DOCKETED USNRC
DEC 11 1979
Office of the Secretary
Docketing & Service
Section

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING APPEAL BOARD

In the Matter of: PACIFIC GAS & ELECTRIC COMPANY (Diablo Canyon Nuclear Power Plant Units 1 & 2)	Docket Nos. 50-275 O.L. 50-323 O.L.

JOINT INTERVENORS' BRIEF IN SUPPORT OF EXCEPTIONS

David S. Fleischaker, Esq.
1735 Eye Street, N.W.
Suite 709
Washington, D.C. 20006
(202) 638-6070

John R. Phillips, Esq.
CENTER FOR LAW IN THE PUBLIC INTEREST
10203 Santa Monica Boulevard
Fifth Floor
Los Angeles, California 90067
(213) 879-5588

ATTORNEYS FOR JOINT INTERVENORS

DECEMBER 7, 1979

7912310 457 G

TABLE OF CONTENTS

TABLE OF AUTHORITIES ii

INTRODUCTION 1

STATEMENT OF FACTS AND COURSE OF THE PROCEEDING. . . . 3

ARGUMENT AND AUTHORITIES 8

A. INTRODUCTORY STATEMENT 8

B. THE LICENSING BOARD'S FINDINGS RELATED TO THE HOSGRI FAULT ARE ERRONEOUS. 13

C. THE LICENSING BOARD ERRED IN FINDING THAT THE RESPONSE SPECTRA USED IN THE REANALYSIS OF THE DCNPP FOR A 7.5 MAGNITUDE HOSGRI EARTHQUAKE WERE APPROPRIATELY CONSERVATIVE AND REASONABLY ASSURE THE PRESERVATION OF THE HEALTH AND SAFETY OF THE PUBLIC 16

D. THE LICENSING BOARD ERRED IN FINDING THAT THE APPLICANT'S PROGRAM OF TESTING AND ANALYSIS DEMONSTRATED THAT CATEGORY I STRUCTURES, SYSTEMS AND COMPONENTS WILL PERFORM AS REQUIRED DURING THE 7.5 MAGNITUDE HOSGRI EARTHQUAKE AND POSSIBLE AFTERSHOCKS 56

E. THE LICENSING BOARD ERRED IN FINDING THAT USE OF AN OBE OF 0.20g MEETS REGULATORY REQUIREMENTS. 69

CONCLUSION, INCLUDING REQUEST FOR ORAL ARGUMENT 82

TABLE OF AUTHORITIES

A. Cases

1. Court Cases:

Environmental Defense Fund v. EPA, 548 F.2d 998 (D.C. Cir. 1976) cert. denied, 431 U.S. 925 (1977)

Greater Boston Television Corp. v. F.C.C., 444 F.2d 841 (D.C. Cir. 1970) cert. denied, 403 U.S. 923 (1971)

NRDC v. NRC, 547 F.2d 663 (D.C. Cir. 1976) rev'd sub nom. Vermont Yankee Nuclear Power Corp. v. NRDC, 435 U.S. 519 (1978)

SEC v. Chenery Corp., 34 U.S. 80 (1943)

WAIT Radio v. F.C.C., 458 F.2d 1153 (D.C. Cir. 1969)

Wingo v. Washington, 396 F.2d 633 (D.C. Cir. 1968)

2. Administrative Cases:

Consumer Power Company (Midland Plant Units 1 & 2), ALAB-283, 2 NRC 11 (1975), on reconsideration, ALAB-315, NRCI-76-2, (1976)

Northern States Power Co. (Prairie Island Nuclear Generating Plants, Units 1 & 2) ALAB-104, 6 AEC 179 (1973)

Pacific Gas & Electric Company (Diablo Canyon Nuclear Power Plant Units 1 & 2), ALAB-519, 9 NRC ___ (January 23, 1979)

Pacific Gas & Electric Company (Diablo Canyon Nuclear Power Plant, Units 1 & 2), LBP-79-26, 9 NRC ___ (September 27, 1979)

Public Service Company of New Hampshire et al. (Seabrook Station, Units 1 & 2), ALAB-422, 6 NRC 33 (1977)

Public Service Company of New Hampshire, et al. (Seabrook Station, Units 1 & 2), ALAB-561, 10 NRC ___ (1979) (Dissenting Opinion of Mr. Farrar, Slip. Op. August 3, 1979)

Vermont Yankee Nuclear Power Corp. (Vermont Yankee Nuclear Power Station), ALAB-138, RAI-73-4

Vermont Yankee Nuclear Power Corp. (Vermont Yankee Nuclear Power Station), ALAB-194, 8 AEC 431 (1974)

B. Statutes

Administrative Procedure Act, 5 U.S.C. § 554

Atomic Energy Act, 42 U.S.C. § 2012

C. Regulations

10 CFR § 100, Appendix A

D. Miscellaneous

Amendment to 10 CFR § 5100, Appendix A, Statement of Considerations, 42 Fed. Reg. 2051 (January 10, 1977)

Letter from Stephen Lawrowski, Chairman, Advisory Committee on Reactor Safeguards to Joseph M. Hendrie, U.S. Nuclear Regulatory Commission (July 14, 1978)

Proposed Appendix A, Statement of Considerations, 39 Fed. Reg. 31279 (December 13, 1973)

Risk and Responsibility, 205 SCIENCE 277 July 20, 1979

SECY-79-300, Attachment F (Memorandum from Robert B. Minogue, Director, Office of Standards Development to Commissioner Marson, NRC, 2 (October 8, 1976)

Sutherland's Statutory Construction, C. Sands, § 25.04 (4th ed. 1973)

DOCKETED USNRC DEC 14 1979 Office of the Secretary Docketing & Service Section

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING APPEAL BOARD

In the Matter of: PACIFIC GAS & ELECTRIC COMPANY (Diablo Canyon Nuclear Power Plant Units 1 & 2)	Docket Nos. 50-275 O.L. 50-323 O.L.

JOINT INTERVENORS' BRIEF
IN SUPPORT OF EXCEPTIONS

INTRODUCTION

Pacific Gas and Electric Company ("Applicant") is seeking a license to operate the Diablo Canyon Nuclear Power Plant ("DCNPP"), located on the California coastline about halfway between Los Angeles and San Francisco.[1/] The central safety issue in this proceeding is whether the DCNPP is adequately protected against earthquakes. This is because of the discovery - after construction was well underway - of the Hosgri fault, an active geologic fault only four and one-half (4-1/2) miles offshore of the plant site.

1/ DCNPP consists of two Westinghouse pressurized reactors located on a 750 acre site, in San Luis Obispo County, California. The units are designed to generate at steady state power levels of 3338 and 3411 megawatts (MWe) thermal with a net total electrical output of approximately 2120 MWe. The Applicant is seeking authorization to operate both units.

The DCNPP's original design incorporated protection against earthquakes. This protection was based on certain assumptions regarding the maximum earthquake that the plant could reasonably be expected to experience during its lifetime.[2/] It turns out that the original earthquake assumptions were wrong.

The discovery of the Hosgri fault and the assignment to it of a 7.5 magnitude earthquake capability imposed a new set of problems that had not been considered in the original design. To deal with those problems, the Applicant and Staff reanalyzed the DCNPP's seismic design. On the basis of that reanalysis, the Applicant and Staff conclude that the DCNPP, with certain modifications, can safely withstand a 7.5 magnitude earthquake on the Hosgri fault.

2/ For the original design, it was assumed that the following earthquakes could occur during the operating lifetime of the plant:

(1) Magnitude 8-1/2 along the San Andreas fault 48 miles from the site.

(2) Magnitude 7-1/4 along the Nacimiento fault 20 miles from the site.

(3) Magnitude 7-1/2 along the off-shore extension of the Santa Ynez fault 50 miles from the site.

(4) Magnitude 6-3/4 aftershock near the site associated with (1).

Safety Evaluation Report ("SER"), Supp. 4 at 2-4.

The Atomic Safety and Licensing Board ("Licensing Board") agrees with the Applicant and Staff. In the partial Initial Decision issued September 27, 1979, the Licensing Board found that the Staff and Applicant's reanalysis demonstrated that the DCNPP can safely withstand a 7.5 magnitude earthquake on the Hosgri fault.[3/]

The Joint Intervenors filed exceptions to the partial Initial Decision. This brief is filed in support of those exceptions.

STATEMENT OF FACTS AND COURSE OF THE PROCEEDING

Construction permits were issued for Unit 1 on April 23, 1968, and for Unit 2 on November 9, 1970. Construction was well underway at both units when, in July, 1973, the NRC Staff learned of the Hosgri fault from a PG&E report.[4/] Safety

---

3/ Pacific Gas & Electric Company (Diablo Canyon Nuclear Power Plant, Units 1 and 2), LBP-79-26, 9 NRC ____ September 27, 1979).

4/ The Hosgri fault was initially discovered in the late 1960's by two exploration geologists working for the Shell Oil Company. The geophysical data revealing the structure was classified proprietary by the oil company and apparently not made public until January, 1971 when the two geologists, Hoskins and Griffiths, published the data and their interpretation of the data in a scientific publication. The NRC Staff, however, remained unaware of the discovery until mid-1973. The Applicant cited the Hoskins and Griffiths publication and provided a map showing the location of the Hosgri fault offshore of the plant site in the operating license application filed in July, 1973. SER, Supp. 4 at 2-2.

Evaluation Report ("SER"), Supp. 4 at 2-2. The Applicant's 1973 report concluded that the newly discovered fault did not pose a threat to the facility. Final Safety Analysis Report ("FSAR"), Sec. 2.5, Amendment 11. Nevertheless, the Staff requested the Applicant to investigate further the earthquake potential of the newly discovered fault. SER, Supp. 4 at 2-2. The Staff also requested the U.S. Geological Survey ("USGS") to review the Applicant's findings and to provide an independent assessment of the Hosgri fault's earthquake potential. SER, Supp. 4 at 2-2.

In November, 1974, based on preliminary assessments of the Hosgri fault's earthquake potential, the Staff requested the Applicant to analyze the plant's ability to withstand an earthquake requiring a "design value" of 0.50g.[5/] SER, Supp. 7 at 1-3. The original seismic design of the facility required a "design value" of 0.40g. SER, Supp. 11 at 2-14.

The initial indications were that the as-built plant would qualify for the 0.50g design. SER, Supp. 7 at 1-3.

5/ Reference to a "design value of 0.50g" is a shorthand description for a complex set of earthquake (seismic) design criteria. These criteria are embodied in the "design response spectrum" - a model that depicts, in terms useful to engineers, the forces associated with an earthquake. This will be discussed in more detail later in the brief. *Infra*. at 16. It is sufficient here to note that the higher the "design value" the greater the earthquake forces the plant must be built to withstand.

However, on January 28, 1975, the USGS forwarded the results of their analysis of the Hosgri fault. The USGS concluded that (a) ". . . an earthquake similar to the November 4, 1927 (7.3 Magnitude) . . . represents the maximum earthquake that is likely to occur near to the site;" and (b) ". . . the design value of 0.5g used as a zero period acceleration in the development of the appropriate response spectra is inadequate." SER, Supp. 1 at D-8,9.

Following receipt of the USGS January, 1975 report, the Staff requested the Applicant to conduct additional investigations of the Hosgri fault. SER, Supp. 4 at 2-3. The Applicant completed those investigations in the fall of 1975 and forwarded its reports to the NRC Staff. The reports (Amendments 19 and 20 to the FSAR) concluded that the maximum earthquake potential for the Hosgri fault was 6.25-6.5 magnitude. The USGS reviewed the report, and on January 26, 1976, informed the Staff that it did not agree with that conclusion. Instead, the USGS reiterated its position that an earthquake - "with a magnitude of about 7.5 - could occur in the future anywhere along the Hosgri fault." SER, Supp. 4 at C-15. In addition, the USGS recommended that

> the ground motion values as exemplified by Table 2 "Near-fault horizontal ground motion" of . . . [Geological Survey Circular 672] . . . for magnitude 7.5 be used to form the basis of a description of the earthquake postulated to have the potential for occurring on the Hosgri

> fault at a point nearest to the Diablo Canyon site subject to the conditions placed on these values in . . . [Geological Survey Circular 672]. The earthquake so described should be used in the derivaton of an effective engineering accelera-tion for input into the process leading to the seismic design analysis.

SER, Supp. 4 at C-16.

In May, 1976, the Staff announced that it had accepted the USGS conclusion and would require the Applicant to determine the DCNPP's ability to withstand a 7.5 magnitude earthquake on the Hosgri fault. SER, Supp. 4. In addition, the Staff outlined the engineering procedures to be used in the reanalysis. SER, Supp. 4 at 3-1. Those procedures, discussed at length later in this brief, are the center of controversy in the proceeding.

Approximately two years of reanalysis followed. During this same period, and up until June, 1978, the Staff and the Applicant met periodically with the Advisory Committee on Reactor Safeguards ("ACRS") to discuss the reanalysis. Dr. Mihailo Trifunac and Dr. Enrique Luco were two consultants deeply involved in that review. Throughout, Drs. Trifunac and Luco submitted comments to the ACRS critical of the pro-cedures adopted by the Staff and the Applicant for reanalysis of the DCNPP. And, in a development perhaps unique to this proceeding, the Joint Intervenors obtained subpoenas compel-ling the testimony of Drs. Trifunac and Luco at the evidentiary hearings.6/ That testimony will be discussed at length later in this brief.

Despite the concerns voiced by these consultants, in July, 1978 the ACRS issued a letter approving the DCNPP reanalysis.

The Committee concluded as follows:

> It is evident from the foregoing that the design bases and criteria utilized in the seismic reevaluation of the Diablo Canyon Station for the postulated Hosgri event are in certain cases less conservative than those that would be used for an original design. The Committee believes, however, that there are offsetting factors that lead to acceptance of these bases and criteria for an already completed plant. They include . . . (1) the fact that the Committee's consultants believe that the choice of magnitude 7.5 for the postulated Hosgri event is relatively more conservative than the values considered acceptable for other plants; (2) because of the extent and depth of the Staff's review of the Applicant's seismic reevaluation, the likelihood of an undetected error in the seismic analyses or design is greatly reduced; and (3) the fact that the population density around the Diablo Canyon site is low. For these reasons, the Committee believes that, without endorsing all details of the NRC seismic design bases and criteria, the use of the Staff approach leads to an acceptable level of safety in this instance.7/

Evidentiary hearings were held in three segments: December 4-22, 1978; January 3-15, 1979; and February 7-15,

---

6/ Pacific Gas and Electric Company (Diablo Canyon Nuclear Power Plant, Units 1 and 2), ALAB-519, 9 NRC ___ (January 23, 1979).

7/ Letter from Stephen Lawrowski, Chairman, Advisory Committee on Reactor Safeguards to Joseph M. Hendrie, U.S. Nuclear Regulatory Commission (July 14, 1978).

1979. A total of 33 days were spent in hearings and over 10,000 pages of testimony taken. Proposed findings were submitted in late March and early April. On September 27, 1979 the Licensing Board issued a partial Initial Decision addressing the seismic safety issues.[8/]

ARGUMENT AND AUTHORITIES

A. Introductory Statement

The only thing standing between Pacific Gas and Electric Company and an operating license are the Commission's regulations. PG&E contends that the plant is safe, as does the Staff and the Licensing Board. Were it up to them the DCNPP would operate. The regulations are the main stumbling block.[9/]

The Commission's regulations are intended to reflect the philosophy that "public safety is the first, last and a permanent consideration" in decisions to license nuclear power plants. Consumer Power Co. (Midland Plant, Units 1 and 2), ALAB-315, NRCI-76-2, 103-4, citing Power Reactor Co. v. Electricians, 367 U.S. 396, 402 (1961). They command that nuclear power plants be built with substantial margins of safety.

8/ LBP-79-26, 9 NRC ___. Issuance of the Licensing Board's partial Initial Decision was apparently delayed by the "licensing moratorium" imposed in the wake of the Three Mile Island-2 accident.

9/ The Atomic Energy Act plainly makes compliance with the Commission's regulation a precondition to obtaining an operating license. See, e.g., Sections 183.d, 186.g, 187, 42 U.S.C. §2233(d), 2236(g) and 2237.

Appendix A embodies that philosophy. The strict criteria that it sets down - designed to assure that nuclear power plants can withstand the effects of earthquakes - were written in recognition that the occurrence of earthquakes and their possible effects on nuclear power plants were not well understood. Public Service Co. of New Hampshire, et al. (Seabrook Station, Units 1 and 2), ALAB-422, 6 NRC 33, 111 (1977, Mr. Farrar dissenting) (ALAB-422). And so Appendix A, in uncompromising terms, demands substantial margins of safety. It requires that seismic design criteria for nuclear power plants reflect the "maximum earthquake potential" of the region, Sec. III(c); that nuclear power plants be designed to withstand the "maximum vibratory ground motion" based on an evaluation of the "most severe earthquake" likely to occur in the region, Sec. V(a); that earthquake faults near the plant be assigned the "greatest magnitude related to that fault," which may be larger than the "maximum" historical earthquake based geologic evidence, Sec. V(a)(1)(i); and that the "maximum vibratory accelerations" at the site "shall be" the design criteria, calculated by assuming that earthquakes occur on the fault at a point closest to the plant, Secs. V(a)(1)(i)&(iv).

The Licensing Board ruled that the DCNPP is designed adequately to safeguard the public from an earthquake induced accident. That finding, however, is not rooted in the standards set out in Appendix A. Instead the Licensing Board applied standards fashioned by the Staff and Applicant, standards fashioned to meet an exceptional circumstance: the discovery of the Hosgri fault and the need to prove that a $2 billion nuclear facility is safe. These standards, embodied in the various reduction factors applied to the Safe Shutdown Earthquake (SSE) analysis - effective acceleration, tau effect, 7% damping - as well as the Operating Basis Earthquake (OBE) approach, reduce the margin of safety below that required by Appendix A.

Nuclear reactors may not be licensed unless they comply with all applicable regulations. Vermont Yankee Nuclear Power Corp. (Vermont Yankee Station), ALAB-138, RAI-73-4, 528-29. (ALAB-138). Staff and Applicant assurances that the plant is safe are not enough. The Commission's regulations are the definition of what is required to protect the public health and safety, and if they are not met, the license cannot be granted. ALAB-138 at 528. The decision below violates that principle and others as well.

Before a nuclear power plant is permitted to operate, the party who wants to run it - not the opponent - must show that it is safe. Consumer Power Co. (Midland Units 1 and 2), ALAB-283, 2 NRC 11, 16-18 (1975), on reconsideration, ALAB-315, 3 NRC 101 (1976); Environmental Defense Fund v. EPA, 548 F.2d 998, 1004-5, 1012-18 (D.C. Cir. 1976), cert. denied 431 U.S. 925 (1977). The Licensing Board's decision neglects that principle. In every case the conflict between the testimony of Intervenor witnesses (including ACRS experts) and that of Staff or Applicant, are resolved favorable to the Applicant and Staff. Their case is cloaked with a presumption of validity, and the burden on the Intervenor to prove that the plant is unsafe.

The Licensing Board's decision ignores regulatory principles that mandate a conservative approach. The task in this proceeding - determining whether the DCNPP is adequately protected during a 7.5 magnitude earthquake - involves concepts at the frontier of scientific thinking and relates to phenomena for which there is little observational data. The Board's decision is critical to public health and safety. Under the circumstances, there is a duty to confront and to explore fully the depth and consequences of the problems presented. NRDC v. NRC, 547 F.2d 633, 653 (D.C. Cir. 1976) rev'd sub nom. Vermont Yankee Nuclear Power Corp. v. NRDC, 435 U.S. 519 (1978).[10/]

The Licensing Board, however, fails to do that. Significant issues were either left hanging or never addressed. In more than one instance, the Licensing Board failed to confront the facts and provide a reasoned basis for the conclusions drawn. Seabrook, ALAB-422.

10/ See also, Risk and Responsibility, 205 SCIENCE 277 (July 20, 1979).

Northern States Power Co. (Prairie Island Nuclear Generating Plants, Units 1 and 2), ALAB-104, 6 AEC 179 (1973).11/

Finally, licensing hearings are convened to test the positions of the respective parties. If hearings are to serve their intended purpose of providing for full and fair inquiry into questions vitally affecting the health and safety of the public - rather than serve as a purely decorative mechanism for placing an automatic stamp of approval on the decisions previously made by the Staff - then it is not enough to simply compile a voluminous record. The evidence must be judged against the Commission's standards, not those of the Staff. The burden of proof must be placed where, by law, it belongs, on the Applicant. And the Licensing Board must confront all of the evidence and provide a reasoned basis for its decision. The failure to follow these principles deprives the hearing process of its meaning.

11/ See also, SEC v. Chenery Corp., 34 U.S. 80, 94 (1943); Greater Boston Television Corp. v. F.C.C., 444 F.2d 841, 851-3 (D.C. Cir. 1970) cert denied 403 U.S. 923 (1971); WAIT Radio v. F.C.C., 458 F.2d 1153, 1156 (D.C. Cir. 1969); Wingo v. Washington, 396 F.2d 633, 636 (D.C. Cir. 1968).

B. THE LICENSING BOARD FINDINGS RELATED TO THE HOSGRI FAULT ARE ERRONEOUS

All parties and the Licensing Board agree that the assignment of a 7.5 magnitude earthquake to the Hosgri fault is acceptably conservative. Decision at 42.[12/] Nevertheless, the Joint Intervenor's filed exceptions to the Licensing Board's findings on the matter.[13/] The evidence offered by the Joint Intervenors paints a very different picture of the Hosgri fault from that portrayed in the Licensing Board's findings. Those findings characterize the Hosgri as a minor, discontinuous and segmented feature, Decision at 27, that has experienced no more than 20 kilometers of slip over the last 20 million years, Decision at 42, and no major cumulative slip during the last 5 million years, Decision at 28. The fault is not associated with significant seismic activity, Decision at 32, and the assignment of a 7.5 magnitude to it is very conservative, Decision at 55.

12/ The Licensing Board's decision is cited as "Decision;" Joint Intervenors' Exceptions, as "J.I. Exceptions;" Joint Intervenors' Exhibits, as "J.I. Ex.;" Licensing Board Exhibits, as "L.B. Ex.;" Applicant Exhibits as "App. Ex.;" and Staff Exhibits as "Staff Ex." All cites to the transcripts are noted as "TR."

13/ J.I. Exceptions at 1-39.

In contrast, the Intervenors' testimony[14/] depicts the Hosgri as a major feature that is part of a through-going system of faults, approximately 400 kilometers in length with an estimated accumulated offset of 80-115 kilometers. TR 6196-8; 6364; 9466-9696. The Hosgri fault is the likely source of the 1927 earthquake - magnitude 7.3, and the assignment of a 7.5 magnitude assignment to it is appropriate.

We will not brief in detail our disagreement with the Licensing Board's findings. It is sufficient to make this one point. In almost every case where there was a conflict in testimony between the Joint Intervenors' witnesses and the Applicant or Staff witnesses, the Licensing Board resolved the question of fact against the Intervenor. We contend that such a one-sided decision is not justified by the record. Rather the Licensing Board "view[ed] the evidence presented by the Intervenors with an unjustifiably jaundiced eye, demanding from them what they do not expect from the Staff or Applicant -- strict proof neither within the grasp of any practitioner of the seismological arts nor demanded by the regulations." ALAB-422 at 112 (dissenting opinion of Mr. Farrar).

14/ Witnesses for the Intervenors included: Dr. Clarence Hall, Chairman, Department of Geology, UCLA; Dr. Eli Silver, Associate Professor, Earth Sciences Department, U.C. Santa Cruz; Dr. Steven Graham, Exploration Geologist, CHEVRON, USA, Inc.; Dr. Hall's qualifications are set out in J.I. Ex. 72; Dr. Silver's in J.I. Ex. 49; and Dr. Graham's, in J.I. Ex. 48.

Because the Licensing Board has concluded that a 7.5 magnitude earthquake ("the Hosgri Earthquake") should be used in the reanalysis, we turn now to the major points of disagreement.

C. THE LICENSING BOARD ERRED IN FINDING THAT THE RESPONSE SPECTRA USED IN THE REANALYSIS OF THE DCNPP FOR A 7.5 MAGNITUDE HOSGRI EARTHQUAKE WERE APPROPRIATE, CONSERVATIVE AND REASONABLY ASSURE THE PRESERVATION OF THE HEALTH AND SAFETY OF THE PUBLIC

Introductory Statement

The central dispute in this proceeding concerns the response spectra used to determine whether the DCNPP can safely withstand the effects of the Hosgri earthquake. The Licensing Board concluded that the response spectra were "appropriate," "conservative," and "reasonably assure the preservation of the health and safety of the public." Decision at 75. That conclusion is based on erroneous fact finding and misapplication of the controlling regulation - Appendix A. The discussion that follows shows why that is the case. It begins with the regulatory requirements, then moves to evidence, and finally addresses the Licensing Board's decision.

Regulatory Requirements

The methodology of Appendix A is based on developing response spectra depicting the earthquake forces[15/] that

15/ The forces associated with an earthquake are described in terms of "acceleration," "displacement" and "velocity." TR 5492-5. These are quantities that can be and are measured by scientific recording devices. TR 5495-6. "Displacement" is the amount of ground movement at any given moment during an earthquake. It is usually measured in centimeters. "Velocity," measured in centimeters per second, is a measure of how fast the ground moves. "Acceleration," usually measured as a fraction of gravity ("g"), is the rate at which velocity changes. TR 5493. During an earthquake these quantities vary as a function of time. Thus, the "time history" for acceleration is a record of the acceleration experienced at a location during a particular earthquake. TR 5496. Each time history will display a "peak" or "maximum" acceleration which is the highest acceleration recorded. Time histories are likewise recorded for velocity and displacement.

nuclear power plants must be built to withstand.16/ One is developed for each structure to estimate earthquake-related stresses that must be taken into account in that structure's design and construction. In addition, the response spectrum for each structure is the starting point for developing criteria to test the ability of systems and components (e.g., piping valves, motors electrical instrumentation) within that structure to perform required safety functions during and following an earthquake. TR 8591; 9006; 9897-9903.

Appendix A requires that seismic design response spectra correspond ". . . to the maximum vibratory accelerations at the elevation of the foundations of the nuclear power plant structures . . ." Sec. VI(a), Appendix A. These maximum vibratory accelerations are determined - in the case of capable faults - by assuming that the most severe earthquake that can be associated with the fault occurs at the point on the fault closest to the site. Sec. V(a)(1)(i), Appendix A. In this case that means the maximum vibratory accelerations at the DCNPP site are to be determined assuming a 7.5 magnitude earthquake occurs within 7 to 10 kilometers. The maximum vibratory accelerations "throughout the frequency range of interest" are to be reflected in the response spectra. Sec. V(a)(1)(iv)&VI(a), Appendix A.

16/ The usefulness of response spectra comes from the ability to model engineering structures by equivalent simple damped oscillators and to estimate stresses induced by the particular ground motion. J.I. Ex. 45 at 15; TR 6502; 8582.

Evidence

There are four principal areas of dispute: (1) the predicted ground motion at the DCNPP site produced by the Hosgri earthquake (magnitude 7.5); (2) the use of an "effective acceleration;" (3) the reduction applied to spectra to account for the "tau" effect; and (4) the use of a 7% damping factor for reinforced concrete structures. The evidence relating to each is discussed in the four sections that follow.

However, because of its significance in this proceeding, a brief summary of the testimony of ACRS consultants, Dr. Enrique Luco and Dr. Mihailo Trifunac is presented here.[17/] Their testimony is discussed in more detail in the following sections.

17/ Dr. Trifunac and Dr. Luco's Curriculum Vitae are L.B. Ex. 2A and 2B, respectively. Their professional qualifications, including educational and professional experience are discussed at TR 8850-62 (Luco) and TR 8966-68 (Trifunac).

In written reports to the ACRS[18/] as well as the sworn testimony before the Licensing Board, Drs. Luco and Trifunac outlined their disagreement with procedures employed to derive the response spectra used in the reanalysis. They conclude that (1) the Newmark and Blume response spectra represent the near source[19/] ground motion associated with a 6.5 magnitude earthquake, not a 7.5 earthquake; (2) there is no satisfactory physical basis for reducing the response spectra to reflect so-called "effective accelerations"; (3) reduction for the "tau effect" has not been proven appropriate for the Diablo Canyon site; and (4) use of 7% damping for reinforced concrete structures is not conservative and not adequately supported by available data.

Dr. Luco concluded that the methods employed in the reanalysis do not permit him to derive reliable conclusions about the structural response at the DCNPP for the Hosgri earthquake. TR 8866. In the event of 7.5 magnitude earthquake, structural response will likely be nonlinear in the inelastic range. TR 8867-70. No analysis has been presented of the response in that range. TR 8867. Dr. Luco's principal recommendation is that an inelastic analysis be performed on the structure. TR 8894; 8933.

18/ Dr. Luco and Dr. Trifunac describe their participation in the ACRS review at TR 8862 (Luco) and TR 8868-70 (Trifunac).

19/ "Near source" as used in the brief means within 10 kilometers of the earthquake source.

Dr. Trifunac's opinion is that a 6.5 magnitude earthquake is reasonable for the reanalysis. On this basis he concludes that the DCNPP structures - with the exception of the turbine building - are reasonably designed to withstand the forces associated with a 6.5 magnitude earthquake on the Hosgri. TR 9187; 9237; 9263. However, this statement is limited to structures and does not apply to the equipment within the structures. TR 9199. Equipment testing and analysis is based on unreliable and non-conservative criteria resulting from reductions to the response spectra for the so-called tau effect and use of 7% damping. TR 9006-7. Dr. Trifunac's principal recommendations are that (1) a three-dimensional soil structure interaction analysis, and/or (2) an inelastic analysis of the structural response be performed.

1. Ground Motion in the Free Field

This section discusses evidence relating to ground motion at the DCNPP site produced by the Hosgri earthquake. We begin with Geological Survey Circular 672, "Ground Motion Values for Use in the Seismic Design of the Trans-Alaska Pipeline System." J.I. Ex. 45.

Circular 672 was written for the seismic design of the Trans-Alaska Pipeline System. It provides the maximum values of ground acceleration, velocity and displacement expected within 10 kilometers of earthquakes of different magnitudes. These values are for ground motion in the free field unaffected by the presence of a structure. They are based solely on seismological data and principles and have not been modified to reflect soil structure interaction, deformational processes within the structure, or the importance of the structures to be designed. J.I. Ex. 45. Circular 672 at 3. Finally, the values "are not the maximum possible." Id. at 4.

The values assigned to earthquakes of magnitude 5.5; 6.5; 7.0; and 7.5 are provided below. 20/

Magnitude	Acceleration (g) Peak absolute values				Velocity (cm/sec) Peak absolute values			Displacement (cm)
7.5	1.15	1.00	0.85	0.65	135	115	100	70
7.0	1.05	0.90	0.75	0.55	120	100	85	55
6.5	0.30	0.75	0.60	0.45	100	80	70	40
5.5	0.45	0.30	0.20	0.15	50	40	30	15

Up to magnitude 6.5, the values are based on strong motion data. For magnitude 6.5, the values were derived from a single record: the Pacoima Dam accelerogram of the 1971 San Fernando Valley earthquake. The peak acceleration - which is of particular concern here - recorded at Pacoima

---

20/ J.I. Ex. 45. Table 2, Circular 672 at 3.

Dam was 1.25g. Because the peak acceleration for that record was dominated by high frequency energy, the record was filtered to remove very high frequencies, reducing the accelerations by 25%. USGS considers the filtered values to be a better estimate of peak accelerations for a 6.5 magnitude earthquake. J.I. Ex. 45. Circular 672 at 6-7.

Values for magnitudes larger than 6.5 were based on extrapolations. J.I. Ex. 45 at 3. The extrapolations are supported by existing strong motion data indicating that peak acceleration increases with magnitude for all distances, as well as theoretical arguments. J.I. Ex. 45. Circular 672 at 8.

The principal witness for the Staff on ground motion was Dr. Nathan Newmark. Dr. Newmark testified that the response spectra derived for the Pacoima Dam record of the 1971 San Fernando Valley earthquake - magnitude 6.5 - represents the upper limit for ground motion in the free field produced at the DCNPP site by the Hosgri earthquake. TR 8618-19; 9322. The explanation for equating ground motion recorded for a 6.5 magnitude earthquake with that predicted for a 7.5 magnitude earthquake has to do with (a) the relation between near source peak acceleration and magnitude and (b) the style of faulting associated with the Hosgri - strike-slip.

Dr. Newmark testified that near the source, peak accelerations are relatively independent of magnitude, for magnitudes greater than 6.0. TR 8596-98. Not magnitude, but stress release during rupture controlled peak accelerations near the fault. Thrust faults are thought to result in greater stress release than strike-slip faults, like the Hosgri. The San Fernando Valley earthquake was caused by thrust faulting. For that reason, Dr. Newmark concludes that ground motion recorded at Pacoima Dam represents the upper limit for the DCNPP site. TR 8624-30.

Dr. Newmark testified that the values in Circular 672 for magnitudes greater than 6.5 "were guesses that have no real validity." TR 8600. These values are based on extrapolations that are not valid. TR 8615. As for magnitude 6.5 and below, Dr. Newmark testified that the Circular 672 provides values that are very conservative, values that are "upper bound" estimates. TR 9288.[21/]

21/ Many of the same views were expressed by Staff witnesses Dr. Carl Stepp and Mr. Renner Hoffman. Stepp Testimony at 35-38; Hoffman Testimony at 3-8.

The Applicant's testimony on ground motion was offered by a panel of experts.[22/] While opinions varied somewhat from witness to witness, there was unanimity on two points. First, the values for free field ground motion presented in Circular 672 for 7.5 magnitude earthquakes were too high. Second, expected peak acceleration for a 7.5 magnitude earthquake, measured in the free field, was in the range of .50g-.80g.

Dr. Bolt testified that a range of .60g to .80g represented his estimate of the expected peak acceleration in the free field for the magnitude range 6.5 to 8.0. TR 5921. Dr. Bolt derived these values for peak accelerations on the basis of extrapolation from values recorded at some greater distance. TR 5922-5925.

Dr. Smith testified that the expected peak accelerations for earthquakes in the magnitude range 5.5 to 8.0 is .50g. This value is the average of the near fault (recorded within 10 km of the earthquake source) peak accelerations for approximately twenty-five (25) earthquakes. TR 5940-5948.

22/ The Applicant offered testimony on near fault ground motion twice during the hearing. Testimony was presented as part of the Applicant's direct case by members of the seismology panel - Drs. Bolt, Frazier and Smith. Testimony was again offered by Drs. Frazier and Seed as part of the Applicant's rebuttal case. The testimony regarding the probability of peak accelerations occurring at the DCNPP site was offered by Dr. Blume of the engineering panel as part of the Applicant's direct case.

Dr. Seed obtained a mean value of .70g for 7.5 magnitude earthquakes. This value was obtained by performing a regression analysis on a plot of near source peak accelerations vs. magnitude, for several earthquakes in the magnitude range 3.3-7.8. TR 10,103-05; App. Ex. 61 & 62. In addition, Dr. Seed obtained an average peak acceleration of .80g for earthquakes in the magnitude range 5.5-7.8. This value was obtained by taking the mathematical average of the four highest peak accelerations recorded within 10 kilometers of the earthquake source.[23/] TR 10,107; App. Ex. 63. It should be noted that for both this and the regression analysis, Dr. Seed reduced the peak acceleration recorded at Pacoima Dam from 1.25g to .80g.

As noted earlier, the Applicant's experts found fault with the estimate of ground motion in Circular 672 for magnitudes greater than 6.5. The reasons for rejecting the USGS values were that (a) they were dependent upon the

---

23/

Earthquake Record	Date	Magnitude	Distance	Peak Horizontal Acceleration
Naghan	1977	5.5	Very close	0.95g
Pacoima	1971	6.5	3 km	0.8g
Koyna	1967	6.5	5 km	0.63g
Gazli	1976	7.2	10 km	0.80g
			Avg.	0.80g

App. Ex. 63.

Pacoima Dam record, TR 5845-49; 5871-72; (b) the extrapolation was dependent on the assumption that, near the source, peak acceleration increased as a function of magnitude, J.I. Ex. 45 at 8; TR 5876-79; 5889-97; 5909; and (c) the methods outlined above provided reliable estimates of peak accelerations substantially lower than USGS values.

The Joint Intervenors presented the testimony of Dr. James Brune. Dr. Brune testified that for earthquakes greater than 7.0 magnitude and at close distances, peak accelerations and velocities could exceed, by a factor of two, those postulated by the USGS in Circular 672. J.I. Ex. 66, 3-2; TR 7963. Dr. Brune testified that for a 7.5 earthquake maximum accelerations near the source could exceed 2g and velocities near the source could exceed 200 cm/sec. Average accelerations may be about 1g. J.I. Ex. 66, at 3-19. These estimates are supported by extrapolations from the existing - although limited - data base. J.I. Ex. 66, at 3-6 through 3-9. Moreover, theoretical modeling of fault rupture provides an explanation for these high accelerations and velocities. They could be caused by one of several phenomenon: focusing of energy (directivity); high stress drop during fault rupture: or fault breakout. J.I. Ex. 3-10 through 3-16; TR 7936-39. Dr. Brune concluded that energy released about 20 km up the

Hosgri fault could be focused nearly directly at the DCNPP site - J.I. Ex. 66, at 3-13 - and that fault breakout could increase ground motion at the site. J.I. Ex. 66, at 3-16.

Dr. Brune specifically considered the arguments against these high values and concluded that they were outweighed by the arguments in favor of the high values. J.I. Ex. 66, at 3-18. Finally, Dr. Brune concluded that the values cited in Circular 672 for near source peak accelerations and velocities for a 7.5 magnitude earthquake have not been proved to be conservative. In his opinion, Circular 672 provides reasonable estimates of the expected ("mean") values for near source peak accelerations and velocities. J.I. Ex. 66, at 3-3 and 3-19.

Dr. Enrique Luco testified that a 7.5 magnitude earthquake on the Hosgri fault would produce ground motion values consistent with those recommended in Circular 672 and in publications by Dr. Trifunac. L.B. Ex. 2-C (Luco comments) at 2. TR 8873.[24/] Dr. Luco concluded that the Newmark and Blume response spectra (without tau reduction)[25/] do not reflect

---

24/ Dr. Luco independently verified Dr. Trifunac's attenuation function and was satisfied that it gave valid results. TR 8874-5. He noted that other researchers had found Dr. Trifunac's attenuation function produced accurate results. TR 8875.

25/ The Staff permitted PG&E to use either the Newmark response spectra or the Blume response spectra for the reanalysis, provided that the Blume spectra did not fall below the Newmark spectra. In most instances the Newmark spectra were used in the reanalysis. TR 6836.

the ground motion at the DCNPP site for a 7.5 magnitude earthquake, but rather the motion for a 6.5M earthquake. L.B. Ex. 2-C at 1. The peak acceleration, velocity and displacements controlling the high, intermediate and low frequency portions of the Newmark spectrum fall short by 40 to 60 percent of the Trifunac estimates of the values for a 7.5 magnitude earthquake - TR 8975 - and are also considerably lower than those suggested in USGS Circular 672. On the other hand, they are generally in agreement with the expected peak values obtained by Dr. Trifunac for a 6.5M earthquake.

COMPARISON OF MAXIMUM GROUND MOTIONS

	Peak values used by Newmark [1]	M = 6.5		M = 7.5	
		Trifunac [2]	USGS [3] No. 672	Trifunac [2]	USGS [3] No. 672
a_{max} (g)	0.75	0.69 (1.29)	0.90	1.07 (2.00)	1.15
v_{max} (in/sec)	24	23 (48)	39	39 (84)	53
d_{max} (in)	8	8 (19)	16	12 (30)	28

[1] Newmark, N. M., "A Rationale for Development of Design Spectra for Diablo Canyon Reactor Facility," Appendix C, Supplement No. 5, SER, Diablo Canyon Nuclear Power Station Units 1 and 2, NRC, 1976.

[2] Average (average + standard deviation) peak motion for rock at an epicentral distance R = 7.5km based on Trifunac, M. D., "Preliminary Analysis of the Peaks of Strong Earthquake Ground Motion - Dependence of Peaks on Earthquake Magnitude, Epicentral Distance and Recording Site Conditions," B.S.S.A., 66, 189-219 (1975).

[3] Page, R.A., et al., "Ground Motion Values for Use in the Seismic Design of the Trans-Alaska Pipeline System," Geological Survey Circular 672, 1972.

Source: J.I. Ex. 2-C, Table 1.

Dr. Luco addresses and rejects the arguments that the thrust fault mechanism and the location of the Pacoima Dam instrument in the San Fernando earthquake produced peak accelerations equivalent to those that the Hosgri earthquake - magnitude 7.5 - would produce at the site, assuming a strike-slip sense of motion. He argues, first, that the peak acceleration for the Pacoima record falls well within the standard deviation of mean values predicted by Circular 672 and Dr. Trifunac. Second, Dr. Luco, cites the Gazli earthquake (1976) - magnitude 7.6 - to support his position. The peak acceleration .80g was recorded at an epicentral distance of 10 kilometers. Correcting for attentuation, using Gutenberg's relation, results in a peak acceleration of 1.0g, an estimate in general agreement with the values obtained by Dr. Trifunac and cited in Circular 672. L.B. Ex. 2-C (Luco comments) at 2.

Dr. Trifunac recommends use of a 6.5 magnitude earthquake to reanalyze the DCNPP. L.B. Ex. D at 5; TR 8970. For that reason, Dr. Trifunac accepts the Newmark spectrum as an acceptable representation of ground motion for use in the DCNPP reanalysis. TR 8984-5.

2. Effective Acceleration

Supplement 4 to the Staff's SER states as follows:

> The ground motion values recommended by the U.S. Geological Survey are based on instrumental data insofar as possible and do not reflect the presence of

structures. These values must be translated into quantitative measures of effective acceleration for design purposes. To develop an effective acceleration for Diablo Canyon, we have obtained the advice of our consultant in this area, Dr. N. M. Newmark of N. M. Newmark Consulting Engineering Services. He has recommended, and we have accepted, that an effective horizontal ground acceleration of 0.75g be used for the development of design response spectra.

* * *

At a meeting on April 20, 1976, we requested that the Applicant evaluate the plant's capability to withstand [a 7.5 magnitude earthquake on the Hosgri fault]. An outline of the procedures that we believe would be appropriate for this evaluation is as follows:

(1) A magnitude 7.5 earthquake on the Hosgri fault should be assumed with horizontal ground response spectra normalized to an effective value of 0.75g for engineering reevaluation of the plant.

Supplement 4 at 2-4 and 2-5.[26/]

In its report to the NRC Staff, the USGS recommended "that the ground motion values . . . [in Circular 672] . . . for magnitude 7.5 be used to form the basis of a description of the earthquake postulated to have the potential for occurring on the Hosgri fault at a point nearest to the

---

26/ *See also* SER, Supp. 7 at 1-3.

In April 1976 we completed our review of the Hosgri fault's earthquake potential. As a result of our evaluation, we once again requested that the applicant reevaluate the plant's seismic design capabilities. The USGS recommended assuming a magnitude 7.5 earthquake on the Hosgri fault and provided instrumental ground motion values to be associated with such an event. As the USGS also recommended, we developed effective ground motion values to be used in the reanalysis. These effective values were based on the recommendation of our consultant, Dr. N. M. Newmark. The ground response spectra involved in the reevaluation are anchored at 0.75g horizontal acceleration, prior to any reduction of the high frequency portion to account for building size effects.

Diablo Canyon site. The earthquake so described should be used in the derivation of an effective engineering acceleration for input into the process leading to the seismic design analysis. SER, Supp. 4 at C-16.

On the witness stand, the USGS representative, Mr. Devine, explained that in the past, USGS had recommended to the NRC Staff a "single g value with the full intention of it being used as it stands, as the scaling high frequency end of the response spectrum." TR 8327; 8330.[27/] In this case, however, USGS, provided a description of ground motion in the free field, leaving it to the engineers to decide whether, and if so, how, the ground motion should be modified for purposes of structural analysis. TR 8329. The USGS <u>did</u> <u>not</u> take a position as to whether the Newmark response spectrum was an acceptable modification of the description of ground motion in Circular 672 - that is, the USGS took no position as to whether the response spectrum for the DCNPP reanalysis should be anchored to 0.75g. TR 8333.

Dr. Newmark provided the rationale for using an "effective acceleration" to the scale response spectra for the reanalysis. Dr. Newmark testified that the absence of significant structural damage near the earthquake source where high accelerations were recorded - and the general

---

27/ Mr. Devine is referring to the scaling function contemplated in Regulatory Guide 1.60.

good performance of structures subjected to earthquakes - is direct evidence that using near source peak accelerations to scale response spectra results in unreliable predictions of structural damage. SER, Supp. 5 at C-2 through C-10; at 2-3; B-1 through B-8. TR 8639-40; 8668. Specific examples where that is the case include Parkfield earthquake of June 27, 1966 (magnitude 5.6, peak acceleration 0.5g); the Melendy Ranch earthquake of September 4, 1972 (magnitude 4.8, peak acceleration 0.79g); the Ancona earthquake of June, 1972 (magnitude range, 4.4-4.9, peak acceleration 0.60g [on rock] and 0.40g [on sediment]). SER, Supp. 5 at C-6. This phenomenon can be attributed to three factors principally: (1) the absence of perfect coupling between the structure and the foundation, which results in energy loss at the interface; (2) the difference between the peak acceleration (as measured by an instrument) and the average acceleration to which a large rigid foundation responds - the so-called tau effect, TR 9345-49; (3) recording instruments tend to overrecord the peak acceleration actually experienced on the ground. TR 9349-50.

The value 0.75g was designated the effective acceleration at the site for the Hosgri earthquake principally on the basis that the Regulatory Guide 1.60 spectra scaled with an effective acceleration of 0.75g, generally envelopes the free field response spectra computed from the record at

Pacoima Dam (peak acceleration, 1.2g) for the 1971 San Fernando earthquake, (6.5 magnitude). SER, Supp. 5 at C-3; TR 8559; 8587-90; Dr. Newmark states. "[T]his is the most direct indication that the effective peak acceleration for the Pacoima Dam record is not in fact the measured value of 1.20g, but actually does not exceed 0.75g. Therefore this is taken as the effective peak acceleration for design." SER, Supp. 5 at C-4.

However, in rebuttal, Dr. Newmark testified that the scaling factor used in the reanalysis - 0.75g - does not involve any consideration of effective acceleration. TR 9275-6; 9321.

> The development of the .75g was based on taking the actual Pacoima Dam record, drawing its response spectrum, and that is shown in Figures 1A and 1B of that rationale report, and showing that the so-called Newmark spectrum, which is very close to the Reg. Guide 1.60 spectrum, envelopes the Pacoima Dam record if one scales that spectrum to .75g.
>
> So actually the .75g involves the use of the 1.2, essentially, measured acceleration at Pacoima Dam.

TR 9286.

The Applicant did not designate 0.75g as the effective acceleration level. Nevertheless, the Applicant's experts provided testimony on the matter.[28/] Applicant's witnesses testified that the concept of "effective acceleration" is a valid engineering concept. Blume Testimony at 2, 19-25; TR 6476; 10,102. Like Dr. Newmark, the Applicant's witnesses cite the absence of structural damage near the source where high accelerations were recorded - and the good performance of structures generally - as direct evidence that using near source peak accelerations to scale response spectra results in overpredicting structural damage. Blume Testimony at 21-25.

The Applicant's position is that a 6.5 magnitude earthquake should be used for the reanalysis. On that basis, Dr. Blume designated an effective acceleration of 0.50g to scale the response spectra.[29/] Blume Testimony at 12; TR 6683; 6495. Dr. Blume testified that for a 7.5 magnitude earthquake and a 1.15g instrumental peak acceleration, he would select an effective acceleration of .60g to anchor the response spectra used for the DCNPP reanalysis. TR 6495.

28/ Testimony as "effective acceleration" was provided principally by Dr. Bolt of the seismology panel and Drs. Blume and Seed of the engineering panel.

29/ The spectral shape was derived from the superimposition of eight earthquakes Dr. Blume selected as appropriate. Blume Testimony at 12-14; TR 6681-83.

Dr. Luco concludes that the reduction from 1.15g to an effective acceleration of 0.75g is out of order. Structures don't fail, when on paper they should, because of ordinary seismic analyses leave a safety margin. They do not account for the effects of soil structure interaction, are based on nominal values for damping and strength, assume linear behavior and do not include the energy dissipation in partitions and other non-structural elements. L.B. Ex. 2-C at 2-3.

In the case of the DCNPP, the seismic analysis has taken these factors into account. Test material strength rather than nominal values, higher than common damping values, allowance for ductility where necessary, has all been included in the reanalysis. L.B. Ex. 2-C at 2. Credit is taken for the scattering of waves by large foundations - "tau effect." L.B. Ex. 2-C at 3; TR 8894; 9124. Soil structure interaction is not likely to significantly reduce the free field ground motion at the levels of foundation. (During an earthquake a good coupling between the surrounding soil and the foundation is expected principally because the foundation is located on hard rock, not soft soil). TR 8894; 8924-6; 8932. Therefore, Dr. Luco concludes these is no physical basis for an effective acceleration reduction. L.B. Ex. 2-C at 3; TR 8894.

If the effective acceleration is a factor to take elastic response into account, Dr. Luco concludes that a study of inelastic response should be conducted. TR 8894. Reduction of response spectra to account for an inelastic response may be acceptable for ordinary facilities. But for a critical facility like the DCNPP, it is inappropriate. L.B. Ex. 2-C at 3. A principal problem is that this reduction means that the input motion into equipment testing is not accurate. TR 8894.

Dr. Trifunac accepts 0.75g based on a 6.5 magnitude Hosgri earthquake. For him the debate over effective acceleration is largely one of engineering principle. And, in principle, he agrees with Dr. Luco: the reduction in ground motion from 1.15g to 0.75g attributed to an effective acceleration has no physical basis. TR 8973.

3. "<u>Tau Effect</u>"

Supplement 4 to the SER provides that "[A] revision of the design response spectra will be accepted depending on the equivalent length of the foundations of individual buildings. This revision recognizes that ground motion waves are not synchronized underneath structures during earthquakes. In other words, different points in the foundation base slab will not experience the maxima in the ground motion at the same time."

Dr. Newmark testified that the tau reduction is based on the observation that structures with large foundations respond "with less intensity" than smaller structures. SER, Supp. 5 at C-10. This could be explained by the fact that at any given moment, acceleration over the base is not uniform. Structural response, therefore, is a function of an average acceleration value - not the maximum value. SER, Supp. 5 at C-10 and 11.

As proof, Dr. Newmark offers the records of two earthquakes from the Hollywood Storage Building and parking lot. The record from the basement is taken to represent motion at the foundation; the record from the parking lot, 112 feet away from the nearest corner of the building, to represent motion in the free field. Comparison of the response spectra derived from recordings at the two locations reveals that in the high frequency range - 2 hertz-25 hertz - the motion at the foundation was reduced from that recorded in the free field. SER, Supp. 5 at C-32 and 33 (Figures 10, 11, 14 and 15).

Dr. Newmark concludes that this reduction effect can be approximated by the following function:

$$A_f = A_o \times R$$

Where A_f = acceleration for foundation; A_o = acceleration for free field; and $R = 1 - 5\gamma$.

τ (or "tau"), the "transit time parameter" is determined by dividing the "effective" width (the square root of the area, in general) of the foundation by the wave velocity. The formula is consistent with analysis of the records from both the San Fernando and Kern County earthquakes. In both cases, the response spectra for the structure, when reduced in the high frequencies by a transit time parameter, approximates the spectra derived for the free field. SER, Supp. 5 at C-13; C-34 (Figure 12); C-35 (Figure 13); C-36 (Figure 14); C-37 (Figure 15).[30/]

At the evidentiary hearing, Dr. Newmark expanded on the physical basis for the tau reduction. Soil under the site is layered and hetergeneous. Waves would be reflected and refracted by this medium and they propagate from the earthquake source. TR 8568; 8656. As a result, the acceleration values at any time instant are different from those that would correspond to a uniform wave motion over the whole foundation. TR 8568; 9333. According to Dr. Newmark, the theory that waves arrive "out of phase" is verified by recordings of underground nuclear explosions where closely

---

30/ In reality the motion of the foundation is governed by an averaging of acceleration over its area, not the wave transit time. Acceleration, at any instant, varies both in the direction considered and in the transverse direction. SER, Supp. 5 at C-14. Newmark Testimony, App. B., Figures 1&2. However, use of the transit time parameter - like many engineering conventions, is a practical way to approximate a highly complex interaction.

spaced instruments have recorded different peak accelerations at the same instant. TR 9329-30.

Dr. Newmark testified that both torsion (horizontally incident waves) and tilting (vertically incident waves) had been analyzed. Neither were a significant problem. TR 8570; 8656; 8661-8666.

Members of the Applicant's engineering panel[31/] offered testimony on the "tau effect." Dr. Seed also testified extensively on the results of soil-structure interaction analyses that he performed. Because of their close relationship, they are discussed together.

The Applicant's witnesses testified that the tau reduction is an acceptable way of taking into account the "excellent performance of large building foundations in earthquakes." Blume Testimony at 42; TR 10,123-27. The formulation used in the DCNPP reanalysis - a reduction of the high frequency portion of the spectrum, as a function of the geometry of the foundation and seismic wave travel time - is a "simplification of a very complex wave motion - structure action problem. Blume Testimony at 42. In that regard, however, it is similar to many "engineering equivalents" used to approximate loading conditions. Blume Testimony at 32.

31/ Dr. Blume, Dr. Frazier and Dr. Seed.

Dr. Blume testified that the Applicant's calculations of the tau effect were conservative because it excluded two factors that would further reduce motion at the foundation level. First, the spectra could be further reduced to reflect that various large foundations at the site would act as a single large foundation reducing the high frequency response significantly more than a single foundation. Second, the Applicant did not reduce the response spectra to reflect the "mitigating effect" of the structures' embedments. TR 10,125-6.

Dr. Seed testified about the results of the soil structure interaction analyses he conducted for the Applicant to determine whether the presence of the soil or rock which underlies the DCNPP has any significant influence on the response of the structure. TR 6748. J.I. Ex. 58 at 1. Dr. Seed's analyses compared the results of three different models.32/

32/ The purpose of the comparison was to determine whether the rigid base analysis for the DCNPP - which ignores soil structure interaction - provides reliable results. Section 3.7 of the NRC Standard Review Plan provides that where the shear wave velocity at the site is greater than 3500 feet per second, as is the case at the DCNPP site, a rigid base analysis may be used to determine structural response. The Applicant performed such an analysis. Drs. Trifunac and Luco, among others, suggested that additional studies needed to be performed. This comparative analysis was performed "to explore the significance of those suggestions and see if indeed those kind of studies needed to be made for . . . [the DCNPP]. J.I. Ex. 58 at 1; TR 6800.

For all three models the property of the structure are those of the DCNPP. TR 6800-1. J.I. Ex. 58 at 2. In the first, the DCNPP is assumed to be located on a rigid base and the time history of ground motion - derived from Newmark's response spectra - 0.75g zero period limit - without reduction for tau effect - is input directly into the foundation. TR 6772-3; J.I. Ex. 58 at 2-3.

In both the second and third model, the soil is modeled as a multilayered medium with the physical properties (stiffness, damping action, and mass density) of the soils underlying the site. J.I. Ex. 58 at 2, 7. The foundation is assigned the physical properties of the DCNPP foundation. TR 6778. In model 2, the input motion is characterized as vertically incident shear waves (S-waves) and compression waves (P-waves). In model 3, the input motion is characterized as horizontally incident Rayleigh waves.[33/] Both model 2 and model 3 take into account soil deformability (effect of the soil on the wave fields) and radiation damping (loss of energy at the interface between the soil and the foundation). Model 1 does not. TR 6774;6778.

Dr. Seed concludes as follows:

> The good agreement between computed responses obtained by the three different methods illustrated in the preceding plots would seem to indicate that the effects

---

33/ See TR 5994-7 for description of the various earthquake waves.

of soil-structure interaction are very small for the case considered. In fact this conclusion would seem to be even more apparent if a comparison is made between the response spectra for motions in the free field and motions developed at nodal point 63 at the center of the base of the foundation. Such a comparison is shown in Fig. 13. In general the response at the base of the structure is slightly less than the free field response, particularly in the high-frequency range, and this is more true for Rayleigh wave excitation than for vertically propagating wave excitation. However, the difference is small and it would apparently be conservative to neglect interaction effects altogether and simply subject the structure to the prescribed control motion as if the base were rigid.34/

On the last day of the hearing, Dr. Seed presented further testimony on his soil structure interaction analysis, offered in part in rebuttal to Dr. Luco's testimony. During his appearance, Dr. Luco cited Dr. Seed's soil structure interaction analysis for the proposition that there is no significant soil structure interaction or "tau effect" at the site. TR 9034.35/ Dr. Seed responded that Dr. Luco had misinterpreted the analysis. TR 10,146.

34/ *See also*, TR 6803-4.

35/ In addition to the language quoted above, Dr. Seed's analysis states:

Essentially similar values of response are obtained for this site whether the base motions are considered to consist of a system of vertically propagating shear and compression waves or a system of horizontally propagating Rayleigh waves and except for a small increase in rocking which affects the outer edges of the foundation slab, the computed responses are essentially similar to those computed for a rigid base analysis where the control motions are used directly as base excitation for the structure.

* * *

On the bases of other analyses made for inclined shear wave propagation leading to phase differences across the base of the structure it seems reasonable to conclude that the results presented in the preceding study would closely simulate these effects also. [emphasis added]. J.I. Ex. 58 at 12-13.

Dr. Seed testified that use of the words "very small" in the report describing the soil structure interaction effects is not the same as "no significant effects" as used by Dr. Luco. TR 10,150. According to Dr. Seed, the report shows that soil structure interaction reduces motion in the base of the structure on average, 20% from motion in the free field. Thus, a free field peak acceleration of .75g corresponds to .60g response in the foundation. TR 10,149.

Dr. Seed also excepts to Dr. Luco's reading that the report concludes there is no significant tau effect. Dr. Seed states that the analyses were not designed to take tau effect into account. TR 10,150. Were the tau effect to be taken into account, further reduction in the foundation response would be proper: first, a reduction to take into account phase differences associated with shear wave travel time across the base (TR 10,152-60; App. Exs. 61, 62, 63 and 64); second, a reduction to take into account random variation in peak accelerations across the base due to non-uniformity of the soil formation under the structure. TR 10,162-166; App. Ex. 65. A reduction of 10% for each would be appropriate. Taking all of the above into account, Dr. Seed concludes that the free field ground motion can be reduced legitimately by 40% to obtain the motion at the foundation level for the DCNPP. The 20%-30% reduction (depending upon the structure) indicated in the DCNPP re-analysis reflects a very conservative approach. TR 10,167.

Drs. Luco and Trifunac presented testimony on the tau effect. Their views were similar. Both agreed that reduction of the foundation response in the high frequency range could be justified under certain circumstances. However, neither the Staff nor Applicant had demonstrated it was justified at the DCNPP site. L.B. Ex. 2-C at 3-6; Ex. 2-D at 3-4; TR 8889; 8975-78.

In Dr. Luco and Trifunac's view the tau effect is an effort to quantify the scattering and diffraction of high frequency energy by a large rigid intrusion in the soil. L.B. Exs. 2-C at 3; Ex. 2-D at 2. This effect is a function of soil characteristics, the dimension of the foundation, the rigidity of the foundation and the type of excitation. TR 8879; 8975. This scattering can be substantial where (1) soft soil conditions prevail; (2) the foundation is rigid and embedment ratio large;36/ and (3) the seismic excitation has a significant high frequency component. TR 8879. In fact, the records of the San Fernando earthquake at the Hollywood Storage building demonstrate that to be the case. TR 8947-50; 9068-70. There the soil is soft, - the shear

---

36/ I.E., the depth of the foundation compared to the foundation dimension facing the earthquake source. TR 9172.

velocity at the site is in the range of 800 ft/sec. - the foundation is deeply embedded and the seismic energy had a large high frequency component. However, at Diablo, the site is hard rock - shear wave velocity of 3500-3600 ft/sec. - and the foundation embedment is relatively shallow. Consequently, scattering and diffraction of high frequency energy (Dr. Trifunac describes the phenomenon as an impedence jump) at the DCNPP is expected to be small TR 8879; 8975.

Assuming there is some "tau effect", Drs. Luco and Trifunac conclude that the Staff and Applicant's reductions are too large and are based on unrealistic models that consider only part of the problem. L.B. Exs. 2-C at 3-6; and 2-D at 2. First, the assumptions of a rigid foundation fails to recognize deviations that lead to localized higher stresses in the lower part of the structure. L.B. Ex. 2-C at 4 and 2-D at 2.

Second the reductions are calculated on the bases of horizontally propagating shear waves. TR 8883. Given the likely orientation of the Hosgri earthquake to the site - short horizontal distance to the fault relative to the focal depth - most of the energy at the site will be in the form of vertically incident waves. TR 8880.[37/] In that case, energy scattering by foundations with a shallow embedment is

---

37/ See also, TR 10,128; 10,168.

"practically nonexistent." L.B. Ex. 2-C at 4; TR 8935. Moreover, even assuming a large component of seismic energy is in the form of horizontally incident shear waves, any reduction in shearing force (translational component of motion) is coupled with a substantial torsional force. The Staff and Applicant have discounted the torsional force. Consequently, the 20%-30% reduction reflects an exaggerated reduction of the shearing force (translational motions) while discounting the full torsional force. L.B. Exs. 2-C at 4-6; 2-D at 2; TR 8890.

The assertion that the vertical waves could produce a 20%-30% reduction, also gets a "not proven" verdict from Drs. Luco and Trifunac. First, the recordings of nuclear explosions by closely spaced instruments are not valid indicators for the situation at Diablo. The recordings are at significant distances. In that case, the nonuniformity in the geologic structures through which the waves travel can introduce phase differences recorded by closely spaced instruments. At Diablo, the earthquake is assumed to occur on a fault only 4-1/2 miles from the site. At that distance, the phase difference likely to be introduced by nonuniformity is practically insignificant. TR 8887. Second, Dr. Luco cites Dr. Seed's studies to support the conclusion that any tau effect due to vertically incident waves is practically insignificant. TR 8889; 8924-26.[38/] Third, the hard soil, and relative depth of embedment preclude any significant scattering and diffraction of high frequency energy at the DCNPP site for vertically incident waves. TR 8879; 8975.

4. Damping

A damping factor of 7% for reinforced concrete structures (including, among others, the containment building, turbine building and intake structures) was used in the reanalysis. A smaller damping value (5%) which yields larger calculated responses and is therefore more conservative, was used in the original analysis. Regulatory Guide 1.61 provides that 7% damping may be used for reinforced concrete structures.

The Staff testified that the 7% damping in Regulatory Guide 1.61 was initially based on a 1973 report co-authored by Dr. Newmark, Dr. Blume and one other engineer. The report was not introduced into evidence. Confirmation of the 7% damping figure is provided by tests on Japanese nuclear power plants which show damping in the range of 20% and by laboratory tests on reinforced concrete shear wall tests showing damping in the range of 7%-10%. TR 9820. The Staff acknowledged that the Japanese data reflect total

---

38/ J.I. Ex. 58 at 13; *supra*. fn. 46 at page 39.

building soil system damping. TR 9820. This includes radiation damping, energy dissipated at the soil-structure interface. Blume Testimony at 47; TR 6553-9; 8895-98. What is sought here is structural damping alone, energy dissipation within the structure. Further, the Staff did not know the character of the soil under the site, nor how it compared to the hard rock under the DCNPP site. The Staff could not identify any data involving tests of an existing structure where the soils are of comparable rigidity to those at Diablo Canyon. TR 9886. Nevertheless, because the total building - soil system damping is as high as 20% - the Staff concludes that 7% is a reasonable estimate of structural damping. TR 9820.

The Staff rejected Dr. Luco's view that results from reinforced masonary tests suggested that the concrete shear wall tests have been misinterpreted, but gave no specific reason for doing so. TR 9824.

In addition, Staff rejected the results of Dr. Luco's and Dr. Trifunac's report on the Miliken Library - a report the Staff had not reviewed (TR 9887) - showing damping values significantly lower than 7% for that reinforced concrete structure. TR 9823. The reasons cited by the Staff were (1) the report was a draft only and had not been subjected to peer review and (2) extrapolation from the damping values determined from very low stress created by the forced vibration test was not a valid indicator of damping obtained during high stress earthquake conditions. TR 9823. Nevertheless, the Staff accepted extrapolations from very low stress conditions created to test damping on the DCNPP piping system. TR 9845.

The Applicant presented results from various tests to support 7% damping. Blume Testimony at 46-49.

Dr. Blume testified that tests of reinforced concrete shear walls are particularly relevant because (1) much of the DCNPP complex is reinforced concrete shear wall and (2) the values are not contaminated by radiation damping. Blume Testimony at 48-9; TR 6827-30; 10,119. The tests measured damping as a function of stress levels. The data points (nine in all) are clustered at low strain levels (four points) and at high strain levels, near the yield point for steel (five points). Blume Testimony, Figure 20. Dr. Blume testified that the results suggest use of 7% damping in the reanalysis. TR 6827-30; 10,118-20.

Both Dr. Luco and Dr. Trifunac dispute the use of 7% damping in the reanalysis; both would prefer using 5%. L.B. Ex. 2-D at 3-4, 6; TR 8895; 8980. They conclude that data cited by the Applicant and Staff cannot be applied to the DCNPP with any confidence because in most cases the measurements are (1) contaminated by radiation damping or (2) are on models unlike the DCNPP structures in geometry and materials. L.B. Ex. 2-D at 6; TR 8895-6; 8980. The only exception are the tests on reinforced concrete shear walls, and those results are based on few data points, require an assumption of linear relationship between stress and damping, and are placed in question by recent results from tests on reinforced masonry. TR 8897. These last tests suggest that the damping does not increase linearly with stress, but remains low and constant until stresses equal to one-half yield. TR 8897.

Discussion

Appendix A requires that seismic design response spectra correspond ". . . to the maximum vibratory accelerations at the elevations of the foundations of the nuclear power plant structures . . ." for the SSE. Sec. VI(a), Appendix A. The Licensing Board contends that this requirement is met by the response spectra it approved. Decision at 75. We disagree.[39/]

First, the Applicant has failed to prove that the response spectra represent[40/] the maximum vibratory accelerations of a 7.5 magnitude earthquake. In fact, a strong case has been made that the response spectra represent a 6.5 magnitude earthquake. This view is held by Dr. Luco, Dr. Brune, and Dr. Trifunac. It is butressed by the values in

---

39/ J.I. Exceptions at 39-52 & 72.

40/ The discussion is directed to the Newmark spectra which established the minimum acceptable standards and was used for analysis instead of Blume's spectra in most instances. TR 6836.

Circular 672 and by the fact that the response spectra correspond to the one derived at Pacoima Dam during the San Fernando earthquake - a 6.5 magnitude earthquake. The Licensing Board either dismisses this evidence improperly or doesn't address it at all.

The Licensing Board assigns three reasons for rejecting Dr. Luco's position. None are valid. First, Dr. Luco's testimony is rejected because it is contradicted by the testimony of Drs. Blume and Newmark. Decision at 58. The fact that Dr. Luco disagrees with Drs. Blume and Newmark is not a valid reason for rejecting his conclusions or accepting theirs. The fact that experts disagree, in and of itself, is not sufficient to accept one view and reject the other.

The Licensing Board states that Dr. Luco's position is contradicted by the USGS. Decision at 58. That's not the case. Circular 672 provides descriptions of ground motion in the free field for a 7.5 magnitude earthquake. The USGS left to the engineers the question of whether those values should be modified for engineering analysis. Further, USGS took no position on the 0.75g response spectra used in the reanalysis.

Dr. Luco used the values in Circular 672 in the way that the USGS intended. He used them to describe ground motion in the free field. Then he made an independent engineering evaluation as to whether those values should be modified to account for soil structure interaction or other phenomenon - tau - that could reduce the motion at the levels of the foundation. He determined that the ground motion in the free field was not significantly modified by soil structure interaction or by a tau effect.

The Licensing Board suggests that Dr. Luco improperly relied on correlations developed by Dr. Trifunac. Decision at 59. Dr. Trifunac's correlations are to be used to estimate ground motion values for large earthquakes. L.B. Ex. 2-F. That is the purpose for which Dr. Luco used them. The fact that Dr. Trifunac would assign a 6.5 magnitude earthquake to the Hosgri fault is a separate matter altogether, and does not bear on Dr. Luco's use of the Trifunac correlations.

The Licensing Board dismissed Dr. Brune's testimony without providing an adequate reason for doing so. Decision at 61. After reciting Dr. Brune's testimony, the Board concludes that "[c]onsidering all of the evidence, the Board is of the opinion that the speculated higher values postulated by Dr. Brune are not of design or analytical significance for the Diablo Canyon Plant." Decision at 61. The

Board did not critique Dr. Brune's analytical methods. It did not identify logical gaps or inconsistencies in his reasoning. The Board did not identify data that he failed to consider. In short, the Licensing Board failed to provide a rational basis for dismissing Dr. Brune's testimony as irrelevant.

The Board briefly discusses Dr. Newmark's basis for selecting 0.75g to represent the "effective acceleration" for the Hosgri earthquake. However, the Board fails to examine the fact that on the last day of hearings, Dr. Newmark jettisoned the concept of effective acceleration. It fails to examine Dr. Newmark's conclusion that the response spectra for a 6.5 magnitude earthquake (Pacoima Dam) is selected to represent a 7.5 magnitude earthquake - or Dr. Luco's criticism of Dr. Newmark's explanation.

The Board cites the testimony of Dr. Seed and others that 0.75g represents the "actual" maximum acceleration at the site and that therefore "there is no need to introduce the concept of effective acceleration." Decision at 60. However, the Board fails to explain how this testimony is consistent with Dr. Blume's assertion that 0.75g effective acceleration "is based" on a peak instrumental acceleration of 1.15g for a 7.5 magnitude earthquake. Furthermore, the Board provides no rational basis for accepting .75g (Newmark, Seed, Frazier, Bolt, Stepp, Hoffman, Blume) which rejecting 1g (Brune, Trifunac, Luco, USGS) to represent "actual" maximum accelerations for a 7.5 magnitude earthquake. Decision at 57.

Dr. Blume's probabilistic studies are cited. Dr. Luco and Dr. Trifunac's criticism of those studies are neither cited or discussed. Decision at 60.

Second, even if the above problems are ignored, and the Newmark spectra (without tau) were acceptable, neither the Applicant nor Staff have demonstrated that a tau reduction is appropriate at the DCNPP site. Dr. Luco and Dr. Trifunac testified that the tau reduction is not appropriate for the DCNPP site. The Licensing Board dismisses Dr. Luco's testimony for reasons that are not valid, Decision at 72. (See, discussion of evidence, supra, at 42-47). Moreover, the Licensing Board doesn't discuss Dr. Trifunac's views at all.

Finally, we turn to damping,[41/] here again, the Licensing Board's decision is devoid of a fair explication of the evidence. The Licensing Board cites mostly general concepts, and fails to discuss the views of Trifunac and Luco.

41/ J.I. Exceptions at 53.

In sum, the evidence in this case does not demonstrate that

(1) the Newmark spectra represent the maximum vibratory accelerations at the DCNPP site for a 7.5 magnitude earthquake on the Hosgri fault;

(2) tau reductions in the range of 20%-30% at the DCNPP site are appropriate and conservative;

(3) 7% damping values are appropriate and conservative.

D. THE LICENSING BOARD ERRED IN FINDING THAT THE APPLICANT'S PROGRAM OF TESTING AND ANALYSIS DEMONSTRATED THAT STRUCTURES, SYSTEMS AND COMPONENTS NECESSARY TO ACHIEVE SAFE SHUTDOWN AND TO MAINTAINING A SAFE SHUTDOWN CONDITION WILL PERFORM THEIR SAFETY FUNCTIONS DURING THE HOSGRI EARTHQUAKE AND AFTERSHOCKS

Introductory Statement

Appendix A requires the Applicant to demonstrate, through analysis and testing, that the structures, systems and components necessary to achieve safe shutdown and to maintain the plant in safe shutdown will perform their safety functions during the Hosgri earthquake. Sec. VI(a), Appendix A. In addition, where the seismic analysis indicates stresses beyond the yield point in safety related structures, systems and components, Appendix A requires the Applicant to demonstrate that the yielding does not affect the performance of necessary safety functions. Id.

The Licensing Board finds that these requirements have been met. Decision at 92-3. The Joint Intervenors disagree. J.I. Exceptions at 54-62. First, the Staff has permitted deviations from standard testing and analytical procedures or has waived regulatory requirements without demonstrating that (a) the substitute procedures provide an adequate margin of safety or (b) the waived requirements have no applicability to safety determinations at the DCNPP.

Second, where as here, the response spectra are legally deficient, the whole scheme of testing and analysis falls short of the Commission's legal requirements. Assurances that the plant is safe because (a) the seismic review was the most extensive ever undertaken by either the Applicant or Staff (Decision at 92) or (b) conservatism is factored into the seismic analysis by standard engineering procedure (Decision at 75-78) cannot cure the basic legal defect: the response spectra do not meet the regulations.

Discussion

1. Deviations From Normal Procedures

Procedures normally used in the design of nuclear plant structures, systems and components that provide conservatism in design were not applied by the Staff in the DCNPP seismic reevaluation. In Supplement 7 of the SER, the Staff acknowledges that

> [t]he generic methods of analysis used by the Applicant in the seismic reevaluation as outlined above contain three significant relaxations relative to the normal, or currently accepted, procedures. One relaxation is reduction of ground response spectra to account for building size effects. [Tau effect] The second is use of actual material strengths rather than code specified minimum material strengths. The third is allowance for ductility in structures which might be used in two specific cases and specifically justified. (SER, Supp. 7 at 3-22 and 3-23)

The technical basis for the "tau effect" reduction was discussed in the previous section. In addition to use of actual material strengths and allowance for ductility, the Staff departed from normal practice by permitting loads to be combined in a manner different from that specified in Regulatory Guide 1.92. We turn to examine these three items below.

10 CFR §50.55(a) requires that nuclear power plant structures, systems and components meet the requirements of professional engineering codes identified by the regulations. Exceptions are permitted (a) where compliance would result in hardship or unusual difficulties without a compensating increase in the level of safety or (b) alternative specifications will provide an acceptable level of quality and safety. Instead of using the values specified in the applicable engineering codes, the Applicant used the <u>average values</u> of material properties for concrete and steel determined from tests. The original analysis was based on code specified values. SER, Supp. 7, at 3-20. The relaxation is approximately a 13% to 27% factor for concrete and an 11% to 24% factor for steel. TR 7194.

Neither the American Concrete Institute Standard Building Code nor the specifications of the American Institute of Steel Construction contemplates the use of average actual material properties. Both specify using minimum specified material strengths for concrete and steel. TR 6944-6945. The Applicant's justification for the departure from standard code practices is summarized in Section 4.1.2, and in Appendix D-LL6 and D-LL21 of the Hosgri Seismic Reevaluation. While it is true that concrete strength increases with time, use of the "average actual value" means that one-half are not expected to fall below the computed average.[42/] The averaging effect is potentially even more misleading for structural steel where a small number of samples are used.

The use of average actual steel and concrete values represents a departure from normal Staff practice. Furthermore, the evidence offered by the Staff and Applicant fails to justify an exemption from the requirements to 10 CFR §50.55(a).

In another departure from normal practice, the Staff permitted an allowance for ductility in testing and analysis that is permitted for stresses or strains beyond the material yield point. In the case of structural analysis, the reanalysis indicated stresses beyond the yield point of the material in (a) the curtain wall of the intake structure;

42/ The Licensing Board's finding that "actual material values were used in the reevaluation is wrong (J.I. Exceptions at 60). Average actual material values were used. TR 7142.

(b) the turbine building (when the crane is parked at either end); (c) certain piers beneath the main turbine generators. Hoch Testimony at 21, 22; TR 6917. For the equipment, analysis indicated stresses or strains beyond the material yield point at (a) locations within the piping system; (b) unspecified pumps and valves; and (c) in the fuel grid. TR 6919-6921; Bucker-Esselman Testimony at 1-7; TR 7661-79; Esselman Testimony at 1-8; TR 7549-86.[43/]

Appendix A permits stresses and strains beyond the yield point, "some" safety related structures, systems and components only where the safety functions are not impaired. Sec. VI(a), Appendix A. With respect to the list, above, the Applicant asserts that (a) structural deformations have been carefully evaluated to assure that all necessary safety functions are maintained, Hoch Testimony at 21, 22; TR 6917; (b) special criteria were developed and applied to assure that material yielding would not impair the safety functions of valves and pumps, TR 6919-21; modification to the piping system - strengthening of the supports and addition of snubbers (equipment designed to minimize pipe vibration) - assures its safety, TR 7679; analysis of the core assures

---

43/ The Licensing Board finds that loads from the seismic event and the worst postulated loss of coolant accident - individually and combined - are below the allowable grid strength. J.I. Ex. at 62. However, if asymmetric loading is considered, the grids are expected to deform. Esselman Testimony at 3-7.

that an adequate cooling will be provided in the event of fuel grid damage. Esselman Testimony at 5-7; TR 7571-76.

The fact that the above analyses were performed is not disputed. Our concern is whether they were adequate. Proof that a piece of equipment, although damaged, can perform its safety function is not a run-of-the-mill task in licensing proceedings. For that reason, such proof requires a degree of scrutiny altogether missing from the Licensing Board's examination of the evidence.

For example, the Licensing Board did not examine the problem of aftershocks. Appendix A expressly requires consideration of aftershocks. Sec. VI(a), Appendix A. The aftershock following a 7.5 magnitude earthquake is expected to be large. As Dr. Newmark testified, a principal reason to require that structural response remain in the elastic range (i.e. that materials do not yield) is to assure that the plant can safely ride out aftershocks. Newmark Testimony at 5. That margin of safety has been admittedly eliminated for the components, structures and systems identified above. In addition, both Dr. Trifunac and Dr. Luco believe that the structural response will be inelastic for the Hosgri earthquake (for either 1.15g or .75g). TR 8867-71; 8873; 8894; 9263. Damage significantly greater than predicted may occur. In view of that fact and the admission that damage is likely to occur to certain parts of the plant, the problem of aftershocks merits significantly more scrutiny than that given by the Licensing Board.

The Licensing Board failed to address Task Action Plan - B-51, which states that when inelastic analytical procedures are used, proper qualification of the methodology and complete understanding of its limitations are important. In addition, the Licensing Board failed to address the deficiencies in the component design criteria of the ASME Codes - NUREG-0471, Generic Task Problem Description, Category B, C, and D Tasks; Staff Exhibit No. 12, and in what manner those deficiencies, in combination with the relaxations mentioned above, affect safety margins.

Finally, the Licensing Board failed to address in what manner the cumulative effect of these departures from normal Staff practice and design requirements affect safety margins. Such analysis is fully justified under the exceptional circumstances of this case. Specifically, the interaction between systems judged most likely to fail during the SSE and important safety related systems should be analyzed to determine whether, and, if so, how, a failed system might impair the safety functions of another. A rigorous study of this kind would provide some of the margin of safety reduced by the analytical and testing techniques employed in the reanalysis. In addition, operator procedures required to cope with system failures judged likely to occur during an earthquake should be studied. Adequate completion of both and adjudication of the conclusions drawn should be required before licensing.

The Joint Intervenors are aware that the Staff is requiring the Applicant to conduct such studies. The TMI-2 accident provided the necessary impetus. The point, however, is that the evidence in this record - taken before the TMI-2 accident - was sufficient to alert a vigilent Licensing Board to the need for these studies. The fact that the Licensing Board failed to act on that evidence and order such studies is another indicator of the deficiency of its decision.

2. Other Regulatory Requirements Waived

For the electrical equipment that must be requalified, the Applicant, at the Staff's request, committed to employ seismic qualification methods that conform to current Staff criteria. (Regulatory Guide 1.100, Revision 1, "Seismic Qualification of Electrical Equipment for Nuclear Power Plants," and IEEE Standard 344-1975, "IEEE Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Generating Station;" SER, Supp. 7 at 3-71). This testing is deficient because the effects of aging have not been considered in the seismic qualification of electrical equipment. The Applicant admits that the seismic testing program does not include the aging requirement as described in IEEE 323-1974. TR 7691. At best, the tests performed to date by the Applicant indicate that new equipment could withstand the accident environment.

The methods used in combining seismic stresses with normal operating loads and stresses for the piping systems were not in accord with the method in Regulatory Guide 1.92. TR 7593. In the original seismic analysis of piping systems, two different seismic load cases were analyzed, each case representing responses due to a combination of a vertical component and one of the two orthogonal horizontal components of the earthquake. The higher response of the two load cases was compared against the allowable response to assure adequacy of the system. In contrast, the current Staff Regulatory Guide 1.92 requires that all three components shall be considered to act on the system simultaneously.

At the request of the Staff, a study was performed by the Applicant to evaluate the differences in the piping system responses when the two different analytical approaches discussed above were used. SER, Supp. 7 at 3-69. The responses at some locations on the systems increased; at other locations they decreased. The sensitivity of the results to the analysis demonstrates that a departure from the methods specified in Regulatory Guide 1.92 is not merited.

The Applicant has yet to identify in the FSAR its amendments, or in testimony offered into evidence, each and every Category 1 44/ structure, system and component.

To begin with, the Applicant has provided only a general list of Category I structures, systems, and components in Section 3.2 of the FSAR and in the equipment lists provided as part of the Hosgri Seismic Reevaluation documented in the FSAR Amendment 50. The Applicant states that "their seismic design classifications comply with the <u>intent</u> of Safety Guide 29." (emphasis added) Safety Guide 29 is the predecessor to Regulatory Guide 1.29. Later in the FSAR, the Applicant acknowledges in a footnote that the Spent Fuel Pool Cooling System at Diablo Canyon is classified Design Class II, which differs from Regulation Guide 1.29 which indicates that the system should be Class I. FSAR, page 3.2-2.

44/ As set forth in Regulatory Guide 1.29, Revision 3, September 1978; Category I structures, systems and components are those necessary to assure

> (i) the integrity of the reactor coolant pressure boundary, (ii) the capability to shut down the reactor and maintain it in a safe condition, or (iii) the capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of this part

Another incomplete list of equipment to be seismically qualified as Category I is tabulated in FSAR Amendment 50. In particular, the listing of Category I electrical items, Table 10-1, is cursory and inadequate for independent analysis. For instance, the Applicant fails to mention all the locally mounted, rack mounted, and panel mounted instruments.

Without a rigorous listing of all Category I structures, systems and components, independent evaluation of the Applicant's seismic qualification program is precluded. There is no way to assure that all structures, systems and components required by the regulations to be seismically qualified have, in fact, been qualified. The Licensing Board failed to demand such a list. Decision at 92-3.

3. The Response Spectra

The process of testing and analysis begins with the design criteria discussed in Section C of this brief - the response spectra that define the earthquake forces at the foundation level. These spectra are used to define the stresses used for testing and analysis.[45/] The modifications

---

45/ From these response spectra, a time history of ground motion is developed, one that would reproduce the response spectra. The time history is used to excite the various locations in the structure where equipment is mounted. The result is a family of floor response spectra that define stresses for testing and analysis. Pursuant to Appendix A, these seismic stresses must then be combined with other stresses which would be caused by normal operation, and, where appropriate, stresses caused by accident loads. The total stress is then compared to the stress criteria in various codes, where stresses exceed the criteria, additional analysis is performed to show that safety functions are not affected. Gormly Testimony 2-5; TR 9830-38.

that have been factored into the response spectra - and are the center of dispute - are, in turn, factored into all subsequent testing and analysis. These modifications result in substantial reduction in the input[46/] to testing and analysis of systems and components. TR 8893-5; 9002-07.

46/ In all instances below, the 1.15g "peak instrumental acceleration" for the 7.5 magnitude earthquake was first reduced to 0.75g "effective acceleration."

- Containment structure - the Blume spectra were reduced from .75g effective acceleration to .67g for the "tau effect," a reduction of approximately 10%; the Newmark spectra from .75g effective acceleration to .60g, a reduction of approximately 20%. (TR 7014-7016). The reduction for tau effect at the fundamental mode for the containment structure is approximately 12%. The reduction at the zero period is approximately 11%. (TR 7170).

- Turbine building - the Blume spectra were reduced from .75g effective acceleration to .54g to reflect the "tau effect," a reduction of approximately 28%; the Newmark spectra, from .75g effective acceleration to .50g, a reduction of 33%. (TR 7182-7183). The tau reduction in the north-south direction at the fundamental mode would be about 17%. (TR 7185).

- The auxiliary building - the Blume spectra was reduced from .75g effective acceleration to .63g, a reduction of approximately .16%. The reduction for the Newmark spectra for tau effect was from .75g to .55g, a reduction of approximately 27%. (TR 7131-7132). There is some tau reduction for the concrete portion of the auxiliary building at the fundamental mode. (TR 7172-7173).

- The intake structure - the Blume spectra were reduced from .75g effective acceleration to .67g for the tau effect, a reduction of 10%. The adjustment for "tau" under the Newmark spectra was from .75g to .60g, a reduction of 20%. The reduction for tau at the fundamental mode would be similar to that of the zero period acceleration. (TR 7225-7227).

The increase in the damping factor to 7% for reinforced concrete (from 5% in the original analysis) structures significantly reduces the response spectra. Newmark Testimony at Figures 8-13.

The Applicant and the Staff testified that (a) conservatisms factored into the seismic analysis by standard engineering procedures, Knight Testimony at 1-5; and (b) the extent of the review provide all the assurance of safety that is necessary. The Licensing Board agrees with that position. Decision at 75-78; 92.

This view ignores a central tenet of nuclear power plant licensing. Nuclear reactors are not licensed unless they meet the Commission regulations. They cannot be licensed on the basis that, although an applicable regulation is not met, the public health and safety will still be protected. For as the Appeal Board has noted, "once a regulation is adopted, the standards it embodies represent the Commission's definition of what is required to protect the public health and safety." ALAB-138. <u>Vermont Yankee Nuclear Power Corp</u>. (Vermont Yankee Station), ALAB-194, 8 AEC 431, 435 (1974). The response spectra do not comply with the Commission's regulations. That defect is not cured by Staff and Applicant assurances that the plant is safe anyway.

E. THE LICENSING BOARD ERRED IN FINDING THAT THE USE OF AN OPERATING BASIS EARTHQUAKE OF 0.20g MEETS REGULATORY REQUIREMENTS

Introductory Statement

Appendix A establishes two levels of seismic design: the Safe Shutdown Earthquake ("SSE") and the Operating Basis Earthquake ("OBE"). Previous sections discussed the SSE (Hosgri earthquake) criteria for the DCNPP. This section discusses the OBE defined by Appendix A as follows:

> "that earthquake which, considering the regional and local geology and seismology and specific characteristics of local subsurface material, could reasonably be expected to affect the plant site during the operating life of the plant; it is that earthquake which produces the vibratory ground motion for which those features of the nuclear power plant necessary for continued operation without undue risk to the health and safety of the public are designed to remain functional."

Sec. III(d), Appendix A.

Appendix A further states that -

> [T]he maximum vibratory ground acceleration of the Operating Basis Earthquake <u>shall be at least one-half the maximum vibratory ground acceleration</u> of the Safe Shutdown Earthquake. (emphasis added)

Sec. V(a)(2), Appendix A.

The OBE adopted by the Applicant and approved by the NRC Staff is 0.20g.[47/] SER, Supp. 7 at 2-3. This is less

---

47/ The value assigned to the OBE for the Hosgri reanalysis - 0.20g - is the same as the original OBE (design earthquake). It was taken as one-half the original SSE (double design earthquake) - 0.40g. "Design Earthquake" and "double design earthquake" were the terminology used for "OBE" and "SSE" before adoption of Appendix A.

than one-half the effective ground acceleration of 0.75g adopted by the Staff as the SSE. SER, Supp. 7 at 2-4. The Licensing Board found that "use of an operating basis earthquake of 0.2g is reasonable for the Diablo Canyon facility." Decision at 65. The Joint Intervenors have a different view. Our's is that use of an operating basis earthquake of 0.20g violates the regulatory requirements. J.I. Exceptions at 63-70.

Discussion

The Licensing Board assigns several reasons for its decision. First, it concludes that "the OBE requirement (the OBE shall be at least one-half (1/2) the SSE) was intended to apply to the original design basis at the construction permit stage, and is not necessarily applicable to the instant case." Decision at 63. The basis for this conclusion is the Statement of Consideration addressing the Section V(a)(2) requirement "together with the total text of Appendix A." Decision at 63. The Statement of Consideration reads as follows:

> Paragraph (a)(2) of Section V has been changed to require the Applicant to specify the Operating Basis Earthquake. A requirement which reflects the seismic design bases for plants recently evaluated for construction permits that the maximum vibratory ground acceleration of the

> Operating Basis Earthquake shall be at least one-half the maximum vibratory ground acceleration of the Safe Shutdown Earthquake has been added.48/

There is nothing in the Statement of Consideration that supports the Licensing Board's reasoning. Design requirements are normally established prior to construction, at the construction permit stage. The language that the OBE requirement "reflects the seismic design basis for plants recently evaluated for construction permits . . ." refers to that fact. Had the Commission intended to base the OBE requirement only on those facts known at the CP stage - as the Licensing Board suggests - it would have said so. Further, limiting the OBE requirement to only those facts known at the CP stage is not conservative. Appendix A is required to be interpreted conservatively. ALAB-561, supra. (Slip. Op. at 32, Mr. Farrar dissenting); see also, 42 Fed. Reg. 2051 (Appendix A is amended to make explicit that earthquakes larger than those in the historical record may be selected as the SSE).

Finally, the Licensing Board gives no hint as to why "the total text of Appendix A" leads it to believe that the Commission intended to base the OBE requirement only on

---

48/ The Joint Intervenors excepted to the Licensing Board's statement that the revision to Appendix A incorporating the OBE requirement was issued September 1, 1978. J.I. Ex. at 64. In fact, it was included in Appendix A as originally adopted by the Commission. The proposed Appendix was published, with the Statement of Consideration on November 13, 1973 and became effective December 13, 1973. 39 Fed. Reg. 31279.

those facts known at the CP stage. In sum, its interpretation should be rejected. It strains the language of the Statement of Consideration, has no basis in any other part of Appendix A, and violates the requirement that Appendix A be interpreted conservatively.

The Licensing Board argues that the OBE requirement is a "guideline for prudent design rather than a non-clad necessity for Regulatory approval." Decision at 63-4. Designating the OBE as one-half the SSE is an arbitrary requirement - based on past Staff practice rather than a rigorous engineering evaluation. Decision at 63. For that reason, a flexible approach is appropriate.

There are two problems with this line of argument. First, the Licensing Board fails to explain why, on the one hand, the OBE requirement is arbitrary, while on the other the Staff's probabilistic criteria are not. If it is arbitrary to select an OBE equal to one-half the SSE, why is it any less so to establish the OBE as an earthquake with a return period of no less than approximately 110 years? Why not 300 years? Why not 1000? The Licensing Board fails to address that question.

Equally important, the Licensing Board has overlooked a fundamental rule of statutory construction: use of the word "shall" in the text indicates a mandatory intent unless convincing arguments to the contrary can be made. C. SANDS, SUTHERLAND'S STATUTORY CONSTRUCTION §25.04 (4th ed. 1973). The Licensing Board's arguments that the OBE requirement is discretionary is not convincing; it is not supported by either the Statement of Consideration or the text of Appendix A.

The Licensing Board argues that establishing an OBE less than one-half (1/2) the SSE on the basis of probabilistic estimates is consistent with past Staff practice and regulatory requirements. Appendix A defines OBE as an earthquake which could reasonably be expected to affect the plant site during the operating life of the plant. Sec. III(d), Appendix A. The Staff considers that an earthquake with an exceedance probability no greater than 30% and a return time less than approximately 110 years qualifies as the OBE. Decision at 64. PG&E introduced studies concluding that the lowest average return time for a peak acceleration of 0.20g at the site is 275 years; the corresponding exceedance probability for a forty (40) year plant lifetime, approximately 14.5%. Hoch Testimony at 11.

This approach raises several problems. First, the relevance of past Staff practice to DCNPP is not established. The record in this case requires that we speculate as to the values assigned to the OBE in cases where it is less than one-half the SSE. The Staff and Applicant assert that such plants exist, but the OBE/SSE values were not provided. It is quite possible that the OBE for these plants, if indeed they are less than one-half (1/2) the SSE, might be extremely close to being one-half (1/2) the SSE. TR at 6896; 6905. For example, for a plant with an SSE equal to 0.25g, OBE might be .10g; instead of .12g. That example would appear to have little bearing on this case - where the OBE is .20g and one-half the SSE is .375g. In addition, plants assigned an OBE less than one-half the SSE may all be located in zones of low to moderate seismic risk. That information is not in the record. The evidence in the record suggests strongly that at West Coast sites - other than Diablo - Staff practice is to require an OBE equal to one-half the SSE.[49/] Hubbard Testimony (OBE) at 5. In short, without more information, the relevance of past Staff practice to the case here is not established.

49/ Examples provided are:

	SSE	OBE
San Onofre 1	0.50g	0.25g
San Onofre 2	0.67g	0.33g
Trojan	0.25g	0.15g

Source: Hubbard Testimony (OBE) at 5.

Second, the Staff states that an exceedance probability of no more than 30% and a return period of no less than approximately 110 years are acceptable criteria for the OBE. The Licensing Board agrees. Decision at 64. However, no rational basis is provided for those criteria. Neither the regulations, the regulatory guides, nor the NRC Standard Review Plan provides for or endorses the use of these criteria - exceedance probabilities and average return periods - for determining the OBE. TR 6905.[50/] Internal documents recently made public indicate a different Staff view: that view is that a recurrence interval to the OBE is in the range of 300 to 1,000 years.

Finally, the Licensing Board accepts the Applicant's estimates of recurrence intervals and exceedance probabilities and fails to discuss evidence that (1) the Applicant's methodology is faulty and (2) better estimates are available. The Applicant's probabilistic studies were strongly criticized by Drs. Luco and Trifunac. L.B. Exs. 2-C at 8-12 and 20 at 4; TR 8926-7;9008-09. Dr. Trifunac's probability studies were introduced into evidence. L.B. Ex. 2-F & J. For 0.20g, Dr. Trifunac estimates an exceedance probability of approximately .30g for a 40-year plant lifetime and a recurrence interval of 110 years. L.B. Ex. 2-F.

50/ SECY-79-300, Attachment F (Memorandum from Robert B. Minogue, Director, Office of Standards Development to Commissioner Marson, NRC, 2 (October 8, 1976)).

The Licensing Board finds that "the safety of the plant and components is measured against codes which exceed the lower OBE values," and then concludes that the safety of the plant is not controlled by the OBE, but by various codes. Decision at 65. The Licensing Board's reference to the testimony and conclusions that it draws is misleading. The OBE value assigned to the DCNPP is of safety significance for several reasons.

First, the OBE, not the SSE, may control the level of design for safety related features of the plant. Appendix A requires the plant to be capable of safe operation after the OBE.51/ The SSE requirement is different. It is that the plant be capable of safe shutdown, even though it may be severely damaged and lost as a power generating facility.52/

51/ Section V(a)(2), Appendix A states -

> (2) Operating Basis Earthquake. The Operating Basis Earthquake shall be defined by response spectra. All structures, systems, and components of the nuclear power plant necessary for continued operation without undue risk to the health and safety of the public shall be designed to remain functional and within applicable stress and deformation limits when subjected to the effects of the vibratory motion of the Operating Basis Earthquake in combination with normal operating loads.

52/ Section V(a)(1), Appendix A states, in part -

> The nuclear power plant shall be designed so that, if the Safe Shutdown Earthquake occurs, certain structures, systems, and components will remain functional. These structures, systems, and components are those necessary to assure (i) the integrity of the reactor coolant pressure boundary, (ii) the capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of this part.

Put another way, OBE criteria must ensure that the plant suffers no damage that would bar it from restarting and operating safely immediately following the OBE.

Thus, while the shear and deformation limits permitted by the various codes for the SSE will be higher absolutely than those permitted for the OBE, TR 8700, relative to the stresses developed in the plant for the OBE, the lower allowable stresses required to ensure continued safe operation may dictate the level of design. For example, the Applicant testified that design for some of the plant piping is controlled by the OBE. TR 7692;8700.

Second, the value of the OBE utilized in the seismic test program by the Applicant is of safety significance. (Joint Intervenors' Exhibit No. 65, at 4-9) For example, IEEE Standard 344-1975 provides that testing for design verification of safety related electrical equipment must meet the following requirements:

> 6.1.4 OBE TESTS. Seismic qualification tests on Class 1E equipment designed to show adequacy of performance during and following an SSE must be preceded by one or more OBE tests. The number of tests shall be justified for each site or shall produce the equivalent effect of 5 OBE's. The purpose of this is to show that the lower intensity earthquake (which has a high probability of occurrence) will neither adversely affect an equipment's performance of its safety function nor cause any condition to exist which, if undetected, would cause failure of such performance during a subsequent SSE. These tests may also provide a part of the aging requirement of IEEE Std 323-1974, IEEE Standard for Qualifying Class 1E Equipment for Nuclear Power Generating Stations.

(Joint Intervenors' Exhibit No. 65, at 4-9.

The Licensing Board found that an OBE equal to or greater than 50% of the SSE was used to test plant electrical equipment for the Hosgri. Decision at 65. That finding is important but addresses only a small part of the overall problem. The Licensing Board failed to determine whether the OBE value has safety significance in any other verification programs, and, if so, what value was used.

Third, non-safety related equipment - designed to the OBE - and safety related equipment - designed to the SSE - may interact in a manner that impairs the plant operator's ability to shutdown the plant safely. In other words, failure in systems designed to the OBE could jeopardize plant safety.

Another problem, the Licensing Board decision glosses over concerns regarding the vertical acceleration values used in the OBE analysis. Instead of performing a dynamic analysis - where the vertical acceleration increases as a function of elevation, the Applicant used constant acceleration values for the containment structure, TR 7041, the auxiliary building, TR 7134, the intake structure, TR 7227; 7228, the turbine building, TR 7196, the outdoor water storage tanks, TR 7304; 7305, and the buried tanks, TR 7335. For some components of the DCNPP, the vertical acceleration was taken as two-thirds of ground acceleration for the OBE.

For some of the piping, the vertical spectrum that was used was two-thirds of the floor acceleration. TR 7672; 7674. A dynamic vertical analysis could result in load calculations greater than those obtained using a constant acceleration by factors of two to six. Hubbard Testimony. J.I. Ex. 65 at 4-10.

The Licensing Board concludes that setting the OBE at 0.2g, rather than a higher level, will require PG&E to shut the plant down for inspection at a lower acceleration than otherwise, thereby adding a further safety feature. Decision at 65. However, the Licensing Board fails to address the problem that adequate post-OBE inspection plans have yet to be developed.

The current requirements for a post-OBE inspection are stated in Section 3.7.4.II.4 of the Standard Review Plan. Since neither the regulations nor the Standard Review Plan provides details on the extent of such inspections, generic safety program Task Action Plan B-50 (TAP B-50) is applicable to the Diablo Canyon OBE assessment.

Neither the FSAR nor the Staff in the SER has identified the plan and schedule for the required research and development on post-OBE inspection that is the subject of TAP B-50. The SER also fails to discuss TAP-49, "Inservice Inspection Criteria and Corrosion Prevention Criteria for

Containments," which addresses the fact that detailed and comprehensive criteria need to be developed for performing inservice inspections of all types of containment. Hubbard Testimony (OBE) at 10-11.

In conclusion, use of an OBE equal to 0.20g violates the Commission's regulations. Appendix A expressly provides for selection of an OBE equivalent to at least one-half (1/2) of the SSE. Exempting nuclear plants from this requirement may be justified in seismically inactive zones, because the regulations provide that:

> [i]f an applicant believes that the particular seismology and geology of a site indicate that some of these criteria or portions thereof, need not be satisfied, the special sections of these criteria should be identified in the license application and supporting data to justify clearly such departures should be presented. 10 CFR Part 100 Appendix A(II)

However, where a nuclear plant is located in an area of high seismicity, as is the case with DCNPP, an exemption to the regulatory requirements is not justified. Indeed, Appendix A specifically provides that "[a]dditional investigation and/or <u>more conservative determinations</u> than those included in these criteria may be required for sites located in areas having <u>complex geology or in areas of high seismicity</u>." (Emphasis added) Sec. II, Appendix A. Thus, if anything, perhaps an OBE greater than one-half the SSE should be required for the DCNPP.

Finally, it bears to repeat here a point made earlier in the brief.[53/] The Commission regulations are the definition of what is required to protect the public health and safety. ALAB-138 at 528. It is not sufficient for the Staff to assure us that their probability standards for determining OBE will make the plant safe enough. Reactors may not be licensed unless they comply with the Commission standards. ALAB-138 at 529. In this case the Staff and Applicant have substituted their own standard for the Commission's. ALAB-194 at 445. For sanctioning that action, the Licensing Board's decision should be vacated.

53/ *Supra*. at 68.

CONCLUSION

WHEREFORE, for all the reasons cited above, the Joint Intervenors respectfully request the Appeal Board to VACATE the decision of the Licensing Board.

The Joint Intervenors further request the Appeal Board to ORDER additional analyses to determine more precisely the nature of structural and equipment response to a postulated 7.5 magnitude earthquake on the Hosgri fault. To that end the Applicant, Staff and Joint Intervenors should be directed to meet with Drs. Trifunac and Dr. Luco to discuss the scope and nature of such studies and report to the Appeal Board.

The Joint Intervenors further request the Appeal Board to ORDER that studies on systems interaction during an earthquake and operator procedures to shutdown the DCNPP safely during an earthquake be undertaken, completed and subject to hearing prior to issuance of an operating license.

Additionally, the Joint Intervenors request the Appeal Board to hear oral argument on the issues discussed herein.

Respectfully submitted,

David S. Fleischaker, Esq.
1735 Eye Street, N.W.
Suite 709
Washington, D.C. 20006
(202) 638-6070

John R. Phillips, Esq.
CENTER FOR LAW IN
THE PUBLIC INTEREST
10203 Santa Monica Boulevard
Los Angeles, California 90067
(213) 879-5588

Attorneys For Joint Intervenors
SCENIC SHORELINE PRESERVATION
CONFERENCE, INC.
SAN LUIS OBISPO MOTHERS FOR PEACE
ECOLOGY ACTION CLUB
SANDRA A. SILVER
GORDON SILVER
JOHN J. FORSTER
ELIZABETH APFELBERG

DECEMBER 7, 1979

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

DOCKETED USNRC DEC 14 1979 Office of the Secretary Docketing & Service Section

In the Matter of: PACIFIC GAS & ELECTRIC COMPANY (Diablo Canyon Nuclear Power Plant, Units 1 & 2)	Docket Nos. 50-275 O.L. 50-323 O.L.

CERTIFICATE OF SERVICE

I hereby certify that on this 7th day of December, 1979, I have served copies of the foregoing JOINT INTERVENORS' BRIEF IN SUPPORT OF EXCEPTIONS, mailing them through the U.S. mails, first-class, postage prepaid.

Elizabeth S. Bowers, Esq.
Chairman
Atomic Safety & Licensing Board
U.S. Nuclear Regulatory Commission
Mail Drop East West 450
Washington, D.C. 20555

Mr. Glenn O. Bright
Atomic Safety & Licensing Board
U.S. Nuclear Regulatory Commission
Mail Drop East West 450
Washington, D.C. 20555

Docket & Service Section
Office of the Secretary
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Dr. William E. Martin
Battelle Memorial Institute
Columbus, Ohio 43201

James R. Tourtellotte, Esq.
L. Dow Davis, Esq.
Marc R. Staenberg, Esq.
Edward G. Ketchen, Esq.
Office of the Executive Legal Director - BETH 042
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Mrs. Elizabeth Apfelberg
1415 Cozadero
San Luis Obispo, CA 93401

Mr. Frederick Eissler
Scenic Shoreline Preservation Conference, Inc.
4623 More Mesa Drive
Santa Barbara, CA 93105

Sandra A. Silver
1760 Alisal Street
San Luis Obispo, CA 93401

Gordon Silver
1760 Alisal Street
San Luis Obispo, CA 93401

John Phillips, Esq.
Center For Law In The
Public Interest
10203 Santa Monica Boulevard
Fifth Floor
Los Angeles, CA 90067

Bruce Norton, Esq.
3216 N. Third Street
Suite 202
Phoenix, Arizona 85012

Mr. Yale I. Jones, Esq.
100 Van Ness Avenue
19th Floor
San Francisco, CA 94102

Andrew Baldwin, Esq.
Friends of the Earth
124 Spear Street
San Francisco, CA 94105

Paul C. Valentine, Esq.
321 Lytton Avenue
Palo Alto, CA 94302

Janice E. Kerr, Esq.
Lawrence Q. Garcia, Esq.
J. Calvin Simpson, Esq.
California Public Utilities
Commission
5246 State Building
350 McAllister Street
San Francisco, CA 94102

Mr. Richard S. Salzman,
Chairman
Mr. Alan S. Rosenthal
Dr. W. Reed Johnson
Atomic Safety & Licensing
Appeal Panel
U.S. Nuclear Regulatory
Commission
4350 East West Highway
Bethesda, Maryland 20014

John C. Morrissey, Esq.
Philip A. Crane, Esq.
Pacific Gas & Electric Company
77 Beale Street, Room 3127
San Francisco, CA 94106

Arthur C. Gehr, Esq.
Snell & Wilmer
3100 Valley Center
Phoenix, Arizona 85073

Mrs. Raye Fleming
1920 Mattie Road
Shell Beach, CA 93449

MHB Technical Associates
1723 Hamilton Avenue
Suite K
San Jose, CA 95125

Carl Neiburger
Telegram Tribune
P.O. Box 112
San Luis Obispo, CA 93402

J. Anthony Kline, Esq.
Legal Affairs Secretary to
the Governor
State Capitol Building
Sacramento, California 95814

Herbert H. Brown, Esq.
Hill, Christopher & Phillips
1900 M Street, N.W.
Washington, D.C. 20036

David S Fleischaker

David S. Fleischaker, Esq.