April 1, 2002

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

TESTIMONY OF C. ALLIN CORNELL

I. BACKGROUND – WITNESS

Q1. Please state your full name

A1. Allin Cornell.

Q2. By whom are you employed and what is your position?

A2. 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 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.

Q3. What are your areas of professional expertise?

A3. Through my education, teaching, research and consulting activities (described below) I have developed professional expertise in earthquake engineering, probabilistic engineering analysis of seismic and other loads on structures, and structural responses to such loads. By virtue of my expertise in these areas, I have been actively involved in the development of structural design guidelines, codes and standards, including the appropriate level of earthquake design required to achieve a desired level of safety. I have been involved in establishing earthquake standards of design for nuclear power plants, radiological waste facilities, offshore oil platforms, and buildings.

Q4. Please summarize your educational and professional qualifications.

A4. My professional and educational experience is summarized in the *Curriculum Vitae* attached to this testimony. 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.

Q5. Please describe your studies and professional experience in structural engineering and earthquake hazard analysis.

A5. 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 200~~2~~1 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 (SSHAC) (sponsored jointly by NRC, EPRI and DOE) to establish "standards" for conducting PSHAs at nuclear facility sites.

- As documented in a brief history of the field in ASCE 4-98 [Ref. 32 (ASCE 4-98 Appendix A)], 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 nuclear power plant 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 followed by a second paper (co-authored by several structural and nuclear engineers) based on the first practical application to a specific NPP (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.

- 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.

Q6. Please describe your involvement in the research and development of industry codes and standards, including earthquake design standards.

A6. 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.

[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.)

- 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 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.

Q7. What is your experience with nuclear facilities and the NRC's requirements for the design and licensing of dry cask storage systems?

A7. As indicated by the above description of my background, nuclear power plants and other nuclear facilities have been a major focus of my professional work on the development and application of methodologies and standards for evaluating earthquake hazards. My professional engagements in the area have included work for the NRC, the DOE and a number of commercial operators of nuclear power plants, defense reactors, and high level radioactive waste storage facilities. While working as a consultant to a company preparing material for ISFSI seismic rulemaking, I had the opportunity to become generally familiar with the technologies and issues applicable to the design of ISFSIs.

Q8. Are you familiar with the Private Fuel Storage Facility (PFSF) and the activities that will take place there?

A8. Yes.

Q9. What is the basis of your familiarity with the PFSF?

A9. In connection with the preparation of my earlier declaration and the preparation of this testimony, I have read relevant filings in this proceeding, reviewed portions of the Safety Analysis Report for the PFSF ("SAR") and the NRC Staff's Safety Evaluation Report ("SER"), reviewed a variety of related technical documents (such as DOE Standards 1020-94, 1020-2002, 1021-93, NUREG/CR-6728, etc., as cited herein) and have had multiple conversations with PFSF project personnel such as Mr. Bruce Ebbeson, Mr. Paul Trudeau, Dr. Robert Youngs, Dr. Alan Soler, and Dr. Krishna Singh. In addition, I attended the deposition of the State's expert witness Dr. Walter Arabasz, and have reviewed the declarations of the State's experts that were filed in support of the State's Opposition to PFS's Motion for Summary Disposition of Utah L, Part B (now Section E of Unified Contention Utah L/QQ). I have also reviewed the recent depositions of Drs. Farhang Ostadan and Steven Bartlett on Utah QQ, the earlier declarations by Dr. Singh et al. of Holtec International ("Holtec") and Mr. Ebbeson of Stone & Webster, Inc. ("Stone & Webster") and the testimony being filed simultaneously by Mr. Ebbeson, Mr. Trudeau, Drs. Singh and Soler of Holtec, Dr. Robert Youngs of Geomatrix, and Dr. Wen Tseng of International Civil Engineering Consultants.

Q10. What is the purpose of your testimony?

A10. The purpose of my testimony is to respond to allegations raised by the State of Utah in Section E of Unified Contention Utah L/QQ involving the exemption requested by Private Fuel Storage ("PFS") to use the 2,000-year return period earthquake as the seismic design basis for the PFSF. In particular, 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 like the PFSF, as the standard for the PFSF seismic design. I shall also address specific issues raised by the State in Section E of the Unified Contention Utah L/QQ.

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

Q11. Please describe how the current NRC regulations provide for the earthquake design of ISFSIs.

A11. The current regulations for the seismic design of ISFSIs at sites west of the Rocky Mountains (10 C.F.R. § 72.102(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 C.F.R. Part 100).

Q12. Please describe PFS's request for an exemption to use a 2,000-year return period earthquake as the design basis for the PFSF?

A12. PFS has requested an exemption from the deterministic methodology currently required by 10 C.F.R. Part 72 to use the Probabilistic Seismic Hazard Analysis methodology, accepted by the NRC for new nuclear power plants, for establishing the design basis ground motions for the PFSF. Specifically, PFS proposes to set the design basis motions for the PFSF at a mean annual probability of exceedance ("MAPE") of 5×10^{-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."

Q13. What is meant by "deterministic" procedures for assessing earthquake design basis ground motions?

A13. Deterministic assessments of the seismic hazard at a site lead to one or a small set (of magnitudes and locations) of representative earthquakes that could affect a site and a corresponding set of ground motion response spectra. As it has been applied in the nuclear field, the deterministic procedure consists of associating a single event magnitude to each identified seismic source, based where possible on the dimensions of the active fault, or where such faults are ill-defined, on the historical seismicity in large regions of assumed uniform seismicity. Single locations (or distances to the site) are associated with each such event. A method of ground motion prediction is then used to project a single value of one or more ground motion measures (e.g., peak ground acceleration and/or spectral acceleration) to the site for each of the magnitude-location pairs. From these ground motion results, the dominant event pair (or set of two or three pairs) is identified together with its (or their) representative response spectra at the site. This becomes the design ground motion.

Q14. Please describe the Probabilistic Seismic Hazard Analysis (PSHA) methodology for assessing earthquake design basis ground motions and explain how does it differ from a deterministic" approach.

A14. A PSHA takes into account the entire range of potential events (magnitudes and locations) that could affect a site 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.

Q15. Is the PSHA methodology commonly used for determining design basis ground motions for earthquake design of building and structures?

A15. Yes. The use of PSHA methodology for establishing structural design basis ground motions is today the dominant nuclear power industry practice. Use of PSHA methodology is also prevalent in the design of other structures and facilities including buildings, bridges, offshore structures and U.S. Department of Energy ("DOE") facilities. Current regulations and guidelines based on probabilistic seismic hazard 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-1020-94, Table 2.1, pp. 2-4)].[2] In the building and offshore area, the use of PSHA-based designs dates to the early 1980s.

Q16. Why is the PSHA methodology so widely used and accepted?

A16. The PSHA methodology has become widely accepted and used today because there are several advantages to using a probabilistic approach to establish design ground motions. These advantages are: (1) the probabilistic approach captures more fully the current scientific understanding of earthquake forecasting than the deterministic method; (2) the probabilistic approach is capable of reflecting the uncertainties in professional knowledge of key elements of the seismic hazard; and (3) the probabilistic approach can be used to set design criteria that are consistent among different regions and among different failure consequences, thus allowing a rational and a equitable allocation of safety resources.

[2] Portions of DOE-STD-1020-94 are attached as PFS Exhibit DDD.

Q17. Has the NRC adopted the use of PSHA methodology?

A17. Yes. The NRC has recognized the advantages of the probabilistic approach and has replaced Appendix A, 10 C.F.R. Part 100, which was based on a deterministic hazard assessment methodology, with regulations and guidance documents that provide for use of PSHA methodology for the seismic design of new nuclear power plants. [Ref. 3 (10 C.F.R. §100.23) and Ref. 4 (Regulatory Guide 1.165)]. As stated in my background, I served on a committee of consultants that advised the NRC and its contractor in its development of these documents. 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)].

Q18. Is it appropriate to use the PSHA methodology for assessing and determining the design basis ground motion for the PFSF as requested by PFS?

A18. Yes. 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 both current NRC policy and practices as well as broader engineering policy and practice. The State's seismic expert witness in this proceeding agrees that a PSHA should be used for the seismic analyses and design of the PFSF. Deposition of Walter J. Arabasz ("Arabasz Dep.") (October 31, 2001) at 44-45, attached as PFS Exhibit EEE.

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

A. General Principles of Risk-Informed Seismic Design

Q19. Please describe the seismic design basis for the PFSF.

A19. 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 $5x10^{-4}$ (i.e., a 2,000-year mean annual return period, or 2,000-year MRP) and applying those ground motions to the design criteria and procedures of the NRC's SRPs for nuclear systems, structures, and components (SSCs).

Q20. Based on your work using PSHA and developing codes and standards for earthquake safety, are there any general principles that provide guidance on the adequacy of PFS's proposed seismic design basis for the PFSF?

A20. Yes. General principles of risk-informed seismic design can be used to judge the adequacy of the seismic design basis proposed for the PFSF. The first such general principle is that there should be a risk-graded approach to seismic safety which allows facilities and structures with lesser failure consequences to have larger mean annual probabilities of failure. A second general principle is that the adequacy of a design basis earthquake ("DBE") to provide the desired level of seismic safety is to be judged by considering both the mean annual probability of exceedance of the DBE and the level of conservatism incorporated into the design criteria and procedures.

Q21. Please describe the first principle that you identified, use of risk-graded approach for establishing seismic design standards.

A21. Most modern seismic design criteria are based on the principle that the probability of SSC failure (where failure is defined as reaching or exceeding a behavior mode that may preclude the SSC from fulfilling its intended function, e.g., containment of hazardous material,) 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 whose seismic failure would cause less severe consequences are designed to allow for higher probabilities of failure. The State's seismic 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 SSCs. Arabasz Dep. at 59-60.

Q22. What are the underlying reasons for applying a risk-graded approach to seismic safety?

A22. 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)], of which I am a co-author, Federal Emergency Management Agency ("FEMA") guidelines for building assessment [Ref. 14 (FEMA 273 pp. 2-5)], and DOE Standard 1020-94 [Ref. 11 (Table B-1, p. B-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)].

Q23. Has the NRC made any determination of the relative risk posed by ISFSIs, such as the PFSF, compared to those posed by operating nuclear plants?

A23. Yes, the NRC has stated that the potential consequences of failure of ISFSIs are much less severe than those for NPPs. For example, the Commission has rejected the notion that licensing standards should be as high for ISFSIs as for NPPs, 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))]. Indeed, the Commission has stated in the context of the PFSF case that "Our flexible approach to financial assurance in nonreactor cases appropriately reflects differing levels of risk." Private Fuel Storage (Independent Spent Fuel Storage Installation), CLI-00-13, 52 NRC 23, 30 (2000). The Commission further supported "the Board's risk calculus [holding that a ISFSI presents safety risks more closely comparable to a uranium enrichment plant is] reasonable." Id. at 31. "[T]he Commission has previously stated that a spent fuel storage facility, which holds fuel that has been cooled for at least 1 year and is not subject to dispersive forces associated with high temperature and pressure, has a much smaller potential for serious accidents than a power reactor." Id. (citations omitted). Thus, the Commission has determined that an ISFSI, by virtue of the largely passive nature of its operation, poses much less risk than a nuclear power plant, which relies on active cooling and safe-shutdown systems to maintain the integrity of the high-pressure reactor coolant boundary and shut down after an earthquake.

Q24. In terms of the appropriate level of seismic safety, what is the significance of the Commission's determination that ISFSIs pose much less risk than an operating nuclear power plant?

A24. Because the Commission has determined that the potential consequences of seismic failure of ISFSIs are much less severe than those for nuclear power plants, under the risk-graded approach to the seismic design, ISFSIs, such as the PFSF, can be allowed higher annual probability of failure due to seismic events than NPPs.

Q25. Please elaborate on the second general principle stated above, that a combination of both the mean annual probability of exceedance of the DBE and the level of conservatism incorporated into the design criteria and procedures determine the adequacy of a DBE to provide the desired level of seismic safety.

A25. 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 ("MAPE") and a set of design procedures and acceptance criteria. Both the design procedures and the acceptance criteria (e.g., applicable codes and standards) include conservatisms that implicitly or explicitly implement "performance goals" (e.g., target levels of the seismic failure probability for the SSCs), which are defined in a manner reflecting the anticipated consequences of the failure. These conservatisms are typically not explicitly stated, but are embedded in the design procedures and the various codes and standards pursuant to which the design of an SSC is accomplished.

Q26. Please describe how the MAPE of the DBE and the level of conservatism incorporated in the applicable codes and standards affect the failure probability of seismically-designed SSCs.

A26. The desired level of seismic safety can be achieved by adjusting either the MAPE of the DBE or the level of conservatism of the design procedures and acceptance criteria, or by adjusting both elements simultaneously. For example, a lower (or higher) failure probability can be achieved by keeping the design procedures and acceptance 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. (A concrete example of the last approach is described below in association with a 2002 revision of DOE-STD-1020-94.) 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 acceptance 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 acceptance criteria, the probability of failure of a seismically-designed facility or SSC is virtually always less than the MAPE of the governing DBE. In other words, virtually all facilities and SSCs 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 SSCs are able to withstand a more severe, i.e., more infrequent, earthquake than that used as the DBE.

Q27. Can you give an example of the application of these principles of risk-graded seismic design?

A27. 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 1020-94. The basis for DOE Standard 1020-94 is a set of "performance categories" (1 to 4) for seismically designed[3] SSCs with increasing consequences of failure, and thus decreasing probabilities of failure as their performance goals [Ref. 11 (DOE-STD-1020-94, p. 1-2, Section B-2, and Table B-1)]. 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 (PC2), (3) important facilities such as ISFSIs that contain hazardous materials[4] (PC3), and (4) critical facilities such as those involving nuclear reactors (PC4).

The performance goals for DOE structures, systems and components in the four performance categories PC1 to PC4 in DOE-STD-1020-94 are set as mean annual failure probabilities of 10^{-3}, $5x10^{-4}$, 10^{-4}, and 10^{-5}, respectively [Ref. 11 (DOE-STD-1020-94, p. Table B-1)] reflecting the increasing consequences of failure. On the other hand, the mean annual probability of exceedance (MAPE) 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.

To bridge the gap between the performance goals and the DBE MAPEs, the DOE-STD-1020-94 standards call for design procedures and acceptance 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-STD-1020-94, p. 2-2, C-4 to C-5)]. The quantitative effect, in

[3] There is a fifth category, PC0, for which there are no seismic requirements.

[4] For PC3 SSCs the performance goal is set relative to "damage beyond which hazardous material confinement and safety-related functions are impaired" [Ref. 11 (DOE-STD-1020-94 at pg B-8)].

terms of reducing earthquake risk, of applying the conservatisms built into these various design procedures and acceptance 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-STD-1020-94, p. C-5)]. The ratios are called "Risk Reduction Ratios", R_R, in DOE-STD-1020-94 parlance. 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-STD-1020-94:

TABLE 1: DOE STD 1020-94 SEISMIC PERFORMANCE GOALS, DBE MAPES AND R_{RS}

Performance Category	Target Seismic Performance Goal (P_F)	DBE Exceedance Probability (MAPE)	Risk Reduction Ratio (R_R)
PC1 (e.g., office building)	$1x10^{-3}$	$2x10^{-3}$	2
PC2 (e.g., essential building that should remain operational, such as hospital or police station)	$5x10^{-4}$	$1x10^{-3}$	2
PC3 (e.g., hazardous waste facilities such as ISFSIs)	$1x10^{-4}$	$5x10^{-4}$ (except $1x10^{-3}$ for Western sites near tectonic boundaries)[5]	5 (except 10 for Western sites near tectonic boundaries)[3]

[5] The actual value of R_R obtained from the design conservatisms for a given SSC is dependent to some degree 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

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PC4 (e.g., nuclear reactor facility)	$1x10^{-5}$	$1x10^{-4}$ (except $2x10^{-4}$ for Western sites near tectonic boundaries)[3]	10 (except 20 for Western sites near tectonic boundaries)[3]

Q28. Has a revised version of DOE-STD-1020-94 recently been issued?

A28. Yes. A revised version of DOE Standard 1020 was approved in January, 2002 [Ref. 18 (DOE-STD-1020-2002]. The modifications have no effect on the use made of the DOE-STD-1020-94 here. The primary change is that PC1 and PC2 are now based on the IBC 2000 building code instead of the older UBC model building code. This newer code calls for a considerably larger, 2500-year, DBE and, appropriately, much less conservative acceptance criteria (e.g., the ground motions are reduced by a new factor of 1.5) (which I discuss further below). This IBC 2000 code has not been based on an explicit Performance Goal or explicit risk reduction, R_R, values, however, and DOE has not made an effort to estimate them. As a result, the Performance Goals and the R_R values on this table have been left blank in DOE-1020-2002 in those categories.[6] A minor change has also

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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 considerably steeper [Ref. 11 (DOE-STD-1020-94, Table C-3 p. C-5)]. 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.

[6] Although the R_R column is left blank for PC1 and PC2, it can be shown (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) that the net R_R is still about 2 for PC 2 and, now, because of the 1.5 reduction referred to above, the net value is only about 0.4 for PC1; it is still 2 before this adjustment. If so, then the performance goal achieved for PC1 has remained effectively unchanged at 10^{-3} and that for PC2 has perhaps been implicitly improved. DOE-STD-1020-2002 recognizes these issues stating that the original PC1 and PC2 goals (still cited in Appendix B, Table B-1) are "no longer exact" [Ref. 18 (DOE-STD-1020-2002 at pg C-6)].

been made to the PC3 category to permit the use for PC3 category structures and components of USGS national probabilistic seismic hazard maps. To meet building code needs, these maps are printed for this 2500-year level. Therefore, the DOE-STD-1020-2002 MAPE of PC3 is modified slightly to this $4x10^{-4}$ value. The PC3 performance goal remains 10^{-4}, however. Therefore, the R_R has been reduced from 5 to 4 by making the acceptance criteria somewhat less conservative.[7] This is the example of a conscious, simultaneous change of MAPE and conservatisms referred to above. For simplicity and clarity, because the DOE-STD-1020-94 and the PFSF both have a 2000-year DBE, I shall continue to refer to the original document.

Q29. How is the level of conservatism or risk reduction factors , R_{R}s, for DOE-STD – 1020-94 achieved?

A29. In DOE-STD-1020-94, for most SSCs the overall conservatism levels are controlled through conventional "deterministic" acceptance criteria to achieve specific R_R levels [Ref. 11 (DOE-STD-1020-94, pg. 1-5)]. For the categories of more interest here, PC3 and PC4, this has been accomplished by specifying certain procedures, parameter values, and material standards [Ref. 11 (DOE-STD-1020-94, Chap. 2)] that permit calculation of a SSC's earthquake resistance capability ("capacity") versus earthquake and other loadings ("demand"). Capacity and demand are compared to determine whether compliance with the acceptance criteria is achieved. In DOE-STD-1020-94, the conservatisms have been "intentionally introduced and controlled" [Ref. 11 (DOE-STD-1020-94, at pg. C-6)]. For example, the seismic portion of the demands is obtained by estimating the force on the SSC due to the design basis earthquake and then multiplying this demand by a factor, SF, whose value has been carefully calibrated by probabilistic calculations (described in the document [Ref. 11 (DOE-STD-1020-94, Section C.2.2)]) to achieve the value of R_R appropriate to the

[7] A factor referred to as SF in Eq. 2-1 and 2-7 [Ref. 18 (DOE-STD-1020-2002)] has been reduced from 1.0 to 0.9 to accomplish this change.

DBE MAPE and performance goal of category PC3 and of category PC4 (5 and 10 respectively, for most regions).

Q30. Do the design acceptance criteria and procedures for NRC-licensed facilities contain similar conservatisms, or risk reductions factors, as those embodied in DOE-STD-1020-94?

A30. Yes. It is well established that the design acceptance criteria and procedures guidelines provided by the NRC SRPs contain many conservatisms that result in risk reduction factors as large as, or larger than, those for PC4 category facilities designed to DOE-STD-1020-94. NRC SRP standards share with DOE's PC3 and PC4 categories many procedures leading to design conservatism [Ref. 11 (DOE-STD-1020-94, pp. C-5, C-6)]. These conservatisms are introduced through prescribed analysis 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, however. Nonetheless, the risk reduction factors achieved through the use of NRC guidelines for typical SSCs have been found to be equal to, or higher than, those called for in DOE-STD-1020-94 for PC4 facilities.

Q31. Is this higher level of conservatism compared to DOE-STD-1020-94 provided by the design criteria embodied in the NRC SRPs expressed anywhere?

A31. Yes. DOE-STD-1020-94 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-STD-1020-94, p. 2-2, C-4 to C5). Further, there is recent independent technical support both for the general conclusion that NRC SRPs provide equal or greater levels of conservatism than DOE-STD-1020-94, and for the quantitative finding that the levels of the risk reduction factor, R_R, for typical systems, structures, and components designed to NRC SRPs are in the range of 5 to 20 or greater [Ref. 20 (NUREG/CR-6728 at Chapter 7)].[8]

Q32. What do you mean by typical systems, structures and components?

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

A32. By typical systems, structures and components I mean those SSCs which are representative of SSCs commonly found in commercial nuclear power plants. These are the SSCs that have been evaluated in the many seismic PRAs and seismic margins studies upon which the experience base has been built to reach these general conclusions about the 5 to 20 or greater range of NPP SSC R_R values. As used here, the term typical SSCs is restricted further to exclude brittle SSCs, which are not found in any case among those in the PFSF.

Q33. What would be expected for other components assuming that they were designed to NRC SRPs?

A33. Given the decades of NRC's concern about seismic safety, and given the code, standards and criteria they call for, one would expect *a priori* similar levels of conservatism in any SSC designed to their SRPs and hence a similar range of R_R levels. For a SSC such as a free-standing storage cask, which is not typical of commercial NPPs, the level of conservatism can be demonstrated by specific analysis. This has been done here by finding a lower bound on R_R based on beyond-design-basis analyses by Holtec and the NRC Staff with respect to the HI-STORM 100 storage system, as discussed further below.

B. Application of General Principles to the PFSF

1. Application of NRC SRP Risk Reduction Factors to ISFSIs

Q34. You stated earlier that PFS has performed the seismic design for important to safety SSCs at the PFSF using a 2,000-year mean annual return period earthquake and applying the design criteria and procedures of the NRC's SRPs for nuclear components. What do you mean by the NRC's SRPs for nuclear components?

A34. I mean the SRPs that the NRC has established for various facilities that it licenses. These SRPs set forth the acceptance criteria and procedures for designing the facility, typically referring to standards and codes specifically developed for the design and construction of nuclear components, such as the code for the Seismic Analysis of Safety-Related Nuclear Structures developed by the American Society of Civil Engineers, ASCE

4-86[9] and the "Code Requirements for Nuclear Safety Related Concrete Structures" of the American Concrete Institute, ACI 349,[10] to which the PFSF has committed. Specifically, the NRC has a Standard Review Plan for nuclear power plants, NUREG-0800,[11] which specifies the design procedure and acceptance criteria for nuclear power plants. Likewise, the NRC has a Standard Review Plan for Independent Spent Fuel ISFSIs, NUREG-1567,[12] and one for dry cask storage systems, NUREG-1536.[13]

Q35. Is the conclusion that the R_R levels for typical systems, structures, and components designed to NRC SRPs are in the range of 5 to 20 or greater premised on the application of any particular SRP?

A35. As stated above, the basis for this conclusion is the history of seismic PRA and margins studies conducted on commercial nuclear power plants designed to NUREG-0800, the SRP for such facilities. However, by virtue of the general commonality of the design procedures and acceptance criteria called for in other SRPs, that the conclusion is equally applicable to SSCs designed to the NRC dry storage SRPs cited above. This commonality is discussed below and in the testimony of other PFS witnesses.

Q36. What is your familiarity with these SRPs?

A36. I have been involved for most of my professional career with the evolution of key parts of the seismic portions of NUREG-0800, the SRP for commercial NPPs. In particular, I am very familiar with the assessment of vibratory ground motions (Section 2.5.2) and seismic design parameters

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9 [Ref. 31 (American Society of Civil Engineers, ASCE 4-86, *Seismic Analysis of Safety-Related Nuclear Structures and Commentary for Seismic Analysis of Safety Related Nuclear Structures*, September 1986)].

10 [Ref. 34 (American Concrete Institute, ACI-349, *Code Requirements for Nuclear Safety-Related Concrete Structures*, 1999)].

11 [Ref. 2 (U.S. Nuclear Regulatory Commission, NUREG-0800, *Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants*, August 1988)].

12 [Ref. 1 (U.S. Nuclear Regulatory Commission, NUREG-1567, *Standard Review Plan for Spent Fuel Dry Storage Facilities*, March 2000)].

13 [Ref. 38 (U.S. Nuclear Regulatory Commission, NUREG-1536, *Standard Review Plan for Dry Cask Storage Facilities*, January 1997)].

(Section 3.7.1), and the documents they refer to. As explained earlier, I participated in the development of Section 100.23 of 10 C.F.R. Part 100, Regulatory Guide 1.165, the EPRI and LLNL PSHA studies of CEUS (Central and Eastern U.S.) sites, and the Senior Seismic Hazard Analysis Committee (SSHAC) report. Other sections of NUREG-0800 relevant to seismic safety, e.g., those defining load combinations, acceptable codes (such as ASME Boiler and Pressure Vessel,[14] ACI 349, [15] AISC,[16]) etc., are similar in content if not in detail to other seismic criteria that I have worked with my entire career. I have reviewed recently the NPP SRP, NUREG-0800. My familiarity with NUREGs-1567 and 1536, the SRPs for dry storage systems, was limited before beginning my work on the PFSF, but I have reviewed them in the context of that work.

Q37. Based on your review of NUREGs-1536 and 1567, do you have any opinion on the similarity of conservatisms embodied in the acceptance criteria and procedures of 1536 and 1567 compared to those encompassed within NUREG-0800 as they relate to seismic design?

A37. Yes. That review confirmed the similarities in the seismic elements of the ISFSI and NPP SRPs. Some specific examples follow. Both set of requirements call for use of Regulatory Guide 1.165 [Ref. 4 "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion," 1997] and accept Regulatory Guide 1.60 [Ref. 37 "Design Response Spectra for Seismic Design of Nuclear Power Plants"]. For damping levels, which introduce important conservatisms, both NUREG-1567 and NUREG-0800 reference the NRC Regulatory Guide 1.61 [Ref. 39 "Damping Values for Seismic Analysis for Nuclear Power Plants," 1974]. For reinforced concrete structures (other than the casks themselves, e.g., as would be used with a cask transfer building)

[14] [Ref. 35 (American Society of Mechanical Engineers, *ASME Boiler and Pressure Vessel Code-Nuclear Power Plant Components*, Section III, 1989)].]

[15] [Ref. 34 (American Concrete Institute, ACI-349, *Code Requirements for Nuclear Safety-Related Concrete Structures*, 1999)].

[16] [Ref. 36 (American Institute of Steel Construction, *Manual of Steel Construction, Allowable Stress Design*, 1989)].

the ISFSI SRPs, like that for NPPs, call for application of ACI-349 [Ref. 34 "Code Requirements for Nuclear Safety Related Concrete Structures"]. Finally all three SRPs cite frequently Section III of the ASME Boiler and Pressure Vessel Code. [Ref. 35] Such similarities explain why one can anticipate very similar levels of conservatism from both the NPP and ISFSI SRPs.

Q38. Do you have any other basis on which to conclude that the SRPs for ISFSIs generally embody the same level of conservatism as NUREG-0800?

A38. Reviewing the testimony filed by PFS of Dr. Alan Soler, Dr. Krishna Singh, Mr. Bruce Ebbeson, Mr. Paul Trudeau, and Dr. Wen Tseng, I see that they used the standards and codes generally applicable for nuclear components, such as those cited above, which are the same standards and codes referenced in NUREG-0800. Further, they have stated that they generally used the same design criteria and procedures applicable to nuclear power plants.

Q39. What conclusion do you draw based on your review and understanding of the SRPs and the testimony of those responsible for the design of the PFSF structures and components?

A39. Because important-to-safety structures, systems and components at the PFSF are designed to the same codes and standards as those for nuclear power plants, the conclusion that the R_R levels for typical systems, structures, and components designed to NRC SRPs are in the range of 5 to 20 (or greater) would apply to such structures systems and components at the PFSF.

Q40. What SSCs important to safety at the PFSF would clearly fall under the rubric of "typical" SSCs designed to NRC SRPs for which a R_R of 5 to 20 (or greater) would apply?

A40. PFSF SSCs in the CTB, including the building itself, its roof, the cranes and the seismic struts, clearly fall under this category because the same (or very similar) SSCs occurring in the NPPs have been analyzed in the many seismic PRAs and margins studies that provided the experience upon which this general range of R_R values is based. Several projects have developed guideline procedures based on such general R_R observations (e.g., the NRC and EPRI margins methods, DOE-STD-1020-94, and most recently NUREG/CR-6728, in which, as cited above, ~~Answer Attachment A,~~ one can find the quote that is the basis for the conclusion that typical NPP R_Rs are 5 to 20 or more). The results of these studies have been evaluated and/or collected and summarized in seismic PRA and margins projects I have been involved in the past, e.g., the Diablo Canyon seismic PRA, the LLNL Seismic Margins project, and the development of NRC seismic margins methodology.

Q41. What about the foundation to the Canister Transfer Building or the storage cask pads for the spent fuel casks?

A41. The NPPs whose seismic PRAs and margins studies form the basis of the R_R values cited have buildings with foundations generally analogous to that of the CTB. While I am personally less familiar with the foundation SPRA results, I am aware that they have been prepared for potential foundation failure modes such as overturning, bearing, and sliding. While it is not entirely clear whether the R_R range conclusion (based on NUREG-6728) was intended to apply to foundations, it can be presumed, nonetheless, that given the NRC's many years of concern for seismic safety and for margins beyond the design basis, that comparable levels of conservatism in foundations have been provided by their criteria and by practice in the field, and hence that comparable levels of R_R likely exist with respect to performance that might jeopardize hazardous materials containment.

Q42. What about the spent fuel storage casks themselves?

A42. As described in the testimony of Dr. Alan Soler, the spent fuel storage casks are designed to the ISFSI SRP NUREG-1536 [Ref. 38] discussed above. They are also designed for other SRP-dictated accident conditions, such as hypothetical drop and tip-over events. With respect to direct seismic inertial forces, it can be expected for the reasons cited above that their R_R values will equal or exceed the 5 to 20 range of typical NPP components. (Indeed, it has been confirmed that for these effects the HI-STORM 100 storage system has very large margins.) As stated above, these casks are not common NPP SSCs but, as will be discussed below, consideration of the Holtec and Sandia analyses of the HI-STORM 100 system with respect to beyond-design-basis earthquake motions and with respect to potential tip-over conditions shows that the effective R_R of the cask system is in excess of 5. Thus, the design of this cask system provides risks reduction factors comparable to those available for typical NPP SSCs.

2. Appropriate Risk Reduction Factors for the PFSF

Q43. Do you have an opinion as to the risk reduction factors applicable to the seismic design of the PFSF?

A43. Yes.

Q44. What in your opinion is an appropriate seismic risk reduction factor to represent the SSCs in the PFSF?

A44. Based on the established and demonstrated margins, I believe that a risk reduction factor of five or more is appropriate for important-to-safety SSCs in the PFSF.

Q45. What is the general basis for your opinion?

A45. The basis for my opinion is (1) my general knowledge and experience regarding risk reduction factors as applied to many different types of structures designed to a wide variety of codes and standards; (2) my general knowledge and experience of risk reduction factors applicable to nuclear power plants designed in accordance with the applicable design codes and standards as specified by the NRC NPP SRP (NUREG-0800); (3) my independent review of the SRPs applicable to ISFSIs and spent fuel storage casks (NUREGs 1567 and 1536) and confirmation that the codes and standards applicable to nuclear power plants are generally applicable to ISFSIs, such as the PFSF; (4) confirmation by those responsible for the design of the structures and components at the PFSF that such structures and components are generally designed to the same codes and standards applicable to nuclear power plants; (5) analytical and qualitative demonstration by those responsible for the design of the PFSF of significant beyond-design-basis margins for structures and components important to safety; (6) the limited fraction of time that certain SSCs are in use; (7) demonstration by Holtec that casks at the PFSF will not tip-over at the

10,000-year earthquake and (8) demonstration by Holtec that a postulated cask tip-over will not result in breach of a cask and release of radioactivity.

Q46. What structures and components have you considered as important to safety in your review?

A46. In my review, I considered the Canister Transfer Building and the cranes and the seismic struts inside the CTB used in transferring the spent fuel canisters from the transportation casks to the storage casks. I also considered the spent fuel storage casks and the storage cask pads on which they are placed.

Q47. On what basis did you decide that these were the appropriate structures and components to consider in your evaluation of risk reduction factors for the PFSF?

A47. I depended on information provided by PFSF personnel, such as the testimony of Mr. Wayne Lewis.

Q48. Please describe the basis of your opinion that the risk reduction factor for the Canister Transfer Building and the cranes and struts inside the building is 5 or more?

A48. The Canister Transfer Building itself and the cranes and seismic struts inside the building are typical of nuclear power plant components for which the risk reduction factor has been shown to be a factor of 5 to 20 or more. That basis alone would be sufficient to conclude that the CTB and the cranes and seismic struts inside the CTB have a risk reduction factor of five or more.

Q49. What else, if anything, do you base your opinion that the risk reduction factor for the Canister Transfer Building and the cranes and struts inside the building have a risk reduction factor of 5 or more?

A49. I rely upon facts described in the testimony of Mr. Bruce Ebbeson, the individual responsible for the design of the CTB, and Mr. Wayne Lewis. First, Mr. Ebbeson's testimony confirms that these components were designed to nuclear power plant standards, where applicable, suggesting that the general conclusion about the R_R values of typical NPP SSCs applies. Second, the beyond-design-basis analyses and margins described in the testimony of Mr. Ebbeson confirm the existence of significant beyond-design-basis margins in the design of the CTB and the cranes and struts therein, which would enable them to survive earthquake ground motions much greater than those of the 2000-year design basis earthquake. Third, as described in the testimony of Mr. Wayne Lewis, the CTB cranes and seismic struts are in use only a fraction of the time, and 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 20%, the annual probability of failure causing release due to earthquake ground motions is 5 times smaller. This implies that, even if their R_Rs due to SRP conservatisms were only unity instead of the factors of 5 to 20 or more estimated above, the relevant frequencies of failure of these SSCs would be less than 10^{-4}. With the predicted R_R of 5 to 20 or more, this estimated failure frequency reduces to about 10^{-5}. In short the effect of the 20% use fraction is, in effect, to increase R_R by a factor of 5.

Q50. What about the foundations for the CTB? Have you considered and determined whether a risk reduction factor of 5 or more is applicable to the CTB foundations?

A50. As discussed earlier, based on the NRC's long concern over seismic safety margins there is *a priori* reason to expect that an R_R comparable to those of typical NPP SSCs is available with respect to those modes of PFSF CTB foundation behavior that might lead to loss of containment of hazardous materials. As presented in the testimony of PFS witnesses Mr. Ebbeson and Mr. Trudeau due to differences such as those between calculated and design safety factors, realistic dynamic and the assumed static behavior, mean and the lower bound soil properties, dynamic and static soil properties, etc., that there is significant margin with respect to the ground motions that might cause overturning or bearing failure of these foundations. They conclude that this total expected margin is greater than that needed to meet the 10,000-year ground motions. Local bearing failure would, in any case, likely be tolerated by the building without impairing the performance of hazardous material containments inside it. Therefore these foundation behavior modes can be estimated to have R_R

levels of 5 or more. It has not been demonstrated that the CTB will not slide under ground motions of, say, the 10,000-year level, but, as Mr. Ebbeson states, this sliding would not have negative consequences with respect to loss of containment of hazardous materials.

Q51. Please describe the basis of your opinion that the risk reduction factor for the storage pads is 5 or more?

A51. As discussed in the testimony of PFS witness Paul Trudeau, there are large quantifiable margins of safety against overturning and soil bearing failure at or approaching MRPs 5 times the 2000 DBE level, as well as other significant non-quantified conservatisms. Together these conservatisms safety allow one to reasonably conclude that no overturning or hazardous-to-release bearing failure would be expected under ground motions with MRPs of more than 5 times the 2000-year DBE level. Also, as ~~these~~ PFS witnesses confirm, sliding of the storage pads is not expected, per se, to cause hazardous material release. The effect of any such pad sliding on the behavior of the storage casks has been considered in the assessment of the cask.

Q52. Please describe the basis of your opinion that the risk reduction factor for the spent fuel storage casks at the PFSF is 5 or more?

A52. As described in the testimony of Drs. Singh and Soler of Holtec, the HI-STORM 100 system storage casks are stubby cylindrical weldments of steel and concrete designed to NRC SRPs to tolerate significant earthquake-induced inertial forces as well as those due to drop and tip-over accidents. Therefore, as discussed above, their margins with respect to the 2000-year design basis motions can be expected to be very significant. As testified by Drs. Soler and Singh, in addition to the assessments required by the NRC SRPs, Holtec and Sandia have conducted 10,000-year ground motion analyses predicting that there will be neither cask tip-over nor cask-cask sliding impacts. They testify further that even should there be tip-over the tip-over analysis conducted by Holtec predicts no breach. As testified by PFS witnesses Drs. Singh and Soler, even should one sliding cask impact another the effects are bounded by the tip-over analysis. Further, Drs. Singh and Soler state that these assessments retain elements of conservatism, e.g., upper and lower bound cask friction coefficients are used, and the cask could suffer even more damage than predicted before breaching. An upper bound on the probability of loss of containment can be estimated easily by use of this information. Given this prediction of no tip-over under a 10,000-year ground motion, the annual probability of tip-over can be judged to be no more than 10^{-4}. Based on the prediction of no breach given tip-over the conditional probability of breach given tip-over can be judged to be significantly less than one. The annual probability of loss of containment of hazardous material due to cask tip-over is simply the product of these two numbers, which is clearly less than 10^{-4}. Based on the information stated above the annual probability of loss of containment due to cask sliding is clearly much smaller than this bound on that due to tip-over. With the $5x10^{-4}$ MAPE of the DBE, the implied R_R for the storage casks is therefore greater than 5.

3. Adequacy of the 2000-year Design Basis Earthquake for the PFSF under a Risk-Graded Approach to Seismic Safety

Q53. Based on your review of the risk reduction factors applicable to the PFSF, do you have an opinion on whether the 2000-year design basis earthquake for the PFSF provides an adequate level of seismic safety?

A53. Yes.

Q54. Please state your opinion and the bases therefore.

A54. I believe that the PFSF 2000-year design basis earthquake (DBE) provides an adequate level of seismic safety because: (1) based on my review of the risk reduction factors (R_R) applicable to the SSCs important to hazardous material containment discussed above I believe that these factors are 5 to 20 or greater; (2) coupled with the 2000-year ($5x10^{-4}$ MAPE) DBE these R_R levels imply that the PFSF SSCs will have achieved a performance goal of $1x10^{-4}$ or better; and (3) I believe, based on the principle of risk-grading discussed above, that $1x10^{-4}$ is an appropriate performance goal for the SSCs of this spent fuel dry storage facility.

Q55. Please state the basis for your opinion that $1x10^{-4}$ is an appropriate performance goal for the PFSF SSCs.

A55. First, applying the risk-graded seismic principle, a performance objective of 1×10^{-4} for SSCs ISFSIs such as the PFSF is consistent with the NRC's performance objectives for operating nuclear plants, which ~~THE~~ the NRC ~~HAS FOUND~~ has found pose higher radiological hazard consequences than ISFSIs. While the NRC nuclear power plant seismic performance goals and the quantitative effects of their design criteria are less explicit than those in DOE Standard 1020-94, 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 1×10^{-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 from seismic PRAs to range from about 4×10^{-6} to about 1×10^{-4}, with most lying between 0.6 and 1×10^{-5}. DOE-STD-1020-2002 [Ref. 18 at p. B-7] quotes NUREG/CR-5042 as finding the same range in 12 more recent NPPs, while 10 of the 12 plants have such frequencies greater than 1×10^{-5}. ~~[Ref. 22 (NUREG/CR-5501)]~~ As discussed above, DOE-STD-1020-94 also uses, explicitly, a performance goal of 1×10^{-5} for nuclear reactor SSCs. The use of a probability of seismic failure or performance goal for the PFSF SSCs, such as 1×10^{-4}, higher than that for nuclear power plants SSCs (about 1×10^{-5}) is consistent with the risk-graded approach of the probabilistic approach.

Second, an SSC performance goal of 1×10^{-4} is consistent with DOE policy as represented by DOE-STD-1020-94 and DOE-STD-1021-93. As discussed above, the performance goal stated in DOE-STD-1020-94 for category PC3 SSCs is 1×10^{-4}. The PFSF important-to-safety SSCs would clearly fall into category PC3. DOE-STD-1021-93 [Ref. 40, "Natural Phenomena Hazards Performance Categorization Guidelines for Structures, Systems and Components," July 1993], which defines such categories, states (at pg. 2-3) "If the adverse offsite consequences of an NPH [Natural Phenomena Hazard] event are significant enough to make them safety-class but are substantially less than those associated with consequences from an unmitigated large Category A reactor severe accident, the SSCs should be placed in PC-3." The State's seismic expert witness, Dr. Arabasz, agreed that ISFSIs, such as the PFSF, would appropriately be classified PC3 facilities under DOE-STD-1020-94 and that the performance objective of 1×10^{-4} for the PFSF SSCs would be an appropriate standard on which to determine the acceptability of its seismic design. Arabasz Dep. at 80-81. I conclude that a performance goal of 1×10^{-4} for the PFSF would be consistent with a risk-graded approach to seismic safety. The proposed PFSF seismic design basis of a 2,000-year MRP DBE and the SRP design procedures and criteria will meet such a goal and therefore provide an appropriate and consistent level of protection to public health and safety.

IV. DISCUSSION OF SPECIFIC ISSUES RAISED BY THE STATE OF UTAH

Q56. What claims does the State of Utah raise with respect to Section E on the Unified Contention?

A56. The State raised seven issues in the bases supporting what is now Section E of the Unified Contention, some of which relate to issues discussed above. In addition, in the State's Opposition to PFS's Motion for Summary Disposition on this aspect of the contention, the State's experts dispute certain aspects of the analysis that I provided in a declaration dated November 9, 2001 supporting the PFS Motion.

A. Claims of State's Experts Raised in State of Utah's Summary Disposition Opposition

Q57. Focusing first on the claims of the State's experts in the State's Summary Disposition Opposition, what were the main responses of the State's experts regarding the analysis provided in your November 9, 2001 declaration supporting the PFS Motion?

A57. The State's primary expert supporting the State's contention, Dr. Walter Arabasz, agreed with the two basic principles that I set forth in my analyses, which I have also explained above. Dr. Arabasz agreed with the concepts of (i) using a risk graded approach to seismic safety, and (ii) determining acceptable earthquake performance of a facility or structure based on a combination of the mean annual exceedance period of the design basis earthquake for the structure and the conservatisms embodied in the standards and codes governing its design and construction.[17] Further, Dr. Arabasz did not take issue with my application of those principles to the PFSF in my November 9, 2001 declaration, although other experts of the State, Drs. Bartlett and Ostadan, did take issue with certain parts of the declaration.

Q58. What issues did Drs. Bartlett and Ostadan raise with respect to your November 9, 2001 declaration?

A58. Generally, their issues involved the risk reduction factors applicable to the PFSF. They claimed that PFS could not rely upon the risk reduction factors specified by DOE-STD-1020-94 or derived from NUREG/CR-6728 because the PFS design does not meet the intent or requirements of either document.[18] They further claimed that the risk reduction factors applicable to typical SSCs at nuclear power plants are not applicable to SSCs at the PFSF because the NRC Standard Review Plan ("SRP") requirements for nuclear power plants are not applicable to important-to-safety SSCs at the PFSF, and that "the SRPs in NUREG 1536 and 1567" applicable to SSCs at the PFSF "may already incorporate less conservatism than" the SRP for nuclear power plants.[19]

Q59. Let's address Drs. Bartlett and Ostadan's claims in the reverse order that you just mentioned. What about their claim that the SRPs applicable to the PFSF "may incorporate less conservatism" than the SRP for nuclear power plants?

A59. Their claim that the SRPs applicable to the PFSF "may incorporate less conservatism" than the SRP for nuclear power plants is erroneous, at least insofar as the design of the PFSF is concerned. As I discussed above, the

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[17] Declaration of Dr. Walter J. Arabasz ("Arabasz Decl.") (Dec. 7, 2001 ¶¶ 18-19).

[18] Joint Declaration of Dr. Steven F. Bartlett, Dr. Moshin R. Khan and Dr. Farhang Ostadan ("Joint Utah Decl.") (Dec. 7, 2001) ¶ 49.

[19] Id. ¶ 49.

design of important-to-safety SSCs at the PFSF is based on essentially the same nuclear codes and standards specified in NUREG-0800, the SRP for nuclear power plants. Therefore, it is appropriate, to utilize the seismic risk reduction factors of 5 to 20 or more for typical nuclear power plant SSCs to the corresponding SSCs at the PFSF.

Q60. What is your response to the claims raised by the Drs. Bartlett and Ostadan that your reliance on NUREG/CR-6728 is inappropriate?[20]

A60. As set forth in paragraph 25 of my November 9, 2001 declaration and Attachment A thereto (which is the same as Attachment A to this testimony except for minor edits and corrections), I rely upon NUREG/CR-6728 for the basic quantitative input that leads directly to the general proposition that the risk reduction factor, in DOE-STD-1020-94 parlance, for "typical ~~components~~ SSCs" designed to the NRC SRP are in the range of 5 to 20 or greater." See Attachment A at 4. As I describe in Attachment A, this range of risk reduction factors is based on the compilation of 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." Attachment A at 3. I have been associated with many of these evaluations as I have described above. As set forth in Attachment A, this experience is summarized in NUREG/CR-6728 as a factor of safety applicable to "typical components SSCs" for nuclear power plants that corresponds in DOE-STD-1020-94 parlance to a risk reduction factor in the range of "5 to 20 or greater." See Attachment A at 4. Therefore, my reliance on NUREG/CR-6728 is appropriate.

[20] Joint Utah Dec. ¶ 56.

Q61. Please define what you mean by a "fragility curve" referred to in your previous answer.

A61. A fragility curve is a quantitative representation of the capacity of a component or structure with respect to seismic ground motion, reflecting both the engineer's best estimate of that capacity and the uncertainty ~~above~~ about the value of that capacity. Graphically, it is an S-shaped curve that plots the probability of failure versus the level of the ground motion. To develop this curve, the engineer must provide, first and by far most importantly, his best estimate (median) of the SSC' capacity. This determines the midpoint of the S-shaped curve. This estimate must be based on removing all conservatisms inherent in customary engineering calculations. The most realistic judgments should be made, even if they are only estimates of what a more detailed analysis might show conclusively. The estimation of the median capacity is unrelated, in principle, to design basis ground motions, codes and standards, etc. It is much more akin to a scientific prediction than to a conventional engineering design assumption. On the other hand, the median capacity, when compared to the capacity as determined instead by codes and standards and standard engineering practices, becomes a quantitative measure of the conservatisms implicit in those standards and practices. Such conservatisms are inevitable because the purpose of customary calculations is to demonstrate compliance to codes and standards, which dictate conservatisms. In addition standard engineering practices introduce additional conservatisms, e.g., selection of a conservative value to represent scattered material property data, and avoidance of making realistic but potentially contentious assumptions simply to avoid delay of acceptance by reviewers.

The second element in a fragility curve is a figure that reflects the uncertainty in the median estimate.[21] This number reflects how narrowly or

[21] In DOE-STD-1020-94 Appendix C ,this is referred to as beta, β, which is formally the standard deviation of the natural logarithm of the capacity, but it is more easily understood as being very roughly the fractional standard deviation of the capacity. A typical nuclear power plant SSC beta is 0.45 [Ref. 21 (NUREG/CR-6728 at pg. 7-15)], implying the standard deviation is about 45% of the median. With typical (e.g., DOE-STD-1020-94) assumptions, this in turn means that there is

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widely the S-shape spreads about the best estimate or median. Its value is based on the scatter in relevant data and the judgment of engineers as to the limitations of the various physical models used to predict the capacity. This number plays a comparatively smaller role in the fragility curve estimation in that conclusions based on the fragility curve are much less sensitive to it than they are to the median (best estimate) that is used. Once the fragility curve is developed for a particular SSC, it can be used together with the site's probabilistic hazard analysis to estimate the annual probability of failure of the SSC in question. With this annual probability of failure and the mean annual probability of exceedance of the design basis earthquake, one can determine the risk reduction factor inherent in the design of the SSC. In nuclear industry practice, there exist guidelines for the preparation of fragility curves, and hundreds of examples of their use. Some of the general conclusions can be distilled from these examples as to the effect of applying particular codes and standards; hence, for example, the ability to make such statements as the risk reduction factor of a typical component designed to nuclear power plant standards and practice are in the range of 5 to 20 or more.

Q62. What about Drs. Bartlett and Ostadan's claim that the risk reduction factors of 5 to 20 derived from nuclear power plant experience do not apply to unanchored dry storage casks that are free standing on concrete pads and which may slide and tip because the fragility curves relied upon in NUREG/CR-6728 did not include fragility curves for unanchored storage casks?

A62. I agree that the fragility curves for sliding and tipping of freestanding casks were not developed as part of the seismic evaluations on which the 5 to 20 factor for typical nuclear power plant components is based. However, as discussed above, given the decades of NRC's concern about seismic safety, and given the codes, standards and criteria they call for, one would expect *a priori* similar levels of conservatism in any SSC designed to their SRPs, such as the HI-STORM 100 casks, and hence a similar

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about a 84% chance that the capacity will be greater than or equal to about 55% of the median (more precisely, under lognormal assumptions, 63%).

range of R_R levels. In such cases, such a factor could be estimated by conducting a fragility analysis, as Drs. Bartlett and Ostadan call for, but it is necessary here only to demonstrate that the R_R factor is larger than 5; this has been affirmatively demonstrated through various analyses conducted by Holtec and the NRC Staff.

Q63. What is your response to the claims raised by the Drs. Bartlett and Ostadan that PFS cannot rely upon DOE-STD-1020-94 because neither the intent nor the requirements of DOE-STD-1020-94 are met?

A63. Contrary to the claims of Drs. Bartlett and Ostadan, it is not necessary to satisfy the requirements of DOE-STD-1020-94 in order to demonstrate acceptable seismic design of the PFSF, and I am not suggesting such a reliance. The purpose of my testimony (both above and in my November 9 declaration) is not to show explicit compliance with the various acceptance criteria embodied in DOE-STD-1020-94. Rather, I use DOE-STD-1020-94 to demonstrate that there is important support in the industry for the use of a risk graded approach to seismic safety, and as a way to demonstrate the general principles involved in applying a risk graded approach.

In this latter respect, DOE-STD-1020-94 clearly demonstrates that in applying a risk-graded approach the level of seismic performance achieved by a facility's design is a function of both the mean annual probability of exceedance ("MAPE"), or mean return period ("MRP"), of the design basis earthquake and the conservatisms embodied in the applicable design codes, standards and acceptance criteria (formally referred to in DOE-STD-1020-94 as the "risk reduction factor incorporated in the design").[22] Thus, as recognized at one point by Drs. Bartlett and Ostadan, I am using DOE-STD-1020-94 as an "analogy."[23] DOE-STD-1020-94 explicit use of a DBE MRP and a "risk reduction factor" shows, by analogy, that it is appropriate to look at the PFSF DBE and the margins inherent in the PFSF

[22] These are principles with which the State's primary expert, Dr. Arabasz, agrees (Arabasz Decl. ¶ 38) and with which Drs. Bartlett and Ostadan also appear to agree. Utah Joint Decl. ¶¶ 22-23.

[23] Id.

seismic design bases as the bases for establishing whether the design of the PFSF SSCs provides an acceptable level of seismic performance.

Q64. You referenced Drs. Bartlett and Ostadan's acknowledgement of your use of DOE-STD-1020-94 as an "analogy." Is there merit in their claim that the conservatisms that PFS believes to exist in its seismic design bases "cannot be analogized to the risk reduction factors in DOE Standard 1020" because PFS has not conducted the "full panoply of analyses required" by DOE-STD-1020-94?[24]

A64. This claim of Drs. Bartlett and Ostadan reflects their mistaken view that I rely upon DOE-STD-1020-94 as the authoritative source for the actual seismic risk reduction factors applicable for the PFSF design. That is not the case. As stated above, I rely upon DOE-STD-1020-94 to demonstrate the interplay between the role of the mean return period for the design basis earthquake of a structure and the level of conservatism in its seismic design. The source of my opinion of the applicable seismic risk reduction factors for the PFSF are, as discussed above, (1) the nuclear codes and standards to which SSCs, important to safety at the PFSF, are designed and the conservatism shown to exist for typical components designed and constructed to those codes and standards, supplemented by the testimony of other PFS witnesses who describe and quantify some of the conservatisms in the PFSF design, and (2) specific analyses undertaken to demonstrate the conservatism inherent in the PFSF design, such as the cask stability analyses performed by Holtec and the NRC Staff for the 10,000-year earthquake. Based on this information, I have concluded with no reliance on DOE-STD-1020-94 that the applicable risk reduction factor for PFSF SSCs, important to safety, is 5 or more, and that, together with the 2000-year DBE, achieves a seismic safety performance goal of 10^{-4}, or lower.

Q65. What about Dr. Bartlett and Dr. Ostadan's specific claim that it is necessary to generate "fragility" curves as described in DOE-STD-1020-94 for each SSC important to safety in order to evaluate its seismic design capacity?

A65. As stated above, fragility curves are quantitative descriptions of the expected conservatisms or margins in the design of components and the un-

[24] Utah Joint Decl. ¶ 23.

certainty in these margins. While a fragility curve can be developed to show quantitatively the value of a component's risk reduction factor, it is not required to generate a fragility curve to confirm that a particular component has a risk reduction factor larger than some specified level or can meet a specified seismic performance level.

First, as discussed above, extensive experience has been developed to show that typical SSCs designed to meet the design codes, standards and acceptance criteria specified in the NRC's standard review plans have seismic risk reduction factors of 5 to 20 or more. It is not necessary to generate fragility curves for such typical SSCs to determine whether that they have a risk reduction factor of at least 5, which, together with the 2000-year PFSF DBE is all that is required here to confirm that they will meet a seismic performance goal of 1×10^{-4}. (DOE-STD-1020-94 does not itself require the generation of fragility curves for such typical SSCs, to confirm a risk reduction factor of 5 or more; it only requires that the component be designed to DOE-STD-1020-94 PC3 criteria.) Further, one can in other ways demonstrate that a SSC meets at least a specified performance goal without generating a fragility curve for the SSC. For example, if the expected (e.g., mean or median) capacity of the component is somewhat larger than a ground motion with a MAPE equal to a performance goal (e.g., 1×10^{-4}), then it meets the goal.[25] Again, it is important to keep in mind the difference between the median capacity in the fragility curve and the design basis arrived at by applying relevant codes and standards. In the former case conservatisms (such as lower bound properties, static and linear behavior assumptions in place of realistic dynamic and nonlinear considerations) are removed and replaced by best engineering judgments. Yet another way to confirm that the performance goal is met is to show that the probability of failure (e.g., failure to maintain containment of hazardous material) is less than the specified performance goal. For example, if the performance goal is 10^{-4} and the component is estimated not likely to fail under a ground motion with an annual probability

[25] This approach is referred to as a "median-centered" in DOE-STD-1020-2002 (Ref. 18 at C-4).

of exceedance that is less than the performance goal, then the goal has been met..

Q66. Based on what you just stated, is it necessary to generate a fragility curve for the HI-STORM 100 cask, as claimed by Drs. Bartlett and Ostadan, to show that the HI-STORM 100 cask would meet a seismic performance goal of 1×10^{-4}?

A66. No. It is not necessary to develop fragility curves to make the judgment that the HI-STORM 100 cask system will achieve a performance goal of 1×10^{-4} or better. Rather, following the logic that I described just above, one can determine that the HI-STORM 100 cask meets a seismic performance goal of 1×10^{-4} based on the Holtec and Sandia evaluations of the HI-STORM 100 cask system. Based on Holtec's prediction of no cask tip-over under the 10,000-year ground motion and of no release should a cask tip over,[26] it can be concluded that the loss of containment of hazardous material is unlikely given a 10,000-year ground motion, and that the annual probability of loss of containment will be less than 1×10^{-4}. Further, the evaluation performed by Sandia shows that under the 10,000 year ground motion no sliding impact between casks will occur[27] and, as testified to by Drs. Singh and Soler, even if such impact were to occur the velocities and damage of such impacts would be much less than those associated with cask tip-over for which it has been shown that there is no release of radioactivity. Therefore, one can judge that the probability of unacceptable seismic performance due to cask sliding is less than that associated with cask tip-over, i.e., less than 1×10^{-4}. Thus, no fragility curves are necessary to make an informed determination that the HI-STORM 100 cask system will achieve a performance goal of 1×10^{-4} or better at the PFSF.

Q67. Do you then disagree with the claim made by Drs. Bartlett and Ostadan that the selection of "appropriate risk reduction factors can only adequately be conducted

[26] See Testimony of Krishna P. Singh and Alan I. Soler on Unified Contention L/QQ (April 1, 2002).

[27] Vincent K. Luk, Jeffrey A. Smith and David A. Aube, "Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage Facility," Sandia National Laboratories, March 2002.

by evaluating a thorough uncertainty analysis of the fragility of each SSC at the PFS site, as outlined in DOE-STD-1020-94 and NUREG/CR-6728?"[28]

A67. Yes, I disagree for the reasons I just stated.

Q68. What about the similar claim by the State's experts, Drs. Bartlett and Ostadan, that PFS has not met DOE-STD-1020-94 requirements for foundation failure through, overturning, or sliding or bearing capacity failure?[29]

A68. As stated above, neither I nor PFS is relying on meeting DOE-STD-1020-94 acceptance criteria, so it is not necessary to satisfy the DOE acceptance criteria discussed by Drs. Bartlett and Ostadan in evaluating whether foundations meet a particular seismic performance goal. *A priori* one would strongly expect foundation designs to have safety levels close to those of other NPP elements. While foundation stability and sliding fragility curve calculation at NPPs have been comparatively limited, foundations under safety-related buildings, tanks, and other structures, etc., are present at every NPP, and their performance is considered in seismic PRA and margins studies. It would seem unlikely that, in the closely monitored NRC process, where margins against seismic failures have been the subject of more than two decades of investigation, foundations would be allowed to have lower levels of safety than these structural/mechanical SSCs. As discussed earlier, the risk reduction factors of structural and mechanical SSCs have been found to be 5 to 20 or more. In any case, using the "median-centered" argument I described above, the expected stability (overturning and bearing failure) margins for the CTB and pad foundations are judged, as discussed previously, to be in excess of that needed to confirm that their risk reduction factors are 5 or greater.

Q69. Dr. Ostadan also claimed that revision of DOE-STD-1020-94 to change the DBE for PC3 SSCs from a 2000 to a 2500 MRP earthquake would invalidate the use of the 2000 MRP as the DBE earthquake for the PFSF.[30] Do you agree?

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[28] Utah Joint Decl. ¶ 59.

[29] Utah Joint Decl. ¶ 41.

[30] Joint Utah Decl. ¶ 31.

A69. No. Dr. Ostadan's claim reflects an apparent fundamental misunderstanding of the risk-graded approach to seismic safety incorporated into DOE-STD-1020-94 and the purpose of my reference to DOE-STD-1020-94. As stated above, under the risk-graded approach satisfactory performance is a function of both the mean return period of the design basis earthquake and the level of conservatism embodied in the design of the SSC. I refer to DOE-STD-1020-94 as an example of how this risk-graded approach is applied. I do not rely upon the DOE Standard for either the appropriate DBE or the risk reduction factor appropriate for the PFSF. Therefore, the recent change in DOE-STD-1020-2002 of the DBE for PC3 SSCs from a 2000 MRP earthquake to a 2500 MRP earthquake does not affect my analysis of the appropriateness of using a 2000-year MRP as the DBE for the PFSF. This is particularly true given that the seismic performance goal for PC3 SSCs remains unchanged in DOE-STD-1020-2002 at 1×10^{-4}. Thus, DOE's conclusion regarding an appropriate performance goal for ISFSI SSCs, which is the final product under a risk-graded approach to seismic safety, has not changed in DOE-STD-1020-2002. Indeed, as discussed above, in raising the DBE for PC3 SSCs to 2500 MRP DOE-STD-1020-2002 simultaneously reduced the level of conservatism required for the design of PC3 SSCs, thereby consciously keeping the performance goal the same.

Q70. Would you please summarize your above responses to the claims raised by Drs. Bartlett and Ostadan?

A70. The general claims made by the State's witnesses are that: (1) PSF and I cannot rely on DOE-STD-1020-94 or NUREG/CR-6728 to confirm that, by selecting a 2000-year DBE and applying NRC SRP design standards, the PFSF SSCs meets a performance goal of 10^{-4} per annum with respect to loss of containment of hazardous materials due to a seismic event, and (2) because of possible differences in the nuclear power plant and ISFSI NRC SRPs, the conclusions based on experience with nuclear power plants may not apply. With respect to the first item, we do not rely on these two documents in the ways alleged by the State's witnesses. We rely on NUREG/CR-6728 only for the range of values it provides for a particular parameter, which in turn confirms our use of the range 5 to 20 or more for the risk reduction factor for typical nuclear power plant components similar to those in use at PFSF. We do not follow the criteria or specific methods of DOE-STD-1020-94, but rely on it only in support of the proposition that a performance goal of 10^{-4} would be appropriate for the PFSF; we also draw an analogy to DOE-STD-1020-94 in that the DOE Standard treats, as we do, the safety or performance goal as a combination of the level of the DBE and the conservatisms in the design of the PFSF. These conservatisms are a direct product of the codes and standards used in the design, as required by the NRC SRPs. With respect to the second item, at least with respect to the PFSF, the nuclear power plant experience is applicable because the same codes and standards applied at the PFSF are those used in nuclear power plants.

B. SPECIFIC CLAIMS RAISED BY THE STATE OF UTAH IN SECTION E OF UNIFIED CONTENTION UTAH L/QQ

Q71. What claims does the State of Utah raise in Section E of the Unified Contention Utah L/QQ?

A71. In Section E of the Unified Contention Utah L/QQ,[31] the State of Utah asserts that:

> 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

[31] Joint Submittal of Unified Geotechnical Contention, Utah L and Utah QQ (Jan. 16, 2002) at 6-7.

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 NRCs 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 NRC 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 NRC 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.

Q72. Which of these bases will you be addressing in your testimony?

A72. I have already discussed the predicate for item 1, in that my testimony shows that the existing design, based on a 2000-year return period earthquake, provides adequate protection against component failure that would risk exceeding regulatory dose limits. I will also address the remaining bases for the State's Contention, except for basis 2.

Q73. Please describe your understanding of the State's Basis 1.

A73. In Basis 1, 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 SECY discussed three different rulemaking options for the Commission for incorporating PSHA methods into 10 C.F.R. Part 72 with one of the three being identified as the "preferred" option.

Q74. Please describe the preferred methodological approach set forth in SECY-98-126?

A74. The preferred approach set forth in SECY-98-126 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 result in radiological doses exceeding the requirements of 10 C.F.R. § 72.104(a) being designated Category 2 SCCs.

Q75. Is this two-tiered DBE approach still the Commission's preferred methodology for the rulemaking plan to amend 10 C.F.R., Part 72 to incorporate PSHA methods?

A75. No. In SECY-01-0178, dated September 26, 2001, the NRC Staff recommended to the Commission that the rulemaking plan be modified to add a fourth option. This fourth option eliminated the two-tiered DBE approach for ISFSI SSCs and proposed the use a single 2,000-year mean return period earthquake as the design basis for all ISFSI SSCs. This is the same DBE as that provided for by the proposed exemption for the PFSF. SECY-01-0178, identified this fourth option that would provide for the use of a single 2,000-year mean return period earthquake as the "preferred" option. In a Staff Requirements Memorandum dated November 19, 2001, the Commission approved the modification to the rulemaking plan proposed by SECY-01-0178, further instructing the NRC Staff that the proposed rule should solicit comment on a range of exceedance levels from 5.0E-04 through 1.0E-04.

Q76. Does the PFS proposed exemption conform to this newly identified "preferred" option of the NRC rulemaking plan for amending 10 C.F.R. Part 72 to incorporate PSHA methods?

A76. Yes. It proposes a single DBE for all PFSF SSCs with a mean return period of 2,000 years identical to the preferred option identified in SECY-01-0178.

Q77. Where does that leave the State's Basis 1

A77. I believe that the NRC Staff's action and its approval by the Commission render Basis 1 obsolete.

Q78. Please describe your understanding of the State's Basis 3.

A78. In Basis 3, the State challenges the exemption on the grounds that the NRC Staff's reliance on the lower radiological hazard posed by stand-alone ISFSIs (as compared to commercial power reactors) is based on "incorrect factual and technical assumptions." The alleged incorrect factual and technical assumptions apparently allude to the State's assertion that, per Regulatory Guide 1.165, nuclear power plant "design ground motions would have to correspond to a median annual probability of exceedance of 10^{-5}", and that for sites in the western U.S. a median of 10^{-5} is not equivalent to a mean of 10^{-4} as generally stated by the NRC Staff in its approval of PFS's exemption request.[32]

Q79. Is the State's assertion that nuclear power plant "design ground motions would have to correspond to a median annual probability of exceedance of 10^{-5}" accurate?

A79. No, the assertion is incorrect. First, Regulatory Guide 1.165, as the title of this series of NRC documents implies, only provides general guidance to applicants as to procedures that the NRC Staff would deem acceptable for satisfying the NRC's new probabilistic seismic criteria in 10 C.F.R. § 100.23. Regulatory Guide 1.165 (Ref. 4 at page 1) specifically provides that "Appendix B describes the procedure used to determine the reference probability for the SSE exceedance level that is acceptable to the Staff." Second, although the Guide 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)], this provision of the Guide is, in my opinion, primarily the result of historical circumstances. There was a significant discrepancy between the two assessments of the mean estimates made by the two major Central & Eastern U.S. ("CEUS") seismic hazard studies available at the time of the Guide's preparation. While the two studies differed with respect to the mean estimates, both studies provided similar median 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. This discrepancy between the two studies has, however, since been largely resolved[33] and it

[32] [Ref. 25 (State's Request for Admission of Late-Filed Modification to Basis 2 of Contention Utah L, pp. 8- 11)].

[33] This history is recounted in Ref. 33 (T.C. Hanks, Imperfect Science: Uncertainty, Diversity, and Experts, EOS, Transactions, American Geophysical Union, Vol. 78, No. 35, Sept. 2, 1997, 369, 373, 377). The author concludes: "When LLNL used elicitation techniques more in line

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has been clearly established that the typical SSE at existing plants across the country has a mean annual probability of exceedance of approximately 10^{-4}.

Q80. Where is it documented that the mean annual probability of exceeding the SSE at existing nuclear power plants is approximately 10^{-4}?

A80. That the mean annual probability of exceeding the SSE at existing nuclear power plants in the CEUS sites is on the average about 10^{-4} is demonstrated in DOE-STD –1020-94 at p. C-17 [Ref. 11], in NUREG/CR-6728 at p. 7-14 [Ref. 20], and in DOE Topical Report for Yucca Mountain TR-003 at App. C [Ref. 26]. A set of the relatively recent CEUS sites were those used in the preparation of Regulatory Guide 1.165. See Ref. 26 at pg 12. It has also been demonstrated more recently in the DOE Topical Report II TR-003 at App. C [Ref. 26, also identified as PFS Exhibit FFF.] that this same number is also approximately representative of Western US nuclear power plant sites for which the average mean annual probability of exceeding the SSE is about 2 x 10^{-4}, or 5,000 years.

Q81. Please explain the significance of the fact that it has been clearly established, since the issuance of Regulatory Guide 1.165, that the typical SSE at existing plants across the country has a mean annual probability of exceedance of approximately 10^{-4}?

A81. The significance is that it can now be shown that, for nuclear power plants, there is a uniform DBE MAPE throughout the United States, as measured by the consistent use of the 10^{-4} number. The mean estimate is commonly preferred to the median estimate when making decisions based on uncertain annual probabilities or frequencies. It is preferred (1) because it is sensitive to that uncertainty, usually leading to more conservative estimates when the uncertainty is greater, and (2) because the mean estimate is consistent with formal decision theory which concludes that mean risks should be the basis for decisions in the face of uncertainty; the

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with the EPRI approach, the resulting answers were, within the likely uncertainties of either study, the same." Ref. 33 at 373.

mean accident risk of a facility is in turn proportional to the mean (not the median) estimate of the (uncertain) probability of that accident.

Q82. What estimate, the mean or the median, does the Commission typically use when estimating probabilities?

A82. When faced with uncertain probability estimates, the Commission has generally chosen 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 p. 14 [Ref. 5] and in SECY-00-0077, "Modifications to the Reactor Safety Goal Policy Statement" at p. 6 [Ref. 22]. Thus, in accordance with common practice, the Commission has clearly stated its general preference for the use of mean estimates as opposed to median estimates.

Q83. Based on the above, what conclusion do you draw regarding Basis 3?

A83. I conclude that $1x10^{-4}$ per annum, which has been found to be the mean estimate of the annual probability of exceedance of the design basis earthquake (DBE) of the typical nuclear power plant in all regions of the United States, is the appropriate basis from which to establish, via the principles of the risk-graded philosophy adopted by the Commission, the mean annual probability of exceedance of the DBE of an ISFSI anywhere in the country, including specifically at the PFSF site. This conclusion is independent of how or why the NRC Staff established the acceptable procedure highlighted in Regulatory Guide 1.165. It should be noted that both the original 10 C.F.R. part 72 rulemaking plan (SECY-98-126) and the modified plan (SECY-01-0178) approved by the Commission call for the use of mean probability estimates. Finally, as the State correctly points out, ratio between mean and median estimates of the probabilities is not the same at typical CEUS sites as it is at most WUS sites. If risk-graded DBE decisions were based on median estimates, the result would be non-uniform mean probabilities of seismic accidents at nuclear power plants across the country.

Q84. Please describe your understanding of Basis 4.

A84. In Basis 4, the State challenges the exemption granted to PFS on the grounds that the NRC Staff inappropriately relied on DOE-STD-1020-94 (or DOE-STD-1020), which also provided for a 2000 MRP earthquake for ISFSIs, because the NRC Staff did not adopt this Standard in SECY-98-126.

Q85. What is the significance of DOE-STD-1020-94?

A85. As discussed above, DOE-STD-1020-94 is illustrative of the risk-graded approach toward seismic analyses. DOE-STD-1020-94 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. Further, DOE-STD-1020-94 also clearly illustrates the general principle, embodied in using a risk-graded approach, that the probability of failure depends on both the DBE MRP and the level of conservatism in design procedures and criteria. It does so by establishing performance goals for acceptable seismic performance that are expressly the product of the DBE MRPs and the level of conservatism in design procedures and criteria, formally referred to in DOE-STD-1020-94 parlance as the risk reduction factor, R_R. It was for these reason that I used DOE-STD-1020-94 above to illustrate the application of a risk graded approach.

Q86. Has DOE-STD-1020-94 been updated?

A86. Yes, the DBE for category PC3 structures (the category in which ISFSIs would fall were they DOE facilities) has recently been changed from 2,000 years to 2,500 years.

Q87. Does this affect your opinion of whether DOE-STD-1020-94 is relevant to and supports the NRC Staff's approval of the PFS exemption?

A87. No. As just stated, under DOE-STD-1020-94 acceptable seismic performance is the product of the DBE MRPs and the level of conservatism in design procedures and criteria. While the DBE MRP for PC3 structures was increased to 2,500 years, the level of conservatism in the applicable design procedures and criteria was reduced such that the performance goal for PC3 structures remains unchanged at 1×10^{-4}. The State's expert witness, Dr. Arabasz, has stated that he supports the use of the DOE PC3 performance goal of 10^{-4} for the PFSF. Arabasz Dep. at 80-81.

Q88. Please describe your understanding of Basis 5.

A88. In Basis 5, the State challenges the grant of the PFSF exemption claiming that the NRC 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 the 2,000-year return period value of 0.30 g.

Q89. Assuming for the sake of the argument that these differences in circumstances between the PFSF and the INEEL ISFSI exist, would they affect the appropriateness of using the 2,000-year MRP earthquake as the DBE for the PFSF?

A89. No. As discussed above, application of well established risk-graded principles to the specific circumstances of the PFSF show that use of a 2,000 MRP DBE for the PFSF provides sufficient protection to the public health and safety in accordance with established Commission use of risk-informed principles in its regulatory functions. The fact that a similar conclusion was reached for the INEEL ISFSI corroborates the appropriateness of this conclusion.

Q90. Please describe your understanding of Basis 6.

A90. In Basis 6, 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 ground motion 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 "IBC-2000") will, when in effect, require a MRP of approximately 2500 years for the DBE, which is greater than the 2,000-year MRP DBE proposed for PFS.

Q91. Does this difference in definition of the DBE imply a lower probability of failure if an SSC is designed to IBC-2000 codes?

A91. No. One should not draw the erroneous conclusion that the 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 I stated previously, the level of safety achieved depends on *both* the DBE MRP and on the design procedures and criteria utilized. The State's witness, Dr. Arabasz, expressly agrees that one needs to consider both the level of DBE MRP and the level of conservatism in the design in determining unacceptable seismic response of a structure. Utah Joint Decl. ¶ 38. 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's witness,[34] 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.

Moreover, even for those "essential structures" for which this reduction is in effect recovered, 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

[34] State of Utah's Objections and Response to Applicant's Seventh Set of Formal Discovery Requests to Intervenor State of Utah (Sept. 28, 2001) at 18.

allowances for the benefits of post-elastic behavior than either DOE-STD-1020-94 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 in net effect quite similar to those in IBC-2000 and to DOE-STD-1020-94 PC1 and PC2, is a risk reduction ratio R_R of only 2, versus a value of 10 for DOE-STD-1020-94 (PC-4) and typically 5 to 20 or more for the facilities designed to the NRC SRPs These differences represent a factor of 2.5 to 10 or more in increased conservatism (as measured by R_R) in the design procedures for nuclear facilities versus those in model building codes, even if the multiplier of two-thirds in the IBC-2000 is ignored.

Q92. What conclusion do you draw with respect to the State's claim in Basis 6 as it relates to the IBC-2000?

A92. 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, the more conservative design procedures and criteria of the ISFSIs SRP will provide that the typical PFSF SSCs have a mean annual probability of failure several times (2 to 8 or more) lower than buildings designed to IBC-2000 standards. Moreover, all PFSF important-to-safety SSCs have a risk reduction factors sufficient to provide a probability of failure of 10^{-4} or lower, i.e., at least two times lower than essential facilities designed to the IBC-2000. In addition, as discussed above, a number of key important-to-safety SSCs in the PFSF have great robustness and/or fractional operating periods, which reduce their probabilities of failure even further

Q93. How does the PFSF design compare to the bridge codes cited by the State?

A93. With bridge codes, like Dr. Bartlett, the State's witness[35], it is my understanding that, the AASHTO (American Association of State Highway Transportation Officials) model bridge code is used almost universally in the U.S. and that the currently governing version requires only a 500-year return period DBE. Further, it is my understanding that they have structural design procedures and criteria similar in conservatism to those of

[35]Deposition of Steven F. Barlett (Nov. 2, 2001) at 75-76.

model building codes such as UBC and IBC-2000. Therefore, assuming that a 2,500-MRP DBE is used in place of the 500-year value for 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.

Q94. The State also 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. Does the fact that the PFSF license may be extended for twenty years have any affect on the appropriate choice of a design basis earthquake?

A94. No. In virtually all areas of public safety hazards are measured in terms of frequency of occurrence (e.g., as measured in annual probabilities, in probabilities per 50-year period, or in per human lifetime units), and the same safety criteria are specified 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 <u>Frequency</u> (CDF) and Large Early Release <u>Frequency</u> (LERF). [Ref. 5 (Reg. Guide 1.174 at p. 10)] and [Ref. 22 (SECY-00-0077 at p. 6)], usually measured in per annum terms. The reasons for focusing on frequencies such as 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.

Q95. Does this conclude your testimony?

A95. Yes, it does.

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 [Ref. 11 and Ref. 18] and the NRC SRPs [e.g., Ref. 2 (NUREG 0800)]. 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-STD-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-STD-1020. (As an example, Figure C-4 on page C-11 of DOE-STD-1020-94 [Ref. 11] shows the range of R_R values for SSCs designed to the criteria specified for category PC4 SSCs in DOE-STD-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-STD-1020-94 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-STD-1020-94 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-STD-1020 R_R determinations a range of values for the hazard curve slope $K_H = 2.1$ to 3.3 [Ref. 21 (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-STD-1020 at pg. C-8-9, and C-12)[36], 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 PFSF site for peak ground acceleration is 2.8, as I determined from [Ref. 28 (Revised Geomatrix Appendix F at Fig. 6-11)]. For simplicity, I use here a typical value[37] of $\beta = 0.4$. (The conclusions are quite insensitive to β as shown in DOE-STD-1020-94 [Ref. 11] 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-STD-1020 and the NRC SRP.

First, I consider the DOE-STD-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-STD-1020-94 at Eq. C-6, pg. C-9 [Ref. 11]. For PC4 the value of the "scale factor" SF is set at 1.25 (and for PC3 it is set[38] at 1.0) in order to achieve the desired risk reduction ratio R_R [Ref. 11 (DOE-STD-1020-94 at pg. 2-13)]. Substitution of the above values

[36] 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-STD-102-94 at pg. C-8 [Ref. 11].

[37] Ref. 21 (NUREG 6728) at pg. 7-15) cites an average value of 0.45.

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 to 17 for DOE-STD-1020 category PC4, as can be seen on Figure C-4 on page C-11 of DOE-STD-1020-94. The results of these and similar calculations were used in DOE-STD-1020 to confirm the conclusion that the DOE-STD-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-STD-1020-94 at p. C-12 [Ref. 11].

Unlike DOE-STD-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 [Ref. 21] 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 [Ref. 21]. 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

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Footnote continued from previous page

[38] As described in the body of my testimony, this number has been changed to 0.9 in DOE-STD-1020-2002.

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-STD-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-STD-1020 calculations above) that the R_R for the NRC NPP 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-STD-1020 PC4 standard does indeed only "approach" those of the NRC NPP SRP, as stated in DOE-STD-1020-94 at page C-5 [Ref. 11].

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 NPP 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 [Ref. 28].

For simplicity in the body of my testimony 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".

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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.

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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 Probabilty 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

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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:

Torres, G. G. B., Brotchie, J. R., and Cornell, C. A., "A Program for the Optimum Design of Prestressed Concrete Highway Bridges", *Journal of the Prestressed Concrete Institute*, Vol. 11, No. 3, June, 1966.

Reinschmidt, K. F., Cornell, C. A., and Brotchie, J. R., "Iterative Design and Structural Optimization", *Journal of the Structural Division*, ASCE, Vol. 92, No. ST6, December, 1966, pp. 281-318.

Sturman, G. M., Albertson, L. C., Cornell, C. A., and Roesset, J. M., "A Computer-Aided Bridge Design System", *Journal of the Structural Division*, ASCE, Vol. 92, No. ST6, December, 1966, pp. 141-165.

Cornell, C. A., "Bounds on the Reliability of Structural Systems", *Journal of the Structural Division*, ASCE, Vol. 93, No. ST1, February, 1967, pp. 171-200.

Ayer, F. and Cornell, C. A., "Grid Moment Maximization by Mathematical Programming", *Journal of the Structural Division*, ASCE, Vol. 94, No. ST2, February, 1968, pp. 529-549.

Cornell, C. A. and Vanmarcke, E. H., "Some Practical Implications of Elementary Safety Analysis", *Journal of the Boston Society of Civil Engineers*, Vol. 55, No. 3, July, 1968.

Cornell, C. A., "Engineering Seismic Risk Analysis", *Bulletin of the Seismological Society of America*, Vol. 58, No. 5, October, 1968, pp. 1583-1606.

Cornell, C. A., "A Probability-Based Structural Code", *Journal of the American Concrete Institute*, No. 12, Proc. Vol 66 December, 1969, pp. 974-985.

Corotis, R. B., Vanmarcke, E. H., and Cornell, C. A., "First Passage of Non-Stationary Random Processes", *Journal of the Engineering Mechanics Division*, ASCE, No. EM2, April, 1972, pp. 401-414.

Peir, J. C. and Cornell, C. A., "Spatial and Temporal Variability of Live Loads", *Journal of the Structural Division*, ASCE, Vol. 99, No. ST5, May, 1973, pp. 923-943.

McGuire, R. K. and Cornell, C. A., "Creep of Concrete Under Stochastic Live Load", *Journal of the Structural Division*, ASCE, Vol. 99, No. ST5, May, 1973, pp. 923-943.

Merz, H. A. and Cornell, C. A., "Seismic Risk Analysis Based on a Quadradic Magnitude-Frequency Law", *Bulletin of Seismological Society of America*, Vol. 63, No. 6, December, 1973, pp. 1999-2006.

McGuire, R. K. and Cornell, C. A., "Live Load Effects in Office Buildings", *Journal of the Structural Division*, ASCE, Vol. 100, No. ST7, July, 1974, pp. 1351-1366.

Ang, A. H. S. and Cornell, C. A., "Reliability Bases of Structural Safety Design", *Journal of the Structural Division*, ASCE, Vol. 100, No. ST9, September, 1974, pp. 1755-1770.

Whitman, R. V., et al., "Seismic Design Decision Analysis", *Journal of the Structural Division*, ASCE, Vol. 101, No. ST5, May, 1975, pp. 1067-1084.

Garson, R. C., Morla-Catalan, J., and Cornell, C. A., "Tornado Design Winds Based on Risk", *Journal of the Structural Division*, ASCE, Vol. 101, No. ST9, September, 1975, pp. 1883-1897.

Cornell, C. A. and Merz, H. A., "Seismic Risk Analysis of Boston", *Journal of the Structural Division*, ASCE, Vol. 101, No. ST10, October, 1975, pp. 2027-2034.

Morla-Catalan, J. and Cornell, C. A., "Earth Slope Reliability by a Level-Crossing Method", *Journal of the Geotechnical Division*, ASCE, Vol. 102, No. GT3, June, 1976.

Veneziano, D., Grigoriu, M., and Cornell, C. A., "Vector-Process Models for System Reliability", *Journal of the Engineering Mechanic Division*, ASCE, Vol. 103, No. EM3, Proc. paper 12981, June, 1977, pp. 441-460.

Ravindra, M. K., Cornell, C. A., and Galambos, T. V., "Wind and Snow Load Factors for Use in LRFD", *Journal of the Structural Division*, ASCE, Vol. 104, No. ST9, Proc. Paper 14006, September, 1978, pp. 1443-1457.

Fardis, M. N. and Cornell, C. A., "Containment Liner Seismic Reliability Under Statistical Uncertainty", *Nuclear Engineering and Design*, Vol. 49, No. 3, September, 1978, pp. 279-294.

Fardis, M. N. and Cornell, C. A., "Seismic Soil-Containment Interaction: Pipe Safety", *Journal of the Engineering Mechanics Division*, ASCE, Vol. 104, No. EM6, Proc. Paper 14218, December, 1978, pp. 1353-1370.

Fardis, M. N., Cornell, C. A., and Meyer, J. E., "Accident and Seismic Containment Reliability", *Journal of the Structural Division*, ASCE, Vol. 105, No. ST1, Proc. Paper 14305, January, 1979, pp. 67-83.

Larrabee, R.D. and Cornell, C.A., "Upcrossing Rate Solution for Load Combinations", *Journal of the Structural Division*, ASCE, Vol. 105, No. ST1, Proc. Paper 14329, January, 1979, pp. 125-132.

Cornell, C. A., Shakal, A., and Banon, H., "Seismic Motion and Response Prediction Alternatives", *Earthquake Engineering and Structural Dynamics*, Vol. 7, 1979, pp. 295-315.

Millman, R., Kilcup, R., and Cornell, C. A., "Design Temperature for Structural Elements", *Journal of the Structural Division*, ASCE, Vol. 106, No. ST4, Proc. Paper 15364, April, 1980, pp. 877-895.

Kennedy, R. P., Cornell, C.A., Campbell, R.D., Kaplan, S. and Perla H.F., "Probabilistic Seismic Safety Study of an Existing Nuclear Power Plant", *Nuclear Engineering and Design*, Vol. 59, No. 2, August, 1980, pp. 315-338.

Cornell, C. A., "Some Thoughts on Systems and Structural Reliability", *Nuclear Engineering and Design*, Vol. 60, No. 1, September, 1980, pp. 115-116.

Cornell, C. A., "Utilization of Present Knowledge of Probabilistic Structural Reliability in Analyses of Nuclear Power Plants", *Nuclear Engineering and Design*, Vol. 60, No. 1, September, 1980, pp. 33-36.

Larrabee, R. D. and Cornell, C. A., "Combination of Various Load Processes", *Journal of the Structural Division*, Vol. 107, No. STI, January, 1981, pp. 223-239.

Fardis, M. N. and Cornell, C. A., "Analysis of Coherent Multistate Systems", *IEEE Transactions on Reliability*, Vol. R-30, No. 2, June, 1981, pp. 117-122.

Galambos, T.V., Ellingwood, B., MacGregor, J.G. and Cornell, C.A., "Probability-Based Load Criteria: Assessment of Current Design Practice", *Journal of the Structural Division*, ASCE, Vol. 108, No. ST5, May, 1982, pp. 959-977.

Galambos, T,V., Ellingwood, B., McGregor, J.G. and Cornell, C.A., "Probability-Based Load Criteria: Load Factors and Load Combinations", *Journal of the Structural Division*, ASCE, V. 108, No. ST5, May, 1982, pp. 978-997.

Winterstein, S. R. and Cornell, C. A. "Load Combinations and Clustering Effects", *Journal of the Structural Division*, ASCE, Vol. 110, No. 11, November, 1984, pp. 2690-2708.

Winterstein, S. R. and Cornell, C. A., "The Energy Fluctuation Scale and Diffusion Models", *Journal of Engineering Mechanics*, ASCE, No. 2, February, 1985, pp. 125-142.

Toro, G. R., and Cornell, C. A., "Extremes of Gaussian Processes with Bimodal Spectra", *Journal of Engineering Mechanics*, ASCE Vol. 112, No. 5, pp. 465-484, May, 1986.

Cornell, C. A., "On the Seismology - Engineering Interface", Presidential Address, *Bulletin of the Seismological Society of America*, April, 1988, Vol. 78, No. 2.

Bjerager, P., Loseth, R., Winterstein, S. R. and Cornell, C. A., "Reliability Method for Marine Structures Under Multiple Environmental Load Processes," *Proceedings of the 5th International Conference on Behavior of Offshore Structures*, BOSS, N.I.T., Trondheim, Norway, June, 1988.

Cornell, C. A., and Winterstein, S. R., "Temporal and Magnitude Dependence in Earthquake Recurrence Models", *Bulletin of the Seismological Society of America*, August, 1988, Vol. 78, No. 4.

Karamchandani, A., and Cornell, C. A., "Sensitivity of Simulation Estimates to Changes in Distribution Parameters", *Submitted to ASCE*, 1990.

De, R. S., Karamchandani, A., and Cornell, C. A., "Offshore Structural System Reliability Under Changing Load Pattern", *Applied Ocean Research*, V. 13, No. 3, June, 1991.

Karamchandani, A., and Cornell, C. A., "An Event-to-Event Strategy for Non-Linear Analysis of Truss Structures", *ASCE Structural Division*, Vol. 118 No. 4, April, 1992, pp. 895-925.

Karamchandani, A., and Cornell, C. A., "Reliability Analysis of Truss Structures with Multi-State Elements", *Jo. of Str. Engrg., Vol. 118, No. 4*, April, 1992.

Karamchandani, A., and Cornell, C. A., "Adaptive Hybrid Conditional Expectation Approaches for Reliability Estimation", *Structural Safety*, Vol. 11, No. 1, November, 1992, pp. 59-74.

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June 27, 2002

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

REBUTTAL TESTIMONY OF C. ALLIN CORNELL TO THE TESTIMONY OF STATE WITNESS DR. WALTER J. ARABASZ ON SECTION E OF UNIFIED CONTENTION UTAH L/QQ

Q1. In Answers 14-15 of his pre-filed testimony, Dr. Arabasz takes issue with the position stated in paragraph 49 of your November 7, 2001 declaration that 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." See "State of Utah Testimony of Walter J. Arabasz Regarding Unified Contention Utah L/QQ (Seismic Exemption)," April 1, 2002 ("Arabasz Direct Testimony"). According to Dr. Arabasz, many standards make use, not of annual probabilities, but of probabilities of exceedence in units such as 10%, 5% or 2% in 50 years. What is your response to Dr. Arabasz's criticism?

A1. Stating probabilities of exceedence in such terms as a 10% probability of exceedence in 50 years (as opposed to annual probability of exceedence of $2x10^{-3}$) is just a different way of presenting the frequency of occurrence. Neither method of specifying frequency is predicated on the lifetime of a facility, nor does the application of the standard vary depending on a facility's projected lifetime. This is clearly reflected in the quotation on page 15 of Dr. Arabasz's direct testimony from the National Research Council's Panel on Seismic Hazard Analysis, which directly equates a design seismic hazard level with a 10% probability of exceedence in 50 years to an annual probability of exceedence of $2x10^{-3}$.

Thus, for example, applying a seismic standard of 10% probability of exceedence in 50 years to two buildings, one constructed for a 10 year lifetime and the other for a 100 year lifetime, would result in the same annual probability of exceedence of $2x10^{-3}$ for each building. Therefore, the examples cited by Dr. Arabasz confirm my basic thesis, which is that in these codes and criteria the frequency of occurrence used is (and should be) independent of the length of the lifetime of the facility or item at risk. All that his examples confirm is that different standards use different units for measuring frequency.

Q2. In his testimony at the hearing (Tr. 10164-10170), Dr. Arabasz acknowledged the potential for logical inconsistencies that might result from adopting a design return period proportional to the duration of the facility lifetime. The two examples discussed were (1) that under a facility lifetime-dependent approach a reduction of the plant design life could lead to perhaps unreasonably reduced design return periods; and (2) that ambiguities could arise in a nuclear power plant ("NPP") re-licensing application of a plant whose original lifetime has expired. Dr. Arabasz further stated that under the DOE 1020 paradigm the lifetime-independent, annual frequency approach would be appropriate and preferable, but that lacking "the pertaining regulatory guidance . . . and clearly established framework for decision making" (such as that in DOE 1020) would apparently lead him in the direction of a duration-dependent safety criterion here. (Tr. 10170) Do you believe that a clearly established framework for decision making based on a lifetime-independent, annual frequency approach is lacking in the NRC arena?

A2. No. In my written testimony I cite several NRC documents that attest to that agency's clear adoption of annual frequency as the appropriate basis for safety criteria and a risk-informed decision-making framework. For example, both the Commission's Reactor Safety Policy Statement [(SECY 00-007), Ref. 22 of my direct testimony] at p.6 and Regulatory Guide 1.174 [Ref. 5] clearly set forth frequency-based risk acceptance guidelines for NPPs where the performance objectives are Core Damage Frequency and Early Large Release. While these statements were made in connection with the adoption of frequency-based guidelines for NPPs, the same principles apply to ISFSIs, such as the PFSF.

Another example of the logical inconsistencies that may arise from tying the frequency standard to lifetimes is in the area of worker safety. Worker safety criteria are typically are measured in terms of the "probability of death per worker lifetime" (not per annum). However, no such standard to my knowledge differentiates between a 65-year-old worker (whose remaining lifetime is likely to be short) and a younger worker. In other words, while the frequency of occurrence in this example is expressed in units "per lifetime," the standards are not applied differently depending on a person's remaining lifetime. Indeed, the use of a duration-dependent worker safety criterion would lead to implications to which many of us of the older generation would not react kindly. Compared to our younger workplace colleagues, those of us with only, say, a decade of work ahead of us could be subjected, by the application of such a duration-dependent standard, to significantly reduced work place protection standards: lesser protection against cancer-inducing activities (e. g., working with asbestos), no shields around dangerous equipment, etc.

Q3. In response to questions by Judge Lam (Tr. 10047-50), Dr. Arabasz agreed "emphatically" with your testimony that, in seeking to achieve an acceptable risk of failure of SSCs for ISFSIs, it was appropriate to use a "two hand approach" which took into account, on the one hand, the robustness and conservatism of the design of the SSCs and, on the other hand, the regulatory standard on hazard probability. Dr. Arabasz, however, opined that the levels of conservatism in the design of some of the SSCs for an ISFSI (such as storage casks) may not have been established to the same level of confidence as for nuclear power plant SSCs. For that reason, he suggested that the desired low level of overall risk might not be achieved unless the hazard probability was set sufficiently high. Do you agree with Dr. Arabasz's position?

A3. I agree with Dr. Arabasz's reasoning but not with the premise on which his position is based. Thus, I agree with Dr. Arabasz that, in assessing what seismic safety level has been achieved, one cannot depend solely on either the conservatism in the design or the mean return period of the design basis ground motion. Dr. Arabasz and I apparently agree that both of the two hands must be recognized to make informed public safety decisions. We also agree that for SSCs typical of NPPs we can have confidence that the NRC SRPs will insure very significant levels of robustness in the design; hence, the 2000 year return period will achieve the desired performance goal (i.e., an SSC failure probability of 10^{-4} or less) with high degree of confidence.

Further, Dr. Arabasz is correct in saying that storage casks for ISFSIs do not fall into this "NPP-typical" category, and that some further analysis is necessary to provide confidence that the desired performance goal for these components has been achieved. However, both the NRC staff and PFS have conducted beyond-design-basis analyses of these casks and their foundations with the aim of achieving such levels of confidence. In my view, the analysts of both PFS and the staff have demonstrated (using conservative assumptions with respect to key parameters such as the friction coefficient) that under the 10^{-4} year return period ground motion, the casks to be used at the PFSF site will not tip over. These demonstrations are in themselves sufficient to provide good evidence that a performance goal in the order of 10^{-4} has been achieved. A further determination has also been made, i.e., that no release would occur even if the casks were to tip over. This further conclusion provides still greater confidence that the annual probability of radioactive releases is less than or equal to 10^{-4}.

Based on the above reasoning, I would answer in the negative Dr. Lam's question as to whether the "design robustness hand" is doing more than its share of heavy lifting vis a vis the "hazard level hand." My negative answer arises from the fact that we are not asking the designs to provide higher levels of performance than what they are naturally capable of providing. For example, as I have testified previously, ductile materials such as steel and reinforced concrete are capable of withstanding dynamic deformations many times larger than their nominal yield levels, and hence are also capable of withstanding ground motion amplitudes multiple times the level that causes the material to reach its nominal yield level. (This capability was recognized by the earthquake engineers when they were for the first time called upon in the 1970s to determine what the realistic seismic margins in existing nuclear power plant SSCs really are.) Thus, the "design robustness hand" is not being unduly emphasized at the expense of the "hazard level definition" hand.

Second, we must keep in mind the chronological sequence of events that have led to the current regulatory standards. Virtually all U.S. NPPs were designed based on Appendix A "deterministic" design basis ground motions and on SRPs that were intentionally more conservative than, for example, corresponding building design standards (e.g., the so-called R or force "reduction factors" in conventional building codes – such as the UBC and IBC, which have been discussed in these proceedings - were not used in the SRPs). Then, PSHA came along and showed that the Appendix A design basis ground motions had a mean return period of about 10,000 years. At about the same time, seismic PRA engineers were conducting the analyses that showed just how robust were the SSCs designed to the NPP SRP. Their conclusion was summarized in R_r ("risk reduction ratios") of 5 to 20 or more. The resulting implication that NPP SSCs achieved a performance goal of about 10^{-5} was a product of those studies; it was not a pre-defined target. Thus, the relative roles of the seismic hand and the robustness hand were not pre-selected, but resulted from the inherent beyond-design-basis capability of these components, embodied in nuclear design criteria and practices.

As we today consider the safety implications of similar SSCs in ISFSIs such as the storage casks, we are simply building on and working within the logical framework established in the past for nuclear power plant SSCs.

Q4. Does that conclude your testimony?

A4. Yes.

April 1, 2002

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

TESTIMONY OF KRISHNA P. SINGH, ALAN I. SOLER, AND EVERETT L. REDMOND II ON RADIOLOGICAL DOSE CONSEQUENCE ASPECTS OF BASIS 2 OF SECTION E OF UNIFIED CONTENTION UTAH L/QQ

I. WITNESSES AND SCOPE OF TESTIMONY

A. Krishna P. Singh ("KPS")

Q1. Please state your full name?

A1. Krishna P. Singh.

Q2. By whom are you employed and what is your position?

A2. (KPS) I am President and CEO of Holtec International ("Holtec"). My educational and professional qualifications are summarized in Testimony of Krishna P. Singh and Alan I. Soler ("Singh/ Soler Testimony") with respect to Sections D and E of Unified contention L/QQ, being filed simultaneously herewith.

B. Alan I. Soler ("AIS")

Q3. Please state your full name?

A3. Alan I. Soler.

Q4. Please summarize your educational and professional qualifications.

A4. (AIS) I am Holtec's Vice-President of Engineering. My educational and professional qualifications are summarized in the Singh/Soler Testimony, being filed simultaneously herewith.

C. **Everett L. Redmond ("ELR")**

Q5. Please state your full name?

A5. Everett L. Redmond, II.

Q6. By whom are you employed and what is your position?

A6. (ELR) I am a Principal Engineer and Manager of the Nuclear Physics Department with Holtec. I am responsible for all shielding, criticality, and confinement analysis work related to Holtec's dry cask storage systems. I am the author of the shielding analyses performed in support of the general NRC certification of Holtec's HI-STORM 100 Cask System under Docket 72-1014. I have also performed site-specific shielding analyses in support of deployment of the HI-STORM 100 Cask System at the Private Fuel Storage Facility ("PFSF") independent spent fuel storage installation ("ISFSI"), the subject of this licensing proceeding.

Q7. Please summarize your educational and professional qualifications.

A7. (ELR) My professional and educational experience is described in the *curriculum vitae* attached to this testimony. As indicated there, my professional background and work experience include significant expertise on matters pertaining to the shielding characteristics of the HI-STORM 100 Cask System and the radiation does associated with the use of the HI-STORM 100 Cask System. My work in those areas has included developing analytical methods and models for conducting shielding analyses and dose calculations, and performing site boundary dose evaluations for ISFSIs.

Q8. What is the basis of your familiarity with the PFSF?

A8. (ELR) Holtec is the supplier of the HI-STORM 100 Cask System that will be used to store spent nuclear fuel at the PFSF. I performed site-specific shielding and radiation site boundary analyses in support of the deployment of the HI-STORM 100 Cask System at the PFSF. Through the performance of those analyses, I have become familiar with the site-specific characteristics of the cask layout arrangement at the PFSF ISFSI, the distance to the site boundary, and other factors used to calculate radiation dose rates at the site boundary due to normal, off-normal, and postulated accident conditions at that facility.

D. **Scope of Testimony**

Q9. What is the purpose of your testimony?

A9. (KPS, AIS, ELR) The purpose of our testimony is to respond on behalf of Private Fuel Storage LLC ("PFS" or "Applicant") to certain radiological dose consequences issues raised by the State, with respect to Basis 2 of Section E of Unified Geotechnical Contention, Utah L QQ, in which the State asserts:

> 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 2000 years, in that:
>
> * * * *
>
> 2. PFS has failed to show that its facility design will provide adequate protection against exceeding the section 72.104(a) dose limits.

Q10. What assertions has the State made in regard to the radiological dose consequences of allowing PFS to use a probabilistic seismic hazard analysis with a 2,000-year return period?

A10. (KPS, AIS, ELR) In a declaration dated December 7, 2001, filed in Support of the State's Opposition to PFS's Motion for Summary Disposition on this part of the contention, State witness Dr. Marvin Resnikoff asserts that PFS has failed to adequately and conservatively calculate the potential increase in dose rates following a beyond design basis seismic event at the PFSF site. Specifically, Dr. Resnikoff asserts that:

i) Multiple cask tipovers at the PFSF will result in exceedance of the 25 mrem dose limit of 72.104(a). Resnikoff Decl. ¶ 14-15.

ii) There are significant differences between the PFSF site and the Holtec Cask Certificate of Compliance ("CoC") (id. ¶ 12) which invalidate the PFS analysis of cask tipover impacts.

iii) PFS has neither quantified the damage to the casks that would result from tipover of the casks, nor calculated the resulting radiation dose to workers or at the boundary; PFS's claim of negligible increase in radiation from tipped over casks is not supportable, and PFS "must calculate a bounding radiation dose at the fence line and to workers" (id. ¶ 19-24).

iv) PFS has not analyzed the effects of an increase in neutron dose due to concrete degradation to on site workers in the event of a prolonged tipover (id. ¶ 25-26).

v) PFS has not analyzed damage to the casks and potential increase in radiation due to collision among sliding casks (id. ¶ 27).

vi) PFS has not analyzed damage to the casks and potential increase in radiation due to lifting up of casks during an earthquake event (id. ¶ 28).

vii) The cask drop calculation of a stainless steel MPC from 25 feet does not evaluate the stresses that would result if the MPC were dropped on its edge (id. ¶ 29).

Q11. (KPS, AIS, ELR) Do you agree with Dr. Resnikoff's claims?

A11. No, we do not.

Q12. Why not?

A12. (KPS, AIS, ELR) First of all Dr. Resnikoff uses the wrong dose limits. His entire position is based upon the incorrect assumption that the applicable dose limit is the 25 mrem limit of 10 CFR § 72.104 for "normal operations and anticipated occurrences". In reality, a cask tipover during a seismic event is a beyond-design basis accident for which the applicable dose limit is the 5 rem limit of 10 C.F.R. § 72.106(b). Under Dr. Resnikoff's own analysis, the 5 rem limit of 10 C.F.R. § 72.106(b) is nowhere close to being exceeded.

Moreover, the assumptions used in the analyses that document the performance of these components also contain high levels of conservatism. These inherent conservatisms built into the PFSF design clearly establish that the radiological consequences of the postulated 10,000-year beyond design basis earthquake would be within all applicable regulatory dose limits. These analyses and conservatisms show the inaccuracy of Dr. Resnikoff's claims, even in the event of a more severe, postulated 10,000-year return period earthquake.

II. APPLICABLE DOSE LIMITS FOR A BEYOND DESIGN BASIS EVENT

Q13. In his analysis, Dr. Resnikoff evaluates dose consequences of cask tip over based on the dose limits found in 10 C.F.R. § 72.104 using 8760 hours per year to calculate the annual dose under 10 C.F.R. § 72.104 "for normal operations and anticipated occurrences". Is this a correct statement of the applicable dose limits for a hypothetical cask tip over event at the PFSF?

A13. (ELR) No, it is not.

Q14. Why not?

A14. (ELR) First, the dose limits of 10 CFR § 72.104 do not apply to accident conditions. The regulation states directly that the limits found in 10 C.F.R. § 72.104(a) are for "normal operations and anticipated occurrences." A cask tip over is not part of normal operations nor is it an anticipated occurrence. Rather it is a beyond-design basis accident.

Q15. Why is cask tip over an accident condition?

A15. (KPS, AIS, ELR) The HI-STORM 100 cask storage system is designed so that it will not tip over in normal operations, nor even under a design basis accident including a design basis earthquake. A cask tip over is a postulated, hypothetical, beyond-design basis accident condition.

Q16. Based on your experience in designing storage casks to meet radiological dose limits, what is your understanding regarding what dose limits apply to what conditions?

A16. (ELR) In designing storage casks, there are two sets of radiological dose requirements that may be applicable: normal dose limits and accident dose limits. In the event of a design basis accident, the dose due to an accident must be less than 5 rem at the controlled area boundary. Section 72.106(b) provides:

> [a]ny individual located on or beyond the nearest boundary of the controlled area may not receive from any design basis accident the more limiting of a total effective dose equivalent of 0.05 Sv (5 rem), or the sum of the deep-dose equivalent and the committed dose equivalent to any individual organ or tissue (other than the lens of the eye) of 0.5 Sv (50 rem). The lens dose equivalent shall not exceed 0.15 Sv (15 rem) and the shallow dose equivalent to skin or to any extremity shall not exceed 0.5 Sv (50 rem). The minimum distance from the spent fuel or high-level radioactive waste handling and storage facilities to the nearest boundary of the controlled area must be at least 100 meters.

Q17. What about beyond-design basis events?

A17. (ELR) While the regulations do not explicitly address beyond-design basis accidents because they are not part of the regulatory requirements that must be satisfied by a licensee, the same limits set by 10 C.F.R. § 72.106 for accident conditions would apply to the extent that such events are considered and evaluated. For example, the Standard Review Plan for Spent Fuel Storage Facilities, NUREG-1567 (March 2000) provides for evaluation of dose consequences for hypothetical accident conditions under 10 C.F.R. § 72.106(b). NUREG-1567 § 9.5.2.2.

III. EVALUATION OF RADIOLOGICAL DOSES FROM HYPOTHETICAL CASK TIPOVER EVENTS

Q18. Has Holtec evaluated the radiological dose consequences of a hypothetical cask tipover event?

A18. (KPS, AIS, ELR) Yes.

Q19. Please describe the nature of Holtec's evaluation.

A19. (KPS, AIS, ELR) As set forth in the Singh/Soler testimony, Holtec performed a hypothetical cask tip-over analysis for the PFSF even though it has been demonstrated that the casks will not tip over under either the design basis 2,000 year return period earthquake for the PFSF or under a beyond-design basis, 10,000 year return period seismic event. The tipover analysis showed that all stresses remained within the allowable values of the HI-STORM 100 Certificate of Compliance ("CoC") assuring integrity of the multi-purpose canister ("MPC") confinement boundary with large safety margins, as described in the Singh/Soler testimony. Holtec has further qualitatively evaluated the potential radiological consequences of the hypothetical tipover event in its Final Safety Analysis Report ("FSAR") for the HI-STORM 100 Cask System. As discussed there, although the tipover has no effect on the MPC confinement function, it could cause localized damage to the radial concrete shield and outer steel shell where the storage cask impacts the surface. HI-STORM FSAR, § 11.2.3.3. However, because the areas of damage will be small and localized, no noticeable increase in the ISFSI site or boundary dose rates would be expected.

Q20. Has Holtec evaluated the potential dose consequences of multiple cask tipover events at the PFSF:

A20. (ELR) Yes.

Q21. How did Holtec do this evaluation?

A21. (ELR) Holtec reviewed qualitatively the effect that multiple cask tipover events would have on radiation doses at the site boundary compared to the normal dose limits that it had previously calculated for the PFSF site boundary of approximately 5.85 mrem. We determined that the dose consequences at the site boundary from multiple cask tipover events would be similar or less than the normal doses previously calculated and far below the 5 rem accident dose limit of 10 C.F.R. § 72.106(b). Because of the large margin between the normal dose limits calculated for the PFSF and the accident dose limit, there is no need to perform further calculation of the dose consequences of multiple cask tip-over events.

Q22. Please describe the calculation of normal dose limits that Holtec performed for the PFSF site and its results?

A22. (ELR) In the design basis analyses for the PFSF, a radiation dose analysis determined the direct radiation dose rate at the controlled area boundary from neutron and gamma (photon) radiation emanating off of the sides and top of the HI-STORM storage casks. The maximum 4000 casks at the ISFSI were considered in the analysis. The calculations were performed with the Monte Carlo radiation transport code MCNP-4A. Section 7.3.3.5 and Table 7.3.7 of the

PFSF SAR present the results of this calculation and show that a maximum value of 5.85 mrem/year was calculated for a 2000 hour/year occupancy time at the controlled area boundary assuming all casks contained fuel with a burnup of 40,000 MWD/MTU and a cooling time of 10 years. These analyses demonstrated that the doses at the boundary are well within the limits deemed acceptable by the NRC in 10 C.F.R. § 72.104(a) and 10 C.F.R. § 72.106(b) for both normal operations and accident conditions.

Q23. Please describe your comparison of the dose limits arrived at by this calculation to the expected radiological doses for casks in a tipped over condition.

A23. (ELR) In the upright position, the side of the storage cask is visible from all equidistant locations from the HI-STORM storage cask and the top is not visible from any location. Therefore, all equidistant locations from an upright HI-STORM storage cask will have the same dose rates. However, in a tipped over position, the profile of the cask would be considerably different from its upright position. If one were to walk around the tipped over storage cask maintaining a constant distance from its center, the 11 ft. diameter circular ends of the cylinder (the top or bottom of the cask) would be visible from some locations and not from others while the 20 ft. long side of the storage cask cylinder (now in the horizontal position) would also be visible from some locations and not others. Therefore, unlike the upright condition, the dose rate profile around a tipped over HI-STORM storage cask would not be uniform at equidistant locations from the cask. Accordingly, the comparison must take into account the following changes in the dose rate profile of the cask:

a. The top of the cask would be visible although no longer facing the sky. Therefore, the radiation leaving the top of the cask would reach certain locations at the controlled area boundary directly (with due consideration of any attenuation and scattering in the intervening air), as opposed to the strictly scattering effect of sky shine. This would be an increase in the dose rate contribution from the top of the cask. However, at the locations along the controlled area boundary where the top of the cask is now easily visible, the dose rate from the side of the storage cask would be greatly reduced because the line-of-sight to the side of the cask would be reduced.

b. The bottom of the cask, which is normally facing the concrete ISFSI pad and the ground below, would now be exposed. This means that radiation emanating from the bottom of the storage cask, which previously was immediately absorbed by the ground, could now reach locations along the controlled boundary directly, again with due consideration of attenuation and scattering provided by the intervening air. This would also cause an increase in the dose rate contribution from the bottom of the cask. However, at the locations along the controlled area boundary where the bottom of the cask was now easily visible, the dose rate from the side of the cask would be greatly reduced because the line-of-sight to the side of the cask was reduced.

c. Since the storage cask would now be lying on its side, a large portion of the outer radial surface of the cask would be shielded by the ground. In the upright position, all radiation that emanated off the side of the cask was able to scatter and reach the site boundary. In the tipped over position, a significant portion of the radiation leaving the side of the cask would now be unable to reach the site boundary because it would be immediately absorbed by the ground below the side of the cask. In addition, as discussed above, not all locations on the controlled area boundary would have line-of-sight to the side of the cask. This would result in a reduction in the dose rate at the controlled area boundary from radiation emanating off the side of the cask.

Overall, the decrease in dose rate from the side of the tipped over storage cask should more than compensate for the increase in dose rate from the top or bottom of the cask. Based on this discussion, it is my opinion that the dose rate at the controlled area boundary from a HI-STORM storage cask lying on its side would be less than the dose rate from a HI-STORM storage cask in the upright position.

Q24. What is the likelihood of multiple cask tipovers at the PFSF?

A24. (ELR) The storage casks at the PFSF ISFSI are positioned in fifty 2x40 arrays. The arrays of casks are positioned parallel to each other with a spacing of 35 feet between arrays. Because of the positioning of the casks, it is improbable that all 4,000 casks could ever completely tip over and come to rest on their sides on the ground. Even assuming the occurrence of an event that could tip over any of the casks, a more plausible scenario would have some casks lying on the ground while the remainder would be upright in one of two positions: free standing, or leaning against other storage casks.

Q25. Is it possible for all 4,000 casks to tip over?

A25. (ELR) In order for all casks to be resting on the ground, the casks in the 2x40 arrays would have to all fall away from each other into the 35 feet wide pathway between the arrays. In any event, tip over of all 4,000 casks would not change the calculated radiation dose limits.

Q26. What effect would all 4,000 casks tipping over have on the overall radiation dose at the boundaries of the facility?

A26. (ELR) Overall, the decrease in dose rate from the side of the tipped over storage cask should more than compensate for the increase in dose rate from the top or bottom of the cask, which I have described above. Based on this discussion, it is my opinion that the dose rate at the controlled area boundary from a HI-STORM storage cask lying on its side would be less than the dose rate from a HI-STORM storage cask in the upright position. For all casks to successfully tip over, they have to fall in such a way that the tops and bottoms of casks would be facing other casks, which would minimize the dose contribution at the controlled area boundary from radiation emanating off the top and bottom of the casks, since this radiation would be directed toward other storage casks. In the upright position for the ISFSI, the sides of the cask are partially shielded by the position of casks next to each other. This self-shielding would still exist to a degree when all casks are tipped over because they would be lying next to each other. Therefore, based on the response for a single cask, it is my opinion that the dose rate from the entire 4,000 casks at PFSF lying on their sides would be similar to that from the ISFSI with all casks in the upright position.

Q27. How does this expected dose rate for 4,000 tipped over casks compare to the accident dose limit in 10 C.F.R. § 72.106(b)?

A27. (ELR) As stated, the normal dose at the site boundary calculated for 4,000 casks in their upright position used in my comparison is 5.85 mrem. Based on the above analysis, the expected dose rate for 4,000 tipped over casks at the site boundary would be of the same order of magnitude. Thus, there is approximately three orders of magnitude of margin between the expected dose rate at the site boundary for 4,000 casks in a tipped over condition compared to the 5 rem accident dose limit in 10 C.F.R. § 72.106(b).

Q28. Are there any other conservatisms built into your evaluation of radiation doses at the site boundary resulting from 4,000 tipped over casks?

A28. (ELR) Yes, there are other significant conservatisms. The analyses that Holtec performed for the PFSF in the PFSF SAR for normal doses include a number of conservative assumptions that tend to result in overstating the doses at the site boundary. These conservatisms would be equally applicable to casks in a tipped over condition. Some of these conservative assumptions are as follows:

- The single most conservative assumption in the analysis that Holtec performed for the PFSF is that all 4,000 casks have the exact same burnup and cooling time. This is impossible, since the MPCs will be delivered over many years and each additional year of cooling further reduces the radiation source term. As an example, if the PFSF received 4 casks per week, 50 weeks per year, it would take 20 years to completely fill the ISFSI. This means that at the completion of the ISFSI, the first casks delivered will have an additional 15 years of cooling time compared to the last casks delivered.

- A conservative burnup of 40,000 MWD/MTU and a cooling time of 10 years was used by Holtec in its analysis. In a separate analysis performed by Stone & Webster, a more realistic value of 35,000 MWD/MTU and a cooling time of 20 years were used, resulting in a reduction of more than 50% in the calculated normal doses at the site boundary, from 5.85 mrem/year to 2.10 mrem/year.

- The analyses use a single design basis fuel assembly, which has the highest gamma and neutron radiation source term in all fuel storage locations.

- The analyses use a single irradiation cycle to calculate the source term. This does not recognize the down time during reactor operations for scheduled maintenance and refueling. This additional down time would reduce the source term by effectively increasing the cooling time.

Q29. Dr. Resnikoff claims that for calculating normal doses at the site boundary, on which you base your comparison, PFS should have assumed that "a hypothetical individual is located at the site boundary the entire year or 8,760 hours/ year" instead of the 2,000-hour per year occupancy time used in the PFSF SAR (referenced above). Resnikoff Decl. ¶ 14. Do you agree, and even assuming Dr. Resnikoff were correct what effect would that have on your conclusions?

A29. (ELR) I disagree with Dr. Resnikoff, and even assuming he were correct his results would not affect my conclusions. The regulations provide that the applicable dose limits are to be calculated for a "real" individual, and not a hypothetical individual as claimed by Dr. Resnikoff. See 10 C.F.R. § 72.104(a). The regulatory guidance provided in the SRP and Interim Staff Guidances (ISG) for ISFSIs further provides for using a "real individual" for calculating radiation doses as opposed to Dr. Resnkoff's hypothetical individual. NUREG-1567 § 11.5.3.2 and ISG 13 revision 0. Here, PFS calculated the annual dose limit at the site boundary assuming that a worker is present at the site boundary 40 hours a week for 50 weeks a year to produce a conservative upper bound 2000 hour per year exposure at the site boundary. PFSF Safety Analysis Report §7.3.3.5.

Moreover, even assuming Dr. Resnikoff's argument that one should consider a hypothetical individual located at the site boundary for the entire year were correct, it would have no effect on my conclusion that the radiological dose at the site boundary would be far less than the accident dose limit of 5 rem in 10 C.F.R. § 72.106(b). It would merely reduce the margin of conservatism somewhat less than an order of magnitude, from the three orders of magnitude of conservatism discussed above to a margin of conservatism of still more than two orders of magnitude. Thus, the dose consequences at the site boundary would continue to be far below the 5 rem accident limit of 10 C.F.R. § 72.106(b).

Q30. What conclusion do you draw of the radiological doses at the site boundary in the event of one or more casks were to tip over at the PFSF due to a beyond design basis seismic event?

A30. (ELR) Based on the responses above for a single cask and 4000 casks, and the other conservative assumptions used in the analyses as documented in the PFSF SAR, it is my opinion that whether the HI STORM storage casks are assumed to remain upright in a severe earthquake or tip over, the radiation dose at the site boundary will remain essentially unchanged regardless of whether one assumes that a single cask, any number of them, or all the casks, tip over. In either case, the dose at the boundary is far below the accident limits of 10 C.F.R. § 72.106(b).

IV. RESPONSE TO OTHER CLAIMS RAISED BY STATE

A. Differences between the HI-STORM 100 Certificate of Compliance and the PFSF Design Basis Analysis for the HI-STORM 100 Storage Cask

Q31. In his December 7, 2001 declaration, Dr. Resnikoff points to differences between the NRC-approved Certificate of Compliance ("CoC") design basis analysis for the HI-STORM 100 Cask System and the design basis analysis of the HI-STORM 100 for PFSF as challenging Holtec's evaluation of cask tipover effects at the PFSF. The cited differences include variations in the number of hours used to calculate the year long exposure dose, the size of the design basis ground motion, and the number of casks involved in a tipover. E.g., Resnikoff Decl. ¶¶ 12-14. Do any of these differences affect the validity of the Holtec's analysis of cask tipover effects at the PFSF site?

A31. (KPS, AIS, ELR) No. Holtec has performed general design analyses in its FSAR for the HI-STORM 100 storage cask which support the CoC that the NRC has issued for the HI-STORM 100 storage cask system under 10 C.F.R. Part 72. Under the CoC, nuclear power plant licensees may use the HI-STORM 100 storage cask system at their sites under the general license provision of 10 C.F.R. § 72.210 as long as they meet the conditions of both 10 C.F.R. § 72.212 and the CoC. However, in addition, satisfactory performance of the HI-STORM 100 cask may be demonstrated by site-specific analyses. Holtec has performed such site specific analyses for the PFSF. Those analyses show satisfactory performance of the HI-STORM 100 Cask System at the PFSF. Thus, differences between the Holtec FSAR and CoC and the PFSF design do not invalidate our analyses of cask tipover effects as claimed by Dr. Resnikoff. Moreover, everything in the PFSF design is consonant with the Holtec CoC.

Q32. Dr. Resnikoff claims that PFS's use a 2,000-hour year occupancy time to calculate radiation dose levels at the site boundary is inconsistent with the use of 8,760 hours for the Holtec CoC. Are these inconsistent?

A32. (ELR) No. While it is true that the number of hours is different, it must be understood that a site-specific evaluation was performed for the PFSF whereas the Holtec FSAR is a generic evaluation for widespread application. The site specific analysis for the PFSF takes into account the particular characteristics found at the PFSF site, as discussed above. Therefore, the assertion by Dr. Resnikoff that the

PFS SAR is not consistent with the Holtec FSAR in its use of 2,000 hours/year occupancy time is irrelevant.

Q33. What effect, if any, would the assumption of 8,760 hours occupancy time at the site boundary, versus the 2,000 hours used in Holtec's analysis, have on the Holtec's evaluation of cask tipover effects?

A33. (ELR) It would have no effect. As discussed above, large margins exist between the dose rates at the PFSF site boundary and the 5 rem accident limit of 10 C.F.R. § 72.106(b) under either assumption.

Q34. Dr. Resnikoff also points to the fact that the PFSF design basis ground motion exceeds that of the Holtec CoC. What, if any, significance does the inclusion of larger design basis ground motion for the PFSF have for the analysis contained in the Holtec CoC?

A34. (AIS) None whatsoever. Holtec's cask stability analyses for the PFSF shows that the larger design basis ground motion at the PFSF site would have no adverse effects on the performance of HI-STORM 100 Cask System at the PFSF.

Q35. Dr. Resnikoff also claims significance in the fact that the Holtec CoC analyzes a single cask tipover, whereas the PFSF will have over 4,000 casks potentially subject to tipover. How, if at all, does this affect Holtec's analysis of cask tipover effects for the PFSF?

A35. (AIS, ELR) It has no effect. Cask tip over is a hypothetical event as confirmed by Holtec's cask stability analyses for the PFSF at both the 2,000 design basis earthquake as well as the 10,000-year beyond-design basis earthquake. Moreover, as shown above, even assuming the 4,000 casks were to tip over, the dose rates at the PFSF site boundary would be far below the 5 rem accident limit of 10 C.F.R. § 72.106(b).

B. Potential for Damage to the Storage Casks or the MPC Resulting from Cask Tipover and the Effect on Radiation Doses

Q36. Dr. Resnikoff asserts that PFS has neither quantified the damage to the casks that would result from tipover of the casks, nor "calculated the resulting radiation dose to workers or at the boundary" and that "PFS's claim of negligible increase in radiation from tipped over casks is not supportable. . . ." Resnikoff Decl. ¶¶ 19-24. Do you agree with Dr. Resnikoff?

A36. (AIS, ELR) No. We have evaluated the damage to the cask that might result from cask tipover and have concluded based on the design of the cask and the shielding characteristics of the concrete that any damage to the cask would be localized and would have negligible effect on the radiation shielding capability of the cask. Further, our comparison above of the radiological doses of casks in a tipped over configuration with casks in an upright configuration shows no significant difference in the radiation doses for the two configurations at the site boundary, therefore the dose rates from casks in a tipped over configuration would be far below the 5 rem accident limit of 10 C.F.R. § 72.106(b).

Q37. Please elaborate on your conclusion that cask tipover would have negligible effect on the radiation shielding provided by the storage cask.

A37. (AIS, ELR) As addressed in Section 11.2.3.3 of the HI-STORM FSAR, a hypothetical tip-over accident could cause localized damage to the radial concrete shield and outer steel shell where the storage cask impacts the surface. The localized damage from this hypothetical event would probably include some local crushing of the concrete contained within the steel enclosure near the point of impact with the target concrete pad. However, it is highly unlikely that any localized crushing and associated micro-cracking would create an uninterrupted radiation streaming path due to the homogeneity of concrete in the HI-STORM storage cask. In addition, since the concrete is fully encased in a steel structure, it is not possible for any concrete that may crush to become dislodged from the cask as it might in other cask systems where the concrete is exposed directly to the environment. Nor will there be any significant settling of damaged concrete since the enclosure shell is filled with concrete when it is poured and the damaged concrete would have nowhere to move. Therefore, any damaged concrete in the storage cask would remain inside the enclosure shell and continue to perform its shielding function.

Q38. Dr. Resnikoff also asserts that metal stretching or flattening and deformation of the cask would occur if the casks were subject to tipover which would adversely impact their shielding capability. Do you agree?

A38. (AIS, ELR) No. We do not agree. Since concrete is not fluid in nature and since there are four large steel ribs between the inner and outer shells of the storage cask it is highly unlikely that the storage cask would experience a general thinning of the concrete shielding as a result of concrete movement within the steel encasement. In addition, any damage due to a side impact (tip-over) will cause only localized damage to the concrete and outer shell of the storage cask in the immediate area of impact, as discussed in Section 11.2.3.2 of the HI-STORM FSAR. Therefore, the roundness of the storage cask could only be reduced in the immediate area of the impact (between the cask and the ground) and this would not significantly affect the shielding performance, since the same mass of steel and concrete would still be present. In the event of a non-mechanistic tipover, we would expect local flattening to occur, but no significant change in thickness. As an estimate for illustration, consider the HI-STORM tipped over and the impact to occur over a 12" diameter circle near the top of the cask, causing a stretching of the outer steel shell by 0.5" in that vicinity. The change in volume introduced by the stretch is approximately equal to the perimeter times the thickness times the stretch, or (3.14 x 12") x 0.75" x 0.5" = 14.137 cubic inches (note 3.14 is the value of "pi"). If we conservatively assume that because of deformations beyond the elastic limit, the material is uniformly incompressible over the entire local region, then the volume change is accommodated by thinning of the plate section in the area of the stretch. This change in thickness, "dt", can be computed by equating the volume change due to stretch to a volume change due to "thinning", or $(3.14 \times (12"+2*0.5")^2 / 4) \times dt = 14.137$ cubic inches. Solving for "dt" gives the thinning as dt = 0.107". A change in thickness by this amount over such a local area would have little consequences to the site boundary dose.

Q39. Dr. Resnikoff claims that Holtec's starting premise of zero initial angular velocity for the cask tipover is unfounded, and that "the angular velocity will be greater than zero" which will cause more flattening of the cask than contemplated. Resnikoff Decl. ¶¶ 19-20. Do you agree ?

A39. (KPS, AIS) No, we do not agree. The assumption of zero angular velocity is appropriate. As discussed in the companion Singh/Soler testimony, under the 10,000-year return period earthquake, the analysis has shown that the HI-STORM storage cask does not tip over, and that the behavior of the cask is characterized by tilting from the vertical resulting in a plane of precession for a certain duration in the course of the earthquake event. The cask experiences an oscillatory rocking motion with limited return to the vertical position until the rocking finally ends when the earthquake subsides. Observation of the simulated motion experienced by the PFSF casks during the 10,000-year event and other non-PFSF simulations of cask tipover leads us to conclude that, if the strength of the seismic event were increased to the point where the cask did tip over the initiating angular velocity propelling the cask towards the ground is quite small. Furthermore, the precessionary motion of the cask enables it to remain stable even while the center of gravity of the cask is well past the corner. As a result of the precessionary motion, the initial height of the cask center of gravity is apt to be much lower than the static tipover scenario (where tipover begins as soon as the center of gravity crosses the vertical plane containing the axis of overturning rotation). With less distance to fall, and a negligible initial angular velocity propelling the tip over, a cask tipping away from precessionary motion is expected to have substantially less kinetic energy of collision than one tipping from zero velocity with center of gravity of over corner.

Therefore, the starting premise used by Holtec in its cask tipover analysis of zero initial angular at the point at which the "center of gravity over corner" is exceeded is reasonable. The velocity might be somewhat increased from the tipover condition already studied, thereby increasing somewhat the deceleration of the cask upon hitting the pad or the point at which the cask initiates tipover might be below the center of gravity over corner velocity which would decrease the deceleration of the cask upon hitting the pad. In either event, the local deformation of the cask would generally be the same. Moreover, as stated above and discussed further below, Dr. Resnikoff's assumption that greater flattening of the cask would decrease its radiation shielding capability is erroneous.

Q40. What about Dr. Resnikoff's related claim made in paragraph 19 of his declaration that because "the angular velocity will be greater than zero" the top of the canister will be decelerating "at greater than 45g, in exceedance [sic] of the 45g design basis, thereby damaging the fuel assemblies . . . ." Do you agree with Dr. Resnikoff's statements?

A40. (KPS, AIS) No we do not. As discussed above, assuming zero initial angular velocity center of gravity over corner is a well-warranted assumption. Moreover, there is significant margin in the 45 g value stated in the HI-STORM FSAR in that the fuel assemblies can withstand g forces up to 63 g's under a side impact (Ref., Chun, Witte, Schwartz, "Dynamic Impact effects on Spent Fuel Assemblies, UCID-21246, Lawrence Livermore National Laboratory, 1987).

This is based on a stress analysis of the fuel assembly as a supported beam between grid straps and has been accepted by the NRC as a meaningful limit to assess the onset of fuel damage under impact decelerations laterally to the axis of the fuel. Thus, decelerations would be potentially damaging to the fuel assemblies only if the decelerations were increased by 33%.

Moreover, even if the fuel assemblies were damaged there would be no release of radioactivity because the damaged fuel would be confined by the MPC. As discussed in the companion Singh/Soler testimony, the MPC design incorporates large margins of safety, enabling the cask to perform its safety function of confining the radioactivity of the spent fuel at accelerations well beyond its design basis. This is exemplified by the hypothetical 25 foot end drop of a loaded canister on a hard concrete foundation discussed in that testimony. In that case the target surface, assumed to be essentially unyielding, was modeled as a 22 ft. thick concrete slab of compressive strength 6,000 psi. The computed strain in the confinement boundary material as a result of this hypothetical drop is only 41% of the failure strain limits for the canister material.

In the case of a side impact with a larger than anticipated deceleration at the top of the MPC, the MPC shell is buttressed by the thick MPC lid in precisely that area where the impact loads would be greatest. Therefore, in our opinion, the MPC strains would be bounded by the values computed in the 25' end drop.

Q41. Dr. Resnikoff also asserts that if deformation occurs to casks during tipover that PFS will have to calculate "the potential increase in dose at the site boundary or to workers from such casks" because the deformations would not necessarily face the ground while the cask is prone and "[w]hen the HI-STORM 100 casks are in fact up righted, the flattened area of the cask (localized deformation) will not face the ground." Do you agree with these conclusions?

A41. (ELR) No. Dr. Resnikoff makes several fundamental errors. First, NRC regulations regarding the radiological consequences of a design basis accident at an ISFSI are applicable to the public, not the workers on the site who are governed by other occupational standards (discussed further below). Second, Dr. Resnikoff misunderstands the nature of shielding provided by the HI-STORM 100 cask. The effectiveness of radioactive shielding is based on the mass of the shielding, not on the thickness. Because there is no-where for concrete that may be deformed to move, it will remain in place. Thus, a local deformation that may change the thickness of the concrete, by increasing the density, at a particular location will not change the mass and radiation shielding will be unaffected regardless of whether the deformation faces the ground. Even if there was a slight thinning of the steel as discussed above, the effect would not be noticeable at the site boundary.

Q42. Dr. Resnikoff also claims that PFS has not calculated the radiation dose at the boundary resulting from the bottoms of tipped over storage casks facing the fence line. Please describe the basis for not calculating such a scenario.

A42. (ELR) If the tipped over HI-STORM casks had been considered in the analysis the accident condition dose rates would not have been significantly affected as discussed above. In order for all casks to be resting on the ground, the casks in the 2x40 arrays would have to all fall away from each other into the 35 feet wide pathway between the arrays. If this were to occur, the tops and bottoms of casks would be facing other casks, which would minimize the dose contribution at the controlled area boundary from radiation emanating off the top and bottom of the casks, since this radiation would be directed toward other storage casks.

Further, the outer row of casks, which is the row Dr. Resnikoff is considering in his assertion, would have to fall inward towards the center of the ISFSI in order for the bottom of the casks to be facing the site boundary. The outer row of casks are positioned immediately adjacent to other casks, therefore, it is extremely improbable that a cask on the outer row would fall inward hitting an adjacent cask and still end up lying horizontally on the ground with the bottom facing the site boundary. In my opinion. it is far more likely that an outer cask would bump an inner cask in its movement and then fall away from the center of the ISFSI and end up resting on the ground with the top of the cask facing the site boundary. The top of the casks are heavily shielded and the resulting dose would be less than if the side of the casks were facing the site boundary. In addition, in the upright position for the ISFSI, the sides of the cask are partially shielded by the position of casks next to each other. This self-shielding would still exist to a degree when all casks are tipped over because they would be lying next to each other. Therefore, based on the response for a single cask, it is my opinion that the dose rate from the entire 4000 casks at PFSF lying on their sides would be similar to that from the ISFSI with all casks in the upright position.

Q43. Are you familiar with the calculations made by Dr. Resnikoff in his analysis of radiation dose at the PFSF site boundary resulting from the bottoms of tipped over casks?

A43. (ELR) Yes. I have reviewed Attachment B to Dr. Resnikoff's December 7, 2001 declaration, entitled "Rough Calculations: Dose Emanating from Bottom of Tipped-Over Cask." In his rough calculations, Dr. Resnikoff estimates the dose rate on the bottom of the HI-STORM overpack and the dose rates at the site boundary in a few steps. His basic approach is to first estimate the dose rate on the bottom of an unshielded MPC and then determine the dose rate on the bottom of the HI-STORM accounting for the shielding between the bottom of the MPC and the bottom of the overpack. Since the MPC sits on a 22 inch tall pedestal, which provides substantial shielding, Dr. Resnikoff assumes for the purposes of his calculation that the only pathway for radiation to reach the bottom of the overpack is through the annular gap between the MPC/pedestal and the inner shell of the HI-STORM overpack. Attachment A to this testimony provides a figure which illustrates this gap. In this annular region, the only shielding is the baseplate of the overpack.

The first step in the calculation was to estimate the dose rate on the bottom of the MPC based on the dose rates on the bottom of a loaded HI-TRAC transfer cask. Since the HI-STORM is always positioned vertically, the dose rates on the bottom of a HI-STORM overpack have never been calculated. Therefore, the only dose rates available to Dr. Resnikoff to use for this calculation were the dose rates on the bottom of the HI-TRAC.

Second, he estimates the percentage of the area on the bottom of the overpack which covers the annulus between the MPC and overpack (<u>see</u> attached figure in Attachment A to this testimony) using the following formula:

Area percentage = $\pi(r_o^2 - r_i^2)/ \pi r_o^2$

where r_0 is the outer radius of the annulus and r_i is the inner radius of the annulus. Using the percentage of area from the second step and the dose rate on the bottom of the MPC from the first step, he calculates the dose rate on the bottom of the overpack assuming that the baseplate of the overpack is the only shielding material.

Lastly, Dr. Resnikoff estimates the dose at the site boundary from the 80 casks in the outer row tipped over with the bottoms of these casks facing the site boundary. (An inherent assumption in his using only the casks in the outer row is that the tipped over casks inside the array are shielded by other casks and do not contribute any additional dose rate, which is consistent with points that I have previously made with regard to 4000 tipped over casks.) His estimates of the accident condition dose rates for the 80 casks in the outer row tipped over with their bottoms facing the site boundary range from 45.1 mrem/year to 451 mrem/year, depending on various assumptions. In either case these values are well below the 5 rem limit in 10 C.F.R. § 72.106(b).

Q44. Is Dr. Resnikoff's methodological approach correct?

A44. (ELR) As stated above, Dr. Resnikoff assumed in his analysis that the bottom of all 80 casks are facing the site boundary. As I have discussed earlier, it is far more likely that the tops of these 80 casks would be facing the site boundary since the casks are more likely to fall away from the ISFSI because the casks would bump into other casks if they fell inward. Therefore, I believe that the assumption that all 80 casks would be facing the site boundary is highly unrealistic. In addition, his estimation of the dose rate on the bottom of the overpack fails to account for the additional attenuation of radiation due to the MPC being positioned 22 inches above the baseplate of the overpack. As stated above, Dr. Resnikoff assumes that, as the worst case, the only shielding in the annular region between the MPC and inner shell of the overpack is the 2 inch thick baseplate of the overpack. In fact, there is considerably more shielding through the geometry where radiation must travel 22 inches from the MPC to the baseplate in an approximately 2.5 inch wide channel. This means that a significant amount of radiation will be scattered and absorbed in the walls of the pedestal and the overpack along this 22 inches. Dr. Resnikoff does not account for this in his worst case analysis, however he does approximate this affect by taking 10% of the calculated area of the annulus in his analysis to produce the lower bound dose rates.

In conclusion, the dose estimates calculated by Dr. Resnikoff are much higher than what would reasonably be expected, even under the unrealistic assumptions that Dr. Resnikoff made in his analysis. In my review of his calculations, I also found some errors in the calculations in the form of material thicknesses, distances, and an error in a formula. The errors in material thicknesses, if corrected would increase the calculated dose rate while the correction to the formula and the distance would decrease the calculated dose rate. The decrease would more than offset the increase.

Q45. Before describing the other errors in Dr. Resnikoff's analysis, please describe generally the design of the bottom of the HI-STORM 100 cask as it relates to radiation shielding.

A45. (ELR) The bottom of the HI-STORM overpack is a 2 inch thick circular steel plate. When the overpack is laying on its side the bottom steel baseplate of the overpack will be visible. Attachment A to this testimony shows a figure of the HI-STORM overpack with an MPC inside and a hatched outline of the bottom of the overpack when tipped over. The hatching in the figure indicates areas of concrete behind the baseplate. Behind the center section of the baseplate there are 17 inches of concrete and 5 inches of additional steel before the MPC is reached. In the outer regions of the baseplate, the concrete extends from the baseplate to the top of the overpack. Therefore, it is clear from the figure that there is only a very small annular region which does not have any concrete or additional steel positioned behind it. This is the annular region between the MPC and the inner shell of the HI-STORM overpack. This is also the area that Dr. Resnikoff calculated the dose rates for. Since there is significant shielding behind the two shaded areas of the baseplate in the form of concrete and steel, the highest region of dose on the baseplate of the overpack will be in the annular region between the MPC and the overpack inner shell.

Q46. Now please describe the errors in Dr. Resnikoff's actual calculation of the doses from the bottom of a tipped over cask assuming no shielding from other casks.

A46. (ELR) I found the following items that were inaccurate in his calculations

a. The thickness of lead that Dr. Resnikoff used for the HI-TRAC when calculating the dose rate on the bottom of the MCP was 1.0 inch. The correct value is 1.5 inches. Since there is more shielding than he assumed, his calculated dose rate on the bottom of the MPC would be higher if this thickness was corrected.

b. The thickness of the base plate on the HI-STORM overpack is 2 inches rather than 3 inches. Assuming a 3 inch thick baseplate in the calculations provides more shielding than is actually there. Reducing this value to 2 inches would result in higher estimated dose rates.

c. The equation $I_2=I_1\theta/h$ below Table 3 in Section D should be $I_2=I_1\theta/(4\pi h)$. This would reduce the dose rates estimated in the calculations. This is an easily made mistake when calculating the dose from a line source.

d. The distance from the casks to the site boundary should be, at a minimum, 600 meters rather than 555 meters. Correcting this would reduce the estimated dose rates.

Q47. What would the results of Dr. Resnikoff's calculations be if these inaccuracies were corrected?

A47. (ELR) If the four inaccuracies discussed above were corrected, Dr. Resnikoff's calculated dose rates would be reduced by approximately a factor of 2.9.

Q48. Based on your review of Dr. Resnikoff's calculation, what is your conclusion regarding his claim that dramatically higher radiation doses at the boundary of the PFSF fence line will occur in the event of a cask tipover event at the PFSF site?

A48. (ELR) I disagree. Both Dr. Resnikoff's methodology and analysis are flawed and therefore his conclusion is similarly flawed. Moreover, even accepting Dr. Resnikoff's inaccurate calculations, he states in his declaration that the dose rates due to gamma rays would increase 1.8 to 18 times those calculated by PFS assuming 2000 hours occupancy at the site boundary and 7.7 to 77 times that calculated by PFS assuming 8,760 hours occupancy per year. The highest number cited by Dr. Resnikoff would result in an annual dose at the controlled area boundary of approximately 450 mrem/year (5.85 x 77). This is well below the 5 rem accident dose set forth in 10 CFR §72.106. In fact, at 450 mrem/year, it would take 11 years before the 5 rem limit were reached.

C. NEUTRON DOSES CALCULATION.

Q49. The State asserts that cask heat-up and loss of concrete shielding has not been adequately addressed by PFS. In particular, the State contends that "after 33 hours of 100% air inlet blockage, the concrete temperature will exceed the short-term limit of 350° F specified in the CoC for the HI-STORM 100 cask, " which will cause water to evaporate from the concrete, "reducing the amount of hydrogen available for neutron capture;" and that "PFS has not analyzed the effects of an increase of neutron dose to on-site workers from the prolonged tip over of HI-STORM 100 casks." Resnikoff's Decl. ¶¶ 25-26. Do you agree with this claim?

A49. (KPS, AIS, ELR) No, Dr. Resnikoff makes several errors in his analysis. First, Dr. Resnikoff fails to consider the proper regulatory provisions and guidance for accident scenarios – the occupational dose applicable to workers are different from those that govern the maximum applicable dose to the public. Second, Dr. Resnikoff misinterprets and misuses the analysis of air inlet blockage in the CoC. Finally, Dr. Resnikoff's analysis assuming all the water from the concrete would evaporate is highly unrealistic. The effect of thermal degradation of a cask in a tipover condition on the water content of the concrete and its neutron shielding capability is insignificant.

Q50. Why is the occupational dose to on-site the workers not pertinent in determining whether the applicable dose limits to members of the public (10 C.F.R. § 72.104 for "normal operations and anticipated occurrences" and 10 C.F.R. § 72.106(b) for "accident" conditions) have been exceeded?

A50. (ELR) The reason why the occupational dose to on-site the workers is not pertinent in determining whether the applicable dose limits to members of the public have been exceeded is the occupational dose applicable to workers is governed by different regulatory provisions than those that govern the maximum applicable dose to the public. The regulations under 10 C.F.R. 72 only address the general public beyond the controlled area boundary, not workers on site. The PFSF site will have to meet the regulatory requirements of 10 C.F.R. 20 which governs the radiation workers.

Q51. How does Dr. Resnikoff misuse and misinterpret the Holtec CoC provisions regarding blockage of the air vents?

A51. (KPS, AIS, ELR) Dr. Resnikoff, in paragraph 25 of his declaration of December 7, 2001 makes an incorrect assumption that for the hypothetical cask tip over, "...the chimney effect is reduced dramatically and this is equivalent to the intake vents being blocked". Blockage of all the intake vents in a tipped over condition is, however, not possible. The HI-STORM overpack is a cylindrical vessel having four intake vents at the bottom (10" high x 15" wide) and four exit vents at the top (6" high x 25" wide). These top and bottom vents are spaced 90° apart around the circumference of the overpack. In a hypothetical tipover event, the overpack cylinder will come to rest on the ground with a line of contact with the cylindrical surface. For a worst case scenario, the projected outline of at most <u>one</u> intake vent and <u>one</u> exit vent can straddle this line of contact. If the vent openings were flat and the ground smooth then the straddled vents would be blocked. But because the openings are formed on a cylindrical surface, areas of the straddled vent openings away from the contact line are not blocked and the three other intake and three exit vents are open. For this reason, to assume that all-inlet-ducts will be blocked as a result cask tip over condition is physically impossible. Therefore Dr. Resnikoff misinterprets the 33 hour time limit provided in the CoC for standing the cask upright as this is assuming that all of the inlet ducts are blocked, which, <u>cannot</u> happen as a result of a tip over. Therefore, the 33 hour time limit provided for by the CoC is inappropriate for this condition.

Q52. Before turning to the next issue, would you please explain the importance of the water in the concrete in regards to the concrete's neutron shielding capability.

A52. (ELR) Yes. High energy fast neutrons must be slowed down (have their energy reduced) and captured in the shielding material in order to reduce the dose rate on the exterior of the cask. Neutrons lose the most energy in collisions with Hydrogen atoms. While collisions with other atoms will reduce the energy, Hydrogen is the best attenuator for neutrons. In concrete, a significant portion of the Hydrogen is in the form of bound water. There may also be Hydrogen contained in the aggregate depending upon the type of aggregate used.

Q53. What effect would an increase of the concrete temperature of a tipped over cask have on the water content of the concrete and its neutron shielding capability?

A53. (KPS, ELR) The effect would be minimal. There is a limited range of temperatures to which the concrete could be subjected in the event of a cask tipover, even assuming that the cask remained in a tipped over condition for a long period of time. This range of temperature would not cause significant evaporation of water, and in turn the impact on the neutron shielding capability of the concrete would be insignificant. In addition, any Hydrogen contained in the aggregate in the concrete would not be affected by the increase in temperatures.

(KPS) It is not easy to evaporate water within concrete, because it is in a confined space, and as the water evaporates, the air pressure increases. In turn, the increased air pressure will convert the water vapor back to liquid water. Likewise, concrete does not lose its moisture content as easily as water might evaporate from a free surface. In order for large, extensive, sustained water evaporation from the concrete to occur, exposure to high temperatures on the order of 600 degrees Fahrenheit or greater for a period of months ("Properties of Concrete", A.M. Neville, 4th Edition, (Pages 385 – 387)) will be necessary. The cask in a tipover condition will not attain this range of temperatures, even if such a condition is assumed to persist for a long time with a bounding assumption that one air vent at both the top and the bottom of the cask were blocked (See response to Q51). Although this particular geometry has not been analyzed, based on our experience modeling comparable scenarios we expect the concrete temperature to remain below 350°F which is far below the 600° F required for extensive water evaporation from the concrete. Even assuming all vents were blocked as claimed by Dr. Resnikoff, the bounding steady state temperature for the concrete would be, well below the 600°F necessary for extensive sustained water evaporation. Therefore, the evaporation of water from the concrete of a tipped over cask would be minimal even if the cask remained in a tipover position for a period of months.

Further, there will be a temperature profile in the concrete body of the tipped over HI-STORM overpack. The hottest concrete will be the inner concrete surface contacting the overpack inner shell which is heated by the MPC. The temperature will decrease radialy outward to approach the overpack enclosure shell surface temperature. The temperature will also be much less in the concrete away from the ends of the MPC. Therefore, there are the heated regions in the overpack where the amount of water loss may be larger and regions in the overpack away from the inner heated regions where the temperatures are such as to preclude any water loss.

(KPS, ELR) Thus, a cask tipover event would not cause a significant increase in neutron radiation because the cask simply will lose very little shielding due to the loss of hydrogen atoms in the water within the concrete even under a worst case scenario.

Q54. Of what consequence therefore is Dr Resnikoff's assertion that if all the water evaporates from a HI-STORM cask, that neutron dose rate will increase 57.3 time for a dose result of 108 mrem per hour?

A54. (KPS, ELR) It is of no consequence. Dr. Resnikoff's analysis, in "Calculation of Neutron Dose at Elevated Concrete Temperatures" on which he bases his claim assumes that all Hydrogen in the concrete was in the form of water and available to be evaporated and in fact would be evaporated. Resnikoff Decl. ¶ 26. As discussed above, evaporation of all of the water is very difficult to achieve. Likewise, neutron shielding capability of the concrete also depends on the aggregate that is used in making the concrete. If that aggregate contains Hydrogen, then a very substantial amount of this Hydrogen would still remain even after assuming that somehow all the water could be removed from the concrete.

Q55. Why is it important if a worker receives the occupational dose limit of 5 rem?

A55. (ELR) 10 C.F.R. 20 § 20.1201 sets the occupational limit for radiation workers at 5 rem per year. Therefore, if a worker receives 5 rem, they are prohibited from working in a radiation environment for the remainder of the year. This may have an impact on the operating entity in that they may have to hire additional workers to perform specific tasks. Therefore, a worker receiving 5 rem is not a problem for the worker but may end up being a logistics problem for the operating entity.

In the case of PFSF, Dr. Resnikoff states that worker may receive the 5 rem limit in just over 46 hours based on his calculation of a contact dose rate of 108 mrem/hr. The implication is that this is a problem for PFSF and is something that should have been considered. In reality, even if his calculations were correct, it is unreasonable to assume that a worker would be in contact with an overpack for an extended period of time. Radiation workers at nuclear utilities have to deal with areas of high radiation (much higher than 108 mrem/hr) on a daily basis and they do so without exceeding the 5 rem per year limit through careful planning and the use of the temporary shielding. The same would be true at PFSF if the cask hypothetically tipped over and all of the water evaporated from the concrete. Therefore, the fact that a worker may reach a limit of 5 rem is of no practical concern for PFSF.

Q56. What is your conclusion regarding Dr. Resnikoff's analysis?

A56. (KPS, ELR) While Dr. Resnikoff tries to make much out of his calculation that a worker would exceed the 5 rem per year dose limit after 46 hours at the postulated neutron radiation dose rate, he ignores common radiation shielding practices that would be used to maintain the dose to an individual as low as possible. In addition, his line of argument has no impact on the conclusions for the general public. Therefore, the discussion revolving around his questionable calculations does not have any bearing on the licensing of the PFSF.

D. OTHER CLAIMS RAISED BY THE STATE OF UTAH

1. <u>Sliding Impacts</u>

Q57. Dr. Resnikoff claims that the HI-STORM cask could slide up to 370 inches in the x direction and 230 inches in the y direction during a 2,000-year earthquake. Do you agree?

A57. (AIS) No. Dr. Resnikoff bases his claim on the results from a calculation by another State expert. In our opinion, as shown in companion testimony, the expert testimony relied on by Dr. Resnikoff is completely erroneous with respect to sliding of the cask. Our calculations show that the casks will not undergo sliding impact during a 2,000-year earthquake. In a hypothetical sliding scenario for a 10,000-year earthquake, confirmatory analyses (by Sandia Laboratory) have indicated that a cask may slide up to 15 inches. Since the casks are nearly 48" apart, this will not result in any collision of casks. Moreover, even if sliding impact of the casks were to be postulated to occur without regard to results from analyses, the velocities of the impact will be much smaller than the velocity of impact determined in the hypothetical cask tipover event. Thus, even if they were to slide and impact one another, any damage would be less than that predicted due to the hypothetical tipover case. Certainly, no diminution of radiation shielding would occur.

2. Potential Effects to Storage Casks Due to Uplifting and Dropping

Q58. The State asserts that the HI-STORM cask can be uplifted by up to 27 inches in a 2,000-year earthquake. Do you agree with this assertion?

A58. (AIS) No. Dr. Resnikoff's claims are contrary to numerous cask stability analyses that we have done for the PFSF at varying design basis earthquakes, and at the 10,000-year beyond design basis earthquake. As noted in the previous answer, the results that Dr. Resnikoff relies on are fundamentally incorrect. On the other hand, our analysis have been confirmed by the analysis done by Sandia Laboratories for the NRC Staff. Based on our analysis, during the design basis earthquake, there could be a maximum uplift of approximately **2.31"** at one corner of the storage cask. No liftoff of the entire cask is indicated.

Q59. Even assuming that an earthquake could cause the cask to be lifted up 27 inches, what effect would the subsequent drop have on the storage cask and MPC capability to perform their safety related functions?

A59. (AIS) None. Even if a storage cask were lifted twenty-seven inches and dropped, there would be no impact to the shielding effectiveness of the storage cask or the confinement function of the MPC. Such a drop would have no impact on the confinement capability of the MPC. As the hypothetical MPC drop analysis shows, the unprotected MPC can be subjected to a twenty-five (25) foot drop without adverse radiological consequences. A mere twenty-seven inch drop, while the MPC is protected by the storage cask, would not result in any significant harm to the storage system and certainly would not have any radiological consequences due to deformation or damage to the storage cask, as discussed above.

Q60. Of what consequence would the fact that the 27 inch you just unrealistically assumed in the above question was greater than the 12 inches referred to in the CoC?

A60. (KPS, AIS) The twelve inch drop limit listed in the HI-STORM is intended to maintain the decelerations within a prescribed regulatory limit which is well below the "failure limit" for the MPC. The failure limit, as observed earlier, could not be reached even when the MPC is assumed to free fall from a height of 300 inches (25 feet). Because the 27 inch drop is claimed for a beyond-the-design-basis event by the State, the 12 inch CoC limit, which is a regulatory limit applicable to normal handling of casks, is entirely inapplicable.

3. Potential Effects on the MPC of an On-Edge Impact

Q61. Dr. Resnikoff claims that Holtec Report HI-2002572, *Evaluation of the Confinement Integrity of a Loaded Holtec MPC Under a Postulated Drop Event* is inadequate, because it assumes that the HI-TRAC cask will drop vertically. He further asserts that it "is more likely that the HI-TRAC cask would drop on edge" as opposed to flat on the surface and that "the shear stresses would then be considerably more severe than in a vertical drop." Do you agree with his claims?

A61. (KPS/AIS) No we do not. The HI-TRAC transfer cask is a geometrically symmetrical structure with a radially symmetric MPC inside it. Moreover, the cask is held by the crane hook along its axis of symmetry. Failure of the hook (itself a counter factual assumption given the margin of safety inherent in its design) however, would lead to a symmetrical fall of the cask. In view of the symmetry in mass and geometry, an inclined drop can not be reasonably postulated. Therefore, should a drop occur because of an earthquake, there would not be enough time for the cask to rotate from the vertical. And in any event, at the PFSF as described in the Testimony of Wayne Lewis, HI-TRAC transfer cask would be supported only by the crane for only a very brief moment in time.

V. CONCLUSION

Q62. Considering all the potential effects and scenarios raised by Dr. Resnikoff, what effect, if any, could a beyond design basis seismic event have on the radiation dose calculations?

A62. (KPS, AIS, ELR) Based on the responses above for a single cask and 4000 casks, and the other conservative assumptions used in the design and applicable analyses, whether the HI-STORM storage casks are assumed to remain upright in a severe earthquake or tip over, or slide into and impact each other, the radiation doses at the site boundary will remain essentially unchanged. Regardless of whether one assumes that a single cask, any number of them, or all the casks tip over or impact each other, the dose to the general public will be several orders of magnitude below the 5 rem accident limit of 10 C.F.R. § 72.106(b).

EVERETT L. REDMOND II, Ph.D.

NUCLEAR ENGINEERING
HOLTEC INTERNATIONAL

EDUCATION

Massachusetts Institute of Technology
Ph.D. in Nuclear Engineering and a Minor in Biology (1997)
GPA: 4.3 out of 5.0

Massachusetts Institute of Technology
M.S. in Nuclear Engineering (1990)
GPA: 4.3 out of 5.0

Massachusetts Institute of Technology
B.S. in Nuclear Engineering (1990)
GPA: 4.3 out of 5.0

PROFESSIONAL EXPERIENCE

HOLTEC INTERNATIONAL
Marlton, New Jersey

1999–Present	Nuclear Engineer and Manager of Nuclear Physics Group
1995-1999	Nuclear Engineer
August 1994-May 1995	Criticality and Shielding Consultant

LOS ALAMOS NATIONAL LABORATORY
Los Alamos, New Mexico

Summers 1993 and 1994	Graduate Research Assistant

RAYTHEON
Sudbury, Massachusetts

Spring 1993	Shielding Consultant

NORTHEAST UTILITIES COMPANY
Hartford, Connecticut

Summer 1992	Engineer

IDAHO NATIONAL ENGINEERING LABORATORY
Idaho Falls, Idaho

Summers 1987, 1988, 1990	Engineer and Co-op Student
June 1989 - January 1990	

PROFESSIONAL SOCIETY MEMBERSHIPS/ACTIVITIES

Member American Nuclear Society (1986-Present)

SPENT FUEL STORAGE TECHNOLOGY

- Developed Holtec's shielding analysis methods for dry cask storage licensing.
- Developed Holtec's shielding analysis methods and models for performing site boundary dose calculations for an ISFSI.
- Performed site boundary dose evaluations in support of 10CFR 72.212 evaluations.
- Developed preferential fuel loading plans for Holtec's dry cask systems to reduce personnel exposure and off-site dose.
- Interacted with NRC on numerous occasions in vigorous technical discussions about shielding issues as they pertain to Holtec's dry cask storage systems.
- Created all computer models of HI-STAR 100, HI-STORM 100, 100-ton and 125-ton HI-TRACs used in the shielding analysis reported in the HI-STAR SAR and HI-STAR and HI-STORM TSARs under Dockets 71-9261, 72-1008, and 72-1014
- Author of Shielding Evaluation Chapters in the HI-STAR SAR and HI-STAR and HI-STORM TSARs under Dockets 71-9261, 72-1008, and 72-1014
- Primary reviewer for Criticality Evaluation Chapters in the HI-STAR SAR and HI-STAR and HI-STORM TSARs under Dockets 71-9261, 72-1008, and 72-1014
- Performed criticality analysis for both PWR and BWR spent fuel pool reracking.
- Served as primary reviewer for numerous criticality analyses for spent fuel pool reracking.

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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-ISFSI
	)	
(Private Fuel Storage Facility)	)	

APPLICANT'S PROPOSED FINDINGS OF FACT AND CONCLUSIONS OF LAW ON UNIFIED CONSOLIDATED CONTENTION UTAH L/QQ (SEISMIC)

Jay E. Silberg
Paul A. Gaukler
Matias F. Travieso-Diaz
Blake J. Nelson
SHAW PITTMAN LLP
2300 N Street, N.W.
Washington, DC 20037
(202) 663-8000

Dated: September 5, 2002

Counsel for Private Fuel Storage L.L.C

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whether or not the factor of safety recommendations in the Standard Review Plan for nuclear power plants are satisfied. Tr. 6594-96, 6739-41 (Ofoegbu). Therefore, the claims raised by the State in Section D of Contention L/QQ with respect to the dynamic stability of the CTB have no licensing significance.

C. Section E of Contention Utah L/QQ

1. Introduction and Background

393. Section E of Contention Utah L/QQ challenges the Staff's granting of an exemption from NRC regulatory requirements so as to allow PFS to design the PFSF based on a probabilistic seismic hazard analysis and a 2,000-year return period earthquake. The contention reads (PFS Exh. 237):

__Section E Seismic Exemption__

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 NRC 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 NRC 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 NRC 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.*

394. Applicable regulations in 10 C.F.R. § 72.102(f) and 10 C.F.R. § 72.102(b), provide for the assessment of design basis seismic ground motions for ISFSIs at sites west of the Rocky Mountains based on the deterministic procedures and criteria formerly used for nuclear power plant seismic design (Appendix A, 10 C.F.R. Part 100). In 1996 the Commission changed the seismic design requirements for new nuclear power plants by issuing regulations and guidance documents that

provide for use of Probabilistic Seismic Hazard Analysis ("PSHA") methodology. 10 C.F.R. §100.23; Regulatory Guide 1.165. The Commission is considering a similar rule change to employ the use of PSHA methodology for the seismic design of ISFSIs. See 67 Fed. Reg. 47745 (July 22, 2002).

395. SECY-98-126 (June 4, 1998), referenced in the State's contention, was the initial rulemaking plan for implementing the change from deterministic methods to PSHA methods for the seismic design of ISFSIs. That SECY document discussed three different rulemaking options for the Commission for incorporating PSHA methods into 10 C.F.R. Part 72. The "preferred" approach set forth in SECY-98-126 proposed a 1,000-year mean return period design basis earthquake for "Category 1" structures, system and components important to safety ("SSCs") (those whose failure would not result in radiological doses exceeding the requirements of 10 C.F.R. § 72.104(a)) and a 10,000-year mean return period design basis earthquake for Category 2 SCCs (those whose failure would result in radiological doses exceeding the requirements of 10 C.F.R. § 72.104(a)).

396. This initial rulemaking plan, however, was essentially superseded by SECY-01-0178, dated September 26, 2001 in which the NRC Staff recommended to the Commission that the rulemaking plan be modified to add another option, which it identified as the "preferred" one, in lieu of the two-tiered approach identified as the preferred option in in SECY-98-126. This new "preferred" option features the use of a 2,000-year mean return period earthquake as the design basis for all ISFSI SSCs. In a Staff Requirements Memorandum dated November 19, 2001, the Commission approved the modification to the rulemaking plan proposed by SECY-01-0178, further instructing the NRC Staff that the proposed rule should

solicit comments on a range of "exceedance levels" from 5.0×10^{-4} through 1.0×10^{-4} to which the failure probability of SSCs should be set.

397. On July 22, 2002, the NRC issued a proposed rule to make the Part 72 regulations compatible with the 1996 revision to Part 100 that addressed uncertainties in seismic hazard analysis. "Geological and Seismological Characteristics for Siting and Design of Dry Cask Independent Spent Fuel Storage Installations and Monitored Retrievable Storage Installations," 67 Fed. Reg. 47745 (July 22, 2002). The proposed rule would require a new specific license applicant for a dry cask storage facility located in either the western U.S. or in areas of known seismic activity in the eastern U.S., and not co-located with a NPP, to address uncertainties in seismic hazard analysis by using appropriate analyses, such as a PSHA or suitable sensitivity analyses, for determining the DBE. The new proposed regulation, 10 C.F.R. § 72.103, would eliminate the current requirement to comply with deterministic methodology of Appendix A to Part 100. As part of the proposed rule, the Commission indicated it is considering using a mean annual probability of exceedance value in the range of 5.0E-04 to 1.0E-4 for ISFSI applications. Draft Regulatory Guide DG-3021, "Site Evaluations and Determination of Design Earthquake Ground Motion for Seismic Design of Independent Spent Fuel Storage Installations and Monitored Retrievable Storage Installations," has been developed to provide guidelines that are acceptable to the NRC staff for determining the DE for an ISFSI. The Draft Regulatory Guide currently recommends a mean annual probability of exceedance value of 5.0E-04 as an appropriate risk-informed value for the design of a dry cask storage ISFSI.

398. On April 2, 1999 PFS filed an exemption request to use PSHA methods for determining the seismic design of the PFSF using a 1,000-year mean return period

earthquake as the PSHA design basis. PFS Exh 247. On August 24, 1999, PFS amended its request for an exemption to seek the use of a 2,000-year mean return period earthquake as the design basis for the PFSF. PFS Exh. 248. In its Safety Evaluation Report of October 2000 the NRC Staff approved PFS's request to use PSHA methodology for the seismic design of the PFSF based on a 2,000-year mean return period design basis earthquake. The final statement of the Staff's reasons for granting the exemption is set forth in the Consolidated SER issued in March 2002. See Staff Exhibit C at 2-50 to 2-51.

399. The State filed its contention challenging the exemption request November 9, 2000. On January 31, 2001 the Board determined that contention would largely be admissible under the Commission's standards for the admission of contentions, but referred the rulings regarding admissibility of the contention to the Commission and certified to the Commission as well the question whether the State challenges should be cognizable in this adjudicatory licensing proceeding.

400. In its decision of June 14, 2001, the Commission affirmed the Board's findings concerning the admissibility of the proffered contentions and held that the State's challenge to the exemption should be heard as part of this licensing proceeding. With respect the State's challenge to the Staff's rationale for granting the exemption, the Commission reasoned, as had the Board, that "although the contentions attacking the Staff's reasons for granting the exemption were not artfully pleaded, the substance of Utah's complaints was that the 2000-year return period has not been shown to be adequately protective." Therefore, "the contentions should not be dismissed simply because they referred to the Staff's reasoning." The Commission went on to say that, although PFS has the "burden to show that the exemption is 'authorized by law, will not endanger life or property or the common

defense or security and [is] otherwise in the public interest,'". PFS had here "essentially adopted the Staff's reasoning when it agreed to use the 2000-year return period the Staff recommended." Therefore, the Commission concluded that it was "appropriate under these circumstances to consider the Staff's bases for granting the exemption." CLI-01-12, 53 NRC at 473.

401. In its testimony and evidence before this Board, the Applicant has fully set forth the reasons why use of the 2,000-year mean return period design basis earthquake will not endanger life or property or the common defense or security, and is otherwise in the public interest. As set forth below, the Applicant's justifications provide full legal and technical bases for granting the exemption, wholly independently of the Staff's rationale, which also provides sufficient technical and legal basis for the granting of the exemption.

402. Our findings with respect to the remainder of Section E of the Unified Contention are organized as follows. First, we will discuss the appropriateness of using PSHA methods for the seismic design basis for the PFSF. Second, we will discuss whether using a 2,000 year design period earthquake in accordance with the applicable design requirements will adequately protect public health and safety. Third, we will address the State's claims concerning radiation dose consequences and the conclusions that can be drawn from them.

2. Appropriateness of Using Probabilistic Seismic Hazard Analysis Methodology for the PFSF Seismic Design

403. The parties are in agreement that use of PSHA methods is appropriate for the seismic design of the PFSF, and should be used instead of the deterministic methods currently provided for by Part 72 of the regulations. Cornell Dir. at A11-A18; PFS Exh EEE at 44-45; Tr. 9116-19 (Arabasz).

404. Deterministic methodology as applied to nuclear power plants under Appendix A of Part 100 typically leads to a small set of representative earthquakes (magnitudes and locations) that could affect a site and a corresponding set of ground motion response spectra. From these, the dominant event pair (magnitude and location) is identified, together with its representative response spectra at the site, which becomes the design basis ground motion. Cornell Dir. at A13.

405. PSHA methods differ from deterministic methods in that a PSHA takes into account the entire range of potential seismic events (magnitudes and locations) that could affect a site and resulting site ground motions, and their corresponding frequencies of occurrence and associated uncertainties. The result is a curve of estimated annual probability of exceedance versus level of ground motion. This curve can be used to identify the design ground motion corresponding to a specified mean annual probability of exceedance. Cornell Dir. at A14. In this manner, probability and risk factors are incorporated into the selection of a design basis earthquake.

406. PSHA methodology is commonly used for determining the design basis ground motions for the seismic design of building and structures, and today is the prevalent methodology in the seismic design of structures and facilities. Current regulations and guidelines based on probabilistic seismic hazard principles include those governing the design of buildings under both the Uniform Building Code ("UBC") and the International Building Code, offshore structures under API RP2A, and Department of Energy ("DOE") facilities under DOE-STD-1020. Cornell Dir. at A15.

407. The PSHA methodology has become widely accepted and used because of the advantages of using a probabilistic approach to establish design ground motions.

These advantages are: (1) the probabilistic approach captures more fully the current scientific understanding of earthquake forecasting than the deterministic method; (2) the probabilistic approach is capable of reflecting the uncertainties in professional knowledge of key elements of the seismic hazard; and (3) the probabilistic approach can be used to set design criteria that are consistent among different regions and among different failure consequences, thus allowing a rational and a equitable allocation of safety resources. Cornell Dir. at A16.

408. The Commission has recognized the advantages of the probabilistic approach and has replaced Appendix A, 10 C.F.R. Part 100 with regulations and guidance documents that provide for use of PSHA methodology for the seismic design of new nuclear power plants. 10 C.F.R. §100.23; Regulatory Guide 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion," March 1997 (Staff Exh. UU). The Commission has also used probabilistic seismic procedures in areas such as re-evaluation of existing nuclear power plants and seismic standards for high-level waste geological repository design. Cornell Dir. at A17. This move towards probabilistic methodologies is consistent with the Commission's general policy of risk-informed regulations and decision making. See, e.g., Regulatory Guide 1.174, "An Approach for Using Probablistic Risk Assessment in Risk-Informed Decisions on Plant-Specific Changes to the Licensing Basis," July 1998; Commission Direction Setting Issue 12, "Risk-Informed, Performance-Based Regulation". In accordance with this use of probabilistic procedures, the Commission has recently undertaken 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. See

"Proposed Rule: Geological and Seismological Characteristics for Siting and Design of Dry Cask ISFSIs and MRSs," 67 Fed. Reg. 47745 (July 22, 2002).

409. Thus, PFS's proposed use of PSHA methods to characterize the seismic hazard at the site and to set the seismic design basis of the PFSF is fully consistent with NRC policy and practices, as well as with the current state of the art in engineering practice. We accordingly conclude that the use of PSHA methods for determining the design basis ground motion for the PFSF, as requested in PFS's exemption request, is warranted.

3. Appropriateness of Using a 2,000-Year Return Period Earthquake for the Seismic Design of the PFSF

410. We next turn to consider the appropriateness of using a 2,000-year return period earthquake for the seismic design of the PFSF, on which there is dispute among the parties. There are two main areas of dispute, one between PFS and the State and a second between the NRC Staff and the State. Those are discussed separately below in subsections b and c. Subsection a discusses general principles of risk informed seismic design. Subsection d discusses the specific issues raised by the State in the various subparts of Section E of the contention (other than subpart 2 concerning radiation does consequences) drawing primarily on our earlier discussion.

a. General Risk-Based Principles for Judging the Adequacy of a 2,000-Year Return Period Earthquake for the PFSF

411. The Applicant's witness, Dr. Cornell, articulated PFS's position on the appropriateness of using a 2,000-year return period earthquake for the seismic design of the PFSF based on accepted principles of risk-informed seismic design. Dr. Cornell has extensive experience in seismic risk analysis and the development of ap-

propriate seismic codes and standards. He has been involved in seismic PRAs and seismic margin studies for dozens of nuclear projects and is among the foremost experts in seismic risk assessment for nuclear facilities. Given Dr. Cornell's recognized expertise and the other parties' general agreement with the risk principles enunciated by Dr. Cornell in his testimony, we will first set forth those general risk-based principles, which we adopt.

412. The first general principle of risk-informed seismic design is that there should be a risk-graded approach to seismic safety that allows facilities and structures with lesser consequences of failure to have larger mean annual probabilities of failure than those allowed for facilities for which the consequences of failure would be more severe. In other words, under a risk-graded approach to seismic safety, the less severe the anticipated consequences of failure, the larger the probability of failure that can be tolerated. Examples of seismic standards that explicitly incorporate a risk-graded approach are the draft International Standards Organization guidelines for offshore structures, Federal Emergency Management Agency ("FEMA") guidelines for building assessment, and DOE Standard 1020. Cornell Dir. at A20-A22; Tr. 8014-18 (Cornell).

413. Such a risk-graded approach was implemented in the Staff's approval of the PFS exemption request. The Staff concluded that, because an ISFSI like the PFSF poses less radiological risk than a nuclear power plant, an ISFSI can be subjected to less stringent licensing requirements for seismic safety than those for an operating nuclear power plant. [Staff Exh. C at 2-50, 2-51] This conclusion is in accordance with the Commission's acknowledgement that the potential consequences of failure of ISFSIs are much less severe than those for nuclear power plants, and

therefore, the licensing standards for ISFSIs need not be as strict as those for operating nuclear power plants. See Cornell Dir. at A23.

414. The State's expert witness, Dr. Arabasz, agreed that it is appropriate to use a risk-graded approach for the seismic analysis and design of facilities and structures. PFS Exh. EEE at 59-60; Arabasz Dir. at A11; Tr. 9122 (Arabasz). Likewise, they agreed with Dr. Cornell and the Staff that it is appropriate to allow a higher probability of seismic failure for ISFSIs, such as the PFSF, than for nuclear power plants, since ISFSIs inherently pose less risk than an operating nuclear power plant. Tr. 9122-24 (Arabasz); Tr. 12831-32 (Bartlett) Thus, the parties are in full agreement that it is appropriate to use a risk-graded approach to seismic safety for licensing the PFSF and that under such a risk-graded approach the PFSF can be subject to less strict seismic safety requirements than those for an operating nuclear power plant.

415. The second general principle of risk-informed seismic design articulated by Dr. Cornell is that the adequacy of a design basis earthquake ("DBE") to provide the desired level of seismic safety is judged based on two considerations or factors, often referred to as the "two-handed approach." The first factor is the mean annual probability of exceedance ("MAPE") of the DBE. The second factor is the level of conservatism incorporated into the criteria and procedures for the design of the facility. Cornell Dir. at A20. Following DOE 1020 parlance, this second factor was referred to by PFS and the State as the risk reduction factor, R_R. See, e.g., id. at A27; State of Utah Testimony of Dr. Steven Bartlett on Unified Contention Utah L/QQ, Part E (Lack of Design Conservatism)(Introduced at Tr. 11822) (revised June 5, 2002) ("Bartlett Section E Dir.") at A11; Tr. 9131-36 (Arabasz); Tr. 12804-05 (Bartlett).

416. Underlying this second general principle is the fact that the design procedures and the acceptance criteria (e.g., applicable codes and standards) for seismic design usually include conservatisms that reduce the risk of failure. These conservatisms are not explicitly identified, but are embedded in the design procedures and in the provisions of the various codes and standards pursuant to which seismic design is accomplished. Because of the conservatisms incorporated in seismic design procedures and acceptance criteria, the probability of failure of a seismically-designed facility is virtually always less than the MAPE of the governing DBE. In other words, virtually all facilities 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 systems, structures and components are able to withstand a more severe, i.e., more infrequent, earthquake than that used as the DBE. Cornell Dir. at A25-A26.

417. This second principle is of great import here, for its means that the actual probability of failure of a seismically-designed facility, such as the PFSF, is a function of both the MAPE of the DBE and the level of conservatism incorporated in the design procedures and the acceptance criteria for seismic design of the facility. This function can be expressed by the simple algorithm MAPE/ R_R. Cornell Dir. at A20, A25-A26.

418. The MAPE is the inverse of the DBE. Cornell Dir. at A19; Tr. at 9145-46 (Arabasz). For example, the MAPE of the PFSF 2,000 year DBE is 5×10^{-4}. Id. Therefore, assuming that the seismic design procedures and acceptance criteria for the PFSF achieved a R_R on the order of 5, the annual probability of seismic

failure for the PFSF would be 1 x 10^{-4}, or 1 in 10,000. See, e.g., Cornell Dir. at A44 & A48; Tr. 9134, 9180-81, 10154 (Arabasz); Tr. 7925-26 (Cornell).-

419. Therefore, the actual level of seismic safety achieved by the seismic design of a facility, such as the PFSF, cannot be determined by simply looking at its DBE. Equally important, the comparative level of seismic safety of two facilities cannot be evaluated solely on the basis of their relative DBEs, unless they are also designed to the same procedures and criteria. Rather, both factors – the MAPE of the DBE as well as the level of conservatism in the design procedures and acceptance criteria – must be considered when comparing the seismic safety of two facilities or structures. Cornell Dir. at A25-A26.

420. For example, the annual probability of seismic failure for a facility or structure with a 2,500-year return period earthquake as its DBE (with a corresponding MAPE of 4 x 10^{-4}) but designed to seismic codes and standards providing a R_R of only 2 would be 2 x 10^{-4}, or 1 in 5,000. Therefore, even though the DBE of such a facility would be an earthquake of higher intensity than that for the PFSF, its annual probability of failure would be twice that for the PFSF (assuming a R_R of 5 for the PFSF seismic design) because the underlying seismic codes and standards for such a facility would embody significantly less conservatisms than those for the PFSF. See, e.g., Cornell Dir. at A91-93; Tr. 12961-63 (Cornell).

421. The State and PFS agree that DOE-STD-1020-94, "Natural Phenomena Hazards Design and Evaluation Criteria for Dept. of Energy Facilities," Jan. 2002 (PFS Exh. DDD), is a good example of the application of a risk-graded approach toward seismic design. This standard establishes a set of "performance categories" for seismically designed SSCs with increasing consequences of failure, and thus decreasing probabilities of failure, as their performance goals. DOE-1020-94 es-

tablished performance goals (reflecting increasingly severe consequences of failure) of 10^{-3} for PC-1 structures (designed to protect occupant safety) $5x10^{-4}$ for PC-2 category structures (essential facilities and buildings, such as hospitals, that should continue functioning after an earthquake with minimal interruption), and 1×10^{-4} and 1×10^{-5} for PC-3 and PC-4 category structures (which correspond to ISFSIs and NPPs respectively). The MAPE for the design basis ground motions under DOE-1020-94 were set as $2x10^{-3}$, 10^{-3}, $5x10^{-4}$, and 10^{-4} for PC-1, PC-2, PC-3 and PC-4 structures respectively.

422. To bridge the gap between the performance goals and the DBE MAPEs, DOE 1020 standards call for design procedures and acceptance 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" PFS Exh DDD (DOE-STD-1020-94, p. 2-2, C-4 to C-5). The quantitative effect of applying the conservatisms built into these various design procedures and acceptance criteria is to reduce the risk reflected in the MAPE of the design basis ground motions so that it meets the corresponding performance goals.

423. The experts for both the Applicant (Dr. Cornell) and the State (Drs. Arabasz and Bartlett) "emphatically" agreed on the appropriateness of applying this two-factor, or two-handed, approach to evaluating the seismic safety of the PFSF. Tr. 9120-21, 9187-89, 9199, 10048, 10150-51 (Arabasz); Tr. 12804-05, 12859-60, 12878 (Bartlett); Tr. 8012-13 (Cornell). The NRC Staff also agreed in principle with the fact that conservatisms in the PFSF seismic design would reduce the probability of seismic failure of the PFSF to be less than the MAPE for the 2,000-year DBE, but the Staff's approach in evaluating those conservatisms, which is challenged by

the State, differed from that of PFS and the State. See Stamatakos/Chen/McCann Dir. at A25, A31; Tr. 12716-17 (Stamakatos). We turn next therefore to the different views of the parties about the application of these principles.

b. PFS-State of Utah Disputes on Adequacy of 2,000 Year DBE

i) Position of PFS and State on Adequacy of 2,000 Year DBE

424. Dr. Cornell's opinion on the adequacy of a 2,000 year DBE for the PFSF is based on two conclusions. The first conclusion is that the risk reduction factors (R_R) applicable to the SSCs important to hazardous material containment for the PFSF are 5 to 20, or greater. These R_R levels, coupled with the 2000-year ($5x10^{-4}$ MAPE) DBE imply that the PFSF SSCs will achieve a performance goal of $1x10^{-4}$ or better. Dr. Cornell's second conclusion is that $1x10^{-4}$ is an appropriate performance goal for the PFSF based on the risk-graded principles for seismic safety discussed above. Cornell Dir. at A54.

425. Dr. Cornell's conclusion that the risk reduction factors (R_R) applicable to the SSCs important to hazardous material containment for the PFSF are 5 to 20 or greater is based on his familiarity with the conservatisms embodied in nuclear codes and standards and evidence of actual conservatisms in the PFSF seismic design. Specifically, Dr. Cornell's conclusion is based on (1) his general knowledge and experience regarding risk reduction factors as applied to many different types of structures designed to a wide variety of codes and standards; (2) his general knowledge and experience of risk reduction factors applicable to nuclear power plants designed in accordance with the applicable design codes and standards as specified by the NRC NPP SRP (NUREG-0800); (3) his independent review of the SRPs applicable to ISFSIs and spent fuel storage casks (NUREGs 1567 and

1536) and confirmation that the codes and standards applicable to nuclear power plants are generally applicable to ISFSIs, such as the PFSF; (4) confirmation by those responsible for the design of the structures and components at the PFSF that such structures and components are generally designed to the same codes and standards applicable to nuclear power plants; (5) analytical and qualitative demonstration by those responsible for the design of the PFSF of significant beyond-design-basis margins for structures and components important to safety; (6) the limited fraction of time that certain SSCs are in use; (7) a showing by Holtec through analysis that casks at the PFSF will not tip over at the 10,000-year earthquake and (8) analyses by Holtec showing that a postulated cask tipover will not result in breach of a cask and release of radioactivity. Cornell Dir. at A45.

426. Dr. Cornell concluded that 1×10^{-4} is an appropriate performance goal for the PFSF is based on several considerations. First, the use of a probability of seismic failure or performance goal for the PFSF of 1×10^{-4} is consistent with the risk-graded probabilistic approach that the Commission has adopted. Second, a performance goal of 1×10^{-4} is consistent with DOE policy as represented by DOE-STD-1020, which provides a performance goal of 1×10^{-4} for ISFSIs, for facilities comparable to the PFSF. Third, a performance goal of 1×10^{-4} provides a lower probability of failure than the performance goals associated with even critical structures, such as bridges and hospitals. Cornell Dir. at A55; Tr. 12961-63 (Cornell).

427. The State's witnesses agreed with Dr. Cornell that 1×10^{-4} is an appropriate performance goal for the PFSF. PFS Exh. EEE at 80-81; Tr. 10154-55 (Arabasz); Tr. 12798-99 (Bartlett). Further, the State's witnesses agreed that if the risk reduction factors (R_R) applicable to the SSCs important to hazardous material containment

for the PFSF are 5 to 20 or more as concluded by Dr. Corenell, then the performance goal of $1x10^{-4}$ would be met. Tr. 9134, 9180-81, 10154 (Arabasz). However, Dr. Bartlett raised issues concerning the risk reduction factors available in the design of the SSCs important to hazardous material containment for the PFSF, which we discuss next.[30]

ii) Appropriate Risk Reduction Factors for Typical SSCs Designed to NRC SRPs

428. As stated, Dr. Cornell's conclusion that the risk reduction factors (R_R) applicable to the SSCs important to radioactive material containment for the PFSF are 5 to 20 or greater is based on his familiarity with the conservatisms embodied in nuclear codes and standards and evidence of actual conservatisms in the PFSF seismic design. The State acknowledges that "Dr. Cornell is a recognized expert in [the] area of evaluating conservatisms that exist in codes and standards." Tr. 10159-62 (Arabasz).

429. It is well established that the NRC guidelines on design acceptance criteria and procedures for nuclear power plants set forth in the Standard Review Plan (NUREG-0800) (Staff Exhs. CC-EE, and 64) contain many conservatisms that result in significant risk reduction factors for typical nuclear power plant components. These conservatisms are introduced through prescribed analysis methods, specification of material strengths, limits on inelastic behavior, etc. However, unlike DOE-1020, the conservatism levels in the NRC acceptance criteria guidelines are not keyed to specific risk reduction factors. Nonetheless, the risk reduc-

[30] Dr. Arabasz did not take issue with the risk reduction factors of 5 to 20 or greater that Dr. Cornell concluded exist for PFSF SSCs and indeed agrees, as set forth in the findings above, that "Dr. Cornell is a recognized expert in [the] area of evaluating conservatisms that exist in codes and standards." Tr. 10159-62 (Arabasz); see also id. at 9180.

tion factors achieved through the use of NRC guidelines for typical nuclear power plant SSCs have been found to be equal to, or higher than, the risk reduction factor of 10 for PC4 category facilities designed to DOE-STD-1020. Cornell Dir. at A30-31; PFS Exh. DD (DOE-STD-1020-94, p. 2-2, C-4 to C-5) ("[c]riteria for PC4 approach the provisions for commercial nuclear power plants").

430. The significant risk reduction factor (of 5 to 20, or more) for typical nuclear power plant SSCs was established by seismic risk analyses performed at many NPPs. Virtually all the current U.S. NPPs were designed based on Appendix A "deterministic" design basis ground motion, prior to the adoption of PSHA methodologies, and on SRP guidelines that were intentionally more conservative than, for example, corresponding building design standards. Subsequent PSHAs for these NPPs established that the Appendix A design basis ground motions had a mean return period of approximately 10,000 years. Further, numerous seismic probabilistic risk analyses ("PRAs") and seismic margin studies were also subsequently performed for SSCs at existing NPPs which established the beyond-design-basis robustness for SSCs designed to the NPP SRP. The results of these PRAs and margin studies provide the data upon which the general range of risk reduction factor values of 5 to 20 or more for typical NPP SSCs designed to the NRC's SRPs is based. These conservatisms in the design of NPP SSCs enable NPPs to achieve a performance goal of about 1×10^{-5}. Rebuttal Testimony of C. Allin Cornell to the Testimony of State Witness Dr. Walter Arabasz on Section E of Unified Contention Utah L/QQ, June 27, 2002 (Introduced at Tr. 12951) ("Cornell Reb.") at A3, following Tr. 12952-53 (Cornell); Cornell Dir. at A31-A32, A40 and Attachment A.

431. The NRC's SRPs for ISFSIs, NUREG-1567,[31] and for dry cask storage systems, NUREG-1536[32] generally provide for use of the same codes and standards employed for NPPs under NUREG-0800. By virtue of this commonality of design procedures and acceptance criteria, similar levels of conservatisms can be expected for SSCs designed to the SRPs for ISFSIs and dry storage systems as for NPP SSCs designed to NUREG-0800. Cornell Dir. at A34-A37. Additionally, those responsible for the PFSF design testified that in designing the PFSF they generally used the same design criteria and procedures applicable to nuclear power plants and applied the standards and codes applicable for nuclear components. Singh/Soler Dir. at A19 & A20; Ebbeson Dir at A7, A14; Trudeau D Dir. at A8 & A9; Young/Tseng Dir. at A30-A34. Because SSCs at the PFSF are designed following the same codes and standards as those for nuclear power plants, the conclusion that the risk reduction factors for typical systems, structures, and components designed to the NPP SRP are in the range of 5 to 20 (or greater) would apply to such structures systems and components at the PFSF. Cornell Dir. at A39.

432. Dr. Bartlett suggested however, that the SRPs for ISFSIs and dry storage systems "may already incorporate less design conservatism" than NUREG-0800 for NPPs, from which he argued that it would be improper to use a risk reduction factor for typical SSCs of 5 to 20 (or greater) based on their design to the SRPs for ISFSIs and dry storage systems. Bartlett Section E Dir. at A27. However, this statement was merely an expression of "concern," and not one of reasoned expert opinion.

[31] U.S. Nuclear Regulatory Commission, NUREG-1567, *Standard Review Plan for Spent Fuel Dry Storage Facilities,* March 2000). (Staff Exh. 53)

[32] U.S. Nuclear Regulatory Commission, NUREG-1536, *Standard Review Plan for Dry Cask Storage Facilities,* January 1997). (Staff Exh. 58)

Tr. 12824 (Bartlett). Unlike Dr. Cornell, who has reviewed and compared the ISFSI and dry storage SRPs against NUREG-0800 and has determined that their levels of conservatism are comparable, Dr. Bartlett has not evaluated the SRPs for ISFSIs and dry storage systems against NUREG-0800. Therefore, he could not opine on the relative conservatisms of the ISFSIs and dry storage systems SRPs compared to those in NUREG-0800. Tr. 12824-25, 12919-20, 12939-40 (Bartlett). Moreover, as stated above, the actual design of the PFSF SSCs did follow the same codes and standards as those used for nuclear power plant design. Therefore it is appropriate to use a R_R, in the range of 5 to 20 (or greater) for typical SSCs at the PFSF.

iii) The CTB Building and the Cranes and Seismic Struts therein are Typical SSCs

433. The CTB (including the building itself and the cranes and seismic struts inside the building) are typical of NPP SSCs for which the risk reduction factor has been shown to be a factor of 5 to 20 or more by the many seismic PRAs and seismic margins studies and evaluations that have been undertaken for NPPs. Cornell Dir. at A40, A48; Cornell Reb. at A3. This is sufficient to conclude that the CTB and the cranes and seismic struts inside the CTB have a risk reduction factor of five or more. Cornell Dir. at A48. The State did not take issue with the appropriateness of using a risk reduction of 5 or more for the CTB and the cranes and struts therein Tr. 9132 (Arabasz); Tr. 12786, 12814 (Bartlett).

434. In addition, the testimony of Mr. Ebbeson describes the existence of significant beyond-design-basis margins in the design of the CTB and the cranes and struts therein. Ebbeson Dir. at A20; see also Tr. 7989 (Cornell). Further, the CTB cranes and seismic struts are in use at most approximately 20% of the time and

thus a canister would be exposed to potential risk of damage due to their failure only for that fraction of the time. Lewis Dir. at A11. For such intermittent-use components, the annual likelihood of failure during a safety-important operation is further reduced 5 times, thereby effectively increasing the R_R factor for these components by a factor of 5. Cornell Dir. at A49. The testimony of Messrs. Ebbeson and Lewis provides additional direct support for the use of a risk reduction factor of five or more for the CTB and the cranes and struts therein.

iv) Appropriate Risk Reduction Factor for Foundations

435. The State did take issue with applying a risk reduction factor of 5 to 20 or more for typical NPP SSCs to the foundations for the CTB and the storage pads for potential foundation failure mechanisms i.e., sliding, loss of bearing capacity and overturning. Bartlett Section E Dir. at A--; Tr. 12785-86 Bartlett (opinions rendered in Section E testimony "limited to conservatisms for foundations" and in "the foundation design"); Id. at 12825 (Bartlett) ("no basis to disagree with Dr. Cornell['s]" conclusion that "the levels of conservatisms are the same with respect to SRPs for nuclear power plants and those for ISFSIs" other than "foundation design" issues); see also Tr. 12819-12824, (Bartlett).

436. Dr. Bartlett made two arguments to support his position that a risk reduction factor of 5 to 20 or more for typical NPP SSCs is inapplicable to the storage pad and CTB foundations. First, Dr. Bartlett asserted that the seismic PRAs and margins studies on which the 5 to 20 risk reduction factor for typical NPP SSCs is based would not have included potential soil failure mechanisms for NPP foundations. Tr. 12812-17 (Bartlett). However, Dr. Bartlett acknowledged on cross-examination that he did not know in fact whether these seismic PRAs and margins studies did or did not include potential failure due to foundation sliding, overturn-

ing and loss of bearing capacity.[33] Id. at 12817. On the other hand, Dr. Cornell testified, based on his extensive knowledge of this area, that the seismic PRAs and seismic margins studies for NPPs did in fact consider NPP foundation failure modes – such as overturning, loss of bearing capacity and sliding – and that these failure modes were not identified "as being critical failure conditions." Cornell Dir. at A41; Tr. 12952-53 (Cornell). Accordingly, it is appropriate to conclude that similar levels of conservatism have been provided by NUREG-0800 for NPP foundation as for other typical NPP SSCs and that a risk reduction factor of 5 to 20 or more is equally applicable to these foundation failure modes. Id.

437. Second, Dr. Bartlett claimed that applying the SRP factor of safety of 1.1 to a smaller earthquake level (as allowed under the PFS exemption) than that of the equivalent safe shutdown earthquake ("SSE") for NPPs reduces the absolute margin terms provided for by the 1.1 factor of safety. Tr. 12835-40 (Bartlett). However, Dr. Bartlett acknowledged that the *proportional* margins would be the same. Id. at 12840. Moreover, the actual margins provided for by the PFSF foundation design are much greater than the 10% suggested by the SRP factor of safety due to numerous conservatism in the PFSF design. Cornell Dir. at A50-51; Trudeau D Dir. at A13-A19; Trudeau Soils Reb. at A2-A3; Ebbeson Dir at A8, A9.

438. Specifically, for example, the factor of safety that PFS calculated for the storage pads against sliding was obtained by applying the following conservatisms:

[33] Of the potential foundation failure mechanisms, the one of "greatest concern" to Dr. Bartlett and the State is the potential sliding of the storage pads. Tr. 12845 (Bartlett). Dr. Bartlett would not expect "overturning of a pad foundation even for a 10,000-year return period" earthquake, and has testified that PFS's "bearing capacity analysis" for the pads for the 2,000-year return period "seems to be adequately conservative." Tr. 12845-46 (Bartlett). Similarly, Dr. Bartlett has no concerns with respect to "catastrophic potential failures of the foundations" for the CTB other than potential sliding of the building. Tr. 12849 (Bartlett). Thus, even for a 10,000 year earthquake event, the primary concern of the State is with respect to potential sliding of the foundations for the storage pads and the CTB.

- The calculated factor of safety of the pads against sliding of 1.27 in the east-west direction and 1.36 in the north-south direction did not take into account the passive resistance provided by the soil cement around the pads. Taking credit for this conservatism would increase the factor of safety from 1.27 to 3.3 in the east-west direction and from 1.36 to 2.35 in the north-south direction without taking other conservatisms into account. Trudeau D Dir. at A18-A19.[34]

- In addition, the calculation for sliding is based upon the static shear strength of the underlying clay silt soils. Trudeau Soils Reb. at A3. It is undisputed that the underlying clayey silt soils will exhibit greater strength under the dynamic loadings experienced under an earthquake of at least 30% and potentially up to 100%. Tr. 11967-68 (Trudeau); Tr. 12858, 12976-77 (Bartlett); Trudeau Dir. on Section D at A15-A16; Bartlett Soils Reb. at R3. Assuming a 50% increase in strength would increase the factor of safety for the east-west base case from 1.27 to 1.9, again without taking other conservatisms into account. Trudeau Soils Reb. at A3.

- PFS computed the minimum 1.27 and 1.36 factors of safety using the lower-bound, worst-case static shear strength for the entire pad storage area. Tr. 11960-62, 11966 (Trudeau); PFS Exh. 238. Further, this lower-bound strength was obtained from the weakest layer of soil underlying the pads whereas the pads will be resting in most cases on the soils above this layer which are much stronger than the weakest layer for which the lower bound shear strength was determined. Trudeau Soils Reb. at A3.

- Any measurement of the strength of soils will disturb the soils and result in soil strength values that are less than the actual strength that the soils will exhibit in place. Therefore, when the measured value of strength is used in the factor of safety computations, there is a "built-in" conservatism because the actual strength of the soil in place will be higher. Trudeau Soils Reb. at A3.

- The minimum factor of safety is applicable only when the earthquake reaches its peak magnitude. At all other times there is considerably more margin available. Trudeau Soils Reb. at A2-A3.

[34] The calculation with the passive resistance of the soil cement was based upon a minimum compressive strength of 250 psi. Trudeau D Dir. at A14. In fact, the compressive strength of the soil cement is likely to be greater providing more passive resistance than that calculated. Id.

- Further, due to the cyclic nature of the seismic loading each of the peak accelerations that impart dynamic loads from the earthquake exist for only one very brief moment of time – typically less than 0.005 seconds – and then the seismic loading reverses direction, which minimizes any sliding displacement that would occur. Trudeau D Dir. at A9.

439. Thus, PFS's calculation of the minimum factors of safety against pad sliding are "exceptionally conservative." Removing the various conservatisms in the calculation would result in a much greater factor of safety against pad sliding (of at least 5 for the east-west base case). Tr. 11968 (Trudeau); Trudeau Soils Reb. at A3; Trudeau D Dir. at A14-A24. Moreover, if pad sliding does occur, it reduces significantly the seismic loading to which the casks are subjected and therefore reduces the potential for radiological release. Singh/Soler Dir. at A70.

440. There is similarly a large margin against pad failure due to the loss of soil bearing capacity. The minimum factor of safety of 1.17 against bearing capacity failure for the storage pads was computed using the extremely conservative assumption that 100% of the earthquake loads act in both horizontal directions at the same time. Trudeau D Dir. at A22; Trudeau Soils Reb. at A3. If the load combinations allowed by ASCE 4-86 were used instead, the factor of safety against loss of bearing capacity would be increased to 2.1. Trudeau D Dir. at A16; Trudeau Soils Reb. at A3; <u>see also</u> Bartlett Soils Reb. at R3 (states ASCE 4-98 would increase safety factor).

441. Another major conservatism in the computation of the factor of safety against loss of bearing capacity is the use of the lower bound static shear strength of the weakest layer of soil underlying the pads. Standard practice for computing bearing capacity is to average the contributions of all soil layers over a depth equal to the shortest dimension of the foundation, in this case the 30 feet width of the pads. Approximately 2/3 of this depth below the pads would have soils or cement-

treated soils that would be much stronger than the weakest layer of soil from which the lower bound static strength was measured. Using the average strength of the cement-treated soil and soil for the 30 ft. below the pad and the soil's dynamic strength rather than its static strength would have significantly increased the factor of safety against loss of bearing capacity failure. Trudeau Soils Reb. at A3. Also, as noted with respect to pad sliding, the laboratory measured strength of the soils would be less than their in situ strength and the maximum earthquake magnitude to which the pads would be subject would be cyclic and of very short duration. Trudeau Soils Reb. at A2-A3; Trudeau D Dir. at A9.

442. Taking into account just two of the above many conservatisms (use of the load combinations allowed by ASCE 4-86 and the dynamic strength of the clayey soils) would increase the factor of safety for the pads against loss of bearing capacity to 3.63, which would provide a factor of safety of 1.0 against loss of bearing capacity for vertical and horizontal earthquake accelerations of 1.24g and 1.27g respectively, essentially the same as the 10,000 year earthquake accelerations for the PFSF site. Trudeau D Dir. at A9. Thus, as acknowledged by Dr. Bartlett, the bearing capacity analysis performed by PFS for the 2,000-year return period earthquake is "adequately conservative." Tr. 12846 (Bartlett). It provides ample margin to conclude that a risk reduction factor of more than 5 applies with respect to the pads' capability to withstand a loss of bearing capacity. Cornell Dir. at A51.

443. The factor of safety against pad overturning is 5.6, without taking into account any conservatism, Trudeau D Dir. at A23, and Dr. Bartlett acknowledged that he would not expect "overturning of a pad foundation even for a 10,000-year return period." Tr. 12846-47 (Bartlett). Thus, the margins against pad overturning are

also sufficient to conclude that a risk reduction factor of more than 5 applies with respect to pad overturning. Cornell Dir. at A51.

444. There are also numerous conservatisms included in the design of the foundations of the CTB such that, as acknowledged by Dr. Bartlett, "catastrophic" failure of the CTB due to overturning or loss of bearing capacity would not occur for a beyond-design basis earthquake event. See Tr. 12849 (Bartlett). For example, removing some of the conservatisms in the analysis results in a factor of safety against loss of bearing capacity of the CTB on the order of 10, and the 2,000-year return period earthquake accelerations would have to increase by a factor of more than four to reduce this factor of safety to 1.0. Trudeau D Dir. at A16, pages 7-8. Similarly, the CTB would not overturn during a 10,000-year earthquake event. Ebbeson Dir. at A16. Therefore, the risk reduction factors applicable to these foundation failure modes would be of 5 or more. Cornell Dir. at A50.[35]

445. Dr. Bartlett suggested at the hearing that one could not conclude that a foundation failure would not occur for a 10,000-year return period earthquake based on the margins for the 2,000-year return period DBE without performing the equivalent, formal design calculation for the 10,000 year event. Tr. 12841-42, 12874-75 (Bartlett). However, in determining the available margins associated with a DBE, such as the 2,000-year return period DBE for the PFSF, the purpose is to strip away the conservatisms and determine at what point failure would realistically occur. Therefore, it would be wholly inappropriate to require performance of a

[35] The margins against sliding of the CTB are not as large. Trudeau D Dir. at A16, page 8. But as already discussed, no negative safety consequences would result from sliding of the CTB. Tr. 7323-24 (Bartlett/Ostadan); Ebbeson Dir. at A18, A25; Cornell Dir. at A50.

10,000 year analysis using the same conservative SRP design assumptions as used for the design. Tr. 12954-56 (Cornell).

446. It is also not necessary to do a formal 10,000-year return period earthquake evaluation to show a lack of SSC failure in the event of a 10,000 year earthquake. One can determine, as reflected by the discussion above, that sufficient conservatisms exist in the design of the SSCs and their foundations to meet the increase in loadings due to the higher ground accelerations for the 10,000-year event. Indeed, if anything, the demands placed on foundations would be proportionally less for higher earthquake levels, due to the higher damping that would be associated with the higher strain levels in the soil for the 10,000-year event so that such an approach would be both appropriate and conservative. Id; see also Ebbeson Dir. at A18.

447. Therefore, risk reduction factors of five or more are appropriate for foundation failures associated with overturning, loss of bearing capacity and sliding of the storage pads. Moreover, foundation failure of the pads would not by itself constitute ultimate failure of the PFSF resulting in radioactive release, but would be part of a chain of events that one would need to analyze to determine whether the ultimate performance goal had been met. Tr. 12802-03 (Bartlett). In this respect, the record shows that the foundation failure mechanism of the pads of most concern to the State, sliding of the storage pads, would in fact reduce the loads transferred to the storage cask on the pad and reduce the likelihood of cask tipover. Singh/Soler Dir. at A70; see also Tr. 10377 (Bartlett). Similarly, the risk reduction factors for turnover and loss of bearing capacity of the CTB would be five or more, and any potential sliding of the CTB that might occur for a 10,000-year event would result in no adverse health ~~of~~ OR safety impact.

v) Appropriate Risk Reduction Factor for the Casks

448. The HI-STORM 100 cask system is designed to the SRP for dry storage systems, NUREG-1536, including SRP-dictated accident conditions, such as hypothetical drop and tip-over events. Singh/Soler Dir. at A43. The cask and canister are not, however, "typical" NPP SSCs for which R_R factors of 5 to 20 or more have been demonstrated. Therefore, some further analysis is necessary to provide confidence that the desired performance goal for the HI-STORM 100 cask system has been achieved. Both Holtec and Sandia have performed beyond design basis analyses of the HI-STORM 100 cask system which demonstrate that the casks will not tip over during a beyond-design basis 10,000-year return period earthquake and that significant margins still remain against tipover even at the 10,000 year earthquake event. These analyses demonstrate that the effective R_R of the HI-STORM 100 cask system is in excess of 5 because the casks can survive both the 2,000 year DBE and the beyond-design-basis 10,000 year earthquake. Accordingly, the design of the HI-STORM 100 provides risks reduction factors comparable to those available for typical NPP SSCs. These demonstrations are in themselves sufficient to provide confidence that a performance goal on the order of 10^{-4} has been achieved. Cornell Dir. at A42, A52; Cornell Reb. at A3; <u>see also</u> Tr. 9134, 9180-81, 10154 (Arabasz); Tr. 12844-45 (Bartlett).[36]

449. Specifically, the Holtec beyond-design basis analyses showed maximum cask rotations for the 10,000-year return period earthquake event of approximately 10 to 12 degrees, still providing a factor of safety against tipover on the order of 2 to 3, as measured against the center-of-gravity over corner location of 29.3 degrees at

[36] Dr. Bartlett premised his agreement on a hypothetical basis, assuming no foundation failure and resolution of the cask stability issues raised by Dr. Khan. These issues have already been dealt with above. <u>See</u> Findings in Section III.B <u>supra</u>.

which the cask would tip over on its own accord. Further, many of the 10,000-year beyond design bases evaluations performed by Holtec assumed unrealistic, "worst-case" assumptions regarding soil damping and other factors. The demonstration under such worst-case assumptions that the casks would not tip over, with significant factors of safety still remaining, provides confidence that the casks would not tipover during even a 10,000-year earthquake event. Singh/Soler Dir. at A169; Cornell Dir. at A52; Cornell Reb. at A3; Tr. 6106-08 (Soler).

450. This conclusion is supported by the Sandia analyses which used sophisticated modeling techniques. The Sandia cask stability analyses showed cask rotations on the order of 1 degree for 10,000-year return period earthquake event, suggesting even larger margins of safety against tipover than those demonstrated by Holtec. Luk/Guttman Dir. at A16; Tr. 11661 (Luk).

451. Assuming, however, the casks were to tipover, it has been demonstrated that no breach of the confinement barrier of the canister containing the spent nuclear fuel would occur. Holtec has performed a hypothetical, non-mechanistic tipover analysis that demonstrates the decelerations at the top of the canister due to tipover would remain within the HI-STORM 100 Cask System's 45g design basis limit. Singh/Soler Dir. at A35. Moreover, as is typical of design basis limits, large conservatisms exist in this analysis.[37] In the first place, the actual g limit

[37] Dr. Bartlett expressed the opinion that a tipover under seismic earthquake conditions would have angular velocities greater than the initial zero angular velocity at the center of gravity over corner position used by Holtec in its hypothetical tipover analysis. Tr. 12870-71, 12913-15 (Bartlett). However, such analysis is beyond his area of expertise and he had done no evaluation or analysis of angular velocity during tipover. Id. at 12915. Contrary to Dr. Bartlett, Drs. Singh and Soler concluded from their evaluation of the PFSF beyond design basis analyses and other analyses they have performed that the angular velocity at impact of casks tipping over under seismic conditions would likely be less than that resulting from assuming an initial angular velocity of zero at the center of gravity over corner assumed by Holtec in its hypothetical tipover analysis because of precession of the casks prior to tipover. Singh/Soler Dir. at A170. In any event, as discussed above, large margins exist that would preclude breach of the canister's confinement boundary even if the angular velocity at impact in a tipover event

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for the fuel cladding in the fuel assemblies is at least 63g. Additionally, there are large margins in the design of the MPC canister system that would prevent the release of radioactive material under much larger loadings. It has been demonstrated that the canister can withstand a 25 ft. straight drop, unprotected by a cask onto a hard concrete surface, maintaining confinement when subject to forces up to 300g and maintaining significant margins against reaching the failure strain limit of the material. Singh/Soler Dir. at A23; Tr. 12075 (Singh). These large margins against breach of the radioactive confinement barrier provide additional confidence that a performance objective of 10^{-4} has been met with respect to the HI-STORM 100 Cask System, since the cask will maintain containment of the radioactive matter even if tips over in a beyond-design-basis earthquake. Singh/Soler Dir. at A170-A171; Cornell Dir. at A52; Cornell Reb. at A3; see also Tr. 12075-76 (Singh).

vi) Asserted Need for Fragility Curves

452. In his pre-filed testimony, Dr. Bartlett asserted that a major deficiency in PFS's beyond-design basis analysis of the risk reduction factors based on the conservatims inherent in the PFSF design was its failure to develop fragility curves for the SSCs at the PFSF. See, e.g., Bartlett Section E Dir. at A21, A27. Fragility curves are curves that show the probability of failure of SSCs as a function of earthquake strength. Id.; see also Tr. 12794 (Bartlett). However, as explained by Dr. Cornell in his prefiled testimony, while a fragility curve can be developed to show quantitatively the value of a component's risk reduction factor, a fragility curve is not

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were larger than that resulting from an initial angular velocity of zero at the center of gravity over corner position.

needed to confirm that a particular component has a risk reduction factor larger than some specified level or can meet a specified seismic performance level. This can be done by various means, including analysis at the desired performance goal level to show such a goal has been met, as was accomplished by Holtec's 10,000-year beyond design basis analysis of cask stability. Cornell Dir. at A65-A66. Dr. Bartlett acknowledged on cross-examination that it was not necessary to develop fragility curves for the SSCs at the PFSF in order to determine whether the specified performance goal was met. Tr. 12852-53, 12874-75 (Bartlett). Therefore, the need for fragility curves is no longer an issue.

vii) Conclusion on State-PFS Disputes on Adequacy of 2,000-Year Return Period DBE for the PFSF

453. Based on our findings above, we conclude that the risk reduction factors, R_R, attributable to the large conservatisms inherent in the design of the SSCs for the PFSF are on the order of 5 or more, and that therefore a performance goal of 10^{-4} against potential failures that might cause radioactive release at the PFSF has been achieved. In particular, the large margins demonstrated against cask tipover and any subsequent breach of the radioactive confinement barrier, even assuming tipover were to occur, provides great confidence that a performance goal of 10^{-4} has been achieved. The large margins against breach of the radioactive confinement boundary provide a practical answer to many of the concerns raised by the State in this proceeding. As aptly expressed by Mr. Guttman, the Staff's witness, even assuming all of the analysis done by Holtec and Sandia is erroneous and the casks do tip over there will be no significant adverse consequences, even for a 10,000 year return period earthquake. Tr. 7062-64 (Guttman). The showing that a performance goal of 10^{-4} has been achieved establishes that the overall risk from a 2,000-year return period design basis earthquake is sufficiently low that its use as

the DBE for the PFSF is consistent with Commission precedent and policy for protecting public health and safety.

c. NRC Staff-State Disputes on Adequacy of 2,000-year MRP DBE

454. Wholly apart from the State's allegations concerning PFS's justification for the exemption, the State raised separate issues challenging the Staff's rationale for approving the exemption. Employing a risk-graded approach, the Staff – like PFS and the State – determined that the mean annual probability of exceedance for the PFSF DBE could be greater than that for a NPP. The Staff set forth its rationale for this conclusion in the Consolidated SER as follows:

- The radiological hazard posed by a dry cask storage facility is inherently lower and the Facility is less vulnerable to earthquake-induced accidents than operating commercial nuclear power plants (Hossain et al., 1997). In its Statement of Consideration accompanying the rulemaking for 10 CFR Part 72, the NRC recognized the reduced radiological hazard associated with dry cask storage facilities and stated that the seismic design basis ground motions for these facilities need not be as high as for commercial nuclear power plants (45 FR 74597, 11/12/80; SECY-98-071; SECY-98-126).

- Seismic design for commercial nuclear power plants is based on a determination of he Safe Shutdown Earthquake ground motion. This ground motion is determined with respect to a reference probability level of 10^{-5} (median annual probability of exceedance) as estimated in a probabilistic seismic hazard analysis (Reference Reg Guide 1.165). The reference probability, which is defined in terms of the median probability of exceedance, corresponds to a mean annual probability of exceedance of 10^{-4} (Murphy et al., 1997). That is, the same design ground motion (which has a median reference probability of 10^{-5}) has a mean annual probability of exceedance of 10^{-4}. Further, analyses of nuclear power plants in the western United States show that the estimated average mean annual probability of exceeding the safe shutdown earthquake is 2.0×10^{-4} (U.S. Department of Energy, 1997).

- On the basis of the foregoing, the mean annual probability of exceedance for the PFS Facility may be defined as greater than 10^{-4} per year.

Staff Exh. C at 2-50 and 2-51. The Staff's SER also cited DOE-STD-1020-94 and the Commission's approval of an exemption authorizing a 2,000-year return period DBE for TMI-2 ISFSI as additional support for approving the PFS exemption:

- The DOE standard, DOE-TD-1020-94 (U.S. Department of energy, 1996), defines four performance categories for structures, systems, and components important to safety. The DOE standard requires that performance Category-3 facilities be designed for the ground motion that has a mean recurrence interval of 2000 yrs (equal to a mean annual probability of exceedance of 5×10^{-4}). Category-3 facilities in the DOE standard have a potential accident consequence similar to a dry spent fuel storage facility.

- The NRC has accepted a design seismic value that envelopes the 2000-yr return period probabilistic ground motion value for the TMI-2 ISFSI license (Nuclear Regulatory Commission, 1998b; Chen and Chowdhury, 1998). The TMI-2 ISFSI was designed to store spent nuclear fuel in dry storage casks similar to the PFS Facility.

Id. at 2-51. The references to DOE-STD-1020 and the TMI-2 ISFSI exemption were considered to be illustrative rather than binding precedents. For example, the Staff used the DOE-STD-1020-94 as illustrative of the acceptability of a MAPE of 5×10^{-4} under a risk-graded approach for ISFSIs, but did not adopt the standard as a regulatory criterion for use in licensing the PFSF. Likewise, the TMI-2 ISFSI – discussed in SECY-98-071 – was not referenced as establishing a regulatory criterion, but as an example of the Commission's general acceptance of PSHA methodology and principles, and of the application of risk-graded approaches to an ISFSI. Stamatakos/Chen/McCann Dir. at A14.

455. In its testimony, the Staff generally referred to the conservatisms inherent in the PFSF design that we have discussed at length above. For example, in discussing DOE-STD-1020, the Staff quoted from SECY-98-071 as follows:

> Considering the minor radiological consequences from a canister failure, and the lack of a credible mechanism to cause a failure, the staff finds that the DOE approach of using the 2000-year return period mean ground motion as the design earthquake for dry storage facilities is adequately conservative.

Stamatakos/Chen/McCann Dir. at A-25. In this respect, the Staff concluded that the HI-STORM System casks would not tip over at the PFSF even under a 10,000 year earthquake event and that, even if cask tipover occurred, no adverse consequences would result. Further, in discussing Basis 6(a) of the contention (concerning whether seismic design requirements for new Utah buildings and highway bridges are more stringent than those under the exemption granted to PFS) the Staff observed that because "SSCs important to safety at the proposed PFS Facility will be designed to NRC seismic design requirement, the resulting factors of safety and conservatism will be greater than those achieved by building codes." Id. at A31.

456. Thus, the Staff recognized the conservatisms in the design of SSCs at the PFSF that enable the SSCs to withstand earthquakes more severe than the DBE 2,000-year mean return period. The Staff did not, however, attempt to formally quantify those conservatisms or to arrive at an applicable risk reduction factor, as done by PFS.

457. The State attacks this absence of a formal determination of risk reduction and the corresponding failure to show the achievement of a specified target performance goal. See, e.g., Tr. 10145 (Arabasz). However, PFS's extensive analysis of the

conservatisms in the PFSF design and its determination of the applicable risk reduction factors would fill any void that may have existed in the Staff's rationale for approving the exemption.

458. In addition, there is no fatal flaw in the Staff's approach, even assuming that PFS had not filled this void. As discussed above, the seismic design criteria and procedures applied to the PFSF were generally the same as those applied to NPPs with which the Staff is thoroughly familiar. The NRC's seismic design criteria and procedures are recognized to contain numerous conservatisms, as reflected by the observation in DOE-STD-1020 that the risk reduction factor of 10 for PC-4 category structures "approaches the provisions for commercial nuclear power plants." The NRC Staff knows and understands the inherent margins in its seismic criteria. Indeed, its reference to DOE-STD-1020 certainly reflects the Staff's awareness of the role and importance of design conservatisms as part of the two-handed approach – explicitly endorsed in DOE-STD-1020 – and, as explained by Dr. Cornell, implicitly embodied as well in the NRC's seismic acceptance criteria.

459. Furthermore, based on its review, the Staff concluded that there were significant conservatisms in the results of the Geomatrix PSHA due to, *inter alia*, a very conservative seismic source characterization. Stamatakos/Chen/McCann Dir. at A8, A12; the Staff Exh. C, Consolidated SER (Sections 2.1.6.1 and 2.1.6.2).

460. Among the factors leading to the Staff's conclusion that the Applicant's PSHA was overly conservative were proprietary industry gravity data that indicated that the West fault near the site was not an independent seismic source as the PSHA had treated it. Stamatakos/Chen/McCann Dir. at A12; Stamatakos, et al. (1999) Staff Exh. Q. The Staff concluded that the West fault is a splay of the larger East fault, incapable of generating large magnitude earthquakes independently. In con-

trast, the Geomatrix PSHA treats the West fault as capable of producing a large magnitude earthquake, and therefore a contributor to the seismic risk at the PFSF. Stamatakos/Chen/McCann Dir. at A12.

461. The Staff also concluded that Applicant's PSHA was conservative in terms of its site-to-source distance models in the ground motion attenuation relationships, and in the development of distributions of maximum earthquake magnitude based on the dimensions of fault rupture. Stamatakos/Chen/McCann Dir. at A12. The Staff undertook an independent "slip tendency analysis," which concluded that the segments of the East fault and the East Cedar Mountain fault nearest the PFSF site have relatively low slip tendency values compared to segments farther north in Skull Valley, making the seismic source characterization in PFS's PSHA overly conservative. Stamatakos/Chen/McCann Dir. at A12; Staff Exh. C at 2-38 to 2-40).

462. The Staff also found Applicant's PSHA to be overly conservative in that it over-estimated the maximum magnitude of the East and East Cedar Mountain faults near the proposed PFSF site. The relatively low slip tendency values found by the Staff would lead to fault models with smaller rupture dimensions – and hence smaller magnitude earthquakes – than those used by PFS. Stamata-kos/Chen/McCann Dir. at A12.

463. The Staff also concluded that the PSHA results obtained by PFS are conservative by comparison of those results to other sites in Utah, especially around Salt Lake City. Despite having fault sources near Salt Lake City that are larger and more seismically active than those near the PFSF site, PFS's PSHA suggests that the seismic conditions at the PFSF site are 1.5 times more likely to produce a peak horizontal ground acceleration of 0.5g or greater than accelerations predicted for

Salt Lake City by the USGS National Earthquake Hazard Reduction Program. Stamatakos/Chen/McCann Dir. at A12; Staff Exh. Q.

464. Likewise, the 2,000-year return period horizontal peak ground acceleration for Skull Valley estimated by PFS was found to be higher than the 2,500-year ground motions for the nine sites along the Wasatch Front, which were evaluated as part of the Utah Department of Transportation I-15 Reconstruction Project. Stamatakos/Chen/McCann Dir. at A12; Staff Exh. Q. The peak horizontal ground acceleration calculated for those nine sites along the I-15 corridor ranged between 0.561g and 0.686g, based on a mean annual probability of exceedance of 4×10^{-4} (2,500-year return period). Despite the fact that the I-15 corridor sites lie close to Wasatch Fault, which has a slip rate nearly ten times that of the Stansbury or East Faults and which is capable of larger magnitude earthquakes, the PSHAs for these sites result in substantially lower ground motions than the .711g horizontal PGA calculated for the PFSF site based on a 2,000-year return period earthquake. Stamatakos/Chen/McCann Dir. at A12.

465. Thus, the Staff concluded that the results of the Applicant's PSHA could be conservative "by as much [as] 50% or more," and that this conservatism "provides an additional margin of safety in the seismic design" of the PFSF. Stamatakos/Chen/McCann Dir. at A12.

466. The State did not challenge the adequacy of the Applicant's PSHA to represent the seismic hazard at the PFSF; indeed Dr. Arabasz concluded that Geomatrix had done a "good job" with respect to the PSHA for the PFSF. Tr. 9119-20, 9965, 9970-71, 9977-78 (Arabasz). Dr. Arabasz also did not take issue with the specific conservatisms that the Staff had identified in the PSHA that Geomatrix performed for the PFSF site (although he did take issue with the comparisons that the Staff

had drawn with the earthquake hazard along the Wasatch front and the earthquake hazard at the PFSF). Tr. 9864-65, 9878-80 (Arabasz).

467. The State also took issue with testimony by the Staff that an appropriate benchmark for a NPP SSE at the PFS site would be a 5,000-year return period earthquake as opposed to a 10,000-year return period earthquake. See Tr. 10091-94 (Arabasz). The State and the Staff agree that whether a 5,000 or 10,000 year earthquake for a NPP at the PFSF site is the appropriate benchmark turns on whether the PFSF is a high seismicity site.

468. In this respect, as testified to by Dr. Stamatakos, the hazard curve produced by Geomatrix for the PFSF site is similar to the hazard curves for many high-seismicity sites along the San Andreas fault. Tr. 12753-54 (Stamatakos). From this similarity, Dr. Stamatakos concluded that if the PFSF is not a high seismicity site, the real hazard curves should not be as high as those produced by Geomatrix, from which it would follow that the PFS facility has been designed to a significantly higher return period than the 2,000 year return period ground motions obtained from the Geomatrix PSHA hazard curves. Tr. 12754 (Stamatakos). If that were the case, the design basis ground motions obtained from the Geomatrix PSHA would be overly conservative.

469. On the other hand, Dr. Stamatakos testified that if the hazard curve produced by Geomatrix accurately reflects the conditions at the PFSF, and the 2,000 year return period earthquake has a horizontal acceleration in excess of 0.7g, then such a high ground acceleration for a 2,000 year return period earthquake would by definition classify the PFSF as a high seismicity site, and it would be appropriate to use a 5,000-year mean return period earthquake as the NPP SSE benchmark. Tr. 12754 (Stamatakos).

470. We do not need to resolve the dispute on whether the PFSF is a high seismicity site such that the appropriate benchmark NNP SSE would be 5,000 years. We note that, although the Staff testified to a 5,000-year NPP benchmark at the hearing, the SER only concludes that, because the PFSF's risk is lower than that of a NPP, the PFSF may have a design basis earthquake that has a mean annual probability of exceedance greater than 1×10^{-4}. The 2,000-year DBE selected for the PFSF design is consistent with the Staff's determination.

471. Further, we note that the Staff has identified what it considers to be many conservatisms in the Geomatrix PSHA. Therefore, the 2,000-year DBE constitutes a conservative prediction of the seismic hazard at the PFSF. This conservatism is above and beyond the inherent conservatisms embodied in the PFSF design, and provides additional confidence that the 2,000-year DBF for the PFSF provides sufficient protection of the public health and safety.

d. Specific Issues Raised in Subparts of Section E Other than Radiological Dose Consequences

472. The State raised six specific bases to support what is now Section E of Contention L/QQ. Basis 2 concerns radiation dose consequences, andis discussed Section III.C.4 below. The remaining bases are addressed specifically here.

i) Section E, Basis 1

473. In Basis 1, the State challenged the exemption granted by the NRC Staff to PFS authorizing the use of a 2,000-year return period DBE for the PFSF on the grounds that the exemption failed to conform to the rulemaking plan set forth in SECY-98-126 (June 4, 1998). That SECY discussed three different rulemaking options for the Commission to incorporate PSHA methods into 10 C.F.R. Part 72, with the "preferred" approach being a 1000-year mean return period design basis

earthquake for Category 1 SSCs (those whose failure would result in radiological doses less than the dose limits specified in 10 C.F.R. § 72.104(a)) and a 10,000-year mean return period design basis earthquake for Category 2 SCCs whose failure would result in radiological doses exceeding the dose limits of 10 C.F.R. § 72.104(a). Cornell Dir. at A74.

474. The two-tiered approach set forth in SECY-98-126 is, however, no longer the Commission's preferred approach. In SECY-01-0178, dated September 26, 2001, the NRC Staff recommended to the Commission that the rulemaking plan be modified to add a fourth option. This fourth alternative proposed, as a new "preferred" option, the use of a single 2,000-year mean return period earthquake as the design basis for all ISFSI SSCs. This is the same DBE proposed by PFS in its exemption request. In a Staff Requirements Memorandum dated November 19, 2001, the Commission approved the modification to the rulemaking plan proposed by SECY-01-0178, further instructing the NRC Staff that the proposed rule should solicit comment on a range of exceedance levels from 5.0E-04 through 1.0E-04. Cornell Dir. at A75. Thus the PFSF proposed exemption conforms with this new preferred methodology, rendering the State's concern in Basis 1 obsolete. Cornell Dir. at A76-A77.

475. Furthermore, in admitting Basis 1, both the Commission and the Licensing Board expressly held that PFS was not bound by the rulemaking plan, and that the ultimate issue to be determined is whether the 2000-year design standard is sufficiently protective of public safety and property.

ii) Section E, Basis 2

476. Basis 2 of Section E of Utah L/QQ is discussed in Section III.C.4, below.

iii) Section E, Bases 3-5

477. Bases 3-5 of Section E concern specific issues raised by the State with respect to the logic used by the Staff in approving PFS's exemption request, which have been carried forward as part this proceeding. Tr. at 9158-63 (Arabasz).[38] These bases do not concern whether the PFSF design is sufficiently conservative to withstand an earthquake with a mean return period on the order of 10,000 years discussed in Section III.C.3(b) above. Tr. 9163-64 (Arabasz). Therefore, they do not challenge the justification put forth by PFS for the use of the 2,000 year design basis earthquake.

(a) Section E, Basis 3

478. The claim raised in Basis 3 is that the NRC 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" concerning the relationship between mean and median probabilities for NPP safe shutdown earthquakes ("SSE"). This issue, however, has evolved significantly from the original contention, and indeed even from the pre-filed testimonies. As phrased in both Dr. Arabasz's pre-filed testimony, the issue has metamorphosized into what is the NPP "benchmark" against which to judge the adequacy of the DBE for the PFSF in applying a risk-graded approach. Arabasz Dir. at A10; see also,Cornell Dir. at A83.

479. There appears to be no dispute between PFS and the State on the appropriate NPP SSE benchmark to judge the adequacy of the DBE for the PFSF. Dr. Arabasz in his initial oral testimony in May 2002 stated his belief that the mean annual prob-

[38] Dr. Arabasz was the author of Bases 3-5, in that he provided the technical input for these bases. Tr. at 9115 (Arabasz).

ability of exceedance for a NPP SSE at the PFS site would probably be $1x10^{-4}$. Therefore, Dr. Arabasz concluded that "it would be appropriate in applying the risk graded approach in determining the appropriate design basis [earthquake] for the PFSF" to compare it to a NPP SSE with a mean annual probability of exceedance of $1x10^{-4}$– i.e., an earthquake with a 10,000 year mean return period. Tr. 9176-79 (Arabasz); see also id. at 9207-08. Dr. Arabasz subsequently reconfirmed his belief that the appropriate NPP benchmark against which to compare a risk graded design basis ground motion for the PFSF would be an earthquake with a 10,000 year mean return period (or a mean annual probability of exceedance of $1x10^{-4}$). Tr. 10124 (Arabasz). This is the same benchmark that Dr. Cornell would use in applying the risk-graded approach to the PFSF. Cornell Dir. at A83. On the other hand, the State does dispute the Staff's use of a 5,000 year mean return period earthquake as the benchmark NPP earthquake against which to judge the adequacy of the PFSF 2,000 year design basis earthquake. However, because we find that PFS has established the sufficiency of a 2,000 year design basis earthquake for the PFSF when judged against a NPP benchmark SSE earthquake of 10,000 years, we do need not resolve the question of whether it would be appropriate to use a lower mean return period earthquake as the applicable NPP SSE benchmark.

(b) Section E, Basis 4

480. In Basis 4, the State challenges the NRC Staff's reliance on DOE-STD-1020 as support for its approval of the exemption. Specifically, the State claims that the Staff inappropriately relied upon DOE-STD-1020 as support for use of a 2,000 year design basis earthquake because it did not couple this design basis earthquake with a target performance goal achieved by conservatisms embodied in the

design acceptance criteria, as called for by DOE 1020. Tr. 9160-61, 9179 (Arabasz). In this respect, Dr. Arabasz acknowledged that if the conservatisms set forth in Dr. Cornell's testimony were "shown to exist," then PFS would have established "a target performance level equivalent to a PC-3 category" structure under DOE-1020. Tr. 9179-81 (Arabasz). Those conservatisms have been "shown to exist." As discussed above, the NRC's SRPs implicitly embody conservatisms that are equal to or greater than those provided for by DOE 1020. In addition, PFS has shown that the PFSF design achieves a performance goal on the order of $1x10^{-4}$, equivalent to the goal for ISFSIs under DOE-STD-1020 (which are classified as category PC-3 structures under DOE-1020). Therefore, the analysis set forth in Dr. Cornell's testimony, which fully embraces the two-handed approach embodied in DOE-1020, addresses and resolves the State's concern raised in Basis 4.

481. While the DBE for category PC3 structures under DOE-1020 has recently been changed from 2,000 years to 2,500 years, the level of conservatism in the applicable design procedures and criteria provided for by DOE 1020 was reduced such that the performance goal for PC3 structures remains unchanged at $1x10^{-4}$. Cornell Dir. at A86-A87; Tr. 9305-06 (Arabasz). Thus, the PFSF would continue to achieve a target performance goal equivalent to that for PC3 structures under DOE 1020. Id.

(c) Section E, Basis 5

482. In Basis 5, the State challenges the NRC 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") spent facility fuel as support for granting the PFSF exemption. The State claims that the NRC's

reliance on the INEEL exemption is misplaced because the grant of the exemption there was based on circumstances not present with respect to the PFSF.

483. As acknowledged by Dr. Arabasz, however, the potential precedential value of the INEEL exemption does not directly affect the substantive issue of whether PFS has shown sufficient basis to justify its proposed 2,000 year design basis earthquake. See Tr. 9181 (Arabasz). In this respect, as discussed above, we have concluded that PFS has justified its use of a 2,000-year mean return period DBE for the PFSF using well established risk-principles, with which the State fully agrees. Thus, while the appropriateness of the conclusion reached here is corroborated by the similar determination reached with respect to the INEEL ISFSI, it is not dependent upon the INEEL determination.

iv) Section E, Basis 6(a)

484. In Basis 6(a), 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 ground motion levels for certain new Utah building construction and highway bridges are more stringent. The State's conclusion was based on the observation that, for example, the International Building Code 2000 ("IBC-2000") will, when in effect, require a MRP of approximately 2500 years for the DBE, which is greater than the 2,000-year MRP DBE proposed for PFS. Cornell Dir. at A90. However, the comparison between the two sets of codes based solely on the MRP DBE is completely erroneous. Cornell Dir. at A91.

485. As discussed above, the State "emphatically" agreed with PFS that in order to determine the level of safety achieved by an applicable design one has to take a two-handed approach, addressing both the mean return period of the DBE and the conservatisms embodied in the applicable design procedures and criteria. Cornell

Dir. at A93; Tr. 9120-21 (Arabasz); Tr. 12805 (Bartlett). Therefore, it would be inappropriate to compare solely the 2000 mean return period DBE of the PFSF with the higher MRP DBE of the IBC-2000 or other codes. Tr. 9187-88 (Arabasz); Tr. 12805-09 (Bartlett).

486. The design procedures and acceptance criteria of the IBC-2000 are much less conservative than those specified by the NRC's SRPs. For example, 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. Cornell Dir. at A93, Tr. 7898-7902 (Cornell). 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. Cornell Dir. at A93.

487. Even for those "essential structures" for which this reduction is in effect recovered, 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-STD-1020-94 PC-3 and PC-4 criteria, or the NRC SRPs. Cornell Dir. at A94; see also Ebbeson Dir. at A12. The net effect of the UBC design and acceptance criteria is a risk reduction ratio R_R of only 2 for essential buildings and structures, which is similar to that achieved by the IBC. Cornell Dir. at A94. By contrast, facilities designed to the NRC SRPs typically have risk reduction ratios of 5 to 20 or more. These differences represent a factor of 2.5 to 10 or more in increased conservatism (as measured by R_R) in the design procedures for nuclear facilities versus those in model building codes, even if the multiplier of two-thirds in the IBC-2000 is ignored. Cornell Dir. at A91. Thus,

the PFSF structures, even though designed using a lower MRP DBE than the starting point for determining the seismic ground motions under the IBC-2000 or UBC model building codes, would be much stronger and able to withstand greater ground motions than a structure designed to the ostensibly higher MRP DBE specified in IBC-2000

488. Thus, while the MRP DBE under the IBC-2000 is 25% larger than the proposed MRP for the PFSF, the more conservative design procedures and criteria of the ISFSIs SRP will ensure that the SSCs at the PFSF have a mean annual probability of failure that is several times (2 to 8 or more) lower than buildings designed to IBC-2000 standards. Moreover, all PFSF important-to-safety SSCs have risk reduction factors sufficient to provide a probability of failure of 10^{-4} or lower, i.e., at least two times lower than essential facilities designed to the IBC-2000. Additionally, as discussed earlier, a number of key important-to-safety SSCs in the PFSF have great robustness and/or fractional operating periods that reduce their probabilities of failure even further. Cornell Dir. at A92. Therefore, structures and components important to public health and safety at the PFSF would be much less likely to fail in an earthquake than would other facilities essential for public health and safety in the event of an earthquake, such as bridges, hospitals, fire stations, etc.

v) Section E, Basis 6(b)

489. In Basis 6(b), the State claims 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. As explained by Dr. Arabasz, this basis originated as a challenge to the Staff's logic set forth in the preliminary SER

that peak ground motion values corresponding to a 2,000-year return period earthquake were adequately conservative because they had a 99% probability of not being exceeded in the 20-year licensing period of the PFSF. Tr. 9183, 9190 (Arabasz). The Staff no longer asserts this rationale as a basis for approving the exemption. See Staff Exh. C. at 2-50, 2-51.

490. As explained by Dr. Cornell, hazards in virtually all areas of public safety are measured in terms of frequency of occurrence (e.g., measured in annual probabilities, in probabilities per 50-year period, or in per human lifetime units), and the same safety criteria are specified regardless of the length of the activity in question, the exposure time, the estimated facility life, or the licensing duration. Cornell Dir. at A94. The purpose of choosing annual risk as a basis for measuring hazard is to avoid logical inconsistencies that would arise from using lifetime risk. For example, under the lifetime risk approach an apartment building with a life of 10 years would be designed to a lesser protective standards (fire, seismic, etc.) than an apartment with a life span of 100 years. This would result in residents living in the "10-year" apartment being exposed to greater annual risk than those living in the "100-year" apartment. Tr. 8004-05 (Cornell). Similarly, for example, under a lifetime risk approach, older workers could logically be subject to greater risks than younger workers, which would lead to reduced work place protection standards for older workers, e.g., less protection against cancer-inducing activities (such as working with asbestos) or no shields around dangerous equipment, etc. Cornell Reb. at A3.

491. Dr. Arabasz in his testimony pointed to standards, such as the national seismic hazard maps, that depict probabilities in units such as 10%, 5% or 2% probability of exceedance in 50 years. Arabasz Dir. at A14-A15. However, as explained by

Dr. Cornell, stating probabilities of exceedance in terms such as a 10% probability of exceedance in 50 years (as opposed to an annual probability of exceedance of $2x10^{-3}$) is just a different way of presenting the frequency of occurrence. Cornell Reb. at A1. This is clearly reflected in Dr. Arabasz's quotation from the National Research Council's Panel on Seismic Hazard Analysis, which directly equates a design seismic hazard level with a 10% probability of exceedance in 50 years to an annual probability of exceedance of $2x10^{-3}$. Arabasz Dir. at 15. The important point is that neither frequency standard is predicated on the lifetime of a facility, nor does the application of the standard vary depending on a facility's projected lifetime. For example, applying a seismic standard of 10% probability of exceedance in 50 years to two buildings, one constructed for a 10-year lifetime and the second for a 100-year lifetime, respectively, would result in the same annual probability of exceedance of $2x10^{-3}$ for each building. Cornell Reb. at A1; see also Tr. 9195-98 (Arabasz).

492. Thus, none of the conventions that are in use for expressing the required seismic safety level are stated in terms that make this level dependent on the life of the building or facility Id. In fact, using a design return period proportional to the duration of the facility lifetime results in potential logical inconsistencies that make such an approach impractical. Cornell Reb. at A2; Tr. 10164-70 (Arabasz).

493. Dr. Arabasz acknowledges that he is not a risk expert and does not have a "firm basis" for saying that one should use an annual or lifetime basis for selecting the appropriate design level earthquake for the PFSF. Tr. 9191-93 (Arabasz). Further, Dr Arabasz agrees that under the DOE-1020 framework, which he generally favors, "the mean annual frequency" would be the "basis for determining the appropriate design basis earthquake," but he questions whether the NRC has a simi-

larly clearly established framework for decision-making based on annual frequencies. Tr. 10170 (Arabasz); see also Cornell Reb. at A2.

494. The NRC has adopted, however, annual frequency risk metrics as the basis for selecting the appropriate level of safety under its risk-informed regulatory framework. For example, both the Commission's Reactor Safety Policy Statement and Regulatory Guide 1.174 clearly set forth annual frequency-based risk acceptance guidelines for NPPs where the performance objectives are Core Damage Frequency and Early Large Release Frequency. While these guidelines are for NPPs, the same general risk-based principles employing frequency based risk metrics as opposed to life-time based risk metrics would apply to the PFSF.

495. Further, adoption of lifetime risk metrics would lead to inconsistent and illogical results. Under lifetime risk metrics, the annual level of risk would change depending on whether the PFSF was planned to be a 10, 20 or 40 year facility, which from a societal risk standpoint is inconsistent with the general risk principles enunciated above. For example, if the spent fuel were not stored at the PFSF it would be stored at another location with attendant risks associated with its storage there. The only way to make such decisions on a comparative risk basis is to use annual risk, and not lifetime risk, as the basis for decision. Further, use of a lifetime risk would raise practical issues on how the appropriate design basis earthquake should be determined in light of potential relicensing of a facility, or how relicensing might affect the already established seismic design basis of a facility. These are practical concerns further support the use frequency risk based metrics in determining the appropriate design basis earthquake for the PFSF. See Cornell Dir. at A94; Cornell Reb. at A2; Tr. 10164-70 (Arabasz).

4. Radiological Dose Consequences

a. Applicable Regulatory Standards for Radiological Dose Consequencess

496. Basis 2 of Section E of Contention Utah L/QQ asserts that "PFS has failed to show that its facility design will provide adequate protection against exceeding the section 72.104(a) dose limits."

497. 10 C.F.R. Section 72.104(a) provides that "[d]uring normal operations and anticipated occurrences, the annual dose equivalent to any real individual who is located beyond the controlled area must not exceed 0.25 mSv (25 mrem) to the whole body . . . ." Thus, notwithstanding the State's claim in Basis 2 of Section E, the radiological dose limits found in 10 C.F.R. § 72.104(a) are for "normal operations and anticipated occurrences," not for seismically–induced events.

498. A cask tipover during a seismic event is a beyond-design-basis accident for which the applicable dose limit is the 5 rem limit of 10 C.F.R. § 72.106(b). See Waters Dir. at A9, A11; Singh/Soler/Redmond Dir. at A14-A15; Tr. 12379 (Resnikoff). For this reason, the dose limits in 10 C.F.R. § 72.104(a) are not applicable to a cask tipover at the PFSF. See Waters Dir. at A11; Singh/Soler/Redmond Dir. at A12-A17; Tr. 12379 (Resnikoff).

499. All parties ultimately agreed that the radiological dose limits in 10 C.F.R. § 72.106(b) would apply to the consequences of a seismic event at the PFSF, not those in 10 C.F.R. § 72.104(a). Waters Dir. at A7; Singh/Soler/Redmond Dir. at A12-A17; Tr. 12379 (Resnikoff). 10 C.F.R. § 72.106(b) provides that "[a]ny individual located on or beyond the nearest boundary of the controlled area may not receive from any design basis accident the more limiting of a total effective dose equivalent of 0.05 Sv (5 rem) . . . ."

500. Although Dr. Resnikoff's prefiled direct testimony discussed the application of the dose limits of 10 C.F.R. § 72.104(a) to a cask tipover accident, the testimony reflected not Dr. Resnikoff's opinion as to the the relevant dose limits, but the directions of the State regarding how to present his testimony. Tr. 12376 (Resnikoff). Dr. Resnikoff testified that he did not believe that § 72.104(a) governs an accident involving a cask tipover during a seismic event, but that § 72.106(b) should apply instead. Tr. 12379 (Resnikoff).

501. Having conceded that 10 C.F.R. §72.106(b) is the controlling regulatory standard, the State raised for the first time during the course of the hearings an issue as to the duration of the postulated accident. This newly-raised issue then became the State's "$64,000.00 Question". Tr. 12367 (Curran). Counsel for the State represented (Tr. 12468) that there was testimony from Dr. Resnikoff that "the accident is a year." But no such testimony existed. Dr. Resnikoff's pre-filed testimony merely calculated dose rates on an annual basis and was silent on the duration of the postulated accident condition. See, e.g., Resnikoff Dir. at A23(b); State Exh. 141; State Exh. 143.

502. The applicable regulations in 10 C.F.R. § 72.106(b) do not place any express limit on the duration of an accident. Tr. 12600 (Resnikoff). There is no regulatory guidance directly on point regarding the duration of an accident for calculation of dose limits, although the NRC assumes a 30 day duration for some analyses, consistent with the loss of containment calculations for accident dose levels for Part 72 facilities described in NUREG-1567, Section 9.5.2. Staff Exh. 53; Tr. 12222 (Waters). Dr. Resnikoff testified that he would consider the dose limit to apply however long the accident condition lasted (Tr. 12600 (Resnikoff)), but did not know how an accident would be defined under NRC regulations or how long it

would last. Tr. 12506-08 (Resnikoff). As discussed below, regardless of the duration assumed for the postulated accident condition, the 5 rem limit of 10 C.F.R. § 72.106(b) will not be exceeded.

b. PFS's Evaluation of Radiological Dose Consequences Arising from a Beyond Design Basis Seismic Event

503. As discussed in Section D above, the analyses undertaken by Holtec and Sandia demonstrate that a cask will not experience any uplift during a design-basis earthquake. Likewise, cask displacements will be on the order of a few inches, precluding cask collision, even during conditions that will maximize sliding of the cask. Cask rotation will be small, with large margins of safety against tipover.

504. During the ground motions associated with a 10,000-year return period earthquake, the Holtec and Sandia analyses show that the casks will not tip over. Findings 132-148, 198-211. Likewise, uplift during such a beyond design basis seismic event was found to be on the order of fractions of an inch. Finding 211. Even under worst case assumptions, neither the Sandia nor the Holtec analyses showed cask-to-cask impacts resulting from sliding. Only in the Holtec simulations that intentionally tried to maximize cask displacements and cask rotations for a 10,000 year beyond-design-basis earthquake did any cask impacts (caused by cask precession or out-of-phase rotations) take place, and the simulations showed that those impacts occurred at relatively low speeds with no damage to the casks or loss of stability. Singh/Soler Dir. at A169; PFS Exh. OO. Even under those unrealistic conditions, maximum cask rotation was on the order of 10 to 12 degrees, representing a factor of safety against tipover of more than 2 when measured against the angle at which a cask would tip over as a result of its own moment. Fingding 135

505. Although it has been demonstrated that the casks will not tip over, PFS analyzed a non-mechanistic hypothetical tipover event in accordance with applicable regulatory guidance. Singh/Soler Dir. at A43; Waters Dir. at A15. The results of this analysis show that all stresses on the storage cask remain within the allowable values of the HI-STORM 100 System Certificate of Compliance ("HI-STORM CoC"), assuring the integrity of the MPC confinement boundary with large margins of safety. Singh/Soler/Redmond Dir. at A19. Therefore, there would be no releases of radioactivity even in the event a of a postulated tipover.

506. Holtec qualitatively evaluated the potential radiological consequences of a hypothetical cask tipover event in its Final Safety Analysis Report for the HI-STORM 100 System and determined that impact of the cask on the pad would only cause localized damage to the concrete and outer shell of the storage cask at the point of impact, reducing somewhat the roundness of the storage cask in the immediate area of impact. Singh/Soler/Redmond Dir. at A19, A38.

507. The HI-STORM 100 System storage cask consists of both a radial concrete shield and an outer steel shell. The concrete is fully encased in a steel structure, and four large steel ribs are located between the inner and outer shell. It is physically impossible for the concrete to be lost in the event of impact damage. A local deformation would not significantly affect the shielding performance of the storage cask, since the same mass of steel and concrete would still be present. Singh/Soler/Redmond Dir. at A38. Because radiation shielding is dependent on mass rather than thickness (Tr. 12479 (Resnikoff)), rearrangement of the mass present in the shielding will not result in significant changes in radiation dose levels, since loss of mass in one location of the cask will be offset by an increase in mass in another location. Tr. 12148-50 (Soler, Redmond); Tr. 12244 (Waters).

Additionally, the local deformations would occur at the top of the storage cask, whereas the radiation doses are greater at the middle of the cask. Tr. 12551-52, 12567-68 (Soler, Redmond). Therefore, any increase in the radiological dose levels due to localized deformation of the cask would at most be minimal. Singh/Soler/Redmond Dir. at A38.

508. Holtec also evaluated the radiological dose consequences resulting from the hypothetical tipover of multiple casks. Singh/Soler/Redmond Dir. at A20-A30. Hypothetical multiple cask tipovers would likely result in similar localized damage for each of the casks tipped over, with no significant aggregate effect on radiological doses at the owner-controlled area ("OCA") boundary. Singh/Soler/Redmond Dir. at A23, A26; Waters Dir. at A18, A19. The greatest potential for increase in radiological doses at the boundary would not be due to damage to the cask or the MPC, but to the possibility that the bottom of the cask, which has less radiation shielding, might face the OCA boundary. Singh/Soler/Redmond Dir. at A23; A26; Waters Dir. at A21; Resnikoff Dir. at A20.

509. Holtec evaluated the effect that 4,000 tipped-over casks would have on the radiation dose at the OCA boundary, compared to the doses due to releases from the casks in their normal upright position. In the upright position, the side of the storage cask is in a direct line of sight from all equidistant locations from the cask, the top is not visible from any location, and the bottom is shielded by the ground. In a tipped-over position, the top or bottom of the cask would be visible from some locations and not from others, while the side of the storage cask cylinder (now horizontal) would also be visible from some locations and not others. Additionally, since the storage cask would be lying on its side, a large portion of the outer radial surface of the cask would be shielded by the ground. From its evaluation of

the geometry of the storage cask Holtec concluded that, overall, the decrease in dose rate from the sides of a tipped-over storage cask should more than compensate for the increase in dose rate from the top or bottom of the cask. Further, in the event of multiple casks tipping over, the orientation of the tipped-over casks would be random and the bottoms and tops of many of the casks would be shielded from the OCA boundary by other casks. Singh/Soler/Redmond Dir. at A23-A26.

510. Thus, in the event of a beyond-design-basis accident that caused the tipover of all, or a significant portion of the 4,000 casks at the PFSF site, the radiological dose levels at the OCA boundary would not be increased from the 5.85 mrem per year for normal operations which had previously been calculated. Thus, there are approximately three orders of magnitude of margin between the expected dose rate at the OCA boundary for 4,000 casks in a tipped-over condition compared to the 5 rem accident dose limit in 10 C.F.R. § 72.106(b). Singh/Soler/Redmond Dir. at A27-A28.

511. In addition, many conservatisms were included in PFS's calculation of the 5.85 mrem/year dose at the OCA boundary. These included:

- The calculation assumed that all 4,000 casks contain fuel with a burnup of 40,000 MWD/MTU and a cooling time of 10 years. This is physically impossible, since the MPCs will be delivered over many years and each additional year of cooling further reduces the radiation source term. A more realistic value of 35,000 MWD/MTU and a cooling time of 20 years has been used in other PFS analyses. These more realistic assumptions result in a greater than 50% reduction in the calculated normal doses at the site boundary, from 5.85 mrem/year to 2.10 mrem/year.

- The calculation assumed that the fuel assemblies inside the casks have the highest gamma and neutron radiation source term in all fuel storage locations, maximizing radiological doses.

- The calculation assumed that the fuel has been subject to a single irradiation cycle in calculating the source term. This ignores the down time during reactor operations for scheduled maintenance and refueling, which would reduce the source term by effectively increasing the cooling time.

Using more realistic assumptions would significantly reduce the calculated radiological dose levels, further decreasing the expected radiation dose consequences of the hypothetical tip over of all 4,000 casks at the PFSF. Singh/Soler/Redmond Dir. at A28.

c. *State Challenges Based on Differences Between the HI-STORM 100 Certificate of Compliance and the PFSF Design Basis Analysis for the HI-STORM 100 Storage Cask*

512. In his prefiled testimony, Dr. Resnikoff noted that there were differences between the HI-STORM CoC and site-specific conditions at the PFSF, and asserted that these differences resulted in a failure of PFS to accurately quantify the consequences of a design basis earthquake at the PFSF. Resnikoff Dir. at A9. Dr. Resnikoff cited three differences between the HI-STORM CoC and the PFSF conditions: differences in ground motion, occupancy time, and the thirty-three hour corrective action time limit in the event of a 100% air inlet duct blockage of storage casks. Resnikoff Dir. at A9 and A22.[39]

513. Dr. Resnikoff's testimony was apparently premised on the assumption that the HI-STORM CoC is supposed to reflect the "fact and conditions" at the PFSF site. Resnikoff Dir. at A8. This assumption is clearly incorrect. Holtec performed general design analyses in its FSAR for the HI-STORM 100 System storage cask, which support the HI-STORM CoC. Singh/Soler/Redmond Dir. at A31. Under

[39] Dr. Resnikoff also asserted that because all the casks could tip over at the PFSF, PFS needed to calculate the dose consequences due to the tipover of an entire field of casks. Id. As discussed above, if such an event were to occur, the dose consequences would be far below the 5 rem limit.

the HI-STORM CoC, nuclear power plant licensees may use the HI-STORM system at their sites under the general license provision of 10 C.F.R. § 72.210 as long as they meet the conditions of both 10 C.F.R. § 72.212 and the CoC. Singh/Soler/Redmond Dir. at A31.

514. However, satisfactory performance of the HI-STORM system may also be demonstrated by site-specific analyses. Holtec has performed such site specific analyses for the PFSF, demonstrating satisfactory performance of the system at the PFSF. Singh/Soler/Redmond Dir. at A31. Thus, the differences claimed by Dr. Resnikoff to exist between the HI-STORM CoC and the PFSF are irrelevant.

i) Design Basis Ground Motion

515. The design-basis ground motion for the PFSF is 0.711g in the horizontal direction and 0.695g in the vertical direction. These values exceed the ground motion limits in the HI-STORM CoC. Resnikoff Dir. at A8a; Singh/Soler/Redmond Dir. at A34. However, Holtec conducted site-specific cask tipover dynamic analyses for the PFSF which demonstrate that the casks do not tip over under the PFSF design basis ground motions, or even under ground motions due to a 10,000-year beyond–design-basis earthquake. See Singh/Soler/Redmond Dir. at A34. Thus, the variance between the ground motions for the PFSF DBE and the analyses supporting the HI-STORM CoC has no significance. Id.; Tr. 12435-36 (Resnikoff).

ii) Occupancy Time

516. The PFS site-specific analysis for radiation dose levels uses a 2,000 hours/year occupancy time for calculating normal operating dose levels (conservatively based on an assumed worker at the site boundary 40 hours a week for 50 weeks a year), whereas the HI-STORM CoC uses 8,760 hours/year to calculate the normal operating dose. Singh/Soler/Redmond Dir. at A29, A32. The dose limits estab-

lished by 10 C.F.R. § 72.104(a) apply to "any real individual who is located beyond the controlled area," not to a hypothetical person at the OCA boundary. Thus, occupancy time for normal operating conditions is determined using a real person standard, which takes into account the site-specific circumstances at a facility. Singh/Soler/Redmond Dir. at A29. This interpretation is endorsed by Staff Regulatory guidance. Id.; see also, Tr. 12067 (Redmond). Likewise, for accident conditions, the 5 rem limit would apply to real individuals, and site-specific circumstances would similarly need to be taken into account, including any remedial measures that may be taken during extended accident conditions (e.g., shielding or moving persons away from OCA boundary). See, Tr. 12072 (Redmond); Tr. 12266-67 (Waters).

517. The PFSF has a buffer zone of two miles on the southern side and a buffer zone of nearly a mile on the eastern side that preclude an individual from being present at the OCA boundary twenty-four hours a day. Tr. 12561-64 (Donnell). The land to the west of the PFSF is owned by the Bureau of Land Management and is used for grazing. Tr. 12564-65 (Donnell). The land immediately to the north of the PFSF is privately owned and used for livestock grazing with concomitantly low expected human occupancy time. Tr. 12564-65 (Donnell). The nearest offsite residence to the PFSF is located over two miles away from the OCA boundary, with intervening high ground blocking any line of sight. Tr. 12557-58 (Redmond); Tr. 12571-72, 12578-79 (Donnell). No witnesses for any party testified as to any plans to change existing land uses surrounding the PFSF. Changes in existing land use are prohibited in the buffer zone surrounding the PFSF. Tr. 12562 (Donnell).

518. Based on the land use surrounding the PFSF, the assumed 2,000 hours per year occupancy time is conservatively high. Tr. 12067-68 (Redmond). The only individuals likely to be present at the OCA boundary would be workers, who are assumed to be present 40 hours a week for 50 weeks a year to produce an upper bound of 2,000 hours per year exposure at the site boundary. Singh/Soler/Redmond Dir. at A29; PFSF SAR §7.3.3.5.

519. Thus, using a conservatively high 2,000 hours/year occupancy time is appropriate for normal operations, given the site-specific circumstances at the PFSF. Singh/Soler/Redmond Dir. at A29; see also Tr. 12263-65 (Waters). Such an occupancy time would also be conservatively high for postulated accident conditions. Tr. 12266-67 (Waters). In addition to measures to limit occupancy of areas of potential radioactive contamination, remedial measures, such as the construction of an earthen berm, could easily be undertaken to assure that radiological dose levels at the boundary of the OCA do not exceed regulatory limits following a beyond-design-basis earthquake. See Tr. 12583-84 (Donnell); Tr. 12622-23 (Resnikoff).

iii) Relevance of Thirty-Three Hour Time Period for Corrective Action of Complete Air Inlet Blockage Under HI-STORM CoC

520. The thermal analysis used to support the HI-STORM CoC provides that in the event of a 100% blockage of the air inlet ducts, the short term temperature limit of the concrete would be expected to be reached in thirty-three hours. Staff Exhibit FF. The thirty-three hour period for correcting a 100% air duct blockage was based on the requirement that the casks be visually inspected every twenty-four hours, allowing an additional eight hours for corrective action to be taken. Tr. 12152 (Singh). The thermal analysis that was used in the HI-STORM CoC makes

the conservative (but unrealistic) assumption that no heat transfer to the surrounding air will occur. In effect, the calculation presumes that the cask not only has its air inlet ducts completely blocked, but that it is shrouded in a "heavy blanket" that prevents any heat transfer. Tr. 12152-53 (Singh). Only under those extreme conditions would the short-term temperature limit of the concrete be reached in thirty-three hours. Id.

521. It is physically impossible for all air inlet ducts of a cask to be blocked due to a cask tip-over. Singh/Soler/Redmond Dir. at A51. Even in a tipped-over condition, heat transfer continues to take place and the air inlet ducts continue to dissipate heat, thus concrete temperature would be expected to remain below the short term limit. Singh/Soler/Redmond Dir. at A53; see also, Tr. 12152-54 (Singh).

522. Further, even assuming all vents were blocked, the bounding steady state temperature for the concrete would be well below the 600°F necessary for extensive sustained water evaporation. Singh/Soler/Redmond Dir. at A53; see also, Tr. 12153-54 (Singh). Both conduction and radiation of heat still occur from a storage cask that has all its air inlet ducts blocked. Tr. 12300-01 (Waters). Therefore, the evaporation of water from the concrete of a tipped-over cask would be minimal, even if the cask remained in a tipped-over position for a period of months. Singh/Soler/Redmond Dir. at A53.

523. Exceedance of the short-term temperature limit of the concrete does not affect public health and safety because it (1) has no effect whatsoever on the containment of the spent fuel within the storage cask; and (2) there would be no significant reduction in the shielding effectiveness of the system. Tr. 12154-55 (Singh); see also, Tr. 12440-41 (Resnikoff).

d. State Challenges to PFS's Evaluation of Cask Damage

524. Dr. Resnikoff identified three possible mechanisms by which damage might occur to a HI-STORM 100 System storage cask during a design basis seismic event: cask tipover, sliding and impact, and uplift. Tr. 12381-83 (Resnikoff). He acknowledged that all the mechanisms that he postulated are based entirely on the Altran Report and the testimony of State witnesses Khan, Ostadan, and Bartlett and, despite language to the contrary in his prefiled testimony, Dr. Resnikoff does not have any independent basis or expertise for assessing whether any of these mechanisms will occur or to what extent. Tr. 12381-85, 12394-98 (Resnikoff). Rather, Dr. Resnikoff presumed a cask tipover (Tr. 12402 (Resnikoff)), despite the fact that the Altran Report did not conclude that such a tipover would occur, and neither did Dr. Khan. See Tr. 12469-73 (Resnikoff). In fact, no State witness has testified that sliding and collision of the casks, tipping of the casks, or uplift of the casks would occur to such an extent as to cause cask tipover.

525. Dr. Resnikoff conceded that he did not know whether a cask impact due to a beyond-design-basis seismic accident at the PFSF would cause flattening or other damage to the storage cask (Tr. 12406 (Resnikoff)), whether or how much cracking of the steel or concrete would occur (Tr. 12407-08 (Resnikoff)), or whether or how much thinning of the steel would occur (Tr. 12406 (Resnikoff)). Dr. Resnikoff acknowledged that the State's allegations relating to damage to the cask, including all the mechanisms postulated in his testimony, were theoretical concerns and that he did not have expertise to determine whether or to what extent they could occur. Tr. 12413-18 (Resnikoff). Nor had he attempted to estimate any effect on radiation doses arising from any postulated damage to the casks. Tr. 12414 (Resnikoff).

526. Dr. Resnikoff speculated that it may be possible for the deformation of a fallen cask to be in a location on the storage cask different than the Holtec analysis suggests due to one cask falling onto another cask, or from some other seismically-induced cask-to-cask interaction. Tr. 12599 (Resnikoff). Dr. Resnikoff, however, did not know whether it is physically possible for one cask to fall on top of another prone cask (Tr. 12613 (Resnikoff)), had no detailed knowledge of the behavior of the casks during a seismic event (Tr. 12613 (Resnikoff)), and had no knowledge of how the casks might interact from a structural engineering standpoint (Tr. 12613 (Resnikoff)).

527. Dr. Resnikoff also acknowledged that he had neither experience nor expertise in measuring or quantifying concrete cracking (PFS Exh. 240 at 42-45, 47, 71), determining the strength of steel or concrete (PFS Exh. 240 at 46), calculating the initial angular velocity of a cask during tipover (PFS Exh. 240 at 70-71; Tr. 12403-04 (Resnikoff)), or measuring or quantifying thinning or flattening of the steel in the cask shell due to impact (PFS Exh. 240 at 80-81). No State witness has provided testimony concerning whether or how much a cask impact from uplift, sliding and collision, or tipover due to a postulated cask tipover event at the PFSF would cause: (1) flattening or other damage to the storage cask, (2) cracking of the steel or concrete, (3) thinning of the steel shell or radial concrete shield, or (4) displacement of the cask lid. Neither has any State witness quantified the effects of any of those mechanisms.

528. Dr. Resnikoff further admitted that he had no background or experience in cask stability analyses (Tr. 12397-98 (Resnikoff)), had not conducted cask stability analyses for the PFSF (Tr. 12396-98 (Resnikoff)), had no knowledge of the behavior of the storage casks from a structural engineering perspective (Tr. 12614

(Resnikoff)), had never modeled or reviewed a simulation of a storage cask drop outside of this case (Tr. 12398-99 (Resnikoff)), and did not know how to evaluate whether a cask lid displacement would occur during tipover (see Tr. 12414-17 (Resnikoff)).

529. Despite this lack of expertise, Dr. Resnikoff testified to three specific concerns that he had with the Holtec cask tipover analysis: (1) the potential unconservativeness of Holtec's assumption of zero initial angular velocity; (2) a related concern that deceleration at the top of the storage cask might exceed 45g; and (3) Holtec's asserted failure to account for the dynamic impulse resulting from displacement of the cask lid upon impact in a tip-over event. Resnikoff Dir. at A16, A21; Tr. 12403 (Resnikoff).

i) Initial Angular Velocity

530. Based on the Altran Report, Dr. Resnikoff postulated that the Holtec analysis of cask tipover was inadequate because the initial angular velocity of a falling cask may be greater than zero. However, Dr. Resnikoff has never calculated an initial angular velocity for any storage cask tipover PFS Exh. 240 at 70-71), nor did he have the expertise to do so. Tr. 12403-04 (Resnikoff). Instead, Dr. Resnikoff testified that he asked "[the State's] other experts what is the angular velocity and is zero correct, and their opinion [was] that the zero initial angular velocity could be greater than zero." Tr. 12403 (Resnikoff).

531. There is no testimony by any State witness that supports the conclusion that an initial angular velocity greater than zero would be either realistic or more appropriate for a cask tipover at the PFSF. State soils expert Dr. Bartlett summarily asserted, in reference to the Holtec non-mechanistic cask tipover analysis, that "the tipover event postulated that the cask would be perched on its edge with zero an-

gular velocity. During an earthquake, that's not true. If we go to tipover, we have some angular velocity." Tr. 12870-71 (Bartlett). However, Dr. Bartlett admitted that he had not been involved in any calculations of cask stability or the results of a tipover event (Tr. 12870 (Bartlett)), and there is no evidence that he has expertise to perform such an analysis.

532. The Holtec analyses of dynamic cask behavior have shown that the behavior of the cask is characterized by tilting from the vertical resulting in a plane of precession for a certain duration in the course of the earthquake event, resulting in an oscillatory rocking motion with limited return to the vertical position until the rocking finally ends when the earthquake subsides. Singh/Soler/Redmond Dir. at A39. If the earthquake ground motions were assumed to be increased to the point at which a cask would tip over, the initiating angular velocity propelling the cask towards the ground would be quite small. Singh/Soler/Redmond Dir. at A39.

533. Furthermore, the precessionary motion of the cask enables it to remain stable after the center of gravity of the cask is well past the "center-of-gravity-over-corner" position. As a result of this precessionary motion, the location of the cask's center of gravity is likely to be much lower than in the static tipover scenario (where tipover begins as soon as the center of gravity crosses the vertical plane containing the axis of overturning rotation). The combination of a shorter distance to fall and a negligible initial angular velocity propelling the tip-over further supports the assumption of an initial angular velocity of zero because a cask tipping away from precessionary motion is expected to have substantially less kinetic energy of collision than one tipping from a zero velocity with the center of gravity over corner. Singh/Soler/Redmond Dir. at A39. Thus, the assumption of an initial angular velocity of zero is appropriate.

ii) Deceleration in Excess of 45g and Design Margin of the MPC

534. Dr. Resnikoff's pre-filed testimony indicated that his concern regarding the possibility of the top of the cask decelerating at a rate in excess of 45g was premised on the initial angular velocity being greater than zero. See Tr. 12410-12 (Resnikoff). He changed his testimony at the hearing and acknowledged that damage to the cladding on fuel rods contained in the fuel assemblies within the storage cask would not be an issue unless the assemblies were subjected to an acceleration of at least 63g. Tr. 12409-10 (Resnikoff). Dr. Resnikoff did not know how large an initial angular velocity would be required to exceed the 63g limit, but conceded that an initial angular velocity of greater than zero would be required. Tr. 12411-12 (Resnikoff).

535. The HI-STORM 100 FSAR places a 45g limit on the deceleration for the top of the HI-STORM 100 storage cask in the event of a cask tipover event. This is a licensing limit that does not represent the actual ability of the storage cask, the MPC, or the fuel assemblies to maintain both containment and radiation shielding. Singh/Soler/Redmond Dir. at A40; Tr. 12158 (Singh). The spent fuel assemblies have design margins that allow them to withstand accelerations up to at least 63g. Singh/Soler/Redmond Dir. at A40; Tr. 12409-11 (Resnikoff); Tr. 12158 (Singh). There has been no analysis of postulated beyond-design-basis accidents that resulted in decelerations greater than the 45g limit in the HI-STORM 100 FSAR, let alone the 63g design limit. Tr. 12411 (Resnikoff).

536. The MPC also has substantial design margins beyond the 45g level. A hypothetical 25 foot end drop of a loaded canister on a hard concrete foundation resulted in a computed strain in the confinement boundary of 41% of the failure strain limits for the canister material. Singh/Soler/Redmond Dir. at A40. The computed strain

showed that the MPC could experience a maximum deceleration of 300g without loss of confinement. Tr. 12075 (Singh).

537. Thus, exceeding the 45g deceleration limit imposed on the top of the canister in the HI-STORM 100 FSAR would not result in increased radiological dose consequences. Decelerations would have to exceed 63g before there was a concern regarding the possible effect of such decelerations on the fuel assemblies contained in the MPC. Tr. 12409-11 (Resnikoff). Moreover, due to the large margins of safety built into the design of the MPC, much larger decelerations than 45g would be required before the containment function of the MPC was compromised. Singh/Soler/Redmond Dir. at A40; Tr. 12158 (Singh).

iii) Cask Lid Displacement

538. In his prefiled direct testimony, Dr. Resnikoff posited that tipover could cause additional "dynamic impulses" to the structure of a cask. He described his concerns as follows:

> In a tipover event, discussed in TSAR Appendix 3.B, the cask walls at the top of the cask are expected to flatten slightly (0.11 inch, p. 3.B-5) when the cask top strikes the ground. On the other hand, the cask lid plate is expected to be displaced as much as 4.9 inches in a tip over event (TSAR, p. 3.A-15). This indicates to me that the 3 ¾ inch thick lid plate is going to strike the ground in a tipover event and send a strong dynamic impulse to the cask wall and canister. It does not appear that this cask detail, that may affect the canister welds, has been modeled.

Resnikoff Dir. at A21. (Footnote omitted.)

539. Dr. Resnikoff's testimony misinterpreted the results of the HI-STORM cask tipover analysis in several significant respects. <u>First</u>, Dr. Resnikoff incorrectly assumed that the displacement reported in the TSAR is a displacement of the cask lid relative to the cask body. Tr. 12549-50 (Soler). In fact, the cask lid and the

cask body move together, not relative to one another, so that the 4.9 inches of displacement applies to both the cask lid and the cask body. Tr. 12550-51 (Soler). Second, Dr. Resnikoff mistakenly assumed that any dynamic forces due to the displacement of the cask lid and cask body are not adequately taken into account in the Holtec analysis, when in fact any dynamic forces due to the impact of the cask lid or body are included in the modeled behavior. Tr. 12551 (Soler). Third, the effect on the canister welds of any such forces are considered in the tipover model and no deleterious effects to the welds occur during a hypothetical tipover event. See Tr. 12551 (Soler). Fourth, to the extent that damage to a cask could hypothetically be caused by a tipover, the analysis demonstrates that any deformations would be small, localized, and would occur within one foot of the top of the cask, where radiation dose consequences are the least significant. Tr. 12551-52 (Soler, Redmond). Thus, Dr. Resnikoff's concern regarding cask lid displacement is unrealistic.

e. State Estimation of Radiological Dose Consequences of a Worst Case, Beyond-Design-Basis-Accident at the PFSF

540. Dr. Resnikoff's prefiled testimony contained two radiation dose calculations: an estimation of the gamma dose from the bottom of eighty prone storage casks, with their bottoms facing the OCA boundary (State Exh. 141), and an estimation of the neutron dose from a cask based on the amount of "water evaporated" from the concrete shielding (State Exh. 143). Beginning with amended State Exh. 141A, Dr. Resnikoff combined both scenarios – cask tip over and loss of hydrogen shielding – to portray a total, worst case radiological dose at the OCA boundary. Both original calculations (as well as the subsequently amended overall dose cal-

culation, State Exh. 141A) contained so many errors that these calculations cannot be given any weight.

541. The dose exposure that Dr. Resnikoff ultimately calculated at the OCA boundary was less than 150 mrem for the first year, assuming a hypothetical person were at the OCA boundary for the entire year (which, as discussed above, is not realistic). Radioactive decay would reduce this dose exposure in subsequent years. Thus, assuming that the casks remained on the ground indefinitely with no remedial actions being taken, the 5 rem limit would never be exceeded for a person continuously stationed at the OCA boundary. Tr. 12619-20 (Resnikoff).

i) Neutron Dose Calculation

542. Dr. Resnikoff's neutron dose calculation, State Exh. 143, purports represent the "increased neutron dose due to reduced shielding" in order to estimate "the increase in dose to workers due to neutrons . . . 1 meter from the cask mid-height if all of the water evaporates from a HI-STORM cask." Resnikoff Dir. at A23(b). In this calculation, Dr. Resnikoff assumed that there is some unspecified temperature at which no hydrogen is present in the concrete or the aggregate material contained in the concrete. Tr. 12420-23 (Resnikoff). Dr. Resnikoff did not try to calculate the actual amount of hydrogen loss that would take place if a HI-STORM 100 cask tipped over, nor did he have any idea how to calculate the thermal degradation of the cask's concrete over time (PFS Exh. 240 at 90-93); nor had he ever used computer programs that computed the temperature of concrete over time (Id.). He also did not know how to estimate the reduction in shielding due to concrete heating up over time (Id. at 93). Indeed, this was his first attempt to examine thermal degradation in concrete and quantify the loss of radiation shielding that may result. Tr. 12418-19 (Resnikoff). Dr. Resnikoff was also not

aware of the actual physics of hydrogen evaporation from concrete when he made his calculations. Tr. 12422 (Resnikoff).

543. The premise of Dr. Resnikoff's calculation of the lack of any hydrogen in the concrete due to evaporation of water is unrealistic. It is not easy to evaporate water within concrete, because it is in a confined space, and as the water evaporates the air pressure increases. In turn, the increased air pressure will convert the water vapor back to liquid. Likewise, concrete does not lose its moisture content as easily as water might evaporate from a free surface. In order for large, extensive, sustained water evaporation from the concrete to occur, exposure to high temperatures for a period of months will be necessary. Moreover, it is physically impossible for cask heat-up to release hydrogen contained in the aggregate within the concrete. Singh/Soler/Redmond Dir. at A53; Waters Dir. at A20. In an actual simulation of the worst case scenario for heat degradation of the HI-STORM 100 cask, the Staff indicated that neutron dose rates due to thermal degradation would result in a much smaller increase of computed neutron dose rates than those predicted using the unrealistic assumptions in Dr. Resnikoff's analysis. Waters Dir. at A20. In addition to the erroneous assumptions made by Dr. Resnikoff, his neutron dose calculation was also in error because he used the wrong neutron dose from the SAR, which inflated his calculated neutron dose by a factor of 2.68. Tr. 12607-08 (Resnikoff).

ii) Gamma Dose and Overall Dose Calculation at the OCA Boundary – St. Exh. 141 and 141A

544. Dr. Resnikoff's gamma dose calculation at the OCA boundary was premised on the bottoms of eighty prone storage casks lined up in a row all facing the OCA boundary. St. Exh. 141 at 3-5, 6-8. Such an arrangement is "highly unrealistic."

Singh/Soler/Redmond Dir. at A24-A26, A44-A53. Further, Dr. Resnikoff made numerous errors in his calculation. After correcting these errors, the 5 rem accident dose limit would never be reached even under the unrealistic conditions assumed in the calculation.

545. Dr. Resnikoff made a total of nine different corrections or changes to his overall dose calculation at four different points in the proceeding. These errors are identified in the testimony of the PFS witness by Dr. Redmond as well as by Dr. Resnikoff in the amendments to his pre-field direct testimony and in oral testimony at the hearing. See e.g., Singh/Soler/Redmond Dir. at A46; Tr. 12428-30 (Resnikoff); State Exh. 141A; Tr. 12374-75 (Resnikoff).

546. It would serve no useful purpose to recite the details of the various errors in Dr. Resnikoff's dose rate calculations. Suffice it to say that they leave this Board with little confidence in the accuracy of his analyses. Even after making several of these corrections to his testimony, Dr. Resnikoff testified that he was "pretty confident" that there were no additional errors in his calculation. Tr. 12430-32 (Resnikoff). Yet additional errors were identified in the course of his examination which required him to make additional adjustments (downwards) to his results. See Tr. 12432, 12503, 12607-08 (Resnikoff).

547. A particularly egregious error in Dr. Resnikoff's dose calculations is that he did not consider the effect of radioactive decay. The majority of the gamma radiation from the spent nuclear fuel comes from the radioactive decay of Cobalt-60 and Cesium-137, with Cobalt-60 being the main gamma emitter for radiation emanating off the bottom of the cask, accounting for ninety percent of the total gamma dose calculated by Dr. Resnikoff. Tr. 12619-20, 12624-25 (Resnikoff). Although the half-life or Cobalt-60 is approximately five years, Dr. Resnikoff neglected to

take radioactive decay into account when arriving at his dose estimates. Tr. 12617-20 (Resnikoff); State Exh. 141, State Exh. 141A.

548. Taking into account only the radioactive decay of just the Cobalt-60 and ignoring the decay of other radioisotopes will result in a total radiation dose over fifty years of 2582.1 mrem, or 2.58 rem.[40] In fact, as Dr. Resnikoff admitted, taking into account radioactive decay, the 5 rem accident limit specified in 10 C.F.R. § 72.106(b) is never reached (Tr. 12620 (Resnikoff)) no matter how long one assumes that the casks remain in a worst case tipover and total loss of hydrogen shielding condition, and disregarding any remedial actions that might be taken in the intervening period by PFS or others.

f. Duration of Accident

549. Upon the State's identification of accident duration as the "$64,000 question," Dr. Resnikoff attempted to testify as to accident duration. Dr. Resnikoff had no idea, however, how long a seismically induced accident condition might exist at the PFSF, indicating only that he was concerned about casks being tipped over for years. Tr. 12440-41, 12507-08 (Resnikoff). The longest duration postulated by Dr. Resnikoff was forty years. Tr. 1257-08 (Resnikoff).

550. Dr. Resnikoff testified that he has no experience with estimating the length of time it would take to correct a seismically induced accident at an ISFSI, nor did he have any knowledge about how long it would take and had not undertaken any analyses to determine what kind of accident durations might occur at the PFSF in

[40] This number is obtained from the data generated by Dr. Resnikoff as follows: 962.1 millirem (cumulative gamma dose from decay of Cobalt-60) + 1068.5 millirem (cumulative gamma dose from Cesium-137 assuming no decay from St. Exh 141A times 50 years) + 551.5 millirem (cumulative neutron dose assuming no radioactive decay from St. Exh 141A times 50 years) = 2582.10 millirem.

the event of a beyond-design-basis accident involving the seismically-induced tipover of storage casks. Tr. 12507-09, 12614-16 (Resnikoff).

551. Further, although he testified that occupational dose limits may prolong the duration of an accident (Tr. 12607 (Resnikoff)), he acknowledged that mitigation measures such use of shielding, can be taken to minimize worker exposure in the event of a beyond design basis accident. Tr. 12607; see also, Singh/Soler/Redmond Dir. at A56. The radiological dose levels for such a beyond design basis accident at the PFSF are lower than radiation dose levels workers in nuclear facilities routinely experience. Singh/Soler/Redmond Dir. at A55.

552. Even assuming a physically impossible, worst case cask tipover and loss of all hydrogen shielding event as postulated by the State, the 5 rem radiological dose limits set by 10 CFR Section 72.106(b) will not be exceeded within at least 50 years of a beyond design basis seismically induced accident. Tr. 12619-20 (Resnikoff). Indeed, the radiation doses resulting from any postulated tipover accident would never reach the regulatory limits no matter how long the accident was assumed to extend, hence the accident duration is not a meaningful parameter for purposes of our decision.

553. The nearest resident to the PFSF is two and a half miles away from the OCA boundary, separated by high ground blocking any line of site (Tr. 12557-58 (Redmond); Tr. 12571-72, 12578-79 (Donnell)), and no changes in land use surrounding the PFSF are planned, and in some cases prohibited. See Tr. 12562 (Donnell). Moreover, remedial actions to lower radiological dose consequences at the OCA boundary, such as the construction of an earthen berm, can easily be taken to assure that radiological dose levels at the boundary of the OCA do not exceed regulatory limits. See Tr. 12583-84 (Donnell); Tr. 12622-23 (Resnikoff).

Therefore, even if concerns remained about potential offsite radiation doses following a beyond-design-basis seismic event, such concerns would be readily alleviated by a number of remedial actions which could, and this Board can reasonably assume would, be taken following the event.

5. SECTION E CONCLUSION

554. The PFS seismic exemption is consistent with well understood and widely accepted risk-graded principles. All parties agreed that the two-handed approach employed – using both the return period of the DBE and the conservatisms of the design procedures and criteria – to determine if a performance goal were met was an appropriate methodology. Likewise, the State agreed that a MAPE of 1×10^{-4} was an appropriate performance goal for the PFSF ISFSI, using a risk-graded approach that takes into account the consequences of the failure of an SSC at an ISFSI

555. The record in the proceeding demonstrates that considerable margins exist in the seismic design of the PFSF due to the design procedures and criteria built into the SRPs and confirmed through numerous seismic PRAs. These SRP design margins apply to all SSCs within the CTB, the CTB structure, and the foundations of both the CTB and the storage pads. In addition, Applicant's witnesses have testified to numerous margins that exist in the design of SSCs within the CTB, the CTB structure, and the foundations of both the CTB and the storage pads.that would enable them to withstand earthquakes with return periods on the order of 10,000 years.

556. Further, with respect to the the HI-STORM 100 storage cask system, cask stability analyses performed by Holtec and Sandia show that the casks will not tip-over

even under a 10,000 year earthquake event with significant excess margin remaining. In addition, even if the casks were to tip-over no breach of the MPC confinement boundary would occur. Significant margins exist with respect to the integrity of the MPC confinement boundary.

557. The State's witnesses agree that if the margins testified to in this proceeding are correct, that the PFSF will meet or exceed the intended performance goal of 1×10^{-4} and that the granting of the exemption would be appropriate. We find that the margins are more than sufficient to enable SSCs at the PFSF to withstand without failure earthquakes with return periods on the order of 10,000 years.

558. Further, ~~be~~ the ultimate issue with respect to the granting of the seismic exemption is whether the facility ~~design~~ design will provide reasonable assurance of protecting public health and safety. As the record indicates, all SSCs at the PFSF and the storage casks themselves meet or exceed performance goals sufficient to protect public health and safety. Under no postulated circumstances would the consequences of a beyond-design-basis accident endanger public health and safety. Even counterfactually assuming that the margins either did not exist or a seismic event sufficiently exceeding the performance goal were to occur, such that a worst-case cask tipover and total loss of hydrogen shielding beyond-design-basis accident would occur, there would be no exceedance of the applicable radiological dose limits. Therefore, there is reasonable assurance that the public health and safety would be protected.