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-ISFSI
	)	
(Independent Spent Fuel Storage Installation)	)	

NRC STAFF'S OUTLINE OF PROPOSED KEY DETERMINATIONS FOR UNIFIED CONTENTION UTAH L/QQ (GEOTECHNICAL)

Part C. Characterization of Subsurface Soils

1. The site-specific investigations and laboratory analyses performed by PFS satisfy geotechnical site characterization requirement of 10 C.F.R. § 72.102(d) by showing that site soil conditions are adequate for the proposed foundation loading.

2. PFS satisfactorily classified the subsurface materials and identified lateral and vertical variations in the properties of those materials (including shear strength and compressibility). As such, it is not necessary that PFS follow the particular spacing of borings or perform continuous sampling as recommended in Reg. Guide 1.132.

3. The specific combination of field and laboratory tests performed by PFS provided the data needed to obtain the soil-strength parameter values used in its stability analyses of the storage pads and canister transfer building foundations. Additional "tested samples" and "strain-controlled cyclic triaxial tests or triaxial extension tests" are therefore not necessary.

4. PFS has described the stress-strain behavior of the native foundation soils in a manner adequate to support the various engineering analyses of the facility structures, systems, and components important to safety.

5. The soil cement around the storage pads and the cement-treated soil under the storage pads are not being relied upon to support any safety function of the pads.

6. PFS has committed to demonstrate through testing that the stiffness of the cement-treated soil under the pads will not exceed the specified design value (*i.e.*, a dynamic Young's modulus of 75,000 psi). PFS has also committed to demonstrate through testing that the soil cement around the CTB will have a minimum unconfined compressive strength of 250 psi.

7. The commitment by PFS to follow the applicable soil-cement standards in ACI 230-1R-90 provides further assurance that the proposed soil-cement layer around the CTB will provide the specified amount of lateral resistance.

8. Any impacts to the native soil caused by construction and placement of the cement-treated soil, including potential changes in the settlement, strength and adhesion properties of the native soils - assuming they occur - would not have an adverse effect on the safety of the proposed facility.

9. If cracking or other degradation of the soil cement/cement-treated soil in the vicinity of the storage pads were to occur, it would not have an adverse effect on the safety functions of the storage pads. Similarly, if cracking of the soil cement around the CTB occurs, it will not significantly affect the passive resistance of the soil cement; and any small lateral movement of the foundation will not impact CTB safety functions.

Part D. Seismic Design and Foundation Stability

1. PFS has properly demonstrated that the proposed PFS Facility structures and foundations have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake ("DBE"), and the seismic design and foundation stability of the proposed PFS Facility satisfy applicable regulatory requirements.

2. Calculations provided by PFS demonstrate that: (1) there are adequate safety margins against bearing capacity failure of the storage pads under combined static loads and potential dynamic loading from the design-basis earthquake; (2) potential sliding of the pads under seismic loading would not constitute a safety hazard; and (3) settlement of the pads does not present a foundation stability concern.

3. The Staff's review of the PFS site-specific analysis -- as well as the Staff's confirmatory analysis, performed by Dr. Luk -- indicate that the HI-STORM 100 storage casks will not tipover or collide due to a design basis ground motion.

4. The static and dynamic pad analyses performed by PFS demonstrate that the cask storage pads are adequately designed to resist the loads based on the site characteristics and environmental conditions during normal operations and during postulated off-normal and accident events. Further, the PFS structural analysis demonstrates that the storage pads are designed to withstand the effects of natural phenomena, such as earthquakes, without impairing the capacity to perform safety functions, in accordance with regulatory requirements.

5. The assumption of vertically propagating in-phase waves is reasonable at the proposed PFS site. Further, the stability of the casks will not be affected by non-vertically out-of-phase seismic waves that may occur at the site.

6. The assumption of a rigid storage pad by PFS will produce conservative results, since no energy will be absorbed in deformation of the storage pad, resulting in an upper bound estimate of the displacement response of the casks.

7. PFS has provided a realistic evaluation of the foundation pad motion with cement-treated soil under and around the pads in relation to motion of the casks sliding on the pads.

8. Lateral variations in the phase of ground motion and their effect on the stability of the storage pads and casks will be insignificant, as confirmed by the Staff's review of applicable Geomatrix calculations.

9. The Applicant's modeling of foundation soils under dynamic loading is consistent with the requirements of ASCE 4-98. The PFS modeling provides an accurate representation of the lowest frequency of response of the storage pads, and therefore accurately predicts the maximum displacement of the storage pads.

10. Taking into account both initial and creep deformations, the amount of deformation of the concrete pad is too insignificant to result in cold-bonding of the cask and storage pad, and it will not have any influence on the overall stability of the casks on the storage pads under seismic load conditions.

11. The potential for pad-to-pad interaction caused by sliding of the storage pads under seismic loading is not a concern, as the influence on the structural integrity of the storage pads and the stability of the casks will be minor given the low magnitude of force that can be transmitted through the soil-cement layer between the storage pads.

12. The time histories used by PFS in its non-linear analysis are consistent with NRC guidance and provide an adequate margin of safety with respect to the Applicant's analysis of the potential for cask tipover.

13. Fault fling is a potential issue for strike-slip faults, whereas the Stansbury and East faults at the proposed PFS site are normal faults.

14. The design of the CTB foundation proposed by PFS satisfies regulatory requirements with respect to the capability of the underlying soil to provide adequate support to the foundation. PFS has demonstrated the ability of the CTB, with its foundation, to perform its safety function and limit the impact on public health and safety. The CTB design satisfies applicable requirements, in that its performance is not influenced by the dynamic loading from the design basis ground motion.

15. The rigid mat foundation assumption used by PFS is conservative, in that no energy will be absorbed in deformation of the mat foundation, resulting in an upper bound estimate of the response of the CTB. The foundation damping assumption used by PFS is acceptable.

16. The amount of motion of the CTB calculated by PFS without inclusion of the soil/cement will be greater than the amount of motion if the additional restraint of the soil/cement was included in the soil impedance function. Thus, the omission of this factor by PFS in its calculation was conservative.

17. Cracking and separation of the soil-cement around the CTB will not adversely affect the ability of the structure to perform its safety function.

18. Dr. Vincent Luk of Sandia National Laboratories performed a confirmatory analysis on behalf of the Staff, of the seismic stability of the casks at the proposed Facility.

19. In the Staff's confirmatory analysis, three-dimensional coupled finite element models were developed, and seismic analyses were performed, to examine the dynamic and nonlinear behavior of the HI-STORM 100 casks at the proposed PFS Facility, including the soil-structure interaction effects during a seismic event.

20. Three different sets of seismic conditions were modeled: (1) the 2,000-year return period earthquake for the PFS Facility site; (2) the 10,000-year return period earthquake for the PFS Facility site; and (3) a sensitivity study based on the 1971 San Fernando Earthquake (Pacoima Dam record).

21. The Staff's confirmatory analysis demonstrated that the casks would not collide into each other or tipover in the event of either the design basis (2,000-year return period), the 1971 San Fernando Earthquake, or the 10,000-year return period seismic event. A maximum horizontal displacement of 15.94 inches, rotation of 1.16 degrees, and vertical displacement of 0.26 inches, of the cask, were obtained for the 10,000-year return period seismic event.

Part E. Seismic Exemption

1. The use of the PSHA methodology and a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period) are acceptable bases to determine the seismic design ground motions of the proposed PFS Facility.

2. Based upon its review and independent analyses, the Staff found the PFS PSHA results to be conservative. Specifically, the Staff's slip tendency analysis indicates smaller predicted maximum earthquake magnitudes than those developed by PFS.

3. The Applicant calculated the seismic hazard in Skull Valley to be higher than the seismic hazards for sites at, or near, Salt Lake City, despite the fact that fault sources near Salt Lake City are larger and more seismically active than fault sources near the PFS site.

4. The 2,000-year horizontal peak ground acceleration for Skull Valley (soil hazard) as estimated by PFS, is actually higher than the 2,500-year ground motions for the nine sites along the Wasatch Front that were evaluated as part of the Utah Department of Transportation I-15 Reconstruction Project.

5. The radiological hazard posed by a dry cask storage facility is inherently lower than operating commercial nuclear power plants. Thus, an ISFSI's design ground motions need not be as large (*i.e.*, improbable) as those used for NPP design

6. Analyses of SSEs at nuclear power plants in the western U.S. show that the average mean annual probability of exceeding the safe shutdown earthquake is 2.0×10^{-4} - which is equivalent to a 5,000-year return period.

7. DOE-STD-1020-94 and the TMI-2 ISFSI exemption were utilized by the Staff as two points of reference, that provide relevant technical and regulatory insights for consideration in deciding that a seismic design based on ground motions that have

a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period) is appropriate for the proposed PFS Facility.

8. The favored option in SECY-01-0178 proposes a seismic design in conjunction with a PSHA methodology, based on ground motions with a mean annual probability of exceedance of 5×10^{-4} (2,000 year return period ground motion).

9. The two-tiered approach proposed in SECY-98-126, and its reference to the dose limits in § 72.104(a), is inapplicable. These dose limits do not apply to design basis earthquakes, for which the dose limits in 10 C.F.R. § 72.106(b) would apply.

10. The Staff determined that the mean annual probability of exceedance of the proposed PFS Facility's seismic design ground motion should be greater than the NPP value of 1×10^{-4} (10,000-year return period), and may be greater than the average mean annual probability of exceeding the SSE at NPPs in the western United States of approximately 2×10^{-4} per year (5,000-year return period).

11. The State's assertion that design levels for new Utah building construction and highway bridges are more stringent is not correct. The State's assertion ignores the relative levels of conservatism in the design of facilities of different types.

12. The occurrence of a design basis earthquake with a mean annual probability of occurrence of 5×10^{-4} (2,000-year return period) would not impair the ability of SSCs important to safety to maintain subcriticality, confinement, and sufficient shielding of the spent nuclear fuel. Accordingly, the dose limits in 10 C.F.R. § 72.106(b) will not be exceeded in the event of a design basis earthquake.

13. The Staff conducted an analysis of potential dose consequences in the event of multiple hypothetical cask tipovers at the proposed PFS Facility, considering (a) the potential damage to the cask shield; (b) potential thermal degradation; and (c) potential effect on offsite dose rates caused by spatial reorientation of the casks.

14. Based on the Staff's calculations, the (design basis) accident offsite dose limit of 5 rem could be exceeded only if the off-site dose rate at the OCA boundary increases to approximately 6.94 mrem/hr. This dose rate corresponds to an increase above the maximum normal off-site dose rate by a factor of about 2,400.

15. In the event of the beyond-design basis hypothetical tipover of multiple casks, any minor irregularities in the shields that might result from shield damage in a tipover would not contribute significantly to the radiation dose rate at the OCA boundary.

16. Assuming that all 4,000 casks tipover and experience thermal degradation (via hydrogen loss) in the radial shield, the off-site dose rates could increase by a factor of approximately 2.4 (far less than 2,400).

17. If all 4,000 casks tipover, dose rates would increase by no more than a factor of 97.6. This predicted worst-case scenario is well below the factor of 2,400 increase needed to exceed an offsite dose of 5 rem.

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PRIVATE FUEL STORAGE, L.L.C.	)	Docket No. 72-22-ISFSI
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PREFACE TO NRC STAFF TESTIMONY OF GOODLUCK I. OFOEGBU
CONCERNING UNIFIED CONTENTION UTAH L/QQ, PART C
(CHARACTERIZATION OF SUBSURFACE SOILS)

The NRC Staff ("Staff") is filing the testimony of Dr. Goodluck I. Ofoegbu, concerning the issues in Unified Contention Utah L/QQ, Part C. Dr. Ofoegbu is a Principal Engineer at the Center for Nuclear Waste Regulatory Analyses ("CNWRA"), which is a division of the Southwest Research Institute ("SwRI"), in San Antonio, Texas. Dr. Ofoegbu has experience with respect to the mechanical analysis of underground excavations, foundations, earthworks, and natural geological processes such as faulting and volcanism, and has served as Principal Investigator for numerous projects involving geological engineering.

Dr. Ofoegbu assisted the NRC Staff in its evaluation of the Applicant's site characterization and geotechnical evaluations for the proposed Private Fuel Storage (PFS) facility, as set forth in the NRC Staff's "Consolidated Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued in March 2002.

In his testimony, Dr. Ofoegbu provides the Staff's views concerning the acceptability of the Applicant's characterization of subsurface soils, which is the subject of Unified Contention Utah L/QQ, Part C. As discussed in the testimony and the Consolidated SER, the Staff finds that the Applicant has satisfied the Commission's requirements related to the characterization of subsurface soils for the design of an ISFSI, as set forth in 10 C.F.R. Part 72. The Staff concludes that the information provided by the Applicant, through its site characterization and geotechnical evaluations concerning the behavior of the native foundation soils, is adequate to support the various engineering analyses of the facility structures, systems, and components important to safety.

Further, with respect to the Applicant's proposed use of soil cement/cement-treated soil in the vicinity of the storage pads and canister transfer building, the Applicant has committed to demonstrate through appropriate testing that any Staff-approved soil cement/cement-treated soil design specifications will be achieved. For example, PFS has committed to demonstrate through testing that the stiffness of the cement-treated soil under the pads will not exceed the specified design value (*i.e.*, a dynamic Young's modulus of 75,000 psi); and it has committed to demonstrate through testing that the soil cement around the CTB will have a minimum unconfined compressive strength of 250 psi. The commitment by PFS to follow the applicable ACI soil-cement standards provides further assurance that the proposed use of soil-cement and cement-treated soil will be acceptable.

Finally, the Staff has considered the mechanisms postulated by the State with respect to potential degradation of the soil-cement and/or cement-treated soil, and concludes that those processes, if they occur, would not have an adverse effect on the safety of the facility.

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PREFACE TO NRC STAFF TESTIMONY OF
DANIEL J. POMERENING AND GOODLUCK I. OFOEGBU
CONCERNING UNIFIED CONTENTION UTAH L/QQ, PART D
(SEISMIC DESIGN AND FOUNDATION STABILITY)

The NRC Staff ("Staff") is filing the joint testimony of Daniel J. Pomerening and Goodluck I. Ofoegbu, concerning the issues in Unified Contention Utah L/QQ, Part D.

Mr. Pomerening is a Principal Engineer in the Mechanical and Materials Engineering Division of the Southwest Research Institute ("SwRI"), in San Antonio, Texas. Mr. Pomerening serves as the principal investigator for projects associated with the evaluation of structural design and environmental testing of systems and components, with an emphasis on dynamic loading. Mr. Pomerening assisted the Staff in its evaluation of design requirements related to the proposed PFS Facility, with emphasis on the review of structural and seismic design.

Dr. Ofoegbu is a Principal Engineer at the Center for Nuclear Waste Regulatory Analyses ("CNWRA"), which is a division of the SwRI, in San Antonio, Texas. Dr. Ofoegbu has experience with respect to the mechanical analysis of underground excavations, foundations, earthworks, and natural geological processes such as faulting and volcanism, and has served as Principal Investigator for numerous projects involving geological engineering. Dr. Ofoegbu assisted the Staff in its evaluation of the Applicant's site characterization and geotechnical evaluations for the proposed PFS Facility. He is also appearing as the Staff's expert witness with respect to Part C of this contention ("Characterization of Subsurface Soils"), filed herewith.

In their testimony concerning Part D of the contention, Dr. Ofoegbu and Mr. Pomerening provide the Staff's views concerning the foundation stability and seismic design of the proposed PFS Facility. As set forth in their testimony, the Staff concludes that the Applicant has properly demonstrated that the proposed PFS Facility structures and foundations have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake, and that the seismic design and foundation stability of the proposed PFS Facility satisfy all applicable regulatory requirements.

More specifically, the Staff's witnesses provide their views that the foundation stability for the storage pads and canister transfer building is adequate. Calculations provided by PFS demonstrate that: (1) there are adequate safety margins against bearing capacity failure of the storage pads under combined static loads and potential dynamic loading from the design-basis earthquake; (2) potential sliding of the pads under seismic loading would not constitute a safety hazard; and (3) settlement of the pads does not present a foundation stability concern. Further, the Staff concludes that the HI-STORM 100 storage casks will not tipover or collide due to a design basis ground motion; that the Applicant's design of the concrete storage pads will not adversely affect the stability of the casks under seismic load conditions; and that the potential for sliding of the storage pads under seismic loading is not a concern. Finally, the Staff concludes that the assumptions used by PFS in its analyses are reasonable and acceptable, and the design of the storage pads and CTB satisfies applicable regulatory requirements.

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PREFACE TO NRC STAFF TESTIMONY OF
VINCENT K. LUK AND JACK GUTTMANN CONCERNING UNIFIED CONTENTION
UTAH L/QQ, Part D (SEISMIC DESIGN AND FOUNDATION STABILITY)

The NRC Staff ("Staff") is filing the joint testimony of Dr. Vincent K. Luk and Jack Guttmann, concerning issues relating to Unified Contention Utah L/QQ, Part D.1.i.

Jack Guttmann is Chief of the Technical Review Section, Spent Fuel Project Office, Office of Nuclear Material Safety and Safeguards, NRC. With respect to proposed PFS Facility, Mr. Guttmann requested, through the NRC Office of Nuclear Regulatory Research, that a confirmatory analysis be performed by Sandia National Laboratories to evaluate the potential for cask sliding, collision and tipover at the proposed PFS facility. This analysis was considered to be confirmatory in nature, in that the Staff had previously concluded, on the basis of its review of the PFS application and supporting analyses, that tipover and collision of the casks on the PFS concrete storage pads will not occur under design basis seismic conditions.

Dr. Vincent K. Luk is a Principal Member of the Technical Staff in the Nuclear Technology Programs Department at Sandia National Laboratories, in Albuquerque, New Mexico. Among his other duties, Dr. Luk serves as the Principal Investigator in an NRC project that seeks to establish criteria and review guidelines in evaluating the seismic behavior of dry cask storage systems, and to examine the dynamic seismic behavior of free-standing dry cask storage systems and soil-structure interaction effects in simulated earthquake events.

Dr. Luk led a research team in conducting an evaluation of the seismic behavior and stability of the freestanding, cylindrical HI-STORM 100 casks to be installed on concrete pads at the proposed PFS facility, as requested by the NRC Staff. Dr. Luk and his team developed a three-dimensional coupled finite element model of the proposed PFS dry cask storage system to examine the nonlinear and dynamic behavior of the casks under prescribed seismic conditions. The team's efforts culminated in the production of the "Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage (PFS) Facility," Rev. 1, dated March 31, 2002.

The Staff's testimony describes the results of this confirmatory analysis, with respect to the potential for cask sliding, collision and tipover under seismic conditions. Three different sets of seismic conditions were modeled: (1) the 2,000-year return period earthquake for the PFS Facility site; (2) the 10,000-year return period earthquake for the PFS Facility site; and (3) a sensitivity study based on the 1971 San Fernando Earthquake (Pacoima Dam record).

The confirmatory analysis demonstrated that the casks would not collide into each other or tipover in the event of either the design basis seismic event (2,000-year return period), the 1971 San Fernando Earthquake (Pacoima Dam record), or the 10,000-year return period seismic event. A maximum horizontal displacement of 15.94 inches, rotation of 1.16 degrees, and vertical displacement of 0.26 inches, of the cask, were obtained for the 10,000-year return period seismic event. Accordingly, the Staff concludes that Part D.1.i. of Unified Contention Utah L/QQ fails present a valid concern with respect to the potential for cask sliding, collision and tipover under seismic conditions at the proposed PFS facility.

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PREFACE TO NRC STAFF TESTIMONY
OF JOHN A. STAMATAKOS, RUI CHEN AND
MARTIN W. McCANN, JR., CONCERNING UNIFIED
CONTENTION UTAH L/QQ, PART E (SEISMIC EXEMPTION)

The NRC Staff ("Staff") is filing the joint testimony of Drs. John A. Stamatakos, Rui Chen, and Martin W. McCann concerning issues in Unified Contention Utah L/QQ, Part E.

Dr. Stamatakos is a Principal Scientist at the Center for Nuclear Waste Regulatory Analysis ("CNWRA"), which is a division of the Southwest Research Institute ("SwRI"), in San Antonio, Texas. Dr. Stamatakos is a structural geologist and geophysicist with international research experience, and is the Principal Investigator for structural deformation and seismicity, including tectonics and neotectonics research.

Dr. McCann is President of Jack R. Benjamin & Associates, Inc., in Menlo Park, California, where he serves as a consultant to the CNWRA. He is also a Consulting Professor of Civil and Environmental Engineering at Stanford University. His professional experience includes probabilistic hazards analysis, including seismic and hydrologic events, reliability assessment, probabilistic risk analysis for critical facilities, systems analysis, and seismic engineering.

Dr. Chen is employed as an independent consultant in geological engineering and geosciences, and has provided technical assistance and consulting services to the CNWRA. Her work experience includes the evaluation of seismic hazard analyses and seismic designs related to proposed spent fuel storage facilities, including the Three Mile Island Unit 2 ISFSI at the Idaho National Engineering and Environmental Laboratory (INEEL) and the proposed PFS Facility. She has also worked on matters involving the technical analysis of mechanical, thermal, and hydrological processes in complex geomechanical and geotechnical engineering systems related to the proposed Yucca Mountain repository.

Drs. Stamatakos, Chen, and McCann assisted the NRC Staff in its evaluation of the Applicant's seismic exemption request. Specifically, they conducted the Staff's evaluation of the Applicant's probabilistic seismic hazard analysis ("PSHA"), including seismic ground motions and faulting hazards. They are the co-authors of a document entitled "Seismic Ground Motion and Faulting Hazard at Private Fuel Storage Facility in the Skull Valley Indian Reservation, Tooele County - Final Report," issued by the CNWRA in September 1999.

The Staff's testimony discusses the acceptability of the Applicant's seismic exemption request, which is the subject of Unified Contention Utah L/QQ, Part E. They conclude that the Applicant's use of a PSHA and ground motions that have a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period) provides an acceptable basis for the seismic design of the proposed PFS Facility. Further, they found that the PFS PSHA results are conservative; and the Staff's slip tendency analysis indicates smaller predicted maximum earthquake magnitudes than those developed by PFS. Indeed, PFS calculated the seismic hazard in Skull Valley to be higher than the seismic hazards for sites at, or near, Salt Lake City, despite the fact that fault sources near Salt Lake City are larger and more seismically active than fault sources near the PFS site. Finally, their testimony explains the bases for the Staff's approval of the Applicant's seismic exemption request, as set forth in the Staff's Consolidated SER, including comparisons between the radiological risk at an ISFSI using dry cask storage systems and commercial nuclear power plants, and other regulatory materials and reference points that provide relevant technical and regulatory insights for consideration in establishing the design ground motions.

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PREFACE TO NRC STAFF TESTIMONY OF MICHAEL D. WATERS
CONCERNING RADIOLOGICAL DOSE CONSIDERATIONS
RELATED TO UNIFIED CONTENTION UTAH L/QQ,
PART E (SEISMIC EXEMPTION)

The NRC Staff ("Staff") is filing the testimony of Mr. Michael D. Waters, concerning certain issues contained in Unified Contention Utah L/QQ, Part E.2.

Mr. Waters is a Health Physicist in the Spent Fuel Project Office ("SFPO"), Office of Nuclear Material Safety and Safeguards, U.S. Nuclear Regulatory Commission ("NRC"). He performed reviews in the areas of shielding and confinement with respect to the HI-STORM 100 storage cask, and a review of shielding for the HI-STAR 100 transportation cask systems, both of which PFS proposes to use at its facility. Further, in response to the State's contention, Mr. Waters performed an analysis of potential offsite dose consequences that may result in the event of a beyond-design-basis hypothetical cask tipover involving multiple casks at the proposed PFS Facility.

The Staff's testimony describes its views with respect to one portion of Unified Contention Utah L/QQ, Part E (Subpart E.2.), insofar as that contention concerns the potential dose consequences that may result in the event of a beyond-design-basis hypothetical cask tipover.

In his testimony, Mr. Waters describes the Staff's analysis of potential dose consequences in the event of multiple hypothetical cask tipovers at the proposed PFS Facility, in which the Staff considered potential damage to the cask shield; potential thermal degradation of the concrete in the shield; and the potential effect on offsite dose rates that may be caused by spatial reorientation of the casks from a vertical to tipped or horizontal position.

Based on the Staff's calculations, the Staff has concluded that in the event of a beyond-design basis hypothetical tipover of multiple casks, any minor irregularities in the shields that might result from shield damage in a tipover would not contribute significantly to the radiation dose rate at the OCA boundary. Further, assuming that all 4,000 casks tipover and experience thermal degradation (via hydrogen loss) in the radial shield, the off-site dose rates would not increase significantly. In addition, even if all 4,000 casks tipover, dose rates would increase by no more than a factor of 97.6, which is well below the increase needed to exceed an offsite dose of 5 rem (*i.e.,* the design basis accident dose limit). Accordingly, the Staff has concluded that the concern raised in Unified Contention Utah L/QQ, Part E (Subpart E.2.), is not valid.

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NRC STAFF TESTIMONY OF GOODLUCK I. OFOEGBU
CONCERNING UNIFIED CONTENTION UTAH L/QQ, PART C

Q1. Please state your name, occupation, and by whom you are employed.

A1. My name is Goodluck I. Ofoegbu. I am employed as a Principal Engineer at the Center for Nuclear Waste Regulatory Analyses ("CNWRA"), which is a division of the Southwest Research Institute ("SwRI"), in San Antonio, Texas. I am providing this testimony under a technical assistance contract between the staff of the Nuclear Regulatory Commission ("NRC Staff" or "Staff") and the CNWRA at the SwRI. A statement of my professional qualifications is attached hereto.

Q2. Please describe your current responsibilities.

A2. In my position as Principal Engineer at the CNWRA, I have served as Principal Investigator for several projects involving geological engineering. My work includes mechanical analysis of underground excavations, foundations, earthworks, and natural geological processes, such as faulting and volcanism.

Q3. Please explain what your duties have been in connection with the NRC Staff's review of the application filed by Private Fuel Storage, L.L.C. ("PFS" or "Applicant") for a license to construct and operate an Independent Spent Fuel Storage Installation ("ISFSI") on the Reservation of the Skull Valley Band of Goshute Indians, geographically located within Skull Valley, Utah (the "proposed PFS Facility").

A3. As part of my official responsibilities, I assisted the NRC Staff in its evaluation of the Applicant's site characterization and geotechnical evaluations for the proposed PFS Facility. Further, I assisted the Staff in the preparation of its "Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued on September 29, 2000 ("SER"). I also assisted in the preparation of Supplement No. 2 to the SER, dated December 21, 2001 ("SSER Supplement No. 2"). Those two documents have been incorporated into the NRC Staff's "Consolidated Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued in March 2002 ("Consolidated SER").

In addition, I assisted the NRC Staff in preparing its responses to several sets of discovery requests filed by the State of Utah ("State"), including the "NRC Staff's Objections and Responses to the 'State of Utah's Eleventh Set of Discovery Requests directed to the NRC Staff,'" dated December 11, 2000; "NRC Staff's Objections and Responses to the 'State of Utah's Eighteenth Set of Discovery Requests directed to the NRC Staff,'" dated February 1, 2002; and "NRC Staff's Objections and Responses to the 'State of Utah's Twentieth Set of Discovery Requests directed to the NRC Staff,'" dated February 27, 2002.

Q4. What is the purpose of this testimony?

A4. The purpose of this testimony is to provide the NRC Staff's views concerning the acceptability of the Applicant's characterization of subsurface soils, which is the subject of Unified Contention Utah L/QQ, Part C. I am also providing separate testimony on selected portions of Part D of this contention in the NRC Staff's testimony of Goodluck I. Ofoegbu and Daniel J. Pomerening, filed herewith.

Q5. Please identify the Commission's requirements related to the characterization of subsurface soils for the design of an ISFSI.

A5. The Commission's requirements governing the characterization of subsurface soils for an ISFSI are set forth in 10 C.F.R. Part 72. In general, 10 C.F.R. § 72.90 requires an evaluation of site characteristics that may directly affect the safety or environmental impact of the proposed facility. Specific requirements for the characterization of the subsurface soils are defined in 10 C.F.R. § 72.102. Specifically, 10 C.F.R. § 72.102(c) states: "Sites other than bedrock sites must be evaluated for their liquefaction potential or other soil instability due to vibratory ground motion." Additionally, 10 C.F.R. § 72.102(d) states: "Site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading."

Pursuant to 10 C.F.R. § 72.122(b)(1), structures, systems, and components important to safety ("SSCs") must be designed to accommodate the effects of, and be compatible with, site characteristics and environmental conditions associated with normal operation, maintenance and testing of the ISFSI, and to withstand postulated accidents. Further, 10 C.F.R. § 72.122(b)(2) requires that SSCs be designed to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform safety functions.

Q6. Are you familiar with Unified Contention Utah L/QQ, Part C.?

A6. Yes. As admitted by the Licensing Board, Unified Contention Utah L/QQ, Part C., states as follows:

> **Unified Contention Utah L/QQ (Geotechnical)**
>
> * * *
>
> **C. <u>Characterization of Subsurface Soils.</u>**
>
> 1. <u>Subsurface Investigations</u>
>
> 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."

2. Sampling & Analysis

The Applicant'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:

a. 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 Part C6, Sampling.

b. The Applicant's design of the foundation systems is based on an insufficient number of tested samples, and on a laboratory shear strength testing program that does not include strain-controlled cyclic triaxial tests and triaxial extension tests.

3. Physical Property Testing for Engineering Analyses

a. The Applicant 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.

b. The Applicant 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 storage pad foundations as required by 10 CFR § 72.102(d).

c. The Applicant has not considered the impact to the native soil caused by construction and placement of the cement-treated soil, nor has the Applicant analyzed the impact to settlement, strength and adhesion properties caused by placement of the cement-treated soil.

d. The Applicant has not shown that its proposal to use cement-treated soil will perform as intended – *i.e.*, provide dynamic stability to the foundation system – and the Applicant has not adequately addressed the following possible mechanisms that may crack or degrade the function of the cement-treated soil over the life of the facility:

(i) shrinkage and cracking that normally occurs from drying, curing and moisture content changes.

(ii) potential cracking due to vehicle loads.

(iii) potential cracking resulting from a significant number of freeze-thaw cycles at the Applicant's site.

(iv) potential interference with cement hydration resulting from the presence of salts and sulfates in the native soils.

(v) cracking and separation of the cement-treated soil from the foundations resulting from differential immediate and long-term settlement.

e. The Applicant has unconservatively underestimated the dynamic Young's modulus of the cement-treated soil when subjected to impact during a cask drop or tipover accident scenario. This significantly underestimates the impact forces and may invalidate the conclusions of the Applicant's Cask Drop/Tipover analyses.

Q7. In Subpart C.1. of the contention, concerning subsurface investigations, 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." Do you agree with this assertion?

A7. No.

Q8. Please provide the bases for this conclusion.

A8. NRC regulatory guidance in Regulatory Guide 1.132 (and Draft Reg. Guide DG-1101) provides <u>general guidelines</u> concerning site investigations, including the spacing and depth of borings for safety-related structures. This guidance document appropriately recognizes that the spacing and depth of borings or other site-characterization activities depend on the complexity of the site-specific subsurface conditions and the particular information needed for the engineering design of structure foundations. Indeed, Reg. Guide 1.132 states:

> Because the details of the actual site investigations will be highly site dependent, the procedures described herein should only be used as guidance and be tempered with professional judgment. Alternative and special investigative procedures that have been derived in a professional manner will be considered equally applicable for conducting foundation investigations.

The specific regulatory requirement for the geotechnical site characterization for an ISFSI is contained in section 72.102(d), which provides that site-specific investigations and laboratory analyses must show that the soil conditions are adequate for the proposed foundation loading. The primary purpose of the site-specific investigation and associated laboratory analyses is to classify the site subsurface materials and to identify variations in important properties of these materials both laterally and with depth. As set forth in the Staff's Consolidated SER and discussed herein, the Staff has determined that the Applicant achieved this purpose through a combination of borings and other test methods, including cone penetrometer testing. Further, the geotechnical site characterization information provided by the Applicant in the PFS Safety Analysis Report ("SAR") satisfies the regulatory requirement of 10 C.F.R. § 72.102(d), by showing that soil conditions are adequate for the proposed foundation loading

The following considerations support these Staff findings:

First, standard penetration and cone-tip resistance data provided in the SAR (*see* Consolidated SER, page 2-55) support the Applicant's classification of the subsurface materials at the site as consisting of a relatively compressible top layer (layer 1) that is approximately 25–30 feet thick and underlain by much denser and stiffer material (layer 2), which is classified as dense sand and silt.

Second, the profiles of cone-tip resistance provided in the SAR Figures 2.6-5 (Sheets 1–14) and 2.6-21 through 2.6-23 illustrate the lateral and vertical variations of shear strength and compressibility for layer-1 soil. As described in the Consolidated SER (page 2-56), the profiles support a subdivision of layer-1 soil into four sublayers, and show that the second sublayer from the top (a mixture of silty clay and clayey silt referred to as layer 1B soil in the Consolidated SER, page 2-56) is the weakest and most compressible sublayer.

Third, the bearing capacity of the storage pads was calculated using the undrained shear strength of layer-1B soil. *See* Consolidated SER, pages 2-58 and 2-61. The permissible value of undrained shear strength for evaluating the bearing capacity of the storage pads consists of the average undrained shear strength through a depth of 30 feet below the base of the pads. *Cf.* Terzaghi et al., 1996, page 406. Because layer-1B is the weakest sublayer, the value of undrained shear strength used by PFS is therefore a conservative lowerbound estimate of the permissible value.

Fourth, the bearing capacity of the canister transfer building ("CTB") foundation was calculated using an average undrained shear strength for layer-1 soil estimated using laboratory data for layer-1B and the cone penetrometer test data. *See* Consolidated SER, pages 2-63 and 2-65. The permissible value of undrained shear strength for evaluating the bearing capacity of the CTB foundation consists of the average undrained shear strength through a depth of 240 feet below the base of the foundation. *Cf.* Terzaghi et al., 1996, page 406. The value of the average undrained shear strength at the proposed PFS site would thus be determined mainly by layer-2 soil, which is much stronger than the layer-1 soil used by PFS to obtain an average undrained shear strength value for its CTB foundation bearing capacity calculations. The value of undrained shear strength used by PFS is therefore a conservative lowerbound estimate of the permissible value.

Fifth, the potential settlement of the storage pads and CTB was estimated using the laboratory compressibility data for layer-1B soil. Because layer-1B is the most compressible sublayer, the estimated settlement values therefore represent the upperbound values. *See* Consolidated SER, pages 2-58 and 2-63.

In sum, the preceding considerations collectively support the Staff's findings that PFS, through its existing site-specific investigations and laboratory analyses, has satisfactorily classified the subsurface materials, identified lateral and vertical variations in the relevant properties of those materials, and demonstrated that the site-specific soil conditions are adequate for the proposed foundation loading. Therefore, because the Applicant has satisfied the regulatory requirement of 10 C.F.R. § 72.102(d), it is not necessary that the Applicant follow the particular spacing of borings recommended in Reg. Guide 1.132.

Q9. In Subpart C.2. of the contention, the State asserts that the "Applicant'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." Do you agree with this assertion?

A9. No.

Q10. Please provide the bases for this conclusion.

A10. As discussed above, the Staff finds that the Applicant has satisfied the geotechnical site characterization requirement set forth in 10 C.F.R. § 72.102(d), including the sampling and analysis to characterize the site. The Applicant has provided sufficient geotechnical data in its SAR to demonstrate that the site-specific soil conditions are adequate for the proposed foundation loading.

Q11. More specifically, in Subpart C.2.a. of this contention, the State asserts 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 Part C6, Sampling." Do you believe that this presents a valid concern?

A11. No.

Q12. Please provide the bases for this conclusion.

A12. The purpose of "continuous sampling" is to determine the continuous variation of soil properties with depth. The continuous sampling of soil layers referred to in Reg. Guide 1.132 represents one method available for determining the continuous variation of soil properties with depth. PFS instead successfully determined the variation of soil properties with depth through the use of an alternative method, *i.e.*, *in situ* cone penetrometer testing. As discussed above, Reg. Guide 1.132 provides guidance [as opposed to establishing a regulatory requirement like 10 C.F.R. § 72.102(d)], and recognizes that alternative procedures which have been derived in a professional manner will be considered equally applicable for conducting foundation investigations. In the Staff's view, *in situ* cone penetrometer testing, as used by the Applicant, is one such alternative procedure for determining the continuous variation of soil properties with depth.

Q13. In Subpart C.2.b. of the contention, the State asserts that the "Applicant's design of the foundation systems is based on an insufficient number of tested samples." Do you agree with this assertion?

A13. No.

Q14. Please provide the bases for this conclusion.

A14. In the Staff's view, the relevant inquiry is whether the geotechnical site characterization data obtained by the Applicant is adequate to support the specific values or parameters used in the Applicant's foundation stability analyses, not how many samples *per se* the Applicant has taken. As set forth in the Staff's Consolidated SER and discussed herein, the Applicant has provided in the PFS SAR sufficient geotechnical data -- in the form of both cone penetrometer and laboratory test data -- to demonstrate that the site-specific soil conditions are adequate for the proposed foundation loading in accordance with 10 C.F.R. § 72.102(d). This is further discussed in response to Questions 8 and 16, herein.

Q15. The State also asserts, in Subpart C.2.b. of this contention, that the Applicant's design of the foundation systems is based "on a laboratory shear strength testing program that does not include strain-controlled cyclic triaxial tests and triaxial extension tests." Do you believe that this presents a valid concern?

A15. No.

Q16. Please provide the bases for this conclusion.

A16. As indicated in response to Question 8, *supra*, the geotechnical information used for the PFS foundation system designs was obtained from laboratory test data for layer-1B soil (including laboratory compression test results) and the cone penetrometer test data. Information presented in a PFS calculation (Stone and Webster, 2001a, Appendix C) supports the undrained shear strength value of 2,200 psf for layer-1B soil. This value, in turn, was combined with information determined from the cone-penetrometer test data to establish the basis for the soil-strength parameter values used for stability analyses of the storage pads and canister transfer building foundation. As stated in the Consolidated SER (page 2-57), the Staff reviewed the geotechnical information provided in the PFS SAR and concluded, *inter alia*, that (1) the index properties and strength and compressibility of the soil layers were determined by the Applicant using an appropriate combination of field and laboratory testing, and (2) the information presented is sufficient to support appropriate engineering analyses of the proposed structures. Thus, the specific combination of tests performed by the Applicant provided the data needed to obtain the soil-strength parameter values used in its stability analyses of the storage pads and canister transfer building foundation; strain-controlled cyclic triaxial tests or triaxial extension tests of site soils are therefore not necessary.

Q17. In Subpart C.3.a. of this contention, concerning physical property testing for engineering analyses, the State asserts that the "Applicant 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." Do you agree with this assertion?

A17. No.

Q18. Please provide the bases for this conclusion.

A18. The information provided by the Applicant in the SAR regarding the stress-strain characteristics of the native foundation soils is sufficient to demonstrate that the soil conditions are adequate for the proposed foundation loading, as required by 10 C.F.R. § 72.102(d). Adequate information on the following aspects of stress-strain characteristics was provided by the Applicant: (1) undrained shear strength, based on laboratory triaxial-compression and direct-shear testing; (2) soil compressibility, based on laboratory oedometer testing; (3) the lateral and vertical variations of shear strength and compressibility at the site, based on in situ cone penetrometer testing data; (4) elastic parameters (Young's modulus, Poisson's ratio, and shear modulus), determined using shear and compressional wave velocities from field seismic reflection, refraction, and cross-hole velocity measurements and cone penetrometer testing; and (5) shear modulus and damping versus cyclic strain relationships, derived from a combination of laboratory data developed by PFS and information available in the literature.

One aspect of the stress-strain behavior of soils is the stiffness of the soils, which can be characterized through shear-wave velocity profiles obtained from field refraction data. Accordingly, the Applicant provided upper and lower bounds of shear-wave velocity profiles, in addition to the best estimate soil profile, to account for uncertainties in the average shear-wave velocity of the native foundation soils. The Applicant also performed sensitivity analyses to define the effects of the variability of the shear modulus and damping versus cyclic strain relationships on the calculated seismic site-response factors. *See* Appendix F of Geomatrix Consultants, Inc., 2001a, *Fault evaluation study and seismic hazard assessment study—final report.* Revision 1. Oakland, CA: Geomatrix Consultants, Inc. (cited in Section 2.3 of the Consolidated SER). The modulus-reduction and damping versus strain curves provided by PFS were generated using accepted engineering practices and are consistent with other curves generated from comparable data.

In sum, the Applicant provided sufficient information on the behavior of the native foundation soils to demonstrate that: (1) the value of soil strength used for foundation-stability analyses is a lowerbound estimate of the applicable value; (2) the value of soil compressibility used for foundation-settlement analyses resulted in upperbound estimates of the potential foundation settlement; and (3) the values of shear-wave velocity used to determine the elastic-parameter values for the soils account for the variability of shear-wave velocity at the site. Based on the foregoing considerations, and as discussed in Section 2.1.6 of the Consolidated SER, the Staff therefore concludes that PFS has adequately described the stress-strain behavior of the native foundation soils, to support the various engineering analyses of the facility structures, systems, and components important to safety.

Q19. The State contends, in Subpart C.3.b. of this contention, that the "Applicant 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 [ ] storage pad foundations as required by 10 CFR § 72.102(d)." Do you agree with this assertion?

A19. No.

Q20. Please provide the bases for this conclusion.

A20. My conclusion is based on several considerations. First and foremost, the soil cement around the storage pads and the cement-treated soil under the storage pads are not being relied upon to support any safety function of the pads. This fact is reflected in the Staff's stability evaluation of the cask-storage-pad foundation in the Consolidated SER (pages 2-57 to 2-62).

Second, PFS has committed to demonstrate through testing that the stiffness of the cement-treated soil under the pads will not exceed the specified design value (*i.e.*, it will have a dynamic Young's modulus not exceeding 75,000 psi).

Third, with respect to the specific requirements of 10 C.F.R. § 72.102(d), the regulation states: "Site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading." The regulation by its terms does not require the use of "case history precedent" or "dynamic analyses," although the Staff does recognize the utility and value of such analytical tools (*see, e.g.,* references to prior uses of soil cement in discussion below concerning the proposed use of soil cement around the CTB foundation to provide additional lateral resistance). Therefore, to the extent that the State might be asserting that the use of these tools is explicitly required by 10 C.F.R. § 72.102(d), the Staff believes that the State's assertion is misplaced. As noted above, the Staff finds that the geotechnical site characterization information provided by the Applicant in the PFS Safety Analysis Report ("SAR") shows that the site-specific soil conditions are adequate for the proposed foundation loading, in compliance with 10 C.F.R. § 72.102(d).

Q21. The State similarly contends, again in Subpart C.3.b. of this contention, that the "Applicant 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 [ ] foundations as required by 10 CFR § 72.102(d)." Do you agree with this assertion?

A21. No.

Q22. Please provide the bases for this conclusion.

A22. My conclusion with respect to the CTB is also based on several considerations, which differ from those discussed in connection with the storage pads. This is due to the fact that the soil cement around the CTB is required to provide additional lateral resistance to increase the factor of safety against sliding of the CTB foundation.

First, to provide the necessary lateral resistance, the soil cement around the CTB foundation must have a minimum unconfined compressive strength of 250 psi, a value based on Staff-reviewed PFS calculations. Therefore, PFS is required (and has committed) to demonstrate through testing that the soil cement will meet this minimum strength requirement.

Second, in support of this proposed use of soil cement, PFS provided references to previous uses of soil cement within and outside of the United States (*see* SAR, Rev. 22, pages 2.6-113 to 114) as precedents for the use of cement stabilization to enhance the engineering characteristics of natural soils. The precedents cited by PFS are supported by other cases gleaned from the literature and reviewed by the Staff, which indicate that: (a) the soil-property changes that result from cement stabilization can be considered long-lasting (*see, e.g.,* Roberts, 1986); and (b) soil cement has been used as a buttress - *i.e.*, as a structure that provides lateral resistance to another structure - in several other engineering projects. *See*, *e.g.,* Van Riessen, 1992; Lambrechts, 1998.

Third, PFS has committed in the SAR (Rev. 22, pages 2.6-117 to 118) to follow the standards, procedures, and recommendations contained in the "State-of-the-Art Report on Soil Cement," developed by ACI Committee 230 [ACI 230-1R-90 (Reapproved 1997)]. This report describes the state-of-the-art procedures and identifies the applicable standards for mix proportioning, construction, quality-control, and testing of soil cement. The report, for example, lists ASTM D 559-82 ("Standard Methods for Wetting-and-Drying Tests of Compacted Soil-Cement

Mixtures") and ASTM D 560-82 ("Standard Methods for Freezing-and-Thawing Tests of Compacted Soil-Cement Mixtures"), which specify test procedures for evaluating the durability of soil cement. Adherence by PFS to these and the other standards contained in the ACI report provides further assurance that the proposed soil-cement layer around the CTB will provide the specified amount of lateral resistance for the proposed duration of the PFS ISFSI facility.

Q23. In Subpart C.3.c. of this contention, 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, nor has the Applicant analyzed the impact to settlement, strength and adhesion properties caused by placement of the cement-treated soil." Do you believe that these represent valid concerns?

A.23 No.

Q24. Please provide the bases for this conclusion.

A24. As I understand Subpart C.3.c. of the contention, the State is raising two principal concerns associated with the construction and placement of the cement-treated soil: (1) that the cement-treated soil will form a relatively impermeable cap over the natural soil, resulting in an increase in the water content of the soil because of reduced evapotranspiration, and consequently, a decrease in shear strength and an increase in compressibility of the natural soil; and (2) that the use of heavy placement equipment for construction of the cement-treated soil may cause significant remolding of the underlying natural soil, which in turn could cause a significant decrease in the shear strength of the natural soil.

Based on this understanding, I do not believe that the State has presented any valid concerns in Subpart C.3.c. of the contention. In my professional opinion, both of the aforementioned concerns are based on phenomena that are either unlikely to occur or, if they were to occur, would not have an adverse effect on the safety of the proposed facility, for the reasons discussed below.

First, the depth to the water table is approximately 120 feet below the base of the facility structures. Therefore, there is no supply of water close enough to feed the postulated water-content increase.

Second, data provided by PFS (*see* SAR, Rev. 22, pages 2.6-42 to 44 and Table 2.6-1) on the effects of inundation of five specimens of the natural soil indicate that an increase in water content is not likely to have any appreciable effect on the compressibility of the soil. Inundation of the specimens during consolidation testing caused an additional vertical strain of only about 0.001 (*i.e.*, an additional settlement of about 0.12 inch for a 10-foot thick soil layer).

Third, a small decrease in shear strength occurring over a large area (such as may result from the postulated water-content change) or a localized larger decrease (such as may result from the postulated remolding) would not have a significant effect on the bearing capacity of either the storage pads or the CTB foundation. It is important to note that the shear strength actually used by the Applicant to determine the bearing capacity of each of the foundations is much smaller than the permissible shear strength for the calculation of bearing capacity, given the foundation widths and depth profile of shear strength below the foundations. As such, it is unlikely that a sufficient decrease in shear strength can occur over an area large enough to significantly affect the average shear strength within the applicable depth for each foundation (30 feet for the pads and 240 feet for the CTB).

Q25. In Subpart C.3.d. of this contention, the State contends that the "Applicant has not shown that its proposal to use cement-treated soil will perform as intended – *i.e.*, provide dynamic stability to the foundation system," citing in support of this assertion the Applicant's alleged failure to adequately address five "possible mechanisms that may crack or degrade the function of the cement-treated soil over the life of the facility." Do you believe that the State presents any valid concerns in this subpart of its contention, with respect to the Applicant's proposed use of cement-treated soil under the storage pads or soil cement around the pads?

A25. No.

Q26. Please provide the bases for this conclusion.

A26. As I noted earlier, the proposed cement-treated soil/soil cement under or around the storage pads is not being relied upon to support any safety function of the pads. As set forth in Section 2.1.6.4 of the Consolidated SER and discussed herein, the Staff's acceptance of the storage-pad design relative to the capability of the underlying soil to provide adequate support to the storage pads is based on the following considerations.

First, calculations provided by PFS demonstrate adequate safety margins against bearing capacity failure of the pads under combined static loads and potential dynamic loading from the design-basis earthquake. The calculations do not rely on any contribution of load-bearing resistance from the soil cement around the storage pads and the cement-treated soil under the storage pads.

Second, calculations provided by PFS demonstrate that the storage pads can be expected to undergo post-construction settlement of about 3 to 4 inches, taking into account both static loads and potential dynamic loading from the design-basis earthquake. The stiffness of the soil cement around the pads and the cement-treated soil under the pads was not relied upon to reduce the potential settlement of the pads. PFS has committed to perform maintenance repair of the pad-emplacement area as necessary to correct any changes caused by settlement. One such type of maintenance repair includes the scraping of aggregates from between the pads to maintain the top surface of the aggregate layer at the same elevation as the top surface of the pads.

Third, calculations provided by PFS demonstrate that potential sliding of the storage pads under seismic loading does not constitute a safety hazard, as there are no safety-related external connections to the pads or casks that may rupture or become misaligned as a result of pad sliding. Indeed, the Staff agrees with the Applicant that the storage casks are less likely to tip over if the pads are free to slide. The Staff's evaluation of the potential effects of sliding of the pads does not rely on any property of the soil cement or cement-treated soil.

For these reasons, even if cracking or other degradation of the soil cement/cement-treated soil in the vicinity of the storage pads were to occur -- and be of the type and occur by the various mechanisms specifically postulated by the State in Subpart C.3.d. of this contention -- it would not have any adverse effects on the safety functions of the storage pads.

Q27. Likewise, do you believe that the State presents any valid concerns in Subpart C.3.d. of this contention with respect to the Applicant's proposed use of soil cement <u>around the canister transfer building foundation</u>?

A27. No.

Q28. Please provide the bases for this conclusion.

A28. Notwithstanding the Applicant's proposal to use soil cement around the CTB to provide additional lateral resistance, the potential cracking or other soil-cement degradation mechanisms adduced by the State in Subparts C.3.d.(i)-(v) of this contention could not have an adverse effect on the safety functions of the CTB foundation, for the following reasons.

First, as I noted previously, PFS has committed (in the SAR, Rev. 22, pages 2.6-117 to 118) to follow the standards, procedures, and recommendations contained in the "State-of-the-Art Report on Soil Cement," developed by ACI Committee 230 [ACI 230-1R-90 (Reapproved 1997)], which describes the state-of-the-art procedures and identifies the applicable standards for mix proportioning, construction, quality-control, and testing of soil cement. These standards and procedures were developed to reduce the likelihood and mitigate the effects of the type of soil-cement cracking/degradation cited by the State in Subpart C.3.d. of its contention. For example, the effects of any salts or sulfates in the native soil would necessarily be considered in the mix design. In this regard, the Applicant has committed to performing the appropriate tests to determine the proportions of natural soil and cement needed to achieve the soil-cement properties specified for the CTB foundation.

Second, even if vertical and/or near-vertical cracks were to form in the soil cement via the various mechanisms identified by the State in Subpart C.3.d. of this contention, the expected vertical/near-vertical orientation of the cracks would allow them to close up, and the small size of the cracks would be such that any resulting increase in the amount of lateral movement of the foundation necessary to close the cracks and mobilize the passive resistance of the soil cement would be small. Therefore, the Staff does not expect such cracking - assuming it occurs - to significantly affect the passive resistance of the soil cement, nor does it expect any associated small lateral movement of the CTB foundation to impact any safety function of the structure, as there are no external safety-related connections associated with the CTB.

Q29. In Subpart C.3.e. of this contention, the State asserts that the "Applicant has unconservatively underestimated the dynamic Young's modulus of the cement-treated soil when subjected to impact during a cask drop or tipover accident scenario." Do you agree with this assertion?

A29. No.

Q30. Please provide the bases for this conclusion.

A30. The State's assertion appears misplaced, insofar as the dynamic Young's modulus of the cement-treated soil underneath the pads is a design specification and not an estimated property. As stated earlier, PFS will be required to demonstrate through testing that the stiffness of the cement-treated soil under the pads will not exceed the specified value -- *i.e.*, a dynamic Young's modulus of 75,000 psi.

Q31. What is your overall conclusion with respect to the various issues described by the State in Part C. of Unified Contention Utah L/QQ, concerning the Applicant's characterization of the subsurface soils underlying the proposed site of the PFS facility and its proposed use of soil cement/cement-treated soil?

A31. For the reasons discussed above and in the Consolidated SER, the Applicant has satisfied the Commission's requirements related to the characterization of subsurface soils for the design of an ISFSI, as set forth in 10 C.F.R. Part 72. The information obtained by the Applicant through its site characterization and geotechnical evaluations concerning the behavior of the native foundation soils is adequate to support the various engineering analyses of the facility structures, systems, and components important to safety. Further, with respect to the Applicant's proposed use of soil cement/cement-treated soil in the vicinity of the storage pads and CTB, the Applicant has committed to demonstrate through appropriate testing that any Staff-approved soil cement/cement-treated soil design specifications will be achieved.

Q32. Does this conclude your testimony?

A32. Yes.

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B.Sc., Geology, University of Nigeria, Nsukka, 1977
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Qualifications:

Dr. Ofoegbu is a geological engineer specializing in the mechanical analyses of geological processes, finite element modeling, and the constitutive modeling of geological materials. He has a background in geoscience, geomechanics and computer software development; and about 20 years of experience in teaching, research, and consulting.

Prior to assuming his current position as Principal Engineer in March 2002, Dr. Ofoegbu served as a senior research engineer at the Southwest Research Institute. In that position, Dr. Ofoegbu led several numerical modeling projects to investigate technical issues related to possible licensing of a geologic repository for high level nuclear waste at Yucca Mountain, such as: Evaluation of a finite element code, ABAQUS, for modeling thermal-mechanical-hydrological coupled processes; and investigations of ground motion patterns resulting from numerically simulated normal fault earthquakes, effects of perched water on thermally driven moisture flow, effects of spatial and time-dependent rock-mass property variations on the stability of underground openings and groundwater flow, and effects of regional crustal density variations on patterns of small-volume basaltic volcanism. Other numerical modeling investigations led by Dr. Ofoegbu include finite element analyses of geologic finite strain for fracture distribution predictions and numerical simulation of a deforming salt body. He has also participated in the development of review procedures for an anticipated license application for the proposed Yucca Mountain repository, technical review of uranium recovery site reclamation plans under the Uranium Mill Tailings Radiation Control Act, and a safety evaluation report for an Independent Spent Fuel Storage Installation.

Dr. Ofoegbu was a research engineer at the University of Toronto for five years, during which time he was the Principal Investigator for an industrial contract on the development and numerical implementation of a constitutive model for geological materials. He developed constitutive models for intact rock, non-lithified soils, and regularly jointed rock mass; implemented the models as user-defined code modules in ABAQUS (a commercially available finite element code); and conducted finite element modeling of the Atomic Energy of Canada Limited's mine-by experiment tunnel.

As an Assistant Professor at the Ahmadu Bello University, Nigeria, in the Department of Civil Engineering, Dr. Ofoegbu taught courses and supervised student research projects in the areas of soil mechanics, earthwork, and foundation engineering, and served as Principal Consultant on industrial site-investigation contracts.

Dr. Ofoegbu has published 25 articles in refereed journals and conference proceedings, as well as several technical reports. He is a member of the International Society for Rock Mechanics and the American Rock Mechanics Association. He is a registered professional engineer in Canada.

Professional Chronology:

Principal Engineer, Southwest Research Institute, March 2002-Present; Senior Research Engineer, Southwest Research Institute, 1993–2002; Consulting Engineer, GI-Johnson Engineering, 1991–93; Research Engineer, University of Toronto, 1987–92; Assistant Professor, Ahmadu-Bello University, 1985–87; Teaching/Research Assistant, University of Toronto, 1980–85; Hydrogeologist, Lower Benue Development Authority, 1978–79; Mathematics/Physics Teacher, Ogun State of Nigeria, 1977–78.

Publications:

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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-ISFSI
	)	
(Independent Spent Fuel Storage Installation)	)	

NRC STAFF TESTIMONY OF
DANIEL J. POMERENING AND GOODLUCK I. OFOEGBU
CONCERNING UNIFIED CONTENTION UTAH L/QQ, PART D
(SEISMIC DESIGN AND FOUNDATION STABILITY)

Q1. Please state your names, occupations, and by whom you are employed.

A1(a). My name is Daniel J. Pomerening ("DJP"). I am employed as a Principal Engineer in the Mechanical and Materials Engineering Division of the Southwest Research Institute ("SwRI"), in San Antonio, Texas. I am providing this testimony under a technical assistance contract between the staff of the Nuclear Regulatory Commission ("NRC Staff" or "Staff") and the Center for Nuclear Waste Regulatory Analyses ("CNWRA"), which is a division of the SwRI. A statement of my professional qualifications is attached hereto.

A1(b). My name is Goodluck I. Ofoegbu ("GIO"). I am employed as a Principal Engineer at the Center for Nuclear Waste Regulatory Analyses ("CNWRA"), which is a division of the Southwest Research Institute ("SwRI"), in San Antonio, Texas. I am providing this testimony under a technical assistance contract between the NRC Staff and the CNWRA of the SwRI. A statement of my professional qualifications is attached hereto.

Q2. Please describe your current responsibilities.

A2(a). (DJP) In my position as Principal Engineer at the Mechanical and Materials Engineering Division, I serve as principal investigator for projects associated with the evaluation of structural design and environmental testing of systems and components, with an emphasis on dynamic loading. Among my responsibilities related to the CNWRA, I currently serve as an Investigator for several projects involving the technical evaluation of facility operation systems, evaluation of the adequacy of design criteria, evaluation of the structural design of the facility, and review of accident analyses.

A2(b). (GIO) In my position as Principal Engineer at the CNWRA, I serve as Principal Investigator for several projects involving geological engineering. My work includes mechanical analysis of underground excavations, foundations, earthworks, and natural geological processes such as faulting and volcanism.

Q3. Please explain what your duties have been in connection with the NRC Staff's review of the application filed by Private Fuel Storage, L.L.C. ("PFS" or "Applicant") for a license to construct and operate an Independent Spent Fuel Storage Installation ("ISFSI") on the Reservation of the Skull Valley Band of Goshute Indians, geographically located within Skull Valley, Utah (the "proposed PFS Facility").

A3(a). (DJP) As part of my official responsibilities, I assisted the Staff in its evaluation of design requirements related to the proposed PFS Facility. My specific role involved conduct of the Staff's evaluation of the Applicant's classification of structures, systems, and components ("SSCs") according to their importance to safety; identification of design criteria and design bases, including external conditions during normal and off-normal operations, accident conditions, and natural phenomena events; assessment of the structural integrity of SSCs, with emphasis on SSCs important to safety; and identification and analysis of hazards for off-normal, accident and design basis events involving SSCs that are important to safety.

In this regard, I assisted in preparation of the Staff's "Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued on September 29, 2000 ("SER"), and Supplement No. 2

to the SER, dated December 21, 2001 ("SSER Supplement No. 2"). Those two documents have now been incorporated into the Staff's "Consolidated Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued in March 2002 ("Consolidated SER").

In addition, among my other duties I assisted the Staff in preparing the "NRC Staff's Response to Applicant's Motion for Summary Disposition of Utah Contention L, Part B," dated December 7, 2001; and I assisted the Staff in preparing responses to various discovery requests which were filed by the State of Utah ("State"), including the Staff's responses of February 1, 2002 (Response to the State's Eighteenth Request); and February 27, 2002 (Response to the State's Twentieth Request).

A3(b). (GIO) As part of my official responsibilities, I assisted the Staff in its evaluation of the Applicant's site characterization and geotechnical analyses of the proposed PFS Facility. Further, I assisted in preparation of the Staff's "Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued on September 29, 2000 ("SER"), and Supplement No. 2 to the SER, dated December 21, 2001 ("SSER Supplement No. 2"). Those two documents have now been incorporated into the Staff's "Consolidated Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued in March 2002 ("Consolidated SER").

In addition, among my other duties I assisted the Staff in preparing its responses to various discovery requests which were filed by the State of Utah ("State"), including the Staff's responses of December 11, 2000 (Response to the State's Eleventh Request); and February 1, 2002 (Response to the State's Eighteenth Request).

Q4. What is the purpose of this testimony?

A4. The purpose of this testimony is to provide the NRC Staff's views concerning the foundation stability and seismic design of the PFS facility, which is the subject of Unified Contention Utah L/QQ, Part D.

Q5. Please summarize the nature of the Staff's evaluation of the Applicant's foundation stability and seismic design of the PFS facility.

A5. As part of the Staff's evaluation of the Applicant's seismic design and the foundation stability of the PFS facility, we conducted a technical review of the Applicant's Safety Analysis Report ("SAR") and supporting documentation. A description of this evaluation is set forth in the Staff's Consolidated SER. Specifically, the Staff's evaluation of these matters may be found *inter alia*, in §§ 2.1.6.4 (Stability of Subsurface Materials); 5.1.1 (Confinement Structures) (and in particular, § 5.1.1.4 (Structural Analysis)); 5.1.3 (Reinforced Concrete Structures) (and in particular, § 5.1.3.4 (Structural Analysis)); 5.1.4 (Other [SSCs] Important to Safety) (and in particular, § 5.1.4.4 (Structural Analysis)); and 15.1.2 (Accidents) (and in particular, § 15.1.2.1 (Cask Tipover), § 15.1.2.2 (Cask Drop), and § 15.1.2.6 (Earthquake)).

Q6. Please identify the Commission's requirements related to the foundation stability and seismic analysis and design for an independent spent fuel storage installation ("ISFSI") that you considered in your review of these matters?

A6. The Commission's requirements governing foundation stability and the seismic analysis and design for an ISFSI are set forth in 10 C.F.R. Part 72. More specifically, with respect to foundation stability, 10 C.F.R. § 72.102(c) requires that an applicant must evaluate the site for its liquefaction potential or other soil instability due to vibratory ground motion. Further, pursuant to 10 C.F.R. § 72.102(c), non-bedrock sites must be evaluated for their liquefaction potential or other soil instability due to vibratory ground motion, and pursuant to 10 C.F.R. § 72.104(d), site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading.

With respect to seismic design, 10 C.F.R. § 72.24(d)(2) requires that an analysis and evaluation be provided of the design and performance of structures, systems, and components important to safety, with the objective of assessing the impact on public health and safety resulting from operation of the ISFSI and including a determination of the adequacy of structures, systems, and components provided for the prevention of accidents and the mitigation of the consequences of accidents, including natural and manmade phenomena and events.

In addition, 10 C.F.R. § 72.122(b)(1) requires that SSCs important to safety be designed to accommodate the effects of, and to be compatible with, site characteristics and environmental conditions associated with normal operation, maintenance, and testing of the ISFSI and to withstand postulated accidents. Further, 10 C.F.R. § 72.122(b)(2) requires, in part, that SSCs important to safety be designed to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform safety functions. The design bases for these SSCs must reflect: (i) appropriate consideration of the most severe of the natural phenomena reported for the site and surrounding area, with appropriate margins to take into account the limitations of the data and the period of time in which the data have accumulated, and (ii) appropriate combinations of the effects of normal and accident conditions and the effects of natural phenomena. In addition, 10 C.F.R. §72.122(b)(2) specifies that an ISFSI should be designed to prevent massive collapse of building structures or the dropping of heavy objects as a result of building structural failure on the spent fuel or onto SSCs important to safety.

Q7. Are you familiar with Unified Contention Utah L/QQ, Part D?

A7. Yes. As admitted by the Licensing Board, Unified Contention Utah L/QO, Part D, states as follows:

Unified Consolidated Contention Utah L/QQ (Geotechnical)

* * *

D. Seismic Design and Foundation Stability.
The Applicant, in its numerous design changes and revisions to the calculations, has failed to demonstrate that the structures and their foundations have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake, and does not

satisfy 10 CFR § 72.102(c) or (d) or § 72.122(b)(2) in the following respects:

1. <u>Seismic Analysis of the Storage Pads, Casks, and Their Foundation Soils</u>

   The Applicant has not demonstrated adequate factors of safety against overturning and sliding stability of the storage pads and their foundation system for the design basis earthquake (DBE) as outlined by NUREG-75/087, Section 3.8.5, "Foundation," Section II.5, *Structural Acceptance Criteria*, because of the following errors and unconservative assumptions made by the Applicant in determining the dynamic loading to the pads and foundations:

   a. In spite of proximity to major active faults, the Applicant's calculations unconservatively assume that only vertically propagating in-phase waves will strike the pads, casks and foundations, and fail to account for horizontal variation of ground motion that will cause additional rocking and torsional motion in the casks, pads and foundations.

   b. The Applicant's calculations incorrectly assume that the pads will behave rigidly during the design basis earthquake. The assumption of rigidity leads to:

      (i) Significant underestimation of the dynamic loading atop the pads, especially in the vertical direction.

      (ii) Overestimation of foundation damping.

   c. The Applicant has failed to provide a realistic evaluation of the foundation pad motion with cement-treated soil under and around the pads in relation to motion of the casks sliding on the pads in that Applicant's evaluation ignores:

      (i) the effect of soil-cement around the pads and the unsymmetrical loading that the soil-cement would impart on the pads once the pads undergo sliding motion,

      (ii) the flexibility of the pads under DBE loading, and

(iii) the variation of the coefficient of sliding friction between the bottom of the casks and the top of the pads due local deformation of the pad at the contact points with the cask.

d. The Applicant has failed to consider lateral variations in the phase of ground motions and their effects on the stability of the pads and casks.

e. The Applicant's calculations for cask sliding do not address the frequency dependency of the spring and damping values used to model the foundation soils.

f. The Applicant has failed to consider the potential for cold bonding between the cask and the pad and its effects on sliding in its calculations.

g. The Applicant has failed to analyze for the potential of pad-to-pad interaction in its sliding analyses for pads spaced approximately five feet apart in the longitudinal direction.

h. In an attempt to demonstrate cask stability, the Applicant's calculations use only one set of time histories in its non-linear analysis. This is inadequate because:

(i) Nonlinear analyses are sensitive to the phasing of input motion and more than one set of time histories should be used.

(ii) Fault fling (*i.e.*, large velocity pulses in the time history) and its variation and effects are not adequately bounded by one set of time histories.

i. Because of the above errors, omissions and unsupported assumptions, the Applicant has failed to demonstrate the stability of the free standing casks under design basis ground motions. Thus, the Applicant's analyses do not support the Applicant's conclusions that excessive sliding and collision will not occur or that the casks will not tip over. 10 CFR § 72.122(b)(2) and NUREG-1536 at 3-6.

2. Seismic Analysis of the Canister Transfer Building and its Foundation

The Applicant has not demonstrated adequate factors of safety against overturning and sliding stability of the CTB and its foundation system for the design basis earthquake as outlined by NUREG-75/087, Section 3.8.5, "Foundation," Section II.5, *Structural Acceptance Criteria*, because of the following errors and unconservative assumptions made by the Applicant in determining the dynamic loadings to the CTB and its mat foundation:

a. The Applicant's calculations incorrectly assume that the CTB mat foundation will behave rigidly during the DBE. The assumption of rigidity leads to:

(i) Significant underestimation of the dynamic loading to the mat foundation.

(ii) Overestimation of foundation damping.

b. The Applicant's calculations ignore the presence of a much stiffer, cement-treated soil cap around the CTB. This soil cap impacts:

(i) Soil impedance parameters.

(ii) Kinematic motion of the foundation of the CTB.

c. The Applicant's calculations are deficient because they ignore the out-of-phase motion of the CTB and the cement-treated soil cap, which potentially can lead to the development of cracking and separation of the cap around the building perimeter.

d. The Applicant's calculations unconservatively assume that only vertically propagating in-phase waves will strike the CTB and its foundations, and fail to account for horizontal variation of ground motion that will cause additional rocking and torsional motion of the CTB and its foundations.

Q8. In Part D of the contention, concerning seismic design and foundation stability, the State asserts that "[t]he Applicant, in its numerous design changes and revisions to the calculations, has failed to demonstrate that the structures and their foundations have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake, and does not satisfy 10 CFR § 72.102(c) or (d) or § 72.122(b)(2)". Do you agree with this assertion?

A8. No.

Q9. Please provide the bases for this conclusion.

A9. Based on the Staff's review of the Applicant's SAR and supporting calculations and analyses, the Staff has concluded that the Applicant has properly demonstrated that the proposed PFS Facility structures and foundations have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake ("DBE"), and that the seismic design and foundation stability of the proposed PFS Facility satisfy the requirements of 10 C.F.R. §§ 72.102(c) and (d), and 72.122(b)(2). These matters are discussed in greater detail in the Staff's Consolidated SER, in sections 2.1.6.4 (Stability of Subsurface Materials), 4.1.3.2 (Structural), 5.1.1.1 (Description of Confinement Structures), 5.1.1.4 (Structural Analysis of Confinement Structures), 5.1.3.1 (Description of Reinforced Concrete Structures), 5.1.3.4 (Structural Analysis for Reinforced Concrete Structures), 5.1.4.1 (Description of Other SSCs Important to Safety), 5.1.4.4 (Structural Analysis of Other SSCs Important to Safety), and 15.1.2.6 (Earthquake).

The bases for this conclusion are further provided in greater detail below, with respect to the specific assertions made in Subparts 1 and 2 of Part D of this contention.

Contention Utah L/QQ, Part D.1: Casks, Storage Pads, and Their Foundation Soils

Q10. The State asserts in Part D.1 of this contention that the Applicant has failed to demonstrate that the storage pads, casks, and their foundation soils have adequate factors of safety to sustain the dynamic loading from the proposed design basis earthquake ("DBE"). Do you believe that this presents a valid concern?

A10. No.

Q11. Please provide the bases for this conclusion, with respect to (a) storage pad foundation stability, and (b) the seismic design of the storage pads and casks, respectively.

Storage Pad Foundation Stability

A11(a). (GIO) As stated in my testimony with respect to Subpart C.3.d of Contention Utah L/QQ, and as set forth in Section 2.1.6.4 of the Consolidated SER, the Staff has concluded that the storage pad design proposed by PFS satisfies the requirements of 10 C.F.R. § 72.102(c) and (d), and § 72.122(b)(2), with respect to the capability of the underlying soil to provide adequate support to the storage pads. This conclusion is based on the following considerations.

First, calculations provided by PFS demonstrate adequate safety margins against bearing capacity failure of the pads under combined static loads and potential dynamic loading from the design-basis earthquake. The existence of adequate safety margins against bearing capacity failure implies adequate safety margins against overturning, because overturning of a storage pad could not occur without a bearing-capacity failure of the underlying soil.

Second, calculations provided by PFS demonstrate that potential sliding of the storage pads under seismic loading would not constitute a safety hazard, as there are no safety-related external connections to the pads or casks that may rupture or be misaligned as a result of pad sliding. Indeed, the Staff agrees with the Applicant that the storage casks are less likely to tip over if the pads are free to slide.

Third, calculations provided by PFS demonstrate that the storage pads can be expected to undergo post-construction settlement of about 3 to 4 inches, taking into account both static loads and potential dynamic loading from the design-basis earthquake. PFS has committed to perform maintenance repair of the pad-emplacement area as necessary to correct any changes caused by settlement (for example, PFS has indicated that it would scrape aggregates from between the pads to maintain the top surface of the aggregate layer at the same elevation as the top surface of the pads). Thus, settlement of the pads does not present a foundation stability concern.

Seismic Design of the Storage Pads and Casks

A11(b). (DJP) Based on a review of the PFS application and supporting analyses and calculations, the Staff has concluded that the Applicant has demonstrated that the ability of the cask and storage pad with its foundations to perform their safety function and limit the impact on public health and safety, as required by 10 C.F.R. §72.24(d) and that performance is not influenced by the dynamic loading from the design basis earthquake, as required by 10 C.F.R. §72.122(b)(1) and (2). This conclusion is based on the following considerations.

First, as discussed by Dr. Ofoegbu above, the Staff has concluded that PFS has demonstrated the stability of the storage pads against overturning, and that potential sliding of the storage pads does not affect the performance of their safety function under dynamic loading from the DBE. Further, the 3 to 4 inches of potential settlement is smaller than the maximum tolerable settlement of the CTB foundation.

Second, as summarized in the Consolidated SER, Sections 5.1.4.4 and 15.1.2.6 (pages 5-30 and 15-32), the HI-STORM 100 storage casks will not tipover, slide excessively or collide into each other on the storage pad as a result of credible natural phenomena, including a design basis seismic event. This conclusion is based on the Staff's review of the PFS site-specific analysis performed by Holtec International ("Holtec"), which demonstrated that the HI-STORM 100 storage casks will not tip over or collide into each other in the event of a PFS Facility design basis ground motion.

In this regard, the Holtec PFS site-specific analysis considered soil-structure interaction, actual storage pad site conditions, and a variety of cask placement configurations on the storage pad, and was performed to provide a bounding solution. Two bounding cases for the interface between the storage pads and supporting soil were considered. For the first case, it was assumed that the concrete pad, the soil-cement layer, and the underlying soil were fully bonded. For the second case, the concrete pad and soil-cement layer were allowed to slide when frictional resistance exceeded the limits. In both cases, two bounding coefficients of friction for the cask-pad interface were analyzed: (1) a coefficient of friction equal to 0.2, for an evaluation of cask sliding potential, and (2) a coefficient of friction equal to 0.8, for an evaluation of cask tipover potential. The acceptance criterion was that the casks must be stable, in the sense that the center of the top cover of the cask must remain within the original contact circle that the cask makes with the pad. The use of these upper and lower bound coefficients of friction between the casks and storage pads is an acceptable approach, in that the response of the casks and storage pads under these bounding conditions will bound the responses for all realistic conditions. As indicated in the Applicant's calculations, the maximum rocking at the top was less than 4 inches. With a cask height of 231.25 inches, the rocking was found to be less than 1 degree. This is significantly less than the angle required for tipover, which is stated to be approximately 29 degrees. The maximum sliding of the cask was found to be less than 3 inches. This is significantly less than the spacing between the casks themselves and the edge of the pad. Consequently, based on its review of the PFS site-specific analysis, the Staff concurred with the Applicant's conclusion that the cask will not tipover, slide off the pad, or impact adjacent casks during a site-specific design basis earthquake. This conclusion is stated in the Staff's Consolidated SER, at pages 5-30 and 15-32.

Further, a non-mechanistic, a hypothetical cask tipover scenario was analyzed by the Applicant for site-specific conditions, to evaluate the performance of the HI-STORM 100 storage cask design at the proposed PFS site. The Staff reviewed Applicant's method of analysis, inputs, assumptions, and conclusions. Based on its review, the Staff concluded that the deceleration in a hypothetical cask tipover event is less than 45 g, and the resulting stresses in the multi-purpose canister ("MPC") within the HI-STORM 100 storage cask will be lower than the stresses evaluated in the HI-STORM 100 Cask System Final Safety Analysis Report ("FSAR"). The Commission has previously approved the HI-STORM 100 storage cask, as reflected in NRC Certificate of Compliance No. 1014 and in the Staff's associated HI-STORM 100 Safety Evaluation Report ("HI-STORM SER"). Based on the Staff's review of the Applicant's hypothetical cask tipover event at the PFS site, the Staff has determined that the conclusions in its HI-STORM 100 SER, with respect to the structural integrity of the MPC, are valid for the PFS Facility.

In addition, the Staff reviewed the Applicant's assessment of the structural adequacy of the PFS storage pads under seismic loading, as summarized in the Consolidated SER, Section 5.1.3.4. The cask storage pads have been designed in accordance with ANSI/ANS–57.9–1992 and American Concrete Institute ("ACI") standard ACI 349–90, as is appropriate. The ultimate strength method of analysis was used with the appropriate load factors, and the design of the storage pads accounts for both the weight of the loaded storage casks and the design earthquake for the proposed PFS site. Accordingly, the PFS analysis shows that the structural capacity of the storage pads exceeds the demand, and the requirements of ACI 349-90 are satisfied.

The Applicant's storage pad analysis and design calculation package includes static analysis with both dead and live loads using the CECSAP computer code. The storage pad was modeled using a three-dimensional, flat-shell finite element model. The gross uncracked stiffness of the storage pad was used for the model. Vertical springs were used to model the upper, best, and lower bounds of the soil support of the pads for the long-term static load conditions. The Staff found that the Applicant's methods of analysis, inputs, and assumptions were acceptable, based on a review of the input files for the static analysis, including geometry, soil parameters, and loading inputs, and the parameters used in the Applicant's analyses are consistent with the physical geometry and characteristics of the structural design and characteristics of the materials, including soil. The results of the static pad analysis for dead and live loads of cask weights are summarized in Table 4.2-7 of the Applicant's SAR. The Staff reviewed the procedures used to determine the ultimate static moment and shear capacity calculation for the reinforced concrete slab and found them to be consistent with industry practice, as identified in ACI 349–90. Considering the static pad analysis, the Staff concluded that the storage pad, as designed, provides adequate strength for accommodating the design loading conditions.

Dynamic analyses for the storage pads were also performed by PFS, for the site-specific probabilistic seismic hazard analysis ("PSHA") design basis earthquake, using both the CECSAP and SASSI computer codes. The results of the dynamic pad analysis are summarized in Table 4.2-8 of the Applicant's SAR. The Staff reviewed the procedures used to determine the moment and shear capacity calculation for the reinforced concrete slab and found them to be consistent with industry practice, as specified in ACI 349-90. Considering the dynamic pad analysis, the Staff concluded that the storage pad as designed provides adequate strength for accommodating the site-specific seismic loading conditions.

These static and dynamic analyses by PFS confirm the foundation stability and structural adequacy of the reinforced concrete storage pad for supporting the storage casks when subjected to the design loading conditions. Based upon its review, the Staff concluded that the structural analysis performed by PFS demonstrates that the cask storage pads are adequately designed to resist the loads based on the site characteristics and environmental conditions during normal operations and during postulated off-normal and accident events, in accordance with the requirements of 10 C.F.R. § 72.122 (b)(1). Further, the Staff concluded that the structural analysis performed by PFS demonstrates that the cask storage pads are designed to withstand the effects of natural phenomena, such as earthquakes, without impairing the capacity to perform safety functions, in accordance with the requirements of 10 C.F.R. § 72.122(b)(2). The bases for the Staff's conclusions are provided in greater detail in the following sections of the Consolidated SER: Sections 2.1.6.4 (Stability of Subsurface Materials); 4.1.3.2 (Structural); 5.1.1.1 (Description of Confinement Structures); 5.1.1.4 (Structural Analysis of Confinement Structures); 5.1.3.1 (Description of Reinforced Concrete Structures); 5.1.3.4 (Structural Analysis for Reinforced Concrete Structures); 5.1.4.1 (Description of Other Structures, Systems, and Components Important to Safety); 5.1.4.4 (Structural Analysis Other Structures, Systems, and Components Important to Safety); and 15.1.2.6 (Earthquake).

Q12. The State asserts in Subpart D.1.a of this contention that in spite of the PFS site's proximity to major active faults, "the Applicant's calculations unconservatively assume that only vertically propagating in-phase waves will strike the pads, casks, and foundations, and fail to account for horizontal variation of ground motion that will cause additional rocking and torsional motion in the casks, pads and foundations." Do you agree that this is a valid concern?

A12. (DJP) No.

Q13. Please provide the bases for this conclusion.

A13. (DJP) As identified in the Staff's Consolidated SER, Section 2.1.6.2 (Ground Vibration and Exemption Request), and specifically, the discussion under "Design-Basis Ground Motion" (page-2-47), the design ground motion response spectra for the proposed PFS site were developed by Geomatrix Consultants, Inc. ("Geomatrix"), based on its site-specific PSHA results and the procedures outlined in Regulatory Guide 1.165. Prior to implementing the Regulatory Guide 1.165 procedure, the site seismic hazard results were modified to account for the near-source effects of rupture directivity and the polarization of ground motions. As stated in the Consolidated SER (page 2-47), the Staff determined that the deterministic approach of shifting the seismic hazard results to account for rupture directivity and ground motion directional effects is conservative for the frequencies to which these adjustments were applied, and was an acceptable step in the development of the Applicant's PSHA. This consideration provides one indication that vertically propagating in-phase waves are accounted for in the Applicant's calculations.

The Applicant recently provided a calculation by Geomatrix on the evaluation of the spatial and temporal variations of ground motions for the proposed PFS Facility site. In this calculation, Geomatrix concludes that: (1) the angle of incidence of the seismic waves is such that the assumption of vertically propagating waves is reasonable for the site; (2) the small time difference for wave arrivals would have a negligible effect on the analysis; (3) the additional rocking and torsional motion of the pad caused by inclined incident waves is insignificant compared to the motion caused by the vertically propagating waves; (4) for the small pad size of interest, nearly all of the power in the ground motion can be represented by a vertically propagating plane wave; and (5) the site's proximity to the major active faults does not require special evaluation of the effects of spatial variation. I have discussed these Geomatrix conclusions with Dr. Martin McCann, who provided seismic/geotechnical input into the Consolidated SER. Dr. McCann indicated that he concurs with the conclusions reached by Geomatrix.

The following considerations support a conclusion that non-vertically propagating waves will not cause additional rocking and torsional motion in the casks, pads and foundations.

First, one must consider the ray path for the seismic waves at the surface relative to source points on the Stansbury and East faults. As identified in the Geomatrix calculation concerning spatial and temporal variations of ground motions for the proposed PFS Facility, the angle of incidence is typically less than 10°. Geomatrix therefore arrived at the conclusion that "the proximity of the site to the major active faults does not result in a high angle of incidence waves measured from vertical (i.e., low angle measured from horizontal) and the assumption of vertically propagating waves is reasonable for the site." Based on my review of the procedure followed by Geomatrix and the results presented in its report, I agree with this conclusion.

Second, one must consider the arrival time for the seismic waves at the surface relative to source points on the Stansbury and East faults. As identified in the Geomatrix calculation concerning spatial and temporal variations of ground motions for the proposed PFS Facility, the difference in arrival time is on the order of 0.001 to 0.002 seconds. This time difference is such that it would affect only frequencies above the highest ground motion frequency of interest (*i.e.,* 50 Hz). Accordingly, the assumption of in-phase waves is reasonable for the proposed PFS Facility site. Based on my review of the procedure followed by Geomatrix and the results presented in its report, I agree with this conclusion.

In sum, based on its review of the Geomatrix calculation with respect to the occurrence of seismic waves in the soil, the Staff concludes that the assumption of vertically propagating in-phase waves is reasonable at the proposed PFS site.

Additional supporting bases for this conclusion include the following considerations, concerning the Applicant's localized inputs into its structural analysis of the storage pads, with respect to seismic waves that may occur in the soil. The seismic input used in the Applicant's calculations consists of three orthogonal and statistically independent time histories to characterize the motion at the surface. Acceleration is defined for two orthogonal horizontal directions as well as the vertical direction. The vector sum of these three components results in input motion that is random with respect to both amplitude and direction. Therefore, the input to the pads, casks, and foundation cannot be characterized as the result of only vertically propagating waves -- and the Applicant's analysis therefore did include motions other than vertically propagating waves.

Further, one must consider the influence of storage pad geometry (30 ft x 67 ft) on the pad motion with respect to the wave length of the seismic waves. The shear and compression wave velocities in the soil will define the velocity at which the seismic waves will propagate through the site. As given in Calculation G(PO18)-2, the upper range, best estimate, and lower range for the compression wave velocities are 2205 ft/sec, 1527 ft/sec, and 1157 ft/sec, respectively. As also given in Calculation G(PO18)-2, the upper range, best estimate, and lower range for the shear wave velocities are 1322 ft/sec, 842 ft/sec, and 579 ft/sec, respectively. The wave velocities, when combined with information on the geometry of the storage pad, define the frequency that will most adversely affects the structural integrity and rocking of the storage pad.

In this regard, a seismic wave whose length is twice the pad length/width will produce maximum bending in the pad, whereas a seismic wave whose length is four times the pad length/width will produce maximum rocking of the pad. Also, the bounding response of the pad can be defined in terms of the peak displacements; and an estimation of the peak displacement response can be calculated from the response acceleration at a given frequency divided by the frequency squared. The response acceleration corresponds to the design basis fault normal (FN), fault parallel (FP), and vertical (V) response spectra.

For the 30 ft (E-W) direction, the lowest frequency (*i.e.*, the frequency that has the highest displacement) is determined based on the lower bound estimate of the wave velocities. Accordingly, the maximum bending in the storage pads will occur at 9.7 Hz with corresponding vertical response displacements of: V = 0.19 inches. The maximum rocking of the storage pads will occur at 4.8 Hz, based on shear wave velocity with corresponding vertical response displacements of: V = 0.60 inches. The maximum vertical deflection satisfies the requirement of Table 9.5(a) of ACI 349 ($\delta_{max} \leq$ 1.8 inches = 30*12/200). The amount of rotation of the surface of the storage pad is less than 0.1 degrees and, therefore, the stability of the cask will not be affected by non-vertically out-of-phase seismic waves that may occur at the site.

For the 67 ft (N-S) direction, the lowest frequency (*i.e.*, the frequency that has the highest displacement) is also determined based on the lower bound estimate of the wave velocities. Accordingly, the maximum bending in the storage pads will occur at 4.3 Hz, based on the shear wave velocity with corresponding vertical response displacement of: V = 0.68 inches. The maximum rocking of the storage pad will occur at 2.2 Hz, based on shear wave velocity with corresponding response displacements of : V = 1.16 inches. The maximum vertical deflection satisfies the requirement of Table 9.5(a) of ACI 349 ($\delta_{max} \leq 4.2$ inches = 67*12/200). The amount of rotation is less than 0.1 degrees and, therefore, the stability of the cask will not be affected by non-vertically out-of-phase seismic waves that may occur at the site.

Q14. The State asserts in Subpart D.1.b. of this contention that "the Applicant's calculations incorrectly assume that the pads will behave rigidly during the design basis earthquake," and that this "assumption of rigidity" leads to "significant underestimation of the dynamic loading atop the pads, especially in the vertical direction," and "overestimation of foundation damping." Do you believe that this is a valid concern?

A14. (DJP) No.

Q15. Please provide the bases for this conclusion.

A15. (DJP) As stated in response to Question 13 above, the bounding displacements of the storage pad due to seismic waves is small relative to the overall geometry of the storage pad. Therefore, the difference between assuming a rigid or flexible storage pad will be minimal. Accordingly, for all practical purposes, the storage pad can be assumed to be a rigid element.

In the Applicant's calculations of the response of multiple casks on the storage pad due to the 2000-yr seismic event (Holtec Report No. HI-2012640), the storage pad is conservatively assumed to the rigid. The HI-2012640 analysis is used to identify the stability of the casks on the storage pad, and follows the guidelines identified in ASCE 4-86, Section 3.1.8. The rigid storage pad assumption will produce conservative results, since no energy will be absorbed in deformation of the storage pad; rather, the energy will be transmitted to the cask, resulting in an upper bound estimate of the displacement response of the casks. The HI-2012640 analysis is also used to define the loads that are used in the Applicant's storage pad analysis and design, in Calculation G(PO17)-2. The assumption of a rigid storage pad will result in a conservative estimate of the loads transmitted to the pad by the casks. The cask motion is maximized relative to the storage pad, so the resulting loads transmitted to the storage pad will represent an upper bound.

In the Applicant's calculations for the storage pad analysis and design (G(PO17)-2), the storage pad is assumed to be flexible, in order to identify the amount and placement or reinforcing required to resist the maximum or bounding loads that need to be considered in the design of the pads. The capacity of the storage pads exceeds the demand that would be imposed. A summary of the maximum displacements are given in Tables S-2, D-1(a), D-1(b), and D-1(c) of G(PO17)-2. The distribution of vertical displacements over the extent of the storage pad show that the storage pad responds as a flexible body under the various load conditions. The maximum relative vertical displacement in the storage pad under static loading is 8.9 inches (Table S-2), and under dynamic loading is 0.4 inches (Table D-1(c)). Again, the bounding displacements calculated for the storage pad is small relative to the overall geometry of the storage pad. Therefore, the difference between assuming a rigid or flexible storage pad will be minimal. Accordingly, for all practical purposes the storage pad can be assumed to be a rigid element. For these reasons, the Applicant did not "incorrectly assume that the pads will behave rigidly during the design basis earthquake," and did not "significant underestimat[e] the dynamic loading atop the pads," in the horizontal or vertical direction.

Finally, the State is incorrect in asserting that the Applicant's "assumption of rigidity" for the pad leads to "overestimation of foundation damping." As stated above, the vertical deformation of the storage pad is small relative to the overall geometry of the pad. Therefore, the soil will effectively "see" the storage slab as a rigid element, and the foundation damping assumption used by the Applicant is acceptable.

Q16. The State asserts in Subpart D.1.c. of this contention that "the Applicant has failed to provide a realistic evaluation of the foundation pad motion with cement-treated soil under and around the pads in relation to motion of the casks sliding on the pads," in three specified respects: (i) unsymmetrical loading on the pads imparted by the soil-cement around the pads; (ii) flexibility of the pads under DBE loading; and (iii) variation of the coefficient of sliding friction between the bottom of the casks and the top of the pads, due to local deformation of the pad at the contact points with the cask. Do you believe that this is a valid concern?

A16. (DJP) No.

Q17. Please provide the bases for this conclusion.

A17. (DJP) First, with respect to unsymmetrical loading on the pads due to the soil-cement around the pads, the soil cement should not result in significant additional loads on the storage pad, or additional rocking instability or excessive sliding of the casks on the storage pad. If separation occurs between the storage pad and the surrounding soil/cement, the resulting impact load that would occur during closure of the gap will not result in significant additional loads on the storage pad, additional rocking instability of the storage casks, or excessive sliding of the storage casks on the storage pad. The storage pad is to be constructed of reinforced concrete with a compressive strength of f_c' = 3000 psi and a density of w = 150 pcf. The modulus of elasticity (E_c) for concrete is computed with reasonable accuracy from the empirical equation $E_c = 33w^{3/2}\sqrt{f_c'}$. For 3000 psi concrete, the modulus of elasticity is 3,120,000 psi. It should be noted that this is a static modulus. The Applicant has indicated that it will used soil cement around the storage pad with a nominal unconfined compressive strength of 250 psi and a density of 100 pcf. Based on the information presented by Doshi and Mesdary on estimations of the dynamic modulus of soil-cement, the soil-cement surrounding the storage pads will have a dynamic modulus of 228,000 psi. There is a significant difference between the compressive strength and modulus of elasticity of the cement storage pad (3000 psi and 3,120,000 psi), and the compressive strength and the dynamic modulus of the soil/cement (250 psi and 228,000 psi). In all cases, the soil /cement is much softer and will tend to crush under impact with the storage pad. This crushing will distribute any loading over an longer period of time resulting in a lower peak force and acceleration. As a result, there will be no significant additional loads on the storage pad, or additional rocking instability or excessive sliding of the casks on the storage pad.

Second, with respect to the flexibility of the pads under DBE loading, as discussed above the storage pad is considered to be rigid for the Applicant's calculation of the response of the storage pads and casks under seismic load conditions in Holtec Report No. HI-2012640. As discussed above, this is a valid assumption.

Third, with respect to variations of the coefficient of sliding friction between the bottom of the casks and the top of the pads due to local deformation of the pad at the contact points with the cask, the Applicant's calculation (Holtec Report No. HI-2012640) assumes a bounding set of coefficients of friction of 0.2 and 0.8 in the analysis of the response of the casks on the storage pad. These values effectively cover the range of friction coefficients that will be present for the steel-to-concrete interface for the contact area between the cask and the storage pad. Friction arises on a microscopic scale because of the roughness and interactions of the surfaces. In addition, friction depends only on the magnitude of the force normal to the surface, and does not depend on the contact area between surfaces. The coefficients of friction utilized by the Applicant represent an average of the values that may be present over the contact area between the cask and the storage pad. Within the overall contact area, there may be regions with different local coefficients of friction. However, when averaged over the entire contact area, the composite is a coefficient of friction that is representative of the bounding values used in the Applicant's calculations (Holtec Report No. HI-2012640). Finally, with respect to the amount of deformation in the storage slab due to the placement of the casks, the Staff does not consider this to be of concern, for the reasons stated in response to Question 23 below.

Q18. The State asserts in subpart D.1.d. of this contention that "the Applicant has failed to consider lateral variations in the phase of ground motions and their effects on the stability of the pads and casks." Do you believe that this is a valid concern?

A18. (DJP) No.

Q19. Please provide the bases for this conclusion.

A19. (DJP) For the reasons stated in response to Question 13 above, the assumption of vertically propagating in-phase waves at the site is appropriate. If the seismic waves arriving at the site are in-phase, there will not be lateral variation in the phase of the ground motion. The Geomatrix calculation concerning spatial and temporal variations of ground motions for the proposed PFS Facility shows that the difference in arrival time for the seismic waves at the surface relative to source points on the Stansbury and East faults is on the order of 0.001 to 0.002 seconds. This time difference is such that it would affect only frequencies above the highest ground motion frequency of interest (*i.e.,* 50 Hz). Accordingly, Geomatrix concludes that the assumption of in-phase waves with respect to the storage pads at the proposed PFS Facility site is reasonable. Based on my review of the procedure followed by Geomatrix and the results presented in its report, I agree with this conclusion. Therefore, lateral variations in the phase of ground motion and their effect on the stability of the storage pads and casks will be insignificant.

Q20. The State asserts in Subpart D.1.e. of this contention that "the Applicant's calculations for cask sliding do not address the frequency dependency of the spring and damping values used to model the foundation soils." Do you believe that this is a valid concern?

A20. (DJP) No.

Q21. Please provide the bases for this conclusion.

A21. (DJP) The Holtec calculation (Holtec Report No. HI-2012640) of the impact of multiple casks on the storage pads due to seismic loading uses a set of three springs and dampers to represent the soil under dynamic loading. The springs represent the site-specific resistance to motion in the two horizontal directions as well as the vertical direction. When coupled with the mass of the storage pad and casks, these springs accurately represent the dynamic response of the first modes of vibration of the rigid storage pad with casks on the soil. (A mode of vibration is defined in the Shock and Vibration Handbook as "a characteristic pattern assumed by the system in which the motion of every particle is simple harmonic with the same frequency.") This procedure is consistent with the requirements of ASCE 4-98, "Standard for Seismic Analysis of Safety Related Nuclear Structures." This modeling provides an accurate representation of the lowest frequency of response of the storage pads, and therefore accurately predicts the maximum displacement of the storage pads. The maximum displacement of the storage pads will tend to reduce the maximum rocking and sliding of the casks on top of the storage pad.

Q22. The State asserts in Subpart D.1.f. of this contention that "the Applicant has failed to consider the potential for cold bonding between the cask and the pad and its effects on sliding in its calculations." Do you believe that this is a valid concern?

A22. (DJP) No.

Q23. Please provide the bases for this conclusion.

A23. (DJP) The stress on the concrete storage pad due to the placement of casks is determined by dividing the weight of the cask by the contact area. For a 360,000 lb cask with a diameter of 132.5 inches, the bearing stress is 26 psi. This stress is well below the allowable bearing stress of 1785 psi, in concrete with a compressive strength of 3000 psi, as calculated pursuant to Section 10.15 of ACI 349-90. The resulting initial strain in the concrete is determined by dividing the stress by the modulus of elasticity. For concrete with a compressive strength of 3000 psi, the modulus of elasticity is 3,120,000 psi, as calculated pursuant to Section 8.5 of ACI 349-90. Therefore, the initial strain in the concrete is 8.33 micro-inches/inch. If one assumes that the initial strain is constant over the full depth of the storage pad (*i.e.,* 36 inches), the resulting deformation is 300 micro-inches. If one also considers the influence of long term creep as identified in ACI 209, "Prediction of Creep, Shrinkage, and Temperature Effect in Concrete Structures," the amount of creep strain for normal concrete is 18.6 micro-inches/inch for a load application duration of 20 years. Again, considering that the creep strain is constant over the full depth of the storage pad, the resulting creep deformation is 672 micro-inches. Combining the initial and creep deformations gives a total deformation of 972 micro-inches. This is an insignificant amount of deformation, which will not result in cold-bonding of the cask and storage pad and will not have any influence on the overall stability of the casks on the storage pads under seismic load conditions.

Q24. The State asserts in Subpart D.1.g. of this contention that "the Applicant has failed to analyze the potential of pad-to-pad interaction in its sliding analysis for pads spaced approximately five feet apart in the longitudinal direction." Do you believe that this is a valid concern?

A24. (DJP) No.

Q25. Please provide the bases for this conclusion.

A25. (DJP) The Applicant has determined that potential sliding of the storage pads under seismic loading would not constitute a safety hazard. For the reasons discussed in response to Question 17 above, the soil-cement between the pads will tend to crush under seismic loading. Crushing of the soil-cement will limit the magnitude of the force that can be transmitted from one pad to another. Because of the low magnitude of force that can be transmitted through the soil-cement layer between the storage pads, the influence on the structural integrity of the storage pads and the stability of the casks will be minor. Therefore the potential of pad-to-pad interaction in its sliding analysis for pads spaced approximately five feet apart in the longitudinal direction, is not a valid concern.

Q26. The State asserts in Subpart D.1.h. of this contention that "in an attempt to demonstrate cask stability, the Applicant's calculations use only one set of time histories in its non-linear analysis." This is asserted to be inadequate because (i) "non-linear analyses are sensitive to the phasing of input motion and more than one set of time histories should be used, and (ii) "fault fling (i.e., large velocity pulses in the time history) and its variation and effects are not adequately bounded by one set of time histories." Do you believe that this is a valid concern?

A26. (DJP) No.

Q27. Please provide the bases for this conclusion.

A27. (DJP) With respect to the number of time histories utilized, the nonlinear analysis performed by Holtec (Holtec Report No. HI-2012640) was based on only one set of time histories. These time histories have been shown to be random with respect to both amplitude and direction, as discussed in response to Question 13 above. The Staff concurs with the statement in Stone & Webster Calculation G(PO18)-3 ("Development of Time Histories for 2000-Year Return Period Design Spectra"), that these time histories satisfy the criteria specified in NRC guidance documents, specifically, Section 3.7.1 ("Seismic Design Parameters") of NUREG-0800 and

Section 5 of NUREG-1567. Under this loading, the maximum calculated cask displacement in the global horizontal directions is 3 inches. This is significantly less than the 48-inch separation between the casks. An approximation of the maximum rocking is given by the arctangent of the ratio of the maximum difference between the position of the top and bottom of the cask and the height of the cask. Under the Applicant's analysis, the difference between the maximum and minimum displacements at the top and bottom of the cask is 4 inches. The bounding value for rotation of the cask can be estimated by the difference in horizontal position (4 inches) divided by the cask height (231.25 inches). This bounding value is less than 1 degree, which is significantly less than the approximately 29 degrees required for tipover. The margin of safety is sufficiently high that multiple sets of time histories are not warranted. It should be noted that comparable results (*i.e.,* insignificant horizontal and vertical displacement of the casks) were obtained in the Staff's confirmatory analysis of cask/pad behavior, performed by Dr. Vincent Luk of Sandia National Laboratories utilizing several sets of time histories, as is discussed in his testimony filed herewith.

With respect to "fault fling," the Staff considers that phenomenon not to be applicable here. More specifically, I have discussed this matter with Drs. John Stamatakos and Martin McCann, who were responsible for the evaluation of seismic issues in the Consolidated SER. They indicated that fault fling is not an appropriate consideration for the for the type of faulting present at the proposed PFS site. In this regard, they indicated that under the general consensus, fault fling is a potential issue for strike-slip faults, whereas the Stansbury and East faults at the proposed PFS site are normal faults. Thus, fault fling is not a concern at the proposed PFS site.

Q28. The State asserts in Subpart D.1.i. of this contention that, for the reasons stated in previous subparts of Part D of the contention, "the Applicant has failed to demonstrate the stability of the free standing casks under design basis ground motions," and therefore, "the Applicant's analyses do not support [its] conclusions that excessive sliding and collision will not occur or that the casks will not tip over." Do you believe that this is a valid concern?

A28. (DJP) No.

Q29. Please provide the bases for this conclusion.

A29. (DJP) The Applicant has demonstrated that the ability of the structures and their foundations to perform their safety function and limit the impact on public health and safety, as required by 10 C.F.R. § 72.24(d), are not influenced by the dynamic loading from the DBE as required by 10 C.F.R. § 72.122(b)(2). The Applicant has demonstrated that the casks will not tip over, impact adjacent casks, or slide off the pads under the dynamic loading from the DBE. In particular, the Staff's review of the Holtec site-specific analysis for the proposed PFS Facility leads it to conclude that the HI-STORM 100 storage casks will not experience excessive sliding, collide with each other, or tipover in the event of a design basis earthquake at the proposed PFS site. Additional bases for this conclusion are stated with respect to the other portions of this contention, as set forth above.

Contention Utah L/QQ, Subpart D.2 (Canister Transfer Building and its Foundation)

Q30. The State asserts in Subpart D.2. of this contention that "the Applicant has not demonstrated adequate factors of safety against overturning and sliding stability of the CTB and its foundation system for the [DBE] as outlined by NUREG-75/087, Section 3.8.5, 'Foundation,' because of . . . errors and unconservative assumptions made by the Applicant in determining the dynamic loadings to the CTB and its mat foundation." Do you believe that this is a valid concern?

A30. No.

Q31. Please provide the bases for this conclusion, with respect to (a) CTB foundation stability, and (b) the seismic design of the CTB.

CTB Foundation Stability

A31(a). (GIO) Pursuant to 10 C.F.R. §§ 72.102(c)-(d), an applicant for an ISFSI at a non-bedrock site is required to conduct an evaluation of the site for its liquefaction potential or other soil instability due to vibratory ground motion, as well as site-specific investigations and laboratory analyses to show that soil conditions are adequate for the proposed foundation loading. In addition, pursuant to 10 C.F.R. § 72.122(b)(2), SSCs important to safety must be designed to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform their safety functions.

As set forth in Section 2.1.6.4 of the Staff's Consolidated SER, and as discussed in my testimony with respect to Subpart C.3.d of Contention Utah L/QQ, the Staff has concluded that the Applicant has provided an adequate geotechnical site characterization, and that the design of the CTB foundation proposed by PFS satisfies the requirements of 10 C.F.R. § 72.102(c) and (d) and § 72.122(b)(2), with respect to the capability of the underlying soil to provide adequate support to the foundation. This conclusion is based on the following considerations.

First, calculations provided by PFS demonstrate adequate safety margins against bearing capacity failure of the CTB foundation under combined static loads and potential dynamic loading from the design-basis earthquake. The existence of adequate safety margins against bearing capacity failure implies adequate safety margins against overturning, because overturning of the CTB foundation could not occur without a bearing-capacity failure of the underlying soil.

Second, calculations provided by PFS demonstrate adequate safety margins against sliding of the CTB foundation under potential dynamic loading from the design-basis earthquake, in accordance with section 3.8.5 of NUREG-0800, Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants (which supersedes Section 3.8.5 of NUREG-75/087, cited by the State in this contention).

Third, calculations provided by PFS demonstrate that the CTB can be expected to undergo post-construction settlement of about 3 to 4 inches, taking into account both static loads and potential dynamic loading from the design-basis earthquake. This magnitude of settlement is smaller than the maximum tolerable settlement of the CTB foundation, based on the testimony provided by Daniel J. Pomerening below. Thus, settlement of the CTB and its underlying concrete mat does not present a foundation stability concern.

Seismic Design of the Canister Transfer Building.

A31(b). (DJP) The Staff has concluded that the Applicant has demonstrated that the ability of the CTB, with its foundation, to perform its safety function and limit the impact on public health and safety, as required by 10 C.F.R. §72.24(d). Further, the Staff has determined that the design of the CTB proposed by PFS satisfies the requirements of 10 C.F.R. § 72.122(b)(1) and (2), in that the performance is not influenced by the dynamic loading from the design basis earthquake equivalent to the 2,000-year return-period ground motion. The bases for these conclusions are provided in the Staff's Consolidated SER, §§ 2.1.6.4 (Stability of Subsurface Materials"); 4.1.3.2 (Structural); 5.1.3.1 (Description of Reinforced Concrete Structures); 5.1.3.4 (Structural Analysis for Reinforced Concrete Structures); 5.1.4.1 (Description of Other Structures, Systems, and Components Important to Safety); 5.1.4.4 (Structural Analysis of Other Structures, Systems, and Components Important to Safety); and 15.1.2.6 (Earthquake). Details concerning the basis for this conclusion are provided in the discussion below concerning specific portions of this part of Contention Utah L/QQ, Subpart D.

First, as discussed by Dr. Ofoegbu above, the Staff has concluded that PFS has demonstrated the stability of the CTB against overturning, and that adequate safety margins exist against sliding of the CTB foundation under potential dynamic loading from the design-basis earthquake, in accordance with section 3.8.5 of NUREG-0800. Further, the 3 to 4 inches of potential settlement is smaller than the maximum tolerable settlement of the CTB foundation.

Second, the Staff has reviewed Section 4.7 of the PFS SAR, and found that PFS has committed to an appropriate structural analysis process for the Canister Transfer Building to mitigate environmental effects. The Applicant utilizes the ultimate strength method of analysis set forth in ACI 349-90, with the appropriate load factors. The reinforced concrete canister transfer building analyses include consideration of normal, off-normal, and accident loading conditions. The procedures for selection of the reinforcement and checks for axial, shear, moment, and torsional resistance of the elements are in conformance with standard engineering practice, as described in ACI 349-90. These analyses ensure that the capacity of the structural elements exceed the demand, and therefore would be able to perform their intended safety functions under the extreme environmental and natural phenomena in accordance with the requirements of 10 C.F.R. §§ 72.122(b)(1) and (2).

A seismic analysis of the structure was performed to determine the seismic loads for the building design and to generate in-structure response spectra for the design of the overhead and semi-gantry cranes supported by the CTB walls. The seismic analysis was performed following the guidelines of ASCE 4-86. The dynamic analysis is based on a lumped mass model of the Canister Transfer Building capable of modeling response in all linear and rotational degrees of freedom, which is an acceptable modeling process as identified in ASCE 4-86 Impedance functions were developed to represent the subgrade, using the layered dynamic soil properties described in Calculations G(P018)-2 and SC-4. These soil characteristics were subsequently used in the seismic analysis of the Canister Transfer Building, as is appropriate.

Pursuant to 10 C.F.R. § 72.24(b), an ISFSI applicant must provide its design criteria, design bases, applicable codes and standards, methods of analyses, and acceptance criteria for evaluation, but is not required to submit a detailed design of its facility prior to licensing. Here, PFS has updated the seismic load analysis to reflect the physical changes in the building design, as a result of updated design-basis seismic conditions. The Applicant also plans to update the detailed design of the CTB prior to construction. Based upon (1) the Staff's detailed review of the Applicant's design criteria and the process utilized in developing its previous building design, and (2) the Applicant's statement in the SAR (section 4.7.1.5.3) that the changes to the detailed design will follow the same design criteria and the process, the Staff concludes that the design of the Canister Transfer Building for the design-basis earthquake loads is acceptable.

The Applicant has also submitted detailed analyses for the upper and lower roof steel (SC-12), and the rolling doors for the transfer cells (SC-14). The Applicant has indicated that these structural steel elements will be designed in accordance with American National Standards Institute ("ANSI")/American Institute of Steel Construction ("AISC") Standard ANSI/AISC N–690. The appropriate sections were selected to insure that they have sufficient capacity to meet the demands under all loading conditions. As identified in the Staff's Consolidated SER, Section 5.1.4.4 (Structural Analysis of Other [SSCs] Important to Safety" and, specifically, subsections entitled "Canister Transfer Building Structural Steel Roof Beams"), the Staff has concluded that the available design strength for the structural roof beams that were evaluated exceeds the strength required for the factored design loads.

Finally, for the reasons stated above, the Staff has concluded that PFS has demonstrated that the adequacy of its analysis is not affected by "errors" or "unconservative assumptions" in determining the dynamic loadings to the CTB and its mat foundation.

Q32. The State asserts in Subpart D.2.a. of this contention that "the Applicant's calculations incorrectly assume that the mat foundation will behave rigidly during the DBE," and that this assumption of rigidity leads to (i) "significantly underestimation of the dynamic loading to the mat foundation," and (ii) "overestimation of foundation damping." Do you believe that this is a valid concern?

A32. (DJP) No.

Q33. Please provide the bases for this conclusion.

A33. (DJP) First, the State is incorrect in asserting that the Applicant's assumption of rigidity leads to "significant underestimation of the dynamic loading to the mat foundation." In the Applicant's calculations concerning the seismic response of the CTB on the mat foundation due to the 2000-yr seismic event (Stone & Webster Calculation SC-5), the mat foundation is conservatively assumed to be rigid. The Applicant's calculation is used to identify the overall response of the CTB on the mat foundation and to develop elevated response spectra. The assumptions made and the calculation procedure are consistent with ASCE-4-98, Standard for Seismic Analysis of Safety Related Nuclear Structures. The results based on the rigid mat foundation assumption will be conservative, in that no energy will be absorbed in deformation of the mat foundation, resulting in an upper bound estimate of the response of the CTB.

This analysis is to be used to define the loads that the Applicant has committed to perform for the design of the CTB (updated versions of SC-6 and SC-7) to demonstrate compliance with the requirements of ACI 349. A determination of loads based on a rigid mat foundation assumption will result in an upper bound design load estimate.

In contrast, in the procedures used for the Applicant's calculations for the CTB finite element analysis (SC-6), the mat foundation is assumed to be flexible. These results (SC-6) will be used as input into the detailed design of the reinforcement steel for the CTB (SC-7).

Using a procedure similar to that identified in response to Question 13 above, the Staff has estimated the maximum deformation that the CTB mat foundation would experience under seismic loading. The influence on the mat foundation motion can be determined by the relationship between the wave length of the seismic waves and the geometry of the mat foundation (240 ft x 279.5 ft). The response will be defined in terms of the peak vertical displacements. An estimation of the peak displacement response can be defined by the response acceleration at a given frequency divided by the frequency squared. The response acceleration for this is assumed to correspond to the design basis fault normal (FN), fault parallel (FP), and vertical (V) response spectra. The bounding displacements of the mat foundation due to seismic waves is small relative to the overall geometry of the mat foundation. Therefore, the difference between assuming a rigid or flexible mat foundation will be minimal.

For the 240 ft (E-W) direction, the lowest frequency (*i.e.,* the frequency that has the highest displacement) is determined based on the lower bound estimate of the wave velocities. The maximum bending in the mat foundations will occur at 1.21 Hz, based on the shear wave velocity. The corresponding response displacement is: V = 1.9 inches. The maximum rocking of the mat foundations will occur at 0.60 Hz, based on shear wave velocity. The corresponding response displacement is: V = 3.4 inches. The maximum vertical deflection satisfies the requirement of Table 9.5(a) of ACI 349 ($\delta_{max} \leq$ 14.4 inches = 240*12/200). Accordingly, the stability and structural integrity of the CTB will not be affected by non-vertically out-of-phase seismic waves that may occur at the site.

For the 279.5 ft (N-S) direction, the lowest frequency (*i.e.,* the frequency that has the highest displacement) is also determined based on the lower bound estimate of the wave velocities. The maximum bending in the mat foundations will occur at 1.03 Hz, based on the shear wave velocity. The corresponding response displacement is: V = 2.3 inches. The maximum rocking of the mat foundation will occur at 0.52 Hz, based on shear wave velocity. The corresponding response displacement is: V = 3.9 inches. The maximum vertical deflection satisfies the requirement of Table 9.5(a) of ACI 349 ($\delta_{max} \leq$ 16.8 inches = 279.5*12/200). Accordingly, the stability and structural integrity of the CTB will not be affected by non-vertically out-of-phase seismic waves that may occur at the site.

Finally, the State is incorrect in asserting that the Applicant's "assumption of rigidity" for the CTB leads to "overestimation of foundation damping." As stated above, the vertical deformation of the storage pad is small relative to the overall geometry of the pad. Therefore, the soil will effectively "see" the CTB mat foundation as a rigid element, and the foundation damping assumption used by the Applicant is acceptable.

Q34. The State asserts in subpart D.2.b. of this contention that "the Applicant's calculations ignore the presence of a much stiffer, cement-treated soil cap around the CTB," and that "this soil cap impacts (i) soil impedance parameters," and "(ii) kinematic motion of the foundation of the CTB." Do you believe that this is a valid concern?

A34. (DJP) No.

Q35. Please provide the bases for this conclusion.

A35. (DJP) The soil-cement cap would provide restraint against lateral motion due to embedment of the CTB mat within the soil cement cap. This effect was conservatively disregarded in the Applicant's calculation of soil impedance parameters in Calculation SC-4, "Development of Soil Impedance Functions for Canister Transfer Building." The amount of motion of the CTB calculated by the Applicant without inclusion of the soil/cement will be greater than the amount of motion if the additional restraint of the soil/cement was included in the soil impedance function. Thus, the Applicant's omission of this factor in its calculation was conservative. This matter is also discussed in response to Question 17 above.

Q36. The State asserts in Subpart D.2.c. of this contention that "the Applicant's calculations are deficient because they ignore the out-of-phase motion of the CTB and the cement-treated soil cap, which potentially can lead to development of cracking and separation of the cap around the building perimeter." Do you believe that this is a valid concern?

A36. (DJP) No.

Q37. Please provide the bases for this conclusion.

A37. (DJP) Cracking and separation of the soil-cement around the CTB will not adversely affect the ability of the structure to perform its safety function. The influence of potential cracking and separation of the cap around the building is the same as has been previously discussed in response to Question 17, concerning the storage casks. As the mat foundation of the CTB contacts the soil-cement cap during seismic motion, it will tend to locally crush the soil-cement. As indicated in the response to Question 17, this would occur because the soil-cement is softer than the mat foundation. Therefore, it will tend to crush and allow motion of the CTB, and will limit the amount of force imposed on the mat foundation by the soil-cement cap. If separation occurs between the storage pad and the surrounding soil/cement, the resulting impact load that would occur during closure of the gap will not result in significant additional loads on the CTB. The majority of the resistance to lateral motion of the CTB is through the soil under the mat foundation that is captured by the perimeter key. The resulting load on the CTB due to the development of cracking and separation of the soil-cement cap will be minor and will not influence the ability of the structure to perform its safety function.

Q38. The State asserts in Subpart D.2.d. of this contention that "the Applicant's calculations unconservatively assume that only vertically propagating in-phase waves will strike the CTB and its foundations, and fail to account for horizontal variation of ground motion that will cause additional rocking and torsional motion of the CTB and its foundations." Do you believe that this is a valid concern?

A38. (DJP) No.

Q39. Please provide the bases for this conclusion.

A39. (DJP) The bases for this conclusion are the same as those provided in response to Questions 13 and 19 above, concerning the casks and cask storage pads. Specifically, the Applicant recently provided a calculation by Geomatrix concerning the spatial and temporal variations of ground motions for the proposed PFS Facility site. Although these calculations pertain specifically to the storage pads, the approach is also applicable to the Canister Transfer Building. In this calculation, Geomatrix reached the following conclusions:

(1) The angle of incidence of the seismic waves is such that the assumption of vertically propagating waves is reasonable for the site. (It should be noted that the Geomatrix calculation is for the site and is directly applicable to the CTB as well as the storage pads.)

(2) The small time difference for wave arrivals would have a negligible effect on the analysis. The arrival time is dependent on the geometry of the structure in questions. Based on the dimensions of the CTB foundation mat (240 ft x 279.5 ft), the difference in the arrival time is on the order of 0.015 to 0.018 seconds. The time difference is such that it will only affect frequencies above the highest ground motion frequency of interest (*i.e.,* 50 Hz). In sum, based on a review of the Geomatrix calculation with respect to the occurrence of seismic waves in the soil, the Staff concludes that the assumption of vertically propagating in-phase waves is reasonable at the proposed PFS site.

Q40. Does this conclude your testimony?

A40. Yes.

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Education:

B.Sc., Geology, University of Nigeria, Nsukka, 1977
M.A.Sc., Geological Engineering, University of Toronto, Canada, 1981
Ph.D., Geological Engineering, University of Toronto, Canada, 1985

Qualifications:

Dr. Ofoegbu is a geological engineer specializing in the mechanical analyses of geological processes, finite element modeling, and the constitutive modeling of geological materials. He has a background in geoscience, geomechanics and computer software development; and about 20 years of experience in teaching, research, and consulting.

Prior to assuming his current position as Principal Engineer in March 2002, Dr. Ofoegbu served as a senior research engineer at the Southwest Research Institute. In that position, Dr. Ofoegbu led several numerical modeling projects to investigate technical issues related to possible licensing of a geologic repository for high level nuclear waste at Yucca Mountain, such as: Evaluation of a finite element code, ABAQUS, for modeling thermal-mechanical-hydrological coupled processes; and investigations of ground motion patterns resulting from numerically simulated normal fault earthquakes, effects of perched water on thermally driven moisture flow, effects of spatial and time-dependent rock-mass property variations on the stability of underground openings and groundwater flow, and effects of regional crustal density variations on patterns of small-volume basaltic volcanism. Other numerical modeling investigations led by Dr. Ofoegbu include finite element analyses of geologic finite strain for fracture distribution predictions and numerical simulation of a deforming salt body. He has also participated in the development of review procedures for an anticipated license application for the proposed Yucca Mountain repository, technical review of uranium recovery site reclamation plans under the Uranium Mill Tailings Radiation Control Act, and a safety evaluation report for an Independent Spent Fuel Storage Installation.

Dr. Ofoegbu was a research engineer at the University of Toronto for five years, during which time he was the Principal Investigator for an industrial contract on the development and numerical implementation of a constitutive model for geological materials. He developed constitutive models for intact rock, non-lithified soils, and regularly jointed rock mass; implemented the models as user-defined code modules in ABAQUS (a commercially available finite element code); and conducted finite element modeling of the Atomic Energy of Canada Limited's mine-by experiment tunnel.

As an Assistant Professor at the Ahmadu Bello University, Nigeria, in the Department of Civil Engineering, Dr. Ofoegbu taught courses and supervised student research projects in the areas of soil mechanics, earthwork, and foundation engineering, and served as Principal Consultant on industrial site-investigation contracts.

Dr. Ofoegbu has published 25 articles in refereed journals and conference proceedings, as well as several technical reports. He is a member of the International Society for Rock Mechanics and the American Rock Mechanics Association. He is a registered professional engineer in Canada.

Professional Chronology:

Principal Engineer, Southwest Research Institute, March 2002-Present; Senior Research Engineer, Southwest Research Institute, 1993–2002; Consulting Engineer, GI-Johnson Engineering, 1991–93; Research Engineer, University of Toronto, 1987–92; Assistant Professor, Ahmadu-Bello University, 1985–87; Teaching/Research Assistant, University of Toronto, 1980–85; Hydrogeologist, Lower Benue Development Authority, 1978–79; Mathematics/Physics Teacher, Ogun State of Nigeria, 1977–78.

Publications:

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Education:

B.S. in Aerospace Engineering, Georgia Institute of Technology, 1975
M.E. in Civil Engineering, Structural Engineering and Structural Mechanics,
University of California, Berkeley, 1977

Qualifications:

Mr. Pomerening is experienced in design, analysis, and testing of structural systems in the fields of Naval Architecture, Aerospace and Civil Engineering. While working for the Naval Ship Research and Development Center, Mr. Pomerening participated in the design, construction, instrumentation, testing, and data reduction of a variety of models tested in wind tunnels. As a research assistant at the University of California, Berkeley, Mr. Pomerening was involved with the testing of building structures on a large scale seismic simulator. His Master's project produced a feasibility study on a boundary layer wind tunnel to study the dynamic effects of the wind on structures immersed in the atmospheric boundary layer.

Since joining Southwest Research Institute, Mr. Pomerening has been involved in the study of structural response of systems under dynamic loading with specific emphasis on transient and shock loading. Investigations of the structural integrity under seismic motion have included the response of floating platforms, industrial plants, electrical racks, valves and other components. Mr. Pomerening has performed seismic qualification programs for components using both analytical and experimental procedures. Work in this area has also included a Nuclear Regulatory Commission (NRC) program designed to evaluate methodology of equipment seismic qualification for nuclear plants. This has included development of procedures for use of hand held analyzers for determination of the in-situ modes of systems. Mr. Pomerening has supported programs in the Center for Nuclear Waste Regulatory Analysis (CNWRA). These have included reviews of safety analysis reports with specific emphasis on identification of design criteria and assessment of the structural integrity of structures, systems and components to with respect to the NRC Standard Review Plans.

Studies of aerospace structures have included structural models of light aircraft for determination of structural-borne noise, the T-37B aircraft wing to determine local crack growth rates, and the dynamic response of a number of missile systems during transportation and flight. Mr. Pomerening has performed several preliminary hazards analysis of electrical systems and reliability studies of space station mechanical systems. Other activities have included ground vibration and flight flutter testing as part of the T-37B structural life extension program, and slosh and crash testing of light aircraft wings. His work in Naval Architecture has been associated with LNG transport ships, the use of reinforced concrete in the marine environment, the study of ship-based missile systems, blast response of submarines and radomes and dynamic response and fatigue assessments of submersibles.

Under Mr. Pomerening's management, a number of programs have been performed to qualify equipment installed on air, sea, and land-based vehicles. The programs have included test tailoring in accordance with the most recent standards. Mr. Pomerening has also managed a number of programs which tested packaging systems used in the shipment of nuclear materials for compliance with 10 CFR, Part 72 requirements.

Professional Chronology:

Student Engineering Trainee, Naval Ship Research and Development Center, 1970-75; Research Assistant, University of California, Berkeley, 1976-77; Southwest Research Institute, 1977 to Present in the positions of Research Engineer, 1977-83, Senior Research Engineer, Department of Mechanical and Fluids Engineering, 1983-96; and Principal Engineer, 1999 to Present.

Memberships:

American Society of Civil Engineering
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Publications

Kana, D.D., D.J. Pomerening, and J.C. Simonis, "Recent Research on Methodology for Seismic Qualification of Nuclear Plant Equipment," Nuclear Engineering and Design, Vol. 79, pp. 229-241, 1984.

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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-ISFSI
	)	
(Independent Spent Fuel Storage Installation)	))	

NRC STAFF TESTIMONY OF VINCENT K. LUK
AND JACK GUTTMAN CONCERNING UNIFIED
CONTENTION UTAH L/QQ (GEOTECHNICAL ISSUES)

Q1. Please state your names, occupations, and by whom you are employed.

A1(a). My name is Jack Guttmann ("JG"). I am employed as Chief of the Technical Review Section, Spent Fuel Project Office ("SFPO"), Office of Nuclear Material Safety and Safeguards ("NMSS"), U.S. Nuclear Regulatory Commission ("NRC"), in Washington, D.C. A statement of my professional qualifications is attached hereto.

A1(b). My name is Vincent K. Luk ("VKL"). I am employed as a Principal Member of the Technical Staff in the Nuclear Technology Programs Department at Sandia National Laboratories ("SNL"), in Albuquerque, New Mexico. I am providing this testimony under a technical assistance contract between the staff of the U.S. Nuclear Regulatory Commission ("NRC Staff" or "Staff") and SNL. A statement of my professional qualifications is attached hereto.

Q2. Please describe your current responsibilities.

A2(a). (JG) As Chief of the Technical Review Section in the Spent Fuel Project Office, my responsibilities include direction and supervision of various technical reviews related to the licensing and certification of radioactive material transportation and storage packages, under 10 C.F.R. Parts 71 and 72, respectively, including technical reviews related to independent spent fuel storage installations ("ISFSIs"). Among my other responsibilities, I routinely direct and supervise the evaluation and use of computer code modeling and analytical methodologies in assessing the safety and performance of radioactive material transportation and storage packages.

A2(b). (VKL) I currently serve as Leader of the Structural Analysis and Evaluation Team for an NRC Integrated Vulnerability Assessment Project, examining the vulnerability and structural integrity of nuclear power plants subjected to external high-energy impacts. In addition, I serve as the Principal Investigator in an NRC project, establishing criteria and review guidelines in evaluating the seismic behavior of dry cask storage systems; and in examining the dynamic seismic behavior of free-standing dry cask storage systems and soil-structure interaction effects in simulated earthquake events.

Q3. Please explain what your duties have been in connection with the NRC Staff's review of the application filed by Private Fuel Storage, L.L.C. ("PFS" or "Applicant") for a license to construct and operate an Independent Spent Fuel Storage Installation ("ISFSI") on the Reservation of the Skull Valley Band of Goshute Indians, geographically located within Skull Valley, Utah (the "proposed PFS Facility").

A3(a). (JG) As Chief of the Technical Review Section in SFPO, I requested, through the Office of Nuclear Regulatory Research, that a confirmatory analysis be performed by Sandia National Laboratories on behalf of the Staff to evaluate the potential for cask sliding, collision and tipover at the proposed PFS Facility. This analysis was considered to be confirmatory in nature, in that the Staff had previously concluded, on the basis of its review of the PFS application and supporting analyses, that tipover and collision of the casks on the PFS concrete storage pads will not occur under design basis seismic conditions. *See* Consolidated SER, §5.1.4.4, at 5-28 to 5-32; and NRC Staff Testimony of Goodluck I. Ofoegbu and Daniel J. Pomerening on Unified Contention Utah L/QQ, Part D. In addition, I and other members of my staff provided information and expertise, as appropriate, to assist in the review of cask stability under seismic conditions at the proposed PFS Facility.

A3(b). (VKL) As part of my official responsibilities, at the Staff's request I conducted an analysis to evaluate the seismic behavior and stability of the freestanding, cylindrical HI-STORM 100 casks to be installed on concrete pads at the proposed PFS facility, including the potential for cask sliding, collision and tipover. As Principal Investigator in this project, my role was to develop a three-dimensional coupled finite element model of the proposed PFS dry cask storage system to examine the nonlinear and dynamic behavior of the casks, and to simulate the effects of soil-structure interaction, under prescribed seismic conditions. I am the principal author of several documents describing this confirmatory analysis, including (1) "Summary Report on Seismic Analysis of HI-STORM 100 Casks at Private Fuel Storage (PFS) Facility," dated February 22, 2002; (2) "Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage (PFS) Facility," dated March 8, 2002; and (3) "Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage (PFS) Facility," Rev. 1, dated March 31, 2002 (herein cited as "Final Report, Rev. 1").

Q4. What is the purpose of this testimony?

A4. The purpose of this testimony is to provide the results of the NRC Staff's confirmatory analysis of the stability of the freestanding HI-STORM 100 casks at the proposed PFS Facility, with respect to the potential for cask sliding, collision and tipover under seismic conditions, as set forth in Unified Contention Utah L/QQ, Part D.1.i.

Q5. Are you familiar with Unified Contention Utah L/QQ, Part D.1.i. ?

A5. Yes. We understand that Part D.1.i. of this contention states:

> Because of the above errors, omissions and unsupported assumptions [stated in preceding portions of Contention Utah L/QQ, Part D], the Applicant has failed to demonstrate the stability of the free standing casks under design basis ground motions. Thus, the Applicant's analyses do not support the Applicant's conclusions that

excessive sliding and collision will not occur or that the casks will not tip over. 10 CFR § 72.122(b)(2) and NUREG-1536 at 3-6.

Q6. Please describe the Staff's analysis of the stability of the HI-STORM 100 casks and the potential for cask sliding, collision and tipover at the proposed PFS Facility?

A6. An ongoing generic program for developing guidance on seismic hazards analysis was established by NRC's Office of Nuclear Regulatory Research. A research team consisting of analysts and engineers from SNL, ANATECH Corporation, and Earth Mechanics, Inc., was assembled for this purpose, under the leadership of Dr. Vincent Luk, as Principal Investigator. As part of this ongoing effort, the Staff requested technical assistance from the Sandia National Laboratories in conducting an analysis of the behavior of loaded HI-STORM 100 storage casks under seismic conditions at the PFS Facility. The Staff provided basic information to the research team, with respect to cask design, pad dimensions, soil-cement layers under and adjacent to the pad, the site-specific soil profile, and time histories of seismic accelerations.

In conducting this analysis, three-dimensional coupled finite element models were developed, and seismic analyses were performed, to examine the dynamic and nonlinear behavior of the HI-STORM 100 casks to be installed on the concrete storage pads at the proposed PFS Facility, including the soil-structure interaction effects during a seismic event. Three different sets of seismic conditions were modeled: (1) the 2,000-year return period earthquake for the PFS Facility site; (2) the 10,000-year return period earthquake for the PFS Facility site; and (3) a sensitivity study based on the 1971 San Fernando Earthquake (Pacoima Dam record). The analyses thus modeled ground motions for the design basis 2,000-year event; the 1971 San Fernando Earthquake (Pacoima Dam record), for which the ground motions are somewhat similar to the ground motions of the PFS 2,000-year event; and ground motions for the PFS 10,000-year event, which significantly exceed the design basis ground motions for the proposed PFS Facility.

Q7. Please describe the nature of the model that was utilized in the analysis.

A7. (VKL) The ABAQUS/ Explicit code was used to analyze the three-dimensional coupled finite element models, that consist of a single cylindrical HI-STORM 100 cask (with the MPC-68 option), a flexible full-sized concrete pad (30-ft x 67-ft x 3-ft), a shallow surface layer of compact aggregate around the pad (5-ft x 10-ft x 8-in), a soil-cement layer under and adjacent to the pad (approximately 2-ft thick), and an underlying layered soil foundation. The layout of the entire coupled model is shown as Figure 1 of the Final Report, Rev. 1 (page 14). The cask was modeled as an elastic solid component, while the gravel, concrete pad, soil-cement, and soil were modeled as flexible linearly elastic materials. Structural damping ratios, whose values are tabulated in each horizontal layer and for each of the three cases of soil profile data (*see* Final Report, Rev. 1, Tables 2 to 7), were used for the soil and soil cement materials, while a zero damping was used for the concrete pad and the cask.

The shallow surface layer and the concrete pad are placed on a continuous 2-ft soil-cement layer that is on top of the soil foundation. The coupled model has three interfaces, which include the (1) cask/pad, (2) pad/soil-cement layer, and (3) soil-cement layer/soil foundation interfaces. In addition to incorporation of the aforementioned structural elements, development and use of the model also required selection of appropriate cask/pad and soil material properties and application of properly prescribed seismic time history sets to the model. To this end, the NRC staff provided the research team with the basic information on cask design, pad dimensions, soil-cement layers under and adjacent to the pad, the site-specific soil profile, and time histories of seismic accelerations. The analytical results obtained from the model address the dynamic and nonlinear response of the cylindrical cask in terms of its wobbling and sliding by examining closely the nonlinear contact behavior at the three interfaces and accounting for soil-structure interaction effects.

Q8. What assumptions did you make with respect to cask rigidity/elasticity and damping in you model?

A8. (VKL) The cask and pad were modeled as elastic bodies with zero damping.

Q9. Please describe the principal factors you considered in modeling and evaluating the dynamic response of the casks during an earthquake event?

A9. (VKL) This particular modeling effort focused on performing sensitivity studies on the cask response with respect to three key factors: (1) prescribed seismic loading, (2) coefficients of friction at the three interfaces in the coupled model, and (3) soil profile data used for the soil foundation model.

Q10. With respect to the first factor you identified (seismic loading), please describe the seismic loading conditions or events that were used in performing dynamic analyses of the cask.

A10. (VKL) Three sets of seismic time histories were used as input excitations in the coupled model analyses. First, a prescribed artificial time history of seismic accelerations with a duration of 30 seconds, using design basis response spectra for the PFS site for a 2,000-year return period earthquake, was used to generate the response of the cask under design basis conditions. Second, a similar site-specific time history of seismic accelerations for a 10,000-year return period with a duration of 30 seconds was used to provide a limiting or upper-bound case assessment of cask response. Third, a sensitivity study was performed using the 1971 San Fernando Earthquake, Pacoima Dam record.

Each set of seismic time histories has one vertical and two horizontal components of statistically independent seismic accelerations. For the 2,000-year return period earthquake, the peak ground accelerations ("PGAs") that were modeled, based on artificial time histories specific to the PFS site, were 0.728 g (horizontal, east-west), 0.707 g (horizontal, north-south), and 0.721 g (vertical); these PGAs envelop the 2,000-year design basis response spectra of 0.711 g (horizontal) and 0.695 g (vertical), stated in the Consolidated SER for the PFS Facility. For the 10,000-year return period event, the PGAs that were modeled, based on site-specific artificial time histories, were 1.25 g and 1.23 g for the horizontal components, and 1.33 g for the vertical component, which envelop the PFS earthquake hazard spectra. For the 1971 San Fernando Earthquake, Pacoima Dam record, the PGAs that were modeled were 0.641 g for the two horizontal components, and 0.433 g for the vertical component; the duration for this event was 41.8 seconds.

Each of the three seismic acceleration components of a set of time-histories was treated with a deconvolution procedure to produce a modified time history of deconvoluted accelerations with properly adjusted amplitudes and frequencies of the surface-defined accelerations. All three components of deconvoluted accelerations were applied simultaneously at the base of the soil foundation in the coupled model. Deconvolution is a mathematically rigorous solution process that applies the wave propagation equation of the free-field surface along with the boundary conditions, that modifies the input to account for the site-specific soil properties (*i.e.,* linear shear modulus and viscous damping model). This serves to preserve the dynamic characteristics of the original seismic motions and achieve the desired (*i.e.,* appropriate) surface shaking intensity.

Q11. With respect to the second factor you mentioned (coefficients of friction at the three interfaces in the model), please describe how such coefficients were used in the coupled model.

A11. (VKL) Three interfaces were used in the coupled model: cask/pad, pad/soil-cement layer, and soil-cement layer/soil foundation. In order to determine the governing cases for both (a) the maximum horizontal sliding displacement, and (b) the angular rotation of the cask, different combinations with upper and lower bound coefficients of friction were used in the analyses. For the 2,000-year (design basis) event, the best estimate soil profile data (*see* discussion *infra*), a lower bound coefficient of friction of 0.20 (for investigating cask sliding) and an upper bound coefficient of friction of 0.80 (for investigating the potential for cask tipover) were used at the cask/pad interface; also, bounding coefficients of friction of either 1.00 or 0.31 were assumed at the other two interfaces, as shown in Table 8 of the Final Report, Rev. 1 (Best Estimate, Model Type 1) (at page 30).

These sensitivity studies showed that the maximum horizontal displacement (sliding) of the cask was obtained when using a coefficient of friction of 0.20 at the cask/pad interface and 0.31 at the pad/soil-cement layer and soil-cement layer/soil foundation interfaces, as shown in Table 8 of the Final Report, Rev. 1 (Best Estimate, Model Type 1). Consequently, this combination of coefficients of friction was selected as the governing case for other seismic analyses reported in Table 8 of the Final Report, Rev. 1 (page 30), for the 2,000-year event.

Similarly, several studies were conducted for the 1971 San Fernando Earthquake (Pacoima Dam record) and the 10,000-year return period event, using a coefficient of friction of 0.20 at the cask/pad interface, and 0.31 at the other two interfaces, in order to maximize the potential for horizontal displacement (sliding) of the cask. The results of these studies are shown in Tables 9 and 10 of the Final Report, Rev. 1 (pages 31-32). Finally, two additional analyses were conducted for the 1971 San Fernando Earthquake and the 10,000-year return period event, using a coefficient of friction of 0.80 at the cask/pad interface, and 1.00 at the other two interfaces, in order to maximize the potential for cask tipover. These results are also shown in Tables 9 and 10 of the Final Report, Rev. 1.

Q12. With respect to the third factor you identified (soil profile data), please describe the soil profile data used for the soil foundation model.

A12. (VKL) As discussed above, the compact aggregate surface layer and concrete pad are placed on top of a 2-ft thick soil-cement layer that is on top of the soil foundation. The soil foundation submodel utilized in the model was 330-ft in the east-west direction and 757-ft in the north-south direction; these lateral dimensions exceed the recommended minimum as defined in

U.S. Corps. of Engineers soil-structure interaction modeling guidelines. Also, the coupled model partitions the soil into six horizontal layers to a depth of 140 feet, to represent the soil foundation; and the top surface was further divided into layers. The 140-ft depth was selected, in part, to reach a level below which the soil stiffness increases monotonically with depth. Sensitivity studies were performed to demonstrate the adequacy of this discretization scheme (using six layers to a depth of 140 feet) to incorporate the depth variation of soil properties such as shear wave velocity and damping profiles. As shown in Section 3.4.1 and Tables 2-7 of the Final Report, Rev. 1 (pages 9-12), specific soil properties considered include Young's Modulus, Poisson's ratio, density, damping ratio and a mass-related damping factor. This foundation modeling and its rationale are discussed in greater detail in sections 3.2.4 to 3.4.1 of the Final Report, Rev. 1 (pages 7-12).

To provide for broad variation in the soil properties, three sets of soil profile data - the best estimate, the lower bound, and the upper bound - were used separately in the analysis. The same soil profile data (best estimate, the lower bound, and upper bound) were used in performing the cask analyses for the seismic event with a 2,000-year return period and the 1971 San Fernando Earthquake, Pacoima Dam record, as shown in Tables 2 to 4 of the Final Report, Rev. 1 (pages 10-11). Different soil profile data were used for the 10,000-year return period seismic event, in which the shear modulus and damping of each layer of the soil foundation were adjusted for shear strains, as shown in Tables 5 to 7 of the Final Report, Rev. 1 (pages 11-12); in contrast, for seismic events with a 2,000-year return period, the low strain shear modulus and damping were used.

13. What does the coupled model predict as the maximum horizontal cask sliding displacements for each of the three seismic events considered?

A13. (VKL) The results from the seismic analyses indicate that the maximum horizontal cask sliding displacements are 3.98 inches for the 2,000-year return period event, 3.00 inches for the 1971 San Fernando Earthquake, Pacoima Dam record, and 15.94 inches for the 10,000-year return period event.

It should be noted that these results are based the original coupled model ("Model Type 1"). However, two other cases of interest were also examined for the seismic event with a 2,000-year return period, using the best estimate soil profile data. In one case ("Model Type 2"), the ground surface preparation with compacted aggregate and soil-cement layers was removed from the coupled model. In the other case ("Model type 3"), the dead loads of the seven adjacent casks and neighboring pads were included in the coupled model. The maximum horizontal sliding displacements of the cask for both additional cases for the 2,000-year return period event were determined to be less than those obtained using the original coupled model. This is shown in Table 8 of the Final Report, Rev. 1 (page 30).

Q14. Based on the maximum horizontal cask sliding displacements predicted by the model, is the collision of adjacent casks likely to occur?

A14. (VKL) No.

Q15. Please provide the basis for this conclusion.

A15. (VKL) The separation distance between neighboring casks is 47.50 inches. Half of this distance, or 23.75 inches, is regarded as the cask collision criterion. Inasmuch as maximum displacements under the design basis 2,000-year earthquake is 3.98 inches, no cask collisions were found to occur. Further, no collisions were found to occur at the PFS site for the 1971 San Fernando earthquake, Pacoima Dam record, for which the maximum displacement was 3.00 inches. Similarly, under 10,000-year seismic conditions, the maximum displacement was 15.94 inches, which is less than the collision criterion of 23.75 inches. Thus, even under the beyond-design basis 10,000-year event conditions, cask collisions were not found to occur.

Q16. What does the coupled model predict as the maximum cask rotation with respect to the vertical axis of the cask?

A16. (VKL) With respect to the 2,000-year return period seismic event, the analysis results indicate that the maximum cask rotation in either horizontal direction with respect to the vertical axis is equal to or less than 0.03 degrees, using a coefficient of friction of 0.20 for the cask/pad interface. Further, using a coefficient of friction of 0.80, in order to maximize the amount of cask rotation, results in a maximum cask rotation of about 0.22 degrees in the east-west direction and about 0.40 degrees in the north-south direction, with respect to the vertical axis, for the 2,000-year earthquake. In sum, the maximum cask rotation, with respect to the vertical axis, is equal to or less than 0.40 degrees under 2,000-year return period seismic conditions.

With respect to the 1971 San Fernando Earthquake (Pacoima Dam record), the maximum cask rotation in either horizontal direction with respect to the vertical axis, using a coefficient of friction for the cask/pad interface of 0.20, results in a maximum cask rotation with respect to the vertical axis, of 0.02 degrees in the east-west direction and 0.01 degrees in the north-south direction. Further, using a coefficient of friction of 0.80, in order to maximize the amount of cask rotation, results in a maximum cask rotation of 0.06 degrees in the east-west direction and 0.07 degrees in the north-south direction for the 1971 San Fernando Earthquake (Pacoima Dam record). In sum, the maximum cask rotation, with respect to the vertical axis, is equal to or less than 0.07 degrees for the 1971 San Fernando Earthquake (Pacoima Dam record).

With respect to the 10,000-year return period seismic event, the maximum cask rotation in either horizontal direction with respect to the vertical axis, using a coefficient of friction for the cask/pad interface of 0.20, results in a maximum cask rotation with respect to the vertical axis, of 0.10 degrees in the east-west direction and 0.05 degrees in the north-south direction. Further, using a coefficient of friction of 0.80, in order to maximize the amount of cask rotation, results in a maximum cask rotation of 0.65 degrees in the east-west direction and 1.16 degrees in the north-south direction, for the 10,000-year earthquake. In sum, the maximum cask rotation, with respect to the vertical axis, is equal to or less than 1.16 degrees even under 10,000-year return period seismic conditions.

Q17. Based on the maximum cask rotation predicted by the model, is cask tipover likely to occur during either the 2,000-year or 10,000-year return period seismic events?

A17. (VKL) No.

Q18. Please provide the basis for this conclusion.

A18. (VKL) The cask rotation that is associated with tipover is approximately 29 degrees. A rotation of less than 29 degrees would be insufficient to result in tipover of a loaded HI-STORM 100 cask.

Q19. How much movement of the cask in the vertical direction did your analyses predict?

A19. (VKL) A detailed evaluation of cask movement in the vertical direction was conducted. This evaluation indicates that the cask does not experience much displacement in the vertical direction in any of the three seismic events. The cask base is never entirely lifted off the top surface of the pad throughout the seismic event with a 2,000-year return period or the 1971 San Fernando Earthquake (Pacoima Dam record). Further, during either the 2,000-year return period seismic event or the 1971 San Fernando Earthquake (Pacoima Dam record), the maximum vertical displacement at any location of the cask base is much less than 1 inch above the top surface of the pad.

During the seismic event with a 10,000-year return period, the analysis results reveal that the cask base will entirely lift off the top surface of the pad by a maximum 0.26 inches, for a total duration of less than 0.30 seconds. Detailed examinations of the analysis results also indicate that the maximum vertical displacement at any point along the perimeter of the cask base is less than 2.7 inches above the top surface of the pad, for the 10,000-year event.

Q20. In your analysis, did you reach any conclusions as to the importance of the dynamic coupling or soil-structure interaction ("SSI") effect of the cask with the soil foundation?

A20. (VKL) Yes. As discussed in section 4.1 of the Final Report, Rev. 1 (pages 27-29), the dynamic coupling or SSI effect of the cask with the soil foundation was examined in detail, using acceleration results in the east-west direction for the governing case. The model analyses indicate the presence of a significant SSI effect, as shown in Figures 17 through 19 in the Final Report, Rev. 1 (pages 34-35). More specifically, as shown in these Figures, when the acceleration results at four locations on the soil surface are compared to the acceleration results at various depths along the central axis of the pad, noticeable differences in acceleration are observed. The SSI effect is further demonstrated by plotting the corresponding response spectra in Figures 20a through 22b. These differences demonstrate the presence of the SSI effect and justify the development of the coupled finite element model in the Staff's research effort.

Q21. What is your overall conclusion with respect to stability of the freestanding HI-STORM 100 casks at the proposed PFS Facility, and the potential for cask sliding, collision, and tipover?

A21. (VKL, JG) For the reasons discussed above and in the Final Report, Rev. 1, it is our conclusion that excessive cask sliding or cask collisions will not occur. Further, it is our conclusion that cask tipover will not occur during either a 2,000-year return period or 10,000-year return period seismic event at the PFS site. Accordingly, we believe that Part D.1.i. of Unified Contention Utah L/QQ does not present a valid concern.

A22. Does this conclude your testimony?

A22. Yes.

Vincent K. Luk

EDUCATION:

Ph.D., Theoretical and Applied Mechanics, Northwestern University, 1978
M.S., Theoretical and Applied Mechanics, Northwestern University, 1975
B.S., Civil Engineering, University of Mississippi, 1974

WORK EXPERIENCE:

December 1993 to Present — Principal Member of Technical Staff
Nuclear Technology Programs Department, 6420
Sandia National Laboratories / New Mexico

- Team Leader of the Structural Analysis and Evaluation Team for an NRC Integrated Vulnerability Assessment Project. This project examines the vulnerability and structural integrity of nuclear power plants subjected to external high-energy impacts.

- Principal Investigator for the International Nuclear Energy Research Initiative (INERI) Project on "Condition Monitoring through Advanced Sensor and Computational Technology." This project is an international joint project with Korea Atomic Energy Research Institute (KAERI) of South Korea. This project focuses on developing and demonstrating advanced sensor and computational technology for continuous monitoring of the condition of components, structures, and systems in advanced and next generation nuclear power plants.

- Task Leader in the Nuclear Energy Research Initiative (NERI) Project on "Development of Advanced Technologies to Reduce Design, Fabrication and Construction Costs for Future Nuclear Power Plants." This task focuses on investigating the feasibility of developing the design-to-analysis tool to be used to enhance the efficiency of design/analysis cycle.

- Principal Investigator of an NRC project to examine the seismic behavior of freestanding dry cask storage systems subjected to earthquake excitations. In this project, coupled finite element models consisting of casks, concrete pad, and soil foundation were developed to investigate the nonlinear dynamic seismic behavior of cask systems and the soil-structure-interaction effect.

- Lead Engineer for the Steel Containment Vessel Project. This project is a part of the Cooperative Containment Program between Nuclear Power Engineering Corporation (NUPEC) of Japan and US NRC. Responsibilities include overall project management and coordination to conduct an overpressurization test of a scale model of a steel containment vessel and to perform finite element analyses to simulate model responses.

- Analysis Coordinator for NUPEC/NRC Cooperative Containment Program. Responsibilities include defining and monitoring pretest and posttest analysis tasks for simulating structural responses of scale models of steel and prestressed concrete containment vessels under severe pressure loading conditions. Additional assignments are to coordinate the Round Robin analysis activities that involve the participation of various US and international groups to perform independent analyses in pretest predictions and post-test evaluation.

April 1985 to November 1993

<u>Senior Member of Technical Staff</u>
Advanced Munitions Department, 9723
Sandia National Laboratories / New Mexico

- Developed analytical penetration models based on spherical and cylindrical cavity-expansion approximations to predict dynamic loads on projectiles, projectile trajectories, and final penetration depths. Penetration problems included penetration and perforation of aluminum and steel targets, penetration of concrete and soil targets, and perforation of concrete slabs.

- Conducted laboratory-scale ballistic tests and full-scale sled-track tests.

- Team Coordinator for Penetration Technology Team, starting in 1991. Responsibilities included serving as a single point of contact for penetrator technology project activities, to interface with customers, to develop new projects and expand customer base, and to provide team networking of communication and interaction among participants of different disciplines.

- Project Manager for the MOU (Memorandum of Understanding) Tandem-Rod Kinetic Energy Projectile Project. This project involved activities from concept definition, system design and analysis, hardware design and fabrication, to the eventual system demonstration for dual penetrators as an anti-armor system.

- Project Manager for the MOU Penetration Technology Project. Principal project tasks included advancement of penetration technology in the common interests of weapon programs for DOE and DoD laboratories.

- Project Manager for the DOE/DP Penetrator Tech Base Project. Project tasks included providing penetrator technology support to the Defense Program to broaden operational options for the development of future penetrating weapons and developing computational codes as reliable weapon design tools.

January 1981 to March 1985

<u>Senior Staff Engineer</u>
Engineering Mechanics Group
Franklin Research Center, Philadelphia, PA

- Performed structural analysis using finite element techniques on nuclear power plant containment vessel, condenser waterbox flange, valve/actuator assemblies, fan pedestals, and cartridge and barrel assembly of machine guns.
- Performed stress analyses, fatigue evaluation, and heat transfer analyses.
- Section Leader in an NRC project to review the feasibility and adequacy of the kinetic expansion process used to repair damaged tubes and to evaluate the performance of expanded tubes in the Once-Through Steam Generators at Three Miles Island Nuclear Power Station (TMI-1).

July 1978 to December 1980

Stress Analyst
Joseph Oat Corporation
Camden, NJ

- Performed seismic analysis and design of heat exchangers and pressure vessels.
- Performed water-hammer analysis of piping system in heat exchangers for flow-induced vibration during start-up condition.
- Performed response spectra analysis and impact evaluation of new and intermediate fuel storage racks.
- Performed thermal fatigue analysis of tubesheets in regenerative heat exchangers.
- Project Leader in an EPRI project to conduct an experimental study on feedwater heater tube erosion; a laser doppler velocimeter was used to measure 3-dimensional turbulent flow profile inside the inlet header of a plexi-glass model of feedwater heater.

September 1974 to June 1978

Research Assistant
Northwestern University
Evanston, Illinois

- Major fields: elasticity, fracture mechanics and solid contact problems.
- Fracture analysis of spot-welded elastic layers subjected to shear loads.
- Fracture analysis of a cylindrical cavity containing a circumferential edge crack.
- Three-dimensional stress analysis of an elastic half-space containing a partially embedded finite rod.

Awards and Honors:

Sandia National Laboratories

- Award for Excellence in November 1999 for outstanding work in executing the PCCV Round Robin Analysis task.
- 1996 President's Quality Award – Turquoise Award as a member of the NUPEC/NRC Containment Project Team.
- Award for Excellence in June 1992 for outstanding leadership of the Tandem Rod Project that resulted in high praise from the project sponsor.
- Award for Excellence in April 1993 for exceptional leadership of the EPW Tech Base Project.

Northwestern University

- Walter P. Murphy Fellowship in 1974-1975.
- Royal E. Cabell Fellowship in 1977-1978.

University of Mississippi

- Foreign Student Scholarship in 1971-1974.
- Faulkner Concrete Pipe Company Scholarship in 1972-1974.
- Recipient of Taylor Medal in Civil Engineering in 1973.
- Recipient of Taylor Medal Citation in Civil Engineering in 1974.
- President of the Student Chapter of the American Society of Civil Engineers in Senior Year.
- Recipient of the Outstanding Civil Engineering Student Award in 1974.
- Student Marshall for the School of Engineering in the 1974 Commencement.
- Chi Epsilon, Tau Beta Phi and Phi Kappa Phi

Professional Society Affiliations:

Member, American Society of Mechanical Engineers.

Journal Publications:

1. L. M. Keer and V. K. Luk, "Stress Analysis of an Elastic Layer Attached to an Elastic Half Space of the Same Material," International Journal of Engineering Science, Vol. 14, pp. 735-747, 1976.

2. L. M. Keer, V. K. Luk, and J. M. Freedman, "Circumferential Edge Crack in a Cylindrical Cavity," Journal of Applied Mechanics, Vol. 99, No. 2, pp. 250-254, 1977.

3. V. K. Luk and L. M. Keer, "Stress Analysis for an Elastic Half Space Containing an Axially-Loaded, Rigid Cylindrical Rod," International Journal of Solids and Structures, Vol. 15, pp. 805-827, 1979.

4. V. K. Luk and L. M. Keer, "Stress Analysis of a Deep Rigid Axially-Loaded Cylindrical Anchor in an Elastic Medium," International Journal for Numerical and Analytical Methods in Geomechanics, Vol. 4, pp. 215-232, 1980.

5. K. P. Singh and V. K. Lük, "An Approximate Analysis of Foundation Stresses in Horizontal Pressure Vessels," Journal of Engineering for Power, Vol. 102, No. 3, pp. 555-557, 1980.

6. K. P. Singh, M. Holtz, and V. K. Luk, "On Minimization of Rad-Waste Carry-Over in an N-Stage Evaporator," Heat Transfer Engineering, Vol. 5, Nos. 1-2, pp. 68-73, 1984.

7. M. J. Forrestal, Z. Rosenberg, V. K. Luk, and S. J. Bless, "Perforation of Aluminum Plates with Conical-Nosed Rods," Journal of Applied Mechanics, Vol. 54, No. 1, pp. 230-232, 1987.

8. V. K. Luk and M. J. Forrestal, "Penetration into Semi-Infinite Reinforced-Concrete Targets with Spherical and Ogival Nose Projectiles," International Journal of Impact Engineering, Vol. 6, No. 4, pp. 291-301, 1987.

9. M. J. Forrestal, V. K. Luk, and H. A. Watts, "Penetration of Reinforced Concrete with Ogival-Nose Penetrators," International Journal of solids and Structures, vol. 24, No. 1, pp. 77-87, 1988.

10. M. J. Forrestal and V. K. Luk, "Dynamic Spherical Cavity-Expansion in a Compressible Elastic-Plastic Solid," Journal of Applied Mechanics, Vol. 55, No. 2, pp. 275-279, 1988.

11. M. J. Forrestal, K. Okajima, and V. K. Luk, "Penetration of 6061-T6 Aluminum Targets with Spherical, Ogival, and Conical Nose Rods," Journal of Applied Mechanics, Vol. 55, No. 4, pp. 755-760, 1988.

12. V. K. Luk and M. J. Forrestal, "Comments on 'Penetration into Semi-Infinite Reinforced-Concrete Targets with Spherical and Ogival Nose Projectiles'," International Journal of Impact Engineering, Vol. 8, No. 1, pp. 83-84, 1989.

13. M. J. Forrestal, A. J. Piekutowski, and V. K. Luk, "Long-Rod Penetration into Simulated Geological Target at an Impact Velocity of 3.0 km/s," Proceedings of the 11th International Symposium on Ballistics, Brussels, Belgium, May 9-11, 1989.

14. M. J. Forrestal, V. K. Luk, and N. S. Brar, "Perforation of Aluminum Armor Plates with Conical-Nose Projectiles," Mechanics of Materials, Vol. 10, No. 1-2, pp. 97-105, 1990.

15. V. K. Luk, M. J. Forrestal, and D. E. Amos, "Dynamic Spherical Cavity-Expansion of Strain-Hardening Materials," Journal of Applied Mechanics, Vol. 58, No. 1, pp. 1-6, 1991.

16. M. J. Forrestal, N. S. Brar, and V. K. Luk, "Penetration of Strain-Hardening Targets with Rigid Spherical-Nose Rods," Journal of Applied Mechanics, Vol. 58, No. 1, pp. 7-10, 1991.

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18. V. K. Luk and A. J. Piekutowski, "An Analytical Model on Penetration of Eroding Long Rods into Metallic Targets," International Journal of Impact Engineering, Vol. 11, No. 3, pp. 323-340, 1991.

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20. M. J. Forrestal and V. K. Luk, "Penetration into Soil Targets," International Journal of Impact Engineering, Vol. 12, No. 3, pp. 427-444, 1992.

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23. L. M. Keer, Y. Xu, and V. K. Luk, "Analysis of High Speed Axially Symmetric Cutting for Stripping Peripheral Coating," Journal of Manufacturing Science and Engineering, Vol. 120, No. 1, pp. 185-191, 1998.

24. L. M. Keer, Y. Xu, and V. K. Luk, "Boundary Effects in Penetration or Perforation," Journal of Applied Mechanics, Vol. 65, No. 2, pp. 489-496, 1998.

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1. G. K. Haritos, L. M. Keer, and V. K. Luk, "Two and Three Dimensional Stress Analysis of an Elastic Half Space Containing a Partially Embedded Finite Rod," presented at the 15th International Congress of Theoretical and Applied Mechanics, Toronto, Canada, August 18-22, 1980.

2. M. J. Forrestal, M. M. Hightower, V. K. Luk, and B. K. Chritensen, "Penetration and Perforation of Reinforced-Concrete Targets," Proceedings from the Workshop on Weapon Penetration into Hard Targets, Norwegian Defense Research Establishment, May 30-31, 1988.

3. V. K. Luk, J. Hickerson, A. E. Hodapp, and A. D. Foster, "System Development of a 120-mm Tandem-Rod Kinetic Energy Projectile," Proceedings of the Second Ballistics Symposium on Classified Topics, Johns Hopkins University, October 26-29, 1992.

4. J. D. Cargile, M. E. Giltrude, and V. K. Luk, "Perforation of Thin Unreinforced Concrete Slabs," Proceedings of the Sixth International Symposium on Interaction of Nonnuclear Munitions with Structures, Panama City Beach, Florida, May 3-7, 1993.

5. T. Matsumoto, K. Takumi, Y. Kobayashi, M. Fujii, S. Nakajima, J. F. Costello, W. A. von Riesemann, M. B. Parks, M. F. Hessheimer, and V. K. Luk, "Plan on Test to Failure of a Steel, a Prestressed Concrete and a Reinforced Concrete Containment Vessel Model," Proceedings of the 13th International Conference on Structural Mechanics in Reactor Technology, Vol. VI, pp. 89-94, Porto Alegre, Brazil, August 13-18, 1995.

6. V. K. Luk, M. F. Hessheimer, T. Matsumoto, K. Komine, and J. F. Costello, "Testing of a Steel Containment Vessel Model," Proceedings of the 14th International Conference on Structural Mechanics in Reactor Technology, Vol. 5, pp. 73-79, Lyon, France, August 17-22, 1997.

7. T. Matsumoto, K. Komine, S. Arai, V. K. Luk, M. F. Hessheimer, and J. F. Costello, "Preliminary Results of Steel Containment Vessel Model Test," Proceedings of the 14th International Conference on Structural Mechanics in Reactor Technology, Vol. 5, pp. 81-87, Lyon, France, August 17-22, 1997.

8. R. A. Dameron, Y. R. Rashid, V. K. Luk, and M. F. Hessheimer, "Preliminary Analysis of a 1:4 Scale Prestressed Concrete Containment Vessel Model," Proceedings of the 14th International Conference on Structural Mechanics in Reactor Technology, Vol. 5, pp. 89-96, Lyon, France, August 17-22, 1997.

9. V. K. Luk, M. F. Hessheimer, V. L. Porter, T. Matsumoto, and J. F. Costello, "Results of 1:10 Scale Steel Containment Vessel Model Test," SMiRT 14 Post Conference Seminar, Saclay, France, August 25-26, 1997.

10. R. A. Dameron and V. K. Luk, "Preliminary Assessment of Potential Liner Tearing Near the Equipment Hatch of a 1:4 Scale PCCV," SMiRT 14 Post Conference Seminar, Saclay, France, August 25-26, 1997.

11. T. Matsumoto, K. Komine, J. F. Costello, V. K. Luk, and M. F. Hessheimer, "Pressurization Test of a 1/10 Steel Containment Vessel Model," Proceedings of the Workshop on Severe Accident Research in Japan (SARJ-97), pp. 210-218, Yokohama, Japan, October 6-8, 1997.

12. D. W. Pace, M. F. Hessheimer, V. K. Luk, R. A. Dameron, M. Iriyama, and J. F. Costello, "Preliminary Analysis and Instrumentation of a Prestressed Containment Vessel Model," Proceedings of the Workshop on Severe Accident Research in Japan (SARJ-97), pp. 219-224, Yokohama, Japan, October 6-8, 1997.

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16. V. K. Luk, M. F. Hessheimer, K. Komine, M. Iriyama, T. Matsumoto, and J. F. Costello, "Steel Containment Vessel Model Test: Results and Evaluation," Proceedings of the 15th SMiRT Conference, Vol. VI, pp. 267- 274, Seoul, Korea, August 15-20, 1999.

17. V. K. Luk, E. W. Klamerus, M. F. Hessheimer, K. Komine, M. Iriyama, T. Matsumoto, and J. F. Costello, "Round Robin Analyses of the Steel Containment Vessel Model," Proceedings of the 15th SMiRT Conference, Vol. VI, pp. 203-210, Seoul, Korea, August 15-20, 1999.

18. J. S. Ludwigsen, V. K. Luk, M. F. Hessheimer, T. Matsumoto, K. Komine, and J. F. Costello, "Posttest Analyses of the Steel Containment Vessel Model," Proceedings of the 15th SMiRT Conference, Vol. VI, pp. 219-226, Seoul, Korea, August 15-20, 1999.

19. V. K. Luk, J. A. Smith, S. K. Shaukat, R. M. Kenneally, R. A. Dameron, Y. R. Rashid, and V. P. Sobash, "Seismic Analysis of Evaluation of Spent Fuel Dry cask Storage Systems," Transactions, SMiRT 16, Paper # 1369, Washington DC, USA, August 12-17, 2001.

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21. M. F. Hessheimer, V. K. Luk, E. W. Klamerus, S. Shibata, S. Mitsugi, and J. F. Costello, "Pretest Round Robin Analysis of 1:4-Scale Prestressed Concrete Containment Vessel Model," Transactions, SMiRT 16, Paper # 1305, Washington DC, USA, August 12-17, 2001.

22. R. A. Dameron, Y. R. Rashid, V. K. Luk, and M. F. Hessheimer, "Pretest Analysis of a 1:4-Scale Prestressed Concrete Containment Vessel Model," Transactions, SMiRT 16, Paper # 1271, Washington DC, USA, August 12-17, 2001.

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Thesis and Dissertation:

1. "Fracture Analysis of a Spot Welded Elastic Layer in Shear," M.S. Thesis, Northwestern University, August 1975.

2. "Elastostatic Load-Diffusion Characteristics of Embedded Axially-Loaded Cylindrical Structures," Ph.D. Dissertation, Northwestern University, June 1978.

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Education:

B.S. in Mechanical Engineering, Michigan Technological University, 1973
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Experience:

Mr. Guttmann has experience in nuclear engineering related to thermal-hydraulic and mechanical engineering analysis. Mr. Guttmann worked at the Idaho National Engineering Laboratory as a contractor to the NRC in the area of thermal-hydraulic computer code validation and analysis. He performed analyses that quantified the conservatism between the accident analysis requirements for licensing nuclear power plants (10 C.F.R. Part 50, Appendix K), validated the computer code RELAP for regulatory application by the NRC, and performed independent confirmatory transient and accident analyses of operating reactor events and safety issues defined by the NRC.

While working at the NRC, Mr. Guttmann was responsible for reviewing and approving the computer codes used by the nuclear industry for transient and accident analysis. He represented the Office of Nuclear Reactor Regulation (NRR) on the Advanced Code Review Committee, the Loss of Fluid Test Facility, and the Semiscale Test Facility. Mr. Guttmann performed independent analyses of plant operating events, including regulatory responses to the TMI-2 accident. He was a member of the BWR Bulletins and Orders Task Force that reviewed the ramifications of the TMI-2 events for boiling water reactors. He reviewed and approved emergency operator procedures for PWR designs and performed quality assurance inspections. Mr. Guttmann developed standard review plans for analyzing reactor transient and accident events, developed regulatory guidance and NUREG documents for implementing Risk-Informed In-Service Testing of Piping, and was on the task force and project manager for developing Risk-Informed regulatory guidance documents (i.e., RG-1.174, -1.175, - 1.176, -1.177, and 1.178).

With respect to policy development, Mr. Guttmann served as a technical assistant to Commissioner Forrest J. Remick. He advised Commissioner Remick on policy development of advanced nuclear power plants, operating reactor issues, research needs, international activities, and represented the Commission as an observer on INPO inspections.

Mr. Guttmann is currently Chief of the Technical Review Section at the Spent Fuel Project Office. His responsibilities include licensing and certification of storage (10 CFR Part 72) and transportation (10 CFR Part 71) packages of radioactive materials, including independent spent fuel storage installations. Mr. Guttmann is also responsible for assessing vulnerabilities of storage and transportation packages to terrorist events.

Professional Chronology:

Jr. Engineer, Detroit Edison Co., Enrico Fermi Atomic Power Plant-I, 1972-73; Research Engineer, Idaho National Engineering Laboratory, 1975-1976; Nuclear Engineer, Office of Nuclear Reactor Regulation, NRC, 1976-1985; Technical Coordinator, Office of the Secretary, NRC, 1985-1990; Technical Assistant, Office of the Commission, NRC, 1990-1994; Sr. Reliability and Risk Assessment Engineer, Office of Nuclear Regulatory Research, NRC, 1994-1999; Sr. Nuclear Engineer, Office of Nuclear Material Safety and Safeguards, NRC, 1999-2000; Chief, Technical Review Section, Spent Fuel Project Office, Nuclear Material Safety and Safeguards, 2000-2002.

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-ISFSI
	)	
(Independent Spent Fuel Storage Installation)	)	

NRC STAFF TESTIMONY OF
JOHN A. STAMATAKOS, RUI CHEN AND
MARTIN W. McCANN, JR., CONCERNING UNIFIED
CONTENTION UTAH L/QQ, PART E (SEISMIC EXEMPTION)

Q1. Please state your names, occupations, and by whom you are employed.

A1(a). My name is John A. Stamatakos ("JAS"). I am employed as a Principal Scientist at the Center for Nuclear Waste Regulatory Analyses ("CNWRA"), which is a division of the Southwest Research Institute ("SwRI"), in San Antonio, Texas. I am providing this testimony under a technical assistance contract between the staff of the Nuclear Regulatory Commission ("NRC Staff" or "Staff") and the CNWRA at the SwRI. A statement of my professional qualifications is attached hereto.

A1(b). My name is Rui Chen ("RC"). I am employed as an independent consultant in geological engineering and geosciences. From April 1995 to August 2000, I was employed as a Research Engineer and Senior Research Engineer at the Center for Nuclear Waste Regulatory Analyses ("CNWRA"), in San Antonio, Texas, where I was involved in various matters including the technical analysis of mechanical, thermal, and hydrological processes in complex geomechanical and geotechnical engineering systems related to the proposed Yucca Mountain repository; and the evaluation of seismic hazard analyses and seismic design related to proposed spent fuel storage facilities, including the proposed PFS Facility. I am providing this testimony under a technical assistance contract between the NRC Staff and the CNWRA of the SwRI. A statement of my professional qualifications is attached hereto.

A1(c). My name is Martin W. McCann, Jr. ("MWM"). I am employed as President of Jack R. Benjamin & Associates, Inc., in Menlo Park, California. I am also a Consulting Professor of Civil and Environmental Engineering at Stanford University. Among my duties at Jack R. Benjamin & Associates, Inc., I serve as a consultant to the Center for Nuclear Waste Regulatory Analyses ("CNWRA"), which is a division of the Southwest Research Institute ("SwRI"), in San Antonio, Texas. I am providing this testimony under a technical assistance contract between the NRC Staff and the CNWRA of the SwRI. A statement of my professional qualifications is attached hereto.

Q2. Please describe your current responsibilities.

A2(a). (JAS) In my position as Principal Scientist at the CNWRA, I currently serve as the Principal Investigator for several projects involving technical evaluation of structural deformation and seismicity, including tectonics and neotectonics research. My work includes field analyses of the structural and tectonic elements of the Basin and Range province in the southwestern United States, and the evaluation of seismic and faulting hazards at various nuclear facilities.

A2(b). (RC) In my position as an independent consultant, I have provided technical assistance and consulting services to the CNWRA at SwRI involving a broad range of problems in underground rock engineering, seismic hazard assessment, and earthquake engineering; including the evaluation of seismic and geotechnical hazards at various nuclear facilities. I also teach graduate and undergraduate courses in the fields of geotechnical engineering and geosciences in the Department of Civil Engineering and College of Natural Sciences at the California State University at Chico, California.

A2(c). (MWM) In my position as President of Jack R. Benjamin & Associates, Inc., I provide consulting services to private industry and government entities, both in the United States and abroad, in the area of risk analysis for critical facilities, development of generic standards and guidelines for use in assessing seismic hazards and in the assessment of seismic hazards at specific sites, with emphasis in the area of probabilistic seismic hazard analysis ("PSHA"). As part of my responsibilities, I have provided technical assistance and consulting services to the CNWRA at SwRI in its review of various PSHAs, including the PSHA for the U.S. Department of Energy's ("DOE") proposed Yucca Mountain repository and other DOE nuclear facilities. In addition, in my position as a Consulting Professor of Civil and Environmental Engineering at Stanford University, I am involved in activities related to the collection and evaluation of data on dams and dam incidents.

Q3. Please explain what your duties have been in connection with the NRC Staff's review of the application filed by Private Fuel Storage, L.L.C. ("PFS" or "Applicant") for a license to construct and operate an Independent Spent Fuel Storage Installation ("ISFSI") on the Reservation of the Skull Valley Band of Goshute Indians, geographically located within Skull Valley, Utah (the "proposed PFS Facility").

A3(a). (JAS) As part of my official responsibilities, I assisted the NRC Staff in its evaluation of the Applicant's request for an exemption from certain regulations pertaining to seismic analyses and requirements related to the Applicant's construction and operation of the proposed PFS Facility. My specific role was to conduct the Staff's evaluation of the Applicant's probabilistic seismic hazard analysis ("PSHA"), including seismic ground motions and faulting hazards. In this regard, I co-authored, with Drs. McCann and Chen, a document entitled "Seismic Ground Motion and Faulting Hazard at Private Fuel Storage Facility in the Skull Valley Indian Reservation, Tooele County - Final Report," issued by the CNWRA in September 1999. Further, I assisted in preparation of the Staff's "Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued on September 29, 2000 ("SER"); and I assisted in preparation of Supplement No. 2 to the SER, dated December 21, 2001 ("SSER Supplement No. 2"). Those two documents have now been incorporated into the NRC Staff's "Consolidated Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued in March 2002 ("Consolidated SER").

In addition, among my other duties, I assisted the NRC Staff in preparing the "NRC Staff's Response to Applicant's Motion for Summary Disposition of Utah Contention L, Part B," dated December 7, 2001; and I assisted the NRC Staff in preparing its responses to various discovery requests which were filed by the State of Utah ("State"), including the Staff's responses of February 14 and July 12, 2000 (Response and Supplemental Response to the State's Sixth Request); December 7, 2000 (Response to the State's Tenth Request); November 16, 2001 (Response to the State's Fourteenth Request); and December 4, 2001 (Supplemental Response to the State's Twelfth Request).

A3(b). (RC) As part of my official responsibilities, I assisted the NRC Staff in its evaluation of the Applicant's request for an exemption from certain regulations pertaining to seismic analyses and requirements, related to the Applicant's construction and operation of the proposed PFS Facility. My specific role was to participate in the Staff's evaluation of seismic hazard analyses and seismic design for the proposed PFS Facility. In this regard, I co-authored a document entitled "Seismic Ground Motion and Faulting Hazard at Private Fuel Storage Facility in the Skull Valley Indian Reservation, Tooele County - Final Report," issued by the CNWRA in September 1999. Further, I assisted in preparation of the Staff's "Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued on September 29, 2000 ("SER"), and I assisted in preparation of Supplement No. 2 to the SER, dated December 21, 2001 ("SSER Supplement No. 2"). Those two documents have now been incorporated into the NRC Staff's "Consolidated Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued in March 2002 ("Consolidated SER").

In addition, among my other duties, I was involved in the NRC Staff's review of the seismic exemption request for the Three Mile Island Unit 2 ("TMI-2") ISFSI at the Idaho National Engineering and Environmental Laboratory ("INEEL") located in southeastern Idaho, and I co-authored a related report entitled, "Seismic Ground Motion at Three Mile Island Unit 2 Independent Spent Fuel Storage Installation Site in Idaho National Engineering and Environmental Laboratory - Final Report," CNWRA-98-007, issued in June 1998 (Chen and Chowdhury, 1998).

A3(c). (MWM) As part of my official responsibilities, I assisted the NRC Staff in its evaluation of the Applicant's request for an exemption from certain regulations pertaining to seismic analyses and requirements, related to the Applicant's construction and operation of the proposed PFS Facility. My specific role was to conduct the Staff's evaluation of the probabilistic seismic hazard analysis for the proposed PFS Facility. In this regard, I co-authored a document entitled "Seismic Ground Motion and Faulting Hazard at Private Fuel Storage Facility in the Skull Valley Indian Reservation, Tooele County - Final Report," issued by the CNWRA in September 1999. Further, I assisted in preparation of the Staff's "Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued on September 29, 2000 ("SER"), and I assisted in preparation of Supplement No. 2 to the SER, dated December 21, 2001 ("SSER Supplement No. 2"). Those two documents have now been incorporated into the NRC Staff's "Consolidated Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued in March 2002 ("Consolidated SER").

Q4. What is the purpose of this testimony?

A4. The purpose of this testimony is to provide the NRC Staff's views concerning the acceptability of the Applicant's seismic exemption request, which is the subject of Unified Contention Utah L/QQ, Part E.

Q5. Please describe the Commission's regulatory requirements related to the seismic analysis and design that you considered in your evaluation of the PFS application for an independent spent fuel storage installation.

A5. (JAS) The Commission's requirements governing the seismic analysis and design for an ISFSI are set forth in 10 C.F.R. Part 72. In general, 10 C.F.R. § 72.90 requires an evaluation of site characteristics that may directly affect the safety or environmental impact of the proposed facility, including an evaluation of the frequency and severity of external natural events that could affect the safe operation of the ISFSI. Pursuant to 10 C.F.R. § 72.92, an applicant must identify and assess the natural phenomena that may exist or can occur in the region of the proposed facility, with respect to their potential effects on safe operation, including consideration of the occurrence and severity of important natural phenomena; and 10 C.F.R. § 72.98(a) requires identification of the regional extent of external phenomena that are used as a basis for the design of the facility.

Pursuant to 10 C.F.R. §72.122(b)(1), structures, systems, and components important to safety ("SSCs") must be designed to accommodate the effects of, and be compatible with, site characteristics and environmental conditions and to withstand postulated accidents. Further, § 72.122(b)(2) requires that SSCs be designed to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform safety functions, and that the design bases for the SSCs must reflect (i) appropriate consideration of the most severe of the natural phenomena reported for the site and surrounding area, with appropriate margins to take into account the limitations of the data and the period of time in which the data have accumulated, and (ii) appropriate combinations of the effects of normal and accident conditions and the effects of natural phenomena.

In addition, pursuant to 10 C.F.R. § 72.102, an ISFSI applicant is required to address the geological and seismological characteristics of its proposed site. For sites located west of the Rocky Mountain Front (west of approximately 104° west longitude) and in other areas of known potential seismic activity, 10 C.F.R. § 72.102(b) requires that "seismicity will be evaluated by the techniques of appendix A of [10 C.F.R. Part 100]." Further, 10 C.F.R. § 72.102(f) requires that for sites which have been evaluated under the criteria in 10 C.F.R. Part 100, Appendix A, the "design earthquake (DE) for use in the design of structures . . . . must be equivalent to the safe shutdown earthquake (SSE) for a nuclear power plant."

Appendix A to 10 C.F.R. Part 100 (which is cited in 10 C.F.R. § 72.102(b) and (f)), establishes seismic and geologic siting criteria for nuclear power plants ("NPPs"). Appendix A sets forth the criteria to be used by NPP license applicants in conducting the geologic and seismic investigations necessary to determine site suitability; it describes "procedures for determining the quantitative vibratory ground motion design basis at a site due to earthquakes" and "information needed to determine whether and to what extent a [NPP] need be designed to withstand the effects of surface faulting"; and it identifies "other geologic and seismic factors required to be taken into account in the siting and design of [NPPs]" (*Id.*, Part II). Part IV of Appendix A describes the geologic, seismic and engineering investigations that are required; Part V describes the process to be followed in determining the seismic and geologic design bases for the facility; and Part VI describes the application of these matters to the facility's engineering design.

In particular, Part V(a) of 10 C.F.R. Part 100, Appendix A, discusses the process to be followed in determining the design basis for vibratory ground motion, including identification of the safe shutdown earthquake for a nuclear power plant. Appendix A, Part III, defines the safe shutdown earthquake as that earthquake, "based upon an evaluation of the maximum earthquake potential" shown in site and regional investigations, which produces "the maximum vibratory ground motion" at the site for which certain SSCs are designed to remain functional; the SSE is commonly referred to as the NPP's "design basis earthquake." The approach specified in Appendix A implies the use of a "deterministic seismic hazard analysis" ("DSHA") to calculate the SSE, because it considers only the largest possible earthquake that could occur at a location closest to the site, and does not consider how frequently the seismic events occur that control the deterministic ground motion. Thus, analyses using the Part 100, Appendix A methodology determine the SSE for a NPP, without regard to the uncertainties associated with the evaluation of earthquakes (*e.g.*, size, location, magnitude) and the assessment of ground motions, and do not consider the probability of occurrence of the SSE.

Q6. Has PFS sought an exemption from any of the regulatory requirements referred to above, with respect to its ISFSI application?

A6. Yes. On April 2, 1999, the Applicant submitted a request for an exemption from the seismic design requirements in 10 C.F.R. § 72.102(f)(1), to allow its use of a PSHA and considerations of risk to establish the design earthquake ground motion levels at the proposed PFS Facility. The Applicant's exemption request also proposed to design the Facility based on ground motions that have a mean annual probability of exceedance of 1×10^{-3} (or the reciprocal 1,000-year return period).

Q7. Did the Staff conduct an evaluation of the Applicant's seismic exemption request?

A7. Yes.

Q8. Please summarize the nature of the Staff's evaluation of the exemption request.

A8. Inherent in the PFS exemption request are two related questions: (1) Should the Applicant be permitted to substitute a PSHA for the DSHA required by the regulations in 10 C.F.R. Part72; and (2) if the PSHA is used, what is the appropriate mean annual probability of exceedance (*i.e.*, return period) for the seismic design ground motions. At the Staff's request, we conducted an evaluation of the Applicant's seismic exemption request, which is described in Section 2.1.6.2 of the Consolidated SER. As part of this evaluation, we provided a detailed analysis of the Applicant's request, in a report entitled "Seismic Ground Motion at the Private Fuel Storage Facility Site in the Skull Valley Indian Reservation," issued in September 1999 (Stamatakos, et al., 1999). This evaluation included an independent technical review of the seismic hazard investigations at the proposed PFS site, as described in the Consolidated SER (Section 2.1.6) and Stamatakos, et al. (1999).

As set forth in the Consolidated SER, in April 1999, after extensive site characterization studies were performed by Geomatrix Consultants, Inc., the Applicant submitted a request for an exemption from the deterministic seismic design requirements of 10 C.F.R. § 72.102(f)(1). The exemption request proposed to instead use a probabilistic seismic hazard analysis ("PSHA") along with considerations of risk to establish the design earthquake ground motions at the proposed PFS site. The original exemption request also proposed to design the PFS Facility to ground motions with a mean annual probability of exceedance of 1×10^{-3} (1,000-year return period).

To support its evaluation of the PFS exemption request, the Staff asked the CNWRA to conduct a technical review of the seismic and faulting hazard investigations at the proposed PFS Facility site. The objectives of the CNWRA seismic and faulting hazard investigations were (1) to conduct an independent review of seismic and faulting hazard studies at Skull Valley and, in particular, to identify seismic and faulting issues important to siting the proposed PFS Facility; (2) to evaluate the adequacy and acceptability of the PFS seismic and faulting design approach; and (3) to make recommendations regarding the PFS proposed seismic design approach and design basis ground motions. These objectives were accomplished through a survey of state-of-the-art literature (including documents submitted by PFS), analyses of relevant NRC regulations, and CNWRA independent analyses of geophysical data, sensitivity studies of model alternatives, and consideration of uncertainties. Seismic issues important to siting the proposed PFS Facility included (a) characterization of potential seismic sources, (b) estimation of ground motion attenuation, (c) assessment of probabilistic and deterministic ground motion hazards, (d) assessment of probabilistic surface faulting hazards, and (e) development of design basis ground motions in compliance with applicable regulations and regulatory guidance.

Based on the review of the PSHA conducted by Geomatrix Consultants, Inc. (1999), the Staff concluded that the PFS seismic and surface faulting hazard results provide an adequate basis for development of the design seismic ground motions for the proposed PFS Facility. In fact, the Staff's analyses concluded that the results of the PSHA are conservative, mainly because of conservative assumptions in the seismic source characterization.

Following issuance of the CNWRA report (Stamatakos, et al., 1999), the Staff continued to evaluate the exemption request in light of the additional site characterization information that was provided by the Applicant. This new information included the Applicant's updates to the PSHA in 2000 and 2001, some of which led the Applicant to increase its estimated seismic hazard at the site. These revisions included modifications to the site velocity model, the ground motion attenuation relationships adopted from the Yucca Mountain study, and the approach used in the site response analysis. In the aggregate, these revisions resulted in an increase in the ground motion hazards estimated at the PFS site. For example, based on the new information, the Applicant increased its estimate of the peak horizontal acceleration (5×10^{-4} mean annual probability of exceedance) from 0.53g (as reported in 1999) to 0.711g (as reported in 2001). The Applicant's PSHA revisions have not affected our conclusions regarding the acceptability of the PFS exemption request. Details concerning the Staff's evaluation and conclusions with respect to the adequacy and results of the Applicant's PSHA are documented in the Consolidated SER (Sections 2.1.6.1 and 2.1.6.2) and in Stamatakos, et al. (1999).

Q9. Please provide a summary of the factors considered by the Staff in determining whether a PSHA may be utilized in lieu of the DSHA required in 10 C.F.R. Part 72.

A9. Although 10 C.F.R. Part 72 requires a deterministic approach for the seismic hazard assessment of an ISFSI site located west of the Rocky Mountain Front, the Commission and Staff have taken certain actions which indicate general approval of the use of PSHA methodology

First, the Commission has indicated that the uncertainty associated with evaluating seismic design ground motions for NPPs must be addressed. In this regard, the Commission has issued regulations and regulatory guidance that approve this approach in determining the SSE for a nuclear power plant, as set forth in 10 C.F.R. § 100.23 and Regulatory Guide 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion" (1997). In addition, the Commission has initiated a rulemaking effort to amend 10 C.F.R. Part 72, to permit the use of a PSHA to establish the design basis ground motions for SSCs important to safety at an ISFSI. *See* SECY-98-126, as modified in SECY-01-0178. Second, as set forth in SECY-98-071, the Commission has previously reviewed and approved a request for an exemption from the deterministic seismic requirements in 10 C.F.R. § 72.102(f)(1), to allow the use of a PSHA to establish the design ground motions at the Three Mile Island Unit 2 ("TMI-2") spent fuel debris ISFSI, located at the Idaho National Engineering and Environmental Laboratory.

The Commission's actions in considering an alternative to the deterministic approach specified in 10 C.F.R. Part 100, Appendix A, appear to reflect the recognition that the PSHA methodology has certain advantages as compared to a DSHA. For example, a DSHA considers only the most significant earthquake sources and events with a fixed site-to-source distance. A PSHA, on the other hand, incorporates the contribution of all potential seismic sources and considers the range of source-to-site distances, earthquake magnitudes, and the randomness of earthquake ground motions. Most importantly, the PSHA methodology evaluates uncertainty in the assessment of seismic hazards. In doing so, it provides a more complete estimate of the earthquake hazards at a proposed site, for use in establishing the design basis ground motions.

Q10. Please identify the mean annual probability of exceedance (*i.e.*, the return period) that the Applicant proposed for use in conjunction with its PSHA.

A10. The Applicant's exemption request, as originally submitted, proposed design ground motions that have a mean annual probability of exceedance of 1×10^{-3} (1,000-year return period). In reviewing the Applicant's exemption request, the Staff found that the Applicant's proposed use of a 1×10^{-3} mean annual probability of exceedance (1,000-year return period) lacked sufficient regulatory and technical bases. Thereafter, in August 1999, the Applicant revised its exemption request to use design ground motions that have a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period).

Q11. Has the Staff reached a conclusion as to whether the Applicant's seismic exemption request is acceptable?

A11. Yes. As set forth in Section 2.1.6.2 of the Staff's Consolidated SER (pages 2-50 to 2-51), the Staff has concluded that the use of the PSHA methodology and a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period) are acceptable bases to determine the seismic design ground motions of the proposed PFS Facility. Accordingly, the Staff has concluded that the Applicant's exemption request should be granted.

Q12. Please describe the bases for this conclusion, insofar as it is based upon the Applicant's PSHA.

A12. The Staff considered a number of technical and regulatory factors in its evaluation of the Applicant's seismic exemption request. These included (1) the Applicant's exemption request and the PSHA submitted in support thereof; (2) our evaluation of the Applicant's PSHA; (3) the Commission's acceptance, in various regulatory documents, of a PSHA approach in determining the seismic design basis for NRC-licensed facilities (as reflected in amendments to 10 C.F.R. Parts 50 and 100, issuance of Reg. Guide 1.165, and approval of the Rulemaking Plan in SECY-98-126); and (4) the Commission's 1998 approval of the exemption request for the TMI-2 ISFSI at INEEL.

With respect to the technical analysis supporting the Applicant's seismic exemption request, we found the Applicant's PSHA results to be conservative. As stated in the Consolidated SER (page 2-48), this determination was based upon our review of the geological and seismotectonic setting, historical seismicity, potential seismic sources and their characteristics, ground motion attenuation modeling, probabilistic and deterministic estimates of ground motion hazards, development of design basis ground motions, and independent Staff analyses.

One aspect of the Staff's review included the interpretations of fault geometries for the newly discovered East and West faults in Skull Valley, based on reflection seismic data and forward modeling of gravity data by Geomatrix Consultants, Inc., developed in 1999. Staff review of the Applicant's fault models (models defining the size, location, and activity of seismogenic faults in the region) shows that the assessment by Geomatrix Consultants, Inc. may have led to an overly conservative hazard result (perhaps by as much 50% or more, based on a comparison to Salt Lake City PSHA results, as discussed below). For example, independent analysis of proprietary industry gravity data (reported in Stamatakos et al., 1999) does not support the interpretation that the West fault (one of the faults very near the site) is an independent seismic source. Rather, the Staff concluded that the West fault is a splay of the larger East fault, incapable of independently generating large magnitude earthquakes. By contrast, in the Geomatrix fault model, the West fault is considered capable of producing large-magnitude earthquakes.

Another aspect of the Applicant's seismic source characterization that appears to be conservative is the site-to-source distance models used in the ground motion attenuation relationships and the development of distributions of maximum earthquake magnitude based on the dimensions of fault rupture. This conclusion of additional conservatism is derived from a slip tendency analysis (Morris et al., 1996) of the Skull Valley fault systems that was performed by the Staff. The Staff's slip tendency analysis shows that segments of the East fault and the East Cedar Mountain fault nearest the PFS site have relatively low slip tendency values compared to segments farther north in Skull Valley. As discussed in the Consolidated SER (pages 2-38 to 2-40), these relatively low slip tendency results indicate that the seismic source characterization of the PSHA study conducted by Geomatrix Consultants, Inc. (Geomatrix Consultants, Inc., 1999a and 2001a) is conservative.

In slip tendency analysis, the underlying assumption is that the regional stress state controls slip tendency and that there are no significant deviations due to local perturbations of the stress conditions. This assumption is supported by a similar slip tendency analysis of the Wasatch fault, which shows the highest slip tendency values for the segments of the fault considered to be most active (Machette et al., 1991).

The Staff's slip tendency analysis was completed using an interactive stress analysis program (3DStress™) that assesses potential fault activity relative to crustal stress. For Skull Valley, the stress tensor is defined with a vertical maximum principal stress (σ_1), a horizontal intermediate principal stress (σ_2) with azimuth of 355°, and a horizontal minimum principal stress (σ_3) with an azimuth of 085°. The stress magnitude ratios are $\sigma_1/\sigma_3 = 3.50$ and $\sigma_1/\sigma_2 = 1.56$. This orientation for the principal stresses was based on recent global positioning satellite information (Martinez, et al., 1998a). The Staff's slip tendency analysis assumed a normal-faulting regime, with rock density equal to 2.7 g/cc, fault dip equal to 60°, water table at a depth of 40 m, and a hydrostatic fluid pressure gradient.

The results of the Staff's slip tendency analysis indicate that fault segments with approximately North-South strikes (azimuth = 175°) are optimally oriented for future fault slip. Faults with north northeast-south southwest strikes have high slip tendency values. In contrast, fault segments with northwest-southeast strikes, such as the East fault near the proposed PFS Facility site and the southern segments of the East Cedar Mountain fault also near the proposed PFS Facility site, have relatively low slip tendency values. Therefore, these fault segments are less likely to slip in the future than fault segments further from the site. In this regard, it should be noted that fault rupture close to the site greatly influences the seismic hazard. The closer the earthquake is to the site, the larger the resulting ground motions will be as compared to an earthquake with an equal magnitude on a fault segment farther away from the site.

In the Applicant's site-to-source distributions used in the ground motion attenuation equations, Geomatrix Consultants, Inc. (1999a) assumed uniform distributions of earthquake ruptures along active fault segments, without regard to the orientation and slip tendency of the fault segment. Given the slip tendency analysis described above, this assumption by Geomatrix Consultants, Inc. (1999a) is conservative. Based on its own slip tendency analysis, the Staff has concluded that seismic source models that incorporate slip tendency would result in a lower ground motion hazard than the one developed by the Applicant.

In addition, the slip tendency results in the Staff's analysis suggest that Geomatrix Consultants, Inc. (1999a) may have overestimated the maximum magnitude of the East and East Cedar Mountain faults near the proposed PFS site. In its Safety Analysis Report ("SAR"), the Applicant first developed conceptual models of the physical dimensions of fault rupture -- either rupture area or trace length of surface fault rupture -- based on the geologic record (Geomatrix Consultants, Inc.,1999a). Second, the Applicant developed distributions of maximum magnitudes for each active fault using empirical scaling relationships developed from the magnitudes and associated rupture dimensions of historical earthquakes (e.g., Wells and Coppersmith,1994). In developing the fault segment models, the Applicant conservatively assumed that the entire mapped length of the surface trace length represents active fault segments. Thus, these maximum fault dimensions produce conservative estimates of maximum magnitude.

The Staff's slip tendency analysis indicates that parts of the East and East Cedar Mountain faults near the proposed PFS Facility site have relatively low slip tendency values. Thus, these faults may actually be smaller than is represented in the fault models used by the Applicant to estimate maximum magnitude. Fault rupture models developed using slip tendency analysis would lead to fault models with smaller rupture dimensions (length or area) than those used by Geomatrix Consultants, Inc. (1999a). Because the Applicant derived distributions of maximum magnitude for each active fault from empirical scaling relationships of rupture area or rupture length, application of the slip tendency analysis would result in smaller predicted maximum magnitudes than those developed by the Applicant. Smaller maximum magnitudes would reduce the overall ground motion hazard.

The conservative nature of the Applicant's source characterization and the PSHA results presented in the SAR is evident when the results are compared to PSHA results for other sites in Utah, especially those in and around Salt Lake City. Such a comparison shows that the seismic hazard in Skull Valley was calculated by the Applicant to be higher than the seismic hazards for sites at, or near, Salt Lake City, despite the fact that fault sources near Salt Lake City are larger and more seismically active than fault sources near the PFS site. For example, the results of the Applicant's PSHA for Skull Valley (Geomatrix Consultants, Inc., 2001a) suggest that it is 1.5 times more likely that a ground motion of 0.5g horizontal peak ground acceleration or greater will be exceeded at the PFS site (assuming hard rock site conditions), than at Salt Lake City, based on the USGS National Earthquake Hazard Reduction Program (Frankel et al., 1997). This is shown in a Figure entitled "Comparison of Western U.S. Hazard Curves," which the Staff intends to submit as an Exhibit in this proceeding.

Similarly, the 2,000-year horizontal peak ground acceleration for Skull Valley (soil hazard) as estimated by the Applicant, is actually higher than the 2,500-year ground motions for the nine sites along the Wasatch Front that were evaluated as part of the Utah Department of Transportation I-15 Reconstruction Project (Dames & Moore, Inc., 1996). For example, the horizontal PGA calculated at the nine sites in the I-15 corridor study range between 0.561g and 0.686g, based on a mean annual probability of exceedance of 4×10^{-4} (2,500-year return period) -- as compared to the Applicant's estimated horizontal PGA of 0.711g, based on a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period) at the PFS site. It should also be noted that the ground motions estimated by the Applicant in Skull Valley are higher than those estimated for the I-15 corridor, despite the close proximity of Salt Lake City to the Wasatch fault -- which has a slip rate nearly ten times greater than the Stansbury or East Faults (cf. Martinez et al., 1998; Geomatrix Consultants, Inc., 1999a), and is capable of producing significantly larger magnitude earthquakes than the faults near the proposed PFS Facility site in Skull Valley (cf. Machette et al., 1991; Geomatrix Consultants, Inc., 1999a). Because the Applicant's estimate of the seismic hazard is conservative, the proposed ground motions based on the mean annual probability of exceedance of 5×10^{-4} (2,000-year return period) provides an additional margin of safety in the seismic design.

As further stated in the Consolidated SER (pages 2-48 to 2-49), the Staff found that the Applicant's exemption request was acceptable in that:

> (1) Seismic events that could potentially affect the site were identified and the potential effects on safety and design were adequately assessed.

(2) Records of the occurrence and severity of historical and paleoseismic earthquakes were collected for the region and evaluated for reliability, accuracy, and completeness.

(3) Appropriate methods were adopted for evaluations of the design basis vibratory ground motion from earthquakes based on site characteristics and current state of knowledge.

(4) Seismicity was evaluated by the techniques of 10 C.F.R. Part 100, Appendix A. The seismic hazard, however, was evaluated using a probabilistic approach as stated in the request for an exemption from the requirements in 10 C.F.R. § 72.102(f)(1).

(5) The liquefaction potential or other soil instability from vibratory ground motions was appropriately evaluated.

(6) The design earthquake was found to have a value for the horizontal ground motion greater than 0.10g with the appropriate response spectrum and, thus, a site-specific analysis was appropriate.

(7) The Applicant's considerations with respect to the approach taken to model the epistemic uncertainty in ground motions and near-source effects were adequate.

(8) As discussed in Stamatakos, et al. (1999), the Applicant adequately applied adjustment factors for the near-fault effect using the state-of-the-art techniques and applied procedures described in Regulatory Guide 1.165 (1997) for developing the design-basis ground motion. The associated response spectra and design basis motion levels were found to be adequate.

Q13. Has the Staff reached a conclusion as to whether the Applicant's exemption request is acceptable, insofar as it is based upon seismic design ground motions that have a mean annual probability of acceptance of 5×10^{-4} (2,000-year return period)?

A13. Yes. The Staff has concluded that the Applicant's use of a PSHA and ground motions that have a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period) provides an acceptable basis for the seismic design of the proposed PFS Facility. Apart from considerations as to the acceptability of the Applicant's PSHA (discussed above), the Staff based its conclusions upon the following considerations with respect to the appropriate probability of exceedance (return period) to be utilized in establishing the seismic design of the proposed PFS Facility, as set forth in the Consolidated SER (pages 2-49 to 2-51).

First, as stated in SECY-98-071, the radiological hazard posed by a dry cask storage facility is inherently lower than operating commercial nuclear power plants, noting that "a major seismic event at an ISFSI storing spent fuel in dry casks or canisters would have minor radiological consequences compared with a nuclear power plant, spent fuel pool, or single massive storage structure." SECY-98-071, at 2. As further stated therein, "the design earthquake for cask and canister technology need not be as high as a nuclear power plant safe shutdown earthquake." *Id.* (citing comments in Statement of Consideration, "Licensing Requirements for the Storage of Spent Fuel in an Independent Spent Fuel Storage Installation," 45 Fed. Reg. 74,693, 74,697 (1980).

Second, as set forth in the Consolidated SER (page 2-50), the seismic design for commercial NPPs is based on a determination of the SSE ground motion. Heretofore, this ground motion has been estimated using a deterministic approach in the initial licensing of a NPP. Based on an analysis of the SSEs for existing nuclear power plants, in Regulatory Guide 1.165 the Staff established the appropriate Reference Probability to determine the SSE at future NPP sites in connection with the use of a PSHA approach under 10 C.F.R. § 100.23. The Reference Probability was determined to be a 1×10^{-5} median annual probability of exceedance (approximately equivalent to a 100,000-year return period). The Reference Probability, which is defined in terms of the median probability of exceedance, corresponds to a mean annual probability of exceedance of 1×10^{-4}. That is, the same design ground motion that has a median Reference Probability of 1×10^{-5}, has a mean annual probability of exceedance of 1×10^{-4}. Further, analyses of SSEs at nuclear power plants in the western United States (where the proposed PFS Facility would be sited), show that the average mean annual probability of exceeding the safe shutdown earthquake is 2.0×10^{-4} -- which is equivalent to a <u>5,000-year return period</u>. U.S. Department of Energy, "Preclosure Seismic Design Methodology for a Geologic Repository at Yucca Mountain, TR-003-NP, Rev. 2 (1997).

Based on the foregoing considerations, the Staff determined that the mean annual probability of exceedance of the seismic design ground motions at the proposed PFS Facility may be greater than 1×10^{-4} (*i.e.,* something less than a 10,000-year return period). Specifically, the Staff found that in considering the reduced risk posed by an ISFSI as compared to a nuclear power plant, a mean annual probability of exceedance of 5×10^{-4} (2,000 year return period) as a basis to determine the seismic design ground motions appropriately may be used for the proposed PFS Facility.

Finally, in addition to the above considerations, as discussed in Consolidated SER (page 2-51), the Staff favorably considered two other instances in which seismic design ground motions with an annual probability of exceedance of 5×10^{-4} (2,000-year return period) was found to be appropriate. These are (a) the U.S. Department of Energy's issuance of a Standard concerning the use of PSHAs for DOE facilities, DOE-STD-1020-94, "Natural Phenomena Hazards Design and Evaluation Criteria for [DOE] Facilities" (April 1994, as revised January 1996), and (b) the Commission's 1998 approval of a 5×10^{-4} mean annual probability of exceedance (2,000-year return period) for seismic design ground motions at the TMI-2 ISFSI at INEEL, described in SECY-98-071.

Q14. Please describe the extent to which the Staff relied upon the "two other instances" referred to in the last paragraph of your response to Question 13, in determining to approve the PFS seismic exemption.

A14. With respect to the first of these two matters, DOE-STD-1020-94 defines four performance categories for SSCs important to safety (in addition to a "PC-0" category that has no associated safety considerations). The Staff considered that DOE-STD-1020-94 provided an appropriate reference for characterizing the grades of radiological hazards at nuclear facilities such as ISFSIs and NPPs. Further, DOE-STD-1020-94 established the mean hazard annual probability of exceedance for seismic design for the range of SSCs at DOE sites, including ordinary structures (such as warehouses and office buildings) to structures presenting various levels of radiological hazards. Within this range of facilities considered by the DOE are nuclear fuel facilities like the proposed PFS ISFSI. In particular, DOE-STD-1020-94 requires that PC-3 SSCs (which are analogous to SSCs at a dry spent fuel storage facility) be designed for ground motions that have a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period). It should be noted, however, that while the Staff referred to DOE-STD-1020-94, the Staff did not adopt that standard as a regulatory criterion for use in licensing or regulating the proposed PFS Facility or any other NRC-licensed facility.

With respect to the second matter identified above, the Staff referred to the Commission's prior acceptance of a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period) as the basis for establishing the seismic design ground motions for the TMI-2 ISFSI at INEEL, which is discussed in SECY-98-071 and CNWRA-98-007 (Chen and Chowdhury, 1998). The TMI-2 ISFSI was designed to passively store spent nuclear fuel debris in dry storage casks, similar to the passive storage of spent nuclear fuel in dry storage casks at the proposed PFS Facility. In referring to the Commission's approval of the TMI-2 ISFSI seismic design ground motion, the Staff found it to constitute an appropriate point of reference, notwithstanding the fact that it did not establish a regulatory criterion having generic applicability.

In summary, the Staff considered that these points of reference provided relevant technical and regulatory insights for consideration in deciding that a seismic design based on ground motions that have a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period) is appropriate for the proposed PFS Facility.

Q15. Are you familiar with Unified Contention Utah L/QQ, Subpart E?

A15. Yes. As admitted by the Licensing Board, Unified Contention Utah L/QO, Subpart E, states as follows:

Unified Consolidated Contention Utah L/QQ (Geotechnical)

* * *

E. Seismic Exemption.

Relative to the PFS seismic analysis supporting its application and the PFS April 9, [sic] 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:

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

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

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

4. In supporting the grant of the exemption based on 2000-year return period, the staff relies upon the United States Department of Energy (DOE) standard,

DOE-STD-1020-94, and specifically the category-3 facility SSC performance standard that has such a return period, notwithstanding the fact the staff categorically did not adopt the four-tiered DOE category scheme as part of the Part 72 rulemaking plan.

5. In supporting the grant of the exemption based on the 2000-year return period, the staff relies upon the 1998 exemption granted to DOE for the Idaho National Engineering and Environmental Laboratory (INEEL) ISFSI for the Three Mile Island, Unit 2 (TMI-2) facility fuel, which was discussed in SECY-98-071 (Apr. 8, 1998), even though that grant was based on circumstances not present with the PFS ISFSI, including (a) existing INEEL design standards for a higher risk facility at the ISFSI host site; and (b) the use of a peak design basis horizontal acceleration of 0.36 g that was higher than the 2000-year return period value of 0.30 g.

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

Q16. Do you agree with the assertion in Part E of this contention, that 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"?

A16. No.

Q17. Please provide the bases for this conclusion.

A17. First, as set forth in the Staff's Consolidated SER and discussed above, the Applicant's use of a PSHA methodology is acceptable. The Commission has indicated that, in adopting a risk-informed philosophy for regulation, probabilistic methods are appropriate, as shown in recent revisions to NRC regulations (*e.g.,* 10 C.F.R. Parts 50 and 100) and the issuance of regulatory guidance (*e.g.,* Regulatory Guide 1.165). In addition, the PSHA methodology has been accepted by the scientific and engineering community as a well-founded approach to evaluate seismic hazards and, in particular, to incorporate the uncertainties in the evaluation process.

Second, for the reasons discussed above, the use of a return period of 10,000 years, or a return period that is "significantly greater than 2,000 years" is not necessary; rather, the use of ground motions with a 2,000-year return period provides an adequate basis for the seismic design of the proposed PFS Facility, particularly when considering the lower radiological risk that an ISFSI with a dry cask storage system presents as compared to a nuclear power plant. This matter is discussed in greater detail below, with respect to Part E, paragraph 3 of the contention.

Q18. In Part E, paragraph 1 of the contention, the State asserts that the Applicant's exemption request "fails to conform to the SECY-98-126 (June 4, 1998) rulemaking plan scheme," in which only 1000-year and 10,000-year return periods are specified for design earthquakes for [SSCs]", and that "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." Do you believe this presents a valid concern?

A18. No.

Q19. Please provide the bases for this conclusion.

A19. First, the Staff does not consider that SECY-98-126 established a regulation or proposed regulation, but only set out a proposed regulatory approach. Further, the Commission has now approved a modification of that proposed regulatory approach in SECY-01-0178, which supercedes the approach proposed in SECY-98-0126. The favored option in SECY-01-0178 proposes a seismic design in conjunction with a PSHA methodology under 10 C.F.R. Part 72, based on ground motions with a mean annual probability of exceedance of 5×10^{-4} (2,000 year return period ground motion). Thus, the Staff does not view the approach specified in SECY-98-126 as having regulatory significance at this time, especially in light of the Commission's approval of the modified rulemaking plan in SECY-01-0178.

Second, the approach proposed in SECY-98-126 was not followed by the Commission in its approval of the TMI-2 ISFSI exemption request, where the Commission approved an exemption from the deterministic criteria of 10 C.F.R. § 72.102(f)(1) and allowed the ISFSI applicant to use a PSHA with a seismic design based on ground motions with an a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period). Thus, in the only other ISFSI seismic exemption approved to date, the Commission did not follow the approach reflected in the original rulemaking plan, but instead followed the approach that is reflected in the subsequent modified rulemaking plan, based on a PSHA methodology with ground motions having a mean annual probability of exceedance of 5×10^{-4} (2,000 year-return period). While the Staff does not view the TMI-2 ISFSI exemption as establishing a generic precedent, the TMI-2 exemption does provide a pertinent reference point.

Third, in adopting the regulations in 10 C.F.R. Part 72, the Commission indicated that the design earthquake for an ISFSI should be determined on a case-by-case basis.

> [F]or ISFSI's which do not involve massive structures, such as dry storage casks and canisters, the required design earthquake will be determined on a case-by-case basis until more experience is gained with the licensing of these types of units.

Statement of Consideration, 45 Fed. Reg. at 74,697. The Staff's approval of the PFS exemption request constitutes such a case-specific approval, as contemplated in the Commission's Statement of Consideration, and is based upon consideration of the safety of the proposed PFS Facility in the event of an earthquake, as compared to the radiological risks of a major seismic event at a nuclear power plant.

Q20. In Part E, paragraph 2 of the contention, the State asserts that "PFS has failed to show that its facility design will provide adequate protection against exceeding the section 72.104(a) dose limits." Do you believe this presents a valid concern?

A20. No.

Q21. Please provide the bases for this conclusion.

A21. We believe that the State's assertion that PFS has failed to show that its facility design will provide adequate protection against exceeding the dose limits in 10 C.F.R. § 72.104(a) is misplaced. As discussed in the NRC Staff's Testimony of Michael Waters, filed herewith, we understand that this regulatory standard applies to normal operations and anticipated occurrences -- *i.e.*, events which are anticipated to occur during the license term -- rather than design basis earthquakes, for which the dose limits specified in 10 C.F.R. § 72.106(b) would apply. Moreover, as set forth in the NRC Staff's testimony of Daniel J. Pomerening, Vincent Luk and Michael Waters filed herewith, and in the Consolidated SER (Chapter 15), we understand that the HI-STORM 100 storage casks are not expected to slide into each other, tipover, or release radiological materials in the event that design basis (or significantly larger) earthquake ground motions occur at the proposed PFS Facility.

Q22. In Part E, paragraph 3 of the contention, the State asserts that "the Staff's reliance on the reduced radiological hazard of stand-alone ISFSIs as compared to commercial power reactors as justification for granting the PFS exemption is based on incorrect factual and technical assumptions 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." Do you believe this presents a valid concern?

A22. No.

Q23. Please provide the bases for this conclusion.

A23. This conclusion is supported by a number of considerations. First, as discussed above, the Commission has recognized that the potential consequences of seismically initiated failures at the proposed PFS Facility would be much less severe than the potential consequences of seismically-induced failures at an operating nuclear power plant. The PFS SAR proposes static operations involving spent nuclear fuel that is continuously contained in a multi-purpose canister ("MPC") inside a shipping, transfer or storage cask, as compared to complex operations at nuclear power plants which involve the fission of nuclear material at elevated temperatures and pressures, and/or the storage of spent nuclear fuel in spent fuel pools without the confinement boundary afforded by an MPC. In this regard, in considering the appropriate mean annual probability of exceedance (return period) for seismic design ground motions, the Staff was guided by the Commission's Statement of Consideration in adopting 10 C.F.R. Part 72, in which the Commission stated:

> Radiological risks to the public result from a release of radioactive materials and their dispersal to the environment. Once in place, spent fuel storage is a static operation and during normal operations the conditions required for the release and dispersal of significant quantities of radioactive materials are not present. There are no high temperatures or pressures present during normal operations or under design basis accident conditions to cause the release and dispersal of radioactive materials. This is primarily due to the low heat generation rate of spent fuel with more than one year of decay before storage in an ISFSI required by the rule [in Part 72] and with the low inventory of volatile radioactive materials readily available for release to the environs.

45 Fed. Reg. at 74,694.

Second, Regulatory Guide 1.165 determined the Reference Probability (e.g., the annual probability of exceedance) to be used to determine the safe shutdown earthquake (SSE) at future nuclear power plant sites, based on an analysis of the SSEs at 29 existing NPP sites. As set forth therein, the 29 plant sites used in the analysis were relatively recent power plant designs that used the Regulatory Guide 1.60 (1973) or similar spectral shapes in their seismic design; further, the use of these 29 NPP sites, and the use of the Reg. Guide 1.60 spectral shapes, was found to ensure "an adequate level of conservatism in determining an SSE consistent with licensing decisions."

Significantly, the analysis performed in Regulatory Guide 1.165 used the median probability of exceedance of the SSEs, and established a Reference Probability for nuclear power plant SSEs of 1 $\times 10^{-5}$ (100,000-year return period).

Further analysis of Regulatory Guide 1.165, provided in "Revision of Seismic and Geologic Siting Criteria (Murphy, et al., 1997), indicates the median-based Reference Probability determined for 29 NPP sites, is approximately an order of magnitude smaller than a similarly determined mean-based Reference Probability that is derived from the mean probability of exceeding the NPP site SSEs. Thus, the median-based Reference Probability in Regulatory Guide 1.165 of 1×10^{-5} (100,000 year return period) is approximately equal to a mean-based Reference Probability of 1×10^{-4} (10,000 year return period).

The above comparison of mean and median values suggests that the appropriate mean-based Reference Probability for SSEs for new nuclear power plants would be 1×10^{-4} (10,000 year return period). A similar conclusion was reached by DOE in Appendix C of its recent revision to DOE-STD-1020-94 (*i.e.,* DOE-STD-1020-2002). In that analysis, DOE analyzed the mean annual probability of exceedance of 69 NPP SSEs, and concluded that the appropriate mean-based reference probability is slightly greater than 1×10^{-4} (10,000 year return period). It should be noted that these 69 NPPs are all in the Eastern United States.

The Staff came to a similar conclusion about the appropriate mean annual probability of exceedance for the seismic design ground motion, in its evaluation of the TMI-2 ISFSI exemption request. In the safety evaluation attached to SECY-98-071, the Staff stated:

> [DOE] Standard 1020 defines four performance categories (PCs) for structures, systems, and components (SSCs) important to safety, with PC 4 facilities being those with potential accident consequences similar to a commercial nuclear power plant. Such facilities must have a design earthquake equal to the mean seismic ground motion with a 10,000-year return period.

SECY-98-071, at 3 (emphasis added).

Similarly, the safety evaluation attached to SECY-98-071 indicated that both DOE PC-4 facilities and commercial nuclear power plants must be designed to withstand "the mean seismic ground motion with a 10,000-year return period," and further stated that under Regulatory Guide 1.165 ("Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion"), "a future NPP licensed by the NRC in the western United States would be allowed to design to this same level." SECY-98-071, Attachment at 3.

In fact, following the procedures of Regulatory Guide 1.165 for NPPs in the western United States, the average annual probability of exceeding the SSE would be greater than 1×10^{-4} (10,000-year return period). Such an evaluation was carried out by DOE, which concluded that the mean annual probability of exceeding the design basis ground motions for NPPs in the western United States is approximately 2×10^{-4} (5,000-year return period).

In light of the fact that the radiological risks of an ISFSI are inherently lower than the radiological risks at NPPs, and thus an ISFSI's design ground motions need not be as large (*i.e.*, improbable) as those used for NPP design, the Staff concluded that the appropriate mean annual probability of exceedance of the proposed PFS Facility's seismic design ground motion should be greater than the NPP value of 1×10^{-4} (10,000-year return period), and may be greater than the average mean annual probability of exceeding the SSE at NPPs in the western United States of approximately 2×10^{-4} per year (5,000-year return period).

In sum, contrary to the State's assertion, the Staff correctly understood the difference between mean and median values, and properly considered the mean annual probability of exceeding the seismic design ground motions.

Q24. In Part E, paragraph 4 of the contention, the State asserts that "in supporting the grant of the exemption based on 2000-year return period, the staff relies upon the United States Department of Energy (DOE) standard, DOE-STD-1020-94, and specifically the category-3 facility

SSC performance standard that has such a return period, notwithstanding the fact the staff categorically did not adopt the four-tiered DOE category scheme as part of the Part 72 rulemaking plan." Do you believe this presents a valid concern?

A 24. No.

Q25. Please provide the bases for this conclusion.

A25. The Staff did not adopt DOE-STD-1020-94 in approving the 2,000-year return period as the basis to determine the seismic design ground motions of the proposed PFS Facility. Rather, as discussed above, the Staff referred to the DOE Standard as a point of reference, in that DOE-STD-1020-94 established a mean reference probability corresponding to a 2,000-year return period as the basis for determining the design ground motions for DOE Performance Category-3 SSCs. DOE PC-3 facilities are generally comparable to NRC-licensed ISFSIs. Accordingly, while the Staff referred to the DOE Standard, it did not attempt to impose DOE-STD-1020-94 as a regulatory standard on the proposed PFS Facility, nor did it find any reason to require an NRC license applicant (here, PFS) to justify its seismic exemption request on the type of analysis that DOE might conduct under the DOE Standard, in order to meet all the specified DOE requirements.

The underlying philosophy of DOE-STD-1020-94 is to use a risk-graded approach in establishing the seismic (or other) hazard's mean annual probability of exceedance, and in establishing design and evaluation criteria to satisfy performance goals for different categories of critical facilities. Although not expressed in the same terminology, the Staff's evaluation and approval of a seismic design ground motion corresponding to a 2,000-year return period for the proposed PFS Facility relies on a consideration of risk. Thus, as discussed above, the Staff considered (a) the Commission's risk-related statements in the Statement of Consideration issued upon its adoption of the regulations in 10 C.F.R. Part 72; (b) the Commission's previous approval of the seismic design ground motion with a 2,000-year return period for the TMI-2 ISFSI, which included a quantitative risk assessment; and (c) the DOE standard which similarly recognizes that PC-3 facilities present lower radiological risks than PC-4 facilities (which are similar to a NPP). For example, in SECY-98-071, the Staff stated as follows:

> The staff also considered the relative risk posed by the ISFSI. The staff examined relative risk by referring to DOE Standard 1020 . . . . This standard takes a graded approach to designing critical facilities, requiring facilities with greater accident consequences to use higher design requirements for phenomena such as earthquakes . . . . Dry spent fuel storage facilities such as the TMI-2 ISFSI, are PC 3 and must have a design earthquake equal to the mean ground motion with a 2000-year return period. Considering the minor radiological consequences from a canister failure, and the lack of a credible mechanism to cause a failure, the staff finds that the DOE approach of using the 2000-year return period mean ground motion as the design earthquake for dry storage facilities is adequately conservative.

SECY-98-071 at 3. Thus, considerations of radiological risk entered into the Staff's determination to approve the use of a seismic design ground motion with a 2,000-year return period, as derived from the Applicant's PSHA for the proposed PFS Facility.

Q26. Are you familiar with DOE's issuance of a revision to DOE-STD-1020-94, in a DOE-STD-1020-2002, dated January 2002?

A26. Yes.

Q27. Are your conclusions affected by this development?

A27. No. In DOE-STD-1020-2002, DOE revised its Standard by changing the hazard annual probability of exceedance for the seismic design ground motion for PC 3 SSCs, from a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period) to 4×10^{-4} (2,500-year return period). The Staff has discussed this development with the DOE official responsible for making this change (Dr. Harish Chander). Based on these conversations, we understand that the revision was not based upon technical considerations, but instead was undertaken in order to make the DOE standard consistent with U.S. Geological Survey National Earthquake Hazard Reduction

Program ("NEHRP") maps, and thereby result in analytical descriptions of seismic hazards that can be more readily used in conjunction with the USGS NEHRP maps.

Notwithstanding DOE's revision of this Standard, the fact that DOE made this change in the hazard annual probability of exceedance for determining the seismic design ground motion for PC-3 facility SSCs from 5×10^{-4} (2,000-year return period) to 4×10^{-4} (2,500-year return period), is inconsequential. This revision results in a small change in the mean probability of exceedance of the seismic design motion, as compared to the uncertainty in the estimate of the probability of exceedance of earthquake ground motions. For these reasons, DOE's revision to DOE-STD-1020-2002 does not affect our conclusion as to the acceptability of the PFS seismic exemption request, insofar as it is based upon an analogy to DOE's PC-3 hazard annual probability level.

Q28. In Part E, paragraph 5 of the contention, the State asserts that "in supporting the grant of the exemption based on the 2000-year return period, the Staff relies upon the 1998 exemption granted to DOE for the [TMI-2 ISFSI], . . . 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 2000-year return period value of 0.30 g." Do you believe this presents a valid concern?

A28. No.

Q29. Please provide the bases for this conclusion.

A29. The Staff's evaluation of the TMI-2 ISFSI exemption request and the reasons for granting that request are clearly described in the TMI-2 ISFSI docket, including SECY-98-071. Specifically, with respect to the assertions in this part of Contention Utah L/QQ, Part E, it should be noted that (a) "existing INEEL design standards for a higher risk facility at the INEEL host site" did not play any role in the approval of the TMI-2 ISFSI exemption request; and (b) although the

TMI-2 ISFSI had been designed to a slightly higher standard than the ground motion that has a 2,000-year return period (5×10^{-4} mean annual probability of exceedance), the Commission approved the lower 2,000-year ground motion as the acceptable seismic design basis for the facility. Both of these conclusions are evident in SECY-98-071.

The State's reference to "existing INEEL design standards" appears to be a reference to the INEEL architectural engineering ("AE") standards. The INEEL AE standards resulted in DOE's selection of a peak design basis horizontal acceleration for the Idaho Chemical Processing Plant ("ICPP") site of 0.36 g, including the effects of soil amplification. Inasmuch as the TMI-2 ISFSI was placed at the ICPP site, DOE utilized the same standard in constructing the lower-risk TMI-2 ISFSI. However, DOE's decision to utilize that same standard in constructing the TMI-2 ISFSI did not result in a determination by the Staff that a 0.36 g ground motion be used as the basis for approving the exemption request.

Second, in approving a design basis ground motion for the TMI-2 ISFSI, the Staff (and Commission) approved the use of design ground motions that have a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period), with an associated peak horizontal acceleration of 0.30 g, as an acceptable design basis for the facility. Thus, SECY-98-071 states, "[g]iven the absence of radiological consequences from any credible seismic event, the Staff finds that the DOE Standard 1020 risk-graded approach of using the 2000-year mean return period ground motion as the DE (design earthquake) is adequately conservative."

The TMI-2 ISFSI exemption is pertinent in another respect. In this regard, we note that the Staff's (and the Commission's) approval of a 2,000-year return period design basis ground motion for the TMI-2 ISFSI was based, in part, on an assessment of the radiological risks at that facility. Thus, in SECY-98-071, the Staff noted that it had considered the public health and safety consequences of a major seismic event occurring at the facility. Accident analyses for the design basis ground motion at the TMI-2 ISFSI showed that the consequences were bounded by a canister drop onto the concrete pad -- and that the casks and canisters were designed to withstand such events with no release of radioactive materials. Likewise, accident analyses for the proposed PFS Facility have similarly concluded that a cask drop event would not result in the release of radioactive materials, as is discussed in the NRC Staff's testimony of Michael Waters, filed herewith. Thus, the TMI-2 ISFSI example is also analogous for this reason, with respect to the Applicant's seismic exemption request for the proposed PFS Facility.

Q30. In Part E, paragraph 6 of the contention, the State asserts that "because (a) design levels for new Utah building construction and highway bridges are more stringent; and (b) the PFS return period is based on the twenty-year initial licensing period rather than the proposed thirty- to forty-year operating period, the 2000-year return period for the PFS facility does not ensure an adequate level of conservatism." Do you believe this presents a valid concern?

A30. No.

Q31. Please provide the bases for this conclusion.

A31. We believe that the State's first assertion ("design levels for new Utah building construction and highway bridges are more stringent") is not correct. A comparison between the probability of exceedance of the design basis ground motions is not the appropriate metric on which to judge the level of conservatism in the design of facilities of different types. Rather, an appropriate comparison should consider all elements of the seismic design process, including the probability of exceeding the seismic design ground motions, the factors of safety and conservatism in the seismic design and evaluation of SSCs, requirements with regard to design details and quality assurance, and the consequences of failure. For example, DOE-STD-1020-94 considers the differences between the factors of safety that are provided in building codes for ordinary structures and those provided for critical facilities such as nuclear reactors. Inasmuch as SSCs important to safety at the proposed PFS Facility will be designed to NRC seismic design requirements, the resulting factors of safety and conservatism will be greater than those achieved by building codes. Thus, the State's assertion is invalid, in that it does not discuss these factors.

Further, the State's second assertion ("the PFS return period is based on the twenty-year initial licensing period rather than the proposed thirty- to forty-year operating period") is misplaced. Consistent with established engineering practice, design basis earthquake ground motions are based on the risk at a facility, which includes a mean annual probability of exceedance, determined using the existing information, and potential consequences of seismically initiated failures. The Staff's approval of the PFS exemption request was based on the use of seismic design ground motions that have a mean annual probability of exceedance of 5×10^{-4} (2000-year return period) -- *i.e.*, this determination was based on the mean annual seismic hazard at the facility, and not upon a consideration of the licensing period. In the event that PFS receives a license and later seeks to extend or renew the license term, the Staff would evaluate that request based on available information and analyses at that time.

Q32. Does this conclude your testimony?

A32. Yes.

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M.S., Geology, Lehigh University, Bethlehem, Pennsylvania, 1988
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GENERAL QUALIFICATIONS:

Dr. Stamatakos is a structural geologist and geophysicist with international research experience in regional and global tectonics. Dr. Stamatakos has conducted research on a range of topics including paleomagnetism, neotectonics, kinematics of fault block rotations in strike-slip, normal, and thrust fault systems, effects of internal strain on the magnetic properties of deformed rocks, evolution of curvature in arcuate mountain belts, and age and sequence of deformation in folded and faulted mountain belts. This research has focused on the northern and central Appalachians in the eastern United States and Canada, the Hercynian mountains in Germany and northern Spain, the Rocky Mountains and Basin and Range in the western United States, and the northern Cordilleran Mountains in Alaska. Other strengths include numerical modeling of deformation, magnetostratigraphy, rock magnetism, and exploration geophysics.

As a Principal Scientist in the Center for Nuclear Waste Regulatory Analyses, Dr. Stamatakos is the Principal Investigator for structural deformation and seismicity, including tectonics and neotectonics research. Tectonics research at CNWRA currently includes compiling a tectonics Geographic Information System (GIS) database, field analyses of the structural and tectonic elements of the Basin and Range province in southwestern United States, evaluation of seismic and faulting hazards at nuclear facilities (including the proposed repository at Yucca Mountain, Nevada; the proposed storage facilities at Skull Valley, Utah; INEEL, Idaho; and Diablo Canyon, California; and the Fuel Fabrication Facilities at Paducah, Kentucky and Savannah River, South Carolina). These investigations, sponsored by the U.S. Nuclear Regulatory Commission, currently support development of the structural, seismological, and tectonic framework for evaluation of risk of earthquakes and volcanic activity, and the effects of structures and tectonic processes on groundwater flow in the region surrounding nuclear facilities.

Prior to coming to CNWRA, Dr. Stamatakos held positions as a visiting faculty at the University of Michigan and as a postdoctoral fellow at the Eidgenössische Technische Hochschule (ETH) in Zurich, Switzerland. At the University of Michigan, Dr. Stamatakos taught courses in field mapping, structural geology, geophysics, and tectonics. Dr. Stamatakos has written or collaborated on nearly 50 papers and reports on seismology, structural geology, tectonics, and geophysics. He has made presentations at international conferences in the U.S., Canada, Asia, and Europe and has won an outstanding paper award from the American Geophysical Union.

Dr. Stamatakos is associate editor of the Geological Society of America Bulletin, former GP Editor for EOS of the American Geophysical Union, and is a regular reviewer of papers for the Journal of Geophysical Research, Earth and Planetary Science Letters, Journal of Geophysics, Journal of Structural Geology, Physics of the Earth and Planetary Sciences, Tectonophysics, Journal of Geology, Geophysical Journal International, Geological Society of America Bulletin, and Geophysical Research Letters as well as grant proposals for the National Science Foundation.

Acquired NSF and similar institutional grant support for research. Taught geology and geophysics at both undergraduate and graduate levels, including two summer field camp sessions. Co-developed and taught advanced field course for petroleum-industry geologists. Supervised undergraduate, master, and Ph.D. research, including service as external committee member on several masters theses and a Ph.D. dissertation.

Memberships: Seismological Society of America, Geological Society of America, American Geophysical Union, Sigma Xi.

PROFESSIONAL EXPERIENCE

2002: Principal Research Scientist, Center for Nuclear Waste Regulatory Analyses, Southwest Research Institute, San Antonio, Texas

1996-2002: Senior Research Scientist, Center for Nuclear Waste Regulatory Analyses, Southwest Research Institute, San Antonio, Texas.

1995-1996: Research Scientist, Center for Nuclear Waste Regulatory Analyses, Southwest Research Institute, San Antonio, Texas.

1995-2001: Adjunct Research Scientist, Department of Geological Sciences, University of Michigan, Ann Arbor, Michigan.

1999-2002: Adjunct Professor, Incarnate Word University, Palo Alto College, University of Texas at San Antonio, all in San Antonio, Texas.

1992-1994: Visiting Assistant Professor, Department of Geological Sciences, University of Michigan, Ann Arbor, Michigan.

1990-1992: Research Associate: Eidgenössische Technische Hochschule (ETH), Institut für Geophysik, Zürich, Switzerland.

1984-1990: Research and Teaching Assistant, Lehigh University, 1984–1990.

1981-1983: Petroleum Geologist, Analex Geosciences, 1981–1983.

RESEARCH INTERESTS:

Global and regional tectonics through the study of earthquake seismology, paleomagnetism, structural geology, neotectonics, magnetostratigraphy, potential-field geophysics (gravity and magnetics), fission-track thermochronology, and numerical modeling.

PUBLICATIONS

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Education

B.S. - Civil Engineering, Villanova University, 1975
M.S. - Structural Engineering, Stanford University, 1976
Ph.D. - Civil Engineering, Stanford University, 1980

Memberships

Association of State Dam Safety Officials
United States Committee on Large Dams
Earthquake Engineering Research Institute
Seismological Society of America

Committees

National Academy of Engineering, National Research Council - Committee on Risk-Based Analyses for Flood Damage Reduction
Association of State Dam Safety Officials Affiliate Member Advisory Committee
U.S. Society on Dams Dam Safety Subccmmittee
American Nuclear Society - ANS-2.29 Subcommittee Chairman - Probabilistic Analysis of Natural Phenomena Hazards for Nuclear Materials Facilities

Awards

1994 Engineering News-Record - Newsmaker
1995 Association of State Dam Safety Officials - Presidents Award

Experience

Jack R. Benjamin & Associates, Inc., Mountain View, CA (since 1979)

Dr. Martin W. McCann, Jr. is President of Jack R. Benjamin & Associates, Inc. (JBA). From 1984 to 1989 he served as Vice President of the corporation. His professional experience includes probabilistic hazards analysis, including seismic and hydrologic events, reliability assessment, probabilistic risk analysis (PRA) for critical facilities, systems analysis, and seismic engineering.

Dr. McCann is currently a consultant to the Southwest Research Institute, Center for Nuclear Waste Regulatory Analyses, in San Antonio, TX, and is participating in the CNWRA review of the probabilistic seismic hazard analysis (PSHA) on behalf of the U.S. Nuclear Regulatory Commission (NRC) with respect to the high level waste repository site proposed by the U.S. Department of Energy (DOE) to be located at Yucca Mountain, Nevada.

Dr. McCann is a consultant to the Swiss nuclear regulatory authority (HSK) in the area of probabilistic seismic hazard assessment. He prepared comprehensive guidelines for conducting detailed probabilistic seismic hazard assessments in Switzerland for nuclear power plant sites.

As a consultant to Bechtel National Corporation, Dr. McCann is leading the effort to perform a PRA for the tank waste processing facility on the DOE Hanford site. As part of this project, the team led by Dr. McCann is developing seismic systems models for the facility, conducting seismic fragility assessments and quantifying the frequency of exceedance of radiological dose levels to facility workers, co-located workers and the public.

As part of seismic studies conducted for nuclear power plants as part of the NRC Individual Plant Examination for External Events, Dr. McCann supported the development of seismic systems models (event and fault trees) and performed seismic risk calculations. These plants included Pilgrim, Kewaunee, Point Beach, Palisades, and Fort Calhoun.

As part of an NRC, DOE and Electric Power Research Institute (EPRI) project, Dr. McCann provided technical support for the Senior Seismic Hazards Advisory Committee that developed improved methods to conduct probabilistic seismic hazard studies. As part of his participation, Dr. McCann contributed to the committee's final report.

As part of a study at the DOE Savannah River Site, Dr. McCann was the project manager of a program to evaluate the risk to nuclear reactor facilities due to seismic events. JBA provided the seismic hazard and fragility input to the risk assessment. In addition, JBA conducted the risk quantification calculations, using software developed at JBA. For the Savannah River Site, Dr. McCann conducted an extensive comparative evaluation of the EPRI and Lawrence Livermore National Laboratory (LLNL) seismic hazard assessments. This study, which involved extensive modification of the EPRI and LLNL seismic hazard software, identified the source of the differences between the two studies and developed a single, composite estimate of the site hazard.

As part of a DOE, industry and EPRI study to evaluate future advanced light water reactor designs, Dr. McCann performed an extensive hazard assessment for the eastern United States. In this analysis the seismic hazard was mapped for the entire eastern United States at a grid spacing of 25 km. These hazard results were used to map the risk of a future nuclear power plant located anywhere in this area.

Dr. McCann was the project manager of a program to conduct an independent review of the EPRI seismic hazard software package, EQHAZARD. Following completion of the software review, JBA maintains the codes for EPRI according to Quality Assurance Standards.

Dr. McCann served on an expert panel organized by the CNWRA that reviewed approaches for fault hazard assessment at high-level waste repository sites.

Dr. McCann was the project manager of a study to evaluate the risk of failure of three lock and dam structures on the Upper Ohio River. This study was concerned with a 25-year projection of the frequency of the loss of function of the navigation locks due to natural and man-made hazards.

Dr. McCann directed a preliminary probabilistic risk assessment for PAR Pond Dam at the DOE Savannah River Site. The study included an assessment of the frequency of dam failure due to seismic, hydrologic, and static load events.

As part of the DOE and LLNL natural phenomena hazards project, Dr. McCann prepared the flood design criteria in Design and Evaluation Guidelines For Department of Energy Facilities Subjected to Natural Phenomena Hazards, UCRL-15910. He was the course lecturer for the flood part of the DOE workshop on natural phenomena hazard. The workshop addresses the DOE flood design guidelines, probabilistic flood hazard assessment and flood design strategies.

Dr. McCann was the project manager of an effort supported by LLNL to review the potential flood hazards at DOE facilities in the United States. The principal objective of this work is to conduct a preliminary, cost-effective review in order to screen those sites that may require an in-depth probabilistic flood hazard analysis. The results of this preliminary effort are a series of recommendations to minimize the risk at each DOE site due to flood hazards. Preliminary flood hazard studies have been performed for nine DOE sites.

Under the direction of Dr. McCann, JBA performed a probabilistic flood hazard assessment for the DOE Hanford Reservation, located adjacent to the Columbia River. The flood hazard assessment considered the possibility of extreme flood events and upstream dam failure as potential causes of onsite flooding.

As a subcontractor to Sandia National Laboratories (SNL) for the NRC Unresolved Safety Issue on Decay Heat Removal, JBA performed probabilistic flood studies at a number of nuclear power plant sites. These studies involved an assessment of the frequency of extreme floods and the frequency of core damage.

Dr. McCann was the project manager of an EPRI-sponsored study to evaluate the engineering characteristics of small-magnitude earthquakes. As part of this study the threshold level of ground motion required to damage nuclear power plant structures and equipment was estimated.

Dr. McCann assisted EPRI in developing an industry position regarding the seismic design basis for future nuclear power plants. As part of this effort, Dr. McCann worked with industry representatives and the NRC to develop an effective, stable approach for seismic siting.

Dr. McCann participated in a project to develop a NRC external event PRA procedures guide and a review document for seismic and external flood hazards.

In the 1980's, Dr. McCann participated in the review of seismic probabilistic risk assessments conducted for the Zion, Indian Point, Limerick, Millstone, and Oconee nuclear power plants.

Department of Civil Engineering, Stanford University (since 1981) - Consulting Professor

Currently, Dr. McCann is the chairman of the National Performance of Dams Program (NPDP) Executive Committee. The NPDP is headquartered at Stanford. The program operates and maintains a library and database on dam incidents. The library contains over 10,000 documents, including the U.S. Committee on Large Dams incident files. Dr. McCann is directing the development of a web-based digital library system. The digital library and database will be an online resource on dams and their performance.

Dr. McCann was the director of a project to develop PRA procedures for the evaluation of dams. The project was supported under a contract with the Federal Emergency Management Agency (FEMA). The objectives of the project included the development of a probabilistic screening procedure to assign priorities to dams in a jurisdiction based on a cost-effectiveness criteria. A methodology to conduct a detailed PRA of existing dams due to all stimuli was also developed.

As part of the FEMA project, Dr. McCann has presented workshops on the probabilistic assessment of dams in the U.S. and in foreign countries.

Working with the Association of State Dam Safety Officials, Dr. McCann was a chairman of a committee to develop a national standard for reporting the performance of dams. The result of this work was the publication of the Guidelines for Reporting the Performance of Dams.

Dr. McCann is the Chairman of the National Dam Safety Information Technology Committee. The purpose of the committee is to develop a strategic plan for the collection, archiving and access to information on dams in the U.S. The committee is comprised of state, federal and private sector engineers.

Partial List of Technical Publications and Reports

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McCann, Jr., M. W., "Historic Seismicity Analysis for the Savannah River Plant," prepared for E.I. du Pont de Nemours & Company, Inc., 1986.

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Laboratory, Livermore, California," JBA Report 105-140, prepared for Lawrence Livermore National Laboratory, Mountain View, California, 1987.

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McCann, Jr., M.W., "SRS PRA of Reactor Operation - Level 1 External Events, Appendix A - Probabilistic Seismic Hazard Evaluation," Westinghouse Savannah River Company, Aiken, South Carolina, July 1990.

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Jack R. Benjamin and Associates, Inc., "A Comparative Assessment of Historic Seismicity and Expert Seismicity Model - Dr. Joseph Litehiser," prepared for Lawrence Livermore National Laboratory, April 1993.

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McCann, Jr., M.W., et al., *Guidelines for Reporting the Performance of Dams,* National Performance of Dams Program and the Association of State Dam Safety Officials, 1994.

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McCann, Jr., M.W. and L. Manuel, L., "Methodology for Seismic Reliability Assessment of Outlet Operations at Dams," Proceedings of the 2nd International Conference on Dam Safety Evaluation, Trivandrum, India, November 1996.

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McCann, M.W., Jr., "Probabilistic Seismic Hazard Analysis for the Anniston Chemical Demilitarization Facility, Anniston, Alabama," prepared for Science Applications International Corporation, October, 2000.

Committee on Risk-Based Analysis for Flood-Loss Reduction (contributing author), *Risk Analysis and Uncertainty,* National Academy Press, Washington, D.C. , 2000.

Rui Chen

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Primary Technical Areas:

* Computational, experimental, and theoretical rock mechanics and geological engineering
* Earthquake engineering and seismology, seismic hazard assessment, structural geology, and neotectonics
* Computer applications and programming

Summary of Qualifications:

* Fifteen years of post-graduate research experience
* Seven years of consulting/industrial experience
* Numerical modeling of rock deformation and fracturing
* Laboratory experience in rock testing and data acquisition
* Consulting experience in rock mechanics and geological engineering
* Application of U.S. regulations in radioactive waste management and disposal and associated compliance review

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EDUCATION

1/89 - 2/93	Ph.D., Civil and Geological Engineering, The University of Manitoba, Winnipeg, Canada Thesis: *In Situ and Laboratory Studies of Potash Deformation - with Reference to Saskatchewan Potash*
9/82 - 7/85	M.Sc., Seismotectonics, Graduate School, the University of Science and Technology of China and Institute of Geology, State Seismological Bureau of China, Beijing, China Thesis: *Earthquake Mechanisms and Repeated Activities of the Western Branch of Xiaojiang Fault Zone, Southeastern Edge of the Qinghai-Tibet Plateau*
9/78 - 7/82	B.Sc., Seismology and Geomechanics, The Geological University of China at Wuhan, China Thesis: *Earthquake Seismicity in Jiangchuang Basin*

EMPLOYMENT HISTORY

09/00 - present	Independent consultant in geological engineering and geosciences
08/01- present	Lecturer, Development of Civil Engineering and College of Natural Sciences, California State University, Chico
09/98 - 08/00	Senior Research Engineer, Center for Nuclear Waste Regulatory Analyses, Southwest Research Institute, San Antonio, Texas, USA.

04/95 - 09/98 Research Engineer, Center for Nuclear Waste Regulatory Analyses, Southwest Research Institute, San Antonio, Texas, USA.

07/93 - 04/95 Staff Engineer, RE/SPEC Inc., Rapid City, South Dakota, USA.

11/92 - 06/93 Consultant, RE/SPEC Inc., Rapid City, South Dakota, USA.

11/88 - 06/93 Graduate Assistant, Department of Geological Engineering, The University of Manitoba, Winnipeg, Manitoba, Canada.

09/85 - 10/88 Research Associate, Institute of Crustal Dynamics, State Seismological Bureau of China.

09/83 - 08/85 Research Assistant, Institute of Geology, State Seismological Bureau of China.

COMPUTER SKILLS

- Finite element, finite difference, and boundary element modeling and programming. Familiar with a number of commercial numerical modeling software packages, including ABAQUS, FLAC, UDEC, 3DEC, ANSYS, MAP3D and special purpose software, including SPECTROM32 and SIMEX.
- Programming for data acquisition and servo-controlling in laboratory rock testing.
- Object-oriented programming for Web and Windows-based applications using html, JAVA, and VISUAL BASIC.
- Familiar with statistical software, BMDP, and programming of non-linear regression analyses of partial differential equations for evaluating model parameters and sensitivity studies for complex constitutive models for geological materials.
- Proficiency in other software tools, including GIS software (e.g., EarthVision).

SELECTED GRADUATE COURSES

Computational Mechanics Topics, Engineering Geology, Rock Engineering and Ground Support, Finite Element Methods, Solid Mechanics, Fracture Mechanics, Advanced Rock Mechanics and Rock Fracture Mechanics, Structural Geology, Seismology, Earthquake Engineering, Tectonophysics, Seismotectonics, Geomechanics, Theoretical Geophysics, Applied Geophysics, and Quaternary Geology and Geomorphology.

MEMBERSHIPS

International Society for Rock Mechanics
American Geophysical Union
Seismological Society of America
International Association for Computer Methods and Advances in Geomechanics

EXPERIENCE

Dr. Chen's experience includes academic-based research and industry-related consulting. Her areas of expertise include experimental, theoretical, and computational rock mechanics; seismology and earthquake engineering; neotectonics; structural geology; and software development.

Dr. Chen's professional activities began in 1983 when she headed the Institute of Geology at the State Seismological Bureau (SSB) of China, in Beijing, as a research assistant. There she was involved in research projects in seismicity and earthquake mechanisms along active fault zones and in active basins in Southwestern and Northwestern China. An early project included seismicity analyses in the Jiangchuang Basin on the Honghe River Fault Zone in Southwestern China. Other projects included mapping geological and geomorphic features along the 1920 Haiyuan Earthquake (M=8.5) Fault Zone and neotectonic and geomorphic mapping, paleoseismic trenching, and seismicity analyses along the Xiaojiang Active Fault Zone. The latter study resulted in the first qualitative evaluation of the slip rate and earthquake recurrence intervals along the fault zone. These results are included in the national database for large-scale active fault zones in China.

Dr. Chen's experience in interdisciplinary research started in 1985 when she joined the Rock Mechanics Division at the Institute of Crustal Dynamics, SSB of China, Beijing, as a research associate. There, she extended her professional activities to include computational and experimental rock mechanics. She managed a project on double-shear rock friction tests for investigating the stability of fractured rock mass. Results from this study were applied to evaluate the stability of concealed fault zones and salt solution-mining induced seismicity in Zigong Anticline, Sichuan Province of China. This work included constructing a structural model for the Zigong Anticline based on geophysical survey data and back analysis of the tectonic stress field using non-linear, viscoplastic finite element methods.

From 10/88 to 06/93, Dr. Chen worked on industrial research projects in the Department of Civil and Geological Engineering at the University of Manitoba as a graduate assistant. There, she led research in two areas of rock mechanics and mining: the yield behavior of deep potash pillars and rock bimodularity. She investigated mining-induced deformation and fracturing in potash mines under contract with the Potash Corporation of Saskatchewan, with matching funding from the Natural Sciences and Engineering Research Council of Canada. The accomplishments in this research included underground mapping of deformational patterns in exposed cross sections of yield pillars after substantial deformation, and correlation of the observed deformation to time-dependent finite strain and shear failure in the pillars using modern techniques in quantitative structural geology. The research led to better understanding of the stress-control design methodology in underground soft rock mining. It also led to several external reports, papers, and presentations to government, commercial, and scientific organizations. She also performed experimental and constitutive research on potash relaxation behavior and on the differences in elastic moduli for rocks in tension and compression, adapted the results on material bimodularity to a special-purpose finite element computer program to study the influence of bimodularity on the behavior of underground structures, and developed computer codes for test control and data acquisition for a sophisticated servo-controlled material testing system. While pursuing her Ph.D. education in the University of Manitoba, she provided technical consulting services to RE/SPEC Inc. on a legal case regarding water inflow into potash mines in Esterhazy, Canada. These services included evaluation of the microseismic monitoring system at the mines, seismic data analyses, and evaluation of the effects of seismicity on mine stability.

During her employment with RE/SPEC Inc. (7/93 - 4/95), Dr. Chen served as Technical Lead on industry and government contracts and performed numerous numerical (finite element and finite difference) simulations in support of experiments at the Waste Isolation Pilot Plant (WIPP) in New Mexico for long-term containment of radioactive wastes. The areas supported include engineering design alternative analyses, and the sealing and backfill programs (crushed salt consolidation and concrete bulkhead experiments). The analyses performed used predictive technology developed for the WIPP to simulate the thermal-mechanical behavior of salt to determine the response of repository seals and closure times for rooms and shafts. She was also involved in the evaluation of consolidation constitutive models for crushed salt in support of the sealing program at the WIPP, including a state-of-the-art literature survey to select candidate models, using BMDP (a statistical software package) and self-developed computer codes to evaluate parameters and their correlations, and constitutive model development. She has also performed numerous two and three dimensional finite element and boundary element modeling studies on underground mining and storage caverns to determine safe operating conditions and to evaluate structural integrity. This work was performed for commercial clients including DynMcDermott Petroleum Operations Company, the Dow Chemical Company, the Warren Petroleum Company, Texas Eastern Products Pipeline Company, and AKZO Mining Company, etc. Another of her interest areas is object-oriented software development for Windows-based applications. She developed a graphical user interface (Visual SalGas) for Solution Mining Research Institute's commercial software known as SALGAS (a numerical leaching model with cavity hydraulics and gaseous pad calculations for design of salt rock solution caverns). A demo-version of Visual SalGas was distributed to SMRI memberships world wild and the full-version has been purchased by companies in the United States, Canada, and Europe.

At the Center for Nuclear Waste Regulatory Analyses (CNWRA) at Southwest Research Institute (SwRI), Dr. Chen performed technical activities related to geological engineering to assist the Nuclear Regulatory Commission (NRC) in licensing the nation's first geological repository for high-level radioactive waste, which is proposed to be developed at Yucca Mountain, Nevada. She was involved in CNWRA analyses of mechanical, thermal, and hydrological processes in complex geomechanical and geotechnical engineering systems. She was involved in review of technical reports produced by the U.S. Department of Energy in supporting its license application for the proposed Yucca Mountain repository in areas such as site geology and seismology, repository design and construction (including seismic design of surface and subsurface facilities), and stability analyses of emplacement drifts and ground support under thermal and dynamic loads for preclosure safety and postclosure performance assessments. She performed independent confirmatory analyses and research to support such reviews. Her role at the CNWRA also includes a Project Manager for geomechanical analyses and their application in gas, oil, and underground storage industries for commercial clients. She was also a Principal Investigator in providing technical support, including evaluation of seismic hazard analyses and seismic design, for the NRC licensing activities related to the nation's spent nuclear fuel dry-storage facilities, including TMI-2 ISFSI and the proposed Private Fuel Storage Facility. These evaluations included review and independent analyses and provided input to the NRC safety evaluation reports, environmental impact statements, and other National Environmental Policy Act documentation required under Title 10 Code of Federal Regulations. She also conducted geotechnical review and independent analyses to support NRC regulation of the uranium mining industry, including slope stability, foundation stability, and liquefaction analyses.

Since relocating to California and becoming an independent consultant, she has provided technical assistance and consulting services to CNWRA at SwRI in solving a broad range of problems in underground rock engineering, seismic hazard assessment, and earthquake engineering. She is also teaching graduate and undergraduate courses at California State University, Chico.

LIST OF PUBLICATIONS

a). Refereed Journals and Conference Proceedings

Chen, R., G.I. Ofoegbu, and S.M. Hsiung. 2000. Modeling drift stability in fractured rock mass at Yucca Mountain, Nevada - Discontinuum approach. *Proceedings, the Fourth North American Rock Mechanics Symposium, Rock Around the Rim,* the University of Washington, Seattle, Washington, July 31 - August 3, 2000.

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Ratigan, J.L., J.D. Nieland, R.Chen, K.D. Mellegard, and T.W. Pfeifle. 1995. Numerical modeling of Weeks Island subsidence and laboratory testing of Weeks Island salt. *Proceedings of the Solution Mining Research Institute 1995 Spring Meeting*, April 30-May 3, New Orleans, Louisiana.

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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-ISFSI
	)	
(Independent Spent Fuel Storage Installation)	)	

NRC STAFF TESTIMONY OF MICHAEL D. WATERS
CONCERNING RADIOLOGICAL DOSE CONSIDERATIONS
RELATED TO UNIFIED CONTENTION
UTAH L/QQ, PART E (SEISMIC EXEMPTION)

Q1. Please state your name, occupation, and by whom you are employed.

A1. My name is Michael D. Waters. I am employed as a Health Physicist in the Spent Fuel Project Office, Office of Nuclear Material Safety and Safeguards, U.S. Nuclear Regulatory Commission ("NRC"). A statement of my professional qualifications is attached hereto.

Q2. Please describe your current responsibilities.

A2. In my position as a Health Physicist, I perform technical reviews of spent nuclear fuel ("SNF") storage casks, independent spent fuel storage installations ("ISFSIs"), and transportation packages, primarily in the areas of shielding, confinement, containment, radiation protection, and criticality. In addition, I continue to be responsible for certain reviews initiated in my former position as a Project Engineer in SFPO, involving management of the safety reviews of applications for these designs and facilities. My safety reviews have included both new ISFSI license applications and amendments to existing licenses.

Q3. Please explain what your duties have been in connection with the NRC Staff's review of the application of Private Fuel Storage, L.L.C. ("PFS" or the "Applicant") for a license to construct and operate an ISFSI on the reservation of the Skull Valley Band of Goshute Indians, geographically located within Skull Valley, Utah (the "proposed PFS Facility").

A3. As part of my official responsibilities, I served as a Project Manager for portions of the NRC Staff's safety evaluation of the proposed PFS Facility, and provided general technical oversight and advice on technical reviews performed by other NRC Staff ("Staff") members. My involvement included review of the Applicant's Safety Analysis Report ("SAR") and participation in the Staff's preparation of the "Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued on September 29, 2000 ("SER"), and Supplement No. 2 to the SER, dated December 21, 2001 ("SER Supplement No. 2"). Those two documents have since been incorporated into the NRC Staff's "Consolidated Safety Evaluation Report Concerning the Private Fuel Storage Facility," issued in March 2002 ("Consolidated SER").

I also assisted the Staff in its preparation of the Final Environmental Impact Statement ("FEIS") for the proposed PFS Facility, NUREG-1714 (December 2001), in which I reviewed general design issues associated with the proposed PFS Facility and its potential radiation impacts on the environment. In addition, I assisted the Staff in preparing the "NRC Staff's Response to Applicant's Motion for Summary Disposition of Utah Contention L, Part B," dated December 7, 2001; and the "NRC Staff's Objections and Responses to the 'State of Utah's Twelfth Set of Discovery Requests directed to the NRC Staff,'" dated October 3, 2001.

Q4. Have you performed any other work among your duties at the NRC that is relevant to the Staff's evaluation of the license application for the proposed PFS Facility?

A4. Yes. On behalf of the Staff, I performed a shielding and radiation protection evaluation of the HI-STORM 100 storage cask system, and a shielding evaluation of the HI-STAR 100 transportation cask system. PFS has proposed to use both of these systems at the PFS Facility. The NRC approved the HI-STORM 100 storage cask design for general use under

Subpart L of 10 C.F.R. Part 72, in the HI-STORM 100 Certificate of Compliance ("CoC") (May 1, 2000). The NRC certified the HI-STAR 100 transportation cask design for SNF transport under 10 C.F.R. Part 71, in CoC No. 9261 for the HI-STAR 100 transportation package (Revision 0, March 31, 1999).

Q5. What is the purpose of this testimony?

A5. The purpose of this testimony is to provide the NRC Staff's views with respect to one portion of Unified Contention Utah L/QQ, Part E, insofar as that contention concerns the potential dose consequences that may result in the event of a beyond-design-basis hypothetical cask tipover.

Q6. Are you familiar with Unified Contention Utah L/QQ, Subpart E.2.?

A6. Yes. As admitted by the Licensing Board, Unified Contention Utah L/QQ, Subpart E.2., states as follows:

> **Unified Contention Utah L/QQ (Geotechnical)**
>
> * * *
>
> **E. <u>Seismic Exemption.</u>**
>
> 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 probablistic rather than a deterministic seismic hazards analysis, PFS should be required either to use a probablistic 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 failed to show that its facility design will provide adequate protection against exceeding the section 72.104(a) dose limits.

Q7. Please identify the Commission's requirements pertaining to the dose limits for an ISFSI, such as the proposed PFS facility, that you considered in your evaluation of this matter.

A7. The Commission's requirements concerning the dose limits for an ISFSI are set forth in 10 C.F.R. Part 72. Specifically, 10 C.F.R. § 72.104(a) provides that annual dose equivalents to any real individual who is located beyond the controlled area boundary will not exceed 25 mrem to the whole body, 75 mrem to the thyroid, or 25 mrem to any other critical organ as a result of exposure to discharges of radioactive material or direct radiation from the ISFSI, during "normal operations and anticipated occurrences." Further, the Commission has established radiation dose limits for individuals located on or beyond the nearest boundary of the controlled area for any "design basis accident," as set forth in 10 C.F.R. § 72.106(b). Specifically, under accident conditions, individuals may not receive the more limiting of a total effective dose equivalent (referred herein as "dose") of 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 50 rem.

Q8. Has the Applicant demonstrated that its proposed facility design will provide adequate protection against exceeding the section 72.104(a) dose limits for normal operations and anticipated occurrences?

A8. Yes. The Staff reviewed the Applicant's dose analysis for normal conditions and anticipated occurrences, set forth in Chapter 7 of the PFS SAR and found it to be acceptable, as documented in Chapters 11 and 15 of the Staff's Consolidated SER.

Q9. In its evaluation, did the Staff consider whether exposures from design-basis accidents or design-basis seismic events at the proposed PFS Facility would exceed the dose limits specified in 10 C.F.R. 72.104(a)?

A9. No. Such an evaluation would have been inappropriate, in that the requirements of 10 C.F.R. § 72.104(a) apply only to "normal operations and anticipated occurrences," and do not apply to design-basis accidents or a design-basis seismic event. Design basis accidents and design basis seismic events do not constitute normal operations or anticipated occurrences; rather, by definition, they constitute "design basis" accidents or events, for which the dose limits in 10 C.F.R. § 72.106(b) apply.

Q10. In Unified Consolidated Contention Utah L/QQ, Subpart E.2, the State of Utah asserts that PFS should be required either to use a probablistic methodology with a 10,000-year return period, comply with the deterministic analysis requirement of 10 C.F.R. § 72.102(f), or use a return period significantly greater than 2000 years, in that "PFS failed to show that its facility design will provide adequate protection against exceeding the section 72.104(a) dose limits." Do you believe that this is a valid concern?

A10. No.

Q11. Please provide the bases for this conclusion.

A11. First, as stated above, the dose limits specified in 10 C.F.R. § 72.104(a) apply to normal operations and anticipated occurrences, and do not apply to design basis accidents or design basis seismic events, for which 10 C.F.R. § 72.106(b) applies.

Second, the State's assertion appears to be premised on certain language in SECY-98-126 (June 4, 1998), in which the Staff had proposed the use of a two-tiered approach in conducting probabilistic seismic hazard analyses ("PSHAs"). Under that proposed approach, a 1,000-year return period might have been proposed for SSCs whose failure would not result in exceedance of the dose limits in 10 C.F.R. § 72.104(a), whereas a 10,000-year return period would have been necessary if the dose limits in § 72.104(a) were exceeded. However, as discussed in the NRC Staff's Testimony of Drs. John Stamatakos, Martin McCann and Rui Chen, filed herewith, the approach proposed in SECY-98-126 has been superseded by SECY-01-0178, in which the Staff proposed (and the Commission approved) the use of a single-level design basis earthquake with a ground motion that is commensurate with the level of risk associated with an ISFSI, instead of the graded approach that was proposed in SECY-98-126 and cited in Part E of this contention.

Accordingly, the Staff considers that the two-tiered approach proposed in SECY-98-126, and its reference to the dose limits in § 72.104(a) to be inapplicable, and there does not exist any valid basis to require PFS to demonstrate that its exemption request satisfies the dose limits specified in 10 C.F.R. § 72.104(a).

Q12. Has the Staff conducted an evaluation to determine if the does limits specified in 10 C.F.R. § 72.106(b) would be satisfied in the event that a design basis earthquake occurs at the proposed PFS Facility?

A12. Yes. The Staff has concluded that in the event that a design basis earthquake occurs at the proposed PFS Facility, the dose limits specified in 10 C.F.R. § 72.106(b) would not be exceeded.

Q13. Please provide the bases for this conclusion.

A13. As set forth in the Staff's Consolidated SER, sections 15.1.2.6 and 15.2 (pages 15-29 to 15-32, and 15-122), the occurrence of a design basis earthquake with a mean annual probability of occurrence of 5×10^{-4} (2,000-year return period) would not impair the ability of SSCs important to safety to maintain subcriticality, confinement, and sufficient shielding of the spent nuclear fuel. Accordingly, the dose limits in 10 C.F.R. § 72.106(b) will not be exceeded in the event that a design basis earthquake occurs at the proposed PFS Facility.

Q14. Did the Staff's evaluation consider whether a cask tipover could occur in the event of a design basis seismic event?

A14. Yes. The Staff reviewed the Applicant's analyses, with respect to whether tipover of a cask at the proposed PFS Facility could occur as a result of a design-basis seismic event at the proposed PFS Facility, which was described in Chapters 2, 3, and 4, and section 8.2.6.1, of the Applicant's SAR. As discussed in section 5.1.4.4 of the Consolidated SER, based on its review of this matter, the Staff concluded that the Applicant had adequately demonstrated that the design-basis seismic event would not cause cask tipover.

Q15. Did the Applicant and Staff also consider the potential consequences of a hypothetical cask tipover, if that event were to occur?

A15. Yes. The Applicant provided an analysis, in accordance with the Standard Review Plan (NUREG-1567), in which it considered whether a hypothetical cask tipover (*i.e.,* a tipover that is non-mechanistically assumed to occur) would impair the cask's ability to maintain subcriticality, confinement and sufficient shielding of the stored fuel.

In its analysis, the Applicant determined that deceleration forces would be less than the HI-STORM 100 design basis values for the MPC. As a result, the MPC would maintain its integrity, maintain its confinement function, and would not release radioactive materials. Therefore, there would be no resultant doses due to a release of radioactive materials. Further, with respect to potential increases in direct radiation, the Applicant stated that localized damage to the radial concrete shield and outer steel shell, where the cask impacts the pad, could result in an increased surface dose rate due to the damage; however, the Applicant indicated that the localized areas would be small and would not produce a "noticeable increase" in the dose rates at the owner controlled area ("OCA") boundary. In sum, the Applicant indicated that there would not be a significant increase in radiation exposures above normal operating conditions as a result of a hypothetical cask tipover. Therefore, based on the Applicant's confinement and shielding analysis of a hypothetical cask tipover, the design basis accident dose limits of 10 C.F.R. § 72.106(b) would not be exceeded. The Applicant's conclusions with respect to this matter are presented in Section 8.2.6 of the PFS SAR.

As set forth in section 5.1.1.4 of the Consolidated SER, the Staff agreed that a hypothetical cask tipover at the proposed PFS Facility would result in stresses in the MPC that are bounded by those evaluated in the Staff's HI-STORM 100 SER, and that the Staff's conclusions in the HI-STORM 100 SER with respect to the structural integrity of the MPC are valid for the proposed PFS Facility. Further, as set forth in section 15.1.2.1 of the Consolidated SER, the Staff reviewed the Applicant's method of analysis, inputs, assumptions and conclusions, and agreed with the Applicant that deformations of the storage cask as a result of a tipover event would not impose unacceptable loads on the MPC. Accordingly, the Staff concluded that a hypothetical tipover would not impair the cask's ability to maintain subcriticality, confinement and sufficient shielding of the stored fuel.

Q16. Did the Staff also consider whether an earthquake that exceeds the design basis seismic event would result in cask tipover at the proposed PFS Facility?

A16. Yes. In this regard, it should be noted that beyond-design basis seismic events are not required to be considered in the licensing or evaluation of a proposed facility. However, in preparing for hearings on this contention, the Staff considered whether a 10,000-year return period seismic event (*i.e.*, a seismic event that is significantly beyond the design basis) would cause the storage casks at the proposed PFS Facility to tipover. As set forth in the NRC Staff's Testimony of Drs. Goodluck I. Ofoegbu and Daniel J. Pomerening, and in the NRC Staff's Testimony of Jack Guttmann and Dr. Vincent Luk, filed herewith, the Staff has concluded that the storage casks would not tipover even in the event of a 10,000-year return period earthquake at the proposed PFS Facility.

Q17. Notwithstanding the Staff's conclusion that neither a design basis seismic event nor a 10,000-year return period seismic event would result in cask tipover at the proposed PFS Facility, did the Staff also analyze the potential offsite dose consequences that might result from a hypothetical multiple cask tipover event, if it were to occur at the proposed PFS Facility?

A17. Yes.

Q18. Please describe the dose consequence analysis that was conducted by the Staff, pertaining to this hypothetical multiple cask tipover event.

A18. On behalf of the Staff, in preparing to address this contention at hearing, I conducted an analysis of a multiple cask tipover event at the proposed PFS Facility. In this analysis, I considered (a) the potential for damage to the cask shield that might result from impact on the concrete storage pad; (b) the potential for thermal degradation of the cask's radial concrete shield in the form of hydrogen loss from the concrete, with the cask assumed to be in a horizontal position; and (c) the potential effect on offsite doses that might be caused by spacial reorientation of the casks from a vertical to tilted or horizontal position (*i.e.,* the potential for direct offsite exposures to the top, side, and/or bottom of the casks).

Q19. Please describe the Staff's evaluation, with respect to the first issue identified in response to Question 18 above, *i.e.,* the potential for damage to the cask shield that might result from impact on the concrete storage pad.

A19. I conducted an analysis, in which I considered the nature, configuration, and amount of shielding provided in the HI-STORM 100 cask system, including the multipurpose canister ("MPC") contained within the cask. Important shielding components within the MPC and/or overpack are shown in Holtec International Drawing No. 1495, Sheet 1, of the HI-STORM 100 FSAR (Staff Exhibit __) and Figure 5.3.11 of the FSAR (Staff Exhibit __). Within the radial sides of the canister and overpack shields, there are a total of approximately 3.25 inches of steel and 26.75 inches of concrete. In the top of the canister and overpack, there are a total of approximately 14.75 inches of steel and 10.5 inches of concrete. In the bottom of the canister and overpack, there are a total of approximately 9.5 inches of steel and 17 inches of concrete. There are openings and penetrations in the overpack shield (such as the small annulus region between the pedestal shield and overpack wall, and the air inlet and outlet vents), and these openings may result in radiation streaming and higher surface dose rates in surrounding areas. However, the openings have an insignificant effect on the overall shielding ability of the top, side, and bottom of the HI-STORM cask, with respect to mitigating OCA boundary dose rates.

As stated above, the hypothetical tipover of a storage cask at the proposed PFS Facility would not impair the cask's ability to maintain subcriticality, confinement and sufficient shielding of the stored fuel. However, it is possible that there could be localized damage near the cask impact area, as noted by the Applicant in its hypothetical tipover analysis. I understand that the State of Utah has postulated that this could be in the form of crushing, "micro-cracking," or flattening of the concrete, or thinning of the steel shell. If this were to occur, the surface dose rates in localized areas of each cask could increase, in theory, due to the postulated damage to the shield.

However, even if one postulates that damage to the shield would occur, this would result in only minor increases in dose rates at the surface of the casks. Each cask would continue to maintain its shielding after a hypothetical tipover to sufficiently attenuate radiation from the spent fuel. In this regard, it should be noted that there would be no significant loss of bulk shielding mass or severe discontinuities in any direction around the radial concrete shield. The concrete is encased in a steel cylindrical shell, with no means of escape after a hypothetical cask tipover. Any postulated minor discontinuities within or damage to the concrete (resulting from crushing, cracking, or flattening of the concrete), or thinning of the steel shell, would have either no effect at all, or would only slightly increase dose rates at the surface of each cask within the vicinity of the damage. These localized dose increases would be minor and confined to a relatively small area of the total surface area of each cask.

In the event such damage occurs and leads to an increased dose rate near the cask, any change in the surface radiation flux near the damaged area likely would not be detectable at the

OCA boundary. This is because the relatively small, additional amount of radiation escaping from these areas would have significantly dissipated at 600 meters (*i.e.*, at the OCA boundary). Radiation dissipation would be caused by scattering and absorption interactions in the surrounding air, and by divergence of the radiation flux as it is emitted in every direction away from the casks.

Further, any minor increases in dose rates would likely be in areas where each horizontal cask is in contact with the storage pad. Therefore, additional radiation that could escape from these areas would be shielded and absorbed by the storage pad underneath. In addition, in any assumed arrangement of tipped casks (whether in a random array or sequential pattern), only the casks located along the "outer periphery" of the arrangement (*i.e.*, the casks on the storage pads located along the outer perimeter of the two pad clusters) could effectively contribute to off-site doses. Except for minor contributions from skyshine, any tipped casks situated on the interior pads of a hypothetical cask arrangement (damaged or undamaged) would be substantially shielded by the casks positioned along the outer periphery of storage pads.

In sum, any minor irregularities in the cask shields that might result from shield damage incurred in a cask tipover, resulting in an increased dose rate at or near the cask surfaces, would not contribute significantly to the total radiation dose rate at the OCA boundary.

Q20. Please describe the Staff's evaluation, with respect to the second issue identified in response to Question 18 above, *i.e.,* hypothetical thermal degradation of the concrete shield in the form of hydrogen loss.

A20. As presented in Section 7.3.3.5 of the PFS SAR and in Holtec Report No. HI-971645, "Radiation Shielding Analysis for the PFS," Rev 1, the maximum normal condition dose rate (for an undamaged cask in the upright position) is estimated by the Applicant to be 0.00293 mrem/hr (*i.e.*, 5.85 mrem/yr ÷ 2,000 hours/yr) at the OCA boundary (~600 meters). This total dose consists of an individual neutron dose rate of approximately 0.000552 mrem/hr (~19% of total) and an individual gamma dose rate of approximately 0.00237 mrem/hr (~81% of total) at the OCA boundary. This is based on the conservative assumption that 4,000 casks are filled with design-basis fuel with a burnup and cooling time of 40,000 MWD/MTU and 10-years, respectively.

The Staff performed sensitivity calculations of the surface dose rates (gamma and neutron) from postulated reductions in hydrogen content within the HI-STORM 100 cask's radial concrete shield. The Staff performed dose rate calculations with SCALE, a state-of-the-art Monte Carlo radiation transport computer code, to compare changes in dose rates from postulated reductions in hydrogen content within the radial concrete shield. The reductions in hydrogen content were based on reference data that lists hydrogen densities in a particular type of concrete at different temperatures. In general, as radial concrete temperature increased, the hydrogen content decreased.

The Staff predicted worst-case temperatures (with the COBRA-SFS computer code) within the radial concrete shield, making the conservative assumptions that the cask rests in a horizontal position, and that all inlet and outlet vents are blocked, so as to minimize convective heat transfer through the HI-STORM annulus. Based on the temperatures predicted for regions of the radial concrete shield, and the hydrogen content data for concrete at various temperatures, the hydrogen content was conservatively reduced in layers within the SCALE shielding model of the concrete radial shield.

The Staff performed multiple dose calculations (for gamma and neutrons) for fuel with a burnup of 40,000 MWD/MTU and cooling time of 10 years (representative of PFS design-basis analysis fuel). Based on these sensitivity calculations, the peak neutron and gamma dose rate at the surface of the cask could increase by factors of 6.77 and 1.39, respectively, as a result of the assumed worst-case thermal degradation (via hydrogen loss). The total dose rate at the surface of the cask increases by a factor of 1.54 when considering the relative contributions of neutrons and gammas to the total normal condition dose rate. Assuming that all 4,000 casks tipover and experience thermal degradation in the radial shield, one could conclude, conservatively, that the off-site dose rates could increase by a factor of approximately 2.4 ([6.77 x 19%] + [1.39 x 81%]). This factor is based on the assumption that increases to off-site neutron and gamma dose rates would be approximately proportional to the respective increases to the peak neutron and gamma dose rates on the radial surface. In addition, this factor accounts for the relative contribution of neutron radiation (~19% of total) and gamma radiation (~81% of total) to off-site dose rates at the OCA boundary. A factor of 2.4 increase for a tipover of 4,000 casks would constitute only a minor increase over the normal dose rate, and would not exceed the (design basis) accident dose limit of 5 rem.

By comparison, in my evaluation, I determined that the (design basis) accident offsite dose limit of 5 rem could only be exceeded if the off-site dose rate at the OCA boundary increases to approximately 6.94 mrem/hr. This assumes that: (1) the only postulated dose to persons off-site would be from direct radiation, as the confinement system of each cask would maintain its integrity (*see* discussion of cask confinement and shielding, *supra*); and (2) a hypothetical person at the site-boundary is unshielded, stationary, and continually exposed to the accident dose rate for thirty days after the hypothetical tipover event (*i.e.,* 6.94 mrem/hr x 30 days x 24 hrs/day = 5 rem). A dose rate of 6.94 mrem/hr corresponds to an increase above the maximum normal condition off-site dose rate by a factor of approximately 2,400 (*i.e.*, 6.94 mrem/hr ÷ 0.00293 mrem/hr = 2,369).

Inasmuch as postulated thermal degradation of the concrete shields of 4,000 casks could lead to no more than a factor of 2.4 increase, it would not exceed the (design basis) accident dose limit.

Q21. Please describe the Staff's evaluation, with respect to the third issue identified in response to Question 18 above, *i.e.,* spacial reorientation of the casks from a vertical to a tilted or horizontal position.

A21. In my analysis, I considered the extent to which dose rates might increase as a result of spacial reorientation of the casks, from a vertical to a tilted or horizontal position. In this regard, as discussed above, I considered the amount of shielding at the top, side, and bottom of the casks that may face off-site and affect off-site dose rates.

Further, based on a review of the "Radiation Shielding Analysis for the PFS," Holtec Report No. HI-971645 (March 14, 2001), and the storage configuration of the 4,000 casks depicted in PFS SAR Figure 1.2-1 (Staff Exhibit __), it is evident that the sides of the casks located along the outer periphery of the storage pads (*i.e.*, the casks which directly face the OCA boundary) are the dominant contributor (~99% of total) to off-site dose rates during normal upright conditions. The tops of all the casks in the entire storage array contribute only a minor amount (~1% of total) to off-site dose rates through skyshine interactions in the atmosphere. The bottoms of the casks do not contribute to off-site dose rates in their normal (upright) position. Also, the sides of the casks that are located on interior storage pads (inside the outer periphery) do not significantly contribute to off-site dose rates because they are shielded by the closely-spaced casks (*i.e.,* 15 to 16 feet cask-to-cask pitch) on the outside periphery of the storage pads. Even if the casks tipped over, the same would be true: Either the top, side, or bottom of tipped casks on the outer periphery of the storage pads would be the dominant contributor to off-site dose rates, in the direction faced by the cask surface.

Also, if the top or bottom of any particular horizontal or tilted cask (located along the outer periphery) increased the off-site dose rates in a particular direction (*e.g.*, the north OCA boundary), the contribution from the side of the cask that had directly faced the OCA boundary under normal upright cask conditions (*i.e.*, prior to reorientation) would obviously decrease. As a result, one would not expect to see a significant increase (relative to a factor of 2,400) in off-site dose rates at any point of the OCA boundary, as a result of the tops or bottoms of multiple casks directly facing off-site on the outer periphery of the storage pads (as compared to the normal condition in which when the sides of these casks, which are the dominant dose contributors during normal upright cask conditions, face off-site).

This conclusion is further supported by an analysis I conducted of the worst-case changes to off-site dose rates, from examination of the peak one-meter dose rates for the tops, sides, and bottoms of the casks and the normal condition off-site dose rates. Based on Holtec Report No. HI-971645, the total peak dose rate at one meter from the top of the cask is 0.87 mrem/hr, consisting of an individual neutron dose rate of 0.62 mrem/hr (~71% of total) and individual gamma dose rate of 0.25 mrem/hr (~29% of total). The total peak dose rate at one meter from the side of the cask is 5.01 mrem/hr, consisting of an individual neutron dose rate of 0.42 mrem/hr (~8% of total) and individual gamma dose rate of 4.59 mrem/hr (~92% of total).

Calculations of the maximum dose rates from the <u>bottom</u> of the HI-STORM cask, at one meter, are not available because the bottom of the cask is normally face-down and does not contribute to off-site doses during normal operations or design-basis accidents. However, the maximum one meter dose rate would be bounded by a consideration of the calculated dose rates at one meter from the bottom of the 125-ton HI-TRAC transfer cask with the pool lid, as shown in Table 5.1.8 of the HI-STORM FSAR.

The shielding configuration at the bottom of the HI-STORM 100 storage cask provides superior attenuation ability as compared to the shielding configuration at the bottom of the HI-TRAC transfer cask. On the bottom of the HI-STORM overpack, there are approximately 7 inches of steel to attenuate gamma radiation and 17 inches of concrete to attenuate both neutron and gamma radiation. By comparison, on the bottom of the HI-TRAC cask with a pool lid, there are approximately 3 inches of steel and 2.5 inches of lead to attenuate gamma radiation, with no concrete or other material to attenuate neutron radiation. In addition, the HI-TRAC dose value is based on "hotter" fuel with a 45,000 MWD/MTU burnup and 9-years cooling time, which bounds the design basis PFS fuel with a 40,000 MWD/MTU burnup and 10-years cooling time.

The bottom of the HI-TRAC transfer cask, at one meter, results in a dose rate of 292 mrem/hr, consisting of an individual neutron dose rate of 167 mrem/hr (~57% of total) and an individual gamma dose rate of 125 mrem/hr (~43% of total). Because this dose rate would exceed the expected dose rate from the bottom of the HI-STORM cask (with its superior shielding), this dose rate may be considered to bound the dose rate at one meter from the bottom of the HI-STORM cask. However, I consider this value to be very conservative for the HI-STORM cask bottom, because of the substantially superior shielding offered by the bottom design of the HI-STORM cask as compared to that of the HI-TRAC transfer cask.

During normal upright conditions, the sides of the casks on the outside periphery of storage pads face off-site toward all sides of the OCA boundary. The normal condition peak neutron and gamma dose rates at one-meter from the side of each cask is 0.42 mrem/hr and 4.59 mrem/hr, respectively. This radiation diverges and is attenuated before it reaches the OCA boundary, leading to bounding OCA boundary neutron and gamma dose rates of 0.000552 mrem/hr and 0.00237 mrem/hr, respectively.

Three hypothetical conditions may be postulated, in which up to 4,000 casks tipover at the proposed PFS Facility, with resulting dose rate effects as follows:

(1) If all casks on the outer periphery tipped over, with their tops facing off-site in the north direction, the off-site dose rates could decrease approximately by a factor of 14.6 (*i.e.*, [(4.59 mrem/hr ÷ 0.25 mrem/hr) x 81%] - [(0.62 mrem/hr ÷ 0.42 mrem/hr) x 19%]).

(2) If all casks on the outer periphery tipped over, with their bottoms facing off-site in the north direction, the off-site dose rates could, conservatively speaking, increase by a factor of 97.6 (*i.e.*, [(125 mrem/hr ÷ 4.59 mrem/hr) x 81%] + [(167 mrem/hr ÷ 0.42 mrem/hr) x 19%]).

(3) If all casks on the outer periphery tipped over, with their sides facing off-site in the north direction, the off-site dose rates would remain essentially the same as during normal upright conditions (i.e., a factor of 1.0).

Therefore, if all casks on the outer periphery of the storage pad area tipped over, or tilted, with the tops, bottoms, and sides facing off-site, the change to the off-site dose rate could range from a decrease by a factor of 14.6, to an increase by a factor of 97.6. These factors are based on the assumption that increases to off-site neutron and gamma dose rates would be approximately proportional to the respective increases to the peak neutron and gamma dose rates at one meter from the top or bottom, as compared to the dose rate at one meter from the side of the cask. Also, these factors account for the relative contribution of neutron radiation (~19% of total) and gamma radiation (~81% of total) to off-site dose rates at the OCA boundary during normal conditions. Finally, it should be noted that this result would not be substantially different if all 4,000 casks tipover, in that the casks which are not on the outer periphery of the storage pads would not significantly contribute to the resulting offsite dose rate.

Based on the worst-case results shown above for all three hypothetical conditions, the off-site dose rates could increase by a factor of 97.6. However, this predicted factor of 97.6 is well below the factor of 2,400 increase which is needed to exceed an offsite dose of 5 rem, as discussed above.

Q22. Based on your considerations and analyses of the effects on off-site dose rates resulting from cask impact damage, cask thermal degradation, and cask spacial reorientation, as discussed above, what is your overall conclusion concerning potential changes in off-site dose rates that might occur in the event that any or all of the 4,000 casks at the proposed PFS Facility were to tipover?

A22. If there is hypothetical tipover of multiple (*i.e.,* up to 4,000) casks with impact damage to the shield, thermal degradation of the radial concrete shield, and/or cask reorientation, off-site dose rates would not increase by more than a conservative factor of 97.6. Therefore, the (design basis) accident dose limit of 5 rem in 10 C.F.R. § 72.106(b) would not be exceeded.

Q24. Does this conclude your testimony?

A24. Yes.

Michael D. Waters

EDUCATION

M.S. Nuclear Engineering Sciences, University of Florida, 1995
B.S. Nuclear Engineering, University of Florida, 1993

WORK EXPERIENCE

U.S. NUCLEAR REGULATORY COMMISSION

March 2002 to present Health Physicist, NMSS, Spent Fuel Project Office (SFPO)

Review the adequacy of spent fuel storage casks, storage facilities, and transportation package designs to provide radiological protection to public and the workers, with a focus on the major technical disciplines of shielding, criticality, containment, and dispersion analysis. Perform detailed technical reviews and conducted independent confirmatory analyses with state-of-the-art methods to determine compliance of proposed designs with federal safety requirements in 10 CFR Parts 20, 71, and/or 72. Prepare written safety evaluations reports, federal rulemakings, and environmental assessments for assigned projects. Develop NRC policy on health physics issues related to storage casks and storage facilities.

May 1996 to February 2002 Project Engineer, NMSS, SFPO

Project Manager (PM): Scheduled, coordinated, and prepared licenses, amendments, and approval certificates for several spent fuel storage installations, spent fuel storage and transportation cask designs, and transportation package designs. Coordinated multi-disciplined technical review teams to make regulatory findings on the adequacy of proposed designs. Developed NRC policy on assigned technical and licensing issues.

Technical Reviewer: Reviewed the adequacy of several spent fuel storage cask, storage facility, and transportation package designs primarily in the major technical disciplines of shielding, criticality, containment, radiological protection, and operating and maintenance procedures. Performed detailed technical reviews and conducted independent confirmatory analyses with state-of-the-art methods to determine compliance of proposed designs with federal safety requirements in 10 CFR Parts 20, 71, and/or 72. Prepared written safety evaluations reports, federal rulemakings, and environmental assessments for assigned projects.

Major NRC Casework and Publications:

SFPO Project Manager (backup) for the Private Fuel Storage Facility (PFSF) license application review. Review team member for the PFSF environmental impact review.

SFPO Project Manager for the Fort St. Vrain independent spent fuel storage installation (ISFSI), Three Mile Island Unit 2 ISFSI, SPEC-300 transportation package.

Primary shielding and radiological safety reviewer of the HI-STAR 100 transportation cask design (Part 71), HI-STAR 100 storage cask design (Part 72), and HI-STORM 100 storage cask design

(Part 72). Primary shielding, radiological safety, criticality, and/or containment reviewer of multiple other storage and transportation cask designs (Part 71 and 72).

Co-author of NUREG-1571, *"Information Handbook on Independent Spent Fuel Storage Installations,"* December 1996. Primary author of *"Reconsideration of Dose Assessments for Future Independent Spent Fuel Storage Installation Multi-Row Cask Arrays,"* Sixth International Conference on Nuclear Engineering,1998.

UNIVERSITY OF FLORIDA

Jan 1993 - April 1996 Research Assistant/Graduate Student, Department of Nuclear Engineering Sciences

Responsibilities: Developed computer codes to analyze and evaluate the characteristics of industrial fuel designs and performed cost-benefit analyses to determine optimum fuel designs. Investigated the University's hazardous mixed waste problem, interviewed research laboratory personnel, identified root causes of waste generation, and determined inexpensive methods to mitigate waste.

OAK RIDGE INSTITUTE FOR SCIENCE AND EDUCATION

May 1993 - August 1993 Professional Intern, Oak Ridge National Laboratory
May 1992 - August 1992

Responsibilities: Performed various studies with the RELAP-5 thermal hydraulic code and commercial plotting software. Developed experiments and standard procedures, as part of a program to confirm criticality design features of the reactor spent fuel storage racks.

NRC AWARDS

NRC Special Act Award - August 1, 2001
NRC Performance Award - July 18, 2000
NRC Performance Award - March 18, 1999