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April 28, 1980

Director of Nuclear Reactor Regulation
Attention: Mr. D. L. Ziemann, Chief
Operating Reactors Branch No. 2
Division of Operating Reactors
U.S. Nuclear Regulatory Commission
Washington, D. C. 20555

Gentlemen:

Subject: Docket 50-206
Seismic Reevaluation Program
San Onofre Nuclear Generating Station, Unit 1

At a meeting with the NRC Staff on March 20, 1980, SCE described a program for initiating the seismic reevaluation of the balance of plant structures and equipment at San Onofre Unit 1. This program is provided as Enclosure 1 to this letter. It should be noted that this program consists of the analysis of structures and the development of instructure response spectra. It is SCE's intention to work with the NRC staff in parallel with the program in Enclosure 1 to develop the scope and schedule of the reevaluation program for the balance of plant equipment at San Onofre Unit 1.

Enclosure 2 provides a discussion of the adequacy of the seismic design of San Onofre Unit 1 during the time that the seismic reevaluation is being performed. A number of independent items are discussed including the conservatism of the design basis response spectrum, the capability of structures and equipment to withstand ground motions higher than the original design basis, the ability of structures to withstand instrumental ground motion significantly larger than the design ground motion, and the low probability of experiencing large ground motion at the San Onofre site. Based on the combination of all these independent considerations, it is concluded that San Onofre Unit 1 can continue to operate during the time required to perform the seismic reevaluation without undue risk to the health and safety of the public.

If you have any questions on this information, please let me know.

Sincerely,

ORIGINAL SIGNED

Enclosures

Enclosure 1

SCOPE OF SEISMIC REEVALUATION PROGRAM

SAN ONOFRE NUCLEAR GENERATING STATION, UNIT 1

The purpose of this document is to describe the tasks to the performed during the initial thirteen month period following recommencement of San Onofre Unit 1 seismic reevaluation work in conjunction with the seismic review topic of the Systematic Evaluation Program (SEP).

Any additional seismic review work which may have to be subsequently performed by SCE upon completion of the tasks described herein will be separately addressed as the SEP requirements are identified.

The tasks to be performed are as follows:

A. Identify the remaining plant features which are subject to seismic review (including the identification and evaluation of potential applications of the declassification of equipment items).

B. Perform a seismic reevaluation of the remaining safety related structures:

   1) Identify the SEP interactive effects to be considered.

   2) Establish the seismic review/reevaluation approach to be employed.

   3) Establish the specific reevaluation criteria and analysis methods to be employed.

   4) Complete the mathematical models of the structures to be analyzed.

   5) Perform the analyses of the structures and evaluate the results.

   6) Perform time history analyses, as required, to develop instructure response spectra for the safety related structures.

A description of these tasks is provided in Appendix A. A preliminary schedule for this work is shown in Figure 1.

APPENDIX A

DESCRIPTION OF TASKS FOR
SEISMIC REEVALUATION OF SAN ONOFRE UNIT 1

A. IDENTIFICATION OF REMAINING PLANT FEATURES WHICH ARE SUBJECT TO SEISMIC REVIEW.

Certain of the San Onofre Unit 1 structures, systems, components and equipment have either been seismically reevaluated, or designed in accordance with current seismic design criteria as part of recent plant modification projects, as identified in Appendix B. The purpose of this task is to develop a listing of plant structures, systems, components, and equipment which will be subject to further seismic review in conjunction with the Systematic Evaluation Program (SEP). The draft criteria which was developed for this purpose in conjunction with the suspended balance of plant (BOP) seismic reevaluation program is provided in Appendix B. This draft criteria will be reviewed and revised, as necessary, prior to its use in developing the aforementioned listing.

In conjunction with the preparation of this listing, declassification of certain portions of Seismic Category A structures, components, and systems will be considered. Approaches to this declassification will include (1) isolating portions that are not essential to the safety related performance of the system, (2) demonstrating by safety analysis that the failure of the portion of the structure, component, or system would not result in loss of the safety related function of the structure, component, or system, or (3) demonstrating by safety analysis that the probability and/or consequences of failure of the portion of the structure, component, or system is acceptably low. Draft criteria which were developed for this purpose in conjunction with the suspended BOP seismic reevaluation program is provided in Appendix B. This criteria will be reassessed and revised, as necessary, prior to its use.

B. PERFORM THE SEISMIC REEVALUATION OF STRUCTURES.

As discussed in Appendix B, the containment sphere, reactor building, sphere enclosure building, and diesel generator building have been either reevaluated or designed to current seismic design criteria and will not be subject to SEP seismic review. The remaining safety related structures identified in Task A above will be subject to the seismic review described in the following subtasks.

B.1 IDENTIFY THE SEP INTERACTIVE EFFECTS TO BE CONSIDERED

Published SEP Status Reports have provided some initial identification of NRC and licensee activities in the seismic review process.

The objectives of this task are as follows:

1. Ascertain and assess the scope definition, current status and available documented results of:

   - All preceding and ongoing SEP seismic review activities which have been conducted by the NRC.

   - All NRC performed SEP topic reviews of associated topics (Table 1).

2. Develop a firm understanding of how the seismic review will be integrated with the other SEP review topics and the ultimate integrated assessment of the overall SEP results.

B.2 ESTABLISH THE SEISMIC REVIEW/REEVALUATION APPROACH TO BE EMPLOYED.

Available published SEP schedules evidence:

- A lack of any extensive integration of the seismic design review, as scheduled, with the review of interrelated SEP topics until the scheduled completion of the seismic analysis and evaluation efforts. It, therefore, appears that the current schedules do not provide for or incorporate benefits which could be derived or afforded from the implementation of the seismic review in the context of the integrated SEP effort.

- Dependence on iterative seismic analysis (e.g. preliminary analyses as a scoping task, structural analyses based on preliminary seismic design input, and confirmatory analyses by LLL), as well as subsequent analyses of defined plant changes.

The purpose of this task is to consider various alternative approaches to the seismic review of the structures which could potentially optimize the integration of the subject work into the overall SEP effort, or otherwise would efficiently attain the ultimate objectives of the SEP seismic review effort.

Identification of the specific alternative approaches to be considered is dependent upon the information gained in the conduct of Task B.1. above, as well as a better understanding of the licensing or regulatory restraints or conditions which could be associated with such alternative approaches.

B.3 ESTABLISH THE SPECIFIC REEVALUATION CRITERIA AND ANALYSIS METHODS TO BE EMPLOYED.

The draft criteria for the reevaluation of structures which were prepared in conjunction with the suspended seismic reevaluation program is provided in Appendix C. This draft criteria will be

reassessed to reflect the results of Tasks B.1 and B.2 above as well as a review of NUREG/CR-0098 and any other available NRC endorsed SEP criteria and guidelines which is obtained during the conduct of the above tasks.

B.4 COMPLETE THE MATHEMATICAL MODELS OF THE STRUCTURES TO BE ANALYZED.

The development of mathematical models for San Onofre Unit 1 structures was undertaken in the suspended BOP seismic reevaluation program. These models will be completed and refined as necessary to be consistent with the specific analysis methods which are established for the work.

B.5 PERFORM THE ANALYSES OF THE STRUCTURES

The remaining structures identified in Task A above, will be analyzed and evaluated using the approach, criteria, and methods established in Tasks B.2 and B.3, and the site specific ground motion which is defined by the NRC.

B.6 DEVELOP INSTRUCTURE RESPONSE SPECTRA FOR THE SAFETY RELATED STRUCTURES.

Instructure response spectra which will be needed in conjunction with the seismic review of safety related systems, components, and equipment will be developed by performing, as appropriate, time history analyses of safety related structures.

TABLE 1

SEP TOPICS WHICH MAY INTERACT WITH
THE SEISMIC DESIGN REVIEW OF STRUCTURES

1.	II-1.C	Potential Site Hazards
2.	II-2.A	Severe Weather Phenomena
3.	II-3.A	Hydrologic Description
4.	II-3.B	Flooding Potential and Protection Requirements
5.	II-3.C	Safety Related Water Supply (Ultimate Heat Sink-UHS)
6.	II-4.D	Stability of Slopes
7.	II-4.F	Settlement of Foundations and Buried Equipment
8.	III-2	Wind and Tornado Loadings
9.	III-3.A	Effects of High Water Level on Structures
10.	III-4.A	Tornado Missiles
11.	III-4.B	Turbine Missiles
12.	III-4.C	Internally Generated Missiles
13.	III-4.D	Site Proximity Missiles (Including Aircraft)
14.	III-5.A	Pipe Break Inside Containment
15.	III-7.A	Inservice Inspection (Category I Structures Integrity)
16.	III-7.B	Design Codes, Design Criteria, Load Combinations, and Reactor Cavity Design Criteria
17.	III-7.D	Containment Structural Integrity Tests
18.	III-10.B	Pump Flywheel Integrity
19.	V-7	Reactor Coolant Pump Overspeed
20.	IX-1	Fuel Storage

Figure 1 **San Onofre Unit 1 Seismic Reevaluation Program Schedule**

Activity	Months 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	1
A. IDENTIFY THE REMAINING PLANT FEATURES WHICH ARE SUBJECT TO SEISMIC REVIEW (INCLUDING THE IDENTIFICATION AND EVALUATION OF POTENTIAL APPLICATIONS OF THE DECLASSIFICATION OF EQUIPMENT ITEMS).																
B. PERFORM A SEISMIC REEVALUATION OF THE REMAINING SAFETY RELATED STRUCTURES.																
1) IDENTIFY THE SEP INTERACTIVE EFFECTS TO BE CONSIDERED.																
2) ESTABLISH THE SEISMIC REVIEW/REEVALUATION APPROACH TO BE EMPLOYED.																
3) ESTABLISH THE SPECIFIC REEVALUATION CRITERIA AND ANALYSIS METHODS TO BE EMPLOYED.																
4) COMPLETE THE MATHEMATICAL MODELS OF THE STRUCTURES TO BE ANALYZED.				+												
5) PERFORM THE ANALYSES OF THE STRUCTURES AND EVALUATE THE RESULTS.																
6) PERFORM TIME HISTORY ANALYSES, AS REQUIRED, TO DEVELOP INSTRUCTURE RESPONSE SPECTRA FOR THE SAFETY RELATED STRUCTURES.																

Reactor Containment Bldg
Other Structures

+Proceeding with Task B.5 at Month 4 is dependent on receiving NRC concurrence on criteria developed in Task B.3 by the end of Month 3.

APPENDIX B

DRAFT CRITERIA* FOR IDENTIFICATION OF
REMAINING PLANT FEATURES WHICH ARE SUBJECT TO SEISMIC REVIEW

I. Identification of Structures, Systems, Components, and Equipment Which Have Been Either Seismically Reevaluated, or Designed In Accordance With Current Seismic Design Criteria.

A. Structures (including the containment sphere and reactor building), piping (the primary reactor coolant system piping), and components (steam generators, reactor coolant pumps, pressurizer, and reactor vessel) which were seismically reevaluated as part of the Seismic Backfit Project (SBP), as discussed in Reference 1.

B. Structures, systems, components and equipment, which have been designed in accordance with current seismic design criteria as part of:

- o Standby Power Addition Project (including the Diesel Generator Building) as discussed in Subsection 1.3.3 and Section 4.3 of Reference (2) and Reference (3).
- o Sphere Enclosure Project (including the Sphere Enclosure Building) as discussed in Sections 3.1 and 3.4 of Reference (4).
- o Residual Heat Removal Project a discussed in Reference (5).

Such design criteria have been determined to be acceptable by the NRC in Reference (6). Accordingly, items so designed will not be subject to seismic review.

II. Draft Criteria For the Identification of the Remaining Plant Features Subject to Seismic Review.

The remaining plant features which are subject to seismic review consist of:

A. The remainder of the reactor coolant system pressure boundary, not reevaluated as part of SBP as described in Reference 1.

B. The structures, systems, components, and equipment necessary to bring the plant to a safe, warm (greater than 350°F) shutdown condition within 24 hours:

1. Boration Function - Those portions of the chemical and volume control system that supply borated water from the refueling water storage tank to the reactor coolant system via the normal charging line, the reactor coolant pump seal water lines, and the auxiliary spray line in the pressurizer.

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* Developed in conjuction with the suspended BOP seismic reevaluation program. Will be revised, as necessary, for use in SEP seismic review work.

2. Heat Removal Function - Removal of decay heat from the reactor coolant system during cooldown from a hot operating condition to a warm shutdown condition may be accomplished by venting steam to the atmosphere using the atmospheric steam dump valves in the main steam relief headers with the safety valves as backup. Makeup to the steam generators is provided by the auxiliary feedwater pumps with water supplied from the condensate storage tank.

   The component cooling water system and a portion of the salt water cooling system are required to support the various equipment cooling requirements during the above-identified cooldown. Those portions of the steam, feedwater, component cooling water, and salt water cooling systems required to effect such heat removal.

3. Depressurization Function - The reactor coolant system pressure is controlled by the auxiliary spray to the pressurizer (paragraph 1) and heat removal.

4. Miscellaneous Supporting Systems - To support the primary functions noted above.

C. The structures, systems, (e.g., the residual heat removal system), and equipment which are necessary to maintain the plant in an extended safe, warm shutdown, and/or to ensure the capability to achieve and maintain a safe, cold shutdown.

D. The structures, systems and components necessary to ensure the capability to prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the guideline exposures of 10CFR100.

   o Spent fuel pool cooling system

   o Safety injection system

   o Containment spray system

   o Hydrazine addition system

   o Containment isolation system

   o Fuel storage building and control room heating, ventilating, and air conditioning system (HVAC)

III. Draft Declassification Criteria

Certain portions of Seismic Category A structures, components, and systems may be declassified (i.e., not classified as Seismic Category A) by: (1) isolating portions that are not essential to the safety related performance of the system, (2) demonstrating by safety analysis that the failure of the portion of the structure, component, or system would not

result in loss of the safety related function of the structure, component, or system, or (3) demonstrating by safety analysis that the probability and/or consequences of failure of the portion of the structure, component, or system is acceptably low.

A. Declassification by System Boundary Reestablishment (Isolation)

In order to isolate (and therefore declassify) portions of components and systems determined to be non-essential to the safety-related performance of Seismic Category A components and systems, system boundaries may be reestablished by one or more of the following:

a. Closing an existing normally open manual or remote operated valve(s) at all times during plant operation.

b. Require remote-manual closure of existing valve(s) by station operators following a seismic event.

c. Addition or relocation of valve(s) to be employed as identified in A and B, above.

d. Addition of a new check valve(s) for automatic isolation of portions of systems where a depressurization/flow reversal might occur following a seismic event.

B. Declassification By Analysis

The declassification of portions of Seismic Category A structures, components, and systems may be accomplished by the performance of a safety analysis which demonstrates that partial structure, component, or system failure will not result in the unacceptable degradation in the safety-related performance of Seismic Category A structures, components, and systems, or the probability or consequences of such a failure are acceptably low.

IV. References

1. "Seismic Reevaluation and Modification," San Onofre Nuclear Generating Station, Unit 1, April 29, 1977.

2. Amendment 38 in Docket 50-206, "Addition of Standby Power and ECCS Modifications (Preliminary Engineering and Safety Analysis Report), San Onofre Nuclear Generating Station, Unit 1," February, 1975.

3. "Supplement 1, Addition of Standby Power and ECCS Modifications (Preliminary Engineering and Safety Analysis Report), San Onofre Nuclear Generating Station, Unit 1," May, 1975.

4. Amendment 52 to Final Safety Analysis, San Onofre Nuclear Generating Station, Unit 1, December 3, 1975.

5. Amendment 30 to Docket 50-206.

6. Safety Evaluation by the Office of Nuclear Reactor Regulation Supporting Amendment No. 52 to Provisional Operating License No. DPR-13," San Onofre Nuclear Generating Station, Unit 1, April 1, 1977.

3.7 SEISMIC DESIGN

In the Balance of Plant (BOP) Phase 1 Seismic Reevaluation, seismic loadings due to a Design Basis Earthquake (DBE) will be considered. The BOP Phase I Seismic Reevaluation will utilize a DBE ground motion, developed and verified as part of the Site Specific Earthquake program described in reference (1).

3.7.1 SEISMIC INPUT

3.7.1.1 Design Response Spectra

Design response spectra are being developed in the Site Specific Earthquake program as described in reference (1). Development of design response spectra considering actual site characteristics is consistent with provisions of Regulatory Guide 1.60, footnote 2.

3.7.1.2 Design Time History

The horizontal and vertical ground-motion components of the DBE will be developed in accordance with the provisions of the Standard Review Plan (SRP), subsection 3.7.1. The table of frequency intervals used to calculate the response spectra are given in SRP subsection 3.7.1.

Free field time histories to be used in the BOP Seismic Reevaluation will be consistent with the design spectra being developed in the Site Specific Earthquake program.

3.7.1.3 Critical Damping Values

The damping values given in table 3.7-1 will be used for the seismic analysis of structures, piping and mechanical equipment. These are identical with the damping values recommended in Regulatory Guide 1.61 except in the case of mechanical equipment and large piping (equal to or greater than 12 inches) for which 4% damping is used. This damping value was established in testing programs reported in reference (2) as permitted by Regulatory Guide 1.61, paragraph C.2.

Soil hysteretic and geometric damping is discussed in paragraph 3.7.2.4.

TABLE 3.7-1

DBE DAMPING VALUES USED FOR SEISMIC REEVALUATION PROGRAM

Item	DBE Damping (Percent of Critical)
Mechanical Equipment and Large piping (greater than or equal to 12 inches)	4[a]
Small piping	2
Welded steel structures	4
Bolted and/or riveted steel structures	7
Reinforced concrete structures	7
Prestressed concrete structures	5

a. Based on reference (2).

3.7.1.4 Supporting Media for Seismic Category A Structures

The foundations for the various soil supported structures analyzed in this study are described in subsection 1.4.3 of reference (1). The supporting media is a San Mateo Sand deposit which is uniform and extends to a depth of approximately 1000 feet (Reference 3).

Work performed by Woodward-Clyde Consultants, formerly Woodward-McNeill and Associates (Reference 3), demonstrates that the soil shear modulus and material (hysteretic) damping properties are dependent upon confining pressures and induced strains as illustrated in figures 3.7-1 and 3.7-2.

A single average value of Poisson's ratio of 0.35 was developed from dynamic laboratory tests and field geophysical tests (Reference 3).

The procedures for how these parameters are used in soil structure interaction analysis were developed from dynamic model tests completed at the site (Reference 4).

3.7.2 SEISMIC SYSTEM ANALYSIS

Major Seismic Category A structures that are considered in conjunction with foundation media in forming a soil-structure interaction model are defined as "Seismic Systems."

3.7.2.1 Seismic Analysis Methods

In general, the analysis methods utilized will be based upon linear dynamic analysis techniques. These techniques are described in sections 3.1, 3.2, and 4 of BC-TOP-4-A, revision 3. Some systems may be analyzed by equivalent static analysis methods where appropriate. In instances where the response of the structure is in the inelastic range an appropriate non-linear analysis may be performed. Among the several possible non-linear techniques which may be applied are time history, simplified inelastic response spectrum, or energy methods. Section 3.8 includes applicable ductility ratios which will form the basis for the non-linear acceptance criteria.

The selection of the minimum number of mass points and the number of degrees-of-freedom per mass point are described in section 3.2 of BC-TOP-4-A, revision 3.

Significant effects such as piping interactions, external structural restraints, and hydrodynamic effects are included in the analysis. Methods employed to account for soil structure interaction effects are described in paragraph 3.7.2.4.

An equivalent static analysis of the reactor auxiliary building and circulating water intake structure will be performed. These structures are embedded and are therefore not subjected to large amplifications of ground motion.

The lateral seismic earth pressures acting upon embedded walls consist of active and passive pressures. The active and passive pressures will be calculated separately and each wall will be evaluated for the most severe of the two conditions.

The embedded structures are expected to track the surrounding soil because the weights of the structures were each found to be less than the soil they displace. Therefore, the seismic amplification of these structures is considered insignificant and the ground motion response spectra will be used in lieu of the amplified spectra.

Floor slabs in the reactor auxiliary building and circulating water intake structure will be analyzed in the vertical direction based upon estimating their period in the vertical direction and selecting an appropriate corresponding acceleration from the vertical ground motion response spectrum curve. The horizontal free field response spectrum will be applied to all elevations for the horizontal analysis of structures, systems, and components. Localized slab stiffnesses will be considered in the vertical response analysis of subsystems utilizing the free field ground response spectra.

The seawall will be analyzed employing equivalent static analysis methods for both the Design Basis Earthquake (DBE) and a site specific Tsunami.

3.7.2.2 Natural Frequencies and Response Loads

A summary of natural frequencies, mode shapes, modal responses, and response loads determined by the seismic analyses and response spectra at selected plant equipment elevations and equipment support points will be available following completion of structural analysis.

3.7.2.3 Procedures Used for Modeling

DECOUPLING CRITERIA FOR SUBSYSTEM - Decoupling of systems and subsystems is done in accordance with section 3.2 of BC-TOP-4-A, revision 3.

LUMPED MASS CONSIDERATIONS - A description of the procedure used to locate lumped masses for the seismic system analyses for Seismic Category A structures and equipment is provided in section 3.2 of BC-TOP-4-A, revision 3.

FINITE ELEMENT MODELING - Three dimensional finite element models may be employed. A detailed description of each finite element model (including the location of mass points) and the number of dynamic degrees-of-freedom will be available following completion of seismic system modeling.

MODELING FOR THREE COMPONENT INPUT MOTION - Modeling of the structure for the three component input motion is done in accordance with section 3.2 of BC-TOP-4-A, revision 3.

3.7.2.4 Soil Structure Interaction

Soil-structure interaction, when used, is taken into account by coupling the structural model with the foundation medium. The method used for representing the structure-foundation interaction is the lumped parameter representation. The effect of embedment is always taken into account in the analysis. The embedment is defined as the vertical distance from the bottom of the structural base slab to the adjacent finish grade. Appendix H of BC-TOP-4-A, revision 3 discusses the applicability of the lumped parameter method and its comparison with the finite element method.

In general, the impedances which represent the foundation media are complex functions of the basemat geometry, structure inertia, structural embedment, elastic properties of the foundation medium, and forcing frequencies. The impedance functions can be approximated by frequency-independent conditions for foundation material with fairly uniform properties.

The impedance functions can be represented by a mechanical analog composed of equivalent springs and dampers. The equivalent dampers represent the radiation, or the geometric damping effect of the seismic wave energy away from the structural base and usually predominate over the internal, or hysteretic damping of the foundation medium; the latter is considered to be additive to geometric damping in the lumped parameter representation and is discussed in paragraph 3.7.1.4.

Figure 3-1 in BC-TOP-4-A, revision 3 shows a schematic lumped parameter model of the structure-foundation system consisting of the foundation impedances. These impedances are represented by the equivalent spring constants, designated as K_z, K_x, K_ψ, K_t, and the radiation dampers, designated as D_z, D_x, D_ψ, D_t, for vertical, horizontal, rocking, and torsional seismic excitations, respectively. The formulation of these parameters is presented in table 3.7-2 which is based upon reference 4.

3.7.2.5 Development of Floor Response Spectra

A multi-mass two or three-dimensional time history analysis method is used to develop floor response spectra, except for the reactor auxiliary building, sea wall, and intake structure (see paragraph 3.7.2.1). The time history method is described in section 4.2 of BC-TOP-4-A, revision 3.

When the seismic analysis is performed separately for each of the three directions in the case of asymmetric structures, the ordinates of the floor response spectrum for a given direction will be obtained by combining the ordinates of the three computed floor response spectra for that direction according to the square-root-of-the-sum-of-the-squares (SRSS) criterion. In the case of symmetric structures, the floor response spectrum for each direction will be the single smoothed floor response spectrum computed for that direction. This provision is in accordance with Regulatory Guide 1.122, paragraph c.2, with the exception that the computed response spectra are used in lieu of the smoothed response spectra for the SRSS combination.

When the mathematical model of the supporting structure is subjected simultaneously to the action of three spatial components of an earthquake, the computed and smooth response spectrum in a given direction will be the floor response spectrum in that direction. This provision is in accordance with Regulatory Guide 1.122, paragraph c.3.

3.7.2.6 Three Components of Earthquake Motion

The three-dimensional earthquake effects will be combined in the same manner as discussed in paragraph 3.7.3.6.

TABLE 3.7-2

EQUATIONS FOR LUMPED STRUCTURE-FOUNDATION INTERACTION ANALYSIS (Sheet 1 of 4)

Base	Motion	Spring Constant	Geometric Damping (% of critical)	Mass Ratio
Circular[a]	Horizontal	$K_x = C_1 C_2 \frac{32\ (1 - \nu)\ GR}{7 - 8\nu}$	$D_x = \frac{0.288}{\sqrt{B_x}}$	$B_x = \frac{(7 - 8\nu)\ M}{32\ (1 - \nu)\ \rho R_{e_x}^3}$
	Rocking	$K_\psi = C_1 C_2 \frac{8GR^3}{3\ (1 - \nu)}$	$D_\psi = \frac{0.15}{(1 + B_\psi)\ \sqrt{B_\psi}}$	$B_\psi = \frac{3\ (1 - \nu)\ \eta I r}{8\rho R_{e_\psi}^5}$
	Vertical	$K_z = C_1 C_2 \frac{4GR}{1 - \nu}$	$D_z = \frac{0.425}{\sqrt{B_z}}$	$B_z = \frac{(1 - \nu)\ M}{4\rho R_{e_z}^3}$
	Torsion	$K_t = C_1 C_2 \frac{16GR^3}{3}$	$D_t = \frac{0.5}{1 + 2B_t}$	$B_t = \frac{I_t}{\rho R_{e_t}^5}$

TABLE 3.7-2

EQUATIONS FOR LUMPED STRUCTURE-FOUNDATION INTERACTION ANALYSIS (Sheet 2 of 4)

Base	Motion	Spring Constant	Geometric Damping (% of critical)	Mass Ratio
			Use the above circular base, formulas for equivalent damping and mass ratio for rectangular bases with an equivalent radius redefined by the following.	
Rectangular[b]	Horizontal	$K_x = C_1C_2 2(1 + \nu)G\beta_x \sqrt{BL}$	$R_x = \sqrt{BL/\pi}$	
	Rocking	$K_\psi = C_1C_2 \frac{G}{1 - \nu} \beta_\psi BL^2$	$R_\psi = \sqrt[4]{BL^3/3\pi}$	
	Vertical	$K_z = C_1C_2 \frac{G}{1 - \nu} \beta_z \sqrt{BL}$	$R_z = \sqrt{BL/\pi}$	
	Torsion	$K_t = C_1C_2 \frac{16GR^3}{3}$	$R_t = \sqrt[4]{BL(B^2 + L^2)/6\pi}$	

TABLE 3.7-2

EQUATIONS FOR LUMPED STRUCTURE-FOUNDATION INTERACTION ANALYSIS (Sheet 3 of 4)

a. Circular	G = Shear modulus of foundation material
	R = Base mat radius
	R_e = Effective base mat radius = C_3R
	M = Total mass of structure
	ν = Poisson's ratio of foundation material
	ρ = Mass density
	I_r = Mass moment of inertia of structure and base mat about the rocking axis at the base
	I_t = Polar mass moment of inertia of structure and base mat
	C_1 = Correction factor for pressure distribution (Reference 4)
	C_2 = Correction factor for embedment (Reference 4)
	η = Correction factor for mass moment of inertia for rocking (Reference 4)
	C_3 = Correction factor for effective base mat radius (Reference 4)
b. Rectangular	ν = Poisson's ratio of foundation material
	G = Shear modulus of foundation material
	B = Width of the base mat parallel to the axis of rocking
	C_1 = Correction factor for pressure distribution (Reference 4)
	C_2 = Correction factor for embedment (Reference 4)
	L = Length of the base mat perpendicular to the axis of rocking

TABLE 3.7-2

EQUATIONS FOR LUMPED STRUCTURE-FOUNDATION INTERACTION ANALYSIS (Sheet 4 of 4)

β_x, β_ψ, β_z =	Constants that are functions of the dimensional ratio, L/B (figure 10-6, Reference 8)
C_3 =	Correction factor for effective base mat radius (Reference 4)

If statistically independent time history components are utilized, codirectional responses due to the three earthquake components are combined by superposition.

3.7.2.7 Combination of Modal Responses

The methods used for combining modal responses are in accordance with the requirements of Regulatory Guide 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," except as noted in paragraph 3.7.3.7.

3.7.2.8 Interaction of Non-Seismic Category A Structures with Seismic Category A Structures

To ensure that Seismic Category A structures will perform their intended functions during and after a DBE, Non-Seismic Category A structures will be evaluated to ensure the following criterion is met:

- The response or collapse of any Non-Seismic Category A structure will not impair the integrity of Seismic Category A structures or components.

3.7.2.9 Effects of Parameter Variations on Floor Response Spectra

To account for variations in the structural frequencies owing to uncertainties in such parameters as the material properties of the structure and soil, damping values, soil-structure interaction techniques, and the approximations in the modeling techniques used in seismic analysis, the computed floor response spectra from the floor time-history motions will be smoothed, and peaks associated with each of the structural frequencies will be broadened by a frequency, $\pm\Delta f_j$, where:

$$\Delta f_j = \left[(0.05 f_j)^2 + \sum_{n=1}^{P} (\Delta f_{jn})^2 \right]^{1/2} \not< 0.10 f_j$$

where Δf_{jn} denotes the variation in Jth mode frequency f_j, due to variation in parameter number n, and P is the number of significant parameters considered. A value of $0.10 f_j$ will be used if the actual computed value

of Δf_j is less than 0.10f. If the above procedure is not used, Δf_j will be taken as $0.15f_j$. This provision is in accordance with Regulatory Guide 1.122, paragraph c.1.

3.7.2.10 Use of Constant Vertical Static Factors

Constant vertical load factors are not used for Seismic Category A structures.

3.7.2.11 Method Used to Account for Torsional Effects

Torsional effects, if significant, are included in the horizontal models. Section 3.2 and Appendix C of BC-TOP-4-A, revision 3 show the technique used to account for torsional effects in lumped parameter models. In three-dimensional finite element models, torsional effects are automatically accounted for.

3.7.2.12 Comparison of Responses

It is not anticipated that both time history and response spectrum stress analysis will be performed for any one structure.

3.7.2.13 Methods for Seismic Analysis of Dams

Dams are not analyzed in this program.

3.7.2.14 Determination of Seismic Category A Structures Overturning Moments

The effects of overturning moments are evaluated by the methods shown in section 4.4 of BC-TOP-4-A, revision 3. This section includes a description of the methods used to compute foundation reactions and to account for vertical earthquake effects.

Codirectional responses are combined using the square-root-of-the-sum-of-squares (SRSS) of the applicable maximum codirectional responses as described in section 4.3 of BC-TOP-4-A, revision 3. The codirectional responses can also be combined using the absolute sum of 100% of the maximum one directional response with 40% of the maximum remaining two directions as described in reference (7).

3.7.2.15 Analysis Procedure for Damping

For a coupled system with different damping and different structural elements, such as would be the case in analysis with coupling between concrete structures and welded steel components, the method to be used for damping is either to: (a) inspect the mode shapes to determine which modes correspond with a particular structural element and then use the damping associated with that element having predominant motion, (b) to use one of the damping methods as described in paragraph 3.7.2.15 of reference (6), or (c) by procedures similar to those described in reference (2).

3.7.3 SEISMIC SUBSYSTEM ANALYSIS

Seismic Category A structures, systems, and components not considered as "seismic systems" (see subsection 3.7.2) are defined as "seismic subsystems."

3.7.3.1 Seismic Analysis Methods

Seismic subsystems consisting of structures, equipment, and piping will be analyzed using methods similar to those described in paragraph 3.7.2.1 for system seismic analysis. In general the analysis methods utilized will be based on linear dynamic analysis techniques, certain examples of which are described in BC-TOP-4-A, revision 3 for structures and BP-TOP-1, revision 3 for piping. In instances where the response of a subsystem item would be in the inelastic range, an appropriate non-linear analysis may be performed.

The analyses performed for piping, equipment, and supports will not include stresses resulting from DBE-induced differential motion. These stresses are secondary in nature, based on current ASME Code rules, and are therefore not required to be evaluated in a faulted condition analysis. The basic characteristic of these stresses is that they are self-limiting. Local yielding and minor distortions will satisfy the initial conditions that caused the stress to occur.

3.7.3.2 Determination of Number of Earthquake Cycles

The BOP Seismic Reevaluation Program considers a DBE in combination with normal operating loads. Fatigue calculations are not required as part of a DBE analysis.

3.7.3.3 Procedures Used for Modeling

Examples of the techniques and procedures to be employed in the modeling of seismic subsystems are contained in BP-TOP-1, revision 3. Equipment reevaluation techniques are discussed in sections 3.9 and 3.10.

3.7.3.4 Basis for Selection of Frequencies

The analysis of equipment subjected to seismic loading involves several basic steps, the first of which is the establishment of the intensity of the seismic loading. Considering that the seismic input originates at the point of support, the response of the equipment and its associated supports, based upon the mass and stiffness characteristics of the system, will determine the seismic accelerations which the equipment must withstand. Three ranges of equipment/support behavior that affect the magnitude of the seismic acceleration are possible:

A. If the equipment is rigid relative to the structure, the maximum acceleration of the equipment mass approaches that of the structure at the point of equipment support. The equipment acceleration value in this case corresponds to the low-period region of the floor response spectra.

B. If the equipment is very flexible relative to the structure, the internal distortion of the structure is unimportant and the equipment behaves as though supported on the ground.

C. If the periods of the equipment and supporting structure are nearly equal, resonance occurs and must be taken into account.

Also, equipment/support systems having natural frequencies greater than 20Hz are considered rigid. The natural frequencies will be determined, based on the as-built condition and appropriately considered in the analysis.

3.7.3.5 Use of Equivalent Static Load Methods of Analysis

The static load equivalent or static analysis method involves the multiplication of the total weight of the equipment or component member by the

specified seismic acceleration coefficient. The magnitude of the seismic acceleration coefficient is established on the basis of the expected dynamic response characteristics of the component. Components which can be adequately characterized as single-degree-of-freedom systems are considered to have a modal participation factor of one. Seismic acceleration coefficients for multi-degree of freedom systems which may be in the resonance region of the amplified response spectra curves are increased by 50 percent to account conservatively for the increased modal participation.

3.7.3.6 Three Components of Earthquake Motion

Seismic subsystem and equipment response to the three components of the earthquake will be determined using either time history or response spectrum analyses.

If the response spectra method is adopted for seismic reevaluation, the representative maximum values of the structural responses to each of the three components of earthquake motion will be combined by taking the square-root-of-the-sum-of-the-squares of the maximum representative values of the codirectional responses caused by each of the three components of earthquake motion at a particular point of the structure or of the mathematical model.

If the time-history analysis method is employed for seismic reevaluation, two types of analysis will be performed depending upon the complexity of the problem:

a. When the maximum responses due to each of the three components are calculated separately, the method for combining the three-dimensional effects is identical to the response spectra method, except that the maximum responses are calculated using the time-history method instead of the spectrum method.

b. If the time-history responses from each of the three components of the earthquake motion are calculated by the step-by-step method and

combined algebraically at each time step, the maximum response will be obtained from the combined time solution.[a]

These provisions are in accordance with Regulatory Guide 1.92, Regulatory Position C.2.

3.7.3.7 Combination of Modal Responses

The combination of modal responses will be in accordance with Regulatory Guide 1.92 or, as an acceptable alternative in accordance with subsection 3.7.3.4 of RESAR-41 as described below. The total seismic response for each analysis shall be obtained by combining the individual modal responses utilizing the square-root-of-the-sum-of-the-squares method.

For systems having modes with closely spaced frequencies, the above method will be modified to include the possible effect of these modes. The groups of closely spaced modes will be chosen such that the difference between the frequencies of the first mode and the last mode in the group does not exceed 10 percent of the lower frequency. Combined total response for systems which have such closely spaced modal frequencies will be obtained in accordance with Regulatory Guide 1.92 or, as an acceptable alternative, in accordance with paragraph 3.7.3.4 of RESAR-41. This alternative is the addition of the square-root-of-the-sum-of-squares of all modes, the product of the responses of the modes in each group of closely spaced modes, and a coupling factor $\varepsilon_{K\ell}$. This can be represented mathematically as:

$$R_T^2 = \sum_{i=1}^{N} R_i^2 + 2 \sum_{j=1}^{S} \sum_{K=M_j}^{N_j-1} \sum_{\ell=K+1}^{N_j} R_K R_\ell \varepsilon_{K\ell} \text{ , for: } \ell \neq K$$

a. When this method is used, the earthquake motions specified in the three different directions will be statistically independent.

where:

ℓ = mode index used in response summation

k = mode index used in response summation

R_T = total response

R_i = absolute value of response of mode i

N = total number of modes considered

S = number of groups of closely spaced modes

M_j = lowest modal number associated with group j of closely spaced modes

N_j = highest modal number associated with group j of closely spaced modes

$\varepsilon_{K\ell}$ = coupling factor with

$$\varepsilon_{K\ell} = \left\{ 1 + \left[\frac{\omega'_K - \omega'_\ell}{(\beta'_K \omega_K + \beta'_\ell \omega_\ell)} \right]^2 \right\}^{-1}$$

and

$$\omega'_K = \omega_K \left[1 - (\beta'_K)^2 \right]^{1/2}$$

$$\beta'_K = \beta_K + \frac{2}{(\omega_K t_d)}$$

ω_K = frequency of closely spaced mode K (rad/sec)

β_K = fraction of critical damping in closely spaced mode K

t_d = duration of the earthquake (seconds)

3.7.3.8 Analytical Procedure for Piping

The response spectra modal analysis technique will be used to analyze piping.

The seismic analyses will be based on the DBE being initiated while the plant is at the normal full power condition.

The percentage of the critical damping value to be used in the analysis of Seismic Category A piping is given in table 3.7-1. The analysis procedures for damping are given in paragraph 3.7.3.15.

For piping systems interconnected between floors of a structure and/or building, the envelope of the respective floor response spectra will be used in the seismic analysis.

The piping will be analyzed for the simultaneous occurrence of two horizontal components and one vertical earthquake input component.

The response spectra associated with each earthquake component will be applied in each direction separately. The combined modal response for each item of interest (e.g., force, displacement, stress) resulting from each component analysis will be combined by the square-root-of-the-sum-of-the-squares method.

For small piping (2" and smaller) as an option to dynamic analysis, either the equivalent dynamic or static rigid range approach will be used. The equivalent dynamic analysis is described in BP-TOP-1, revision 3. The static rigid range approach is used for rigid piping systems which are defined as having natural frequencies greater than 20Hz. In this case, the piping system is analyzed with static equivalent loads corresponding to acceleration in the rigid range of the applicable response spectrum curves. Both horizontal and vertical static equivalent loads are applied to rigid piping systems. The response of the piping system for two orthogonal horizontal directions and one vertical direction are combined on a square-root-of-the-sum-of-squares basis.

For any piping that can be shown to be rigid (lowest natural frequency greater than 20Hz), as an option to performing a dynamic analysis, the static rigid range approach may be used.

3.7.3.9 Multiple Supported Equipment and Components with Distinct Inputs

The DBE, being a very low probability, single occurrence event, is treated as a faulted condition. Therefore, consistent with present ASME Code rules, the secondary stresses associated with the DBE-induced differential motion will not be evaluated in the seismic analysis.

3.7.3.10 Use of Constant Vertical Static Factors

Constant vertical load factors are not used in the seismic reevaluation of safety-related components and equipment.

3.7.3.11 Torsional Effects of Eccentric Masses

The effect of eccentric masses, such as valves and extended structures, are considered in the seismic piping analyses. These eccentric masses are modeled in the system analysis, and the torsional effects caused by them are evaluated and included in the total system response. The total response must meet the limits of the criteria applicable to the safety class of the piping.

3.7.3.12 Buried Seismic Category A Piping Systems and Tunnels

Section 6.0 of BC-TOP-4-A, revision 3 discusses the techniques to calculate the stresses from seismic loadings for buried Seismic Category A piping. The buried Seismic Category A piping will be seismically reevaluated to ensure structural integrity when subjected to DBE loads. This is accomplished by comparing the calculated stresses in the pipe material under faulted condition loading combinations, including the DBE and normal operating pressure, to the faulted condition stress limits presented in table 3.9-5.

Stress limits presented in table 3.9-5 are based on current code requirements and consideration of original design codes and quality requirements. Compliance with such criteria will represent adequate reevaluation without further analysis; if component or system response does not comply with such criteria, alternate stress criteria based upon further consideration of original design codes, original quality requirements and failure probabilities and consequences will be utilized.

The methodology presented in section 6.0 of BC-TOP-4-A, revision 3 will be employed in the analysis of Seismic Category A tunnels.

3.7.3.13 Interaction of Other Piping with Seismic Category A Piping

In certain instances, Seismic Category A piping may be connected to non-Seismic Category A piping at locations other than a piece of equipment. These transition points typically occur at Seismic Category A isolation boundary valves. Since a dynamic analysis must be modeled from pipe anchor point to anchor point (or seismic restraint), it is necessary to analyze the system from the anchor point in the Seismic Category A system through the valve to the first anchor point or equivalent seismic restraint beyond the Seismic Category A system boundary.

Where small Non-Seismic Category A piping is attached to Seismic Category A piping, its effect on the Seismic Category A piping is accounted for by lumping a portion of its mass with Seismic Category A piping at the point of attachment or alternately, the non-seismic piping will be included in the seismic piping model out to the first seismic restraint or anchor point.

3.7.3.14 Seismic Analysis for Reactor Internals

The analysis of the reactor internals has been completed as discussed in reference (1).

3.7.3.15 Analysis Procedure for Damping

See paragraph 3.7.2.15.

REFERENCES

1. "Seismic Reevaluation Program, San Onofre Nuclear Generating Station Unit 1," March, 1978.

2. "Damping Values of Nuclear Plant Components," Westinghouse Electric Corporation, WCAP-7921-AR, Pittsburgh, Pennsylvania, May, 1974.

3. "Material Property Studies, San Onofre Nuclear Generating Station," San Onofre Nuclear Generating Station Units 2 and 3 PSAR, Amendment 11 Attachment A3 to Appendix 2E, March 13, 1972.

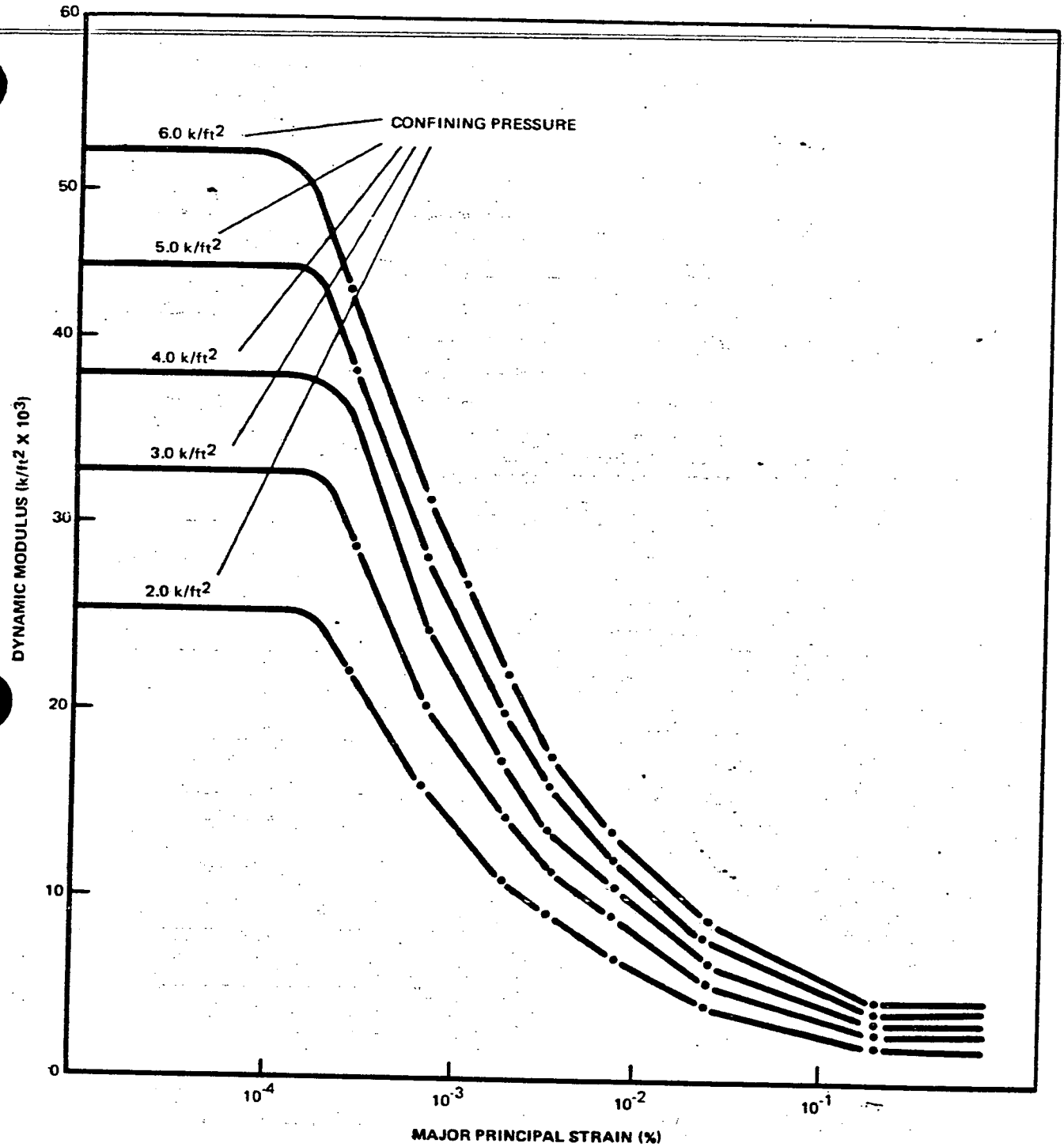
4. Woodward-McNeill and Associates, Development of Soil Structure Interaction Parameters Proposed Units 2 and 3, San Onofre Nuclear Generating Station, San Onofre, California, January 31, 1974.

5. Seed, H., and Whitman, R., "Design of Earth Retaining Structures for Dynamic Loads," Proceedings of ASCE specialty conference on Lateral Stresses in the Ground and Design of Earth Retaining Structure, 1970.

6. "Seismic Reevaluation and Modification," San Onofre Nuclear Generating Station, Unit 1, NRC Docket 50-206, Bechtel Power Corporation, Los Angeles, California, April 29, 1977.

7. "Combination of Co-directional Responses Due to Three Earthquake Input Components by the Component Factor Method," Seismic Committee Newsletter No. 6, Bechtel Power Corporation, Los Angeles, California, July 1, 1976.

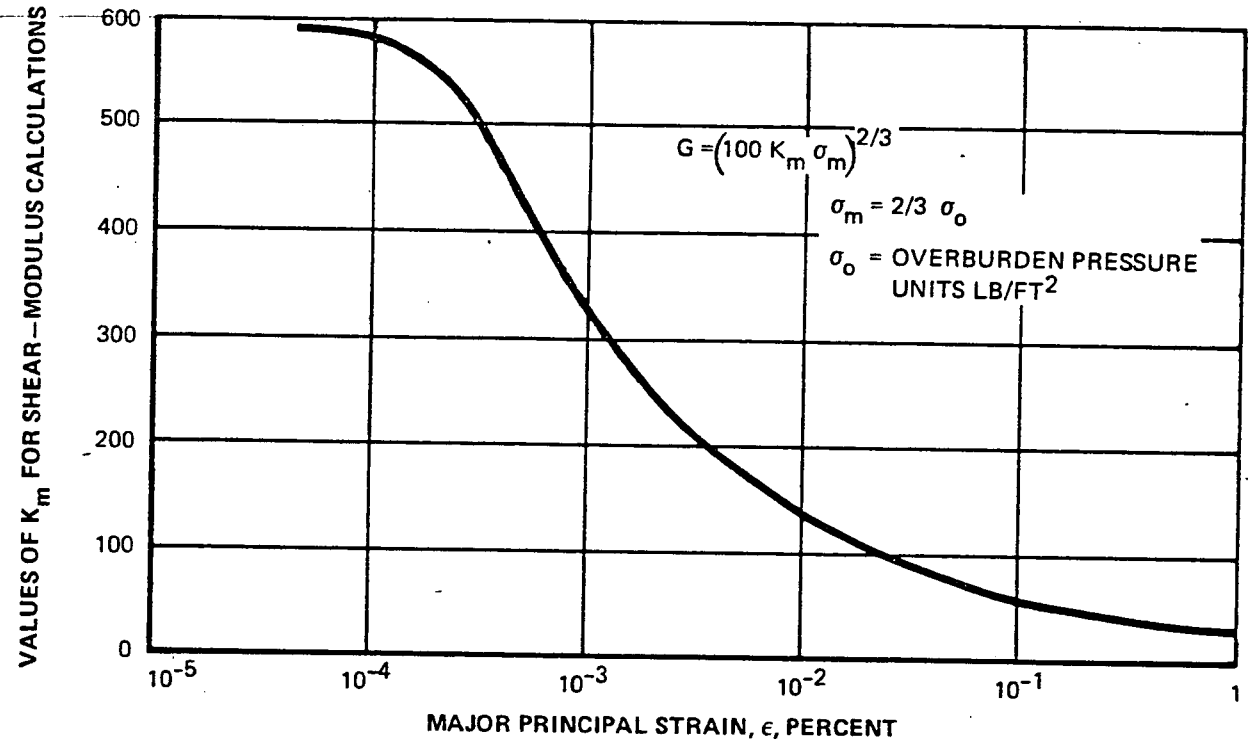
8. Richart, F. E., Jr., Hall, J. R., and Woods, R. D., Vibration of Soil and Foundations, Prentice-Hall, Inc., Englewood, New Jersey, 1970.

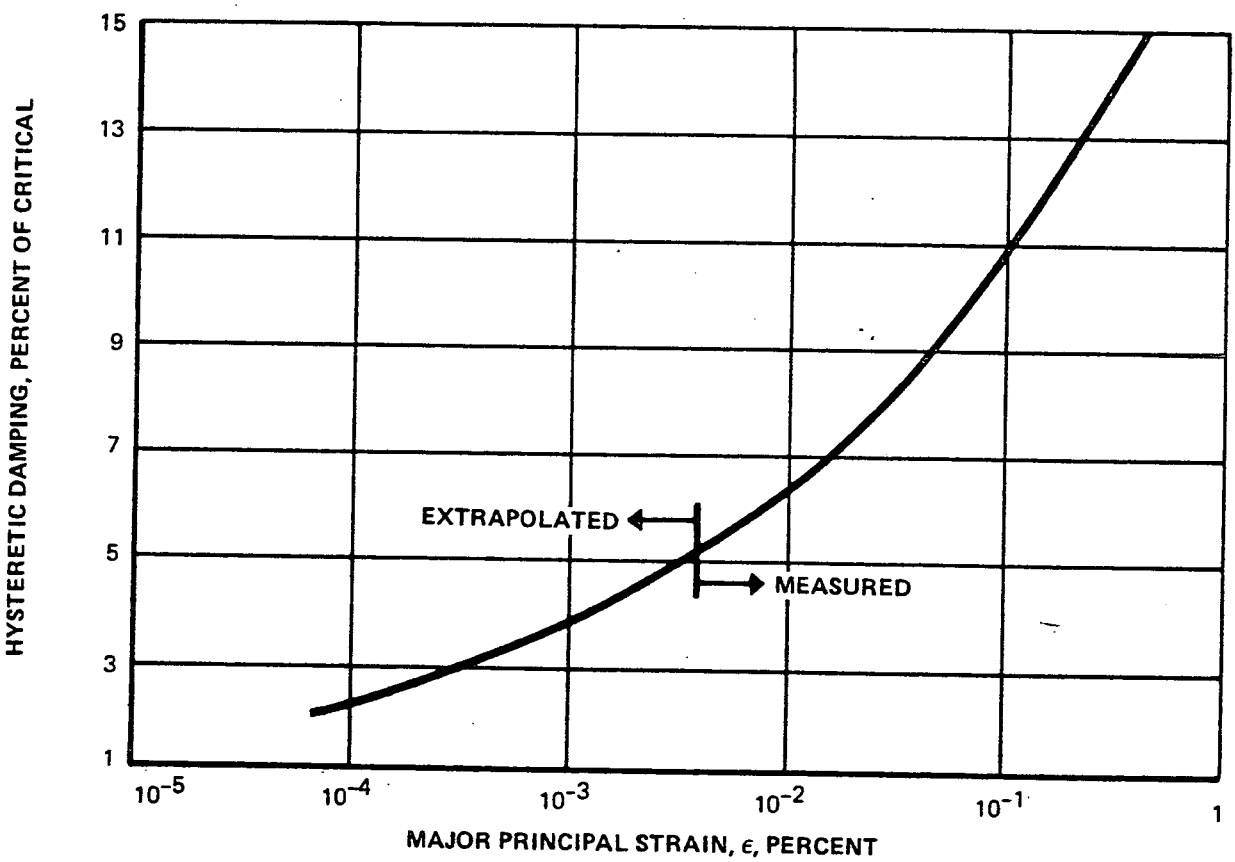


a. BASED UPON REFERENCE (4).

Figure 3.7-1
EFFECTIVE DYNAMIC MODULUS[a] FOR THE SAN ONOFRE SITE

3/1/78





a. BASED ON REFERENCE (4)

Figure 3.7-2

MODULUS AND DAMPING VS STRAIN[a]
SAN MATEO FORMATION SAND

3.8.4 STRUCTURES OTHER THAN CONTAINMENT

The following section describes the seismic reevaluation of structures as part of the BOP Phase I scope as identified in subsection 1.2.1. The structures to be reevaluated in BOP Phase 1 are identified in table 3.2-1. Structures identified in table 3.2-1 as Seismic Category A will be analyzed to ensure that the DBE will not impair their ability to perform their particular safety-related functions. The safety related functions of Seismic Category A structures identified in table 3.2-1 are given in table 1.4.4-1 of reference (1).

3.8.4.1 Description of Structures

A description of structures is included in subsection 1.4.2 of reference (1).

3.8.4.2 Applicable Codes, Standards, and Specifications

The following codes, specifications, and project reports constituted the basis for the design, fabrication, and construction of the existing structures.

A. Uniform Building Code (UBC), 1961 Edition

B. "Manual of Steel Construction," American Institute of Steel Construction (AISC), 1963 Edition.

C. "Building Code Requirements for Reinforced Concrete," ACI Standard 318-63, American Concrete Institute (ACI).

D. "Specification for the Design of Light Gage, Cold-Formed Steel Structural Members," 1963 Edition, American Iron and Steel Institute (AISI).

E. "Final Safety Analysis Report - San Onofre Unit 1," Bechtel Power Corporation, Los Angeles, California.

F. "Foundation Investigation at the San Onofre Unit 1 Site," Report No. 176, Engineering Department, Southern California Edison Company, Los Angeles, California, October 22, 1963.

Specific sections of the following codes and standards will be employed in the seismic reevaluation of the existing structures (see paragraphs 3.8.4.3 and 3.8.4.5 for application).

A. Uniform Building Code (UBC), 1976 Edition.

B. "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings," including Supplements 1, 2, and 3, American Institute of Steel Construction (AISC), February 12, 1969.

C. "Code Requirements for Nuclear Safety Related Concrete Structures," ACI Standard 349-76, American Concrete Institute (ACI).

D. American Society for Testing and Materials (ASTM) Standards.

E. "Building Code Requirements for Minimum Design Loads in Buildings and Other Structures," ANSI A58.1-1972, American National Standards Institute (ANSI).

F. "Building Code Requirements for Reinforced Concrete," ACI Standard 318-71, American Concrete Institute (ACI).

3.8.4.3 Loads and Load Combinations

The Balance of Plant seismic reevaluation will consider the occurrence of a DBE during normal plant operation. The loading combinations that will be considered for structures are shown in table 3.8-1. The normal loads and extreme environmental loads that will be considered are described in the following paragraphs. The loading combinations are consistent with ACI-349-1976, for concrete structures and Standard Review Plan, for steel structures.

TABLE 3.8-1

LOAD COMBINATIONS FOR STRUCTURES

Definitions and Nomenclature for Load Combinations

D = Dead Loads or their related internal moments and forces.

L = Applicable live loads or their related internal moments and forces.

F = Lateral and vertical pressure of liquids, or their related internal moments and forces.

H = Lateral earth pressure, or its related internal moments and forces.

T_o = Thermal effects and loads during normal operating conditions based on the steady state condition.

R_o = Maximum pipe and equipment reactions during normal operating conditions based on the steady-state condition, if not included in the above loads.

E' = Loads generated by the Design Basis Earthquake (DBE).

Load Combinations for Concrete, Masonry, and Steel Structures

$$D + L + F + H + T_o + R_o + E'^{(a)(b)(c)(d)}$$

Load Combination for Overall Structural Effects

D + F + H + E' for sliding, overturning, and flotation

(a) For the load combinations where D or L reduce the effects of other loads, the corresponding coefficients shall be taken as 0.90 for D and zero for L. The vertical pressure of liquids shall be considered as dead load, with due regard to variation in liquid depth.

(b) Hydrodynamic loads (F) will also be included for the spent fuel storage pool, refueling water storage tank, and condensate storage tank.

(c) T_o will not be considered when it can be shown that the loads secondary and self-limiting in nature.

(d) Where applicable, impact effects of moving loads shall be included with the live load L.

3.8.4.3.1 Normal Loads (all the loads encountered during normal plant operation)

Normal loads include dead loads (D), live loads (L), lateral and vertical pressure of liquids (F), and lateral earth pressure (H), as well as appropriate thermal operating loads (T_o), and pipe reaction loads (R_o) as defined in table 3.8-1.

In the determination of dead loads, actual as-built data will be used in lieu of estimation normally used in the design of a new plant. Actual as-built data will be collected by means of the following methods:

A. Use of as-built drawings.

B. Field investigation, survey, and measurements as required.

C. Use of final certified vendor drawings.

Live loads shall be based upon actual loadings and table 3 of ANSI A58.1-1972.

Thermal operating loads (T_o) and pipe reaction loads (R_o) will be considered where applicable. In general no significant loads of this type are anticipated.

3.8.4.3.2 Extreme Environmental Loads (Loads that are credible, but are highly improbable)

The extreme environmental loads considered are the Design Basis Earthquake (DBE) loads (E'). The DBE seismic loads will be generated in accordance with the provisions of section 3.7.

3.8.4.4 Reevaluation Analysis Procedures

The initial analysis procedures will be based upon elastic analysis techniques. For certain structures, an additional inelastic analysis as discussed in section 3.7.2.1, may be performed in order to demonstrate that the structural response to earthquake excitation will not result in impairment of the safety functions of the structure.

All structures will be analyzed by dynamic analysis techniques with the exception of the reactor auxiliary building, circulating water intake structure, and the seawall. See paragraph 3.7.2.1 for a detailed discussion of the dynamic analysis procedures.

The following are the principal computer programs that may be used in the seismic reevaluation and analysis of structures.

A. Bechtel Structural Analysis Program (BSAP)

B. Spectra Computer Program (SPECTRA)

C. Symbolic Matrix Interpretive System Program (SUPER SMIS)

D. Optimum Concrete Design Program (OPTCON)

E. Reinforced Concrete Design for Axial Force and Biaxial Bending (BIAX)

F. Generalized Equivalent Modal Damping (GEMD)

A description of each computer program, along with information pertaining to the validation and extent of application for each program, is presented in San Onofre Units 2 and 3 FSAR, Appendix 3C.

3.8.4.5 Structural Acceptance Criteria

The limiting values of stress, strain, and gross deformations are established by the following general criteria:

A. To maintain the structural integrity as required to achieve its Seismic Category A safety function when subjected to the DBE load combination.

B. To prevent structural deformations from impairing the safety function of Seismic Category A systems and equipment.

If a linear elastic analysis is used, the specific acceptance criteria stated in table 3.8-2 will be employed for the critical portions of Seismic Category A structures.

Acceptance criteria presented in table 3.8-2 are based on current code requirements and consideration of original design codes and quality requirements. Compliance with such criteria will represent adequate reevaluation without further analysis; if structural response does not comply with such criteria, alternate acceptance criteria based upon further consideration of original design codes, original quality requirements and failure probabilities and consequences will be utilized.

Typical values of ductility ratios that may be used for non-linear analysis are presented in table 3.8-3.

The acceptance criteria for non-critical portions of Seismic Category A structures will be the demonstration that their response or collapse will not impair the integrity or function of Seismic Category A structures, systems, or components.

Where justified, actual material properties and increased allowable stresses may be utilized. Justification may include in-situ tests, material tests performed during the original construction, or reference to technical literature.

Table 3.8-2

ACCEPTANCE CRITERIA FOR SEISMIC CATEGORY A STRUCTURES

DEFINITIONS AND NOMENCLATURE FOR ACCEPTANCE CRITERIA

U = FOR CONCRETE STRUCTURES, U IS THE REQUIRED SECTION STRENGTH TO RESIST THE DESIGN LOADS BASED ON THE STRENGTH DESIGN METHODS AS DESCRIBED IN ACI STANDARD 349-76.[a] FOR MASONRY BLOCKWALL STRUCTURES, U IS THE REQUIRED SECTION STRENGTH TO RESIST THE DESIGN LOADS BASED ON THE STRENGTH DESIGN METHODS SIMILAR TO THE METHODS DESCRIBED IN ACI STANDARD 349-76[a] FOR CONCRETE STRUCTURES, WITH CONCRETE RELATED MATERIAL PROPERTIES (SUCH AS f_c' AND E_c, ETC) SUBSTITUTED BY MASONRY PROPERTIES (SUCH AS f_m' AND E_m, ETC).

S = FOR STRUCTURAL STEEL STRUCTURES, S IS THE REQUIRED SECTION STRENGTH BASED ON ELASTIC DESIGN METHODS AND THE ALLOWABLE STRESSES DEFINED IN PART 1 OF THE AISC "SPECIFICATION FOR THE DESIGN, FABRICATION, AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS." FEBRUARY 12, 1969.[a]

Y = FOR STRUCTURAL STEEL, Y IS THE SECTION STRENGTH REQUIRED TO RESIST DESIGN LOADS AND BASED ON PLASTIC DESIGN METHODS DESCRIBED IN PART 2 OF THE AISC "SPECIFICATION FOR THE DESIGN, FABRICATION, AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS," FEBRUARY 12, 1969.[a]

ACCEPTANCE CRITERIA FOR THE CRITICAL PORTIONS OF THE CATEGORY A CONCRETE AND MASONRY STRUCTURES

THE STRENGTH DESIGN METHOD WILL BE USED AND THE FOLLOWING ACCEPTANCE CRITERIA WILL BE CONSIDERED:

$$U \geq D + F + L + H + T_0 + R_0 + E'$$ [b]

ACCEPTANCE CRITERIA FOR THE CRITICAL PORTIONS OF CATEGORY A STEEL STRUCTURES [c]

IF THE ELASTIC WORKING STRESS DESIGN METHOD IS USED, THE FOLLOWING ACCEPTANCE CRITERIA WILL BE CONSIDERED:

$$1.6\,S \geq D + F + L + H + T_0 + R_0 + E'$$ [b]

ACCEPTANCE CRITERIA FOR OVERALL STRUCTURAL EFFECTS

FOLLOWING IS THE ACCEPTANCE CRITERIA AGAINST OVERTURNING, SLIDING AND FLOTATION.

LOADING COMBINATION	MINIMUM FACTORS OF SAFETY		
	AGAINST OVERTURNING	AGAINST SLIDING	AGAINST FLOTATION
D + F + H + E'	1.1	1.1	1.1

a. WHERE JUSTIFIED, ACTUAL MATERIAL PROPERTIES AND/OR INCREASED ALLOWABLE STRESSES WILL BE UTILIZED BASED UPON IN-SITU TESTS, MATERIAL TESTS PERFORMED DURING THE ORIGINAL CONSTRUCTION, OR REFERENCE TO GENERIC INFORMATION.

b. T_0 AND R_0 WILL BE NEGLECTED WHEN IT CAN BE SHOWN THAT THEY ARE SECONDARY AND SELF-LIMITING IN NATURE.

c. IF PLASTIC DESIGN METHODS ARE USED, THE FOLLOWING ACCEPTANCE CRITERIA WILL BE CONSIDERED:

$$0.9Y \geq D + L + F + H + T_0 + R_0 + E'$$

Table 3.8-3

STRUCTURAL DUCTILITY RATIOS FOR ANALYSIS[a][b][c]

Material	Structure Type	Ductility Ratio (μ) Without Moment Resisting Frame	Ductility Ratio (μ) With Moment[d] Resisting Frame
1. HORIZONTAL EXCITATION			
Concrete	Frame	-	2.0
	Shear Wall	-	1.3
Steel	X-brace[e]	5.0	6.0
	X-brace[f]	3.0	4.0
	K-brace	2.0	3.0
	Single Diagonal[g]	2.0	3.0
	Moment Resisting Frame	-	3.0
2. VERTICAL EXCITATION			
All structures		1.2	

a. Ductility ratio (μ) is defined as the ratio of maximum member deformation to the member deformation at yield.

b. Damping ratio for various types of structures = 5% of critical damping.

c. Note these ductility ratios assume the existence of adequate connections. All connections will be checked for adequacy prior to utilizing these ductility ratios.

d. The frame moment capacity shall be equal to or greater than 25% of seismic shear.

e. The diagonal bracing member buckling load is nearly equal to the yield load (i.e., $\ell/r \leq 65$ for A36 steel; for steels with other yield points ℓ/r is such that $F'_e \cong F_y$ where F'_e is as defined in AISC-1969 specification, section 1.6.1).

f. The diagonal bracing member is slender (i.e., $\ell/r > 65$ for A36 steel; and as described in note (e) for steels with other yield points).

g. If there are equal numbers of diagonals in each direction, the μ value for X-bracing will be utilized.

3.8.4.6 Materials and Special Construction Techniques

Basic materials used in the construction of structures identified in table 3.2-1 and their specified minimum design strengths are presented below.

A. Concrete[a]

1.	Slabs on grade, building and equipment foundations		$f'c$ (lb/in.^{2}) = 2,500
2.	Supported floor slabs, beams, walls, retaining walls, turbine pedestal foundation, intake structure, shielding concrete		$f'c$ (lb/in.^{2}) = 3,000
3.	Prestressed decks, circulating water system gates, turbine pedestal superstructure		$f'c$ (lb/in.^{2}) = 4,000
4.	Grout		$f'c$ (lb/in.^{2}) = 2,000
5.	Concrete block masonry, Grade A	UBC-63 ASTM C-90	$f'm$ (lb/in.^{2}) = 1,350
6.	Mortar for concrete blocks	ASTM C270	$f'm_o$ (lb/in.^{2}) = 2,000

B. Reinforcing steel[b]

1.	Intermediate Grade No. 2 size round bars	ASTM A15	f_y (lb/in.^{2}) = 40,000
2.	No. 3 thru 11	ASTM A15 ASTM A305	f_y (lb/in.^{2}) = 40,000
3.	No. 14 and 18	ASTM A408	f_y (lb/in.^{2}) = 40,000

a. $f'c$ = specified compressive strength of concrete at 28 days
$f'm$ = specified compressive strength of masonry blocks at 28 days
$f'm_o$ = specified compressive strength of mortar at 28 days

b. fy = specified yield strength of steel
fpu = ultimate strength of prestressed tendons

4.	Welded Wire Mesh			
	10 gage and larger	ASTM A185	fy (lb/in.^{2})	= 65,000
	11 gage and smaller	ASTM A185	fy (lb/in.^{2})	= 56,000
5.	Prestressed tendons	ASTM A421-59T Type BA	f_{pu} (lb/in.^{2})	= 240,000
C.	Structural steel	ASTM A36	fy (lb/in.^{2})	= 36,000
D.	Miscellaneous steel			
1.	High-strength bolts			
	$\geq$ 1-1/8 inch	ASTM A325	fy (lb/in.^{2})	= 81,000
	$\leq$ 1 inch	ASTM A325	fy (lb/in.^{2})	to 92,000
2.	High-strength anchor bolts	ASTM A193, Grade B7	fy (lb/in.^{2})	= 105,000
3.	Anchor bolts	ASTM A307, Grade A	fy (lb/in.^{2})	= 36,000
4.	Stainless Steel plates	ASTM A167 Type 304	f_y (lb/in.^{2})	= 30,000
		ASTM A240 Type 304L	f_y (lb/in.^{2})	= 25,000
		ASTM A276 Type 304	fy (lb/in.^{2})	= 30,000
5.	Insert plates	ASTM A36	fy (lb/in.^{2})	= 36,000

The structures listed in table 3.2-1 were built of reinforced concrete, reinforced concrete block masonry (with special inspection), and/or structural steel, using methods common to heavy industrial construction.

3.8.5 Foundations

3.8.5.1 Description of Foundations

A description of the foundations of structures is presented in subsection 1.4.3 of reference (1).

3.8.5.2 Applicable Codes, Standards, and Specifications

The applicable codes, standards, and specifications, used in the structural design, fabrication, and construction of foundations, and the applicable codes, standards, and specifications to be used in the seismic reevaluation of foundations are discussed in paragraph 3.8.4.2.

3.8.5.3 Loads and Load Combinations

Foundation loads and loading combinations for structures are discussed in paragraph 3.8.4.3.

3.8.5.4 Reevaluation Analysis Procedures

Reevaluation analysis procedures, including computer programs to be employed in the reevaluation of foundations are discussed in paragraph 3.8.4.4.

3.8.5.5 Structural Acceptance Criteria

The structural acceptance criteria for foundations of structures will be the same as that which is discussed in paragraph 3.8.4.5.

A minimum factor of safety of 1.1 against overturning and sliding is maintained for all structures.

3.8.5.6 Materials and Special Construction Techniques

The foundations are built of reinforced concrete using conventional methods for heavy industrial construction. Materials utilized in the construction of foundations are discussed in paragraph 3.8.4.6.

REFERENCES

1. "Seismic Reevaluation Program, San Onofre Nuclear Generating Station Unit 1," March, 1978.

Enclosure 2

ADEQUACY OF SEISMIC DESIGN OF

SAN ONOFRE NUCLEAR GENERATING STATION

UNIT 1

April 21, 1980

TABLE OF CONTENTS

I. INTRODUCTION

The purpose of this report is to discuss the adequacy of the seismic design of San Onofre Unit 1 during the time that the seismic reevaluation is being performed. The specific items to be discussed are:

A. The conservatism in the use of a 0.67g Housner response spectrum for seismic reevaluation.

B. The capability of structures and equipment at San Onofre Unit 1 to withstand ground motion greater than the original seismic design based on a 0.5g Housner response spectrum.

C. Other factors which collectively indicate that since the original seismic design is based on a 0.5g Housner response spectrum, the plant can withstand instrumental ground motion in excess of that.

D. The probability of exceeding the 0.5g Housner design response spectrum and the corresponding instrumental response spectrum.

Based on the discussions in this report it is concluded that San Onofre Unit 1 can continue to operate during the time required to perform the seismic reevaluation without undue risk to the health and safety of the public.

II. CONSERVATION OF 0.67G HOUSNER RESONSE SPECTRUM

The San Onofre Unit 1 design was based on a 0.5g Housner response spectrum as shown in Figure 1, (Reference 1). However, for the purposes of seismic reevaluation, a 0.67g Housner response spectrum will be utilized. The geologic basis for the higher design acceleraion to be used for seismic reevaluation is the hypothesized Offshore Zone of Deformation (OZD). The USGS and NRC Staff have postulated that this feature extends from the Santa Monica Mountains in the north to Baja California in the south and is capable of large earthquakes resulting in accelerations of 0.67g at the San Onofre site. Geologic evidence, on the other hand, supports the fact that this zone is made up of short discontinuous segments no more than 40 km long. Structures which are capable of producing large earthquakes at the site are more than 18 miles away and are capable of producing accelerations of less than 1/4g at the site. Nevertheless, SCE has proceeded with seismic reevaluation of San Onofre Unit 1 based on a maximum ground motion of 0.67g by adopting the 0.67g Housner response spectrum for reevaluation purposes. Use of this spectrum is certainly conservative with respect to the geologic evidence. Moreover, this spectrum is also a conservative representation of an earthquake on the hypothesized OZD as discussed in the following paragraphs.

It is noted that the 0.67g Housner response spectrum is a design response spectrum. Instrumental response spectra (i.e., ground motion) are expected to exceed design response spectra. However, some degree of

conservatism of 0.67g Housner, a design response spectrum, can be established by comparison of this spectrum with instrumental spectra obtained in two separate ways: (1) source-modeling studies, and (2) historical data.

Source-modeling computations have been done for a very large earthquake on the hypothesized OZD adjacent to the plant (Reference 2). The results of these studies are shown in Figure 2 for the period range of interest, along with the 0.67g Housner spectrum. The computed results yield instrumental maximum accelerations of 0.31g for the mean, and 0.36g for the 84th percentile. Thus, the calculated instrumental maximum accelerations are significantly lower than the design value of 0.67g utilized in seismic reevaluation. Figure 2 also shows that the computed instrumental spectra for the mean and the 84th percentile lie at all points below the 0.67g Housner design spectrum.

The 1979 Imperial Valley earthquake furnished a great amount of high-quality instrumental data from a large earthquake representative of that which has been postulated offshore from San Onofre. In particular, fourteen records in the distance range of 6 to 13 km from the fault are of interest to San Onofre Unit 1. The spectral time histories of these records were analyzed at a number of periods, with the results shown for the mean and the 84th percentile in Figure 3. The instrumental maximum accelerations of the recorded data seem to be asymptotic about values of about 0.3g or less for the mean, and 0.35g or less for the 84th percentile. Thus, these measured instrumental maximum accelerations are significantly lower than the design value of 0.67g utilized in seismic reevaluation. The measured results also agree well with the source modeling computations of 0.31g and 0.36g for the mean and 84th percentile. For ease of comparison on Figure 3 the computed instrumental and the 0.67g Housner design spectra are also shown. The instrumental data for this earthquake are in general agreement with the computed instrumental spectra in the period range of interest. The 84th percentile instrumental data for this earthquake exceed the 0.67g Housner design spectrum at only one point, at a period of about .15 seconds.

As noted previously, the 0.67g Housner response spectrum is a design spectrum, whereas the spectra it is being compared against in the preceeding paragraphs (the source-modeling computations and the Imperial Valley earthquake data) are instrumental spectra. Nevertheless, the Housner spectrum is a conservative representation of instrumental ground motion in both cases and, therefore, is a conservative design response spectrum. The conservatism associated with using 0.67g Housner as a design response spectrum rather than an instrumental response spectrum is further discussed in Section IV.

III. APPRAISAL OF SEISMIC CAPABILITY OF STRUCTURES AND EQUIPMENT

The capacity of structures and equipment at San Onofre Unit 1 to withstand seismic inputs greater than 0.5g Housner is discussed in Appendix A. In particular, the effect of increasing the seismic input from the 0.5g Housner spectrum to a 0.67g Housner spectrum is addressed.

As described in Appendix A, an objective assessment of the existing data demonstrates that, if the design basis earthquake is increased from 0.5g to 0.67g Housner, the possible increase in stresses will be mitigated by a number of factors not considered in the original design. Furthermore, even in the absence of such mitigating factors, additional capacity in terms of ductility, increased material strength, factors of safety incorporated in the design, and other factors should enable engineered structural systems to respond without significant damage. These items are discussed briefly in the following paragraphs.

A. Material Properties

A structure is always intentionally designed and constructed to be stronger than the design criteria would indicate. This is due to such things as the use of standard sizes for construction materials, the fact that concrete mixes are intentionally designed to exceed the minimum design strengths to ensure passing of strength tests, and the fact that concrete strength increases with age due to the continuation of the hydration process.

For structures using mild steel, significant reserve capacity can exist due to inelastic response. Calculations show that a ductility factor of only 1.4 is by itself adequate to account for an increase in design criteria from 0.5g Housner to 0.67g Housner. Since allowable ductility values for typical mild steel are in the range of 2 to 6, and for concrete structures are 1.5 or more, it can be concluded that the ductility which exists in structures at San Onofre Unit 1 is sufficient to compensate for the increase in design criteria. In addition, the high ductility of mild steels allows for redistribution of loads to stronger members from members which have reached or approached yield.

For structural steel and reinforcing steel, the minimum values of yield stress specified by governing codes are used as the basis for allowable stress values with appropriate safety factors applied. Typically the actual yield stress and ultimate stress are greater than the specified minimum yield stress.

B. Damping

Much has been learned generically about structure and equipment damping since San Onofre Unit 1 was designed. This has led to the use of higher damping values than those used in the original design. For example, in the original design, a damping value of 1/2 percent was used for piping. However, currently a value of 2 or 3 percent is permitted by Regulatory Guide 1.61. A comparison of response spectra associated with these damping values is shown in Figure 4. As can be seen from this figure, the 0.67g Housner spectrum with current damping values is less than the original design spectrum for piping in the frequency range of interest (less than 20 Hz).

Another example of the conservative damping used in the design of San Onofre Unit 1 is the conduit and raceway systems. Recent tests have shown that damping values up to 50% of critical do occur in raceway systems similar to those installed at San Onofre Unit 1. Due to this high level of damping these raceway supports can be expected to survive severe earthquakes (up to .75g) with no loss of function in the circuits they support (Reference 3).

C. Piping Systems and Equipment

The original design of piping systems at San Onofre Unit 1 was based on the 1955 version of the B31.1 Code for Power Piping. The fundamental basis for this code is the development of a balance between flexibility and control. An inherent property of piping designed with this concept is the ability to absorb large amounts of energy. A large amount of contemporary evidence is available, as summarized in Appendix A, which shows that piping systems designed with controlled flexibility have the capacity to withstand forces far in excess of the forces for which they were designed. Power plants have experienced ground accelerations in excess of twice the original design basis while sustaining virtually no damage to piping systems. Moreover, this same historical evidence allows us to draw similar conclusions with respect to mechanical and electrical equipment. Equipment has experienced ground motion far in excess of that to which it was designed and survived without loss of function. In general, power plants have either remained on line or have returned to service in a very short time following seismic events. The ability to continue to operate demonstrates that the equipment in the plants survived the event without loss of integrity or function.

An example of the capacity of San Onofre Unit 1 to withstand seismic input greater than the design basis 0.5g Housner spectrum as a result of the various effects discussed above is the recent reevaluation of the containment, reactor building and nuclear steam supply system (NSSS), otherwise known as the Seismic Backfit Project (SBP). The reanalysis of the containment, the reactor building, structural steel framing and NSSS piping revealed that these structures and equipment have resistance capacities in excess of those required to meet the 0.67g Housner spectrum; and therefore, no modifications were required. Additional restraints were required for large NSSS equipment which was base-supported. However, this latter condition is unique to the NSSS components with large height-to-diameter ratios and is not typical for base-supported equipment in the balance of the plant. Since the remaining Category A structures and equipment at San Onofre Unit 1 were designed and constructed in the same manner as the above, it is likely that seismic reevaluation of the balance of plant will not result in stresses in excess of allowable for the remaining structures and equipment.

In addition, SCE has initiated a significant effort in the area of demonstrating the seismic adequacy of electrical equipment. Specifically, all safety-related electrical equipment is being inspected during the April-June 1980 refueling outage to ensure that positive anchorage or support exists. Instances where such positive anchorage cannot be demonstrated will be evaluated and required modifications will be implemented during the refueling outage. In addition to the in-plant inspections, the design adequacy of the anchorage for all safety-related electrical equipment will be verified by September of 1980.

IV. OTHER FACTORS

In addition to the more quantitative factors discussed in the previous sections, there are a number of other more qualitative factors which have a bearing on the seismic adequacy of San Onofre Unit 1 during the reevaluation period. Although these other factors are not necessarily completely independent of each other, they are addressed individually in the following paragraphs.

A. Generic Observations

Accelerations which are at wavelengths which are short compared to the characteristic plan dimension of a structure will have less than a one-to-one effect on the structure. In principle, a large structure only sees the average effects of such short-period waves, whereas it responds more faithfully to wavelengths greater than the characteristic plan dimensions (References 4, 5 and 6). Thus, it would be necessary to lower the short-period end of the instrumental ground motion spectrum to define the design spectrum, as shown in Figure 5. For this effect, short-period means less than 1/2 second (Reference 7). At short periods, the ratio between design and instrumental accelerations can be less for large structures than for small structures, especially for nearby earthquakes.

A structure which is embedded is expected to experience lower response than one with its foundation near the surface because, among other factors, the deeper structure tends to scatter the incoming waves, and to respond interactively with the soil surrounding it (References 8 and 9). The result is a decrease of response from the instrumental values, at short periods, as shown qualitatively in Figure 5. Even for the relatively shallow depths of embedment of the structures which are embedded at San Onofre Unit 1, some effect is still expected.

A material or structure can sustain a higher load for a few cycles of loading than it can for many cycles. The effect is basic to machine design and is recognized as a fundamental aspect of soil-mass behavior. Thus it is generally agreed that it is reasonable to base the design of structures on those values of maximum acceleration which are repeated several times (Reference 10), rather than on the anticipated maximum peak.

Although earthquake calculations are done for mathematical convenience by assuming that the waves are plane, it is generally agreed that they are in fact incoherent. The heterogeneity of the faulting process and the natural heterogeneities in the transmitting earth materials virtually preclude planeness of the wave front. For this reason, two seismometer stations very close to each other would be expected to record different time histories; and a large structure would respond to a time average of the two time histories because the peak accelerations would not be applied simultaneously to all points on the structure. Thus, the maximum acceleration of a large structure would a priori be lower than an instrumental maximum acceleration based on seismometer time-histories.

The original design of San Onofre Unit 1 was based on dampings for the structures alone, and did not take into account the combined soil-structure system. Vibration (plucking) tests performed at the site (Reference 11) on large test foundations, at the surface and embedded, have shown that the combined soil-structure dampings are much larger than those used in the original design. Using the logarithmic decrement of the decaying vibrations, it was possible to compute the combined radiation and hysteretic damping for the test foundations. Then, using the theory of the elastic half-space, the test dampings were used to calculate the dampings for actual structures. Those dampings were substantially higher than those used in the original design or to be used in the reanalysis. Thus, the combined soil-structure systems have a conservative energy-dissipating capability not accounted for in the original design response spectrum or the reanalysis response spectrum.

B. Quantitative Assessment

Although significant advances have been made in the mathematical analyses of the several factors discussed above, the generic modeling of the entire physical processes is by no means complete. For this reason, it appears prudent to rely on actual data for a general assessment of the sum of all the factors. Such an assessment must be done carefully, because the effects will vary for large and small earthquakes, with greater or lesser distances from the event, with the size, mass, and depth of burial of the structure, with the type of faulting, and with the intervening geology. With these limitations in mind, a limited study was made of available data.

Valuable data on the performance of heavy reactor structures were obtained during the June 6, 1975 Ferndale, California earthquake. Seismometers had been located outside a reactor structure, on the ground surface, to record the free-field instrumental motions; and inside the reactor structure to record the structure's response.

The in-structure maximum acceleration was only 46 percent of the instrumental maximum acceleration in the free-field. Perhaps more importantly, the spectrally amplified structural motions were also always less than the instrumental spectral accelerations, as shown in Figure 6. The in-structure spectral accelerations ranged from as low as 25 percent to as much as 75 percent of the instrumental free-field values.

Similar results have evolved from studies of the situation from a different viewpoint (Reference 12). Figure 7 shows the experience of six reinforced-concrete structures during the February 9, 1971 San Fernando earthquake. The structures were designed by Uniform Building Code procedures for base-shear coefficients from 4 to 8 percent gravity (shown as squares). Of the six structures, three sustained no structural damage (open circles), and three sustained some structural damage (open circles with tick marks). Those which experienced no structural damage experienced responses during the earthquake which were from 2 to 3 times the values for which they had been designed, using UBC procedures.

These data argue strongly that modern reinforced-concrete structures, designed and built according to current practices, have actual earthquake resistance capacities which are conservatively greater than those for which they were designed. Some reasons for this have been mentioned above: sizing actual members to the next greater standard size; the actual material strengths of concrete and steel are greater than specified in the design; aging of concrete; less than one-to-one response to short wavelength components of the earthquake; embedment effects; lack of effects on the structure of only a few applications of the higher peak accelerations; probabilistic incoherence of the incoming waves so that peak accelerations are not applied simultaneously to all points on the structure; and soil-structure interactive damping which causes the soil-structure system to have extra energy-absorbing capabilities.

The influence of some of these factors is demonstrated in Table 1, which presents data from five well recorded earthquakes. In each case, records from seismometers in the structure and in the adjacent free-field were available, so that the ratio between in-structure maximum acceleration and free-field instrumental maximum acceleration could be compared. The results are shown in the right column of Table 1. The ratio of in-structure to free-field maximum acceleration varies from as low as 46 percent to as high as 78 percent. The lowest value, 46 percent, is for the heavy reactor structure, as would be expected based on the physical arguments developed above. If the ratio of in-structure to free-field acceleration is noted as r, then the relationship of structural acceleration, S, to applied free-field acceleration, F, would be:

$$S = rF$$

TABLE 1

Earthquake	Date	Magnitude (M_L)	Distance (km)	Station Location	Ratio of In-Structure to Free-Field Acceleration
Kern County	07-21-52	7.2	107	Hollywood Storage Bldg.	78
San Fernando	02-09-71	6.4	35	Hollywood Storage Bldg.	62
			39	3407 W. 6th, LA 616 S. Normandie, LA 3345 Wilshire, LA 3411 Wilshire, LA 3550 Wilshire, LA	66
Lillis Ranch	09-03-75	4.9	18	Pleasant Valley Pump Plant	62
Ferndale	06-07-75	5.3	25	Humboldt Bay Power Plant	46
El Centro	10-15-79	6.6	5	El Centro Station 9 El Centro Diff. Array	75

For the present situation, where the structures have been designed in terms of the design maximum acceleration, S_d, it is convenient to rewrite the above expression in terms of the equivalent instrumental maximum acceleration, F_t, which corresponds to the design value:

$$F_t = 1/r\ S_d$$

From this viewpoint, the data which is available to date, as summarized in Table 1, indicate that equivalent instrumental accelerations range from 1.3 to 2 times the maximum accelerations for which a structure is designed. Using these ratios, the San Onofre Unit 1 design value of 0.5g is expected to be equivalent to instrumental maximum accelerations from 0.65g to 1.0g. Using these values as the zero-period acceleration, referring to Figure 5, and noting (Reference 7) that the point of tangency of the instrumental spectrum to the design spectrum is 1/2 second or more for large structures and nearby earthquakes, approximate equivalent instrumental spectra can be drawn. This has been done, as shown in Figure 8, based on the 0.5g Housner design spectrum. Based on the physical principles and the data discussed above, that spectrum range shown in Figure 8 is the instrumental ground motion which a large structure designed to the 0.5g Housner spectrum would be expected to withstand due to a large nearby earthquake. It can be further observed that the 0.67g Housner response spectrum is within the range of equivalent instrumental spectra shown on Figure 8. Therefore, a large structure designed to the 0.5g Housner spectrum could be expected to withstand an instrumental spectrum which is identical to 0.67g Housner.

V. PROBABILITY STUDY

A study was performed to determine the probability of exceeding instrumental maximum spectral accelerations. Appendix B provides a detailed evaluation of the probabilities of exceeding the design spectrum for San Onofre Unit 1. Specifically, instrumental response spectra with equal probabilities of exceedance are provided. Using the results of Appendix B the probability of exceeding any part of the instrumental spectrum band associated with the design 0.5g Housner spectrum (i.e., Figure 8) is about 6×10^{-4}. Additionally, sensitivity studies performed on the input parameters to these calculations as discussed in Appendix B show only a small variation in probability.

VI. CONCLUSIONS

The preceeding sections of this report have discussed several independent factors which relate to the seismic adequacy of San Onofre Unit 1 during the time that seismic reevaluation is being performed. Specifically, the following geologic, engineering and probabilistic evidence has been presented:

A. The 0.67g Housner response spectrum is a conservative representation of ground motion which would be expected from an earthquake on the hypothesized OZD and, therefore, a conservative design response spectrum based on

1. recent source-modeling studies which indicate that the site maximum isntrumental acceleration due to a large nearby earthquake will be 0.31g for the mean and 0.36g for the 84th percentile,

2. data from the 1979 Imperial Valley earthquake which indicate that at a distance of 8 km from a fault, the site maximum instrumental acceleration is about 0.4g for the 84th percentile,

3. the 0.67g Housner response spectrum lies at all points above the 84th percentile instrumental spectrum from the source-modeling study, and

4. the 0.67g Housner response spectrum generally lies above the 84th percentile instrumental spectrum developed from data measured at 6 to 13 km from the Imperial Valley earthquake.

B. Consideration of the OZD in establishing the seismic design basis with which to compare the existing seismic design is conservative based on

1. the fact that although the hypothesized OZD has been postulated by the NRC and USGS it remains the licensee's position that the geologic evidence indicates that the hypothesized OZD is made up of short discontinuous segments, and

2. structures capable of producing large earthquakes are distant and capable of producing less than 1/4g at the San Onofre site.

C. Structures and equipment at San Onofre Unit 1 can withstand ground motion in excess of the 0.5g Housner design response spectrum based on

1. the fact that reanalyses of the containment, reactor building, steel framing and NSSS piping revealed that those structures, as they were built and exist today, have resistance capacities in excess of that required to meet the 0.67g Housner spectrum,

2. the fact that additional capacity in terms of ductility, increased material strength and other factors should enable engineered structural systems to respond to seismic input larger than the design basis without significant damage,

3. recent experience of power plants in large earthquakes has demonstrated the ability of piping systems and equipment to survive seismic forces far in excess of the forces for which they were designed without loss of integrity or function, and

4. the fact that the original 0.5g design maximum acceleration corresponds to an equivalent instrumental maximum acceleration of at least 0.65g.

D. The probability of having an earthquake during the reevaluation period which exceeds the design basis of the plant is small based on

1. the fact that the probability of exceeding the instrumental maximum acceleration (0.65g or greater) is less than about 1×10^{-4} per year, and

2. the fact that the probability of exceeding the instrumental spectrum equivalent to the 0.5g Housner design spectrum is less than about 6×10^{-4} per year.

Based on the combination of all of these separate considerations it is concluded that San Onofre Unit 1 can continue to operate without undue risk to the health and safety of the public during the time required to perform the seismic reevaluation.

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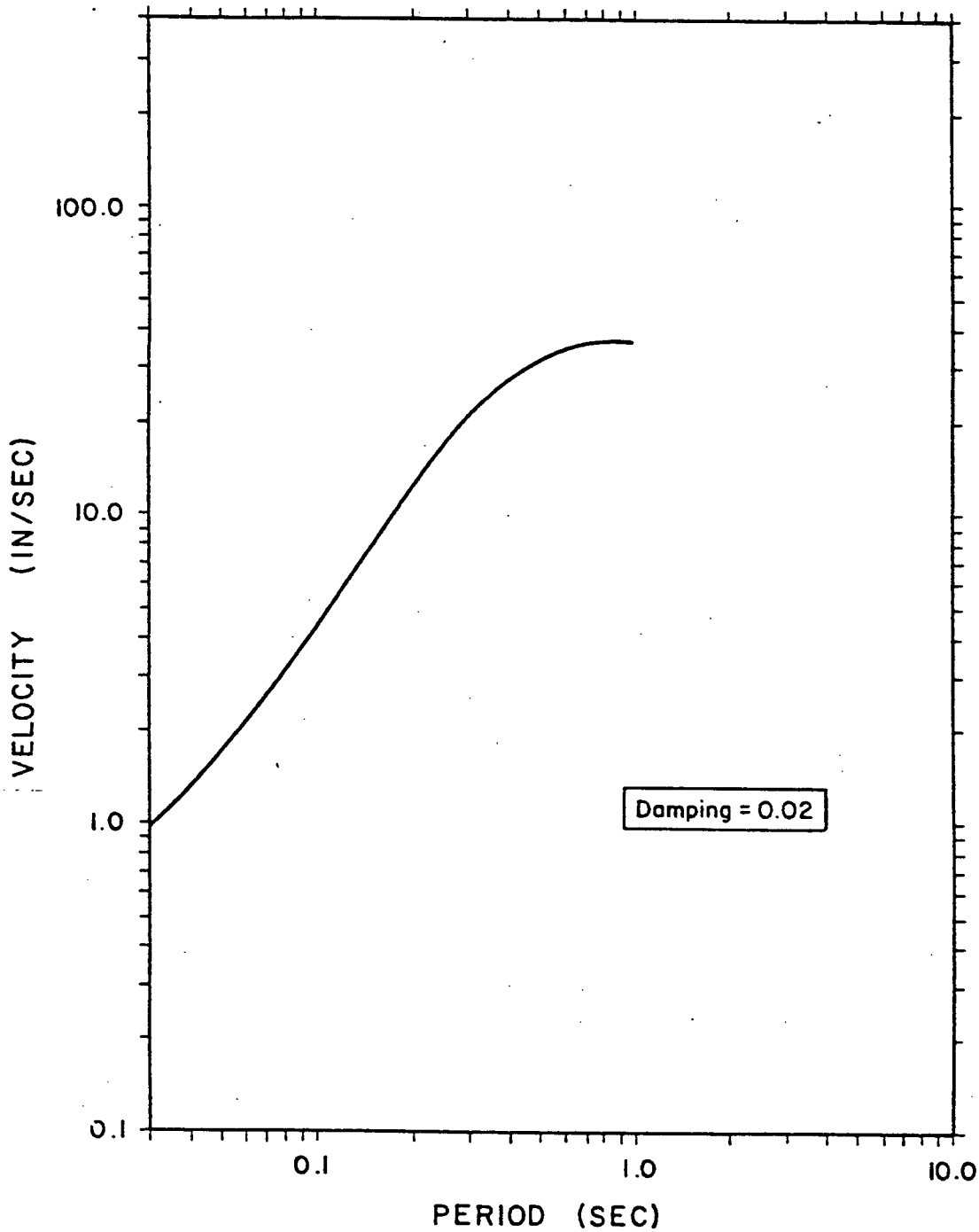


Fig. 1. Design Spectrum for SONGS Unit 1

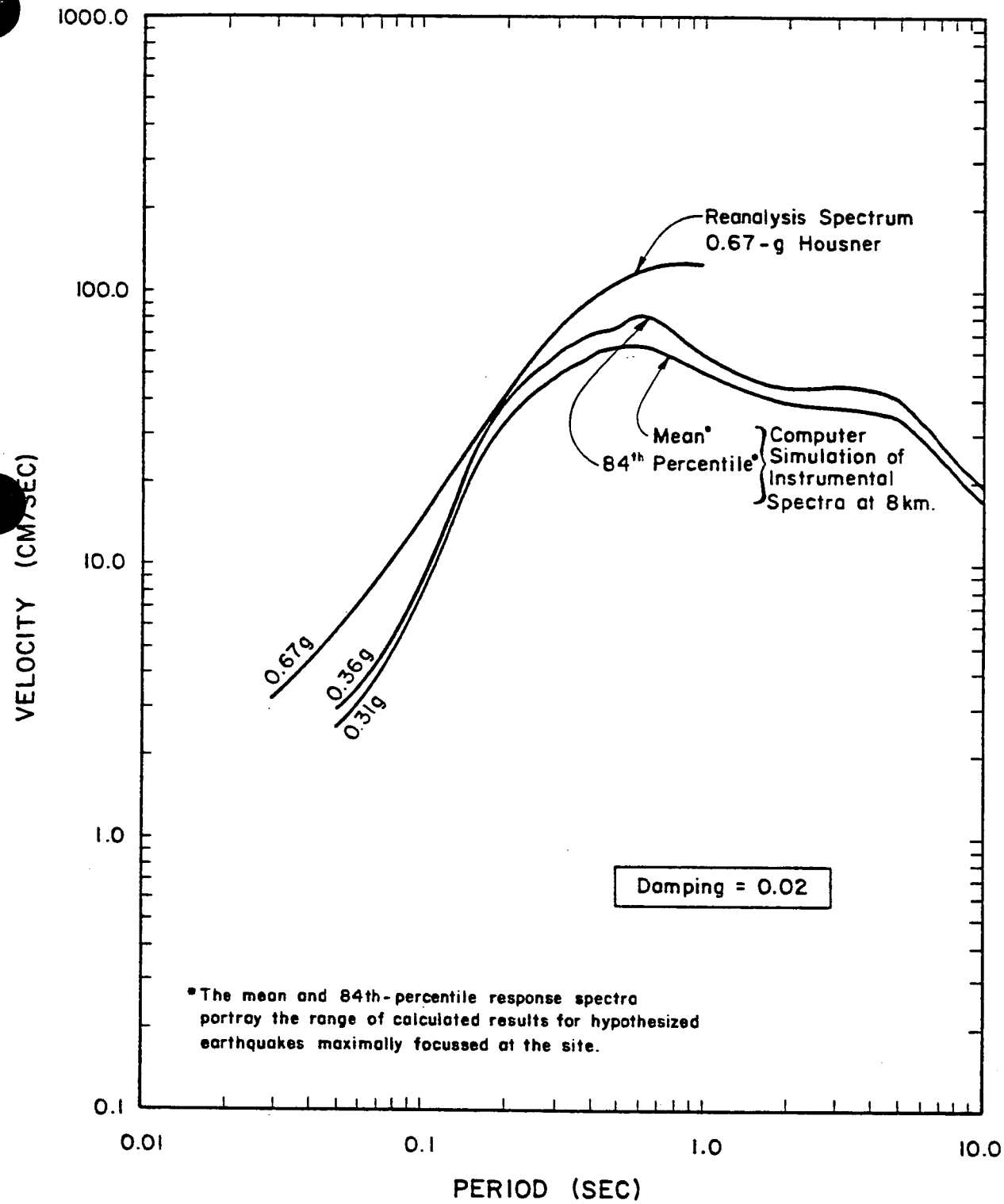


Fig. 2. Computer Simulation of Site-Specific Earthquake

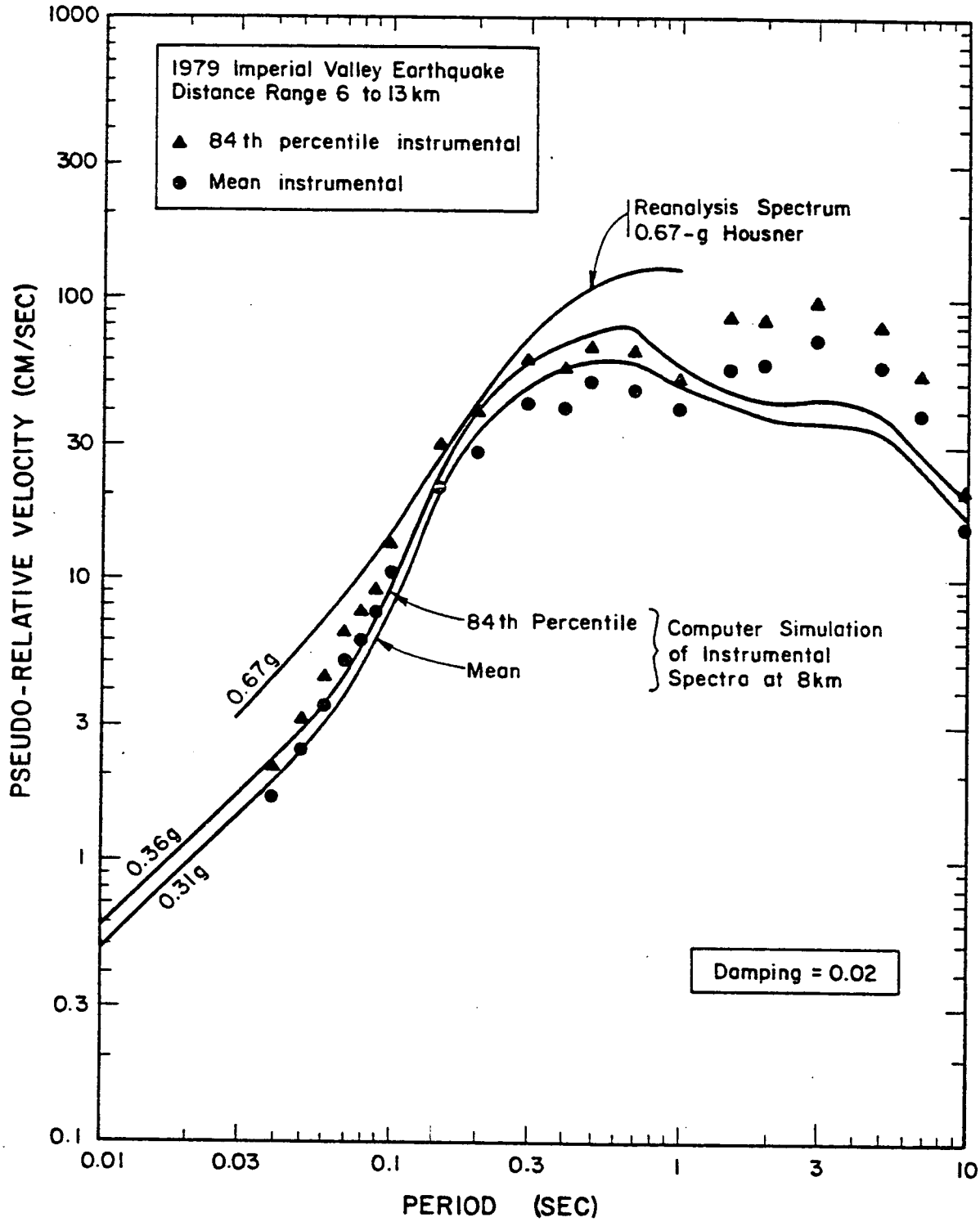
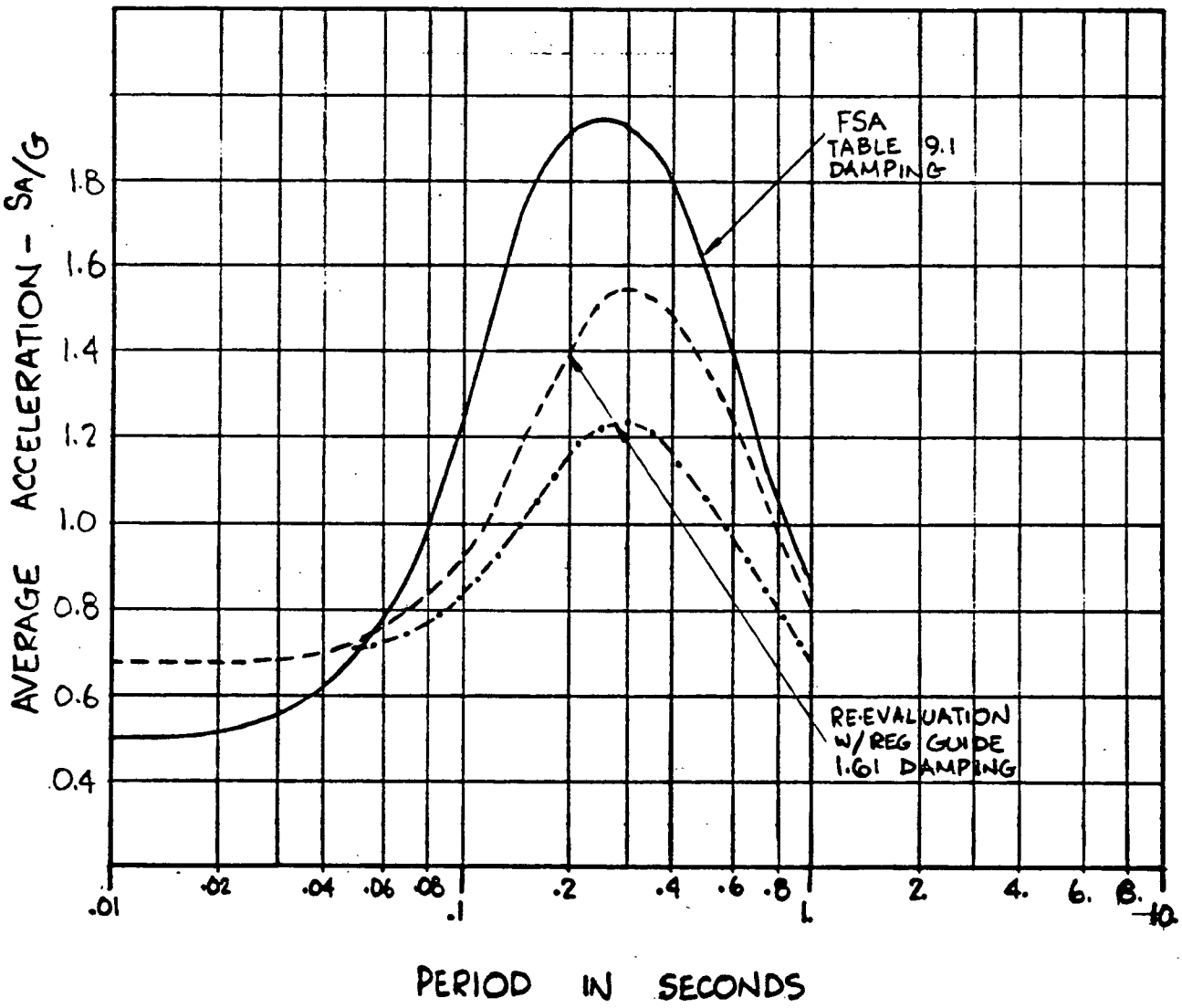


Fig. 3. Comparison of SONGS Unit I Computer Simulation Spectra with 1979 Imperial Valley Earthquake Spectra and Reanalysis Spectrum.

FIGURE 4. PIPING
RESPONSE SPECTRA



——— .5g HOUSNER @ ½% DAMPING (VITAL PIPING)
- - - - .67g HOUSNER @ 2% DAMPING (SMALL DIAMETER PIPING)
-·-·-·- .67g HOUSNER @ 3% DAMPING (LARGE DIAMETER PIPING)

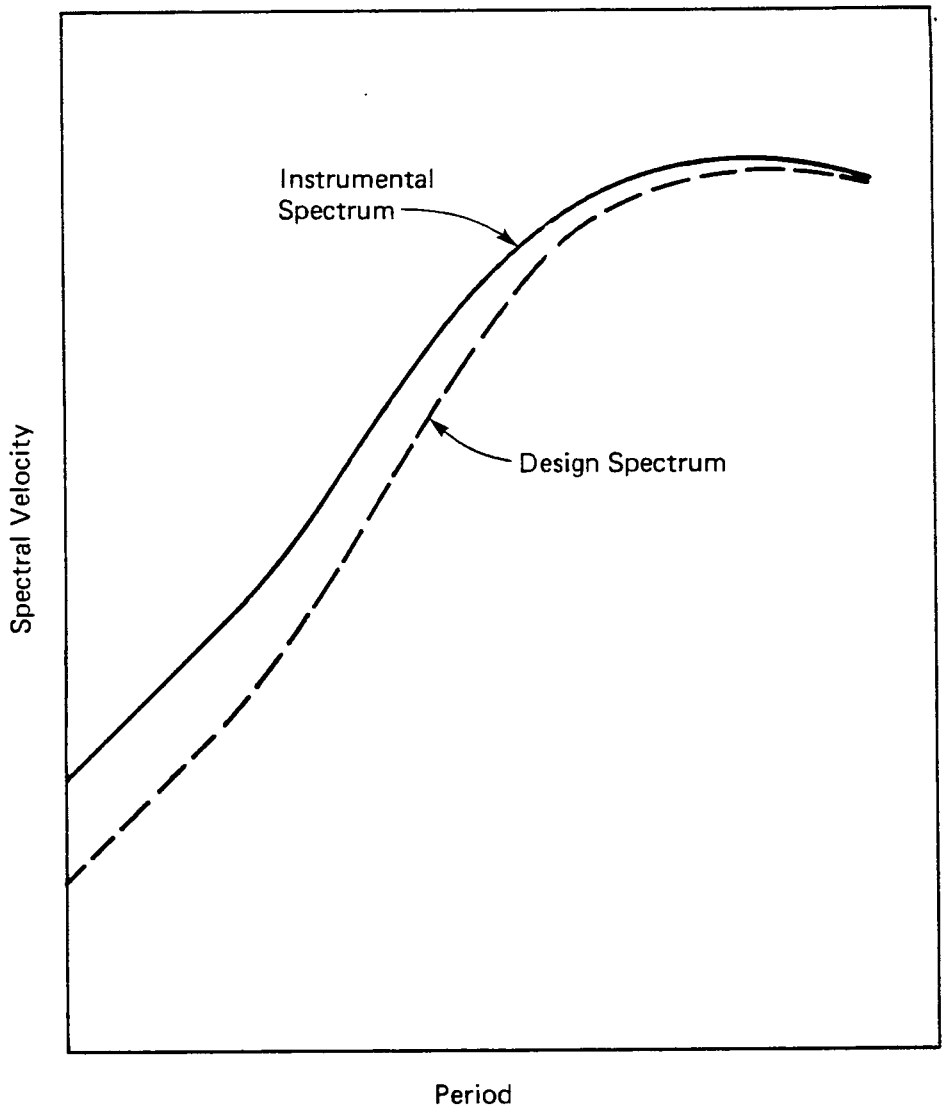


Fig. 5 Comparison of Instrumental Spectrum to Design Spectrum for Nearby Large Earthquake

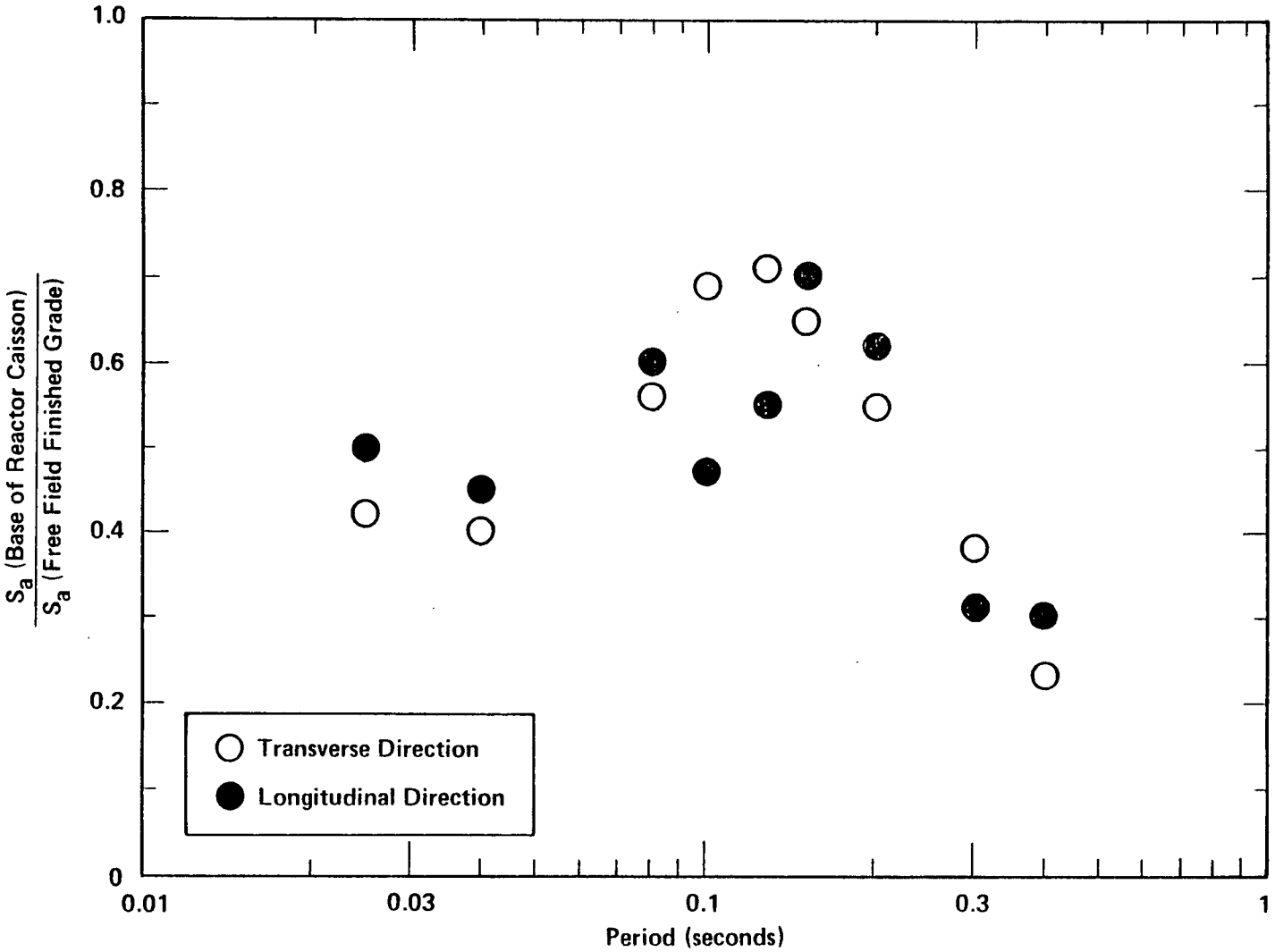


Fig. 6 Ratios of Response Spectra at the Base of the Reactor Caisson at the Humboldt Bay Power Plant to Response Spectra at Finished Grade in the Free Field from Recordings obtained during the June 6, 1975, Ferndale, California, Earthquake

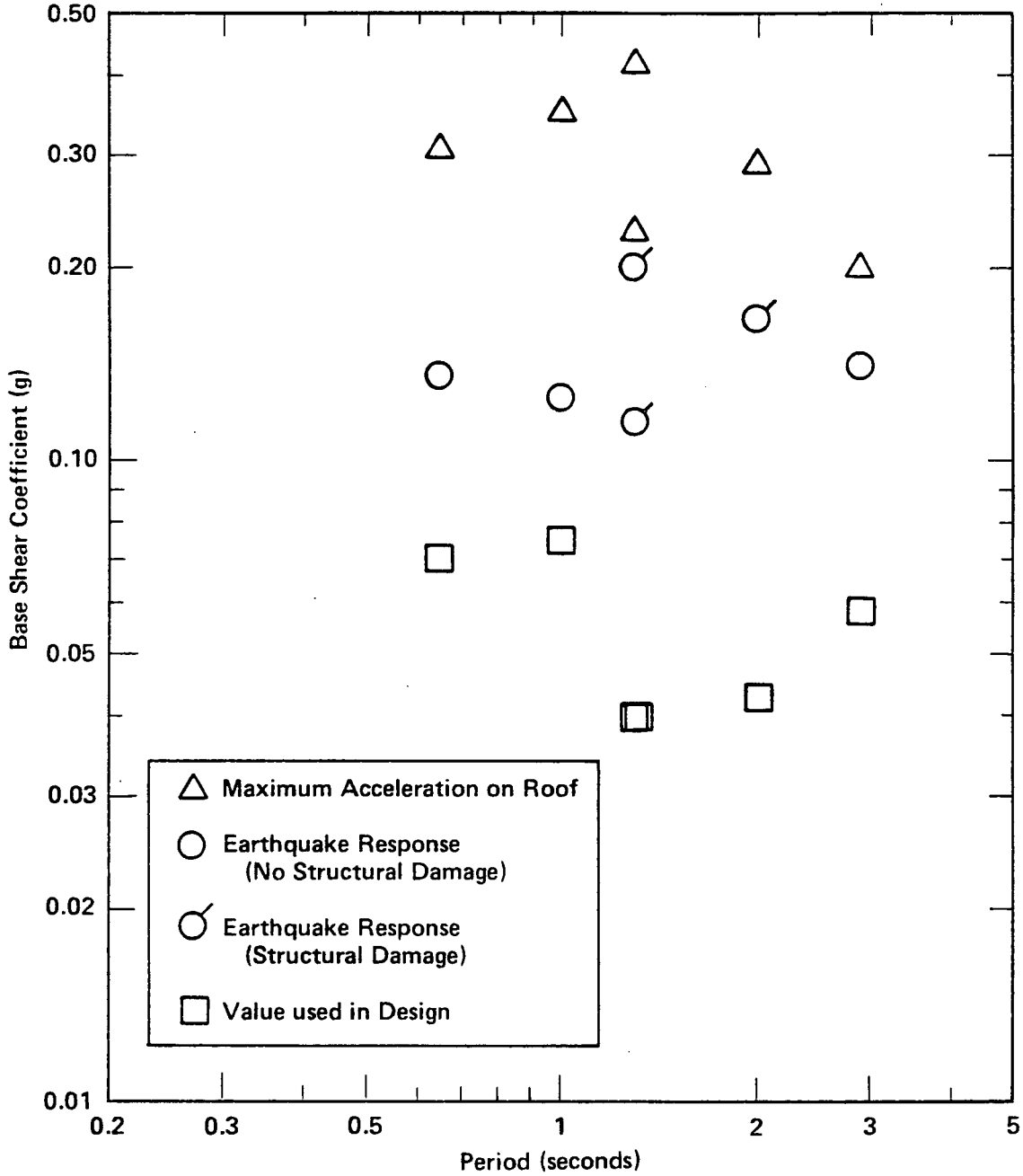


Fig. 7 Capacity of Reinforced Concrete Structures Demonstrated during the San Fernando Earthquake

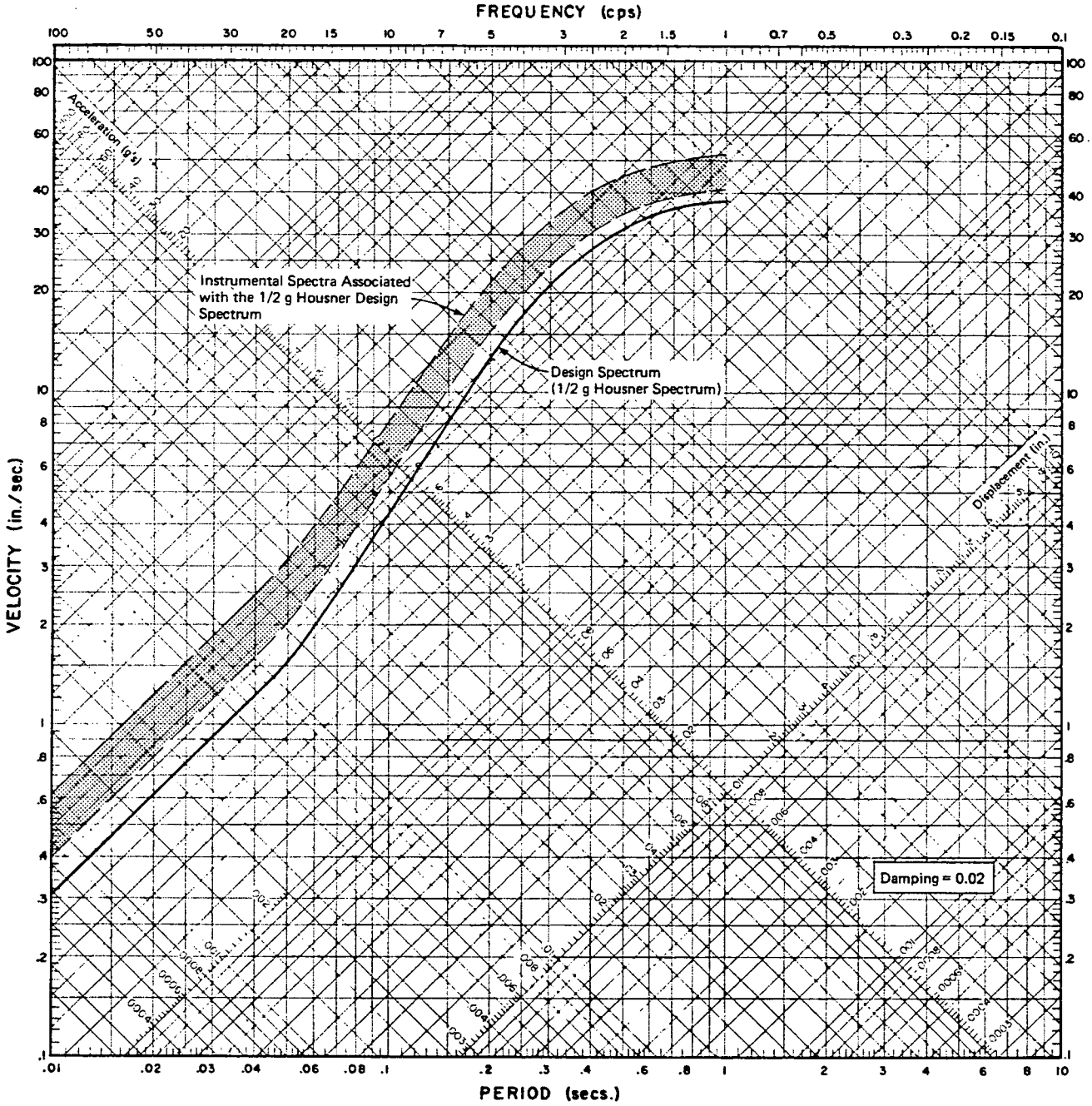


Fig. 8 Comparison of Housner Design Spectrum with Associated Instrumental Spectra

APPENDIX A

EXPECTED CAPACITY OF STRUCTURES AND EQUIPMENT DESIGNED TO ½G HOUSNER SPECTRUM

SAN ONOFRE NUCLEAR GENERATING STATION

UNIT 1

EXPECTED CAPACITY OF STRUCTURES AND EQUIPMENT DESIGNED TO 1/2G HOUSNER SPECTRUM

SAN ONOFRE NUCLEAR GENERATION STATION

UNIT 1

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APRIL 21, 1980

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1.0 INTRODUCTION

The purpose of this report is to provide an assessment of the capacity of structures and equipment at San Onofre Unit 1 to resist increased seismic inputs. In particular, the effect of increasing the seismic input from a 1/2g Housner Spectrum to a 2/3g Housner Spectrum is addressed.

The basis for inferring that modern engineered structures (especially nuclear power plant structures) have the capacity to survive earthquakes whose magnitudes and peak instrumental accelerations exceed the design basis acceleration levels has been well established both through observation and analytical studies, which consider factors not included in conventional analyses. Recent work by Newmark (reference 1), Blume (reference 2), and Cloud (reference 7), have identified and defined some of the parameters which provide for increased capacity.

Although the concept of significant reserve capacity is well established, there have been recent examples of modern structures suffering severe damage, if not outright collapse, during major earthquakes. Two examples of this were the Olive View Hospital during the 1971 San Fernando Earthquake and the County Services building in El Centro during the 1979 Imperial Valley earthquake. Both of these structures were characterized by a soft first story consisting of columns at the periphery of the structure and more rigid upper stories with heavy curtain walls at the periphery of the building. The damage to these structures was clearly a result of the soft first story. This feature is not found in structures at San Onofre Unit 1. Therefore, the probability of significant reserve capacity in structures at San Onofre is not affected by the observed

damage of these structures.

The following sections of this report describe the experience gained in the seismic reanalysis of portions of San Onofre Unit 1 to a 2/3g Housner Spectrum and discuss the potential for reserve capacity in the balance of the plant based upon the previous analysis, the current state-of-the-art, and the criteria used in the original design of San Onofre Unit 1.

2.0 THE IMPACT ON STRUCTURES AND EQUIPMENT OF INCREASING THE DBE FROM 1/2g HOUSNER TO 2/3g HOUSNER

The effect of increasing the DBE from 1/2g Housner to 2/3g Housner may be evaluated in several ways. The assessment provided in this discussion is based upon the results of the seismic reevaluation and modification of the containment, reactor building and NSSS, referred to as the Seismic Backfit Project (SBP), and upon a comparison of the current state-of-the-art with the original design basis.

The study phase of the SBP was performed in the period from 1974 through 1975. The purpose of the study was to determine the feasibility of qualifying the containment sphere, reactor building and NSSS to resist a 2/3g Housner Spectrum. The original design basis for these structures and equipment was the 1/2g Housner Spectrum. The results of the SBP provide a basis that may be used to gauge the likely impact of increasing the seismic design basis to 2/3g. As a result of the reevaluation no modifications to structures were required, The reasons for this are discussed in detail below. Some modifications to equipment supports were required to increase resistance to overturning. These modifications were installed during an extended refueling outage in 1976.

In addition to evaluating the results of the SBP, a comparison of the design assumptions, criteria, and analysis techniques can be made. Specifically, the effects of increased seismic inputs may be mitigated by other factors. For example, the damping values currently used for piping and electrical raceway systems are higher than the values cited in Table 9-1 of the San Onofre Unit 1 FSA. Also, in-situ material properties as

documented by testing can provide a basis for establishing increased allowable stresses in a manner that is consistent with code requirements.

The following sections discuss the likely impact on structures and equipment of increasing the DBE from 1/2g to 2/3g, based upon the results of the SBP. The expected effects of increased seismic input on piping and electrical raceways are also discussed in light of current criteria and recent testing.

2.1 Reevaluation of Existing Structures

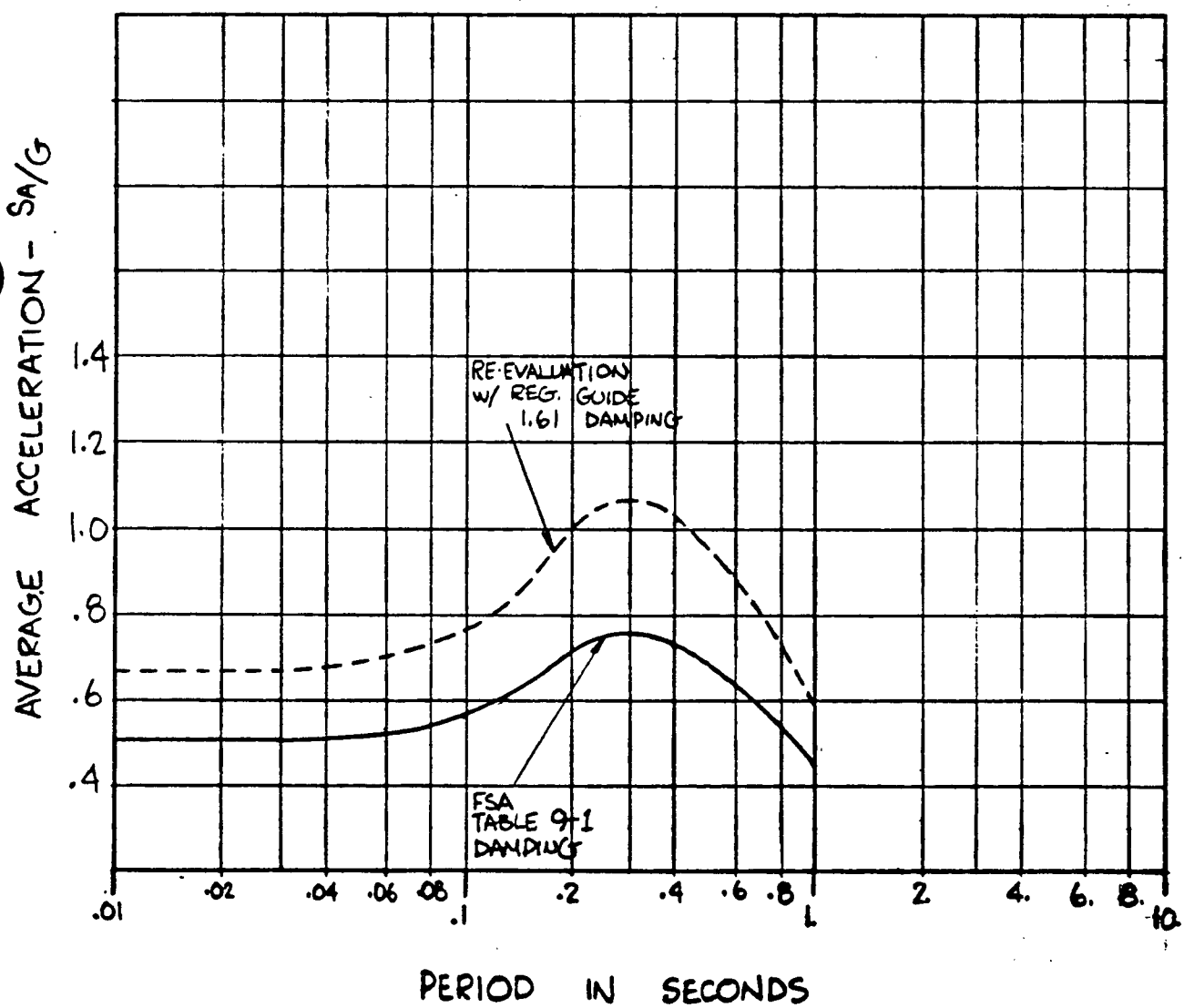
2.1.1 Steel Containment Sphere

The seismic reanalysis of the steel containment sphere was performed in 1974-75. A comparison of inputs for the original design and the seismic reanalysis is shown in Figure 1.

In the reanalysis the containment structure, the reactor building, their foundations, and the surrounding soil media were modelled with an axisymmetric finite-element computer code. The modelling and analysis techniques conformed to currently accepted practice and are documented in detail in Reference 3. A conservative damping of 4% was assumed for all modes in the reanalysis.

As part of the reanalysis an evaluation of in-situ material properties was made. Documented test data were obtained which indicated that, in accordance with code philosophy, the actual allowable stresses could be increased by about 12% over the allowable determined from specified minimum design material properties. Based upon these increased allowable stresses it

FIGURE 1. CONTAINMENT VESSEL RESPONSE SPECTRA



——— .5g HOUSNER @ 4% DAMPING

– – – .67g HOUSNER @ 4% DAMPING

was demonstrated that the stresses in the steel sphere, including penetrations, would not exceed allowable stresses for an increased seismic input from 1/2g to 2/3g (see Figure 1). Primary membrane tensile stresses were limited to approximately 2/3 yield while secondary tensile stresses were generally less than 1/2 ultimate. Compressive stresses were within the limits determined in accordance with paragraphs NB3133.4, NB3133.6, and NE313102 of the ASME Boiler and Pressure Vessel Code. These conditions indicate that the allowable stresses provide substantial margins against failure.

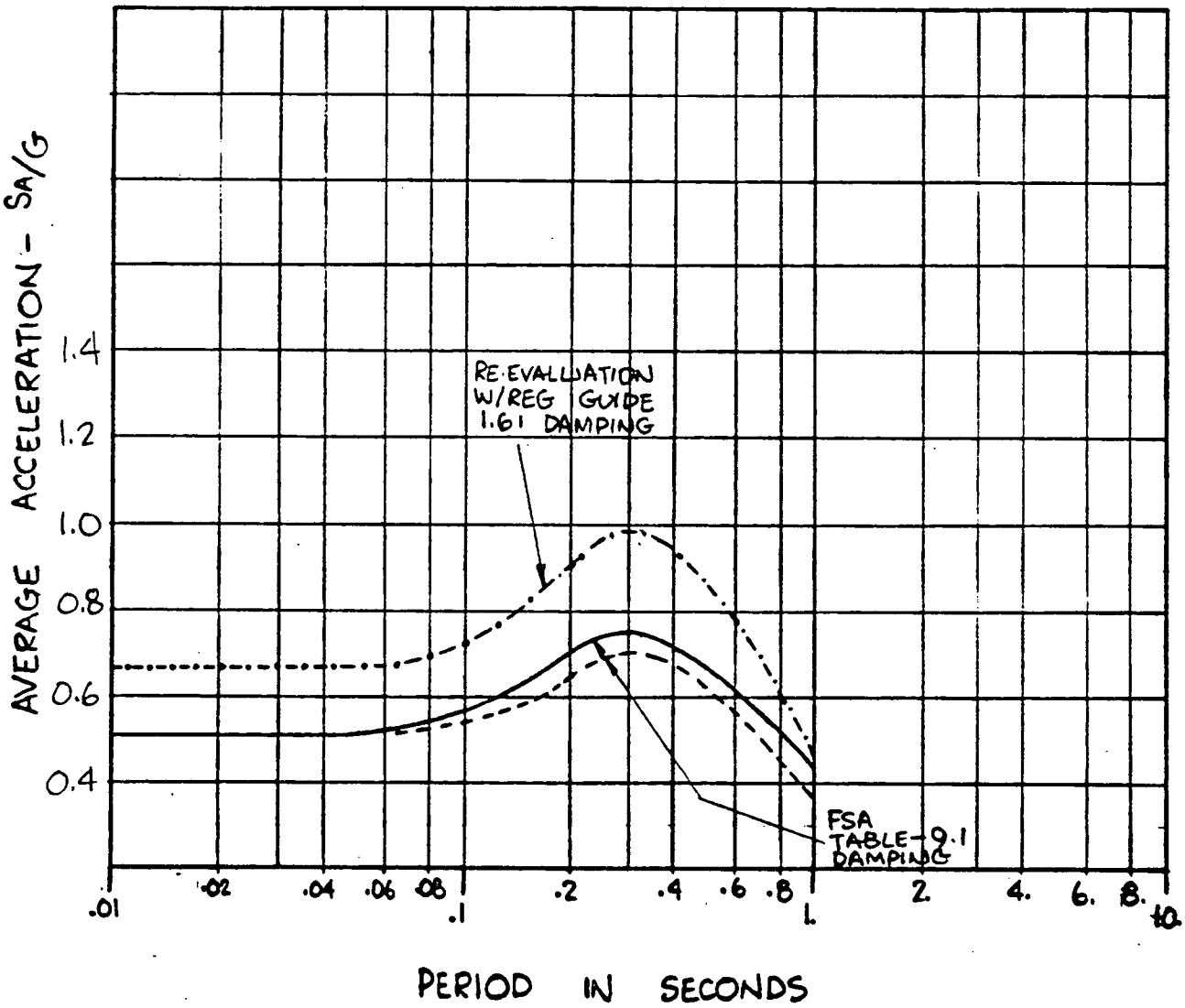
2.1.2 Concrete Reactor Building

The seismic reanalysis of the concrete reactor building was performed in 1974-75. A comparison of inputs is difficult since the original design was based upon simplified analysis techniques while the reanalysis utilized a three-dimensional finite element model, including soil structure interaction. In addition, the equipment loads in the reanalysis were based upon a nonlinear analysis which included the effects of impact. Taking these factors into account an approximate comparison of inputs is presented in figure 2.

The reanalysis models and the analysis methodologies conformed to currently accepted practice and are discussed in detail in reference 3.

The results of the reanalysis, as documented in reference 3, demonstrate that stresses in the concrete reactor structure will not exceed allowable stresses for a 2/3g Housner Spectrum.

FIGURE 2. CONCRETE STRUCTURES RESPONSE SPECTRA



——— .5g HOUSNER @ 5% DAMPING (RIGID CONCRETE FRAMES)

- - - - - .5g HOUSNER @ 7% DAMPING (CONCRETE SHEAR WALLS)

-·-·- .67 HOUSNER @ 7% DAMPING (REINF. CONCRETE STRUCTURES)

No credit was taken for in-situ material properties in making this determination. Subsequent to the reanalysis, independent tests were performed on several of the concrete structures at San Onofre, including the reactor building, using the Windsor Probe and the Schmidt Hammer. Both tests indicated that the in-situ concrete strengths exceeded the design value (3000 psi) by a factor of 2 or more [1]. In view of this, shear capacity would be substantially greater than for the specified strength of 3000 psi. The effect of increased concrete strength on the capacity of structural members is described below in Section 3.3.1.

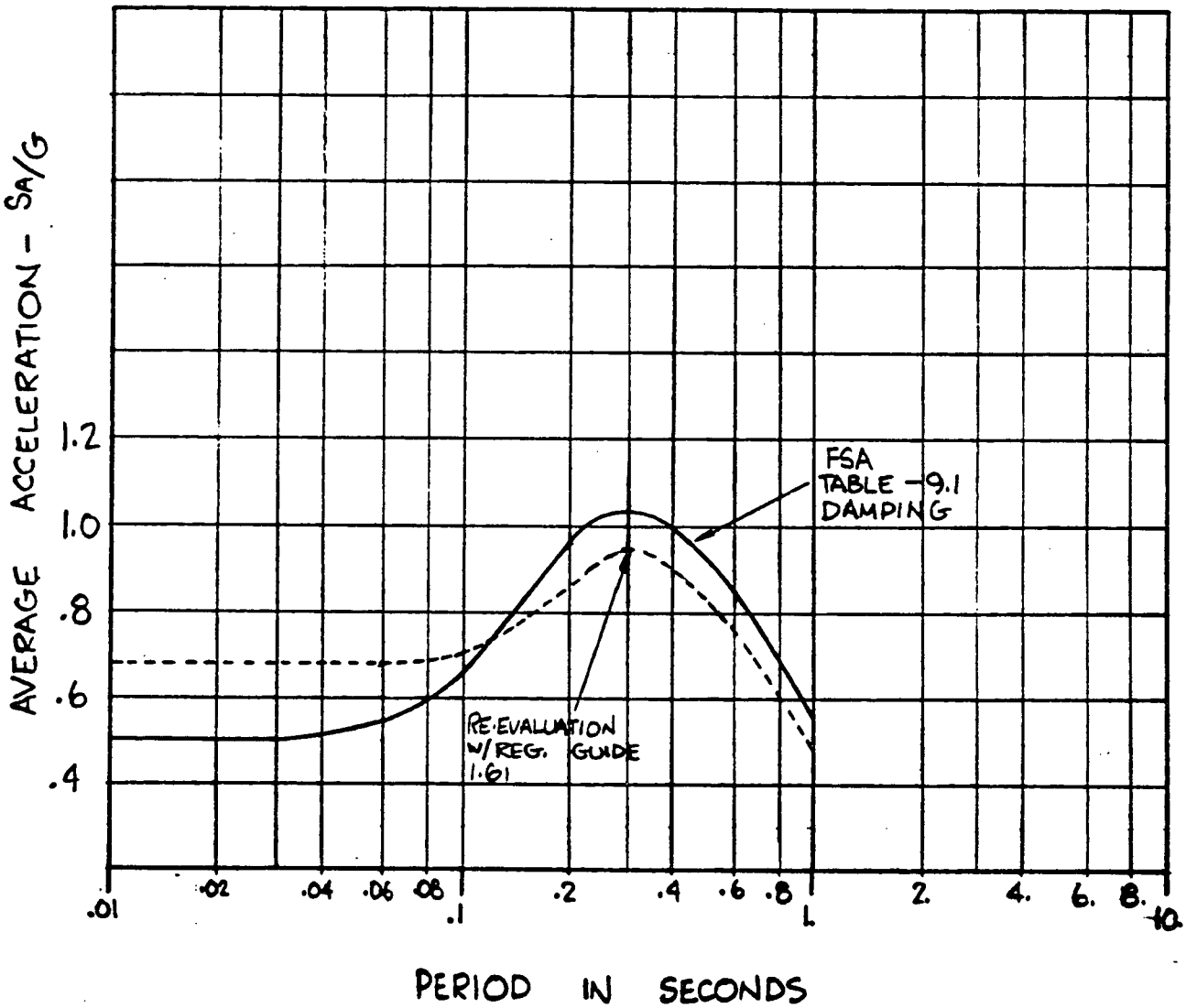
In summary, the concrete reactor building was demonstrated to be within allowable stresses for a 2/3g Housner earthquake. This represents approximately a 33% increase in the design input (see figure 2). Additional capacity beyond 2/3g Housner is available due to in-situ material properties and the margins against failure inherent in design allowables.

2.1.3 Structural Steel Framing

The Seismic Backfit Project scope included an evaluation of structural steel framing within the reactor building. This framing consisted of the supports for the reactor service crane. Figure 3 illustrates the inputs to steel framing systems for the original design basis and the 2/3 Housner reevaluation.

1. The increased material strength, and thus the modulus of elasticity, will not cause a shift in frequency since the fundamental frequencies are influenced primarily by soil-structure interaction.

FIGURE 3. STEEL STRUCTURES RESPONSE SPECTRA



——— .5g HOUSNER @ 2½% DAMPING (FRAMED STEEL STRUCTURES)

- - - - .67g HOUSNER @ 7% DAMPING (BOLTED STEEL STRUCTURES)

The reactor service crane structural steel framing is supported from the operating deck. A response spectrum analysis was performed utilizing in-structure response spectra calculated from three-dimensional finite-element model of the concrete reactor building. The results of the analysis demonstrated that the stresses in this framing would not exceed allowable stresses for a 2/3g Housner Spectrum input to the reactor building. No credit was taken for in-situ material properties.

2.2 Reevaluation of Equipment: Reactor Coolant System (NSSS)

The reanalysis of the NSSS was performed by Westinghouse Electric Corporation in 1974-75.

The analytical model and analysis methodology included coupling between the structure and the NSSS, and nonlinearities such as gaps and constant force hangers. This analysis was generally recognized to represent the state-of-the-art and as such exceeded the requirements of current practice.

The results of the reanalysis indicated that additional laterial restraints were required, primarly to resist overturning. This was because the large NSSS equipment was base-supported, and the supports were configured to allow a maximum thermal growth potential. By strengthening existing supports and providing some additional restraints, stresses were brought within allowable limits.

In general, the modifications required for the NSSS were unique in that the large height to diameter ratios encountered in this equipment

is not typical for base supported equipment in the balance of the plant.

2.3 Piping Systems and Equipment

The original design of piping systems at San Onofre Unit 1 was based on the 1955 version of the ANSI (formerly USAS) B31.1 Code for Power Piping and a .5g Housner Spectrum input. The magnitude of the design acceleration necessitated special provisions (including hydraulic snubbers) to resist horizontal and vertical seismic loads.

The fundamental basis of the 1955 version of the B31.1 Code is to develop a piping system that has a balance of flexibility and control. It is this concept of controlled flexibility that is in use today in the design of power plant piping. An inherent property of piping systems designed with controlled flexibility is the ability to absorb large amounts of energy such as is created by seismic ground motion.

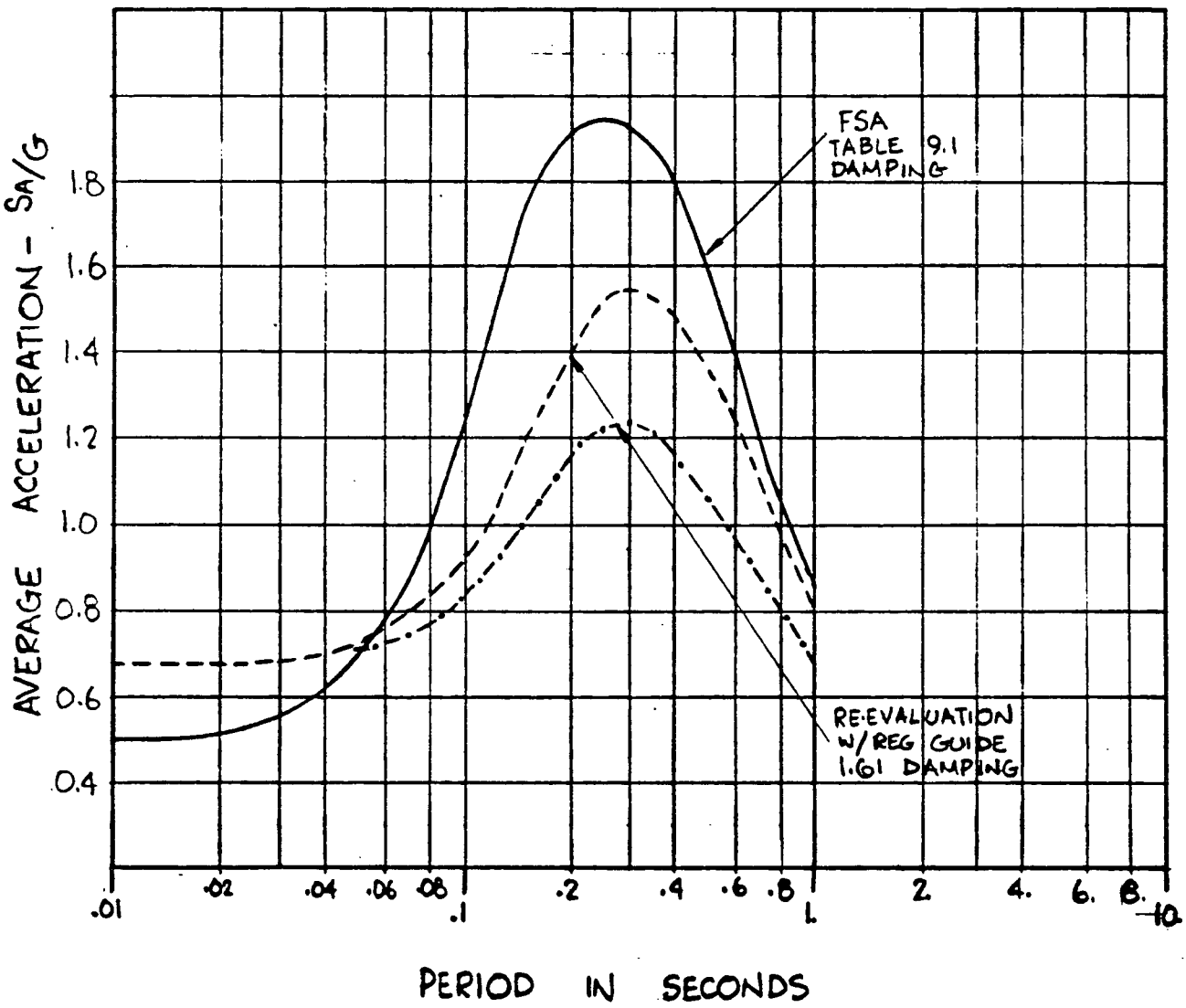
A formidable quantity of contemporary evidence is available, some of which has been collected in Reference 7, showing that piping systems designed with controlled flexibility have the capacity to withstand forces far in excess of the forces for which they were designed. Reference 7 includes data collected from more than twenty power plants and industrial facilities which were subject to severe seismic motion. A typical example is the ESSO refinery in Managua, Nicaragua which was designed to meet provisions of the Uniform Building Code for a .2g seismic acceleration. During the 1972 Managua, Nicaragua earthquake the peak acceleration measured at the refinery was .39g E-W and .34g N-S. Despite the fact the ground acceleration exceeded by nearly 100% the acceleration for which the systems were designed, virtually

no damage was sustained by the piping systems. The plant was shut down for inspection but was operating at full capacity within 24 hours. Even more impressive evidence can be found at the ENALUF Power Plant which was subject to an estimated .6g ground motion during the same earthquake. This plant sustained no damage to its piping, despite a probably non-existant seismic design.

Although it was not the intent of reference 7 to address equipment operability this report offers a great deal of evidence to the effect that mechanical and electrical equipment will survive without loss of function following a seismic event much greater than that for which it is designed. The examples of the ESSO refinery and ENALUF Power Plant cited above are cases in point. Still other examples are found in reference 7 of power plants that either remained on line or tripped off due to a loss of load. Typically tripped units were returned to service in a very short time, anywhere from a few minutes to a few hours following the seismic event. The ability of these power plants to continue to operate demonstrates that the equipment in the plants survived this seismic event without loss of integrity or function.

In addition to the evidence cited above and in Reference 7, there are other factors which imply the ability of the existing piping systems at San Onofre Unit 1 to withstand a .67g Housner Spectrum input. Regulatory Guide 1.61 recommends 2 to 3 percent damping for vital piping as opposed to the San Onofre Unit 1 FSA which specifies 1/2 percent damping for vital piping. The effect of this increase in recommended damping is illustrated in Figure 4. As can be seen in this figure, piping systems with fundamental frequencies of less than 20 Herz experience smaller accelerations with the .67 Housner

FIGURE 4. PIPING
RESPONSE SPECTRA



——— .5g HOUSNER @ ½% DAMPING (VITAL PIPING)
- - - - .67g HOUSNER @ 2% DAMPING (SMALL DIAMETER PIPING)
-·-·-·- .67g HOUSNER @ 3% DAMPING (LARGE DIAMETER PIPING)

Spectrum input than with the .5g Housner Spectrum input. This is due to the increase in recommended damping valves. Therefore, for piping systems with fundamental frequencies of 20 Herz and less, the .67g Housner Response Spectrum input is actually less conservative than the original .5g Housner criteria. A review of piping stress analyses that were performed as part of the original design and for subsequent modifications revealed that fundamental frequencies fall below 20 Herz.

Since the original design and construction of the plant, various piping systems and portions thereof have been reevaluated to higher seismic input than the original .5g Housner Spectrum criteria. Among the systems which have been partially reanalyzed are:

1. The Safety Injection System inside containment from the header connection at elevation 27'-0" down to the floor penetration at elevation 11'-6" has been analyzed for a 2/3g amplified Housner Response Spectrum.

2. The Boric Acid System is currently being analyzed as part of the program for replacement of stainless steel pipe. The analysis involves that portion of the system which runs from the Boric Acid Tank to the transfer pumps and from the Boric Acid Injection Pumps to the Batching Tank. The analysis of this piping was based on the 2/3g Response Spectrum input as developed for San Onofre Units II and III.

3. The Safety Injection Recirculation Piping outside containment has been reanalyzed from the sphere penetration to the heat exchanger then to the Refueling Water Pumps. This system was analyzed based on a 2/3 g amplified Housner Response Spectrum.

4. Various portions of the Primary Coolant Loop Main Piping have been reevaluated by Westinghouse Electric Corporation. This analysis included a time history analysis whose Spectrum enveloped a .67g Housner Response Scpectrum. The portions of the system analyzed in this manner include the hot leg (from the reactor to the steam generator), the cross over leg (from the steam generator to the reactor coolant pump), the cold leg (from the reactor to the reactor coolant pump).

5. The Main Steam and Feedwater System was analyzed from the Steam Generator to the Main Steam Stop Valve. This analysis was performed using a statically applied input of 1.0g horizontal and 0.67g vertical.

In summary, there is ample basis to conclude that piping and mechanical equipment at San Onofre will survive earthquake inputs corresponding to a 2/3g maximum instrumental ground acceleration. This conclusion is supported by actual experience with power plant and industrial facilities that were less stringently designed than San Onofre Unit 1 and by the conservative damping (1/2%) that was assumed for the piping stress analysis.

2.4 Conduit and Raceway Systems

Cable raceway systems, with unbraced supports, rod or strut, have been observed to survive actual earthquakes at several non-nuclear power installations with only light to moderate damage. A particular example of this occured at the Sylmar Converter Station during the 1971 Sylmar earthquake. Although these cable tray systems were rod hung and generally unbraced, they remained essentially intact during and after severe shaking. By contrast to the observed inherent capacity of unbraced tray systems the trend in seismic design of raceway systems in nuclear power plants has been toward more rigid and heavily braced support systems.

This trend was a natural result of rather conservative seismic criteria (especially in the area of damping) and lack of information concerning tray and conduit system dynamics. In order to bridge the gap between observed performance and current criteria the Cable Tray and Conduit Raceway Program was undertaken by Bechtel with support from several utilities and manufacturers.

The principal goal of the program was to develop a better understanding of raceway system dynamics. In order to achieve this goal an extensive test program including a wide variety of system configurations and hardware that are used at San Onofre Unit 1 were included in the test program.

The conclusions of the test program were significant and are based upon substantial data taken from over 2000 individual dynamic tests conducted during 11 months of testing. Of particular significance is the conclusion that raceway systems similar to those installed at San Onofre Unit 1 can be expected to survive severe earthquakes (up to 0.75g SSE) with no loss of function in the circuits they support. Therefore, based upon this recent testing, the cable and conduit raceway systems at San Onofre Unit 1 would still be expected to perform satisfactorily in the event that the DBE were increased from a 1/2g Housner Spectrum to a 2/3g Housner Spectrum.

3.0 GENERIC FACTORS AFFECTING CAPACITY

This section describes how ductility, damping and in-situ material properties might be expected to affect the capacity of structures and equipment.

3.1 Ductility

For structures designed using mild steel, significant reserve capacity can exist due to inelastic response. Those category A structures that were neither designed to or reanalyzed to a 2/3g Housner Spectrum at San Onofre Unit 1 were designed for a 1/2g Housner Spectrum utilizing mild steel (A36 and Grade 40 rebar). Newmark and Hall in reference 6 have indicated that for small excursions into the inelastic range an equivalent reduction in the elastic spectrum of $1/\sqrt{2u-1}$ could be expected for structures having a frequency range of 2 to 8 Hz (the frequency range for structures at San Onofre Unit 1). Since a 2/3g Housner Spectrum is equivalent to a 33% increase from the 1/2g Housner Spectrum, it can be directly calculated from the preceeding relationship that a ductility factor of only 1.4 is sufficient to ensure that structures designed elastically for a 1/2g Housner Spectrum will have adequate capacity to withstand a 2/3g Housner Spectrum input. Since allowable ductility values for typical mild steel are in the range of 2 to 6 and the allowable ductility for individual members are typically 8 or more, it can be concluded that adequate ductility probably exists in affected steel structures at San Onofre Unit 1 to permit the increase of the DBE from 1/2g Housner Spectrum to a 2/3g Housner Spectrum. In the case of concrete structures, the

allowable ductility is 1.5 or more. This is also adequate to resist a 33% higher ground motion input.

3.2 Damping

For structures in the linear range of response to dynamic loading, energy absorption is due primarily to damping. Damping levels determined from observation and measurement show a fairly wide spread. In recent years, testing has expanded the data base of damping values and additional opportunities for observation of response to earthquakes have occurred. Partly due to this expanding data base, damping values suggested for design purposes have increased. For example, in the mid-1960's the San Onofre Nuclear Generating Station's FSA indicated 2-1/2% damping would be assumed for welded steel structures. Since 1973 the value of damping suggested in Regulatory Guide 1.61 for a Safe-Shutdown Earthquake was 4% of critical damping for welded steel structures. In May 1978 N. M. Newmark and W. J. Hall recommended 7% as the damping available in welded steel structures (Reference 6). This increase in recommended values of damping is also typical for other materials (see Table 1). This increase in recommended damping values results in a corresponding decrease in structural response to dynamic loading. In many cases when using current recommended damping values, response due to the application of a .67g Housner Spectrum input may be less than the level of response resulting from the application of a .5g Housner Spectrum input that assumed the damping values cited in the San Onofre Unit 1 FSA (Table 9-1).

3.3 In-Situ Material Properties

The probable effects of in-situ material properties on structural

TABLE 1

RECOMMENDED DAMPING VALUES

	San Onofre Unit 1-FSA (Mid-1960's)	Reg. Guide 1.61 (Oct. 1973)	Newmark (Ref. 6) (May 1978)
Steel (Welded)	2.5	4	7
Steel (bolted)	2.5	7	15
Vital Piping	0.5	2 or 3(1)	3
Prestressed Concrete	-	5	7 or 10(2)
Reinforced Concrete	5 or 7(3)	7	10

Damping is expressed as a per cent of critical viscous damping.

(1) 2% for pipe with diameter of 12 inches or less, 3% for pipes with larger diameters.

(2) 7% without complete loss of prestress and 10% if no prestress is remaining.

(3) 5% for rigid frame structures and 7% for shear wall structures.

capacity are described in the following paragraphs.

3.3.1 Concrete

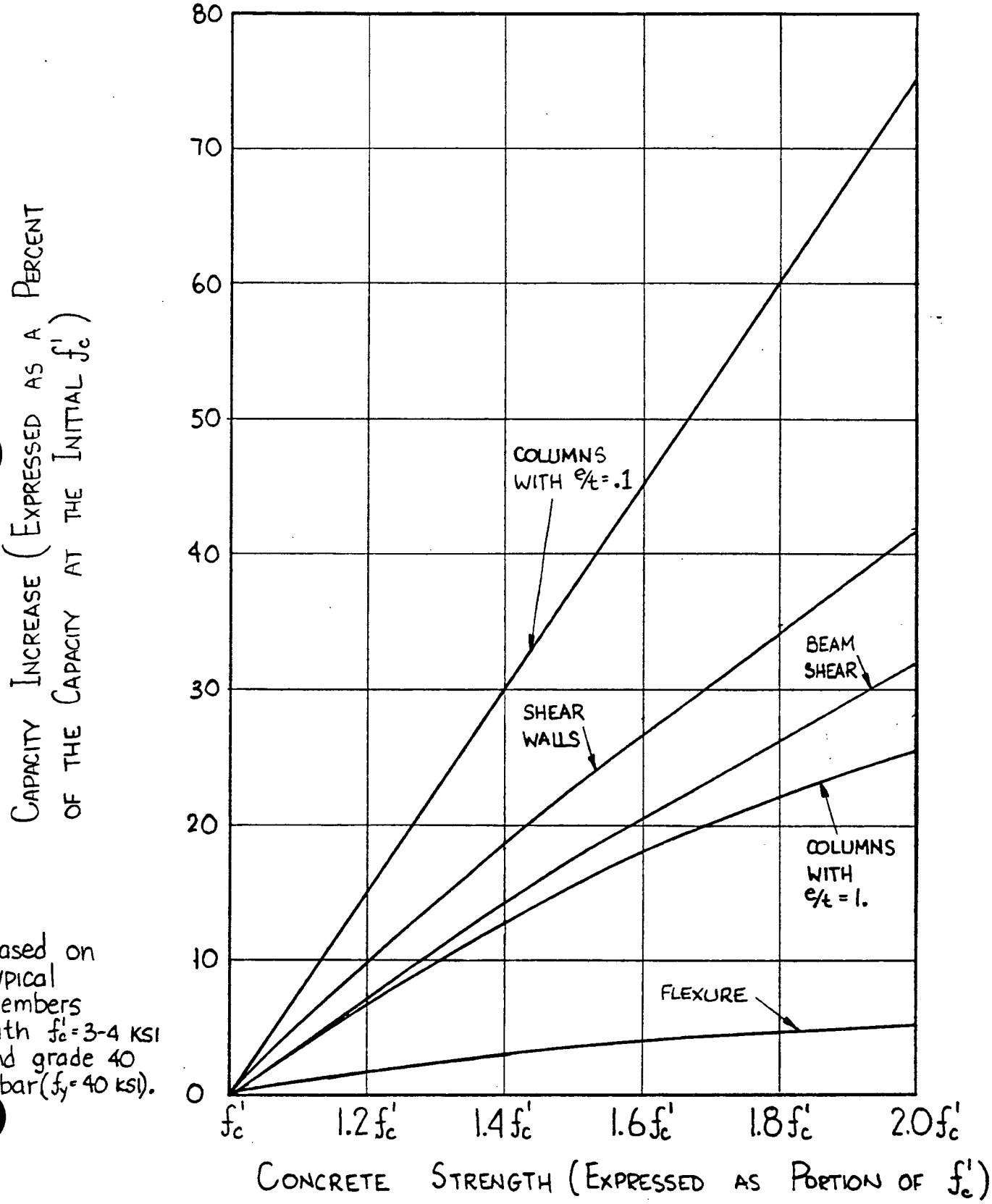
The nominal design strength of concrete is based on the 95% confidence value of concrete compressive strength for a particular batch of concrete after 28 days of curing. Typically the actual strength of the batch is higher than that used in design. In addition, and of greater importance, concrete strength increases as hydration of the mix water and curing water takes place. This hydration results in a marked increase in strength, particularly in the first ten years (Reference 1 and 2). The 28 day strength of concrete can increase 100% and more after ten years. The amount of increase in strength with age depends on a number of variables, among which are; type of cement, curing history, climate, size of concrete member, mix quality control and construction techniques. Due to the nonhomogeneous nature of reinforced concrete, the effects of higher concrete strenght on capacities of structural members varies with the type of forces resisted. For example, compression capacity is most directly affected since it is primarily a function of the concrete strength. A 100% increase in concrete strength (typical after 10 years) will increase the ultimate strength capacity of columns with an eccentricity ratio of 1/10 by 75%. For an eccentricity ratio of one, the capacity increase is 26%. By contrast, flexural capacity is less directly influenced by an increase in concrete strength. This is due to the fact that the flexural capacity is more dependent on the

amount of reinforcement in the member. The ultimate strength bending capacity of typical members would probably be increased by less than 10% when concrete strength is doubled. Shear capacity is a function of the square root of concrete strength and the amount of shear reinforcement present. Consequently, the effects of increased concrete strength on shear capacity lies between the effects on compression capacity and flexural capacity. Increasing concrete strength 100% produces a 32% increase in concrete shear capacity in beams and a 41% increase in concrete shear capacity for shear walls. Since both concrete and shear reinforcement contribute to shear capacity, the increase in shear capacity due to higher concrete strength on the capacity of various types of concrete members is shown in the accompanying figure 5. It is apparent that, with the exception of bending, the increased capacities are significant for the concrete strength expected after ten or more years.

3.3.2 Steel

Minimum specification values of yield stress for structural steel and reinforcing steel have been set by the American Society for Testing and Materials (ASTM). These 'minimum yield stresses' are used in design as the basis for allowable stress values, with safety factors applied according to governing codes. Typically, the actual yield stress of a steel number is significantly higher than the value used in design. For example, the actual mean yield stress for A36 steel is typically in the range of 44-48 ksi. However, the specified minimum yield stress for A36 steel is 36 ksi.

FIGURE 5. EFFECT OF INCREASED CONCRETE STRENGTH ON MEMBER CAPACITY



Based on typical members with f'_c = 3-4 KSI and grade 40 rebar (f_y = 40 KSI).

Mild steels have large ductility ratios and ultimate stresses significantly greater than the specified minimum yield stress. (For example A36 steel has an ultimate stress of 58-80 ksi.). These two factors allow for excellent response to dynamic loads. In addition, the high ductility and increase in strength with increased strain beyond yield can allow for redistribution of loads to stronger members from members which have reached yield.

4.0 CONCLUSIONS

An objective assessment of existing data demonstrates that, if the design basis earthquake is increased from 1/2g Housner to 2/3g Housner, the possible increase in stresses will be mitigated by a number of factors not considered in the original design. Furthermore, even in the absence of such mitigating factors, additional capacity in terms if ductility, increased material strength, and other factors should enable engineered structural systems to respond without significant damage. The basis for these conclusions is as follows:

(1) Structural elements already analyzed have been qualified to 2/3g Housner Spectrum without modification. These include the steel containment sphere, the primary shield wall, the secondary shield wall, the structural steel framing supporting the reactor service crane, and foundations.

(2) Although modifications to equipment were required, these were primarily to resist overturning in large equipment that was base supported and had significant height to width ratios. This equipment was originally designed to provide significant thermal growth capability. The conditions encountered in this equipment are unique to the reactor coolant system.

(3) There is ample basis to conclude that piping and mechanical equipment at San Onofre Unit 1 will survive earthquake inputs corresponding to a 2/3g maximum ground motion. This conclusion is supported by actual experience of power plants and industrial facilities that were less stringently designed that San Onofre

Unit 1 and subjected to ground motion input several times the system design basis. This experience is described in detail in Reference 7.

(4) Damping values in the original design (FSA Table 9.1) for steel structures and piping were less than those recommended in Regulatory Guide 1.61. Additionally Newmark and Hall in reference 6 have recommended still larger values for damping in the seismic reevaluation of existing structures. Additionally, some complex systems such as conduit and raceway systems have been tested recently and show to have damping well in excess of 10%. Therefore, damping would clearly tend to reduce response expected by increasing from 1/2g to 2/3g.

(5) The design process has inherent margins built into it. Capacity reduction factors and discrete size increments (i.e. rebar diameter, rolled steel shape, or pipe wall thickness). Typically the required size will fall between available sizes and the next largest will be selected or spacing of supports will be dictated by other considerations, such as interferences. Also standardization may result in use of conservatively sized structural elements.

(6) A ductility factor of only 1.4 is sufficient to account for the increase from 1/2g to 2/3g for structural elements designed elastically to 1/2g. Since mild steel was used in the design of affected structures a ductility ratio of 2 to 3 is probable.

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APPENDIX B

DEVELOPMENT OF INSTRUMENTAL RESPONSE SPECTRA WITH EQUAL PROBABILITY OF EXCEEDANCE

SAN ONOFRE NUCLEAR GENERATING STATION

UNIT 1

8005150294
P

Development of Instrumental Response Spectra with Equal Probability of Exceedance for SONGS Unit 1

PREPARED FOR

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April 18, 1980

Woodward-Clyde Consultants

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1.0 SUMMARY AND FINDINGS

1.1 Summary

The objective of this study was to provide basic input data to be used in evaluating the probabilities of exceeding the design spectrum for San Onofre Nuclear Generating Station (SONGS) Unit 1. Specifically, this study provides instrumental response spectra with equal probabilities of exceedance for use in judging the probabilities of exceedance of the SONGS Unit 1 design spectrum.

The approach to evaluating probabilities of exceeding various levels of a ground motion parameter is shown schematically in Figure 1-1 and described in Section 2. The basic input requires the following information:

- Identification of seismicity sources and characterization of activity of each source;
- Specification of magnitude-rupture length relationships;
- Selection of attenuation relationships for peak ground acceleration and spectral ordinates at selected structural periods.

The assessment of these input items is discussed in Section 3. The results of the probability of exceedance calculations are described in Section 4.

The analyses results presented herein are a product of a carefully carried out evaluation of existing data. Best estimate and in some cases conservative input parameters with appro-

proriate probability distributions were considered in the initial analysis as described to the NRC staff in Washington on 20 March 1980 (results shown in Sections 2 through 4). Subsequent to that meeting and in response to NRC concerns expressed during the meeting, sensitivity analyses were carried out on various input parameters. The results of these studies are discussed in Section 5. All cited references are listed in Section 6.

1.2 Findings

The results of the probability analyses as described in Sections 2 through 4 are summarized in Figure 1-2 in the form of equal probability spectra. Specifically, instrumental response spectra with equal probabilities of exceedance corresponding to peak accelerations of 4/10-, 1/2- and 2/3-g are presented. The corresponding probabilities of exceedance are 2×10^{-3}, 6×10^{-4} and 1×10^{-4}. These results are consistent with those presented in the 20 March 1980 NRC meeting. After consideration of the results of sensitivity studies on various input parameters (Section 5), it is concluded that the results presented in Figure 1-2 reflect a reasonable but conservative estimate of equal probability of exceedance spectra for the SONGS site consistent with the regional historic seismicity data as well as the geologic constraints on prehistoric seismicity.

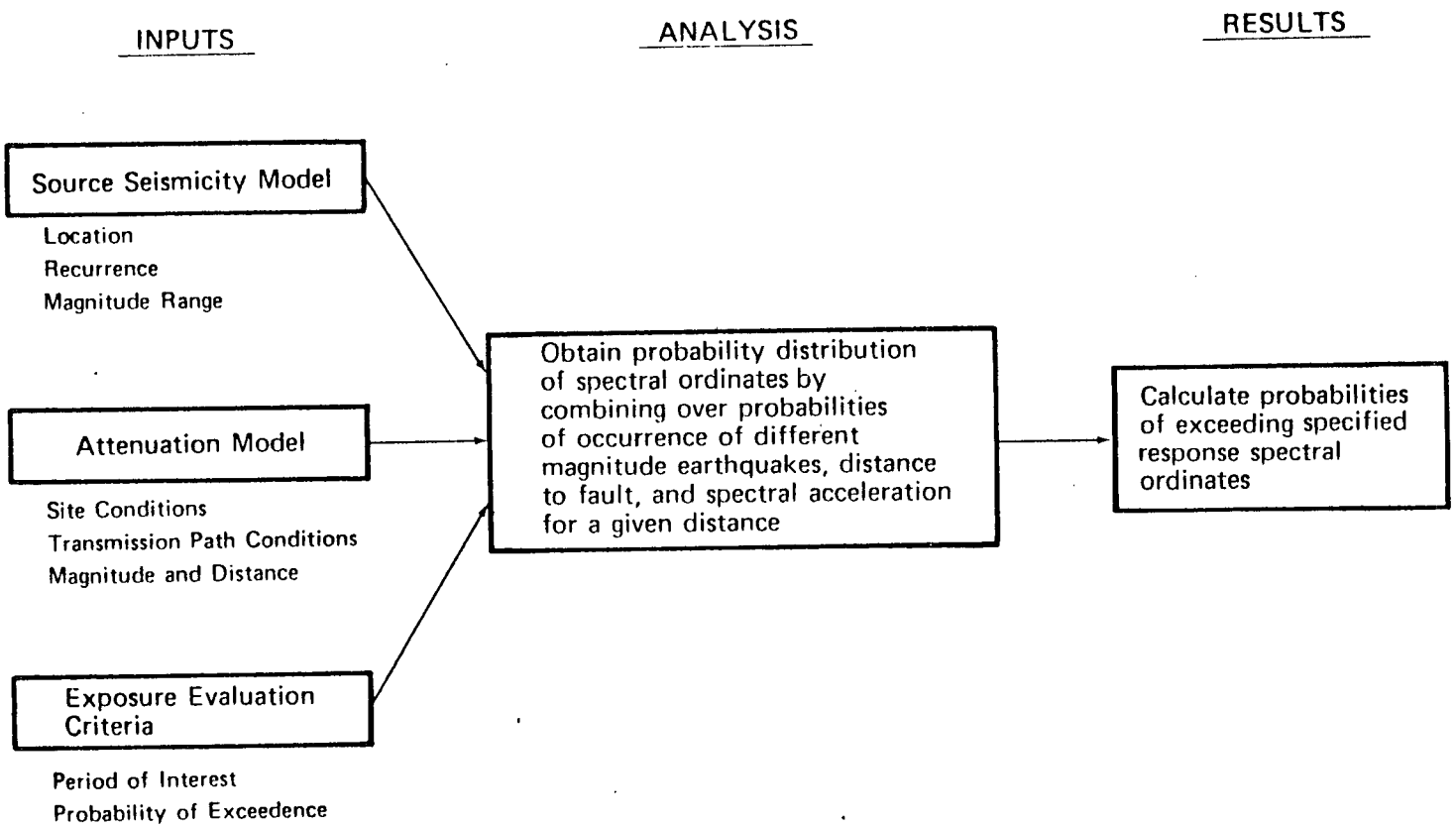

Figure 1-1 – Schematic Representation of Approach

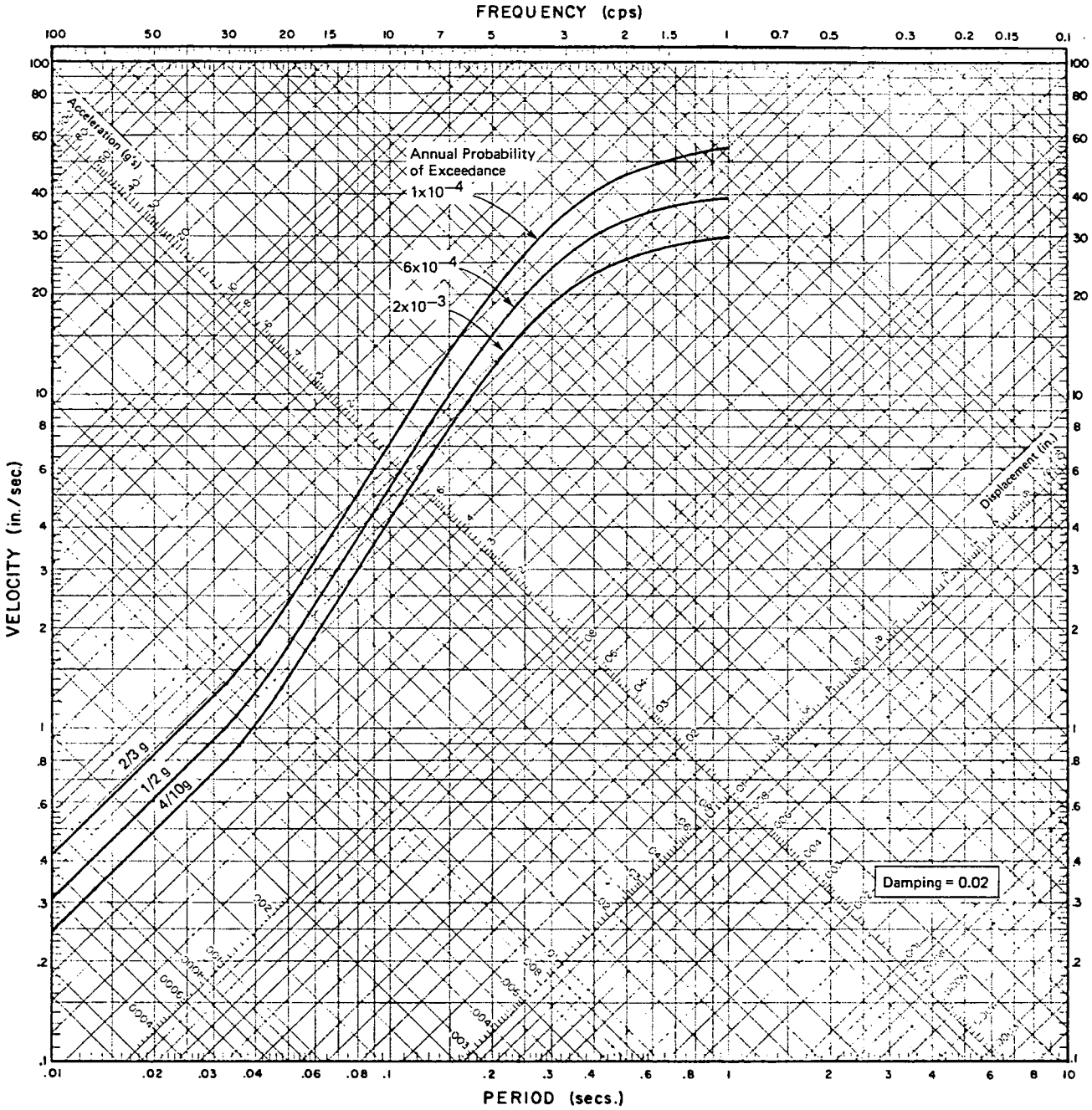

Figure 1-2 — Instrumental Response Spectra Associated with Various Annual Probability of Exceedance Levels

2.0 METHODOLOGY

This section provides a description of the methodology that was used to calculate probabilities of exceeding different levels of ground motion transmitted to the site from future earthquakes. The basic components of this methodology are described below.

2.1 Identification of Seismicity Sources

The sources which can generate earthquakes within the study region were identified and the geometry of these sources was delineated.

2.2 Characterization of Activity of Seismicity Sources

A Poisson model was assumed for the occurrence of earthquakes. The parameter of this model is the average number of earthquakes of different magnitude per unit time. This parameter is assessed from the Gutenberg-Richter relationship expressed as

$$\log N(M) = a - bM \qquad (2\text{-}1)$$

in which $N(M)$ = number of earthquakes greater than or equal to magnitude M in unit time

a and b = empirical constants.

The constants a and b were estimated from the analysis of historic seismicity and geologic data. For engineering purposes, bounds on earthquake magnitude, M, may be used. The lower bound is the minimum magnitude of engineering significance (for example, a magnitude 4). The upper bound represents the magnitude of a maximum credible earthquake that each fault is capable of generating. Equation 2-1 was modified to account for the lower and upper bounds on earthquake magnitude.

2.3 Characterization of Attenuation of Ground Motion

The attenuation of ground motion was assumed to follow a relationship of the type shown below:

$$S = b_1 \, e^{b_2 M} \, (R + C)^{-b_3} \, \varepsilon \qquad (2\text{-}2)$$

in which S = ground motion parameter of interest (peak acceleration and spectral velocity for several periods up to 1 sec)

M = earthquake magnitude

R = source-to-site distance

b_1, b_2, b_3 = coefficients

C = magnitude-dependent constant = $K_1 \, e^{K_2 M}$ (2-3)

ε = random error term

2.4 Specification of Earthquake Magnitude Versus Rupture Length Relationship

The approach used in the present analysis incorporates a line-source model which includes the length of rupture during the earthquake in a manner similar to that suggested by Der Kiureghian and Ang., 1977. The release of earthquake energy was assumed to be along a line rupture of the fault. Closest distance from the site to the fault rupture was used in calculating the attenuation of earthquake ground motion. The following form of relationship was used for the rupture length, L_R for an earthquake of magnitude M:

$$L_R = \exp\,(C_1 + C_2 M), \text{ for } M \leq M_1 \qquad (2\text{-}4)$$

$$= \exp\,(C_3 + C_4 M), \text{ for } M > M_1 \qquad (2\text{-}5)$$

The constants C_1, C_2, C_3 and C_4 were estimated from historic data.

2.5 Calculation of Probabilities of Exceedance

The mean number of events in which a given level of ground motion parameter is exceeded at the site during the period of interest was calculated. Contributions of different magnitude earthquakes occurring on various sources and at different distances from the site were included in this calculation. The mean number of events was then used to calculate the probability that at least one such event would occur at the site during the specified period of interest.

The computer program SEEP developed by Woodward-Clyde Consultants (Kulkarni et al., 1979) was used to perform the probability calculations. The program displays the overall probability of exceeding a specified level of a ground motion parameter. In addition, contributions to the overall probability from the different sources, earthquake magnitudes, and distances are also displayed.

3.0 ASSESSMENT OF INPUT DATA

3.1 Geometry of Sources

Table 3-1 lists fault lengths, closest distance to each fault from the site of SONGS Unit 1, and the maximum magnitude assessed for each fault in the study region. The San Andreas, San Jacinto, Whittier-Elsinore, Palos Verdes faults, and the hypothesized OZD were selected for this analysis because the maximum earthquakes on these faults are expected to produce significant ground motions at the SONGS site.

Fault lengths were obtained principally from Jennings (1975), with more recent maps consulted as applicable. The closest point from the site to each fault was obtained by measuring a line from the site to the fault and perpendicular to the trend of the fault. In order to measure the endpoints of the faults, the faults were approximated by straight lines, with no bends. This approximation is generally close to the actual fault geometry. The approximate lines were structured such that the distance to the part of the fault closest to the site remained unchanged.

3.2 Activity of Sources

The magnitude-frequency relationship for each fault was determined from its moment rate using the method of Anderson (1979). In this method, all slip is assumed to occur seismically. Then the total moment rate (i.e. the product of slip rate, fault length, fault width, and shear modulus) is partitioned among events of different magnitude assuming the Gutenberg-Richter magnitude-frequency relationship, which must be truncated at some maximum magnitude.

The magnitude scale used with the method of Anderson (1979) is the moment magnitude scale (Hanks and Kanamori, 1979). These magnitude values do not differ significantly from M_S except for

magnitudes greater than 7.5. A moment magnitude of 8.0 was used to represent the maximum magnitude on the San Andreas fault. This value is compatible with the moment magnitude of 7.9 for the 1857 Fort Tejon earthquake obtained by Hanks and Kanamori (1979).

Using the slip rate, maximum magnitude, fault length values, and an assumed fault width of 15 km, shear modulus of $3x10^{11}$ dyne/cm^2, and 'b' value of 0.85, the 'a' value of each fault was calculated. Table 3-2 shows the inputs used in the probability analysis.

The central segment of the San Andreas fault (north of Wrightwood) is judged to be more active than the southern segment. However, for the analysis, an average 'a' value was assumed for the entire fault. This is conservative, since the more active central segment is farther away from the site.

The selected 'a' value and the 'b' value for the combined faults and the corresponding number of large earthquakes calculated from the Gutenberg - Richter relationship are compared with other estimates for Southern California in Table 3-3. It can be seen that the expected number of large earthquakes calculated using the selected 'a' and 'b' values is only exceeded significantly by the number obtained from Anderson (1979) for an equivalent set of faults. Therefore, the selected recurrence parameters are conservative compared to those obtained from seismicity data. It should be noted that the estimates from siesmicity data generally pertain to much larger areas than that of the present study.

3.3 Attenuation Relationships

The form of the attenuation relationship given in Equation 2-2 was used for the present study. The parameters in this relationship were estimated for tectonic environment, transmission path characteristics, and local site conditions appropriate to SONGS. Expressions were derived for attenuation

of peak ground acceleration and spectral velocities for several selected periods. Figure 3-1 shows the attenuation relationships for peak ground acceleration. Figure 3-2 illustrates the attenuation relationships for spectral velocity at a period of 0.4 seconds. An upper bound on each ground motion parameter was specified through a truncated distribution as described in Kulkarni et al., 1979. The upper bound values of the ground motion parameters used in this truncation accommodate an uncertainty band of greater than 3σ about the mean.

3.4 Magnitude-Rupture Length Relationship

The calculation of the closest distance to site requires the extent of fault rupture length for a given magnitude earthquake. For magnitudes greater than approximately 6-1/4, the dimension of the rupture surface was established based on the relationship for strike-slip faults given in Slemmons (1977). For magnitudes less than 6-1/4 the relationship given in Patwardhan and others (1975), which was derived on the basis of correlation between earthquake magnitude and length of aftershock zone, was selected.

TABLE 3-1

FAULT PARAMETERS USED IN SEISMIC EXPOSURE ANALYSIS

Fault or Zone of Deformation	Length (km)	Distance from SONGS Site to Closest Point on Fault (km)	Estimated Maximum Magnitude
San Andreas	540	92	8-1/4
San Jacinto	260	69	7-1/2
Whittier-Elsinore	230	37	7
Hypothesized OZD	200	8	6-1/2
Palos Verdes	100	18	6-1/2

TABLE 3-2

DATA ON SEISMIC ACTIVITY OF SOURCES USED IN SEISMIC EXPOSURE ANALYSIS

Fault or Zone of Deformation	Maximum Earthquake	Average Number of Earthquakes Greater than $M_s = 4$ in One Year on the Entire Fault	Slope 'b' for the Gutenberg-Richter Relationship $\log_{10}N(M) = a - bM$
Central and Southern San Andreas	8-1/4	15.04	0.85
San Jacinto	7-1/2	3.39	0.85
Whittier-Elsinore	7	2.04	0.85
Hypothesized OZD	6-1/2	0.82	0.85
Palos Verdes	6-1/2	0.41	0.85

TABLE 3-3

COMPARISON OF DIFFERENT ESTIMATES OF a AND b VALUES

FOR SOUTHERN CALIFORNIA

Source	Region	Time Interval	a	b	Normalized w.r.t. this study N(M>6)	N(M>7)
1. This study	S. California (specific faults)	(Pleistocene slip rate)	4.74	0.85	(0.44/yr) 1	(0.062/yr) 1
2. Anderson (1979)	S. California	(Pleistocene)	4.99	0.86	1.56	1.52
3. Richter (1958)	S. California (300,000 km^2)	1934 - 1943	4.77	0.85	1.09	1.07
4. Ryall et al. (1966)	Southern California (60,000 km^2)	before 1932	2.53	0.55	0.39	0.78
5. Ryall et al. (1966)	Southern California (60,000 km^2)	1932 - 1961	4.30	0.79	0.83	0.96
6. Hileman et at. (1973)	S. California (238,600 km^2)	1932 - 1971	5.36	0.98	0.69	0.52
7. Hileman et al. (1973)	Los Angeles Area Imperial Valley Parkfield	1932 - 1971	4.33 4.27 3.64	0.93 0.85 0.80	0.63	0.62

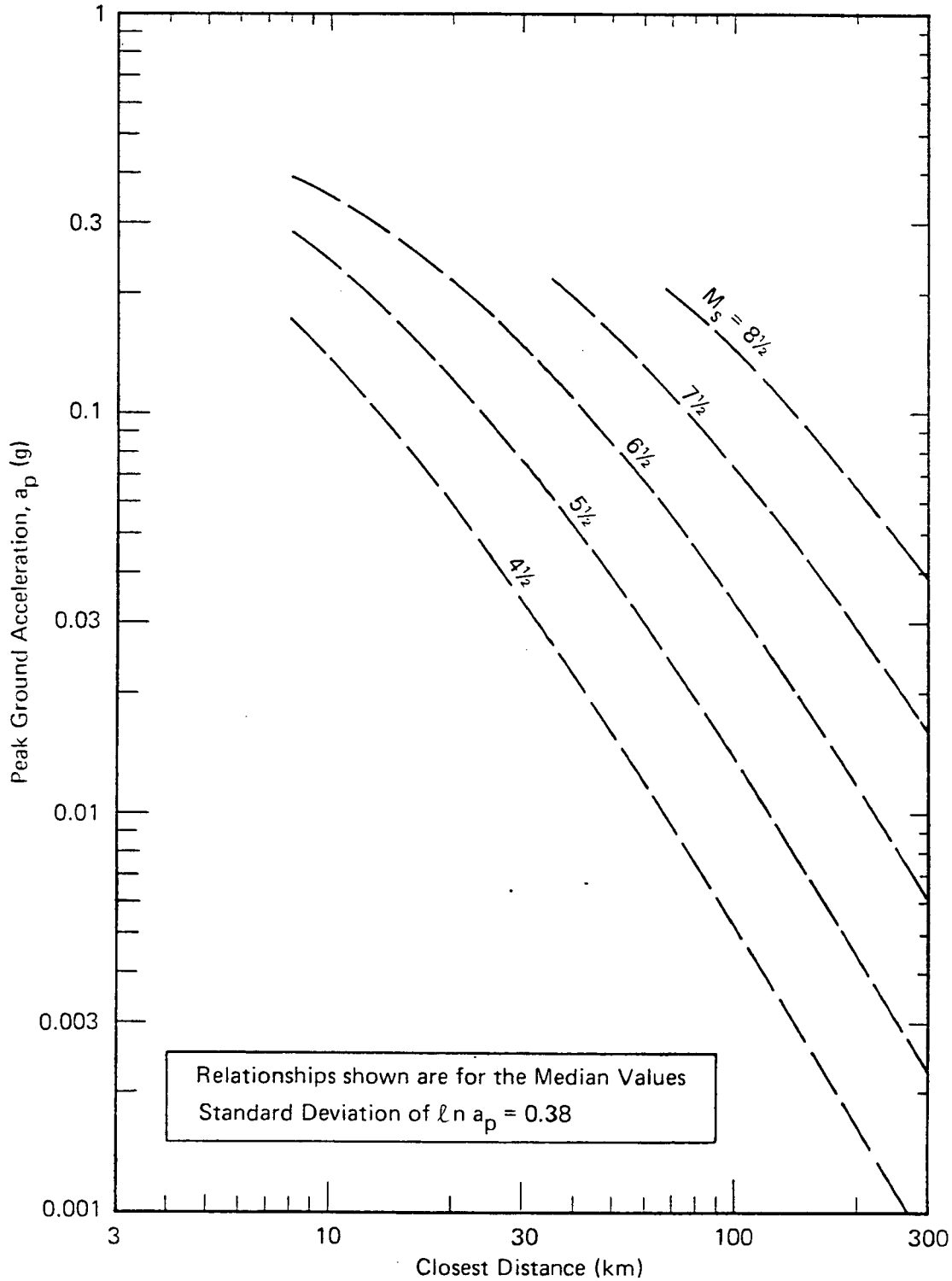

Figure 3-1 — Attenuation Relationships for Peak Ground Acceleration

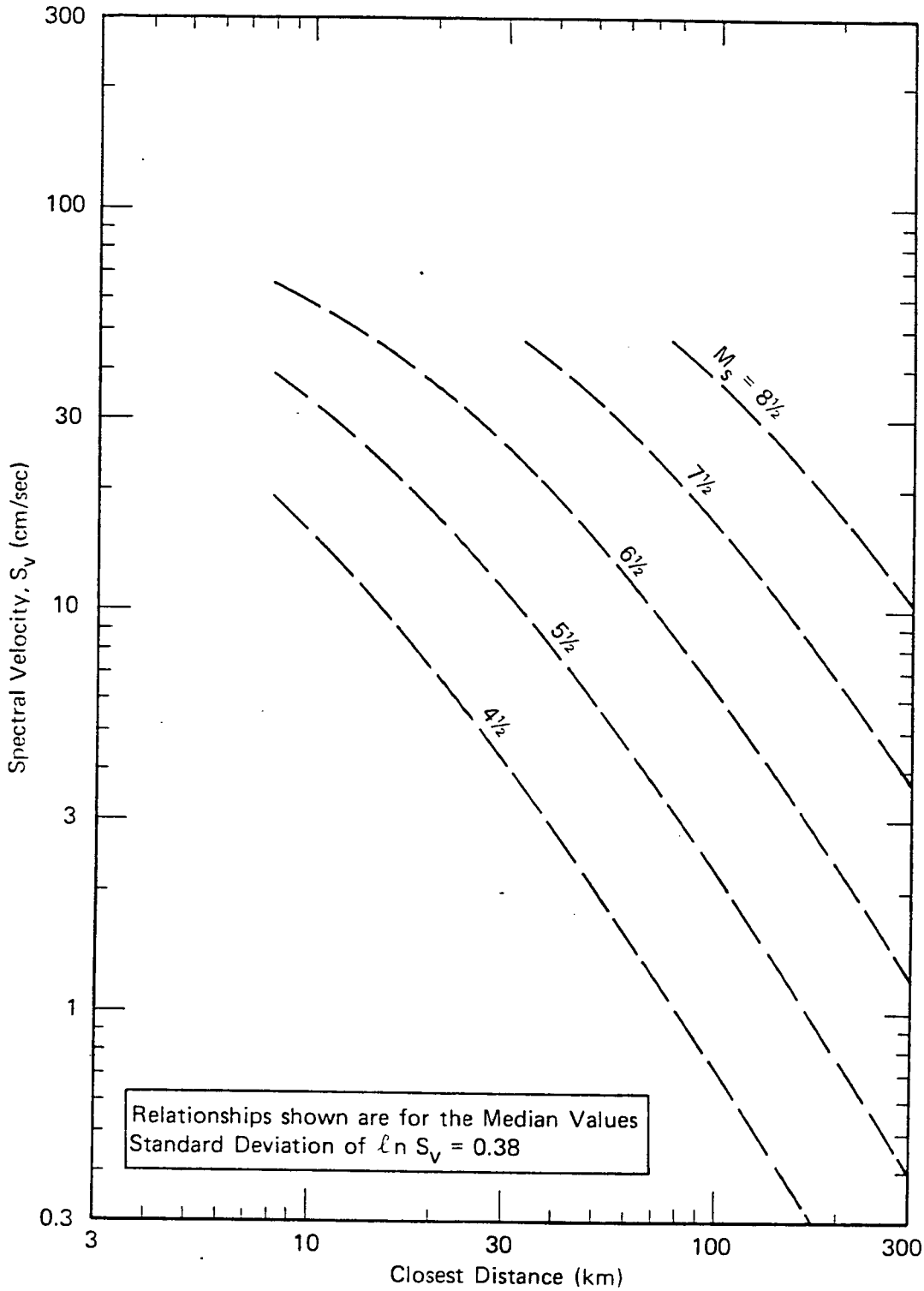

Figure 3-2 — Attenuation Relationships for Response Spectral Velocity
(Period = 0.4 seconds, Damping = 0.02)

4.0 RESULTS

This section describes briefly the main results of the probabilistic evaluation of seismic exposure.

4.1 Probability of Exceedance Relationships for Ground Motion Parameters

The mean number of events per year, λ, in which a given level of a ground motion parameter is exceeded at the site was obtained directly from the output of the computer program. The annual probability of exceedance, p_A, was then calculated from

$$p_A = 1 - e^{-\lambda}. \qquad (4\text{-}1)$$

The annual probabilities of exceeding different levels of peak ground acceleration are shown in Figure 4-1. Figures 4-2 through 4-5 show the annual probabilities of exceeding different levels of spectral velocity at selected periods.

4.2 Development of Equal Probability Instrumental Response Spectra

Instrumental response spectra associated with desired annual probability of exceedance levels were obtained from relationships shown in Figures 4-1 through 4-5 for various periods. The spectral values corresponding to annual probability of exceedance levels of 2×10^{-3}, 6×10^{-4} and 1×10^{-4} are presented below.

	S_v (cm/sec) corresponding to		
Period	$P_A = 2 \times 10^{-3}$	$P_A = 6 \times 10^{-4}$	$P_A = 1 \times 10^{-4}$
0	ZPA = 0.4g	ZPA = 0.5g	ZPA = 2/3g
0.1	10.8	13.5	17.6
0.2	30	38	50
0.4	59	76	103
1.0	75	100	140

The instrumental response spectra corresponding to these three annual probability of exceedance levels are shown in Figure 1-3.

4.3 Contribution of Source, Magnitude and Distance to Probability of Exceedance

The annual probability of exceedance values presented above include all of the contributions from the different magnitude earthquakes which could occur along the lengths of the sources examined in this study. Each source, distance, and earthquake magnitude combination contributed a different amount to the total exposure. As can be observed from the table below, the total probability of exceedance was dominated by the contribution from the Hypothesized OZD.

Period	Relative Contribution from the Hypothesized OZD to the Total Probability of Exceedance
0-0.04 sec.	94 - 99 percent
0.1	93 - 99
0.2	92 - 98
0.4	84 - 96
1.0	50 - 60

Earthquakes with magnitudes 5-1/2 to 6-1/2 rupturing to distances within 8 to 12 kilometers of the site provided the vast majority of these contributions from the Hypothesized OZD. For each other source, most of its contributions similarly came from the larger magnitude earthquakes with rupture at the closest distance from the site.

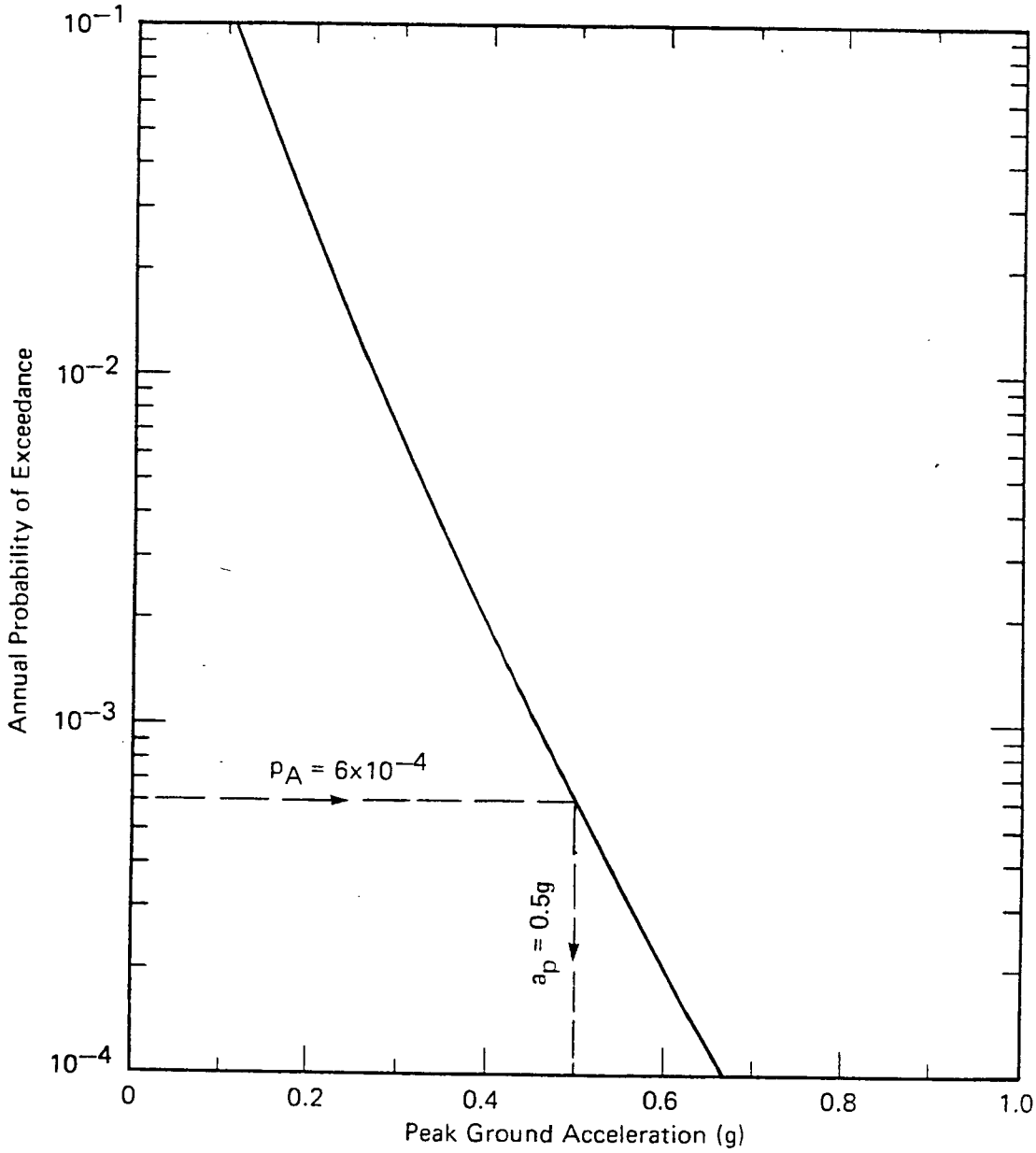

Figure 4-1 — Annual Probabilities of Exceeding Different Levels of Peak Ground Acceleration

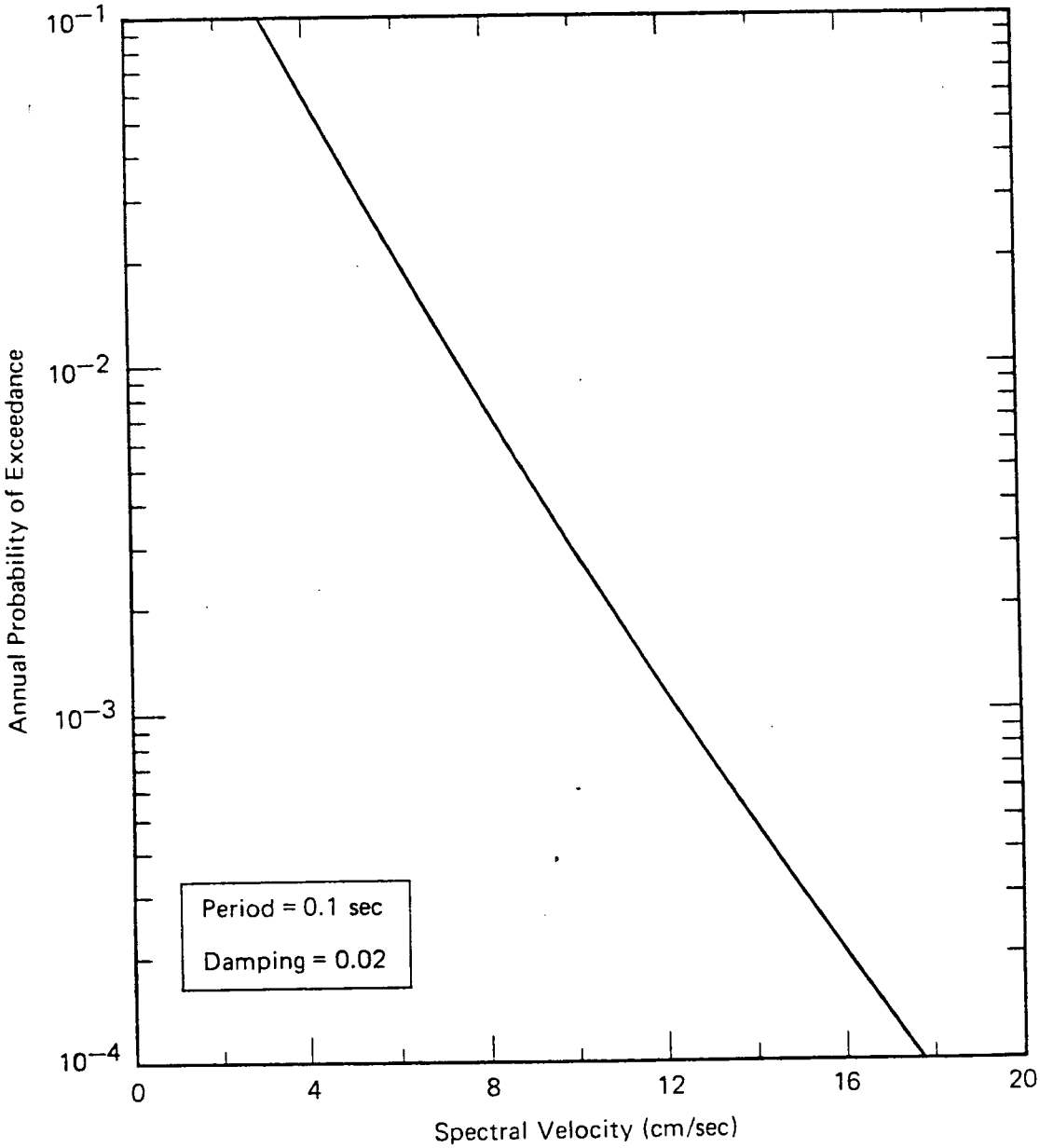

Figure 4-2 — Annual Probabilities of Exceeding Different Levels of Spectral Velocity (Period = 0.1 sec, Damping = 0.02)

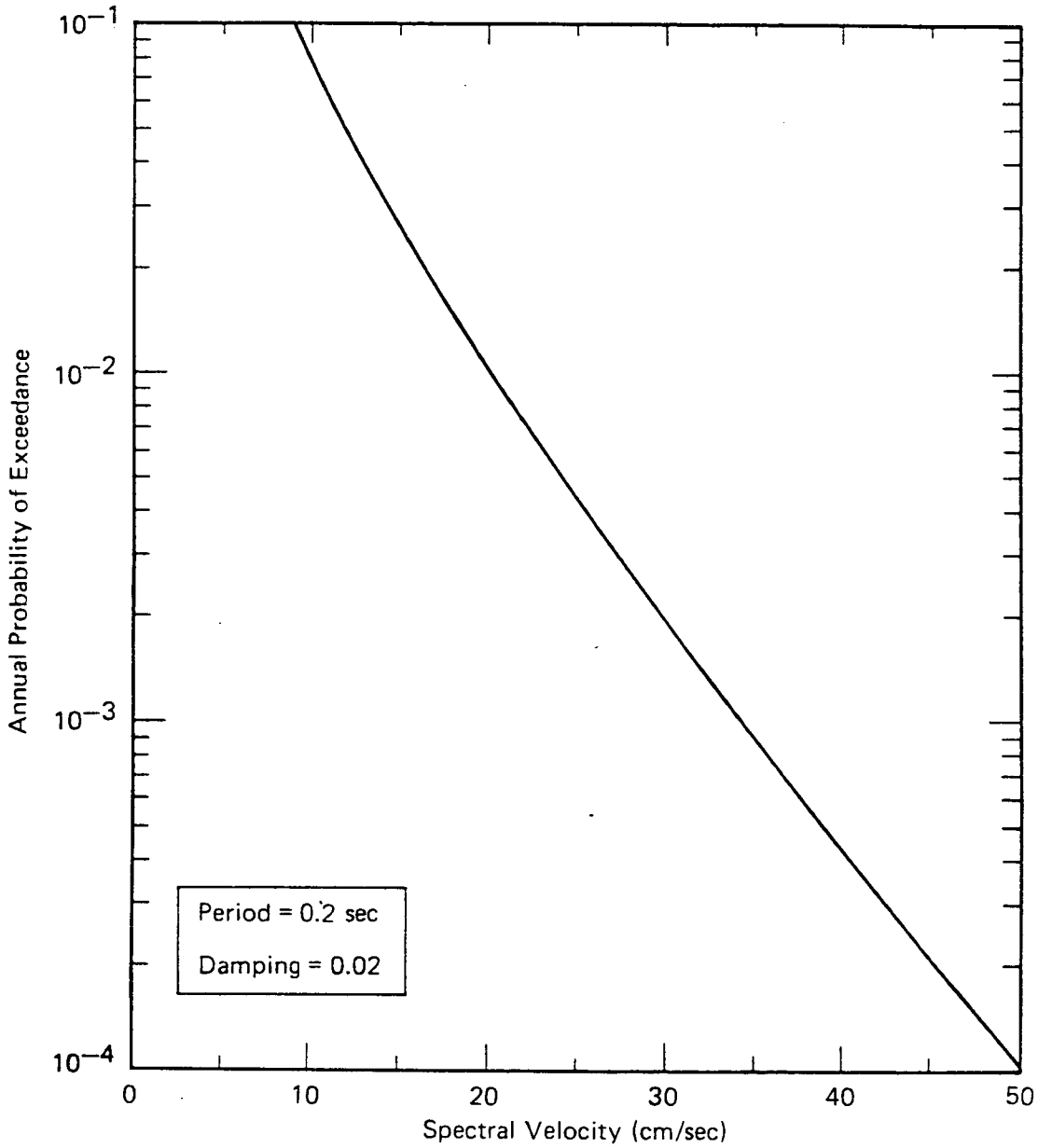

Figure 4-3 — Annual Probabilities of Exceeding Different Levels of Spectral Velocity (Period = 0.2 sec, Damping = 0.02)

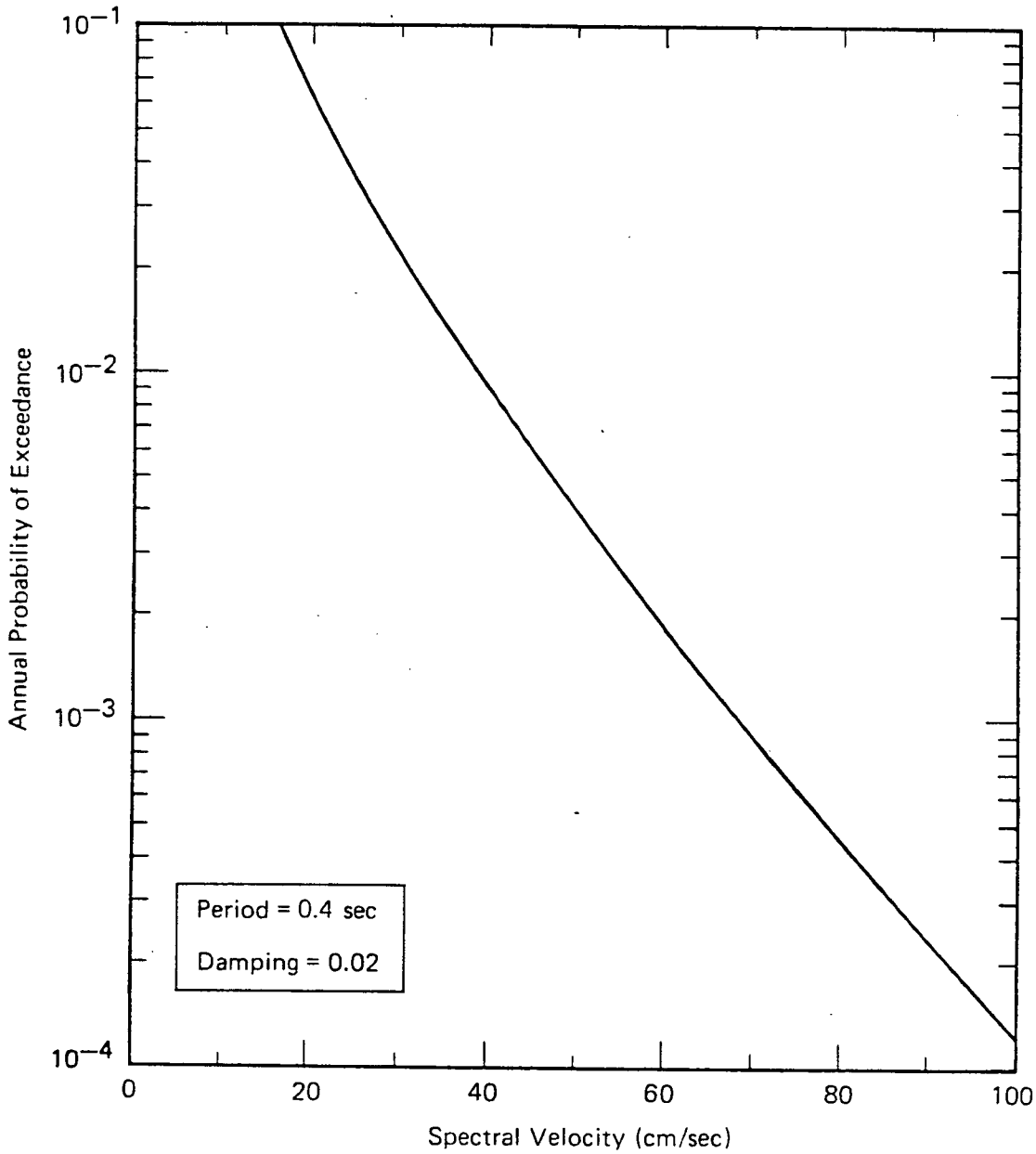

Figure 4-4 – Annual Probabilities of Exceeding Different Levels of Spectral Velocity (Period = 0.4 sec, Damping = 0.02)

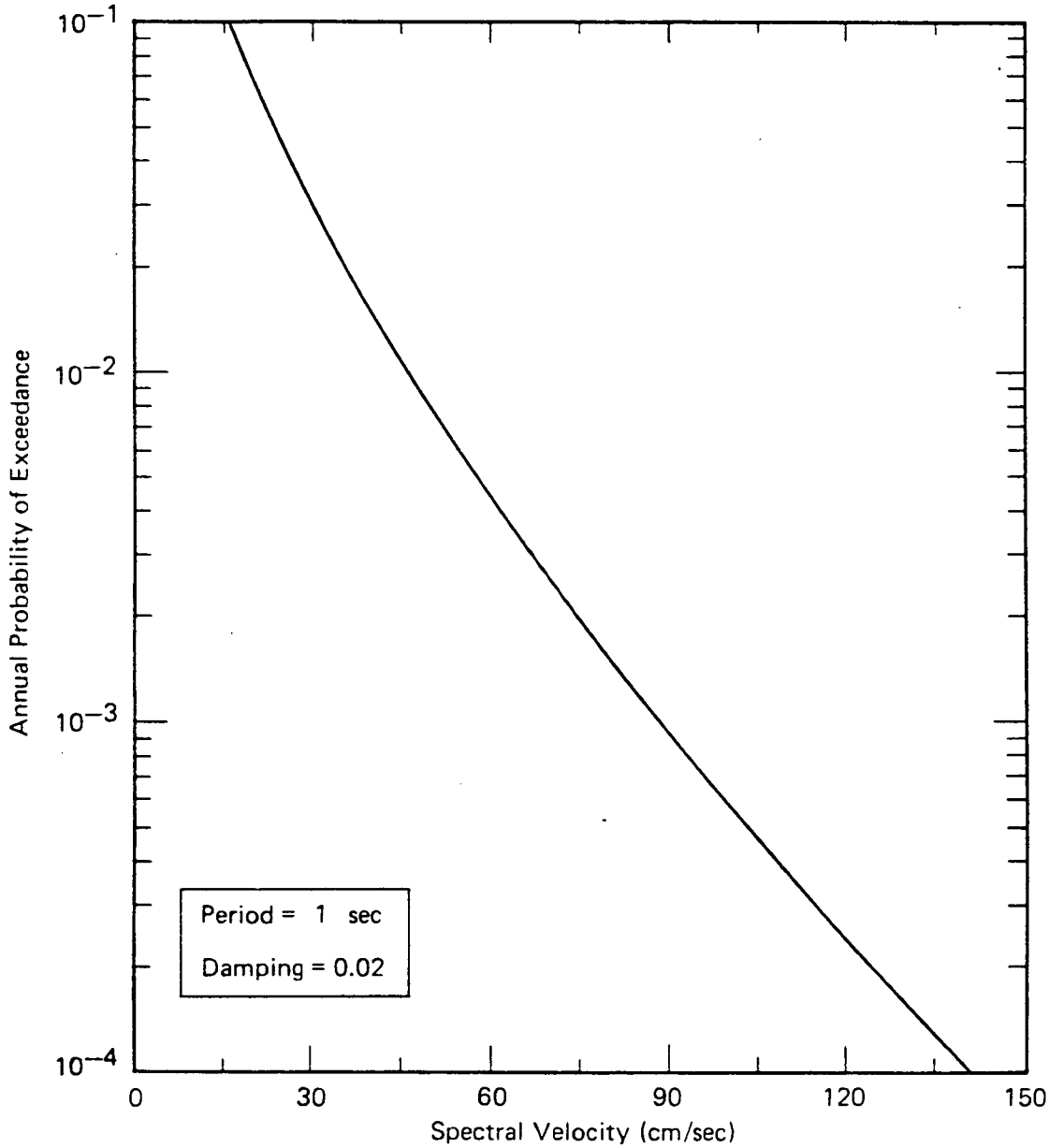

Figure 4-5 — Annual Probabilities of Exceeding Different Levels of Spectral Velocity (Period = 1 sec, Damping = 0.02)

5.0 SENSITIVITY OF THE PROBABILITY OF EXCEEDANCE TO INPUT DATA VARIATIONS

5.1 Introduction

The expected values of annual probability of exceedance presented above were based on the best estimates of the input data from the assessment described in Section 3. To examine the sensitivity of those results to variations of the input data, the probabilities of exceeding various levels of instrumental peak ground accelerations have also been evaluated for extreme ranges of the various input parameters. In these sensitivity analyses, variations have been limited to the data pertaining to the Hypothesized OZD rather than the entire study region because, as discussed in Section 4.3, the Hypothesized OZD dominates the total exposure of the site.

5.2 Results of Sensitivity Analyses

The results of the analyses for an instrumental peak acceleration of 0.5 g, which are typical for other levels of instrumental peak acceleration, are summarized below:

- for slip rate variations within the 0.30 to 0.68 mm/year range assigned to the Hypothesized OZD (value in analysis documented in Section 4 was 0.5 mm/year), the annual probability of exceedance ranges between 4×10^{-4} and 8×10^{-4};

- for values of the parmeter 'b' (of the Gutenberg-Richter relationship) between 0.7 and 1.0, the annual probability of exceedance varies from 3×10^{-4} to 8×10^{-4};

- for a maximum magnitude earthquake of M_s = 7 on the Hypothesized OZD together with the maximum slip rate for the Hypothesized OZD, the annual probability of exceedance was calculated to be within 10% of that calculated for M_s 6-1/2;

- using dispersion values varying with magnitude from 0.62 for magnitudes 4 to 4-1/2 to 0.38 for magnitudes 6 to 6-1/2 with the mean attenuation relationships presented in Section 3, has minor effect (approximately 20 percent increase) on the annual probability of exceedance for the magnitude range 5-1/2 to 6-1/2 which, as discussed in Section 4.3, provides the vast majority of the Hypothesized OZD contribution;

The use of this type of dispersion relationship, i.e. decreasing with increasing magnitude, is consistent with the observed trend of the empirical data. Donovan and Bornstein (1978) provide dispersion values as a function of acceleration level (varying from 0.3 for accelerations greater than 0.3 to 0.48 for an acceleration of 0.05 g). Also, the use of dispersion values (larger than 0.38) with the mean attenuation relationship selected for SONGS would be too conservative. In general, published relationships with higher reported dispersions exhibit a correspondingly lower mean peak acceleration and cover a wider range in site conditions than the attenuation relationship used in the present study (Idriss, 1978 contains a summary of widely used attenuation relationships). As a result, the calculated probabilities of exceedance using a lower mean acceleration level would be decreased providing little or no effect on the calculated probabilities.

Considering the observations above, the annual probability of exceedance for an instrumental peak ground acceleration of 0.5 g was found to be in the range of 3×10^{-4} to 9×10^{-4} for reasonable variations of the input data. Because this represents a small variation in probability from the calculated 6×10^{-4} and because the variations evaluated above are considered extreme, the calculated equal probability spectra shown on figure 1-3 are considered reasonable and conservative for the SONGS site.

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