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UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	)	
	)	
PRIVATE FUEL STORAGE, L.L.C.	)	Docket No. 72-22-ISFSI
	)	
(Independent Spent	)	
Fuel Storage Installation	)	

NRC STAFF'S FINDINGS OF FACT AND
CONCLUSIONS OF LAW CONCERNING UNIFIED
CONTENTION UTAH L/QQ (GEOTECHNICAL ISSUES)

Sherwin E. Turk
Robert M. Weisman
Counsel for NRC Staff

September 5, 2002

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I. INTRODUCTION

1.1 These findings and rulings address all outstanding issues with respect to Unified Contention Utah L/QQ (Geotechnical Issues) concerning the application filed on June 20, 1997, by Private Fuel Storage, L.L.C. ("PFS" or "Applicant"), for a license under 10 C.F.R. Part 72 to possess spent fuel and other radioactive materials associated with spent fuel storage in an away from reactor independent spent fuel storage installation ("ISFSI"), which PFS proposed to construct and operate on the Reservation of the Skull Valley Band of Goshute Indians in Skull Valley, Utah.

1.2. Notice of the Nuclear Regulatory Commission ("NRC")'s receipt and consideration of the PFS ISFSI license application was published in the Federal Register on July 31, 1997. 62 Fed. Reg. 41,099 (1997). If granted, the license would authorize PFS to store up to 40,000 metric tons uranium (MTUs) of spent nuclear fuel ("SNF") in dry cask storage systems at its proposed ISFSI. The Notice advised the Applicant and any person whose interest may be affected by the proceeding of their right to request a hearing by filing such a request and a petition for leave to intervene. In response to the Notice, petitions for leave to intervene were timely filed by various parties and entities, including the State of Utah.[1]

1.3. On September 15, 1997, the Atomic Safety and Licensing Board was established to rule on petitions for hearing and for leave to intervene and to preside over any adjudicatory proceeding that might be held in connection with the ISFSI application. 62 Fed. Reg. 49,263 (1997).[2] The petitioners then timely filed approximately 100 (initial) contentions which they sought to litigate in this proceeding.

1.4. On April 22, 1998, the Licensing Board issued its "Memorandum and Order (Rulings on Standing, Contentions, Rule Waiver Petition, and Procedural/Administrative Matters)," in which the Board determined, among other things, that the State of Utah and four other petitioners had demonstrated their standing to intervene in this matter,[3] and that many of their contentions, in whole or in part, satisfied the Commission's requirements for admission as contested issues in this proceeding. *Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), LBP-98-7, 47 NRC 142, 169-72, 183-238, 251-58 (1998).

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[1] Petitions to intervene were filed initially by: (1) the State of Utah ("State"); (2) Ohngo Gaudadeh Devia ("OGD"); (3) Castle Rock Land and Livestock, L.C., Skull Valley Company, Ltd. (collectively referred to as "Castle Rock"); and Ensign Ranches of Utah, L.C.; (4) the Confederated Tribes of the Goshute Reservation ("Confederated Tribes") and David Pete; and (5) the Skull Valley Band of Goshute Indians ("Skull Valley Band").

[2] The Licensing Board was reconstituted twice during the course of the proceeding. *See* 62 Fed. Reg. 52,364 (1997); 66 Fed. Reg. 67335 (2001).

[3] The Licensing Board granted the petitions of the State of Utah, Castle Rock, OGD, the Confederated Tribes, and the Skull Valley Band; but it denied the petitions of David Pete, Ensign Ranches of Utah, L.C., and Scientists for Secure Waste Storage ("SSWS"). LBP-98-7, 47 NRC at 147. Castle Rock later withdrew from the proceeding. *See Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), LBP-99-6, 49 NRC 114 (1999). The Licensing Board subsequently granted a late-filed petition filed by the Southern Utah Wilderness Alliance ("SUWA"). *See Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), LBP-99-3, 49 NRC 40, *aff'd,* CLI-99-10, 49 NRC 318 (1999).

1.5. Among the contentions admitted in the Licensing Board's ruling was Contention Utah L, which raised certain geotechnical issues associated with the PFS site. Contention Utah L asserted as follows:

> **Utah L - Geotechnical**
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> CONTENTION: The Applicant has not demonstrated the suitability of the proposed ISFSI site because the License Application and SAR [Safety analysis Report] do not adequately address the site and subsurface investigations necessary to determine geologic conditions, potential seismicity, ground motion, soil stability and foundation loading.

LBP-98-7, 47 NRC at 191, 253. Four bases were provided in support of this contention, concerning (1) surface faulting, (2) ground motion, (3) characterization of subsurface soils (including subsurface investigations, sampling and analysis, and physical property testing for engineering analysis), and (4) soil stability and foundation loading.[4] At the same time, the Board excluded Late-Filed Contention Utah EE, concerning the seismic stability of the HI-STORM 100 storage casks and storage pads which PFS proposes to use at its facility. *See id.* at 206-09.[5]

1.6. One year later, on April 2, 1999, PFS filed a request for exemption, pursuant to 10 C.F.R. § 72.7,[6] from certain of the Commission's regulations pertaining to the seismic design of an ISFSI set forth in 10 C.F.R. § 72.102(f)(1). Specifically, PFS requested that it be granted an

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[4] "State of Utah's Contentions on the Construction and Operating License Application by Private fuel Storage, LLC for an Independent Spent Fuel Storage Facility" ("Utah Contentions"), dated November 23, 1997, at 80-95. *See Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), LBP-01-39, 54 NRC 497, 501 (2001).

[5] The Board admitted portions of another contention, Late-Filed Contention Utah GG, concerning cask-pad stability for another cask system. *See* LBP-98-7, 47 NRC at 210-11. Subsequently, the vendor of that cask system withdrew its request for NRC review, and PFS then removed the references to that system from its license application, thus mooting that contention.

[6] In accordance with 10 C.F.R. § 72.7, the Commission may grant exemptions from the specific requirements in 10 C.F.R. Part 72, where it determines that the exemptions "are authorized by law and will not endanger life or property or the common defense and security and are otherwise in the public interest."

exemption to allow it to utilize probabilistic seismic hazard analysis ("PSHA") methodology and consideration of risk to establish the design earthquake ground motion levels at its facility, in lieu of the deterministic approach required by 10 C.F.R. § 72.102(f) and 10 C.F.R. Part 100, Appendix A; in addition, the exemption request proposed that the facility's seismic design be based upon the ground motions produced by a 1,000-year return period earthquake (PFS Exh. 247). On August 24, 1999, PFS revised its exemption request, to substitute a 2,000-year return period ground motion as its seismic design basis, in lieu of the 1,000 year ground motion it had proposed earlier (PFS Exh. 248).

1.7 Following the submittal of the Applicant's seismic exemption request, the State filed two requests to modify Contention Utah L to challenge the exemption request, in motions dated April 30, 1999, and January 26, 2000, respectively. The Licensing Board denied these requests on the grounds that the issue was not yet ripe, prior to any action by the NRC Staff ("Staff") on the exemption request.[7]

1.8. Evidentiary hearings were held in this proceeding in June 2000, concerning three issues (financial assurance, decommissioning funding and emergency planning) that are not pertinent to this decision. Those matters, as well as other matters addressed at the April - July 2002 evidentiary hearings, are not discussed herein, but have been or will be the subject of other Licensing Board decisions in this proceeding.

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[7] *See Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), LBP-99-21, 49 NRC 431, 437-38 (1999), and *Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), LBP-00-15, 51 NRC 313, 318 (2000). The Licensing Board subsequently denied a further contention filed by the State, concerning a possible co-seismic rupture of certain faults in the vicinity of the proposed site. *See Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), LBP-00-16, 51 NRC 320 (2000).

1.9. On September 29, 2000, the Staff issued its "Safety Evaluation Report Concerning the Private Fuel Storage Facility" ("SER").[8] Therein, the Staff concluded, *inter alia*, that the Applicant's use of a PSHA with a 2,000-year return period is acceptable for the PFS facility.[9] *See* Staff Exh. C, § 2.1.6.2, at 2-51. On November 9, 2000, the State filed a third motion to amend Contention Utah L to challenge the Applicant's exemption request, and the Staff's bases for deciding to approve that request as set forth in the Staff's SER.[10]

1.10 In late December 2000, while the State's request to challenge the Applicant's exemption request was pending before the Board, PFS notified the Board and parties that new developments had led it to revise its seismic design and stability analyses to account for higher design ground motions than it had previously calculated.[11] As discussed *infra*, these developments would lead the Applicant to revise its seismic design basis from a peak ground acceleration ("PGA") of 0.53g to a PGA of 0.711g horizontal and 0.695g vertical. Staff Exh. C, at 2-48.

1.11. On January 31, 2001, the Licensing Board ruled on the State's request to amend Contention Utah L, finding most of the bases offered in support thereof to be admissible; and, consistent with 10 C.F.R. §§ 2.718(i) and 2.730(f), the Board referred its ruling to the Commission

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[8] The Staff's SER replaced the Staff's Preliminary Safety Evaluation Report ("Preliminary SER"), issued on December 15, 1999 (revised and reissued on January 4, 2000). The Preliminary SER was introduced into evidence during evidentiary hearings in June 2000, as Staff Exh. A.

[9] The Staff's SER was later amended and supplemented, upon the Staff's issuance of SER Supplement Nos. 1 and 2, on November 13 and December 21, 2001, respectively. The SER and the two SER Supplements were then integrated into a "Consolidated Safety Evaluation Report," which was admitted into evidence as Staff Exh. C.

[10] *See* "State of Utah's Request for Admission of Late-Filed Modification to Basis 2 of Contention Utah L," dated November 9, 2000.

[11] *See* (1) Letter from Jay E. Silberg, Esq. to the Licensing Board, dated December 28, 2000, enclosing letter from John D. Parkyn (Chairman, PFS) to Mark Delligatti (NRC), dated December 22, 2000; (2) letter from Sherwin E. Turk, Esq., to the Licensing Board, dated January 23, 2001 (enclosing letter from E. William Brach (Director, NRC Spent Fuel Project Office) to John D. Parkyn, dated January 19, 2001; and (3) letter from John D. Parkyn to NRC Document Control Desk, dated March 30, 2001 (License Application Amendment No. 22).

with a certified question as to whether the State's challenge to the exemption request is litigable in this proceeding. *Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), LBP-01-3, 53 NRC 84, 100-01 (2001).

1.12 On June 14, 2001, the Commission affirmed the Licensing Board's decision, finding that the State's proposed modification of Contention Utah L to challenge the Applicant's exemption request was admissible and may be litigated in this proceeding, in that the State's request raised a material issue that was directly related to the PFS license application pending before the agency. *Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), CLI-01-12, 53 NRC 459 (2001). The Licensing Board then admitted this issue, along with six stated bases, as "Part B" of Contention Utah L.[12]

1.13. Meanwhile, proceedings continued with respect to other portions of the State's geotechnical and seismic challenges to the PFS application, including extensive discovery conducted by the parties. On December 30, 2000, PFS moved for summary disposition of previously admitted portions of Contention Utah L (herein described as "Part A" of the contention),[13] to which the Staff and State responded on January 30, 2001. On February 9, 2001, PFS moved to strike certain portions of the State's response to its summary disposition motion, based on its assertion that it included matters which were beyond the scope of Contention Utah L;[14] in response, the State argued, *inter alia*, that the matters were within the scope of Contention Utah L, and that

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[12] *See* "Memorandum and Order (Requesting Joint Scheduling Report and Delineating Contention Utah L"), dated June 15, 2001.

[13] *See* "Applicant's Motion for Summary Disposition of Contention Utah L," dated December 30, 2000, and "Correction to Applicant's Motion for Summary Disposition of Contention Utah L," dated January 2, 2001.

[14] "Applicant's Motion to Strike Portions of State of Utah's Response to Applicant's Motion for Summary Disposition of Utah Contention L," dated February 9, 2001.

this was not a new position, as shown by evidenced by its discussion of these matters in its responses to PFS's discovery requests.[15]

1.14. While these threshold "scope of contention" questions were pending, the State raised additional seismic design issues. On May 16, 2001, the State submitted late-filed Contention Utah QQ, in which it challenged the application of PFS's newly revised design basis ground motions to the seismic design of its Canister Transfer Building ("CTB") and reinforced concrete storage pads and their foundations; PFS's intended use and redesign of soil cement around the CTB and under and around the storage pads; and the foundation design of the CTB, storage pads, and underlying soils, and the stability of the storage casks, to safely withstand the newly revised design basis ground motions.[16] Further, by motions dated June 19 and August 23, respectively, the State proposed two modifications of late-filed Contention Utah QQ, based upon its recent receipt of revised calculations from the Applicant concerning its seismic design.[17]

1.15. On December 19, 2001, the Licensing Board in this proceeding was reconstituted, and Administrative Judge Michael C. Farrar was appointed to serve as Board Chairman in place of Chief Administrative Judge G. Paul Bollwerk, except with respect to certain retained issues.[18] Shortly thereafter, this reconstituted Licensing Board ruled upon all pending motions concerning the litigation of Contention Utah L and Late-Filed Contention Utah QQ (including PFS's motion for

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[15] "State of Utah's Response to Applicant's Motion to Strike Portions of State of Utah's Response to Applicant's Motion for Summary Disposition of Utah Contention L," dated February 20, 2001, at 4-9.

[16] *See* "State of Utah's Request for Admission of Late-Filed Contention Utah QQ (Seismic Stability)," dated May 16, 2001, at 1.

[17] *See* "State of Utah's Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to Further Revised Calculations From the Applicant," dated June 19, 2001; and "State of Utah's Second Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to More Revised Calculations From the Applicant," dated August 23, 2001.

[18] "Notice of Reconstitution," dated December 19, 2001.

summary disposition of Contention Utah L, its motion to strike portions of the State's response thereto, the admissibility of Contention Utah QQ and the two proposed modifications thereof, and certain pending discovery motions). *Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), LBP-01-39, 54 NRC 497 (2001). In sum, the Board's ruling denied the Applicant's motion for summary disposition and its motion to strike, and admitted the State's late filed issues in Contention Utah QQ -- finding, *inter alia*, that the State had shown a genuine dispute of material fact with respect to the motion for summary disposition, that it had timely filed an admissible contention, and that the issues should proceed to hearing. *Id.* at 512-21.

1.16. On January 9, 2002, the Board denied a further PFS motion for summary disposition filed on November 9, 2001,[19] with respect to "Part B" of Contention Utah L (concerning PFS's seismic exemption request). *See Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), LBP-02-1, 55 NRC 11 (2002).

1.17. As directed by the Licensing Board, the parties then proposed a unified contention that incorporates all remaining seismic/geotechnical issues in the proceeding,[20] as Unified Contention Utah L/QQ. As formulated, Unified Contention Utah L/QQ asserts as follows:

> **Unified Consolidated Contention Utah L/QQ (Geotechnical)**
> The Applicant has not demonstrated the suitability of the proposed independent spent fuel storage installation (ISFSI) site because the License Application and the Safety Analysis Report do not adequately address site and subsurface investigations necessary to determine geologic conditions, potential seismicity, ground motion, soil stability and foundation loading; and
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> The Applicant's site specific investigations, laboratory analyses, characterization of seismic loading, and design calculations, including redesign of cement-treated soil (or soil cement) fail to demonstrate that a) the newly revised probabilistic seismic hazard design basis ground motions have been adequately and consistently applied to the Canister Transfer

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[19] "Applicant's Motion for Summary Disposition of Part B of Utah Contention L," dated November 9, 2001.

[20] *See* "Joint Submittal of Unified Geotechnical Contention, Utah L and Utah QQ," dated January 16, 2002. *See also* LBP-01-39, 54 NRC at 521; LBP-02-1, 55 NRC at 18.

> Building ("CTB"), storage pads, and their foundations; b) PFS's general design approach, including the redesign of soil cement, for the CTB, storage pads, or storage casks can safely withstand the effects of earthquakes; and c) the foundation design of the CTB, storage pads, and the underlying soils, or the stability of the storage casks, are adequate to safely withstand the newly revised probabilistic seismic hazard design basis ground motions (10 CFR §§ 72.102(c), (d); 72.122(b)), *in that* . . . .

The contention then identified five specific areas in which the Applicant's efforts were asserted to be deficient, as follows:

A. Surface Faulting;

B. Ground Motion;

C. Characterization of Subsurface Soils;

D. Seismic Design and Foundation Stability; and

E. Seismic Exemption.[21]

1.18. On January 31, 2002, the parties informed the Board that Parts A (Surface Faulting) and B (Ground Motion) of Unified Contention L/QQ were no longer in dispute.[22] *See* discussion *infra*, at 22. As a result, three portions of the contention (Parts C, D and E) remained to be addressed in evidentiary hearings.

1.19. In accordance with a notice of hearing published in the *Federal Register,*[23] evidentiary hearings with respect to Parts C, D and E of the contention were held in Salt Lake City, UT, on April 29-May 17, June 3-8, and June 17-27, 2002. Numerous witnesses appeared on behalf

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[21] Each of these parts of the contention is set forth in full, in the discussion *infra.*

[22] "Joint Stipulation of Facts and Issues Not in Dispute With Respect to Unified Contention Utah L/QQ (Geotechnical)," dated January 31, 2002.

[23] *See* "Notice of Evidentiary Hearing and of Opportunity to Make Limited Appearance Statements," 67 Fed. Reg. 10,448 (March 7, 2002). Evidentiary hearings were also held in 2002 concerning Contentions Utah K/ Confederated Tribes B (Credible Offsite Accidents), Contention Utah O (Hydrology), SUWA Contention B (Low Rail Line Alternatives). Contention Utah O was settled by the parties; the other two contentions are addressed in other sets of proposed findings of fact and conclusions of law, and are not addressed herein.

of PFS, the Staff and the State during the 28 days of hearing on this contention, as summarized below.

1.20. These proposed findings of fact and conclusions of law present the Licensing Board's findings of fact with respect to the evidence presented at the 2002 hearings concerning Unified Contention Utah L/QQ, and the Board's conclusions of law with respect thereto.

II. FINDINGS OF FACT

A. Factual Background.

The Proposed PFS Facility

2.1. PFS proposes to construct and operate a dry cask storage ISFSI in which up to 4,000 steel and concrete casks, each containing 10 metric tons of spent nuclear fuel, would be placed on reinforced concrete storage pads at its proposed site. Under PFS's proposal, up to eight loaded storage casks would be placed on each pad, which in turn would be arranged in a 25 x 20 array (*i.e.,* up to 500 pads) occupying approximately 99 acres. Each pad would be constructed of reinforced concrete, and would be 30 ft wide, 67 ft long and 3 ft thick. Staff Exh. C at 1-1, 1-2, and 5-8; Staff Exh. X.

2.2. In accordance with 10 C.F.R. § 72.42, the PFS Facility would be initially licensed for 20 years. Before the end of this 20-year term, PFS could submit an application to renew the license. If granted, all spent fuel will be transferred offsite and the Facility will be ready for decommissioning by the end of the second term. *Id.* at 1-1.

2.3. The proposed PFS site is located in the northwest corner of the Reservation of the Skull Valley Band of Goshute Indians, and will cover 820 acres of the Reservation's 18,000 acres. The Reservation is geographically located in Tooele County, Utah, 27 miles west-southwest of Tooele City, Utah, about 50 miles southwest of Salt Lake City, Utah, and 14 miles north of the entrance to the Dugway Proving Ground in Tooele County, Utah. *Id.* at 1-1, 2-3. No large towns are located within 10 miles of the proposed site. The Skull Valley Band of Goshute Indians' Village, which has about 30 residents, is 3.5 miles east-southeast of the site. *Id.* at 1-1. The nearest residence is located about two miles from the site. Approximately 36 persons live within five miles of the site. No transient or institutional populations are present within five miles of the site, and no public facilities are expected to be located in the vicinity. *Id.* at 2-4.

2.4. Interstate Highway 80 and the Union Pacific Railroad main line are approximately 24 miles north of the site. Shipping casks approved under 10 C.F.R. Part 71 will be used to transport the spent nuclear fuel to the Facility. The shipping casks will either be off-loaded at an intermodal transfer point near Timpie, Utah, and loaded onto a heavy haul tractor/trailer for transporting to the Facility, or transported via a new railroad line connecting the Facility directly to the Union Pacific main line. The Facility will be accessed by a new road from the Skull Valley Road as shown in Figure 1.2 of the Staff's Final Environmental Impact Statement ("FEIS"). *Id.*; Staff Exh. E. at 1-3.

2.5. The Facility is designed to store up to 40,000 metric tons of uranium (MTU) in the form of spent fuel from commercial nuclear power plants in sealed metal canisters. The spent fuel assemblies are placed in sealed canisters, which are then placed inside a steel and concrete storage cask. The ISFSI, consisting of approximately 4,000 storage casks, is passive and does not rely on active cooling systems. Staff Exh. C at 1-1.

2.6. The Facility's restricted area is approximately 99 acres surrounded by a chain link security fence and an outer chain link nuisance fence. An isolation zone and intrusion detection system are located between the two fences. The cask storage area that surrounds the concrete cask storage pads that support the storage casks is surfaced with compacted gravel that slopes slightly to allow for runoff of storm water. Each concrete pad supports up to eight storage casks in a 2 × 4 array. The Canister Transfer Building ("CTB"), where canisters are transferred from the shipping cask to the storage cask, is located within the restricted area. An overhead bridge crane and a semi-gantry crane are located within the Canister Transfer Building to facilitate shipping cask loading/unloading operations and canister transfer operations. *Id.* at 1-1 to 1-2.

2.7. The dry cask storage system that has been identified for use at the Facility is the HI-STORM 100 Cask System, designed by Holtec International, Inc. ("Holtec"). The cask system is a canister-based storage system that stores spent fuel in a vertical orientation. It consists of three discrete components: the multi-purpose canister ("MPC"), the HI-TRAC transfer cask, and the HI-STORM 100 storage overpack. The MPC is the confinement system for the stored fuel. The HI-TRAC transfer cask provides radiation shielding and structural protection of the MPC during transfer operations. The storage overpack provides radiation shielding and structural protection of the MPC during storage. The cask system stores up to 24 pressurized water reactor (PWR) fuel assemblies or 68 boiling water reactor (BWR) fuel assemblies. The HI-STORM 100 Cask System is passive and does not rely on any active cooling systems to remove spent fuel decay heat. *Id.* at 1-2.

2.8. The spent fuel is loaded into the MPCs at the originating nuclear power plant. Before transport, the MPC's lid is welded in place and the canister is drained, vacuum dried, filled with an inert gas, sealed, and leak tested. Shipping casks that are approved under 10 C.F.R. Part 71 (*e.g.,* the HI-STAR 100 transportation cask) are used to transport the MPCs from the originating power plants to the Facility. At the Facility, the shipping cask is lifted off the transport vehicle and placed in a shielded area of the Canister Transfer Building, called a transfer cell. The MPC is transferred from the shipping cask to the transfer cask, then from the transfer cask into the storage cask. The storage cask, loaded with the MPC, is then closed, and moved to the storage area using a cask transporter and placed on a concrete pad in a vertical orientation. *Id.*

2.9. The HI-STORM 100 storage cask is approximately 20 feet tall (239.5 inches) and about 11 feet in diameter (132.5 inches); when loaded with a spent fuel canister, it will weigh approximately 180 tons. The steel and concrete cylindrical walls of the cask form a heavy steel weldment, consisting of an inner and outer steel shell within which the shielding concrete is installed; these walls in the radial direction are approximately 30 inches thick. The cask has four air inlets at the bottom and four air outlets at the top, to allow air to circulate naturally through the annular cavity to cool the MPC within the storage cask. *See* "Testimony of Krishna P. Singh and Alan I. Soler on Unified Contention Utah L/QQ" (hereinafter referred to as "Singh/Soler"), Post Tr. 5750, at 7; Staff Exh. W. Further details concerning the construction of the HI-STORM 100 cask system are provided *infra*, in our findings of fact concerning Part D of this contention.

2.10. The HI-STORM 100 Cask System has been approved by the NRC for use under the general license provisions of 10 C.F.R. Part 72, Subpart K. The HI-STORM 100 Cask System is approved under Certificate of Compliance No. 1014, effective May 31, 2000 (Docket No. 72-1014) (Staff Exh. FF). The Staff's evaluation of the cask system for general use is documented in the NRC's safety evaluation report for the HI-STORM 100 Cask System, issued with the certificate of compliance. *Id.*; *see* Staff Exh. FF.

2.11. Notwithstanding the NRC's issuance of a general certificate of compliance for the HI-STORM 100 Cask System, PFS evaluated the cask system against the parameters and conditions specific to the PFS Facility, in order to establish the acceptability of that system for site-specific use at the PFS Facility. Based on the Applicant's evaluation and its own evaluation (discussed in its SER), the Staff has found that the HI-STORM 100 Cask System is acceptable for use at the PFS Facility under the site-specific license provisions of 10 C.F.R. Part 72. Staff Exh. C at 1-2, 1-3.

2.12. PFS has identified certain organizations as responsible for providing the licensed spent fuel storage and transfer systems and engineering, design, licensing, and operation of the Facility -- certain officials or employees of which testified in this proceeding. Holtec International is responsible for the design of the HI-STORM 100 Cask System. Stone & Webster Engineering Corporation ("Stone & Webster") is responsible for the design of the Facility. PFS has overall responsibility for planning and design of the Facility, using Stone & Webster as a contractor. PFS is also responsible for the operation of the Facility and for providing quality assurance (QA) services. *Id.* at 1-3.

The Geologic Setting of the Proposed Site

2.13. The proposed site is located on a typical valley floor of the local Basin and Range topography. Skull Valley is a north-trending valley that extends from the Onaqui Mountains to the southwest shore of the Great Salt Lake. The Stansbury Mountains lie to the east of the site and separate the site from Tooele City, Utah, about 27 miles to the northeast. The Cedar Mountains are approximately 14 miles to the west and separate the Facility from portions of the Utah Test and Training Range within the Great Salt Lake Desert. Skull Valley, Utah, has little population and limited agriculture, although a cattle ranch is located on the north border of the Facility. *Id.* at 2-3, 2-22.

2.14. As summarized in the Applicant's Safety Analysis Report ("SAR"), the proposed site is located in the northeastern margin of the Basin and Range Province, a wide zone of active extension and distributed normal faulting that extends from the Wasatch Front in central Utah to the Sierra Nevada Mountains in western Nevada and eastern California. Topography within the Basin and Range Province reflects Miocene to recent, east-west extensional faulting, in which tilted and exhumed footwall blocks form subparallel north-south striking ranges separating elongated and internally drained basins. Ranges are up to several hundred kilometers long with elevations up to 6,500 feet above the basin floors. Much of the surface faulting took place at the base of the ranges along normal faults that dip moderately (~60°) beneath the adjacent basins (herein defined as range-front faults), although complex faulting within the basins is also common [i.e., the fault-rupture patterns of the 1954 Rainbow Mountain-Stillwater or 1959 Hebgen Lake earthquakes. *Id.* at 2-28.

2.15. The proposed site in Skull Valley lies in one of the typical basins of the province, bounded on the east by the Stansbury Mountains and the Stansbury fault and on the west and south by the Cedar Mountains and the East Cedar Mountain fault. The basin is underlain by late Quaternary lacustrine deposits laid down from repeated flooding of the valley during transgressions of intermontane lakes, most notably Lake Bonneville, which flooded Skull Valley several times during the Pleistocene and Holocene. These deposits form the basis for paleoseismic evaluations of the Skull Valley site. Topography of the proposed site is relatively smooth, reflecting the origin of the valley floor as the bottom of Lake Bonneville. The site gently slopes to the north with a slope of less than 0.1°. Detailed topographic maps of the region and the site were provided in the SAR. This smooth valley floor contains small washes up to 4 feet deep and soil ridges up to 4 feet high. *Id.* at 2-29.

2.16. The geomorphology of Skull Valley in the vicinity of the site is typical of a semiarid to arid desert setting. The adjacent mountain ranges are affected by mass-wasting processes and stream erosion that deliver sediment loads to a complex of alluvial fans (aprons) situated at the bases of the ranges. Runoff is conveyed down the ranges and over the alluvial fans through a series of small channels to the valley floor. Stream and spring flows are absorbed into the fan and the valley floor near the fan-floor interface, resulting in minimal surface runoff reaching the central valley near the site. There is no evidence of flash-flooding near the site nor are there deposits indicative of geologically recent [last 2 Ma (million years)] mudflows or landslides. *Id.*

2.17. The valley floor near the site comprises beach ridges and shoreline deposits interrupted by bedrock outcrops, such as Hickman Knolls rising about 400 feet above the valley bottom. The valley bottom relief comprises a series of braided, northerly flowing dry washes. The washes are disrupted and convey runoff for only short distances before merging into other washes or open space. This network of shallow washes extends offsite to the north where it confluences with the central valley drainage system and from there flows to the Great Salt Lake. The only perennial surface water is located approximately 10 miles north of the site. The central valley in the vicinity of the Facility is unaffected by fluvial processes. *Id.*

2.18. In the southern and eastern parts of the proposed site, numerous north-trending linear sand ridges interrupt the otherwise smooth valley floor. The ridges, which are typically 8 feet high and 100 feet wide, were originally mapped as possible fault traces. In the SAR, PFS reviewed the available surficial information and concluded that these features constitute sandy beach ridges deposited by southward longshore transport within the Stansbury shoreline coastal zone of Lake Bonneville. The applicant provided technical information about the nature and origin of the ridges which substantiated its conclusion that these ridges have a depositional origin. *Id.*

2.19. In a few locations, bedrock composed of Paleozoic carbonate rocks crop out of the smooth valley floor. The largest of these is a small group of hills 1.3 miles south of the proposed site known as Hickman Knolls. Rocks of this outcrop are medium to dark gray dolomite breccia. The origin and stratigraphic correlation of the Hickman Knolls carbonate rocks within the Paleozoic section is not well known. The preferred interpretation put forth by Geomatrix Consultants, Inc. ("Geomatrix") is that they are rooted bedrock outcrops. The alternative interpretation based on independent modeling of gravity data by the Staff is that they are landslide deposits, resting unconformably on the Tertiary sediments in the valley.[24] The differences in these two interpretations lead to differences in the estimated seismic hazard, discussed *infra* with respect to Part E of this contention. In the Geomatrix preferred interpretation, rooted bedrock requires a significant and seismogenic fault just west of Hickman Knolls. In the alternative interpretation, no such fault is necessary. Therefore, the Geomatrix preferred interpretation leads to a slightly more conservative seismic hazard. *Id.* at 2-29 to 2-30; *see* Staff Exh. Q.

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[24] *See generally,* "Seismic Ground Motion and Faulting Hazard at Private Fuel Storage Facility in the Skull Valley Indian Reservation, Tooele County, Utah -- Final Report," by John A. Stamatakos, Rui Chen, Martin W. McCann, Jr., and Asadul H. Chowdhury, Center for Nuclear Waste Regulatory Analysis (September 1999) (Staff Exh. Q).

2.20. The SAR discusses the geological history of the site and surrounding region. The discussion includes background information about the tectonic setting of the region in the Precambrian and Paleozoic that led to the deposition of the bedrock stratigraphy presently exposed in the Stansbury and Cedar Mountains. In brief, the structural framework of bedrock across the region reflects overprinting of several major periods of North American tectonic activity. These include contractional deformation structures such as thin- and thick-skinned thrusts and folds associated with the Devonian Antler, Jurassic to Cretaceous Sevier, and Cretaceous-Tertiary Laramide orogenies, and extensional normal and detachment faults associated with the Eocene to the current Basin and Range extension. Staff Exh. C, at 2-30.

2.21. As noted above, the proposed site lies near the center of a typical Basin and Range valley, situated between roughly north-south and northwest-southeast elongated ranges of exhumed bedrock. Exhumation of the ranges was accomplished by extensional faulting along range-front normal faults. Faulting tilted the ridges to the east. The adjacent basins subsided concomitant with exhumation while they accumulated sediment shed from the eroding ranges. In Skull Valley, as in much of central and western Utah, the valleys are also flooded by transgressions of the intramontane saline lakes. Tertiary and Quaternary deposits in and around the site document numerous transgressions associated with Lake Bonneville and pre-Lake Bonneville lacustrian cycles. Most important to the evaluations of seismic and faulting hazards was identification and characterization of a detailed Quaternary stratigraphy, that provided critical constraints on faulting activity and local and regional active faults. *Id.* at 2-30 to 2-31.

2.22. Valley-fill consists of inter-stratified colluvium, alluvium, lacustrine, and fluvial deposits with minor ash and some Eolian material. The coarser deposits are generally near the perimeter of the valley, grading into well-sorted sand and gravel and interlayered with lacustrine silt and clay toward the center of the valley. Thick beds of clay exist in some areas, with sand and gravel along the alluvial fans. The Salt Lake Group of the Tertiary age comprises most of the valley-fill with a thickness ranging from 2,000 to over 8,000 feet. *Id.* at 2-22. PFS has classified the subsurface material at the proposed site as a relatively compressible top layer, approximately 25 to 30 feet thick, that is underlain by much denser and stiffer material. The underlaying layer is classified as dense sand and silt. *Id.* at 2-23.

2.23. Valley fill sediments in Skull Valley consist of Tertiary age siltstones, claystones, and tuffaceous sediments overlain by Quaternary lacustrian deposits. Late Miocene to Pliocene deposits of the Salt Lake Formation were exposed in Trench T1 and in Boring C-5. Microprobe analyses of glass shards from vitric tuffs (ash fall deposits) within the sediments were used to correlate the tuffs with volcanic rocks of known age. The analyses indicate ages for the stratigraphic units between 16 and 6 Ma consistent with the known age of the Salt Lake Formation. During the Quaternary (approximately the last 2 Ma), especially the last 700 ka (thousand years), sedimentation in Skull Valley was dominated by fluctuations associated with lacustrian cycles in the Bonneville Basin. The SAR provides a detailed analysis of these deposits from trenches, test pits, and borings, including two radiocarbon ages on ostracodes and charophytes. *Id.* at 2-31.

2.24. The stratigraphy was also critical to interpretations of the reflection seismic profiles. Two prominent paleosols were developed during inter pluvial periods near the Tertiary-Quaternary boundary (~2 Ma) and between the Lake Bonneville and Little Valley cycles (130–28 Ka). These buried soils are characterized by relatively well-developed pedogenic carbonate, both in the soil matrix and as coatings on pebbles. As such, these paleosols form strong reflectors that are readily apparent on the seismic reflection profiles. These horizons were also correlated with cores from the borings drilled directly beneath the seismic profile lines. These constraints on the Quaternary stratigraphy and the high quality seismic reflection profiles provided in the Geomatrix report are sufficient to document the Quaternary faulting record of the site and to provide a stratigraphic framework for reliable paleoseismic analyses of active faults in and around Skull Valley. *Id.*

2.25. PFS has investigated the structural geologic conditions affecting its proposed site, with considerable attention given to the faults and structures identified therein.[25] Classical structural models for the Basin and Range envision a simple horst and graben framework in which range-front faults are planar and extend to the base of the transition between the brittle and ductile crust, 9–12.5 miles below the surface. More recent work has shown that many normal faults are not planar but curved or listric, and they sole into detachments that may or may not coincide within the brittle-ductile transition in the crust. In Skull Valley, the detachment model places the Stansbury fault as the master or controlling fault of a half graben. The other side of the half graben would include the antithetic East Cedar Mountain fault and a series of antithetic and synthetic faults within the basin, all of which would sole into the Stansbury fault 1–12.5 miles deep in the crust. *Id.*

2.26. Geomatrix developed a model with two regional cross sections that depict the overall structural framework of Skull Valley and the surrounding ranges. These cross sections were constructed from a compilation and analysis of existing geological map data, reprocessed and new seismic profiles across the valley, and interpretation of proprietary gravity data. The cross sections depict a series of pre-Tertiary folds and thrusts related to the Sevier and older contraction deformation that have been cut by a series of Tertiary and Quaternary normal faults related to Basin and Range extension. The normal faults are considered moderately dipping (~60°) planar features following the horst and graben model described previously. *Id.* at 2-31 to 2-32.

2.27. As discussed in Staff Exh. Q, the Staff considers that this horst and graben model is conservative for predicting a maximum earthquake potential for these faults. Faults that extend all the way to the base of the seismogenic crust define a larger area for earthquake rupture and

[25] As discussed *infra*, the State had originally challenged the Applicant's fault analysis, in Part B of this contention. That portion of the contention has since been resolved by stipulation of the parties. *See* discussion *infra*, at 22.

thus greater maximum magnitude earthquakes than those that terminate into a detachment above the brittle-ductile transition. *Id*. at 2-32.

2.28. The cross sections show three first-order, west-dipping normal faults and one east-dipping fault (the East Cedar Mountain fault). The west-dipping faults are the Stansbury and two previously unknown faults in the basin informally named the East and West faults. These new faults were interpreted based mainly on analyses of the gravity and seismic reflection data and by analogy to other faults in the Basin and Range. Discovery of these new faults and related structures was found to have important implications to both the seismic and fault displacement hazard assessments. *Id.* The significance of the various faults is discussed *infra* with respect to Part E of this contention. [26]

2.29. Finally, within the valley fill, the Applicant's SAR documented several additional secondary faults designated as fault zones A to F. Each fault zone has a number of secondary splays that are designated with numeral subscripts (*e.g.,* A1 to A7, B1 and B2, and so forth). These fault zones are all considered secondary faults related to deformation of the hanging wall above the larger East and West faults. They are too small to be independent seismic sources but large enough to be considered important in the fault displacement analysis. The largest of the secondary faults is the F fault, which appears to be a splay of the East fault. The characteristics

[26] A critical aspect of the interpretation of the East and West faults centers on the origin and nature of rocks exposed at Hickman Knolls, which are composed of monolithologic carbonate breccias. Under one alternative, the breccias are part of a detached landslide block of a bedrock dislodged from one of the nearby ranges by Tertiary or Quaternary earthquake activity along the range fronts; under the second alternative, the breccias are rooted to the Paleozoic basement beneath the basin fill. The difference between these alternatives is important to structural interpretations of Skull Valley. Geomatrix favored the alternative in which the West fault becomes a significant contributor to the overall seismic hazard. The Staff's independent analysis of EDCON gravity data provided in the SAR favored that alternative, as well, in that the West fault appears to be a splay of the East fault and, therefore, is not capable of independently triggering earthquakes. Given that Geomatrix included the West fault coupled with other conservative assumptions about seismicity, the Staff concluded that the Geomatrix assessment has led to a conservative hazard assessment, in terms of the seismic source characterization. *See* Staff Exh. C at 2-32 to 2-33.

of these secondary faults and their contributions to the surface faulting hazard at the proposed site are discussed in § 2.1.6.3 of the Staff'S SER. *Id.* at 2-33, 2-53.

2.30. With this background in mind, we turn to consider the various issues raised in Subparts A through E of Unified Contention Utah L/QQ.

B. Unified Contention Utah L/QQ, Parts A and B.

3.1. Parts A and B of Unified Contention Utah L/QQ asserted as follows:

> **A. Surface Faulting**.
>
> 1. The Applicant's approach to surface faulting is neither integrated nor comprehensive and is inadequate to assess surface rupture at the site in that:
>
> a. The Applicant has not used soil velocity data obtained from its seismic cone penetration tests in order to convert the seismic reflection data to show depth of marker beds.
>
> b. The Applicant's conclusion that the structural grain of the valley runs northwest does not account for the east-west Pass Canyon and the topographic embayment at the east-west trending Rydalch Pass.
>
> c. The Applicant has failed to collect any seismic tie lines perpendicular to the east-west lines shot in 1998 in order to correlate the 1998 lines among themselves or with the Geosphere and GSI lines, nor are the placement and number of seismic lines adequate to determine the length and projected locations of the East or West faults and other unnamed faults.
>
> **B. Ground Motions.**
>
> 1. The Applicant's failure to adequately assess ground motion places undue risk on the public and the environment and fails to comply with 10 CFR § 72.102(c) in that:
>
> a. The Applicant has not conducted a fully deterministic seismic hazard analysis that meets the requirements of 10 CFR Part 100 Appendix A.

3.2. As discussed *supra* at 9, Parts A and B of Unified Contention Utah L/QQ were resolved by stipulation of the parties; these matters were not addressed in the evidentiary hearings, and no issues remain to be adjudicated concerning these portions of the contention. Accordingly, these portions of the contention are not addressed further herein.

C. Unified Contention Utah L/QQ, Part C.[27]

The Contention

4.1. Part C of this contention combined certain issues raised in both Contention Utah L and Contention Utah QQ. As formulated, this portion of Unified Contention Utah L/QQ challenges the acceptability of the Applicant's geotechnical site characterization and of its proposed uses of cement-stabilized soil in its ISFSI design. *See* PFS Exh. 237, at 1-3. It asserts as follows:

> **C. Characterization of Subsurface Soils.**
>
> 1. Subsurface Investigations
> 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

[27] The NRC Staff's proposed findings of fact concerning Part C of Unified Contention Utah L/QQ were prepared principally by former Staff Counsel Martin J. O'Neill, prior to his filing a Notice of Withdrawal in this proceeding on August 19, 2002.

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.

Applicable Legal Standards

4.2 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.[28] Specific requirements for the characterization of the subsurface soils are defined in 10 C.F.R. § 72.102. In particular, 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."

4.3. Minimum general design criteria (GDC) applicable to the design, fabrication, construction, testing, maintenance, and performance of structures, systems, and components important to safety (SSCs) at an ISFSI are set forth in 10 C.F.R. § 72.120 *et seq*. Pursuant to

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[28] The types and magnitudes of the events which must be accommodated in a facility's design are established in the facility's design bases. The design bases for an ISFSI are defined in 10 C.F.R. § 72.3, in pertinent part, as follows:

> *Design bases* means that information that identifies the specific functions to be performed by a structure, system, or component of a facility or of a spent fuel storage cask and the specific values or ranges of values chosen for controlling parameters as reference bounds for design. These values may be derived from generally accepted state-of-the-art practices for achieving functional goals or requirements derived from analysis (based on calculation or experiments) of the effects of a postulated event under which a structure, system, or component must meet its functional goals. The values for controlling parameters for external events include–
>
> (1) Estimates of severe natural events to be used for deriving design bases that will be based on consideration of historical data on the associated parameters, physical data, or analysis of upper limits of the physical processes involved . . . .

10 C.F.R. § 72.122(b)(1) ("Protection against environmental conditions and natural phenomena"), structures, systems, and components important to safety ("SSCs") "must 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 that SSCs be designed to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform their intended design functions.

Evidence Presented

4.4. Evidentiary hearings on this portion of the contention were held on June 17-18 and June 20-21, 2002. A total of five witnesses appeared on behalf of PFS, the Staff, and the State of Utah, as set forth below. In addition to their prefiled testimony, each of the witnesses also provided oral and/or written rebuttal or surrebuttal testimony. All of the witnesses were found to be qualified to present testimony on the matters they addressed.

Applicant Witnesses

4.5. The Applicant presented two witnesses with respect to Part C of this contention. These were: (1) Paul J. Trudeau, a Senior Lead Geotechnical Engineer at Stone & Webster, Inc., whose responsibilities included, *inter alia*, the conduct of the Applicant's geotechnical site characterization and evaluation of the stability of the cask-storage pad foundations (*i.e.*, the potential for bearing-capacity failure, settlement, and sliding) under static and dynamic loading conditions; and (2) Dr. Anwar E.Z. Wissa, President of Ardaman & Associates, Inc., in Orlando, Florida, who was retained by PFS to review its intended uses of cement-stabilized soils (*i.e.*, soil cement and cement-treated soil) at the PFS site and PFS's associated proposed testing program and construction techniques. *See* "Joint Testimony of Paul J. Trudeau and Anwar E.Z. Wissa on Section C of Unified Contention Utah L/QQ" (hereinafter referred to as "Trudeau/Wissa"), Post

Tr. 10834, at 1-4.[29] At the hearing, Mr. Trudeau testified individually with respect to site characterization issues, whereas Mr. Trudeau and Dr. Wissa testified jointly as a panel on soil cement issues. In addition, PFS presented oral rebuttal testimony by Mr. Trudeau and Dr. Wissa and two pieces of written rebuttal testimony pertaining to soil cement and soils characterization issues, respectively. *See* "Rebuttal Testimony of Paul J. Trudeau and Anwar E.Z. Wissa to Direct Testimony of State of Utah Witnesses Dr. Steven F. Bartlett and James K. Mitchell on Section C of Unified Contention L/QQ" (hereinafter referred to as "Trudeau/Wissa Rebuttal", Post Tr. 11232; and "Rebuttal Testimony of Paul J. Trudeau to Testimony of State of Utah Witness Dr. Steven F. Bartlett on Section C of Unified Contention L/QQ (Soils Characterization)" (hereinafter referred to as "Trudeau Rebuttal"), Post Tr. 11954.

4.6. Applicant witness Paul J. Trudeau received an M.S. degree in Civil Engineering from the Massachusetts Institute of Technology (MIT). Trudeau Qualifications, at 2. He is a registered professional engineer with 29 years of experience in geotechnical engineering. Trudeau/Wissa Post Tr. 10834, at 1. This experience encompasses subsurface soil investigations; analyses of foundations in support of structural designs; laboratory testing of soils; analyses of soil and structure performance under static and dynamic conditions; and the development of geotechnical design criteria for other engineering disciplines. *Id.* at 1-2. Mr. Trudeau has conducted geotechnical investigatory and design work in connection with numerous other facilities, including a number of nuclear facilities. Trudeau Qualifications, at 3-5. The Licensing Board finds Mr. Trudeau to be well-qualified as an expert witness on the subjects of subsurface soil investigations, laboratory testing of soils, and the analysis of soil and foundation behavior under loading conditions.

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[29] PFS witness Trudeau's Qualifications are attached to his testimony on Part D issues, entitled "Testimony of Paul J. Trudeau on Section D of Unified Contention Utah L/QQ."

4.7. Applicant witness Dr. Anwar E.Z. Wissa received an M.S. degree in Civil Engineering and a D.Sc. Degree in Geotechnical Engineering from MIT, in addition to B.A. and M.A. degrees from Oxford University. Wissa Qualifications, at 1. He is a registered professional engineer in Massachusetts and Florida with over 40 years of experience that includes consulting work as well as teaching and research at MIT. *Id.* at 1-2. Dr. Wissa has been a Fellow of the American Society of Civil Engineers (ASCE) since 1983, and served on the Committee on Placement and Improvement of Soil for nine years. *Id*. at 4. He has also been a member of Committee D-18 on Soil and Rock for the American Society of Testing and Materials ("ASTM") since 1966. Trudeau/Wissa Post Tr. 10834, at 3-4. Further, he has been extensively involved in projects utilizing soil cement, including reservoir and pavement projects, and has authored or co-authored publications on soil stabilization. *Id.* at 4. Dr. Wissa was also a member (from 1985 to 2000) of American Concrete Institute (ACI) Committee 230, which published ACI 230.1R-90 (Reapproved 1997), "State-of-the-Art Report on Soil Cement" (PFS Exh. HHH). Wissa Qualifications, at 3. The Licensing Board finds Dr. Wissa to be well-qualified as an expert witness on the use of cement-stabilized soils to improve subsurface conditions and the development and testing of cement-soil mixtures.

4.8. The State presented two witnesses in support of Part C of its contention. These were: (1) Dr. Steven F. Bartlett, an Assistant Professor in the Civil and Environmental Engineering Department at the University of Utah; and (2) Dr. James K. Mitchell, a University Distinguished Professor Emeritus at Virginia Polytechnic Institute and State University ("Virginia Tech") and Professor Emeritus at the University of California at Berkeley. State Exh. 92, at 1; State Exh. 105, at 1-2. The State submitted two pieces of prefiled written testimony concerning Part C of its contention: one relating to soils characterization issues, and the other relating to the Applicant's proposed use of cement-stabilized soils. *See* "State of Utah Testimony of Dr. Steven F. Bartlett on Unified Contention L/QQ (Soils Characterization)" (hereinafter referred to as "Bartlett"), Post

Tr. 11822; "State of Utah Testimony of Dr. Steven F. Bartlett and Dr. James K. Mitchell on Unified Contention Utah L/QQ (Soil Cement)" (hereinafter referred to as "Bartlett/Mitchell"), Post Tr. 11033. Accordingly, Dr. Bartlett testified individually with respect to soils characterization issues and jointly with Dr. Mitchell on cement-stabilized soil issues. Dr. Bartlett also provided oral rebuttal and written surrebuttal testimony with respect to soils characterization issues. *See* "Surrebuttal of Dr. Steven Bartlett to PFS Witness Paul Trudeau's Rebuttal Testimony on Section C of Unified Contention Utah L/QQ" (hereinafter referred to as "Bartlett Surrebuttal"), Post Tr. 11982. Finally, Drs. Bartlett and Mitchell presented oral rebuttal and surrebuttal testimony on specific issues related to the proposed use of cement-stabilized soils at the PFS site.

4.9. State witness Dr. Steven F. Bartlett received a B.S. degree in Geology and a Ph.D. in Civil Engineering from Brigham Young University. Bartlett Post Tr. 11822, at 1. Dr. Bartlett is currently an Assistant Professor in the Civil and Environmental Engineering Department at the University of Utah, where he teaches undergraduate and graduate courses in geotechnical engineering and conducts research. *Id.* He is a licensed professional engineer in Utah and has worked for the Utah Department of Transportation, Woodward-Clyde Consultants, and Westinghouse Savannah River Co. *Id.* The Licensing Board finds Dr. Bartlett to be well-qualified as an expert witness on the subjects of subsurface soil investigations, laboratory testing of soils, and the analysis of soil and foundation behavior under different loading conditions.

4.10. State witness Dr. James K. Mitchell received a B.S. in Civil Engineering from Rensselaer Polytechnic Institute, and an S.M. and Sc.D. in Civil Engineering from MIT. State Exh. 105, at 1. He is currently a University Distinguished Professor Emeritus at Virginia Tech and Professor Emeritus at the University of California at Berkeley. Bartlett/Mitchell Post Tr. 11033, at 1-2. Dr. Mitchell is licensed as a civil and geotechnical engineer in California and as a professional engineer in Virginia. *Id.* at 2. He has over 40 years of experience in the field of geotechnical engineering, and serves as an individual consultant on geotechnical problems and earthworks projects of various types, including projects involving soil stabilization and ground improvement for seismic risk mitigation. *Id.* He has conducted research on a number of geotechnical and soil behavior topics germane to this proceeding, and has authored more than 350 publications. *Id.* He is a Fellow and Honorary Member of the ASCE, among other affiliations, and has served as an officer to the Geotechnical Engineering Division of the ASCE. *Id.* He has also served on the ASCE Committee on Soil Properties and the Committee on Placement and Improvement of Soils. *Id.* at 3. The Licensing Board finds Dr. Mitchell to be well-qualified as an expert witness on the use of cement-stabilized soils to improve subsurface conditions and the development and testing of cement-soil mixtures.

4.11 With respect to Part C of this contention, the Staff presented the testimony of Dr. Goodluck I. Ofoegbu, 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. *See* "NRC Staff Testimony of Goodluck I. Ofoegbu Concerning Unified Contention Utah L/QQ, Part C" (hereinafter referred to as "Ofoegbu"), Post Tr. 11001, at 1; Tr. 6736.[30] His testimony, which was provided under a technical assistance contract between the NRC Staff and the CNWRA, addresses the acceptability of the Applicant's characterization of subsurface soils at the PFS site and its proposed uses of cement-stabilized soils. *Id.* at 2. During the evidentiary hearings, Dr. Ofoegbu also provided oral rebuttal testimony concerning certain aspects of the Applicant's subsurface soils characterization.

4.12. Staff witness Dr. Goodluck I. Ofoegbu received a B.Sc. in Geology from the University of Nigeria and an M.A.Sc. and Ph.D. in Geological Engineering from the University of

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[30] The CNWRA was established as a Federally-funded research and development center to assist the NRC in the technical analysis of nuclear waste disposal projects. It currently performs about 70 percent of its work for the NRC, and 30 percent for private industry. The SwRI, of which the CNWRA is a part, is not-for-profit private research institute that conducts research for both government and private entities, in the United States and abroad. Tr. 6735-37.

Toronto. Ofoegbu Qualifications, at 1. He is a registered professional engineer in Canada. *Id.* Dr. Ofoegbu specializes in the mechanical analyses of geological processes, finite element modeling, and the constitutive modeling of geological materials. *Id.* His work includes mechanical analysis of underground excavations, foundations, earthworks, and natural geological processes such as faulting and volcanism. *Id.*; Ofoegbu Post Tr. 11001, at 1. He has approximately 20 years of professional experience that includes teaching, research, and consulting. Ofoegbu Qualifications, at 1. He has published numerous articles in refereed journals and conference proceedings, as well as several technical reports. *Id.* at 1-3. The Licensing Board finds Dr. Ofoegbu to be well-qualified as an expert witness on the issues raised in this proceeding concerning the acceptability of the Applicant's geotechnical site characterization and its proposed uses of cement-stabilized soils.

4.13. For the reasons more fully set forth below, having considered the testimony and other evidence presented by the parties, we find that the evidence supports a conclusion 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 information obtained by the Applicant through its geotechnical site characterization 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. Namely, the geotechnical investigations and laboratory tests performed by the Applicant support the conclusion that the soil conditions at the PFS site are adequate for the proposed foundation loadings. Further, with respect to the Applicant's proposed use of cement-stabilized soils in the vicinities of the storage pads and Canister Transfer Building (CTB), we find that the evidence supports a conclusion that the specified design properties of these materials can be achieved by the Applicant (subject to post-licensing test verification, which PFS has committed to perform), and that the resulting cement-stabilized soils would be capable of performing their intended functions. [31]

1. GEOTECHNICAL SITE CHARACTERIZATION

4.14. As noted above, the specific regulatory requirement for the geotechnical site characterization for an ISFSI is contained in 10 C.F.R. § 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. PFS sought to satisfy this regulatory requirement through a combination of geotechnical field investigations and laboratory testing, discussed herein.

Subsurface Investigations

4.15. With respect to its subsurface field investigations, PFS utilized multiple techniques in characterizing the subsurface soils at the proposed site and assessing their adequacy for the proposed foundation loadings. These techniques included, *inter alia*, soil borings (including visual field classification of drill cuttings and split-spoon samples and the collection of undisturbed soil samples), standard penetration tests (SPTs), dilatometer tests (DMTs), *in situ* cone penetrometer tests (CPTs), seismic CPTs (which provided measurements of pressure and shear wave velocities in addition to penetration resistance data), downhole geophysical measurements, and the excavation of test pits and trenches. *See* Trudeau/Wissa Post Tr. 10834, at 5-8; Staff Exh. C at 2-55. The locations of the various soil borings, CPTs and DMTs, and test pits at the PFS site

[31] In this regard, we note that the Commission, in adopting its rules in 10 C.F.R. Part 72, clearly established a "one-step licensing procedure" for ISFSIs. Statement of Consideration, "Licensing Requirements for the Storage of Spent Fuel in an Independent Spent Fuel Storage Installation," 45 Fed. Reg. 74693 (1980). Accordingly, it should be expected that tests which verify that an applicant has satisfied its design commitments would be performed post-licensing where, as here, the tests and the criteria to be satisfied have been clearly defined, and the test results may be verified objectively. *See generally, Cleveland Electric Illuminating Co.* (Perry Nuclear Power Plant, Unit 1), CLI-96-13, 44 NRC 315, 327-28, 330 (1996), *citing Union of Concerned Scientists v. NRC*, 735 F.2d 1437, 1451 (D.C. Cir. 1984), *cert. denied*, 469 U.S. 1132 (1985);*Metropolitan Edison Co.* (Three Mile Island Nuclear Station, Unit No. 1), ALAB-729, 17 NRC 814, 886-87 (1983) (post- hearing procedures may be used for confirmatory tests).

are shown in Figures 2.6-2, 2.6-18, and 2.6-19 of the SAR. Trudeau/Wissa Post Tr. 10834 at 6; PFS Exh. 235. The results of these various investigations are presented in Section 2.6 and Appendix 2A of the SAR and visually manifested in the form of geologic maps and site stratigraphy or "foundation" profiles that are also provided in the SAR. *See* Trudeau/Wissa Post Tr. 10834 at 6.

4.16. The 17 foundation profiles provided by PFS in Figures 2.6-5 (14 sheets) and 2.6-20 to 2.6-22 of the SAR (two diagonal, six east-west, and six north-south lines in the pad emplacement area, and two east-west lines and one north-south line in the CTB area) depict the subsurface soil composition in the vicinity of all safety-related structures at the proposed site. *See* Trudeau/Wissa Post Tr. 10834 at 6; PFS Exh. 233, 233A. These profiles demonstrate the nature, location, and thickness of the various soil layers underlying the proposed PFS site. *See* Trudeau/Wissa Post Tr. 10834 at 6-8.

4.17. Based on the information obtained from its geotechnical investigations, PFS effectively characterized the subsurface soil profile -- for geotechnical engineering purposes -- as consisting of two primary layers. *See* Ofoegbu Post Tr. 11001, at 6. Layer 1, a relatively compressible top layer that is approximately 25-30 feet thick, consists of a mixture of clayey silt, silt, and sandy silt with occasional silty clay and silty sand. *See id.*; Staff Exh. C, at 2-55 to 2-56. As reflected in the 17 foundation profiles (*see* PFS Exh. 233, 233A), Layer 1 can be further subdivided into several sublayers, (in top-down order): layer 1A, classified as eolian silt, is typically about 3–5 feet thick; layer 1B, a silty clay/clayey silt mixture that varies in thickness from about 5 to 10 feet;[32] layer 1C, a mixture of clayey silt, silt, and sandy silt, with thickness of about

[32] In the Applicant's prefiled testimony on Part C issues, Mr. Trudeau referred to layer 1B as "Layer 2." Trudeau/Wissa, Post Tr. 10834, at 8. The parties acknowledged this difference in nomenclature during the hearing, and Mr. Trudeau explained that his reference to "layer 2" was in fact a reference to the layer-1B or upper Lake Bonneville soils (located approximately 3 to 10 feet below the ground surface). *See* Tr. 11732, 11815, 11834-35.

7.5–12 feet; and layer 1D, a silty clay/clayey silt mixture with maximum thickness of about 5 feet. Ofoegbu Post Tr. 11001, at 7; Staff Exh. C, at 2-56.

4.18. Layer 1 is underlain by much denser and stiffer sand and silt, referred to by the Staff as layer 2. *See id.* at 6; Staff Exh. C, at 2-55. The Applicant's geotechnical investigations established that the approximately top 30 feet of the subsoil profile (*i.e.,* layer 1) is of primary interest from a geotechnical standpoint, as the soils below this depth consist of very dense sands or silty sands overlying very dense silts. Trudeau/Wissa Post Tr. 10834, at 8. As evidenced by their high SPT blow counts (N > 100 blows/ft.), these soils have considerable strength, and are not expected to be a source of potential instability with respect to the proposed structures. *Id.*; Staff Exh. C at 2-55. The Staff agrees that the standard penetration and cone-tip resistance data provided in the SAR support the Applicant's classification of the subsurface materials at the site. *See* Ofoegbu Post Tr. 11001, at 6. The State does not challenge this general classification of the site subsoils. *See* Tr. 11832-33.

4.19. With respect to variations in soil properties, the profiles of cone-tip resistance contained in the foundation profiles reflect any lateral and vertical variations in the shear strength and compressibility of the layer 1 soil. Ofoegbu Post Tr. 11001, at 6-7. PFS interprets the CPT data as showing that the upper or layer-1 soils have fairly consistent properties across the pad emplacement area and beneath the CTB. *See, e.g.,* Trudeau/Wissa Post Tr. 10834, at 7; Trudeau Rebuttal Post Tr. 11954, at 1, 6-7. Similarly, Staff witness Ofoegbu noted that while there is some horizontal variation, the soil properties, including shear strength, vary more with depth than horizontally. Tr. 11785. PFS witness Trudeau also noted that shear wave velocities have extremely low variability across the site, which is further evidence that the soil properties are fairly uniform in the horizontal direction. Tr. 11726. Moreover, subsurface data obtained in 1999 by PFS consultant Geomatrix, as part of an extensive fault evaluation study, confirm that the soils in approximately the upper 30 feet of the subsoil are fairly uniform and consistent in the horizontal direction across the site. Trudeau/Wissa Post Tr. 10834, at 7.

Laboratory Testing and Analysis

4.20. In addition to performing various in situ subsurface field investigations, PFS also collected undisturbed soil samples for subsequent laboratory testing and analysis. *See id.* at 11; Tr. 11730. As shown in Table 1 of the Trudeau/Wissa prefiled testimony, PFS took a total of 33 undisturbed samples from eight borings in the pad emplacement area and from seven borings in the CTB area. Trudeau/Wissa Post Tr. 10834, at 10. Roughly two-thirds of these undisturbed samples were taken from layer-1B soils (approximately 3 to 10 feet below ground surface), as this is the layer of primary concern from the standpoint of soil strength and compressibility. *Id.* at 11.

4.21. The laboratory tests performed on the soil samples collected included stress and strain-controlled dynamic testing of the samples. *Id.* at 12. The laboratory testing program included 10 consolidation tests, 19 triaxial shear strength tests, 5 cyclic triaxial tests, 2 resonant column tests (at three different confining pressures), and 11 direct shear tests. *Id.* at 9. PFS used the laboratory test results, which are presented in Appendix 2A of the SAR, to ascertain parameters related to the static and dynamic properties of the soils, including grain size, triaxial shear strength, consolidation characteristics, Atterberg limits, water content, direct shear strength, shear moduli, damping, and strength under cyclic loading. *Id.* at 12.

4.22. PFS witness Trudeau testified that the collective laboratory and field test results are sufficient to determine the static and dynamic properties of the site subsoils, including their shear strength and compressibility, and that these results were conservatively interpreted to develop the design parameters for the Applicant's various engineering analyses. *Id.* at 13.

4.23. Based on its review, the Staff found that PFS provided sufficient geotechnical data to support the engineering analyses of the proposed structures, and that it satisfied the site characterization requirement set forth in 10 C.F.R. § 72.102(d) through its existing site investigations and laboratory analyses. *See* Ofoegbu Post Tr. 11001, at 8. Having considered all of the evidence, we find no reason to disagree with this conclusion. Our analysis of the specific site characterization concerns raised by the State is set forth below.

Specific Challenges to the Applicant's Geotechnical Site Characterization Program

4.24. In its prefiled testimony, the State adduced a series of interrelated concerns pertaining to the adequacy of the Applicant's field investigations, sampling and laboratory testing of site subsoils, and analysis of test data for purposes of its foundation system designs. Specifically, the State alleged that: (1) PFS has not performed the recommended spacing of borings for the pad emplacement area, as outlined in NRC Regulatory Guide (RG) 1.132; (2) PFS has not performed continuous sampling of critical soil layers important to foundation stability for each major structure, as recommended by RG 1.132; (3) PFS's design of the foundation systems is based on an insufficient number of tested samples; (4) PFS has not accounted for the potential horizontal variation of shear strength properties of the upper Lake Bonneville sediments across the pad emplacement area; (5) PFS has used potentially unconservative estimates of the undrained shear strength in the dynamic bearing capacity calculations for the CTB; (6) PFS should have included strain-controlled cyclic triaxial tests in its laboratory shear strength testing program; (7) PFS has not adequately analyzed the stress-strain behavior of the native foundation soils under a range of cyclic strains imposed by the design earthquake; and (8) PFS should have included triaxial extension tests in its laboratory shear strength testing program. *See* Bartlett Post Tr. 11822, at 6-12.

Spacing and Density of Soil Borings

4.25. The State alleges that the number of borings made by PFS for the pad emplacement area is insufficient because the borehole and CPT spacing is approximately 221 feet for this area, whereas NRC Regulatory Guide 1.132, Appendix C recommends a spacing of 100 feet for linear structures. *See id.* at 6-7. On cross-examination, however, State witness Bartlett indicated that this concern relates to the density or areal spacing of the borings at the site, "not particularly looking at the cone penetration data, but the amount of borings . . . , and the amount of undisturbed sampling that was done." Tr. 11855. Indeed, CPT data were collected at numerous locations, in a far denser pattern than the borings. *See* Trudeau Rebuttal Post Tr. 11954 at 7-8; PFS Exh. 233.

4.26. RG 1.132 provides general guidelines or recommendations concerning site investigations for foundations of nuclear power plants, including the spacing and depth of borings for safety-related structures. *See* Trudeau/Wissa Post Tr. 10834, at 13; Ofoegbu Post Tr. 11001, at 5-6. Indeed, this guidance document is not directly applicable to Part 72 facilities such as the PFS facility. *See* Trudeau/Wissa Post Tr. 10834, at 13-14; Trudeau Rebuttal Post Tr. 11954, at 5-6. Therefore, RG 1.132 need not be used in soils investigations for storage pads, which differ significantly from nuclear power plant structures with respect to foundation size and loadings and the types of SSCs involved. [33] *See* Trudeau Rebuttal Post Tr. 11954, at 5-6.

4.27. In any event, as noted by the Staff, RG 1.132 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. Ofoegbu Post Tr. 11001, at 5-6. Further, RG 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.

[33] With respect to the density of borings in the CTB area, PFS elected to follow the guidance in RG 1.132 because it considered the CTB to be more analogous to a nuclear power plant structure than a storage pad. *See* Trudeau Rebuttal Post Tr. 11954, at 5-6. The State did not challenge the density of borings in the CTB area as being deficient. *See* Bartlett Post Tr. 11822, at 7.

PFS Exh. 234, at 1.132-1. Dr. Bartlett conceded that RG 1.132 is a guidance document which is not strictly applicable to ISFSIs, and that professional judgment is required in designing a soils investigation program. Bartlett Surrebuttal Post Tr. 11982, at 3.

4.28. On rebuttal, PFS summarized the considerations underpinning the professional judgment it exercised in designing and implementing its soils investigation program. Trudeau Rebuttal Post Tr. 11954, at 6-7. In particular, PFS noted that with respect to the storage pads, it "exercised professional judgment and developed a subsurface investigation program which combined the drilling of boreholes and the performance of cone penetrometer tests and geophysical testing to the extent warranted by site conditions and the size, loading, and isolation of the storage pads." *Id.* at 6.

4.29. Mr. Trudeau testified that the initial boring work done in 1996 served to establish that the soil properties were reasonably uniform across the pad emplacement area of the PFS site, and that the results of subsequent SPTs and CPTs confirmed that the stratigraphy and subsoil characteristics across the pad emplacement area are relatively uniform. *See* Trudeau Rebuttal Post Tr. 11954, at 6-7. Further, he testified that the grid of borings and CPTs covered the entire pad emplacement area. Tr. 11767. Accordingly, because PFS "found no evidence of significant horizontal variations in the thickness or properties of the soil layers in the pad emplacement area," and felt assured that it had properly characterized this area, it did not find it necessary to establish a denser set of borings. *Id.* at 7; Trudeau/Wissa Post Tr. 10834, at 14.

4.30. In regard to this issue, the Staff expressed its view that the Applicant's existing site-specific investigations and laboratory analyses have allowed for satisfactory classification of the subsurface materials, identification of lateral and vertical variations in the relevant properties of those materials, and confirmation that the site-specific soil conditions are adequate for the proposed foundation loading. Ofoegbu Post Tr. 11001, at 8. We agree based on our review of the evidence. Therefore, we do not find it necessary that PFS follow the particular spacing of borings recommended in RG 1.132 with respect to the pad emplacement area.

Continuous Sampling and Number of Tested Samples

4.31. The State claims that PFS did not perform "continuous sampling" of critical soil layers important to foundation stability for each major structure, again citing recommendations contained in RG 1.132 in support of this assertion. Bartlett Post Tr. 11822, at 5, 7. More specifically, State witness Bartlett testified that "[t]he upper Lake Bonneville sediments have not been continuously sampled and characterized with depth," and that "[t]his incomplete characterization adds additional uncertainty to the Applicant's estimate of the shear strength of this important layer and subsequently to the factors of safety calculated for seismic sliding and bearing capacity of the pads." *Id.* at 8. Dr. Bartlett testified that he understands "continuous sampling," as referred to in RG 1.132, to mean the collection of undisturbed soil samples from boreholes and the testing of those samples in the laboratory. *See* Tr. 11868-69.

4.32. Further, the State asserts that the Applicant's design of the foundation systems is based on an insufficient number of tested samples. *See* Bartlett Post Tr. 11822, at 5, 8. Specifically, Dr. Bartlett opined that "[t]he most egregious weakness of the Applicant's sampling program is the extreme undersampling that has been performed of the upper Lake Bonneville sediments," since "[t]he Applicant has calculated the sliding resistance of the pads on one set of direct shear tests obtained from borehole C-2 at a depth interval of 5.7 to 6 feet [citation omitted]." *Id.* at 8. Hence, the "undersampling" alleged to exist by the State refers again to the Applicant's collection of undisturbed samples from the pad emplacement area for laboratory measurement of undrained shear strength. Thus, given the interrelatedness of these issues – *i.e.*, continuous sampling and the number of tested samples – we address these issues together.

4.33. The purpose of "continuous sampling" is to determine the continuous variation of soil properties with depth. Ofoegbu Post Tr. 11001, at 9. In particular, it serves to identify "suspect zones," *i.e.*, "[r]elatively thin zones of weak or unstable soils [that] may be contained within more competent materials and may affect the engineering characteristics or behavior of the soil or rock." Trudeau Rebuttal Post Tr. 11954, at 8-9; PFS Exh. 234, at 1.132-5. In the Staff's view, *in situ* cone penetrometer testing, as used by the Applicant, is an acceptable "alternative procedure" for ascertaining the continuous variation of soil properties with depth. Ofoegbu Post Tr. 11001, at 9. State witness Bartlett recognized that "the CPT data were gathered to . . . gain more information about the layering and the strength and compressibility [of the site subsoils]," and that the data are "invaluable" in that it "really helps us to understand the site." Tr. 12008.

4.34. PFS performed 37 CPT soundings in the pad emplacement area. *See, e.g.,* Trudeau Rebuttal Post Tr. 11954, at 8. These CPTs gave continuous profiles of tip resistance and sleeve friction, which in turn were interpreted to obtain continuous profiles of relative soil strength and compressibility. *See* Staff Exh. C, at 2-55.; PFS Exh. 233. These tests yielded essentially the same values of tip resistance for comparable depths at various locations across the pad emplacement area, thus indicating that the stratigraphy across the site is uniform. *See* Trudeau Rebuttal Post Tr. 11954, at 7.

4.35. With respect to potential variation of soil properties in the vertical direction, the CPT data confirm that there are no weak layers that have been missed by the soil sampling that was performed in the borings drilled in the pad emplacement area. *Id.* at 8. Such layers would have been detected through changes in the cone tip resistance measured during the CPTs, which included readings at 0.2-foot intervals within the layer-1B soils at 37 locations across the pad emplacement area. *Id.* at 9; Tr. 11773. Moreover, continuous undisturbed sampling of layer 1-B soils in boreholes drilled within the adjacent CTB area did not reveal any zones of weak or unstable soils. *See* Trudeau Rebuttal Post Tr. 11954, at 9.

4.36. We agree with PFS and the Staff that PFS has successfully determined the vertical variation of soil properties -- including shear strength -- through the use in situ cone penetrometer testing, thereby achieving the purpose of "continuous sampling." With respect to shear strength in particular, the witnesses agreed that this soil property is proportional to the cone tip resistance measured as part of such tests. *See* Tr. 11773, 11790; Bartlett Post Tr. 11822, at 9. Although the CPTs do not provide absolute values of undrained shear strength, the tip resistance profiles generated by the CPTs within the pad emplacement and CTB areas indicate how the undrained shear strength varies with depth and laterally at the site; this is illustrated in Figures 2.6-5 (14 sheets) and 2.6.21 through 2.6.23 of the SAR. Tr. 11790; Ofoegbu Post Tr. 11001, at 6-7.

4.37. Notwithstanding the extensive CPT data obtained by PFS, the State maintains that additional continuous sampling in the critical layer for laboratory testing is required, and that PFS has failed to collect a sufficient number of samples for foundation design purposes. *See, e.g.,* Tr. 11863; Bartlett Post Tr. 11822, at 8. The evidence indicates, however, that PFS used the CPT data in conjunction with laboratory test data to determine appropriate soil-strength parameters for its stability analyses of the cask storage pads and CTB. Ofoegbu Post Tr. 11001, at 10. Indeed, PFS drilled a total of sixteen borings (A-1 to A-4, B-1 to B-4, C-1 to C-4, and D-1 to D-4) in or near the pad emplacement area and collected disturbed and undisturbed soil samples from these boreholes. Trudeau Rebuttal Post Tr. 11954, at 7. Of the nine undisturbed samples taken (and subsequently tested) from boreholes located within the pad emplacement area, five of the samples were taken from layer 1-B. *See Id.* at 8; Trudeau/Wissa Post Tr. 10834, at 10 (Table1). As discussed below, this approach resulted in conservative values for the soil parameters used for stability analysis.

4.38. Significantly, with respect to the Applicant's use of the data derived from these undisturbed samples, Mr. Trudeau testified that "for those analyses that required soil properties such as strength and compressibility as inputs, PFS generally used the least favorable value of each of the measured properties (e.g., lowest peak strength and highest compressibility) of the subsoil from the weakest soil layer to represent the *entire* top thirty feet of soil." Trudeau/Wissa Post Tr. 10834, at 14-15.

4.39. In fact, PFS measured the undrained shear strength of the upper Lake Bonneville from Sample U-1 from Boring C-2, which it collected from the weakest portion (the 5 to 7-foot depth range) of the weakest, most compressible soil layer (layer-1B) of the weakest section of the pad emplacement area (the northeast quadrant). *See* Tr. 11769-71. This sample had the highest void ratio and lowest density -- which are indicators of low shear strength -- of any of the samples collected in the pad emplacement area. *See* Tr.11769-70. The CPT tip resistance profiles obtained by PFS, which indicate the relative variation of shear strength with depth, support the conclusion that the sample tested by the PFS is from the "critical layer," and that it provided a conservatively low undrained shear strength value for use in the foundation stability analyses of the storage pads. *See* Tr. 11772, 11789; Ofoegbu Post Tr. 11001, at 10.

4.40. As summarized by the Staff, 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. *Id.* Information presented in a PFS calculation 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. *Id.* As stated in the Consolidated SER (Staff Exh. C, at 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. *Id.* 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 CTB foundation. *Id.*

4.41. For the foregoing reasons, we find that additional "continuous sampling" of the type called for by the State is unnecessary for purposes of the Applicant's site characterization and engineering analyses. Further, we conclude that the Applicant's design of the storage pad foundation system is adequate and based on a sufficient number of tested samples.

Potential Variation of Shear Strength Properties Across the Pad Emplacement Area

4.42. In a related argument, the State alleges that PFS has failed to account for "the potential horizontal variation of shear strength properties of the upper Lake Bonneville sediments across the pad emplacement area." Bartlett Post Tr. 11822, at 8. This concern appears to relate largely to the parties' respective interpretations of the aforementioned CPT data. In particular, State witness Dr. Bartlett testified that these data suggest that the CPT tip resistance values (and hence the shear strength of the corresponding soils) may vary by about a factor of two across the pad emplacement area in the depth interval between three and ten feet below ground surface (*i.e.,* the layer-1B or upper Lake Bonneville soils). *Id.* at 9; Tr. 11874-84. Further, he testified that PFS has neither (1) made a "statistical assessment" of the alleged horizontal variation for purposes of assessing how this variation may affect the single shear strength value of 2.1 ksf used in the pad sliding stability analyses, nor (2) established a site-specific correlation that relates the CPT tip resistance data to actual values of undrained shear strength measured by direct shear tests. Bartlett Post Tr. 11822, at 9; Tr. 11856-60, 11939-44.

4.43. The relevant inquiry is whether the Applicant has obtained and appropriately analyzed adequate geotechnical site characterization data to support its engineering analyses of the storage pads. *See* Ofoegbu Post Tr. 11001, at 9. We find that the Applicant has in fact done so, and that even if the State's criticisms regarding interpretation of the CPT data have some merit, they do not undermine this conclusion, for the reasons set forth below.

4.44. The evidence indicates that there is ample geotechnical data, derived from multiple investigatory techniques, to demonstrate that the critical upper soil layers have consistent properties across the pad emplacement area. *See, e.g.,* Trudeau Rebuttal Post Tr. 11954, at 6-7. Indeed, it is this "horizontal consistency" that led PFS to conclude that there is no need for any type of statistical analysis of the deviation or variation of cone tip resistance values across the various horizontal locations at the site. *See* Tr. 11771-72. As stated by Mr. Trudeau, the CPT data support the conclusion that PFS selected a "lower bound strength" or "minimum strength value for use in the sliding stability analyses of the soils in the pad emplacement area." Tr. 11966; Trudeau Rebuttal Post Tr. 11954, at 9.

4.45. The State challenges this conclusion, maintaining that "to estimate the undrained shear strength from the CPT data, normally one would develop a correlation." Tr. 11939. The correlation referred to by the State is a site-specific correlation that relates the CPT tip resistance data to actual laboratory-measured values of undrained shear strength. *See id.* According to State witness Dr. Bartlett, such a correlation requires the use of "paired data" obtained from CPTs and adjacent boreholes -- *i.e.*, the calibration of CPT data with known measured values of undrained shear strength from adjacent boreholes. Tr. 11940; State Exh. 100, at 4-56. This permits the development of a site-specific cone bearing or correction factor (N_k or N_{kt}) that, in turn, can be used in an equation to calculate undrained shear strengths from cone tip resistance values. *See* Tr. 11940; State Exh. 100, at 4-55; PFS Exh. 238. The State maintains that a site-specific correlation or N_k factor has not been developed for the soils at the PFS site. Tr. 11940.

4.46. On surrebuttal, however, PFS indicated that ConeTec, Inc., a Salt Lake City-based PFS subcontractor that has experience working with Bonneville clays, developed an N_k factor of 12.5 for the PFS site based on laboratory tests results provided by PFS. *See* Tr. 11955-62; PFS Exh. 238. Specifically, the N_k factor was determined based on the average of the individual N_k factors calculated from the laboratory shear strength tests performed on samples from borings B-1, B-3, B-4, C-2, CTB-N and CTB-S, and corresponding cone tip resistance values observed in the nearest CPTs. *See* PFS Exh. 238. Significantly, Mr. Trudeau indicated that the lowest or worst-case shear strength value obtained from the ConeTec calculations is consistent with the value that he would expect to exist under the pad based on the laboratory shear strength measurements made by PFS. Tr. 11962.

4.47. Further, Staff witness Dr. Ofoegbu testified that a site-specific correlation relating the CPT tip resistance data to actual values of undrained shear strength measured by direct shear tests is unnecessary. Tr. 11790-91. Rather, he concluded, "[a]ll [PFS] needed to show was that the value [of undrained shear strength] used for their design calculation was less than the value they could have used based on an interpretation of the CPT data." Tr. 11791. Like Mr. Trudeau, Dr. Ofoegbu found this to be the case, testifying that the value of soil strength used for the foundation-stability analyses is a lowerbound estimate of the applicable value. Tr. 11966; Ofoegbu Post Tr. 11001, at 12. Thus, the State's view of the data is at odds with that of PFS and the Staff.

4.48. The notion of an "applicable" value of soil strength reflects the fact that the actual shear strength of the soil under the cement-treated soil beneath a storage pad depends on the average strength of the soil in the area under the pad. Trudeau Rebuttal Post Tr. 11954, at 11. The existence of discrete pockets of weak soils would not adversely impact the validity of the PFS analyses because the foundations for the cask storage pads and the Canister Transfer Building are such wide foundations that the superstructure loads are distributed over a large volume of soil. Trudeau/Wissa Post Tr. 10834, at 15. Further, it is extremely unlikely that the average shear strength of the soil in the 30-ft. x 67-ft. area under a pad would be less than the minimum value measured by PFS. Trudeau Rebuttal Post Tr. 11954, at 11. Therefore, any effects that might be associated with shear strength values that are lower than the minimum value calculated by the PFS, assuming such lower values existed, would be minor and "localized" in nature. *See id.*

4.49. Further, assuming *arguendo* that there is some variability in the layer-1B soils that could result in shear strength values lower than the minimum value estimated by PFS, the evidence nonetheless leads us to conclude that such lower values would not result in an "unsafe sliding condition," as the State posits could occur. *See* Bartlett Post Tr. 11822, at 9. That is, the evidence demonstrates that there are sufficient countervailing conservatisms inherent in the Applicant's determination of the applicable undrained shear strength value and calculation of the factor of safety against sliding of the storage pads. *See* Trudeau Rebuttal Post Tr. 11954, at 1-5. These include, *inter alia*, the facts that: (1) the value of undrained shear strength measured in the laboratory for the U-1 sample is likely to be less than -- not greater than -- the *in situ* shear strength of that sample prior to its collection (*see id.* at 4-5);[34] (2) PFS calculated the minimum factor of safety against sliding of the pads without taking into account the increase in strength of clayey soils that occurs under cyclic dynamic loadings [*i.e.,* strain-rate effects] (*see i.d.* at 3); and (3) PFS calculated the minimum factor of safety against sliding of the pads without taking into account the passive resistance of the soil cement around the pads. *Id.*

4.50. Finally, the minimum factor of safety against sliding that PFS is seeking to achieve -- a value of 1.1 -- is the value that has been deemed acceptable by the NRC for structural foundations at nuclear power plants. *See* Tr. 11966; Staff Exh. EE, at 3.8.5-7. The evidence indicates, however, that there are no external safety-related connections to either the storage pads or casks of the kind that are required at nuclear power plants, which could rupture or become misaligned as a result of pad sliding. *See* Tr. 11966-67; Trudeau/Wissa Post Tr. 10834, at 13-14; Ofoegbu Post Tr. 11001, at 18; Staff Exh. C, at 2-60. Further, PFS's calculations demonstrate that

[34] Any measurement of the strength of soils that is obtained from laboratory testing on soil samples will disturb the samples to some degree and result in a strength measurement that is less than the actual strength that the soils exhibit *in situ*. Studies conducted at MIT found that carefully conducted, unconsolidated, undrained triaxial tests on high quality undisturbed samples of saturated clays yielded undrained shear strengths that ranged from 50% to 80% of field measured strengths. Trudeau Rebuttal Post Tr. 11954, at 4-5.

potential sliding of the storage pads under seismic loading does not constitute a safety hazard, and such sliding of the storage pads would actually tend to increase the stability of the casks against sliding or tipover. *See* Ofoegbu Post Tr. 11001, at 18; Staff Exh. C, at 2-60.

Estimates of Undrained Shear Strength Used in the Dynamic Bearing Capacity Calculations

4.51. In its prefiled direct testimony, the State contended that PFS has not demonstrated acceptable factors of safety against dynamic bearing capacity failure of the storage pad and CTB foundations. Bartlett Post Tr. 11822, at 10, 12. However, Dr. Bartlett later retreated from this view, ultimately conceding that: (1) "the calculated factor of safety of 1.17 against bearing capacity failure [of the storage pads] is conservative for the design basis earthquake" (Bartlett Surrebuttal Post Tr. 11982, at 2-3); and (2) the rebuttal testimony proffered by PFS "satisfactorily addresse[d]" his concerns relating to the Applicant's alleged use of "potentially unconservative estimates of the undrained shear strength in the dynamic bearing capacity calculations for the CTB." *Id.* at 4; *see* Trudeau Rebuttal Post Tr. 11954, at 12.

4.52. Staff witness Dr. Ofoegbu's testimony discussing the PFS bearing capacity analyses support a conclusion that PFS has demonstrated acceptable factors of safety against dynamic bearing capacity failure of the storage pad and CTB foundations. He observed that PFS calculated the bearing capacity of the storage pads using the undrained shear strength of layer-1B soil, *i.e.,* the weakest sublayer (about 3 to 10 feet below ground surface). Ofoegbu Post Tr. 11001, at 7. This was conservative because, under bearing capacity theory, the permissible value of undrained shear strength would consist of the *average* undrained shear strength through a depth of 30 feet below the base of the pads. *Id.* Similarly, for the CTB foundation, the permissible value would consist of the average undrained shear strength through a depth of 240 feet below the base of the foundation. 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 which

PFS used to obtain an average undrained shear strength value for the CTB bearing capacity calculations. *Id.*

4.53. Accordingly, based on our review of the testimony, and Dr. Bartlett's concessions set forth above, we conclude that PFS has demonstrated acceptable factors of safety against dynamic bearing capacity failure of the storage pad and CTB foundations.

Strain-Controlled Cyclic Triaxial Testing and Stress-Strain Behavior of the Native Soils

4.54. The State also asserted that (1) PFS should have included strain-controlled cyclic triaxial tests in its laboratory shear strength testing program, and (2) PFS has not adequately analyzed the stress-strain behavior of the native foundation soils under a range of cyclic strains imposed by the design earthquake. *See* Bartlett Post Tr. 11822, at 10-12. Of particular concern to the State is that PFS "ensure that there is no significant degradation of the shear strength at shear strain levels caused by the design basis earthquake" (*Id.* at 11) and that it "consider the magnitude of the cyclic strains imposed by the earthquake and the effects that these cyclic strains have on the soil's shear strength properties." *Id.* at 12. Based on our review of the evidence, as discussed herein, we find that PFS has satisfactorily addressed the State's concerns.

4.55. Although PFS did not conduct "strain-controlled" cyclic triaxial tests of the specific type called for by the State, it did perform "stress-controlled" cyclic triaxial tests to determine the collapse potential of the soils under repeated, cyclic dynamic loading. *See* Trudeau Rebuttal Post Tr. 11954, at 11. Significantly, the results of these tests did not show any degradation of the shear strength of the samples throughout 500 cycles of loading at extremely high cyclic ratios; and the resulting cyclic strains were very small, indicating essentially elastic response throughout the tests. *Id.* Accordingly, PFS concluded that, "since the cyclic stresses applied during the tests (500 cycles) are greatly in excess of those that take place during the design basis earthquake for the PFSF (approximately 7 to 11 cycles), no significant degradation of shear strength is anticipated to take place, and strain-controlled cyclic triaxial tests are unnecessary." *Id.*

4.56. The Staff's testimony supports this conclusion. Specifically, the Staff concluded that PFS had provided adequate information on the following aspects of stress-strain characteristics: (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 curves. Ofoegbu Post Tr. 11001, at 11. PFS provided upper and lower bounds of shear-wave velocity profiles, in addition to the best estimate soil profile. *Id.* at 9. Dr. Ofoegbu testified that the values of shear-wave velocity used to determine the elastic parameters for the soils account for the uncertainties of the average shear-wave velocity at the site. *Id.* at 10.

4.57. Staff witness Dr. Ofoegbu testified that geotechnical analysis of foundations, of the type conducted by PFS, is based on the strength of the foundation soils, not on the strain developed in those soils. *See* Tr. 11791-92. Dr. Ofoegbu also referred to the elastic behavior and lack of shear strength degradation that one would expect for the PFS subsoils under design earthquake loading conditions, testifying that the elastic assumption can be used to analyze the stress-strain behavior of the soils at the site because the soils would not reach their peak shear strength under such loading conditions. *See* Tr. 11792. Further, he testified that under loading conditions, "there is a combination of recoverable [elastic] deformation and non-recoverable [plastic] deformation," and that if the stress level is below the peak -- as PFS and the Staff maintain it would be -- then "the percentage of plastic deformation is very small compared to the total deformation." Tr. 11793. As such, the Staff concluded that the specific combination of 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 CTB foundation, and that strain-controlled cyclic triaxial tests of site soils are therefore not necessary. Ofoegbu Post Tr. 11001, at 10. The Staff also concluded that PFS has provided adequate information on stress-strain characteristics of the native foundation soils to demonstrate that the soil conditions are adequate for the proposed foundation loading. *Id.* at 11.

4.58. Further, among the tests PFS conducted are two sets of resonant column tests, which PFS has characterized as a form of strain-controlled, cyclic triaxial testing -- though not the same type of strain-controlled cyclic triaxial test referred to by the State. Trudeau/Wissa Post Tr. 10834, at 18. Mr. Trudeau testified that these tests achieved the same objectives sought by the State. *Id.* In particular, he noted that resonant column tests are capable of measuring shear moduli and damping characteristics over a wide range of strains, and that the resonant column tests which PFS performed were sufficient to cover the range of strains applicable for the design earthquake at the site. Tr. 11736.

4.59. PFS used the data from the resonant column tests in combination with information available in the literature to derive shear modulus degradation and damping curves. Ofoegbu Post Tr. 11001, at 11. The modulus degradation and damping versus strain curves were generated using accepted engineering practices and are consistent with other curves generated from comparable data. *Id.* at 12. These curves, in turn, were used as input to the site response analyses performed by Geomatrix. Trudeau/Wissa Post Tr. 10834, at 19. Specifically, Geomatrix 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. Ofoegbu Post Tr. 11001, at 11-12.

4.60. The results of the Geomatrix analyses indicate that the greatest *effective* shear strains occur for the layer-1B soils (depths of 5 to 12 feet), and range from 0.04% to 0.13%. Trudeau/Wissa Post Tr. 10834, at 19. Significantly, these values are within the range of strains measured in the resonant column tests. *Id.* PFS tested the sample obtained from layer-1B

(Sample U-3C, obtained from a depth of approximately 8 feet in Boring CTB-1) to shear strains as high as 0.07%; and it tested another sample (Sample U-7C, obtained from a depth of about 20 feet in Boring CTB-1) to shear strains as high as 0.15%. *Id.* at 18. Although these two samples were obtained from different depths within the same boring, a comparison of the plots of shear modulus degradation and damping versus shear strain derived in part from the two sets of test data indicates that they are very similar. *Id.* Therefore, it is reasonable to extrapolate the results from the testing of Sample U-3C along the same curves as those measured in the resonant column testing of Sample U-7C. *Id.* at 18-19. State witness Dr. Bartlett conceded as much, stating that he does not have "too much of an issue extrapolating the shear modulus and damping curves to high strain levels for the ground response analysis, because there are other published curves upon which one can make this extrapolation in a reasonable manner." Tr. 11991-92.

4.61. In sum, PFS has provided adequate information on stress-strain characteristics of the native foundation soils. Through "stress-controlled" cyclic triaxial tests, PFS demonstrated that the soils of interest are not likely to experience any significant degradation of shear strength. In addition, PFS performed resonant column tests, the results of which encompass the range of strains applicable to the PFS site subsoils for design earthquake conditions. Based on the evidence presented, we find that PFS has adequately analyzed the stress-strain behavior of the native foundation soils under these conditions, and that "strain-controlled" cyclic triaxial tests of the type referred to by the State are not necessary.

Soil Anisotropy and Triaxial Extension Testing

4.62. Finally, the State also asserts that the Applicant's laboratory shear strength testing program should have included strain-controlled triaxial extension tests due to the allegedly "anisotropic shear strength properties" of the upper Lake Bonneville sediments. *See* Bartlett Post Tr. 11822, at 11. Dr. Bartlett testified that these soils are strongest in triaxial compression and weakest in triaxial extension, and that PFS primarily used triaxial compression tests to calculate the soil's resistance to bearing capacity failure. *Id.* This aspect of the PFS approach, he claimed, "is unconservative and overestimates the average shear resistance along the potential failure plane if significant anisotropy is present," and "has the greatest significance in analyzing the bearing capacity of the storage pads." *Id.* at 11-12.[35]

4.63. First and foremost, we note that Dr. Bartlett conceded that "the calculated factor of safety of 1.17 against bearing capacity failure of [the storage pads] is conservative for the design basis earthquake." Bartlett Surrebuttal Post Tr. 11982, at 2-3. This concession would appear to moot -- or at least severely undermine -- his argument that triaxial extension testing is necessary in analyzing the bearing capacity of the storage pads. Nonetheless, we find that PFS and the Staff have sufficiently addressed the State's concerns regarding triaxial extension testing and the alleged anisotropic shear strength properties of the upper Lake Bonneville soils.

4.64. According to PFS, triaxial extension tests typically are not performed to assess the bearing capacity of foundations. Trudeau/Wissa Post Tr. 10834, at 19-20. Rather, they typically are used to assess situations where foundation soils are unloaded, such as at the base of deep excavations, or to determine the strength applicable for soils at the toes of slopes that might be subject to a deep, circular arc-type failure. *Id.* at 20. Neither of these situations is present at the PFS site, which is essentially level and will require only very shallow excavations. *Id.*

4.65. Furthermore, the State's perceived need for triaxial extension testing appears to stem largely from the alleged "anisotropic shear strength properties" of the upper Lake Bonneville soils, which Dr. Bartlett attributes to the "microfabric" formed by the "distinct layering" of clays and silts in these soils. *See* Tr. 11831-32; 11983-84. In contrast, Staff witness Dr. Ofoegbu testified that "there is evidence that the anisotropy actually does not exist," and that "even if [it] were to exist,

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[35] Anisotropy is the exhibition of a property that has different values when measured along axes in different directions. *See* State Exh. 103; Tr. 11983-85; *see also* Webster's New Collegiate Dictionary, G. & C. Merriam Co. (1977).

it would be of no concern," because if failure conditions were to occur at the PFSF site, a potential failure surface would "zig-zag" through the soil as it proceeded to find the "weakest link" in the soil. *See* Tr. 12018, 12021; Staff Exh. ZZ. That is, any "micro-level" heterogeneity in the soils "would drive a potential [ ] failure surface to go from one layer to another in a real case." *Id.* The result, he testified, would be a "composite failure surface" -- portions of which would be "inclined" and not parallel to the base of the pads -- along which the resistance to sliding can be represented by the strength of the soil measured in (1) confined compression, (2) triaxial extension, and (3) direct shear. Tr. 12018-19; 12031. Further, on average, over the entire failure surface, the undrained shear strength of the soil measured using any of the available methods gives a representation of the strength of the soil along that failure surface. Tr. 12019.

4.66. In determining the value of undrained shear strength for use in its foundation stability analyses, PFS performed compression tests, direct shear tests, and in situ tests using the cone penetrometer. *See* Tr. 12020. Dr. Ofoegbu testified that the undrained shear strength determined by PFS using these methods is suitable, even given shear strength anisotropies that might exist in the soils. Tr. 12020-21. Because we find the Staff's testimony on this issue to be persuasive, we decline to accept the State's assertion that triaxial extension tests are necessary.

2. THE USE OF CEMENT-STABILIZED SOILS

4.67. PFS has proposed to use cement-stabilized soils[36] at its facility to improve subsurface conditions in the vicinities of the cask storage pads and CTB. *See* Trudeau/Wissa Post Tr. 10834, at 23-24. Specifically, PFS intends to excavate the surficial eolian silt -- which, in its *in situ* state, is not suitable as a foundation for these structures -- and to mix it with sufficient portland cement and water to form "soil-cement" and "cement-treated soil." *See id.* at 23; PFS Exh. JJJ,

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[36] Like the Applicant, we use the term "cement-stabilized soils" here to refer to both "soil-cement" and "cement-treated soil." These terms are more fully defined, *infra*.

at 2.6-108. The primary purpose is to form a strong cement-stabilized soil subgrade underneath and around the cask storage pads, as well as around the CTB. *See id.* In essence, from an engineering standpoint, PFS is relying on the cement-stabilized soils to improve the shear and compressive strengths of the surficial native soils at the site. Trudeau/Wissa Post Tr. 10834, at 24.

4.68. According to PFS, the use of cement-stabilized soils at the proposed site would serve three specific purposes. *Id.* at 23.[37] First, in the area directly underneath the concrete storage pads upon which the storage casks rest, cement-treated soil is to be used as a cohesive material that will be strong enough to resist the sliding forces generated by the design basis earthquake. The cement-treated soil is intended to provide bonding with the bottom of the concrete storage pad above it and with the clay soils beneath, so as to transfer the horizontal earthquake forces downwards from the pad and into the underlying clay soils. *Id.*

4.69. Second, soil cement is to be used in the area around and between the cask storage pads. In these areas, the intended function of the soil cement is to support the weight of the transporter vehicle that would be used to deliver storage casks to the pad area. *Id.*

4.70. Third, soil cement is to be placed around the CTB foundation mat, extending outward from the mat to a distance equal to the associated mat dimension. Here, the intended function is to provide additional passive resistance[38] against sliding forces acting on the CTB foundation mat in the event of a design basis earthquake. *Id.*

4.71. The cement-treated soil underlying the pads will have a minimum unconfined compressive strength of 40 psi. *Id.* at 24. This layer of cement-treated soil must be no greater

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[37] As noted by PFS, mixing cement with the eolian silts also allows these soils to be utilized in the construction of the facility, thereby avoiding waste of these materials and the need to replace them with structural fill. Trudeau/Wissa Post Tr. 10834, at 23.

[38] Passive resistance is a term that refers to the ability of soils to resist horizontal forces, which in this case, are the result of earthquake ground motions. Trudeau/Wissa Post Tr. 10834, at 23-24.

than 2-feet thick and have a Young's modulus (modulus of elasticity) less than or equal to 75,000 psi. *Id.* at 25. This is to ensure that the decelerations associated with a hypothetical cask tipover event or vertical end drop accident do not exceed the HI-STORM 100 cask system's design criteria. PFS Exh. JJJ, at 2.6-108a; Tr. 10992. PFS has committed to demonstrate through testing that the stiffness of the cement-treated soil under the pads will not exceed 75,000 psi. Trudeau/Wissa Post Tr. 10834, at 33-34 (citing Section 2.6.4.11 of the SAR); Ofoegbu Post Tr. 11001, at 13.

4.72. The soil cement to be placed around and between the cask storage pads will have a thickness of 28 inches, *i.e.,* it will extend upwards from the bottom of the storage pads to a level that is 28 inches above the bottoms of the 36-inch thick storage pads. Trudeau/Wissa Post Tr. 10834, at 25; PFS Exh. JJJ, at 2.6-108a. The remaining 8 inches, from the top of the soil cement to grade, will be filled with coarse aggregate. Trudeau/Wissa Post Tr. 10834, at 25. This aggregate will be placed and compacted to be flush with the tops of the pads so as to permit easy access by the cask transporter. Since it will be within the frost zone, the soil cement placed adjacent to the storage pads must have minimum unconfined compressive strength of at least 250 psi to ensure that it meets the applicable durability requirements (*i.e.,* wet/dry and freeze/thaw tests). *Id.*

4.73. The soil cement to be placed around the CTB will be 5-feet thick and overlain by eight inches of compacted coarse aggregate similar to that intended for use in the pad emplacement area. Trudeau/Wissa Post Tr. 10834, at 25. This soil cement must also have a minimum unconfined compressive strength of at least 250 psi, to ensure that the soil cement can (1) meet the applicable durability requirements and (2) provide the additional passive resistance required to achieve an adequate factor of safety against sliding of the CTB in the event of a design basis earthquake. *Id.* PFS has also committed to demonstrate through testing that the soil cement around the CTB will have a minimum unconfined strength of 250 psi. Trudeau/Wissa Post

Tr. 10834, at 33-34; Ofoegbu Post Tr. 11001, at 14. Staff witness Dr. Ofoegbu indicated that this is an achievable value. Tr. 11027.

4.74. The appropriate soil-cement formulation for each of the aforementioned applications will be established through a laboratory testing program. Trudeau/Wissa Post Tr. 10834, at 25. PFS is conducting this program in accordance with a document entitled "Engineering Services Scope of Work for Laboratory Testing of Soil-Cement Mixes, ESSOW 05996.02-G010 (2001) ("ESSOW") (PFS Exh. GGG). *Id.* at 25-26. The laboratory testing program is being conducted by a PFS contractor, Applied Geotechnical Engineering Consultants, Inc. ("AGEC"), in accordance with the ESSOW. *Id.* at 30. This includes full compliance by AGEC with the Quality Assurance (QA) Category I requirements of the ESSOW. PFS Exh. JJJ, at 2.6-109; Tr. 10968.

4.75. As set forth in the ESSOW, the laboratory testing program being implemented by PFS to develop soil-cement mixtures that meet applicable design requirements is in accordance with well-established regulatory guidance and industry standards. Trudeau/Wissa Post Tr. 10834, at 31. In particular, the ESSOW cites RG 1.138 as a source of guidance with respect to laboratory test methods for soils, in addition to numerous other standards issued by the American Society for Testing and Materials ("ASTM") and the Portland Cement Association. *Id.* at 29. In addition, PFS has committed to follow the standards, procedures, and recommendations contained in the industry standard publication "State-of-the-Art Report on Soil Cement," American Concrete Institute Report ACI 230.1R-90 (1998) ("ACI Committee 230 Report") (PFS Exh. HHH) with respect to mix proportioning, testing, construction and quality control for soil cement. Trudeau/Wissa Post Tr. 10834, at 29.

4.76. PFS witness Dr. Wissa, who is one of the developers of the ACI Committee 230 Report, testified that the design, placement, testing and performance of soil cement are well-established technologies, and that this fact provides reasonable assurance that the program proposed by PFS can be executed successfully. *Id.* at 32. 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. Ofoegbu Post Tr. 11001, at 18-19.

4.77. The ESSOW sets forth a series of tests to be conducted in several phases. Trudeau/Wissa Post Tr. 10834, at 26. These tests include, *inter alia*, soil index property tests, moisture-density tests, and durability tests. *Id.* PFS intends to conduct additional tests beyond those identified in the Laboratory Testing ESSOW. *Id.* For instance, it has committed to performing direct shear tests to demonstrate that adequate bond strength exists at the interfaces between the in situ clay and cement-treated soil and between the cement-treated soil and the bottom of the cask storage pads. *Id.* at 26, 29, 34.

4.78. The index property tests are used to ascertain basic properties of the site soils, including, *inter alia*, water content, the Atterberg limits (*i.e.,* liquid and plastic limits), and particle size and gradation. *Id.* at 26. The water contents of the soils are determined in accordance with ASTM D2216, whereas the Atterberg limits of the soils are measured in accordance with ASTM 4318. *Id.* Sieve analyses (ASTM D422 and D1140) and hydrometer analyses (ASTM D422) are used to determine the gradation of particle sizes and the percentages of various clay-sized particles, respectively, in the soil samples. *Id.* AGEC has provided preliminary test results for the index property tests; though preliminary, Dr. Wissa indicated that these tests "appear to be reliable and adequate to describe the on-site surficial soils that will be stabilized with cement." *Id.* at 30.

4.79. Moisture-density tests, which are conducted in accordance with ASTM D558, establish for each soil-cement mixture the relationship between the moisture content of the mixture and the resulting density when the mixture is compacted. *Id.* at 27. In particular, these tests establish the optimum moisture content and maximum density for molding laboratory test specimens for further testing. AGEC has provided preliminary results for these tests. *Id.* at 30.

4.80. Once PFS has identified those soil-cement mixes with the optimal combination of properties, it will perform durability tests in accordance with ASTM D559 and D560 to determine the durability of soil cement specimens subjected to repeated cycles of exposure to the elements during extreme conditions. *Id.* at 27. These tests include "wet-dry" and "freeze-thaw" tests to determine moisture/volume changes and soil cement losses due to (1) repeated exposures to inundation and drying and (2) alternate cycles of freezing and thawing. *Id.* at 27-28. PFS witness Trudeau testified that "successful completion of the durability tests establishes that the soil cement mixture tested is adequate to provide a durable soil cement mix, one that will not lose compressive strength over time due to the effects of weather and normal wear and tear." *Id.* at 28.[39]

4.81. PFS indicated that AGEC has performed a set of durability tests, but that a review of these tests determined that they failed to demonstrate the durability of the tested samples. *Id.* at 30. PFS witness Trudeau opined that this failure was likely due to insufficient compaction of the test specimens prior to performance of the tests. *Id.* He further testified that the test program is currently on hold, pending determination of the causes for the failure of the durability tests that were performed by AGEC. *Id.*

4.82. The next step in the proposed soil-cement testing program is the performance of compressive strength tests in accordance with ASTM D1633 and D558. *Id.* at 28. Specifically, for those soil cement mix formulations shown to meet the durability tests, compressive strength tests will be performed on cured test specimens to determine whether the formulations meet the design requirements for compressive strength. *Id.* If the compressive strength of a given soil cement sample is determined to be adequate, then the soil-cement mixture will be deemed appropriate for use at the PFS site. *Id.*

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[39] The cement-treated soil to be placed under the cask storage pads will not be subjected to durability tests because it is to be located beneath the three-foot thick concrete pads and is therefore not exposed to the elements. Trudeau/Wissa Post Tr. 10834, at 28. Also, the cement-treated soil would not be susceptible to freezing and thawing cycles due to its location below the depth of frost penetration at the PFS site. *Id.*

4.83. Finally, as noted above, the cement-treated soil will be subject to direct shear tests to confirm that the bond (a) at the interfaces between the concrete bottom of the cask storage pad and the cement-treated soil, (b) at the interfaces between lifts of cement-treated soil, and (c) at the interfaces between cement-treated soil and the in situ clayey soil, exceed the strength of the clay soils at the site. *Id.* at 29. According to PFS, such confirmation will demonstrate that the cement-treated soil provides sufficient resistance against seismic sliding forces. *Id.*

4.84. Following completion of the testing phase, PFS will develop procedures for the placement and treatment of the soil cement/cement-treated soil, lift surfaces, and foundation contact in accordance with the recommendations of the ACI Committee 230 Report. *Id.* at 31; PFS Exh. JJJ, at 2.6-118. Specific construction techniques and field quality control requirements will also be identified in the construction specifications developed by PFS during this phase of the project. *Id.* at 2.6-118.

4.85. To ensure that sufficient bonding is achieved, PFS will utilize the techniques described in the ACI Committee 230 Report and DeGroot, G., 1976, "Bonding Study on Layered Soil Cement," REC-ERC-76-16, U.S. Bureau of Reclamation, Denver, CO, September 1976. Trudeau/Wissa Post Tr. 10834, at 31; PFS Exh. JJJ, at 2.6-118. These techniques include, *inter alia*, (1) minimizing the time between placement of successive layers or "lifts" of soil cement, which will have a compacted thickness of approximately six inches; (2) moisture conditioning to facilitate the proper curing of the soil cement; (3) producing a roughened surface on the soil cement prior to the placement of additional lifts or concrete foundations; and (4) using a dry cement or cement slurry to enhance the bonding of concrete or new soil cement layers to underlying layers that have already set. *Id.*

Specific Challenges to the Applicant's Proposed Uses of Cement-Stabilized Soils

4.86. The State has challenged various aspects of the Applicant's proposal to use cement-stabilized soils as part of its cask storage pad and CTB foundation designs. The State's numerous concerns appear to fall into five general categories: (1) The specific applications of cement-stabilized soils proposed by PFS are "unique" and without "direct precedent" (*see* Bartlett/Mitchell Post Tr. 11033, at 3); (2) the existing soil cement program developed by PFS is inadequate to provide "proof of its design concept" (*see id.* at 13); (3) PFS has not performed the tests and analyses necessary to demonstrate that the cement-stabilized soils will perform their intended design functions (*see id.* at 3, 5); (4) PFS has not demonstrated that it can achieve the specific combination of strength and stiffness properties required for the cement-stabilized soil that would underlie the storage pads (*see id.* at 10); and (5) PFS has not adequately evaluated the potential impacts of construction and placement of the soil cement and cement-treated soil on the underlying native soils. *See id*. at 3, 11-13. We evaluate the State's concerns below.

Precedent for the Specific Applications of Cement-Stabilized Soil Proposed by PFS

4.87. The State asserts that the proposal by PFS "to use soil-cement and cement-treated soil to provide additional seismic sliding resistance and stability to shallowly embedded foundations subjected to intense strong ground motion is a new and unique application of this technology." Bartlett/Mitchell Post Tr. 11033, at 3. Hence, the State characterizes the Applicant's proposed uses of cement-stabilized soils as lacking "precedent." *Id.* at 3, 5-6. The State equates "precedent" with the existence of "case histories," which it argues demonstrate "how the system actually responded and performed after an earthquake." Tr. 11263.

4.88. Applicable NRC regulations do not explicitly require an applicant to use "case history precedent" in demonstrating that subsurface conditions are adequate for the proposed foundation loadings, or that SSCs important to safety will perform their intended design functions. *See* Ofoegbu Post Tr. 11001, at 13; 10 C.F.R. §§ 72.102(d), 72.122(b). Rather, an applicant must show that the proposed design satisfies the regulatory requirements. *See* Tr. 11016. This includes an assessment of whether a material with a specified property would be adequate for the proposed foundation loading, and whether the specified property is achievable for that material based on available information. *See* Tr. 11018.

4.89. The value of a particular application as a precedent is not the specific use that is made of the soil cement, but whether the application draws upon the same mechanical properties of the soil cement. Trudeau/Wissa Rebuttal Post Tr. 11232, at 1. With regard to its design, PFS proposes to use the shear strength and compressive strength of the soil cement to resist sliding for the CTB and the shear strength of the cement-treated soil under the storage pads to resist sliding due to the seismic loadings. *Id.* Mr. Trudeau testified that the shear and compressive strength of soil cement are well-known and easily measured properties, "relied upon in numerous previous applications." *Id.* These are properties that "any other kind of material can have," including compacted soil, concrete, or steel. Tr. 11003-04.

4.90. It is clear from the evidence presented that the material properties being relied upon by PFS in its design -- *i.e.,* shear strength and compressive strength -- have in fact been relied upon in numerous previous applications.[40] For example, though its primary use has been as a base material underlying bituminous and concrete pavements, soil cement has also been used in providing slope protection for dams and embankments, and in constructing liners for channels, reservoirs, and lagoons; coal-retaining berms; and mass soil-cement placements for dikes and foundation stabilization. *See* Trudeau/Wissa Post Tr. 10834, at 22; Trudeau/Wissa Rebuttal Post Tr. 11232, at 1-2; PFS Exh. HHH, at 230.1R-3. PFS considers its proposed uses of cement-stabilized soils to constitute additional examples of foundation stabilization. Trudeau/Wissa Post Tr. 10834, at 32-33; Tr.10845.

4.91. With respect to foundation stabilization in particular, soil cement has been used as a massive fill to provide foundation strength and uniform support under large structures. *See* PFS

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[40] State witness Mitchell acknowledged that soil cement has "been used for [a] larger or increasing number of purposes over time." Tr. 11190.

Exh. HHH, at 230.1R-5. For example, in Koeberg, South Africa, soil cement was used to replace an approximately 18 ft thick layer of medium-dense, liquefiable saturated sand under two 900-MW nuclear power plants. *See id.* According to PFS witness Trudeau, the shear strength of the loose sandy soils at Koeberg was increased by the use of soil cement to preclude potential liquefaction due to seismic shear stresses. Trudeau/Wissa Post Tr. 10834, at 33; Trudeau/Wissa Rebuttal Post Tr. 11232, at 3.[41]

4.92. Additionally, a more recent example includes the use of deep soil-cement buttresses to restrain large lateral loads and form permanent foundations for the five highway tunnels for Interstate-90 and Interstate-93, that converge at the Fort Point Channel crossing of the Boston Central Artery/Tunnel Project ("CA/T Project"). *See* Trudeau/Wissa Rebuttal Post Tr. 11232, at 2. In this instance, resistance to lateral loads is provided by the shear and compressive strength of the deep soil-cement buttresses.[42] *Id.*

4.93. We conclude that the foregoing examples -- whether or not one considers them to be "precedent" for the PFS design -- provide sufficient reason to believe that cement stabilization can be effectively used to enhance the engineering characteristics (*i.e.,* shear and compressive strengths) of the natural soils in the manner called for by the PFS design. Indeed, State witness Dr. Mitchell stated that he did not see anything inherently wrong with the Applicant's proposed uses of soil cement, and that "[i]t appears that this is a situation where cement-treated soil and soil

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[41] Dr. Mitchell distinguished the Koeberg and PFS sites as not analogous, because the soils at Koeberg were loose saturated sands, whereas the soils at PFS are plastic, fine-grained cohesive materials that are not liquefiable. Bartlett/Mitchell Post Tr. 11033, at 6. Further, he testified that PFS intends to use cement-stabilized soils to provide sliding resistance and buttressing, not to prevent liquefaction, unlike the Koeberg uses. *Id.* He conceded, however, that cement was used to increase the cohesion (a component of shear strength) of the native soils at Koeberg. Tr. 11267.

[42] Dr. Mitchell testified that the Boston CA/T Project involved "deep soil mixing," and "a very thick layer" of "soft clay soil at a quite high water content," "with considerably different properties." Tr. 11257-58. He conceded, however, that soil cement has been and is being used to resist lateral earth pressures, and that "there are some similarities" in the PFS and Boston CA/T Project applications, insofar as they both involve resistance to lateral loads. Tr. 11193.

cement could be used for the intended purposes." Tr. 11054, 11187. Accordingly, we do not find the asserted lack of "case history precedent" to be dispositive with respect to the viability or acceptability of the proposed PFS design.

Adequacy and Timing of the PFS Soil Cement Testing Program

4.94. The State maintains that "PFS's proposal to conduct a soil cement testing program after, rather than before, it obtains a license will not prove the design concept that will form the basis of a licensing decision." Bartlett/Mitchell Post Tr. 11033, at 3. Further, the State claims that "if in the future PFS finds that soil cement and cement-treated soil will not support PFS's seismic design, then the licensing basis for approving the PFS facility design will be invalid." *See id.* at 5. We recognize the State's preference that PFS "perform testing and analyses now rather than at some future date." *Id.* However, as State witness Dr. Mitchell stated, while conducting such tests prior to licensing may prudent from an engineer's viewpoint, that does not equate to a regulatory requirement. *See* Tr. 11097-100. For the reasons cited below, we do not find that soil cement testing must be performed prior to issuance of a license for the proposed PFS facility.

4.95. PFS has formulated a soil cement testing program; identified design criteria; specified the test standards, methodologies, and acceptance criteria to be used; and performed some testing to date. Trudeau/Wissa Post Tr. 10834, at 33-34. Its testimony establishes that the properties of the soil cement are within well-established, attainable parameters, and will be achieved in accordance with standard industry procedures. *Id.* at 34. In addition, the construction techniques available for ensuring proper placement and curing of the soil cement, and for preventing disturbance of the underlying soils, have been utilized in numerous construction projects. *Id.*

4.96. Although he was not involved in developing the test program, PFS witness Dr. Wissa reviewed the soil cement laboratory testing program developed by PFS and the standards and methodologies it contains. *Id.* at 30. He testified that if properly implemented, the program will lead to the identification of suitable soil cement and cement-treated soil mixes and construction specifications that will meet the specified design requirements and give adequate performance for the life of the proposed facility. *Id.* Similarly, Staff witness Dr. Ofoegbu testified that there is information in the literature which indicates that the soil-property changes that result from cement-stabilization can be considered long-lasting. Ofoegbu Post Tr. 11001, at 14.

4.97. PFS has committed to developing a soil-cement mix design using standard industry practice, and to performing a soil-cement testing program in accordance with specified industry standards. *Id.* at 35. This program follows industry-accepted protocols designed to address environmental factors that may affect the long-term performance of the cement-stabilized soils. *Id.* Significantly, PFS has committed to demonstrate through appropriate testing that any Staff-approved soil cement/cement-treated soil design specifications will be achieved. *See* Ofoegbu Post Tr. 11001, at 20. Significantly, in response to an inquiry from the Board as to whether there is anything that "necessarily precludes [PFS] from coming up with the right mix of soil-cement and cement-treated soil," State witness Dr. Mitchell stated, "I am unaware of any at this point." Tr. 11212.[43]

4.98. The Staff has reviewed the design submitted by PFS, including the associated calculations and specified material properties, to determine compliance with the applicable regulatory requirements. *See* Tr. 11016. The Staff determined that the analyses submitted by PFS demonstrated that the design would be safe, and that based on information available in the

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[43] We find no reason to believe that PFS will not adhere to commitments that have been made as part of its license application and reiterated in statements made by its witnesses under oath. *Cf. Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), LBP-00-35, 52 NRC 364, 405-06, 410 (2000) (in the absence of evidence to the contrary, the Board will not presume that an applicant or licensee, and those who work for them, will not adhere to applicable regulations or standards). Moreover, applicants or licensees that make material false statements in an NRC proceeding run the risk of criminal penalties and agency enforcement actions. *See* Tr. 10958; *see generally*, 10 C.F.R. §§ 72.12, 72.60; Atomic Energy Act of 1954, as amended, §§ 186, 223 and 234, 42 U.S.C. §§ 2236, 2273, 2282.

technical literature, the material properties used by PFS in its design are achievable. *See* Tr. 11021. The Staff's analysis is documented in the Consolidated SER (Staff Exh. C). Tr. 11022. In this regard, the Staff concluded that PFS has "proven its design," even without having completed the soil cement testing program. Tr. 11021.

4.99. As noted above, the soil cement laboratory testing program will be conducted by the PFS contractor in accordance with the laboratory testing ESSOW, which includes full compliance by the contractor with the QA Category I requirements of the ESSOW. *See* PFS Exh. JJJ, at 2.6-109; Tr. 10968. Applicable NRC quality assurance requirements are set forth in Subpart G of 10 C.F.R. Part 72 (§§ 72.140 - 72.176). These requirements apply, *inter alia*, to the testing of SSCs important to safety, and include provisions for design control, control of the test program, and QA audits. *See* 10 C.F.R. §§ 72.140 and 72.176; *see also* Tr. 10958-59. Additionally, any post-licensing construction activities conducted by PFS would be subject to NRC Staff inspection. *See* 10 C.F.R. § 72.82; Tr. 10947.

4.100. Finally, PFS, like all NRC applicants and licensees, has a duty to inform the NRC of any information identified by it as having "a significant implication for public health and safety or common defense and security." *See* 10 C.F.R. § 72.11; Tr. 10958. Additionally, if in the course of its post-licensing testing activities, PFS perceives a need to modify its testing procedures or evaluations demonstrating that the intended functions of SSCs will be accomplished, it would have to do so in accordance with 10 C.F.R. § 72.48. *See*, *e.g.*, Tr. 10959, 10996. This could require a license amendment, in appropriate circumstances. *See* Tr. 10996.[44]

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[44] 10 C.F.R. § 72.48 ("Changes, tests and experiments") provides, in pertinent part:

> (c)(1) A licensee . . . may make changes in the facility . . . as described in the FSAR. . . , make changes in the procedures as described in the FSAR . . . , and conduct tests or experiments not described in the FSAR . . . , without obtaining . . . [a] license amendment pursuant to §72.56 . . . if: (A) A change to the technical

(continued...)

<u>Performance of Site-Specific Testing and Analyses</u>

4.101. The State also asserts that PFS has not conducted the site-specific testing and analyses necessary to determine whether the cement-stabilized soil will perform their intended design functions. *See* Bartlett/Mitchell Post Tr. 11033, at 5. In support of its argument, the State maintains that PFS has failed to consider properly the potential effects of a number of postulated environmental mechanisms or loading phenomena, which the State posits could crack or otherwise degrade the cement-stabilized soils and consequently impede performance of their intended

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[44](...continued)

> specifications incorporated in the specific license is not required; . . . and (C) The change, test, or experiment does not meet any of the criteria in paragraph (c)(2) of this section.
>
> (2) A specific licensee shall obtain a license amendment pursuant to §72.56, . . . prior to implementing a proposed change, test, or experiment if the change, test, or experiment would:
>
> > (i) Result in more than a minimal increase in the frequency of occurrence of an accident previously evaluated in the FSAR . . . ;
> >
> > (ii) Result in more than a minimal increase in the likelihood of occurrence of a malfunction of a system, structure, or component (SSC) important to safety previously evaluated in the FSAR. . . ;
> >
> > (iii) Result in more than a minimal increase in the consequences of an accident previously evaluated in the FSAR . . . ;
> >
> > (iv) Result in more than a minimal increase in the consequences of a malfunction of an SSC important to safety previously evaluated in the FSAR . . . ;
> >
> > (v) Create a possibility for an accident of a different type than any previously evaluated in the FSAR (as updated);
> >
> > (vi) Create a possibility for a malfunction of an SSC important to safety with a different result than any previously evaluated in the FSAR . . . ;
> >
> > (vii) Result in a design basis limit for a fission product barrier as described in the FSAR . . . being exceeded or altered; or
> >
> > (viii) Result in a departure from a method of evaluation described in the FSAR . . . used in establishing the design bases or in the safety analyses.

functions. *See* Subpart C.3.d. of Unified Contention Utah L/QQ; Bartlett/Mitchell Post Tr. 11033, at 8-10. These include: (1) interference with cement hydration by salts and sulfates present in the native soils; (2) shrinkage cracking during curing and drying; (3) frost penetration and expansion cracking: (4) settlement cracking resulting from differential settlement at the perimeter of the pads and the CTB mat foundation; (5) cracking or overstressing due to vehicle loads; (6) delamination or debonding along the various material interfaces during a seismic event; and (7) bending, torsional, and tensile stresses due to dynamic loading conditions and inertial interaction.

Presence of Salts and Sulfates in the Native Soils

4.102. In Subpart C.3.d.(iv) of the unified contention, the State asserts that the presence of salts and sulfates in the native soils could interfere with hydration of the cement used in constructing the cement-stabilized soils. For the reasons indicated below, however, this potential problem – to the extent it might exist – does not strike us as significant, in light of the Applicant's proposed test program and available remedial construction measures.

4.103. First, preliminary testing of the site soils for sulfates indicates that very low levels of sulfate are present in the soils of concern, *i.e.*, the eolian silts that will be treated with cement. *See* Trudeau/Wissa Post Tr. 10834, at 43-45. Second, in the event that higher concentrations of sulfates were to be present and interfere with the cement hydration process, the problem would be evidenced by the failure of the soil cement test sample to pass the required durability tests. *Id.* at 45. Third, the effects of any salts or sulfates in the native soils would necessarily be considered in the mix design. Ofoegbu Post Tr. 11001, at 19. For example, should the presence of salts or sulfates be determined to be a concern, there are a number of measures that can be used to address the problem, including use of a sulfate resistant cement, increasing the treatment levels, or conducting chemical treatment of the soil (*e.g.*, adding barium compounds or lime to the mix). Trudeau/Wissa Post Tr. 10834, at 45.

Shrinkage Cracking During Curing and Drying

4.104. Of the potential cracking mechanisms identified by the State, the State characterizes shrinkage cracking of the soil cement between and around the pads and around the CTB foundation to be "of most concern." Bartlett/Mitchell Post Tr. 11033, at 8-9. The State indicates that shrinkage cracks form during the process of curing and aging of soil cement, and are relatively thin, generally vertical to subvertical cracks. Bartlett/Mitchell Post Tr. 11033, at 9. State witness Dr. Mitchell indicated that these cracks "are the ones most likely to form." Tr. 11111-12. Further, the State argues that if these cracks are somewhat continuous, the tensile resistance will be completely lost along the surface of the crack; and it posits that such a "loss of tensile capacity in the mat is extremely deleterious when the mat has to resist dynamic tensile stresses." Bartlett/Mitchell Post Tr. 11033, at 9. Of particular concern to the State is the postulated reduction in the soil cement mat's capacity to act as an integral mat and resist "out-of-phase motion," and the consequent introduction of "inertial interaction." *Id.* [45]

4.105. We are unpersuaded by the State's arguments relative to shrinkage cracking, from two perspectives. First, we are not convinced that continuous shrinkage cracks would in fact occur in the soil cement. Second, as discussed below in connection with bending and tensile stresses, we are not persuaded that such cracks -- if they occurred -- would have "deleterious" effects on the performance of the soil cement or impair any safety function of the relevant structures.

4.106. With respect to shrinkage crack formation, PFS has committed to taking appropriate measures during the soil cement installation and curing process to minimize the potential for crack formation and to take appropriate remedial measures to address any significant cracks that form

[45] The State's concerns about possible "out-of-phase" motion and "inertial interaction" are directed at the adequacy of the Applicant's dynamic analyses, and were raised in Part D of this contention. As such, they were addressed by the parties primarily in that context. Therefore, while we address the implications of dynamic stresses with respect to soil cement integrity and performance to some extent here, we limit that discussion accordingly.

during curing and drying. Trudeau/Wissa Rebuttal Post Tr. 11232, at 6. For example, as PFS noted, there are shrink-resistant types of cement (Type K cements) which can be used to inhibit the formation of cracks. Trudeau/Wissa Post Tr. 10834, at 39. Moreover, during the curing, a sealing coat, such as a geomembrane, can be put on the soil cement to minimize the formation of cracks. *Id.* Finally, PFS would have the opportunity to seal such cracks in the soil cement surrounding the CTB prior to placement of the layer of compacted aggregate in this area. *Id.* at 40.

4.107. Any shrinkage cracks that might form would be near vertical in orientation and only a fraction of an inch wide, thereby making any potential effects on the compressive strength or shear strength of the soil cement negligible. Trudeau/Wissa Rebuttal Post Tr. 11232, at 6. Further, shrinkage cracks are unlikely to run continuously through the soil cement layers, given the thicknesses of these layers (28 inches and 5 feet). *Id.* State witness Mitchell, in speaking generally about the various cracking mechanisms identified by the State, agreed that “[i]t could be unlikely that you would have one continuous crack going all the way through [the soil cement and cement-treated soil layers].” Tr. 11111. Also, the soil cement layers are to be constructed in lifts, with each lift being laid down separately and bonded with cement to the next applied lift, so as to allow each lift to cure independently of each other. Trudeau/Wissa Rebuttal Post Tr. 11232, at 6-7. This would further impede any crack continuity. These considerations are significant, given the State’s recognition that “[f]rom a seismic performance standpoint, the real issue is not thickness of the crack, but its potential for continuity.” Bartlett/Mitchell Post Tr. 11033, at 9.

Frost Penetration and Expansion Cracking

4.108. Another mechanism for potential cracking of the soil cement and cement-treated soil referred to by the State is frost penetration and expansion cracking, which the State attributes to a significant number of freeze-thaw cycles at the PFS site. *Id.* at 8. However, the evidence shows that PFS has anticipated this potential cracking mechanism and has committed to take appropriate countervailing measures. As discussed below, we find that the commitments made by PFS, in addition to certain other mitigating factors, address the State's concern.

4.109. First, the soil cement mixture to be used at the PFS site would be subjected to standard durability tests that demonstrate the soil cement's ability to withstand freeze-thaw and wet-dry cycles without degradation in performance. Trudeau/Wissa Rebuttal Post Tr. 11232, at 5. PFS has committed in the SAR to follow the standards, procedures, and recommendations contained in the ACI Committee 230 Report, including 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"). Ofoegbu Post Tr. 11001, at 14-15. This will help assure that the soil cement remains durable and free from cracks due to freeze-thaw cycles. Trudeau/Wissa Rebuttal Post Tr. 11232, at 5. Indeed, soil cement has performed well under more severe climatic conditions than those present at the PFS site; and it is used extensively for protection from wave erosion on the slopes of earthen dams and reservoirs. *Id.* at 6. Dr. Mitchell agreed that if a given soil cement mixture passes the durability tests, freeze-induced cracks are "unlikely" to occur. *See* Tr. 11142.

4.110. Second, PFS has indicated that it will take appropriate measures to minimize the potential for crack formation during drying and curing. *Id.* at 5-6. This will reduce the possibility that water seeping into shrinkage cracks might freeze and further extend such cracks. *Id.* at 5.

4.111. Third, with respect to the cement-treated soil under the cask storage pads, the top of the layer of cement-treated soil will be six inches below the frost level for the site; thus, it will not be exposed to freeze-thaw cycles. Trudeau/Wissa Post Tr. 10834, at 42. Further, the cement-treated soil beneath the storage pads would likely be warmed to some extent by heat released from storage casks, when such casks are present. *Id.* Therefore, freeze-thaw cracking of the cement-treated soil under the pads does not present a significant concern.

Settlement Cracking Resulting from Differential Settlement

4.112. The State also claims that there could be potential cracking of the soil cement located along the perimeter of storage pads and CTB foundation as a result of differential settlement. Bartlett/Mitchell Post Tr. 11033, at 9. The State suggests that the total settlement estimated by PFS (approximately 2 inches for the storage pads and 3 inches for the CTB) will be distributed around the perimeter of the pads and CTB due to the abrupt change in vertical static loading conditions between relatively heavily loaded foundations and the adjacent unloaded perimeter area. *Id.*

4.113. Neither PFS nor the Staff perceive potential cracking of the soil cement around the pads due to differential settlement to be a concern. The long-term settlement of the pads has been conservatively estimated by PFS to be 1.75 inches. Trudeau/Wissa Rebuttal Post Tr. 11232, at 5. Significantly, in its calculations, PFS did not rely upon the stiffness or load-bearing resistance of the soil cement around the pads or the cement-treated soil under the pads to reduce the potential settlement of the pads. Ofoegbu Post Tr. 11001, at 17. Moreover, the value of soil compressibility used for foundation-settlement analyses resulted in upperbound estimates of the potential foundation settlement. *Id.* at 12. PFS witness Trudeau opined that the actual settlement would more likely be on the order of one-half inch, given the conservatisms in the pad settlement analysis; and that such a small amount of settlement would not result in any significant cracking in the soil cement. Trudeau/Wissa Rebuttal Post Tr. 11232, at 5. State witness Dr. Mitchell conditionally agreed, stating that "[i]f indeed, settlement were only a maximum of one half inch differential between the center of the pad and the soil cement, it would alleviate my concern a great deal." Tr. 11125.

4.114. Further, if any settlement of the pads were to occur, it would be less pronounced at the periphery of the pads than near the center, thus reducing the potential for formation of cracks at the interface between the pads and the adjacent soil cement. Trudeau/Wissa Rebuttal Post

Tr. 11232, at 5. Also, because the loading is distributed over a widening area as one moves deeper into the soil profile, the settlement that occurs in the soils adjacent to foundations will tend to approximate the settlement at the foundation's edge. Trudeau/ Wissa Post Tr. 10834, at 46. Therefore, the resulting settlement profile would be "dish-shaped," and there would be no abrupt differential settlement at the joint between the foundation and the adjacent soil cement. *Id.*[46]

4.115. Likewise, with respect to potential settlement of the CTB mat, such settlement would be more pronounced near the center of the mat. Trudeau/Wissa Rebuttal Post Tr. 11232, at 5. Also, as noted by PFS, the soil cement to be placed adjacent to the CTB is likely to be installed after most of the building has been constructed. *Id.* Such a construction sequence would minimize the potential for differential settlements to occur at the interface between soil cement and the building mat, because much of the building settlement would already have occurred by the time the soil cement is installed. *Id.*

4.116. In light of the foregoing considerations, we are not persuaded that significant cracking in the soil cement around the storage pads and CTB foundation will occur as a result of differential settlement. As discussed more fully below, we also find that such cracking, if it occurs, would not have any adverse effects on the safety functions of the storage pads or the CTB.

Cracking or Overstressing Due to Vehicle Loads

4.117. In addition to other cracking mechanisms suggested by the State, the State asserts that vehicle loading – caused by the cask transporters that will move the storage casks from the CTB to their locations in the pad emplacement area -- could cause cracking or overstressing of the soil cement. Bartlett/Mitchell Post Tr. 11033, at 8. However, State witness Dr. Mitchell indicated that studies demonstrating that the PFS design takes into account the loading imparted by such

[46] PFS has committed to perform maintenance repair of the pad-emplacement area as necessary to correct any changes caused by potential settlement. Ofoegbu Post Tr. 11001, at 17. One such type of 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. *Id.*

vehicles would resolve this concern. *See* Tr. 11144. Similarly, State witness Dr. Bartlett described this problem as "something that can be analyzed and calculated." Tr. 11144.

4.118. The evidence indicates that PFS has indeed analyzed this problem. With respect to the soil cement layer around and between the cask storage pads and around the CTB, PFS demonstrated by calculation that a 2-feet thick layer of compacted structural fill would be sufficient for distribution of the fully loaded transporter loads down to the underlying clayey soils. Trudeau/Wissa Post Tr. 10834, at 41-42; Tr. 11237-38. An approximately 5-feet thick layer of soil cement has since been substituted for the structural fill. *Id.* This soil cement layer, with a proposed unconfined compressive strength of at least 250 psi (36 ksf), would be several times stronger than the structural fill that it replaces and greater than the 10 ksf loading that would occur at the bottom of the transporter crawler tracks. Trudeau/Wissa, Post Tr. 10834, at 42. Accordingly, we find that this concern has been adequately addressed.

Delamination or Debonding Along the Various Material Interfaces

4.119. The State also asserted that there could be delamination or debonding along a soil cement lift interface or an interface with the concrete pad or the native soil during a seismic event. Bartlett/Mitchell Post Tr. 11033, at 8. PFS witnesses, however, citing the methodology outlined in its SAR, stated that PFS "will make certain that adequate bond strength exists at all interfaces between lifts, and between the soil cement and the pad and the underlying soil, to ensure that no delamination or debonding takes place." Trudeau/Wissa Rebuttal Post Tr. 11232, at 4. Similarly, PFS committed to perform interface strength tests to ensure that adequate bonding is achieved. *See*, *e.g.*, Trudeau/Wissa Post Tr. 10834, at 26. Also, PFS will use cement surface treatments as part of its construction procedures if the interface strength test results indicate that such treatments are necessary; these treatments, which consist of placing small amounts of cement on the lift line as each lift is applied, are extremely effective in creating a bond along the interface that exceeds the shear strength of the soil cement itself. Trudeau/Wissa Rebuttal Post Tr. 11232, at 4-5.

4.120. The State provided very little in the way of additional testimony on the issue of possible delamination or debonding. State witness Mitchell, however, did concede that the measures PFS has proposed to ensure adequate bonding between all surfaces constitute a correct approach to dealing with this potential problem. Tr. 11129. Accordingly, we find that this concern has been adequately addressed.

Dynamic Bending, Torsional, and Tensile Stresses and the Consequences of Cracking in the Cement-Stabilized Soils

4.121. The State asserts that during a design basis earthquake, possible "out-of-phase motion between individual pads or out-of-phase motion between the CTB concrete mat foundation and the perimeter soil cement mat" could impart bending, torsional, and tensile stresses in the soil cement and cement-treated soil. Bartlett/Mitchell Post Tr. 11033, at 8-9. The State claims that "even rather low tensile stresses can cause cracking," given the assertedly "very low tensile strength of the soil-cement and cement-treated soil." *Id.* at 8. Further, the State suggests that cracks formed by the various mechanisms heretofore discussed could further reduce the tensile capacity of the cement-stabilized soils and hence their ability to resist out-of-phase motion. *Id.* at 9.

4.122. The State argues that because PFS has not calculated the magnitude and orientation of these bending, torsional, and tensile stresses, a "rational assessment" cannot be made of the seismic performance of the proposed cement treatment. *Id.* at 8. From the standpoint of seismic performance, the State identifies two potential consequences in particular, which it attributes to possible significant cracking of the cement-stabilized soils and inertial interaction. *Id.* at 9. These include (1) "a reduction or loss of the cement-treated soil's ability to transfer shear stresses to the underlying upper Lake Bonneville sediments;" and (2) "a loss of the buttress effect (*i.e.,* passive earth pressure) that is relied upon by the Applicant to resist sliding of the CTB foundation." *Id.*

4.123. In addressing these concerns, PFS maintains that it does not rely on the tensile strength of the soil cement in its design. *See*, *e.g.*, Trudeau/Wissa Rebuttal Post Tr. 11232, at 4; Tr. 11296-97. Rather, PFS relies on the soil cement to resist horizontal movement, for which the important attributes are the compressive and shear strength of the soil cement, not the tensile strength. *Id.* PFS further contends that tensile forces and their effect, if any, on the soil cement will not decrease the compressive strength of the soil cement. *Id.* In response, the State disputes the accuracy of these assertions, suggesting that there are "other load paths" which PFS has failed to consider in its design calculations, and that tensile capacity is important given these load paths. *See, e.g.,* Tr. 11113-14.

4.124. From a regulatory standpoint, the ultimate concern is whether the storage pads and CTB foundation can perform their intended design or safety functions. *See* 10 C.F.R. § 72.122(b)(2)(i). As set forth below, based on the evidence presented, we find that they can perform their intended functions, notwithstanding the concerns presented by the State.

4.125. With respect to the cask storage pads, insofar as PFS seeks to use the cement-treated soil under the pads to transfer shear stresses to underlying soil strata, such a design function will not necessarily be impaired by tensile or bending stresses. *See* Trudeau/Wissa Rebuttal Post Tr. 11232, at 4. As discussed above, PFS has committed to taking appropriate measures and performing tests to ensure that adequate bonding is achieved between individual lifts of cement-treated soil and at the various material interfaces. In this regard, PFS submits that the cement-treated soil under the pads will adhere to the concrete pad, which, being much stiffer than the underlying cement-treated soil and being heavily reinforced, will resist tensile stresses. *Id*. State witness Dr. Mitchell indicated that he would expect such bonds in the cement-treated soils and at the material interfaces to help resist shear or horizontal forces generated by an earthquake. *See* Tr. 11114-15.

4.126. The Staff does not rely upon the proposed cement-treated soil/soil cement under or around the storage pads to support any safety function of the pads. Ofoegbu Post Tr. 11001, at 17. Rather, the Staff's acceptance of the storage pad design relative to foundation load bearing capacity is based upon calculations provided by PFS which demonstrate adequate safety margins against bearing capacity failure of the pads under combined static loads and potential dynamic loading from the design-basis earthquake. *Id.* These calculations do not rely on any contribution of load-bearing resistance from the soil cement around the storage pads or the cement-treated soil under the storage pads. *Id.* In addition, the Staff cites PFS calculations that show a projected pad settlement of only 3 to 4 inches, without considering the potential reduction of settlement that may result from placement of the cement treated soil or soil cement. *Id.*

4.127. Further, the Staff relies upon PFS calculations which 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. *Id.* at 18.[47] Indeed, the Staff concluded that the storage casks are less likely to tip over if the pads are free to do some sliding. *See id.*; Staff Exh. C, at 2-60; Tr. 11024.

4.128. For these reasons, we conclude, like Staff witness Dr. Ofoegbu, that even if cracking or other degradation of the soil cement/cement-treated soil in the vicinity of the storage pads were to occur as postulated by the State, it would not have any adverse effect on the safety functions of the storage pads. Ofoegbu Post Tr. 11001, at 17.

4.129. We reach a similar conclusion with respect to the CTB foundation. According to PFS, if tensile stresses that exceed the tensile strength of the soil cement occur, they will tend to open up existing shrinkage cracks, rather than create new cracks. Trudeau/Wissa Rebuttal Post Tr. 11232, at 4. Insofar as these cracks are thin, vertical cracks that are randomly oriented

[47] The Staff's evaluation of the potential for sliding of the pads did not rely on any property of the soil cement or cement-treated soil. Ofoegbu Post Tr. 11001, at 18.

throughout the soil cement, they would have to be lined up parallel to the edge of the foundation to have the greatest impact on the passive resistance of the soil cement. *Id.*; Trudeau/Wissa Post Tr. 10834, at 40. Further, even if such cracks develop, the passive resistance of the soil cement adjacent to the building will be engaged after a slight horizontal motion to close any cracks that happen to be aligned parallel to the edge of the foundation mat. *Id.,* Post Tr. 11232, at 4. This process will not significantly affect the ability of the soil cement to provide the passive resistance required to prevent sliding of the CTB. *Id.*

4.130. Staff witness Ofoegbu similarly testified that even if vertical and/or near-vertical cracks were to form in the soil cement via the various mechanisms identified by the State, 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. Ofoegbu Post Tr. 11001, at 19. Accordingly, like PFS, the Staff does not expect such cracking -- if it occurs -- to significantly affect the passive resistance of the soil cement, or that any associated small lateral movement of the CTB foundation would impact any safety function of the structure, as there are no external safety-related connections associated with the CTB. *Id.* Therefore, we find that the potential cracking or other soil-cement degradation mechanisms identified by the State would not have an adverse effect on the safety functions of the CTB foundation.

<u>Strength and Stiffness Properties of the Cement-Treated Soil Under the Storage Pads</u>

4.131. Another concern raised by the State is the Applicant's alleged failure to demonstrate that it can achieve the specific combination of strength and stiffness properties required for the cement-treated soil that would underlie the storage pads. *See* Bartlett/Mitchell Post Tr. 11033, at 10. These required material properties are a minimum unconfined compressive strength of 40 psi and a maximum Young's modulus of 75,000 psi. *Id.* The State submits that PFS has not provided any site-specific test data that demonstrate this rather low modulus can be achieved for a cement-treated soil with a minimum compressive strength of 40 psi, as “there is not very much published test data for these low modulus values.” *Id.* [48]

4.132. These criticisms fail to identify any fatal flaws with respect to the acceptability of the Applicant’s proposed uses of cement-stabilized soils in its foundation system designs. To be sure, PFS has yet to provide site-specific data definitively demonstrating the achievability of this particular strength-stiffness combination; however, PFS is confident that it can achieve this combination of properties. *See, e.g.,* Tr. 10939. As noted by Dr. Ofoegbu, PFS will have to show through testing that it has achieved the minimum compressive strength/maximum Young’s modulus values specified for the cement-treated soil. Tr. 11024. In this regard, PFS has committed to demonstrate through appropriate testing that any Staff-approved soil cement/cement-treated soil design specifications will be achieved. Ofoegbu Post Tr. 11001, at 20. In the absence of any evidence that PFS will not do so, we find that the commitments made by PFS effectively resolve the State’s concern.

4.133. We recognize that the State has expressed doubt as to the technical feasibility of achieving this specific combination of strength and stiffness properties. *See*, *e.g.*, Bartlett/Mitchell Post Tr. 11033, at 10; Tr. 11159. The State observes that the strength/stiffness requirement for the soils of concern falls within “a low cement, low modulus range where there’s not a lot of data to give us confidence that it’s going to be readily achievable.” Tr. 11159. However, both the PFS and Staff witnesses are satisfied that this combination of material properties is achievable. *See*, *e.g.*, Tr. 10939, 10992, 11023. Indeed, Dr. Ofoegbu, citing two specific technical papers, testified

[48] In subpart C.3.e. of this contention, the State asserted that PFS “unconservatively underestimated the dynamic Young’s modulus of the cement-treated soil when subjected to impact during a cask drop or tipover accident scenario,” which “significantly underestimates the impact forces and may invalidate the conclusions of the Applicant’s Cask Drop/Tipover analyses.” That issue was addressed under Part D of the contention rather than here. *See, e.g.*, Tr. 10915-18.

that there is information in the literature that shows that these properties are within the range that can be achieved. *See* Tr. 11023, 11025-26. In contrast, the State's witnesses did not claim that this specific combination of strength and stiffness properties is unattainable. To the contrary, Dr. Mitchell "agree[d] that it is a condition that is achievable," noting that the "information sources" cited by Dr. Ofoegbu "suggested it shouldn't be too much of a problem." Tr. 11159-60.

4.134. Dr. Mitchell further testified that PFS must consider changes in compressive strength and stiffness over time, because these properties "are likely to continually increase for some time period," even months or years after the cement-treated soil is initially constructed. Tr. 11216-17. In his view, this could be significant relative to the 75,000 psi limit that has been prescribed by PFS for the cement-treated soil. Tr. 11216. However, on cross-examination, Dr. Mitchell indicated that the rate of increase in the Young's modulus decreases with time, and that he would expect the greatest amount of change or increase to occur within the first 28 days of curing. Tr. 11227.

4.135. In response to this concern, PFS witness Dr. Wissa testified that he has "done a lot of work in this area of curing and strength change," including work relating to changes in modulus with time. Tr. 11240-41. Dr. Wissa was well aware of the correlation between Young's modulus and compressive strength, as well as the behavior of these properties as a function of time. *See* Tr. 10984-85, 11240-43. Significantly, he indicated that Dr. Mitchell's concerns about increases in strength and stiffness with time can be taken into account when designing a soil-cement mix of the type proposed by PFS. *See* Tr. 11242.

4.136. In sum, the evidence supports a conclusion that the specific combination of strength and stiffness properties required for the cement-treated soil under the storage pads is achievable, and that the time-dependent behavior of these properties can be accounted for in designing a soil-cement mix. Further, we find that the commitments made by PFS to perform appropriate testing effectively resolves these issues.

Potential Impacts of Cement-Stabilized Soil Construction/Placement on Native Soils

4.137. The State also claims that PFS has not adequately evaluated the potential impacts of construction and placement of the soil cement and cement-treated soil on the underlying native soils. Bartlett/Mitchell Post Tr. 11033, at 3 and 11-13. The State asserts that the engineering properties of the native clays – *i.e.*, upper Lake Bonneville sediments – are very important because PFS relies on the shear strength of this layer to provide resistance to sliding. *Id.* at 11. As such, the State reasons that "[a]ny disturbance or remolding of these clays could substantially decrease their shear strength." *Id.* In addition to remolding and recompacting of the native clays, the State postulates that increases in moisture content resulting from the placement of the cement-stabilized soils could cause shear strength decreases in the native soils. *Id.* at 11-13.

4.138. PFS will use compacted clayey soils only in the event that the eolian silt layer extends to a depth greater than two feet below the bottoms of the storage pads, in which case it would be necessary to raise the elevation of the subgrade under the cement-treated soil layer. *See* PFS Exh. JJJ, at 2.6-108. The foundation profiles show that for most of the pad emplacement area, the eolian silt layer extends less than two feet below the pad; only in the far southeastern corner of the pad emplacement area, the eolian silt layer to be removed may extend more than two feet below the bottom of the pads. Trudeau/Wissa Rebuttal Post Tr. 11232, at 7.

4.139. The State asserts that recompaction of the native clays could result in shear strengths that are less than the value PFS is relying upon for the native soils – *i.e.*, less than the 2.1 ksf determined by PFS using the lowest strength measured in the unconsolidated, undrained triaxial tests for the soils in the pad emplacement area. Bartlett/Mitchell Post Tr. 11033, at 11. However, State witness Dr. Bartlett admitted that "I don't have a good feel for what the strengths of the Bonneville clay are recompacted," and "I just don't have a lot experience with remolded and recompacted Bonneville clay." Tr. 11164. Thus, he could not draw a conclusion as to whether the desired shear strengths of the recompacted clays are "achievable" or "not achievable." *Id.*

4.140. In contrast, State witness Dr. Mitchell expressed the view that if the Bonneville clay is processed appropriately and compacted to the right condition, "you could get a reasonably high strength," but that "you might not be able to do that quite so easily in the field." Tr. 11166. He attributed this potential "construction challenge" to his belief that PFS would be compacting the clays over a "deformable subgrade" (*i.e.*, the Bonneville clays) as opposed to a "firm subgrade," which could result in reduced compaction, and he stated that PFS "would have to look at that." *Id.*

4.141. Having reviewed the evidence, we conclude that PFS has anticipated and adequately addressed these potential concerns. PFS indicated its understanding that "the soil cement construction techniques to be used could potentially impair the surface of the native soils under the soil cement or the cement-treated soil layer . . . if it is not properly protected." Trudeau/Wissa Post Tr. 10834, at 36. Further, PFS indicated that it intends to demonstrate at the start of construction that the techniques it allows the contractor to use will not have an adverse impact on the strength of the soils. *Id.*

4.142. Further, PFS observed that it is "well established" that recompacted clays can have shear strengths comparable to 2.1 ksf, particularly if a high degree of compaction is used. Trudeau/Wissa Rebuttal Post Tr. 11232, at 7. Moreover, PFS indicated that it expects to control the compaction of these clayey soils based on Modified Proctor densities, which require a higher degree of compactive effort and will result in a stronger compacted clay. *Id.* In response to Dr. Mitchell's concern regarding reduced compaction capabilities due to a "deformable subgrade," PFS witness Trudeau testified that he does not expect such difficulties, in that the soils of concern are not soft clays, but rather, are "2 ksf undrained shear strength soils" that will not respond in the manner" cited by Dr. Mitchell. Tr. 11243. Finally, with respect to potential disturbance of the clay subgrade by construction equipment, PFS has indicated that there are a number of construction techniques available to prevent damaging the native soils beneath the cask storage pads, and that it intends to use appropriate measures to prevent such damage. Trudeau/Wissa Post Tr. 10834, at 36-37. On the basis of this evidence, we find that the State's concerns regarding compaction of the soils during construction have been adequately addressed.

4.143. The State has also expressed a concern about possible reductions in the shear strengths of the native clay soils as a result of increases in their moisture content caused by placement of the cement-stabilized soils. Bartlett/Mitchell Post Tr. 11033, at 12-13. The State noted that when clays gain moisture, they soften and there is a decrease in their undrained shear strength. *Id.* at 12. In their testimony, the parties discussed several purported mechanisms by which the moisture content of these clays could increase, including: (1) exposure of the subgrade to rainfall or runoff; (2) the upward migration of soil moisture that can no longer evaporate because of the "sealed surface" or "impermeable cap" formed by the concrete foundations; and (3) the infiltration of surface water through cracks in the soil cement. For the reasons stated below, we are not convinced that such moisture increases will occur by the mechanisms postulated; nor are we persuaded to believe that such moisture increases – if they were to occur – would have any significant consequences from an engineering and safety standpoint.

4.144. With regard to the first mechanism, exposure to the elements will be minimized through the use of proper construction procedures and scheduling. Trudeau/Wissa Post Tr. 10834, at 36. Soil excavation will not take place until the first lift of soil cement or cement-treated soil is ready to be placed; once in place, this first lift will serve to shelter the underlying clay soils from rain. *Id.* Moreover, if the underlying clay soils are exposed to rainfall during construction, one of several available options will be utilized to remove excess moisture from the soil. *Id.*

4.145. The second mechanism involves postulated increases in moisture content resulting from the redistribution (*i.e.*, the accumulation) of moisture in the clay soils beneath the concrete storage pads or CTB foundations. *See* Bartlett/Mitchell Post Tr. 11033, at 12-13. The State claims that changes in the evapotranspiration environment caused by placement of the pads or CTB foundation could change the moisture content, and therefore, the strength of those soils, which "will have a detrimental consequence on the engineering properties of the clay layer." *Id.* at 12.

4.146. We are not persuaded that this concern is valid. The proposed site is located in a semi-arid environment, with annual precipitation of approximately nine inches. Tr. 11139, 11236. Moreover, the depth to the water table is approximately 120 feet below the base of the facility structures. Ofoegbu Post Tr. 11001, at 16. As such, there is no available water supply that could feed the postulated water content increase that the State suggests could occur beneath the cement-treated soil. *Id; see also* Trudeau/Wissa Post Tr. 10834, at 37. Moreover, with respect to moisture that is present in the soils, thermal gradients will tend to redistribute the moisture away from the cement-treated soil. *See* Trudeau/Wissa Post Tr. 10834, at 38-39.[49]

4.147. The third mechanism by which water could purportedly reach the underlying clay soils is "infiltration through cracks" in the soil cement located around the storage pads and the CTB. *See* Bartlett/Mitchell Post Tr. 11033, at 9; Tr. 11149. State witness Dr. Mitchell indicated, however, that such infiltration "would be reasonably localized," and that "unless it's close to the canister transfer building, it probably would not be a major consequence." Tr. 11153. Similarly, Dr. Bartlett conceded that an increase in soil moisture potentially resulting from infiltration of water through a gap produced by differential settlement would be "a relatively localized effect, and that "[f]or something as large as the canister transfer building, . . . this effect is not one that I'd worry about."

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[49] PFS estimated the average temperature for the surface of a loaded storage pad to be approximately 120°F, which is about fifty degrees warmer than the average ambient temperature at the site. *See* Trudeau/Wissa Post Tr. 10834, at 38. As a result, heat from the cask bottoms will be conducted downward through the concrete pads and underlying cement-treated soil, making the soils beneath the pads warmer relative to surrounding soils. *Id.* Because water vapor tends to move from warmer areas to colder areas in response to a drop in air pressure as the moisture condenses, PFS states that moisture will migrate away from the cement-treated layer beneath the pads to the surrounding areas. *Id.* Dr. Ofoegbu testified that this is a "well-known phenomenon" and that moisture flows down the temperature gradient, which in this case would tend to drive moisture away from the foundation. Tr. 11012, 11030. State witness Dr. Mitchell also acknowledged the existence of this phenomenon. *See, e.g.,* Tr. 11150.

Tr. 11157. Rather, Dr. Bartlett indicated that his concern is more with moisture infiltration through the soil cement around the storage pads, given their smaller dimensions or "footprint." *See id.*

4.148. In response, PFS expressed the view that such water is unlikely to reach the underlying clay soils via cracks in the soil cement located around the storage pads, for a number of reasons. *See* Tr. 11234-37. First, the cement-stabilized soils would be constructed in lifts that would cure at different times, and the surface of each lift that has set or cured would most likely receive a cement surface treatment before the overlying soil-cement lift is placed -- such that the formation of continuous cracks would be unlikely. *See* Tr. 11235-36. Also, the continuous layer of cement-treated soil underneath each pad extends beyond the pad, resulting in a total thickness of almost five feet for the cement-stabilized soils between the pads. *See* Tr. 11236-37. In addition, water entering any cracks in the soil-cement may evaporate due to the semi-arid conditions at the site. *See* Tr. 11236.[50]

4.149. Having considered the evidence, we conclude that regardless of the specific mechanism involved, the soil moisture increases postulated by the State would not have any detrimental consequence on the engineering properties of the clay layer. As explained by Staff witness Dr. Ofoegbu, 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. Ofoegbu Post Tr. 11001, at 16. The shear strength that was used by PFS 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

[50] Data provided by PFS in its SAR 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). Ofoegbu Post Tr. 11001, at 16.

widths and depth profile of shear strength below the foundations. *Id.* As such, it is unlikely that a sufficient decrease in shear strength would 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). *Id.*

<u>Summary of Findings Regarding Part C of Unified Contention Utah L/QQ</u>

4.150. Based on our consideration of all of the evidence, we have reached the following factual conclusions regarding the matters raised in Part C of Unified Contention Utah L/QQ, in addition to the specific findings set forth above.

4.151. First, we find that the site-specific investigations and laboratory analyses performed by PFS satisfy the 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. In this regard, PFS has 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. Further, the specific combination of field and laboratory tests performed by PFS provided the data needed to obtain the values for soil strength and compressibility used in its stability analyses of the storage pads and canister transfer building foundations. In addition, we find that 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. Additional "tested samples" and "strain-controlled cyclic triaxial tests or triaxial extension tests" are therefore not necessary.

4.152. Second, with respect to the soil-cement and cement-treated soils issues raised in this contention, 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, and it has committed to follow the applicable soil-cement standards in ACI 230-1R-90. We find that these commitments, when satisfied, provide adequate assurance that the cement-treated soil and soil-cement will perform their intended safety functions.

4.153. Further, we find that 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, can be precluded or mitigated by the use of proper construction techniques and would not have an adverse effect on the safety of the proposed facility. Similarly, if cracking or other degradation of the cement-treated soil under the storage pads, or cracking of the soil-cement around the pads or the CTB were to occur, it would not have an adverse effect on the safety functions of the pads or the CTB.

4.154. For these reasons, as well as the other reasons discussed above, we find that the Applicant's geotechnical site characterization and its proposed use of cement-stabilized soils at the PFSF site are adequate, and that the concerns expressed in Part C of this contention have been resolved. We turn now to consider Part D of Unified Contention Utah L/QQ.

D. Unified Contention Utah L/QQ, Part D.

The Contention

5.1. Part D of this contention presented numerous issues concerning the Applicant's seismic design and cask stability analyses, which were first explicitly raised in Late-Filed Contention Utah QQ and were then incorporated in Unified Contention Utah L/QQ. As formulated, this portion of the unified contention asserts as follows:

D. <u>Seismic Design and Foundation Stability.</u>

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. <u>Seismic Analysis of the Canister Transfer Building and its Foundation</u>
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.

Applicable Legal Standards

5.2. 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. 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. § 72.104(d), site-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading.

5.3. 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.

5.4. 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.

Evidence Presented

5.5. Seventeen days of evidentiary hearings were held on this portion of the contention, on April 29 - May 9, June 3-5, June 7-8, and June 18-20, 2002. Numerous witnesses appeared on behalf of the parties, as set forth below. All of the witnesses were found to be qualified to present testimony on the matters they addressed.

Applicant Witnesses

5.6. The Applicant presented six witnesses with respect to Part D of the contention, in support of its application. These were as follows: Drs. Robert Youngs and Wen Tseng ("Joint Testimony of Robert Youngs and Wen Tseng on Unified Contention Utah L/QQ" (hereinafter referred to as "Youngs/Tseng"), Post Tr. 5529); Drs. Krishna P. Singh and Alan I. Soler ("Testimony of Krishna P. Singh and Alan I. Soler on Unified Contention Utah L/QQ" (hereinafter referred to as "Singh/Soler"), Post Tr. 5750); Paul J. Trudeau (hereinafter referred to as "Testimony of Paul J. Trudeau on Unified Contention Utah L/QQ" ("Trudeau"), Post Tr. 6135); and Bruce E. Ebbeson ("Testimony of Bruce E. Ebbeson on Unified Contention Utah L/QQ" (hereinafter referred to as "Ebbeson"), Post Tr. 6357). In addition, rebuttal and/or surrebuttal testimony (either written and oral) was presented by each of these witnesses.

5.7. Applicant witness Dr. Robert Youngs is a Principal Engineer employed by Geomatrix Consultants Inc., in Oakland, California. He has a Ph.D. in Geotechnical Engineering from the University of California; and he is registered as a Geotechnical Engineer and Civil Engineer in the State of California. He has over 25 years of professional consulting experience, primarily focused in the analysis of seismic hazards. Dr. Youngs' experience encompasses, among other areas, the characterization of earthquake ground motions and the performance of probabilistic and deterministic analyses to develop seismic design criteria for ground motion and fault displacement. He has conducted these types of analyses for many nuclear power plants throughout the country and the world and has performed similar studies for existing and proposed U.S. Department of Energy ("DOE") nuclear facilities at Hanford, Washington; INEEL, Idaho; Rocky Flats, Colorado; Savannah River, South Carolina; and Yucca Mountain, Nevada. Youngs/Tseng, Post Tr. 5529, at 1-2; Youngs Professional Qualifications ("Qualifications") at 1.

5.8. Applicant witness Dr. Wen Shou Tseng is President of International Civil Engineering Consultants, Inc. ("ICEC"), which provides specialty consulting services in the general areas of civil and structural engineering with special emphasis on earthquake engineering. Dr. Tseng has a Ph.D. in Civil Engineering from the University of California, Berkeley, and is a registered Civil Engineer in the State of California. He has been doing research and development, and performing consulting services for more than 30 years in the area of earthquake engineering and soil-structure interaction effects on structures. He has published many technical papers and technical and project reports on soil-structure interaction. During his 12 years of experience at ICEC, Dr. Tseng has work for numerous nuclear facilities. For the 12 years prior to his joining ICEC, Dr. Tseng was head of Bechtel's Special Structures Group performing research and development and providing technical consulting services to many nuclear power generating facilities, including a large number of U.S. nuclear power plants. The work on all these plants involved elements of seismic analysis and design of the plant structures, systems and components, including soil-structure interaction. Youngs/Tseng, Post Tr. 5529, at 2-3; Tseng Qualifications at 1.

5.9. Applicant witness Dr. Krishna P. Singh is President and CEO of Holtec International ("Holtec"), the vendor of the HI-STORM 100 Storage Cask System which PFS proposes to use at its facility, and bears the ultimate corporate responsibility for Holtec's spent fuel dry cask storage systems. Dr. Singh has a Ph.D. in Mechanical Engineering from the University of Pennsylvania, and is a registered Professional Engineer. He has extensive experience in the design and licensing of nuclear spent fuel systems since 1979. Over the past 23 years, Dr. Singh has led the design and licensing of spent fuel storage systems for over forty nuclear plants, and for Holtec's HI-STAR 100 System and HI-STORM 100 Storage Cask System. He is also the inventor of the honeycomb basket design utilized in the HI-STAR 100/HI-STORM Systems and the METCON™ construction used in the HI-STORM System overpack. He has published extensively in the field of applied heat transfer and structural mechanics and taught academic courses in this field at the University of Pennsylvania. Singh/Soler, Post Tr. 5750, at 1-2.

5.10. Applicant witness Dr. Alan I. Soler is the Executive Vice President and Vice-President of Engineering for Holtec International. He is responsible for all corporate engineering activities by the company, including overseeing the analyses performed to establish the stability of the HI-STORM 100 System under postulated seismic events. Dr. Soler is the lead structural discipline expert responsible for the design of the HI-STORM System, including supporting analyses, and he has acted in this capacity since the design was conceptualized in the early 1990s.

Dr. Soler either performed or reviewed all HI-STORM System seismic analyses conducted in support of deployment of the HI-STORM System at the PFS Facility ("PFSF"). Prior to Dr. Soler's employment with Holtec International, he was a tenured Professor of Mechanical Engineering and Applied Mechanics at the University of Pennsylvania for over 26 years; and he has a Ph.D. in Mechanical Engineering from that institution. *Id.* at 4-5; Soler Qualifications at 1.

5.11. As discussed above with respect to Part C of this contention, Applicant witness Paul J. Trudeau is a Senior Lead Geotechnical Engineer at Stone & Webster, Inc., a Shaw Group Company ("S&W") in Stoughton, Massachusetts. Mr. Trudeau has a M.S. degree in Civil Engineering from the Massachusetts Institute of Technology, and is a licensed Professional Engineer in Massachusetts and elsewhere. He has 29 years of experience in geotechnical engineering, including the performance of subsurface soil investigations; the performance and supervision of the analysis of foundations in support of the design of structures; the performance of laboratory tests of soils including index property tests, consolidation tests, static and dynamic triaxial tests, and other tests; the performance of analyses of the performance of soils and structures under static and dynamic conditions; the development of geotechnical design criteria for other engineering disciplines, such as Structural, Environmental, Engineering Mechanics, and Electrical; and the preparation of the geotechnical sections of Preliminary and Final Safety Analyses Reports and Environmental Reports. Trudeau, Post Tr. 6135, at 1-2; Trudeau Qualifications at 2.

5.12. Applicant witness Bruce E. Ebbeson is a Senior Lead Structural Engineer with Stone & Webster in Cherry Hill, New Jersey. He has a M.S. degree in Civil Engineering from Tufts University, and is a licensed Professional Engineer in New Jersey and elsewhere. He has approximately 30 years of experience as a Civil/Structural Engineer, specializing in the structural design and analysis, including seismic analysis, of nuclear facilities. Mr. Ebbeson is currently the supervisor of the structural division for S&W's Cherry Hill office and serves as structural engineering consultant on various projects performed by S&W in its Cherry Hill, Boston, Denver and Taiwan offices. Mr. Ebbeson has been the Principal Structural Engineer on many nuclear facility projects. Among other activities, Mr. Ebbeson has performed and supervised the performance of original designs and design modifications for those nuclear facility projects, as well as safety evaluations to meet licensing requirements; and he has performed independent design reviews of nuclear facilities at various stages of their licensing and operation. Ebbeson, Post Tr. 6357, at 1-2; Ebbeson Qualifications at 1.

Staff Witnesses

5.13. The Staff presented a total of four witnesses, in two panels, with respect to Part D of this contention. First, Daniel J. Pomerening and Dr. Goodluck I. Ofoegbu provided testimony concerning the Staff's evaluation of the seismic design and foundation stability of the PFSF, including evaluations of various analyses submitted by PFS in its Safety Analysis Report ("SAR") and other documents ("NRC Staff Testimony of Daniel J. Pomerening and Goodluck I. Ofoegbu Concerning Unified Contention Utah L/QQ, Part D (Seismic Design and Foundation Stability)" (hereinafter referred to as "Pomerening/Ofoegbu"), Post Tr. 6496). Second, Dr. Vincent K. Luk and Jack Guttmann provided testimony concerning an independent analysis performed at the Staff's request, concerning the potential for cask tipover and/or sliding during a seismic event at the PFSF ("NRC Staff Testimony of Vincent K. Luk and Jack Guttmann Concerning Unified Contention Utah L/QQ (Geotechnical Issues)" (hereinafter referred to as "Luk/ Guttmann"), Post Tr. 6760). In addition, Dr. Luk presented surrebuttal testimony in response to the rebuttal testimony of State witness Dr. Steven F. Bartlett.

5.14. Staff witness Daniel J. Pomerening is a Principal Engineer in the Mechanical and Materials Engineering Division of the Southwest Research Institute ("SwRI"), in San Antonio, Texas. Mr. Pomerening provided his testimony under a technical assistance contract between the NRC Staff and the Center for Nuclear Waste Regulatory Analyses, which is a division of the SwRI.

He has an M.E. degree in Civil Engineering, Structural Engineering and Structural Mechanics from the University of California, Berkeley. In his position at the SwRI, he serves 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. He has had considerable experience in the study of structural response of systems under dynamic loading with specific emphasis on transient and shock loading, and has reviewed safety analysis reports with specific emphasis on the identification of design criteria and assessment of the structural integrity of structures, systems and components with respect to the NRC Standard Review Plans. Pomerening/Ofoegbu, Post Tr. 6496, at 1-2; Pomerening Qualifications at 1.

5.15. As discussed above, Staff witness Dr. Goodluck I. Ofoegbu is a Principal Engineer at the CNWRA in San Antonio, Texas, and provided his testimony under a technical assistance contract between the NRC Staff and the CNWRA. Dr. Ofoegbu has a Ph.D. in Geological Engineering from the University of Toronto, and is a registered professional engineer in Canada. Dr. Ofoegbu specializes 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 he has about 20 years of experience in teaching, research, and consulting. At the CNWRA, he serves as Principal Investigator for several projects involving geological engineering, including mechanical analysis of underground excavations, foundations, earthworks, and natural geological processes such as faulting and volcanism. He has published 25 articles in refereed journals and conference proceedings, as well as several technical reports. Pomerening/Ofoegbu, Post Tr. 6496, at 1-2; Ofoegbu Qualifications at 1.

5.16. Staff witness Dr. Vincent K. Luk is a Principal Member of the Technical Staff in the Nuclear Technology Programs Department at Sandia National Laboratories ("SNL") in Albuquerque, New Mexico. Dr. Luk provided his testimony under a technical assistance contract between NRC Staff and SNL. He has a Ph.D. in Theoretical and Applied Mechanics from Northwestern University. In his position at SNL, he serves 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, he serves as the Principal Investigator in a generic 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, in which 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. Luk/ Guttmann, Post Tr. 6760, at 1-2; Luk Qualifications at 1.

5.17. Staff witness Jack Guttmann is Chief of the Technical Review Section in the Spent Fuel Project Office ("SFPO"), in the NRC Office of Nuclear Material Safety and Safeguards ("NMSS"), in Rockville, Maryland. He has a M.S. degree in Nuclear Engineering from the University of Michigan, and has approximately 30 years of experience in nuclear engineering and risk assessment. During his employment at the NRC, Mr. Guttmann's work has included the direction and supervision of technical reviews related to the licensing and certification of radioactive material transportation and storage packages, including technical reviews related to ISFSIs; and has included direction and supervision of the evaluation and use of computer code modeling and analytical methodologies in assessing the safety and performance of radioactive material transportation and storage packages. Luk/Guttmann, Post Tr. 6760, at 1-2; Guttmann Qualifications at 1.

State Witnesses

5.18. The State presented three witnesses with respect to Part D of this contention, appearing in two witness panels. These were as follows: Drs. Mohsin R. Khan and Farhang

Ostadan ("State Testimony of Dr. Mohsin R. Khan and Dr. Farhang Ostadan on Unified Contention Utah L/QQ, Part D (Cask Stability)" (hereinafter referred to as "Khan/Ostadan"), Post Tr. 7123); and Drs. Steven F. Bartlett and Farhang Ostadan ("State of Utah Testimony of Dr. Steven F. Bartlett and Dr. Farhang Ostadan on Unified Contention Utah L/QQ (Dynamic Analyses)" (hereinafter referred to as "Bartlett/Ostadan"), Post Tr. 7268). In addition, rebuttal and/or surrebuttal testimony was presented by each of these witnesses.

5.19. State witness Dr. Mohsin R. Khan is an Engineering Manager at Altran Corporation ("Altran"), in San Francisco, CA. He has a Ph.D. in Solid Mechanics (Structures), from Clarkson College of Technology, and is a registered Professional Mechanical Engineer in the State of California. Dr. Khan serves as Manager of Altran's Structural Mechanics group in San Francisco, which is involved in equipment, piping, and structural analysis and design. He has about 22 years of experience using response spectral data and finite element analysis to predict seismic performance of various structures, systems and components; and his work assignments have included work related to other nuclear facilities, including dry cask storage projects for the Diablo Canyon and Humboldt Bay nuclear power plants. Dr. Khan was previously employed as a Principal Engineer by Pacific Gas & Electric Co., and Bechtel Power Corp. (where his work included various nuclear power plant-related projects); and he was an Assistant Professor at Clarkson College of Technology. Khan/Ostadan, Post Tr. 7123, at 3-4; Khan Qualifications (State Exh. 119) at 1-5.

5.20. State witness Dr. Farhang Ostadan has a Ph.D. in Civil Engineering from the University of California, Berkeley. He is employed as a Chief Soils Engineer at Bechtel Power Corp. in San Francisco, where he supervises about five other persons.[51] In addition, he is a consultant in the field of soil dynamics and geotechnical earthquake engineering, and is a visiting lecturer at the University of California at Berkeley in the field of soil dynamics and soil-structure

[51] Although Dr. Ostadan is employed by Bechtel, he pointed out that he was providing testimony in this proceeding as an individual rather than as a Bechtel employee. Tr. 7800.

interaction. Dr. Ostadan has over 20 years' experience in dynamic analysis and seismic safety evaluations of structures and subsurface materials, and he co-developed SASSI, a computer program for seismic soil-structure interaction analysis. Dr. Ostadan has participated in seismic studies and reviews of numerous nuclear structures and facilities, and has published a number of papers in the area of soil structure interaction and seismic design for nuclear and other structures. Khan/Ostadan, Post Tr. 7123, at 2; Ostadan Qualifications (State Exh. 110) at 1-2; Tr. 7735-36.

5.21. As discussed above with respect to Part C of this contention, State witness Dr. Steven F. Bartlett is an Assistant Professor in the Civil and Environmental Engineering Department of the University of Utah, where he teaches undergraduate and graduate courses in geotechnical engineering and conducts research. He has a Ph.D. in Civil Engineering from Brigham Young University, and is a licensed professional engineer in the State of Utah. Previously, he worked for the Utah Department of Transportation ("UDOT") as a research project manager, and has held various positions at UDOT and elsewhere involving geotechnical engineering, earthquake engineering, and geotechnical design. *See* Bartlett/Ostadan, Post Tr. 7268, at 1; Bartlett Qualifications (State Exh. 92) at 1-2.

State of Utah's Testimony

5.22. State of Utah witness Dr. Khan provided testimony in which he advanced many of the State's claims concerning the adequacy of the Applicant's seismic design. Specifically, he asserted that the Applicant's design, in which free-standing (unanchored) cylindrical casks are placed on a reinforced concrete storage pad, is "unprecedented" and "unconventional." Further, he asserted that the design lacks redundancies; that the design lacks a comprehensive seismic analysis; that the cask stability analysis performed by Holtec should be verified or benchmarked with shake table tests and other computer codes; that the analysis fails to account for all potential conditions that may amplify the movement of the casks and pads; and that Holtec used input parameters which may significantly underestimate cask behavior. Khan/Ostadan, Post Tr. 7123, at 1-2, 5-6, 13-14.

5.23. With respect to the Holtec analysis, Dr. Khan criticized Holtec's use of its in-house non-linear DYNAMO code, a lump mass mathematical code, which Holtec utilized in modeling cask behavior for the 2,000-year design earthquake. Dr. Khan asserted that DYNAMO is a "small deformation code" that cannot process cask rotations greater than 15 degrees, that it has never been used to model dry cask ISFSIs where the zero period acceleration was as great as the PFS 2,000-year return period DBE ground motion, and was never benchmarked for ground motions greater than 0.4 g. He further observed that Holtec realized that its DYNAMO code might produce errors for large ground motions, and it therefore used the Visual/Nastran code for analysis of the 10,000-year earthquake at the PFSF site, having previously used that code to model cask behavior at the Diablo Canyon ISFSI, where the zero period accelerations are large. *Id.* at 4-5.

5.24. Most of Dr. Khan's testimony, however, consisted of a description of an analysis which he conducted, to check the validity of the PFS cask stability analysis, which provided the basis for his conclusion that the HI-STORM 100 casks at the PFSF site may excessively slide, uplift, and "potentially tipover" in a 2,000-year return period ground motion at the site. *Id.* at 2, 6. That analysis (commonly referred to as "the Altran report") was introduced into evidence by the State and became the focus of considerable testimony in the proceeding. *See* State Exh. 122.[52]

5.25. In his analysis, Dr. Khan modeled aspects of cask behavior under 2,000-year design earthquake conditions at the PFS site. Therein, he first attempted to estimate the maximum,

[52] As discussed below, Dr. Khan's analysis used unrealistic and inappropriate input parameters, leading to grossly unreliable results. *See generally* Singh/Soler, Post Tr. 5750, at 73-98; Tr. 6691-92. State witness Dr. Ostadan observed that Dr. Khan did not include consideration of soil-structure interaction effects, which can be important, Tr. 7736-37. Further, he distanced himself from Dr. Khan's analysis, stating it was not Dr. Khan's intent to conduct a thorough seismic analysis that could be used for design -- and he characterized Dr. Khan's analysis as merely providing a sensitivity analysis which shows how results could vary if different values for the input parameters are used. Khan/Ostadan, Post Tr. 7123, at 14; Tr. 7391-92.

cumulative cask displacement, without allowing the cask to overturn. He found that cask displacement varies significantly with the contact stiffness value used; and that non-linear mathematical models predicting cask movement are highly sensitive to the contact stiffness parameter selected for the analysis. He contended that high contact stiffness values in the model would artificially absorb energy and could reduce instantaneous velocities used in the next iteration in non-linear time history analysis, thus underestimating vertical cask displacement. Khan/ Ostadan, Post Tr. 7123, at 7-9, 11.

5.26. In addition, Dr. Khan found that cask displacement varies significantly with the amount of damping used, and he contended that the amount of damping should be insignificant for a rigid cask, because friction should be the primary energy dissipation mechanism. He input what he considered to be "reasonable" contact stiffness values -- which were, in fact, quite small (0.01% and 1%). *Id.* at 9, 11-12; State Exh. 122, Table 3, at 13. His results led him to conclude that cask displacements may be significantly higher than those reported in Holtec's cask stability analyses, in which a larger 5% damping value was used. Khan/Ostadan, Post Tr. 7123, at 9, 12.

5.27. In sum, Dr. Khan predicted horizontal and vertical displacements up to 31 feet and 19 feet in the x and y horizontal directions, and 27 inches in the vertical direction, using a damping value of 0.01% and a coefficient of friction (*mu*) of 0.8. Khan/Ostadan, Post Tr. 7123, at 13; State Exh. 122 at 13 (Table 3).[53] Further, Dr. Khan calculated the minimum vertical uplift and horizontal

[53] In effect, Dr. Khan's analytical results -- were we to credit them -- would produce a phenomenon in which the 175-ton stubby cylindrical casks at the PFS site literally "jump" or take off and "fly" in the event of a design basis earthquake at the site. Singh/Soler, Post Tr. 5750, at 93; Tr. 7391-92 (Lam, J.). Mr. Pomerening, for the Staff, indicated that Dr. Khan used input values that were not "realistic." Tr. 6691-92. PFS witnesses Drs. Singh and Soler were more emphatic, stating that Dr. Khan's results "defy physical reality," and are "nonsensical," "contrary to the laws of physics," and contrary to established guidance. *See* Singh/Soler, Post Tr. 5750, at 73, 74, 75, 76, 78, 79, 85, 88; Tr. 9637-39. Moreover, even State witness Dr. Ostadan testified that he did not believe the numbers produced by Dr. Khan's analysis were accurate, and opined that the Licensing Board should not be concerned about Dr. Khan's results or the potential for "flying casks." Tr. 7391-92, 10406-07. On the basis of the evidence presented, we share that opinion.

velocity required to tip over the HI-STORM 100 cask, and concluded that the cask has the potential to tip over if subject to the 2,000-year design earthquake at the PFS site. Khan/Ostadan, Post Tr. 7123, at 10, 13.

5.28. For his part, Dr. Ostadan considered Dr. Khan's determination that the HI-STORM 100 casks may excessively slide and uplift, as well as his own view that the PFS design lacks redundancies and has small design margins -- and concluded that PFS had not evaluated the cumulative effect of applicable input parameters in evaluating the safety of the casks, and had not proven the stability of the free-standing casks. *Id.* at 5-6, 6-7 and 14-15.[54]

5.29. In addition, testimony was presented by State witnesses Drs. Ostadan and Bartlett, concerning Part D of the contention, based on their criticisms of the Applicant's seismic design and supporting analyses. First, they asserted (similar to the State's criticisms in Part C of the contention), that PFS's seismic design is unprecedented and unproven, insofar as it involves unanchored storage casks, shallowly embedded foundations, the use of soil cement adjacent to the pads and CTB to provide resistance to sliding, and the use of cement treated soil under the storage pads to strengthen the soils. Bartlett/Ostadan, Post Tr. 7268, at 4-5. They further asserted that the design involves conflicting requirements, insofar as the storage pads need to act rigidly to allow "controlled" cask sliding but not be too rigid in the event of cask tipover. *Id.* at 5, 11. In addition, Dr. Ostadan contended the PFS design lacks redundancies. *Id.* at 8.

5.30. Their testimony further contends that because the native soils at the PFS site are compressible, deformable and of relatively low strength, PFS introduced soil cement and cement

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[54] In order to find a "cumulative" or "synergistic" effect, we must first find that errors exist in the Applicant's analyses, and that those errors may have a cumulative effect. However, as discussed in the text above, the evidence does not support a conclusion that the alleged errors exist. Further, Staff witnesses Pomerening and Ofoegbu did not find any basis to believe that a cumulative or synergistic effect exists -- and, in any event, if there was such an effect it could be additive, subtractional, multiplicative, or divisional. *See* Tr. 6680-90, 6698, 6699-6702, 6709-13. In other words, some of the alleged errors (if any exist) might actually cancel out other alleged errors, so that no cumulative effect may be deemed to arise.

treated soil as an "engineering mechanism" in a attempt to improve poor soil conditions. *Id.* at 5, 9-10. However, Dr. Bartlett asserts that foundations overlying compressible soils will settle with time, which may crack the soil cement and affect the resistance to sliding. *Id.* at 9. Also, he contends that PFS has failed to adequately consider the interaction of the foundations and supported structures -- *i.e.*, soil structure interaction ("SSI"), involving both kinematic and inertial interaction; and that PFS should conduct strain-controlled cyclic tests to characterize the soils and determine their strength/deformation at levels of strain from the design basis earthquake. *Id.* at 10.

5.31. In the main, Drs. Bartlett and Ostadan challenge numerous aspects of the Applicant's calculations concerning the stability of the casks and concrete storage pads, including two calculations performed by Holtec, one calculation performed by International Civil Engineering Consultants, Inc., and two calculations performed by Stone & Webster -- claiming that the calculations are flawed and/or fail to show an adequate margin of safety in the seismic design.[55]

5.32. These criticisms involve, *inter alia*, the following alleged errors: (a) the assumption that the pads will behave relatively rigidly; (b) the potential that the dynamic Young's modulus will exceed the required limit of 75,000 psi; (c) conflicting uses of static vs. dynamic Young's modulus of the cement treated soil; (d) ICEC's alleged failure to consider long-term settlement of the pads; (e) ICEC's use of Holtec's estimated dynamic forces acting on the pad, which assertedly do not represent total dynamic load; (f) Stone & Webster's failure to consider "pad-to-pad interaction" in its dynamic analysis of the pads, leading to an incorrect calculation of dynamic forces; (g) potential

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[55] These calculations were as follows: (1) Holtec Calc. No. HI-2012653, Rev. 2, "PFSF Site Specific HI-STORM Drop/Tipover Analyses," October 31, 2001; (2) Holtec Calc. No. HI-2012640, Rev. 2, "Multi-Cask Response at the PFS ISFSI from 2000-Yr. Seismic Event," August 20, 2001 (State Exh. 173 (proprietary)); (3) ICEC Calc. No. G(PO17)-2, Rev. 3, "Storage Pad Analysis and Design," April 5, 2001) (PFS Exh. 85); (4) Stone & Webster Calc. No. G(B)-04, Rev. 9, "Stability Analyses of Cask Storage Pads," July 26, 2001 (PFS Exh. UU; State Exh. 95); and (5) Stone & Webster Calc. No. G(B)-13, Rev. 6, "Stability Analyses of Canister Transfer Building," July 26, 2001 (PFS Exh. VV). Other calculations performed on behalf of the Applicant were also challenged by the State during this proceeding.

underestimation of the seismic loads and assumptions in Holtec's analysis, and its alleged use of inappropriate soil springs and damping values; (h) Stone & Webster's alleged failure to consider gapping between the pads and the adjacent soil-cement, and cracking of the soil-cement, allegedly invalidates its assumption that cement treated soil beneath the pads will adhere to the underlying natives soils, and that soil-cement around the pads will move in unison with the pads, providing resistance to sliding of a longitudinal column of pads; (i) Stone & Webster's alleged underestimation of the dynamic forces for the pads, insofar as its analysis uses peak ground acceleration values;[56] (j) Stone and Webster's alleged failure to consider the potential for resonance in the pads; (k) the alleged failure of PFS's simplified Newmark sliding block analysis to meet a factor of safety against sliding of 1.1; (l) the failure to consider cold bonding between the casks and pad; and (m) the failure to consider all potential variations in the motion of the pads and casks due to the phasing of motions and potential pad-to-pad interaction. *See id.*, at 10-20, 21-22, and 23.

5.33. In addition, Drs. Bartlett and Ostadan point to alleged flaws in the CTB stability analysis, claiming it suffers many of the same asserted impediments as the storage pad analysis. They allege that the soil cement adjacent to the CTB will crack and separate from the building due to out-of-phase motion of the CTB foundation mat and soil cement, such that the soil-cement

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[56] Drs. Bartlett and Ostadan pointed to Figures 17 to 22b in a report by Staff witness Dr. Vincent Luk (discussed *infra*), which they believed pointed to larger peak ground acceleration values than Stone & Webster had used in its stability analyses. Bartlett/Ostadan, Post Tr. 7268, at 18; Tr. 7542, 7803-06. However, they apparently misunderstood the values cited in the Luk report: Dr. Luk explained that these figures were included in his report to show the importance of the soil-structure interaction effect, and the plotted values do not constitute a proper basis upon which to design a facility. *See* Tr. 6795-98, 6804-06, 11563-65. Rather, these are acceleration values at single node points, Tr. 6804-05; they are of very short duration (*i.e.*, impulse loading), Tr. 11563; and they are not significant with respect to the structural response of the casks, Tr. 6804, 11563-65. Further, the node points where these values were observed are at the boundary of a plane, so that the values are different from the accelerations that would be observed elsewhere on that plane. Tr. 11563-66. In addition, these values included the accelerations resulting from high frequencies, which appeared because Dr. Luk had used only one of two terms for damping and, accordingly, they have no structural significance. Staff Exh. P at 9; Tr. 6805-07.

buttress will be ineffective during a seismic event; that the stiff soil cement perimeter around the CTB affects the soil spring and damping parameters, and reduces the factor of safety to less than 1.1; and that PFS's calculations do not establish that the CTB foundation mat is rigid -- and if it is not, the soil damping in the dynamic analysis may be excessive and CTB seismic loads may be underestimated. *Id.* at 20-21. Further, they assert that the Applicant's seismic analysis does not comply with ASCE 4-98,[57] in that PFS did not consider non-vertically propagating waves, and did not utilize multiple sets of time histories in a non-linear analysis. *Id.* at 22.

5.34. We turn now to consider the Applicant's and Staff's evaluation of these criticisms. As discussed below, we find that the Applicant's analyses are acceptable, for the reasons provided by PFS, as supported by the Staff's thorough evaluation thereof. Further, we find that the additional computer modeling and analyses performed by Holtec and Staff witness Dr. Vincent Luk provide important confirmatory results which support our conclusion as to the adequacy of PFS's seismic design.

Applicant's Testimony

5.35. Applicant witnesses Drs. Singh and Soler provided a detailed explanation of the Applicant's seismic design and supporting analyses for the concrete storage pads and storage casks, in response to claims made by the State with respect to the adequacy of Holtec's seismic analyses of the HI-STORM 100 Cask System to be deployed at the PFSF. In response to these allegations, the testimony of Drs. Singh and Soler sought to (1) summarize the design of the HI-STORM System; (2) describe the features and conservatisms in the design of the HI-STORM System that enhance the ability of the casks and the fuel canisters inside the casks to withstand the forces imparted on them during a severe seismic event; (3) report the results of analyses

[57] ASCE 4-98, "Seismic Analysis for Safety-Related Nuclear Structures and Commentary on Standard for Seismic Analysis of Safety-Related Nuclear Structures" (American Society of Civil Engineers, 1998). *See* State Exh. 118; PFS Exh. XX.

performed of the casks' response to a 2,000 year return period earthquake at the PFSF and other, more severe seismic events; (4) respond to claims raised by the State in Section C.3(e) and portions of Section D of the Unified Contention; (5) respond to claims concerning the modeling of the stability of the HI-STORM System under earthquake forces raised by State witness Dr. Khan; and (6) address the capability of the HI-STORM System to withstand earthquake forces significantly beyond those imparted by the 2,000 year return period design basis earthquake for the PFSF, including the forces due to the 10,000 year return period earthquake for the site. *See* Singh/Soler, Post Tr. 5750, at 3, 6-7.

5.36. With respect to the design capability of the HI-STORM 100 cask system to withstand seismic events, Drs. Singh and Soler testified to the ruggedness of the HI-STORM System and the analyses that Holtec has performed to show that the system can withstand seismic events far more severe than the 2,000 year design basis earthquake, including the postulated 10,000 year earthquake. They presented computerized simulations of the HI-STORM 100 cask, which show that the casks will not tip-over under the postulated 10,000 year ground motions, with significant margins remaining, even under worst case assumptions. They further described how their DYNAMO computer code, used to model the HI-STORM cask system, has been validated and approved by the NRC and has been used as the licensing basis for spent fuel technology at numerous nuclear plants throughout the world. Finally, they testified to the large margin in the multi-purpose canister ("MPC") in which the spent fuel is sealed, that would serve to confine the spent fuel even assuming hypothetically that a HI-STORM storage cask would tip over under a seismic event. Singh/Soler, Post Tr. 5750, at 7 *et. seq*.

5.37. More specifically, Dr. Singh testified that the HI-STORM 100 cask is engineered to minimize local area radiation doses and to provide a robust structural enclosure for the MPC located within it; and that it is designed to withstand extreme natural phenomena, including strong earthquakes. He further stated that a loaded HI-STORM 100 storage cask exhibits excellent resistance to overturning under seismic events, due to its low height-to-diameter ratio (239.5 inches to 132.5 inches, or a ratio of 1.8); the energy absorbing internal channels within the cask; and by a state of internal dissonance produced by the vibration of the MPC within the cask and by the individual fuel assemblies in their respective storage locations. *Id.* at 7-8.

5.38. The HI-STORM cask system is designed and constructed, as applicable, in accordance with Section III of the American Society of Mechanical Engineers ("ASME") Boiler and Pressure Vessel Code (the "ASME Code") -- which governs the design of pressure vessels for safety-related applications at nuclear power plants. The MPC is engineered in accordance with Subsection NB of the ASME Code, which governs the construction of Class 1 nuclear components. Class 1 nuclear components include such items as reactor pressure vessels and primary coolant system piping. Dr. Singh considers the use of Subsection NB for the construction of the MPC to be highly conservative, since the MPC design pressure is much lower than the design pressure for a typical reactor coolant system (*i.e.,* 100 psig versus 2,500 psig or higher) and there is no significant cycling of the stress state in the service condition of the MPC, eliminating fatigue as a concern. The internal fuel basket within the MPC is designed to Subsection NG of the ASME Code, which governs the construction of nuclear component core support structures. The HI-STORM 100 storage cask is designed in accordance with Subsection NF of the Code, which governs the construction of nuclear component supports, such as spent fuel racks and reactor coolant piping supports. Thus, the MPC and the storage casks are designed and built to the same standards, as applicable, as safety-related components used in nuclear power plants. In addition, the HI-STORM System components are designed in accordance with the standards specified in the governing NRC Standard Review Plan, NUREG-1536, "Standard Review Plan for Dry Cask Storage Systems" (January 1997). *Id.* at 8-9.

5.39. Drs. Singh and Soler compared the standards specified in NUREG-1536 for dry storage cask systems with the standards specified in NUREG-0800, the NRC Standard Review

Plan for nuclear power plants. From the standpoint of seismic/structural considerations, they stated that NUREG-1536 incorporates the lessons learned from the evolutionary development of NUREG-0800. Any differences between these two NUREGs principally lie in the difference in their technical missions, in that each must address the specific concerns that apply to the particular types of structures, systems and components which they are intended to address. In particular, they noted that the amplification of an earthquake by the interplay between the flexibility of a fuel storage building and the free-field seismic motion is given considerable attention in NUREG-0800, whereas NUREG-1536 focuses on the effects of free-standing massive rigid bodies under seismic events since vertical ventilated casks (like HI-STORM) are essentially rigid structures with respect to seismic input. They concluded that NUREG-1536 calls for application of the same codes, standards and design procedures as does NUREG-0800, and any differences are almost entirely due to differences in the type, nature, and anticipated loadings between dry storage casks and reactor installations. *Id.* at 9-10.

5.40. Dr. Singh testified that, as required by NUREG-1536 and other applicable codes and standards, the design of the HI-STORM 100 storage cask has significant built-in conservatisms and design margins that assure its ability to perform in accordance with design basis requirements and to withstand events well beyond its design basis. The cask is a stubby steel weldment with homogeneous concrete (without rebars or other potential sources of crack propagation), designed to tolerate very large earthquake-induced forces without tipping over. The storage cask has been designed as a buttressed ASME Section III, Class 3, Subsection NF cylindrical structure. The 1¼-inch thick inner steel shell and ¾ inch thick outer steel shell are both welded to a 2 inch thick baseplate, and are joined by four full-length inter-shell radial support plates, each ¾ -inch thick and welded to the inner and outer shells. The cask provides an internal cylindrical cavity, 191½ inches in height and 73½ inches in diameter, to house the MPC. The top steel closure plate is also a steel weldment with confined concrete. Finally, a steel pedestal with enclosed concrete is provided for shielding, missile penetration, canister drop, and cooling flow considerations. Steel channels are located on the interior surface of the inner shell to minimize g-loadings imparted to the MPC under a hypothetical cask tip-over scenario. *Id.* at 10.

5.41. The multi-purpose canisters, in which the spent fuel assemblies are placed, are filled with an inert gas (helium) and welded shut at the reactor site prior to shipment off-site. The MPC consists of a stainless steel enclosure vessel and a fuel basket. The enclosure vessel is a cylindrical container with flat ends designed to meet the applicable provisions of Subsection NB of the ASME Code. The fuel basket is a stainless steel, continuously welded, stiff honeycomb structure that is designed to meet Subsection NG of the ASME Code, as applicable, and serves to position the fuel inside the MPC enclosure vessel. The MPC has the same relative design margins as those imposed by Subsection NB of the ASME Code for reactor operation service, even though the MPC is not subject to the stresses that result from an operating reactor environment. The MPC is designed for transportation as well as storage, giving it a ruggedness that allows it to resist large earthquake-induced forces. Thus, like the storage casks, the MPC has significant built-in conservatisms and design margins that assure its ability to perform in accordance with its design basis requirements and to withstand events well beyond its design basis. *Id.* at 11.

5.42. Holtec performed an analysis to determine whether the confinement boundary of the MPC would be breached in the hypothetical, postulated case of a crane failure or other malfunction that causes a drop of an MPC that is in the process of being loaded into a cask.[58] In that analysis, the MPC is assumed to free-fall over a distance of 25 feet, representing the height

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[58] During transfer operations at the PFSF, a loaded MPC will be transferred from the transportation cask in which it is shipped to the site to the HI-STORM 100 storage cask, inside the Canister Transfer Building. To perform this transfer, the following process occurs: (1) the HI-TRAC transfer cask is placed on top of the transportation cask; (2) the MPC is lifted up into the transfer cask; (3) the loaded transfer cask is moved by a crane over to the storage cask; and (4) the MPC is lowered into the storage cask. Singh/Soler, Post Tr. 5750, at 11; *see* “Testimony of Donald Wayne Lewis on Section E of Unified Contention Utah L/QQ,” Post Tr. 8968, at 2-4.

of the storage cask cavity plus an allowance for the thickness of the transfer cask bottom lid. The target surface was assumed to be essentially unyielding and was modeled as a 22 ft thick concrete slab of compressive strength 6,000 psi. The analysis demonstrated that the MPC confinement boundary integrity is maintained and radioactive material is not released into the environment even under this severe, hypothetical drop accident -- which is far more severe than either the drop accident analysis or hypothetical tip-over analysis performed as part of the design basis of the HI-STORM 100 cask system.[59] Based on these results, Drs. Singh and Soler concluded that due to the large margins provided by the ASME Code and design criteria, the MPC is able to withstand forces much greater than the design basis forces and still perform its safety function. Id. at 11-12.

5.43. In order to demonstrate that the HI-STORM 100 cask satisfies the requirements in 10 C.F.R. § 72.122(b)(2) and the regulatory guidance in NUREG-1536,[60] Holtec constructed a comprehensive, non-linear dynamic model of the casks, the supporting pad, and the soil foundation, and performed a series of dynamic simulations using input loading from the specified three-dimensional seismic acceleration time histories for the design basis earthquake. Drs. Singh and Soler explained that performing a non-linear analysis was appropriate, and that Holtec

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[59] This 25-foot MPC end-drop accident analysis performed by Holtec was not evaluated in the Staff's SER, is not addressed in the testimony of the Staff's witnesses, and does not appear to be part of the licensing basis for the proposed PFSF. Accordingly, we do not rely upon the results of that analysis herein, except insofar as it is cited as one of the bases for the opinion of PFS witnesses Singh and Soler.

[60] NUREG-1536, § 3.V.1.d.(i)(3)(g), at 3-14, states as follows:

> Cask designs must satisfy the load combinations that encompass earthquake, including those for sliding and overturning in ANSI/ ANS-57.9, Section 6.17.4.1. The applicant should demonstrate that no tip-over or drop will result from an earthquake. In addition, impacts between casks should either be precluded, or should be considered an accident event for which the cask must be shown to be structurally adequate.

See Singh/Soler, Post Tr. 5750, at 13.

obtained a solution over the duration of the seismic event, modeling the position and orientation of each cask at each instant in time, in order to draw conclusions about cask stability and cask-to-cask impact. Further, in order to encompass a variety of configurations and the potential for sliding and/or overturning of one or more casks, multiple simulations were performed with upper and lower bound cask-to-pad coefficients of friction, and for varying numbers of casks on the pad. Id. at 13.

5.44. Holtec developed a specialized computer code, known as "DYNAMO", to perform this dynamic analysis of the spent fuel systems, to demonstrate their compliance with NRC seismic requirements. Drs. Singh and Soler testified that the DYNAMO code has been validated, and has been reviewed and accepted by the NRC for the licensing of spent fuel storage systems.[61] In addition to using its DYNAMO code in its seismic analyses for the PFSF, Holtec has utilized that code in site-specific analyses for use of the HI-STORM cask system at ISFSIs that do not fall under the general license provisions of 10 C.F.R. Part 72, including ISFSIs at a number of nuclear power plants. *Id.* at 14. Further, Holtec has used the DYNAMO code in analyses supporting the seismic qualification of free-standing spent fuel racks used in the spent fuel pools at numerous nuclear power plants; during a seismic event, those racks may slide, tip, and rotate with respect to the spent fuel pool in a manner similar to the potential motions of a spent fuel cask on a concrete storage pad. *Id.* at 14-17.

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[61] In order for DYNAMO to be approved by the NRC for use in licensing analyses, the code had to be validated to demonstrate that it produces acceptable results for the applicable class of problems. A series of classical problems having known solutions were modeled using the code and were shown to give results in good agreement with the analytical results. Problems that had no simple analytical solutions were also evaluated and shown to give good agreement with numerical solutions using finite element codes such as ANSYS. Finally, some features of DYNAMO were validated by comparing results from experiments designed to be capable of simulation using DYNAMO. During the course of certain wet storage license submittals, DYNAMO was subjected to additional validation at the request of NRC's reviewers. In each case, the DYNAMO code proved capable of providing acceptable resolutions to the problem. The NRC has accepted the results from DYNAMO as the basis for NRC licensing action in numerous dockets. Thus, DYNAMO has been extensively benchmarked to confirm its veracity as a non-linear dynamic code. Singh/Soler, Post Tr. 5750, at 19-20.

5.45. Dr. Soler provided a detailed description of the model used by Holtec for analyzing spent fuel storage systems, including the HI-STORM storage cask system. As pertinent here, he stated that contact is defined to occur at a finite number of locations around a storage cask's circular perimeter. The HI-STORM 100 cask is modeled as a two-body system, in which the MPC and storage overpack are treated separately. The complete system, including multiple casks on a pad, is characterized by the two components' respective degrees of freedom (six for the overpack, five for the MPC), the mass and inertia properties of the component parts, and the stiffness elements (linear and non-linear) that are used to characterize contact and friction between components and to characterize underlying pad and soil properties. In addition, the storage pad is subject to seismic movements at the base of "soil springs", which represent the resistance of the soil foundation to pad translations and rotations. Three components of ground acceleration time histories of the earthquake, multiplied by the mass of the component, are applied as specified inertia forces at the mass center of each moving body. The model thus simulates the application of earthquake forces, with the pad, cask and canister being free to respond to the earthquake forces in any of their respective directional degrees of freedom. *Id.* at 17-19.

5.46. Holtec performed seismic analyses for the HI-STORM 100 cask system to be used at the PFSF using the general design parameters for the cask system along with the site-specific earthquake ground motions for the PFSF site and other site-specific parameters. Over time, a number of time history analyses were performed using different seismic events. In each analysis, the casks and their loaded internals were modeled as rigid bodies, the pad was modeled as a rigid slab, and the soil/soil cement foundation was modeled with appropriate springs and dampers to characterize the soil resistance in deflection and rotation. Further, the casks were modeled as free-standing structures with compression-only contact and with friction elements modeling the interfaces between casks and the pad. Seismic design input (acceleration time histories and soil properties to characterize the soil springs and dampers) were provided by Geomatrix. *Id.* at 20-21.

5.47. In its PFSF analysis, Holtec used site-specific ground motions and related inputs for the 2,000-year return period design basis seismic event at the PFSF site, provided by Geomatrix. These included three acceleration time histories for 5% damping, developed from response spectra having Peak Ground Acceleration ("PGA") values of 0.711g for the two horizontal directions and 0.695g for the vertical direction. Holtec also utilized soil property values provided by Geomatrix for the soil under the pad (including the effect of soil cement, as applicable), described as "Best Estimate," "Lower Range," and "Upper Range" soil properties. These properties took into account variations in the magnitude of the soil spring constants caused by the use of different input values for Young's Modulus, Shear Modulus, and Poisson's Ratio. Holtec computed the values of the spring constants and damping coefficients using the soil property values supplied by Geomatrix and applying the formulas provided in ASCE Standard 4-86, "Seismic Analysis of Safety Related Nuclear Structures." *Id.* at 21-22.

5.48. Holtec's PFSF site-specific analysis incorporated the pad dimensions and other design information, and modeled a single pad with the effect of the underlying soil foundation included by virtue of the six soil spring/dampers, calculated by Holtec based on soil properties provided by Geomatrix. The effect of soil cement under the pads was included in the moduli values used to model the springs. An effective soil mass or inertia was also included by Holtec in the model for each pad degree of freedom. *Id.* at 22-23.

5.49. Using these inputs, Holtec modeled various configurations of one to eight casks, using the lower bound, best estimate and upper range soil properties and an upper bound coefficient of friction of 0.8 at the cask/pad interface to emphasize the possibility of cask tipping, and a lower bound coefficient of friction of 0.2 to emphasize the possibility of sliding. Nine cases were run for the upper bound coefficient of friction of 0.8 (which previous analyses had shown to provide the bounding solution for cask displacement, as measured at the top of the casks); and one case was run for the lower-bound coefficient of friction of 0.2, for the configuration that gave the limiting results based on soil property values, to identify the range of potential sliding. *Id.* at 23-24.

5.50. Holtec's analysis showed that under design basis earthquake conditions for the PFSF, the loaded HI-STORM System casks have large safety margins against overturning or sliding. In no case did the analyses predict that there will be any cask tip-over or cask-to-cask impacts. Further, the maximum accelerations experienced by the casks (less than 8 g) are well below the design basis limit of 45 g specified in the Final Safety Analysis Report ("FSAR") for the HI-STORM 100 cask system. These results confirm that the forces experienced by the cask and its internals in a design-basis earthquake do not produce stresses that exceed the allowable limits. Indeed, an examination of the analytical results for the nine cases using a coefficient of friction of 0.8 shows that the maximum displacement measured at the top of the casks was 3.24 inches. The maximum angle of tilt or rotation of the cask from the vertical plane, measured by the net maximum displacement of the top of the cask in the horizontal X-Y plane, using a coefficient of friction of 0.8, was found to be 1.026 degrees. *Id.* at 24-26.

5.51. The 1.026 degree angle of tilt predicted in the Holtec analysis is significantly less than the 29.3 degree angle of tilt that is required to result in cask tipover -- *i.e.*, the angle of tilt at which a cask would tip over from its own moment with no other force applied, where it is positioned at that angle of tilt with its center of gravity positioned directly over a corner of the cask. A ratio of these two numbers shows a safety factor against cask tipover of 29.3/1.026 = 28.6. *Id.* at 26

5.52. Further, comparing the maximum net cask horizontal displacement with the displacement required to result in cask-to-cask impact (*i.e.*, 50% of the distance between adjacent casks) shows a safety factor against sliding impact of 24" / 4.142" = 5.79. Thus, cask-to-cask sliding impacts are not expected to occur in the event of a 2,000-year design basis earthquake at the PFS site. *Id.*

5.53. In addition to performing its 2,000-year return period design basis analyses, Holtec performed a site-specific analysis of the HI-STORM cask system, in which a loaded HI-STORM 100 storage cask was subjected to accelerations from a postulated, beyond-design basis 10,000-year return period earthquake for the PFSF site having a vertical PGA of 1.33g and horizontal PGAs of 1.25g and 1.23g. The analysis used a conservative estimate of the coefficient of friction between the base of the cask and the top surface of the pad of 0.8, in order to maximize the possibility of tipping by the cask. The earthquake motion was assumed to be applied directly to the base of the pad so that soil springs were not included in the simulation. The results of this analysis showed that the loaded cask would exhibit larger rotations relative to the pad (approximately 10.89 degrees from the vertical) than seen in the 2,000-year analyses -- but a significant margin against tip-over still existed, with a safety factor against tipover equal to 29.3/10.89 = 2.69. Thus, even at the 10,000-year earthquake ground motion level, large margins of safety against cask tip-over still exist. *Id.* at 27.[62]

5.54. In sum, based on the above (and other) analyses performed by Holtec, Drs. Singh and Soler concluded that under the design-basis 2,000-year return period seismic event at the PFS site, the casks will remain vertical and not tip over, and will not impact each other. Moreover, they concluded that a very large margin exists and the HI-STORM cask system at the PFSF can withstand earthquakes with return periods significantly greater than the 2,000-year design basis

[62] Holtec used the Visual Nastran ("VN") code in evaluating the beyond-design basis 10,000-year earthquake. As compared to DYNAMO, Visual Nastran is better able to model the large rotations of the cask that would be expected to occur under the 10,000-year earthquake event. Singh/Soler, Post Tr. 5750, at 62.

earthquake, including earthquakes with 10,000-year return period ground motion, and will not tip over in such events. *Id.* at 28.[63]

5.55. In addition to the cask stability analyses described above, Holtec performed analyses in accordance with the guidance in NUREG-1536, in which it analyzed the consequences of both (a) cask drop and (b) a non-mechanistic postulated tip-over of a loaded HI-STORM 100 cask at the PFSF site. The purpose of these analyses was to demonstrate that the deceleration experienced by the stored fuel in the HI-STORM 100 cask during each of the postulated vertical drop and tip-over accidents remains below the design basis deceleration of 45 g limit as specified in the HI-STORM 100 CoC. *Id.* at 29. The analyses demonstrated acceptable results, taking into account PFS site-specific conditions and design limits *(e.g.*, a Young's modulus of no more than 75,000 psi for the cement-treated soil under the storage pads). *Id.* at 30-32.[64]

5.56. In particular, a hypothetical non-mechanistic tipover analysis was conducted in accordance with NRC "defense-in-depth" regulatory guidance, to evaluate the results of a hypothetical cask tip-over event with the attendant impact of the cask on the pad. *Id.* at 32; Staff Exh. C at 15-8 to 15-9. The HI-STORM 100 storage cask and a representative portion of the pad,

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[63] In support of their conclusion, Drs. Singh and Soler also cited a confirmatory analysis performed at the Staff's request by Dr. Vincent Luk of Sandia National Laboratories, to investigate cask behavior in the event of a 2,000-year or 10,000-year return period seismic event at the PFSF. Singh/Soler, Post Tr. 5750, at 19-20. As they observed, the Sandia analysis used a sophisticated finite element analysis model (rather than a soil spring model), in which the cask, pad, soil-cement, cement-treated-soil, and soil foundations were modeled. *Id.; See* Staff Exh. P, at 5-6. The Sandia analysis confirmed Holtec's findings that the casks will not slide excessively, collide into each other, or tipover in either a 2,000-year or a 10,000-year seismic event. *See* Staff Exh. P, at 39-40. The Sandia analysis is discussed in greater detail *infra,* at 154-65.

[64] For a cask drop from a height of 10 inches, Holtec calculated a longitudinal deceleration experienced by fuel of 45.15 g, which slightly exceeds the design basis deceleration limit in the Holtec CoC of 45 g. As a result, PFS will impose procedural limits such that a HI-STORM storage cask may not be lifted by a transporter more than 9 inches above the ground (resulting in a maximum deceleration of 42.6 g). Staff Exh. C at 15-4; Singh/Soler, Post Tr. 5750, at 31. A larger margin of safety exists against the release of radioactivity, in that to actually breach a canister requires deceleration levels far in excess of those predicted by these analyses. *Id.*

soil-cement, and soil substrate were modeled to the extent required to accurately predict the post-impact system response. The hypothetical tip-over analysis demonstrated that the decelerations experienced by the fuel contained in the MPC are bounded by the 45 g design basis limits for fuel stated in the HI-STORM 100 FSAR. As in the case of the cask drop, this provides assurance that a cask tipover will not result in a breach of the confinement barrier and no release of radioactivity would occur. *Id.* at 32-33.

5.57. Drs. Singh and Soler further addressed, *seriatim*, numerous claims raised by the State in Parts C and D of this contention concerning the adequacy of the Applicant's seismic design and supporting analyses.[65] In general, they showed that those claims are either incorrect or constitute insignificant "second order" effects that have no bearing given the large safety margins inherent in the HI-STORM 100 design. *See id.* at 34-61.

5.58. In addition, Drs. Singh and Soler described additional computer simulations which they conducted, using the Visual Nastran code, to address various claims raised by the State in Part D of this contention, including the failure to consider non-vertically propagating waves; the lack of sufficient time histories; overestimation of soil damping; and failure to consider resonance. *See id.,* at 62. These further analyses generally used a 10,000-year return period earthquake as the ground motion input, to eliminate any issue as to whether the analyses used a bounding input;[66] in addition, the analyses conservatively utilized a soil damping factor of 1% of critical damping (based on the spring constant determined and the vibrating weight), and included consideration of resonance effects. *Id.* at 63-64. Drs. Singh and Soler testified that because these analyses used

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[65] In their testimony, Drs. Singh and Soler specifically addressed the claims raised by the State in Subparts C.3.e, and D.1.b, c, e, f, g, h, and i of this contention. *See* Singh/Soler, Post Tr. 5750, at 34-62. With respect to Subparts D.1.a and D.1.d, they deferred to the testimony of Drs. Youngs and Tseng, which is discussed *infra*. *See id.* at 37, 47-48.

[66] Drs. Singh and Soler included some analyses using the 2,000-year return period seismic event, in order to demonstrate the dramatic difference in results which that variation produces, and to provide an independent check of their DYNAMO results. Singh/Soler, Post Tr. 5750, at 63.

the 10,000-year return period earthquake, they bound the 2,000-year design basis seismic event and would bound, by virtue of their increased strength, any issues raised by the State concerning the appropriateness of PFS's evaluation of the response to the 2,000-year design basis earthquake. *Id.* at 62-63.

5.59. Eleven cases were analyzed in these computer simulations, in which different values were utilized for the number of casks on a pad, the stiffness, damping, and coefficient of friction. *See* Singh/Soler, Post Tr. 5750, at 66 (Table). The results of these analyses were described by Drs. Singh and Soler in their testimony, and were also presented in the form of computer animated videos in which various cask motions were visible (PFS Exh. OO). The animation illustrated the following results:

(1) The results of the VN simulation using a 2,000-year return period event and the lower bound set of soil stiffness and damping elements, agree with the results predicted by DYNAMO. To the extent that there may be differences, these are due to the fact that VN recomputes the equilibrium equations at each instant in time and accounts for the changes in orientation (even though they are small) throughout the entire run duration. DYNAMO, by contrast, uses the original equilibrium equations and does not update them continuously. Thus, the results from VN more accurately display slightly larger rotations than those predicted from DYNAMO if the rotations reach the upper end of "small rotations."

(2) The VN simulations using the 10,000 year return period event experience significant rocking behavior and out of phase motion of the casks when the coefficient of friction is 0.8. At certain instants, some casks impact each other with the net result that one of the two casks involved in the impact, slows down almost completely for a period of time following the contact.

(3) For coefficients of friction of 0.2, the casks move in phase and there are no contacts between casks.

(4) No overturning of any cask was experienced in any of the analyses.

(5) Random coefficients of friction reduced the rocking behavior of the casks.

(6) While there was some effect on the system behavior due to "tuning" the soil spring stiffness values to match a input seismic frequency, the major contribution to the large motions was the earthquake strength.

(7) The use of conservatively low soil damping values, while increasing the cask response, does not lead to a condition where severe pad oscillations occur.

(8) Maximum excursions of the pad horizontally are generally below 0.5”.

Singh/Soler, Post Tr. 5750, at 68-69.

5.60. While the design basis event (Case 1) showed very little cask movement, *Id.* at 69, large motions were observed in a 10,000-year event (case 8) in which conservatively “tuned” soil stiffness and 1% soil damping is assumed, with a significant contribution from out-of-phase effects. Despite the orientations observed, however, at the end of the simulation, all of the casks were in a vertical orientation, although perhaps in a new location (e.g., cask 1 came to rest approximately 8” away from its starting point). *Id.* at 70.

5.61. Significantly, the computer simulations performed by Drs. Singh and Soler used input values for earthquake, soil stiffness, and soil damping that were chosen to maximize any deleterious effects (as opposed to using expected real-world values). The results of these analyses demonstrated that the casks and the storage pad, under worst-case scenarios, show no significant detrimental effects that would lead to cask tipover. Accordingly, these bounding analyses confirmed their conclusion that the HI-STORM 100 storage casks will perform satisfactorily in a design basis earthquake at the PFSF site, and are capable of withstanding much larger earthquake events, up to and beyond the 10,000-year return period earthquake. Accordingly, they concluded that none of the State's claims have any merit. *Id.* at 71.

5.62. Finally, Drs. Singh and Soler responded to claims raised by State witness Dr. Khan. They testified that Dr. Khan, who has never before modeled large free standing objects such as the HI-STORM 100 Cask System, ignored authoritative guidance on the modeling of friction contact problems and made fundamentally flawed assumptions in his model that cause the model to predict results that defy the laws of physics. In addition, they explained how Holtec’s computer code has been benchmarked to provide results that correspond to physical reality for the modeling of contact friction problems, and that the model has been reviewed and approved by the NRC as the licensing basis for spent fuel storage systems throughout the country. *See id.* at 71-94.

5.63. Applicant witness Dr. Youngs was part of a Geomatrix team that performed the seismic hazard analysis for the PFSF, and co-authored a Geomatrix Report entitled, "Fault Evaluation Study and Seismic Hazard Assessment, Private Fuel Storage Facility" (*see* State Exh. 185). He was specifically responsible for conducting the probabilistic seismic hazard analysis and developing the design basis ground motions for the PFSF from the results of that study; and was responsible for developing a set of "time histories" to represent the design basis ground motions, and for developing dynamic soil properties for use in the dynamic analyses of the storage cask pads and the Canister Transfer Building at the PFSF. Youngs/Tseng, Post Tr. 5529, at 2.

5.64. Applicant witness Dr. Tseng is President of ICEC, which is the designer of the reinforced-concrete storage pads to be constructed at the PFSF site on which the HI-STORM 100 storage casks will be placed. Dr. Tseng served as an independent reviewer for the ICEC design calculation for the storage pads, and was responsible for assuring the technical adequacy of the design calculations and the design in accordance with ICEC's quality assurance requirements specified in ICEC's Quality Assurance Manual for Nuclear Projects. *Id.* at 3-4.

5.65. In particular, Dr. Youngs described the design basis parameters developed by Geomatrix for the PFSF site, including ground motions; soil properties; soil mass, spring, and damping values to be used for dynamic analyses of the pads; and the time histories to be used for these analyses. He testified that the Geomatrix PSHA followed the general guidance provided for such analyses in NRC Regulatory Guide 1.165; that three-component time histories were developed in accordance with § 3.7.1.2 of NUREG-0800; and that dynamic soil properties were developed which incorporated the uncertainty ranges recommended in § 3.7.2 of NUREG-0800 and ASCE 5-86 for the seismic analysis of safety-related nuclear structures. As he observed, Geomatrix thus followed the same codes and standards that apply to the design of nuclear power plants, except with respect to the "reference probability" used for establishing the PFSF design ground motions (discussed with respect to Part E of this contention, *infra*). *Id.* at 4-9.

5.66. For his part, Dr. Tseng described the analyses performed by ICEC to support the design of the concrete storage pads at the PFSF site. In particular, ICEC prepared a static and dynamic model of the pad/soil system and performed analyses of the pad/soil system under static and dynamic loading conditions to determine the internal stresses in the storage pad. In its pad dynamic analyses, it utilized the cask dynamic response forcing functions at the cask/pad interface boundaries, which were provided to it by Holtec. The internal stresses calculated in the ICEC analysis were then used to determine the amount of reinforcing steel bars required for the reinforced concrete pad to resist the combined stresses in accordance with the project design criteria. Since the design calculation is used to determine internal stresses under design loadings, the pad itself was modeled as a flexible pad supported on flexible soil foundations using a finite-element model for the pad and soil spring representation for the soil foundation. In its design and analyses, ICEC followed American Concrete Institute ("ACI") standard ACI 349-85 (1990), "Code Requirements for Nuclear Safety Related Concrete Structures," and (2) ASCE Standard 4-86; and it followed the guidelines in NUREG-0800, with respect to its seismic soil-structure interaction analyses of the pad/soil system. Thus, ICEC followed the codes and standards that apply to the design and analysis of similar structures for nuclear power plants. *Id.* at 9-11.

5.67. Drs. Youngs and Tseng addressed a number of issues related to claims raised by the State concerning (1) the effect on the storage pad of non-vertically propagating waves and lateral variations in phase, raised in Subparts D.1.a and D.1.d of the Unified Contention; (2) the flexibility or rigidity of the storage pads, raised in D.1.b; (3) the effect of soil cement around the storage pads on pad sliding, raised in D.1.c; (4) the frequency dependency of soil springs and damping values for the storage pad, raised in D.1.e; and (5) the use of multiple time histories and fault fling, raised in D.1.h of the contention. Drs. Youngs and Tseng testified, based on their evaluation that, even if these claims were addressed as sought by the State, the resulting variations in the design analyses would be inconsequential and would not affect the adequacy of the final design. *Id*. at 12.

5.68. With respect to non-vertically propagating waves, Drs. Youngs and Tseng testified that seismic waves will impinge the storage pads with small angles of incidence off the vertical and that within the dominant frequency range of interest for the cask response, the effect of earthquake motions on the pads and the casks resting on the pads at the PFSF may be represented by the use of vertically propagating earthquake waves. The effect of non-vertically propagating waves is insignificant. Youngs/Tseng, Post Tr. 5529, at 12-17.

5.69. With respect to the rigidity of the storage pad, Dr. Tseng testified that the State's reliance on ICEC's accounting for pad flexibility in performing its detailed structural design, to claim that the pad should be considered flexible in evaluating the global dynamic response of the casks and the pads, is misplaced because the purposes of the two analyses are different. He testified that in accordance with recognized authorities in the field of soil structure interaction, the storage pads may be treated as rigid bodies in evaluating their global response. The effect of pad flexibility will be small. *Id.* at 19-25.

5.70. With respect to the effect of the soil cement on pad sliding, Dr. Tseng testified that the pads have a minimum safety factor against sliding of 1.27 and are not expected to slide. The soil cement around the pads will contribute to resisting sliding of the pad on the soil and will limit the amount of sliding if sliding were to occur. However, if the pads slide, Holtec's calculation shows that cask movement would be reduced; accordingly, sliding of the pads would reduce the loads on the casks and would be beneficial, not detrimental, to the stability of the casks. *Id.* at 26. In any event, however, Dr. Tseng testified that the loads imparted by the soil cement surrounding the pads would provide only a relatively small amount of resistance to sliding, and will have only a second order effect on the stability of the casks. *Id.* at 25-27.

5.71. A further issue addressed by Drs. Youngs and Tseng was the frequency dependency of soil spring and damper values. In this regard, they testified that the soil input parameters used by Holtec do not improperly ignore frequency dependency; rather, the foundation soil springs, masses and dampers used by Holtec were developed by Geomatrix in a manner that took into account the frequency-dependency of the soil foundation system. Thus, the resulting foundation impedance functions used by Holtec are a good approximation of the soil foundation impedances for the fundamental frequency of the soil foundation system. *Id.* at 28-30.

5.72. Finally, Dr. Youngs explained the time histories provided by Geomatrix for use in seismic analyses of the PFSF site, and explained how Geomatrix incorporated the effects of "fault fling"[67] in its development of those time histories. In this regard, he explained that Geomatrix accounted for such near-fault effects (described as forward directivity and fault-normal effects) in the set of time histories that it developed for the PFSF site, by using a deterministic worst-case rupture geometry (worst possible rupture location) that maximized their effect, in the PSHA developed for the PFS site. Thus, fault fling was conservatively incorporated into the set of time histories that was used for the design of the PFSF. *Id.* at 31-32.

5.73. Applicant witness Paul Trudeau served as Stone and Webster's lead geotechnical engineer for the PFSF, and was responsible for preparing two of the calculations challenged by the State in Part D of this contention: Calculation No. 05996.02-G(B)-04, Rev. 9, "Stability Analyses of Cask Storage Pads" (July 26, 2001) (PFS Exh. UU); and Calculation No. 05996.02-G(B)-13, Rev. 6, "Stability Analyses of Canister Transfer Building" (July 26, 2001) (PFS Exh. VV). Trudeau, Post Tr. 6135, at 2.

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[67] "Fault fling" is a term that is generically used to describe enhanced ground motions that have been observed in a number of earthquake recordings obtained very near to the causative fault rupture. Youngs/Tseng, Post Tr. 5529, at 31. It has also been described as a phenomenon involving large velocity pulses in the time history. Pomerening/Ofoegbu, Post Tr. 6496, at 26.

5.74. In his testimony, Mr. Trudeau explained that the goal of seismic stability analyses such as these two calculations is to evaluate three potential failure modes for the structures involved: sliding stability, overturning stability, and bearing capacity stability. Sliding failure occurs if the structure moves horizontally, parallel to the ground. Overturning failure occurs if the structure rotates as a rigid body about a horizontal axis. Bearing capacity failure takes place if the soils beneath the structure become overloaded in the vertical direction, leading to excessive settlement or rotation of the structure's foundation. The intent of the analyses is to establish what margin or "factor of safety" is provided by the design of the structure's foundations against each of these three failure modes. In this regard, the NRC's regulatory guidance in NUREG-0800, § 3.8.5 ("Foundations") indicates that the factors of safety against overturning and sliding are acceptable if they exceed 1.1 for load combinations that include seismic loads due to the design basis earthquake. *Id.* at 3; Staff Exh. EE, at 3.8.5-7. *See also* NUREG-0800, § 3.8.4 ("Other Seismic Category I Structures") (Staff Exh. 64), at 3.8.4-7 to 3.8.4-8 ("Loads and Load Combinations").[68]

5.75. Mr. Trudeau explained the various cases analyzed in the Geomatrix calculations, and the conservatisms contained therein. For example, the base case did not take credit for the strength of the soil cement around the pads to resist sliding forces, and it conservatively used the shear strength of the underlying clayey soils based on static strength direct shear tests, despite the fact that the shear strength would be substantially higher when the soil is subjected to rapid

[68] Mr. Trudeau explained that where a design does not meet a factor of safety of 1.1, sliding or overturning failure would not necessarily occur; rather, failure might occur only when the results of the analysis predict a factor of safety of less than 1.0. Mr. Trudeau testified, however, that even then, sliding may not occur because (a) the analyses include additional conservatisms, and (b) due to the cyclic nature of the seismic loading, each of the peak accelerations used to estimate the dynamic loads from the earthquake exists only for one very brief moment in time (typically less than 0.005 seconds) before the earthquake accelerations reverse direction. Therefore, the structures are not expected to experience significant horizontal displacement. Also, he observed that there would be only one point in time where the acceleration equals the maximum value, whereas the analyses assume that the forces due to these peak accelerations act continuously for purposes of computing the factor of safety. Trudeau, Post Tr. 6135, at 4.

loadings such as would be imparted by the design basis earthquake; this would result in increased factors of safety against sliding and overturning of the pads, beyond the values submitted in the PFS analyses. Other conservatisms were utilized as well, in accordance with applicable codes and standards, such as the use of lower bound, rather than upper bound or best estimate values for soil properties, such that the foundations will have greater factors of safety than the calculations predict. Trudeau, Post Tr. 6135, at 4-9.

5.76. Stone and Webster calculated the factors of safety against sliding of the pads in the design basis earthquake (ignoring the passive resistance to sliding of the soil cement adjacent to the pads) as 1.27 in the east-west direction and 1.36 in the north-south direction. Greater factors of safety against sliding were calculated if the passive resistance of the adjacent soil-cement is considered. In addition, Stone and Webster considered the sliding stability of a north-south column of 10 pads, and found that the sliding stability is increased over that of an individual pad. Other calculations were performed, including a calculation in which the cohesive portion of the strength of the clayey soils at the interface with the cement treated soil under the pads was ignored (such that resistance to sliding was provided only by the frictional portion of the soils' shear strength); in that case, sliding was found to occur, for both a single pad and a row of pads. *Id.* at 9-10.

5.77. Mr. Trudeau concluded, based on Stone and Webster's analyses of sliding stability, bearing capacity, and overturning stability of the foundations of the storage pads, that significant margins are available for those foundations in the event of a design basis earthquake. These factors of safety, which incorporate a number of conservative assumptions, assure that the pads and the storage casks will remain stable under the loads imparted by the design basis earthquake. Moreover, based on the results of these analyses and consideration of the conservatisms contained therein, he opined that the storage pads will not experience failure under the loadings from an earthquake far more severe than the design basis earthquake. *Id.* at 12-13.

5.78. In addition, Mr. Trudeau addressed various allegations raised by the State in Part D of this contention, concerning PFS's seismic analyses of the storage pads, casks, and their foundation soils, and the CTB and its foundation. Specifically, he addressed allegations concerning (1) the potential lack of rigidity of the storage pads and its effect on the stability analysis of the pads, raised in Subpart D.1.b(i) of the contention; (2) the potential effect of soil cement around the pads once the pads undergo sliding motion, raised in D.1.c(i); (3) the effect of potential pad-to-pad interaction on the sliding analysis of the pads, raised in D.1.g; and (4) potential out of phase motion between the CTB and the soil cement placed around the building's foundations, raised in D.2.c. *Id.* at 13-22.

5.79. With respect to the State's allegation that the stability analyses incorrectly assume that the storage pads behave rigidly under design basis earthquake loads, Mr. Trudeau cited the testimony of Dr. Tseng, in support of the view that the pads are essentially rigid, such that the assumption of pad rigidity is proper. Accordingly, the earthquake dynamic loads have not been underestimated in the stability analyses. In addition, he demonstrated that the use of peak ground acceleration in the seismic analyses was appropriate and did not underestimate the accelerations to which the pads will be subjected. Moreover, even if an alternative calculation is performed, using the time history of forces from Holtec's SSI analysis, the factor of safety against pad sliding would be 1.25 rather than 1.27 (*i.e.*, it still meets the 1.1 value stated in § 3.8.5 of NUREG-0800). *Id.* at 13-16.

5.80. Mr. Trudeau also refuted the State's claim in Subpart D.1.c(i) of the contention, that the presence of soil cement and cement-treated soil adjacent to the storage pads will introduce unsymmetrical loadings on the pads once the pads undergo sliding motion in an earthquake; and he explained why the Newmark sliding block analyses for the pads is not rendered invalid for failure to consider unsymmetrical loadings. *Id.* at 16-18. Further, he addressed the State's claim that the Applicant had failed to consider the potential for pad-to-pad interaction, explaining that shear distortions within the soil cement and concrete pads due to the upward propagation of seismic waves should be very small; therefore, the pad and soil cement plug between the pads should deflect in phase with the underlying soils, such that interaction between the pads will be insignificant. *Id.* at 18-19.

5.81. In addition, Mr. Trudeau addressed the State's claim in Subpart D.2.c of the contention, concerning the potential for crack formation due to out-of-phase motion of the CTB relative to the adjacent soil-cement cap. In this regard, he examined the potential formation of cracks in the soil cement that is to be placed around the foundations of the CTB due to out of phase motion between the soil cement and the building, and showed that such cracks, if they form, will have little or no impact on the soil cement's ability to provide passive resistance against sliding of the CTB. *Id.* at 19-21.

5.82. Based on Stone and Webster's analyses of the sliding stability, bearing capacity, and overturning stability of the CTB, Mr. Trudeau concluded that adequate factors of safety are available for those foundations in the event of a design basis earthquake. These factors of safety, which incorporate a number of conservative assumptions, assure that the CTB will not be subject to failure under the loads imparted by the design basis earthquake. Moreover, based on the results of the analyses and the conservatisms contained therein, he concluded that the CTB will not experience failure under the loadings from an earthquake significantly more severe than the design basis earthquake. *Id.* at 21-22.

5.83. The Applicant's final piece of pre-filed testimony on Part D of this contention was provided by Bruce Ebbeson, Senior Lead Structural Engineer at Stone and Webster in Cherry Hill, N.J. Mr. Ebbeson was responsible for the seismic analysis and structural design of the CTB at the PFSF. In his testimony, he described the purpose and uses of the CTB, as well as its structural design and the ability of the CTB to withstand seismic loadings. Ebbeson, Post Tr. 6357, at 2-12.

5.84. Mr. Ebbeson explained that the CTB is a reinforced concrete structure with thick walls which provide tornado-generated missile protection and radiation shielding. The CTB's main function is to facilitate the safe performance of canister transfer operations at the PFSF. The important-to-safety SSCs in the CTB include a 200 ton overhead bridge crane, a 150 ton semi-gantry crane, seismic support struts, the spent fuel canisters, shipping and storage casks, and transfer casks used during the canister transfer operation. *Id.* at 3.

5.85. Pertinent structural features of the CTB include the following elements. The roof consists of an eight-inch thick reinforced concrete slab supported on structural steel beams spanning in the N-S direction, which are in turn supported by plate girders spanning in the E-W direction. There are studs on the beams and girders to prevent the roof slab from uplifting during a design basis tornado. The CTB is supported by a heavily reinforced concrete foundation mat, which is 240 ft in the E-W direction, 279.5 ft in the N-S direction, and 5 ft thick. A reinforced concrete key, 1.5 ft deep by 6.5 ft wide, will be constructed around the perimeter of the foundation mat; the purpose of this key is to ensure that the full shear strength of the clayey soils beneath the foundation is available to resist sliding of the structure due to the loads from the design basis ground motion. Soil cement is to be placed around the base mat to help resist earthquake sliding forces; it will extend approximately 240 ft out from the mat in the E-W direction and approximately 280 ft out in the N-S direction. The soil cement around the CTB will be 5 ft thick and have a minimum unconfined compressive strength of 250 psi; the top 8 inches will be filled with compacted aggregate, similar to that used in the pad emplacement area. *Id.* at 5.

5.86. Mr. Ebbeson testified (similar to other Applicant witnesses) that in its seismic design of the CTB, PFS followed the guidance in NUREG-1567 (the Standard Review Plan for ISFSIs); and that the CTB seismic design used the same criteria that are used to meet the safe shutdown earthquake loads specified for nuclear power plants in NUREG-0800. Both of these SRPs provide load combinations and acceptance criteria which, for the loads applicable to the PFSF, are very similar; and both codes provide similar degrees of conservatism. The codes and standards followed by PFS in its seismic analysis and design of the CTB include other codes and standards that apply to nuclear power plants, including ASCE 4-86 and AISC N-690. To the extent pertinent for ISFSIs, the load combinations and acceptance criteria for the CTB under seismic loadings are the same as those specified in NUREG-0800 for the safe shutdown earthquake loadings for nuclear power plants. *Id.* at 3-4.

5.87. The codes and standards utilized in the seismic design of the CTB include various conservatisms, such as a requirement that stresses resulting from the design basis earthquake are to be limited to levels below the specified yield point of the materials -- thus giving no credit to the fact that concrete and steel structures have substantial deformation capacity above and beyond the point of first yielding. In contrast, codes used to design conventional buildings, such as the Uniform Building Code, increase allowable seismic loads for ductile structures. For this and other reasons, the Applicant's use of these codes results in conservative design values, far greater than if the CTB was designed to the 1994 Uniform Building Code. *Id.* at 6-7.

5.88. Mr. Ebbeson testified that the CTB and the structures it contains have a reserve capacity that would allow them to resist seismic loadings beyond those from the design basis earthquake. This reserve capacity exists due to many factors, including a redistribution of stresses from highly stressed areas of the structure to adjacent areas which occurs after yielding; the fact that the actual material yield strength (for concrete, the compressive strength) exceeds the nominal yield strength values; the fact that the materials' ultimate strength is significantly greater than its yield strength; and the nature of the seismic loads, which are of short duration and reverse direction several times each second, so that the load will likely reverse direction before significant distortion can occur and the stresses will return to the elastic range. *Id.* at 7. In addition, he found a number of conservatisms in the seismic design of the foundations of the CTB, and concluded that the CTB will not overturn under beyond-design basis 10,000-year return period earthquake loadings.

Id. at 8-9. In addition, conservatisms in the CTB design assure that the CTB roof, cranes, seismic struts and other important-to-safety SSCs will be able to withstand a beyond-design-basis earthquake much more severe than the 2,000-year return period. *Id*. at 9-11.[69]

5.89. In sum, Mr. Ebbeson concluded that the CTB and all important-to-safety SSCs it contains possess far greater seismic loading capacities than those for which they were nominally designed, and have margins greater than those which have been calculated. Consequently, the CTB and the important-to-safety SSCs it houses can withstand acceleration levels well in excess of those associated with the design basis earthquake and have a high likelihood of surviving without loss of safety function in an earthquake with a return period significantly greater than the 2000 years of the design basis earthquake. *Id.* at 12.

5.90. Mr. Ebbeson also addressed the allegations raised by the State in Section D.2 of the contention concerning the seismic design of the CTB and its foundation. In particular, he addressed (1) the allegations concerning the potential lack of rigidity of the CTB basemat and its effect on the underestimation of the dynamic loading on the foundation, raised by the State in Subpart D.2.a(i) of the contention; (2) the potential lack of rigidity of the CTB basemat and its effect on the overestimation of foundation damping, raised in D.2.a(ii); (3) the potential effect of the soil cement on the soil impedance parameters, raised in D.2.b(i); (4) the potential interaction between the CTB foundation mat and the surrounding soil cement, raised in D.2.b(ii); and (5) the potential effect of non-vertically propagating waves on the rocking and torsional motion of the CTB and its foundations, raised in D.2.d. *Id*. at 12-19.

5.91. Mr. Ebbeson testified that the Applicant's treatment of the CTB base mat as a rigid body is correct and in accordance with industry codes and standards, including § 3.3.1.6 of

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[69] We note that the Staff has found the Applicant's design to be acceptable, but did not find it necessary under current NRC regulatory practice to reach a conclusion as to the extent of the margins present in the design of important-to-safety SSCs.

ASCE 4-86. Further, the maximum amount of deflection calculated for the CTB basemat along the centerline of the 279.5-ft N-S direction is 0.164 inch -- which amounts to a 0.005% deflection; in the 240-ft E-W direction, 0.334 inch has been calculated -- amounting to a 0.05% deflection. This small amount of deflection further justifies the assumption of rigidity and the conclusion that it will have no impact on the dynamic loadings on the CTB or foundation damping. Nor would this assumption of rigidity lead to an overestimation of damping, but in any event, other margins in the CTB foundation design would compensate for it. *Id.* at 13-15.

5.92. With respect to the effect of the soil cement adjacent to the CTB on soil impedance parameters (i.e., frequency-dependent spring and damping parameters used to characterize the soil in soil-structure interaction analyses), Mr. Ebbeson testified that any impact from soil cement around the foundations would be minimal and can be disregarded in accordance with standard industry practice and § 3.3.4.2.4 of ASCE 4-86. Moreover, the SSI analysis was done with three sets of impedance functions to cover possible variations in soil properties, and the most conservative (i.e., least favorable) results were used for design of the CTB. This enveloping technique would account for any minor variations in soil impedance, caused by soil cement or other conditions. *Id.* at 15-17.

5.93. Mr. Ebbeson discussed two other issues raised by the State: kinematic interaction between the CTB and the soil-cement around the building; and non-vertically propagating waves. With respect to the first of these issues, he explained that the presence of soil cement was included by Geomatrix in developing the free-field motion and strain-dependent soil property inputs to Stone and Webster's CTB seismic analyses. His review of the Geomatrix results led him to conclude that the Geomatrix input was conservative, as was the Stone and Webster analysis. *Id.* at 17-18.

5.94. With respect to the latter issue (non-vertically propagating waves), Mr. Ebbeson endorsed the testimony of Drs. Youngs and Tseng concerning this issue and its alleged effect on the concrete storage pads. Further, he demonstrated that Stone and Webster followed the guidance in ASCE 4-86, which allows an assumption that incoming seismic waves are vertically-propagating as long as a mass eccentricity factor of 5% is incorporated into the actual design -- which applies here; and he cited deposition testimony by State witness Dr. Ostadan in agreement, thus eliminating this issue from concern. *Id.* at 18-19.

NRC Staff's Testimony

5.95. As stated above, the NRC Staff presented two panels of witnesses: The first panel (consisting of Mr. Pomerening and Dr. Ofoegbu), addressed the adequacy of the Applicant's seismic design and supporting analyses, while the second panel (consisting of Dr. Luk and Mr. Guttmann) discussed the Staff's independent confirmatory analysis of cask behavior during a seismic event at the PFSF site. As discussed below, the Staff's testimony provides firm support for a conclusion that the Applicant's seismic design and supporting analyses are acceptable, and that the HI-STORM 100 storage casks at the proposed PFSF site will not tipover or experience excessive sliding in the event of either a 2,000-year return period design basis ground motion or a more severe 10,000-year earthquake.

(a) Staff Evaluation of the PFS Application

5.96. Staff witnesses Mr. Daniel Pomerening and Dr. Goodluck Ofoegbu provided the Staff's views concerning the foundation stability and seismic design of the PFSF. Mr. Pomerening participated in the Staff's review of the PFS application, with respect to design requirements related to the proposed PFSF. He was involved in the Staff's evaluation of structures, systems, and components ("SSCs") important to safety at the proposed PFSF; identification of design criteria and design bases, including natural phenomena events; and identification and analysis of hazards for off-normal, accident and design basis events involving SSCs that are important to safety. Dr. Ofoegbu participated in the Staff's evaluation of the Applicant's site characterization and geotechnical analyses of the proposed PFS Facility. Pomerening/Ofoegbu, Post Tr. 6496, at 2-3.

5.97. Both Mr. Pomerening and Dr. Ofoegbu assisted in preparation of the Staff's SER of September 29, 2000, and SER Supplement No. 2 dated December 21, 2001 (incorporated in the Staff's Consolidated SER (Staff Exh. C). The Staff's evaluation of the seismic design and the foundation stability of the PFS facility is set forth, *inter alia*, in the following sections of the Consolidated SER: §§ 2.1.6.4 (Stability of Subsurface Materials); 5.1.1 (Confinement Structures); 5.1.3 (Reinforced Concrete Structures); 5.1.4 (Other [SSCs] Important to Safety); and 15.1.2 (Accidents) (in particular, § 15.1.2.1 (Cask Tipover), § 15.1.2.2 (Cask Drop), and § 15.1.2.6 (Earthquake)). *See* Pomerening/Ofoegbu, Post Tr. 6496, at 3-4.

5.98. Dr. Ofoegbu and Mr. Pomerening testified that the Staff has concluded that the Applicant properly demonstrated that the proposed PFSF structures and foundations have adequate factors of safety to sustain the dynamic loading from the proposed 2,000-year design basis earthquake, and that the seismic design and foundation stability of the proposed PFS Facility satisfy all applicable regulatory requirements. *Id.* at 9.

5.99. In particular, the Staff determined that the foundation stability for the storage pads and canister transfer building is adequate. Calculations provided by PFS were found to 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 concluded that the HI-STORM 100 storage casks will not tipover or collide due to a design basis ground motion; that the concrete storage pads will perform their safety functions and 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 concluded 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. *Id.* at 9-15, 28-30.

5.100. In sum, Dr. Ofoegbu and Mr. Pomerening disagreed with the State's assertion in Part D of this contention, that the Applicant had "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)". To the contrary, they explained that, 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 PFSF 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). *Id.* at 8-9.

5.101. Dr. Ofoegbu and Mr. Pomerening addressed each of the matters raised in Part D of this contention, as summarized below.

Part D.1: Casks, Storage Pads, and Their Foundation Soils

5.102. First, Dr. Ofoegbu and Mr. Pomerening disagreed with the State's assertion in Subpart 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"). In this regard, they addressed the issues of both (a) storage pad foundation stability, and (b) the seismic design of the storage pads and casks. *Id.* at 9-10.

5.103. With respect to storage pad foundation stability, Dr. Ofoegbu testified (as described in his testimony on Part C.3.d of the contention and in § 2.1.6.4 of the Consolidated SER), that the Staff has concluded that the Applicant's storage pad design 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. *Id.* at 10.

5.104. 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. *Id.*

5.105. 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. *Id.*

5.106. 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. Thus, settlement of the pads does not present a foundation stability concern. *Id.* at 10-11.

5.107. With respect to the seismic design of the storage pads and casks, Mr. Pomerening testified that, based on a review of the PFS application and supporting analyses and calculations, the Staff has concluded that PFS demonstrated 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). *Id.* at 11.

5.108. This conclusion is based on the following considerations. First, as discussed above, the Staff 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 storage pad foundation. *Id.*

5.109. Second, as summarized in §§ 5.1.4.4 and 15.1.2.6 of the Consolidated SER, the Staff has concluded that 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, at the PFSF site. This conclusion is based on the Staff's review of the Holtec PFS site-specific analysis, which demonstrated that the HI-STORM 100 storage casks will not tip over or collide into each other in the event of a PFSF design basis ground motion. *Id.*

5.110. The Staff observed that 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 displacement 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 (Staff Exh. C), at pages 5-30 and 15-32. *Id.* at 11-12.

5.111. Further, the Staff reviewed a site-specific analysis of a non-mechanistic, hypothetical cask tipover scenario, which the Applicant performed to evaluate the performance of the HI-STORM 100 storage cask design at the proposed PFS site. 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 100 SER"). In its review of the PFS site-specific tipover analysis, the Staff reviewed the Applicant's method of analysis, inputs, assumptions, and conclusions. Based on its review of the PFS analysis, the Staff concluded that the deceleration in a hypothetical cask tipover event is less than the 45 g limit of the multi-purpose canister, and the resulting stresses in the MPC within the HI-STORM 100 storage cask will be lower than the stresses evaluated in the HI-STORM 100 FSAR. Accordingly, the Staff 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. *Id.* at 12-13.

5.112. 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, § 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. *Id.* at 13.

5.113. 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. *Id.* at 13-14.

5.114. Dynamic analyses for the storage pads were also performed by PFS, for the PFS 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 to accommodate the site-specific seismic loading conditions. *Id.* at 14.

5.115. As explained by Mr. Pomerening, 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, as set forth in the Consolidated SER (§§ 2.1.6.4, 4.1.3.2, 5.1.1, 5.1.3, 5.1.4, and 15.1.2.6), the Staff concluded that PFS's 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 the requirements of 10 C.F.R. § 72.122(b)(2). *Id.* at 14-15.

5.116. Mr. Pomerening further disagreed with the State's assertion in Part D.1.a of the contention that the Applicant's calculations unconservatively fail to account for non-vertically propagating waves. He explained, as set forth in the Consolidated SER, § 2.1.6.2 (Ground Vibration and Exemption Request), the design ground motion response spectra for the proposed PFS site were developed by Geomatrix, based on its site-specific PSHA results and the procedures outlined in NRC 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 PFS PSHA. This indicates that non-vertically propagating in-phase waves are accounted for in the PFS calculations. *Id.* at 15-16.

5.117. Mr. Pomerening addressed a calculation by Geomatrix concerning the spatial and temporal variations of ground motions for the PFSF site, in which it concluded: (1) the assumption of vertically propagating waves is reasonable for the site, considering the angle of incidence of the seismic waves; (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. Mr. Pomerening discussed these Geomatrix conclusions with Dr. Martin McCann (a Staff witness with respect to Part E of the contention), who concurred in the Geomatrix conclusions. *Id.* at 16.

5.118. Mr. Pomerening provided a number of reasons in support of his agreement with Geomatrix that non-vertically propagating waves will not cause additional rocking and torsional motion in the casks, pads and foundations. These included consideration of: (1) the ray path and low angle of incidence (measured from the vertical) for seismic waves at the surface relative to source points on the Stansbury and East faults, such that an assumption of vertically propagating waves is reasonable for the site; and (2) the extremely small (0.001 to 0.002 second) variations in arrival times for seismic waves at the surface relative to source points on the Stansbury and East faults, such that a difference in arrival time would affect only frequencies above the highest ground motion frequency of interest (*i.e.,* 50 Hz), and an assumption of in-phase waves is therefore reasonable for the PFSF site. *Id.* at 16-17, 23.

5.119. In addition, Mr. Pomerening testified that this conclusion is supported by consideration of the Applicant's localized inputs into its structural analysis of the storage pads, with respect to seismic waves that may occur in the soil, including the Applicant's use of three orthogonal and statistically independent time histories to characterize the motion at the surface (whereby acceleration is defined for two orthogonal horizontal directions and the vertical direction). The vector sum of these three components results in input motion that is random with respect to amplitude and direction, such that the Applicant's input to the pads, casks, and foundation may be considered to have included motions other than vertically propagating waves. *Id.* at 17.

5.120. Further, Mr. Pomerening explained that the storage pad's geometry (30 ft x 67 ft) has an influence on pad motion with respect to the wave length of the seismic waves. He considered the shear and compression wave velocities in the soil (which define the velocity at which the seismic waves will propagate through the site), and the frequencies having the greatest displacement potential in either the 30 ft (E-W) direction or 67 ft (N-S) direction; and he calculated the maximum amount of rocking of the storage pads that may result. He concluded that the maximum vertical deflection satisfies the requirement of Table 9.5(a) of ACI 349 both for the 30 ft (E-W) direction ($\delta_{max} \leq$ 1.8 inches = 30*12/200) and for the 67 ft (N-S) direction ($\delta_{max} \leq$ 4.2 inches = 67*12/200). Inasmuch as the amount of rotation of the surface of the storage pad is less than 0.1 degrees in either the E-W or N-S direction, the stability of the cask will not be affected by non-vertically out-of-phase seismic waves that may occur at the site. *Id.* at 17-19.[70]

5.121. Mr. Pomerening also disagreed with the State's assertion in Part D.1.b of the 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." As stated 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.

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[70] In effect, the bounding displacements of the storage pad due to seismic waves is small relative to the overall geometry of the storage pad. Pomerening/Ofoegbu, Post Tr. 6496, at 19.

Accordingly, for all practical purposes, the storage pad can be assumed to be a rigid element. *Id.* at 19.

5.122. Two different assumptions were made by the Applicant with respect to pad rigidity. First, in the Applicant's calculations of the response of multiple casks on the storage pad due to the 2000-year seismic event (Holtec Report No. HI-2012640), the storage pad was conservatively assumed to be rigid.[71] The rigid storage pad assumption produces 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. *Id.* at 19-20.

5.123. Second, in the calculations for the storage pad analysis and design, G(PO17)-2, the storage pad was assumed to be flexible, in order to identify the amount and placement of reinforcing required to resist the maximum or bounding loads that need to be considered in the design of the pads. A summary of the maximum displacements is 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 are small relative to the overall geometry

[71] Staff witness Mr. Pomerening indicated that the HI-2012640 analysis, concerning the stability of the casks on the storage pad, follows the guidelines in ASCE 4-86, Section 3.1.8. Pomerening/Ofoegbu, Post Tr. 6496, at 19.

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 Staff concluded that PFS did not incorrectly assume that the pads will behave rigidly during the design basis earthquake, and did not significantly underestimate the dynamic loading atop the pads, in the horizontal or vertical direction. *Id.* at 20.[72]

5.124. Next, Mr. Pomerening provided the Staff's views with respect to the State's assertion in Part D.1.c. of the 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. Mr. Pomerening disagreed with the State's assertion as discussed below. *Id.* at 21-23.

5.125. First, with respect to unsymmetrical loading on the pads due to the soil-cement around the pads, the Staff concluded that 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

[72] Similarly, the Applicant's "assumption of rigidity" for the pad does not lead 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. Pomerening/Ofoegbu, Post Tr. 6496, at 20-21.

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 (static) modulus of elasticity was calculated to be 3,120,000 psi. PFS has indicated that it will use soil cement around the storage pad with a nominal unconfined compressive strength of 250 psi and a density of 100 pcf. Based on information in published literature, it is estimated that 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). Accordingly, the soil/cement is much softer than the concrete storage pad, and will tend to crush under impact with the storage pad. This crushing will distribute any loading over a 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, due to the use of soil/cement around the storage pads. *Id.* at 21-22.

5.126. 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, and the pads need not be assumed to be flexible in this calculation. *Id.* at 22.

5.127. 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). *Id.* at 22-23. [73]

5.128. Mr. Pomerening further testified that the State's assertion in Part D.1.d. of the 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," does not present a valid concern. In this regard, he reiterated the Staff's view that the assumption of vertically propagating in-phase waves at the site is appropriate. Accordingly, any lateral variations in the phase of the ground motion and their effect on the stability of the storage pads and casks will be insignificant. *Id.* at 23.

5.129. Mr. Pomerening also disputed the State's assertion, in Part D.1.e. of the 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." *Id.* at 24. The Holtec calculation of the impact of multiple casks on the storage pads due to seismic loading (Holtec Report No. HI-2012640) 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

[73] 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. *See* ¶ 5.130, *infra*.

with casks on the soil.[74] 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. *Id.* at 24.

5.130. In addition, the Staff disputed the State's assertion in Part D.1.f. of the 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." Mr. Pomerening explained that 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 -- *i.e.*, 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. Mr. Pomerening calculated the resulting initial strain in the concrete, utilizing Section 8.5 of ACI 349-90, as well as the influence of long term creep as identified in ACI 209, "Prediction of Creep, Shrinkage, and Temperature Effect in Concrete Structures." 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. *Id.* at 24-25.

5.131. Mr. Pomerening then considered the State's assertion, in Subpart D.1.g. of the 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." He

[74] 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." Pomerening/Ofoegbu, Post Tr. 6496, at 24.

concluded that this is not a valid concern, and agreed with PFS that potential sliding of the storage pads under seismic loading would not constitute a safety hazard. As discussed in ¶ 5.125, *supra,* the soil-cement between the pads will tend to crush under seismic loading, and that crushing will limit the magnitude of the force that can be transmitted from one pad to another.[75] 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 for pad-to-pad interaction need not be considered in the Applicant's sliding analysis and structural design of the storage pads, for pads spaced approximately five feet apart in the longitudinal direction (or even greater distances in the latitudinal direction). *Id.* at 24-25., for pads spaced approximately five feet apart in the longitudinal direction (or even greater distances in the latitudinal direction). *Id.* at 24-25.

5.132. The Staff further disputed the State's assertion, in Part D.1.h. of the contention, that more than one set of time histories should have been used in the Applicant's non-linear analysis, due to the sensitivity of non-linear analyses to the phasing of input motion and the effects of fault fling (*i.e.*, large velocity pulses in the time history). Holtec's nonlinear analysis (Holtec Report No. HI-2012640) was based on only one set of time histories. These time histories were shown to be random with respect to both amplitude and direction, as discussed above. The Staff concurred 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, § 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, which is significantly less than the 48-inch separation

[75] Significantly, both Dr. Bartlett and Dr. Ostadan agreed that the soil-cement between the concrete storage pads will crush in the event of pad movement during a seismic event, thus dissipating some of the energy that might have been transferred from one pad to its neighboring pad. Tr. 10214, 10708-10.

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. Thus, the margin of safety is sufficiently high that multiple sets of time histories are not warranted. *Id.* at 26-27.[76]

5.133. With respect to "fault fling," the Staff concluded that this phenomenon is not applicable here, in that it is not an appropriate consideration for the type of faulting (*i.e.,* normal faulting) present at the proposed PFS site. Rather, 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. *Id.* at 27.

5.134. Finally, the Staff disagreed with the State's assertion in Part D.1.i of the contention, that the Applicant failed to demonstrate the stability of the free standing casks under design basis ground motions. The Staff determined that the Applicant's analyses support its conclusions that excessive sliding and cask collisions will not occur and that the casks will not tip over. The Staff concluded that PFS 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

[76] As noted by Mr. Pomerening, 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. Pomerening/Ofoegbu, Post Tr. 6496, at 27. *See* discussion *infra* at 154-65; Staff Exh. P at 30 (Table 8), 32 (Table 10), and 39-40; Tr. 7104-05.

adjacent casks, or slide off the pads under the dynamic loading from the DBE. In sum, based on its review of Holtec's site-specific analysis for the proposed PFSF, the Staff concluded 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 PFS site. *Id.* at 28.

Part D.2 (Canister Transfer Building and its Foundation).

5.135. Subpart D.2 of the contention concerns the Applicant's seismic analysis for the Canister Transfer Building and its foundation. Here, the State asserts that PFS "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,'"[77] because of certain alleged "errors and unconservative assumptions" made by PFS in determining the dynamic loadings to the CTB and its mat foundation. Dr. Ofoegbu and Mr. Pomerening disputed these assertions, with respect to both the foundation stability and the seismic design of the CTB. Pomerening/Ofoegbu, Post Tr. 6496, at 29-37.

CTB Foundation Stability

5.136. An applicant for an ISFSI at a non-bedrock site is required, pursuant to 10 C.F.R. §§ 72.102(c)-(d), 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. *See* Pomerening/Ofoegbu, Post Tr. 6496, at 29.

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[77] NUREG-75/087 has been superseded NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants." Pomerening/Ofoegbu, Post Tr. 6496, at 29. In turn, Section 3.8.5 ("Foundation") in NUREG-75-087 has been superseded by Section 3.8.5 of NUREG-0800 ("Foundations"), which was admitted into evidence as Staff Exh. EE.

5.137. Dr. Ofoegbu presented the Staff's conclusions (as set forth in § 2.1.6.4 of the Consolidated SER) that PFS has provided an adequate geotechnical site characterization, and the proposed design of the CTB foundation satisfies the requirements of 10 C.F.R. §§ 72.102(c)-(d) and 72.122(b)(2) with respect to the capability of the underlying soil to provide adequate support to the foundation.[78] This conclusion is based on three considerations. First, the PFS calculations 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; this 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, the PFS calculations demonstrate adequate safety margins against sliding of the CTB foundation under potential dynamic loading from the design-basis earthquake, in accordance with § 3.8.5 of NUREG-0800. Third, PFS calculations 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 -- which is smaller than the maximum tolerable settlement of the CTB foundation; thus, settlement of the CTB and its underlying concrete mat does not present a foundation stability concern. Pomerening/Ofoegbu, Post Tr. 6496, at 29-30.

<u>Seismic Design of the Canister Transfer Building</u>.

5.138. Mr. Pomerening addressed the CTB seismic design. The Staff concluded that 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, as required by 10 C.F.R. § 72.24(d). Further, the Staff determined that the proposed CTB design 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. Details concerning the

[78] *See also* Ofoegbu Testimony Post Tr. 11011, at 16-19, concerning Subpart C.3.d of this contention.

bases for this conclusion are provided in the Consolidated SER, §§ 2.1.6.4, 4.1.3.2, 5.1.3, 5.1.4, and 15.1.2.6, and are discussed below. Pomerening/Ofoegbu, Post Tr. 6496, at 30.

5.139. First, as discussed by Dr. Ofoegbu above (¶ 5.138), the Staff 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 § 3.8.5 of NUREG-0800. Second, PFS has committed to an appropriate structural analysis process for the CTB to mitigate environmental effects, utilizing the ultimate strength method of analysis set forth in ACI 349-90, with appropriate load factors. The CTB 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 exceeds the demand, and they would therefore be able to perform their intended safety functions under extreme environmental and natural phenomena conditions in accordance with 10 C.F.R. §§ 72.122(b)(1)-(2). *Id.* at 30-31.

5.140. Mr. Pomerening further testified that 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 CTB 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 CTB, as is appropriate. *Id.* at 31.

5.141. 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 updated its seismic load analysis to reflect the physical changes in the building design, which were made as a result of the updated design-basis seismic conditions. (The Applicant also plans to update the detailed design of the CTB prior to construction.) Based upon the Staff's review of the Applicant's design criteria and the process utilized in developing its previous building design, and the Applicant's statement in the SAR (§ 4.7.1.5.3) that the changes to the detailed design will follow the same design criteria and the process, the Staff concluded that the design of the CTB for the design-basis earthquake loads is acceptable. *Id.* at 31-32.[79]

5.142. For the foregoing reasons, the Staff has concluded that the adequacy of PFS's analysis is not affected by "errors" or "unconservative assumptions" in determining the dynamic loadings to the CTB and its mat foundation. *Id.* at 32.

5.143. Next, Mr. Pomerening disputed the State's assertion, in Part D.2.a. of the contention that PFS's calculations "incorrectly assume that the mat foundation will behave rigidly during the DBE," allegedly leading to a significant underestimation of the dynamic loading to the mat foundation and overestimation of foundation damping. *Id.* at 32-33.

5.144. In this regard, Mr. Pomerening provided the Staff's views that the Applicant's assumption of rigidity does not lead 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

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[79] PFS also submitted detailed analyses for the upper and lower roof steel (SC-12), and the rolling doors for the transfer cells (SC-14). PFS 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 these elements have sufficient capacity to meet the demands under all loading conditions. As identified in § 5.1.4.4 of the Consolidated SER, 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. Pomerening/Ofoegbu, Post Tr. 6496, at 32.

foundation due to the 2000-year seismic event (Stone & Webster Calculation SC-5), the mat foundation is conservatively assumed to be rigid. This 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." This analysis is to be used to define the loads in analyses that PFS 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. 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. Thus, a determination of loads based on a rigid mat foundation assumption will result in an upper bound design load estimate. *Id.* at 33.

5.145. In contrast, in the procedures used for PFS'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). *Id.*

5.146. Using a procedure similar to that identified in ¶ 5.121 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. *Id.* at 33-34.

5.147. 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 Staff's calculation showed that the maximum response displacement is: V = 3.4 inches; this maximum vertical deflection satisfies the requirement of Table 9.5(a) of ACI 349 ($\delta_{max} \leq$ 14.4 inches = 240*12/200). Similarly, for the 279.5 ft (N-S) direction, the Staff's calculation showed that the maximum response displacement (deflection) is V = 3.9 inches; this, too, 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. *Id.* at 34-35.

5.148. The Staff further disputed the State's assertion that the Applicant's "assumption of rigidity" for the CTB leads to "overestimation of foundation damping." The soil will effectively "see" the CTB mat foundation as a rigid element, and the foundation damping assumption used by the Applicant is acceptable. *Id.* at 35.

5.149. Next, Mr. Pomerening disputed the State's assertions 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 soil impedance parameters and kinematic motion of the CTB foundation. In this regard, he stated that the soil-cement cap would provide restraint against lateral motion due to embedment of the CTB mat within the soil cement cap -- which 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 CTB motion 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 Applicant's omission of this factor in its calculation was conservative. *Id.* at 35.

5.150. Similarly, Mr. Pomerening disputed the State's assertion 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." In this regard, he explained that 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 is discussed above, with respect to the storage casks. As the CTB mat foundation contacts the soil-cement cap during seismic motion, it will tend to locally crush the soil-cement, because the soil-cement is softer than the mat foundation. Therefore, the soil/cement 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. *Id.* at 35-36.

5.151. Finally, Mr. Pomerening disputed the State's assertion in Subpart D.2.d. of the contention, that PFS'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. The bases for these conclusions are the same as those provided above concerning the casks and cask storage pads. First, PFS has provided a calculation by Geomatrix concerning the spatial and temporal variations of ground motions for the proposed PFSF site (although these calculations pertain specifically to the storage pads, the approach is also applicable to the CTB)). *Id*. at 37.

5.152. Mr. Pomerening explained that in its calculation, Geomatrix concluded that: (1) the angle of incidence of the seismic waves is such that the assumption of vertically propagating waves is reasonable for the site,[80] and (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. *Id.*

(b) NRC Staff's Confirmatory Analysis (Cask Tipover and Sliding)

5.153. As discussed above, Part D.1.i. of Unified Contention Utah L/QQ asserts that "the Applicant has failed to demonstrate the stability of the free standing casks under design basis ground motions," and that 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." The Staff evaluated those claims in its SER and in the testimony of Dr. Ofoegbu and Mr. Pomerening, as discussed above. In addition, in evaluating and responding to those claims, the Staff performed an independent confirmatory analysis to evaluate the potential for storage cask tipover or sliding in the event that a 2,000-year design basis earthquake, or a larger, 10,000-year earthquake, occurs at the PFSF site. This confirmatory analysis was addressed in the testimony of Staff witnesses Dr. Vincent K. Luk of Sandia National Laboratories and Mr. Jack Guttmann of the NRC Spent Fuel Project Office.

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[80] 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.

As discussed below, the Staff's confirmatory analysis provided persuasive evidence that the storage casks will not tipover or experience excessive sliding in the event that a design basis earthquake, or larger 10,000-year earthquake, occurs at the PFSF site. *See* Luk/ Guttmann, Post Tr. 6760.

5.154. In his capacity as Chief of an SFPO Technical Review Section, 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, to verify the conclusions in 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; and this analysis was conducted separately from the Staff's evaluation of the Applicant's calculations and analyses (which were evaluated in the SER and addressed in the testimony of Dr. Ofoegbu and Mr. Pomerening). Luk/Guttmann, Post Tr. 6760, at 1; *see* Consolidated SER, §5.1.4.4, at 5-28 to 5-32.

5.155. Dr. Vincent K. Luk served as Principal Investigator in this PFS site-specific confirmatory analysis. Among his other duties at SNL, 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. Luk/ Guttmann, Post Tr. 6760, at 1-2. That effort commenced as a generic study, and was then expanded to include several site-specific studies, including dry cask storage systems at the Hatch and San Onofre nuclear power plants and, more recently, the PFS ISFSI. *Id.* at 4; Tr. 6763-65, 7023-25.

5.156. Dr. Luk assembled and directed a team of experts in conducting an evaluation of the seismic behavior and stability of the freestanding, cylindrical HI-STORM 100 casks to be installed on concrete storage pads at the proposed PFSF. He 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, and to simulate the effects of soil-structure interaction, under prescribed seismic conditions. These efforts culminated in the publication of a final report on March 31, 2002, entitled "Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage (PFS) Facility," Rev. 1" (Staff Exh. P). Luk/ Guttmann, Post Tr. 6760, at 2.

5.157. Dr. Luk's testimony describes the results of this confirmatory site-specific 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). *Id.* at 4. As discussed in greater detail below, this confirmatory analysis demonstrated that the HI-STORM 100 storage casks at the PFS site 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 -- which exceed the maximum rotation and displacements that would occur with the smaller, 2,000-year design basis event. Accordingly, the Staff concluded 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. *Id.* at 9-13. For the reasons discussed herein, we share this conclusion.

5.158. Dr. Luk first became involved in NRC efforts to model storage cask behavior in a seismic event in an ongoing generic program for developing guidance on seismic hazards analysis, established by the NRC 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 Dr. Luk's leadership, as Principal Investigator. As part of this ongoing effort, the Staff requested technical assistance from 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. Luk/ Guttmann, Post Tr. 6760, at 4.

5.159. 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 PFSF, 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. *Id.*

5.160. 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 adjacent to the pad (2' 4" thick), 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 in Figure 1 of Staff Exh. P, at 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* Staff Exh. P, 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. Luk/ Guttmann, Post Tr. 6760, at 5. In other words, the cask and pad were modeled as elastic bodies with zero damping. *Id.* at 6.

5.161. 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 structural elements discussed above, 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. *Id.* at 5.

5.162. The Staff's 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. Luk/ Guttmann, Post Tr. 6760, at 6.

5.163. With respect to the first of these factors (seismic loading), 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. *Id.*

5.164. 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. *Id.* at 6-7.

5.165. 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. *Id.* at 7.

5.166. Coefficients of friction at the three interfaces were modeled as follows. 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 Staff Exh. P (Best Estimate, Model Type 1), at 30. Luk/Guttmann, Post Tr. 6760, at 7-8.

5.167. 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 Staff Exh. P (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 Staff Exh. P, at 30, for the 2,000-year event. *Id.* at 8.

5.168. 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 Staff Exh. P, at 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 Staff Exh. P. *Id.*

5.169. With respect to the use of soil profile data, 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 Staff Exh. P, at 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 Staff Exh. P, at 7-12. Luk/Guttmann, Post Tr. 6760, at 8-9.

5.170. 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 (Staff Exh. P, at 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 report (id., at 11-12); in contrast, for seismic events with a 2,000-year return period, the low strain shear modulus and damping were used. Luk/Guttmann, Post Tr. 6760, at 9.

5.171. The results from the Staff's 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. Luk/ Guttmann, Post Tr. 6760, at 9-10.[81]

5.172. The results predicted by the coupled model with respect to the maximum horizontal cask sliding displacements render it unlikely that collisions of adjacent casks would occur at the PFSF site. The separation distance between neighboring casks is 47.5 inches. Half of this distance, or 23.75 inches, is regarded as the cask collision criterion. Inasmuch as maximum displacement 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. *Id.* at 10.

5.173. Similarly, the model predicts that tipover of the HI-STORM 100 storage cask is unlikely to occur at the PFSF site during a seismic event. In this regard, with respect to the 2,000 year return period seismic event, the coupled model analysis predicts that the maximum cask

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[81] 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 Staff Exh. P (Staff Exh. P, at 30). Luk/Guttmann, Post Tr. 6760, at 10.

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. *Id.* at 11.

5.174. 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). *Id.* at 11.

5.175. 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. *Id.* at 11-12.

5.176. Based on the maximum cask rotation predicted by the model, Dr. Luk concluded that cask tipover is unlikely to occur during either the 2,000-year or 10,000-year return period seismic events at the PFSF site. The cask rotation that is associated with a cask tipover is approximately 29 degrees. A rotation of less than 29 degrees (as is predicted here) would be insufficient to result in tipover of a loaded HI-STORM 100 cask. *Id.* at 12.

5.177. A detailed evaluation of cask movement in the vertical direction was also conducted. This evaluation indicates that the cask does not experience much displacement in the vertical direction in any of the three seismic events. During either the 2,000-year return period seismic event or the 1971 San Fernando Earthquake (Pacoima Dam record), the cask base is never entirely lifted off the top surface of the pad, and the maximum vertical displacement at any location of the cask base is much less than 1 inch above the top surface of the pad. *Id.* at 12. During the 10,000-year return period seismic event, 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; and the analysis results for the 10,000-year event 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. *Id.* at 12-13.[82]

5.178. In sum, based on its confirmatory analysis, the Staff concluded that excessive cask sliding, cask collisions, and/or cask tipover will not occur during either a 2,000-year return period

[82] The Staff's analysis also demonstrated the importance of the dynamic coupling or soil-structure interaction ("SSI") effect of the cask with the soil foundation. *See* Staff Exh. P, at 27-29. The model analyses indicate the presence of a significant SSI effect. *See* Staff Exh. P, at 29, 34-35 (Figures 17-19). 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. *See id.* The SSI effect is further demonstrated by plotting the corresponding response spectra in Figures 20a through 22b. *Id.* at 13. In contrast, State witness Dr. Khan did not include consideration of the SSI effect in the Altran report, Tr. 7737 -- although both Drs. Bartlett and Ostadan recognized that soil-structure interaction effects are important. Tr. 7594, 7638.

or 10,000-year return period seismic event at the PFS site -- and Part D.1.i. of Unified Contention Utah L/QQ does not, therefore, present a valid concern. *Id.* at 13.

5.179. The Sandia analysis performed by Dr. Luk did not attempt to duplicate the Holtec analysis, nor did it do so. The Sandia analysis utilized a methodology (a state-of-the-art three-dimensional finite element analysis, Tr. 6934) that differed from the damper-and-spring model constructed by Drs. Singh and Soler; indeed, Dr. Luk performed his analysis without ever having seen the Holtec analysis prior to testifying in this proceeding. Tr. 6937-41. These independent analyses resulted in specific quantitative results that differed to some extent. However, while Dr. Luk's specific quantitative results varied somewhat from the results obtained by Drs. Singh and Soler, using a wholly independent approach and different methodology, his analysis clearly confirmed their conclusion that the HI-STORM 100 casks will not collide into each other or tipover in the event of either the design basis (2,000-year return period) earthquake or the 10,000-year return period seismic event. *See* Luk/Guttmann, Post Tr. 6760, at 13; Staff Exh. P, at 39-40.

Summary of Findings Regarding Part D of Unified Contention Utah L/QQ

5.180. We have carefully considered all of the evidence presented by the parties with respect to the numerous seismic design and foundation stability issues raised in Part D of this contention. Like the parties, we have considered the adequacy of the Applicant's analyses and calculations supporting the seismic design of the storage pads and Canister Transfer Building, and their foundations. We have also considered the Applicant's beyond-design-basis evaluations of cask stability, presented by Drs. Singh and Soler, as well as the Staff's independent confirmatory analysis of both the design basis and beyond-design-basis events, presented by Dr. Luk. We have reached the following factual conclusions regarding the matters raised in Part D of Unified Contention Utah L/QQ, in addition to the specific findings set forth above.

5.181. First, on the basis of all the evidence, we have concluded that the PFSF seismic design is adequate, and that the Facility will satisfy the Commission's requirements governing foundation stability and the seismic analysis and design for an ISFSI, set forth in 10 C.F.R. Part 72. Further, the Applicant has conducted adequate analyses of its seismic design and the performance of structures, systems, and components important to safety, considering the impact on public health and safety resulting from operation of the ISFSI and the consequences of accidents, including natural phenomena and events, as required in 10 C.F.R. § 72.24(d)(2). In addition, we are satisfied that SSCs important to safety have been designed to accommodate the effects of, and to be compatible with, site characteristics and environmental conditions so as to withstand postulated design basis seismic events, and to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform safety functions, as required in 10 C.F.R. § 72.122(b)(1) - (2).

5.182. More specifically, 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. 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) the potential for sliding of the pads under seismic loading is small, and would not constitute a safety hazard; and (3) settlement of the pads does not present a foundation stability concern.

5.183. 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 their capacity to perform their safety functions, in accordance with regulatory requirements.

5.184. We have weighed specific criticisms of the Applicant's calculations and analyses proffered by State witnesses Ostadan, Khan and Bartlett, and find them wanting, for the reasons discussed above. On the basis of the evidence, we find, *inter alia*, that (a) the assumption of vertically propagating in-phase waves is reasonable at the proposed PFS site; (b) the assumption of a rigid storage pad produces 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; (c) the Applicant's modeling of foundation soils under dynamic loading is consistent with the requirements of ASCE 4-98, and provides an accurate representation of the most significant frequency for pad response, thus accurately predicting their maximum displacement; (d) lateral variations in the phase of ground motion and their effect on the stability of the storage pads and casks will be insignificant; (e) the amount of deformation of the concrete pad is insignificant and will not result in cold-bonding of the casks and storage pads; (f) the potential for pad-to-pad interaction caused by sliding of the storage pads under seismic loading is not a concern, given the low magnitude of force that can be transmitted through the soil-cement layer between the storage pads and the likelihood of energy dissipation through the soil-cement; (g) the time histories used by PFS in its non-linear analysis are consistent with NRC guidance; (h) fault fling is not a potential issue for normal faults such as exist at the PFS site; (i) PFS has provided a realistic evaluation of the foundation-pad motion considering the cement-treated soil underlying the pad emplacement area; and (j) the PFS design provides a factor of safety against sliding of the pads of at least 1.1, consistent with the regulatory guidance in § 3.8.5 of NUREG-0800 -- but even if pad sliding occurs, that would result in greater cask stability, thus affording greater protection against cask tipover and sliding on the storage pads.

5.185. With respect to the Canister Transfer Building, we find that PFS has demonstrated the ability of the CTB, with its foundation, to perform its intended safety function in the event of a design basis earthquake. In this regard, in addition to our specific findings discussed above, (a) the proposed CTB foundation design satisfies regulatory requirements with respect to the capability of the underlying soil to provide adequate foundation support; (b) the CTB design will perform its intended safety functions under dynamic loading from the design basis ground motion; (c) the rigid mat foundation assumption used by PFS is conservative, as it results in an upper bound estimate of the response of the CTB; (d) the foundation damping assumption used by PFS is acceptable; (e) PFS's omission of the soil-cement around the CTB in calculating the soil impedance function was conservative; and (f) any cracking or separation of the soil-cement around the CTB will not adversely affect the ability of the structure to perform its safety function.

5.186. The Applicant's analyses satisfactorily demonstrate that the seismic design of the PFSF satisfies applicable NRC regulatory requirements and guidance. Further, the computer simulations presented by Drs. Singh and Soler demonstrate that the casks will not tip over or collide into each other in the event of a design basis earthquake at the PFSF. Moreover, their analyses demonstrate that the casks will not tipover or collide into each other with adverse consequences even in a 10,000-year return period beyond-design-basis event,[83] demonstrating that significant margins are incorporated in the PFS design for a 2,000-year return period earthquake.

5.187. Finally, the Staff's independent confirmatory analysis, performed by Dr. Luk of Sandia National Laboratories, provides firm support for the conclusion that the HI-STORM 100 storage casks will not tipover or collide under design basis ground motion conditions. That analysis examined the dynamic and nonlinear behavior of the HI-STORM 100 casks at the PFSF, including the soil-structure interaction effects during a seismic event, using three-dimensional finite element analysis methodology that is quite different from the damper-and-spring model constructed by Drs. Singh and Soler. While Dr. Luk's specific quantitative results varied somewhat from the results obtained by Drs. Singh and Soler, his analysis demonstrated that the casks would not

[83] Although their computer simulation depicts a case in which the tops of two casks collide in a 10,000-year beyond-design-basis event, that case involved unreasonable assumptions; and further, even in that event, no tipover or significant cask damage occurred. *See* PFS Exh. OO.

collide into each other or tipover in the event of either the design basis (2,000-year return period) earthquake, the 1971 San Fernando Earthquake, or the 10,000-year return period seismic event.

5.188. Having considered all of the evidence, we are satisfied that the concerns raised in Part D of this contention have been resolved and that the Applicant has demonstrated the adequacy of the seismic design and foundation stability of the proposed PFS Facility.

5.189. We turn now to consider the claims raised in Part E of this contention, concerning the Applicant's request that it be granted an exemption from the deterministic requirements in 10 C.F.R. § 72.102(f).

E. Unified Contention Utah L/QQ, Part E.

6.1. Part E of this contention incorporated, without modification, the State's challenge to the PFS seismic exemption request previously set forth in Part B of Contention Utah L. It asserts as follows:

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

Applicable Legal Standards

6.2. 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.

6.3. 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.

6.4. 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^{o} 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."

6.5. 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.

6.6. 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 on a seismogenic fault or within a seismic source at the closest possible distance to the site. Moreover, the DSHA methodology does not consider how frequently the seismic events occur, including the earthquake that is considered to control the deterministic ground motion. In addition, DSHA methods do not consider uncertainties associated with the identification and characterization of an earthquake at the site or uncertainties in ground motion modeling. Thus, analyses using the Part 100, Appendix A <u>deterministic</u> methodology would establish the SSE for a NPP without regard to the uncertainties associated with the evaluation of earthquakes (*e.g.*, size, location, magnitude) and with the assessment of ground motions, and do not consider the probability of occurrence of the SSE within any period of time.[84]

6.7. As discussed herein, PFS has requested an exemption from the deterministic seismic requirements in 10 C.F.R. Part 72. Where, as here, an exemption is sought from the requirements in 10 C.F.R. Part 72, the regulations provide that the Commission "may . . . grant such exemptions from the requirements of the regulations in this part as it determines are authorized by law and will not endanger life or property or the common defense and security and are otherwise in the public interest." 10 C.F.R. § 72.7.

Evidence Presented

6.8. Ten days of evidentiary hearings were held on Part E of this contention, on May 11, May 13, May 16-17, June 5-6, and June 24-27, 2002. Numerous witnesses appeared on behalf of the parties, as set forth below. All of the witnesses were found to be qualified to present testimony on the matters they addressed.

6.9. The evidence was presented in two parts: First, the parties presented evidence concerning subparts E.1 and E.3-E.6, in which they addressed the geotechnical bases and technical merits of the exemption request; second, they presented evidence concerning subpart E.2 of the contention, involving the projected radiological dose consequences of a seismic event in which the HI-STORM 100 storage casks are hypothetically assumed to tipover on the storage

[84] As Applicant witness Dr. C. Allin Cornell explained, deterministic methodology involves the selection of a design ground motion based on the magnitude of a seismic source and its distance from a site *(i.e.*, "the dominant event pair"), that has the potential to result in the greatest seismic hazard at the site. *See* Cornell Post Tr. 7856, at 8. The deterministic methodology is not time dependent -- *i.e.*, no consideration is given to the probability that the seismic event will occur during a particular time period. Staff Exh. C, at 2-49. In contrast, probabilistic seismic hazard analyses involve identification of all known seismic sources within the zone of interest, accounting for uncertainties and frequencies of occurrence, and consideration of the probability of exceeding a particular ground motion value within a specified return period, based upon the cumulative probabilities obtained for each seismic event. This results in the development of a single hazard curve which plots the probability of exceedance per year of various levels of ground motion. *Id.*; Cornell, Post Tr. 7856 at 8-9; Tr. 5814-18, 5824-32, 5837-38, 7868-69.

pads at the PFSF site. In the discussion which follows, we adopt, as logical, this bifurcation of the various subparts of the contention.

1. <u>Geotechnical Bases for the Exemption.</u>

<u>Applicant Witnesses</u>

6.10. The Applicant presented two witnesses with respect to Subparts E.1 and E.3 - E.6 of Unified Contention Utah L/QQ. These were as follows: Dr. C. Allin Cornell ("Testimony of C. Allin Cornell" (hereinafter referred to as "Cornell"), Post Tr. 7856); and Donald Wayne Lewis ("Testimony of Donald Wayne Lewis on Section E of Unified Contention Utah L/QQ" (hereinafter referred to as "Lewis"), Post Tr. 8968). In addition, rebuttal testimony was presented by Dr. Cornell ("Rebuttal Testimony of C. Allin Cornell to the Testimony of State Witness Dr. Walter J. Arabasz on Section E of Unified Contention Utah L/QQ" (hereinafter referred to as "Cornell Rebuttal"), Post Tr. 12951.

6.11. Dr. C. Allin Cornell is currently a research professor at Stanford University in Stanford, California, and an independent engineering consultant; previously, during the course of a distinguished career, he was also employed as a Professor at MIT. Dr. Cornell has a Ph.D. in Civil Engineering (Structures) from Stanford University. He has developed extensive professional expertise in earthquake engineering, probabilistic engineering analysis of seismic and other loads on structures, and structural responses to such loads. Due to Dr. Cornell's expertise in these areas, he has been actively involved in the development of structural design guidelines, codes and standards, including determining the appropriate level of earthquake design required to achieve a desired level of safety. Dr. Cornell has been involved in establishing earthquake standards of design for nuclear power plants, radiological waste facilities, offshore oil platforms, and buildings. Nuclear power plants and other nuclear facilities have been a major focus of Dr. Cornell's professional work on the development and application of methodologies and standards for evaluating earthquake hazards. His professional engagements in the area have included work for the NRC, DOE and a number of commercial operators of nuclear power plants, defense reactors, and high level radioactive waste storage facilities. Dr. Cornell has also been in the forefront of addressing, in probabilistic terms, the problems that arise at the interface between scientists who characterize natural hazards that threaten facilities and the engineers responsible for designing those facilities in a safe and cost-effective way. The majority of this work has been with earth scientists and structural engineers engaged in earthquake engineering. Cornell, Post Tr. 7856, at 1-6; Cornell Qualifications, at 1-2.

6.12. PFS witness Donald Wayne Lewis is employed by S&W as the Lead Mechanical Engineer for the PFSF project, a position he has held since 1996. He received his undergraduate engineering degree from the Montana State University, majoring in Civil/Structural Engineering. Mr. Lewis has 19 years of experience in the nuclear power industry, including 10 years of experience with the design, licensing, construction, and operation of independent spent fuel storage installations. He is a registered professional engineer in the states of New York, Colorado, Utah, Iowa, and Maine. Mr. Lewis' technical contribution to the PFS project focuses on the mechanical aspects of ISFSI operations, including cask handling and transportation equipment and operations, building services and utility systems, and fire protection. For the PFS project, he is also responsible for the preparation of the principal design criteria, design installation, and operating systems portions of the PFSF Safety Analysis Report. Lewis, Post Tr. 8968, at 1-2; Lewis Qualifications, at 1-2.

Staff Witnesses

6.13. The Staff presented three witnesses with respect to Subparts E.1 and E.3 - E.6 of this contention. These were Drs. John A. Stamatakos, Rui Chen and Martin W. McCann, Jr. ("NRC Staff Testimony of John A. Stamatakos, Rui Chen and Martin W. McCann, Jr., Concerning Unified Contention Utah L/QQ, Part E (Seismic Exemption)" (hereinafter referred to as "Stamatakos/

Chen/McCann"), Post Tr. 8050). In addition, Dr. Stamatakos presented rebuttal testimony in response to some of the testimony presented by State witness Dr. Walter J. Arabasz. ("NRC Staff Rebuttal Testimony of Dr. John A. Stamatakos Concerning Unified Contention Utah L/QQ, Part E (Seismic Exemption))" (hereinafter referred to as "Stamatakos Rebuttal"), Post Tr. 12648. [85]

6.14. Staff witness Dr. John A. Stamatakos is 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. He provided his testimony under a technical assistance contract between the NRC Staff and the CNWRA. Dr. Stamatakos has a Ph.D. degree in Geology from Lehigh University. He is a structural geologist and geophysicist with international research experience in regional and global tectonics. In his position as Principal Scientist at the CNWRA, he serves as the Principal Investigator for several projects involving the technical evaluation of structural deformation and seismicity, including tectonics and neotectonics research. His 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. Stamatakos/Chen/McCann, Post Tr. 8050, at 1-3; Stamatakos Qualifications, at 1.

6.15. Staff witness Dr. Martin W. McCann, Jr. is President of Jack R. Benjamin & Associates, Inc., in Menlo Park, California, and a Consulting Professor of Civil and Environmental Engineering at Stanford University. He has an M.S. degree in Structural Engineering and a Ph.D. degree in Civil Engineering from Stanford University. Among his duties at Jack R. Benjamin & Associates, Inc., he has served as a consultant to the CNWRA, in San Antonio, Texas; and he

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[85] In addition, on April 30, 2002, at the Board's request, Staff witnesses Drs. John A. Stamatakos and Martin W. McCann provided a general introductory "tutorial" discussion of probabilistic seismic hazard analysis methodology; how it differs from the deterministic approach set forth in 10 C.F.R. Part 100, Appendix A; and at what point the "hand-off" from the hazard analysts to facility design engineers occurs. *See* Tr. 5808-09, 5812-46.

provided his testimony under a technical assistance contract between the NRC Staff and the CNWRA. As President of Jack R. Benjamin & Associates, Inc., he has provided 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 his responsibilities, he provided technical assistance and consulting services to the CNWRA in its review of various PSHAs, including the PSHA for the DOE's proposed Yucca Mountain repository and other DOE nuclear facilities. Stamatakos/Chen/McCann, Post Tr. 8050, at 1-3; McCann Qualifications, at 1.

6.16. Staff witness Dr. Rui Chen is employed as an independent consultant in geological engineering and geosciences. Dr. Chen has a M.Sc. degree in Seismotectonics from the Graduate School at the University of Science and Technology of China and Institute of Geology, State Seismological Bureau of China, in Beijing, China; and a Ph.D. degree in Civil and Geological Engineering from the University of Manitoba in Winnipeg, Canada. From April 1995 to August 2000, she was employed as a Research Engineer and Senior Research Engineer at the CNWRA in San Antonio, Texas, where she 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. She provided her testimony under a technical assistance contract between the NRC Staff and the CNWRA. She has provided technical assistance and consulting services to the CNWRA 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. She has also taught graduate and undergraduate courses in geotechnical engineering and geosciences in the Department of Civil

Engineering and College of Natural Sciences at the California State University at Chico, California. Stamatakos/Chen/McCann, Post Tr. 8050, at 1-3; Chen Qualifications, at 1-2.

6.17. Drs. Stamatakos, Chen and McCann assisted the NRC Staff in its evaluation of the Applicant's request for an exemption from certain NRC regulations pertaining to seismic analyses and requirements related to the Applicant's construction and operation of the proposed PFSF, and participated in conducting the Staff's evaluation of the Applicant's PSHA. [86] In this regard, they co-authored a report 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 (Staff Exh. Q). Further, they assisted in preparation of the Staff's SER concerning the proposed PFS Facility (September 2000), and SER Supplement No. 2 (December 2001), which were incorporated into the Staff's Consolidated SER concerning the PFS Facility, issued in March 2002 (Staff Exh. C).[87] Stamatakos/ Chen/McCann, Post Tr. 8050, at 3-5.

State Witnesses

6.18. The State presented three witnesses with respect to Subparts E.1 and E.3 - E.6 of this contention. These were as follows: Barry Solomon ("State of Utah Testimony of Barry Solomon on Unified Contention Utah L/QQ - Geotechnical (Geologic Setting))" (hereinafter referred

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[86] While Drs. Stamatakos, Chen and McCann participated in various aspects of the Staff's evaluation of the PFS seismic exemption request, they described their respective areas of emphasis as being in the evaluation of seismic ground motions and faulting hazards, seismic hazard analyses and seismic design, and probabilistic seismic hazard analysis. Stamatakos/ Chen/McCann, Post Tr. 8050, at 3-5.

[87] In addition, Dr. Chen 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. The TMI-2 spent fuel debris ISFSI is referred to in Unified Contention Utah L/QQ, and was addressed in some of the testimony in this proceeding. In this regard, Dr. Chen co-authored a 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). *See* Stamatakos/ Chen/McCann, Post Tr. 8050, at 5. That report is discussed in the Staff's TMI-2 ISFSI Safety Evaluation Report (Staff Exh. SS), at 2-18.

to as “Solomon”), Post Tr. 8965); Dr. Steven F. Bartlett (“State of Utah Testimony of Dr. Steven Bartlett on Unified Contention Utah L/QQ, Part E (Lack of Design Conservatism)”) (hereinafter referred to as “Bartlett”), Post Tr. 12776);[88] and Dr. Walter J. Arabasz (“State of Utah Testimony of Dr. Walter J. Arabasz Regarding Unified Contention Utah L/QQ (Seismic Exemption))” (hereinafter referred to as “Arabasz”), Post Tr. 9098. In addition, oral rebuttal and/or surrebuttal testimony was presented by Drs. Arabasz and Bartlett.

6.19. State witness Barry Solomon has an M.S. degree in Geology from San Jose State University, and is employed as a Senior Geologist by the Utah Geological Survey (“UGS”). Among his responsibilities, he is a principal investigator for the National Earthquake Hazards Reduction Program grants to study active faults and seismic hazards in northern Utah, in which he has evaluated the potential for geologic hazards caused by an earthquake scenario on the Wasatch Fault Zone. He has 27 years of experience in developing, implementing, and managing geologic studies; and has studied geologic hazards and geologically characterized, screened and selected sites for hazardous, nuclear waste, construction, and mining projects. Solomon, Post Tr. 8965, at 1-2; Solomon Qualifications (State Exh. 91), at 1-2.

6.20. State witness Dr. Walter J. Arabasz is a Research Professor of Geology and Geophysics, and Director of the University of Utah Seismograph Stations, in Salt Lake City, Utah. He has an M.S. degree in Geology and a Ph.D. degree in Geology and Geophysics, from the California Institute of Technology. He is also a member and past Chair of the Utah Seismic Safety Commission. Dr. Arabasz has more than 30 years of professional experience in research, project management, consulting, and occasional teaching in observational seismology, tectonics, and earthquake hazard evaluation. He is the author or co-author of 37 published papers, 77 published

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[88] Dr. Bartlett’s written testimony on Part E of the contention was originally submitted as joint testimony with Dr. Farhang Ostadan. Dr. Ostadan subsequently became unavailable to present that testimony; his name and certain portions of the testimony were then deleted, so that the remaining testimony was presented by Dr. Bartlett alone. *See* Tr. 12771-73.

abstracts and numerous technical reports. His present responsibilities at the University of Utah include seismological research and extensive project management, chiefly relating to the operation and modernization of a 160-station regional/ urban seismic network covering Utah and neighboring parts of the Intermountain area. He has been affiliated, in various capacities, with the U.S. National Earthquake Hazards Reduction Program, and has served on national and state advisory and policy-making committees of the National Research Council, the Seismological Society of America, and the Council of the National Seismic System (a senior advisory group to the U.S. Geological Survey, concerning seismic monitoring). He has also served as a consultant on earthquake hazard evaluations for dams, nuclear facilities, and other critical structures and facilities in the private and governmental sectors. Arabasz, Post Tr. 9098, at 1; Arabasz Qualifications (State Exh. 123) at 1.

6.21. As noted above, Dr. Steven F. Bartlett also provided direct and rebuttal testimony on certain issues raised in Part E of Unified Contention Utah L/QQ, focusing on the issue of conservatisms (or the lack thereof) in the PFS seismic design. His professional qualifications are discussed above, with respect to his testimony on Parts C and D of the contention.

State of Utah's Testimony

6.22. State of Utah witness Barry Solomon provided an uncontested overview of the geologic setting of Skull Valley, Utah. Solomon, Post Tr. 8965, at 2-5.[89] In brief, his testimony explained that the proposed PFS site is located in the center of Skull Valley. For purposes of geological and geotechnical interpretation, Skull Valley lies within three regional zones: the Basin

[89] Mr. Solomon's corrected testimony was admitted by stipulation of the parties, without cross-examination. *See* Tr. 8961-64. To be sure, much of Dr. Solomon's testimony describing the geologic setting of the proposed PFS site duplicates portions of the Staff SER's description of the site's geologic setting, set forth above in section II.A of these proposed findings of fact. A more complete description of the geologic setting of Skull Valley and the surrounding area may be found in Chapter 2 in the Staff's SER (Staff Exh. C) and in the 1999 CNWRA report (Staff Exh. Q). *See* Staff Exh. C, at 2-24 to 2-33; Staff Exh. Q, at 2-1 to 2-27.

and Range physiographic province; the Intermountain seismic belt; and the Bonneville Lake basin. *Id.* at 2-3.

6.23. The Basin & Range physiographic province extends east-west from the Wasatch Range in central Utah to the Sierra Nevada, and north-south from southern Oregon and Idaho to northern Mexico. The northern part of this province, including Skull Valley, is characterized by asymmetrical mountain ranges separated by intervening valleys, both with north-south axes; this extension has range-bounding faults with significant Quaternary displacement, commonly with active faults scarps on adjacent piedmont slopes. The Stansbury fault, located on the east side of Skull Valley, is an active fault of this nature. Another active fault of this nature is the Wasatch fault zone, located about 50 miles east of the proposed site; it comprises one of the longest (230 miles) and most active normal-slip faults in the world, and is capable of generating earthquakes as large as surface magnitude 7.5 (M7.5). *Id.* at 3.

6.24. The Stansbury fault and Skull Valley are located along the western edge of the Intermountain Seismic Belt ("ISB"). The ISB is a prominent north-tending zone of mostly shallow earthquakes about 60-120 miles wide, that extends at least 900 miles from southern Nevada/ northern Arizona to northwestern Montana. Since 1900, 49 moderate to large earthquakes (M5.5 to 7.5) have been generated in the ISB, including the largest historic earthquake in Utah (the 1934 Hansel Valley M6.6 earthquake, located at the northern end of the Great Salt Lake), and the largest recorded earthquake in the ISB (the 1959 Hebgen Lake M7.5 earthquake near Yellowstone National Park, in Montana). There is a lack of correlation in the ISB between scattered background seismicity and mapped Cenozoic faults. The upper bound of background seismicity appears to be in the range of M6.0 to 6.5, representing the threshold of surface faulting; earthquakes up to this size can occur anywhere in the ISB, including Skull Valley, even where no geologic evidence exists for Quaternary surface faults. *Id.* at 3-4.

6.25. Skull Valley is also a geomorphic subbasin of the Bonneville Lake basin, which primarily occupies northwestern Utah. The Bonneville Lake basin consists of a number of topographically closed structural basins that were hydrologically connected during major lacustral episodes. Lake Bonneville was the most recent major lake to form in this basin; it persisted from about 30,000 to 10,000 radiocarbon years ago (essentially coincident with the last major ice age). The Great Salt Lake and the connecting Utah Lake are remnants of Lake Bonneville. Late Pleistocene deposits of Lake Bonneville are a significant component of foundation soils at the proposed PFS facility site. The lake level varied throughout its existence, due to climatic and other factors, and important information for estimating the age of latest Quaternary fault movements is provided by the variations in lake level and shorelines resulting from major periods of persistent lake levels. Two such shorelines, the Stansbury and Provo, are present within the proposed PFS site area. Another datum useful for estimating the age of late Pleistocene fault movement is the Promontory soil which formed on alluvium and eolian deposits prior to the start of the Bonneville lake cycle. *Id.* at 4-5.

6.26. Mr. Solomon also described capable faults found in the area of the proposed PFS site. These include the Stansbury Fault, located about 6 miles east of the site at the base of the Stansbury Mountains; the East Cedar Mountain fault, located about 10 miles west of the site, at the base of the Cedar Mountains. In addition, two mid-valley faults were identified by PFS contractor Geomatrix in a 1998 geologic investigation, and a zone of fault offset between the two: These are the "East" fault, about 0.5 miles east of the proposed site, and the "West" fault, about 1.2 miles west of the site. *Id.* at 5. In addition, the "Springline" fault lies in the northern part of Skull Valley. As indicated by Mr. Solomon, and in the testimony of the PFS and Staff witnesses, the Applicant's PSHA identified the Stansbury and East faults as the dominant seismic sources with respect to seismic hazard at the proposed PFS site. *Id.* at 5-6; Staff Exh. C at 2-31; *see also* Staff Exhibit Q, at 2-5; Staff Exh. C at 2-28 to 2-33 and 2-35 to 2-37.

6.27. Moving on to the issues in dispute, the State presented the testimony of Dr. Walter J. Arabasz concerning numerous issues raised in Part E of the contention.[90] In his testimony, Dr. Arabasz largely criticized the Staff's stated bases and rationale for finding the Applicant's seismic exemption request to be acceptable. His testimony mirrored and amplified Part E of this contention and the sworn declaration which he had filed concerning this contention at the summary disposition stage of this proceeding. *See* Tr. 9832-37, 9848-49, 9850-52; Staff Exh. MM. Thus, unlike other testimony that is typically presented in NRC adjudicatory proceedings, the focus of Dr. Arabasz' testimony was not the adequacy of the Applicant's proposal but, rather, the adequacy of the Staff's bases and rationale for finding the exemption request to be acceptable. *See* Tr. 9850-52; Tr. 9333-34 (Farrar, J.).[91]

6.28. Dr. Arabasz cites the Commission's proposed Rulemaking Plan in SECY-98-126 as providing guidance in the selection of a design basis earthquake for the PFS site, when using a probabilistic approach. *See* Arabasz, Post Tr. 9098, at 5. That initial rulemaking plan (set forth in Staff Exh. T) would allow the use of probabilistic methodology in lieu of the deterministic analysis

[90] Dr. Arabasz addressed each portion of Part E of this contention, other than the issues of (1) whether PFS has demonstrated adequate conservatism in the design of its SSCs (with respect to which he deferred to other State testimony including that of Dr. Bartlett on Parts C, D and E of the contention (Dynamic Analysis, Cask Stability and Lack of Design Conservatism)); and (2) the radiological consequences of a seismic event (contention subparts E.1 and E.2), to which he deferred to the testimony of State witness Dr. Resnikoff. Arabasz, Post Tr. 9098, at 5, 6; Tr. 9852.

[91] We accepted this approach in line with the Commission's determination that this part of the contention should be admitted. Thus, in *Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), CLI-01-12, 53 NRC 459 (2001), the Commission observed that "[a]t bottom, what Utah proposes to litigate is whether PFS's ISFSI design, which is dependent on an exemption from otherwise controlling seismic regulations, is adequate to withstand plausible earthquake risks," *Id*. at 465-66; and the Commission agreed with the Licensing Board's finding that "the substance of Utah's complaints was that the 2,000-year return period has not been shown to be adequately protective." *Id*. at 473. The Commission concluded that because "PFS essentially adopted the Staff's reasoning when it agreed to use the 2,000-year return period the Staff recommended," it is "appropriate under these circumstances to consider the Staff's bases for granting the exemption." *Id.*

required in 10 C.F.R. Part 100, Appendix A; further, it would establish a two-tiered approach, whereby structures, systems and components important to safety ("SSCs") would be designed to a 1,000-year return period ground motion if their failure would not result in radiological exposures greater than the 25 mrem normal dose limit set forth in 10 C.F.R. § 72.104(a); however, where the failure of an SSC would result in dose consequences which exceed that standard, they would have to be designed to a 10,000-year return period ground motion. *See* Staff Exh. T; Arabasz, Post Tr. 9098, at 5. Dr. Arabasz did not, however, take issue with the use of a probabilistic approach in establishing the seismic design basis for an ISFSI. [92]

6.29. Dr. Arabasz recognized that the initial rulemaking plan in SECY-98-126 has been superseded by the modified rulemaking plan set forth in SECY-01-0178 (*see* Staff Exh. U), in which a single-tier approach (*i.e.*, a single design basis ground motion) is followed. However, he observed that while the Staff had proposed a 2,000-year ground motion in the modified rulemaking plan, the Commission instructed the Staff to solicit comments on a range of probability of exceedance levels from 5.0E-4 (2,000-year return period) through 1.0E-04 (10,000-year return period), and it directed Staff to undertake further analysis to support a specific proposal. Arabasz, Post Tr. 9098, at 5-6.[93] Dr. Arabasz asserts that the validity of PFS's claim that it has met the

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[92] *See, e.g.,* Cornell Post Tr. 7856, at 31-32.

[93] Following the close of evidentiary hearings in this proceeding, the Commission published a proposed rule in the *Federal Register*, which follows the modified rulemaking plan set forth in SECY-01-178 and the Commission's related Staff Requirements Memorandum ("SRM") (Staff Exh. U). *See* Proposed Rule, "Geological and Seismological Characteristics for Siting and Design of Dry Cask Independent Spent Fuel Storage Installations and Monitored Retrievable Storage Installations," 67 Fed. Reg. 47745 (July 22, 2002). The proposed rule would allow an ISFSI applicant to utilize probabilistic methodology in establishing its seismic design basis, with a 2,000-year return period ground motion; however, the Commission requested comments on the return period, and that issue remains under study pending publication of the final rule. *See* 67 Fed. Reg. at 47751-52. While the Commission may ultimately publish a final rule that resolves this matter on a generic basis, the pendency of the generic rulemaking proceeding does not bar our resolution of this issue on a case-specific basis in this proceeding. *See generally*, Staff Exh. U, at 1-2.

Commission's requirement to show that the 2000-year design standard is sufficiently protective of public safety and property depends on both the probability of occurrence of the seismic event and the level of conservatism incorporated into design of SSCs. While Dr. Arabasz does not challenge the Applicant's PSHA -- and he defers to other State witnesses on the question of whether the design is sufficiently conservative -- he challenges the Staff's alleged failure to link (at least explicitly) the issues of probability and design conservatisms. *Id.* at 6.

6.30. With respect to Subpart E.3 of the contention, Dr. Arabasz asserts that the Staff misunderstood and misapplied Regulatory Guide 1.165, "Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion (*see* Staff Exh. UU), insofar as that guidance document discusses the use of median and mean probabilities of exceeding the safe shutdown earthquake ("SSE") for a nuclear power plant. In this regard, he contends that the Staff erred in its use of a mean annual probability of exceedance of 1×10^{-4} to establish the SSE for nuclear power plants. Arabasz, Post Tr. 9098, at 7-8.

6.31. With respect to Subpart E.4 of the contention, Dr. Arabasz observed that the Staff's determination to approve the PFS exemption request, described in the Staff's SER (Staff Exh. C, at 2-51), cites a U.S. Depart of Energy Standard, DOE-STD-1020-94, "Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities" (April 1994, as revised January 1996) (PFS Exh. DDD). He further observed that DOE-STD-1020-94 establishes a 2,000-year return period design basis ground motion for DOE Performance Category 3 ("PC-3") facilities, *i.e.,* facilities equivalent to the proposed PFS ISFSI. However, he asserted that the Staff's reliance on the DOE standard in determining to approve the PFS exemption request was flawed in that (a) the 2,000-year return period design basis stated in this standard was not relied upon in SECY-98-126, where the Staff followed a 1,000-year/10,000-year two tier approach, and (b) the Staff did not properly follow the DOE-STD-1020-94 approach, in that it did not couple the design standard to a target seismic performance goal of 1×10^{-4} as was done in the DOE document.

Further, according to Dr. Arabasz, a determination of the mean annual exceedance probability of a DBE for the proposed PFS facility, and whether it ensures sufficient protection, cannot be made independent of an evaluation of the facility's level of design conservatism. Arabasz, Post Tr. 9098, at 9-10.

6.32. With respect to Subpart E.5 of the contention, Dr. Arabasz opined that the Staff's SER reference to the Commission's prior grant of a seismic exemption to INEEL for the TMI-2 ISFSI is misplaced. He believes that the circumstances applicable to the INEEL site do not present a good precedent for the PFS exemption, in that (a) the design basis ground motion for the TMI-2 ISFSI was based on the design basis for another, higher risk facility at the TMI-2 ISFSI site; (b) the Staff has not performed a site-specific radiological risk analysis here, unlike the TMI-2 ISFSI example; (c) he understands the design ground motion value at the INEEL site was 0.36g, which is higher than the TMI-2 ISFSI's 2000-year return period mean ground motion (0.30g) and would equate to a 3,000 to 4,000-year ground motion there; (d) he views the TMI-2 example to have been disavowed by SECY-98-126, issued two months after the INEEL exemption was granted, in that it provided a 1,000-year/10,000-year two tier approach rather than following the approach used for the TMI-2 ISFSI. Arabasz, Post Tr. 9098, at 11-12.

6.33. Finally, with respect to Subpart E.6 of the contention, Dr. Arabasz criticized the Staff SER's reference to the PSHAs conducted for new Interstate-15 highway bridges in Utah (which use lower design basis ground motions), as erroneous and irrelevant. *Id.* at 13. In addition, he criticized an analogy used by the Staff in its December 1999 Preliminary SER -- but not in the final SER -- in which the Staff compared a 2,000-year return period seismic design for an ISFSI (99% probability of not being exceeded during a 20-year license term), to other facilities based on their operating life. He argued that a comparison of the PFS ISFSI (which is expected to seek license renewal and to have a 40-year operational life) compared to the pre-closure facility at Yucca Mountain with a 100-year operating period), would require establishment of a DBE with a mean return period of 3,980 years. *Id.* at 13-14; *see* Tr. 9988-89.[94] Lastly, he criticized Applicant witness Dr. Cornell's discussion of risk considerations, and Dr. Cornell's comparison of a risk-acceptance decision specific to the PFS site to a societal global safety strategy for the storage of spent fuel at Yucca Mountain. *Id.* at 14-16.[95]

6.34. State witness Dr. Steven Bartlett testified concerning the conservatism (or lack of conservatism) in the PFSF design. Dr. Bartlett asserted that the use of a 2,000-year design earthquake is inconsistent with other standards. In this regard, he pointed to a 2001 draft version of the U.S. Department of Energy's DOE-STD-1020-94, in which DOE proposed to revise its 2,000-year design basis for PC-3 facilities to a 2,500-year return period ground motion. Bartlett,

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[94] In its Preliminary SER of December 1999, the Staff compared a 2,000-year return period for an ISFSI with a 20-year license (99% probability of not being exceeded in 20 years) to the International Building Code ("IBC") and National Earthquake Hazards Reduction Program ("NEHRP") standards (90% probability of not being exceeded in 50 years). *See* Staff Exh. A, at 2-45; Arabasz, Post Tr. 9098, at 13-14. Similarly, as Dr. Arabasz noted, the modified rulemaking plan in SECY-01-178 compares an ISFSI with a 20-year "operational period" (20 years x 5.0E-04 = 1.0E-02) with the Yucca Mountain pre-closure facility with a 100-year operational period (100 years x 1.0E-04 = 1.0E-02). *Id.* at 14; Tr. 9989. Despite Dr. Arabasz' criticism of these comparisons, however, they do not appear in the Staff's final SER or its bases for approving the PFS exemption request -- as Dr. Arabasz has recognized. See Staff Exh. C, at 2-50 to 2-51; Tr. 9989. Accordingly, we need not reach a conclusion as to whether such comparisons are appropriately relied upon by the Staff outside of this proceeding.

[95] Dr. Arabasz also complained that PFS presented a "moving target" as to the appropriate seismic design basis ground motion. He noted that the Applicant's original SAR, submitted in 1997, used a deterministic approach; that PFS then proposed a probabilistic approach in its April 1999 seismic exemption request, based on a PSHA and a 1,000-year mean return period; that PFS then proposed a 2,000-year ground motion following the Staff's suggestion; that PFS revised its DBE from a peak ground acceleration of 0.53 g to 0.711g (horizontal), in March 2001; and that PFS submitted a new rationale in support of its proposed 2,000-year return period, in November 2001. *See* Arabasz Post Tr. 9098, at 3; Tr. 9158-60, 9183. While Dr. Arabasz may be correct in his assessment that the Applicant's proposal has changed with time, we see no flaw in this; rather, it reflects the Applicant's due regard for new information, and a sharpening of its analyses and thought processes. The same is true for the Staff's revision of its thought processes concerning the PFS seismic exemption request, after issuing its Preliminary SER in December 1999.

Post Tr. 12776, at 4.[96] Similarly, he pointed to seismic hazard maps published by U.S. Geological Survey and the National Earthquake Hazard Reduction Program, which utilize a 2,500-year ground motion; and the use of a 2,500-year ground motion by the most recent International Building Code; and he testified that Utah interstate highway bridges must be designed to levels of strong ground motion equivalent to an average return period of 2,500 years. *Id.*

6.35. Dr. Bartlett also testified about the factors which he believes should affect the selection of a design earthquake ("DE"). In this regard, he stated that the DE and the seismic performance of SSCs are inextricably linked; thus, any claims of conservatism are meaningless if a DE is selected without adequately determining the site specific seismic performance of SSCs and risk reduction ratios. *Id.* at 6. He further asserted that in determining the appropriate DE for the PFSF, consideration of the seismic performance of the SSCs at PFS is even more critical because the PFS site is located in a highly seismic area with high 2,000-year DE ground accelerations; that the use of a 2,000-year DE significantly reduces the design standard from the deterministic design basis earthquake standard; and that there is no precedent for PFS's proposed design (addressed in Part D of this contention), so that PFS cannot rely on any direct experience to support its proposed DE. *Id.*

6.36. Dr. Bartlett argued that, if followed in its entirety, DOE Standard 1020 would provide an acceptable methodology for selection of a DE, where (a) documented and peer-reviewed

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[96] The 2001 draft version of the DOE standard would be later be published, with certain important changes, as DOE-STD-1020-2002. Bartlett, Post Tr. 12776, at 5; *see* Staff Exh. II, Staff Exh. QQ. Dr. Bartlett had not seen the final document when he filed his testimony asserting that DOE's adoption of a 2,500-year return period was more conservative than its prior use of a 2,000-year return period for PC-3 facilities. Tr. 12883-85. Significantly, one of the changes in the 2002 final document, as compared to the 2001 draft, was that DOE reduced the seismic scale factor ("SF") for PC-3 facilities, from 1.0 to 0.9, with the result that the design standard for PC-3 facilities effectively did not change from the 1994 version of the document in which DOE had used a 2,000-year return period for PC-3 facilities. *See* Staff Exh. QQ at C-10 to C-11; Staff Exh. II at iv; Tr. 9305-06, 12895. Dr. Bartlett was not aware of this modification of the SF factor when he filed his testimony asserting that DOE had adopted a more conservative design basis standard in its 2001 draft document. *See* 12895-96.

performance goals and the requisite conservatism (risk reduction ratios) are established based on the classification hazard of the SSCs; (b) unacceptable performance is defined as damage to the SCC, beyond which hazardous material confinement and safety-related functions are impaired; and (c) fragility curves are developed for the SSCs, which show the expected damage or unacceptable performance of an SSC as a function of the amplitude of strong ground motion, to determine whether the performance goal is met for the DE. *Id.* at 8-9.

6.37. Dr. Bartlett then presented his view that PFS has failed to demonstrate that the seismic performance of its SSCs is adequate to accommodate a 2,000 year DE. In this regard, he asserted that PFS has not generated any fragility curves or other mechanism to demonstrate the seismic performance of its SSCs; and that its seismic analysis is fraught with errors, omissions, and a lack of conservative assumptions -- with the result, in his view, that PFS failed to show that adequate conservatism has been applied in its design of foundations for the storage pads and CTB and the seismic stability of the pads and casks for the proposed DE. In addition, he reiterated the State's views, presented with respect to Part D of this contention, that PFS's cask stability analysis is inadequate and the casks may tipover in the event of a DE at the PFS site. *Id.* at 9-12.

6.38. Finally, Dr. Bartlett presented his view -- contrary to the opinion of PFS witness Dr. Cornell (Cornell, Post Tr. 7856, at 20-25; *id.*, Attachment at 1) -- that performance goals are not clearly "inherent" in the NRC Standard Review Plans ("SRPs") for ISFSIs and dry cask storage systems.[97] In this regard, he observed that the PFS storage casks are designed to NUREG-1536, "Standard Review Plan for Dry Cask Storage Systems" (*see* Staff Exh. 58), while the CTB "must be" designed to NUREG-1567, "Standard Review Plan for Dry Cask Storage Systems," rather than

[97] Dr. Bartlett's view in this regard is contrary to the detailed testimony presented by PFS witness Dr. Cornell, concerning conservatisms inherent in NRC design standards and guidance documents. *See, e.g.,* Cornell, Post Tr. 7856, at 20-31.

guidance applicable to nuclear power plants ("NPPs"). Bartlett, Post Tr. 12776, at 13.[98] He further asserted that the standards in NUREG-1536 and NUREG-1567 "may be" lower than the standards for NPPs (*Id.*) -- a view which he later effectively retracted.[99]

Applicant's Testimony

6.39. Applicant witness Dr. C. Allin Cornell has developed, over the course of a highly distinguished career (including participation in numerous NRC, DOE, and other organizations' advisory or standard-setting committees) , an extensive and deep familiarity with NRC and industry seismic design standards, and probabilistic methodology. *See* Cornell, Post Tr. 7856, at 1-6, 7-11. His testimony provided a thorough analysis and explanation of the conservatisms inherent in the PFS seismic design and the adequacy thereof, based on the use of a probabilistic seismic hazard analysis (PSHA) with a 2,000-year return period; and discussed the seismic related design procedures and criteria contained in NRC guidance documents, such as the Standard Review Plans applicable to NRC-licensed ISFSIs, which were used in establishing the PFSF seismic design. *See id.,* at 8-31, and Attachment at 1. Finally, he provided a thoughtful and persuasive

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[98] While Dr. Bartlett described the Staff's Standard Review Plans as establishing regulatory requirements (Bartlett Post Tr. 12776, at 13), they are, in fact, regulatory guidance documents -- with which strict compliance is not required, where an applicant can show alternative means of meeting the requirements prescribed by regulation. *See, e.g., Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), CLI-01-22, 54 NRC 255, 264 (2001) ("NUREGs, such as the Standard Review Plan, like all guidance documents, are not legally binding regulations.") (citations omitted).

[99] Dr. Bartlett based his view that the regulatory guidance documents for ISFSIs establish lower standards than the regulatory guidance applicable to nuclear power plants based on the Applicant's and Staff's stated views that the consequences of seismic failure at an ISFSI are less severe than the consequences of seismic failure at NPPs, and that the PFS ISFSI design basis DE ground motion may be lower than that used for a NPP. *See, e.g.,* Tr. 12820-21. Significantly, on cross-examination, Dr. Bartlett indicated he had no basis to state that the regulatory guidance for ISFSIs resulted in any lower standard than applied to nuclear power plants -- and he recognized that the ISFSI guidance documents often cited and relied on guidance for NPPs found in NUREG-0800 and other regulatory guidance documents for NPPs. *See* Tr. 12896-905, 12920-22, 12932-42. Accordingly, we find no basis for his attack on NUREG-1536, NUREG-1567, or any other NRC regulatory guidance document applicable to an ISFSI.

response to the specific assertions raised by the State in Subparts E.1 and E.3 - E.6 of this contention. *See id.,* at 31-53.

6.40. More specifically, Dr. Cornell testified that the proposed use by PFS of a PSHA, both to characterize the seismic hazard at the site and to set the seismic design basis of the PFSF, is fully consistent with NRC policy and practices as well as broader engineering policy and practice. *Id.* at 9-10. In this regard, current NRC policy with respect to the licensing of nuclear power plants permits the use of PSHA methodology in establishing the seismic design basis, as set forth in 10 C.F.R. § 100.23. *See id.* at 10.[100]

6.41. Dr. Cornell explained that there are two general principles of risk informed seismic design. The first such general principle is that there should be a risk-graded approach to seismic safety which allows facilities and structures with lesser failure consequences to have larger mean annual probabilities of failure. A second general principle is that the adequacy of a design basis earthquake (DBE) to provide the desired level of seismic safety is to be judged by considering both the mean annual probability of exceedance ("MAPE") of the DBE and the level of conservatism incorporated into the design criteria and procedures. *Id.* at 11.

6.42. With respect to the first general principle, Dr. Cornell testified that because the Commission has determined that ISFSIs pose less risk than nuclear power plants, it is appropriate for ISFSIs, such as the PFSF, to have a higher MAPE than nuclear power plants, due to their relatively lower radiological consequences in the event of a seismic failure. *Id.* at 12-13.

6.43. With respect to the second general principle enunciated in ¶ 6.41above, Dr. Cornell testified that in determining the MAPE, it is appropriate to consider the high levels of margin of safety embodied in nuclear safety acceptance criteria and design procedures. *Id.* at 13-15. These

[100] The Commission's acceptance of the use of PSHA methodology is further reflected in its recent publication of a proposed rule which would similarly allow the use of PSHA methodology in establishing the design basis for an ISFSI.

are reflected in DOE criteria such as DOE-STD-1020-94 and STD-1002-2002, as well as in NRC Standard Review Plans, including NUREG-0800, NUREG-1536, and NUREG-1567. *Id.* at 15-25.[101]

6.44. Large safety margins on the order of magnitude of a factor of five (more typically, factors of five to twenty) exist and have been demonstrated for important to safety systems, structures and components at the PFSF. Based on a number of pertinent considerations,[102] Dr. Cornell concluded that a risk reduction factor of five is appropriate for important-to-safety SSCs at the PFS Facility, including the CTB, the cranes and seismic struts inside the CTB, the HI-STORM 100 storage casks, and the concrete storage pads. *Id.* at 25-28.

6.45. Because of the large margins inherent in the standards and guidance used in the PFSF design, Dr. Cornell concluded that the PFSF design could withstand an earthquake well in excess of the 2000 year DBE. Indeed, inasmuch as cask tipover is not likely to occur in a 10,000-year return period earthquake, the probability of tipover can be described as no greater than 10^{-4} -- and further, even if cask tipover occurs, a release of radioactivity is unlikely. Thus, the annual probability of a release of radioactivity from a storage cask due to a 10,000-year return period seismic event is clearly less than 10^{-4}. *Id.* at 28-29, 39. Accordingly, for a DBE with a MAPE of

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[101] *See* (1) NUREG-0800, "Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants" (August 1988); (2) NUREG-1536, "Standard Review Plan for Spent Fuel Dry Storage Facilities" (March 2000); and (3) NUREG-1567, "Standard Review Plan for Dry Cask Storage Systems" (January 1997)." As Dr. Cornell noted, NUREG-1536 and NUREG-1567 are applicable to the PFS Facility and the HI-STORM 100 storage casks it proposes to use. These SRPs, in turn, refer to NUREG-0800 and/or many of the same codes and standards that are cited therein. Cornell, Post Tr. 7856, at 22-23.

[102] In reaching this conclusion, Dr. Cornell considered (1) risk reduction factors applied to different types of structures designed to a wide variety of codes and standards; (2) risk reduction factors applicable to nuclear power plants under NUREG-0800; (3) the SRPs applicable to an ISFSI such as the PFSF (NUREG-1567 and NUREG-1536); (4) confirmation by persons responsible for design of the PFS SSCs, that they were generally designed to the same codes and standards as apply to NPPs; (5) a demonstration by persons responsible for the design of the PFSF of significant beyond-design-basis margins for important to safety SSCs; (6) the limited fraction of time that certain SSCs would be in use at the PFSF; (7) a demonstration by Holtec that the casks will not tipover in a 10,000-year earthquake; and (8) a demonstration by Holtec that a postulated cask tipover would not result in a release of radioactivity. Cornell, Post Tr. 7856, at 25-26.

5×10^{-4}, the implied risk reduction factor for the storage casks is greater than a factor of 5. Using the parlance of DOE-STD-1020, this results in meeting a performance goal of 1×10^{-4}. *Id.* at 28-29. Moreover, use of a performance goal of 1×10^{-4} is appropriate for the SSCs at the PFSF, and is consistent with DOE-STD-1020, which similarly sets a performance goal of 1×10^{-4} for PC-3 SSCs (which would embrace the PFSF important-to-safety SSCs). *Id.* at 29-31.

6.46. In sum, Dr. Cornell concluded that a performance goal of 1×10^{-4} for the PFSF would be consistent with a risk-graded approach to seismic safety; and that a DBE at the PFSF based on a 2000-year return period ground motion provides adequate protection of public health and safety. *Id.* at 29-31.

6.47. Dr. Cornell also responded to claims raised by the State with respect to Part E of this contention. He testified that DOE-STD-1020 provides an illustrative example of the risk graded approach, but is not the source of the margins on which he bases his opinions; rather, the source of the margins on which he bases his expert opinion are the conservatisms inherent in typical nuclear power plant design and acceptance criteria, as well as actual demonstration of the capability of key SSCs at the PFSF to withstand a beyond design basis earthquake of 10,000 years or more without the release of radioactivity to the environment. *Id.* at 36-37, 41-42. He further stated that the construction of fragility curves is not required to demonstrate such conservatism, as claimed by the State;[103] rather, that the conservatism may be shown by demonstrating that SSC failure will not occur. *Id.* at 37-39.

6.48. Finally, Dr. Cornell testified that the PFSF meets the requirements of the current preferred approach of the rulemaking plan as modified, and that the issues raised in Part E of the contention, concerning the Staff SER's stated bases for approving the exemption request, do not

[103] A "fragility curve" is a quantitative representation of the capacity of a component or structure with respect to seismic ground motion, reflecting the engineer's best (realistic) judgment of that capacity and the uncertainty about that capacity. Generally, it is an S-shaped curve that plots the probability of failure versus the level of ground motion. Cornell, Post Tr. 7856, at 34.

affect the bases for his opinion that the 2000 year DBE for the PFSF provides adequate protection of the public health and safety. *Id.* at 42-53. Specifically, his testimony shows:

(a), as to Subpart E.1,the State's reliance on the original rulemaking plan is now obsolete, *Id*. at 45;

(b) as to Subpart E.3, the historic discrepancy between different mean estimates has been largely resolved, and it is now clearly established that the typical SSE at nuclear power plants across the country has a mean annual probability of exceedance of approximately 10^{-4} (or, in the Western United States, a lower MAPE of about 2×10^{-4}, or 5,000 years), as was stated in the Staff's SER, as well as in various DOE and NRC publications,[104] -- and the use of a mean (rather than median) value is appropriate and has previously been utilized by the Commission, *Id*. at 46-48;

(c) as to Subpart E.4, DOE-STD-1020-94 (later revised in DOE-STD-1020-2002) provides an appropriate model to illustrate the application of a risk-graded approach for seismic design, and DOE's revision of this standard to adopt a 2,500-year ground motion for PC-3 SSCs left the performance goal for PC-3 SSCs unchanged at 1×10^{-4}, *Id*. at 49-50;

(d) as to Subpart E.5, the TMI-2 ISFSI exemption corroborates the appropriateness of a conclusion here that a DBE with a MAPE of 2,000-years provides sufficient protection of public health and safety in accordance with risk-informed principles, *Id*. at 50; and

(e) as to Subpart E.6, the design procedures and criteria in the International Building Code (2000) ("IBC-2000) are much less conservative than the design procedures and criteria in the NRC Standard Review Plan, such that a simple comparison to the 2,500-year return period specified

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[104] In this regard, Dr. Cornell cited DOE-STD-1020-94; the 1997 DOE Topical Report for Yucca Mountain (TR-003); and a recent assessment of NRC regulatory guidance on seismic design standards NUREG/CR-6728, "Technical Basis for Revision of Regulatory Guidance on Design Ground Motions: Hazard- and Risk-consistent Ground Motion Spectra Guidelines" (Risk Engineering, Inc.) (October 2001). Cornell, Post Tr. 7856, at 47.

therein would not be proper[105] -- and indeed, despite the larger return period in IBC-2000, the design criteria and procedures of the Commission's SRPs for an ISFSI result in important-to-safety SSCs at the PFSF with a 2,000-year DBE that have a mean annual probability of failure at least two times lower than essential facilities designed to IBC-2000 standards. *Id*. at 50-52; Tr. 12961-62.[106]

6.49. In sum, Dr. Cornell provided a persuasive and informed analysis of design conservatisms inherent in the PFSF design, and well-developed support for a finding that the PFSF seismic design, based upon the Applicant's PSHA with a 2,000-year return period ground motion, provides adequate protection of the public health and safety.

6.50. Applicant witness Donald Wayne Lewis provided more limited testimony, concerning specific details of the PFS Facility operations and SSCs important to safety at the PFSF.[107] He described the positioning of shipping, transfer, and storage casks within the CTB transfer cells, and indicated that each would be attached to the CTB walls with seismic struts during times when the cask would be involved in transfer operations. Lewis, Post Tr. 8968, at 2-4; Tr. 8971-72,

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[105] For example, a first step in the IBC-2000 design procedures is to multiply the DBE by two-thirds (which, at the PFS site, would reduce the DBE from 2,500 years to 800 years), and only for "essential structures" would an "importance factor" of 1.5 would be applied, to effectively recover this deduction. Cornell, Post Tr. 7856, at 51. Further, even for "essential structures," IBC-2000 and the Utah Building Code ("UBC") are significantly less conservative (by a factor of 2.5 to 10) than either DOE-STD-1020 for PC-3 SSCs or the NRC's Standard Review Plans, Cornell, Post Tr. 7856, at 51-52.

[106] Similarly, highway bridges in the United States are generally designed to a model bridge code that requires only a 500-year return period DBE; and they have structural design procedures and criteria similar in conservatism to those of model building codes like the UBC and IBC-2000. Therefore, the PFSF design with a 2,000-year DBE provides a higher safety level than the essential bridges in Utah referred to by State witness Dr. Bartlett. Cornell, Post Tr. 7856, at 52-53.

[107] Mr. Lewis also explained the safety classification of SSCs at the PFSF that are relevant to Unified Contention Utah L/QQ. *See* Lewis, Post Tr. 8968, at 5-6.

9067-71.[108] He further explained the process for transferring spent fuel canisters from their shipping casks to their intended storage casks, the time involved in such transfer operations, and the methodology utilized in calculating the time involved in canister transfer operations. Lewis, Post Tr. 8968, at 2-6. He also explained that throughout such operations, potential drops and tipovers of the fuel canister or shipping, transfer, and storage casks is precluded by the safety measures taken during transfer operations and the use of seismically qualified SSCs, including cranes and associated lifting equipment. Lewis, Post Tr. 8968, at 2-4, 6; Tr. 9047-49.

6.51. With respect to the amount of time involved in transfer operations, Mr. Lewis testified that the time allotted for such operations (totaling approximately 9 hours) was conservatively calculated in order to provide a benchmark for estimating worker doses; the actual time involved in such operations, however, is likely to be considerably lower. *Id.* at 4-6; Tr. 8974-75, 9039.[109] Thus, the transfer of spent fuel canisters from the shipping cask to the storage casks at the PFSF will occur only during a small fraction of time that the facility is in operation; at all other times, the canister is protected within a closed shipping or storage cask. Lewis, Post Tr. 8968, at 4-5; Tr. 9073-74.

6.52. On this basis, as discussed by PFS witness Dr. Cornell, we are satisfied that the Applicant's calculations of the risk associated with operation of the PFSF, which do not take such time considerations and in-cask protections into account, overstate the seismic risk, thereby affording an additional layer of conservatism in considering the safety of the facility. *See* Cornell, Post Tr. 7856, at 25.

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[108] To eliminate any misunderstanding, the Applicant committed to revise a Table in its SAR, which describes transfer cell operations, to clarify the points at which seismic struts would be attached and removed from the casks. Tr. 9070-71.

[109] The Applicant has conservatively estimated a total of about 20 hours from the time it inspects a shipping cask on its incoming railroad car or other transportation vehicle, to the time at which the loaded storage cask is placed on the concrete storage pad. Tr. 9074-75.

NRC Staff's Testimony

6.53. The NRC Staff's testimony provided an independent assessment of the merits of the PFS seismic exemption request and explained the bases for the Staff's determination that the Applicant should be permitted to utilize probabilistic seismic hazard analysis (PSHA) methodology and a 2,000-year return period ground motion in establishing the seismic design basis for the PFS Facility -- in other words, that issuance of a license for the PFS Facility, incorporating its requested exemption from the seismic requirements of 10 C.F.R. § 72.102(f)(1), would adequately protect the public health and safety.

6.54. Staff witnesses Drs. Stamatakos, Chen and McCann described the Staff's review of the PFS exemption request. As discussed above, after extensive site characterization studies were performed by Geomatrix Consultants, Inc., on April 2, 1999, PFS submitted a request in which it sought 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). Stamatakos/Chen/McCann Post Tr. 8050, at 8-9; PFS Exh. 247. 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). Stamatakos/Chen/McCann Post Tr. 8050, at 12; PFS Exh. 248.

6.55. Two related questions were inherent in the Applicant's exemption request: (1) whether the Applicant should be permitted to substitute a PSHA in lieu of the DSHA required by the regulations in 10 C.F.R. Part 72, 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. The Staff requested the assistance of the CNWRA in evaluating the seismic exemption request, as described in Section 2.1.6.2 of the Staff's Consolidated SER (Staff Exh. C). This evaluation was performed by Drs. Stamatakos, Chen and McCann, as described in a September 1999 report entitled "Seismic Ground Motion at the Private Fuel Storage Facility Site in the Skull Valley Indian Reservation," issued by the CNWRA (Stamatakos, et al., 1999) (Staff Exh. Q), and included an independent technical review of the seismic hazard investigations at the proposed PFS site, as described in the Consolidated SER, § 2.1.6, and in the CNWRA report. Stamatakos/Chen/McCann Post Tr. 8050, at 8-9.

6.56. To support its evaluation of the PFS exemption request, the Staff requested that the CNWRA conduct a technical review of the seismic and faulting hazard investigations at the proposed PFS Facility site. The objectives of these 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. *Id.* at 9-10.

6.57. 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. *Id.* at 10.[110]

6.58. 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 did not affect the Staff's 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 (Staff Exh. C, §§ 2.1.6.1 and 2.1.6.2) and in Stamatakos, et al. (1999) (Staff Exh. Q). *Id.*

6.59. In determining whether a PSHA may be utilized in lieu of the deterministic approach required in 10 C.F.R. Part 72 for the seismic hazard assessment of an ISFSI site located west of the Rocky Mountain Front, the Staff considered that the Commission (and Staff) have previously taken certain actions which indicate general approval of the use of PSHA methodology. *Id.* at 11.

6.60. First, the Staff observed that the Commission had previously indicated that the uncertainty associated with evaluating seismic design ground motions for NPPs must be addressed. In this regard, the Commission issued regulations and regulatory guidance that

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[110] State witness Dr. Arabasz similarly concluded that the PFS seismic hazard curve developed by Geomatrix was conservative. Tr. 9973-75.

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 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 (Staff Exh. T), as modified in SECY-01-0178 (Staff Exh. U). Second, as set forth in SECY-98-071, the Staff observed that the Commission had 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 TMI-2 spent fuel debris ISFSI, located at the Idaho National Engineering and Environmental Laboratory. *Id.*

6.61. The Staff reviewed the Commission's actions in considering an alternative to the deterministic approach specified in 10 C.F.R. Part 100, Appendix A, and observed that those actions 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. *Id.* at 11-12.

6.62. As set forth in Section 2.1.6.2 of the Staff's Consolidated SER, pages 2-50 to 2-51 (Staff Exh. C), the Staff 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 concluded that the Applicant's exemption request should be granted. The Staff considered a number of technical and regulatory factors in its evaluation of this matter. These included (1) the Applicant's exemption request and the PSHA submitted in support thereof; (2) its 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. *Id.* at 12-13.

6.63. With respect to the technical analysis supporting the Applicant's seismic exemption request, the Staff found the Applicant's PSHA results to be conservative. As stated in the Consolidated SER (Staff Exh. C, at 2-48), this determination was based upon a 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. *Id.* at 13.

6.64. 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 as 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. *Id.*

6.65. Another aspect of the Applicant's seismic source characterization that appeared 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 of the Skull Valley fault systems that was performed by the Staff. The Staff's slip tendency analysis, performed by Dr. Stamatakos, 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 is conservative. *Id.* at 13-14.[111]

6.66. 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 by Martinez, *et al.* (1998)[112] and optimization of slip tendency values for segments of faults such

[111] 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. Stamatakos/Chen/McCann Post Tr. 8050, at 14.

[112] Martinez, L., C. M. Meertens, and R. B. Smith, "Anomalous intraplate deformation of the Basin and Range-Rocky Mountain transition from initial GPS measurements," *Geophysical Research. Letters 24*: 2741-2744 (1998). *See* Staff Exh. C at 2-37, 2-72.

as the Wasatch that are known to be produce earthquakes.[113] 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. *Id.* at 14.

6.67. 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. *Id.* at 15.

6.68. By contrast, in the Applicant's site-to-source distributions used in the ground motion attenuation equations, Geomatrix 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 was conservative. Based on its own slip tendency analysis, the Staff concluded that seismic source models that incorporate slip tendency would result in a lower ground motion hazard than the one developed by the Applicant. *Id.*

6.69. In addition, the slip tendency results in the Staff's analysis suggest that Geomatrix may have overestimated the maximum magnitude of the East and East Cedar Mountain faults near

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[113] As discussed *infra* at 205, Dr. Arabasz disagreed with the reliability of the Martinez study, insofar as Dr. Stamatakos used GPS data to derive slip rates. Dr. Arabasz did not criticize the Martinez paper insofar as it relates to the orientations of the principal stresses. *See generally,* Tr. 9865-78, 10128-30.

the proposed PFS site. In its Safety Analysis Report, 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 as described by Geomatrix. 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. 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 produced conservative estimates of maximum magnitude. *Id.* at 15-16.

6.70. 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. *Id.* at 16.

6.71. As described by Dr. Stamatakos, 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 developed by

Geomatrix 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 a seismic hazard curve for Salt Lake City developed with USGS National Earthquake Hazard Reduction Program data. This was graphically and clearly shown in a Figure entitled "Comparison of Western U.S. Hazard Curves," prepared by Dr. Stamatakos, in a highly illustrative scientific notebook entry. *Id.* at 16-17; Staff Exh. JJ, at 5.

6.72. Similarly, the Staff observed that 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 by Dames & Moore, Inc., in 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. Likewise, the Staff observed that the ground motions estimated by Geomatrix 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, and is capable of producing significantly larger magnitude earthquakes than the faults near the proposed PFS Facility site in Skull Valley.[114] In sum, the Staff found that because the

[114] As noted above, Dr. Arabasz criticized the Martinez paper's means of establishing the rate and amount of fault slip, based on GPS data. However, while this may constitute a new application of GPS technology to which Dr. Arabasz or others may not subscribe, we have been provided no reason to conclude here that such use is truly improper. *See generally*, Tr. 9865-78, 10128-30. Moreover, the State has conceded that the Geomatrix analysis disregarded slip rate variances along different fault segments, thus rendering the analysis conservative, in its view, by a factor of three. Thus, while Dr. Stamatakos relied on GPS data in finding the Geomatrix analysis to be conservative (by a factor of ten), the only issue in dispute here is the degree to which the Geomatrix analysis may be conservative due to its treatment of slip rate. See Tr. 9875-80.

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. Stamatakos/Chen/McCann Post Tr. 8050, at 17.

6.73. As further stated in the Consolidated SER (Staff Exh. C), 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.

Id. at 17-18.

6.74. For the reasons set forth above, the Staff concluded that the Applicant's exemption request is acceptable, insofar as it is based upon use of the Applicant's PSHA and seismic design ground motions that have a mean annual probability of acceptance of 5×10^{-4} (2,000-year return period), and that this provides an acceptable basis for the seismic design of the proposed PFS Facility. *Id.* at 18.

6.75. In addition to concluding that the PFS exemption request is acceptable based on 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 (Staff Exh. C), pages 2-49 to 2-51. *Id.* at 18-19.

6.76. 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: In this regard, SECY-98-071 stated 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 (Staff Exh. S), 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.* at 19.[115]

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[115] The referenced statements in SECY-98-071 cited an NRC 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 (Nov. 12, 1980), which the Commission published when it adopted the regulations in 10 C.F.R. Part 72. That Statement of Consideration contained the following discussion concerning the comparative radiological risks of spent fuel storage in an ISFSI and operation of a nuclear power plant:

> 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 the one year of decay before storage in an ISFSI required by the rule and with the

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6.77. Second, as set forth in the Consolidated SER (Staff Exh. C) at 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. In Regulatory Guide 1.165, based on an analysis of the SSEs for existing NPPs, 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). *Id.* As the Staff explained, this 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}. *Id.*[116]

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[115](...continued)

> low inventory of volatile radioactive materials readily available for release to the environs.
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> The controlled area for an ISFSI is not the same as the exclusion area for a reactor because the design basis accidents are different. Reactor accidents involve a potential release of radioactive materials, including short-lived species such as ^{131}I. Design basis accidents of concern at an ISFSI primarily involve direct radiation from exposure to the spent fuel rather than releases of radioactive materials. The areas requiring control or protective action measures for the protection of the public are quite different . . . .

Id., 45 Fed. Reg. at 74694, 74696.

[116] The Staff's conversion of the Reference Probability median value to a mean value was challenged by State witness Arabasz, who believed the Staff had misunderstood the Reference Probability in Regulatory Guide 1.165. *See* Arabasz, Post Tr. 9098, at 7-8; Tr. 9156-57. We find no such problem. As Staff witness Dr. McCann explained, if the analysis in Regulatory Guide 1.1.65 had used SSE mean values instead of median values, the reference probability for nuclear power plants would be 1.0E-4. *See* Tr. 8148, 8331-32, 8338. Further, the concern raised by Dr. Arabasz is of no importance, in that Dr. Arabasz clearly agrees with the conclusion reached by the Staff -- *i.e.,* that the mean annual reference probability for the nuclear power plants in Regulatory Guide 1.165 would be 1.0E-4. *See* Tr. 9157-58, 9309.

6.78. 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 x 10^{-4} -- which is equivalent to an SSE with a 5,000-year return period. This is demonstrated in a Department of Energy publication entitled, "Preclosure Seismic Design Methodology for a Geologic Repository at Yucca Mountain," TR-003-NP, Rev. 2 (1997), Table C-2 at C-18 (State Exh. 202). Stamatakos/Chen/McCann, Post Tr. 8050, at 19-20.[117]

6.79. 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. *Id.* at 20.[118]

6.80. Finally, in addition to the above considerations, the Staff's Consolidated SER indicates that 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

[117] Specifically, the mean annual probability of exceedance for SSEs at the five western United States nuclear power plants listed in the DOE TR-003 report, were reported to be as follows: Diablo Canyon - 1.7E-04/year (5,882-year return period); Palo Verde - 3.8E-05/year (26,316-year return period); San Onofre - 3.0E-04/year (3,333-year return period); Washington Nuclear Plant No. 2 - 2.8E-04/year (3,571-year return period); Washington Nuclear Plant No. 3 - 2.2E-04/year (4,545-year return period). State Exh. 202; Tr. 8031, 8327-28.

[118] Dr. Arabasz contested the Staff's view that existing nuclear power plants in the western United States have an average mean annual probability of exceedance of approximately 2 x 10^{-4} (5,000-year return period). We are satisfied, however, that the Staff has correctly analyzed this matter, as set forth in DOE's Yucca Mountain TR-003 report and as more fully explained in the Staff's testimony. *See* Stamatakos Rebuttal, Post Tr. 12648, at 3-8; Tr. 8154, 8338-42, 12702-16, 12759-61; *see also* Staff Exh. 62.

to be appropriate. These were (a) the Department of Energy's issuance of 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. *Id.* at 19-20; *see* Staff Exh. C, at 2-51.

6.81. 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). Stamatakos/ Chen/McCann Post Tr. 8050, at 20-21.[119]

6.82. With respect to the second matter identified above (*i.e.,* the TMI-2 ISFSI exemption), the Staff referred to the Commission's 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 (designed to passively store spent nuclear fuel debris in dry storage casks), which is discussed in SECY-98-071 and CNWRA-98-007 (Chen and Chowdhury, 1998). In this

[119] As the Staff's testimony points out, although the Staff referred to DOE-STD-1020-94 in its SER, it 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. Stamatakos/ Chen/McCann Post Tr. 8050, at 21.

regard, the Staff explained that it found the Commission's approval of the TMI-2 ISFSI seismic design ground motion to constitute an appropriate point of reference, notwithstanding the fact that it did not establish a regulatory criterion having generic applicability. Stamatakos/ Chen/McCann Post Tr. 8050, at 21.

6.83. In summary, the Staff considered that the DOE standard and the TMI-2 ISFSI exemption 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. *Id.* at 21.

6.84. Further, the Staff witnesses expressed their disagreement with each of the State's assertions in Part E of Unified Contention Utah L/QQ. First, they addressed the State's assertion 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." In this regard, they noted that the Staff's Consolidated SER had found the Applicant's use of PSHA methodology to be 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).[120] 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. *Id.* at 23-24.

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[120] As discussed above, in July 2002, the Commission published a proposed rule in the *Federal Register*, which explicitly provides for the use of PSHA methodology in establishing the seismic design basis for an ISFSI. This development reinforces the view, expressed in the Staff's testimony, that the use of PSHA methodology in the licensing of an ISFSI is consistent with current NRC licensing philosophy. *See* Stamatakos/Chen/McCann, Post Tr. 8050, at 23-24.

6.85. Second, for the reasons discussed above, the Staff concluded that 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 PFSF, particularly when considering the lower radiological risk that an ISFSI with a dry cask storage system presents as compared to a nuclear power plant. *Id.* at 24. This matter is further discussed *infra,* with respect to Subpart E.3 of the contention.

Unified Contention Utah L/QQ, Subpart E.1

6.86. The Staff disputed the State's assertions in Subpart E.1 of the contention, that the PFS exemption request should be denied because it "fails to conform to the SECY-98-126 (June 4, 1998) rulemaking plan scheme," and that if the failure of an SSC would not meet 10 C.F.R. § 72.104(a) dose limits, it must be designed for a 10,000-year event. In this regard, they noted (as do we) that SECY-98-126 did not establish a regulation or even a proposed regulation, but only set out a proposed regulatory approach -- which, in any event, was superseded by the Commission's approval of a modified rulemaking plan in SECY-01-0178. As the Staff observed, the favored option in SECY-01-0178 proposes the use of PSHA methodology in establishing a seismic design 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).[121] Thus, wholly apart from the fact that a rulemaking plan does not establish a binding regulatory requirement, the approach specified in SECY-98-126 has been superseded and has absolutely no regulatory significance at this time. *Id.*

6.87. 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

[121] Also, as noted above, the proposed rule published in July 2002 follows the single-tier approach of SECY-01-178, rather than the two-tiered approach of SECY-98-126.

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 the design earthquake based on ground motions having a mean annual probability of exceedance of 5×10^{-4} (2,000 year-return period). The Staff cited this example as a pertinent reference point, although it noted that the TMI-2 ISFSI exemption does not establish a generic precedent. *Id.* at 25.

6.88. Third, the Staff observed that in adopting the Part 72 regulations, the Commission indicated that the design earthquake for an ISFSI should "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 constituted just such a case-specific approval, based upon a consideration of the relative safety of the proposed PFSF in the event of an earthquake, as compared to the radiological risks of a major seismic event at a nuclear power plant. *Id.*

Unified Contention Utah L/QQ, Subpart E.2

6.89. With respect to Subpart E.2 of the contention, Dr. Stamatakos expressed the view that the State's reliance on the dose limits in 10 C.F.R. § 72.104(a), is misplaced. He cited the testimony of Michael Waters (discussed *infra*), in which this regulatory standard was explained to apply 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. Further, he cited the testimony of Staff witnesses Daniel Pomerening, Dr. Vincent Luk and Michael Waters, and the accident analyses in Chapter 15 of the Consolidated SER, 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. *Id.* at 26.

6.90. We have previously addressed the issues of cask tipover and sliding in our analysis of Part D of this contention; radiological dose issues are discussed below, in our analysis of Subpart E.2 of this contention. Accordingly, we do not address those matters here, in connection with the testimony of Drs. Stamatakos, Chen and McCann.

Unified Contention Utah L/QQ, Subpart E.3

6.91. With respect to Subpart E.3 of the contention, the Staff witnesses (Dr. McCann, in particular) supported, as reasonable and proper, the Staff's reliance on the reduced radiological hazard of stand-alone ISFSIs as compared to commercial power reactors, as a justification for granting the PFS exemption, contrary to the State's assertion that such reliance was "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." *Id.*

6.92. 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 an ISFSI like the proposed PFSF 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 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, which 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.

Stamatakos/ Chen/McCann Post Tr. 8050, at 26-27, *citing* 45 Fed. Reg. at 74,694.[122]

6.93. 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 <u>median</u> probability of exceedance of the SSEs, and established a Reference Probability for nuclear power

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[122] The Commission also described the lower relative risk of an ISFSI as compared to an operating nuclear power plant more recently, upon amending the regulations in Part 72. *See* Statement of Consideration, "Interim Storage of Spent Fuel in an Independent Spent Fuel Storage Installation at a Reactor Site; Site-Specific License to a Qualified Applicant," 60 Fed. Reg. 20,879, 20,883 (1995) ("the public health and safety risks posed by ISFSI storage . . . are very different from the risks posed by the safe irradiation of the fuel assemblies in a commercial nuclear reactor," in which high operating temperatures and pressures, and "a driving force" for dispersion are present; "the absence of such a driving force . . . in an ISFSI . . . substantially eliminates the likelihood of accidents involving a major release of radioactivity from spent fuel stored in an ISFSI."). *See also, Private Fuel Storage, L.L.C.* (Independent Spent Fuel Storage Installation), CLI-01-22, 54 NRC 255, 265 (2001) (discussing the relative radiological risks of ISFSIs and NPPs).

plant SSEs of 1 $\times 10^{-5}$ (100,000-year return period). Stamatakos/ Chen/McCann Post Tr. 8050, at 27-28.

6.94. 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, ground motions corresponding to the median-based Reference Probability in Regulatory Guide 1.165 of 1×10^{-5} (100,000 year return period) are approximately equal to ground motions that correspond to a mean-based Reference Probability of 1×10^{-4} (10,000 year return period). Stamatakos/ Chen/McCann Post Tr. 8050, at 28.

6.95. 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 exceedance probabilities of the SSEs for 69 NPPs, 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 all of these 69 NPPs are located in the Eastern United States. *Id.*

6.96. Indeed, the Staff previously 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. Thus, in the safety evaluation attached to SECY-98-071 (Staff Exh. S), 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.

Stamatakos/Chen/McCann Post Tr. 8050, at 28, *citing* SECY-98-071, Enclosure at [2] (emphasis added).

6.97. In addition to indicating 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," the safety evaluation attached to SECY-98-071 stated that under Regulatory Guide 1.165, "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, however -- as the Staff's witnesses observed -- 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 <u>greater</u> than 1×10^{-4} (10,000-year return period). Indeed, as discussed above, such an evaluation was carried out by DOE, which concluded that the <u>mean</u> annual probability of exceeding the design basis ground motions for NPPs in the western United States is approximately <u>2×10^{-4}</u> (5,000-year return period). *See* discussion *supra*, at ¶ 6.78; Stamatakos/Chen/McCann Post Tr. 8050, at 29; Stamatakos Rebuttal, Post Tr. 12648, at 3-6; Tr. 8154, 8338, 12702-09.

6.98. 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 <u>mean</u> 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, further, 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). *Id.*

6.99. In sum, contrary to the State's assertion, Drs. McCann and Stamatakos explained that 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. *See id.*;

Stamatakos/ Chen/McCann Post Tr. 8050, at 28. Based on our consideration of the Staff's testimony, we share this conclusion.

Unified Contention Utah L/QQ, Subpart E.4

6.100. Drs. McCann and Stamatakos further responded to the State's assertion, in Subpart E.4 of this contention, that the Staff's reference to DOE-STD-1020-94 fails to support its determination to approve the PFS exemption request, in that this standard was not followed in the original Part 72 rulemaking plan. The Staff did not "adopt" DOE STD-1020-94 in approving the 2,000-year return period for use in the design of the proposed PFSF. Rather, as discussed above, the Staff cited the DOE Standard as a reference point, in that it established a mean reference probability (corresponding to a 2,000-year return period) as the basis for determining the design ground motions for SSCs at DOE Performance Category-3 facilities, which are generally comparable to NRC-licensed ISFSIs. Further, the Staff did not attempt to impose DOE STD-1020-94 as a regulatory standard on the proposed PFSF, 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. *Id.* at 30.

6.101. As Drs. McCann and Stamatakos also explained, 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 also relies on considerations 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 NPPs or PC-4 facilities (which present risks similar to a NPP). *Id.* at 30-31. 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.

Id. at 31, *citing* SECY-98-071 [Enclosure at 2-3]. Thus, contrary to the State's apparent belief, 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. *See id.*; Stamatakos Rebuttal Post Tr. 12648, at 2-3; Tr. 8069-71, 8074, 8334, 12678, 12695-96, 12697-700.

6.102. The Staff further addressed DOE's revision of DOE-STD-1020-94, in DOE-STD-1020-2002, dated January 2002. In this regard, the Staff observed that, in the 2002 revision of the 1020-94 Standard, DOE revised 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). Further, the responsible DOE official had informed the Staff that this 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. *Id.* at 31-32. The Staff's description of DOE's reason for revising this aspect of DOE STD-1020-94 is directly supported by DOE's published explanation of this matter. *See* Staff Exh. II, at iv.

6.103. 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 did not affect the Staff's 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. Stamatakos/Chen/McCann Post Tr. 8050, at 32.

6.104. We share the Staff's view that DOE's revision of the return period for PC-3 facilities is not significant, for the reasons stated. Further, we find that DOE's substitution of a 2,500-year return period ground motion in place of the previous 2,000-year return period for DOE PC-3 facilities is of no consequence for a wholly different reason: As discussed above, In revising the specified return period, DOE simultaneously revised its seismic scale factor from an SF of 1.0 to an SF of 0.9, thus effectively leaving the seismic design standard for PC-3 facilities unchanged. *See* Staff Exh. QQ at C-10 to C-11; Staff Exh. II at iv. In light of DOE's revision of the SF factor, we find no basis for the State's criticism of the Staff's citation to DOE STD-1020-94 and the 2,000-year return period established therein (Arabasz, Post Tr. 9098, at 10-11), inasmuch as the design standard has effectively remained the same in DOE-STD-1020-2002 with its use of a nominal 2,500-year return period and a 0.9 SF factor. *See* Tr. 7910-11.

Unified Contention Utah L/QQ, Subpart E.5

6.105. Next, Dr. Chen addressed the State's assertion, in Subpart E.5 of the contention, that the Staff's reliance on the TMI-2 ISFSI exemption was misplaced, in that the TMI-2 ISFSI was designed to a higher horizontal peak ground acceleration (0.36 g) than the 2000-year return period value (0.30 g). Dr. Chen (who was involved in the agency's review of the TMI-2 ISFSI exemption request) observed that 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. Referring to SECY-98-071, she explained 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 2,000-year return period (5×10^{-4} mean annual probability of exceedance) ground motion, the Commission in fact approved the lower <u>2,000-year</u> ground motion as the acceptable seismic design basis for the facility. Stamatakos/Chen/McCann, Post Tr. 8050, at 32-33.[123]

6.106. The Staff's witnesses explained that in approving a design basis ground motion for the TMI-2 ISFSI, the Staff (and Commission) had 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

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[123] Dr. Chen observed that existing INEEL architectural engineering standards led to 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; and because the TMI-2 ISFSI was placed at the ICPP site, DOE utilized that same standard in constructing the lower-risk TMI-2 ISFSI. She further explained, however, that the Staff approved the TMI-2 ISFSI exemption based on a 2,000-year ground motion, even though the TMI-2 ISFSI was designed for a higher, 0.36 g ground motion. Stamatakos/Chen/McCann, Post Tr. 8050, at 32-33; Tr. 8182-84.

2000-year mean return period ground motion as the DE (design earthquake) is adequately conservative." *Id.* at 33.

6.107. Finally, the Staff observed that the TMI-2 ISFSI exemption is also pertinent insofar as 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. Similarly, accident analyses for the proposed PFS Facility have concluded that a cask drop event would not result in the release of radioactive materials (as discussed in testimony of Staff witness Michael Waters). Thus, the TMI-2 ISFSI example also provides a useful analogy here, with regard to the issue of relative radiological consequences. *Id.* at 33-34.

Unified Contention Utah L/QQ, Subpart E.6

6.108. With respect to Subpart E.6 of the contention, the Staff disputed the State's assertion 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." *Id.* at 34.

6.109. In this regard, they observed that the State was incorrect in its claim that design levels for new Utah building construction and highway bridges are more stringent than the design standard at issue here. As they explained, an *a priori*, simple 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 must 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, the relative risks to public health and safety, 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. *Id.* at 34-35.

6.110. Further, Dr. McCann explained that the State's second assertion -- *i.e.*, that the PFS return period should be based on a 30 to 40-year operating period rather than on the 20-year initial licensing 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, that request would be subject to review and evaluation based on available information and analyses at that time. *Id.* at 35.[124]

6.111. In sum, having conducted a detailed evaluation of this matter as set forth in the 1999 CNWRA report (Staff Exh. Q), the Staff's SER (Staff Exh. C), and their testimony in this proceeding, all three of the Staff's witnesses concluded that the use of a PSHA with a 2,000-year return period ground motion establishes an adequate level of conservatism for the PFS Facility. Tr. 8323-26.

Summary of Findings Regarding Subpart E.1 of Unified Contention Utah L/QQ

6.112. Based on our consideration of all of the evidence, we have reached the following factual conclusions regarding the matters raised in Subpart E.1 of Unified Contention Utah L/QQ, in addition to the specific findings set forth above.

6.113. First, we are satisfied that the use of the PSHA methodology provides an acceptable basis upon which to determine the seismic design ground motions of the proposed PFS Facility. The use of PSHA methodology has been approved by the Commission for use in establishing the seismic design of nuclear power plants in 10 C.F.R. § 100.23, and the Commission has indicated its approval of this approach for dry cask ISFSI facilities, as well, in recent rulemaking activities.

6.114. Second, we are satisfied that the PFS PSHA results are acceptable -- and, like the staff, we consider those results to be conservative. In this regard, we take note of the Staff's

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[124] As noted above, Dr. Arabasz had criticized the Staff's use of a "metric" in its Preliminary SER (Staff Exh. A) and the modified rulemaking plan in SECY-01-178, in which facilities were compared based on a cumulative probability of exceeding a design ground motion over the course of their operational lives. Arabasz, Post Tr. 9098, at 13-14. However, the Staff does not rely on this type of "operational life" comparison in its final SER for the PFS Facility, and it does not proffer it as a basis for granting the PFS exemption request. *See, e.g.,* Staff Exh. C at 2-51; Tr. 8161-62. Moreover, witnesses for each of the parties -- including Dr. Cornell, Drs. McCann and Stamatakos, and Dr. Arabasz -- provided support for the conclusion that seismic hazard is best calculated on the basis of the mean annual probability of exceeding the design ground motion, consistent with other risk acceptance guidelines, rather than consideration of the risk over some period of operation. *See* Cornell, Post Tr. 7856, at 53; Tr. 8161-64; Tr. 9999-10001, 10008.

independent analyses (*e.g.*, gravity modeling and slip tendency analysis) which demonstrate the potential conservatism in the Applicant's seismic hazard assessment. We also consider the fact that the Applicant calculated the seismic hazard in Skull Valley to be higher than the seismic hazards for other sites with large seismic hazard curves -- including sites in the Salt Lake City vicinity, despite the fact that fault sources near Salt Lake City are larger and more seismically active than fault sources near the PFS site.

6.115. Third, while the State would have us require an ISFSI applicant to follow the two-tiered approach of the rulemaking plan contained in SECY-98-126, that approach has no current regulatory significance. Rather, the establishment of a single design earthquake level is favored in the revised rulemaking plan contained in SECY-01-178, as well as in a recently proposed revision to 10 C.F.R. Part 72. *See* 67 Fed. Reg. at 47751-52.

6.116. We come now to the pivotal question upon which the outcome of this subpart of the contention depends: What is the appropriate level at which to set the design basis earthquake for the PFS Facility. As set forth below, it is our conclusion that the PFS seismic design may be established based on the use of the Applicant's PSHA with a design basis earthquake having a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period).

6.117. As we begin to address this question, we note, as stated above, that the PFS PSHA is conservative, so that the 2,000-year return period earthquake predicted by Geomatrix and PFS is, in fact, larger than the 2,000-year return period earthquake for the site. Indeed, the 2,000-year horizontal peak ground acceleration for Skull Valley estimated by PFS is actually greater than the 2,500-year ground motions considered for the nine sites along the Wasatch Front that were evaluated as part of the Utah Department of Transportation I-15 Reconstruction Project.

6.118. Moreover, it is beyond dispute that the radiological hazard posed by a dry cask storage facility is inherently lower than the radiological hazard of an operating commercial nuclear power plant. For this reason, an ISFSI's design basis ground motion need not be as large (*i.e.*, improbable) as those used for NPP design. The evidence shows that the safe shutdown earthquake at nuclear power plants across the United States having an average mean annual probability of exceedance of 1.0×10^{-4}. Further, 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 the 5,000-year return period ground motion. The exceedance probability of the design basis earthquake for an ISFSI at the PFS site may therefore be set at a value that is larger *(i.e.*, less improbable) than these values established for NPPs.

6.119. The appropriateness of selecting a 2,000-year return period ground motion to establish the design basis ground motion for an ISFSI is confirmed by various considerations. First, the U.S. Department of Energy has adopted the use of a 2,000-year return period ground motion in DOE-STD-1020-94 for Performance Category-3 facilities, which would include a dry cask storage ISFSI like the proposed PFS Facility. While the 2002 revision of that document adopts a 2,500-year earthquake, DOE coupled its selection of that earthquake with application of a safety factor of 0.9, which all parties agree effectively leaves the design standard unchanged. Thus, in NRC parlance, where an "SF" factor is not applied to the DE, a 2,000-year earthquake would remain appropriate for a facility like the proposed PFSF.

6.120. Further, the State's assertion that design levels for new Utah building construction and highway bridges are more stringent than the use of a 2,000-year earthquake for the PFS design is not correct. The State's assertion ignores the relative levels of conservatism required in the design of facilities of different types, and the design conservatisms inherent in NRC codes and standards used in the construction of a nuclear facility such as the PFS ISFSI. Design conservatism and margins of safety are substantially higher for nuclear facilities than highways, bridges, or normal buildings.

6.121. Finally, the acceptability of a 2,000-year return period earthquake in establishing the design of the PFS Facility is conclusively shown by the parties' evidence concerning the dose consequences of a cask tipover at the PFS Facility. As discussed below, however improbable a cask tipover event may be, the evidence clearly supports a conclusion that a tipover of all 4,000 casks at the PFS Facility would not result in offsite dose consequences in excess of the dose limits stated in 10 C.F.R. § 72.106(b). Accordingly, the occurrence of a design basis earthquake with a MAPE of 5×10^{-4} (2,000-year return period) will not result in adverse consequences to public health and safety. We turn now to consider the radiological dose consequence issues raised in this contention.

2. Radiological Dose Considerations.

6.122. As stated above, in Part E of Unified Contention Utah L/QQ, the State asserted that the Applicant's request for an exemption from the deterministic seismic requirements in 10 C.F.R. § 72.102(f) should be denied, based in part on radiological dose considerations:

> 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.

Id., emphasis added.

6.123. We addressed the State's assertions in Subpart E.1 of this contention above, in which we concluded that the Applicant's failure to follow the initial rulemaking plan in SECY-98-126 was of no consequence, and that the PFSF need not be designed to a 10,000-year return period ground motion. We now come to the issue of whether PFS should nonetheless be required to meet the offsite dose limits set forth in 10 C.F.R. § 72.104(a). For the reasons set forth below, we conclude that the dose limits in § 72.104(a) apply only to normal operations and anticipated occurrences, and do not apply to accidents or design basis events such as a design basis earthquake -- which are governed by the dose limits in 10 C.F.R. § 72.106(b). Further, we find that the occurrence of either a 2,000-year return period DBE, or a beyond-design-basis 10,000-year return period earthquake, will not result in offsite radiological doses that exceed the limits in 10 C.F.R. § 72.106(b). Accordingly, we find that issuance of the exemption requested by PFS affords adequate protection of the public health and safety, and will not endanger life or property or the common defense and security.

Applicable Legal Standards

6.124. The Commission's regulations in 10 C.F.R. Part 72 contain two sets of requirements pertaining to offsite radiological dose consequences associated with the licensing of an ISFSI. First, the following requirements apply to "normal operations and anticipated occurrences":

> **§ 72.104. Criteria for radioactive materials in effluents and direct radiation from an ISFSI or MRS.**
> (a) During normal operations and anticipated occurrences, the annual dose equivalent to any real individual who is located beyond the controlled area must not exceed 0.25 mSv (25 mrem) to the whole body, 0.75 mSv (75 mrem) to the thyroid and 0.25 mSv (25 mrem) to any other critical organ as a result of exposure to:
>
> > (1) Planned discharges of radioactive materials, radon and its decay products excepted, to the general environment,
> >
> > (2) Direct radiation from ISFSI or MRS operations, and
> >
> > (3) Any other radiation from uranium fuel cycle operations within the region.

6.125. Second, the Commission has established the following requirements in 10 C.F.R. § 72.106(b), with respect to design basis accidents:

> **§ 72.106. Controlled Area of an ISFSI or MRS.**
>
> * * *
>
> (b) Any individual located on or beyond the nearest boundary of the controlled area may not receive from any design basis accident the more limiting of a total effective dose equivalent of 0.05 Sv (5 rem), or the sum of the deep-dose equivalent and the committed dose

> equivalent to any individual organ or tissue (other than the lens of the eye) of 0.5 Sv (50 rem). The lens dose equivalent may not exceed 0.15 Sv (15 rem) and the shallow dose equivalent to skin or any extremity may not exceed 0.5 Sv (50 rem). . . .

6.126. In sum, for normal operations and anticipated occurrences at an ISFSI, the Commission has established an offsite radiological dose limit of 25 mrem to the whole body, whereas for design basis accidents a limit equal to a total effective dose equivalent of 5 rem has been established.

6.127. The State argues that the approach of the rulemaking plan in SECY-98-126 should be applied to the exemption request and that a design basis earthquake must not result in dose consequences that exceed the dose limits in 10 C.F.R. § 72.104(a). We are aware of no legal basis for this suggestion. First, the Commission has abandoned the two-tiered approach contained in SECY-98-126, in favor of the single-tier approach in SECY-01-0178 -- and in the proposed rule published in July 2002 (67 Fed. Reg. 47745). Second, while the Applicant has sought an exemption from the requirement that it use deterministic methodology to establish the design basis earthquake for its facility, its use of a different (probabilistic) methodology to establish the design basis does not alter the Commission's standard of protection with respect to the offsite dose limits for design basis accidents and events at an ISFSI, set forth in 10 C.F.R. § 72.106(b). The State has presented no sound reason why the dose consequences of a DBE at the PFS site must meet the dose standard for "normal operations and anticipated occurrences" set forth in § 72.104(a), nor are we aware of any reason why we should require them to do so.

6.128. Accordingly, we conclude, as a matter of law, that the standard set forth in § 72.106(b) applies to the dose consequences resulting from a design basis earthquake -- regardless which ground motion is selected for the design basis earthquake -- and this dose standard applies to the Applicant's requested exemption. While our conclusion as to which standard applies is essentially a legal one, we also rely upon the evidence adduced at hearing, discussed below, which strongly supports our conclusion that the limits of § 72.106(b) governs the dose consequences resulting from a DBE. Finally, as further set forth below, we find that the Applicant's request for exemption satisfies the dose standard set forth in 10 C.F.R. § 72.106(b). We turn now to consider the parties' evidentiary presentations concerning this matter.

Evidence Presented

6.129. Of the ten days of evidentiary hearings held on Subpart E of the contention, three days (June 24 - 26, 2002) were devoted to evidence concerning radiological dose consequences. The parties presented six witnesses with respect to this issue, as discussed below.

Applicant Witnesses

6.130. The Applicant presented four witnesses with respect to Subpart E.2 of Unified Contention Utah L/QQ. Direct testimony was presented by Drs. Krishna P. Singh, Alan I. Soler, and Everett L. Redmond II ("Testimony of Krishna P. Singh, Alan I. Soler, and Everett L. Redmond II on Radiological Dose Consequence Aspects of Basis 2 of Section E of Unified Contention Utah L/QQ") (hereinafter referred to as "Singh/Soler/Redmond"), Post Tr. 12044. In addition, Mr. John Donnell joined Drs. Soler and Redmond in presenting rebuttal testimony on behalf of the Applicant. *See* Tr. 12549-67.

6.131. The professional qualifications of Applicant witnesses Drs. Krishna P. Singh and Alan I. Soler are set forth in our discussion of Section D of Unified Contention Utah L/QQ, *supra*. Applicant witness John Donnell provided his professional qualifications (summarized below) with his testimony regarding another contention heard previously in this proceeding. *See* "Testimony of John Donnell on Contention SUWA B–Railroad Alignment Alternatives" ("Donnell"), Post Tr. 4564. Applicant witness Dr. Everett L. Redmond's professional qualifications are also discussed below.

6.132. Applicant witness John Donnell received a Bachelor of Science degree from the University of Toledo where he majored in Electrical Engineering. Donnell, Post Tr. 4564, at 1;

Curriculum Vitae attached to Donnell Post Tr. 4564 ("Donnell Qualifications"), at 2. Mr. Donnell is a registered professional engineer. *Id.* He has 21 years of experience in nuclear project management and engineering, with experience in site selection; ISFSI storage technology assessment, selection, and bid specification; ISFSI project scoping, staffing, and licensing; design; and budget and schedule control. *Id*. at 1-2. As Project Manager for the proposed PFSF, he is responsible for the engineering, design, budget, and schedule control for the project. *Id*.

6.133. Applicant witness Dr. Everett L. Redmond II is a Principal Engineer and Manager of the Nuclear Physics Department at Holtec International. Dr. Redmond has a Ph.D. in Nuclear Engineering (with a minor in Biology) from the Massachusetts Institute of Technology. He is responsible for all shielding, criticality, and confinement analysis work related to Holtec's dry cask storage systems. He is the author of the shielding analyses performed in support of the general NRC certification of Holtec's HI-STORM 100 cask system under NRC Docket No. 72-1014. Dr. Redmond also performed site-specific shielding analyses in support of deployment of the HI-STORM 100 cask system at the proposed PFS facility. Dr. Redmond has significant expertise on matters pertaining to the shielding characteristics of the HI-STORM 100 cask system and the radiation dose associated with the use of this cask system. His work in those areas has included developing analytical methods and models for conducting shielding analyses and dose calculations, and performing site boundary dose evaluations for ISFSIs. Singh/Soler/ Redmond, Post Tr. 12044, at 2; *Curriculum Vitae* attached to Singh/Soler/Redmond Post Tr. 12044, at 1.

<u>Staff Witness</u>

6.134. The Staff presented one witness, Mr. Michael D. Waters, with respect to Subpart E.2 of this contention. *See* "NRC Staff Testimony of Michael D. Waters Concerning Radiological Dose Considerations Related to Unified Contention Utah L/QQ, Part E (Seismic Exemption)" (hereinafter referred to as "Waters"), Post Tr. 12215.

6.135. Mr. Waters is a Health Physicist in the Spent Fuel Project Office in the NRC Office of Nuclear Material Safety and Safeguards. He has an M.S. degree in Nuclear Engineering Sciences from the University of Florida. In his position as a Health Physicist, Mr. Waters performs technical reviews of spent nuclear fuel storage casks, ISFSIs, and transportation packages, primarily in the areas of shielding, confinement, containment, radiation protection, and criticality. He is also responsible for certain reviews initiated in his former position as a Project Engineer in SFPO, involving management of the safety reviews of applications for these designs and facilities. Waters, Post Tr. 12215, at 1; Professional Qualifications attached to Waters, Post Tr. 12215, at 1.

6.136. As part of his official responsibilities, Mr. Waters 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 Staff members. His involvement included review of the Applicant's Safety Analysis Report and participation in the Staff's preparation of its SER (September 2000) and SER Supplement No. 2 (December 2001) -- both of which were incorporated into the Staff's Consolidated SER for the PFS facility (Staff Exh. C). Waters, Post Tr. 12215, at 2. In addition, Mr. Waters assisted the Staff in preparing the Final Environmental Impact Statement ("FEIS") for the proposed PFS Facility, NUREG-1714 (December 2001), in which he reviewed general design issues associated with the proposed PFS Facility and its potential radiation impacts on the environment. *Id.*

<u>State Witness</u>

6.137. The State presented one witness, Dr. Marvin Resnikoff, with respect to Subpart E.2 of this contention. *See* "Amended State of Utah Testimony of Dr. Marvin Resnikoff Regarding Unified Contention Utah L/QQ (Seismic Exemption - Dose Exposure)" (hereinafter referred to as "Resnikoff"), Post Tr. 12349. In addition, Dr. Resnikoff presented some limited rebuttal testimony. *See* Tr. 12598-612.

6.138. Dr. Resnikoff is the Senior Associate of Radioactive Waste Management Associates ("RWMA"), a private technical consulting firm based in New York City. He holds a Ph.D. in high-energy theoretical physics from the University of Michigan. Dr. Resnikoff has researched radioactive waste issues for the past 28 years and has experience in the study of nuclear waste management, storage, and disposal. His work at RWMA includes the calculation of radiation exposures. *See* Resnikoff, Post Tr. 12349, at 1. He assisted the State in its efforts concerning dose issues raised in Contention Utah L, Part B, which has now been incorporated as Part E of Unified Contention Utah L/QQ. *Id.*

State of Utah's Testimony

6.139. Dr. Resnikoff's testimony sought to establish the lack of reasonable assurance that public health and safety will be protected if the HI-STORM 100 casks are subjected to the peak ground accelerations from a 2,000-year return period design earthquake at the PFS site. *Id*. at 3. Dr. Resnikoff's specific claims are summarized below.[125]

6.140. First, Dr. Resnikoff claimed that there are shortcomings in PFS's site-specific analysis, as stated by the State's other witnesses, such that multiple casks "will likely tipover" in a 2,000-year return period seismic event at the PFS site, *Id.* at 4, 7, 9.[126]

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[125] We note that Dr. Resnikoff, the State's expert witness on dose consequences, did not agree with the State's legal argument that § 72.104(a) should apply to a design basis earthquake. Rather, he shared the other parties' view that 10 C.F.R. § 72.106(b) applies to doses resulting from earthquake events, while § 72.104(a) applies to non-accident events. Tr. 12377, 12379, 12450.

[126] In his testimony, Dr. Resnikoff offered various conclusions related to engineering or structural issues involving the HI-STORM 100 cask and/or the PFS Facility -- based either on his reading of testimony presented by other State witnesses or on his own unsupported, "common sense" opinions. *See, e.g.,* Tr. 12384-85, 12394-98, 12402-04, 12427. We afford no weight to his testimony concerning such engineering or structural matters. Dr. Resnikoff conceded that he is not a structural engineer, mechanical engineer, nuclear engineer, or civil engineer, and he is not a licensed or professional engineer. Tr. 12443. Thus, he lacks the requisite qualifications for his personal opinions concerning such matters to be accepted as expert testimony. Further, as discussed *infra*, his reading of the other witnesses' statements was often significantly flawed.

6.141. Second, he claimed that a tipover of a HI-STORM 100 cask at the PFS site would exceed the deceleration limits of the CoC issued for the cask system. *Id.* at 4-5. He asserted that in the event of a cask tipover at the PFS site, the initial angular velocity would exceed zero (contrary to Holtec's assumption), such that the MPC's 45 g deceleration design basis will be exceeded and the damage resulting to the cask will be greater than PFS predicts, *Id.* at 7-8. In that event, he asserted that the HI-STORM 100 cask will flatten, causing the dose rate to increase. *Id*. at 8-9. He further claimed that PFS has not quantified the amount of steel stretching and concrete cracking that would result from a cask striking the ground, or whether such stretching or cracking will result in an increase in dose. *Id*. at 9.

6.142. Third, Dr. Resnikoff asserted that If the casks do not tip over in a 2,000-year earthquake, they will likely be uplifted by up to 27 inches; and he asserted that the HI-STORM CoC analysis determined that the cask could withstand a drop of only 11 inches. *Id*. at 12. He therefore concluded that the canister will experience deceleration greater than the 45 g design basis if the cask experiences a drop greater than 11 inches. *Id*. In addition, he stated that PFS's drop calculations fail to assume the cask will drop at an angle, *e.g.,* corner drop, which would result in shear stresses on the MPC inside the cask greater than those calculated by Holtec. *Id*. at 12-13.

6.143. Fourth, he asserted that PFS must address the issue of cask heat up. *Id*. at 10. In this regard, he asserted that Holtec's calculations supporting the CoC show that after 33 hours of 100% air inlet blockage, the concrete temperature will exceed the short term limit of 350° F specified in the CoC for the HI-STORM 100 cask; further, he asserted that PFS cannot upright the casks within the 33 hours, and that this would result in evaporation of the water in the concrete shielding, and an increase in neutron doses of "up to 57.3 times" greater than Holtec's calculated value of 1.88 mrem/hour at one meter from the cask. *Id*. at 11-12.

6.144. Fifth, he asserted that in the event of a cask tipover, offsite doses would increase if the cask bottoms face the controlled area boundary, because there is less shielding on the bottom of the cask. In addition, he claimed, based on his (flawed) reading of a Holtec document, that the cask lid may be displaced. *Id*. at 10.

6.145. Sixth, Dr. Resnikoff described, at some length, the "preliminary rough calculations" which he and his associate had performed, using his various assumptions, in which they concluded that a multiple cask tipover event at the PFSF would result in controlled area boundary doses of 128.71 mrem per year. *Id.* at 6, 10; State Exh. 141a at 3, 4.[127]

6.146. Based on these various assertions, Dr. Resnikoff concluded that the Applicant's dose consequence analyses are not bounding or conservative. *Id*. at 13.[128]

6.147. Finally, during the evidentiary hearing, Dr. Resnikoff asserted that accident doses should be calculated over the lifetime of the facility, rather than for a 30-day period used by the Staff and Applicant. In this regard, he asserted that accident exposures might continue for 30-50 years. Tr. 12370-71. Thus, using this approach, his calculations show that the § 72.106(b) 5-rem

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[127] In his testimony, Dr. Resnikoff referred to possible increases in doses to workers. Resnikoff, Post Tr. 12349, at 3, 9-13. The State, however, had not raised the issue of occupational doses in this contention, and such matters are therefore beyond its proper scope. Accordingly, we do not address that issue herein. *See* Tr. 12465-66 (Farrar, J.); *see also* Tr. 12448-65.

[128] Dr. Resnikoff also asserted that the Applicant's use of a 2,000-hour controlled area boundary occupancy time in its dose calculations for normal operations and anticipated occurrences is inadequate, and that an 8,760-hour occupancy time should have been used -- which would result a dose at the controlled area boundary of 25.6 mrem/year (0.6 mrem over the § 72.104(a) dose limit of 25 mrem/year for normal operations and anticipated occurrences). Resnikoff, Post Tr. 12349*,* at 5, 6-7. In this regard, he asserted that (a) the dose calculations for the HI-STORM 100 CoC had utilized an 8,760-hour occupancy time, at the request of the Staff, and (b) PFS does not control the property outside the PFS fence line, and it therefore should conservatively assume an occupancy time of 8,760 hours per year. *Id.* at 5-6.

An exposure time of 8,760 hours was utilized in calculating normal doses for the HI-STORM CoC, in accordance with Staff practice, because the CoC involved a generic approval for unknown sites, for which the site boundary distance and nearby populations were not known. *See* Tr. 12595. For this reason, the Staff requested a bounding calculation. *Id.* In contrast, for the PFS Facility, the PFS site parameters and the remote location of the nearest residence (two miles southwest of the facility) were known, and the use of a 2,000-hour residence time at the OCA boundary for dose calculations under § 72.104(a) was conservative. *See* Staff Exh. 59; Tr. 12263, 12320.

dose limit for a design basis accident would be exceeded 33 years after a multiple cask tipover event (assuming a dose rate of 150 mrem/year). Tr. 12370-71, 12380, 12602.[129]

6.148. Before proceeding to evaluate each of Dr. Resnikoff's claims, we note that our review of his testimony leads us to conclude that his opinions often lack proper foundation and/or are seriously flawed. While we admitted Dr. Resnikoff's testimony on dose consequences, we note that he described the analyses which he presented for our consideration as "preliminary rough calculations." Indeed, he conceded that his dose calculations often lacked sound foundation -- and on cross-examination, they were shown to be flawed and unreliable. *See, e.g.,* Tr. 12427, 12415-27,12429-32, 12446-47,12477-78, 12481-83, 12486-87, 12493-94, 12496-97, 12499-501, 12502-03. Further, on cross-examination, he testified that he has never conducted a Monte Carlo analysis -- although he conceded that this is the preferred technique for estimating direct radiation doses from neutron and gamma radiation, which are at issue here. Tr. 12443-44, 12446, 12618, 12628. Similarly, he has had no prior experience in calculating direct radiation doses over long distances (on the order of 600 meters) such as are involved here. Tr. 12447. Thus, while we address his various claims below, it is our conclusion that very little, if any, weight should be given to his testimony on dose matters.

6.149. With respect to Dr. Resnikoff's repeated claim that the HI-STORM 100 casks at the PFSF <u>will likely</u> tipover, based on his reading of other State witnesses' testimony, a careful reading of those witnesses' testimony reveals that they did not reach the conclusion ascribed to them by

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[129] In support of his view that a period of many years should be used in calculating the accident dose, Dr. Resnikoff cited a Protective Action Guide ("PAG") published by the Environmental Protection Agency, concerning the "intermediate phase" following an accident in which <u>deposited</u> radioactive materials are present. Tr. 12602; State Exh. 218. As discussed *infra*, this PAG is irrelevant here, where only direct radiation is at issue, and there is no release or deposition of radioactive materials.

Dr. Resnikoff. Rather, their testimony, and even the Altran report, stated only that there is a potential for cask tipover to occur.[130] *See, e.g.*, Tr. 12384-85, 12472-73.

6.150. With respect to Dr. Resnikoff's assertion that a tipover of a HI-STORM 100 cask at the PFS site would exceed the deceleration limits of the CoC issued for the cask system, in that the initial angular velocity would exceed zero, we find that he lacks the requisite expertise or familiarity with the PFS site-specific ground motions to render him qualified to provide an opinion on this subject. Further, Dr. Resnikoff conceded under cross examination that while the 45 g design basis deceleration was a value stated in the CoC, damage would occur not at the 45 g deceleration (as stated in his original testimony), but at 63 g. Tr. 12409-10. Moreover, he admitted that he had not calculated the initial angular velocity that would be required to cause deceleration of the fuel assemblies at 63 g, Tr. 12411; and he did not know what the initial angular velocity should be. Tr. 12406.

6.151. Similarly, Dr. Resnikoff lacks the requisite expertise and knowledge to conclude that a HI-STORM cask would be uplifted by 27 inches in the event of a 2,000-year return period earthquake at the PFS site; and he lacks the requisite expertise and knowledge to proffer an expert opinion regarding the structural consequences of a cask drop event. Accordingly, we afford no weight to his testimony on such matters. *See* note 126, *supra*.

6.152. Likewise, with respect to his assertion that cask tipover would cause the HI-STORM 100 cask to flatten, causing the dose rate to increase, Dr. Resnikoff lacks the requisite qualifications to offer an expert opinion on such matters. Moreover, he admitted that he did not know how much flattening or subsequent damage to the cask would occur. Tr. 12406. In addition, he admitted that he had not estimated the radiological consequences of cask flattening or

[130] Dr. Resnikoff did not independently determine whether the casks would tip over, but assumed cask tipover as the starting point for his analysis. Resnikoff, Post Tr. 12349, at 7; Tr. 12381-396, 12522, 12532-33.

deformation, concrete cracking, or steel thinning or stretching. Tr. 12407-08. He also did not know whether PFS had analyzed the effect of out-of-roundness of the cask on radiation shielding. Tr. 12408-09.

6.153. With respect to Dr. Resnikoff's assertions concerning the potential for cask heat up, which he believed may lead to evaporation of the water in the concrete shielding and a factor of 57 increase in neutron doses one meter from the cask above Holtec's calculation, again we find that he lacks the requisite expertise to address such structural engineering issues. Moreover, his dose calculations regarding this matter were shown to lack proper foundation. Thus, Dr. Resnikoff assumed that cask heating would result in all water being lost from the concrete shielding, thus resulting in increased neutron doses. Resnikoff, Post Tr. 12349, at 12. However, on cross-examination he admitted that he could not say whether it is physically possible to lose all hydrogen shielding from the concrete, and he did not know if all the hydrogen can be boiled off. Tr. 12421-22. Further, while his dose calculations did not take credit for hydrogen shielding of neutrons, he admitted that the hydrogen may not have escaped from the cask, Tr. 12420; that he did not try to model how much hydrogen would actually be present, Tr. 12421; and that his calculation did not take credit for hydrogen in the aggregate in the concrete. Tr. 12423.

6.154. With respect to Dr. Resnikoff's claim that in the event of a cask tipover, offsite doses would increase if the cask bottoms face the controlled area boundary, Dr. Resnikoff primarily relied upon his understanding of the HI-STORM 100 cask design and his "preliminary rough" dose calculations. We therefore turn to consider those calculations.

6.155. First, Dr. Resnikoff's understanding of the HI-STORM design, and his representation of that design in his calculations, was shown to be significantly flawed. For example, on cross-examination, it became apparent that he had used the wrong values to represent the amount of steel and concrete shielding provided at the bottom of the HI-STORM cask. Specifically, Dr. Resnikoff indicated that the steel base plate of the HI-STORM 100 cask was three inches thick

(State Exh. 140), but it is actually two inches thick. Tr. 12432. In addition, he indicated that his calculations accounted for shielding provided by 5 inches of steel in the pedestal of the HI-STORM 100 cask system through consideration of steel in the base of the MPC (Tr. 12485-86); however, the MPC's 2.5 inch steel base is in addition to the 5 inches of steel on the pedestal. Tr. 12486; *see* Tr. 12164; Staff Exh. V.

6.156. Second, wholly apart from using incorrect structural input values, Dr. Resnikoff's dose calculations were shown to be significantly flawed.[131] In his "preliminary rough calculations," Dr. Resnikoff analyzed a tipover accident in which the bottoms of a row of casks face the site boundary. Resnikoff, Post Tr. 12349 at 10; Tr. 12353-54. This analysis accounted for 80 casks tipped over, with each cask bottom facing the site boundary. State Exh. 141a, at 1, 4. Further, these calculations included his assumption that cask heating would cause all water to be lost from the concrete shielding, thus resulting in increased neutron doses. *Id.* at 4; Tr. 12420-23. Dr. Resnikoff testified that his calculations showed the dose rate will increase by 5 times that calculated by PFS at the site boundary, assuming an 8,760-hour occupancy time per year -- but that the dose may also be ½ of the dose calculated by PFS Resnikoff, Post Tr. 12349, at 10. According to Dr. Resnikoff's calculations, the total dose rate at the site boundary would be 128.71 mrem/year. State Exh. 141a at 4. At the hearing, however, Dr. Resnikoff stated that this was an underestimate, and the total dose rate would be approximately 150 mrem/year. Tr. 12360.

6.157. These calculated doses were shown to be unreliable. With respect to cask orientation, while his analysis assumed that 80 cask bottoms would face the controlled area boundary, Dr. Resnikoff admitted that if earthquake accelerations were high, the casks would be

[131] We note that Dr. Resnikoff corrected the dose calculations set forth in his pre-filed written testimony in response to various errors identified by the Applicant. *See* State Exhibit 141a, at 1; Tr. 12355-56. However, during cross-examination, additional errors in his calculations beyond those accounted for in his earlier corrections, became apparent. *See, e.g.*, Tr. 12428, 12618, 12623-26, 12540, 12542-44.

oriented in various different directions. Tr. 12428. Further, he admitted that his calculations overestimated the dose in that he did not account for decay, Tr. 12618. In this regard, he calculated the initial gamma dose to be 117 mrem/year, or approximately 78% of the total dose (State Exh. 141a at 3, 4); however, inasmuch as 10 percent of the gamma dose is from Cesium-137 and 90 percent is from Cobalt-60, there would be a very substantial reduction over time if decay were taken into account. Tr. 12623-26.

6.158. Dr. Resnikoff also admitted under cross-examination that he based his calculations on a 1.88 mrem dose rate at one meter from the cask -- but in doing so, he used the dose rate provided by Holtec for a point adjacent to the cask. Resnikoff, Post Tr. 12349, at 12; Tr. 12540. He then claimed that he had also performed the calculation based on dose rates one meter from the cask, and found no difference in the result as compared to the calculation based on the dose rate at the cask surface, set forth in State Exhibit 141a. Tr. 12542-44. This assertion defies reality and lacks credibility: Indeed, Dr. Resnikoff conceded that it would be "inconsistent" to use the surface dose rate to represent the dose rate at one meter. Tr. 12502-03, 12545. Subsequently, Dr. Resnikoff confirmed that if he had erroneously used the dose rate at the surface as if it were the rate one meter from the surface, then his results would require correction by a factor of 0.78/1.88 (approximately 0.41). Tr. 12607-08; 12559.

6.159. Finally, with respect to Dr. Resnikoff's stated view that some period of time (on the order of many years) should be used to calculate accident doses, we find that opinion to be lacking in support. Dr. Resnikoff testified that he did not know the duration of the postulated cask tipover event -- *i.e.*, whether it is two years, 20 years, 30 or 40 years, or the life of the facility. Tr. 12508. He further stated that he had not constructed scenarios to estimate the length of time it would take to right the casks. Tr. 12509. In addition, he acknowledged that measures could be taken to minimize radiation doses, if necessary. Tr. 12623.[132]

Applicant's Testimony

6.160. The Applicant's witnesses addressed the various points raised by the State in this portion of the contention. Most significantly, they testified that, even assuming the maximum 4,000 casks were to tip over during a postulated beyond design basis earthquake, the limits at the site boundary will be far below the 5 rem limit found in 10 C.F.R. § 72.106(b), and, in fact, will remain essentially unchanged regardless of whether one assumes that a single cask, any number of them, or all of the casks tip over. Singh/Soler/Redmond, Post Tr. 12044, at 5, 7-8, 10-11.

6.161. Further, with respect to the State's testimony, the Applicant's witnesses testified that differences between the Holtec CoC and the PFSF do not affect the validity of plant-specific analyses performed by Holtec for the PFSF. *Id*. at 13-14.

6.162. With respect to the State's claim that initial angular momentum will be greater than zero, Drs. Singh and Soler testified that a HI-STORM storage cask does not tip over even under a 10,000 year return period earthquake. *Id.* at 17. They explained that the cask experiences an oscillatory rocking motion with limited return to the vertical position until the rocking finally ends when the earthquake subsides. *Id*. Based on their observations, they concluded that, if the strength of the seismic event were increased to the point where the cask did tip over, the initiating angular velocity propelling the cask towards the ground is quite small. *Id*. They stated further that the precessionary motion of the cask enables it to remain stable even while the center of gravity

[132] In addition, as noted above, the Environmental Protection Agency PAG cited by Dr. Resnikoff is inapplicable here. That PAG specifically applies to an event in which there has been a release of radiological materials, with subsequent deposition on the ground in the vicinity of the nuclear facility. *See* State Exh. 218. Where such deposited radioactive materials are present, it is conceivable that remedial measures to clean up and dispose of the contaminated soil may take some lengthy period of time. However, that situation is inapplicable to the accident postulated here, which involves only direct radiation and no deposition of radioactive materials.

of the cask is well past the corner. *Id*. Drs. Singh and Soler stated that as a result of the precessionary motion, the initial height of the cask center of gravity is apt to be much lower than the static tipover scenario (where tipover begins as soon as the center of gravity crosses the vertical plane containing the axis of overturning rotation). *Id*. They reasoned that, with less distance to fall, and a negligible initial angular velocity propelling the tip over, a cask tipping away from precessionary motion is expected to have substantially less kinetic energy of collision than one tipping from zero velocity with center of gravity of over corner. *Id*. They concluded, therefore, that their assumption of zero initial angular velocity at the point at which the "center of gravity over corner" is exceeded, is reasonable. *Id*. at 18.

6.163. With respect to the maximum deceleration experienced by the MPC in a 25-foot drop, Dr. Singh testified that such deceleration was about 300 g. Tr. 12075. He testified that he believed the confinement barrier, *i.e.*, the MPC, would remain intact if subjected to such a drop. Tr. 12075-76. He testified further that the scenario presented by the State, namely, a 27-inch drop due to seismic lifting of the pads, is entirely unrealistic. Tr. 12077.

6.164. With respect to evaporation of water out of concrete, Dr. Singh testified that for large, extensive, sustained water evaporation to occur, exposure to high temperatures on the order of 600° F or greater would be necessary for a period of months. Singh/Soler/Redmond, Post Tr. 12044, at 27. Further, a tipped cask would not attain this range of temperatures, even if such a condition is assumed to persist for a long time with a bounding assumption that one air vent at both the top and bottom of cask were blocked. *Id*. Also, even assuming all vents were blocked as claimed by Dr. Resnikoff, the bounding steady state temperature for the concrete would be well below the 600 °F necessary for extensive sustained water evaporation. *Id*. Dr. Singh concluded, therefore, that evaporation of water from the concrete of a tipped over cask would be minimal even if the cask remained in a tipover position for a period of months. *Id*.

6.165. With respect to potential cask damage resulting from tipover, Drs. Soler and Redmond testified that tipover of the cask would result in only localized, limited damage to the cask and canister and would not result in adverse radiological consequences at the PFSF. *Id*. at 15-16. Further, sliding or other impacts of casks would not threaten the confinement function of the MPC, and there would be no release of radioactivity. *Id*. at 29-31.

6.166. Dr. Redmond further described an evaluation which Holtec performed concerning the potential dose consequences resulting from a multiple cask tipover event at the proposed PFSF. In this regard, Holtec qualitatively reviewed the effect that a multiple cask tipover event would have on radiation doses at the site boundary, as compared to the normal dose rate of about 5.85 mrem/year at the site boundary. Holtec determined that the dose consequences at the site boundary from a multiple cask tipover event would be similar to or less than the normal doses, and far below the 5 rem accident dose limit of 10 C.F.R. § 72.106(b). Because of the large margin between the normal doses calculated for the PFSF and the accident dose limit, Holtec determined there is no need to perform further calculations of the dose consequences of a multiple cask tipover event. Singh/Soler/Redmond, Post Tr. 12044, at 7-8.

6.167. With respect to normal operational doses at the PFSF, Holtec performed an analysis in which it determined the direct radiation dose rate at the controlled area boundary from neutron and gamma (photon) radiation emanating from the sides and top of the HI-STORM storage casks. The analysis considered the maximum PFSF capacity of 4,000 casks. The calculations were performed with the Monte Carlo radiation transport code MCNP-4A. The results of this calculation show a maximum dose rate of 5.85 mrem/year for a 2000 hour/year occupancy time at the controlled area boundary, assuming all casks contained fuel with a burnup of 40,000 MWD/MTU and a cooling time of 10 years. These analyses demonstrated that the doses at the boundary are well within the limits deemed acceptable by the NRC in 10 C.F.R. § 72.104(a) and 10 C.F.R. § 72.106(b) for both normal operations and accident conditions. *Id*. at 8.

6.168. Dr. Redmond compared the doses calculated for normal operation to those expected for tipped over casks. In the upright position, the side of the HI-STORM 100 storage cask is visible from all equidistant locations from the cask, while the top is not visible from any location. Therefore, all equidistant locations from an upright HI-STORM 100 storage cask will have the same dose rates. However, in a tipped over position, the profile of the cask would be considerably different from its upright position, in that the side, top and bottom of the cask would be visible from some locations but not others, and the dose rate profile around a tipped over cask would not be uniform at equidistant locations from the cask. *Id.* at 8-9.

6.169. Accordingly, the comparison must take into account the following changes in the dose rate profile of the cask:

> a. The top of the cask would be visible although no longer facing the sky. Therefore, the radiation leaving the top of the cask would reach certain locations at the controlled area boundary directly (with due consideration of any attenuation and scattering in the intervening air), as opposed to the strictly scattering effect of sky shine. This would be an increase in the dose rate contribution from the top of the cask. However, at the locations along the controlled area boundary where the top of the cask is now easily visible, the dose rate from the side of the storage cask would be greatly reduced because the line-of-sight to the side of the cask would be reduced.
>
> b. The bottom of the cask, which is normally facing the concrete ISFSI pad and the ground below, would now be exposed. Therefore, radiation emanating from the bottom of the storage cask, which previously was immediately absorbed by the ground, could now reach locations along the controlled boundary directly, with due consideration of attenuation and scattering provided by the intervening air. This would also cause an increase in the dose rate contribution from the bottom of the cask. However, at the locations along the controlled area boundary where the bottom of the cask was now easily visible, the dose rate from the side of the cask would be greatly reduced because the line-of-sight to the side of the cask was reduced.
>
> c. Since the storage cask would now be lying on its side, a large portion of the outer radial surface of the cask would be shielded by the ground. In the upright position, all radiation that emanated off the side of the cask was able to scatter and reach the site boundary. In the tipped over position, a significant portion of the

> radiation leaving the side of the cask would now be unable to reach the site boundary because it would be immediately absorbed by the ground below the side of the cask. In addition, as discussed above, not all locations on the controlled area boundary would have line-of-sight to the side of the cask. This would result in a reduction in the dose rate at the controlled area boundary from radiation emanating off the side of the cask.

Id. at 9.

6.170. Overall, Dr. Redmond concluded that the decrease in dose rate from the side of the tipped over storage cask should more than compensate for the increase in dose rate from the top or bottom of the cask. He therefore concluded that the dose rate at the controlled area boundary from a HI-STORM storage cask lying on its side would be less than the dose rate from a HI-STORM storage cask in the upright position. *Id.* at 10.

6.171. Dr. Redmond also testified as to the likely orientation of the casks in the event of a tipover accident. The storage casks at the PFSF ISFSI are positioned in fifty 2x40 arrays that are parallel to each other with a spacing of 35 feet between arrays. Because of the positioning of the casks, it is improbable that all 4,000 casks could ever completely tip over and come to rest on their sides on the ground. Rather, a more plausible scenario would have some casks lying on the ground while the remainder would be upright in one of two positions: free standing, or leaning against other storage casks. *Id*.

6.172. Dr. Redmond further concluded that tipover of all 4,000 casks would not change the calculated radiation dose limits. In the event of a tipover of all 4,000 casks, the decrease in dose rate from the side of the tipped over storage cask should more than compensate for the increase in dose rate from the top or bottom of the cask. In the upright position at the ISFSI, the sides of the cask are partially shielded by the position of casks next to each other. Similarly, this self-shielding would still exist to a degree if all casks are tipped over because they would be lying next to each other. Therefore, based on the response for a single cask, it was Dr. Redmond's opinion that the dose rate from the entire 4,000 casks at PFSF lying on their sides would be similar to that from the ISFSI with all casks in the upright position. *Id*. at 10-11.

6.173. Dr. Redmond also compared the normal dose rate at the site boundary calculated for 4,000 casks in their upright position, which as stated above is 5.85 mrem/year, to the accident dose limit in 10 C.F.R. § 72.106(b). Based on the above analysis, the expected dose rate for 4,000 tipped over casks at the site boundary would be of the same order of magnitude as the normal dose rate. Thus, there is approximately three orders of magnitude of margin between the expected dose rate at the site boundary for 4,000 casks in a tipped over condition compared to the 5 rem accident dose limit in 10 C.F.R. § 72.106(b). *Id*. at 11.

6.174. Dr. Redmond described the following significant conservatisms in the analyses for normal doses as being equally applicable to casks in a tipped over condition:

> (1) Holtec assumed that all 4,000 casks have the exact same burnup and cooling time. This is impossible, since the MPCs will be delivered over many years and each additional year of cooling further reduces the radiation source term. As an example, if the PFSF received 4 casks per week, 50 weeks per year, it would take 20 years to completely fill the ISFSI. This means that at the completion of the ISFSI, the first casks delivered will have an additional 15 years of cooling time compared to the last casks delivered.
>
> (2) Holtec used a conservative burnup of 40,000 MWD/MTU and a cooling time of 10 years in its analysis. In a separate analysis, Stone & Webster used a more realistic value of 35,000 MWD/MTU and a cooling time of 20 years, resulting in a reduction of more than 50% in the calculated normal doses at the site boundary, from 5.85 mrem/year to 2.10 mrem/year.
>
> (3) The analyses use a single design basis fuel assembly, which has the highest gamma and neutron radiation source term in all fuel storage locations.
>
> (4) The analyses use a single irradiation cycle to calculate the source term. This does not recognize the down time during reactor operations for scheduled maintenance and refueling. This additional down time would reduce the source term by effectively increasing the cooling time.

Id. at 11-12.

6.175. Dr. Redmond disputed Dr. Resnikoff's argument that one should consider a hypothetical individual located at the site boundary for the entire year. Specifically, Dr. Redmond pointed out that § 72.104(a) requires calculation of the dose for a "real" individual, not a hypothetical individual as claimed by Dr. Resnikoff. The regulatory guidance provided in the NUREG-1567, § 11.5.3.2, and Interim Staff Guidance 13 ("ISG 13"), Rev. 0 also provides for use of a "real individual" for calculating radiation doses as opposed to Dr. Resnikoff's hypothetical individual.[133]

6.176. In addition, even if it is assumed that a hypothetical individual is located at the site boundary an entire year (8,760 hours) as Dr. Resnikoff argues is appropriate, this would not affect Dr. Redmond's conclusion that the radiological dose at the site boundary would be far less than the accident dose limit of 5 rem in 10 C.F.R. § 72.106(b). Rather, that assumption would reduce the margin of conservatism by somewhat less than an order of magnitude, to a margin of conservatism that is still more than two orders of magnitude. Thus, the dose consequences at the site boundary would continue to be far below the 5 rem accident dose limit of 10 C.F.R. § 72.106(b). *Id*. at 12-13.

6.177. Based on the responses above for a single cask and 4,000 casks, and the other conservative assumptions used in the analyses as documented in the PFSF SAR, it was

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[133] A "real individual" is defined in Interim Staff Guidance Memorandum No. 13 ("ISG 13"), Rev. 0" (undated) (PFS Exh. 239), as follows:

> The *real individual* is an individual at or beyond the controlled area, and dose to any real individual must not exceed the limits specified in 72.104 from both the storage facility and other surrounding fuel cycle activities. For example a real individual may be anyone living, working, or recreating close to the facility for a significant portion of the year.

PFS Exh. 239, Attachment at [unnumbered] 1.

Dr. Redmond's opinion that regardless whether the HI STORM storage casks are assumed to remain upright in a severe earthquake or tip over, the radiation dose at the site boundary will remain essentially unchanged regardless of whether one assumes that a single cask, any number of them, or all the casks, tip over. In either case, the dose at the boundary is far below the accident limits of 10 C.F.R. § 72.106(b). *Id*. at 13.

6.178. Finally, Dr. Redmond testified that the radiological dose calculations performed by Dr. Resnikoff are based on flawed methodologies and assumptions, and contain numerous mistakes. *See Id*. at 22-24.[134]

Staff's Testimony

6.179. The NRC Staff's testimony of Mr. Waters provided an independent assessment as to whether the PFS Facility design will provide adequate protection against exceeding the applicable dose limits set forth in the Commission's regulations in the event of an earthquake. As discussed below, the Staff's testimony provides firm support for the conclusion that the cask tipover accident postulated by the State, if it occurs, would have to meet the limits in 10 C.F.R. § 72.106(b). Further, based on his independent calculations, Mr. Waters' testimony establishes that doses from a postulated multiple cask tipover event at the PFS site would not result in dose consequences in excess of the limits in § 72.106(b).

6.180. Mr. Waters' testimony described the Staff's review of whether the proposed PFSF would satisfy the Commission's requirements in the event of a design-basis earthquake, as follows. 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

[134] Dr. Redmond also provided testimony concerning the limits on neutron doses to workers. *Id*. at 24-25. As stated above, such matters are outside the proper scope of this contention, and we therefore do not address that issue in this decision. *See* n.127, *supra*.

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 to 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. Waters, Post Tr. 12215, at 4.

6.181. 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 (Staff Exh. C). *Id*.

6.182. In its evaluation, the Staff did not 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). 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. *Id*. at 4-5.

6.183. The Staff concluded that the State's assertion, in Unified Contention Utah L/QQ, Subpart E.2, that PFS should be required either to use a probabilistic 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," is not a valid concern. *Id*. at 5. The bases for this conclusion were stated as follows.

6.184. 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. *Id*.

6.185. 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 in the event of such a failure. However, as discussed above with respect to Subpart E.1 of the contention, 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 two-tiered approach proposed in SECY-98-126, with its reference to the dose limits in § 72.104(a) is 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). *Id*. at 5-6.

6.186. Further, the Staff 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. *Id*. at 6. In this regard, the Staff has determined that 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. *See* Staff Exh. C, §§ 15.1.2.6, 15.2, at 15-29 to 15-32 and 15-122. 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. Waters, Post Tr. 12215, at 6.

6.187. 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 (Staff Exh. C), 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. *Id*. at 6-7.

6.188. Further, 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. *Id*. at 7.

6.189. 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, as stated in Section 8.2.6 of the PFS SAR. 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. *Id*.

6.190. As set forth in § 5.1.1.4 of the Consolidated SER (Staff Exh. C), 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 § 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. *Id*. at 7-8.

6.191. With regard to beyond-design basis seismic events, it should be noted that such 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 tip over. As set forth in the testimony of Dr. Goodluck I. Ofoegbu and Daniel J. Pomerening, and in the Staff's testimony of Jack Guttmann and Dr. Vincent Luk, the Staff has concluded that the storage casks would not tip over even in the event of a 10,000-year return period earthquake at the proposed PFS Facility. *Id*. at 8.

6.192. 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, Mr. Waters analyzed the potential offsite dose consequences that might result from a hypothetical multiple cask tipover event, if it were to occur at the facility. *Id.* Mr. Waters' dose consequence analysis pertaining to this hypothetical multiple cask tipover event is described below.

6.193. In Mr. Waters' analysis, he 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 spatial 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). *Id*. at 9.

6.194. In his analysis, Mr. Waters 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. He testified further as follows: 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 V) and Figure 5.3.11 of the FSAR (Staff Exhibit W). 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. *Id*. at 9-10.

6.195. 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. The Staff considered 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. *Id*. at 10.

6.196. 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. *Id*.

6.197. 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. *Id*. at 10-11.

6.198. 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. *Id*. at 11.

6.199. 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. *Id*.

6.200. Mr. Waters also described the Staff's evaluation of hypothetical loss of hydrogen from the concrete shield due to thermal effects, as set forth below.

6.201. As presented in § 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. *Id*. at 11-12.

6.202. 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. *Id*. at 12.

6.203. 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. *Id*.

6.204. 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 1.37, 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 tip over 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%] + [1.37 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. *Id.* at 12-13.

6.205. By comparison, the Staff 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 calculated by the Applicant by a factor of approximately 2,400 (*i.e.*, 6.94 mrem/hr ÷ 0.00293 mrem/hr = 2,369). *Id*. at 13.

6.206. 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. *Id*.

6.207. Mr. Waters further described the Staff's evaluation with respect to spatial reorientation of the casks from a vertical to a tilted or horizontal position, as set forth below.

6.208. In the Staff's analysis, the Staff considered the extent to which dose rates might increase as a result of spatial reorientation of the casks, from a vertical to a tilted or horizontal position. In this regard, as discussed above, the Staff 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. *Id*. at 14.

6.209. 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 X), 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. *Id*.

6.210. 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). *Id*. at 14-15.

6.211. This conclusion is further supported by the Staff's analysis 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). *Id*. at 15.

6.212. 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. *Id*.

6.213. 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. *Id*. at 15-16.

6.214. 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, this value is 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. *Id*. at 16.

6.215. 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. *Id*.

6.216. The Staff determined that 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 <u>tops</u> facing off-site in the north direction, the off-site dose rates could <u>decrease</u> approximately by a factor of 3.1 (*i.e.*, 1 ÷ {[(0.25 mrem/hr ÷ 4.59 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 <u>bottoms</u> facing off-site in the north direction, the off-site dose rates could, conservatively speaking, <u>increase</u> 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). *Id*. at 16-17.

6.217. 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 3.1, 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, this result would not be substantially different if all 4,000 casks tip over, 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. *Id*. at 17.

6.218. 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. *Id*.

6.219. Mr. Waters testified that 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. He concluded, therefore, that the (design basis) accident dose limit of 5 rem in 10 C.F.R. § 72.106(b) would not be exceeded. *Id*. at 18.

6.220. Finally, Mr. Waters provided oral direct testimony, in which he summarized the regulatory guidance followed by the NRC Staff (and the Commission) with regard to the dose standard applicable to design basis earthquake events. *See* Tr. 12216-38. Although we have resolved this question above, as a matter of law, we find that Mr. Waters' description of the agency's established regulatory practice provides useful insight into this matter and demonstrates the correctness of our legal conclusion that the Applicant's request for exemption is to be evaluated with respect to the accident dose limit stated in 10 C.F.R. § 72.106(b).

6.221. First, we note that Regulatory Guide ("Reg. Guide") 3.60, "Design of an Independent Spent Fuel Storage Installation (Dry Storage)" (March 1987) (Staff Exh. 55) states that ANSI/ANS 57.9-1984 is acceptable to the NRC staff for use in the design of an ISFSI that uses a dry environment as the mode of storage (subject to certain conditions, none of which are relevant here). Staff Exh. 55 at 1.

6.222. We have examined ANSI/ANS 57.9-1984, "American National Standard Design Criteria for an Independent Spent Fuel Storage Installation (Dry Storage Type)" (Dec. 31, 1984) (Staff Exh. 56), and compared it to ANSI/ANS 57.9-1992 (Staff Exh. 57), which is a subsequent revision of the standard. We find that these two standards are substantively identical, and, for clarity, we will refer to the 1992 revision.

6.223. The ANSI/ANS 57.9-1992 standard defines four conditions, namely Design Events I, II, III, and IV, which are "occurrences that need to be considered in system and installation design." Staff Exh. 57 at 2. Design Event I consists of that set of events that are expected to occur regularly or frequently in the course of normal operation of the ISFSI. *Id*. Design Event II consists of that set of events that, although not occurring regularly, can be expected to occur with moderate frequency or on the order of once during a calendar year of ISFSI operation. *Id*. Design Event III consists of that set of infrequent events that could reasonably be expected to occur during the lifetime of the ISFSI. *Id*. at 3.

6.224. In particular, ANSI/ANS 57.9-1992 states that Design Event IV:

> consists of the events that are postulated because their consequences may result in the maximum potential impact on the immediate environs. Their consideration establishes a conservative design basis for certain systems that are important to confinement. Typically, this set of events consists of plant-specific design events as defined in Design Phenomena.

Id. Finally, the ANSI standard defines "Design Phenomena" as "[t]hose natural phenomena . . . for which the ISFSI is designed." *Id*. Applying these definitions to the Applicant's exemption request, it is readily apparent that a design basis earthquake, by definition, is a matter that is properly classified as Design Event IV. This is true regardless of the specific return period or ground motion magnitude that is ultimately found to be appropriate to establish the DBE.[135]

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[135] Alternatively, looking at the specific design basis earthquake proposed here, it is apparent that a DBE with a 2,000 year return period is not expected to occur "regularly or frequently in the course of normal operation of the ISFSI" (Design Event I) or "with moderate frequency or on the order of once during a calendar year of ISFSI operation" (Design Event II); and it could not "reasonably be expected to occur during the lifetime of the ISFSI" (Design Event III). Therefore, a design basis earthquake with a 2,000 year return period can only be a Design Event IV under the ANSI/ANS 57.9-1992 standard.

6.225. Further, the Staff uses the guidance in NUREG-1567, "Standard Review Plan for Spent Fuel Dry Storage Facilities" (March 2000) (Staff Exh. 53) in evaluating the adequacy of ISFSI applications. Tr. 12058, 12212, 12218. NUREG-1567 refers to normal, off-normal, or accident conditions. Staff Exh. 53 at 9-14, 9-15.[136] More specifically, NUREG-1576 defines off-normal and accident events as follows:

> Off-normal events are those expected to occur with moderate frequency or once per calendar year. ANSI/ANS 57.9 refers to these events as Design Event II.
>
> Accident events are considered to occur infrequently, if ever, during the lifetime of the facility. ANSI/ANS 57.9 subdivides this class of accidents into Design Event III, a set of infrequent events that could be expected to occur during the lifetime of the ISFSI, and Design Event IV, events that are postulated because they establish a conservative design basis for SSCs important to safety. For purposes of this chapter of the Facilities Standard Review Plan (FSRP), no distinction is made between these two classes of events. <u>The effects of natural phenomena, such as earthquakes, tornadoes, hurricanes, floods, tsunami, and seiches, are considered to be accident events</u>.

Staff Exh. 53, at 15-1 (emphasis added).

6.226. In addition, NUREG-1536, "Standard Review Plan for Dry Cask Storage Systems" (Jan. 1997) (Staff Exh. 58), summarizes the guidance to the Staff with respect to the definitions of "design-basis" events (Staff Exh. 58, at xi) and "normal" and "off-normal" conditions (*id*. at xiii), and the dose limits for such events and conditions (*id*. at 11-2). This guidance similarly establishes that design basis earthquakes are to be considered under the accident dose limits in 10 C.F.R. § 72.106(b). It states, in pertinent part, as follows (*id*.):

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[136] To be sure, NUREG-1567 does not refer to direct radiation, but rather refers only to doses resulting from loss of confinement. Tr. 12329. However, this distinction is irrelevant to determining the proper classification of a DBE, because the Commission's regulations in Part 72 concerning dose limits make no distinction between direct radiation and radiation resulting from the loss of confinement. Moreover, NUREG-1567 is the only guidance on the subject, in that no guidance exists for the evaluation of dose consequences resulting from direct radiation. Tr. 12218.

> [A]nticipated occurrences (off-normal conditions) are distinguished, in part, from accidents or natural phenomena by the appropriate regulatory guidance and design criteria. For example, the radiation dose from an off-normal event must not exceed the limits specified in 10 CFR Part 20 and 10 CFR 72.104(a), whereas the radiation dose from an accident or natural phenomenon must not exceed the specifications of 10 CFR 72.106(b). . . . .

6.227. In sum, the regulatory guidance published by the Commission, and routinely followed by this agency in its review of applications under 10 C.F.R. Part 72, clearly establishes that the dose consequences of design basis earthquakes are to be evaluated with respect to whether they meet the 5 rem dose limit provided in 10 C.F.R. § 72.106(b). With this in mind, we now turn to consider the totality of the evidence.

Summary of Findings Regarding Subpart E.1 of Unified Contention Utah L/QQ

6.228. Based on our consideration of all of the evidence, we have reached the following factual conclusions regarding the matters raised in Subpart E.2 of Unified Contention Utah L/QQ, in addition to the specific findings set forth above.

6.229. First, we are satisfied that 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.

6.230. Second, we have considered the various dose calculations performed by the parties. In particular, we are mindful of the Staff's detailed analysis of potential dose consequences in the event of a multiple hypothetical cask tipover event at the proposed PFS Facility, in which the Staff, using sophisticated Monte Carlo methodology, considered potential damage to the cask shield, potential thermal degradation, and the potential effect on offsite dose rates caused by spatial reorientation of the casks. In view of these calculations, we find that the design basis accident offsite dose limit of 5 rem could only be exceeded only if the off-site dose rate at the OCA boundary increases to approximately 6.94 mrem/hr (which corresponds to an increase above the maximum normal off-site dose rate by a factor of about 2,400). Such a dose rate is far greater than any party, including the State, has predicted to result from a multiple cask tipover event at the proposed PFS Facility.

6.231. Third, we are satisfied that in the event of a beyond-design basis hypothetical tipover event involving 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. In addition, assuming that all 4,000 casks tip over and experience hydrogen loss (via thermal degradation) in the radial shield, the off-site dose rates could increase by a factor of approximately 2.4 -- which is far less than the factor of 2,400 required to exceed the 5 rem dose limit established in § 72.106(b).

6.232. In sum, even if all 4,000 casks tip over, 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.

6.233. Further, we find that Dr. Resnikoff's analysis, as described above, provides an unreliable, overly large estimate of dose consequences in that, *inter alia*, it: (a) does not account for decay (*see* ¶ 6.157, *supra*); (b) is erroneously based on dose rates at the surface of the cask, rather than one meter, and thus require correction by a factor of 0.41 (0.78/1.88) (*see* ¶ 6.158, *supra*); (c) does not take credit for any shielding of neutrons by hydrogen (*see* ¶¶ 6.153, 6.156, *supra*); and (d) does not account for varying cask orientation (*see* ¶ 6.157, *supra*). We conclude, therefore, that Dr. Resnikoff's analysis, on its own terms, is not useful to determine whether doses resulting from a DBE or cask tipover accident at the proposed PFSF will satisfy the dose limits of § 72.106(b).

6.234. We are also cognizant of Dr. Resnikoff's view that accident doses should be calculated based on an occupancy time of 8,760 hours per year. Indeed, his accident dose calculations utilize that occupancy time, which leads in part to his calculation of an OCA boundary accident dose of from "½ to 5" times the dose from normal operations, or up to 150 mrem/year. We find this to be an unsupportable assumption.

6.235. In this regard, we note that Commission regulatory guidance provides for the use of a 30-day occupancy time in accident dose calculations.[137] Thus, in contrast to Dr. Resnikoff's assumption, the Staff's accident dose calculations under 10 C.F.R. § 72.106(b) use a residence time at the OCA boundary of 720 hours (30 days). Waters, Post Tr. 12215, at 13.[138] This is appropriate, as set forth in NUREG-1567, in that protective actions could be taken to assure that any persons located near the OCA boundary are evacuated or otherwise protected in the event that the Applicant is unable to upright all of the casks that might be involved in a hypothetical tipover event within 30 days.[139] In the event of an accident, there would be immediate dose mitigation, followed by actions to place the facility in a safe condition. Tr. 12327. Thereafter, actions would be taken to return the facility to compliance. Tr. 12328. The use of a 30 day occupancy period in accident dose calculations affords sufficient time to apply fundamental principles of radiological dose protection, such as time, distance, and shielding. Tr. 12266-67. In addition, direct radiation

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[137] As set forth in NUREG-1567 (Staff Exh. 53):

> A bounding exposure duration assumes that an individual is also present at the controlled area boundary for 30 days. This time period is the same as that used to demonstrate compliance with 10 CFR 100 for reactor facilities licensed per 10 CFR 50 and provides good defense in depth since recovery actions to limit releases are not expected to exceed 30 days.

Id. at 9-15.

[138] This is the same time period the Staff used in evaluating accident consequences in the HI-STORM 100 Certificate of Compliance. *See* 65 Fed. Reg. at 25245.

[139] Indeed, if an accident were to occur, there is no reason to expect that some person would be at the boundary fence for a period of 30 days, other than people who had an absolute need to be at the site. Tr. 12311.

doses could be reduced by uprighting the casks, removing persons from the site boundary, and providing temporary shielding (such as lead blankets, steel plates, or a berm). Tr. 12202-03, 12267, 12268-89, 12583, 12589.[140]

6.236. In sum, we are satisfied that the postulated multiple cask tipover event, if it were to occur at the PFSF site during a seismic event, would not adversely affect public health and safety. As all parties agree -- and even under Dr. Resnikoff's analysis -- the 5 rem dose limit stated in § 72.106(b) is nowhere close to being exceeded. *See* Singh/Soler/Redmond, Post Tr. 12044, at 5, 8, 11-12, 20-22. This is true even if the duration of exposure is assumed to be one year. *Id*. at 12-13. Moreover, Dr. Resnikoff admitted that even under his calculations, it would take more than 33 years for a cask tipover accident to exceed the dose limits of § 72.106(b) -- assuming no corrective actions are taken and dose rates do not decrease due to decay, which we find to be absolutely unrealistic.

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[140] We further note that Dr. Resnikoff asserted that in calculating doses for normal operations and anticipated occurrences, the occupancy time for a "real individual" at the OCA boundary should be greater than 2,000 hours per year, in that future land development in the vicinity of the site could result in an increase in population at the OCA boundary. Resnikoff, Post Tr. 12349, at 7; Tr. 12439. We find no basis for this assertion.

At present, the nearest residence to the PFSF is located two miles southwest of the facility. Tr. 12320; Staff Exh. 59. Only about 30 persons live on the Reservation, and only about 150 persons live in Skull Valley. Staff Exh. E, § 3.5.2.2, at 3-41. Further, only 36 persons live within a 5-mile radius of the proposed PFSF. Tr. 12330; Staff Exh. E. at 3-42. There are no transient or institutional populations within 5 miles of the proposed PFSF, and no public facilities are located or planned within that radius. Staff Exh. E at 3-42. Dr. Resnikoff was not familiar with any potential future land use development in the area. Tr. 12474-75. Moreover, Mr. Donnell testified that land to the east of the OCA boundary is tribal land would be under PFS control under the lease, and would be designated as "buffer zones" within which changes to land use would effectively be prohibited. Tr. 12562. Land to the west of the OCA is controlled by the Bureau of Land Management ("BLM"), and is used for grazing; and land to the north of the OCA boundary is private land that is also used for grazing. Tr. 12560, 12565; *see* PFS Exh. 243. Mr. Donnell was not aware of any planned changes in the use of that land. Tr. 12565. For these reasons, we find no reason to require the use of an occupancy time for a "real individual" located at the OCA boundary that is greater than 2,000 hours per year in calculations for doses resulting from normal operations and anticipated occurrences.

6.237. Having considered the opportunities for implementing mitigation or recovery actions in the event of a cask tipover accident, as set forth above, and the very conservative dose consequence analysis performed by the Staff, we find that § 72.106(b) would be satisfied in the event of a tipover accident involving multiple casks. In addition, Dr. Resnikoff's calculations, to the extent they show anything, demonstrate that the dose limit in § 72.106(b) would not be exceeded for many years. Accordingly, we find that the accident dose limits of § 72.106(b) will not be exceeded in the event of a multiple cask tipover event at the proposed PFS Facility.

III. CONCLUSIONS AND ORDER

7.1 The Licensing Board has considered all of the evidence presented by the parties on Unified Contention Utah L/QQ. Based upon a review of the entire record in this proceeding and the proposed findings of fact and conclusions of law submitted by the parties, and based upon the findings of fact set forth herein, which are supported by reliable, probative, and substantial evidence in the record, the Board has decided all matters in controversy concerning this contention and reaches the following conclusions with respect to the contested issues raised in Unified Contention Utah L/QQ in this proceeding.

7.2. The Applicant has satisfied the Commission's requirements governing the characterization of subsurface soils for an ISFSI, set forth in 10 C.F.R. Part 72, in that it has:

(a) evaluated site characteristics that may directly affect the safety or environmental impact of the proposed facility, in accordance with 10 C.F.R. § 72.90;

(b) characterized the subsurface soils in accordance with 10 C.F.R. § 72.102 (c) and (d), including the satisfactory performance of site-specific investigations and laboratory analyses showing that soil conditions are adequate for the proposed foundation loading;

(c) demonstrated that it has designed its structures, systems, and components important to safety "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," in accordance with 10 C.F.R. § 72.122(b)(1); and

(d) demonstrated that its important-to-safety SSCs are designed to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform their intended design functions, in accordance with 10 C.F.R. § 72.122(b)(2).

7.3. The Applicant has satisfied the Commission's requirements governing foundation stability and the seismic analysis and design for an ISFSI, set forth in 10 C.F.R. Part 72, in that it has:

(a) evaluated the foundation stability of its proposed site for "its liquefaction potential or other soil instability due to vibratory ground motion," in accordance with 10 C.F.R. § 72.102(c);

(b) conducted site-specific investigations and laboratory analyses which show that soil conditions are adequate for the proposed foundation loading, in accordance with 10 C.F.R. § 72.104(d);

(c) conducted analyses and evaluations of its seismic design and the 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, 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 accordance with 10 C.F.R. § 72.24(d)(2);

(d) demonstrated that its important-to-safety SSCs are 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, in accordance with 10 C.F.R. § 72.122(b)(1); and

(e) demonstrated that its important-to-safety SSCs are designed to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform safety functions, in accordance with 10 C.F.R. § 72.122(b)(2).

7.4. Notwithstanding its request for an exemption from the deterministic requirements in § 72.102(f), the Applicant has satisfied the Commission's other requirements pertaining to the identification of a design basis earthquake, in that it has:

(a) evaluated the 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, in accordance with 10 C.F.R. § 72.90;

(b) identified and assessed 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, in accordance with 10 C.F.R. § 72.92;

(c) identified the regional extent of external phenomena that are used as a basis for the design of the facility, in accordance with 10 C.F.R. § 72.98(a);

(d) designed structures, systems, and components important to safety to accommodate the effects of, and be compatible with, site characteristics and environmental conditions and to withstand postulated accidents, in accordance with 10 C.F.R. §72.122(b)(1); and

(e) designed important-to-safety SSCs to withstand the effects of natural phenomena, including earthquakes, without impairing their capability to perform safety functions, in accordance with 10 C.F.R. § 72.122(b)(2).

7.5. Pursuant to the provisions of 10 C.F.R. § 72.7, the Applicant has requested an exemption from the deterministic requirements specified in 10 C.F.R. § 72.102(f) and 10 C.F.R. Part 100, Appendix A. While the Applicant has sought an exemption from those requirements, it has also provided an acceptable alternative thereto, in requesting that the seismic design for the PFS Facility be based on its site-specific probabilistic seismic hazard analysis (PSHA) with a design basis earthquake ground motion having a mean annual probability of exceedance of 5×10^{-4} (2,000-year return period).

7.6. Based on our consideration of all of the evidence in this proceeding, we conclude, pursuant to 10 C.F.R. § 72.7, that the Applicant's exemption request is authorized by law and will not endanger life or property or the common defense and security and is otherwise in the public interest.

7.7. For the reasons stated above, it is our conclusion that, with respect to the matters contested in Unified Contention Utah L/QQ, the application for license submitted by Private Fuel Storage, L.L.C. may be granted pursuant to 10 C.F.R. § 72.40, in that:

(a) it complies with 10 C.F.R. Part 72, Subpart E (Siting Evaluation Factors), in accordance with 10 C.F.R. § 72.40(a)(2);

(b) it complies with 10 C.F.R. Part 72, Subpart F (General Design Criteria), except insofar as it relies upon its request for an exemption from the deterministic requirements stated therein, in accordance with 10 C.F.R. § 72.40(a)(1);

(c) there is reasonable assurance that: (i) the activities authorized by the license can be conducted without endangering the health and safety of the public and (ii) these activities will be conducted in compliance with the applicable regulations of this chapter, in accordance with 10 C.F.R. § 72.40(a)(13); and

(d) the issuance of the license will not be inimical to the common defense and security, in accordance with 10 C.F.R. § 72.40(a)(14).

7.8. This conclusion depends upon the Applicant's satisfaction of the various commitments set out in the findings above. We leave verification of the Applicants' conformance with these provisions to the experts of the NRC Staff under the direction of the Director of Nuclear

Material Safety and Safeguards, and the Commission's inspection and enforcement staff located in Rockville, Maryland and the NRC Region IV office in Arlington, Texas. Except where specified time requirements have been placed on the Applicant's conformance, the Director has broad discretion in the timing and manner of conformance consistent with the discussion in the findings above and in furtherance of the Board's intent.

7.9. Upon the issuance of this Initial Decision and other pending decisions concerning other contested matters, all issues remaining in controversy in this license proceeding will have been decided. Upon favorable resolution of those other issues and the NRC Staff's favorable resolution of any outstanding uncontested issues, we conclude that pursuant to the Atomic Energy Act, as amended, and the Commission's regulations, the Director of Nuclear Material Safety and Safeguards may issue to the Applicant a license to construct and operate the proposed PFS Facility, subject to the Commission's authorization as provided in 10 C.F.R. § 2.764(c).

7.10. Pursuant to 10 C.F.R. § 2.786(b)(1), petitions for review of this Initial Decision may be filed within 15 days from the date of its service, unless the Commission directs otherwise. *See also* 10 C.F.R. §§ 2.760 and 2.764.

7.11. Any petition for review must be no longer than 10 pages, and must contain: (i) a concise summary of the decision or action of which review is sought; (ii) a statement (including record citation) where the matters of fact or law raised in the petition for review were previously raised before the presiding officer and, if they were not why they could not have been raised; (iii) a concise statement why in the petitioner's view the decision or action is erroneous; and (iv) a concise statement why Commission review should be exercised.

7.12. Any other party to the proceeding may, within ten (10) days after service of a petition for review, file an answer supporting or opposing Commission review. This answer must be no longer than ten (10) pages and should concisely address the matters set forth in paragraph 7.11

above, to the extent appropriate. The petitioning party shall have no right to reply, except as permitted by the Commission. *See* 10 C.F.R. § 2.786.

/RA/

Respectfully submitted,

Sherwin E. Turk
Robert M. Weisman
Counsel for NRC Staff

Dated at Rockville, Maryland
this 5th day of September 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)	)	

CERTIFICATE OF SERVICE

I hereby certify that copies of "NRC STAFF'S FINDINGS OF FACT AND CONCLUSIONS OF LAW CONCERNING UNIFIED CONTENTION UTAH L/QQ (GEOTECHNICAL ISSUES)," in the above captioned proceeding have been served on the following through deposit in the NRC's internal mail system, with copies by electronic mail, as indicated by an asterisk, or by deposit in the U.S. Postal Service, as indicated by double asterisk, with copies by electronic mail this 5th day of September, 2002:

Michael C. Farrar, Chairman*
Administrative Judge
Atomic Safety and Licensing Board
U.S. Nuclear Regulatory Commission
Washington, DC 20555
(E-mail copy to MCF@NRC.GOV)

Dr. Jerry R. Kline*
Administrative Judge
Atomic Safety and Licensing Board
U.S. Nuclear Regulatory Commission
Washington, DC 20555
(E-mail copy to JRK2@NRC.GOV)

Dr. Peter S. Lam*
Administrative Judge
Atomic Safety and Licensing Board
U.S. Nuclear Regulatory Commission
Washington, DC 20555
(E-mail copy to PSL@NRC.GOV)

Atomic Safety and Licensing Board
Panel
U.S. Nuclear Regulatory Commission
Washington, DC 20555

Office of the Secretary*
ATTN: Rulemakings and Adjudications
Staff
U.S. Nuclear Regulatory Commission
Washington, DC 20555
(E-mail copies to SECY@NRC.GOV
and HEARINGDOCKET@NRC.GOV)

Office of the Commission Appellate
Adjudication
Mail Stop: 16-C-1 OWFN
U.S. Nuclear Regulatory Commission
Washington, DC 20555

James M. Cutchin, V*
Atomic Safety and Licensing Board
U.S. Nuclear Regulatory Commission
Washington, DC 20555
(E-mail to JMC3@NRC.GOV)

Jay E. Silberg, Esq.**
Ernest Blake, Esq.
Paul A. Gaukler, Esq.
Sean Barnett, Esq.
Shaw Pittman
2300 N Street, N.W
Washington, DC 20037-8007
(E-mail copy to jay_silberg,
paul_gaukler, sean_barnett, and
ernest_blake@shawpittman.com)

Tim Vollmann, Esq.**
3301-R Coors Road N.W.
Suite 302
Albuquerque, NM 87120
(E-mail copy to tvollmann@hotmail.com)

Leon Bear, Chairman
Skull Valley Band of Goshute Indians
3359 South Main
Box 808
Salt Lake City, Utah 84115

Denise Chancellor, Esq.**
Fred G. Nelson, Esq.
Laura Lockhart, Esq.
Utah Attorney General's Office
160 East 300 South, 5th Floor
P.O. Box 140873
Salt Lake City, UT 84114-0873
(E-mail copies to dchancellor, fnelson,
llockhart, jsoper, and jbraxton@utah.gov,
and attygen@xmission.com

Connie Nakahara, Esq.**
Utah Dep't of Environmental Quality
168 North 1950 West
P. O. Box 144810
Salt Lake City, UT 84114-4810
(E-mail copy to
cnakahara@utah.gov)

Diane Curran, Esq.**
Harmon, Curran, Spielberg & Eisenberg
1726 M Street, N.W., Suite 600
Washington, D.C. 20036
(E-mail copy to
dcurran@harmoncurran.com)

John Paul Kennedy, Sr., Esq.**
David W. Tufts, Esq.
Durham, Jones & Pinegar
111 East Broadway, Suite 900
Salt Lake City, UT 84105
(E-mail copy to dtufts@djplaw.com)

Joro Walker, Esq.**
Land and Water Fund of the Rockies
1473 South 1100 East, Suite F
Salt Lake City, UT 84105
(E-mail copy to utah@lawfund.org

Paul C. EchoHawk, Esq.
EchoHawk Law Offices
151 North 4th Avenue, Suite A
P.O. Box 6119
Pocatello, Idaho 83205-6119
(E-Mail copies to: paul, larry and
mark@echohawk.com)

/RA/

Sherwin E. Turk
Counsel for NRC Staff