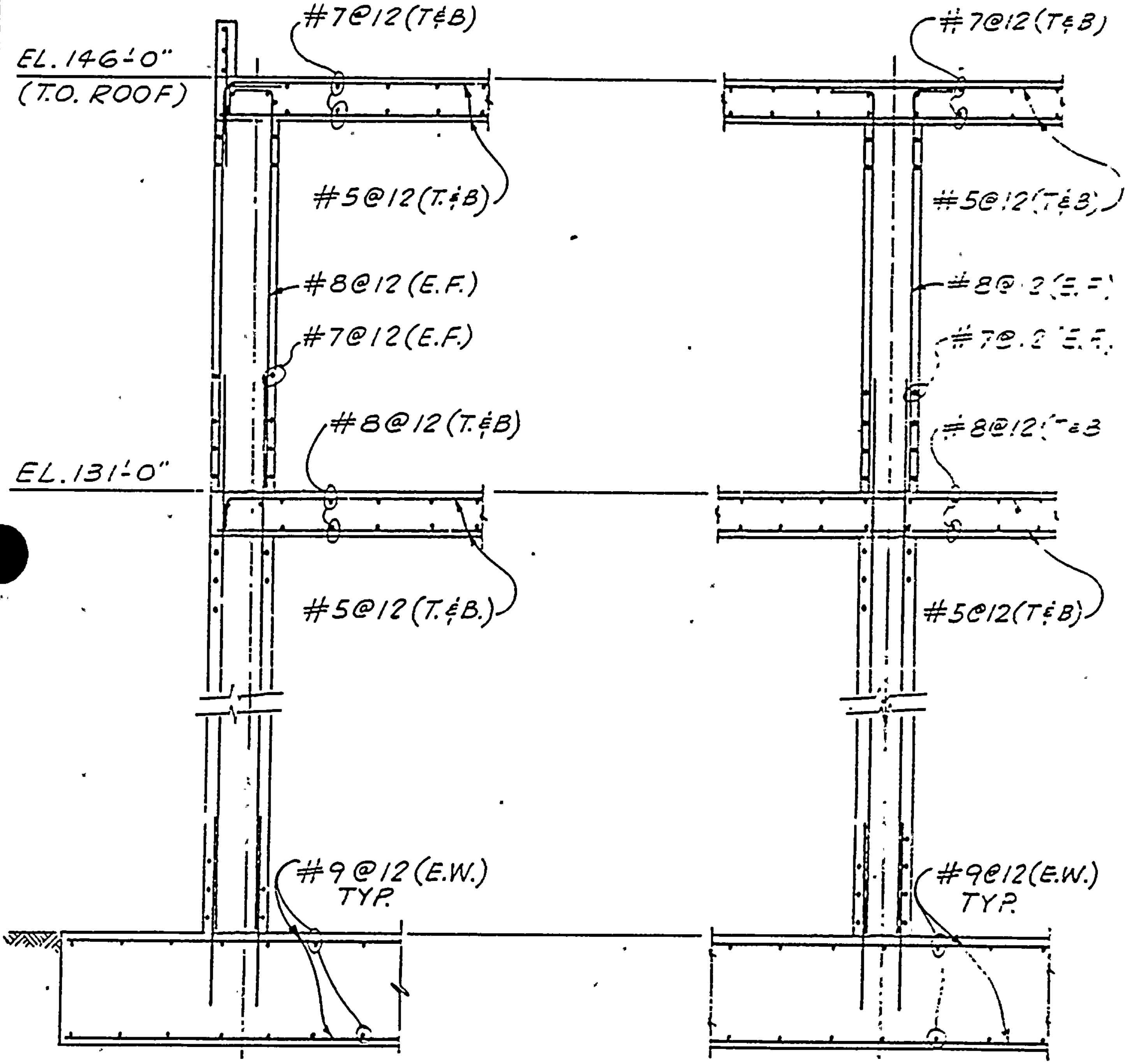


ATTACHMENT "A"

DIESEL GENERATOR BUILDING NATURAL FREQUENCIES

		Mode	Frequency (Hz)	Participation Factor
Horizontal (N-S)	OBE	1 2 3	7.98 21.9 47.8	17.1 6.36 0.41
	SSE	1 2 3	6.98 19.3 47.7	17.4 5.00 0.27
Horizontal (E-W)	OBE	1 2 3	6.72 19.1 43.8	16.5 7.79 0.44
	SSE	1 2 3	6.17 17.3 43.7	16.7 7.33 0.33
Vertical	OBE	1 2 3	11.6 62.8 126.8	19.3 0.86 0.033
	SSE	1 2 3	10.3 62.7 126.8	18.2 0.66 0.025

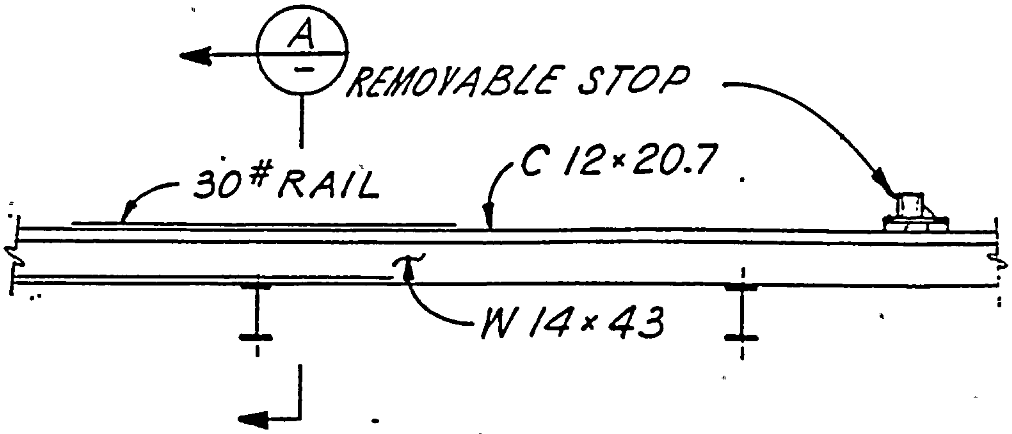
ATACHMENT "B"



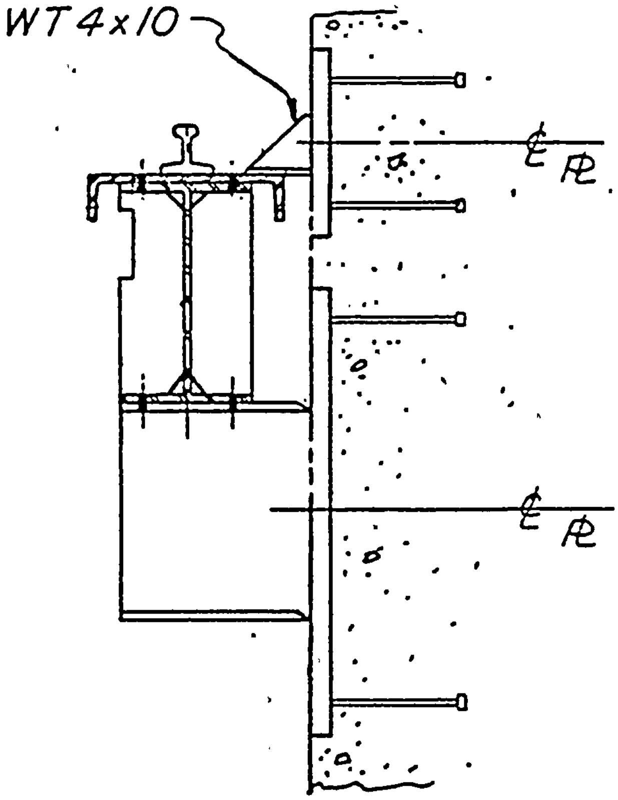
DIESEL GENERATOR BLDG.
TYPICAL WALL SECTIONS

ATACHMENT "C"

DIESEL GENERATOR BLDG.



ELEVATION



SECTION A

ATTACHMENT "D"

Table 2.5-15

STATIC BEARING CAPACITY OF CATEGORY I STRUCTURES

Structure	Average Static Design Load q_s (k/ft^2)	Ultimate Bearing Capacity q_o (k/ft^2)	Factor of Safety (q_o/q_s)
Containment Building	7.9	35.7	4.5
Auxiliary Building (deep section)	6.2	34.9	5.6
Main Steam Support Structure	7.1	64.8	9.1
Control Building	3.3	45.3	13.7
Fuel Building	5.3	54.9	10.4
Diesel Generator Building	3.1	79.5	25.6
Refueling Water Tank	4.4	90.4	20.5
Condensate Storage Tank	3.5	112.4	32.1

Table 2.5-16

DYNAMIC BEARING CAPACITY OF CATEGORY I STRUCTURES[a]

Structure	Equivalent Uniform Vertical Stress q_d (k/ft^2)	Ultimate Bearing Capacity q_o (k/ft^2)	Factor of Safety (q_o/q_d)
Containment Building	16.1	32.2	2.0
Auxiliary Building (deep section)	10.3	25.8	2.5
Main Steam Support Structure	25.3	60.6	2.4
Control Building	9.8	39.8	4.1
Fuel Building	19.1	50.3	2.6
Diesel Generator Building	5.6	75.5	13.5
Refueling Water Tank	13.2	58.7	4.4
Condensate Storage Tank[b]	13.2	30.2	2.3

a. Based upon maximum dynamic loads derived from analyses described in section 3.7.

b. Condensate storage tank loads were conservatively chosen to be equal to the dynamic design load for the refueling water tank. Actual loads will be less.

1

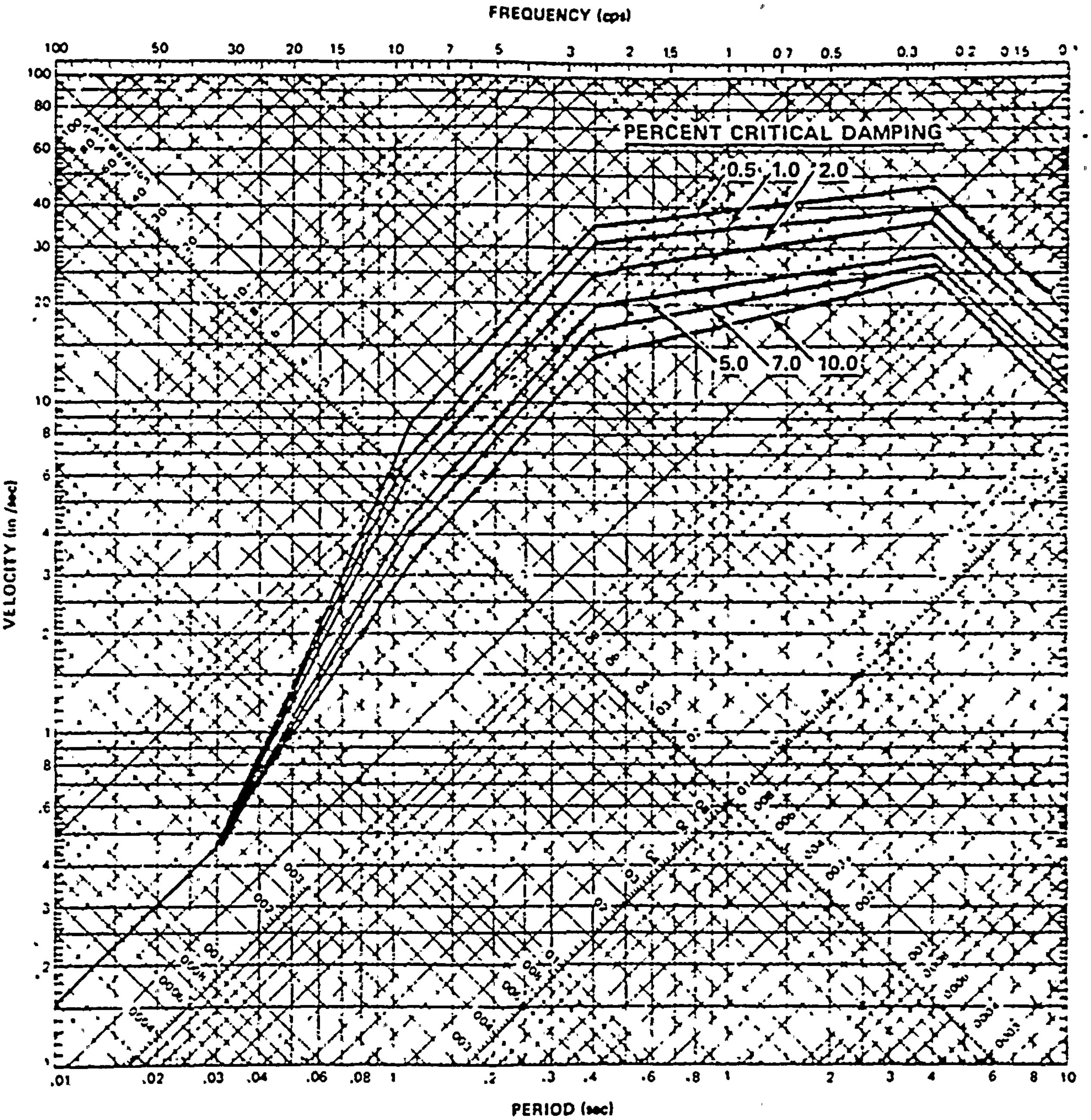
Table 3.5-8

TORNADO-GENERATED MISSILES CONSIDERED IN DESIGN OF SAFE SHUTDOWN STRUCTURES

Description	Weight (lbs)	Impact Area (ft^2)	Maximum Velocity (ft/s)	Kinetic Energy (ft-lbs)
(A) A 12-foot wood plank, 4 x 12 inches in cross-section, traveling end on at a speed of 240 mi/h.	200	0.333	352	3.85×10^5
(B) A steel pipe, Schedule 40, 3 inches in diameter by 10 feet long, traveling end on at 120 mi/h.	78	0.063	176	3.75×10^4
(C) A steel rod, 1 inch in diameter, 3 feet long, traveling end on at 180 mi/h.	8	0.005	264	8.66×10^3
(D) A steel pipe, Schedule 40, 6 inches in diameter by 15 feet long, traveling end on at 120 mi/h.	285	0.24	176	1.37×10^5
(E) A steel pipe, Schedule 40, 12 inches in diameter by 15 feet long, traveling end on at 120 mi/h.	743	0.886	176	3.57×10^5
(F) A utility pole, 13-1/2 inches in diameter, 35 feet long, traveling end on at 120 mi/h.	1490	0.994	176	7.17×10^5
(G) An automobile of 4,000 pounds weight, striking the structure at 60 mi/h.	4000	20.0	88	4.81×10^5

17

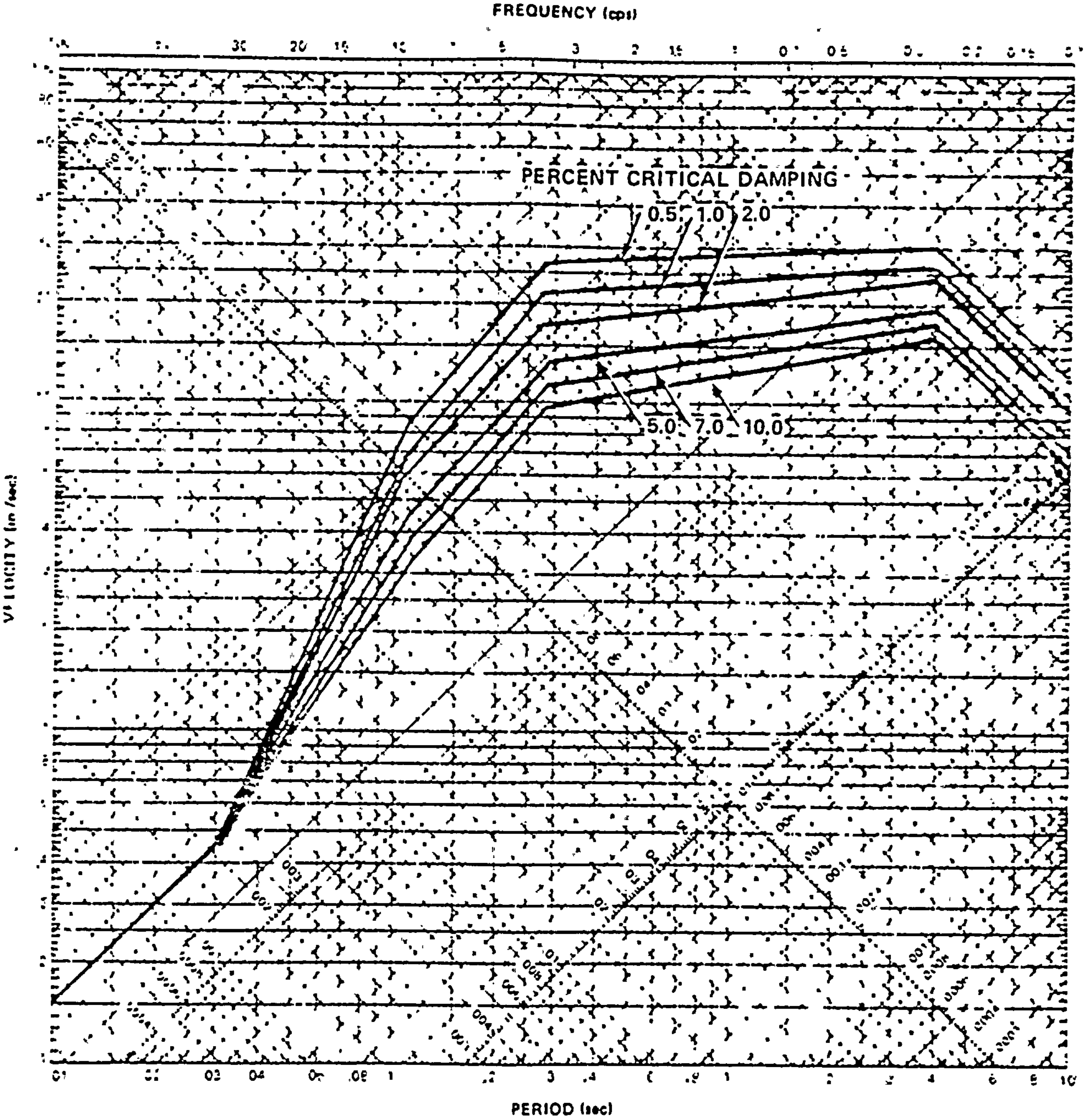
450



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR SSE 0.25 g

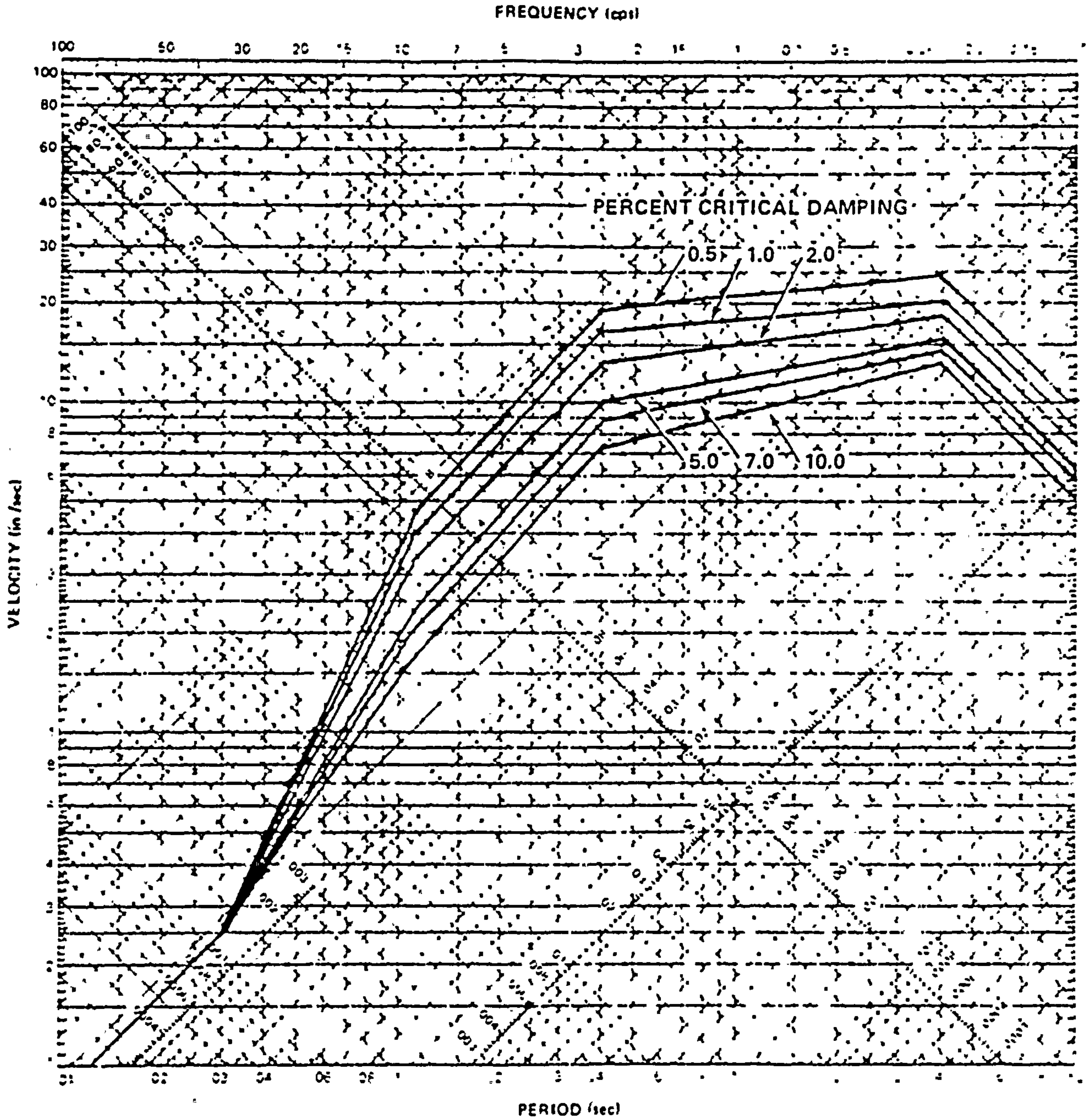
Figure 3.7-1



Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR SSE 0.25 g

Figure 3.7-2

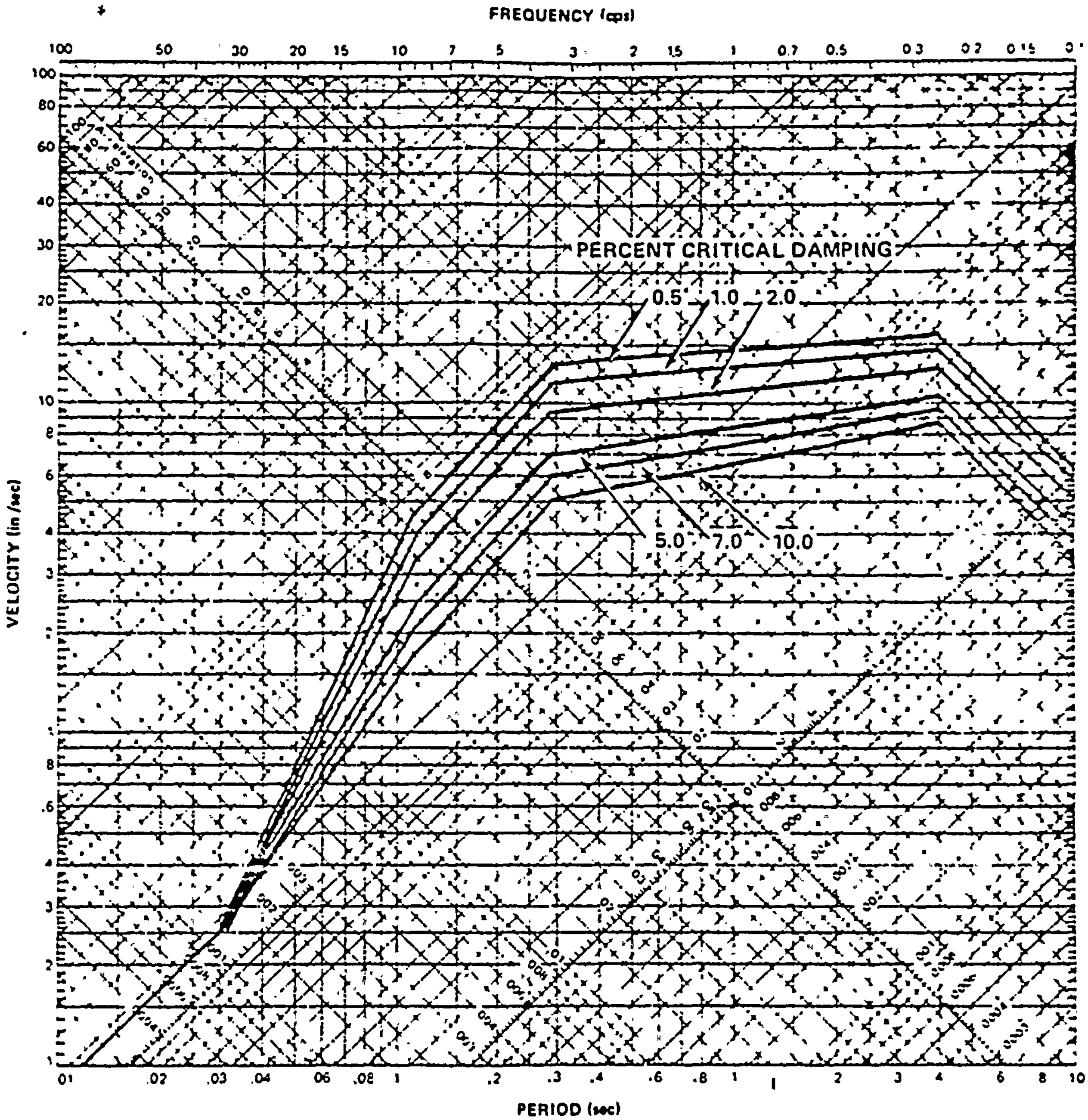


Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-3

C1



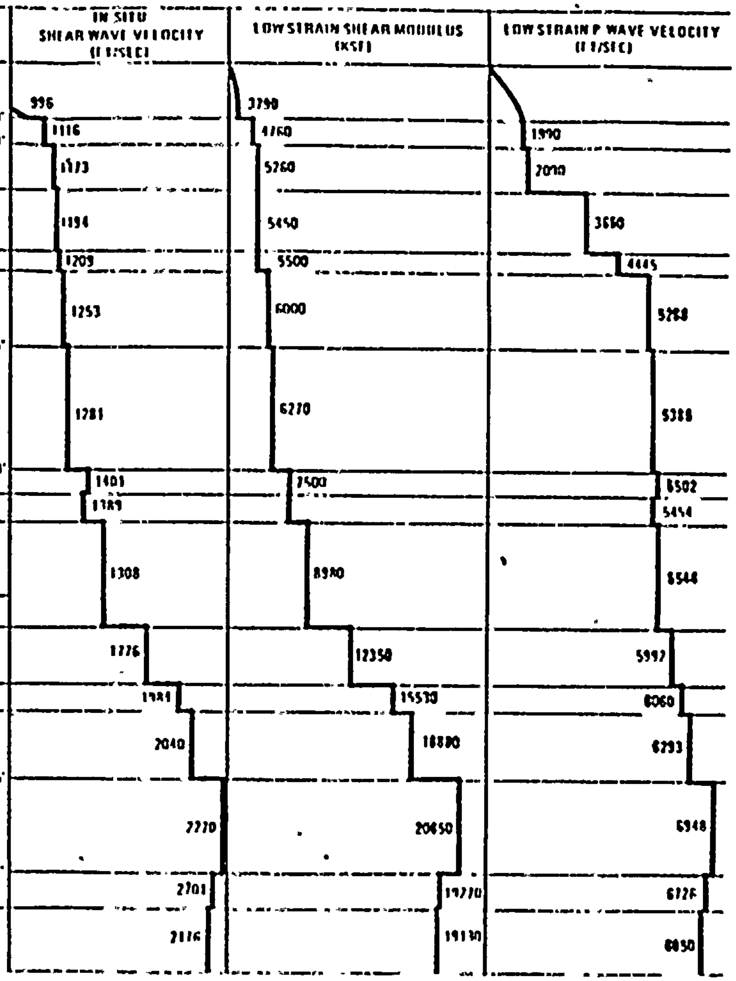
Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-4

AVERAGE PERCHED WATER LEVEL 44'

DEPTH (FT)	LAYER DEPTH (FT)	DESCRIPTION	LAYER THICKNESS	UNIT WEIGHT (PCF)	POISSON'S RATIO
		SAND (2)	47'	123	22
50	47'	CLAY (1)	23'	121	44
	70'	SAND (2)	7'	121	.46
100	77'	CLAY (1)	82'	123	.47
150	159'	SAND (2)	10'	125	465
	169'	CLAY (1)	27'	127	.46
200	196'	SAND (2)	12'	127	46
	206'	CLAY (1)	20'	126	.45
	228'	SAND (2)	10'	127	44
250	238'	CLAY (1)	73'	128	43
300	311'	SAND (2)	23'	130	45
350	334'	BEDROCK			



NOTES: 1 LOW STRAIN SHEAR MODULI AT VARIOUS DEPTHS ARE AVERAGE VALUES

2 SHEAR WAVE VELOCITY CALCULATED FROM $V_S = (G/\rho)^{1/2}$

3 P WAVE VELOCITY CALCULATED FROM V_P V_S (2 2v)/(1 2v)$^{1/2}$

Palo Verde Nuclear Generating Station
FSAR

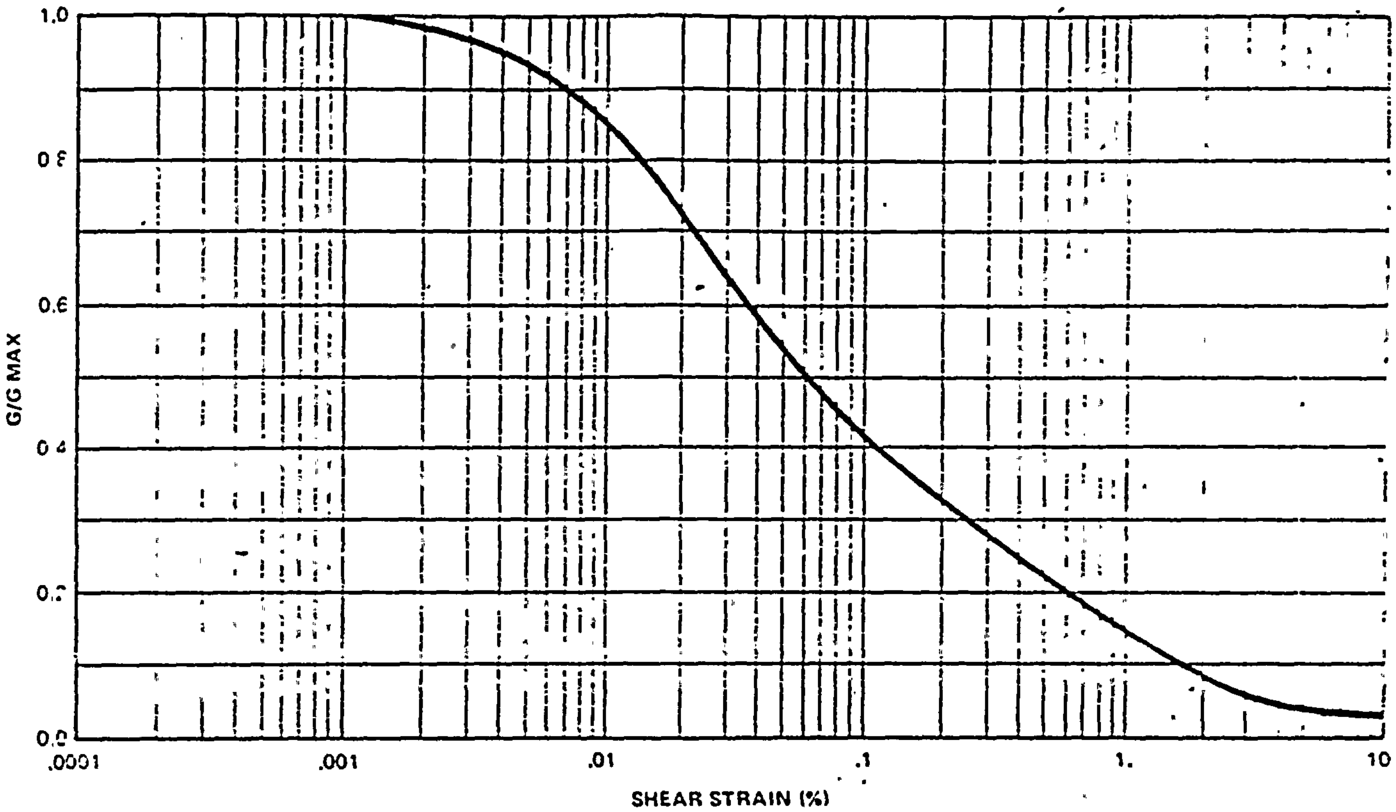
SOIL PROFILE

FIGURE 2.?-?

CHANGE

455

82



Palo Verde Nuclear Generating Station
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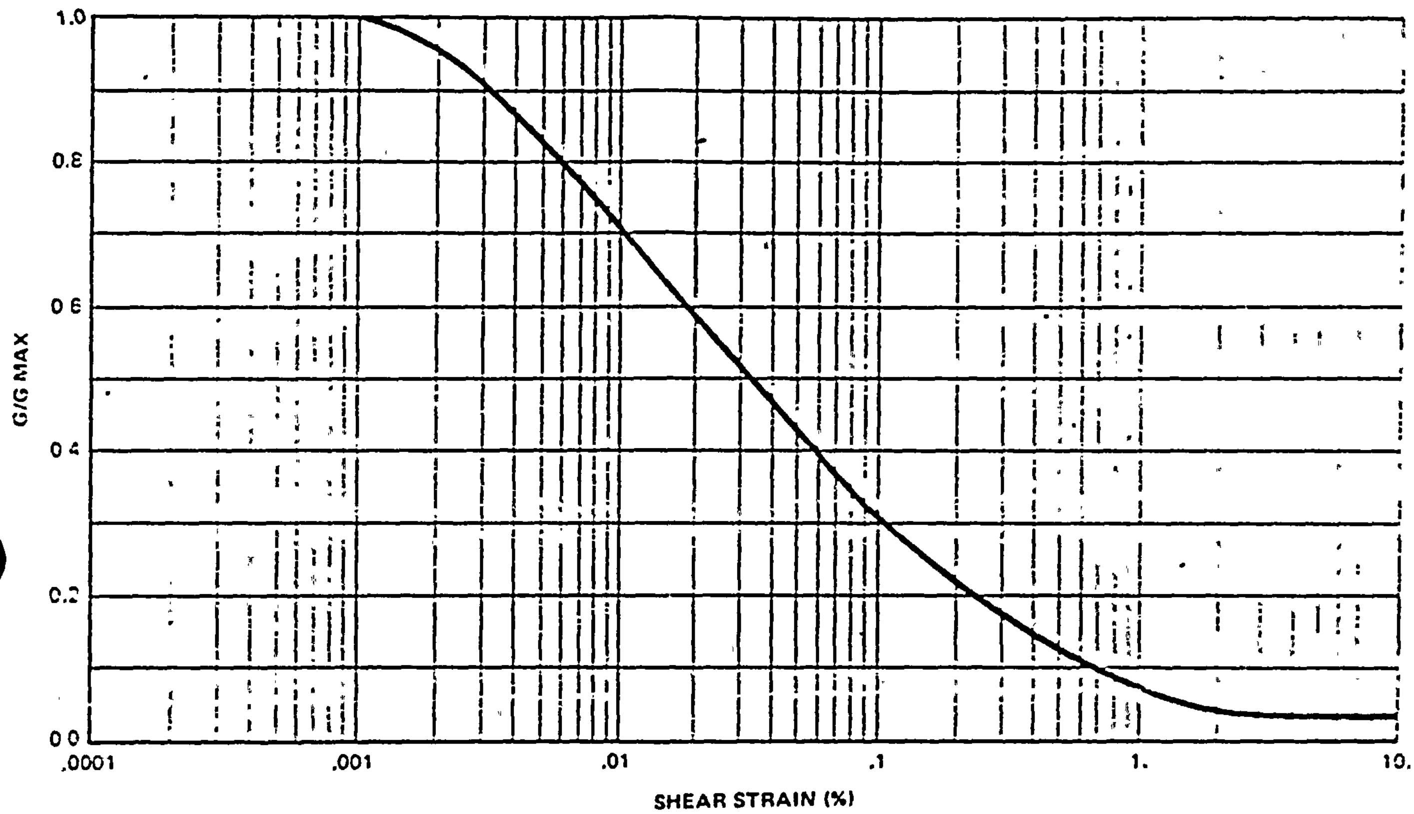
SHEAR MODULUS VS. STRAIN - CLAY

FIGURE 3.7-8



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SHEAR MODULUS VS. STRAIN - SAND

FIGURE 3.7-9

Table 3.8-4J

DIESEL GENERATOR BUILDING SUMMARY OF GOVERNING LOAD INTERACTIONS FOR PRINCIPAL REINFORCED CONCRETE MEMBERS

Description of Principal Member	Location of Principal Member	Governing Load Combination Number (a)	Calculated Axial Load (P_u) and Flexural Load (M_u)		Maximum Flexural Interaction Capacity (M_u), Given Axial Load (P_u) (b)(c)	Calculated Shear Load (V_u) (b)	Maximum Shear Capacity (V_u) (b)
			P_u (b)	M_u (c)			
1'-9" x 60'-0" wall - vertical and horizontal reinforcement	Exterior wall @ El. 100'-0"	2	-1,042	53,671	153,503	1,548	4,986
1'-9" x 19'-6" wall - vertical and horizontal reinforcement	Interior wall @ El. 100'-0"	2	-497	8,323	16,869	166	1,619
1'-9" x 60'-0" wall - vertical and horizontal reinforcement	Interior wall @ El. 131'-0"	2	-556	5,772	146,549	385	4,986
1'-4" thick slab - E-W reinforcement	El. 115'-0"	2	-(d)	13	34	-(d)	-(d)
1'-4" thick slab - N-S reinforcement	El. 131'-0"	2	-	16	34	-	-
4'-0" thick basemat - E-W reinforcement	El. 100'-0"	2	-	247	249	-	-
4'-0" thick basemat - E-W or N-S reinforcement	El. 100'-0"	2	-	210	240	-	-

a. Refer to Section 3.8.3.3.2.A(2) for description of load combination number.
b. P_u and V_u are in kips; sign convention for P_u: Compression (-), Tension (+).
c. M_u is in ft-k/ft for slabs and ft-K for walls.
d. Negligible.

Table 3.8-4K

DIESEL GENERATOR BUILDING SUMMARY OF GOVERNING COMBINED STRESS RATIOS FROM THE BEAM/COLUMN INTERACTION EQUATION FOR PRINCIPAL STRUCTURAL STEEL MEMBERS

Description of Principal Members	Location of Principal Members	Governing Load Combination Number[a]	Combined Stress Ratio (≤1.0)
W 24 X 84	Floor Beam @ El. 115'-0"	2	0.47
W 12 X 45	Floor Beam @ El. 115'-0"	2	0.71
W 24 X 130	Main Girder @ El. 131'-0"	2	0.87
W 36 X 160	Main Girder @ El. 131'-0"	2	0.85
W 24 X 100	Floor Beam @ El. 146'-0"	2	0.58
S 24 X 120	Monorail Beam @ El. 126'-5"	1	0.98

a. Refer to section 3.8.3.3.3.A(1) for description of load combination number.

3

3.8.5.5 Structural Acceptance Criteria

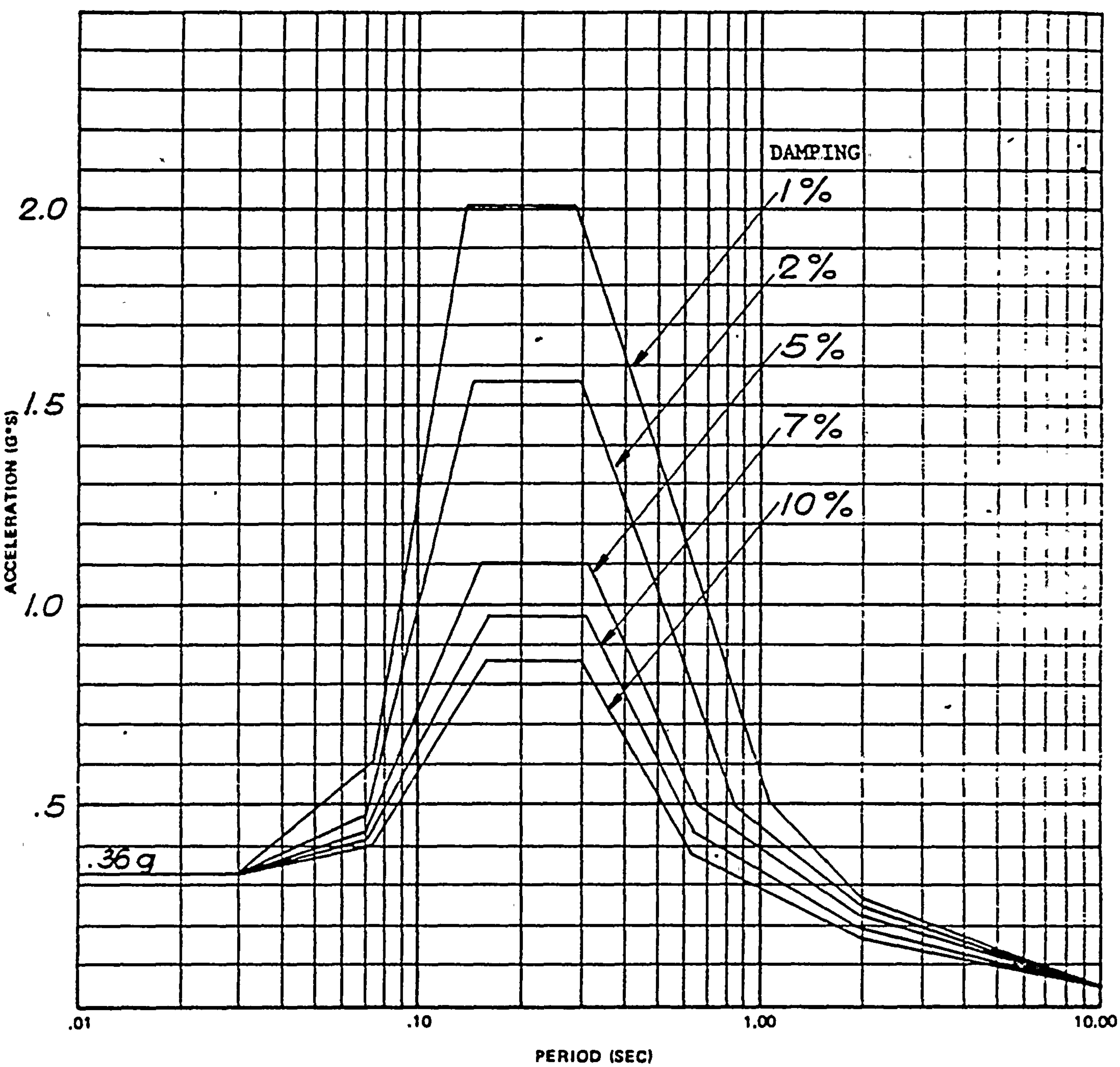
The foundations of Seismic Category I buildings are designed to meet the same structural acceptance criteria as the buildings themselves. These criteria are discussed in sections 3.8.1.5, 3.8.3.5, and 3.8.4.5. The limiting conditions for the foundation medium, together with a comparison of actual capacity and estimated structure loads, are found in sections 2.5.4.10 and 2.5.4.11. Computed factors of 1
safety against overturning, sliding, and flotation for
Category I structures are given in table 3.8-5. 1

Table 3.8-5
COMPUTED FACTORS OF SAFETY

Structure	Overturning		Sliding		Flotation
	OBE	SSE	OBE	SSE	
Auxiliary	3200	830	2.2	1.3	4.7
Containment	3400	1200	1.7	1.2	4.5
Control	1500	420	1.6	1.2	4.8
Diesel Generator	1200	400	2.2	1.1	NA[a]
Fuel	1600	400	1.9	1.1	NA
Main Steam Support	340	91	1.6	1.1	NA
Condensate Storage and Refueling Water Tanks	500	150	1.7	1.4	NA

a. Not applicable

460



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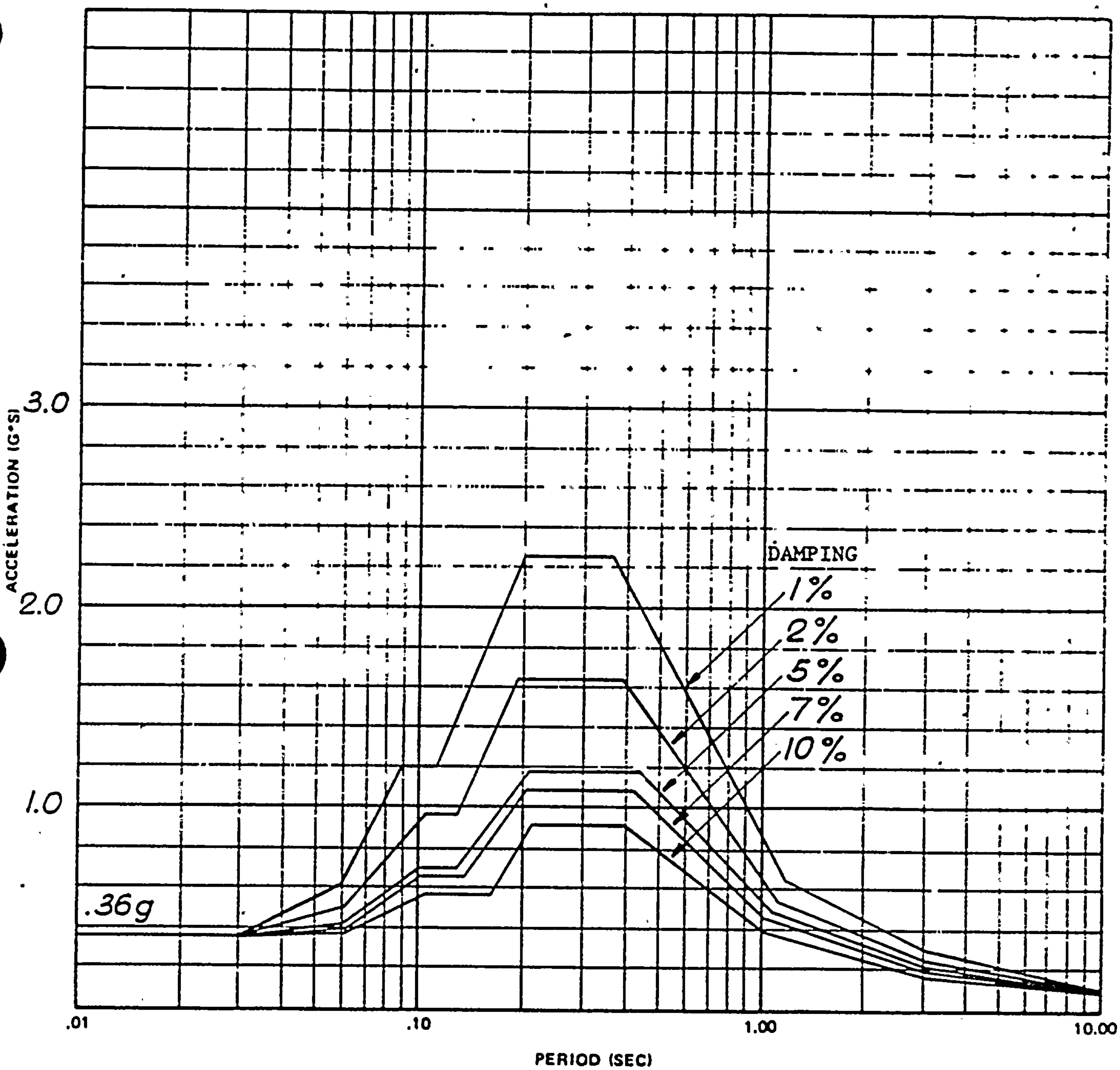
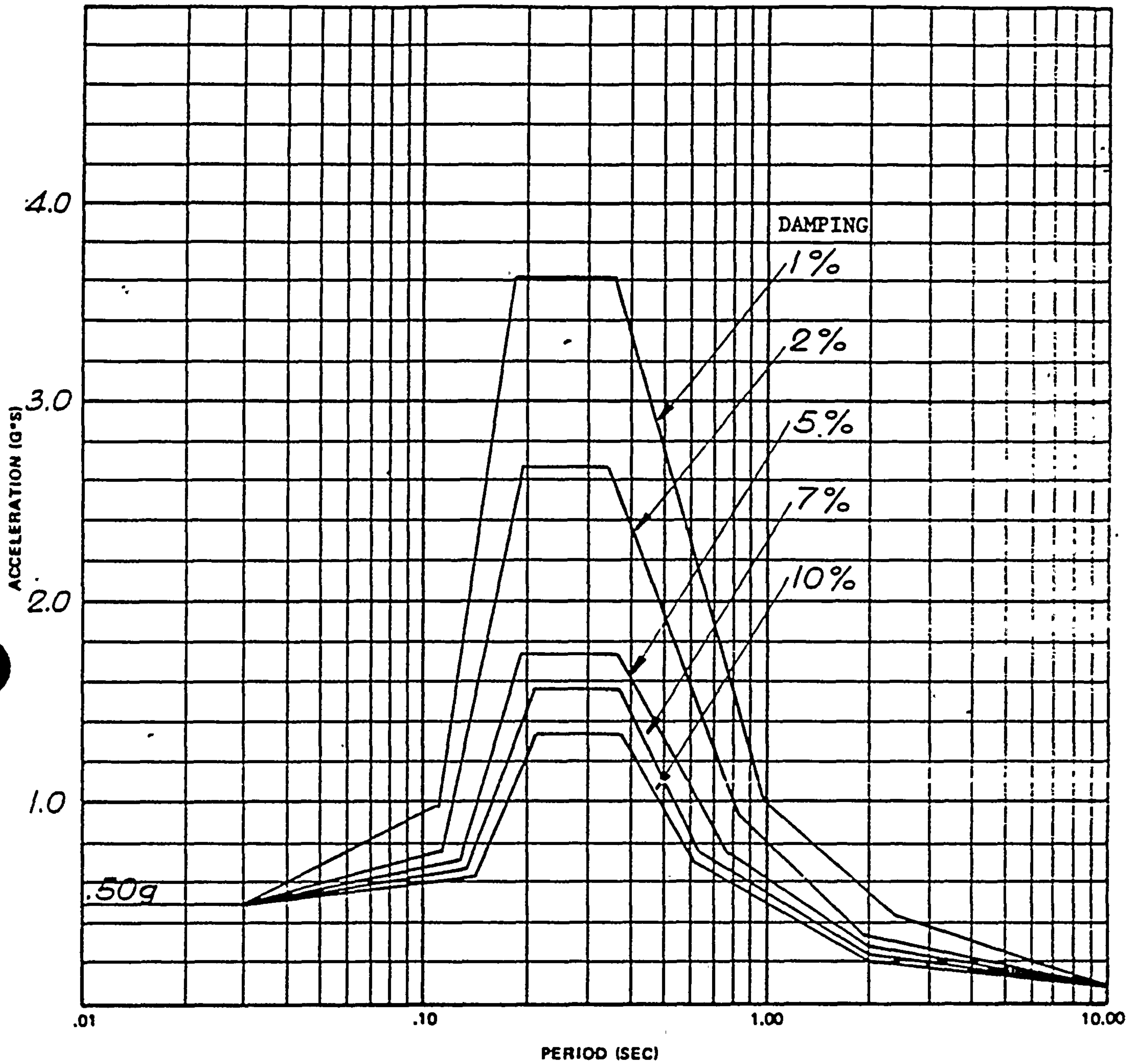
DIESEL GENERATOR BUILDING
SSE VERTICAL
ACCELERATION RESPONSE SPECTRA

Figure 3D-34



Palo Verde Nuclear Generating Station
FSAR

DIESEL GENERATOR BUILDING
SSE HORIZONTAL ACCELERATION
RESPONSE SPECTRA, EL 100'-0",
BASE MAT
Figure 3D-35



Palo Verde Nuclear Generating Station
FSAR

DIESEL GENERATOR BUILDING
SSE HORIZONTAL ACCELERATION
RESPONSE SPECTRA, EL 146'-0", ROOF

Figure 3D-36

PALO VERDE NUCLEAR STATION UNITS 1, 2, 3
DESIGN ADEQUACY AUDIT

STRUCTURAL AUDIT OF CATEGORY I TANKS

Part I - General Analysis

I. BASIC DESIGN CRITERIA

(1) 'g' value - free field

	Seismic level based on construction permit license	Seismic level used in design of structures and equipment
SSE	.20g	.25g
OBE	.10g	.13g

Reference: FSAR Section 3.7

(2) Spectra (attached fig. for all damping values, ductilities)

A. Zero period acceleration

SSE .25g

OBE .13g

Reference: FSAR, Figures 3.7-1 - 3.7-4 (Pages 490, 1, 2, 3) and Section 3.7.1.1
This is consistent with Reg Guide 1.60

B. Frequency (or period) interval

Refer to BC-TOP-4A, Section 2.5.1(c)

C. Damping

Refer to FSAR, Section 3.7.1.3.
This is consistent with Reg Guide 1.61
Refer to FSAR figures 3.7-5 and 3.7-6 (Pages 92A & 92B)

D. Artificial time history and corresponding spectra (attach figures)

1. original time history and its composition i.e., rising time, strong motion and tail end.

A time history analysis was not used.

2. base line correction, check the integrated velocity and displacement time histories.

3. time interval - compatible with the highest frequency considered in the spectral calculation.

E. Motion duration

F. Components of motion including their relative motion amplitudes. Analysis was performed for the three principle directions with equal amplitudes. Due to symmetry, only one horizontal analysis was performed.

G. Dead and live loads

~~Refer to Project Design Criteria Part II, Sections 3.5.1 and 3.5.2 and Part III, Section 1.4.1~~.
Dead load includes all the structures and equipment.
there is no live load

H. Internal Pressure

In addition to the normal hydrostatic pressure, the following pressures were considered:

TYPE OF PRESSURE	CAUSE OF PRESSURE	REFUELING WATER TANK (psig)	HOLD-UP TANK (psig)	CONDENSATE STORAGE TANK (psig)
Normal Operating Pressure, Po	(a) Due to Suction or Discharge of Water, or	± 0.5	± 0.5	± 0.5
	(b) Due to Temp. Change	± 0.1	± 0.1	± 0.1
Accident Pressure, Pa	Due to Suction in LOCA	- 1.5	- 1.5	± 0.5

I. Ground water level

The groundwater design level is at plant El. 70′-0". The actual water level is at approximately plant El. 60′-0".

Reference: FSAR, Section 2.4.13.2.4 and 2.4.13.5.

J. Back fill earth pressure, wind, overpressure due to postulated external explosions (as applicable).

~~Refer to Project Design Criteria Part II, Sections 3.4.5.3 and 3.5.5 for lateral earth pressure, wind and tornado loads.~~
For lateral earth pressure see pages 208A & 208B.
Wind does not govern.

K. Other considerations

II. METHOD OF ANALYSIS

(a) Method of analysis used (Time history, response spectrum methods, etc.) and consideration of torsional and translation response.

A modal response spectrum analysis was performed to determine seismic loads for the design of structural elements.

(i) general description

The Category I tank analysis used a planar lumped parameter model with 9 nodes. Soil springs and hydrodynamic effects (modeled per US AEC TID 7024, Nuclear Reactor and Earthquakes) were incorporated. Only the most critical tank was modeled for the seismic analysis. The results were applied to all three tanks.

See Attachment A for a sketch of the mathematical model. (pg.481)

(ii) findings and comments

(b) Selection of number of masses and degrees of freedom

(i) general description

Total number of nodes considered = 9

Total number of degrees of freedom = 22

(ii) findings and comments

(c) number of modes considered

Six modes

Frequency of 6th mode = 39.92 cps (SSE)
= 40.24 cps (OBE)

(i) general description

See Attachment B for modal frequencies and participation factors. (pg. 482)

(ii) findings and comments

(d) combining modal responses

(i) actual procedures used

Refer to FSAR, Section 3.7.2.7.

(ii) general findings

(e) consideration of three components of motion

(i) actual procedures used

Refer to FSAR, Section 3.7.2.6
This is consistent with Reg Guide 1.92

(ii) general findings

STRESS ANALYSIS

1. Computer programs used in analysis

   BSAP, OPTCON

   A. Assumptions and limitations

      Refer to FSAR, Appendix 3B.

   B. Applicability

      Refer to FSAR, Appendix 3B.

   C. Verification

      • Sensitivity study in case of numerical solutions (e.g., finite element analysis)

      Refer to FSAR, Appendix 3B.

   D. Load input (include all cases)

PROGRAM	INPUT
BSAP	Finite element model (nodes and elements), response spectra, damping
OPTCON	Actual wall reinforcement, temperatures, section properties, element loadings.

E. Output (include all cases)

Refer to FSAR, Appendix 3B.

F. Other discussions

(2) Overall Stability

A. forces and moments from seismic analysis (computer output)

	SSE	OBE
Horizontal Force (K)	3,302	1,770
Vertical Force (K)	4,833	2,501
Over-turning Moment (FT-K)	142,557	76,514

B. Applicable loads other than those in item A above

None.

C. Various cases considered

Seismic event loading combinations considered OBE or SSE applied in N-S, E-W and vertical directions simultaneously.

D. bearing pressure versus bearing capacity and safety factor against bearing failure

Refer to FSAR, Section 2.5.4.10 and Tables 2.5-15, -16. (Pages 488, 9)

E. factors of safety

Refer to FSAR, Section 3.8.5.5 and Table 3.8-5. (Pg. 494)

a. sliding

Factor of Safety = 1.40 (SSE)

b. overturning

Factor of Safety = 150 (SSE)

(3) interaction of non-category I structures or pipings with the storage tanks

(a) identification of pertinent non-category I structures or pipings

None. There are no non-Category I structures adjacent to the tanks.

(b) consideration given to potential failure of non-category I pipings on Category I tank elements

(c) general findings and comments

III. Conformance to current NRC Criteria

(1) Identification of deviations, if any

None.

(2) Justification of deviations and disposition of the deviations

(3) Comparison of reevaluation results with the original design ....
and discussions.

(4) general comments

(5) Tank wall-base plate or mat junction design

A. design requirements and model (as applicable)

Refer to FSAR, Sections 3.8.4.2 and 3.8.4.4.

B. Design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

C. forces and moments at key sections

TANK WALL MOMENTS AT BASE

Mu = 61.4 K-Ft/Ft (Tension inside Face)
Mu = 49.3 K-Ft/Ft (Tension outside Face)

TANK WALL RING TENSION

Tmax = 164.3 K/Ft

TANK WALL RADIAL SHEAR

Vu = 20.9 K/Ft

PART II - AUDIT OF KEY DESIGNS

For each key design area audited, the design calculations should be reviewed together with applicable drawings, sketches, etc. Also, key details and/or sections, as appropriate, in this audit report should be included.

1. Storage Tank Liner Design

   (1) conformance with

   AISC Code.

   (2) specific check of key liner locations

   A. cylinder-base mat junction

   (a) sketch

   See Attachment C. (Pg. 483)

   (b) forces and displacements obtained from computer analysis

   The liner is not designed as a load carrying member. It acts only as the water-tight membrane for the tank.

F. Key penetration design

Refer to Attachments D and G. (Pages 484,5)

G. preliminary audit findings

(6) Tank roof or dome-to-cylinder junction design

See Attachment E. (Pg. 485)

A. design requirements and model

Refer to FSAR, Sections 3.8.4.2, 3.8.4.4, 3.8.4.1.7, and 3.8.4.1.8

B. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

C. forces and moments at key sections

Maximum Load in Radial Beams

Axial: 279^K (tension) Moment: 158.8 Ft-K

D. detailed design of rebar placement or steel connections at key sections

See Attachments E and F. (Pg. 485, 6)

E. Conformance to applicable codes and standards

Refer to FSAR, Section 3.8.4.2.

F. general comments and preliminary audit findings

(7) Piping penetrating design

See Attachment G. (Pg. 487)

A. design requirements and model

Refer to FSAR, Sections 3.8.4.2 and 3.8.4.4.

B. design loads (from general analysis)

Refer to FSAR Section 3.8.3.3.

C. forces and moments at key sections

Applied load on shell manhole cover = 26.26 K

D. Detailed design of the penetration

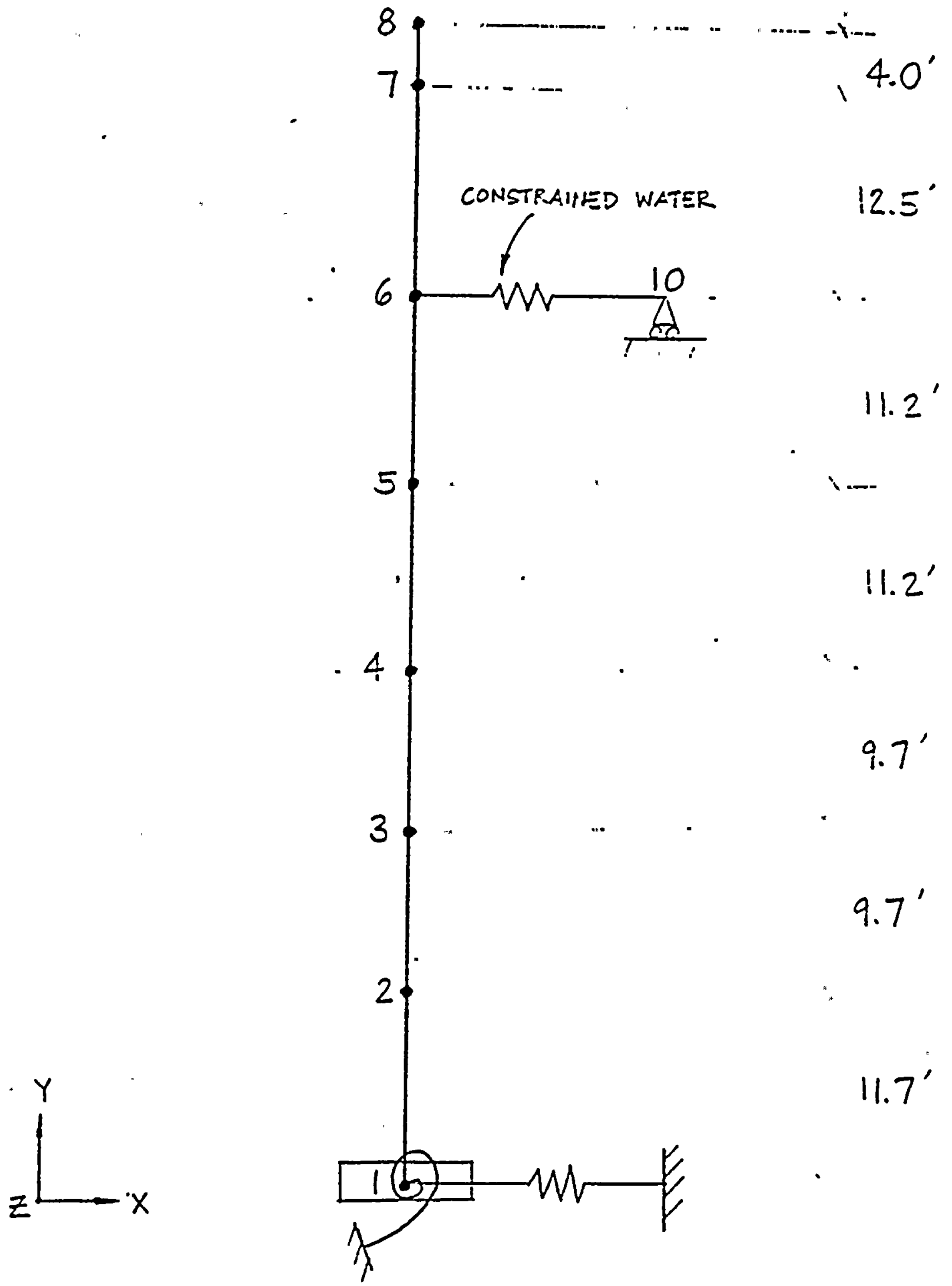
~~Refer to Project Design Criteria Part II, Section 3.3 and Part III, Section 4.0.~~

E. conformance to applicable codes and standards

Refer to FSAR, Section 3.8.4.2.

F. general comments and preliminary audit findings

ATTACHMENT A



CATEGORY I TANK MATH MODEL

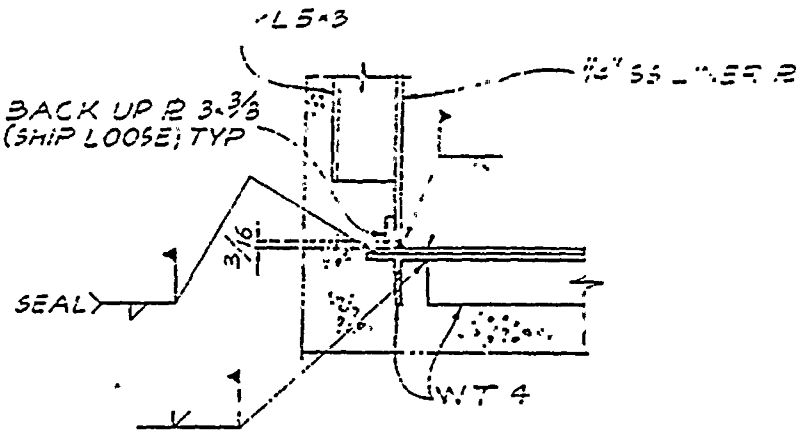
PVNGS

ATTACHMENT. B SEISMIC DESIGN

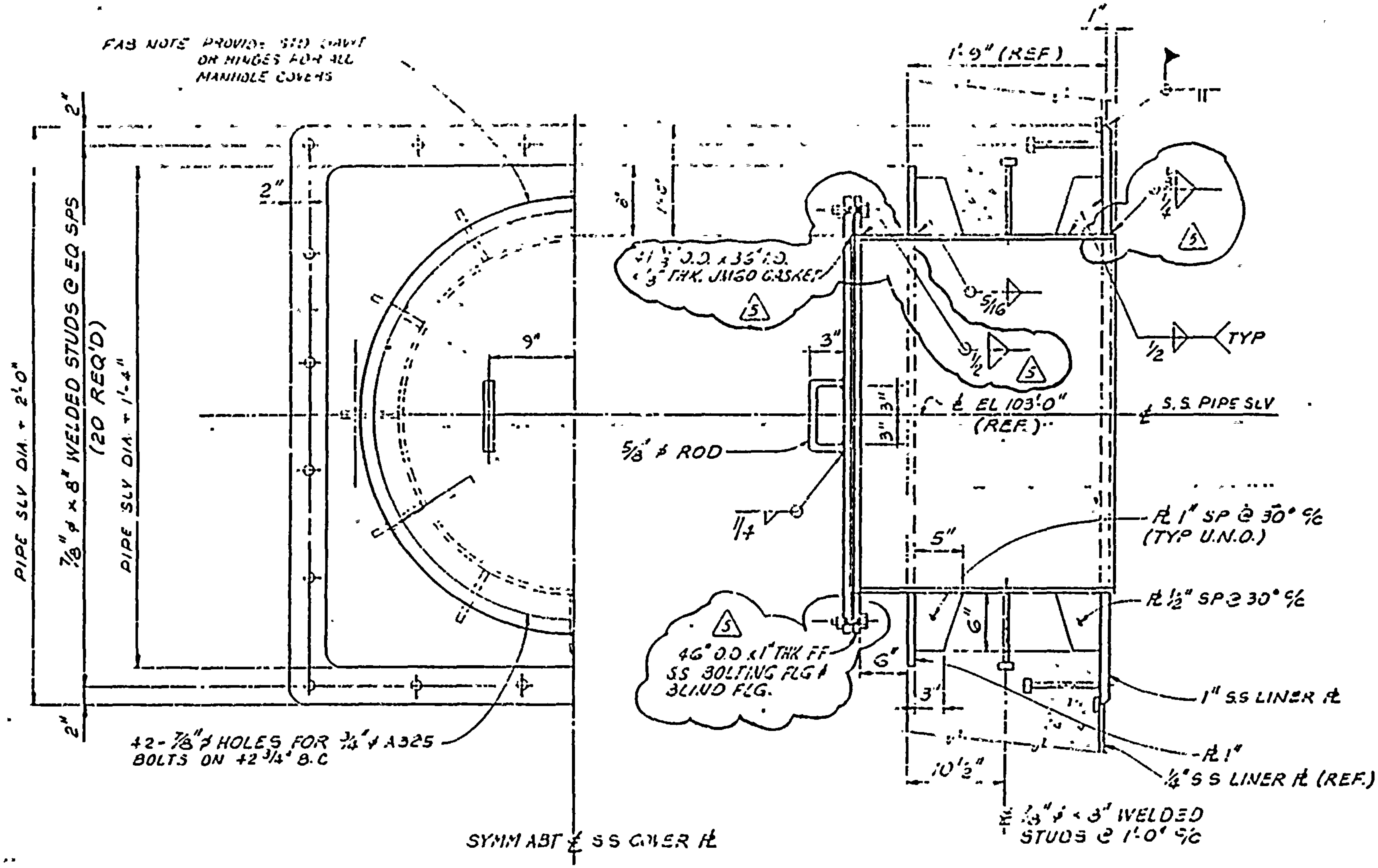
CAT. I TANKS ~~BUILDING~~ NATURAL FREQUENCIES [illegible] & PARTICIPATION FACTORS

		Mode	Frequency (Hz)		PARTICIPATION FACTOR
	OBE 2-D MODEL	1	0.27	SLOSHING	4.975
		2	3.70	ROCKING	-14.824
		3	7.34	VERTICAL	-17.600
		4	10.90	HORIZ.	-9.437
		5	35.90	HORIZ.	0.754
		6	40.24	[illegible]	[illegible]
	SSE 2-D MODEL	1	0.27	SLOSHING	4.979
		2	3.38	ROCKING	-14.847
		3	6.63	VERTICAL	17.600
		4	9.92	HORIZ.	-9.409
		5	35.68	HORIZ.	0.609
		6	39.92	HORIZ.	-0.155

ATTACHMENT C



CYLINDER - BASE MAT JUNCTION



SHELL MANHOLE

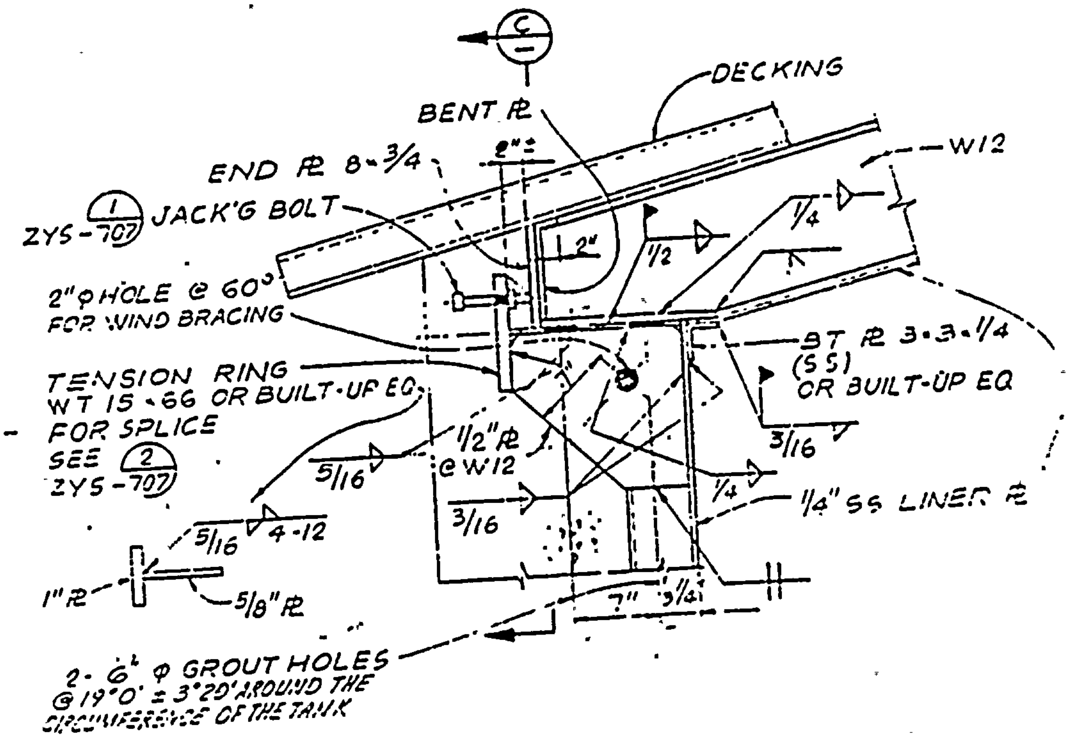
NTS

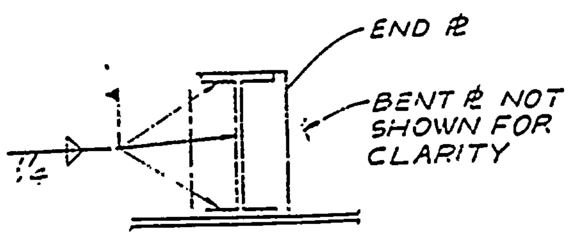
ATTACHMENT D

KEY PENETRATION DESIGN

4494

ATTACHMENT E

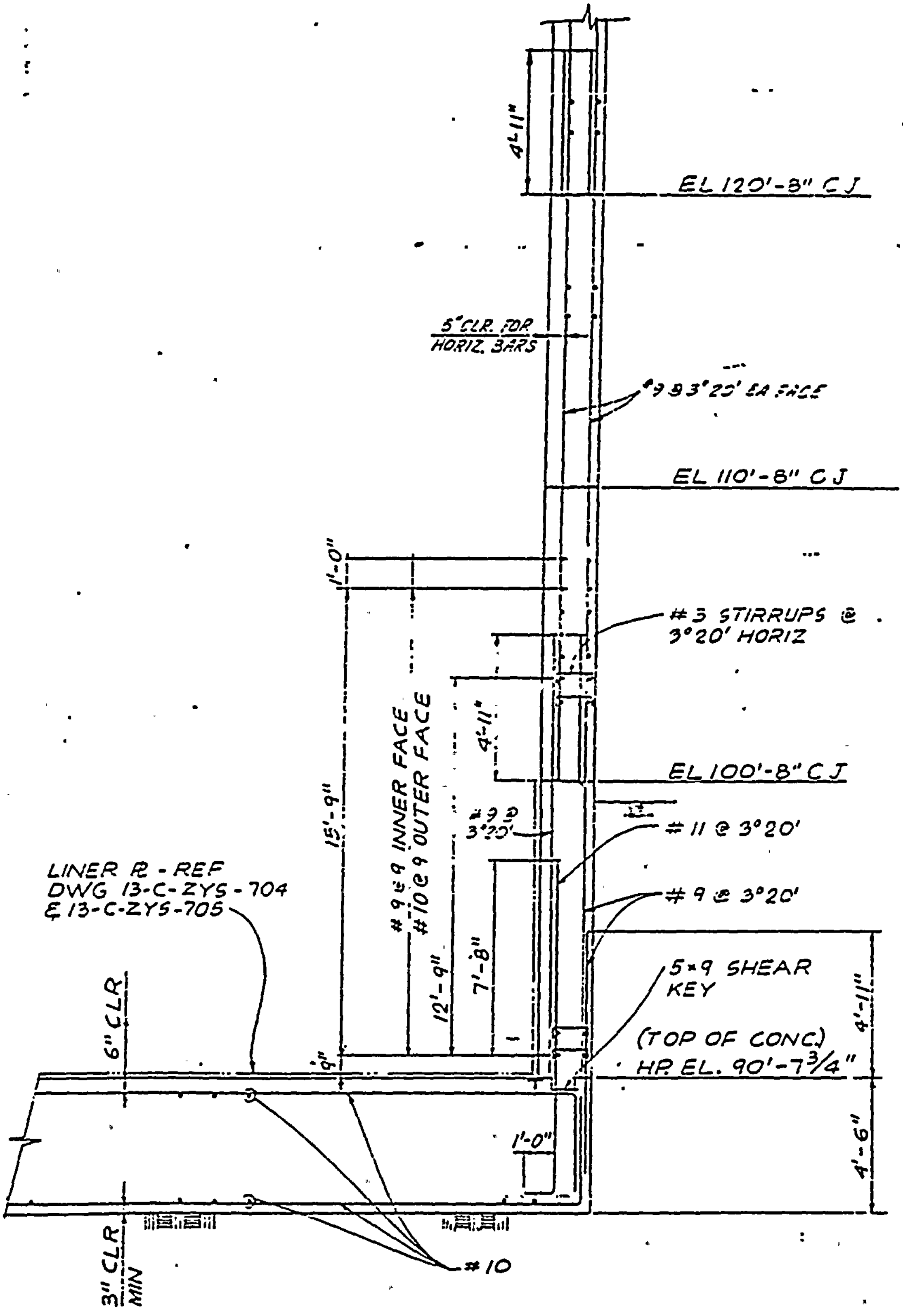




SECTION C
1 1/2" = 1'-0"

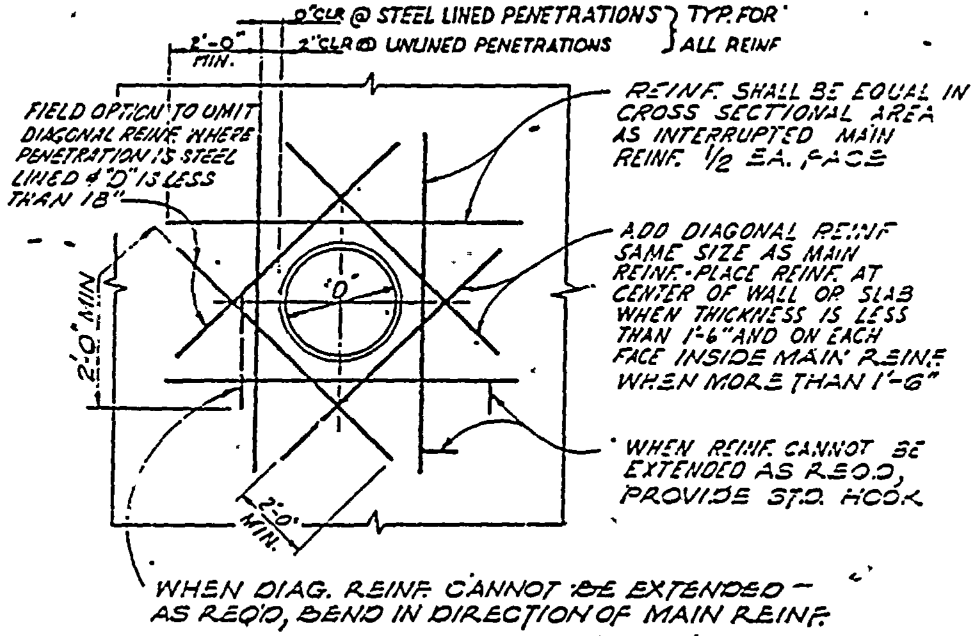
TANK ROOF - TO - CYLINDER JUNCTION DESIGN

ATTACHMENT F

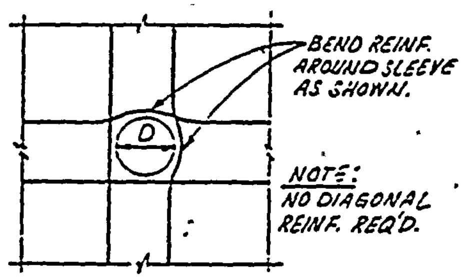


DESIGN OF REBAR PLACEMENT

ATTACHMENT G



REINF. AROUND SLEEVES IN WALLS & SLAB
WHEN "D" IS LARGER THAN MAIN REINF. SPACING



REINF. AROUND SLEEVE IN WALLS OR SLABS
WHEN 'D' IS LESS THAN MAIN REINF. SPACING

PIPING PENETRATION DESIGN

Table 2.5-15

STATIC BEARING CAPACITY OF CATEGORY I STRUCTURES

Structure	Average Static Design Load q_s (k/ft^2)	Ultimate Bearing Capacity q_o (k/ft^2)	Factor of Safety (q_o/q_s)
Containment Building	7.9	35.7	4.5
Auxiliary Building (deep section)	6.2	34.9	5.6
Main Steam Support Structure	7.1	64.8	9.1
Control Building	3.3	45.3	13.7
Fuel Building	5.3	54.9	10.4
Diesel Generator Building	3.1	79.5	25.6
Refueling Water Tank	4.4	90.4	20.5
Condensate Storage Tank	3.5	112.4	32.1

Table 2.5-16

DYNAMIC BEARING CAPACITY OF CATEGORY I STRUCTURES[a]

Structure	Equivalent Uniform Vertical Stress q_d (k/ft^2)	Ultimate Bearing Capacity q_o (k/ft^2)	Factor of Safety (q_o/q_d)
Containment Building	16.1	32.2	2.0
Auxiliary Building (deep section)	10.3	25.8	2.5
Main Steam Support Structure	25.3	60.6	2.4
Control Building	9.8	39.8	4.1
Fuel Building	19.1	50.3	2.6
Diesel Generator Building	5.6	75.5	13.5
Refueling Water Tank	13.2	58.7	4.4
Condensate Storage Tank[b]	13.2	30.2	2.3

a. Based upon maximum dynamic loads derived from analyses described in section 3.7.

b. Condensate storage tank loads were conservatively chosen to be equal to the dynamic design load for the refueling water tank. Actual loads will be less.

489

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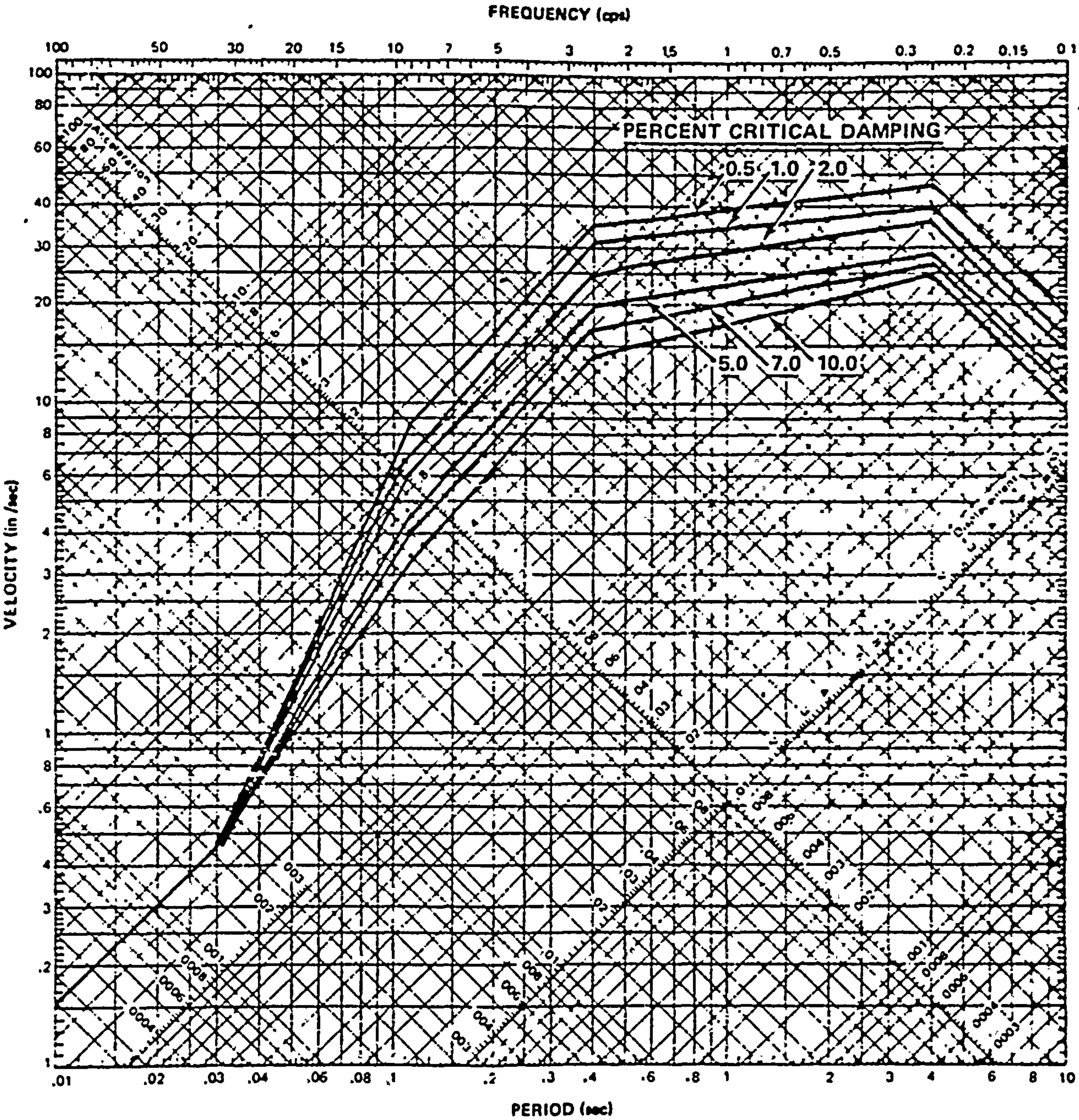
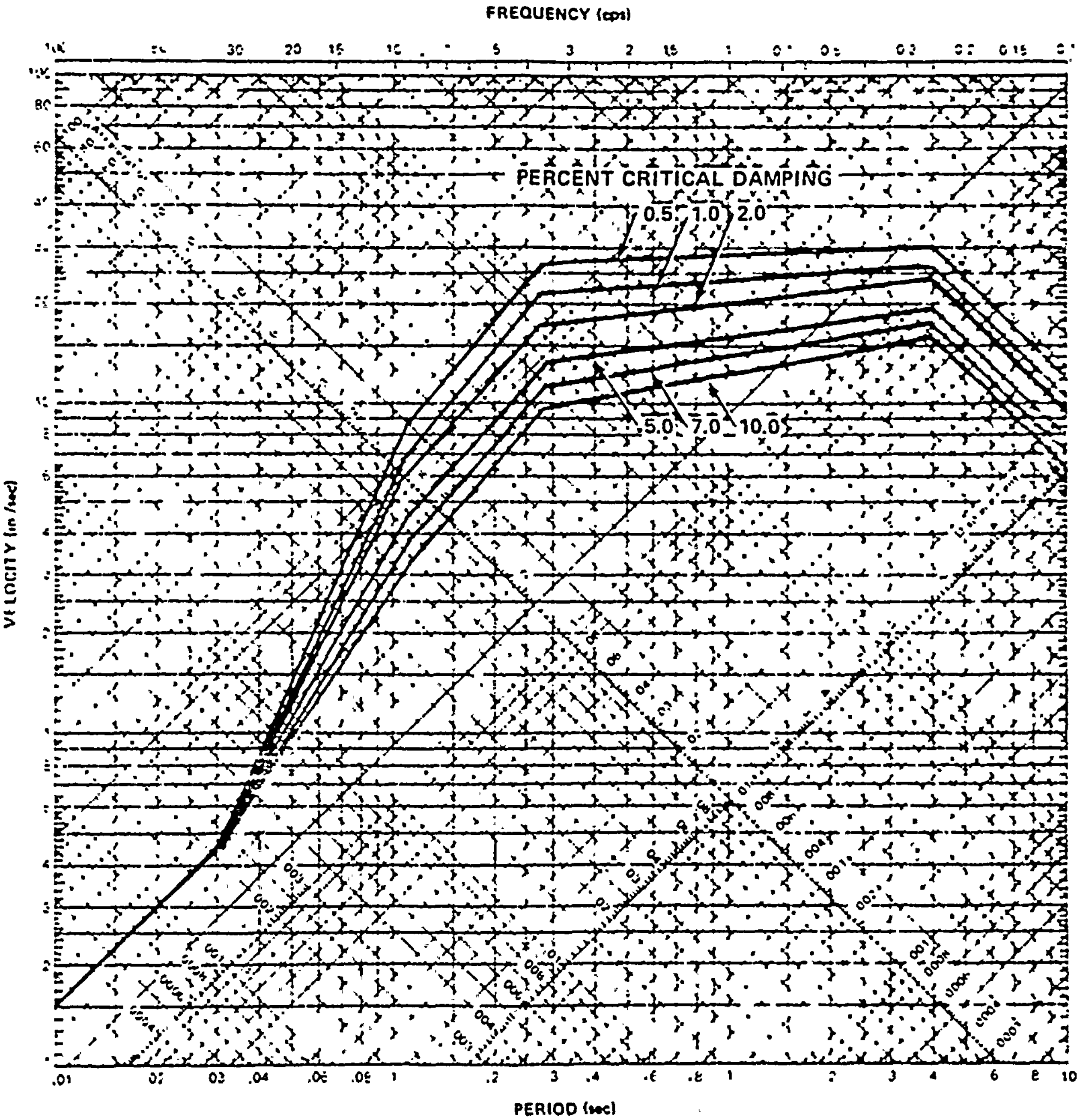


Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR SSE 0.25 g

Figure 3.7-1

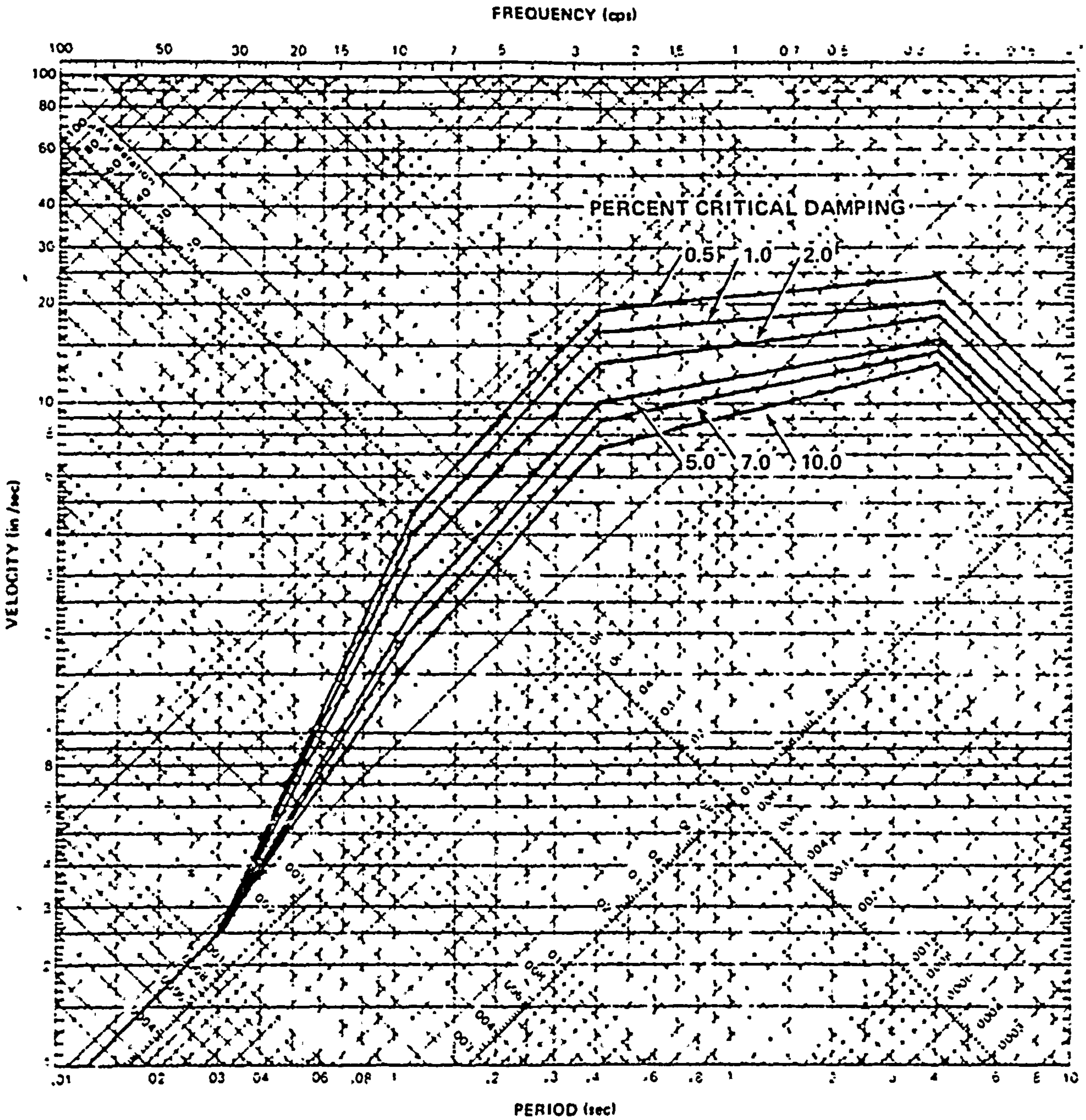


Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR SSE 0.25 g

Figure 3.7-2

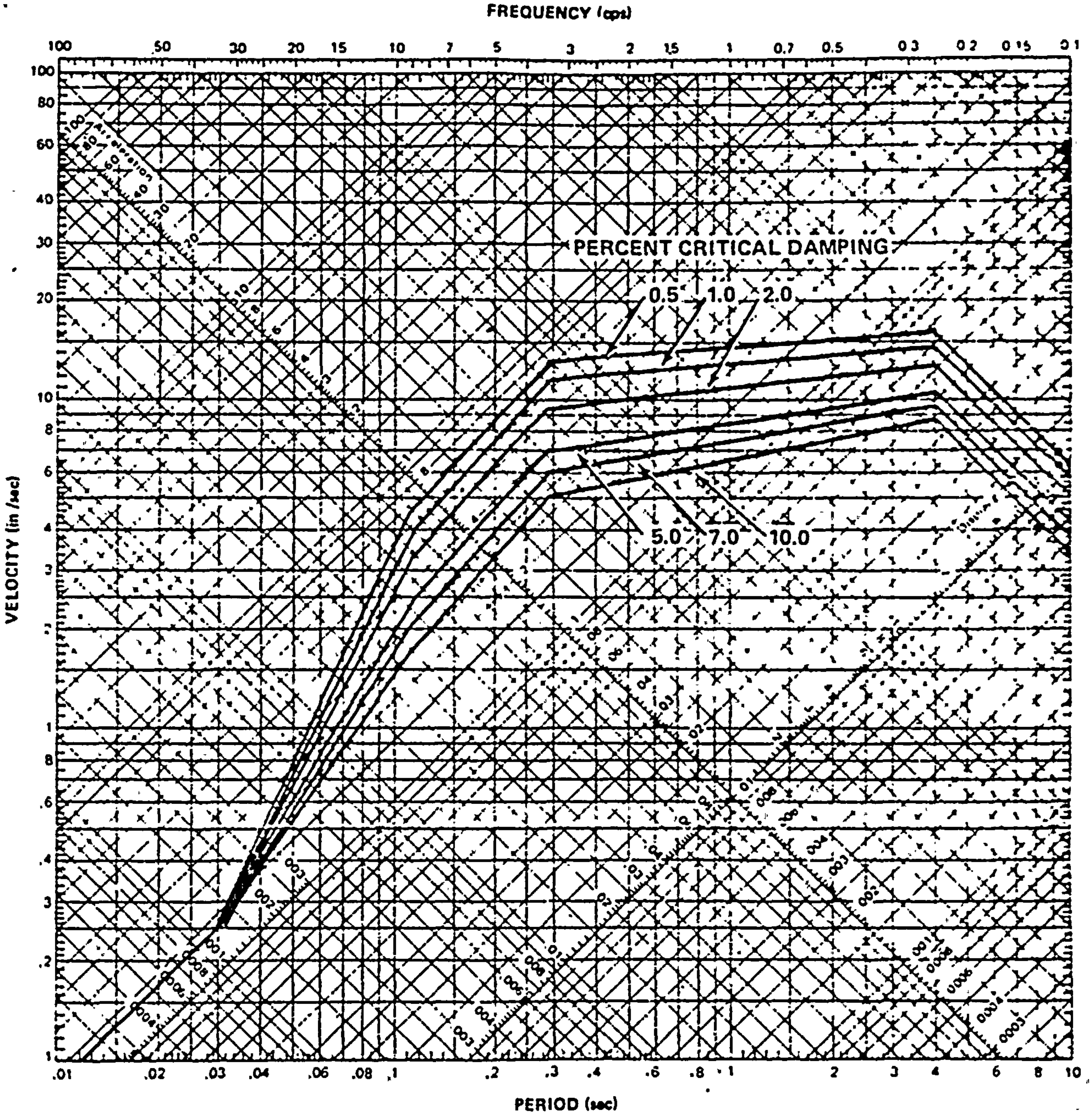
491



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-3



Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-4

3.8.5.5 Structural Acceptance Criteria

The foundations of Seismic Category I buildings are designed to meet the same structural acceptance criteria as the buildings themselves. These criteria are discussed in sections 3.8.1.5, 3.8.3.5, and 3.8.4.5. The limiting conditions for the foundation medium, together with a comparison of actual capacity and estimated structure loads, are found in sections 2.5.4.10 and 2.5.4.11. Computed factors of safety against overturning, sliding, and flotation for Category I structures are given in table 3.8-5. 1

Table 3.8-5
COMPUTED FACTORS OF SAFETY

Structure	Overturning OBE	Overturning SSE	Sliding OBE	Sliding SSE	Flotation
Auxiliary	3200	830	2.2	1.3	4.7
Containment	3400	1200	1.7	1.2	4.5
Control	1500	420	1.6	1.2	4.8
Diesel Generator	1200	400	2.2	1.1	NA[a]
Fuel	1600	400	1.9	1.1	NA
Main Steam Support	340	91	1.6	1.1	NA
Condensate Storage and Refueling Water Tanks	500	150	1.7	1.4	NA

a. Not applicable

PALO VERDE NUCLEAR STATION UNITS 1, 2, 3
DESIGN ADEQUACY AUDIT

STRUCTURAL AUDIT OF SPRAY PONDS
Part I - General Analysis

I. BASIC DESIGN CRITERIA

A. 'g' value - free field

	Seismic level based on construction permit license	Seismic level used in design of structures and equipment
SSE	0.20g	0.25g
OBE	0.10g	0.13g

Reference: FSAR, Section 3.7

B. Spectra (attach figs. for all damping values, ductilities)

1. zero period acceleration

SSE 0.25g
OBE 0.13g

Pages 539-542

Reference: FSAR, Figures 3.7-1 - 3.7-4 and Section 3.7.1.1
This is consistent with REG. GUIDE 1.60

Frequency (or period) interval

Refer to BC-TOP-4A, Section 2.5.1 (c)

C. Damping

Refer to FSAR, Section 3.7.1.3
This is consistent with Reg. Guide 1.61
Refer to FSAR Figures 3.7-5 and 3.7-6 (Pages 216 A & 216B)

D. Artificial time history and corresponding spectra (attach figures)

1. original time history and its composition, i.e., rising time, strong motion and tail end.

A time history was not used in the design of the Spray Ponds and Spray Pond Pump House.

2. base line correction, check the integrated velocity and displacement time histories

3. time interval - compatible with the highest frequency considered in the spectral calculation

E. Motion duration

F. components of motion including their relative motion amplitudes

Analysis was performed for the three principle directions with equal amplitudes.

G. Dead and live loads for various operating floors and base slab

~~Refer to Project Design Criteria Part II, Section 3.0 and Part III, Section 1.4.~~

Dead Load: includes all structures, major equipment load and 50 psf equivalent for small equipment.

Live Load: see action item 3

H. Ground water level

The groundwater design level is at plant El 70'-0". The actual water level is at approximately plant El. 60'-0".

Reference: FSAR, Sections 2.4.13.2.4 and 2.4.13.5.

I. Backfill earth pressure, wind, overpressure due to postulated external explosions (as applicable)

~~Refer to Project Design Criteria Part II, Sections 3.4.5.3 and 3.5.5 for lateral earth pressure, wind and tornado loads.~~

For lateral earth pressure see pages 208A & 208D
Wind does not govern.

J. Other considerations

The concrete walls and roof slab of the Spray Pond Pump house were designed for missile protection.

II. ANALYSIS METHOD

A. Seismic Analysis

1. Mathematical model-general description with sketch.

See Attachment A. (Pages 534 & 535)

a. parameters used

(1) concrete modulus

E_c = 3605 Ksi f'c = 4000

(2) rebar modulus and yield strength

E_s = 29000 Ksi fy = 60 Ksi

(3) Poisson's ratio

ν = .24

~~Reference: Project Design Criteria~~

(4) damping

Refer to FSAR, Section 3.7.1.3.

This is consistent with Reg. Guide 1.61

(5) properties of foundation materials.

- shear modulus

  Refer to FSAR, Figures 3.7-7, -8, -9 (Pages 543-545)

- subgrade reactions

  Coefficient of Subgrade Reaction: 30 K/Ft^3

- bearing capabilities

  Static Capacity: 8 KSF

  Dynamic Capacity: 17.2 KSF

(6) other parameters

b. stiffness calculations

(1) exterior walls

Stiffness calculations were performed manually using standard engineering methods.

(2) interior walls

Same as exterior walls.

2. method of Analysis

a. method of analysis used (time history, response spectrum methods, etc.) and consideration of torsional and translational response

(1) general description

A response spectrum analysis was performed for each of the two principle horizontal directions of the Spray Pond Pump House. Due to the rigidity of the pump house in the vertical direction, the ZPA from the free-field was used. The calculations were performed manually.

(2) findings and comments

b. selection of number of masses and degrees of freedom

(1) general description

Each of the two planar models consisted of two masses with two degrees of freedom.

(2) findings and comments

c. number of modes considered

(1) general description

N-S 2 modes

E-W 2 modes

(2) findings and comments

d. combining modal responses

(1) actual procedures used

Refer to FSAR, Section 3.7.2.7.

(2) general findings

e. consideration of three components of motion

Refer to FSAR, Section 3.7.2.6.

(1) actual procedures used

The component factor method was used to combine the three components of motion.

It is consistent with Reg Guide 1.92

(2) general findings

f. consideration of soil-structure interaction

Soil - structure interaction was not considered.

(1) general description

(2) findings and comments

g. decoupling criteria for subsystems

(1) general procedure

Refer to BC-TOP-4A, Section 3.2.

(2) key examples

(3) The other criteria pertaining to frequency ratio as defined in SRP 3.7.3.II.3.b are also met

g) general findings and comments

h. modeling of hydrodynamic effects in spent fuel pool

The Spray Pond Pump House and Spray Ponds were analyzed to withstand hydrodynamic effects in accordance with U.S. AEC TID 7024, Nuclear Reactors and Earthquakes.

i. modeling of spent fuel pool wells and interior floor slabs and equipment thereof

Not applicable.

3. development of in-structure response spectra

In-structure Response Spectra was not developed for the Spray Ponds and Pump House. The Pump Vendor was given generic In-structure response spectra for the design of the pump. See Page 545-A

a. general procedures

(1) smoothing (describe specific smoothing method used)

(2) peak widening

b. typical results (attach figures)

B. Stress Analysis

1. shear walls and floors

a. mathematical model - general description w/sketch

Shear wall and floor stresses are computed by performing standard manual calculations. Vertical loads are distributed in the structure by conventional methods. Lateral loads were obtained from the response spectrum analysis. These lateral loads are then distributed among the shear walls according to their relative stiffness and location.

b. method of analysis--incorporation of torsion

The torsional moment is determined by resolving the couple due to eccentricity between the center of mass and center of rigidity at each floor. Shear derived from this torsional moment is added directly to the forces considered for the individual shear walls.

c. load combinations

Refer to FSAR, Section 3.8.3.3.

1. foundation mat

   a. mathematical model - description of boundary conditions

   b. method of analysis

      Manual calculations using "Beam on Elastic Foundation" theory were performed to determine the design moments and forces.

   c. load combinations

      Refer to FSAR, Section 3.8.3.3.

   d. key results (figures, etc.)

      Spray Pond Pump House Basemat:

      M_u = 128 Ft-K/Ft

      V_u = 54 K/Ft

      Basemat of Pond Perimeter Wall

      M_u = 83 Ft-K/Ft

      V_u = 12 K/Ft

3. Material to protect against structure - structure interaction

Not applicable. The Spray Ponds and Pump House do not have any other structures adjacent to them.

a. mechanical properties

b. additional pressure on walls

c. findings and comments

4. vertical dynamic analysis

The structure was analyzed.for the free field vertical accelerations due to its rigidity.

b. development of stiffnesses, including floor stiffness, as applicable

c. method of analysis

Manual calculations were used in the analysis.

C. Computer Programs Used in Analysis

None.

1. assumptions and limitations

2. applicability

3. verification

   - sensitivity study in case of numerical solutions (e.g., finite element analysis)

4. load input (include all cases)

5. output (include all cases)

6. other discussions

D. Overall Stability

1. forces and moments from seismic analysis

Forces and Moments acting at top of Pump House basemat:

	OBE	SSE
Shear	273 K	519 K
Overturning Moment	7977 Ft-K	15156 Ft-K

2. various cases considered

Seismic event loading combinations considered SSE or OBE applied in the North-South, East-West, and vertical directions simultaneously.

3. bearing pressure versus bearing capability and safety factor against bearing failure

For Pump House Foundation:

	Actual	Capacity
Static	3.6 KSF	8 KSF
Dynamic	6.9 KSF	17.2 KSF

4. factors of safety

a. sliding

The Pump House Foundation is integral with the Spray Pond basemat. Due to the size of the Spray Pond basemat, sliding was not considered to be a problem.

b. overturning

Factor of Safety = 3.8 (SSE)

E. Interaction of Non-category I Structures with the Structure Considered

1. identification of pertinent non-Category I structures

   None. There are no non-Category I structures close to the Spray Ponds or Pump House.

2. consideration given to potential failure of non-Category I systems on Category I systems

3. general findings and comments

F. Design Consideration for Tornado Missiles

1. design requirements

Refer to FSAR, Table 3.5-8. (Page 538)

2. models for

a. local damage

Refer to FSAR, Section 3.5.3

b. overall response

Refer to FSAR, Section 3.5.3.

3. load combinations

Refer to FSAR, Section 3.8.3.3.

4. forces

The governing structural members are of sufficient thickness to preclude perforation by postulated missiles and maintain structural integrity.

6. general comments and preliminary audit findings

III. CONFORMANCE TO ACCEPTABLE CRITERIA

A. Identification of deviations, if any

None.

B. Justification of deviations and disposition of the deviations

D. general comments

Part II Audit of Key Designs

A. Exterior Shear Walls

1. design requirements

The exterior walls of the Pump House are designed to satisfy structural requirements as bearing walls, shear walls and protection against tornado missiles.

2. design loads (from general analysts)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

M = 1195 Ft-K

P = 357 K

4. detailed design of rebar placement at key sections

See Attachment B. (page 536)

5. general comments and preliminary audit findings

B. Interior Shear Walls

1. design requirements

Same as exterior walls.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

The forces and moment in the interior walls are less than those of the exterior walls. The reinforcement is the same for both walls.

4. detailed design of rebar placement at key sections

See Attachment B. (page 536)

5. general comments and preliminary audit findings

C. Main Floors and Roofs

1. design requirements

Main floors and roofs were primarily designed for dead, live, and seismic loads, ~~as defined in the Project Design Criteria~~. Roofs are also designed to satisfy minimum thickness to preclude perforation by tornado generated missiles.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

For elevated slab:

M_u = 34 Ft-K/Ft

V_u = 13 K/Ft

4. detailed design of rebar placement at key sections

See Attachment B. (page 536)

5. general comments and preliminary audit findings

D. Steel Structural Bracing Systems (if any)

1. design requirements

The Spray Pond Pump House floor and roof slabs are supported by structural steel beams which are primarily designed for construction loads.

2. design loads

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

	Moment	Shear
El. 120′ - 7-1/2"	35 Ft -K	8.6 K
El. 105′ - 7-1/2"	22 Ft -K	8.5 K

6. general comments and preliminary audit findings

E. Foundation Mat

1. design requirements

- Refer to FSAR, Sections 3.8.5.4.2 and 3.8.5.5.

2. design loads (from general analysis)

   Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

   Basemat of perimeter wall: M_u = 83 Ft-K/Ft   V = 12 K/Ft

   Basemat of Pump House: M_u = 128 Ft-K/Ft   V = 54 K/Ft

4. detailed design of rebar placement at key sections

   See Attachments B and C. (pages 536 & 537)

5. general comments and preliminary audit findings

F. Main Frame Concrete Column Design (Key Columns)

Not applicable. There are no concrete columns in this structure.

1. design requirements

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design of rebar placement at key sections

5. general comments and preliminary audit findings

G. Secondary Floors

Not applicable.

1. design requirements

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design of rebar placement at key sections

5. general comments and preliminary audit findings

H. Detailing at Floor-Wall Joints

1. design requirements

As per ACI 318-71 Code, Chapter 6 and 17.

2. design loads (from general analysis)

Refer to FSAR, Section 3.8.3.3.

3. forces and moments at key sections

M_u = 34 Ft-K/Ft

V_u = 12 Ft-K/Ft

4. detailed design of rebar placement at key sections

See Attachments B and C. (pages 536 & 537)

5. general comments and preliminary audit findings

I. Dynamic Effects Applied to Floors and Walls by Machinery

1. design requirements

The natural frequency of the floor/structure support is about twice that of the operating frequency of the pump/motor units. Since resonance is not a problem, dynamic effects have been neglected.

2. design loads (from general analysis)

3. forces and moments at key sections

4. detailed design

5. general comments and preliminary audit findings

J. Crane & Support

1. design of bents (columns and roof trusses)

Not applicable. There are no cranes in this structure.

a. design requirements

b. design loads (from general analysis)

c. forces and moments at key sections

d. detailed design

e. general comments and preliminary audit findings

2. design of girders supporting crane rails

Not applicable.

a. design requirements

b. design loads (from general analysis)

c. forces and moments at key sections

d. detailed design

e. general comments and preliminary audit findings

3. design of spent fuel bridge

   Not applicable.

   a. design requirements

   b. design loads (from general analysis)

   c. forces and moments at key sections

   d. detailed design

   e. general comments and preliminary audit findings

K. Fuel Pool Liner Design

Not applicable.

1. stresses and strain controls

2. conformance to code requirements

3. analysis procedure and results

4. consideration of accidental drop of crane loads

5. corrosion effects (e.g., pitting) on liner integrity

6. preliminary findings of audit results

ATTACHMENT A

ESSENTIAL SPRAY POND CONCRETE WALLS

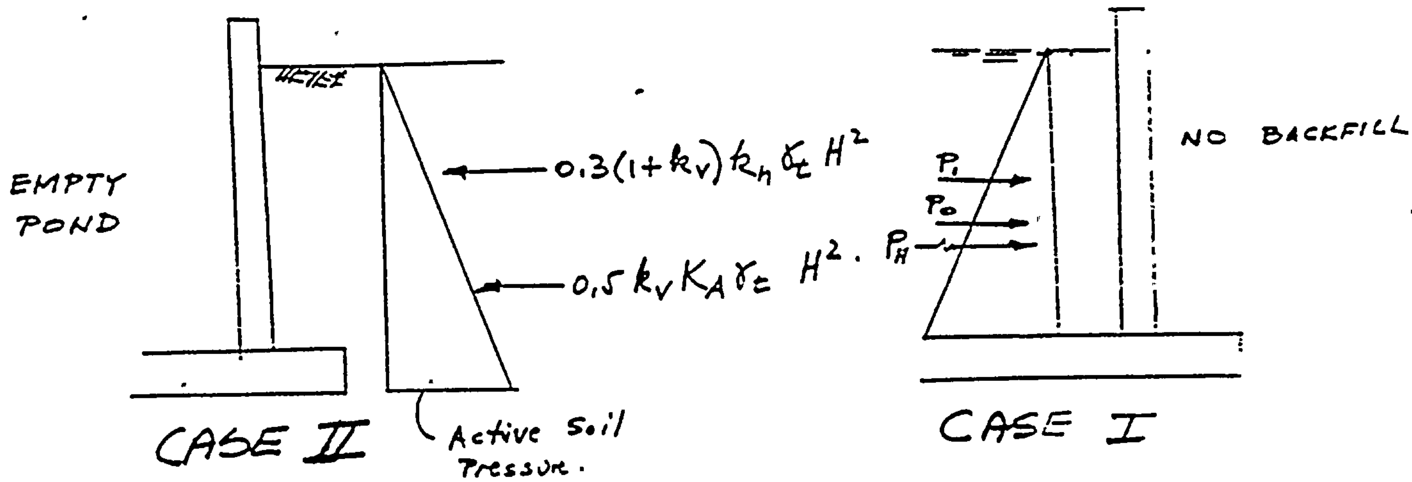
The reinforced concrete walls and their connections to the base slab were designed manually as free standing cantilevered walls. Two load cases were considered (see Sheet 2 for sketch):

1) Hydrostatic Pressure - included hydrodynamic effects under an OBE or SSE. In this analyses, the presence of the soil embankment outside the wall was conservatively neglected.

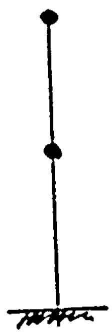
2) Loading from the active soil pressure included effects from an OBE or SSE. The pond was assumed to be empty for this load case. The dynamic lateral forces were determined in accordance with the Project Design Criteria Part II, Section 3.4.5.3.

ESSENTIAL SPARY POND PUMP HOUSE

Two lumped parameter planar models were used for a response spectrum dynamic analysis. See Sheet 2 for a sketch of the model.

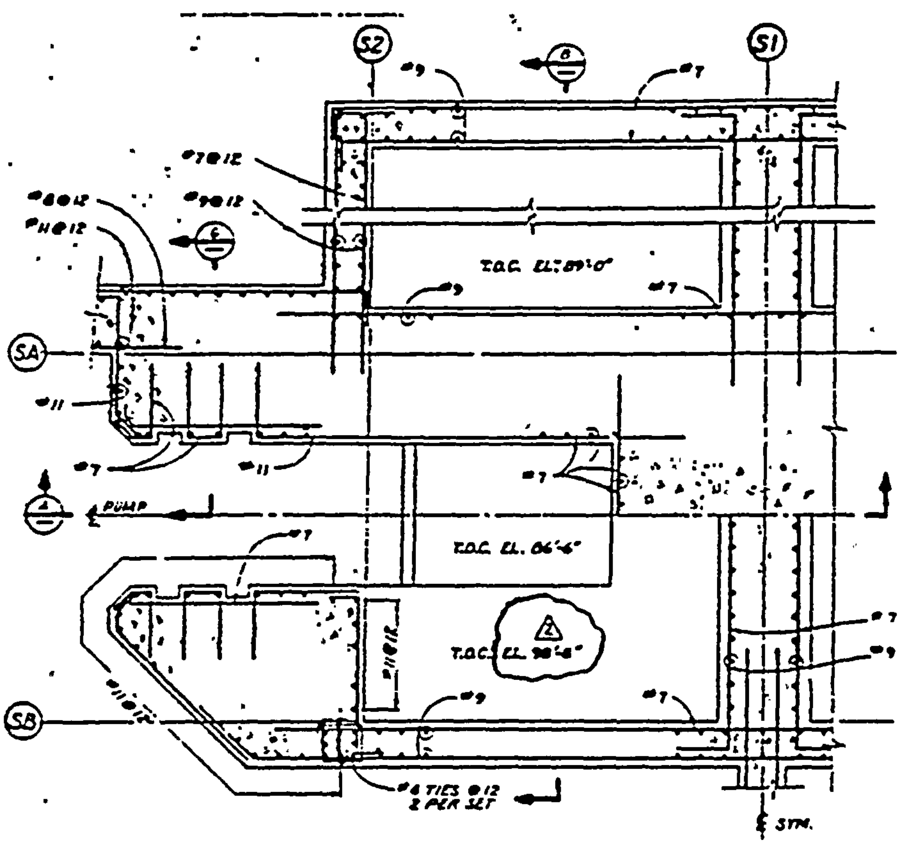


CONCRETE WALL MODELS

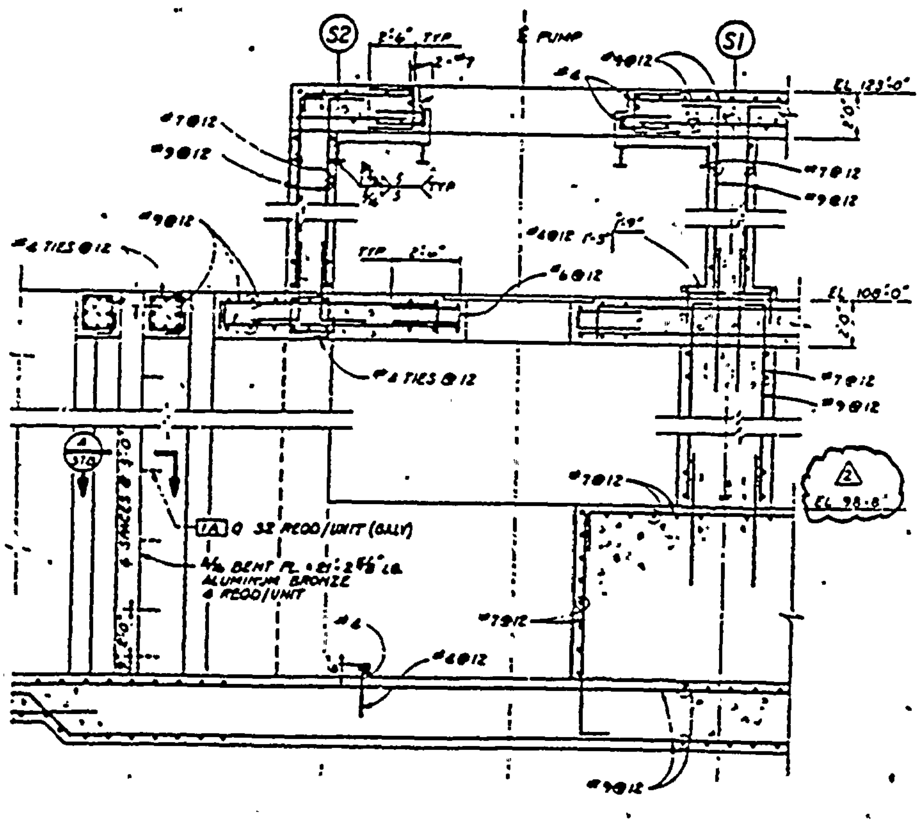


PUMP HOUSE LUMPED PARAMETER MODEL

ATTACHMENT B
DETAILS OF
SPRAY POND
PUMP HOUSE



PLAN



SECTION A
3/8" = 1'-0"

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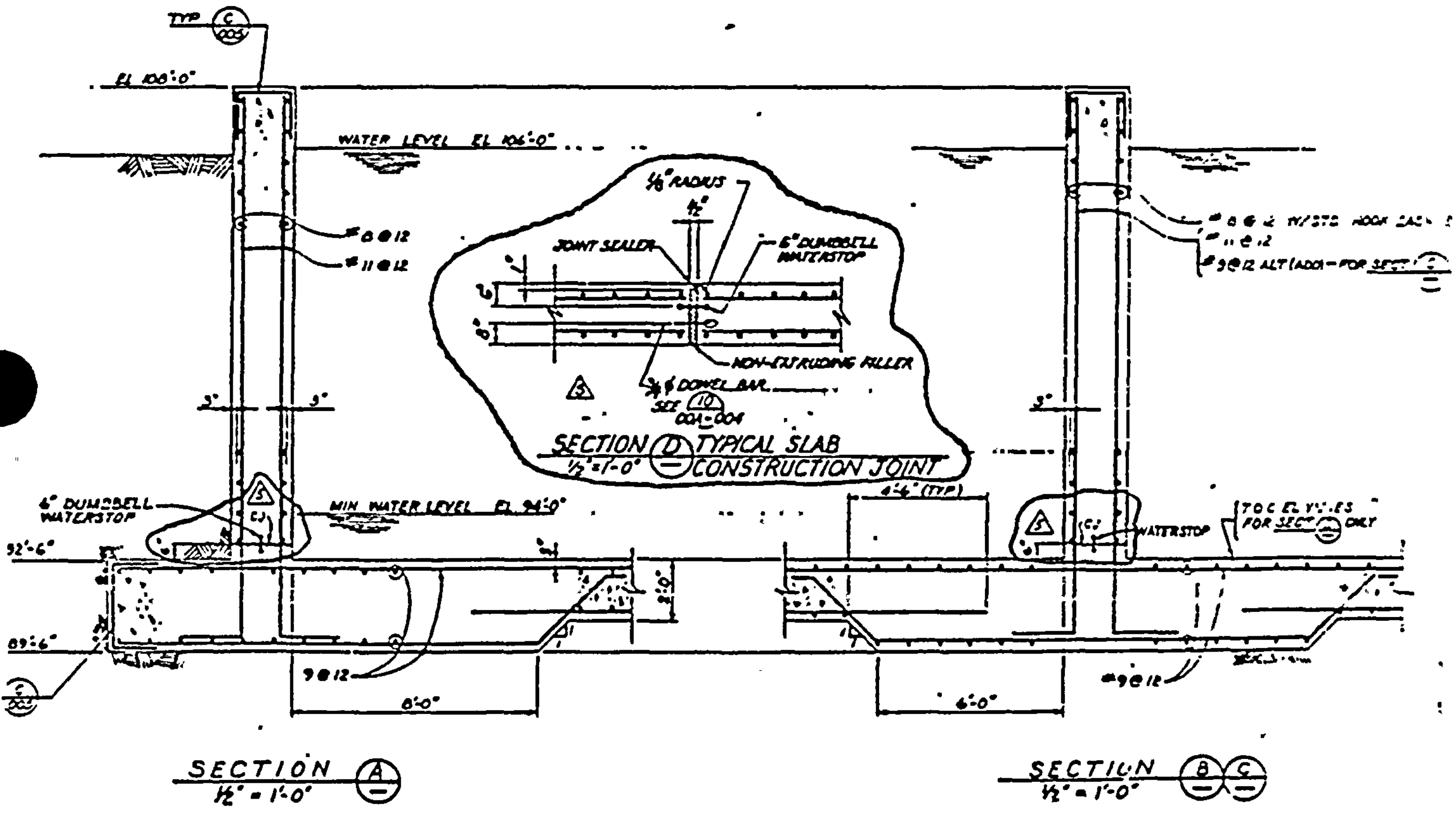
ATTACHMENT C



DETAILS OF SPRAY PONDS

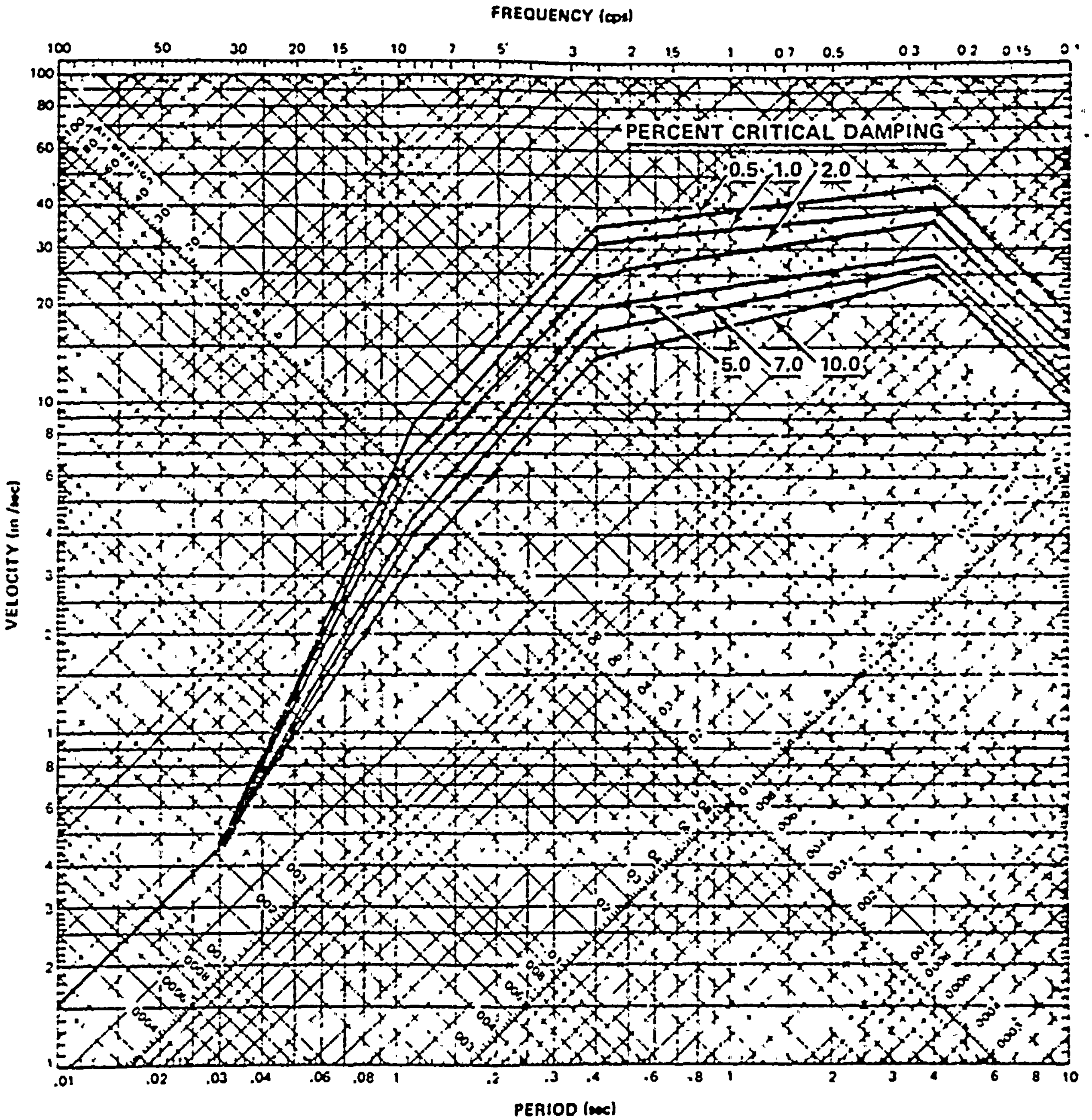
Table 3.5-8

TORNADO-GENERATED MISSILES CONSIDERED IN DESIGN OF SAFE SHUTDOWN STRUCTURES

Description	Weight (lbs)	Impact Area (ft^2)	Maximum Velocity (ft/s)	Kinetic Energy (ft-lbs)
(A) A 12-foot wood plank, 4 x 12 inches in cross-section, traveling end on at a speed of 240 mi/h.	200	0.333	352	3.85×10^5
(B) A steel pipe, Schedule 40, 3 inches in diameter by 10 feet long, traveling end on at 120 mi/h.	78	0.063	176	3.75×10^4
(C) A steel rod, 1 inch in diameter, 3 feet long, traveling end on at 180 mi/h.	8	0.005	264	8.66×10^3
(D) A steel pipe, Schedule 40, 6 inches in diameter by 15 feet long, traveling end on at 120 mi/h.	285	0.24	176	1.37×10^5
(E) A steel pipe, Schedule 40, 12 inches in diameter by 15 feet long, traveling end on at 120 mi/h.	743	0.886	176	3.57×10^5
(F) A utility pole, 13-1/2 inches in diameter, 35 feet long, traveling end on at 120 mi/h.	1490	0.994	176	7.17×10^5
(G) An automobile of 4,000 pounds weight, striking the structure at 60 mi/h.	4000	20.0	88	4.81×10^5

17

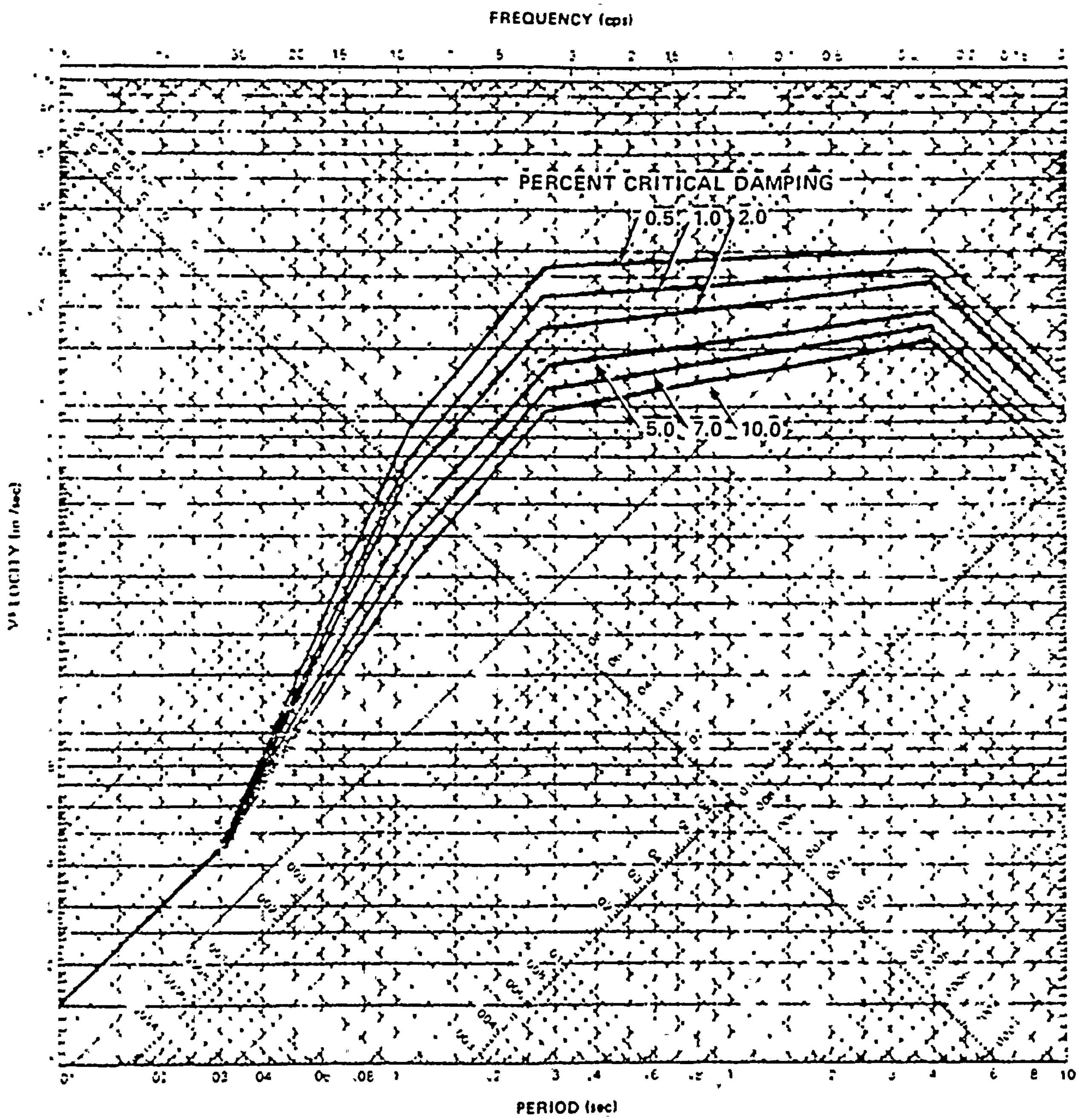
538



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR SSE 0.25 g

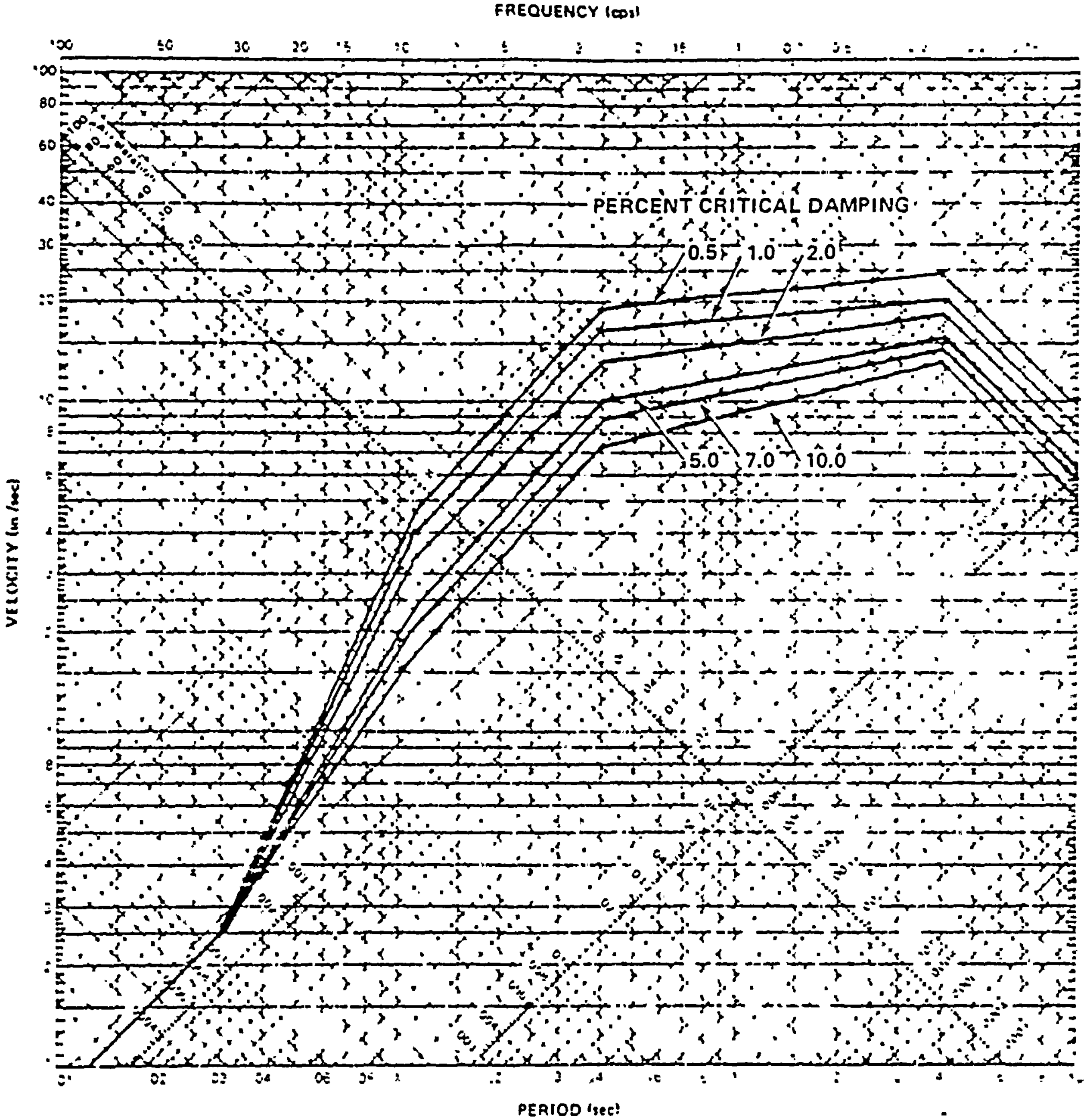
Figure 3.7-1



Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR SSE 0.25 g

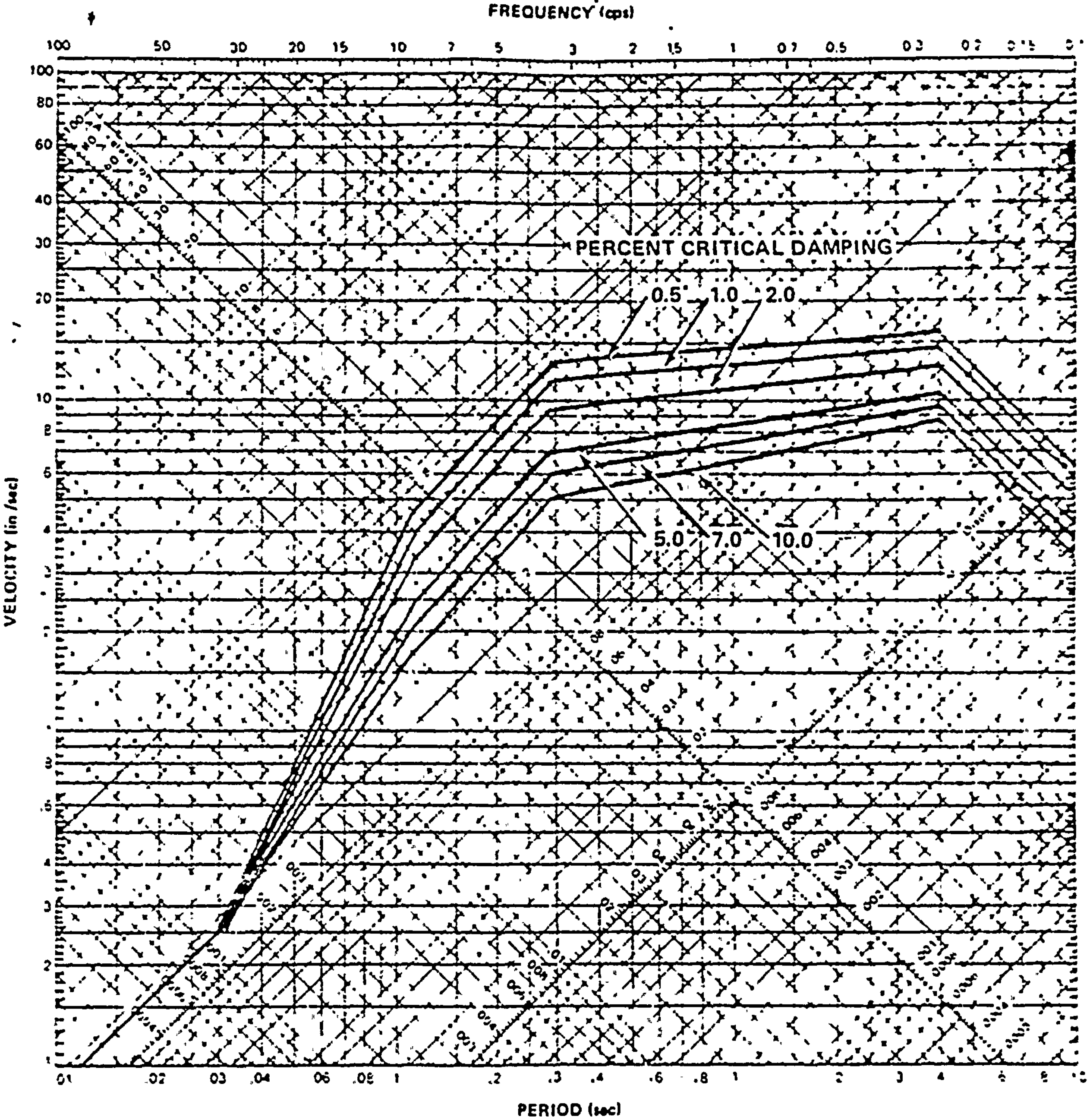
Figure 3.7-2



Palo Verde Nuclear Generating Station
FSAR

HORIZONTAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-3

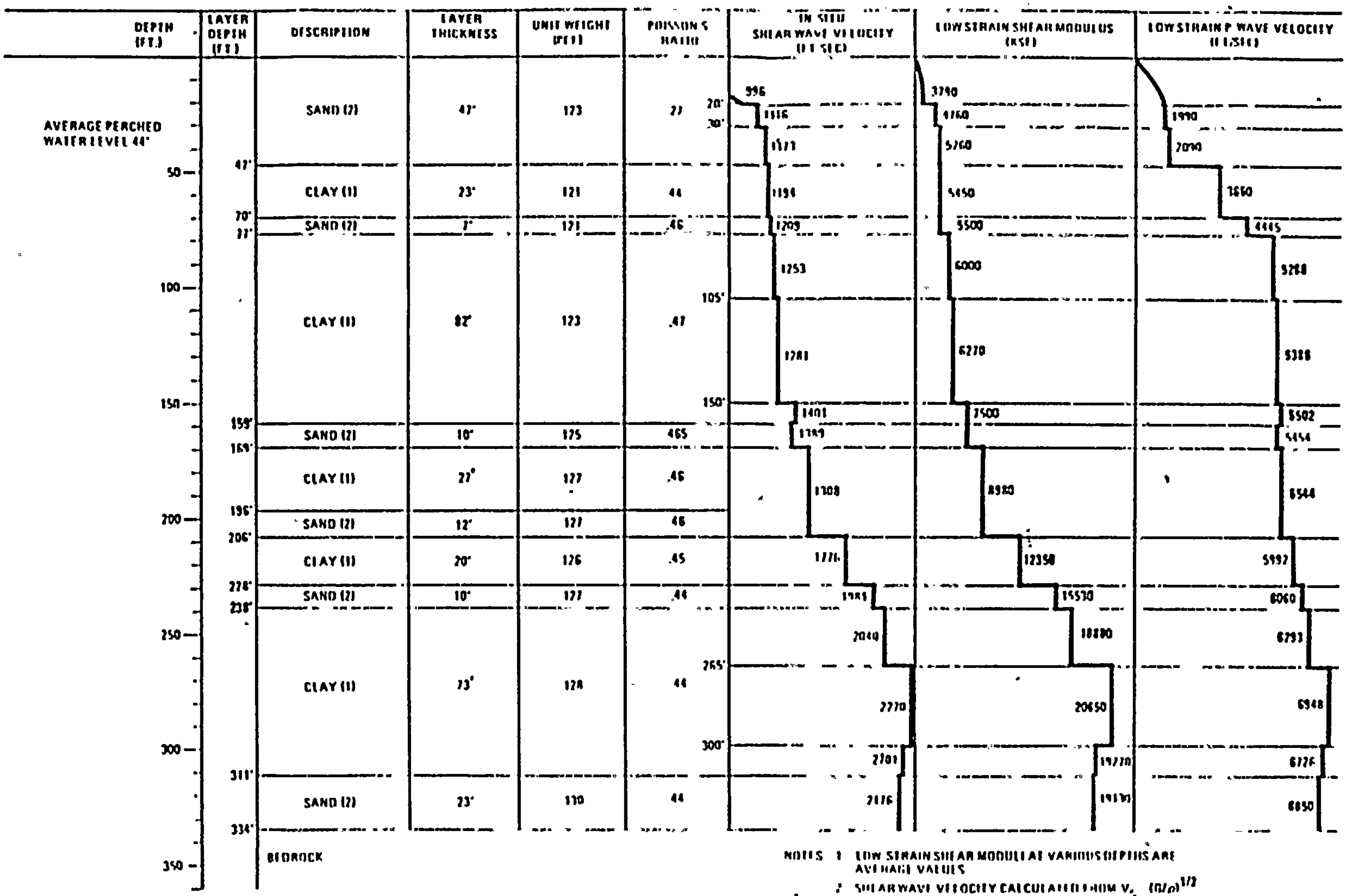


Palo Verde Nuclear Generating Station
FSAR

VERTICAL DESIGN SPECTRA
FOR OBE 0.13 g

Figure 3.7-4

542



NOTES 1 LOW STRAIN SHEAR MODULI AT VARIOUS DEPTHS ARE AVERAGE VALUES
2 SHEAR WAVE VELOCITY CALCULATED FROM V_s [illegible] $(G/\rho)^{1/2}$
3 P WAVE VELOCITY CALCULATED FROM V_p [illegible]

Palo Verde Nuclear Generating Station
FSAR

SOIL PROFILE

FIGURE [illegible]

CHANGE

82

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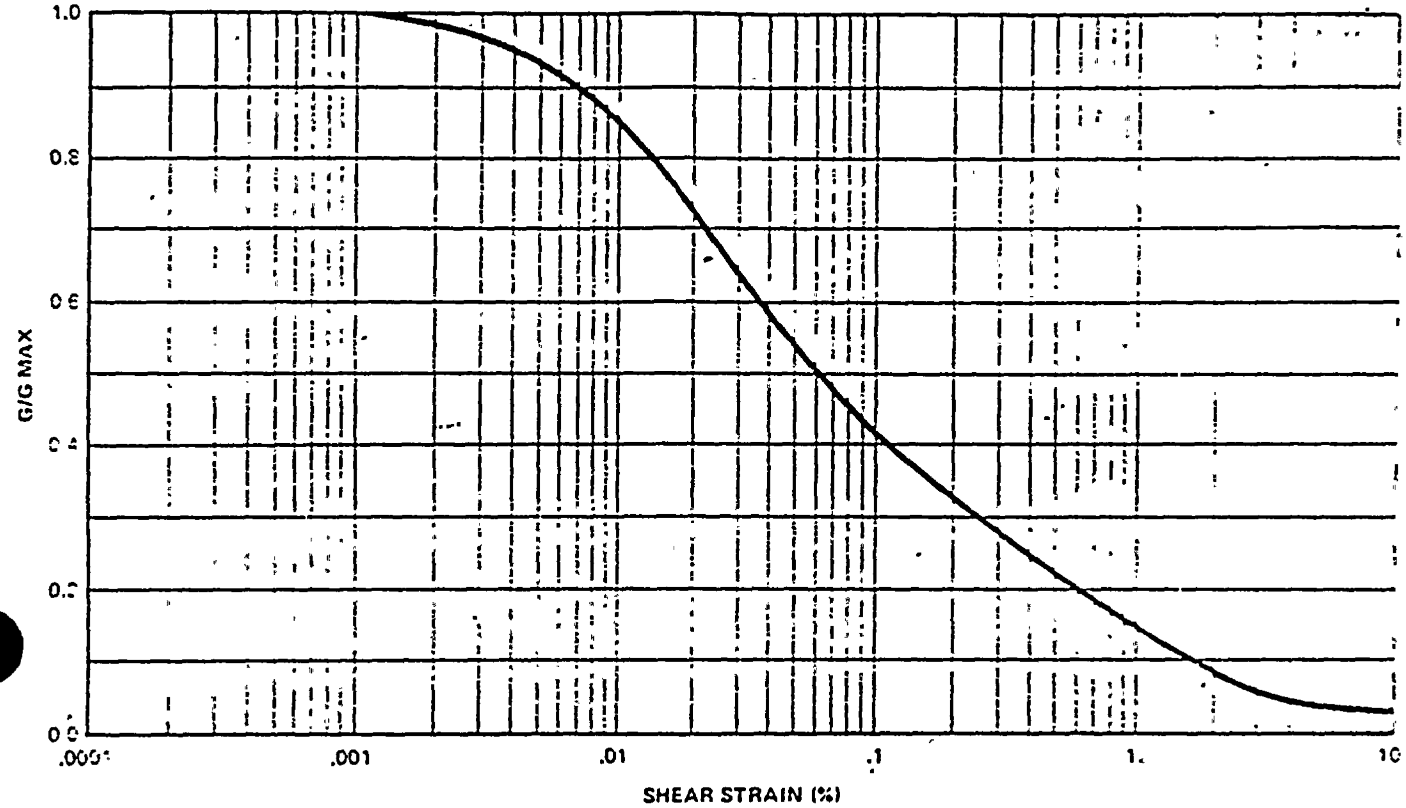
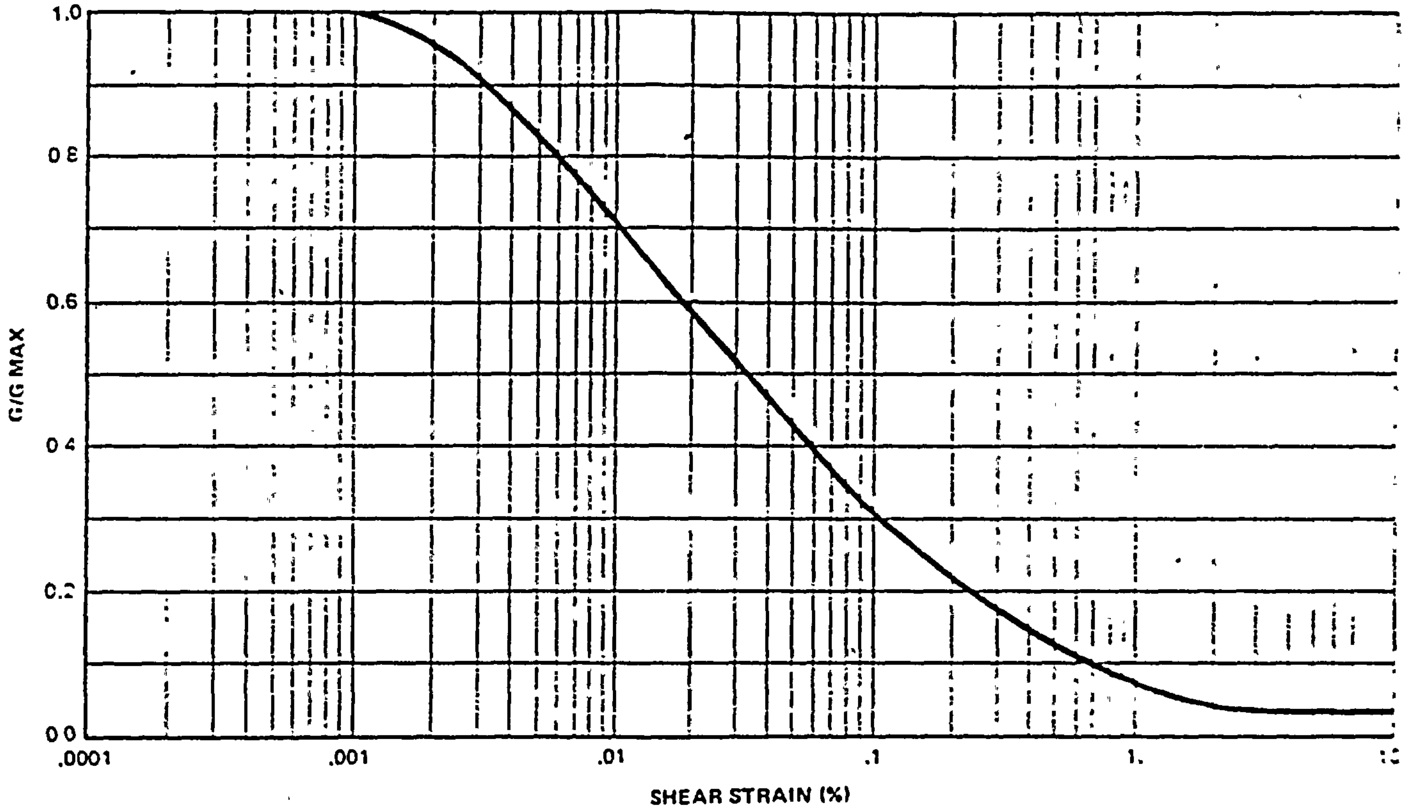


Palo Verde Nuclear Generating Station
FSAR

SHEAR MODULUS VS. STRAIN - CLAY

FIGURE 3.7-8



Palo Verde Nuclear Generating Station
FSAR

SHEAR MODULUS VS. STRAIN - SAND

FIGURE 3.7-9

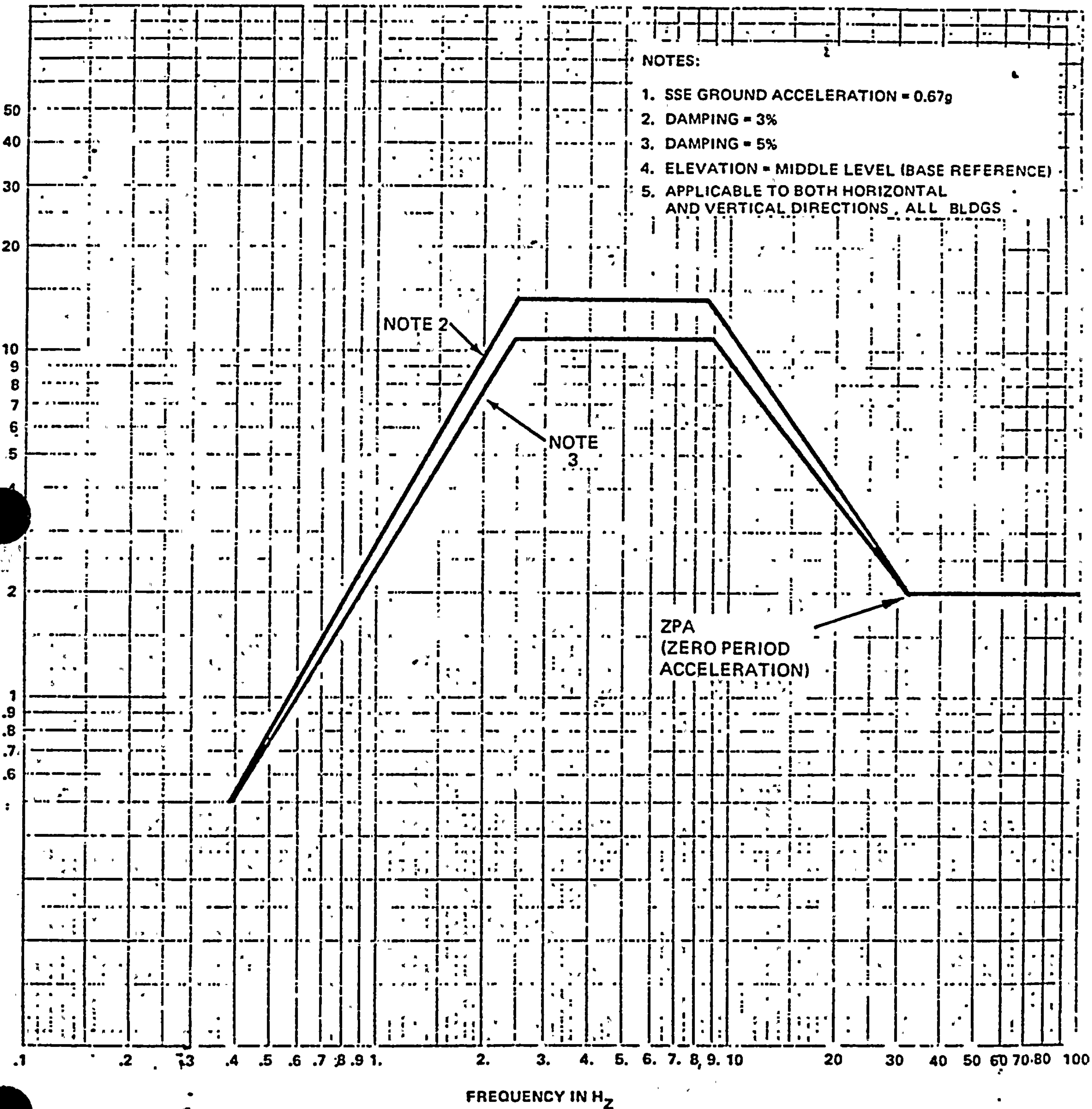


FIGURE 4E-1
REQUIRED RESPONSE SPECTRUM (RRS) FOR THE MAJORITY OF NUCLEAR POWER PLANT LOCATIONS IN THE CONTINENTAL UNITED STATES.

10407-85H
05MY5-13