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October 20, 2011

PG&E Letter DCL-11-097

10 CFR 50.90

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555-0001

Diablo Canyon Units 1 and 2
Docket No. 50-275, OL-DPR-80
Docket No. 50-323, OL-DPR-82
License Amendment Request 11-05, "Evaluation Process for New Seismic Information and Clarifying the Diablo Canyon Power Plant Safe Shutdown Earthquake"

Dear Commissioners and Staff:

Pursuant to 10 CFR 50.90, Pacific Gas and Electric Company (PG&E) hereby requests approval of the enclosed proposed amendment to Facility Operating License Nos. DPR-80 and DPR-82 for Units 1 and 2 of the Diablo Canyon Power Plant (DCPP), respectively. The enclosed license amendment request (LAR) proposes to revise the current licensing basis, as described in the Final Safety Analysis Report Update (FSARU) and Technical Specifications (TS), to provide requirements for the actions, evaluations, and reports necessary when PG&E identifies new seismic information relevant to the design and operation of DCPP.

Through this LAR, PG&E proposes to: (1) clearly define an evaluation process for newly identified seismic information and incorporate ongoing commitments associated with the Long Term Seismic Program (LTSP) into the FSARU; and (2) clarify, consistent with the NRC Supplemental Safety Evaluation Report 7, that the 1977 Hosgri earthquake is the equivalent of DCPP's safe shutdown earthquake, as defined in 10 CFR 100, Appendix A.

The enclosure to this letter contains the evaluation of the proposed change. Attachments 1 and 2 of the enclosure include proposed TS markup and retyped pages, respectively. Attachment 3 of the enclosure includes FSARU markup pages. Attachment 4 of the enclosure includes a summary of regulatory commitments and changes. Attachments 5 through 7 of the enclosure include Chapters 5 through 7 of the 1988 LTSP Final Report, respectively.

PG&E has determined that this LAR does not involve a significant hazard consideration per 10 CFR 50.92. Pursuant to 10 CFR 51.22(b), no environmental

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impact statement or environmental assessment is required in connection with the issuance of this amendment.

PG&E requests approval of this LAR by September 29, 2012. PG&E requests that the license amendments be made effective upon NRC issuance, to be implemented within 180 days from the date of issuance.

During a June 29, 2011, telephone conference call with the NRC staff, PG&E was requested to provide a comparison of the current Standard Review Plan (SRP) with DCPP's licensing basis. The SRP comparison will be provided in a separate letter.

PG&E is making a regulatory commitment (as defined by NEI 99-04) in this letter. This letter also includes a revision to an existing regulatory commitment. Attachment 4 of the enclosure summarizes the regulatory commitment and the revision to an existing regulatory commitment made in this letter.

In accordance with 10 CFR 50.91, PG&E is notifying the State of California of this LAR by transmitting a copy of this letter and enclosure to the designated State Official.

If you have any questions or require additional information, please contact Mr. Tom Baldwin at (805) 545-4720.

I state under penalty of perjury that the foregoing is true and correct.

Executed on October 20, 2011.

Sincerely,

James R. Becker
Site Vice President

mjrm/gwh2/50350163

Enclosure:	Evaluation of the Proposed Change
cc/enc:	Gary W. Butner, California Department of Public Health
	Elmo E. Collins, NRC Region IV
	Michael S. Peck, NRC, Senior Resident Inspector
	James T. Polickoski, NRR Project Manager
	Alan B. Wang, NRR Project Manager
cc:	Diablo Distribution

Evaluation of the Proposed Change

Subject: License Amendment Request 11-05, "Evaluation Process for New Seismic Information and Clarifying the Diablo Canyon Power Plant Safe Shutdown Earthquake"

ATTACHMENTS:

1. Technical Specification Page Markups
2. Retyped Technical Specification Pages
3. FSAR Update Changes
4. Summary of Regulatory Commitments
5. Chapter 5 of the 1988 Long Term Seismic Program Final Report
6. Chapter 6 of the 1988 Long Term Seismic Program Final Report
7. Chapter 7 of the 1988 Long Term Seismic Program Final Report

1. SUMMARY DESCRIPTION

This letter is a request to amend Facility Operating License Nos. DPR-80 and DPR-82 for Units 1 and 2 of the Diablo Canyon Power Plant (DCPP), respectively.

This License Amendment Request (LAR) proposes to address licensing basis issues with respect to evaluations of new seismic information and to clarify that the 1977 Hosgri Earthquake spectrum (HE) is the equivalent of DCPP's safe shutdown earthquake (SSE).

The current DCPP licensing basis lacks a clear process for evaluating new seismic information. The proposed change would clearly define the evaluation to be performed upon discovery of new seismic information, and addresses Unresolved Item 05000275; 323/2011002-03, "Requirement to Perform an Operability Evaluation Following Receipt of New Seismic Information." (Reference 15)

The proposed amendment would add the following new Technical Specification (TS) Administrative Controls Programs:

(1) TS 5.5.20, "Long Term Seismic Program" (LTSP) to provide for ongoing review and evaluation of new seismic information and associated methodologies. The proposed evaluation process for new seismic information follows the seismic margin assessment performed by the LTSP compared to the HE. As described in Section 4 of this enclosure, "Technical Evaluation," under Subheading "1991 LTSP DGMRS as the comparison for new ground motion spectra," new seismic information will only be compared to the 1991 LTSP ground motion spectrum.

(2) TS 5.6.11, "Long Term Seismic Program Report" to inform the NRC of new, peer-reviewed seismic information that might affect the seismic risk to DCPP.

PG&E proposes to use the square-root-of-the-sum-of-squares (SRSS) method for the evaluation of load combinations of seismic with loss-of-coolant accident (LOCA). This method of combination is consistent with NUREG-0484, "Combining Dynamic Loads," Revision 1.

PG&E is in the process of performing evaluations for the combination of HE seismic loads with LOCA loads for reactor coolant system (RCS) loop piping and certain primary equipment. The above proposal to use SRSS as the methodology for evaluating the combination of seismic and LOCA loads is needed to address a non-conforming condition (DCPP corrective action program

Notification 50403189 and 50403377). PG&E anticipates that the evaluations will be completed after NRC review of this LAR and issuance of a license amendment.

The proposed amendment clarifies that the HE is the equivalent of DCPP's SSE to be used to demonstrate that the design basis requirements associated with the SSE continue to be met. The proposed amendment clarifies ongoing commitments to evaluate new seismic information for its significance to DCPP, maintain a seismic instrumentation program, and to design and construct future additions and modifications to DCPP in accordance with the existing seismic design basis.

The current DCPP Final Safety Analysis Report Update (FSARU) contains inconsistencies with documents issued by the NRC identifying the SSE for DCPP. While DCPP was licensed prior to 10 CFR 100, the definition of SSE (10 CFR 100, Appendix A) is:

> *[SSE] is that earthquake which is based upon an evaluation of the maximum earthquake potential considering the regional and local geology and seismology and specific characteristics of local subsurface material. It is that earthquake which produces the maximum vibratory ground motion for which certain structures, systems, and components are designed to remain functional. These structures, systems, and components are those necessary to assure: (1) The integrity of the reactor coolant pressure boundary, (2) The capability to shut down the reactor and maintain it in a safe shutdown condition, or (3) The capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of this part.*

Although DCPP is not a 10 CFR 100 licensed plant, the HE fits this definition for DCPP and; therefore, has appropriately been identified by the NRC as the equivalent of the DCPP SSE. In order to align the FSARU with the NRC conclusions in Supplemental Safety Evaluation Report (SSER) 7 during the DCPP licensing reviews and to eliminate regulatory uncertainty, PG&E proposes to incorporate the NRC's position that the HE (not the double design earthquake (DDE)) is the equivalent of DCPP's SSE, as defined in 10 CFR 100, Appendix A.

2. BACKGROUND

The NRC's predecessor agency, the Atomic Energy Commission (AEC), issued a construction permit (CP) for DCPP Unit 1 on April 23, 1968, and for Unit 2 on December 9, 1970. In 1975, the regulatory functions of the AEC were assumed by the NRC. After construction was complete, the NRC issued operating licenses (OLs) for DCPP. The NRC issued a full-power OL for Unit 1 on November 2, 1984, and for Unit 2 on August 25, 1985.

Before the NRC issued the DCPP CP, PG&E conducted geological and seismic investigations to validate the acceptability of the site. These investigations included regional studies and detailed onshore site investigations consisting of trenching, core drilling, and geological mapping in the vicinity of the site. During the time of the DCPP CP review, the NRC regulation that currently governs seismic design (10 CFR 100, Appendix A) was in the early stages of development, and the concepts of the SSE and operating basis earthquake (OBE) were still being developed.

At the time the CP was issued, PG&E concluded, and the AEC concurred, that the earthquake design bases for Diablo Canyon would be a peak horizontal ground acceleration (PGA) of 0.4 g for safety-related structures and a PGA of 0.2 g for operational-related structures. These seismic design criteria were based on consideration of two design-basis earthquakes: a magnitude 7.25 earthquake on the Nacimiento fault 20 miles from the site, and a magnitude 6.75 aftershock at the site associated with a large earthquake on the San Andreas fault. It was also concluded at that time that there was no surface displacement hazard (capable fault) in the site vicinity. This conclusion was based on the absence of any displacement of the 80,000 year-old and 105,000 year-old marine terraces underlying the site area.

Later, while geological investigations in support of the DCPP OL applications were under way, oil company geoscientists discovered that a major zone of faulting existed a few miles off shore from the plant site. This proprietary offshore geophysical information was made public in 1971. When the DCPP Final Safety Analysis Report (FSAR) was initially submitted for NRC review in 1973, it briefly described the offshore fault zone, calling it the East Boundary Fault Zone.

During the next few years, in response to NRC Staff requests for additional information, PG&E investigated this fault zone. In addition, the U.S. Geological Survey (USGS), with NRC funding, conducted numerous offshore investigations of the fault zone. The zone was later renamed the Hosgri fault. Based on the results of these studies, recommendations by the USGS, and the issuance of 10 CFR 100, Appendix A (1973), the NRC established that the equivalent SSE for DCPP is a horizontal PGA of 0.75g based on a postulated magnitude 7.5 earthquake on the Hosgri fault 5 kilometers (km) (3 miles) from the DCPP site.

Subsequently, PG&E reanalyzed and upgraded the plant to accommodate the new (Hosgri) seismic design basis in compliance with General Design Criteria (GDC) 2 (1967) and Safety Guide (SG) 29. The Hosgri earthquake is the most severe natural phenomena (earthquake) (GDC 2) that produces the largest vibratory ground motion at the plant site (10 CFR 100, Appendix A). All safety-related SSCs at DCPP have been designed to remain functional if an SSE occurs (SG 29). These SSCs are those necessary to assure: (1) the integrity of the reactor coolant pressure boundary, (2) the capability to shutdown the reactor and maintain it in a safe shutdown condition, or (3) the capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of 10 CFR Part 100 (SG 29 and Appendix A to Part 100).

The DCPP seismic design basis was reviewed and accepted by the NRC Staff in SSER 7. The NRC stated that:

> *. . . although the applicant does not agree, we now consider the Hosgri event to be the safe shutdown earthquake for the site, or at least its equivalent.*
>
> *The structures, systems and components that are being qualified for the Hosgri event in the seismic reevaluation are described in the various chapters of the Hosgri reevaluation report (Amendment 50 and subsequent amendments to the operating license application). These plant features are those necessary to assure (1) the integrity of the reactor coolant pressure boundary, (2) the capability to shutdown the reactor and maintain it in a safe shutdown condition, or (3) the capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of 10 CFR Part 100.*

The Atomic Safety and Licensing Board (ASLB) hearing of September 27, 1979, also concluded that a 7.5 magnitude earthquake on the Hosgri fault was conservative for the SSE for DCPP (LBP-79-26, 10 NRC 453 (1979). The ASLB stated:

> *Accordingly, the Board concludes that a 7.5 magnitude earthquake is a very conservative value for the safe shutdown earthquake. We also find that the requirement imposed by the Staff that a 7.5 magnitude earthquake be used by the applicant in its seismic analyses is reasonable and meets regulatory requirements.*
>
> *The Board finds that the Applicant has demonstrated through appropriate analysis and tests that Category I structure, systems and components will perform as required during the seismic load of the safe shutdown earthquake.*
>
> *The Board finds that Category I structure, systems and components will be adequate to assure (a) the integrity of the reactor coolant pressure boundary, and (b) the capability to shutdown the reactor and maintain it in a safe condition.*

The seismic design basis for DCPP was reviewed by the NRC's Advisory Committee on Reactor Safeguards (ACRS). On July 14, 1978, the ACRS issued a letter report to the Commission stating that it had completed its review of the OL application. The ACRS letter concluded that if due consideration were given to the items in its report, and subject to completion of the necessary plant modifications and preoperational testing, there was reasonable assurance that Units 1 and 2 could be operated at full power without undue risk to the health and safety of the public.

With regard to seismic issues, the ACRS stated:

> *The ACRS notes that, for distances less than 10 km from the earthquake source, there are currently no strong motion data for shocks larger than magnitude 6 and few reliable data for shocks of magnitude 5 and 6. Also, the theory and analyses of earthquake and seismic wave generation, of seismic wave transmission and attenuation, and of soil-structure interaction are in a state of active development. The Committee recommends that the seismic design of Diablo Canyon be reevaluated in about ten years taking into account applicable new information.*

It was this recommendation that eventually led to issuance of the conditions on the DCPP Unit 1 low-power and full-power OLs requiring a reevaluation of the seismic design bases of the plant. After public hearings before the NRC's ASLB and Atomic Safety and Licensing Appeal Board, and meetings with the NRC, OLs were issued for both DCPP units 1 and 2. License condition, Item 2.C.(7) was placed on Unit 1 Facility OL DPR-80 and reads as follows:

> *"Seismic Design Bases Reevaluation Program*
>
> *PG&E shall develop and implement a program to reevaluate the seismic design bases used for the Diablo Canyon Nuclear Power Plant.*
>
> *The program shall include the following Elements:*
>
> 1) *PG&E shall identify, examine, and evaluate all relevant geologic and seismic data, information, and interpretations that have become available since the 1979 ASLB hearing in order to update the geology, seismology and tectonics in the region of the Diablo Canyon Nuclear Power Plant. If needed to define the earthquake potential of the region as it affects the Diablo Canyon Plant, PG&E will also reevaluate the earlier information and acquire additional new data.*
>
> 2) *PG&E shall reevaluate the magnitude of the earthquake used to determine the seismic basis of the Diablo Canyon Nuclear Plant using the information from Element 1.*
>
> 3) *PG&E shall reevaluate the ground motion at the site based on the results obtained from Element 2 with full consideration of site and other relevant effects.*
>
> 4) *PG&E shall assess the significance of conclusions drawn from the seismic reevaluation studies in Elements 1, 2, and 3, utilizing a*

probabilistic risk analysis and deterministic studies, as necessary, to assure adequacy of seismic margins.

PG&E shall submit for NRC staff review and approval a proposed program plan and proposed schedule for implementation by January 30, 1985. *The program shall be completed and a final report submitted to the NRC three years following the approval of the program by the NRC staff.*

PG&E shall keep the staff informed on the progress of the reevaluation program as necessary, but as a minimum will submit quarterly progress reports and arrange for semi-annual meetings with the staff. PG&E will also keep the ACRS informed on the progress of the reevaluation program as necessary, but not less frequently than once a *year."*

The license condition was imposed because of: (1) the substantial amount of offshore exploration for hydrocarbons, (2) significant advances in geology, seismology, and geophysics that had occurred since the beginning of the site review, and (3) the ACRS recommendation quoted above.

The NRC's review and acceptance of PG&E's response to License Condition 2.C.(7), are discussed in NUREG-0675, "Safety Evaluation Report Related to the Operation of Diablo Canyon Nuclear Power Plant, Units 1 and 2," Supplement No. 34, dated June 1991 (SSER 34), and in NRC letter dated April 14, 1992, "Transmittal of Safety Evaluation Closing Out Diablo Canyon Long-Term Seismic Program (TAC Nos. M80670 and M80671)." SSER 34, Section 2.5.2.4, "Seismology Conclusions," included a restatement of two PG&E commitments with respect to ongoing activities associated with the implementation of the LTSP. These commitments are based on PG&E Letter DCL-91-091, "Benefits and Insights of the Long Term Seismic Program," dated April 17, 1991.

PG&E's reevaluation effort was named the "Long Term Seismic Program." The objective of the LTSP was to satisfy the license condition set forth above, using new techniques and data developed since 1979, to reevaluate the seismic design bases. The LTSP consisted of three phases. The Program Plan was developed in Phase I. In Phase II, the Program Plan was refined and the scope of work was focused and priorities established. In Phase III, the program tasks were implemented and documented.

With regard to the License Condition, Element 1, the NRC reviewed PG&E submittals and concluded that PG&E identified, examined, and evaluated all relevant geologic and seismic data and interpretations since the 1979 ASLB hearing. PG&E updated the geology, seismology, and tectonic characteristics of the DCPP region. PG&E reevaluated selected earlier information and acquired new data relating to the earthquake potential in the region as it affects DCPP. (Reference 2)

The NRC Staff found that the geological, seismological, and geophysical investigations and analyses conducted by PG&E and its consultants for the LTSP were the most extensive, thorough, and complete ever conducted for a nuclear facility in the United States, and advanced the state of knowledge in these disciplines significantly. On this basis, the NRC found that PG&E complied with License Condition Element 1 in an acceptable manner. (Reference 2)

License Condition Element 2 required that PG&E reevaluate the magnitude of the earthquake used to determine the seismic design basis at DCPP using the information developed for Element 1. The NRC reviewed the information submitted by PG&E and found that the conclusion reached during the Staff's review of the DCPP OL application, that the 1977 characterization of the Hosgri fault is the seismic source that could cause the maximum vibratory ground motion at the DCPP site, is still valid. The maximum credible earthquakes that could occur on any other fault or fault zone in the site vicinity would produce smaller ground motions at the site. PG&E concluded that the maximum earthquake associated with the Hosgri fault zone has a magnitude of 7.2 and could be located on the strand of the Hosgri that is nearest the site (the closest epicentral distance from the DCPP site is 4.5 km). The NRC reviewed the PG&E conclusion and found it acceptable. On this basis, the Staff found that PG&E met License Condition, Element 2. (Reference 2)

License Condition, Element 3 required that PG&E reevaluate the ground motion at the site with full consideration of site and other relevant effects. In order to determine the ground motion at the site, one necessary piece of data is an estimate of the style of faulting on the controlling fault. This is important because regression analyses of the empirical ground-motion database show that reverse-slip motion on the Hosgri fault would produce higher ground motion at the site than strike-slip motion, for the same earthquake magnitude. In the 1988 LTSP Final Report, PG&E concluded that earthquake motion on the Hosgri fault is best characterized as 65 percent strike-slip, 30 percent oblique-slip (midway between strike-slip and reverse-slip), and 5 percent thrust-slip (reverse-slip with a low dip angle). On the basis of its review and the advice of its consultants, the NRC found that the style of faulting on the Hosgri fault is predominantly right-lateral strike-slip, with a subordinate but substantial reverse (vertical) component. Specifically, the NRC concluded that ground motion at the site should be evaluated for an earthquake on the Hosgri fault that is two thirds strike-slip and one third reverse-slip.

The NRC reviewed PG&E's empirical ground-motion attenuation model and numerical modeling studies and performed an independent attenuation study to estimate ground motion at the DCPP site. The NRC's analysis was based on the NRC's estimate (described above) of the ratio of strike-slip to reverse-slip motion expected from an earthquake on the Hosgri fault. The resulting independently estimated ground-motion spectra at the plant site were compared to the spectra developed by PG&E for the LTSP.

The results showed that the NRC's estimates of both the 50th and 84th percentile horizontal ground-motion spectra at the site is equal to or less than the PG&E spectra at frequencies above 1 Hz, but exceed the PG&E spectra at frequencies at and below 1 Hz. For vertical ground motion, the NRC's 84th percentile vertical spectra exceed the PG&E vertical spectra over the frequency range from 1 to 10 Hz. PG&E met License Condition Element 3 by its reevaluation of ground motion at the site. To fully satisfy License Condition, Element 4, PG&E had to demonstrate that the plant structures can withstand these exceedances. PG&E submitted additional analyses to confirm LTSP conclusions that the seismic margins for structures and equipment at DCPP are adequate to accommodate the NRC's spectral estimates of horizontal and vertical ground motions defined in SSER 34 in PG&E Letters DCL-91-313 and DCL-92-077, dated December 26, 1991, and April 3, 1992, respectively. NRC letter, dated April 17, 1992, "Transmittal of Safety Evaluation Closing out Diablo Canyon Long-Term Seismic Program (TAC Nos. M80670 and M80671)," documented the NRC's review of these confirmatory analyses, concluding that the seismic margins of the structures, systems, and components (SSCs) at DCPP reported in the 1988 LTSP Final Report are adequate even after considering the NRC's estimate of increased seismic ground motions.

License Condition, Element 4 required PG&E to assess the significance of the conclusions drawn from License Condition, Elements 1, 2, and 3 using probabilistic and deterministic methods, as necessary, to assess seismic margin adequacy. PG&E performed a deterministic analysis as well as a probabilistic risk assessment (PRA) and concluded that the plant seismic margins are adequate.

PG&E performed detailed soil-structure interactions (SSI) analyses to determine the effects of dynamic interaction between the plant structures and the foundation rock underlying the plant on the seismic response of plant structures. The analyses showed that the effects of ground motion incoherence and embedment of structures lumped into the "tau effect" in previous studies reduce the seismic response of some plant structures, but not others. The NRC found, based on its review of PG&E analyses and on analyses conducted by NRC consultants, that the PG&E SSI analyses were comprehensive, thorough, and acceptable.

The PRA analysis conducted by PG&E included both internal and external events. The objectives of the PRA were to: (1) assess the importance of various structures and items or equipment to seismic risk; and (2) put the seismic risk in perspective by comparing it to the risk from other external and internal initiators. Risk in this context refers primarily to the estimated core damage frequency (CDF). The PRA results indicated that the mean overall CDF for DCPP was similar to that of other nuclear plants.

PG&E performed deterministic comparisons using its LTSP ground-motion estimates and showed that the major plant structures at DCPP have adequate seismic margins. As described above, the NRC's estimates of horizontal and vertical ground-motion spectra exceed PG&E's estimates and resulted in the NRC requiring PG&E to perform additional analyses to confirm LTSP conclusions that the seismic margins for structures and equipment at DCPP are adequate to accommodate the NRC's spectral estimates of horizontal and vertical ground motions. The NRC's review of these additional analyses concluded that the seismic margins of the SSCs at DCPP reported in the 1988 LTSP Final Report are adequate even after considering the NRC's estimate of increased seismic ground motions.

The NRC reviewed PG&E's PRA and deterministic analyses of selected SSCs and found them acceptable and concluded that PG&E met License Condition, Element 4, and therefore finding that License Condition 2.C.(7) of OL-DPR-80 was met. The NRC Staff summarized their review and conclusions about the LTSP in Supplement No. 34 to the Safety Evaluation Report (SER) (Reference 2), and NRC letter dated April 14, 1992, "Transmittal of Safety Evaluation Closing out Diablo Canyon Long-Term Seismic Program (TAC Nos. M80670 and M80671)." In these conclusions, the NRC also noted that the seismic qualification basis for Diablo Canyon will continue to be the original design basis plus the HE evaluation basis, along with the associated analytical methods, initial conditions, etc. For future plant design modifications, the NRC concluded that the LTSP spectra, increased to envelope the exceedances in the vertical and horizontal spectra discussed in Section 2.5.2.3 of SSER 34, should be used to verify that the plant high-confidence-of-low-probability of failure (HCLPF) values remain acceptable.

As part of the ongoing review process, PG&E made the following commitments at a public meeting on March 15, 1991, and in a letter to the NRC (Reference 3): (1) continue to maintain a strong geosciences and engineering staff to keep abreast of new geological, seismic, and seismic engineering information and evaluate it with respect to its significance to DCPP, and (2) continue to operate a strong-motion accelerometer array and the coastal seismic network, although likely with fewer stations than are currently operating. Since some issues (i.e., slip type of the Hosgri, the characterization of the Southwest Boundary Zone, and ground-motion estimates for oblique-slip earthquakes) are controversial because of the lack of definitive evidence, future geoscience discoveries may allow a more robust conclusion for these issues.

PG&E has continued to fulfill these commitments through its ongoing geosciences research and evaluations. However, an evaluation process for new seismic information with NRC approved acceptance criteria is not specifically defined in DCPP's current licensing basis. The purpose of the proposed change to the FSARU and TS is to clearly define an evaluation process for newly identified seismic information.

3. DETAILED DESCRIPTION

Proposed Amendment

The following changes are proposed to TS 5.0, "Administrative Controls":

A new TS Program 5.5.20, "Long Term Seismic Program" stating:

> This program provides ongoing review and evaluation of new seismic information and associated methodologies. The program shall include the following:
>
> a. A staff to keep abreast of new geological, seismic, and seismic engineering information and evaluate it with respect to its significance to DCPP;
>
> b. Operation of a strong-motion accelerometer array and the coastal seismic network;
>
> c. Verification that plant seismic margins remain acceptable for plant additions and modifications when checked against insights and knowledge gained from the Long Term Seismic Program, as identified in FSARU Section 3.7.6;
>
> d. Deterministic seismic margin acceptance criteria for operability determinations;
>
> e. Peer review process requirements for seismic probabilistic risk assessment revisions;
>
> f. Peer review process requirements for seismic model or methodology revisions; and
>
> g. Minimum requirements for the Seismic Advisory Board (SAB).

The above incorporates existing commitments into the TS as 5.5.20.a and 5.5.20.b, with "strong geosciences and engineering staff" revised to "staff" in 5.5.20.a.

A new TS Reporting Requirement 5.6.11, "Long Term Seismic Program Report" stating:

> A report shall be submitted once every 10 years, based on the submittal date of the previous update. An updated report will be submitted in less than 10 years if new peer reviewed seismic information becomes available that would significantly increase the risk to DCPP. The report shall include the following information:

a. Geology/seismology/geophysics/tectonics investigations,

b. Seismic source characterization,

c. Characterization of ground motions,

d. Soil/structure interaction analysis,

e. Probabilistic risk analysis,

f. Deterministic evaluations,

g. Assessment of the adequacy of seismic margins,

h. Documentation of the review performed by the Seismic Advisory Board (SAB) and the resolution of the SAB's comments if performed in less than 10 years, and

i. Documentation of the review performed by the Senior Seismic Hazards Analysis Committee for 10 year updates.

There is no change to the TS Bases associated with this LAR.

The proposed FSARU changes generally can be sorted into the following categories:

1. Clarifying the 1977 Hosgri earthquake spectrum as DCPP's SSE

2. Proposed method of evaluation of new seismic information

3. Clarification of ongoing commitments associated with LTSP

The specific changes are included in Attachment 3 of this enclosure. Some of the markup pages address multiple categories (from above), provide reference to another change, or provide clarification based on historic information. In these cases, they are listed based on the best fit. These following lists are provided for convenience only.

Clarifying the 1977 Hosgri Earthquake Spectrum as DCPP's SSE

The following FSARU sections are revised to address the SSE:	
FSARU Section	**Title**
1.2.1.6	Seismology
2.5.1	Basic Geologic and Seismic Information
2.5.2.2	Underlying Tectonic Structures
2.5.2.5	Earthquake History

The following FSARU sections are revised to address the SSE:	
FSARU Section	**Title**
2.5.2.7	Identification of Active Faults
2.5.2.8	Description of Active Faults
3.1.2.2	Criterion 2, Performance Standards (Category A)
3.2	Classification of Structures, Systems, and Components
3.2.1	Seismic Classification
3.2.2	Design Classification
3.2.3	Quality Assurance Classification
3.2.4	Piping Classification Symbols
3.2.5	System Quality Group Classifications
3.2.5.1	Design Class I, Quality/Code Class I Fluid Systems and Fluid System Components
3.2.5.2	Design Class I, Quality/Code Class II Fluid Systems and Fluid System Components
3.2.5.3	Design Class I, Quality/Code Class III Fluid System Components
3.2.5.4	Other Fluid Systems and Fluid System Components
3.2.5.5	Summary of System Quality Group Classifications
3.2.6	References
3.2.7	Reference Drawings
3.7.1.1.1	Design Earthquake (DE)
3.7.1.1.2	Double Design Earthquake (DDE)
3.7.1.1.3	1977 Hosgri Earthquake (HE)
3.7.3.15.3	Control Rod Drive Mechanism Evaluation
3.7.3.15.4	CRDM Support System Evaluation
3.8.1.1	Description of the Containment
3.9.3.1	Core and Internals Integrity Analysis (Mechanical Analysis)
3.9.3.5.1	Blowdown Forces Due to Cold and Hot Leg Break
3.10.2.7.1	4160 V Metal-Clad Switchgear
3.10.2.32.1	RVLIS/Incore Thermocouple Cabinets
Table 4.1-3	Design Loading Conditions for Reactor Core Components
5.2.1.5.4	Faulted Conditions
5.2.1.7	Design of Active Pumps and Valves
5.2.1.11	Analysis Method for Faulted Condition
5.2.1.14	Stress Analysis for Faulted Condition Loadings (DDE and LOCA)
5.2.1.15 (New)	Stress Analysis for Faulted Condition Loadings (Hosgri and LOCA)
5.2.1.15.1	Integrated Reactor Coolant Loop Analysis
5.2.1.15.2	Steam Generator Evaluation
5.2.1.15.3	Reactor Coolant Pump Evaluation
5.2.1.15.4	Reactor Vessel Evaluation
5.2.1.1.15.8	Primary Equipment Support Evaluation
5.2.1.15.9	Pressurizer Evaluation
Table 5.2-6	Load Combinations and Stress Criteria for Westinghouse

The following FSARU sections are revised to address the SSE:	
FSARU Section	**Title**
	Primary Equipment
Table 5.2-8	Loading Combinations and Acceptance Criteria for Primary Equipment Supports
Table 5.2-16	Reactor Coolant Boundary Leakage Detection System
6.3.1.4.3	Seismic Requirements
9.1.1.2	Facilities Description
Appendix 9.5B	Regulatory Compliance Summary
Appendix 9.5C	Reactor Coolant Pump Oil Collection System, Evaluation to 10 CFR 50, Appendix R, Section III.O
Section 15	Accident Analysis
15.4.5.1.2	Probability of Activity Release
Table 15.4.1-7A	Unit 1 Plant Operating Range Allowed by the Best-Estimate Large Break LOCA Analysis
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Proposed Method of Evaluation of New Seismic Information

Proposed Method of Evaluation of New Seismic Information	
FSARU Section	**Title**
2.5.6.1 (New)	Ongoing Geological and Seismological Investigations
2.5.6.2 (New)	Evaluation of Updated LTSP Ground Motions
2.5.6.2.1 (New)	Seismic Margin Evaluation
2.5.6.2.1.1 (New)	Approved Minimum Seismic Margins Less Than 1.3
2.5.6.2.2 (New)	Probabilistic Risk Assessment Evaluation
2.5.6.3 (New)	LTSP Configuration Control
2.5.6.4 (New)	Elements of a Seismic Margins Evaluation
2.5.7	References
Figure 2.5-38 (New)	Flowchart for Evaluation of Updated LTSP Ground Motion
Figure 2.5-39 (New)	1991 LTSP Fragility Curve Representation
Figure 2.5-40 (New)	Schematic Illustration for the Determination of Seismic Margins

Clarification of Ongoing Commitments Associated with LTSP

Clarification of Ongoing Commitments Associated with LTSP	
FSARU Section	**Title**
2.5	Geology and Seismology
2.5.2.9	“Design and Licensing Basis Earthquakes
2.5.2.9.1	Maximum Earthquake (Design Earthquake)
2.5.2.9.2 (New)	Double Design Earthquake

Clarification of Ongoing Commitments Associated with LTSP	
FSARU Section	**Title**
2.5.2.9.3	1977 Hosgri Earthquake
2.5.2.9.4 (New)	1991 Long Term Seismic Program Spectra
2.5.2.10 (New)	Ground Accelerations and Response Spectra
2.5.2.10.1	Maximum Earthquake (Design Earthquake)
2.5.2.10.2 (New)	Double Design Earthquake
2.5.2.10.3	1977 Hosgri Earthquake
2.5.2.10.4 (New)	1991 Long Term Seismic Program Spectra
2.5.4.9	Earthquake Design Basis
2.5.6	Long Term Seismic Program
Figure 2.5-33 (New)	Free Field Spectra – Horizontal 1991 LTSP (84th Percentile Nonexceedance) As Modified Per SSER-34
Figure 2.5-34 (New)	Free Field Spectra – Vertical 1991 LTSP (84th Percentile Nonexceedance) As Modified Per SSER-34
Figure 2.5-35 (New)	Free Field Spectra – Horizontal LTSP (PG&E 1988) Ground Motion vs. Hosgri (Newmark 1977)
Figure 2.5-36 (New)	1988 LTSP Seismic Hazard Curve
Figure 2.5-37 (New)	1991 LTSP Uniform Hazard Spectrum
3.7.1	Seismic Input
3.7.1.1	Design Response Spectra
3.7.1.1.4 (New)	1991 Long Term Seismic Program Earthquake (LTSP)
3.7.1.2	Design Response Spectra Derivation
3.7.1.2.1	Design Earthquake (DE) and Double Design Earthquake (DDE) Derivation
3.7.1.2.2	1977 Hosgri Earthquake Derivation
3.7.1.2.3 (New)	1991 Long Term Seismic Program Earthquake (LTSP)
3.7.4	Seismic Instrumentation Program" (Section 3.7.4 subsections renumbered)
3.7.4.1	Seismic Monitoring System
3.7.4.1.1	Comparison with NRC Regulatory Guide 1.12, Revision 2
3.7.4.1.2	Description of Instrumentation
3.7.4.1.2.1	Strong Motion Triaxial Accelerometers
3.7.4.1.3	Control Room Operator Notification
3.7.4.1.4	Comparison of Measured and Predicted Responses
3.7.4.2 (New)	Central Coast Seismic Network
3.7.6 (New)	Application of the LTSP to Modifications and Additions (previous content deleted – redundant to FSARU Section 3.2)
3.7.6.1 (New)	Basis for Selection of LTSP Evaluation Scope
3.7.6.1.1 (New)	Modifications and Additions in the LTSP Evaluation Scope
3.7.6.1.2 (New)	Modifications and Additions Excluded from LTSP Evaluation Scope
3.7.6.2 (New)	LTSP Evaluation Process
3.7.6.2.1 (New)	Fragility Analysis Method

Clarification of Ongoing Commitments Associated with LTSP	
FSARU Section	**Title**
3.7.6.2.2 (New)	Conservative Deterministic Failure Margins Method
3.7.6.2.3 (New)	Earthquake Experience Data Method
3.7.7	References
Table 3.7-25 (New)	High Confidence Low Probability of Failure ($HCLPF_{84}$) Capacities and Seismic Margins for Civil Structures
Table 3.7-26 (New)	High Confidence Low Probability of Failure ($HCLPF_{84}$) Capacities and Seismic Margins for Equipment and Components
Figure 3.7-29 (New)	Sample Free Field Ground Motion LTSP Analysis Longitudinal Component
Figure 3.7-30 (New)	Sample Free Field Ground Motion LTSP Analysis Transverse Component
Figure 3.7-31 (New)	Sample Free Field Ground Motion Comparison to Target Spectrum - LTSP Analysis - Longitudinal Component – 5% Damping Ratio
Figure 3.7-32 (New)	Sample Free Field Ground Motion Comparison to Target Spectrum - LTSP Analysis – Transverse Component – 5% Damping Ratio
Figure 3.7-33 (New)	LTSP Evaluation Process for Plant Additions and Modifications
Table 3.9-9	List of Active Valves (Notes revised)

4. TECHNICAL EVALUATION

Incorporating the NRC's Position from SSER 7 / SSER 34 on the 1977 Hosgri Earthquake Spectrum as DCPP's SSE

The ASLB in a Partial Initial Decision, dated September 27, 1979, concluded that (page 490) the 0.75g acceleration assigned to the SSE to be an appropriately conservative value for the maximum vibratory ground acceleration that could occur at the DCPP site. As discussed in the Background section, the NRC stated in SSER 7 (pages 2-5) that they consider the Hosgri event to be the SSE for this site, or at least its equivalent. Accordingly, PG&E is updating the FSARU to align with the conclusions in the licensing reviews that the maximum ground acceleration that could occur at the site, the HE, is the equivalent of the DCPP SSE.

The SSCs necessary to assure: (1) the integrity of the reactor coolant pressure boundary, (2) the capability to shutdown the reactor and maintain it in a safe shutdown condition, or (3) the capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of 10 CFR Part 100, were qualified for the HE as described in the various chapters of the Hosgri reevaluation report (Amendment 50 and subsequent amendments to the operating license application). Because PG&E used criteria that was current at the time of the reevaluation in determining what items should be qualified, there were some differences between the set of items identified as part of the reevaluation and the set that was originally designated Seismic Category I. The NRC reviewed the set of designations developed as part of the reevaluation in two ways. Note that the classification system for SSCs is unique to DCPP and does not explicitly use the term "Seismic Category I." Equivalencies between DCPP's classification system and that used by the NRC in Regulatory Guide (RG) 1.29 are described in FSARU Section 3.2.1.

In the first review, the NRC applied the criteria as documented in RG 1.29, "Seismic Design Classification," to determine the SSCs that need to be designed to withstand the effects of the HE and remain functional. As in the original review, the NRC concluded that SSCs important to safety that are designed to withstand the effects of the HE and remain functional have been properly classified in conformance with the NRC's regulations, the applicable RG, and industry standards. Note that the SG 29, "Seismic Design Classification," is DCPP's current licensing basis, not RG 1.29, but these two documents contain similar provisions. In accordance with the NRC's normal acceptance criteria, qualification of these items for the HE provides reasonable assurance that the plant will perform in a manner providing adequate safeguards for the health and safety of the public with respect to earthquake safety. The criteria, as documented in RG 1.29, continues to be applied to ensure that the required SSCs are designed to withstand the effects of the HE and remain functional.
In the second review, at the NRC's request, PG&E considered the equipment and procedures necessary to achieve long-term cold shutdown conditions after the HE, assuming that: (1) only equipment qualified for the event would be available, (2) single failures may occur in that equipment, and (3) offsite power may be lost for an extended period of time. PG&E submitted the results of this evaluation to the NRC in a letter dated January 26, 1978.

The NRC reviewed the capability to cool the plant to cold shutdown conditions and provide long-term cooling and concluded that PG&E demonstrated that sufficient systems are available for residual heat removal with or without offsite power and assuming a single failure in accordance with Criterion 34 of the GDC. Similarly, these systems are qualified for operation in the event of the HE in accordance with Criterion 2 of the GDC (SSER 7 pages 3-1 thru 3-4, and SSER 8 pages 3-1 thru 3-3).

The evaluation methods used for HE were generally the same as those used in the original seismic design with the approval of different methods of analysis for the items listed in SSER 7 Sections 3.8.5.3, 3.9.3.2, and 3.10.2, which the NRC concluded are conservative and provide adequate safety margins in the design of Category I components.

The HE evaluation methods for structures use the following deviations from the methods used for the DE and DDE seismic analyses, as discussed in SSER 7:

(1) The use of damping values recommended in RG 1.61, "Damping Values for Seismic Design of Nuclear Power Plants," Revision 0, unless the use of a later revision has been approved by the NRC for specific applications. In most cases, the damping values used for the DE and DDE analyses were conservative relative to the values recommended in RG 1.61, resulting in larger calculated responses.

The NRC concluded, in SSER 7, that the use of higher damping values consistent with the guidance provided in RG 1.61 is realistic and acceptable.

(2) Average values of material properties, from tests of the actual materials installed, are used to determine allowable stress levels instead of using code specified minimum material properties, as was used for the DE and DDE seismic analyses.

The NRC concluded in SSER 7 that the use of actual material strengths is acceptable since some margin remains. For concrete, the appropriate average 28-day test strength is used. Since concrete continues to gain strength with age after 28 days, the installed concrete will be stronger. For steel, average mill test strength is used. Since the steel is ductile and the structures are designed to remain below yield (with a limited number of exceptions), margin remains.

(3) Ductility (yielding) in structures is allowed in certain cases. Structural ductility has not generally been used. Where used, it is justified for each specific case. Appropriate assurance is made that seismic inputs to systems and equipment is not underestimated due to structural ductility in such instances. (This can readily be done by calculating the inputs to systems and equipment separately, assuming the structure does not yield). Based on these conditions, the NRC considered the use of structural ductility acceptable. Ductility was not used in the DE and DDE seismic analyses.

(4) Fixed base mathematical models are used for structures and above ground tanks. A SSI analysis, as was done for DE and DDE, is not necessary for the HE evaluation, due to the stiffness of the rock foundation material.

The use of fixed base analyses is consistent with the NRC's current criteria for rock sites such as the DCPP site. The NRC concluded in SSER 7 that this method precludes the reduction of the ground motion that often results from SSI analyses using deconvolution and is considered acceptable.

(5) The horizontal ground response spectra are adjusted to account for foundation size effects in relation to ground motion waves (nonsynchronized ground motion or spatial incoherence). Such

adjustments to the horizontal ground response spectra were not included in the DE and DDE seismic analyses.

The NRC concluded in SSER 7 that where such credit is taken in relation to the usual procedure of assuming synchronized ground motion; an appropriate consideration of other effects of nonsynchronized ground motion such as torsion is also included.

This is accomplished by assuming an artificial eccentricity between the center of mass and center of rigidity of the building. This is above and beyond any actual (geometric) eccentricity, which is also computed and accounted for. The effect of this artificial eccentricity is to force the horizontal ground motion that is used in the analysis to create additional torsional motions about the vertical axis.

An eccentricity of five percent or seven percent of the width of the structures is assumed, depending on the technique used to combine the torsional with the translational responses. This is in addition to any actual eccentricity of the structures. The five percent eccentricity is used when the torsional and translational responses are combined by the absolute sum rule, and the seven percent eccentricity is used when the two responses are combined by the SRSS rule. The greater of the combined responses is used.

With regard to floor response spectra, the torsional floor response spectra at the center of mass is calculated using actual (geometric) eccentricity of the structure in addition to an assumed eccentricity equal to five percent of the structural dimension. The NRC concluded that these approximating techniques represented a step towards more realistic modeling of structural responses and were therefore found to be acceptable.

(6) A vertical response dynamic analysis is performed rather than assuming an invariant vertical acceleration throughout the structures as was done for the DE and DDE analyses. This is consistent with the structural dynamic analysis methods applicable at the time of the Hosgri evaluation.

The NRC concluded in SSER 7 that the use of a vertical response dynamic analysis is more accurate than the methodology used for the DE and DDE analyses and is therefore acceptable.

(7) A modified procedure is used for smoothing and widening of the raw floor response spectra. The smoothing is done by averaging of floor response spectra, except at the peaks, where it is widened by 15 percent on the low frequency side and five percent on the high frequency side without reduction of the peaks. In the analysis for the DE and DDE, the peaks were widened by 10 percent on both sides after being lowered by 10 percent.

The purpose of widening the peaks is to account for possible variations in the predicted structural frequencies. At the time of the Hosgri reevaluation, the NRC's criteria indicate widening by 15 percent on both sides of the peaks. However, since actual material strengths are being used in the reevaluation, the calculated structural stiffness is closer to the maximum stiffness than usual, indicating a lesser need for peak broadening on the high frequency side. For these reasons, the NRC considered the approach used for the HE evaluation as acceptable.

(8) In combining structural responses at each point, responses due to horizontal excitation in two directions are combined with the response due to vertical excitation by the SRSS rule. In the analysis for DE and DDE, one response due to horizontal excitation and one response due to vertical excitation were combined by the absolute sum method. The process was repeated for the other horizontal component and the more limiting result was employed for design.

This approach corresponds to the recommendations of RG 1.92, "Combining Modal Responses and Spatial Components in Seismic Response Analysis," December 1974. The NRC determined that this approach is acceptable (SSER 7, Section 3.8.5.3).

The HE evaluation methods for mechanical systems and components use the following deviations from the methods used for the DE and DDE seismic analyses:

(1) Damping values recommended in RG 1.61, Revision 0, are generally used. Damping values from a later revision for RG 1.61 has been approved by the NRC for specific applications. In most cases, the damping values used for the DE and DDE analyses were conservative relative to the values recommended in RG 1.61, resulting in greater calculated responses.

The NRC concluded in SSER 7 that the use of higher damping values consistent with the guidance provided in RG 1.61 is realistic and acceptable.

(2) Actual material properties are used, where available, in lieu of code specified minimum properties to establish allowable stress limits to justify structural integrity where the calculated stress exceeded the limits of The American Society of Mechanical Engineers Boiler and Pressure Vessel Code (ASME Code). The ASME Code allowable values were used in the DE and DDE analysis.

Allowable stress values are established using the bases prescribed by Appendix III of Section III of the ASME Code so that the factors of safety used in the code are preserved. For this reason, the NRC considered the use of actual material properties acceptable.

(3) The responses to HE loads or the DDE loads (whichever is more limiting) are combined with the response due to normal operation and the response due to LOCA loads.

This is a conservative method which results in the RCS being designed for loads well in excess of those calculated for a seismic event alone without a pipe break. Even though the assumed seismic event is not expected to cause a pipe break in a seismically designed piping system, these loads are combined for design purposes to produce extra margin (SSER 7, Section 3.9.3.2).

In a letter dated November 10, 1977, the NRC requested the following:

> In assessing the design adequacy of piping, other pressure retaining components and their supports, the combination of loads due to Hosgri earthquake and the loss of coolant accident (LOCA) has to be considered. In the subject report, however, this effect was not considered. Submit the results of your analysis which consider the effects of combining the normal operating loads, the earthquake loads and the LOCA loads. Explain and justify the method of load combination.

In response to this request, PG&E submitted the Westinghouse report titled, "Response to Combinations of Calculated Loads for Pipe Break and Earthquake," as part of Appendix F to the PG&E report, "Seismic Evaluation for Postulated 7.5M Hosgri Earthquake." As part of this evaluation the combination of a postulated LOCA and seismic event were considered in the following four ways:

a) No combination, seismic alone
b) No combination, LOCA alone
c) Seismic and LOCA by absolute summation
d) Seismic and LOCA by SRSS

The evaluation of the effects of these various combinations showed that calculated stresses for the RCS were below allowable values with the exception of the following:

a) For both the absolute summation and SRSS combinations of LOCA and seismic, the stress in the fuel grid exceeds the strength based on testing. Exceeding grid strength allowables cause minor deformation in the grid. The resulting flow reduction has been evaluated and shown not to significantly affect the emergency core cooling system (ECCS) performance of the system.

b) For the absolute summation combination of LOCA and seismic, the computed stresses for the reactor internals, the reactor coolant pump supports, and the reactor vessel support were over the allowable values by a small amount. However, these exceedances do not affect the function of the supports or internals.

The evaluation demonstrated that the entire primary system is capable of withstanding the simultaneous occurrence of the peak loads of the HE or DDE and a LOCA without compromising its ability to safely shut down the system and retain it in a shutdown condition.

SSER 7 documents the NRC's review and approval of this evaluation. The NRC concluded that the evaluation was acceptable based on the conservative process of requiring that the peak responses to the seismic and LOCA loads be combined on an absolute summation basis. However, the NRC's acceptance of the evaluation is inconsistent with the results of the evaluation determining that four components exceeded allowable values when using the absolute sum method.

PG&E proposes to use the SRSS method for the evaluation of load combinations of seismic with LOCA. This method of combination is consistent with NUREG-0484, "Combining Dynamic Loads," Revision 1.

The HE evaluation methods for electrical equipment use the seismic qualification methods of RG 1.100, Revision 1, "Seismic Qualification of Electrical Equipment for Nuclear Power Plants," and IEEE Standard 344-1975, "IEEE Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations." Electrical equipment was originally qualified for the DDE in accordance with IEEE Standard 344-1971, "IEEE Guide for Seismic Qualification of Class I Electrical Equipment for Nuclear Power Generating Stations." The methods identified in RG 1.100, Revision 1, and IEEE Standard 344-1975 were used, per the NRC's request, during the Hosgri reevaluation of components where the original qualification level did not envelope the required seismic inputs to equipment for the HE.

With the use of the previously approved methods of analysis to the original design analysis, it has been demonstrated that the SSCs necessary to assure: (1) the integrity of the reactor coolant pressure boundary; (2) the capability to shutdown the reactor and maintain it in a safe shutdown condition; or (3) the capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of 10 CFR Part 100 were qualified for the HE. Therefore the DCPP SSE can be clarified as the HE.

<u>Proposed Method of Evaluation of Updated Seismic Information, Seismic Margin Evaluation, and 1991 LTSP Design Ground Motion Response Spectrum (DGMRS) as the comparison for new ground motion spectra</u>

The use of the 1991 LTSP DGMRS as the comparison spectra for new ground motion spectra is justified because the 1977 HE (DCPP SSE) envelopes the

1991 LTSP with the exception of minor exceedances in the high frequency range (greater than approximately 15 Hz), which were approved by the NRC in SSER 34, Section 3.8.1.1. This conclusion is supported by SSER 34, Section 3.8.1.4, "Comparison of LTSP and Seismic-Qualification-Basis Response Spectra":

> *Originally, PG&E did not make a one-to-one comparison of the response spectra resulting from LTSP site-specific ground motions with the seismic-qualification-basis spectra (that were prepared at different times) from two other earthquakes, the double-design earthquake and the Hosgri earthquake. PG&E based this decision on the lack of direct comparability due to the differences between the LTSP assumptions and analysis methodology and those adopted for the Hosgri reevaluations. Although generally agreeing with PG&E's arguments in this area, during its review of the LTSP, the staff required PG&E to make this comparison to be able to judge the level of demand resulting from the LTSP ground motion. If the Hosgri responses were found to be greater than the LTSP responses, then no additional evaluation would be needed.*

In order to be consistent with the approach used by PG&E under the initial implementation of the LTSP as discussed above, the updated DGMRS associated with new seismic information will be compared to the 1991 LTSP DGMRS, and no additional evaluations to the DE or DDE DGMRS need be performed. The comparison of the updated DGMRS to the 1991 LTSP DGMRS implicitly includes a comparison to the 1977 HE DGMRS, based on the comparison of the 1991 LTSP DGMRS and the 1977 HE DGMRS discussed in SSER 34, Section 3.8.1.1.

The DE and DDE seismic design basis is based on historical predictions of two specific faults, the Nacimiento and the San Andreas. The seismic design basis criteria for the DE and DDE were established in the original September 28, 1973, FSARU, Section 3.7. Proposed changes to FSARU Sections 3.7.1.1.1, 3.7.1.1.2, and 3.7.1.1.3, included in Attachment 3 of this enclosure, describe the DE, DDE, and HE. The DDE design response spectra were made twice that of the DE. The design damping values were established as follows:

Type of Structure	% Critical Damping - DE	% Critical Damping - DDE
Containment structures and all internal concrete structures	2.0	5.0
Other conventionally reinforced concrete structures above ground, such as shear walls or rigid frames	5.0	5.0
Welded structural steel assemblies	1.0	1.0

Bolted or riveted steel assemblies	2.0	2.0
Vital piping systems	0.5	0.5
Foundation rocking *	5.0	5.0

* Five percent of critical damping is used for structures founded on rock for the purpose of computing the response in the rocking mode, and seven percent of critical damping is used for the purpose of computing the response in the translation mode.

There are situations in the current seismic design evaluations indicating that in-building response spectra for the DE and DDE have larger peak accelerations than those for the HE. These occurrences are due to the use of original DE and DDE seismic design basis criteria for SSI modeling and the use of damping values based on seismic design knowledge of the early 1970 time period. The DE and DDE seismic design criteria remain unchanged to this day and will remain fixed design criteria, not subject to modification as a result of new seismic information. Updated ground motions based on new seismic information will be compared to the 1991 LTSP DGMRS that was compared to the 1977 Hosgri DGMRS per the LTSP process. The design criteria for the HE does not include SSI modeling (replaced with a fixed based model for free-field ground motion/structure interaction) and used the more contemporary damping values based on RG 1.61. The NRC acknowledged the conservatisms in the original design criteria for the DDE in SSER 7, Section 3.7:

In a few individual cases, the applicant has demonstrated that the double design earthquake loads determined from the original analysis are more limiting than Hosgri event loads. This may at first appear confusing and raise a question as to how the original can be so conservative as to exceed the Hosgri event loads. It can happen in a few cases due to highly conservative assumptions or methods in the original analysis. In any event, if the applicant has used a load in the original design and can now demonstrate that the Hosgri event load is less, we consider this to be a sufficient load determination.

Where the original analysis is more limiting, the applicant has chosen not to take credit for the lesser Hosgri event loads, but rather to use the more limiting double design earthquake loads.

Also, in SSER 7, Section 3.7, the NRC stated:

[W]e discussed the applicant's original seismic design methods and procedures and found them acceptable in relation to the original seismic design criteria. This conclusion has not been changed.

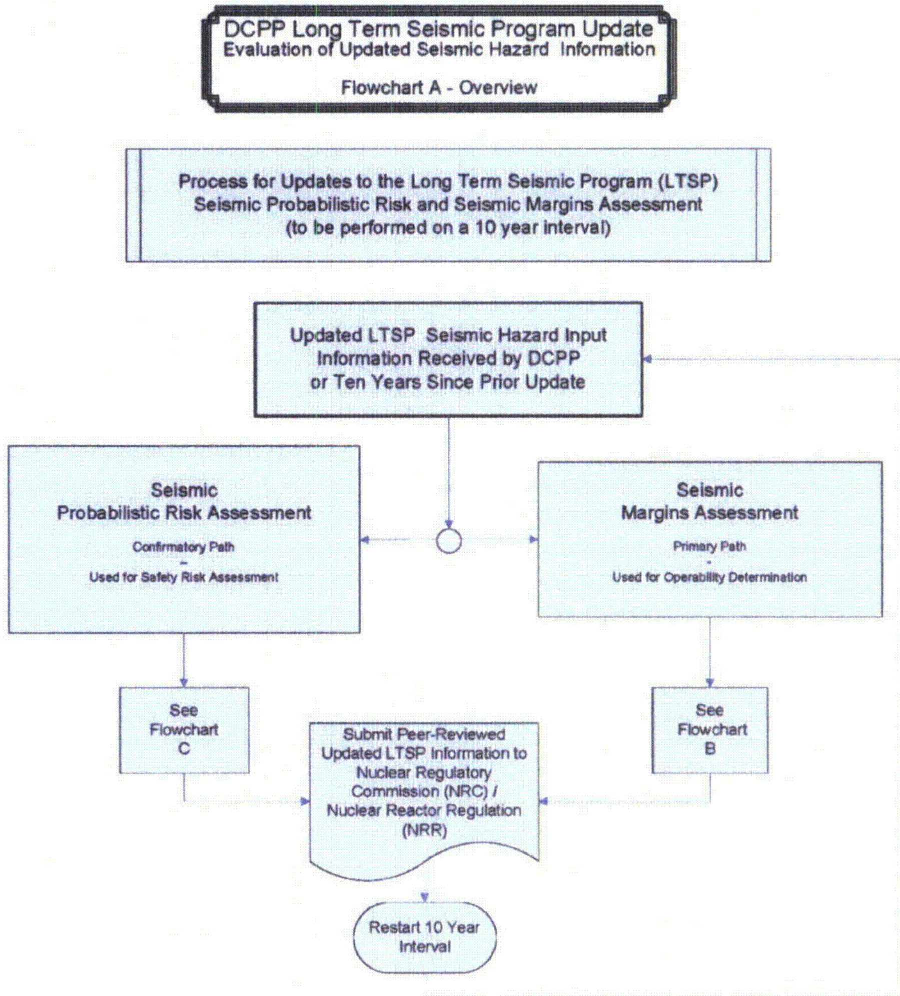
With regard to the design earthquake or operating basis earthquake, we have concluded in Section 2.5 of this supplement that the original operating basis earthquake remains unchanged for this site. Accordingly, there is no need for *any further work by the applicant with regard to operating basis earthquake design matters.*

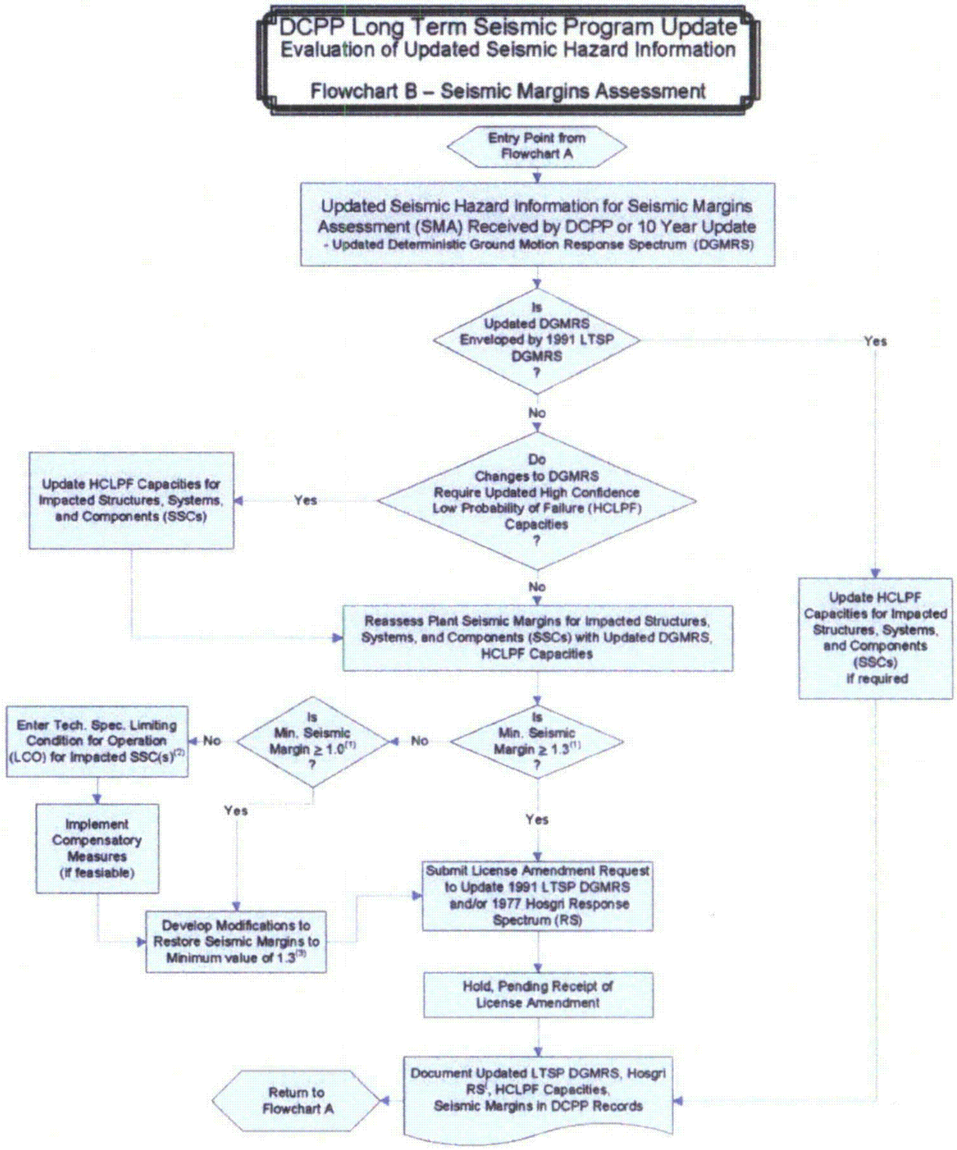
In SSER 34, Section 1.4, the NRC noted that the seismic qualification for DCPP will continue to be the original design basis (i.e., DE and DDE) plus the Hosgri evaluation basis, along with the associated analytical methods, initial conditions, etc.

The objective of the proposed LTSP deterministic seismic margin evaluation is different from that of the DCPP seismic design basis evaluations associated with the DE, DDE, and HE. The objective of the DCPP seismic design basis evaluations associated with the DE, DDE, and HE is to demonstrate that the capacity of plant SSCs meet or exceed specified seismic criteria, rather than to quantify seismic margins. The objective of the proposed LTSP deterministic margin evaluation method for new ground motion information is to evaluate the plant seismic margins by comparing the HCLPF capacity of SSCs (not the code based capacities) with the seismic demands associated with the updated ground motions. The 1991 LTSP deterministic, horizontal, ground-motion spectra were compared to the 1977 Hosgri evaluation spectra that were used as the licensing basis for DCPP (1988 LTSP Final Report, Figure 7-2). The 1991 LTSP deterministic spectra were used to provide assurance that the plant HCLPF capacity estimates are at least equal to the seismic demand (SSER 34 page 3-42).

The identification of new seismic information will not impact the requirement that the design of DCPP SSCs satisfy the design criteria associated with the DE, DDE, and HE. The DE and DDE design criteria, and input ground-motion response spectra, will remain unchanged by the identification of new seismic information. As a result, there will be no impact on the DE and DDE evaluations of SSCs. The HE design criteria will remain unchanged by the identification of new seismic information. However, if the ground motion spectra associated with the new seismic information were to exceed both the 1991 LTSP spectra and the 1977 HE spectra, at any frequency, it will be necessary to revise the HE input ground-motion response spectra to envelop that associated with the new seismic information and update the HE evaluations of SSCs. In this case, a LAR will be required to revise the HE input ground-motion response spectra. The proposed process for the evaluation of new seismic information is based on the seismic margins approach, as was previously utilized under PG&E's initial implementation of the LTSP (1985 through 1991), which ensures that SSCs can perform their safety functions as specified in GDC 2 and 10 CFR 100, Appendix A, during and after a SSE.

The process for evaluation of updated seismic hazard information is as follows (proposed FSARU Figure 2.5-38):


DCPP Long Term Seismic Program Update
Evaluation of Updated Seismic Hazard Information
Flowchart A - Overview
Process for Updates to the Long Term Seismic Program (LTSP)
Seismic Probabilistic Risk and Seismic Margins Assessment
(to be performed on a 10 year interval)
Updated LTSP Seismic Hazard Input
Information Received by DCPP
or Ten Years Since Prior Update
Seismic
Probabilistic Risk Assessment
Confirmatory Path
-
Used for Safety Risk Assessment
Seismic
Margins Assessment
Primary Path
-
Used for Operability Determination
See
Flowchart
C
See
Flowchart
B
Submit Peer-Reviewed
Updated LTSP Information to
Nuclear Regulatory
Commission (NRC) /
Nuclear Reactor Regulation
(NRR)
Restart 10 Year
Interval

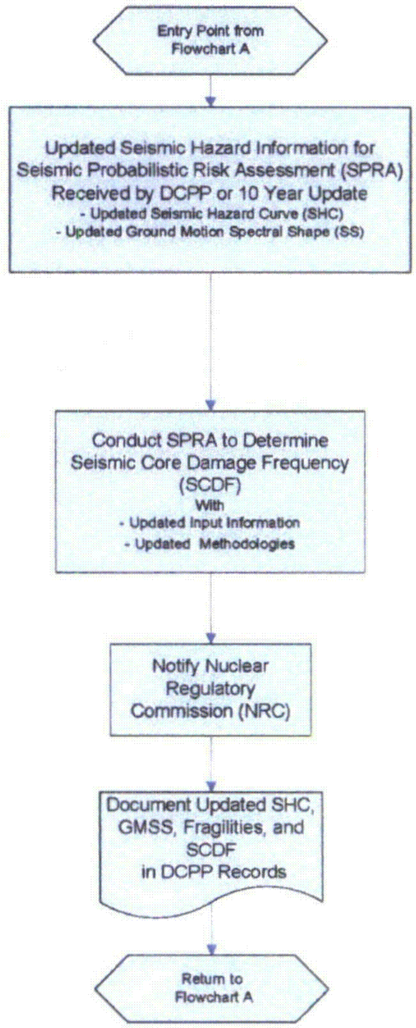


Notes:

1) Or greater than or equal to the approved seismic margin exceptions for certain SSCs discussed in FSARU Section 2.5.6.2.1.1.
2) Unless the SSC is one of the approved seismic margin exceptions below 1.0 discussed in FSARU Section 2.5.6.2.1.1.
3) Or to achieve the minimum approved seismic margin exception discussed in FSARU Section 2.5.6.2.1.1.

DCPP Long Term Seismic Program Update
Evaluation of Updated Seismic Hazard Information

Flowchart C – Seismic Probabilistic Risk Assessment



<u>Methods for recomputing the HCLPF capacities of affected SSCs</u>

The determination of the HCLPF capacities of individual SSCs is dependent on a number of factors, including the shape of the ground-motion response spectrum and the fundamental frequency of the SSC. Each SSC within the scope of the LTSP seismic margins evaluation will be screened, considering the shape of the ground-motion response spectrum associated with the new seismic information, and the SSC's fundamental frequency, to determine if recomputation of the HCLPF capacity is required. If the recomputation of the HCLPF capacity of an SSC is required, the use of the fragility analysis, conservative deterministic failure margin (CDFM), and earthquake experience data methods are acceptable.

The fragility analysis method used during the original implementation of the LTSP is based on the methods described in Chapter 6 of the 1988 LTSP Final Report. The fragility curves (see FSARU Figure 2.5-39 for sample curve) are tied to the 5 percent damped spectral acceleration value, averaged between 3 and 8.5 Hz. The computation of fragilities going forward for evaluation of updated LTSP seismic hazards input will be based on the methods described in ASME/ANS RA-Sa-2009, as modified by Regulatory Guide 1.200, Revision 2.

General guidelines of the application of the CDFM method are provided in EPRI NP-6041-SL. The CDFM method used during the original implementation of the LTSP are as described in PG&E Report titled, "Additional Deterministic Evaluations Performed to Assess Seismic Margins of the Diablo Canyon Power Plant Units 1 and 2," with the HCLPF capacities tied to the 5 percent damped spectral acceleration value, averaged between 3 and 8.5 Hz. The same methodology may be used for the computation of CDFM going forward for evaluation of updated LTSP seismic hazards. This CDFM method was reviewed and audited by the NRC and was concluded to be acceptable in SSER 34.

The earthquake experience data method was previously implemented under the LTSP for developing the HCLPF capacities of components associated with the 230 kV switchyard (e.g., transformers, breakers, switches) in response to the NRC's request for PG&E to reassess the 230 kV switchyard fragility with component performance information available from the Loma Prieta earthquake. Details of the application of the earthquake experience data method at DCPP are described in PG&E report "Long Term Seismic Program – Seismic Capacity of the 230 kV Switchyard" submitted to the NRC as part of PG&E Letter DCL-90-205, dated August 10, 1990. The reassessment resulted in the fragility of the 230 kV switchyard to be revised with the median capacity of 1.40g and an HCLPF of 0.70g. The NRC conducted an additional sensitivity study and, in general, concurred with this finding, as documented in SSER 34 (pages 23-68). The same methodology may be used for the computation of HCLPF capacities for new components, modifications to existing components, or as input to the evaluation of updated LTSP seismic hazards input.

<u>The scope of the SSCs to be evaluated for the impact of exceedances</u>

The SSCs within the scope of the LTSP, including the current seismic margins, is provided in FSARU Tables 3.7-25 and 3.7-26. The scoping was initially developed based on the methods and evaluations described in PG&E Letter DCL-86-022, "Long Term Seismic Program - Results of Phase II Scoping Study," which identified the SSCs for which the seismic fragility data were required for LTSP Phase II PRA studies by the principal PRA investigator Pickard, Lowe, and Garrick, Inc. (PL&G) on the basis of their earlier experience with similar Westinghouse plants, a study of DCPP design documents, as well as physical review of the plant systems. The Phase II PRA scoping studies identified the dominant risk contributors to the overall seismic risk.

Subsequent to the initial scoping, SSCs have been and will continue to be added to these tables to meet the criteria identified in PG&E Letter DCL-91-178, "Long Term Seismic Program - Future Plant Modifications," dated July 16, 1991, for the application of the LTSP to modifications and additions implemented at DCPP after 1991 (restated and updated in FSARU Section 3.7.6). In addition, the seismic margins for the individual SSCs will be updated, as necessary, based on modifications to DCPP or the recomputation of HCLPF capacities.

Maintaining a minimum seismic margin of 1.3

If the deterministic seismic margin evaluation determines that the minimum seismic margin remains greater than 1.3, except for SSCs identified having an acceptable seismic margin below 1.3 in FSARU Tables 3.7-25 and 3.7-26, the updated response spectrum is acceptable. Consistent with the commitment made in PG&E Letter DCL-91-178, the target HCLPF84 capacity of 2.6 g is maintained at DCPP, based on use of a seismic demand of 1.94 g (84th percentile site specific spectrum) and an upper bound seismic load factor of 1.3.

The exceptions for SSCs identified having an acceptable seismic margin below 1.3 in FSARU Tables 3.7-25 and 3.7-26 falls into two categories. The first category is SSCs that are maintained with a seismic margin between 1.3 and 1.14. The second category is SSCs that are maintained with a seismic margin below 1.0.

For the first category, the basis for maintaining the SSCs seismic margin below 1.3, but above 1.14 is consistent with the acceptable seismic margins committed to in PG&E Letter DCL-91-178 to review future plant modifications in light of the findings of the LTSP. DCL-91-178 stated the following:

Step 3: *The HCLPF84 capacities for the "screened-in" items (from Step 2) will be checked using either the Fragility Analysis method or the Conservative Deterministic Failure Margin method. If the new capacities are significantly less than those reported in the Tables 7-l and 7-2 of the Long Term Seismic Program Final Report, consideration will be given to redesign of the modifications so that capacities are consistent with those reported in the Final Report, including the guidelines given below. If redesign is not possible, proceed to Step 4.*

A modification is considered to reduce the HCLPF84 capacities significantly if any of the following occurs:

- *Turbine Building: The revised HCLPF84 capacity is reduced from that reported in the Final Report (Table 7-l).*

- *New and other existing structures: The revised HCLPF84 capacities are less than 2.6 g.*

- *New and existing equipment: The revised HCLPFU capacities are less than 2.6 g (See Commentary A).*

- *230 kV Switchyard: The revised HCLPFU capacities are reduced from those reported in the PG&E Letter DCL-90-205.*

Step 4: *The overall Plant seismic margin or the Plant seismic risk is reviewed under this step. Either of the following two alternatives can be followed to ensure continued seismic adequacy of the Plant.*

Alternative 1 - Deterministic Studies

The HCLPF84 capacity of the modified item shall be at least 1.14 times the Program seismic demand.

Alternative 2 - Probabilistic Risk Assessment Studies

The revised fragility of the modified item shall be such that the calculated risk of core damage due to the Program's seismic events is comparable to that shown in Table 6-54 of the Final Report.

SSER 34 documented the NRC's review of the seismic margins resulting from the 1988 LTSP Final Report deterministic evaluations confirming that the major plant structures and equipment at DCPP have adequate seismic margins. In SSER 34, the NRC acknowledged PG&E's commitment in DCL-91-178 to review future plant modifications in the light of the findings of the LTSP. As part of this LAR, the commitment of Step 4 from DCL-91-178 will be revised to be consistent with the proposed evaluation process for new seismic information. The evaluation process proposed in this LAR requires that the seismic margin for plant additions and plant modifications be maintained at or above 1.3, unless the minimum seismic margin below 1.3 is identified in FSARU Table 3.7-25 and 3.7-26 due to previous review and approval by the NRC, while Step 4 of DCL-91-178 permitted a minimum seismic margin of 1.14 for plant additions and plant modifications..

For the second category, SSCs with a seismic margin below 1.0 are limited to SSCs within the 4160 V (Vital) and 230 kV electrical power systems. PG&E identified relays that affect components necessary for safe shutdown using the circuit analyses as discussed in Section 23.3 of SSER 34. Functional failure fragilities of relays were evaluated only for those relays considered to be chatter sensitive. The median strength factor for chatter mode was estimated using the generic equipment ruggedness spectrum, the cabinet amplification factor, and the floor spectral acceleration. The relay chatter failure mode fragilities were derived to be 1.57 g as compared to the 84 percent site-specific ground-motion demand of 1.94 g, equating to a seismic margin of 0.81. The NRC found this seismic margin to be acceptable because the relays have reset capability from the control room, and therefore the relay chatter failure mode does not significantly impact the estimate of CDF (SSER 34 – Page 3-26).

The 230 kV switchyard has a HCLPF capacity of 0.84 g equating to a seismic margin of 0.43. A key feature in the PRA is the treatment of the fragility for the 230 kV switchyard. Given the loss of offsite power, the diesel generators are expected to have to function (with some chance of not functioning) for at least 24 hours. Recovery of seismically failed offsite power within 24 hours of the earthquake was assumed in the quantification performed in Chapter 6 of the 1988 LTSP Final Report. Based on previous California earthquake experience, the 230 kV line is expected to survive an earthquake. The NRC reviewed the treatment of the fragilities for the 230 kV switchyard in the PRA and determined that this contribution to the CDF as being acceptable in SSER 34.

Therefore the SSCs will maintain a seismic margin that is consistent with margins previously reviewed and found acceptable by the NRC.

<u>SSCs operable with seismic margin of greater than or equal to 1.0</u>

If the evaluation determines that the minimum seismic margin is below 1.3, but equal to or above 1.0, or is an SSC identified in FSARU Tables 3.7-25 and 3.7-26 as being acceptable to have a minimum seismic margin below 1.0, the applicable SSCs are determined to be operable. The seismic margin for an SSC is determined by comparing the SSCs HCLPF capacity to the 84th percent site-specific, 5 percent damped, spectral acceleration averaged from 3 to 8.5 Hz associated with the updated ground motion information. The proposed evaluation process for new seismic information would allow SSCs with a seismic margin of 1.0 or greater to be considered operable, but would require the implementation of modifications to impacted SSCs to achieve a minimum seismic margin of 1.3, or restore the SSCs seismic margin to the level that is identified in FSARU Tables 3.7-25 and 3.7-26 if its seismic margin is identified as being below 1.3. The proposed evaluation process also requires a seismic probabilistic risk assessment (SPRA) be conducted to determine the impact that the new seismic information has on the SCDF, which will be communicated to the NRC.

If the engineering evaluations determine that the seismic margin for applicable SSCs is less than 1.0, except for those SSCs identified in FSARU Tables 3.7-25 and 3.7-26 as being acceptable having a seismic margin less than 1.0, the need for an operability determination shall be addressed in accordance with the DCPP operability determination procedure and documented in the corrective action program. For SSCs already identified as having a seismic margin of less than 1.0, and seismic margin is further reduced, an operability determination is also required.

Seismic Probabilistic Risk Assessment Evaluation

Method for development of fragility curves

The fragility analysis method used during the original implementation of the LTSP is described in Chapter 6 of the 1988 LTSP Final Report. The fragility curves (see FSARU Figure 2.5-39 for sample curve) are tied to the 5 percent damped spectral acceleration value, averaged between 3 and 8.5 Hz. The computation of fragilities going forward for evaluation of updated LTSP seismic hazards input will be based on the methods described in ASME/ANS RA-Sa-2009, as modified by RG 1.200, Revision 2.

Method of conducting a Seismic Probabilistic Risk Assessment

A gap assessment of the DCPP PRA is currently underway. Outstanding gaps against Capability Category II of ASME/ANS RA-Sa-2009 will be addressed as part of any SPRA update. The next SPRA update will be completed within 2 years following issuance of (currently draft) NRC Generic Letter 2011-XX, Seismic Risk Evaluations for Operating Reactors.

Documentation of LTSP Update

The periodic updates of the LTSP evaluation being documented in a peer-reviewed report is proposed to be submitted to the NRC on a 10-year interval, and more frequently based on significant peer reviewed information justifying it. These updates will contain the information identified in the proposed TS Reporting Requirement 5.6.11, "Long Term Seismic Program Report." The reporting interval is consistent with the seismic hazards technical communities recommended maximum update interval. The proposed report content is consistent with the content of the 1988 LTSP Final Report, with the addition of documentation of the review performed by the SAB and the resolution of the SAB's comments for updates less than ten years and review by the SSHAC process and resolution of SSHAC comments for ten year updates. The first report will be submitted no later than 10 years from the date of issuance of the license amendment associated with this LAR.

Clarification to Ongoing Commitments Associated with LTSP

On March 15, 1991, the NRC Staff met with PG&E to discuss the LTSP. This meeting included a presentation by PG&E Staff describing the continuing LTSP activities for DCPP (Reference 4, Enclosure 5):

- *Maintain a high level of technical expertise in geology, seismology, and earthquake engineering to effectively address future seismic issues*
- *Operate the strong-motion array at and near the Diablo Canyon site*

- *Operate the Central Coast Seismic Network in the region of Diablo Canyon*
- *Maintain knowledge of earthquakes occurring elsewhere sufficient to rapidly evaluate their significance to Diablo Canyon*

As a follow-up to the public meeting, PG&E provided a summary of the benefits and insights of the LTSP in a letter to the NRC dated April 17, 1991 (Reference 3). This letter included a summary of PG&E plans for ongoing activities in support of the LTSP, described as the "Framework for the Future" (Enclosure to Reference 3):

PG&E recognizes the value of the Long Term Seismic Program in the future operation of the Diablo Canyon Power Plant, and we plan to support key technical activities and associated personnel into the future.

Data Bases and Instrumentation

One of the more significant benefits of the Long Term Seismic Program has been the creation of a comprehensive data base. We will continue to use and develop this data set in assessments related to the Diablo Canyon site, as well as the sites of other PG&E facilities in the South-Central California region. We plan to monitor and evaluate technological advances and new data as they become available. We will use the probabilistic risk assessment developed during the Program as a tool to provide insight into the continued safe operation of the Plant. The Central Coast Seismic Network will continue to monitor micro earthquake activity in the region, and will assist us in accurately locating and characterizing relevant earthquakes. The strong-motion array will continue to operate to help us assess site response to ground motions.

Focus for Addressing Seismic Issues

The Long Term Seismic Program will allow PG&E to anticipate and respond in a timely manner to new issues and concerns as they arise. For example, through the expertise available in the Long Term Seismic Program, we were able to test and verify the results of the Program's ground motion evaluation by using the new data from the October 17, 1989, Loma Prieta earthquake, a well-recorded event. This ability provided increased confidence that new earthquakes are not likely to produce surprising or conflicting data. The Long Term Seismic Program will continue to provide a focus for addressing seismic issues related to Diablo Canyon.

The NRC Staff summarized its review and conclusions about the LTSP in SSER 34 (Reference 2).

- The NRC Staff's conclusions regarding the relationship between the LTSP and the seismic qualification basis for DCPP are given in SSER 34, Section 1.4, Summary of Staff Conclusions (Reference 2):

  The staff notes that the seismic qualification basis for Diablo Canyon will continue to be the original design basis plus the Hosgri evaluation basis, along with the associated analytical methods, initial conditions, etc. The LTSP has served as a useful check on the adequacy of the seismic margins and has generally confirmed that the margins are acceptable. For future plant design modifications, the staff concludes that LTSP spectra, increased to envelope the exceedances in the vertical and horizontal spectra discussed in Section 2.5.2.3 of this SSER, should be used to verify that the plant high confidence of low probability of failure (HCLPF) values remain acceptable (Section 3.3 of this SSER). PG&E has agreed (Shiffer, 1991l) to review future plant modifications in the light of the findings of the LTSP, and is currently developing an implementation procedure for that purpose.

- The NRC Staff recognized that the LTSP response spectrum exceeded the design basis 7.5M Hosgri response spectrum at high frequencies, but indicated that revisions to the design basis earthquakes were not required. This is discussed in SSER 34, Section 3.8.1.1 (Reference 2):

  The ground-motion input data used in the deterministic evaluation were the 84th percentile, 5 percent damped, horizontal and vertical, site-specific LTSP acceleration response spectra resulting from the MME. The LTSP site-specific, horizontal, ground-motion response spectra (SSRS) for 5 percent damping due to the MME and the 1977 Hosgri evaluation spectrum are compared in Figure 7-2 of the LTSP Final Report (reproduced here as Figure 3.1). It may be seen from this figure that the Hosgri evaluation spectrum is greater than the LTSP 50th percentile (median) spectrum at all frequencies, and is greater than the 84th percentile spectrum (called the "LTSP spectrum" in this section of the SSER) at all frequencies less than about 15 Hz. The magnitude of the exceedance at frequencies above 15 Hz is approximately 10 percent. On the basis of PG&E's margins evaluation discussed in Section 3.8.1.7 of this SSER, the staff concludes that these high-frequency spectral exceedances are not significant.

- PG&E's commitments for the ongoing activities in support of the LTSP are restated in SSER 34, Section 2.5.2.4, Seismology Conclusions (Reference 2):

  PG&E made the following commitments at the public meeting on March 15, 1991, and in a letter from PG&E to the NRC (Shiffer, 1991f): (1) to continue to maintain a strong geosciences and engineering staff to keep abreast of new geological, seismic, and seismic engineering *information and evaluate it with respect to its significance to Diablo Canyon, and (2) to continue to operate a strong-motion accelerometer array and the coastal seismic network, although likely with fewer stations than are currently operating. Since some issues (i.e., slip type of the Hosgri, the characterization of the Southwest Boundary Zone, and ground-motion estimates for oblique-slip earthquakes) are controversial because of the lack of definitive evidence, future geoscience discoveries may allow a more robust conclusion for these issues.*

PG&E's commitment to review certain future plant modifications in light of the findings of the LTSP is provided in FSARU Sections 2.5 and 3.7. The proposed change captures the commitments for ongoing activities in support of the LTSP in the FSARU, and provides details of the process employed by PG&E in implementing these commitments. By including this information in the FSARU, any change to the program would be subject to the provisions of 10 CFR 50.59; and evaluated to determine if the change requires prior NRC approval.

5. REGULATORY EVALUATION

5.1 Applicable Regulatory Requirements/Criteria

In Regulatory Issue Summary 2000-17, "Managing Regulatory Commitments Made by Power Reactor Licensees to the NRC Staff," dated September 21, 2000, the NRC informed licensees that the Nuclear Energy Institute document NEI 99-04, "Guidelines for Managing NRC Commitment Changes," contains acceptable guidance for controlling regulatory commitments and encouraged licensees to use the NEI guidance or similar administrative controls to ensure that regulatory commitments are implemented and that changes to the regulatory commitments are evaluated and, when appropriate, reported to the NRC. NEI 99-04 defines a "regulatory commitment" as an explicit statement to take a specific action agreed to, or volunteered by, a licensee and submitted in writing on the docket to the NRC. This proposed change ensures that the commitments made to the NRC at the public meeting on March 15, 1991, and in a letter from PG&E to the NRC, dated April 17, 1991, regarding ongoing activities in support of the LTSP are effectively controlled.

10 CFR 50.59, "Changes tests and experiments," establishes the conditions under which licensees may make changes to the facility or procedures and conduct tests or experiments without prior NRC approval. Proposed changes, tests, and experiments that satisfy the definitions and one or more of the criteria in the rule must be reviewed and approved by the NRC before implementation. The proposed change revises the licensing basis, as described in the FSARU and TS, to include discussions of the actions and requirements associated with PG&E's ongoing activities

in support of the LTSP (response to Condition No. 2.C.(7) of Facility Operating License DPR-80). A codified change process for evaluating any future changes to the LTSP is provided in 10 CFR 50.59.

Appendix A to 10 CFR 100, "Seismic and Geologic Siting Criteria for Nuclear Power Plants," establishes that the nuclear power plant shall be designed so that, if the SSE occurs, certain SSCs will remain functional. These SSCs are those necessary to assure: (1) the integrity of the reactor coolant pressure boundary, (2) the capability to shut down the reactor and maintain it in a safe condition, or (3) the capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of this part. In addition to seismic loads, including aftershocks, applicable concurrent functional and accident-induced loads shall be taken into account in the design of these safety-related SSCs. The design of the nuclear power plant shall also take into account the possible effects of the safe shutdown earthquake on the facility foundations by ground disruption, such as fissuring, differential consolidation, cratering, liquefaction, and landsliding. FSARU Sections 2.5, 3.2.1, and 3.7.1.1 discuss that DCPP's design had been established prior to the issuance of 10 CFR 100, Appendix A, and provide a comparison to 10 CFR 100, Appendix A requirements.

Paragraph 50.36(c)(5) of 10 CFR, "Administrative controls," establishes that "[a]dministrative controls are the provisions relating to organization and management, procedures, recordkeeping, review and audit, and reporting necessary to assure operation of the facility in a safe manner. Each licensee shall submit any reports to the Commission pursuant to approved technical specifications as specified in 10 CFR 50.4."

GDC 2, "Design Bases for Protection Against Natural Phenomena," establishes those systems and components of reactor facilities that are essential to the prevention of accidents which could affect the public health and safety, or to mitigation of their consequences, shall be designed, fabricated, and erected to performance standards that will enable the facility to withstand, without loss of the capability to protect the public, the additional forces that might be imposed by natural phenomena such as earthquakes, tornadoes, flooding conditions, winds, ice, and other local site effects. The design bases so established shall reflect: (1) appropriate consideration of the most severe of these natural phenomena that have been recorded for the site and the surrounding area, and (2) an appropriate margin for withstanding forces greater than those recorded to reflect uncertainties about the historical data and their suitability as a basis for design. FSARU Section 3.1.2.2 and Appendix 3.1A discuss compliance with GDC 2.

SG 29, "Seismic Design Criteria" establishes "a method acceptable to the NRC staff for identifying and classifying those features of light-water-cooled nuclear power plants that should be designed to withstand the effects of the SSE." FSARU Section 3.7 discusses compliance with SG 29.

With the proposed revisions to the DCPP TS and FSARU, DCPP continues to meet the requirements of 10 CFR 50.59, 10 CFR 50.36, GDC 2, and SG 29.

With the use of the previously approved HE methods of analysis, which are different than those used for the original design analysis for the DE and DDE, as identified in SSER 7 Sections 3.8.5.3 and 3.9.3.2, it has been demonstrated that the SSCs necessary to assure: (1) the integrity of the reactor coolant pressure boundary, (2) the capability to shutdown the reactor and maintain it in a safe shutdown condition, or (3) the capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of 10 CFR 100, were qualified for the HE.

5.2 Precedent

The LTSP, developed in response to License Condition No. 2.C.(7) of DCPP Facility OL DPR-80, provides an acceptable regulatory mechanism for reevaluating the DCPP seismic design bases taking into account new seismic information, and for assessing the significance of the conclusions of the seismic reevaluation to ensure adequacy of seismic margins.

During the time period between the issuance of SSER 34 (1991) and the present, PG&E has continuously gathered seismic data, both locally (through the seismic instrumentation in the vicinity of DCPP) and globally (through review of earthquake records and field reconnaissance associated with major earthquakes worldwide), participated in technical research (through participation in the Pacific Earthquake Engineering Research Center, through a cooperative research agreement with the USGS, and through collaboration with other industry and governmental agencies), and evaluated the significance of any new information relative to the seismic evaluation of DCPP.

5.3 Significant Hazards Consideration

The proposed change would revise the licensing basis as documented in the Final Safety Analysis Report Update (FSARU) and technical specifications (TS) to include discussions of the actions and requirements associated with PG&E's ongoing activities in support of the Long Term Seismic Program (LTSP) (response to Condition No. 2.C.(7) of Facility Operating License DPR-80) and clarify the Hosgri Earthquake spectrum (HE) as Diablo Canyon Power Plants (DCPP's) equivalent safe shutdown earthquake (SSE). The LTSP provides an acceptable regulatory mechanism for reevaluating the DCPP seismic design bases taking into account new seismic information, and for assessing the significance of the conclusions of the seismic reevaluation to assure adequacy of seismic margins.

Consistent with NUREG-0484, "Combining Dynamic Loads," Revision 1, PG&E proposes to use the square-root-of-the-sum-of-squares (SRSS) method for evaluations of seismic and loop pipe rupture load combinations.

The NRC's review and acceptance of PG&E's response to License Condition 2.C.(7), are discussed in NUREG-0675, "Safety Evaluation Report Related to the Operation of Diablo Canyon Nuclear Power Plant, Units 1 and 2," Supplement No. 34, dated June 1991 (SSER 34), and in NRC letter dated April 14, 1992, "Transmittal of Safety Evaluation Closing Out Diablo Canyon Long-Term Seismic Program (TAC NOS. M80670 and M80671)." SSER-34, Section 2.5.2.4, "Seismology Conclusions," included a restatement of two PG&E commitments with respect to ongoing activities associated with the implementation of the LTSP. These commitments are based on PG&E Letter DCL-91-091, J.D. Shiffer (PG&E) to NRC, "Benefits and Insights of the Long Term Seismic Program," dated April 17, 1991. PG&E proposes to incorporate the implementation of these commitments with the evaluation of new seismic information into the FSARU, Section 2.5, "Geology and Seismology," and Section 3.7, "Seismic Design." PG&E also proposes to incorporate the ongoing review and evaluation of new seismic information and methodologies, and reporting requirements, associated with the LTSP into new administrative control TS 5.5.20, "Long Term Seismic Program," and 5.6.12, "Long Term Seismic Program Report."

PG&E has evaluated whether or not a significant hazards consideration is involved with the proposed amendment by focusing on the three standards set forth in 10 CFR 50.92, "Issuance of amendment," as discussed below:

1. *Does the change involve a significant increase in the probability or consequences of an accident previously evaluated?*

The changes proposed by this LAR clarify the licensing basis as documented in the FSARU and TS to incorporate the HE as DCPP's equivalent SSE consistent with the NRC conclusions in Supplemental Safety Evaluation Report (SSER) 7, describe an acceptable methodology for evaluating the effect of new seismic information on DCPP's ability to achieve safe shutdown, and for assessing the significance of the conclusions of the seismic reevaluation to assure adequacy of seismic margins. The proposed changes include specifying the use of the SRSS method for evaluations of seismic and loop pipe rupture load combinations.

The changes proposed by this LAR do not change design requirements and does not involve any physical change to any structures, systems, and components (SSC), nor does it affect the ability of any SSC to function in response to design-basis seismic events or other previously evaluated accidents, including the previous definitions and assumptions regarding design earthquake (DE), double design earthquake (DDE), and HE. It is unrelated to the probability of occurrence or the consequences of those events or accidents.

Therefore, the proposed change does not involve a significant increase in the probability or consequences of an accident previously evaluated.

2. *Does the change create the possibility of a new or different kind of accident from any accident previously evaluated?*

The changes proposed by this LAR do not change design requirements and do not involve changes to any plant SSCs, nor do they involve changes to any plant operating practice or procedure. No credible new failure mechanisms, malfunctions, or accident initiators not considered in the design and licensing bases are created that would create the possibility of a new or different kind of accident. The proposed changes would provide an agreed to process for evaluating new seismic information.

Therefore the proposed changes do not create the possibility of a new or different kind of accident from any accident previously evaluated.

3. *Does the change involve a significant reduction in a margin of safety?*

The changes proposed by this LAR change do not change design requirements and do not involve any physical changes to the plant or alter the manner in which plant systems are operated, maintained, modified, tested, or inspected. The proposed changes do not alter the manner in which safety limits, limiting safety system settings, or limiting conditions for operation are determined. The safety analysis acceptance criteria are not affected by this change. The proposed changes will not result in plant operation in a configuration outside the design basis. The proposed LAR change does not adversely affect systems that respond to safely shutdown the plant and to maintain the plant in a safe shutdown condition. The proposed evaluation methodology, based on that used during the original implementation of the LTSP, will ensure that the seismic margins, relative to the 1977 HE response spectra, accepted by the NRC in 1991, are maintained.

The use of the SRSS method for evaluating the combination of seismic and LOCA load combinations is commonly used in seismic analysis and is appropriate when combining statistically independent transient functions. This method is consistent with NUREG-0484, Revision 1.

Therefore, the changes proposed by this LAR do not involve a significant reduction in a margin of safety.

Based on the above evaluation, PG&E concludes that the changes proposed by this LAR satisfies the no significant hazards consideration standards of 10 CFR 50.92(c), and accordingly a no significant hazards finding is justified.

5.4 Conclusions

In conclusion, based on the considerations discussed above: (1) There is reasonable assurance that the health and safety of the public will not be endangered by operation in the proposed manner, (2) such activities will be conducted in compliance with the Commission's regulations, and (3) the issuance of the amendment will not be inimical to the common defense and security or to the health and safety of the public.

6. ENVIRONMENTAL CONSIDERATION

PG&E has evaluated the proposed amendment and has determined that the proposed amendment does not involve: (i) a significant hazards consideration, (ii) a significant change in the types or significant increase in the amounts of any effluents that may be released offsite, or (iii) a significant increase in individual or cumulative occupational radiation exposure. Accordingly, the proposed amendment meets the eligibility criterion for categorical exclusion set forth in 10 CFR 51.22(c)(9). Therefore, pursuant to 10 CFR 51.22(b), no environmental impact statement or environmental assessment need be prepared in connection with the proposed amendment.

7. REFERENCES

1. Diablo Canyon Power Plant, Units 1 and 2, Final Safety Analysis Report Update, Revision 19

2. NRC, NUREG-0675, "Safety Evaluation Report Related to the Operation of Diablo Canyon Nuclear Power Plant, Units 1 and 2," Supplement No. 34, dated June 1991

3. PG&E Letter DCL-91-091, J.D. Shiffer (PG&E) to USNRC, "Benefits and Insights of the Long Term Seismic Program," dated April 17, 1991

4. NRC, "Summary of March 15, 1991, Public Meeting to Discuss Diablo Canyon Long Term Seismic Program (TAC Nos. 55305 and 68049)," Docket Nos. 50-275 and 50-323, dated March 22, 1991

5. NRC, Safety Evaluation Report Related to the Operation of Diablo Canyon Nuclear Power Plant, Units 1 and 2," Supplement No. 7, dated May 26, 1978

6. PG&E Letter DCL-90-226, "Long Term Seismic Program Additional Deterministic Evaluations," dated September 18, 1990

7. PG&E Letter DCL-91-143, J.D. Shiffer (PG&E) to USNRC, "Long Term Seismic Program - Implementation of the Results of the Program," dated May 29, 1991

8. PG&E Letter DCL-91-178, J.D. Shiffer (PG&E) to USNRC, "Long Term Seismic Program - Future Plant Modifications," dated July 16, 1991

9. Atomic Safety and Licensing Board, Partial Initial Decision LBP-79-26, "Pacific Gas and Electric Company – Diablo Canyon Nuclear Power Plant (Units 1 and 2), dated September 27, 1979

10. NCR Safety Guide 29, "Seismic Design Classification," dated July 7, 1972

11. PG&E Letter DCL-88-192, "Long Term Seismic Program Completion," dated July 31, 1988

12. PG&E Letter DCL-91-027, "Addendum to Long Term Seismic Program Final Report," dated February 13, 1991

13. NRC, "Transmittal of Safety Evaluation Closing Out Diablo Canyon Long-Term Seismic Program (TAC Nos. M80670 and M80671)," dated April 17, 1992

14. PG&E Letter to NRC, "Amendment No. 50 to the Operating License Application for Diablo Canyon Power Plant Units 1 and 2; 'Seismic Evaluation for Postulated 7.5M Hosgri Earthquake,'" dated June 5, 1977

15. NRC Letter to PG&E, "Diablo Canyon Power Plant – NRC Integrated Inspection 05000275/2011002 AND 05000323/2011002," dated May 11, 2011, Unresolved Item: 05000275; 323/2011002-03

Technical Specification Page Markups

Insert 5.5.20

5.5.20 Long Term Seismic Program

This program provides ongoing review and evaluation of new seismic information and associated methodologies. The program shall include the following:

a. A staff to keep abreast of new geological, seismic, and seismic engineering information and evaluate it with respect to its significance to DCPP;

b. Operation of a strong-motion accelerometer array and the coastal seismic network;

c. Verification that plant seismic margins remain acceptable for plant additions and modifications when checked against insights and knowledge gained from the Long Term Seismic Program, as identified in FSARU Section 3.7.6;

d. Deterministic seismic margin acceptance criteria for operability determinations;

e. Peer review process requirements for seismic probabilistic risk assessment revisions;

f. Peer review processes requirements for seismic model or methodology revisions; and

g. Minimum requirements for the Seismic Advisory Board (SAB).

Insert 5.6.11

5.6.11 Long Term Seismic Program Report

A report shall be submitted once every 10 years, based on the submittal date of the previous update. An updated report will be submitted in less than 10 years if new peer reviewed seismic information becomes available that would significantly increase the risk to DCPP. The report shall include the following information:

a. Geology/seismology/geophysics/tectonics investigations,

b. Seismic source characterization,

c. Characterization of ground motions,

d. Soil/structure interaction analysis,

e. Probabilistic risk analysis,

f. Deterministic evaluations,

g. Assessment of the adequacy of seismic margins,

h. Documentation of the review performed by the Seismic Advisory Board (SAB) and resolution of the SAB's comments if performed in less than 10 years, and

i. Documentation of the review performed by the Senior Seismix Hazards Analysis Committee for 10 year updates.

5.5 Programs and Manuals (continued)

5.5.19 Control Room Envelope Habitability Program

A Control Room Envelope (CRE) Habitability Program shall be established and implemented to ensure that CRE habitability is maintained such that, with an OPERABLE Control Room Ventilation System (CRVS), CRE occupants can control the reactor safely under normal conditions and maintain it in a safe condition following a radiological event, hazardous chemical release, or a smoke challenge. The program shall ensure that adequate radiation protection is provided to permit access and occupancy of the CRE under design basis accident (DBA) conditions without personnel receiving radiation exposures in excess of 5 rem whole body or its equivalent to any part of the body for the duration of the accident. The program shall include the following elements:

a. The definition of the CRE and the CRE boundary.

b. Requirements for maintaining the CRE boundary in its design condition, including configuration control and preventive maintenance.

c. Requirements for (i) determining the unfiltered air inleakage past the CRE boundary into the CRE in accordance with the testing methods and at the Frequencies specified in Sections C.1 and C.2 of Regulatory Guide 1.197, "Demonstrating Control Room Envelope Integrity at Nuclear Power Reactors," Revision 0, May 2003, and (ii) assessing CRE habitability at the Frequencies specified in Sections C.1 and C.2 of Regulatory Guide 1.197, Revision 0.

d. Measurement, at designated locations, of the CRE pressure relative to all external areas adjacent to the CRE boundary during the pressurization mode of operation by one train of the CRVS, operating at the flow rate required by the VFTP, at a Frequency of 24 months on a STAGGERED TEST BASIS. The results shall be trended and used as part of the 24 month assessment of the CRE boundary.

e. The quantitative limits on unfiltered air inleakage into the CRE. These limits shall be stated in a manner to allow direct comparison to the unfiltered air inleakage measured by the testing described in paragraph c. The unfiltered air inleakage limit for radiological challenges is the inleakage flow rate assumed in the licensing basis analyses of DBA consequences. Unfiltered air inleakage limits for hazardous chemicals must ensure that exposure of CRE occupants to these hazards will be within the assumptions in the licensing basis.

f. The provisions of SR 3.0.2 are applicable to the Frequencies required by paragraphs c and d for determining CRE unfiltered inleakage and assessing CRE habitability, and measuring CRE pressure and assessing the CRE boundary.

(continued)

Insert 5.5.20 on new page 5.0-17b

5.6 Reporting Requirements (continued)

5.6.10 Steam Generator (SG) Tube Inspection Report

A report shall be submitted within 180 days after the initial entry into MODE 4 following completion of an inspection performed in accordance with the Specification 5.5.9, Steam Generator (SG) Program. The report shall include:

a. The scope of inspections performed on each SG,

b. Active degradation mechanisms found,

c. Nondestructive examination techniques utilized for each degradation mechanism,

d. Location, orientation (if linear), and measured sizes (if available) of service induced indications,

e. Number of tubes plugged during the inspection outage for each active degradation mechanism,

f. Total number and percentage of tubes plugged to date, and

g. The results of condition monitoring, including the results of tube pulls and in-situ testing.

Insert 5.6.11

Retyped Technical Specification Pages

Remove Page	Insert Page
5.0-17a	5.0-17a
	5.0-17b
5.0-23	5.0-23
	5.0-23a

5.5 Programs and Manuals (continued)

5.5.19 Control Room Envelope Habitability Program

A Control Room Envelope (CRE) Habitability Program shall be established and implemented to ensure that CRE habitability is maintained such that, with an OPERABLE Control Room Ventilation System (CRVS), CRE occupants can control the reactor safely under normal conditions and maintain it in a safe condition following a radiological event, hazardous chemical release, or a smoke challenge. The program shall ensure that adequate radiation protection is provided to permit access and occupancy of the CRE under design basis accident (DBA) conditions without personnel receiving radiation exposures in excess of 5 rem whole body or its equivalent to any part of the body for the duration of the accident. The program shall include the following elements:

a. The definition of the CRE and the CRE boundary.

b. Requirements for maintaining the CRE boundary in its design condition, including configuration control and preventive maintenance.

c. Requirements for (i) determining the unfiltered air inleakage past the CRE boundary into the CRE in accordance with the testing methods and at the Frequencies specified in Sections C.1 and C.2 of Regulatory Guide 1.197, "Demonstrating Control Room Envelope Integrity at Nuclear Power Reactors," Revision 0, May 2003, and (ii) assessing CRE habitability at the Frequencies specified in Sections C.1 and C.2 of Regulatory Guide 1.197, Revision 0.

d. Measurement, at designated locations, of the CRE pressure relative to all external areas adjacent to the CRE boundary during the pressurization mode of operation by one train of the CRVS, operating at the flow rate required by the VFTP, at a Frequency of 24 months on a STAGGERED TEST BASIS. The results shall be trended and used as part of the 24 month assessment of the CRE boundary.

e. The quantitative limits on unfiltered air inleakage into the CRE. These limits shall be stated in a manner to allow direct comparison to the unfiltered air inleakage measured by the testing described in paragraph c. The unfiltered air inleakage limit for radiological challenges is the inleakage flow rate assumed in the licensing basis analyses of DBA consequences. Unfiltered air inleakage limits for hazardous chemicals must ensure that exposure of CRE occupants to these hazards will be within the assumptions in the licensing basis.

f. The provisions of SR 3.0.2 are applicable to the Frequencies required by paragraphs c and d for determining CRE unfiltered inleakage and assessing CRE habitability, and measuring CRE pressure and assessing the CRE boundary.

(continued)

5.5 Programs and Manuals (continued)

5.5.20 Long Term Seismic Program

This program provides ongoing review and evaluation of new seismic information and associated methodologies. The program shall include the following:

a. A staff to keep abreast of new geological, seismic, and seismic engineering information and evaluate it with respect to its significance to DCPP;

b. Operation of a strong-motion accelerometer array and the coastal seismic network;

c. Verification that plant seismic margins remain acceptable for plant additions and modifications when checked against insights and knowledge gained from the Long Term Seismic Program, as identified in FSARU Section 3.7.6;

d. Deterministic seismic margin acceptance criteria for operability determinations;

e. Peer review process requirements for seismic probabilistic risk assessment revisions;

f. Peer review processes requirements for seismic model or methodology revisions; and

g. Minimum requirements for the Seismic Advisory Board (SAB).

5.6 Reporting Requirements (continued)

5.6.10 Steam Generator (SG) Tube Inspection Report

A report shall be submitted within 180 days after the initial entry into MODE 4 following completion of an inspection performed in accordance with the Specification 5.5.9, Steam Generator (SG) Program. The report shall include:

a. The scope of inspections performed on each SG,

b. Active degradation mechanisms found,

c. Nondestructive examination techniques utilized for each degradation mechanism,

d. Location, orientation (if linear), and measured sizes (if available) of service induced indications,

e. Number of tubes plugged during the inspection outage for each active degradation mechanism,

f. Total number and percentage of tubes plugged to date, and

g. The results of condition monitoring, including the results of tube pulls and in-situ testing.

5.6.11 Long Term Seismic Program Report

A report shall be submitted once every 10 years, based on the submittal date of the previous update. An updated report will be submitted in less than 10 years if new peer reviewed seismic information becomes available that would significantly increase the risk to DCPP. The report shall include the following information:

a. Geology/seismology/geophysics/tectonics investigations,

b. Seismic source characterization,

c. Characterization of ground motions,

d. Soil/structure interaction analysis,

e. Probabilistic risk analysis,

f. Deterministic evaluations,

g. Assessment of the adequacy of seismic margins,

h. Documentation of the review performed by the Seismic Advisory Board (SAB) and resolution of the SAB's comments if performed in less than 10 years, and

(continued)

5.6 Reporting Requirements

5.6.11 Long Term Seismic Program Report (continued)

i. Documentation of the review performed by the Senior Seismic Hazards Analysis Committee for 10 year updates.

Final Safety Analysis Report Update Changes

1.2.1.6 Seismology

Seismological investigations were undertaken to determine the potential for earthquakes in the site area, to form a basis of the establishment of seismic design criteria, and to evaluate the adequacy of seismic design margins for the plant (Section 2.5). Records indicate that seismic activity within 20 miles of Diablo Canyon has been very low compared to other parts of California. Until PG&E's seismological investigation of the Hosgri fault zone located approximately 3 miles offshore, the seismically significant fault system nearpest the site was considered to be the Nacimiento Fault located about 20 miles away as discussed in Section 2.5.2.9. The largest earthquake known to have been associated with this fault system occurred at an epicentral distance to the site of about 44 miles. It is listed with a Richter magnitude 6. A Richter magnitude 7.5 earthquake was postulated for the Hosgri fault, as discussed in Section 2.5.2.9.3. At its closest point, the San Andreas Fault passes some 48 miles from the site.

~~PG&E's reevaluation of the plant's capability to withstand a postulated Richter magnitude 7.5 "Hosgri" earthquake is discussed in Section 3.7.~~

2.5 GEOLOGY AND SEISMOLOGY

This section presents the findings of the regional and site-specific geologic and seismologic investigations of the Diablo Canyon Power Plant (DCPP) site. ~~Information presented is in compliance with the criteria in Appendix A of 10 CFR 100[52] and meets the format and content recommendations of Regulatory Guide 1.70, Revision 1[39].~~

In order to capture the historical progress of the geological and seismological investigations associated with the DCPP site, information pertaining to the following three time periods are described herein:

(1) Pre-Construction/Early-Construction Phase: investigations performed in support of the Preliminary Safety Analysis Report (circa 1967), prior to the issuance of the Unit 1 construction permit, through the early stages of the construction of Unit (circa 1971). See Sections 2.5.1 through 2.5.2.8, 2.5.2.9.1, 2.5.2.9.2, 2.5.2.10.1, and 2.5.2.10.2.

(2) Hosgri Evaluation Phase: investigations performed in response to the identification of the offshore Hosgri fault zone (circa 1971) through the issuance of the Unit 1 operating license (circa 1984). See Sections 2.5.1 through 2.5.2.8, 2.5.2.9.3 and 2.5.2.10.3

(3) Long Term Seismic Program (LTSP) Phase: investigations performed in response to the License Condition Item No. 2.C.(7) of the Unit 1 operating license (circa 1985) through the removal of the License Condition (circa 1991), including current on-going investigations. See Sections 2.5.2.9.4, 2.5.2.10.4, and 2.5.6.

Overview

Location of earthquake epicenters within 200 miles of the plant site, and faults and earthquake epicenters within 75 miles of the plant site for either magnitudes or intensities, respectively, are shown in Figures 2.5-2, 2.5-3, and 2.5-4 (through 1971). A geologic and tectonic map of the region surrounding the site is ~~given in two sheets of~~shown in Figure 2.5-5, and detailed information about site geology is presented in Figures 2.5-8 through 2.5-16. Geology and seismology are discussed in detail in Sections 2.5.1 through 2.5.4. Additional information on site geology is contained in References 1 and 2.

~~On November 2, 1984, the NRC issued the Diablo Canyon Unit 1 Facility Operating License DPR-80. In DPR-80, License Condition Item 2.C.(7), the NRC stated, in part:~~

> ~~"PG&E shall develop and implement a program to reevaluate the seismic design bases used for the Diablo Canyon Power Plant."~~

~~PG&E's reevaluation effort in response to the license condition was titled the "Long Term Seismic Program" (LTSP). PG&E prepared and submitted to the NRC the "Final Report of the Diablo Canyon Long Term Seismic Program" in July 1988[40]. Between 1988 and 1991, the NRC performed an extensive review of the Final Report, and PG&E prepared and submitted written responses to formal NRC questions. In February 1991, PG&E issued the "Addendum to the 1988 Final Report of the Diablo Canyon Long Term Seismic Program"[41]. In June 1991, the NRC issued Supplement Number 34 to the Diablo Canyon Safety Evaluation Report (SSER)[42], in which the NRC concluded that PG&E had satisfied License Condition 2.C.(7) of Facility Operating License DPR-80. In the SSER the NRC requested certain confirmatory analyses from PG&E, and PG&E subsequently submitted the requested analyses. The NRC's final acceptance of the LTSP is documented in a letter to PG&E dated April 17, 1992[43].~~

~~The LTSP contains extensive data bases and analyses that update the basic geologic and seismic information in this section of the FSAR Update. However, the LTSP material does not address or alter the current design licensing basis for the plant, and thus is not included in the FSAR Update.~~

~~A complete listing of bibliographic references to the LTSP reports and other documents may be found in References 40, 41 and 42.~~

Detailed supporting data pertaining to this section are presented in Appendices 2.5A, 2.5B, 2.5C, and 2.5D of Reference 27 in Section 2.3. Geologic and seismic information from investigations that responded to Nuclear Regulatory Commission (NRC) licensing review questions are presented Appendices 2.5E and 2.5F of the same reference (Hosgri evaluation phase). A brief synopsis of the information presented in Reference 27 (Section 2.3) is given below.

The DCPP site is located in San Luis Obispo County approximately 190 miles south of San Francisco and 150 miles northwest of Los Angeles, California. It is adjacent to the Pacific Ocean, 12 miles west-southwest of the city of San Luis Obispo, the county seat. The plant site location and topography are shown in Figure 2.5-1.

The site is located near the mouth of Diablo Creek which flows out of the San Luis Range, the dominant feature to the northeast. The Pacific Ocean is southwest of the site. Facilities for the power plant are located on a marine terrace that is situated between the mountain range and the ocean.

The terrace is bedrock overlain by surficial deposits of marine and nonmarine origin. Seismic Category I structures at the site are situated on bedrock that is predominantly stratified marine sedimentary rocks and volcanics, all of Miocene age. A more extensive discussion of the regional geology is presented in Section 2.5.1.1 and site geology in Section 2.5.1.2.

Several investigations were performed at the site and in the vicinity of the site during the pre-construction/early-construction investigation phase to determine: potential vibratory ground motion characteristics, existence of surface faulting, and stability of subsurface materials and cut slopes adjacent to Seismic Category I structures. Details of these investigations are presented in Sections 2.5.2 through 2.5.5. Consultants retained to perform these studies included: Earth Science Associates (geology and seismicity), John A. Blume and Associates (seismic design and foundation materials dynamic response), Harding-Lawson and Associates (stability of cut slope), Woodward-Clyde-Sherard and Associates (soil testing), and Geo-Recon, Incorporated (rock seismic velocity determinations). The findings of these consultants are summarized in this section and the detailed reports are included in Appendices 2.5A, 2.5B, 2.5C, 2.5D, 2.5E, and 2.5F of Reference 27 in Section 2.3.

Geologic investigation during the pre-construction/early-construction phase of the Diablo Canyon coastal area, including detailed mapping of all natural exposures and exploratory trenches, yielded the following basic conclusions:

(1) The area is underlain by sedimentary and volcanic bedrock units of Miocene age. Within this area, the power plant site is underlain almost wholly by sedimentary strata of the Monterey Formation, which dip northward at moderate to very steep angles. More specifically, the reactor site is underlain by thick-bedded to almost massive Monterey sandstone that is well indurated and firm. Where exposed on the nearby hillslope, this rock is markedly resistant to erosion.

(2) The bedrock beneath the main terrace area, within which the power plant site has been located, is covered by 3 to 35 feet of surficial deposits. These include marine sediments of Pleistocene age and nonmarine sediments of Pleistocene and Holocene age. In general, they are thickest in the vicinity of the reactor site.

(3) The interface between the unconsolidated terrace deposits and the underlying bedrock comprises flat to moderately irregular surfaces of Pleistocene marine planation and intervening steeper slopes that also represent erosion in Pleistocene time.

(4) The bedrock beneath the power plant site occupies the southerly flank of a major syncline that trends west to northwest. No evidence of a major fault has been recognized within or near the coastal area, and bedrock relationships in the exploratory trenches positively indicate that no such fault is present within the area of the power plant site.

(5) Minor surfaces of disturbance, some of which plainly are faults, are present within the bedrock that underlies the power plant site. None of these breaks offsets the interface between bedrock and the cover of terrace deposits, and none of them extends upward into the surficial cover. Thus, the latest movements along these small faults must have antedated erosion of the bedrock section in Pleistocene time.

(6) No landslide masses or other gross expressions of ground instability are present within the power plant site or on the main hillslope east of the site. Some landslides have been identified in adjacent ground, but these are minor features confined to the naturally oversteepened walls of Diablo Canyon.

(7) No water of subsurface origin was encountered in the exploratory trenches, and the level of permanent groundwater beneath the main terrace area probably is little different from that of the adjacent lower reaches of the deeply incised Diablo Creek.

2.5.1 BASIC GEOLOGIC AND SEISMIC INFORMATION

This section presents the basic geologic and seismic information for DCPP site and surrounding region, resulting from investigations performed during the pre-construction/ early-construction phase. Information contained herein has been obtained from literature studies, field investigations, and laboratory testing and is to be used as a basis for evaluations required to provide a safe design for the facility. The basic data contained in this section and in Reference 27 of Section 2.3 are referenced in several other sections of this FSAR Update. Additional information, developed during the Hosgri evaluation and LTSP evaluation phases are described in Sections 2.5.2.9.3 and 2.5.6, respectively.

2.5.2 VIBRATORY GROUND MOTION

2.5.2.1 Geologic Conditions of the Site and Vicinity

DCPP is situated at the coastline on the southwest flank of the San Luis Range, in the southern Coast Ranges of California. The San Luis Range branches from the main coastal mountain chain, the Santa Lucia Range, in the area north of the Santa Maria Valley and southeast of the plant site, and thence follows an alignment that curves toward the west. Owing to this divergence in structural grain, the range juts out from the regional coastline as a broad peninsula and is separated from the Santa Lucia Range by an elongated lowland that extends southeasterly from Morro Bay and includes Los Osos and San Luis Obispo Valleys. It is characterized by rugged west-northwesterly trending ridges and canyons, and by a narrow fringe of coastal terraces along its southwesterly flank.

Diablo Canyon follows a generally west-southwesterly course from the central part of the range to the north-central part of the terraced coastal strip. Detailed discussions of the lithology, stratigraphy, structure, and geologic history of the plant site and surrounding region are presented in Section 2.5.1.

2.5.2.2 Underlying Tectonic Structures

Evidence pertaining to tectonic and seismic conditions in the region of the DCPP site, developed during the pre-construction/early-construction phase is summarized later in the section, and is illustrated in Figures 2.5-2, 2.5-3, 2.5-4, and 2.5-5. Table 2.5-1 includes a summary listing of the nature and effects of all significant historic earthquakes within 75 miles of the site that have been reported through the end of 1971. Table 2.5-2 shows locations of 19 selected earthquakes that have been investigated by S. W. Smith. Table 2.5-3 lists the principal faults in the region that were identified during the pre-construction/early-construction phase and indicates major elements of their histories of displacement, in geological time units.

Prior to the start of construction of DCPP, Benioff and Smith[5] ~~have~~ assessed the maximum earthquakes to be expected at the site, and John A. Blume and Associates[6,7] ~~have~~ derived the site vibratory motions that could result from these maximum earthquakes (see Section 2.5.2.9.1). An extensive discussion of the geology of the southern Coast Ranges, the western Transverse Ranges, and the adjoining offshore region is presented in Appendix 2.5D of Reference 27 of Section 2.3. Tectonic features of the central coastal region are discussed in Section 2.5.1.1.2, Regional Geologic and Tectonic Setting.

Additional information of the tectonic and seismic conditions was gathered during the Hosgri evaluation and LTSP evaluation phases, as discussed in Sections 2.5.2.9.3 and 2.5.2.9.4, respectively.

2.5.2.3 Behavior During Prior Earthquakes

Physical evidence that indicates the behavior of subsurface materials, strata, and structure during prior earthquakes is presented in Section 2.5.1.2.5. The section presents the findings of the exploratory trenching programs conducted at the site.

2.5.2.4 Engineering Properties of Materials Underlying the Site

A description of the static and dynamic engineering properties of the materials underlying the site is presented in Section 2.5.1.2.6, Site Engineering Properties.

2.5.2.5 Earthquake History

The seismicity of the southern Coast Ranges region is known from scattered records extending back to the beginning of the 19th century, and from instrumental records dating from about 1900. Detailed records of earthquake locations and magnitudes became available following installation of the California Institute of Technology and University of California (Berkeley) seismograph arrays in 1932.

A plot of the epicenters for all large historical earthquakes and for all instrumentally recorded earthquakes of Magnitude 4 or larger that have occurred within 200 miles of DCPP site, through the end of 1971, is given in Figure 2.5-2. Plots of all historically and instrumentally recorded epicenters, through the end of 1971, and all mapped faults within about 75 miles of the site, known through the end of 1971, are shown in Figures 2.5-3 and 2.5-4.

A tabulated list of seismic events through the end of 1971, representing the computer printout from the Berkeley Seismograph Station records, supplemented with records of individual shocks of greater than Magnitude 4 that appear only in the Caltech records, is included as Table 2.5-1. Table 2.5-2 gives a summary of revised epicenters of a representative sample of earthquakes off the coast of California near San Luis Obispo, as determined by S. W. Smith.

2.5.2.7 Identification of Active Faults

Faults that have evidence of recent activity and have portions passing within 200 miles of the site, as known through the end of 1971, are identified in Section 2.5.1.1.2.

2.5.2.8 Description of Active Faults

Active faults that have any part passing within 200 miles of the site, as known through the end of 1971, are described in Section 2.5.1.1.2. Additional active faults were identified during the Hosgri and LTSP evaluation phases, as described in Sections 2.5.2.9.3 and 2.5.2.9.4, respectively.

2.5.2.9 ~~Maximum Earthquake~~ Design and Licensing Basis Earthquakes

The seismic design and evaluation of DCPP is based on the earthquakes described in the following four subsections. Refer to Section 3.7 for the design criteria associated with the application of these earthquakes to the structures, systems, and components.

2.5.2.9.1 Maximum Earthquake (Design Earthquake)

During the pre-construction phase, Benioff and Smith, in reviewing the seismicity of the region around DCPP site, determined the maximum earthquakes that could reasonably be expected to affect the site. Their conclusions regarding the maximum size earthquakes that can be expected to occur during the life of the reactor are listed below:

(1) Earthquake A: A great earthquake may occur on the San Andreas fault at a distance from the site of more than 48 miles. It would be likely to produce surface rupture along the San Andreas fault over a distance of 200 miles with a horizontal slip of about 20 feet and a vertical slip of 3 feet. The duration of strong shaking from such an event would be about 40 seconds, and the equivalent magnitude would be 8.5.

(2) Earthquake B: A large earthquake on the Nacimiento (Rinconada) fault at a distance from the site of more than 20 miles would be likely to produce a 60 mile surface rupture along the Nacimiento fault, a slip of 6 feet in the horizontal direction, and have a duration of 10 seconds. The equivalent magnitude would be 7.5.

(3) Earthquake C: Possible large earthquakes occurring on offshore fault systems that may need to be considered for the generation of seismic sea waves are listed below:

Location	Length of Fault Break	Slip, feet	Magnitude	Distance to Site
Santa Ynez Extension	80 miles	10 horizontal	7.5	50 miles
Cape Mendocino, NW Extension of San Andreas fault	100 miles	10 horizontal	7.5	420 miles
Gorda Escarpment	40 miles	5 vertical or 7 horizontal		420 miles

(4) Earthquake D: Should a great earthquake occur on the San Andreas fault, as described in "A" above, large aftershocks may occur out to distances of about 50 miles from the San Andreas fault, but those aftershocks which are not located on existing faults would not be expected to produce new surface faulting, and would be restricted to depths of about 6 miles or more and magnitudes of about 6.75 or less. The distance from the site to such aftershocks would thus be more than 6 miles.

~~A further assessment of the seismic potential of faults mapped in the region of DCPP site has been made following the extensive additional studies of on- and offshore geology of the last few years that are reported in Appendix 2.5D of Reference 27 of Section 2.3. This was done in terms of observed Holocene activity, to achieve assessment of what seismic activity is reasonably probable, in terms of observed late Pleistocene activity, fault dimensions, and style of deformation.~~

~~PG&E was requested by the NRC to evaluate the plant's capability to withstand a postulated Richter Magnitude 7.5 earthquake centered along an offshore zone of geologic faulting, generally referred to as the "Hosgri fault." The detailed methods, results, and plant modifications performed based on this evaluation are dealt with in Section 3.7.~~

The available information ~~suggests~~ suggested that the faults in this region can be associated with contrasting general levels of seismic potential. These are as follows:

(1) Level I: Potential for great earthquakes involving surface faulting over distances on the order of 100 miles: seismic activity at this level should occur only on the reach of the San Andreas fault that extends between the locales of Cajon Pass and Parkfield. This was the source of the 1857 Fort Tejon earthquake, estimated to have been of Magnitude 8.

(2) Level II: Potential for large earthquakes involving faulting over distances on the order of tens of miles: seismic activity at this level can occur along offshore faults in the Santa Lucia Bank region (the likely source of the

Magnitude 7.3 earthquake of 1927), and possibly along the Big Pine and Santa Ynez faults in the Transverse Ranges.

Although the Rinconada-San Marcos-Jolon, Espinosa, Sur-Nacimiento, and San Simeon faults do not exhibit historical or even Holocene activity indicating this level of seismic potential, the fault dimensions, together with evidence of late Pleistocene movements along these faults, suggest that they may be regarded as capable of generating similarly large earthquakes.

(3) Level III: Potential for earthquakes resulting chiefly from movement at depth with no surface faulting, but at least with some possibility of surface faulting of as much as a few miles strike length and a few feet of slip: Seismic activity at this level probably could occur on almost any major fault in the southern Coast Ranges and adjacent regions.

From the observed geologic record of limited fault activity extending into Quaternary time, and from the historical record of apparently associated seismicity, it can be inferred that both the greater frequency of earthquake activity and larger shocks from earthquake source structures having this level of seismic potential probably will be associated with one of the relatively extensive faults. Faults in the vicinity of the San Luis Range that may be considered to have such seismic potential include the West Huasna, Edna, and offshore Santa Maria Basin East Boundary zone.

(4) Level IV: Potential for earthquakes and aftershocks resulting from crustal movements that cannot be associated with any near-surface fault structures: such earthquakes apparently can occur almost anywhere in the region.

This information forms the basis of the Design Earthquake, described in Section 2.5.2.10.1.

2.5.2.9.2 Double Design Earthquake

In order to assure adequate reserve seismic resisting capability of safety related structures, systems, and components, an earthquake producing two-times the acceleration values of the Design Earthquake ~~is~~ was also considered (Reference 51).

2.5.2.9.3 1977 Hosgri Earthquake

In 1976, subsequent to the issuance of the construction permit of Unit 1, PG&E was requested by the NRC to evaluate the plant's capability to withstand a postulated Richter Magnitude 7.5 earthquake centered along an offshore zone of geologic faulting, generally referred to as the "Hosgri fault." Details of the investigations associated with this fault are provided in Appendices 2.5D, 2.5E, and 2.5F of Reference 27 in Section 2.3. An overview is provided in Section 2.5.2.10.3.

During the Hosgri evaluation phase, a further assessment of the seismic potential of faults mapped in the region of DCPP site was made following the extensive additional studies of on- and offshore geology, and are reported in Appendix 2.5D of Reference 27 of Section 2.3. This was done in terms of observed Holocene activity, to achieve assessment of what seismic activity is probable, in terms of observed late Pleistocene activity, fault dimensions, and style of deformation.

2.5.2.9.4 1991 Long Term Seismic Program Earthquake

License Condition No. 2.C.(7) of the Unit 1 Operating License included the following elements pertaining to the seismic design basis for DCPP:

(1) PG&E shall identify, examine, and evaluate all relevant geologic and seismic data, information, and interpretations that have become available since the 1979 ASLB hearing in order to update the geology, seismology and tectonics in the region of the Diablo Canyon Nuclear Power Plant. If needed to define the earthquake potential of the region as it affects the Diablo Canyon Plant, PG&E will also reevaluate the earlier information and acquire additional new data.

(2) PG&E shall reevaluate the magnitude of the earthquake used to determine the seismic basis of the Diablo Canyon Nuclear Plant using the information from Element 1.

(3) PG&E shall reevaluate the ground motion at the site based on the results obtained from Element 2 with full consideration of site and other relevant effects.

(4) PG&E shall assess the significance of conclusions drawn from the seismic reevaluation studies in Elements 1, 2 and 3, utilizing a probabilistic risk analysis and deterministic studies, as necessary, to assure adequacy of seismic margins.

PG&E's evaluations in response to these elements of the license condition included the development of significant additional data applicable to the geology, seismology, and tectonics of the DCPP region. Based on this data, PG&E identified four capable faults, the Hosgri, Los Osos, San Luis Bay, and Wilmer Avenue faults, requiring evaluation as potential seismic sources (Reference 40, Chapter 3). However, PG&E determined that the governing earthquake for the LTSP deterministic seismic margins review of DCPP (84th percentile ground motion response spectrum) is a Richter Magnitude 7.2 earthquake centered along an offshore zone of geologic faulting, generally referred to as the "Hosgri fault." Details are provided in References 40 and 41. New faults were introduced and evaluated in the 1988 LTSP Report. Details are provided in Reference 40, Chapter 2.0 and summarized in SSER 34, Section 2.5.1, "Geology" and 2.5.2, "Seismology".

The NRC's review of PG&E's evaluations is documented in References 42 and 43.

2.5.2.10 Ground Accelerations and Response Spectra

The seismic design and evaluation of DCPP is based on the earthquakes described in the following four subsections. Refer to Section 3.7 for the design criteria associated with the application of these earthquakes to the structures, systems, and components.

2.5.2.10.1 Maximum Earthquake (Design Earthquake)

During the pre-construction/early-construction phase, t~~T~~he maximum ground acceleration that would occur at DCPP site ~~has been~~was estimated for each of the postulated earthquakes listed in Section 2.5.2.9, using the methods set forth in References 12 and 24. The plant site acceleration ~~is~~ was primarily dependent on the following parameters: Gutenberg-Richter magnitude and released energy, distance from the earthquake focus to the plant site, shear and compressional velocities of the rock media, and density of the rock. Rock properties are discussed under Section 2.5.1.2.6, Site Engineering Properties.

The maximum rock accelerations that would occur at the DCPP site ~~are~~ were estimated as:

Earthquake A	0.10 g	Earthquake C	0.05 g
Earthquake B	0.12 g	Earthquake D	0.20 g

In addition to the maximum acceleration, the frequency distribution of earthquake motions ~~is~~ was important for comparison of the effects on plant structures and equipment. In general, the parameters affecting the frequency distribution are distance to the rupture plane, properties of the transmitting media, length of faulting, focus depth, and total energy release. Radiated ~~E~~earthquake~~s~~ energy that might reach the site after traveling over great distances will~~would~~ tend to have the~~ir~~ high frequency waves filtered out. Earthquake~~s~~ ruptures that might be centered close to the site ~~would~~ will tend to produce wave forms at the site having minor low frequency characteristics.

In order to evaluate the frequency distribution of earthquakes, the concept of the response spectrum wa~~i~~s used.

Using the attenuation relations available at the time, f~~F~~or nearby earthquakes, the resulting response spectra accelerations ~~would~~ peaked sharply at short periods and ~~would~~ decayed rapidly at longer periods. Earthquake D ~~would~~ produced such response spectra. The March 1957 San Francisco earthquake as recorded in Golden Gate Park (S80°E component) was the same type. It produced a maximum recorded ground acceleration of 0.13 g (on rock) at a distance of about 8 miles from the epicenter. Since Earthquake D had~~s~~ an assigned hypocentral distance of 12 miles, it ~~would~~ was ~~be~~ expected to produce response spectra similar in shape to those of the 1957 event.

Large earthquakes centered at some distance from the plant site ~~would~~ tended to produce response spectra accelerations that peaked at longer periods than those for nearby smaller shocks. Such spectra maintained a higher spectral acceleration throughout the period range beyond the peak period. Earthquakes A and C ~~a~~were events that ~~would~~ tended to produce this type of spectra. The intensity of shaking as indicated by the maximum predicted ground acceleration showed~~s~~ that Earthquake C would always have lower spectral accelerations than Earthquake A.

Since the two shocks ~~would~~ have approximately the same shape spectra, Earthquake C would always have lower spectral accelerations than Earthquake A, and it wa~~i~~s therefore eliminated from further consideration. The north-south component of the 1940 El Centro earthquake produced response spectra that emphasized the long period characteristics described above. Earthquake A, because of its distance from the plant site, w~~ould~~as ~~be~~ expected to produce response spectra similar in shape to those produced by the El Centro event. Smoothed response spectra for Earthquake A were constructed by normalizing the El Centro spectra to 0.10 g. These spectra, however, showed smaller accelerations than the corresponding spectra for Earthquake B (discussed in the next paragraph) for all building periods, and thus Earthquake A wa~~i~~s also eliminated from further consideration.

Earthquake B ~~would~~ tended to produce response spectra that emphasize the intermediate period range in as much as the epicenter wa~~i~~s not close enough to the plant site to produce large high frequency (short-period) effects, and it wa~~i~~s too close to the site and too small in magnitude to produce large low frequency (long-period) effects. The N69°W component to the 1952 Taft earthquake produced response spectra having such characteristics. That shock was therefore used as a guide in establishing the shape of the response spectra that was~~ould be~~ expected for Earthquake B.

Following several meetings with the AEC staff and their consultants, the following two modifications were made in order to make the criteria more conservative:

(1) The Earthquake D time-history was modified in order to obtain better continuity of frequency distribution between Earthquakes D and B.

(2) The accelerations of Earthquake B were increased by 25 percent in order to provide the required margin of safety to compensate for possible uncertainties in the basic earthquake data.

Accordingly, Earthquake D-modified was derived by modifying the S80°E component of the 1957 Golden Gate Park, San Francisco earthquake, and then normalizing to a maximum ground acceleration of 0.20 g. Smoothed response spectra for this earthquake are shown in Figure 2.5-21. Likewise, Earthquake B was derived by normalizing the N69°W component of the 1952 Taft earthquake to a maximum ground acceleration of 0.15 g. Smoothed response spectra for Earthquake B are shown in Figure 2.5-20. The maximum vibratory motion at the plant site ~~would be~~as produced by either Earthquake D-modified or Earthquake B, depending on the natural period of the vibrating body.

2.5.2.10.2 Double Design Earthquake

The maximum ground acceleration and response spectra for the Double Design Earthquake are twice those associated with the design earthquake, as described in Section 2.5.2.10.1 (Reference 51).

2.5.2.10.3 1977 Hosgri Earthquake

As mentioned earlier, based on a review of the studies presented in Appendices 2.5D and 2.5E (of Reference 27 in Section 2.3) by the NRC and the USGS (acting as the NRC's geological consultant), Supplement No. 4 to the NRC Safety Evaluation Report (SER) was issued in May 1976. This supplement included the USGS conclusion that a magnitude 7.5 earthquake could occur on the Hosgri fault at a point nearest to the Diablo Canyon site. The USGS further concluded that such an earthquake should be described in terms of near fault horizontal ground motion using techniques and conditions presented in Geological Survey Circular 672. The USGS also recommended that an effective, rather than instrumental, acceleration be derived for seismic analysis.

The NRC adopted the USGS recommendation of the seismic potential of the Hosgri fault. In addition, based on the recommendation of Dr. N. M. Newmark, the NRC prescribed that an effective horizontal ground acceleration of 0.75g be used for the development of response spectra to be employed in a seismic evaluation of the plant. The NRC outlined procedures considered appropriate for the evaluation including an adjustment of the response spectra to account for the filtering effect of the large building foundations. An appropriate allowance for torsion and tilting was to be included in the analysis. A guideline for the consideration of inelastic behavior, with an associated ductility ratio, was also established.

The NRC issued Supplement No. 5 to the SER in September 1976. This supplement included independently-derived response spectra and the rationale for their development. Parameters to be used in the foundation filtering calculation were delineated for each major structure. The supplement prescribed that either the spectra developed by Blume or Newmark would be acceptable for use in the evaluation with the following conditions:

(1) In the case of the Newmark spectra no reduction for nonlinear effects would be taken except in certain specific areas on an individual case basis.

(2) In the case of the Blume spectra a reduction for nonlinear behavior using a ductility ratio of up to 1.3 may be employed.

(3) The Blume spectra would be adjusted so as not to fall below the Newmark spectra at any frequency.

The development of the Blume ground response spectra, including the effect of foundation filtering, is briefly discussed below. The rationale and derivation of the Newmark ground response spectra is discussed in Appendix C to Supplement No. 5 of the SER.

The time-histories of strong motion for selected earthquakes recorded on rock close to the epicenters were normalized to a 0.75g peak acceleration. Such records provide the best available models for the Diablo Canyon conditions relative to the Hosgri fault zone. The eight earthquake records used are listed in the table below.

Earthquake	M	Depth, km	Recorded at	Epicentral Distance, km	Component	Peak Acceleration g
Helena 1935	6	5	Helena	3 to 8	EW	0.16
Helena 1935	6	5	Helena	3 to 8	NS	0.13
Daly City 1957	5.3	9	Golden Gate Park	8	N80W	0.13
Daly City 1957	5.3	9	Golden Gate Park	8	N10E	0.11
Parkfield 1966	5.6	7	Temblor 2	7	S25W	0.33
Parkfield 1966	5.6	7	Temblor 2	7	N65W	0.28
San Fernando 1971	6.6	13	Pacoima Dam	3	S14W	1.17
San Fernando 1971	6.6	13	Pacoima	3	N76W	1.08

The magnitudes are the greatest recorded thus far (September 1985) close in on rock stations and range from 5.3 to 6.6. Adjustments were made subsequently in the period range of the response spectrum above 0.40 sec for the greater long period energy expected in a 7.5M shock as compared to the model magnitudes.

The procedure followed was to develop 7 percent damped response spectra for each of the eight records normalized to 0.75g and then to treat the results statistically according to period bands to obtain the mean, the median, and the standard deviations of spectral response. At this stage, no adjustments for the size of the foundation or for ductility were made. The 7 percent damped response spectra were used as the basis for calculating spectra at other damping values.

Figures 2.5-29 and 2.5-30 show free-field horizontal ground response spectra as determined by Blume and Newmark, respectively, at damping levels from two to seven percent.

Figures 2.5-31 and 2.5-32 show vertical ground response spectra as determined by Blume and Newmark, respectively, for two to seven percent damping. The ordinates of vertical spectra are taken as two-thirds of the corresponding ordinates of the horizontal spectra. These response spectra, finalized in 1977, are described as the "1977 Hosgri response spectra ".

2.5.2.10.4 1991 Long Term Seismic Program Earthquake

As discussed in Section 2.5.2.9.4, the Long Term Seismic Program, in response to License Condition No. 2.C.(7) determined that the governing earthquake for the deterministic seismic margins evaluation of DCPP (84th percentile ground motion response spectrum) is a Richter Magnitude 7.2 earthquake centered along an offshore zone of geologic faulting, generally referred to as the "Hosgri fault."

Ground motions, and the corresponding free-field response spectra for the LTSP earthquake, were developed by PG&E, as documented in Reference 40. As part of their review of Reference 40, the NRC concluded that spectra developed by PG&E could underestimate the ground motion (Reference 42). As a result, the final spectra, applicable to the LTSP evaluation of DCPP, is an envelope of that developed by PG&E and that developed by the NRC. Figures 2.5-33 and 2.5-34 show the 84th percentile ground motion response spectrum at 5% damping for the horizontal and vertical directions, respectively, described as the "1991 LTSP response spectra". These spectra define the current licensing basis for the LTSP.

Figure 2.5-35 shows a comparison of the horizontal 1991 LTSP response spectrum with the 1977 Newmark Hosgri spectrum (based on Reference 40, Figure 7-2). This comparison indicates that the 1977 Hosgri spectrum is greater than the 1991 LTSP spectrum at all frequencies less than about 15 Hz, but the 1991 LTSP spectrum exceeds the 1977 Hosgri spectrum by approximately 10 percent for frequencies above 15 Hz. This exceedance was accepted by the NRC in SSER-34 (Reference 42), Section 3.8.1.1 (Ground-Motion Input for Deterministic Evaluations):

> "On the basis of PG&E's margins evaluation discussed in Section 3.8.1.7 of this SSER, the staff concludes that these high-frequency spectral exceedances are not significant."

In addition, the NRC states in SSER-34 (Reference 42), Section 1.4 (Summary of Staff Conclusions):

> "The staff notes that the seismic qualification basis for Diablo Canyon will continue to be the original design basis plus the Hosgri evaluation basis, along with the associated analytical methods, initial conditions, etc. The LTSP has served as a useful check of the adequacy of the seismic margins and has generally confirmed that the margins are acceptable."

Therefore, the 1991 LTSP ground motion response spectra supplements, but does not replace or modify, the DE, DDE, or 1977 Hosgri response spectra described above.

2.5.4.9 Earthquake Design Basis

The earthquakes postulated for DCPP site are discussed in Section 2.5.2.9, ~~and~~ a discussion of the design response spectra is provided in Section 2.5.2.10, and the application of the earthquake ground motions to the seismic analysis of structures, systems, and components is provided in Section 3.7. ~~Response acceleration curves for the site resulting from Earthquake B and Earthquake D-modified are shown in Figures 2.5-20 and 2.5-21, respectively. Response spectrum curves for the 7.5M Hosgri earthquake are shown in Figures 2.5-29 through 2.5-32.~~

2.5.6 Long Term Seismic Program

On November 2, 1984, the NRC issued the Diablo Canyon Unit 1 Facility Operating License DPR-80. In DPR-80, License Condition Item 2.C.(7), the NRC stated, in part:

> "PG&E shall develop and implement a program to reevaluate the seismic design bases used for the Diablo Canyon Power Plant."

PG&E's reevaluation effort in response to the license condition was titled the "Long Term Seismic Program" (LTSP). PG&E prepared and submitted to the NRC the "Final Report of the Diablo Canyon Long Term Seismic Program" in July 1988 (Reference 40). Between 1988 and 1991, the NRC performed an extensive review of the Final Report, and PG&E prepared and submitted written responses to formal NRC questions. In February 1991, PG&E issued the "Addendum to the 1988 Final Report of the Diablo Canyon Long Term Seismic Program" (Reference 41). In June 1991, the NRC issued Supplement Number 34 to the Diablo Canyon Safety Evaluation Report (SSER) (Reference 42) in which the NRC concluded that PG&E had satisfied License Condition 2.C.(7) of Facility Operating License DPR-80. In the SSER the NRC requested certain confirmatory analyses from PG&E, and PG&E subsequently submitted the requested analyses. The NRC's final acceptance of the LTSP is documented in a letter to PG&E dated April 17, 1992 (Reference 43).

The LTSP contains extensive data bases and analyses that update the basic geologic and seismic information in this section of the FSAR Update. The LTSP material does not address or alter the current design licensing basis for the plant. In SSER-34 (Reference 42), the NRC stated, "The Staff notes that the seismic qualification basis for Diablo Canyon will continue to be the original design basis plus the Hosgri Evaluation basis, along with associated analytical methods, initial conditions, etc.

As a condition of the NRC's final acceptance of the LTSP, PG&E committed to ongoing activities in support of the LTSP, described as the "Framework for the Future," in a letter to the NRC, dated April 17, 1991 (Reference 50). These ongoing activities include the following (Reference 42, Section 2.5.2.4):

(1) To continue to maintain a strong geosciences and engineering staff to keep abreast of new geological, seismic, and seismic engineering information and evaluate it with respect to its significance to Diablo Canyon.

(2) To continue to operate a strong-motion accelerometer array and the coastal seismic network, although likely with fewer stations than currently operating.

The implementation of Activity (1) is described in the following sections; the implementation of Activity (2) is described in Section 3.7.4.

A complete listing of bibliographic references to the LTSP reports and other documents may be found in References 40, 41 and 42.

2.5.6.1 Ongoing Geological and Seismological Investigations

As discussed in Section 2.5.6, PG&E committed to ongoing geological and seismological investigations in support of the LTSP, and to evaluate the findings with respect to their significance to DCPP (Reference 42, Section 2.5.2.4).

These investigations are performed by the PG&E Geosciences Department, and include the following:

(1) Maintain knowledge of major earthquakes occurring worldwide in order to evaluate their significance to DCPP

(2) Review near-fault recordings from any large magnitude earthquakes which occur near DCPP, collected through the seismic monitoring system, operated by PG&E at DCPP (Section 3.7.4) and operated by other agencies in the area

(3) Review and/or participate in the development of new ground motion models (e.g., attenuation relationships)

(4) Review and evaluate potential changes to source characterization for faults near DCPP

(5) Monitor ground motion data for small and moderate earthquakes occurring near DCPP, collected through PG&E's Central Coast Seismic Network

The results of these investigations are used by the PG&E Geosciences Department to develop updated estimates of the ground motion applicable to both the deterministic seismic margin and the seismic probabilistic risk assessment (SPRA) parts of the LTSP evaluation of DCPP, as described in Sections 2.5.6.2.1 and 2.5.6.2.2, respectively.

The development of the updated estimates of ground motion response spectra for each fault under consideration, for use in the deterministic seismic margins evaluation is based on the following:

(1) The source characterization is developed describing the magnitudes, locations, rates, and faulting styles of future potential earthquakes in the DCPP region. Alternative models are developed to capture the center, body, and range of the scientific (epistemic) uncertainty in the source characterization and are modeled using logic trees. The source characterization will be peer reviewed.

(2) The deterministic earthquake magnitude is selected based on the 90th fractile of the mean characteristic magnitude from the alternative models defined by the logic tree.

(3) The distance is established based on the shortest distance from the fault to the DCPP power block

(4) The ground motion characterization is developed describing the median and standard deviation of the ground motion for a given magnitude, distance, style-of-faulting for the DCPP site condition. Alternative ground motion prediction equations (GMPEs) are developed to capture the center, body, and range of the scientific (epistemic) uncertainty in the ground motion models using logic trees. The ground motion characterization will be peer reviewed.

(5) The deterministic ground motion is computed for each GMPE using the 84th percentile level from the aleatory variability.

(6) The final deterministic ground motion spectrum is given by the weighted geometric mean of the 84th percentile ground motions from the alternative GMPEs.

The development of the updated estimates of spectral shapes and seismic hazard curves for the SPRA evaluation is based on one of the following:

(1) A probabilistic seismic hazard analysis (PSHA) is conducted using the source characterization and ground motion characterization described above.

(2) The uniform hazards spectra (UHS) are computed based on the mean hazard for a suite of hazard levels (e.g. 1E-3, 1E-4, 1E-5, 1E-6, 1E-7).

(3) At each hazard level, the spectral shape will be based on either the UHS or on a suite of scenario spectra that represent realistic earthquakes.

If the scenario spectra approach is used, the suite of scenario spectra is checked to show that the seismic hazard computed from these spectra envelop the mean hazard curves over frequencies from 0.5 to 33~~0~~ Hz.

The calculations of the ground motions will follow the PG&E Geosciences Department Quality Assurance (QA) procedure (see Section 17.2.1(4)). In addition, the updated ground motion estimates will be peer reviewed by PG&E's Seismic Advisory Board (SAB). The SAB is comprised of a selection of outside industry experts, and members of the academic community, in the following areas of knowledge:

- Ground motions

- Seismic hazards
- Seismic source characterization
- Seismic risk
- Seismic fragilities

The charter of the SAB is to review the updated seismic hazards calculations for changes in methodologies and key modeling assumptions. In most cases, the full SAB perform their review and document the results in a single consensus report. However, in some cases, it may only be necessary to have the review performed by those members with expertise in the technical topic under consideration. In such cases, individual reviews, rather than a consensus review, will be provided. A minimum of two SAB members are required for a specific topic.

An official review letter, documenting the SAB's review and conclusions, is required as part of the peer review process. A written response to the SAB's comments will be prepared by PG&E, documenting how the SAB comments were addressed. The official review letter, and written response to the SAB's comment, will be submitted to the NRC as a part of LTSP update process. The regular ten year update to the LTSP Report will be performed consistent with the recommendations of NUREG/CR-6372, "Recommendations for Probabilistic Seismic Hazard Analysis: Guidance on Uncertainty and Use of Experts," for a Level 3 Senior Seismic Hazard Analysis Committee.

It should be noted that since these results are associated with the LTSP, and as discussed in Section 2.5.2.10.4, the NRC has indicated that the LTSP does not redefine the seismic design basis for DCPP, the updated estimates of the ground motion are compared with the current licensing basis for the LTSP, as outlined in Sections 2.5.6.2.1 and 2.5.6.2.2. The 1991 LTSP spectra is enveloped by the 1977 Hosgri Earthquake spectrum (Figure 2.5-35) with the exception of exceedances at certain frequencies, as approved by the NRC in SSER 34.

In no case shall the results of the ongoing investigations in support of the LTSP result in changes to the design basis earthquakes: the DE, as described in Section 2.5.2.10.1; the DDE, as described in Section 2.5.2.10.2; or the 1977 Hosgri earthquake, as described in Section 2.5.2.10.3, except if new ground motion spectra were to exceed both the LTSP spectra and the 1977 Hosgri spectra at any frequency. A license amendment would be required to address these exceedances.

2.5.6.2 Evaluation of Updated LTSP Ground Motions

As a result of the ongoing geological and seismological investigations associated with the LTSP, the PG&E Geosciences Department provides updated ground motion information to DCPP, either on a ten year interval or more frequently as the result of significant new discoveries. The updated ground motions for the LTSP earthquake are defined by each of the following:

(1) 84th percentile ground motion response spectrum (spectral acceleration vs. frequency). See Figures 2.5-33 and 2.5-34 for examples.

(2) Mean probabilistic seismic hazard curves (annual frequency of exceedance vs. average spectral acceleration for the 3.0 to 8.5 Hz frequency range) and ground motion spectral shapes. See Figures 2.5-36 and 2.5-37 for examples.

These two characterizations of the updated ground motion serve as input to the seismic margins evaluation and the seismic probabilistic risk assessment evaluation, as described in Sections 2.5.6.2.1 and 2.5.6.2.2, respectively. An overview of the evaluation for updated LTSP ground motions is shown in Figure 2.5-38.

2.5.6.2.1 Seismic Margin Evaluation

The seismic evaluations performed in support of the LTSP (References 40 and 41) demonstrated that DCPP has adequate seismic margins for the ground motions defined by the current licensing basis 1991 LTSP ground motions (Figures 2.5-33 and 2.5-34). The process for the evaluation of the impact of updated deterministic ground motion response spectra on the LTSP seismic margins evaluation is illustrated in Figure 2.5-38, sheet 2. Guidance in the performance of seismic margins evaluations is provided in EPRI NP-6041-SL (Reference 56). An overview of the seismic margins evaluation performed for the 1991 LTSP is provided in Section 2.5.6.4 and details are provided in References 40 and 41.

Upon receipt of an updated 84th percentile ground motion response spectrum (horizontal and vertical directions, as applicable) from the PG&E Geosciences Department, ~~,~~ the updated spectrum will be compared to the current licensing basis LTSP spectrum. The ~~two~~ possible outcomes of this comparison will be addressed as follows:

(1) If the updated spectrum is enveloped by the current licensing basis LTSP spectrum, the seismic margins remain adequate and the results of the comparison shall be documented, as described in ~~Section 2.5.6.3~~Technical Specification 5.6.11, "Long Term Seismic Program Report." Otherwise, proceed to Step (2).

(2) If the updated spectrum exceeds the current licensing basis LTSP spectrum at any frequency, engineering evaluations are required to assess the impact of the updated ground motions on the seismic margins for DCPP and to determine if changes to the current licensing basis LTSP spectrum is required. The engineering evaluations will include the following:

(a) A review of the frequency range of the exceedance to determine which structures, systems, or components (SSCs) are impacted. At this point, it may be necessary to regenerate the in-structure response spectra and/or recompute the high-confidences-low-probability-of-failure (HCLPF) capacities of affected SSCs (see Section 3.7.6.2 for discussion of HCLPF capacities).

(b) An evaluation of the impact of the exceedances on the seismic margins for the affected SSCs. Note that the seismic margins for all SSCs that have the potential to impact SCDF were in the scope of the LTSP and are listed in Tables 3.7-25 and 3.7-26.

(c) If the minimum seismic margin remains greater than or equal to 1.3 (or greater than or equal to the approved seismic margin exceptions for certain SSCs discussed in Section 2.5.6.2.1.1), the updated response spectrum is acceptable and proceed to Step (3). Otherwise, proceed to Step (d).

(d) If the minimum seismic margin is greater than or equal to 1.0 (or greater than or equal to the approved seismic margin exceptions for certain SSCs discussed in Section 2.5.6.2.1.1), the SSC can perform its safety function, proceed to Step (f). Otherwise, proceed to Step (e).

(e) The applicable TS Limiting Condition for Operation shall be entered for the SSCs having a minimum seismic margin less than 1.0 (unless the SSC is one of the approved seismic margin exceptions below 1.0 discussed in Section 2.5.6.2.1.1). Appropriate compensatory measures are to be implemented if feasible.

(f) Develop and implement modifications to impacted SSCs to achieve a minimum seismic margin of 1.3 (or to achieve the approved seismic margin exception discussed in Section 2.5.6.2.1.1).

(3) Process a change to the licensing basis 1991 LTSP spectrum (and 1977 Hosgri spectrum if it is exceeded at any frequency or justify why a change is not necessary) through the license amendment request process. Once the license amendment has been issued, proceed to Section 2.5.6.3.

2.5.6.2.1.1 Approved Minimum Seismic Margins Less Than 1.3

Even though the target minimum seismic margin for SSCs within the scope of the LTSP is 1.3, exceptions to this value have been accepted on a case-by-case basis for certain SSCs. The following provides a summary of these exceptions:

(1) Exceptions previously approved by the NRC

The following exceptions to the target minimum seismic margin of 1.3 are associated with SSCs as the existed during the 1991 LTSP evaluation. These exceptions were previously approved by the NRC (Reference 42).

(a) Turbine Building

As indicated in Table 3.7-25, the $HCLPF_{84}$ capacity of the turbine building is 2.21 g, based on the fragility analysis method, giving a seismic margin of 1.14, which is less than the target minimum margin of 1.3. The limiting capacity is associated with the onset of severe structural distress (significant strength degradation) to the major east-west shear walls. Due to the fact that the turbine houses various components associated with the vital electric power system (e.g., emergency diesel generators and 4160V vital switchgear) and the vital cooling water system (e.g., component cooling water heat exchangers), coupled with the fact that this is the structure with the lowest seismic capacity, the overall plant fragility is governed by this building.

In order to evaluate the conservatism of the reported HCLPF84 capacity, a rigorous seismic evaluation was performed using state-of-the-art analytical methods beyond those normally employed for the fragility analysis method. This evaluation utilized multiple non-linear time history analyses (Reference 57) to estimate the seismic capacity associated with the ultimate failure of the structure. The results of these analyses indicated that a realistic estimate of the seismic margin is likely in excess of 1.40. These analyses were reviewed and acceptable by the NRC (Reference 43).

(b) 4160V Vital Switchgear Relay Chatter

As indicated in Table 3.7-26, the HCLPF84 capacity associated with chatter of the overcurrent relays in the 4160V vital switchgear is 1.57 g, giving a seismic margin of 0.81, which is less than the target minimum margin of 1.3. However, the failure mode associated with this chatter is recoverable by operator action from the Control Room (resetting the relays), and the probabilities associated with operator action have been included in the PRA model for the system. The PRA model indicates that this failure mode does not have a significant impact on the core damage frequency. This evaluation was reviewed and accepted by the NRC (Reference 43).

(c) 230kV Offsite Power System/Switchyard

As indicated in Table 3.7-26, the HCLPF84 capacity associated with 230kV offsite power system is 0.84 g, giving a seismic margin of 0.43, which is less than the target minimum margin of 1.3. This capacity is limited by the failure of ceramic insulators, transformers, and circuit breakers, and is based on the earthquake experience data method (Section 3.7.6.2.3), as documented in Reference 58.

Since this system is the primary source of offsite power, it is assumed to be lost due to a major earthquake, with back-up power provided by the emergency diesel generators. However, in order to allow rapid recovery of offsite power, key spare parts are stored onsite. These parts include items such as conductors, connectors, insulators, and transformer bushings. The maintenance of the spare parts is a licensing commitment made in References 40 and 58, as acknowledged by the NRC in Reference 42.

(2) Exceptions Associated with Additions and Modifications

The following exceptions to the target minimum seismic margin of 1.3 are associated with additions and modifications implemented subsequent to the completion of the 1991 LTSP evaluation. The acceptance of the lower seismic margin is based on the requirements of Reference 59, which permitted the acceptance of seismic margins as low as 1.14 for plant modifications and additions.

(a) Integrated Head Assembly

Integrated head assemblies (IHAs) were installed in Units 1 and 2 during refueling outage nos. 2R15 and 1R16, respectively. The IHAs are classified as new components which could significantly impact the seismic margins of existing safety-related structures (see Section 3.7.6.1.1), since they are attached to the reactor vessel closure heads and provide support to the control rod drive mechanisms (CRDMs), small bore piping, instrumentation, and cables. An assessment of their impact on the seismic PRA indicated that the key function is the lateral support of the CRDMs, since excess deflection of the CRDMs could impair the downwards movement of the control rods, required for reactor trip.

The HCLPF84 capacity associated with the limiting element of the CRDM lateral support function of the IHAs, developed based on the conservative deterministic failure margins method (Section 3.7.6.2.2), is 2.40 g, giving a seismic margin of 1.24.

2.5.6.2.2 Seismic Probabilistic Risk Assessment Evaluation

The LTSP evaluation for DCPP also included a Seismic Probabilistic Risk Assessment (SPRA), which estimated the annual seismic core damage frequency (SCDF) (References 40 and 41). The process for the SPRA evaluation of the updated seismic hazard information is illustrated in Figure 2.5-38, sheet 3.

If the UHS approach is used, the input to the SPRA evaluation includes:

(1) Seismic hazard curves provided by the PG&E Geosciences Department. See Figure 2.5-36 for an example.

(2) Ground motion spectral shapes provided by the PG&E Geosciences Department. See Figure 2.5-37 for an example.

(3) Fragilities developed in accordance with ASME/ANS RA-Sa-2009 (Reference 54). See Figure 2.5-39 for an example.

Other methods for developing seismic hazard information are allowed provided they are peer reviewed.

The evaluation of the updated seismic hazards information will proceed as follows:

(1) Conduct SPRA to determine current SCDF value. Note that the SPRA is classified as Capability Category II per ASME/ANS RA-Sa-2009 (Reference 54), as modified by Regulatory Guide 1.200, rev. 2 (Reference 55) and is subject to a peer-review process.

(2) Report the calculated SCDF to the NRC.

(3) Document updated seismic hazard information, fragilities, and SDCF in DCPP records.

(4) Update LTSP documentation.

2.5.6.3 LTSP Configuration Control

The implementation of the LTSP seismic PRA relies on several key items to assure an acceptable level of core damage frequency. The following items must be maintained in the proper configuration to assure continued validity of the seismic PRA (Reference 41):

(1) Diesel Fuel Oil Transfer System

In order to assure a reliable supply of fuel oil for the diesel generators, the following features associated with the diesel fuel oil system shall be maintained:

(a) Recirculation lines to allow the system to operate continuously once a start demand has been received for any day tank level.

(b) Provisions for the manual operation of the level control valves on the day tanks.

(c) Provisions for the connection of a portable engine-driven pump to the transfer system.

(2) Centrifugal Charging Pump Backup Cooling

In order to assure adequate cooling of the centrifugal changing pump lube oil and seal coolers, in the event of the complete loss of component cooling water, provisions are provided for the use of firewater to cool the pumps. This is accomplished through the use of dedicated hoses to interconnect the firewater header and the charging pump coolers. This feature is in support of reactor coolant pump seal injection and seal cooling.

(3) 230kV Offsite Power System Spare Parts

In order to ensure post-earthquake restoration of this system in a timely manner, key spare parts for the 230kV offsite power system shall be stored on site.

(4) 4160V Overcurrent Relay Remote Reset

In order to recover from breaker trips in the 4160V switchgear, the capability to reset an overcurrent trip from the control room shall be maintained.

(5) Component Cooling Water and Safety Injection Valve Control Switches

In order to prevent relay chatter-induced position changes for the component cooling water pump discharge valves and the safety injection pump suction valves, the two-position valve control switches (with maintained contacts) shall be maintained.

2.5.6.4 Elements of a Seismic Margins Evaluation

The elements of the seismic margins evaluation are as follows:

(1) Determine the seismic demand associated with the deterministic ground motion (Figure 2.5-33). The seismic demand for the ground motion is

defined as the 5 percent damped spectral acceleration averaged between 3 and 8.5 Hz. This is illustrated on Figure 2.5-40.

(2) Determine the seismic capacity of each structure, system, or component (SSC) within the LTSP scope. The seismic capacity for SSCs at DCPP is defined based on the High Confidence Lower Probability of Failure (HCLPF) 5 percent damped spectral acceleration capacity averaged between 3 and 8.5 Hz. This value can be determined using the fragility analysis method (Section 3.7.6.2.1), the conservative deterministic failure margins method (Section 3.7.6.2.2), or the earthquake experience data method (Section 3.7.6.2.3). This is also illustrated on Figure 2.5-40.

(3) In general, the seismic margin is defined as the ratio of the capacity of the SSC to the demand. However, this value must be adjusted to account for the demand contributions associated with other applicable loads (e.g., deadweight, pressure, thermal).

Note that the process for the seismic margins evaluation, described above, is in terms of the horizontal ground motion and the capacity of the SSC relative to horizontal input motion. A similar approach can be applied to the vertical ground motion and the capacity of the SSC relative to vertical input motion. However, as discussed in Chapter 6 of Reference 40, the capacities of most SSCs are dominated by their response to horizontal input motion, and the contribution due to vertical input motion is generally small. Therefore, the consideration of the impact of vertical input motion on the seismic margin of a specific SSC will be addressed on a case-by-case basis.

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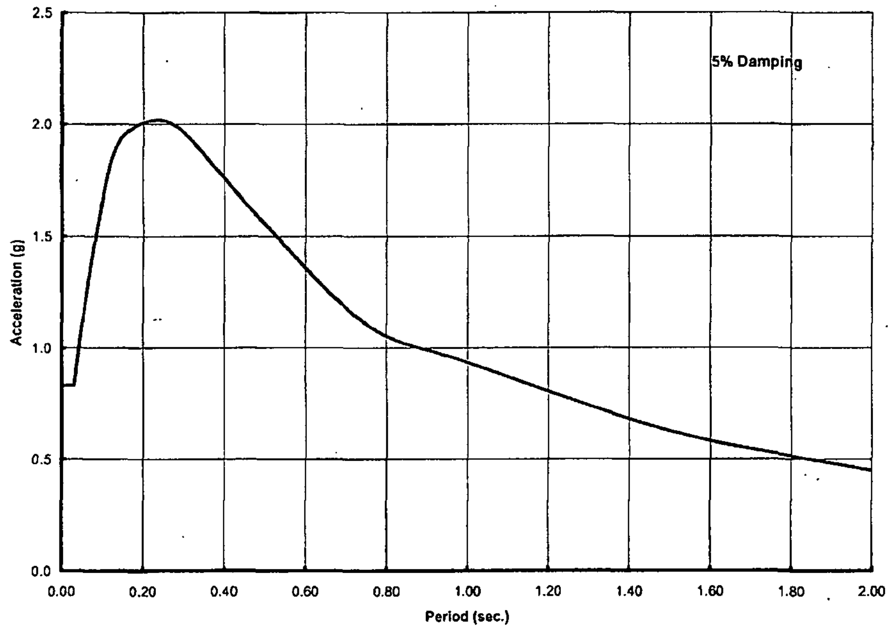
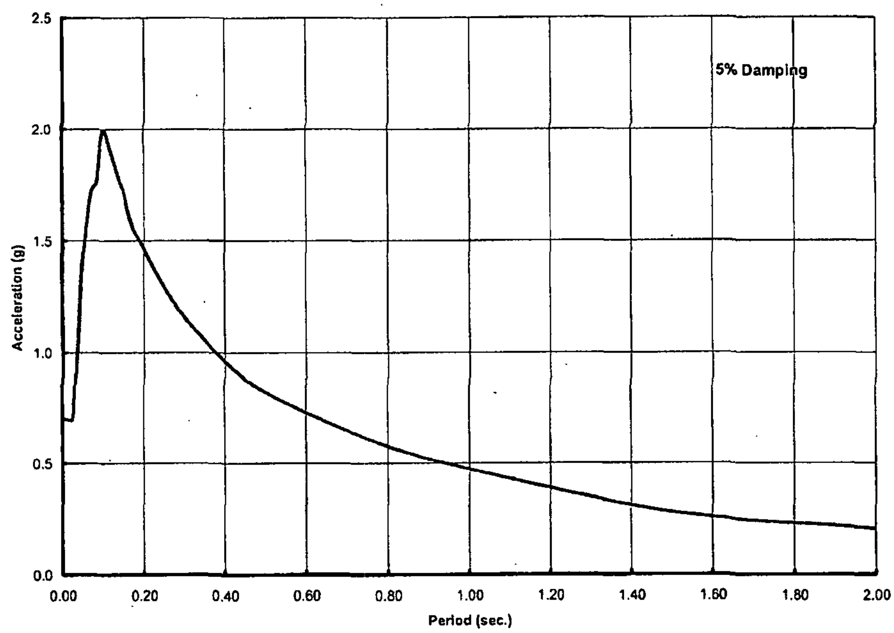


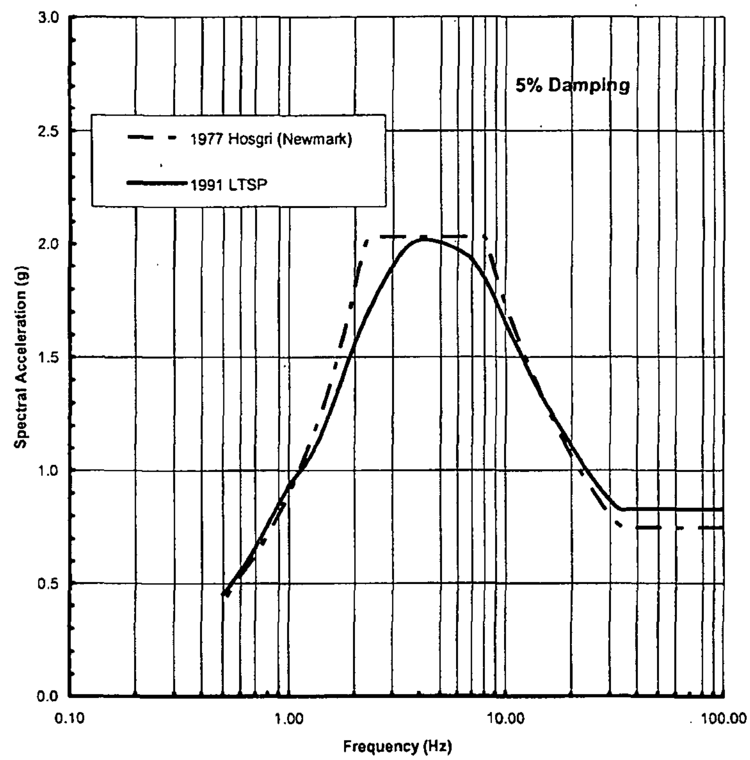
Notes:
1. This figure is based on Reference 42, Figure 2.4.

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-33 FREE FIELD SPECTRA HORIZONTAL 1991 LTSP (84TH PERCENTILE NON-EXCEEDANCE) AS MODIFIED PER SSER-34



Notes:
1. This figure is based on Reference 42, Figure 2.5.

FSAR UPDATE
UNITS 1 AND 2 **DIABLO CANYON SITE**
FIGURE 2.5-34 FREE FIELD SPECTRA VERTICAL 1991 LTSP (84TH PERCENTILE NON-EXCEEDANCE) AS MODIFIED PER SSER-34



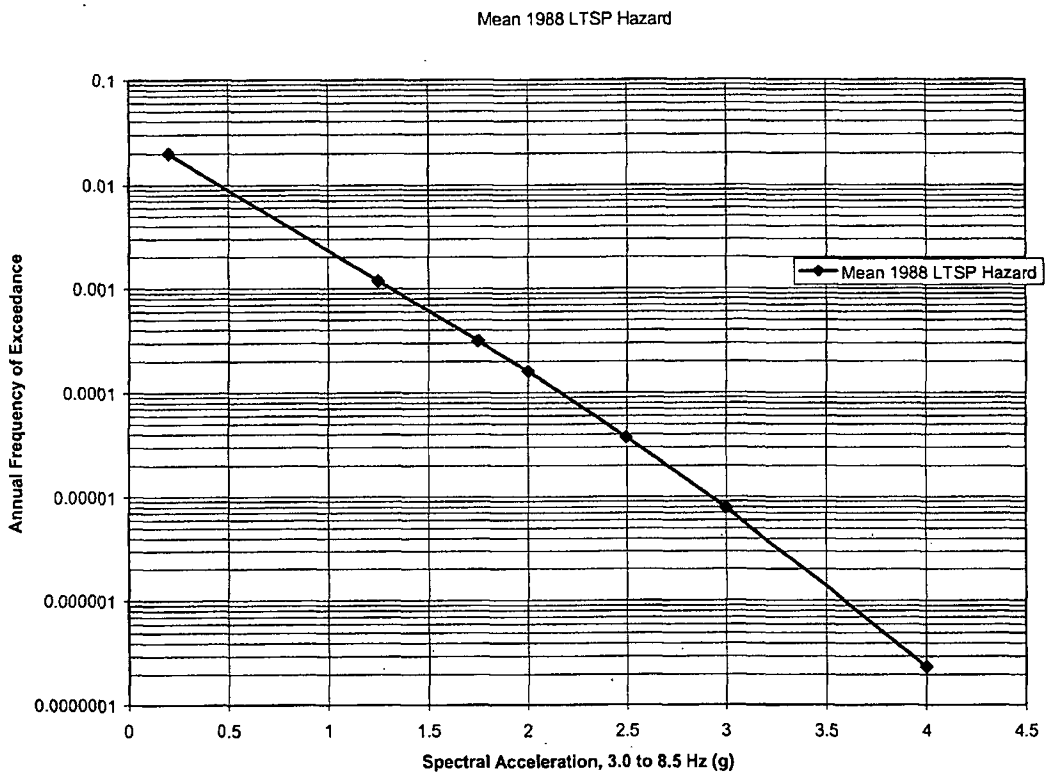
Notes:

1. This figure is based on Reference 40, Figure 7-2, but the LTSP response spectrum has been adjusted in accordance with Reference 42, Figure 2.5

2. This figure is for comparison purposes only and shall not be used for design

3. Legend:
   - 1977 Hosgri (Newmark) corresponds to the spectrum shown in Figure 2.5-30
   - 1991 LTSP corresponds to the spectrum shown in Figure 2.5-33
   -

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-35 FREE FIELD SPECTRA - HORIZONTAL LTSP (PG&E 1988) GROUND MOTION VS. HOSGRI (NEWMARK 1977)



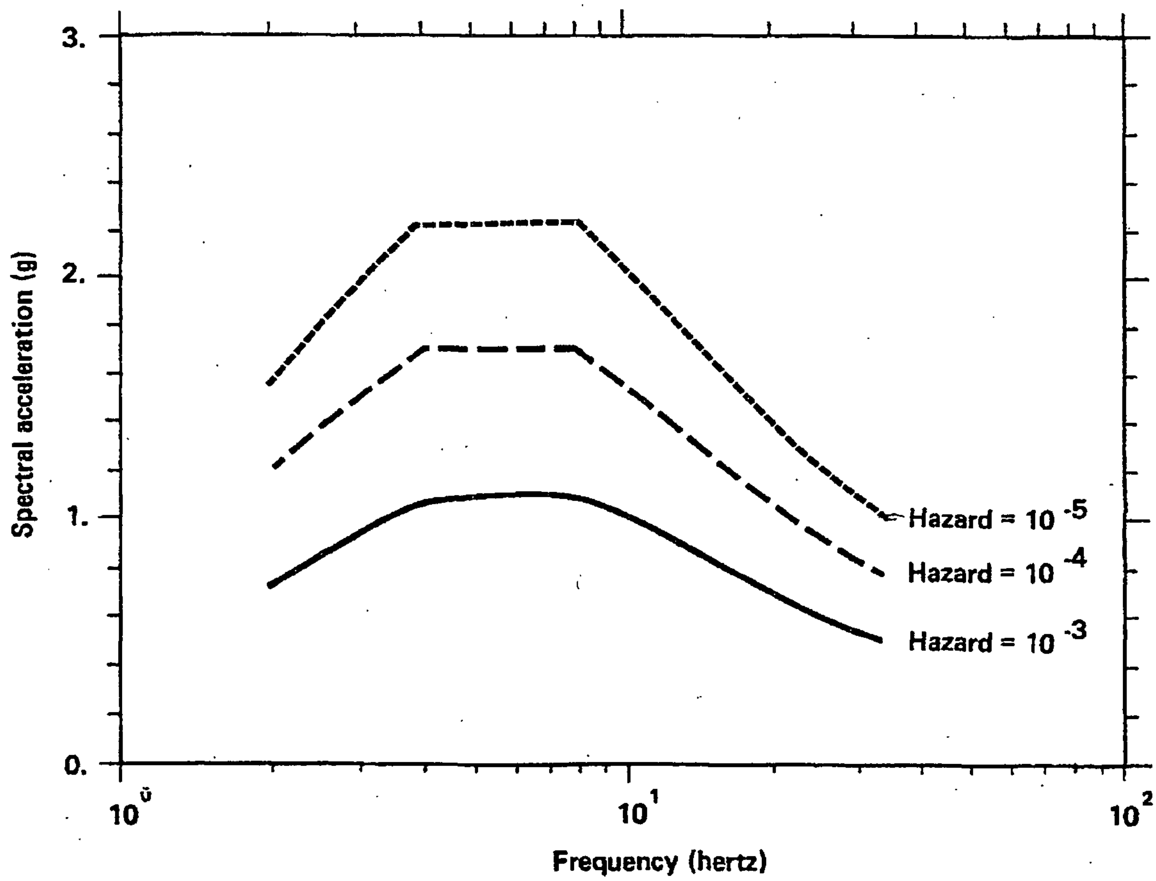
Notes:

1. This figure is based on Reference 40, Figures 6-6 and 6-8.

2. The seismic hazard curve defined in Reference 40 (1988 LTSP) was not affected by the adjustments to the LTSP ground motion response spectra described in Reference 42. Therefore, a seismic hazard curve is not defined for the 1991 LTSP ground motion.

3. This figure is for information only and should not be used for the evaluation of the DCPP seismic probabilistic risk assessment.

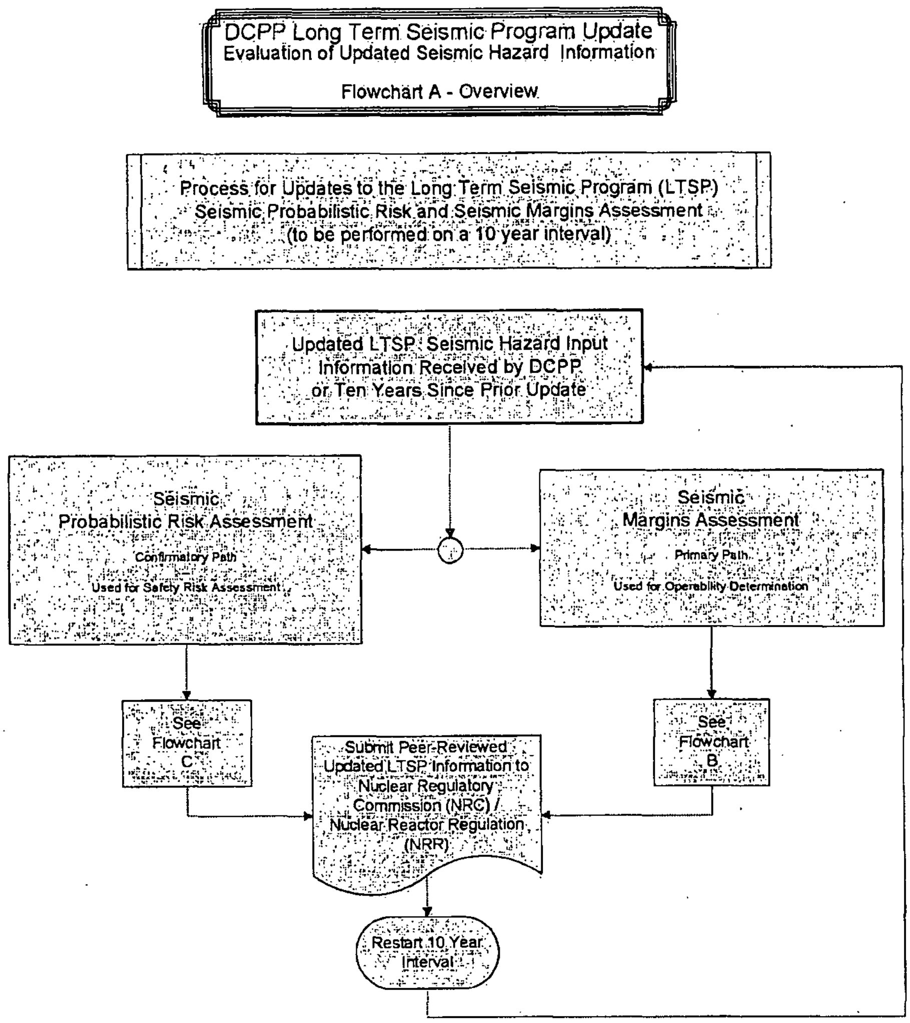
FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-36 1988 LTSP SEISMIC HAZARD CURVE



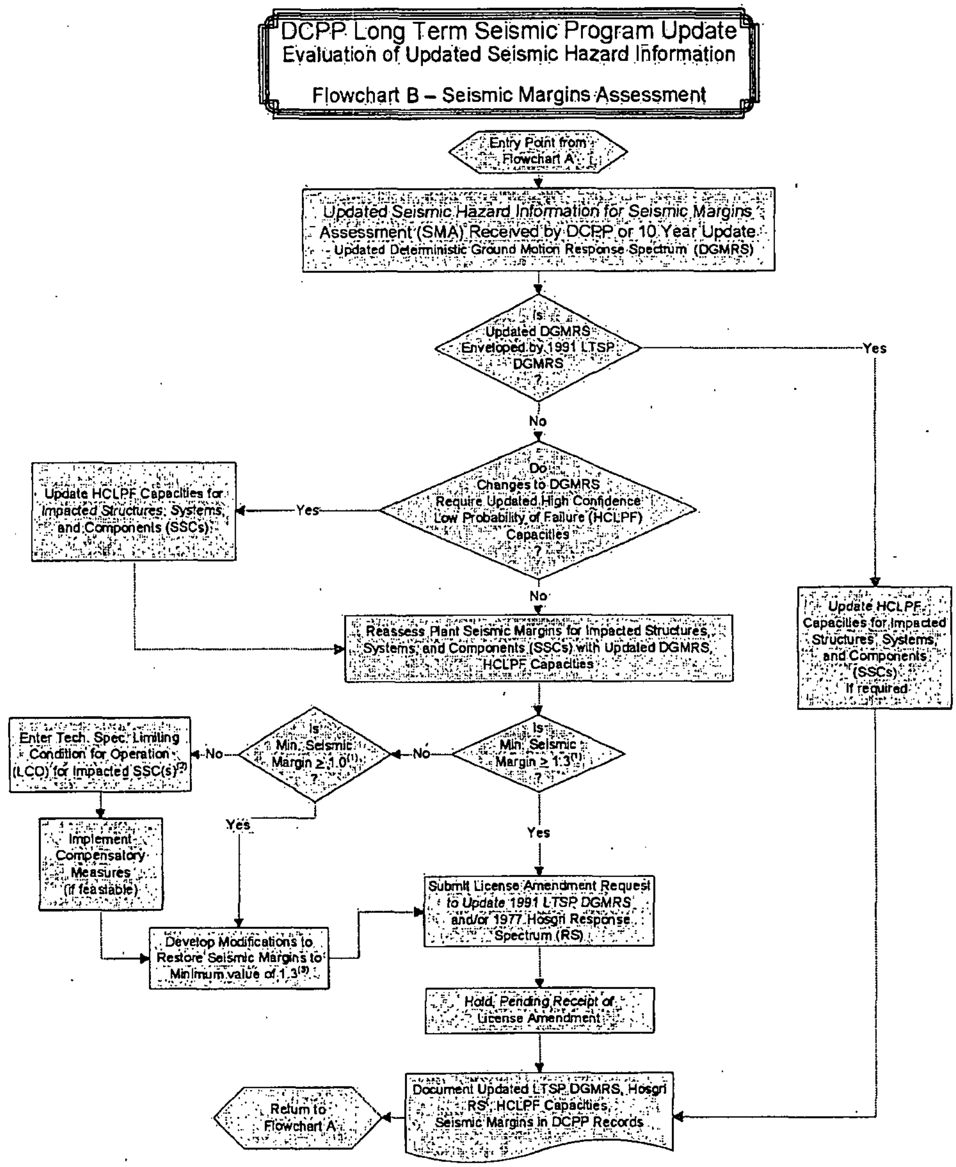
Notes:
1. This figure is based on Reference 40, Figure 6-9.

2. This figure is for information only and should not be used for the evaluation of the DCPP seismic probabilistic risk assessment.

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-37 1991 LTSP UNIFORM HAZARD SPECTRUM



FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-38 FLOWCHART FOR EVALUATION OF UPDATED LTSP GROUND MOTION (SHEET 1 OF 3)



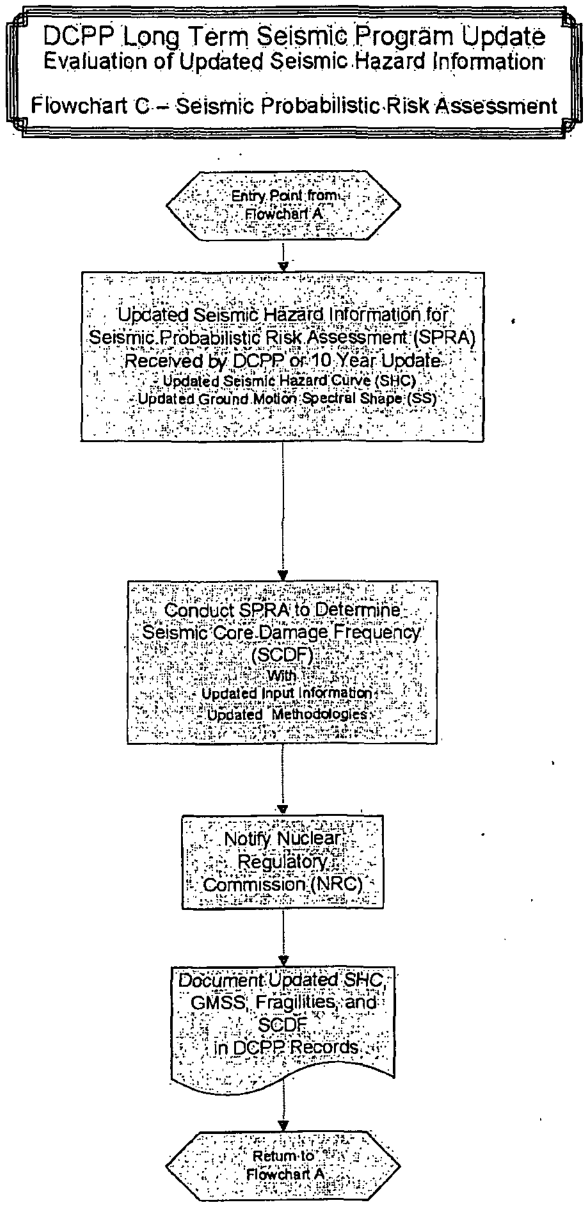
Notes:

1) Or greater than or equal to the approved seismic margin exceptions for certain SSCs discussed in FSARU Section 2.5.6.2.1.1.
2) Unless the SSC is one of the approved seismic margin exceptions below 1.0 discussed in FSARU Section 2.5.6.2.1.1.
3) Or to achieve the minimum approved seismic margin exception discussed in FSARU Section 2.5.6.2.1.1.

FSAR UPDATE

UNITS 1 AND 2
DIABLO CANYON SITE

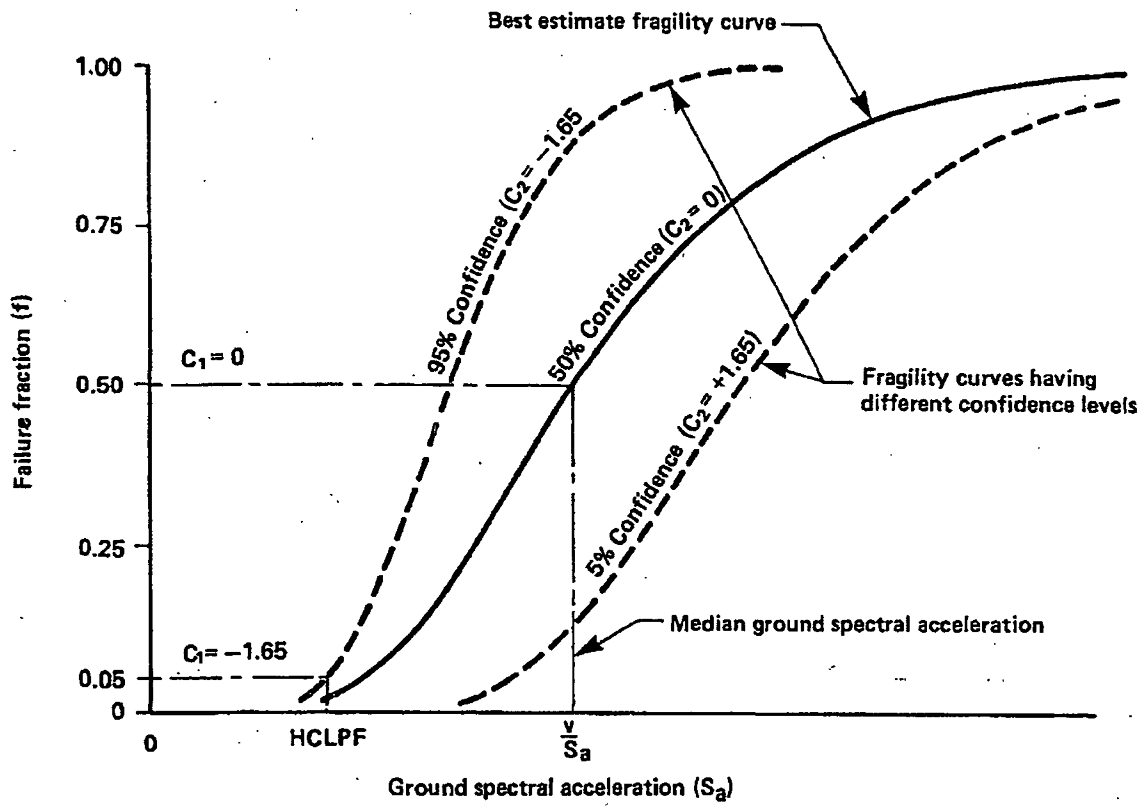
FIGURE 2.5-38
FLOWCHART FOR EVALUATION
OF
UPDATED LTSP GROUND MOTION
(SHEET 2 OF 3)



FSAR UPDATE

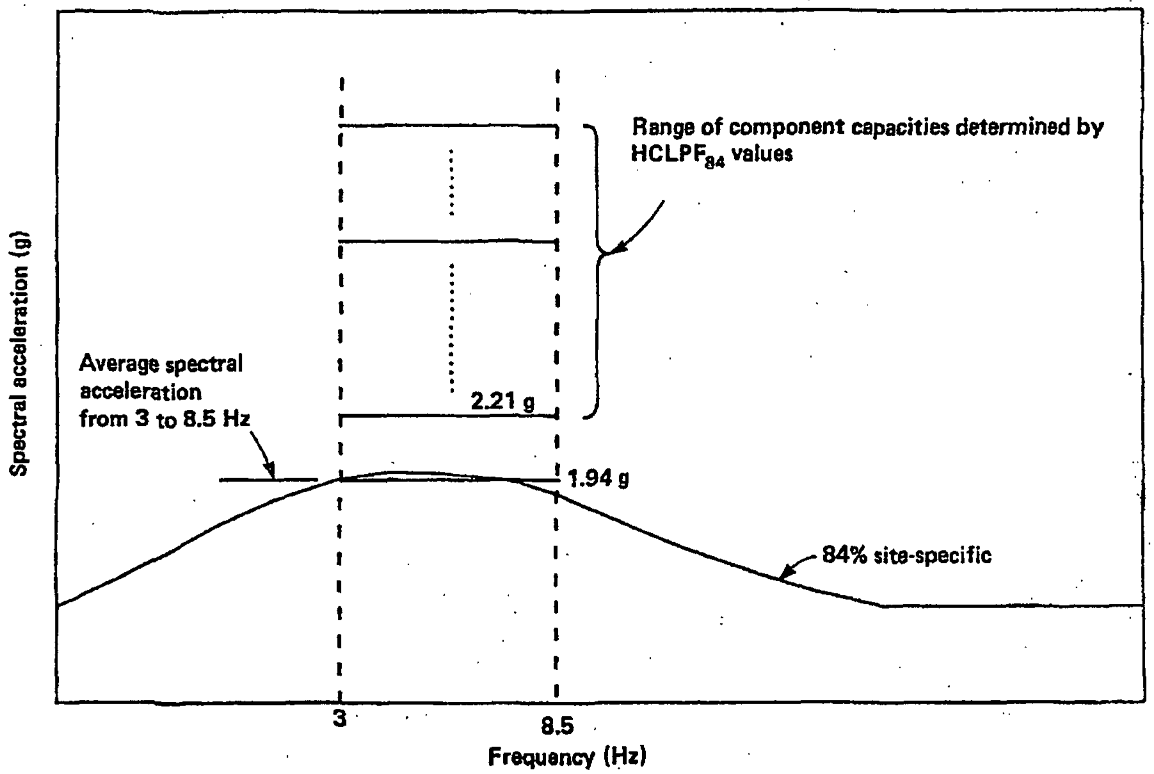
UNITS 1 AND 2
DIABLO CANYON SITE

FIGURE 2.5-38
FLOWCHART FOR EVALUATION
OF
UPDATED LTSP GROUND MOTION
(SHEET 3 OF 3)



Notes:
1. This figure is based on Reference 40, Figure 6-10.
2. This figure is for information only and should not be used for the evaluation of the DCPP seismic probabilistic risk assessment.

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-39 1991 LTSP FRAGILITY CURVE REPRESENTATION



Notes:
1. This figure is based on Reference 40, Figure 7-40.

2. This figure is for information only and should not be used for the evaluation of the DCPP seismic margins.

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 2.5-40 SCHEMATIC ILLUSTRATION FOR THE DETERMINATION OF SEISMIC MARGINS

3.1.2.2 Criterion 2 - Performance Standards (Category A)

Those systems and components of reactor facilities that are essential to the prevention of accidents which could affect the public health and safety, or to mitigation of their consequences, shall be designed, fabricated, and erected to performance standards that will enable the facility to withstand, without loss of the capability to protect the public, the additional forces that might be imposed by natural phenomena such as earthquakes, tornadoes, flooding conditions, winds, ice, and other local site effects. The design bases so established shall reflect (a) appropriate consideration of the most severe of these natural phenomena that have been recorded for the site and the surrounding area, and (b) an appropriate margin for withstanding forces greater than those recorded to reflect uncertainties about the historical data and their suitability as a basis for design.

Discussion

All systems and components designated Design Class I are designed so that there is no loss of function for ground acceleration associated with the ~~two times the design earthquake~~Double Design earthquake (DDE) and the Hosgri earthquake (HE), acting in the horizontal and vertical directions simultaneously. The ~~ESF is~~engineered safety features are included in the above. The working stresses for Class I items are kept within code allowable values for the Design Earthquake (DE). Seismic classification and seismic design criteria are discussed in Sections 3.2 and 3.7 through 3.10, respectively. Similarly, measures are taken in the plant design to protect against possible effects of tsunamis, lightning storms, strong winds, and other natural phenomena.

The site characteristics are discussed in Chapter 2. Wind design criteria and flood design criteria are found in Sections 3.3 and 3.4, respectively.

3.2 CLASSIFICATION OF STRUCTURES, SYSTEMS, AND COMPONENTS

This section provides a guide to the classification of the DCPP structures, systems, and components (SSCs).

Criterion 1 of the July 1967 GDC requires that systems and components essential to the prevention of accidents be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed. This section describes how Criterion 1 has been implemented by relating the classifications of SSCs to the various criteria, codes, regulations, and standards that dictate specific quality requirements.

In this regard, it is recognized that during the design and construction of DCPP Units 1 and 2, significant industry and regulatory changes were made in establishing common methods of classification, e.g., ANSI N18.2 (Reference 1)~~[1]~~, SG 26 (Reference 2)~~[2]~~, SG 29 (Reference 3)~~[3]~~, and NRC Regulatory Guide (RG) 1.143 (Reference 6)~~[6]~~. These methods all differ slightly in detail from those used for the DCPP, but the form and intent of all are equivalent, as will be shown in ~~the following discussion of: (a) the seismic classification of SSCs, and (b) the system quality group classification of pressure-containing components of fluid systems~~ Sections 3.2.1 through 3.2.5.

Classifications of instruments and controls and the associated requirements ~~for them~~ are discussed in Section 7.1.

The general applicability and requirements of the DCPP classification systems are provided in Tables 3.2-1 and 3.2-2. The classifications of specific SSCs are provided in the DCPP Q-List~~[8]~~ (Reference 8). The DCPP Q-List is controlled by a written PG&E procedure. The procedure requires that all non-editorial changes to the contents of the Q-List be reviewed pursuant to the requirements of 10 CFR 50.59. Access to the Q-List is available through hard copy or electronically at PG&E.

3.2.1 SEISMIC CLASSIFICATION

Criterion 2 of the July 1967 GDC, and Appendix A to 10 CFR 100, Seismic and Geologic Siting Criteria for Nuclear Power Plants (Reference 11)~~[11]~~, require that nuclear power plant SSCs important to safety be designed to withstand the effects of earthquakes. Specifically, Appendix A to 10 CFR 100 requires that all nuclear power plants be designed for the following two earthquakes:

(1) ~~so that, if t~~The safe shutdown earthquake (SSE) is that earthquake which is based on an evaluation of the maximum earthquake potential considering the regional and local geology and seismology and specific characteristic of local subsurface material. It is the earthquake which produces the maximum vibratory ground motion for which certain structures, systems, and components are designed to remain functional. These structures, systems, and components are those necessary to

assure:~~occurs, all structures and components important to safety remain functional. Plant features important to safety are those necessary to ensure~~

(a) the integrity of the reactor coolant pressure boundary,

(b) the capability to shut down the reactor and maintain it in a safe shutdown condition, or

(c) the capability to prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the guideline exposures of 10 CFR 100.

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

Since DCPP design and construction had progressed substantially prior to the issuance of Appendix A to 10 CFR 100, different terminology is used for the design basis earthquakes. The following equivalencies have been established between the DCPP design basis earthquakes and those defined in Appendix A to 10 CFR 100:

~~The SSE of Appendix A to 10 CFR 100 is equivalent to the DCPP double design earthquake (DDE) (see References 9 and 10 for final resolution of issues raised in Supplemental Safety Evaluation Reports 7, 8, and 31 relative to the SSE). Similarly, the operating basis earthquake (OBE) of Appendix A to 10 CFR 100 is equivalent to the DCPP DE.~~

(1) The DCPP design earthquake (DE), as described in Section 3.7.1.1.1, is the equivalent of the event that was later defined as the OBE in Appendix A to 10 CFR 100 (see SSER No. 7).

(2) The DCPP double design earthquake (DDE), as described in Section 3.7.1.1.2, was originally the equivalent of the event that was later defined as the SSE in Appendix A to 10 CFR 100, prior to the discovery of the Hosgri fault.

(3) The DCPP 1977 Hosgri earthquake (HE), as described in Section 3.7.1.1.3, replaced the DDE as the maximum vibratory ground acceleration that could occur at the site, comparable to the SSE.

~~DCPP's capability to withstand a postulated Richter magnitude 7.5 earthquake centered along an offshore zone of geologic faulting known as the "Hosgri Fault" has been reviewed.~~ Guidance for determining the seismic classification of SSCs is provided in SG 29 (Reference 3)~~(3)~~, specifically:

(1) Those SSCs required ~~designed~~ to remain functional in the event of an SSE ~~is provided in SG 29. These plant features~~, including their foundations and supports, are designated as Seismic Category I ~~in SG 29~~.

(2) Those SSCs not required to remain functional in the event of an SSE are designated as Non-Seismic Category I.

~~DCPP SSCs, and their seismic design classifications comply with the intent of SG 29. However, s~~Since DCPP design and construction had progressed substantially prior to the issuance of SG 29, different terminology is ~~often~~ used for the classification of SSCs. The seismic design classification of SSCs is not explicitly identified, instead it is determined based on the combination of several DCPP-specific classification systems:

(1) Design Classification (see Section 3.2.2)

(2) Quality Assurance Classification (see Section 3.2.3)

(3) Piping Symbol (see Section 3.2.4)

(4) Quality Group/Code Classification (see Section 3.2.5)

(5) Instrument Classification (see Section 7.1)

3.2.2 DESIGN CLASSIFICATION

The design classification system for SSCs is defined in Table 3.2-1. The design classifications of specific SSCs are provided in the DCPP Q-List. The relationships between the DCPP design classifications and the SG 29 seismic categories are as follows:

(1) Design Class I: Plant features important to safety, including plant features required to assure (1) the integrity of the reactor coolant pressure boundary, (2) the capability to shut down the reactor and maintain it in a safe shutdown condition, or (3) the capability to prevent or mitigate the consequences of accidents which could result in potential offsite exposures comparable to the guideline exposures of 10 CFR 100.

Plant features designated as Design Class I ~~that~~ correspond to a subset of the Seismic Category I features, as identified in SG 29 (the remaining Seismic Category I features are designated as either Design Class II or III, as decribed below). The seismic design requirements for the Design

Class I plant features are dependent on whether the design basis function of the equipment ~~determines whether it is qualified for~~ is active or passive.

Passive components are not required to perform any function during an earthquake. Passive components ~~Design Class I plant features~~ are designed to maintain their structural integrity when subjected to ~~in the event of both~~ the DE,~~/~~DDE, and 1977 HE. They are not required ~~may or may not~~ to be designed to remain functional during an earthquake. ~~operable for the DE/DDE or HE; the function for a DE/DDE and/or an HE~~.

Active components must be able to perform a function (ability to operate and/or change state), during an earthquake. Active components are designed to:

(a) maintain their structural integrity when subjected to ~~in the event of both~~ the DE,~~/~~DDE, and HE and

(b) remain functional during one or more of the design basis earthquakes ~~that they are required to withstand:~~ (the DE ~~(equivalent to the OBE of SG 29)~~, the DDE ~~(equivalent to the SSE of SG 29)~~, and/or the ~~postulated Hosgri earthquake (~~1977 HE). The earthquakes applicable to specific components are defined in the Q-List.

The following Design Class I SSCs, including their foundations and supports, are designed to maintain structural integrity and to remain functional when subjected to a DDE or HE, and are subject to the requirements of the Quality Assurance Program (see Section 3.2.3):

(a) The reactor coolant pressure boundary

(b) The reactor core and reactor vessel internals

(c) Systems [see Note (i) at the end of this list] or portions of systems that are required for emergency core cooling, post-accident containment heat removal, or post-accident containment atmosphere cleanup [see Note (iv) at the end of this list]

(d) Systems or portions of systems that are required for reactor shutdown and residual heat removal

(e) Those portions of the main steam, feedwater, and steam generator blowdown systems extending from and including the secondary side of the steam generators up to and including the outermost containment isolation valves, and connected piping up to and including the first valve (including a safety or relief valve) that is either normally closed or capable of automatic closure during all modes of normal reactor operation [see Note (iv) at the end of this]

(f) Auxiliary saltwater, component cooling water, and auxiliary feedwater systems or portions of these systems that are required for emergency core cooling, post-accident containment heat removal, post-accident containment atmosphere cleanup, and residual heat removal

(g) Component cooling water system and seal water systems, or portions of these systems that are required for functioning of other systems or components important to safety

(h) Those portions of systems (other than the radioactive waste management systems) that contain or may contain radioactive material and whose postulated failure could result in conservatively calculated potential offsite exposures in excess of 0.5 rem whole body (or its equivalent to parts of the body) at the site boundary or beyond

(i) Systems or portions of systems that are required to supply fuel for emergency equipment

(j) Systems or portions of systems that are required for (a) post accident monitoring of RG 1.97 Category 1 variables and (b) actuation of systems important to safety

(k) The protection system [see Note (ii) at the end of this list]

(l) The spent fuel storage pool structure, including the spent fuel racks.

(m) The reactivity control systems, i.e., control rods, control rod drives, and boron injection system, that are required to achieve safe shutdown of the plant

(n) The control room, including its associated vital equipment and life support systems, and any structures or equipment inside or outside of the control room whose failure could result in incapacitating injury to the operators

(o) Reactor containment structure, including penetrations [see Note (iv) at the end of this list]

(p) Systems or portions of systems that are required to provide heating, ventilating, and/or air conditioning for safety-related equipment/areas

(q) Portions of the onsite electric power system, including the onsite electric power sources, that provide the emergency electric power needed for functioning of plant features included in Items (a) through (p) above

(r) Portions of the spent fuel pool cooling system used to remove spent fuel decay heat from the spent fuel pool; and portions of the refueling water purification system used to recirculate and cleanup the contents of the refueling water storage tank

Notes:

(i) A system boundary includes those portions of the system required to accomplish the specified safety function and connected piping up to and including the first valve (including a safety or relief valve) that is either normally closed or capable of automatic closure when the safety function is required.

(ii) For purposes of these criteria, the protection system encompasses all electrical and mechanical devices and circuitry (from sensors to actuation devices input terminals) involved in generating those signals associated with the protective function. These signals include those that actuate reactor trip and, in the event of a serious reactor accident, that actuate ESFs such as containment isolation, safety injection, pressure reduction, and air cleaning.

(iii) ~~SSCs that form interfaces between Design Class I and Design Class II or III features are designed to Design Class I requirements.~~Not Used.

(iv) Certain valves in these systems that are used for accident mitigation only, and do not support safe shutdown following an HE, were qualified for active function for an HE to provide increased conservatism in accordance with Reference 7.

~~All plant features designated as Design Class I are also Seismic Category I.~~

(2) Design Class II: Plant features ~~SSCs~~ important to reactor operation but not essential to ~~safe shutdown and isolation of the reactor, and failure of which would not result in the release of substantial amounts of radioactivity, are classified as Design Class II.~~ safety, including plant features not required to be Design Class I.

In general, plant features designated as Design Class II correspond to ~~SSCs not identified as~~ Non-Seismic Category I features, as identified in SG 29 and are not designed to withstand the effects of the design basis earthquakes. However, based on specific licensing requirements, certain

Design Class II plant features, as indentified in the Q-List, have been designed to withstand one or more of the design basis earthquakes and form a subset of Seismic Category I features. These licensing requirements are addressed in Sections 3.2.3, 3.2.4, and/or 3.2.5, as applicable.~~, are referred to by the guide as Nonseismic Category I features. Under the DCPP classification system, Design Class II features may or may not be Seismic Category I.~~ Seismically qualified Design Class II features include, but are not limited to, the following:

(a) Architectural Platforms supporting Design Class I components

(b) Spent Fuel Pool Liner

(c) Turbine Building

(d) Turbine Pedestals

(e) Intake Structure

(f) Pipe Vaults at Outdoor Water Storage Tanks

(g) Reactor Coolant Pump Motors

(h) Reactor Coolant Pump Oil Collection Tank and Pans

(i) Reactor Vessel Support Coolers

(j) Firewater Pumps

(k) Containment Penetration Overcurrent Protection

(l) Main and Remote Annunciator Cabinets

(m) Seismic Monitoring System

(n) Containment Fan Cooler Ductwork and Annular Ring Duct

(o) Post-LOCA Sampling Room Ventilation System

(p) Technical Support Center Ventilation System

(3) Design Class III: Plant features~~SSCs~~ not related to reactor operation or safety.~~ are classified as Design Class III.~~

In general, plant features designated as Design Class III correspond to Non-Seismic Category I, as identified in SG 29 and are not designed to withstand the effects of the design basis earthquakes. However, based on specific licensing requirements, certain Design Class III plant features, as indentified in the Q-List, have been designed to withstand one or more of the design basis earthquakes. These licensing requirements are addressed in Sections 3.2.3, 3.2.4, and/or 3.2.5, as applicable. Seismically qualified Design Class III features include, but are not limited to, the following:

(a) Containment Dome Service Crane

(b) New Fuel Elevator

Power and auxiliary service piping systems (as defined in ANSI B31.1, Paragraph 100.1), which might otherwise be considered as Design Class III, are classified as Design Class II (i.e., Design Class III is not used for power and auxiliary service piping systems).

3.2.3 QUALITY ASSURANCE CLASSIFICATION

~~In addition,~~ Appendix B to 10 CFR 50, Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants (Reference 13)~~[13]~~, requires that SSCs important to safety be designed and constructed in accordance with the quality assurance requirements described in Appendix B. Therefore, as described in Chapter 17, the requirements of the DCPP Quality Assurance (QA) Program apply to all SSCs classified as Design Class I. This ensures that plant features important to safety have met the requirements of Appendix B. Specific quality assurance requirements may also be applied to selected Design Class II and III features, described as the "graded" QA Program.

~~The general applicability and requirements of the design class, quality/code class classification, and seismic category are provided in Tables 3.2-1 and 3.2-2.~~

~~The classifications of specific SSCs are provided in the DCPP Q-List (see Reference 8). The DCPP Q-List is controlled by a written PG&E procedure. The procedure requires that all non-editorial changes to the contents of the Q-List be reviewed pursuant to the requirements of 10 CFR 50.59. Access to the Q-List is available through hard copy or electronically at PG&E.~~

The QA classification of individual SSCs is identified in the Q-List. The following QA classes are used at DCPP:

QA Class	Description
Q	Equipment and structures to which the QA provisions of Appendix B to 10 CFR 50 apply for design, procurement, and construction.
"Blank"	Design Class II or III equipment that is not subject to nuclear quality assurance requirements.

QA Class	Description
R	Those radioactive waste management items which require application of graded quality assurance requirements including Regulatory Guide 1.143 (Reference 14). See Section 2.2.2 of the Q-List for further details.
G	Those portions of the fire protection systems and emergency lighting and communication equipment which require application of a quality program as described in Appendix A to NRC Branch Technical Position APCSB 9.5-1 (Reference 15). See Section 2.2.2 of the Q-List for further details.
S	Design Class II and III equipment that requires seismic qualification to satisfy license or FSAR Update commitments or to assure the functionality of Design Class I components. This includes, but is not limited to equipment in the following categories: • SSCs required to achieve Mode 5 for both units following a 1977 HE, assuming a single failure and the loss of offsite power (Section 5.1 and Appendix J of the Hosgri Report (Reference 12), Section 3.2 of SSER-7 (Reference 17) and Section 3.2 of SSER-8 (Reference 18)) • SSCs associated with electrical isolation in certain 120 VAC power circuits, • SSCs required for compliance with the requirements of RG 1.97. • SSCs associated with certain inputs to the Solid State Protection System.
T	Regulatory Guide 1.97 (Reference 16) Category 2 and 3 instrumentation which requires application of a graded Quality Assurance Program. (Note: Other Category 2 and 3 instrumentation which is within the Environmental Qualification (EQ) Program, is part of the pressure boundary of a Design Class I System, or is treated as a Class 1E electrical devices, is QA Class Q.)

3.2.4 PIPING CLASSIFICATION SYMBOLS

The piping schematic drawings ~~are illustrated in~~ (see Figures 3.2-1 through 3.2-27) employ a system of symbols ~~The piping symbol system that appears on all piping schematics and drawings~~ to indicate piping fabrication, erection, and test criteria. Their ~~can be~~ correlationed to the design class (Section 3.2.2) and quality group/code classes (Section 3.2.5) is as follows:

Piping Schematic Correlation

Piping Symbol	Design Class	Quality Group/Code Class[c]
A	I	I

Piping Schematic Correlation

Piping Symbol	Design Class	Quality Group/Code Class[c]
B	I	II
@[a]	I/II	II/None
C	I	III
D	I	III
E	II	None
F	II	None
G	II	None
G1	II	None
H	II	None
J	I	III
-[b]	I	Not Applicable
-[b]	II or III	None

Notes:

[a] The symbol '@' is referred to in the FSAR Update and the Q-List. However, this symbol is not used on the piping schematics for Code Class designation; the line is bubbled (i.e., -0-0-) and the notes describe the applicable code(s).
[b] For HVAC system ductwork symbols, see Figures 3.2-1A and 3.2-2A.
[c] See Section 3.2.5 for Quality Group/Code Classification system

3.2.~~2~~5 SYSTEM QUALITY GROUP/CODE CLASSIFICATIONS FOR FLUID SYSTEMS AND FLUID SYSTEM COMPONENTS

GDC 1 requires that systems and components essential to the prevention of accidents be designed, fabricated, erected, and tested to quality standards commensurate with the importance of the safety functions to be performed. This section describes the quality classification system that has been used to implement quality standards that satisfy Criterion 1 for DCPP fluid systems and fluid system components. The discussion also shows the relationship of this classification system to fluid system and fluid system components classification systems in ANSI N18.2, Nuclear Safety Criteria for the Design of Stationary Pressurized Water Reactor Plants (Reference 1)~~[1]~~, and SG 26.

DCPP SSCs are classified as Design Class I, II, or III (Section 3.2.2). Design Class I ~~is Seismic Category I and~~ fluid systems and fluid system components are~~is~~ further categorized as PG&E Quality Group/Code Class I, II, or III.

Design Classes II or III fluid systems and fluid system components ~~are usually Nonseismic Category I and~~ have no PG&E quality group/code class designation.

Specific requirements as dictated by the quality standards applicable to the respective commercial (ASME, ANSI, or ASA) code classes are also applicable. However, some Design Class II and III components have been seismically designed, e.g., items in the Seismically Induced Systems Interaction Program (Section 3.7.3.13), specific components required for post-HE shutdown, CCW header C components, and items that were designed for the DE pursuant to RG 1.143 (Reference 14). For this reason, there is not a direct correlation between design class and seismic category (except that all Design Class I features are Seismic Category I). In addition, the design class ~~ification of Seismic Category I~~ does not indicate which of the three design basis earthquakes a feature has been qualified for, nor whether that qualification is for passive or active function (except that all electrical Class 1E and Instrument Class IA components are qualified to remain operable for all three design basis earthquakes). The design basis function of the equipment determines the type of seismic qualification required. These classifications and their relationships are illustrated in Table 3.2-2 and discussed below.

3.2.~~2~~5.1 Design Class I, Quality Group/Code Class I Fluid Systems and Fluid System Components

10 CFR 50.55a requires that certain components of the reactor coolant pressure boundary be designed, fabricated, erected, and tested in accordance with the requirements for Class A[a] components of Section III of the ASME Boiler and Pressure Vessel Code, or the most recently available industry codes and standards. Code Class I has been applied to those components of the reactor coolant pressure boundary and implements the quality standards that satisfy the requirements of Section 50.55a, 10 CFR 50. DCPP Code Class I components of the reactor coolant pressure boundary are listed in the DCPP Q-List (Reference 8), along with the industry codes and standards used for their design, fabrication, erection, and test. The Code Class I classification includes the components of the reactor coolant pressure boundary identified as Safety Class I in ANSI N18.2 and Quality Group A in SG 26.

3.2.~~2~~5.2 Design Class I, Quality Group/Code Class II Fluid Systems and Fluid System Components

Generally, Code Class II has been applied to include fluid systems and fluid system components that are either:

(1) Part of the reactor coolant boundary, but excluded from Code Class I requirements by Section 50.55a of 10 CFR 50

(2) Not part of the reactor coolant pressure boundary, but part of:

[a] The 1971 edition of the ASME Boiler and Pressure Vessel Code, Section III, Nuclear Power Plant Components, uses the term Class I in lieu of Class A.

(a) Systems or portions of systems[b] that are required for emergency core cooling, postaccident containment heat removal, or postaccident containment atmosphere cleanup

(b) Systems or portions of systems that are required for reactor shutdown and residual heat removal

(c) Those portions of the main steam, feedwater, and steam generator blowdown systems extending from and including the secondary side of steam generators up to and including the outermost containment isolation valves, and connected piping up to and including the first valve (including a safety or relief valve) that are either normally closed or capable of automatic closure during all modes of normal reactor operation

(d) Systems or portions of systems that are connected to the reactor coolant pressure boundary and are not capable of being isolated from the boundary during all modes of normal reactor operation by two valves, each of which is either normally closed or capable of automatic closure

Code Class II fluid systems and fluid system components are listed in the DCPP Q-List (see Reference 8), along with the industry codes and standards used for their design, fabrication, erection, and testing.

3.2.~~2~~5.3 Design Class I, Quality Group/Code Class III Fluid Systems and Fluid System Components

Generally, Code Class III has been applied to include fluid systems and fluid system components not part of the reactor coolant pressure boundary, nor included in Code Class II, but part of:

(1) Auxiliary saltwater, component cooling water, and auxiliary feedwater systems, or portions of these systems that are required for (a) emergency core cooling, (b) postaccident containment heat removal, (c) postaccident containment atmosphere cleanup, and (d) residual heat removal from the reactor

(2) Systems or portions of systems that are connected to the reactor coolant pressure boundary and are capable of being isolated from the boundary during all modes of normal reactor operation by two valves, each of which is either normally closed or capable of automatic closure

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(b) The system boundary includes those portions of the system required to accomplish the specified safety function and connected piping up to and including the first valve (including a safety or relief valve) that is either normally closed or capable of automatic closure when the safety function is required.

(3) Those portions of systems other than radioactive waste management systems that contain or may contain radioactive material, and whose postulated failure could result in conservatively calculated potential offsite exposures in excess of 0.5 rem whole body (or its equivalent to parts of the body) at the site boundary or beyond

(4) Component cooling water system and seal water systems, or portions of these systems, that are required for functioning of other systems or components important to safety

(5) Portions of the spent fuel pool cooling system required for spent fuel cooling, and the refueling water purification system whose postulated failure could result in a loss of refueling water storage tank inventory

Code Class III fluid systems and fluid system components are listed in the DCPP Q-List (see Reference 8), along with the industry codes and standards used for their design, fabrication, erection, and testing.

3.2.~~2~~5.4 Other Fluid Systems and Fluid System Components

Fluid systems and fluid system components that are not part of the reactor coolant pressure boundary, not essential to shut down the reactor and maintain it in a safe condition, and not essential to prevent or mitigate the consequences of accidents that could result in potential offsite exposures comparable to the guideline exposures of 10 CFR 100, are not included in the Design Class I classification.

These other systems and components are classified as Design Class II or III and are listed in the DCPP Q-List (see Reference 8), along with the industry codes and standards used for their design, fabrication, erection, and testing. They comprise a design class, but have not been assigned a code class. Design Class II includes the fluid systems and fluid system components identified as Quality Group D in SG 26 and as radioactive waste management system in RG 1.143, i.e., those fluid systems and fluid system components that contain or may contain radioactive material, but whose failure would not result in calculated potential exposures in excess of 0.5 rem whole body (or its equivalent to parts of the body) at the site boundary. These fluid systems and fluid system components are in conformance with the accepted industry codes and standards in effect during the design and construction of DCPP. If they were designed and constructed to codes and standards outside of the requirements of SG 26 or RG 1.143, additional quality standards have normally been applied so that the intent has been met.

3.2.~~2~~5.5 Summary of System Quality Group/Code Classifications

Table 3.2-2 summarizes the design and quality group classifications applied to the DCPP ~~SSCs~~ fluid systems and fluid system components, and their relationships to the other methods of classification.

Generally, codes and standards were applied prior to issuance of the latest codes and standards, such as the 1971 edition of the ASME Boiler and Pressure Vessel Code, Section III, Nuclear Power Plant Components. In some cases, fluid systems and components were designed and built to codes and standards outside the requirements of SG 26, ANSI N18.2, and RG 1.143 definitions. The classification for those fluid systems and fluid system components that do not fall within the strict definition of SG 26, ANSI N18.2, and RG 1.143 were established prior to ANSI N18.2, SG 26, RG 1.143, and the issuance of revised industry codes and standards. For these fluid systems and fluid system components, the design specifications specified the accepted industry codes and standards in effect during the design and construction of DCPP.

While some portions of the fire protection system components are designated Design Class I, the system is not required to ensure the integrity of the reactor coolant pressure boundary or to shut down the reactor and maintain it in a safe shutdown condition. Fire protection features meet the requirements defined in BTP APCSB 9.5-1 (Reference 5) after 1979 and, where designated Design Class I, are designed to withstand the effects of an HE.

3.2.~~3~~6 REFERENCES

1. Nuclear Safety Criteria for the Design of Stationary Pressurized Water Reactor Plants. N18.2 American Nuclear Society, August 1970 Draft.

2. Quality Group Classifications and Standards for Water, Steam, and Radioactive Waste Containing Components of Nuclear Power Plants, SG 26, Atomic Energy Commission.

3. Seismic Design Classification, SG 29, US Atomic Energy Commission, June 7, 1972.

4. Spent Fuel Storage Facility Design Basis, RG 1.13, Nuclear Regulatory Commission.

5. Guidelines for Fire Protection for Nuclear Power Plants, BTP APCSP 9.5-1, Nuclear Regulatory Commission.

6. Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants, RG 1.143, Nuclear Regulatory Commission.

7. PG&E Letter to the NRC, "Inadequate Maintenance of Hosgri Report Commitments," DCL-92-198 (LER 1-92-015), September 11, 1992.

8. Classification of Structures, Systems, and Components for Diablo Canyon Power Plant Units 1 and 2 (Q-List), PG&E.

9. ~~Letter from NRC (L. F. Miller) to PG&E (G. M. Rueger), dated December 13, 1993, Subject: NRC Inspection of Diablo Canyon Units 1 and 2 (Report No. 50-275, 50-323/93-31) [pages 1 and 2].~~Deleted in revision xx.

10. ~~Letter from NRC (A. W. Beach) to PG&E (G. M. Rueger), dated August 15, 1994, Subject: NRC Inspection Report 50-275/94-18; 50-323/94-18 (Notice of Violation) [pages 14 and 15].~~Deleted in revision xx.

11. Seismic and Geologic Siting Criteria for Nuclear Power Plants, Appendix A to 10 CFR 100.

12. Seismic Evaluation for Postulated 7.5M Hosgri Earthquake, DCPP Units 1&2, PG&E, (Amendment Nos. 50, 53, 54, 56, 59, 60, 62, 64, 66, 68, 70, 72, 75, 76, 77, 79, 82, and 83 to the DCPP Final Safety Analysis Report dated September 28, 1973)

13. Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants, Appendix B to 10 CFR 50.

14. Design Guidance for Radioactive Waste Management Systems, Structures, and Components Installed in Light-Water-Cooled Nuclear Power Plants, RG 1.143, Nuclear Regulatory Commission, Revision 1 (October 1979)

15. Guidelines for Fire Protection for Nuclear Power Plants Docketed prior to July 1, 1976, NRC Branch Technical Position APCSB 9.5-1, Appendix A, (August 23, 1976)

16. Criteria for Accident Monitoring Instrumentation for Nuclear Power Plants, RG 1.97, Nuclear Regulatory Commission

17. Supplement No. 7 to the Safety Evaluation Report Related to the Operation of Diablo Canyon Nuclear Power Plant, Units 1 and 2, NUREG-0675, Nuclear Regulatory Commission, May 1978.

18. Supplement No. 8 to the Safety Evaluation Report Related to the Operation of Diablo Canyon Nuclear Power Plant, Units 1 and 2, NUREG-0675, Nuclear Regulatory Commission, November 1978.

3.2.~~7~~4 REFERENCE DRAWINGS

Figures representing controlled engineering drawings are incorporated by reference and are identified in Table 1.6-1. The contents of the drawings are controlled by DCPP procedures.

3.7 SEISMIC DESIGN

3.7.1 SEISMIC INPUT

This section describes the three design basis earthquakes, the Design Earthquake (DE), the Double Design Earthquake (DDE), and the postulated 7.5M Hosgri Earthquake (HE).

In addition to the above three earthquakes, in response to Unit 1 Operating License Condition No. 2.C.(7), PG&E conducted, as described in Sections 2.5.2.9.4 and 2.5.2.10.4 ~~below~~, a program to reevaluate the seismic design basis for DCPP. ~~On November 2, 1984, the NRC issued the DCPP Unit 1 Facility Operating License DPR-80. In License Condition 2.C(7) of DPR-80, the NRC stated, in part: "PG&E shall develop and implement a program to reevaluate the seismic design bases used for the Diablo Canyon Power Plant."~~ This reevaluation effort was titled the "Long Term Seismic Program".

~~PG&E's reevaluation effort in response to the license condition was titled the "Long Term Seismic Program" (LTSP). PG&E prepared and submitted to the NRC the "Final Report of the Diablo Canyon Long Term Seismic Program" in July 1988 (Reference 19). The NRC reviewed the Final Report between 1988 and 1991, and PG&E prepared and submitted written responses to NRC questions resulting from that review. In February 1991, PG&E issued the "Addendum to the 1988 Final Report of the Diablo Canyon Long Term Seismic Program." (Reference 20) In June 1991, the NRC issued Supplement 34 to the Diablo Canyon Safety Evaluation Report (SSER) (Reference 21), in which the NRC concluded that PG&E had satisfied License Condition 2.C(7) of DPR-80. In the SSER the NRC requested certain confirmatory analyses from PG&E, and PG&E subsequently submitted the requested analyses. The NRC's final acceptance of the LTSP is documented in a letter to PG&E dated April 17, 1992 (Reference 22).~~

~~The LTSP contains extensive databases and analyses that update the basic geologic and seismic information in this FSAR Update. However, the LTSP material does not alter the design bases for DCPP. In SSER 34 (Reference 21), the NRC states, "The Staff notes that the seismic qualification basis for Diablo Canyon will continue to be the original design basis plus the Hosgri evaluation basis, along with associated analytical methods, initial conditions, etc."~~

As a condition of the NRC's final acceptance of the LTSP, PG&E committed to ongoing activities in support of the LTSP, as follows:

(1) The "Framework for the Future," per letter to the NRC, dated April 17, 1991 (Reference 32). These ongoing activities include the following (Reference 21, and Section 2.5.6.1):

(a) To continue to maintain a strong geosciences and engineering staff to keep abreast of new geological, seismic, and seismic engineering information and evaluate it with respect to its significance to Diablo Canyon. See Section 2.5.6 for additional details.

(b) To continue to operate a strong-motion accelerometer array and the coastal seismic network, although likely with fewer stations than currently operating. See Section 3.7.4 for additional details.

(2) "Future Plant Additions and Modifications," per letter~~PG&E committed~~ to the NRC~~, in a letter~~ dated July 16, 1991 (Reference 23). This commitment requires~~,~~ that certain plant additions and modifications, as identified in that letter, would be checked against insights and knowledge gained from the LTSP to verify that the plant margins remain acceptable. See Section 3.7.6 for additional details.

A completed listing of bibliographic references to the LTSP reports and other documents are provided in References 19, 20, and 21.

3.7.1.1 Design Response Spectra

Section 2.5.2 provides a discussion of the earthquakes postulated for the DCPP site and the effects of these earthquakes in terms of maximum free-field ground motion accelerations and corresponding response spectra at the plant site. The ground motion response spectra associated with each of these earthquakes are described in the following sections.

3.7.1.1.1 Design Earthquake (DE)

The original (pre-construction permit) geological and seismological investigations determined that the maximum vibratory accelerations at the plant site would result from either Earthquake B (a magnitude 7.5 earthquake on the Nacimiento fault) or Earthquake D-modified (a magnitude 6.75 aftershock, on an unknown fault directly below DCPP, associated with a magnitude 8.5 earthquake on the San Andreas fault), depending on the natural period of the vibrating body (See Section 2.5.2.9.1). Response acceleration spectra curves for horizontal free-field ground motion at the plant site from Earthquake B and~~,~~ Earthquake D-modified~~, and HE~~ are presented in Figures 2.5-20~~,~~ and 2.5-21~~, and 2.5-29 through 32, respectively~~.

For design purposes, the response spectra for each damping value from Earthquake B and Earthquake D-modified ~~are~~ were combined to produce an envelope spectrum. The acceleration value for any period on the envelope spectrum is equal to the larger of the two values from the Earthquake B spectrum and the Earthquake D-modified spectrum. Vertical free field ground accelerations, and the vertical free-field ground motion response spectra ~~are~~ were assumed to be two-thirds of the corresponding horizontal spectra.

The DE is the hypothetical earthquake that would produce these horizontal and vertical vibratory accelerations. As discussed in Section 3.2.1, the~~The~~ DE corresponds to the operating basis earthquake (OBE), as described in Appendix A to 10 CFR 100 (Reference 7) (SSER 7, Section 2.5.2, "Operating Basis Earthquake").

Note that the DE is a hypothetical earthquake that is not to be revised based on any insights from new geotechnical information from the Long Term Seismic Program (LTSP). The process for the evaluation of updated LTSP ground motions is described in Section 2.5.6.2. Updated ground motions are compared to the 1991 LTSP spectra, which is bounded by the 1977 HE spectra. The HE is the SSE.

3.7.1.1.2 Double Design Earthquake (DDE)

To ensure adequate reserve energy capacity, Design Class I structures and equipment are ~~reviewed~~ also designed for the DDE. The DDE is the hypothetical earthquake that would produce accelerations twice those of the DE. ~~The DDE corresponds to the SSE, as described in Appendix A to 10 CFR 100 (Reference 7).~~ The horizontal free-field response spectra for the DDE correspond to twice the envelope of the spectra shown in Figures 2.5-20 and 2.5-21. The vertical free field ground accelerations and the vertical free-field ground motion response spectra are assumed to be two-thirds of the corresponding horizontal spectra.

Note that the DDE is a hypothetical earthquake that is not to be revised based on any insights from new geotechnical information from the Long Term Seismic Program. The process for the evaluation of updated LTSP ground motions is described in Section 2.5.6.2. Updated ground motions are compared to the 1991 LTSP spectra, which is bounded by the 1977 HE spectra. The HE is the SSE.

3.7.1.1.3 1977 Hosgri Earthquake (HE)

PG&E was requested by the NRC to evaluate the plant's capability to withstand a postulated Richter magnitude 7.5 earthquake centered along an offshore zone of geologic faulting, generally referred to as the Hosgri Fault. This evaluation is discussed in the various chapters when it is specifically referred to as the Hosgri evaluation or Hosgri event evaluation.

Acceleration response spectra curves for horizontal and vertical free field ground motion at the plant site from the HE in 1977 are the Newmark and Blume spectra described in Section 2.5. The vertical free field response spectra are two-thirds of the corresponding horizontal spectra. As discussed in Section 3.2.1, the 1977 HE spectrum corresponds to the SSE, as described in Appendix A to 10 CFR 100 (Reference 7). The horizontal free-field ground motion response spectra for the HE (Blume) and HE (Newmark) are shown in Figures 2.5-29 and 2.5-30, respectively. The vertical free-field ground motion response spectra for the HE are shown in Figures 2.5-31 and 2.5-32.

3.7.1.1.4 1991 Long Term Seismic Program Earthquake (LTSP)

As discussed in Sections 2.5 and 3.7, the Long Term Seismic Program was developed in response to Unit 1 Operating License Condition 2.C.(7). The acceleration response spectra curves for horizontal and vertical free field ground motion at the plant site are the 84th percentile ground motion response spectrum, as modified per SSER-34 (Reference 21), as described in Section 2.5.2.10.4, are shown in Figures 2.5-33 and 2.5-34. Note that, unlike the DE, DDE, or 1977 HE, the vertical free field response spectrum is not based on a scale factor times the corresponding horizontal spectrum.

The ongoing activities in support of the LTSP, described in Section 2.5.6, may result in changes to the 84th percentile ground motion response spectrum for the LTSP. The methods for the evaluation of the significance of any changes are described in Section 2.5.6.

3.7.1.2 Design Response Spectra Derivation

3.7.1.2.1 Design Earthquake (DE) and Double Design Earthquake (DDE) Derivation

The free-field ground motion acceleration time-histories used in the dynamic analyses of the containment structure, auxiliary building, turbine building, and intake structure are developed by the following procedure: The response spectra for 2 percent damping for Earthquake B and Earthquake D-modified are enveloped to produce a single response spectrum (DE intensity). A time-history is then developed that produces a spectrum with no significant deviation from the smooth DE-envelope spectrum. This procedure eliminates undesirable peaks and valleys that exist in the response spectrum calculated directly from Earthquake B and Earthquake D-modified records.

A similar procedure is used to obtain a free-field ground motion acceleration time-history for the DDE. The free-field ground motion acceleration time-histories for the DE and DDE are shown in Figures 3.7-1 and 3.7-2, respectively. Comparison of the response spectrum computed from the time-history with the smoothed envelope spectrum is shown in Figure 3.7-3 (2 percent damping) and in Figure 3.7-4 (5 percent damping). These spectra are calculated at period intervals of 0.01 seconds, which adequately define the spectra.

The dynamic analyses of the containment structures and auxiliary building consider the interactions between their embedded foundations and the surrounding soil through the inclusion of soil-structure interaction effects in the finite element models (see Section 3.7.2.1.7.1). As a result, the calculated response at ground level is not the same as the free-field ground motion. Soil-structure interaction effects are not considered in the dynamic analysis of the turbine building and intake structure.

3.7.1.2.2 1977 Hosgri Earthquake (HE) Derivation

For the HE evaluation of containment structure, auxiliary building, turbine building, and intake structure, the horizontal input motions are reduced from free-field motions to account for the presence of the structures that have large foundations. These reduced inputs have been derived by spatial averaging of acceleration across the foundations of each structure by the Tau filtering procedure (Reference 12). The resulting horizontal response spectra for these structures are shown in Figures 3.7-4A through 3.7-4F.

For the HE evaluation of outdoor water storage tanks and smaller structures, the horizontal design response spectra are the free-field horizontal response spectra. HE vertical design response spectra are the free-field vertical response spectra. ~~For design purposes, the Newmark spectra are used, or alternately the Blume spectra are used, with adjustment in certain frequency ranges as necessary so that they do not fall below the corresponding Newmark spectra.~~

For the design of structures, the seismic response parameters (e.g., forces, moments, displacements, accelerations) are determined based on either of the following methods:

(1) The response to the Newmark and Blume ground motions are developed separately, and then the response parameters are enveloped.

(2) The response to an envelope of the Newmark and Blume ground motions is developed

Acceleration time-histories used in the analysis of the containment and intake structures, auxiliary building, and turbine building are shown in Figures 3.7-4G through 3.7-4M. Comparison of the response spectrum computed from each time-history with the corresponding design response spectrum for 7 percent damping is shown in Figures 3.7-4N through 3.7-4T.

3.7.1.2.3 1991 Long Term Seismic Program Earthquake (LTSP)

The free-field ground motion acceleration time-histories used in the dynamic analyses of the containment structure, auxiliary building, and turbine building are developed by the following procedure (Reference 19, Chapter 5, and Reference 33, Question DE-2):

(1) Two sets of strong-motion recordings of three-component actual earthquakes were selected.

(2) The original recorded motions were adjusted to conform to source-specific and site-specific conditions, such as the maximum earthquake magnitude, source-to-site distance, and site conditions.

(3) The two horizontal components of the motions were transformed, as necessary, into longitudinal and transverse horizontal components to provide motions in the directions normal and parallel to the strike of the causative fault.

(4) The longitudinal and transverse time histories were both modified by adjusting the Fourier amplitudes, but keeping the Fourier phase-angles unchanged, so that the resulting time history response spectra closely matched the median site-specific target spectra at several damping ratios. Likewise, the vertical component time histories were modified to match the median site-specific target vertical spectra at several damping values.

(5) The three-component time histories were scaled upwards by a constant scaling factor common to all three components to envelop the LTSP 84th percentile ground motion response spectrum (Figures 2.5-33 and 2.5-34)

Sample free-field ground motion acceleration time-histories for the LTSP 84th percentile ground motion response spectrum are shown in Figures 3.7-29 and 3.7-30. Comparison of the response spectrum computed from the time-history with the target spectrum is shown in Figures 3.7-31 and 3.7-32 (5 percent damping).

The dynamic analyses of the containment structures, auxiliary building, and turbine building consider the interaction between their embedded foundations and surrounding soil through the inclusion of soil-structure interaction effects in the finite element models (see Chapter 5 of Reference 19). As a result, the calculated response at ground level is not the same as the free-field ground motion. A dynamic analysis of the intake structure was not performed for the LTSP earthquake.

3.7.3.15.3 Control Rod Drive Mechanism Evaluation

The replacement CRDMs were evaluated using a combination of linear and nonlinear finite element models which included the CRDM housings, RPV head adapters, and the integrated head assembly. The following models and analysis methods were employed for the specified earthquakes:

(1) DE and DDE: The horizontal analyses for the DE and DDE were based on a nonlinear model. The horizontal DE and DDE acceleration time-histories at the seismic plate elevation and the reactor vessel support elevation were used as inputs to the model. The vertical analyses for the DE and DDE were based on a linear model. The vertical DE and DDE response spectra at the reactor vessel head elevation were used as input to the model.

(2) HE: The horizontal and vertical analyses for the HE were based on a linear model. The horizontal and vertical HE response spectra at the seismic plate elevation and the reactor vessel head elevation were used as input to the model.

The DDE and the HE seismic loads were combined by the square root sum of the squares (SRSS) methodology with the LOCA loads. The resulting stress levels satisfied the code requirements.

3.7.3.15.4 CRDM Support System Evaluation

The integrated head assembly CRDM seismic support structure, tie rods, and head lifting legs were evaluated using linear elastic 3-D finite element models of the support system. Tension-only capability of the tie rods was modeled. The loading from the CRDMs was addressed through the inclusion of a simplified representation of the pressure housings, including the appropriate lumped masses.

In general, the qualification was based on the response spectrum superposition method using the envelope of the spectra at the 140 foot elevation of the containment interior concrete (attachment point for the tie rods for the tie rods to the reactor cavity walls) and on the reactor vessel lifting lugs and pads (attachment point for the integrated head assembly ring beam to the head) for the DE, DDE, HE, and LOCA load cases. These analyses were supplemented with the time history modal superposition method for the determination of DDE loads for selected connections.

The DDE and the HE seismic loads were combined by the square root sum of the squares (SRSS) methodology with the LOCA loads. The resulting stress levels satisfied the code requirements.

3.7.4 SEISMIC INSTRUMENTATION PROGRAM

The seismic instrumentation program for DCPP includes two independent systems, the Seismic Monitoring System and the Central Coast Seismic Network. Descriptions of these systems are provided in the following sections.

3.7.4.1 Seismic Monitoring System

3.7.4.1.1 Comparison With NRC Regulatory Guide 1.12, Revision 2

The seismic monitoring system instrumentation consists of strong motion triaxial accelerometers that sense and record ground motions. The licensing basis for the seismic monitoring system instrumentation is Safety Guide 12, "Instrumentation for Earthquakes," dated March 10, 1971. ~~This~~The seismic monitoring system instrumentation consists of a Basic and Supplemental System.~~, meets the intent of RG 1.12, Revision 2.~~ The Basic System is consistent with, but not committed to, RG 1.12 Revision 2. Enhancements to the Basic seismic ~~instrumentation~~ monitoring system have been made to improve the system effectiveness. The enhancements, described as the Supplemental System, include supplemental accelerometers and rapid processing of the ground motion data. The enhancements exceed the ~~intent~~recommendations of RG 1.12, Revision 2, and are not considered part of the licensing basis. However, as discussed in Section 3.7, one of the ongoing commitments associated with the LTSP requires that the entire system, including both the basic and supplemental accelerometers, be maintained.

3.7.4.1.2 ~~Location and~~ Description of Instrumentation

Seismic instrumentation is provided in accordance with RG 1.12, Revision 2, paragraph 1.2. All instruments are rigidly mounted so their records can be related to movement of the structures and ground motion. All are accessible for periodic servicing and for obtaining readings.

3.7.4.1.2.1 Strong Motion Triaxial Accelerometers

Strong motion triaxial accelerometers provide time-histories of acceleration for each of three orthogonal directions. These histories are recorded in the accelerometer housings. The instruments start recording upon actuation of a seismic trigger which has an adjustable threshold. Six strong motion triaxial accelerometers are provided in accordance with RG 1.12. Revision 2, paragraph 1.2. Supplemental accelerometers provide ground motion data beyond the regulatory guidance and are not part of the licensing commitment,

3.7.4.1.3 Control Room Operator Notification

Operation of the strong motion triaxial accelerometers (ESTA01 or ESTA28) will activate an annunciator in the control room and provide indications on the earthquake force monitor (EFM) in the ~~RSI~~ seismic instrumentation panel. The EFM will display the acceleration levels for all areas of both the Unit 1 containment base sensor (ESTA01) and the free field sensor (ESTA28). For the Emergency Plan event classification, it also provides a status of level exceedance for any axis on both sensors within a few minutes. The setpoint thresholds are set in accordance with Emergency Plan Action Levels. .

3.7.4.1.4 Comparison of Measured and Predicted Responses

In the event of an earthquake that produces significant ground motions, all seismic instruments are read and the readings compared to the corresponding design values. This comparison, together with information provided by other plant instrumentation and an inspection of safety-related systems, forms the basis for a judgment on severity, level, and the effects of the earthquake.

In addition, the recorded time histories, and the associated response spectra, are used by the PG&E Geosciences Department, as input to the ongoing LTSP activities. See Section 2.5.6.

3.7.4.2 Central Coast Seismic Network

The PG&E Geosciences Department operates and maintains an array of seismometers located primarily along the south-central California coast, between Ragged Point and Point Sal, and the recording equipment located at the PG&E Geosciences Department office in San Francisco. The Central Coast Seismic Network (CCSN) was installed in 1987 as part of the LTSP to provide continuous real-time monitoring of earthquake activity in the vicinity of DCPP. Data from the CCSN are transmitted directly to the PG&E Geosciences Department, where it is stored, processed, and archived, and to the United State Geological Survey in Menlo Park, CA.

These data are used by the PG&E Geosciences Department, as input to the ongoing LTSP activities. See Section 2.5.6.

~~3.7.6 SEISMIC EVALUATION TO DEMONSTRATE COMPLIANCE WITH THE HOSGRI EARTHQUAKE REQUIREMENTS UTILIZING A DEDICATED SHUTDOWN FLOWPATH~~

~~3.7.6.1 Post-Hosgri Shutdown Requirements and Assumed Conditions~~

~~In response to a request from the NRC, PG&E evaluated the ability of DCPP to shut down following the occurrence of a 7.5M earthquake due to a seismic event on the Hosgri fault. This evaluation is presented in Reference 15, which was amended several times after it was first issued in order to respond to questions by the NRC and reflect agreements made at meetings with the NRC. The final document describes the method proposed by PG&E to shut down the plant after the earthquake, assuming a loss of all offsite power, but no concurrent accident, using only equipment qualified to remain operable following such an earthquake.~~

~~For this purpose, valves that are required to operate to achieve shutdown following the earthquake were qualified for active function to the Hosgri parameters, whereas other valves, which might have an active function for postaccident mitigation, but were not required to operate to achieve shutdown following the earthquake, were qualified for passive function (pressure boundary integrity) to the Hosgri parameters. This is consistent with the DCPP design basis stated in FSAR Section 3.7.1.1 that the DDE is the SSE for DCPP, and that the guidelines presented in RG 1.29 apply to the DDE.~~

~~In addition, pursuant to the NRC request, it was necessary to demonstrate that DCPP could be shut down following an HE in order to protect the health and safety of the public. The Hosgri evaluation presented in Reference 15 demonstrated this. To provide increased conservatism, PG&E has subsequently qualified all active valves for active function for an HE pursuant to a commitment made in Reference 17.~~

~~3.7.6.2 Post-Hosgri Safe Shutdown Flowpath~~

~~The flowpath qualified to enable shutdown of the plant following an HE is defined in Chapter 5 of Reference 15. For this purpose, safe shutdown was defined as cold shutdown. It assumes concurrent loss of offsite power, a single active failure, but no concurrent accident or fire. Local manual operation of equipment from outside the control room is acceptable for taking the plant from hot standby to cold shutdown.~~

~~3.7.6.2.1 Hot Standby~~

~~Hot standby is achieved by feeding the steam generators using the auxiliary feedwater system and by release of steam to the atmosphere through the 10 percent steam dump valves. Although other long term cooling water sources may be available, only the seismically qualified condensate storage tank and firewater storage tank are assumed to be available.~~

~~3.7.6.2.2 Cold Shutdown~~

~~Cold shutdown is achieved by use of the normal charging system flow path. Depressurization is performed using auxiliary spray (alternatively, the PORVs may be used). Boration to cold shutdown concentration is accomplished using boric acid from the boric acid storage tanks via the emergency borate valve 8104 and using a centrifugal charging pump (CCP1 or CCP2) charging through valves FCV-128, HCV-142, 8108, 8107, and 8146 or 8147. Sampling capability to verify boron concentration is available. While reactor coolant pump seal injection flow would be available, the seal water return flow path and the normal letdown flow path are assumed not to be available. Calculations have shown that even with letdown unavailable, by taking credit for shrinkage of the reactor coolant during cooldown, sufficient volume is available in the reactor coolant system to borate to cold shutdown using 4 percent boric acid.~~

~~Once the RCS is less than or equal to 390 psig and 350°F, the normal RHR system is placed into service, along with the portions of the component cooling water and auxiliary salt water systems which support RHR operation.~~

~~3.7.6.2.3 Single Active Failure~~

~~Systems and components used to perform the post-Hosgri shutdown described above have redundant counterparts except for components along the normal charging flowpath, which lacks redundancy since its redundant flow path for emergency boration is the high-pressure safety injection flow path. Use of that redundant flow path is not postulated for post-Hosgri shutdown, however, so adequate redundancy had to be incorporated into the normal charging flowpath to enable cold shutdown following the HE. For this purpose, the Hosgri evaluation assumed that manual bypass valves 8387B or 8387C would be used in the event that fail-open valve FCV-128 was to fail closed. Manual bypass valve 8403 would be used in the event that fail-closed valve HCV-142 was to fail closed. Fail-open valve FCV-110A and manual bypass valve 8471 would be used in the event that motor-operated valve 8104 was to fail closed. Valves 8146 and 8147 were assumed redundant for normal charging, and valves 8145 and 8148 were assumed redundant for pressurizer auxiliary spray. Valves with pneumatic operators, which are required to operate to achieve shutdown, were fitted with seismically qualified air or nitrogen accumulators to enable their operation in spite of the loss of their instrument air or nitrogen supply. Although some of these valves do not have safety-related operators since they are not required for accident mitigation, they are seismically qualified to ensure their operability for post-Hosgri shutdown.~~

~~3.7.6.2.4 Equipment Required for Post-Hosgri Shutdown~~

~~The equipment determined to be required to achieve post-Hosgri cold shutdown in the manner described above is presented in Sections 7.3 and 9.2 of Reference 15. Some minor revisions to the list of valves required have been made, and are reflected in the latest revision of the active valve list, FSAR Table 3.9-9. Instrument Class IA,~~

~~Instrument Class IB, Category 1, and on a case-by-case basis, Instrument Class ID instrumentation are qualified to the Hosgri parameters, and assumed to be operable following an HE. Additional instrumentation determined to be required is presented in Section 7.3 of Reference 15. Some revisions have been made to that list; the revised list of required instrumentation is presented in Reference 16. The electrical Class 1E system is also qualified to the Hosgri parameters, and is assumed to be operable following an HE.~~

3.7.6 APPLICATION OF THE LTSP TO MODIFICATIONS AND ADDITIONS

As indicated in Section 3.7.1, one of the on-going commitments associated with the LTSP requires that certain plant additions and modifications, as identified in Reference 23, would be checked against insights and knowledge gained from the LTSP to verify that the plant seismic margins remain acceptable (Reference 23).

The LTSP findings have demonstrated that the use of the original DCPP design criteria and methodology for DE, DDE, and 1977 HE consistently produces an adequately conservative design. Therefore, future additions and modifications to DCPP will be designed and constructed in accordance with the existing seismic qualification basis. This includes the following:

(1) The earthquake motions defined for the DE,-DDE, and 1977 HE.

(2) The acceptance criteria and methodology corresponding to each of these earthquakes.

In addition, in order to take advantage of the insights and knowledge gained from the LTSP, certain additions and modifications will be evaluated under the LTSP, to verify that the seismic margins remain acceptable. The basis for this selection process is described in Section 3.7.6.1.

3.7.6.1 Basis for Selection of LTSP Evaluation Scope

3.7.6.1.1 Modifications and Additions in the LTSP Evaluation Scope

The additions and/or modifications of plant structures and components that will be evaluated under the LTSP are selected based on the following:

(1) The seismic probabilistic risk assessment studies have identified certain structures and components that are significant contributors to the Plant seismic risk (e.g., turbine building and diesel control panel. The seismic capacities are defined in terms of average spectral acceleration in 3 to 8.5 hertz frequency range of the 5 percent damped horizontal ground response spectrum having the same spectral shape as the 1991 LTSP spectrum shown in Figure 2.5-33.

(2) Major modifications to structures particularly important to plant safety are included (e.g., the containment structures and auxiliary building). Major modifications are defined as those changes that significantly affect the dynamic properties (such as mass, stiffness) and strengths of the structures.

(3) New/unique structures (such as non-safety related structures which could interact with safety related structures) that significantly impact the seismic margins of the existing safety-related structures.

(4) Masonry walls (all new construction and significant modifications to existing walls).

(5) Specific issues determined to be important in the LTSP margins evaluation (e.g. relay chatter and electrical panel anchorage).

(6) New major safety related equipment that may significantly impact seismic risk (e.g., diesel generator no. 2-3).

Tables 3.7-25 and 3.7-26, list all structures, systems, and components (SSCs) that have the potential to impact SCDF and were evaluated under the 1991 LTSP. The tables also indicate which of the listed SSCs require LTSP evaluations for the impact of additions or modifications. The original (1991) scope of these tables, as listed in *References 19 and 20, was developed based on the methods and evaluations* described in Reference 39, which identified the SSCs that were the dominant contributors to the overall seismic risk. Based on estimates of the fragilities for these SSCs, a subset were modeled in the seismic PRA, while the remainder were not modeled, based on their relatively high seismic capacities.

SSCs have been, and will continue to be, added to Tables 3.7-25 and 3.7-26 if the SSC meets the criteria for requiring an LTSP evaluation, described above.

3.7.6.1.2 Modifications and Additions Excluded from LTSP Evaluation Scope

Specific categories of additions and/or modifications to the structures, equipment, and components need not be evaluated under the LTSP. These categories are as follows:

(1) Seismic like-for-like replacement of structures, equipment, and components. These replacements will not change the SSC's seismic margin.

(2) Minor additions or modification to structures (such as access platforms, typical core drills, modifications to nonstructural elements, etc.). These additions and modifications will not significantly affect the structure's seismic margin.

(3) Additions or modifications to electrical raceways and supports. These commodities have high seismic margins due to redundancies in design and high damping.

(4) Additions or modifications to HVAC ducts and duct supports. These commodities have high seismic margins due to redundancies in design.

(5) Additions or modifications to piping and supports. These commodities have high seismic margins due to redundancies in design.

(6) Additions or modifications to hand-operated valves, relief valves, solenoid valves and check valves and air- and motor-operated valves, due to their high seismic margins. Specific exceptions are noted in Table 3.7-26.

(7) Other additions and modifications not meeting the criteria defined in Section 3.7.6.1.1

3.7.6.2 LTSP Evaluation Process

The following provides a summary of the key steps of the LTSP evaluation process applied to additions and modifications to DCPP structures, systems, and components. An overview of the LTSP evaluation process is shown in Figure 3.7-33.

(1) Additions and modifications are designed in accordance with the DCPP design change process, considering the applicable seismic qualification bases (e.g., DE, DDE, HE, as applicable), and reviewed under the Licensing Basis Impact Evaluation process.

(2) The scope of the addition or modification is checked against the criteria defined in Section 3.7.6.1 to determine if an LTSP evaluation is required. If an LTSP evaluation is required, proceed to Step (3), otherwise, the process is complete at this stage.

(3) Calculate the 84th percentile ground motion response spectrum High-Confidence-Low-Probability-of-Failure capacity (HCLPF84) capacities for the in-scope items using either the Fragility Analysis method (see Section 3.7.6.2.1), the Conservative Deterministic Failure Margin (CDFM) method (see Section 3.7.6.2.2), or the earthquake experience data method (see Section 3.7.6.2.3).

(4) If the in-scope item is a new SSC, skip to Step (6). Otherwise, for modifications to existing SSCs, proceed to Step (5).

(5) If the revised capacity for a modified SSC is greater than or equal to the value listed in Tables 3.7-25 or 3.7-26, skip to Step (9). Otherwise proceed to Step (6).

(6) Calculate the seismic margin (ratio of the HCLPF capacity to the seismic demand associated with the 1991 LTSP ground motion spectrum) for the SSC. If the seismic margin is greater than or equal to 1.3*, skip to Step (8). Otherwise, proceed to Step (7).

* A seismic margin of less than 1.3 is acceptable for certain SSCs (see Section 2.5.6.2.1.1). For these SSCs, skip to Step (8).

(7) Determine if a license amendment request will be pursued to allow a seismic margin below 1.3 for SSC. If a license amendment request is submitted, hold design pending receipt of license amendment, then proceed to Step (8). Otherwise, redesign the new/modified SSC to increase the seismic margin and return to Step (1).

(8) Calculate the fragility curve for the new/modified SSC (see Section 3.7.6.2.1) and conduct a seismic probabilistic risk assessment, in accordance with ASME/ANS RA-Sa-2009 and RG 1.200, Rev. 2, to determine the seismic core damage frequency.

(9) The LTSP evaluation for the new/modified SSC is complete.

Note that the process for the LTSP evaluation, described above, is in terms of the horizontal ground motion and the fragility/capacity of the SSC relative to horizontal input motion. A similar approach can be applied to the vertical ground motion and the fragility/capacity of the SSC relative to vertical input motion. However, as discussed in Chapter 6 of Reference 19, the fragilities of most SSCs are dominated by their response to horizontal input motion, and the contribution due to vertical input motion is generally small. Therefore, the consideration of the impact of vertical input motion on the LTSP evaluation of a specific SSC will be addressed on a case-by-case basis.

3.7.6.2.1 Fragility Analysis Method

During the initial implementation of the LTSP (1985 through 1991), the HCLPF capacities of most SSCs were developed using the fragility analysis method. Details of the fragility analysis method are described in Chapter 6 of the 1988 LTSP Final Report (Reference 19). The fragility curves (see Figure 2.5-39 for sample curve) are tied to the 5% damped spectral acceleration value, averaged between 3 and 8.5 Hz.

The computation of fragilities for new components, modifications to existing components, or as inputs to the ~~evaluation of updated LTSP seismic hazards input~~ probabilistic risk assessment and/or deterministic seismic margin evaluation (See Section 2.5.6) shall be based on the methods described in ASME/ANS RA-Sa-2009 (Reference 36), as modified by Regulatory Guide 1.200, Revision 2 (Reference 37).

3.7.6.2.2 Conservative Deterministic Failure Margins Method

During the initial implementation of the LTSP (1985 through 1991), the HCLPF capacities of certain SSCs were developed using the CDFM method, and compared to the HCLPF capacities developed based on the fragility method. This comparison validated the approximate equivalency of the two methods. General guidelines of the application of the CFDM method are provided in EPRI NP-6041-SL (Reference 35).

Details of the application of the CDFM method at DCPP are described in PG&E report "Additional Deterministic Evaluations Performed to Assess Seismic Margins of the Diablo Canyon Power Plant" (Reference 38).

The HCLPF capacities are tied to the 5% damped spectral acceleration value, averaged between 3 and 8.5 Hz. The same methodology may be used for the computation of HCLPF capacities for new components, modifications to existing components, or as input to the evaluation of updated LTSP seismic hazards input (See Section 2.5.6).

3.7.6.2.3 Earthquake Experience Data Method

During the initial implementation of the LTSP (1985 through 1991), the HCLPF capacities of components associated with the 230kV switchyard (e.g., transformers, breakers, switches) were developed using the earthquake experience data method. General guidelines of the application of the earthquake experience data method are provided in Appendix A to EPRI NP-6041-SL (Reference 35). Details of the application of the earthquake experience data method at DCPP are described in PG&E report "Long Term Seismic Program – Seismic Capacity of the 230 kV Switchyard" (Reference 34).

The HCLPF capacities are tied to the 5% damped spectral acceleration value, averaged between 3 and 8.5 Hz. The same methodology may be used for the computation of HCLPF capacities for new components, modifications to existing components, or as input to the evaluation of updated LTSP seismic hazards input (See Section 2.5.6).

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TABLE 3.7-25

HIGH CONFIDENCE LOW PROBABILITY OF FAILURE ($HCLPF_{84}$) CAPACITIES AND SEISMIC MARGINS FOR CIVIL STRUCTURES[(e)]

Structure	$HCLPF_{84}$ Capacity (g)[(a)]	Seismic Margin[(b)]	In Scope for LTSP Review of Modifications[(c)]?
Containment Structures	4.30	2.22	Y
Containment Interior Structures	3.58	1.85	Y
Intake Structure	3.88	2.00	Y
Auxiliary Building	3.19	1.64	Y
Turbine Building (including Turbine Pedestals)	2.21	1.14[(d)]	Y
Refueling Water Storage Tanks	4.21	2.17	
Condensate Storage Tank	≥5	≥2.58	
Diesel Generator Fuel Oil Storage Tanks	≥5	≥2.58	
Safety Related Masonry Walls	2.83	1.46	Y

Notes:

(a) The $HCLPF_{84}$ capacity is equal to 1.20 times the $HCLPF_{50}$ (median) capacity

(b) The seismic margin equals $HCLPF_{84}$ capacity divided by 1.94 g (applicable to horizontal input motion).

(c) Per Reference 23. See Section 3.7.6.1.1.

(d) Seismic margin of less than 1.3 acceptable for Turbine Building, see Section 2.5.6.2.1.1.

(e) The $HCLPF_{84}$ capacities and seismic margin value provided in this table are associated with horizontal input motion. The corresponding values associated with vertical input motion are not reported. and must be evaluated on a case-by-case basis, if required, as discussed in Section 3.7.9.2.

TABLE 3.7-26 Sheet 1 of 4

HIGH CONFIDENCE LOW PROBABILITY OF FAILURE ($HCLPF_{84}$) CAPACITIES AND SEISMIC MARGINS FOR EQUIPMENT AND COMPONENTS[f]

System/Component	$HCLPF_{84}$ Capacity (g)[a]	Seismic Margin[b]	In Scope for LTSP Review of Modifications[c]?
Nuclear Steam Supply System			
Reactor Pressure Vessel	4.01	2.07	
Reactor Internals	4.85	2.50	
- Integrated Head Assembly[e]	2.40	1.24[d]	Y
Steam Generators	3.16	1.63	Y
Pressurizer	4.00	2.06	
Pressurizer Safety Valves	>3	>1.55	
Power Operated Relief Valves	2.78	1.43	Y
Reactor Coolant Pumps	3.40	1.75	Y
Control Rod Drives	4.08	2.10	
NSSS Piping	>3	>1.55	
Residual Heat Removal			
RHR Pumps	4.02	2.07	
RHR Heat Exchangers	4.18	2.15	
Safety Injection			
SI Accumulators	5.44	2.80	
SI Pumps	5.57	2.87	
Boron Injection Tank	4.75	2.45	
Component Cooling Water			
CCW Pumps	4.49	2.31	
CCW Heat Exchangers	3.06	1.58	Y
CCW Surge Tank	3.31	1.71	Y
Chemical and Volume Control			
ECCS Centrifugal Charging Pumps	5.34	2.75	
Auxiliary Saltwater			
ASW Pumps	>3	>1.55	
ASW Piping	5.45	2.81	
Containment Spray			
CS Pumps	4.62	2.38	
Spray Additive Tank	3.68	1.90	

TABLE 3.7-26 | Sheet 2 of 4

System/Component	$HCLPF_{84}$ Capacity (g)[a]	Seismic Margin[b]	In Scope for LTSP Review of Modifications[c]?
Main Steam			
MS Isolation Valves	>3	>1.55	
MS Safety Valves	>3	>1.55	
MS PORVs	4.21	2.17	
Auxiliary Feedwater			
AFW Pumps (Motor Driven)	>3	>1.55	
AFW Pumps (Turbine Driven)	4.06	2.09	
Diesel Generator			
DG Fuel Oil Day Tank	>3	>1.55	
DG Fuel Oil Pumps/Filters	4.39	2.26	
DG Fuel Off Shutoff Valve	>3	>1.55	
DG Air Start Compressor	>3	>1.55	
DG Air Start Receiver	>3	>1.55	
Diesel Generators	4.37	2.25	
DG Radiator/Water Pump	4.39	2.26	
DG Inlet Silencer/Air Filter	>3	>1.55	
DG Excitation Cubical	3.08	1.59	Y
DG Control Panel			
- Chatter	5.51	2.84	Y
- Structural	2.69	1.39	Y
DG Main Lead Terminal/Box	>3	>1.55	
Containment Building Ventilation			
Containment Fan Cooler	3.38	1.74	Y
Control Room Ventilation			
Supply Fans	4.58	2.36	
AC Units/Compressor	>3	>1.55	
Control Cabinets	>3	>1.55	
480V Switchgear/Inverter/DC Switchgear/Spreading Room Ventilation			
Supply/Return Fans	4.74	2.44	
Backdraft and Shutoff Dampers	>3	>1.55	
4160V (Vital) Electric Power			
Switchgear			
- Chatter	1.57	0.81[d]	Y
- Structural	3.84	1.98	Y
Potential Transformers			
- Bus F	4.16	2.14	
- Buses G & H	>3	>1.55	

TABLE 3.7-26

System/Component	HCLPF$_{84}$ Capacity (g)[a]	Seismic Margin[b]	In Scope for LTSP Review of Modifications[c]?
Safeguard Relay Panel	4.07	2.10	
125V DC Electric Power			
Batteries	3.29	1.70	Y
Battery Racks	6.48	3.34	
Battery Chargers	3.52	1.81	Y
Switchgear/Breaker Panels	2.83	1.46	Y
120V AC Electric Power			
Instrument Breaker Panels	>3	>1.55	
Inverters	3.30	1.70	Y
480-V (Vital) Electric Power			
4160-V/480-V Transformers	2.90	1.49	Y
Breaker Cabinets (Load Centers)	>3	>1.55	
Auxiliary Relay Panel	4.28	2.21	
Control Room			
Main Control Boards			Y
- Switch Function	>3	>1.55	
- Structural	3.58	1.85	
Hot Shutdown Panel			
- Switch Function	4.36	2.25	
- Structural	4.22	2.18	
Auxiliary Safeguards Cabinet	>3	>1.55	
NSSS Control			
Process Control and Protection System	4.28	2.21	
Solid State Protection System	5.18	2.67	
Reactor Trip Switchgear	3.77	1.94	
Resistance and Temperature Detectors	>3	>1.55	
Pressure and DP Transmitters	4.93	2.54	
Miscellaneous Components			
Auxiliary Relay Rack	>3	>1.55	
Local Starter Boards	>3	>1.55	
Molded Case Circuit Breakers	>3	>1.55	
Valve Limit Switches	>3	>1.55	
Impulse Lines (which affect LOCA)	3.16	1.63	Y
Containment Purge Valves	>3	>1.55	

TABLE 3.7-26

System/Component	HCLPF$_{84}$ Capacity (g)[a]	Seismic Margin[b]	In Scope for LTSP Review of Modifications[c]?
Generic Components			
230 kV Off-Site Power			
- Circuit Breakers	0.84	0.43[d]	Y
- Switches	0.84	0.43[d]	Y
- Transformers	0.84	0.43[d]	Y
Penetrations, Penetration Boxes	3.40	1.75	
BOP Piping and Supports	3.60	1.86	
Hand, Relief, Solenoid, and Check Valves	$\geq$3	$\geq$1.55	
Air and Motor Operated Valves	4.28	2.21	
Cable Trays and Supports	$\geq$3	$\geq$1.55	
HVAC Ducting and Supports	2.99	1.55	

Notes:

(a) The HCLPF$_{84}$ capacity is equal to 1.20 times the HCLPF$_{50}$ (median) capacity

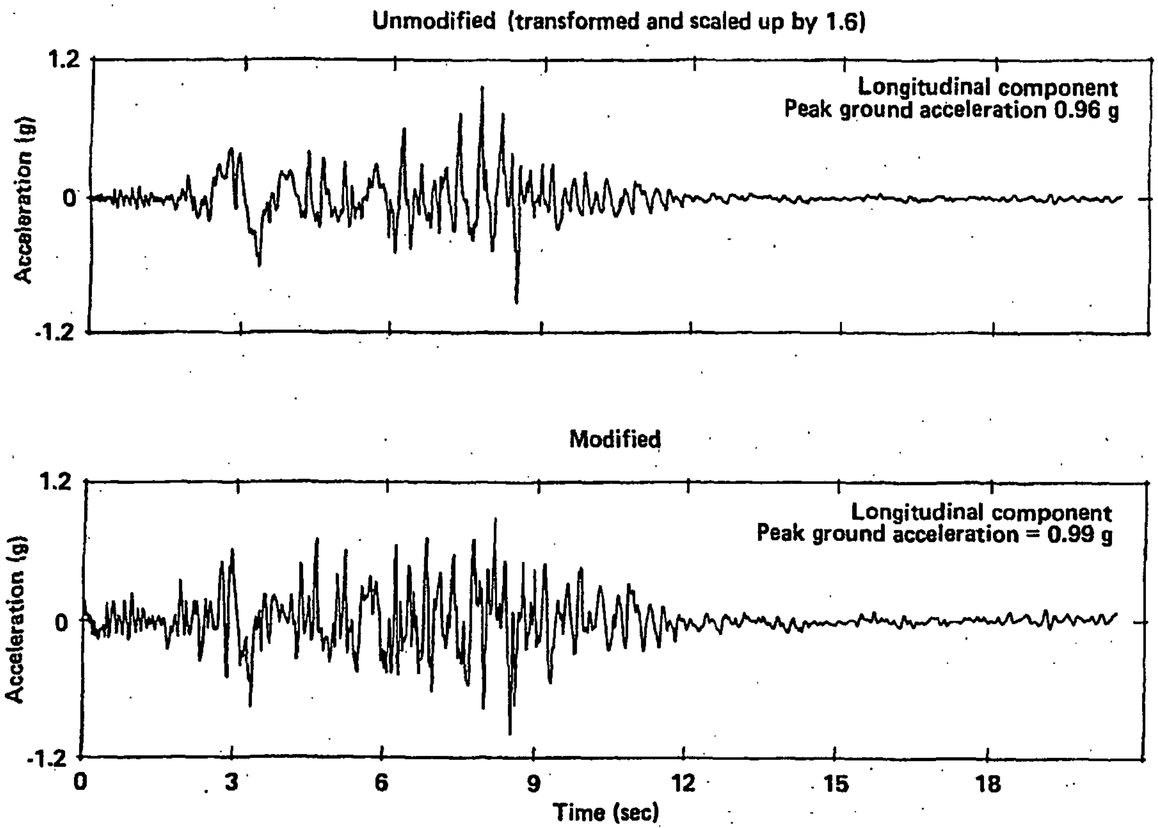
(b) The seismic margin equals HCLPF capacity divided by 1.94 g.

(c) Per Reference 23. See Section 3.7.6.1.1.

(d) Seismic margin of less than 1.3 acceptable for this component, see Section 2.5.6.2.1.1.

(e) The integrated head assembly (IHA) provides lateral support to the control rod drive mechanisms. Seismically induced failure of the IHA could impair control rod drop required for reactor trip. Since the reactor trip function is modeled as part of the reactor internals in the seismic PRA, the IHA is treated as a subcomponent of the reactor internals.

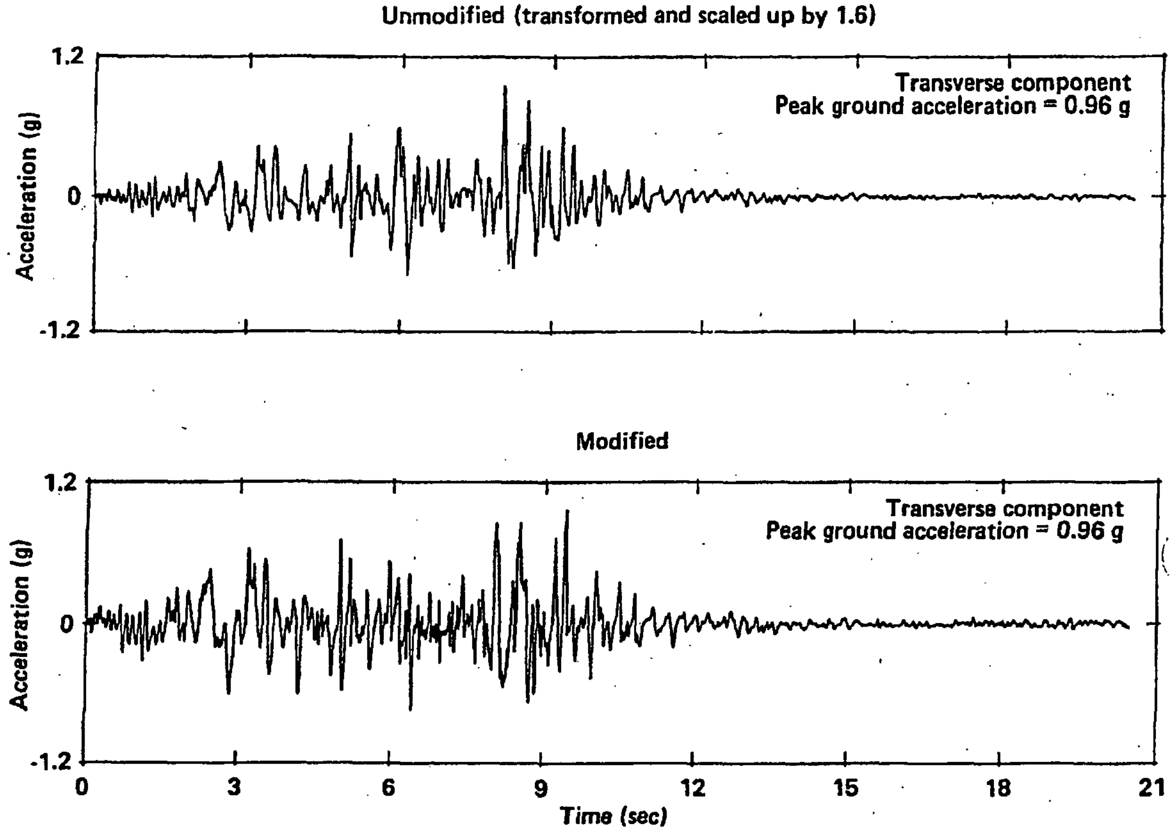
(f) The HCLPF$_{84}$ capacities and seismic margin value provided in this table are associated with horizontal input motion. The corresponding values associated with vertical input motion are not reported, and must be evaluated on a case-by-case basis, if required, as discussed in Section 3.7.9.2.



Notes:
1. This figure is reproduced from Reference 19, Figure 5-23

2. This figure is for comparison purposes only and shall not be used for design

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 3.7-29 SAMPLE FREE FIELD GROUND MOTION LTSP ANALYSIS LONGITUDINAL COMPONENT



Notes:
1. This figure is reproduced from Reference 19, Figure 5-24

2. This figure is for comparison purposes only and shall not be used for design

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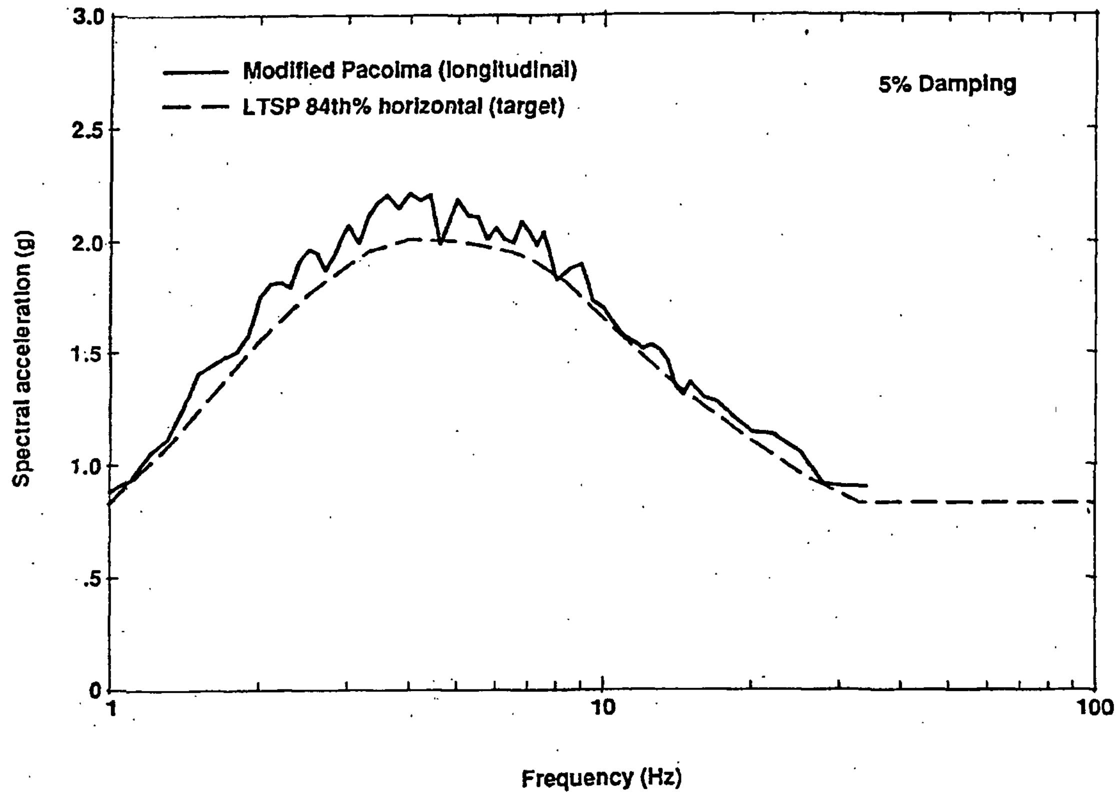
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 3.7-30 SAMPLE FREE FIELD GROUND MOTION LTSP ANALYSIS TRANSVERSE COMPONENT



Notes:
1. This figure is reproduced from Reference 33, Figure DE Q2-3

2. This figure is for comparison purposes only and shall not be used for design

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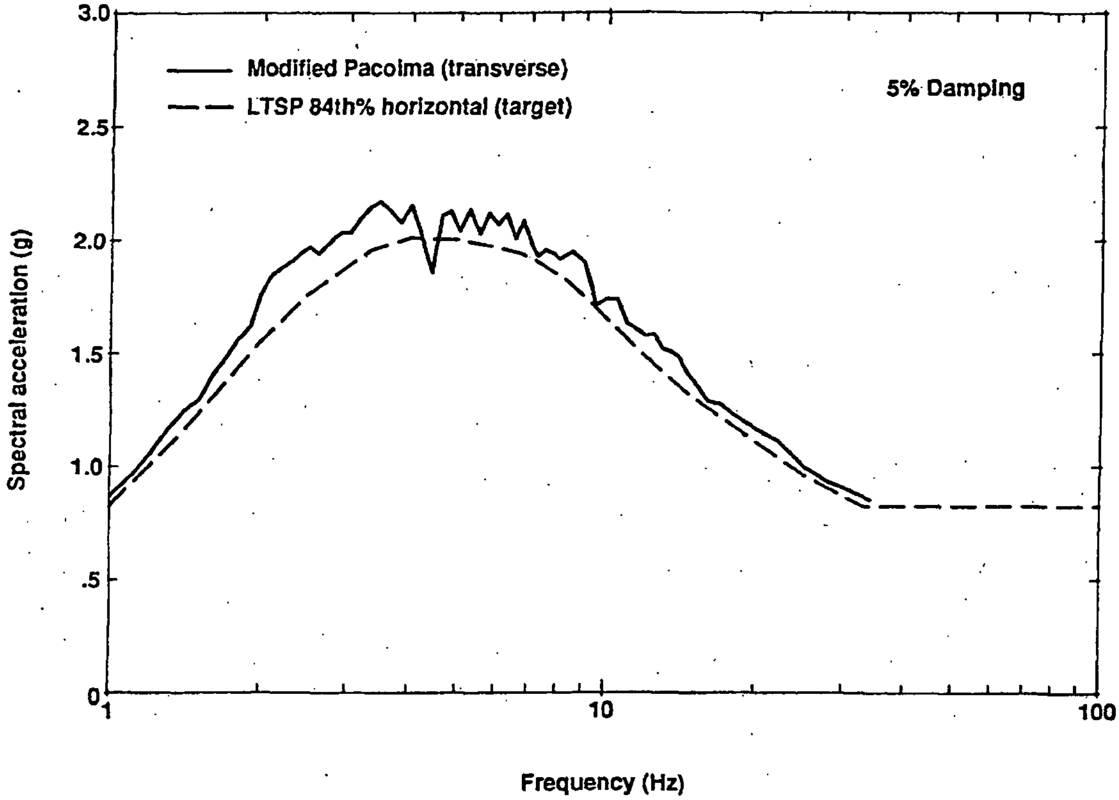
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 3.7-31 SAMPLE FREE FIELD GROUND MOTION COMPARISON TO TARGET SPECTRUM LTSP ANALYSIS LONGITUDINAL COMPONENT 5% DAMPING RATIO



Notes:
1. This figure is reproduced from Reference 33, Figure DE Q2-4

2. This figure is for comparison purposes only and shall not be used for design

FSAR UPDATE
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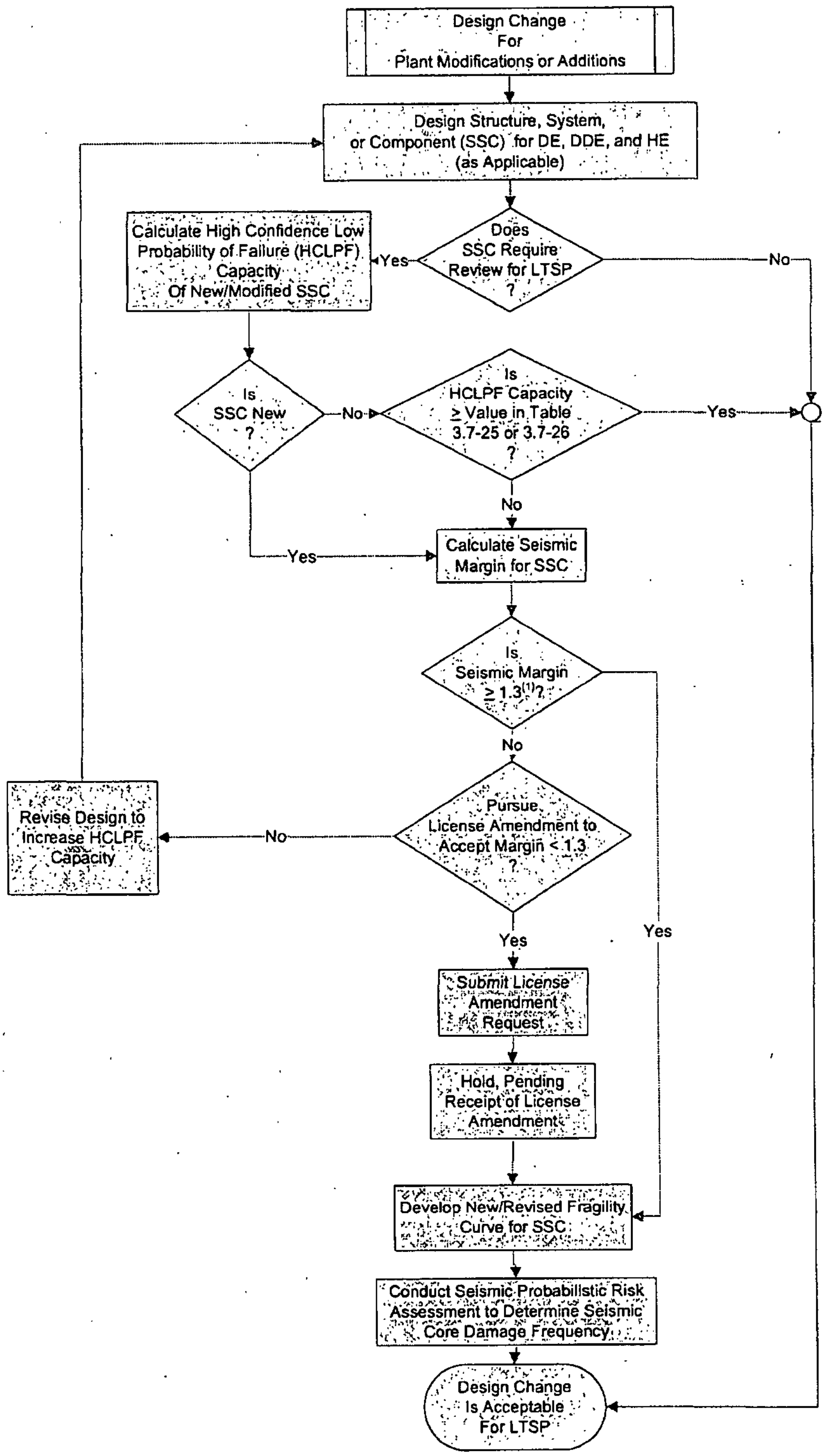
FIGURE 3.7-32 SAMPLE FREE FIELD GROUND MOTION COMPARISON TO TARGET SPECTRUM LTSP ANALYSIS TRANSVERSE COMPONENT 5% DAMPING RATIO



Notes:

1. A seismic margin below 1.3 is acceptable for certain SSCs (see Section 2.5.6.2.1.1). Therefore, redesign or a License Amendment Request is not required for these SSCs.

FSAR UPDATE
UNITS 1 AND 2 DIABLO CANYON SITE
FIGURE 3.7-33 LTSP Evaluation Process for Plant Additions and Modifications

3.8.1 CONTAINMENT STRUCTURE

3.8.1.1 Description of the Containment

The reactor containment for each unit is a steel-lined, reinforced concrete building of cylindrical shape with a dome roof that completely encloses the reactor and RCS. It ensures that essentially no leakage of radioactive materials to the environment would result even if gross failure of the RCS were to occur simultaneously with an earthquake of intensity twice the maximum postulated.

The containment structures for Units 1 and 2 are essentially identical, as mirror images. The following discussion applies to either unit:

The concrete outline and equipment locations are shown in Chapter 1. The exterior shell consists of a 142-foot-high cylinder, topped with a hemispherical dome. The minimum thickness of the concrete walls is 3.6 feet, and the minimum thickness of the concrete roof is 2.5 feet. Both have a nominal inside diameter of 140 feet and a nominal inside height of 212 feet. The concrete floor pad is 153 feet in diameter with a minimum thickness of 14.5 feet, with the reactor cavity near the center. The inside of the dome, cylinder, and base slab is lined with welded steel plate, which forms a leaktight membrane. The nominal thickness of the steel liner is 3/8-inch on the wall and dome and the nominal thickness of the steel liner on the base slab is 1/4-inch.
The containment is designed and will be maintained for a maximum internal pressure of 47 psig and a temperature of 271°F, coincident with a Double Design Earthquake or Hosgri Earthquake.

The internal concrete structure approximates a 106-foot-diameter, 51-foot-high cylinder, with a slab on top. However, there are multiple openings and walls, such as the reactor support and the stainless steel lined refueling canal, which complicate the shape. The walls and top slab are generally 3 feet thick. This structure provides support for the reactor and components of the RCS, provides radiation shielding, and provides protection for the liner from postulated missiles originating from the RCS.

A polar crane is mounted on top of the internal concrete cylinder wall. The support of the polar crane, its connection to the concrete, and provisions to resist seismic forces are shown in Figure 3.8-23 and described in Section 9.1.4. Seismic analysis for the polar crane is discussed in Section 3.7.

The piping and electrical connections between equipment inside the containment structure and other parts of the plant are made through specially designed, leaktight penetrations. In addition to the piping and electrical penetrations, other penetrations are the 18-foot 6-inch diameter equipment hatch, the 9-foot 7-inch diameter personnel hatch, the 5-foot 6-inch diameter personnel emergency hatch, and the fuel transfer tube.

The 6-foot 7-inch by 13-foot ventilation duct is attached to the outside of the structure, extending from an elevation 25 feet above the base slab to the top of the dome. The duct is fabricated from steel plate with stiffeners.

A system of lightning rods is installed on the dome to protect against lightning damage.

The following paragraphs describe the various parts of the structure:

3.9.3 CORE AND REACTOR INTERNALS

3.9.3.1 Core and Internals Integrity Analysis (Mechanical Analysis)

Stainless steel clad silver-indium-cadmium alloy absorber rods are resistant to radiation and thermal damage, thereby ensuring their effectiveness under all operating conditions. Rods of similar design have been successfully used in the original and reload cores of San Onofre, Connecticut Yankee, and others.

Two burnable poison rods (Reference 6) of smaller length but similar in design to those used in DCPP were exposed to in-pile test conditions in the Saxton Test Reactor in October 1967. A visual examination of the rods was made in early June 1968 and a visual and profilometer examination was made on July 30, 1968, after an exposure of 1900 effective full power hours (approximately 25 percent B^{10} depletion). The rods were found to be in excellent condition and profilometry results showed no dimensional variation from the initial condition.

An experimental verification of the reactivity worth calculations for borosilicate glass tubing has been accomplished. Similar rods have been successfully operated in the Ginna Reactor (Reference 7) with no evidence of deficiency.

Manufacturing defects did not appear during the hot functional tests because any manufacturing defects were detected in the shop or during the assembly period. The basic program that is currently being used to ensure adequacy of manufacturing practices consists of:

(1) Extremely thorough nil ductility temperature and quality assurance programs at the internals vendors

(2) Extensive visual examination at the plant site prior to hot functional testing of the primary system

(3) Running the hot functional test with full flow for 240 hours that accumulates approximately 10^7 cycles on the majority of the core structure components

(4) Reexamining all areas of the internals after the 240-hour hot functional test

The response of the reactor core and vessel internals under excitation produced by a simultaneous complete severance of a reactor coolant pipe and seismic excitation for a typical Westinghouse pressurized water reactor plant internals was determined. The following mechanical functional performance requirements applied:

(1) Following the DBA, the basic operational or functional requirement to be met for the reactor internals is that the plant shall be shut down and cooled in an orderly fashion so that fuel cladding temperature is kept within

specified limits. This implies that the deformation of certain critical reactor internals must be kept sufficiently small to allow core cooling.

(2) For large breaks, the reduction in water density greatly reduces the reactivity of the core, thereby shutting down the core whether the rods are tripped or not. The subsequent reflooding of the core by the ECCS with borated water maintains the core in a subcritical state. Therefore, the main requirement is to ensure effectiveness of the ECCS. Insertion of the control rods, although not needed, gives further assurance of the ability to shut the plant down and keep it in a safe shutdown condition.

(3) The functional requirements for the core structures during the DBA are shown in Table 3.9-10. The inward upper barrel deflections are controlled to ensure no contacting of the nearest rod cluster control guide tube. The outward upper barrel deflections are controlled in order to maintain an adequate annulus for the coolant between the vessel inner diameter and core barrel outer diameter.

(4) The rod cluster control guide tube deflections are limited to ensure operability of the control rods.

(5) To ensure no column loading of rod cluster control guide tubes, the upper core plate deflection is limited to the value shown in Table 3.9-10.

(6) The reactor has mechanical provisions that are sufficient to maintain the design core and internals and to ensure that the core is intact with acceptable heat transfer geometry following transients arising from the DBA operating conditions (References 2, 8, and 13).

(7) The core internals are designed to withstand mechanical loads arising from DE, DDE, HE, and pipe ruptures (References 2, 4, 8, and 13).

While these performance requirements originally had to be met for load combinations that included the contribution from a main RCS loop line break, with the acceptance of the DCPP leak-before-break analysis by the NRC (Reference 14), dynamic loads resulting from pipe rupture events in the main reactor coolant loop piping no longer have to be considered in the design basis structural analyses and included in the loading combinations; only the much smaller loads from RCS branch line breaks have to be considered (see Section 3.6.2.1.1.1).

3.9.3.5.1 Blowdown Forces Due to Cold and Hot Leg Break

A USNRC approved FORTRAN-IV computer program called MULTIFLEX (Reference 3) is *used to calculate the local fluid pressure, flow, and density transients that occur during a* LOCA. MULTIFLEX is an extension of the BLODOWN-2 computer code and includes mechanical structure models and their interaction with the thermal-hydraulic system.

The analysis is performed for the subcooled decompression period of the transient, where the hydraulic loads are the greatest. These loads are used for the structural evaluation of the reactor pressure vessel support system, in conjunction with other loads associated with a LOCA and with the Hosgri earthquake (HE). (Previous calculations using LOCA and DDE loads that bound the LOCA and HE loads would be conservative.)

System or Service Description	Valve Identification	FSAR Fig. No.	Body Type	Size in.	Actuator Type	Valve Position On Failure	Position for Safe Shutdown[(a)]	Failure Analysis Comments
MAIN STEAM LEAD FOUR 10% STEAM DUMP ISOLATION	MS-4015	3.2-4	Gate	8	Manual	NA	Closed Note 24	

(a) The valves whose positions are listed in this column are those valves whose operability is relied on to perform an active function such as safe shutdown of the reactor or mitigation of the consequences of a Design Basis Accident coincidental with loss of offsite power. An entry of "functional" or equivalently "operable" means that the valve must be capable of being opened and/or closed to perform its active function. For DCPP, safe shutdown is defined as Mode 3 following an accident (SSER 7 and SSER 22), Mode 5 following a Hosgri earthquake ~~(Section 3.7.6.2)~~, and Mode 3, followed by Mode 5 within 72 hours, following an Appendix R fire (10 CFR 50, Appendix R).

Failure Analysis Comment Notes:

1. Deleted in Revision 9.
2. Deleted in Revision 9.
3. Deleted in Revision 9.
4. Valve is provided for control. Failure, open or close, is remedied by redundant train and EOP RNO actions.
5. Valve provides isolation. Failure to close is remedied by valve in series.
6. Deleted in Revision 9.
7. Locally mounted air accumulators protected against compressed air system failure by check valves can hold open the main steam isolation valves for a short duration of time after the compressed air system is lost. In the event of loss of all air to the main steam isolation valves, the valves will fail closed.
8. These valves are provided for controlled steam release. Failure to open is remedied by redundant valves. Failure to close is remedied by closure of series valve or system shutdown.
9. These valves provide vessel protection. Failure to open is remedied by redundant valves in parallel. Valve size limits flow on failure to close.
10. Valve provides isolation. Failure to close (or stay closed) is remedied by a redundant valve in series. Failure to open (or stay open) is remedied by a redundant line (or system).
11. Valve opens to start device. Failure to open is remedied by use of redundant system.
12. Air-operated valve operation is not required for safe shutdown.
13. Used during recirculation mode.

Failure Analysis Comment Notes (continued)

14. Valve provides isolation. Failure to stay open could defeat system function. "Hot" short could close valve, but is not considered credible.

15. Deleted in Revision 9.

16. Deleted in Revision 9.

17. Deleted in Revision 9.

18. Deleted in Revision 9.

19. Valves operated (opened) during changeover from cold leg recirculation to hot leg injection. Failure to stay closed during cold leg injection or cold leg recirculation could defeat system function. "Hot" short could open valve but is not considered credible.

20. Valve 8809A operated (closed) during the changeover from cold leg injection to cold leg recirculation. Valve 8809B operated (closed) during the changeover from cold leg recirculation to hot leg recirculation. Failure of one valve to stay open during cold leg injection remedied by redundant system.

21. Air operated valves required to operate or maintain position after a loss of the compressed air system are supplied with compressed gas from the backup air/nitrogen supply system. See Section 9.3.1.6 for details.

22. If one of the CCW heat exchangers is valved out-of-service, then backup air is supplied to the respective CCW heat exchanger saltwater inlet valve to maintain the valve closed. This ensures all ASW flow is directed to in-service CCW heat exchangers.

23. Valve does not have an active safety function to support accident mitigation or Mode 3~~-safe shutdown~~. Valve is active to support achieving Mode 5 following a Hosgri earthquake and muste~~post-Hosgri cold shutdown in the manner defined in the Hosgri Report. Valve needs to~~ be seismically qualified for active function for Hosgri earthquake loading~~only~~.

24. Valve has an active safety function to support accident mitigation or Mode 3~~-safe shutdown~~. Valve is passive to support achieving Mode 5 following a Hosgri earthquake,~~post-Hosgri cold shutdown in the manner defined in the Hosgri Report.~~

25. Normal position for Safe Shutdown is Open. For Containment Isolation and the condition described in section 6.5.3.4, valve must be Operable.

Abbreviations:

FCV	= Flow control valve	RCP	= Reactor coolant pump	B'fly	= Butterfly
LCV	= Level control valve	FAI	= Fail as is	RC	= Reactor coolant
PCV	= Pressure control valve	PP & PPS	= Pump(s)	CCW	= Component cooling water
HCV	= Hand control valve	CNT	= Containment	RHR	= Residual heat removal
RV	= Relief valve	CHG	= Charging	AFW	= Auxiliary feedwater
TCV	= Temperature control valve	DSL FO	= Diesel fuel oil	NA	= Not applicable

3.10.2.7.1 4160 V Metal-Clad Switchgear

The original 4160 V metal-clad switchgear with General Electric (GE) 250 mVA 4.16 kV magneblast circuit breakers was seismically qualified by a combination of testing and analyses.

Later, it was discovered that 350 mVA circuit breakers should be used in place of the GE 250 mVA 4.16 kV magneblast circuit breakers. GE could not supply such breakers to the same switchgear. Consequently, PG&E decided to procure 350 mVA 4.16 kV breakers from NTS/PDS, which converted Japanese-made Yaskawa SF6 circuit breakers to fit the existing 4 kV switchgear. The new circuit breakers were installed during refueling outages 1R8 and 2R7.

New circuit breakers were seismically qualified by shake table testing (NTS report No. TR60431-95N-FR). The shake table testing was intended to achieve the following objectives.

(1) Demonstrate the structural integrity and functionality of the Yaskawa breakers.

(2) Demonstrate the structural integrity of as-installed 4 kV switchgear cubicles at DCPP with the Yaskawa breakers.

(3) Demonstrate the functional performance of the existing components (i.e., various relays and switches) installed in the existing 4 kV switchgear cubicles with replacement Yaskawa breakers.

(4) Instrument the test 4 kV switchgear cubicles with sufficient number of accelerometers to obtain accurate information on the dynamic response (response frequencies, test response spectra) at various cubicle locations. This information is to be used for further/future testing and analyses.

(5) Take immediate corrective actions to address significant anomalies observed during the test.

The initial seismic testing was performed at Wyle Laboratories in Huntsville, Alabama. Three seismic mock-up 4 kV switchgear cubicles were built to duplicate the design, material, and construction of cubicles G-5, G-12, and G-13 of Unit 1. A total of 18 ~~OBE~~ DE and ~~SSE~~ DDE/HE (envelope of the applicable DDE and HE response spectra) test runs were performed, including three runs of resonance search. Test results showed that the new breakers and mock-up cubicles successfully passed the minimum required 5 ~~OBE~~ DE tests.

For the ~~SSE~~ DDE/HE tests performed at Wyle Laboratories, excessive relay chatter at certain frequencies were noted. The excessive chatter was due to over-testing the equipment, which in turn was a result of Wyle Laboratories being unable to accurately control the test table response at 10 Hz and above due to resonance of the table. The over-test produced a significant amount of relay chatter, which caused the tripping and closing of breakers. The post test functional check showed that the breakers were functioning properly and had no structural damage.

To properly test the relays, supplemental ~~SSE~~ DDE/HE testing was performed at Farwell and Hendricks (F&H) Laboratories. The upper front doors of the G-12 and G-13 cubicles, where a majority of relays are mounted, were mounted on the F&H rigid test fixture. One 1200A breaker and one 2000A breaker, located adjacent to the test table, were fed by the relays. The ~~SSE~~ DDE/HE RRS obtained at relay locations on the G-12 and G-13 cubicles from the previous Wyle testing were reduced with the appropriate scaling factor to eliminate unnecessary over-testing. The supplemental ~~SSE~~ DDE/HE testing was successful. However, certain modifications (such as adding chokes to the breakers and removing the seal-ins from certain relays) were made when the new breakers were installed in the 4-kV switchgear.

Based on the above, the switchgear and its contents are qualified for the DE, DDE, and HE~~Hosgri, and LTSP postulated~~ seismic events at DCPP.

3.10.2.32.1 RVLIS/Incore Thermocouple Cabinets

Two RVLIS/incore thermocouple cabinets (PAMs 3 and 4) are provided for DCPP application. Located within each cabinet are the microprocessor electronics, reactor coolant pump (RCP) status panel, and a remote display. The above RVLIS instrumentation is only required to operate normally before and after seismic excitation. The RCP status panel assembly is shown to be operational by the signals recorded during testing and the functional checks made after each simulated ~~SSE~~ DDE/HE (envelope of the applicable DDE and HE response spectra). The remote display electronics must function normally by providing microprocessor output display formatted information.

The results of seismic testing of the original RVLIS/incore thermocouple cabinets are provided in Reference 27. The original remote display was not included in the cabinet tested. The original remote display was tested later to worst-case (maximum) in-cabinet response for the RVLIS/incore thermocouple cabinets. The seismic testing of the original remote display is documented in Reference 28.

Because the original Westinghouse-supplied system is obsolete and due to the lack of availability of replacement components, the obsolete RVLIS/incore thermocouple systems were replaced. The replacement processors, signal conditioners, and displays are seismically qualified by testing and analysis as documented in References 47 and 48 and PG&E Calculation IS-66.

TABLE 4.1-3

DESIGN LOADING CONDITIONS FOR REACTOR CORE COMPONENTS

(1) Fuel assembly weight

(2) Fuel assembly spring forces

(3) Internals weight

(4) Control rod scram (equivalent static load)

(5) Differential pressure

(6) Spring preloads

(7) Coolant flow forces (static)

(8) Temperature gradients

(9) Differences in thermal expansions

- (a) Due to temperature differences
- (b) Due to expansion of different materials

(10) Interference between components

(11) Vibration (mechanically or hydraulically induced)

(12) One or more loops out of service

(13) All operational transients listed in Table 5.2-4

(14) Pump overspeed

(15) Seismic loads (DE, ~~and~~ DDE, and HE)

(16) Blowdown forces (due to RCS branch line breaks)(a)

(a) In the original analysis, the blowdown forces used were those resulting from breaks in the RCS cold and hot legs. However, with the acceptance of the DCPP leak-before-break analysis by the NRC, the blowdown forces resulting from pipe rupture events in the main reactor coolant loop piping no longer have to be considered in the design basis structural analyses and included in the loading combinations. Only the much smaller forces from RCS branch line breaks have to be considered (see Section 3.6.2.1.1.1).

5.2.1.5.4 Faulted Conditions

The following transients are considered faulted conditions:

(1) RCS Boundary Pipe Break

This accident involves the postulated rupture of a pipe belonging to the RCS boundary. It is conservatively assumed that system pressure is reduced rapidly and the emergency core cooling system (ECCS) is initiated to introduce water into the RCS. The safety injection signal will also initiate a turbine and reactor trip.

The criteria for locating design basis pipe ruptures for the design of RCS supports and restraints, thus ensuring continued integrity of vital components and engineered safety features (ESF), are presented in Section 3.6. They are analyzed in Reference 7. With the acceptance of the DCPP leak-before-break analysis by the NRC (Reference 31), the dynamic effects of breaks in the main reactor coolant loop piping no longer have to be considered in the design basis analyses. Only the dynamic effects from RCS branch line breaks have to be considered (see Section 3.6.2.1.1.1).

(2) Steam Line Break

For RCS component evaluation, the following conservative conditions are considered:

(a) The reactor is initially in hot, zero power subcritical condition assuming all rods in, except the most reactive rod, which is assumed to be stuck in its fully withdrawn position.

(b) A steam line break occurs inside the containment.

(c) Subsequent to the break, there is no return to power and the reactor coolant temperature cools down to 212°F.

(d) The ECCS pumps restore the reactor coolant pressure.

The above conditions result in the most severe temperature and pressure variations that the component will encounter during a steam break accident.

The dynamic reaction forces associated with circumferential steam line breaks are considered in the design of supports and restraints to ensure continued integrity of vital components and ESFs. Criteria for protection against dynamic effects associated with pipe breaks are covered in Section 3.6.

(3) Double Design Earthquake (DDE)

The mechanical stress resulting from the DDE is considered on a component basis. As discussed in Sections 2.5.2.9.2 and 3.2.1, the DDE is part of the original ~~The~~ design basis for the plant~~ is the DDE~~ and is still applicable to the design of the reactor coolant system. The seismic analysis is described in Section.3.7.

(4) Hosgri Earthquake

As discussed in Sections 2.5.2.9.3 and 2.5.2.10.3, s~~S~~tudies subsequent to the original seismological survey of the site region have resulted in the development of the Hosgri earthquake, producing ground motions at DCCP greater than those associated with the DDE. ~~a postulated earthquake of greater magnitude.~~ The characteristics and consequences of the~~is~~ ~~postulated~~ Hosgri earthquake are discussed in Section 5.2.1.15.

The design transients and the number of cycles of each are shown in Table 5.2-4.

5.2.1.7 Design of Active Pumps and Valves

The design criteria for active safety-related pumps outside the RCS boundary are discussed in Section 3.9.2. All these safety-related pumps are designated either ASME B&PV Code Class II or III.

Active pumps were qualified for operability by first being subjected to rigid tests both prior to installation in the plant and after installation in the plant. The in-shop test included (a) hydrostatic tests of pressure-retaining parts to 150 percent of the product of the design pressure times the ratio of material allowable stress at room temperature to the allowable stress value at the design temperature, (b) seal leakage tests, and (c) performance tests to determine total developed head, minimum and maximum head, net positive suction head (NPSH) requirements and other pump parameters. Bearing temperature and vibration levels were monitored during these operating tests. Bearing temperature limits and vibration levels were established by the manufacturer based on bearing materials, clearances, oil type and rotational speed. After a pump was installed in the plant, it underwent cold hydrostatic tests, and hot functional tests, and will undergo the required periodic inservice inspection operation. These tests demonstrated that a pump will function as required during all normal operating conditions for the design life of the plant.

In addition to these tests, the active pumps were qualified for operability by assuring that they will start, continue operating and not be damaged during the ~~postulated~~ Hosgri earthquake.

It was shown that the pumps will perform their design functions when subjected to loads imposed by the maximum seismic accelerations and maximum nozzle loads. It was required that test or analysis be used to show that the lowest natural frequency of each pump was greater than 33 Hz. A pump having a natural frequency above 33 Hz was considered rigid. This consideration avoids amplification between the component and structure for all seismic areas. A static shaft deflection analysis of rotors was performed with horizontal and vertical accelerations acting simultaneously. The deflections, determined from the static shaft analyses, were compared to the allowable rotor clearances. Pump and motor bearing loads were determined and shown to be below the manufacturer's specified levels.

To avoid damage during the postulated earthquake, the stresses caused by the combination of normal operating loads, earthquake, and dynamic system loads were limited to the limits indicated in Section 3.9.2. Pump casing stresses caused by the maximum nozzle loads were limited to the stresses outlined in Section 3.9.2. The maximum seismic nozzle loads combined with the loads imposed by the seismic accelerations were considered in the analysis of pump supports. Furthermore, calculated misalignment was shown to be less than that which could hinder pump operation. The stresses in the supports were below those of Section 3.9.2. Therefore, support distortion is elastic and of short duration (no longer than the duration of the seismic event).

Performing these analyses with the loads and the stress limits of Section 3.9.2, assures that critical parts of pumps will not be damaged during the postulated earthquake.

If the natural frequency was found to be below 33 Hz, an analysis was performed to determine the amplified input accelerations necessary to perform the static analysis. The adjusted accelerations were determined with the same conservatisms used for rigid structures. The static analysis was performed using the adjusted accelerations; the stress limits stated in Section 3.9.2 were satisfied.

To complete the seismic qualifications procedures, the pump motors were qualified for operation during the maximum seismic event. Any auxiliary equipment which is vital to the operation of the pump or pump motor, and which was not qualified for operation with the pump or motor was qualified separately.

The above program gives assurance that the active pumps and motors would not be damaged and would continue operating under seismic loadings. These requirements demonstrate that the active pumps will perform their intended functions.

Since it has been demonstrated that the pumps would not be damaged during the earthquake, the functional ability of the active pumps after the earthquake is assured. Normal operating loads and steady state nozzle loads are the most probable conditions following an earthquake. The ability of the pumps to function under these loads is demonstrated during normal plant operation.

The valves were designed to function at normal operating conditions, maximum design conditions, and DDE/Hosgri conditions. Active valves that serve a post-earthquake safe shutdown and/or ~~are used for~~ an accident mitigation function~~only, and do not serve to support safe shutdown following a Hosgri earthquake,~~ were qualified for active function for a Hosgri earthquake ~~to provide increased conservatism in accordance with~~ (Reference 30). The design meets the requirements of the ANSI B31.1, ANSI B16.5, and MSS-SP-66 codes.

The stress limits for the valves in the RCS pressure boundary are indicated in Table 5.2-5. The design criteria and allowable stress limits for safety-related valves outside the RCS pressure boundary (i.e., valves considered to be ASME B&PV Code Class II or III components) are indicated in Section 3.9.2.

In addition, all valves 1 inch and larger within the RCPB were checked for wall thickness to ANSI B16.5, MSS-SP-66, or ASME B&PV Code, Section III (1968, some 1974) requirements, as applicable, and subjected to nondestructive tests in accordance with ASME and ASTM codes.

The valves were designed to the requirements of ANSI B16.5 or MSS-SP-66 pertaining to minimum wall thickness for pressure containing components. Analyses were performed to qualify active valves. These valves were subjected to a series of stringent tests prior to service and during the plant life. Prior to installation, the following tests were performed: shell hydrostatic tests to MSS-SP-61 requirements, backseat and main seat leakage tests. Cold hydrostatic tests, hot functional qualification tests, periodic inservice inspections and operability tests have been and are performed to verify and assure the functional ability of the valves. These tests assure reliability of the valves for the design life of the plant.

On all active valves, an analysis of the extended structure was performed for static equivalent seismic loads applied at the center of gravity of the extended structure. The minimum stress limits allowed in these analyses will assure that no significant permanent damage occurs in the extended structures during the earthquake.

Motor operators and other electrical appurtenances necessary for operation were qualified.

The natural frequencies of all active valves were determined by test or by analysis. If the natural frequencies of the valves were shown to be less than 33 Hz, one of the following options was employed:

(1) The valve was qualified by dynamic testing.

(2) The valve was modified to increase the minimum frequency to greater than 33 Hz.

(3) The valve was qualified conservatively using static accelerations that are sufficiently in excess of accelerations it might experience in the plant to take into account any effect due to both multifrequency excitation and multi-mode response (a factor of 1.5 times peak acceleration is generally accepted, although lower coefficients can be used when shown to yield conservative results).

(4) A dynamic analysis of the valve was performed to determine the equivalent acceleration to be applied during the static analysis. The analysis provided the amplification of the input acceleration considering the natural frequency of the valve and the frequency content of the applicable plant floor response spectra. The adjusted accelerations were then used in the static analysis and the valve operability was assured by the methods outlined above, using the modified acceleration input.

Swing check valves are characteristically simple in design and their operation is not affected by seismic accelerations or applied nozzle loads. The check valve design is compact and there are no extended structures or masses whose motion could cause distortions which could restrict operation of the valve. The nozzle loads due to seismic excitation do not affect the functional ability of the valve since the valve disc is typically designed to be isolated from the casing wall. The clearance available around the disc prevents the disc from becoming bound or restricted due to any casing distortions caused by nozzle loads. Therefore, the design of these valves is such that once the structural integrity of the valve is assured using standard design or analysis methods, the ability of the valve to operate is assured by the design features. For the faulted condition evaluations, since piping stresses are shown to be acceptable, the check valves are qualified.

The valves have undergone the following tests: (a) in-shop hydrostatic test, (b) in-shop seat leakage test, and (c) periodic in-plant exercising and inspection to assure functional ability.

By the above methods, all active valves are qualified for operability for the faulted condition seismic loads. These methods simulate the seismic event and assure that the active valves will perform their safety-related functions when necessary.

5.2.1.11 Analysis Method for Faulted Condition

The analysis of the RCLs and support systems for blowdown loads resulting from a LOCA is based on the time-history response of simultaneously applied blowdown forcing functions on a broken and unbroken loop dynamic model. The forcing functions are defined at points in the system loop where changes in cross section or direction of flow occur such that differential loads are generated during the blowdown transient. Stresses and loads are checked and compared to the corresponding allowable stress.

The stresses in components resulting from normal sustained loads and the worst case blowdown analysis are combined with the DDE seismic analysis (see Section 5.2.1.15 for a discussion of the Hosgri seismic analysis) to determine the maximum stress for the combined loading case. This is considered a very conservative method since it is highly improbable that both maxima will occur at the same instant. These stresses are combined to ensure that the main reactor coolant piping loops and connected primary equipment support system will not lose their intended functions under this highly improbable situation.

For faulted conditions, the limits are provided in Table 5.2-7.

Further details of the stress analysis for faulted conditions are presented in Section 5.2.1.14. With the acceptance of the DCPP leak-before-break analysis by the NRC (Reference 31), the dynamic thrust forces and blowdown loads resulting from pipe rupture events in the main reactor coolant loop piping no longer have to be considered in the design basis analyses. Only the thrust forces and blowdown loads resulting from RCS branch line breaks have to be considered (see Section 3.6.2.1.1.1). For the RCL reanalysis performed for the RSGs, thrust forces and blowdown loads were determined for RCS branch line breaks identified in Section 5.2.1.10.1. Details of the stress analyses performed to evaluate the effects of the postulated Hosgri earthquake are presented in Section 5.2.1.15.

Protection criteria against dynamic effects associated with pipe breaks are covered in Section 3.6. With the acceptance of the DCPP leak-before-break analysis by the NRC (Reference 31), the dynamic effects of breaks in the main reactor coolant loop piping no longer have to be considered in the design basis analyses. Only the dynamic effects from RCS branch line breaks have to be considered (see Section 3.6.2.1.1.1).

5.2.1.14 Stress Analysis for Faulted Condition Loadings (DDE and LOCA)

Stress analyses of the RCS for faulted conditions employ the displacement (stiffness) matrix method and lumped-parameter, multimass representation of the system. The analyses are based on adequate and accurate representation of the system using an idealized, mathematical model. See Section 5.2.1.15 for a discussion of the faulted condition loading associated with Hosgri and LOCA.

5.2.1.15 Stress Analysis for Faulted Condition Loadings (Hosgri and LOCA)

This section describes the supplemental analyses of the faulted condition load combination to address the Hosgri earthquake. Differences between this analysis, and the faulted condition load combination evaluation for DDE and LOCA described in Section 5.2.1.14 are discussed.

5.2.1.15.1 Integrated Reactor Coolant Loop Analysis

Analysis of the reactor coolant loop piping was performed using the response spectra method. The RCL model was constructed for the WESTDYN computer program.

The horizontal response spectrum at 140 feet in the inner containment structure, corresponding to the steam generator upper support elevation, and the horizontal spectrum at 114 feet in the inner containment structure, corresponding to the reactor coolant pump support and reactor vessel elevation, was used in the analysis. A vertical response spectrum envelope from elevation 114 ft to the base slab of elevation 87 ft was used in the analysis. With mode, the results due to the vertical shock were combined by direct addition with the results of the horizontal shock directions. The modal contributions were then added by the square-root-sum-of-the-squares (SRSS) method.

Two seismic cases were considered; north-south plus vertical and east-west plus vertical. Each horizontal shock was combined with the vertical shock and the worst combined response was used in the evaluation of the system.

The stresses and loads associated with the LOCA loading case are taken from the analysis described in Section 5.2.1.14.1.

The results of the analysis are as follows: The results of the Hosgri seismic evaluation were combined with the pressure, ~~and~~ deadweight, and LOCA stresses. The ~~revised~~ combined piping stresses were all under the allowable of 2.4 S_h, or, for loop piping, 3.6 S_h.

5.2.1.15.2 Steam Generator Evaluation

The seismic spectra at the elevations of the steam generator upper support and vertical support were used as the seismic input. The horizontal spectra at the upper support and the vertical spectra at the vertical support were used as input. The model was used to evaluate the shell, tube bundles, upper and lower internals, and other pressure boundary components.

The nozzles and support feet of the steam generator were analyzed using static stress analysis methods with externally applied design loads. Loadings on the inlet and outlet nozzles of the steam generator for the Hosgri earthquake were calculated as part of the reactor coolant loop piping analysis. The stresses and loads associated with the LOCA loading case are taken from the analysis described in Section 5.2.1.14.1. The results of the Hosgri seismic evaluation were combined with the pressure, deadweight, and LOCA stresses. The combined loadings calculated by this analysis were compared with ~~previous~~ faulted condition loads associated with the DDE and LOCA combination. The ~~new~~ Hosgri and LOCA faulted condition loads were shown to be lower than the loads that were used initially to evaluate the nozzles or was shown to be less than the applicable stress allowable. Therefore, the stresses ~~caused by~~associated with the Hosgri ~~spectra~~earthquake are within the design basis of these nozzles.

The loads on the steam generator support feet and upper seismic support were ~~supplied for~~taken from the Hosgri evaluation by the reactor coolant loop analysis. The LOCA loads were taken from the analysis described in Section 5.2.1.14.1. The results of the Hosgri seismic evaluation were combined with the deadweight, pressure, and LOCA loads. These combined loadings are below the loading originally calculated for the DDE and LOCA faulted condition analysis or was shown to be less than the applicable stress allowable.

~~A long-term seismic program (LTSP) seismic margin assessment was performed by Westinghouse for the DCPP RSGs and associated supports. The assessment shows that the limiting LTSP seismic margin for the components affected by the RSGs is greater than the controlling value of 3.06 contained in the LTSP final report (Reference 34). In addition, the assessment confirms a minimum elastic seismic margin scale factor (FS_E) greater than 1.65 for RSG components. A lower value of FS_E (1.33) was calculated for the RSG vertical support; however, the resulting 84 percent nonexceedance high confidence, low probability of failure is greater than 3.06 (i.e., 3.22 g), when the standard ductility factor of 1.25 is applied. Details of the margin assessment are provided in Supplement 1 to Reference 33.~~

~~An LTSP seismic margin assessment was also performed for the Unit 2 RSG support anchorages. An FS_E of 1.31 corresponding to an LTSP seismic capacity of 2.6 g was determined for the RSG vertical support anchorages. Higher LTSP seismic capacities were calculated for the RSG upper and lower support anchorages.~~

5.2.1.15.3 Reactor Coolant Pump Evaluation

The Hosgri seismic analyses of the reactor coolant pump were performed using dynamic modal methods with a finite element computer program. The seismic response spectra corresponding to the elevation of the reactor coolant pump support structure were used. The LOCA loads were taken from the analysis described in Section 5.2.1.14.1. The results of the Hosgri seismic evaluation were combined with the deadweight, pressure, and LOCA loads.

The nozzles and support feet of the reactor coolant pump were analyzed by static stress analysis methods with externally applied design loads. For the ~~Hosgri spectra~~ faulted condition including Hosgri seismic loads, the combined external loads applied to the inlet and outlet nozzles of the reactor coolant pump by the reactor coolant loop piping

~~are all below the load for which the nozzles previously were shown acceptable. No further analysis was necessary for the nozzles.~~resulted in the code stress allowables being met.

The loads resulting from piping reactions for the ~~Hosgri spectra~~Hosgri and LOCA faulted load combination resulted in the code stress allowables being met. ~~were lower than the DDE loads for which the reactor coolant pump support feet were analyzed. No further analysis was necessary for the support feet.~~

5.2.1.15.4 Reactor Vessel Evaluation

Several portions of the reactor vessel were evaluated using static stress analysis methods with externally applied design loads. The control rod drive mechanism head adapter, closure head flange, vessel flange, closure studs, inlet nozzle, outlet nozzle, vessel support, vessel wall transition, core barrel support pads, bottom head shell juncture and bottom head instrumentation penetrations were analyzed by this method. The design loads for all areas evaluated except the inlet and outlet nozzles and vessel supports were chosen to be more conservative than any actual load the component would ever experience. The design loads for the inlet and outlet nozzles and vessel supports were umbrellas of loads experienced by past plants. In cases where the actual plant loads exceed the design loads, separate analyses were performed to assure adequacy. All stresses and fatigue usage factors were found to be acceptable

The LOCA loads were taken from the analysis described in Section 5.2.1.14.1. The results of the Hosgri seismic evaluation of the reactor coolant loop were combined with the deadweight, pressure, and LOCA loads and code stress allowables were met. ~~The Hosgri loads calculated by the reactor coolant loop analysis were compared with the DDE seismic loads and are lower. Thus, the previous reactor vessel analysis ensures adequacy for the Hosgri seismic event.~~

5.2.1.15.5 Reactor Vessel Internals Evaluation

The reactor vessel internals evaluation is presented in Section 3.7.3.15.

5.2.1.15.6 Fuel Assembly Evaluation

The fuel assembly evaluation is presented in Section 3.7.3.15.

5.2.1.15.7 Control Rod Drive Mechanism and Support System Evaluation

The evaluation of the control rod drive mechanism and its support system is presented in Section 3.7.3.15.

5.2.1.15.8 Primary Equipment Support Evaluation

Reactor coolant system component supports were shown adequate for the Hosgri seismic event by evaluating the supports for the loads determined in the integrated reactor coolant loops seismic analysis.

The STASYS and NASTRAN computer programs were used to obtain support stiffness matrices and member influence coefficients for the equipment supports.

Loads acting on the supports obtained from the reactor coolant loop analysis, support structure member properties, and influence coefficients at each end of each member were input to the THESSE program. The LOCA loads were taken from the analysis described in Section 5.2.1.14.1. The results of the Hosgri seismic evaluation of the integrated reactor cool loop analyses were combined with the deadweight, pressure, and LOCA loads.

A finite element stress analysis of the steam generator upper support structure was performed with the WECAN (Reference 18) computer program. The STRUDL program was used to analyze the pressurizer support frame.

In summary, stresses in all reactor coolant system component support members are below yield and buckling values for the Hosgri ~~seismic event~~and LOCA faulted load condition. The integrity of the supports has therefore been demonstrated for this ~~postulated event~~loading combination.

5.2.1.15.9 Pressurizer Evaluation

Hosgri seismic loading on the pressurizer is based on 4 percent damped Hosgri response spectra at elevation 140 ft. on the containment interior concrete structure. When the Hosgri seismic loads are combined with the deadweight, pressure, and LOCA loads, the total loading met code allowable stresses.~~The Hosgri response spectra for 4 percent damping at the 140 ft. elevation has a peak of 5 g horizontally, well below the value used to qualify the pressurizer. Therefore, the original pressurizer analysis is conservative for the Hosgri earthquake.~~

A dynamic reactor coolant loop analysis, which included a surge line model and was performed with the Hosgri response spectra, produced total loads (forces and moments), when combined with deadweight, pressure, and LOCA loads, on the support skirt, surge nozzle, and upper seismic lug which met code allowable stresses.~~were less than those produced by the original surge line analysis. Therefore, the loads on these components are acceptable.~~

TABLE 5.2-6

LOAD COMBINATIONS AND STRESS CRITERIA FOR WESTINGHOUSE PRIMARY EQUIPMENT [(a)]

CONDITION	LOAD COMBINATION	STRESS CRITERIA[(e)]
Design	Deadweight + Pressure ± DE	$P_m \leq S_m$ $P_L + P_b \leq 1.5\ S_m$
Normal	Deadweight + Pressure + Thermal	$P_L + P_b + P_e + Q \leq 3\ S_m$ [(b)]
Upset - 1	Deadweight + Pressure + Thermal ± DE	$U_T \leq 1.0$ [(b)] $P_L + P_b + P_e + Q \leq 3\ S_m$
	Deadweight + Pressure + Thermal	$U_T \leq 1.0$ [(b)] $P_L + P_b + P_e + Q \leq 3\ S_m$
Faulted - 1	Deadweight + Pressure ± DDE	Table 5.2-7
Faulted - 2	Deadweight + Pressure ± DDE + LPR[(c, d, g)]	Table 5.2-7
Faulted - 3	Deadweight + Pressure ± Hosgri + LPR[(c, d, g)]	Table 5.2-7
Faulted - 4	Deadweight + Pressure + Other Pipe Rupture[(f)]	Table 5.2-7

(a) Steam generators, reactor coolant pumps, pressurizer.
(b) Based on elastic analysis. For simplified elastic-plastic analysis, the stress limits of the 1971 ASME Code Section III, NB-3228.3 apply.
(c) LPR = reactor coolant loop pipe rupture
(d) DDE or Hosgri and LPR combined by SRSS method
(e) For definition of stress criteria terms, see Additional Notes.
(f) Pipe rupture other than LPR.
(g) While the original stress analysis considered this load combination, with the acceptance of the DCPP leak-before-break analysis by the NRC, loads resulting from ruptures in the main reactor coolant loop no longer have to be considered in the design basis structural analyses and included in the loading combinations, only the loads resulting from RCS branch line breaks have to be considered.

P_m = General membrane; average primary stress across solid section. Excludes discontinuities and concentrations. Produced only by mechanical loads.
P_L = Local membrane; average stress across any solid section. Considers discontinuities, but not concentrations. Produced only by mechanical loads.
P_b = Bending; component of primary stress proportional to distance from centroid of solid section. Excludes discontinuities and concentrations. Produced only by mechanical loads.
P_e = Expansions; stresses which result from the constraint of "free end displacement" and the effect of anchor point motions resulting from earthquakes. Considers effects of discontinuities, but not local stress concentration. (Not applicable to vessels).
Q = Membrane Plus Bending; self-equilibrating stress necessary to satisfy continuity of structure. Occurs at structural discontinuities. Can be caused by mechanical loads or by differential thermal expansion. Excludes local stress concentrations.
U_T = Cumulative usage factor.

TABLE 5.2-8

LOADING COMBINATIONS AND ACCEPTANCE CRITERIA FOR PRIMARY EQUIPMENT SUPPORTS

CONDITION	LOADING COMBINATIONS	STRESS LIMITS
Normal	Deadweight + Temperature + Pressure	1969 AISC Specification, Part 1
Upset	Deadweight + Temperature + Pressure ± DE	1969 AISC Specification, Part 1
Faulted - 1	Deadweight + Pressure ± DDE + LPR[a, b, f]	1969 AISC Specification, Part 2[c] or S_y after load redistribution, whichever is higher
Faulted - 2	Deadweight + Pressure ± HOSGRI + LPR[a, b, f]	1969 AISC Specification, Part 2[c] or S_y[e] after load redistribution, whichever is higher
Faulted - 3	Deadweight + Pressure + Other Pipe Rupture[d]	1969 AISC Specification, Part 2[c] or S_y after load redistribution, whichever is higher

(a) LPR = Reactor coolant loop pipe rupture.
(b) DDE or HOSGRI and LPR combined by SRSS method (or more conservative method).
(c) For supports qualified by load test, allowable loads = 0.8 times L_t per Table 5.2-7.
(d) Pipe rupture other than LPR.
(e) For the pressurizer upper lateral supports and the reactor vessel supports, the allowable S_y is based on average value of actual yield stress of the material.
(f) While the original stress analysis considered this load combination, with the acceptance of the DCPP leak-before-break analysis by the NRC, loads resulting from ruptures in the main reactor coolant loop no longer have to be considered in the design basis structural analyses and included in the loading combinations, only the loads resulting from RCS branch line breaks have to be considered.

TABLE 5.2-16

REACTOR COOLANT BOUNDARY LEAKAGE DETECTION SYSTEMS

Radioactivity Detection Systems

Detector Location or Process	Medium	Type	Range	Approximate Time to Detect 1-gpm Leak	Identified[c] Leak Detection	Instrument Class[a] ~~Seismic[a] Category~~	Indicator in Control Room
Containment	Air	G-M	10^{-1} to 10^{4} mR/hr	Less responsive than other detection systems	No	II	Yes
Incore inst area	Air	G-M	10^{-1} to 10^{4} mR/hr	Less responsive than other detection systems	No	II	Yes
Containment air particulate	Air	NaI Scintillator	10 to 10^{6} cpm	See Fig. 5.2-9	No	II[b]	Yes
Containment radiogas	Air	G-M	10 to 10^{6} cpm	See Fig. 5.2-9	No	II[b]	Yes
Plant vent radiogas	Air	Beta Scintillator	10 to 5E6 cpm	Less responsive than other detection systems	No	II	Yes
Condenser air ejector	Air	Beta Scintillator	10 to 5E6 cpm	See Fig. 5.2-10	Yes	II	Yes
Component cooling liquid	Liquid	NaI Scintillator	10 to 10^{6} cpm	See Fig. 5.2-12	No	IC	Yes
Steam generator blowdown	Liquid	NaI Scintillator	10 to 10^{6} cpm	See Fig. 5.2-11	Yes	II	Yes

TABLE 5.2-16

Other Detection Systems

Detector Location or Process	Medium	Type	Range and Repeatability[e]	Approximate Time to Detect 1-gpm Leak[q]	Identified[c] Leak Detection	Instrument Class[a] ~~Seismic[a] Category~~	Indicator in Control Room
Containment[d] condensation	Liquid	Change in time required to accumulate fixed volume	see note (m)	1 hr [g][h][l]	No	II	Yes
Containment sumps	Liquid	Liquid level and quantity of liquid	1 to 48 in. W.C. [n] 1 to 35 in. W.C. [p] ±1 in.	<1 hr [h]	No	II	Yes
Reactor vessel flange leakoff	Liquid	Temperature	50 to 300 °F ±5 °F	<30 sec [f]	Yes	II	Yes
Reactor coolant drain tank	Liquid	Liquid level and quantity of liquid	0-100% ±2%	<20 min [h]	Yes	II	No
Pressurizer relief valve discharge	Liquid	Temperature	50 to 400 °F ±7 °F	<30 sec [f]	Yes	II	Yes
Pressurizer relief tank	Liquid				Yes	II	Yes
		Liquid level	0 to 100 % ±2%	<12 hrs [h]			

Systems Used to Quantify Leakage [i]

Detector System	Medium	Type	Range/Sensitivity	Instrument Class[a] ~~Seismic Category~~	Indicated in Control Room
Pressurizer level	Liquid	Liquid level	0 to 100% [g][j] ~125 gal/% level	I	Yes
Volume control tank level	Liquid	Liquid level	0 to 100% [g][j] ~19 gal/% level	II	Yes
Charging pump flow	Liquid	Flow	0 to 200 gpm [k] ± 10% span when flow >60 gpm (channel uncertainty value)	II	Yes
Pressurizer relief tank level	Liquid	Liquid level	0 to 100% [h] min. 127, max. 154 gal/% level (20 < % level < 80)	II	Yes

(a) See Section 7.1 for the definition of Instrument Class. Instrument Class ~~Seismic Category~~ I systems are designed to perform required safety functions following a DDE or HE (whichever is larger). Instrument Class ~~Category~~ II instrument systems were designed to function under conditions up to DE. Instrument Class IC instrument systems refer to maintenance of pressure boundary integrity of Category I fluid systems. Also refer to Section 3.2.

(b) These units were not constructed to withstand DDE accelerations; however, they will be housed in a ~~Seismic Category~~Design Class I structure and protected from external damage associated with a seismic event. Therefore, it is considered that these units can be returned to operational status within 36 hours of a DDE or HE.

(c) Leakage is defined as identified or unidentified in accordance with Regulatory Guide 1.45.

(d) Containment condensation measures moisture condensed by the fan cooler drip collection system.

(e) Repeatability, including the operators ability to read the same value at another time, is included in this column; this is a true measure of ability to detect a change in system conditions over a period of time.

(f) Automatically alarmed.

TABLE 5.2-16

(g) Requires operator action - (i.e., close valve, start-stop pump, etc., and operator monitoring and logging).

(h) Requires operator monitoring and logging to note changes in rate, level, flow, etc.

(i) Systems listed here would be used to quantify true leakage rate in the event systems listed on Sheets 1 & 2 above detected an unidentified leak. These systems also provide additional capability for detecting leak rates of 1-gpm within short periods of time.

(j) Normal variations in process variable or automatic control systems will mask this change. Operator must take action as in (g) above to detect leakage.

(k) Insufficient accuracy/repeatability to ever detect a 1-gpm change in flowrate.

(l) Dependent on initial conditions. May take longer for fan cooler drip level if humidity is initially low.

(m) Level switches (HI and HI-HI) are provided in each CFCU drain line. The level switches have a fixed location in each drain line providing a repeatable alarm. The time intervals between the receipt of the HI level and HI-HI level alarms are monitored and logged by the operator. Alarm intervals less than a conservative pre-defined value directs the operator to perform an RCS water inventory balance to quantify the RCS leakage rate.

(n) This range refers to the containment structure sumps.

(o) Not used.

(p) This range refers to the reactor cavity sump.

(q) This column refers to the capability of the detection system to sense a leak.

6.3.1.4.3 Seismic Requirements

The ECCS is designed to perform its function of ensuring core cooling and providing shutdown capability following an accident ~~under~~ with simultaneous seismic (larger of DDE and HE) loading. ~~ECCS operability during and following a Hosgri event has been verified.~~ The seismic requirements are defined in Sections 3.7 and 3.10.

9.1.1.2 Facilities Description

There are two new fuel storage racks for each unit. A rack is approximately 9 feet 6 inches wide, 13 feet long, and 13 feet 6 inches high (excluding centering cones). It is built from Type 304 stainless steel.

The storage cells in the racks are in seven rows, five deep, and are spaced to have a nominal center-to-center distance of 22 inches. They are of Type 304 stainless steel and have a cone shaped top entrance to facilitate loading of fuel elements. They are shaped in a 9-inch square (cross section) hollow beam configuration, standing upright. At the base, they have a 1-inch thick bearing plate made of neoprene-impregnated fabric.

The new fuel storage racks and the anchorage of racks to the floor are designed for the design earthquake (DE), ~~and~~ double design earthquake (DDE), and Hosgri Earthquake (HE) loading conditions ~~and checked for a postulated Hosgri seismic event (Reference 1)~~with the racks containing fuel assemblies at the corners.

The racks are designed to withstand a vertical (uplift) force of 4000 pounds in the unlikely event that an assembly would bind in the rack while being lifted by the spent fuel bridge crane.

[redacted]

TABLE B-1

COMPARISON OF DCPP TO APPENDIX A OF BTP APCSB 9.5-1
A. OVERALL REQUIREMENTS OF NUCLEAR PLANT FIRE PROTECTION PROGRAM

Guideline Statement	DCPP Compliance to Commitment
2. Design Bases	
The overall fire protection program should be based upon evaluation of potential fire hazards throughout the plant and the effect of postulated design basis fires relative to maintaining ability to perform safety shutdown functions and minimize radioactive releases to the environment.	The overall fire protection program is based on the evaluation of potential fire hazards throughout the plant. The Appendix R Reports for DCPP Units 1 and 2 analyze the effect of a postulated design basis fire relative to safe shutdown functions and minimize radioactive releases to the environment.
3. Backup	
Total reliance should not be placed on a single automatic fire suppression system. Appropriate backup fire suppression capability should be provided.	In areas of the plant where automatic fire suppression systems are employed, appropriate backup fire suppression capability is provided by installation of manual hose stations, portable fire extinguishers and portable fire pumps. Each backup method is surveilled as per procedure to ensure equipment availability so total reliance is not dependent upon a single automatic fire suppression system.
4. Single Failure Criterion	
A single failure in the fire suppression system should not impair both the primary and backup fire suppression capability. For example, redundant fire water pumps with the independent power supplies and controls should be provided. Postulated fires or fire protection system failures need not be considered concurrent with other plant accidents or the most severe natural phenomena. The effects of lightning strikes should be included in the overall plant fire protection program	A single failure in the fire suppression system will not impair both the primary and backup suppression capability due to the nature of the primary and backup water supplies, the independence of power supplies for the associated pumps and valves, and the provision for portable backup fire pumps. Portions of the fire water system have been analyzed in regard to the design basis earthquake and are seismically qualified so that all hose-reels in safety-related areas of the plant will be available following a ~~safe shutdown~~DDE/Hosgri earthquake. The seismically qualified portion of the fire system can be readily isolated from the rest of the fire system. Other than those areas required to be available after the design basis earthquake, postulated fires or fire protection system failures are not considered concurrent with other plant accidents or the most severe natural phenomena. Lightning rods are installed at the high points of the containment, and lightning arrestors are installed on each of the phases of the main and auxiliary transformers. The effects of lightning strikes are included in the overall plant fire protection program

F.2 Basis for Deviation Request (Unit 1)

a. The RCP lube oil collection tank overflow pipe discharges downward to a recessed trench in the elevation 91 feet floor, along the outside of the shield wall. This trench is sloped so that an RCP lube oil overflow would flow to the containment drain sump.

b. The overflow pipe of the oil collection tank has pickup from 3 inches above the tank bottom. Thus, in the remote likelihood of a multiple RCP motor lube oil spill and fire propagation to the oil collection tank, such a fire would not be extended to the oil discharges to the floor trench.

c. The Westinghouse RCP CS VSS motor currently utilizes a high flash point lubricating oil (425°F). The fire point of this oil is 520°F. Therefore, a high-energy ignition source would be necessary to sustain combustion in the unlikely event that a multiple RCP lube oil spill occurs and oil is discharged through the overflow pipe. An additional evaluation on the impact of the flash point temperature is included in FHARE 115.

d. Because an oil-to-water heat exchanger serves each bearing assembly, and the heat exchanger discharge water and bearing temperatures are monitored and alarm in the continuously manned control room, it is not deemed credible for the RCP lube oil to reach temperatures within 50 percent of its flash point.

e. There are various components and circuits necessary for safe shutdown in the vicinity of this floor trench. Power cable is routed in conduit. Other circuits are not considered to present a high-energy ignition source.

F.3 Basis for Deviation Request (Unit 2)

a. The RCP oil collection system, including the oil collection tank and overflow piping, has been designed to withstand the ~~safe shutdown~~DDE/Hosgri earthquake.

b. The RCP lube oil collection tank overflow pipe discharges downward to a recessed trench in the floor at elevation 91 feet, along the outside of the shield wall. This trench is sloped so that any RCP lube oil overflow would flow to the containment drain sump.

c. The inlet of the overflow pipe of the oil collection tank, located 3 inches above the tank bottom, will drain water off the bottom of the tank while containing the entire oil inventory of one RCP. The discharge is piped to the containment annulus trench such that splashing of the tank overflow in the trench is precluded.

Chapter 15

ACCIDENT ANALYSES

Since 1970, the ANS classification of plant conditions has been used to divide plant conditions into four categories in accordance with anticipated frequency of occurrence and potential radiological consequences to the public. The four categories are as follows:

(1) Condition I: Normal Operation and Operational Transients

(2) Condition II: Faults of Moderate Frequency

(3) Condition III: Infrequent Faults

(4) Condition IV: Limiting Faults

The basic principle applied in relating design requirements to each of the conditions is that the most frequent occurrences must yield little or no radiological risk to the public, and those extreme situations having the potential for the greatest risk to the public shall be those least likely to occur. Where applicable, reactor trip system and engineered safety features functioning is assumed, to the extent allowed by considerations such as the single failure criterion, in fulfilling this principle.

In the evaluation of the radiological consequences associated with initiation of a spectrum of accident conditions, numerous assumptions must be postulated. In many instances these assumptions are a product of extremely conservative judgments. This is due to the fact that many physical phenomena, in particular fission product transport under accident conditions, are not understood to the extent that accurate predictions can be made. Therefore, the set of assumptions postulated would predominantly determine the accident classification.

The specific accident sequences analyzed in this chapter include those required by Revision 1 of Regulatory Guide 1.70, Standard Format and Content of Safety Analysis Reports for Nuclear Power Plants, and others considered significant for the Diablo Canyon Power Plant (DCPP). Because the DCPP design differs from other plants, some of the accidents identified in Table 15-1 of Regulatory Guide 1.70, Revision 1, are not applicable to this plant; some comments on these items are as follows:

(Item 10) - There are no pressure regulators or regulating instruments in the Westinghouse pressurized water reactor (PWR) design whose failure could cause heat removal greater than heat generation.

(Item 11) - Reactor coolant flow controller is not a feature of the Westinghouse PWR design. Treatment of the performance of the reactivity controller in a number of accident conditions is offered in this chapter.

(Item 12) - The reactor coolant system (RCS) components whose failure could cause a Condition III or Condition IV loss-of-coolant accident (LOCA) are Design Class I components, that is, they are designed to withstand consequences of the ~~safe shutdown earthquake (SSE) which is equivalent to the~~ double design earthquake (DDE) and the Hosgri earthquake (HE) ~~occurrence~~. In addition, the ~~analyses of the design LOCA includes~~ analyses of the design LOCA include the assumption of unavailability of offsite power.

(Item 22) - No instrument lines from the RCS boundary in the DCPP design penetrate the containment[a].

(Item 24) - The analysis of the consequences of such small spills and leaks is included within the cases evaluated in Chapter 11, and larger leaks and spills are analyzed in Section 15.5.

(Item 25) - The radiological consequences of this event are analyzed in Chapter 11, for the case of "Anticipated Operational Occurrences."

(Item 26) - Habitability of the control room following accident conditions is discussed in Chapter 6, and potential radiological exposures are reported in Section 15.5. In addition, Chapter 7 contains an analysis showing that the plant can be brought to, and maintained in, the hot shutdown condition from outside the control room.

(Item 27) - Overpressurization of the residual heat removal system (RHRS) is considered extremely unlikely. PG&E reviewed possible RHRS overpressure scenarios and qualified the system for all credible high pressure transients in DCPP design change package N-049118.

(Item 28) - This event is covered by the analyses of Section 15.2.7.

(Item 29) - Same as Item 28 above.

(Item 30) - Malfunctions of auxiliary saltwater system and component cooling water system (CCWS) are discussed in Chapter 9, Sections 9.2.7 and 9.2.2 respectively.

(Item 31) - There are no significant safety-related consequences of this event.

(Item 33) - The effects of turbine trip on the RCS are presented in Section 15.2.7.

(Item 34) - Malfunctions of this system are discussed in Section 9.3.2.

(Item 35) - The radiological effects of this event are not significant for PWR plants. Minor leakages are within the scope of the analysis cases presented in Chapter 11.

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[a] For definition of the RCS boundary, refer to the 1972 issue of ANS N18.2, Nuclear Safety Criteria for the Design of Stationary PWR Plants.

15.4.5.1.2 Probability of Activity Release

In the above operations, there exists the remote possibility that one or more fuel assemblies will sustain some mechanical damage. There exists an even more remote possibility that this damage will be severe enough to breach the cladding and release some of the radioactive fission products contained therein.

Both the fuel handling procedure and the fuel handling equipment design adhere to the following safety criteria:

(1) Fuel handling operations must not commence before short-lived core activity has decayed, leaving only relatively long-lived activity. Equipment Control Guidelines for refueling operations specify the minimum waiting time.

(2) Fuel handling operations must preclude any critical configuration of the core, spent fuel, or new fuel.

(3) The fuel handling system design must ensure an adequate water depth for radiation shielding of operating personnel.

(4) Active components of the fuel handling systems must be designed such that loss-of-function failures will terminate in stable modes.

(5) The design of fuel handling equipment must minimize the possibility of accidental impact of a moving fuel assembly with any structure.

(6) The design of fuel handling equipment and procedures must minimize the possibility of any massive object damaging a stationary fuel assembly.

(7) Fuel assembly design must minimize the possibility of damage in the event that portable or hand tools come into contact with a fuel assembly.

(8) The design of structures around the fuel handling system must minimize the possibility of the structures themselves failing in the event of a double design earthquake (DDE) or Hosgri earthquake (HE). ~~, which is the safe shutdown earthquake~~. Furthermore, the structures must minimize the possibility of any external missile from reaching fuel assemblies.

(9) Fuel handling equipment must be capable of supporting maximum loads under seismic conditions. Furthermore, fuel handling equipment must not generate missiles during seismic conditions. The earthquake loading of the fuel handling equipment is evaluated in accordance with the seismic considerations addressed in Section 9.1.4.3.2.

TABLE 15.4.1-7A

UNIT 1 PLANT OPERATING RANGE ALLOWED BY THE BEST-ESTIMATE LARGE BREAK LOCA ANALYSIS

Parameter		Operating Range
1.0	Plant Physical Description	
	a. Dimensions	No in-board assembly grid deformation ~~assumed due to~~during LOCA + DDE or LOCA + HE~~SSE~~ (which ever is more limiting)
	b. Flow resistance	N/A
	c. Pressurizer location	N/A
	d. Hot assembly location	Anywhere in core
	e. Hot assembly type	Fresh 17X17 V5, ZIRLO, or Zircaloy cladding, 1.5X IFBA or non-IFBA
	f. SG tube plugging level	≤15%
	g. Fuel assembly type	Vantage 5, ZIRLO, or Zircaloy cladding, 1.5X IFBA or non-IFBA
2.0	Plant Initial Operating Conditions	
	2.1 Reactor Power	
	a. Core average linear heat rate	Core power ≤ 102% of 3411 MWt
	b. Peak linear heat rate	$F_Q \leq 2.7$
	c. Hot rod average linear heat rate	$F_{\Delta H} \leq 1.7$
	d. Hot assembly average linear heat rate	$\overline{P}_{HA} \leq 1.57$
	e. Hot assembly peak linear heat rate	$F_{QHA} \leq 2.7/1.04$

TABLE 15.4.1-7B

UNIT 2 PLANT OPERATING RANGE ALLOWED BY THE BEST-ESTIMATE LARGE BREAK LOCA ANALYSIS

Parameter		Operating Range
1.0	**Plant Physical Description**	
	a) Dimensions	No in-board assembly grid deformation during LOCA + DDE or LOCA + HE (which ever is more limiting)~~SSE~~
	b) Flow resistance	N/A
	c) Pressurizer location	N/A
	d) Hot assembly location	Anywhere in core interior (149 locations)[a]
	e) Hot assembly type	Fresh 17x17 V5+ fuel with ZIRLO™ cladding
	f) Steam generator tube plugging level	≤ 15%
	g) Fuel assembly type	17x17 V5+ fuel with ZIRLO™ cladding, non-IFBA or IFBA
2.0	**Plant Initial Operating Conditions**	
	2.1 Reactor Power	
	a) Core average linear heat rate	Core power ≤100.3% of 3,468 MWt
	b) Peak linear heat rate	$F_Q \leq 2.7$
	c) Hot rod average linear heat rate	$F_{\Delta H} \leq 1.7$
	d) Hot assembly average linear heat rate	$\overline{P}_{HA} \leq 1.7/1.04$
	e) Hot assembly peak linear heat rate	$F_{QHA} \leq 2.7/1.04$
	f) Axial power distribution (PBOT, PMID)	See Figure 15.4.1-15B.
	g) Low power region relative power (PLOW)	0.3 ≤PLOW ≤0.8
	h) Hot assembly burnup	≤75,000 MWD/MTU, lead rod[a]
	i) Prior operating history	All normal operating histories
	j) Moderator temperature coefficient	≤0 at HFP
	k) HFP boron (minimum)	800 ppm (at BOL)
	2.2 Fluid Conditions	
	a) T_{avg}	$565 - 5°F \leq T_{avg} \leq 577.6 + 5°F$
	b) Pressurizer pressure	$2250 - 60\ psia \leq P_{RCS} \leq 2250 + 60\ psia$

Summary of Regulatory Commitments

New Commitment:

1) Any outstanding gaps in the probabilistic risk assessment model when compared to the Capability Category II of ASME/ANS RA-Sa-2009 will be addressed as part of any seismic probabilistic risk assessment (SPRA) update. The SPRA update will be completed within 2 years following issuance of (currently draft) NRC Generic Letter 2011-XX, Seismic Risk Evaluations for Operating Reactors.

Revision to Existing Commitment:

In PG&E Letter DCL-91-178, PG&E made the following commitment:

Future additions and modifications to the plant will be designed and constructed in accordance with this existing seismic qualification basis. In addition, certain future plant additions and modifications as specified in enclosed Table 1 will be checked against insights and knowledge gained from the LTSP to verify that the plant "high-confidence-of-low-probability-of-failure" values remain acceptable.

- DCL-91-178 included an implementing procedure which stated that in order to take advantage of the insights and knowledge gained from the [LTSP], certain future additions and modifications will be checked against the [LTSP] spectra described in the U. S. Nuclear Regulatory Commission's Supplemental Safety Evaluation Report (SSER) No. 34, June 1991, to verify that the [DCPP] high-confidence-of-low-probability-of-failure (HCLPF) values remain acceptable.

PG&E is revising the commitment above to the following:

This commitment is being revised to be consistent with the proposed evaluation process for new seismic information. The evaluation process proposed in this license amendment request (LAR) requires that the seismic margin for plant additions and plant modifications be maintained at or above 1.3, unless the minimum seismic margin below 1.3 is identified in Final Safety Analysis Report Update (FSARU) Tables 3.7-25 or 3.7-26 due to previous review and approval by the NRC.

Chapter 5 of the 1988 Long Term Seismic Program Final Report

Chapter 5
SOIL/STRUCTURE INTERACTION ANALYSIS
To Partially Address
Element 4 of the License Condition

ELEMENT 4 OF THE LICENSE CONDITION

PG&E shall assess the significance of conclusions drawn from the seismic reevaluation studies in Elements 1, 2, and 3, utilizing a probabilistic risk analysis and deterministic studies, as necessary, to assure adequacy of seismic margins.

OBJECTIVES

The objectives of the soil/structure interaction analysis conducted for the Diablo Canyon Power Plant Long Term Seismic Program were to examine the effects of dynamic interaction between the Plant structures and the supporting rock medium on the seismic response of the structures, and to generate seismic responses for the Plant structures required for the seismic fragility evaluation and seismic margin assessment. This analysis was conducted in response to Element 4 of the license condition.

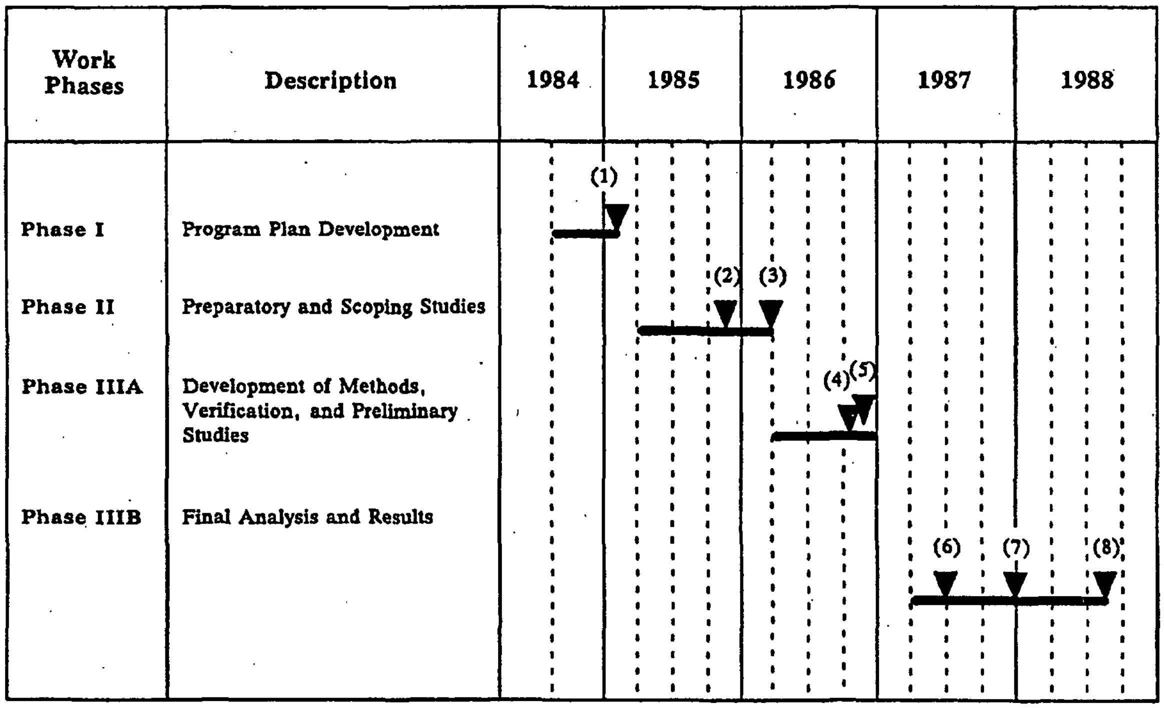
SCOPE

The soil/structure interaction analysis started in late 1984 and continued through mid-1988. The analysis was carried out in three phases, namely, Phase I: Program Plan development; Phase II: preparatory work and Scoping Study; Phase III: method development, implementation, and verification; preliminary results; and final analysis and results.

The progress and results of the soil/structure interaction analysis obtained in various phases were reviewed and discussed with the Nuclear Regulatory Commission (NRC) Staff and its consultants through several NRC/PG&E meetings, three specific NRC/PG&E workshops on soil/structure interaction analyses, and one NRC audit on soil/structure interaction analysis calculations. The schedule and milestones of the soil/structure analysis program are summarized in Figure 5-1. Comments received to date from the NRC at various stages of review have been incorporated into the program wherever applicable, and they are reflected in the final results of the analysis.

The scope of the soil/structure interaction analysis that has been carried out for the Long Term Seismic Program consists of the following major activities:

- Assemble, review, and determine appropriate site rock profiles and properties.
- Develop suitable three-dimensional dynamic models for the power block structures.
- Implement, modify, and validate dual soil/structure interaction analysis computer programs, CLASSI and SASSI.
- Perform parametric studies to assess the sensitivities of soil/structure interaction response and identify significant parameters to be considered for modeling and analysis.
- Perform analyses of on-site recorded earthquake data and extract information useful for correlation and calibration of model parameters.
- Perform soil/structure interaction analyses to generate the responses for the power block structures subjected to coherent, vertically



Milestones:

1. NRC approval of the Long Term Seismic Program Plan, January 31, 1985
2. NRC/PG&E meeting on Long Term Seismic Program, October 21, 1985
3. First NRC/PG&E soil/structure interaction workshop, April 14-16, 1986
4. NRC/PG&E ground-motion workshop to review soil/structure interaction work, October 24, 1986
5. Second NRC/PG&E soil/structure interaction workshop, December 10-12, 1986
6. NRC audit of PG&E soil/structure interaction calculations, June 9-11, 1987
7. Third NRC/PG&E soil/structure interaction workshop, November 4-6, 1987
8. PG&E submittal of Long Term Seismic Program Final Report to NRC, July 31, 1988

Figure 5-1
Soil/structure interaction assessment schedule and milestones.

- Develop and validate the method and computer programs for incorporating the spatial incoherence of seismic ground motions for soil/structure interaction analysis; perform analyses to develop the soil/structure interaction response adjustment factors to account for the spatial incoherence of ground motions.

- Modify and validate the method and computer program for nonlinear soil/structure interaction analysis, taking into account the nonlinear base-uplifting response behavior, and perform analyses for the containment structure to assess the effect on soil/structure interaction response due to partial uplifting of the containment base from the rock foundation.

METHOD OF ANALYSIS AND SUMMARY OF RESULTS

The general configuration of the Plant power block structures, which include the containment structures of both units, the auxiliary building, and the turbine building, is shown schematically in Figure 5-2. An elevation view of a section through the Plant is shown in Figure 5-3. To achieve the objective of the soil/structure interaction analysis, a complete reevaluation of the seismic soil/structure interaction effects on the power block structures was carried out, using state-of-the-art analysis techniques. The analysis has also incorporated all available relevant new information that became available after 1978. This includes the additional site investigation data obtained during 1977 to 1978, and the on-site recorded actual earthquake data available since 1980.

As stipulated in the Program Plan, the Long Term Seismic Program soil/structure interaction analyses have specifically included the following elements:

- Three-dimensional soil/structure analysis methods have been used.

- All components of free-field ground motions at the site have been considered in the determination of seismic response of interest.

- The effect of spatial variation of free-field seismic ground motion, including the apparent wave passage effect, has been properly evaluated.

- The effect of nonlinear base uplifting behavior on the seismic response of the most critical containment structure under the fragility evaluation strong ground motion input has been assessed.

- Recorded earthquake data at the Diablo Canyon site and on the power block structures have been utilized to the extent practicable to assist in calibrating the low amplitude dynamic characteristics of the site rock and dynamic models.

The free-field seismic ground-motion inputs for the soil/structure interaction analyses were obtained from the ground-motion studies, as summarized in Chapter 4. The results of the soil/structure interaction analyses provided the Plant responses required for the probabilistic Plant fragility evaluation and the deterministic seismic margin assessment. The overall soil/structure interaction analysis method, from the ground-motion input to the generation of Plant response output, is shown schematically in Figure 5-4.

Prior to performing the soil/structure interaction analysis, an extensive effort was conducted to characterize the soil/structure interaction systems for the power block structures and to prepare the appropriate analytical methods and computer programs required by various phases of analysis. The effort spent on the characterization of the systems includes: (a) the characterization of site rock profile and properties; (b) the development of suitable three-dimensional dynamic models for the power block structures; and (c) parametric studies to evaluate the sensitivities of soil/structure interaction response and identify important soil/structure interaction parameters to be considered. The effort on preparation of appropriate analytical tools for the soil/structure interaction analysis includes: (a) the implementation and validation of the CLASSI and

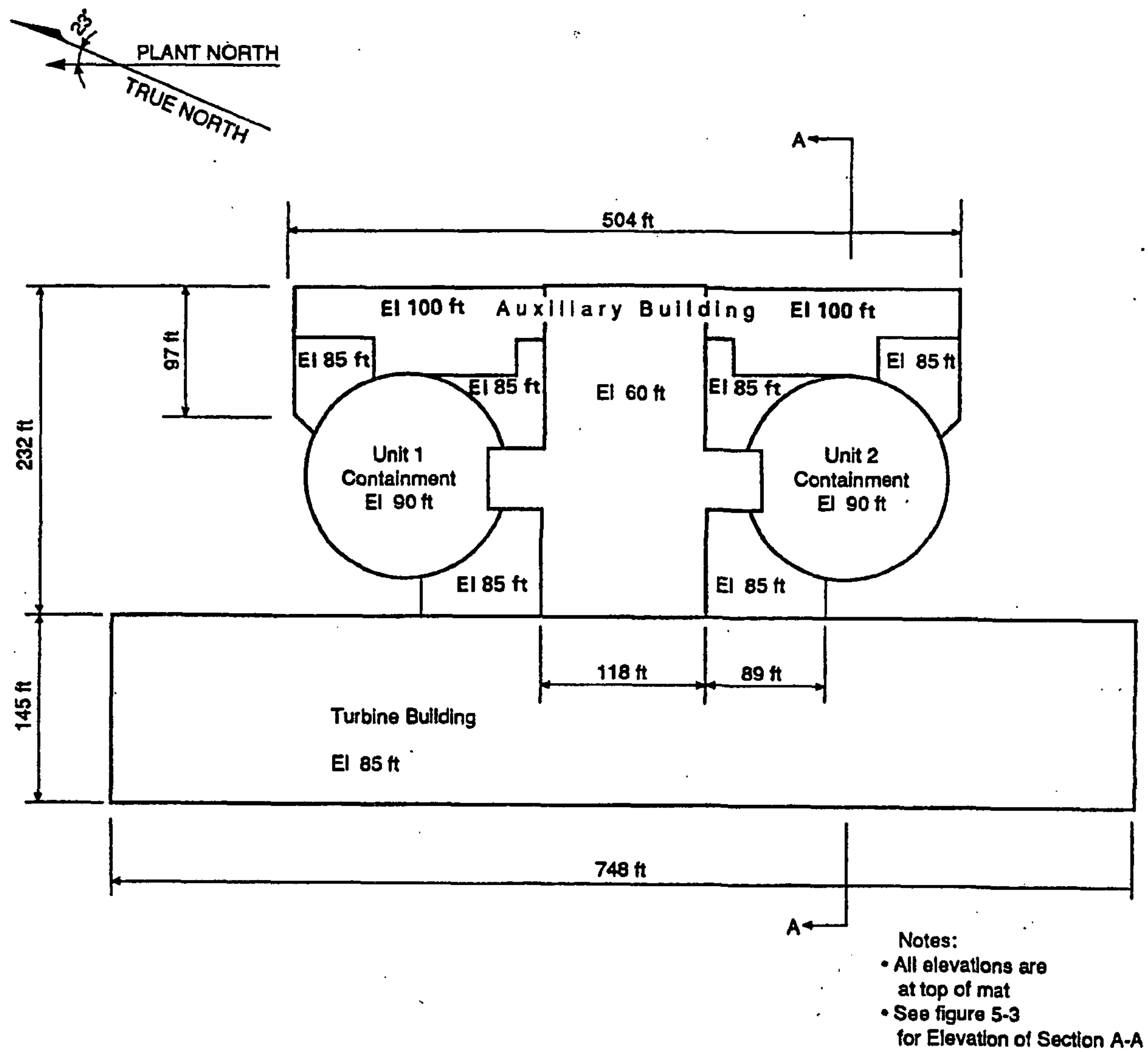


Figure 5-2

Foundation configuration of power block structures.

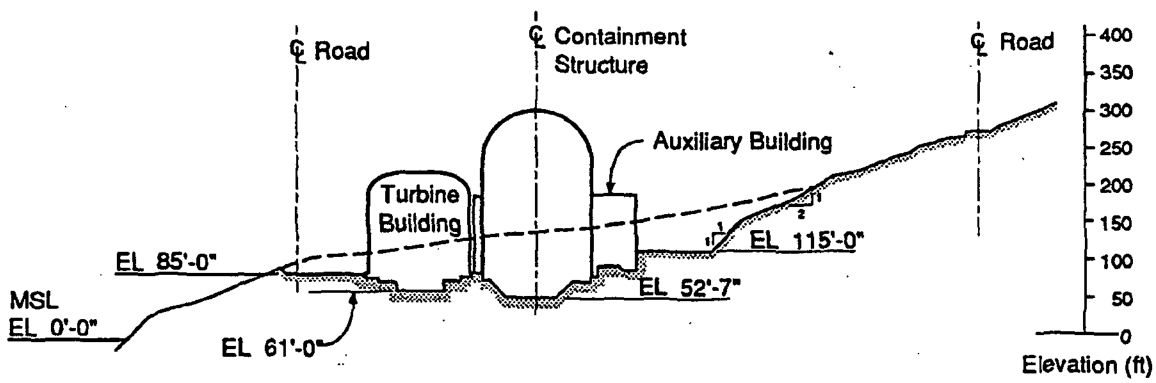


Figure 5-3

Elevation view of section A-A of Figure 5-2.

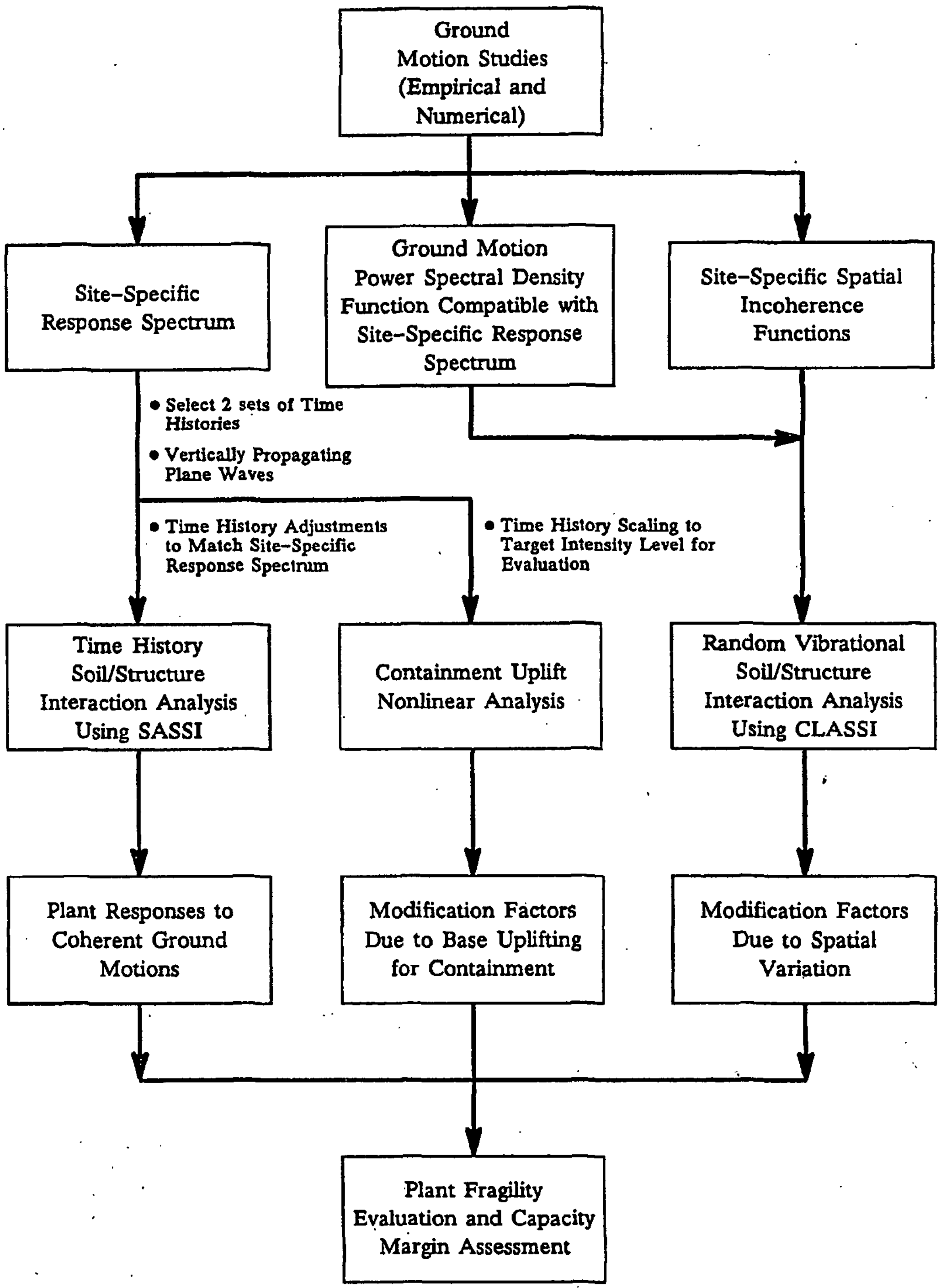


Figure 5-4

Overall soil/structure interaction analysis method.

SASSI computer programs for three-dimensional analysis; (b) the development, implementation, and validation of analysis method and computer programs for soil/structure interaction analysis incorporating the spatial incoherence of seismic ground motions; and (c) the modification and validation of the soil/structure analysis method and computer program for analyzing the nonlinear dynamic response due to base-uplifting.

Characterization of Site Rock Properties

Recognizing the importance of fixing the site rock properties at the beginning of the Long Term Seismic Program, a priority task was performed to assemble and review all available site rock data and, based on this review, to assess the appropriate rock profile and properties for soil/structure interaction analysis. The rock data that have been assembled include two sets of data: one set consists of data contained in the source references of the Diablo Canyon Power Plant FSAR Section 2.5, which were obtained from the site investigations conducted from 1967 to 1973; the second set consists of data obtained from the additional site investigations conducted from 1977 to 1978. Both sets of data have been reviewed in detail.

The rock data available from the FSAR references consist of data obtained from both field geophysical surveys and laboratory tests of rock samples. These data were applicable mainly for rocks at shallow depths, that is, down to a depth of about 40 feet below the finished grade at El 85 feet. The rock data available from the 1977 to 1978 site investigations consist of data from borehole logging, field geophysical surveys, and laboratory tests of rock samples obtained from four deep boreholes drilled around the Plant to a depth of approximately 300 feet below grade.

Review of data from both sets indicated that the data from field-measured shear and compression wave velocities and rock densities are more mutually consistent and these data are considered to be more representative of the in situ properties of the rock mass below the plant foundation; the laboratory test values represent only very local rock conditions and the test results are marked with uncertainties resulting from the specimen saturation procedures used and the test equipment flexibilities. Thus, in deriving the low-strain rock property profiles for soil/structure interaction analysis purposes, emphasis was placed on field-measured data, especially the data taken from the depth below El 50 feet, because the foundations of the power block structures are located at elevations between 50 feet and 80 feet.

Based on the review of rock data assembled, representative profiles and the ranges of variation of rock shear wave velocity, Poisson's ratio, rock density, damping ratio at low-strain, and the strain-dependent variations of shear modulus and damping ratio, were derived. Figure 5-5 shows the mean shear wave velocity profile and the upper-bound and lower-bound of data developed from the assembled site rock data.

Because the rock shear wave velocity profiles developed from the assembled data showed relatively large scattering, a study was carried out to assess the sensitivity of soil/structure interaction response due to the variation of rock shear wave velocity profile. The sensitivity study was performed using a simplified soil/structure interaction model for the containment structure and the CLASSI computer program for soil/structure interaction analyses. The results of this sensitivity study indicated that, as the foundation rock shear wave velocity profile varies from the upper-bound to the mean and then to the lower-bound, the fundamental soil/structure interaction frequency for the coupled horizontal translation and rocking mode of the containment shell shifts from 4.6 hertz to 4.0 hertz, and then to 3.3 hertz. Despite the relatively large variation in the rock shear wave velocity profile, the frequency variation was found to be within approximately ±15 percent.

To provide an independent confirmation of the appropriateness of the rock property profiles developed for soil/structure interaction analysis, the fundamental soil/structure interaction frequency of the containment shell, which was sensitive to the variation of rock shear wave

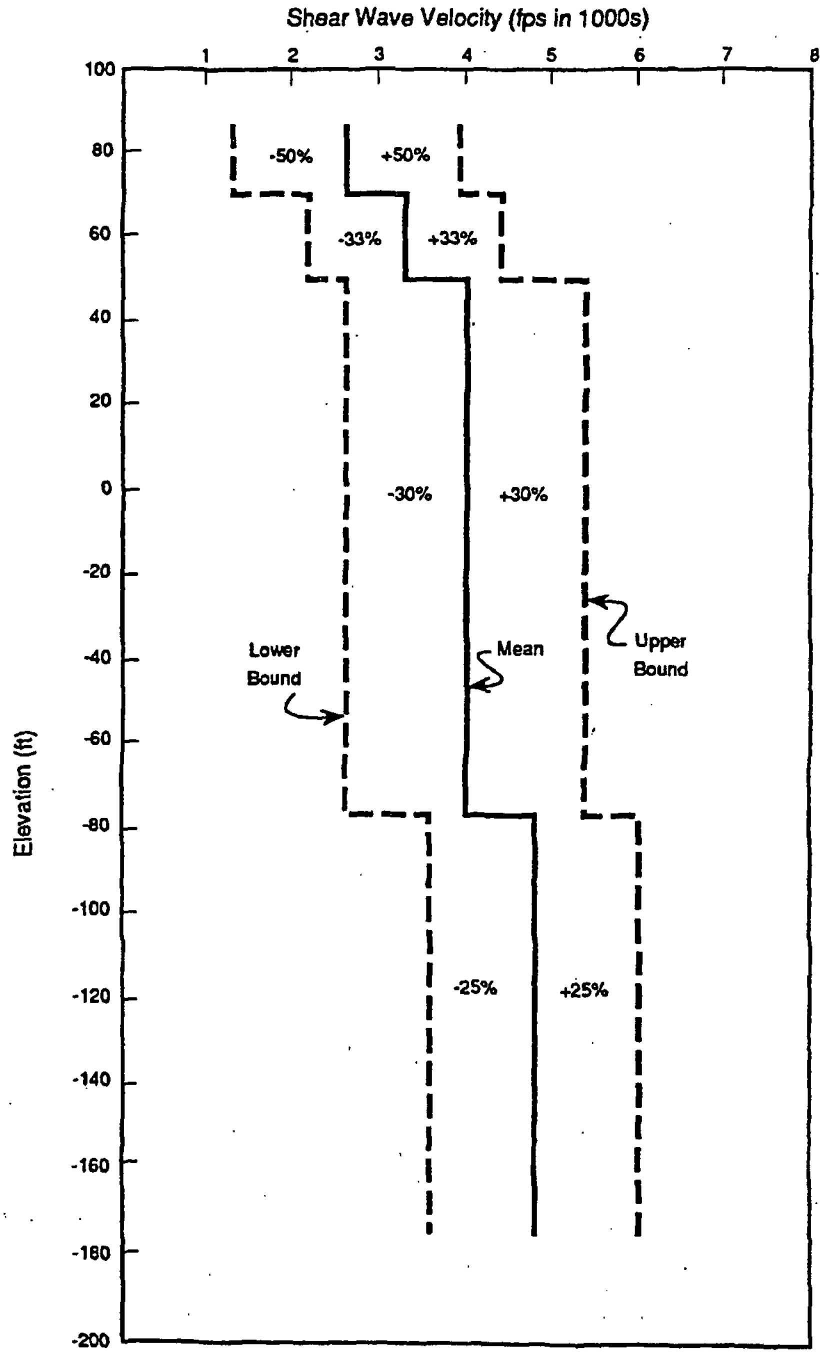


Figure 5-5

Site shear wave velocity profiles (based on 1978 downhole velocity measurements).

velocity profile, was selected as the parameter for a correlation study using the available on-site recorded earthquake data in the free-field and on the Unit 1 containment structure for three very low intensity earthquakes (the maximum ground accelerations recorded were between 0.01 to 0.03 g). The results of this correlation study showed that the analytical soil/structure interaction frequency based on the mean shear wave velocity profile and the associated properties correlates very well (within ±5 percent) with the corresponding soil/structure interaction frequencies determined from the analysis of recorded data for all three earthquakes. This good correlation confirms that the mean shear wave velocity profile along with other associated elastic properties of the rock as developed from the assembled rock data provides an appropriate representation of the characteristics of the foundation rock at the Diablo Canyon site for strain levels (2×10^{4} percent to 4×10^{-4} percent) consistent with the low-intensity earthquakes considered in the study. It can be concluded from this result that values of shear wave velocity above those represented by the mean profile shown in Figure 5-5 need not be considered for soil/structure interaction analyses with input seismic intensities higher than those considered in this correlation study.

Soil/Structure Interaction Analysis Methods and Computer Programs

To adequately address the issues relating to soil/structure interaction raised in the NRC SER Supplement No. 27, the analysis adopted the newly developed three-dimensional soil/structure interaction analysis methods and the associated computer programs, CLASSI and SASSI. Both these programs are capable of handling three-dimensional soil/structure interaction problems with seismic inputs in the form of general incidence plane wave fields. Although some limitations still exist in the use of the individual computer codes, the effects of these limitations can be evaluated through the concurrent use of both analysis techniques and reconciliation of the results with each other.

In the early stages of the Long Term Seismic Program, both these computer programs (that is, CLASSI and SASSI) were obtained from the program developers and they were implemented to test their suitability for Program applications. As a result of these tests, desirable modifications to both programs were identified to suit the Program application requirements; these modifications were subsequently implemented with the aid of the program developers. At this stage, an extensive code verification program was performed to validate the modified versions of the computer codes. The results of the program modifications and validation for both programs have been fully documented in the Theoretical, User's, and Validation Manuals for CLASSI and SASSI (Bechtel, 1988).

CLASSI (Continuum Linear Analysis for Soil/Structure Interaction) is a linear three-dimensional seismic soil/structure interaction analysis computer code developed at the University of California, San Diego (Wong and Luco, 1976). The analysis method used in CLASSI is based on the substructuring technique that separates the analysis of kinematic interaction (foundation scattering of seismic motions) from that of inertial interaction (dynamic coupling of structure and foundation impedances), as shown schematically in Figure 5-6. The foundation medium is represented in CLASSI by a uniform or a horizontally layered, elastic or viscoelastic continuum halfspace. The most significant limitation of the version of CLASSI implemented for our Program applications is that the structural foundation must be rigid, flat, and founded on the surface of the halfspace. Thus, the foundation embedment and basemat flexibility effect cannot be evaluated. This version of CLASSI has been validated by benchmarking the CLASSI solutions against available published solutions for 18 validation test problems, and by cross-benchmarking with the SASSI solutions available for the common validation test problems.

SASSI (Systems for Analysis of Soil/Structure Interaction) is a finite-element computer program for two- and three-dimensional linear

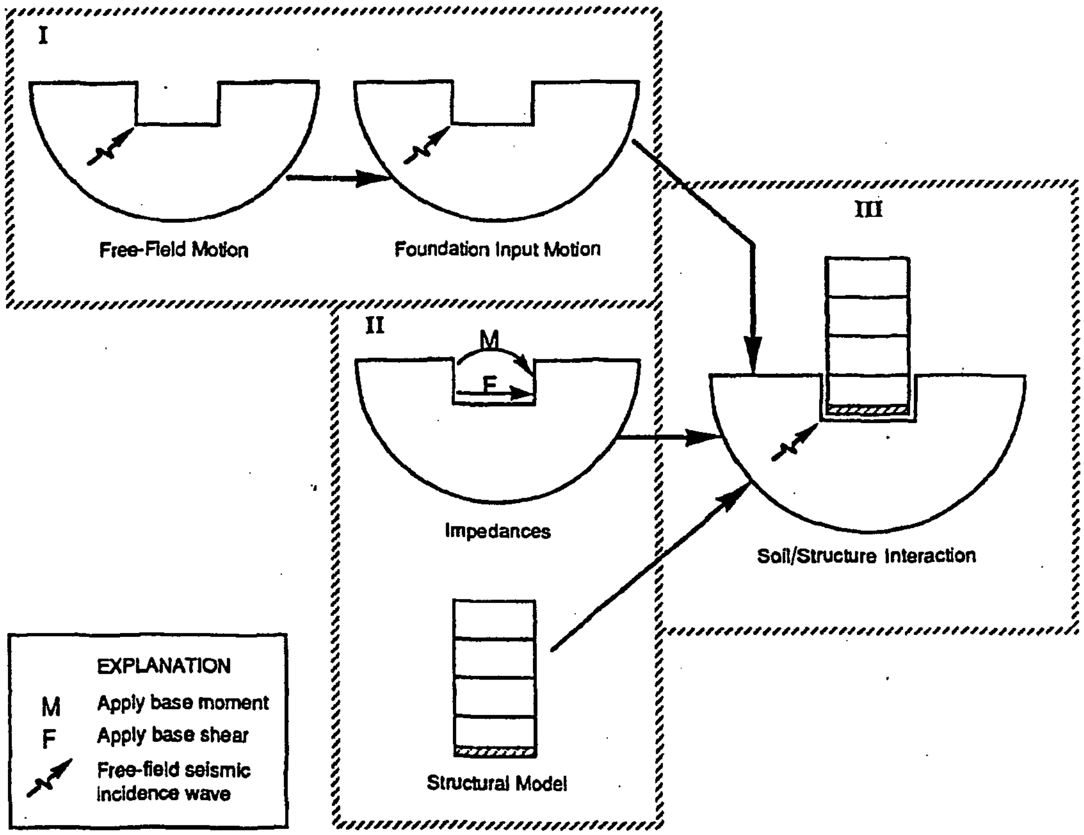


Figure 5–6

CLASSI substructuring technique.

soil/structure interaction analyses developed at the University of California, Berkeley (Lysmer and others, 1981). The program uses the complex response method and the flexible-volume substructuring technique as shown schematically in Figure 5-7. The soil material is modeled using complex moduli and a hysteretic damping mechanism. The foundation medium is represented by a horizontally layered soil system overlaying an elastic halfspace. Due to the unique flexible-volume substructuring technique employed and the use of finite-element models, SASSI can rigorously handle the soil/structure interaction effects due to foundation embedment and basemat flexibility. However, because of the large number of degrees-of-freedom that usually result from the use of three-dimensional finite-element models, the most significant limitation of the SASSI program is the soil/structure interaction model size and the computational costs. The SASSI version implemented for Program applications has been validated by benchmarking SASSI solutions against available published solutions for 20 validation test problems, and by cross-benchmarking with the CLASSI solutions available for the common validation test problems.

Three-Dimensional Dynamic Models for Power Block Structures

For the purpose of three-dimensional soil/structure interaction analysis for the power block structures using either CLASSI or SASSI, three-dimensional dynamic models were developed for the containment structure, auxiliary building, and turbine building including the turbine pedestal. The development of these models used as much as possible the model data available from the dynamic models used for seismic analysis prior to the Long Term Seismic Program.

For the containment structure, the three-dimensional dynamic model developed for the analysis is a three-dimensional lumped-mass, multiple-stick model, as shown in Figure 5-8. The model consists of a 9-lumped-mass, single stick for representing the exterior containment shell and a 16-lumped-mass, multiple-branch single-stick for representing the interior concrete structure. An extra single degree-of-freedom vertical lumped-mass model was developed and attached to the containment shell stick at the containment springline location to represent the fundamental vertical drumming mode of the containment hemispherical dome. Due to its asymmetric configuration, the three-dimensional stick model for the interior concrete structure includes both the mass eccentricities and the proper locations and orientations of the centers of rigidity of the structure.

For the auxiliary building, two three-dimensional dynamic models were developed for analysis applications. One of these models was a three-dimensional finite-element dynamic model, which was developed by modifying the three-dimensional finite-element static model that existed prior to the Program. The second model was a three-dimensional, 25-lumped-mass, five-stick model. The three-dimensional finite-element dynamic model was developed primarily for studying the dynamic characteristics of the building in relation to its irregular configuration. The knowledge gained from this study provided a basis for developing the three-dimensional lumped-mass stick model. In addition to this application, the three-dimensional finite-element dynamic model was also used for soil/structure interaction parametric studies to assess the effect of foundation basemat flexibility.

The three-dimensional lumped-mass stick model for the auxiliary building was developed with the specific intent of CLASSI and SASSI analysis applications. The development was based on the conventional dynamic stick model development technique aided with the understanding of the dynamic characteristics of the building obtained from the three-dimensional finite-element dynamic model.

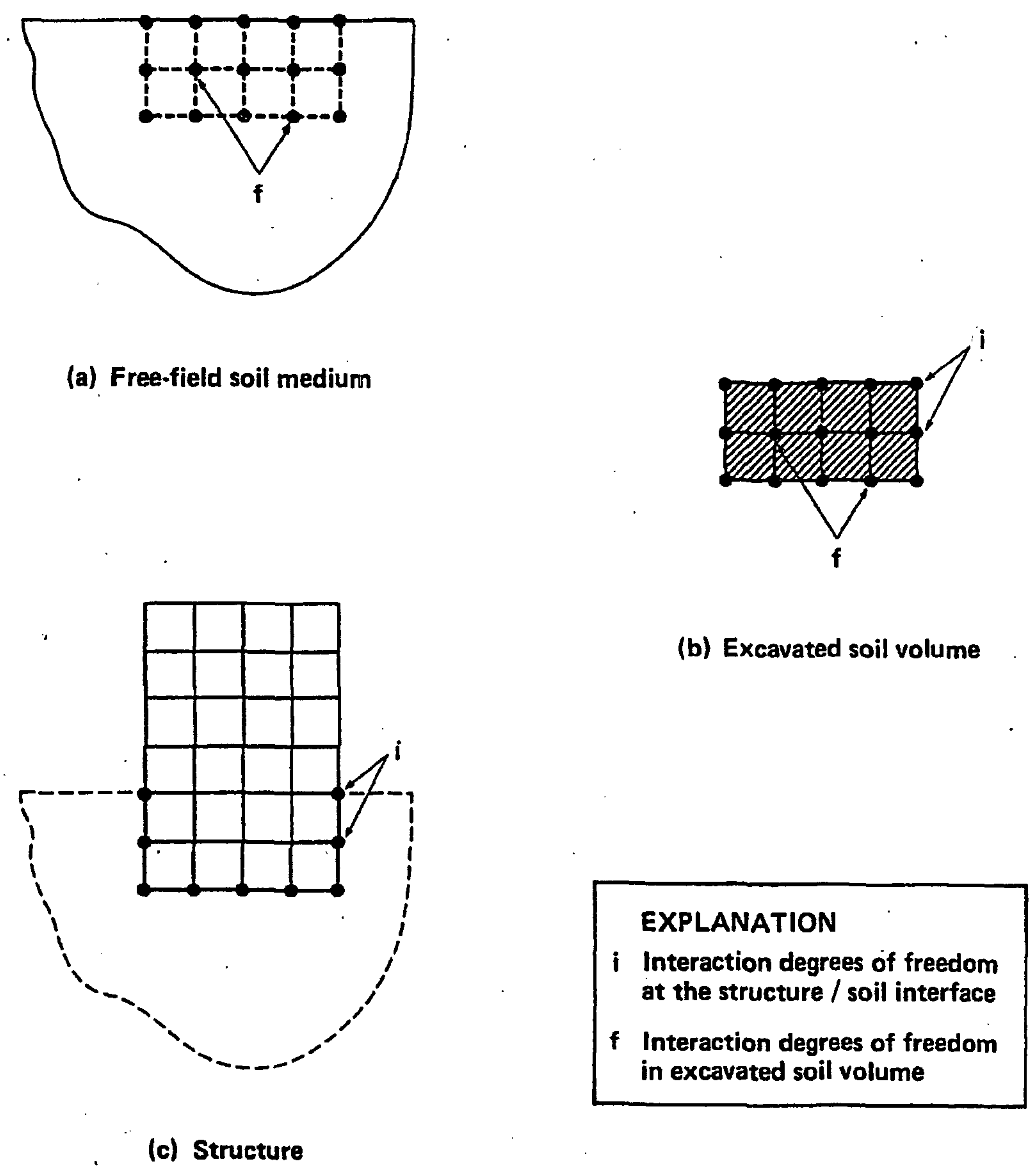


Figure 5–7

SASSI flexible–volume substructuring technique.

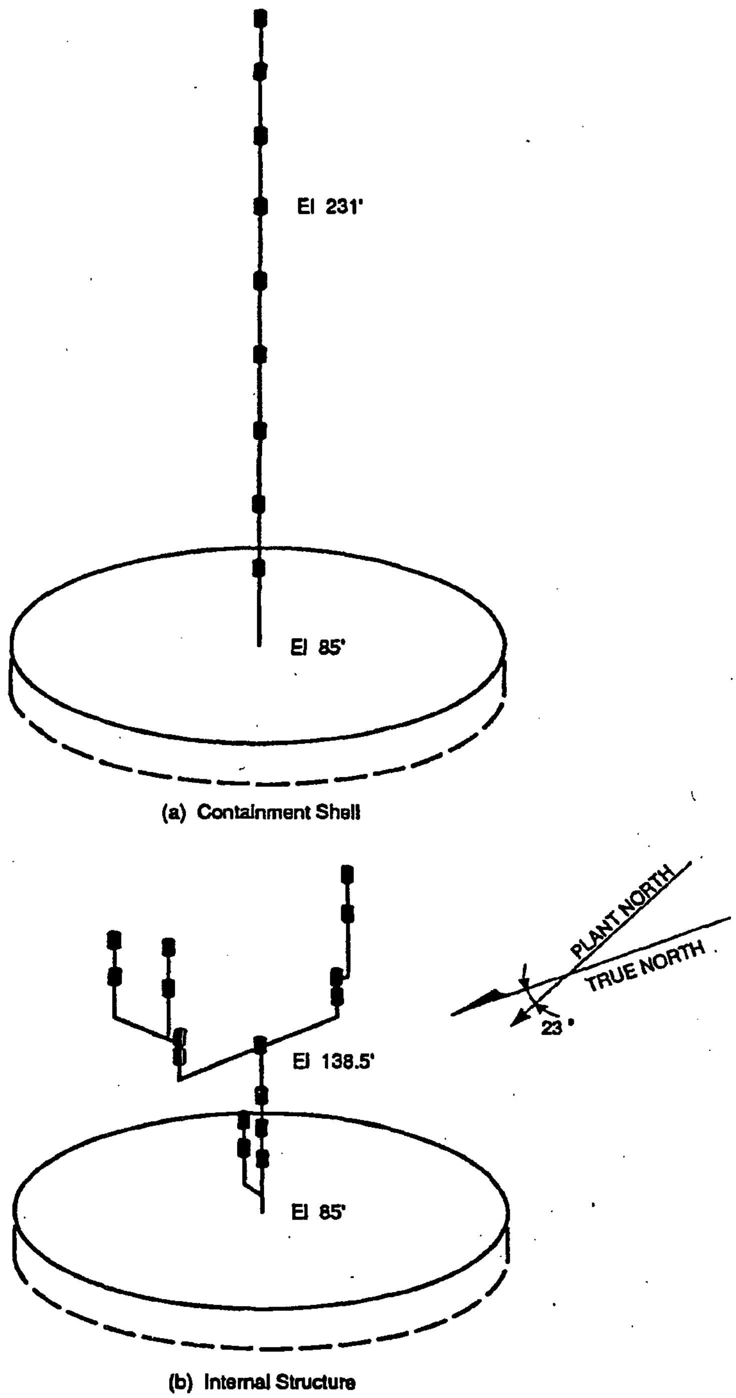


Figure 5-8

Three-dimensional lumped-mass dynamic model for the containment structure.

The configuration of the three-dimensional lumped-mass stick model developed for the auxiliary building is shown in Figure 5-9. Modal analysis performed using the three-dimensional stick model and the three-dimensional finite-element dynamic model, both with the same fixed-base conditions, showed that they are dynamically equivalent with each other in terms of providing comparable modal characteristics for the significant response modes.

For the turbine building, because of the complexity of the building structural system and the lack of continuous rigid diaphragm action due to the presence of turbine pedestal openings in the floors, the three-dimensional dynamic model selected for analysis applications was a three-dimensional finite-element dynamic model, as shown in Figure 5-10 for the Unit 2 turbine building. This model was developed by modifying the detailed three-dimensional finite-element model used in studies prior to the Program. The three-dimensional dynamic model for the turbine pedestal developed for Program applications is a single lumped-mass stick model. This simple model was considered adequate, because the dynamic characteristics of the turbine pedestal as indicated by the existing refined model were found to be dominated by the fundamental modes in each of the three directions.

Soil/Structure Interaction Parametric Studies

Prior to the development of suitable soil/structure interaction models for the power block structures and the selection of the more appropriate computer programs between CLASSI and SASSI to be applied for final soil/structure interaction analysis, a series of parametric studies were carried out. The objectives of these parametric studies was to assess the soil/structure interaction response sensitivities as affected by various parameters and to identify those parameters which are important for the soil/structure interaction modeling and analysis for power block structures.

The soil/structure interaction parameters studied included the foundation embedment effect, the multiple-structure-to-structure interaction effect, the effect of nonvertically incident seismic wave inputs, the foundation basemat flexibility effect, and the sensitivity of results to the CLASSI/SASSI solution techniques. In addition, a separate study was performed to assess the importance of strain-dependency of the site rock shear modulus under high intensity earthquake conditions and the effects of variations in Poisson's ratio and material damping ratio for the foundation rock.

For the purpose of the parametric studies, the horizontal soil/structure interaction responses of the containment structure and the auxiliary building were analyzed using either CLASSI or SASSI, or both, for seven parametric cases, each with a different combination of the following parameters: surface-supported versus embedded foundations; single versus multiple foundations; rigid versus flexible foundation. The seismic input for the analysis considered three different type of seismic wave fields, namely, vertical SV plane waves; SV plane waves inclined at a 30-degree angle from the vertical; and horizontally propagating SH plane waves. The seven parametric cases with different types of seismic input analyzed for the parametric studies are summarized in Table 5-1. Except the study for the foundation basemat flexibility effect, for which the analysis was based on the three-dimensional finite-element dynamic model of the auxiliary building coupled with a finite- element foundation model, all analyses for the parametric studies were based on simplified soil/structure interaction models of both the containment structure and the auxiliary building. As an example, the simplified model for the containment structure used for the studies is shown in Figure 5-11. The seismic input time history for the parametric studies was a horizontal acceleration time history with a maximum acceleration of either 0.75 g or 0.96 g, prescribed at the rock surface of the Plant's finished grade at El 85 feet.

Based on the assessment of the soil/structure interaction response sensitivities indicated by the results of the parametric studies, the following conclusions were made:

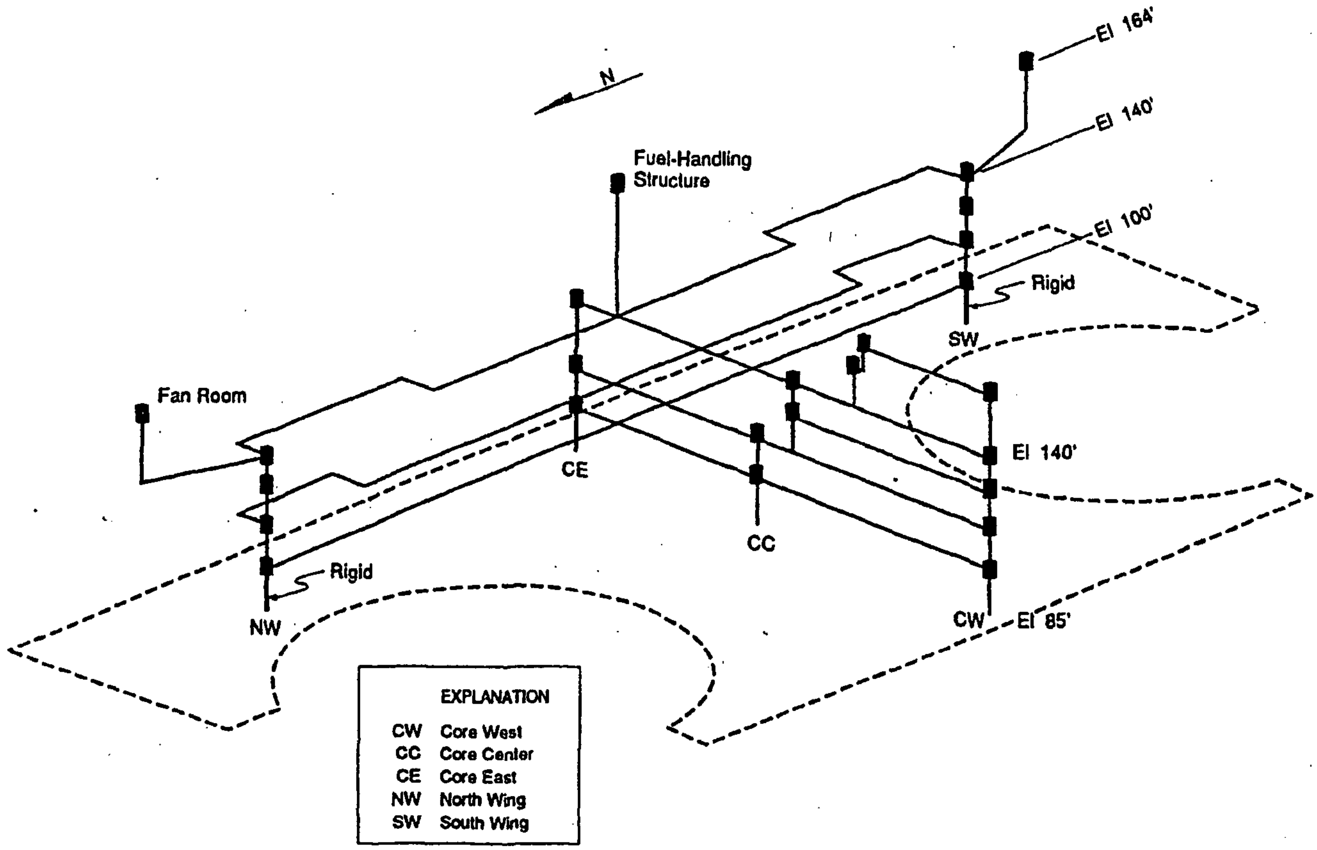


Figure 5-9

Three-dimensional 25-lumped-mass, 5-stick model for auxiliary building.

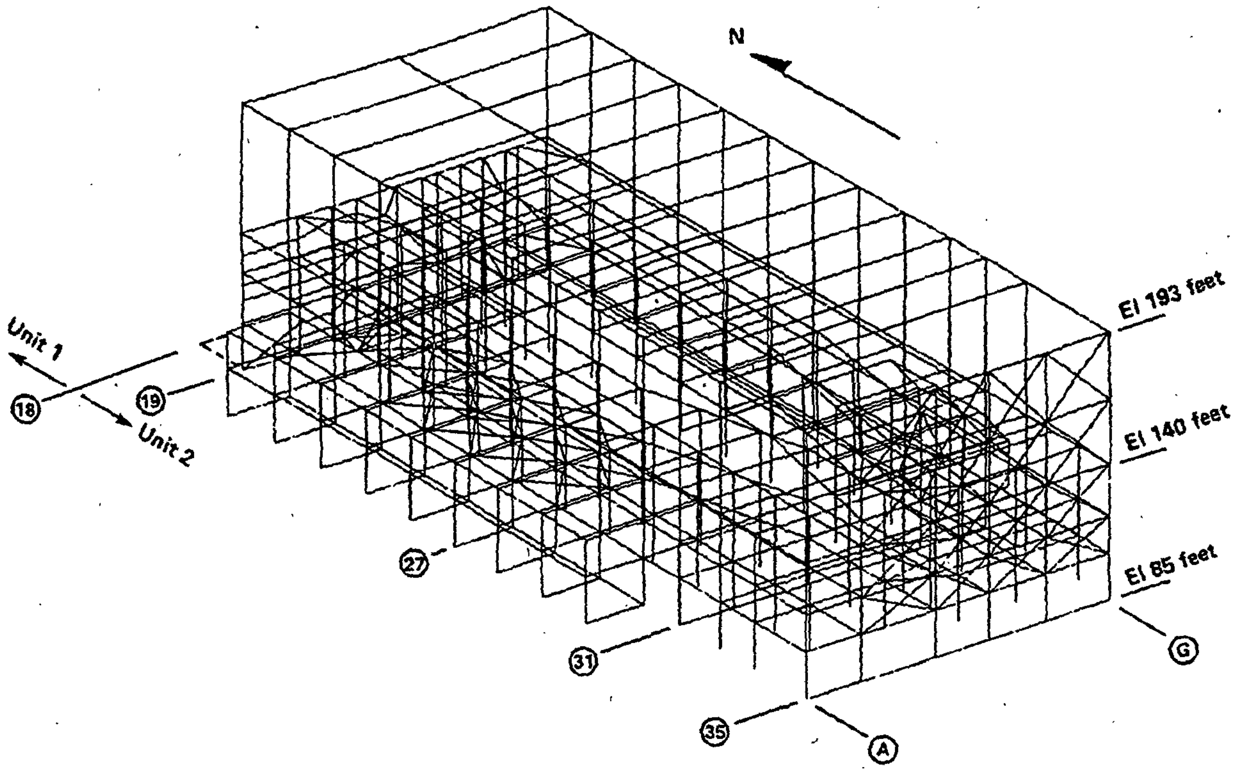


Figure 5-10

Three-dimensional finite-element dynamic model for Unit 2 turbine building above El 85 feet.

Table 5-1

PARAMETRIC CASES STUDIED AND COMPUTER PROGRAMS USED

	Ground Motion Input		
Parametric Cases	**Vertical SV**	**SV-30 Degrees**	**Horizontal SH**
(1) Fixed-Base Condition for Containment and Auxiliary Building	Standard Structural Dynamics Programs	-	-
(2) Single Surface Rigid Foundation for Containment and Auxiliary Building	CLASSI/SASSI	CLASSI/SASSI	CLASSI
(3) Single Embedded Rigid Foundation for Containment and Auxiliary Building	SASSI	SASSI	-
(4) Containment and Auxiliary Building Surface Rigid Foundation	SASSI/CLASSI	SASSI/CLASSI	CLASSI
(5) Containment and Auxiliary Building Embedded Rigid Foundation	SASSI	SASSI	-
(6) Auxiliary Building Embedded Flexible Foundation	SASSI	-	-
(7) Containment with Embedded Rigid Foundation and Rock Property Variations	SASSI	-	-

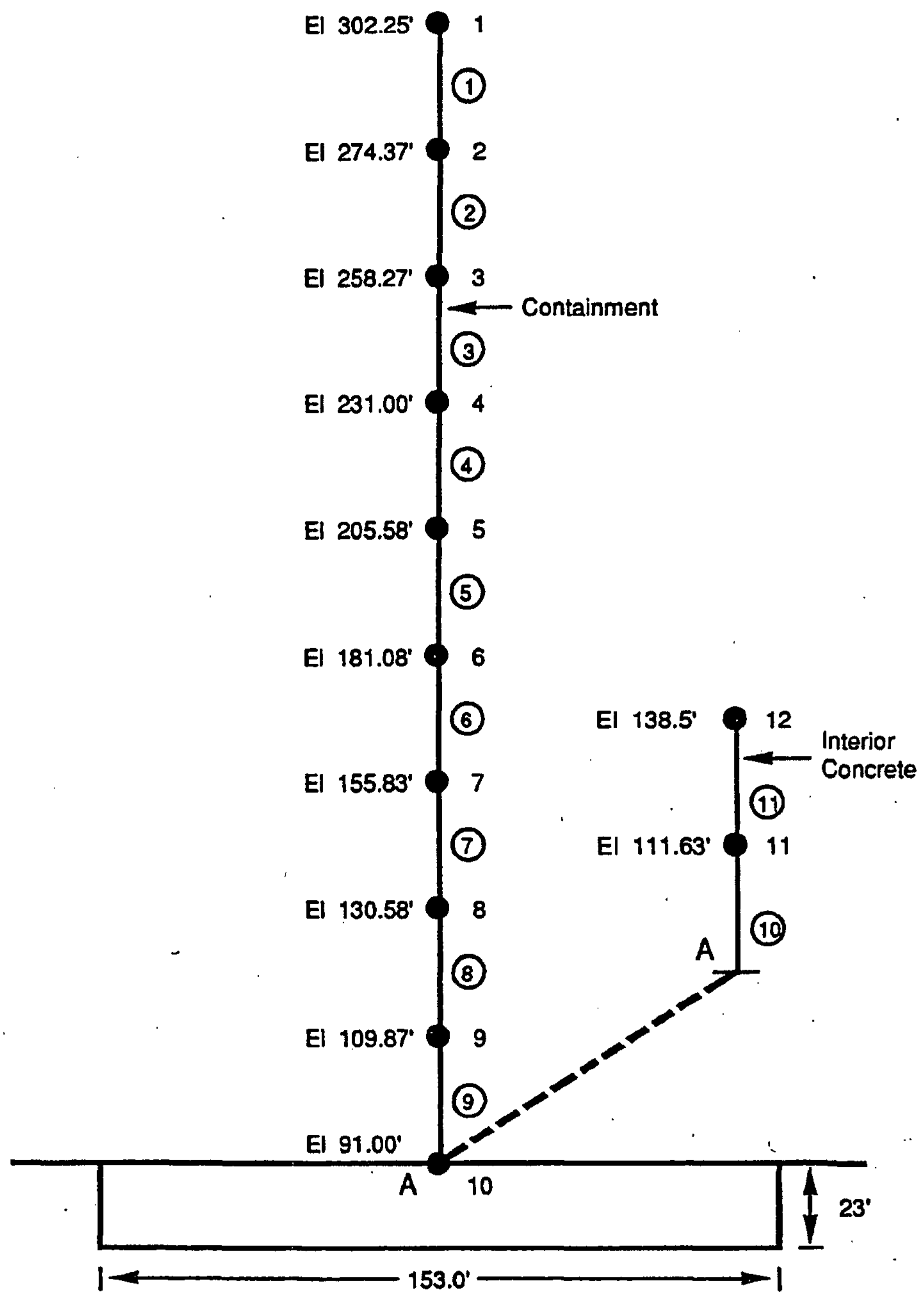


Figure 5-11

Simplified lump-mass stick model of the containment structure used in the parametric studies.

CLASSI/SASSI Solution Techniques. CLASSI and SASSI produce solutions that are closely comparable with each other so that the choice of either solution method and computer program for a specific application can be based simply on the suitability of foundation model assumptions for the specific application. Representative comparisons of the floor response spectra determined from CLASSI and SASSI analyses for a common parametric case involving the response of the containment base and top of the interior structure to vertically propagating SV wave inputs are shown in Figures 5-12 and 5-13, respectively.

Foundation Embedment Effect. The foundation embedment effect is relatively important and, thus, should be considered in the final soil/structure interaction models for the power block structures. This is demonstrated by the comparison shown in Figure 5-14 of floor response spectra at El 140 feet of the auxiliary building obtained from SASSI analyses assuming surface-supported versus embedded foundation conditions.

Structure-to-Structure Interaction Effect. The through-rock, multiple-structure-to-structure interaction effect is relatively unimportant; thus, it can be neglected in the soil/structure interaction analyses for the power block structures. This is demonstrated by the comparison shown in Figure 5-15 of the floor response spectra at the top of containment interior concrete structure obtained from SASSI analyses assuming single-embedded versus multiple-embedded foundation conditions.

Non-Vertical Wave Propagation Effect. The use of non-vertical seismic wave input motions was found to generally result in reductions in the seismic response; thus, the use of vertical plane wave input for soil/structure interaction analysis is conservative. This is demonstrated by the comparison shown in Figure 5-16 of the floor response spectra at El 140 feet of the auxiliary building obtained from SASSI analyses assuming vertical SV wave versus inclined SV-30-degree wave inputs. Furthermore, the use of vertically propagating wave input precludes double counting of the effect of horizontal spatial variations of ground motions when such a variation is included in the ground-motion spatial incoherence model and incorporated in the soil/structure interaction analysis.

Basemat Flexibility Effect. The effect of foundation basemat flexibility was shown to be relatively important for the auxiliary building. This is demonstrated by the comparisons shown in Figure 5-17 of the transfer function amplitudes at the core west location of the floor at El 140 feet of the auxiliary building obtained from SASSI analysis assuming five different basemat flexibility conditions as shown in Figure 5-18. Thus, for those structures having basemats of large plan dimensions such as the auxiliary and turbine buildings, the basemat flexibility should be considered in the soil/structure interaction models.

Rock Property Variation Effect. The effect of strain-dependency of site rock shear modulus was found to be insignificant (maximum reduction of containment fundamental soil/structure interaction frequency was less than 8 percent) for seismic input intensities involving maximum ground acceleration as high as 1.0 g. The effects of variations in the Poisson's ration and material damping ration of the rock within the ranges of values considered appropriate was found to be negligible.

Based on the above conclusions, the SASSI computer program was selected for the final soil/structure interaction analysis application because of its capability to include the effects of foundation embedment and basemat flexibility. The SASSI finite-element foundation models developed for the power block structures for the final analysis are shown in Figure 5-19 for the containment structure, in Figure 5-20 for the auxiliary building, and in Figure 5-21 for the turbine building.

Ground-Motion Input for Soil/Structure Interaction Analysis

The basic data of seismic ground-motion input for soil/structure interaction analysis were provided by the ground-motion studies (Chapter 4). These data consisted of the median and 84th percentile, horizontal and vertical site-specific

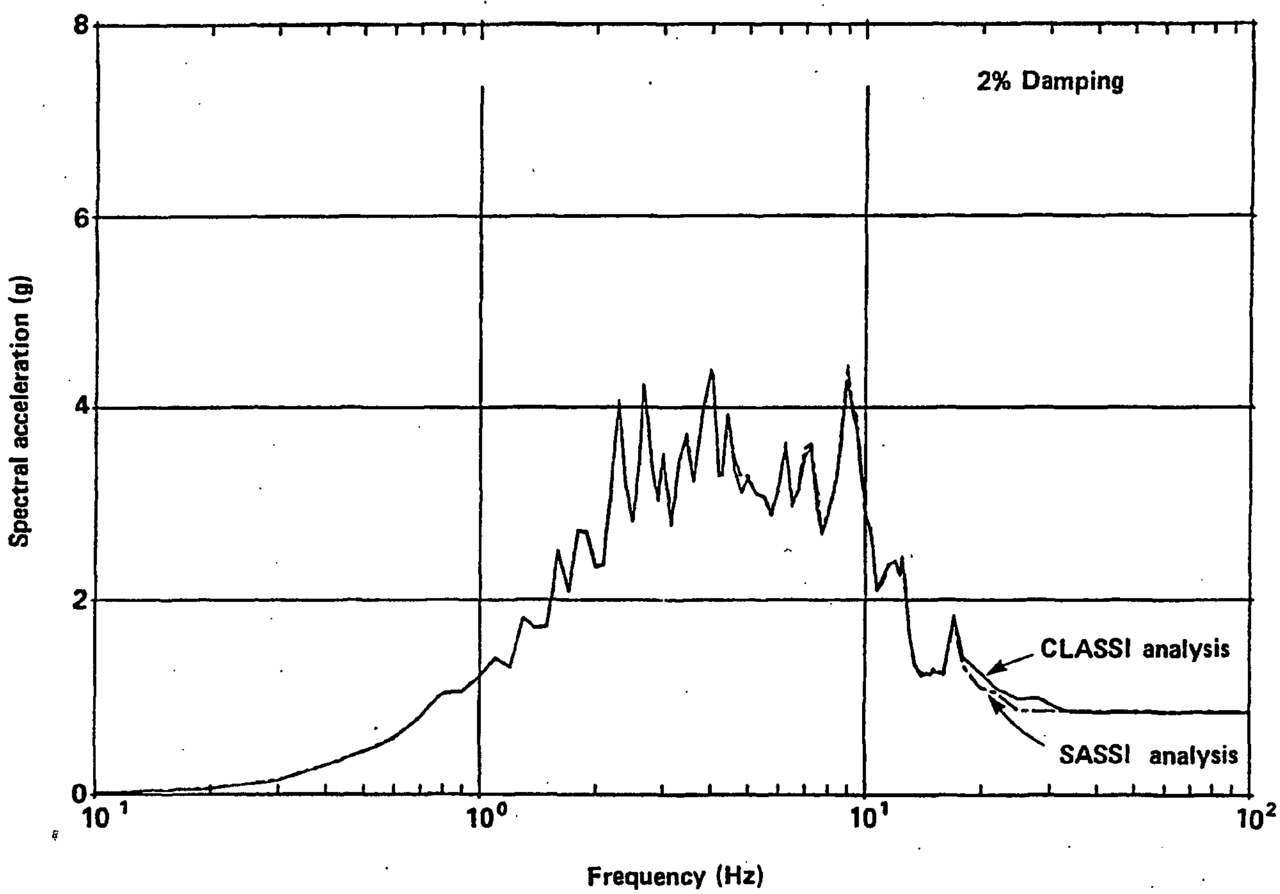


Figure 5-12

Comparisons of floor response spectra obtained from CLASSI and SASSI analysis for the east/west response at the containment base.

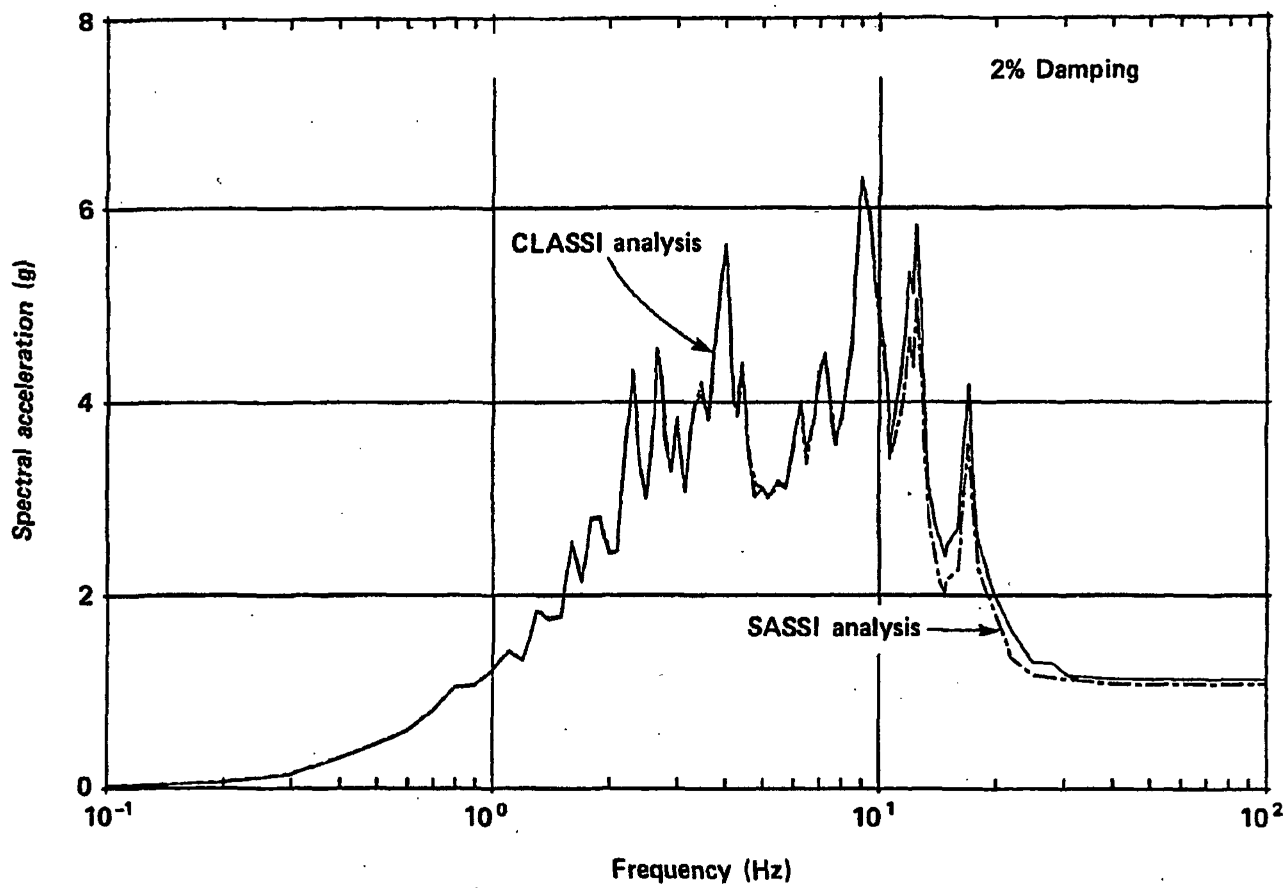


Figure 5-13

Comparisons of floor response spectra obtained from CLASSI and SASSI analyses for the east/west response at the top of interior concrete structure.

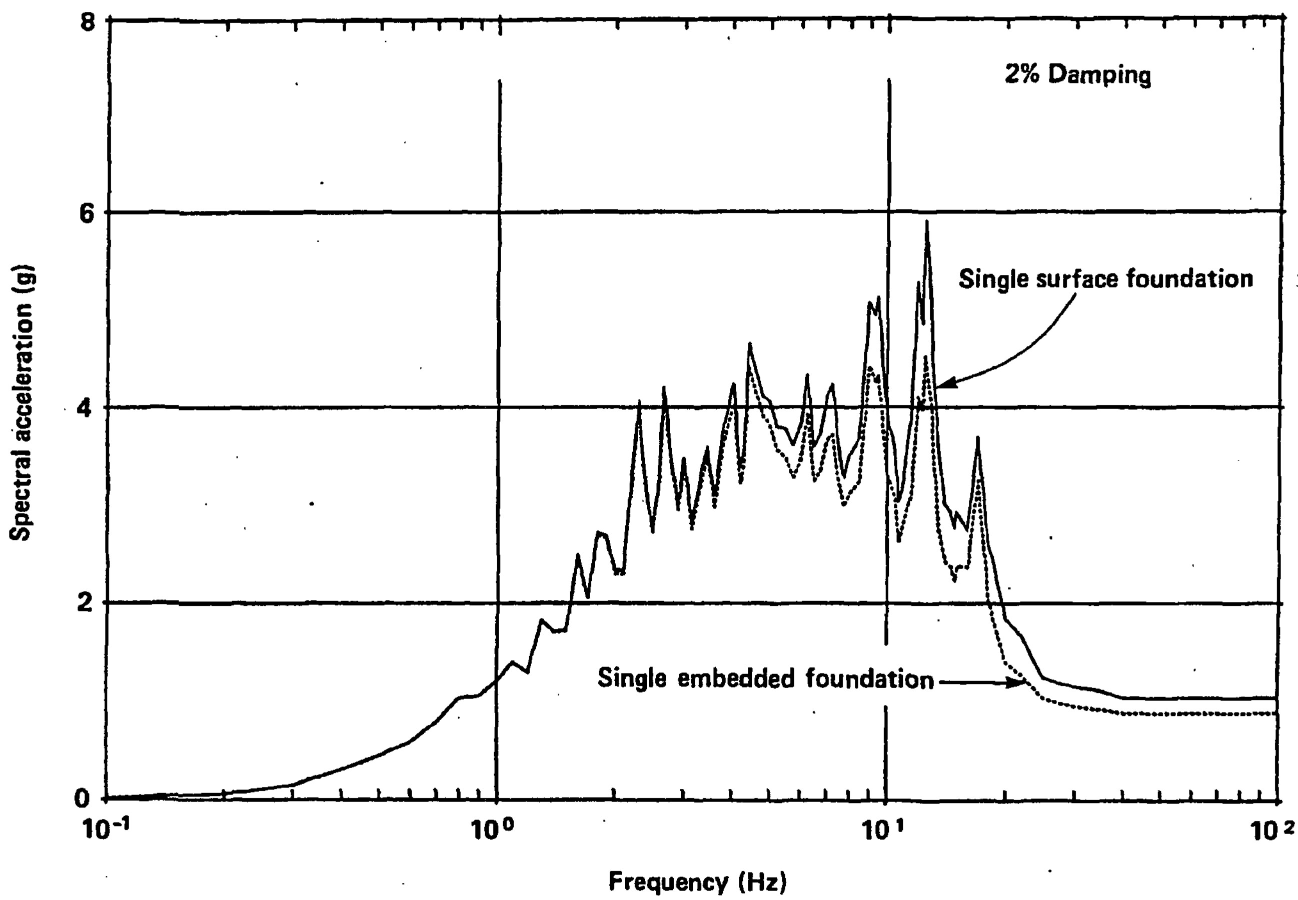


Figure 5-14

Comparison of floor response spectra obtained from SASSI analysis assuming surface-supported versus embedded foundation conditions for the auxiliary building at El 140 feet.

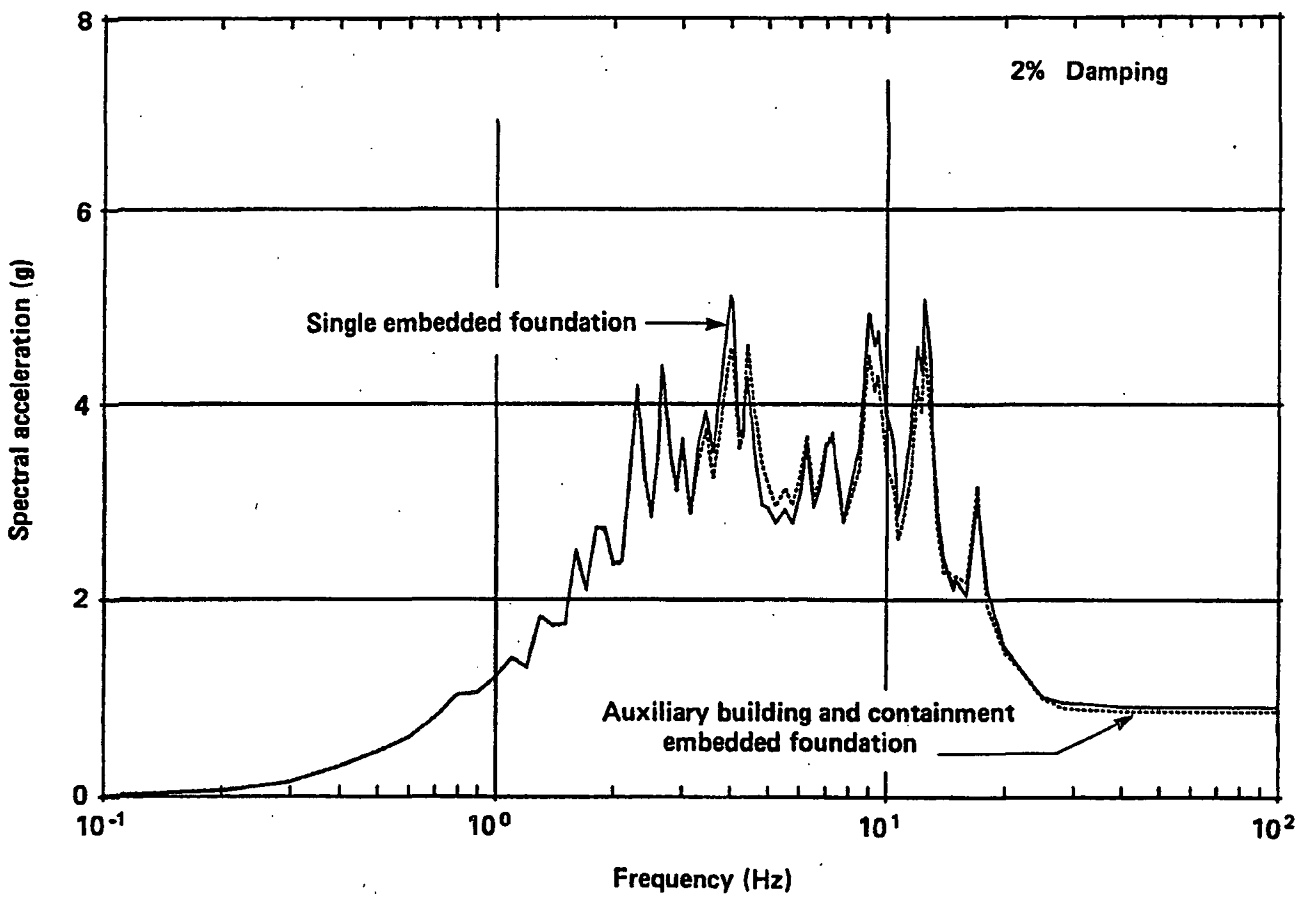


Figure 5-15

Comparison of floor response spectra obtained from SASSI analyses assuming single embedded foundation versus multiple-embedded foundations.

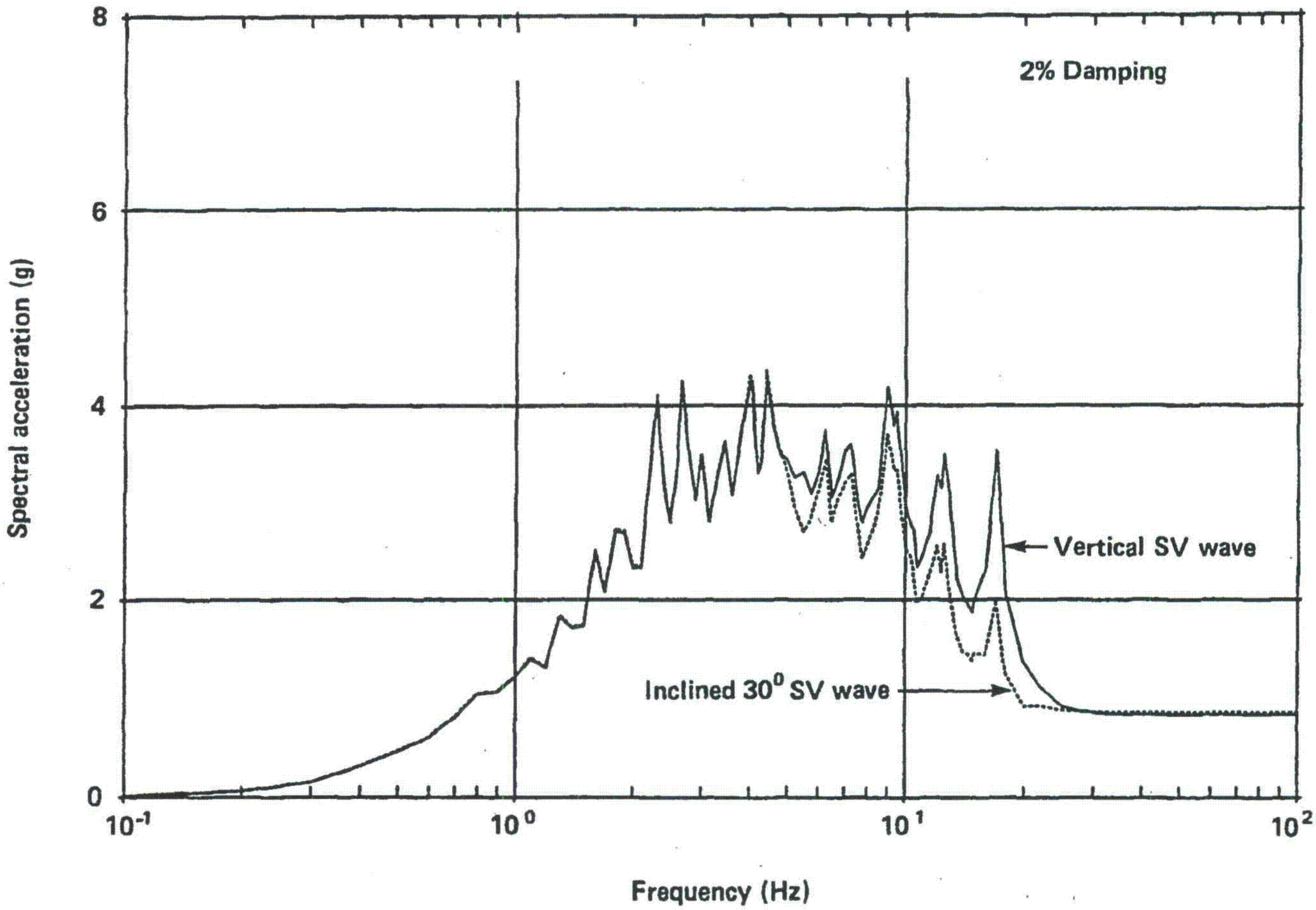


Figure 5-16

Comparison of floor response spectra at El 140 feet of the auxiliary building obtained from SASSI analyses with vertical SV wave input versus inclined SV wave input.

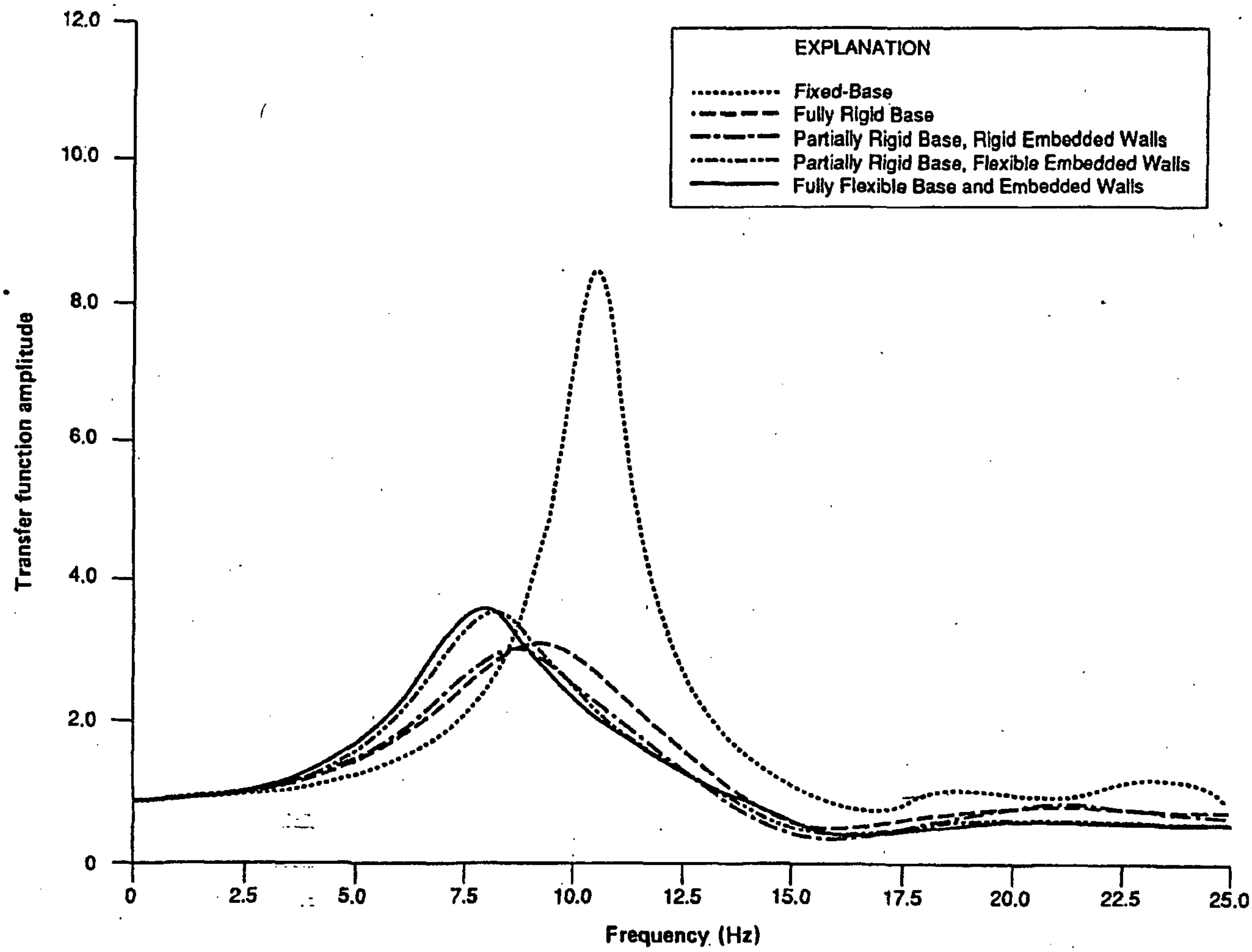


Figure 5–17

Transfer functions for east/west response at core west El 140 feet of the auxiliary building for various conditions of foundation systems.

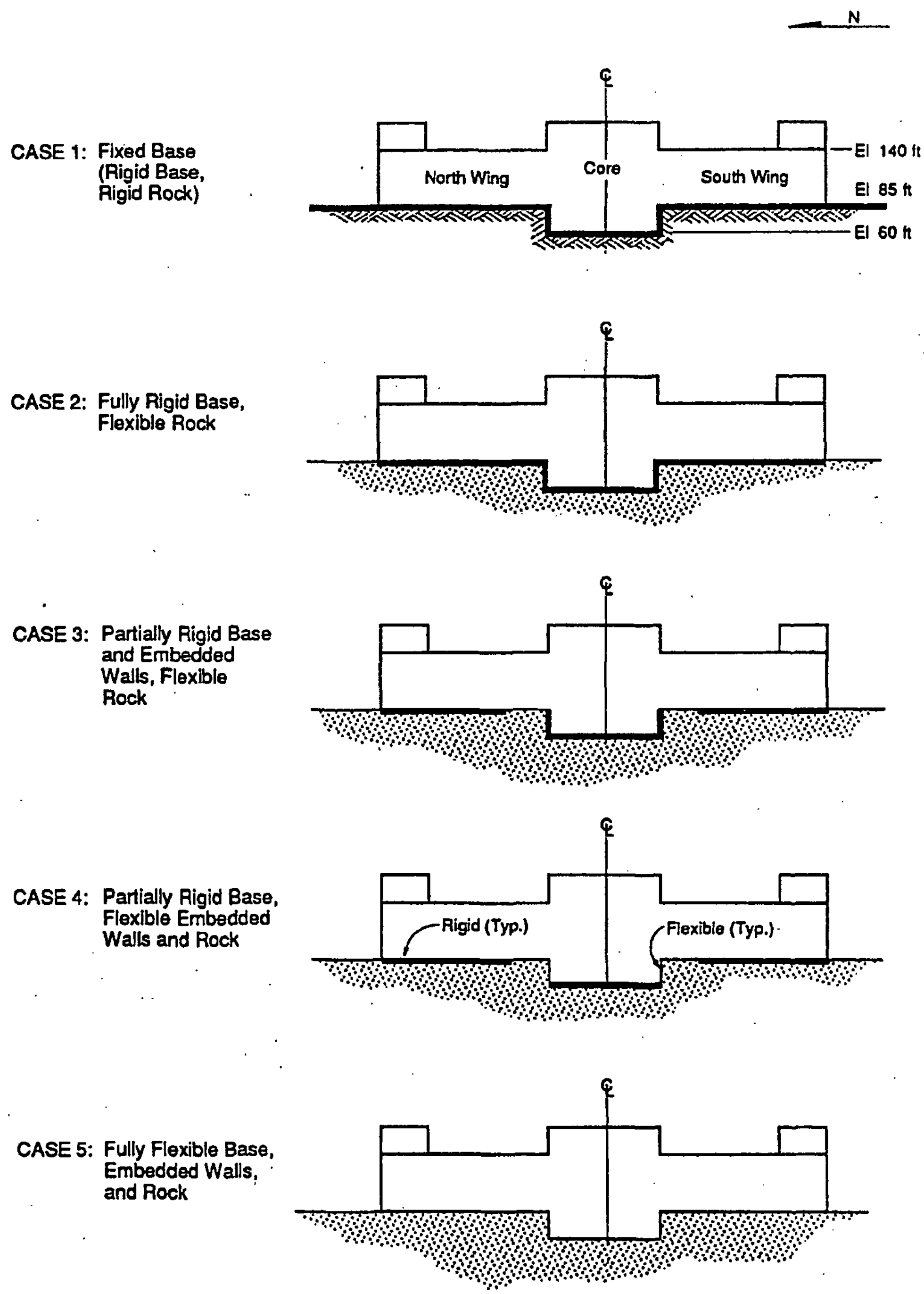


Figure 5–18

Various foundation basemat flexibility assumptions for the auxiliary building considered in the parametric studies.

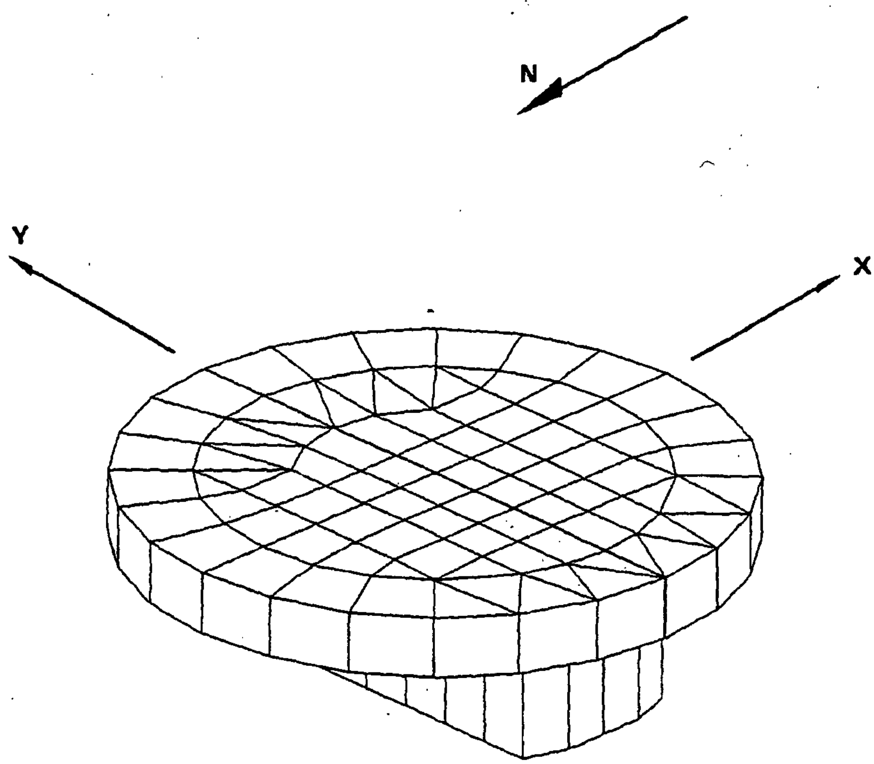


Figure 5-19

SASSI foundation model for containment structure.

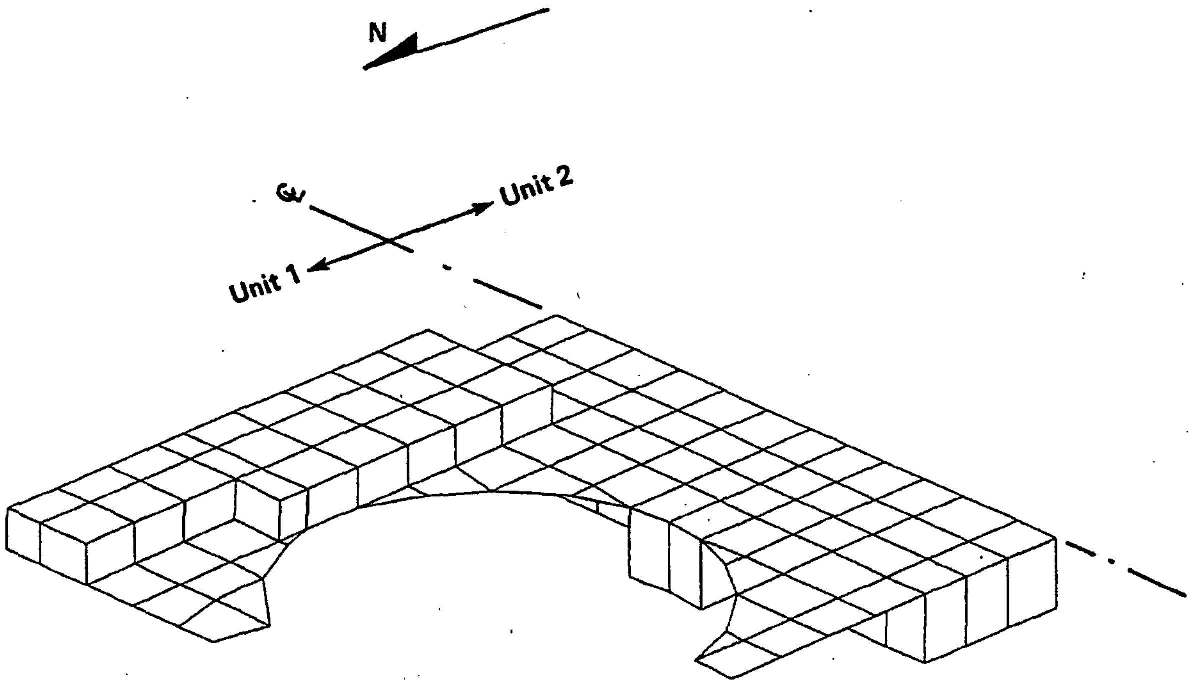
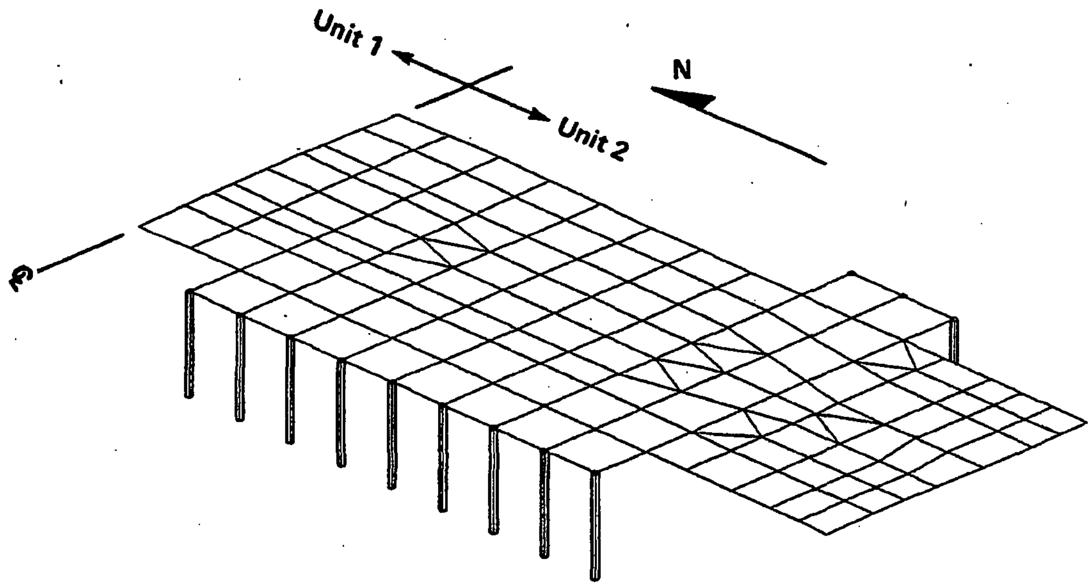
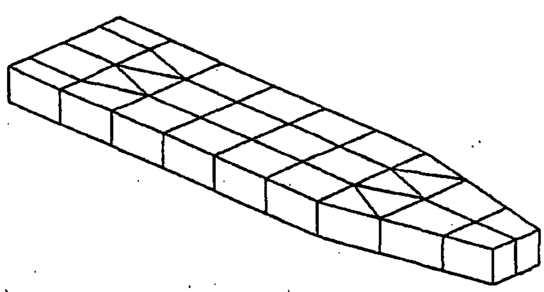


Figure 5-20

SASSI foundation half-model for auxiliary building.



Unit 2 turbine building foundation model



Unit 2 turbine pedestal foundation model

Figure 5-21

SASSI foundation model for Unit 2 turbine building.

earthquake acceleration response spectra for the Diablo Canyon site. Both the median and the 84th percentile spectra, normalized with respect to peak ground acceleration values, have almost the same spectral shape. Thus, it is only necessary to consider one set (median or 84th percentile) of these spectra for linear soil/structure interaction analysis, because the responses so obtained can be linearly scaled up or down, based on the peak ground acceleration ratio, to obtain the soil/structure interaction responses for any desired level of input.

Associated with the site-specific response spectra, three sets of three-component actual earthquake ground-motion time histories were selected and provided by the ground-motion study for soil/structure interaction analysis applications. These three sets of ground-motion records are: (a) the Pacoima Dam records of the 1971 San Fernando earthquake; (b) the Tabas records of the 1978 Tabas earthquake; and (c) the El Centro No. 4 records of the 1979 Imperial Valley earthquake. Two of the three sets of ground-motion records provided (Pacoima and Tabas) were actually used for final soil/structure interaction analyses.

Before these motions were applied for soil/structure interaction analysis, the following step-by-step procedure was used to adjust the motions:

(1) The original recorded motions were adjusted to conform to site-specific conditions, such as the maximum earthquake magnitude, source-to-site distance, and site condition.

(2) The two horizontal components of the motions were transformed, as necessary, into the longitudinal and transverse horizontal components to provide motions in the directions normal and parallel to the strike of the causative fault.

(3) The longitudinal and transverse time histories were both modified by adjusting the Fourier amplitudes, but keeping the Fourier phase-angles unchanged, so that the resulting time history response spectra closely matched the median site-specific horizontal spectra of several damping values. Likewise, the vertical component time histories were modified to match the median site-specific vertical spectra of several damping values.

(4) The three-component time histories were scaled upward by a constant scaling factor common to all three components to correspond to a reference seismic input level for Plant fragility evaluation purposes.

(5) Because the Plant north/south direction is approximately parallel to the strike of the Hosgri fault zone, the modified and scaled three-component time histories were applied as the input for soil/structure interaction analyses; first, with the longitudinal component applied in the Plant north/south direction, and the transverse component in the plant east/west direction; then vice versa, the vertical component was applied in the Plant vertical direction. The interchanging of the two horizontal components for input was done to allow for uncertainties in the time history phasing, because both the Pacoima and Tabas motions were initiated by thrust events.

For Plant fragility evaluation applications, the constant scaling factor used in step (4) above, was derived in such manner that the average spectral value of the 5 percent damped site-specific horizontal spectral acceleration in the frequency range from 4.8 hertz to 14.7 hertz, equal to the fragility evaluation reference spectral acceleration of 2 g. The frequency range was chosen considering the fragility evaluations described in Chapter 6. This procedure is illustrated in Figure 5-22. The resulting scaling factor was 1.6, and the peak spectral acceleration of the resulting horizontal spectrum was about 2.2 g. The fragility evaluation reference spectra so obtained are

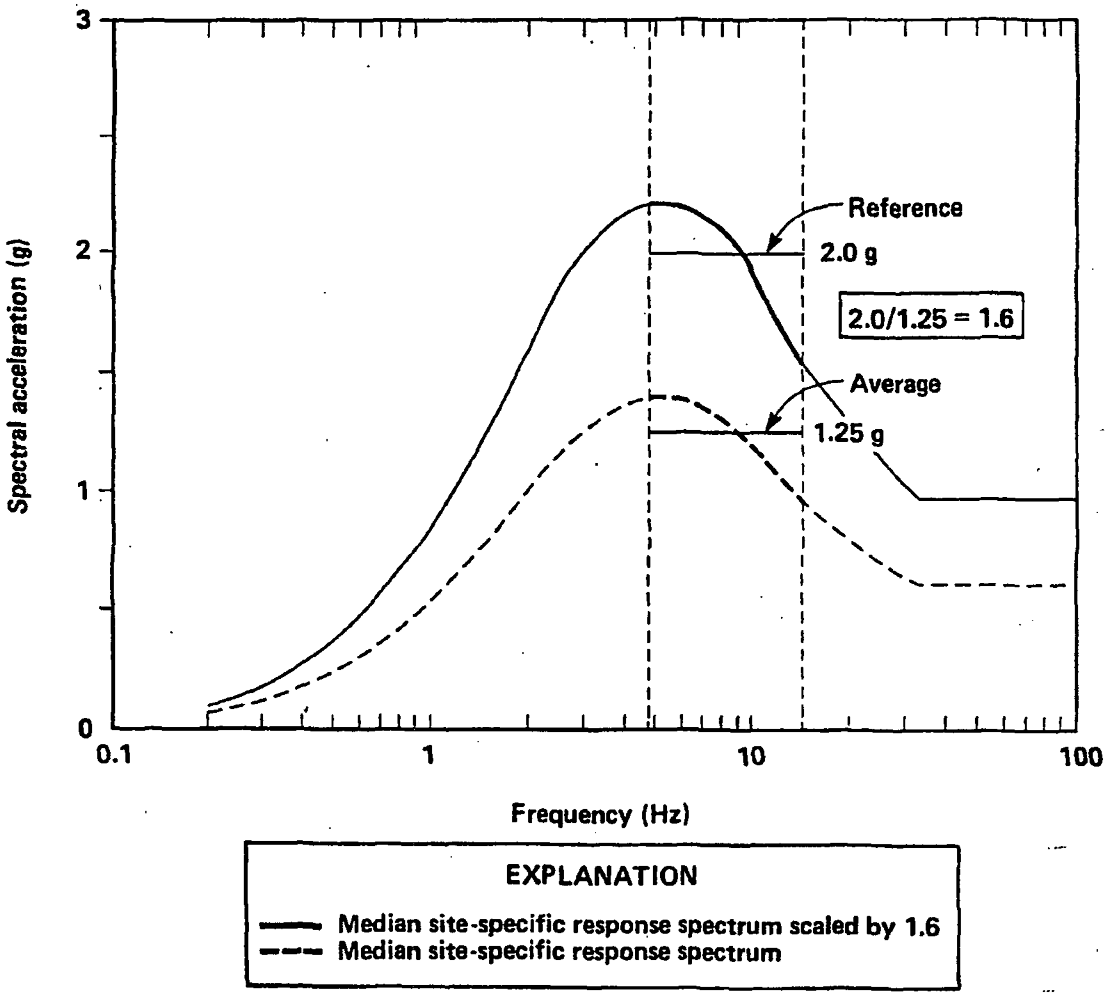


Figure 5-22

Illustrative procedure for obtaining the 5 percent damped horizontal reference spectrum for soil/structure interaction analyses.

slightly higher than the 84th percentile response spectra for the site.

The final three-component Pacoima time histories, which have been modified to match the median spectra shapes and subsequently scaled up by the 1.6 factor, are shown and compared with the unmodified, time histories from step (2) above (but scaled to the maximum acceleration of 0.96 g for the horizontal components) in Figures 5-23, 5-24, and 5-25, respectively for the longitudinal, transverse, and vertical components. The comparisons of the 5 percent damped final Pacoima time history response spectra with the 5 percent damped fragility evaluation reference response spectra are shown in Figures 5-26, 5-27, and 5-28. Similar comparisons for the three-component Tabas time histories are shown in Figures 5-29, 5-30, and 5-31; and similar comparisons for response spectra are shown in Figures 5-32, 5-33, and 5-34.

As shown in these comparisons, the modified final time history response spectra closely match the corresponding reference response spectra, which are about 10 percent higher than the 84th percentile response spectra discussed previously. Furthermore, as a result of keeping the time history Fourier phases unchanged during the time history modifications for spectrum compatibility, the final spectrum-compatible time histories maintain realistic characteristics and appearances, and resemble the time histories of the motions before modifications.

Generation of Soil/Structure Interaction Responses to Coherent Ground-Motion Inputs

To generate the soil/structure interaction responses required for the Plant fragility evaluations, soil/structure interaction analyses were performed using the SASSI computer program, the soil/structure interaction models developed for the power block structures, and the ground motions described previously. Because equipment fragilities are mostly dominated by horizontal responses (Chapter 6), only the horizontal north/south and east/west responses of the power block structure were generated.

For these analyses, the final scaled-up three-component spectrum-compatible Pacoima and Tabas time histories, shown in Figures 5-23 through 5-25, and Figures 5-29 through 5-31, respectively, were directly used as inputs for analyses. These input motions were assumed in the analyses to be the free-field surface motions prescribed at the plant grade (El 85 feet). The incident seismic wave field was assumed to be coherent, vertically propagating plane seismic shear and compression waves, respectively, for the horizontal and vertical components of the free-field motion. Because only the horizontal north/south and east/west responses were generated, the coupling between the two horizontal responses that exists for non-symmetrical structures was considered by combining the co-directional time history responses or by combining the floor response spectra using the rule of square-root-of-the-sum-of-squares. Under the vertically propagating plane wave assumption, the contributions to the horizontal responses due to the vertical input motion are negligible; thus, they were not considered in the response combinations to obtain the north/south and east/west horizontal responses.

The results of the soil/structure interaction analyses were obtained and provided for use in the Plant fragility evaluation in terms of 5 percent damped horizontal north/south and east/west floor response spectra at selected locations in the power block structures. Floor response spectra for both sets of input motions, namely, the Pacoima and the Tabas inputs, were generated. Representative results obtained from both sets of input motions are shown in Figures 5-35 and 5-36 for the north/south response of the containment at the base (El 85 feet) and at the top of the interior structure (El 138.5 feet), respectively. Similarly, the results for the north/south response of the auxiliary building at El 85 feet and El 140 feet of the core west

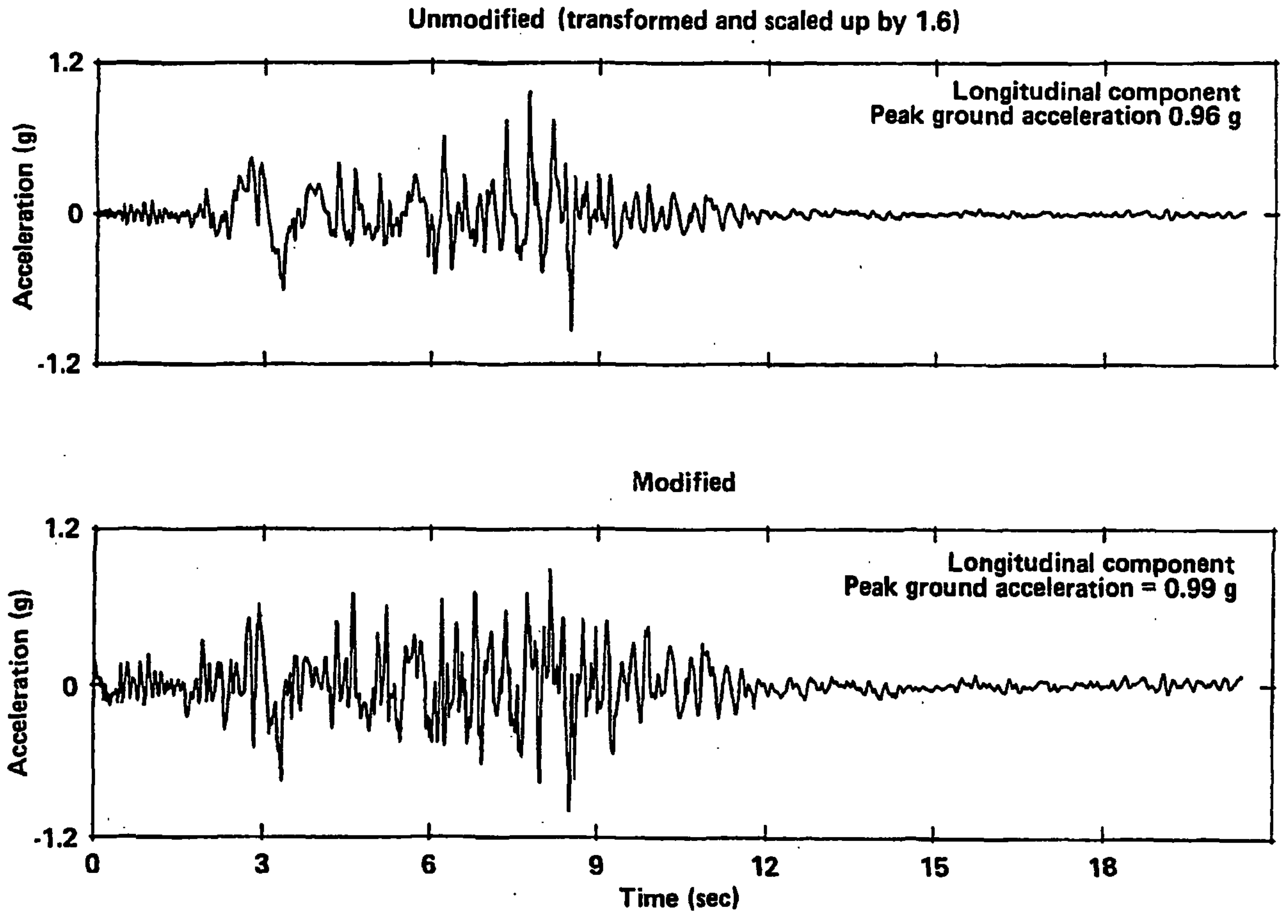


Figure 5-23

Comparisons of unmodified and modified Pacoima acceleration time histories, longitudinal component.

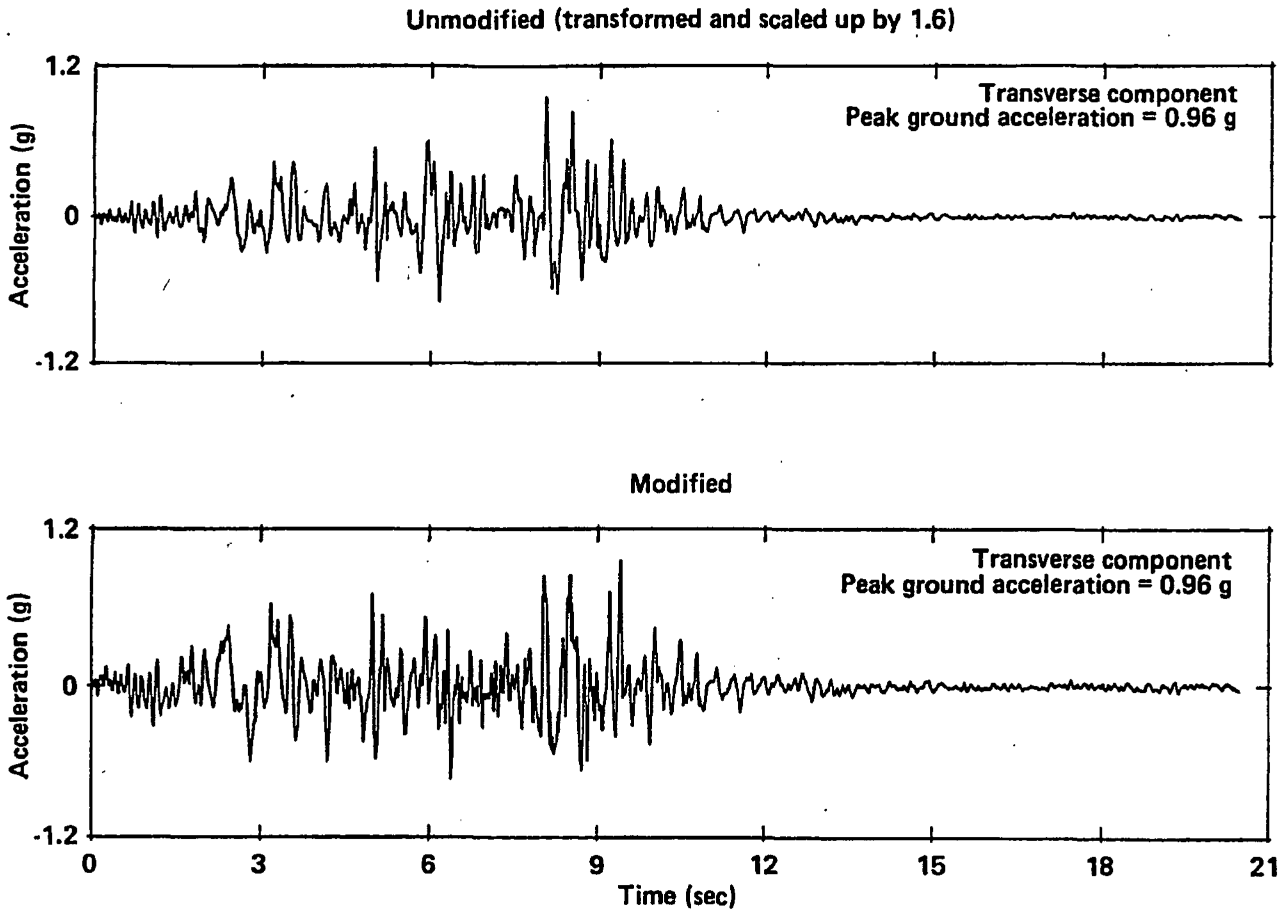


Figure 5-24

Comparisons of unmodified and modified Pacoima acceleration time histories, transverse component.

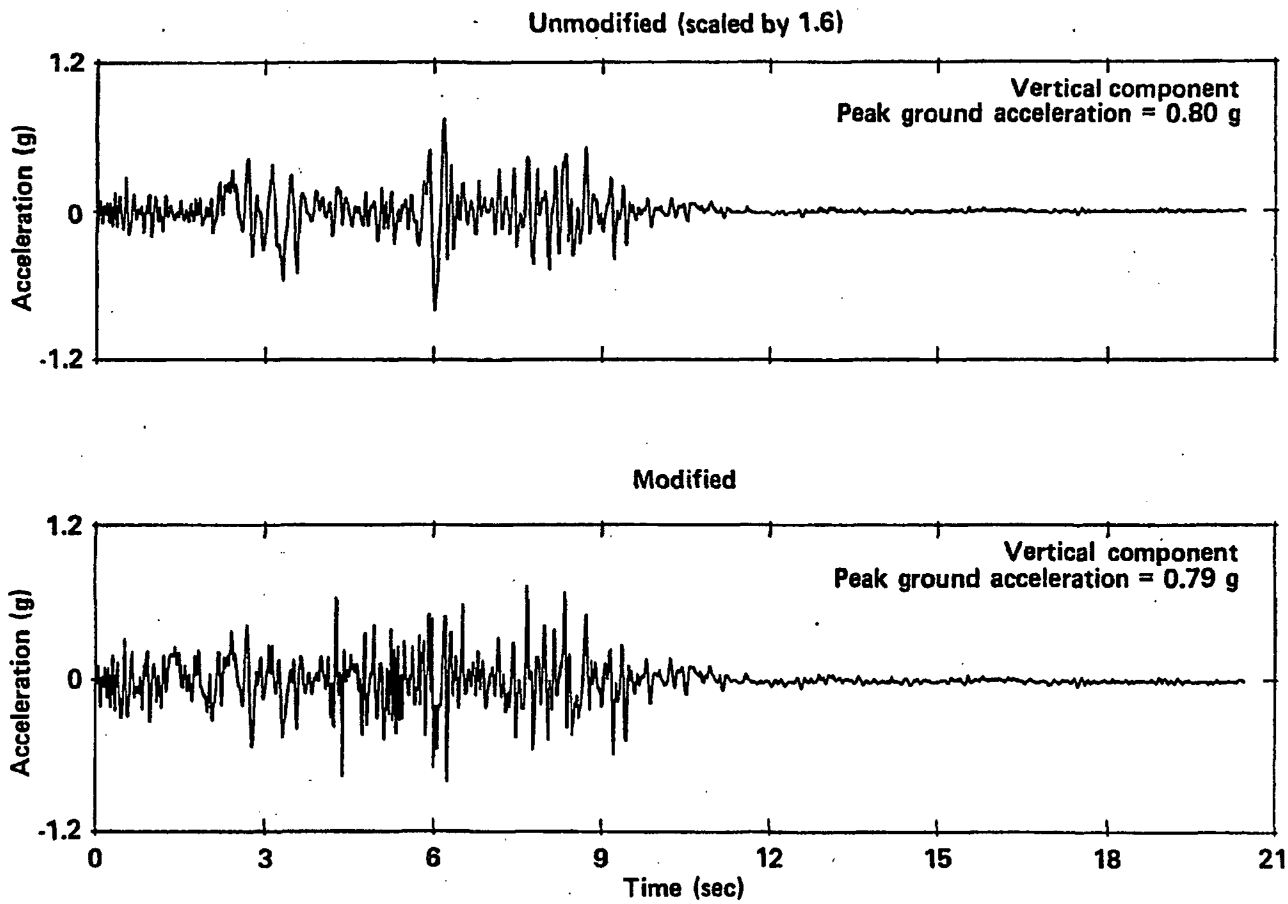


Figure 5-25

Comparisons of unmodified and modified Pacoima acceleration time histories, vertical component.

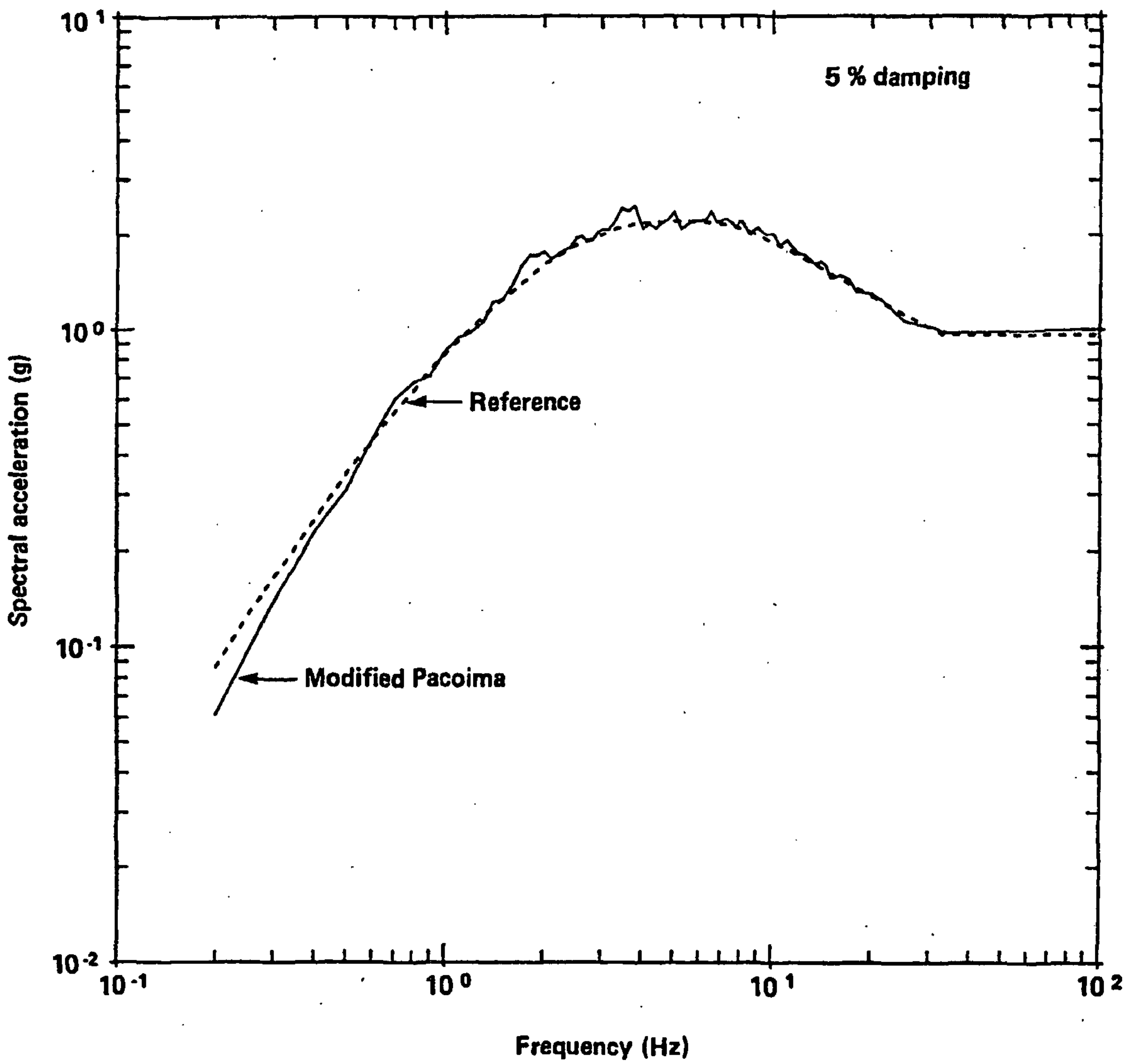


Figure 5-26

Comparisons of modified Pacoima time history response spectrum and fragility evaluation reference response spectrum, longitudinal component.

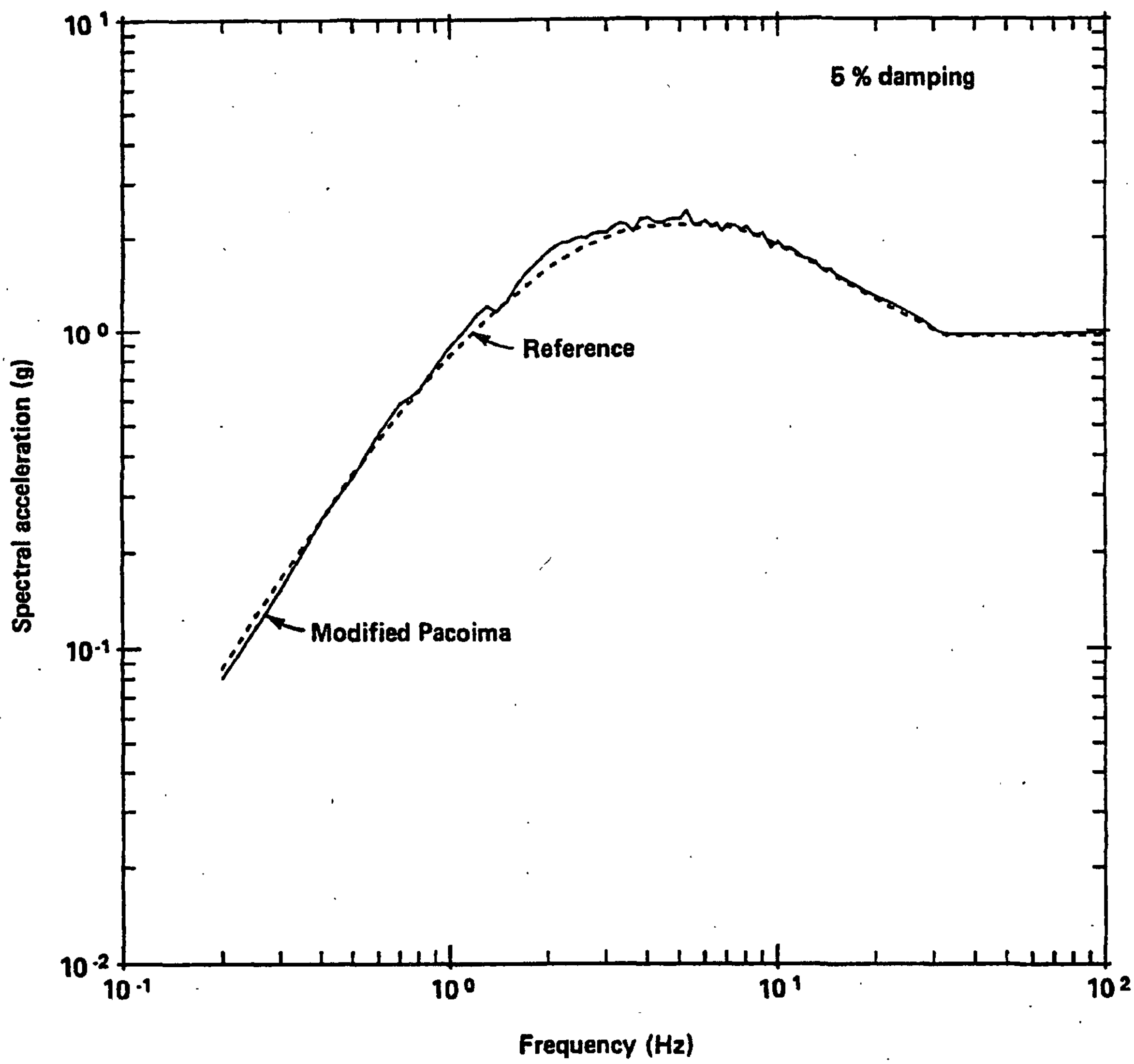


Figure 5-27

Comparisons of modified Pacoima time history response spectrum and fragility evaluation reference response spectrum, transverse component.

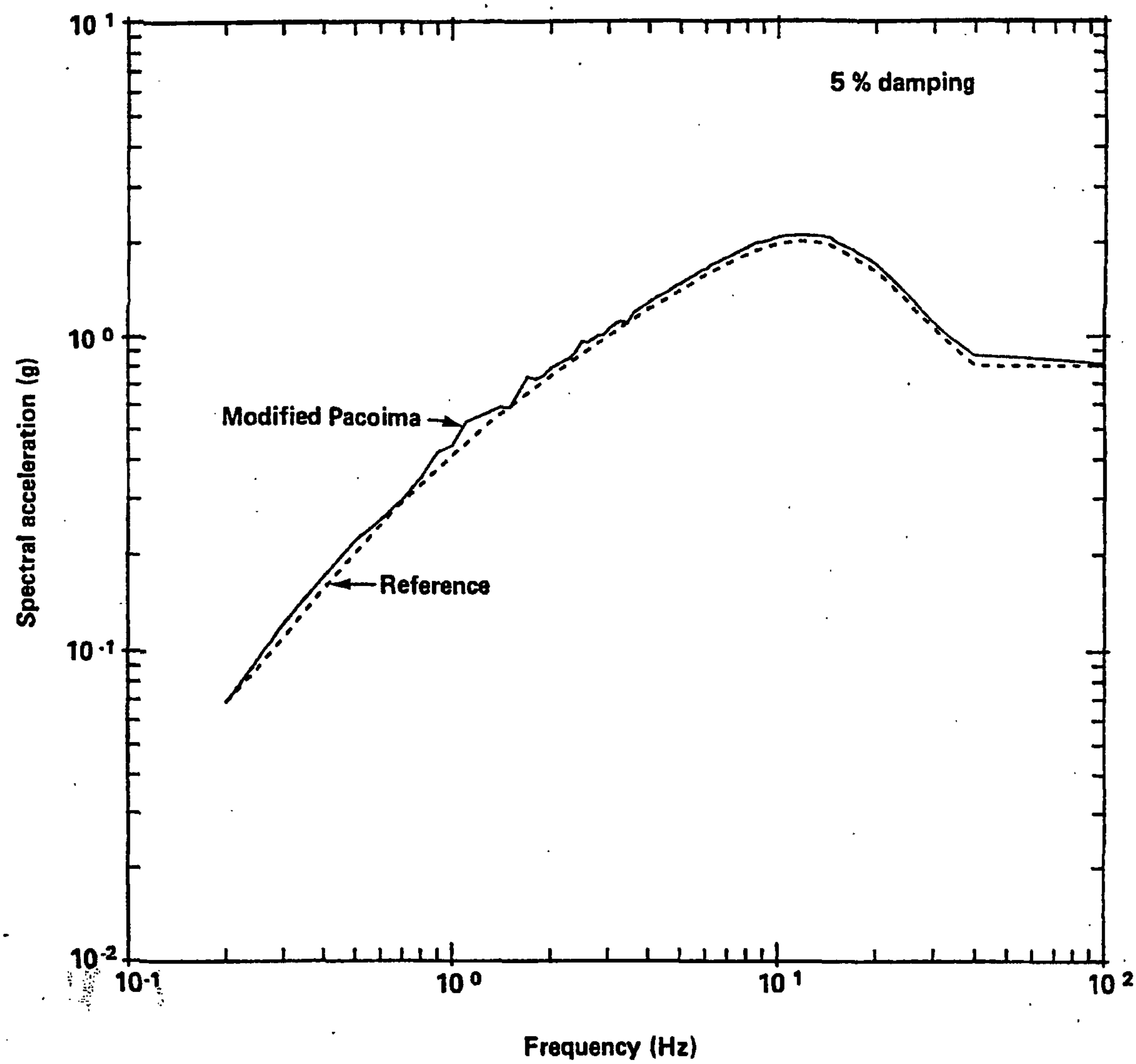


Figure 5-28

Comparisons of modified Pacoima time history response spectrum and fragility evaluation reference response spectrum, vertical component.

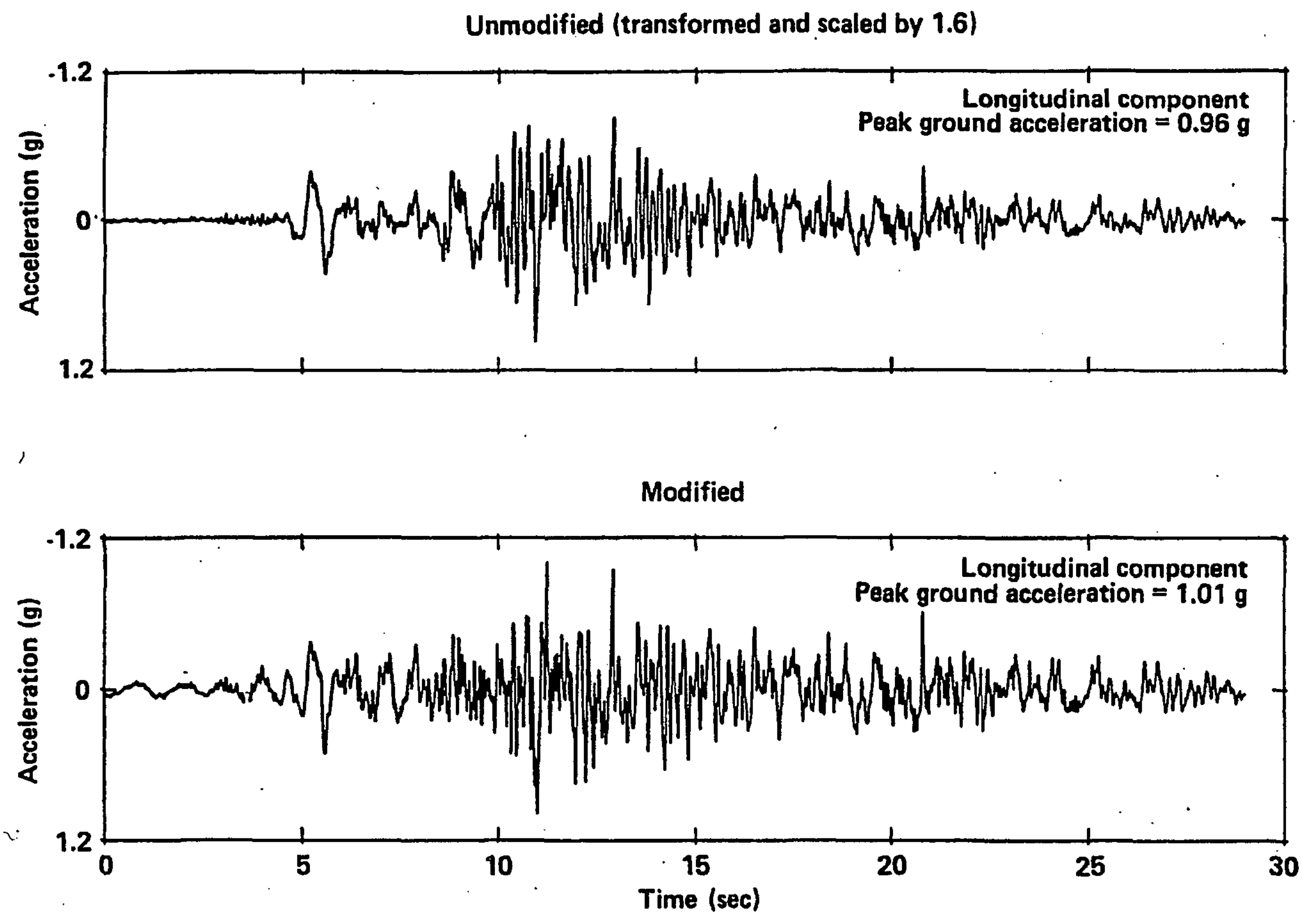


Figure 5-29

Comparisons of unmodified and modified Tabas acceleration time histories, longitudinal component.

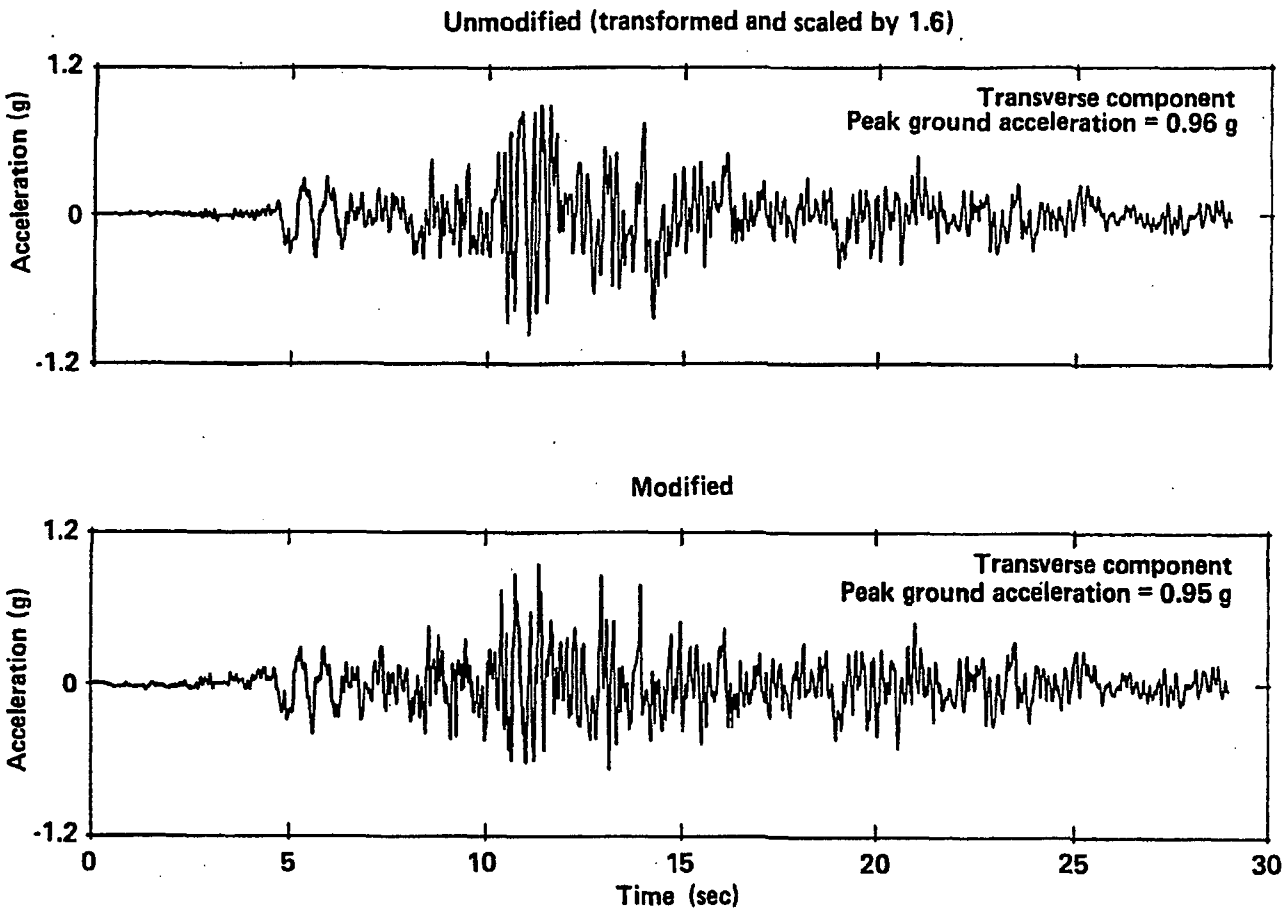


Figure 5-30

Comparisons of unmodified and modified Tabas acceleration time histories, transverse component.

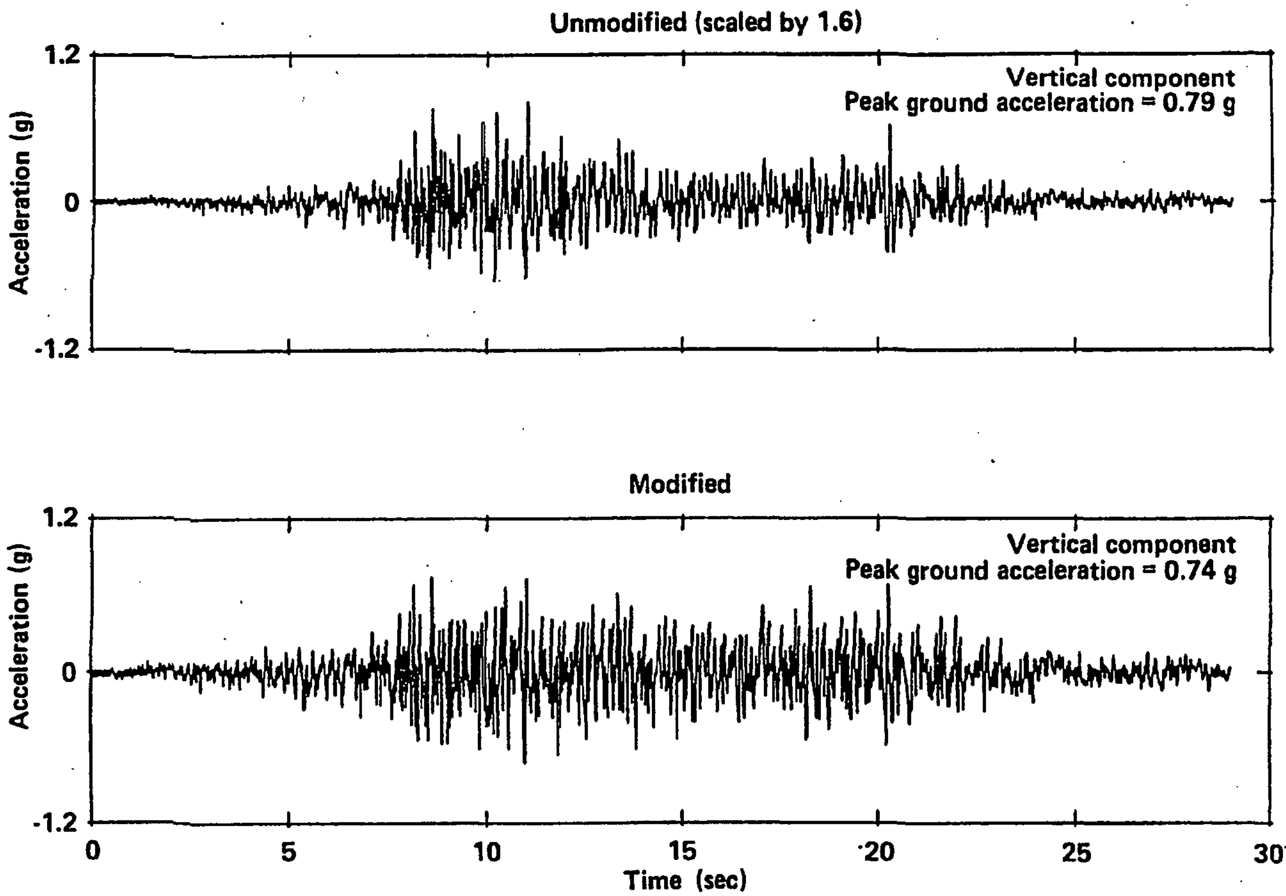


Figure 5-31

Comparisons of unmodified and modified Tabas acceleration time histories, vertical component.

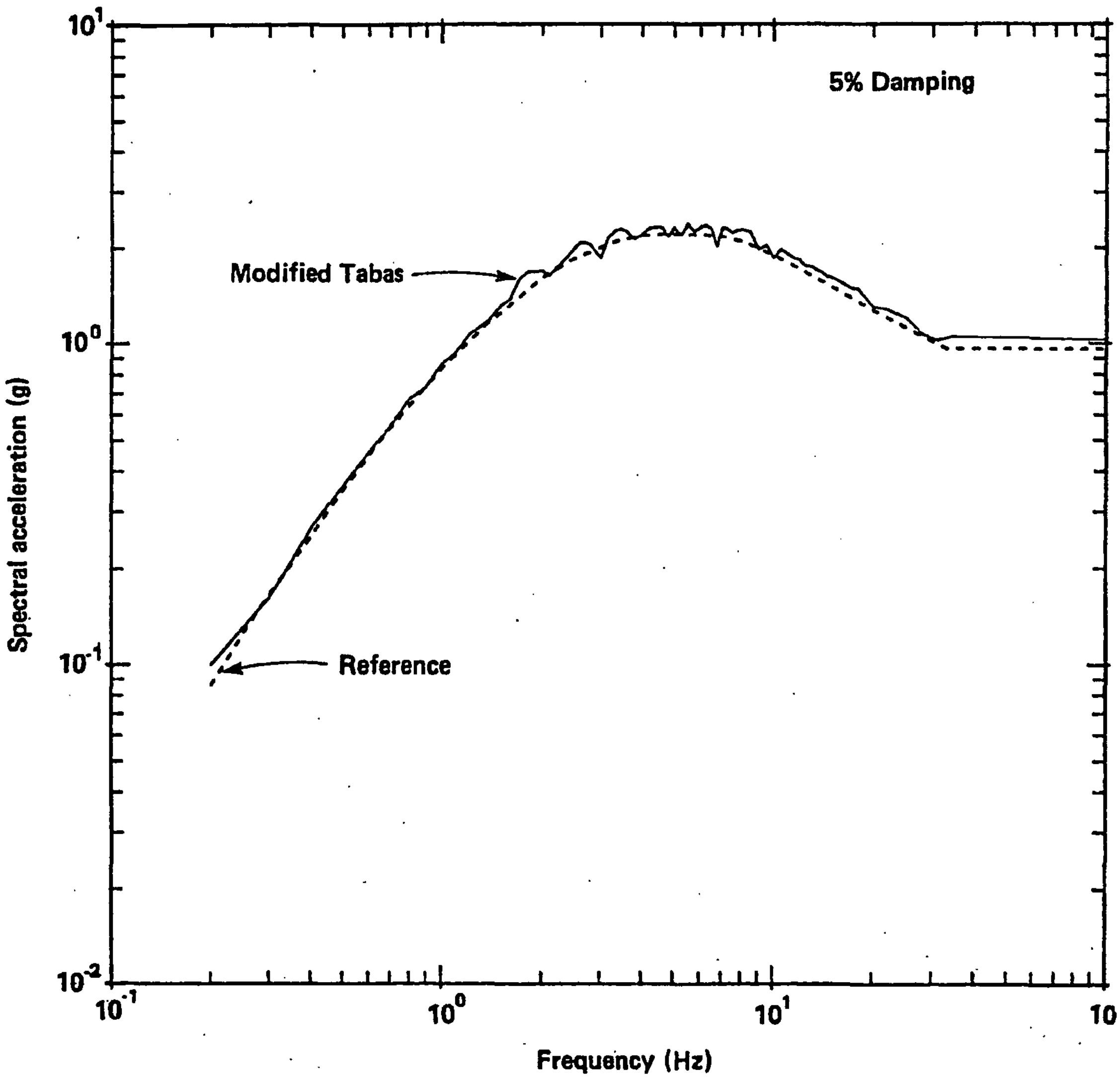


Figure 5-32

Comparisons of modified Tabas time history response spectrum and fragility evaluation reference response spectrum, longitudinal component.

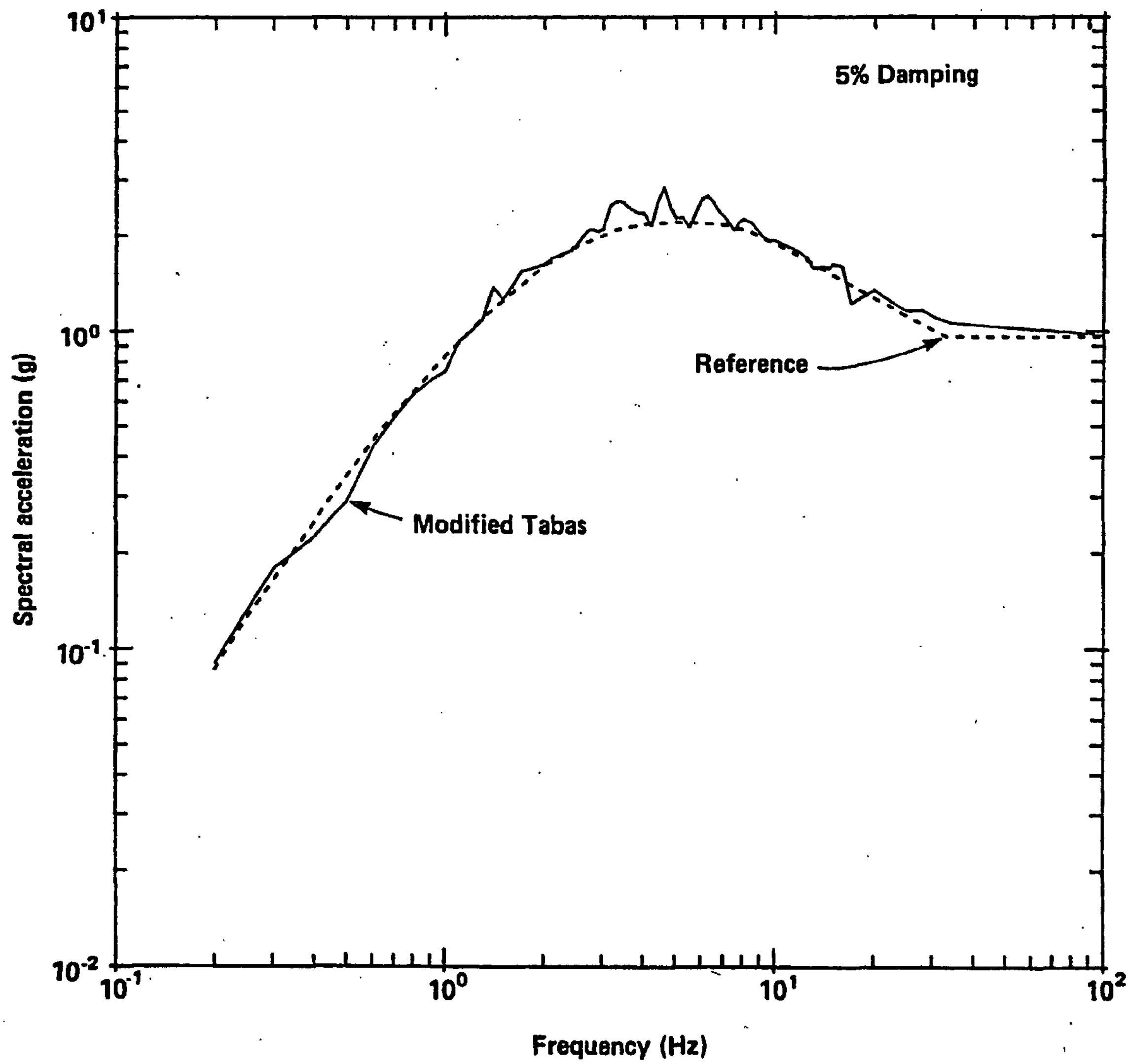


Figure 5-33

Comparisons of modified Tabas time history response spectrum and fragility evaluation reference response spectrum, transverse component.

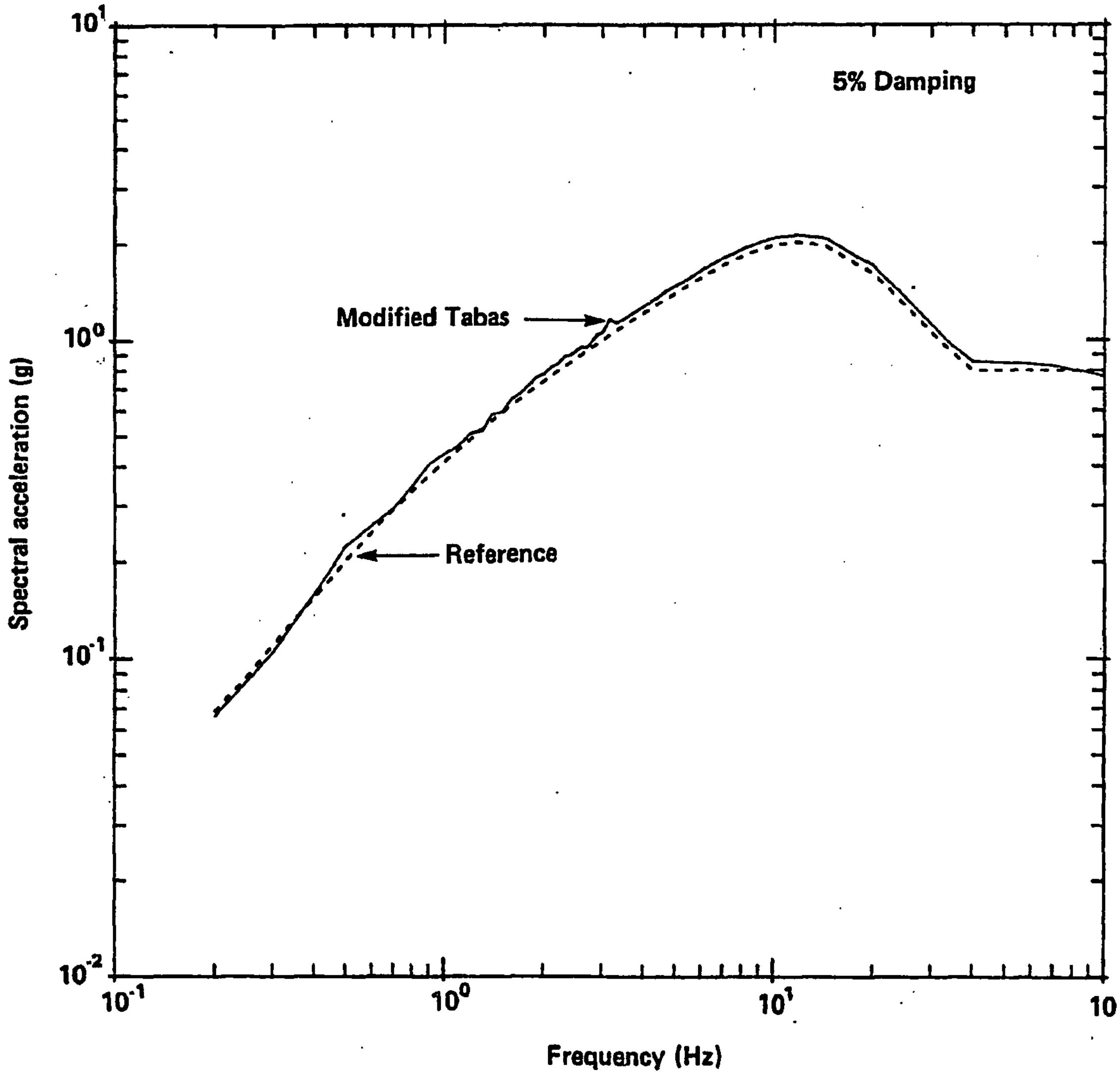


Figure 5-34

Comparisons of modified Tabas time history response spectrum and fragility evaluation reference response spectrum, vertical component.

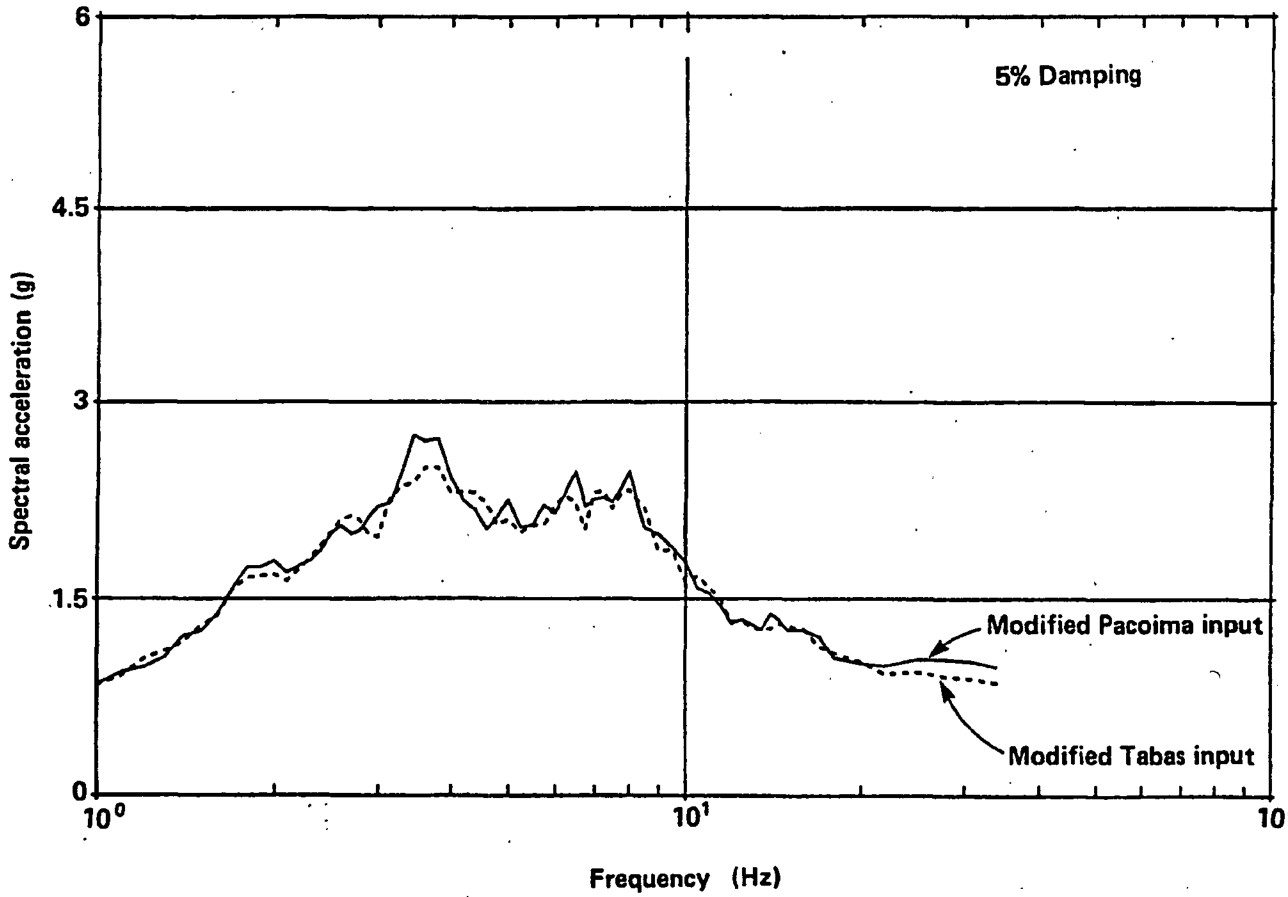


Figure 5-35

Floor response spectra for the north/south response of the containment at the base (El 85 feet) obtained from SASSI analyses with coherent ground motion input.

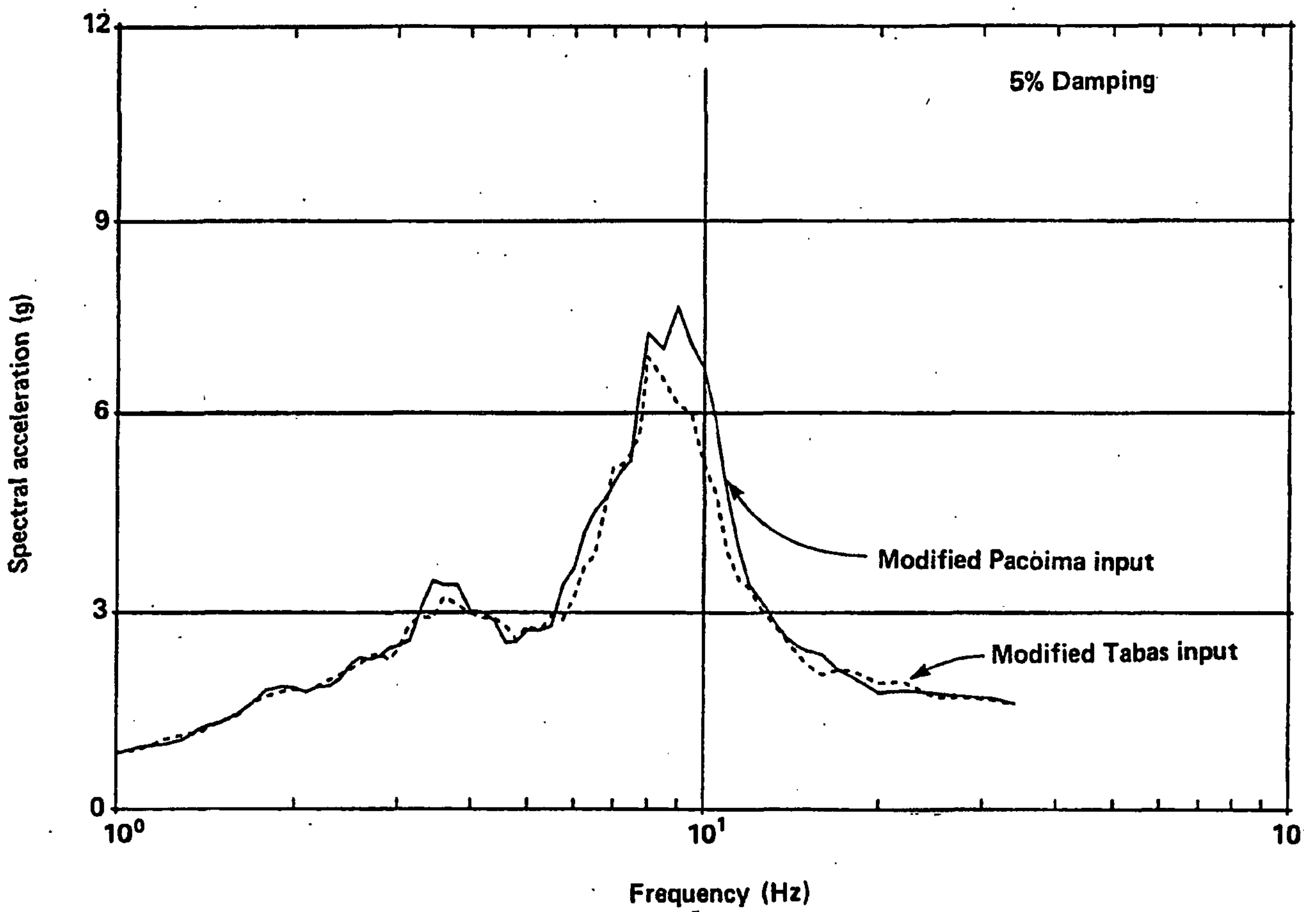


Figure 5-36

Floor response spectra for the north/south response of the containment at the top of interior concrete structure (El 138.5 feet) obtained from SASSI analyses with coherent ground motion input.

location are shown in Figures 5-37 and 5-38, respectively.

The soil/structure interaction responses resulting from the two sets of input motions (Pacoima and Tabas) were found to be consistent with each other for all response locations, as shown in Figures 5-35 through 5-38. Thus, the use of more sets of such motions as input was considered unnecessary. It was also found that interchanging the directions of the horizontal motion components had no significant effect on structural responses. The soil/structure interaction responses generated using the spectrum-compatible input motions as used herein also can be shown to be consistent with the responses that would be obtained from the ensemble averages of the responses to the individual inputs of the time history ensemble that forms the basis of the site-specific earthquake spectra.

Adjustment of Soil/Structure Interaction Responses Due to Spatial Incoherence of Ground Motions

The soil/structure interaction responses based on the assumption of vertical coherent plane wave input do not consider the effects of horizontal spatial variation of free-field ground motions. Thus, separate soil/structure interaction analyses were performed to develop the response adjustment factors that could be used to adjust the soil/structure interaction responses obtained from the coherent ground-motion input to account for the effect of spatial variations.

The characterization of spatial variation of free-field surface motions at the Diablo Canyon site was achieved using a set of site-specific spatial incoherence functions, as described in Chapter 4. Such functions consist of ground-motion coherency amplitudes (Figure 5-39), and the corresponding phase angles (Figure 5-40). These functions vary with the Fourier frequency of the surface motions and the separation distance between two points on the ground surface.

To use such spatial incoherence functions for soil/structure interaction analysis, the free-field ground surface motions at various points of the site within the foundation region were represented in the frequency domain, using a 3x3 ground-motion covariance matrix in which the on-diagonal elements represent the auto-power spectral density and the off-diagonal elements represent the cross-power spectral density for the three-components of the ground motions.

The ground-motion covariance matrix for the Diablo Canyon site was derived from the time history ensemble used for deriving the site-specific spectra. Thus, it is consistent with the site-specific earthquake spectra. The amplitude of one element of the covariance matrix, scaled-up by the factor of $(1.6)^2$ to correspond to the fragility evaluation reference input, is shown in Figure 5-41.

To incorporate the ground-motion covariance matrix in conjunction with the spatial incoherence functions for soil/structure interaction analyses, an analysis method was developed that is based on the random vibration theory of structural dynamics and uses the covariance matrix of the ground motions directly as the input.

Because the site-specific spatial incoherence functions were developed only for free-field surface motions, only the spatial variations of surface motions need be considered for soil/structure interaction analyses. Consequently, the analysis method developed to incorporate the site-specific spatial incoherence functions used the CLASSI method of soil/structure interaction analysis, which is applicable for surface-supported rigid foundations. The total method, which includes applying the CLASSI computer code for generating the scattered foundation input motions and soil/structure interaction response transfer functions, and the PROSPEC computer code (Lilhanand and Tseng, 1983) for generating the probabilistic floor response spectra based on random vibration theory, is shown schematically in Figure 5-42. Using this method, the spatial incoherence functions are incorporated into the ground-motion input at the step when the ground-motion covariance matrices for various points on the ground surface covered by the CLASSI foundation model are calculated, and then integrated to generate the scattered foundation

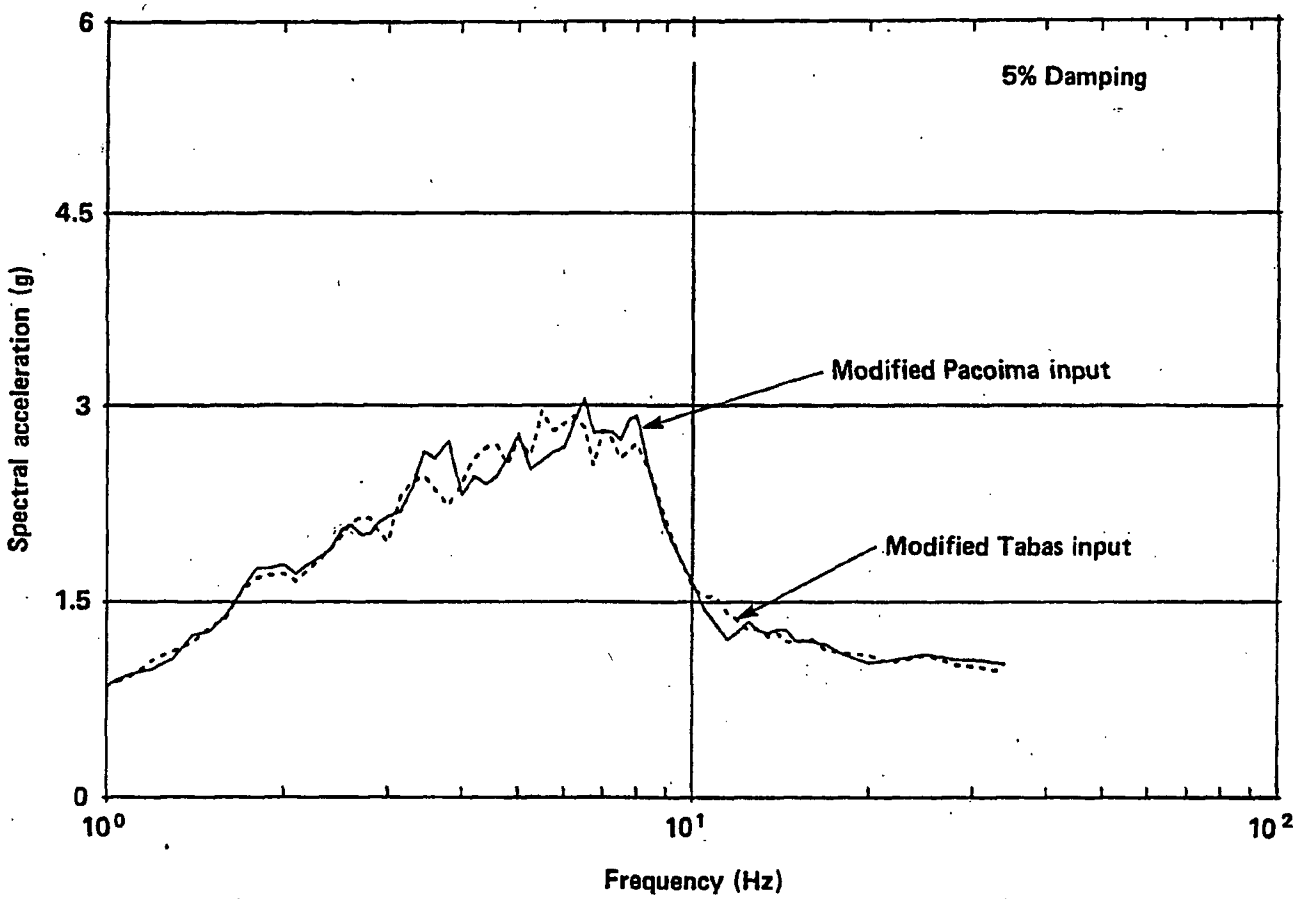


Figure 5-37

Floor response spectra for the north/south response of the auxiliary building at the core west (El 85 feet) obtained from SASSI analyses with coherent ground motion input.

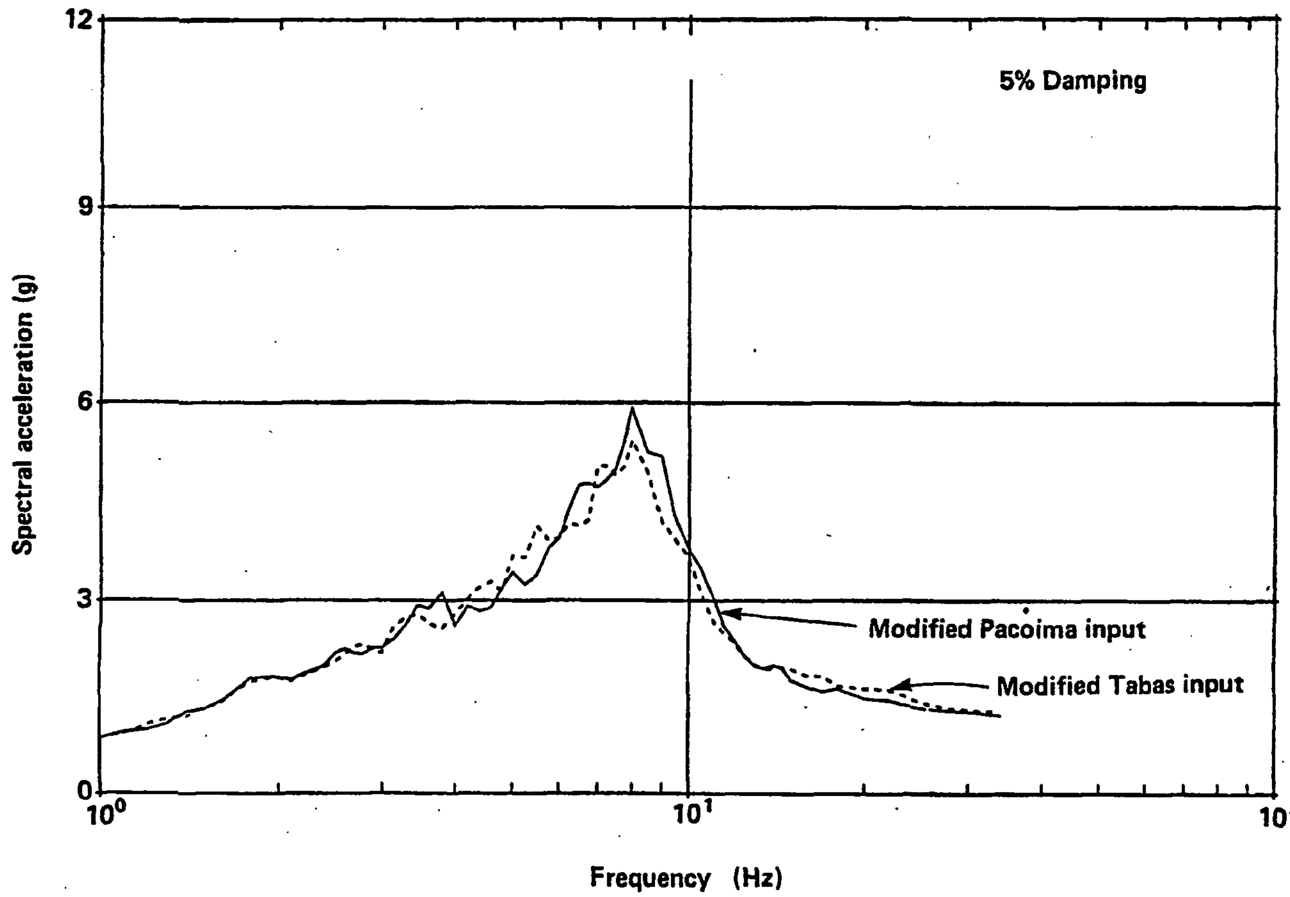


Figure 5-38

Floor response spectra for the north/south response of the auxiliary building at core west (El 140 feet) obtained from SASSI analyses with coherent ground motion input.

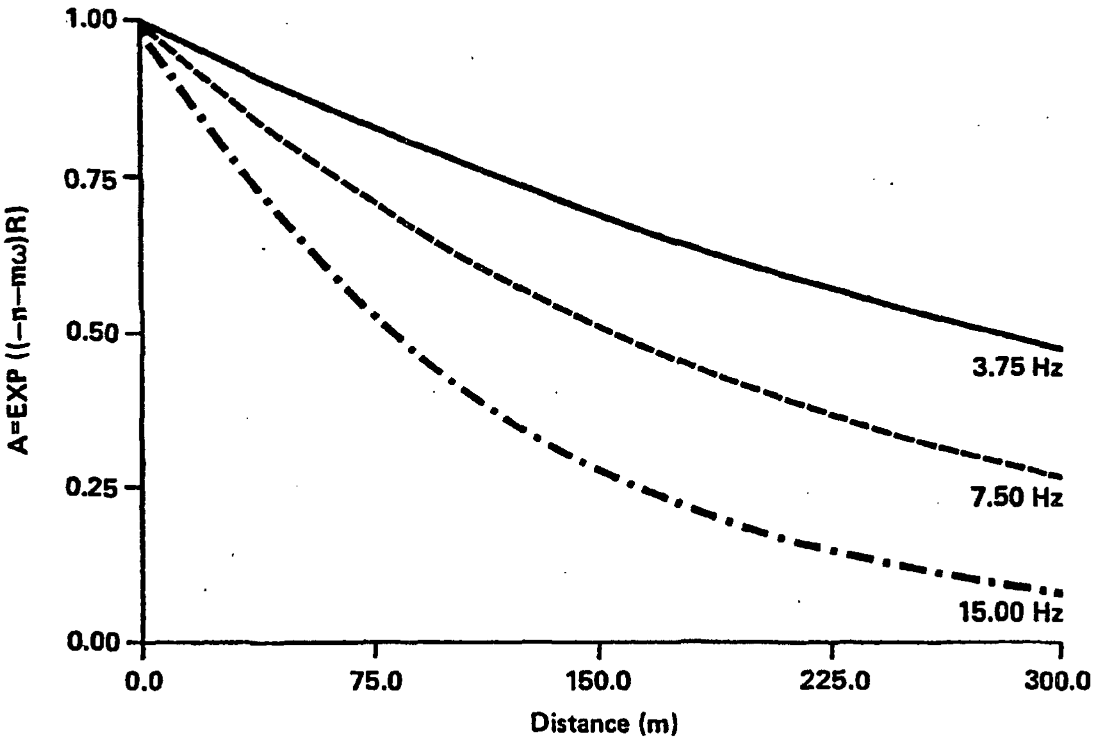


Figure 4—12 Horizontal coherence spectral amplitude functions.

Figure 5-39

Amplitudes of horizontal site-specific spatial incoherence functions.

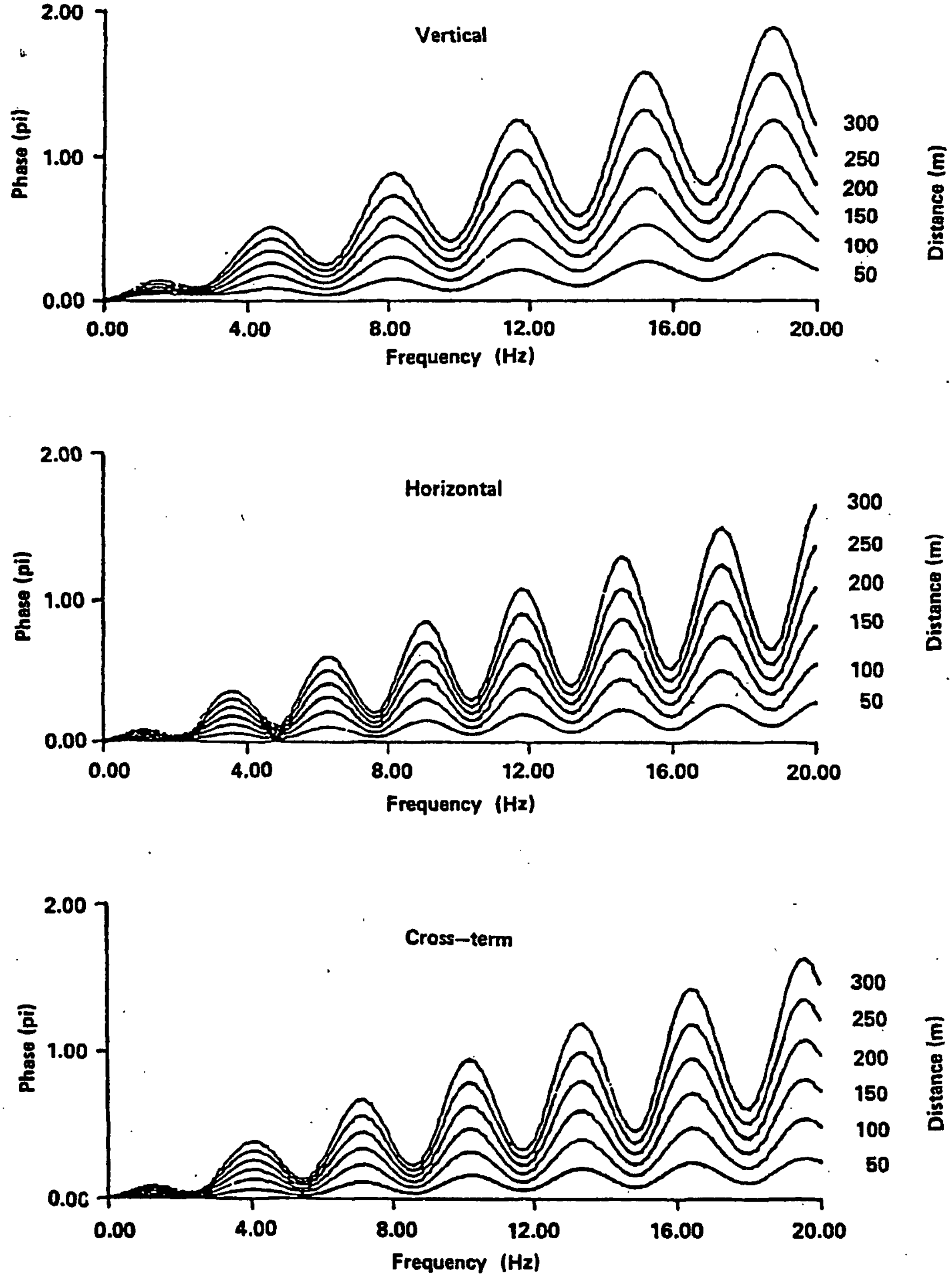


Figure 5-40

Phase angle of site-specific spatial incoherence functions.

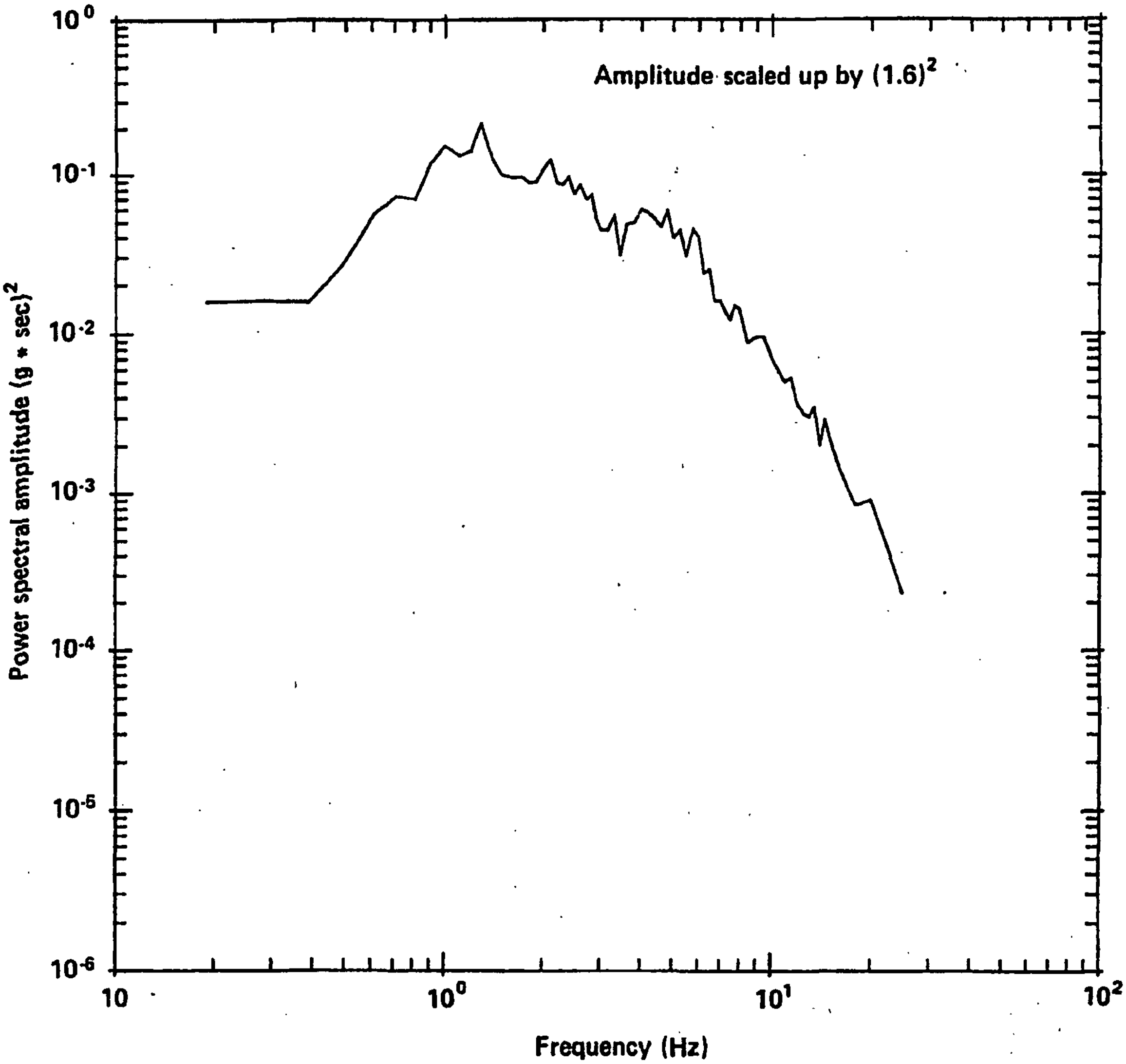


Figure 5-41

Amplitude of one element of the ground motion covariance matrix used for the soil/structure interaction analysis with incoherent ground motion input.

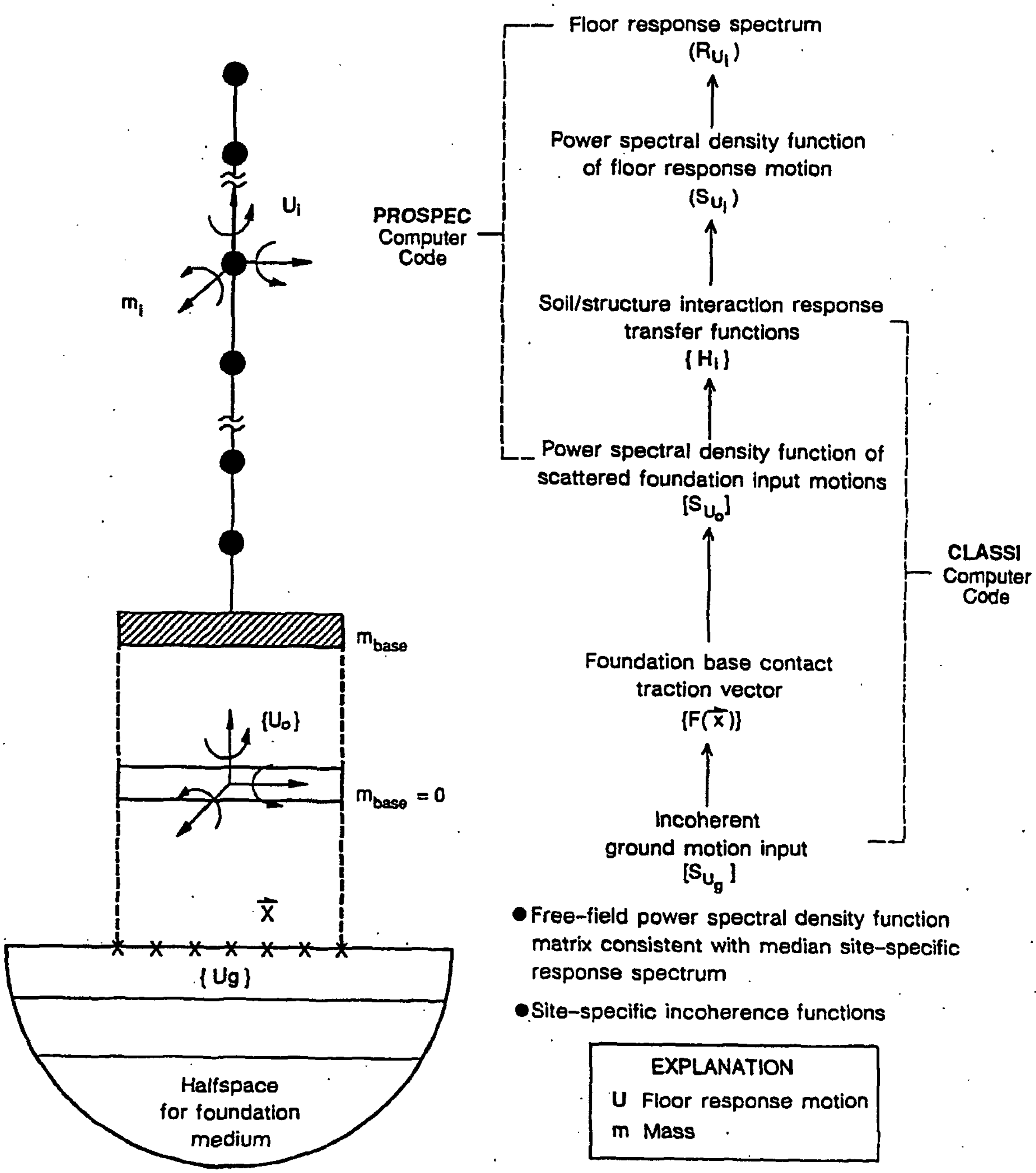


Figure 5-42

Schematic diagram of random vibrational soil/structure interaction analysis for incoherence ground motion input.

input motions. This method and the associated computer programs have been benchmarked against the available published solutions (Luco and Wong, 1986; Mita and Luco, 1986).

Using this method and the CLASSI soil/structure interaction models for the power block structures, analyses were performed with input conforming to the fragility evaluation reference average spectral acceleration of 2 g, as shown in Figure 5-22. Soil/structure interaction responses (including the effects of spatial incoherence) in terms of 5 percent damped floor response spectra were developed for each of the locations in the power block structures where the responses to the coherent ground-motion inputs were generated earlier. To isolate the effect of spatial incoherence using the same analysis method, soil/structure interaction analyses in which the spatial incoherence functions were set equal to unity, were also performed to generate the responses to the coherent ground motions at the same locations. Values of the floor response spectral ratio, which is the ratio of the 5 percent damped floor response spectral value resulting from the incoherent ground-motion input to the corresponding spectral value resulting from the coherent ground-motion input, were determined. The floor response spectral ratios for various response locations, which represent only the effect on the soil/structure interaction response due to the spatial incoherence of ground motions, were then provided for use in the Plant fragility evaluations. Representative results of the 5 percent damped floor response spectra and the corresponding floor response spectral ratios to be used as the response adjustment factors, obtained from both the coherent and incoherent ground motion inputs consistent with the fragility evaluation reference response spectra, are shown in Figures 5-43 and 5-44 for north/south responses of the containment, in Figures 5-45 and 5-46 for north/south responses of the auxiliary building, and in Figures 5-47 through 5-50 for north/south and east/west responses of the turbine building.

The results obtained from soil/structure interaction analyses of the power block structure, incorporating site-specific spatial incoherence ground motion effects, indicate the following:

(1) Spatial incoherence of ground motions generally results in reductions in the foundation base translational motions as indicated by the floor response spectral ratios for the basemat responses shown in Figures 5-43, 5-45, 5-47, and 5-48, and such reductions are proportional to the plan area of the foundation. For the basemats of the power block structures, the magnitudes of these reductions increase gradually with increasing frequency. For frequencies above 10 hertz, these reductions, as indicated by the analytical studies, are about 6 percent for the containment structure, 15 percent for the auxiliary building, and between 0 and 30 percent for the turbine building.

(2) Due to the accompanying rocking and torsional motions induced as a result of spatial incoherence, the reductions in response are less at the locations within the structures where the response is affected by rocking or torsional response motions, and in the specific frequency ranges of the rocking and torsional response modes of the structure. This is illustrated by comparing the floor response spectral ratios for the north/south and east/west responses as shown, respectively, in Figures 5-49 and 5-50 for the switchgear location near the south end of the Unit 2 turbine building. The comparisons indicate that the spectral ratio for the north/south response, which is close to the north/south centerline of the foundation mat and thus has little contribution from the torsional response, is similar to the spectral ratio of the north/south response near the center of the basemat, as shown in Figure 5-47. The spectral ratio for the east/west response, which is away from the east/west centerline and thus is sensitive to torsional response, is different from that of the east/west response of the basemat, as shown in Figure 5-48.

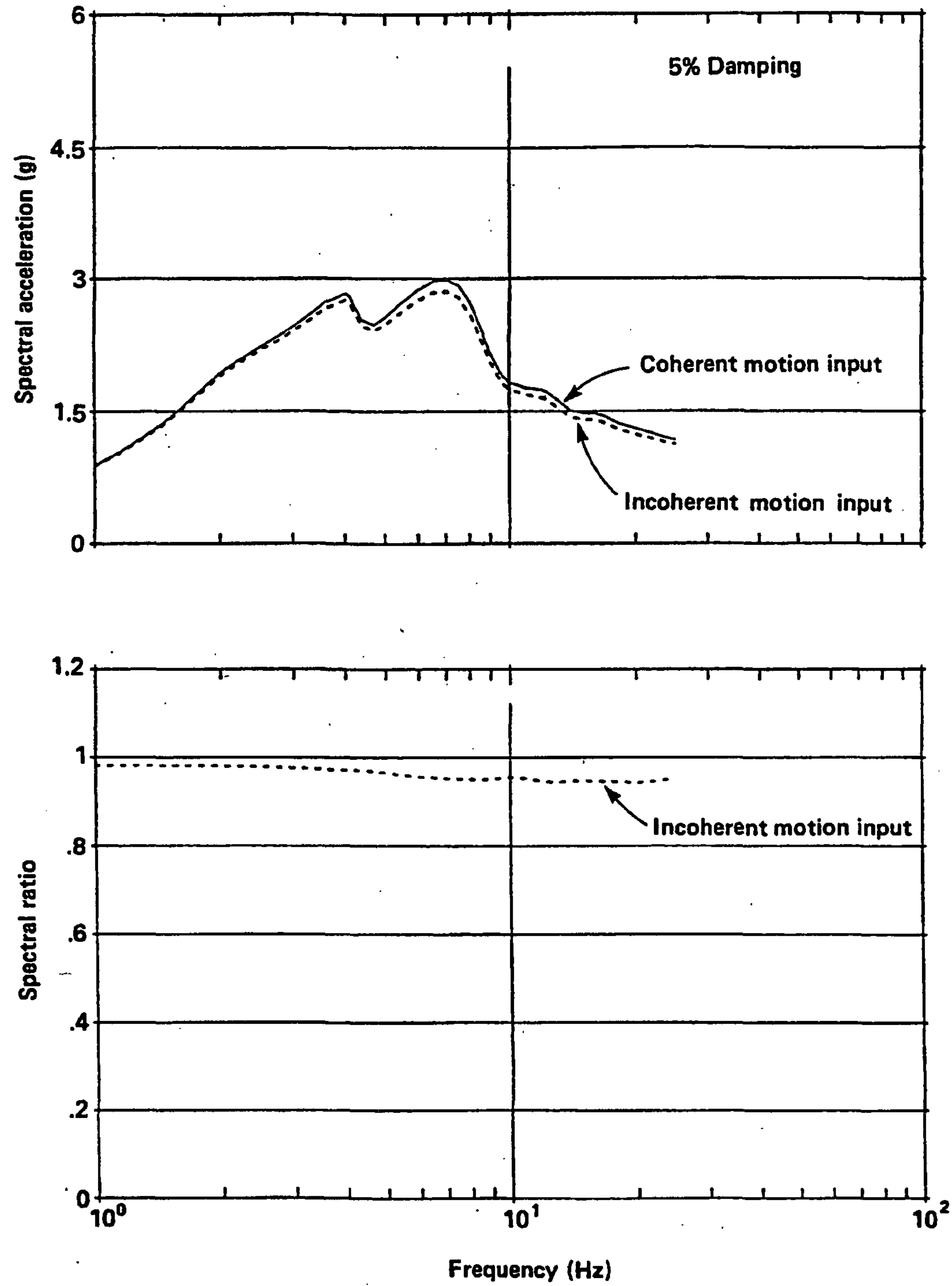


Figure 5-43

Floor response spectra and floor response spectral ratio for the north/south response of the containment at the base, El 85 feet.

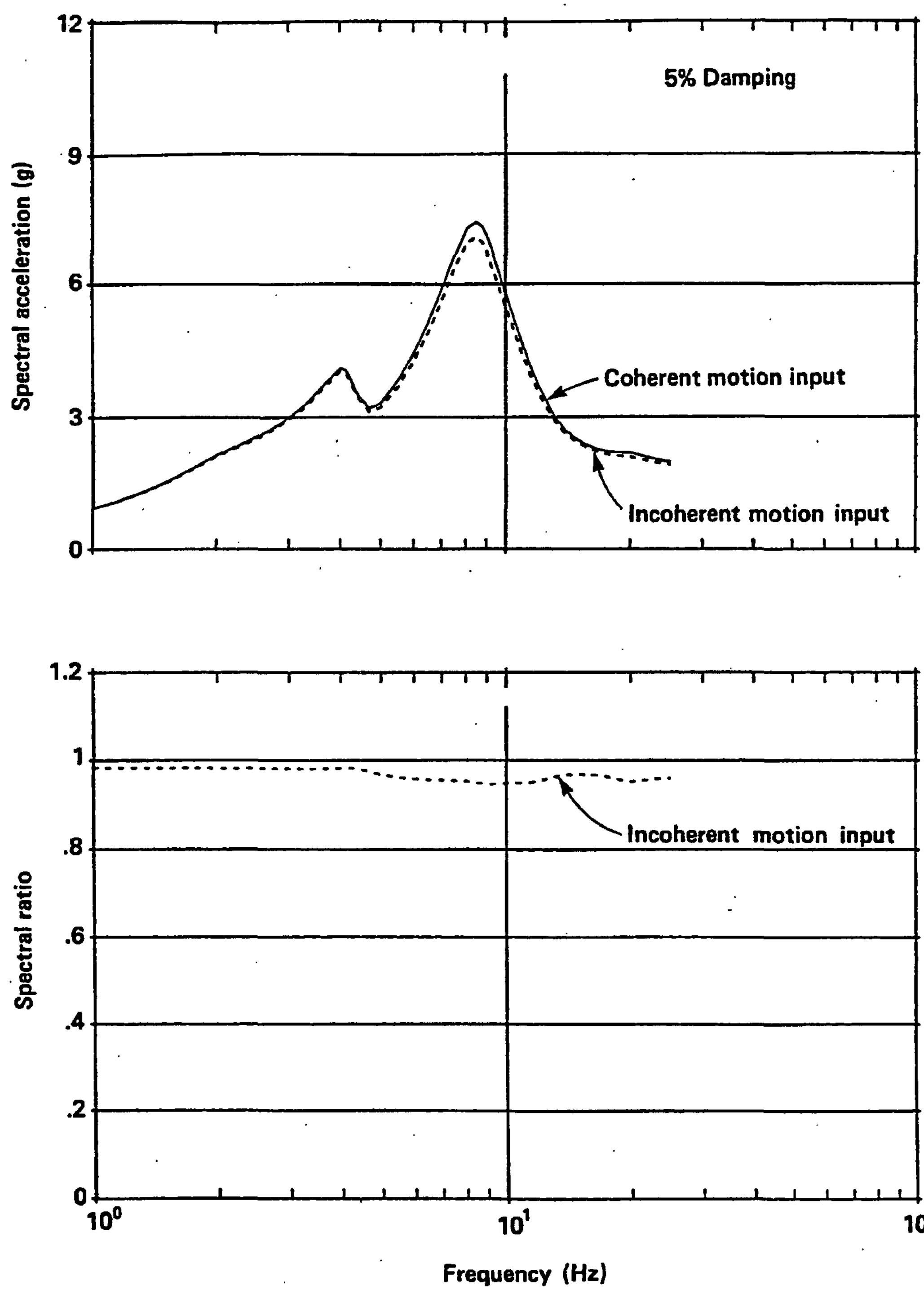


Figure 5-44

Floor response spectra and floor response spectral ratio for the north/south response of the containment at the top of interior concrete structure, El 138.5 feet.

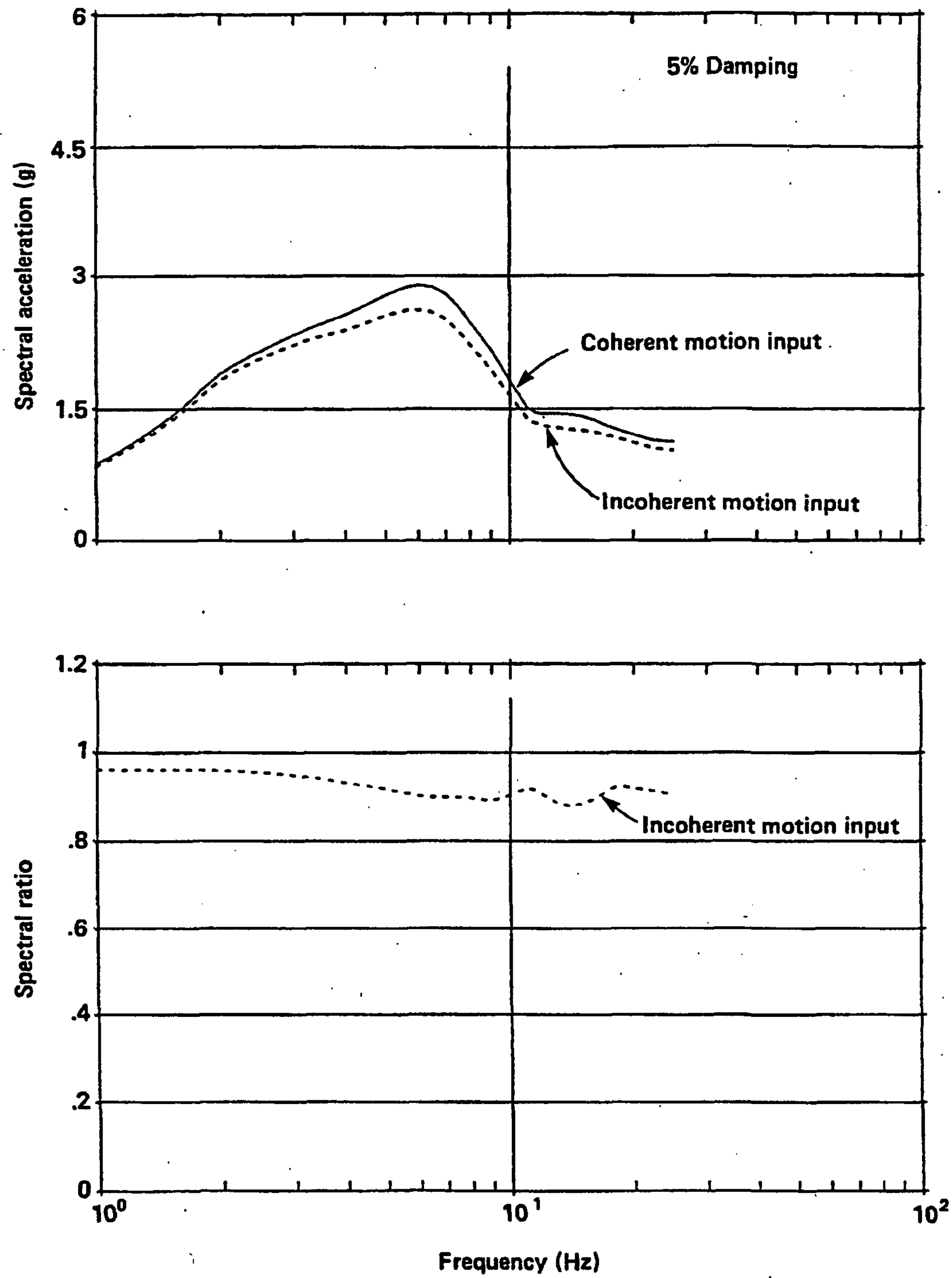


Figure 5-45

Floor response spectra and floor response spectral ratio for the north/south response of the auxiliary building at El 85 feet.

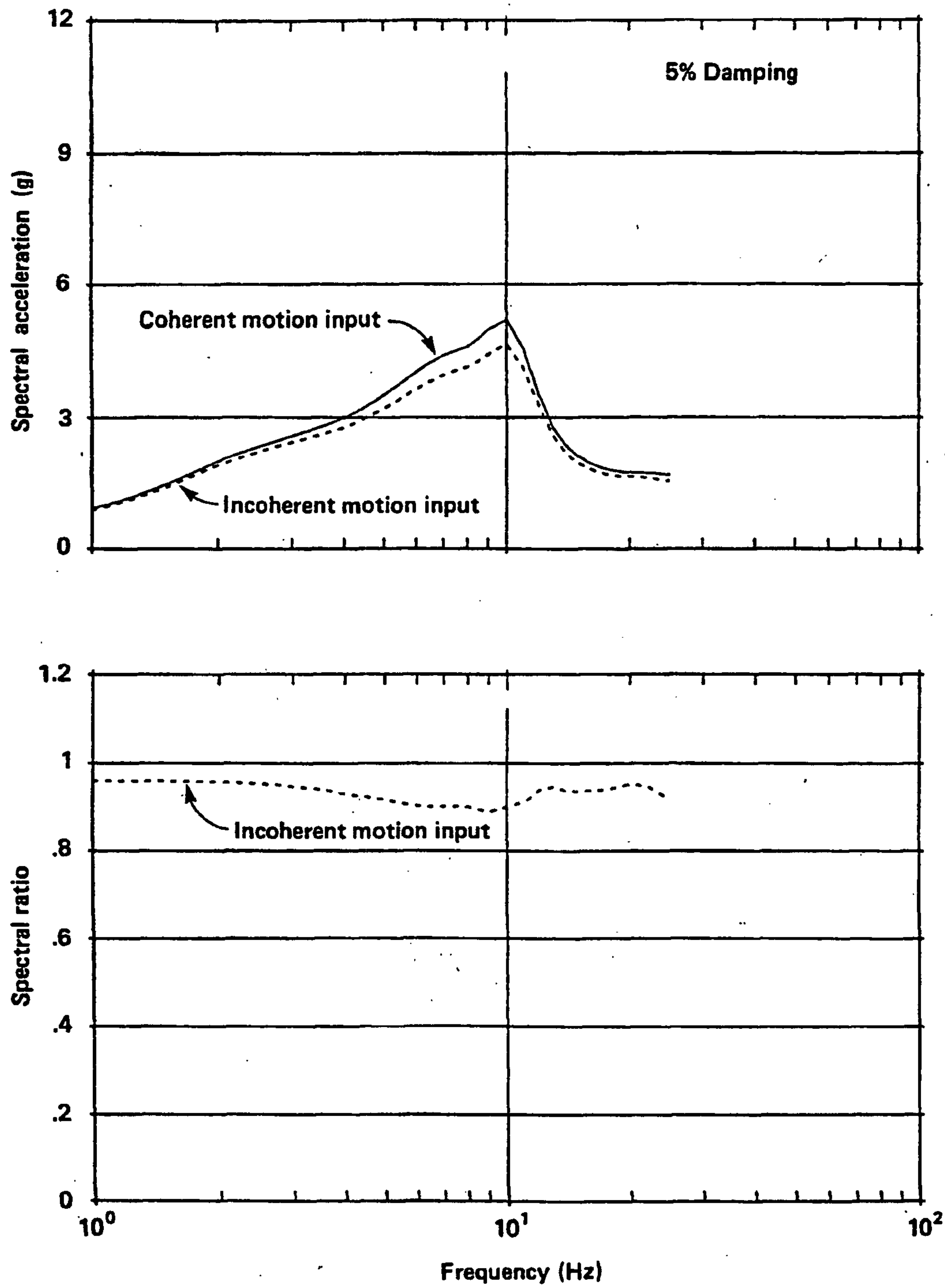


Figure 5-46

Floor response spectra and floor response spectral ratio for the north/south response of the auxiliary building at El 140 feet.

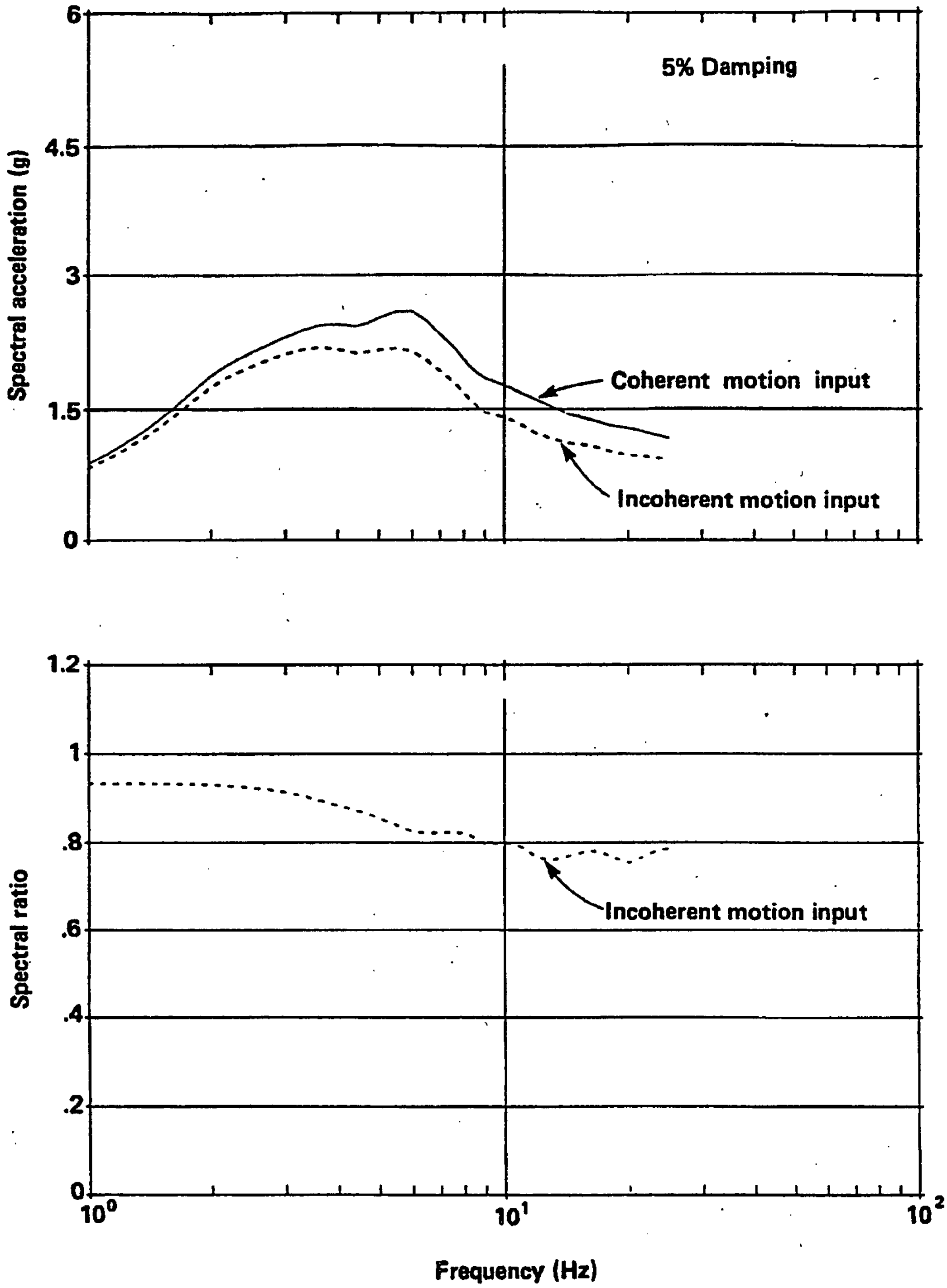


Figure 5-47

Floor response spectra and floor response spectral ratio for the north/south response of the turbine building at CCW heat exchange location, El 85 feet.

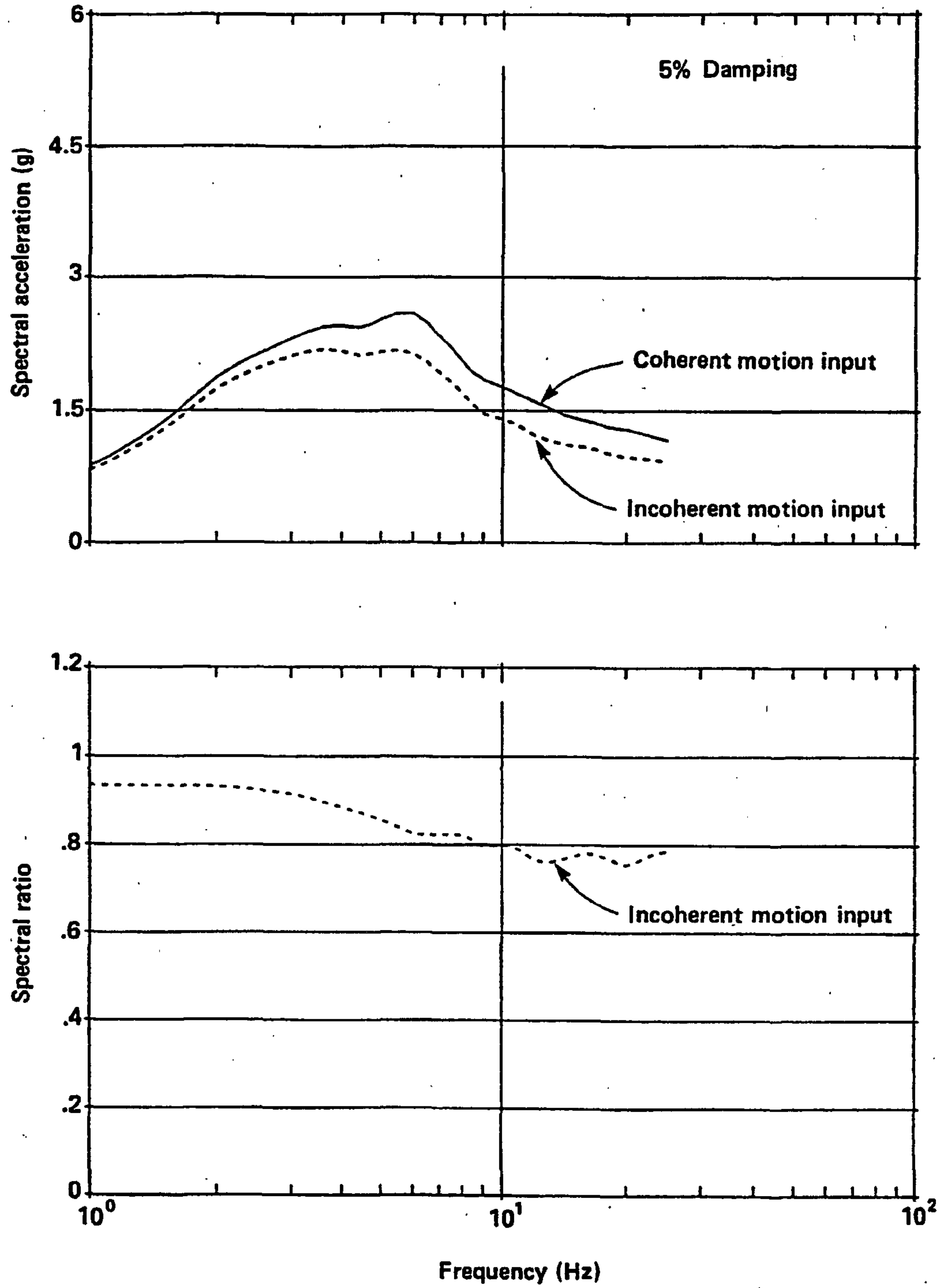


Figure 5-48

Floor response spectra and floor response spectral ratio for the east/west response of the turbine building at CCW heat exchange location, El 85 feet.

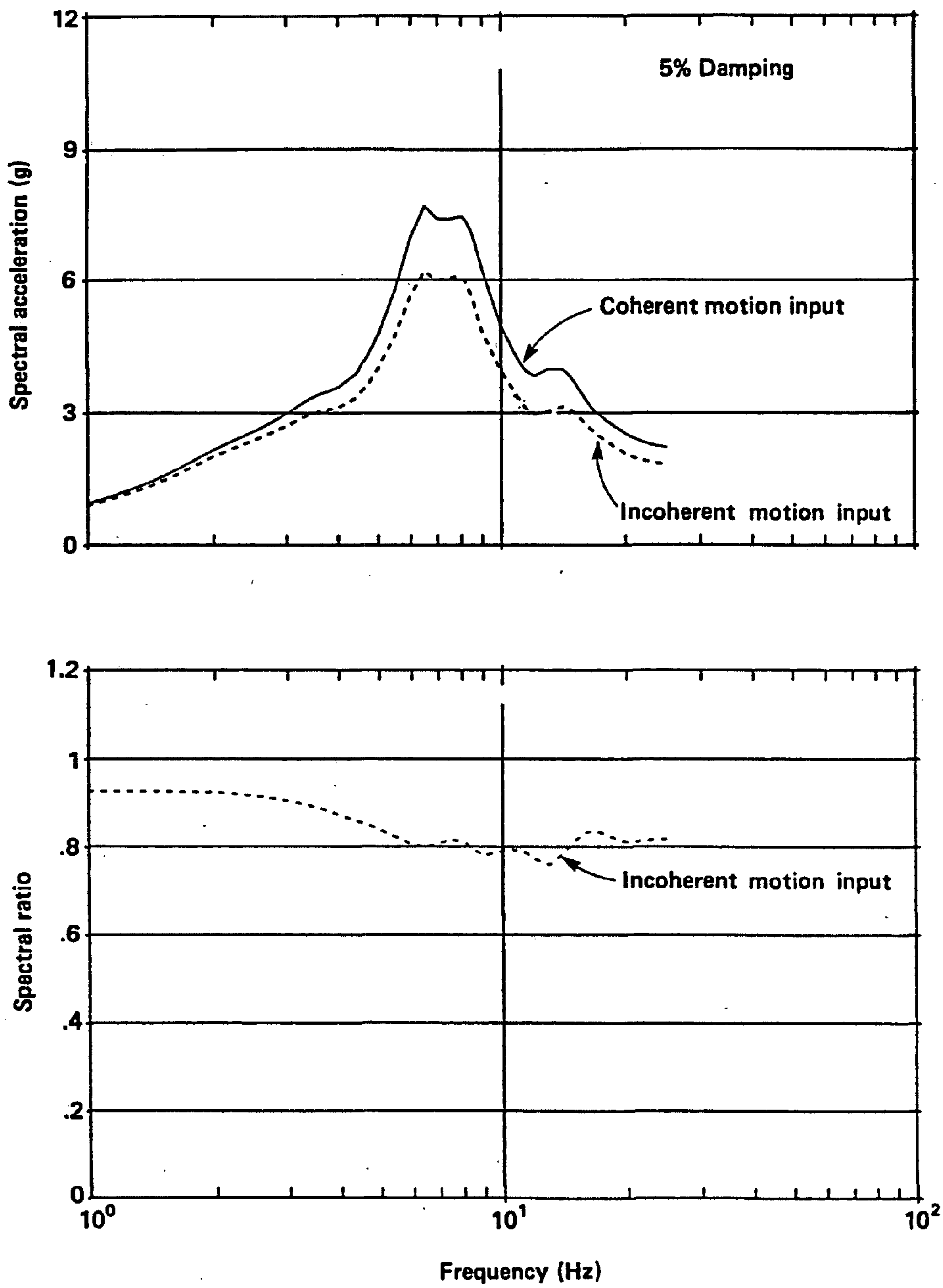


Figure 5-49

Floor response spectra and floor response spectral ratio for the north/south response of the turbine building at switchgear location, El 119 feet.

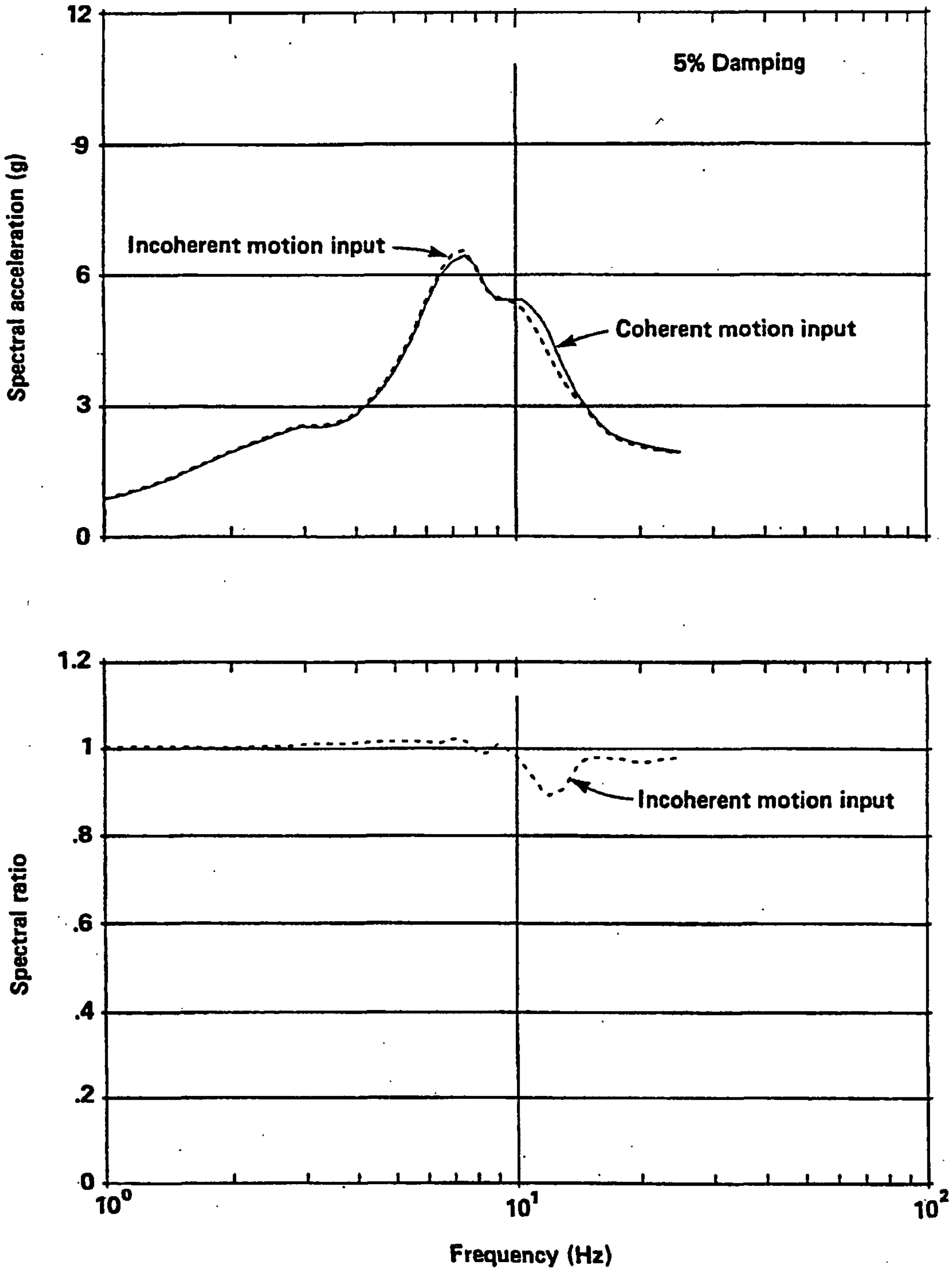


Figure 5-50

Floor response spectra and floor response spectral ratio for the east/west response of the turbine building at switchgear location, El 119 feet.

Assessment of Soil/Structure Interaction Responses of the Containment Structure Due to Basemat Uplifting

The effect on the containment seismic response due to partial uplift of the containment basemat from the rock foundation under strong seismic ground motions was investigated in a separate study using a two-dimensional nonlinear time history analysis method.

The analysis was based on a soil/structure interaction model for the containment formed by coupling the lumped-mass stick model for the structure with a Winkler foundation model (uniformly distributed discrete foundation springs and dampers) which has no tension capability. This model is shown schematically in Figure 5-51. For the Diablo Canyon Power Plant containment which has foundation embedment, the Winkler foundation model was further extended to simulate the foundation embedment effect by incorporating a set of Winkler-type side-rock springs and dampers. Furthermore, a method was also developed to incorporate the energy dissipation associated with the base "slapdown" which occurs following base uplift. The mechanism of energy dissipation was simulated using an equivalent viscous damping for the foundation model which becomes effective when base uplift occurs.

The nonlinear base uplifting analysis methodology and the associated UPLIFT computer program (Tseng and Wing, 1984) used for the analysis of containment have been benchmarked against available published solutions for the effects of base uplifting in dynamic response problems (Psycharis, 1981).

The free-field input motions used for the containment base uplift response analyses were the rock surface motions assumed in the form of coherent, vertically incident, plane waves. Since a two-dimensional analysis was used, one horizontal component together with the vertical component of the three-component prescribed earthquake motions were simultaneously applied as the input for each analysis. Both horizontal components were used in this manner in two separate analyses.

The ground motions that were used as the input for the containment base uplift response analysis consisted of three sets of three-component recorded motions as selected by the ground-motion studies, which are: (a) the Pacoima Dam records of the 1971 San Fernando earthquake; (b) the 1978 Tabas records of the Tabas earthquake; and (c) the El Centro No. 4 records of the 1979 Imperial Valley earthquake. Before applying these as-recorded motions for the analysis, the motions were adjusted in the following manner:

(1) The original recorded motions were adjusted to conform with the site-specific conditions such as the maximum earthquake magnitude, source-to-site distance, and site condition.

(2) The two horizontal components of the adjusted three-component motions were transformed, as necessary, into two longitudinal and transverse horizontal components to provide motions in the directions normal and parallel to the strike of the causative fault.

(3) The three-component time histories were scaled by a constant scaling factor common for all three components, to correspond to the reference seismic input level used for fragility evaluation purposes.

(4) The scaled three-component time histories were then applied as the input for the base uplift response analyses, first, with the longitudinal component applied in the Plant north/south direction and the transverse component in the Plant east/west direction, and then vice versa; the vertical component was applied in the Plant vertical direction in each case.

To be conservative for the containment base uplift response analyses, the constant scaling factor used for step (3) above was derived such that the average value of the 5 percent damped acceleration response spectral values of the two horizontal time histories in the frequency range of 3 to 8.5 hertz, inclusively, was equal to 2.25 g. This procedure is illustrated in Figure 5-52. The scaling factors as derived for the three sets of

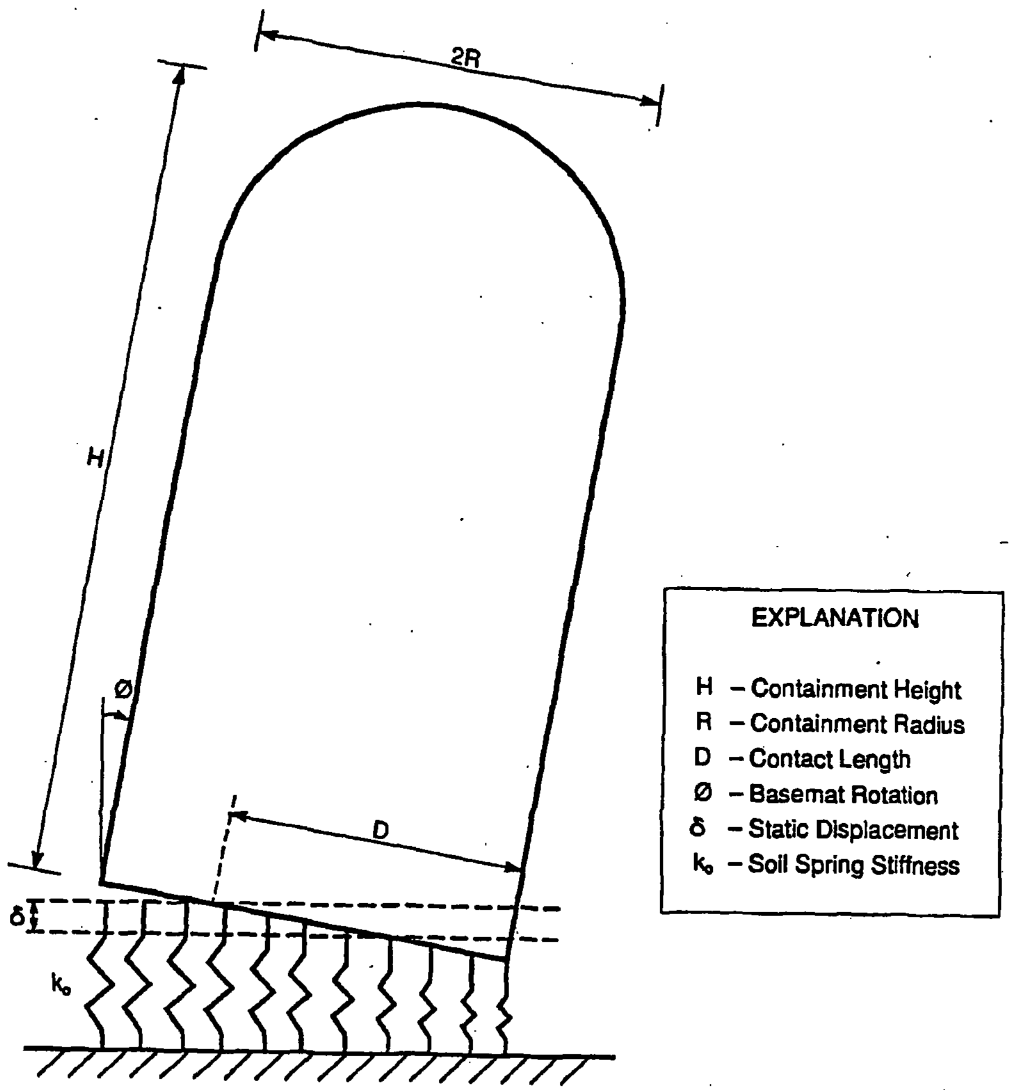


Figure 5-51

Schematic configuration of containment on Winkler foundation with base uplift.

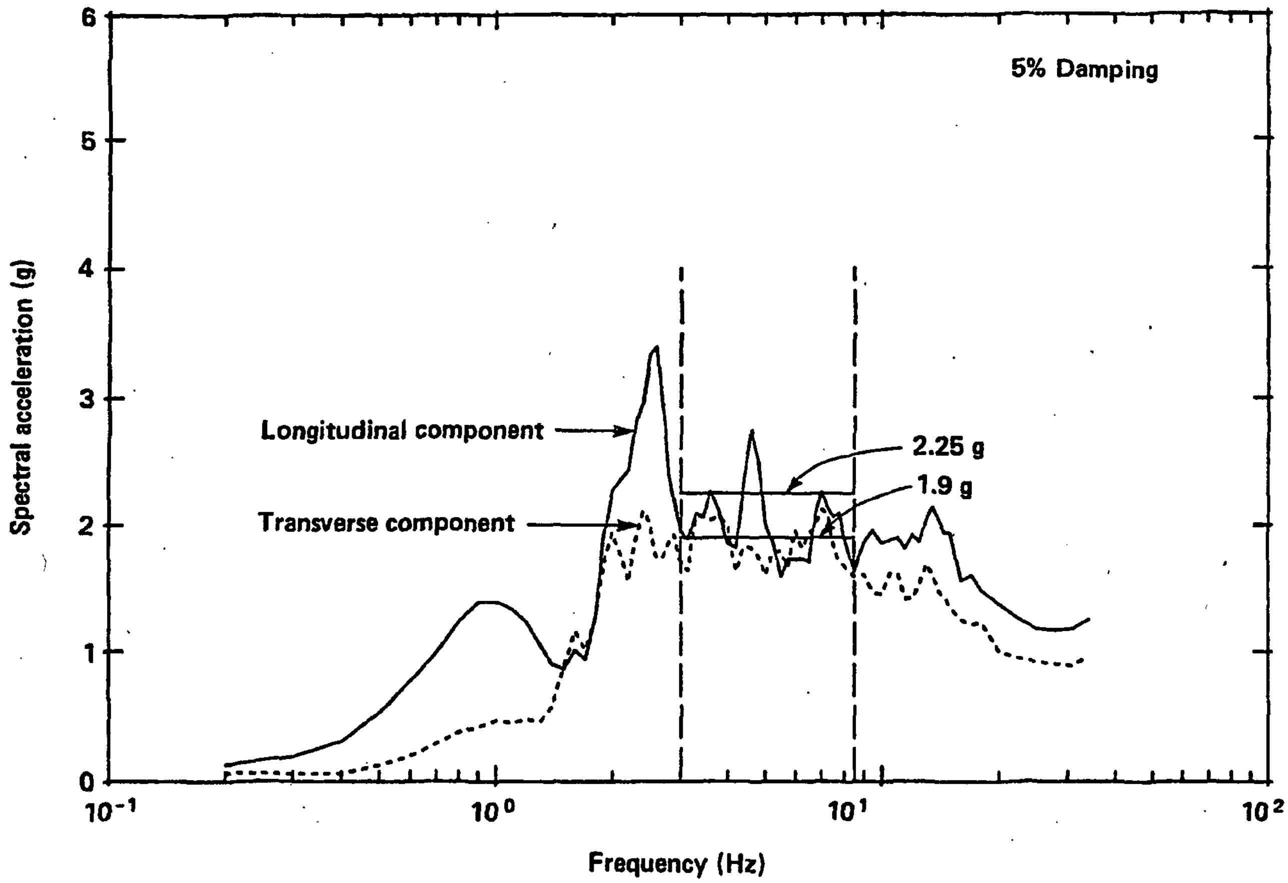


Figure 5-52

Illustration of the procedure used to derive the constant scaling factor for the input motions using the Pacoima motions for containment base uplift analyses.

ground motion inputs considered were: 1.2 for the Pacoima input; 0.9 for the Tabas input; and 2.5 for the El Centro No. 4 input. The scaled final time histories used for analyses of base uplift effects are shown in Figures 5-53 through 5-55. The 5 percent damped acceleration response spectra of these time histories are shown in Figures 5-56 through 5-58.

Containment base uplift analyses were performed for three foundation model assumptions: (a) a Winkler base foundation model with the full amount of side-rock impedances to simulate the condition of full contact between the side rock and the embedded containment basemat wall; (b) a Winkler base foundation model with one-half the side-rock impedances to simulate the partial loss of side-rock support up to one-half the basemat wall perimeter; and (c) a Winkler base foundation model with one-half the side-rock impedances and with added viscous damping to simulate the base slapdown impact energy dissipation. For comparison purposes, linear response analyses, in which base uplift was suppressed, were also performed for all base uplift analysis cases.

Representative horizontal and vertical response results obtained from the analyses for all three foundation model cases and all three sets of three-component time histories used as input motions, are presented in Figures 5-59 through 5-64 in terms of the 5 percent damped floor response spectra at the containment shell springline location and at the top floor of the containment interior structure.

The results of the containment base uplift analyses, as presented in these figures, show that: (a) allowance for base uplift generally leads to small reductions in the horizontal acceleration response, shear, and overturning moment; and small increases in the horizontal displacement and the vertical acceleration response in the high frequency range, as compared to the response obtained without including base uplift effects; (b) a reduction in the side-rock impedances to one-half the full values, to account for the partial separation of the embedded wall from the surrounding rock over one-half the basemat wall perimeter, produced relatively small variations in the response; and (c) consideration of the base slapdown impact energy dissipation, as proposed by Psycharis (1981), resulted in further reductions in both horizontal and vertical response; however, the effect was found to be relatively small. In view of these results, it was concluded that base uplift had no significant effects on the dynamic response of the containment structure.

SUMMARY AND CONCLUSIONS

A complete reevaluation of the seismic soil/structure interaction effect on the power block structures was carried out as part of the Long Term Seismic Program. The conclusisons from these studies are described below.

CLASSI/SASSI Solution Techniques. The reevaluation used state-of-the-art three-dimensional analysis techniques and computer programs, CLASSI and SASSI. An extensive effort was spent implementating, upgrading, validating, and documenting these two programs for our Program's applications. Plant-specific applications of these two programs have demonstrated that they produce essentially the same solutions for the same soil/structure interaction problems.

Soil/Structure Interaction Parametric Studies. Prior to performing the soil/structure interaction analysis, extensive studies were made to characterize the soil/structure interaction systems for the power block structures. These studies included the assemblage, review, and characterization of the foundation rock profile and properties, the development of appropriate three-dimensional dynamic models for the power block structures, and the performance of a series of soil/structure interaction parametric studies. In these studies, the additional site investigation data that became available in 1978, and the on-site earthquake recordings that became available after 1980 have been used to assist in calibrating the dynamic characteristics of the site rock and the soil/structure interaction systems for the power block structures.

The results of the soil/structure interaction parametric studies indicated that the effects of

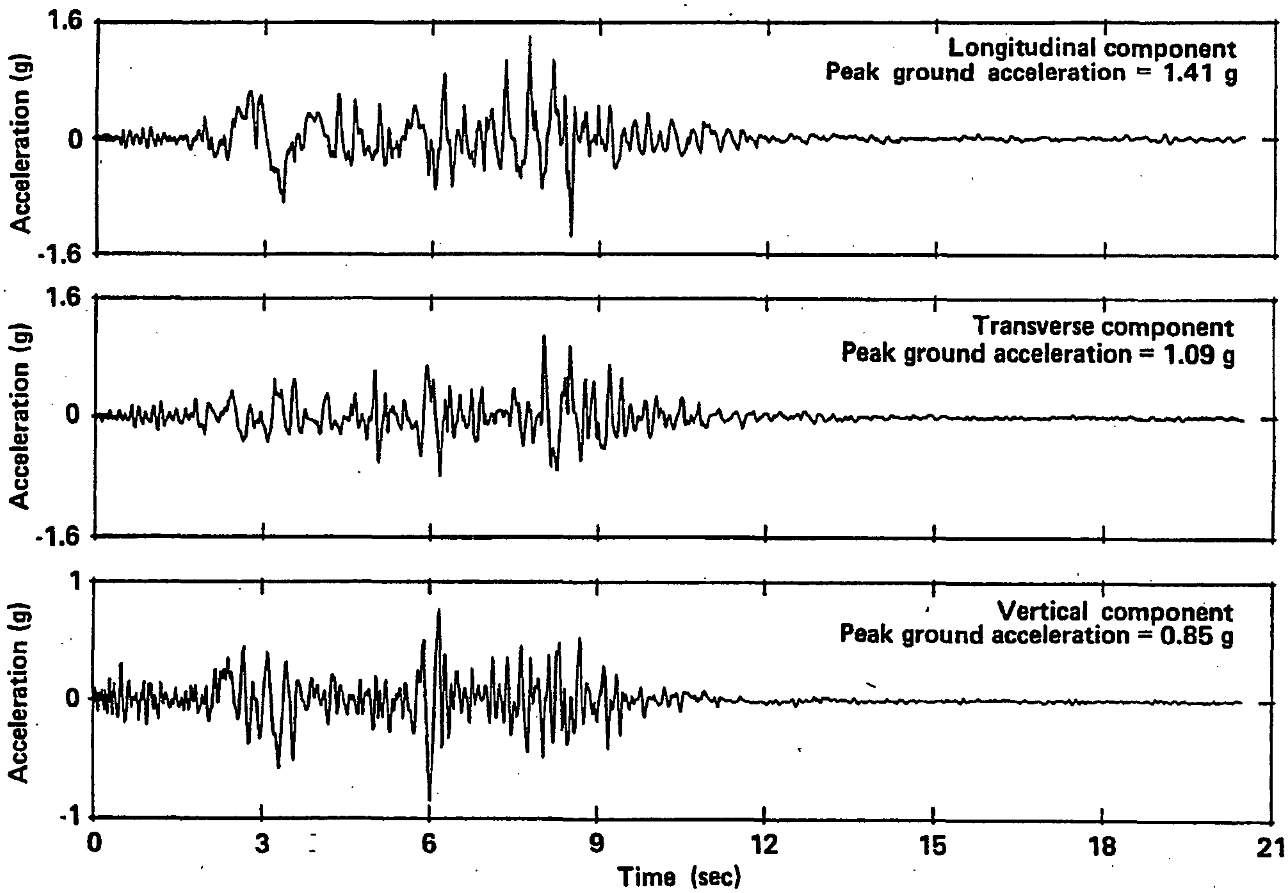


Figure 5-53

Scaled Pacoima acceleration time histories used for containment base uplift analyses, longitudinal, transverse, and vertical components.

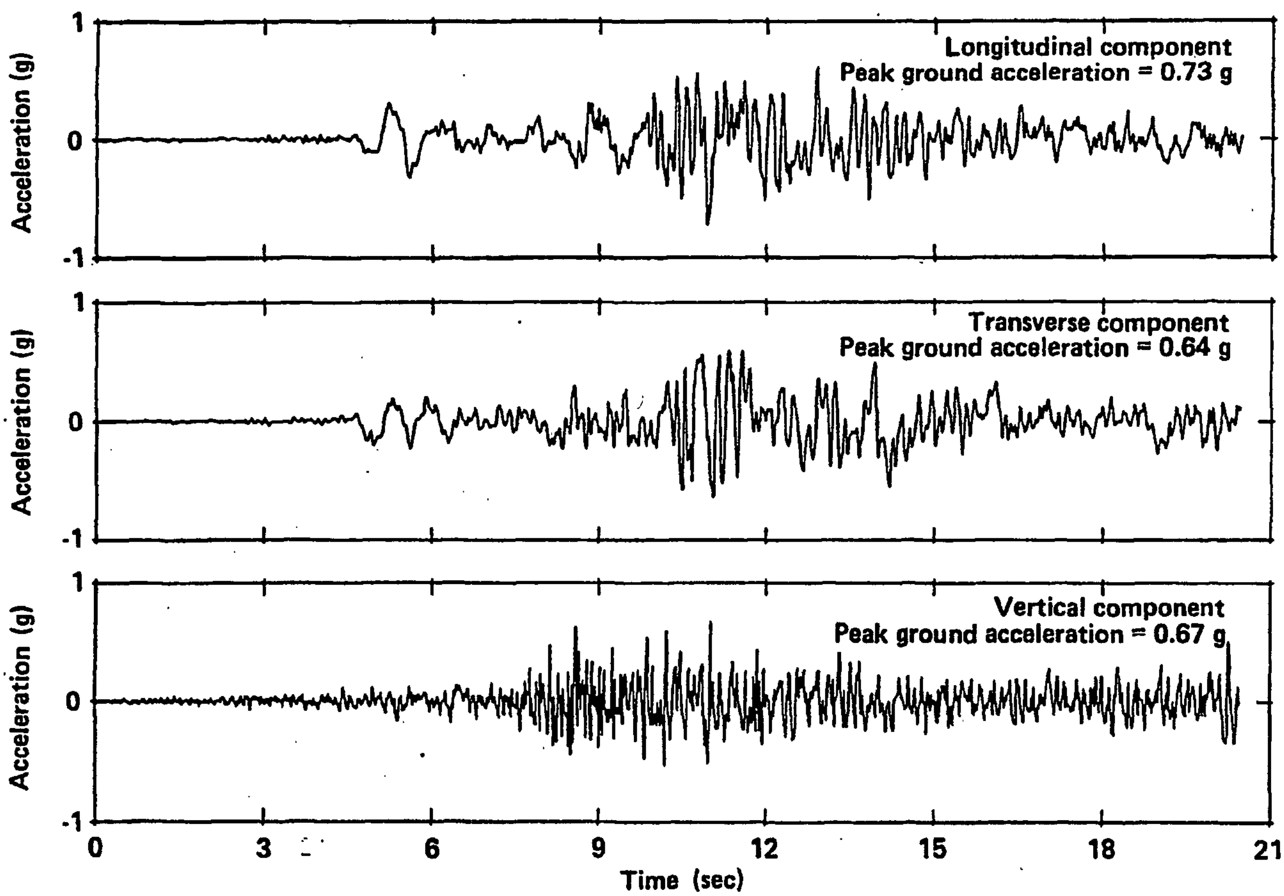


Figure 5-54

Scaled Tabas acceleration time histories used for containment base uplift analyses, longitudinal, transverse, and vertical components.

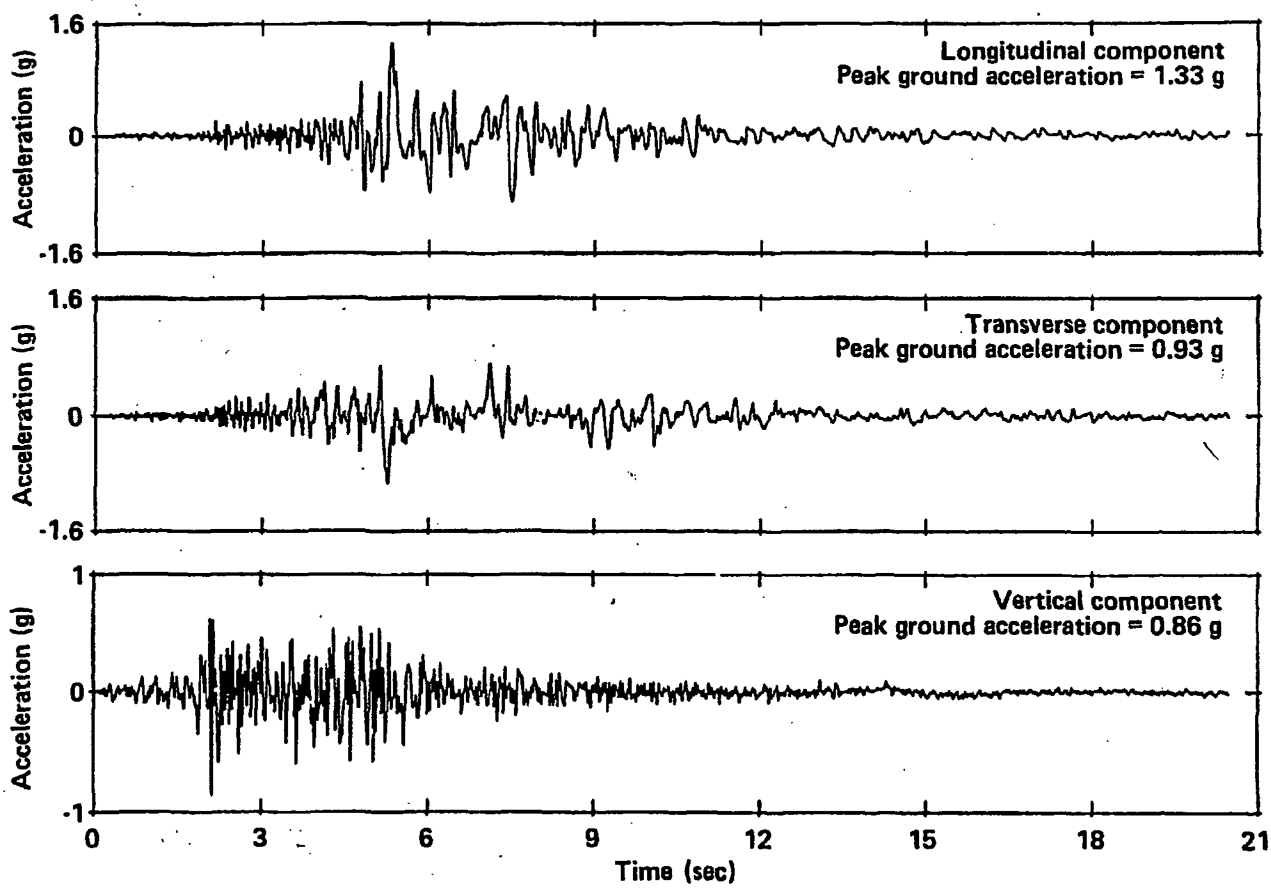


Figure 5-55

Scaled El Centro No. 4 acceleration time histories used for containment base uplift analyses, longitudinal, transverse, and vertical components.

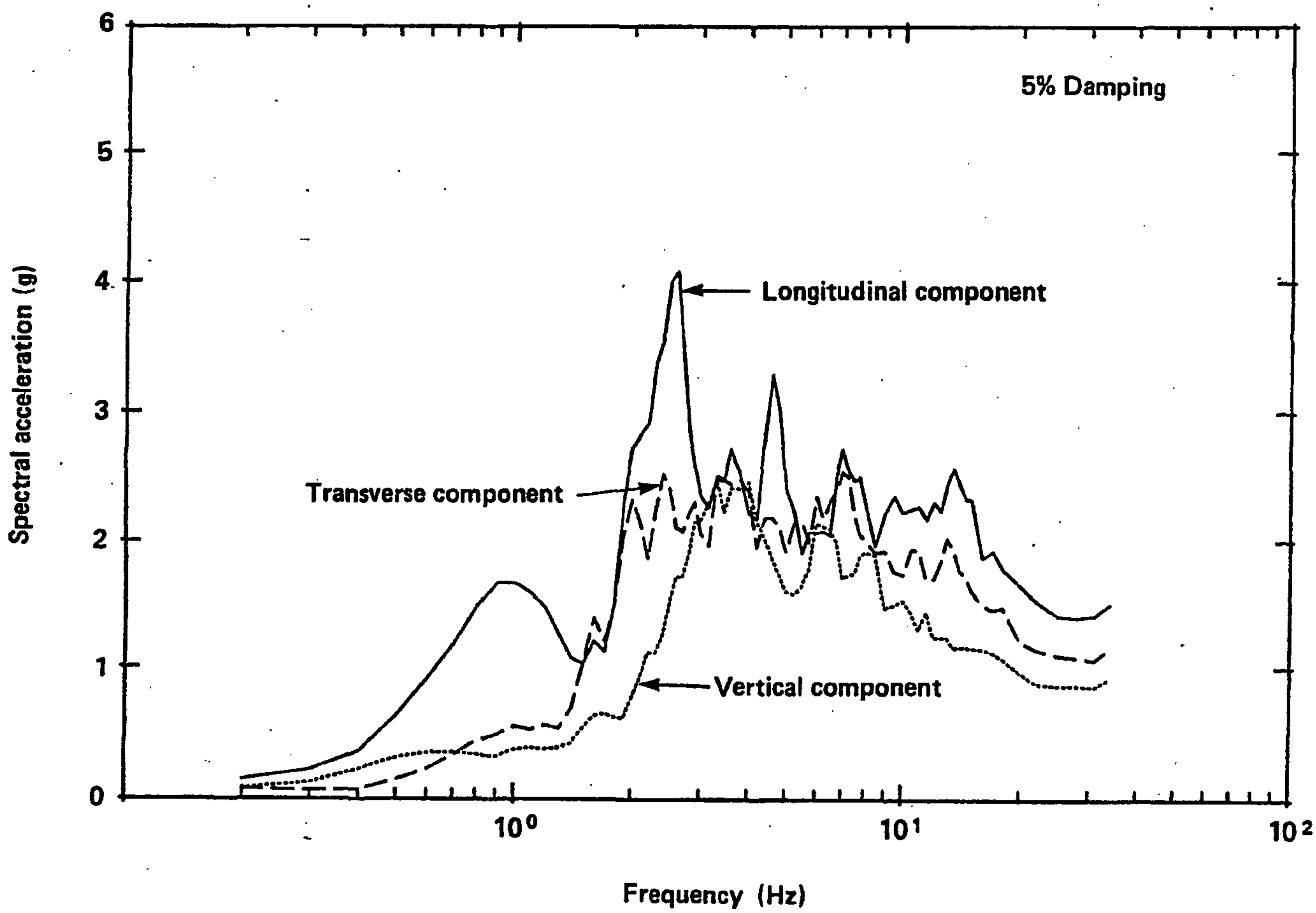


Figure 5-56

Acceleration response spectra of scaled Pacoima time histories used for containment base uplift analyses.

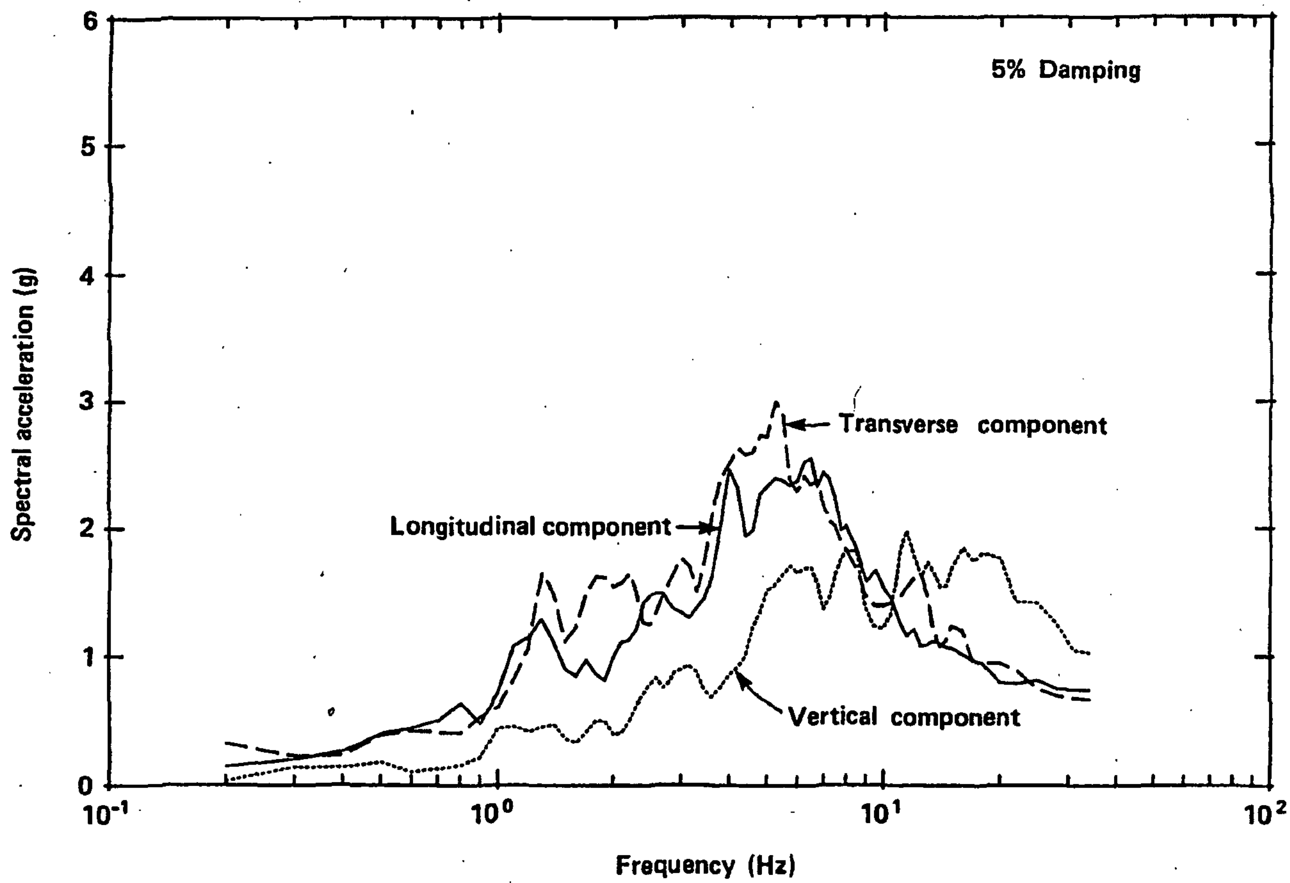


Figure 5-57

Acceleration response spectra of scaled Tabas time histories used for containment base uplift analyses.

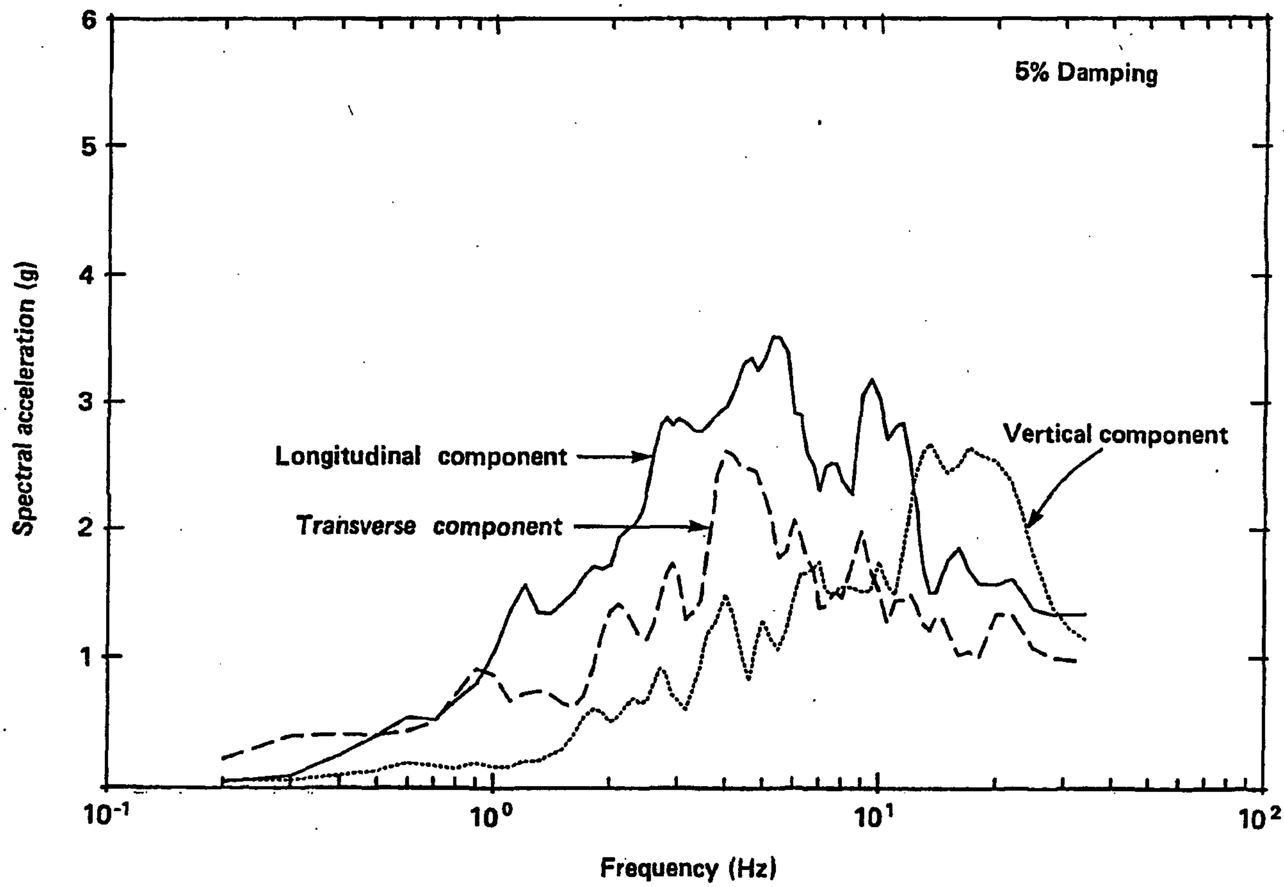


Figure 5-58

Acceleration response spectra of scaled El Centro No. 4 time histories used for containment base uplift analyses.

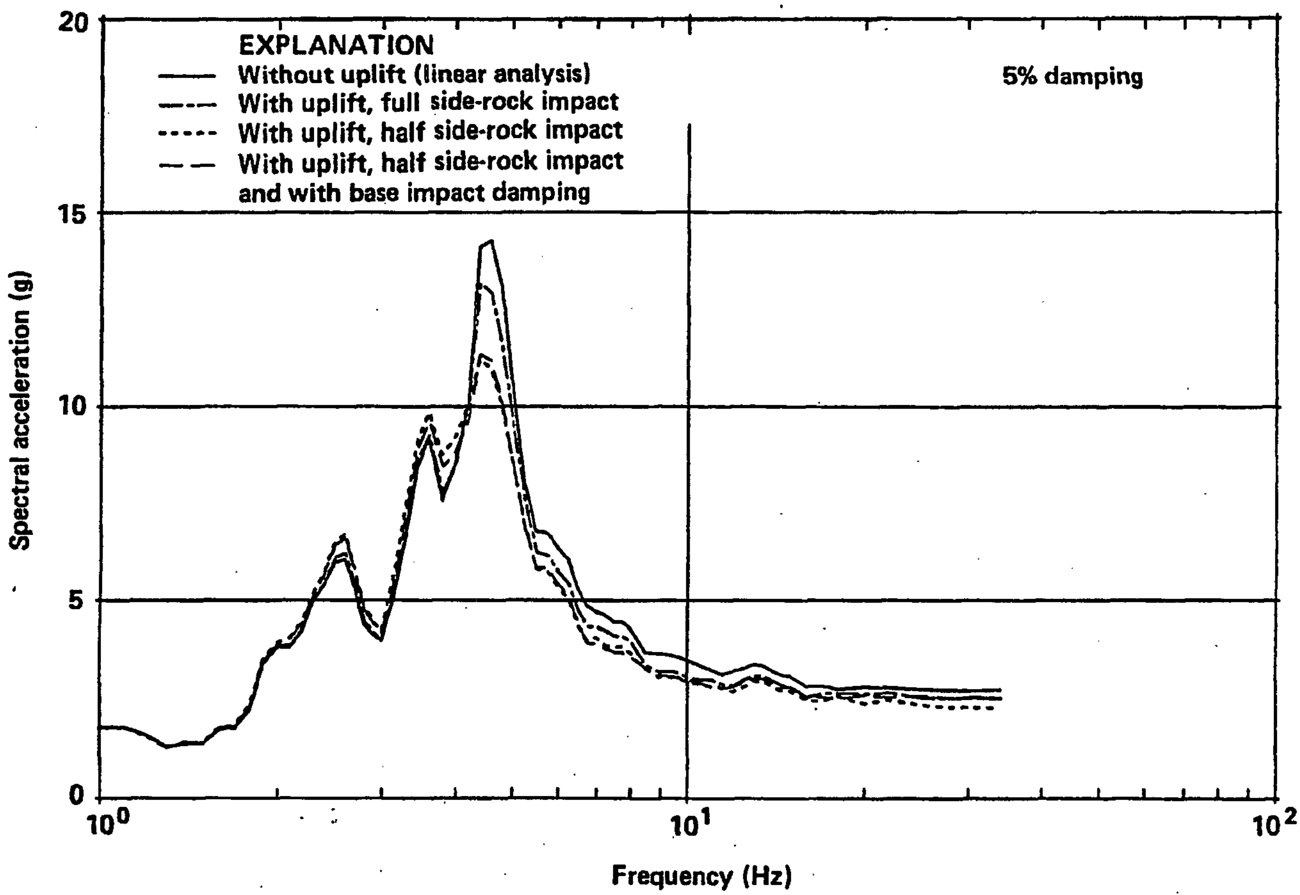


Figure 5-59

Floor response spectra for the north/south response of containment shell at El 231 feet due to scaled Pacoima input.

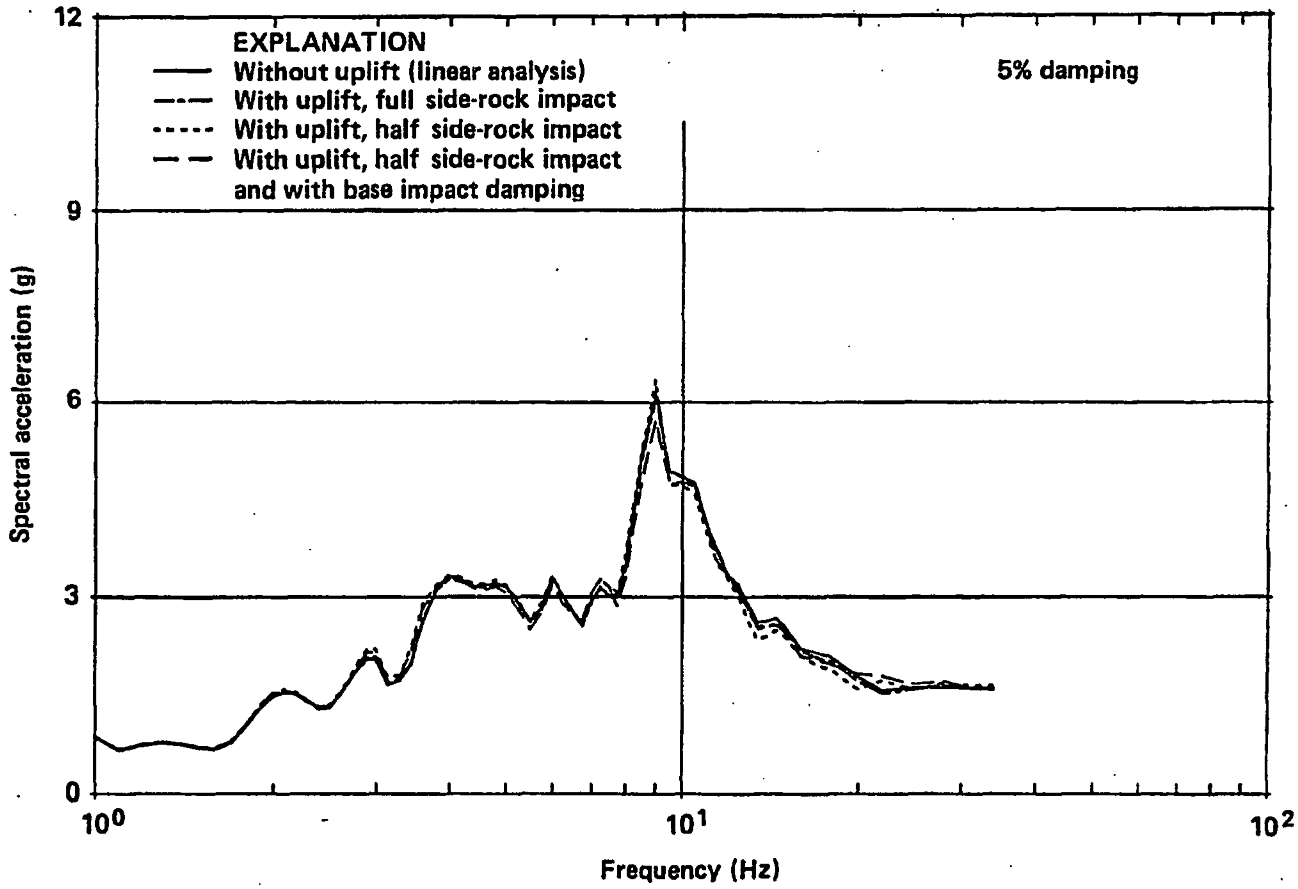


Figure 5-60

Floor response spectra for the east/west response of containment interior structure at El 138.5 feet due to scaled El Centro 4 input.

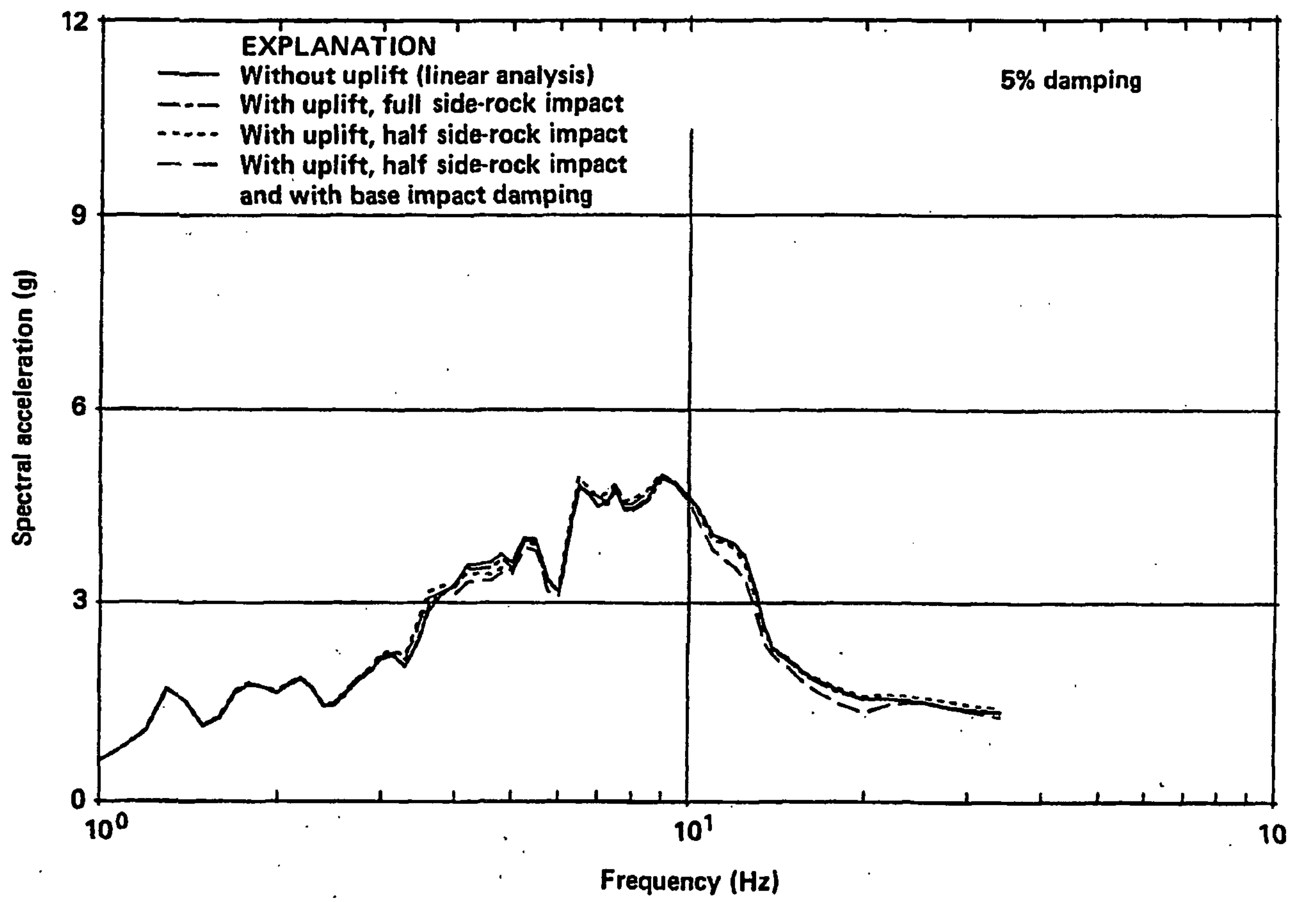


Figure 5-61

Floor response spectra for the east/west response of containment interior structure at El 138.5 feet due to scaled Tabas input.

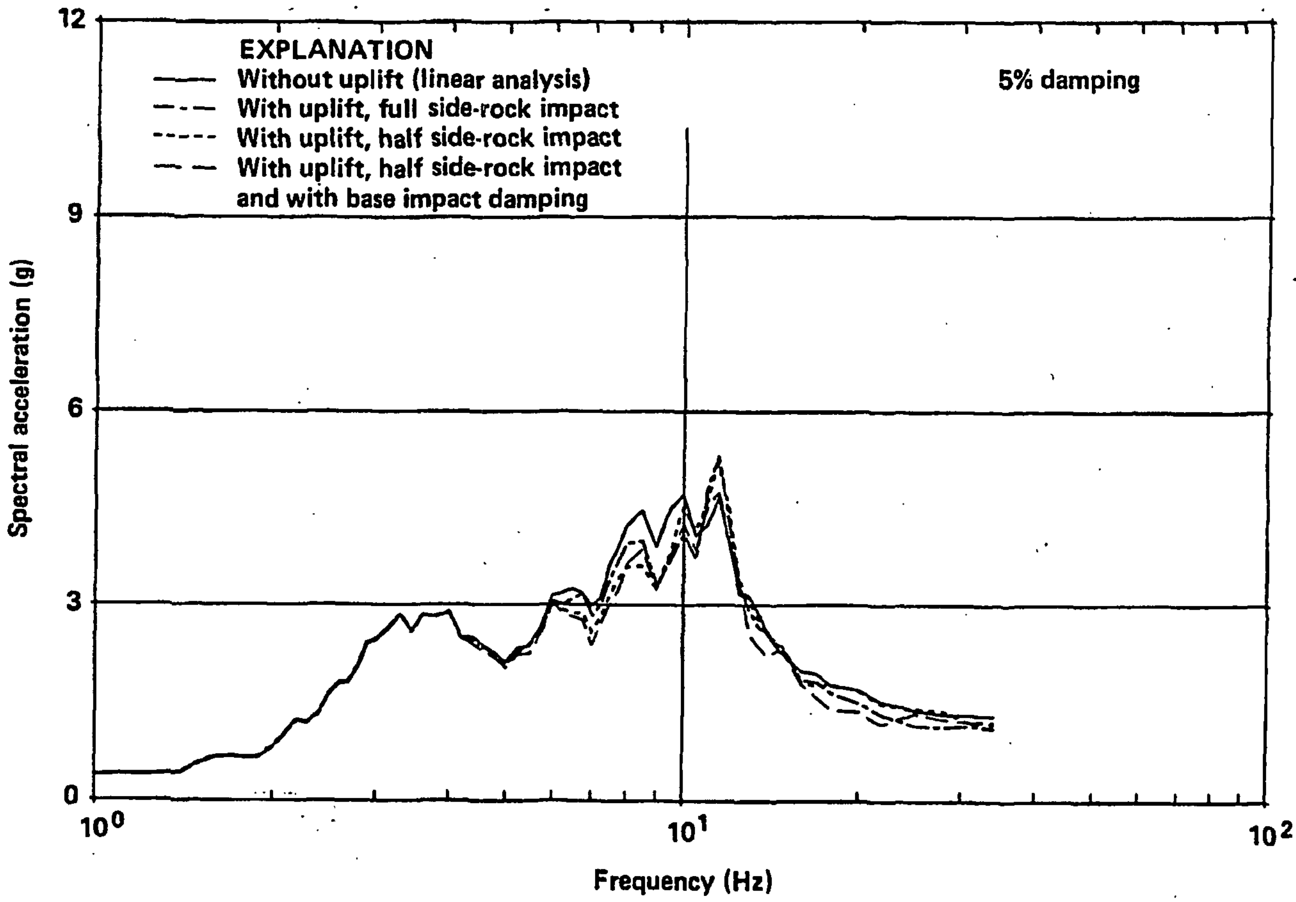


Figure 5-62

Floor response spectra for the vertical response of containment shell at El 231 feet due to scaled Pacoima input.

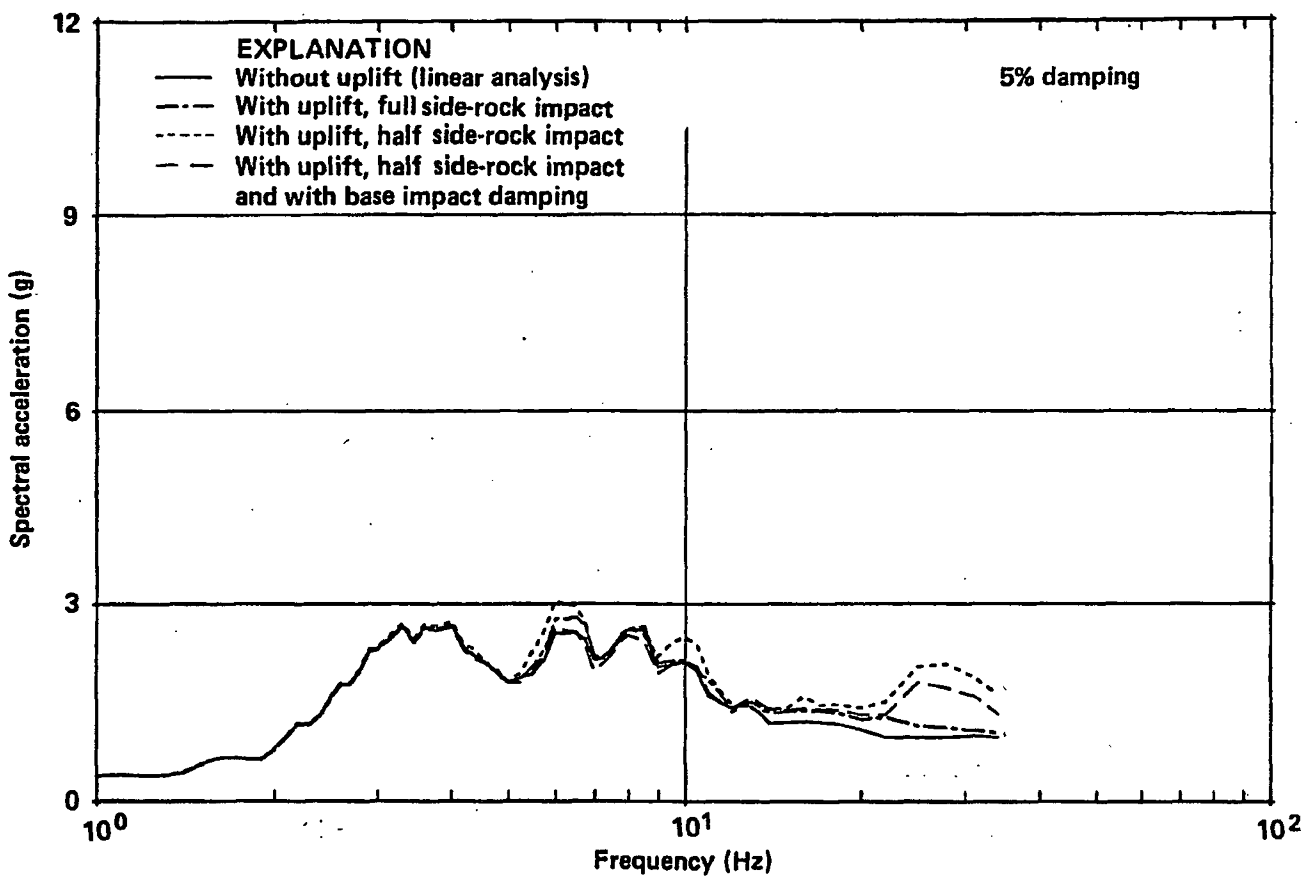


Figure 5-63

Floor response spectra for the vertical response of containment interior structure shell at El 138.5 feet due to scaled Pacoima input.

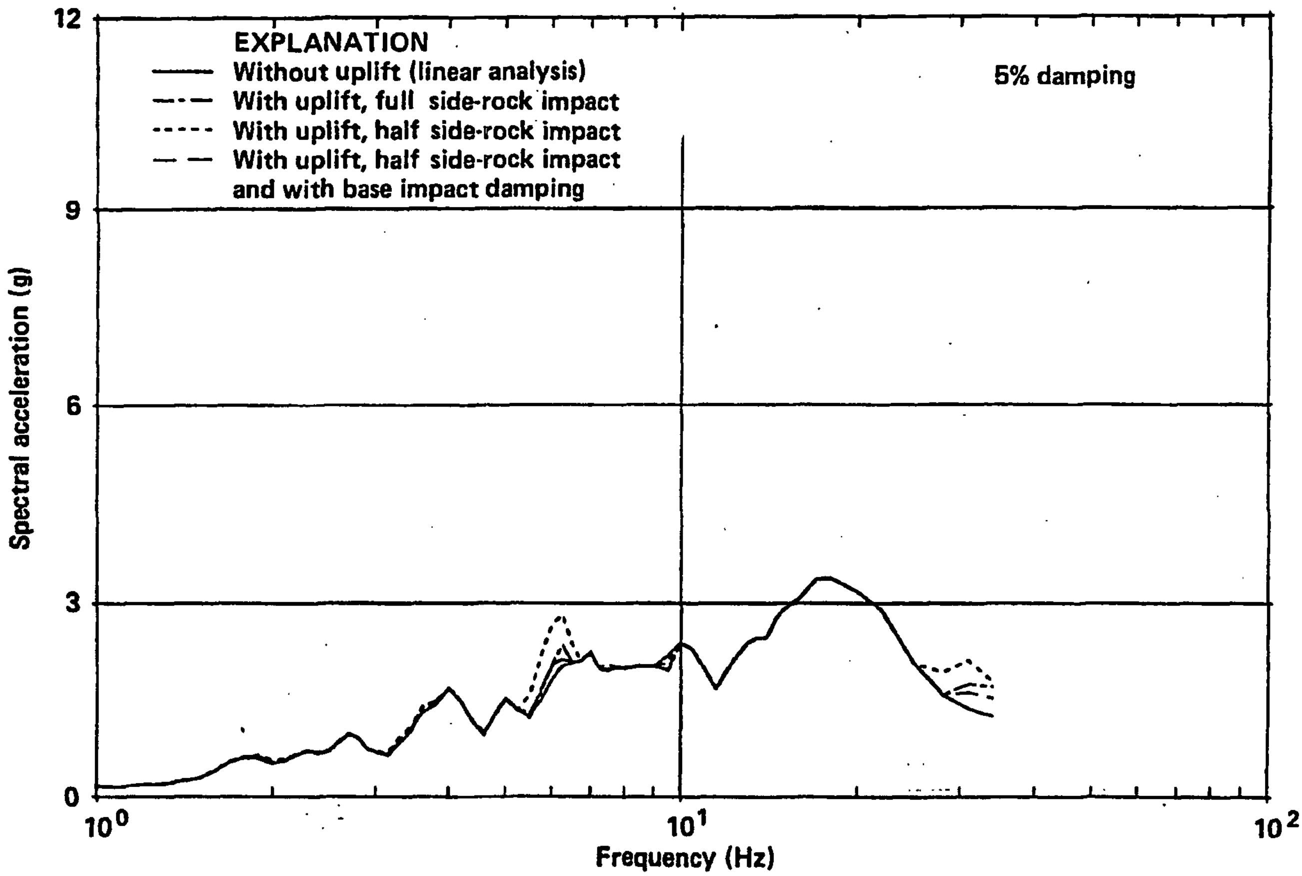


Figure 5-64

Floor response spectra for the vertical response of containment interior structure at El 231 feet due to scaled El Centro No. 4 input.

structure-to-structure interaction, degradation of rock shear modulus, and variations of Poisson's ratio and material damping ratio of the foundation rock are relatively unimportant;however, the effects of foundation embedment and foundation basemat flexibility are relatively important for the power block structures. The important parameters, such as foundation embedment and basemat flexibility, were incorporated into the models of the power block structures for the final soil/structure interaction analyses.

Soil/Structure Interaction Response to Coherent Ground-Motion Inputs. The basic soil/structure interaction responses of the power block structures required for the Plant fragility evaluations and seismic margin assessment were generated using the three-dimensional SASSI time history response analyses with coherent ground-motion inputs; the input motions were consistent with the site-specific earthquake response spectrum and at a level slightly higher than the site-specific 84th percentile response spectum. The results of these analyses indicated substantial soil/structure interaction effects, mainly due to inertial interaction, for the short, stiff containment interior structure and the auxiliary building. The soil/structure interaction effects due to coherent ground-motion excitation was, however, found to be relatively small for the taller and more flexible containment shell and the turbine building.

Adjustment of Soil/Structure Interaction Responses Due to Special Incoherence of Ground Motions. To account for the effect of spatial variations of ground motions on soil/structure interaction response, separate analyses, using the CLASSI analysis technique and random vibration theory, were performed incorporating site-specific spatial incoherence functions. Soil/structure interaction response adjustment factors, in the form of floor response spectral ratios applicable to specific response directions and locations, were developed to adjust the floor response spectra resulting from the coherent ground-motion analyses to give the final soil/structure interaction responses for the Plant fragility evaluations. The results of these analyses showed that spatial incoherence of ground motions generally resulted in a reduction in the soil/structure interaction responses. However, the amount of reduction varied from point to point within the structure. These variations resulted from rocking and torsional response motions induced by spatial incoherence. At the structural base near the center region (which is not affected by rocking and torsion), in the frequency range above 10 hertz, such reductions are about 6 percent for the containment, 15 percent for the auxiliary building, and 20 percent for the turbine building.

Containment Base Uplift Effects. The effect of base uplift on the containment seismic response was investigated using a separate study that used a two-dimensional nonlinear time history base-uplift response analysis procedure. This study considered the seismic input from three sets of three-component actual earthquake ground-motions adjusted to an intensity level higher than the site-specific 84th percentile ground motion level. It also considered foundation model parameter variations including the partial loss of side rock support for embedded basemat wall and the base slapdown impact energy dissipation. The results of the study indicated that base-uplift has no significant effect on the dynamic response of the containment structure, even under the strong input motions considered in the study.

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Chapter 6 of the 1988 Long Term Seismic Program Final Report

Chapter 6
PROBABILISTIC RISK ANALYSIS
To Partially Address
Element 4 of the License Condition

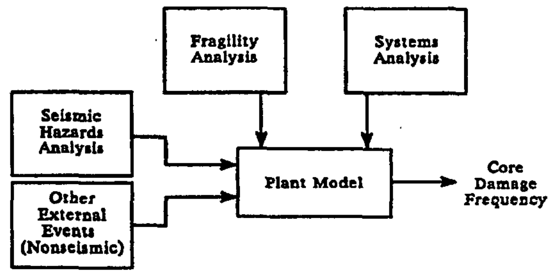
ELEMENT 4 OF THE LICENSE CONDITION

PG&E shall assess the significance of conclusions drawn from the seismic reevaluation studies in Elements 1, 2, and 3, utilizing a probabilistic risk analysis and deterministic studies, as necessary, to assure adequacy of seismic margins.

INTRODUCTION

Element 4 of the license condition calls for an assessment of the significance of conclusions drawn from the seismic studies in Elements 1, 2, and 3, utilizing a probabilistic risk analysis and deterministic studies, as necessary, to assure adequacy of seismic margins. This chapter summarizes our approach to and key findings from the probabilistic risk analysis. The approach and findings related to deterministic studies are summarized in Chapter 7.

The results presented in the earlier chapters have been integrated to develop seismic hazard curves and fragilities of Plant structures and items of equipment that are important to evaluating probabilities of seismic risk. The seismic hazards and fragilities are combined to perform a systems analysis on the Plant risk model as part of the probabilistic risk analysis.



This chapter details the processes and results of each component of the probabilistic risk assessment and how these components are combined to produce the results. The Seismic Hazards Analysis is described first, followed by the Seismic Fragility Analysis. Finally, the remaining components are described in the Probabilistic Risk Assessment.

SEISMIC HAZARDS ANALYSIS

Objectives

The objective of the seismic hazards analysis was to provide a probabilistic representation of the earthquake ground motions at the Diablo Canyon Power Plant site, in a format suitable for use in the probabilistic risk analysis. A secondary objective of the seismic hazards analysis was to calculate constant hazard spectra over the frequency range of interest to Plant structures and equipment.

Scope

The seismic hazards analysis included consideration of all seismic sources that can affect ground motions at the Diablo Canyon Power Plant site. Logic trees were developed for the Hosgri, Los Osos, San Luis Bay, Santa Lucia Bank, West Huasna, offshore Lompoc, Rinconada, Nacimiento, and San Andreas faults. Seismic hazards calculations were performed and it was shown that the Hosgri fault dominates the seismic hazard at the site, and that the Los Osos and San Luis Bay faults taken together add only a few percent to the total seismic hazard. Relative contributions to the total hazard from the other faults are insignificant.

The seismic hazards analysis for the Hosgri, Los Osos and San Luis Bay faults was performed in terms of response spectral acceleration, in order to provide consistency with the fragility estimates of Plant structures and equipment.

The development of ground-motion attenuation relationships applicable to the Diablo Canyon Power Plant site is described in Chapter 4. For use in the seismic hazards analysis, attenuation relationships were developed for spectral acceleration at 5 percent damping, at frequencies of vibration of 33, 25, 14, 8, 4, and 2 hertz, and for average spectral acceleration in the ranges of 3 to 8.5 hertz and 5 to 14 hertz. These relationships include factors that represent the different styles of faulting included in the logic tree representation (strike-slip, oblique-slip and thrust) based on results derived from the numerical modeling program, from the empirical ground-motion studies, and from review of available literature.

Seismic hazards analyses for the Hosgri, Los Osos, and San Luis Bay faults were performed for each of the structural frequencies mentioned above (33, 25, 14, 8, 4, and 2 hertz), and for the frequency ranges of 3 to 8.5 hertz, and 5 to 14 hertz. From these multiple hazards analyses, the hazards curves representing the frequency range of 3 to 8.5 hertz were selected for use in the probabilistic risk assessment, and are presented herein. In addition, the analyses at individual frequencies were used to construct constant hazard response spectra as presented herein.

The results of the seismic hazards analyses are presented in terms of fractile hazard curves, which show at each spectral acceleration amplitude the distribution of hazard from the entire family of hazard curves, and in the form of aggregate hazard curves, which reduce the large number in the total family of hazard curves to a limited number of curves (about 8 to 12) for input into the probabilistic risk assessment.

Method of Analysis

The procedures used to calculate seismic hazard for the case when faults can be identified as the potential sources of earthquakes are documented in detail (for example, Der Kiureghian and Ang, 1977; McGuire, 1978). The steps involved in a seismic hazards analysis are illustrated in Figure 6-1. The calculation of seismic hazard is made with the following equation:

$$\nu(a^*) = \sum_i \nu_i \iint G_{A|m,d}(a^*)\, f_{M(i)}(m)\, f_{D(i)}(d|m)\, dm\, dd \qquad (6-1)$$

in which the summation is performed over all faults i that affect the site, ν_i is the mean annual rate of damaging earthquakes for fault i. The probability-density function of magnitude and distance for fault i are $f_{M(i)}(m)$ and $f_{D(i)}(d|m)$. (The distance distribution depends on magnitude because the rupture length is explicitly taken into account.) The ground-motion or attenuation model allows calculation of, for a given magnitude m and distance r, the probability $G_{A|m,d}(a^*)$ that a ground motion amplitude a* is exceeded. The hazard defined in equation 6-1 represents the annual rate ν at which ground-motion amplitude a* is exceeded at the site; because it is much smaller than unity, this rate can be interpreted as the probability that ground-motion amplitude a* is exceeded in any one year. As is common in probabilistic risk assessments, we refer to this rate as an "annual frequency of exceedance." The calculation of equation (6-1) is performed for several values of a* and the resulting values can be plotted as a "hazard curve," illustrated on Figure 6-1(D).

This is a standard formulation of seismic hazard; the application takes proper account of randomness in the following factors:

- Fault geometry in three dimensions,
- All possible locations of the rupture surface, both horizontally and vertically,

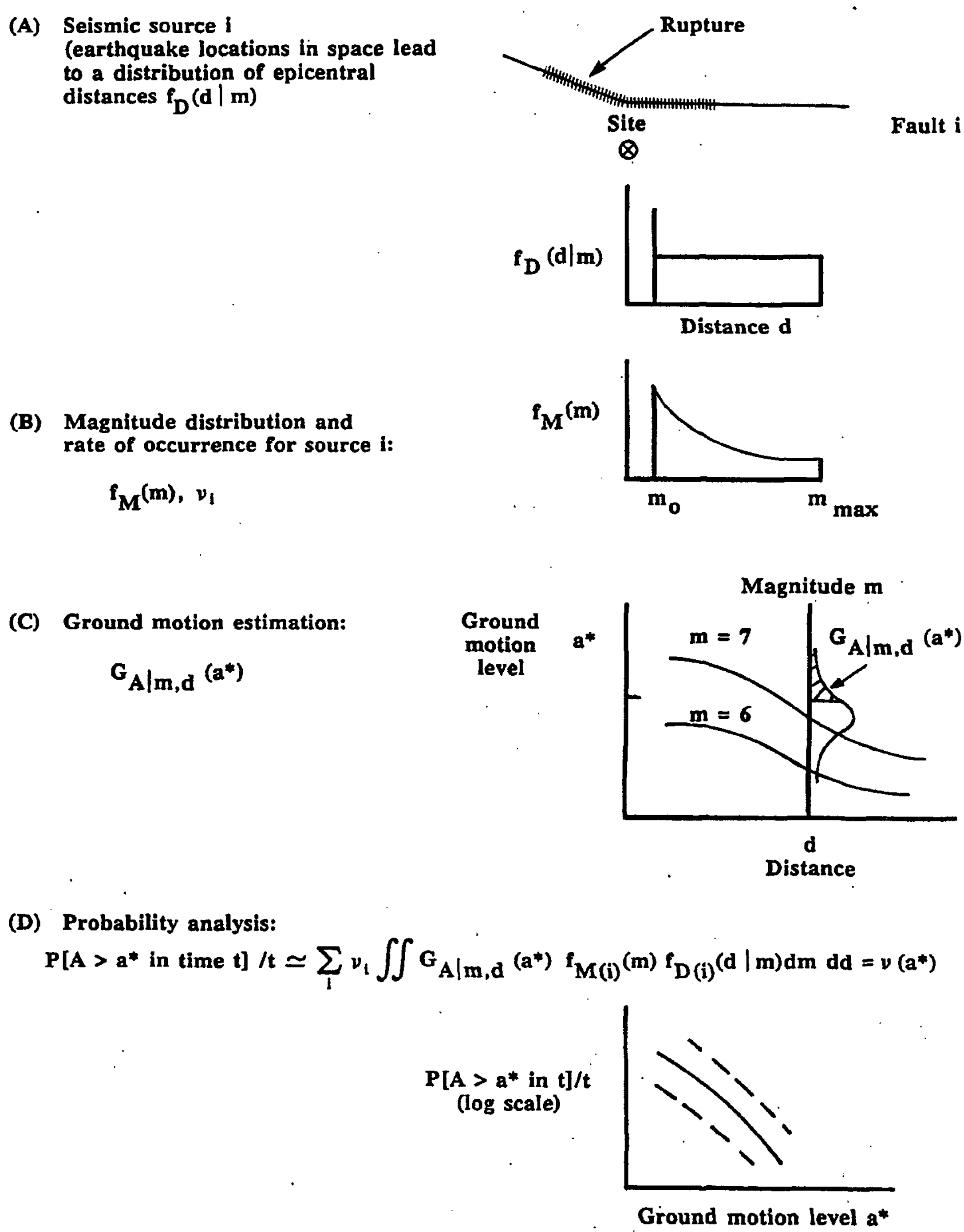


Figure 6-1

Steps involved in seismic hazard analysis.

- Sizes (magnitudes of earthquakes that might occur on the fault),
- Size of rupture as a function of earthquake magnitude,
- Closest distance of the site to the rupture, as required by the ground-motion estimation equations,
- Ground motions at the site as a function of the earthquake magnitude and its location relative to the site, and
- Possible amplification or reduction of the ground motions as a result of the sense of fault slip and geometry of the fault.

Thus, for a given fault geometry and style of faulting, the calculation integrates over all possible magnitudes of earthquakes, generates a rupture surface for each magnitude, and integrates over all possible locations of the rupture surface on the fault plane. For each possible rupture location, the procedure calculates the distance to the Plant site; estimates the distribution of site ground motions, accounting for any amplification or reduction caused by faulting style and geometry; and integrates over randomness in ground motions, given the earthquake magnitude and location with respect to the site. The result is a calculation of annual rates (probabilities) that specified levels of ground shaking will be exceeded. The procedure accounts for randomness in the models used to represent earthquake occurrences: earthquake magnitudes, rupture locations, times of occurrence, and ground-motion levels given the occurrence of the event.

Uncertainties are distinct from randomness in the sense that they involve parameters and models that are chosen to describe earthquake occurrences; in concept, uncertainties can be reduced as more data are collected and physical processes are better understood. Uncertainties are treated by performing separate hazard calculations (equation 6-1) for different sets of models and parameter values. Hence, uncertainty in the input results in uncertainty in the hazard curve, which may be represented by a family of hazard curves or by fractiles of hazard at all ground-motion amplitudes. The uncertainties in input were represented using the logic tree format, an example of which is shown on Figure 6-2. Each element in the logic tree consists of a set of nodes representing an uncertain state of nature, and each branch represents discrete possible values for that state. Probabilities were assigned to each branch using subjective assessments, and the end branch probabilities were calculated as the product of all the intermediate branch probabilities. A single seismic hazard analysis was performed for each end branch resulting in a single hazard curve for the set of assumptions that led to that end branch. The eight hazard curves for the logic tree on Figure 6-2 are illustrated at the right side of the figure. The uncertainty in hazard is represented by this family of hazard curves, the size of the family being equal to the number of end branches.

Typically, large numbers (several thousand) of hazard curves result from practical applications of the logic-tree concept. This large number is reduced to summary curves, both for examination and analysis and for input to other Plant evaluations. One simple representation of the uncertainty in hazard is gained through fractile hazard curves, which show, at each ground-motion amplitude, the distribution of hazard from the family. A second representation is through aggregate hazard curves, which reduce the large number in the total family of hazard curves to a limited number of curves (about 8 to 12) for input into a probabilistic risk assessment of the Plant systems.

The logic tree approach has several important advantages over others that might be pursued. First, the complete enumeration of all possible states of nature ensures that all hypotheses have been accounted for properly, with appropriate weights assigned to each. As a result of the efficient algorithms used to calculate seismic hazard, no compromises need be made to keep the number of combinations small or to reduce the number of hypotheses that can be considered. The procedure allows consideration of all suggestions made about tectonics, fault behavior, seismicity, and ground-motion characteristics,

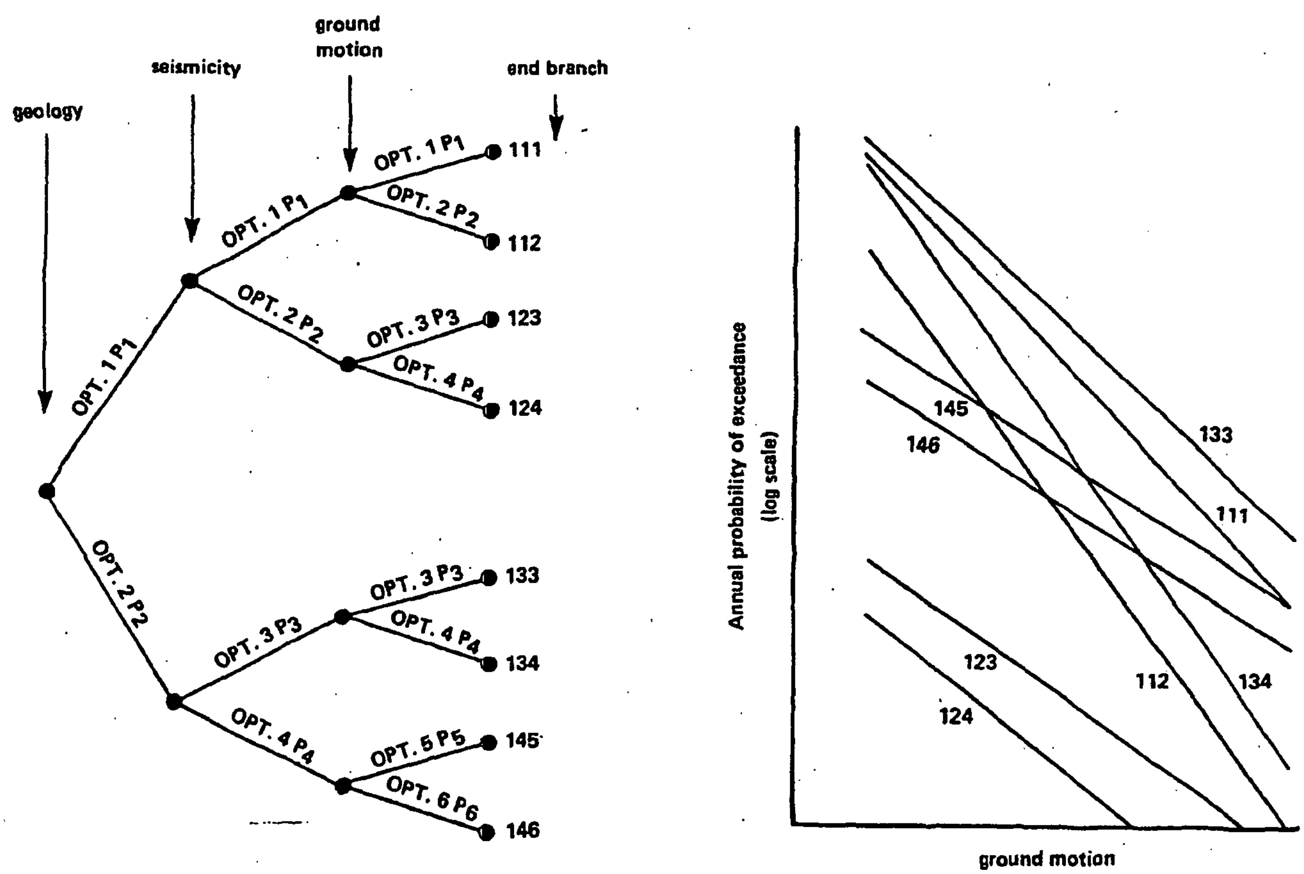


Figure 6-2
Example of logic tree and resulting family of hazard curves.

however unlikely, and puts them in the proper context along with all other interpretations.

Second, the procedure provides a logical means of identifying those elements that contribute importantly to uncertainty in seismic hazard and those that do not. This allows priorities for investigations on appropriate input models and parameters to be established on a logical basis.

Third, the entire procedure is documentable and trackable, so that decisions (for example, which faults to investigate further can be justified and defended.

The analyses considered here calculated hazard from each fault separately. Although several faults are currently active in south-central coastal California, the Hosgri fault dominates the seismic hazard at the Diablo Canyon site (as will be demonstrated below), so that consideration of multiple faults acting simultaneously is not required. The total hazard can be accurately calculated by considering each fault characterization separately, and combining hazards to evaluate the total hazard.

The logic tree used to represent input for the Hosgri, Los Osos, and San Luis Bay faults is shown on Figure 6-3.

A total of 20,700 end branches of the logic tree resulted from the input specification. The resulting family of hazard curves is too numerous to interpret, or even to illustrate on a single plot. As described above, one summary of this family can be constructed by determining, at each ground motion amplitude, the distribution of annual frequency of exceedance, and identifying the frequencies that are associated with certain preselected fractiles. For example, at each ground-motion amplitude the *median* frequency of exceedance can be determined, meaning that curves below that frequency have 50 percent of the total weight. Constructing a plot of frequency of exceedance versus ground-motion, and drawing these medians, gives an indication of the median seismic hazard for all ground motions. This procedure can be applied to other fractiles as well.

For probabilistic risk analyses, it is necessary to construct a sophisticated representation of the family of hazard curves. The reason is that probabilistic risk assessment procedures treat uncertainty by conditioning on alternative interpretations (in this case seismic hazard curves), convolving these with alternative representations of Plant response, and calculating the resulting uncertainty in Plant state frequency. Therefore, if several hazard curves represent the uncertainty in geological and seismological interpretations, and these curves have different slopes, the character of the curves (slopes) must be maintained for probabilistic risk assessment input. Fractile hazard curves do not transmit this information.

To derive hazard results appropriate for probabilistic risk assessment, an aggregation process is employed that reduces the large number of hazard curves (20,700) to a few (typically 8 to 12), using a procedure that optimally determines how to combine pairs of curves sequentially so that the character of the original curves will be maintained, and the set of aggregate curves will represent as much of the original uncertainty in hazard as possible for each ground-motion amplitude. The procedure uses the following steps:

1) A contribution to variance analysis is used to select nodes on the logic tree that do not contribute significantly to uncertainty in hazard. The logic tree is then restructured to reduce the number of end branches by combining hazard results for end branches that are identical except for branches at nodes that contribute little to the uncertainty in hazard. By this mechanism the family of hazard curves is reduced to several hundred in number. These hazard curves typically represent greater than 96 percent of the total uncertainty in hazard.

2) The hazard curves are characterized by the frequency of exceedance at three ground-motion amplitudes, chosen as those most critical to the determination of Plant response and system state. The total variance in frequency of exceedance at these three amplitudes is calculated.

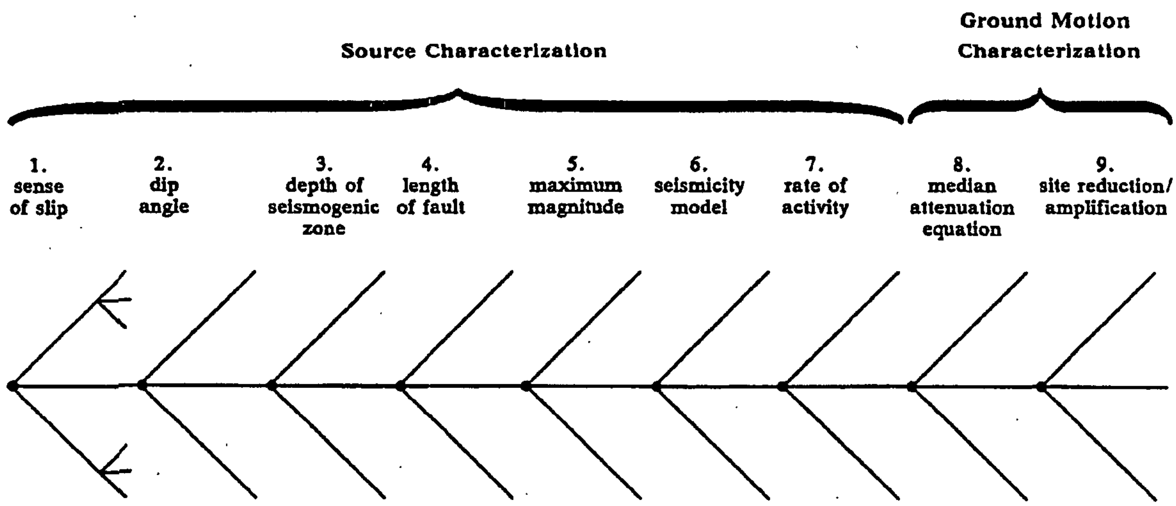


Figure 6-3

Elements in logic tree used for Hosgri, Los Osos, and San Luis Bay faults.

3) A small number of possible aggregate curves (for example, 64) is estimated by dividing the ranges of frequencies of exceedance into intervals and constructing a first set of aggregates at the centers of these intervals.

4) Each of the hazard curves is assigned to a tentative aggregate curve, based on its proximity in frequency-of-exceedance for the three amplitudes.

5) The tentative aggregate curves are recomputed as the conditional mean or the assigned curves.

6) Steps 4 and 5 are repeated, because step 5 may change the assignments based on proximity, until the tentative aggregate curves are stable (that is, until there are no changes in assignments). A weight for each tentative aggregate curve is calculated as the sum of weights of the assigned curves.

7) All possible pairs of tentative aggregate curves are examined as candidates for combination; the pair that, when combined, will result in the minimum reduction in variance is selected and combined by computing the weighted average frequency of exceedance for all three amplitudes. The combined curve is assigned a weight equal to the sum of the weights of the two curves used to calculate it.

8) Steps 4 through 7 are repeated to reduce sequentially the number of tentative aggregate curves. The process ends when the desired number of aggregate curves is reached.

9) The curve assignments are used to calculate aggregate hazard curves for all ground-motion amplitudes; the weight given to each aggregate is the sum of the weights of the assigned curves.

There are no general solution techniques for aggregating a discrete, multidimensional distribution, but the above algorithm has been tested for a number of seismic hazard problems and works well. It is efficient for up to several hundred initial hazard curves (which is the reason for Step 1). Typically, 8 to 12 aggregate curves can be constructed with this algorithm that replicate about 90 percent of the total variance of the original data set, for all ground-motion amplitudes (that is, the standard deviation of frequency of exceedance is 95 percent of the original). Figure 6-4 illustrates how this procedure would work for the case of reducing nine hazard curves. Three aggregate curves adequately represent the amplitude and slope of the original nine curves.

Input Data

As illustrated schematically on Figure 6-1, input data for the seismic hazards analysis consisted of seismic source characteristics (location and recurrence) and ground-motion attenuation relationships.

SEISMIC SOURCE CHARACTERISTICS

The logic trees for the Hosgri, Los Osos, and San Luis Bay faults are given in Chapter 3. The range of parameters and associated probabilities provide a description of the uncertainties associated with the characteristics of each earthquake source. Included in the analysis of the logic tree are calculations of the distribution of maximum magnitudes and recurrence relationships for each source. In addition, the calculations of seismic hazard include the ranges of fault geometries given in the logic trees in defining source locations.

The input data for the Hosgri, Los Osos, and San Luis Bay faults are summarized as follows:

Hosgri Fault. Geologic data, were provided for the first four nodes of the Hosgri fault logic tree (Figure 6-3). These are summarized as follows:

Style of Faulting	Dip (Degrees)	Depth (km)	Fault Length (km)
Strike-slip	90, 70	9, 12, 15	410
Oblique	90, 60, 45	9, 12, 15	110, 250, 410
Thrust	60, 30, 15	9, 12, 15	110, 160, 250

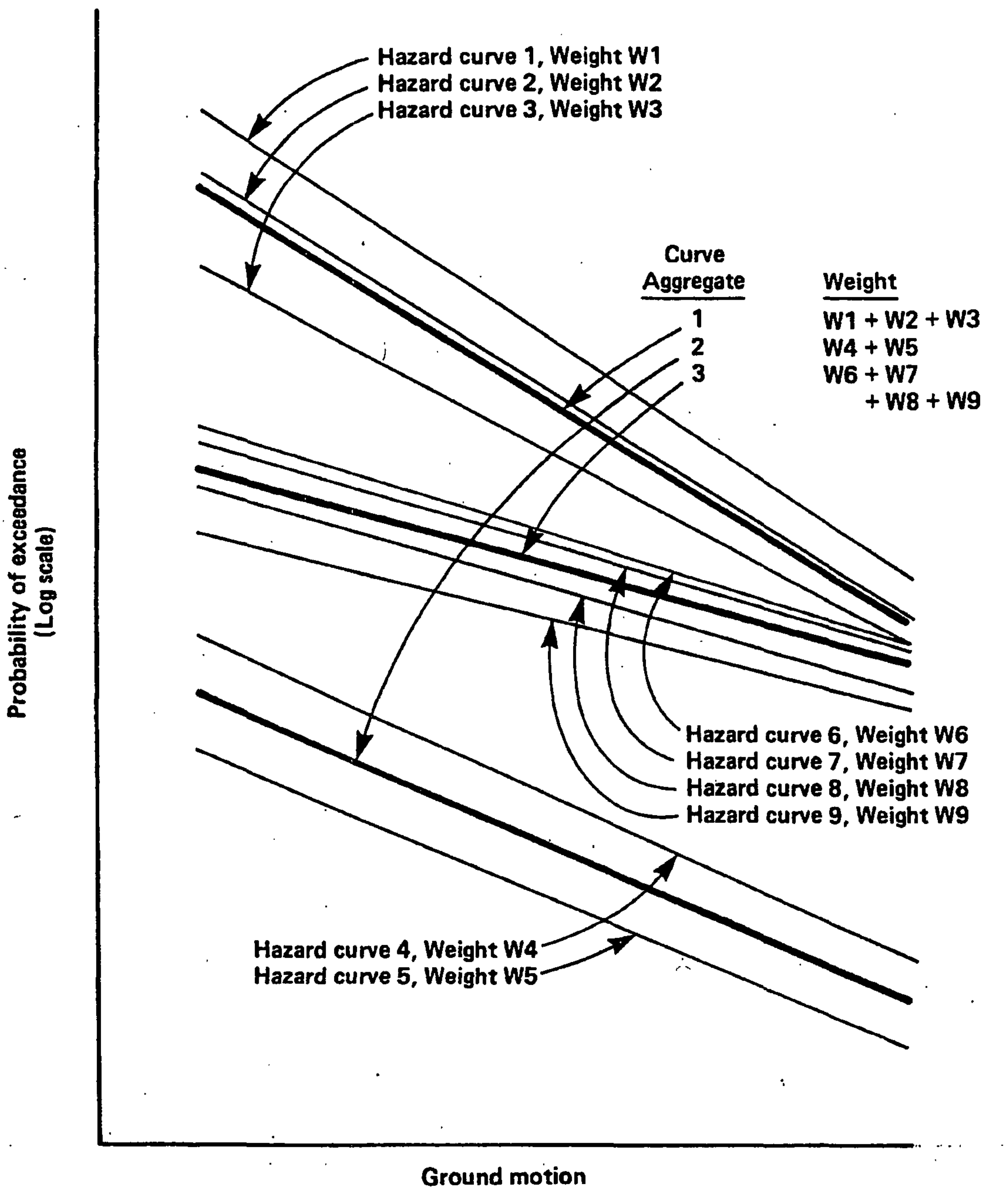


Figure 6-4
Example of aggregation of nine hazard curves to obtain three curves.

Weights assigned to the style of faulting interpretations are as follows: strike-slip = 0.65; oblique = 0.30; thrust = 0.05. Weights assigned to the subsequent interpretations are conditional on style of faulting.

Seismological input constituted the next three sets of nodes on the logic tree of Figure 6-3. The assessments of maximum magnitudes (the fifth element of the logic tree) and their probabilities are conditional on previous branches; values chosen for maximum magnitude range from 6.5 to 7.75 M_W. Two seismicity models (logic tree element 6) were used, exponential and characteristic; these were weighted 0.4 and 0.6, respectively, for all faults. The rate of seismic activity (element 7) was discretized and estimated from interpretations of fault slip rate; the values and their probabilities are conditional on previous branches of the logic tree.

Los Osos Fault. The geologic input for the Los Osos fault is summarized as follows:

Style of Faulting	Dip (Degrees)	Depth (km)	Fault Length (km)
Oblique	75, 45	9, 12, 15	16, 24, 36, 44, 49, 57
Thrust	60, 30	9, 12, 15	16, 24, 36, 44, 49, 57

Weights assigned to the style of faulting interpretations are as follows: oblique = 0.1; thrust = 0.9. Weights assigned to the subsequent interpretations are conditional on the style of faulting.

San Luis Bay Fault. The geologic input for the San Luis Bay fault is summarized as follows:

Style of Faulting	Dip (Degrees)	Depth (km)	Fault Length (km)
Thrust	70, 40	9, 12, 15	6, 12, 19

In this case a weight of unity was assigned to the thrust interpretation.

Other Faults. Other faults considered in the hazard analysis are the Santa Lucia Bank fault, the Rinconada fault, the Nacimiento fault, the offshore Lompoc fault, and the West Huasna fault. Input for these faults was specified using the logic tree format. The hazard from these faults is several orders of magnitude lower than for the Hosgri, as will be documented below. Thus, the total seismic hazard at the Plant can accurately be calculated by considering only the Hosgri, the Los Osos, and the San Luis Bay faults.

GROUND-MOTION CHARACTERISTICS

Ground-motion input constitutes the last two elements of the logic tree. Three median ground-motion attenuation relationships (element 8) were used for all faults. The attenuation equations for the eight frequencies and frequency bands investigated are listed in Table 6-1. Note that for use in probabilistic seismic hazard analyses, the nonlinear magnitude scaling of spectral ordinates (presented in Chapter 4) was simplified into a bilinear form to provide linear magnitude scaling within two magnitude ranges, $M_W < 6.5$ and $M_W \geq 6.5$. The coefficients for this bilinear form provide essentially identical spectral values in the magnitude range M_W 5.5 to 7.5, which is the range of interest to the seismic hazard analysis.

The coefficients given in Table 6-1 represent the average amplitudes for two horizontal components. The variability in amplitude was expressed as the standard deviation of ln (spectral acceleration) = 0.36 for magnitude greater than or equal to 6.5, and 1.27 - 0.14M for magnitude less than 6.5. This is the variability specified for the frequency bands 3 to 8.5 hertz and 5 to 14 hertz, and does not include frequency-to-frequency variations (these variations have been averaged by calculating the average spectral acceleration for a frequency band). Because the amplitudes desired for the probabilistic risk assessment are spectral accelerations (average of two horizontal components, without peak-to-valley variability from frequency to frequency), the above variability was used for all frequencies.

The site factor (element 9) represents the portion of empirical ground-motion variability that can be attributed to variability in site characteristics. As

Table 6-1

ATTENUATION EQUATIONS* FOR SPECTRAL ACCELERATION (5% DAMPING) FOR THRUST FAULTING**

$$\ln(S_a[f]) = c_0 + c_1M + c_2\ln[D + c_3\exp(c_4M)]$$

$c_2 = -2.1$ for all frequencies and magnitudes

$c_3 = 3.656$ and $c_4 = 0.25$ for M<6.5

$c_3 = 0.616$ and $c_4 = 0.524$ for M>6.5

f(Hz)	c_0(for M≥6.5)	c_0(for M<6.5)	c_1(for M≥6.5)	c_1(for M<6.5)
33	-1.092	-0.442	1.1	1.0
25	-0.943	-0.293	1.1	1.0
14	-0.280	+0.695	1.05	0.90
8	-0.327	+0.323	1.1	1.0
4	-0.872	-0.840	1.184	1.179
2	-1.902	-2.624	1.286	1.397
3-8.5	-0.537	-0.154	1.136	1.077
5-14	-0.374	+0.276	1.1	1.0

* *Coefficients above represent the best-estimate equation, which is assigned a weight of 0.5; alternative equations, which were assigned weights of 0.25 each, provide acceleration values 1.15 times the above values, and 1/1.15 times the above values.*

** *Equations for strike-slip faulting are obtained by multiplying the reverse/thrust amplitudes by 0.833. Equations for oblique faulting are obtained by multiplying the reverse/thrust amplitudes by 0.913.*

Note: M is moment magnitude, D is closest distance to rupture surface, in kilometers.

such, this variability is treated as an uncertainty for any specific site. The variance representing total ground-motion variability discussed above (represented by the variance of ln [spectral acceleration]) was divided into two parts:

$$\sigma_{tot}^2 = \sigma_r^2 + \sigma_{site}^2 \qquad (6\text{-}2)$$

where σ_{tot}^2 is the total variance of ground-motion amplitude (that is, of response spectral amplitude at a given frequency), σ_r^2 is variance attributed to randomness and σ_{site}^2 is variance attributed to uncertainty in site conditions. As discussed above, σ_{tot} was specified as 0.36 for magnitude greater than 6.5. We divide the total variance equally between σ_r and σ_{site}, so that both are equal to 0.255. As σ_{site} is treated as uncertainty, we represent it with element 10 of the logic tree (Figure 6-3) and use five discrete factors of 0.682, 0.869, 1.00, 1.15, and 1.47, weighted equally, to represent this uncertainty in site response. The total variability in ground motions was truncated at three standard deviations, but this truncation has almost no influence on the final hazard results.

Results of Analysis

Results of the hazard calculations are shown for the Hosgri fault on Figure 6-5 in the form of fractile hazard curves for spectral acceleration in the frequency range 3 to 8.5 hertz (5 percent damping). These fractile curves illustrate the range of uncertainty in hazard that results from uncertainty in the geologic, seismologic, and ground-motion input.

Figure 6-6 compares the mean hazard from the Hosgri fault to mean hazards from the Los Osos fault and the San Luis Bay faults (for spectral acceleration in the same frequency range of 3 to 8.5 hertz), and to approximate mean hazards from the Nacimiento, West Huasna, Rinconada, offshore Lompoc, and Santa Lucia Bank faults. The approximate mean curves were constructed by determining the ratios of hazards from these faults to that of the Hosgri under the same ground-motion assumptions, and applying these ratios to the current mean Hosgri hazard curve that uses the most current ground-motion assumptions. This approximation is justified in light of the low hazards that these curves indicate, compared to the Hosgri fault. It is clear that the Hosgri fault zone is the dominant contributor to the seismic hazard, with the Los Osos and San Luis Bay faults contributing a minor fraction of this hazard (about 3 to 5 percent in aggregate) and the remaining faults contributing hazards that are several orders of magnitude lower.

To calculate aggregate hazard curves for input to probabilistic assessment, the family of 20,700 Hosgri hazard curves based on spectral acceleration for 3 to 8.5 hertz were aggregated to eight curves, using the method presented in the previous section. For this aggregation process, hazards at 1.5 g, 2.0 g, and 3.0 g spectral acceleration were used, as these levels of ground motions contribute most to Plant seismic risk studied in probabilistic risk assessments and therefore are the most important to represent accurately. To these eight aggregate curves were added the mean hazards from the Los Osos and San Luis Bay faults. This procedure preserves the mean total hazard from all three faults, and incurs almost no loss of accuracy in representing the uncertainty in hazard, because of the low contribution of these faults relative to the Hosgri. Figure 6-7 shows the resulting eight aggregate hazard curves. The seismic hazard is highly skewed, with a high probability at relatively low hazards and a small probability of relatively high hazards. This characteristic is properly portrayed by the aggregate hazard curves. As discussed in the previous section, the amplitudes presented on Figure 6-7 are spectral accelerations for the average of two components, with frequency-to-frequency (peak and valley) variation removed.

A second set of hazard curves is presented on Figure 6-8 as fractile curves of total hazard. These curves were obtained in a manner similar to the aggregate curves; that is, fractile curves were calculated for the Hosgri fault, and mean hazards were added to represent the Los Osos and San Luis Bay faults. Thus, these fractile curves are approximate for the lower fractiles; they are very accurate for fractiles above the median.

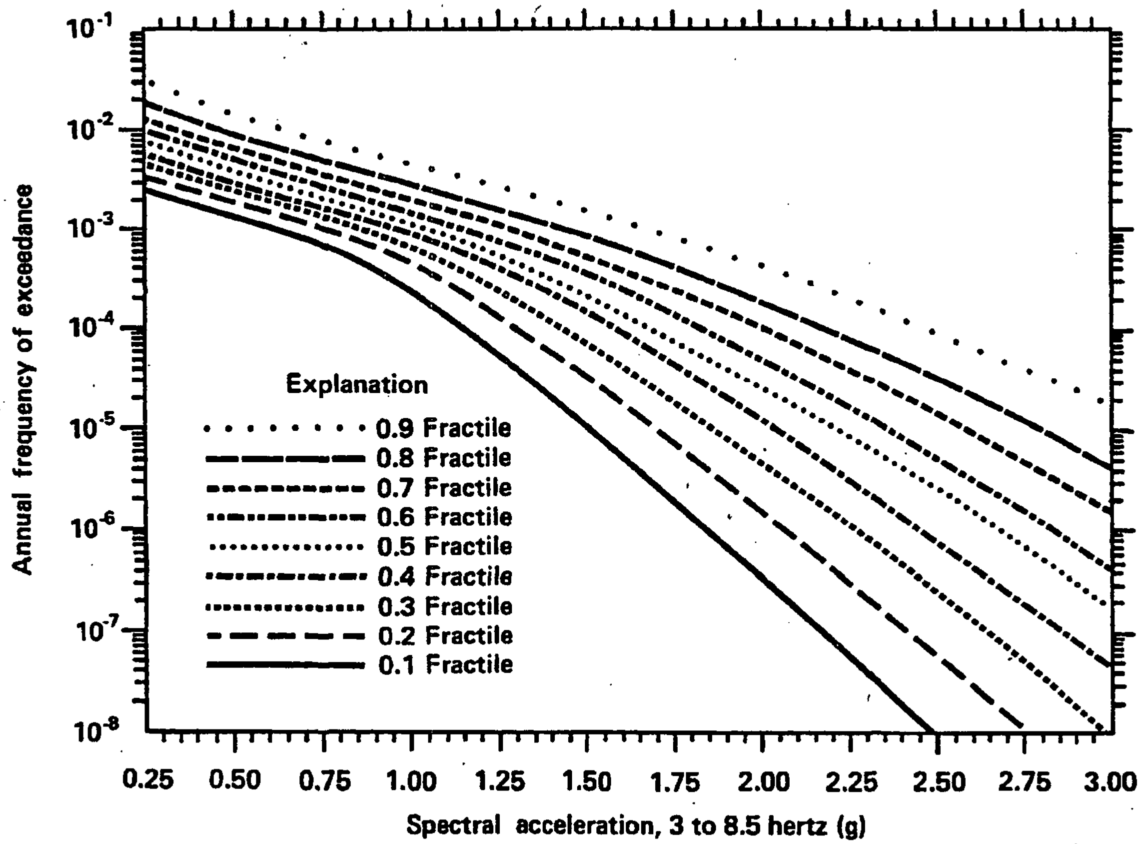


Figure 6-5
Fractile seismic hazard curves for Hosgri fault zone.

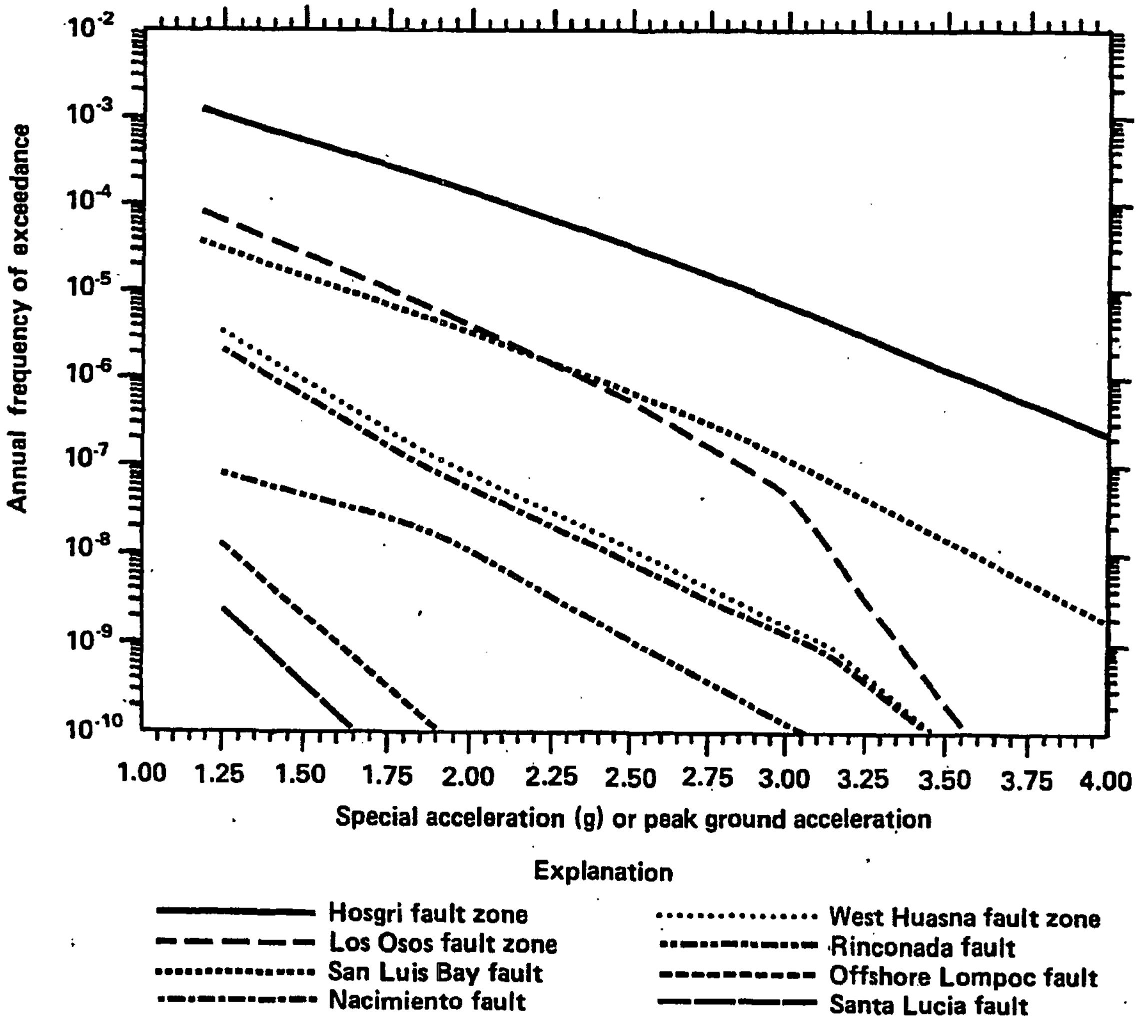


Figure 6-6

Comparison of mean hazard from Hosgri fault zone to mean hazards from Los Osos and San Luis Bay faults, and to approximate mean hazards from other faults.

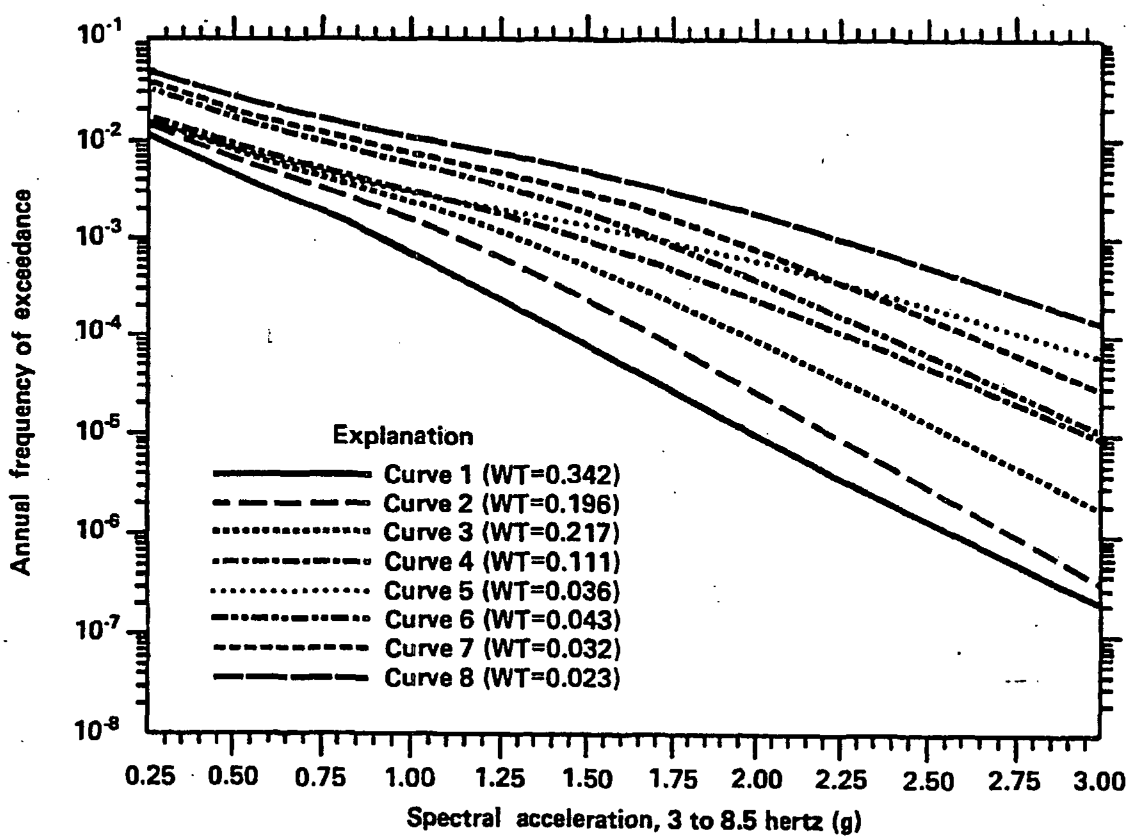


Figure 6-7
Total aggregate hazard curves.

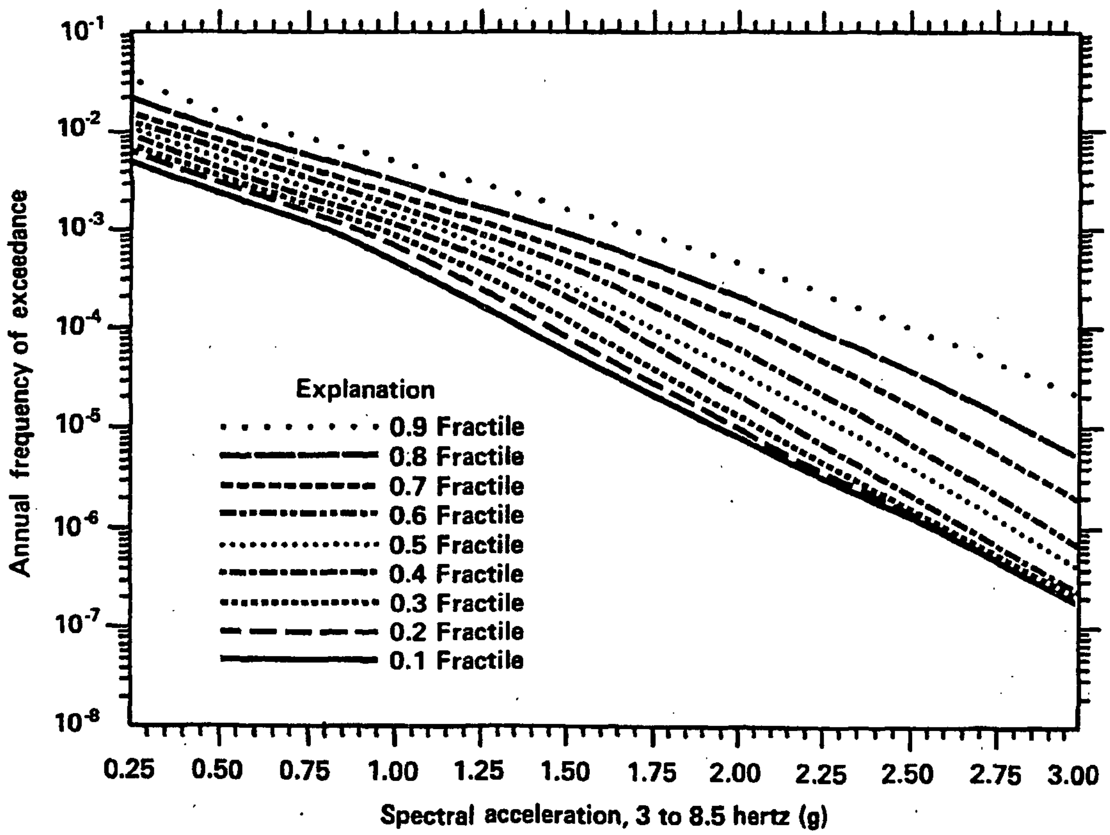


Figure 6-8
Curves representing approximate fractiles of total hazard.

The final set of results was obtained from the hazard calculations at all six frequencies shown in Table 6-1. For each frequency, total fractile hazard results were prepared (as illustrated on Figure 6-8 for the frequency range 3 to 8.5 hertz) and spectra were calculated for 10^{-3}, 10^{-4}, and 10^{-5} annual frequencies of exceedance, for median results. These spectra are shown on Figure 6-9.

SEISMIC FRAGILITY ANALYSIS

Objectives

As part of the probabilistic risk assessment, a seismic fragility evaluation of key safety related structures and equipment was conducted. The seismic fragility evaluation consisted of a probabilistic definition of seismic capacity which, together with a probabilistic definition of the seismic hazard and an event-tree and fault-tree characterization of the operating system, provided the necessary data for the probabilistic risk assessment. The objective of the fragility analysis was to carefully evaluate each of the structures and components which are included in the risk model to define those failure modes that have the lowest seismic capacities and which, therefore, may constitute the most important or dominant contributors to Plant seismic risk.

Scope

The Diablo Canyon seismic fragility evaluation studies were conducted over a period of approximately 3 years in a phased approach designed to clearly identify and reevaluate those components whose failure most substantially contribute to plant risk. Appropriate aspects of the various Diablo Canyon Long Term Seismic Program studies, including the site-specific geotechnical and soil/structure interaction investigations, the median in-structure response spectra evaluation, and the structural response variability investigation were incorporated into the fragility evaluations.

The fragility description of structures consisted of the identification and evaluation of controlling failure modes associated with the important structures (Table 6-2). Similarly, the fragility description of mechanical and electrical equipment consisted of the identification and evaluation of controlling failure modes related to elements of the major safe shutdown reactor plant systems (Table 6-3). In every case, the fragility analyses were based upon Plant-specific structure or component seismic qualification analyses directly related to elements in place at the Diablo Canyon Plant. Even the fragility for generic component categories, whose elements are too numerous to evaluate individually, were based upon a sampling of Plant-specific seismic qualification analyses for components in the category. Typical generic component categories are listed in Table 6-4.

Method Of Analysis

The definition of failure is vitally important to the development of median fragilities for structures and equipment. For purposes of this study, Category I structure failure was defined in terms of inelastic lateral drifts generally corresponding to the onset of significant strength degradation of major structural elements. The exception is the containment building where lateral drifts were limited to lower levels consistent with the need of the containment building to remain pressure-tight. Equipment housed in the important structures was assumed to fail when the structure reached lateral drifts corresponding to the onset of significant strength degradation or severe distress. The fragility estimates for structures correspond to distress levels short of partial or total collapse, but are treated as total collapse in the probabilistic risk assessment. The degree of margin between the onset of significant strength degradation and total collapse is uncertain and difficult to estimate. However, the benefits of this margin, which in most cases is likely to be large, has been conservatively ignored.

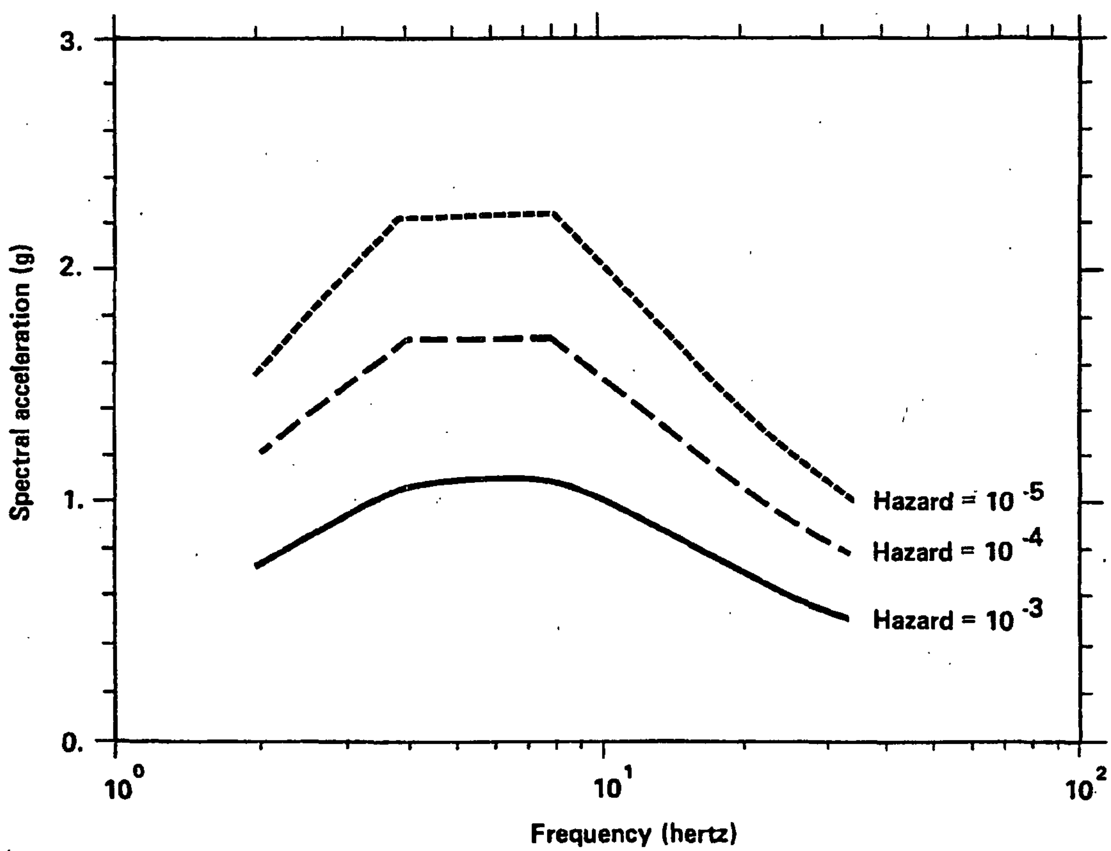


Figure 6-9
Median constant-hazard spectra (5 percent damping).

Table 6-2

IMPORTANT STRUCTURES

Containment Building
Concrete Internal Structure
Auxiliary Building
Turbine Building
Intake Structure
Refueling Water and Condensate Storage Tanks
Diesel Generator Fuel Oil Storage Tank (Buried)
Auxiliary Saltwater System Piping (Buried)

Table 6-3

MAJOR REACTOR PLANT SYSTEMS

Nuclear Steam Supply System (NSSS)	Containment Building Ventilation System
Residual Heat Removal System	Control Room Ventilation System
Safety Injection System	Vital Electrical Room Ventilation System
Component Cooling Water System	4160 V (Vital) Electrical System
Chemical and Volume Control System	480 V (Vital) Electrical System
Auxiliary Saltwater System	125 V DC Electrical System
Containment Spray System	120 V AC Electrical System
Main Steam System	Operator Instrumentation and Control System
Auxiliary Feedwater System	NSSS Instrumentation and Control System
Diesel Generator and Auxiliaries	Off-Site Power System

Table 6-4

TYPICAL GENERIC COMPONENT CATEGORIES

Electrical Penetrations
Balance-of-Plant Piping and Supports
Air- and Motor-Operated Valves
Cable Tray, Conduits, and Supports
HVAC Ducting and Supports

Piping, electrical, mechanical, and electro-mechanical equipment vital to safe shutdown of the Plant or mitigation of an accident were considered to fail when it was judged they were no longer able to perform their designated functions. Therefore, for mechanical equipment, the fragility definition represents failure to function, loss of anchorage, or rupture of the pressure boundary. For electrical equipment, the fragility represents loss of function due to acceleration-sensitive failure (for example, relay chatter) or loss of function due to structural failure of the cabinet, anchorage, or internals. For ductile systems such as piping, HVAC ducting, and electrical conduits, fragility represents crimping, choking of flow, or rupture due to failure of the supports, as it has been shown that failure of these systems is virtually impossible apart from failure of the supports.

Fragility of a structure or a component is defined as the conditional frequency of its failure for a given value of the ground-motion parameter (for example, spectral acceleration). Thus, the fragility evaluation is based on the estimation of the median ground spectral acceleration value for which the seismic response of a given structure or component exceeds its capacity, resulting in failure. Because there are many sources of variability in the estimation of the median ground spectral acceleration capacity, the component fragility is described by means of a family of fragility curves. Figure 6–10 depicts such curves, showing the best estimate (50 percent confidence, $C_2 = 0$) curve with its shape governed by randomness variability, (β_R), and showing the relative position of the curve for other confidence levels greater than or less than 50 percent. The properties of the fragility curves and the general approach to their development are defined in previous works (Kennedy, 1980; Kennedy, 1984). Employing the characteristics of the lognormal distribution as described in these references, the entire family of fragility curves for any mode of failure is defined in terms of a median estimate of the ground spectral acceleration capacity, $\check{\overline{S}}_a$ (Figure 6–10), times the product of randomness and uncertainty variables, ϵ_R and ϵ_U, which have unit median values and are lognormally distributed with logarithmic standard deviations of β_R and β_U, respectively.

$$\overline{S}_a = \check{\overline{S}}_a \epsilon_R \epsilon_U \qquad (6\text{–}3)$$

The spectral ground acceleration capacity, $\overline{S}_a$, is computed as:

$$\overline{S}_a = F \cdot \overline{S}_{aRef} \qquad (6\text{–}4)$$

where F equals the overall factor of safety based on response to the reference earthquake, and $\overline{S}_{aRef}$ equals the average spectral ground acceleration of the reference earthquake. The overall factor of safety has a median value, $\check{F}$, and randomness and uncertainty variabilities (β_R and β_U). In contrast, the average reference spectral acceleration is a deterministic quantity determined over a specified frequency range of the reference ground spectrum. Thus, the product of these terms, shown in equation (6–4), results in a spectral acceleration capacity which has a median value, $\check{\overline{S}}_a$, and randomness and uncertainty variabilities which are equal to the corresponding variabilities associated with the overall factor of safety (Figure 6–10). As a result, the spectral acceleration capacity at any point within the family of fragility curves is computed as:

$$\overline{S}_a = \check{\overline{S}}_a \, e^{(C_1\beta_R + C_2\beta_U)} \qquad (6\text{–}5)$$

where C_1 and C_2 are the statistical constants associated with the failure fraction and confidence level of interest (Figure 6-10).

It must be noted then, that the term $\overline{S}_a$ as used in this chapter refers to an average spectral acceleration capacity defined over the same frequency range as $\overline{S}_{aRef}$. This is in contrast to the normal usage of the term S_a, which refers to a spectral acceleration at a specific frequency.

The Diablo Canyon site-specific median horizontal and vertical ground spectra were established as part of the ground-motion studies documented in Chapter 4. These are shown on Figure 6-11 and define the median spectral shape

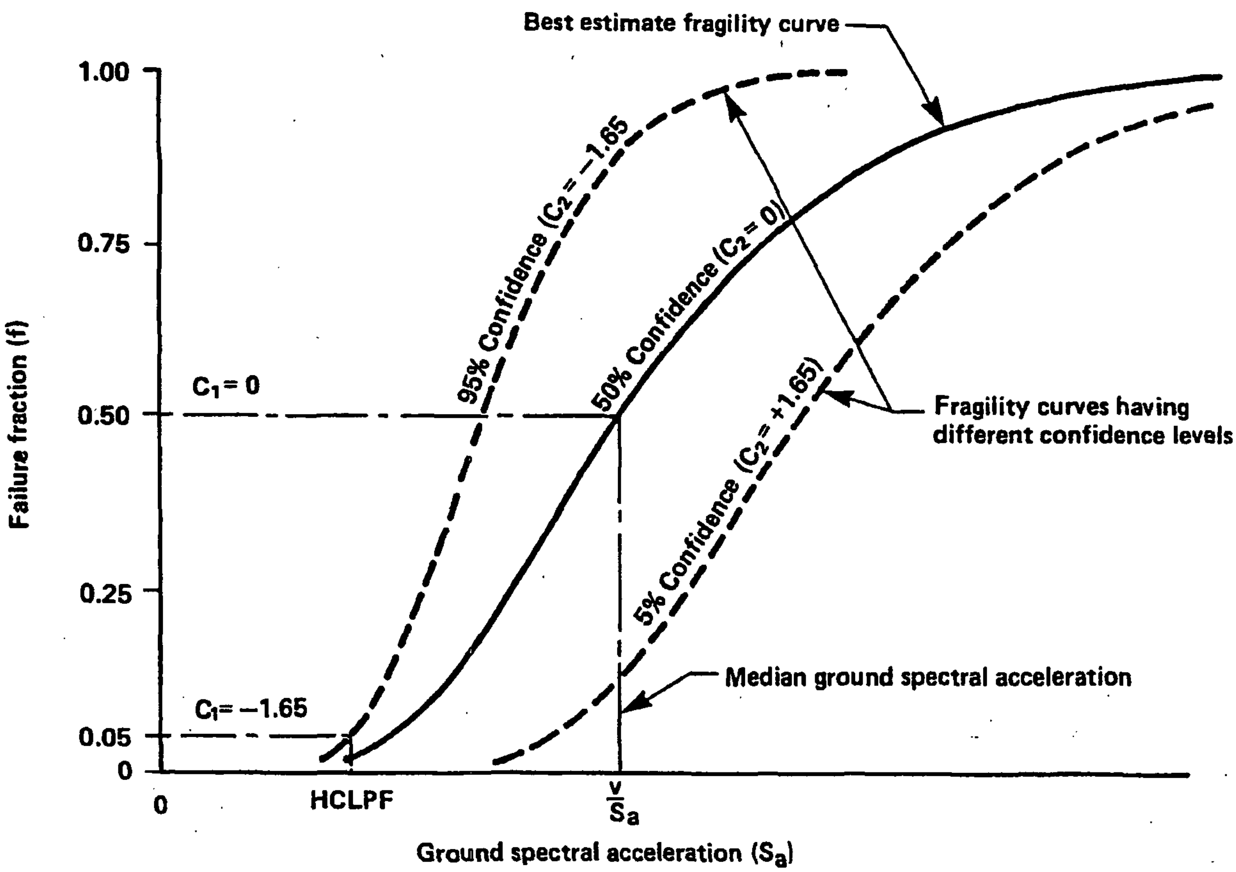


Figure 6-10
Fragility curve representations.

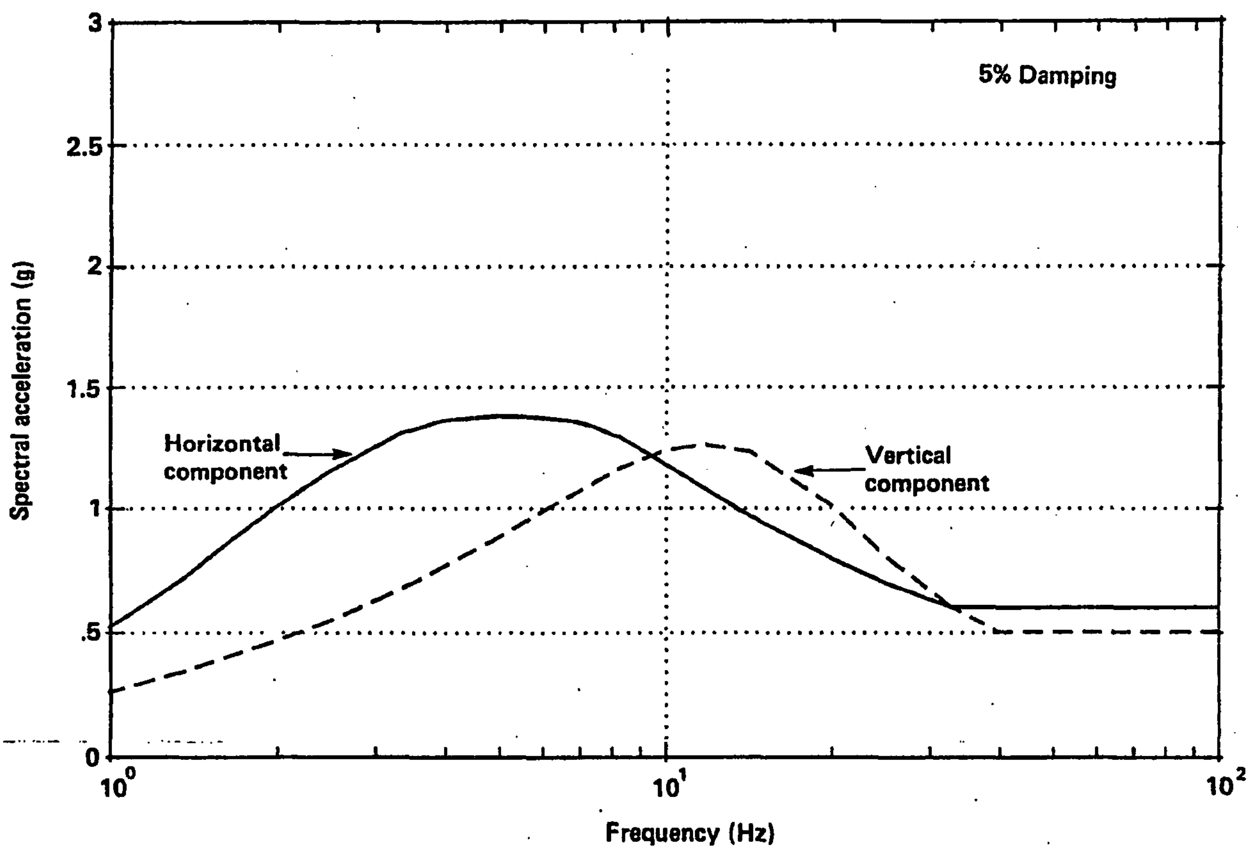


Figure 6-11
Diablo Canyon site-specific median ground-motion acceleration response spectra.

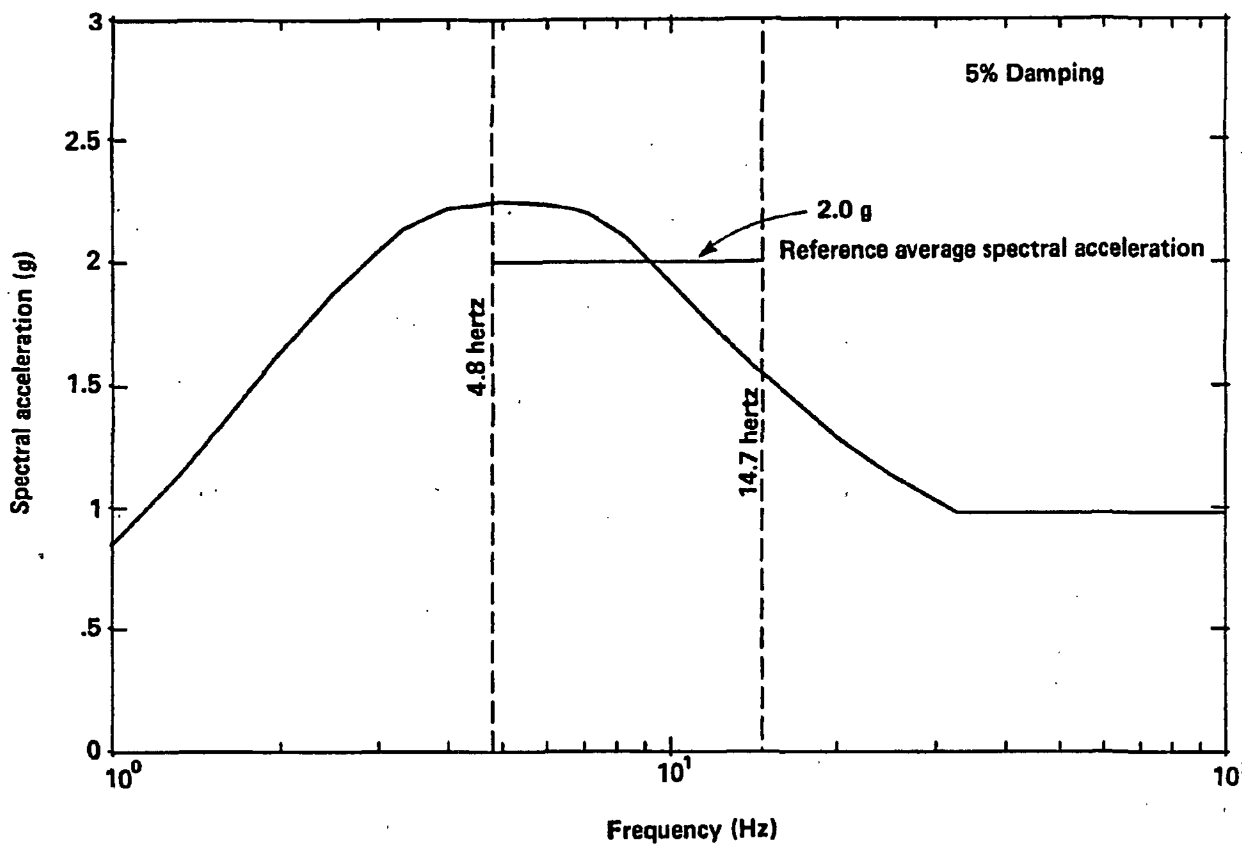


Figure 6-12
Reference horizontal ground-motion response spectrum.

and relative amplitude between the horizontal and vertical components on a frequency-by-frequency basis. For use in the fragility evaluation of the Diablo Canyon structures and equipment, the reference horizontal ground spectrum (Figure 6-12) was established by scaling the median horizontal ground spectrum such that the average spectral acceleration, $\overline{S}_{a_{Ref}}$, over the frequency range between 4.8 and 14.7 hertz was several key variables together with the randomness and uncertainty variability associated with each. The key factors involved are listed below and were appropriately applied for structures and/or components.

Because seismically induced fragility data are generally unavailable for most Plant components and all structures, fragility curves were developed primarily from design analysis data, equipment qualification test data, and engineering judgment. The overall median factor of safety, $\check{F}$, based on these data sources, was established by considering

1) The Strength factor, F_S, comparing the median strength available to resist seismic motion (or strength at loss of function) to the response level due to either the reference seismic event or the design seismic event. Where possible, based upon the form of the available seismic qualification data, the Strength factor was based upon a revised calculation of the critical response using the reference spectra, median-centered property values, and median-centered combination methods. For such cases the response factors discussed below were unity and only the associated variabilities were evaluated. This was done to minimize the uncertainty variabilities associated with the various response parameters. Where the form of the available data did not permit recomputation of the median-centered response, the responses from the design event (usually Hosgri reevaluation data) were used to evaluate the Strength factor of safety. For such cases, the response factors were evaluated as necessary.

2) The Inelastic Energy Absorption factor, F_μ (ductility), accounting for the fact that an earthquake represents a limited energy source and many structures and components are capable of absorbing substantial amounts of energy beyond yield without loss of function.

3) The Qualification Method factor, F_{QM}, comparing the acceleration values used in the equipment design analysis (when F_S is based on the design seismic event) to those obtained from the reference floor response spectrum.

4) The Damping factor, F_D, comparing response accelerations from the reference floor spectra at structure or equipment design damping to that associated with the damping level expected at or near failure.

5) The Modeling factor, F_M, assessing the ability of the design mathematical model to accurately determine the fundamental frequencies and mode shapes of the structures or equipment modeled; for tested components, assessing the similarity of the dynamic test boundary conditions to the in-Plant anchorage.

6) The Mode Combination factor, F_{MC}, assessing the conservatism or unconservatism in the mode combination method used in the design process; for components qualified by test, assessing the ability of the test method to simultaneously excite all dynamic modes.

7) The Earthquake Component Combination factor, F_{ECC}, evaluating the conservatism or unconservatism in the method used to combine the responses from the various earthquake component directions during the design analysis; for tested equipment, evaluating the unconservatism in the use of uniaxial or biaxial tests to duplicate actual earthquake response.

8) The Spectral Shape factor, F_{SS}, evaluating the randomness and uncertainty associated with peaks and valleys in the reference ground spectra.

9) The Ground Motion Incoherency factor, F_{GMI}, evaluating the conservatism in assuming coherent ground motion in establishing the reference floor spectra.

10) The Inelastic Structural Response factor, F_{IR}, evaluating the potential for increased high frequency floor acceleration response due to nonlinear structural behavior. This factor is applicable to equipment fragility evaluation only.

The median overall factor of safety and its variabilities are computed as:

$$\check{F} = \check{F}_S \cdot \check{F}_\mu \cdot \check{F}_{QM} \cdot \check{F}_D \cdot \check{F}_M \cdot \check{F}_{MC} \cdot \check{F}_{ECC} \cdot \check{F}_{SS} \cdot \check{F}_{GMI} \cdot \check{F}_{IR} \qquad (6\text{-}6)$$

$$\beta_R = \left[\beta^2_{R_S} + \beta^2_{R_\mu} + \beta^2_{R_{QM}} ----\cdot + \beta^2_{R_{IR}} \right]^{1/2}$$

$$\beta_U = \left[\beta^2_{U_S} + \beta^2_{U_\mu} + \beta^2_{U_{QM}} ----\cdot + \beta^2_{U_{IR}} \right]^{1/2}$$

Although the Diablo Canyon fragility evaluation of safety-related structures and equipment essentially followed the basic approach used for previous seismic probabilistic risk assessments of nuclear power plants, substantial work went into the more rigorous determination of certain factors and their variabilities. This was accomplished with the intent of minimizing the variabilities associated with the various parameters. A discussion of the important differences between previous fragility estimation efforts and the approaches used for the Diablo Canyon evaluation is included in the following sections.

Reference Ground-Motion Parameter

The fragilities for all Diablo Canyon structures and equipment, except the turbine building, were estimated as a function of the 5 percent damped average spectral acceleration of the horizontal ground-motion components averaged over the frequency range of 4.8 to 14.7 hertz. Most previous seismic probabilistic risk assessments of nuclear plants have defined fragilities as a function of the peak ground acceleration. However, damage to structures and equipment is more a function of the spectral accelerations within the elastic and inelastic frequency ranges of the structure and equipment than it is a function of the peak ground acceleration. For nearly all of the structures (except the turbine building) and the equipment, the frequency range of primary interest was from about 3.5 hertz to about 35 hertz. From the 38 sets of time histories defined in Chapter 4 and used for the fragility evaluations, it was found that the ratio of spectral acceleration, at any specific frequency of interest in the 3.5 hertz to 35 hertz frequency range, to the average spectral acceleration over the 4.8 to 14.7 hertz range, showed lesser and more consistent variability than did the ratio of spectral acceleration at any specific frequency to peak ground acceleration. Over the entire frequency range of 3.5 hertz to 35 hertz, the ratio of spectral acceleration at any specific frequency to the average spectral acceleration over 4.8 to 14.7 hertz had a nearly constant logarithmic standard deviation that averaged about $\beta_R = 0.18$. However, the ratio of spectral acceleration at a specific frequency to the peak ground acceleration was highly variable over this important frequency range. The logarithmic standard deviation of this ratio ranged from close to zero at 35 hertz to more than 0.25 below 5 hertz. The frequency-dependent nature of spectral peak-and-valley or spectral shape variability is difficult to accommodate in the fragility analysis of a large number of components and equipment so that seismic fragility estimates anchored to peak ground acceleration have tended to use a conservative, frequency-independent spectral shape randomness variability of 0.25 or greater. Anchoring the fragility estimates to the average spectral acceleration from 4.8 to 14.7 hertz eliminates this difficulty and has enabled the use of a lesser β_R for peak-and-valley or spectral shape variability for frequencies equal or greater than about 3.5 hertz.

As will be noted later, the turbine building fragility was initially estimated to be sensitive to spectral accelerations in the 3 to 8.5 hertz frequency range, and was later found to be sensitive to spectral accelerations in the 1.7 to 9.5 hertz frequency range. To enable a better incorporation of spectral shape variability within the frequency range of interest for the turbine building, its fragility estimate was developed as a function of the average spectral acceleration in the 3 to 8.5 hertz range.

When convolving the seismic hazard and fragilities together in the seismic probabilistic risk assessment, it is desirable for all fragilities and the seismic hazard to be expressed in terms of one common ground-motion parameter. Because the median and 84 percent nonexceedance probability site-specific spectra, the probabilistic seismic hazard spectra with the annual probability range of interest, and the median horizontal spectrum shape used in the fragility evaluations all showed essentially the same ratio of average spectral acceleration in the 3 to 8.5 hertz range to average spectral acceleration in the 4.8 to 14.7 hertz frequency range, it was immaterial which average spectral acceleration frequency range was used for the common ground-motion parameter. The ratio between these average 5 percent damped spectral accelerations was:

$$\frac{\overline{S}_{a\,3-8.5}}{\overline{S}_{a\,4.8-14.7}} = 1.125$$

Because 3 to 8.5 hertz is the frequency range over which spectral accelerations are maximum, it was judged to be most descriptive to define the average spectral acceleration over the 3 to 8.5 hertz range as the common ground motion parameter for convolving hazard and fragility estimates. All fragility median and high-confidence-of-low-probability-of-failure estimates included in this report were converted so as to be defined in terms of the average spectral acceleration in the 3 to 8.5 hertz frequency range using the above defined conversion ratio.

Median Horizontal Floor Spectra

In many previous probabilistic risk assessments, the factors for equipment capacities and equipment responses were based upon the floor spectra used during the Plant design phases. Various factors were then generated in an attempt to account for conservatism or unconservatism in the generation of the design floor spectra due to differences between the design and median site-specific ground spectra, effects of soil/structure interaction, and differences between design structural damping and structural damping expected at or near failure. In contrast, as discussed in Chapter 5, reference median horizontal floor spectra were generated for selected elevations of the Diablo Canyon safety-related structures corresponding to the location of important safety-related equipment. These floor spectra were generated using the reference ground motion, together with median soil/structure interaction and building structural parameters. Thus, the Strength factor of safety, F_S, is generally based upon the reference median horizontal floor spectra, together with a clear understanding of the associated variabilities.

Relationship Between Horizontal and Vertical Ground Spectra

The vertical ground spectrum used in the design of most nuclear plants is usually based upon some specified factor (for example, 2/3 or 1) times the design horizontal spectrum evaluated on a frequency-by-frequency basis. For the probabilistic risk assessments for such plants, the potential for higher than the designed-for vertical to horizontal ground-motion ratio is either ignored or included as a randomness variability based upon the vertical direction contribution to the response of interest. The Diablo Canyon site-specific horizontal and vertical 5 percent damped median ground spectra are shown on Figure 6-11. As discussed earlier, the reference horizontal ground spectrum for use in the fragility evaluations was established by scaling the median horizontal spectrum such that the average spectral acceleration over the 4.8 to 14.7 hertz range was equal to 2.0 g (Figure 6-12). This same scale factor was applied to the median vertical ground response spectrum to establish a reference vertical ground response spectrum that properly corresponded to the reference horizontal spectrum. The resulting 5 percent damped reference vertical ground response spectrum is depicted on Figure 6-13 and is shown in comparison with the Hosgri reevaluation vertical ground spectrum.

From Figure 6-11 it can be seen that the vertical ground acceleration exceeds the horizontal acceleration over the frequency range of about 9.5 to 30 hertz. In addition, Figure 6-13 shows that the reference vertical ground spectrum exceeds the Hosgri reevaluation vertical spectrum for frequencies greater than about 4.5 hertz. The effects of this reference vertical spectrum were included in the evaluation of equipment fragilities.

Equipment fragilities are mostly dominated by horizontal reponses. As discussed above, reference median horizontal floor spectra were developed for the safety-related structures (Chapter 5). Reference median vertical floor spectra were not similarly generated. Reference vertical floor spectra were developed by scaling the Hosgri reevaluation vertical spectral acceleration at the floor by the ratio of the reference vertical ground spectrum to the Hosgri reevaluation vertical ground spectrum (Figure 6-13). Since the vertical direction contribution to seismic fragilities of components is generally small, this approach for the generation of reference vertical floor spectra was considered adequate.

Structural Response Variability

In most previous seismic probabilistic risk assessments of nuclear power plants, the evaluation of the Structural Response factor used in developing fragility descriptions for structures and equipment has employed simplified methods using the separation-of-variables approach. Because of the significant variabilities associated with each of the factors that would make up the Structural Response factor and the uncertainties associated with the simplified approach (how the individual variabilities combine), a more rigorous approach was undertaken to establish structural response variability, as part of the Diablo Canyon Long Term Seismic Program.

The Structural Response factor is a measure of the conservatism introduced in the development of the reference in-structure floor response spectra. The important variables used in the development of equipment fragilities, which affect the generation of in-structure floor spectra include:

1) Ground-motion spectral shape
2) Structural damping
3) Structural frequency
4) Structural mode combination
5) Earthquake directional combination
6) Soil/structure interaction
7) Structural mode shape
8) Ground-motion incoherency
9) Inelastic structural response

The first six variables, which constitute the majority of the randomness and uncertainty variability, were included in the structural response variability study described herein; the last three variables were added to the structure and component fragility analyses based on the normal separation-of-variables approach.

The variables associated with ground-motion spectral shape (peaks and valleys), structural mode combination, and earthquake directional combination were represented in the variability study using a large suite of 38 sets of two orthogonal horizontal components of earthquake time histories that provided a broad characterization of the ground motions which might occur at the Plant site in the event of a very large earthquake. The 38 sets of earthquake time histories used in the variability study consisted of a set of 24 empirical earthquake time histories and 14 numerically simulated acceleration-time records. The variables associated with structural damping, structural frequency, and rock modulus are model parameters that characterize the behavior of the soil/structure system under a given ground motion. These parameters were represented by employing a random selection procedure (Latin Hypercube simulation) to select model parameter values which were then randomly mixed for use with the suite of earthquake time history input ground motions.

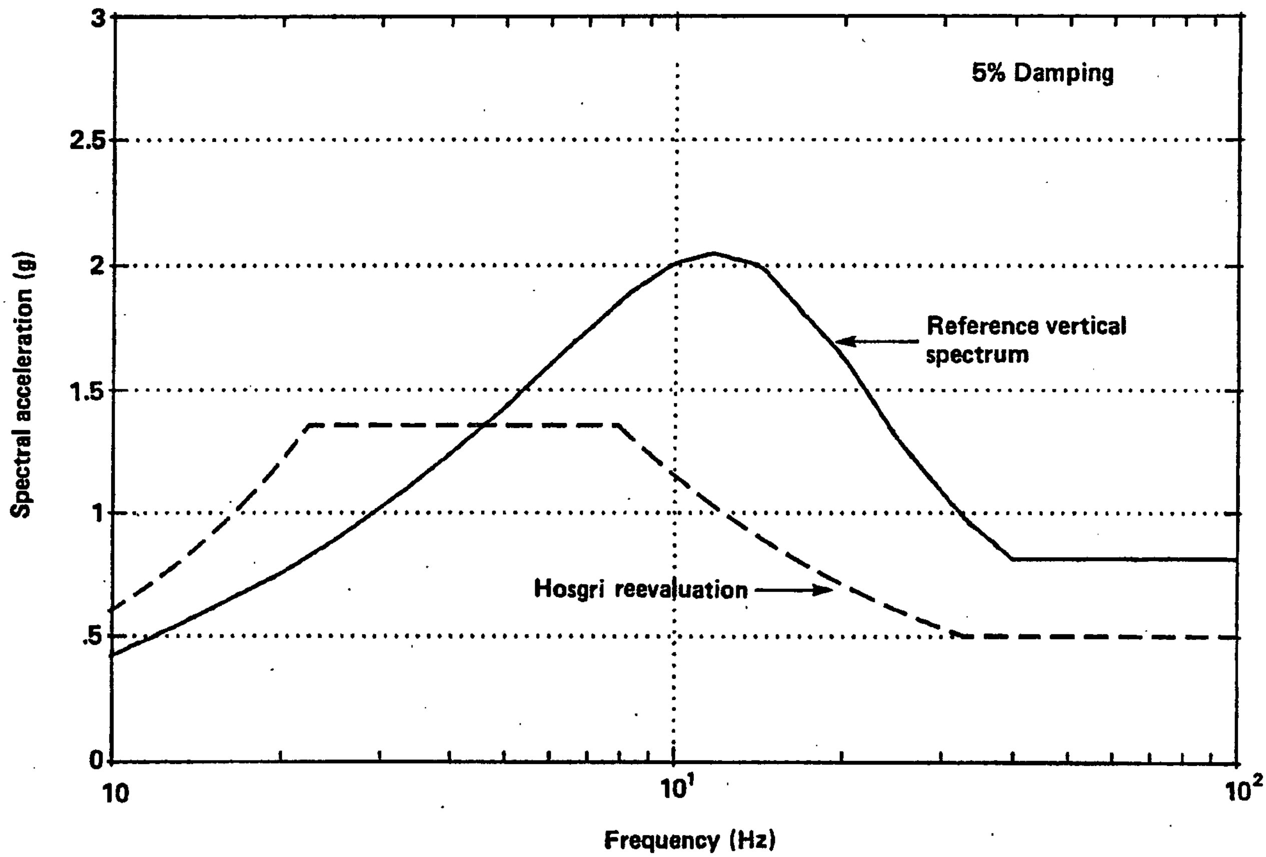


Figure 6-13

Diablo Canyon reference vertical ground spectrum.

Each set of randomly selected and mixed model parameter values and its associated north/south and east/west earthquake time histories were input into a simplified soil/structure interaction system model of the auxiliary building, which was analyzed using the CLASSI computer code to generate 38 sets of deterministic floor response spectra at various elevations. The floor response spectra from the 38 earthquake runs were then statistically analyzed to generate median and 84th percentile probabilistic floor spectra. At any frequency, the combined variability, β_c, associated with the six variables included in the study was estimated from the ratio of the 50th and 84th percentile spectral accelerations

ANALYSIS MODEL

As noted above, a simplified soil/structure interaction model of the auxiliary building was used in this study. This structure was chosen because it is a large structure that houses a substantial portion of the important Plant equipment. Emphasis in this study was placed upon assessing the response variability of the western core of the auxiliary building because a majority of the Plant safety-related equipment is located in the western core.

As discussed in Chapter 5, detailed soil/structure interaction analyses of the auxiliary building were conducted using the SASSI computer code based upon a three-dimensional, 5-stick representation of the structure above El 85 feet and a three-dimensional finite element plate representation below El 85 feet (Figure 6-14, SASSI model). A large number of time-history soil/structure interaction response analyses and varying model parameters were required in the structural response variability study; thus, it was desired that the model be simple and easily amendable to model parameter adjustment. The SASSI 5-stick model was considered too detailed for the structural response variability study, and as a result, a simplified CLASSI model was developed (Figure 6-15). The transformation to the simplified 3-stick model of the auxiliary building superstructure was accomplished by deleting the stick representation of the north and south wings. The north/south stiffnesses of the deleted wings were accounted for by adjusting the north/south stiffness of the core east stick. Comparison of fixed-base modal properties between the two models, both fixed at El 85 feet, showed close agreement at the lower modes. The embedded portion of the auxiliary building, the foundation, basement, and the underlying rock medium, were represented by equivalent foundation base mass, mass moments of inertia, and impedances in the simplified model. The frequency-dependent foundation impedances associated with the rigid rectangular base were calculated using the CLASSI code, based on the same rock profile and properties used for the soil/structure interaction study (Chapter 5). The frequency-dependent soil spring stiffnesses and damping coefficients were taken as the CLASSI calculated impedance functions at about 8 hertz, which closely corresponds to the fundamental north/south and east/west frequencies from the soil/structure interaction model. These parameters were then adjusted for the embedment effect of the core structure. The simplified soil/structure interaction model was formed by coupling the 3-stick core structure model, the foundation base mass properties, and the soil spring and damping coefficients into the soil/structure interaction system.

To validate the simplified CLASSI soil/structure interaction model, two response parameters were compared with results from the more detailed SASSI model. The comparison of the north/south and east/west horizontal seismic response transfer functions at El 140 feet is shown on Figure 6-16 and the comparison of the 2 percent damped north/south and east/west floor response spectra at El 140 feet (core west) for the same free-field ground-motion time-history input is shown on Figure 6-17. Both show very good agreement.

The effect of conrete cracking on structural response was considered by adjusting the frequencies of the fixed base model by a factor of 0.9 (stiffness reduction of approximately 0.8).

The median fundamental frequencies of the simplified soil/structure interaction model, for both the north/south and east/west directions, were approximately 8 hertz taking concrete cracking into account.

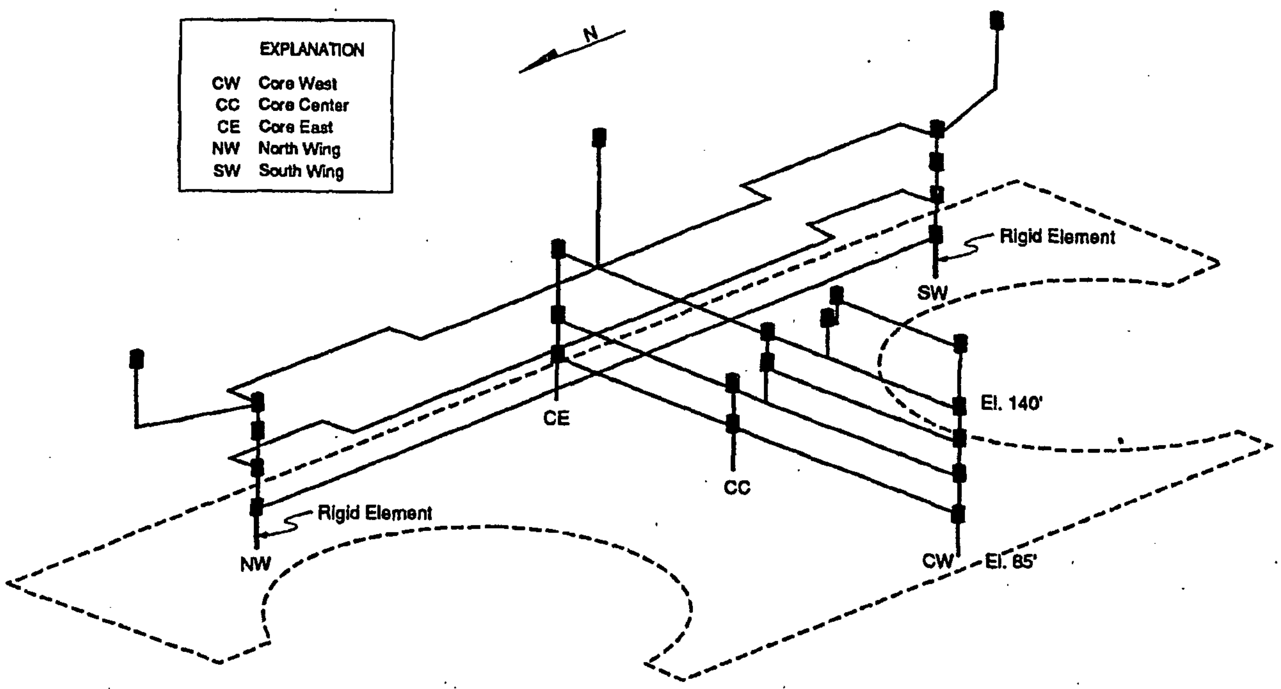


Figure 6-14
SASSI structural model for the auxiliary building.

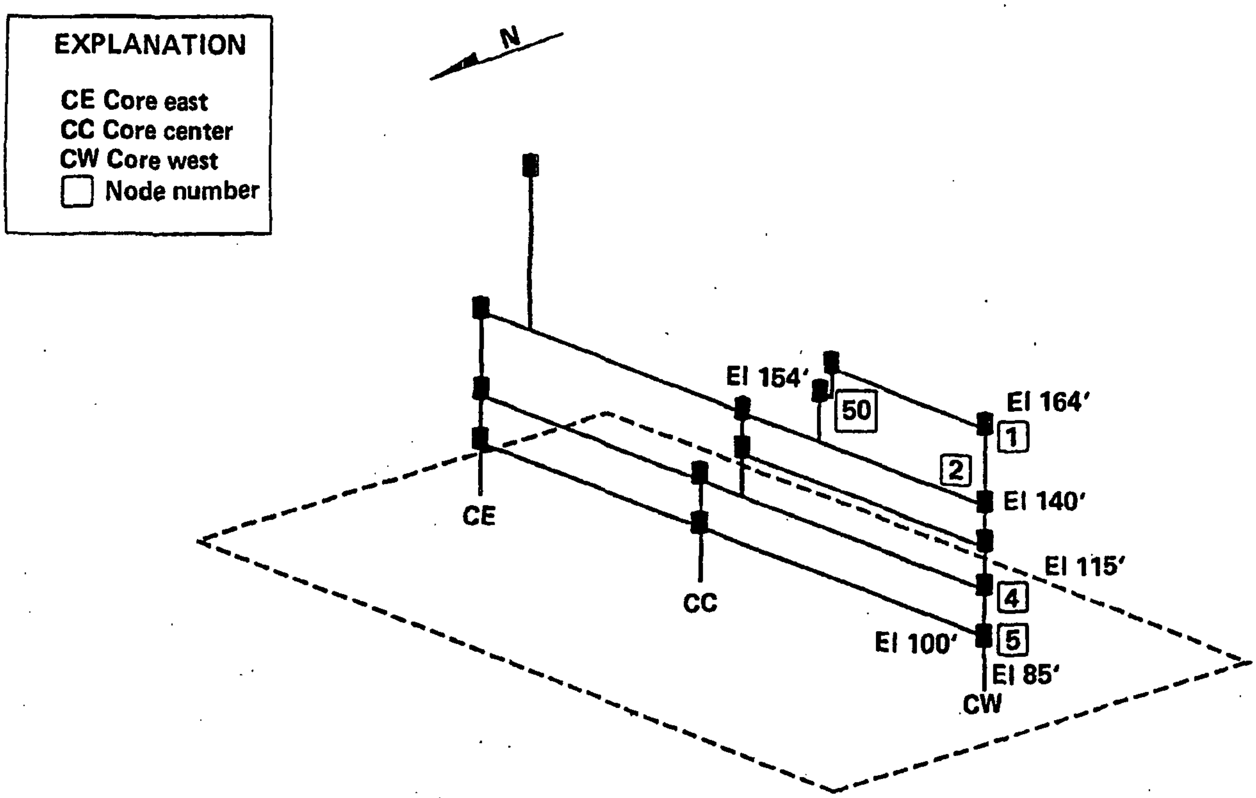


Figure 6-15
Configuration of the simplified CLASSI model for the auxiliary building core structure.

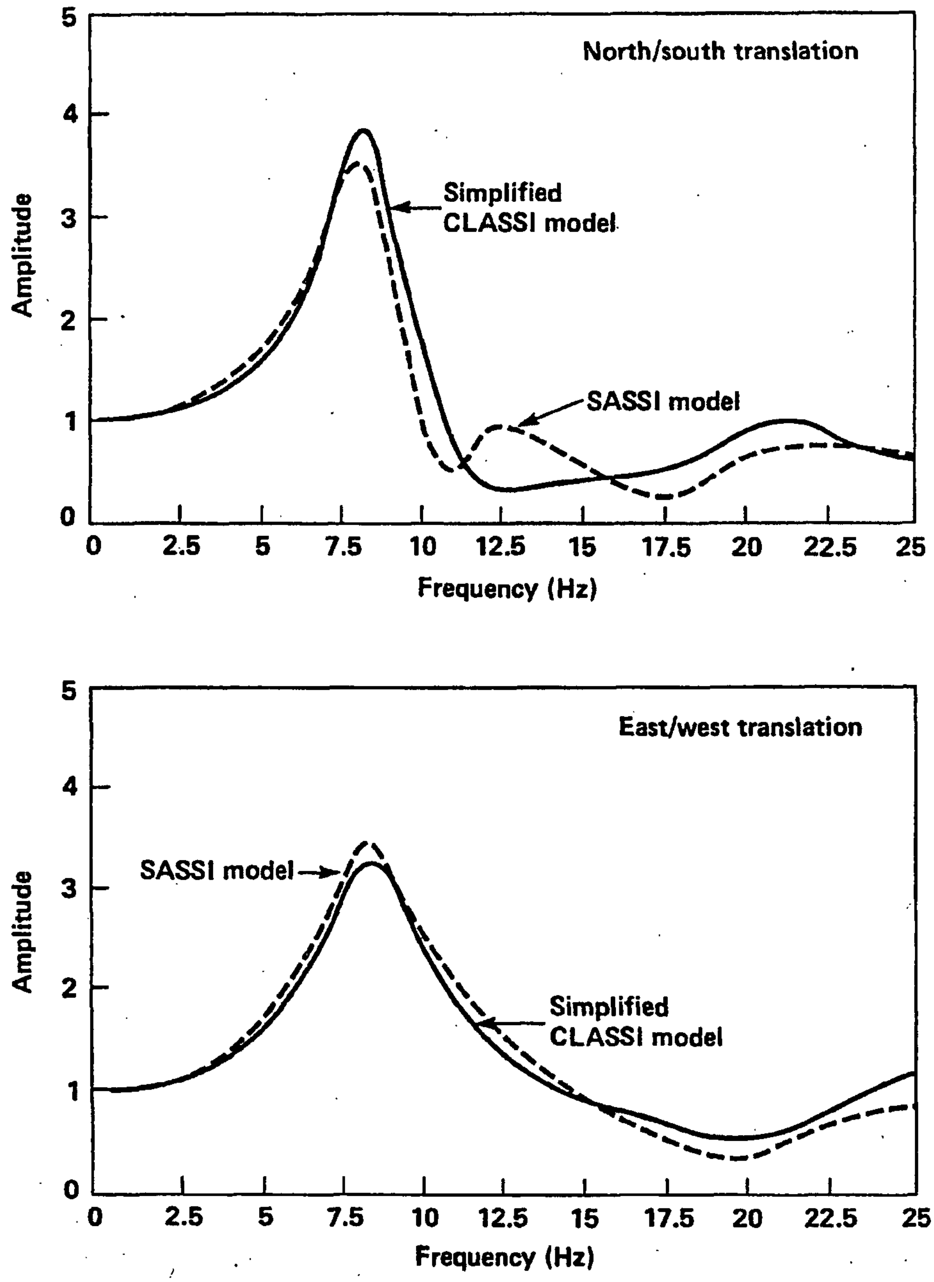
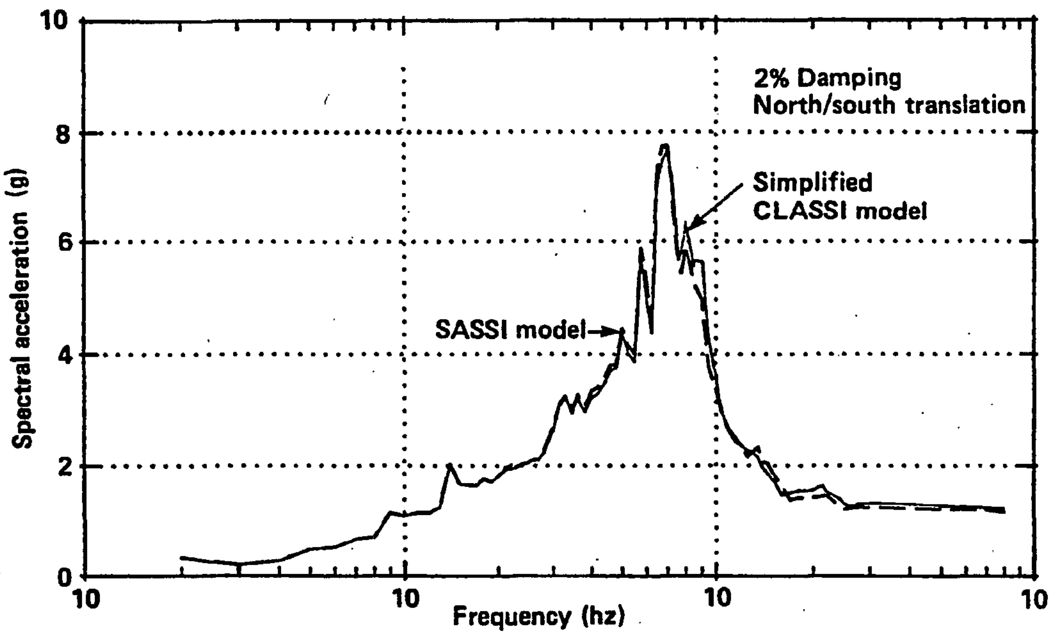


Figure 6-16

Comparison of transfer functions from the simplified CLASSI soil/structure interaction model with those from the SASSI model at El 140 feet (core west).



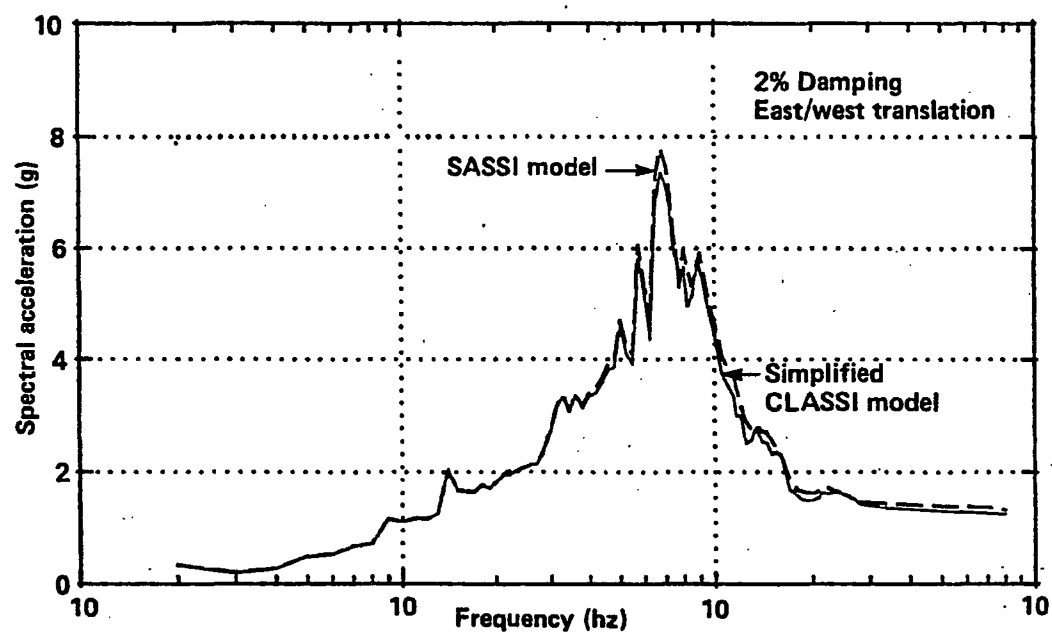


Figure 6-17

Comparison of 2 percent damped response spectra from the simplified CLASSI soil/structure interaction model with those from the SASSI model at El 140 feet (core west).

INPUT MOTION

Acceleration time histories used in the structural response variability study were developed to represent ground motions that might be expected at a rock site within 10 kilometers of the fault rupture surface due to shallow crustal earthquakes having magnitudes in the range of 6.5 to 7.5 and having strike-slip, oblique, or reverse faulting mechanisms.

A total of 52 horizontal ground–motion time–history records were used in this study. Twelve pairs of orthogonal empirical time histories derived from actual recordings of eight past earthquakes were selected and are shown in Table 6-5. Because directions of ground motions for the suite of empirical time histories are random with respect to the north/south and east/west directions of the Plant, the two components of each of the 12 empirical records were interchanged to produce 24 empirical earthquake time-history sets. To provide a more balanced representation of potential fault mechanisms at the site, and to increase the size of the suite of time histories for a better overall distribution, 14 pairs of orthogonal numerically simulated time histories were also generated. Because the numerical set of earthquake histories (Table 6-6) were specifically generated to correspond to the Plant north/south and east/west directions respectively, they were applied in accordance with their specified directions.

As discussed earlier, average spectral acceleration over a broad frequency range is a substantially better descriptor of damage than is peak ground acceleration. To maintain an approximately uniform variability over the entire frequency range of interest in the earthquake ground motion, each time–history pair was scaled such that the average 5 percent damped spectral acceleration over the frequency range of 4.8 to 14.7 hertz was 2.0 g for the average of the two horizontal components. This scaling method is identical to that used in the detailed soil/structure interaction analyses described in Chapter 5. The scaling factors used are shown in Table 6–5 and Table 6-6. The frequency range of 4.8 to 14.7 hertz covers approximately the median auxiliary building soil/structure system frequency (about 8 hertz) plus or minus two logarithmic standard deviations. Tables 6-5 and 6-6 present details of each of the empirical and numerical time-history records, respectively, including fault type, scaling factor to achieve an average spectral acceleration of 2.0 g, and the nature of adjustments to the empirical records necessary so that they would be appropriate for the Diablo Canyon site.

VARIABLE MODEL PARAMETERS

Variability in structural response due to variation in structural damping, structural frequency, and rock modulus were included in the auxiliary building variability study. A Latin Hypercube simulation was used to select the random variables (model parameter values) used in the analysis. Since the earthquake time histories selected were assumed to be equally likely, the sample size was set equal to the number of earthquake records provided. The damping ratios, frequencies, and rock modulus values were assumed to be lognormally distributed with medians and variabilities as shown in Table 6–7. Two sets of model parameter samples were created: one for the set of 24 empirical earthquakes, and one for the set of 14 numerically simulated earthquakes.

Table 6-7

MEDIANS AND VARIABILITIES FOR MODEL PARAMETERS

Parameter	Median	β
Structure Frequency Ratio	1.0	0.25
Structure Damping	0.07	0.35
Rock Modulus Ratio	1.00	0.45

The domain of each model parameter was divided into N + 2 strata (where N is equal to the number of sample points to be selected, that is, the number of earthquakes) such that each of the strata is of equal probability (Figure 6–18). Parameter values within the first and (N + 2)th strata (that is, the tails of the probability distribution function) were considered to be extreme, unrealistic values; thus sampling was

Table 6-5

EARTHQUAKE RECORDS USED TO DEVELOP TIME HISTORIES FOR FRAGILITY STUDIES

Time History Number	Earthquake	Recording Station	Record Name	Magnitude Used	Distance (km)	Style of Faulting	Adjustment	Scaling Factor[1]
1 2	1978 Tabas	Tabas	Tabas N74E Tabas N16W	7.4	3	Thrust	None	0.98
3 4	1971 San Fernando	Pacoima Dam	SFPAC S16E SFPAC S74W	6.6	3	Thrust	None	1.12
5 6	1971 San Fernando	Lake Huges No. 12	SFLH12 N21E SFLH12 N69W	6.6	20	Thrust	Distance	1.07
7 8	1971 San Fernando	Castaic	CAS N69W CAS N21E	6.6	25	Thrust	Distance	1.25
9 10	1979 Imperial Valley	Differential Array	IVDA N00E IVDA N90W	6.5	5	Strike-slip	Site response	1.46
11 12	1979 Imperial Valley	El Centro No. 4	IVEC S50W IVEC S40E	6.5	4	Strike-slip	Site response	1.80
13 14	1984 Morgan Hill	Coyote Lake Dam	CLD N75W CLD S15W	6.2	0.1	Strike-slip	Magnitude	1.21
15 16	1983 Coalinga	Pleasant Valley Pump Station (Switchyard)	PVPP 045 PVPP 135	6.5	10	Reverse	Distance	1.31
17 18	1985 Nahanni	Site 1	NAH1 010 NAH1 280	6.8	6	Thrust	None	0.84
19 20	1976 Gazli	Karakyr Point	Gazli EAS Gazli NOR	6.8	3	Reverse	None	1.24
21 22	1966 Parkfield	Temblor	TEM N65W TEM S25W	6.1	10	Strike-slip	Distance and magnitude	2.13
23 24	1978 Tabas	Dayhook	Dayhook N10E Dayhook N80W	7.4	17	Thrust	Distance	1.45

[1]This scaling factor was used to bring the empirical records to an average 5 percent damped spectral acceleration of 2.0 g in the 4.8 to 14.7 hertz range and is in addition to the scaling necessary to make the records appropriate for the Diablo Canyon site (Chapter 4).

Table 6-6

FAULT MODELS USED TO GENERATE SIMULATED TIME HISTORIES FOR FRAGILITY STUDIES

Time History Number	Record Name	Style of Faulting	Rupture Mode	Source Functions	Scaling Factor
25	FILE1–C2E	Strike-slip	Bilateral	Coalinga aftershock	1.38
26	FILE1–C2N				
27	FILE1–I3N	Strike-slip	Bilateral	Imperial Valley aftershock	2.06
28	FILE1–I3E				
29	FILE2–I9N	Strike-slip	Unilateral-N	Imperial Valley aftershock	2.53
30	FILE2–I9E				
31	FILE3–C6N	Strike-slip	Unilateral-S	Coalinga aftershock	1.68
32	FILE3–C6E				
33	FILE3–I6N	Strike-slip	Unilateral-S	Imperial Valley aftershock	2.33
34	FILE3–I6E				
35	FILE4–C4N	Oblique	Bilateral	Coalinga aftershock	1.09
36	FILE4–C4E				
37	FILE4–C5N	Oblique	Bilateral	Coalinga aftershock	1.33
38	FILE4–C5E				
39	FILE4–I7N	Oblique	Bilateral	Imperial Valley aftershock	2.63
40	FILE4–I7E				
41	FILE5–C5N	Oblique	Unilateral-N	Coalinga aftershock	1.39
42	FILE5–C5E				
43	FILE5–I6N	Oblique	Unilateral-N	Imperial Valley aftershock	2.25
44	FILE5–I6E				
45	FILE6–C4N	Oblique	Unilateral-S	Coalinga aftershock	1.12
46	FILE6–C4E				
47	FILE6–I1N	Oblique	Unilateral-S	Imperial Valley aftershock	1.96
48	FILE6–I1E				
49	FILE7–C1N	Thrust	Bilateral	Coalinga aftershock	1.23
50	FILE7–C1E				
51	FILE8–C2N	Thrust	Unilateral-N	Coalinga aftershock	1.05
52	FILE8–C2E				

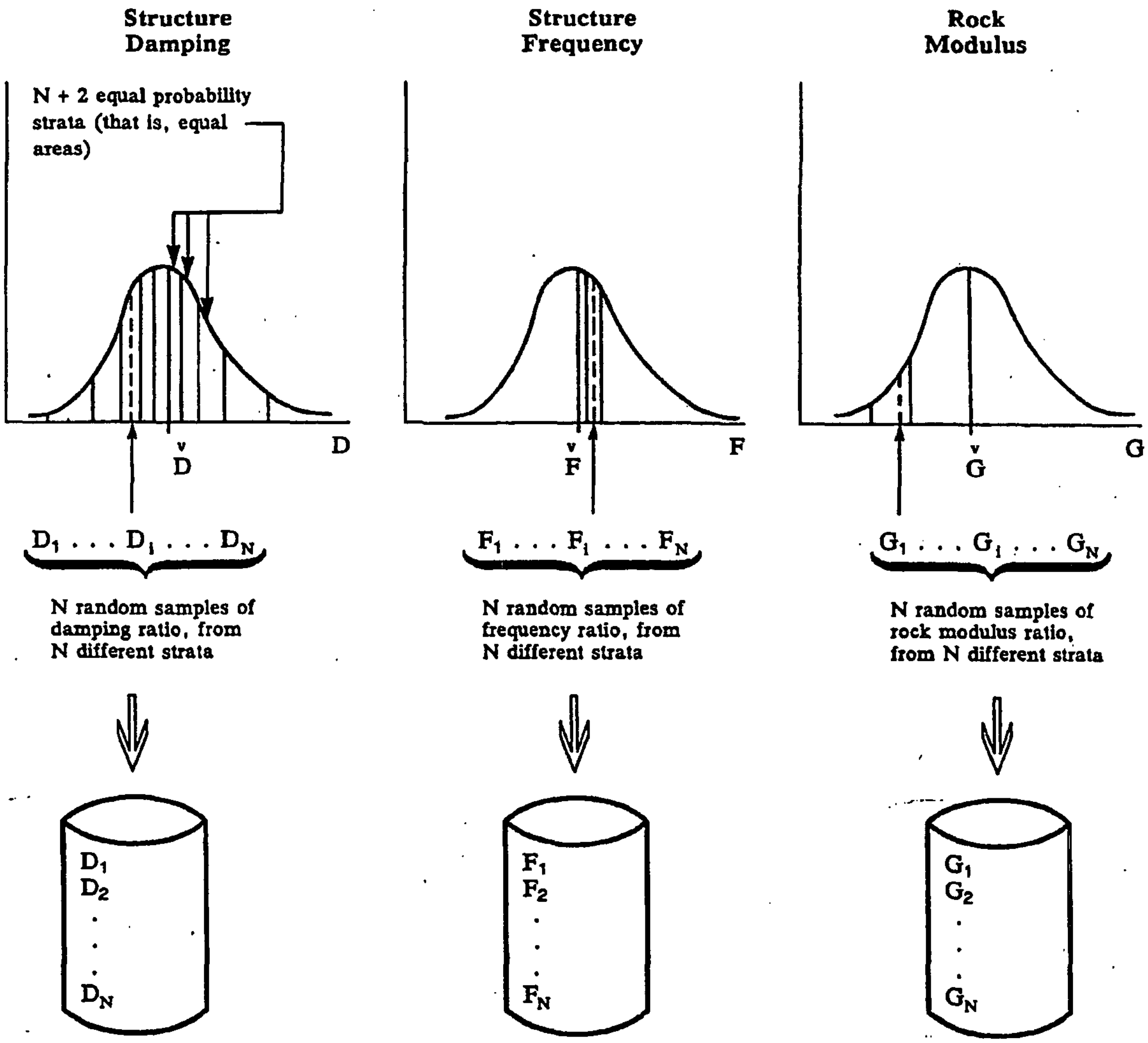


Figure 6-18
Sampling of model parameter values.

limited to the N strata lying between the first and last strata. For each model parameter, one sample value was chosen at random within each of the N strata (by using the model parameter medians and variabilities given in Table 6-7) based on the properties of the lognormal distribution. The result is a set of model parameter values consisting of N values of damping ratio, N values of frequency, and N values of rock modulus as illustrated schematically on Figure 6-18. The three sets of model parameter values were then randomly mixed. This might be visualized asplacing the N damping values, N frequency values, and N rock modulus values into three separate bins, then drawing one damping, frequency, and rock modulus value at random, without replacement, until all values have been chosen. As a result, N sets of model parameter values, each containing a damping, frequency, rock modulus value are obtained as shown on Figure 6-19. Each of the N equally likely parameter values were assigned to one of the N equally likely earthquake pairs; the resulting sets are given in Tables 6-8 and 6-9 for the 24 empirical and 14 numerical records, respectively.

The dynamic properties of the superstructure portion of the simplified soil/structure interaction model were input in each CLASSI run in the form of modal masses, structure damping, and mode shapes and frequencies. A frequency cut-off point of 33 hertz for the superstructure resulted in a total of 56 modes, with cumulative effective modal masses of 87 percent and 95 percent in the north/south and east/west directions, respectively. The balance of the modal masses were treated by the CLASSI program as rigid masses.

From the given set of model parameters, the sampled structure damping was applied to all 56 modes. The frequency ratios, along with the 0.9 concrete cracking factor, were used to scale each of the 56 fixed base frequencies. The rock modulus ratio was applied to the median rock modulus value (that is, the value at the top layer of the soil profile, to which all other layers have been normalized) to determine the input value for each analysis. The shear wave velocity, which is not independent of the rock modulus, must also be specified for the CLASSI program; thus the rock modulus ratio was also used to compute the corresponding shear wave velocity for each analysis.

One deterministic soil/structure interaction analysis was then performed for each of the 38 earthquake/model parameter value sets.

RESULTS

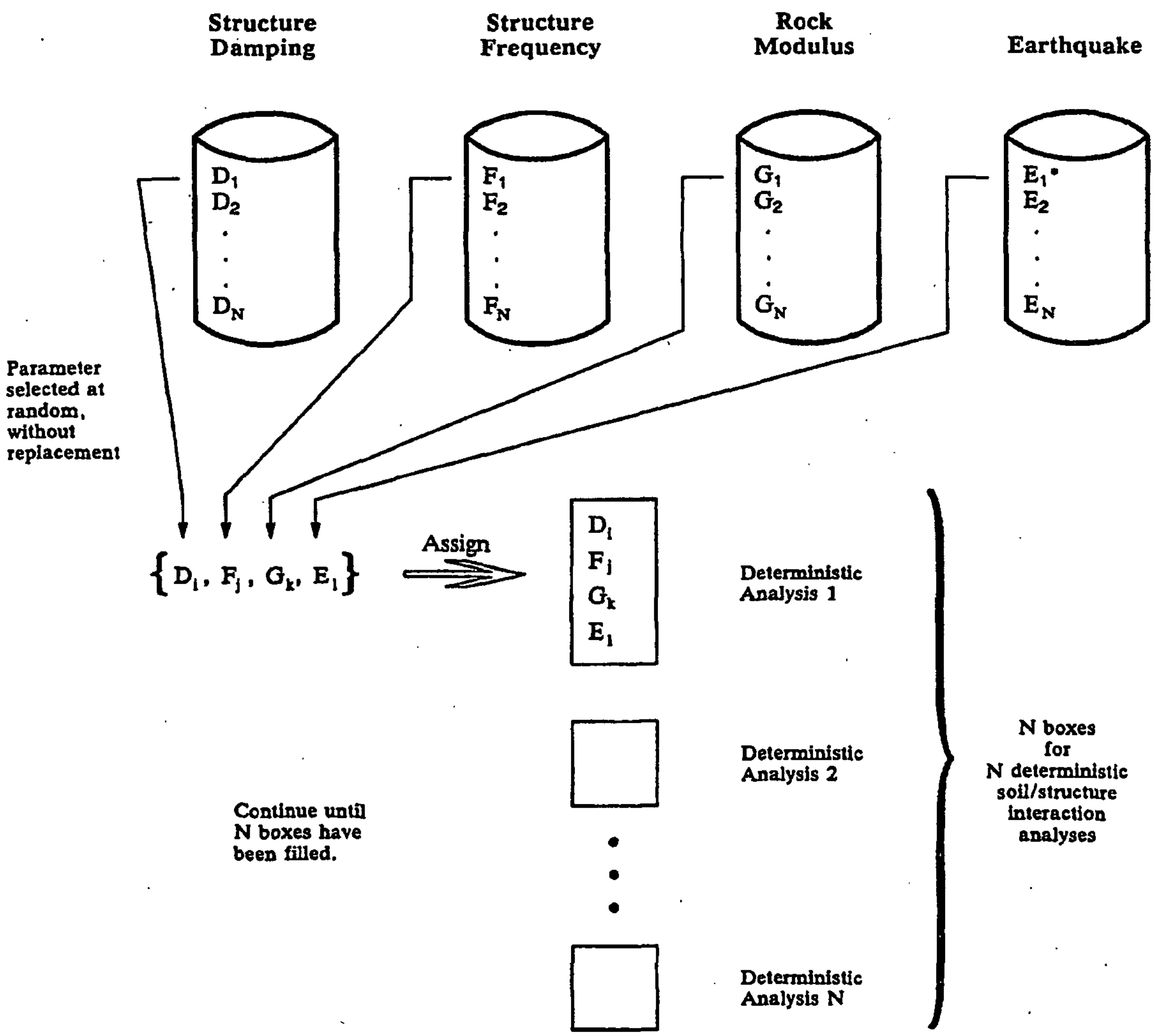
The time-history output from each of the 38 deterministic analyses was obtained for both horizontal directions for six selected locations in core west of the auxiliary building. Referring to Figure 6-15, the selected locations included El 164 feet (node 1), El 154 feet (node 50), El 140 feet (node 2), El 115 feet (node 4), El 100 feet (node 5), and El 85 feet (structure base). From the floor response time histories, floor response spectra were generated for four specified damping ratios (3, 5, 7, and 15 percent). As an illustration of the results of the 38 CLASSI runs, the 5 percent damped north/south response spectra from all 38 runs, at El 140 feet of the core west stick, were plotted on the same frame on Figure 6-20. The spectral accelerations were arranged in descending order at each of the selected frequency points and the median and 84th percentile values were extracted. The resulting median (50th percentile) and 84th percentile floor spectra were then plotted and digitized for use in the fragility evaluations. The north/south and east/west median and 84th percentile spectra for El 140 feet are depicted on Figures 6-21 through 6-24.

APPLICATION OF RESULTS

The combined variability associated with variation of the six parameters included in the auxiliary building variability study was determined by comparing the 5 percent damped median and 84th percentile floor spectra.

$$\beta_c = \ln (S_{a84} / S_{a50}) \qquad (6\text{-}7)$$

In a comparison of the 50th and 84th percentile floor spectra for the various auxiliary building core west elevations, it was found that the variabilities tended to be consistent over certain frequency bands. The resulting combined variabilities are shown in Tables 6-10 and 6-11, respectively, for the north/south and east/west directions.



*The earthquake time histories were randomly mixed by virtue of the random selection of the other three parameters.

Figure 6-19
Random mixing of model parameters.

Table 6-8

MODEL PARAMETER VALUES AND SCALING FACTORS FOR THE EMPIRICAL RECORDS

Analysis Number	Input Time History Number NS	Input Time History Number EW	Structure Damping (%)	Structure Frequency Ratio[1]	Rock Modulus Ratio	Time History Scaling Factor[2]
1	1	2	6.80	0.950 x 0.9 = 0.855	1.335	0.98
2	2	1	4.71	0.915 x 0.9 = 0.824	1.124	0.98
3	3	4	9.46	0.983 x 0.9 = 0.885	0.771	1.12
4	4	3	12.45	0.801 x 0.9 = 0.721	1.737	1.12
5	5	6	4.34	0.903 x 0.9 = 0.813	1.081	1.07
6	6	5	5.10	1.174 x 0.9 = 1.057	1.238	1.07
7	7	8	5.82	0.814 x 0.9 = 0.733	1.486	1.25
8	8	7	6.33	1.009 x 0.9 = 0.908	0.618	1.25
9	9	10	10.09	1.217 x 0.9 = 1.095	2.187	1.46
10	10	9	10.71	1.509 x 0.9 = 1.358	0.986	1.46
11	11	12	4.05	0.644 x 0.9 = 0.580	1.434	1.80
12	12	11	8.07	0.871 x 0.9 = 0.784	0.900	1.80
13	13	14	6.28	0.855 x 0.9 = 0.770	0.540	1.21
14	14	13	9.97	1.344 x 0.9 = 1.210	1.033	1.21
15	15	16	7.29	1.068 x 0.9 = 0.961	1.651	1.31
16	16	15	7.68	0.750 x 0.9 = 0.675	0.853	1.31
17	17	18	5.49	1.428 x 0.9 = 1.285	0.934	0.84
18	18	17	8.02	1.134 x 0.9 = 1.021	0.672	0.84
19	19	20	5.33	0.957 x 0.9 = 0.861	1.167	1.24
20	20	19	7.01	1.121 x 0.9 = 1.009	0.512	1.24
21	21	22	6.08	1.047 x 0.9 = 0.942	0.697	2.13
22	22	21	8.57	0.734 x 0.9 = 0.661	0.738	2.13
23	23	24	8.73	1.264 x 0.9 = 1.138	1.311	1.45
24	24	23	6.72	1.097 x 0.9 = 0.987	0.830	1.45

[1] 0.9 factor accounts for concrete cracking (typical).
[2] For both north/south and east/west time histories.

Table 6-9

MODEL PARAMETER VALUES AND SCALING FACTORS FOR THE NUMERICAL RECORDS

Analysis Number	Input Time History Number NS	Input Time History Number EW	Structure Damping (%)	Structure Frequency Ratio[1]	Rock Modulus Ratio	Time History Scaling Factor[2]
25	26	25	9.28	0.892 x 0.9 = 0.803	0.954	1.38
26	27	28	5.42	0.865 x 0.9 = 0.779	0.566	2.06
27	29	30	8.77	1.061 x 0.9 = 0.955	0.669	2.53
28	31	32	7.90	1.218 x 0.9 = 1.096	1.510	1.68
29	33	34	5.08	1.265 x 0.9 = 1.139	1.693	2.33
30	35	36	10.57	0.801 x 0.9 = 0.721	0.924	1.09
31	37	38	5.56	0.928 x 0.9 = 0.835	1.016	1.33
32	39	40	7.08	0.811 x 0.9 = 0.730	1.190	2.63
33	41	42	9.77	1.025 x 0.9 = 0.923	1.470	1.39
34	43	44	6.05	1.180 x 0.9 = 1.062	0.747	2.25
35	45	46	7.56	0.712 x 0.9 = 0.641	1.299	1.12
36	47	48	6.58	1.430 x 0.9 = 1.287	1.098	1.96
37	49	50	6.75	0.986 x 0.9 = 0.887	0.701	1.23
38	51	52	4.35	1.129 x 0.9 = 1.016	0.864	1.05

[1]0.9 factor accounts for concrete cracking (typical).
[2]For both north/south and east/west time histories.

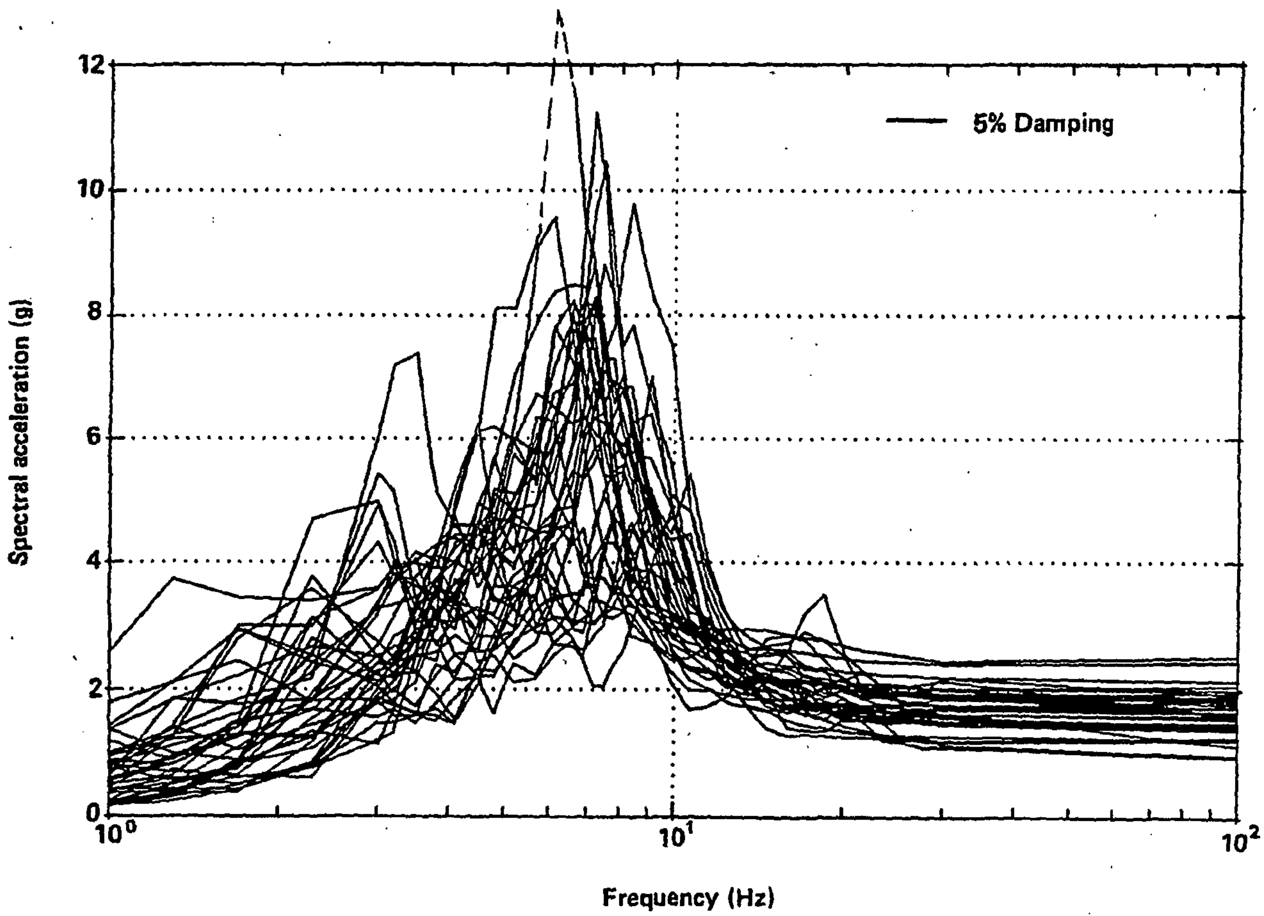


Figure 6-20

North/south response spectra at El 140 feet from all 38 deterministic analyses.

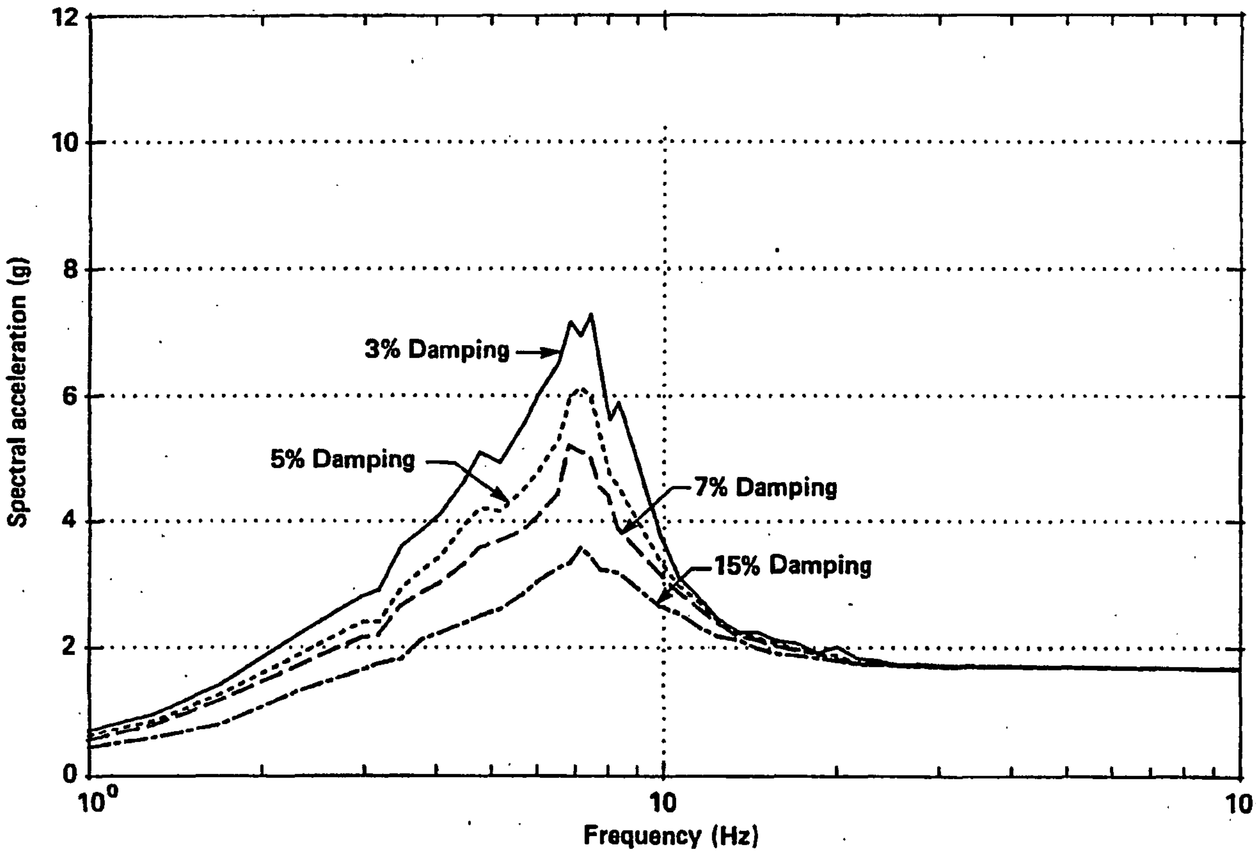


Figure 6-21

50th percentile north/south response spectra for El 140 feet.

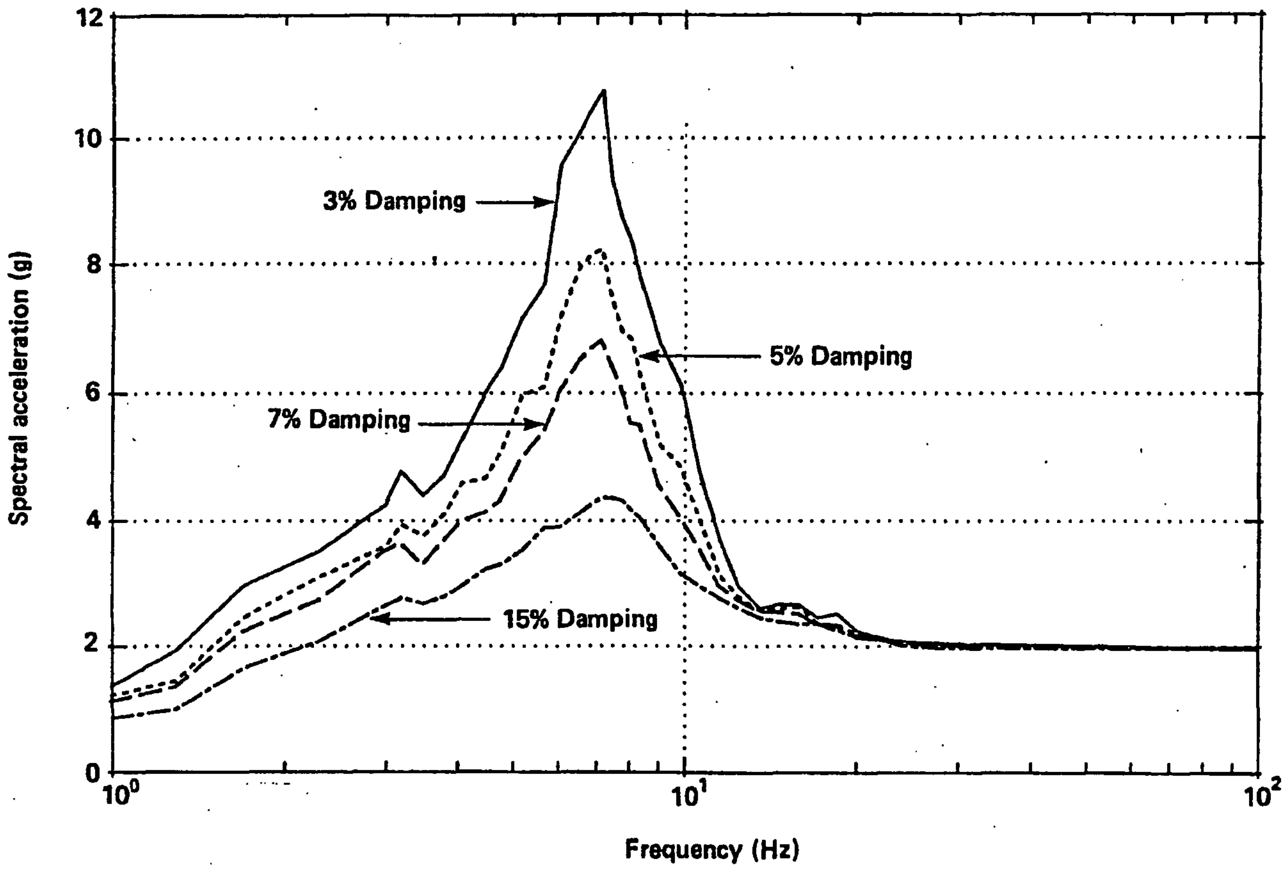


Figure 6-22
84th percentile north/south response spectra for El 140 feet.

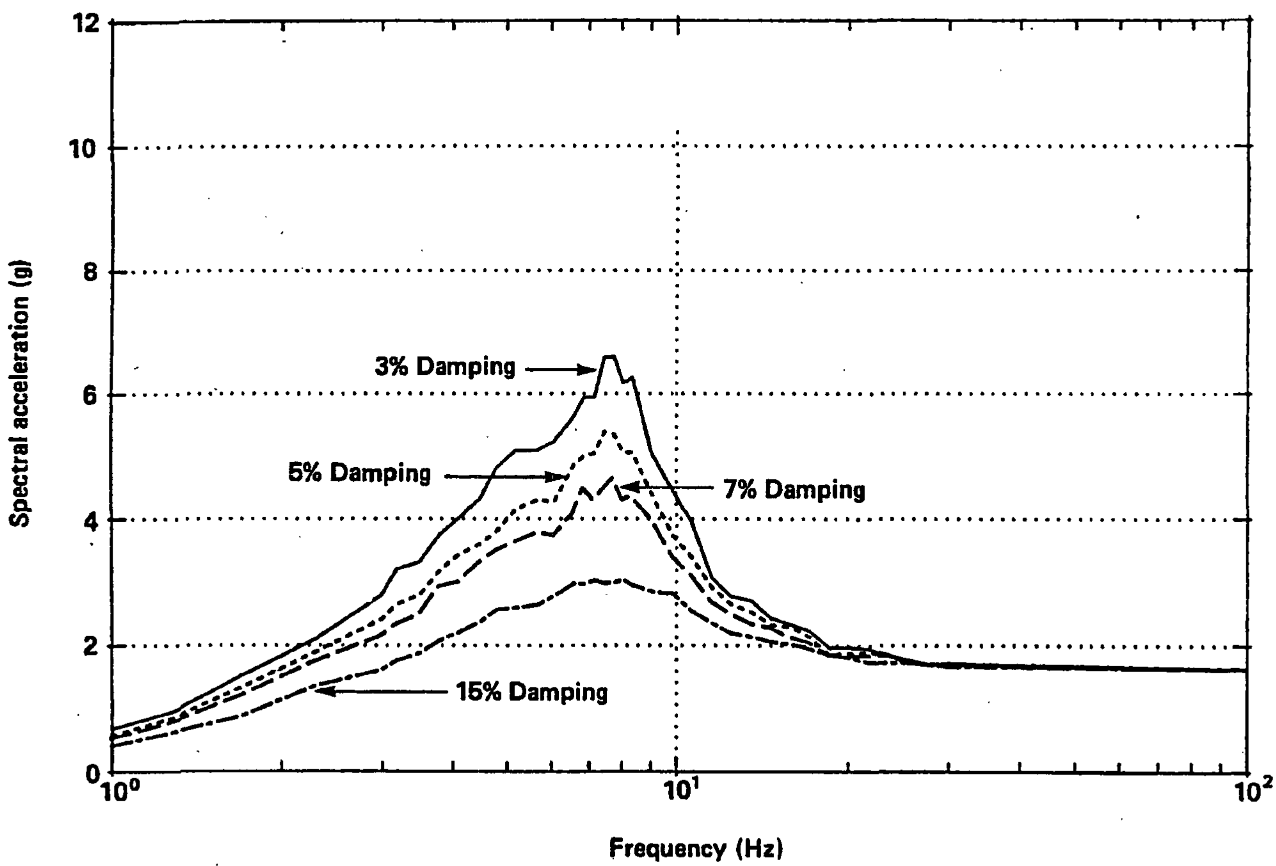


Figure 6-23
50th percentile east/west response spectra for El 140 feet.

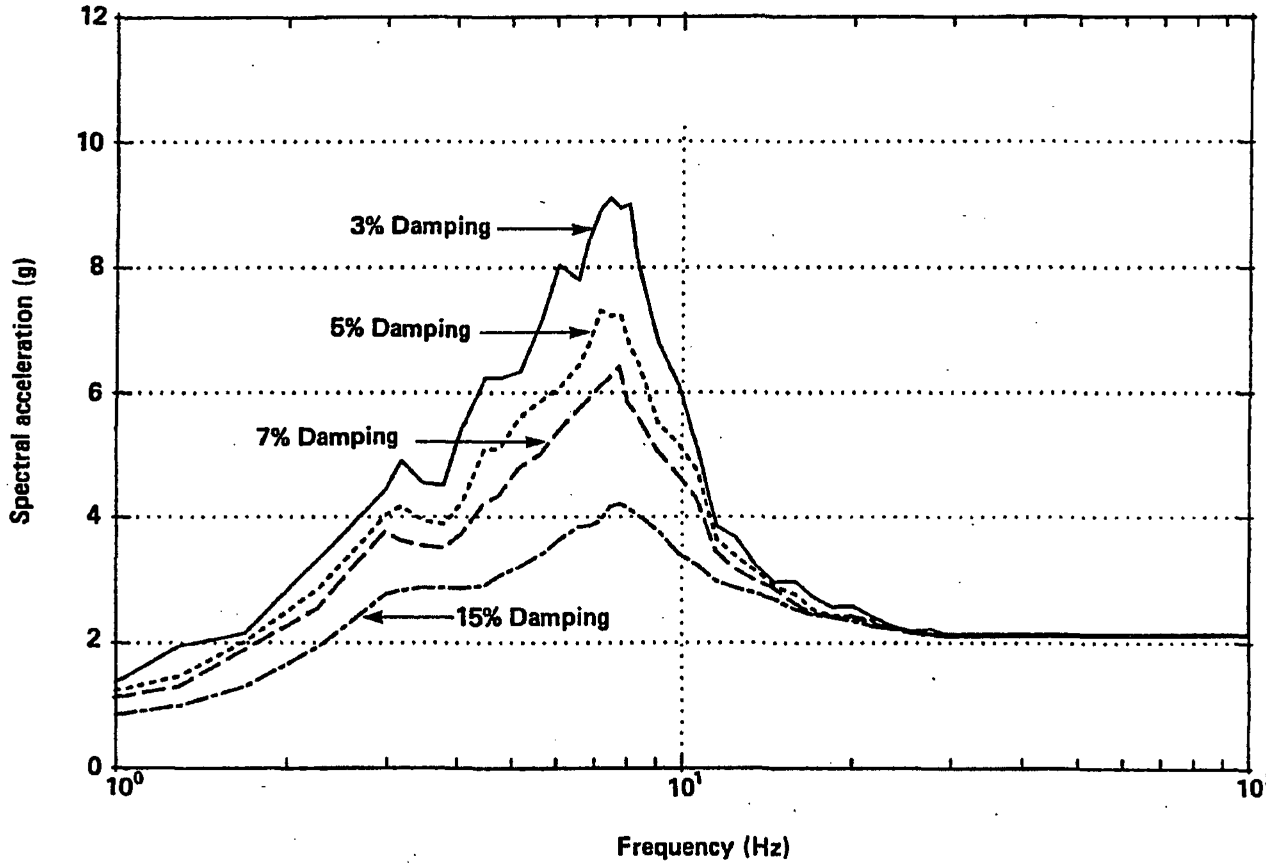


Figure 6-24
84th percentile east/west response spectra for El 140 feet.

At El 85 feet, corresponding to the basemat of the structure, the entire combined variability over each frequency band is taken to be due to randomness, that is, β_R. It can be seen that the combined variability is relatively insensitive to changes in floor level in the low and high frequency ranges, and thus, in these frequency ranges, the combined variability is also virtually all due to randomness. However, in the frequency bands near the fundamental frequency of the auxiliary building, it can be seen that at higher elevations, the combined variability increases substantially. The majority of the increase in the combined variability is due to the uncertainty associated with the structural property values and is assigned to β_u.

Specific structural response variabilities were not conducted for the containment building, concrete internal structure and the turbine building. The structural response variabilities for equipment located in structures other than the auxiliary building were based upon a conservative application of the result of the auxiliary building evaluation. Referring again to Tables 6-10 and 6-11, the structural variabilities at the basemat (El 85 feet) and high in the structures (approximately El 164 feet) were taken to be as shown below:

ELEVATION	FREQUENCY RANGE		
	Low $<0.6\ f_n$	Mid $0.6\ f_n$ to $1.4\ f_n$	High $>1.4\ f_n$
BASEMAT	0.24	0.26	0.24
HIGH IN STRUCTURE	0.34	0.41	0.26

where f_n is the median frequency of the appropriate structure, and the low, mid, and high frequency range correspond to the ranges given in Tables 6-10 and 6-11.

Values for other floor levels were interpolated accordingly. The variabilities for equipment located in these other structures were applied as shown above in terms of the ratio of the equipment fundamental frequency to the fundamental frequency of the appropriate structure.

As noted earlier, the factors and variabilities associated with the remaining three structural response parameters not included in the auxiliary building variability study were applied in accordance with the normal separation-of-variables approach.

As part of the soil/structure interactions analysis described in Chapter 5, median reference floor response spectra were developed for various locations of the containment, auxiliary, and turbine building structures. The 5 percent damped median reference floor spectra developed for selected locations in the west core of the auxiliary building from the soil/structure interaction deterministic study were compared with those developed in the structural response variability study. It was found from the comparisons that the spectra showed good agreement. A representative comparison is depicted on Figure 6-25. The peak frequencies of the two spectra were found to be approximately the same, and the spectral accelerations from the soil/structure interaction spectra were found to be only slightly higher than those from the response variability study in the frequency range of interest. Thus, it was judged that the median reference spectra developed in the soil/structure interaction deterministic study were adequate for use in estimating equipment fragilities.

As noted above, the auxiliary building variability study results were used as the basis for the structural response variabilities for the other structures. The median reference floor spectra from the soil/structure interaction study for the containment building, concrete internal structure, and turbine building tended to be somewhat sharply peaked. Therefore, to be certain that the equipment response near the peak of the reference floor spectra was adequately represented for structures other than the auxiliary building, an additional uncertainty variability on the fundamental frequency of the structures was introduced. This additional uncertainty variability of 0.15 was combined with the equipment frequency uncertainty variability in the assessment of the equipment modeling factor.

Table 6-10

NORTH/SOUTH RESPONSE COMBINED VARIABILITY (β_c)

<table>
<tr><th rowspan="2">Floor Elevation (feet)</th><th colspan="4">Frequency Band (Hz)</th></tr>
<tr><th>3.5 to 5</th><th>5 to 7</th><th>7 to 11</th><th>11 to 30</th></tr>
<tr><td>85</td><td>0.24</td><td colspan="2">0.24</td><td>0.18</td></tr>
<tr><td>100</td><td>0.24</td><td colspan="2">0.27</td><td>0.18</td></tr>
<tr><td>115</td><td>0.24</td><td>0.32</td><td>0.27</td><td>0.18</td></tr>
<tr><td>140</td><td>0.24</td><td>0.37</td><td>0.29</td><td>0.18</td></tr>
<tr><td>154</td><td>0.25</td><td>0.40</td><td>0.29</td><td>0.18</td></tr>
<tr><td>164</td><td>0.26</td><td>0.41</td><td>0.30</td><td>0.18</td></tr>
</table>

Table 6-11

EAST/WEST RESPONSE COMBINED VARIABILITY (β_c)

<table>
<tr><th rowspan="2">Floor Elevation (feet)</th><th colspan="3">Frequency Band (Hz)</th></tr>
<tr><th>3.5 to 6</th><th>6 to 11</th><th>11 to 30</th></tr>
<tr><td>85</td><td>0.24</td><td>0.28</td><td>0.25</td></tr>
<tr><td>100</td><td>0.24</td><td>0.30</td><td>0.25</td></tr>
<tr><td>115</td><td>0.24</td><td>0.30</td><td>0.25</td></tr>
<tr><td>140</td><td colspan="2">0.31</td><td>0.25</td></tr>
<tr><td>154</td><td colspan="2">0.32</td><td>0.26</td></tr>
<tr><td>164</td><td colspan="2">0.35 *</td><td>0.26</td></tr>
</table>

*Except for 6.9 to 7.5 hertz, where $\beta_c = 0.47$

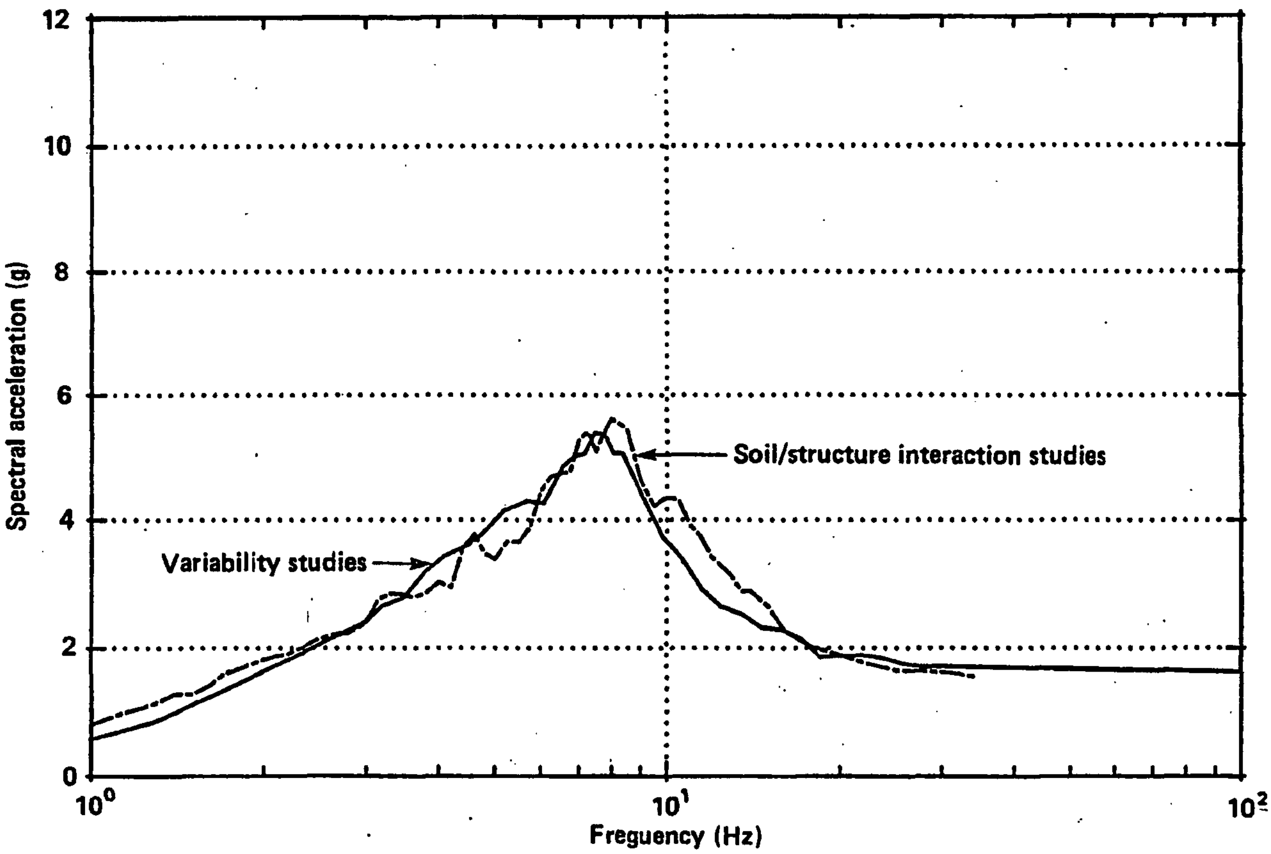


Figure 6-25

Representative comparison of median reference response spectra from the soil/structure interaction and structural response variability studies.

Turbine Building Nonlinear Analysis

In the Diablo Canyon Long Term Seismic Program, the fragilities (probabilistic seismic capacities) of all major structures (Table 6-2) were obtained using the standard separation-of-variables approach (Kennedy, 1980; Kennedy and Ravindra, 1984) as summarized earlier. From these analyses, it was found that the turbine building has the lowest seismic capacity of the structures and is the only one that could possibly be a significant contributor to the seismically induced risk of core damage. Thus, it was determined that a probabilistically based, nonlinear evaluation of the turbine building would be extremely valuable for the purposes of:

- Improving the probabilistic seismic capacity (fragility) estimates for severe overall distress of the turbine building for use in the seismic probabilistic risk assessment.
- Comparing the fragility estimate based upon multiple nonlinear analyses with the estimate extrapolated from a single median-centered elastic response spectrum analysis obtained using the standard separation-of-variables fragility evaluation method.

As a by-product, it was found that the nonlinear analysis provided an understanding of the relationship between turbine building shear wall distress and various earthquake ground-motion characteristics.

It should be noted that the nonlinear evaluation of the Diablo Canyon turbine building provided both probabilistic and deterministic estimates of the turbine building capacity. It is the intention of this portion of the report to only briefly summarize those aspects of the study leading to the development of the fragility parameters. Details are included in the full report entitled "Probabilistic Evaluation of the Diablo Canyon Turbine Building Seismic Capacity Using Nonlinear Time-History Analysis" (Kennedy and others, 1988).

In a manner similar to that used for the auxiliary building variability study, the variables associated with ground motions spectral shape were represented using a suite of 25 earthquake time histories that provide a broad characterization of the ground motions which might occur at the Diablo Canyon site. Further, the variables associated with structural damping, stiffness, and strength were represented by randomly selecting model parameters for use with the suite of earthquake time-history ground motions.

The 25 earthquake time histories used in the turbine building nonlinear analysis consisted of 21 actual recorded ground motions, some of which have been scaled and modified to correspond to Diablo Canyon magnitude, source-to-site distance, and site conditions, and four semi-numerically generated ground-motion records developed to simulate the magnitude of a strike-slip earthquake on the Hosgri fault.

A total of 200 deterministic nonlinear analyses (25 each at average spectral accelerations of 3.0 g and 6.0 g with median structural properties, and 50 each at average spectral accelerations of 3.0 g, 4.0 g, and 6.0 g using variable structural properties) was performed using a simplified model of the turbine building, which was analyzed using the DRAIN-2D computer code (Kanaan and Powell, 1975). The resulting inelastic structure drift from each deterministic run was compared with a criterion relating inelastic drift to the probability of severe distress and strength degradation. The probabilities of severe distress were then statistically evaluated as a function of the three average spectral acceleration levels and the median seismic capacity and variabilities were estimated. The structural response variables associated with structural modeling, earthquake directional effects, and ground-motion incoherency were then added using the normal separation-of-variables approach.

It should be emphasized again that this study is concerned with the prediction of ground-motion levels associated with the onset of severe structural distress and significant strength degradation of the turbine building and not the prediction of failure capacity. In the Diablo Canyon seismic probabilistic risk assessment, the onset of severe structural distress was conservatively used as a surrogate for a structure-induced failure of *all* safety equipment housed in the turbine building.

ANALYSIS MODEL

During Phase II of the Long Term Seismic Program, several possible failure modes that could lead to overall severe distress of the turbine building were investigated using the standard fragility evaluation method. It was concluded that the most probable cause of overall severe distress was substantial inelastic drift and strength degradation of the two major east/west load-carrying shear walls spanning from the foundation level (El 85 feet) to the operating floor (El 140 feet). Thus, the nonlinear analyses consisted of an assessment of the east/west response of the Unit 2 turbine building, with emphasis on the two major east/west load-carrying shear walls below the operating floor.

Figure 6-26 shows a plan view of the Unit 2 turbine building; Figure 6-27 presents a schematic elevation view, emphasizing the major east/west shear walls at column lines 19 and 31 (herein called wall 19 and wall 31), which support the heavy operating floor at El 140 feet. Essentially, walls 19 and 31 are the only two major walls available to resist east/west drift of the heavy operating floor. In turn, nearly all the in-plane lateral loads imposed on these two walls come from the east/west inertial loads of the operating floor, plus their own weight. Although some additional in-plane loads enter due to east/west inertial loads from the intermediate floors, these floor masses are small compared with that of the operating floor, and much of their east/west inertial load is carried by external buttresses added to the turbine building. The inertial loads transferred into Walls 19 and 31 from the superstructure above the operating floor are also small; they were approximated by a slight increase in the weight at the operating floor level. Each wall is 55 feet high by approximately 137 feet long, and contains several openings (particularly wall 19). The thickness of wall 19 varies from 20 inches to about 36 inches over its height. Wall 31 is 24 inches thick over its entire height. Thus, these walls are long relative to their height and are rather thick.

The operating floor consists of a 12-inch concrete slab supported on a steel beam framing system. It is 139 feet wide and 267 feet long between Walls 19 and 31, plus a 77-foot overhang beyond Wall 31. The slab contains a cutout for the independently supported turbine pedestal which is approximately 59 feet wide by 212 feet long. Thus, for east/west lateral forces, the operating floor was treated as two independent 267-foot-long by about 40-foot-deep beams between Walls 19 and 31.

A minimum gap of 3.375-inch exists between the turbine pedestal and the operating floor. This gap is insufficient to preclude impact between the turbine pedestal and the operating floor at the high ground-motion levels of interest in the fragility evaluation. Furthermore, the effective inertial mass to be lumped at the top of the turbine pedestal exceeds the entire inertial mass supported by wall 19 plus wall 31; therefore, impact of the turbine pedestal potentially could lead to additional distress in the shear walls. Thus, the turbine pedestal was included in the nonlinear model together with a gap element interconnecting it to the operating floor beam elements on each side.

Due to their relative ductility, severe distress of the shear walls was expected to occur well before failure of either the operating floor beam elements or the turbine pedestal. For this reason, walls 19 and 31 were modeled in more detail than either the operating floor beam elements or the turbine pedestal. The operating floor and turbine pedestal were only modeled in sufficient detail to approximate their potential for distributing inertial loads to the shear walls. The shear walls were modeled into three segments each along their height, corresponding to points where both the stiffness and strength of the walls greatly change. Because of the low height-to-length ratio, the wall shear stiffness is generally greater than the flexural stiffness and the shear capacity is generally less than the flexural capacity. Each shear wall segment was modeled with both a nonlinear shear element and a nonlinear flexural element combined in series, because each element has different nonlinear properties.

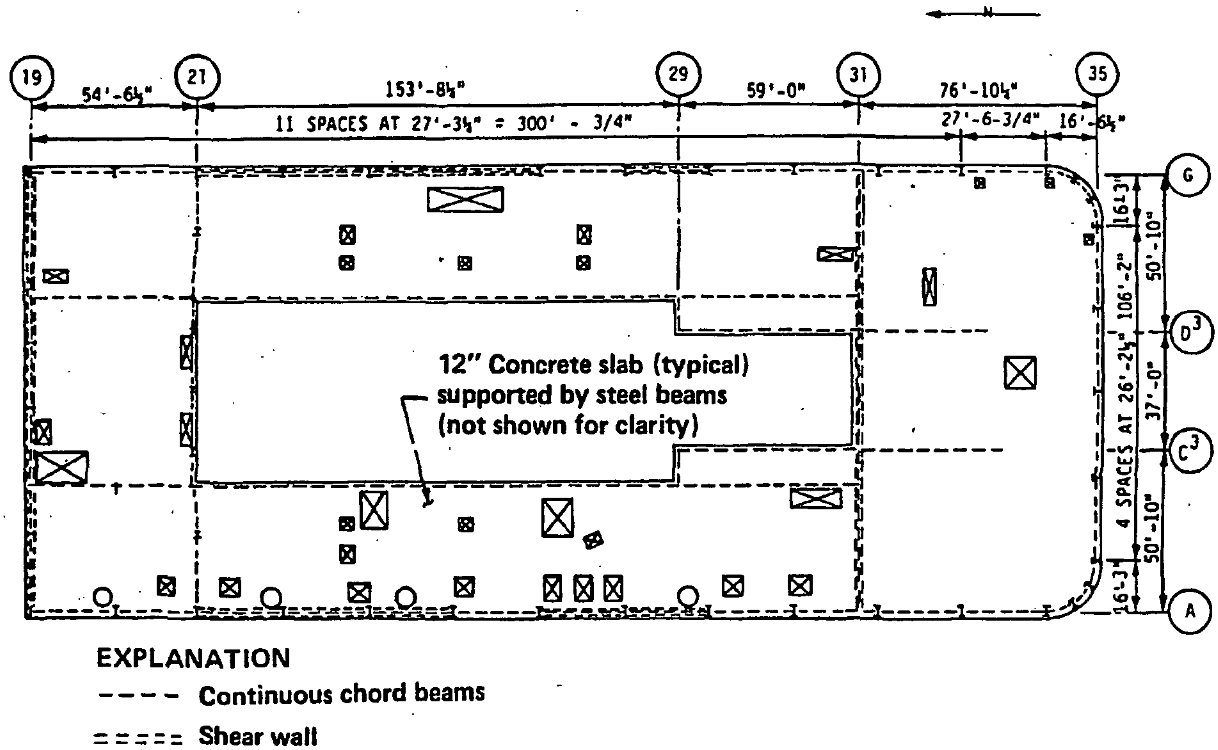


Figure 6-26

Turbine building Unit 2 concrete outline at El 140 feet.

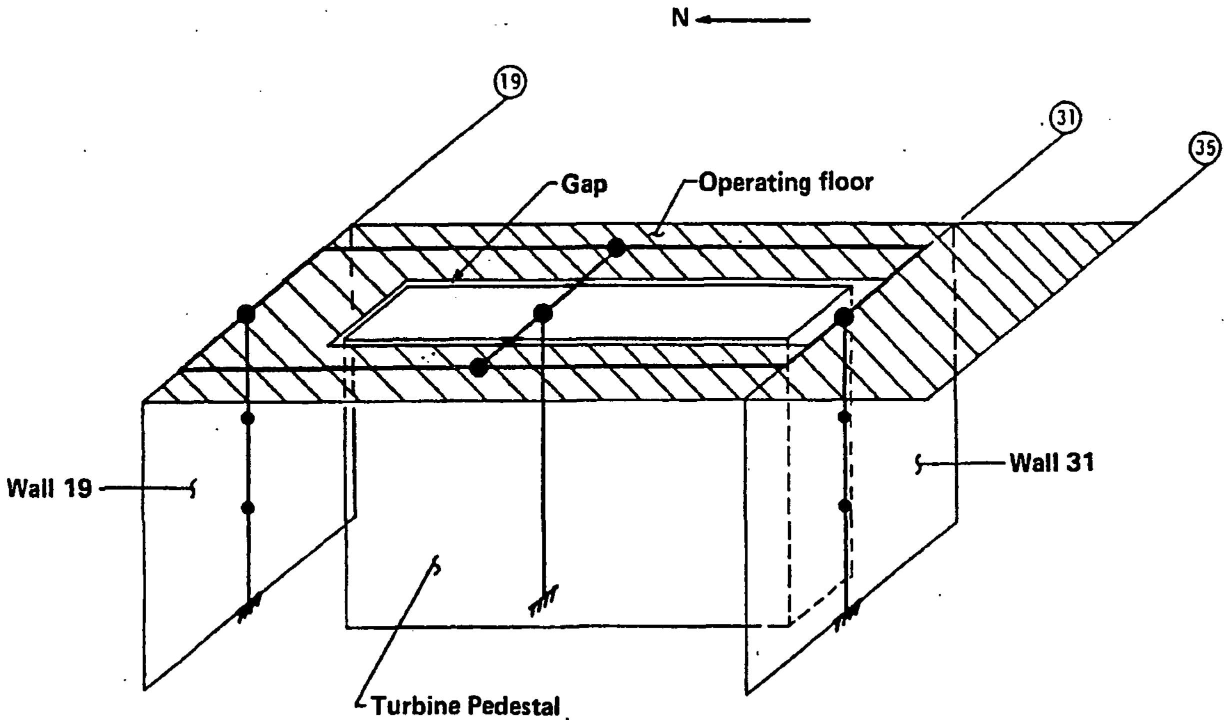


Figure 6-27
Schematic illustration of turbine building nonlinear model.

The analysis was concerned only with east/west response due to an east/west input; therefore, the schematic model (Figure 6-27) was simplified into a two-dimensional model (Figure 6-28). This model consists of the two shear walls subdivided into three segments (stories) each, two operating-floor beam elements, and the turbine pedestal with a 3.375-inch separation gap between the pedestal and the operating floor beam elements. Model properties, including masses, element strengths, and stiffnesses are summarized in Tables 6-12 through 6-14, respectively. Elastic modal characteristics of this model are summarized in Table 6-15.

FORCE-DEFLECTION DIAGRAM FOR SHEAR DRIFT

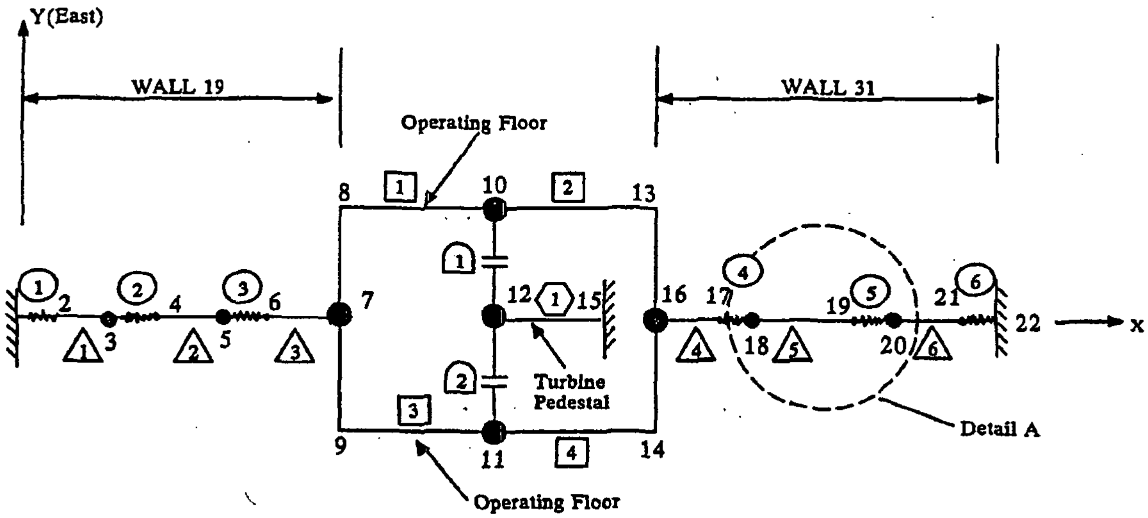
Reinforced concrete walls resist shear through various mechanisms. Initially, the wall is elastic and shear resistance is developed according to elastic beam theory. Inclined shear cracks develop when the principal tensile stresses exceed the concrete tensile strength. Once shear cracks open, the shear force is resisted mainly by the reinforcing bars and aggregate interlock. Other mechanisms such as dowel action, truss action, and the flexural compression zone also contribute to the shear resistance. The opening and closing of cracks under load reversals causes a pinching behavior to be noted in the hysteresis loops. Also, as shear cracks open wider and damage to the concrete increases, the contribution of concrete, through aggregate interlock, to shear resistance decreases. This effect causes strength degradation under large displacement cycles. A typical shear force-shear distortion diagram obtained during a structural wall test is shown on Figure 6-29 (Wang, 1975), which illustrates the reverse-cycle loading behavior characterized by stiffness degradation and pinching of the hysteresis loops. This behavior was approximated by the 10 Rule hysteretic model shown on Figure 6-30. The shear force-deformation curves used for the operating floor beams and the turbine pedestal are shown on Figures 6-31 and 6-32, respectively.

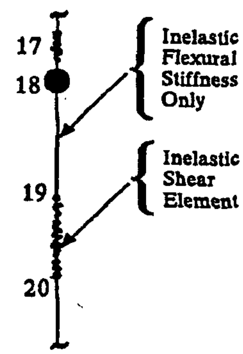
VARIABLE STRUCTURE PROPERTIES

To study the dispersion in the response due to uncertainty in structure properties, a Monte Carlo technique was used in the turbine building nonlinear analysis. Important structure variables affecting structure response (damping, stiffness, and strength), were assumed to be lognormally distributed with median and logarithmic standard deviations as shown below:

Variable	Median Value	Logarithmic Standard Deviation		
		Random	Uncertainty	Composite
Damping	7%	0	0.35	0.35
Stiffness Ratio	1.0	0	0.50	0.50
Strength Ratio	1.0	0	0.25	0.25

Note that the stiffness and strength ratios were used to scale the median stiffnesses and median strengths of each of the structural elements of the nonlinear model. For each nonlinear analysis, the median stiffnesses and strengths of the shear walls, operating floor, and turbine pedestal were multipled by a probabilistically defined stiffness and strength ratio. Stiffness and strength ratios were independently defined for each element type (shear walls, operating floors, and turbine pedestals). Thus, a given element could simultaneously have a high stiffness ratio and a low strength ratio. Similarly, shear walls could have a low strength ratio and the operating floor have a high strength ratio. However, all six shear wall elements in shear and flexure had the same stiffness and strength ratios in a given analysis. Similarly, the four operating floor elements had the same stiffness and strength factors in a given analysis. The 50 sets of stiffness ratios, strength ratios and damping shown in Table 6-16 were independently selected.





DETAIL A

(i) - Inelastic Shear Elements (Shear Deformation Only)

△i - Inelastic Flexural Beam Element (Flexural Deformation Only)

[i] - Operating Floor Element

⟨i⟩ - Turbine Pedestal

(i) - Gap Element

Figure 6-28
Turbine building DRAIN-2D model.

Table 6-12

NODAL MASSES OF TURBINE BUILDING NONLINEAR MODEL

Node No.	Weight (kips)	Comment
3	1,573	Wall 19 and Floor at El 104
5	832	Wall 19 and Floor at El 123
7	4,219	Wall 19 and Operating Floor
10	2,250	Operating Floor
11	2,250	Operating Floor
12	25,000	Turbine Pedestal
16	6,331	Wall 31 and Operating Floor
18	2,130	Wall 31 and Floor at El 119
20	2,460	Wall 31 and Floor at El 107

Table 6-13

MEDIAN CAPACITIES OF SHEAR WALL ELEMENTS

Concrete Shear Wall	Shear Capacities: Concrete Only V_C (kips)	Shear Capacities: Ultimate V_U (kips)	Flexural Capacities: Yield Moment M_U (kip-ft)	Flexural Capacities: Equivalent Yield Shear V_M (kips)
WALL 19				
El 140 to El 123	10,600	12,800	0.23×10^6	13,700
El 123 to El 104	11,000	13,300	0.39×10^6	11,200
El 104 to El 85	9,200	13,500	0.71×10^6	14,100
WALL 31				
El 140 to El 119	13,200	16,600	0.64×10^6	30,700
El 119 to El 107	17,000	21,700	0.72×10^6	24,800
El 107 to El 85	15,000	19,200	1.04×10^6	22,300

Table 6-14

EFFECTIVE ELASTIC SHEAR AND FLEXURAL STIFFNESS OF SHEAR WALLS

Concrete Shear Wall	Effective Shear Stiffness (kips/ft)	Effective Flexural Stiffness (kips/ft)
WALL 19		
El 140 to El 123	1.14×10^6	6.13×10^7
El 123 to El 104	1.22×10^6	7.55×10^7
El 104 to El 85	2.25×10^6	5.05×10^7
WALL 31		
El 140 to El 119	1.71×10^6	24.2×10^7
El 119 to El 107	3.10×10^6	99.0×10^7
El 107 to El 85	1.60×10^6	16.0×10^7

Table 6–15

ELASTIC MODAL PROPERTIES OF THE TURBINE BUILDING MODEL WITH MEDIAN STRUCTURE PROPERTIES

(A) MODAL FREQUENCIES

Mode	Natural Frequency (Hz)	Remarks
1	3.1	Turbine Pedestal
2	4.0	Operating Floor
3	8.6	Wall at Line 31
4	9.5	Wall at Line 19

(B) MODAL SHEARS, AND MOMENTS

	Modal Shears (kips/g)					Modal Moments (kip-ft/g)				
Element	Mode 1	Mode 2	Mode 3	Mode 4	Total Higher Modes	Mode 1	Mode 2	Mode 3	Mode 4	Total Higher Modes
Turbine Pedestal	25,000	—	—	—	—	—	—	—	—	—
WALL 19										
Operating Floor (Per Beam)	—	1,410	-20	-260	0	—	—	—	—	—
El 123+	—	3,470	-390	3,820	-420	—	59,000	-7,000	65,000	-7,000
El 104+	—	3,550	-430	4,360	-160	—	126,000	-15,000	148,000	-10,000
El 85+	—	3,600	-460	4,740	1,010	—	195,000	-24,000	238,000	9,000
WALL 31										
Operating Floor (Per Beam)	—	1,460	-310	-40	0	—	—	—	—	—
El 119+	—	3,660	5,580	160	-840	—	77,000	117,000	4,000	-18,000
El 107+	—	3,820	7,020	230	-380	—	123,000	201,000	6,000	-22,000
El 85+	—	3,950	8,190	280	740	—	210,000	381,000	12,000	-6,000

(C) MODAL DISPLACEMENTS

	Drifts (inches/g)			
Location	Mode 1	Mode 2	Mode 3	Mode 4
Top of Turbine Pedestal	1.040	—	—	—
Center of Operating Floor		0.768	-0.019	-0.015
WALL 19				
El 140	—	0.098	-0.011	0.111
El 123	—	0.056	-0.007	0.070
El 104	—	0.020	-0.003	0.026
WALL 31				
El 140	—	0.071	0.129	0.004
El 119	—	0.044	0.090	0.003
El 107	—	0.030	0.062	0.002

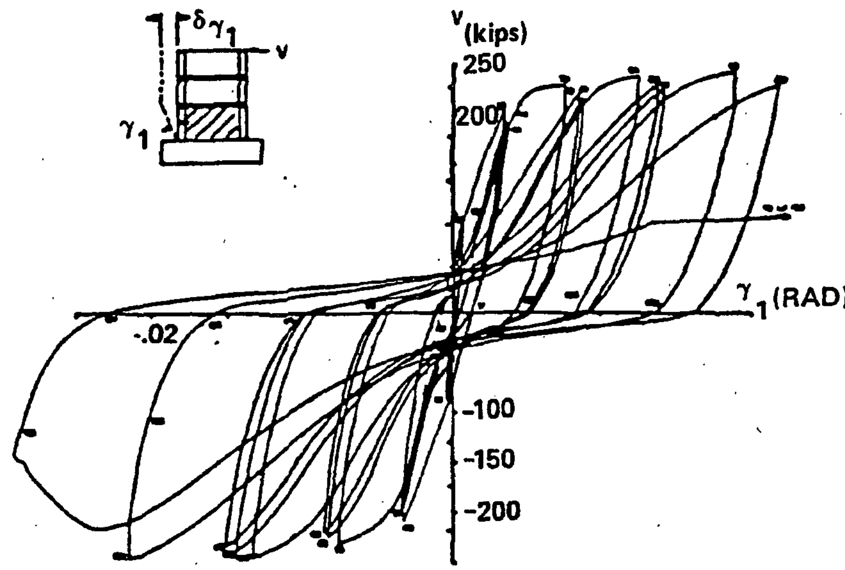


Figure 6-29

Cyclic load-deflection behavior of concrete shear walls (Wang, 1975).

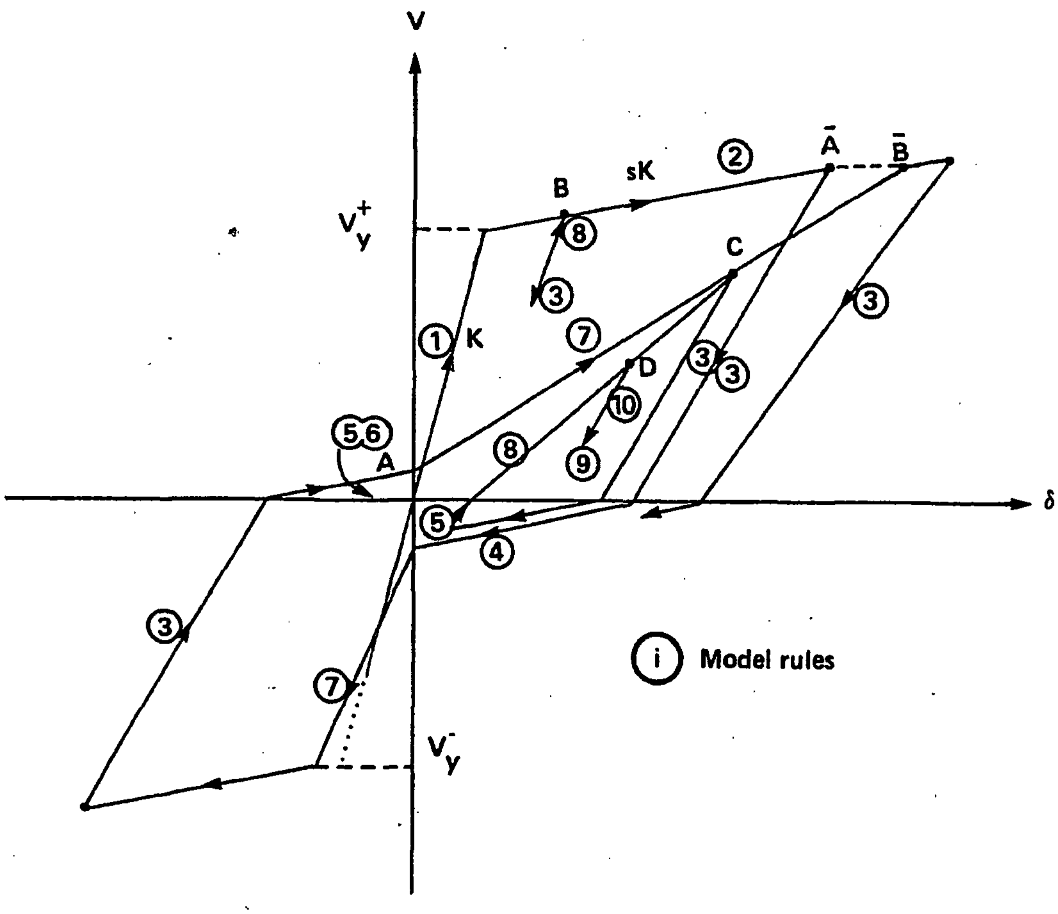


Figure 6-30

Shear deformation hysteretic behavior.

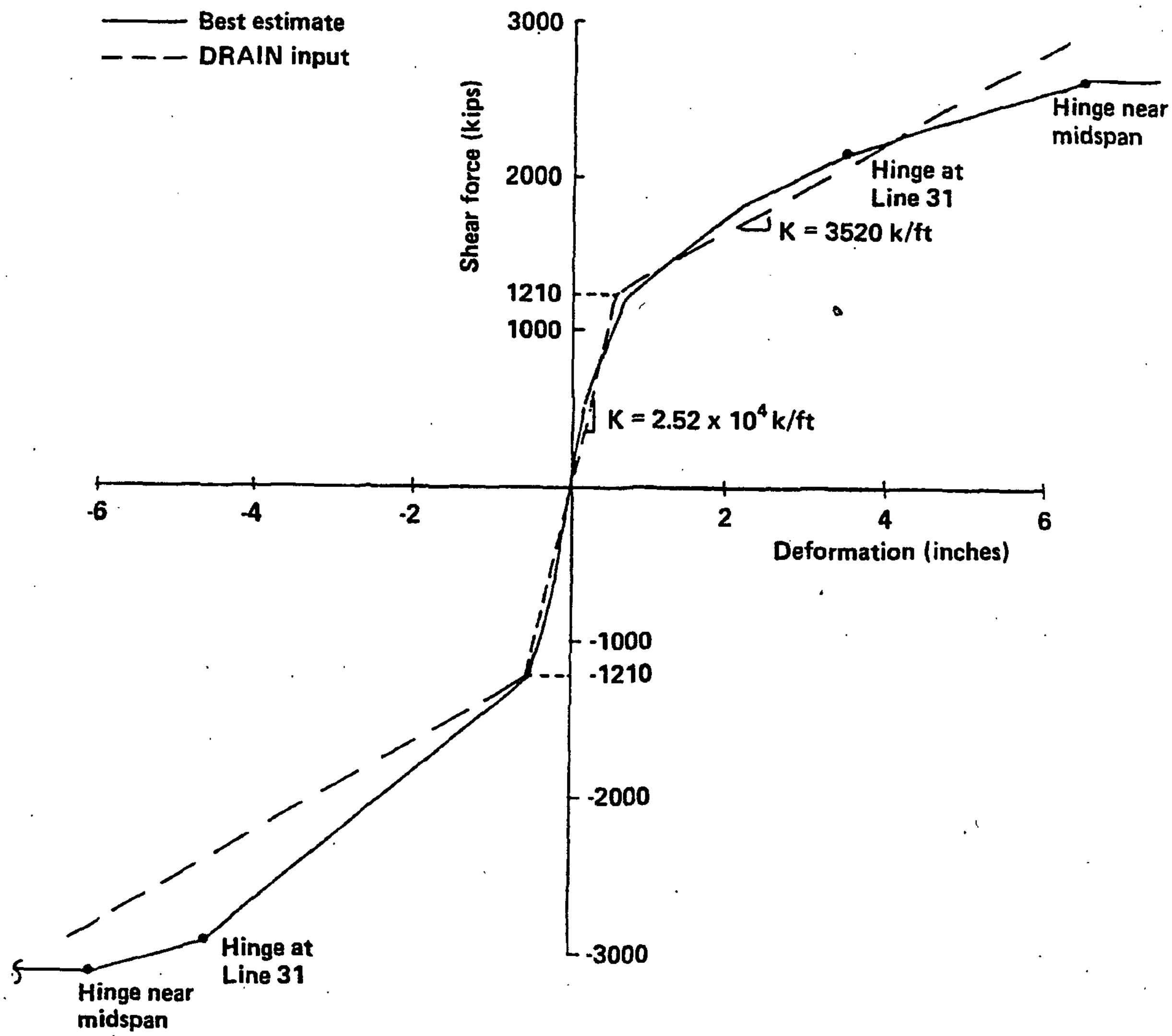


Figure 6-31

Shear deformation curve of the beam-like portion of the operating diaphragm at the midspan for each of four beam elements.

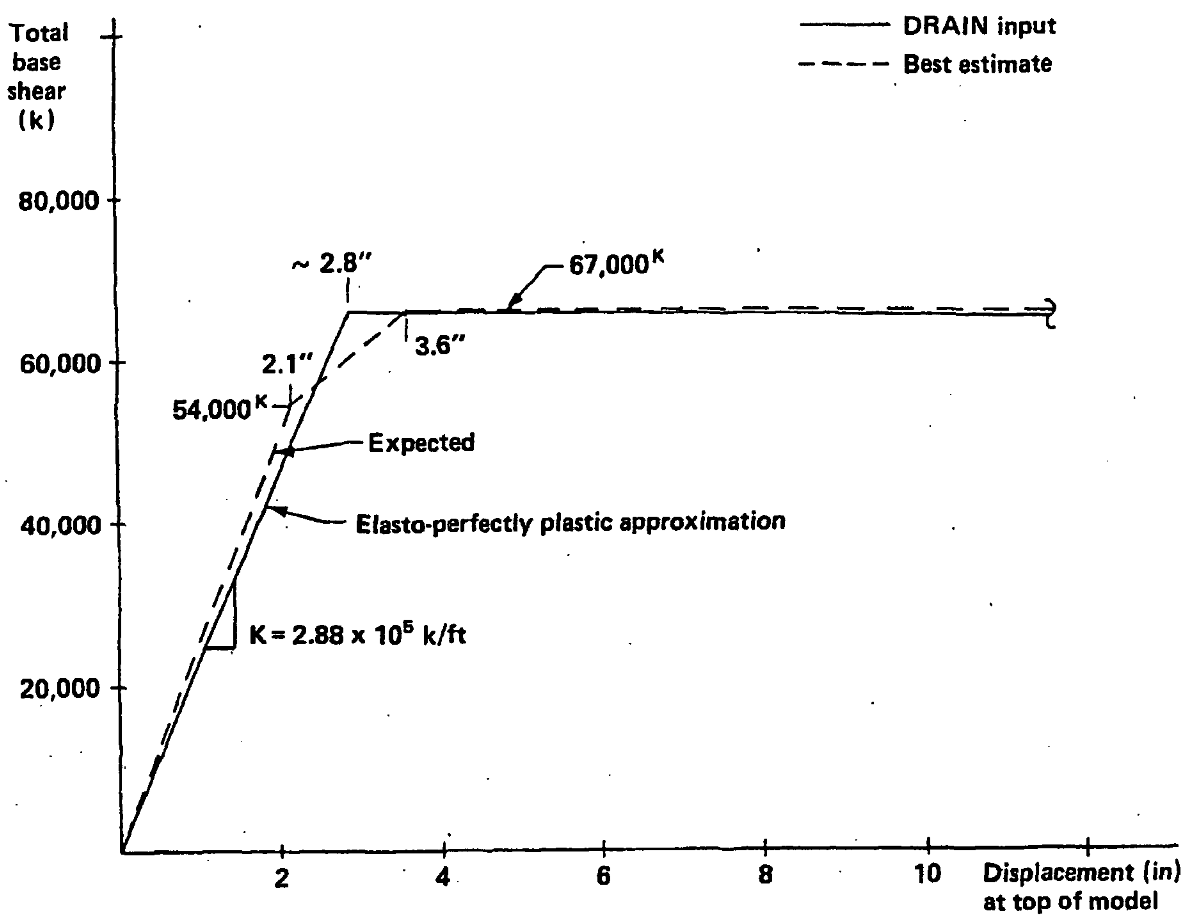


Figure 6-32

Shear-deformation curve of the turbine pedestal.

Table 6-16

VARIABLES OF MODEL STRUCTURE PROPERTIES

Trial Number	System Damping Value	Stiffness Ratio: Shear Walls	Stiffness Ratio: Operating Floor	Stiffness Ratio: Turbine Pedestal	Strength Ratio: Shear Walls	Strength Ratio: Operating Floor	Strength Ratio: Turbine Pedestal
1	.0601	.9343	.8421	1.4495	1.1481	.9319	1.1148
2	.0793	1.0607	.9839	.6679	1.0732	.9263	1.2484
3	.1155	.9275	1.0205	.4355	.9692	1.8888	1.0005
4	.1009	1.0914	1.7003	.7948	.8729	1.2734	1.0282
5	.1023	2.1317	1.5155	1.2248	1.4069	1.1721	1.1336
6	.0582	3.0935	.6578	1.3189	.6618	1.3248	1.4670
7	.0585	1.0504	1.1815	.5977	1.0734	.7686	1.4773
8	.0568	.9974	1.5550	1.0877	1.2396	.7181	.7032
9	.0684	1.5548	.5104	.8385	1.3007	1.2217	1.5525
10	.0698	1.1254	1.3876	2.2731	1.4946	.8750	.7939
11	.0704	.4634	1.1817	.4356	.7953	1.0006	1.1623
12	.1493	1.4327	1.3365	1.3245	.9122	1.0389	1.5407
13	.0572	.6004	1.3288	1.0898	.7238	.8229	1.1988
14	.0927	1.5996	.9293	2.1709	.9210	1.3397	1.3248
15	.1123	.4682	1.0784	.6464	.8982	.6652	.8286
16	.0652	1.2137	1.0849	1.1087	.7419	1.4439	.8299
17	.1053	1.1349	1.7651	2.0897	.6416	1.1718	.9177
18	.0609	2.1395	.9588	.5845	1.0878	.8770	.9039
19	.1096	1.2604	1.6396	2.5241	1.1662	1.6452	1.1995
20	.1074	1.6790	.8242	2.5171	.8942	.8675	1.0032
21	.0596	.6275	.7439	.6320	.8245	.7250	.7423
22	.0760	1.0896	1.0855	.5099	.8584	1.4128	1.0756
23	.1369	3.5920	.8568	1.3599	1.2019	.9804	1.1332
24	.0831	1.0797	.9291	.6403	1.5220	.5480	1.0474
25	.1240	1.0087	.9222	1.8344	.6911	.6206	1.0911
26	.0772	.5653	.8310	.8607	1.0071	.8561	.8499
27	.1136	1.3648	.5680	.6208	1.3277	.7726	1.3637
28	.0910	.6796	1.1320	1.4513	.9571	1.0147	.7058
29	.0496	2.2296	1.8930	1.1704	1.0630	1.1235	1.3609
30	.0486	.9323	3.7765	2.3605	1.4893	.6491	1.6148
31	.0538	.4250	.5502	2.0228	1.1503	.6496	.9712
32	.1009	1.1350	.5983	2.5905	.8772	.8909	.6511
33	.0949	.8769	2.0427	.9961	.7191	1.2164	.6773
34	.0365	1.1243	1.9010	.7875	1.5064	.7779	1.0995
35	.1507	1.6397	3.7699	1.4291	.9167	1.0773	1.2275
36	.0334	.8274	.9919	.5106	.9336	1.0267	.6908
37	.0523	.6222	1.4331	.9317	.9753	.8322	1.2863
38	.0357	.6568	1.4558	1.4051	.9462	1.0556	1.5023
39	.0603	1.0507	1.2858	1.2992	1.0741	.6953	.8159
40	.0753	.9401	1.7601	1.0112	.7298	.7261	1.5294
41	.0637	1.2890	.8582	.8481	.8885	.7238	.8793
42	.0391	.5772	.8199	.9413	.5133	.8863	1.2748

Table 6-16 *(Continued)*

VARIABLES OF MODEL STRUCTURE PROPERTIES

Trial Number	System Damping Value	Stiffness Ratio: Shear Walls	Stiffness Ratio: Operating Floor	Stiffness Ratio: Turbine Pedestal	Strength Ratio: Shear Walls	Strength Ratio: Operating Floor	Strength Ratio: Turbine Pedestal
43	.1107	1.1182	1.3084	.4553	.9878	1.1935	1.1383
44	.1180	2.6135	.8649	1.0102	.7662	1.1832	.9608
45	.1142	3.3185	1.5687	.8905	1.1715	.9960	.9420
46	.1162	.8227	1.1182	.7581	.8490	1.6054	.7599
47	.0783	1.0485	3.6193	.9812	1.0399	1.2617	.8452
48	.0538	1.3866	.6226	.5341	.8526	1.0049	.9262
49	.0403	.6810	1.2759	.5473	1.1852	1.0856	1.2072
50	.0616	.3445	.4155	.8854	1.1008	1.1917	1.2165

INPUT MOTION

A single ground-motion parameter was used to define the fragilities of Diablo Canyon structures and equipment. The 5 percent damped average spectral acceleration over the 3 to 8.5 hertz range was chosen to convolve the seismic hazard and seismic fragilities for use in the probabilistic risk assessment.

Twenty-one earthquake time-history records (Table 6-17) representing actual recorded events were selected for use in this study based upon the following selection criteria:

1) The records should be appropriate for shallow crustal earthquakes in the magnitude range from 6.5 to 7.5 with recording distances appropriate for the Hosgri fault zone.

2) The records should be appropriate for rock-site conditions.

3) The records should represent, in the aggregate, about a 50-50 mixture of thrust and strike-slip faulting.

4) The records should be appropriate for ground motions having very high average spectral accelerations (defined as the average 5 percent damped spectral acceleration in the 3 to 8.5 hertz range), of 2.0 g or greater. Ground motions with average spectral acceleration less than about 2.0 g are undamaging to the turbine building and are thus of little interest.

Only the Tabas and Pacoima Dam records (Records 3 through 6) met the above criteria in their original unmodified form. Although average spectral acceleration was too low, the Gazli records (Records 1 and 2) clearly met Criteria 1 and 2. All other empirical records had to be modified for distance (frequency-independent scaling) and/or magnitude and site conditions (non-constant, frequency-dependent correction). After modification, all 21 empirical records met Criteria 1 and 2. Table 6-17 lists the characteristics of both the original and the modified records and the average spectral acceleration for each record after modification.

Even after modification, only a few of the empirical records met Criterion 4; it was assumed the records could be further modified by frequency-independent upward scaling to achieve desired values of average spectral acceleration. Due to the paucity of near-source, strong-motion records from rock sites for magnitude approximately 7.0 strike-slip earthquakes, records 22 through 25 were added (Criterion 3). These are simulated ground-motion records generated by semi-numerical methods to represent a magnitude 7.0 Ms strike-slip earthquake on the Hosgri fault.

To study the randomness variability of the ground motions on the shear wall drifts, each of the 25 modified earthquake ground-motion time histories listed in Table 6-17 were constant-amplitude (frequency-independent) scaled to obtain the same average spectral acceleration in the frequency range of 3 to 8.5 hertz. Using median structural properties, shear wall drifts were computed from the nonlinear analyses for average spectral acceleration values of 3.0 g and 6.0 g (25 trials each). Figure 6-33 presents the 5 percent damped response spectra for three of the records, each scaled to an average spectral acceleration of 3.0 g to illustrate the diversity of spectral shapes included. Figure 6-34 depicts the mean, median, 84 percent probability of non-exceedance, and upper-bound spectra for the ensemble of 25 records scaled to an average spectral acceleration of 2.25 g.

To study the combined influence of the randomness variability associated with the ground motions and the uncertainty variability associated with the structural properties, each of the 25 modified ground-motion records was scaled to average spectral acceleration values of 3.0 g, 4.0 g, and 6.0 g, and each was used twice (Trials 1 through 25 and Trials 26 through 50), in combination with the 50 sets of variable structural properties shown in Table 6-16 (150 total trials).

SHEAR WALL DRIFT LIMIT

The drifts associated with Walls 19 and 31 were established from each of the 200 nonlinear trials using median and variable structural properties. To calculate the corresponding probability of

Table 6-17
EARTHQUAKE TIME HISTORIES

Earthquake Date	Magnitude	Style of Faulting	Recording Station Distance	Time History Number	Component	Site Conditions	Time History Adjustments	$\bar{S}_a\vert_5$ 3-8.5 Hz (g)
Gazli, U.S.S.R. May 17, 1976	6.8	Reverse	Karakyr Point 3 km	2 1	East North	Rock/stiff alluvium	None	1.33 1.31
Tabas, Iran Sept. 16, 1978	7.4	Thrust	Tabas 3 km	4 3	N16W N74E	Stiff alluvium/rock	None	2.48 2.27
San Fernando, CA Feb. 9, 1971	6.6	Thrust	Pacoima Dam 3 km	5 6	S16E S74W	Rock	None	2.00 1.89
			Lake Huges No. 12 20 km	7 8	N21E N69W	Rock	Distance	2.38 2.27
			Castaic 25 km	9	N69W	Stiff alluvium	Distance	1.69
Imperial Valley, CA Oct. 15, 1979	6.5	Strike-Slip	Differential Array 5 km	10 11	N00E N90W	Deep alluvium	Site response	1.38 1.55
			El Centro No. 4 4 km	12 13	S50W S40E	Deep alluvium	Site response	0.75 1.16
Parkfield, CA Jun. 27, 1966	6.1	Strike-Slip	Temblor 10 km	14 15	N65W S25W	Rock	Distance and Magnitude	1.27 1.33
Morgan Hill, CA Apr. 24, 1984	6.2	Strike-Slip	Coyote Lake Dam 0.1 km	16 17	N75W S15W	Rock	Magnitude	2.29 1.95
Coalinga, CA May 2, 1983	6.5	Reverse	Pleasant Valley Pump Station (Switchyard) 10 km	18 19	N45E S45E	Stiff alluvium/rock	Distance	1.63 2.38
Tabas, Iran Sep. 16, 1978	7.4	Thrust	Dayhook 17 km	20 21	N10E N80W	Rock	Distance	1.12 1.67
Hosgri Simulations	7.0	Strike-Slip Bilateral	-	22 23	North East	—		1.16 1.47
		Strike-Slip Unilateral	-	24 25	North East	—		0.98 1.56

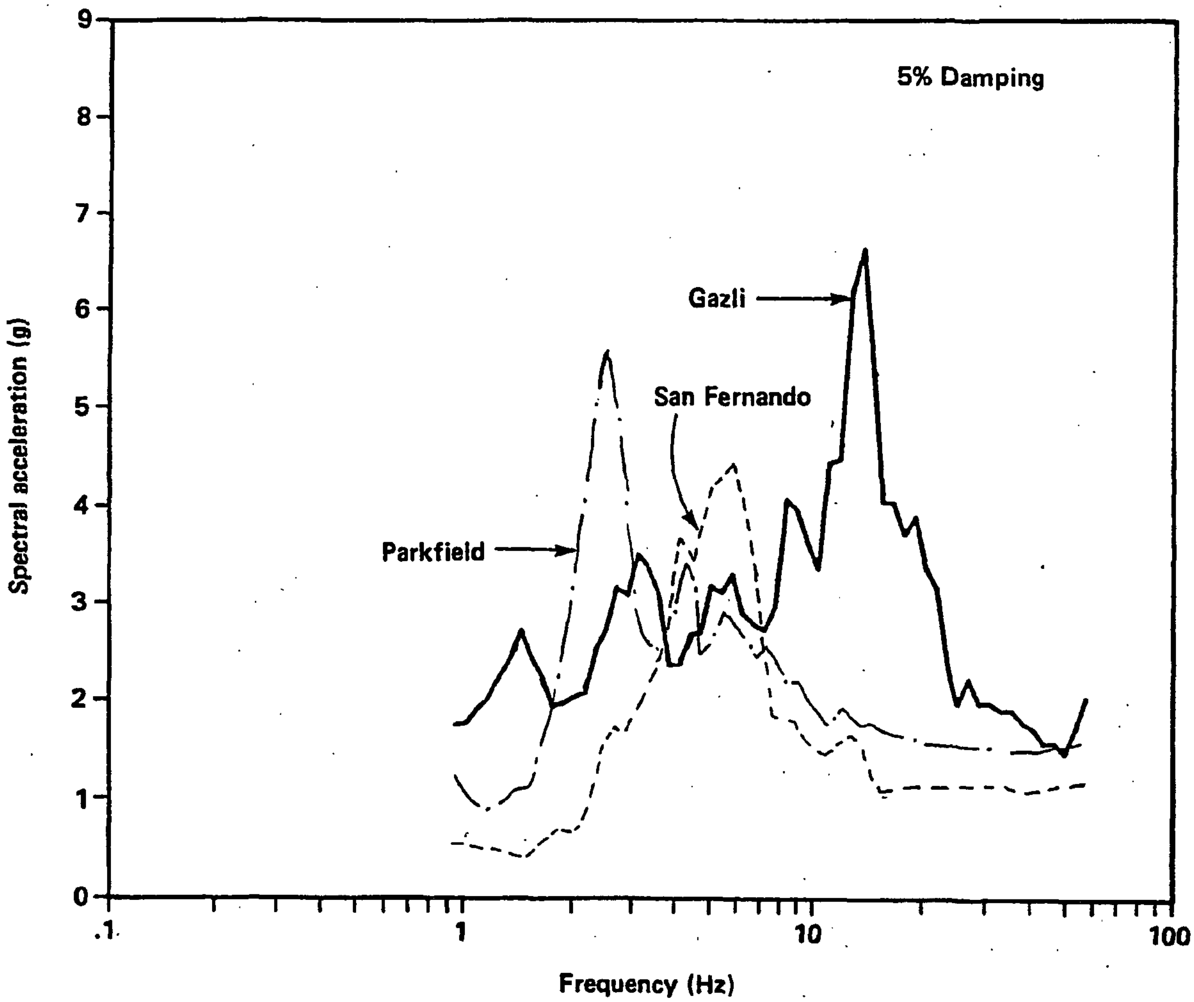


Figure 6-33

Acceleration response spectra for three empirical records scaled to an average spectral acceleration of 3.0 g over the frequency range of 3.0 to 8.5 hertz.

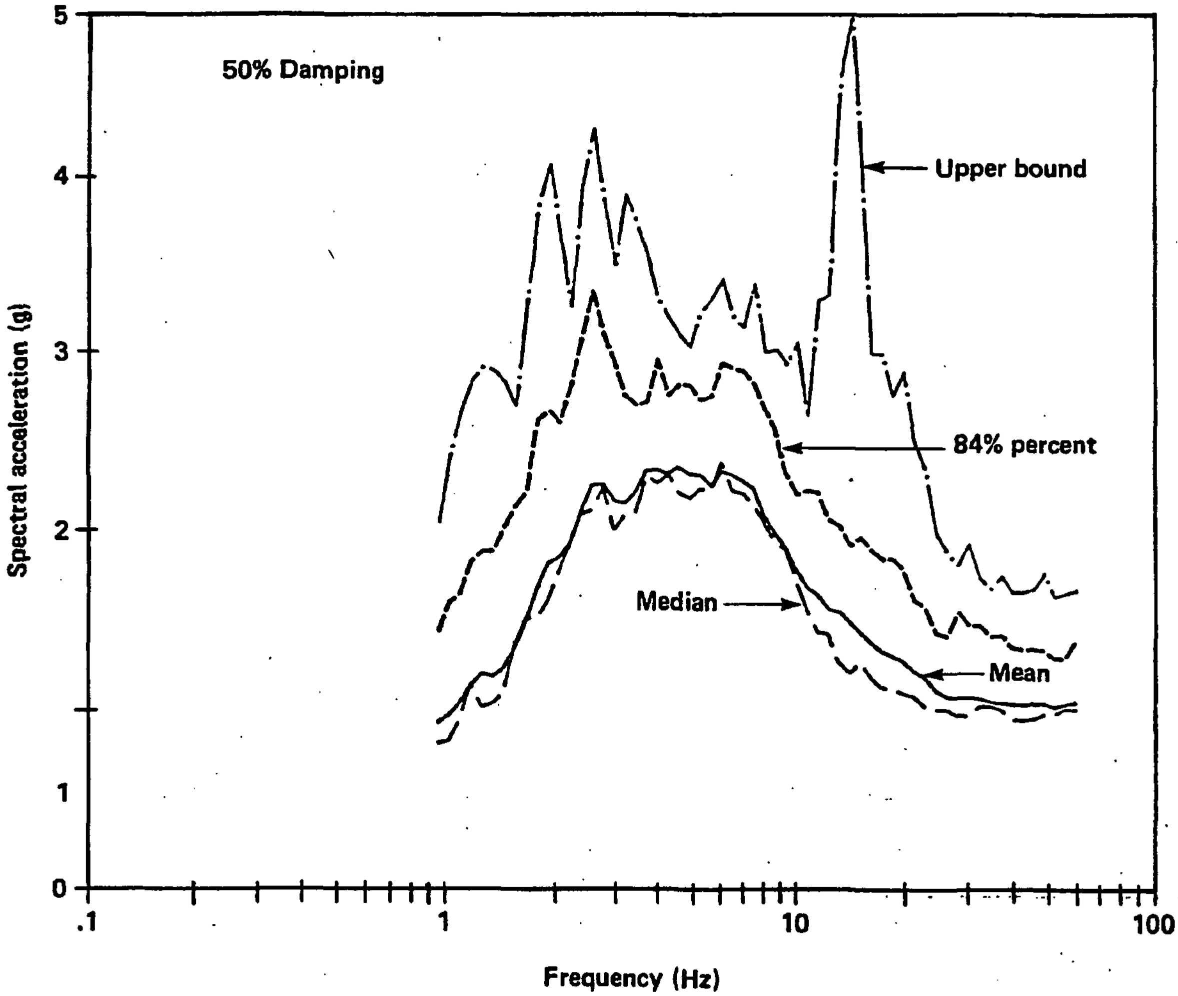


Figure 6-34

Mean, median, 84 percent probability of nonexceedance, and upper-bound spectra for 25 records scaled to an average spectral acceleration of 2.25 g over the frequency range of 3.0 to 8.5 hertz.

severe distress, the onset of severe shear wall damage (significant strength degradation) was defined in terms of shear wall drift limits. Based upon a study of the results of a number of shear wall strength investigations, both in the United Stated and Japan, the median estimate of shear wall drift (expressed as a percentage of wall height) corresponding to the onset of significant strength degradation and the associated logarithmic standard deviations were taken as:

$\check{D} = 0.7\%$ (median drift limit)

$\beta_R = 0.15$

$\beta_U = 0.30$

$\beta_C = 0.335$

When treated on a composite basis (using βc), there is about a 16 percent probability of severe distress at 0.5 percent drift and about an 84 percent probability of severe distress at 1.0 percent drift. These estimates might be more conservative than necessary.

Both walls 19 and 31 were segmented into three elements along their height because of changing capacities and stiffnesses. With the shear capacities listed in Table 6-13, drift percentages tend to be greatest within the lower element in wall 19 or within the lower or upper element of wall 31. It was conservatively decided to limit the element having the greatest drift percentage to the limits specified above. Thus, the probability of severe distress was based upon the shear element having the largest drift percentage obtained as a percent of the element height, such that the limit criterion was essentially treated as an element drift criterion. The total drift of either wall 19 or 31 was less than the maximum element drift percentage times the total wall height of 55 feet (often substantially less).

ANALYSIS RESULTS

First, an elastic response spectrum analysis was performed using the 5 percent damped median response spectrum scaled to an average spectral acceleration of 2.25 g (Figure 6-34) and median structural properties (note that 7 percent median damping was used in this analysis). The results of this analysis are presented in Table 6-18. Based upon these results, it was concluded that the lower segment of both wall 19 and wall 31 will yield slightly in shear at an average spectral acceleration of 2.25 g because the elastic demand to yield capacity ratios (V_R/V_Y) are slightly greater than unity. Based on the median ground spectrum shape and median structural properties, inelastic behavior is expected to initiate at about an average spectral acceleration of 1.90 g and 2.05 g for the lower segment of walls 19 and 31, respectively. However, at an average spectral acceleration of 2.25 g, with median properties, yielding in the shear walls will be slight and limited to the lowest segment of each wall. With median properties, the turbine pedestal is expected to remain elastic up to an average spectral acceleration of 3.30 g. At an average spectral acceleration of 2.25 g, the median drift of the turbine pedestal was estimated to be about 1.9 inches; that for the operating floor was estimated to be about 2.0 inches. Combining the drift responses by square-root-sum-of-the-squares, the gap closure between the pedestal and operating floor was estimated to be about 2.75 inches, which is less than the available gap of 3.375 inches. Thus, at an average spectral acceleration of 2.25 g, it is not expected that the turbine pedestal will impact the operating floor for the median spectrum shape case.

Each of the 25 modified time histories, scaled to an average spectral acceleration of 3.0 g and 6.0 g, were applied to the nonlinear structure model with median strength, stiffness, and damping properties. Tables 6-19 and 6-20 list the maximum total drift at the top of both Walls 19 and 31, and for the operating floor and turbine pedestal for the two acceleration levels. Also shown are the maximum story drifts for each wall defined as a percentage of the wall segment (story) height. In nearly every case, the maximum story drifts occurred in the lowest segment of each wall. These tables also indicate for which cases the turbine pedestal impacted the operating floor. Lastly, the probability of severe shear wall distress is estimated for each trial using the random shear wall distress criteria defined above. Defining P_{F_1} as the probability P_F of severe distress for

Table 6-18

ELASTIC COMPUTED RESPONSE FOR FIGURE 6-28 MEDIAN SPECTRUM SCALED TO AN AVERAGE 5 PERCENT DAMPED SPECTRAL ACCELERATION OF 2.25 G

(A) DRIFTS

Location	Drifts (inches)
Top of pedestal	1.89
Center of operating floor	1.57* (2.00)
Wall 19	
El 140	0.26
El 123	0.16
El 104	0.06
Wall 31	
El 140	0.27
El 119	0.18
El 107	0.12

* The operating floor is actually highly inelastic, so this elastic computed drift is too small. Value in parenthesis is more realistic for the inelastic operating floor.

(B) SHEARS AND MOMENTS

Element	Shear V_R (kips)	$\frac{V_R}{V_Y}$	Moment M_R (kip-ft) x 10^6	$\frac{M_R}{M_Y}$
Turbine pedestal	45,400	0.68	—	—
Wall 19				
Operating Floor (Per Beam)	2,910	2.41	—	—
El 123+	9,520	0.90	0.16	0.70
El 104+	10,240	0.93	0.36	0.91
El 85+	10,820	1.18	0.56	0.79
Wall 31				
Operating Floor (Per Beam)	3,030	2.50	—	—
El 119+	12,330	0.93	0.26	0.40
El 107+	14,560	0.86	0.43	0.60
El 85+	16,460	1.10	0.79	0.76

Table 6–19

NONLINEAR RESULTS FOR MEDIAN STRUCTURAL MODEL AT AN AVERAGE SPECTRAL ACCELERATION OF 3.0 G

Trial No.	Wall 19 Top Drift (inches)	Wall 19 Max Story Drift (%)	Wall 31 Top Drift (inches)	Wall 31 Max Story Drift (%)	Operating Floor Drift (inches)	Turbine Pedestal Drift (inches)	Pedestal Impact Cases[1]	Probability of Severe Wall Distress (%)
1	0.58	0.18	0.60	0.18	3.06	3.22		0
2	1.01	0.35	1.30	0.42	5.15	2.47	Y	0
3	0.36	0.09	0.61	0.18	2.29	1.86		0
4	0.24	0.04	0.29	0.06	1.58	2.20		0
5	0.52	0.17	0.83	0.26	3.54	2.35	Y	0
6	0.79	0.26	0.79	0.26	4.57	2.40	Y	0
7	0.22	0.04	0.43	0.11	1.98	1.58		0
8	0.20	0.04	0.24	0.05	1.81	2.12		0
9	0.89	0.30	1.18	0.38	4.00	2.70	Y	0
10	0.64	0.20	0.70	0.22	2.71	2.45		0
11	0.54	0.16	0.74	0.24	1.70	1.37		0
12	0.36	0.10	0.52	0.17	2.84	2.24		0
13	0.59	0.18	0.58	0.18	3.78	2.81		0
14	0.28	0.06	0.25	0.05	3.18	3.43		0
15	1.39	0.43	1.81	0.61	7.03	4.80	Y	17.9
16	1.03	0.35	1.10	0.37	3.71	2.28		0
17	0.65	0.20	0.89	0.28	5.39	3.50		0
18	1.69	0.53	2.36	0.69	5.77	2.48	Y	46.0
19	0.24	0.04	0.25	0.05	2.57	3.47		0
20	1.62	0.51	2.11	0.59	5.37	3.12	Y	12.7
21	0.25	0.03	0.48	0.15	1.66	1.86		0
22	0.41	0.11	0.62	0.19	3.47	3.07	Y	0
23	0.65	0.21	0.97	0.32	4.18	3.76	Y	0
24	1.13	0.43	0.90	0.29	2.95	1.88		0
25	0.23	0.04	0.62	0.19	3.84	3.88		0

$\Sigma = 76.6$

$$P_F = \frac{\Sigma}{25} = \frac{76.6}{25} = 3.1\%$$

[1]Y indicates that the turbine pedestal did impact the operating floor. For all other cases, no impact occurred.

Table 6-20

NONLINEAR RESULTS FOR MEDIAN STRUCTURAL MODEL AT AN AVERAGE SPECTRAL ACCELERATION OF 6.0 G

	Wall 19		Wall 31					
Trial No.	Top Drift (inches)	Max Story Drift (%)	Top Drift (inches)	Max Story Drift (%)	Operating Floor Drift (inches)	Turbine Pedestal Drift (inches)	Pedestal Impact Cases[1]	Probability of Severe Wall Distress (%)
1	4.8	0.89	5.9	1.46	8.8	6.1		100
2	6.4	0.97	7.7	2.05	14.0*	10.6		100
3	2.1	0.59	4.2	0.97	7.4	4.0		99
4	2.4	0.66	3.1	0.90	7.3	4.6		95
5	3.2	0.84	5.8	1.20	8.6	5.2		100
6	4.6	0.82	6.3	1.50	11.5*	8.1		100
7	1.5	0.48	2.0	0.65	4.4	3.1		31
8	1.3	0.43	1.9	0.64	3.6	3.5	N	27
9	7.2	1.16	9.1	1.89	13.1*	9.7		100
10	2.8	0.71	4.0	1.13	7.3	5.4		100
11	1.5	0.48	1.8	0.57	3.9	2.8	N	9
12	3.6	0.81	5.9	1.45	9.3*	5.9		100
13	3.8	0.74	5.6	1.41	10.7*	7.3		100
14	3.0	0.73	4.2	1.21	8.8*	6.3		100
15	6.6	1.05	9.4	2.08	11.8*	9.9		100
16	6.6	1.00	8.2	1.67	11.8*	8.4		100
17	6.1	1.09	8.1	1.72	10.3	8.3		100
18	10.1	1.82	12.2	2.76	18.5*	15.1		100
19	1.6	0.55	2.8	0.95	5.6	5.3	N	98
20	7.7	1.23	8.8	1.91	14.2*	10.8		100
21	1.7	0.55	2.0	0.65	4.9	4.0		31
22	4.3	0.77	5.3	1.33	10.0*	6.6		100
23	3.8	0.82	5.2	1.45	9.6*	6.2		100
24	4.2	0.81	5.2	1.33	7.9	5.0		100
25	2.2	0.68	5.0	1.28	8.0	6.8		100

$\sum = 2190$

$P_F = \frac{\sum}{25} = \frac{2190}{25} = 87.6\%$

* Relative diaphragm drift exceeds the limits of applicability of the bilinear force-deflection relationship used for the operating floor so that diaphragm drifts are likely to be underpredicted and wall drifts are likely to be overpredicted to some extent for these cases.

[1]N indicates that the turbine pedestal *did not* impact the operating floor. For all other cases there was impact.

Trail i, the median estimate of the probability each average spectral acceleration value is obtained from:

$$P_F = \frac{\sum_{i=1}^{N} P_{F_i}}{N} \qquad (6\text{-}8)$$

where N is the number of trials.

In a similar manner, 50 nonlinear analyses were conducted at average spectral acceleration values of 3.0 g, 4.0 g, and 6.0 g, incorporating the randomly selected structure damping, stiffness, and strength ratios shown in Table 6-16. These analyses include both input motion randomness variability and structural property uncertainty. Table 6-21 tabulates the maximum story drift as a percentage of the wall segment height for both walls 19 and 31. Again, in nearly every case, the maximum story drifts occurred in the lowest segment of each wall and again the composite probability of severe wall distress for each trial was estimated based upon the median drift limit of 0.7 percent and composite $\beta c = 0.335$. The overall composite probability of severe wall distress is computed using equation (6-8) for each average spectral acceleration level as shown in Table 6-21. Those trials in which turbine pedestal and operating floor impact occurred are also indicated.

The overall probability estimates for each case studied (randomness only at an average spectral acceleration of 3.0 g and 6.0 g, and randomness plus uncertainty at an average spectral acceleration of 3.0 g, 4.0 g, and 6.0 g) are presented in Tables 6-19 through 6-21. These results were then fit by a "best-fit" lognormally distributed fragility estimate using linear regression (least-square error fitting). The result is a lognormally distributed fragility estimate defined in terms of the median, $\check{\bar{S}}_a$, and logarithmic standard deviations for randomness variability, β_R, composite variability, β_C, and uncertainty variability, β_U. The high-confidence-low-probability-of-failure (HCLPF) capacity, defined as a 95 percent confidence of less than 5 percent probability of failure, is calculated from:

$$\text{HCLPF } \bar{S}_a = \check{\bar{S}}_a\, e^{-1.65(\beta_R + \beta_U)} \qquad (6\text{-}9)$$

Thus, the turbine building fragility estimate becomes:

$\check{\bar{S}}_a = 4.59$ g

$\beta_C = 0.37$ (from randomness and uncertainty runs)

$\beta_R = 0.23$ (from randomness only runs)

$\beta_U = (0.37^2 - 0.23^2)^{1/2} = 0.29$

$$\text{HCLPF } \bar{S}_a = 4.59\, e^{-1.65\,(.23\, +\, .29)} = 1.95 \text{ g}$$

As noted earlier, three structural response factors were not included in the nonlinear time-history analyses and their effects were added by means of the separation-of-variables approach.

1) *Modeling:* Only a single mathematical model was used. Structure properties were varied, but the model was not varied. The model which was used is judged to be median-centered. It is further judged that modeling uncertainty is about $\beta_{U_M} = 0.15$, which is equivalent to stating that the 95 percent nonexceedance probability responses near the base of the shear walls are estimated to be as much as 1.28 times those reported herein if differing models had been used.

2) *Earthquake Component Variation:* Within this study, the fragility of east/west shear walls were defined in terms of average spectral acceleration associated with east/west ground motions. However, in the seismic probabilistic risk assessment, the seismic hazard was defined in terms of the average horizontal component ($\bar{S}_a$). The east/west component is expected to have the same median value as the average horizontal component ($\check{F}_{DIR} = 1.0$); however, the random variability ($\beta_{R_{DIR}}$) for the east/west component, given an average horizontal component spectral acceleration, is estimated to be about 0.12.

3) *Incoherence of Ground Motion:* At any instant in time, the ground acceleration is not the same at every location under the turbine building foundation. The soil/structure

Table 6-21

NONLINEAR RESULTS FOR UNCERTAIN STRUCTURAL PROPERTIES MODEL

Trial No.	Average Spectral Acceleration 3.0 g: Max Story Drift (%) Wall 19	Max Story Drift (%) Wall 31	Prob. Severe Distress (%)	Pedestal Impact (1)	Average Spectral Acceleration 4.0 g: Max Story Drift (%) Wall 19	Max Story Drift (%) Wall 31	Prob. Severe Distress (%)	Pedestal Impact (1)	Average Spectral Acceleration 6.0 g: Max Story Drift (%) Wall 19	Max Story Drift (%) Wall 31	Prob. Severe Distress (%)	Pedestal Impact (1)
1	0.19	0.15	0	Y	0.29	0.64	39.4	Y	0.94	1.27	96.2*	Y
2	0.45	0.37	9.3		1.10	0.65	91.1	Y	1.76	1.62	99.7*	Y
3	0.19	0.23	0		0.40	0.46	10.6	Y	1.23	0.99	95.4	Y
4	0.04	0.05	0		0.18	0.25	0		0.63	0.86	72.9	Y
5	0.02	0.04	0		0.06	0.11	0		0.37	0.60	67.4	Y
6	0.19	0.33	1.3	Y	0.47	0.52	18.7	Y	1.68	0.92	99.5*	Y
7	0.03	0.09	0		0.16	0.29	0.4		0.42	0.66	57.1	
8	0.02	0.05	0		0.05	0.09	0		0.27	0.44	8.2	
9	0.06	0.31	0.8	Y	0.24	0.40	4.7	Y	1.15	1.09	93.1*	Y
10	0.03	0.06	0		0.06	0.20	0		0.47	0.66	42.9	Y
11	0.22	0.19	0		0.75	0.35	58.3		0.72	0.86	72.9	Y
12	0.14	0.16	0		0.29	0.44	8.2		0.53	0.97	83.4	Y
13	0.42	0.71	51.6	Y	0.67	1.24	95.6	Y	1.20	1.97	99.9*	Y
14	0.09	0.06	0		0.28	0.40	4.7	Y	0.53	0.90	77.3	Y
15	0.72	0.96	82.6		1.18	1.84	99.8		1.97	2.81	100.0	Y
16	0.45	0.63	37.8		0.87	0.99	84.8	Y	1.44	1.51	98.9	Y
17	0.51	0.83	69.5	Y	0.73	0.96	82.6	Y	1.11	1.42	98.3	Y
18	0.45	0.26	9.3	Y	0.38	0.79	63.7*	Y	1.46	1.15	98.6*	Y
19	0.02	0.04	0		0.20	0.17	0		0.45	0.71	51.6	Y
20	0.40	0.46	10.6	Y	0.45	0.70	50.0	Y	0.80	1.21	94.8*	Y
21	0.24	0.31	0.8		0.45	0.45	9.3	Y	0.69	1.02	86.9	Y
22	0.33	0.52	18.7	Y	0.58	0.74	56.8	Y	0.84	1.25	95.8	Y
23	0.01	0.01	0		0.02	0.03	0		0.24	0.35	2.0	Y
24	0.05	0.24	0		0.32	0.36	2.3	Y	0.86	0.78	72.9*	Y
25	0.31	0.43	7.4	Y	0.49	0.82	68.1	Y	0.71	1.15	93.1	Y

(1) Y indicates that turbine pedestal did impact the operating floor. For all other cases, no impact occurred.

* Relative diaphragm drift exceeds the limits of applicability of the bilinear force-deflection relationship used for the operating floor so that wall drifts and probability of severe wall distress are likely to be overpredicted to some extent for these cases.

Table 6-21 *(Continued)*

NONLINEAR RESULTS FOR UNCERTAIN STRUCTURAL PROPERTIES MODEL

Trial No.	Average Spectral Acceleration 3.0 g: Max Story Drift (%) Wall 19	Wall 31	Prob. Severe Distress (%)	Pedestal Impact (1)	Average Spectral Acceleration 4.0 g: Max Story Drift (%) Wall 19	Wall 31	Prob. Severe Distress (%)	Pedestal Impact (1)	Average Spectral Acceleration 6.0 g: Max Story Drift (%) Wall 19	Wall 31	Prob. Severe Distress (%)	Pedestal Impact (1)
26	0.17	0.41	5.6		0.69	0.85	71.9	Y	1.36	1.76	99.7	Y
27	0.37	0.17	2.9		0.30	0.40	4.7	Y	0.57	0.76	59.9	Y
28	0.18	0.20	0		0.43	0.46	10.6		0.79	1.12	65.5	Y
29	0.05	0.18	0		0.06	0.13	0		0.35	0.43	7.4	Y
30	0.06	0.11	0		0.25	0.27	0.2*		0.87	1.07	89.8*	Y
31	0.14	0.45	9.3	Y	0.79	1.37	97.7	Y	1.71	2.54	100.0*	Y
32	0.03	0.12	0		0.17	0.29	0.4		0.41	0.62	35.9	
33	0.33	0.30	1.3		0.40	0.51	17.1		0.50	0.66	42.9	
34	0.08	0.09	0	Y	0.35	0.40	4.7	Y	0.80	1.11	91.6*	Y
35	0.12	0.15	0		0.27	0.39	4.0		0.70	0.67	50.0	
36	0.19	0.23	0		0.30	0.35	2.0		0.51	0.61	34.1	
37	0.16	0.28	0.3		0.51	0.90	77.3	Y	1.31	1.99	99.9*	Y
38	0.25	0.36	2.4	Y	0.51	0.99	84.8	Y	2.11	1.90	100.0*	Y
39	0.04	0.05	0		0.13	0.27	0.2		0.91	1.17	93.7*	Y
40	0.75	1.10	91.1	Y	1.20	1.70	99.6	Y	1.70	2.20	100.0*	Y
41	0.31	0.43	7.4	Y	0.53	0.90	77.3	Y	1.03	1.41	98.2*	Y
42	0.67	0.98	84.1	Y	1.50	1.72	99.6	Y	2.86	3.14	100.0*	Y
43	0.41	0.67	44.8	Y	0.74	1.20	94.6	Y	1.69	2.30	100.0*	Y
44	0.08	0.06	0		0.14	0.20	0		0.35	0.46	10.6	Y
45	0.11	0.13	0		0.22	0.39	4.0		0.51	0.67	44.8*	
46	0.06	0.15	0		0.22	0.32	1.0		0.57	0.76	59.9	
47	0.35	0.25	1.9		0.80	0.92	79.4	Y	1.23	1.42	98.3*	Y
48	0.15	0.36	2.4		0.51	0.79	63.7	Y	0.59	1.15	93.1*	Y
49	0.27	0.29	0.4	Y	0.58	0.51	28.8	Y	1.14	1.73	99.7*	Y
50	0.11	0.22	0	Y	0.43	1.04	88.1	Y	1.49	2.00	100.0	Y
		Σ =	553.6			Σ =	1860.8			Σ =	3833.8	

P_F = 553.6/50 = 11.1% P_F = 1860.8/50 = 37.2% P_F = 3833.8/50 = 76.7%

(1) Y indicates that turbine pedestal did impact the operating floor. For all other cases, no impact occurred.

* Relative diaphragm drift exceeds the limits of applicability of the bilinear force-deflection relationship used for the operating floor so that wall drifts and probability of severe wall distress are likely to be overpredicted to some extent for these cases.

interaction analysis considered this aspect for Diablo Canyon, and it was estimated that east/west shear wall responses are reduced by a median factor of $\check{F}_{GMI}$ = 1.06, with estimated randomness $\beta_{R_{GMI}}$ = 0.02, and uncertainty $\beta_{U_{GMI}}$ = 0.06.

Table 6-22 includes the effects of these three additional parameters on the fragility estimate for the turbine building. The final fragility estimate for the turbine building for use in the seismic probabilistic risk assessment is:

$\check{\overline{S}}_a$ = 4.87 g

β_C = 0.26

β_R = 0.33

HCLPF $\overline{S}_a$ = 1.84 g

The median and HCLPF capacities are in terms of an average 5 percent damped spectral acceleration averaged over the 3 to 8.5 hertz range.

Results and Conclusions

The fragility evaluation established that the Diablo Canyon safety-related structures and equipment that are important to evaluating the probability of core damage, generally have high median seismic capacities relative to the median reference ground motion. In addition, the important structures and equipment have HCLPF capacities that are generally in excess of 2.25 g average spectral acceleration. (The exceptions are noted below.)

The following summarizes the findings of the fragility evaluation with regard to several categories of structures and equipment, and highlights those items that may contribute to seismic risk due to relatively low demonstrated capacity. Only the salient information that is specific to the Diablo Canyon fragility evaluation is summarized. Details are included in the comprehensive technical reports (Kennedy, 1988; Kipp, 1988) where descriptions of the methods used, example calculations, interpretation of the fragilities, and failure consequences are discussed. Again, it should be noted that the reported median fragility capacities, $\check{\overline{S}}_a$, are in terms of the 5 percent damped average spectral acceleration averaged over the 3 to 8.5 hertz range.

STRUCTURAL FRAGILITY RESULTS

The fragility parameters associated with the important structures are presented in Table 6-23. The fundamental frequency of the structure, failure mode, fragility parameters ($\check{\overline{S}}_a$, β_R, and β_U), and HCLPF capacity are included in the table.

Table 6-22
TURBINE BUILDING FRAGILITY ESTIMATE INCORPORATING ADDITIONAL VARIABLE PARAMETERS

	Median $\check{\overline{S}}_a$, (g) or $\check{F}$	Randomness β_R	Uncertainty β_U	HCLPF $\overline{S}_a$ (g)
Nonlinear Time History Results	4.59 g	0.23	0.29	1.95 g
Modeling	1.0	—	0.15	—
Directional Effects	1.0	0.12	—	—
Incoherence of Ground Motion	1.06	0.02	0.06	—
Fragility Estimate	4.87 g	0.26	0.33	1.84 g

Table 6-23

DIABLO CANYON STRUCTURE FRAGILITIES

(Based on hazard defined over 3 to 8.5 hertz range.)

Structure	Fundamental Frequency Hertz	Failure Mode	Spectral Acceleration Capacity $\bar{S}_a$ (g)	β_R	β_U	HCLPF (g)
Containment Building	4.1	Exterior Shell Shear	8.42	0.26	0.30	3.34
Concrete Internal Structure	8.9	Internal Structure Shear	6.91	0.20	0.31	2.98
Intake Structure	23.3	North Wall Shear	8.55	0.28	0.31	3.23
Auxiliary Building	8.2	North/South Shearwalls	5.79	0.21	0.26	2.66
Turbine Building	8.6	Shear Wall, Column 31	4.87	0.26	0.33	1.84
	9.0	Block Wall	>10.0	—	—	—
Refueling Water Storage Tank	7.6	Concrete/Bedrock Flexure	9.92	0.29	0.36	3.40
Condensate Storage Tank	—	Comparison to RWST	>10.0	—	—	—
DG Fuel-Oil Storage Tank	Buried	Rupture	>10.0	—	—	—
Auxiliary Saltwater Piping	Buried	Rupture	9.23	0.18	0.21	4.85

The containment building, concrete internal structure and intake structure all have very high median and HCLPF capacities, and thus contribute very little to overall Plant risk. The auxiliary building fragility evaluation shows median and HCLPF capacities of 5.79 g and 2.66 g, respectively, which, although not as high as the three concrete structures identified above, are sufficiently high so as not to contribute significantly to Plant seismic risk.

The turbine building has the lowest median seismic capacity of all the civil structures. The median spectral acceleration capacity for the turbine building is estimated to be 4.87 g, based upon a shear-wall failure due to east/west response, with randomness and uncertainty variabilities of 0.26 and 0.33, respectively. The resulting HCLPF spectral acceleration capacity is 1.84 g. Because the turbine building houses the diesel generators, the component cooling water heat exchanger, and the 4160 V (vital) electrical system, the potential for severe distress of the turbine building is likely to be a significant contributor to overall Plant risk. However, it should be noted that in searching for actual earthquake records for use in the turbine building nonlinear analysis, very few were found that met Criterion 4 (see page 6-64), related to high spectral acceleration in the 3.0 to 8.5 hertz range. This fact alone demonstrates the lack of seismic vulnerability of the Diablo Canyon turbine building. Two of strongest ground motions that have ever been recorded anywhere in the world (Tabas and Pacoima Dam) only have a slight potential of causing measureable damage to the turbine building. It should be further noted that the fragility estimate of the turbine building was heavily influenced by the selection and equal weighting of the 25 time histories used in the study. The highest probability of severe distress was related to those records that required substantial frequency-dependent modifications to scale them up to the level required for the Diablo Canyon site. In contrast, those very strong motion records requiring only minor frequency-independent scaling resulted in relatively small potential for severe distress. Thus, the turbine building fragility estimate is likely to be conservatively biased. Particularly, both β_R and β_U are likely to be too large.

The main outdoor storage tanks for refueling water and condensate, the buried diesel generator fuel oil storage tank, and buried piping all have very high seismic capacities, and thus are negligible contributors to overall Plant risk.

EQUIPMENT FRAGILITY RESULTS

Table 6-24 contains fragility descriptions for all the equipment that was included in the probabilistic risk assessment. The table includes the component location, frequency, method of seismic qualification, critical failure mode, sources of information, and fragility parameters ($\overline{S}_g$ β_R, and β_U). As a means of reference, the resulting HCLPF capacity is also listed. Fragility derivations were conducted for each of the components and are reported for those items that have a median spectral ground acceleration capacity less than 10.0 g. Based upon a review of the seismic capacity of a sample of such equipment, it was determined that equipment that possessed median spectral ground acceleration capacities greater than about 10.0 g also possessed HCLPF capacities in excess of 3.0 g. Due to the fact that there is an extremely low frequency of occurrence of a 3.0 g average spectral acceleration earthquake at the Diablo Canyon site, and that other lower capacity equipment would govern the Plant seismic risk, it was judged that the high capacity equipment would not contribute to the overall Plant risk. Therefore, detailed fragility descriptions were not included for components having median capacities greater than 10.0 g based on the capacity factor alone. Equipment in this category are labeled with a ">10.0" in the spectral acceleration capacity column.

The seismic capacity for most of the safety-related equipment items is relatively high with respect to the median reference ground motion. In all cases, the equipment fragilities were based upon Plant-specific component analyses or qualification test data and involved the use of very little generic information. Even for generic component categories, the fragilities were based upon the review of sample calculations from specific Diablo Canyon qualification analyses. The piping and major equipment components associated with the reactor coolant loop have generally high capacities with respect to the reference ground-motion spectra demands. The most critical component is the steam generator, which has a median spectral acceleration capacity of 6.96 g, based upon the failure of the upper lateral support due to the formation of a plastic hinge in the ring band. The fundamental frequency of the steam generator is 8.8 hertz, which corresponds closely to the frequency of the concrete internal structure. Excessive movement of the steam generator after loss of the upper support is assumed to result in rupture of the main steam system piping and other attached lines.

The components for the major balance-of-Plant safety systems, such as the residual heat removal, safety injection, and component cooling water systems were found to have high capacities. The failure modes for these components were generally associated with anchorage. Pumps, piping, and valves are estimated to have median capacities greater than 7.7 g and HCLPF capacities greater than 3.4 g. Similarly, tanks and vessels have high capacities, with median capacities and HCLPF values estimated to be greater than 6.7 g and 3.0 g, respectively. The component cooling water heat exchanger was found to be the weakest of all mechanical system components, based upon the failure of the longitudinal strut anchor bolts. The median seismic capacity of the heat exchanger was estimated to be 6.31 g, with randomness and uncertainty variabilities of 0.27 and 0.28, respectively, providing a HCLPF capacity of 2.55 g.

Components of the diesel generator system exhibit high seismic capacities. For the diesel generator itself, fragility is based upon failure of the skid anchor bolts and seismic stays, which occur at a spectral acceleration of 7.79 g. The most critical element of the diesel generator system is the diesel generator control panel which was computed to have median and HCLPF capacities of 4.55 g and 2.24 g, respectively, based upon a generic structural failure. This fragility is based upon the seismic qualification dynamic testing of the cabinet and, as such, is not based on actual fragility testing leading to an actual failure state. Because the control panel is situated on the basemat of the turbine building, the demand acceleration for the seismic qualification test was relatively low. Thus, the reported fragility may be

Table 6-24

DIABLO CANYON EQUIPMENT FRAGILITIES

(Based on hazard defined over 3 to 8.5 hertz range.)

System and Component	[redacted]	Fundamental Frequency	Method of Seismic Qualification	Failure Mode	Information Source	Spectral Acceleration Capacity $\check{S}_a$(g)	β_R	β_U	HCLPF (g)
NUCLEAR STEAM SUPPLY									
Reactor Pressure Vessel		12-14 Hz (H)	Dynamic Analysis	Support Pin Shear	W Summary Data	8.71	0.25	0.33	3.34
Reactor Internals		16-20 Hz (V)	Dynamic Analysis	Lower Core Plate	W Summary Data	10.54	0.40	0.26	3.55
Steam Generators		9 Hz (H)	Dynamic Analysis	Upper Lateral Support	W Summary Data	6.96	0.31	0.29	2.55
Pressurizer		15 Hz (H)	Dynamic Analysis	Seismic Support Lug	W Summary Data	11.46	0.31	0.44	3.33
Pressurizer Safety Valves		Flexible Piping	Static Analysis/Test	Generic Function	M397, M401	>10.0	-	-	-
Power Operated Relief Valves		Flexible Piping	Static Analysis/Test	Generic Function	M397, M401	7.62	0.30	0.42	2.32
Reactor Coolant Pumps		7 Hz (H)	Dynamic Analysis	Lower Motor Stand	M355, M428, M429	8.82	0.37	0.32	2.83
Control Rod Drives		7-10 Hz (H)	Dynamic Analysis	Head Adapter Yield	W Summary Data	11.71	0.41	0.34	3.40
NSSS Piping		7-9 Hz (H&V)	Dynamic Analysis	Rupture	W Summary Data	>10.0	-	-	-
RESIDUAL HEAT REMOVAL									
RHR Pumps		Flexible Piping	Dynamic Analysis	Pump Hold Down Bolts	W Summary Data	8.31	0.33	0.22	3.35
RHR Heat Exchangers		12 Hz (H)	Static Analysis	Anchor Bolts & Upper Lateral Support	W Summary Data, M462, M474	8.09	0.24	0.27	3.48
SAFETY INJECTION									
SI Accumulators		23-34 Hz (H)	Static Analysis	Anchor Studs	W Summary Data, M316	10.01	0.29	0.19	4.53
SI Pumps		>33 Hz (H)	Static Analysis	Pump Hold Down Bolts	W Summary Data	10.94	0.34	0.18	4.64
Boron Injection Tank		15-17 Hz (H)	Static Analysis	Anchor Bolts	W Summary Data	8.46	0.27	0.19	3.96
COMPONENT COOLING WATER									
CCW Pumps		Flexible Piping	Static Analysis	Pump Hold Down Bolts	M006, M007, M318	8.53	0.29	0.21	3.74
CCW Heat Exchangers		13 Hz (H)	Dynamic Analysis	Longitudinal Strut Bolts	M008, M336, M475	6.31	0.27	0.28	2.55
CCW Surge Tank		32 Hz (H)	Static Analysis	Seismic Lateral Brace	M319	7.22	0.33	0.22	2.91
CHEMICAL AND VOLUME CONTROL									
Charging Pumps (centrifugal)		>33 Hz (H)	Static Analysis	Motor Hold Down Bolts	W Summary Data	10.16	0.31	0.19	4.45
Charging Pumps (reciprocal)		>33 Hz (H)	Static Analysis	Pump Hold Down Bolts	W Summary Data	>10.0	-	-	-
AUXILIARY SALTWATER									
Auxiliary Saltwater Pumps		43 Hz (H)	Static Analysis	Pump Mounting Bolts	M009	>10.0	-	-	-
CONTAINMENT SPRAY									
CS Pumps		>33 Hz (H)	Static Analysis	Foundation Bolts	W Summary Data	8.65	0.29	0.20	3.85
Spray Additive Tank		24 Hz (H)	Static Analysis	Support Pad/Shell	W Summary Data	6.78	0.30	0.18	3.07

Table 6-24 *(Continued)*

DIABLO CANYON EQUIPMENT FRAGILITIES

(Based on hazard defined over 3 to 8.5 hertz range.)

System and Component	[redacted]	Fundamental Frequency	Method of Seismic Qualification	Failure Mode	Information Source	Spectral Acceleration Capacity			
						$\bar{S}_a$(g)	β_R	β_U	HCLPF (g)
MAIN STEAM									
MS Isolation Valves	[redacted]	Flexible Piping	Dynamic Analysis/Test	Actuator Support	M067, M463, M469	>10.0	-	-	-
MS Safety Valves	[redacted]	Flexible Piping	Dynamic Analysis/Test	Generic Function	M397	>10.0	-	-	-
MS PORV'S	[redacted]	Flexible Piping	Dynamic Analysis/Test	Generic Function	M397	11.51	0.34	0.38	3.51
AUXILIARY FEEDWATER	[redacted]								
AFW Pumps (Motor Driven)	[redacted]	42 Hz (H)	Static Analysis	Pump Hold Down Bolts	M320A	>10.0	-	-	-
AFW Pumps (Turbine Driven)	[redacted]	43 Hz (H)	Static Analysis	Pump Hold Down Bolts	M320, M321	7.71	0.29	0.21	3.38
DIESEL GENERATOR	[redacted]								
D.G. Fuel Oil Day Tank	[redacted]	10 Hz (V)	Static Analysis	Bottom Plate Rupture.	M323	>10.0	-	-	-
D.G. Fuel Oil Pumps/Filters	[redacted]	Flexible Piping	Static Analysis	Filter Anchor Bolts	M324, M326	8.33	0.27	0.23	3.65
D.G. Fuel Oil Shutoff Valve	[redacted]	Flexible Piping	None	Fusible Link	Data Base	>10.0	-	-	-
D.G. Air Start Compressor	[redacted]	>33 Hz (H)	Generic Anchorage Analysis	Hold Down Bolts	M323	>10.0	-	-	-
D.G. Air Start Receiver	[redacted]	26 Hz (H)	Dynamic Analysis	Hold Down Bolts	M329	>10.0	-	-	-
Diesel Generators	[redacted]	17 Hz (H)	Dynamic Analysis	Skid Anchor Bolts	M423-M426	7.79	0.26	0.20	3.64
D.G. Radiator/Water Pump	[redacted]	17 Hz (H)	Dynamic Analysis	Anchor Bolting	M323	8.78	0.29	0.24	3.66
D.G. Inlet Silencer/Air Filter	[redacted]	Flexible Piping	Dynamic Analysis	Filter Support Rod Weld	M271, M449	>10.0	-	-	-
D.G. Excitation Cubicle	[redacted]	13 Hz (H)	Test	Structural	M346, M364	7.40	0.29	0.35	2.57
D.G. Control Panel	[redacted]	8 Hz (H)	Test	Chatter	M347, M364, M464, M482	7.77	0.25	0.14	4.08
	[redacted]		Test	Structural		4.55	0.30	0.13	2.24
D.G. Main Lead Terminal/Box	[redacted]	10 Hz (H)	Static Analysis	Attachment-Fillet Weld	M348	>10.0	-	-	-
CONTAINMENT BUILDING VENTILATION	[redacted]								
Containment Fan Cooler	[redacted]	23 Hz (H)	Dynamic Analysis	Foot Plate/Embed. Weld	M399, M499, M420, M421, M448	8.10	0.31	0.33	2.82
CONTROL ROOM VENTILATION	[redacted]								
Supply Fans	[redacted]	>33 Hz (H)	Static Analysis	Support Bolting	M056	9.79	0.33	0.24	3.82
AC Units/Compressors	[redacted]	>33 Hz (H)	Static Analysis	Anchor Bolt	M288, M312	>10.0	-	-	-
Control Cabinets	[redacted]	21 Hz (H)	Test	Structural	M455	>10.0	-	-	-
480V SWITCHGEAR/INVENTER/DC SWITCHGEAR/SPREADING ROOM VENTILATION									
Supply/Return Fans	[redacted]	>33 Hz (H)	Static Analysis	Expansion Anchor	M310	11.16	0.33	0.30	3.95
Backdraft and Shut-Off Dampers	[redacted]	>33 Hz (H)	Static Analysis	Structural	M388	>10.0	-	-	-

Table 6-24 *(Continued)*

DIABLO CANYON EQUIPMENT FRAGILITIES

(Based on hazard defined over 3 to 8.5 hertz range.)

System and Component	[redacted]	Fundamental Frequency	Method of Seismic Qualification	Failure Mode	Information Source	Spectral Acceleration Capacity $\bar{S}_a$(g)	β_R	β_U	HCLPF (g)
4160V (VITAL) ELECTRIC POWER									
Switchgear	[redacted]	7 Hz (H)	Test	Chatter	M049, M315, M356,	3.53	0.35	0.25	1.31
			Static Analysis	Guide Rod Bending	M373, M377-380, M482	7.44	0.31	0.25	2.95
Potential Transformers (Bus F)		21 Hz (H)	Static Analysis	Support Leg/Embed. Weld	M049, M375, M416, M450	10.83	0.31	0.38	3.47
(Bus G & H)		35 Hz (H)	Static Analysis	Support Leg/Embed. Weld	M049, M375, M416, M450	>10.0	-	-	-
Safeguard Relay Panel		11 Hz (H)	Static Analysis	Anchor Welds	M012, M373, M414, M430	10.76	0.34	0.36	3.39
125V DC ELECTRIC POWER									
Batteries		>33 Hz (H)	Test	Structural	M050, M054, M364	6.04	0.30	0.18	2.74
Battery Racks		>33 Hz (H)	Static Analysis	Longitudinal End Restraint	M013, M032, M050, M207	11.91	0.26	0.22	5.40
Battery Chargers		12 Hz (H)	Test	Structural	M054, M364, M453, M462	9.93	0.34	0.40	2.93
Switchgear/Breaker Panels		7 Hz (H)	Test	Structural	M014, M051, M364	6.67	0.35	0.28	2.36
120V AC ELECTRIC POWER									
Instrument Breaker Panels		>20 Hz (H)	Static Analysis	Slip-Nut Failure	M051A	>10.0	-	-	-
Inverters		5 Hz (H)	Test	Structural	M015, M016, M355, M415, M436, M451, M467	6.82	0.31	0.24	2.75
480V (VITAL) ELECTRIC POWER									
4160V/480V Transformers		3 Hz (H)	Static Analysis	Structural	M052, Walkdown	5.34	0.28	0.20	2.42
Breaker Cabinets (Load Centers)		13 Hz (H)	Static Analysis	Anchor Stitch Weld	M017, M364	>10.0	-	-	-
Auxiliary Relay Panel		29 Hz (H)	Test	Structural	M315, M364	7.25	0.28	0.15	3.57
CONTROL ROOM									
Main Control Boards		>33 Hz (H)	Test	Switch Function	W Summary Data,	>10.0	-	-	-
			Dynamic Analysis	Structural	M456, M482	7.77	0.31	0.27	2.98
Hot Shutdown Panel		>33 Hz (H)	Test	Switch Function	M317, M383, M342,	7.60	0.27	0.25	3.22
			Static Analysis	Structural	M479, M482	7.27	0.30	0.14	3.52
Auxiliary Safeguards Cabinet		9-13 Hz (H)	Test	Structural	M317, M354, M359	>10.0	-	-	-

Table 6-24 *(Continued)*

DIABLO CANYON EQUIPMENT FRAGILITIES

(Based on hazard defined over 3 to 8.5 hertz range.)

System and Component	[redacted]	Fundamental Frequency	Method of Seismic Qualification	Failure Mode	Information Source	Spectral Acceleration Capacity $\check{S}_a$(g)	β_R	β_U	HCLPF (g)
NSSS CONTROL									
Process Control and Protection System	[redacted]	8-10 Hz (H)	Test	Structural	M317, M355	10.78	0.39	0.28	3.57
Solid State Protection System	[redacted]	8-11 Hz (H)	Test	Structural	M317, M355	12.63	0.37	0.28	4.32
Reactor Trip Switchgear	[redacted]	8 Hz (H)	Test	Structural	M317, M354	7.90	0.30	0.26	3.14
Resistance & Temperature Detectors	[redacted]	Not Given	Test	Structural	M345	>10.0	-	-	-
Pressure & ΔP Transmitters	[redacted]	>33 Hz (H)	Test	Structural	M341	8.93	0.27	0.20	4:11
MISCELLANEOUS COMPONENTS	[redacted]								
Auxiliary Relay Rack	[redacted]	12-20 Hz (H)	Static Analysis	Anchor Bolts	M317, M350, M359	>10.0	-	-	-
Local Starter Boards	[redacted]	18 Hz (H)	Test	Structural	M454	>10.0	-	-	-
Molded Case Circuit Breakers	[redacted]	>33 Hz (H)	Test	Structural	M476	>10.0	-	-	-
Valve Limit Switches	[redacted]	>33 Hz (H&V)	Test	Generic Function	M344	>10.0	-	-	-
Impulse Lines	[redacted]	5-20 Hz (H&V)	None	Rupture From Impact	Data Base	7.09	0.28	0.32	2.63
Containment Purge Valves	[redacted]	>33 Hz (H&V)	Static Analysis	Actuator Attach. Bolts	M432	>10.0	-	-	-
GENERIC COMPONENTS	[redacted]								
Off-Site Power 230KV	[redacted]	Flexible	None	Generic Failure	Data Base	1.69	0.24	0.20	0.82
500KV	[redacted]					0.81	0.24	0.20	0.39
Penetrations/Penetration Boxes	[redacted]	24 Hz (H)	Test	Generic Structural	M054	7.38	0.31	0.27	2.83
BOP Piping and Supports	[redacted]	Flexible Piping	Dynamic Analysis	Generic Support	M020, M381	11.03	0.40	0.39	3.00
Hand, Relief, Solenoid, & Check Valves	[redacted]	Flexible Piping	Dynamic Analysis	Generic Function	Data Base	>10.0	-	-	-
Air and Motor Operated Valves	[redacted]	Flexible Piping	Dynamic Analysis	Generic Function	M067, M401	17.10	0.35	0.60	3.57
Cable Trays and Supports	[redacted]	Flexible Trays	Static Analysis	Generic Support	M209-M213	>10.0	-	-	-
HVAC Ducting and Supports	[redacted]	Flexible Ducting	Static Analysis	Generic Support	M214-M218	9.78	0.35	0.48	2.49

excessively conservative; however, it is difficult to justify higher values based upon qualification test data alone.

The fragility description of electrical cabinets was based upon the documented results of their corresponding seismic qualification tests. The loss of function due to acceleration-sensitive failures (for example, relay chatter), when important, and the loss of function due to generic structural failure were generally based upon a conservative factor applied to the qualification acceleration test level. The structural capacities of the important electrical components are high, and have adequate factors of safety. The weakest of the electrical elements is the 4160 V/480-V transformer which has median and HCLPF capacities estimated to be 5.34 g and 2.42 g, respectively.

Loss of function due to acceleration-sensitive failures were considered to be of sufficient importance to warrant fragility estimates for the following electrical cabinets:

Diesel Generator Control Panel

4-kV Switchgear

4-kV Safeguard Relay Panel

Main Control Boards

Hot Shutdown Panel

Except for the 4-kV switchgear, the chatter failure mode capacities, when evaluated by means of relay-specific Generic Equipment Ruggedness Spectra, are sufficiently high so as not to contribute significantly to Plant seismic risk. The 4-kV switchgear, however, contains a large number of overcurrent relays, which are primarily sensitive to vertical excitation. The median and HCLPF chatter failure capacities were estimated to be 3.53 g and 1.31 g, respectively. The 4-kV switchgear chatter failure mode is recoverable by operator action, and the probabilities associated with operator action were included in the model of the system.

The main components of the various critical safety-related ventilation supply systems have relatively high seismic capacities. The failure of the heating, ventilating, and air conditioning (HVAC) ducting is based upon the generic failure of the ducting supports. The supports have a median spectral acceleration capacity of 9.78 g. Bending or slight buckling of the HVAC ducts is likely at accelerations less than the support capacity, but is not expected to result in failure of the ventilation systems.

The fragility of offsite power is based upon the failure of ceramic insulators, transformers, and circuit breakers, and is generated from a data base pertaining to the performance of power transmission components for both nuclear and non-nuclear power stations in real earthquakes. Review of these data shows clear evidence of superior performance of the lighter 230-kV systems over the 500-kV systems. This is particularly true where live-tank, air-blast circuit breakers are used in the 500-kV systems. The median capacities for the 230-kV and 500-kV switchyards are 1.69 g and 0.81 g, respectively.

Several items were treated in a generic manner due to the quantity of such items in the Plant. These included balance-of-Plant piping, air and motor-operated valves, cable trays, and heating, ventilating, and air conditioning ducting and supports. In general, these had relatively high capacities, with median spectral acceleration capacities of approximately 6.0 g or greater. The basis for the fragility of balance-of-Plant piping is generic failure of the piping supports.

Conclusion

In summary, based upon the estimated fragility capacities of the important safety-related structures and equipment, it is judged that the largest individual contributor to seismic risk is the turbine building, because the probable loss of function of the 4-kV switchgear due to acceleration-sensitive failure is recoverable by operator action. Several other components constitute much lesser contributors to overall Plant risk, and no other structures contribute to the seismic risk.

Pages 6.83 through 6.220 of Attachment 6 to PG&E Letter DCL-11-097
have been redacted from public disclosure

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Chapter 7 of the 1988 Long Term Seismic Program Final Report

Chapter 7
DETERMINISTIC EVALUATIONS
To Partially Address
Element 4 of the License Condition

ELEMENT 4 OF THE LICENSE CONDITION

PG&E shall assess the significance of conclusions drawn from the seismic reevaluation studies in Elements 1, 2, and 3, utilizing a probabilistic risk analysis and deterministic studies, as necessary, to assure adequacy of seismic margins.

OBJECTIVE

The objective of the deterministic evaluations was to augment the probabilistic risk assessment to assure the adequacy of Plant seismic margins, as specified by Element 4 of the license condition. This objective was achieved by:

- Comparing Plant responses as calculated from the site-specific ground motions due to the maximum earthquake on the Hosgri fault zone with those used as the bases for Plant design or for the earlier Hosgri evaluation, as appropriate (note that we will use the term "qualification basis" to mean the combination of the original design basis and the subsequent Hosgri evaluation basis).
- Assessing the Plant capacity margins over the demands (Plant responses) resulting from the 84th percentile ground motions due to the maximum magnitude earthquake.

SCOPE

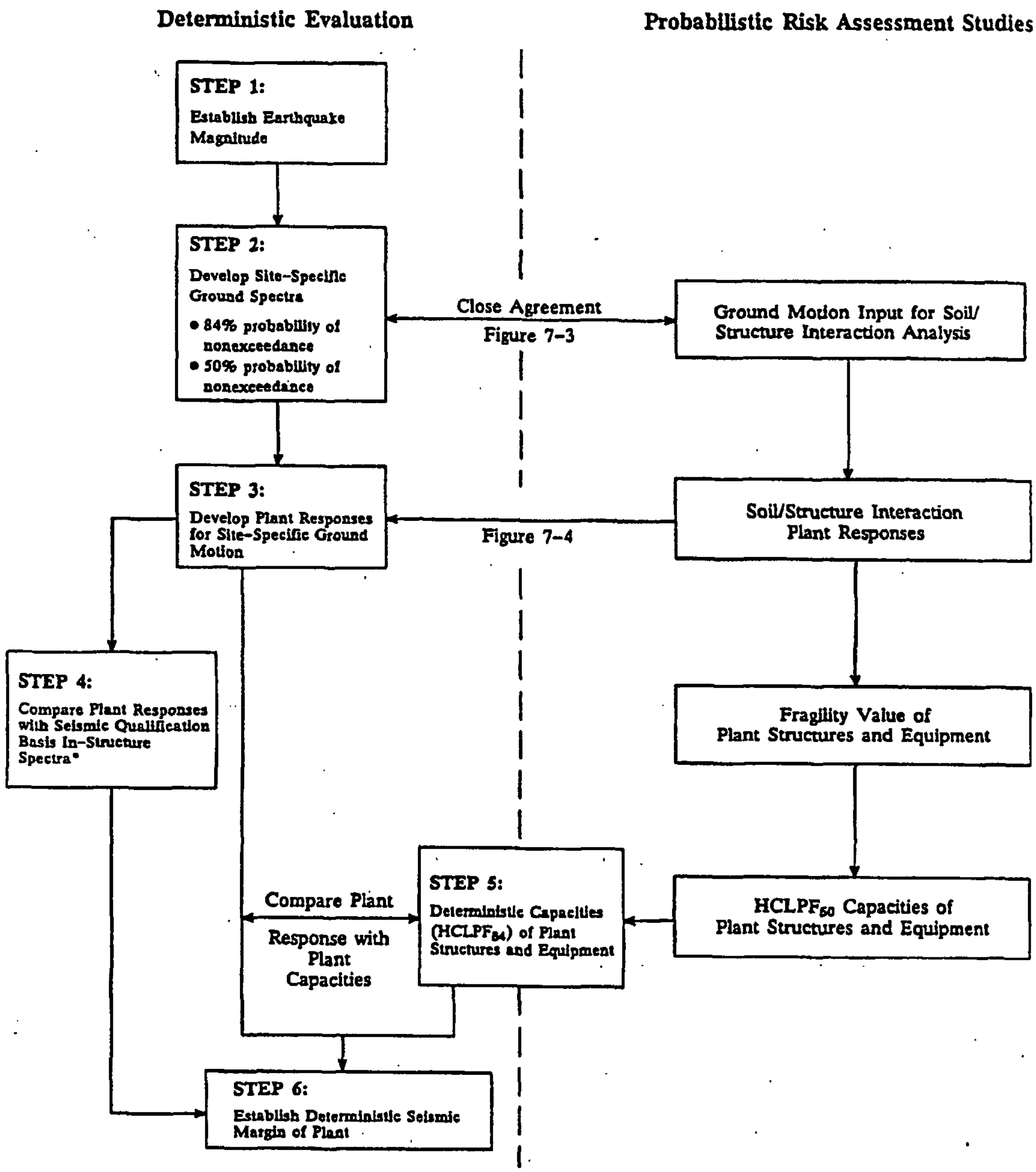
The deterministic evaluation of the Plant drew from essentially all activities of the Long Term Seismic Program (Figure 7-1). The evaluation consisted of six steps. In Step 1, the maximum earthquake magnitude was quantified, as described in Chapter 3. Step 2, which is described in Chapter 4, involved the development of the site-specific ground motions for the 50 percent and 84 percent probability of nonexceedance levels. Step 3 used information from the soil/structure interaction studies (Chapter 5) and applied it to develop Plant responses resulting from the site-specific ground motions. Step 4 compared these responses with the seismic qualification basis responses for the Plant. It also addressed the effects of responses due to the site-specific ground motions that exceed those for the seismic qualification basis. Step 5 involved the determination of the capacities for plant structures and components. These capacities were derived from the fragility evaluations described in Chapter 6. Finally, in Step 6, the capacities were compared with the demands (Plant responses) to assess the seismic margin of the Plant above the demand resulting from the 84th percentile ground motions due to the maximum magnitude earthquake.

DETERMINISTIC COMPARISONS

Plant Responses to Site-Specific Ground Motions

DEVELOPMENT OF SITE-SPECIFIC GROUND MOTIONS

The confirmation of the controlling seismic source and its tectonic environment are described in Chapter 2 of this report. The source was identified as the Hosgri fault located at a distance of about 4.5 kilometers from the Plant site. The maximum



*Seismic qualification basis is a combination of original design and Hosgri evaluation basis.

Figure 7–1

Deterministic evaluation process.

magnitude earthquake on this source, as established in Chapter 3, is an earthquake of magnitude 7.2 M_W.

This earthquake was used in the ground-motion study to develop appropriate ground response spectra. Because there is a lack of agreement in the nuclear industry on the selection of the level of ground motions for Plant reevaluations, the site-specific ground motions have been specified at both the 50 percent and 84 percent probability of nonexceedance levels. The details of the development of those ground motions are provided in Chapter 4. Site-specific horizontal and vertical ground-motion response spectra for 5 percent damping corresponding to the maximum earthquake magnitude are shown in Figures 4-22 and 4-23, respectively.

A comparison of the site-specific response spectra (for 5 percent damping) corresponding to the horizontal ground motions due to the maximum magnitude earthquake and the 1977 Hosgri (Newmark) evaluation spectrum is shown in Figure 7-2. It may be seen that the Hosgri evaluation spectrum envelops the site-specific 50th percentile spectrum at all frequencies and the 84th percentile spectrum at all frequencies less than about 15 hertz. The magnitude of the exceedance at frequencies above 15 hertz is approximately 10 percent.

DEVELOPMENT OF PLANT RESPONSES

To generate in-structure response spectra for use in the fragility evaluations for the probabilistic risk assessment, detailed soil/structure interaction analyses were performed, as described in Chapter 5. These soil/structure interaction analyses were performed deterministically, using the most current site-specific acceleration response spectra available at the time.

Although the primary purpose of these analyses was to generate inputs to the fragility analysis and the probabilistic risk assessment, it was recognized that they could also provide data useful in the Plant deterministic evaluation, if the ground-motion spectral shape used for the deterministic evaluation was similar to that used in the probabilistic risk assessment. At the time the soil/structure interaction analyses were performed, the site-specific ground-motion spectra had not been finalized. To support the fragility analyses, a "best estimate" spectrum was established, recognizing that the soil/structure interaction analysis results could be adjusted for compatibility with the site-specific ground spectra, as appropriate, at a later stage in the Program.

Figure 7-3 shows a comparison of the ground-motion spectral shape used in the soil/structure interaction analyses with the site-specific ground-motion spectrum at the 84 percent probability of nonexceedance level. To permit a meaningful comparison of the shapes of these two ground-motion spectra, the ground response spectrum used in the soil/structure interaction analyses (Figure 5-22) has been scaled uniformly (frequency-independent scaling) such that the average spectral acceleration between 3 and 8.5 hertz is the same as that of the 84 percent probability of nonexceedance site-specific ground-motion spectrum (1.94 g). A comparison of these spectra shows that the site-specific ground-motion spectrum closely matches the soil/structure interaction analysis input spectral shape. Because the soil/structure interaction analyses are linear elastic, their results can be scaled uniformly. Accordingly, the soil/structure interaction analysis results can be used with small adjustment factors, to obtain Plant responses to the selected final site-specific ground motions.

The procedure used to convert the results of the soil/structure interaction analyses into Plant responses for the site-specific ground motions is illustrated in Figure 7-4. It requires the use of two factors:

1) A spectral shape factor (F_{ss}) that accounts for the minor variations between the site-specific ground-motion spectrum and the soil/structure interaction input spectrum. This factor is determined from the ratio of the

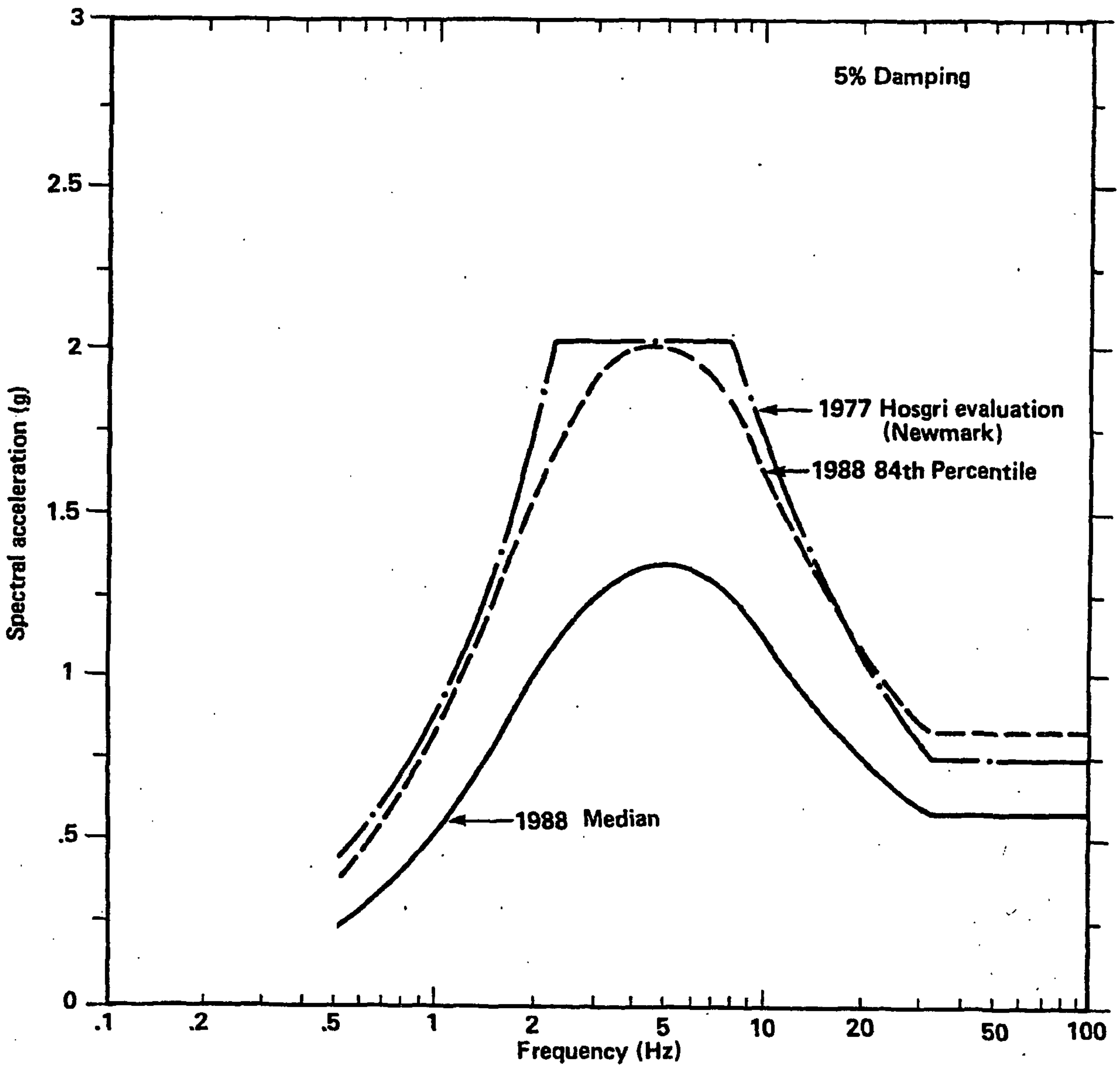


Figure 7-2
Comparison of the 1988 site-specific median and 84th percentile horizontal response spectra with the 1977 Hosgri evaluation (Newmark) response spectrum.

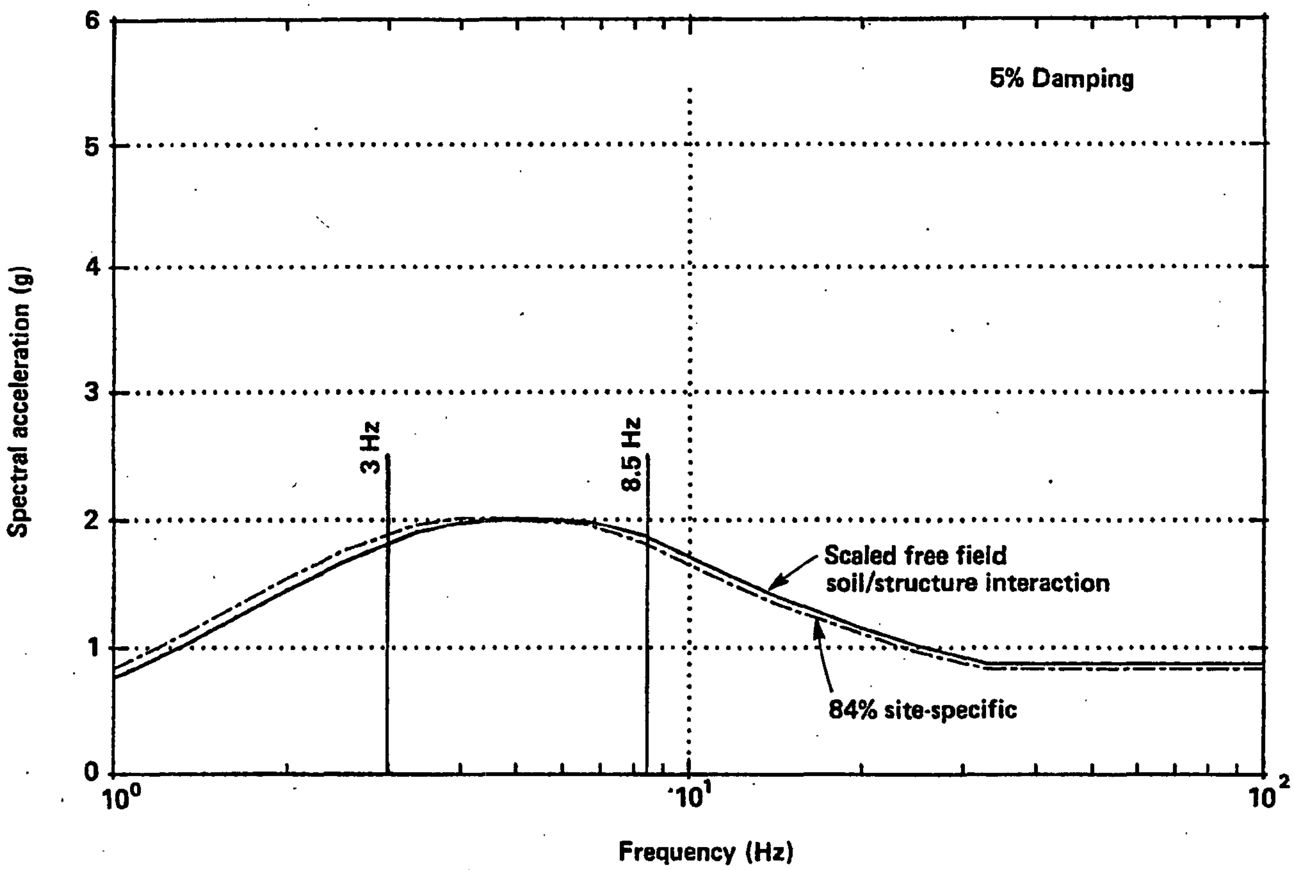


Figure 7-3

Comparison of scaled soil/structure interaction input spectrum with 84 percent probability of nonexceedance site-specific spectrum.

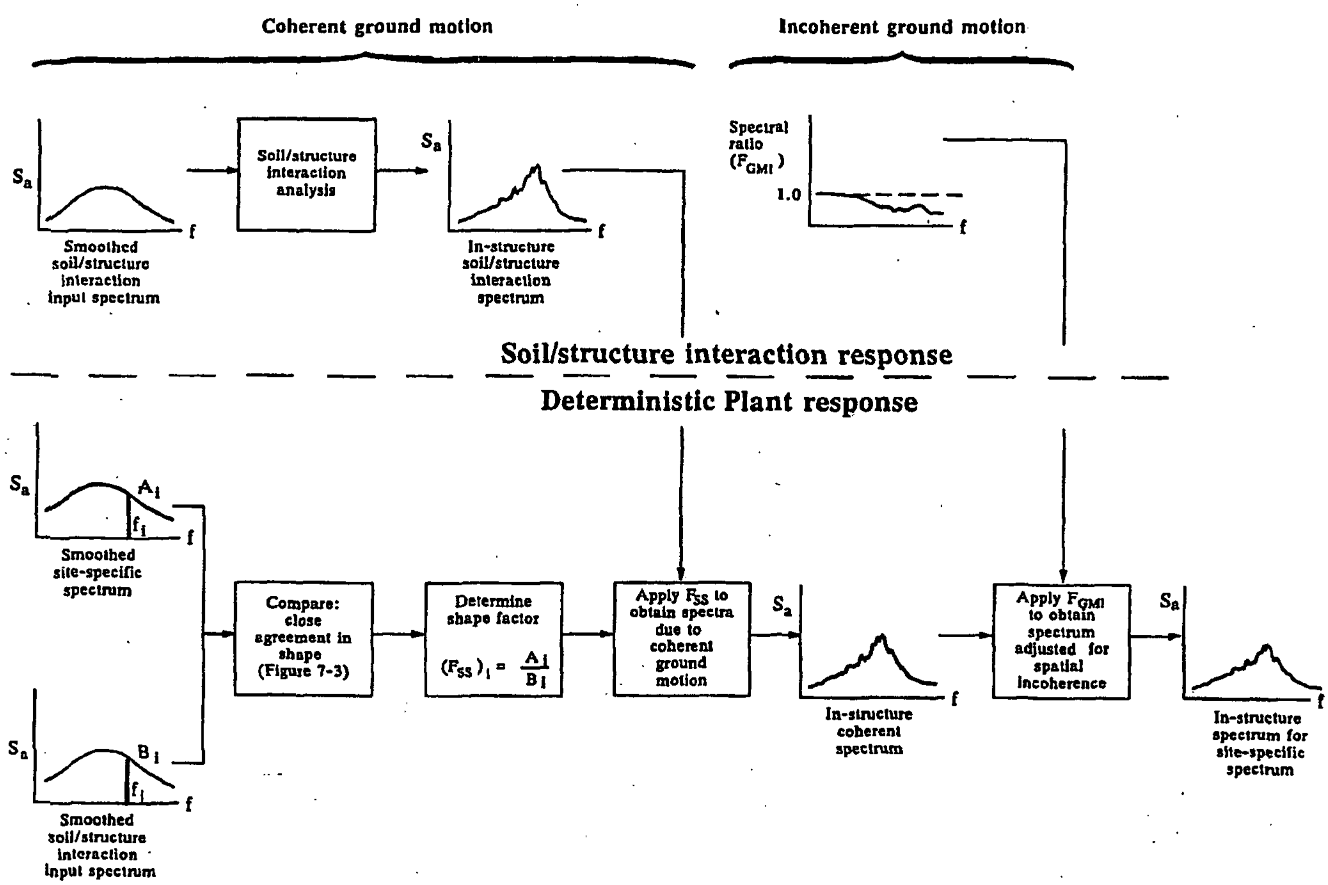


Figure 7-4
Overall process of developing plant responses.

response spectral ordinates of the free-field site-specific ground-motion spectrum (Figure 7-2) and the spectral ordinates of soil/structure interaction analysis smoothed input spectrum (Figure 5-22). For the 84 percent probability of nonexceedance site-specific ground-motion spectrum, this factor ranges from about 0.86 to 1.0.

The factors F_{SS} are applied to the soil/structure interaction in-structure response spectra to obtain the Plant responses at various floor levels due to coherent site-specific ground motions. Figures 7-5 through 7-10 show plots of the site-specific free-field ground-motion spectrum (at the 50 and 84 percent probability of nonexceedance levels) with the corresponding basemat response spectra (El 85 feet) of the major structures.

For frequencies above about 10 hertz, the soil/structure interaction effect (coherent motion) results in a reduction of the input motion to the basemat from the free-field motion. For the frequencies lower than 10 hertz, that basemat motion is slightly amplified.

2) A spatial incoherence factor (F_{GMI}) that accounts for the spatial variations of ground motion. The development of this factor is discussed in Chapter 5. For a specific site, this factor, in general, results in a reduction of building responses and is dependent on the plan area of the building foundation and the frequency of vibration of the building, among other parameters. For the frequency range above about 5 hertz, the reduction in translational motion is about 6 percent for the containment, 15 percent for the auxiliary building, and 20 percent for the turbine building. When considered in conjunction with rocking and torsional motions, there is generally a decrease in the above effects.

The F_{GMI} factors are applied to the response spectra developed for coherent site-specific ground motions to obtain the composite effect of soil/structure interaction. Figures 7-11 through 7-18 show comparisons of free-field and basemat spectra, including the effects of ground-motion incoherence.

Comparisons of Plant Responses for Site-Specific Ground Motions and Seismic Qualification Bases Motions

The Plant seismic qualification basis events included two large earthquakes: the double design earthquake and the Hosgri earthquake. The seismic design and qualification requirements associated with those two earthquakes were developed at different times during the plant-licensing process. As a result, the corresponding analysis parameters, method, and criteria (for example, structural damping, modeling assumptions, treatment of soil/structure interaction, and so forth), differ not only from those used in the current deterministic evaluations, but also from one another. Because of these differences, one-to-one comparison of response spectra due to the site-specific ground motions with the governing seismic qualification bases spectra is not always appropriate. However, comparisons of response spectra are provided, in response to a request from Nuclear Regulatory Commission (NRC) Staff.

FREE-FIELD AND BASEMAT RESPONSE SPECTRA

During the previous Hosgri evaluations, Plant response was evaluated for both a Hosgri spectrum recommended by Newmark and a Hosgri spectrum recommended by Blume. Seismic evaluations were based on whichever of these spectra proved to be more conservative for any given structure or equipment item at any given frequency. Figure 7-19 shows comparisons of the enveloped Hosgri (Newmark and Blume) 0.75 g free-field response spectrum with the site-specific ground-motion spectra, at the 50 percent and 84 percent probabibility of nonexceedance levels. This figure shows that the 50 percent probability of nonexceedance of site-specific ground-motion

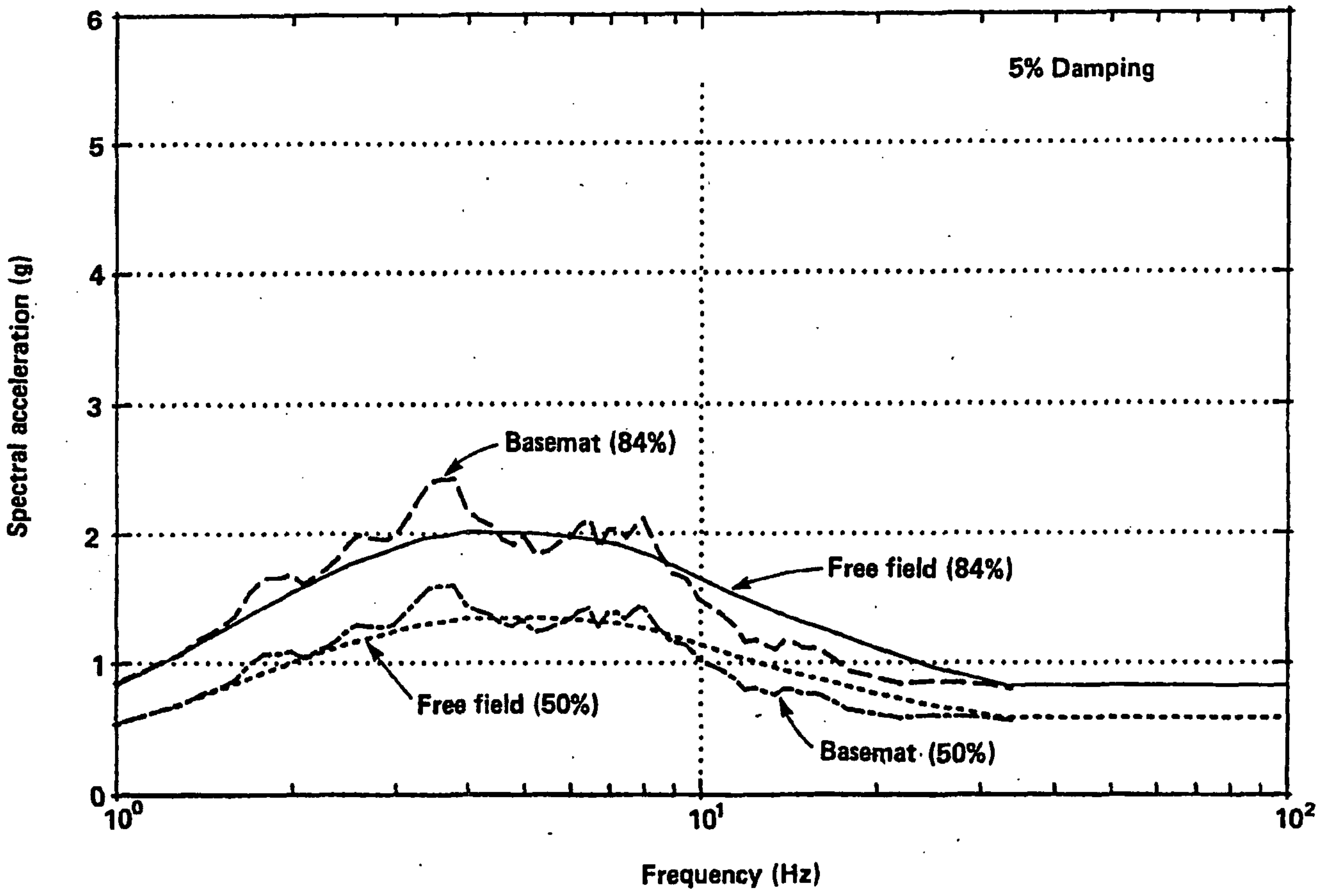


Figure 7-5

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, containment building, north/south response, using coherent ground motions.

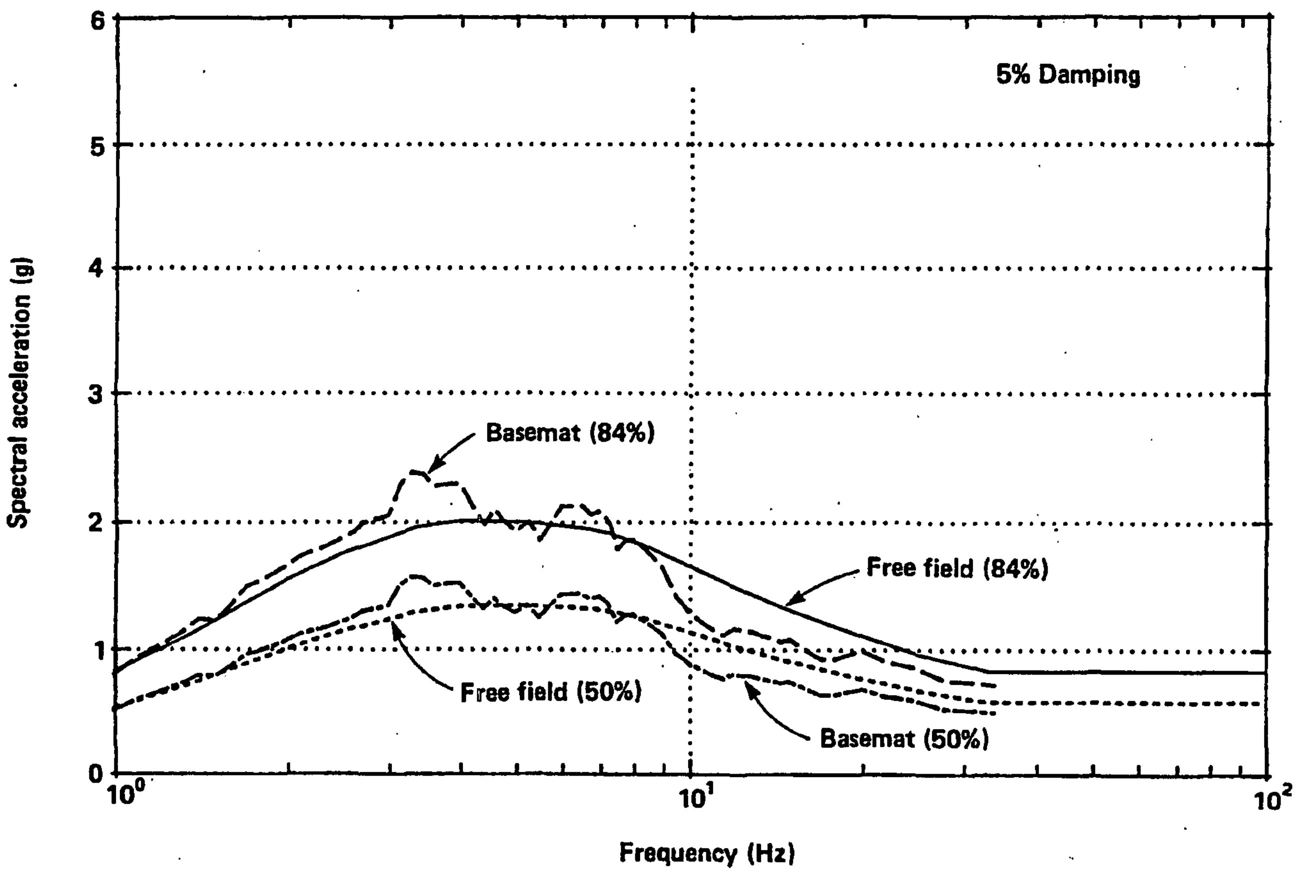


Figure 7-6

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, containment building, east/west response, using coherent ground motions.

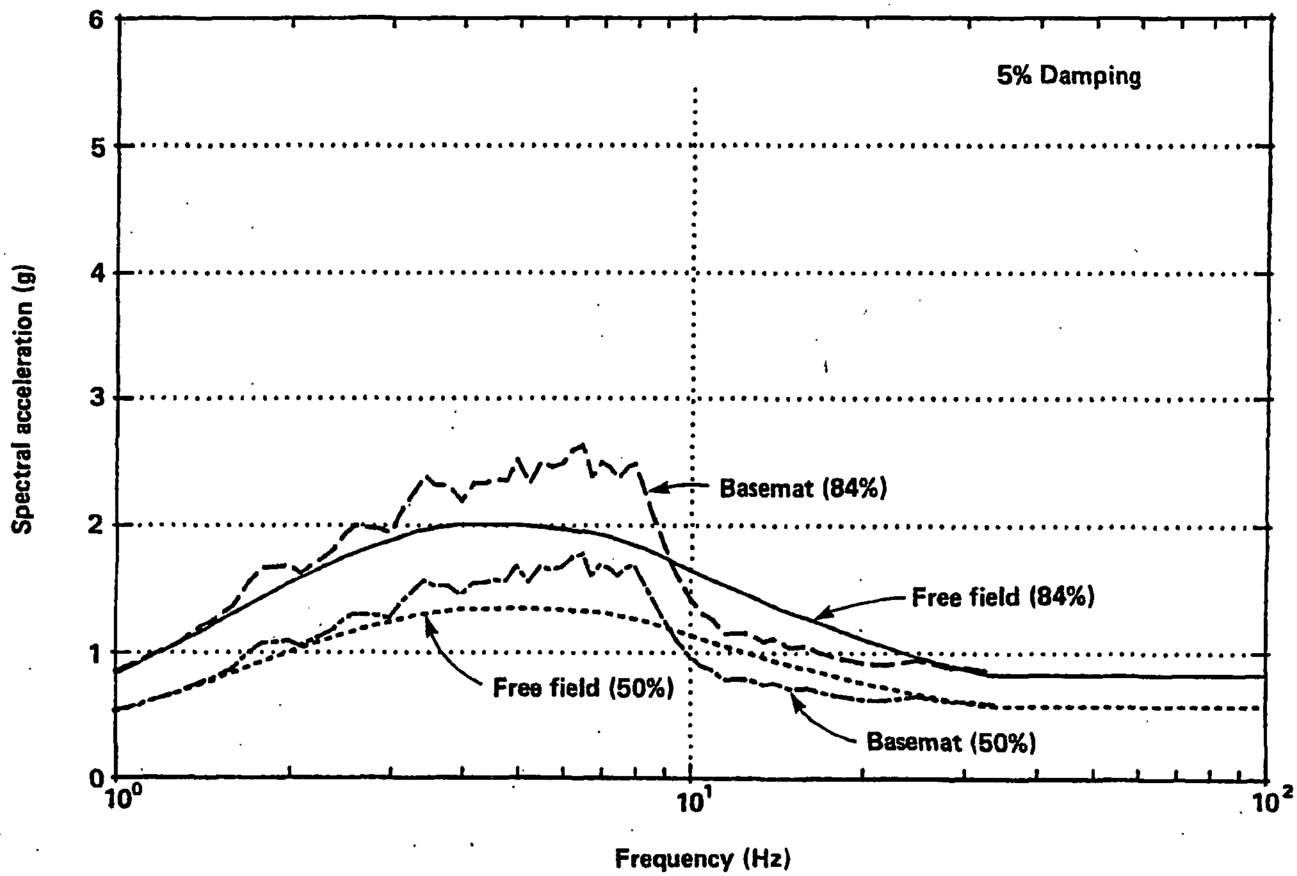


Figure 7-7

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, auxiliary building, north/south response, using coherent ground motions.

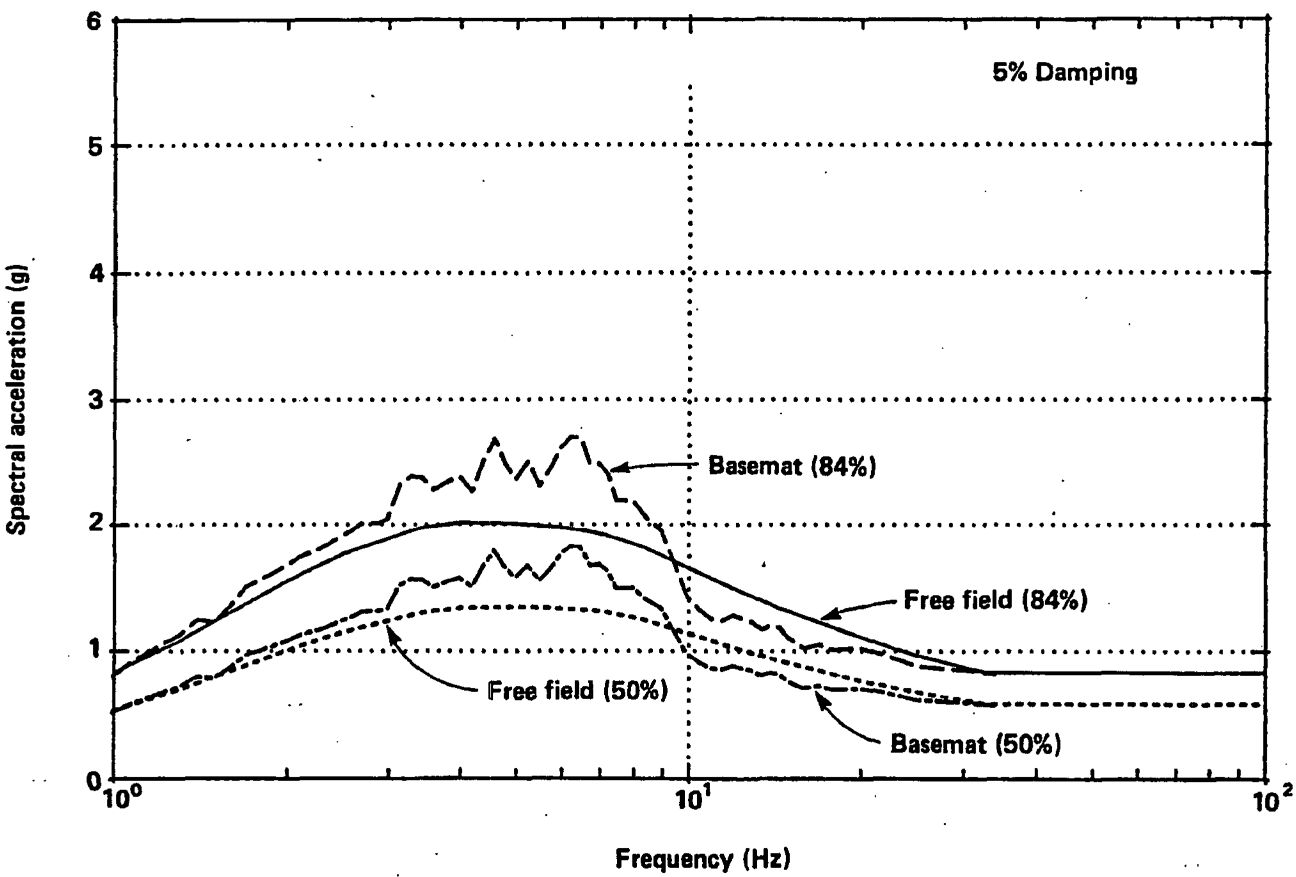


Figure 7-8

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, auxiliary building, east/west response, using coherent ground motions.

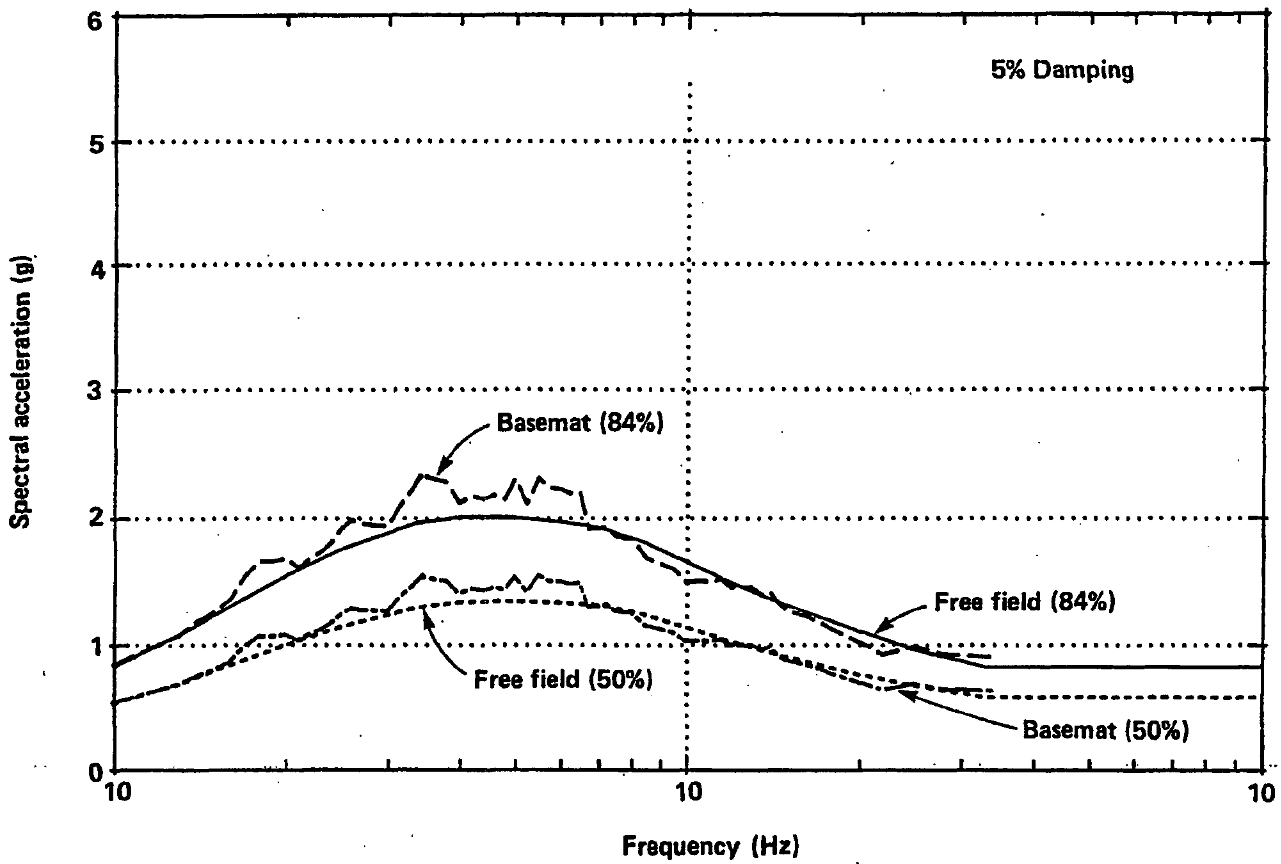


Figure 7-9

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, turbine building, north/south response, using coherent ground motions.

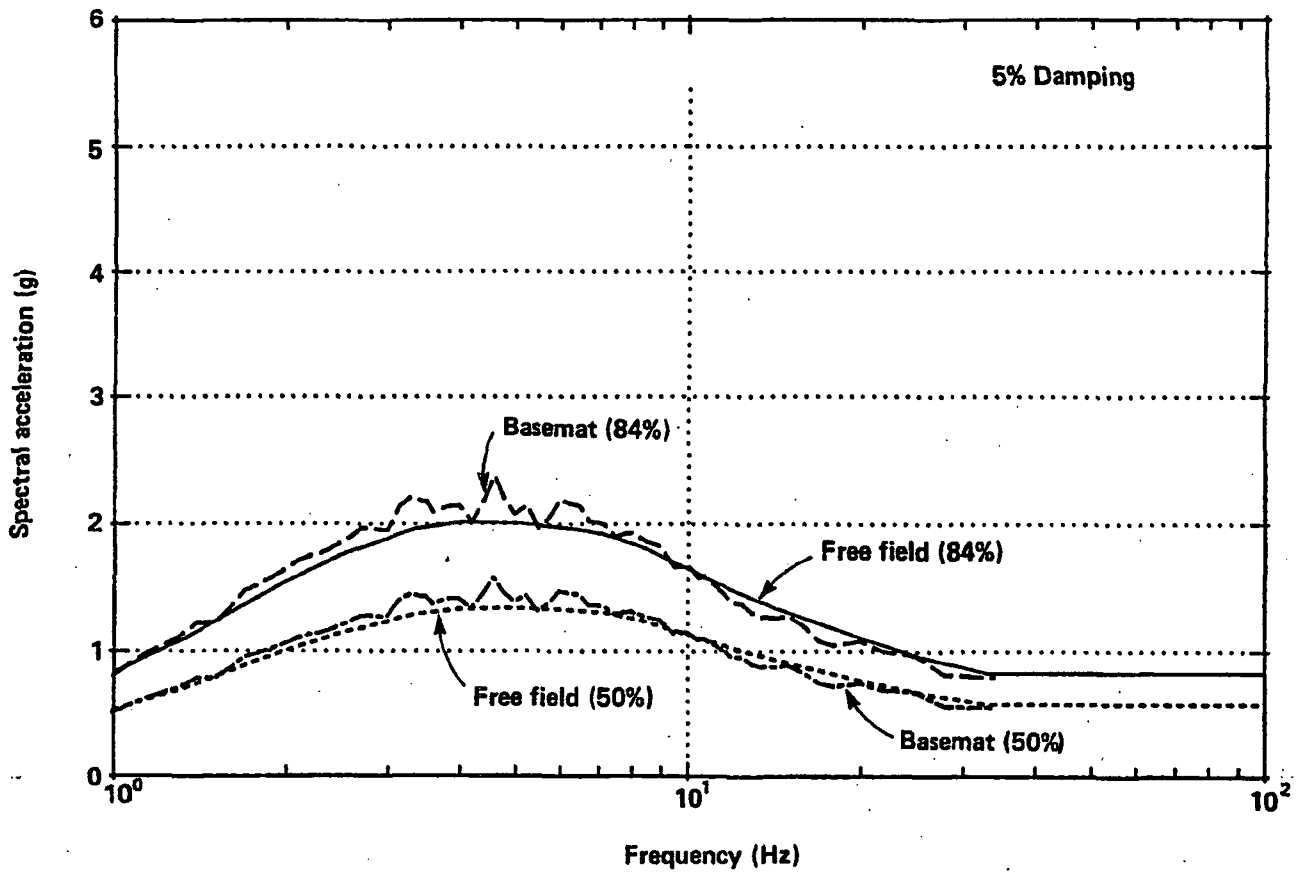


Figure 7-10

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, turbine building, east/west response, using coherent ground motions.

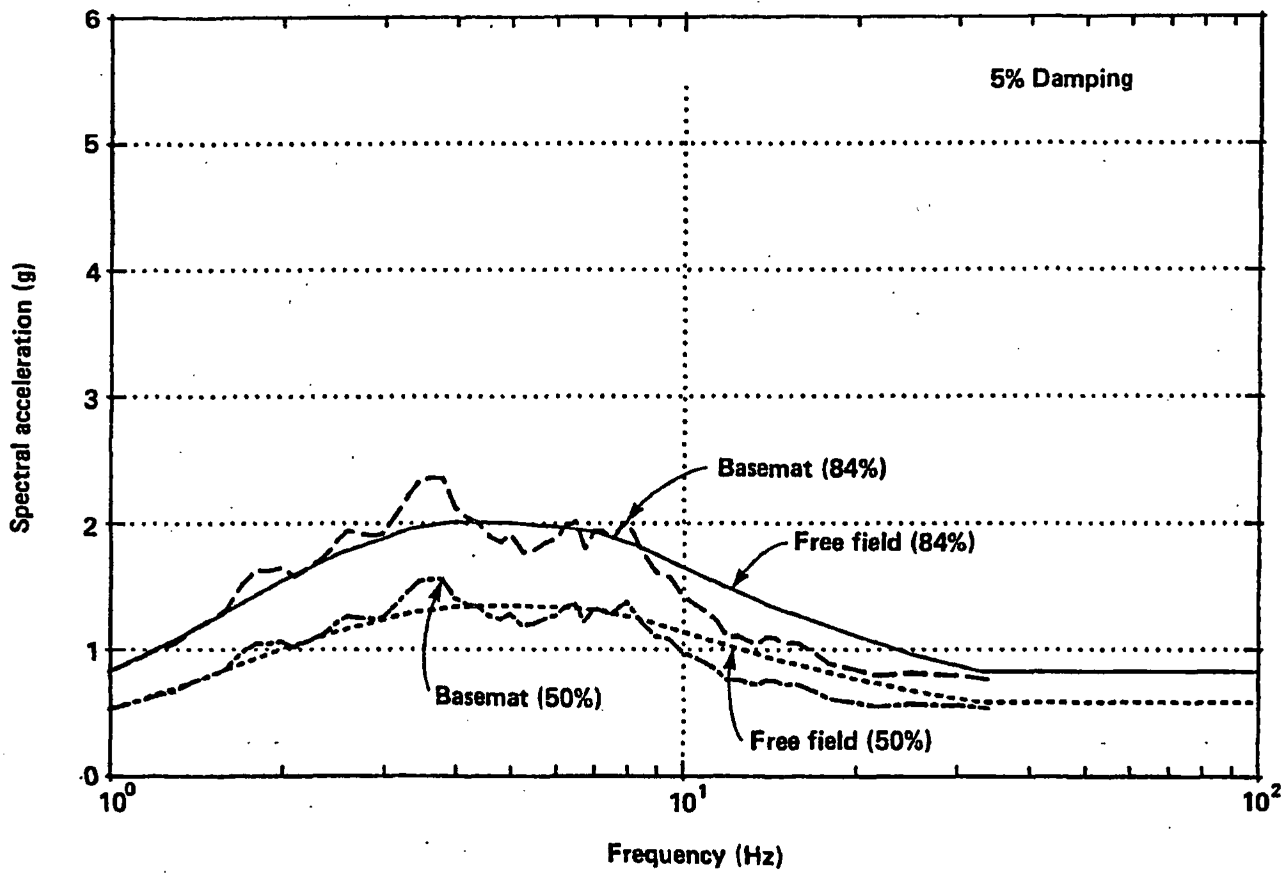


Figure 7-11

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, containment building, north/south response, using incoherent ground motions.

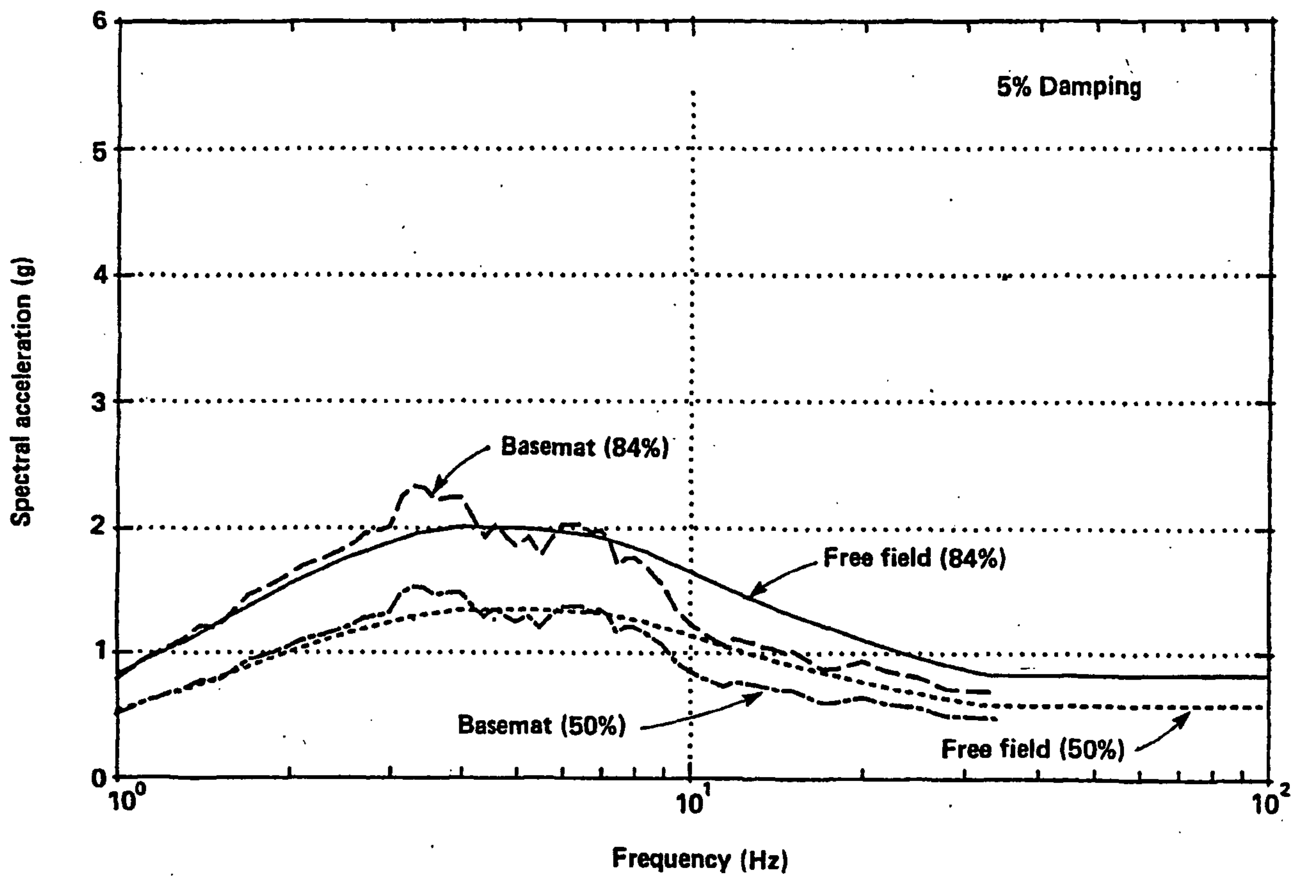


Figure 7-12

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, containment building, east/west response, using incoherent ground motions.

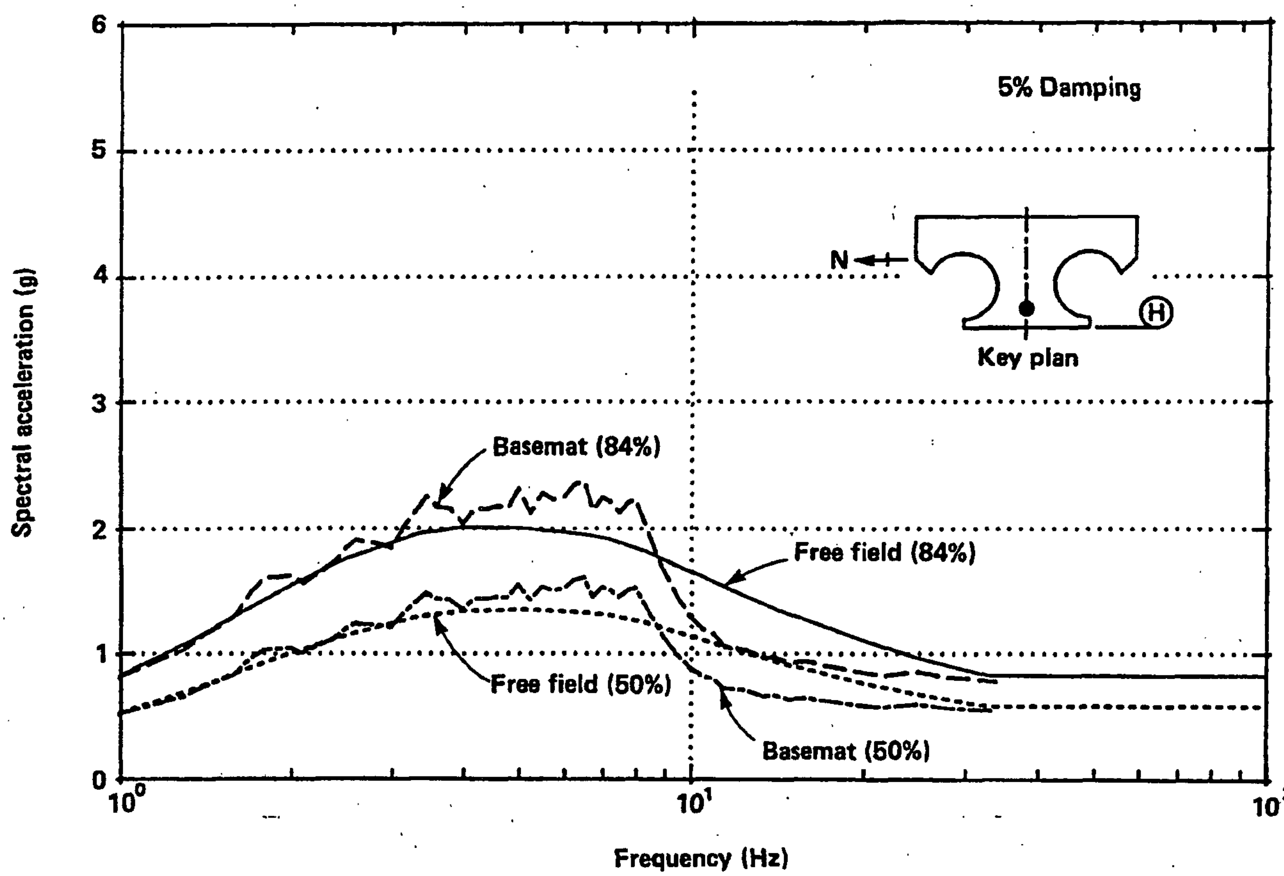


Figure 7-13

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, auxiliary building, north/south response, using incoherent ground motions.

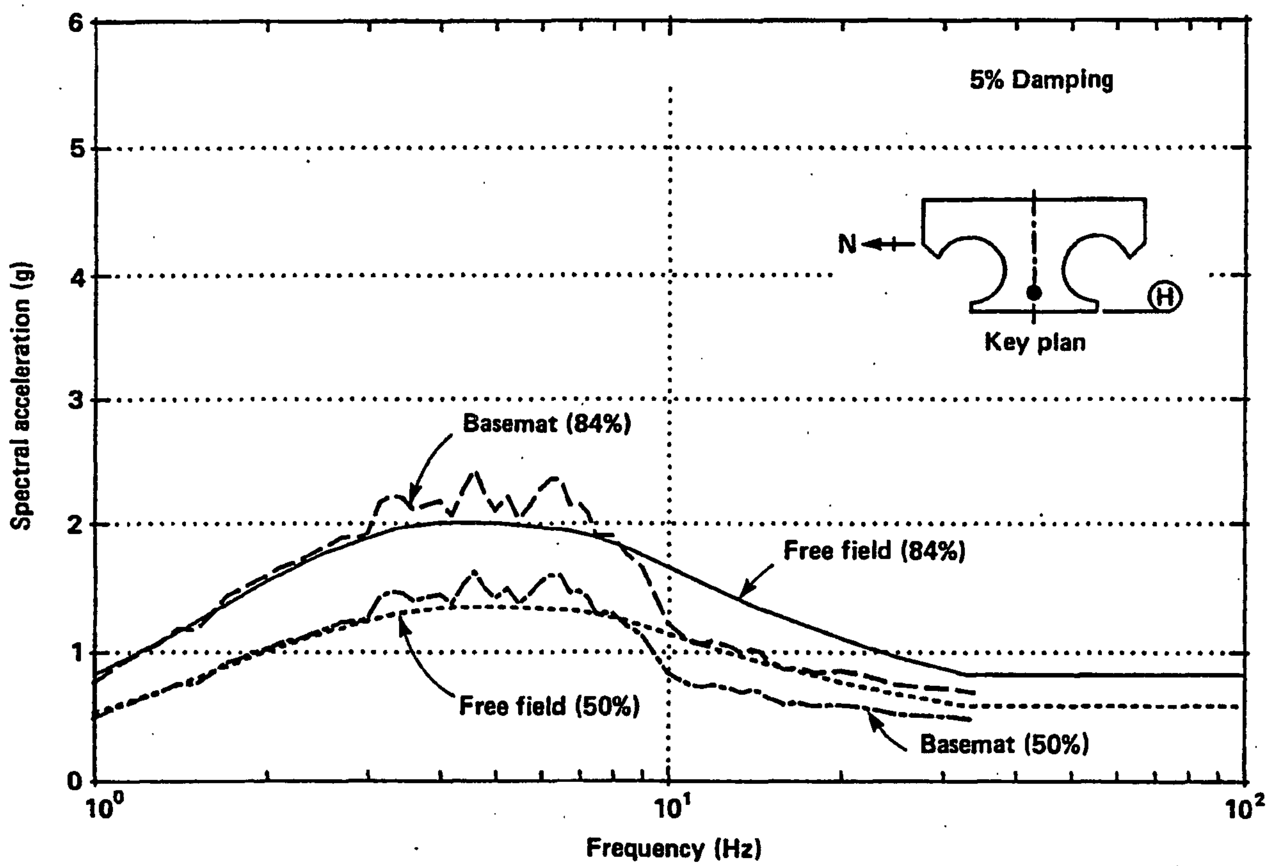


Figure 7-14

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, auxiliary building, east/west response, using incoherent ground motions.

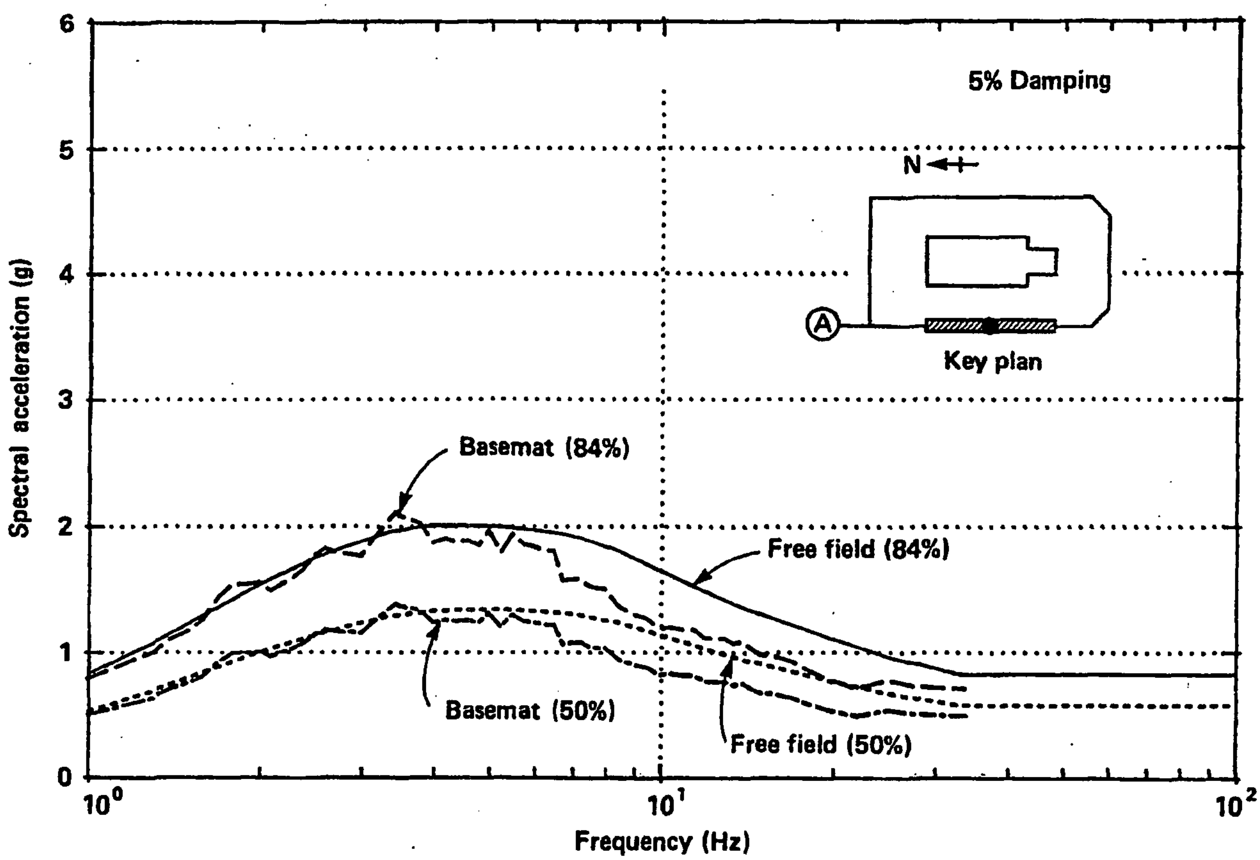


Figure 7-15

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, turbine building, wall A area, north/south response, using incoherent ground motions.

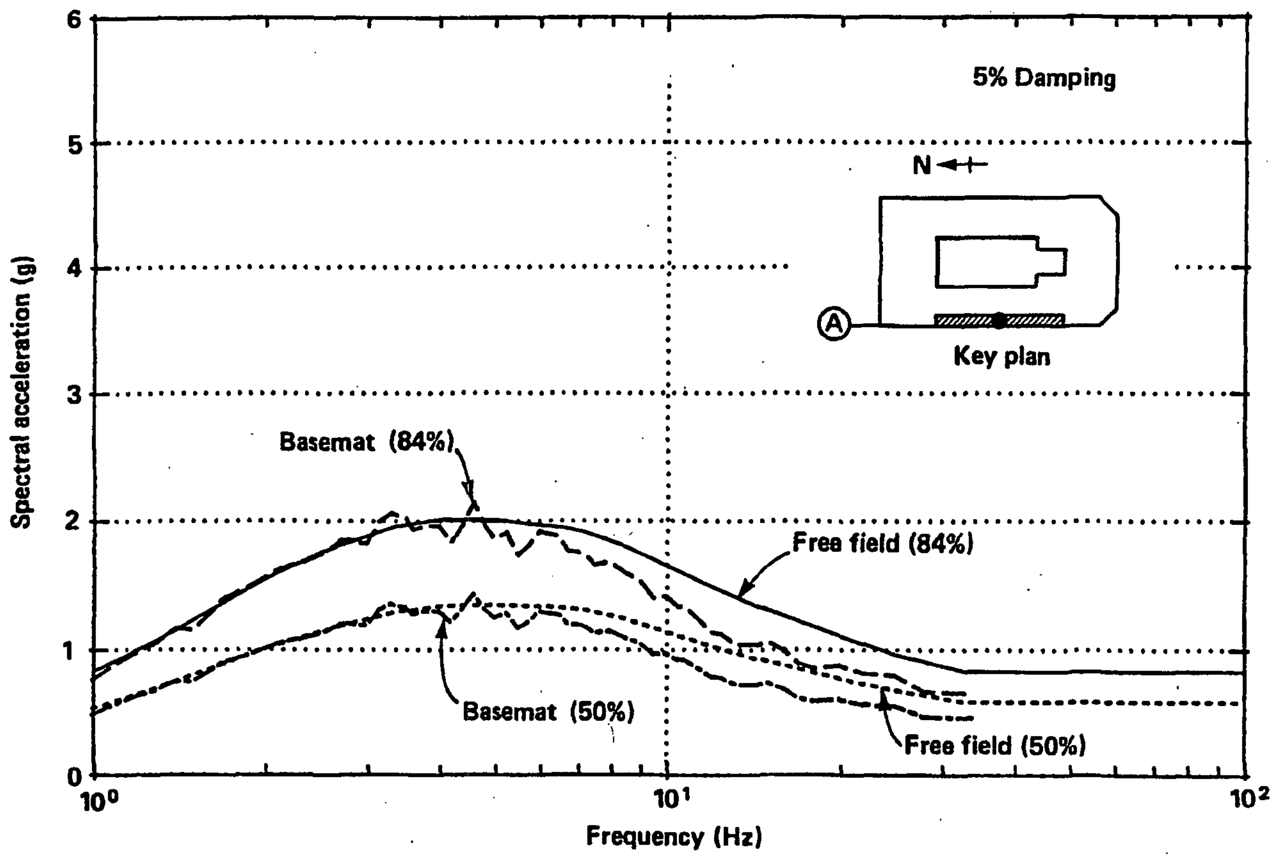


Figure 7-16

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, turbine building, wall A area, east/west response, using incoherent ground motions.

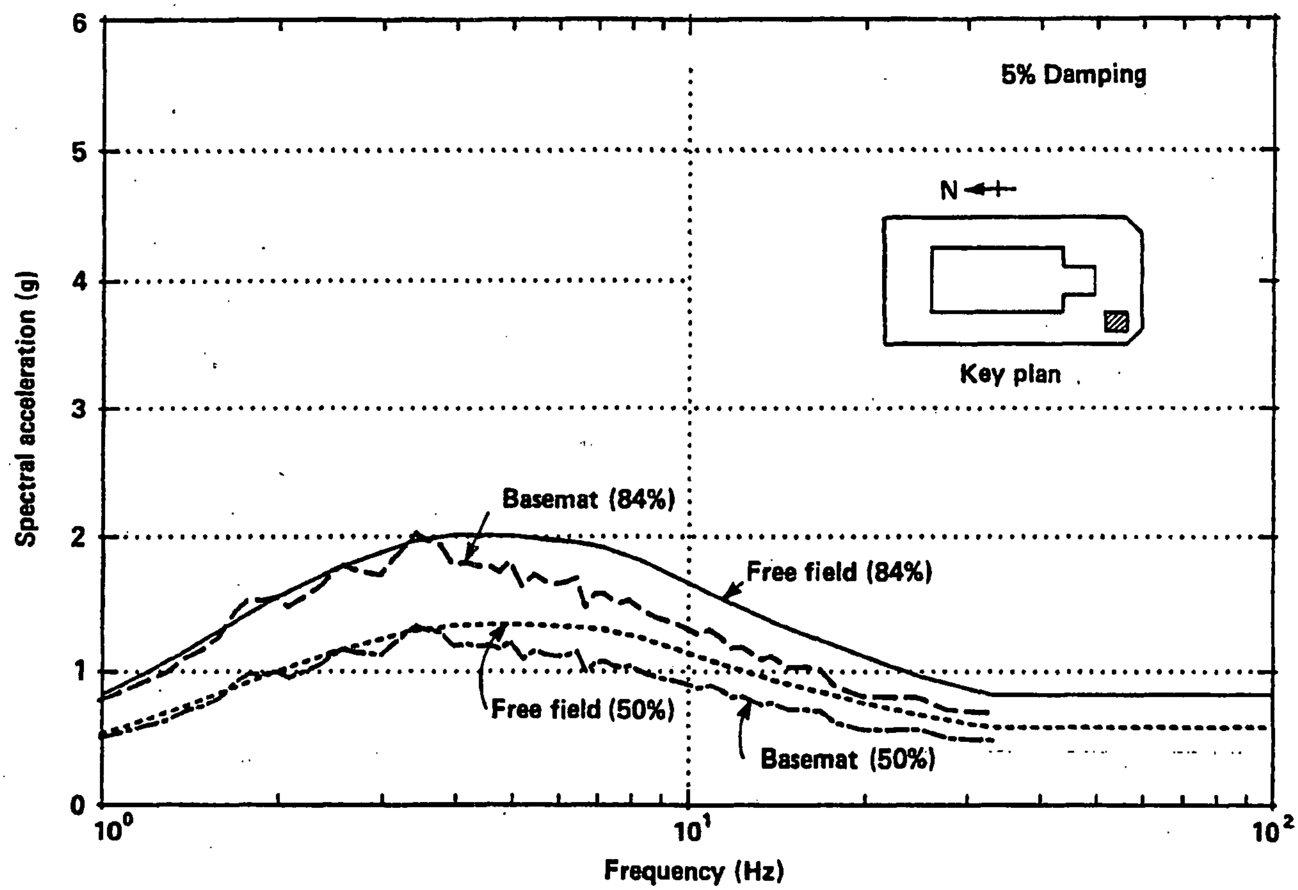


Figure 7-17

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, turbine building, diesel generator area, north/south response, using incoherent ground motions.

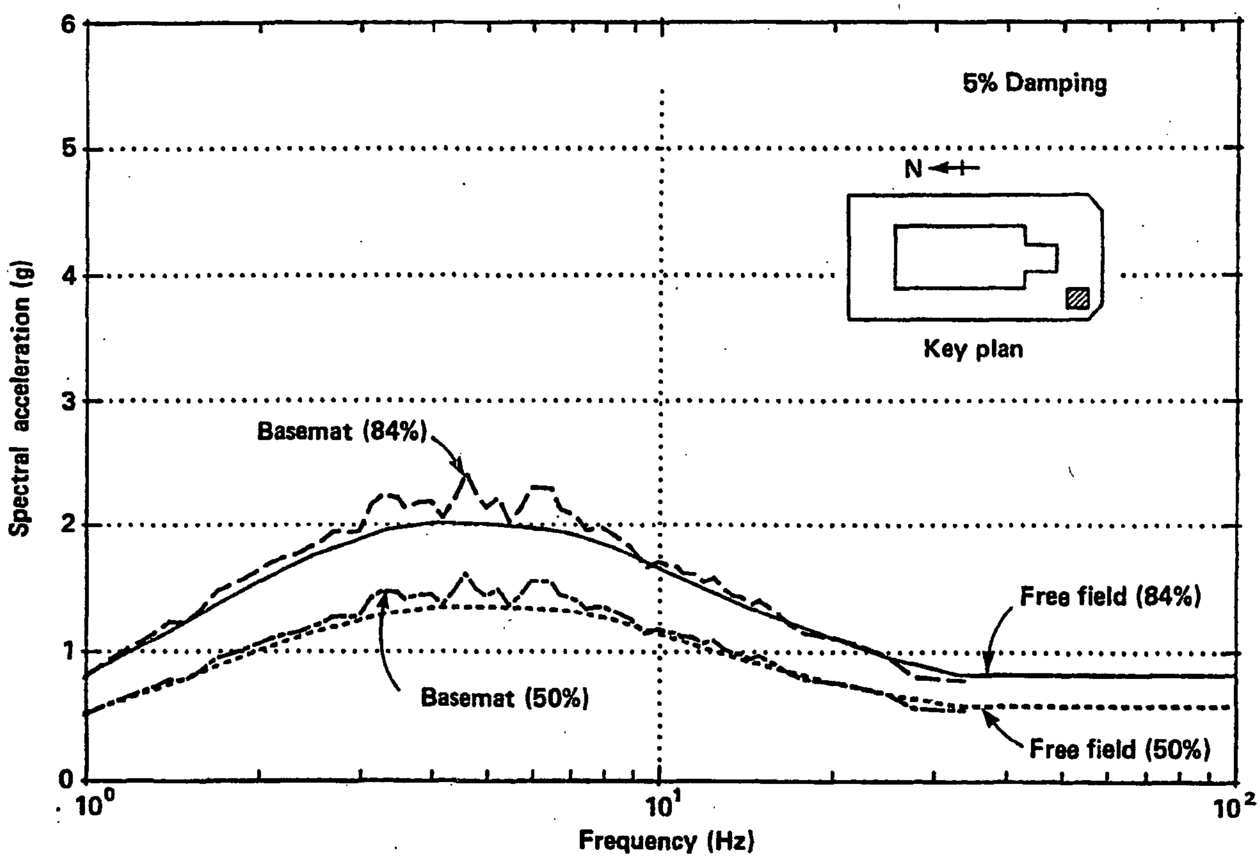


Figure 7-18

Comparison of spectra for free-field ground motions with basemat spectra determined by soil/structure interaction analysis, turbine building, diesel generator area, east/west response, using incoherent ground motions.

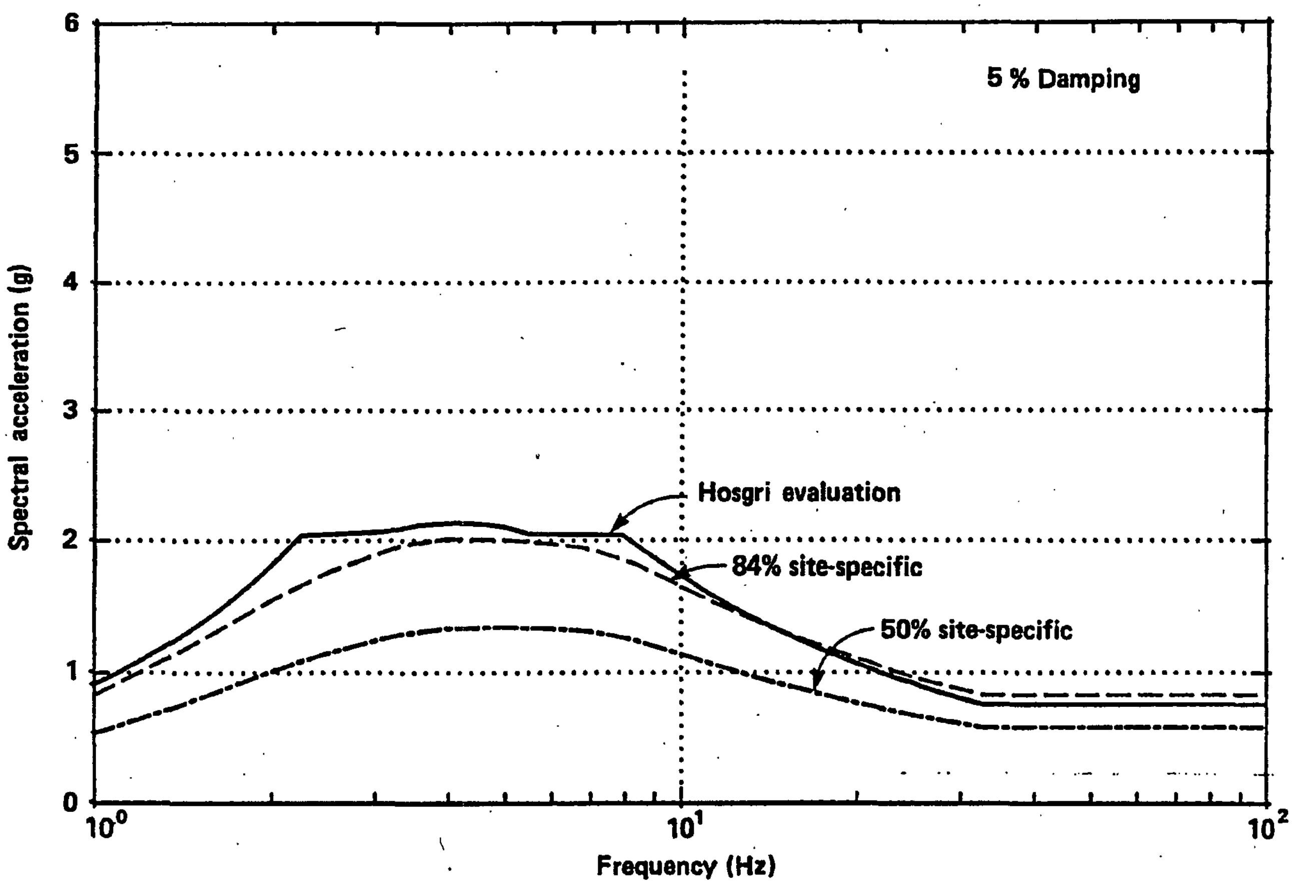


Figure 7-19
Comparison of free-field, site-specific spectra with 1977 Hosgri evaluation spectrum for horizontal ground motion.

spectrum is enveloped by the Hosgri evaluation spectrum. The 84 percent probability of nonexceedance site-specific ground spectrum exceeds the Hosgri evaluation spectrum only at frequencies greater than about 15 hertz, and the exceedance is only about 10 percent.

Figures 7-20 through 7-29 show a comparison of the Hosgri (envelope of Newmark and Blume spectra) evaluation basemat (El 85 feet) response spectra, which had been reduced from the free-field spectra to account for the tau-filtering effect (USNRC, 1976), with the basemat spectra computed from the site-specific ground motions in this study including soil/structure interaction incoherence effects.

It should be noted that the effects of tau-filtering in the Hosgri evaluation studies were generally analogous to the combined effect of soil/structure interaction, foundation embedment, and incoherent ground-motion effects in the current soil/structure interaction analysis. However, these effects vary from point to point on the foundation basemats in the current study results. This is reflected in the difference in spectral amplitudes shown in Figures 7-20 through 7-29 for different locations on a building structure. For the Hosgri evaluation, the dynamic models used for analysis were considered to be fixed at the base and, therefore, the variations in the tau-filtering reduction for different locations on the basemat could not be considered. Instead, an average motion was assumed for all points on the foundation of a building. Becuase of this, a one-to-one comparison of basemat motions for the Hosgri evaluation spectra with those due to the site-specific ground motions in this study will show some differences.

The comparison shows that the responses to the 84th percentile site-specific spectra exceed the responses to the Hosgri evaluation spectra at various structural frequencies. The average values of these exceedances at key frequencies range from about 5 percent for the containment building interior structure to about 10 percent for the auxiliary building. These exceedances are not significant as they can be accommodated by the existing design margin, as discussed later.

IN-STRUCTURE RESPONSE SPECTRA

In-structure response spectra at selected locations in the major structures are shown in Figures 7-30 through 7-39. These spectra include the effects of soil/structure interaction, foundation embedment, and spatial incoherence of ground motions, and are compared with Plant seismic qualification basis (Hosgri or double design earthquake) in-structure response spectra. The locations for which these comparisons are shown were based on their importance in terms of structural design, or locations of critical safety-related components. These spectra are for:

- Containment interior structure at El 140 feet (operating floor level)
- Auxiliary building at El 100, 115, and 140 feet (various equipment of interest)
- Turbine building at El 119 feet (4-kV switchgear area)

The primary purpose of these spectral comparisons is to assess the effect of the site-specific ground motion in-structure response spectra on seismic qualification of equipment. Comparison of the spectra shows the following:

- For the containment building operating floor at El 140 feet (Figures 7-30 and 7-31), the seismic qualification basis spectra are well above (by up to 100 percent) the site-specific ground motion in-structure spectra between 8 and 18 hertz. For other frequencies that are significant for qualification of equipment (5 to 8 hertz), the site-specific ground-motion in-structure spectra, in general, exceed the seismic qualification basis spectra by approximately 15 percent. This exceedance is not significant and can be accommodated in design margin.

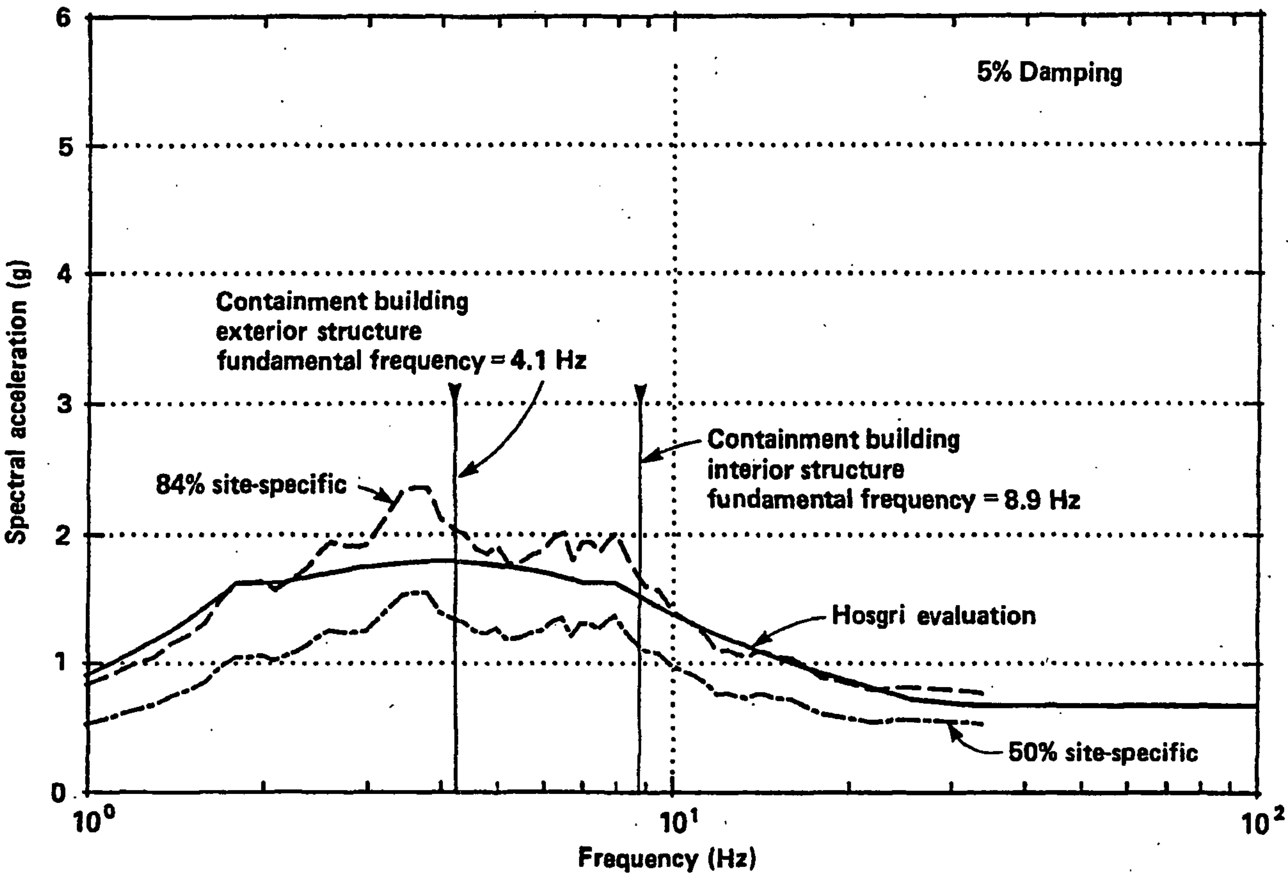


Figure 7-20

Comparison of spectra for motion at top of basemat, containment building, north/south response.

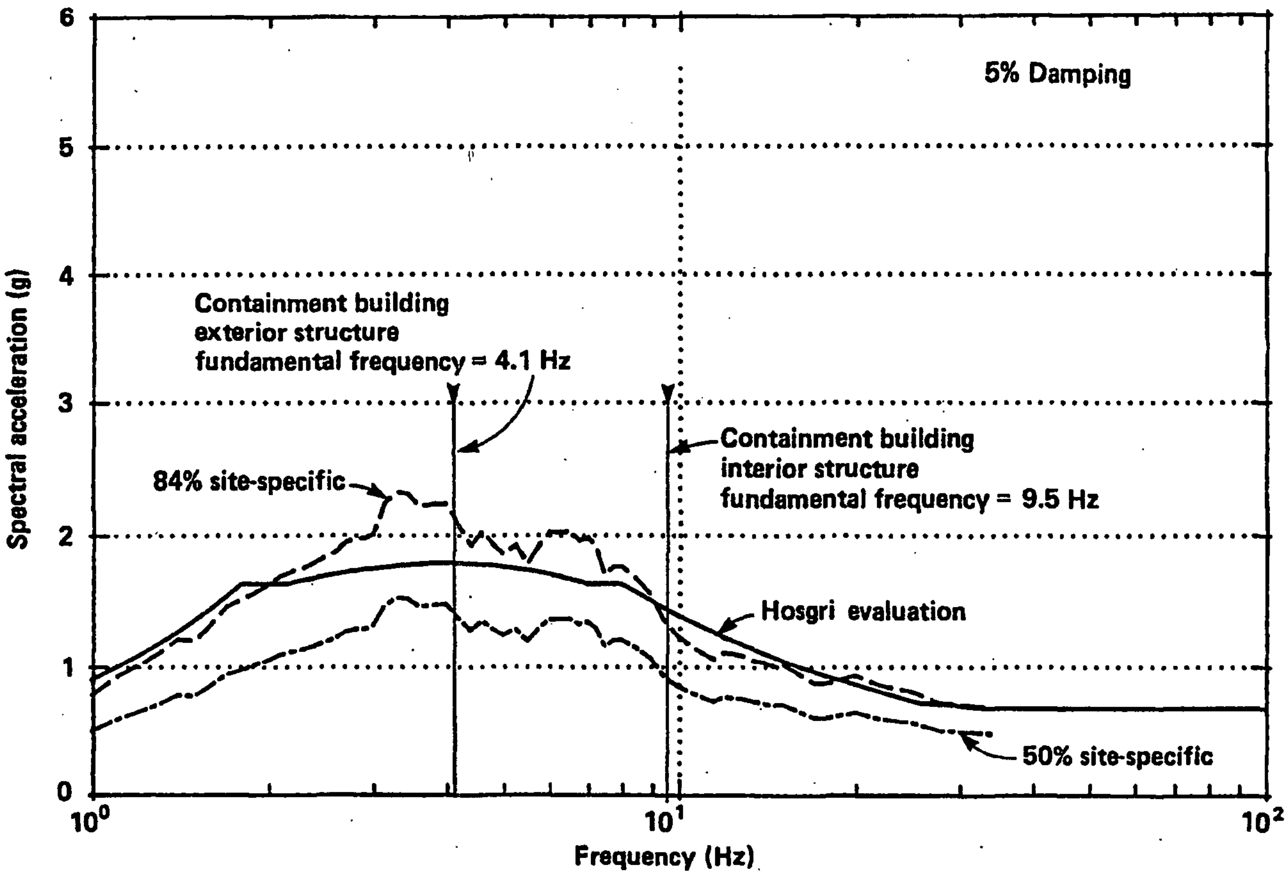


Figure 7-21

Comparison of spectra for motion at top of basemat, containment building, east/west response.

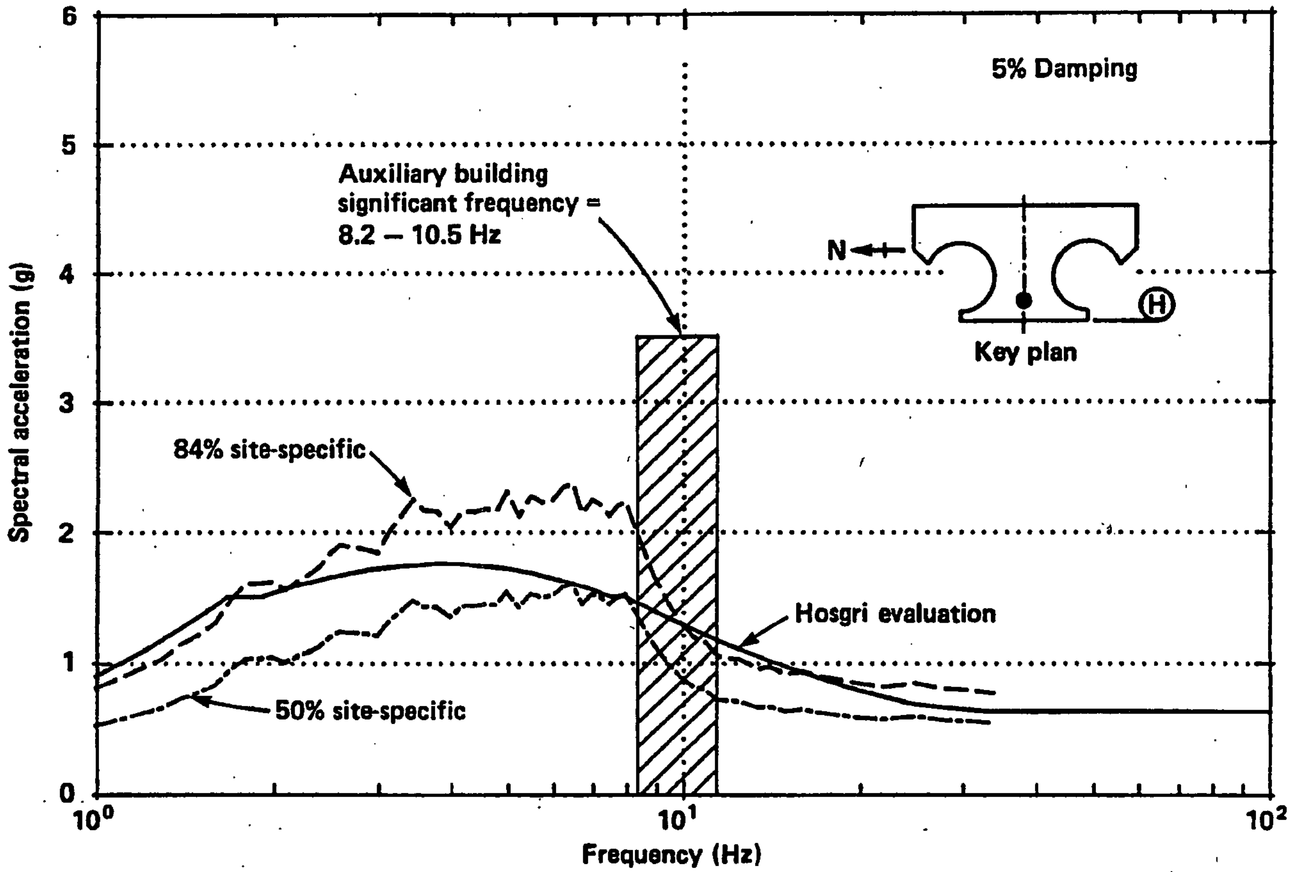


Figure 7-22
Comparison of spectra for motion at top of basemat, auxiliary building, north/south response.

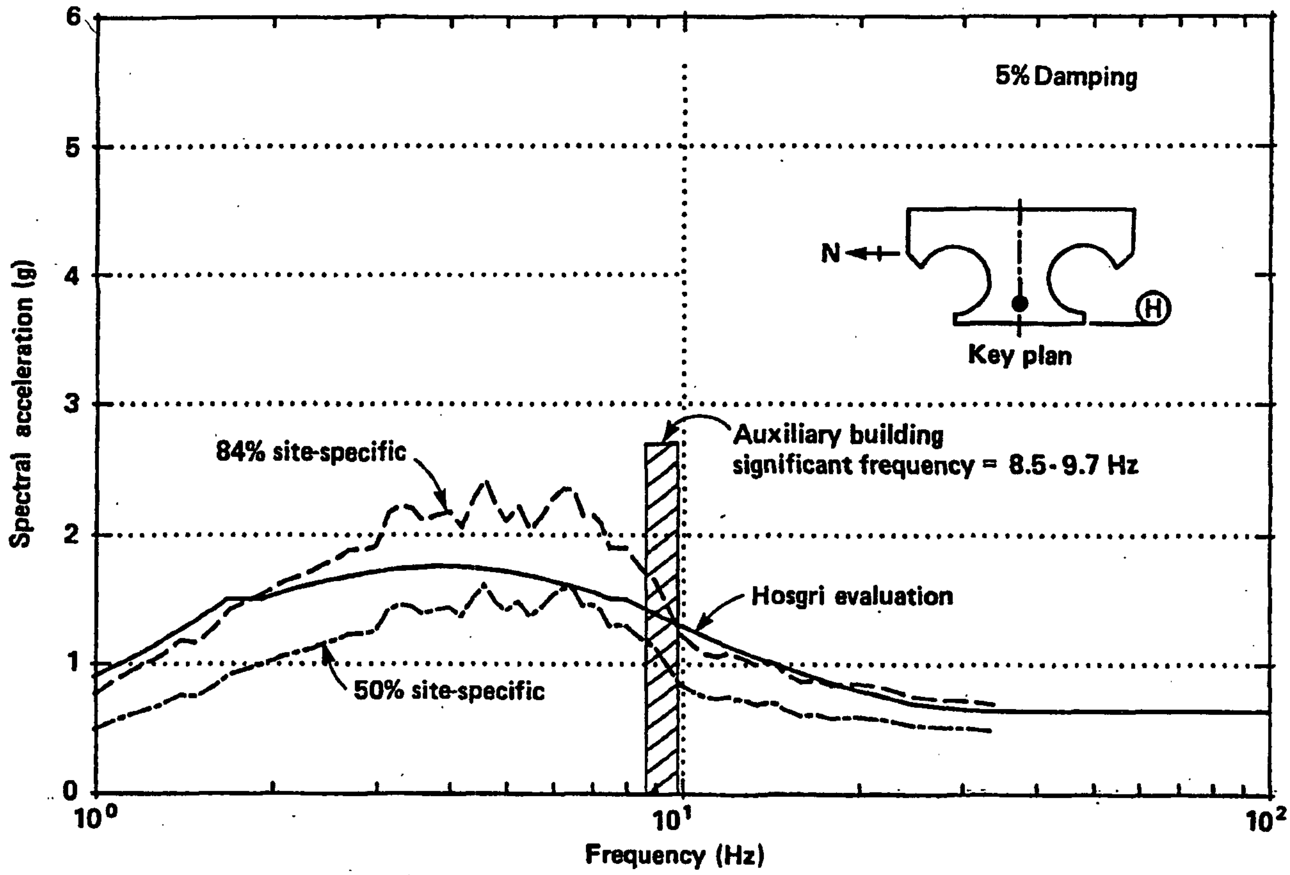


Figure 7-23
Comparison of spectra for motion at top of basemat, auxiliary building, east/west response.

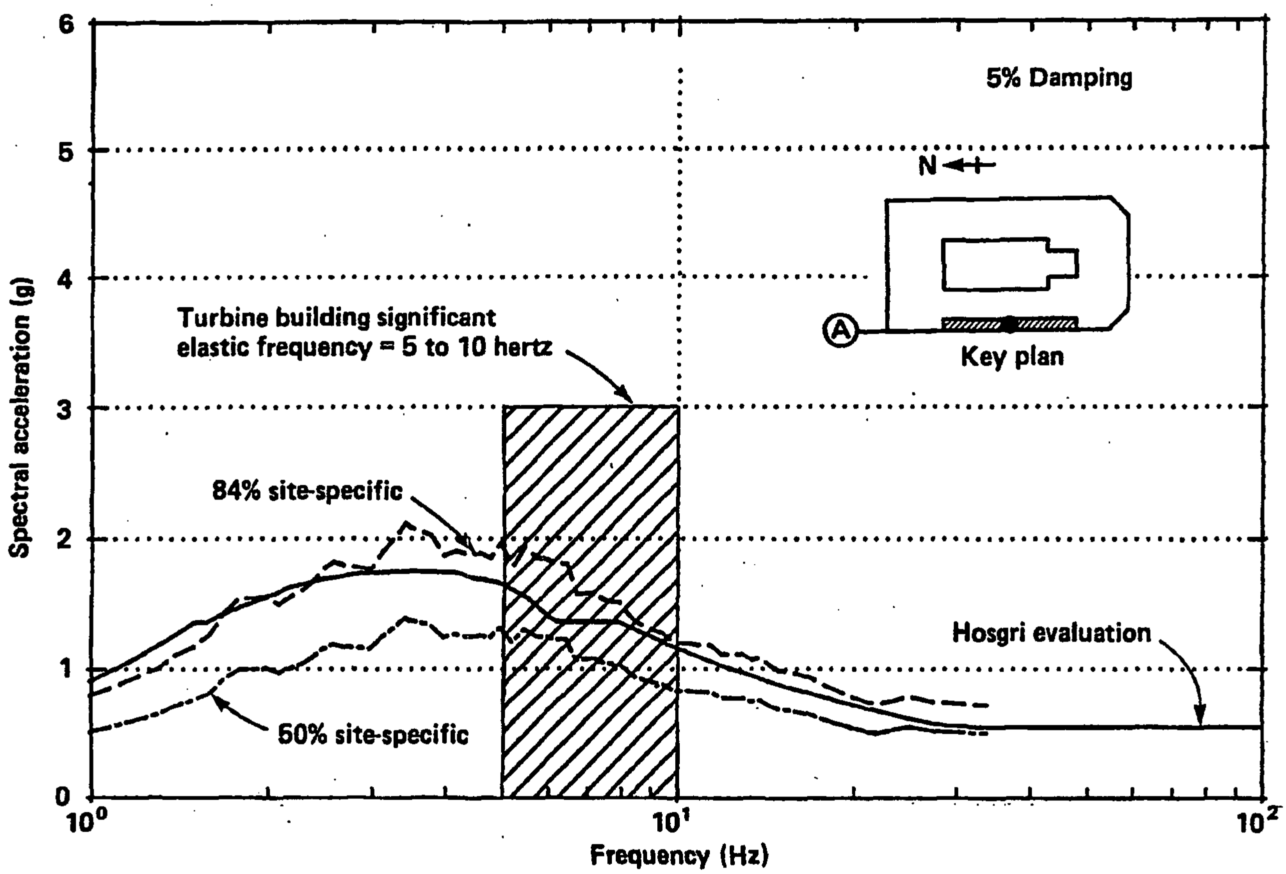


Figure 7-24

Comparison of spectra for motion at top of basemat, turbine building, wall A area, north/south response.

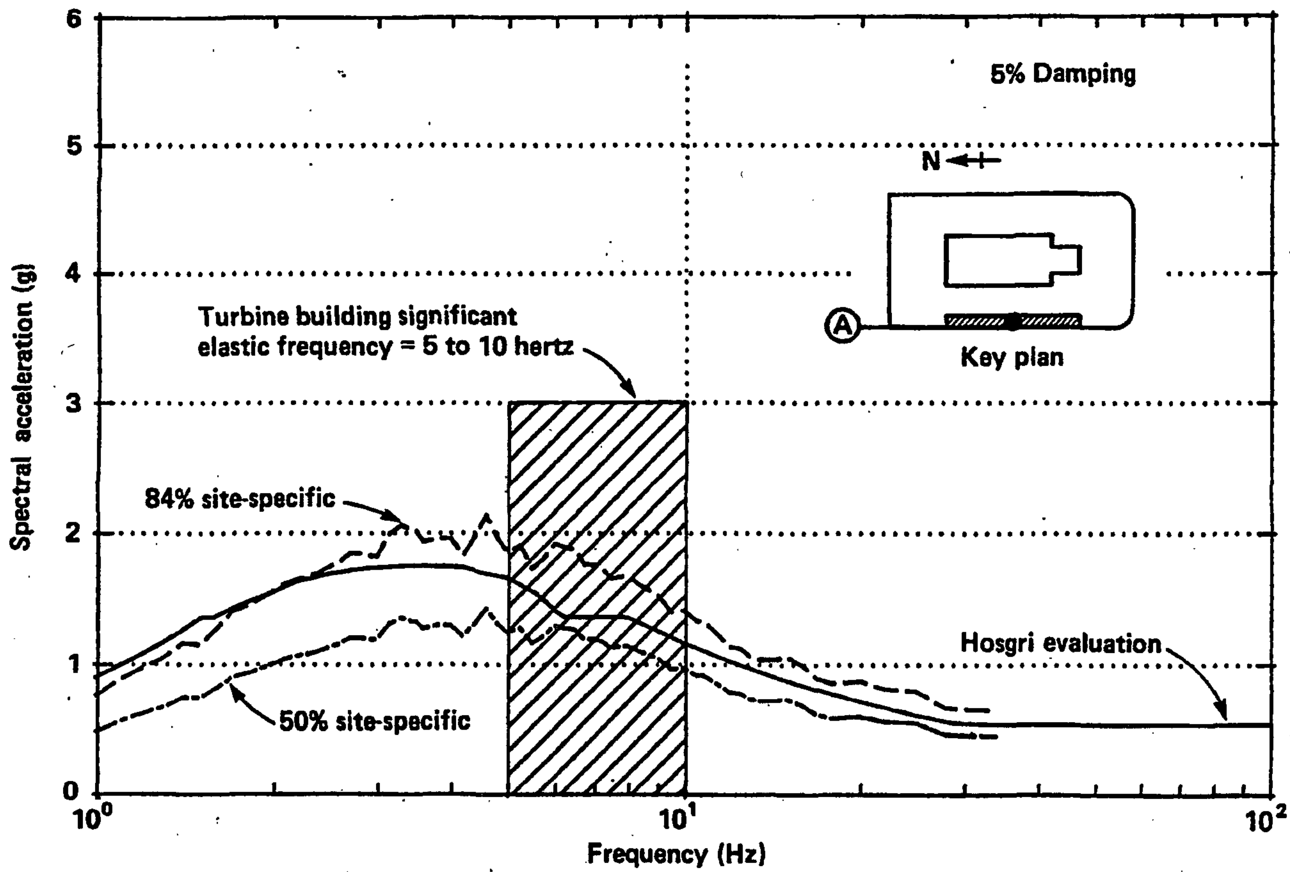


Figure 7-25

Comparison of spectra for motion at top of basemat, turbine building, wall A area, east/west response.

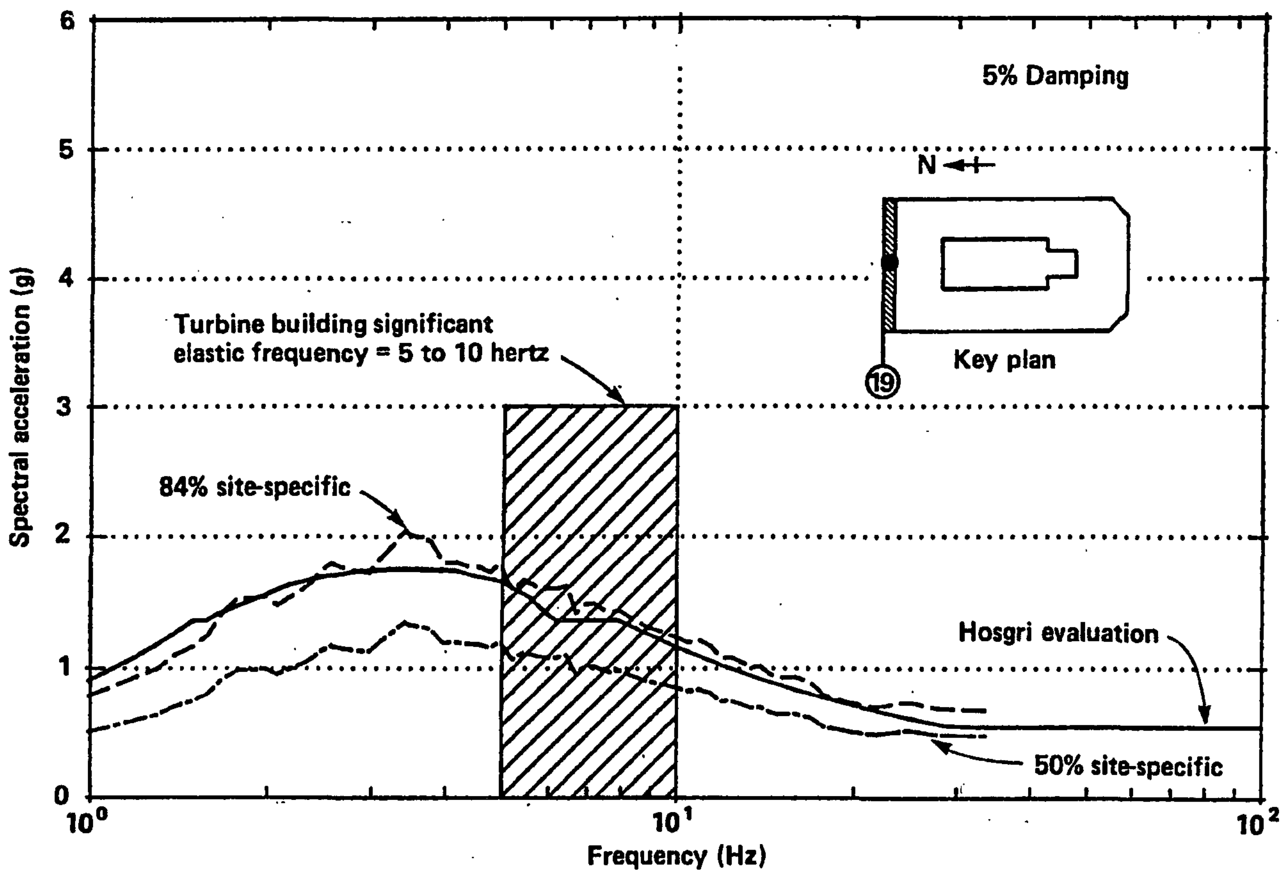


Figure 7-26

Comparison of spectra for motion at top of basemat, turbine building, wall 19 area, north/south response.

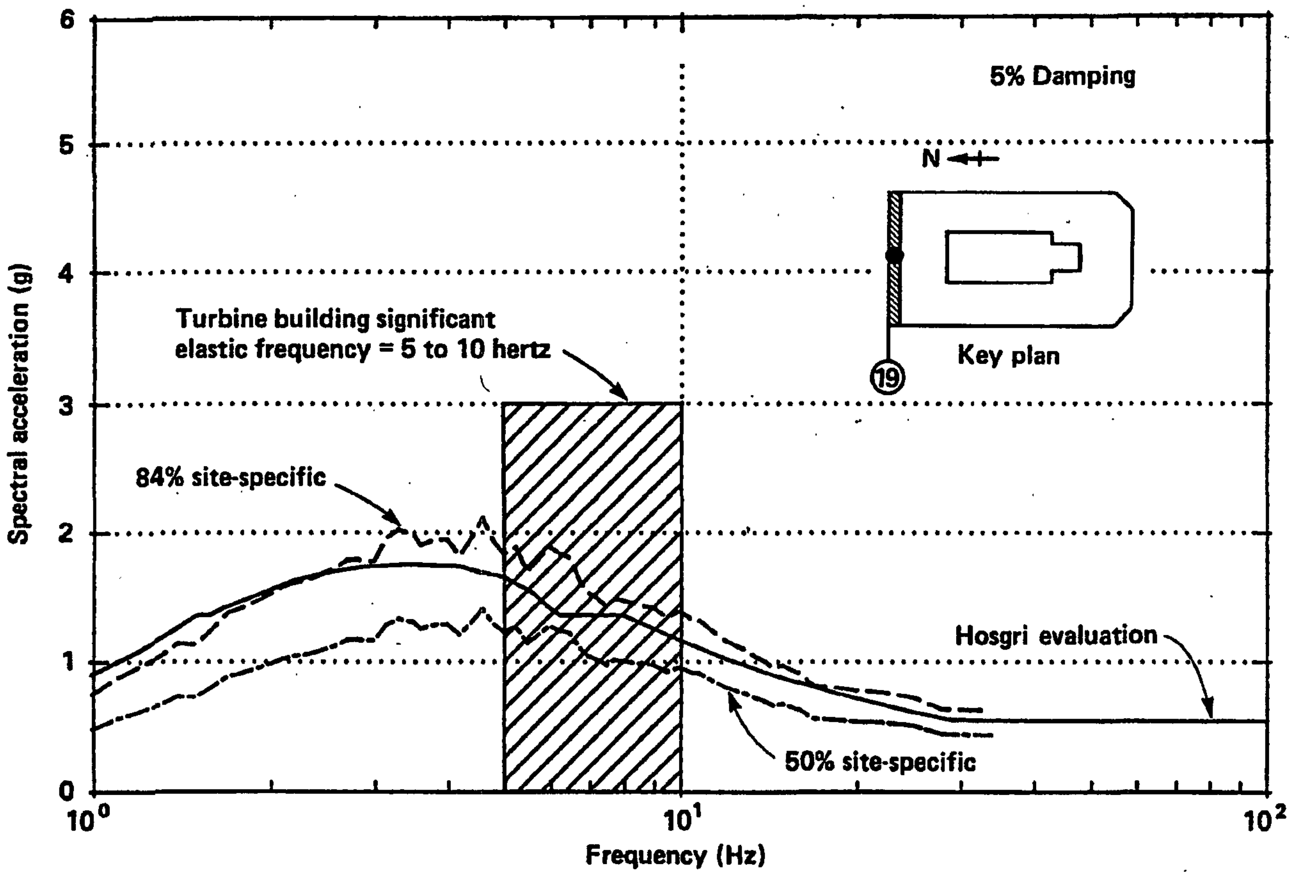


Figure 7–27

Comparison of spectra for motion at top of basemat, turbine building, wall 19 area, east/west response.

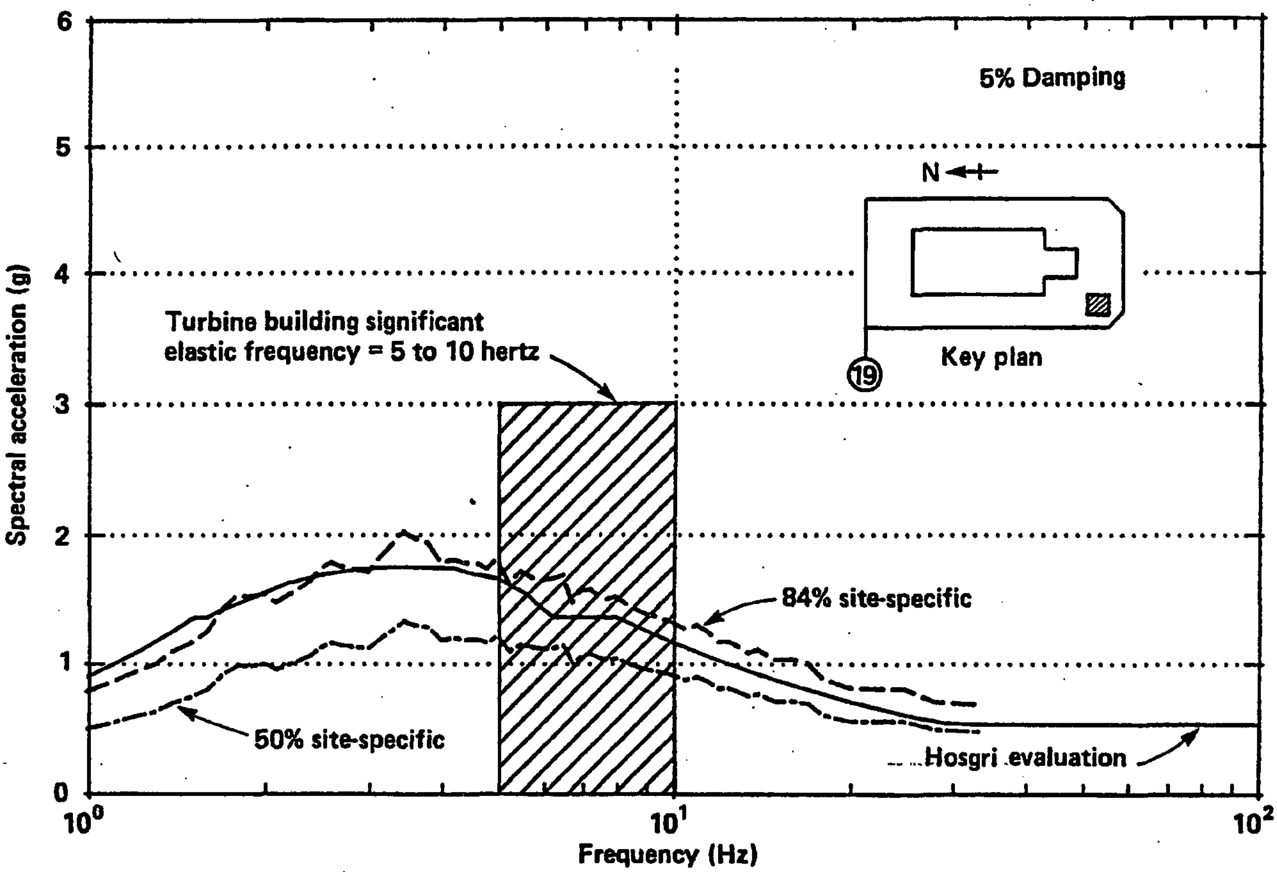


Figure 7–28

Comparison of spectra for motion at top of basemat, turbine building, diesel generator area, north/south response.

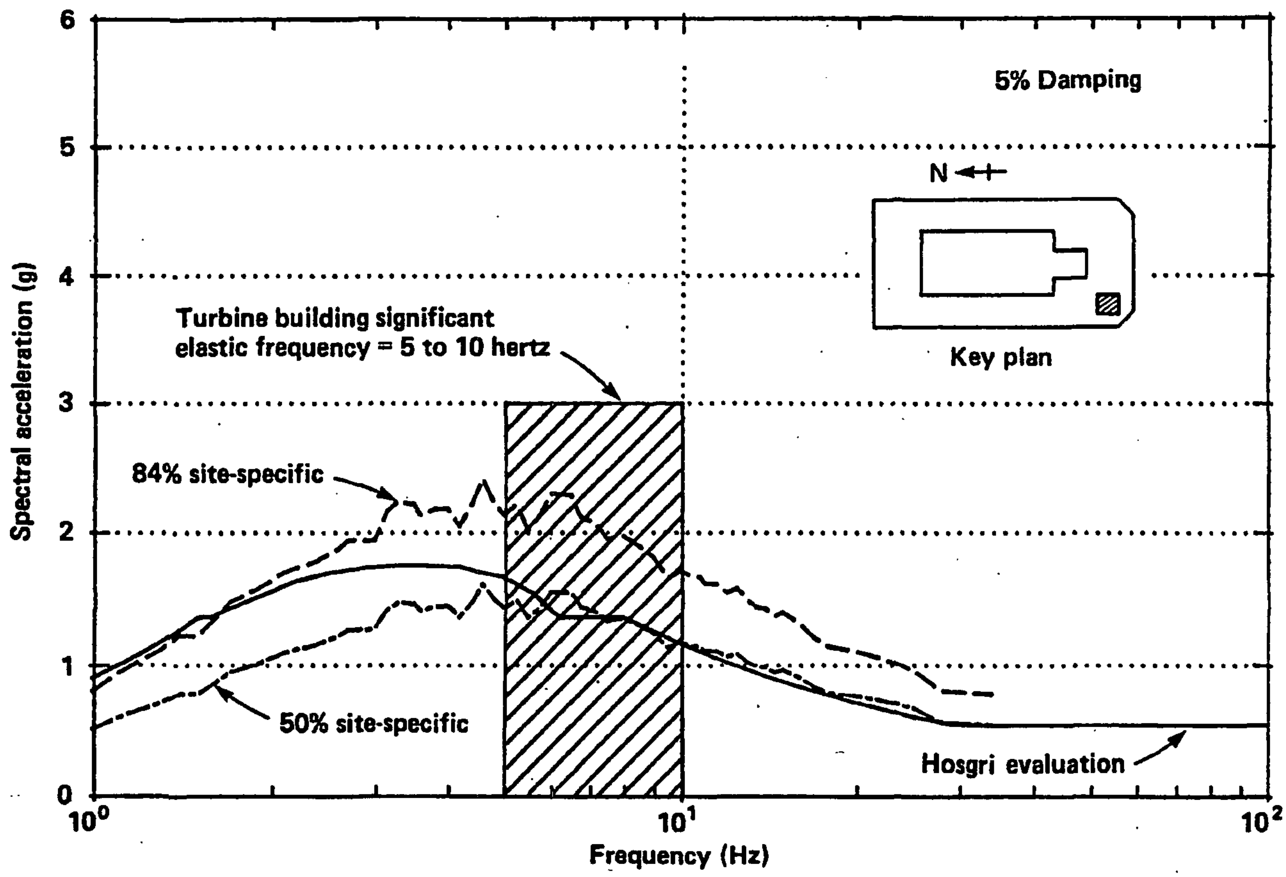


Figure 7-29

Comparison of spectra for motion at top of basemat, turbine building, diesel generator area, east/west response.

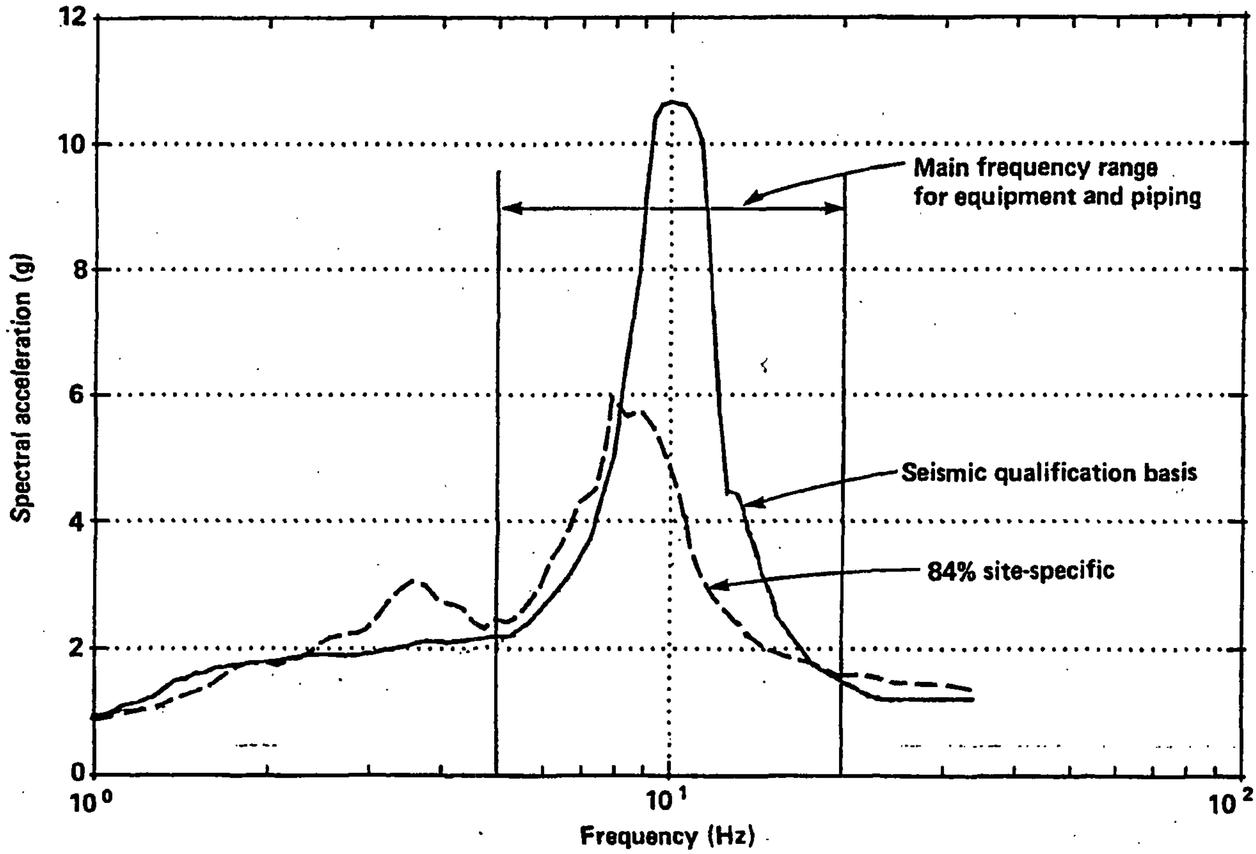


Figure 7-30

Comparison of floor response spectra for equipment qualification for the containment building, interior structure, El 140 feet, north/south response.

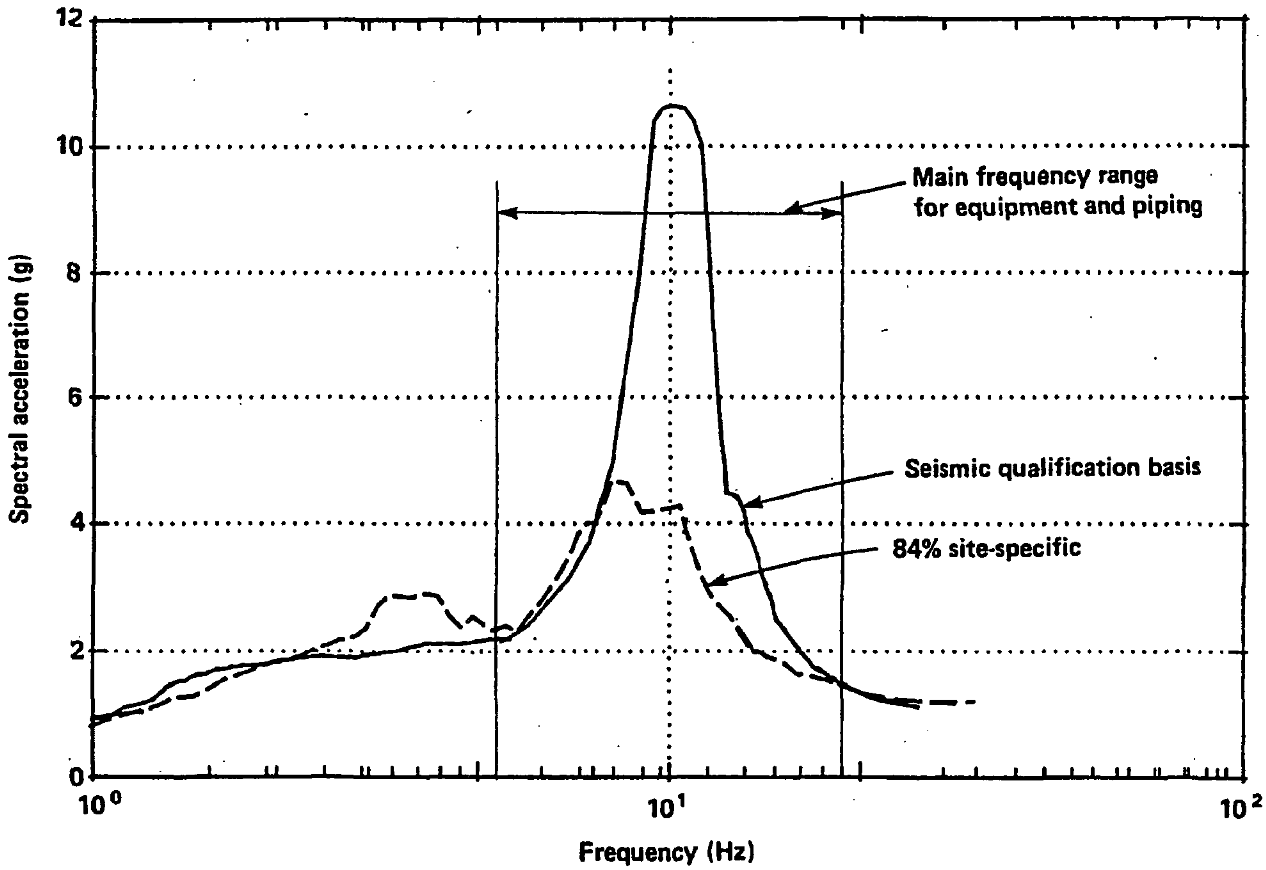


Figure 7-31

Comparison of floor response spectra for equipment qualification for the containment building, interior structure, El 140 feet, east/west response.

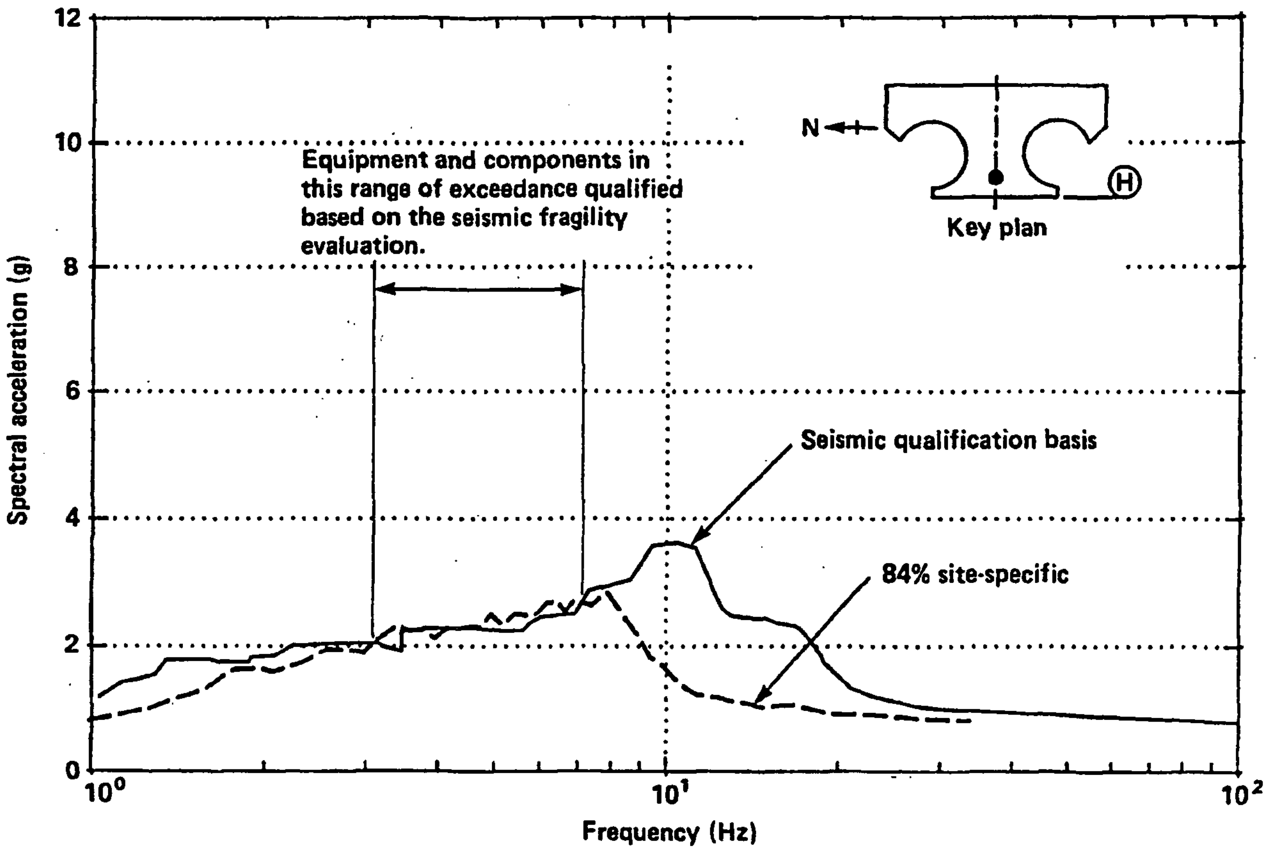


Figure 7-32

Comparison of floor response spectra for equipment qualification for the auxiliary building, El 105 feet, north/south response.

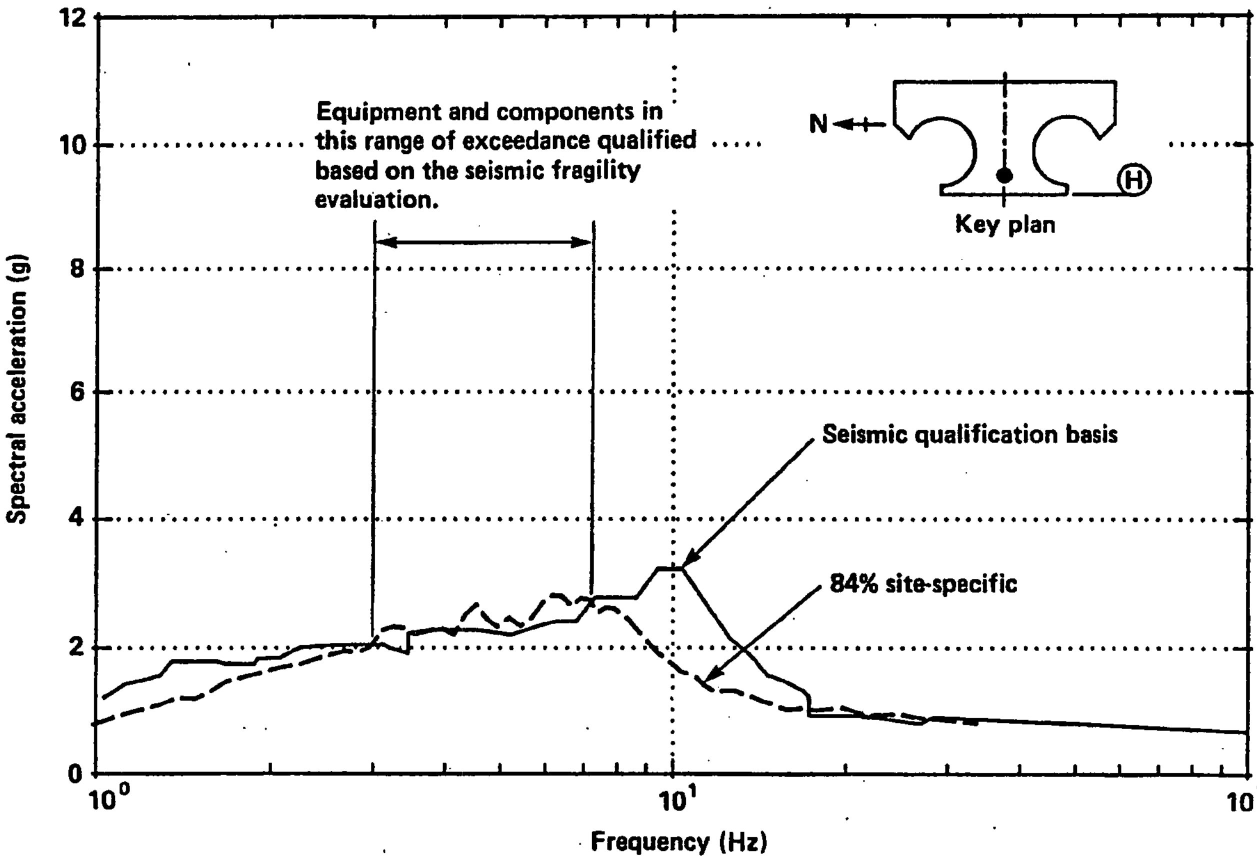


Figure 7–33

Comparison of floor response spectra for equipment qualification for the auxiliary building, El 105 feet, east/west response.

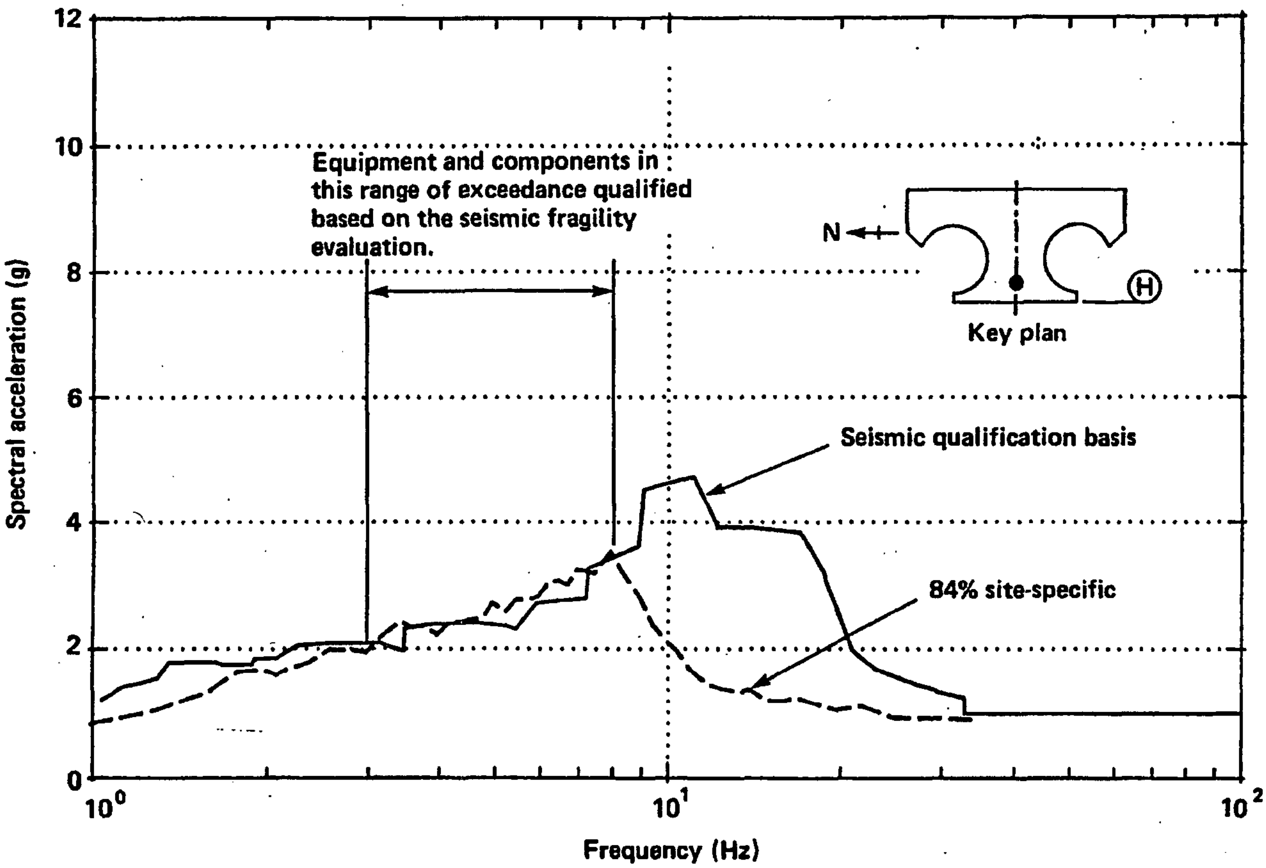


Figure 7-34

Comparison of floor response spectra for equipment qualification for the auxiliary building, El 115 feet, north/south response.

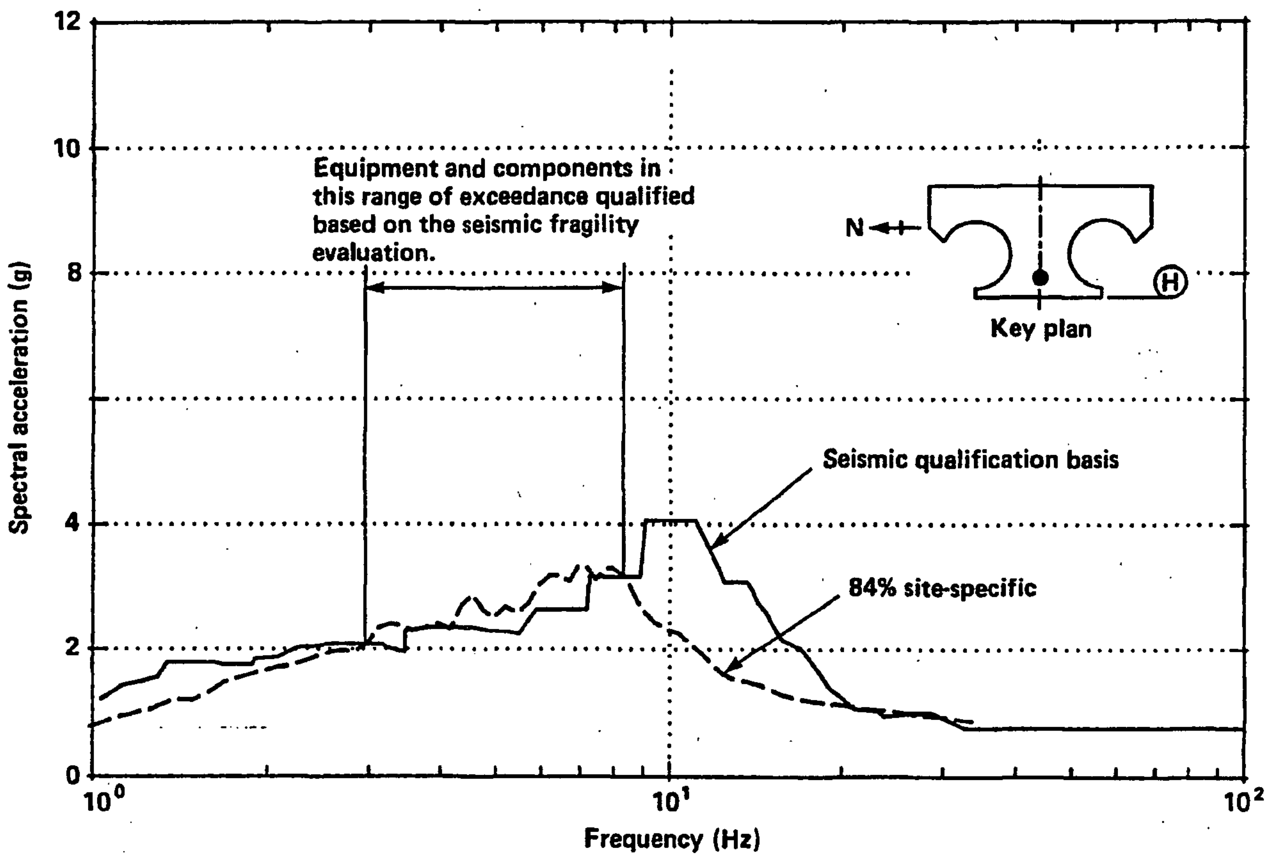


Figure 7-35

Comparison of floor response spectra for equipment qualification for the auxiliary building, El 115 feet, east/west response.

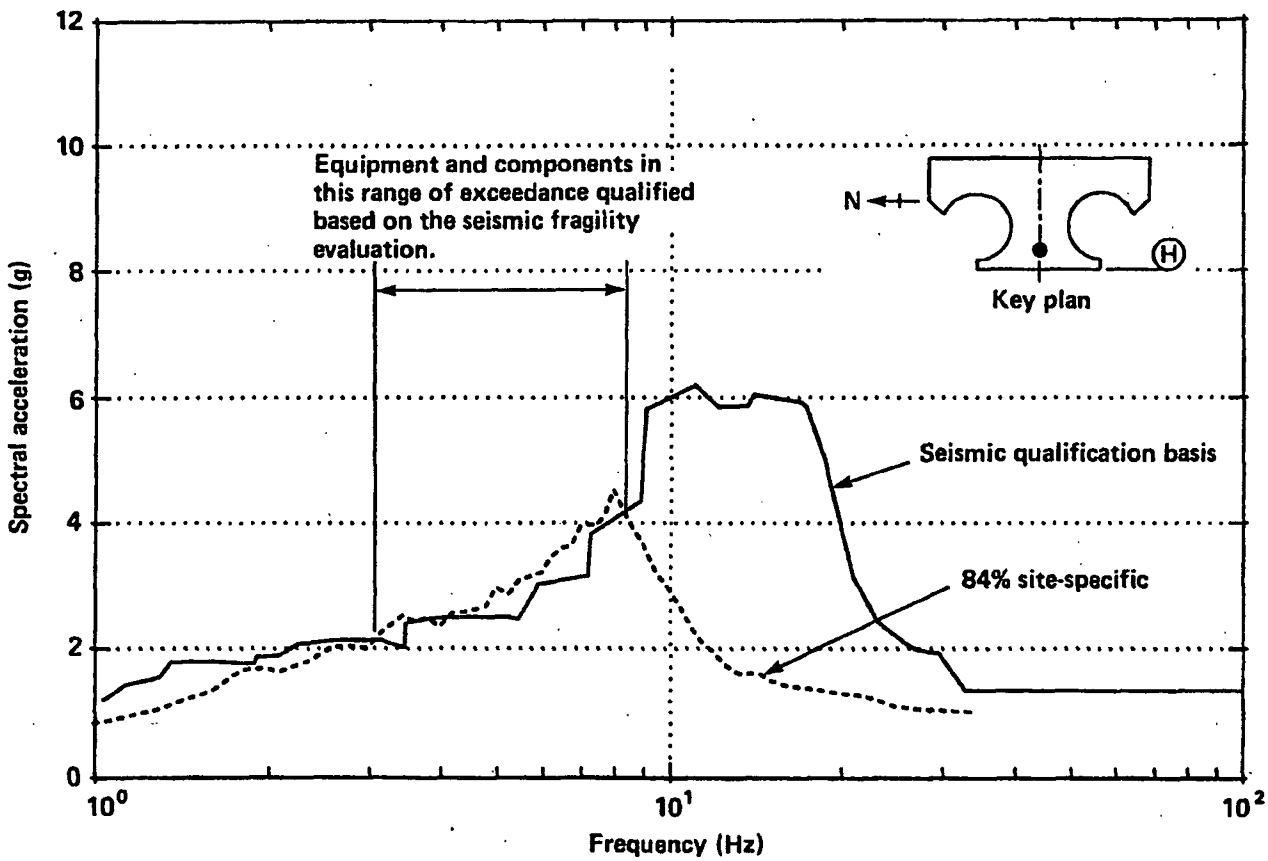


Figure 7-36

Comparison of floor response spectra for equipment qualification for the auxiliary building, El 140 feet, north/south response.

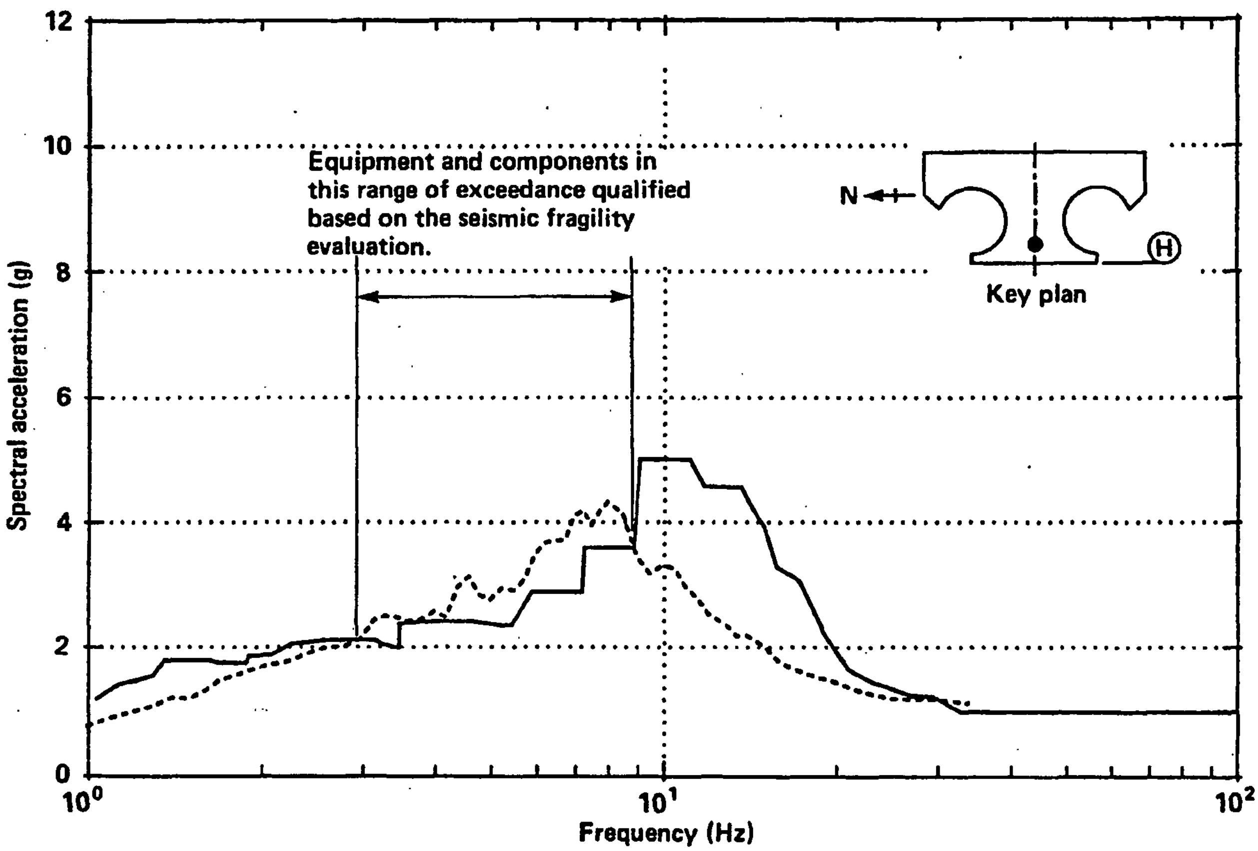


Figure 7-37

Comparison of floor response spectra for equipment qualification for the auxiliary building, El 140 feet, east/west response.

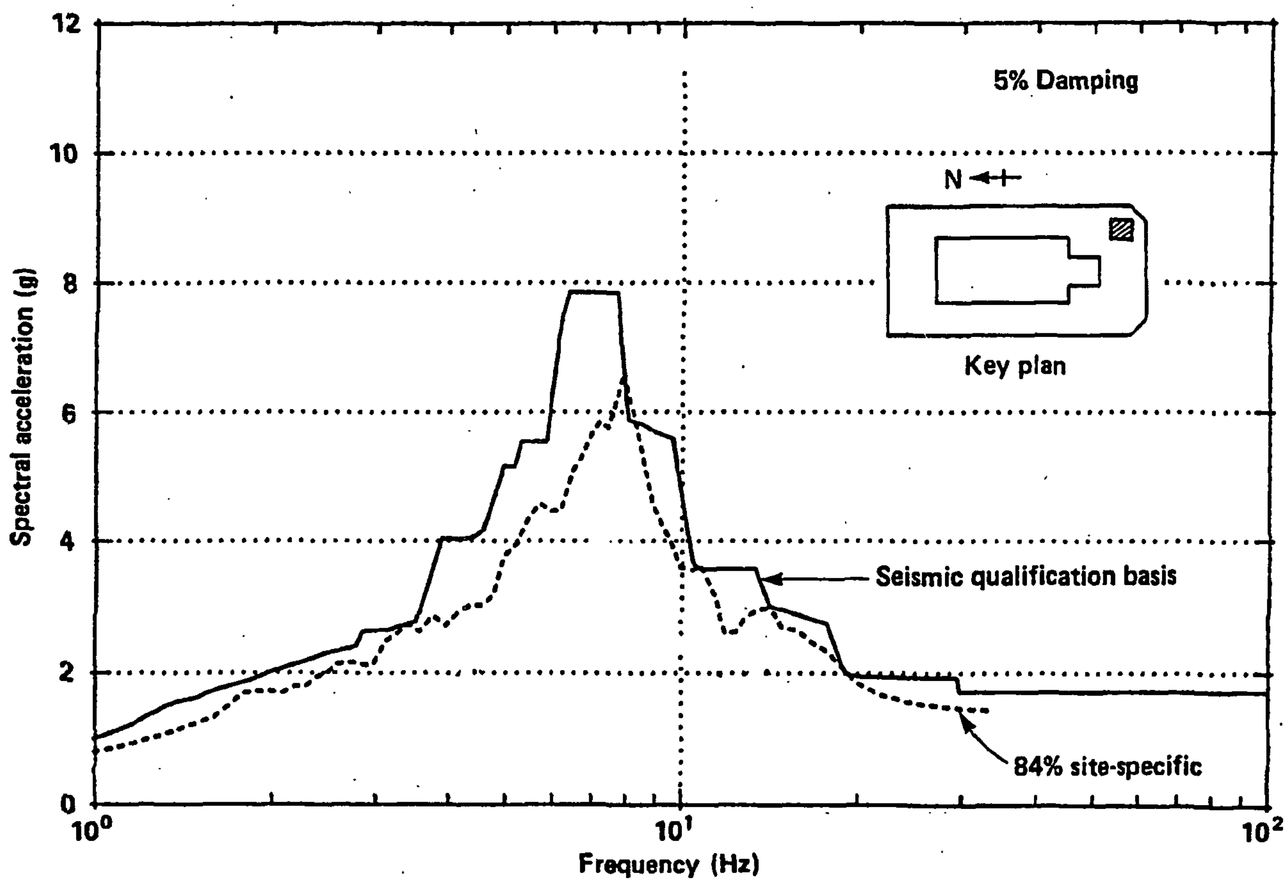


Figure 7-38

Comparison of floor response spectra for equipment qualification for the turbine building, El 119 feet, switchgear area, north/south response.

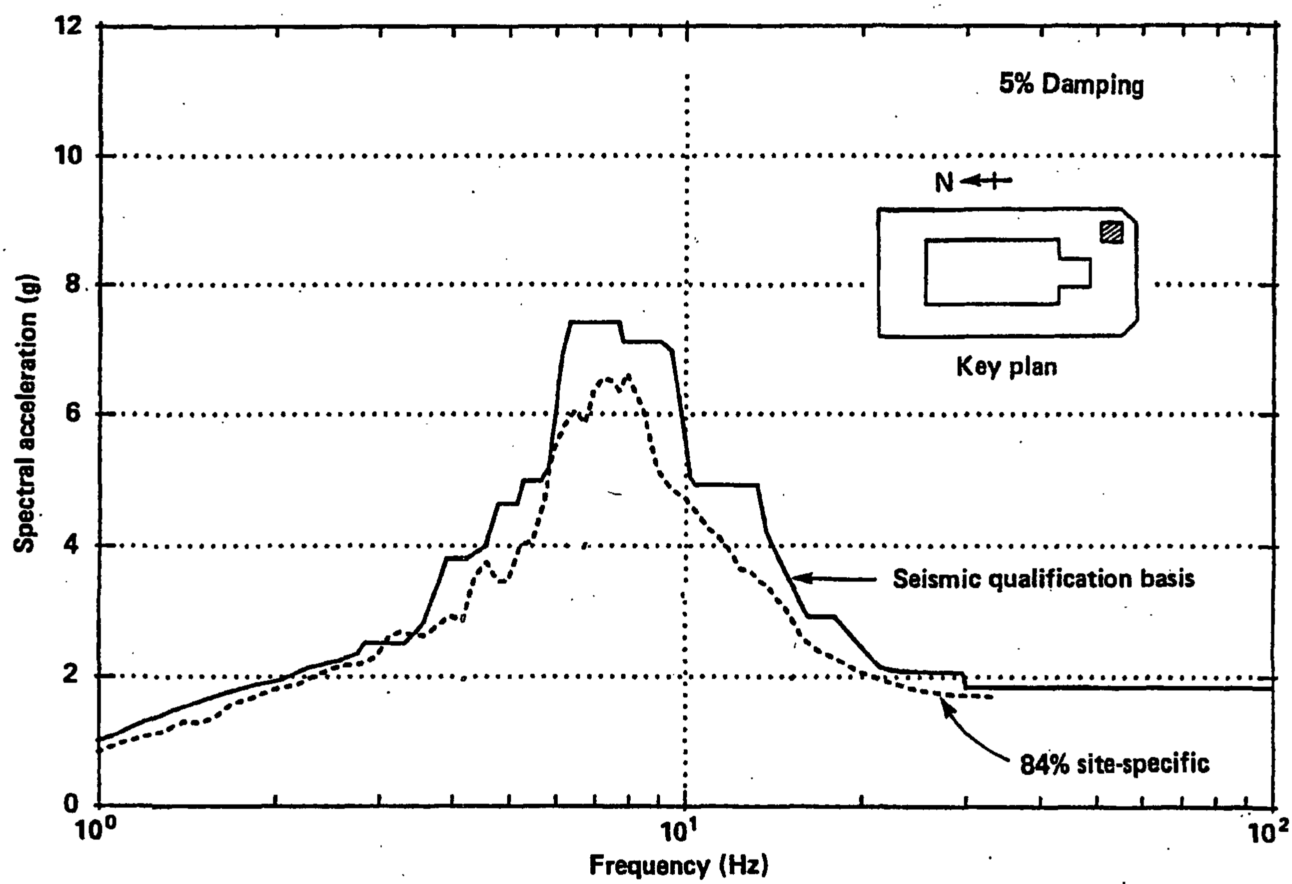


Figure 7-39

Comparison of floor response spectra for equipment qualification for the turbine building, El 119 feet, switchgear area, east/west response.

- For the auxiliary building (Figures 7-32 to 7-37), the seismic qualification basis spectra exceed the site-specific ground-motion in-structure spectra at all frequencies above about 8 hertz (by amounts up to 100 percent) and at frequencies between 3 and 8 hertz, they fall below the site-specific ground-motion in-structure spectra by an average of about 5 percent. However, equipment in the latter range is qualified on the basis of the seismic fragility evaluations as discussed later.

- For the turbine building (El 119 feet) (Figures 7-38 and 7-39), the site-specific ground-motion in-structure spectra, in general, are enveloped by the seismic qualification basis spectra. Any exceptions are insignificant and they can be accommodated in the design margin.

- For the majority of items of equipment that are essential to Plant seismic safety (therefore, included in the Plant system model used for the probabilistic risk assessment studies), the existing seismic qualification is unaffected, because the in-structure spectra are enveloped by the corresponding seismic qualification basis spectra.

- For those essential items of equipment whose seismic qualification basis spectra do not envelop the site-specific ground-motion in-structure spectra, seismic fragility evaluations (Chapter 6) and the seismic margin assessment described later in this chapter show that each of these items is qualified for the site-specific ground-motion spectra.

DETERMINISTIC SEISMIC MARGIN ASSESSMENT

Capacities for Structures, Systems, and Components

The Expert Panel on Quantification of Seismic Margins organized and funded by the Nuclear Regulatory Commission has recommended that high-confidence-of-low-probability-of-failure (HCLPF) seismic capacity estimates be used in seismic margin evaluations of nuclear power plants (Budnitz and others, 1985). Several authors have suggested that these seismic capacities can be back-calculated from full fragility curves used in seismic probabilistic risk assessment studies (Campbell, 1987; Kennedy, 1984; Prassinos and others, 1986). This method has been endorsed by the NRC Seismic Design Margins Working Group.

The HCLPF capacities of structures, systems and components back-calculated from full fragility curves are presented in this section. As part of the probabilistic risk assessment, fragility descriptions were developed for structures and major mechanical and electrical systems required for safe shutdown. In all cases, the fragility analyses were based on Plant-specific structures or equipment seismic qualification analyses directly related to elements in place at the Diablo Canyon Plant. The structure, system, and component fragility descriptions were used as inputs to systems analysis models and HCLPF capacities were developed for each.

All fragility estimates presented in Chapter 6 and used in the seismic probabilistic risk assessment were defined in terms of a free-field ground-surface control motion response spectral shape anchored to an average 5 percent-damped spectral acceleration ($\bar{S}_a$), averaged over the 3.0 to 8.5 hertz frequency range. Therefore, all HCLPF capacities are also defined in terms of average spectral acceleration and this same spectral shape. Because the spectral shape used for fragility estimates is nearly identical to the site-specific ground-motion 84 percent probability of nonexceedance spectral shape when both are anchored to the same 5 percent damped average spectral acceleration (Figure 7-3), the appropriate HCLPF capacities in terms of average spectral acceleration can be compared directly to the average spectral acceleration for the site-specific ground-motion spectra:

Site-Specific Ground-Motion Spectra:

50 percent probability of nonexceedance $\bar{S}_a$ = 1.30 g

84 percent probability of nonexceedance $\bar{S}_a$ = 1.94 g

It should be noted that the HCLPF capacities represent a conservative estimate of seismic capacity and that direct comparisons of appropriate HCLPF capacities with the site-specific ground motion $\bar{S}_a$ provide a very conservative estimate of the seismic margin of the plant. In this regard the Expert Panel on Quantification of Seismic Margins (Page 5-2 of Budnitz and others, 1985) has stated:

> *The measure of margin adopted by the Panel is a high-confidence-of-low-probability-of-failure (HCLPF) capacity. This is a conservative representation of capacity and in simple terms corresponds to the earthquake level at which, with considerable confidence, it is extremely unlikely that failure of the component will occur. From the mathematical perspective of a probability distribution on capacity developed in seismic PRA calculations, the HCLPF capacity values are approximately equal to a 95 percent probability of not exceeding about a five percent probability of failure.*
>
> *There is a margin above the conservative capacity values selected by the Panel. The median capacity, which corresponds to the 50 percent probability of exceedance, is generally at least a factor of 2 greater than the HCLPF capacity. Thus, there is no proverbial "cliff" or sudden failure which is expected to occur immediately beyond the HCLPF capacity. From another perspective, the conservative capacities are close to the lower-bound cutoff values below which there is no significant likelihood of failure.*

These points should also be considered in evaluating the comparisons made in the following sections of this report.

DEVELOPMENT OF COMPONENT HIGH-CONFIDENCE-OF-LOW-PROBABILITY-OF-FAILURE CAPACITIES

High-confidence-of-low-probability-of-failure capacity estimates may be directly computed from the fragility estimates (Chapter 6) by (Budnitz and others, 1985; Kennedy, 1984):

$$\text{HCLPF } \bar{S}_a = \check{\bar{S}}_a e^{-1.65(\beta_R + \beta_U)}$$

where $\check{\bar{S}}_a$ = median spectral acceleration capacity
β_R = logarithmic standard deviation for randomness
β_U = logarithmic standard deviation for uncertainty

However, the fragility estimates provided in Chapter 6 include consideration of both peak-and-valley variability and directional variability of spectral response at any given frequency for any given response direction, and assume a 50 percent probability that these sources of variability will increase the response of any component above its median response estimate. Inclusion of both peak-and-valley and directional variabilities in the fragility analysis method results in a reduction in the estimated HCLPF capacity. The fragility analysis method is primarily intended for use in seismic probabilistic risk assessment studies, and defining the site-specific ground-motion spectrum at the 50 percent probability of nonexceedance level is consistent with this usage.

Such HCLPF capacity estimates may be directly compared to the 50 percent probability of nonexceedance site-specific ground-motion average spectral acceleration. Both the HCLPF capacity estimate and the site-specific ground-motion spectrum assume that at any frequency, the spectral acceleration is equally likely either to exceed or to fall below the smoothed spectrum shape.

On the other hand, direct comparison of HCLPF capacities from the fragility analysis method with an 84 percent probability of nonexceedance site-specific ground-motion average spectral acceleration results in unintentional double-counting of the effects of peak-and-valley and directional variabilities, because these variabilities are considered both in reducing the HCLPF capacity and in increasing the 84 percent probability of nonexceedance site-specific ground-motion level. To avoid this double-counting of the effects of peak-and-valley and directional variabilities, the fragility median and resultant HCLPF capacities must be modified before being compared to an 84 percent probability of nonexceedance site-specific ground-motion average spectral acceleration. This point has been recognized and the appropriate modifications have been made in past seismic margin reviews.

The 84 percent probability of nonexceedance site-specific spectrum defined in Chapter 4 is for the average of the two horizontal components. As such, it does not include directional variability. Therefore, for comparison with this spectrum, the HCLPF capacities from the fragility analysis method only needs to be corrected for peak-and-valley variability effects, and not for directional variability effects.

When median and HCLPF capacities from the fragility analysis method are to be directly compared with a desired 84 percent probability of nonexceedance site-specific ground-motion level, these median and HCLPF capacities must first be redefined so that they are appropriate for the control motion response spectrum being defined at the 84 percent level instead of the 50 percent level. Redefined $HCLPF_{84}$ and $MEDIAN_{84}$ capacities appropriate for an 84 percent probability of nonexceedance site-specific ground-motion level are given by:

$$HCLPF_{84} = (R_{84/50}) \bullet HCLPF$$

$$MEDIAN_{84} = (R_{84/50}) \bullet MEDIAN \qquad (7\text{-}1)$$

where $(R_{84/50})$ represents the ratio of the 84 percent probability of nonexceedance response spectral acceleration for the average of the two horizontal response components at a given frequency to the 50 percent probability of nonexceedance response spectral acceleration for the average of the two horizontal components considering only the peak-and-valley variabilities that have been included in the fragility evaluations.

The peak-and-valley variabilities included in the fragility evaluations for Diablo Canyon were based upon a study of these sources of variability using 38 pairs of ground-motion records for two horizontal components considered appropriate for the spectral accelerations at the Diablo Canyon site. All 38 pairs of two horizontal components were first scaled linearly over all frequencies to produce the same average spectral acceleration over the frequency range of 4.8 to 14.7 hertz for the average of the two horizontal components. The ratio $(R_{84/50})$, to account for peak-and-valley variabilities, was then obtained at many different frequencies from these 38 pairs of two horizontal components. For frequencies of 3.5 hertz and greater, the ratio $(R_{84/50})$ associated with peak-and-valley variability is reasonably constant and averages 1.20. At frequencies below 3.5 hertz, this ratio rapidly increases to about 1.55 near 3 hertz. Thus:

$$R_{84/50} = 1.20 \qquad (7\text{-}2)$$

The ratios defined by equation (7-2) were included in the current fragility evaluations, which are conditional on definition of the site-specific ground motions at the 50 percent probability of nonexceedance level.

When the fragility evaluation HCLPF capacities are compared to a desired 84 percent probability of nonexceedance level, they should first be scaled by equation 7-2 to obtain $HCLPF_{84}$ seismic capacities that are conditional on 84 percent probability of nonexceedance ground motions.

Tables 7-1 and 7-2 present HCLPF capacities for each structure and equipment items included in the seismic probabilistic risk assessment. Capacities appropriate for comparison with both the 50 and 84 percent probability of nonexceedance ground motions are presented. In accordance with past practice, the comparison at the 84 percent probability of nonexceedance level will be emphasized. These HCLPF capacities are reported in terms of the 5 percent damped average spectral acceleration in the 3 to 8.5 hertz frequency range.

Table 7-1

STRUCTURE HCLPF CAPACITIES

Structure	HCLPF Spectral Acceleration Capacity (g)[1] 50%[2]	84%[3]
Containment Building	3.34	4.01
Concrete Internal Structure	2.98	3.58
Intake Structure	3.23	3.88
Auxiliary Building	2.66	3.19
Turbine Building	1.84	2.21
Refueling Water Storage Tank	3.40	4.08
Condensate Storage Tank	>5	>5
Diesel Generator Fuel-Oil Storage Tank	>5	>5
Auxiliary Saltwater Piping	4.85	5.82

NOTES:

[1]Values quoted are referenced to average spectral acceleration between 3 and 8.5 hertz for free-field motions.

[2]Values quoted from fragility evaluation in Table 6-23, Chapter 6.

[3]Values determined from $HCLPF_{50}$ multiplied by 1.20 (see text for explanation). These values are to be compared with site-specific ground-motion demand $[\overline{S}_a]_{3\text{-}8.5\ \text{hertz}} = 1.94$ g.

Table 7-2

EQUIPMENT HCLPF CAPACITIES

System and Component	[redacted]	HCLPF Spectral Acceleration Capacity (g)[1] 50%[2]	HCLPF Spectral Acceleration Capacity (g)[1] 84%[3]
Nuclear Steam Supply	[redacted]		
Reactor Pressure Vessel		3.34	4.01
Reactor Internals		3.55	4.26
Steam Generators		2.55	3.06
Pressurizer		3.33	4.00
Pressurizer Safety Valves		>3	>3
Power Operated Relief Valves		2.32	2.78
Reactor Coolant Pumps		2.83	3.40
Control Rod Drives		3.40	4.08
MSSS Piping		>3	>3
Residual Heat Removal			
RHR Pumps		3.35	4.02
RHR Heat Exchangers		3.48	4.18
Safety Injection			
SI Accumulators		4.53	5.44
SI Pumps		4.64	5.57
Boron Injection Tank		3.96	4.75
Component Cooling Water			
CCW Pumps		3.74	4.49
CCW Heat Exchangers		2.55	3.06
CCW Surge Tank		2.91	3.49
Chemical and Volume Control			
Charging Pumps (Centrifugal)		4.45	5.34
Charging Pumps (Reciprocal)		>3	>3
Auxiliary Saltwater			
Auxiliary Saltwater Pumps		>3	>3
Containment Spray			
CS Pumps		3.85	4.62
Spray Additive Tank		3.07	3.68
Main Steam			
MS Isolation Valves		>3	>3
MS Safety Valves		>3	>3
MS PORV's		3.51	4.21
Auxiliary Feedwater			
AFW Pumps (Motor Driven)		>3	>3
AFW Pumps (Turbine Driven)		3.38	4.06

Table 7-2 *(Continued)*

EQUIPMENT HCLPF CAPACITIES

System and Component	[redacted]		HCLPF Spectral Acceleration Capacity (g)[1] 50%[2]	84%[3]
Diesel Generator				
DG Fuel Oil Day Tank	[redacted]		>3	>3
DG Fuel Oil Pumps/Filters	[redacted]		3.65	4.38
DG Fuel Oil Shutoff Valve	[redacted]		>3	>3
DG Air Start Compressor	[redacted]		>3	>3
DG Air Start Receiver	[redacted]		>3	>3
Diesel Generators	[redacted]		3.64	4.37
DG Radiator/Water Pump	[redacted]		3.66	4.39
DG Inlet Silencer/Air Filter	[redacted]		>3	>3
DG Excitation Cubical	[redacted]			3.08
DG Control Panel	[redacted]	Chatter	4.08	4.90
	[redacted]	Structural	2.24	2.69
DG Main Lead Terminal/Box	[redacted]			
Containment Building Ventilation				
Containment Fan Cooler	[redacted]		2.82	3.38
Control Room Ventilation				
Supply Fans	[redacted]		3.82	4.58
AC Units/Compressor	[redacted]		>3	>3
Control Cabinets	[redacted]		>3	>3
480V Switchgear/Inverter/DC Switchgear/Spreading Room Ventilation				
Supply/Return Fans	[redacted]		3.95	4.74
Backdraft and Shutoff Dampers	[redacted]		>3	>3
4160V (Vital) Electric Power				
Switchgear	[redacted]	Chatter	1.31	1.57
	[redacted]	Structural	2.95	3.54
Potential Transformers (Bus F)	[redacted]		3.47	4.16
(Bus G & H)	[redacted]		>3	>3
Safeguard Relay Panel	[redacted]		3.39	4.07
125V DC Electric Power				
Batteries	[redacted]		2.74	3.29
Battery Racks	[redacted]		5.40	6.48
Battery Chargers	[redacted]		2.93	3.52
Switchgear/Breaker Panels	[redacted]		2.36	2.83
120V AC Electric Power				
Instrument Breaker Panels	[redacted]		>3	>3
Inverters	[redacted]		2.75	3.30
480V (Vital) Electric Power				
460V/480V Transformers	[redacted]		2.42	2.90
Breaker Cabinets (Load Centers	[redacted]		>3	>3
Auxiliary Relay Panel	[redacted]		3.57	4.28

Table 7-2 *(Continued)*

EQUIPMENT HCLPF CAPACITIES

System and Component	[redacted]	HCLPF Spectral Acceleration Capacity (g)[1] 50%[2]	84%[3]
Control Room	[redacted]		
Main Control Boards	[redacted]	>3	>3
	[redacted]	2.98	3.58
Hot Shutdown Panel	[redacted]	3.22	3.86
	[redacted]	3.52	4.22
Auxiliary Safeguards Cabinet	[redacted]	>3	>3
MSSS Control	[redacted]		
Process Control and Protection System	[redacted]	3.57	4.28
Soild State Protection System	[redacted]	4.32	5.18
Reactor Trip Switchgear	[redacted]	3.14	3.77
Resistance and Temperature Detectors	[redacted]	>3	>3
Pressure and ΔP Transmitters	[redacted]	4.11	4.93
Miscellaneous Components	[redacted]		
Auxiliary Relay Rack	[redacted]	>3	>3
Local Starter Boards	[redacted]	>3	>3
Molded Case Circuit Breakers	[redacted]	>3	>3
Valve Limit Switches	[redacted]	>3	>3
Impulse Lines	[redacted]	2.63	3.16
Containment Purge Valves	[redacted]	>3	>3
Generic Components	[redacted]		
Penetrations/Penetration Boxes	[redacted]	2.83	3.40
80P Piping and Supports	[redacted]	3.00	3.60
Hand, Relief, Solenoid, and Check Valves	[redacted]	>3	>3
Air and Motor Operated Valves	[redacted]	3.57	4.28
Cable Trays and Supports	[redacted]	>3	>3
HVAC Ducting and Supports	[redacted]	2.49	2.99

NOTES:

[1]Values quoted are referenced to average spectral acceleration between 3 and 8.5 hertz for free-field motions.

[2]Values quoted from fragility evaluation in Table 6-24, Chapter 6.

[3]Values determined from $HCLPF_{50}$ multiplied by 1.20 (see text for explanation). These values are to be compared with site-specific ground-motion demand $[\bar{S}_a]_{3\text{-}8.5 \text{ hertz}} = 1.94$ g.

Margin Assessment

When compared to the average 5 percent damped spectral acceleration in the 3 to 8.5 hertz frequency range for the site-specific ground-motion spectra ($\bar{S}_a$ = 1.30 g and 1.94 g for the 50 percent and 84 percent probability of nonexceedance site-specific ground-motion spectra, respectively), it may be seen from Tables 7-1 and 7-2 that all structures and equipment items are found to have capacities greater than the earthquake review level, except for the 4-kV switchgear. As discussed above, this comparison may be made for either the 50 percent or 84 percent probability of nonexceedance level capacities. Figure 7-42 illustrates schematically the relationship of the structure and equipment items HCLPF capacities in reference to the site-specific ground-motion spectrum for the 84 percent probability of nonexceedance level, and the procedure for evaluating seismic margins.

The 4-kV switchgear relay chatter mode has the lowest $HCLPF_{84}$ capacity, 1.57 g, or 81 percent of the 84 percent probability of nonexceedance site-specific 5 percent damped average spectral acceleration of 1.94 g. However, the relay chatter function mode has a median capacity about 2.7 times its HCLPF capacity, 2.2 times as great at the 84 percent probability of nonexceedance site-specific average spectral acceleration. Thus, at this earthquake level, relay chatter of the 4-kV switchgear is highly unlikely. Furthermore, the consequences of 4-kV switchgear relay chatter are easily recoverable by operator action, as discussed in Chapter 6. Even though the 4-kV switchgear relay chatter fragility estimate was included in the seismic probabilistic risk assessment, it did not turn out to be a significant contributor to seismic risk because operators can reset any tripped circuits. Therefore, the 4-kV switchgear relay chatter $HCLPF_{84}$ capacity is not an appropriate descriptor of the plant seismic margin.

The second lowest $HCLPF_{84}$ capacity, 2.21 g, is for the overall turbine building structure. The reported $HCLPF_{84}$ capacity is a factor of 1.14 times greater than the 84 percent probability of nonexceedance site-specific average spectral acceleration, so that a 14 percent margin exists before this $HCLPF_{84}$ capacity is reached. Even if the demand were to reach this level, failure is unlikely, because the median capacity is estimated to be a factor of 2.65 greater than the HCLPF capacity. Furthermore, as will be discussed in the following section, there are several sources of conservatism in the estimation of the turbine building $HCLPF_{84}$ capacity, so the actual $HCLPF_{84}$ capacity margin over the 84 percent probability of nonexceedance site-specific ground-motion average spectral acceleration is likely to be more than 40 percent, rather than 14 percent.

The diesel generator control panel has the third lowest $HCLPF_{84}$ capacity, 2.69 g, which is a factor of 1.39 greater than the 84 percent probability of nonexceedance site-specific average spectral acceleration. The only other components having reported $HCLPF_{84}$ capacities less than 3.0 g are the power-operated relief valves on the primary system (2.78 g), and the 125-volt DC electric power switchgear/breaker panel (2.83 g), both of which exceed the site-specific ground-motion average spectral acceleration by a factor greater than 1.43.

Thus, except for the turbine building, all components whose failure could lead to seismic risk to the Plant have at least a 40 percent margin between the $HCLPF_{84}$ capacity and the 84 percent probability of nonexceedance site-specific ground motion. Conservatisms in the turbine building capacity evaluation are discussed below.

CONSERVATISMS IN TURBINE BUILDING STRUCTURE CAPACITY EVALUATION (FRAGILITY ESTIMATES)

Because the Plant fragility is governed by that for the turbine building, a further evaluation of the conservatism used in the turbine building analysis has been made (Kennedy and others, 1988). The results of this evaluation are summarized here.

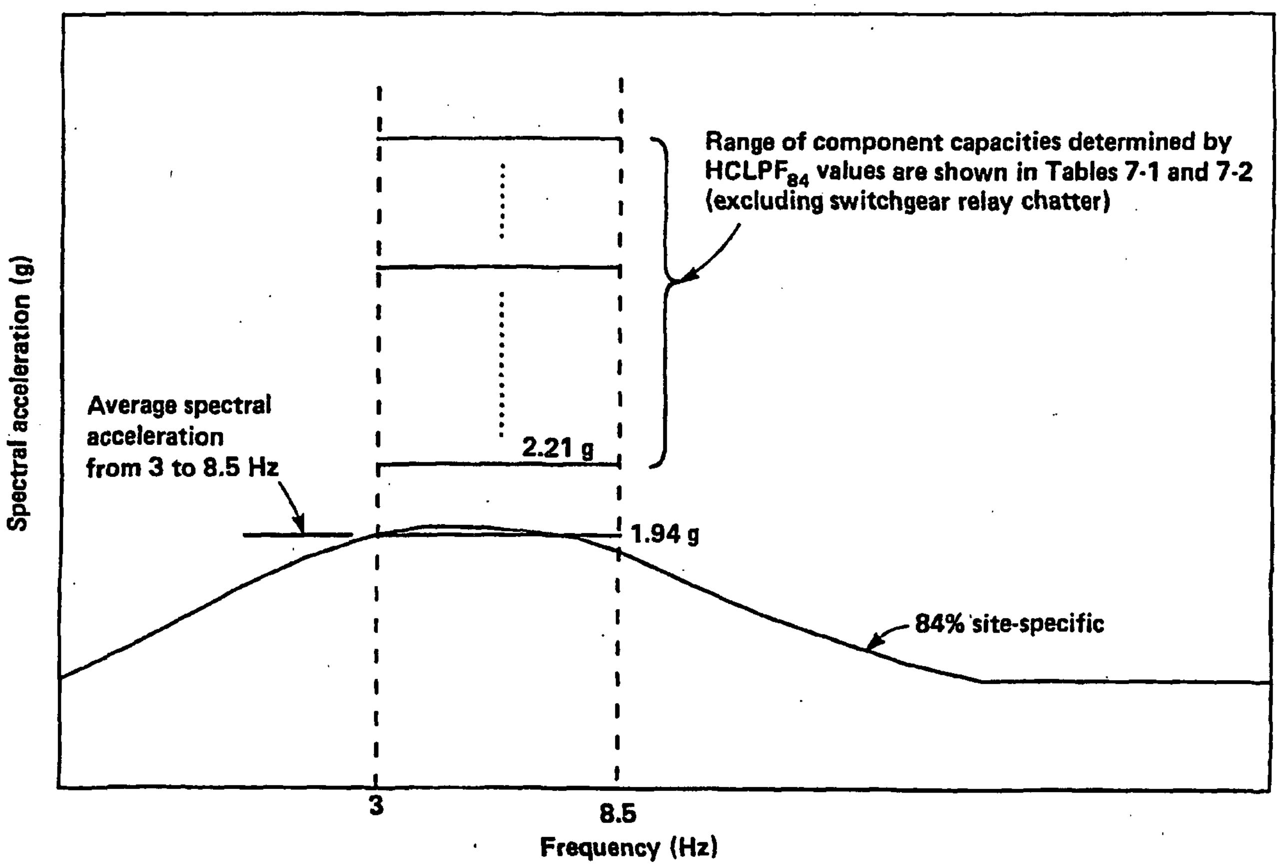


Figure 7–40
Schematic illustration for determining seismic margins.

A number of possible failure modes that could lead to overall severe distress of the turbine building were investigated using the standard fragility evaluation method for the seismic probabilistic risk assessment. It was concluded that the most probable cause of overall severe distress of the turbine building was substantial inelastic drift and strength degradation of the two major east/west load-carrying shear walls spanning from foundation level (El 85 feet) to the operating floor (El 140 feet). An extensive study was then performed to define the fragility estimate for the major east/west load-carrying shear walls. This study is summarized in Chapter 6. In this study, 200 nonlinear time-history analyses, using 25 different ground-motion time history inputs, were performed to define the fragility estimate.

The turbine building fragility estimate specifically applies to the onset of severe structural distress (significant strength degradation) to the major east/west shear walls. Structural distress generally does not correspond to partial collapse, depending on the power of the ground-motion record that remains after this state of distress is reached. Furthermore, partial collapse is likely to be well short of total collapse, even if partial collapse occurs. Even so, in the seismic probabilistic risk assessment study, and in this margin evaluation, the onset of structural distress of these shear walls has been conservatively used as a surrogate for a structure-induced failure of all safety equipment housed in the turbine building. This substitution introduces an indeterminate, but probably substantial conservatism.

Fragility statements on the onset of shear wall distress were anchored to the average 5 percent damped spectral acceleration in the 3 to 8.5 hertz frequency range for use in the seismic probabilistic risk assessment. Therefore, all 25 ground-motion records that were used in the nonlinear analyses were scaled upward so that each had the same average spectral accelerations $\bar{S}_a$. A total of 75 nonlinear analyses were performed at $\bar{S}_a = 3.0$ g and 6.0 g, and 50 nonlinear analyses were performed at $\bar{S}_a = 4.0$ g, with each ground-motion recording being used an equal number of times.

It was found that considerable differences existed in the computed shear wall drifts (measure of damage) for different ground-motion records when each was scaled to the same average spectral acceleration level. In fact, the records such as Tabas, Pacoima Dam, and Karakyr Point (Gazli) which actually had the highest average spectral acceleration and thus had to be scaled and modified the least to achieve a reference $\bar{S}_a$ such as 3.0 g, consistently produced lesser drifts (damage) than did the records that had greater scaling and modification. The records such as Temblor (Parkfield), Coyote Lake Dam (Morgan Hill, 1984), Pleasant Valley Pump Station (Coalinga), and Dayhook (Tabas) which had to be scaled upward and modified the most to produce a reference $\bar{S}_a = 3.0$ g consistently produced the largest drifts (damage) after being scaled upward to that level. Results using each ground-motion record were equally weighted; this decision produced a much lower HCLPF $\bar{S}_a$ capacity estimate than would have resulted if only the highest ground-motion records had been used. Basically, average spectral acceleration is one of the best single ground-motion parameters; it does not serve as a highly accurate descriptor of the capability of ground motions to damage the turbine building shear walls. Because of the large scatter of computed drifts for the same average spectral acceleration, the fragility variability factors β_R and β_U were significantly increased. This resulted in significant reduction of the HCLPF $\bar{S}_a$ capacity. When compared to a specific ground-response spectrum, such as the 84 percent probability of nonexceedance site-specific ground-motion spectrum, the single-parameter fragility method $HCLPF_{84}$ $\bar{S}_a$ capacity is conservatively biased, because it must also cover ground motions having differing spectral shapes.

The 75 deterministic time history analyses performed for the 25 ground-motion records each scaled to $\bar{S}_a = 3.0$ g provide a more precise multiple parameter description of the seismic margin of the turbine building shear walls than could be incorporated into the single parameter average spectral acceleration used in the seismic probabilistic risk assessment. It was found that large shear wall drifts (and thus, damage) only resulted when the ground motions produced high spectral accelerations, both at high frequency (8.6 to 9.5 hertz) associated with the elastic frequency of the shear walls, and at low frequency (1.7 to 2.8 hertz). Substantial spectral accelerations were needed at high frequency to drive the shear walls into the inelastic regime. Substantial low-frequency spectral acceleration was then needed to produce damaging levels of shear wall drift as the shear wall frequencies shifted due to inelastic behavior. In other words, very broad frequency content (1.7 to 9.5 hertz) was necessary within the ground-motion record to produce severe damage at $\bar{S}_a$ = 3.0 g. The required breadth of frequency content was not adequately captured by a single ground-motion parameter. An improved deterministic seismic margin HCLPF capacity statement is obtained by requiring both the following high frequency and either of the two low frequency limits to be exceeded:

HCLPF Limits

High Frequency Limit

Maximum $S_{a5\%} \geq 1.6$ g (within 8.6 to 9.5 hertz)

and

Low Frequency Limit

Maximum $S_{a5\%} \geq 2.8$ g (within 2.4 to 2.8 hertz)

or

Maximum $S_{a5\%} \geq 2.25$ g (within 1.7 to 2.0 hertz)

The 84 percent probability of nonexceedance site-specific ground motion 5 percent damped response spectrum gives spectral accelerations of 1.79 g, 1.83 g, and 1.55 g at 8.6 hertz, 2.8 hertz and 2.0 hertz, respectively. Thus, the high frequency HCLPF limit of the turbine building shear wall is exceeded and nonlinear drift is possible. However, the low-frequency HCLPF limits are well above the 84 percent probability of nonexceedance site-specific ground-motion response spectrum, so nonlinear drifts cannot become large. The ratio at 2.8 hertz is 2.8/1.83 = 1.53, and at 2.0 hertz, is 2.25/1.55 = 1.45. Thus, there is a 45 percent margin between the 84 percent site-specific ground-motion response spectrum and the deterministically defined turbine building shear wall HCLPF capacity.

Multiple nonlinear time-history analyses were not performed for other failure modes of the turbine building. These failure modes have $HCLPF_{84}$ $\bar{S}_a$ capacities back-calculated from fragility estimates that are greater than the 2.21 g obtained for the turbine building east/west shear walls. It is our judgment that these other failure modes would have multiple parameter deterministic HCLPF capacities also greater than those for the turbine building east/west shear walls.

Defining seismic margin as the difference between the appropriate HCLPF capacity and the 84 percent probability of nonexceedance site-specific ground-motion average spectral acceleration, the following conclusions can be reached:

- The seismic margin for the turbine building is at least 14 percent, and most likely in excess of 40 percent.
- The HCLPF capacity does not represent a "cliff" beyond which failure immediately occurs. Instead, it is close to a lower-bound, below which there is no significant likelihood of failure. The median capacity for the turbine building is more than 2.5 times its HCLPF capacity, and the probability of failure would only gradually increase if the ground motion were to exceed the HCLPF capacity.
- Other than the turbine building, all structures and equipment items whose failure could lead to seismic risk to the Plant have at least a 40-percent margin between the HCLPF capacity and the 84 percent probability of nonexceedance site-specific ground motion.

Conclusions of Margin Assessment

The deterministic evaluation discussed in this chapter demonstrates that the Diablo Canyon Plant has adequate margin to accommodate the site-specific ground motions for the maximum earthquake on the Hosgri fault, as evidenced by the following:

- The containment building has a seismic margin of at least 100 percent and the auxiliary building has a seismic margin of at least 70 percent. The most critical structure was identified as the turbine building, which has a seismic margin at least 14 percent, and most likely in excess of 40 percent.

- Among the plant components, relay chatter of the 4-kV switchgear may occur. However, the switchgear structure has ample margin to accommodate demands due to the site-specific ground motions. Therefore, no structural failure will occur. Also, the ease of recovery and specific plant procedures essentially eliminate any concern due to relay chatter.

- For all components, except the 4-kV switchgear, the minimum seismic margin is shown to be in excess of 40 percent.

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