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

APPLICANT'S PREFACE OF THE TESTIMONY OF DONALD WAYNE LEWIS ON SECTION E OF UNIFIED CONTENTION UTAH L/QQ

I. WITNESS

Donald Wayne Lewis is employed by S&W as the Lead Mechanical Engineer for the PFSF project, a position he has held since 1996. He received his undergraduate engineering degree from the Montana State University, majoring in Civil/Structural Engineering. Mr. Lewis has 19 years of experience in the nuclear power industry, including 10 years of experience with the design, licensing, construction, and operation of independent spent fuel storage installations (ISFSIs). He is a registered professional engineer in the states of New York, Colorado, Utah, Iowa, and Maine. Mr. Lewis' technical contribution to the PFS project focuses on the mechanical aspects of ISFSI work, including cask handling and transportation equipment and operations, building services (HVAC, plumbing, etc.), and fire protection. For the PFS project, he is also responsible for the preparation of the principal design criteria, design installation, and operating systems portions of the PFSF Safety Analysis Report.

II. TESTIMONY

Mr. Lewis will testify regarding the process for transferring spent fuel canisters from the shipping casks in which they arrive at the PFSF to the storage casks, and describe the methodology for calculating the time involved in canister transfer operations. He will also testify as to the the safety classification of the structures, systems and components ("SSCs") relevant to Unified Contention Utah L/QQ. The purpose of Mr. Lewis' testimony is to show that the transfer of the spent fuel from shipping to storage casks at the PFSF will occur during a small fraction of time that the facility is in operation. Likewise, Mr. Lewis will show that throughout such operations drops and tipovers of the fuel canister or shipping, transfer, and storage casks themselves is precluded by the safety measures taken during transfer operations.

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TESTIMOMY OF DONALD WAYNE LEWIS ON SECTION E OF UNIFIED CONTENTION UTAH L/QQ

I. WITNESS

Q1. Please state your full name.

A1. Donald Wayne Lewis.

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

A2. I am currently employed by Stone & Webster, Inc., a Shaw Group Company, as the Lead Mechanical Engineer for the PFSF project. I have held this position since 1996.

Q3. Please summarize your educational and professional qualifications.

A3. My professional and educational experience is summarized in the Curriculum Vitae attached to this testimony. As indicated there, I have 19 years of experience in the nuclear power industry, including 10 years of experience with the design, licensing, construction, and operation of independent spent fuel storage installations ("ISFSIs"). My technical contribution focuses on the mechanical aspects of ISFSI work, including cask handling and transportation equipment and operations, building services (HVAC, plumbing, etc.), and fire protection.

Q4. What aspects of your role in the PFSF project are relevant to Unified Contention Utah L/QQ?

A4. As Lead Mechanical Engineer, it is my responsibility to establish the design basis and review all design activities of the mechanical systems at the PFSF, including those located in the Canister Transfer Building ("CTB"). Those systems include cask handling systems -- including the cranes, lifting devices and seismic support struts, -- the fire protection system, the compressed air system, the HVAC system, and the plumbing systems. I am also responsible for the preparation of the principal design criteria, facility design, and operation systems portions of the PFSF Safety Analysis Report ("SAR").

Q5. What is the purpose of your testimony?

A5. The purpose of my testimony is to describe the process for transferring spent fuel canisters from the shipping casks in which they arrive at the PFSF to the storage casks, and describe the methodology for calculating the time involved in canister transfer operations. Additionally, I will describe the safety classification of the structures, systems and components ("SSCs") relevant to Unified Contention Utah L/QQ.

II. CANISTER TRANSFER OPERATIONS

Q6. Please describe the process of transferring canisters containing spent fuel from the shipping casks to the storage casks.

A6. Transfer of the canister containing spent fuel from the shipping cask to the storage cask takes place entirely within the CTB. After the receipt inspection, the overhead bridge crane is used to remove the impact limiters from the shipping cask. A lifting yoke is attached to the crane and hooked to the shipping cask, the cask is placed upright on the cradle, lifted off the transport vehicle, and moved into one of three canister transfer cells. The shipping cask is secured in place by attaching seismic support struts between the cask and the transfer cell walls. The shipping cask lid is unbolted and removed. The canister is then accessible through the top of the shipping cask where the canister lifting attachments and hoist slings are installed onto the canister.

The HI-TRAC transfer cask used in the canister transfer operation is then placed onto the shipping cask by the overhead bridge crane or the semi-gantry crane. The transfer cask is secured in place by attaching seismic support struts between the cask and the transfer cell walls. In order to assure cask stability in the event of an earthquake, the crane is not disconnected from the transfer cask until the seismic support struts are attached to the transfer cask. (The HI-TRAC transfer cask can remain connected to the crane throughout the canister transfer operation since the transfer cask has a canister downloader that raises and lowers the canister, so the crane is not needed to hoist the canister. In this configuration, it is not necessary to connect the seismic support struts since continuous connection of the transfer cask to the crane provides assurance that the transfer cask cannot topple in the event of an earthquake.)

Q7. What happens after the transfer cask is coupled to the shipping cask?

A7. Once the seismic support struts are attached to the transfer cask, shield doors installed on the bottom of the transfer cask are opened, the hoist slings are pulled up through the transfer cask and attached to the downloader, and the canister is lifted up into the transfer cask, just above the shield doors. The doors are then closed and the canister is lowered onto the doors, which support the weight of the canister. The lifting yoke and crane are reattached to the transfer cask. Next, the support struts are disconnected from the transfer cask, and the transfer cask is lifted from the shipping cask by the crane and placed on top of the storage cask, which has already been secured in place to the transfer cell walls by the seismic support struts. The support struts are again attached between the transfer cask and the transfer cell walls; the canister is lifted slightly to remove its weight from the transfer cask shield doors, and the shield doors are opened and the canister is lowered into the storage cask. Finally, the transfer cask is removed from the top of the storage cask, the storage cask lid is installed, and temporary shielding is removed from the cask transfer area. The detailed steps required to perform the canister transfer operation, the number of personnel required, and the duration of each step are provided in Table 5.1-1 of the PSFS SAR, which is identified as

PFS Exhibit ZZ. The operations described above would essentially be reversed in order to ship spent fuel offsite.

Q8. How long does it take to complete a transfer operation?

A8. The total estimated time to complete a single canister transfer operation is approximately 20 hours.

Q9. What portion of that total time is the fuel canister not completely sealed within a shipping or storage cask?

A9. The total time that the MPC is not completely sealed within either a shipping cask or storage cask is nine (9) hours per operation (from initiation of the removal of the HI-STAR cask closure plate bolts to completion of the installation of the HI-STORM cask lid and bolts).

The canister is always secured or protected from a seismic event by a shipping , transfer, or storage cask. The shipping , transfer or storage cask is either connected to the crane or secured to the transfer cell walls by seismic struts, so that there is never a point in time when a seismic event could cause a cask or the MPC to topple.

Q10. What is the total time that the canister is being held by a crane?

A10. The total time the canister is being lifted directly or in the transfer cask and held by the crane in the transfer cell while being transferred from the shipping cask to the storage cask is approximately 3 hours per transfer operation. Of this duration, the transfer cask is sitting on top of the shipping cask or storage cask and is supported by either the crane or both the crane and the seismic support struts. The duration of the actual movement of the loaded transfer cask from the shipping cask to storage cask, or vice-versa, is approximately 0.7 hours (42 minutes).

Q11. What do these durations represent in annual hours?

A11. In order to achieve the ultimate capacity of 4,000 casks over a 20-year loading cycle, the PFSF would receive on average approximately 200 spent fuel casks per year (4 casks per week). Thus, on average, transfer operations would occur for approximately 4,000 hours during the year. The total time a canister is not completely sealed within a shipping or storage cask is approximately 45% of the transfer time, or 1,800 hours a year, which is approximately 20% of the year's duration. The total time the canister is being lifted and held by a crane is a total of 600 hours annually, which represents 15% of the transfer time and approximately 7% of the total time in a year. The total time that the loaded transfer cask is being moved from the top of the shipping cask over to the top of the storage cask, or vice-versa, is approximately 140 hours annually, which represents about 4% of the transfer time and approximately 2% of the total time in a year.

Q12. How did you arrive at the calculation of the time it takes to complete the cask transfer?

A12. The summary of the steps and times involved in the HI-STORM canister transfer operations is found in Table 5.1-1 of the PFS SAR. I arrived at these figures by using the operation durations published in the early revisions of the Holtec HI-STORM and HI-STAR SARs and through discussions with ISFSI personnel at Palisades Nuclear Plant in Michigan who have actual experience with cask transfer operations. The conservatism of these operation times can be seen by comparing Table 5.1-1 of the PFS SAR with Table 10.3.3a of the HI-STORM FSAR, "MPC Transfer Into the HI-STORM 100 System Directly from Transport Using the 125-Ton HI-TRAC Transfer Cask," which is identified as PFS Exh. AAA. From initiation of the removal of the HI-STAR cask closure plate bolts to completion of the installation of the HI-STORM cask lid and bolts, the PFS SAR provides for approximately nine (9) hours to complete the operation. By contrast, the HI-STORM 100 FSAR provides for approximately 3.2 hours to complete this phase of the transfer.

III. SAFETY CLASSIFICATION OF STRUCTURES, SYSTEMS, AND COMPONENTS

Q13. Please describe how SSCs are classified in terms of their importance to safety.

A13. SSCs are classified as "Important to Safety" or "Not Important to Safety." The tabulation of the PFSF SSCs by their classification is contained in Table 3.4-1 of the SAR, which is identified as PFS Exhibit BBB. There are three classification subcategories – A, B, and C – for SSCs that are important to safety. These subcategories are defined in the PFS SAR at 3.4-2, which is also included as part of PFS Exhibit CCC.

Q14. What do these categories represent?

A14. Category A items are those designated as critical to safe operation of the facility, whose failure or malfunction could directly result in a condition adversely affecting public health and safety. Category B items have a major impact on safety and their failure could indirectly result in a condition adversely affecting public health and safety. Category C items have a minor impact on safety, such that their failure would not be likely to create a situation adversely affecting public health and safety.

Q15. What is the safety classification of the PFSF SSCs of relevance to Unified Contention Utah L/QQ?

A15. The only Category A SSC is the canister itself. The CTB, the storage casks, the transfer cask, the cranes, lifting device, and seismic support struts are all Category B. The storage cask pads are Category C SSCs.

Q16. Does this conclude your testimony?

A16. Yes, it does.

DONALD WAYNE LEWIS | **LEAD ENGINEER MECHANICAL DIVISION**

EDUCATION

Montana State University - Bachelor of Science, Civil Engineering - 1980
Daniel International Corp. - Course in ASME Section III - 1982
Daniel International Corp. - Course in Welding - 1983

REGISTRATIONS

Professional Engineer - New York (1988)
Colorado (1997)
Maine (1999)
Utah (2001)
Iowa (2002)

EXPERIENCE SUMMARY

Mr. Lewis has 20 years of engineering experience in the power generation industry, and has participated in all phases of power plant engineering from design through construction, pre-operational testing to on-line modifications.

Mr. Lewis has experience on several nuclear facilities. Assignments include the design of spent nuclear fuel storage facilities, plant systems design modifications, and on-site engineering of mechanical systems installation. Spent fuel storage facility design involved preparation of the design of mechanical aspects and related licensing of the facilities, including an on-site assignment as project engineer for the client for construction of one of the facilities. Plant systems modification assignments involved resolving system design problems, preparing design changes and supporting analyses, revising drawings and preparing specifications. On-site engineering of mechanical systems installation involved resolving pipe and equipment installation conflicts, reviewing and revising design drawings, ensuring code compliance, procuring system components, and developing start-up procedures.

Mr. Lewis has experience on four coal-fired boiler plants. Assignments included the design of mechanical systems on a flue gas scrubber project, development of system descriptions and operating instructions; and the evaluation of a coal to natural gas conversion design. Work involved design of piping systems, component selection and sizing, preparing calculations and specifications, reviewing proposal submittals, initiating process flow and layout drawings; writing plant operation instructions; and preparing cost analyses.

Mr. Lewis is currently assigned to two spent fuel storage projects: the Duane Arnold Energy Center and Private Fuel Storage Facility where he is Lead Mechanical Engineer, responsible for mechanical design and licensing of the facilities.

DETAILED EXPERIENCE RECORD
LEWIS, DONALD WAYNE

STONE & WEBSTER ENGINEERING CORPORATION, DENVER, COLORADO
(Apr 1988 - Present)
Appointments:
Lead Engineer, Mechanical Division - Jan 1998
Senior Mechanical Engineer, Mechanical Division - Nov 1990
Mechanical Engineer, Mechanical Division - Jan 1989

Duane Arnold Energy Center, Cedar Rapids, Iowa – Nuclear Management Company
(July 2000 - Present)
LEAD MECHANICAL ENGINEER

Indian Point 1, Buchanan, New York – Entergy Nuclear Northeast
(April 2001 – January 2002)
PROJECT ENGINEER

Indian Point 2 Nuclear Plant, Buchanan, NY – Consolidated Edison
(January 1999 - January 2000)
PROJECT ENGINEER

Maine Yankee Atomic Plant, Wiscasset, ME – Maine Yankee Power Company
(November 1998 – October 2001)
LEAD MECHANICAL ENGINEER

Yucca Mountain Project, Las Vegas, NV - U.S. Department of Energy
(June 1998 - August 1998)
SYSTEMS ENGINEER

Rocky Flats Environ. Tech. Site, Golden, CO - Rocky Flats Engineers & Contractors, L.L.C.
(May 1998 - Sept 1998)
RADIOLOGICAL CONSULTANT

Prairie Island Generating Plant, Red Wing, MN - Northern States Power Company
(Oct 1997 - Present)
PROJECT ENGINEER

National Wind Technology Center, Golden, CO - National Renewable Energy Laboratory
(Oct 1997 - Apr 1998)
SENIOR MECHANICAL ENGINEER

Rocky Flats Environmental Technology Site, Golden, CO - BNFL
(July 1997 - Oct1997)

SENIOR MECHANICAL ENGINEER

Private Fuel Storage Facility, Goshute Indian Res., UT - Private Fuel Storage
(Oct 1996 - Present)
LEAD MECHANICAL ENGINEER

Goodhue County ISFSI, Frontenac, MN - Northern States Power Company
(Aug 1995 - Sept 1996)
PROJECT ENGINEER

Navajo Generating Station, Page AZ - Salt River Project
(Sept 1993 - Nov 1995)
SENIOR MECHANICAL ENGINEER

Prairie Island Generating Plant, Red Wing, MN - Northern States Power Company
(Jan 1992 - Aug 1993)
SENIOR MECHANICAL ENGINEER

Neil Simpson Station, Gillette, WY - Black Hills Power Company
(Sept 1991 - Dec 1991)
SENIOR MECHANICAL ENGINEER

North Omaha Station, Omaha, NE - Omaha Public Power District
(July 1991 - Aug 1991)
SENIOR MECHANICAL ENGINEER

Fort Calhoun Power Station, Ft Calhoun, NE - Omaha Public Power District
(Apr 1988 - June 1990) (Nov 1990 - Aug 1991)
SENIOR MECHANICAL ENGINEER

Prairie Island Generating Plant-Unit 2, Red Wing, MN - Northern States Power Company
(July 1990 - Oct 1990)
LEAD MECHANICAL ENGINEER

EG&G Rocky Flats Inc., Golden, CO - U. S. Department of Energy
(July 1990)
MECHANICAL ENGINEER

U. S. Department of Energy, Hanford, WA
(June 1990)
MECHANICAL ENGINEER

STONE & WEBSTER ENGINEERING CORP., CHERRY HILL, NEW JERSEY
(Sept 1983 - Mar 1988)
Appointments:

Engineer, Mechanical Division - Aug 1987
Construction Engineer - Oct 1985
Senior Field Engineer - Oct 1984
Field Engineer - Sept 1983

Nine Mile Point Nuclear Station, Unit 2, Lycoming, NY - Niagara Mohawk Power Corporation
(Sept 1983 - Mar 1988)
ENGINEER, Mechanical Division (Aug 1987 - Mar 1988)
ENGINEER, Construction Division (Sept 1983 - July 1987)

Oswego Steam Station Units 5 & 6, Oswego, NY - Niagara Mohawk Power Corporation
(Dec 1986)
CONSTRUCTION ENGINEER

DANIEL INTERNATIONAL CORPORATION, GREENVILLE, SOUTH CAROLINA
(June 1982 - Aug 1983)

Wolf Creek Nuclear Plant, New Strawn, KS - Kansas Gas & Electric
CONSTRUCTION ENGINEER II

J.A. JONES CONSTRUCTION COMPANY, CHARLOTTE, NORTH CAROLINA
(Oct 1981 - Apr 1982)

Washington Nuclear Plant No. 1, Handford, WA - Washington Public Power Supply System
FIELD ENGINEER

WRIGHT SCHUCHART HARBOR-BOECON-GERI, RICHLAND, WASHINGTON
(Mar 1981 - Oct 1981)

Washington Nuclear Plant No. 2, Handford, WA - Washington Public Power Supply System
ASSOCIATE STRUCTURAL ENGINEER

MONTANA STATE HIGHWAY DEPARTMENT, HELENA, MONTANA
(July 1979 - Sept 1979, July 1980 - Mar 1981)
CIVIL ENGINEER I (Traffic Division, Jan 1981 - Mar 1981)
ENGINEER AIDE (July 1979 - Sept 1979)

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APPLICANT'S PREFACE OF THE TESTIMONY OF C. ALLIN CORNELL ON UNIFIED CONTENTION UTAH L/QQ

I. WITNESS

Dr. C. Allin Cornell is a research professor at Stanford University in Stanford, California and an independent engineering consultant. Dr. Cornell has developed extensive professional expertise in earthquake engineering, probabilistic engineering analysis of seismic and other loads on structures, and structural responses to such loads. Due to Dr. Cornell's expertise in these areas, he has been actively involved in the development of structural design guidelines, codes and standards, including determining the appropriate level of earthquake design required to achieve a desired level of safety. Dr. Cornell has been involved in establishing earthquake standards of design for nuclear power plants, radiological waste facilities, offshore oil platforms, and buildings. Nuclear power plants and other nuclear facilities have been a major focus of Dr. Cornell's professional work on the development and application of methodologies and standards for evaluating earthquake hazards. His 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. Dr. Cornell has also been in the forefront of addressing, 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.

II. TESTIMONY

A. Scope of Testimony

Dr. Cornell will testify regarding the 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. Dr. Cornell 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 applicable to NRC-licensed facilities like the PFSF, as the standard for the PFSF seismic design. Dr. Cornell will also testify regarding specific issues raised by the State in Section E of the Unified Contention Utah L/QQ.

B. Appropriateness of Using PSHA

Dr. Cornell will testify that the proposed use by PFS of a Probability Seismic Hazard Analysis 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.

C. Appropriateness of Using a 2,000-Year Return Period DBE for the PFSF

Dr. Cornell will testify that there are two general principles of risk informed seismic design. 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 ("MAPE") of the DBE and the level of conservatism incorporated into the design criteria and procedures.

With respect to the first general principle, Dr. Cornell will testify that because the Commission has determined that ISFSIs pose less risk than nuclear power plants, it is appropriate for ISFSIs, such as the PFSF, to have a higher MAPE than nuclear power plants. In accordance with the second general principle of risk informed seismic design, in determining the MAPE it is appropriate to consider the high levels of margin or safety embodied in nuclear safety acceptance criteria. Large safety margins on the order of magnitude of five both exist and have been demonstrated for important to safety systems, structures and components at the PFSF Because of these large margins, design the PFSF could withstand an earthquake well in excess of the 2000 year DBE Therefore, Dr. Cornell will conclude that a 2000 year DBE for the PFSF provides adequate protection for the public health and safety.

D. Response to State's Claims

Dr. Cornell will also respond to claims raised by the State with respect to the contention. He will testify that DOE-1020 as an illustrative example of the risk graded approach, not the source of the margins on which he bases his opinions. Rather, the source of the margins on which he bases his expert opinion are the conservatisms inherent in typical nuclear power plant design and acceptance criteria as well as actual demonstration of the capability of key SSCs at the PFSF to withstand a beyond design basis earthquake of 10,000 years or more without the release of radioactivity to the environment. He will further testify that the construction of fragility curves are not required to demonstrate such conservatism as claimed by the State but that the conservatism may be shown by demonstrating that SSC failure will not occur. Dr. Cornell will also testify that the PFS meets the requirements of the current preferred approach of the rulemaking plan as modified and that the issues raised in the Section E of the contention do not contradict the basis of his opinion that the 2000 year DBE for the PFSF provides adequate public health and safety.

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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 2002 Medal of the Seismological Society of America. Various other accomplishments and studies relevant to this matter include the following:

- In 1968, I published a seminal paper in the Bulletin of the Seismological Society on characterizing earthquake hazards using probabilistic seismic hazard analysis ("PSHA"). Improved and elaborated by more than thirty years of subsequent application and research (by myself and by many others), PSHA has become the standard method for earth scientists to characterize and report the earthquake threat at a site. For example, the USGS has used the method for two decades to study the entire US and to produce maps of seismic hazard that appear in all model building codes.

- I have participated directly, commonly as a senior advisor, in many prominent PSHA studies. These include the PSHA for the Diablo Canyon Nuclear Power Plant ("NPP"), the major EPRI Seismic Owners Group PSHA of the Central and Eastern US ("CEUS") NPP sites, the Caltrans-sponsored PSHA studies of all major California bridges, and PSHAs for the INEEL and LLNL DOE national lab sites and the Yucca Mountain site. I was also a member of the Senior Seismic Hazard Analysis Committee (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 $5x10^{-4}$. Another way of referring to these design basis motions is to say that they correspond to the 2,000-year mean return period ("MRP") level, or "the 2,000-year MRP earthquake."

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

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[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

Footnote continued on next page

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_Rs, 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 NRCs 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

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)

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[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 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 $1\text{x}10^{-4}$ for SSCs ISFSIs such as the PFSF is consistent with the NRC's performance objectives for operating nuclear plants, which THE NRC 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 $1\text{x}10^{-4}$ [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 $4\text{x}10^{-6}$ to about $1\text{x}10^{-4}$, with most lying between 0.6 and $1\text{x}10^{-5}$. 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 $1\text{x}10^{-5}$. [Ref. 22 (NUREG/CR-5501)] As discussed above, DOE-STD-1020-94 also uses, explicitly, a performance goal of $1\text{x}10^{-5}$ for nuclear reactor SSCs. The use of a probability of seismic failure or performance goal for the PFSF SSCs, such as $1\text{x}10^{-4}$, higher than that for nuclear power plants SSCs (about $1\text{x}10^{-5}$) is consistent with the risk-graded approach of the probabilistic approach.

Second, an SSC performance goal of $1\text{x}10^{-4}$ 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 $1\text{x}10^{-4}$. 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 $1x10^{-4}$ 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 $1x10^{-4}$ 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 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 <u>no</u> 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 x 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 x 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 x 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 x 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 x 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 x 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

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[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 basis for the State's Contention.

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 standalone 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 2×10^{-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 $1x10^{-4}$. 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 started 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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2001, Rio de Janeiro, Brazil, June 2001.

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Reference 40: Department of Energy, DOE-STD-1021-93., "*Natural Phenomena Hazards Performance Categorization Guidelines for Structures, Systems and Components*", July1993.

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

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

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

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

1993 *Seismic Studies (Seismic Hazard Analysis;*
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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
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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. STl, 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.

Karamchandani, A., and Cornell, C. A., "Sensitivity Estimation Within First and Second Order Reliability Methods", *Structural Safety*, Vol. 11, No. 2, 1992, pp. 95-108.

Cornell, C.A., Wu, S.C., Winterstein, S.R., Dieterich, J.H., and Simpson, R.W., "Seismic Hazard Induced by Mechanically Interactive Fault Segments", *BSSA*, Vol. 83, No. 2, pp. 436-449, April, 1993.

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Bazzurro, P. and Cornell, C.A., "Seismic Hazard Analysis for Non-Linear Structures. I: Methodology", *Jo. of Str. Engrg.*, ASCE, Vol. 120, No. 11, November, 1994.

Bazzurro, P. and Cornell, C.A., "Seismic Hazard Analysis for Non-Linear Structures. II: Applications", *Jo. of* S *Engrg.*, ASCE, Vol. 120, No. 11, November, 1994.

Banon, H., Bea, R.G., Bruen, F.J., Cornell, C.A., and Krieger, W.F., "Assessing Fitness for Purpose of Offshore Platforms. I: Analytical Methods and Inspections", *Jour. of Struct. Engrg.*, ASCE, Vol. 120, No. 12, Dec, 1994.

Wu, S.-C., Cornell, C.A., and Winterstein, S.R., "A Hybrid Recurrence Model and Its Implication on Seismic Hazard Results", *BSSA*, Vol. 85, No. 1, pp. 1-16, February, 1995.

Jackson, D.D., Aki, K., Cornell, C.A., Dieterich, J.H., Henyey, T.L., Mahdyiar, M., Schwartz, D., and Ward, S.N. (Working Group on California Earthquake Probabilities), "Seismic Hazard in Southern California: Probable Earthquakes, 1994 to 2024 ", *BSSA,* Vol. 85, No. 2, pp. 379-439, April, 1995.

Bazzurro, P., Cornell, C.A., Diamantidis, D., and Manfredini, G.M., "Seismic Damage Hazard Analysis for Requalification of Nuclear Power Plant Structures: Methodology and Application", *Nuclear Engineering and Design,* Vol. 160, pp. 321-332, 1996.

Manuel, L., and Cornell, C.A., "The Influence of Alternative Wave Loading and Support Modeling Assumptions on Jack-up Rig Response Extremes", *Transactions of ASME, Journal of Offshore Mechanics and Arctic Engineering*, PP. 109-114, Vol. 118, No. 2, May, 1996.

Shome, N., Cornell, C.A., Bazzurro, P., and Carballo, J.E., "Earthquakes, Records and Nonlinear Responses", *Earthquake Spectra*, Vol. 14, No. 3, pp. 469-500, August, 1998.

Bazzurro, P., Cornell, C.A., Shome, N., and Carballo, J.E., "Three Proposals for Characterizing MDOF Nonlinear Seismic Response", *Journal of Structural Engineering, ASCE,* Vol. 124, No. 11, pp. 1281-1289, November, 1998.

Budnitz, R.J., Apostolakis, G., Boore, D.M., Cluff, L.S., Coppersmith, K.J., Cornell, C.A., and Morris, P.A., "Use of Technical Expert Panels: Applications to Probabilistic Seismic Hazard Analysis", *Risk Analysis*, Vol. 18, No. 4, pp. 463 –469, 1998.

Manuel, L., Schmucker, D.G., Cornell, C.A. and Carballo, J.E., "A Reliability-Based Design Format For Jacket Platforms Under Wave Loads", *Marine Structures*, Vol.11, No. 10, pp. 413-428, 1998.

Bazzurro, P., and Cornell, C.A., "On Disaggregation of Seismic Hazard", *Bulletin of Seismological Society of America, B.S.S.A..,* Vol. 89, No.2, pp. 501-520, April, 1999.

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Stahl, Bernhard, Gebara , Joseph M., Aune, Stig and Cornell, C.Allin, "Acceptance Criteria for Offshore Platforms", *Journal of Offshore Mechanics & Arctic Engineering,* Vol. 122,No. 3, pp. 153-156, August, 2000.

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Yun, S-Y., Hamburger, R. O., Cornell, C. A., and Foutch, D. A., "Seismic Performance for Steel Moment Frames", accepted for publication, *ASCE Structural Journal* for publication in 2001

Vamvatsikos, D. and Cornell, C. A., "Incremental Dynamic Analysis", accepted for publication, March, 2002, *Structural Dynamics and Earthquake Engineering*, 2001.

Luco, N. and Cornell, C. A., "Structure-Specific Scalar Intensity Measures for Near-Source and Ordinary Earthquake Ground Motions", *Earthquake Spectra*, Submitted, 2001.

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Hamburger, R. O., Foutch, D. A., and Cornell, C. A., "Translating Research to Practice: FEMA/SAC Performance-based Design Procedures", submitted to *EERI*, for special publication on SAC/FEMA Guidelines, September, 2001.

Conference Proceedings and Book Chapters:

Cornell, C. A., Benjamin, J. R., and Gabrielsen, B. L., "A Stochastic Model of the Creep Deflection of Reinforced Concrete Beams", *Proceedings of International Symposium in the Flexural Mechanics of Reinforced Concrete*, Miami, Florida, November, 1964.

Cornell, C. A., Reinschmidt, K. F., and Brotchie, J. R., "A Method of Structural Optimization", *Proceedings of International Symposium on the Use of Computers in Structural Engineering*, University Newcastle, England, February, 1966.

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Rascon, O. A. and Cornell, C. A., "A Physically Based Model to Simulate Strong Motion Earthquake Records on Firm Ground", *Proceedings of the Fourth World Conference on Earthquake Engineering*, Santiago, Chile, January, 1969.

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Cornell, C. A., "Structural Safety Specifications Based on Second-Moment Reliability Analysis", *Final Report*, IABSE Symposium on Concepts of Safety of Structures and Methods of Design, London, England, September, 1969.

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Vanmarcke, E. H. and Cornell, C. A., "Seismic Risk and Design Response Spectra", *Proceedings of the ASCE Conference on the Safety and Reliability of Metal Structures*, Pittsburgh, Pennsylvania, November, 1972.

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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

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

I. WITNESSES

A. Dr. Krishna P. Singh

Krishna P. Singh is President and CEO of Holtec International ("Holtec") and bears the ultimate corporate responsibility for the accuracy and correctness of Holtec's spent fuel dry storage systems. Dr. Singh has a Ph.D. in Mechanical Engineering and has extensive experience in the design and licensing of nuclear spent fuel systems extending back to 1979. Over the past twenty-three years, Dr. Singh has personally led the design and licensing of spent fuel storage systems for over forty nuclear plants, and for Holtec's HI-STAR 100 System and HI-STORM 100 Storage Cask System. He is also the inventor of the honeycomb basket design utilized in the HI-STAR 100/HI-STORM Systems and the METCON™ construction used in the HI-STORM System overpack. His professional work in the field of applied heat transfer and structural mechanics consists of over 500 industry reports, over fifty published papers in the refereed technical literature, and academic courses taught at the University of Pennsylvania.

B. Dr. Alan I. Soler

Dr. Alan I. Soler is the Executive Vice President and Vice-President of Engineering for Holtec International. He is responsible for Holtec corporate engineering activities, including overseeing the analyses performed to establish the stability of the HI-STORM 100 System under postulated seismic events. Dr. Soler is the lead structural discipline expert responsible for the design of the HI-STORM System, including supporting analyses, and he has acted in this capacity since the design was conceptualized in the early 1990s. Dr. Soler either performed or reviewed all HI-STORM System seismic analyses conducted in support of deployment of the HI-STORM System at the PFSF. Prior to Dr. Soler's employment with Holtec International, he was a tenured Professor of Mechanical Engineering and Applied Mechanics at the University of Pennsylvania for over 26 years.

C. Dr. Everett L. Redmond

Everett L. Redmond is a Principal Engineer and Manager of the Nuclear Physics Department with Holtec. Dr Redmond is responsible for all shielding, criticality, and confinement analysis work related to Holtec's dry cask storage systems. He is 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. Dr. Redmond has also performed site-specific shielding analyses in support of deployment of the HI-STORM 100 Cask System at the Private Fuel Storage Facility Dr. Redmond has 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. His 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

II. TESTIMONY

A. Scope

Drs. Singh, Soler and Redmond will testify on radiological dose consequences issues raised by the State with respect to Basis 2 of Section E of the Unified Contention, Utah L QQ. They will show that the radiological dose consequences for a postulated 10,000 year beyond design basis earthquake would be far below the 5 rem accident limit 10 C.F.R. § 72.106(b) and that no adverse radiological consequences would be expected to occur from such an event.

B. Applicable Dose Limit for Beyond Design Basis Accident Event

Drs Singh, Soler and Redmond will testify and show that the applicable dose limit for a beyond design basis event ,such as a postulated 10,000 year beyond design basis earthquake, is the 5 rem limit found in 10 C.F.R. § 72.106(b).

C. Evaluation of Radiological Doses from Hypothetical Cask Tipover Events

Even though such an occurrence is highly unlikely, Drs Singh, Soler and Redmond will evaluate the radiological consequence from hypothetical cask tip-over events. They will testify that even assuming the maximum 4,000 cask were to tipover during a postulated beyond design basis earthquake, the limits at the site boundary will be far below the 5 rem limit found of 10 C.F.R. § 72.106(b), and in fact will remain essentially unchanged regardless of whether one assumes that a single cask, any number of them, or all the casks tipover.

D. Response to Claims of Dr. Marvin Resnikoff

Drs. Singh, Soler and Redmond will respond to the myriad of claims raised by Dr. Resnikoff of alleged dose consequences from a beyond design basis earthquake. Specifically:

- Differences between the Holtec CoC and the PFSF do not affect validity plant of specific analyses performed by Holtec for the PFSF.
- Tipover of the casks would result in only localized, limited damage to cask and canister and would not result in adverse radiological consequences at the PFSF.

- The radiological dose consequences calculations of Dr. Resnikoff are based on flawed methodologies and precepts and contain mistakes.
- Limitations on neutron doses to workers are governed by different rules than public dose limits, and moreover cask tipover events would not cause significant increases in worker neutron dose exposures.
- Sliding or other impacts of casks would not threaten the confinement function of the multi-purpose containers and there would be no release of radioactivity.

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 (see 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_o 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 one intake vent and one 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, cannot 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. Sliding Impacts

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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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

APPLICANT'S PREFACE TO THE TESTIMONY OF PAUL J. TRUDEAU AND ANWAR E. Z. WISSA ON SECTION C OF UNIFIED CONTENTION UTAH L/QQ

I. WITNESSES

A. Paul J. Trudeau

Paul J. Trudeau is a Senior Lead Geotechnical Engineer at Stone & Webster, Inc., a Shaw Group Company ("S&W") in Stoughton, Massachusetts. Mr. Trudeau has twenty-nine years of experience in geotechnical engineering, including the performance of subsurface soil investigations; the performance and supervision of the analysis of foundations in support of the design of structures; the performance of laboratory tests of soils including index property tests, consolidation tests, static and dynamic triaxial tests, and other tests; the performance of analyses of the performance of soils and structures under static and dynamic conditions; the development of geotechnical design criteria for other engineering disciplines, such as Structural, Environmental, Engineering Mechanics, and Electrical; and the preparation of the geotechnical sections of Preliminary and Final Safety Analyses Reports and Environmental Reports.

B. Anwar E.Z. Wissa

Dr. Anwar E.Z. Wissa is President of Ardaman and Associates ("A&A") in Orlando, Florida, a professional corporation that provides numerous services, including subsurface investigations, foundation engineering, laboratory testing, construction materials testing and inspection, and contamination remediation. Dr. Wissa received his D.Sc. from the Massachusetts Institute of Technology in 1965. He has been a Fellow of the American Society of Civil Engineers since 1983, serving on the Committee on Placement and Improvement of Soil for nine years. Dr. Wissa has also been a member of Committee D-18 on Soil and Rock for the American Society of Testing and Materials ("ASTM") since 1966. He has been extensively involved in projects employing soil cement, including reservoirs and pavements over his forty year professional career, and is the author of several publications on the use of soil cement.

II. TESTIMONY

A. SCOPE

Mr. Trudeau will address the allegations raised by the State in Section C of Unified Contention Utah L/QQ concerning: (1) the characterization of subsurface soils at the PFSF site through subsurface investigations, sampling and analyses and (2) the stress/strain behavior of the soils under design basis earthquake conditions. Mr. Trudeau and Dr. Wissa will testify on the proposed use of soil cement and cement-treated soil to enhance the seismic behavior of the soils beneath and adjacent to the foundations of the safety-related structures at the PFSF. In this testimony, Mr. Trudeau will respond to the allegations raised by the State in Sections C.1 (with respect to the number of geotechnical borings for the pad emplacement area), C.2.a (with respect to the sampling and analysis of critical soil layers), and C.3.a (regarding the characterization of the stress-strain behavior of foundation soils). Jointly, Mr. Trudeau and Dr. Wissa will address the various soil cement related issues raised in Subsections C.3.b, C.3.c and C.3.d of the Unified Contention.

B. SUBSURFACE INVESTIGATIONS

Mr. Trudeau will describe the varied and extensive investigations that PFS has conducted to characterize the soils at the PFSF site and demonstrate that the density of borings in the pad emplacement area is sufficient given the reasonable uniformity in the horizontal direction of the properties of the soils at the site.

C. SAMPLING AND ANALYSIS

Mr. Trudeau will testify that PFS has conducted continuous sampling of the critical soil layers at the site and has conducted borings to sufficient depths to properly characterize the soil conditions. He will also describe the laboratory soils testing program carried out by PFS and explain that the number of samples tested and the kinds of tests conducted are appropriate.

D. STRESS-STRAIN BEHAVIOR OF FOUNDATION SOILS UNDER EARTHQUAKE LOADS

Mr. Trudeau will testify that PFS has performed resonant column tests that provide sufficient information to describe the stress-strain behavior of the soils under the range of cyclic strains imposed by the design basis earthquake.

E. USE OF SOIL CEMENT

Mr. Trudeau and Dr. Wissa will explain the composition, properties and intended uses of soil cement and cement-treated soil at the PFSF site, will describe the testing program that PFS is carrying out to develop suitable soil cement and cement-treated soil mixtures, and will address the construction techniques that may be used to ensure proper placement of those mixtures. Mr. Trudeau and Dr. Wissa will also refute the claims raised by the State as to the alleged lack of showing (via case precedent or site-specific testing and analysis) that the soil cement and cement-treated soil will be able to resist the design basis loadings for the storage pads and the CTB.

Mr. Trudeau and Dr. Wissa will further describe the measures that PFS may take if warranted to prevent adverse impacts on the native soils at the site from the placement of soil cement and cement-treated soil. Finally, Mr. Trudeau and Dr. Wissa will testify that the mechanisms the State has postulated for the degradation of the performance of the soil cement and cement treated soil are either not credible or are addressable in the construction program.

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**

JOINT TESTIMONY OF PAUL J. TRUDEAU AND ANWAR E. Z. WISSA ON SECTION C OF UNIFIED CONTENTION UTAH L/QQ

I. WITNESSES

A. Paul J. Trudeau ("PJT")

Q1. Please state your full name.

A1. Paul J. Trudeau.

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

A2. I am a Senior Lead Geotechnical Engineer at Stone & Webster, Inc., a Shaw Group Company ("S&W") in Stoughton, Massachusetts.

Q3. Please summarize your educational and professional qualifications.

A3. My professional and educational experience is described in the *Curriculum Vitae* attached to the testimony I am filing simultaneously herewith with respect to Section D of Unified Contention Utah L/QQ ("the Unified Contention.") As indicated there, I have twenty-nine years of experience in geotechnical engineering, including the performance of subsurface soil investigations; the performance and supervision of the analysis of foundations in support of the design of structures; the performance of laboratory tests of soils including index property tests, consolidation tests, static and dynamic triaxial tests, and other tests; the performance of analyses of the performance of soils and structures under static and dynamic conditions; the development of geotechnical design criteria for other engineering disciplines, such as Structural, Environmental, Engineering Mechanics, and Electrical; and the preparation of the geotechnical sections of Preliminary and Final Safety Analyses Reports and Environmental Reports.

Q4. What is the basis of your familiarity with the Private Fuel Storage Facility?

A4. S&W is the Architect/Engineer for the Private Fuel Storage Facility ("PFSF") under contract with Private Fuel Storage, L.L.C. ("PFS" or "Applicant"). As such, it coordinates the facility design activities, including the studies needed to characterize the PFSF site and establish its suitability. My particular areas of concentration on the PFSF project are the analysis of soils – settlement, bearing capacity, and stability of foundations – as well as the conduct of soils investigations, laboratory testing of soils to measure static and dynamic properties, and the performance of computer-aided analyses of the behavior of soils and structures under static and dynamic loading conditions.

Q5. What is the purpose of your testimony?

A5. The purpose of my testimony is to respond to allegations raised by the State of Utah in the Unified Contention concerning: (1) the characterization of subsurface soils at the PFSF site through subsurface investigations, sampling and analyses; (2) the stress/strain behavior of the soils under design basis earthquake conditions;

and (3) the use of soil cement and cement-treated soil to enhance the seismic behavior of the soils beneath and adjacent to the foundations of the safety-related structures at the PFSF. Specifically, I will address herein the allegations raised in Section C of the Unified Contention. As indicated earlier, I am also filing separate testimony in which I address the seismic analysis of the cask storage pads, casks, and their foundation soils and the seismic analysis of the Canister Transfer Building and its foundation. That testimony addresses some of the allegations raised by the State in Section D of the Unified Contention.

B. Anwar E. Z. Wissa ("AEZW")

Q6. Please state your full name.

A6. Anwar E. Z. Wissa.

Q7. By whom are you employed and what is your position?

A7. I am President of Ardaman & Associates ("A&A") in Orlando, Florida. A&A is a professional corporation founded in 1959. It provides numerous services, including subsurface investigations, foundation engineering, laboratory testing, construction materials testing and inspection, and contamination remediation. The company employs a staff of over 360 professional engineers, scientists, technicians, drilling personnel, technical assistants and support staff, and maintains a state-of-the-art geotechnical laboratory at its headquarters.

Q8. Please summarize your educational and professional qualifications.

A8. My professional and educational experience is described in the *curriculum vitae* attached to this testimony. Of particular relevance is the fact that I have been a Fellow of the American Society of Civil Engineers since 1983, serving on the

Committee on Placement and Improvement of Soil for nine years. I have also been a member of Committee D-18 on Soil and Rock for the American Society of Testing and Materials ("ASTM") since 1966. I have been extensively involved in projects employing soil cement, including reservoirs and pavements over my professional career, and have authored several publications on the use of soil cement.

Q9. What is the basis of your familiarity with the Private Fuel Storage Facility?

A9. I was retained by PFS to review the program being implemented by PFS to use soil cement to improve subsurface conditions at the PFSF site. In the process of my review, I have examined a number of documents relating to the design of the facility and, specifically, to the proposed use of soil cement at the site.

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 the Unified Contention concerning the use of soil cement and cement-treated soil to enhance the seismic behavior of the soils beneath and adjacent to the foundations of the safety-related structures at the PFSF. Specifically, I will address herein the soil cement-related allegations raised in Section C of the Unified Contention.

II. GEOTECHNICAL INVESTIGATIONS CONDUCTED AT THE PFSF SITE

Q11. Please describe the investigations that PFS has conducted to characterize the soils at the PFSF site.

A11. (PJT) The initial geotechnical investigations were performed in late 1996. The results of those initial investigations were reflected in the initial version (Revision

0) of the Safety Analysis Report ("SAR") for the PFSF, which was filed in June 1997. Later, in 1999, PFS performed considerable additional soil investigations, including borings in the Canister Transfer Building ("CTB") area and a series of cone penetration test soundings to better assess soil strength and compressibility, as well as the faulting study performed by Geomatrix Consultants, Inc. ("Geomatrix"). Specifically, in 1999, 12 additional borings were drilled and sampled, 39 cone penetration tests were performed (16 of which included measurements of pressure and shear wave velocities in addition to the penetration resistance data), and 18 dilatometer soundings were performed. Those investigations were supplemented with further soils investigations performed in January 2001. The January 2001 investigations were conducted in part by Northland Geophysical LLC, which made downhole geophysical measurements in two borings, which corroborated the geophysical measurements that were made in the seismic cone penetration tests. At the same time, S&W performed additional sampling at sixteen test pits excavated at the PFS facility site to obtain bulk samples of the soils for use in the soil cement testing program. As they stand today, the soils investigations performed at the PFSF are sufficient to properly characterize the site from the geotechnical standpoint.

The results of the geotechnical investigations conducted by PFS are presented in Section 2.6 and Appendix 2A of the SAR, as revised through April 2001 (Rev. 22). That section, 219 pages long plus attachments and appendices, presents a comprehensive description of the various investigations that have been conducted, and includes geologic maps, profiles of the site stratigraphy, and discussions of structural geology, geologic history, and engineering geology.

The locations of the borings made to study subsurface conditions at the PFSF site are summarized in three location plans (which are Figures 2.6-2, 2.6-18, and 2.6-19 of the SAR). Boring logs are provided in Attachment 1 to Appendix 2A of the SAR.

Figure 2.6-5 of the SAR includes 14 sheets of "foundation profiles" that depict the composition of the PFSF subsoil layers at various locations in the pad emplacement area and Figures 2.6-20 through 2.6-22 present foundation profiles under the CTB. Seventeen foundation profiles are provided: 2 diagonal, 6 east-west, and 6 north-south in the pad emplacement area and 2 east-west, and 1 north-south in the Canister Transfer Building area. These profiles cover all safety-related structures and encompass all borings made by PSF in the vicinity of those structures.

The initial set of borings was drilled in the pad emplacement area, following a uniform, grid-like pattern, with the borings spaced approximately 600 feet apart. A determination was made after the initial tests that the soil properties at the PFSF site are reasonably uniform in the horizontal direction (that is, across the various site locations). Because of this uniformity, it was unnecessary to establish a denser set of borings than the one initially provided.

Q12. How did you determine that the soils were reasonably uniform in the horizontal direction?

A12. (PJT) The test data, as presented in SAR Figure 2.6-5, Sheets 1 through 14, demonstrate the horizontal consistency of the materials at the site. This consistency was further demonstrated by the cone penetration test data, which show that the upper soil layers have fairly consistent properties across the pad emplacement area and beneath the CTB.

Moreover, data on the properties of the soils in a trench dug by PFS consultant Geomatrix Consultants, Inc. confirmed that the soils in approximately the upper 30 feet of the subsoil are fairly uniform and consistent in the horizontal direction across the site. The site investigations conducted by Geomatrix for PFS since the SAR was prepared in 1997 are described in the Geomatrix report "Fault Evaluation Study & Seismic Hazard Assessment, February 1999." This report includes two plates, Plates 3 and 4, which present geologic profiles that provide an unambiguous geological characterization of the site and set forth the details of the site's geologic conditions. These geological plates prepared by Geomatrix can be correlated with the data on subsurface conditions presented in the foundation profiles developed under my supervision. Comparison of the Geomatrix plates with the foundation profiles in SAR Fig. 2.6-5 demonstrates that the nature, location, and thickness of the various layers of the profile are identically presented in both documents.

Q13. What methodology was used to characterize the soils at the PFSF site?

A13. (PJT) Soil classification was performed through various methods, including: visual inspection of the samples obtained, in accordance with American Society of Testing and Materials ("ASTM") standards; performance of laboratory tests on soil samples; and interpretation of cone penetration test results. These methods provided a consistent and accurate characterization of the thickness, extent and composition of the subsoil at the site.

Q14. What are the main characteristics of the soils at the PFSF site?

A14. (PJT) Our investigations established that the top 30 feet or so of the subsoil profile are the only ones of interest from the geotechnical standpoint, since below 30 feet, the soils are comprised of very dense sands or silty sands overlying very dense silts, which have great strength, as evidenced by their high standard penetration test blow counts ($N > 100$ blows/ft).

The investigations also established the thickness and extent of the layers of soil comprised within the top 30 feet of the profile. As shown in the foundation profiles in SAR Fig. 2.6-5, within the first 30 feet of the profile, there are five distinct soil layers. Of these, the topmost "eolian soil" layer is of only limited interest because the design intent is to remove it and mix it with cement to form cement-treated soil. The second layer, which runs generally 3 to ~10 feet beneath the surface (sometimes referred to as "Layer 2") was found through the boring and laboratory testing programs to have the lowest strength and highest compressibility of the soils at the PFS site. Subsequently, cone penetration tests confirmed that the Layer 2 soils are the weakest and most compressible soils. Layer 2 is, therefore, the main layer of concern from the standpoint of soil strength and compressibility. The other three layers in the first 30 feet of subsoil have considerably greater strength and less compressibility than the top two layers.

III. METHODOLOGY AND RESULTS IN THE PFSF LABORATORY TESTING PROGRAM

Q15. Would you please describe the objectives of the laboratory testing program that was conducted with regard to the PFSF soils?

A15. (PJT) The purpose of the tests conducted on the samples of soil collected at the PFSF site was to establish certain properties of the soils that are needed as inputs in the design of the site structures. The design activities supported by the test program include the establishment of geotechnical design criteria, the analyses of settlements and bearing capacity of the foundations, and the seismic stability of the structures.

Q16. How many soil samples were obtained for testing in the laboratory testing program?

A16. (PJT) PFS has conducted a comprehensive laboratory testing program that has included taking 33 undisturbed samples, as shown on Table 1 below. Also, there have been 10 consolidation tests, 19 triaxial shear strength tests, 5 cyclic triaxial tests, 2 resonant column tests (at 3 different confining pressures), and 11 direct shear tests.

TABLE 1: UNDISTURBED SOIL SAMPLES TAKEN AT THE PFSF SITE

Boring ID	Sample	Depth to:		Date Taken
		Top	Bottom	
A-2	U2	5.00	7.00	Oct 1996
B-1	U2	5.00	7.00	Oct 1996
B-2	U1	8.00	10.00	Oct 1996
B-3	U1	5.00	7.00	Oct 1996
B-3	U2	10.00	12.00	Oct 1996
B-4	U3	10.00	12.00	Oct 1996
C-1	U3	10.00	12.00	Oct 1996
C-2	U1	5.00	7.00	Oct 1996
C-2	U2	10.00	12.00	Oct 1996
E-2	U1	5.00	7.00	Dec 1998
CTB-1	U3	7.00	9.00	Jan 1999
CTB-1	U5	11.00	13.00	Jan 1999
CTB-1	U7	20.00	22.00	Jan 1999
CTB-4	U1	6.00	8.00	Dec 1998
CTB-4	U2	8.00	10.00	Dec 1998
CTB-4	U7	12.00	13.50	Dec 1998
CTB-4	U9	16.00	17.50	Dec 1998
CTB-4	U11	20.00	21.50	Dec 1998
CTB-4	U13	24.00	25.50	Dec 1998
CTB-4	U15	28.00	29.50	Dec 1998
CTB-5(OW)	U6	10.00	12.00	Jan 1999
CTB-5(OW)	U8	14.00	16.00	Jan 1999
CTB-5(OW)	U10	18.00	20.00	Jan 1999
CTB-5(OW)	U12	22.00	24.00	Jan 1999
CTB-5(OW)	U14	26.00	28.00	Jan 1999
CTB-6	U3	7.00	8.50	Dec 1998
CTB-7	U3	7.00	9.00	Dec 1998
CTB-N	U1	5.00	7.00	Oct 1998
CTB-N	U2	7.00	9.00	Dec 1998
CTB-N	U3	9.00	11.00	Dec 1998
CTB-S	U1	5.00	7.00	Dec 1998
CTB-S	U2	7.00	9.00	Dec 1998
CTB-S	U3	9.00	11.00	Dec 1998

Q17. How were these samples taken?

A17. (PJT) Samples were taken and tested in accordance with procedures established under the general guidance of ASTM standards. Detailed, quantitative criteria were used to ensure that the drilling and sampling of the PFSF site soils was conducted as recommended by the ASTM standards referenced in those procedures. The procedures required, among other things, that an engineer from S&W confirm that the samples were taken in accordance with ASTM standards and project procedures.

PFS obtained samples of all soil strata, from the ground surface to depths as great as 226.5 feet below the ground surface, beneath the foundations of the Canister Transfer Building ("CTB" in the table) and the pad emplacement areas. As indicated in Table 1 above, a total of 33 undisturbed samples were collected – from eight borings in the pad emplacement area and from seven borings in the CTB area.

As I mentioned earlier, initial tests on samples collected in 1996 determined that Layer 2 soil is the main layer of concern from the standpoint of soil strength and compressibility. This determination was later confirmed through laboratory testing and cone penetration tests. Thus, for purposes of supporting the structural design of the facility, it was appropriate to focus the testing program on the samples of Layer 2 soils. Table 1 shows that two-thirds of the undisturbed samples were collected from Layer 2 (about 3 to 10 feet below the ground surface).

Q18. Why is the number of samples tested sufficient?

A18. (PJT) The number of samples tested is sufficient because the soil properties are reasonably uniform across the various site locations. Moreover, the soil layer of primary interest (Layer 2) exhibits great uniformity across the site, as evidenced by the consistency in standard penetration test blow count values and the cone penetration testing tip resistance values. All *in situ* testing performed at the site, and the laboratory tests performed on samples of the soils obtained from the upper 30 feet of the profile, demonstrated that the soils beneath Layer 2 are stronger and less compressible. Thus, it was conservative to concentrate the sampling and testing program on samples obtained from Layer 2.

Q19. What tests were conducted on the soil samples collected at the PFSF site?

A19. (PJT) The laboratory tests that were conducted on the soil samples identified in Table 1 included dynamic testing of samples in both stress and strain-controlled manner, and they were sufficient to determine the properties of materials at the site and establish the design parameters. Among the parameters investigated in the laboratory tests were those that relate to the static and dynamic properties of the soil including grain size, triaxial shear strength, consolidation characteristics, Atterberg limits, water content, direct shear strength, shear moduli, damping, and strength under cyclic loading. Because the soil tests performed by PFS provide sufficient information on the soils at the site, no other tests would be needed to adequately characterize these soils.

The manner in which the laboratory tests were conducted and the test results are fully documented in the test reports in the attachments to Appendix 2A of the SAR. Soil sample preparation for testing is adequately described in the

Engineering Services Scope of Work documents ("ESSOWs") (with respect to field testing) and in Attachments 2 through 8 of Appendix 2A of the SAR (with respect to laboratory testing).

The results of the laboratory tests conducted on PFSF soils are included in the attachments to Appendix 2A of the SAR. Taken together, the test results are sufficient to ensure that the soil characteristics were conservatively interpreted to develop the design parameters. The tests conducted and their results show that the soil conditions are adequate for the proposed foundation loadings, both static and dynamic; that the static and dynamic properties of the soils, such as their compressibility and shear strength, have been properly defined; and that reasonably conservative values of those properties were used in the design.

IV. RESPONSE TO THE STATE OF UTAH'S CLAIMS IN SECTIONS C.1, C.2 AND C.3.a OF THE UNIFIED CONTENTION UTAH L/QQ

Q20. In Subsection C.1 of the Unified Contention, the State asserts that the Applicant has not performed the recommended spacing of borings for the pad emplacement area as outlined in NRC Reg. Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants," Appendix C. Why is the number of borings taken in the pad emplacement area sufficient?

A20. (PJT) First of all, Reg. Guide 1.132 is only a guidance document, and one that applies to nuclear power plants, which have larger and more heavily loaded foundations than are applicable for this ISFSI. In addition, nuclear power plants have entirely different categories of safety-related systems and components that do not exist at the PFS ISFSI, such as buried piping and electrical power and control systems. These interconnected systems sometimes carry radioactive fluids and high-pressure steam and power and control systems that are required for the safe shutdown of the reactors, and, thus, these systems arguably have much greater sensitivity to movements of the ground and the enclosing structures than the components of an ISFSI, which have no such interconnected systems. The applicable guidance for Part 72 facilities, which is NUREG-1567, does not provide any guidelines on the number or placement of borings for foundation analyses.

At any rate, the PFSF boring program conforms to the general guidance in Reg. Guide 1.132. The Guide states at p. 1.132-3:

> Subsurface conditions may be considered favorable or uniform if the geologic and stratigraphic features to be defined can be correlated from one boring or sounding* location to the next with relatively smooth variations in the thicknesses or properties of the geologic units. An occasional anomaly or a limited number of unexpected lateral variations may occur. Uniform conditions permit the maximum spacing of borings for adequate definition of the subsurface conditions at the site.

We found no evidence of significant horizontal variations in the thickness or properties of the soil layers in the pad emplacement area, so it is appropriate to characterize the PFSF site as "uniform" and thus, as Reg. Guide 1.132 suggests, a maximum spacing of borings is sufficient for the adequate characterization of the subsurface conditions. Indeed, there is no reason to believe that a denser set of borings would have yielded any different results from the ones we obtained. Moreover, for those analyses that required soil properties such as strength and compressibility as inputs, PFS generally used the least favorable value of each of the measured properties (e.g., lowest peak strength and highest compressibility) of the subsoil from the weakest soil layer (Layer 2) to represent the *entire* top thirty feet of soil. (The only exception to this was that a weighted average strength, based on the increase in strength noted in the cone penetration tests that were performed within the CTB area, was used in the bearing capacity analysis of the CTB because of the large size of the foundation mat relative to the thickness of the upper 30 feet of soil.) In addition, even if undetected pockets of subsoil existed in which the soil strength was lower than the value used in the design, the existence of such discrete pockets of weak soils would not adversely impact the validity of the PFS analyses because the foundations for the cask storage pads and the Canister Transfer Building are such wide foundations that the superstructure loads are distributed over a large soil volume. Thus, it is the average soil strength, rather than the strength at discrete points, that determines the foundations' behavior.

Q21. Subsection C.2.a of the Unified Contention asserts that PFS's sampling and analysis are inadequate to characterize the site and do not demonstrate that the soil conditions are adequate to resist the foundation loadings from the design basis earthquake in that the Applicant has not performed continuous sampling of critical soil layers important to foundation stability for each major structure as recommended by Reg. Guide 1.132 Section C6, Sampling. Is this a valid concern?

A21. (PJT) No. Again, the recommendations in Reg. Guide 1.132 are not applicable to Part 72 facilities, and the applicable guidance in NUREG-1567 does not call for any particular method of sampling. Moreover, the State's allegations are in error in several respects. First, in two instances we took a series of samples for testing throughout the first 30 feet of soil, so we did conduct "continuous sampling" of the critical soil layers. As discussed earlier, we needed to go no further down with our sampling because the soils beneath 30 feet or so consist of very dense sands or silty sands overlying very dense silts; these soils have great strength.

Also, PFS obtained, through standard penetration testing, samples of all soil strata, from the ground surface to depths as great as 226.5 feet below the ground surface. (This depth was determined based on recommendations provided in Appendix C of Reg. Guide 1.132). At such depths, the soils are extremely dense. From the standpoint of geotechnical engineering and the design of foundations for the site's structures, proceeding further down with the sampling (arguably to bedrock, which is many hundreds of feet below the surface) is unnecessary. Reg. Guide 1.132 states at p. 1.132-21: "Where soils are very thick, the maximum required depth for engineering purposes, denoted d_{max}, may be taken as the depth at which the change in the vertical stress during or after construction for the combined foundation loading is less than 10% of the in situ effective overburden stress." At the PFSF, the maximum depth d_{max} beyond which no additional sampling is required in accordance with the Reg. Guide's recommendations is 226.5 feet.

Q22. Subsection C.2.b of the Unified Contention faults the laboratory testing program carried out by PFS for being based on an insufficient number of tested samples, and for failing to include strain-controlled cyclic triaxial tests and triaxial extension tests as part of the laboratory shear strength testing program. Do you agree with the State's assertion that the number of tested samples was insufficient?

A22. (PJT) No. All of the data acquired during the various soils investigations conducted at the PFSF consistently indicate that the subsurface profile at the site is fairly uniform and that the area of concern, from a geotechnical perspective, is the Layer 2 soils. Our testing has concentrated, therefore, on determining the strengths and compressibilities of the Layer 2 soils, and our analyses have conservatively used these lower-bound strengths and upper-bound compressibilities in designing and assessing the performance of these foundations.

Q23. Did PFS perform cyclic triaxial tests?

A23. PFS did perform cyclic triaxial tests in the form of stress-controlled cyclic triaxial tests. The purpose of these tests was to determine whether the soils will likely deform under repeated, cyclic earthquake loading. The stress-controlled cyclic triaxial tests that were performed by PFS show very little deformation, indicating no significant reduction in shear strength, even after 500 cycles of loading (versus about 8 to 15 for the PFS design earthquake).

Q24. What other cyclic triaxial tests does the State contend PFS should have performed?

A24. The State contends that PFS should also have conducted strain-controlled cyclic triaxial tests. These tests are intended to measure the dynamic properties of the soils – the shear modulus vs. shear strain (also referred to as the shear modulus degradation curve, because the shear modulus decreases (i.e., "degrades") for higher levels of shear strain) and the damping vs. shear strain – at high shear strain levels.

Q25. Would you please define what you mean by "shear modulus," "damping" and "shear strain"?

A25. For shear forces, that is forces applied on the horizontal plane, the shear modulus is a measure of elasticity, defined as the ratio of the force (stress) applied to the resulting deformation (strain). If the forces are applied vertically, the ratio of applied stress to resulting strain or deformation is known as the Young's modulus.

Damping is a measure of the amount of energy that is dissipated by a body (in this discussion, a soil sample) due to the dynamic excitation applied to it (in this case, during a test.) Shear strain is the straining that occurs as the sample resists application of a shear stress; axial strain is straining that occurs as the sample resists application of an axial stress.

Q26. Is the State's criticism valid?

A26. No. PFS performed resonant column tests, which achieved the same objectives sought by the State. Resonant column tests are a form of strain-controlled, cyclic triaxial testing (although not the same type of strain-controlled cyclic triaxial test referred to by the State). The resonant column tests are in fact the only form of strain-controlled cyclic triaxial testing that is recommended in Appendix B, "Laboratory Test Methods for Soil and Rock," of US NRC Regulatory Guide 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants" ("Reg. Guide 1.138") for use in developing curves of shear moduli and damping versus shear strain.

The resonant column test results can also be easily extrapolated to establish the high-strain behavior of the PFSF site soils. For example, if one compares the resonant column test results (included in Attachment 6 of Appendix 2A of the SAR) for Sample U-3C, obtained from a depth of about 8 feet in Boring CTB-1 and tested to shear strains as high as 0.07%, with those for Sample U-7C, obtained from a depth of about 20 feet in Boring CTB-1, and tested to shear strains as high as 0.15%, it is evident by looking at the plots of G/G_{max} and damping vs. shear strain from the two sets of tests that they are very similar; and therefore, it is reasonable to extrapolate the results from the testing of Sample U-3C along the same curves as those measured in the resonant column testing of Sample U-7C. Moreover, the curves that depict the test results have the expected, characteristic shape of plots of moduli and damping vs. shear strain, providing further evidence that minor extrapolation of the data from testing Sample U-3C is reasonable.

The modulus degradation and damping curves are used as input to the site response analyses, which were performed by Geomatrix in PFS Calculation 05996.02-G(PO18)-2-1. The Geomatrix results indicate that the greatest effective shear strains occur for the Layer 2 soils (depths of 5 to 12 feet). For this layer, the average effective shear strains range between 0.04% and 0.13%. These values are within the range of strains measured in the resonant column tests, which confirms that the results of the resonant column tests adequately encompass the appropriate range of effective strains for these soils for the design earthquake. Therefore, strain-controlled cyclic triaxial tests to measure shear moduli and damping at higher levels of strain than were measured in the resonant column tests are not required.

Q27. What is the State's claim with regard to triaxial extension tests and how do you respond to it?

A27. The State also contends that PFS should have conducted triaxial extension tests for use in assessing the bearing capacity of the Layer 2 soils. In this form of the triaxial test, the specimen is failed in axial tension by decreasing the vertical load on the specimen while maintaining a constant cell pressure so that the specimen ultimately fails in extension. However, such tests typically are not performed to assess the bearing capacity of foundations, nor are they mentioned in Appendix B, "Laboratory Test Methods for Soil and Rock," of Reg. Guide 1.138. Such tests typically are used to assess situations where foundation soils are unloaded, such as at the base of deep excavations. They also are sometimes used to determine the strength applicable for soils at the toe of slopes that might be subject to a deep, circular arc-type failure. These situations are not present at the PFSF site, which is essentially level and will require only very shallow excavations.

Q28. In section C.3.a of the Unified Contention, the State asserts that PFS has not adequately described the stress-strain behavior of the native foundation soils under the range of cyclic strains imposed by the design basis earthquake. Would you please explain the concern expressed by the State in this paragraph and respond to it?

A28. (PJT) This concern is related to the one I just discussed. The State claims that PFS has not performed strain-controlled, cyclic triaxial testing at large strains to show that the shear modulus and damping values used in development the design basis ground motion are appropriate. However, as indicated earlier, the shear strains imposed on the specimens in the resonant column tests that PFS performed were higher than the effective shear strains that the soils will experience during the design basis earthquake. The resonant column test specimens obtained from a depth of 8 feet were not subjected to shear strains quite as high as those expected at that depth in the profile due to the design basis earthquake; however, the shear strains imposed on the specimen of similar soil from a depth of ~20 feet were as high as 0.15%, a value that exceeds the average effective shear strain determined at any depth in the profile in the site response analyses included in Calculation 05996.02-G(PO18)-2, Rev. 1.

Moreover, the modulus-degradation and damping vs. shear strain data from the two sets of resonant column tests are very similar and follow expected trends based on historical data of this type; therefore, it is appropriate to extrapolate these data to encompass the slight increase in the shear strain above the maximum shear strain measured for the specimens obtained at a depth of ~8 feet.

V. USE OF SOIL CEMENT TO IMPROVE SUBSURFACE CONDITIONS

Q29. What is soil-cement?

A29. (AEZW, PJT) Soil cement is a material produced by blending, compacting and curing a mixture of soil, portland cement, other possible admixtures, and water to form a hardened material with specific engineering properties. Soil cement typically has far greater strength than that of the soil that is its main constituent.

Q30. Are all soils suitable for the formulation of soil cement mixtures?

A30. (AEZW, PJT) Almost all types of soils can be used in the formulation of soil cement. The exceptions to this include organic soils and poorly reacting sandy soils, which do not exist at the PFSF site, and highly plastic clayey soils, which will not be used to make soil cement at the PFSF site. There are tests to determine the suitability of soils for the construction of soil cement, including primarily the durability tests, ASTM D559 and 560, the wet/dry and freeze/thaw tests, as well as the compression tests, ASTM D1633. These tests are included in the soil cement testing program that PFS has underway.

Q31. Are the properties of the soil a factor in the manner in which soil cement is prepared?

A31. (AEZW) Yes. Given a desired set of soil cement properties, the mixture of materials that go into constructing the soil cement will differ depending on the soil properties. However, there is usually little difficulty in obtaining a particular set of soil cement properties, and the question is one of varying the proportions of the ingredients. For example, fine-grained soils generally require a higher proportion of cement than other soils in order to achieve a desired strength.

Q32. The term "cement-treated soil" has sometimes been used in this proceeding to denote a different material than soil cement. What is the difference between the two terms?

A32. (AEZW) In general, referring to a particular mixture as a "soil cement" or as a "cement-treated" soil is a function of the durability of the mixture of soil, portland cement, and/or other admixtures that has been formulated. Mixtures with greater degrees of stabilization and/or durability are generally referred to as soil cement, as opposed to cement-treated soil. Soil cement is typically expected to be able to pass durability tests that measure the ability of the stabilized soil to retain its properties after long periods of exposure to the elements. When addressing both soil cement and cement-treated soils, I shall refer to them as cement stabilized soils.

Q33. What are some of the industrial uses of soil cement?

A33. (AEZW) The most frequent use of soil cement has been as a base material underlying bituminous and concrete pavements. Due to its properties, however, soil cement has a wide-range of uses, including slope protection for dams and embankments; liners for channels, reservoirs and lagoons; and, as in the case here, for foundation stabilization.

Q34. What use does PFS intend to make of soil cement at the PFSF site?

A34. (PJT) The topmost layer of soil at the PFSF site is a layer of loose eolian silt. This eolian silt layer would need to be removed and replaced with some other material to provide a suitable foundation subgrade for the pads, as well as for the areas surrounding the pads. Mixing cement with these soils allows them to be utilized as part of the construction of the facility, instead of wasting the soil materials and replacing them with structural fill.

The use of soil cement at the PFSF site serves three specific purposes. In the area directly underneath the concrete pads upon which the storage casks rest, soil cement is to be used as a cohesive material that will be strong enough to resist the sliding forces generated by the design basis earthquake. The soil cement will provide bonding with the bottom of the concrete pad above it and with the clay soils beneath, so as to transfer the horizontal earthquake forces downwards from the pad and into the underlying clay soils.

Soil cement is also to be used in the area around and between the cask storage pads. There, the function of the soil cement is to support the weight of the transporter vehicle that is used to deliver storage casks to the pad area. Again, soil cement was chosen so that the soil materials would not need to be wasted and replaced with structural fill.

Finally, soil cement is to be placed around the Canister Transfer Building foundation mat, extending outward from the mat a distance equal to the associated mat dimension, to provide additional passive resistance against sliding forces in the event of a design basis earthquake. (Passive resistance is a term that refers to the ability of soils to resist horizontal forces, which in this case, are the result of earthquake forces.)

Q35. Is soil cement suitable for each of the functions assigned to it in the PFSF design?

A35. (AEZW) Yes. The PFSF design is relying on the cement stabilized soils to improve the shear and compressive strengths of the surficial native soils at the site. Soil cement has been used to improve these specific soil properties for over half a century.

Q36. Are the engineering functions that the soil cement will serve at the PFSF analogous to the uses soil cement has been given in other projects?

A36. (AEZW) While the specific application of soil cement to an ISFSI is new, the type of foundation stabilization that is proposed is not. Soil cement was used as a massive fill to provide foundation strength and uniform support at Koeberg, South Africa, for example, where an 18 foot thick layer of saturated sand under two 900-MW nuclear power plants was replaced with soil cement. In that particular case, the soils were prone to liquefaction and the soil cement was designed to provide enough shear strength to resist cyclic shear stresses due to an earthquake and, thus, prevent liquefaction. In the PFSF design, the soil cement provides increased shear strength to resist the shear stresses imposed on the cask storage pads by the design earthquake. In both instances, the design relies on the compressive and shear strength of the soil cement to stabilize the foundations.

Q37. What are the design requirements for the soil cement to be placed in each of the areas you mentioned?

A37. (PJT) The soil cement underlying the pads will have a minimum unconfined compressive strength of 40 pounds per square inch (psi). As discussed earlier, given the relatively low strength of this mix, it is referred to as "cement-treated soil" instead of soil cement. This cement-treated soil is required to have a thickness no greater than 2 feet and have a modulus of elasticity or Young's modulus (that is, a vertical stress to strain ratio) less than or equal to 75,000 psi. This modulus value is achievable with cement-treated soils.

The soil cement to be placed around and between the cask storage pads will have a thickness of 28 inches (3 feet height of the pads, minus the top 8 inches, which will be filled with compacted aggregate). This soil cement adjacent to the pads is expected to have a minimum unconfined compressive strength of at least 250 psi, in order to meet the durability requirements (wet/dry and freeze/thaw), since it will be within the frost zone.

The soil cement to be placed around the CTB will have a thickness of 5 feet (plus 8 inches to be filled with aggregate). It also is expected to have a minimum unconfined compressive strength of at least 250 psi, in order to provide the passive resistance to sliding required and to meet the durability requirements (wet/dry and freeze/thaw), since the upper half of it will be within the frost zone.

The aggregate to which I am referring is a coarse aggregate, such as crushed stone, that is to be placed and compacted to be flush with the top of the pads to permit easy access by the cask transporter.

Q38. How will PFS develop an appropriate soil-cement mix for each of the applications you just described?

A38. (PJT) The appropriate soil-cement formulation for each of the applications will be established by means of a program of laboratory tests. A laboratory testing program is being performed in accordance with a document entitled Engineering Services Scope of Work for Laboratory Testing of Soil-Cement Mixes, ESSOW 05996.02-G010 (2001) ("Laboratory Testing ESSOW") (PFS Exh. GGG).

Q39. What are the elements of the soil cement test program being conducted by PFS?

A39. (PJT) The Laboratory Testing ESSOW sets forth a series of tests to be conducted in several phases that will include soil index properties, moisture-density tests, durability tests, and other tests. Additional tests will also be conducted beyond those defined in the Laboratory Testing ESSOW, particularly the direct shear tests that PFS is committed to performing to demonstrate that adequate bond strength exists at the interfaces between the in situ clay and the cement-treated soil and between the cement-treated soil and the bottom of the cask storage pads.

Q40. Would you please describe the index property tests?

A40. (PJT, AEZW) The index property tests determine basic properties of the site soils, such as water content, liquid limit, plastic limit, particle size, etc. Each of these tests is conducted in accordance with well-established industry standards and procedures. The water content of the soils is determined in accordance with ASTM D2216. The Atterberg limits (liquid limits and plastic limits) of the soil are measured according to ASTM D 4318. The sieve analysis test is used to determine the gradation of the particle sizes in the soil samples, in accordance with ASTM D422 and D1140. The hydrometer analyses are conducted in accordance with ASTM D422 to measure the percentages of various clay-size particles in the soils.

These tests provide a basic understanding of the properties of the soil, primarily the moisture contents, the Atterberg limits, and the particle gradation as determined by sieve analysis and hydrometer analysis. Knowing these soil properties for these soils permits comparisons of results of the moisture-density, durability, and strength tests of soil cement specimens from PFSF with empirical data available in the literature that has been developed since the early part of the 1900s.

Q41. What tests are conducted after the index property tests?

A41. (PJT) After the completion of the index property tests, moisture-density tests are conducted in accordance with ASTM D558. This is an appropriate second step in testing. These tests establish, for each soil-cement mixture, the relationship between the moisture content of the mixture and the resulting density when the mixture is compacted. The moisture-density tests establish the optimum moisture content and maximum density for molding laboratory test specimens. This provides data used in formulating a range of soil cement mixtures to be subjected to further testing, to determine which mixes have the optimal combination of properties.

Q42. What tests will be performed on those mixes that have the optimal combination of properties?

A42. (PJT) The next series of tests to be performed are the durability tests. These tests, known as "wet-dry" and "freeze-thaw" tests, determine the durability of soil cement specimens subjected to repeated cycles of exposure to the elements during extreme conditions. For example, the wet-dry tests, which are conducted in accordance with ASTM D559, are used to determine moisture/volume changes and soil cement losses due to repeated exposures to inundation and drying. The freeze-thaw tests, conducted in accordance with ASTM D560, similarly evaluate moisture/volume changes and soil cement losses due to alternate cycles of freezing and thawing.

Successful completion of the durability tests establishes that the soil cement mixture tested is adequate to provide a durable soil cement mix, one that will not lose compressive strength over time due to the effects of weather and normal wear and tear.

The cement-treated soil to be placed under the cask storage pads will not be subjected to durability tests because it is to be located beneath a three-foot thick concrete pad and therefore will not exposed to the elements. The cement-treated soil also will be beneath the depth of frost penetration at the PFSF site and, thus, will be immune from freezing and thawing cycles.

Q43. What additional tests will be performed on the soil cement mixtures that pass the durability tests?

A43. (PJT) For those soil cement mix formulations shown to meet durability tests, compressive strength tests will be performed on cured test specimens to determine whether the formulations meet the design requirements for compressive strength. These tests will be conducted in accordance with ASTM D1633 and D558. If the compressive strength of a soil cement sample is determined to be adequate, the soil cement mixture will be deemed appropriate for use at the PFSF.

The test program may include other tests, such as permeability tests and splitting tensile strength tests. However, the design and performance of the foundations is not dependent on these properties.

The cement-treated soil will also be subject to direct shear tests to confirm that the bond at the interfaces between the concrete bottom of the cask storage pad and the cement-treated soil, the bond at the interfaces between lifts of cement-treated soil, and the bond at the interfaces between cement-treated soil and the in situ clayey soil, exceed the strength of the clay soils at the site. Such confirmation will demonstrate that the cement-treated soil provides sufficient resistance against seismic sliding forces.

Q44. What standards will be used to assure the proper performance of the various tests?

A44. (PJT) The Laboratory Testing ESSOW cites Reg. Guide 1.138 as controlling the performance of the tests, as well as nearly twenty standards issued by the American Society for Testing and Materials and the Portland Cement Association. More generally, the guidance and recommendations in the industry standard publication "State-of-the-Art Report on Soil Cement," American Concrete Institute Report ACI 230.1R-90 (1998) ("State-of-the-Art Report") (PFS Exh. HHH) will be followed with respect to mix proportioning, testing, construction and quality control. Dr. Wissa is one of the developers of the State-of-the-Art Report. State's soil cement expert, Dr. James Mitchell, endorses the use of the procedures contained in the State-of-the-Art Report. Mitchell Dep., PFS Exh. III, at 46-47, 49-50.

Q45. Dr. Wissa, do you have an opinion on the adequacy of the soil cement laboratory testing program developed by PFS?

A45. (AEZW) Yes. Based on my review of the proposed program and the standards and methodology it includes, I am of the opinion that the program, if properly implemented, will lead to the identification of suitable soil cement and cement-treated soil mixes and construction specifications that will meet the specified design requirements and will give adequate performance for the life of the PFSF.

Q46. What is the current status of the soil cement laboratory testing program?

A46. (PJT) PFS has retained a contractor, Applied Geotechnical Engineering Consultants, Inc. ("AGEC"), to conduct the laboratory testing program in accordance with the Laboratory Testing ESSOW. AGEC has provided preliminary test results for the index property tests and the moisture-density tests. AGEC also performed a set of durability tests, but my review determined that these tests failed to demonstrate the durability of the tested samples, likely due to insufficient compaction of the test specimens prior to performance of the tests. The test program is currently on hold, pending determination of the causes for the failure of the durability tests that were performed by AGEC.

Q47. Dr. Wissa, how would you characterize the results of the laboratory testing program conducted so far?

A47. (AEZW) The index property tests completed to date appear to be reliable and adequate to describe the on-site surficial soils that will be stabilized with cement. On the other hand, these and other soil cement test results are preliminary.

I fully expect that when the tests are resumed to completion they will identify several acceptable soil cement mixes, from which one or more can be selected for further testing. Thus, I see nothing so far that would preclude the site soils from being incorporated into a suitable soil cement mixture.

Q48. Do you foresee any difficulty in PFS implementing a successful soil cement construction program?

A48. (PJT, AEZW) No. The soil cement design requirements have been defined by S&W and do not provide any special engineering difficulties. The compressive strengths of the soil cement (250 psi and 40 psi) are not difficult to obtain for soil cement generally. The State's soil cement expert agrees. Mitchell Dep., PFS Exh. III, at 41, 53-54, 90-91, 173-176. The laboratory testing program in place to design a soil cement mix to meet those requirements is set forth in the ESSOW and is in accordance with well-established regulatory guidance and industry standards. That program is in the process of being implemented.

Following completion of the testing phase, procedures for placement and treatment of soil cement will be developed. For example, the two-foot thick layer of cement-treated soil underlying the cask storage pad will be constructed of lifts approximately six-inches thick. This technique will allow adequate compaction of the cement-treated soil layer using low ground pressure equipment. As discussed in the SAR, the time between placing lifts will be minimized to the extent practicable. In any case, PFS will utilize the techniques described in DeGroot, G., 1976, "Bonding Study on Layered Soil Cement," REC-ERC-76-16, U.S. Bureau of Reclamation, Denver, CO, September 1976 (e.g., dry cement or cement slurry between lifts, roughening of surface before placement of soil cement lift) for enhancing the bond between fresh soil cement and soil cement that has already set to ensure sufficient bonding is achieved.

Thus, all the elements of the program exist or can be readily developed in accordance with established industry standards and practices.

Q49. Is the use of soil cement in the manner in which PFS intends a novel technology?

A49. (AEZW) No. The design, placement, testing and performance of soil cement are well-established technologies. There is also precedent in the industry for using soil cement for foundation stabilization in the manner proposed by PFS. The fact that the use of soil cement is an established technology provides reasonable assurance that the program proposed by PFS can be executed successfully.

VI. RESPONSE TO THE STATE OF UTAH'S CLAIMS IN SECTIONS C.3.b, C.3.c AND C.3.d OF UNIFIED CONTENTION UTAH L/QQ

Q50. In Paragraph C.3 of the Unified Contention, the State alleges several concerns about the use of soil cement at the PFSF. Are you familiar with those allegations?

A50. (AEZW, PJT) Yes.

Q51. What is your general response to the State's allegations?

A51. (AEZW, PJT) In general, the concerns raised by the State and its witnesses are well-known potential problems that can be anticipated and dealt with in the testing and construction phases of the program. In fact, the State's soil cement expert, Dr. James Mitchell, agrees that the concerns raised by the State are issues that he would like to see resolved through testing, but are not technically unachievable. Mitchell Dep., PFS Exh. III, at 186.

Q52. In subsection C.3.b of the Unified Contention, the State claims that PFS has not shown by case history precedent or by site-specific testing and dynamic analyses that the cement-treated soil will be able to resist earthquake loadings for the CTB and cask storage pad foundations. How do you respond to the claim that there is no case history precedent for the manner in which PFS proposes to use soil cement?

A52. (AEZW) While the application of soil cement in the design of the PFSF has some particular features that may be uncommon (mainly the need to maintain the cement-treated soil's Young's modulus at or below 75,000 psi), the use of soil cement for foundation stabilization is not. As discussed above, there is ample precedent for the use of soil cement for foundation stability. Some of the instances of the use of soil cement are described on pages R-2 through R-7 of the State-of-the-Art Report. In particular, there is an analogous instance in which soil cement was used to improve the seismic performance of the subsoil at a nuclear power plant site in Koeberg, South Africa, and increase the soil strength against earthquake dynamic loads. While the types of soil were different at both sites, the application is essentially the same for which soil cement is to be used at the PFSF. At the Koeberg, South Africa nuclear power plant, the shear strength of loose sandy soils was increased by the use of soil cement to preclude potential liquefaction due to seismic shear stresses from the design earthquake. At PFS, the shear strength of the eolian silt is being increased by mixing it with cement to provide sufficient shear strength to resist seismic shear stresses due to the design earthquake. The ability of cement stabilized soils to withstand dynamic loads is being demonstrated every day in pavements where they are continuously being subjected to such loads from traffic.

Q53. What is your answer to the assertion that there has been no demonstration by site-specific testing and dynamic analyses that the soil cement to be used at the PFSF will be able to withstand the anticipated earthquake loadings for the CTB and the cask storage pad foundations?

A53. (PJT) With respect to the alleged lack of site-specific testing, I have described above in detail the soil cement testing program being conducted by PFS. The program has been formulated, the design criteria identified, the test standards, methodology and acceptance criteria specified, and some testing has been performed. PFS is committed to performing these tests, as well as tests that demonstrate that the necessary bonding can be achieved and that this bonding is achieved at the various interfaces that are important to providing the resistance to sliding of the cask storage pads. There is nothing else that is required in advance of licensing of the PFSF. These commitments are reflected in Section 2.6.4.11 of the SAR (PFS Exh. JJJ).

The dynamic analyses of the cask storage pads and the CTB are addressed in my testimony (and that of other PFS witnesses) with regard to Section D of the Unified Contention.

Q54. The State has asserted that "proof of design" testing needs to be conducted before the design is finalized and before construction can proceed to final design stage, contrary to PFS's plans. Is that so?

A54. (PJT, AEZW) No. It is unclear what the State means by "proof of design" testing. There is nothing questionable or requiring "proof" about the concept being proposed in the design of the PFSF. The properties of the soil cement are within well-established, attainable parameters, and will be achieved in accordance with standard industry procedures. The construction techniques that may be used to ensure proper placement and curing of the soil cement, and to prevent damage to the underlying soils, have been utilized in numerous construction projects. Likewise, the design functions of the soil cement and the properties relied on to perform those functions are not new. Thus, there is nothing in the design that has not already been proven.

As stated in Section 2.6.4.11 of the SAR, PFS has committed to developing a soil-cement mix design using standard industry practice, and has further committed to performing a soil cement testing program in accordance with specified industry standards. That program follows industry-accepted protocols designed to address environmental factors that may affect long-term soil cement performance including, among others, the methodology set forth in industry codes ASTM D 558 (1996); ASTM D 559 (1996) and ASTM D 560 (1996). Design and implementation of a soil cement and cement-treated soil application that takes into account the results of the referenced soil cement testing program will assure adequate performance of the soil cement and cement-treated soil over the 40-year life of the facility. Thus, PFS has specified the tests it intends to perform and the acceptance criteria for the test results.

Once the test program has demonstrated the achievability of the design criteria, PFS will lay out a program to demonstrate field construction techniques that achieve the required bond strength in the field. As stated in the SAR, PFS also is committed to performing field testing during construction to demonstrate that we have, indeed, achieved in the field the bond strengths that are required.

These commitments are sufficient to provide reasonable assurance that the soil conditions at the PFSF will be adequate for the foundation loading that will be imparted by the design basis earthquake.

Q55. In subsection C.3.c, the State asserts that the Applicant has not considered the impact to the native soil caused by construction and placement of the cement-treated soil. Is PFS addressing this concern?

A55. (PJT, AEZW) Yes. We have always understood that the soil cement construction techniques to be used could potentially impair the surface of the native soils under the soil cement or the cement-treated soil layer ("subgrade") if it is not properly protected. So we intend to demonstrate at the start of construction that the techniques we allow the contractor to use will not have an adverse impact on the strength of the soils.

There are two main mechanisms by which the underlying soils may be disturbed during the placement of soil cement: exposure to the elements and deformation ("remolding") by construction equipment. Neither mechanism provides an insurmountable problem.

Exposure to the elements will be minimized through the use of proper construction procedures and scheduling. Those procedures will require that soil excavation not take place until the first lift of soil cement is ready to be placed. That first lift of soil cement can be pushed out onto the surface of the subgrade with low ground pressure equipment that won't have an adverse impact on the underlying clay. Once in place, the first lift of soil cement will shelter the underlying soil from rain.

If there is a heavy rainfall during construction, one of several available options will be utilized to remove excess moisture from the soil. One option is to let it dry out before placing soil cement over it. It is also possible to accelerate drying by applying quicklime to the exposed surface.

The main area of concern with respect to remolding of the native soils is with respect to the cask storage pads, for which the cohesive strength of the clay under the cement-treated soil is required to provide sliding resistance. However, the pads are only about 30 feet wide. There is construction equipment that can be located on either side of the pads at the placement locations and reach out to make a cut to the final subgrade surface, if necessary. All other construction equipment can be kept off of the exposed subgrade. Through these means, the subgrade can be sufficiently protected during the soil cement installation.

In short, there are a number of construction techniques available to prevent damaging the native soils beneath the cask storage pads, and we intend to use appropriate measures to prevent such damage. We will also test the bond strength achieved at the critical interfaces, which will prove the adequacy of the construction techniques being employed.

Q56. In subsection C.3.c, the State also asserts that the Applicant has not analyzed the impact to settlement, strength and adhesion properties caused by placement of the cement-treated soil. What is your view on these asserted impacts?

A56. (AEZW, PJT) In this issue, the State expresses a concern that the concrete pads and the soil cement to be placed underneath them at the site may serve as an impermeable barrier that will trap moisture in the underlying soils, but it does not appear that such a problem, if existing, will be significant due to the great depth to the groundwater table at the site and because of the semiarid conditions out in Skull Valley.

Q57. State witnesses have asserted that moisture may migrate to the clay soils beneath the cement-treated soil layer and reduce the strength and adhesion properties of those soils. Do you think moisture accumulation in the soils beneath the cement-treated soil layer is likely?

A57. (AEZW, PJT) No. The placement of a cement-treated soil layer and the presence of the cask storage pads may affect the mechanism of moisture migration from the soils adjacent to and underneath the cement-treated soil layer. However, water vapor tends to move from warmer areas to colder areas in response to a drop in air pressure as the moisture condenses. At the PFSF, the storage casks on top of the pads will provide a source of heat that will be conducted down through the concrete pad and underlying cement-treated soil. Therefore, the area beneath the pads on which casks rest will be warmer than surrounding areas. Moisture migration, therefore, will be away from the cement-treated layer beneath the pads to the surrounding areas due to heat gradient effects, as the State's expert Dr. Mitchell recognizes. Mitchell Dep., PFS Exh. III, at 112.

Holtec's "HI-STORM Thermal Analysis Report for PFS," HI-992134, analyzes the thermal characteristics of the casks supported on the pads at the PFSF site. The analyses indicate that the bottom of the storage casks could be as high as 195°F; however, the average temperature for the surface of the pad will be 120°F, which is approximately fifty degrees warmer than the average ambient temperature at PFSF throughout the year. This temperature differential will cause a warming of the cask storage pads, and the transfer of heat through the concrete in the pads towards the underlying soil cement. This heat transfer will in turn cause water to be transported away from the warmer soils underneath the pad to the cooler soils adjacent and beneath them. Thus, there will not be an increase, but a reduction in the water content of the soils underlying the cask storage pads once the casks are placed on the pads, which if anything, is expected to increase the strength of the clayey soils underlying the cement-treated soils.

Q58. In section C.3.d of the Unified Contention, the State argues that PFS has not adequately addressed several possible mechanisms that may crack or degrade the function of the soil cement or cement-treated soil over the life of the facility. The first such alleged mechanism, set forth in subsection C.3.d(i), is shrinkage and cracking that normally occurs from drying, curing and moisture content changes. How serious a problem is shrinkage and cracking of soil cement and cement-treated soil?

A58. (AEZW, PJT) Shrinkage cracking is a normal phenomenon in soil cement and cement-treated soil. Shrinkage cracking has been extensively investigated over the years and shown to not generally affect the performance of cement stabilized soils. Steps can be taken during the curing and placement process to minimize the amount of shrinkage and the potential for crack formation. For example, there are shrink resistant types of cement – known as Type K cements – which can inhibit the formation of shrinkage cracks. Also, during curing, a sealing coat (such as a geomembrane) can be put on the soil cement, to minimize the formation of cracks.

In our professional opinion, the existence of cracks will not adversely affect the ability of the soil cement and cement-treated soil to perform their design functions. The design does not rely on the cement-stabilized soil layers to transmit tension, but on lateral compression and shear. The ability to transmit compression and shear is not affected by shrinkage cracks, which develop in a vertical direction. If required, the amount of lateral movements needed to close the cracks in order for the soil cement to resist compressive forces can be substantially reduced by filling the cracks with grout after they have developed.

Q59. Why do you believe that the existence of cracks will not adversely affect the performance of the soil cement and the cement-treated soil?

A59. (AEZW, PJT) The cracks that form in soil cement and cement-treated soil due to shrinkage and curing are very narrow (fractions of an inch wide), occur at random locations, and are vertically propagating. Such cracking does not impair the compressive strength of the soil cement or the cement-treated soil.

With respect to the passive resistance of soil cement, which is relied upon for providing resistance to sliding of the CTB, such resistance is not diminished by the presence of vertical cracks. All of these cracks would have to be lined up parallel to the edge of the foundation to have the greatest impact on the passive resistance; however, such a lining up is highly unlikely because of the random orientation of the cracks. The presence of these cracks will not affect the magnitude of the horizontal resistance that the soil cement is capable of providing. The aggregate width of the cracks is small (on the order of few inches), and the potential effect of such cracks relates to the amount of horizontal displacement required to reach full passive resistance; thus, the cracks have no effect on the amount of sliding resistance available from the soil cement. In addition, PFS has the opportunity to seal these cracks in the soil cement surrounding the CTB, where the soil cement is relied upon to provide passive resistance, prior to placement of the layer of compacted aggregate in the area.

A slight horizontal movement may be required to close such vertical cracks if they are aligned nearly parallel to edge of the foundation before the compressive strength of the soil cement can once again provide the full resistance. Such a horizontal movement of the CTB is of no consequence because there are no safety-related connections between the CTB and the surrounding yard area.

Q60. The State witnesses assert that tensile loads may tend to impart bending stresses on the soil cement and the cement-treated soil, and that the presence of cracks will further reduce whatever little resistance the soil cement and the cement-treated soil may have to tensile loads. Is this a valid concern?

A60. (AEZW, PJT) No. The cement-treated soil layer under the cask storage pads will be subjected to very limited bending stresses because the heavily reinforced concrete pads will carry most of those stresses. In addition, the design function of the cement-treated soil is to transmit shear stresses to underlying strata and not for resistance to bending.

For the soil cement surrounding the CTB, any bending of the soil cement cap is only going to change the shape of the gaps of existing shrinkage cracks. Under bending loads, the width of the gap across the crack at one of its ends will increase, while at the opposite end it will decrease. Thus, there will be no permanent effect on the soil cement cap or its ability to provide passive resistance against sliding of the CTB. As noted earlier, if a crack exists and the building exhibits forces that would cause it to tend to slide, then the soil cement will move to close the crack, after which the soil cement will still be able to provide the resistance that it needs to keep the building in place.

Q61. Another mechanism posited by the State in subsection C.3.d(ii) of the Unified Contention for the potential degradation in performance of the soil cement at the PFSF is potential cracking due to vehicle loads. Are vehicle loads potentially capable of causing cracks in the soil cement and the cement-treated soil at the PFSF?

A61. (PJT) No. The vehicles in question are the cask transporters that will move the storage casks from the CTB to their locations in the pad emplacement area. With respect to the soil cement layer around and between the cask storage pads and surrounding the CTB, PFS Calculation 05996.02-G(B)-18-1 demonstrated that a 2-feet thick layer of compacted structural fill would be sufficient for distributing the transporter loads down to the underlying clayey soils. That structural fill layer has now been replaced by approximately 5 feet of soil cement, which has an unconfined compressive strength that will exceed 250 psi, or 36 ksf. Such soil cement is several times stronger than the structural fill that it replaces.

The loading at the bottom of the transporter crawler tracks is less than 10 ksf. Thus, the soil cement (with a compressive strength of 36 ksf) provides a firm foundation for the transporter to travel, and it will not be subject to cracking due to the loads imparted by those vehicles.

Q62. Another mechanism posited by the State in subsection C.3.d(iii) of the Unified Contention for the potential degradation in performance of the soil cement at the PFSF is potential cracking resulting from a significant number of freeze-thaw cycles at the Applicant's site. Is this a valid concern?

A62. (PJT) No. As I explained earlier, the soil cement mixture to be used at the PFSF will have been subjected to durability tests that demonstrated the mixture's ability to withstand freeze-thaw and wet-dry cycles without degradation in performance. For many years, soil cement has been used for erosion protection of reservoir slopes and has proven to able to perform satisfactorily under far more severe environmental conditions than those applicable for the PFSF. With respect to the cement-treated soil under the cask storage pads, the top of the layer of cement-treated soil will be six inches below the frost level for the site; thus, it will not be exposed to freeze-thaw cycles. In addition, when storage casks are present, the cement-treated soil will be warmed by the heat released from the storage casks.

Q63. Another mechanism posited by the State in subsection C.3.d(iv) of the Unified Contention for the potential degradation in performance of the soil cement at the PFSF is interference with cement hydration resulting from the presence of salts and sulfates in the native soils. How can the presence of sulfates potentially affect the performance of soil cement or cement-treated soils?

A63. (AEZW) The presence of sulfates can have two potential deleterious effects on soil cement. First, sulfates may affect the properties of the soil cement itself. Second, sulfates can potentially affect soil cement by attacking the soil cement after placement. This may occur through soluble forms of sulfates in underlying soils being carried upwards to the soil cement layer by moisture migration.

Q64. Do you have any information on the presence of sulfate in the soils at the PFSF site?

A64. (PJT) Preliminary testing of the site soils for the presence of sulfates indicates that very low levels of sulfates are present in the eolian layer of soil that will be used to fabricate the soil cement or cement-treated soil. The preliminary testing for sulfates of soil samples from the PFSF site yielded the following results:

Summary of Sulfate Test Results

PFSF Soil Cement Testing Program

Test Pit No.	Sample No.	Depth (Feet)	Bucket	Water Soluble Sulfate (ppm)
1	1	0 – 2	1 of 4	65
4	1	0 – 2	4 of 4	
3	1	0 – 2	3 of 4	100
3	1	0 – 2	4 of 4	
2	1	0 – 2	3 of 4	530
13	1	0 – 2	n/a	560
14	1	0 – 2	n/a	
15	1	0 – 2	n/a	120
16	1	0 – 2	n/a	
5	1	0 – 2	n/a	110
6	1	0 – 2	n/a	140
7	1	0 – 2	n/a	375
8	1	0 – 2	n/a	< 10
9	1	0 – 2	n/a	210
10	1	0 – 2	n/a	250
11	1	0 – 2	n/a	430
12	1	0 – 2	n/a	110

I should note that the above table excludes the tests on two samples, drawn from depths of 2 to 4 feet, which showed higher levels of sulfates. These were likely

Layer 2, Upper Bonneville clays, which PFS does not intend to use for making soil cement or cement-treated soil.

Q65. What conclusions do you draw from those preliminary sulfate test results?

A65. (AEZW) The test results indicate that, for all the samples of the eolian soil material, the sulfate content is less than 600 parts per million. There should be no problem in constructing soil cement or cement-treated soil out of such material.

Although additional tests are necessary, it would appear that the potential presence of sulfates will not pose an obstacle to the hydration of the soil cement and the cement-treated soil. In any event, should sulfates be present in the soil in such high concentrations as might interfere with the hydration process, the problem would be evidenced by the failure of the soil cement test samples to pass the durability tests discussed above. For example, the presence of sulfate in the form of ettringite (calcium aluminum sulfate) can result in expansion of the ettringite over time in the soil cement mixture. This effect can be readily discernible in the testing program by monitoring strength gain as a function of curing time.

Should the presence of sulfates be determined to be a concern, there are a number of alternatives that can be implemented to address the problem, including: using a sulfate resistant cement, increasing the treatment levels, or conducting chemical treatment on the soil. For example, barium compounds can be added to the mix to immobilize the sulfates, or lime or lime ash can be added, since they will react with the sulfates before the sulfates can attack the cement. An increase in the cement content of the mixture, say from five percent to seven percent cement content, will also increase resistance to sulfate attacks. A certain amount of sulfate can only react with a certain amount of cement, so even if there is some cement loss due to sulfate attack, there would still be adequate cement to maintain the compressive strength required.

Additionally, because water will migrate away from the cask storage pads and the cement-treated soil layer for the reasons discussed earlier, soluble sulfates in the underlying soils would be precluded from reaching the cement-treated soil.

Q66. The last mechanism posited by the State in subsection C.3.d(v) of the Unified Contention for the potential degradation in performance of the soil cement at the PFSF is potential cracking and separation of the cement-treated soil from the foundations resulting from differential immediate and long-term settlement. Would you please address this concern?

A66. (AEZW, PJT) Our earlier general discussion of cracks and their limited impact on the performance of soil cement and cement-treated soil also applies to settlement cracks. We would add that settlement cracks occur when the foundation mat of a building or structure is loaded. As this happens, the soils adjacent to the foundation also experience increases in stresses, as the loading is distributed over a widening area as one moves deeper into the soil profile. Through this mechanism, the settlement that occurs in the soils adjacent to the foundations will tend to approximate the settlement level at the edge of the foundation, so that there will be no abrupt differential settlement at the joint between the edge of the foundation and the soil cement. Soil settlement will gradually diminish with increased distance from the edge of the foundations.

The resulting settlement profile will be dish-shaped, extending some distance away from the edge of the mat. Therefore, the differential settlement between the edge of the foundation and the surrounding soil will be minimal, and crack formation due to differential settlement will be inconsequential.

Q67. Does that conclude your testimony?

A67. (AEZW, PJT) Yes, it does.

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Bachelor of Arts, Engineering Science, Oxford University, Oxford, England, 1957.

Master of Science, Civil Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts, 1961.

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Doctor of Science, Geotechnical Engineering, Massachusetts Institute of Technology, Cambridge, Massachusetts, 1965.

PROFESSIONAL HISTORY:

1975 to Present	President and Chairman of the Board of Directors Ardaman & Associates, Inc., Orlando, Florida Responsible for the overall engineering and business activities of twelve offices in Florida and the Middle East with a staff of over four hundred. International consultant on earthen dams, industrial and mining waste disposal facilities, pavements, soil stabilization, geosynthetics, and construction materials. Guest lecturer at leading universities.
1977 to Present	Director and Senior Consultant Ardaman-ACE, S.A.E., Cairo, Egypt
1961 to Present	Principal and Chairman of the Board of Directors Geotechniques International, Inc., Middleton, Massachusetts Responsible for the design and development of geotechnical field instrumentation and specialized soil testing equipment.
1978 to 1983	Director and Senior Consultant, Saudi Geotechnical Services, Ltd., Jubail, Saudi Arabia
1973 to 1975	Senior Vice President and Chief Engineer Ardaman & Associates, Inc., Orlando, Florida
1965 to 1973	Principal Wissa Associates, Marblehead, Massachusetts Independent consultant on foundations, pavements, earthen dams, soil stabilization, and construction materials.

1969 to 1972 — Associate Professor of Civil Engineering, Dept. of Civil Engring.
Massachusetts Institute of Technology, Cambridge, Massachusetts

Taught graduate and undergraduate courses and conducted seminars in soil mechanics, soil behavior, pavements, soil stabilization, experimental soil mechanics, and instrumentation. Director of soils research laboratory. Supervised doctoral and master student theses and several $100,000.00 of sponsored research per year on soil behavior, soil stabilization and frost action. Developed and holds a patent on laboratory and field instrumentation.

1965 to 1969 — Assistant Professor of Civil Engineering, Dept. of Civil Engring.
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Taught graduate and undergraduate courses in soil stabilization, instrumentation, civil engineering materials, asphalt and Portland cement concrete, soil and materials testing. Director of soil stabilization laboratory. Supervised graduate thesis students and sponsored research in soil stabilization, asphaltic concrete, pavements and experimental soil mechanics.

1962 to 1965 — Instructor of Civil Engineering, Department of Civil Engineering
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Taught undergraduate and graduate course in experimental soil mechanics. Supervised research in soil stabilization.

1959 to 1962 — Research Assistant in Soil Engineering, Dept. of Civil Engineering
Massachusetts Institute of Technology, Cambridge, Massachusetts

Conducted research in soil stabilization, soil technology, and soil behavior.

1957 to 1958 — Junior Civil Engineer
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Designed prestress concrete bridge deck, supervised subsurface investigations and worked in soil testing laboratory.

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Fellow, American Society of Civil Engineers	Present
Member, Committee on Placement and Improvement of Soil	1969-1978
American Society of Testing and Materials	1966 - present
Member, Committee D-18 on Soil and Rock	
Association of Soil and Foundation Engineers	1978 - present
Boston Society of Civil Engineering	1968 - 1972
Member, Executive Committee, Geotechnical Section	
International Society of Soil Mechanics and Foundation Engineering, Member	1966 - present
Transportation Research Board, Member	1965 - 1994
Member, Committee on Soil-Bituminous Stabilization	1965 - 1978
Chairman, Committee on Soil-Bituminous Stabilization	1966 - 1975
Member, Committee on Soil-Cement Stabilization	1965 - 1975
Member, Committee on Flexible Pavement Design	1970 - 1973
Member, Committee on Soil and Rock Instrumentation	1976 - 1991
Member, Committee on Physicochemical Phenomena in Soils	1982 - 1994
Florida Engineering Society	1974 - Present
National Society of Professional Engineers	1974 - Present
Society of Mining Engineers of AIME	1975 - Present
American Concrete Institute	1985 - 2000
Member, Committee 230, Soil-Cement Stabilization	
Florida Institute of Phosphate Research	1985 - Present
Member, Technical Advisory Committee on Beneficiation	1985 - 1989
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American Society of Civil Engineers	1985 - Present
National Task Committee on Response to Disaster Situations	1985-1987
International Geosynthetics Society	Present

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