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

November 5, 1981

MEMORANDUM FOR: Chairman Palladino
Commissioner Gilinsky
Commissioner Bradford
Commissioner Ahearne
Commissioner Roberts

FROM: Forrest J. Remick

SUBJECT: ANALYSIS OF ALAB-644: DIABLO CANYON SEISMIC ISSUES

OPE has prepared an analysis of ALAB-644 on Diablo Canyon seismic issues as an aid to the Commission in deciding whether or not to take review of ALAB-644. Our analysis addresses the major technical issues, including magnitude saturation, the Pacoima Dam record, effective acceleration, and the tau effect. This memorandum provides a brief introduction to and synopsis of the OPE analysis of the record on each of the issues, and it provides our overall conclusions. A brief list of technical terms used in our analysis is appended to this memorandum. The record on each issue is analyzed in Enclosure 1. Because much of the controversy on the record relates to NRC staff practice regarding seismology and seismic engineering, we have provided a brief summary of that practice as Enclosure 2. An overview of the general methodology of seismic design is given in Enclosure 3. In Enclosure 4 we present a list of the witnesses cited in our Enclosure 1 analysis. For certain key witnesses, namely, Drs. Newmark, Blume, Trifunac and Luco, we provide some discussion of their qualifications and roles in the proceeding.

As requested by the Commission, we have utilized the services of two consultants, Dr. Robin McGuire of Ertec Rocky Mountain, Inc., and Dr. S.T. (Ted) Algermissen of the U.S. Geological Survey (USGS). Dr. McGuire is an expert in both seismology and seismic engineering, and Dr. Algermissen is an expert on various aspects of seismic risks. As discussed in our memo to you of September 22, 1981, the roles of Dr. McGuire and Dr. Algermissen have not been to provide expert opinions in addition to those already on the record regarding the matters in controversy. Rather, they have shared with us their general knowledge on seismology and seismic engineering to ensure that we have not misunderstood the nature of the highly technical areas under consideration. In other words, these two consultants did not provide their own assessment of the merits of the specific issues of the Diablo Canyon case, but they helped our understanding of the record by clarifying, where necessary, the essence of controversy between the various parties.

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We are grateful to the Office of Nuclear Regulatory Research for allowing two of its senior staff, Ms. Sandra Wastler and Dr. James Costello, to assist us in our review. Ms. Wastler, a structural geologist, and Dr. Costello, a structural engineer, were not previously associated with the Diablo Canyon licensing review and their assistance has been invaluable. We emphasize, however, that the opinions expressed in this memorandum are OPE's. We thank also Ms. Ann Thomas (ADM) for her editorial assistance.

We have looked closely at the ALAB decision, the proposed findings, and comments of the parties. We have reviewed a substantial portion of the supporting record including prepared and oral testimony and exhibits. Our review necessarily included the extensive Licensing Board record, because the Appeal Board decision is based in large part on that Licensing Board record. We have also reviewed the six-volume Hosgri reanalysis submitted to the staff by the applicant, as well as the staff's evaluation of this reanalysis as described in Supplements 7 and 8 to the Safety Evaluation Report for Diablo Canyon. We have not reviewed the recent information regarding the use of incorrect drawings and component weights related to the implementation of the seismic design criteria accepted by the staff. We believe that matter is a separate issue and should not be a basis for the Commission decision whether or not to take review of ALAB-644.

OGC has no legal objections to the Commission considering the facts in this memorandum in making its decision on whether or not to undertake review of ALAB-644.

NRC Seismic Practice

Under Appendix A to 10 CFR Part 100, in a case like Diablo Canyon, ground motion at the site due to earthquakes on a given fault is determined by assuming that the largest earthquake for which the plant must be designed may occur on that fault at the point nearest to the site. The next step is to assess what ground motion each of these earthquakes can produce. The ground motion can be represented in a number of ways. Usually ground acceleration (expressed in terms of a "g" value, i.e. a percent of acceleration due to gravity) is used to define the level of ground motion. (Ground velocity, displacement, and frequency may also be used.) The peak acceleration at the site corresponding to the postulated maximum potential earthquake is determined. In the western United States, the peak acceleration is determined from conservative correlations with earthquake magnitude, fault length, and epicentral distance. Once the peak acceleration is determined, it is usually used to fix a point at 33 Hz on the standard response spectrum (for a given damping value) given in Regulatory Guide 1.60. This point then becomes the "anchor point" for the rest of the design response spectrum. Although the method described in Regulatory Guide 1.60 for determining the design response spectrum usually has governed plant designs, the applicant may choose to use an alternate approach, if justified.

Overview of the Diablo Canyon Case

The Diablo Canyon site differs from other nuclear power plant sites in being located so near a capable fault. The Hosgri fault, on which the SSE is postulated to originate, is within six kilometers of the reactor. The fault was discovered after construction was well underway, requiring a seismic reanalysis of the Diablo Canyon plant. The reanalysis involved the examination of close-in (or near-field) phenomena, including consideration of such matters as the attenuation of the high frequency components of earthquake-induced ground motion, the extrapolation of correlations used in seismic analyses of sites at greater distances from capable faults, and the influence of such matters on the determination of the design response spectrum. Of necessity, the seismic reanalysis done to establish a conservative design for the Diablo Canyon plant involved the use of some developing concepts that are not yet generally accepted elements of seismic design practice. In the absence of well-established and accepted methods and data, differences of views are bound to be present. In the present proceeding most of the strong differences are not related to the magnitude of the postulated safe shutdown earthquake (SSE) (although there are varying opinions as to just how conservative it is), but rather to the methods used to estimate the design response spectrum for ground motion from the SSE.

DIABLO CANYON SEISMIC ISSUES

Magnitude of SSE on the Hosgri

Two measures of earthquake magnitude are surface wave motion (M_S) which is measured at a distance far from the earthquake site, and local ground motion (M_l) in the vicinity of the earthquake. The magnitude based on surface wave motion more closely correlates with total energy release, while the magnitude based on local ground motion relates more to potential damage in the near field. For the most part, witnesses in the proceeding refer to "magnitude" without specifying which measure of magnitude is being discussed. Thus, the record is often unclear as to whether certain points of controversy represent differences in opinion or differences in definition.

All parties and witnesses accept as given the 7.5 M earthquake assigned by the USGS to the Hosgri fault as the safe shutdown earthquake (SSE) for the Diablo Canyon plant. Nevertheless, several witnesses (Drs. Blume, Smith, Frazier, and Bolt for the applicant; Dr. Hoffman for the staff; and Dr. Trifunac, who was subpoenaed by the Appeal Board) indicated that the 7.5 M earthquake was too high, and that the SSE would be better characterized as 6.5 M. Indeed, Dr. Blume indicated that it might be as low as 6.25 M. We note that the 7.5 M earthquake proposed by the USGS was primarily based on a 1927 event (the Lompoc earthquake) which USGS attributed to the Hosgri fault. Based on the record, we conclude that, given the differing professional opinions as to whether an assignment of this event to the Hosgri fault is correct, the use of a 7.5 M earthquake on the Hosgri fault is conservative, i.e., the SSE is no greater than 7.5 M and, if the Lompoc earthquake did not occur on the Hosgri fault, the SSE may be as low as 6.5 M.

Magnitude Saturation

Magnitude saturation, as discussed in the record, involves a near-field phenomenon in which, as earthquake magnitude increases, local high frequency ground response may increase less than linearly or not at all. All parties appeared to agree that a magnitude saturation phenomenon may sometimes occur, but there was little agreement regarding its quantification. The Appeal Board agreed with the NRC staff position that, on a fault like the Hosgri, the difference in peak ground accelerations for a 6.5 M and a 7.5 M earthquake would be small, and that a 7.5 M event on the Hosgri would not cause significantly larger ground motion accelerations than the 1971 6.5 M San Fernando earthquake. (The importance of quantifying magnitude saturation is reduced, if the magnitude of the SSE were to be reduced to less than 7.5 M.) OPE believes that the record does support the existence of magnitude saturation. We believe, however, that the sparsity of data forces primary reliance on engineering judgment in determining the degree of the effect of magnitude saturation on peak acceleration, and that the Board relied on the engineering judgment of the staff's principal witness, Dr. Newmark, and the applicant's principal witness, Dr. Blume.

Focusing and High-Stress Drop

Focusing and high-stress drop are two phenomena which could lead to amplified ground motion at a site, given the necessary kinds of geometric and geologic circumstances. However, we believe, based on the record, that those circumstances are lacking for Diablo Canyon and, therefore, that these phenomena were properly disposed of by the Appeal Board.

The Pacoima Dam Record

Dr. Newmark argued that, due to geophysical anomalies at the recording site, the Pacoima Dam time history of ground motion during the 6.5 M 1971 San Fernando earthquake would be representative of the ground motion which would be expected from an earthquake larger than 6.5 M. Among the considerations that underlay Dr. Newmark's engineering judgment that the Pacoima Dam data were representative of the motion at the Diablo Canyon site from a 7.5 M earthquake on the Hosgri, he identified the following: (1) the San Fernando earthquake occurred on a thrust fault which would give greater ground motion for a given magnitude than an earthquake on a strike-slip fault such as the Hosgri; (2) Pacoima Dam was on a ridge that may have amplified the ground motion (corroborated by the lack of damage to a nearby simple structure); and (3) magnitude saturation would be more pronounced at Diablo Canyon given an earthquake on the Hosgri than at Pacoima Dam for the 1971 San Fernando earthquake. Newmark then attempted to roughly match the standard response spectrum to the Pacoima data. The best match occurred when the anchor point (33 Hz) was set at 0.75 g. In all but three places the matched spectrum envelopes the Pacoima Dam data.

We believe that the hearing record supports a conclusion that the peak acceleration at about 33 Hz of 1.15 g (the highest ever recorded) at Pacoima Dam was probably representative of an earthquake greater than 6.5 M. However, there is a lack of information in the record concerning whether, in the frequency range most likely to affect safety-related structures, i.e. 1 to 10 Hz, the Pacoima Dam data are representative of an earthquake greater than 6.5 M. Newmark's testimony is not clear with respect to justifying his use of Pacoima Dam data to bound the 1-10 Hz range of the design response spectrum. Furthermore, authors of the USGS Circular 672 believed that there was no conclusive demonstration that Pacoima ground motion in the range of 1 to 10 Hz was anomalous.

The Appeal Board based its assessment of the adequacy of the 0.75 g anchor point, in part, on the effect of magnitude saturation on peak acceleration and the fact that Pacoima Dam had the highest peak acceleration ever recorded. We found no direct testimony on the record to support a conclusion that, in the particular frequency range of 1-10 Hz, Pacoima Dam represented an earthquake greater than 6.5 M, although Newmark states repeatedly that he is addressing an entire spectrum, not just a portion of it. For this frequency range, the Board's logic in determining the adequacy of the 0.75 g anchor point is not specifically discussed in their decision.

Soil Sites Versus Rock Sites

The issue here relates to whether, in the near-field, ground response motions in rock and soil differ significantly. It is generally accepted that in the far field, there will usually be differences in the ground response. This issue is particularly important because the two seismic records which have been extensively referenced in this proceeding, Bond's Corner and Pacoima Dam, are founded on soil and rock, respectively, and the applicability of each record to use in the near field at Diablo Canyon (a rock site) must be understood. Based on the weight of the evidence on the record, we believe that the Appeal Board's conclusion that, in the near-field, rock and soil accelerations are about equal, is reasonable.

Ratio of Vertical to Horizontal Peak Accelerations

At issue here was whether the vertical accelerations to be accounted for in the Diablo Canyon design basis should be as large as, or even higher than, the horizontal accelerations. The staff's position was that a 2/3 vertical-to-horizontal ratio for western U.S. sites was permissible as an alternate to the specific method outlined in Regulatory Guide 1.60. On the basis of this and other evidence on the record that the vertical acceleration components, even if increased 50 percent over design levels, would increase the total calculated stress by only 1 percent, we believe that the Appeal Board concluded properly that the vertical motion phenomenon will have no consequence for Diablo Canyon.

Anchor Point of 0.75 g Effective Acceleration

As used in the Diablo Canyon proceedings, effective acceleration implies the elimination from the time history of an earthquake of certain peaks of frequency greater than 10 Hz. We note that, based on the record, narrow peaks at such frequencies do not impart much energy to a structure and thus are unlikely to greatly affect large structures. The Appeal Board's subpoenaed witnesses, Drs. Luco and Trifunac, argued that there exists no physical basis for a reduction from peak measured acceleration to effective acceleration. However, Dr. Luco indicated that some structures which experienced high accelerations might have been expected to fail but did not, and that effective acceleration may account for this. In any event, the Appeal Board concluded, on the basis of testimony by other witnesses, that the concept of effective acceleration is reasonable. We agree with the Appeal Board in this regard.

As discussed in the Pacoima Dam section above, in his application of the concept of effective acceleration, based on the representativeness of the Pacoima data to Diablo Canyon and the 1.15 g instrument peak acceleration at Pacoima Dam, Newmark proposed an anchor point of 0.75 g as an effective acceleration for the Diablo Canyon design response spectrum. Another approach discussed by Newmark in his testimony was a calculation he made (using a paper which he identifies) that corroborated his choice of 0.75 g as an anchor point. Using Newmark's testimony and references cited therein, we have performed a calculation which duplicates Newmark's value of 0.75 g. If we are correct in this reconstruction of his calculation, then certain questions arise about the assumptions Newmark may have used to make his calculations, particularly the choice of distance to the Hosgri fault. Other assumptions appropriate to Diablo Canyon can result in numbers higher than 0.75 g. Newmark's choice of 0.75 g appears, however, to have been based primarily on his engineering judgment. It is clear that the Boards relied heavily on that engineering judgment in adopting the 0.75 g as an acceptable anchor point of the response spectrum.

Tau Effect

The "tau effect" is a phrase employed during the course of these hearings to symbolize a phenomenon by which the higher frequencies of earthquake motion are reduced in large structures. The phenomenon appears to be a function of earthquake wave velocities and building dimensions. The tau effect has been described variously in the record as resulting either from the time required for a seismic wave to transit the length of a structure, or from the incoherence of the traveling wave-front as a result of inhomogeneities in the ground. Evidence was offered that in certain instances a tau effect of some sort has been observed. The Appeal Board concluded that response spectrum reductions due to the tau effect were reasonable on the basis of the testimony. Thus, it is difficult to conclude that the Board was not justified in giving, as it did, some credit to this effect. However, given the obscurity of the record concerning the explanations of and methods for quantifying the tau effect, the Appeal Board appears to have relied primarily on the engineering judgment of the witnesses concerning the particular degree of reduction to be allowed.

Damping Value

The damping value of a structure is used to account for the degree to which that structure can dissipate vibratory energy during an earthquake. The damping value varies with material type and construction practices. The applicant proposed in his Hosgri reanalysis to increase the damping value from 5 to 7 percent in accordance with the then-newly revised Regulatory Guide 1.61, "Damping Values for Seismic-Design of Nuclear Power Plants." Drs. Luco and Trifunac, the witnesses subpoenaed by the Appeal Board, argued that the 7-percent value is not sufficiently conservative. The staff testified, however, that recent studies showed that the 7-percent damping value is conservative. The Licensing and Appeal Boards supported the staff's and applicant's positions that a 7-percent damping value is acceptable. We concur with the Appeal Board.

Assessment of Plant SSE Design Margins

We have reviewed the applicant's seismic reanalysis provided in the six-volume report entitled "Hosgri Seismic Evaluation" which was submitted to the staff following the discovery of the Hosgri fault. We also have reviewed the staff's evaluation of this reanalysis as described in Supplements 7 and 8 to the Safety Evaluation Report for Diablo Canyon. Our review indicates that the applicant seems to have approached the reanalysis properly, with the understanding that as-built, rather than design, material parameters were used in the reanalysis. We believe further that the staff's review of this reanalysis was intensive. Moreover, based on the record, we can conclude that there is evidence of the ability of the plant structures to accommodate loads greater than those used in the Hosgri reanalysis.

Operating Basis Earthquake (OBE)

The Joint Intervenors argued that the intent of Appendix A to 10 CFR Part 100 was that the OBE level should be at least half that of the SSE. The NRC staff testified that the value of 0.2 g chosen for the OBE level was acceptable as "being one that could reasonably be expected to affect the plant site during the operating life of the plant." The Appeal Board held that the choice of OBE based on return period is consistent with the intent of Appendix A and that, based on estimates given in testimony of the return period of such an earthquake, an OBE of 0.2 g was acceptable.

The Appeal Board noted that it expected that a contention addressing this issue (i.e., the effect of the OBE on systems interaction) would be addressed by the Licensing Board. The Licensing Board rejected this contention at both the low- and full-power stages. Those rejections are now under appeal.

CONCLUSIONS

OPE has reviewed the record on the issues discussed in ALAB-644 and in the earlier Licensing Board decision referenced in ALAB-644. We note that there are uncertainties in several areas. For example, the designation of the 7.5 M event for the SSE seems to be very conservative. On the other hand, there may exist non-conservatisms relating to magnitude saturation, effective acceleration, and the tau effect. Overall, we believe there is no basis for concluding that the Appeal Board erred in its decision regarding the adequacy of the seismic design criteria. On some issues, the Appeal Board appears to have relied on the engineering judgment of key witnesses and found the preponderance of the evidence in favor of the positions of the applicant and of the staff.

As a result of our review of the record and Board decisions, we have identified areas where the evidence is not as fully developed as one might wish. However, on balance, we do not find technical reasons to recommend Commission review of ALAB-644. The issues are interrelated, very technical, and frequently at the forefront of developing technology. The existing record is already replete with expert viewpoints. Further review could lead merely to a restatement of those views, by the same or other experts, with little new information provided.

Appendix:
Brief Listing of Technical Terms
Used in OPE Analysis

Enclosures:
1. OPE Analysis of Diablo Canyon Seismic Issues: ALAB-644
2. Summary of NRC Licensing Seismic Practice
3. General Methodology of Seismic Design and Analysis
4. Witnesses Cited in OPE Analysis of Diablo Canyon Seismic Issues

cc: L. Bickwit
S. Chilk

ENCLOSURE 1

OPE ANALYSIS OF DIABLO CANYON SEISMIC ISSUES: ALAB-644

ENCLOSURE 1

OPE ANALYSIS OF DIABLO CANYON SEISMIC ISSUES: ALAB-644

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OPE ANALYSIS OF DIABLO CANYON SEISMIC ISSUES: ALAB-644

SAFE SHUTDOWN EARTHQUAKE

• Choice of the Safe Shutdown Earthquake Magnitude

While the Appeal Board did not have before it a contention on the appropriate magnitude of a safe shutdown earthquake (SSE) at Diablo Canyon, the magnitude of the SSE is the starting point in analyzing the adequacy of the Diablo Canyon seismic design. There was, in fact, no controverted issue, before the Appeal Board as to whether the 7.5 M* earthquake on the Hosgri fault** is the appropriate value to be used for the controlling earthquake to estimate the ground motion to be used in the design of Diablo Canyon. As presented before the Licensing Board, the 7.5 M value was originally assigned as the maximum-potential magnitude for the Hosgri fault by U.S. Geological Survey (USGS) consultants to the staff and was adopted by all parties. The USGS

*Earthquake magnitude can be measured either as surface wave magnitude (M_s) or local magnitude (M_l), and reference to both types of magnitudes has been made in the record. Unfortunately, the Appeal Board and most witnesses were ambiguous regarding which definition of magnitude was being discussed and the record reflects this ambiguity. Due to the extent of the record, in our analysis we have not attempted to reconstruct each reference to provide the specific type of magnitude discussed by each witness. Therefore, throughout this document reference to magnitude (M) only is made unless the record specifies M_s or M_l. While interpretation of the record is more difficult, the issues discussed are not critically affected by this problem.

**The Hosgri fault is actually a fault zone which consists of a 6.5-8 km wide zone of subparallel faults (i.e., 2-5) and folds. The Hosgri fault's closest approach to the Diablo Canyon site is 5.8 km. In accordance with Appendix A to 10 CFR Part 100, the safe shutdown earthquake (SSE) for the site is taken to occur on the Hosgri at its closest approach to the site to determine the vibratory ground motion.

determination was based on an indepth review of the overall tectonic framework of southern California, the postulated length of the Hosgri fault, the relationship of the Hosgri to the San Andreas fault system, the potential relationship between the Hosgri fault and the 1927 Lompoc Earthquake, and geophysical evidence of Late Quarternary movement on the Hosgri. The strongest argument used by the USGS to support the choice of a 7.5 M event on the Hosgri fault was the fact that the USGS could not eliminate the Hosgri fault as the source of the 1927 Lompoc Earthquake of 7.3 M. In contrast to the USGS, the remaining witnesses presented evidence that a 7.5 M event postulated on the Hosgri was overly conservative. A list of earthquakes significant to Diablo Canyon appears in Table 1.

Dr. Blume, a consultant for the applicant, testified that a 7.5 M earthquake postulated for the Hosgri was too conservative and that, based on all evidence, an earthquake in the range of 6.25 to 6.5 M was adequate (following Tr. 6099, p. 12). In addition, witnesses for the applicant (Pacific Gas & Electric), Drs. Smith, Frazier, and Bolt, testified that, based on their analysis of the slip rate of the Hosgri fault in Late Quarternary, the local stress conditions, the distance from the Hosgri fault to the San Andreas fault, and the regional seismicity, there have not been recurrent earthquakes above 6.5 M on the Hosgri in the past 17,000 years. Dr. Smith associated the 1927 Lompoc Earthquake with the Lompoc structure, not the Hosgri fault (Tr. 5483-84, Tr. 5635-45).

A witness for the Joint Intervenors, Dr. Silver, originally testified that he computed the maximum earthquake on the Hosgri fault, based on the same equations as Dr. Smith, as an 8.25 M event. During cross-examination, Dr. Silver was unable to substantiate the assumptions used to develop his estimates (i.e., a 400-mi fault rupture length) of maximum magnitudes (Tr. 6250-59, 6264-97). Finally, Dr. Silver could not state within a reasonable degree of geologic certainty that an earthquake as large as 6.5 M had or would ever occur on the Hosgri fault (Tr. 6333-44, 6437-42, 6447-53).

Dr. Trifunac, an ACRS consultant subpoenaed by the Appeal Board, based on the weight of the record, did not consider a 7.5 M earthquake appropriate for the

Table 1 List of Earthquakes Relevant to the Diablo Canyon Proceedings

Earthquake	M	Cited by	Relevant to	Recording station	Peak acceleration
1927 Lompoc	7.3	USGS	SSE	None identified in record	
Gazli, Russia	7.0	Luco	Magnitude saturation		0.85 g horiz 1.3 g vert (soft rock)
Tabas, Iran	7.7	Luco	Magnitude saturation		0.8 g alluvium
1979 Imperial Valley	6.5	Joint Intervenors	Design spectra	Bond's Corner	0.81 g
1971 San Fernando	6.6	Newmark	Design spectra and tau effect	Pacoima Dam	1.17 g
				Hollywood Storage Bldg Basement	0.10 g
				Hollywood Storage Bldg Parking Lot	0.17 g
1952 Kern County		Newmark	Tau effect	Cal Tech Athenaeum	0.047 g
				Taft Lincoln School Tunnel	0.156 g
				Santa Barbara Court House	0.089 g
				Hollywood Storage Bldg Basement	0.055 g
				Hollywood Storage Bldg Parking Lot	0.059 g
1972 Ancona, Italy		Newmark	Rock-vs-soil acceleration	Rocca	0.6 g
				Palombina	0.4 g

Hosgri. Dr. Trifunac's testimony supported 6.5 M as more appropriate for the Hosgri fault (Tr. 8971).*

Mr. Hoffman, an NRC staff witness, concluded that the 1927 Lompoc Earthquake did not occur on the Hosgri fault and that the assignment of a 7.5 M earthquake to the Hosgri fault was extreme or ultraconservative (following Tr. 8522, pp. 1-5; Tr. 8529). Dr. Stepp, another NRC staff witness, testified that the 1927 Lompoc earthquake could have occurred on the Hosgri fault, but that, on balance, it was probably associated with the Transverse Range Structures and that 7.5 M on the Hosgri was very conservative (Stepp testimony, pp. 12; 31, 32).

During the Appeal Board hearing, reference was made to the USGS geologic and seismic evaluation of the Point Conception liquid natural gas (LNG) site and the USGS conclusion that:

> ...existing evidence favors association of the 4 Nov., 1927 (M 7.3) Lompoc earthquake with an east dipping reverse fault such as the Offshore Lompoc or similar reverse fault 10 km to the south that offsets the seafloor.

According to an NRC staff witness, R. McMullen, the document does not represent a change in the USGS position as presented at the Licensing Board hearings. The USGS still concludes:

> The 1927 earthquake, therefore, cannot be unequivocally located on any one of these faults. The Hosgri fault, however, is closer to the center of the estimate of error than the other faults and, therefore, must be considered as a possible fault on which to locate the earthquake (following Tr. 922, pp. 3-5).

Based on an evaluation of the data presented as evidence in the hearing, the Licensing Board concluded that a 7.5 M earthquake on the Hosgri is very conservative for the safe shutdown earthquake as used in the Diablo Canyon seismic design (ALAB-644, p. 11). The Appeal Board, based on subsequent review of the data, reaffirmed the decision of the Licensing Board.

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*Tr. refers to the transcript of the Licensing Board hearing; R.Tr. refers to the transcript of the reopened seismic hearing before the Appeal Board in October 1980.

Based on the testimony, it is apparent that during the Licensing Board hearing, the USGS consultants to the staff were the only witnesses to support a 7.5 M on the Hosgri fault. As described above, the remaining witnesses, including the NRC staff members, either testified that they considered the 7.5 M very conservative to ultraconservative or indicated that a lower M value was more appropriate.

The strongest argument used to support the USGS estimate was that the Hosgri could not be eliminated as a source of the 7.3 M 1927 Lompoc Earthquake. Yet during the Appeal Board hearing, evidence was presented by the applicant which indicated that the USGS had altered its position. In the USGS geologic and seismic review of the Point Conception site the USGS concluded: "...the existing evidence favors association of the November 4, 1927 Lompoc (7.3 M) earthquake with an east dipping reverse fault such as the offshore Lompoc or similar reverse faults 10 km to the south that offset the seafloor." The USGS contended that its position had not changed, that the Hosgri fault could not unequivocally be eliminated as a source of the 1927 Lompoc earthquake. The applicant considered this document a reversal of the USGS opinion. We would suggest that while the evidence does not support a reversal of USGS opinion it does weaken the strength this argument played in determining the maximum potential magnitude on the Hosgri.

It should be noted that the USGS is considered the authority with regard to assigning earthquake parameters (magnitude, epicentral distance, and location) to specific earthquakes. Therefore, the NRC staff may have adopted the USGS value as the overall NRC opinion based on this authority. As can be seen in the testimony, both R. Hoffman and Dr. Stepp considered the 7.5 M for the Hosgri as very conservative.

Based on the substantial portion of the record we have reviewed, we believe that there is sufficient evidence to conclude that the use of a 7.5 M earthquake on the Hosgri fault may be very conservative.

• Estimate of the Ground Motion for the SSE at the Site

MAGNITUDE SATURATION AND PEAK GROUND ACCELERATIONS: Would the peak ground accelerations in the near field resulting from a strong earthquake on the Hosgri fault be close to values corresponding to an earthquake of lower magnitude due to magnitude saturation?

During the reopened hearings on Diablo Canyon, the Appeal Board evaluated the concept of magnitude saturation as one of several parameters critical to the reanalysis of the Diablo Canyon seismic design. All participants agree magnitude saturation exists, but there is disagreement over the degree of saturation as magnitude increases above 6.5 M. The NRC staff presented the concept of magnitude saturation in support of its belief that free-field motion acceleration values for a postulated 7.5 M earthquake on the Hosgri fault would not be significantly larger than those that occurred for the 6.5 M San Fernando earthquake. The NRC staff proposed that the unusually high acceleration-value records at Pacoima Dam for the San Fernando earthquake were representative of a 7.5 M earthquake on the Hosgri fault.

Magnitude saturation occurs in near-field (0-10 km) during large earthquakes. When a strong earthquake occurs, the seismic energy released causes oscillations (compression and shear waves) in the earth over a wide range of frequencies. The higher the frequency of the seismic waves, the greater the attenuation of the seismic waves as they travel through the earth. In the near field of a strong earthquake, the very high frequencies* (greater than 10 Hz) are quickly attenuated, and the high frequencies between 1-10 Hz (the most likely to damage nuclear power plant structures) predominate (Tr. 5970, 5877; following R.Tr. 1138, Trifunac p. II-2). (We will come back to this statement frequently.) The low frequencies (less than 1 Hz) are attenuated very slowly and travel long distances. Because the seismic energy is being released over an extended fault length, and because the very high frequency oscillations emanating from all but the

*The frequency range of seismic waves is measured in Hertz, or cycles per second (Hz). The Appeal Board and most witnesses were ambiguous regarding the frequency range being referred to when discussing very high, high, and low frequencies. As can be seen by the nature of several points of controversy, this ambiguity has made assessment of the record difficult.

nearest fault segment will be preferentially attenuated, any location near a fault will not be greatly affected by more than a small amount of these very high frequency waves, regardless of fault length. Thus, because the participating fault length generally is longer for larger earthquakes, peak acceleration at a near-field site does not continue to increase significantly with increasing earthquake magnitude. This phenomenon is one form of magnitude saturation.

Another form of peak ground motion saturation is associated with large earthquakes. The release of seismic energy generally occurs at some depth beneath the surface of the earth. At the surface within the near field there is little difference in the distance between any point on the surface and the nearest region of seismic energy release. Thus, in the near field, the peak ground motion is not strongly affected by distance to surface expression of the fault (Tr. 5922, 8637). Based on the review of the record, the parties involved and the Appeal Board supported the concept that peak ground motion accelerations in the near field saturate due to this effect and are not strongly dependent on distance to the fault. No further consideration to the concept of distance saturation was given in those proceedings.

Dr. Luco's testimony before the Appeal Board provided direct evidence of the phenomenon of magnitude saturation by graphically comparing surface wave magnitude (M_s) to local (M_l) magnitude (Figure 1). The types of magnitude measurements are functions of the frequency ranges in which they are measured (Tr. 5970). A surface magnitude (M_s) can be determined by measuring "low frequency" waves at great distances (greater than 400 km), and these measurements give an indication of the total or integrated energy released along the entire length of the ruptured fault. The local magnitude (M_l) is measured closer to the source at distances from 20-400 km and is directly influenced by the higher frequency spectrum of seismic waves.

Dr. Luco's testimony provides graphical comparison of M_s versus M_l values for a range of historical earthquakes. This comparison shows that for an M_s or M_l value of up to 6, there is little difference between values. For larger values (severe earthquakes over long faults), the M_l values are lower than the M_s values and reach a maximum in the range of 7.2-7.5 M. As the earthquake size or total energy increases, the high frequency motion at distances of 20-400 km saturates

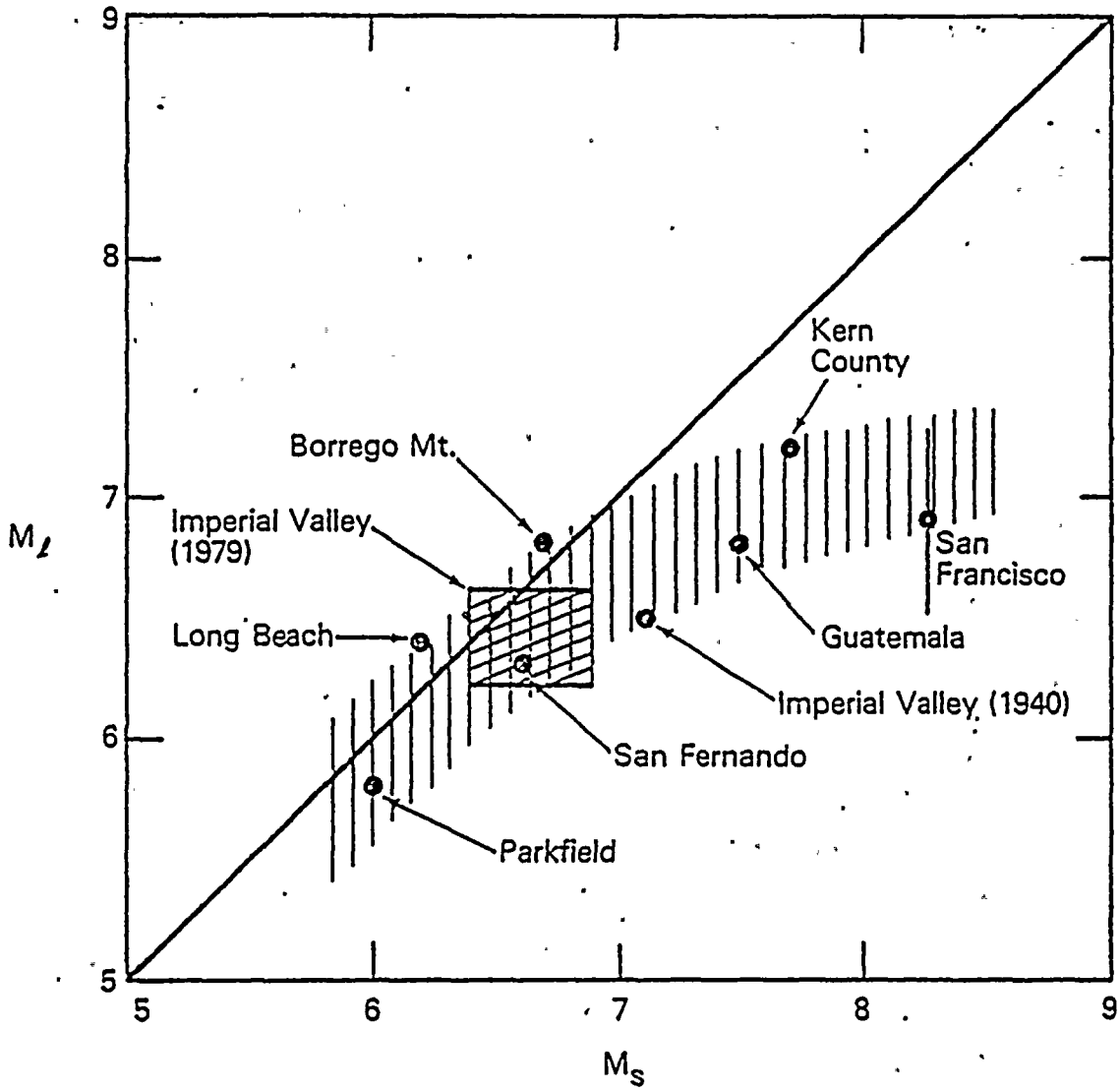

Figure 1 Surface Wave Magnitude (M_s) versus Local Wave Magnitude (M_ℓ)

Source: Dr. Luco's testimony

(Tr. 5970). Although some scatter exists in the data, Dr. Luco's testimony provides significant evidence that the concept of magnitude saturation exists.

The NRC staff and Newmark referenced a paper by Hanks and Johnson* that provided additional support for the concept of magnitude saturation in the near field. This reference was cited by several other witnesses in the reopened proceedings. The Hanks and Johnson paper presented a comparison of near-field acceleration for a series of historic seismic events, suggesting that measured peak accelerations are to some extent dependent on the magnitude of an earthquake below 4.5 M. Hanks and Johnson have concluded, recognizing the scatter of the data, that little to no magnitude dependence occurs above 4.5 M. With the exception of the Pacoima Dam peak acceleration, no peak acceleration values were cited by the authors greater than approximately 0.80 g. The overall conclusion reached by Hanks and Johnson is that in the near field the "high frequency" acceleration from earthquakes is a function of physical processes related to the fault region near the measuring point and is not dependent on earthquake magnitude.

Dr. Seed, in his testimony before the Appeal Board on behalf of the applicant, presented an extension of the Hanks and Johnson work referenced above, to which he added five additional earthquakes in the near field. Seed concluded that his extension fully supports the conclusions reached by Hanks and Johnson on magnitude saturation. Of the new data points, the highest acceleration is 0.80 g, with the exception of one maximum acceleration of 0.95 g.

Dr. Blume, a witness for the applicant, provided similar evidence of magnitude saturation derived from the near-field accelerations recorded for the 6.5 M 1979 Imperial Valley earthquake as a further refinement of the Hanks and Johnson paper. Of these new data points, the maximum acceleration measured was 0.80 g.

Dr. Newmark, an NRC staff consultant, in testimony before the Appeal Board, presented evidence in which he implied that peak acceleration in the near field is totally independent of magnitude and approaches an asymptotic value for magnitudes greater than 6.5 (Figure 2).

*Hanks, T. C. and D. C. Johnson, "Geophysical Assessment of Peak Accelerations," Bull. Seism. Soc. Am. 66, pp. 959-968, 1976.

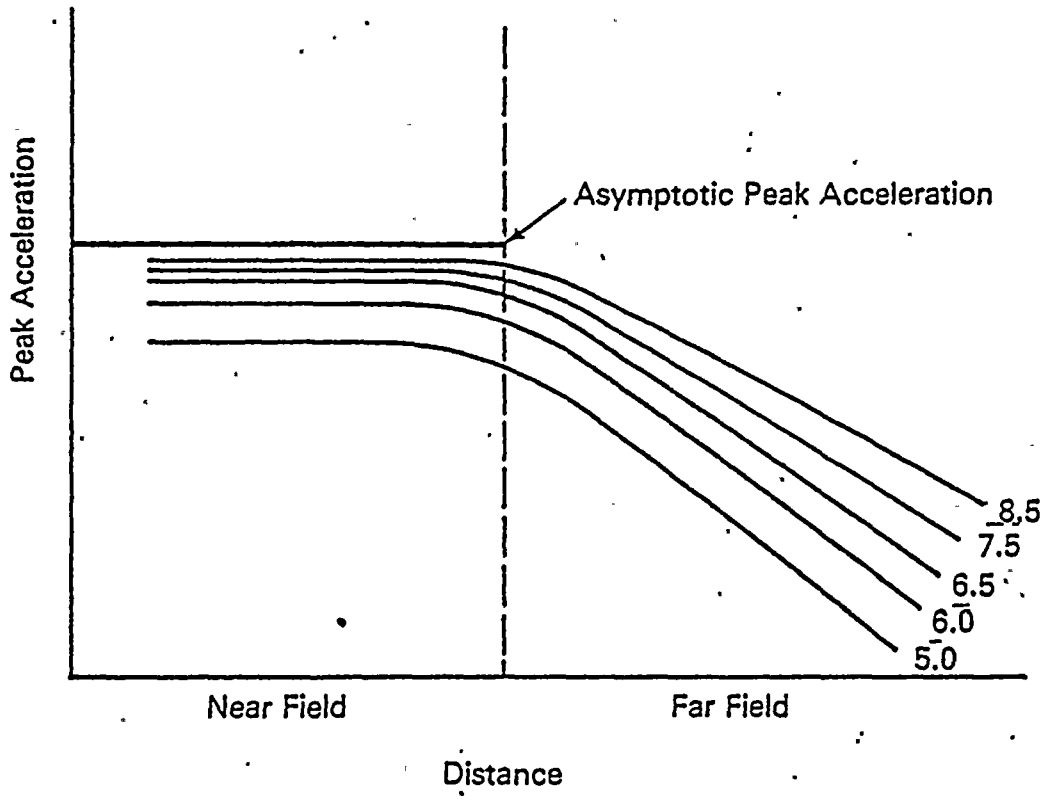

Figure 2 Representation of Newmark's Concept of Magnitude Saturation

Source: OPE

As discussed above, a number of witnesses support the concept of magnitude saturation and agree that peak high frequency and motion in the near field are primarily dependent on the nature of fault rupture. However, these same witnesses do not state that the data presented in the Hanks and Johnson paper and the refinements of the data reflect the maximum acceleration possible in the near field.

Contrary to the testimony discussed above, Governor Brown and the Joint Intervenors presented evidence to support their contention that peak acceleration in the near field is not independent of magnitude, but continues to increase between 6.5 M and 7.5 M (Figure 3). In support of their contention, they presented evidence that the data presented in the Hanks and Johnson paper (USGS) in support of magnitude saturation, when properly represented by a straight-line fit to the data, indicate an increase in peak acceleration of a factor of 1.4 per magnitude unit between 6.5 and 7.5 M. They argued that this straight-line fit is more appropriate than the log-normal fit used by Hanks and Johnson and by others. In addition, Governor Brown and the Joint Intervenors referred to the USGS conclusion from Circular 672* that, in the absence of data, the use of 7.5 M as an SSE requires the presumption of some increase in ground motion with increasing earthquake magnitude.

Based on the testimony, the Appeal Board concluded that the physical description that was developed for the nature of the earthquake motion in the near field and the data on peak ground motion that have been presented in these proceedings provided a convincing case for magnitude saturation. The Appeal Board concluded that in the near field, peak high frequency ground acceleration is largely independent of earthquake magnitude. At the same time, the Appeal Board did not discount the possibility that future ground motion records may exceed those previously measured. Nor did the Board ignore the fact that larger magnitude earthquakes may give rise to higher peak acceleration measurements.

Thus, the Appeal Board supported the NRC staff position that in the near field (within 10 km), given a strong earthquake on a long fault such as the Hosgri,

*Page, R., D. Boore, W. Joyner, H. Coulter, "Ground Motion Values for Use in the Seismic Design of the Trans-Alaska Pipeline System" (USGS Circular 672), 1972.

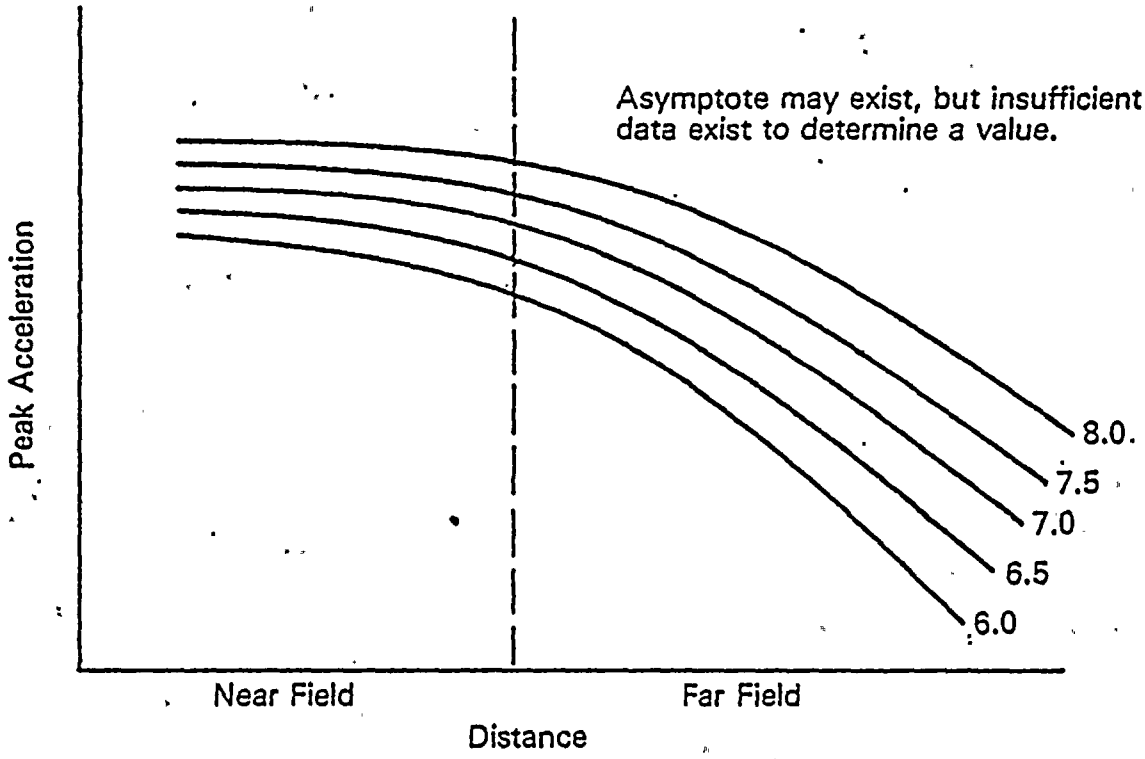

Figure 3 Representation of Intervenor's Concept of Magnitude Saturation

Source: OPE

the difference between the peak ground motions of 6.5 M and 7.5 M earthquakes is small. The Board also supported the NRC staff position that free-field ground motion acceleration values for a 7.5 M event on the Hosgri fault would not be expected to be significantly larger than the free-field ground motion acceleration values that occurred for the 6.5 M San Fernando earthquake.

We note that following the reopened seismic hearings in October 1980, USGS published a report, "Peak Horizontal Acceleration and Velocity from Strong-Motion Records Including Records from the 1979 Imperial Valley, California Earthquake" (Open File Report (OFR) 81-365). Two of the authors of this report, W. Joyner and D. Boore, were also authors of USGS Circulars 672 and 795,* which was the basis for the NRC staff's, the applicant's, and the Boards' acceptance of the instrumental acceleration for a 7.5 M earthquake for Diablo Canyon. Therefore, the Intervenors petitioned the Appeal Board to reopen the hearing, stating that the report has analysis and conclusions relevant to Diablo Canyon. OFR 81-365 included the near-field data from the 1979 Coyote Lake and Imperial Valley earthquake, and introduced new statistical techniques (beyond those used in USGS Circulars 672 and 795) and new modeling techniques and assumptions. The net result claimed by the USGS authors is that they now have "a much improved basis for making ground-motion predictions at small distances from the source" (Figure 4). The importance of the report lies in its relation to near-field magnitude saturation and its indication that magnitude saturation is not evident in the sense of approaching an asymptote as the earthquake magnitude approaches 7.5 M.

Because OFR 81-365 was written by the authors of USGS Circulars 672 and 795 and adds new near-field data, the Joint Intervenors requested that the Appeal Board reopen the record to include it. Although preliminary, OFR 81-365 contains unique statistical analyses that appear to have potential for predicting attenuated accelerations in the near field. The authors of that report state: "The newly available close-in data permit us to extend the prediction equation to zero distance." The staff response stated that strong

*Boore, D., W. Joyner, A. Oliver, R. Page, "Estimation of Ground Motion Parameters" (USGS Circular 795), 1978.

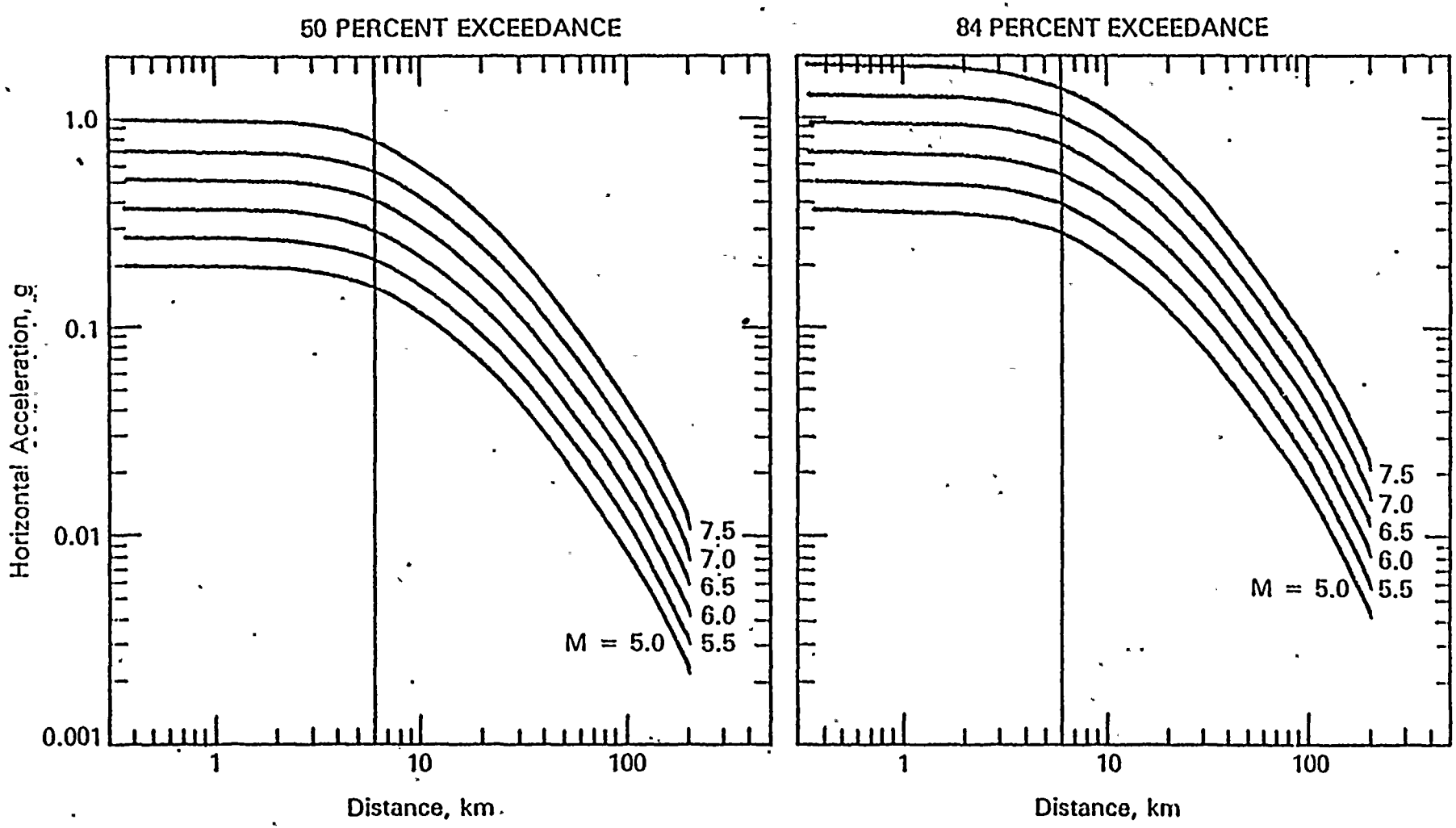

Figure 4 Predicted Values of Peak Horizontal Acceleration for 50 and 84 Percent Exceedance Probability as Functions of Distance and Moment Magnitude

Source: OFR 81-365

ground motion seismology is an evolving science, with assumptions and interpretations on which there is not always a complete consensus. Furthermore, the staff and the applicant both noted that OFR 81-365 does not contain new information but rather develops a new analytical techinque based on existing data. The Appeal Board does not describe the technical basis for its conclusion. Nevertheless the Appeal Board ruled (ALAB-644, pp. 176-178) that while the analysis was new, the seismic records underlying it were not. The Appeal Board concluded that it would not have changed its conclusions had it admitted OFR 81-365 into the record.

Testimony was given that indicated the effect of magnitude saturation is to preferentially attenuate frequencies higher than 10 Hz in the near field for large earthquakes. However, the frequencies of 1-10 Hz, those most likely to damage a building, predominate and are attenuated less than those over 10 Hz (Figure 5). We have not found any reference to this point in the testimony. Data do not exist in sufficient quantity to resolve questions about amplification in the 1-10 Hz region for near-field earthquakes. We feel, however, that the record supports the existence of magnitude saturation. Unfortunately, the lack of data does not allow us to quantify the effect of magnitude saturation on peak acceleration.

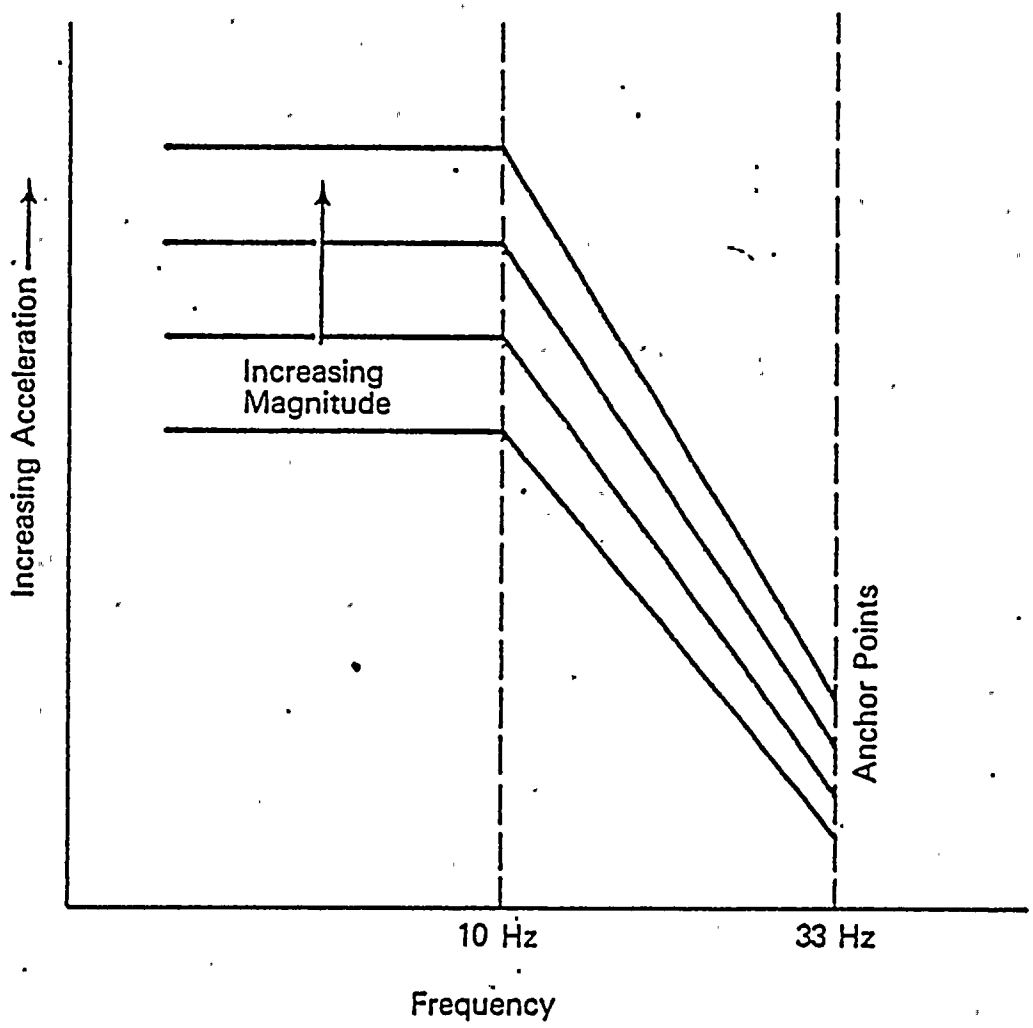

Figure 5 Representation of General Effect of Attenuation on Response Spectra as Earthquake Magnitude Increases

Source: OPE

EFFECTS ON GROUND MOTION OF FOCUSING AND HIGH-STRESS DROP: <u>In the event of a strong earthquake on the Hosgri fault, would focusing or high-stress drop phenomena increase ground motion at the site?</u>

Focusing involves a local reinforcement of propagating seismic waves by means of a form of constructive interference. In ALAB-644, the Appeal Board referred to several experts who provided descriptions of focusing in the transcript. A staff witness, Dr. Rothman, explained in the reopened hearing that "...(t)he focusing effect results from constructive interference of signals whose velocity is close to that of the rupture propagation velocity" (following R.Tr. 536, p. 13). The Intervenors' case on focusing was presented principally through one of their witnesses, Dr. Brune. In his prepared testimony before the Licensing Board, Dr. Brune stated that focusing "...can lead to accelerations and velocities amplified by more than a factor of 2 in a sector of about ±5° from the direction of fault propagation" (Joint Intervenors' Ex. 66, p. 3-2).

It seems clear from the testimony and evidence before both the Licensing Board and the Appeal Board that the phenomenon of focusing is governed by geometric constraints, by local geological properties, and by the mechanism and manner of fault rupture. Much of the Diablo Canyon record on focusing relates to the adequacy of various computer calculations of the extent of expected focusing at Diablo Canyon from an earthquake on the Hosgri fault. As the Appeal Board noted in ALAB-644, Dr. Brune's direct testimony on focusing was essentially theoretical, citing only two earthquakes (the 1971 San Fernando and 1940 Parkfield earthquakes) where focusing led to enhanced ground velocities. However, the Appeal Board noted: "...the evidence indicates that the important structures and systems at Diablo Canyon have relatively high natural frequencies. At high frequencies, the critical seismic motion parameter is not velocity or displacement but acceleration." Under questioning, Dr. Brune stated that, for the Parkfield and 1940 Imperial Valley earthquakes, seismic recordings within 20 km of the rupturing faults and in the direction of their propagation had revealed no unusual values of acceleration due to focusing (Tr. 8030-39). However, the Appeal Board noted (ALAB-644, footnote 157) that an applicant witness, Dr. Frazier, agreed that focusing was recorded at Parkfield.

Dr. Brune testified that the phenomenon of focusing had been demonstrated and that focusing would probably be more noticed at low frequencies than at high frequencies (Tr. 8016-17). However, he indicated that the effectiveness of focusing was not known (Tr. 8011-12, 8028-30, 8040-42), and he said that further modeling and field evidence are needed (Tr. 8028-29, 8042, 8091, 8104). He also stated that the model he used as one basis for his focusing predictions could not be considered reliable and that more realistic calculations should be made (Tr. 8135-36).

In opposition, the applicant and the staff held that the effects of focusing are already incorporated in the existing seismic records (Tr. 5878-81; following 5490, p. 26; following R.Tr. 536, p. 13). Dr. Brune countered that these records may not have been made in the maximum direction of focusing (Tr. 8075). Regarding the Pacoima Dam record of the 1971 San Fernando event, the Appeal Board noted (ALAB-644, footnote 164) that witnesses for the applicant, Drs. Bolt and Frazier (Tr. 5880-87), appeared to agree with a study which hypothesized that the focusing which occurred there caused a large velocity pulse in the early stages of the record but that the large acceleration value for which that record is best known occurred later in the ground motion record and was not related to the focusing. In that same footnote, the Appeal Board noted that the Newmark spectrum based on the Pacoima Dam record included the entire ground motion record and thus incorporated the effects of both the velocity pulse due to focusing and the high-frequency peak acceleration.

Under cross-examination, Dr. Brune concluded that the probability of focusing at any particular site is low (Tr. 8140-44). However, he indicated that the limitation of the effects of focusing to a ±5° sector around the direction of rupture propagation represented a misconstruing of his previous testimony. He also pointed out that different maps indicated different distances between Diablo Canyon and that portion of the fault aligned toward the plant (R.Tr. 619-23). At question here apparently is a short splay of the Hosgri, ending about 3.8 km from the site and oriented to permit direct focusing. Regarding this splay, a witness for the applicant, Mr. Hamilton, had the opinion that such a splay would not participate in a major earthquake; Dr. Frazier, another witness for the applicant, believed that the splay would not generate much more than a 3 M event; and a witness for the staff, Mr. Devine of the USGS, did not believe the feature

existed and thus did not consider it capable (see ALAB-644, footnote 170). The Board stated in ALAB-644 that it did not believe that "the splay represents a reasonable source of earthquake motion beyond those already being considered in the plant's design."

Based on Dr. Brune's testimony (Joint Intervenors' Ex. 66), the Appeal Board noted that because of the orientation of the Hosgri fault relative to the Diablo Canyon site, any focused ground motion would have to travel about 20 km (ALAB-644, footnote 169). The Board also recognized that the high frequency accelerations (which are those most damaging to a nuclear plant) would be "preferentially attenuated in traveling this distance."

The Appeal Board reaffirmed the Licensing Board's conclusion regarding focusing at Diablo Canyon. The Appeal Board did point out, however, that the Licensing Board could have been more explicit in stating its reasons for rejecting the Joint Intervenors' (i.e., Dr. Brune's) position.

The Joint Intervenors postulated that the effects of an earthquake along the Hosgri fault could be enhanced if a short segment of the fault were to rupture under high-stress conditions. Because more energy would be released if such a high-stress drop rupture occurred than if the segment were under only mild stress, the existence of such a high-stress region before the earthquake would have implications regarding the adequacy of the plant design. The Joint Intervenors contended, relying on the testimony of Dr. Brune (Tr. 7930-31, Joint Intervenors' Ex. 66), that this phenomenon might produce unusually high ground acceleration at the Diablo Canyon site. Dr. Brune postulated that accelerations on the order of 2 g might occur.

Testimony was offered by staff and applicant witnesses (following Tr. 5490, pp. 9-11; following Tr. 8522, p. 23; following Tr. 8617-18) to the effect that there are no indications of high-stress areas along the Hosgri fault. Moreover, this fault is expected to exhibit a strike slip/dip slip motion rather than a thrust motion, which is associated with the highest stress-drop values. The Appeal Board noted (ALAB-644, footnote 189) that of the numerous examples of high-stress drops cited by Dr. Brune, only one, that at Pacoima Dam, resulted in high measured accelerations (1.2 g). The Board further noted that, even

for this one case, more persuasive arguments have been offered for the cause of the high acceleration.

Dr. Brune further postulated that the high acceleration (0.81 g) recorded at Bond's Corner during the 1979 Imperial Valley earthquake was the result of a large fraction of the energy of that event being released from a high-stress region. The Appeal Board noted that, although other recording sites were equally distant from the region of energy release postulated by Dr. Brune (R.Tr. 613), these other sites did not experience this high acceleration value, undercutting the idea of a localized high-stress zone. The Board observed: "The Bond's Corner measurement of 0.81 g is commensurate with measured near-field values reported for earthquakes above 5 M_s in other recent studies" (Joint Intervenors' Ex. 47, 61, and 62). The Appeal Board concluded: "Dr. Brune's theory is inconsistent with the evidence of record and (found) the position of the intervenors and the Governor on high stress drop not well taken."

Focusing and high-stress drop were extensively addressed in the record and, based on the record, we believe that the Diablo Canyon site probably does not lend itself to effects of focusing and high-stress drop which exceed the present design basis requirements.

THE USE OF PACOIMA DAM DATA: Have the large peak accelerations observed at Pacoima Dam been properly interpreted and do they constitute a sound basis for developing the response spectrum for a 7.5 M earthquake at Diablo Canyon?

Dr. Newmark's testimony provided the rationale for using the Pacoima Dam record to characterize a 7.5 M event. Drs. Luco and Trifunac, subpoenaed by the Appeal Board, objected to Dr. Newmark's rationale. They stated that the 1971 San Fernando earthquake was recorded as a 6.5 M event, and, therefore, the Pacoima Dam record of the 1971 San Fernando earthquake showing peak acceleration of 1.2 g is representative of only a 6.5 M event. Both witnesses believed that for a 7.5 M event, the design should be anchored at a peak acceleration of 1.15 g (Tr. 8872-77, 8974-75). However, Dr. Trifunac stated that he believed a 6.5 M earthquake was adequate for the Diablo Canyon design, and on this basis Trifunac accepted Newmark's design spectrum anchored at 0.75 g (Tr. 8971, 8985).

The NRC staff presented evidence that the unusually high acceleration values recorded at Pacoima Dam during the 6.5 M 1971 San Fernando earthquake were representative of acceleration values expected from a postulated 7.5 M earthquake on the Hosgri fault. A key element of the staff's argument was the unusual nature of the Pacoima Dam record.

The 6.5 M 1971 San Fernando earthquake was recorded at many locations, including a station (referred to as Pacoima Dam) located on a rocky ridge adjacent to an abutment of the dam itself, approximately 3 km from the fault. The peak acceleration values for all other locations, none closer than 18 km from the fault, ranged from 0.027 g to 0.31 g (NUREG-0003*). The peak acceleration value recorded at Pacoima Dam was 1.2 g. This record provided the greatest peak acceleration ever recorded from an earthquake and has been widely studied by various experts.

As a result of investigations of the Pacoima Dam records, many of the experts have concluded that the large peak accelerations recorded at Pacoima Dam resulted from the anomalous amplification of the motion due to the ridge as a rock structure combined with the adjacent dam having an influence on the response of the

*"Statistical Studies of Horizontal and Vertical Earthquake Spectra," January 1976.

ridge. Because the rocky ridge itself suffered some damage during the earthquake, some also conclude that this damage may have further amplified the record motion (Tr. 10,085; DLL-12). One investigator, Boore 1973, indicated that the high frequencies (greater than 10 Hz) are amplified, but the influence of ridge on the lower frequencies in unimportant. Page, Boore, Joyner, and Coulter, 1972, also indicated that they knew of no conclusive investigations which demonstrate an anomalous amplification of recorded motion in the frequency range 1-10 Hz.

The investigations of Pacoima Dam records have provided estimates of the amount of the amplification by the ridge (DLL-12; Tr. 10,088). Based on these investigations, the estimated amplification of the ridge over the bedrock at valley-floor level ranged from 30 to 310 percent (DLL-12; Tr. 10,086-088); the average amplification factor was 100 percent. If the 310 percent estimate is deleted, the average amplification factor was 48 percent. Therefore, the peak acceleration in the Pacoima Dam area, without amplification by the ridge or the dam, based on the averaged results of all investigators, was in the range of 0.6 g to 0.8 g instead of the 1.2 g commonly used as Pacoima ground motion (Tr. 10,088, DLL-12). The record does not indicate over what spectral range these amplification factors should be applied.

The Appeal Board, based on the testimony given, affirmed the use of Pacoima Dam as the basis for a near-field ground motion spectrum for a 7.5 M event. The Board based its conclusion on its previous findings on magnitude saturation, on the unique nature of the Pacoima Dam record, and on the fact that the 1971 San Fernando event involved a thrust fault, which they felt causes near-field motion greater than an event on a strike slip fault such as the Hosgri.

As we noted, based on the evidence in the record, it appears that the Pacoima Dam record represents an unusually high peak acceleration of 1.2 g, and this peak acceleration may actually be more representative of the ground motion actually associated with some higher magnitude earthquake. It is not clear, however, from the Boards' records if the Pacoima Dam record in the frequency range of interest (1-10 Hz) represents a deviation from that expected for a 6.5 M earthquake. Most of the testimony on Pacoima Dam centered on a frequency range of little practical interest (i.e., near 33 Hz) regarding excitation of structures important to safety. We found no supporting statement on the record

which indicated that the Pacoima Dam record substantially exceeded that expected for a 6.5 M earthquake, the frequency range of 1-10 Hz. USGS Circular 672 (p. 7) indicated that in the frequency range of 1-10 Hz, the Pacoima Dam record closely resembled what one would expect for a 6.5 M earthquake.

There are no other near-field data in the San Fernando earthquake record besides those recorded at Pacoima Dam. If one wishes to determine if Pacoima Dam represents a 7.5 M earthquake, one method is to compare its record to those at sites with reasonably similar distances to their epicenters and hopefully similar ground conditions (e.g., alluvial soil or rock). Therefore, no other San Fernando earthquake records can provide the information required. The Imperial Valley earthquake of 1979 (approximately 6.5 M) had several near-field records which have reasonably similar distances to their causative fault as compared to that of Pacoima Dam to its causative fault. Testimony was provided that the Bond's Corner response spectrum (the highest near-field peak acceleration recorded during the Imperial Valley earthquake) was quite similar to Pacoima Dam. We have been unable, however, to locate on the record a composite response spectrum for all the applicable Imperial Valley near-field records in order to compare them to Pacoima Dam. Imperial Valley is a soil site while Pacoima Dam is a rock site.

GROUND MOTION AT SOIL AND ROCK SITES: <u>In the near field of earthquakes of comparable magnitude, would ground motion on a rock site be greater than or similar to motion on a soil site?</u>

In developing the design spectrum for Diablo Canyon, Dr. Newmark utilized a comparison of the Pacoima Dam record (on rock) to his design spectrum as evidence supporting the adequacy of the Diablo Canyon design.

Governor Brown and the Joint Intervenors contended that in the near field of earthquakes of comparable magnitude, ground motion on a rock site would be greater than on a soil site (Governor Brown Finding of Fact, Joint Intervenors Finding of Fact, Tr. 863-64). Therefore, the values in the Bond's Corner record, which was recorded on deep soil, would have been higher if the recording had been done on rock. Because the Bond's Corner and Pacoima Dam records are similar, the Intervenors contended that the Bond's Corner record adjusted to rock would exceed the Newmark spectrum and, therefore, the Newmark spectrum with a 0.75 g anchor point is not adequate for Diablo Canyon. The reports specifically put into evidence by the applicant and staff in this regard (USGS 795, TERA Report, and NUREG-1175) do not agree with the Intervenors' position. Dr. Blume, a witness for the applicant, provided additional evidence using near-field data (Tr. 6650-51), that for comparable seismic events, peak accelerations measured on rock would be generally equivalent to or less than those measured on soil. However, other reports not entered into evidence on this point, but referenced by Newmark (Donovan* and Ambraseys**), indicate that rock sites have higher accelerations in the near field than soil sites. On the basis of the evidence in the record, the Appeal Board concluded that near-field rock and soil accelerations should be expected to be about equal. The Appeal Board supported the NRC staff and applicant, who cautioned that it would be inappropriate to use the Imperial Valley data to predict the absolute value of ground motion at Diablo

*Donovan, Neville C., "A Statistical Evaluation of Strong Motion Data Including the February 9, 1971 San Fernando Earthquake," Proceedings - Fifth World Conference on Earthquake Engineering, Volume 1, pp. 1252-1261, June 1973 (published April 1974).

**Ambraseys, N., "Trends in Engineering Seismology In Europe," Invited Lecture, Fifth Conference, European Committee for Earthquake Engineering, Istanbul, September 1975. (Note: Newmark misquoted the title of Ambraseys' paper.)

Canyon because of the geological differences between the two sites. The Board also concluded that the Imperial Valley data are important because they include a large number of strong-motion records.

Based on the evidence on the record we believe that the Appeal Board's conclusion, that the near-field rock and soil accelerations should be about equal, is reasonable.

RATIO OF VERTICAL TO HORIZONTAL PEAK ACCELERATIONS: Is the assumption that the vertical peak acceleration is 2/3 of the horizontal peak acceleration consistent with actual data (e.g., from the Imperial Valley earthquake) and adequately conservative for Diablo Canyon?

An NRC staff witness, Dr. Kuo, testified before the Appeal Board that on the basis of an unpublished NRC branch technical position, applicants in the western U.S. were given the option of using a 2/3 (vertical-to-horizontal) ratio for peak acceleration in lieu of Regulatory Guide 1.60. The branch position resulted from conclusions reached by Dr. Newmark as a result of his study of an extensive compilation of earthquake records.

Governor Brown and the Joint Intervenors presented evidence to support their contention that peak vertical acceleration is equal to or higher than horizontal acceleration for frequencies exceeding 10 Hz, as shown by the Imperial Valley earthquake. Drs. Trifunac and Luco felt this phenomenon should be investigated more fully in the near field (Tr. 1138).

A witness for the applicant, Mr. Hanusiak, testified before the Licensing Board (Tr. 7045, 7048-51) that stress in the containment shell due to vertical seismic excitation is only 1.9 percent of the total stress.

Based on the testimony in the record, the Appeal Board concluded that even if vertical accelerations were equal in magnitude to those in the horizontal direction (increased by 50 percent over design levels), the resulting increase in total calculated stress would be only 1 percent (Tr. 7050). Therefore, the Appeal Board concluded that the vertical motion phenomenon will have no significant consequence for Diablo Canyon. We agree with the Appeal Board.

· Development of the Design Response Spectrum

USE OF 0.75 g AS EFFECTIVE ACCELERATION FOR ANCHORING DESIGN SPECTRUM: Is the use of 0.75 g as an effective acceleration instead of peak acceleration values in anchoring the seismic design spectrum an adequately conservative procedure?

A key issue with regard to the seismic design of Diablo Canyon was whether the 0.75 g effective acceleration value used to anchor the design spectrum as proposed by the NRC staff and its consultant, Dr. Newmark, is adequately conservative. During both the Licensing Board and Appeal Board hearings, Governor Brown contended that the use of "effective acceleration" was not within the meaning of Appendix A to 10 CFR Part 100. The Joint Intervenors challenged the use of "effective acceleration" on technical grounds, stating specifically that although a physical explanation for effective acceleration has been given in testimony, no physical theory has been used to reduce peak acceleration to an effective acceleration.

During an earthquake, the energy released causes the earth to oscillate, which, in turn, causes ground acceleration, velocity, and displacement. The acceleration, velocity, and displacement not only vary with time, but they reverse directions. Accelerographs record three perpendicular components of the ground motion and how it varies with time. This record is called a time history. The peak or maximum acceleration is the highest acceleration value recorded by the accelerograph during the entire record of strong motion, and it usually is represented by an extremely short duration spike on a time history. The "effective acceleration" which has been proposed for use in developing the design criteria for Diablo Canyon is equivalent to eliminating the effects of the high acceleration peaks from the time history. Narrow spikes of peak acceleration are low in energy and are not likely to substantially affect large structures. With regard to Diablo Canyon, the NRC staff used the 0.75 g effective acceleration derived by its consultant, Dr. Newmark, to anchor the response spectrum from a 1.15 g peak acceleration value for a 7.5 M event on the Hosgri fault (Tr. 8550, Joint Intervenors' Ex. 45).

Dr. Blume, an applicant witness, presented evidence for the use of effective acceleration in design during the Licensing Board hearings (following TR 6099,

pp. 21-25; DLL-26*). Dr. Blume's rationale relied on four observations which are not mutually independent: (1) the observation that damage during large earthquakes does not correspond to the damage expected from the peak accelerations recorded; (2) there exist safety margins inherent to design theory, analysis, testing, and experiments that are not considered in design, e.g., material strength and smoothed rather than actual response spectra (DLL-5, 6, 9, 18B, 18C, 26, 36); (3) engineering judgment based on consideration of (1) and (2) above and overall experience; and (4) evidence showing that response spectra developed from accelerograms are not significantly altered by modifying the accelerograms to remove the higher frequency peaks. Dr. Blume also testified that in his opinion a 0.6 g acceleration was adequately conservative for Diablo Canyon (Licensing Board Partial Initial Decision, p. 59).

The NRC staff's major testimony in this regard was provided by Dr. Newmark. He showed that the seismic design for Diablo Canyon (anchored at 0.75 g) generally enveloped the Pacoima Dam record, which had a peak acceleration of 1.2 g (Tr. 5846-48, 8638-39). In addition, the USGS agreed with the use of effective acceleration for engineering design (USGS Circular 672, Tr. 8330-31); however, the USGS witness declined to define "effective acceleration" to be used in connection with USGS Circular 672.

The Intervenors' contended that the use of 0.75 g effective acceleration for the seismic design of Diablo Canyon was not conservative based on legal and technical considerations. The Intervenors presented legal arguments that use of effective acceleration did not meet the requirements of Appendix A to 10 CFR Part 100. The Intervenors contended that Appendix A requires the use of the maximum vibratory ground motion. The technical objection of the Intervenors, advanced by the Appeal Board witnesses Drs. Luco and Trifunac, was that no physical methodology exists for a reduction from peak measured acceleration to effective acceleration (Tr. 8867-95, 8973). The only argument for the existence

*Refers to a series of reports prepared by the applicant on specific topics and submitted as part of Appendix D to Amendments 50 and 53 to the Diablo Canyon Final Safety Analysis Report. DLL identifies a report in the series; the numeral following, the specific report number.

of effective acceleration which Dr. Luco considered to have any merit is that there have been buildings subjected to earthquakes for which calculations based on measured peak accelerations would predict failure but which, in fact, did not fail.

Dr. Newmark was the principal architect of the design response spectrum for Diablo Canyon. In Newmark's 1976 paper to the staff,* he discussed two major bases (the paper by Neville Donovan cited in our analysis and the Pacoima Dam record) and two supporting ones (USGS Circular 672 and ductility) for his choice of a response spectrum for Diablo Canyon.

The Appeal and Licensing Boards placed strong emphasis on the report proffered by Newmark into evidence on the appropriate anchor point for the Diablo Canyon seismic response spectrum. Dr. Newmark proposed a value of 0.75 g as appropriate for a 7.5 M earthquake on the Hosgri fault, which is 5.8 km from the plant. According to the records, the Boards were unable to reconstruct these calculations. In our review of the Diablo Canyon seismic question, we believe we have duplicated Dr. Newmark's calculation of 0.75 g.

Dr. Newmark, in his 1976 report to the staff, cited a paper by Dr. Donovan (following Tr. 8552, Newmark's testimony, p. C-3) which provided the basis for Newmark's "attenuation of maximum ground acceleration as a function of magnitude and hypercentral distance from the source." Donovan's report included data from the 1971 San Fernando earthquake (6.5 M). Donovan used the empirical relation

$$y = b_1 \exp (b_2 m)(R + 25)^{-b_3} \qquad (1)$$

to predict mean peak horizontal accelerations. The parameter y is acceleration in gals (1 gal = 1 cm/sec^2, 980 gal = 1 g), m is the instrumentally measured earthquake magnitude, R is the distance to the earthquake source in kilometers,**

*Following Tr. 8552, Newmark, Nathan M., "A Rationale for Development of Design Criteria for Diablo Canyon Reactor Facility - A Report to the USNRC," September 3, 1976.

**In the far field, epicentral and hyperfocal distances are very similar. Donovan defines the distance to the energy center of an earthquake as the hyperfocal distance. However, in the near field, Donovan indicates setting R equal to the epicentral distance may be more appropriate.

and b_1, b_2, and b_3 are empirical constants. Newmark stated: "...(w)ith this relationship, involving an exponent of decay of acceleration with distance of -1.32 and a geometric standard deviation of 2.0, the maximum ground acceleration for one standard deviation from the median is approximately 0.75 g, for a horizontal distance of 7 km and a focal depth of 12 km from the earthquake source."* Newmark was apparently referring to equation 3 of Donovan's paper:

$$y = 1080 \exp (0.5\ m)(R + 25)^{-1.32} \quad . \qquad (2)$$

We calculated the distance (along the hypotenuse of a triangle) to the earthquake source using $R = (7^2 + 12^2)^{0.5} = 13.89$ and set $m = 7.5$. Using these values in equation 2 gives

$$y = 365.97 \text{ gals} = 365.97 \frac{\text{cm}}{\text{sec}^2} \doteq 0.373 \text{ g} \quad .$$

From Donovan's report, we are given that the log normal standard deviation of equation 2 above is 0.71 (the antilog of Donovan's 0.71 is Newmark's geometric standard deviation 2, so this also correlates). To find the mean +1 standard deviation, we solve the following for X:

$$\ln (0.373) + 0.71 = \ln X$$

then,

$$X = 0.759 \text{ g} \cong 0.75 \text{ g} \quad .$$

We believe this is the probable calculational path pursued by Dr. Newmark which the Boards were unable to reconstruct.

There are some questions about Newmark's use of Donovan's numerical relationships:

*We note that 5.8 km rather than 7 km is the epicentral distance accepted by the Boards for Diablo Canyon, and 10 km (plus or minus) represents the average focal depth of earthquakes in California. Newmark has chosen to input slightly larger distances.

(1) Donovan specifically states that "(a)ttenuation comparisons between magnitude and distance cannot be extrapolated to distances close to the source."* Also, the least-squares calculational technique Donovan used to derive his correlation best fits data which are near the mean; the technique worst fits data near the end points. Diablo is near an extreme of the correlation. It appears from the figures in Donovan's paper that there is only one recorded strong motion data point (a rock site, probably Pacoima Dam; Figure 6) in his data set within 17 km of earthquake epicenters.

(2) There are differing opinions on and off the record about the appropriate distance to input as R in the near field into equation 2 above from Donovan's paper. This is especially important, given the unanimous agreement on the record of near-field distance saturation for an earthquake of a given magnitude. Donovan states the following:

> Where known, the distance to the energy center [for far-field events] is considered a better measure of distance for moderate sized earthquakes than the closest distance to the causative fault. For large events where a fault-rupture of many tens of kilometers is involved, the closest distance to the fault may be more appropriate.*

The choice of an appropriate R can make a considerable difference in the value of the mean +1 standard deviation acceleration used for a response spectrum anchor. An R = 13.89 km value as used by Newmark gives about 0.75 g, while an R = 5.8 km value (the closest horizontal distance to the Hosgri fault) gives a mean +1 standard deviation acceleration from equation 2 of 1.03 g.

*Several witnesses testified that extrapolation of data from the far field to the near field tends to overestimate the accelerations in the near field. From our reading of a paper (Ambraseys, N. N., "Trends in Engineering Seismology in Europe," Fifth European Conference on Earthquake Engineering, Istanbul, 1975) referenced in Dr. Newmark's testimony to the Licensing Board (Tr. 8553), we quote:

> Existing attenuation laws are inadequate to predict peak ground motions near the source of a medium-magnitude earthquake ($M_1 \leq 5.0$).
>
> Maximum ground accelerations and velocities recorded during the last few years in the near-field of moderate magnitude earthquakes, but not used by Esteva and others, are significantly underestimated by almost all empirical formulae commonly used in earthquake engineering.

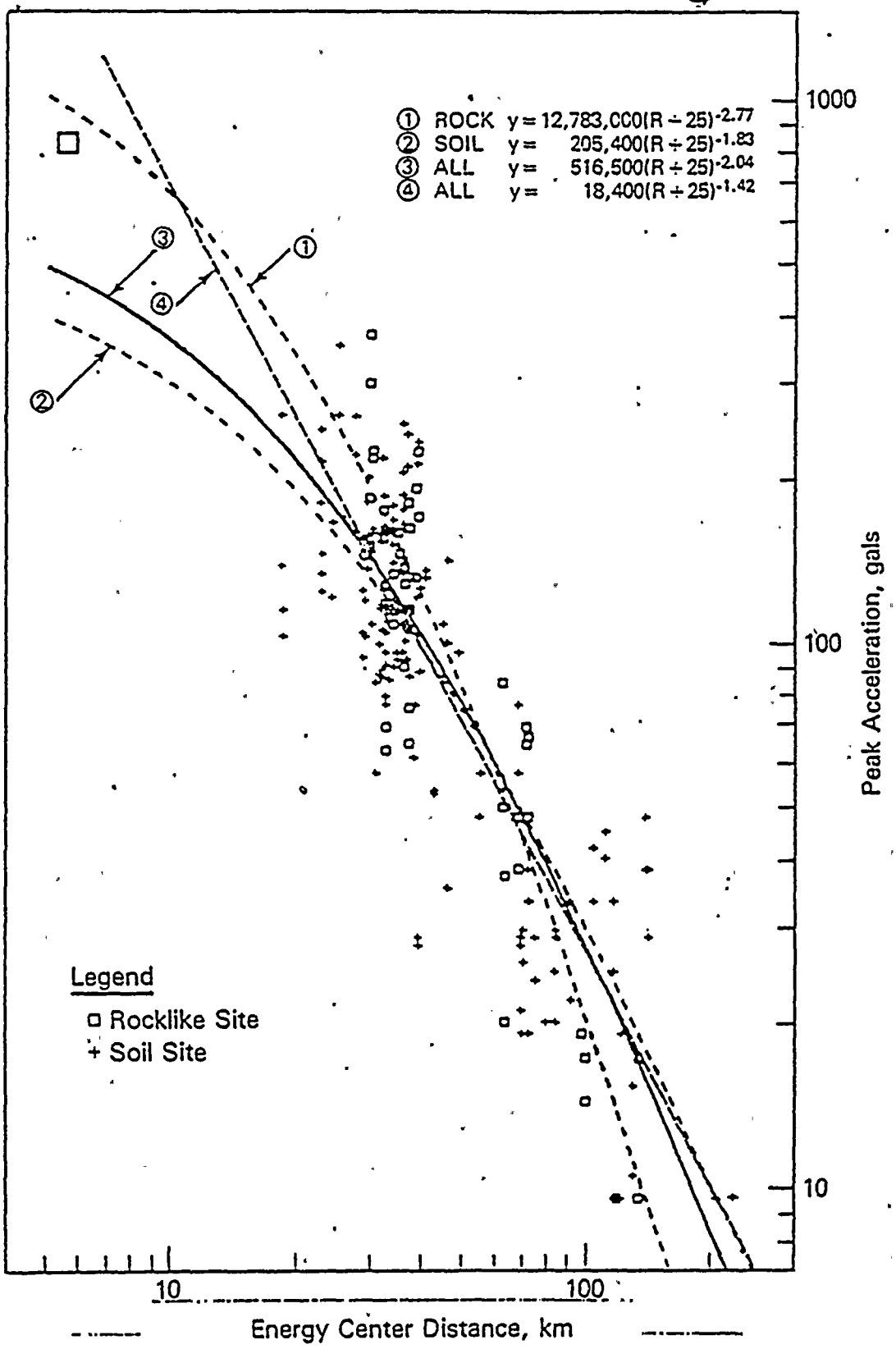

Figure 6 Least Squares Fit to Peak Ground Accelerations. February 9, 1971 San Fernando Earthquake.

Source: Donovan, N. C., "A Statistical Evaluation of Strong Motion Data", Proceedings, Fifth World Conference on Earthquake Engineering," Vol. 1, April 1974.

It is unclear which value of R is more appropriate in the near field for Diablo Canyon.

(3) The value of b_3 chosen to be input into equation 1 by Dr. Newmark was 1.32. Donovan's paper indicates that 1.32 corresponds to a soil site and, in particular, equation 2 above "represents a conservative estimate of mean peak acceleration on sites with 20 feet or more of soil overlying the rock." He also notes that at large acceleration levels, higher damping and lower strength values of soil reduce soil motion, thereby preventing extrapolation of measured site response from small earthquakes to large ones. Donovan says that rock sites will have higher acceleration than soil sites as the distance to the epicenter decreases.

In particular, Donovan's paper gives one equation for rock sites for a 6.5 M earthquake: $y = 12{,}783{,}000\ (R + 25)^{-2.77}$. Using R = 13.89 km, the mean +1 standard deviation for a rock site is 1.05 g. Using R = 5.8 km, the mean +1 standard deviation is 2.00 g. Testimony was given by Brune that peak accelerations as high as 2.0 g are possible due to focusing (Tr. Joint Intervenors' Ex. 66, p. 3-2). Testimony was also given that peak accelerations in the range of 2.0 g are physically unrealistic. We note that Diablo Canyon is anchored on rock.

Newmark also proposed two areas to support his contention that 0.75 g was an adequate anchor point for a Diablo Canyon design spectrum.

- Newmark concluded that a 0.75 g anchor point "...is not inconsistent with the values in USGS Circular 672 (reference deleted) for near-field strong motions, considering a repeated acceleration peak of several times, rather than one isolated peak." We note on referring to USGS Circular 672 (Table 2) it is apparent that the "several" peaks Newmark was referring to was the seventh or eighth highest peak. That is, he neglected the first six peak accelerations in the USGS report for a 7.5 M earthquake when indicating that his computation of 0.75 g was not inconsistent with the USGS (Tr. 6074). Maximum peaks of short duration impart little energy to a structure.

Table 2.—Near-fault horizontal ground motion

Magnitude	Acceleration (g) Peak absolute values				Velocity (cm/sec) Peak absolute values			Displacement (cm)	Duration[1] (sec)
	1st	2d	5th	10th	1st	2d	3d		
8.5	1.25	1.15	1.00	0.75	150	130	110	100	90
8.0	1.20	1.10	0.95	0.70	145	125	105	85	60
7.5	1.15	1.00	0.85	0.65	135	115	100	70	40
7.0	1.05	0.90	0.75	0.55	120	100	85	55	25
6.5	*0.90*	*0.75*	*0.60*	*0.45*	*100*	*80*	*70*	*40*	*17*
5.5	*0.45*	*0.30*	*0.20*	*0.15*	*50*	*40*	*30*	*15*	*10*

[1]Time interval between first and last peaks of absolute acceleration equal to or greater than 0.05 g.

Notes—1. Italic values are based on instrumental data.
The values in this table are for a single horizontal component of motion at a distance of a few (3-5) km of the causative fault; are for sites at which ground motion is not strongly altered by extreme contrasts in the elastic properties within the local geologic section or by the presence of structures; and contain no factor relating to the nature or importance of the structure being designed.

2. The values of acceleration may be exceeded if there is appreciable high-frequency (higher than 8 Hz) energy.
The values of displacement are for dynamic ground displacements from which spectral components with periods greater than 10 to 15 seconds are removed.

Source: USGS Circular 672

- Newmark also discusses the relationship between the response spectrum he has developed for Diablo Canyon and "...actual response of a nonlinear or inelastic structure." He discusses how the ductility of the structure provides margin beyond the design rating (0.75 g) because the building could yield at high stress points.

Blume ("Seismic Evaluation for Postulated 7.5 M Hosgri Earthquake," p. D11.7 and Appendix D11B) and Newmark (following Tr. 5882), working with similar data and equations, seemingly claim to calculate different "types" of acceleration. Newmark uses Donovan's equation 1 above to calculate an "effective acceleration" of 0.75 g for Diablo Canyon. Blume, using an equation form identical to Donovan, data similar to Donovan,* and coefficients modified to account for site conditions, instead calculates an estimated peak (instrumental) acceleration of 0.89 g. Blume's calculation assumes an earthquake wave velocity at Diablo Canyon of 5300 ft/sec (Tr. 6710). In Blume's calculations, the higher the velocity, the lower the calculated peak acceleration. In another calculation, Blume uses a wave velocity of 3750 ft/sec (Tr. 6096).

Governor Brown and the Joint Intervenors challenged the adequacy of the Newmark design spectrum in light of actual ground motion records, which exceeded the Newmark spectra for certain frequencies. During the initial Licensing Board hearing, evidence was presented by all parties regarding the adequacy of the Newmark spectrum in comparison to the Pacoima Dam record of the 6.5 M 1971 San Fernando earthquake. During the reopened Appeal Board hearing, similar arguments were made by Governor Brown and the Joint Intervenors with regard to the Bond's Corner record of the 6.5 M 1979 Imperial Valley earthquake (Joint Intervenors Proposed Findings of Fact, Governor Brown's Proposed Findings of Fact).

Dr. Newmark, the NRC staff consultant, in comparing the Bond's Corner record with the design spectrum for Diablo Canyon, indicated that the Bond's Corner record of the 1979 Imperial Valley earthquake gives results not greater than used in the Diablo Canyon design spectrum.

*Blume's data base does not include the 1971 San Fernando earthquake; it was unavailable at the time of his analysis and he feels it might bias the results.

Evidence presented by Dr. Blume for the applicant indicated that a large disparity occurs when the Bond's Corner record is compared to the other near-field records for the Imperial Valley earthquake (Tr. 185). However, an NRC staff witness indicated that no geological attributes similar to Pacoima Dam have been noted which explain the large magnitude of the record at Bond's Corner (Tr. 563-64). We note that Bond's Corner has not been so widely studied as Pacoima Dam. Dr. Blume also testified that spectra derived by his methodology represented the proper way to incorporate available data and assess the Diablo Canyon design. He felt the use of a single high-acceleration record such as Bond's Corner would statistically misrepresent the data.

The Appeal Board concluded that both Pacoima Dam and Bond's Corner represent distorted records. Considering the distortions, the Appeal Board found that the Newmark spectrum is approximately equivalent and therefore compatible to both records. Based on this rationale and on the earlier findings on magnitude saturation, the Board found that the fact that the Pacoima Dam and Bond's Corner records exceeded the Newmark response spectrum was not cause to find the Newmark spectrum an unconservative representative of the SSE proposed for Diablo Canyon.

The second basis discussed by Newmark concerned the Pacoima Dam record from the 1971 San Fernando earthquake. The Pacoima Dam record shows the largest horizontal ground motion acceleration ever recorded (approximately 1.2 g), although 1979 Imperial Valley, California and Gazli, Russia both had recorded vertical peak accelerations of larger magnitude than Pacoima Dam's horizontal acceleration. The San Fernando earthquake was measured as a 6.5 M earthquake. A number of seismologists believe that the Pacoima Dam record peak acceleration is an anomaly which was amplified by the physical location (a ridge) on which the Pacoima Dam recording device was attached. Pacoima Dam provides the only record of the San Fernando earthquake closer than 17 km of the epicenter. Therefore, it is the only near-field record available from this earthquake.

Dr. Newmark proposed that the Pacoima Dam record represented the record of an earthquake substantially larger than 6.5 M. In particular, he implied that it represents a 7.5 M earthquake and is appropriately conservative to use for Diablo Canyon given the existence of the Hosgri fault. Newmark then used the response

spectrum for Pacoima Dam to act as a guide for the Diablo Canyon design spectrum. We believe Newmark took a slightly modified Regulatory Guide 1.60 design spectrum shape and slid it up the ordinate (y axis) until it bounded the Pacoima Dam response spectrum in the region from 2-10 Hz. This anchored the Regulatory Guide 1.60 design spectrum at 0.75 g (Figure 7). The design spectrum enveloped the Pacoima Dam spectrum everywhere but at about 5 Hz, 7-8 Hz, and higher than 13 Hz.

Inherent in Newmark's argument is an assumption that the Pacoima Dam record can be used to represent a 7.5 M earthquake, even though the associated San Fernando earthquake is not a 7.5 M event. However, there is evidence in the record that the Pacoima Dam record represents an unusually high peak acceleration for a 6.5 M earthquake. Newmark relates magnitude saturation in the near field and his opinion that the Pacoima record represents an earthquake larger than the San Fernando earthquake, and then asserts that his design spectrum is, therefore, adequate.

Dr. Blume provided an independently calculated response spectrum anchored at 0.75 g applicable to Diablo Canyon (following Tr. 6099, pp. 40-41). He combined and scaled eight close by, rocky-site records for the largest earthquakes recorded under such conditions at the time of his analysis. He indicated he would have used more records if other good records were available. Blume felt this technique represented an excellent model for the important higher frequencies of the spectral diagrams. Blume's response spectrum is bounded over most of the frequency range by Newmark's spectrum (see Figure 8). Newmark's spectrum allows for only elastic deformation, while Blume's spectrum allows structure deformation 30 percent greater than yield point deformation. We note that the acceptability of Blume's spectrum greatly depends on the acceptability of the 0.75 g anchor point to which it was scaled.

The Appeal Board's decision held that the peak acceleration for the SSE is not necessarily the "maximum vibratory acceleration for the purposes of Appendix A (to 10 CFR 100)." The Appeal Board also supported the use of effective acceleration as a physically valid and acceptable procedure for structures in the near field. The Appeal Board found particular confirmation in that the Newmark

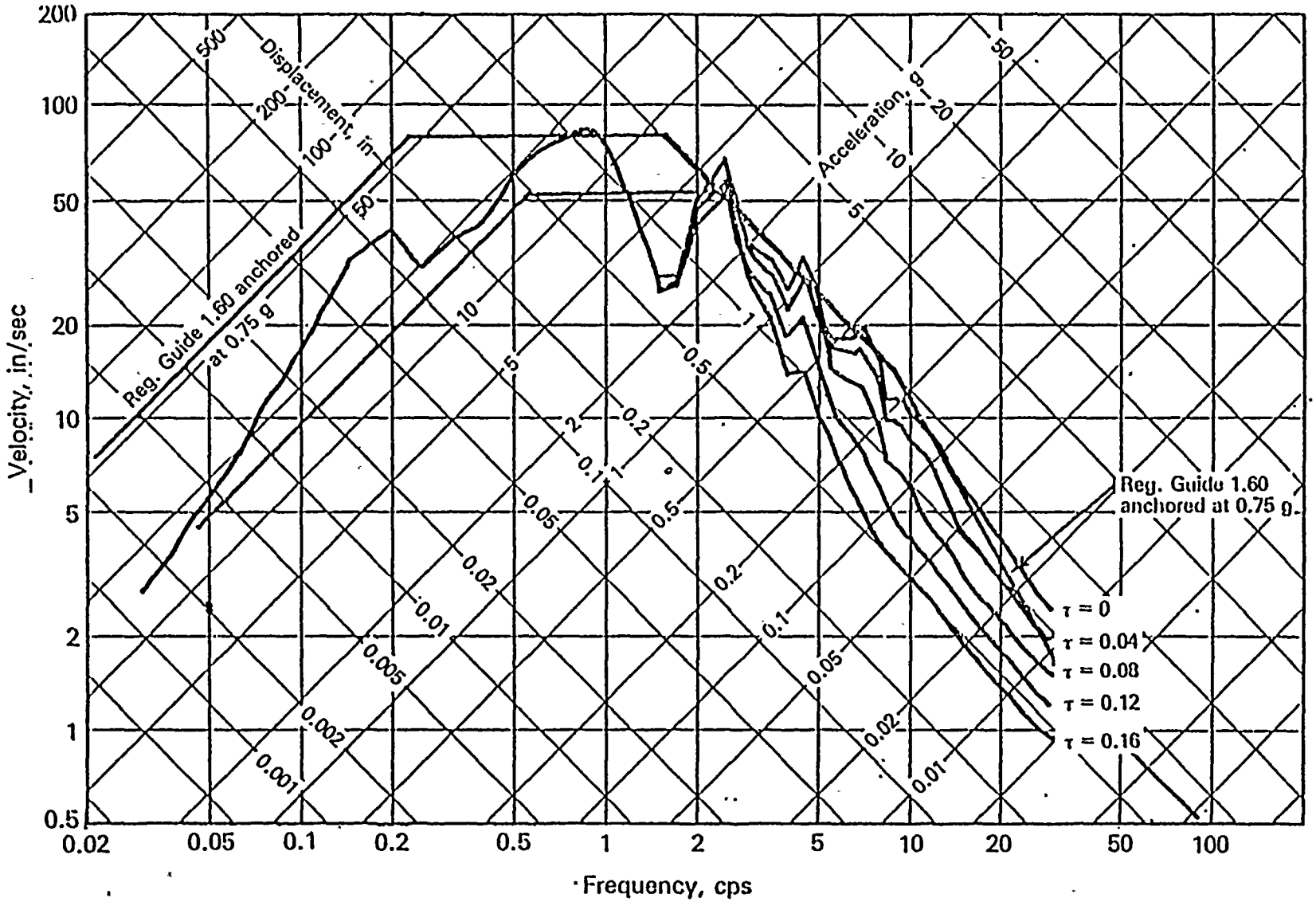

Figure 7 Pacoima Dam Response Spectrum 9 Feb. 1971, S16E, Damping 5% of Critical, τ = 0, 0.04, 0.08, 0.12, 0.16 Sec. Compared with Design Spectra

Source: Dr. Newmark's testimony

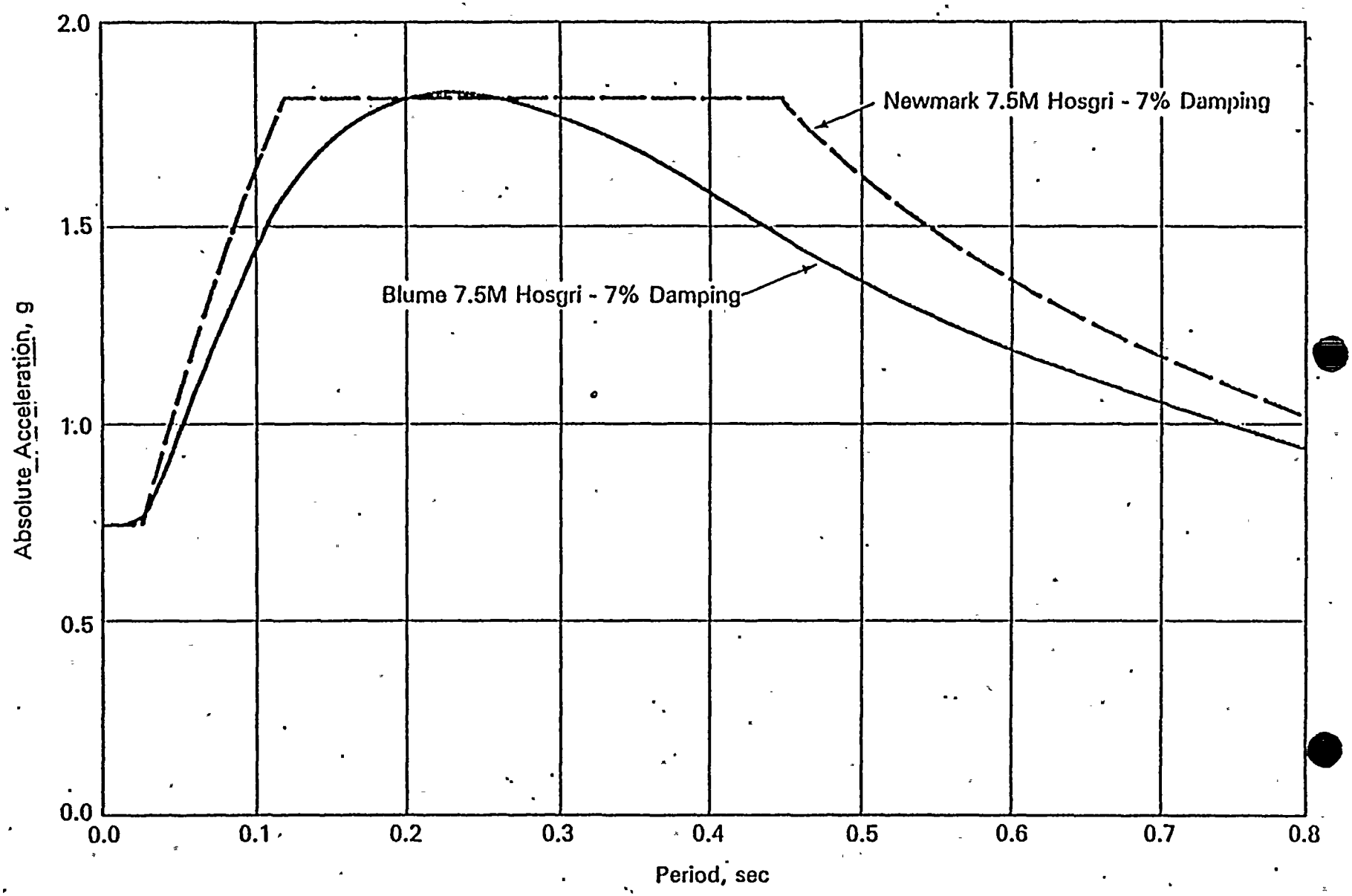

Figure 8 Diablo Canyon Units 1 and 2 Design Spectra

Source: Dr. Blume's testimony.

spectrum, anchored at 0.75 g, essentially enveloped the Pacoima Dam response spectrum, which had a peak acceleration of 1.2 g.

From the substantial portions of the record we have studied, we believe there is insufficient evidence in the record except for Dr. Newmark's engineering judgment to allow us to conclude that 0.75 g is the appropriate anchor point for a 7.5 M earthquake on the Hosgri fault, 5.8 km from Diablo Canyon. The following weigh heavily in our judgment:

- the input values and assumptions we believe Newmark used in Donovan's equation to predict the "effective acceleration"

- the similar data and techniques used by Newmark to calculate an "effective acceleration" and used by Blume to calculate peak acceleration

- the uncertainty about the degree of magnitude saturation in the near field

- the discussions on the record for magnitude saturation which almost exclusively focused on accelerations near 33 Hz rather than the 1-10 Hz range

- the uncertainties whether the Pacoima Dam record, with accelerations between 1 and 10 Hz, represents a 6.5 M or 7.5 M earthquake.

This does not necessarily indicate that 0.75 g is an inappropriate value, just that it is difficult to demonstrate its validity from Newmark's use of Donovan's paper or the Pacoima Dam record.

THE TAU EFFECT: Is it justifiable to account for observations that the damage attributable to high frequency components of acceleration is sometimes less than expected by postulating a "tau effect" and, if so, how should the effect be quantified?

The most rudimentary approach to designing buildings to resist earthquakes involves approximating the inertial forces developed in the building by static forces assumed to act horizontally on the building. This method, often referred to as the quasi-static or Uniform Building Code (UBC) approach, is still routinely used for earthquake design of minor buildings. However, major buildings, including those at nuclear power plants, are usually designed by methods incorporating concepts of dynamic analysis. This is necessary to account more accurately for the distribution of forces throughout the building.

Current structural engineering practice, when estimating the dynamic forces associated with an earthquake, utilizes admitted oversimplifications of the physical phenomena. The building is represented by a "stick model," i.e., a linear element with concentrated masses used to represent the mass distribution of the building (Figure 9). The earthquake is assumed to excite the building by causing motion of the rigid base used to represent the foundation. Representative earthquake motions for use with this model can be, and have been, derived from earthquake records measured at surface stations. Both the model and the limitation that data are measured only on the ground surface neglect the three-dimensional nature of both the building and its foundation; the foundation is effectively treated as a point. However, the model is a significant improvement over past practice which did not explicitly account for the dynamic nature of earthquake response.

In recent years, attempts have been made to improve dynamic modelling techniques. In particular, it has been observed that, whatever the free-field earthquake motion measured at the surface in the vicinity of a building, the motion of a foundation, usually at some depth below the ground surface, will not be the same (Figure 10). Because the building is actually excited by the motion of its foundation, research in recent years has focused on the development of methods to predict foundation motions which are consistent with measured free-field (surface) motions. Efforts have been made to check the utility of a

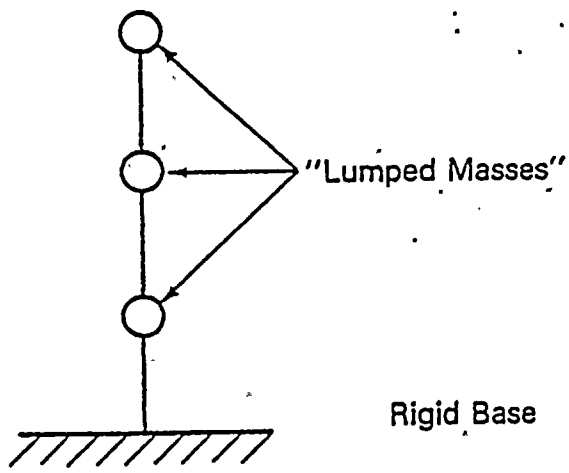

Figure 9 "Stick Model" of a Building

Source: OPE

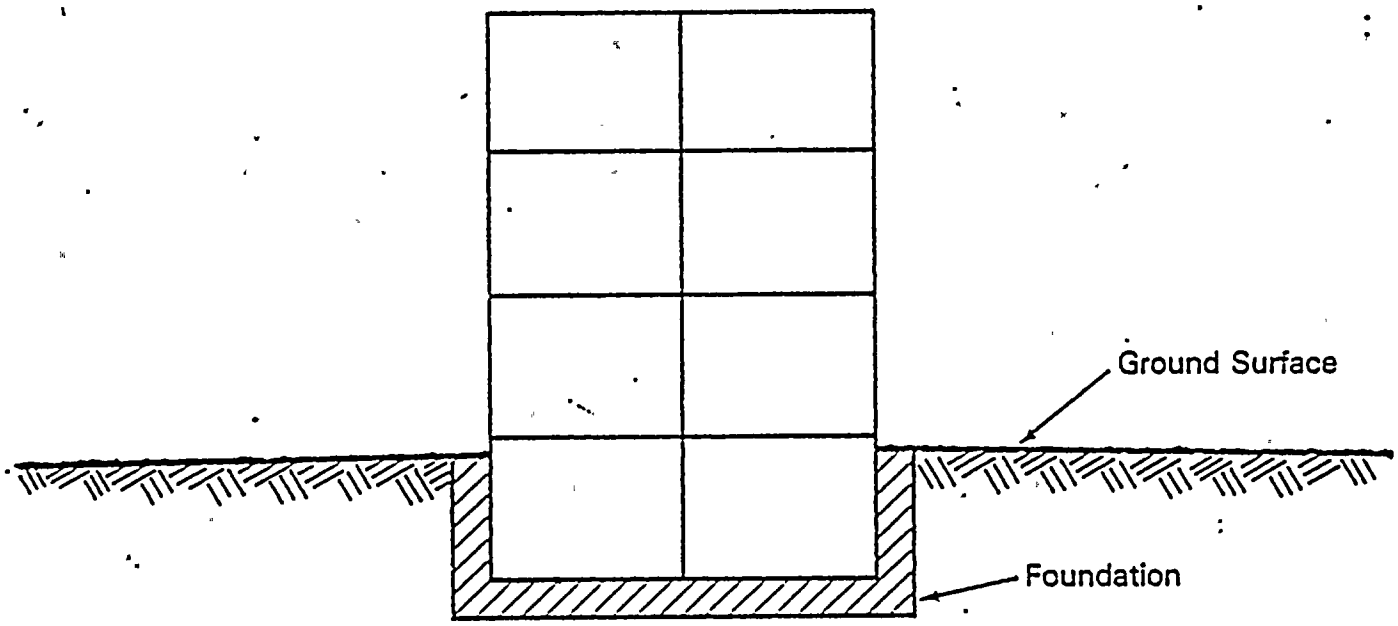

Figure 10 Planar Model of Building and Foundation

Source: OPE

number of these improved models against the relatively small amount of available and relevant data. These comparisons have been noted in ALAB-644.*

The name "tau effect" has been used to describe methods of reducing the high frequency acceleration portion of a free-field response spectrum to account for observations that even when peak ground accelerations in excess of design values are recorded, structures do not necessarily fail or suffer significant damage. We note that this observation was also applied to partly explain why peak accelerations should be reduced to "effective accelerations." Because of these observations, several seismologists have attempted to explain the discrepancy between expected and observed structural damage. Based on the record, we believe that the phenomenon called the tau effect probably exists. We are unclear, however, as to the entire physical explanation of this effect as it relates to propagation of shear waves from the earthquake source through a rigid foundation.

Among the witnesses who testified in the hearings and the references on the record which we checked, we found that at least two methods are used to determine a tau effect: one includes characteristic building length and wave transit time; the other, soil-structure interaction. Some experts insist tau is a function of horizontally propagating seismic waves; other experts insist tau is the result of vertically propagating seismic waves. In general, the view seems to be that wave incoherence impairs the ability of the ground to transmit high frequency accelerations to a building through a large rigid foundation.**

Just as data in the near field for large earthquakes are limited, so are the analyses of paired records of a building foundation and the free field near the building. The Diablo Canyon record refers to only three sets of data points:

*Dr. Newmark's model was discussed on pp. 119-123. Dr. Yamahara's model was discussed on p. 124. Dr. Seed's model was discussed on pp. 142-145. These models differ in the mechanisms they use to predict foundation motions.

**This is like looking at a sine wave traveling through time where different parts of a building would see different parts of the sine wave passing underneath at any instant, depending on the length and velocity of the wave and the dimensions of the building.

Yamahara's basementless building in Japan (Hachinoke Technical College (HTC)), the Hollywood Storage Building (HSB), and the Imperial County Services Building (ICSB).

The HTC building* showed a definite pattern of reduction between the free-field ground acceleration and building foundation acceleration. This building does not appear to be typical of a power plant (odd shape, no basement, less rigid, less mass) (Figure 11).

The HSB (following Tr. 8552, p. 11) is approximately 150 feet high, 51 feet wide, and 218 feet long and has been described as having a rigid foundation. Unlike the HTC building, the HSB has a basement. Accelerographs are located in the building basement and in a parking lot about 110 feet away. Records from several earthquakes recorded at the HSB and its parking lot showed a definite pattern of reduction between the free-field ground and building foundation accelerations.

We have uncovered little data on the record on the ICSB. The applicant claims the building did not have a very rigid foundation. The building sustained heavy damage during the 1979 Imperial Valley earthquake. Comparisons of the building accelerograms with those taken in the free field showed that the building did not reduce accelerations and may, in fact, have amplified them. The applicant presented testimony why the ICSB did not show a tau effect on high frequency accelerations.

In making its decision on tau, the Appeal Board relied most heavily upon the testimony of Newmark. Newmark in turn referenced Yamahara, Ambraseys, and Scanlan on tau effects (Newmark, following Tr. 8552, p. 610). The building Yamahara studied was long and oddly shaped with no basement (Figure 11). From the information available to us, we are unable to determine if Yamahara and Newmark used identical techniques to reduce the calculated input ground motion to a building. They both used a characteristic building length (long side of building (Yamahara), square root of area of building basement (Newmark)) and

*Yamahara, H., "Ground Motions During Earthquakes and the Input Loss of Earthquake Power to an Excitation of Buildings, Soils, and Foundations," Vol. 10, No. 2, pp. 145-61, 1970, Tokyo.

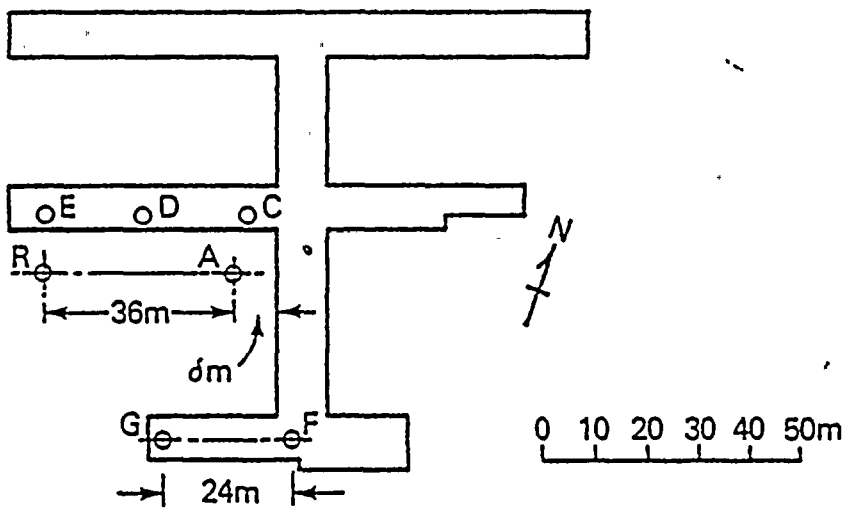

Figure 11 Locations of Measured Points in Hachinoke Technical College

Source: Yamahara, H., "Ground Motions During Earthquakes and the Input Loss of Earthquake Power to an Excitation of Buildings, Soil and Foundations," Vol. 10, No. 2, 1970, Tokyo.

wave transit time. The maximum ground acceleration for the earth tremors Yamahara studied (based on the samples of data given in his paper) were from 0.025 g to 0.1 g. No discussion is provided by Newmark regarding the applicability of scaling up from low-g earthquakes. Nor did Newmark discuss if any of his or Yamahara's results, other than the existence of some sort of "tau effect," were confirmed by each other's work.

Ambraseys' paper discusses a simple tau-like "structural interaction" which he notes has "many questionable assumptions." Without providing any analysis or data or quoting any references, he introduces a diagram which relates the ratio of accelerations of a slab and free-field motion to k, a function of foundation length, wave frequency, and wave velocity. Scanlan's paper* concerns soil-structure interaction. Scanlan models his building as "resting on the top of the soil." (Diablo Canyon is anchored to rock and not soil.) Scanlan also notes in his conclusion that his model is oversimplified and additional refinements and analysis need to be made. Although neither paper provides the basis for Newmark's approach, the observations in both papers compare with Newmark's.

Dr. Blume offered testimony (following Tr. 6100) for the applicant concerning his independent calculation of a tau effect for Diablo Canyon. Blume provided little information in his testimony concerning the methodology and assumptions used in determining the appropriate value of tau for Diablo Canyon. He did reference a paper by Ray and Jhaveri** which provided an analytical technique for calculating tau. But as the Appeal Board noted, no basis was provided in this paper or by Blume as to why this approach to calculating tau was applicable to the real world and Diablo Canyon in particular. He also discussed the tau effect in connection with his justification of why 0.75 g was an acceptable "effective acceleration" ("Seismic Acceleration for Postulated 7.5 M Hosgri Earthquakes," pp. D 26.10-11). We note that Blume appears to have assumed earthquake wave velocities which are low when calculating the tau effect. This increases the tau effect. Blume assumed high wave velocities when calculating peak accelerations. This decreases the peak (Tr. 6096, 6710).

*Scanlan, R. H., "Seismic Wave Effects on Soil-Structure Interaction," Earthquake Engineering and Structural Dynamics, Vol. 4, 1976, pp. 379-388.

**Ray, D. and D. Jhaveri, "Effective Seismic Input Through Rigid Foundation Filtering," Nuclear Engineering and Design, Vol. 45 (1978), pp. 185-95.

Dr. Seed, a witness for the applicant (ALAB-644, p. 142), proposed a model, incorporating vertically propagating waves, to explain the tau effect.

The applicant suggests that any reductions in foundation motion predicted by Seed's model represent additional conservatism (PG&E, Proposed Findings, pp. 43-44). That assumption has been questioned by the Joint Intervenors on two grounds:

- Seed's reduction at high frequencies cannot be justified if one assumes horizontally propagating waves (Joint Intervenors' Proposed Findings, p. 34)

- Newmark's reduction at high frequencies cannot be justified if one assumes vertically propagating waves (Joint Intervenors' Proposed Findings, p. 35)

The Appeal Board concluded (ALAB-644, pp. 126-127) that tau effect motion reductions, which include inhomogeneity and wave passage effects, are reasonable and supported by the record--without factoring in soil-structure interactions as a conservatism. In any case, the Board noted that calculations performed by Dr. Seed using a vertically propagated wave model for the applicant suggested ground motion reductions comparable to those arrived at by Drs. Newmark and Blume.

Based on the record, it appears that a phenomenon exists which at times limits the damage to structures in the near field during earthquakes. However, we have not been able to find an empirical or analytical approach which provides justification as to why the tau effect should be calculated in one specific manner over another. Analyzed or existing data are so sparse that the actual reason for the observed tau effect may still not have been recognized within the engineering community. Except for the judgment of Drs. Blume and Newmark, there is no evidence to demonstrate an ability to predict tau effects over a range of earthquake magnitudes, structural configurations, and site conditions.

On the other hand, the suggestion (Joint Intervenors' Proposed Findings, p. 50) that a "three-dimensional complete soil structure interaction analysis" could resolve the issue is not supported by the record. Three-dimensional modelling

of earthquake response is in its infancy and no systematic attempts have been made to assess the accuracy of the few models that are currently being developed.

The physical explanations of the "effective acceleration" and "tau effect" are different. However, the motivation behind their study is very similar (structures in the near field do not always fail when expected based on their design and the apparent accelerations in the free field). We believe there may be some double counting in these two reductions of high frequency accelerations, but we are unable to confirm this.

DAMPING FACTORS FOR FACILITY STRUCTURES: Is the use of a 7 percent damping factor instead of the former 5 percent factor for analysis of facility structure response adequately conservative?

The Appeal Board decision considered the question of damping values as follows:

> A vibrating system will slow and eventually cease oscillating because of unavoidable energy losses from friction or analogous phenomena operating to dissipate its original energy. The rate or degree of that energy loss varies with different materials and systems and is usually expressed as a percentage of "critical damping," i.e., that amount of damping at which vibratory motion could not exist. The significance of damping for this case centers on the need to factor into the Diablo Canyon seismic response spectra appropriate damping values for the facility's bolted steel and reinforced concrete structures. Regulatory Guide 1.61 (October 1973) states that a 7 percent damping factor should be appropriate for that purpose in the absence of documented tests that would support a higher value. That value was in fact applied in the Hosgri reanalysis. The Joint Invervenors challenged the figure as too high, relying on testimony of Drs. Luco and Trifunac. The Board below, however, accepted the views of the applicant's and staff's experts and found the 7 percent figure to be both appropriate and conservative. LBP-79-26, 10 NRC at 496-97 (ALAB-644, p. 146).

The Appeal Board ruled that "...the weight of the evidence supports the use of 7 percent damping. There is, therefore, no occasion to disturb the Licensing Board's finding that, as recommended by Regulatory Guide 1.61, 7 percent damping was appropriately used in the reanalysis."

The issue has been raised again (Joint Intervenors' Petition for Review of ALAB-644, pp. 11-12). The issue seems, simply, to be that Drs. Luco and Trifunac feel that the use of a 7 percent damping value is not sufficiently conservative, based on their reading of the available data.

Recent studies performed for the staff, and reported in NUREG/CR-1161,* would indicate that, if anything, the 7 percent damping value used may be on the conservative side. Ranges of damping values are recommended in that report for use in the design of bolted steel and reinforced concrete structures, based on compilations of test data more comprehensive than those used as the basis for Regulatory Guide 1.61. Ranges are recommended for damping values, presumably to reflect the scatter in the experimental data. The recommendations, given in NUREG/CR-1161, are:

- reinforced concrete structures - 7 percent to 10 percent
- bolted steel structures - 10 percent to 15 percent

The Board concluded that the evidence presented indicates that the use of a 7 percent damping value is conservative and we concur.

*"Recommended Revisions to Nuclear Regulatory Commission Seismic Design Criteria," May 1980.

• Assessment of Margins in the Plant As Designed

The previous sections primarily treat specific questions, related to the choice of design loads in the Hosgri reanalysis, which were raised in the Petitions for Review filed by Governor Brown and the Joint Intervenors. No specific challenges have been raised in the Petitions for Review concerning the way in which the major structures were analyzed to determine their capacity to resist the increased earthquake loads. However, the ability of the plant to accommodate loads greater than those used in the Hosgri reanalysis depends on the safety margins that exist in the as-built plant. This subject can be put in context by examining the way in which structural engineers decide on the adequacy of a structure or structural component. There are two elements involved: estimating the forces that must be resisted (demand) and estimating the capacity of the structure or component. A measure of the conservatism is then calculated, using either safety factor or safety margin. These are alternate ways of indicating the amount by which the estimated capacity exceeds the estimated demand:

safety factor = estimated capacity/estimated demand
safety margin = safety factor - 1

For example, if capacity were estimated to be 120 and demand 100, the safety factor would be 1.2 and the safety margin would be 0.2. The two measures of conservatism are equivalent; the use of one or the other depends on personal choice of the engineer. The term "margin" is used to indicate that capacity exceeds demand.

When speaking of margins, a distinction is made between "actual" margins and "calculated" margins. The "calculated" margin refers to the actual calculations performed by the engineer to demonstrate that the design meets specified criteria, such as those of building codes, or for nuclear power plant design, those acceptable to the NRC staff as sufficiently conservative. In both building codes and NRC staff practice, there is a tendency to insert conservative factors into the ways in which capacities and demands are estimated. Thus, there is a recognition that "actual" margins exceed "calculated" margins.

Two of the factors that usually contribute to the excess of "actual" margins over "calculated" margins are not present in a reanalysis. Actual measured material strengths are used in lieu of the lower design values, to estimate capacities. This is not uncommon in structural engineering practice. It is applied when bridges are reevaluated for heavier vehicular loadings and when buildings are reevaluated for heavier loads due to changes in occupancy or remodelling. The rationale is simple: the uncertainties about material strengths that prompt prudence at the design stage have been resolved by the material tests during construction. However, in the majority of buildings where the material strengths turn out to exceed those used for design, that prudence becomes an additional protection against unexpected overloads. Another conservative design practice, a tendency to round up to the next convenient size (of pipes, beams, hangers, etc.) after calculating the necessary size, leads to actual capacities greater than estimated capacities and usually contributes to "actual" margins but does not contribute in a reanalysis procedure.

There are, however, other factors which still contribute to "actual" margins in reanalysis procedures.

The net effect of the reanalysis approach is to remove two of the conservative practices normally utilized in structural design. Other conservative practices remain, but are difficult to quantify. The NRC staff referred to some of them (following Tr. 8697) but made no estimate of the amount of conservatism. Newmark (Tr. 8578) estimated that the net conservatism in NRC seismic design requirements, when compared to conventional building practice, was, at a minimum, a factor of 5. Dr. Blume attempted quantitative estimates of the conservatism involved in different steps of the Hosgri reanalysis (following Tr. 6099, pp. 25-29).

Blume's quantification cites conservative practices that lead to overestimates of demand and underestimates of capacity. Blume's list, along with his quantification, is as follows:

Factors that Overestimate Demand

- The use, in estimating seismic forces, of two horizontal components of earthquake motion (say, north-south and east-west) with both components taken as equal to the larger of the two (10-30 percent overestimated).

- The assumption that, even under earthquake excitation, a building's natural frequencies remain constant (10-30 percent overestimate).

- The use of smoothed response spectra for the calculation of forces (10-20 percent overestimate).

Factors that Underestimate Capacity

- The practice, when using test data to estimate capacity, of using a lower bound envelope of the data (15-30 percent underestimate).

- The practice of neglecting the gain in strength, with age, of concrete (20-60 percent underestimate).

- The practice of generally neglecting the ductility, or ability of a structure to absorb energy inelastically (30-100 percent underestimate).

Calculated safety margins and safety factors appear in the record. It should be borne in mind that those calculations, performed for the major structures designated by the applicant as Class I or Class IS, do not reflect the additional conservatism mentioned by Drs. Newmark and Blume. They are simply calculations performed using the conservative estimates of capacity and demand. A summary of those calculations is provided below.

Containment Building (Ghio testimony, following Tr. 6993)

The reevaluation of the containment building led to the conclusion that the most limiting condition was stress in the diagonal reinforcing at approximately mid-height. In that case, the seismic margin is 16 percent (i.e., a seismic stress 16 percent larger than that calculated for the Hosgri event could be

sustained without exceeding the allowable stress). In most locations, the margins are considerably higher. The only modification performed in the containment structure was to an interior crane wall and was a minor one involving changes in the pipe support framing into the wall.

Auxiliary Building (Ghio testimony, following Tr. 7130)

The reevaluation of the auxiliary building indicated that the reinforced concrete components of the building could accommodate the effects of the postulated Hosgri event. Of the steel-framed elements, only the supports for the fuel handling crane required modification. The minimum factors of safety cited are 1.1 for both the most limiting concrete wall element and the steel supports for the cable trays. These factors can be interpreted to mean that the members could carry an additional load of 10 percent without exceeding allowable capacity. Because the ratio of seismic loads to total loads is not given, the seismic margin cannot be estimated directly. It is, however, at least 10 percent and could be higher if the ratio of other loads to seismic loads is significant.

Turbine Building (Ghio testimony, following Tr. 7181)

Major modifications were required on the turbine building as a result of the Hosgri reanalysis. Basically the modifications were of three types: (1) external buttresses and a major shear wall were added to increase the resistance of the building as a whole; (2) significant interior bracing was required at many locations within the building; and (3) the turbine pedestal required post-tensioning of its concrete columns. The calculated safety margins for the most limiting members are small. For example, rock bolts with an allowable load of 84,000 lbs are calculated to have a load of 83,000 lbs under the postulated Hosgri event. This indicates very little margin to accommodate greater effects. However, it is not at all clear that the Category I equipment inside the turbine building would be compromised if some structural elements exceeded capacity.

Intake Structure (Ghio testimony, following Tr. 7224)

The Hosgri reanalysis for the intake structure indicated that most of the main structural elements had significant margins against earthquake forces. Only one set of walls, the lower flow straightener walls, was calculated to be overstressed. For those walls, consideration of the ductility expected in the walls and the overall capacity of the structure led to the conclusion that no modification was necessary.

Water Storage Tanks (Ghio testimony, following Tr. 7235)

The Hosgri reanalysis led to major modifications of the outdoor water storage tanks. The existing steel tanks were encased in concrete and the modified tanks connected by rock anchors to a new concrete foundation. Some locations in the modified tanks show very little margin but in most locations there is significant margin. Again, it is difficult to determine the extent to which the safety function of the tanks would be compromised by localized exceedance of allowable capacities.

Reactor Coolant System (Esselman testimony, following Tr. 7538)

The Hosgri reanalysis indicated significant calculated safety margins for most of the components in the primary system. The results are summarized in Table 6-2 of the Hosgri Report and cited in Esselman's testimony. The table indicates the most highly stressed areas of the primary system components and provided data from which safety factors can be calculated. Some representative values are

Components	Safety factors
Reactor pressure vessel	1.33
Reactor pressure vessel internals	4.55
Steam generator	2.08
Reactor pressure vessel support	2.50
Primary system piping	3.00
Fuel assemblies	1.14
Reactor coolant pump	1.33
Reactor coolant pump support	1.85

In summary, calculations indicate a margin of safety for performance under the safe shutdown earthquake, even within the restrictive criteria agreed to by the applicant and the NRC staff. Further, there is evidence in the record from the staff's, Newmark's, and Blume's testimony that the safety margins calculated by the applicant greatly underestimate actual safety margins.

For comparison with Newmark's minimum estimate of 5, one can use Blume's numbers as follows: Assuming that the calculated capacity is C, Blume indicates that the actual capacity would range from (1.15)(1.2)(1.3)C to (1.3)(1.6)(2.0)C or from 1.79C to 4.16C. Assuming that the calculated demand is D, Blume indicates that the actual demand would range from (.7)(.7)(.8)D to (.9)(.9)(.9)D or from .392D to .729D. Thus, the amount of conservatism would range from (1.79/.729) to (4.16/.392) or from 2.46 to 10.6.

We can conclude, from the record, that there is evidence of the ability of plant structures to accommodate loads greater than those used in the Hosgri reanalysis. Other margins have not been quantified in the record. For example, in the containment analysis the governing load combination postulates that SSE and loss-of-coolant accident (LOCA) effects will occur simultaneously and have an additive effect. This is unrealistic from a physical viewpoint and, in effect, understates the capacity of the containment to sustain an earthquake. In the most limiting situation, a reduction of the LOCA load by a factor of 2 would increase the calculated earthquake capacity by about one-third. For other structures, conservatisms also included the loads and load combinations considered for earthquake design, but their effect may not be as significant as for the containment.

OPERATING BASIS EARTHQUAKE (OBE)

The major safety concern in seismic design of nuclear power plants--the ability to maintain a safe shutdown condition after a major earthquake--centers on the choice of both an appropriate value for the SSE and a design to accommodate the effects of that earthquake. Staff thinking on how to ensure that safe conditions will exist after a major earthquake have evolved over time.

It was originally thought that the specification of a design earthquake (DE) and a double design earthquake (DDE) was the best approach. The intent was that a plant should be designed so that it could undergo a DE commensurate with those used for local building design without any noticeable effect. As a practical matter, that requirement mandated that all plant structures and components should respond elastically to the DE. Then, an analysis was to have been undertaken to ensure that, given whatever inelastic deformation might occur under an earthquake twice as great (DDE), the plant could still be brought to a safe shutdown. It was in that context that the choice of a design value of 0.2 g was made for the Diablo Canyon plant (see Hoch testimony, following Tr. 6871, pp. 68-71). The DDE level was 0.4 g.

A major shortcoming of the DE/DDE approach became apparent and led to its abandonment when Appendix A to 10 CFR Part 100 was revised in 1973. Previously, there was no explicit requirement for applicants to attempt to estimate the most damaging plausible earthquake. Further, applicants were not required to provide evidence that doubling the level of the DE would envelope the effects of the most damaging plausible earthquake. The 1973 revision shifted the focus to the determination, based on geological and seismological evidence, of the most damaging earthquake that could possibly occur at a site. Based on this determination, an SSE would be chosen and the plant would be designed to ensure safe shutdown at that earthquake level. It was recognized that safe shutdown could be accomplished even if some plant structures and components were damaged (i.e., deformed) (10 CFR Part 100; VI(a)(1)). Determination of an operating basis earthquake (OBE) was also mandated with the intent that a plant could continue to operate during and after such an earthquake. While the ability of a plant to operate during and after an OBE may be limited by

other factors, the concept did serve as a basis for developing design criteria to ensure essentially elastic response to the OBE. For design purposes, this is significant. Conceptually, a plant could be designed for the OBE in combination with other normal loads and then checked for SSE conditions.

Some argue that there is no need for a fixed relationship between OBE and SSE levels. One viewpoint (SECY-79-300) is that the OBE level can be an economic choice of the applicant, based on a trade-off between expected shutdown costs and construction costs. The lower the OBE level chosen, the higher the likelihood of having an earthquake exceeding OBE level, and the higher the expected shutdown costs. The higher the OBE level chosen, the higher the earthquake forces to be resisted, and the higher the construction cost. Thus, a "minimum cost" OBE level could be chosen.

The Joint Intervenors argued that the intent of Appendix A to 10 CFR Part 100 was that the OBE level should be at least half that of the SSE (ALAB-644, p. 171). The NRC staff had testified (following Tr. 8183) that the value of 0.2 g chosen for the OBE level was acceptable as "being one that could reasonably be expected to affect the plant site during the operating life of the plant." The grounds for acceptability cited were estimates of the return period of an earthquake with accelerations exceeding 0.2 g. The Appeal Board has held (ALAB-644, p. 170) that the choice of OBE based on return period is consistent with the intent of Appendix A, even as currently written.

The Appeal Board deferred to the Licensing Board the question of whether balance-of-plant equipment designed to an OBE level of 0.2 g might fail and compromise the ability to shut down under the SSE (ALAB-644, p. 175). That issue has been raised in the Petition for Review by the Joint Intervenors (p. 14) but was not raised in the record. Subsequently, the issue was not admitted for contention in the Licensing Board deliberations on the system interaction question. That rejection is currently being reviewed by the Appeal Board. In their responses to the Petition for Review, the NRC staff (p. 7) and the applicant (p. 9) do not speak directly to this issue but implicitly dismiss it by contending that no new issues of fact have been raised.

ENCLOSURE 2

SUMMARY OF NRC LICENSING SEISMIC PRACTICE

The purpose of this enclosure is to provide a primer on NRC licensing practice in the areas of geology, seismology, and seismic engineering. Licensing practice in the geologic and seismic area relies on Appendix A to 10 CFR Part 100, Section 2.5 of the Standard Review Plan, and Regulatory Guides 1.60, 4.7, and 1.132. Because of the variability from site to site of geologic and seismic characteristics, emphasis in the review is given site-specific analysis rather than applying a more generic approach. The primary required investigations are described in Appendix A to 10 CFR Part 100. In licensing practice, a nuclear power plant seismic design is predicted on a reasonable, conservative determination of the safe shutdown earthquake (SSE) and the operating basis earthquake (OBE). The SSE and OBE, in turn, are based on consideration of the regional and local geology and seismology and characteristics of subsurface materials at the site and are described in terms of the vibratory ground motion which they would produce at the site.

The initial source of this data base is the applicant's Safety Analysis Report. The reviewer also relies on other literature sources, his own knowledge and that of other experts including those from universities, state agencies, and other federal agencies.

Geologic Review

With respect to evaluating the existence of potentially hazardous geologic conditions at a site and assisting in the assessment of the site seismic potential, the staff's effort is directed towards a review of the geologic and tectonic characteristics of the site and region. The regional geology is studied primarily to obtain a understanding of the overall "geologic setting" of the site region and its relationship to both ancient and currently active tectonic features and processes and is used to determine tectonic provinces. The site

geology is studied to provide reasonable assurance that there is no potential for surface faulting.

During a site review, extensive amounts of data are submitted by the applicant in the Preliminary Safety Analysis Reports. Visits are made to the site and its environs during the course of a review to examine the regional geology, bedrock exposures, soil samples, and core borings. When applicable, local geological experts, the applicant's geologic consultants, state Geological Surveys and our advisors, the U.S. Geological Survey and the U.S. Army Corps of Engineers, are consulted.

The site geology review assesses the site stratigraphy, the structural geology of the site, the geologic history of the site, and, most importantly, the significance of geologic features which may be present in the immediate vicinity of the site.

The predominant effort expended by reviewers for most sites is a review of the local faulting. The information reviewed is usually detailed surface and subsurface geological and geophysical data gathered by the applicant. When faults are identified in the site vicinity, a comprehensive geologic investigation must be conducted to determine whether or not the faults should be considered to be capable within the meaning of Appendix A to 10 CFR Part 100. This involves the use of absolute age dating techniques to date the most recent movement experienced by the fault, an analysis of stratigraphic evidence, in some cases seismic monitoring, or by associating the faulting with regional tectonic activity of known age. Although the regulations do not expressly prohibit construction on a capable fault, no nuclear plant has ever been constructed over one and it is an open question whether it is possible to design for significant surface displacement with confidence such that the safety-related features of the plant would remain functional. Our practice is to recommend relocation of a proposed plant if a site is found to be located on a capable fault as determined by detailed investigations.

The term "capable fault" as set forth in Appendix A to 10 CFR Part 100 is unique to the Commission and was developed in order to differentiate faults capable of causing movement at or near the ground surface or generating high vibratory

ground motion from faults not possessing these characteristics of activity. Capable faults are usually found in regions subjected to high rates of tectonism (active earthquake areas). Techniques for identifying capable faults are based on recency of fault displacement and on the rate of seismic activity on the fault. The emphasis on rate of activity is expressed more specifically in the criteria in that "capability" requires tectonic movement at or near the ground surface at least once within the past 35,000 years or movements of tectonic origin of a recurring nature within the past 500,000 years.

Macroseismicity is another criterion for determining fault capability which is also a term unique to the Commission. The term is not defined in Appendix A. The staff practice is to consider macroseismicity as a level of seismicity which implies significant, sustained, and coherent tectonic activity representative of major deformational movement within the earth's crust. The staff has not assigned a specific size earthquake or frequency of earthquake occurrence which constitutes macroseismicity; rather, we require a review using judgment on a site-specific basis.

No capable faults are known to exist in the United States east of the Rocky Mountains. It is a general staff view that faults in the eastern United States with a few possible exceptions (Charleston, SC, and New Madrid, MO) are not likely to be found as being capable. However, detailed investigations are still required to confirm that faults found near sites in the eastern U.S. are not capable.

Seismic Review

The staff's primary effort in the seismic review is directed towards identification of potential earthquake sources and regions of equivalent earthquake potential. For this purpose, the staff evaluates tectonic provinces and tectonic structures defined by the applicant. Further, the staff requires that the applicant tabulate all significant earthquakes which have occurred in these tectonic provinces or in association with the tectonic structures.

In assessing an applicant's tectonic province models and his correlations of earthquakes with geologic features, the staff considers the significance of

such factors as geologic structure, tectonic history, present and past stress regimes, and the pattern and level of earthquake activity. The earthquake catalog is also reviewed for completeness and accuracy. Exceptions to this practice are sometimes allowed if a complete and adequate justification for any differences is provided in the Safety Analysis Report.

Having identified historical earthquake activity, potential earthquake sources and tectonic provinces, the applicant is required to determine the association between historical earthquake activity and these sources and provinces. For tectonic structures which might localize earthquake activity, the applicant must identify the historical earthquakes associated with the structures and provide the basis for the association. The staff does not require that an absolute association of historical earthquake activity with structures be established. What is sought is a reasonable association. In reviewing the basis for the association, the staff considers such factors as the nature of the structure, accuracy of location of epicenters near the structure, geologic age of most recent displacement on the structure relative to that for other structures in the region, the mechanism for earthquake generation and its relation to regional tectonics, the uniqueness of the structure relative to others in the tectonic province, the background seismicity of the region, the rate of seismicity on the structure, and the geologic history of the region. The staff treats historical earthquakes which cannot be reasonably associated with tectonic structures as being associated with tectonic provinces as required by Appendix A to 10 CFR Part 100. The tectonic province is determined for the purpose of identifying a region of equivalent earthquake potential. Thus, historical earthquakes associated with a tectonic province are assumed by the staff to have the potential for occurring anywhere in the province, as required by Appendix A.

The applicant is required by Appendix A to identify the most severe earthquakes associated with tectonic structures and tectonic provinces in the region surrounding the site. Normal staff practice with regard to tectonic provinces is to accept the largest historical earthquake associated with the tectonic province as the "most severe earthquake." To assure appropriate conservatism in light of the short historical record of earthquake activity, the staff has required that the geographical area encompassed by a tectonic province be large, e.g.,

similar in dimension to generally accepted provinces existing in the literature proposed by King (1969), Eardley (1973), and Hadley and Devine (1974) for eastern North America. The staff considers that, by so doing, the earthquake data sample has a much greater likelihood of including earthquakes large enough so that they have a low probability of occurring in the site vicinity during the lifetime of the facility and are thus consistent with the desired conservatism for a safe shutdown earthquake. For earthquakes associated with tectonic structures, the staff may require that the "most severe earthquake" be larger than the largest historical earthquake associated with the structure. Assessment of the size of the most severe earthquake in this case is required to take into account not only the earthquake history but also other geologic factors such as type of faulting, fault length, fault displacement, and stress field. The basis for considering these factors is that the energy released in an earthquake, and hence the magnitude, is dependent on the area over which the fault rupture occurs and on the stress drop resulting from that rupture. Therefore, the staff's efforts are directed towards assuring that, in assessing the size of the most severe earthquakes associated with structure, adequate consideration has been given to the dimension of the fault surface expected to rupture in a single earthquake and the prevailing tectonic conditions near the fault and in the region.

Appendix A to 10 CFR Part 100 requires that ground motion at the site due to earthquakes be determined assuming that the epicenters or locations of highest intensity, of earthquakes associated with tectonic structures or tectonic provinces, are situated at the point on the tectonic structures or tectonic provinces nearest to the site. It is staff practice to require that the applicant follow this procedure even though its implementation sometimes results in a situation where the proposed plant site is hundreds of miles from the epicenter of the earthquake on which the safe shutdown earthquake is based, and often tens of miles from the nearest earthquake activity of any size. The staff views this practice as providing the design margin specified in 10 CFR Part 50.

The next step is to assess what ground motion these earthquakes can produce. The ground motion can be represented in a number of ways. Usually an acceleration (percent of gravity) is used to define the level of ground motion, although ground velocity, displacement, and frequency content may also be considered.

The acceleration level at the site corresponding to the postulated maximum potential earthquake is determined.

Where the earthquake is associated with specific structures, as in the western United States, the acceleration level is determined using correlations between earthquake magnitude or fault length, epicentral distance, and acceleration. Relationships, such as those published by Schnabel and Seed (1972) and Toucher (1958), are typical of these correlations. It is our current practice to approve values which are as conservative as those which would be derived from the Schnabel and Seed relationship. Since the relations published by Schnabel and Seed are for acceleration on rock, some consideration of modification of these values for soil sites may be appropriate depending on soil conditions.

Once the acceleration level is determined, it is usually used to scale standard response spectra such as those in NRC Regulatory Guide 1.60. The staff also accepts alternate approaches to the above determination. For example, we permit a site-specific spectrum to be used with sufficient justification.

For plant sites in the eastern United States, relatively little information is available on magnitudes (i.e., few instrumental determinations exist) for the larger historical earthquakes, and we must rely on their descriptions in historical reports of earthquake effects (i.e., intensity). In most cases, the safe shutdown earthquake for the site is based on the largest earthquake associated with the tectonic province in which the site is located. In this case the maximum intensity corresponding to that earthquake is assumed to occur at the site. In a few cases, more distant earthquakes may be significant in determining the maximum ground motion expected at the site, e.g., the well known New Madrid earthquakes of 1811-1812 produced large intensities at great distances from the source. For such cases the attenuation of intensity with distance is considered. The intensity at the site is related to ground motion using intensity-acceleration relationships such as those published by Trifunac and Brady (1975), Neumann (1954), Computer Sciences Corporation (1978), or Ambraseys (1974). Staff practice in recent licensing actions has been to use the mean value of the relationship published by Trifunac.

NRC Staff Practice in Seismic Engineering

(1) Design Ground Motion

For the seismic design of nuclear power plants it is customary to specify the earthquake ground motion which is exerted on the structure or on the soil-structure interaction system. The design ground motion, sometimes known as the seismic input, is based on the seismicity and geologic conditions at the site and expressed in such a manner that it can be applied to the dynamic analysis of structures. The design ground motions for the operating basis earthquake (OBE) and safe shutdown earthquake (SSE) are reviewed. They should be consistent with the information on seismic environment at the site, which includes the variation in and distribution of peak ground acceleration in the free field at different depths across the soil profile, sources and directions of motion, propagation and transmission of seismic waves, and other response characteristics.

(a) Design Reponse Spectra

A response spectrum is a plot of the maximum response of a family of single-degree-of-freedom damped oscillators with different frequency characteristics when the base of the oscillator is subjected to vibratory motion indicated by an appropriate time motion record. The response spectra are usually displayed on tripartite log-log graph paper. When obtained from a recorded earthquake, the response spectrum tends to be irregular, with a number of peaks and valleys. A design response spectrum is a relatively smooth plot, obtained from a number of individual response spectra derived from records of past earthquakes. For high frequencies, spectral acceleration approaches the bound set by the maximum ground acceleration. For intermediate frequencies, spectral velocity is amplified relative to the ground velocity. For low frequencies, spectral displacement is amplified relative to the ground displacement.

Design response spectra for the OBE and SSE are considered to be acceptable if the associated amplification factors are in accordance with Regulatory Guide 1.60, "Design Response Spectra for Nuclear Power Plants," for all damping values.

As noted in Regulatory Guide 1.60, there are site circumstances where the design response spectra are more appropriately developed to suit the particular site characteristics. Design response spectra based on site-dependent analysis must be derived considering in situ variable soil properties, a representative number of site earthquake records, vertical amplification, possible slanted soil layers, and the influence of any predominant soil layers. Variable soil properties and nonlinear stress-strain relations in the soil media should be considered.

If site-dependent design response spectra are used, the data and bases from which the spectra are derived should be consistent with the information on seismic environment at the site.

To be acceptable, the design response spectra should be specified for three mutually orthogonal directions: two horizontal and one vertical. Current practice is to assume that the maximum ground accelerations in the two horizontal directions are equal, while the maximum vertical ground acceleration is 2/3 of the maximum horizontal acceleration. For the western United States (west of the Rockies), the response spectrum for vertical motion can be taken as 2/3 the response spectrum for horizontal motion over the entire range of frequencies.

(b) Design Time History

When a recorded or specified time history is not available as input motion for seismic system analysis, an artificial time history may be generated from the design response spectra for the purpose of carrying out a time history analysis. However, the response spectra obtained from such artificial time history of motion should generally envelope the design response spectra for all damping values to be used.

For the analysis of interior equipment, where the eqipment is decoupled from the building, a compatible time history is needed for computation of the time-history response of each floor. The design floor spectra for equipment are obtained from this time history information.

(2) Critical Damping Values

The specific percentage of critical damping values used for Category I structures, systems, and components are reviewed for both the OBE and the SSE. Critical damping is the amount of damping that would completely eliminate vibration. Although the use of critical damping is of little practical importance in itself, it assumes great significance as a measure of the damping capacity of a structure. Damping is conveniently expressed in the form of some percentage of critical damping.

Vibrating structures have energy losses which depend on numerous factors, such as material characteristics, stress levels, and geometric configuration. This dissipation of energy, or damping effect, occurs because a part of the excitation input is transformed into heat, sound waves, and other energy forms. The response of a system to dynamic loads is a function of the amount and type of damping existing in the system. A knowledge of appropriate values to represent this characteristic is essential for obtaining realistic results in dynamic analysis.

The specific percentages of critical damping values used in the analyses of Category I structures, systems, and components are considered to be acceptable if they are in accordance with Regulatory Guide 1.61, "Damping Values for Seismic Design of Nuclear Power Plants." Higher damping values may be used in a dynamic seismic analysis if documented test data are provided to support them. These values would be reviewed and accepted by the staff on a case-by-case basis. The damping value for soil must be based upon actual measured values or other pertinent laboratory data considering variation in soil properties and strains within the soil.

(3) Supporting Media for Category I Structures

The description of the supporting media for each Category I structure is reviewed, including foundation embedment depth, depth of soil over bedrock, soil layering characteristics, width of the structural foundation, total structural height, and soil properties to permit evaluation of the applicability of finite element or lumped spring approaches for soil-structure interaction analysis.

To be acceptable, the description of supporting media for each Category I structure must include foundation embedment depth, depth of soil over bedrock, width of the structural foundation, total structural height, and soil properties such as shear wave velocity, shear modulus, and density as a function of depth.

(4) Seismic Analysis Methods

For all Category I structures, systems, and components, the applicable seismic analysis methods (response spectra, time history, equivalent static load) are reviewed. The manner in which the dynamic system analysis method is performed, including the modeling of foundation torsion, rocking, and translation, is reviewed. The method chosen for selection of significant modes and an adequate number of masses or degrees of freedom is reviewed. The manner in which consideration is given in the seismic dynamic analysis to maximum relative displacements between supports is reviewed. In addition, other significant effects that are accounted for in the dynamic seismic analysis such as hydrodynamic effects and nonlinear response are reviewed. If tests or empirical methods are used in lieu of analysis for any Category I structure, the testing procedure, load levels, and acceptance basis are also reviewed.

(a) Dynamic Analysis Method

A dynamic analysis (e.g., response spectrum method, time history method, etc.) should be used when the use of the equivalent static load method cannot be justified. To be acceptable, such analyses should consider the following items:

1) Use of either the time history method or the response spectrum method.

2) Use of appropriate methods of analysis to account for effects of soil-structure interaction.

3) Consideration of the torsional, rocking, and translational responses of the structures and their foundations.

4) Use of an adequate number of masses or degrees of freedom in dynamic modeling to determine the response of all Category I and applicable

non-Category I structures and plant equipment. The number is considered adequate when additional degrees of freedom do not result in more than a 10 percent increase in responses. Alternately, the number of degrees of freedom may be taken equal to twice the number of modes with frequencies less than 33 cps.

5) Investigation of a sufficient number of modes to assure participation of all significant modes. The criterion for sufficiency is that the inclusion of additional modes does not result in more than a 10 percent increase in responses.

6) Consideration of maximum relative displacements among supports of Category I structures, systems, and components.

7) Inclusion of significant effects such as piping interactions, externally applied structural restraints, hydrodynamic (both mass and stiffness effects) loads, and nonlinear responses.

(b) Equivalent Static Load Method

An equivalent static load method is acceptable if:

1) Justification is provided that the system can be realistically represented by a simple model and the method produces conservative results in terms of responses. Typical examples or published results for similar structures may be submitted in support of the use of the simplified method.

2) The design and associated simplified analysis account for the relative motion between all points of support.

3) To obtain an equivalent static load of a structure, equipment, or component which can be represented by a simple model, a factor of 1.5 is applied to the peak acceleration of the applicable floor response spectrum. A factor of less than 1.5 may be used if adequate justification is provided.

(5) Soil-Structure Interaction

The design earthquake motion is defined at the foundation level of the structure in the "free field," i.e., the effect of the presence of structures is not included. When plants are founded on soil deposits or soft media, the resulting motions of the base slab will differ from those defined at the same elevation in the free field, due to deformability of the foundation and soil. This difference between the base slab motion and the free-field motion is known as soil-structure interaction effect.

As applicable, the modeling methods of soil-structure interaction analysis used in the seismic system analysis and their bases are reviewed. The factors to be considered in accepting the validity of a particular modeling method are: (1) the extent of embedment, (2) the depth of soil over rock, and (3) the layering of the soil strata.

If the half-space (lumped parameter) modeling method is used, the parameters used in the analysis are reviewed. Also, the procedures by which strain-dependent soil properties (damping, shear modulus), layering, and variation of soil properties are incorporated in the analysis are reviewed.

If the finite boundary modeling method for soil media is used, the criteria for determining the location of the bottom boundary and side boundary are reviewed. The procedures by which strain-dependent soil properties (damping, shear modulus) are incorporated in the analysis are also reviewed.

Any other modeling methods used for soil-structure interaction analysis are also reviewed as is any basis for not using soil-structure interaction analysis. The procedures used to account for effects of adjacent structures on structural response in the soil-structure interaction analysis are reviewed.

At present, most commonly used methods are the half-space and the finite boundaries modeling methods, and there is no indication as to which one is more reliable, especially when too many assumptions are involved. Therefore, modeling methods for implementing the soil-structure interaction analysis should include both the half-space and finite boundaries approaches. Category I structures,

systems, and components should be designed to accommodate responses obtained by one of the following:

(a) Envelope of results of the two methods.

(b) Results of one method with conservative design considerations of effects from use of the other method.

(c) A combination of (a) and (b) with provision of adequate conservatism in design.

The acceptance criteria for the constituent parts of the entire soil-structure interaction system are as follows:

For structures supported on rock, a fixed base assumption is acceptable.

The effect of embedment of structure and the layering effect of soil should be accounted for. For the half-space modeling of the soil media, the lumped parameter (soil spring) method and the compliance function methods are acceptable. For the method of modeling soil media with finite boundaries, all boundaries should be properly simulated and the use of types of boundaries should be justified and reviewed on a case-by-case basis. Finite element and finite difference methods are acceptable methods for discretization of a continuum. The properties used in the soil-structure interaction analysis should be those corresponding to the low strains which are consistent with the realistic soil strain developed during the design earthquake. Use of high strain parameters needs to be adequately justified on a case-by-case basis.

ENCLOSURE 3

GENERAL METHODOLOGY OF SEISMIC DESIGN AND ANALYSIS

A general methodology for estimating forces and stresses in structures that may be subjected to earthquake motions has evolved over several decades. Specific structures and specific anticipated earthquakes sometimes require special scientific approaches and substantial engineering judgment, but the general steps involved in the analysis are similar to those applied to most seismically designed civil structures.

The most common representation of seismic ground motion for design purposes is the response spectrum. This represents the maximum responses to seismic ground motion in one direction of a suite of single-degree-of-freedom oscillators with specified viscous damping, linear force-deformation behavior, and a range of frequencies. Maximum responses are computed mathematically, rather than with physical models. The maximum responses may be in terms of the maximum acceleration, velocity, or displacement experienced by each oscillator during the earthquake motion. To represent response spectra graphically, maximum responses are plotted versus the natural period or frequency of the oscillator, and may be plotted on linear or logarithmic scales (both for frequency and for maximum response). Often on one plot, response spectra show curves for different values of viscous damping. One convenient and popular representation is to compute the maximum displacement d_{max} of each oscillator, and to estimate the maximum velocity v_{max} and maximum acceleration a_{max} by

$$v_{max} = \omega\, d_{max}$$
$$a_{max} = \omega^2\, d_{max} \qquad (1)$$

where ω is the oscillator's natural frequency in radians/sec. These <u>estimated</u> velocities and accelerations are terms "pseudo-relative velocity" and "pseudo-absolute acceleration." They are good approximations of the actual maximum

velocity and acceleration experienced by each oscillator; their importance is that they allow the three quantities to be plotted on one figure. This is accomplished by plotting pseudo-relative velocity versus natural frequency or period, on logarithmic scales. Additional scales can then be constructed on diagonal lines to indicated maximum relative displacement and maximum pseudo-absolute acceleration. (Relative in this sense means displacement of the oscillator relative to the ground.)

When strong ground motion during a real earthquake is recorded, it is generally on an instrument with a high natural frequency that registers acceleration. The record is processed and corrected to remove any instrument effects, so that a true representation (or a close approximation to it) is obtained of the ground acceleration as a function of time. Modern accelerometers simultaneously record ground motion in three orthogonal directions (two horizontal and one vertical). To represent the ground motion by response spectra, three such spectra must be computed and plotted, one for each of the orthogonal components.

Response spectra for recorded ground motions are quite nonuniform, i.e., the computed responses as a function of frequency vary greatly from one frequency to the next. This variation is greatest for response spectra computed for low damping, and less for spectra with higher damping. It results from the complicated ground motions observed during earthquakes; these motions result in oscillators at some frequencies having motions which are greatly amplified or deamplified when compared to motions of oscillators with slightly different frequencies.

For purposes of seismic design, smooth response spectra are used to represent the ground motion. The frequency-to-frequency variations that are observed for recorded ground motions cannot be predicted for future earthquakes, so there would be little point in representing them for design. An exception would be if response spectrum amplification in a specific frequency band were anticipated as a result; for example, of soil amplification at a specific site. In this case this specific effect, which would be repeatable from earthquake to earthquake, would be represented in the response spectrum used for design.

Once a response spectrum is selected for seismic analysis and design (more about this below), it is used in the following manner: The structures and structural components are modeled mathematically as dynamic systems, from which natural frequencies of vibration can be determined for each orthogonal direction. The fundamental mode and enough higher modes are considered to ensure that the total response can be determined. The maximum response in each mode is estimated from the design response spectrum, having a natural frequency and an estimated damping value (which is determined from structural tests). From maximum displacement of the mathematical model, displacements of the structure and its components can be determined for that mode. The total response of the structure can be calculated knowing the modal responses and the modal participation factors. Once the response of the structure in one direction is known, it can be combined with responses in the other two orthogonal directions, if appropriate, using methods such as the square root of the sum of the squares (this is justified because the maximum responses in each direction will generally not occur at the same instant in time). Stresses in members can be calculated after dynamic displacements have been estimated. In addition to stresses caused by foundation translation, those induced by foundation rotation are added, where this is seen to be important.

It is apparent that the critical step in designing and analyzing structures for earthquake motions is selection of the design response spectrum. Because the ground motions in the two horizontal directions produce similar response spectra during real earthquakes, a single spectrum is specified for the two horizontal components. The vertical design spectrum is often specified by scaling the horizontal spectral amplitudes at all frequencies by a fraction, e.g., 2/3, because vertical motions are generally observed to produce lower spectra than do horizontal motions.

Response spectra for design are, in most cases, determined using the following general procedure. Earthquake accelerograms (acceleration records) are gathered for a wide range of motions (a range of earthquake magnitudes, source-to-site distances, and site conditions), and response spectra are calculated for these. Usually the records are segregated using a crude classification scheme into "rock" and "alluvium" site conditions. The computed response amplitudes are then "normalized" in one of several ways. The simplest is to scale response

amplitudes to a common peak ground acceleration such as 1 g. In effect this amounts to computing response amplitudes per unit of peak ground acceleration; it is accomplished very simply by multiplying the computed spectral amplitudes by the ratio of 1 g to the actual recorded peak ground acceleration. (This procedure is possible because of the linear relationship between spectral amplitudes and ground acceleration.)

With normalized spectral amplitudes computed, it is a simple matter to calculate statistics of response amplitude for each frequency. The mean and standard deviation of response, over the range of records considered, is obtained for each frequency and damping. An appropriately conservative set of values, typically the mean +1 standard deviation, is selected. This then provides a design spectrum for a 1 g peak ground acceleration. The last step in the procedure is to select an appropriate peak ground acceleration at the site, considering the seismicity of the region, the degree of conservatism desired for the structure, and the conservatism expected in the structural design. The selection of peak ground acceleration for design is discussed below.

An alternative method of scaling response spectra from recorded ground motions is to do so using the peak acceleration a_g, velocity v_g, and displacement d_g of the ground, rather than merely the peak ground acceleration. In this method, the frequency range of interest is divided into three bands: high frequencies (typically about 3-10 Hz), intermediate frequencies (typically 0.25-3 Hz), and low frequencies (below 0.25 Hz). For each frequency band, spectral "amplification factors" are computed; these are ratios of spectral (maximum oscillator) acceleration to a_g in the high frequency range, spectral velocity to v_g in the intermediate frequency range, and spectral displacement to d_g in the low frequency range. For the suite of earthquake motions analyzed, statistics are calculated of the spectral amplification factors, and appropriately conservative factors are selected for design (e.g., the mean +1 standard deviation factors). Ratios of v_g to a_g and of d_g to a_g are also computed from the records used in the spectrum scaling procedure. The process of calculating a spectrum for design can then proceed in one of two ways. Values of a_g, v_g, and d_g can be selected for the design earthquake, and spectral amplitudes scaled from these or (most often the case) a_g alone can be estimated, v_g and d_g can be estimated from a_g, and spectral amplitudes can be scaled from these values. The latter method is

equivalent to the first method described above of scaling all spectral amplitudes by a_g.

The last step required to determine a design response spectrum at a site is to determine appropriate values of a_g (and perhaps v_g and d_g, if scaling by these is required). This is accomplished by three steps. First, a mathematical equation is derived to estimate a_g (and perhaps v_g and d_g) as a function of earthquake magnitude M, distance R, and, perhaps, site conditions. This is most frequently done by compiling data on a_g, M, R, and site conditions, selecting a functional form to represent the dependence of a_g on M, R, and site conditions, and performing regression analysis to determine the values of constants on the selected function.

The second step required is to select appropriate values of M and R, and to determine underground conditions at the site to represent the design earthquake. This involves evaluating seismicity in the region, the conservatism required (which influences the degree of conservatism influencing choices of magnitude and distance), and the expected conservatism inherent in the structural design. For the purposes of seismic design, deterministic methods are usually used to select design values of M and R. Sometimes (particularly for the analysis of existing and/or operating facilities) a probabilistic method is used, either to specify M and R, or to specify ground motion values at the site.

The final step is to use the magnitude, distance, and site conditions determined in step 2 above to estimate values for a_g (and perhaps v_g and d_g) with the equation developed in step 1. The common procedure is to specify the mean value of a_g, because the mean +1 standard deviation spectrum is used, rather than some other value.

In special cases, the above procedures may be deemed invalid and are modified. For example, the response spectrum from this technique is a general one and will not represent special conditions such as short source-to-site distances. (The reason for this is that the frequency content of seismic ground motion changes with distances.) Special site conditions often require modification of spectra.

In the case of Diablo Canyon, several considerations were made to account for special conditions which, under usual procedures, are not considered. The peak ground acceleration value for the design earthquake was deemed unappropriate to scale response spectra because of its high frequency nature (caused by the proximity of the site to the causative fault). This led to the development of an "effective" peak acceleration by consultants to the staff and to the applicant, which they considered more appropriate to use for scaling response spectra. Further, the massive, rigid foundation of structures associated with the power plant were deemed capable of filtering out some of the high frequency motion normally associated with peak ground accelerations. Thus the peak acceleration and response spectrum used for analysis of the superstructure is not the same as would be predicted for a building with a smaller and/or less rigid foundation. This effect (called the tau effect) resulted in reduction of a_g at the foundation level from those values determined as "effective" peak accelerations. In these two areas, then, special considerations resulted in reduction of both peak acceleration and the design response spectrum, because the general procedure developed heretofore was deemed inappropriate.

ENCLOSURE 4

WITNESSES CITED IN OPE ANALYSIS OF
DIABLO CANYON SEISMIC ISSUES

Listed below are the academic and professional credentials of the witnesses before the Licensing and Appeal Boards in the matter of Diablo Canyon Seismic Issues who are cited in the OPE analysis of those issues.

The seismic design of nuclear plants has been governed by the specifications of Revision 1 to Regulatory Guide 1.60, "Design Response Spectra for Seismic Design of Nuclear Power Plants." Works by Dr. Nathan Newmark and others, most notably Dr. John Blume, formed the basis for Regulatory Guide 1.60.

Dr. Newmark was hired originally by the Atomic Energy Commission as a consultant in the mid-1960s to perform case-related seismic design reviews. The seismic design criteria were at that time guided by the earlier work of Dr. George Housner. Starting in about 1970, Dr. John Blume also became a consultant to the Atomic Energy Commission and, based on efforts by Dr. Blume, the general approach to developing seismic design spectra was modified to produce more stringent design criteria.

The continual interactions between Dr. Newmark, Dr. Blume, and the Commission led to indepth studies for the Commission by these two consultants of numerous earthquake records and responses. These studies led directly to the development of Regulatory Guide 1.60. Following the initial issuance of the regulatory guide, Dr. Newmark continued his work in this area in collaboration with his graduate students. This effort led to the issuance of Revision 1 to Regulatory Guide 1.60 in December 1973. This version of the regulatory guide continues in effect today.

Drs. Enrique Luco and Mihailo Trifunac have served as consultants to the Advisory Committee on Reactor Safeguards and were subpoenaed by the Appeal Board to testify before its hearing.

Resumes

Dr. John A. Blume	Ph.D., Structural/Earthquake Engineering, Stanford University Licensed civil engineer and licensed structural engineer, President of the consulting firm of URS/John A. Blume and Associates, Engineers
Dr. Bruce A. Bolt	Ph.D. and Sc.D., University of Sydney, Australia Professor of Seismology, University of California, Berkeley
Dr. James N. Brune	Ph.D., Geophysics, Columbia University Professr of Geophysics, University of California, San Diego
Mr. James F. Devine	B.S., Geology, West Virginia University Deputy for Engineering, Office of Earthquake Studies, USGS, and USGS Geological Coordinator for NRC Nuclear Power Plant Siting
Dr. Thomas C. Esselman	Ph.D., Mechanical Engineering, Case Western Reserve University Manager, Support Structure Design, Equipment Engineering, Westinghouse
Dr. Gerald Frazier	Ph.D., Civil Engineering, Montana State University President, Del Mar Technical Associates Corporation
Mr. Vincent J. Ghio	B.S., University of California Registered Civil Engineer, Senior Civil Engineer, PG&E
Mr. Douglas H. Hamilton	M.S., Stanford University Engineering Geologist, Vice President and Principal Geologist, Earth Sciences Associates, Inc.

Dr. Stanley A. Hanusiak	M.S., Civil Engineering, Structures, Cracow Technical University, Poland Registered Civil Engineer and Structural Engineer, Civil Engineer, PG&E
Mr. John B. Hoch	B.S., Mechanical Engineering, University of Idaho Registered Mechanical Engineer and Registered Nuclear Engineer, Project Engineer, PG&E
Mr. Renner B. Hofman	M.S., Geophysics (seismology option), St. Louis University Seismologist, Site Safety Standards Branch, Office of Standards Development, NRC
Dr. Pao-Tsin Kuo	Ph.D., Civil Engineering, Rice University Structural Engineer, Structural Engineering Branch, NRC
Dr. Enrique J. Luco	Ph.D., Applied Mechanics, University of California, Los Angeles Associate Professor of Applied Mechanics, University of California, San Diego
Dr. Nathan M. Newmark	M.S., Civil Engineering, Ph.D., Engineering, University of Illinois It is with sincere sorrow that we note the death of Dr. Newmark a few weeks after he appeared as a consultant to, and witness for, the NRC staff at our reopened hearing in October 1980.
Dr. H. Bolton Seed	Ph.D., Civil Engineering, Kings College, London University Professor of Civil Engineering, Geotechnical Engineering, University of California, Berkeley

Dr. Stewart W. Smith	Ph.D., Geophysics and Mathematics, California Institute of Technology Professor and Chairman, Graduate Program in Geophysics, University of Washington
Dr. Mihailo D. Trifunac	Ph.D., Civil Engineering and Geophysics, California Institute of Technology Associate Professor of Civil Engineering, University of Southern California