RAS 3749

November 9, 2001

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

DOCKETED
USNRC

2002 JAN -8 AM 10: 44

OFFICE OF THE SECRETARY
RULEMAKINGS AND
ADJUDICATIONS STAFF

In the Matter of	)	
	)	Docket No. 72-22
PRIVATE FUEL STORAGE L.L.C.	)	
	)	ASLBP No. 97-732-02-ISFSI
(Private Fuel Storage Facility)	)	

APPLICANT'S MOTION FOR SUMMARY DISPOSITION OF PART B OF UTAH CONTENTION L

Applicant Private Fuel Storage, L.L.C. ("Applicant" or "PFS") moves, pursuant to 10 C.F.R. § 2.749, for summary disposition of Part B of State of Utah's ("State") Contention L (Geotechnical) ("Part B of Utah L").[1] Summary disposition is warranted because there exists no genuine issue as to any relevant material fact and PFS is entitled to a decision as a matter of law. This motion is supported by a Statement of Material Facts on Which No Genuine Dispute Exists ("Statement"), the Joint Declaration of Krishna P. Singh, Alan I. Soler and Everett L. Redmond II ("Holtec Dec."), the Declarations of C. Allin Cornell ("Cornell Dec.") and Bruce E. Ebbeson ("Ebbeson Dec."), and relevant discovery materials, including the depositions of State witnesses for Part B of Utah L.

I. STATEMENT OF THE ISSUE

Part B of Contention Utah L, as admitted by the Board in this proceeding, asserts that:

> B. Relative to the PFS seismic analysis supporting its application and the PFS April 9, 1999 request for an exemption from the requirements of 10 C.F.R. § 72.102(f) to allow PFS to employ a probabilistic rather than a

[1] Part B of Utah L was admitted by the Atomic Safety and Licensing Board ("Board" or "ASLB") as a contention on June 15, 2001, pursuant to a Commission order. See Memorandum and Order (Requesting Joint Scheduling Report and Delineating Contention Utah L) (June 15, 2001). Part A of Utah Contention L is subject to a pending motion for summary disposition filed by PFS. See Applicant's Motion for Summary Disposition of Utah Contention L, dated December 30, 2000.

Template = SECY-041

SECY-02

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deterministic seismic hazards analysis, PFS should be required either to use a probabilistic methodology with a 10,000-year return period or comply with the existing deterministic analysis requirement of section 72.102(f), or, alternatively, use a return period significantly greater than 2000 years, in that:

1. The requested exemption fails to conform to the SECY-98-126 (June 4, 1998) rulemaking plan scheme, i.e., only 1000-year and 10,000-year return periods are specified for design earthquakes for safety-important systems, structures, and components (SSCs) -- SSC Category 1 and SSC Category 2, respectively -- and any failure of an SSC that exceeds the radiological requirements of 10 C.F.R. § 72.104(a) must be designed for SSC Category 2, without any explanation regarding PFS SSC compliance with section 72.104(a).

2. PFS has failed to show that its facility design will provide adequate protection against exceeding the section 72.104(a) dose limits.

3. The staff's reliance on the reduced radiological hazard of stand-alone ISFSIs as compared to commercial power reactors as justification for granting the PFS exemption is based on incorrect factual and technical assumptions about the PFS facility's mean annual probability of exceeding a safe shutdown earthquake (SSE), and the relationship between the median and mean probabilities for exceeding an SSE for central and eastern United States commercial power reactors and the median and mean probabilities for exceeding an SSE for the PFS facility.

4. In supporting the grant of the exemption based on 2000-year return period, the staff relies upon the United States Department of Energy (DOE) standard, DOE-STD-1020-94, and specifically the category-3 facility SSC performance standard that has such a return period, notwithstanding the fact the staff categorically did not adopt the four-tiered DOE category scheme as part of the Part 72 rulemaking plan.

5. In supporting the grant of the exemption based on the 2000-year return period, the staff relies upon the 1998 exemption granted to DOE for the Idaho National Engineering and Environmental Laboratory (INEEL) ISFSI for the Three Mile Island, Unit 2 (TMI-2) facility fuel, which was discussed in SECY-98-071 (Apr. 8, 1998), even though that grant was based on circumstances not present with the PFS ISFSI, including (a) existing INEEL design standards for a higher risk facility at the ISFSI host site; and (b) the use of a peak design basis horizon-

tal acceleration of 0.36 g that was higher than the 2000-year return period value of 0.30 g.

6. Because (a) design levels for new Utah building construction and highway bridges are more stringent; and (b) the PFS return period is based on the twenty-year initial licensing period rather than the proposed thirty- to forty-year operating period, the 2000-year return period for the PFS facility does not ensure an adequate level of conservatism.

Part B of Utah L attacks PFS's reliance on a probabilistic seismic hazard analysis ("PSHA") as the basis for designing the PFSF, and the sufficiency of using a 2,000 year return period earthquake as the design basis earthquake ("DBE") for the facility. The NRC Staff granted a PFS request from an exemption from the requirements of 10 C.F.R. §72.102(f) to allow the PFSF to be designed using a PSHA methodology and a 2,000 year return period DBE. The State's contention challenges the Staff's granting of this exemption request. However, the PSHA methodology, as discussed below, is widely recognized as representing the state-of-the-art in the assessment of seismic hazards. Moreover, the NRC has selected this dominant methodology for use in the design of new nuclear power plants, whose safety requirements are greater than those for independent spent fuel storage installations ("ISFSIs") such as the PFSF.

There is also ample support for the use of a 2,000 year return period earthquake as the DBE for the PFSF. This support includes: (a) current NRC policy and practice, (b) proposed draft rulemaking plans by the NRC, (c) current industry practices, (d) the current state of scholarly research, and (e) U.S. Department of Energy ("DOE") facility design criteria and practices. In addition to these general sources of support, it can be demonstrated (as discussed below) that utilization of a 2,000-year return period earthquake as the DBE for the PFSF provides an adequate level of protection for the health and safety of the public, consistent with established NRC safety objectives and procedures.

Even though designed on the basis of the 2,000 year return period earthquake, the systems, structures and components ("SSCs") that are important to safety at the PFSF incorporate such conservatism in their design that they are capable of surviving, as demanded by the State in Part B of Utah L, an earthquake with "a return period significantly greater than 2000 years." Indeed, the storage casks to be used at the PFSF have been demonstrated by analysis to be able to withstand, without tipping over, the accelerations imparted by the 10,000-year return period earthquake called for in Part B of Utah L.

The State raises six bases in support of is contention that the 2,000 year return period earthquake is the wrong standard to use, and hence the NRC Staff should not have granted the exemption from 10 C.F.R. § 72.102(f) sought by PFS. However, the asserted bases are incorrect or inconsequential, and do not invalidate the granting of PFS's exemption request. Accordingly, there is no genuine dispute of material fact warranting a hearing, and the Board should order the summary disposition of Part B of Utah L.

II. LEGAL STANDARDS

The Board has previously stated the applicable standards for motions for summary disposition in this proceeding.[2] The legal requirements concerning expert opinions in support of a contention are particularly relevant here. These requirements include 1) demonstration that the affiant is an expert and 2) an explanation of facts and reasons in the affidavit supporting the affiant's expert opinion.[3] Mere unsupported conclusions or assertions are insufficient to support a contention.[4] As the Supreme Court has held, reliable expert opinion must be based on "more than subjective belief or unsupported speculation."[5]

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[2] See Private Fuel Storage, L.L.C. (Independent Fuel Storage Installation), LBP-99-23, 49 NRC 485, 491 (1999); Applicant's Motion For Summary Disposition of Utah Contention C – Failure to Demonstrate Compliance With NRC Dose Limits," dated April 21, 1999, at 4-16.

[3] Id. at 10-15; Mid-State Fertilizer Co. v. Exchange Nat'l Bank, 877 F.2d 1333, 1339 (7th Cir. 1989); Carolina Power & Light Co. (Shearon Harris Nuclear Plant, Units 1 and 2), LBP-84-7, 19 NRC 432, 447 (1984).

[4] Public Service Co. of New Hampshire (Seabrook Station, Units 1 and 2), LBP-83-32A, 17 NRC 1170, 1177 (1983); Private Fuel Storage, L.L.C. (Independent Spent Fuel Storage Installation), LBP-99-35, 50 NRC 180, 194 (1999).

[5] Daubert v. Merrell Dow Pharmaceuticals, Inc., 509 U.S. 579, 590 (1993).

These admonitions are particularly relevant with respect to Part B of Utah L, because the "experts" tendered by the State in support of its contention are either not experts at all on the topics they purportedly address, or have expertise that is circumscribed to only discrete aspects of the areas of their proposed testimony.[6] Accordingly, the allegations made by the State in Part B of Utah L, and those that the State can be anticipated to make in response to the instant motion, are insufficiently supported by expert opinions to create issues of fact that need to be litigated.

III. DISCUSSION

A. NO MATERIAL DISPUTE EXISTS REGARDING THE APPLICABILITY OF THE PROBABILISTIC SEISMIC HAZARDS ANALYSIS METHODOLOGY TO THE PFSF DESIGN

In Part B of Utah L, the State suggests that the seismic analyses conducted by PFS are flawed because they do not use the deterministic analysis methodology specified in 10 CFR § 72.102(f). This claim, however, does not give rise to a litigable contention. The State has failed to provide any basis for a ruling that the PSHA methodology used by PFS is inadequate for the seismic design of the facility. In fact, the PSHA methodology is being adopted by the NRC in its current policy and practices, and is widely used and well-established in the technical community. Cornell Dec. ¶¶7-10.

The NRC has adopted the PSHA methodology in 10 C.F.R. § 100.23 and Regulatory Guide 1.165, which call for the use of PSHA in the seismic design of new nuclear

[6] For example, State witness Dr. Resnikoff asserts that he will evaluate the potential for the concrete in the HI-STORM storage casks to crack under seismic loads, even though he has never evaluated before the potential cracking of concrete. Deposition of Marvin Resnikoff ("Resnikoff Dep.") at 43-44, 50. Similarly, he asserts that he will evaluate the potential for thermal degradation of the concrete under postulated cask tipover conditions, though he has never done such an evaluation. Id. at 44-47. When asked how he planned to undertake these and other related evaluations, he acknowledged that he had no idea. Id. at 70-75, 80-82, 90-93. Further, with respect to the major point at issue in Part B of Utah L, the establishment of appropriate design standards, none of the State's expert witnesses have previous experience in the setting of standards, the development of codes and standards, or the selection of appropriate margins of safety. Deposition of Walter J. Arabasz ("Arabasz Dep.") at 21-23, 31, 40, 99; Deposition of Farhang Ostadan ("Ostadan Dep.") at 81-82; Deposition of Steven Barlett ("Barlett Dep.") at 20-21. In contrast, PFS expert Dr. Cornell has extensive experience in developing codes and standards in a wide variety of settings. Cornell Dec. ¶¶1-3.

power plants. Id. ¶10. The NRC is also using PSHA procedures in other regulatory areas. Id. This move towards probabilistic analyses is consistent with the NRC's policy of risk-informed regulations and decision making. Id.

Moreover, use of the PSHA methodology has been widely adopted in a variety of other technical settings; it has, for example, been incorporated in the design of buildings, offshore structures, and other facilities. Id. ¶11. Even the State's expert agrees that PSHA is the appropriate methodology to use for the seismic analyses of the PFSF. Arabasz Dep. at 44-45. Thus, the State's claim that a deterministic methodology should be used is without support and raises no litigable contention.

B. USE OF A 2,000 YEAR RETURN PERIOD EARTHQUAKE AS THE BASIS OF THE PFSF DESIGN IS SUFFICIENTLY PROTECTIVE OF PUBLIC HEALTH AND SAFETY

Use of the PSHA methodology to perform the seismic design of facilities such as the PFSF requires: (1) adopting a risk-graded approach to safety, under which the probability of failure of a facility or structure is deemed acceptable or not depending on the gravity of the consequences of such a failure, and (2) taking into consideration that the risk of failure of a facility or structure depends on both the probability of occurrence of the seismic event (often expressed as the mean annual probability of exceedence or "MAPE" of a given earthquake level) and the level of conservatism incorporated in the design procedures and criteria. Cornell Dec. ¶13.

Adoption of a risk-graded approach to the seismic analysis and design have been endorsed by the NRC Staff, the U.S. Department of Energy ("DOE"), the Federal Emergency Management Agency and other national and international standard-setting organizations. Id. ¶15. Applying a risk-graded approach to the seismic design of ISFSIs such as the PFSF means that such facilities can be allowed to have a higher probability of failure than nuclear power plants, for which the consequences of failure are far more severe. Id. ¶¶16, 17.

The U.S. Department of Energy, in its Standard 1020, has implemented the risk-graded approach to seismic analysis and design by dividing its facilities into four "performance categories," with increasing performance goals (that is, decreasing probability of failure) depending on the potential consequences of facility failure. Id. ¶20. ISFSIs are classified as Performance Category ("PC")3 facilities; critical facilities such as nuclear reactors are classified as PC4 facilities. Id. DOE has set performance goals for PC3 and PC4 facilities of, respectively, 10^{-4} and 10^{-5}. (This means that the performance goal for a PC3 category is to have an annual probability of failure of one in ten thousand or 10^{-4}.) Id. ¶21. DOE has also set MAPEs for the design basis ground motions of 5×10^{-4} for PC3 facilities and 1×10^{-4} for PC4 facilities. Id. The 5×10^{-4} MAPE for PC3 facilities under DOE Standard 1020 is identical to the DBE for PFSF under the exemption granted by the Staff; that is, a 2,000 return period earthquake.

In order to achieve the desired performance goals given the postulated MAPEs, DOE 1020 provides "risk reduction ratios" (R_Rs) for each facility category. The risk reduction ratios measure the conservatisms incorporated into the design of a facility by which the desired performance goals are achieved. Id. ¶¶24, 44. For PC3 facilities, the DOE 1020 standard sets a R_R of 5; for PC4 facilities, the R_R is 10. Id. ¶22 and Table 1.

Although the NRC has not set explicit performance goals or risk reduction ratios for ISFSIs or nuclear power plants, its standards and guidelines also contain many conservatisms that result in risk reduction factors as large as or larger than those specified in DOE 1020. Id. ¶25. The "performance goal" for nuclear power plants is a probability of seismic "failure" (core damage) of approximately 10^{-5}, the same set by DOE for PC4 facilities. Id. ¶27. The typical safe shutdown earthquake for a nuclear power plant has a MAPE of 10^{-4}. Id. ¶¶27, 38. From those two values, it is possible to infer that nuclear power plants design standards, set forth in the Standard Review Plan ("SRP"), achieve a risk reduction ratio R_R of approximately 10. Id. This R_R figure is independently confirmed by NRC studies that show that components designed to NRC SRP standards have risk reduction ratios in the range of 5 to 20, or greater. Id. ¶25.[7]

The important-to-safety SSCs of an ISFSI such as the PFSF are subject to essentially the same SRP design requirements as nuclear power plants. See Holtec Dec. ¶¶11-13 (storage casks and canisters); Ebbeson Dec. ¶¶12 Canister Transfer Building ("CTB"), 13 (structural steel members), 14 (cranes). Accordingly, the design of a facility like the PFSF would provide a risk reduction factor of approximately 5 to 20. Since the MAPE for the PFSF is 5×10^{-4}, the annual probability of failure of PFSF's SSCs is in the range of 2.5×10^{-5} to 10^{-4} or lower (i.e., mean return periods of failure of 10,000 to 40,000 years or more). These figures are at least as conservative, or more so, than the performance goal set by DOE 1020 for comparable PC3 facilities, which is 10^{-4}. Cornell Dec. ¶26. Accordingly, the DBE selected for the PFSF, with a 2,000 year return period, provides a level of protection against failure which is comparable to or greater than that provided by DOE standards. Id.

Moreover, the level of protection against failure (2.5×10^{-5} to 10^{-4}) for an ISFSI is appropriate in comparison to the level of protection against failure for a nuclear power plant (10^{-5}), under the risk-graded approach to PSHA design. Id. ¶27. It is consistent with a risk-graded approach that an ISFSI such as the PFSF have a probability of failure 4 to 10 times greater than that of a nuclear power plant.

The risk reduction ratios achieved by the conservatisms in the designs of the IFSF SSCs are confirmed by analyses recently conduced by PFS contractors and vendors, which show that the facility's SSCs can survive earthquakes with return periods significantly greater than the 2,000 years of the PFSF DBE; specifically, the storage casks and canisters and the CTB have been shown to be able to survive, without loss of safety

[7] The State's expert witness Dr. Arabasz acknowledged that he was not familiar with the risk reduction factors achieved for facilities designed to the NRC SRPs. Arabasz Dep. at 83, 116-117.

function, the ground motions of a 10,000 year return period earthquake. Thus, they easily meet or surpass the 10^{-4} performance goal set in DOE 1020. Id. ¶29.

These results demonstrate, both in absolute terms and by comparison to nuclear power plant standards, that the PFSF seismic design basis consisting of a 2,000 year return period earthquake and compliance with SRP design procedures and criteria provide an appropriate and consistent level of protection to public health and safety. Id. ¶30. The PFSF seismic designs, therefore, "are sufficiently protective of public safety and property," as called for by the Commission in CLI-01-12.

C. THE BASES ASSORTED BY THE STATE IN SUPPORT OF ITS CONTENTION ARE INCORRECT AND RAISE NO MATERIAL ISSUES OF FACT

In addition to advocating the use of a deterministic analysis methodology, the State also alleges that, should a PSHA be used in the seismic analyses of the PFSF, the return period of the design basis earthquake should be 10,000 years or "a return period significantly greater than 2000 years." However, the State and its experts have provided no valid reason for imposing a requirement to analyze the PFSF against a seismic event with a longer return period than 2,000 years, and the specific bases cited in support of the State's contention are erroneous and unpersuasive.

1. The Failure of the Staff to Conform the Seismic Exemption to the Rulemaking Plan in SECY-98-126 Does Not Create a Litigable Issue

The State's first basis for contesting the use of a 2,000 year return period earthquake as the DBE for the PFSF is that a 10,000 year earthquake was specified in the rulemaking plan for 10 C.F.R. Part 72 developed by the NRC Staff in 1998.[8] This contention, however, raises no factual or legal issue fit for adjudication. A rulemaking plan is only a planning tool by the Staff that is subject to modification in the course of devel-

[8] SECY-98-126, from L. Joseph Callan ("EDO") to the Commissioners, "Rulemaking Plan: Geological and Seismological Characteristics for Siting and Design of Dry Cask Independent Spent Fuel Storage Installations, 10 C.F.R. Part 72," dated June 4, 1998.

oping a proposed rule, and which can be set aside in individual cases if circumstances warrant.[9] The Commission's order that admitted Part B of Utah L for litigation recognized that there was no obligation to comply with the rulemaking plan in seeking or granting an exemption. The NRC stated that all an applicant need do "in seeking an exemption from [its] existing regulations . . . [is] only to justify the seismic hazard analysis and design standards it proposes to use." Private Fuel Storage, LLC (Independent Spent Fuel Storage Installation), CLI-01-12, 53 NRC 459, 471 (2001).

Moreover, the State's assertion that failure to comply with the draft rulemaking plan should be of some import in this proceeding is mooted by the fact that the NRC Staff has recommended to the Commission a modification of the rulemaking plan in SECY-98-126 that would use a 2000-year mean return period design basis earthquake as the basis for ISFSI design; this is the same DBE approved by the Staff in the exemption it granted for the PFSF. SECY-01-0178 (September 26, 2001). Therefore, the grounds asserted by the State in this basis are no longer valid, if they ever were.

Finally, the showing that the Commission required of the PFSF is that "the 2000 year design standard is sufficiently protective of public safety and property." CLI-01-12, 53 NRC at 472. As discussed in Subsection B above, such a showing has been sufficiently made. Therefore, nothing remains to be litigated with respect to this basis of the State's contention.

2. The State Has Raised No Disputed Issues of Fact Regarding the Ability of the PFSF's Design to Provide Adequate Protection Against Exceeding Regulatory Dose Limits

The State asserts in Basis 2 of Part B of Utah L that PFS has failed to show that the facility design will provide adequate protection against exceeding the 10 C.F.R. §72.104(a) dose limits. As observed by the Licensing Board, this claim is "an adjunct" to Basis 1 in which the State claims that the use of a 2,000-year mean return period design

[9] See, e.g., Yankee Atomic Electric Co. (Yankee Nuclear Power Station), CLI-96-7, 43 NRC 235 (1996).

basis earthquake is improper because it fails to conform to the rulemaking plan in SECY-98-126. Private Fuel Storage, LLC (Independent Spent Fuel Storage Installation), LBP-01-03, 53 NRC 84, 96 (2001). Thus, this basis is insufficient for the same reasons that Basis 1 is inadequate. In fact, PFS has shown that the design of the PFSF, on the basis of the 2000 year return period earthquake and the SRP guidelines, provides adequate protection against exceeding the regulatory dose limits, because the important-to-safety SSCs at the PFSF have a high likelihood of surviving without loss of safety function a significantly more severe earthquake than the 2,000 year return period DBE. Ebbeson Dec. ¶27; Holtec Dec. ¶34. In fact, critical components and structures, such as the spent fuel storage casks and fuel canisters and the Canister Transfer Building, have been shown by analysis to be able to withstand the forces imparted on them by a 10,000 year return period earthquake, such as the one postulated by the State. Holtec Dec. ¶34; Ebbeson Dec. ¶19.

In reality, the State is once again trying to get its often-raised geotechnical claims into litigation through the back door, by alleging that the PFSF seismic design has not been shown to provide adequate protection against a 2,000 year return period seismic event. The State has attempted on several occasions to attack the adequacy of certain aspects of the PFSF seismic design: it sought, in an untimely manner, to expand the scope of Contention Utah L[10], and it has sought repeatedly to have admitted into this proceeding its late-filed Contention QQ.[11] This is the State's third untimely attempt to raise a seismic design contention. However, Part B of Utah L is about whether the exemption

[10] State of Utah's Response to Applicant's Motion for Summary Disposition of Utah Contention L (January 30, 2001).

[11] See State of Utah's Request for Admission of Late-Filed Contention Utah QQ (May 16, 2001); State of Utah's Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to further Revised Calculations from the Applicant (June 19, 2001); State of Utah's Second Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to More Revised Calculations for the Applicant (August 23, 2001); State of Utah's Objections and Responses to Applicant's Seventh Set of Formal Discovery Requests, response to Interrogatory No. 4(b); Ostadan Dep. 55-65; Barlett Dep. 66-70.

allowing PFS to utilize PSHA and a 2,000 year MRP DBE was appropriate. This contention does not concern whether the design of the PFSF to satisfy the design basis earthquake requirements is adequate. Therefore, the State's arguments against the design are irrelevant here.

3. <u>The State Has Failed to Raise a Valid Factual Dispute Regarding the Staff's Differentiation Between Radiological Hazards Associated with ISFSIs and Those Associated with Nuclear Power Plants</u>

In Basis 3 of Part B of Utah L, the State challenges the Staff's justification for granting the PFS exemption request by claiming that the Staff's reliance on the reduced radiological hazard of stand-alone ISFSIs in comparison to commercial power reactors is based on "incorrect factual and technical assumptions." The incorrect factual and technical assumptions referred to by the State appear to stem from the State's assertion that the Staff used an improper standard for comparing the probability of exceeding a design basis safe shutdown earthquake ("SSE") at a nuclear power plant versus the probability that the DBE for the PFSF will be exceeded. The State claims that for a nuclear power plant the "design ground motions would have to correspond to a median annual probability of exceedance of 10^{-5}." State of Utah's Request for Admission of Late-Filed Modification to Basis 2 of Contention Utah L at 8 (November 9, 2000). This assertion, however, is demonstrably incorrect.

Regulatory Guide 1.165, relied on by the State, provides that the annual probability level of the SSE may be based on a median estimate of 10^{-5}. However, the State fails to recognize that the typical SSE at existing nuclear power plants across the country has a mean annual probability of exceedance of 10^{-4}. Cornell Dec. ¶38. While the Staff referred to using median estimates in Regulatory Guide 1.165, the Staff currently would use mean estimates because they provide a better way to deal with uncertainty in probabilistic estimates. <u>Id</u>. ¶35. Indeed, the Staff has recently made statements and drafted regulations that reflect a preference for using a mean estimate. <u>Id</u>. ¶37.

Furthermore, the Staff's use of the mean probability estimate of 10^{-4} in granting PFS's exemption request is appropriate. This value represents the average mean annual probability of exceeding the SSE at existing nuclear power plants. Id. at ¶38. Therefore, there was no error in the Staff's analysis, and the State's challenge to it in Basis 3 is without merit.

4. The Staff's Reliance on the DOE Standard is Appropriate

Basis 4 of Part B of Utah L attacks the Staff's reliance on DOE Standard 1020 and its PC3 facility category to characterize ISFSIs such as the PFSF, because the SECY-98-126 Rulemaking Plan did not adopt the various DOE facility performance categories. Yet, the State has asserted no grounds -- other than the reference to the 1998 Rulemaking Plan -- for disregarding the DOE standard, which uses a 2,000 year DBE for facilities comparable to the PFSF. See Cornell Dec. ¶¶20-22.

As indicated above, the DOE 1020 standard employs different design procedures and evaluation criteria for each category of facility, reflecting the differences in facility importance and acceptable earthquake risk. The State does not show why using an analogous approach for the PFSF would be improper. To the contrary, the State's own expert witness acknowledges that reliance on the DOE 1020 standard is appropriate, and that such a standard, as it pertains to PC3 structures, should be applied at the PFSF. Arabasz Dep. at 80-81.

The State, in its responses to PFS interrogatories, claims that PFS has not demonstrated that its use of a 2000-year DBE would meet the DOE-1020 performance goal for Performance Category 3 (i.e., a mean annual probability of failure of 10^{-4}). As discussed in Subsection B above, however, the DOE-1020 performance goal for PC3 facilities would be met and even bettered at the PFSF, because compliance with the SRP provides a level of conservatism equal to or greater than that achieved using DOE 1020 risk reduction ratios.

For these reasons, the State's challenge to the Staff's reliance on DOE 1020 is unfounded and Basis 4 does not set forth issues requiring adjudication.

5. The Staff appropriately relied on the TMI-2 ISFSI facility exemption

Basis 5 of Part B of Utah L asserts that the Staff improperly relied on the precedent set by the exemption it granted in1998 to DOE's Idaho National Engineering and Environmental Laboratory ("INEEL") for the ISFSI for Three Mile Island Unit 2 ("TMI-2"). The ground for the State's challenge is the allegation that factual differences between the TMI-2 ISFSI facility and the PFSF make it improper for the Staff to rely on the TMI-2 exemption as precedent. The two factual differences asserted in the contention are: (a) the existing INEEL design standards are for a higher risk facility at the ISFSI host site; and (b) INEEL used a peak design basis horizontal acceleration of 0.36 g, which was higher than the 2000-year return period value of 0.30 g used at PFSF. The State has never explained the significance of these differences and has retreated to the unpersuasive assertion that "an exemption does not prove the rule..." State of Utah's Objections and Response to Applicant's Seventh Set of Formal Discovery Requests to Intervenor of State of Utah, Answer to Interrogatory No. 7. However, there is no doubt that at the time the INEEL exemption was approved, the NRC Staff and the Commission expected (and intended) that it would serve as a precedent towards the granting of similar exemptions in the future. The NRC Staff's final statement to the Commission in SECY-98-071 states: "If the staff grants the exemption to 10 CFR 72.102 (f)(1), this may impact the licensing process for other ISFSIs in the western United States. Until the ISFSI seismic requirement in Part 72 is amended by rulemaking, the staff may receive similar exemption requests for other ISFSIs to be sited west of the Rocky Mountain front." This language shows that the Staff intended that the exemption it was granting for INEEL would be relied upon as precedent for other exemption applications, as was done in the case of the PFSF.

Moreover, as the State recognizes, the TMI-2 exemption shows that the use of a 2,000-year return period earthquake is not unprecedented. The State has failed to give any explanation why it is necessary to litigate over the undisputed precedential value of the INEEL exemption. As such, there is no material dispute of relevant fact.

6. <u>Use of the PSHA with a 2,000-Year Return Period Ensures Adequate Conservatism</u>

Finally, the State attacks the 2,000-year return period earthquake exemption by asserting that design levels for new Utah buildings and highway bridges are more stringent than the standards allowed by the exemption. The State bases this assertion on standards such as the International Building Code ("IBC") 2000, which will require a return period of approximately 2500 years for the design basis earthquake when it goes into effect. This return period is longer than the one proposed for the PFS. The State argues that the difference in the definition of DBE implies a lower probability of failure for SSCs designed under IBC-2000 than an ISFSI designed against a 2,000 year return period earthquake. However, the State overlooks that the design procedures and criteria used by IBC-2000 are much less conservative than those under the NRC's SRP that govern the design of the PFSF. Cornell Dec. ¶46. As discussed above, the level of safety achieved depends on both the earthquake threat definition and the design procedures and criteria utilized to protect against that threat; thus, looking only at the earthquake return period is incorrect.

Indeed, the model building codes cited by the State are much less conservative than the SRP. For example, the building codes permit more liberal allowances for the benefits of post-elastic behavior than the SRP. Cornell Dec. ¶47. The net result is that, because the procedures used for the design of the PFSF are far more conservative, typical PFSF SSCs have a mean annual probability of failure several times lower than buildings designed to model building code standards. <u>Id</u>. ¶49. In addition, as discussed above, a number of key SSCs in the PFSF have great robustness and/or fractional operating periods, which reduces their failure probabilities even further. Id. ¶47. Therefore, the allegation that model building codes provide greater protection against the effects of earthquakes is simply not true.

The State further claims in Basis 6 that the 2,000-year mean return period for the PFS facility does not ensure an adequate level of conservatism because the return period was chosen based on the twenty-year initial licensing period rather than a potential thirty to forty-year operating period. This claim ignores the fact that the length of the license at issue is immaterial, since in virtually all areas of public safety, hazards are measured as annual probabilities (or frequencies) of occurrence, regardless of the length of the license. Cornell Dec. ¶49. This is also the case with respect to risk acceptance guidelines promulgated by the NRC. Id. Therefore, this portion of Basis 6 also fails to raise a material issue of fact needing adjudication.

D. THE DESIGN OF THE PFSF RESULTS IN SAFETY STRUCTURES CAPABLE OF WITHSTANDING EVEN THE SEVERE SEISMIC EVENTS POSTULATED BY THE STATE

The main thrust of the allegations in Part B of Utah L is that designing the PFSF against a 2,000 year return period DBE is insufficiently conservative, and that the important-to-safety SSCs at the facility should be able to meet the accelerations produced by a 10,000 year seismic event, or at least one having a return period "significantly greater than 2000 years." However, even taking the contention's allegations at face value, they do not set forth an issue of material fact requiring litigation at a hearing because the SSCs at the PFSF are capable of accommodating the higher earthquake loadings that the State contends should be applied.

The HI-STORM storage casks and canisters are designed to have excellent resistance against seismic events, being designed to the same industry codes used for the manufacture of reactor pressure vessels, primary systems piping, and supports. Holtec

Dec. ¶¶9, 11-13. As a result, their design is extremely conservative and has very high margins against failure. Id. ¶¶11-13.

Indeed, the cask manufacturer has demonstrated by analysis that the storage casks will not overturn when subjected to the forces imparted by a 10,000 year return period earthquake. Id. ¶16. Even if one assumes hypothetically that the casks do tip over, the accelerations imparted upon the casks are within design allowables and do not significantly affect the integrity of the casks or the canisters containing spent fuel. Id. ¶18. Moreover, the margins in the design of storage casks and fuel canisters are so large that it is highly unlikely that the canisters would be breached even under significantly more severe conditions than those resulting from a 10,000 year return period earthquake. Id. ¶20.

From the standpoint of radiation doses at the PFSF site boundary, under normal conditions with all storage casks upright, extremely conservative dose calculations performed by the storage cask vendor show that the annual radiation doses at the site boundary are 5.85 mrem/year, only a fraction of the dose limit of 25 mrem/yr set by 10 C.F.R. §72.104(a) for normal operations and an even smaller fraction of the allowable dose of 5 rem/yr set in 10 C.F.R. §72.106(b) for accident conditions. See Section 7.3.3.5 and Table 7.3.7 of the PFSF SAR; Holtec Dec. ¶¶23, 31.

The occurrence of a 10,000 year return period earthquake does not materially alter these dose calculation results. In fact, if one assumes a storage cask tips over as a result of such an earthquake and remains in a horizontal position, the doses at the site boundary are less than if the cask had remained upright. Holtec Dec. ¶28. Indeed, the radiation doses at the boundary remain essentially unchanged regardless of whether one assumes that a single cask, a number of them, or all of the casks tip over, and regardless of the length of time the casks remain tipped over. Id. ¶¶32, 33.

Another accident scenario that could be postulated involving fuel storage canisters is the potential drop of one such canister due to the occurrence of a beyond-design basis earthquake (such as the 10,000 year return period seismic event) while the canister is in the process of being transferred from a shipping cask to a storage cask in the CTB.

The occurrence of such an event is extremely improbable, due to the very high margins incorporated into the design of components whose failure could cause the drop, such as the CTB cranes. These cranes are single-failure-proof and are capable, due to the conservative nature of their designs, to withstand the loadings of an earthquake far more severe than the 2,000 year return period DBE. See Holtec Dec. ¶21; Ebbeson Dec. ¶25. Nonetheless, assuming a postulated failure of one of these components causes a canister to drop a distance of 25 feet into an unyielding surface, analyses by the canister manufacturer demonstrate that the canister will readily survive the drop, because the strain to which it will be subjected is only 41% of the failure limit for the material. Holtec Dec. ¶22.

Finally, analyses of other important SSCs in the CTB, such as the building itself, its roof, and the seismic struts that provide restraints for the casks during canister transfer operations, demonstrate that these components have a high likelihood of surviving without loss of safety function a significantly more severe earthquake than the design basis 2,000 year return period event. See Ebbeson Dec. ¶¶16-27. These analyses do not even include margins inherent in the designs of the structures and components that are known to exist but are not easily quantifiable, such as the reserve capacity that exists in steel structures above the onset of yielding. Ebbeson Dec. ¶21.

For these reasons, the use of a 2,000 year return period DBE, coupled with the conservatisms inherent in the design of SSCs to NRC standards, assure that the important-to-safety SSCs at the PFSF will be capable of surviving the extreme loadings postulated by the State in Part B of Utah L. Therefore, there is no material issue of fact remaining with respect to the contention that requires adjudication.

IV. CONCLUSION

For the above reasons, the Board should grant summary disposition of Part B of Contention Utah L.

Respectfully submitted,

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Dated: November 9, 2001

November 9, 2001

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	)	
	)	Docket No. 72-22
PRIVATE FUEL STORAGE L.L.C.	)	
	)	ASLBP No. 97-732-02-ISFSI
(Private Fuel Storage Facility)	)	

STATEMENT OF MATERIAL FACTS ON WHICH NO GENUINE DISPUTE EXISTS

Applicant submits, in support of its motion for summary disposition of Part B of Utah Contention L (Geotechnical) ("Part B of Utah L"), this statement of material facts as to which the Applicant contends there is no genuine issue to be heard.

I. BACKGROUND

1. Part B of Utah L was admitted by the Atomic Safety and Licensing Board ("Board" or "ASLB") as a contention on June 15, 2001, pursuant to a Commission order. See Memorandum and Order (Requesting Joint Scheduling Report and Delineating Contention Utah L) (June 15, 2001). As admitted, Part B of Utah L asserts:

    B. Relative to the PFS seismic analysis supporting its application and the PFS April 9, 1999 request for an exemption from the requirements of 10 C.F.R. § 72.102(f) to allow PFS to employ a probabilistic rather than a deterministic seismic hazards analysis, PFS should be required either to use a probabilistic methodology with a 10,000-year return period or comply with the existing deterministic analysis requirement of section 72.102(f), or, alternatively, use a return period significantly greater than 2,000 years, in that:

    1. The requested exemption fails to conform to the SECY-98-126 (June 4, 1998) rulemaking plan scheme, i.e., only 1000-year

and 10,000-year return periods are specified for design earthquakes for safety-important systems, structures, and components (SSCs) – SSC Category 1 and SSC Category 2, respectively – and any failure of an SSC that exceeds the radiological requirements of 10 C.F.R. § 72.104(a) must be designed for SSC Category 2, without any explanation regarding PFS SSC compliance with section 72.104(a).

2. PFS has failed to show that its facility design will provide adequate protection against exceeding the section 72.104(a) dose limits.

3. The staff's reliance on the reduced radiological hazard of stand-alone ISFSIs as compared to commercial power reactors as justification for granting the PFS exemption is based on incorrect factual and technical assumptions about the PFS facility's mean annual probability of exceeding a safe shutdown earthquake (SSE), and the relationship between the median and mean probabilities for exceeding an SSE for central and eastern United States commercial power reactors and the median and mean probabilities for exceeding an SSE for the PFS facility.

4. In supporting the grant of the exemption based on 2,000-year return period, the staff relies upon the United States Department of Energy (DOE) standard, DOE-STD-1020-94, and specifically the category-3 facility SSC performance standard that has such a return period, notwithstanding the fact the staff categorically did not adopt the four-tiered DOE category scheme as part of the Part 72 rulemaking plan.

5. In supporting the grant of the exemption based on the 2,000-year return period, the staff relies upon the 1998 exemption granted to DOE for the Idaho National Engineering and Environmental Laboratory (INEEL) ISFSI for the Three Mile Island, Unit 2 (TMI-2) facility fuel, which was discussed in SECY-98-071 (Apr. 8, 1998), even though that grant was based on circumstances not present with the PFS ISFSI, including (a) existing INEEL design standards for a higher risk facility at the ISFSI host site; and (b) the use of a peak design basis horizontal acceleration of 0.36 g that was higher than the 2,000-year return period value of 0.30 g.

6. Because (a) design levels for new Utah building construction and highway bridges are more stringent; and (b) the PFS return period is based on the twenty-year initial licensing period rather than the proposed thirty to forty-year operating period,

the 2,000-year return period for the PFS facility does not ensure an adequate level of conservatism.

2. The current regulations for the seismic design of Independent Spent Fuel Storage Installations ("ISFSIs") at sites west of the Rocky Mountains (10 C.F.R. § 72.102) provide for a seismic assessment of the design basis seismic ground motions based on the deterministic procedures formerly used for nuclear power plant design (Appendix A, 10 CFR Part 100).

3. On April 2, 1999 PFS filed an exemption request with the NRC to use a probabilistic seismic hazard analyses ("PSHA") for determining the seismic design basis of the Private Fuel Storage Facility ("PFSF") instead of the deterministic approach currently provided for by 10 C.F.R. § 72.102.

4. In its Safety Evaluation Report for the PFSF, the NRC Staff found the use of the PSHA methodology with a 2,000-year return period for the seismic design basis of the PFSF to be acceptable.

5. The PSHA methodology is better able to incorporate risk considerations and uncertainties in establishing seismic design bases than is the deterministic methodology and therefore is in accordance with the Commission's movement toward risk-informed regulation. Declaration of C. Allin Cornell ("Cornell Dec.") ¶ 8-10; Deposition of Walter J. Arabasz ("Arabasz Dep.") at 44-45.

6. The use of the PSHA methodology for establishing seismic design basis ground motions is the dominant and favored methodological choice in other fields. Current regulations and guidelines based on probabilistic principles include those governing the design of buildings, offshore structures, and DOE facilities. Cornell Dec. ¶ 11; see also Arabasz Dep. at 44-45.

7. In 1997, the Commission amended Parts 50 and 100 of its regulations to provide for the use of the PSHA methodology for the seismic design of new nuclear power plants. 100 C.F.R. § 100.23.

8. Thus, use of a PSHA to characterize the seismic hazard at the PFSF site and to set the seismic design basis of the PFSF is appropriate and fully consistent with both NRC and broader engineering policy and practice. Cornell Dec. ¶¶ 8-12; Arabasz Dep. at 44-45.

9. Most modern seismic design criteria are based on a graded approach to seismic safety, permitting facilities or structures with less severe failure consequences to have larger mean annual probabilities of failure. Cornell Dec. ¶¶ 14-15; Arabasz Dep. at 59-60.

10. Dry cask ISFSIs, such as the PFSF, are recognized by the NRC as being inherently less hazardous than operating nuclear power plants and less vulnerable to earthquake-initiated accidents than an operating nuclear power plant. See, e.g.,

60 Fed. Reg. 20,883 (1998); 45 Fed. Reg. 74,697 (1980). See also Cornell Dec. ¶¶ 16-17; Arabasz Dep. at 58-59.

11. Because of the lower radiological hazards posed by dry cask ISFSIs, it is appropriate to allow a higher probability of failure for such facilities due to an earthquake than for operating nuclear power plants. Cornell Dec. ¶ 16; Arabasz Dep. at 59.

12. Two factors are relevant to determining the likelihood of seismic failure of a facility or structure due to an earthquake event. These are (1) the seismic design basis earthquake ("DBE") for the facility or structure and (2) the conservatisms embodied in the codes and standards applicable to its seismic design. Cornell Dec. ¶¶ 18-19; see also Arabasz Dep. at 41-42, 81-84, 115-117.

13. The average mean Safe Shutdown Earthquake ("SSE") for the seismic design basis for nuclear power plants located in the Eastern and Central part of the United States has been calculated to be $1x10^{-4}$, or a return period earthquake of 10,000 years. Cornell Dec. ¶ 38, Arabasz Dep. at 61-62, 70.

14. The average mean SSE for the seismic design basis for nuclear power plants located in the Western United States has been calculated to be $2x10^{-4}$, or a return period earthquake of 5,000 years. Cornell Dec. Exhibit 3; Arabasz Dep. at 69-71.

15. Accordingly, using an average mean SSE of $1x10^{-4}$ approximately represents the seismic design basis for nuclear power plants in the Eastern, Central and Western regions of the United States. Arabasz Dep. at 70-71; Cornell Dec. ¶ 38.

16. The mean estimate is generally preferred to the median estimate when making decisions based on uncertain annual probabilities and frequencies because the mean estimate better captures and reflects uncertainties than the median estimate. Cornell Dec. ¶ 37; Arabasz Dep. at 62-63.

17. The NRC has chosen as a general matter to use mean estimates to express uncertain annual probabilities and frequencies. Cornell Dec. ¶ 36.

18. The use of a median estimate in Regulatory Guide 1.165 was based on a discrepancy in the mean estimates for nuclear power plant SSEs in the Central and Eastern United States between two major studies at the time of its issuance. This discrepancy has since been resolved. Cornell Dec. ¶ 36.

19. The State's argument for the use of a median estimate in lieu of the mean estimate for the design of nuclear power plants, and similarly for ISFSIs, would lead to inconsistent mean SSE probabilities across the country for such facilities because the ratio of the mean to the median is not constant across all regions of the country. Cornell Dec. ¶ 38; see also Arabasz Dep. at 60-63.

20. The Department of Energy Standard 1020 ("DOE-1020") provides a good example of a graded approach to seismic safety using both factors referred to in

Material Fact Statement No. 12 for determining seismic safety. Cornell Dec. ¶ 20; Arabasz Dep. at 59-60, 70.

21. DOE-1020 has four performance categories for seismically designed facilities and structures with increasing consequences of failure and decreasing probabilities or seismic failure as their performance goals. Cornell Dec. ¶¶ 20-22; Arabasz Dep. at 74-78.

22. Under DOE-1020 the seismic performance goals, i.e., the probability of seismic failure are less than the probability of exceedance of the ground motions for the DBE. Cornell Dec. ¶¶ 21-22; Arabasz Dep. at 80-81.

23. Under DOE-1020, the PFSF would be classified as a PC3 facility. Cornell Dec. ¶ 26; Arabasz Dep. at 80-81.

24. Under DOE-1020, PC3 structures systems and components have a seismic performance goal or failure probability of $1x10^{-4}$. This is a rationale and appropriate performance goal for the PFSF. Cornell Dec. ¶¶ 21-22, 26-27; Arabasz Dep. at 80-81.

25. The difference between the mean probability of exceedance of the DBE and the performance goal is obtained by conservatisms incorporated into the applicable design procedures and criteria. Cornell Dec. ¶¶ 19, 22.

26. The conservatisms in DOE-1020 embodied in the risk reduction factor R_R for PC3 structures systems and components is 5. Together with the mean probability of exceedance of the DBE in DOE 1020-94 for category PC3 of $5x10^{-4}$, this R_R meets the performance objectives for PC3 structures, systems and components of $1x10^{-4}$. Cornell Dec. ¶¶ 21-22.

27. The risk reduction factor for DOE-1020 category PC4 facilities, such as nuclear reactor facilities, is 10. Cornell Dec. ¶¶ 21-22.

28. In DOE-1020, the required risk reduction levels, R_R, are achieved through use of the DOE design and evaluation criteria specified in Chapter 2 of the Standard and related appendices. Cornell Dec. ¶ 24.

29. The design guidelines provided by NRC Standard Review Plans ("SRPs") contain numerous conservatisms that result in risk reduction factors as large as, or larger than, those provided for PC4 facilities under DOE-1020. Cornell Dec. ¶ 25 and Attachment A.

30. The State's expert witness Dr. Arabasz acknowledge that he was not familiar with the risk reduction factors achieved for facilities designed to NRC SRPs. Arabasz Dep. at 83, 116-117.

31. Important to safety structures systems and components at the PFSF are designed in accordance with the NRC SRPs and other nuclear industry standards that

provide comparable conservatisms. Cornell Dec. ¶¶ 13, 26; Joint Declaration of Krishna P. Singh, Alan I. Soler, and Everett L. Redmond, II. ("Holtec Dec.") (November 9, 2001) ¶¶ 11-13, 21; Declaration of Bruce E. Ebbeson ("Ebbeson Dec.") (November 9, 2001) ¶¶ 12-16.

32. Designing the PFSF important to safety structures, systems and components ("SSCs") using the NRC SRPs means that the PFSF important to safety structures, systems and components have seismic failure probabilities 5 to 20 or more times lower than the 2,000 mean return period DBE, i.e., seismic failure mean return periods of 10,000 to 40,000 years or more. Cornell Dec. ¶¶ 25-26 and Attachment A.

33. Further, certain PFSF important-to-safety SSCs - the seismic support struts and the Canister Transfer Building ("CTB") cranes - are in an important safety operational mode only approximately 20% of the time or less. Ebbeson Dec. ¶¶ 8-11, 27. For such intermittent use components, the annual probability of failure is at least five times smaller. Cornell Dec. ¶ 29.

34. Based on Material of Fact Statements 29-33 above, the PFSF would meet the performance objective of DOE-1020 $1x10^{-4}$ for PC3 facilities. This performance objective for the PFSF is consistent with the NRC's performance objective which pose higher radiological consequences than ISFSI. Cornell Dec. ¶¶ 26-27.

35. Both in absolute terms and by comparison to nuclear power plant standards, the proposed PFSF seismic design basis of a 2,000 MRP DBE and SRP design procedures and criteria provide an appropriate and consistent level of protection to the Public health and safety. Cornell Dec. ¶ 30.

36. The design procedures and acceptance criteria for the International Building Code 2,000 are significantly less conservative than those in the NRC's SRPs. Cornell Dec. ¶ 47.

37. PFSF important-to-safety structures, systems and components will have a mean annual probability of failure approximately 2.5 or more lower than "essential structures" designed to IBC-2,000 standards. Cornell Dec. ¶¶ 46-47.

38. Assuming that a 2500-mean return period earthquake is used in the design of certain essential bridges in Utah, the design of the PFSF using a 2,000-year return period earthquake and NRC seismic SRP design criteria will provide higher level of safety, than that for the bridges. Cornell Dec. ¶ 48.

39. The proper focus in making facility safety decisions is on annual probabilities or frequencies of occurrence. Cornell Dec. ¶ 49; see also Arabasz Dep. at 51-52.

40. The HI-STORM Storage casks and multi-purpose canisters have significant built-in conservatisms and design margins that assure their ability to perform beyond design requirements and to resist very large earthquake induced forces. Holtec Dec. ¶¶ 11-13.

41. Loaded HI-STORM storage casks will not tipover under a beyond design basis, 10,000 year return period seismic event postulated to occur at the PFSF site. Holtec Dec. ¶¶ 15-17.

42. Even assuming hypothetical cask tipover under 10,000 earthquake return period conditions, there would be no breach of the multipurpose canister confinement boundary and no risk of radioactivity release. Holtec Dec. ¶¶ 18, 20.

43. The concrete of the HI-STORM storage casks will not crack under accelerations produced by the 2,000 year design basis earthquake for the PFSF. Holtec Dec. ¶ 24.

44. The concrete of the HI-STORM storage casks will not crack under accelerations produced by a 10,000 year beyond design basis event at the PFSF. Holtec Dec. ¶ 24.

45. The localized damage to the radial concrete shield and outer steel shell of the HI-STORM storage cask in an hypothetical tipover event would result in no noticeable increase in radiation dose at the ISFSI site boundary. Holtec Dec. ¶¶ 25-28.

46. The dose rate at the PFSF site boundary will remain essentially unchanged regardless of whether one assumes that a single cask, any number of casks, or all the casks tipover. Holtec. Dec. ¶¶ 25-32.

47. There are significant conservatisms in the calculated dose rate of 5.85 mrem per year at the PFSF boundary. Holtec Dec. ¶ 31.

48. Taking one of the many conservatisms into account reduces the calculated dose rate of 5.85 mrem per year by more than 50%. Holtec Dec. ¶ 31.

49. Potential thermal degradation of the HI-STORM storage cask in a tipover condition would have no significant effect on the radiation shielding function of the storage cask. Holtec Dec. ¶ 33.

50. The important-to-safety structures, systems and components of the CTB posses for greater seismic loading capacities than the seismic loads imposed by the 2,000 year mean return period earthquake. Ebbeson Dec. ¶¶ 16-26.

51. Directly quantifiable margins in the capacity of the CTB roof to withstand accelerations well in excess of those produced by the 2,000 year return period earthquake include the following:

    a. The maximum calculated bending moment of a typical girder is only 71% of the code allowable stresses.

b. The ultimate bending moment capacity of the roof is more than 50% greater than the bending moment capacity based on code allowable stresses.

c. Composite behavior of the studs on the beams and girders would allow the design to resist vertical accelerations of at least 3g.

d. The load carrying capacity of the girders is increased by about 30% because of the connection of the girders to the roof slab and the integrated construction of the roof slab with the walls.

Ebbeson Dec. ¶ 23.

52. Directly quantifiable margins in the CTB cranes include:

a. The ultimate strength of the mechanical component materials is five times that required to support the lifted load.

b. If failure of a mechanical component could cause a load to drop, the ultimate strength of the material is ten times that required to support the lifted load.

Ebbeson Dec. ¶ 25; see also Holtec Dec. ¶ 21.

53. Even assuming a hypothetically postulated drop of a MPC canister being lifted by the CTB cranes from a height of 25 feet, the MPC confinement boundary integrity would be maintained with no radioactive release. Holtec Dec. ¶ 22.

54. The ultimate capacity of the seismic support struts are 45% greater than the seismic loads imposed by the 2,000 year return period earthquake. Ebbeson Dec. ¶ 26.

55. In addition to the directly quantifiable or conservatisms in Material Fact Statements 50-53 above, other significant non-quantifiable conservatisms are also present with respect to important-to-safety CTB structures systems and components. Ebbeson Dec. ¶¶ 12-15, 21.

56. The combination of quantifiable and non-quantifiable margins establish that CTB important to safety SSCs can withstand an earthquake with a return period significantly greater than the 2,000 year DBE. Ebbeson Dec. ¶ 27.

57. The State’s witnesses have confirmed that the issues identified in State's Response to Applicant's Seventh Set of Formal Discovery Requests, Interrogatory No. 4(b) have been raised previously by the State in Proposed Contention Utah QQ and

were not intended to introduce any new issues. Deposition of Farhang Ostadan ("Ostadan Dep.") at 55-65; Deposition of Steven F. Bartlett ("Bartlett Dep.") at 66-70.

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	)	
	)	
PRIVATE FUEL STORAGE L.L.C.	)	Docket No. 72-22
	)	
(Private Fuel Storage Facility)	)	ASLBP No. 97-732-02-ISFSI

CERTIFICATE OF SERVICE

I hereby certify that copies of the Applicant's Motion for Summary Disposition of Part B, Utah Contention L, Statement of Material Facts on Which No Genuine Issue Exists, and the Supporting Declarations, Exhibits and Attachments were served on the persons listed below (unless otherwise noted) by e-mail with conforming copies by U.S. mail, first class, postage prepaid, this 9th day of November, 2001.

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Paul Gaukler
Paul A. Gaukler

November 9, 2001

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	)	
	)	
PRIVATE FUEL STORAGE L.L.C.	)	Docket No. 72-22
	)	
(Private Fuel Storage Facility)	)	ASLBP No. 97-732-02-ISFSI

ATTACHMENTS FOR APPLICANT'S MOTION FOR SUMMARY DISPOSITION OF PART B OF UTAH CONTENTION L

Tab No.	Subject
A	Declaration of C. Allin Cornell
B	Joint Declaration of Krishna P. Singh, Alan I. Soler, and Everett Redmond II (HOLTEC)
C	Declaration of Bruce E. Ebbeson
D	Deposition of Walter J. Arabasz
E	Deposition of Marvin Resnikoff
F	Deposition of Farhang Ostadan
G	Deposition of Steven F. Bartlett

A

November 9, 2001

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	)	
	)	
PRIVATE FUEL STORAGE L.L.C.	)	Docket No. 72-22
	)	
(Private Fuel Storage Facility)	)	ASLBP No. 97-732-02-ISFSI

DECLARATION OF C. ALLIN CORNELL

C. Allin Cornell states as follows under penalty of perjury:

I. WITNESS CREDENTIALS AND SCOPE OF TESTIMONY

1. I am currently a professor (research) at Stanford University in Stanford, California and an independent engineering consultant. In the former capacity I perform research and supervise a Senior Research Associate and several Ph.D.-level graduate students in the areas of probabilistic analysis of structural engineering and earthquake engineering. As a consultant, I assist engineering and earth sciences firms, industrial concerns, and government agencies in developing and applying methodologies and standards for probabilistic seismic hazard analysis, engineering safety assessments, natural hazards analyses, and earthquake engineering. Through my teaching, research and consulting activities (described below) I have developed an expertise in earthquake engineering, probabilistic engineering analysis of seismic loads on structures and structural responses to such loads, and the development of structural design guidelines and codes. I am providing this declaration in support of Applicant's Motion for Summary Disposition of Part B of Contention Utah L in the above captioned proceeding concerning the Private Fuel Storage Facility ("PFSF").

2. My professional and educational experience is summarized in the *curriculum vitae* attached as Exhibit 1 to this declaration. My graduate education was in civil structural engineering. After nearly two decades as a faculty member at M.I.T., I entered about twenty years ago into an arrangement with Stanford University whereby I could continue conducting research and supervising advanced graduate students while devoting half-time to a professional practice as an independent consultant. A primary objective of this arrangement was to use my consulting activities to encourage and guide the rapidly emerging practice of employing probabilistic methods in engineering applications, while also being able to return to the university to study at an academic level some of the challenging technical problems identified in that practice. A focus of my efforts has been to address, through the common language of probability, the problems that arise at the interface between the scientists who characterize the natural hazards that threaten facilities and the structural and other engineers responsible for designing those facilities in a safe and cost-effective way. The majority of this work has been with earth scientists and structural engineers engaged in earthquake engineering.

3. I have been studying structural engineering since about 1956 as an undergraduate in architecture, methods of probability and statistics since graduate school, and the earth sciences through almost four decades of research and practice. My Ph.D. dissertation, which was entitled "Stochastic Process Models in Structural Engineering," included studies of earthquake engineering. I have subsequently published more than 150 papers in both engineering and scientific journals and conference proceedings. In 1970 I co-authored the first textbook designed to educate civil engineers in probability, statistics and decision theory under uncertainty. Major recognition for my professional contributions includes election to the National Academy of Engineering in 1981, several medals of the American Society of Civil Engineering, a number of invited annual lectures (for example, that of the Earthquake Engineering Research Institute in 1999) and, most recently announced, the 2002 Medal of the Seismological Society of America. Various other accomplishments and studies relevant to this matter include the following:

- In 1968 I published a seminal paper in the Bulletin of the Seismological Society on characterizing earthquake hazards using probabilistic seismic hazard analysis ("PSHA"). Improved and elaborated by more than thirty years of subsequent application and research (by myself and by many others), PSHA has become the standard method for earth scientists to characterize and report the earthquake threat at a site. For example, the USGS has used the method for two decades to study the entire US and to produce maps of seismic hazard that appear in all model building codes.
- I have participated directly, commonly as a senior advisor, in many prominent PSHA studies. These include the PSHA for the Diablo Canyon Nuclear Power Plant ("NPP"), the major EPRI Seismic Owners Group PSHA of the Central and Eastern US ("CEUS") NPP sites, the Caltrans-sponsored PSHA studies of all major California bridges, and PSHAs for the INEEL and LLNL DOE national lab sites and the Yucca Mountain site. I was also a member of the Senior Seismic Hazard Analysis Committee (sponsored jointly by NRC, EPRI and DOE) to establish "standards" for conducting PSHAs at nuclear facility sites.
- I was one of the originators of seismic probabilistic risk analysis ("SPRA")[1] for nuclear power plants, beginning with informal advice to MIT colleague Norman Rasmussen who directed the first PRA, WASH 1400. I was co-author with Nathan Newmark of the first published SPRA paper (presented by invitation at the annual meeting of the American Nuclear Society); this was then followed by a second paper (co-authored by several structural and nuclear engineers) based on the first practical application to a specific plant (Oyster Creek). I have been involved in a number of SPRA studies for nuclear facilities, including the Diablo Canyon NPP, and was a member of the NRC-sponsored Senior Seismic Margins Research Project committee responsible for directing a major project conducted by the LLNL studying the fragility curves of NPP SSCs.

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[1] SPRA couples the results of a PSHA with seismic "fragility curves" (that is, curves that depict the vulnerability of plant structures, systems, and components ("SSCs") to various levels of earthquake excitation) and a PRA model of the plant SSC interactions to produce results such as the mean annual seismically-induced core damage frequency (CDF). (The CDF is used as a subsidiary safety goal by the NRC.)

- I have had extensive involvement in the research and development of industry codes and standards. This involvement has included activities as:
    - Developer of methods to facilitate the introduction of probabilistic safety assessment directly into professional engineering codes of practice, including development of the methodology adopted by the American Institute of Steel Construction ("AISC") in the first probability-based structural code introduced in the US.
    - Co-author of report for specifying loads for building design that became the basis for the American National Standards Institute ("ANSI") model building loads code.
    - Member of an NRC-sponsored committee that produced the recommended guidelines for conducting the seismic margins studies of existing NPPs in the IPEEE (Individual Plant Evaluation for External Events) program.
    - Member of an advisory committee to the NRC on replacement of Part 100 Appendix A with 10 C.F.R. 100.23 and Regulatory Guide 1.165, providing for probabilistic seismic standards for NPPs and setting the recommended annual probability level.
    - Member of a DOE committee responsible for producing guidelines for seismic evaluation of the high-level radioactive waste tanks at DOE nuclear weapons facilities. This group worked in parallel with the DOE committee that produced DOE Standard 1020-94 for seismic evaluation of all DOE facilities. The two committees shared a key member, Robert P. Kennedy, and co-authored many concepts.
    - Member of a four-person panel of senior earthquake engineers requested by the American Petroleum Institute to prepare the bases and recommendations for the selection of the mean return period of the design basis earthquake for offshore structures.
    - Developer of new probability-based seismic code procedures adopted for use in the 2000 FEMA-sponsored guidelines for the design and assessment of steel-moment resisting frame buildings (a common structural system that behaved unexpectedly badly in the 1994 Northridge earthquake);
    - Co-author of 2000 draft of the International Standards Organization guidelines for seismic design of offshore oil production platforms.

- Member of a National Science Foundation-sponsored, multi-university earthquake engineering research center that is studying "performance-based earthquake engineering," which will couple PSHA, modern scientifically-based predictions of highly nonlinear dynamic building behavior, and risk-cost-benefit analysis.

- I have also served as an engineering consultant on the seismic safety assessment of major individual structures, including recently the Golden Gate Bridge, the new Pac Bell baseball park in San Francisco, the Keenleyside Dam in British Columbia, and offshore platforms in California and around the world.

4. In Contention Utah L Part B, as admitted,[2] the State of Utah asserts that:

> B. Relative to the PFS seismic analysis supporting its application and the PFS April 9, 1999 request for an exemption from the requirements of 10 C.F.R. § 72.102(f) to allow PFS to employ a probabilistic rather than a deterministic seismic hazards analysis, PFS should be required either to use a probabilistic methodology with a 10,000-year return period or comply with the existing deterministic analysis requirement of section 72.102(f), or, alternatively, use a return period significantly greater than 2,000 years, in that:
>
> 1. The requested exemption fails to conform to the SECY-98-126 (June 4, 1998) rulemaking plan scheme, i.e., only 1000-year and 10,000-year return periods are specified for design earthquakes for safety-important systems, structures, and components (SSCs) --- SSC Category 1 and SSC Category 2, respectively --- and any failure of an SSC that exceeds the radiological requirements of 10 C.F.R. § 72.104(a) must be designed for SSC Category 2, without any explanation regarding PFS SSC compliance with section 72.104(a).
>
> 2. PFS has failed to show that its facility design will provide adequate protection against exceeding the section 72.104(a) dose limits.

[2] Private Fuel Storage, L.L.C. (Independent Fuel Storage Installation), Memorandum and Order (Requesting Joint Scheduling Report and Delineating Contention Utah L) (June 15, 2001).

3. The staff's reliance on the reduced radiological hazard of stand-alone ISFSIs as compared to commercial power reactors as justification for granting the PFS exemption is based on incorrect factual and technical assumptions about the PFS facility's mean annual probability of exceeding a safe shutdown earthquake (SSE), and the relationship between the median and mean probabilities for exceeding an SSE for central and eastern United States commercial power reactors and the median and mean probabilities for exceeding an SSE for the PFS facility.

4. In supporting the grant of the exemption based on 2,000-year return period, the staff relies upon the United States Department of Energy (DOE) standard, DOE-STD-1020-94, and specifically the category-3 facility SSC performance standard that has such a return period, notwithstanding the fact the staff categorically did not adopt the four-tiered DOE category scheme as part of the Part 72 rulemaking plan.

5. In supporting the grant of the exemption based on the 2,000-year return period, the staff relies upon the 1998 exemption granted to DOE for the Idaho National Engineering and Environmental Laboratory (INEEL) ISFSI for the Three Mile Island, Unit 2 (TMI-2) facility fuel, which was discussed in SECY-98-071 (Apr. 8, 1998), even though that grant was based on circumstances not present with the PFS ISFSI, including (a) existing INEEL design standards for a higher risk facility at the ISFSI host site; and (b) the use of a peak design basis horizontal acceleration of 0.36 g that was higher than the 2,000-year return period value of 0.30 g.

6. Because (a) design levels for new Utah building construction and highway bridges are more stringent; and (b) the PFS return period is based on the twenty-year initial licensing period rather than the proposed thirty- to forty-year operating period, the 2,000-year return period for the PFS facility does not ensure an adequate level of conservatism.

5. In this declaration, I will address the appropriate standard for earthquake design of the PFSF. I will discuss the appropriateness of using a probabilistic seismic hazard analysis as the basis for designing the PFSF and the sufficiency of the 2,000 year return period earthquake and the seismic related design procedures and criteria contained in NRC guidance documents such as the Standard Review Plans ("SRPs") applicable to NRC-licensed facilities such as the PFSF (hereinafter "NRC SRPs") [e.g., Ref. 1 (NUREG-1567) and Ref. 2 (NUREG-0800)][3] as the standard for the PFSF seismic design. I shall also address specific issues raised by the State in Part B of Utah Contention L.

6. In connection with the preparation of this declaration, I read relevant filings in this proceeding, reviewed a variety of related technical documents (such as DOE Standard 1020-94, NUREG/CR-6728, etc., as cited herein), attended the deposition of the State's expert witness Dr. Walter Arabasz, and reviewed the declarations being filed simultaneously with mine by Dr. Krishna P. Singh et al. of Holtec International ("Holtec") and Mr. Bruce Ebbeson of Stone & Webster, Inc. ("Stone & Webster").

II. APPROPRIATENESS OF USING A PROBABILISTIC SEISMIC HAZARDS ANALYSIS METHODOLOGY FOR THE EARTHQUAKE DESIGN OF THE PFSF

7. The current regulations for the seismic design of ISFSIs at sites west of the Rocky Mountains (10 C.F.R. § 72.1029(b)) call for the assessment of the design basis seismic ground motions based on the deterministic procedures formerly used for nuclear power plant design (Appendix A, 10 CFR Part 100). Deterministic assessments of the seismic hazard at a site lead to one or a small set (of magnitudes and locations) of representative earthquakes and a corresponding set of ground motion response spectra. Private Fuel Storage ("PFS") has requested an exemption to these regulations in order to base the design ground motions for the PFSF on the probabilistic seismic hazard analysis methodology that I helped develop. A PSHA of a site takes into account the entire range

[3] Complete citations to cited references are provided at the end of this Declaration.

of potential events and resulting site ground motions (as measured by peak ground acceleration and spectral acceleration) with their corresponding frequencies of occurrence and uncertainties. The result is a curve of estimated annual probability of exceedance versus level of ground motion. This curve can be used to select the design ground motion at a level corresponding to a pre-specified mean annual probability of exceedance. PFS proposes to set the design basis motions for the PFSF at a mean annual probability of exceedance ("MAPE") of $5x10^{-4}$. Another way of referring to these design basis motions is to say that they correspond to the 2,000 year mean return period ("MRP") level, or "the 2,000-year MRP earthquake". This PSHA approach has replaced Appendix A, 10 CFR Part 100 in the design of nuclear power plants. See 10 C.F.R. §100.23.

8. The use of a PSHA methodology for establishing structural design basis ground motions is today the dominant industry practice. The proposed use by PFS of a PSHA both to characterize the seismic hazard at the site and to set the seismic design basis of the PFSF is fully consistent with current NRC policy and practices. Use of PSHA methodology is also prevalent in the design of other engineering facilities including buildings, bridges, offshore structures and U.S. Department of Energy ("DOE") facilities.

9. The advantages of using a probabilistic approach to establish design ground motions are widely recognized. The probabilistic approach: (1) captures more fully the current scientific understanding of earthquake forecasting than the deterministic method; (2) is capable of reflecting the uncertainties in professional knowledge of key elements of the seismic hazard; and (3) can be used to set design criteria that are consistent among different regions and among different failure consequences, thus allowing an equitable and effective allocation of seismic safety resources.

10. The NRC has recognized the advantages of the probabilistic approach and has revised the regulations and guidance for the seismic design of new nuclear power plants to be based on PSHA [Ref. 3 (10 C.F.R. §100.23) and Ref. 4 (Regulatory Guide 1.165)]. The NRC has also used probabilistic seismic procedures in areas such as re-evaluation of existing nuclear power plants and norms for high-level waste geological

repository design. This move towards probabilistic methodologies is consistent with the NRC's general policy of risk-informed regulations and decision making. [e.g., Ref. 5, (Reg. Guide 1.174 on Risk Informed Decisions) and Ref. 6 (Commission Direction Setting Issue 12, "Risk-Informed, Performance-Based Regulation")]. In accordance with this use of probabilistic procedures, the Commission is considering a proposed rulemaking to modify the current provisions of 10 C.F.R. § 72.102 to employ probabilistic procedures for the seismic design of ISFSIs [Ref. 7 (SECY-01-0178)].

11. Use of a PSHA methodology and probabilistic seismic structural design bases is also the overwhelming methodological choice in other fields. Current regulations and guidelines based on probabilistic principles include those governing the design of buildings [Ref. 8 (97 Uniform Building Code ("UBC"), p. 2-17, § 1631.2) and Ref. 9 (International Building Code ("IBC"), p. 353 § 1615.2.1], offshore structures [Ref. 10 (API RP2A, p. 125, § C.2.3.6b)], and DOE facilities [Ref. 11 (DOE-STD-1020, Table 2.1, pp. 2-4)].[4] In the building and offshore area, the use of PSHA-based designs dates to the early 1980s.

12. Thus, use of a PSHA to characterize the seismic hazard at the site and set the seismic design basis of the PFSF is appropriate and fully consistent with both NRC and broader engineering policy and practice. The State's expert witness in this proceeding agrees that a PSHA should be used for the seismic analyses of the PFSF, rather than the deterministic procedures of 10 C.F.R. § 72.102. Deposition of Walter J. Arabasz ("Arabasz Dep.") (October 31, 2001) at 44-45.

III. APPROPRIATENESS OF USING A 2,000 YEAR RETURN PERIOD EARTHQUAKE FOR THE SEISMIC DESIGN OF THE PFSF

13. PFS has performed the seismic analysis and design of important-to-safety structures, systems, and components at the PFSF using design basis earthquake (or "DBE") ground motions associated with a mean annual probability of exceedance of

[4] Portions of DOE-STD-1020 are attached as Exhibit 2 to my declaration.

$5x10^{-4}$ (i.e., a 2,000-year mean annual return period, or 2,000-year MRP) and applying the design procedures and criteria of the NRC's SRPs. It is my opinion that, taken together, use of this DBE earthquake and the SRP design procedures and criteria will achieve a level of seismic safety that is appropriate for a facility such as the PFSF. In the discussion that follows I will present the general principles of risk-informed seismic design and explain how their application to the PFSF demonstrates the validity of my opinion. The general principles that I will discuss further below are:

- In accordance with risk-informed principles, there should be a risk-graded approach to seismic safety, permitting facilities and structures with lesser failure consequences to have larger mean annual probabilities of failure.
- The seismic failure probability of a facility or structure is dictated by the combination of both the MAPE of the DBE and the level of conservatism incorporated into the design procedures and criteria.

A. Risk-Graded Approach to Seismic Safety

14. Most modern seismic design criteria are based on the principle that the probability of SSC failure (where failure is defined as exceeding a behavior limit state that may preclude the SSC from fulfilling its intended function) that needs to be addressed in the design is inversely related to the consequences of such failure. In other words, the less severe the anticipated consequences of SSC failure, the larger the probability of failure that can be tolerated. Thus, SSCs or facilities whose seismic failure would cause less severe consequences are designed to allow for higher probabilities of failure. The State's expert witness in this proceeding agrees that it is appropriate to use the risk-graded approach underlying the use of PSHA for the seismic analysis and design of structures and facilities. Arabasz Dep. at 59-60.

15. The fundamental reasons supporting the use of a risk-graded approach to seismic analysis and design are notions of equity and efficiency: the public should be provided comparable levels of safety for various societal activities, and the greatest overall safety is obtained if seismic safety resources are distributed rationally among different projects [Ref. 12 (Paté-Cornell, –Structural Safety Journal)]. Examples of

seismic standards that explicitly use this principle include the draft International Standards Organization ("ISO") guidelines for offshore structures [Ref. 13 (Banon et al., OMAE 2001 on ISO)], Federal Emergency Management Agency ("FEMA") guidelines for building assessment [Ref. 14 (FEMA 273 pp. 2-5)], and DOE Standard 1020 [Ref. 11 (Table C-3, p. C-5)]. Further, the NRC staff has stated, with respect to the seismic design of nuclear facilities: "The use of probabilistic techniques and a risk-graded approach are compatible with the direction provided by the Commission on Direction Setting 12, 'Risk-Informed, Performance-Based Regulation.'" [Ref. 15 (SECY-98-071 pp. 3-4)].

16. Under the risk-graded approach to the seismic design, ISFSIs such as the PFSF, can be assigned a higher probability of failure than a nuclear power plant because the potential consequences of seismic failure of ISFSIs are much less severe than those for nuclear power plants. The radioactive inventory that potentially could be released to the environment from an ISFSI is less because the spent fuel has decayed significantly and because a spent fuel canister is under much lower pressures than a reactor's coolant boundary; higher pressures will disperse any released radioactivity farther from the source. The NRC has rejected the notion that licensing standards should be as high for ISFSIs as for nuclear power plants, noting that "[t]he potential ability of irradiated fuel to adversely affect the public health and safety and the environment is largely determined by the presence of a driving force behind dispersion. Therefore, it is the absence of such a driving force, due to the absence of high temperature and pressure conditions at an ISFSI (unlike a nuclear reactor operating under such conditions that could provide a driving force), that substantially eliminate the likelihood of accidents involving a major release of radioactivity from spent fuel stored in an ISFSI." [Ref. 16 (60 Fed. Reg. 20,883 (1995))].

17. Further, an ISFSI facility as a whole is inherently less vulnerable to earthquake-initiated accidents than a nuclear power plant. An ISFSI is largely passive; it does not have active cooling and safe-shutdown systems necessary for maintaining the integrity of the high-pressure reactor coolant boundary and for shutting down after a large earthquake, as does a nuclear power plant. The NRC has recognized the reduced seismic

vulnerability of an ISFSI by stating that for ISFSIs, such as dry storage casks, which do not involve massive storage structures, "the required design earthquake will be determined on a case-by-case basis until more experience is gained with licensing these types of units." [Ref. 17 (45 Fed. Reg. 74,697 (1980), as cited in Ref. 15 (SECY-98-071 p. 2).]

B. Factors Determining Failure Probability for Facilities and Structures

18. While the risk-graded approach is implemented in somewhat different ways in the various fields of seismic design, the standards of practice almost invariably utilize a DBE defined at some mean annual probability of exceedance and a set of design procedures and acceptance criteria. Both the procedures and the acceptance criteria include conservatisms that, implicitly or explicitly, are intended to implement "performance goals" (e.g., target levels of the seismic failure probability for the facility or structure), which are defined in a manner reflecting the anticipated consequences of the failure. These conservatisms are typically embedded in the various codes and standards pursuant to which the design of a structure or facility is accomplished.

19. Both the MAPE of the DBE and the level of conservatism incorporated in the design procedures and criteria affect the failure probability of seismically-designed facilities and structures. A lower (or higher) failure probability can be achieved by keeping the design procedures and criteria fixed while reducing (or increasing) the MAPE of the DBE; or, alternatively, by fixing the MAPE while making the design procedures more or less conservative; or by adjusting both elements simultaneously. Whichever choice is made among these alternatives, it is important to understand that both the MAPE and the level of conservatism in the design procedures and criteria must be considered when assessing and comparing the safety implications of various seismic design standards. One fact remains true, however: because of the conservatisms incorporated in all seismic design procedures and criteria, the probability of failure of a seismically-designed facility or structure is virtually always less than the MAPE of the DBE. In other words, virtually facilities and structures designed against a given DBE have a mean return period to failure that is longer than the mean return period of the

earthquake for which they are designed. In practical terms, this means that seismically-designed facilities and structures are able to withstand a more severe, i.e., more infrequent, earthquake than that used as the DBE.

20. The application of these principles of risk-graded seismic design is perhaps most clearly and explicitly seen in the U.S. Department of Energy's Standard 1020. The basis for DOE Standard 1020 is a set of "performance categories" (1 to 4) for seismically designed facilities and structures with increasing consequences of failure, and thus decreasing probabilities of failure as their performance goals [Ref. 11 (DOE 1020, p. 1-2 and p. C-2)]. DOE is responsible for (1) facilities such as ordinary buildings (Performance Category 1 or PC1) designed to protect occupant safety, (2) essential facilities and buildings that should continue functioning after an earthquake with minimal interruption (PC 2), (3) important facilities such as ISFSIs that contain hazardous materials (PC3), and (4) critical facilities such as those involving nuclear reactors (PC4).

21. The performance goals for DOE structures, systems and components in the four performance categories PC1 to PC4 are set as mean annual failure probabilities of 10^{-3}, $5x10^{-4}$, 10^{-4}, and 10^{-5}, respectively [Ref. 11 (DOE 1020, p. C-5)] reflecting the increasing consequences of failure. On the other hand, MAPEs for the design basis ground motions are set as $2x10^{-3}$, 10^{-3}, $5x10^{-4}$, and 10^{-4}, respectively. These values are uniformly larger than the performance goals.

22. To bridge the gap between the performance goals and the DBE MAPEs, the DOE 1020 standards call for design procedures and evaluation criteria that vary among the categories, ranging from those "corresponding closely to model building codes" for PC1 and PC2 , to those for PC4 which "approach the provisions for commercial nuclear power plants" [Ref. 11 (DOE 1020, p. 2-2, C-4 to C-5)]. The quantitative effect, in terms of reducing earthquake risk, of applying the conservatisms built into these various design procedures and criteria is reflected in the ratios between the MAPE of the design basis ground motions and the corresponding performance goal probabilities. These ratios are 2, 2, 5 and 10, respectively [Ref. 11 (DOE 1020, p. C-5)]. The ratios are called "Risk Reduction Ratios", R_R, in DOE 1020. The following table

summarizes these three parameters, the DBE MAPE, the Performance Goal, and the R_R for the four performance categories PC1 through PC4 in DOE 1020:

Table 1: DOE Std. 1020-94 Seismic Performance Goals, DBE MAPEs and R_Rs[5]

Performance Category	**Target Seismic Performance Goal (P_F)**	**DBE Exceedance Probability (MAPE)**	**Risk Reduction Ratio (R_R)**
PC1 (e.g., office building)	1×10^{-3}	2×10^{-3}	2
PC2 (e.g., essential building that should remain operational, such as hospital or police station)	5×10^{-4}	1×10^{-3}	2
PC3 (e.g., hazardous waste facilities such as ISFSIs)	1×10^{-4}	5×10^{-4} (except 1×10^{-3} for Western sites near tectonic boundaries)	5 (except 10 for Western sites near tectonic boundaries)
PC4 (e.g., nuclear reactor facility)	1×10^{-5}	1×10^{-4} (except 2×10^{-4} for Western sites near tectonic boundaries)	10 (except 20 for Western sites near tectonic boundaries)

[5] A revised draft version of DOE Standard 1020 was released in August of this year for comment [Ref. 18 (DOE-1020-2001)]. The primary change is that PC1 and PC2 will be based on the IBC 2000 instead of the UBC model building code. As a result, this table would differ under the proposed standard in that the MAPE of PC1 and PC2 categories would change to 4×10^{-4}. To be consistent, the MAPE of PC3 is modified slightly to the 4×10^{-4} value. The performance goals remain the same in all categories. The R_R for PC3 would therefore be changed from 5 to 4, although no change would be made to the design procedures and criteria for PC3. The R_R column is left blank for PC1 and PC2, but it can be shown that the R_R is still about 2, using the information in NERHP Recommended Provisions for Seismic Regulations for New Buildings and Other structures [Ref. 19 (FEMA-303, at p. 37)] and the procedures outlined in Attachment A hereto. These proposed revisions to DOE 1020, if adopted, would not in any way alter the analyses and conclusions in this Declaration.

23. The actual value of R_R obtained from the design conservatisms for a given SSC is dependent on the shape or slope of the ground motion hazard curve. For example, the PC4 value of 10 cited in the table is representative of locations in the Central and Eastern United States. However, higher risk reduction ratios, e.g., 20 for PC4 facilities, are achieved in western US sites near tectonic boundaries, where hazard curves are steeper [Ref. 11 (DOE 1020, Table 2-1 p. 2-4)]. The higher achievable R_R values have allowed the DOE to specify that higher DBE MAPE levels can be used for PC4 facilities as well as for PC3 facilities in these regions.

24. In DOE 1020, the overall conservatism levels are controlled through acceptance criteria to achieve specific R_R levels [Ref. 11 (DOE 1020, pg. 1-5)]. The document states: "These design and evaluation criteria have been developed such that the target performance goals of the [Natural Phenomenom Hazard] Implementation Guide [set forth in Table 1 above] are achieved" [Ref. 11 (DOE 1020, p. 2-1)]. In other words, the risk reduction levels in DOE 1020 are achieved through use of the DOE design and evaluation criteria specified in Chapter 2 of DOE Standard 1020 and related appendices.[6] For PC4 facilities the risk reduction factor achieved is 10 in most regions.

25. The design guidelines provided by the NRC SRPs also contain many conservatisms that result in risk reduction factors as large as, or larger than, those for PC4 category facilities designed to DOE 1020. NRC SRP standards share with DOE's PC3 and PC4 categories many procedures leading to design conservatism [Ref. 11 (DOE 1020, pp. C-5, C-6)]. These conservatisms are introduced through prescribed analysis

[6] The State's witness has suggested that the risk reduction ratio does not measure the conservatism in a DOE PC category's design procedures and criteria, but rather that it is simply defined as the ratio of the DBE MAPE to the Performance Goal, and hence it is only the ratio required to achieve the goal. Although one might arguably draw that conclusion from the statement in DOE 1020 that the "required degree of conservatism in the deterministic acceptance criteria is a function of the specified risk reduction ratio," [Ref. 11 (DOE 1020, p. C-5)], the quote in the body of the text clearly confirms that, upon selecting the required ratio DOE then established the prerequisite design and evaluation criteria in Chapter 2 of the DOE-1020 to achieve the goals. Therefore, the ratio also becomes a measure of the conservatism provided for by the design and evaluation criteria set forth in Chapter 2 of DOE Standard 1020 and the related appendices.

methods, specification of material strengths, limits on inelastic behavior, etc. The conservatism levels in NRC seismic SRPs are not explicitly keyed to values of R_R. Nonetheless, the risk reduction factors achieved through the use of NRC guidelines for typical SSCs have been found in application to be equal to, or higher than, those called for in DOE 1020 for PC4 facilities, since they are greater than 10 in most regions. DOE 1020 acknowledges the higher R_R levels provided by the NRC SRPs by stating that the "[c]riteria for PC4 approach the provisions for commercial nuclear power plants". [Ref. 11 (DOE 1020, p. 2-2, C-4 to C5). There is recent independent technical support both for the general conclusion that NRC SRPs provide equal or greater levels of conservatism than DOE 1020, and for the quantitative finding that the R_R levels for typical systems, structures, and components designed to NRC SRPs are in the range 5 to 20 or greater [Ref. 20 (NUREG/CR-6728 at Chapter 7)]. [7]

C. Application of General Principles to the PFSF

26. At the PFSF, designing for the 2,000-year MRP DBE ground motion and using the NRC SRPs means that typical important-to-safety systems, structures and components can be expected to have seismic failure probabilities 5 to 20 or more times lower than the DBE MAPE, i.e., $2.5\text{x}10^{-5}$ to $1\text{x}10^{-4}$ or lower (i.e., seismic failure MRPs of 10,000 to 40,000 years or more). Therefore, the PFSF would easily meet the DOE performance objectives of $1\text{x}10^{-4}$ for PC-3 facilities under which ISFSIs, such as the PFSF, would fall. The State's expert witness, Dr. Arabasz, agreed that ISFSIs, such as the PFSF, would appropriately be classified PC-3 facilities under DOE-1020 and that the performance objective of $1\text{x}10^{-4}$ for the PFSF would be an appropriate standard on which to determine the acceptability of its seismic design. Arabasz Dep. at 80-81.

27. Applying a risk-graded seismic approach, a performance objective of $1\text{x}10^{-4}$ for ISFSIs such as the PFSF is consistent with the NRC's performance objectives for operating nuclear plants, which pose higher radiological hazard consequences than

[7] Demonstration of these conclusions requires a somewhat detailed technical discussion, which is presented in Attachment A to this Declaration.

ISFSIs. While the NRC seismic performance goals and the quantitative effects of their design criteria are less explicit than those in DOE Standard 1020, inferences can be made from existing NRC standards. The NRC's quantitative safety objective with respect to core damage is a mean annual frequency of 10^{-4} [Ref. 21 (SECY-00-0077 at p. 6)] ("Mean annual frequency" and "mean annual probability" are effectively equivalent). Some undefined fraction of this "budget" is available for seismically induced core damage. Past NRC seismic standards for nuclear power plants have provided a mean annual seismically-induced core damage frequency of about 10^{-5}. [Ref. 22 (NUREG/CR-5501 (1989) at p. 26)] In NUREG/CR-5501, a study prepared for the NRC, the mean annual seismic core damage frequency of seven existing plants was estimated to range from about 4×10^{-6} to about 1×10^{-4}, with most lying between 0.6 and 1×10^{-5}. Thus, in order to achieve a probability of seismic "failure" (core damage) of 10^{-5}, and noting that the typical safe shutdown earthquake for a nuclear power plant has an MAPE of 10^{-4} (see discussion of Basis 3 of Part B of Utah L below), a risk reduction ratio of 10 or more is implied for most U.S. nuclear power plant sites. This number is consistent with the R_R of 5 to 20 or more cited above for the conservatism inherent in NRC SRP design procedures and criteria. Also, the use of a higher probability of seismic failure goal for the PFSF (i.e., 10^{-4}) than that for nuclear power plants (10^{-5}) is consistent with the risk-graded approach of the probabilistic approach.

28. The R_R levels for the PFSF SSCs cited above are confirmed by the beyond-design-basis analyses and margins descriptions provided in the Declarations of Bruce Ebbeson and Holtec for critical PFSF structures, systems and components The dry storage casks used to store spent fuel at the facility are stubby cylindrical weldments of steel and concrete designed to tolerate significant earthquake induced forces, including those resulting from their tipping over. The spent fuel canisters for the HI-STORM storage system are designed for transportation as well as storage, giving them a ruggedness that allows them to resist earthquake accelerations. The transfer casks associated with the transfer of spent fuel canisters from transportation to storage casks are in use only a fraction of the time and, if damaged in an earthquake, can be repaired or replaced without adverse safety consequences.

29. I understand that analyses performed by the cask manufacturer demonstrate that the storage casks used at the PFSF will not tip over if subjected to the ground motions caused by an earthquake with an MAPE of 10^{-4} (that is, a 10,000 year return period earthquake). In addition, even if they should tip over, the conservatisms in the design of casks and canisters will prevent the release of radioactivity. Other SSCs at the PFSF, including the Canister Transfer Building ("CTB") and the important-to-safety SSCs therein, are also likely, due to their conservative design to the NRC SRPs, to be able to survive the ground motions from a 10,000 year return period earthquake. Further, some of those SSCs (such as the CTB crane and the seismic struts) are in use only a fraction of the time, thus a canister would be exposed to potential risk of damage due to their failure only a fraction of the time. For such intermittent use components, the annual likelihood of failure during a safety-important operation is reduced further. For example, even if the fraction of time they are used is as high as 20%, the annual probability of failure causing release due to earthquake ground motions is at least 5 times smaller. This implies that, even if their R_Rs were only unity instead of the factors of 5 to 20 or more estimated above, their relevant frequencies of failure would be better than a 10^{-4} goal. With the predicted R_R of 5 to 20 or more, this estimated failure frequency reduces to about 10^{-5}.

30. The foregoing analysis demonstrates, both in absolute terms and by comparison to nuclear power plant standards, that the proposed PFSF seismic design basis of a 2,000-year MRP DBE and the SRP design procedures and criteria provide an appropriate and consistent level of protection to public health and safety.

IV. DISCUSSION OF SPECIFIC ISSUES RAISED IN THE STATE'S CONTENTION

31. In Part B of Contention Utah L, the State raises several challenges to the use of a 2,000 year MRP earthquake as the basis for the PFSF seismic design. The State's contentions are generally responded to by the analysis presented in Section III above. Nonetheless, I will next address each of the State's specific arguments.

32. In Basis 1 to Part B of Utah L, the State challenges the exemption granted by the NRC Staff to PFS authorizing the use of a 2,000 year return period DBE on the grounds that such an exemption fails to conform to the rulemaking plan set forth in SECY-98-126 (June 4, 1998). That plan proposed a 1000-year mean return period design basis earthquake for Category 1 SSCs and a 10,000-year mean return period design basis earthquake for Category 2 SCCs, with SCCs whose failure would results in radiological doses exceeding the requirements of 10 C.F.R. § 72.104(a) being designated Category 2 SCCs. The State's challenge appears to be legal rather than technical, in that the State is contending that it was inappropriate for the NRC Staff to grant the exemption in view of the existing rulemaking plan. However, the NRC Staff has now recommended to the Commission a modification of the rulemaking plan that would use a 2,000-year mean return period earthquake as the basis for ISFSI design, the same DBE as that provided for by the proposed exemption for the PFSF. [Ref. 7 (SECY-01-0178 (September 26, 2001))]. Therefore, the grounds asserted by the State in this objection to the granting of the exemption to PFS are no longer valid. Moreover, as I discussed in Section III above, use of a 2,000-year MRP together with the NRC SRP design procedures and criteria provide sufficient protection of the public health and safety. This result satisfies the rulings of both the Licensing Board and the Commission to the effect that neither PFS nor the NRC Staff were bound by the SECY-98-126 rulemaking plan and could use a 2,000-year mean return period for the PFSF provided that it was demonstrated that a design based on a 2,000-year return period earthquake was sufficiently protective of public health and safety. [Ref. 23 (CLI-01-12 at Section II.C.1 (June 14, 2001))].

33. In Basis 2 to Part B of Utah L, the State challenges the exemption granted by the Staff to PFS on the grounds that "PFS has failed to show that its facility design will provide adequate protection against exceeding the section 72.104(a) dose limits." As observed by the Licensing Board, this claim is "an adjunct" to Basis 1 in which the State claims that the technical basis for a 2,000-year mean return period design basis earthquake has not been adequately established in light of SECY-98-126. [Ref. 24 (LBP-01-03, slip op. at 15)]. The discussion in Section III and the response to Basis 1 above

demonstrate that, given the design bases and the inherent ruggedness of the SSCs in the PFS facility, the PFSF design will provide appropriate seismic safety.

34. In Basis 3 to Part B of Utah L, the State challenges the exemption on the grounds that the Staff's reliance on the reduced radiological hazard of stand-alone ISFSIs as compared to commercial power reactors as justification for granting the PFS exemption is based on "incorrect factual and technical assumptions." This statement apparently alludes to the State's assertion that for a nuclear power plant "design ground motions would have to correspond to a median annual probability of exceedance of 10^{-5}" [Ref. 25 (State's Request for Admission of Late-Filed Modification to Basis 2 of Contention Utah L, pp. 8- 11)].

35. This assertion is incorrect. Regulatory Guide 1.165 provides general guidance to applicants as to procedures that the Staff would deem acceptable for satisfying the NRC's new probabilistic seismic criteria in 10 C.F.R. § 100.23. It does state that the annual probability level of the SSE may be based on a *median estimate of* 10^{-5}. [Ref. 4 (Reg. Guide 1.165, Appendix B, p. 1.165-12)]. However, there is ample evidence that the staff today would both select and prefer an SSE based on a *mean estimate of* 10^{-4}. Moreover, it has been shown that the typical SSE at existing plants across the country has a mean annual probability of exceedance of 10^{-4}. The mean estimate is a preferred measure in the face of the uncertainty in the estimate.

36. The provision in Regulatory Guide 1.165 that a median value of 10^{-5} could be used is only the result of historical circumstances. At the time of preparation of the probabilistic NRC criteria in 10 C.F.R. § 100.23 and Regulatory Guide 1.165, there was a significant discrepancy in the assessment of the mean estimates between the two major CEUS seismic hazard studies then available. While both studies provided similar median estimates, they differed with respect to the mean estimates. Therefore, the median estimate was adopted for the purposes of establishing in Regulatory Guide 1.165 an acceptable quantitative basis for satisfying 10 C.F.R. § 100.23. The discrepancy between two studies has, however, since been resolved and the two studies now provide similar mean estimates of the SSE earthquakes for nuclear plants located in the CEUS.

37. The mean estimate is preferred to the median when making decisions based on uncertain annual probabilities or frequencies. When faced with uncertain probability estimates, the NRC has accordingly chosen as a general matter to use the mean probability estimate. For example, the Commission's "Safety Goals for Operations of Nuclear Power Plants; Safety Policy Statement" states: "The Commission has adopted the use of the mean estimates for purposes of implementing the quantitative objectives of this safety goal policy (i.e., the mortality risk objectives)." [Ref. 30 (51 Fed. Reg. 28,044, 28,046 (1996)]. The NRC's choice of the mean estimate for all such risk objectives, including the subsidiary core melt damage frequency, is discussed in Regulatory Guide 1.174, "An Approach for Using PRA in Risk-Informed Decisions on Plant-Specific Changes to the Licensing Basis", at pg. 14 [Ref. 5] and in SECY-00-0077, "Modifications to the Reactor Safety Goal Policy Statement" at pg. 6 [Ref. 22]. Thus, in accordance with well accepted practices, the Commission has clearly stated its preference for the use of mean estimates. Given this preference along with the fact that the underlying reason for using the median rather than the mean in Regulatory Guide 1.165 is no longer applicable, the Commission can reasonably be expected to revert to its preferred use of the mean estimate. In this respect, both the original 10 CFR part 72 rulemaking plan (SECY-98-126) and the modified plan (SECY-01-0178) call for the use of mean probability estimates.

38. The mean annual probability of exceeding the SSE at existing nuclear power plants is on the average about 10^{-4}. For CEUS sites, this is demonstrated in DOE 1020 at p. C-17 [Ref. 11], in NUREG/CR-6728 at p. 7-15 [Ref. 20], and in DOE Topical Report for Yucca Mountain TR-003 at App. C [Ref. 26]. These were the sites used in the preparation of Regulatory Guide 1.165. It has also been demonstrated more recently in the DOE Topical Report II TR-003 at App. C [Ref. 26] that this same number is also representative of Western US nuclear power plant sites.[8] Therefore, if the Staff were to repeat today the calibration procedure used in Appendix B of Regulatory Guide 1.165 to establish the median value of 10^{-5}, it would likely conclude that a mean of 10^{-4} was more

[8] Portions of DOE Tropical Report TR-003 are attached as Exhibit 3 to my declaration.

appropriate. Indeed, if the NRC were to adopt the State's argument and decided to use a median 10^{-5} value, would lead to inconsistent mean SSE probabilities across the country because, as the State has correctly pointed out, the ratio of mean to median is not constant across all regions. Because the mean is the preferred estimate in risk-consistent procedures, this lack of uniformity in the mean SSE probabilities would be inappropriate.

39. Thus, under the circumstances described above, it is to be expected that the NRC would today not only accept but prefer that a new nuclear power plant, whether in the Eastern or Western United States, have an SSE value based, not on the median 10^{-5}, but on the mean 10^{-4} annual frequency of exceedance. There is evidence to this effect in recent NRC actions and writings. In SECY-98-071 at pg. 2, the Staff states "Based on 10 CFR 100.23 requirements, as described in Regulatory Guide 1.165, . . ., a future nuclear power plant in the western United States can use as a safe shutdown earthquake the 10,000-year return period mean ground motion." [Ref. 15]. SECY-01-0178 (at pg. 6) states: "The present design earthquake (equivalent to the SSE for an NPP) has a mean annual probability of exceedance of approximately 1.0E-04" [Ref. 7]. The NRC has accepted the use of the mean 10^{-4} ground motions as a basis for (Category 2 SSCs) at the Yucca Mountain high level waste storage facility [Ref. 7] (SECY-01-0178, pg. 5).

40. For these reasons, the argument raised by the State in Basis 3 is inconsequential and irrelevant to the issue whether a 2,000-year earthquake should be used at the PFSF.

41. In Basis 4 to Part B of Utah L, the State challenges the exemption granted to PFS on the grounds that the Staff inappropriately relied on DOE-STD-1020-94 (or DOE 1020), given that the Staff did not adopt this Standard in SECY-98-126. The exemption is, however, consistent with the Staff's proposed amendment to the Rulemaking Plan set forth in SECY-01-0178. In any event, the document is relevant in that it represents the considered judgement of a major federal agency, and the PSFS design basis meets and surpasses the PC3 performance goals.

42. DOE 1020 is an important seismic standards document. It has been carefully prepared, with the support of recognized experts in the field, by a major federal agency that has experience with a broad spectrum of nuclear facilities, has authority to set standards, and has responsibility for public safety. The document is considered a model of explicit, graded, risk-consistent seismic criteria. It was for this reason that I used it to illustrate the use of such standards in Section III above. DOE 1020 is therefore a relevant document that supports the exemption. Its Performance Category 3 represents a facility like the PFSF ISFSI, and the PFSF – with a 2,000-year MRP DBE and use of the NRC's SRPs – will meet the performance criteria of DOE 1020 for PC-3 facilities. The State's expert witness, Dr. Arabasz, has stated that he supports the use at the PFSF of a DOE 1020 PC3 performance goal. Arabasz Dep. at 80-81.

43. The State, in its September 28, 2001 discovery responses [Ref. 27] to PFS Interrogatory No. 6 claims, however, that PFS has not demonstrated that its use of a 2,000-year DBE would meet the DOE-1020 performance goal for Performance Category 3 (i.e., a mean annual probability of failure of 10^{-4}). To recapitulate, my previous discussion, I showed that the DOE-1020 performance goal for Performance Category 3 facilities will be met and even bettered at PFSF. In Section III, I showed that the mean annual failure probability depends on both the DBE MRP and the level of conservatism in design procedures and criteria. I then demonstrated that for typical SSCs the NRC SRPs provide a level of conservatism equal or better than DOE 1020's PC4 criteria, which are in turn more conservative than DOE 1020's PC3 criteria. It was shown that the design to the NRC SRPs produces R_R values of 5 to 20 or more; the lowest value in this range, 5, is sufficient to meet the performance goal of 10^{-4}. Wholly independently, I next discussed that specific important-to-safety SSCs have been shown to meet the goal. For a storage cask the 10^{-4} goal is confirmed simply by noting that it will not tip until it experiences an earthquake with an annual probability of less than 10^{-4} and, furthermore, its probability of radiation release assuming tipping is very small, given the conservatisms in the design of the cask and the canisters. For some other important to safety SSCs, such as the CTB crane, I pointed out that their annual probability of seismic failure leading to release is less than the product of the MAPE of the DBE ($5x10^{-4}$) and

the fraction of time they are operating. I am informed that this fraction at the PSF IFSFI is approximately 20% or less, yielding a product of $1x10^{-4}$. In fact the actual annual failure rate leading to release will be less than this number by the a factor equal to R_R, which is 5 to 20 or more, yielding a value of about 10^{-5}.

44. In the same discovery response (September 28, 2001 discovery response to PFS Interrogatory No. 6 [Ref. 27]) the State claims that "[I]n the context of DOE-STD-1020-94, PFS has not demonstrated for its proposed ISFSI facility that use of a 2,000-year return period would achieve DOE's target performance goal, which requires consideration of such factors as the slope of the site-specific hazard curve over the annual probability range of 10^{-3} to 10^{-5}, seismic fragility curves, and quantified uncertainties in the fragility curves. (DOE-STD-1020-94) at section C." This assertion is not true. In the context of DOE-1020, to meet the performance goal one has only to follow the design procedures and acceptance criteria specified in Chapter 2 of the document; these procedures and criteria have been specifically established to meet the desired goal. No hazard curve slopes or fragility curves or quantification of uncertainty are required. Therefore, Section C of DOE Standard 1020 cited by the State is in fact merely an appendix which demonstrates, by using these slopes, fragility curves, etc., that the procedures and criteria set forth in Chapter 2 of the Standard do achieve the desired performance goal. I have further demonstrated in Attachment A that, because of the generally greater conservatism of the NRC SRP design procedures and acceptance criteria used at the PFSF, the PFSF will meet or surpass the performance goal of PC3 in DOE Standard 1020.

45. In Basis 5 to Part B of Utah L, the State challenges the grant of the PFSF exemption claiming that the Staff's reliance on the 1998 exemption granted to DOE for the Idaho National Engineering and Environmental Laboratory ("INEEL") ISFSI for the Three Mile Island, Unit 2 ("TMI-2") facility fuel is misplaced because the grant of the exemption there was based on circumstances not present with the PFS ISFSI, including (a) existing INEEL design standards for a higher risk facility at the ISFSI host site; and (b) the use of a peak design basis horizontal acceleration of 0.36 g that was higher than

46. the 2,000-year return period value of 0.30 g. However, this decision by the Staff and Commission was not intended to be so narrowly interpreted. This is evident in the NRC Staff's final statement to the Commissions in SECY-98-071, pg. 4 [Ref. 15]: "If the staff grants the exemption to 10 CFR 72.102 (f)(1), this may impact the licensing process for other ISFSIs in the western United States. Until the ISFSI seismic requirement in Part 72 is amended by rulemaking, the staff may receive similar exemption requests for other ISFSIs to be sited west of the Rocky Mountain front." Thus, the Staff was advising the Commission that the granting of the exemption for INEEL would be relied upon as precedent for other exemption applications, as was done in the case of the PFSF.

47. In Basis 6 to Part B of Utah L, the State claims that the 2,000-year mean return period for the PFS facility does not ensure an adequate level of conservatism because design levels for certain new Utah building construction and highway bridges are more stringent. As set forth in the State's September 28, 2001 discovery response [Ref. 27] to Interrogatory No. 8, this conclusion was based on the observation that, for example, the International Building Code 2000 (or [Paul; OK?]"IBC-2000") will, when in effect in the future, require a MRP of approximately 2500 years for the DBE, which is greater than the 2,000-year MRP DBE proposed for PFS. One should not draw the erroneous conclusion, however, that this difference in the definition of the DBE implies a lower probability of failure for SSCs designed to IBC-2000 versus those, such as the PFSF, designed to the 2,000-year MRP and the NRC's SRP design procedures and criteria. As described in detail in Section III, the safety achieved depends on *both* the DBE MRP and on the design procedures and criteria utilized. The design procedures and criteria of the IBC-2000 are much less conservative than those of the SRP. For example, as described by the State, a first step of the IBC-2000 design procedures and criteria is to multiply the DBE by two-thirds, which at the PFSF site would reduce the effective IBC-2000 DBE MRP from 2500 years to about 800 years. Only in the case of those "essential structures" that merit the IBC-2000 "importance factor" of 1.5 is this two-thirds reduction, in effect, recovered.

48. Further, the model building codes' design procedures and acceptance criteria are significantly less conservative than those in the SRP. The IBC-2000 and

UBC model building codes permit much more liberal allowances for the benefits of post-elastic behavior than either DOE 1020 PC-3 and PC-4 criteria or the NRC SRPs. As shown in Table 1, the net effect of the UBC design and acceptance criteria, which are quite similar to those in IBC-2000 and to DOE 1020 PC1 and PC2, is a risk reduction ratio R_R of only 2, versus a value of 10 for DOE-1020 (PC-4) and typically 5 to 20 or more for the facilities designed to the NRC SRP. This represents a factor of 2.5 to 10 or more in increased conservatism in the design procedures of the latter standards over model building codes, even if the multiplier of two-thirds in the IBC-2000 is ignored. Therefore, even though the use of IBC-2000 for essential or hazardous buildings will imply a DBE with a 25% larger MRP than that for the PFSF (assumes applicability of the effective 2500 MRP for "essential structures" in IBC-2000), the more conservative design procedures and criteria of the SRP will provide that the typical PFSF SSCs have a mean annual probability of failure several times (2.5 to 5 or more) lower than buildings designed to IBC-2000 standards. In addition, as discussed above, a number of key safety-to-important SSCs in the PFSF have great robustness and/or fractional operating periods, which reduce their failure probabilities even further.

49. While I am less familiar with bridge codes, it is my general understanding that they have design procedures and criteria similar to those of model building codes such as UBC and IBC-2000. Therefore, assuming that a 2500-MRP DBE is used in the design of certain essential bridges in Utah, my discussion of IBC-2000 standards is equally applicable to bridges; the design of the PFSF under a 2,000-year return period earthquake and NRC seismic SRP design criteria provides higher safety levels than those available in the design of these special Utah bridges.

50. The State also claims in Basis 6 to Part B of Utah L that the 2,000-year mean return period for the PFS facility does not ensure an adequate level of conservatism because the return period was chosen based on the twenty-year initial licensing period rather than a potential thirty to forty-year operating period. This contention is unfounded because in virtually all areas of public safety hazards are measured as annual probabilities (or frequencies) of occurrence, regardless of the length of the activity in question, the exposure time, the estimated facility life, or the licensing duration [Ref. 12 (Paté-Cornell

paper)]. This is also the case with respect to the risk acceptance guidelines promulgated by the NRC where the subsidiary performance objectives are the risk metrics Core Damage Frequency (CDF) and Large Early Release Frequency (LERF). [Ref. 5 (Reg. Guide 1.174 at p. 10)] and [Ref. 22 (SECY-00-0077 at p. 6)]. The reasons for focusing on annual risks in making facility safety decisions include the fact that any facility providing a needed service will, at the end of its operating life, most likely be replaced by some other facility used for the same purposes with its own, similar risks. The spent fuel to be stored at the proposed PFSF is currently being stored in or near nuclear power plants, and after leaving the PFSF it will likely be stored at the proposed Yucca Mountain facility.

V. SUMMARY

50. In this Declaration I have explained why the use of probabilistic seismic hazard analysis to establish the design basis ground motions at the PFSF site is consistent with current NRC practice and that in other technical fields. I have showed that the 2000-year mean return period ground motions (i.e., those with mean annual probabilities of exceedance of $5x10^{-4}$) together with the NRC SRPs design procedures and acceptance criteria will provide an appropriate level of public safety for the PFS ISFSI. Finally, I have addressed each of the bases asserted by the State in support of Part B of Contention Utah L and established that they do not undercut or controvert my conclusions.

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

Executed on November 9, 2001.

C. Allin Cornell

ATTACHMENT A

DETERMINATION OF RISK REDUCTION FACTORS FOR SSCs AT FACILITIES DESIGNED USING NRC SEISMIC SRP STANDARDS

The objective of this Attachment is to show the analytical process used to determine quantitatively the degree of conservatism inherent in the design procedures and acceptance criteria found in both DOE Standard 1020 and the NRC SRPs. This level of conservatism is captured in the risk reduction factor or ratio R_R. By calculating the values of R_R resulting from DOE Standard 1020 and the NRC SRPs, the risk reduction factors implicit in the SRP design procedures and criteria can be compared to risk reduction factors expressly provided for in DOE 1020. The precise calculated value of R_R depends on several technical parameters (defined below) whose values may vary from site to site and from SSC to SSC. Accordingly, one can produce only a representative range of R_R values for both the SRP and DOE 1020. (As an example, Figure C-4 on page C-11 of DOE-1020 [Ref. 11] shows the range of R_R values for SSCs designed to the criteria specified for category PC4 SSCs in DOE-1020.)

The risk reduction ratio, R_R, is defined in NUREG/CR-6728 [Ref. 21 pp. 7-9] by the equation:

$$R_R = F_R^{K_H} \left(e^{x_p \beta}\right)^{K_H} e^{-\frac{1}{2}(K_H \beta)^2}$$

A different formulation of this same equation appears also in DOE-1020 at page C-9. In this equation, the variables are as follows:

- K_H, a measure of the slope of the PSHA seismic hazard curve;
- β, a measure of the degree of uncertainty in the response and capacity of SSCs;

- F_R, a measure of the margin (achieved by the procedures and criteria) between the level of the DBE and a reference SSC capacity; and
- x_p, a measure of the margin between this reference capacity and the median value of the SSC capacity.

These variables are defined in more detail in both of the references cited above (DOE 1020 at Appendix C.2 and NUREG/CR-6728 at Section 7.2).

For the purposes of this comparison, I will use for both the SRP and the DOE 1020 R_R determinations a range of values for the hazard curve slope K_H = 2.1 to 3.3 (NUREG/CR-6728 at pg. 7-6). These values are representative of the relevant hazard interval (10^{-4} to 10^{-5}) for nuclear power plants at CEUS sites (DOE 1020 at pg. C-8-9, and C-12)[1], and also of the relevant hazard interval (10^{-3} to 10^{-4}) for DOE PC3 (i.e., ISFSI) SSCs at the PFSF site (e.g., the K_H at the PSFF site for peak ground acceleration is 2.8, as determined from [Ref. 28 (Revised Geomatrix Appendix F at Fig. 6-11 __)]. For simplicity, I use here a typical value of β = 0.4. (The conclusions are quite insensitive to β as shown in DOE 1020 at Figure C-4 on page C-11.) These values for K_H of 2.1 to 3.3 and for β of 0.4 are common to the calculations below of the R_R for both DOE 1020 and the NRC SRP.

First, I consider the DOE 1020 R_R standards. For these standards, the appropriate value of x_p is 1.28 and the appropriate value of F_R is 1.5 SF, both of which appear in DOE 1020 at Eq. C-6, pg. C-9. For PC4 the value of the "scale factor" SF is set at 1.25 (and for PC3 it is set at 1.0) in order to achieve the desired risk reduction ratio R_R [DOE 1020 at pg. 2-13]. Substitution of the above values for K_H, β, x_p, and F_R into the equation for R_R leads to a range of values of R_R from 8

[1] For clarity, if one uses this reference, it needs to be pointed out that the K_H range above corresponds precisely to the A_R range of 2 to 3 that will be found at this citation; A_R is an alternative hazard curve slope measure, DOE 1020, at pg. C-8).

to 17 for DOE 1020 category PC4, as can be seen on Figure C-4 on page C-11 of DOE 1020. The results of these and similar calculations were used in DOE 1020 to confirm the conclusion that the DOE 1020 design procedures and acceptance criteria set forth in Chapter 2 would achieve a value of R_R of about 10, as required to meet the PC4 performance goal. DOE 1020 at p. C-12.

Unlike DOE 1020, the NRC SRPs have not been "tuned" to give a particular R_R (or more precisely a representative value, such as 10 above, applicable to a range of sites). Accordingly, it has been necessary to depend on the numerous engineering evaluations of safety margins and "fragility curves" of SSCs designed to the SRP that have been conducted over the last 20 years in the course of research by the industry and NRC contractors, and on the seismic probabilistic risk assessments and seismic margins studies that have been undertaken at virtually all nuclear power plants in the US (via the NRC IPEEE program). These evaluations have been made by earthquake engineers familiar with nuclear power plant SSC designs prepared to the NRC SRP procedures and criteria, and with the actual behavior of such SSCs in earthquakes as observed in the field and tested in the lab. This experience is summarized in NUREG/CR-6728 at pg. 7-3 by the conclusion: "For nuclear power plant design the factor of safety has typically been 1.25 to 1.5." NUREG/CR-6728 (at pg. 7-4). This "factor of safety" is the variable F_R in the above equation. This factor is, however, coupled with a value of x_p of 2.33. NUREG/CR-6728 (at Ch. 7), which determines the definition of the reference capacity (referred to as a "HCLPF" or C_1) used in engineering evaluations of SRP conservatisms. This value of x_p is much more conservative than that used in DOE-1020.

Using this value of x_p and this range of F_R values one finds (for the same β value and range of K_H values used for the DOE 1020 calculations above) that the R_R for the SRP is in the range 8 to 32. Compared to the range of 8 to 17 calculated for DOE 1020, this result confirms that the

DOE 1020 PC4 standard does indeed only "approach" those of the NRC SRP, as stated in DOE-1020 at page C-5.

If one looks, not at the range of hazard curve slope values of 2.1 to 3.3 used for K_H in the above calculations, but rather at the specific value $K_H = 2.8$ associated with peak horizontal ground acceleration at the PFSF site, the range of NRC SRP R_R values is 12 to 21. For the subset of SSCs sensitive to 1 second spectral accelerations, the ratios range from 8 to 12 based on the reduced slope of the hazard curve for this period. Revised Geomatrix Appendix F at Fig. 6-11.

For simplicity in the body of the declaration and in the [Ref. 29] Applicants Response to the State's Int. 15, Item 9, I have summarized such detailed results in the statement that "the R_R's for typical components SSCs designed to the NRC SRP are in the range 5 to 20 or greater".

REFERENCES

Reference 1: U.S. Nuclear Regulatory Commission, NUREG-1567, *Standard Review Plan for Spent Fuel Dry Storage Facilities*, March 2000.

Reference 2: U.S. Nuclear Regulatory Commission, NUREG-0800, *Standard Review Plan*, August 1988.

Reference 3: 10 Code of Federal Regulations § 100.23.

Reference 4: U.S. Nuclear Regulatory Commission, Regulatory Guide 1.165, *Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion*, March 1997.

Reference 5: U.S. Nuclear Regulatory Commission, Regulatory Guide 1.174, *An Approach for Using Probabilistic Risk Assessment in Risk-Informed Decisions on Plant-Specific Changes to the Licensing Basis*, July 1998.

Reference 6: U.S. Nuclear Regulatory Commission, Strategic Assessment Issue Paper, Direction Setting Issue 12, *Risk-Informed, Performance-Based Regulation Strategic Assessment*, September 16, 1996.

Reference 7: U.S. Nuclear Regulatory Commission, SECY-01-0178, *Rulemaking Plan: Geological and Seismological Characteristics for Siting and Design of Dry Cask Independent Spent Fuel Storage Installations, 10 CFR Part 72*, September 26, 2001.

Reference 8: Uniform Building Code, Vol. 2, 1997.

Reference 9: International Building Code, 2000.

Reference 10: American Petroleum Institute, API Recommended Practice 2A-WSD (RP 2A-WSD), *Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms – Working Stress Design*, twentieth ed., July 1, 1993.

Reference 11: U.S. Department of Energy, DOE-STD-1020-94, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*, January 1996.

Reference 12: Paté-Cornell, M.E., *Quantitative Safety Goals for Risk Management of Industrial Facilities*, Structural Safety Journal 13, 1994.

Reference 13: Banon, H, Cornell, C.A., Crouse, C.B., Marshall, P.W., Nadim, F, and Younan, A. H., *ISO Seismic Design Guidelines for Offshore Platforms*, Proceedings of the 20th Offshore Mechanics and Artic Engineering Conference – OMAE 2001, Rio de Janeiro, Brazil, June 2001.

REFERENCES

Reference 14: Federal Emergency Management Agency (FEMA-273), *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*, October 1997.

Reference 15: U.S. Nuclear Regulatory Commission, SECY-98-071, *Exemption to 10 CFR 72.(f)(1) Seismic Design Requirement for Three Mile Island Unit 2 Independent Spent Fuel Storage Installation*, April 8, 1998.

Reference 16: 60 Federal Register 20,883 (1995).

Reference 17: 45 Federal Register 74,697 (1980).

Reference 18: U.S. Department of Energy, DOE-STD-1020-2001, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*, Rev. 1, 2001.

Reference 19: Federal Emergency Management Agency (FEMA-303), *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, Part 2 – Commentary, 1997 ed., February 1998.

Reference 20: U.S. Nuclear Regulatory Commission, NUREG/CR-5501, *Selection of Review Level Earthquake for Seismic Margin Studies Using Seismic PRA Results*, October 1989.

Reference 21: U.S. Nuclear Regulatory Commission, NUREG/CR-6728, *Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines*, October 2001.

Reference 22: U.S. Nuclear Regulatory Commission, SECY-00-0077, *Modifications to the Reactor Safety Goal Policy Statement*, March 30, 2000.

Reference 23: PFS Memorandum and Order, CLI-01-12, June 14, 2001.

Reference 24: Private Fuel Storage, L.L.C. (Independent Spent Fuel Storage Installation) LBP-01-03, 53 NRC 84 (2001).

Reference 25: State of Utah's Request for Admission of Late-Filed Modification to Basis 2 of Contention Utah L, November 9, 2000.

Reference 26: U.S. Department of Energy, Topical Report YMP/TR-003-NP, *Preclosure Seismic Design Methodology for a Geologic Repository at Yucca Mountain*, Rev. 2, August 1997.

Reference 27: State of Utah's Objections and Responses to Applicant's Seventh Set of Formal Discovery Requests to Intervenor State of Utah, September 28, 2001.

REFERENCES

Reference 28: Geomatrix Consultants, Inc., *Fault Evaluation Study and Seismic Hazard Assessment*, Rev. 1, Final Report, Vol. 1, Private Fuel Storage Facility, Skull Valley, Utah, March 2001.

Reference 29: Applicant's Objections and Responses to the State of Utah's Eleventh Set of Discovery Requests Directed to the Applicant, October 2, 2001.

Reference 30: 51 Federal Register 28,044 (1986).

EXHIBIT 1

Resume of C. Allin Cornell

C. ALLIN CORNELL

EDUCATION:

Stanford University, Architecture	AB	1960
Stanford University, Civil Engineering (Structures)	MS	1961
Stanford University, Civil Engineering (Structures)	PhD	1964

Doctoral Thesis: "Stochastic Process Models in Structural Engineering"

PROFESSIONAL EMPLOYMENT:

Stanford University :	Acting Assistant Professor	1963-1964
Universidad Nacional Autonoma de Mexico :	Visiting Professor	Summer 1966
University of California, Berkeley :	Visiting Associate Professor	1970-1971
Basler and Hofmann, Zurich:	Research Engineer	Summer 1972
Laboratorio Nacional de Engenharia Civil, Lisbon:	Visiting Research Investigator	1974-1975
Massachusetts Institute of Technology:	Assistant Professor and Ford Post-Doctoral Fellow	1964-1966
	Assistant Professor	1966-1968
	Associate Professor	1968-1974
	Holder of Gilbert Winslow Career Development Chair	1971-1974
	Professor	1974-1983
Stanford University :	Visiting Professor	1981-1983
	Professor (Research) - Half-Time	1983-present
	Co-Director, Reliability of Marine Structures Program	1988-present
	Fellow, SU-USGS Institute on Earthquake Engineering and Seismology	1986-1996
Consulting Practice:	Part-Time	1965-1981
	Half-Time	1981-present
Cygna, Inc., San Francisco	Senior Vice President	1984-1985
C. Allin Cornell, Co.	President	1981-present

PROFESSIONAL ORGANIZATIONS AND COMMITTEES (Current and Former):

American Iron and Steel Institute:
- Advisory Committee on Load-Factor Building Design

American National Standards Institute:
- Building Loads Code Committee A58

American Society of Civil Engineers:
- Committee on Structural Safety
- Committee on Nuclear Power Plant Safety
- Committee on Bridge Safety

Committee on Offshore Structure Safety

Earthquake Engineering Research Institute:
Editorial Board: Earthquake Spectra, 1991-1993
Seismic Risk Committee
Planning Committee, 50th Anniversary Annual Meeting, 1998-99

Joint European Committee on Structural Safety

National Academy of Engineering (Elected 1981)

Phi Beta Kappa

Seismological Society of America:	Board of Directors,	1984-1987
	Vice-president	1985-1986
	President	1986-1987

Sigma Xi

Society of Risk Analysis:
Senior Advisory Board, 1991 P.S.A.M. Conference

JOURNAL EDITORIAL BOARDS:

Structural Safety; Risk Abstracts; Probabilistic Engineering Mechanics; Engineering Structures; Earthquake Spectra, Uncertainties in Structural Mechanics

GOVERNMENT COMMITTEES AND SERVICE:

NBS, Consultant	1967-1975
USGS, Advisory Committee to Seismicity and Risk Analysis Branch	1974
UNESCO, Working Group on Definition of Seismicity and Ground Motion	1974
USGS, Workshop on Earthquake Prediction and Engineering Hazards	1977
NAE/NRC-Marine Board	
Committee on Offshore Technology	1979-1981
Committee on Marine Structures,	
Loads Advisory Group	1986-1987
Parent Committee	1987-1989
NAS Committee on Seismology	1981-1984
Panel on Science of Earthquakes	1996-2001
NAS-Water Board	
Committee on Techniques for Estimating Probabilities of Extreme Floods	1986-1988
NAE/NRC-Geotechnical Board - Comm. for Workshop on Reliability Methods for Risk Mitigation in Geotechnical Engineering	1992-1994

NRC Seismic PRA Seminar Technical Coordinator	1982
OECD-CSNI Specialist Meetings: Probabilistic Methods in SRA for NPP's	
Chairman	1980
Technical Organizing Committee	1983
NATO, Advanced Study Institute, Reliability of Structures and Soils, Lecturer, (Seismic Safety of NPP's)	1982

AWARDS RECEIVED:

Huber Research Prize, American Society of Civil Engineers	1971
Guggenheim Fellowship	1974-1975
Fulbright-Hayes Advanced Research Grant	1974-1975
Moisseiff Award, American Society of Civil Engineers	1977
Norman Medal, American Society of Civil Engineers	1983
(First) ICASP Award, Committee of Inter. Conference on Applications of Statistics and Probability in Soils and Structures	1987
Fruedenthal Medal, American Society of Civil Engineers	1988
Offshore Technology Research Center Honors Lecture, OTC	1995
EERI Distinguished Lecturer	1999
EERI Outstanding Paper of 1998 (Earthquake Spectra) (Co-authors: Shome, Bazzurro, and Carballo)	2000

SOME REPRESENTATIVE RECENT SPONSORED UNIVERSITY RESEARCH CONTRACTS:

SPONSOR:

NSF Stochastic Models of Structural Loads.
Spatial and Temporal Memory in Earthquake Recurrence and Hazard.
Nonlinear Seismic Assessment Procedures for Buildings
Probabilistic Prediction of Near-Source Strong Ground Motion and Nonlinear Structural Response

PEER (NSF Earthquake Engineering Center): Technical Foundation for Performance-Based Design

SAC Nonlinear Seismic Demands in Fracturing Steel Moment-Resisting Frames

ONR Reliability Analysis of Moored Marine Structures.

EPRI Multi-site Wind Record Analysis for Transmission Lines Structural Loads.

Effectiveness of Strong Ground Motions.

MMS Probability-Based Design Procedures for Offshore Structures

NRC Hazard-Consistent Nonlinear Analysis of Structures and Soils

JOINT INDUSTRY PROJECT
(36 company consortium, managed by Amoco Production Company)
Structural Systems Reliability Analysis for Offshore Structures.

INDUSTRIAL AFFILIATES PROGRAM 1986-present
Reliability of Marine Structures.

[resumes\largeparts\log.vitae\04\00]

REPRESENTATIVE CONSULTING PROJECTS

1999 *Seismic Studies (Seismic Hazard Analysis;*
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;
Policy Advising, etc.):
DOE/Woodward-Clyde (Yucca Mountain PSHA Peer Review)
NRC/REI (Ground Motions Procedures Peer Review Panel)
SAC (Reliability-based Building Assessment Guidelines)
BP Amoco/EQE (ISO Offshore Seismic Guidelines)
Westinghouse (Savannah River Seismic Review)
NRC/ICF (Advisory Committee: New Dry Storage Cask Guidelines)
Offshore Platform Structures/Marine Reliability
REI/JIP (Riser Reliability)
E&P Forum JIP/REI (Low Probability Storm Assessment)
ABS (M.O.B.: Probability-based Design Procedures)
BP-Amoco (Prob. Asses. Of Extreme Ice Effects)
Other
DOE/Geomatrix (Design Decision Process: Yucca Mtn.)
BC Hydro (Dam Safety Guidelines; review)
WES/Ben Gerwick (Dam PRA Methodology)

1998 *Seismic Studies (Seismic Hazard Analysis;*
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;
Policy Advising, etc.):
DOE/Woodward-Clyde (Yucca Mountain PSHA Peer Review)
DOE/Geomatrix (Yucca Mountain Volcano Hazard Analysis)
NRC/REI (Ground Motions Procedures Peer Review Panel)
B.C. Hydro (Keenleyside Dam Seismic Risk, Peer Review Panel)
Bechtel (Hanford Vitrification Plant PSHA)
Offshore Platform Structures/Marine Reliabiity
REI/JIP (Riser Reliability)
ABS (Risk-Based Ship Criteria)
Mobil (Seismic Design Frequency)
E&P Forum JIP/REI (Low Probability Storm Assessment)
EPR (Reliability Tutorial)
ABS (M.O.B.: Probability-based Design Procedures)
Other
DOE/Geomatrix (Design Decision Process: Yucca Mtn.)
BC Hydro (Dam Safety Guidelines; review)

1997 *Seismic Studies (Seismic Hazard Analysis;*
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;
Policy Advising, etc.):
DOE/Woodward-Clyde (Yucca Mountain PSHA Peer Review)
DOE/Geomatrix (Yucca Mountain Volcano Hazard Analysis)
NRC/REI (Ground Motions Procedures Peer Review Panel)
NRC/Waterways Exper.Sta. (Probabilistic Liquefaction Analysis)
B.C. Hydro (Keenleyside Dam Seismic Risk, Peer Review Panel)

DOE/Pacific Engineering (Probabilistic Soil Amplification; Savannah River Site)
Cal. Earthquake Authority (Expert testimony)
Offshore Platform Structures/Marine Reliabiity
Amoco (Offshore Reliability)
REI/JIP (Riser Reliability)
ABS (Risk-Based Ship Criteria)
Bechtel (M.O.B.: Extreme Environment Characterization; Reliability)
ABS (M.O.B.: Probability-based Design Procedures)
Exxon Production Research (Seismic Criteria)
Other
EPRI/Sargent and Lundy (Temporary Loads Reliablity)
BC Hydro (Dam Safety Guidelines; review)

1996 *Seismic Studies (Seismic Hazard Analysis;*
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;
Policy Advising, etc.):
USGS/DOE (Review of U.S. Hazard Maps)
DOE/Woodward-Clyde (Yucca Mountain PSHA Peer Review)
DOE/Geomatrix (Yucca Mountain Volcano Hazard Analysis)
NRC/REI (Ground Motions Procedures Peer Review Panel)
NRC/Waterways Exper.Sta. (Probabilistic Liquefaction Analysis)
Warburg Pincus (Seismic Insurance Risk Methods)
Aon Insurance Services (Seismic Insurance Risk Analysis)
Seattle Seahawks (King Dome Seismic Review)
B.C. Hydro (Keenleyside Dam Seismic Risk, Peer Review Panel)
Offshore Platform Structures/Marine Reliabiity
Chevron (Hurricanes)
Amoco (Offshore Reliability)
REI/JIP (Riser Reliability)
Shell/PMB (Maui A and B Seismic Reliability)
ABS (Risk-Based Ship Criteria)

1995 *Seismic Studies (Seismic Hazard Analysis;*
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;
Policy Advising, etc.):
DOE/Woodward-Clyde (Yucca Mountain PSHA Peer Review)
DOE/Geomatrix (Yucca Mountain Volcano Hazard Analysis)
DOE/BNL (Short-term Seismic Exposure)
MMS/LLNL (Santa Barbara Channel PSHA)
B.C. Hydro (Seismic Risk Methods)
NRC/LLNL (Seismic Source Characterization)
EQE (Review of Cal. Eq. Auth. Analysis)
USGS/ATC (Paper/Workshops on PSHA)
Offshore Structures Reliability
Exxon Pro.Res. (Seismic Hazard and Response: Caspian Sea/Sakalin Island)
Mobil (Seismic Hazard and Response: Holly Platform)
PMB/JIP (Hurricane Andrew Bayesian Update of
Structural Loads and Capacities II)

1994 *Seismic Studies (Seismic Hazard Analysis;*
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;

Policy Advising, etc.):
DOE/LLNL (Senior Hazard Advisory Committee;
site hazard revisions)
DOE/BNL (Tanks Seismic Expert Panel; site reviews)
NRC/LLNL (Appendix B Revision; expert committee)
DOE/High-Level Waste Review Board
Commonwealth Edison Co. (Short-Term Criteria)
Woodward-Clyde (Hazard Methodology Update)
SRI/EDF (France) (SPRA Methodology)
Westinghouse Hanford (Safety Class Definition)
REI/DOE (SHA review)
Guy Carpenter Inc. (Loss estimation review)
ISEC/Golden Gate Bridge Retrofit
Offshore Structures Reliability:
PMB/JIP (Hurricane Andrew Bayesian Update of
Foundation Capabilities)
PMB/JIP (Hurricane Andrew Bayesian Update of
Structural Loads and Capacities II)
REI/JIP (Reliability Software Development Advice)
Chevron (Hurricane Statistics)
Exxon Production Research (Response Analysis)
Statoil (Failure Probability Bases}

1993 *Seismic Studies (Seismic Hazard Analysis;*
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;
Policy Advising, etc.):
DOE/LLNL (Senior Seismic Hazard Advisory Committee; site reviews)
DOE/BNL (Tanks Seismic Expert Panel)
Woodward-Clyde (SHA)
San Diego Gas & Electric (SHA Review)
EPRI (Max. Magnitude Project)
NRC/CNWRA (HLW Seismic Criteria)
ISEC/Golden Gate
REI/NRC(Seismic Motions/PRA)
EPRI (Max. Magnitude Project)
Ofshore Structures Reliability:
PMB/JIP (Hurricane Andrew Bayesian Update of
Structural Loads and Capacities)
Unocal (Seismic safety review; SHA reviews)
Chevron (Extreme Wave Reliability-Methodology)
Statoil (Norway) (North Sea SHA review)
PMB/JIP (Dynamic Capacity)

1992 *Seismic Studies (Seismic Hazard Analysis;*
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;
Policy Advising, etc.):
DOE/LLNL (NPR Senior Advisory Committee; Interim Criteria, site reviews)
DOE/BNL (TSEP; site SHA reviews)
NRC/LLNL (Appendix B Revision, expert panel)
EPRI (Maximum Magnitude Project)
Geomatrix (CalTrans SHA reviews)
Woodward-Clyde (CalTrans SHA reviews)

Portland General Electric (Senior Seismic Panel)
ISEC/Golden Gate
REI/NRC (Seismic Motions/PRA)
ESA (Aqueduct Analysis)
REI/NSF (Loma Prieta Motions Analysis)

Offshore Structures Reliability:
Unocal (SHA review; SHA and criteria)
REI (TLP-LRFD JIP)
PMB/USN
PMB/JIP (Dynamic Capacity)
PMB/JIP (Andrew Bayesian Update)
Chevron (Reliability Methodology)
API (Seismic Requalification Criteria)

1991 *Seismic Studies (Seismic Hazard Analysis; Seismic Probability Risk Assessment; Seismic Margins; Criteria Development; Policy Advising, etc.):*
DOE/LLNL (Natural Hazards; NPR Senior Advisory Committee; Interim Criteria, site reviews)
BC Hydro (Seismic Hazard Committee)
Portland General Electric (Senior Seismic Panel)
EPRI (Maximum Magnitude Project)
NRC
REI/CGMG (Seismic Motion Analysis)
REI/NRC (Seismic Motions/PRA)
Offshore Structures Reliability:
PMB/USN (Underwater Array Reliability)
EPR (Seismic Review)
API (Seismic Requalification Criteria)
Other:
Paul, Hastings, Janofsky and Wal (Fiber Pipe Reliability)

1990 *Seismic Studies (Seismic Hazard Analysis; Seismic Probability Risk Assessment; Seismic Margins; Criteria Development; Policy Advising, etc.):*
DOE/LLNL/BNL (NPR Senior Advisory Committee; Interim Criteria; Site Reviews; High-Level Waste Tanks)
EPRI/NUMARC/IPEEE
Exxon Production Research (Reliability)
USGS/NEPEC (Bay Area Seismic Hazard)
NRC/ACNW
Portland General Electric
Woodward-Clyde Consultants
Offshore Structures Reliability:
Exxon Production Research (EPR) (reliability software)
PMB/NCEL
ELF Aquitaine (France)/LRFD Development

Other:
NASA/Veritas Research (Structural Reliability)

1989 *Seismic Studies (Seismic Hazard Analysis;
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;
Policy Advising, etc.):*
DOE/LLNL (Senior Review Group: External Events Assessment and Criteria; NPR Criteria)
Pacific Gas and Electric
Portland General Electric
Electric Power Research Institute
(Severe Accident Policy, Seismic Hazard, High Frequency Ground Motion Effects)
Nuclear Regulatory Commission/ANL
Woodward Clyde Consultants
Risk Engineering, Inc.
Geomatrix
Offshore Structures Reliability:
Joint Industry Project (12 sponsors); Full-scope Reliability ("MCAPS"); Amoco Production Co., Manager.
ELF Aquitaine (France)
Exxon Production Research
Statoil (Norway)

1988 *Seismic Studies (Seismic Hazard Analysis;
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;
Policy Advising, etc.):*
Pacific Gas and Electric Co. (Diablo Canyon Long-Term Seismic Program, Advisory Board and Consultant)
Electric Power Research Institute (Senior Advisory Group: Eastern U.S. Seismic Hazards Project)
Risk Engineering, Inc.
U.S. Nuclear Regulatory Commission/ANL
Portland General Electric (Senior Seismic Panel)
Bechtel Corporation
Canada Oil and Gas Administration
Statoil (Norway)
Offshore Structures Reliability:
Joint Industry Project (36 sponsors); Structural Systems
Reliability; Amoco Production Co., Manager
Joint Industry Project (12 sponsors): Full-Scope Systems
Reliability ("MCAPS"); Amoco Production Co., Manager
ELF Aquitaine (France)
Amoco Production Co.
Exxon Production Research
Bridge Loadings:

NCHRP (Jointly with Imbsen and Associates, Inc.)

1987 *Seismic Studies (Seismic Hazard Analysis;*
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;
Policy Advising, etc.):
Pacific Gas and Electric Co. (Diablo Canyon Long-Term Seismic Program, Advisory Board and Consultant)
Electric Power Research Institute (Senior Advisory Group: Eastern U.S. Seismic Hazards Project)
(Non-Poissonian Earthquake Recurrence Analysis Project)
U.S. Nuclear Regulatory Commission
Geomatrix
Offshore Structural Reliability:
Joint Industry Project (36 sponsors); Systems Reliability; Amoco Production Co., Manager
Joint Industry Project (12 sponsors): Full-Scope Systems Reliability ("MCAPS"); Amoco Production Co, Manager
ELF Aquitaine (France)
Site-Specific Bridge Loads:
NCHRP (Jointly with Imbsen and Associates, Inc.)

1986 *Seismic Studies (Seismic Hazard Analysis;*
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;
Policy Advising, etc.):
Pacific Gas and Electric Co. (Diablo Canyon Long-Term Seismic Program, Advisory Board and Consultant)
Electric Power Research Institute (Senior Advisory Group: Eastern U.S. Seismic Hazards Project)
Woodward-Clyde
Impell
Bechtel Corp.
Yankee Atomic Electric Co.
U.S. Nuclear Regulatory Commission
Offshore Structures Reliability:
Joint Industry Project (36 sponsors); Systems Reliability; Amoco Production Co., Manager
Joint Industry Project (12 sponsors): Full-Scope Systems Reliability ("MCAPS"); Amoco Production Co., Manager
ELF Aquitaine (France)
Amoco Production Co.

1985 *Seismic Studies (Seismic Hazard Analysis;*
Seismic Probability Risk Assessment;
Seismic Margins; Criteria Development;
Policy Advising, etc.):

Pacific Gas and Electric Co. (Diablo Canyon Long-Term Seismic Program, Advisory Board and Consultant)
Electric Power Research Institute (Senior Advisory Group: Eastern U.S. Seismic Hazards Project)
(Non-Poissonian Earthquake Recurrence Analysis Project)
Maine Yankee Power Co.
Yankee Atomic Electric Co.
U.S. Nuclear Regulatory Commission (Design Margins and SPRA Validation Senior Advisory Committees)
Bechtel Corp.
Sandia (Long-Term Nuclear Waste Disposal)
Electricite de France
Structural Systems Reliability:
G.A. Technologies (through DOE) (HTGR Probability-Based Design Criteria Advisory Board)
Offshore Structures Reliability:
ELF Aquitaine (France)
Joint Industry Project (36 sponsors); Structural Systems Reliability; Amoco Production Co., Manager
Joint Industry Project (12 sponsors): Full-Scope Systems Reliability ("MCAPS"); Amoco Production Co., Manager
Statistical Analysis of Construction Quality Sampling:
Anolik et al (Shelter Ridge Condominiums)
Fairfield et al (Hunters Point Housing Project)

1984 *Seismic Studies (Seismic Hazard Analysis; Seismic Probability Risk Assessment; Seismic Margins; Criteria Development; Policy Advising, etc.):*
Maine Yankee Power Co. (Maine Yankee)
Lawrence Livermore National Laboratory
Pacific Gas and Electric Co. (Diablo Canyon)
Yankee Atomic Electric Co. (Yankee Rowe, et al)
Niagara Power (through Dames and Moore)
NRC (Design Margins and SPRA Validation Senior Advisory Committees)
Dames and Moore (Millstone)
Electric Power Research Institute (Senior Advisory Group: Eastern U.S. Seismic Hazards Project)
Probabilistic Extreme Precipitation and Flood Analysis:
Yankee Atomic Electric Co.
Risk Analysis Tutorials, Short Courses, etc.:
Woodward-Clyde Consultants
ACTA, Inc.
Offshore Structures Design Criteria:
PMB Systems (SOHIO, Shell)

1983 *Seismic Studies (Seismic Hazard Analysis: Seismic Probability Risk Assessment; Seismic Margins; Criteria Development; Policy Advising, etc.):*
Maine Yankee Power Co. (Maine Yankee)

Lawrence Livermore National Laboratory
NRC, (ACRS)
Yankee Atomic Electric Company
Cygna, Inc.
Boston Edison (through Yankee Atomic Electric Co.)
Pickard, Lowe & Garrick, Inc. (Seabrooke)
Niagara Power (through MPR, and Dames and Moore)
Electric Power Research Institute (Research through Yankee Atomic Electric Co.)
Electric Power Research Institute (Eastern Seismic
Hazard Project Senior Advisory Committee)
Law Engineering and Testing Co. (Duke Power Co.)
Office of Naval Research
A. Anolik (Westborough Housing Study)

Structural Code Development:
Electric Power Research Institute/Col. State Univ. (Transmission Lines)
ACTA, Inc.

Probabilistic Extreme Precipitation and Flood Analysis:
Yankee Atomic Electric Co.

Risk Analysis Tutorials, Short Courses, etc.:
Woodward-Clyde Consultants (Probabilistic Methods)
ACTA, Inc. (Extreme Events)

Offshore Structures Design Criteria (Waves, Ice, System Reliability, etc.):
PMB Systems (SOHIO, Shell)

1982 *Seismic Studies (NPP Sites):*
Pickard, Lowe and Garrick (Zion, Indian Point, Seabrooke)
Yankee Atomic Electric Co. (Yankee Rowe)
Maine Yankee Power Co. (Maine Yankee)
Woodward-Clyde Consultants
Stone and Webster Corp. (Millstone)
Dames and Moore (Millstone)
Electric Power Research Institute (through Yankee
Atomic Electric Co.: Development of Historic SHA)
NRC, Lawrence Livermore National Laboratory

Pile Foundation System Reliability:
NUCLEN, (Brazil)

Structural Code Development:
Electric Power Research Institute/Colorado State Univ.
ACTA, Inc.

Load Combination Analysis:
Lawrence Livermore National Laboratory

Risk Analysis Tutorials, Short Courses, etc.:
NRC (through Sandia National Laboratory)
Woodward-Clyde Consultants

1980-81 *Seismic Studies:*
Pickard, Lowe and Garrick
Yankee Atomic Electric Power Co.
Lawrence Livermore National Laboratory

Pile Foundation System Safety:
NUCLEN, (Brazil)

Load Combination Analysis:
Lawrence Livermore National Labortory

1979-80 *Seismic Studies*:
Pickard, Lowe and Garrick
Weston Geophysical Research
Woodward-Clyde Consultants
Lawrence Livermore National Laboratory/NRC
Yankee Atomic Electric Co.
Air Pollution Hazard Study:
Pickard, Lowe and Garrick
Structural Safety Short Course:
Raytheon Co.
Load Combination Analysis:
G.E. Mark II Reactor Owners Group (through N.M. Newmark)

1978-79 *Seismic Studies*:
T.V.A.
Weston Geophysical Research
Southern California Edison Co.
Woodward-Clyde Consultants
Lawrence Livermore National Labortory/NRC
Load Combination Studies:
G.E. Mark II Reactor Owners Group (through N.M. Newmark)

1977-78 *Seismic Risk Analysis and Ground Motion Predictions*:
T.V.A.
Pacific Gas and Electric Co.
Woodward-Clyde Consultants
Seismic Reliability Studies of Nuclear Power Plant Systems:
Southern California Edison Co. Pacific Gas and Electric Co.
Pickard, Lowe and Garrick
Technical Chairman; one-week seminar for German Government (BAM)
Nuclear Regulatory Commission and Lawrence Livermore National Laboratory;
Senior Advisory Group: Seismic Safety Margins Research Project

1976-77 *Seismic Risk Analysis and Ground Motion Consultation*
Bell Laboratories
Pacific Gas and Electric Co.
Law Engineering
U.S. Army Corps of Engineers
Boston Edison Co.
Weston Geophysical Research, Inc.
Statistical Analysis of Fires:
NFPA

1975-76 *Probabilistic Systems Analysis; Dutch Oosterschelde Closure Project*:
T. W. Lambe and Associates
Seismic Risk Analysis and Ground Motion Consultation:
Nuclear Fuel Services
Dames and Moore

Weston Geophysical Research, Inc.
Boston Edison Co.
Basler and Hofmann
Advisory Committee on NFPA Project on Probabilistic Fire Safety Analysis

1974-75 *Seismic Risk Analysis Consultation*:
Dames and Moore
Weston Geophysical Research, Inc.
Aircraft Crash Risk Consultation:
Pickard and Lowe

1973-74 *Aircraft Crash Risk Studies for Nuclear Power Plants*
for PEPCO and Stone and Webster through Weston Geopysical Research, Inc. and others
Seismic Risk Analyses and Artificial Design Motions
for Several Engineering Projects
Assorted Hazard Study Reviews
for Pickard and Lowe
Refinement and Documentation of Seismic Risk Analysis Programs
for J. A. Blume and Associates
Wind-Loading Studies on Boston's John Hancock Building
for Hansen, Holley and Biggs
National Bureau of Standards Building Live Loads Survey
Report Preparation; and (through J. H. Wiggins and Company)
Survey Implementation Review

1972-73 *Through Weston Geophysical Research, Inc., American Electric Power; Stone and Webster; et al.*:
Design Response Spectra and Probabilistic Artificial Motions for Several
Nuclear Power Plant Projects
For Pickard and Lowe:
Wind-Induced Wave Risks on Great Lakes
Review of Seismic Risk Analysis for Dames and Moore
Consultation to NBS on Live Load Survey Implementation
Aircraft Crash Risk Analysis for Nuclear Power Plants
for Oregon Nuclear and Thermal Energy Council

1971-72 *Design of a Building Live Loads Survey*
for National Bureau of Standards
Through Weston Geophysical Research, Inc.:
a) Response Spectra and Seismic Design Criteria for Several Nuclear Power Plants
b) Development of Seismic Risk Map for American Electric Power
Retained as Seismic Consultant to Environmental Research, Inc., Las Vegas, Nevada
Through Hansen, Holley and Biggs:
Seismic Design Levels and Response Spectra for Drydock Sites on West Coast
for Crandall Drydocks, Inc.
Wind Dispersion Analysis
for Pickard and Lowe
Advisor to University of Mexico Earthquake Engineering Project
for UNESCO

1970-71 *Review of Fire Loads Survey Analysis for CEACM, Paris*

Through Weston Geophysical Research, Inc.: Seismic Design Criteria for several Nuclear Power Plants
Aircraft Crash Risk Analysis for Pickard and Lowe

PUBLICATIONS

Book:

Benjamin, J. R. and Cornell, C. A., *Probability, Statistics, and Decision for Civil Engineers*, McGraw-Hill Book Company, 1970.

Papers in Referred Journals:

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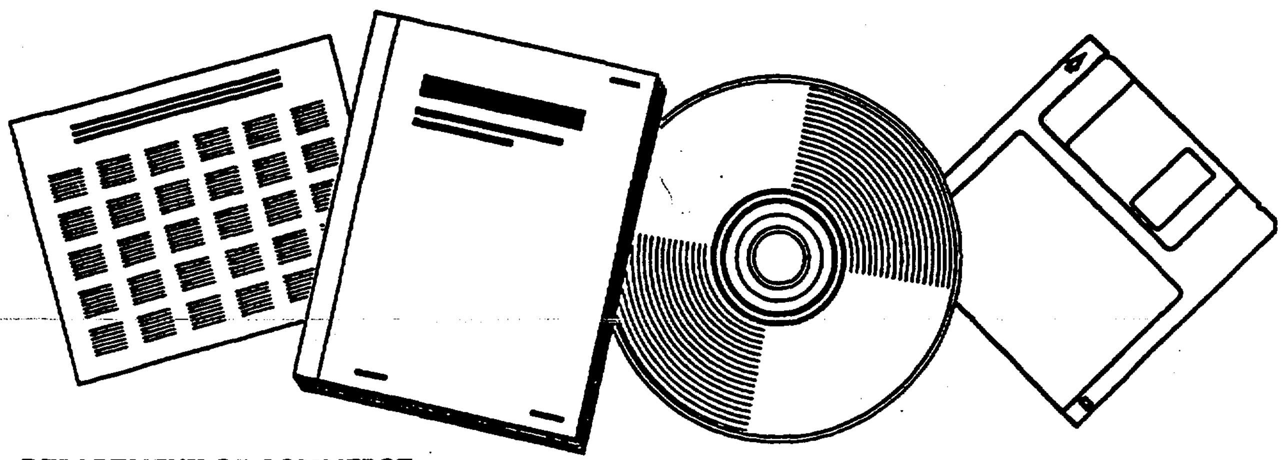
EXHIBIT 2

DOE-STD-1020-94

NATURAL PHENOMENA HAZARDS DESIGN AND EVALUATION CRITERIA FOR DEPARTMENT OF ENERGY FACILITIES

DEPARTMENT OF ENERGY
WASHINGTON, DC

JAN 1996



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The design and evaluation criteria presented herein control the level of conservatism introduced in the design/evaluation process such that earthquake, wind, and flood hazards are treated on a consistent basis. These criteria also employ a graded approach to ensure that the level of conservatism and rigor in design/evaluation is appropriate for facility characteristics such as importance, hazards to people on and off site, and threat to the environment. For each natural phenomena hazard covered, these criteria consist of the following:

1. Performance Categories and target performance goals as specified in the DOE Order 420.1 NPH Implementation Guide, and DOE-STD-1021.

2. Specified probability levels from which natural phenomena hazard loading on structures, equipment, and systems is developed.

3. Design and evaluation procedures to evaluate response to NPH loads and criteria to assess whether or not computed response is permissible.

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Chapter 1
Introduction

1.1 Overview of DOE Natural Phenomena Hazards Order, Standards, and Guidance

It is the policy of the Department of Energy (DOE) to design, construct, and operate DOE facilities so that workers, the general public, and the environment are protected from the impacts of natural phenomena hazards on DOE facilities. DOE Order 420.1, "Facility Safety" (Ref. 1-1) and the associated Implementation Guides, "Implementation Guide for the Mitigation of Natural Phenomena Hazards for DOE Nuclear Facilities and Non-nuclear Facilities" (Ref. 1-2), "Implementation Guide for Nonreactor Nuclear Safety Design Criteria and Explosives Safety Criteria" (Ref. 1-3), and "Implementation Guide for use with DOE Orders 420 and 470 Fire Safety Program" (Ref. 1-4) identify the responsibilities and requirements to execute this policy in a consistent manner throughout DOE which includes: (1) providing safe work places; (2) protecting against property loss and damage; (3) maintaining operation of essential facilities; and (4) protecting against exposure to hazardous materials during and after occurrences of natural phenomena hazards. There is an established hierarchy in the set of documents that specify NPH requirements. In this hierarchy, DOE Order 420.1 is the highest authority. The next set of controlling documents are the associated Implementation Guides followed by the set of NPH standards. The NPH requirements have been developed to provide the necessary information that assess the NPH safety basis for DOE facilities, which is documented in Safety Analysis Reports (SARs), if available. DOE 5480.23 (Ref. 1-5) and the guidance provided in the associated Standard, DOE-STD-3009-94 (Ref. 1-6) prescribed the use of a graded approach for the effort expended in safety analysis and the level of detail presented in associated documentation. DOE NPH mitigation requirements are also consistent with the National Earthquake Hazards Reduction Program and Executive Orders 12699 (Ref. 1-7) and 12941 (Ref. 1-8).

The overall approach for NPH mitigation shall be consistent with the graded approach embodied in the SAR. The application of NPH design requirements to structures, systems, and components (SSCs) shall be based on the life-safety or the safety classifications for the SSCs as established by safety analysis. The application of the most rigorous design requirements should be limited to those SSCs classified by safety analysis as Safety-Class or Safety-Significant consistent with DOE-STD-3009-94. Although DOE-STD-3009-94 is specifically

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applicable to non-reactor nuclear facilities, it is DOE's intention to apply DOE-STD-3009-94 definitions for "Safety-Class" and "Safety-Significant" to all nuclear reactor and other hazardous facilities, and this broader approach is applied here. Mission importance and economic considerations should also be used to categorize SSCs which require NPH design. Once the SSCs have been classified, DOE Order 420.1 and the associated Implementation Guides specifies the NPH requirements to ensure that the SSCs are adequately designed to resist NPH. The NPH requirements utilize a graded approach in order to provide a reasonable level of NPH protection for the wide variety of DOE facilities. A graded approach is one in which various levels of NPH design, evaluation and construction requirements of varying conservatism and rigor are established ranging from common practice for conventional facilities to practices used for more hazardous critical facilities.

Five DOE Standards have been developed to provide specific acceptance criteria for various aspects of NPH to meet the requirements of DOE Order 420.1 and the associated Implementation Guides. These requirements should be used in conjunction with the NPH Implementation Guide and other pertinent documents which provide more detailed methods on specific NPH design and evaluation subjects such as DOE guidance documents, consensus national standards, model building codes, and industry accepted codes and specifications. Figure 1-1 presents a conceptual NPH design framework which identifies how the DOE NPH standards are used to assess NPH design requirements.

The following national consensus codes and standards have been referred to in this standard:

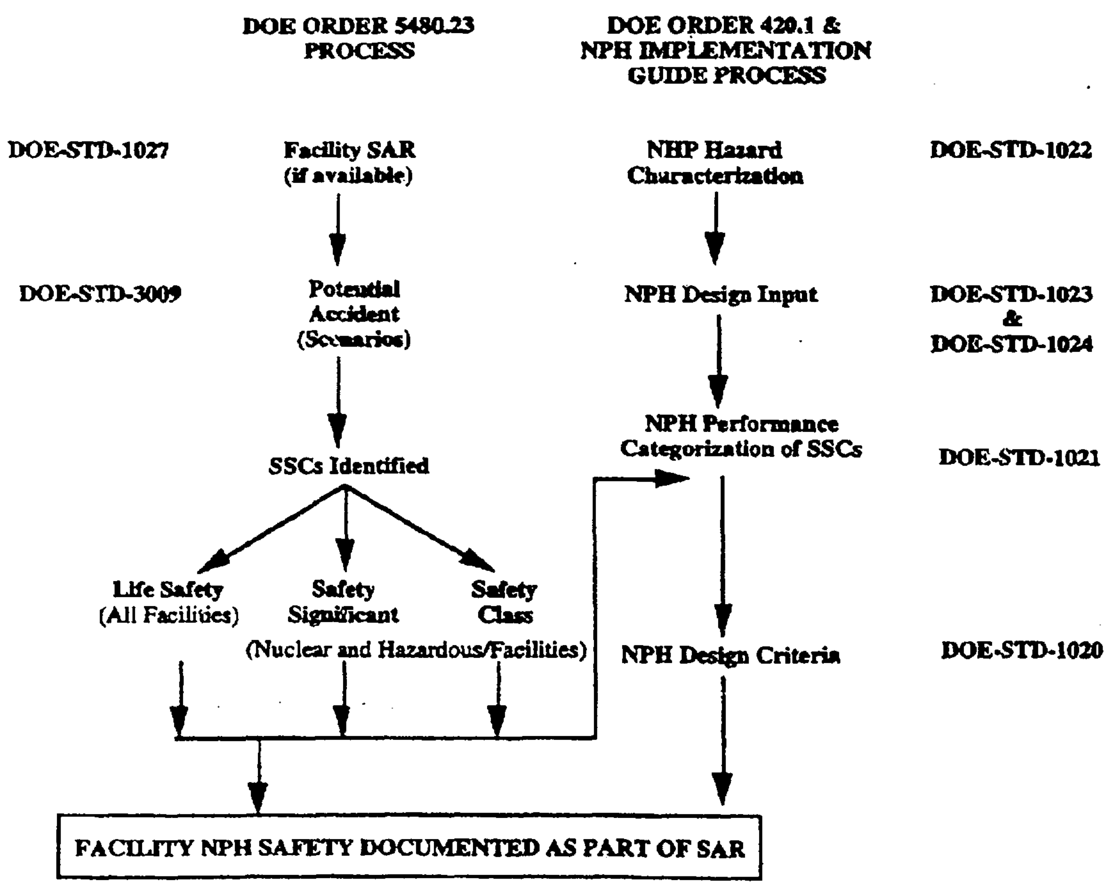
ACI 318	—	Building Code Requirements for Reinforced Concrete
ACI 349	—	Code Requirements for Nuclear Safety-Related Concrete Structures
AISC N690	—	Nuclear Facilities - Steel Safety Related Structures for Design, Fabrication, and Erection
AISC (LRFD)	—	Manual of Steel Construction, Load & Resistance Factor Design
AISC (ASD)	—	Manual of Steel Construction, Allowable Stress Design
ASCE 4	—	Seismic Analysis of Safety-Related Nuclear Structures
ASCE 7	—	Minimum Design Loads for Buildings and Other Structures
ASME	—	Boiler and Pressure Vessel Code
ATC-14	—	Evaluating the Seismic Resistance of Existing Buildings
ATC-22	—	A Handbook for Seismic Evaluation of Existing Buildings
IEEE 344	—	IEEE Recommended Practice for Seismic Qualification of Class IE Equipment for Nuclear Power Generating Stations
UBC	—	Uniform Building Code

NBC	—	National Building Code
SBC	—	Standard Building Code
FEMA 222A	—	NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings
ICSSC RP3	—	Guidelines for Identification and Mitigation of Seismically Hazardous of Existing Federal Buildings
ICSSC RP4	—	Standards of Seismic Safety for Existing Federally Owned or Leased Buildings
ICSSC RP5	—	ICSSC Guidance on Implementing Executive Order 12941 on Seismic Safety of Existing Federally Owned or Leased Buildings

Figure 1-1

Natural Phenomena Design Input

Conceptual Framework



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The NPH Implementation Guide of DOE Order 420.1 has established Performance Categories and target probabilistic performance goals for each category. Performance goals are expressed as the mean annual probability of exceedance of acceptable behavior limits of structures and equipment due to the effects of natural phenomena. Five Performance Categories (PC) have been established in the NPH Implementation Guide of DOE Order 420.1. Performance Categories and performance goals range from those for conventional buildings to those for facilities with hazardous materials for operations. The selection of NPH Performance Categories for SSCs is dependent on several factors including the overall risk of facility operation and the assigned function to the SSC. An SSC's safety classification is based on its function in accident prevention or mitigation as determined by safety analysis. The safety classification should be applied to specific SSCs on a case-by-case basis and need not apply to an entire facility. Experience to date has demonstrated that only a few nuclear facilities are likely to contain Safety-Class SSCs. This indicates that most SSCs in nuclear facilities should be assigned to NPH Performance Category 3 and lower. DOE is revisiting the approach used to assign NPH Performance Categories, and is likely to develop a direct link between NPH Performance Categories and accident dose (radiological or toxicological) criteria. Once this is completed, DOE-STD-1021 will be revised as necessary. The use of NPH Performance Category 4 should be reserved for those facilities whose accident dose potential is similar to that of commercial nuclear reactors.

1.2 Overview of the NPH Design and Evaluation Criteria

This natural phenomena hazard standard (DOE-STD-1020), developed from UCRL-15910 (Ref. 1-9), provides criteria for design of new structures, systems, and components (SSCs) and for evaluation, modification, or upgrade of existing SSCs so that Department of Energy (DOE) facilities safely withstand the effects of natural phenomena hazards (NPHs) such as earthquakes, extreme winds, and flooding. DOE-STD-1020 provides consistent criteria for all DOE sites across the United States. These criteria are provided as the means of implementing DOE Order 420.1 and the associated Implementation Guides, and Executive Orders 12699 and 12941 for earthquakes.

The design and evaluation criteria presented in this document provide relatively straightforward procedures to evaluate, modify, or upgrade existing facilities or to design new facilities for the effects of NPHs. The intent is to control the level of conservatism in the design/evaluation process such that: (1) the hazards are treated consistently; and (2) the level of conservatism is appropriate for structure, system, and component (SSC) characteristics related to safety, environmental protection, importance, and cost. The requirements for each hazard are presented in subsequent chapters. Terminology, guidelines, and commentary material are included in appendices which follow the requirement chapters.

Prior to applying these criteria, SSCs will have been placed in one of five Performance Categories ranging from PC-0 to PC-4. No special considerations for NPH are needed for PC-0; therefore, no guidance is provided. Different criteria are provided for the remaining four Performance Categories, each with a specified performance goal. Design and evaluation criteria aimed at target probabilistic performance goals require probabilistic natural phenomena hazard assessments. NPH loads are developed from such assessments by specifying natural phenomena hazard mean annual probabilities of exceedance. Performance goals may then be achieved by using the resulting loads combined with deterministic design and evaluation procedures that provide a consistent and appropriate level of conservatism. Design/Evaluation procedures conform closely to industry practices using national consensus codes and standards so that the procedures will be easily understood by most engineers. Structures, systems, and components comprising a DOE facility are to be assigned to a Performance Category utilizing the approach described in the DOE performance categorization standard (Ref. 1-10). These design and evaluation criteria (DOE-STD-1020) are the specific provisions to be followed such that the performance goal associated with the Performance Category of the SSC under consideration is achieved. For each category, the criteria include the following steps:

1. NPH loads are determined at specified NPH probabilities as per DOE-STD-1023 (Ref. 1-11).
2. Design and evaluation procedures are used to evaluate SSC response to NPH loads.
3. Criteria are used to assess whether or not computed response in combination with other design loads is permissible.
4. Design detailing provisions are implemented so that the expected performance during a potential NPH occurrence will be achieved.
5. Quality assurance and peer review are applied using a graded approach.

For each Performance Category, target performance goals are provided in the NPH Implementation Guide of DOE Order 420.1 in terms of mean annual probability of exceedance of acceptable behavior limits. In Item 1, the annual probability of exceedance of an NPH parameter such as ground acceleration, wind speed, or water elevation is specified. The level of conservatism in Items 2, 3, 4, and 5 above is controlled such that sufficient risk reduction from the specified NPH probability is achieved so that the target performance goal probability is met. DOE-STD-1020 provides an integrated approach combining definition of loading due to natural phenomena hazards, response evaluation methods, acceptance criteria, and design detailing requirements.

Performance goals and NPH levels are expressed in probabilistic terms; design and evaluation procedures are presented deterministically. Design/evaluation procedures specified in this document conform closely to common standard practices so that most engineers will readily understand them. The intended audience for these criteria is the civil/structural or mechanical engineer conducting the design or evaluation of facilities. These NPH design and evaluation criteria do not preclude the use of probabilistic or alternative design or evaluation approaches if these approaches meet the specified performance goals.

1.3 Evaluation of Existing Facilities

Evaluations of existing SSCs must follow or, at least, be measured against the NPH criteria provided in this document. For SSCs not meeting these criteria and which cannot be easily remedied, budgets and schedule for required strengthening must be established on a prioritized basis. A back-fit analysis should be conducted. Prioritization criteria for evaluation and upgrade of existing DOE facilities are currently being developed. Priorities should be established on the basis of Performance Category, cost of strengthening, and margin between as-is SSC capacity and the capacity required by the criteria. For SSCs which are close to meeting criteria, it is probably not cost effective to strengthen the SSC in order to obtain a small reduction in risk. As a result, some relief in the criteria is allowed for evaluation of existing SSCs. It is permissible to perform such evaluations using natural phenomena hazard exceedance probability of twice the value specified for new design. For example, if the natural phenomena hazard annual probability of exceedance for the SSC under consideration was 10^{-4}, it would be acceptable to reconsider the SSC at hazard annual probability of exceedance of 2×10^{-4}. This would have the effect of slightly reducing the seismic, wind, and flood loads in the SSC evaluation by about 10% to 20%. This amount of relief is within the tolerance of meeting the target performance goals and is only a minor adjustment of the corresponding NPH design and evaluation criteria. In addition, it is consistent with the intent of the Federal Program (Ref. 1-8) being developed by the Interagency Committee on Seismic Safety in Construction. The Implementation Guide provides guidance for facilities with a remaining service life of less than 5 years.

1.4 Quality Assurance and Peer Review

All DOE structures, systems, and components must be designed or evaluated utilizing a formal quality assurance plan as required by 10 CFR 830.120 (Ref. 1-12). The QA and peer review should be conducted within the framework of a graded approach with increasing level of rigor employed from Performance Category 1 to 4. Specific details about a formal quality assurance plan for NPH design and evaluation should be similar to the seismic plan described in the Commentary, Appendix C. The major features of a thorough quality assurance plan for design or evaluation for natural phenomena hazards are described below.

In general, it is good practice for a formal quality assurance plan to include the following requirements. On the design drawings or evaluation calculations, the engineer must describe the NPH design basis including (1) description of the system resisting NPH effects and (2) definition of the NPH loading used for the design or evaluation. Design or evaluation calculations should be checked for numerical accuracy and for theory and assumptions. For new construction, the engineer should specify a program to test materials and inspect construction. In addition, the engineer should review all testing and inspection reports and visit the site periodically to observe compliance with plans and specifications.

For Performance Categories 2, 3, and 4, NPH design or evaluation must include independent peer review. The peer review is to be performed by independent, qualified personnel. The peer reviewer must not have been involved in the original design or evaluation. If the peer reviewer is from the same company/organization as the designer/evaluator, he must not be part of the same program where he could be influenced by cost and schedule consideration. Individuals performing peer reviews must be degreed civil/mechanical engineers with 5 or more years of experience in NPH evaluation.

For more information concerning the implementation of a formal engineering quality assurance program and peer review, Chapter 19 of Reference 1-9 should be consulted. This reference should also be consulted for information on a construction quality assurance program consistent with the implementation of the engineering quality assurance program.

1.5 References

1-1. U. S. Department of Energy, **Facility Safety**, DOE Order 420.1, Washington, DC, October 13, 1995.

1-2. U. S. Department of Energy, **Implementation Guide for the Mitigation of Natural Phenomena Hazards for DOE Nuclear Facilities and Non-Nuclear Facilities** (draft for interim use), Washington, DC, November 13, 1995.

1-3. U. S. Department of Energy, **Implementation Guide for Nonreactor Nuclear Safety Design Criteria and Explosives Safety Criteria** (draft for interim use), Washington, DC, November 13, 1995.

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Chapter 2
Earthquake Design and Evaluation Criteria

2.1 Introduction

This chapter describes requirements for the design or evaluation of all classes of structures, systems, and components (SSCs) comprising DOE facilities for earthquake ground shaking. These classes of SSCs include safety class and safety significant SSCs per DOE-STD-3009-94 (ref. 1-6) and life-safety SSCs per Uniformed Building Codes. This material deals with how to establish Design/Evaluation Basis Earthquake (DBE) loads on various classes of SSCs; how to evaluate the response of SSCs to these loads; and how to determine whether that response is acceptable. This chapter also covers the importance of design details and quality assurance to earthquake safety. These earthquake design and evaluation provisions are equally applicable to buildings and to items contained within the building, such as equipment and distribution systems. These provisions are intended to cover all classes of SSCs for both new construction and existing facilities. These design and evaluation criteria have been developed such that the target performance goals of the NPH Implementation Guide are achieved. For more explanation see the Commentary (Appendix C) herein and the Basis Document (Ref. 2-1).

2.2 General Approach for Seismic Design and Evaluation

This section presents the approach upon which the specific seismic force and story drift provisions for seismic design and evaluation of structures, systems, and components in each Performance Category (as described in Section 2.3) is based. These provisions include the following steps:

1. Selection of earthquake loading
2. Evaluation of earthquake response
3. Specification of seismic capacity and drift limits, (acceptance criteria)
4. Ductile detailing requirements

It is important to note that the above four elements taken together comprise seismic design and evaluation criteria. Acceptable performance (i.e., achieving performance goals) can only be reached by consistent specification of all design criteria elements as shown in Figure 2-1. In order to achieve the target performance goals, these seismic design and evaluation criteria specify seismic loading in probabilistic terms. The remaining elements of the criteria (see Fig. 2-1) are deterministic design rules which are familiar to design engineers and

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which have a controlled level of conservatism. This level of conservatism combined with the specification of seismic loading, leads to performance goal achievement.

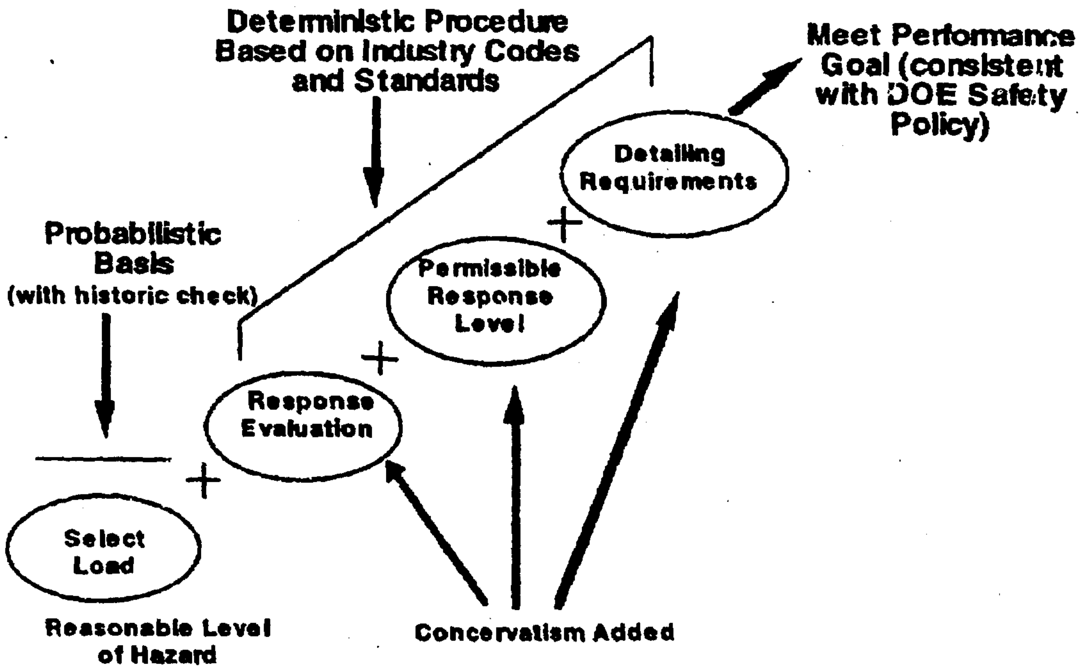

Figure 2-1. DOE-STD-1020 Combines Various Steps to Achieve Performance Goals

Criteria are provided for each of the four Performance Categories 1 to 4 as defined in the NPH Implementation Guide of DOE Order 420.1 and DOE-STD-1021 (Ref. 1-6). The criteria for Performance Categories 1 and 2 are similar to those from model building codes, with the exception that DOE requirements specity a 1000 year return period in the case of PC-2. Criteria for PC-3 are similar to those for Department of Defense Essential Facilities (Ref. C-5) Tri-Services Manual. Criteria for PC-4 approach the provisions for commercial nuclear power plants.

Seismic loading is defined in terms of a site-specified design response spectrum (the Design/Evaluation Basis Earthquake, [DBE]). Either a site-specific design response spectrum specifically developed for the site, or a generic design response spectrum that is appropriate or conservative for the site may be used. Seismic hazard estimates are used to establish the DBE per DOE-STD-1023 (REF. 2-22).

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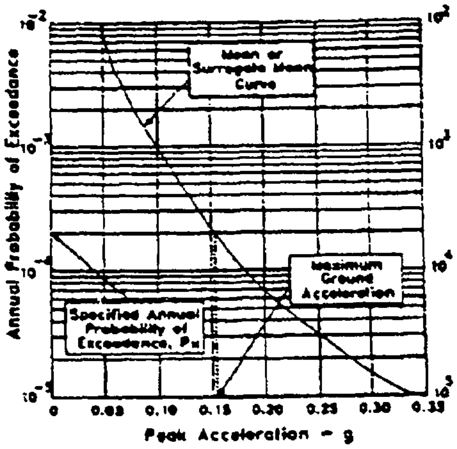
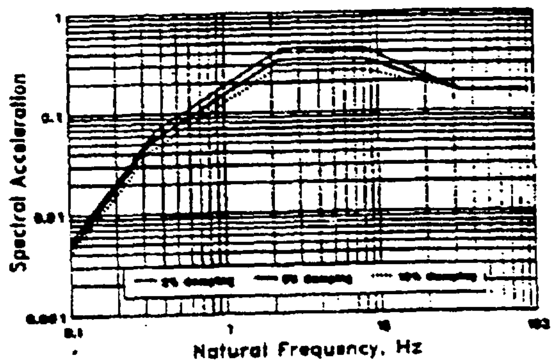
For each Performance Category, a mean annual exceedance probability for the DBE, P_H is specified from which the maximum ground acceleration (and/or velocity) may be determined from probabilistic seismic hazard curves, see Table 2-1. Evaluating maximum ground acceleration from a specified mean annual probability of exceedance is illustrated in Figure 2-2a. Earthquake input excitation to be used for design and evaluation by these provisions is defined by a median amplification smoothed and broadened design/evaluation response spectrum shape such as that shown in Figure 2-2b (from Ref. 2-2) anchored to the maximum ground acceleration and/or velocity. Such spectra are determined in accordance with DOE-STD-1023 (Ref. 2-22).

It should be understood that the spectra shown in Figure 2-2 or in-structure spectra developed from them represent inertial effects. They do not include differential support motions, typically called seismic anchor motion (SAM), of structures, equipment, or distribution systems supported at two or more points. While SAM is not usually applicable to building design, it might have a significant effect on seismic adequacy of equipment or distribution systems.





a) Evaluating Peak Acceleration from Annual Probability of Exceedance with a Seismic Hazard Curve

b) Median Amplification, smoothed and broadened, Design/Evaluation Response Spectra

Figure 2-2. Earthquake Input Excitation Is Defined by Maximum Ground Acceleration Anchoring Site-Specific Response Spectra

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Table 2-1 Seismic Performance Categories and Seismic Hazard Exceedance Levels

Performance Category	Mean Seismic Hazard Exceedance Levels, P_H	Return Period
0	No Requirements	
1	2×10^{-3}	500yr
2	1×10^{-3}	1000yr
3	5×10^{-4} (1×10^{-3})[1]	2000yr (1000yr)[1]
4	1×10^{-4} (2×10^{-4})[1]	10,000yr (5000yr)[1]

[1] For sites such as LLNL, SNL-Livermore, SLAC, LBL, and ETEC, which are near tectonic plate boundaries.

Performance Category 2 and lower SSCs may be seismically designed or evaluated using the approaches specified in building code seismic provisions. However, for Performance Category 3 or higher, the seismic evaluation must be performed by a dynamic analysis approach. A dynamic analysis approach requires that:

1. The input to the SSC model be defined by either a design response spectrum, or a compatible time history input motion.
2. The important natural frequencies of the SSC be estimated, or the peak of the design response spectrum be used as input. Multi-mode effects must be considered.
3. The resulting seismic induced inertial forces be appropriately distributed and a load path evaluation (see Section C.4.2) for structural adequacy be performed.

The words "dynamic analysis approach" are not meant to imply that complex dynamic models must be used in the evaluation. Often equivalent static analysis models are sufficient if the above listed three factors are incorporated. However, use of such simplified models for structures in Performance Category 3 or higher must be justified and approved by DOE. This dynamic analysis approach should comply with the seismic response analysis provisions of ASCE 4 (Ref. 2-3) except where specific exceptions are noted.

The maximum ground acceleration and ground response spectra determined in the manner illustrated in Figure 2-2 are used in the appropriate terms of the UBC equation for base shear. The maximum ground acceleration is also used in the UBC equation for seismic force on equipment and non-structural components. Use of modern site-specific earthquake ground motion data is considered to be preferable to the general seismic zonation maps from the UBC and should be applied according to the guidance provided in DOE-STD-1023 (Ref. 2-22). For structures, UBC provisions require a static or dynamic analysis approach in which loadings are scaled to the base shear equation value. In the base shear equation, inelastic energy absorption capacity of structures is accounted for by the parameter, R_W. Elastically computed seismic response is reduced by R_W values ranging from 4 to 12 as a means of accounting for inelastic energy absorption capability in the UBC provisions and by these criteria for Performance Category 2 and lower SSCs. This reduced seismic response is combined with non-seismic concurrent loads and then compared to code allowable response limits (or code ultimate limits combined with code specified load factors). The design detailing provisions from the UBC, which provide ductility, toughness, and redundancy, are also required such that SSCs can fully achieve potential inelastic energy absorption capability. Normally, relative seismic anchor motion (SAM) is not considered explicitly by model building code seismic provisions. However, SAM should be considered for SSCs in PC-2 or higher categories.

The Uniform Building Code (UBC) has been followed for Performance Categories 1 and 2 because it is believed that more engineers are familiar with this code than other model building codes. The Interagency Committee on Seismic Safety in Construction (ICSSC, Ref. 2-4) has concluded that the following seismic provisions are equivalent for a given DBE:

1. 1994 Uniform Building Code (Ref. 2-5)
2. 1991 NEHRP Recommended Provisions (Ref. 2-6)
3. 1993 BOCA National Building Code (Ref. 2-7)
4. 1994 SBCCI Standard Building Code (Ref. 2-8)

These other model building codes may be followed provided site-specific ground motion data is incorporated into the development of earthquake loading in a manner similar to that described in this document for the UBC.

For Performance Category 3 and 4 SSCs, these seismic design and evaluation criteria specify that seismic evaluation be accomplished by dynamic analysis. The recommended

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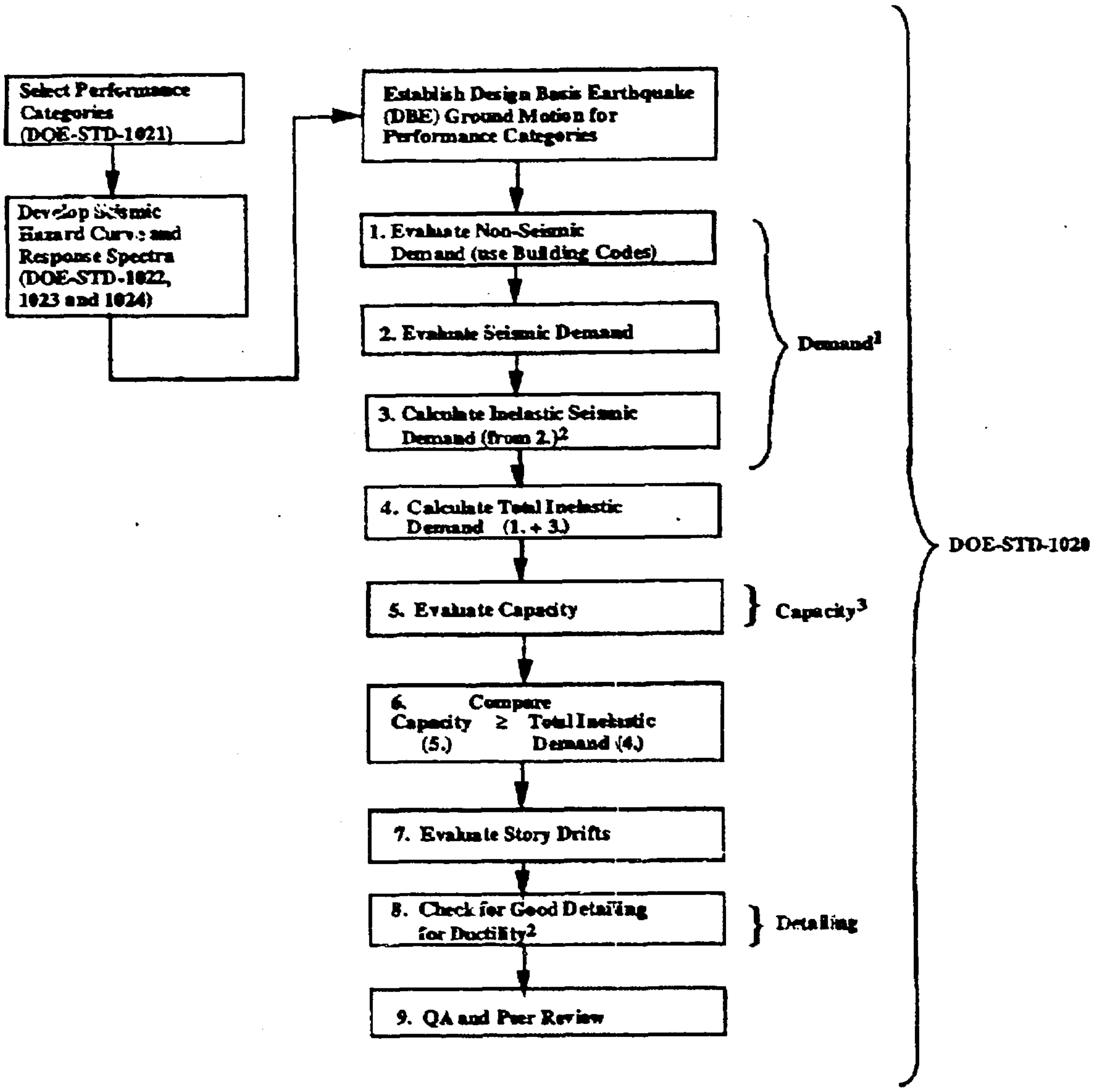
approach is to perform an elastic response spectrum dynamic analysis to evaluate elastic seismic demand on SSCs. Inelastic energy absorption capability is allowed by permitting limited inelastic behavior. By these provisions, inelastic energy absorption capacity of structures is accounted for by the parameter, F_μ. However, strength and ductile detailing for the entire load path should be assured. Elastically computed seismic response is reduced by F_μ values ranging from 1 to 3 as a means of accounting for inelastic energy absorption capability. The same F_μ values are specified for both Performance Categories of 3 and 4. In order to achieve the conservatism appropriate for the different Performance Categories, the reduced seismic forces are multiplied by a scale factor. Scale factors are specified for Performance Category 3 and 4. The resulting factored seismic forces are combined with non-seismic concurrent loads and then compared to code ultimate response limits. The design detailing provisions from the UBC, which provide ductility, toughness, and redundancy, are also required such that SSCs can fully achieve potential inelastic energy absorption capability. Also, explicit consideration of relative seismic anchor motion (SAM) effects is required for Performance Category 3 and higher.

The overall DOE Seismic Design and Evaluation Procedure is shown in Figure 2-3. In addition to the general provisions described in this chapter, the topics discussed in Appendix C should be considered before commencing design or evaluation.

2.3 Seismic Design and Evaluation of Structures, Systems, and Components

- Select Performance Categories of structure, system, or component based on DOE-STD-1021 (Ref. 1-10).
- For sites with Performance Category 3 or 4 structures, systems, and components, obtain or develop a seismic hazard curve and design response spectra in accordance with DOE-STD-1023 (Ref. 2-22) for all performance categories based on site characterization discussed in DOE-STD-1022 (Ref. 1-15). In the interim, Eastern U.S. sites may use DOE-STD-1024. (Ref. 2-23)
- Establish design basis earthquake from P_H, (see Table 2-1) mean seismic hazard curve, and median response spectra.

  For sites with only PC-1 or 2 SSC, and no site-specific seismic hazard curve, obtain seismic coefficients from model building codes.



1. See Section C.4 for further discussion.
2. For evaluation of existing facilities, the strength and detailing of the entire load path must be checked prior to assignment of ductility reduction factors.
3. See Section C.5 for further discussion.

Figure 2-3. DOE Seismic Design and Evaluation Procedure

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Minimum values of peak ground acceleration (PGA) shall be:

0.06g for Performance Category 3
0.10g for Performance Category 4

2.3.1 Performance Category 1 and 2 Structures, Systems, and Components.

Seismic design or evaluation of Performance Category 2 and lower SSCs is based on model building code seismic provisions. In these criteria, the current version of the Uniform Building Code shall be followed. Alternatively, the other equivalent model building codes may be used. All UBC seismic provisions shall be followed for Performance Category 2 and lower SSCs (with modifications as described below).

In the UBC provisions, beginning with the 1988 edition, the lateral force representing the earthquake loading on buildings is expressed in terms of the total base shear, V, given by the following equation:

$$V = \frac{ZICW}{R_w} \qquad (2\text{-}1)$$

where:

	Z	=	a seismic zone factor equivalent to peak ground acceleration,
	I	=	a factor accounting for the importance of the facility,
	C	=	a spectral amplification factor,
	W	=	the total weight of the facility,
	R_W	=	a reduction factor to account for energy absorption capability of the facility which results in element forces which represent inelastic seismic demand, D_{SI}

The steps in the procedure for PC-1 and 2 SSCs are as follows:

- Evaluate element forces for non-seismic loads, D_{NS}, expected to be acting concurrently with an earthquake.
- Evaluate element forces, D_{SI}, for earthquake loads.

a. Static force method, where V is applied as a load distributed over the height of the structure for regular facilities, or dynamic force method for irregular facilities as described in the UBC.

b. In either case, the total base shear is given by Equation 2-1 where the parameters are evaluated as follows:

1. Z is the peak ground acceleration from site-specific seismic hazard curves at the following exceedance probabilities if available:

Performance Category 1 - $2x10^{-3}$
Category 2 - $1x10^{-3}$

Otherwise, Z is obtained using UBC and adjusted per the procedures provided in DOE-STD-1023.

2. C is the spectral amplification at the fundamental period of the facility from the 5 percent damped median site response spectra. For fundamental periods lower than the period at which the maximum spectral acceleration occurs, ZC should be taken as the maximum spectral acceleration. See Fig. 2-4 below:

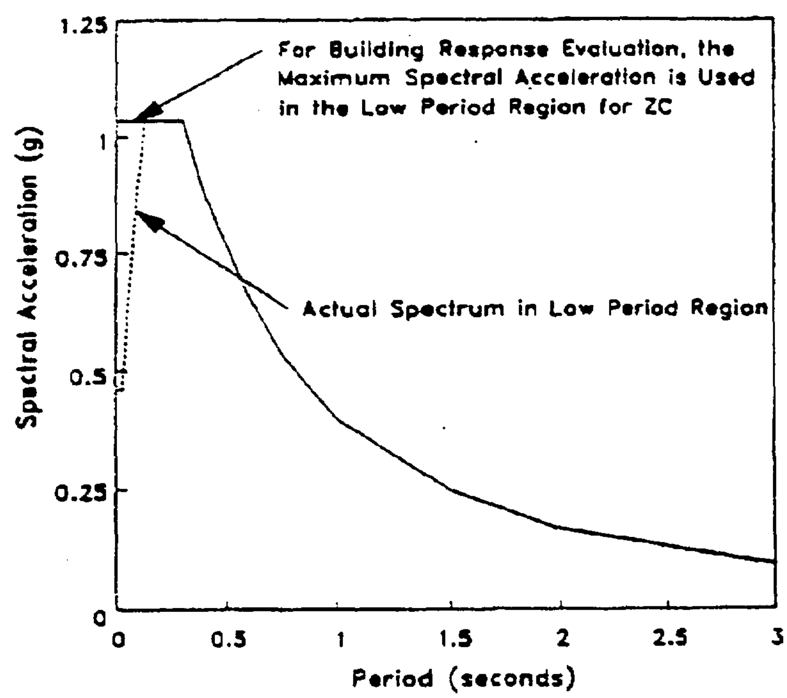

Figure 2-4. Example Design/Evaluation Earthquake Ground Motion Response Spectrum

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For systems and components, spectral amplification is accounted for by C_p in the UBC equipment force equation as discussed in Section 2.4.1.

3. If a recent site-specific seismic hazard assessment is not available, it is acceptable to determine ZC from Table C-5 values and appropriate response spectra. For eastern U.S. sites DOE -STD-1024 provides guidance. If ZC, determined from a recent site-specific assessment is less than that given by UBC provisions, any significant differences with UBC must be justified. Final earthquake loads are subject to approval by DOE.

4. Importance factor, I, should be taken as:

   Performance Category 1, I = 1.0
   Performance Category 2, I = 1.25

5. For structures, reduction factors, R_W, are shown in Table 2-2. For systems and components, the reduction factor is implicitly included in C_p.

- Combine responses from various loadings (D_{NS} and D_{SI}) to evaluate demand, D_{TI}, by code specified load combination rules (e.g., load factors for ultimate strength design or unit load factors for allowable stress design).
- Evaluate capacities of SSCs, C_C, from code ultimate values when strength design is used (e.g., UBC Chapter 19 for reinforced concrete or LRFD for steel) or from allowable stress levels (with one-third increase) when allowable stress design is used. Minimum specified or 95% non-exceedance in-situ values for material strengths should be used for capacity estimation.
- Compare demand, D_{TI}, with capacity, C_C, for all SSCs. If D_{TI} is less than or equal to C_C, the facility satisfies the seismic force requirements. If D_{TI} is greater than C_C, the facility has inadequate seismic resistance.

Table 2-2. Code Reduction Coefficients, R_W

Structural System (Terminology is identical to the UBC)	R_W
MOMENT RESISTING FRAME SYSTEMS - Beams	
Steel Special Moment Resisting Frame (SMRF)	12
Concrete SMRF	12
Concrete Intermediate Moment Frame (IMRF)	8
Steel Ordinary Moment Resting Frame	6
Concrete Ordinary Moment Resisting Frame	5
SHEAR WALLS	
Concrete or Masonry Walls	8(6)
Plywood Walls	9(8)
Dual System, Concrete with SMRF	12
Dual System, Concrete with Concrete IMRF	9
Dual System, Masonry with SMRF	8
Dual System, Masonry with Concrete IMRF	7
STEEL ECCENTRIC BRACED FRAMES (EBF)	
Beams and Diagonal Braces	10
Beams and Diagonal Braces, Dual System with Steel SMRF	12
CONCENTRIC BRACED FRAMES	
Steel Beams	8(6)
Steel Diagonal Braces	8(6)
Concrete Beams	8(4)
Concrete Diagonal Braces	8(4)
Wood Trusses	8(4)
Beams and Diagonal Braces, Dual Systems	
Steel with Steel SMRF	10
Concrete with Concrete SMRF	9
Concrete with Concrete IMRF	6

Note: Values herein assume good seismic detailing practice per the UBC along with reasonably uniform inelastic behavior. Otherwise lower values should be used.

Values in parentheses apply to bearing wall systems or systems in which bracing carries gravity loads.

- Evaluate story drifts (i.e., the displacement of one level of the structure relative to the level above or below due to the design seismic forces), including both translation and torsion. Calculated story drifts should not exceed $0.04/R_W$ times the story height nor 0.005 times the story height for buildings with a fundamental period less than 0.7 seconds. For more flexible buildings, the calculated story drift should not exceed $0.03/R_W$ nor 0.004 times the story height. Note that these story drifts are calculated from seismic loads reduced by R_W in accordance with Equation 2-1; actual drift can be estimated by multiplying calculated drifts by 3 ($R_W/8$). These drifts limits may be exceeded when it is demonstrated that greater drift can be tolerated by both structural systems and non-structural elements.

- Elements of the facility shall be checked to assure that all detailing requirements of the UBC provisions are met. The basic UBC seismic detailing provisions must be met if Z is 0.11g or less. UBC Seismic Zone No. 2 provisions shall be met when Z is between 0.12 and 0.24g. UBC Seismic Zone Nos. 3 & 4 provisions shall be followed when Z is 0.25g or more.
- A quality assurance program consistent with model building code requirements shall be implemented for SSCs in Performance Categories 1 and 2. In addition, peer review shall be conducted for Performance Category 2 SSCs.

2.3.2 Performance Category 3 and 4 Structures, Systems, and Components

The steps in the procedure for PC-3 and 4 SSCs are as follows:

- Evaluate element forces, D_{NS}, for the non-seismic loads expected to be acting concurrently with an earthquake.
- Calculate the elastic seismic response to the DBE, D_S, using a dynamic analysis approach and appropriate damping values from Table 2-3. Response Level 3 is to be used only for justifying the adequacy of existing SSCs with adequate ductile detailing. Note that for evaluation of systems and components supported by the structure, in-structure response spectra are used. For PC-3 and PC-4 SSCs, the dynamic analysis must consider 3 orthogonal components of earthquake ground motion (two horizontal and one vertical). Responses from the various direction components shall be combined in accordance with ASCE 4. Include, as appropriate, the contribution from seismic anchor motion. To determine response of SSCs which use $F\mu > 1$, note that for fundamental periods lower than the period at which the maximum spectral amplification occurs, the maximum spectral acceleration should be used. For higher modes, the actual spectral accelerations should be used.

- Calculate the inelastic seismic demand element forces, D_{SI}, as

$$D_{SI} = SF \frac{D_S}{F_\mu} \qquad (2\text{-}2)$$

where: F_μ = Inelastic energy absorption factor from Table 2-4 for the appropriate structural system and elements having adequate ductile detailing

SF = Scale factor related to Performance Category
= 1.25 for PC-4
= 1.0 for PC-3

Variable scale factors, based on the slope of site-specific hazard curves, may be used as discussed in Appendix C to result in improved achievement of performance goals. SF is applied for evaluation of structures, systems, and components. At this time, $F\mu$ values are not provided for systems and components. It is recognized that many systems and components exhibit ductile behavior for which $F\mu$ values greater than unity would be appropriate (see Section C.4.4.2). Low $F\mu$ values in Table 2-4 are intentionally specified to avoid brittle failure modes.

- Evaluate the total inelastic-factored demand D_{TI} as the sum of D_{SI} and D_{NS} (the best-estimate of all non-seismic demands expected to occur concurrently with the DBE).

$$D_{TI} = D_{NS} + D_{SI} \qquad (2\text{-}3)$$

- Evaluate capacities of elements, C_C, from code ultimate or yield values

    Reinforced Concrete

        Use UBC Chapter 19

    Steel

        Use UBC Chapter 22 Standards

        — LRFD provisions, or
        — Plastic Design provisions, or
        — Allowable Stress Design provision scaled by 1.4 for shear in members and bolts and 1.7 for all other stresses.

Refer to References 2-9 and 2-10 for related industry standards. Note that strength reduction factors, ϕ, are retained. Minimum specified or 95%

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nonexceedence in-situ values for material strengths should be used to estimate capacities.

- The seismic capacity is adequate when C_C exceeds D_{TI}, i.e.:

$$C_C \geq D_{TI} \qquad (2\text{-}4)$$

- Evaluate story drifts due to lateral forces, including both translation and torsion. It may be assumed that inelastic drifts are adequately approximated by elastic analyses (note that lateral seismic forces are not reduced by F_μ when computing story drifts). Calculated story drifts should not exceed 0.010 times the story height for structures with contribution to distortion from both shear and flexture. For structures in which shear distortion is the primary contributer to drift, such as those with low rise shear walls or concentric braced-frames, the calculated story drift should not exceed 0.004 times the story height. These drift limits may be exceeded when acceptable performance of both the structure and nonstructural elements can be demonstrated at greater drift.

- Check elements to assure that good detailing practice has been followed (e.g., see sect. C.4.4.2). Values of F_μ given in Table 2-4 are upper limit values assuming good design detailing practice and consistency with recent UBC provisions. Existing facilities may not be consistent with recent provisions, and, if not, must be assigned reduced F_μ. Basic UBC seismic detailing provisions shall be followed if the PGA at P_H is 0.11g or less. UBC Seismic Zone No. 2 provisions should be met when the PGA at P_H is between 0.12 and 0.24g. UBC Seismic Zone Nos. 3 & 4 provisions should be followed when the PGA at P_H is 0.25g or more.

- Implement peer review of engineering drawings and calculations (including proper applicaton of F_μ values), increased inspection and testing of new construction or existing facilities.

2.3.3 Damping Values for Performance Category 3 and 4 Structures, Systems, and Components

Damping values to be used in linear elastic analyses are presented in Table 2-3 at three different response levels as a function of D_T/C_C.

D_T is the elastically computed total demand,

$$D_T = D_{NS} + D_S \qquad (2\text{-}5)$$

and C_C is the code specified capacity.

When determining the input to subcomponents mounted on a supporting structure, the damping value to be used in elastic response analyses of the supporting structure shall be based on the response level reached in the majority of the seismic load resisting elements of the supporting structure. This may require a second analysis.

In lieu of a second analysis to determine the actual response of the structure, Response Level 1 damping values may be used for generation of in-structure spectra. Response Level 1 damping values must be used if stability considerations control the design.

When evaluating the structural adequacy of an existing SSC, Response Level 3 damping may be used in elastic response analyses independent of the state of response actually reached, because such damping is expected to be reached prior to structural failure.

When evaluating a new SSC, damping is limited to Response Level 2. For evaluating the structural adequacy of a new SSC, Response Level 2 damping may be used in elastic response analyses independent of the state of response actually reached.

The appropriate response level can be estimated from the following:

Response Level	D_T/C_C
3**	≥ 1.0
2*	≈ 0.5 to 1.0
1*	≤ 0.5

* Consideration of these damping levels is required only in the generation of floor or amplified response spectra to be used as input to subcomponents mounted on the supporting structure. For analysis of structures including soil-structure interaction effects (sec C.4.3), D_T/C_C ratios for the best estimate case shall be used to determine response level.

** Only to be used for justifying the adequacy of existing SSCs with adequate ductile detailing. However, functionality of SSCs in PC-3 and PC-4 must be given due consideration.

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Table 2-3 Specified Damping Values

	Damping (% of critical)		
Type of Component	Response Level 1	Response Level 2	Response Level 3
Welded and friction bolted metal structures	2	4	7
Bearing-bolted metal structures	4	7	10
Prestressed concrete structures (without complete loss of prestress)	2	5	7
Reinforced concrete structures	4	7	10
Masonry shear walls	4	7	12
Wood structures with nailed joints	5	10	15
Distribution systems***	3	5	5
Massive, low-stressed components (pumps, motors, etc.)	2	3	--*
Light welded instrument racks	2	3	--*
Electrical cabinets and other equipment	3	4	5**
Liquid containing metal tanks			
Impulsive mode	2	3	4
Sloshing mode	0.5	0.5	0.5

* Should not be stressed to Response Level 3. Use damping for Response Level 2.

** May be used for anchorage and structural failure modes which are accompanied by at least some inelastic response. Response Level 1 damping values should be used for functional failure modes such as relay chatter or relative displacement issues which may occur at a low cabinet stress level.

*** Cable trays more than one half full of loose cables may use 10% of critical damping.

Table 2-4 Inelastic Energy Absorption Factors, F_μ

Structural System (terminology is identical to Ref. 2-5)	F_μ
MOMENT RESISTING FRAME SYSTEMS - Beams	
Steel Special Moment Resisting Frame (SMRF)	3.0
Concrete SMRF	2.75
Concrete Intermediate Moment Frame (IMRF)	1.5
Steel Ordinary Moment Resting Frame	1.5
Concrete Ordinary Moment Resisting Frame	1.25
SHEAR WALLS	
Concrete or Masonry Walls	
In-plane Flexure	1.75
In-plane Shear	1.5
Out-of-plane Flexure	1.75
Out-of plane Shear	1.0
Plywood Walls	1.75
Dual System, Concrete with SMRF	2.5
Dual System, Concrete with Concrete IMRF	2.0
Dual System, Masonry with SMRF	1.5
Dual System, Masonry with Concrete IMRF	1.4
STEEL ECCENTRIC BRACED FRAMES (EBF)	
Beams and Diagonal Braces	2.75
Beams and Diagonal Braces, Dual System with Steel SMRF	3.0
CONCENTRIC BRACED FRAMES	
Steel Beams	2.0
Steel Diagonal Braces	1.75
Concrete Beams	1.75
Concrete Diagonal Braces	1.5
Wood Trusses	1.75
Beams and Diagonal Braces, Dual Systems	
Steel with Steel SMRF	2.75
Concrete with Concrete SMRF	2.0
Concrete with Concrete IMRF	1.4
METAL LIQUID STORAGE TANKS	
Moment and Shear Capacity	1.25
Hoop Capacity	1.5

Note: 1. Values herein assume good seismic detailing practice per Reference 2-5, along with reasonably uniform inelastic behavior. Otherwise, lower values should be used.

2. $F\mu$ for columns for all structural systems is 1.5 for flexure and 1.0 for axial compression and shear. For columns subjected to combined axial compression and bending, interaction formulas shall be used.

3. Connections for steel concentric braced frames should be designed for at least the lesser of:
   The tensile strength of the bracing.
   The force in the brace corresponding to $F\mu$ of unity.
   The maximum force that can be transferred to the brace by the structural system.

4. Connections for steel moment frames and eccentric braced frames and connections for concrete, masonry, and wood structural systems should follow Reference 2-5 provisions utilizing the prescribed seismic loads from these criteria and the strength of the connecting members. In general, connections should develop the strength of the connecting members or be designed for member forces corresponding to $F\mu$ of unity, whichever is less.

5. $F\mu$ for chevron, V, and K bracing is 1.5. K bracing requires special consideration for any building if Z is 0.25*g* or more.

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2.4 Additional Requirements

2.4.1 Equipment and Distribution Systems

For Performance Category 2 and lower systems and components, the design or evaluation of equipment or non-structural elements supported within a structure may be based on the total lateral seismic force, F_p, as given by the UBC provisions (Ref. 2-5). For Performance Category 3 and higher systems and components, seismic design or evaluation shall be based on dynamic analysis, testing, or past earthquake and testing experience data. In any case, equipment items and non-structural elements must be adequately anchored to their supports unless it can be shown by dynamic analysis or by other conservative analysis and/or test that the equipment will be able to perform all of its safety functions without interfering with the safety functions of adjacent equipment. Anchorage must be verified for adequate strength and sufficient stiffness.

Evaluation by Analysis

By the UBC provisions for PC-1 and 2, parts of the structures, permanent non-structural components, and equipment supported by a structure and their anchorages and required bracing must be designed to resist seismic forces. Such elements should be designed to resist a total lateral seismic force, F_p, of:

$$F_p = ZI_pC_pW_p \qquad (2\text{-}8)$$

where: W_p = the weight of element or component

C_p = a horizontal force factor as given by Table 16-O of the UBC for rigid elements, or determined from the dynamic properties of the element and supporting structure for nonrigid elements (in the absence of detailed analysis, the value of C_p for a nonrigid element should be taken as twice the value listed in Table 16-O, but need not exceed 2.0)

The lateral force determined using Equation 2-8 shall be distributed in proportion to the mass distribution of the element or component. Forces determined from Equation 2-8 shall be used for the design or evaluation of elements or components and their connections and anchorage to the structure, and for members and connections that transfer the forces to the seismic-resisting systems. Forces shall be applied in the horizontal direction that results in the most critical loadings for design/evaluation.

Note that DOE-STD-1020 takes one exception to the UBC provisions. By the UBC for equipment located above grade, the value Cp for non-rigid or flexibly supported items is twice the value for rigid and rigidly supported equipment. However, by the UBC for equipment located at or below grade, the value Cp for non-rigid or flexibly supported items is the same as the value for rigid and rigidly supported equipment. By DOE-STD-1020 for equipment located at or below grade, the value Cp for non-rigid or flexibly supported items (except for piping, ducting or conduit systems made of ductile materials and connections) is specified to be twice the value for rigid and rigidly supported equipment. An alternative methodology is contained in the 1994 NEHRP Provisions (Ref. 2-24) which accounts for the dynamic properties of the equipment, the location of the equipment within the primary structure, and the response of the primary supporting structure.

For PC-3 and PC-4 subsystems and components, support excitation shall be represented by means of floor response spectra (also commonly called in-structure response spectra). Floor response spectra should be developed accounting for the expected response level of the supporting structure even though inelastic behavior is permitted in the design of the structure (see Section 2.3.3). It is important to account for uncertainty in the properties of the equipment, supporting structure, and supporting media when using in-structure spectra which typically have narrow peaks. For this purpose, the peak broadening or peak shifting techniques outlined in ASCE 4 shall be employed.

Equipment or distribution systems that are supported at multiple locations throughout a structure could have different floor spectra for each support point. In such a case, it is acceptable to use a single envelope spectrum of all locations as the input to all supports to obtain the inertial loads. Alternatively, there are analytical techniques available for using different spectra at each support location or for using different input time histories at each different support.

Seismic Anchor Motion

The seismic anchor motion (SAM) component for seismic response is usually obtained by conventional static analysis procedures. The resultant component of stress can be very significant if the relative motions of the support points are quite different. If all supports of a structural system supported at two or more points have identical excitation, then this component of seismic response does not exist. For multiply-supported components with different seismic inputs, support displacements can be obtained either from the structural response calculations of the supporting structure or from spectral displacement determined from the floor response

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spectra. The effect of relative seismic anchor displacements shall be obtained by using the worst combination of peak displacements or by proper representation of the relative phasing characteristics associated with different support inputs. In performing an analysis of systems with multiple supports, the response from the inertial loads shall be combined with the responses obtained from the seismic anchor displacement analysis of the system by the SRSS rule $\left[R=\sqrt{(R_{inertia})^2+(R_{SAM})^2}\right]$, where R = response parameter of interest.

Evaluation by Testing

Guidance for conducting testing is contained in IEEE 344 (Ref. 2-11). Input or demand excitation for the tested equipment shall be based on the seismic hazard curves at the specified annual probability for the Performance Category of the equipment (OBE provisions of Ref. 2-11 do not apply). When equipment is qualified by shake table testing, the DBE input to the equipment is defined by an elastic computed required-response-spectrum (RRS) obtained by enveloping and smoothing (filling in valleys) the in-structure spectra computed at the support of the equipment by linear elastic analyses. In order to meet the target performance goals established for the equipment, the Required Response Spectrum (RRS) must exceed the In-Structure Spectra by:

$$
\begin{array}{lll}
RRS \geq (1.1)(\text{In-Structure Spectra}) & \text{for PC-2 and lower} & \\
RRS \geq (1.4SF)(\text{In-structure Spectra}) & \text{for PC-3 and higher} & (2\text{-}6)
\end{array}
$$

where SF is the seismic scale factor from Equation 2-2.

The Test Response Spectrum (TRS) of test table motions must envelop the RRS. If equipment has been tested and shown to meet NRC requirements, then it need not be subjected to further testing.

Evaluation by Seismic Experience Data

For new design of systems and components, seismic qualification will generally be performed by analysis or testing as discussed in the previous sections. However, for existing systems and components, it is anticipated that many items will be judged adequate for seismic loadings on the basis of seismic experience data without analysis or testing. Seismic experience data has been developed in a usable format by ongoing research programs sponsored by the nuclear power industry. The references for this work are the Senior Seismic Review and Advisory Panel (SSRAP) report (Ref. 2-12) and the Generic Implementation Procedure (GIP) for Seismic Verification of Nuclear Plant Equipment (Ref. 2-13). Note that there are numerous restrictions ("caveats") on the use of this data as described in the SSRAP report and the GIP. It is necessary to conduct either seismic analyses or shake table testing to demonstrate sufficient seismic capacity for those items that cannot be verified by seismic experience data or for items that are not obviously inherently rugged for seismic effects. There is an ongoing DOE program on the application of experience data for the evaluation of existing systems and components at DOE facilities. Currently, use of experience data is permitted for existing facilities and for the items specified in the two references, (Ref. 2-12) and (Ref. 2-13).

Anchorage and Supports

Adequate strength of equipment anchorage requires consideration of tension, shear, and shear-tension interaction load conditions. The strength of cast-in-place anchor bolts and undercut type expansion anchors shall be based on UBC Chapter 19 provisions (Ref. 2-5) for Performance Category 2 and lower SSCs and on ACI 349 provisions (Ref. 2-14) for Performance Category 3 and higher SSCs. For new design by AC1 349 provisions, it is required that the concrete pullout failure capacity be greater than the steel cast-in-place bolt tensile strength to assure ductile behavior. For evaluation of existing cast-in-place anchor bolt size and embedment depth, it is sufficient to demonstrate that the concrete pullout failure capacity is greater than 1.5 times the seismic induced tensile load. For existing facility evaluation, it may be possible to use relaxed tensile-shear interaction relations provided detailed inspection and evaluation of the anchor bolt in accordance with Reference 2-15 is performed.

The strength of expansion anchor bolts should generally be based on design allowable strength values available from standard manufacturers' recommendations or sources such as site-specific tests or Reference 2-15. Design-allowable strength values typically include a factor of safety of about 4 on the mean ultimate capacity of the anchorage. It is permissible to utilize strength values based on a lower factor of safety for evaluation of anchorage in existing facilities, provided the detailed inspection and evaluation of anchors is performed in accordance with Reference 2-15. A factor of safety of 3 is appropriate for this situation. When anchorage is modified or new anchorage is designed, design-allowable strength values including the factor of safety of 4 shall be used. For strength considerations of welded anchorage, AISC allowable values (Ref. 2-10) multiplied by 1.7 shall be used. Where shear in the member governs the connection strength, capacity shall be determined by multiplying the AISC allowable shear stress by 1.4.

Stiffness of equipment anchorage shall also be considered. Flexibility of base anchorage can be caused by the bending of anchorage components or equipment sheet metal.

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Excessive eccentricities in the load path between the equipment item and the anchor is a major cause of base anchorage flexibility. Equipment base flexibility can allow excessive equipment movement and reduce its natural frequency, possibly increasing dynamic response. In addition, flexibility can lead to high stresses in anchorage components and failure of the anchorage or equipment sheet metal.

2.4.2 Evaluation of Existing Facilities

It is anticipated that these criteria would also be applied to evaluations of existing facilities. General guidelines for the seismic evaluation of existing facilities are presented in National Institute of Standards and Technology documents (Refs. 2-16 and 2-17), a DOD manual (Ref. 2-18), and in ATC-14, "Evaluating the Seismic Resistance of Existing Buildings" (Ref. 2-19) and ATC-22, "A Handbook for Seismic Evaluation of Existing Buildings" (Ref. 2-22). In addition, guidelines for upgrading and strengthening equipment are presented in Reference 2-23. Also, guidance for evaluation of existing equipment by experience data is provided in Reference 2-13. These documents should be referred to for the overall procedure of evaluating seismic adequacy of existing facilities, as well as for specific guidelines on upgrading and retrofitting.

Once the as-is condition of a facility has been verified and deficiencies or weak links have been identified, detailed seismic evaluation and/or upgrading of the facility as necessary can be undertaken. Obvious deficiencies that can be readily improved should be remedied as soon as possible. Seismic evaluation for existing facilities would be similar to evaluations performed for new designs except that a single as-is configuration is evaluated instead of several configurations in an iterative manner (as is often required in the design process). Evaluations should be conducted in order of priority. Highest priority should be given to those areas identified as weak links by the preliminary investigation and to areas that are most important to personnel safety and operations with hazardous materials. Input from safety personnel and/or accident analyses should be used as an aid in determining safety priorities.

The evaluation of existing facilities for natural phenomena hazards can result in a number of options based on the evaluation results. If the existing facility can be shown to meet the design and evaluation criteria presented in Sections 2.3.1 or 2.3.2 and good seismic design practice had been employed, then the facility would be judged to be adequate for potential seismic hazards to which it might be subjected. If the facility does not meet the seismic evaluation criteria of this chapter, a back-fit analysis should be conducted. Several alternatives can be considered:

1. If an existing SSC is close to meeting the criteria, a slight increase in the annual risk to natural phenomena hazards can be allowed within the tolerance of meeting the target performance goals (See Section 1.3). Note that reduced criteria for seismic evaluation of existing SSCs is supported in Reference 2-16. As a result, some relief in the criteria can be allowed by performing the evaluation using hazard exceedance probability of twice the value recommended in Table 2-1 for the Performance Category of the SSC being considered.
2. The SSC may be strengthened such that its seismic resistance capacity is sufficiently increased to meet these seismic criteria. When upgrading is required it should be designed for the original Performance Goal.
3. The usage of the facility may be changed such that it falls within a less hazardous Performance Category and consequently less stringent seismic requirements.
4. It may be possible to conduct the aspects of the seismic evaluation in a more rigorous manner that removes conservatism such that the SSC may be shown to be adequate. Alternatively, a probabilistic assessment might be undertaken in order to demonstrate that the performance goals can be met.

Requirements of Executive order 12941 (Ref. 1-6), as discussed in the Implementation Guide are to be implemented.

2.4.3 Basic Intention of Dynamic Analysis Based Deterministic Seismic Evaluation and Acceptance Criteria

The basic intention of the deterministic seismic evaluation and acceptance criteria defined in Section 2.3 is to achieve less than a 10% probability of unacceptable performance for a structure, system, or component (SSC) subjected to a Scaled Design/Evaluation Basis Earthquake (SDBE) defined by:

$$SDBE = (1.5SF)(DBE) \qquad (2\text{-}7)$$

where SF is the appropriate seismic scale factor from Equation 2-2.

The seismic evaluation and acceptance criteria presented in this section has intentional and controlled conservatism such that the target performance goals are achieved. The amount of intentional conservatism has been evaluated in Reference 2-1 such that there should be less than 10% probability of unacceptable performance at input ground motion defined by a scale

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factor of 1.5SF times the DBE. Equation 2-7 is useful for developing alternative evaluation and acceptance criteria which are also based on the target performance goals such as inelastic seismic response analyses. To evaluate items for which specific acceptance criteria are not yet developed, such as overturning or sliding of foundations, or some systems and components; this basic intention must be met. If a nonlinear inelastic response analysis which explicitly incorporates the hysteretic energy dissipation is performed, damping values that are no higher than Response Level 2 should be used to avoid the double counting of this hysteretic energy dissipation which would result from the use of Response Level 3 damping values.

2.5 Summary of Seismic Provisions

Table 2-5 summarizes recommended earthquake design and evaluation provisions for Performance Categories 1 through 4. Specific provisions are described in detail in Section 2.3. The basis for these provisions is described in Reference 2-1.

Table 2-5 Summary of Earthquake Evaluation Provisions

	Performance Category (PC)			
	1	2	3	4
Hazard Exceedance Probability, P_H	$2x10^{-3}$	$1x10^{-3}$	$5x10^{-4}$ $(1x10^{-3})^1$	$1x10^{-4}$ $(2x10^{-4})^1$
Response Spectra	Median amplification (no conservative bias)			
Damping for Structural Evaluation	5%		Table 2-3	
Acceptable Analysis Approaches for Structures	Static or dynamic force method normalized to code level base shear		Dynamic analysis	
Analysis approaches for systems and components	UBC Force equation for equipment and non-structural elements (or more rigorous approach)		Dynamic analysis using in-structure response spectra (Damping from Table 2-3)	
Importance Factor	I=1.0	I=1.25	Not used	
Load Factors	Code specified load factors appropriate for structural material		Load factors of unity	
Scale Factors	Not Used		SF = 1.0	SF = 1.25
Inelastic Energy Absorption Ratios	Accounted for by R_w from Table 2-2		$F\mu$ from Table 2-4 by which elastic response is reduced to account for permissible inelastic behavior	
Material Strength	Minimum specified or 95% non-exceedance in-situ values			
Structural Capacity	Code ultimate strength or allowable behavior level		Code ultimate strength or limit-state level	
Quality Assurance Program	Required within a graded approach (i.e., with increasing rigor ranging from UBC requirements from PC-1 to nuclear power plant requirements for PC-4)			
Peer Review	Not Required	Required within a graded approach (i.e., with increasing rigor ranging from UBC requirements from PC-2 to nuclear power plant requirements for PC-4)		

[1]For sites such as LLNL, SNL-Livermore, SLAC, LBL, & ETEC which are near tectonic plate boundaries

2.6 References

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Appendix B
Commentary on General NPH Design and Evaluation Criteria

B.1 NPH Design and Evaluation Philosophy

The natural phenomena hazard (NPH) design and evaluation criteria presented in this document (DOE-STD-1020) implement the requirements of DOE Order 420.1, "Facility Safety" (Ref. B-1) and the associated Implementation Guides: "Implementation Guide for the Mitigation of Natural Phenomena Hazards for DOE Nuclear Facilities and Non-nuclear Facilities" (Ref. B-2), "Implementation Guide for Nonreactor Nuclear Safety Design Criteria and Explosives Safety Criteria" (Ref. B-3), and "Implementation Guide for Use with DOE Orders 420.1 and 440.1 Fire Safety Program" (Ref. B-4) which are intended to assure acceptable performance of DOE facilities in the event of earthquake, wind/tornado, and flood hazards. As discussed in Chapter 1, performance is measured by target performance goals expressed as an annual probability of exceedance of acceptable behavior limits (i.e., behavior limits beyond which damage/failure is unacceptable). DOE Order 420.1 and the associated Implementation Guides establish a graded approach for NPH requirements by defining performance categories (numbered 0 through 4) each with a qualitative performance goal for behavior (i.e., maintain structural integrity, maintain ability to function, maintain confinement of hazardous materials) and a qualitative target probabilistic performance goal. DOE-STD-1020 provides four sets of NPH design and evaluation criteria (explicit criteria are not needed for Performance Category 0). These criteria range from those provided by model building codes for Performance Category 1 to those approaching nuclear power plant criteria for Performance Category 4.

DOE-STD-1020 employs the graded approach by following the philosophy of probabilistic performance goal-based design and evaluation criteria for natural phenomena hazards. Target performance goals range from low probability of NPH-induced damage/failure to very high confidence of extremely low probability of NPH-induced damage/failure. In this manner, structures, systems, and components (SSCs) are governed by NPH criteria which are appropriate for the potential impact on safety, mission, and cost of those SSCs. For example, a much higher likelihood of damage would be acceptable for an unoccupied storage building of low value than for a high-occupancy facility or a facility containing hazardous materials. SSCs containing hazardous materials which, in the event of damage, threaten public safety or the environment, and/or which have been determined to require special consideration, should have a very low probability of damage due to natural phenomena hazards (i.e., much lower probability of damage than would exist from the use of model building code design and evaluation procedures). For ordinary SSCs of relatively low cost, there is typically no need or requirement to add conservatism to the design beyond that of model building codes. For these SSCs, it is also typically not cost-effective to strengthen structures more than required by model building codes that consider extreme loads due to natural phenomena hazards.

Performance goals correspond to probabilities of structure or equipment damage due to natural phenomena hazards; they do not extend to consequences beyond structure or equipment damage. The annual probability of exceedance of SSC damage as a result of natural phenomena hazards (i.e., performance goal) is a combined function of the annual probability of exceedance of the event, factors of safety introduced by the design/evaluation procedures, and other sources of conservatism. These criteria specify hazard annual probabilities of exceedance, response evaluation methods, and permissible behavior criteria for each natural phenomena hazard and for each performance category such that desired performance goals are achieved for either design or evaluation. The ratio of the hazard annual probability of exceedance and the performance goal annual probability of exceedance is called the risk reduction ratio, R_R in DOE-STD-1020. This ratio establishes the level of conservatism to be employed in the design or evaluation process. For example, if the performance goal and hazard annual probabilities are the same ($R_R = 1$), the design or evaluation approach should introduce no conservatism. However, if conservative design or evaluation approaches are employed, the hazard annual probability of exceedance can be larger (i.e., more frequent) than the performance goal annual probability ($R_R > 1$). In the criteria presented herein, the hazard probability and the conservatism in the design/evaluation method are not the same for earthquake, wind, and flood hazards. However, the accumulated effect of each step in the design/evaluation process is to aim at the performance goal probability values which are applicable to each natural phenomena hazard separately.

Design and evaluation criteria are presented in Chapters 2, 3, and 4 for earthquake, wind, and flood hazards, respectively. These criteria are deterministic procedures that establish SSC loadings from probabilistic natural phenomena hazard curves; specify acceptable methods for evaluating SSC response to these loadings; provide acceptance criteria to judge whether computed SSC response is acceptable; and to provide detailing requirements such that behavior is as expected as illustrated in Figure B-1. These criteria are intended to apply equally for design of new facilities and for evaluation of existing facilities. In addition, the criteria are intended to cover buildings, equipment, distribution systems (piping, HVAC, electrical raceways, etc.), and other structures.

DOE-STD-1020 primarily covers (1) methods of establishing load levels on SSCs from natural phenomena hazards and (2) methods of evaluating the behavior of structures and equipment to these load levels. These items are very important, and they are, typically, emphasized in design and evaluation criteria. However, there are other aspects of facility design that are equally important and that should be considered. These aspects include quality assurance considerations and attention to design details. Quality assurance requires peer review of design drawings and calculations; inspection of construction; and testing of material strengths, weld quality, etc. The peer reviewers should be qualified personnel who were not involved in the original design. Important design details include measures to assure ductile behavior and to provide redundant load paths, as well as proper anchorage of equipment and nonstructural building features. Although quality assurance and design details are not discussed in this report to the same extent as NPH load levels and NPH response evaluation and acceptance criteria, the importance of these parts of the design/evaluation process should not be underestimated. Quality assurance and peer review are briefly addressed in Section 1.4, in addition to discussions in the individual chapters on each natural phenomena hazard. Design detailing for earthquake and wind hazards is covered by separate manuals. Reference B-5 describes earthquake design considerations including detailing for ductility. Reference B-6 gives structural details for wind design.

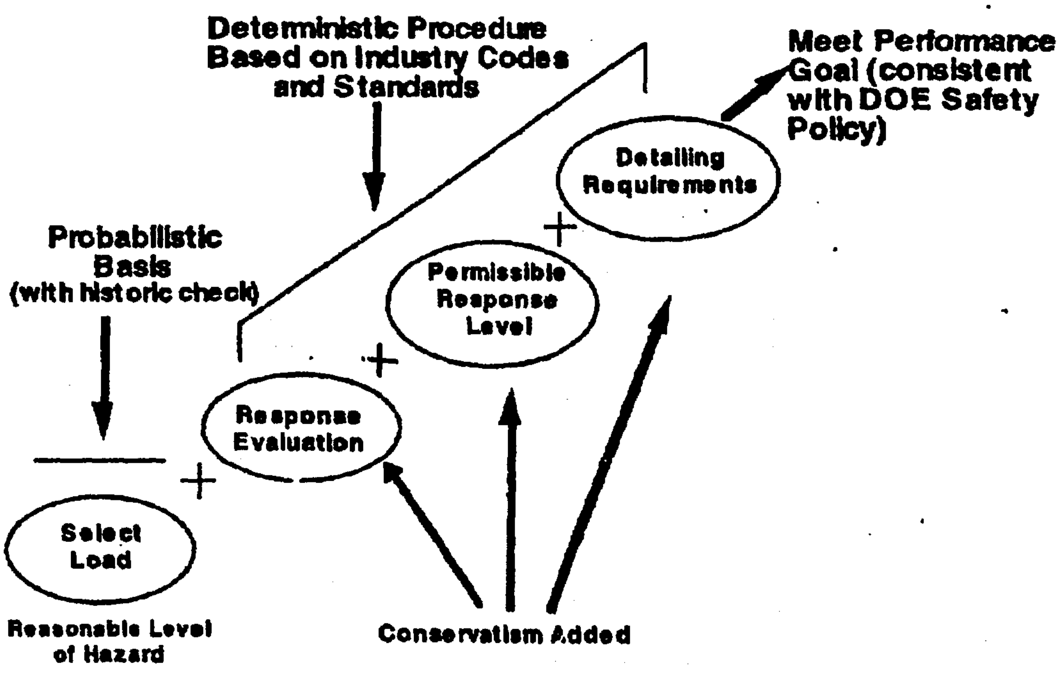

Figure B-1 DOE-STD-1020 Combines Various Methods to Achieve Performance Goals

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B.2 Graded Approach, Performance Goals, and Performance Categories

As stated above, DOE Order 420.1 and the associated Implementation Guides establish a graded approach in which NPH requirements are provided for various performance categories each with a specified performance goal. The motivation for the graded approach is that it enables design or evaluation of DOE structures, systems, and components to be performed in a manner consistent with their importance to safety, importance to mission, and cost. There are only a few "reactor" facilities in the DOE complex and many facilities with a wide variety of risk potential, mission, and cost. Also, the graded approach enables cost-benefit studies and establishment of priorities for existing facilities. There are few new designs planned for the DOE complex and the evaluation of existing facilities requires cost benefit considerations and prioritizing upgrading and retrofit efforts. Finally, the graded approach is common practice by model building codes such as the Uniform Building Code (Ref. B-7), Department of Defense earthquake provisions (Ref. B-8), and even by the Nuclear Regulatory Commission which provides graded criteria from power plants to other licensed nuclear facilities.

The motivation for the use of probabilistic performance goals by the NPH Implemenation Guide for DOE Order 420.1 and DOE-STD-1020 is that accomplish the graded approach using a quantified approach consistent with the variety of DOE facilities as well as meeting the risk-based DOE safety policy. Furthermore, the use of probabilistic performance goals enables the development of consistent criteria both for all natural phenomena hazards (i.e., earthquakes, winds, and floods) and for all DOE facilities which are located throughout the United States. The use of performance goal based criteria is becoming common practice as: it is embedded in recent versions of the Uniform Building Code and in the DOD seismic provisions for essential buildings; it has been used for DOE new production reactor NPH criteria; and it has been utilized in recent Nuclear Regulatory Commission applications such as for the advanced light water reactor program and for revisions to commercial reactor geological siting criteria in 10CFR100, Appendix A.

Five performance categories are specified in the Implementation Guide for DOE Order 420.1 for design/evaluation of DOE structures, systems, and components for natural phenomena hazards ranging from 0 through 4. Table B-1 presents both the qualitative and quantitative descriptions of the performance goals for each performance category. Both the qualitative description of acceptable NPH performance and the quantitative probability value for each performance category are equally significant in establishing these NPH design and evaluation criteria within a graded approach. SSCs are to be placed in categories in accordance with DOE-STD-1021-93 (Ref. B-9) Additional guidance on performance categorization is available in Reference B-10.

As mentioned previously, the quantitative performance goal probability values are applicable to each natural phenomena hazard (earthquake, wind, and flood) individually. The earthquake and flood design and evaluation criteria presented in this document are aimed at meeting the target performance goals given in Table B-1. The extreme wind and tornado design and evaluation criteria presented in this document are conservative compared to earthquake and flood criteria in that they are aimed at lower probability levels than the target performance goals in Table B-1. It is estimated that for extreme winds, the probabilities of exceeding acceptable behavior limits are less than one order of magnitude smaller than the performance goals in Table B-1. For tornado criteria, the probabilities of exceeding acceptable behavior limits are greater than one but less than two orders of magnitude smaller than the performance goals for Performance Categories 3 and 4. This additional conservatism in wind and tornado criteria for design and evaluation of DOE facilities is consistent with common practice in government and private industry. Furthermore, this additional conservatism can be accommodated in the design and evaluation of SSCs without significantly increasing costs. SSCs in Performance Categories 3 and 4 should be designed for tornadoes at certain sites around the country where tornado occurrences are high. The tornado hazard probability must be set lower than necessary to meet the performance goals in order for tornadoes rather than straight w. ¹ˢ or hurricanes to control the design criteria.

Table B-1 Structure, System, or Component (SSC) NPH Performance Goals for Various Performance Categories

Performance Category	Performance Goal Description	NPH Performance Goal Annual Probability of Exceeding Acceptable Behavior Limits, P_F
0	No Safety, Mission, or Cost Considerations	No requirements
1	Maintain Occupant Safety	$\approx 10^{-3}$ of the onset of SSC[(1)] damage to the extent that occupants are endangered
2	Occupant Safety, Continued Operation with Minimum Interruption	$\approx 5\times10^{-4}$ of SSC damage to the extent that the component cannot perform its function
3	Occupant Safety, Continued Operation, Hazard Confinement	$\approx 10^{-4}$ of SSC damage to the extent that the component cannot perform its function
4	Occupant Safety, Continued Operation, Confidence of Hazard Confinement	$\approx 10^{-5}$ of SSC damage to the extent that the component cannot perform its function

(1) These performance goals are for each natural phenomena hazard (earthquake, wind, and flood).
(2) SSC refers to structure, distribution system, or component (equipment).

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The design and evaluation criteria for SSCs in Performance Categories 0, 1, and 2 are similar to those given in model building codes. Performance Category 0 recognizes that for certain lightweight equipment items, furniture, etc., and for other special circumstances where there is little or no potential impact on safety, mission, or cost, design or evaluation for natural phenomena hazards may not be needed. Assignment of an SSC to Performance Category 0 is intended to be consistent with, and not take exception to, model building code NPH provisions. Performance Category 1 criteria include no extra conservatism against natural phenomena hazards beyond that in model building codes that include earthquake, wind, and flood considerations. Performance Category 2 criteria are intended to maintain the capacity to function and to keep the SSC operational in the event of natural phenomena hazards. Model building codes would treat hospitals, fire and police stations, and other emergency-handling facilities in a similar manner to DOE-STD-1020 Performance Category 2 NPH design and evaluation criteria.

Performance Category 3 and 4 SSCs handle significant amounts of hazardous materials or have significant programmatic impact. Damage to these SSCs could potentially endanger worker and public safety and the environment or interrupt a significant mission. As a result, it is very important for these SSCs to continue to function in the event of natural phenomena hazards, such that the hazardous materials may be controlled and confined. For these categories, there must be a very small likelihood of damage due to natural phenomena hazards. DOE-STD-1020 NPH criteria for Performance Category 3 and higher SSCs are more conservative than requirements found in model building codes and are similar to DOD criteria for high risk buildings and NRC criteria for various applications as illustrated in Table B-2. Table B-2 illustrates how DOE-STD-1020 criteria for the performance categories defined in DOE Order 420.1 and the associated Implementation Guides compare with NPH criteria from other sources.

Table B-2 Comparison of Performance Categories from Various Sources

Source	SSC Categorization			
DOE-STD-1020 - DOE Natural Phenomena Hazard Criteria	1	2	3	4
Uniform Building Code	General Facilities	Essential Facilities	-	-
DOD Tri-Service Manual for Seismic Design of Essential Buildings	-	-	High Risk	-
Nuclear Regulatory Commission	-		Evaluation of NRC Fuel Facilities	Evaluation of Existing Reactors

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The design and evaluation criteria presented in this document for SSCs subjected to natural phenomena hazards have been specified to meet the performance goals presented in Table B-1. The basis for selecting these performance goals and the associated annual probabilities of exceedance are described briefly in the remainder of this section.

For *Performance Category 1 SSCs*, the primary concern is preventing major structural damage or collapse that would endanger personnel. A performance goal annual probability of exceedance of about 10^{-3} of the onset of significant damage is appropriate for this category. This performance is considered to be consistent with model building codes (Refs. B-7, B-11, B-12, and B-13), at least for earthquake and wind considerations. The primary concern of model building codes is preventing major structural failure and maintaining life safety under major or severe earthquakes or winds. Repair or replacement of the SSC or the ability of the SSC to continue to function after the occurrence of the hazard is not considered.

Performance Category 2 SSCs are of greater importance due to mission-dependent considerations. In addition, these SSCs may pose a greater danger to on-site personnel than Performance Category 1 SSCs because of operations or materials involved. The performance goal is to maintain both capacity to function and occupant safety. Performance Category 2 SSCs should allow relatively minor structural damage in the event of natural phenomena hazards. This is damage that results in minimal interruption to operations and that can be easily and readily repaired following the event. A reasonable performance goal is judged to be an annual probability of exceedance of between 10^{-3} and 10^{-4} of structure or equipment damage, with the SSC being able to function with minimal interruption. This performance goal is slightly more severe than that corresponding to the design criteria for essential facilities (e.g., hospitals, fire and police stations, centers for emergency operations) in accordance with model building codes (e.g., Ref. B-7).

Performance Category 3 and higher SSCs pose a potential hazard to public safety and the environment because radioactive or toxic materials are present. Design considerations for these categories are to limit SSC damage so that hazardous materials can be controlled and confined, occupants are protected, and functioning of the SSC is not interrupted. The performance goal for Performance Category 3 and higher SSCs is to limit damage such that DOE safety policy is achieved. For these categories, damage must typically be limited in confinement barriers (e.g., buildings, glove boxes, storage cannisters, vaults), ventilation systems and filtering, and monitoring and control equipment in the event of an occurrence of severe earthquakes, winds, or floods. In addition, SSCs can be placed in Performance Categories 3 or 4 if improved performance is needed due to cost or mission requirements.

For Performance Category 3 SSCs, an appropriate performance goal has been set at an annual probability of exceedance of about 10^{-4} of damage beyond which hazardous material confinement and safety-related functions are impaired. For Performance Category 4 SSCs, a reasonable performance goal is an annual probability of exceedance of about 10^{-5} of damage beyond which hazardous material confinement and safety-related functions are impaired. These performance goals approaches and approximates, respectively, at least for earthquake considerations, the performance goal for seismic-induced core damage associated with design of commercial nuclear power plants (Refs. B-14, B-15, B-16, and B-17). Annual frequencies of seismic core damage from published probabilistic risk assessments (PRA) of recent commercial nuclear plants have been summarized in Reference B-18. This report indicates that mean seismic core damage frequencies ranged from 4×10^{-6}/year to 1×10^{-4}/year based on consideration of 12 plants. For 10 of the 12 plants, the annual seismic core damage frequency was greater than 1×10^{-5}. Hence, the Performance Category 4 performance goals given in the NPH Implementation Guide for DOE Order 420.1 are consistent with Reference B-18 information.

B.3 Evaluation of Existing Facilities

New SSCs can be designed by these criteria, but existing SSCs may not meet these NPH provisions. For example, most facilities built a number of years ago in the eastern United States were designed without consideration of potential earthquake hazard. It is, therefore, likely that some older DOE facilities do not meet the earthquake criteria presented in this document.

For existing SSCs, an assessment must be made for the as-is condition. This assessment includes reviewing drawings and conducting site visits to determine deviations from the drawings and any in-service deterioration. In-place strength of the materials can be used when available. Corrosive action and other aging processes should be considered. Evaluation of existing SSCs is similar to evaluations performed of new designs except that a single as-is configuration is evaluated instead of several configurations in an iterative manner, as required in the design process. Evaluations should be conducted in order of priority, with highest priority given to those areas identified as weak links by preliminary investigations and to areas that are most important to personnel safety and operations with hazardous materials. Prioritization criteria for evaluation and upgrade of existing DOE facilities are currently being developed.

If an existing SSC does not meet the natural phenomena hazard design/evaluation criteria, several options (such as those illustrated by the flow diagram in Figure B-2) need to be considered. Potential options for existing SSCs include:

1. Conduct a more rigorous evaluation of SSC behavior to reduce conservatism which may have been introduced by simple techniques used for initial SSC evaluation. Alternatively, a probabilistic assessment of the SSC might be undertaken in order to demonstrate that the performance goals for the SSC can be met.
2. The SSC may be strengthened to provide resistance to natural phenomena hazard effects that meets the NPH criteria.
3. The usage of the SSC may be changed so that it falls within a lower performance category and consequently, less stringent requirements.

If SSC evaluation uncovers deficiencies or weaknesses that can be easily remedied, these should be upgraded without considering the other options. It is often more cost-effective to implement simple SSC upgrades than to expend effort on further analytical studies. Note that the actions in Table B-2 need not necessarily be accomplished in the order shown.

Evaluations of existing SSCs must follow or, at least, be measured against the NPH criteria provided in this document. For SSCs not meeting these criteria and which cannot be easily remedied, budgets and schedule for required strengthening must be established on a prioritized basis. As mentioned previously, prioritization criteria for evaluation and upgrade of existing DOE facilities are currently being developed. Priorities should be established on the basis of performance category, cost of strengthening, and margin between as-is SSC capacity and the capacity required by the criteria. For SSCs which are close to meeting criteria, it is probably not cost effective to strengthen the SSC in order to obtain a small reduction in risk. As a result, some relief in the criteria is allowed for evaluation of existing SSCs. It is permissible to perform such evaluations using natural phenomena hazard exceedance probability of twice the value specified for new design. For example, if the natural phenomena hazard annual probability of exceedance for the SSC under consideration was 10^{-4}, it would be acceptable to reconsider the SSC at hazard annual probability of exceedance of $2x10^{-4}$. This would have the effect of slightly reducing the seismic, wind, and flood loads in the SSC evaluation. This amount of relief is within the tolerance of meeting the target performance goals and is only a minor adjustment of the corresponding NPH design and evaluation criteria.

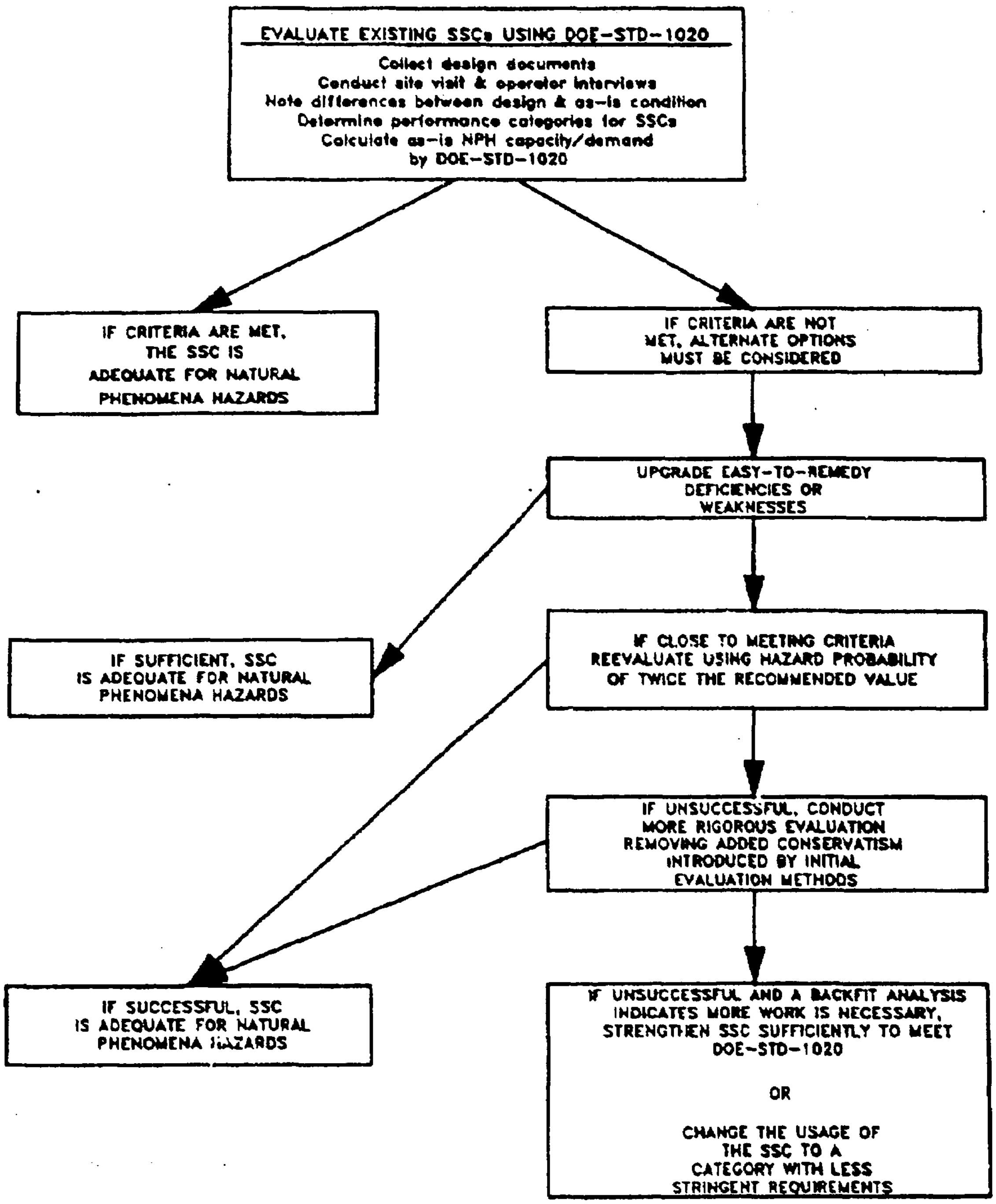

Figure B-2 Evaluation Approach for an Existing SSC

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B.4 References

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Appendix C
Commentary on Earthquake Design and Evaluation Criteria

C.1 Introduction

Earthquake design and evaluation criteria for DOE structures, systems, and components are presented in Chapter 2 of this standard. Commentary on the DOE earthquake design and evaluation provisions is given in this appendix. Specifically, the basic approach employed is discussed in Section C.2 along with meeting of target performance goals, seismic loading is addressed in Section C.3, evaluation of seismic response is discussed in Section C.4, capacities and good seismic design practice are discussed in Section C.5, special considerations for systems and components and for existing facilities are covered in Sections C.6 and C.7, respectively, and quality assurance and peer review are addressed in Section C.8. Alternate seismic mitigation measures are discussed in Section C.9.

These seismic criteria use the target performance goals of the NPH Implementation Guide for DOE Order 420.1 (Ref. C-67) to assure safe and reliable performance of DOE facilities during future potential earthquakes. Design of structures, systems, and components to withstand earthquake ground motion without significant damage or loss of function depends on the following considerations:

1. The SSC must have sufficient strength and stiffness to resist the lateral loads induced by earthquake ground shaking. If an SSC is designed for insufficient lateral forces or if deflections are unacceptably large, damage can result, even to well-detailed SSCs.
2. Failures in low ductility modes (e.g., shear behavior) or due to instability that tend to be abrupt and potentially catastrophic must be avoided. SSCs must be detailed in a manner to achieve ductile behavior such that they have greater energy absorption capacity than the energy content of earthquakes.
3. Building structures and equipment which are base supported tend to be more susceptible to earthquake damage (because of inverted pendulum behavior) than distributed systems which are supported by hangers with ductile connections (because of pendulum restoring forces).
4. The behavior of an SSC as it responds to earthquake ground motion must be fully understood by the designer such that a "weak link" that could produce an unexpected failure is not overlooked. Also, the designer must consider both relative displacement and inertia (acceleration) induced seismic failure modes.
5. SSCs must be constructed in the manner specified by the designer. Materials must be of high quality and as strong as specified by the designer. Construction must be of high quality and must conform to the design drawings.

By the NPH Implementation Guide for DOE Order 420.1 (Ref. C-67) and this standard, probabilistic performance goals are used as a target for formulating deterministic seismic design criteria. Table C-1 defines seismic performance goals for structures, systems, or components (SSCs) assigned to Performance Categories 1 through 4. SSCs are to be assigned to performance categories in accordance with DOE-STD-1021-93 (Ref. C-26). The seismic performance goals are defined in terms of a permissible annual probability of unacceptable performance P_F (i.e., a permissible failure frequency limit). Seismic induced unacceptable performance should have an annual probability less than or approximately equal to these goals.

Table C-1 Structure, System, or Component (SSC) Seismic Performance Goals for Various Performance Categories

Performance Category	Performance Goal Description	Seismic Performance Goal Annual Probability of Exceeding Acceptable Behavior Limits, P_F
1	Maintain Occupant Safety	$\approx 10^{-3}$ of the onset of SSC[1] damage to the extent that occupants are endangered
2	Occupant Safety, Continued Operation with Minimum Interruption	$\approx 5 \times 10^{-4}$ of SSC damage to the extent that the component cannot perform its function
3	Occupant Safety, Continued Operation, Hazard Confinement	$\approx 10^{-4}$ of SSC damage to the extent that the component cannot perform its function
4	Occupant Safety, Continued Operation, Confidence of Hazard Confinement	$\approx 10^{-5}$ of SSC damage to the extent that the component cannot perform its function

(1) SSC refers to structure, distribution system, or component (equipment).

The performance goals shown in Table C-1 include both quantitative probability values and qualitative descriptions of acceptable performance. The qualitative descriptions of expected performance following design/evaluation levels of earthquake ground motions are expanded in Table C-2. These descriptions of acceptable performance are specifically tailored to the needs in many DOE facilities.

The performance goals described above are achieved through the use of DOE seismic design and evaluation provisions which include: (1) lateral force provisions; (2) story drift/damage control provisions; (3) detailing for ductility provisions; and (4) quality assurance provisions. These provisions are comprised of the following four elements taken together: (1) seismic loading; (2) response evaluation methods; (3) permissible response levels; and (4) ductile detailing requirements. Acceptable performance (i.e., achieving performance goals) can only be reached by consistent specification of all design criteria elements as shown in Figure C-1.

Table C-2 Qualitative Seismic Performance Goals

PC	Occupancy Safety	Concrete Barrier	Metal Liner	Component Functionality	Visible Damage
1	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation	Confinement not required.	Confinement not required.	Component will remain anchored, but no assurance it will remain functional or easily repairable.	Building distortion will be limited but visible to the naked eye.
2	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation	Concrete walls will remain standing but may be extensively cracked; they may not maintain pressure differential with normal HVAC. Cracks will still provide a tortuous path for material release. Don't expect largest cracks greater than 1/2 inch.	May not remain leak tight because of excessive distortion of structure.	Component will remain anchored and majority will remain functional after earthquake. Any damaged equipment will be easily repaired.	Building distortion will be limited but visible to the naked eye.
3	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation	Concrete walls cracked; but small enough to maintain pressure differential with normal HVAC. Don't expect largest cracks greater than 1/8 inch.	Metal liner will remain leak tight.	Component anchored and functional.	Possibly visible local damage but permanent distortion will not be immediately apparent to the naked eye.
4	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation	Concrete walls cracked; but small enough to maintain pressure differential with normal HVAC. Don't expect largest cracks greater than 1/8 inch.	Metal liner will remain leak tight.	Component anchored and functional.	Possibly visible local damage but permanent distortion will not be immediately apparent to the naked eye.

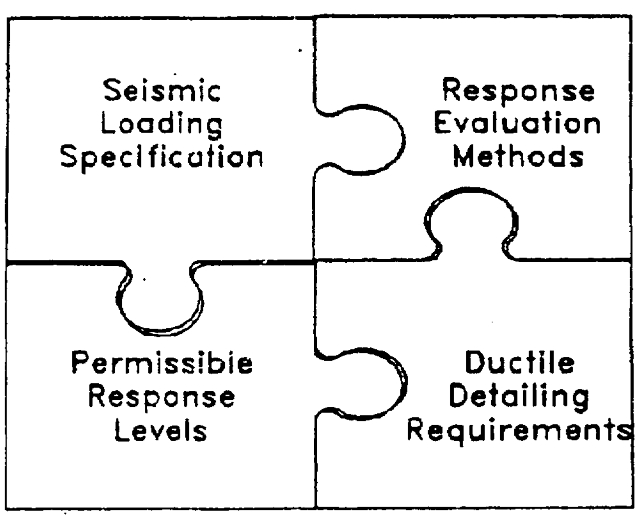

Figure C-1 Consistent Specification of All Seismic Design/Evaluation Criteria Elements

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C.2 Basic Approach for Earthquake Design and Evaluation and Meeting Target Performance Goals

C.2.1 Overall Approach for DOE Seismic Criteria

Structure/component performance is a function of: (1) the likelihood of hazard occurrence and (2) the strength of the structure or equipment item. Consequently, seismic performance depends not only on the earthquake probability used to specify design seismic loading, but also on the degree of conservatism used in the design process as illustrated in Figure C-2. For instance, if one wishes to achieve less than about 10^{-4} annual probability of onset of loss of function, this goal can be achieved by using conservative design or evaluation approaches for a natural phenomena hazard that has a more frequent annual probability of exceedance (such as 10^{-3}), or it can be achieved by using median-centered design or evaluation approaches (i.e., approaches that have no intentional conservative or unconservative bias) coupled with a 10^{-4} hazard definition. At least for the earthquake hazard, the former alternate has been the most traditional. Conservative design or evaluation approaches are well-established, extensively documented, and commonly practiced. Median design or evaluation approaches are currently controversial, not well understood, and seldom practiced. Conservative design and evaluation approaches are utilized for both conventional facilities (similar to DOE Performance Category 1) and for nuclear power plants (similar to DOE Performance Category 4). For consistency with these other uses, the approach in this standard specifies the use of conservative design and evaluation procedures coupled with a hazard definition consistent with these procedures.

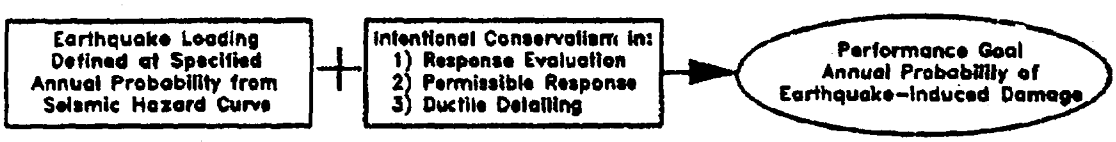


Figure C-2 Performance Goal Achievement

The performance goals for Performance Category 1 SSCs are consistent with goals of model building codes for normal facilities; the performance goals for Performance Category 2 SSCs are slightly more conservative than the goals of model building codes for important or essential facilities. For seismic design and evaluation, model building codes utilize equivalent static force methods except for very unusual or irregular facilities, for which a dynamic analysis method is employed. The performance goals for Performance Category 3 SSC's are consistent with DOE essential facilities and Pu handling facilities. The performance goals for Performance Category 4 SSC's approach those used for nuclear power plants. For these reasons, this standard specifies seismic design and evaluation criteria for PC-1 and PC-2 SSC's corresponding closely to model building codes and seismic design and evaluation criteria for both PC-3 and PC-4 SSC's based on dynamic analysis methods consistent with those used for similar nuclear facilities.

By this standard, the DBE is defined at specified hazard probability P_H and the SSC is designed or evaluated for this DBE using an adequately conservative deterministic acceptance criteria. To be adequately conservative, the acceptance criteria must introduce an additional reduction in the risk of unacceptable performance below the annual risk of exceeding the DBE. The ratio of the seismic hazard exceedance probability, P_H to the performance goal probability P_F is defined herein as the risk reduction ratio R_R, given by:

$$R_R = \frac{P_H}{P_F} \qquad \text{(C-1)}$$

The required degree of conservatism in the deterministic acceptance criteria is a function of the specified risk reduction ratio. Table C-3 provides a set of seismic hazard exceedance probabilities, P_H and risk reduction ratios, R_R for Performance Categories 1 through 4 required to achieve the seismic performance goals specified in Table C-1. Note that Table C-3 follows the philosophy of:

1) gradual reduction in hazard annual exceedance probability
2) gradual increase in conservatism of evaluation procedure as one goes from Performance Category 1 to Performance Category 4 (PC 1 to PC 4).

Table C-3 Seismic Performance Goals & Specified Seismic Hazard Probabilities

Performance Category	Target Seismic Performance Goal, P_F	Seismic Hazard Exceedance Probability, P_H	Risk Reduction Ratio, R_R
1	1×10^{-3}	2×10^{-3}	2
2	5×10^{-4}	1×10^{-3}	2
3	1×10^{-4}	5×10^{-4} (1×10^{-3})[1]	5 (10)[1]
4	1×10^{-5}	1×10^{-4} (2×10^{-4})[1]	10 (20)[1]

[1] For sites such as LLNL, SNL-Livermore, SLAC, LBL, and ETEC which are near tectonic plate boundaries.

Different structures, systems, or components may have different specified performance goal probabilities, P_F. It is required that for each structure, system, or component, either: (1) the performance goal category; or (2) the hazard probability (P_H) or the DBE together with the appropriate R_R factor will be specified in a design specification or implementation document that invokes these criteria. As shown in Table 2-3, the recommended hazard exceedance probabilities and performance goal exceedance probabilities are different. These differences indicate that conservatism must be introduced in the seismic behavior evaluation approach to achieve the required risk reduction ratio, R_R. In earthquake evaluation, there are many places where conservatism can be introduced, including:

1. Maximum design/evaluation ground acceleration and velocity.
2. Response spectra amplification.
3. Damping.
4. Analysis methods.
5. Specification of material strengths.
6. Estimation of structural capacity.
7. Load or scale factors.
8. Importance factors.
9. Limits on inelastic behavior.
10. Soil-structure interaction (except for frequency shifting due to SSI).
11. Effective peak ground motion.
12. Effects of a large foundation or foundation embedment.

For the earthquake evaluation criteria in this standard, conservatism is intentionally introduced and controlled by specifying (1) hazard exceedance probabilities, (2) load or scale factors, (3) importance factors, (4) limits on inelastic behavior, and (5) conservatively specified material strengths and structural capacities. Load and importance factors have been retained for the evaluation of Performance Category 2 and lower SSCs because the UBC approach (which includes these factors) is followed for these categories. Importance factors are not used for Performance Category 3 and higher SSCs. However, a seismic scale factor SF is used to provide the difference in risk reduction ratio R_R between Performance Categories 3 and 4. Material strengths and structural capacities specified for Performance Category 3 and higher SSCs correspond to ultimate strength code-type provisions (i.e., ACI 318-89 for reinforced concrete, LRFD, or AISC Chapter N for steel). Material strengths and structural capacities specified for Performance Category 2 and lower SSCs correspond to either ultimate strength or allowable stress code-type provisions. It is recognized that such provisions introduce conservatism. In addition, significant additional conservatism can be introduced if considerations of effective peak ground motion, soil-structure interaction, and effects of large foundation or foundation embedment are ignored.

The differences in seismic evaluation criteria among categories in terms of load and importance factors, limits on inelastic behavior, and other factors by this standard are summarized below:

1.	PC 1 and PC 2	From PC 1 to PC 2, seismic hazard exceedance probability is lowered and importance factor is increased. All other factors are held the same.
2.	PC 2 and PC 3	From PC 2 to PC 3, load and importance factors are eliminated, damping is generally increased, and limits on inelastic behavior are significantly reduced. All other factors are essentially the same, although static force evaluation methods are allowed for PC 2 SSCs and dynamic analysis is required for PC 3 SSCs.
3.	PC 3 and PC 4	From PC 3 to PC 4, seismic hazard exceedance probability is lowered and a seismic scale factor is used. All other factors are held the same.

The basic intention of the deterministic seismic evaluation and acceptance criteria presented in Chapter 2 is to achieve less than a 10% probability of unacceptable performance for a structure, system, or component (SSC) subjected to a Scaled Design/Evaluation Basis Earthquake (SDBE) defined by:

$$SDBE = (1.5SF)(DBE) \qquad \text{(C-2)}$$

where SF is the appropriate seismic scale factor (SF is 1.0 for PC 3 and 1.25 for PC 4). The seismic evaluation and acceptance criteria presented in this standard has intentional and controlled conservatism such that the required risk reduction ratios, R_R, and target performance goals are achieved. The amount of intentional conservatism has been evaluated in Reference C-20 as that there should be less than 10% probability of unacceptable performance at input ground motion defined by a scale factor of 1.5SF times the DBE. Equation C-2 is useful for developing alternative evaluation and acceptance criteria which are also based on the target performance goals.

It is permissible to substitute alternate acceptance criteria for those criteria defined in Chapter 2 so long as these alternate criteria will also reasonably achieve less than about a 10% probability of unacceptable performance for the combination of the SDBE defined by Equation C-2 with the best-estimate of the concurrent non-seismic loads. This relief is permitted to enable one to define more sophisticated alternate acceptance criteria than those presented in Chapter 2 when one has a sufficient basis to develop and defend this alternate criteria.

C.2.2 Influence of Seismic Scale Factor

The target performance goals of the Implementation Guide for DOE Order 420.1 are the basis of the seismic design and evaluation criteria presented in this standard. It is known that for PC 1 and PC 2, target performance goals, P_F, of 1×10^{-3} and 5×10^{-4}, respectively, are met relatively closely. However, for PC 3 and PC 4, target performance goals, P_F, of $1x10^{-4}$ and $1x10^{-5}$, respectively, are met in a more approximate manner as illustrated in this section. The variability in performance goal achievement can be most significantly attributed to the uncertainty in the slopes of seismic hazard curves from which DBE ground motion is determined. Seismic hazard curve slope does not have a significant effect on performance for PC 1 and PC 2 because P_F and P_H do not differ greatly (i.e. $R_R = P_H/P_F = 2$).

Over any ten-fold difference in exceedance probabilities, seismic hazard curves may be approximated by:

$$H(a) = K a^{-k_H} \tag{C-3}$$

where H(a) is the annual probability of exceedance of ground motion level "a," K is a constant, and k_H is a slope parameter. Slope coefficient, A_R is the ratio of the increase in ground motion corresponding to a ten-fold reduction in exceedance probability. A_R is related to k_H by:

$$k_H = \frac{1}{\log(A_R)} \tag{C-4}$$

The Basis for Seismic Provisions of DOE-STD-1020 (Ref. C-20) presents estimates of seismic hazard curve slope ratios A_R for typical U.S. sites over the annual probability range of 10^{-3} to 10^{-5}. For eastern U.S. sites, A_R typically falls within the range of 2 to 4 although A_R values as large as 6 have been estimated. For California and other high seismic sites near tectonic plate boundaries with seismicity dominated by close active faults with high recurrence rates, A_R typically ranges from 1.5 to 2.25. For other western sites with seismicity not dominated by close active faults with high recurrence rates such as INEL, LANL, and Hanford, A_R typically ranges from 1.75 to 3.0. Therefore, seismic design/evaluation criteria should be applicable over the range of A_R from 1.5 to 6 with emphasis on the range from 2 to 4.

DOE seismic design and evaluation criteria presented in Chapter 2 is independent of A_R and, thus, does not reflect its effect on meeting target goals. The performance of structures, systems, and components in terms of annual probability of exceeding acceptable behavior limits can be evaluated by convolution of seismic hazard and seismic fragility curves. Seismic fragility curves describe the probability of unacceptable performance versus ground motion level. The fragility curve is defined as being lognormally distributed and is expressed in terms of two parameters: a median capacity level, C_{50}, and a logarithmic standard deviation, β. β expresses the uncertainty in the capacity level and generally lies within the range of 0.3 to 0.6. For DBE ground motion specified at annual probability, P_H, it is shown in Ref. C-20 that the risk reduction ratio, R_R, between the annual probability of exceeding the DBE and the annual probability of unacceptable performance is given by:

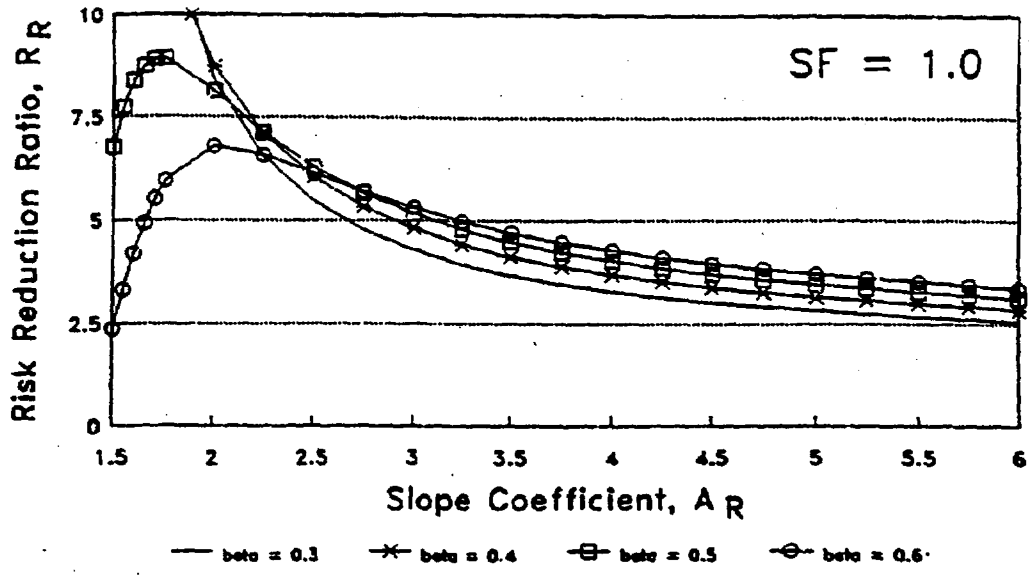
$$R_R = (C_{50}/\mathrm{DBE})^{k_H} e^{-\frac{1}{2}(k_H\beta)^2} \tag{C-5}$$

where C_{50} and β define the seismic fragility curve and DBE and k_H define the seismic hazard curve.

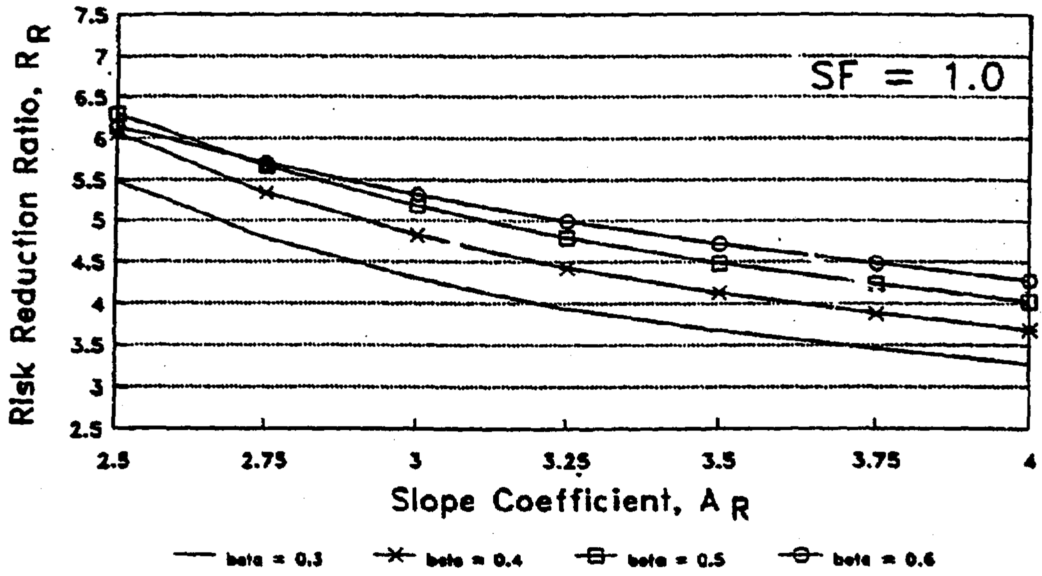
Using the basic criterion of DOE-STD-1020 that target performance goals are achieved when the minimum required 10% probability of failure capacity, C_{10} is equal to 1.5 times the seismic scale factor, SF, times the DBE ground motion, Equation (C-5) may be rewritten as:

$$R_R = (1.5\mathrm{SF})^{k_H} e^{\left[1.282k_H\beta-\frac{1}{2}(k_H\beta)^2\right]} \tag{C-6}$$

Equation (C-6) demonstrates the risk reduction ratio achieved by DOE seismic criteria as a function of hazard curve slope, uncertainty, β, and seismic scale factor, SF. Note from Table C-3 that for Performance Category 4 (not near tectonic plate boundaries), the hazard probability is 1×10^{-4} and the performance goal is 1×10^{-5} such that the target risk reduction ratio, R_R is 10 and for Performance Category 3, the hazard probability is 5×10^{-4} and the performance goal is 1×10^{-4} such that the target risk reduction ratio, R_R is 5. The actual risk reduction ratios from Equation (C-6) versus slope coefficient A_R are plotted in Figures C-3 and C-4 for Performance Categories 3 and 4, respectively. In these figures, SF of 1.0 is used for PC 3 and SF of 1.25 is used for PC 4 and the range of β from 0.3 to 0.6 has been considered. For the hazard curves considered by DOE-STD-1024-92 (Ref. C-13), A_R values average about 3.2 in the probability range associated with PC 3 and about 2.4 in the probability range associated with PC 4. More recent seismic hazard studies (Ref. C-6) gives A_R values which average about 3.8 in the probability range associated with PC 3 and about 3.0 in the probability range associated with PC 4. As a result, Figure C-3 includes a blown-up view for the 2.5 to 4 A_R range and Figure C-4 includes a blown-up view for the 2 to 3 A_R range.

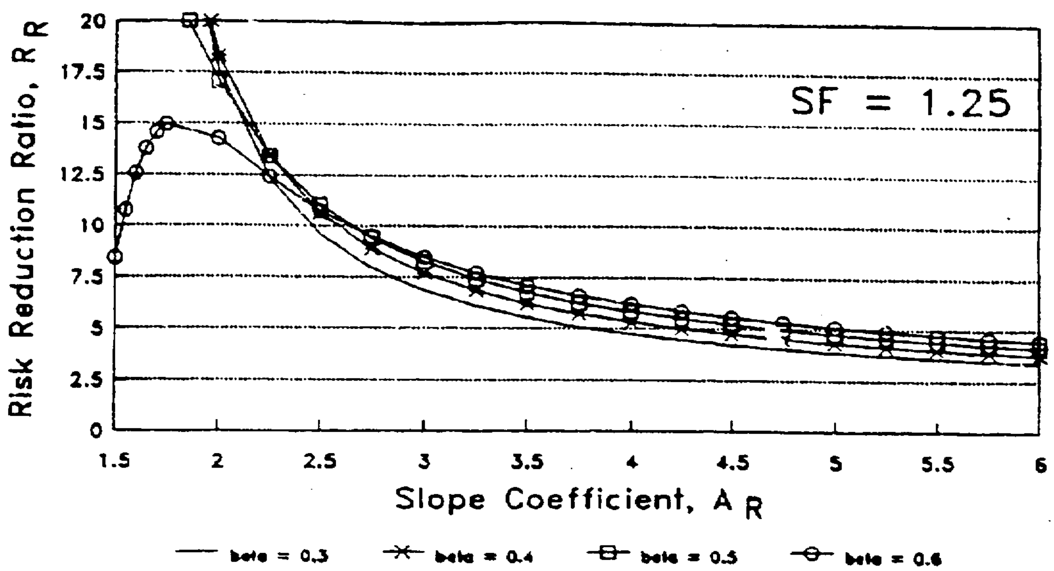


a) A_R from 1.5 to 6

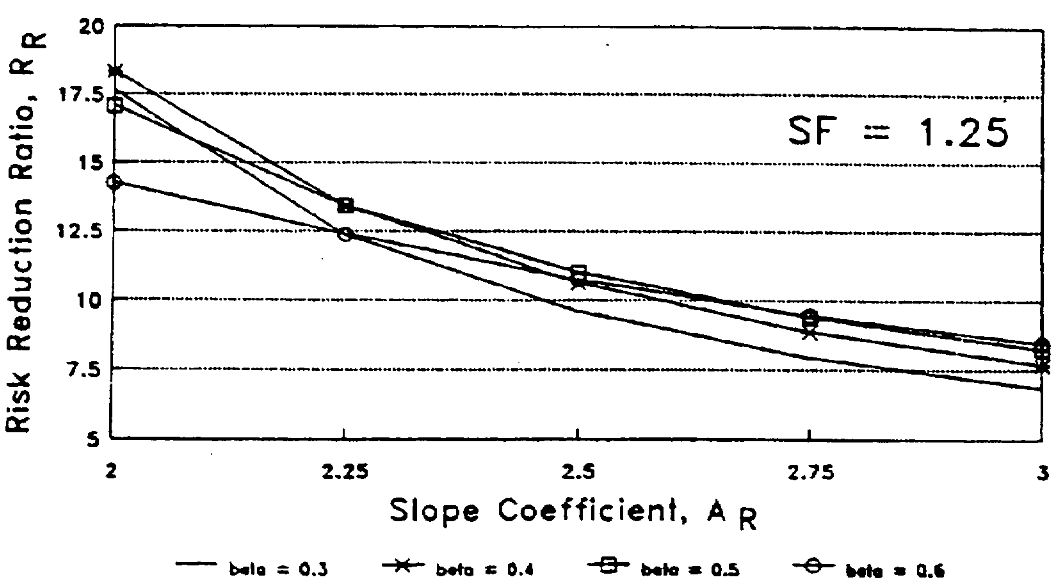


b) A_R from 2.5 to 4

Figure C-3 Value of R_R vs A_R for SF = 1.0 (PC 3)



a) A_R from 1.5 to 6



b) A_R from 2 to 3

Figure C-4 Value of R_R vs A_R for SF = 1.25 (PC 4)

Figure C-3 demonstrates that for SF = 1.0, risk reduction ratios between about 3 and 10 are achieved over the A_R range from 2 to 6. These risk reduction ratios support achieving performance goals between about $2x10^{-4}$ to $5x10^{-5}$. In the primary region of interest of A_R between 2.5 and 4, risk reduction ratios from 4 to 6 are achieved as compared to the target level of 5 for PC 3 and sites not near tectonic plate boundaries. Figure C-4 demonstrates that for SF = 1.25, risk reduction ratios between about 3 and 20 are achieved over the A_R range from 2 to 6. These risk reduction ratios support achieving performance goals between about $3x10^{-5}$ to $5x10^{-6}$. In the primary region of interest of A_R between 2 and 3, risk reduction ratios from about 8 to 17 are achieved as compared to the target level of 10 for PC 4 and sites not near tectonic plate boundaries.

The risk reduction ratio achieved may be improved by using a variable formulation of SF which is a function of A_R. In order to justify use of the variable scale factor approach, the site specific hazard curve must have a rigorous pedigree. Reference C-20 demonstrates that the SF factors shown in Figure C-5 give the best fit of R_R over the A_R range of primary interest from about 2 to about 6. The use of the scale factors given in Figure C-5 combined with Equation C-6 improves the R_R values compared to target values as shown in Figures C-6 and C-7 for PC 3 (R_R = 5) and PC 4 (R_R = 10), respectively. Figures C-6 and C-7 demonstrate that when the variable scale factors from Figure C-5 are used, risk reduction factors achieved are within about 10% of the target values of 5 and 10, respectively. As a result, target performance goals would be met within about the same 10%.

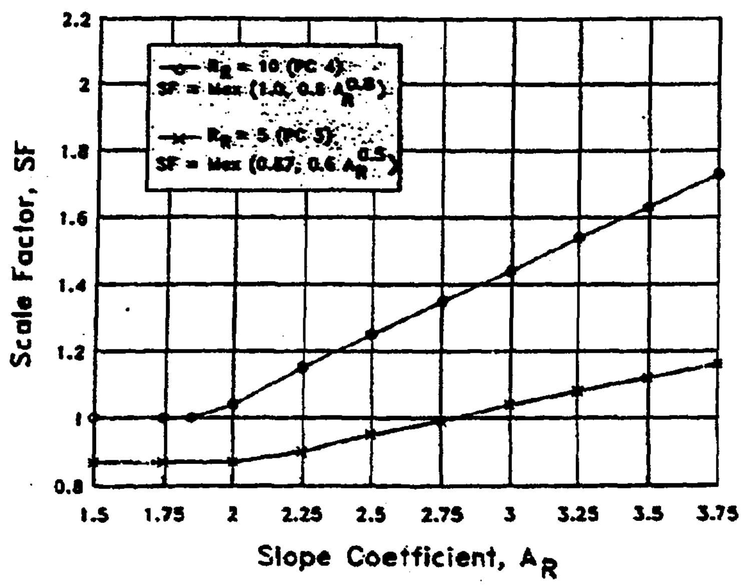


Figure C-5 Variable II Seismic Scale Factor for PC 3 and PC 4

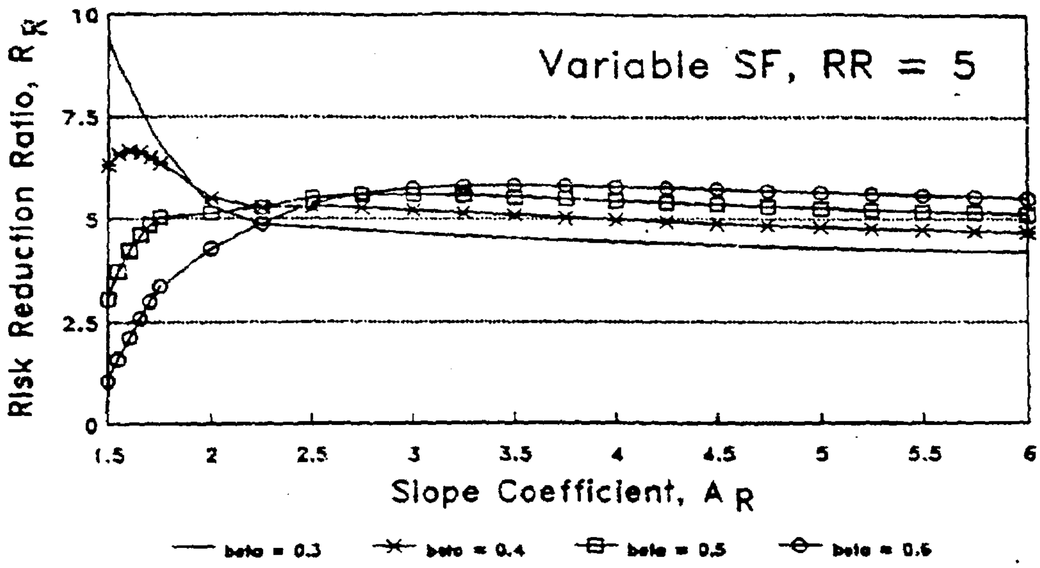


Figure C-6 Value of R_R vs A_R for Variable SF (Fig. C-5 for PC 3)

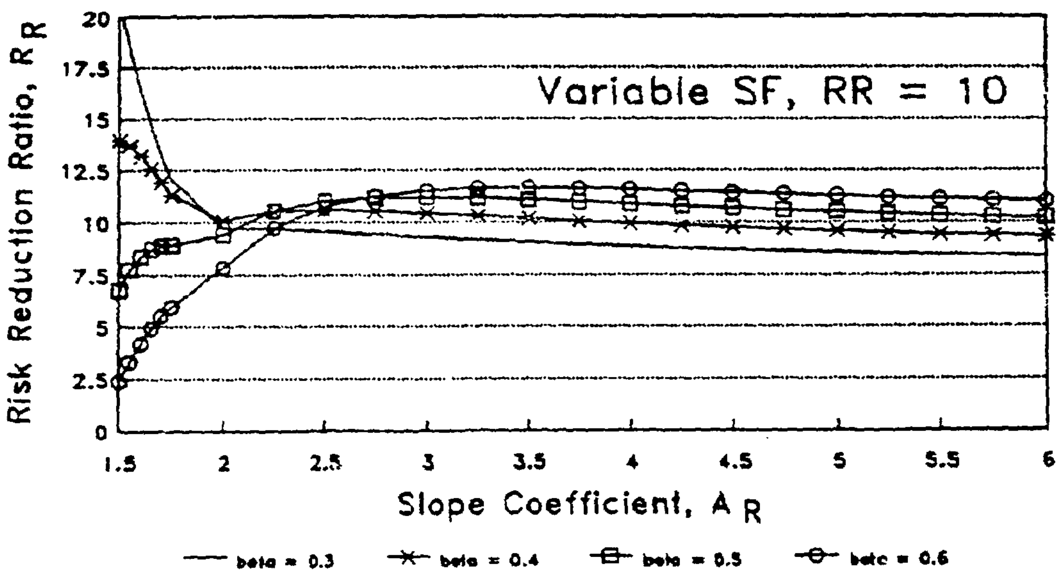


Figure C-7 Value of R_R vs A_R for Variable SF (Fig. C-5 for PC 4)

For sites near tectonic plate boundaries for which A_R is in the range of about 1.5 to 2.25, such as LLNL, SNL-Livermore, SLAC, LBL, and ETEC. Figures C-3a and C-4a demonstrate that larger risk reduction ratios are achieved than the target levels of 5 for PC 3 and 10 for PC 4, respectively. Therefore, it is acceptable to use twice the hazard probabilities for these sites combined with the appropriate constant scale factors. Hence, for sites near tectonic plate boundaries, target performance goals may be adequately achieved with hazard probabilities and seismic scale factors of 1×10^{-3} and 1.0 for PC 3 and 2×10^{-4} and 1.25 for PC 4.

C.3 Seismic Design/Evaluation Input

The seismic performance goals presented in Tables C-1 and C-2 are achieved by defining the seismic hazard in terms of a site-specified design response spectrum (called herein, the Design/Evaluation Basis Earthquake, [DBE]). Either a site-specific design response spectrum specifically developed for the site, or a generic design response spectrum that is appropriate or conservative for the site may be used as the site-specified design response spectrum. Probabilistic seismic hazard estimates are used to establish the DBE. These hazard curves define the amplitude of the ground motion as a function of the annual probability of exceedance P_H of the specified seismic hazard.

For each performance category, an annual exceedance probability for the DBE, P_H is specified from which the maximum ground acceleration (or velocity) may be determined from probabilistic seismic hazard curves. Evaluating maximum ground acceleration from a specified annual probability of exceedance is illustrated in Figure C-8. Earthquake input excitation to be used for design and evaluation by these provisions is defined by a median amplification smoothed and broadened design/evaluation response spectrum shape such as that shown in Figure C-8 anchored to this maximum ground acceleration. Note that the three spectra presented in Figure C-8 are identical; the top spectrum has spectral acceleration plotted against natural frequency on a log scale, the middle spectrum is on what is termed a tripartite plot where spectral velocities and displacements as well as accelerations are shown, and the bottom spectrum has spectral acceleration plotted against natural period on a linear scale.

It should be understood that the spectra shown in Figure C-8 represent inertial effects. They do not include relative or differential support motions of structures, equipment, or distribution systems supported at two or more points typically referred to as seismic anchor motion (SAM). While SAM is not usually applicable to building design, it might have a significant effect on seismic adequacy of equipment or distribution systems.

Seismic design/evaluation criteria based on target probabilistic performance goals requires that Design/Evaluation Basis Earthquake (DBE) motions be based on probabilistic seismic hazard assessments. In accordance with DOE Order 420.1 and the associated NPH Implementation Guide (Refs. C-27 and C-67), it is not required that a site-specific probabilistic seismic hazard assessment be conducted if the site includes only Performance Category 2 and lower SSCs. If such an assessment has not been performed, it is acceptable to determine seismic loads (as summarized in Section C.3.2.2) from the larger of those determined in accordance with the UBC (Ref. C-2) and with UCRL-53582, Rev. 1 (Ref. C-14). Design/evaluation earthquake ground motion determined from a recent site-specific probabilistic seismic hazard assessment is considered to be preferable to the UBC for determining ZC. Therefore, the DBE response spectrum for Performance Category 2 and lower may be developed from a new probabilistic seismic hazard assessment following the guidance given herein for Performance Category 3 and higher. However, when design/evaluation earthquake ground motion is based on recent site-specific geotechnical studies and the resulting seismic loads are less than that determined by the UBC, the differences must be justified and approval of seismic loads must be obtained from DOE.

For design or evaluation of SSCs in Performance Category 3 and higher, it is strongly recommended that a modern site-specific seismic hazard assessment be performed to provide the basis for DBE ground motion levels and response spectra. DOE Order 420.1 and the associated NPH Implementation Guide (Refs. C-27 and C-67), require that the need for updating the site seismic hazard assessment be reviewed at least every 10 years. The DOE seismic working group interim standard, DOE-STD-1024-92 (Ref. C-13), indicates that the approach used for the seismic hazard assessments summarized in UCRL-53582 (Ref. C-14), which are more than 10 years old, are out of date relative to the current state of the art. However, in accordance with DOE-STD-1024-92, it is permissible to establish DBE ground motion levels and response spectra for Performance Categories 3 and 4 based on UCRL-53582 in the interim until a modern site-specific seismic hazard assessment becomes available. DBE ground motion levels for Performance Categories 3 and 4 based on UCRL-53582 are also provided in Section C.3.2.2.

Minimum values of the DBE are provided in Section 2.3 to assure a minimum level of seismic design at all DOE sites. Such a minimum level of seismic design is believed to be necessary due to the considerable uncertainty about future earthquake potential in the lower seismicity regions of the United States where most DOE sites are located.

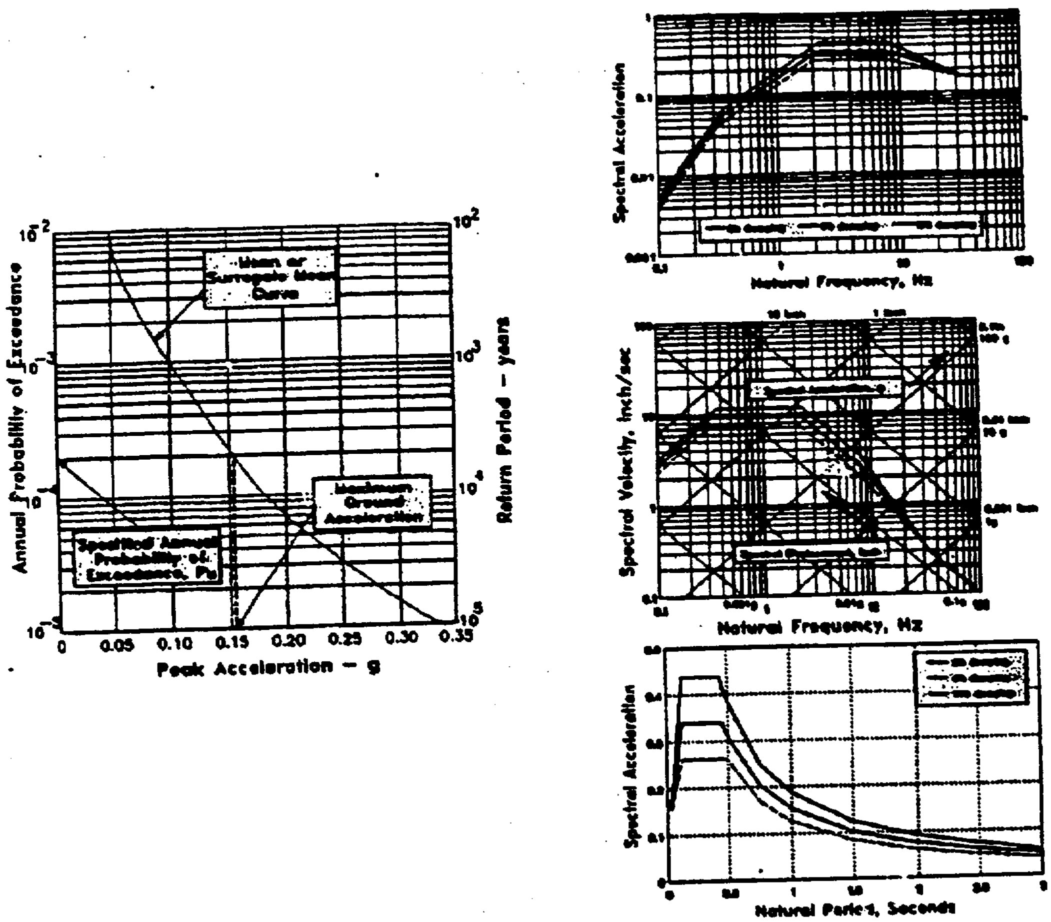


Figure C-8 Earthquake Input Excitation is Defined by Maximum Ground Acceleration Anchoring Site-Specific Response Spectra

C.3.1 Earthquake Hazard Annual Exceedance Probabilities

Historically, non-Federal Government General Use and Essential or Low Hazard facilities located in California, Nevada, and Washington have been designed for the seismic hazard defined in the Uniform Building Code. Other regions of the U.S. have used the UBC seismic hazard definition, other building code requirements, or have ignored seismic design. Past UBC seismic provisions (1985 and earlier) are based upon the largest earthquake intensity that has occurred in a given region during about the past 200 years. These provisions do not consider the probability of occurrence of such an earthquake and thus do not make any explicit use of a probabilistic seismic hazard analysis. However, within the last 15 years there have been developments in building codes in which the seismic hazard provisions are based upon a consistent annual probability of exceedance for all regions of the U.S. In 1978, ATC-3 provided probabilistic-based seismic hazard provisions (Ref. C-1). From the ATC-3 provisions, changes to the UBC (Ref. C-2) and the development of the National Earthquake Hazards Reduction Program (NEHRP, Ref. C-3) have resulted. A probabilistic-based seismic zone map was incorporated into the UBC beginning with the 1988 edition. Canada and the U.S. Department of Defense have adopted this approach (Refs. C-4 and C-5). The suggested annual frequency of exceedance for the design seismic hazard level differs somewhat between proposed codes, but all lie in the range of 10^{-2} to 10^{-3}. For instance, UBC (Ref. C-2), ATC-3 (Ref. C-1), and NEHRP (Ref. C-3) have suggested that the design seismic hazard level should have about a 10 percent frequency of exceedance level in 50 years which corresponds to an annual exceedance frequency of about $2x10^{-3}$. The Canadian building code used $1x10^{-2}$ as the annual exceedance level for their design seismic hazard definition. The Department of Defense (DOD) tri-services seismic design provisions for essential buildings (Ref. C-5) suggests a dual level for the design seismic hazard. Facilities should remain essentially elastic for seismic hazard with about a 50 percent frequency of exceedance in 50 years or about a $1x10^{-2}$ annual exceedance frequency, and they should not fail for a seismic hazard which has about a 10 percent frequency of exceedance in 100 years or about $1x10^{-3}$ annual exceedance frequency.

On the other hand, nuclear power plants are designed so that safety systems do not fail if subjected to a safe shutdown earthquake (SSE). The SSE generally represents the expected ground motion at the site either from the largest historic earthquake within the tectonic province within which the site is located or from an assessment of the maximum earthquake potential of the appropriate tectonic structure or capable fault closest to the site. The key point is that this is a deterministic definition of the design SSE. Recent probabilistic hazard studies (e.g., Ref. C-6) have indicated that for nuclear plants in the eastern U.S., the design SSE level generally corresponds to an estimated annual frequency of exceedance of between $0.1x10^{-4}$ and $10x10^{-4}$ as is illustrated in Figure C-9. The probability level of SSE design spectra (between 5 and 10 hz) at the 69 eastern U.S. nuclear power plants considered by Ref. C-6 fall within the above stated range. Figure C-9 also demonstrates that for 2/3 of these plants the SSE spectra corresponds to probabilities between about $0.4x10^{-4}$ and $2.5x10^{-4}$. Hence, the specified hazard probability level of $1x10^{-4}$ in this standard is consistent with SSE levels.

These seismic hazard definitions specified in this standard are appropriate as long as the seismic design or evaluation of the SSCs for these earthquake levels is conservatively performed. The level of conservatism of the evaluation for these hazards should increase

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as one goes from Performance Category 1 to 4 SSCs. The conservatism associated with Performance Categories 1 and 2 should be consistent with that contained in the UBC (Ref. C-2), ATC-3 (Ref. C-1), or NEHRP (Ref. C-3) for normal or essential facilities, respectively. The level of conservatism in the seismic evaluation for Performance Category 4 SSCs should approach that used for nuclear power plants when the seismic hazard is designated as shown above. The criteria contained herein follow the philosophy of a gradual reduction in the annual exceedance probability of the hazard coupled with a gradual increase in the conservatism of the evaluation procedures and acceptance criteria as one goes from Performance Category 1 to Performance Category 4.

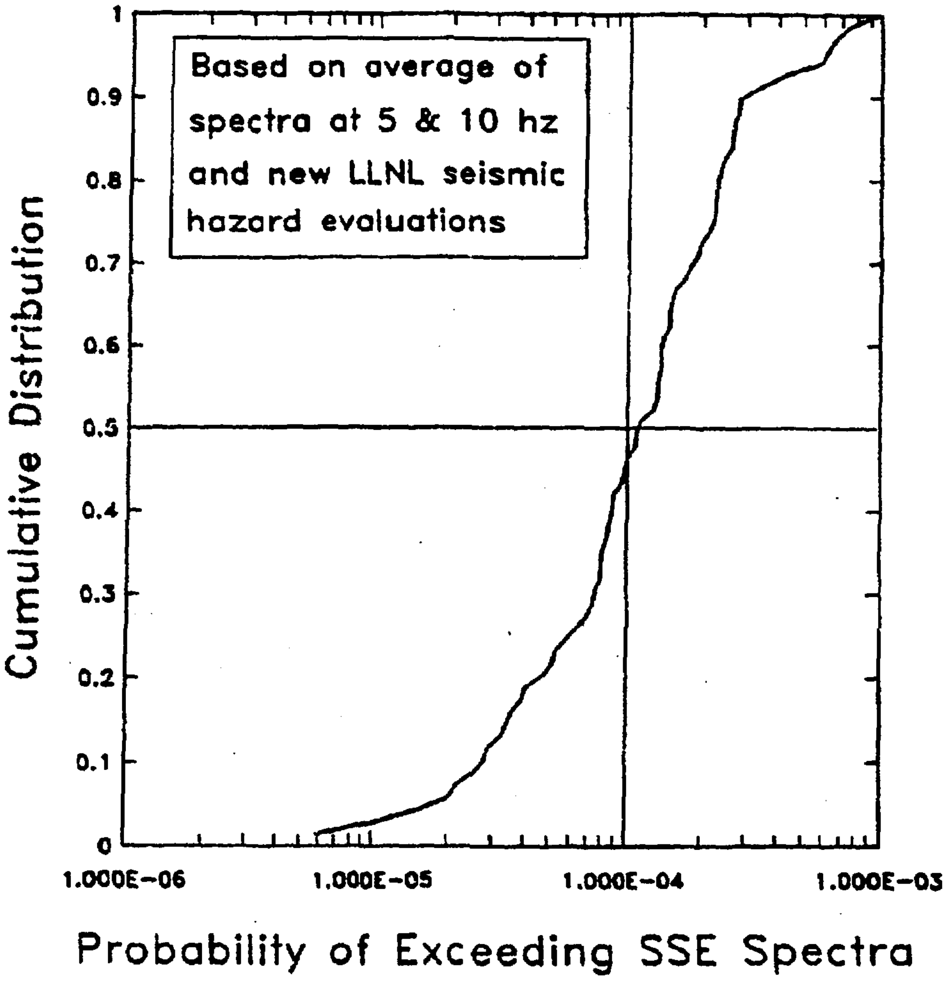

Figure C-9 Probability of Exceeding SSE Response Spectra

EXHIBIT 3

Topical Report
YMP/TR-003-NP

Yucca Mountain Site Characterization Project

TOPICAL REPORT YMP/TR-003-NP

PRECLOSURE SEISMIC DESIGN METHODOLOGY FOR A GEOLOGIC REPOSITORY AT YUCCA MOUNTAIN

Revision 2

August 1997

U.S. Department of Energy
Office of Civilian Radioactive Waste Management
North Las Vegas, NV 89036

Topical Report : Preclosure Seismic Design Methodology for a Geologic Repository at Yucca Mountain

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1.2 CONTENT OF THE SEISMIC TOPICAL REPORTS

This topical report is the second in a series of three reports that the DOE has planned that together will describe the preclosure seismic design process. The relationship of the three topical reports is illustrated in Figure 1-1. Topical Report I, *Methodology to Assess Fault Displacement and Vibratory Ground Motion Hazards at Yucca Mountain* (DOE 1994a), describes the DOE methodology for assessing vibratory ground motion and fault displacement hazards. Topical Report II (this report) describes the DOE preclosure seismic design methodology and design acceptance criteria and establishes seismic hazard levels that are appropriate for design. The DOE anticipates that a third report, currently scheduled for fiscal year 1998, will describe the results of the assessment of the vibratory ground motion and fault displacement hazards at Yucca Mountain and the determination of the appropriate design bases for these hazards.

The content of the three seismic reports is described in more detail in the following paragraphs.

Topical Report I--Topical Report I describes the DOE methodology for probabilistic assessment of vibratory ground motion and fault displacement hazards. The methodology involves a series of workshops structured so that multiple experts can interact to evaluate hypotheses and models using the Yucca Mountain site and area geological, geophysical, and seismological data sets. The data sets will be made available to all participant experts uniformly. Importantly, the methodology requires that the experts specifically evaluate all hypotheses and models that have credible support in the data. The product of the methodology is multiple interpretations by the experts of seismic sources, source properties, and evaluations of ground motion, all of which include specific expressions of uncertainty. The methodology does not involve expert opinion, which implies judgments unconstrained by data or normal scientific rigor, but instead employs normal earth science procedures and practice, and carries the usual past practice one step further by requiring uncertainty in the interpretations to be specifically expressed. Moreover, it forces a consistent level of scientific rigor, a comprehensive and consistent consideration of data, and documentation of all interpretations.

Additional information on the methodology is contained in *Probabilistic Analyses of Ground Motion and Fault Displacement at Yucca Mountain*, Yucca Mountain Study Plan 8.3.1.17.3.6 (DOE 1995a).

Topical Report I does not provide the values of vibratory ground motion and fault displacement hazards for design of the facility SSCs; it describes only the methodology for hazard assessment. The application of this methodology at the Yucca Mountain site will yield hazard estimates that will, together with planned deterministic evaluations, comprise the information base considered in determining preclosure design basis vibratory ground motion and fault displacement values. The hazard estimates will also be used in the assessment of postclosure waste containment and isolation performance.

Topical Report II--Topical Report II (this report) describes the design methodology and criteria that the DOE intends to implement to provide reasonable assurance that vibratory ground motions and fault displacements will not compromise the preclosure safety functions of SSCs important to safety. The seismic design methodology and criteria implement the requirements of 10 CFR 60, including the requirement in the recent ruling (61 FR 64257) to identify Category-1 and -2 design basis events. This report establishes hazard probability levels that are appropriate for determining the two levels of design basis vibratory ground motions and the two levels of design basis fault displacements. Acceptance criteria for both surface and underground facilities are provided for vibratory ground motion and fault displacement design. In addition, the report provides criteria for fault avoidance, which is the DOE preferred approach to mitigating fault displacement hazards. Seismic design considerations for waste packages, which will function on the surface and underground and which have a number of unique performance requirements, are discussed. NRC guidance documents for the seismic design of nuclear power reactors that can appropriately be applied to preclosure seismic design of the repository are identified.

Topical Report III--A third seismic topical report is planned for completion in fiscal year 1998. The DOE intends to conduct and document the probabilistic seismic hazard assessment during fiscal year 1997 using the methodology of Topical Report I. Using the results of the hazard assessment, preclosure seismic design inputs will be developed and documented in a Seismic Design Report, which is scheduled for the second quarter of fiscal year 1998. The third topical report would document the results of both of these efforts for formal NRC staff review.

It is expected that seismic design inputs will be determined from controlling earthquakes identified from a disaggregation of the probabilistic seismic hazard results and from a consideration of deterministic hazard assessments. Disaggregation of the hazard results will be carried out for hazard exceedance probability levels established in Topical Report II and for ground motion frequencies of interest. Different earthquakes may control the hazard in different frequency ranges. Ground motions from the controlling earthquakes will be evaluated deterministically.

In addition to conducting the probabilistic hazard assessment, the DOE intends to perform deterministic evaluations of Type I faults and candidate Type I faults that lie within 5 km of the Yucca Mountain site, including estimations of maximum earthquake magnitudes for the faults. The DOE intends to evaluate where the hazards from these deterministic evaluations fall within the probabilistic results. This comparison will provide a check on the reasonableness of the vibratory ground motion and fault displacement design bases.

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Topical Report : Preclosure Seismic Design Methodology for a Geologic Repository at Yucca Mountain

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3.0 DESIGN OF STRUCTURES, SYSTEMS, AND COMPONENTS FOR VIBRATORY GROUND MOTION

This section presents and rationalizes the reference exceedance probabilities that the DOE plans to use in identifying Frequency-Category-1 and -2 design basis vibratory ground motions. It then discusses the design acceptance criteria that the DOE plans to apply in the preclosure seismic design of structures, systems and components (SSCs) that are important to safety. Design acceptance criteria are discussed specifically for SSCs on the ground surface, for underground openings, and for other underground SSCs.

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3.1 HAZARD LEVELS FOR DESIGN BASIS GROUND MOTIONS

In accordance with the recent 10 CFR 60 rulemaking discussed in Section 2.1.1, the DOE will identify SSCs that are important to (radiological) safety. The DOE procedure for identifying these SSCs is summarized in Appendix B. The classification process involves the identification of Frequency-Category-1 and Frequency-Category-2 design basis events and event-initiated accident scenarios and the calculation of corresponding exposures to workers and the public. The calculated exposures are compared to regulatory limits, and any SSC that must continue to function after a design basis event to ensure the exposure limits are not exceeded is classified as important to safety. No SSCs have yet been classified. Note that SSCs may be important to safety for both Frequency-Category-1 and Frequency-Category-2 design basis events. Where this occurs, the most stringent (i.e., Frequency-Category-2) design basis will apply.

The regulatory definitions of Category-1 and -2 design basis events are qualitative descriptions of the likelihood of occurrence before permanent closure of the geologic repository operations area. For use in SSC classification, which requires knowledge of the design basis events and calculation of radiation exposures, these definitions require quantitative interpretations. As discussed next, the DOE intends to use mean annual exceedance probabilities of 1.0E-03 and 1.0E-04, respectively, as reference values in determining the Frequency-Category-1 and -2 design basis vibratory ground motions. These reference values will be used in the disaggregation of probabilistic seismic hazard estimates to identify those earthquakes that control the seismic hazard at the reference probabilities. The identification of controlling earthquakes and the DOE determination of the design basis ground motions are planned to be detailed in the third seismic topical report.

3.1.1 Frequency-Category-1 Reference Probability

The DOE intends to use a reference mean annual probability of exceedance of 1.0E-03 in determining the Frequency-Category-1 design basis ground motion. The DOE considers that this probability, which corresponds to a 1,000-year return period, represents a conservative quantitative translation of the qualitative frequency description for Category-1 design basis events in the revised 10 CFR 60, i.e., "events that are reasonably likely to occur regularly, moderately frequently, or one or more times before permanent closure of the geologic repository operations area." Assuming a Poisson temporal occurrence model (see Section 3.3.2.2), events with a 1.0E-03/yr recurrence rate would have an 86 percent chance of not occurring, a 13 percent chance of occurring once, and a 1 percent chance of occurring twice in 150 years. For facilities with a 100-year design lifetime, events with this recurrence rate would have a 90 percent chance of not occurring, a 9 percent chance of occurring once, and a 0.4 chance of occurring twice.

An annual occurrence rate of 1.0E-03 for Frequency-Category-1 design basis ground motions are more conservative than what is required by model building codes for ordinary structures, in terms of the annual probability of occurrence of the design basis earthquake, and is comparably conservative in terms of the probability of occurrence during the facility lifetime. The Uniform Building Code (ICBO 1994) and the National Earthquake Hazards Reduction Program (BSSC 1995) both recommend using peak ground motion values that have a 90 percent chance of not being exceeded in 50 years for the life-safety seismic design of new buildings; this corresponds to a return period of about 500 years. DOE Standard 1020-94 (DOE 1994b) is not being applied to the mined geologic disposal system program, but it documents a general DOE policy that a 500-year return period is to be used in establishing design basis ground motions for general facilities. This return period corresponds to an annual exceedance probability of about 2.0E-03 and a 90 percent chance of not occurring during a typical 50-year facility lifetime.

3.1.2 Frequency-Category-2 Reference Probability

For Frequency-Category-2 design basis ground motion, the DOE intends to use a reference mean annual exceedance probability of 1.0E-04. The DOE considers that this mean value is appropriate and conservative based on the observations that (1) it is comparable to the mean exceedance probabilities of the seismic design bases of operating nuclear power reactors in the United States, (2) these accepted reactor design bases and their associated design-acceptance criteria have resulted in acceptably safe seismic designs, (3) design acceptance criteria will be used in repository design that are the same as or comparable to those used in reactor designs, and (4) an operating mined geologic disposal system is inherently less hazardous and less vulnerable to earthquake-initiated accidents than is an operating nuclear power reactor.

3.1.2.1 Comparison with Nuclear Power Reactor Seismic Design Bases

In Regulatory Guide 1.165 (NRC 1997) NRC staff states that a reference median annual exceedance probability of 1.0E-05 will be acceptable for use in determining the safe shutdown earthquake for new nuclear power reactors. The cited rationale for this reference probability is that it is the annual probability level such that 50 percent of a set of currently operating plants (selected by the NRC) has an annual median probability of exceeding the safe shutdown earthquake that is below this level. In other words, 1.0E-05 is the median of the distribution of median exceedance probabilities. The selected plants represent relatively recent designs that used design response spectra in accordance with Regulatory Guide 1.60, *Design Response Spectra for Seismic Design of Nuclear Power Plants* (AEC 1973), or similar spectra. All of the plants selected are located in the central or eastern United States (CEUS). Regulatory Guide 1.165 provides an option for the applicant to use a different reference probability, to be reviewed and accepted on a case-by-case basis, considering the slope of the site-specific hazard curve, the overall uncertainty in hazard estimates, including differences between mean and median hazard estimates, and knowledge of the seismic sources that contribute to the hazard.

In developing Regulatory Guide 1.165, NRC staff considered whether to define the reference probability as a mean or median value. The mean value has the advantage of better reflecting the uncertainty in the seismic hazard evaluation (i.e., it is sensitive to the range of interpretations of seismic source zone configurations, earthquake magnitude recurrence relationships, and ground motion attenuation relationships). However, precisely because the median is less sensitive to uncertainties, it provides a more stable regulatory benchmark than does the mean. Another consideration leading to the staff's preference for the median was the finding that, when median hazard curves were disaggregated, the magnitudes and distances of the controlling earthquakes tended to be more sharply defined and to agree better with the safe shutdown earthquakes of the selected plants than when mean hazard curves were disaggregated (Bernreuter et al. 1996).

For the reasons discussed next, the DOE plans to use mean, rather than median, target annual exceedance probabilities in establishing design basis vibratory ground motions.

To identify the earthquakes that control the Frequency-Category-2 design basis ground motion, the DOE plans to use a mean annual exceedance probability of 1.0E-04. NRC-sponsored research has shown that a mean value of 1.0E-04 corresponds to a median value of 1.0E-05 at sites in the CEUS (NRC 1994b). That is, while 1.0E-05 is the median of the distribution of median exceedance probabilities of the safe shutdown earthquakes of the more recently designed nuclear power reactors in the CEUS, 1.0E-04 is the median of the distribution of means. So, 50 percent of the nuclear power reactors in the selected set have an annual mean probability of exceeding the safe shutdown earthquake that is below this level. Thus, using a mean value of 1.0E-04 to determine the safe shutdown earthquake for a new nuclear power reactor in the CEUS would be risk-consistent with using a median value of 1.0E-05.

In contrast to sites in the CEUS, the equivalency of 1.0E-04 mean and 1.0E-05 median annual probabilities of exceedance does not generally hold in the western United States and is not expected to hold at Yucca Mountain. Because the distributions of probabilistic seismic hazard estimates typically are skewed about the median towards higher probability levels, mean exceedance probabilities usually are greater than median probabilities, and the greater the uncertainty (i.e., spread of the distribution of hazard curves), the greater the difference between the mean and median values. This fact, together with the fact that the uncertainty in seismic hazard evaluations is almost always greater at CEUS sites than at western sites, indicates that mean values normally are closer to median values at western sites than at CEUS sites. Thus, if one were siting a nuclear power reactor at a typical western U.S. site, choosing a mean annual exceedance probability of 1.0E-04 would be consistent with the mean hazard levels associated with the seismic design bases of more recently designed power reactors in the CEUS, but choosing a median annual probability of 1.0E-05 would not be.

As a further check on the reasonableness of using a mean annual exceedance probability of 1.0E-04 as the reference probability for determining the Frequency-Category-2 design basis ground motion, the DOE compiled published probabilistic seismic hazard estimates for the sites of nuclear power plants in the western United States. The objective of the compilation was to determine whether a mean exceedance probability of 1.0E-04/yr is representative of the accepted seismic design response spectra of these plants, as it is for the more recently designed power plants in the CEUS.

Because the shapes of design response spectra rarely match the shapes of uniform hazard spectra, the probabilities of exceeding design response spectra vary with frequency. Therefore, an averaging convention is required to associate a single probability of exceedance with each design response spectrum. To assure comparability of results, this study used the same convention that was used in the study of CEUS plants (NRC 1994b) and that is recommended in Regulatory Guide 1.165 (NRC 1997), i.e., the average of the exceedance probabilities at 5 Hz and 10 Hz[1].

Footnote [1] There is no tacit assumption here that the 5 to 10 Hz frequency range is representative of the natural frequencies of SSCs in a repository. Repository design response spectra will be developed that cover a broad frequency range from 0.33 Hz to more than 20 Hz.

The power plants for which information was compiled are the Diablo Canyon Power Plant (Units 1 and 2) in Port San Luis, California; Palo Verde Nuclear Generating Station (PVNGS) in Wintersburg, Arizona; San Onofre Nuclear Generating Station (Units 2 and 3) in Southern California; Washington Nuclear Plant 2 near Hanford, Washington; and Washington Nuclear Plant 3 at Satsop, Washington. All of these power reactors are currently operating, with the exception of Washington Nuclear Plant 3, which was only partially constructed and which has now been canceled. It is included in this analysis because its seismic design basis was completed and accepted provisionally by NRC staff (NRC 1991a).

Results of the compilation are presented in Appendix C. As shown there the estimated mean annual probability of exceeding the safe shutdown earthquake of each western plant is greater than 1.0E-04/yr, with the single exception of the PVNGS, which is located in a low-seismic-hazard region. The average mean annual probability of exceeding the safe shutdown earthquake of each plant is 2.0E-04, which is twice the value of the reference probability to be used in determining the Frequency-Category-2 design basis ground motion.

3.1.2.2 Conservatism of the Frequency-Category-2 Reference Probability

As noted earlier, the use of NRC-accepted seismic design bases for nuclear power reactors as a benchmark for Frequency-Category-2 design basis ground motion is based on the premise that reactor design bases correspond to acceptable seismic risk levels. The seismic design bases of all nuclear power reactors operating in the United States have been reviewed extensively by NRC staff, using standardized review criteria, and all have been found to satisfy applicable regulatory requirements by NRC licensing boards. In addition, a substantial body of recently developed information indicates that these plants have adequate margins of safety against potential accidents and that they have acceptably safe seismic designs. In June 1991 the NRC requested that its nuclear power reactor licensees perform a plant-specific Individual Plant Examination of External Events (IPEEE) to identify vulnerabilities, if any, to earthquakes, fires, winds, floods, and nearby transportation and other-facility accidents (NRC 1991b). The IPEEE program corroborated the adequacy of the seismic design bases of the Nation's operating nuclear power reactors. For example, specific IPEEE findings for operating reactors in the western United States were as follows:

- In the IPEEE study of the Diablo Canyon Power Plant, Pacific Gas and Electric Company found that the mean core damage frequency due to external events is about 6.7E-05/yr (PG&E 1994) . The component of this risk due to earthquake-initiated accident scenarios was estimated to be 4.0E-05/yr.

- The PVNGS is located in Wintersburg, Arizona, and is operated by the Arizona Public Service Company (APS). The PVNGS site is in a region of low seismic hazard relative to most other regions of the western United States; the PVNGS horizontal design basis response spectrum is anchored at 0.25 g peak ground acceleration (APS 1988). Given the relatively low seismic hazard, APS successfully persuaded NRC staff to have the PVNGS review-level earthquake reduced from 0.5 g (NRC 1991b) to 0.3g. APS elected to conduct a seismic margins analysis for the IPEEE program, rather than a seismic risk assessment. The margins analysis found that at least one safe-shutdown path exists for a peak horizontal ground acceleration in excess of 0.3 g (APS 1995).

- The IPEEE study conducted by Southern California Edison (SCE 1995) for the San Onofre Nuclear Generating Station found that the mean core damage frequency due to external-event initiators is approximately 3.3E-05/yr. The component of this risk due to earthquake-initiated accident scenarios was estimated to be about 1.7E-05/yr.

- In the IPEEE study of the Washington Nuclear Plant 2, the Washington Public Power Supply System (WPPSS 1995) estimated that the mean core damage frequency due to external-event initiators is 2.1E-05/yr and that this risk is dominated by the seismic contribution.

The conservatism of 1.0E-04/yr as a target exceedance probability for the Category-2 design basis ground motion also is based on an assumption that repository design acceptance criteria will reduce the probability of a severe seismically initiated accident below the probability of the design basis ground motions by a "risk-reduction" factor that is comparable to or greater than the factor that is provided by the design acceptance criteria for power reactors. This assumption itself has two bases. The first basis is that the DOE intends to use design acceptance criteria that are the same as or comparable to those used in reactor designs. The DOE has evaluated the NRC standard review plans for the seismic design of nuclear power reactors and has determined that many of the acceptance criteria are applicable to the design of repository surface facilities (see Section 3.2). These facilities are anticipated to include the majority of SSCs important to safety. Acceptance criteria for underground facilities are detailed in Sections 3.3 and 3.4 of this report. The second basis is that a repository is inherently less hazardous and less vulnerable to seismic shaking (or fault displacement) than is an operating nuclear power reactor. As noted by the NRC in the Section-by-Section Analysis of Section 60.136, *Preclosure Controlled Area*, in the Supplementary Information published with the final rule for 10 CFR 60 (61 FR 64257):

> ". . . in comparison with a nuclear power plant, an operating repository is a relatively simple facility in which the primary activities are in relation to waste receipt, handling, storage, and emplacement. A repository does not require the variety and complexity of systems necessary to support an operating nuclear power plant. Further, the conditions are not present at a repository to generate a radioactive source term of a magnitude that, however unlikely, is potentially capable at a nuclear power plant (e.g., from a postulated loss of coolant event). As such, the estimated consequences resulting from limited source term generation at a repository would be correspondingly limited."

In summary, use of a mean annual probability of exceedance of 1.0E-04 as a reference probability for the Frequency-Category-2 vibratory ground motion is quite conservative. This probability is comparable to the probabilities of exceeding the accepted seismic design bases of more recently designed operating nuclear power reactors in the CEUS. A compilation of the mean annual exceedance probabilities of the safe shutdown earthquakes of nuclear power reactors in the western United States indicates that the average mean exceedance probability for this set of reactors exceeds 1.0E-04 by about a factor of two. The DOE considers that use of this value for the preclosure seismic design of the geologic repository operations area is very conservative, given that a repository is inherently less hazardous and less vulnerable to seismic shaking than is an operating nuclear power reactor. The seismic safety of the operating power reactors and, by extension, the adequacy of their seismic design bases, has been confirmed by in-depth, site-specific analyses conducted under the IPEEE program.

3.1.3 Use of Reference Probabilities in Establishing Design Response Spectra

The DOE intends to establish design response spectra that correspond to the Frequency-Category-1 and -2 reference probabilities in a manner similar to that described in Regulatory Guide 1.165 (NRC 1997). This is done by first disaggregating the hazard results to identify the magnitudes and distances of earthquakes that control the hazard at frequencies of engineering interest. Controlling earthquakes will be identified for both of the reference mean annual exceedance probabilities, 1.0E-03 (Frequency Category 1) and 1.0E-04 (Frequency Category 2). Site-specific response spectra will be developed for these controlling earthquakes and will be scaled by the hazard at the reference probability level, at one or more specified frequencies. Finally, smooth design response spectra will be developed that envelope the controlling-earthquake response spectra and that provide sufficient energy over the frequency range of significance to repository SSCs. The details of this process will be developed as part of the development of the repository seismic design and will be fully described in the third seismic topical report.

3.1.4 Use of Reference Probabilities for Other Types of Events

The 10 CFR 60.2 defines Category 1 design basis events as "those natural and human-induced events that are reasonably likely to occur regularly, moderately frequently, or one or more times before permanent closure of the geologic repository operations area," and Category 2 design basis events as "other natural and man-induced events that are considered unlikely, but sufficiently credible to warrant consideration, taking into account the potential for significant radiological impacts on public health and safety." The DOE interprets the frequencies of Frequency Category 1 events (using the DOE's terminology) to be one every 100 years for infrastructure systems (ventilation, surface facilities, etc.) and one every 150 years for ground support systems; events with frequencies less than these values but greater than one every million years are interpreted to be Frequency Category 2 events. This interpretation is consistent with the NRC's statement (61 FR 64257) that the upper probability bound for Category 2 design basis events is roughly 1.0E-02 per year and the lower bound is on the order of 1.0E-06 per year. To ensure conservatism and consistency in the preclosure repository seismic design, the DOE has adopted lower probability levels for design basis seismic loads, as noted above (i.e., annual probabilities of 1.0E-03 and 1.0E-04 for Frequency-Category-1 and -2 vibratory ground motions, respectively, and 1.0E-04 and 1.0E-05 for Frequency-Category-1 and -2 fault displacements, respectively).

The reference probabilities proposed here for seismic loads are not intended to be applicable to other types of design basis external events such as severe winds, fires, or floods, or to design basis internal events. The probabilities for seismic loads are based on professional practice in seismic design, engineering judgment, and industry-wide experience in the licensing of nuclear power reactor seismic designs. Other criteria can be expected to apply to other types of design basis events, considering the degree of uncertainty in characterizing the frequency and severity of events; the potential consequences of exceeding design basis events; the incremental cost of increasing the basis for design; the methodology used to identify the design basis events; and established standards, codes, guidelines, and professional practices.

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APPENDIX C
PROBABILITIES OF EXCEEDING THE SEISMIC DESIGN BASES OF NUCLEAR POWER PLANTS IN THE WESTERN UNITED STATES

This appendix documents estimated composite mean annual probabilities of exceeding the accepted seismic design bases of nuclear power plants in the western United States. Specifically, it documents the exceedance probabilities and seismic design bases of the Diablo Canyon Power Plant (DCPP), the Palo Verde Nuclear Generating Station (PVNGS), the San Onofre Nuclear Generating Station (SONGS) (Units 2 and 3), the Washington Nuclear Plant 2 (WNP-2), and the Washington Nuclear Plant 3 (WNP-3). With the exception of WNP-3, these are currently operating plants whose seismic design bases were reviewed and accepted by U.S. Nuclear Regulatory Commission (NRC) staff, then adjudicated and found acceptable by NRC licensing boards. WNP-3 was partially constructed and then canceled, but its seismic design basis was reviewed and provisionally accepted by NRC staff. The composite exceedance probability is defined as the arithmetic average of the probabilities of exceeding the design basis response spectral ordinates at 5 and 10 Hz, for 5% damping. This convention for the composite exceedance probability is that used by Lawrence Livermore National Laboratory (LLNL) in evaluating the exceedance probabilities of the seismic design bases of a selected set of nuclear power reactors in the central and eastern United States (NRC 1994b). The convention is used here to assure that the results of the compilation can be compared directly to the LLNL results[1]. Mean, rather than median, probabilities are used for the reasons discussed in Section 3.1.2.1 of this report.

Footnote [1] There is no tacit assumption here that the 5-10 Hz frequency range is representative of the natural frequencies of SSCs in a repository. Repository design response spectra will be developed that account for the vibratory ground motion hazard over a broad frequency range, 0.33 Hz to 20 Hz.

Plant-specific evaluations are presented in the following sections.

C.1 DIABLO CANYON POWER PLANT

The two units of the DCPP in Port San Luis, California, are operated by the Pacific Gas and Electric Company (PG&E). PG&E conducted a probabilistic seismic hazard analysis of the DCPP site under its Long Term Seismic Program (PG&E 1988). Results of this study were input to the seismic risk assessment that PG&E conducted for the Individual Plant Examination of External Events (IPEEE) program, described in Section 3.1.2.2 of this report. The Long Term Seismic Program estimated the probability of exceeding the average spectral accelerations at the DCPP site over the 3- to 8.5-Hz frequency range. The mean hazard results, taken from the IPEEE report (PG&E 1994) are shown in Figure C-1.

The accepted earthquake for the seismic evaluation of Diablo Canyon Units 1 and 2 is a postulated magnitude 7.5 event on the Hosgri fault, generating a peak horizontal acceleration at the site of 0.75 g. Both Blume and Newmark standard design spectral shapes were "anchored to" this peak acceleration value to provide a basis for evaluating the DCPP seismic design, as shown in Figures C-2 and C-3 (PG&E 1985). From these figures, the 3- to 8.5-Hz (0.12- to 0.33-sec period) average spectral acceleration for 5% damping for both spectral shapes is approximately 2.0 g.

Referring to Figure C-1, the estimated mean probability of exceeding 2.0 g for the 3- to 8.5-Hz average spectral acceleration is approximately 1.7E-04/yr. Exceedance probabilities for the spectral values at 5 and 10 Hz are not available. However, the calculated average probability of exceeding the DCPP design spectra at 5 and 10 Hz should differ little from the calculated probability of exceeding the 3- to 8-Hz average acceleration. Therefore, it is estimated that the 5-and 10-Hz composite (average) mean exceedance probability for the DCPP is about 1.7E-04/yr, and it can safely be concluded that it is greater than 1.0E-04/yr.

C.2 PALO VERDE NUCLEAR GENERATING STATION

The PVNGS in Wintersburg, Arizona, is operated by the Arizona Public Service Company (APS). The PVNGS site is in a region where the seismic hazard is lower than in most other regions of the western United States, and the free-field horizontal design-basis response spectrum for the PVNGS is anchored at 0.25-g peak ground acceleration (APS 1988). The free-field, horizontal-component design response spectra for the PVNGS are shown in Figure C-4. From this figure, it is estimated the design-basis spectral velocities at 5 Hz and 10 Hz, for 5% damping, are 7.5 in/sec (17.55 cm/sec) and 3.2 in/sec (7.5 cm/sec), respectively.

Risk Engineering, Inc. (1993), conducted a probabilistic seismic hazard evaluation of the PVNGS site. The resulting seismic hazard curves for 5-Hz and 10-Hz spectral velocities (5% damping) are shown in Figures C-5 and C-6. Comparing the design response spectrum with the hazard curves, it can be seen that the mean probabilities of exceeding the PVNGS seismic design basis at 5 Hz and 10 Hz are approximately 4.5E-05/yr and 3.2E-05/yr, respectively. Thus, the composite mean probability of exceeding the design-basis spectrum at 5 and 10 Hz is 3.8E-05/yr. This low exceedance probability is consistent with the low seismic hazard at the PVNGS site.

C.3 SAN ONOFRE NUCLEAR GENERATING STATION

The SONGS is located on the Southern California coast, between San Diego and Los Angeles, and is operated by Southern California Edison (SCE).

To support the IPEEE study, SCE conducted a probabilistic seismic hazard analysis of the SONGS site. Figure C-7 (SCE 1995) shows mean and fractile seismic hazard curves for the average spectral acceleration over the frequency band from 1 to 10 Hz, and Table C-1 (SCE 1995) provides the estimated mean and median horizontal spectral accelerations at various probabilities of exceedance. (The probability value of 1.386E-04/yr in Table C-1 is labeled "SSE" because it corresponds to the probability of exceeding the safe shutdown earthquake peak horizontal ground acceleration, 0.67 g.)

The horizontal-component design-basis earthquake for SONGS (Units 2 and 3) is specified as a smoothed (modified Newmark) response spectrum that is anchored to 0.67 g. This spectrum, for 5% damping, is plotted in Figure C-8 (modified from SCE 1995). From this figure, the design basis spectral accelerations at 5 and 10 Hz are 1.5 g and 1.1 g, respectively.

Using the slope of the mean hazard curve in Figure C-7 to interpolate the 5-Hz and 10-Hz exceedance probabilities provided in Table C-1, it is estimated that the mean probability of exceeding the SONGS design-basis spectral acceleration value of 1.5 g at 5 Hz is 3.0E-04/yr. Similarly, it is estimated that the mean probability of exceeding 1.1 g at 10 Hz is also 3.0E-04/yr. Thus, the composite mean probability of exceeding the SONGS design-basis spectrum at 5 and 10 Hz is estimated to be 3.0E-04/yr.

C.4 WASHINGTON NUCLEAR PLANT 2

WNP-2 is located near Hanford, Washington, and is operated by the Washington Public Power Supply System (WPPSS).

As an input to the IPEEE study for WNP-2, Geomatrix Consultants (1994) conducted a probabilistic seismic hazard analysis of the site. The resulting mean uniform hazard spectra (5% damping) are plotted in Figure C-9.

The horizontal-component design basis response spectra for WNP-2 are shown in Figure C-10, taken from the WNP-2 IPEEE report (WPPSS 1995). From this figure, the design basis spectral accelerations (5% damping) at 5 Hz and 10 Hz are 0.6 g and 0.4 g, respectively.

Figure C-9 shows that the return period for 0.6 g spectral acceleration at 5 Hz is about 4500 yr (i.e., the exceedance probability is approximately 2.2E-04/yr). Similarly, the return period of 0.4 g spectral acceleration at 10 Hz is about 2950 yr, for an exceedance probability of approximately 3.4 E-04/yr. Taking the average of the two probabilities, the composite mean exceedance probability is approximately 2.8E-04/yr.

C.5 WASHINGTON NUCLEAR PLANT 3

WNP-3 at Satsop, Washington, was only partially constructed and has now been canceled. However, it is included in this analysis because its seismic design basis was provisionally accepted by NRC staff (NRC 1991a). (NRC acceptance was given with the caveat that the safe shutdown earthquake would be reviewed again should the request for an operating license be reactivated, in light of any new information that becomes available.)

Geomatrix Consultants (1988) developed fractile seismic hazard curves for peak horizontal ground acceleration and spectral accelerations at periods of 0.15, 0.80, and 2.00 seconds. The hazard curves for peak ground acceleration and 0.15-sec (6.67 Hz) spectral velocities are shown in Figures C-11 and C-12.

The design basis response spectrum for WNP-3 was a Regulatory Guide 1.60 (AEC 1973) standard response spectral shape anchored to a peak horizontal ground acceleration of 0.32 g (WPPSS 1982). The Regulatory Guide 1.60 horizontal-component design spectra, scaled to 1-g horizontal acceleration, are reproduced in Figure C-13 . The peak spectral velocity of the 5% damped standard spectrum at 6.67 Hz (corresponding to a period of 0.15 sec) is close to 30 in/sec. Scaling this value by 0.32 and converting to metric units, 22.5 cm/sec is obtained for the WNP-3 design basis spectral velocity at 6.67 Hz.

Figure C-11 indicates that the median probability of exceeding 0.32-g peak horizontal acceleration at the WNP-3 site is about 1.3E-03/yr. Figure C-12 shows that the median probability of exceeding 22.5 cm/sec peak spectral velocity at 0.15 sec (6.67 Hz) is about 2.2E-04/yr. The median probabilities of exceeding the design-basis spectral velocities at 5 Hz and 10 Hz are expected be close to this value, with the 10-Hz value probably being higher, given that the exceedance probability appears to increase between 6.67 Hz and the high frequency that is represented by the peak ground acceleration. In addition, mean values are expected to be higher than median values. It is concluded, therefore, that the composite mean probability of exceeding the WNP-3 design basis spectrum at 5 and 10 Hz is greater than 2.2E-04/yr.

C.6 SUMMARY

Published information was compiled regarding the probabilities of exceeding the NRC-accepted 5%-damped design response spectra of nuclear power plants in the western United States. From this information, the composite (arithmetic average) mean probability of exceeding the design basis spectra at 5 and 10 Hz. was calculated or estimated. Table C-2 summarizes results of the compilation.

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