UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

DOCKETED 05/22/03

ATOMIC SAFETY AND LICENSING BOARD

SERVED 05/22/03

Before Administrative Judges:

Michael C. Farrar, Chairman
Dr. Jerry R. Kline
Dr. Peter S. Lam

In the Matter of	Docket No. 72-22-ISFSI
PRIVATE FUEL STORAGE	ASLBP No. 97-732-02-ISFSI
(Independent Spent Fuel Storage Installation)	May 22, 2003

PARTIAL INITIAL DECISION
(Regarding Geotechnical Issues)

As we have noted in many previous decisions, Private Fuel Storage (PFS) is a consortium of electric utility companies that applied for an NRC license to build and to operate, on the Reservation of the Skull Valley Band of Goshute Indians some 50 miles southwest of Salt Lake City, an aboveground facility for the temporary storage of spent fuel rods from the nation's nuclear reactors. The Band would derive substantial income from making its Reservation available to the Applicant for the facility, which is intended to serve as the spent fuel's way-station before the coming to fruition of the permanent underground repository long planned for Nevada's Yucca Mountain.

The State of Utah and the Southern Utah Wilderness Alliance (SUWA), among others, challenged a number of aspects of the proposed facility. During a nine-week evidentiary trial, which was held in Salt Lake and at NRC Headquarters and ended in mid-2002, the Applicant PFS -- responding to the State's and SUWA's contentions -- attempted to demonstrate that its proposal was acceptable in terms of meeting certain safety and environmental regulatory

criteria established under federal law, including the Atomic Energy Act and the National Environmental Policy Act (NEPA).

One of those issues, which we resolve today, stemmed from the State's so-called "geotechnical" contentions (denominated Utah L and QQ), involving whether the design of the proposed facility is sufficient to withstand any seismic forces it is likely to face as a consequence of earthquakes that might affect it.[1] Those contentions derive from the Commission's regulations governing site evaluations, under which proposed sites must be examined in terms of the "frequency and severity of external natural and man-induced events that could affect [the facility's] safe operation." 72 CFR § 72.90(b). In terms of seismic forces, this in turn requires the facility be designed to withstand the so-called "design basis earthquake," or "safe shutdown earthquake" [2] (a term used in this field to similar purpose as the "credible accident" concept which underlay our recent decision on aircraft crash likelihood).

For purposes of understanding and deciding the seismic issues, the proposed facility's design can be simply described. Being a facility for storage only, it consists essentially -- insofar as seismic risks are involved -- of a canister transfer building (CTB) and an array of 500 concrete pads on which the spent fuel storage casks would sit. Other onsite structures that support the facility's storage mission raise no seismic risk concerns.

In the CTB, canisters -- sealed at the nuclear power plant at which they originated -- containing spent fuel rods would be transferred from (1) the transportation casks within which they traveled by rail to the facility, to (2) the storage casks which will sit upon the concrete pads.

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[1] Apart from certain issues about potential aircraft accidents that have arisen since our March 10 decision on that subject and that will be considered at two May 29 Board sessions, this Board has remaining before it only one issue, SUWA's environmental contention concerning the placement down Skull Valley of the proposed rail line that would service the facility. For its part, our sister Board has before it several issues about the Applicant's financial qualifications, which it anticipates will be decided shortly.

[2] 10 C.F.R. Part 100, Appendix A, referenced in 10 C.F.R. § 72.102(b).

Those transfers of canisters into the storage casks will be facilitated by the CTB's overhead bridge crane and semi-gantry crane. The application envisions the eventual emplacement of up to 4000 of the storage casks on the concrete pads.

The seismic issues before us concern the stability, during possible earthquakes, of those storage casks, which will be some 20 feet tall and 11 feet in diameter, made of concrete sandwiched between layers of stainless steel. Openings in the top and bottom of the casks are designed to create natural air circulation that would provide the level of cooling then needed by the canisters of spent fuel (which before being transported to Skull Valley would have been cooled for any number of years in pools of water -- and perhaps in dry storage as well -- at the various nuclear power plants at which the fuel originated). Concern was expressed at the hearing about the casks tipping over, either in the CTB (which can hold only five casks at a time) or on the concrete pads, with consequent potential rupturing of the fuel rods or diminution of the natural cooling they need.

A very large, dual-tracked transporter vehicle would straddle and lift a storage cask in order to move it (at 2 miles per hour and 4 inches off the ground) from the CTB to the concrete pad upon which it will sit. The pads -- each 67 feet long and 30 feet wide -- will be made of 3 foot thick reinforced concrete. The pads will be separated from each other by 5 feet in the long direction; in the other direction, they will be separated by 35 feet in order to provide a travel lane for the transporter vehicle to place the casks in a 4 x 2 array on each pad.

To provide support to the pads and the CTB, the Applicant proposes to underlay and to surround them with a mixture of soil and cement. Depending on the proportions of each constituent contained therein, the various mixtures are known as "soil-cement" or "cement-treated soil," the former being of more substantial quality and of greater strength.

We have previously described Skull Valley as being framed by the Stansbury Mountains to the East and the Cedar Mountains to the West: our recent decision on the risk of aircraft

crashes (LBP-03-04, 57 NRC___) provides that and other information about the local geography. For purposes of geotechnical analysis, important nearby features, in addition to the faults associated with those mountains, include the Wasatch Fault, just East of Salt Lake City, as well as two previously-unknown faults (informally named the East and West faults) discovered through the Applicant's investigations.

As will be seen, the State filed a number of contentions that were eventually reshaped into the specific issues which came to hearing before us. In essence, the major issues the State raised involved the following six topics:

- the characterization of the site's subsurface soils, which the State charges was inadequate;
- the proposed uses of soil-cement to overcome foundation sliding, which the State asserts involve novel and untested techniques;
- the assumptions about facility behavior which underlie the seismic design, which assumptions the State says are flawed;
- the stability of the casks during a design basis earthquake, which the State urges has not been adequately demonstrated;
- the exemption from the long-standing "deterministic" standard for predicting ground motion in favor of a "probabilistic" one, which the State challenges as unsupported; and
- the ability of the facility to comply, after a design basis earthquake, with established radiological dose consequences standards, which the State believes will not be met.

On the facts presented, we find that the Applicant has met its burden of proof on all these seismic-related issues. Although the State presented thoughtful, valuable evidence that tested many aspects of the Applicant's presentation, the Applicant's position essentially withstood that scrutiny.

In this decision, we explore at some length all the sub-issues which underlie the main topics outlined above, and explain why we reject the conclusions the State would have us reach. At the same time, we recognize the seriousness of the questions the State raised; the

extensive exploration of those questions in the hearing should provide reassurance to the State's citizens that the merits of the Applicant's proposal have been thoroughly scrutinized. In addition, the State brought to the fore two conflict-of-interest concerns that, although eventually found not to undercut the evidence to which those concerns related, plainly warranted analysis.

We set forth herein all the subsidiary findings needed to address the six major issues that the State raised about the scope and result of the Applicant's seismic investigations and analyses. Our determinations on those six major issues lead inexorably to the ultimate safety findings: based on the preponderance of the evidence in the record and taking into account the nature of the seismic forces the facility is predicted to encounter and the investigations and analyses that have been conducted, the Applicant's proof on the issues in controversy -- which was essentially supported by the NRC Staff based on its lengthy pre-hearing review of the application and related materials -- enables us to say, with the required degree of certainty, that (1) the spent fuel casks would not tip over during a design basis seismic event; and (2) even if one or more casks were to tip over, the spent fuel canister inside would not break or melt. It bears mention as well that, as both the Commission and this Board have previously indicated, even if a canister were to break or melt, the absence of significant dispersive forces would mitigate the consequences of such an event.

Not surprisingly given the complex nature of the contentions and the evidence, our decision today, so briefly summarized above, is a very long one. In Parts I and II, we set forth certain preliminary information about the genesis, development and reshaping of the State's contentions (including the efforts the parties made to put forward a consensus re-statement of those contentions, which arose over a lengthy period of time), and about the facility design and the State's concerns. We then turn in Parts III through VIII to address each of the six major concerns the State raised.

Those first eight parts of the decision -- ending on page 105 -- provide in narrative form an overview of the underlying reasoning that led us to the results we reach. Those eight parts, in turn, are keyed to Part IX, in which we provide a lengthy "Detailed Analysis of Record and Findings of Fact" [hereinafter referred to as "Findings"] that reviews the evidence and includes determinations either providing support for, or resulting from, the opinions and holdings expressed in the earlier, narrative portion of the decision. Finally, in Part X, we recite briefly our formal Conclusions of Law and our Order.

An outline of the entire decision's contents is provided below. A separate table of contents for Part IX begins on page 106.

I. BACKGROUND

A. Contention Utah L

The State's first geotechnical challenge to the application, Contention Utah L, was submitted in November 1997 and admitted into the proceeding in April 1998. See LBP-98-7, 47 NRC 142, 253, reconsideration granted in part and denied in part on other grounds, LBP-98-10, 47 NRC 288, aff'd on other grounds, CLI-98-13, 48 NRC 26 (1998). As admitted, Utah L framed the State's geotechnical concern as follows:

> The Applicant has not demonstrated the suitability of the proposed ISFSI site because the License Application and [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.

LBP-98-7, 47 NRC at 253. In support of its contention, the State submitted bases that addressed the following issues: (1) surface faulting; (2) ground motion; (3) characterization of subsurface soils; and (4) soil stability and foundation loading. LBP-01-39, 54 NRC 497,501 (2002).

According to the current Commission regulation governing the ISFSI seismic design, potential licensees, such as PFS, proposing facilities located west of the Rocky Mountain Front, must comply with the standards regarding seismic stability requirements for nuclear power plants contained in 10 C.F.R. Part 100, App. A. See 10 C.F.R. § 72.102(b). Appendix A

requires nuclear power plants to be designed to withstand the ground motions of a "safe shutdown earthquake."[3] 10 C.F.R. Part 100, App. A. The regulation requires potential licensees to determine the safe shutdown earthquake -- or what has also been referred to as a design basis earthquake (DBE) -- using a deterministic methodology established in Appendix A. Id.

In 1997, the Commission amended sections of Part 100 to allow nuclear power plants to use a probabilistic analysis that accounts for the probability that an earthquake of a particular intensity will occur within a given time span rather than limiting the analysis to the intensity of the earthquake. See CLI-01-12, 54 NRC 459, 461 (2001). See also 10 C.F.R. § 100.23 (establishing the Commission's probabilistic seismic analysis for nuclear power plants). This amendment, however, was not made applicable to the methodology established for ISFSI licenses in Appendix A.

Later, in 1998, the Staff proposed a new rulemaking plan that would conform the regulations governing ISFSIs to the amended rule for nuclear power plants, allowing ISFSIs as well to use a probabilistic methodology in their seismic analysis. See SECY-98-126, Rulemaking Plan: Geological and Seismological Characteristics for Siting and Design of Cask [ISFSIs], 10 C.F.R. Part 72 (June 4, 1998). According to the Commission, this new approach would allow ISFSI applicants to design facilities based either on a 1000-year return period or on a 10,000-year return period (depending upon the amount of potential radiation a person outside the facility's proposed boundary would receive if the structure in question were to fail).

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[3] The safe shutdown earthquake is defined by 10 C.F.R. Part 100, App. A as:

> that earthquake which is based upon an evaluation of the maximum earthquake potential considering the regional and local geology and seismology and specific characteristics of local subsurface material. It is that earthquake which produces the maximum vibratory ground motion for which certain structures, systems, and components are designed to remain functional.

CLI-01-12, 53 NRC at 462. Under the new rule, structures that would cause radiation doses to exceed the maximum limits prescribed by Commission regulations would be designed to withstand a 10,000-year DBE, while all other facilities would be designed to withstand a 1000-year DBE. Id.

On April 2, 1999, the Applicant submitted an exemption request to the NRC Staff that would allow the Applicant to use a probabilistic seismic hazard analysis (PSHA) based upon a 1000-year DBE. LBP-99-21, 49 NRC 431, 433-34 (1999). According to the Applicant, its preliminary probabilistic analysis indicated that the "relative risk" at the proposed site warranted a DBE with much lower ground accelerations than required under Part 100's deterministic approach. Id. at 434. This exemption request was of particular concern to the State because, as noted by the Staff, the Applicant's facility could not meet the deterministic seismic qualifications applicable under the existing regulations.[4]

In response to the Applicant's exemption request, the State filed a motion requesting that the Board either (1) require the Applicant to make its exemption request under the provisions of 10 C.F.R. § 2.758(b), which governs the consideration of Commission rules in adjudicatory proceedings, or (2) allow the State to amend its contention to contest the Applicant's exemption request. LBP-99-21, 49 NRC at 434-35. Both the Applicant and the Staff opposed these requests. Id.

We denied both of the State's requests. The first request was denied because section 2.758(b) was found to be inapplicable to the proceeding at that time. Id. at 439. And the request to allow the State to modify its contention was also denied, because we found it to be premature. Id. In that regard, we informed the State that this matter would not be ripe for consideration unless and until the Staff took favorable action on the Applicant's request. Id.

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[4] See id. at 435-35; NRC Staff's Response to [State] Request for Admission of Late-Filed Modification to Basis 2 of Contention Utah L (Nov. 29, 2000) at 2.

In August 1999, the Applicant modified its request to reflect a 2000-year return period earthquake. And, on December 15, 1999, the Staff issued its Safety Evaluation Report (SER) in which it noted that it planned to grant the exemption request based upon this 2000-year return period interval. LBP-00-15, 51 NRC 313, 315 (2000).

In response to the Staff's SER, the State filed another request to modify Utah L to address the Applicant's exemption request. Id. at 316. The modified contention requested that the Board require the Applicant either (1) to use a probabilistic methodology based upon a 10,000-year return period earthquake or (2) to comply with the current deterministic analysis requirement of 10 C.F.R. § 72.102(f). Id. at 316.

Both the Applicant and Staff opposed the State's request but for different reasons. Id. at 316-17. The Applicant argued that the State's request was outside the scope of the proceeding, an impermissible challenge to the Commission's regulations, and immaterial. Id. Taking a slightly different approach, the Staff contended that the request was not yet ripe because the SER did not grant the exemption but merely indicated the Staff's intended approval. Id. at 317. The Staff also asserted that the State's proposed modification was an impermissible challenge to Staff activity and outside the scope of the exemption request. Id. We denied the State's request, this time advising the State that such a request would not be ripe for adjudication until the Staff officially granted the Applicant's request. Id. at 318.

On September 29, 2000, the Staff issued its final SER, which noted that the Staff had completed its review of the Applicant's exemption request and concluded that the use of PSHA methodology with a 2000-year return period is acceptable for the proposed PFS facility. See [SER] Concerning the [PFS] Facility at 2-42 (Sept. 29, 2000). In response, the State again filed a request to modify Utah L to address the Applicant's exemption, and the State requested that if the Board found that it did not have the authority to address the State's concerns, the Board certify or refer the matter to the Commission. See LBP-01-03, 53 NRC 84, 89-90 (2001).

According to the State, the Staff's support for its decision to grant the Applicant's exemption is deficient because:

> (1) it fails to comply with the 1998 rulemaking plan, which provides only for 1000-year and 10,000-year design basis ground motion return periods, and fails to take into account (a) the radiological consequences of a failed design, or (b) the PFS failure to demonstrate that the PFS facility and its equipment will protect against exceeding the dose limitations of 10 C.F.R. § 72.104(a) or can withstand a 2000-year return period earthquake; (2) the reasons relied upon by the Staff for permitting the 2000-year return period -- lower hazard compared to commercial power reactors, Department of Energy (DOE) category-3 facility performance characteristics, an exemption granted to DOE relative to ISFSI storage of Three Mile Island, Unit 2 fuel at DOE's Idaho National Engineering and Environmental Laboratory (INEEL) -- are flawed or not compelling; and (3) a 2000-year return interval does not provide an adequate level of conservatism given the higher Utah new building construction/highway bridge design levels and the 30 to 40-year facility operating period. See id. at 6-14.

Id. at 90 (citing [State] Request for Admission of Late-Filed Modification to Basis 2 of Contention Utah L (Nov. 9, 2000) at 6-14).

In opposing the State's request, the Applicant argued that (1) the Board lacked jurisdiction to hear the issue; (2) it was an improper challenge to a Commission regulation; and (3) the State failed to support the modification with an adequate basis. LBP-01-3, 53 NRC at 90. For its part, the Staff requested that the Board certify or refer the issue to the Commission to definitively answer the question of whether the State's attack on the Applicant's exemption request was permissible in this adjudicatory proceeding or in the alternative that we deny the State's request outright for failing to provide a litigable contention. Id. at 91.

On January 31, 2001, we issued a Memorandum and Order admitting in part and denying in part the State's proffered modifications to Utah L. Id. at 101. In addition, having decided that the exemption matter was one that we believed could only be resolved by the Commission, we certified to the Commission the question of "whether the State's contention Utah L challenge to the April 1999 PFS seismic exemption request should be litigated in this proceeding." Id.

In accordance with its policy to accept Board certifications of issues that warrant early resolution, the Commission granted review of the Board's certified question and found that the State's exemption-related claims could be litigated in this proceeding. CLI-01-12, 53 NRC at 461. In doing so, the Commission classified the State's challenge in the following manner:

> what [the State] 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. Viewed this way, [the State's] proposed revised Contention [Utah L] plainly puts into play safety issues that are material to licensing and suitable for consideration at an NRC hearing.

Id. at 465-66. Having decided the jurisdictional issue, the Commission remanded the matter to us, because it concluded that the Board, not the Commission, was the proper forum to hear a State challenge in the first instance. Id. at 476. Following the Commission decision, the Board, in an attempt to clarify the terms of Utah L in light of the recent decisions, split the issues into two parts. The State's exemption request was denominated Part B of Utah L, with the original four bases of the contention being Part A.[5]

After discovery was held on Part B, the Applicant filed for summary disposition thereon on November 9, 2001, insisting that there no longer remained a genuine dispute of material fact concerning Part B. LBP-02-01, 55 NRC 11, 14 (2002). The Staff supported the Applicant's request, while the State opposed it. Id. at 14-15.

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[5] See Memorandum and Order (Requesting Joint Scheduling Report and Delineating Contention Utah L) (Jun. 15, 2001). The Board further explained this division in a subsequent decision:

> Part A challenges the Applicant's efforts to show that its facility design generally *meets the requirements* of the NRC's rules and regulations regarding seismic risk. Part B challenges the Applicant's efforts to rely upon an *exemption from meeting* one part of those rules and regulations and to substitute another method for demonstrating that the potential seismic risk is being properly addressed.

LBP-02-01, 55 NRC 11, 14 (2002).

After reviewing the arguments, we found that the State's experts had presented sufficient evidence to create doubt about the Applicant's claims that there no longer remained a dispute of material fact. Id. at 18. In that regard, we determined that Part B of Utah L could be resolved only after a full hearing on the matter, so we rejected the Applicant's motion. Id. We also ordered the parties to combine Part B of Utah L with Part A of Utah L and the newly-admitted Utah QQ (see discussion below) to create a unified geotechnical contention that we believed would aid the presentation and understanding of the issues in the upcoming hearing. Id.

B. Contention Utah QQ

While we were dealing with the Applicant's exemption request as raised in Part B of Utah L, discovery was completed on the remaining seismic issues that comprised Part A of Utah L, and the Applicant, on December 30, 2000, filed for summary disposition of Part A. See LBP-01-39, 54 NRC 497, 502 (2001). In support of its claim that there no longer remained a dispute of material fact concerning Part A, the Applicant pointed to several geotechnical tests and analyses that it argued addressed the issues raised in the State's contention. Id. at 512-16. The State, in opposing the Applicant's motion, submitted a list of material facts that it claimed were still in dispute and a collection of expert affidavits to support this claim. Id. at 503. In its response, the Staff submitted its own collection of expert affidavits in support of the Applicant's motion. Id.

As we were deliberating over the parties' submissions, on March 30, 2001, the Applicant filed its 22nd of 23 license amendments, to incorporate revised design basis ground motions anticipated at the proposed site. In response to this amendment, the State submitted a motion to add a new contention -- Contention Utah QQ (Seismic Stability) -- dealing with the Applicant's

revised calculations. See [State]'s Request for Admission of Late-Filed [Utah QQ] (May 16, 2001) [hereinafter State Request].

As submitted, the Utah QQ, titled "Seismic Stability," states:

> PFS's site specific investigations, laboratory analyses, characterization of seismic loading, and design calculations, including redesign of soil cement, [fn.] fail to demonstrate that a) the newly revised probabilistic seismic hazard design basis ground motions have been correctly and consistently applied to the Canister Transfer 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 safety 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 C.F.R. §§ 72.102(c), (d); 72.122(b).
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> [fn.] PFS uses the term "soil cement" but the more correct term is "cement-treated soil." *See* Mitchell Dec. ¶ 12. The use of the term "soil cement" in this filing does not imply the State accepts that PFS will, in fact, use soil cement.

State Request at 2-3. In support of its contention, the State proffered a "basis presentation" that alleged that the Applicant's revised design "is unsupportable and creates significant safety concerns." Id. The basis presentation was divided into four areas of concern:

> (1) application of the new design basis ground motion to the [Canister Transfer Building] and its foundation system;
> (2) application of the new design basis ground motion to the storage casks and the storage pads; (3) survivability and durability of cement-treated soil for the redesigned [Canister Transfer Building] and storage pad foundation systems; and
> (4) overestimation of the sliding resistance provided by the clayey-silt and silty-clay underlying the [Canister Transfer Building] and storage pads.

See LBP-01-39, 54 NRC at 518.

On June 19, 2001, the State submitted a motion to revise Utah QQ to reflect another set of recently submitted Application revisions. The Applicant opposed this motion, but the Staff believed that the request was, at least in part, valid. On August 23, the State submitted a

second request to modify its contention, to address yet another set of Applicant recalculations. Again, the Applicant opposed the State's motion and the Staff indicated that the request was valid in part. Id. at 504.

On December 26, 2001, we issued a Memorandum and Order admitting Utah QQ, as well as denying the Applicant's motion for summary disposition of Part A of Utah L. Id. at 524. The Board also granted the State leave to amend the bases of Utah QQ to reflect the revisions submitted by the Applicant. Id. at 521. In addition, the Board ordered the parties to create a statement that combined the thrust of both Part A of Utah L and Utah QQ (and later Part B of Utah L, see discussion above) to help the parties better to prepare for the then-upcoming hearing. Id. at 521.

C. Unified Contention L/QQ

In response to our order, on January 16, 2002, the parties submitted Unified Geotechnical Contention, Utah L and Utah QQ (Utah L/QQ) setting forth the remaining geotechnical issues and their supporting bases. Joint Submittal of Unified Geotechnical Contention, Utah L and Utah QQ (Jan. 16, 2002) (PFS Exh. 237) [hereinafter Unified Utah L/QQ]. The new unified contention Utah L/QQ contained five sections.

Section A, which deals with surface faulting and Section B, which deals with ground motion, were drawn from bases 1 and 2 of the original Part A of Utah L. They read 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 C.F.R. Part 100 Appendix A.

Id. at 1-2.

Prior to the hearing, the parties stipulated to the facts and issues in Section A and B. See Joint Stipulation of Facts and Issues Not in Dispute with Respect to [Utah L/QQ] (Geotechnical) (Jan. 31, 2002). Thus, no further consideration was given to them.

Section C of Utah L/QQ, which addresses the Applicant's characterization of the proposed site's subsurface soils, includes both the original basis 3 of Utah L, Part A and the portion of Utah QQ that deals with the Applicant's proposed use of soil-cement. As submitted, Section C reads:

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 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 [DBE].

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 [Canister Transfer Building (CTB)] and storage pad foundations as required by 10 C.F.R. § 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.

Unified Utah L/QQ at 2-3.

Section D, which deals with the facility's proposed seismic design and foundation stability, covers the remainder of Utah L, Part A, while Section E, which deals with the seismic exemption, covers what was originally Utah L, Part B. They read 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 [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 [DBE]. 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 [sic] 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 C.F.R. § 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 [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 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.

E. <u>Seismic Exemption</u>.

Relative to the PFS seismic analysis supporting its application and the PFS April 9, 1999 request for an exemption from the requirements of 10 C.F.R § 72.102(f) to allow PFS to employ a 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 [S]taff'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 [S]taff 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 [S]taff 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.

Id. at 3-7.

Hearings on the three sections of the Unified Contention that remained active (C, D, and E) were held in Salt Lake City from April 29, 2002 through May 13, May 16 and May 17, and June 3 through June 8, 2002. An additional two weeks of hearings were held in Rockville, Maryland from June 17 through June 27, 2002.

D. Witness Qualifications

Over the course of our hearing, we heard testimony from a total of twenty-two expert witnesses for the various parties, each of whom assessed the Applicant's seismic design and analysis. The Board finds that each of the experts proffered are well qualified in their fields of expertise.

In support of its seismic design and analysis, the Applicant proffered eleven witnesses in eight panels of one to three witnesses each. These witnesses had degrees in mechanical, civil, structural, and nuclear engineering as well as countless years of expertise analyzing the suitability of structures to withstand the effects of earthquake conditions. The Board finds each of the Applicant's witnesses to be well qualified in their particular fields of expertise.

For its part, the Staff presented eight expert witness in five panels to support its analysis and subsequent approval of the Applicant's proposed seismic design. These witnesses had degrees in various engineering disciplines as well as numerous years of hands-on experience

evaluating and analyzing structures and facilities similar in nature to the proposed PFS facility. The Board also finds the Staff's experts to be well qualified in their particular areas of expertise.

Like the Applicant and the Staff, the State also relied upon the testimony of expert witnesses to support its claims. To support its challenges to the Applicant's seismic design, the State relied upon the expertise of six witnesses. The State's experts had various areas of expertise ranging from geotechnical engineering to nuclear physics and they too have logged countless years of experience analyzing structural response to earthquake conditions. As we have found with the Applicant and Staff experts, the Board also finds the State's experts to be well qualified in their fields of expertise.

II. FACILITY DESIGN AND LAYOUT

A. Design and Location

The Applicant proposes to construct and to operate a dry cask storage ISFSI that will store up to 4000 concrete and steel casks of spent nuclear fuel (SNF). If approved, the license would allow the Applicant to store SNF at the site for 20 years, with an option to renew the license for an additional 20 years if needed.

The proposed facility is to be located in the northwest corner of the Reservation of the Skull Valley Band of Goshute Indians. The Reservation itself is located approximately 50 miles southwest of Salt Lake City, Utah. There are no large towns within 10 miles of the proposed facility; the city of Tooele is 27 miles away. The nearest small town, the Goshute Indian Village, which consists of roughly thirty residents, is located 3.5 miles from the facility. See Findings A.3.

The proposed facility will contain a restricted zone of approximately 99 acres surrounded by a chain link security fence and an outer link nuisance fence. An isolation zone and intrusion detection system will be located between the two fences as a further security measure. In addition to the storage pads, a Canister Transfer Building (CTB), where the SNF will be transferred from temporary shipping casks to permanent storage casks, will also be located within the restricted area. An overhead bridge crane and a semi-gantry crane that will be used to transfer fuel from shipping to storage casks will be housed in the CTB. See Findings A.6.

According to the Applicant's proposal, several organizations are responsible for the design and testing of the proposed facility; representatives of these organizations testified on behalf of the proposed design in this licensing hearing. Holtec International (Holtec) is responsible for the design of the HI-STORM 100 Cask System (HI-STORM 100). Stone & Webster Engineering Corporation (Stone & Webster) is responsible for the proposed facility's

design. PFS has the overall responsibility for the planning, design, and operation of the facility and for providing quality assurance services. See Findings A.12.

B. General State Concerns

Throughout the course of this licensing proceeding, the State has raised six major areas of concern with the Applicant's seismic design, each of which is discussed herein at the pages listed in the respective footnotes: (1) there is an inadequate characterization of the subsurface soils at the proposed PFS site;[6] (2) the Applicant's proposed use of soil-cement to overcome foundation sliding during an earthquake is a novel and untested technique;[7] (3) the Applicant's seismic design is flawed due to several assumptions concerning the behavior of the facility during a seismic event;[8] (4) the Applicant has not adequately demonstrated the stability of the storage casks during a DBE;[9] (5) there is a lack of support for the Applicant's exemption request from the deterministic standard that establishes the ground motions for the design of the proposed storage facility;[10] and (6) the proposed facility does not comply with the Commission's established standards concerning radiological dose consequences in the event of a design basis accident at the proposed facility.[11]

These six State concerns were thoroughly litigated during the course of this proceeding. In the subsequent sections of this decision (Parts III-VIII), we describe and discuss each one

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[6] See pp. 30-39.

[7] See pp. 40-50.

[8] See pp. 51-62.

[9] See pp. 63-86.

[10] See pp. 87-99.

[11] See pp. 100-105.

and render our rulings. In most instances, further explanation of those rulings is reflected in the detailed analysis and findings contained in Part IX (pp.106-369).

III. CHARACTERIZATION OF SUBSURFACE SOILS

Section C of the State's unified contention challenges the Applicant's characterization of the subsurface soils located beneath the proposed facility's structures and questions how those soils will perform in the event of a design basis earthquake (DBE). In particular, the State contends that the Applicant has not sufficiently characterized the subsurface soils and should be required to conduct additional sampling and analyses to demonstrate that the subsurface soils have an adequate margin of safety to protect against potential failure during a DBE.

The Commission's regulations establishing the comprehensive requirements for subsurface soils that are to be used to support proposed ISFSI facilities are found in 10 C.F.R Part 72. These regulations require an extensive site-specific evaluation of subsurface soils if the proposed site's soil characteristics directly affect the safety or environmental impacts of the proposed facility. See 10 C.F.R. § 72.102.

In particular, sites located on areas other than bedrock require an evaluation to determine their potential for instability due to vibratory ground motions and site-specific investigations must be conducted to demonstrate that site soil conditions are adequate to sustain the proposed foundation loads. See 10 C.F.R. § 72.102(c)-(d). It is with this regulatory framework in mind that the Board examines the State's challenges to the subsurface soil characteristics of the proposed facility.

A. Subsurface Soils at the Proposed Facility

The Applicant used several techniques to characterize the proposed facility's subsurface soils and to determine their ability to sustain the facility's anticipated foundation loads. These techniques included: (1) soil borings; (2) standard penetration tests; (3) dilatometer tests; (4) cone penetration tests (CPT); (5) seismic CPTs; (6) downhole measurements; and (7) excavating test pits and trenches. See Findings A.3.

The upper layer of the subsurface soil profile, which the Applicant labeled Layer 1, was of the most interest during our proceeding. According to the Applicant's characterization, Layer 1 is approximately 25 to 30 feet thick, consisting of a mixture of clayey silt, silt, and sandy silt that is occasionally intermingled with silty clay and silty sand. In its analyses, the Applicant was able to further divide this significant layer into several sublayers. These sublayers include: layer 1A, a layer of eolian soils roughly 3 to 5 feet thick; layer 1B,[12] a layer of silty clay/clayey silt varying from 5 to 10 feet thick; layer 1C, a mixture of clayey silt, silt, and sandy silt, roughly 7 to 12 feet thick; and layer 1D, a silty clay/clayey silt mixture with a thickness not exceeding 5 feet. See Findings B.5.

B. Factors of Safety of Foundation Soil

Generally, factors of safety are expressed as the capacity of the system to resist failure divided by the demand placed upon the system by foundation loads during a seismic event. The capacity of the foundation is primarily a function of the soil's shear strength and the type, flexibility, and embedment of the foundation. The demand on the system is primarily a function of the intensity of earthquake ground motion and the mass and frequency of vibration of the system. See Findings B.6. Relying on NUREG-0800,[13] the State contends that for extreme environmental events, such as a DBE, a factor of safety of 1.1 is inviolable. The State challenges the Applicant's demonstration that its proposed facility design can provide a 1.1 factor of safety. In particular, the State insists that the Applicant's description and use of

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[12] During the proceeding, Layer 1B, which was the most important sublayer for the purposes of this proceeding, was identified differently by each party. The Staff referred to it as Layer 1B, the Applicant as "Layer 2," and the State as the "upper Lake Bonneville clays."

[13] U.S. Nuclear Regulatory Commission, Standard Review Plan for the Review of Safety Analysis Reports for Nuclear Power Plants, NUREG-0800 (Aug. 1989). Several sections of NUREG-0800 that pertain specifically to the review of the seismic portion of an applicant's SAR were introduced by the NRC Staff as Exhibits CC, DD, and EE.

both the capacity of the soils and the dynamic forces involved should be subjected to further scrutiny because there is only a 6% to 15% margin in the Applicant's calculations. See Findings B.7-.8.

During the hearing, the Staff's expert, Dr. Goodluck Ofoegbu, testified that it is not necessary to meet a factor of safety of 1.1 against soil failure to satisfy NRC requirements in 10 C.F.R. Part 72. See Findings B.9. Regardless, the Applicant goes on to demonstrate that its foundation stability analysis of the minimum factors of safety against sliding and bearing capacity of the pad exceed the recommended 1.1 factor of safety. Thus, the Board is satisfied that, irrespective of whether the Applicant must meet a 1.1 factor of safety, the Applicant's analyses demonstrate that its design meets and indeed exceeds that value. See Findings B.9-.10.

C. Importance of the Shear Strength of the Upper Lake Bonneville Clays

The State insists that an "accurate and adequate" characterization of the upper Lake Bonneville clays is essential to the Applicant's demonstration that the pads and CTB will be supported on a stable foundation during a seismic event. See Findings B.12. The parties agree that the soils in the upper Lake Bonneville clay layer are the soils of interest for establishing the minimum value of soil strength, but the parties disagree as to what extent an "accurate" computation of the strength of those soils is necessary. See Findings B.11.

In response, the Applicant acknowledged that it has focused its soils investigations -- borings, samplings, and laboratory tests -- on the upper Lake Bonneville clay layer, and emphasized the conservative approach it used to establish the minimum strength and other characteristics of the PFSF site soils. Thus, even if there were some inaccuracies in the Applicant's determination of the strength of the upper Lake Bonneville clays, the conservatisms built into its methodology for determining the soil properties and the factors of safety against

soil failure are more than sufficient to assure that the soil conditions are adequate to meet the anticipated foundation loadings. See Findings B.13.

D. Specific State Concerns with the Applicant's Testing of the Subsurface Soils

1. Density of Soil Borings

The State asserts that PFS's sampling program does not conform to the density recommended by Site Investigations for Foundations of Nuclear Power Plants, reflected in Regulatory Guide 1.132, Appendix C. In this regard, there is no disagreement that PFS has met the recommended borehole density for the CTB; the issue concerns the borehole spacing used in the pad emplacement area. See Findings B.15.

Appendix C of Regulatory Guide 1.132, which is specific to nuclear power plants, provides a table of spacing and depth of subsurface explorations for various types of safety-related foundations. For linear structures such as a row of storage pads, Regulatory Guide 1.132 recommends a spacing of one boring per every 100 linear feet for favorable, uniform geologic conditions, where continuity of subsurface strata is found. See Findings B.16.

According to the State, the Applicant drilled nine boreholes (A1, B1, C1, A2, B2, C2, A3, B3, C3) in or near the pad emplacement area for the purpose of retrieving samples for laboratory testing and analysis. These borings, taken together with the CPT soundings, result in a spacing of about one boring or sounding every 221 feet in the pad area.

In rebuttal testimony, the Applicant's expert, Mr. Trudeau, claims that seven additional borings were drilled in or near the pad emplacement area (i.e., boreholes A4, B4, C4, D1, D2, D3, and D4). Reviewing Figure 2.6-19 of the Applicant's Safety Analysis Report (SAR), the State insists that borings A4, B4, and C4 are south of the rail spur and are about 200 feet from the edge of the southern-most row of pads. Furthermore, the State contends that borings D1, D2, and D3 (outside the eastern boundary of the perimeter fence) and D4 (adjacent to the CTB) are about 375 feet or more from the edge of the eastern-most row of pads. See Findings B.17.

Therefore, these seven additional borings do not change the State's estimate of borehole spacing of about 221 feet for the pad area. See Findings B.17-18.

The Board is not persuaded, however, that additional boreholes are necessary. As correctly pointed out by both the Applicant and Staff, Regulatory Guide 1.132 is a guidance document applicable to nuclear power plants and is not necessarily binding upon spent fuel facilities, whose structures are quite different and do not involve inter-connected safety systems sensitive to ground motion. See Findings B.19-.21. Moreover, Regulatory Guide 1.132 acknowledges that borehole spacing and depth often vary due to the complex subsurface conditions at the each individual site. Thus, applicants are encouraged to "temper" their recommendations with actual site investigations, as performed by the Applicant in this case. See Findings B.22. In this instance, the Applicant followed the regulatory guidance for the CTB but developed a different subsurface investigation program for the storage pads. See Findings B.23.

Furthermore, the Applicant's investigations established the horizontal uniformity of the soils and have properly documented the vertical layering of the upper Lake Bonneville clays, and the State has not demonstrated that additional boring holes are necessary. See Findings B.24-.34. Thus, the Board finds the Applicant's borehole spacing to be adequate.

2. Continuous Soil Sampling

The State also claims that the Applicant's investigation did not continuously sample the upper Lake Bonneville clays as recommended by Regulatory Guide 1.132. By not continuously sampling the upper Lake Bonneville clay layer, the State insists that the Applicant has introduced an additional unnecessary level of uncertainty into its estimate of shear strength for the upper Lake Bonneville clays and into the factors of safety calculated for the sliding and bearing capacities of the storage pads. According to the State, the CPTs -- conducted by the Applicant in lieu of continuous sampling -- do not obtain undisturbed samples of soil for

laboratory testing and are not as accurate a measure of the soil shear strength as continuous sampling. See Findings B.43-.46. The State also contends that the CPT testing was conducted after the limited laboratory samples were obtained and, therefore, the CPT data could not have been used to designate the weakest soil zone for laboratory shear testing, as claimed by the Applicant. See Findings B.48.

The purpose of continuous sampling, as recommended by Regulatory Guide 1.132, is to identify potential relatively thin zones of weak or unstable soil contained within otherwise stable soil zones. See Findings B.50. The soil characterizations conducted by the Applicant, both through borehole drillings and CPT tests, indicate that no such zones of weak or unstable soil exist under the pad emplacement area. See Findings B.50-.51. Both the Applicant and the Staff agree that such layers would have been detected through changes in cone tip resistance measured by the CPT tests. See Findings B.52. Furthermore, as previously indicated, Regulatory Guide 1.132 is to be used only as a guidance document and is not binding upon this proceeding. See Findings B.49. For these reasons, the Licensing Board finds that the Applicant’s cone penetrometer tests achieve the objective of the tests recommended in Regulatory Guide 1.132 and additional continuous sampling is not necessary in this case.

3. Undrained Shear Strength Determination

All three parties agree that the undrained shear strength is an important characteristic of soils in the seismic analysis with respect to horizontal and vertical loadings. In testing for shear strength, the Applicant selected a single soil sample of the upper Lake Bonneville clays from the quadrant in the pad emplacement area that was determined by CPT to be the weakest portion of the weakest layer of the soil profile. The Applicant then performed laboratory tests on three specimens taken from this soil sample. Using its CPT tests at thirty-seven different locations in the pad emplacement area, the Applicant later confirmed that the sample tested

had the minimum value of shear strength for the entire pad emplacement area. See Findings B.54-.56.

The State argues, however, that the Applicant's reliance on the laboratory analysis of only a single borehole sample is insufficient to establish a lower bound undrained shear strength for the upper Lake Bonneville layer, and that because of the potential for considerable variability in the upper Lake Bonneville layer, locations may exist in the pad area that have lower shear strengths than that established by the Applicant's one sample. See Findings B.57. Moreover, the State contends that although the Applicant claims that the borehole sample came from the weakest zone of the pad emplacement area, the Applicant should have conducted additional laboratory tests on samples from other locations to confirm this assertion. See Findings B.58. The State also attacks the Applicant's reliance on CPT tests to obtain its shear strength values. See Findings B.59-63.

We find the Applicant's process for determining the minimum shear strength to be technically sound. Using a predetermined location to obtain a single borehole sample based on the weakest portion of the weakest layer of the soil profile (Layer 1B) is a sensible approach if the goal is simply to determine a lower limit of shear strength for the pad emplacement area. The Applicant's choice of this location for its borehole sample is independently supported both by the fact that the soil sample obtained from this location exhibits the highest void ratio of all the samples tested in the pad emplacement area, indicating the lowest density and hence lowest shear strength among the tested samples, and by the subsequent CPT measurements at thirty-seven locations in the pad emplacement area, which correlate well with the measured shear strength at the single borehole sample. See Findings B.54-.55.

The State claims that there can be considerable horizontal variability in the shear strength of the upper Lake Bonneville soils across the pad emplacement area. But the CPT data demonstrates relatively low variability in the pad emplacement area. See Findings B.64.

We, therefore, find the number of samples obtained by the Applicant sufficient to demonstrate the minimum soil shear strength for the proposed facility. Moreover, the Board agrees with the Applicant that even if soils of lower strength were to exist in the pad emplacement area, the conservatisms in the overall seismic design of the pad would more than compensate for any difference in the soil strength. See Findings B.65.

4. Additional Tests (Cyclic Triaxial and Triaxial Extension Tests)

Finally, the State contends that the Applicant has failed to conduct a complete analysis of the subsurface soils, because the Applicant failed (1) to include a strain-controlled cyclic triaxial test in its laboratory shear strength testing program and (2) to analyze fully the stress-strain behavior of the native foundation soils under a range of cyclic strains imposed by the design earthquake. See Findings B.65.

According to the State, earthquake motions are cyclic in nature and may reverse the direction of loads several times during a large earthquake. The State claims that the Applicant's tests do not adequately describe this cyclic stress-strain behavior of the upper Lake Bonneville clays and suggests, as a remedy, that the Applicant should conduct strain-controlled cyclic triaxial tests that will ensure that there is no significant loss or degradation of shear strength due to cycling. See Findings B.67-.68. The State believes this test is important, because if earthquake cycling does cause a degradation in the shear strength of the Lake Bonneville clays, then an "unconservatism" will be introduced into the Applicant's sliding calculations. See Findings B.68.

Although the Applicant did not conduct the strain-controlled cyclic triaxial tests requested by the State, it did conduct resonant column tests, which are a form of strain-controlled cyclic triaxial tests recommended in NRC Regulatory Guide 1.138, Appendix B. These resonant column tests provide the information requested by the State for the range of strain levels adequate to account for any potential degradation in the shear strength of the Lake Bonneville

clays. This is demonstrated by the Applicant's site response analysis conducted by Geomatrix which indicates that, for soils in the greatest effective shear strain layer (Layer 1B), the effective shear strains under design basis seismic loadings are within the range of strains measured directly in the resonant column tests. While strain-controlled triaxial tests can measure soil properties at a much higher strain level than the resonant column tests, these tests are unnecessary for the PFS site because the resonant column tests demonstrate that such high strain levels will not be reached there. See Findings B.69-.70.

Finally, the Applicant has conducted stress-controlled cyclic triaxial tests to assess the collapse potential of the soil, and no degradation of the shear strength of the samples tested was observed throughout 500 cycles of loading at extremely high cyclic stress ratios. Therefore, it can be concluded that the resulting cyclic strains were small, hence no strain-controlled cyclic triaxial tests are needed. See Findings B.71.

The State also contends that the Applicant has failed to test the soils adequately to determine whether the soils are subject to failure due to tension loadings. The Applicant used triaxial compression tests to determine the soils' resistance to bearing capacity failure or tension loading. The State argues, however, that if significant anisotropy is present, then the triaxial compression tests will overestimate the average shear strength resistance and undermine the Applicant's bearing capacity calculations. See Findings B.73. Instead of the triaxial compression tests, the State insists that the Applicant should have used triaxial extension tests, which measure the degree of anisotropy in the soils by causing them to fail in tension. See Findings B.76.

The Applicant's tests demonstrate that the minimum vertical and horizontal shear strengths are almost identical, which in turn establishes that the anisotropy at the proposed facility is insignificant. See Findings B.74-.75. Furthermore, the Applicant's bearing capacity analysis provides a large margin of safety against bearing capacity failure and eliminates the

need for additional tests such as the triaxial extensions. See Findings B.76. Thus, the Board finds that the State requests for the additional tests are unnecessary.

E. Board Conclusions

The Board finds that the soil tests conducted by the Applicant are adequate to demonstrate that the subsurface soils at the proposed site will withstand the proposed foundation loadings during a DBE. Thus, the Board finds that the Applicant's geotechnical site characterization is sufficient to demonstrate compliance with the Commission requirements established in 10 C.F.R. § 72.102(c)-(d) and § 72.122(b).

IV. USE OF SOIL-CEMENT AND CONSTRUCTION PROCESS

A. Background and Proposed Uses

1. Design Description

Both soil-cement and cement-treated soil are created by blending, compacting, and curing a mixture of soil, portland cement, other admixtures, and water to produce a hardened mixture with a greater strength than the original native soil. See Findings C.1. As explained by the Applicant's experts during the hearing, there is a distinction between the two types of mixtures. Soil-cement has greater degrees of stabilization and/or durability and is expected to pass durability tests by reason of its ability to retain its properties after long periods of exposure to the weather. On the other hand, cement-treated soil has less strength than soil-cement, and so is not expected to pass durability tests. See Findings C.2.

The Applicant intends to use soil-cement and cement-treated soil at the site to perform three basic functions: (1) placed directly underneath the cask storage pads, cement-treated soil will act as a cohesive material that will resist the sliding ground forces generated by a seismic event; (2) placed between the pads, soil-cement will provide support for the transporter vehicle that will deliver the storage casks to the pad area; and (3) placed around the CTB, soil-cement will provide additional passive resistance to sliding during a DBE. See Findings C.3.

2. State's General Concerns

The State raises several general concerns with the Applicant's proposed use of soil-cement[14] to bolster the foundations of the storage pads and the CTB. First, the State contends that to satisfy the requirements of 10 C.F.R. § 72.102, the Applicant must demonstrate that the soil conditions at the site are adequate, with the addition of soil-cement, to sustain the proposed foundation loadings. The State believes the Applicant's planned testing programs contain too

[14] In its arguments concerning this matter, the State makes no distinction between soil-cement and cement-treated soil, and we discuss its arguments accordingly.

many uncertainties to allow the Board to find that the Applicant has satisfied this burden. Moreover, the State argues that because the NRC inspection programs are not designed to detect latent defects or to serve as a “construction watchdog,” the Applicant and Staff cannot rely upon the Staff’s post-licensing inspection programs to assure the Licensing Board that the Applicant’s programs will meet their intended safety goals. See Findings C.7.

In addition, the State claims that several of the tests already completed by the Applicant either cannot be relied upon to support its assertions, or indeed cut against them by demonstrating that the proposed design will not satisfy the Commission’s safety requirements. First, the State insists that Applicant’s sliding stability calculations are unreliable because they are not based upon site-specific investigations and laboratory analysis. The State also claims that the Applicant has not adequately demonstrated that the shear strength of the cement-treated soil will meet the necessary 1.1 factor of safety required by the NRC Staff. Finally, the State claims that, even if the Applicant can demonstrate that its design concept is adequate, there is evidence to indicate that significant degradation due to cracking, debonding along interface layers, and moisture infiltration will erode its ability to meet the proposed foundation loadings. See Findings C.8.

B. Specific State Challenges

1. Potential Problems with the Construction Process

As explained by the Applicant’s experts during the hearing, the cement-treated soil that will be placed underneath the concrete pads will be created by removing the top layer of soil at the site and mixing it with the appropriate portions of cement at a processing plant constructed on the site. The proposed design requires between a 1 and 2 foot thickness of cement-treated soil to be placed under each storage pad, depending upon the pad location. See Findings C.11-.12.

The State asserts that the process of removing the overlying soils may inadvertently cause the underlying soils to lose strength, which in turn could undermine the validity of the Applicant's soil test results. For example, the State claims that once the overlying soil layer is removed, the underlying soil will be exposed to the elements, which may cause the soil to dry out due to excessive heat or to gain moisture due to rain. If these events occur, the State believes that the underlying soil would weaken and the Applicant's original soil tests would be no longer representative of the soil's strength. Thus, the State argues that the upper Lake Bonneville clay layers used by the Applicant in its study may not be representative of the soil that will ultimately underlie the pad emplacement. See Findings C.9-.10.

The Board finds inadequate support for these allegations regarding the adverse impact of the construction process on soils. Instead, we find the Applicant's commitment to use proper construction techniques to minimize potential damage to the underlying soil and to establish field-quality control requirements that will ensure that any potential contractor will be mindful of the potential adverse effects of the construction process, sufficient to address the State's concerns. See Findings C.13-.14.

2. Design Problems Affecting the Native Soil and Concrete Storage Pad

The State raises several potential problems that may occur after the soil-cement is added to the pad emplacement area. First, the State claims that the soil-cement is prone to cracking which could affect its ability to function as intended. The State also claims that the infiltration of moisture could potentially cause problems for the design. Finally, the State asserts that the different masses of the pads, casks, and soil-cement will behave differently during a DBE, which would affect the transfer of the anticipated dynamic loads from the casks to the pads. We discuss each of these potential problems below.

a. Cracking

The State is concerned that the soil-cement will crack, causing a loss of tensile strength. The State argues that this loss in tensile strength will decrease the structural competency of the soil-cement layer. See Findings C.15. Furthermore, the cracks in the soil-cement could allow water infiltration, which the State believes could also have adverse affects upon the foundation. See Findings C.17.

If water infiltration were to occur, as discussed below, it would not have a significant impact upon the cement-treated soil or the underlying soils. See Findings C.18. Thus, the only remaining consequence of the potential cracking is the loss of tensile strength. The Applicant does not rely, however, upon the tensile strength of the soil-cement for any of its safety analyses, so the potential loss of this tensile strength is of no consequence. See Findings C.19. The Board finds the State's concerns about tensile strength to be unfounded.

b. Moisture

The State argues that the soil-cement or the cement-treated soils are susceptible to water infiltration through cracks in the slabs, shrinkage cracks between the soil-cement and the structure, or standing water that may form in the rows between the pads. Such infiltration, the State believes, could potentially degrade the soil-cement and underlying soil and affect the soil's ability to maintain the proposed foundation loads. See Findings C.20.

The Board finds, however, that water infiltration will not be a problem at the site for two reasons. First, the Applicant has sufficiently demonstrated that the potential mechanisms of water infiltration are either extremely unlikely or inconsequential. See Findings C.22-.26. In addition, because the storage casks provide a source of heat that will be transmitted downward through the soil-cement, the area beneath the pads will be warmer than the surrounding areas. The warmer soil-cement will cause the moisture to migrate to the surrounding areas and away from the underlying soil-cement. See Findings C.21.

c. Pad-to-Pad Interactions

The State argues that the casks, pads, soil-cement, and the underlying soils have different masses that will behave differently during a DBE. The State insists that the inertial effect of these different masses will introduce tension and compression into the system and cause the various masses to have out-of-phase motions. Consequently, the State contends that because the pad and the soil-cement will be acting out of phase, the weaker soil-cement will act as a strut for the dynamic load and transfer it laterally instead of downward to the underlying soil as predicted by the Applicant. See Findings C.27-.28.

The record before us indicates that a concrete storage pad will not slide in the event of a design basis earthquake. See Findings C.30. One would, therefore, expect the pad and the underlying soils to move together under seismic loadings. In turn, we find that the Applicant has adequately demonstrated that there will be no out-of-phase motion of the pads relative to the underlying soil. In addition, the Applicant performed computer simulations to demonstrate that even if the out-of-phase motions were to occur as hypothesized by the State, the dynamic load transfer between the pads would be minor. See Findings C.31-.32. In sum, the Board finds that pad-to-pad interactions are unlikely to happen during a design basis earthquake and the effects of pad-to-pad interactions, if they were to occur, would be of no safety consequence to the proposed PFS site.

C. Testing of the Soil-cement

1. Adequacy

The Applicant plans to establish the appropriate soil-cement formulation for each of its proposed applications through a series of laboratory tests. These tests include, inter alia, soil index property tests, moisture-density tests, and durability tests. See Findings C.33-.44. All the parties agree that the Applicant has developed a suitable program, based on appropriate industry standards, for testing the properties of the soil-cement. See Findings C.47-.49. The

disagreement centers instead on the timing of the testing. As we discuss below, the State insists that the proposed tests, to demonstrate that the soil-cement will perform as intended, should be performed prior to facility licensing, and the Applicant and Staff believe that such testing can be conducted in a post-licensing period.

2. Proof of Design and Timing

The State argues that allowing the Applicant to defer the testing, analysis, and implementation of the soil-cement, as the Staff would do, effectively truncates the State's hearing rights guaranteed to it by the Atomic Energy Act (AEA). According to the State, the AEA, as interpreted by the Court of Appeals for the District of Columbia Circuit, requires a hearing that offers the intervening party a meaningful opportunity to participate. <u>See</u> [State] Reply to Proposed Findings of Fact and Conclusions of Law of the Applicant and NRC Staff on Unified Contention Utah L/QQ (Oct. 16, 2002) at 27 [hereinafter State Reply] (citing <u>Union of Concerned Scientists v. NRC</u>, 735 F.2d 1437 (D.C. Cir. 1984)). The State contends that by relying upon the Applicant's commitments to provide adequate assurance that the soil-cement will achieve its intended safety function, the Staff is denying the State its opportunity to address the results of the Applicant's final analysis of the soil-cement. State Reply at 27-28.

The State also believes the Applicant's commitments do not address the requirements of 10 C.F.R. § 72.102, which call for a site-specific investigation and laboratory analysis showing that the soil conditions will sustain the proposed foundation loadings. <u>See</u> State Reply at 28. It is the State's opinion that the regulation requires an adequate demonstration of soil suitability before the granting of a license. <u>Id.</u>

In support of its argument, the State points to prior Commission decisions, which establish that post-hearing resolution should be employed sparingly and only to resolve minor procedural deficiencies. <u>See</u> State Reply at 32-33 (citing <u>Consolidated Edison Company of New York, Inc.</u> (Indian Point Station, Unit No. 2), CLI-74-23, 7 AEC 947, 951-52 (1974); <u>Long</u>

Island Lighting Co. (Shoreham Nuclear Power Station, Unit 1), LBP-83-57, 18 NRC 445, 543-544 (1983). According to the State, in a previous ruling in this case, the Commission has established the test for determining whether post-hearing deliberations are appropriate to be "whether the NRC Staff inspectors are expected to engage in 'ministerial'-type compliance checks not suitable for hearings or are expected to themselves exercise a form of adjudicatory discretion." State Reply at 33 (quoting CLI-00-13, 52 NRC 23, 33 n.3 (2000)). The State contends that the analysis and tests proposed by the Applicant, which will be reviewed by the Staff, are far too complex to be "rubber stamped" by Staff inspectors. See State Reply at 36-37. Thus, the State urges the Board to require the Applicant to conduct its tests, the results of which would be subject to further adjudicatory proceedings before this Board, prior to the issuance of the Applicant's license. Id. at 40.

In responding to the State concerns, the Applicant insists that there is no regulation that requires testing of the soil-cement prior to the issuing of a license. To the contrary, as the Staff sees it, once the proposed design requirements have been established and accepted, the actual testing of the soil-cement may be postponed until after the license has been issued. See Findings C.52-.53. In this regard, the Applicant notes that, assuming the design requirements for its proposed facility are accepted, the Applicant has committed itself to performing the necessary tests to demonstrate that the soil-cement will meet these requirements. The Applicant believes that these commitments provide us the necessary assurance that the soil conditions at the PFS site will sustain the proposed foundation loadings and that there is no need for us to impose any additional requirements on its pending license.

Commission precedent appears to support the Applicant's position. The former NRC Appeal Board confronted a similar issue when a Licensing Board allowed testing to be conducted after the hearing that confirmed the ability of emergency diesel generators to operate pressurized heaters that the Applicant proposed to use in the event of a reactor

emergency. Metropolitan Edison Co. (Three-Mile Island Nuclear Station, Unit No. 1), ALAB-729, 17 NRC 814, 885-87 (1983). In that case, after a full hearing on the Applicant's proposed design, the Licensing Board had concluded that the pressurized heaters could be connected to the emergency power supply without harming capacity and that the actual tests confirming this proposal could be left to the monitoring of the Staff after the license was granted. Id. at 886. On appeal, the Intervenors argued that the Licensing Board had improperly "delegated" to the Staff responsibility to resolve this "disputed substantive technical" issue. Id. at 885. The Appeal Board found, however, that the monitoring and evaluation of the Applicant's test results does not "involve decisional responsibility, and is within the authority conferred upon the Staff." Id. at 887.

Here we are faced with a similar situation, in which the Applicant proposes to defer the testing of the soil-cement until after the license has been granted. We find this approach to be fully supported by the Appeal Board's decision in Metropolitan Edison Co..

This practice of post-hearing verification finds further support in the regulatory history surrounding the Commission's promulgation of the rules governing ISFSIs in Part 72. In the Federal Register Notice adopting Part 72, the Commission noted that spent fuel storage in an ISFSI is "a simple operation [that] does not require a complex plant and is subject to few controversial technical issues." 45 Fed. Reg. 74,693, 74,964 (Nov. 12, 1980). And for this reason, "a one step licensing procedure requiring only one application and one SAR was adopted in Part 72." Id.

Although we agree in part with the State's concerns and have considered the benefits of a license condition in this situation, we cannot overlook the Commission precedent that weighs heavily in favor of the Applicant and Staff's proposed post-licensing testing of the soil-cement. Thus, given the strength of the support for the Applicant and Staff's position, we find a license condition unwarranted and the Applicant's proposal to test the suitability of the soil-cement after

the issuing of the license, subject to Staff review, to be sufficient to satisfy the requirements of 10 C.F.R. § 72.102.

D. "Unique" Use of Soil-cement

The State argues the Applicant's use of soil-cement at the PFS site is unique. The State contends that although soil-cement may have been used in previous projects, the Applicant's proposal to use the soil-cement to create in shallowly embedded foundations additional seismic sliding resistance to, and stability in the face of, strong ground motions is an unprecedented application. See Findings C.63. Because of this proposed unusual use of the soil-cement, the State insists that the Applicant's proposal to complete testing of the soil-cement post-licensing should be denied and the Applicant should be required to perform its tests prior to the issuance of its facility license. See Findings C.64.

With the assistance of the Staff's review and the State's critique, the Applicant's design has been thoroughly scrutinized over the course of this proceeding.[15] As our discussion on the soil-cement demonstrates, the Board has also carefully reviewed the Applicant's proposal and found the design to be adequate. There is no Commission regulation that requires the suitability of a proposed design, if otherwise found acceptable, to have been demonstrated through prior use. See Findings C.67. Instead, the Board relies on the requirement that the proposed uses -- unique though they may be -- will be completely tested to support its finding that the soil-cement will respond adequately in the event of a DBE at the PFS site.

[15] For a more complete discussion on this matter, see Findings B.1-.76, and C.1-.67.

E. Young's Modulus

The State raises two issues regarding Young's Modulus.[16] First, the State argues that it will be difficult for the Applicant to achieve its design requirements for cement-treated soil that has a minimum compressive strength of 40 pounds per square inch (psi) and a Young's Modulus having a maximum value of 75,000 psi. See Findings C.68. Second, the State asserts that the test to determine the soil's Young's Modulus must be performed using a dynamic rather than a static load, because the static load will be much lower. See Findings C.71.

Contrary to the State's arguments, the Board agrees with the Applicant's expert testimony that obtaining a Young's Modulus of less than 75,000 psi for cement-treated soil with a compressive strength of more than 40 psi is achievable. See Findings C.68-.70. Furthermore, the Board finds immaterial the State's contention that a distinction exists between the static and dynamic loads. As indicated by the Applicant's experts, the important difference is the proper strain level that will be achieved by the proposed test. The Applicant plans to determine Young's Modulus by using the soil strain level as a reference point. Furthermore, the Sandia National Laboratories paper that provided the necessary data for the cask drop analysis used static moduli of elasticity for the soils underlying the pad. This demonstrates a good agreement between the analytical results and the experimental results, which indicates that large-strain moduli are appropriate for such analyses. See Findings C.71. Thus, the Board finds that the Applicant has adequately addressed the State's concerns regarding this issue.

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[16] Named after Thomas Young, the Young's Modulus, defined as the ratio of stress over strain, provides a measure of material stiffness and strength. For more discussion on the Young's Modulus, see Findings C.68-.72.

F. Board Conclusions

After carefully reviewing the evidence presented by all parties, we are confident that the soil-cement and cement-treated soil will adequately sustain the Applicant's proposed foundation loadings. Although the State raised several important concerns during the course of this proceeding regarding the Applicant's use of the soil-cement and cement-treated soil, the Applicant has met its burden in addressing each of these concerns. Thus, the Board concludes that the Applicant's design and use of soil-cement and cement-treated soil will adequately support the facility's anticipated foundation loads.

V. SEISMIC DESIGN AND FOUNDATION STABILITY

A. Overview of the Pad Storage System

1. Proposed Design Concept for the Pad Storage System

The Applicant plans to store the SNF in large storage casks placed on 3 foot thick reinforced concrete pads.[17] Each pad will be 30 feet wide and 67 feet long and will support 8 storage casks, arranged in a 4 x 2 array.[18] The pads will be placed 35 feet apart in the east-west direction and 5 feet apart in the north-south direction. At maximum capacity, the facility will contain 500 such pads. See Findings D.1-.2.

2. State's General Concerns with the Applicant's Proposed Pad Design System

The State contends that the Applicant's design is unprecedented and unconventional and highlights several unproven features that the Applicant relies upon in its design proposal. According to the State, these features include: unanchored casks; acceptance of cask sliding on the pads and use of this as a design feature in its seismic design; shallowly embedded pads on compressible clay; and use of soil-cement as a structural element. Furthermore, the State claims that the Applicant uses the nonlinear cask stability analysis conducted by its cask vendor, Holtec, to support most of its design calculations, which the State asserts is highly sensitive to input parameters. The State notes that there are tests available that could supply the necessary experimental test data to verify the Applicant's nonlinear models and input parameters and that the Board should require such testing rather than allow the Applicant to rely on its asserted "engineering judgment" to support its analysis. See Findings D.3-.8.

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[17] The Applicant has contracted with Holtec to use its HI-STORM 100. For a complete description of the HI-STORM 100 and an analysis of the State's concerns regarding this cask system, see Section VI on Cask Stability, below.

[18] For a more in-depth description of the Applicant's storage pad system, see section 4.2.1.5.2 of the Applicant's SAR.

B. Specific State Concerns with the Applicant's Pad Stability Analysis

1. Concerns with the Applicant's Methodology

The State asserts that the Applicant's design is full of disparate pieces that have evolved in response to cost cutting measures and, therefore, have not been fully integrated into a cohesive and rigorous design. For example, the State notes that there is a lack of independent verification or checks on the input parameters used in various design calculations, because many of the Applicant's consultants received input parameters from other consultants on the Applicant's team without independently verifying the data source. The State also notes concern with the Applicant's decision to treat the foundation soils with cement rather than bypass the weaker Lake Bonneville clay soils and embed the pad in deeper, stiffer soil. In addition, the State believes that the Board should be reluctant to approve the Applicant's analysis of the complex soil properties and its reliance upon engineering judgment concerning the proposed, unprecedented design features. The State also cautions the Board to be hesitant to rely upon Holtec's nonlinear analysis of the soil behavior beneath the proposed foundation system during a DBE. See Findings D.9-.14.

In responding to the State's claims, the Applicant asserts they were never introduced at trial and are outside the scope of the State's original contention. Furthermore, the Applicant argues that the State's claims have no support in the record and should be rejected on this basis as well. See Applicant's Reply to the Proposed Findings of Fact and Conclusions of Law of the State of Utah and the NRC Staff on [Unified Utah L/QQ] (Oct. 16, 2002) at 99-100 [hereinafter Applicant Reply].

The Board need not address the Applicant's complaints on scope and timeliness. For, although we agree that the Applicant has taken a somewhat unconventional overall approach to its design and analysis, we have examined each of the component parts of its approach and found them to pass muster. Although the overall approach may not have previously been

tested in practice, the State has failed to provide any specific evidence demonstrating deficiencies in particular aspects of the Applicant's presentation that would cause it not to satisfy the Commission's licensing regulations. In this circumstance, we cannot rely on general methodological concerns to avoid finding that the Applicant's proposal is adequate to protect public health and safety.

2. Cask Sliding as a Design Concept

As previously indicated, the State challenges the Applicant's reliance upon cask sliding as a mechanism to reduce seismic loadings. According to the State, if the casks were not allowed to slide freely on the pad, the forces transmitted to the pad and the underlying soils would be significantly greater. The State asserts that the sliding of safety structures in earthquake resistance design is not a common practice, especially with no experimental or reliable performance data to support the Applicant's prediction of cask performance and its reliance on a credit for a reduction in forces due to the anticipated cask sliding. See Findings D.15-.16.

The evidence presented during the course of the hearing demonstrates that the sliding of the casks on the concrete storage pads will involve small displacements, and such small displacements will also reduce the loads to which the cask is subjected. See Findings E.17-.18. In any event, we note that sliding is not a feature of the Applicant's design, but is rather a beneficial consequence of cask reaction during a DBE. See Findings D.17-.18. Therefore, we conclude that the record supports a finding that the Applicant's design is adequate.

3. Flexibility of the Storage Pads

The State has two apparent concerns regarding the Applicant's characterization of the flexibility of the storage pads. First, the State contends that the Applicant's pad design has conflicting requirements, i.e., that the pads be rigid enough to allow smooth cask sliding yet flexible enough to allow for tip-over without damaging the cask. In a similar argument, the State

contends that although the Applicant has not demonstrated that the pads are rigid, it takes full credit for a significant amount of radiation damping from a rigid pad, which allows the soils to play a major role in dissipating energy. The State argues that this assumption is contradicted by the Applicant's conclusion that the pads are also flexible enough to absorb a sufficient amount of energy from cask impact to prevent damage in the event of a cask drop or tip-over event. To solve this apparent contradiction, the State notes that the Applicant could have easily quantified the pad stiffness through an industry standard computer program for soil structure interaction analysis, such as SASSI, that analyzes soil structure interaction (SSI) rather than rely on its assumptions. See Findings D.20-.25.

Although the State raises a pertinent issue, the Board is persuaded that this issue has been satisfactorily resolved for two reasons. First, the seemingly conflicting requirements regarding pad characteristics can be resolved by the use of appropriate materials and substantiated by appropriate tests. See Findings D.29. Second, and more importantly, as highlighted by the Applicant, there is no design requirement that the pad be rigid to assure smooth sliding of the cask and, in fact, the effect of pad flexibility on the sliding of the casks is insignificant. See Findings D.27, .30.

Furthermore, regarding the Applicant's use of radiation damping in its analysis, we find that the Applicant has adequately demonstrated through its dynamic analyses that the pads are rigid enough to limit the maximum displacements of the pad during earthquake conditions to on the order of 3/8 of an inch. See Findings D.28. Finally, the Applicant's analysis demonstrates that the effect of the pad's flexibility on its foundation-damping properties is insignificant in the range of frequency important to the cask response. See Findings D.27. Thus, the Board finds the record demonstrates that the Applicant's characterization of the flexibility of its storage pads is sufficient.

4. Soil-Structure Interaction Analysis

According to the State, when an external force caused by an earthquake is applied, both the structure in question and the ground will deform and move in a compatible manner because neither the structural displacement nor the ground displacements are independent of each other. Because of this SSI, the State claims the motion of the foundation will be different from the motion of the supporting soil without the structure located on top. The State argues that the Applicant, in accounting for this notion, has failed to conduct a comprehensive and accurate SSI analysis of the proposed site. Furthermore, the State contends that in response to its concerns, the Applicant relies upon the testimony of the Applicant's expert, Mr. Trudeau, who, the State argues, has no expertise regarding SSI. See State Reply at 41-47.

In response, the Applicant claims that the alleged need for an SSI analysis and the alleged incorrect use of peak ground acceleration in calculating pad stability, are outside of the scope of Utah L/QQ. The Applicant argues that these two issues were raised for the first time in the hearing in the testimony of State witnesses. See Applicant Reply at 112. Despite the Applicant's claim that the State's concerns are new, late, and outside the scope of Utah L/QQ, the Applicant goes on to address and dismiss the merits of these claims. Without ruling on the timeliness of these two issues, we focus our attention on their merits below.

a. Geomatrix Analysis of Soil Column

During its testing of the facility, the Applicant performed a soil column analysis to obtain the strain-compatible soil properties in the free field using a common industry computer program, SHAKE. The State argues that the SHAKE program, being done in the free field, does not account for the SSI. Due to the complexity of SSI, the State contends that a SHAKE analysis cannot not be substituted for an SSI analysis. See Findings D.31-.32.

Although the State argues that the Applicant's testing program should have included an SSI analysis, there is no regulatory requirement for such a test. In that regard, the State does

not claim that the design inputs for SHAKE provided by Geomatrix were incorrect. More importantly, there is no claim by the State that the Holtec analyses of the cask and pad stability were deficient, nor that the pad is ultimately incorrectly designed, due to this particular alleged SSI deficiency. Thus, the Board denies the State's request for an SSI analysis. See Findings D.33.

b. Pad Acceleration

The State also attacks the Applicant's pad stability analysis. According to the State, instead of obtaining the pad acceleration from Holtec in the cask stability design calculations, the Applicant witness Paul Trudeau assumed a peak ground acceleration of 0.7g for a design input in the pad sliding analyses, based upon a presumed high value of radiation damping at the site. Because peak ground acceleration is the ground motion in the free field and does not account for SSI effects, the State argues that the use of peak ground acceleration for the pads is not appropriate for the PFS site unless it is a bedrock site. See Findings D.39. The State contends that its expert on SSI, Dr. Ostadan, also found Mr. Trudeau's assertion of such high damping values unusual for this type of foundation system. See Findings D.40. The State thus contends that the record contains ample evidence to suggest that the actual pad accelerations may be much higher than estimated by the Applicant. See Findings D.41.

During the hearing, the Applicant defended the use of peak ground acceleration by conducting a confirmatory analysis using the forces developed by Holtec cask stability analysis. The factor of safety against sliding of pads would be reduced only by a small amount (from 1.27 to 1.25) when the Holtec data is considered. See Findings D.46. Additionally, after the issue was raised by the State, the Applicant reran its original analysis using the value for horizontal response acceleration suggested by Dr. Ostadan's critique. Using this number, the Applicant determined that, although the factor of safety against sliding would decrease slightly (from 1.27

to 1.22), it would still exceed the 1.1 factor of safety recommended by the Commission. See Findings D.43.

Here, the State did nothing more than suggest that the pad accelerations might be higher than those used in the Applicant's analysis without providing data to demonstrate the actual increase. With the Applicant providing two confirmatory analyses to support its original analysis, the Board finds the Applicant's analysis regarding pad acceleration to be adequate. See Findings D.43-.50.

5. Pad-to-Pad Interaction

During the hearing, the State's experts also argued that the Applicant's analysis did not account for potential pad-to-pad interaction. According to the State, the Applicant's assumption that 100% of the load forces will be transferred straight down to the underlying soil instead of laterally is neither realistic nor conservative given the unprecedented nature of the PFS design. See Findings D.51-.56.

Furthermore, the State argues that the Applicant's analysis incorrectly assumes that the storage pads will move in phase with the surrounding pads and the underlying soil. The State also believes that the Applicant has not accounted for the potential that the underlying soil-cement will act as a strut and transfer the loads horizontally from one pad to another. See Findings D.53. The State contends that this transferring of the inertial force through pad-to-pad interaction could significantly undermine the Applicant's analysis, given what the State claims is the already slim margin for error in the Applicant's design. See Findings D.55.

Moreover, the State argues that the Applicant wrongfully assumed that its high factor of safety against pad sliding will counter any potential for pad-to-pad interaction, because the State's experts contend that the pads can still interact even without pad sliding. According to the State, this seismically-induced interaction can occur between adjacent pads even if the pads do not slide because of two different mechanisms: (a) the weakness, deformability and

lack of uniformity of the soils beneath the pads; and (b) the differences in the number of casks loaded in adjacent pads. The State believes that both of these mechanisms can lead to out-of-phase motion of adjacent pads and to dynamic loadings of one pad on another pad. See Findings D.56.

In response to the State's concerns, both the Applicant and Staff produced experts who insisted that the soils beneath the pad foundations are essentially uniform across the pad emplacement area and have sufficient strength to withstand the forces of the DBE without significant deformation (i.e., seismically-induced strain). The Applicant's testimony established an estimated value of this deformation of the order of 0.1%, which refutes the first aspect of the State's attack, discussed above. See Findings D.57-.58.

Regarding the second mechanism -- the number of casks loaded in adjacent pads -- the Applicant highlights two Holtec simulations that modeled two adjacent pads, 5 feet apart. One pad was fully loaded with eight casks, the other had only a single cask. The simulations also included a representation of the soil-cement between the two pads. In one simulation, the soil-cement between the pads was assumed to retain its integrity and therefore be able to transmit both tensile and compressive forces. In the other the soil-cement was assumed to be cracked and thus able to transmit only compressive forces. See Findings D.60.

In these two simulations, the Applicant also maximized the potential for pad-to-pad forces in the following fashion: (1) no forces were absorbed by the soil-cement; (2) no forces were transmitted downwards to the cement-treated soil and to the soils beneath; (3) no damping was included in the model; (4) a maximum value of Young's Modulus was assumed for the soil-cement; and (5) no credit was taken for the potential crushing of the soil-cement by the forces going from one pad to the other. Yet even with these conservative assumptions to maximize pad-to-pad interactive forces, the maximum estimated force in the soil beneath the pads was less than the minimum required to initiate pad sliding. See Findings D.61.

Finally, the Applicant compared the forces observed above and associated cask motions with prior simulations that did not account for pad-to-pad interactions, and found that cask motions in both cases are of the same order -- mere inches. This, the Applicant argues, resolves in its favor the State's second mechanism for pad-to-pad interactions. See Findings D.61.

Furthermore, the Applicant goes on to address the State's concern that the pad-to-pad interactive forces resulting from the two Holtec analyses referenced above would add to those forces in the Applicant's sliding stability calculation and cause the interactive forces to exceed the available resisting forces, thereby inducing pad sliding. First, the Applicant notes that the Holtec model already accounts both for the seismic forces acting directly on the pads and for the effects of pad-to-pad interaction. In addition, the Applicant explains that the maximum seismic forces acting on the pad and the maximum pad-to-pad interaction forces would occur at different times and, depending on the direction of the pad motion, would not necessarily be additive. See Findings D.64.

In summary, the Board is persuaded that the evidence presented by the Applicant adequately responded to the State's concern of pad-to-pad interaction by demonstrating both qualitatively and quantitatively, as described above, why pad-to-pad interactions do not undermine the Applicant's analysis.

6. Pad Settlement

The State's experts also attacked the Applicant's analysis because of a purported failure to consider long-term pad settlement in its structural design or in its subsequent analysis of pad behavior during a DBE. The State notes that the Applicant's estimation of pad settlement has gradually decreased over the course of this proceeding from an initial 5 inches down to ½ inch during the Applicant's rebuttal case. See Findings D.71-.72. The State believes that a few inches of pad settlement is a significant number, because the Applicant's stability analysis

assumed a perfectly planar surface for its cask sliding and stability analyses. See Findings D.74.

The Board finds the State concerns regarding long-term pad settlement to be unfounded. As demonstrated by the Applicant, given the stiffness contrast between the pads and the underlying soil, the long-term settlement of the pads will likely be uniform, thereby reducing the supposed effect of “dishing”[19] or “tilting.” See Findings D.81. Moreover, it is apparent that the impacts of long-term settlement on the pads will be minimal as well. As noted by the Applicant, the long-term settlement of the pads was computed to be approximately 1.75 inches, and may be realistically expected to be approximately ½ inch. This range of values for pad settlement -- keeping in mind each concrete pad is a structure measuring 67 feet long, 30 feet wide, and 3 feet thick -- introduces a very small angle of tilting or a very small amount of “dishing.” The Applicant also accounted for a slight amount of tilting in its cask stability analysis and the results demonstrated that slight tilting had no effect upon the cask stability. Thus, without further evidence to demonstrate that significant settlement will occur and will have a negative effect on the Applicant’s analysis, the Board finds the Applicant’s design has adequately addressed any potential problems regarding foundation settlement. See Findings D.75-.85.

7. CTB Analysis

The CTB, the parties seem to agree, conforms with the industry’s standards regarding buildings of its design and intended function. There is no concern about potential overturning of the CTB under DBE loadings. Nor is there concern about CTB bearing capacity failure. See Findings D.91. The State’s major concern regarding the CTB is its potential for sliding during a DBE. In that regard, the State argues that the Applicant cannot meet the Commission’s

[19] “Dishing” refers to a phenomenon in which the middle of the pad settles more than the edges, deforming the pad into a concave-up shape.

recommended 1.1 factor of safety against sliding without the buttressing effects of the soil-cement and that the Applicant will not acquire any data that can be relied upon to support its use of soil-cement until after the Commission has issued its license. Thus, the State insists that the proposed tests be completed prior to the licensing of the proposed facility. See Findings D.87-.90.

Contrary to the State's argument, the Board finds the Applicant has adequately demonstrated that the design for the CTB has a sufficient factor of safety to resist sliding in the event of a DBE. Although the Applicant has not completed the analysis for the soil-cement, the analysis done for the CTB does demonstrate that it will be able to meet the Commission's recommended factor of safety of 1.1. Furthermore, even if the CTB were to slide during an earthquake, there will be no safety consequence because there are no safety-related structures connected to the building that could be adversely impacted by the sliding. Finding D.91-.92. For these reasons, the Board finds the Applicant's CTB design to be adequate.

8. Transfer Operations

According to the Applicant's SAR, the Applicant claims that a single cask transfer (transferring a multi-purpose canister (MPC) from the shipping cask to a storage cask) will require 20 hours to complete. See Findings D.93. The State, however, believes that this estimate contains several shortcomings. The State argues that the Holtec study, which the Applicant relies upon to determine its transfer times, is not based upon actual Holtec cask transfer operations, but was intended only to estimate on-site worker dose assessment. Furthermore, the State contends that although the Applicant claims that a single-cask transfer will require 20 hours of operations to complete, those 20 hours will occur over a 3-day period. In addition, the State notes that there is no regulatory requirement prohibiting the Applicant

from leaving a MPC in an unsealed HI-TRAC[20] transfer cask for an extended period of time, such as overnight. See Findings D.96-.100.

The State's argument implies that the alleged shortcomings in the Applicant's estimate could lead to an increase in the dose exposure for site workers and could increase the amount of time the unsealed cask is vulnerable in the event of a DBE. At the hearing, however, the Applicant established that the transfer operation will be completed in 20 hours and has documented the entire operation in SAR Table 5.1-1. See Findings D.98. Furthermore, the Applicant stated that there was no condition under which the MPC would be allowed to remain outside the protection of a shipping or storage cask overnight. See Findings D.99. The State's insistence that there are shortcomings with the Applicant's analysis is not supported by the record. Thus, the Board finds that the Applicant's analysis of the cask transfer operation time is supported by the preponderance of the evidence.

C. Board Conclusions

After carefully reviewing the record before us, we find that the Applicant has demonstrated that the proposed facility's storage pad, CTB, foundation system, and storage casks demonstrate an adequate factor of safety to sustain the anticipated dynamic loads from a 2000-year return period DBE. Although the State raises several concerns in its attempt to support its claim that the Applicant's proposed design is unconventional, unprecedented, and unproven, we find that the Applicant has demonstrated that its seismic design will adequately withstand a 2000-year return period DBE. In that regard, we also find that, contrary to the State claims, no further tests of the Applicant's seismic design or its input parameters are necessary. We find the Applicant's seismic design and proposed foundation system to be suitable to sustain the dynamic loads anticipated as a result of a 2000-year return period DBE.

[20] For a full description of the various components of the MPC, including the HI-TRAC transfer cask, see Findings A.7.

VI. CASK STABILITY

A. General Overview

According to its design proposal, the Applicant intends to store the SNF in the HI-STORM 100 designed by Holtec. The HI-STORM 100 is a massive steel and concrete cylindrical storage cask that surrounds the multi-purpose canister (MPC) in which the spent nuclear fuel (SNF) was sealed at the originating reactor site. Each cask is approximately 20 feet tall and 11 feet in diameter and will weigh approximately 180 tons when fully loaded with SNF. See Findings E.1. The casks are configured with four air inlets at the bottom of the cask and four air outlets at the top of the cask. This configuration circulates the air through the cask and cools the MPC by allowing the air to enter the inlets at the bottom, rise as it is heated by the MPC, and exit the outlets at the top. See Findings E.2.

Holtec used its own computer program, DYNAMO, to conduct the seismic analysis of its HI-STORM 100 at the PFS site. The DYNAMO system was previously approved as support for Holtec's SAR, which was submitted to the Agency in support of Holtec's request for a Certificate of Compliance (CoC) for its HI-STORM 100. Approval required DYNAMO to be validated through a series of NRC Staff tests and experiments. See Findings E.10-.12.

To perform its review of the PFS site, Holtec used the data it received from Geomatrix, a group hired by the Applicant to conduct a soil analysis of the proposed PFS site, to characterize the earthquake excitation and the soil response, but selected its own damping coefficients and spring constants. Using this data, Holtec modeled various configurations of casks, up to eight casks on a pad, using coefficients of friction that varied from .2 to .8. See Findings E.14-.17.

According to the results of the Holtec analysis, the maximum cask displacement for a 2000-year return interval DBE was on the order of 3 to 4 inches with a maximum angle of tilt of about 1 degree. Holtec hypothesized that this result provides a factor of safety in the angle of tilt of approximately 28 against cask tip-over (by observing that the angle of tilt for cask tip-over

is about 29 degrees). See Findings E.18. Holtec also computed several analyses for a 10,000-year return interval earthquake using a computer code, VisualNastran. This analysis demonstrated some cask rotations of roughly 10 to 12 degrees, but these large cask rotations still left the casks with a factor of safety in the angle of tilt against cask tip-over -- of the order of 2. See Findings E.19, .21.

B. Drs. Singh and Soler

To support its cask stability analysis, the Applicant relied upon the expertise of Drs. Krishna P. Singh and Alan I. Soler. Dr. Singh, who holds a Ph.D. and Masters of Science in Mechanical Engineering, is the President and CEO of Holtec. Dr. Soler, who also holds a Ph.D. and Masters of Science in Mechanical Engineering, is the Executive Vice President and Vice President of Engineering for Holtec. Dr. Soler is also the lead structural discipline expert responsible for the design and analysis of the HI-STORM 100 system. See Applicant's Proposed Findings of Fact and Conclusions of Law on [Utah L/QQ] (Sept. 5, 2002) at 14-15 [hereinafter Applicant Findings].

1. Asserted Conflict of Interest for Drs. Singh and Soler

The State argues that Drs. Singh and Soler have a unique interest in the outcome of the hearing, because they have an extensive financial interest in the Applicant receiving a license to construct and operate the PFS facility. Drs. Singh and Soler and one other unnamed individual hold sole interest in the privately owned Holtec company. See Findings E.22. At the time of the hearing, Holtec had only 12 storage casks in use, but if the Applicant's facility is approved, Holtec could sell up to 4000 HI-STORM casks to PFS. According to the State, the sale of these casks could total hundreds of millions of dollars, which would produce a substantial financial benefit for the three sole owners of Holtec. See Findings E.23. Accordingly, the State requests that the Board consider the biases and interest of the Holtec witnesses in our deliberation of the weight to accord their testimony.

In response, the Applicant raised numerous objections to the State's attack on its witnesses. Although the Board recognizes the Applicant's need to defend its witnesses, its claim that the State's attacks are "irresponsible if not reprehensible" is itself overstated. See Applicant Reply at 36. In that regard, the Board notes that the State has a legitimate concern that the potential for such a substantial financial gain may cloud the judgment of the Applicant's witnesses, and that this potential is a legitimate subject of our consideration.

The answer is not to reject the accusations but to scrutinize the witnesses' demeanor and the substance of their testimony with particular care. Having done so, the Board perceives no disqualifying bias on the part of Dr. Singh and Dr. Soler. Indeed, as the Applicant correctly notes, it is typical in Commission proceedings to have equipment vendors testifying on the technical capacity of their equipment, even if those vendors may receive substantial benefits as a result of a decision in their favor. Furthermore, previous Licensing Boards have also refused to act on allegations of bias without substantial evidentiary support. See Long Island Lighting Co. (Shoreham Nuclear Power Station, Unit 1), LBP-85-12, 21 NRC 644, 665 (1985). Thus, although the Board acknowledges the State's concern, the Board finds no reason to disregard the testimony of Drs. Singh and Soler.[21]

2. Experience of Drs. Singh and Soler

In addition to its claims of bias on the part of Drs. Singh and Soler, the State also contends the Board should be mindful of the fact that they do not have any relevant experience regarding site-specific cask stability analysis for sites similar to the PFS site. According to the State, Drs. Singh and Soler do not have previous experience conducting nonlinear seismic analyses of free-standing casks that equal or exceed the expected ground motions for a DBE at

[21] For these same reasons, the State's challenge (see State Reply at 77) to Dr. Soler's use of a program that he authored, DYNAMO, to analyze the cask response to a DBE does not succeed.

the PFS site. Nor do they have experience conducting seismic analyses of sites that store SNF in unanchored casks on concrete pads supported by soil-cement. Finally, the State argues that neither Dr. Soler nor anyone who assisted him in authoring the various cask stability reports had experience in analyzing soil dynamics and foundation design. Thus, the State requests that the Board be mindful of Holtec's limited experience in assessing the weight to be given the various issues the Holtec witnesses address. See Findings E.27-.31.

Although the State raises several claims regarding the purported lack of experience of Drs. Singh and Soler, the Board finds that they have ample experience to support their analyses. See Findings E.32. Both Drs. Singh and Soler have demonstrated specialized technical experience analyzing the response of Holtec storage casks during a seismic event. Although, as noted by the State, the parameters may not have been exactly the same as those expected at the PFS site, the technical experience and expertise demonstrated by Drs. Singh and Soler in their professional careers performing similar types of technical analyses are sufficient to provide the Board with confidence in Drs. Singh's and Soler's analyses in this case. Furthermore, both Dr. Singh and Dr. Soler each have extensive experience in the design, construction, and installation of the HI-STORM 100 system. Thus, the Board finds the witnesses' experience adequate to support their seismic analysis of the HI-STORM 100 at the PFS site. See Findings E.32.

C. Reliability of the Analysis

1. DYNAMO Program

Among its many criticisms of the Applicant's analysis, the State criticizes Holtec for using its DYNAMO code to generate the results in its 2000-year return interval report. According to the State, the DYNAMO program is a "small deflection" program that is not capable of processing large cask rotations. Moreover, the State continues, Holtec has never used the DYNAMO program to analyze the stability of free-standing casks where the ground motions are

equal or greater than those for the 2000-year DBE at the PFS site. See Findings E.33-.35. The State also contends that it did not have ample opportunity to test the reliability of the DYNAMO program due to Holtec's proprietary claim. Thus, the State insists that the Board view Holtec's findings using DYNAMO as suspect. See Findings E.37.

The Board agrees that the DYNAMO program is a small deflection program and is, as the State claims, not capable of handling large cask rotations. Precisely for this reason, however, we find that it was adequate for the seismic analysis of the casks at the PFS site for the DBE. As demonstrated by the Holtec analysis, in the event of a DBE, the casks will undergo only small rotations with displacements of a few inches and maximum rotation of about 1 degree. See Findings E.49. This is well within DYNAMO's capabilities. Furthermore, Holtec performed a confirmatory analysis of the DYNAMO results using another industry recognized program, VisualNastran. VisualNastran predicted small cask displacements similar to those predicted by DYNAMO, thus confirming for the Board that the DBE analysis was well within DYNAMO's capabilities. See Findings E.48-.50.

2. VisualNastran Results

As a result of the State's concerns, raised in Part D of Contention Utah L/QQ, Drs. Singh and Soler conducted additional simulations using the computer program, VisualNastran, to address the issues raised by the State. See Findings E.51. Using VisualNastran, Holtec analyzed eleven different computer simulations that addressed cask reaction to ground motions with varying pad stiffness, damping, and coefficients of friction. The results of the VisualNastran study demonstrate that the casks, even under the worst possible conditions, would not tip over. See Findings E.52.

The State argues, however, that the Board should limit its reliance on the VisualNastran results, because the State was not given an opportunity to challenge sufficiently these results through an informed cross examination. According to the State, no document in evidence lists

all of the input values for the VisualNastran simulations. Without these input values, the State insists that its ability properly to cross examine the Applicant's witnesses on the results of the VisualNastran analysis was severely limited. See Findings E.55-.56.

In addition, the State argues that the VisualNastran results are not testable. The State notes that when asked, Dr. Soler could not produce the critical damping value for a specified analysis case nor could he remember the equations for the equilibrium of rigid bodies built into the VisualNastran code. Finally, the State criticizes Holtec for failing to identify the actual deflection or angle of rotation for each of the casks used in its VisualNastran simulations. See Findings E.55-.56.

Because of the limited data presented by the Applicant to support its VisualNastran analysis and the "untestability" of the VisualNastran results, the State argues that its ability to probe Holtec's results has been severely limited. Thus, the State requests the Board to limit the support that it finds in the Holtec VisualNastran analysis. See Findings E.57-.58.

The Board finds the State's challenge to Holtec's use of VisualNastran to fall short. The Board notes that although the State did not have the input parameters for the VisualNastran simulations prior to the hearing, the Applicant did provide the State with these reports during the hearing.[22] Furthermore, as indicated by the Applicant, Dr. Soler was not able to produce the percentage of critical damping used in the analysis because the VisualNastran program uses the actual damping values instead of the percentages. In that regard, Holtec did provide the actual value of critical damping for each analysis run. See Findings E.62. Moreover, regarding the State's request for the equations for the equilibrium of rigid bodies, the Board notes that the Applicant's witness responded by claiming that these equations were built into the computer

[22] The Board recognizes the State's frustration with being given the Applicant's report at trial without as much time as ideally could have been used to prepare with such a report. The Board does note, however, that because the trial spanned the course of several months, the State did have the opportunity to use the data during recross at the later stages of the trial.

program. Therefore, he did not need to remember the equations, because they were performed by the commercially-acquired, preprogramed VisualNastran computer program. See Findings E.63. Finally, the Board notes that although the Applicant did not provide the cask displacements for each of the casks, Dr. Soler did provide the displacements for one of the casks and offered, if needed, to provide the displacements for the remaining casks.[23] See Findings E.59-.60.

Although the Board has addressed each of the individual claims raised by the State, the Board stresses that such a response was not necessary for the 10,000-year return interval earthquake. As indicated by the Applicant, the VisualNastran program was used to predict the response of the pad system during a 10,000-year return interval earthquake. As we discuss later, because the Staff granted the Applicant an exemption which we also find in this order to be adequately justified, the Applicant did not need to confirm that the pad system would resist such an earthquake.

For these reasons, the Board finds that the Applicant's use of VisualNastran was technically sound. The Applicant's uses of VisualNastran (a) to confirm that the seismic responses of the casks during the 2000-year return interval DBE involve only inches of displacement as predicted by DYNAMO; and (b) to determine if the casks would tip over under the 10,000-year earthquake under different bounding, worst case assumptions, have been well justified.

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[23]The Board delayed its request for the additional displacement values due to the amount of time required to produce such values. As the hearing progressed, the Board realized that such values were not necessary for the Board to determine that the VisualNastran analysis was adequate. For this reason, the Board saw no need to require Dr. Soler to produce the additional results as offered.

3. Input Parameters

In addition to challenging the use of DYNAMO and the testability of VisualNastran, the State also asserts that nonlinear analyses -- such as the one used by Holtec to analyze the seismic stability of the PFS site -- are sensitive to some input parameters, particularly the choice of contact stiffness and damping value. We address the State's concerns below.

a. Contact Stiffness

The State's expert Dr. Khan argues that nonlinear mathematical models are highly sensitive to an assumed contact stiffness between the cask and the storage pad. According to Dr. Khan, high contact stiffness values can absorb high amounts of energy before actual sliding occurs, thereby reducing instantaneous velocities. As a result, Dr. Khan argues that high contact stiffness can cause a study to underestimate the actual vertical displacement of the subject casks. See Findings E.67-.68.

Dr. Khan acknowledged that prior to his study, in preparation for this proceeding, he had never attempted to select a contact stiffness value for cask sliding or tipping, but the State argues the same can be said for the Applicant's experts, Drs. Singh and Soler. See Findings E.70. In that regard, Dr. Khan insists that the contact stiffness selected by Holtec is too high because it makes the vertical frequency of the casks too rigid, which in turn may underestimate the vertical displacement of the casks. Furthermore, Dr. Khan notes that the Applicant has offered no test data, such as the results of a shake table test, to support its stability results. A more appropriate contact stiffness for unanchored casks, such as those proposed at the PFS site, Dr. Khan contends, should correlate with a frequency that falls within the amplified range of the response spectra curve. See Findings E.71-.73.

Using an established method for calculating contact stiffness between two objects, Holtec calculated the contact stiffness used in its DYNAMO design basis analysis for the PFS site. This calculation is consistent with the guidance given in the computer code ANSYS training

manual. See Findings E.81. During their testimony, Drs. Singh and Soler noted that as a practical matter, analysts often use contact stiffness values that are much less than the actual calculated value of the stiffness to reduce the computing time necessary to compile the results, but yet not so low as to corrupt the results. For the 2000-year DBE, Holtec was able to arrive at a converging solution even when it used a contact stiffness value high enough to resemble its actual value. Hence, Holtec did not significantly lower the contact stiffness value. See Findings E.91-.92. For the 10,000-year earthquake, because of the amount of data involved, Holtec used a low contact stiffness value to reduce the amount of computing time using VisualNastran. But Holtec notes that it did several test runs to ensure that the lower contact stiffness would not significantly alter the test results. Holtec observes that there is a relatively large range of contact stiffness values over which the solution does not change appreciably.   See Findings E.90-.92.

The Board finds that the Applicant's use of a lower contact stiffness value was, contrary to the State's claims, technically defensible. The Applicant's argument demonstrates that its use of contact stiffness values was justified both on fundamental principles and practical engineering considerations. The Board is persuaded that the Applicant has performed its analysis in a deliberate and rational way to achieve an appropriate balance between obtaining computational convergence and ensuring the integrity of its technical solution. See Findings E.91-.92.

b. Damping Values

For its 2000-year DBE analysis, Holtec used a 5% value of damping to represent the dissipation of energy that occurs when the cask impacts the concrete pad during a seismic event. This 5% damping value is used in Holtec's analysis to account for the loss of energy in the impact between the cask and the pad, but is not involved in any horizontal sliding calculations. See Findings E.93. For its 10,000-year earthquake analysis, however, Holtec used a 40% damping value.   Holtec explained this disparity in its input values by noting that an increase in ground motion caused by the 10,000-year earthquake will increase the damping

value between the cask and pad. See Findings E.93. The State disagrees with this claim and notes that an overestimate in the damping value could cause an underestimation of the actual dynamic response of the casks. See Findings E.102, E.104.

Furthermore, the State contends that the Applicant presented no support for the damping values it used. During the hearing, the Applicant explained that it correlated the damping values used in its analysis with a study performed by the NRC Staff that measured the damping value of steel "billets" dropped onto a concrete pad. The State argues, however, that the Applicant has not offered any evidence to support the impact test and to demonstrate how the Staff's steel "billet" test correlates to the HI-STORM casks. See Findings E.97.

In response to the State's concerns, the Applicant's experts produced a computer animation that depicted spheres with varying damping percentages dropped from a height of 18 inches to demonstrate the effects of damping. See Findings E.94-.96. The State argues, however, that the animation does not support the Applicant's damping values. See Findings E.98.

According to the State, although the Applicant's animation varied either the damping value or the contact value, it never varied both. The State contends that such an analysis is thus insufficient to demonstrate that the results are not sensitive to both a low contact stiffness and a low damping value. See Findings E.107. Furthermore, the State argues that because the casks will move horizontally as well as vertically, an animation such as the one used by the Applicant that depicts cask movement only in the vertical direction is not an accurate representation of the actual cask movements. For these reasons, the State believes that the evidence presented does not support the damping values used by Holtec for its 2000-year DBE and 10,000-year earthquake analyses. See Findings E.98.

In response to the State's claims, the Applicant's witnesses turned to the Commission's Regulatory Guides for support. In particular, the Applicant points to Regulatory Guide 1.61,

which allows a greater percentage of critical damping for a safe shutdown earthquake than for an operational earthquake. See Findings E.103.

The Board is persuaded by the evidence before us that a 5% impact damping value is not so high that it supports the State's concern that the high-damping value would underestimate dynamic response of the casks. The State has not offered any specific evidence that established there is a deficiency if a 5% impact damping value is used in the 2000-year DBE analysis. Instead, the State focused its critique on the use of a 40% impact damping value in the 10,000-year earthquake analysis.

The 40% impact damping may well be an acceptable value to use in the 10,000-year earthquake, based on the explanation provided by the Applicant's witnesses (see Findings E.105-.106, E.108-.110); however, the Board finds it unnecessary to rule on this point given that the issue is mooted by the Board's finding that the Staff's granting of the exemption request is appropriate. See below, p. 99.

4. Angle of Rotation

According to the Applicant's expert Dr. Soler, the HI-STORM 100 will tip over if the cask is tilted at an angle of roughly 29 degrees from vertical. See Findings E.111. The State contends, however, that in a publication concerning the HI-STAR 100 system, Drs. Singh and Soler recommended that the maximum rotation of the cask be set to 25% of the ultimate cask tip-over value. Based upon this recommendation, the State argues that the maximum allowable tip-over value for the HI-STORM 100 cask should be 8.15 [24] degrees from vertical and the that Holtec's estimated maximum cask rotation for its 10,000-year earthquake analysis exceeds this maximum allowable cask rotation value. See Findings E.112-.114.

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[24] See Findings E.113.

Assuming that the maximum angle of rotation of 8.15 degrees applies as claimed by the State, the Board finds that the Applicant has demonstrated that the HI-STORM cask will meet this standard in the event of a DBE at the PFS facility. Because of the seismic exemption granted by the Staff, the Applicant need only design its facility to withstand a 2000-year DBE, and the results of the Applicant's analysis for this DBE demonstrate that the maximum angle of rotation from vertical will only be between 1 to 2 degrees, well within the 8.15 degrees limit. See Findings E.117. Furthermore, although the Applicant's study demonstrates that the casks may exceed the 8.15 degrees in a 10,000-year earthquake, the Board finds that the 8.15 degree standard is a conservative lower bound and the analysis demonstrates that the casks will not exceed the actual calculated tip-over value of 29 degrees from vertical. See Findings E.111.

5. Time Histories

Section D.1.h of Utah L/QQ contends that the Applicant's cask stability calculations are insufficient because they use only one set of seismic time histories in their analysis. See above p. 22. The State contends that more than one set of time histories should be used, because non-linear analyses are sensitive to the phasing of input motion and large velocity pulses and their variation and effects are not adequately bounded by one set of time histories. See Findings E.118.

The NRC's regulatory guidance has established two methods for developing time histories.[25] Applicants can either use a multiple set of time histories that in the aggregate envelop the design response spectra or, in the alternative, applicants can use a single set of time histories that envelop the design response spectra as well as power spectral density functions. See Findings E.120. In this case, the Applicant chose the latter and developed a single set of time histories consistent with Agency recommendations. See Findings E.121. The

[25] See NUREG-0800, § 3.7.1 and NUREG-1567, § 5.

Board finds this approach to be adequate and in compliance with regulations. See Findings E.172.

6. Cold Bonding

The State also raised the issue that the Applicant has failed to consider the potential for cold bonding between the cask and the pad and its effects on cask sliding. See above, p. 22. According to the State's witness, cold bonding occurs when two bodies with large loads are in contact with one another for a long period of time. As a result, local deformations and redistribution of stresses may occur, which may create a bond between the two objects. See Findings E.123.

The Applicant argues that cold bonding between the cask and pad will not occur for two reasons. First, the pressure at the interface between the steel base of the cask and the pad is between 26 to 40 psi, well below the allowable bearing stress of 1785 psi in a concrete pad with a compressive strength of 3000 psi. The 26 to 40 psi pressure is comparable to a 200-pound man standing on one foot, hence it is not expected to create cold bonding between the steel bottom of the cask and the concrete pad. See Findings E.124. Secondly, the Applicant highlighted an analysis performed by the Staff, which established that the total deformation in the concrete caused by the cask being on top of the pad to be in the order of 972 micro-inches, a very small deformation insufficient to cause cold bonding. See Findings E.126. Therefore, the Board is persuaded that cold bonding will not occur.

Even assuming that cold bonding may occur, however, the Board finds that it will not have a significant impact on cask sliding. Because the anticipated seismic forces during a 2000-year DBE are significant, the seismic forces would be expected to break a cold bond formed between the cask and the pad, allowing the cask to slide. Thus, the cold bonding effect would be limited. Finally, any cold bonding effects, if they were to occur, would ultimately be subsumed in the Applicant's use of a conservative upper bound coefficient of friction estimate in

which frictional forces represented by a high friction coefficient would certainly dominate any cold bonding effects. See Findings E.52, .124-.127. For this reason, the Board finds cold bonding will not materially affect the casks during a DBE.

D. Khan Report

In response to the Holtec study, the State requested that its expert, Dr. Mohsin Khan, conduct a parametric study modeling the seismic reaction of a HI-STORM 100 cask during a DBE. Dr. Khan used a finite element structural analysis code, SAP2000, to model a single HI-STORM 100 cask as beam elements in which the base of the cask was depicted as connected to the pad using nonlinear elements.[26] See Findings E.128. Dr. Khan performed several case studies using three mathematical single cask models by varying the input parameters for contact stiffness, coefficient of friction, and damping. See Findings E.129.

The results of Dr. Khan's analysis ranged widely from a cask displacement of several inches to a cask displacement of 40 feet. During the hearing, Dr. Khan admitted that such large cask displacements were unrealistic, but he asserted that his analysis served as a valid example of the sensitivity of nonlinear analyses to input parameters in these types of simulations. Tr. at 7177-79 (Khan). Although his analysis demonstrates the importance of using proper input parameters, he did not demonstrate that the input parameters used by Holtec were invalid. See E.133.

More importantly, as noted by the Applicant's experts, the SAP2000 program, used by Dr. Khan, is not designed to accurately model nonlinearities, such as those involved in large cask motions. Thus, when the SAP2000 computer code, which is a small deflection program, predicts large displacements, the results should be closely examined. See Findings E.131-.132.

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[26] For a complete description of Khan's analysis as well as a full report of the results, see Analytical Study of HI-STORM 100 Cask System Under High Seismic Condition, Technical Report No. 01121-TR-000, Rev. 0 (Dec. 2001) (State Exh. 122).

The Applicant's experts also highlighted the fact that Dr. Khan did not use his models to benchmark his solutions with known classical solutions.[27] See Findings E.133-.135. These flaws in Dr. Khan's analysis are illustrated by several of the SAP2000 results which depicted casks lifting roughly 1 to 2 feet off the ground and moving roughly 30 to 40 feet. See Findings E.133. These casks, which are massive structures that measure 11 feet in diameter, are 20 feet tall, and weigh some 180 tons, are not expected to have this type of movement under the postulated seismic loads. See Findings E.1.

Finally, the Applicant re-ran VisualNastran, using Dr. Khan's input parameters (1 million pounds/inch contact stiffness and 1% damping at the cask-pad interface) for run 3 of Dr. Khan's third model, and did not observe the large displacements Dr. Khan obtained. Instead of observing Dr. Khan's prediction of cask vertical movement of 1 to 2 feet with lateral displacement of 25 feet, the Applicant's analysis showed a slight bouncing of the cask, with lateral displacement of a foot or two. See Findings E.136-.137. Thus, the Board is not persuaded by the results of Dr. Khan's analysis. To the contrary, based on the evidence before us, the Board finds that the Applicant's cask analysis conducted by Holtec is adequate to demonstrate that the casks will not tip over during a 2000-year DBE.

E. State's Request for a Shake Table Analysis

The State claims that the only way accurately to validate the Holtec cask stability analysis is to benchmark the cask displacement with actual shake table test data. See Findings E.139. As the principal support for this assertion, the State points to the Applicant's own expert's

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[27] The NRC Staff required Holtec to benchmark DYNAMO's solutions in the manner of ASME NQA-2a-1990. See Findings E.135.

testimony about the need for validation when conducting complex nonlinear studies such as the cask stability analysis.[28] Thus, the State contends that a proper cask stability analysis cannot be completed without a validation of the analysis results with actual shake table test data. See Findings E.147.

In this instance, the Licensing Board finds that a shake table test is unnecessary to validate the results of Holtec's cask stability analysis. Although the State claims that the Applicant's experts support such an analysis, a review of the testimony indicates that the degree of such support is quite limited.

During his testimony, Dr. Luk indicated that a shake table test might be helpful, but he also stressed that a shake table test of the PFS facility would have limitations that could not be technically overcome. For example, Dr. Luk explained that in-situ soil conditions cannot be recreated on a shake table test, hence one would not be able to incorporate the effects of SSI in a shake table test. Additionally, he declared a shake table test large enough to accommodate a full sized cask may not be available. See Findings E.148.

Dr. Cornell also expressed concerns about a shake table test. During his testimony, Dr. Cornell noted that although a shake table test may provide some useful information, because it is a model with its own uncertainties, it would also introduce a different set of uncertainties to the analysis. Thus, Dr. Cornell stressed that it may not be as useful as the State's experts imply. See Findings E.149-.150.

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[28] See Findings E.139-.141. The State also argues that although the Holtec experts testified at trial that a shake table test was unnecessary, that in a November 1997 letter, Dr. Singh sought Commission funding to validate its analysis through a model shake table test. See Findings E.144. The Board notes that Dr. Singh's request for funding for a shake table test was in conjunction with Holtec's application for a Certificate of Compliance (CoC). The Staff ultimately approved Holtec's application for a CoC for the HI-STORM 100 without requiring a shake table test to validate Holtec's supporting analysis. See Findings Staff Exh. FF. Similarly, the Board is not persuaded that shake table test is necessary here.

In his testimony, Dr. Singh stressed the difficulty in simulating the condition of a free-standing cask on a pad in a shake table test. As a key example, he emphasized the difficulty in experimentally controlling the coefficient of friction between the free-standing cask and the pad to provide a meaningful correlation with a computer analysis. He noted that one could not design a shake table test that accounted for all of the critical variables that were used in the Holtec computer simulation. Absent such correlation, Dr. Singh stated that the use of the shake table test data would not be a "reliable benchmark" for analysis of free-standing casks on a pad. See Findings E.151.

The Board is persuaded by the testimony provided by the Applicant and Staff experts, Drs. Luk, Cornell, and Singh. Although at first glance a shake table test appears to offer some hope of validating a complex, nonlinear seismic analysis of cask displacement, closer examination of the shake table test indicates that it has its own limitations, including: (1) the inability to model SSI; (2) the unfeasibility of controlling the cask-pad friction coefficient; (3) the unavailability of an appropriately-sized table; and (4) the introduction of uncertainties associated with the shake table test itself. Given these limitations and the large safety margin in the Applicant's design discussed throughout this opinion, the Board concludes that a shake table test is unnecessary to establish reasonable assurance that the cask would not tip over during a 2000-year DBE.

F. The Staff-Sponsored Sandia Report Conducted by Dr. Vincent Luk

To confirm the Holtec cask stability analysis, the Staff commissioned Dr. Vincent Luk to conduct an independent evaluation of the PFS project. Dr. Luk directed a team of experts in developing a three-dimensional finite element model of the proposed dry cask storage system to examine the nonlinear dynamic behavior of the casks and to simulate the effects of SSI under the postulated seismic conditions. See Findings E.152. Dr. Luk modeled the casks using three different sets of seismic conditions at the PFS site: (1) the 2000-year return period earthquake

at the PFS facility; (2) the 10,000-year return period at the PFS facility; and (3) a sensitivity study based on the 1971 San Fernando Earthquake (Pacoima Dam record). See Findings E.154.

The results of Dr. Luk's analysis confirmed Holtec's conclusion that the casks would not tip over during either a 2000-year return or 10,000-year return earthquake. Furthermore, the Luk study demonstrated that at no time during either the 2000-year or the 10,000-year earthquake would the casks collide. Dr. Luk's study also showed that the maximum cask rotation during the 10,000-year event simulation was 1.16 degrees, which is well within Holtec's recommendation that the maximum rotation of the cask be set to 25% of the cask tip-over angle of 29 degrees from vertical (which is 7.25 degrees from vertical). Thus, based on the Luk Report, the Staff concluded that Part D.1.i of Utah L/QQ does not present a valid concern. See Findings E.171-.174.

Although the State contests the validity of the Luk study on several grounds, as we discuss below, we do not find its claims to be supported by the record.

1. Conflict of Interest with Study's Advisory Panel

The State first argues that because certain members of the panel who served as advisors for Dr. Luk's study were affiliated with the Applicant, there was a conflict of interest in Dr. Luk's assessment of the PFS facility. During the hearing, Dr. Luk testified that a review panel consisting of three NRC Staff members and four industry representatives provided technical advice and input to his cask stability analysis. According to Dr. Luk, the advisory panel members provided recommendations concerning the analysis methodology and range of input parameters used in his study. Further inquiry by the State revealed, however, that two members of the advisory panel were from Southern Company and Southern California Edison, both members of the PFS consortium. See Findings E.175.

Because two PFS member company employees serving as members of the Luk study advisory review panel was not disclosed until the hearing, the State argues that it had little or no

opportunity to probe the background and influence of the panel members upon Luk's study. As a consequence, the State asserts that, in assessing the Luk study, the Board should weigh negatively the potential conflict of interest from the PFS member companies. See Findings E.176.

Although this potential for a conflict of interest should have been disclosed prior to the hearing, the Board has no reason to believe, based on the record developed at the hearing, that the results of the Luk Report have been tainted. The potential conflict was probed extensively by the State during the trial, and Dr. Luk answered all of the State's questions with apparent candor and objectivity. Dr. Luk indicated that the choice of representatives from Southern Company and Southern California Edison as members of the review panel had noting to do with the PFS proceeding. Rather, the choice was made in anticipation that the Sandia National Laboratory (Sandia) would be performing seismic stability analyses for their respective plants, Hatch and San Onofre, and before it was even contemplated that there would be a PFS-related analysis. Sandia ultimately did perform such an analysis for these two plants. See Tr. at 7081-82 (Luk). By the time the site-specific analysis for the PFS site was initiated, the composition of the review panel was established and already included representatives from Southern Company and Southern California Edison. See id. During his testimony, Dr. Luk also indicated that the review panel did not provide any inappropriate advice. See Findings E.179. In addition, the Staff provided the minutes of the meetings in which the panel discussed the Luk Report, which contained no indication that there was undue influence upon the Luk study from any industry member. See Findings Staff Exh. GG.

Although the Board is fully aware of the appearance of a conflict of interest here, after intensively probing Dr. Luk and the Staff during the trial and examining the record before us, we find no evidence that improper bias has had any actual impact on the report. The advice provided by the review panel -- as described by Dr. Luk under cross-examination by the State

and Board questioning, and confirmed by the written meeting minutes -- does not evidence any inappropriate influence that would taint the technical independence of the Luk Report. See Findings E.178-.179. Indeed, the State did not ask the Board to dismiss the Luk Report because of the alleged conflict of interest, but rather it asked the Board to weigh the potential conflict in our assessment of the Luk Report. See Findings E.176. The Board has done so, both during the hearing and here in this decision, and has concluded that it does not impact the validity of the report.

2. Dr. Luk's Relative Experience

The State also argues that Dr. Luk lacks the necessary experience to develop a credible analysis of the PFS site. According to the State, Dr. Luk's experience in modeling free-standing casks does not include any sites with a design or conditions similar to those proposed at the PFS site. See Findings E.180. In addition, the State argues that Dr. Luk lacks the proper qualifications and experience to conduct a proper SSI analysis. See Findings E.181-.182.

We find no merit in the State's claim that Dr. Luk lacks the experience necessary to conduct a successful analysis of the PFS facility. On the contrary, we find Dr. Luk has extensive knowledge and experience in finite element analysis and in the modeling of nonlinear dynamic seismic behavior. Furthermore, although Dr. Luk may not have been an expert in each and every element of this particular site-specific PFS study, he compiled a team of experts to assist him in those areas in which he lacked expertise. See Findings E.185-.186. Thus, the Board finds no reason to doubt the expert credentials of Dr. Luk or of the specialists he assembled on his team.

3. Comparison of Dr. Luk's Report and the Holtec Report

The State also argues that although Dr. Luk reached a conclusion similar to that reached by Holtec -- namely that the casks would not tip over -- his report does not confirm the methodology used by Holtec. Furthermore, the State notes that like the Holtec analysis results, the results of Dr. Luk's study have not been benchmarked or compared with physical data such as a shake table test. Thus, the State argues that Dr. Luk's report should not be seen as a confirmation of the results of the Holtec analysis. See Findings E.187-.190.

For the reasons stated in our previous discussion on the need for a shake table test, the Board finds no reason to require a shake table test to validate Dr. Luk's nonlinear dynamic analysis. See above, pp. 77-79. The Board is also persuaded by the Applicant's assertion that Dr. Luk's analysis model, the ABAQUS code, has been benchmarked against a wide variety of classical problems. Dr. Luk also verified the analytical results of his study against available test data in the area of damping results against drop test data. See Findings E.194.

Moreover, the Board recognizes that Dr. Luk's report employs an entirely independent method from Holtec to analyze the dynamic behavior of the pad and the casks during earthquake conditions. Rather than undercutting Holtec's results, we believe this difference supports it. Having used an entirely different set of assumptions and methodologies and still arriving at the same result, Dr. Luk's report provides further assurance for Holtec's conclusion that the casks will not tip over. Furthermore, the Board notes that many of the assumptions questioned by the State in Holtec's analysis were not used by Dr. Luk. See Findings E.192-.193. For these reasons, the Board finds the State's concerns to be unsupported.

4. State's Concerns with Luk's Model

The State asserts that Dr. Luk's modeling of the foundation soils at the PFS site has six principal defects: (1) Dr. Luk's analysis models the interface between model elements as a frictional interface rather than the clay soils at the PFS facility; (2) Dr. Luk's model does not

represent the reliance of the PFS design on cohesion from bonding at the interface layers to transfer horizontal seismic loads downwards from the storage pad to soils below; (3) Dr. Luk's model does not account for post-yield behavior of the upper Lake Bonneville clays; (4) Dr. Luk's model should have used soil characteristics developed by Stone & Webster, instead of those by Geomatrix; (5) Dr. Luk's analysis uses a uniform thickness of 2 feet for the cement-treated soil; and (6) Dr. Luk's analysis uses an improper value for the Young's Modulus. See Findings E.197-.207, .215, .218. Thus, the State argues that the Luk analysis does not accurately model the design proposed by the Applicant and its results are not a good verification of the Holtec analysis. See Findings E.206-.207.

Contrary to the State's objections, the Board finds that Dr. Luk's analysis accurately models the design proposed by the Applicant for the PFS facility. In describing the first asserted defect, the State maintained that Dr. Luk's use of an interface coefficient of friction treats the interface layers under the storage pad as granular materials and models the interface nodes as a frictional material such as sand. This, the State asserts, does not at all represent the actual behavior of the clay soils. During the hearing, Dr. Luk responded to the State's claims by explaining that modeling the interfaces above and below the cement-treated soil with a coefficient of friction is a well-established method in finite-element analysis. Such use of a coefficient of friction does not characterize the properties of the materials, but rather that of the interface between the materials. See Findings E.208.

Regarding the State's complaint that the Luk model does not account for the cohesion of the soil, the Board finds that Dr. Luk's model in fact does take the cohesion of the soil into account. During the hearing, Dr. Luk explained that in his finite element soil foundation model, horizontal layers were developed, with specific soil properties to account for the cohesion in the soils. Furthermore, the Board notes that even if the State is correct and the soil was modeled as

cohesionless -- a notion contrary to the evidence of record -- this would only maximize the soil's potential for sliding and provide a more conservative result. See Findings E.211-.212.

With respect to the alleged deficiency of not accounting for post-yield behavior of the upper Lake Bonneville clays, the Board finds dispositive Dr. Luk's testimony during the hearing that his team has examined the post-yield behavior of the upper Lake Bonneville clays and determined that the effects are not significant. See Findings E.213.

Regarding the State's concern that Dr. Luk did not use the soil properties developed by Stone and Webster in his analysis, the Board notes that Stone and Webster did not develop dynamic soil properties. Instead, Stone and Webster used the soil properties developed by Geomatrix, which were the same soil properties used by Dr. Luk. More importantly, using the soil properties developed by Geomatrix allowed Dr. Luk to incorporate soil cohesion into his analysis, which addresses one of the State's earlier concerns. See Findings E.214.

The State also asserted that the thickness of the cement-treated soil under the storage pad can vary from 1 to 2 feet, but the Luk analysis assumed a uniform thickness of 2 feet. We agree with the Applicant's explanation that the use of a 2 foot thickness provides a more conservative result because a thicker layer leads to more energy associated with ground excitations going to the storage pad and cask, thus causing a higher level of dynamic behavior of the storage pad and cask. See Findings E.215.

Finally, the Luk Report uses a 270,000 psi Young's Modulus for the cement-treated soil underneath the pads. See Findings E.216. According to the State, Dr. Luk claims to have received this number from Mr. Mahandra Shah, but the State contends that Mr. Shah had no basis for this number. Furthermore, the State argues that this number is arbitrary and incorrect. Instead, the State claims that the Applicant should have used the actual value of the Young's Modulus for the cement-treated soil. See Findings E.218-.219.

The Board is sensitive to the State's assertion that Dr. Luk has committed a significant error in his analysis by using a higher Young's Modulus value (270,000 psi instead of 75,000 psi called for by the design) and for that reason this issue was thoroughly examined during the trial. Although the Board finds that indeed it was an input error for Dr. Luk not to use the 75,000 psi design Young's Modulus, as it turns out such an error produces a more conservative result. The higher Young's Modulus value increases the seismic loads transferred from the underlying soils to the storage pad and cask, leading to greater cask displacements. The Board also notes that Dr. Luk has determined after careful examination that his analyses are not critically sensitive to the value of Young's Modulus. See Findings E.221-.222. Thus, the Board is persuaded that Dr. Luk's use of a higher value for Young's Modulus did not adversely impact the validity of his analyses.

G. Board Conclusions

We find that, based on the evidentiary record before us, the Applicant has demonstrated by the preponderance of the evidence that the storage casks will not tip over in the event of a 2000-year return period DBE. We are further assured of this notion by Dr. Luk's independent analysis that similarly demonstrates that in the event of a 2000-year return DBE, the storage casks will not tip over. Although the State raises several important concerns with each of these studies, after a careful review of the record we are assured that the parameters and methodologies are sound. For these reasons, we find that the Applicant has presented sufficient evidence to meet its burden of proof to demonstrate that the HI-STORM 100 will not tip over in the event of a 2000-year DBE.

VII. SEISMIC EXEMPTION REQUEST

A. Background

As previously discussed, the Applicant requested from the Staff an exemption from the NRC regulations that establish design criteria for ISFSI sites located west of the Rocky Mountains range. Without the exemption, the PFS facility would need to be designed to withstand seismic ground motions based on the deterministic criteria for nuclear power plants established in Appendix A, 10 C.F.R. Part 100. See Findings F.1, .5. As indicated in the Applicant's exemption request, the Applicant sought to use a PSHA methodology for a 2000-year mean return period as the design basis earthquake.[29] In October 2000, the Staff approved the Applicant's request. See Findings F.5.

B. Legal Standards Governing the Site-Specific Analysis Necessary to Obtain an ISFSI License

The Commission's regulations governing the seismic analysis and design criteria for an ISFSI are established in 10 C.F.R. Part 72. According to 10 C.F.R. § 72.92, all applicants must evaluate the natural phenomena that may exist or may occur in the proposed facility's region to determine the potential effect the phenomena may have upon the safe operation of the proposed facility. Furthermore, applicants must also design all SSCs important to safety to withstand events such as a potential earthquake. See 10 C.F.R. § 72.122.

Commission regulations also require all applicants to address the geological and seismological characteristics of their proposed site, and for those sites -- such as the proposed PFS site -- that are located west of the Rocky Mountain range, the facility must meet the criteria established in 10 C.F.R. Part 100, Appendix A. Appendix A requires the applicant to determine the maximum credible earthquake, labeled the Safe Shutdown Earthquake (SSE), that could occur at the site, based upon the location of surrounding faults, and design their facility to

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[29] See Findings F.5. Initially the Applicant sought an exemption to use an 1000-year mean return period, but that request was later modified to the 2000-year mean return period.

withstand such an earthquake. This deterministic approach does not take into account the probability of such an occurrence, but rather requires the applicant simply to design against the largest credible vibratory ground motions associated with the site. See 10 C.F.R. Part 100, App. A.

As previously noted, PFS has requested an exemption from the deterministic approach established in Part 72 to allow it instead to use a probabilistic analysis to determine the appropriate seismic evaluation and design standards. A probabilistic analysis allows the applicant to account for both the intensity and likelihood of occurrence of the postulated seismic event, while the deterministic approach uses the maximum credible earthquake as its design basis SSE. Commission regulations allow such an exemption if the Commission determines that the exemption is 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. Pursuant to this criteria, and after an extensive evaluation, the Staff accepted and granted the Applicant's request for such an exemption.

C. Basis for the Applicant's Exemption Request

The Applicant's exemption request relies upon two key elements: (1) a risk-informed approach that is applied to the seismic design and (2) "risk reduction factors," which are significant conservatisms embedded in the design codes, standards, and procedures. We address both of these elements below.

1. Use of a Risk-Informed Seismic Design

The first principle of a risk-informed seismic design is the use of a risk-graded approach to the design. The risk-graded approach imposes graded requirements on a safety structure. Under this approach, facilities and structures with more severe failure consequences are required to have low probabilities of failure, while facilities and structures with lesser failure consequences can have larger probabilities of failure. In other words, more important facilities

and structures are designed to fail less frequently, while less important facilities and structures are allowed to have a higher failure probability. See Findings F.33. The Staff adopted this approach in approving the Applicant's exemption request, because the Staff determined that the radiological consequences of a failure at an ISFSI would be less than a design basis accident at a nuclear power plant. The Staff's determination primarily rests on two key considerations, namely that an accident involving a spent fuel storage cask would have a much smaller fission product inventory than a nuclear power plant and that such an event would also involve lesser dispersal forces than a nuclear power plant. As a result, the Staff determined that the seismic requirements for the licensing of an ISFSI do not need to be as strict as those for licensing a commercial nuclear power plant. See Findings F.34-.35.

2. Use of Risk Reduction Factors

The second principle of the risk-informed seismic safety analysis is to apply a "two-handed approach" to assess seismic safety. This "two-handed approach" involves the consideration of both the mean annual probability of exceedance (MAPE)[30] of the DBE and the level of conservatism incorporated in the design codes, standards and procedures (also referred to "risk reduction factors").[31] See Findings F.36. Under this "two-handed approach" if there is significant conservatism in the second hand (risk reduction factors), then a lower standard can be permitted to be set by the first hand (MAPE). See Findings F.37.

The State has raised concerns regarding the sufficiency of the risk reduction factors in four general areas: (1) the applicability of risk reduction factors in a nuclear power plant to an

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[30] The MAPE is exactly the inverse of the mean return period (MRP). For example, for a 2000-year MRP earthquake, the MAPE is 1/2000 = 5 x 10^{-4}. See Findings F.39.

[31] A risk reduction factor expresses the degree to which the likelihood of failure of a system or a component in a facility is reduced by the conservatism imbedded in the codes, standards, and procedures of the design. For example, a risk reduction factor of 3 would reduce the design basis earthquake MAPE of a facility by the same amount, say from a MAPE of 6 x 10^{-4} to 2 x 10^{-4}.

ISFSI; (2) the lack of fragility curves for the SSCs; (3) the safety margins involved with the free-standing casks; and (4) the safety margins for the CTB. We address each of them in turn below.

a. Risk Reduction Factors -- ISFSI versus a Nuclear Power Plant

The State has two challenges regarding the risk reduction factors. First, the State argues that because the design concept of the PFS facility -- namely using soil-cement and allowing free-sliding casks -- is different from that of a nuclear power plant, the risk reduction ratios encompassed in the Standard Review Plan for nuclear power plants, which are in the range of 5 to 20, cannot be utilized for free-standing dry storage casks. In other words, the State contends that the Applicant has failed to demonstrate that nuclear power plant risk reduction factors are applicable to an ISFSI, especially one as uniquely designed as the PFS facility. See Findings F.45-.46.

The State also emphasizes that the 2000-year DBE results in a larger reduction in the seismic demand placed upon the proposed facility than that anticipated under the original deterministic method. Because under the two-handed approach a factor of safety is a function of the capacity divided by the demand, the State argues that when the factor of safety is kept constant, as it is here, and the demand is reduced, than ultimately capacity is also reduced. This, the State asserts, will lead to a smaller design margin for the 2000-year earthquake than for the 10,000-year earthquake. See Findings F.47-.48.

The Board finds relative to those concerns, however, that the Applicant has provided sufficient evidence to support its conclusion that the risk reduction factors in the design, manufacture, installation, and operation of major safety systems in nuclear power plants are applicable to the safety systems in an ISFSI. The risk reduction factors, which primarily rest with the embedded conservatism in the design codes, standards, and procedures, have been shown to be similar both to design procedures and acceptance criteria in ISFSIs and nuclear power

plants. See Findings F.50. In contrast, the Board finds no support in the record for the State's claims of significant dissimilarity. Accordingly, the Board is persuaded that similar levels of conservatism can be expected for SSCs designed for ISFSIs as for SSCs designed for nuclear power plants. See Findings F.53.

b. Fragility Curves for the SSCs

During the hearing, the State asserted that the Applicant did not develop any fragility curves for the SSCs in the proposed ISFSI, which the State insisted would be needed to provide the probability of failure for the SSCs when combined with the seismic hazard curve. See Findings F.54. In its proposed findings of fact, however, the State concludes that although fragility curves would provide some comfort, a properly justified DBE does not mandate the determination of fragility curves. See Findings F.55. The Board views this statement as a State withdrawal of its request for the Applicant to develop fragility curves and thus finds no reason to further consider this issue further at this time.

c. Risk Reduction Factors of Free-Standing Casks

The State also claims that the risk reduction factors for the free-standing HI-STORM casks have not been demonstrated by the Applicant due to the following asserted deficiencies: (1) failure to conduct a probabilistic risk assessment to evaluate the design margins or the consequences of casks tipping over; (2) failure to determine the uncertainties in the SSI involving cement-treated soil; and (3) failure to determine the impact that nonlinear soil behavior will have upon the slope of the hazard curve. See Findings F.56, .60-.61.

In its response, the Applicant relies upon the testimony of its expert Dr. Cornell, who supports the Applicant's assertion that the free-standing casks have a risk reduction ratio of at least 5 based on his analysis showing that the casks, which have been designed for a 2000-year earthquake would not tip over even during a 10,000-year earthquake. See Findings F.66. Furthermore, the Applicant stresses three additional factors that support its claim it has used a

conservative design. First, the Holtec simulations for the 10,000-year earthquake employ bounding assumptions that include (1) a range of values of 0.2 and 0.8 for coefficients of friction, as well as random selection of coefficients of friction; (2) radiation soil damping of 1 to 5%; and (3) a selected stiffness value of the soil springs to provide resonance of the cask-pad system. See Findings F.67. The Applicant also points to Dr. Luk's analysis commissioned by the Staff that, using an entirely different methodology in modeling the 10,000-year earthquake, also shows the casks would not tip over. See Findings F.64. Finally, the record demonstrates that, even if the casks were to tip over, there would be no radiological release. Collectively, the Applicant maintains that these significant margins more than account for the potential uncertainties alleged by the State. See Findings F.65.

From the record of evidence developed by the Applicant and Staff, the Applicant's claim that the free-standing casks have a risk reduction factor of at least 5 is fully supported based on the Holtec and Luk analyses that demonstrate that the casks would not tip over in a 2000-year DBE or a 10,000-year DBE. See F.62-.67. A probabilistic assessment was not relied upon by the Applicant to establish this claim, nor would such an assessment be expected to resolve any disputes between the Applicant and the State because any probabilistic assessment would introduce its own sets of uncertainties. The Applicant's argument is also significantly buttressed by the demonstration of additional conservatisms in the Holtec analysis, including the showing that there would not be any radiological releases even if a cask were to tip over. The potential uncertainties alleged by the State, therefore, do not diminish the risk reduction factor claimed by PFS.

d. CTB Foundations and Storage Pads

The State further asserts that applying a risk reduction factor of 5 to 20 or more for typical nuclear power plant SSCs to the foundations for the CTB and storage pads is not appropriate. The potential foundation failure mechanisms are sliding, loss of bearing capacity,

and overturning. See Tr. at 12,952-53 (Cornell). The State's reasoning is straightforward: there are no engineering calculations to support the Applicant's claim that the CTB foundations and storage pads can withstand a 10,000-year earthquake. See Findings F.68.

The Applicant acknowledges that it has not designed the CTB foundations and storage pads to withstand a 10,000-year earthquake, but insists that such measures are not necessary to claim a risk reduction factor of 5. The Applicant offers an explanation for this position. The Applicant has quantified several major conservatisms with respect to CTB foundations and the storage pads, which would allow the CTB foundations and storage pads to withstand the seismic loads of a 10,000-year earthquake. See Findings F.70-.72. Morever, when realistic considerations are included in the Applicant's calculations, these conservatisms manifest in much higher factors of safety against sliding, loss of bearing capacity, and overturning. For example, the factor of safety of the storage pads against sliding was calculated to be 1.27 in the east-west direction. Taking credit for passive resistance of the soil-cement around the pads would increase this factor of safety to 3.3. Or taking credit for greater soil shear strength under dynamic loading would increase the factor of safety from 1.27 to 1.9. Additionally, there are other conservatisms such as using worst-case static soil shear strength for the entire pad area; the cyclic nature of seismic loadings; and the difference between measured disturbed soil strength and in-situ soil strength. See Findings F.72.

As to the factor of safety against bearing failure, using load combinations allowed by Publication ASCE 4-86 written by the American Society of Civil Engineers, would increase the factor of safety for storage pads against soil bearing capacity failure (using 100% earthquake loads as horizontal loads) from 1.17 to 2.1. See Findings F.73. Taking credit for the dynamic strength of soils would further increase this factor of safety to 3.63. See Findings F.75.

The Applicant has also demonstrated that the factor of safety against pad overturning is 5.6 without taking into account any conservatism. See Findings F.76.

After reviewing the relevant evidence, the Board is persuaded that a risk reduction factor of at least 5 is available for the CTB foundations and storage pads. In this regard, for each of the three potential foundation failure mechanisms -- sliding, loss of bearing capacity, and overturning -- the Applicant has quantified and explained the major conservatisms in its assessment of the factors of safety involved. These conservatisms relate to concrete and verifiable principles such as passive resistance of soil-cement around the pads, dynamic strength of soils, cyclic nature of seismic loads, and the difference between measured disturbed soil strength and in-situ soil strength. The Applicant's quantification and explanation of these conservatisms demonstrate that sufficient margins exist to allow the CTB foundations and storage pads to withstand a 10,000-year earthquake and, therefore supporting a risk reduction factor of at least 5.

e. Transfer Time Estimates

The Applicant's expert Dr. Cornell testified during the hearing that the risk reduction factors for the CTB have been shown to be a factor of 5 to 20 or more because, as he explained during the proceeding, the CTB and its components are typical of buildings at nuclear power plants which have a risk reduction factor in that range. The State, however, challenges this assertion, claiming that the Applicant's presumed risk reduction factor of 5 to 20 is based on an incorrect transfer time estimate. According to the State, the Applicant's anticipated transfer time underestimates the time in which the canister potentially will be exposed and SSCs will be in use during transfer operations. See Findings F.80-.81.

We do not agree. As previously indicated, we find the Applicant's transfer time estimates reasonable, well explained, and accurate. Furthermore, the Board believes that the State's concern here regarding the transfer time estimates overlooks the primary reason the Applicant used a risk reduction factor of 5 to 20, namely that the CTB is similar to nuclear power plant

buildings, which use a risk reduction factor in that range. Thus, the Board is persuaded that the CTB risk reduction factor will be in the range of 5 to 20, which the Board finds to be adequate.

D. NRC Staff's Justification for Granting Exemption

To fully evaluate the Applicant's seismic exemption request, the Staff conducted a technical review of the seismic and faulting hazard investigation proposed by the Applicant. See Findings F.82. Based on its review of the Applicant's analysis, the Staff determined that the Applicant's probabilistic seismic hazard analysis (PSHA) was conservative and adequately assessed the risks to the site.

The Staff's determination that the Applicant's PSHA was adequate was based upon several factors. The Staff found support for the PSHA from previous Commission actions that indicated apparent Commission approval for the use of such a method. The Staff was also influenced by its conclusion that the Applicant used an overly conservative seismic hazard assessment, which added an additional margin of safety to the Applicant's design. See Findings F.97-.98. An example of this conservative estimate can be found in the Applicant's conservative fault mode estimates used to create its hazard result calculations. See Findings F.90. The Staff also found the Applicant's site-to-source distance models used in determining the ground attenuation relationships and its fault-rupture-related maximum earthquake magnitude distributions to be conservative. See Findings F.91.

The Staff's decision to accept the exemption request was also based in part on the Applicant's demonstration that it has used an appropriate probability of exceedance for the DBE used in the design proposal. See Findings F.101. In that regard, the Staff was reassured by the fact that the radiological hazards associated with an ISFSI are much lower than those associated with a commercial nuclear power plant. See Findings F.102. The Staff also found support in previous instances in which seismic design ground motions with an annual probability of exceeding 5×10^{-4} were found to be appropriate. These instances included the Department of

Energy's (DOE) issuance of DOE-STD-1020-94, "Natural Phenomena Hazards Design and Evaluation Criteria for [DOE] Facilities" and the Commission's 1998 approval of a 5 x 10^{-5} MAPE for seismic design ground motions at the TMI-2 ISFSI at INEEL. See Findings F.106. In examining these two instances, the Staff asserted that they provided significant relative technical and regulatory insight in deciding that a seismic design based on ground motions that have a MAPE of 5 x 10^{-4} is appropriate for the proposed PFS facility. See Findings F.107-.109. We examine both the DOE standard and the INEEL exemption in the following sections.

1. DOE Standard

The State attacks the Staff's reliance on the DOE standard, DOE-STD-1020-94, to support its determination and the staff's subsequent disapproval of adopting the standard. Contrary to the State's claims, however, the Staff did not adopt the DOE standard, but instead used it as a reference point for its final decision. See Findings F.110. As the Staff's experts indicated during the hearing, the DOE standard uses a risk-graded approach in establishing the seismic hazard's mean probability of exceedance and the Staff's evaluation of the Applicant's request also relied upon the consideration of risk. See Findings F.111. Furthermore, although the DOE standard has been revised, the Staff, as it indicated during the hearing, is not obligated to follow such revisions. See Findings F.112-.113. For this reason, the Board finds that the revision of DOE-STD-1020-94, in DOE-STD-1020-2002 did not affect the Staff's conclusion to grant the seismic request.

2. INEEL Exemption for TMI facility

The State also attacks the Staff's reliance on the exemption approval of the TMI ISFSI at INEEL as support for the Applicant's request, because, according to the State, the facts and site conditions are vastly different for the two facilities. For example, the State notes that the INEEL site is located on a federal reservation that is much larger than the Goshute Reservation and the nearest resident is located tens of miles from the facility. In contrast, the nearest neighbor is

located only a few miles from the Skull Valley site. In addition, the State claims that the anticipated ground motions at the INEEL site are much lower than those anticipated at the PFS facility. For these reasons, the State insists that the INEEL facility cannot serve as support for the Staff's approval of the Applicants's exemption request. See Findings F.115-.116.

The Board finds the Staff was justified in using the INEEL ISFSI as support for granting the exemption request. Although, as the State points out, there are a number of differences between the two sites, the Board finds this does not undermine the support the INEEL exemption provides to the Staff's decision. This finding is based on the observation that while INEEL was designed to a higher ground motion than the 2000-year return period, the Staff's approval of the INEEL exemption was based on the 2000-year return period. See Findings F.120. Thus, the Board finds that the INEEL exemption provides an adequate reference point for the Staff's decision to approve the Applicant's exemption request.

3. The Geomatrix Probabilistic Seismic Analysis

The State contends that the Staff's assertion that the Applicant's PSHA was conservative in nature is founded on: (1) erroneous premises; (2) questionable speculation about what the relative PSHA outcome should have been; and (3) a one-party analysis that is subject to scientific challenge. See Findings F.121. In presenting this challenge, the State notes that the Staff did not conduct its own PSHA. Instead, the Staff reviewed the geological and seismological inputs to the Applicant's PSHA and performed some independent analysis, notably the slip tendency analysis. See Findings F.122. According to the State, the Staff's analysis and its PSHA comparisons do not substantiate the Staff's claim that the Applicant's PSHA results are conservative. As an example, the State points to the Staff's assessment of the slip tendency and conclusion that the Applicant's analysis arrived at a conservative result; the State notes, however, that this blanket endorsement fails to recognize that other experts may draw different conclusions. See Findings F.122-.123. Furthermore, the State attacks the Staff's claim that the

Applicant's PSHA is more conservative than the PSHA for nearby Salt Lake City, because as the State argues, this premise overlooks the fact that the fault sources near Salt Lake City are larger and more seismically active than fault sources near the proposed PFS site. See Findings F.125. As a result, the State argues that although the Applicant's PSHA model may be adequate, there is no support for the Staff's conclusion it is conservative. Thus, the State insists that we reject the Staff's decision to grant the Applicant's exemption request. See Findings F.129.

During the hearing, the Staff has explained its conclusion that the Geomatrix PSHA appears to be conservative in some detail. First, the Staff's expert insisted that the slip rate on the Wasatch Fault (near Salt Lake City) is roughly a factor of ten greater than the slip rate on the Stansbury Fault (near the PFS site), even though the data in the Geomatrix report reflected only a factor of three difference. Notwithstanding this insistence, the Staff indicates that even a factor of three difference represents significant conservatism for the Geomatrix analysis, because a higher slip rate would cause a stronger earthquake. See Findings F.131. In addition, the Staff declares that on a crude basis, a valid comparison can be made to assess whether or not a seismic hazard curve is more conservative than another produced by a different PSHA, without a detailed scrutiny of the two different methodologies used in the PSHAs. The Staff also notes that each of the two seismic hazard curves in question was prepared by professional organizations, the PFS site by Geomatrix, and the Salt Lake City site by U.S. Geological Survey. See Findings F.132.

The Board agrees with the Staff's conclusion that the Geomatrix PSHA is conservative. The issue here does not need to be decided with extreme precision. The Staff's explanations that the slip rate for the Wasatch Fault near Salt Lake City is likely to be 3 to 10 times larger than that of the Stansbury Fault near the PFS site is supported by expert testimony with appropriate analysis and available data. Perhaps more importantly, the Board is persuaded that a reasonable comparison can be made between two seismic hazard curves performed by

competent professional organizations to assess which one is likely to produce a larger earthquake and without the need for detailed scrutiny of every detail in the two different models.

4. Comparison of the Applicant's Design Proposal with Utah's Standards for Highway Bridges

During the hearing, the State also raised concerns because certain Utah buildings and highway bridges are designed for a 2500-year earthquake, which is more stringent than the 2000-year mean return period approved by the Staff for the PFS facility. See Findings F.133. This comparison fails to account, however, for the "two-handed approach" used by the Applicant in designing the proposed facility, which results in the design procedures and criteria for highway bridges designed for a 2500-year earthquake being much less conservative than those specified for nuclear power plant SSCs. See Findings F.134-.135. Ultimately, because the power plant SSCs contain much higher risk reduction factors, these structures are much stronger and able to withstand much larger ground motions than a structure such as a highway bridge that has a more frequent mean return period. See Findings F.136. Although the Board finds that the State was justified in raising this concern, we conclude, based on our exhaustive questioning during the hearing, that the State's concerns have been adequately addressed.

E. Board Conclusions

The Board finds that the Staff's approval of the seismic exemption request was justified. Despite the Board's initial discomfort with the perception that the requested seismic exemption would utilize a 2000-year mean return period, less stringent than the 2500-year earthquake standard for certain Utah buildings and highway bridges, we are persuaded that in reality the significant safety margins embedded in the "two handed approach" provide reasonable assurance that the 2000-year mean return period is not only adequate, but is in practice more stringent than the Utah 2500-year standard. Thus, the Utah standard provides no basis for disapproval of the seismic exemption request.

VIII. COMPLIANCE WITH THE RADIATION DOSE LIMITS

The Board finds that the Commission standards governing offsite dose consequence are quite clear. The Commission has established an offsite radiological dose limit of 25 millirem (mrem) to the whole body during normal operations and a total effective dose equivalent of 5 rem for a design basis accident. See 10 C.F.R. §§ 72.104(a), 72.106(b). See also Findings G.2-.4.

During the course of the proceeding the State suggested that a DBE must not result in dose consequences that exceed the standard of 25 mrem established in 10 C.F.R. § 72.104(a). See Findings G.1. The Board finds this suggestion to be without merit. The Commission's regulations clearly establish that in the event of a design basis accident, such as a cask tip-over event or a DBE, the dose consequences must not exceed a total effective dose of 5 rem as established in 10 C.F.R. § 72.106(b). The Board may not hold the Applicant to a more stringent dose limit than what the regulations require.

A. Dose Consequences Analysis Conducted by the Applicant

To examine the normal operational dose rates at the proposed facility, Holtec performed a study in which it determined the direct radiation dose rate at the controlled area boundaries based on the amount of radiation emanating from the sides and tops of 4000 casks. The results of the study demonstrate a maximum dose rate of 5.85 mrem per year.[32]

Holtec also analyzed a hypothetical tip-over event to demonstrate the dose consequences of such an event. According to Holtec's results, a cask tip-over would not release any radioactivity to the surrounding environment. Holtec also determined that a cask tip-over

[32] See Findings G.5. This calculation assumed a 2000 hour/year occupancy rate at the controlled border and that all 4000 casks contained fuel with a burn up of 40,000 MWD/MTU and a cooling time of 10 years. See id.

would cause some local deformation to the cask, but that it would not affect the cask's shielding performance. See Findings G.6-.8.

In addition, Holtec evaluated the radiological dose consequences of a hypothetical multiple cask tip-over event. The results of this analysis also demonstrated some localized deformation to each of the casks, but there was no significant aggregate increase of radiological doses at the facility boundary. This analysis also examined the effect of all the casks tipping over at once and determined that there would be minimal effect on the overall dose consequence rate. See Findings G.9-.11. Thus, the Applicant argues that the proposed design meets the requirements established in 10 C.F.R. § 72.106(b).

1. Time Spent at the Boundary

The Applicant's analysis for radiation dose limits used a 2000 hours/year occupancy time to calculate normal operating dose levels.[33] The Applicant based its calculation on the Commission's regulations, which state that the dose limits are established for any real individual that is located along the boundary of the facility. Thus, the Applicant contends that the dose-limit calculations must take site specifications into account -- namely that there is no one living or likely to live near the facility and therefore the average site worker is the real person targeted by the regulation. See Findings G.13.

Contrary to the Applicant's position, the State argues that the radiation dose limits should be based upon 8760 hours/year. The State's calculations assume that an individual is situated at the facility boundary for 24 hours a day, 365 days a year. According to the State, this calculation is consistent with the language in 10 C.F.R. § 72.106(b) that refers to any individual located near or beyond the facility's boundary. Furthermore, the State contends that its position

[33] This 2000-hour/year level was based upon a worker at the site boundary for 40 hours per week for 50 weeks per year. See Findings G.13.

finds support in the fact that PFS has no ability to keep people from its border because it does not own the property. See Findings G.14-.16.

The Board finds that the Applicant's use of 2000 hours is adequate. The State's reliance on 10 C.F.R. 72.106(b) as support for its calculation is incorrect, because, as previously discussed, 10 C.F.R. 72.106(b) applies to accident dose rates -- not the dose rates for everyday operations. In addition, although the Applicant has no control over the land at its borders, given the current condition of that land and the belief that few individuals will desire such land in the future, the Board finds insufficient support for this argument. Thus, the Board concludes that the Applicant has established adequate support for its use of 2000 hrs/yr as the occupancy time for its radiological dose calculations.

2. Tip-over Analysis

Although none of the studies demonstrate that the casks will tip over, the Applicant and the Staff nonetheless have performed radiological dose calculations assuming that such an event will occur. These calculations demonstrate that, even in the unlikely event that the casks do tip over, any resulting radiological release will be well within the Commission's allowable limits. See Findings G.19-.38.

a. Duration of the Event

During the hearing, the Applicant and the Staff testified that they both assumed that an accident event such as a cask tip-over would not last more than 30 days. The State argues that there is no evidence in the record to support such an assumption. Further, the State argues that the Applicant does not have a contingency plan for uprighting casks once they tip over. In this regard, the State contends that even if a contingency plan were in place, the Applicant could not rely upon such a plan when calculating the radiation dose consequences for a facility accident. See Findings G.19-.20. According to the State, the standards in 10 C.F.R. § 72.106(b) are for facility design, and to allow an Applicant to insert a contingency measure into

its design criteria would violate the Commission's principle of defense-in-depth. See Findings G.22-.23.

Even assuming that the State is correct and an accident could not be fixed in 30 days, the 5 rem dose limit would not be exceeded, given the estimated dose rate, even if it took the Applicant as long as 36 years to correct the situation. Furthermore, the Board finds the State's concern that the Applicant cannot take credit for a contingency plan to rectify an accident scenario is without merit. See Findings G.24. A reasonable assumption in an accident scenario is that the Applicant will take whatever steps are necessary to mitigate the situation -- such as by building a protective berm or by evacuating the surroundings -- as soon as possible. Thus, the Board finds the Applicant's 30-day estimate to be adequate.

b. Multiple Cask Tip-over versus Single Cask Tip-over

The State argues that in the event of a multi-cask tip-over, the orientation of the casks -- namely the number of casks facing in the outward direction -- will have a significant effect on the radiological dose rates. See Findings G.25. Both the State and Staff's analyses of multi-cask tip-over assumed that one full row of casks tipped over with the cask bottoms faced in the outward direction. See Findings G.26-.27. The State contends that assuming this configuration allows the analyst to create a conservative but reasonable estimate of the potential dose levels one might experience in the event of a multi-cask tip-over. The State requests that the Board require the Applicant to redo its multi-cask tip-over analysis using this more conservative configuration. See Findings G.28.

The State's request is without merit. In the event of a multi-cask tip-over event, the orientation of the tipped over casks will likely be random, as assumed by the Applicant's analysis. See Findings G.29. Even if such an event were to result in all of the cask bottoms facing in the outward direction, the radiological dose levels would not exceed the levels

established by 10 C.F.R. 72.106(b). See Findings G.30. Thus, the Board finds the Applicant's analysis to be adequate.

c. Angular Velocity

The State experts also attack the Holtec tip-over analysis because they believe that Holtec incorrectly assumed that the initial angular velocity of a falling cask would be zero. Although the State contests Holtec's angular velocity, the State experts failed to provide an angular velocity of their own. See Findings G.31-.32. Furthermore, we believe that, contrary to the State's claim, the Holtec analysis provides an adequate explanation to support the use of a zero angular velocity. For these reasons, the Board finds the Applicant's use of an initial velocity of zero in its tip-over analysis to be correct. See Findings G.33-.34.

d. Deceleration

In his pre-filed direct testimony, the State's expert Dr. Resnikoff indicated a concern that the top of the HI-STORM cask may decelerate at a rate in excess of 45g. This was based on the understanding that the Commission has placed a 45g limit on the deceleration of the top of the cask in the HI-STORM SAR. During the hearing, however, the testimony of the Staff and Applicant witnesses demonstrated that although the licensing limit is 45g, the spent fuel assemblies have been constructed to withstand accelerations of at least 63g. See Findings G.35-.36. Moreover, the Applicant's analysis demonstrates convincingly that under no circumstances will an accident result in accelerations that exceed 45g. For this reason, the Licensing Board finds Dr. Resnikoff's concern to be unfounded. See Findings G.37-.38.

3. Dose Calculations

Dr. Resnikoff's pre-filed direct testimony also contained two radiation dose calculations that the State asserts refute the dose calculations of the Holtec analysis. Dr. Resnikoff's two calculations were an estimation of the gamma dose released from the bottom of eighty casks that had been tipped over and an estimation of the neutron doses from a cask based on the

amount of water evaporation from the concrete shielding. See Findings G.40. During the hearing, however, Dr. Resnikoff corrected his calculations in numerous places, due in part to his own analysis and due also to the Applicant's cross examination. See Findings G.43-.44.

Due to the discovered errors to Dr. Resnikoff's own testimony in which he admitted that it is unlikely that the accidental dose rate at the facility would ever reach the 5 rem limit, the Board finds his attack on the Applicant's dose rate calculations to be unfounded.

This concludes the narrative portion (Parts I through VIII) of our decision. A table of contents for Part IX, which goes into these matters in more detail, begins on the next page.

IX. DETAILED ANALYSIS OF RECORD AND FINDINGS OF FACT

A. Site Design and Layout

1. Design and Location

A.1 PFS proposes to construct and operate a dry cask storage ISFSI in which up to 4000 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 the PFS 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. Consolidated Safety Evaluation Report for the Proposed Private Fuel Storage Facility (Mar. 2002) 1-1, 1-2, and 5-8 (Staff Exh. C) [hereinafter SER]; Staff Exh. X.

A.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. SER at 1-1.

A.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 2 miles from the site. Approximately 36 people live within

5 miles of the site. No transient or institutional populations are present within 5 miles of the site, and no public facilities are expected to be located in the vicinity. Id. at 2-4.

A.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 SNF 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 1.1-1 of the Applicant's SAR. SER at 1-1.

A.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 (NPPs) 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 4000 storage casks, is passive and does not rely on active cooling systems. SER at 1-1.

A.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 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 CTB to facilitate shipping cask loading/unloading operations and canister transfer operations. SER at 1-1 to 1-2.

A.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. The cask system is a canister-based storage system that stores spent fuel in a vertical orientation. It consists of three discrete components: the 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 twenty-four pressurized water reactor fuel assemblies or sixty-eight boiling water reactor 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. SER at 1-2.

A.8 The spent fuel is loaded into the MPCs at the originating NPP. 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 CTB, 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. SER at 1-2.

A.9 The HI-STORM 100 storage cask is approximately 20 ft. tall (239.5 in.) and about 11 ft. in diameter (132.5 in.); 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 in. 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.

A.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. See SER at 1-2. 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 SER report for the HI-STORM 100 Cask System, issued with the CoC. SER at 1-2.

A.11 Notwithstanding the NRC's issuance of a general CoC for the HI-STORM 100, 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, the Staff also found that the HI-STORM 100 Cask System acceptable for use at the PFS facility under the site-specific license provisions of 10 C.F.R. Part 72. SER at 1-2.

A.12 The Applicant 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 is responsible for the design of the HI-STORM 100. Stone & Webster is responsible for the design of the facility. The Applicant has overall responsibility for planning and design of the facility, using Stone & Webster as a contractor. The Applicant is also responsible for the operation of the facility and for providing quality assurance (QA) services. SER at 1-3.

2. The Geologic Setting of the Proposed Facility

A.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. SER at 2-3, 2-22.

A.14 As summarized in the Applicant's 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 700 kilometers long with elevations up to 6500 ft. above the basin floors. Much of the surface faulting took place at the base of the ranges along normal faults that dip moderately (approximately 60 degrees) beneath the adjacent basins (herein defined as range-front faults), although complex faulting within the basins is also common. SER at 2-28.

A.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 degree. Detailed topographic maps of the region and the site were provided in the SAR. This smooth valley floor contains small washes up to 4 ft. deep and soil ridges up to 4 ft. high. SER at 2-29.

A.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 mudflows or landslides occurring within the last 2 million years (Ma). SER at 2-29.

A.17 The valley floor near the site comprises beach ridges and shoreline deposits interrupted by bedrock outcrops, such as Hickman Knolls rising about 400 ft. 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. SER at 2-29.

A.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 ft. high and 100 ft. wide, were originally mapped as possible fault traces. In the SAR, the Applicant 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. SER at 2-29.

A.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.[34] The differences in these two interpretations lead to differences in the estimated seismic hazard. 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. SER at 2-29 to 2-30.

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[34] 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 and Regulatory Analysis (Sept. 1999) (Staff Exh. Q) [hereinafter CNWRA Report].

A.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. SER at 2-30.

A.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. SER at 2-30 to 2-31.

A.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 2000 to over 8000 ft. SER at 2-22. The Applicant has classified the subsurface material at the proposed site as a relatively compressible top layer, approximately 25 to 30 ft.-thick, that is underlain by much denser and stiffer material. The underlaying layer is classified as dense sand and silt. SER at 2-23.

A.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 thousand years (ka), sedimentation in Skull Valley was dominated by fluctuations associated with lacustrian cycles in the Lake 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. SER at 2-31.

A.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 (approximately 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. SER at 2-31.

A.25 The Applicant has investigated the structural geologic conditions affecting its proposed site, with considerable attention given to the faults and structures identified therein. 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 to 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 to 12.5 miles deep in the crust. SER at 2-31.

A.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 (approximately 60 degrees) planar features following the horst and graben model. SER at 2-31 to 2-32.

A.27 As discussed in the seismic study of the PFS facility conducted by the Center for Nuclear Waste and Regulatory Analysis (CNWRA), the Staff considers that this horst and graben model is conservative for predicting a maximum earthquake potential for these faults. See SER at 2-32; CNWRA Report. Faults that extend all the way to the base of the seismogenic crust define a larger area for earthquake rupture and thus greater maximum magnitude earthquakes than those that terminate into a detachment above the brittle-ductile transition. SER at 2-32.

A.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. SER at 2-32.

A.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. Id. at 2-33.

B. Characterization of Subsurface Soils

B.1 "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." NRC Staff Testimony of Goodluck I. Ofoegbu Concerning Unified Contention Utah L/QQ, Part C [hereinafter Ofoegbu] Post Tr. 11,001, at 3. Specific requirements for the characterization of the subsurface soils are found in 10 C.F.R. § 72.102. See id.

B.2 10 C.F.R. § 72.102(c) states "[s]ites 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 "[s]ite-specific investigations and laboratory analyses must show that soil conditions are adequate for the proposed foundation loading."

1. Subsurface Soils at the Proposed Facility

B.3 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 Joint Testimony of Paul J. Trudeau and Anwar E.Z. Wissa on Section C of Unified Contention L/QQ [hereinafter Trudeau/Wissa] Post Tr. 10,834, at 5-8; SER at 2-55. The locations of the various soil borings, CPTs and DMTs, and test pits at the PFS site are shown in Figures 2.6-2, 2.6-18, and 2.6-19 of the SAR. Trudeau/Wissa Post Tr. 10,834, 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. 10,834 at 5-6.

B.4 The seventeen foundation profiles provided by PFS in Figures 2.6-5 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. 10,834, at 6. These profiles demonstrate the nature, location, and thickness of the various soil layers underlying the proposed PFS site. See Trudeau/Wissa Post Tr. 10,834, at 6-8.

B.5 Based on the information obtained from its geotechnical investigations, the Applicant was able to characterize the subsurface soil profile -- for geotechnical engineering purposes -- as consisting of two layers. See Ofoegbu Post Tr. 11,001, at 6. Layer 1, a relatively compressible top layer that is approximately 25 to 30 ft. thick, consists of a mixture of clayey silt, silt, and sandy silt with occasional silty clay and silty sand. See id.; SER at 2-55 to 2-56. As reflected in the seventeen foundation profiles (see PFS Exh. 233, 233A), Layer 1 can be further subdivided into four sublayers (in top-down order): layer 1A, classified as eolian silt, is typically about 3 to 5 ft. thick; layer 1B, a silty clay/clayey silt mixture that varies in thickness from about 5 to 10 ft.;[35] layer 1C, a mixture of clayey silt, silt, and sandy silt, with thickness of about 7.5 to 12 ft.; and layer 1D, a silty clay/clayey silt mixture with maximum thickness of about 5 ft.

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[35] In the Applicant's prefiled testimony on Part C issues, Mr. Trudeau referred to layer 1B as "Layer 2." Trudeau/Wissa, Post Tr. 10,834, 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 ft. below the ground surface). See Tr. at 11,732; 11,815-16; 11,834-35 (Trudeau).

2. Factors of Safety for the Applicant's Design

B.6 The primary purpose of soil characterization is to gather sufficient information on the characteristics, properties, and variability of the soils to establish their capacity to resist foundation loading with an acceptable factor of safety. State of Utah Testimony of Dr. Steven F. Bartlett on Unified Contention L/QQ (Soil Characterizations) [hereinafter Bartlett] Post Tr. 11,822, at 4. In general, factors of safety are expressed as the capacity of the system to resist failure divided by the demand placed on the system by the seismic event and other foundation loads. Id. at 3. The capacity of the foundation is primarily a function of the soil's shear strength and the type, flexibility, and embedment of the foundation. The demand on the system is primarily a function of the intensity (i.e., amplitude) of earthquake strong ground motion and the mass and frequency of vibration of the foundation and the overlying structure. Id.

B.7 The State insists that for extreme environmental events, such as earthquakes, a factor of safety of at least 1.1 is considered inviolable. Tr. at 11,845-48 (Bartlett). During the hearing, the State pointed out that a factor of safety of 1.1, or 10%, is widely used in the engineering profession (Tr. at 10,802 (Ebbeson); 6163-64 (Trudeau)); it is the acceptance criterion in NUREG-0800, § 3.8.5, Section II, Subpart 5, Structural Acceptance Criteria for Seismic Category I Structures, at 3.8.5-7 (State Exh. 93); and has been adopted by PFS as a minimum design requirement in its seismic stability calculations for the storage pads and CTB. Stability Analysis of Canister Transfer Building (PFS Exhs. VVV) at 21-24 [hereinafter CTB Analysis]; Stability Analyses of Cask Storage Pads (PFS Exhs. UUU) at 22-26 [hereafter Pad Analysis]; Tr. at 6163, 6169 (Trudeau).

B.8 The State challenges the Applicant's demonstration that the dynamic forces and the capacity of the soils have been properly described and used in the Applicant's calculation of a 1.1 factor of safety against sliding, bearing capacity, and overturning of the pads and the CTB.

Tr. at 11,845 (Bartlett). According to the State, the Applicant is relying primarily on the shear strength of the soils to resist earthquake forces. See Tr. at 11,849-50 (Bartlett). In its seismic calculations, the Applicant computes factors of safety against sliding of 1.27 for the pads and 1.26 for the CTB, and 1.17 against bearing capacity failure of the pads for its design basis case. Bartlett Post Tr. 11,822, at 5-6; Tr. at 11,843 (Bartlett). The State argues that, based on the PFS calculated design values, there is a 6 to 15% margin in the PFS assumed capacity of the soils used in its design calculations before it would reach unacceptable performance. Bartlett Post Tr. 11,822, at 5-6. For this reason, the State insists that the soundness of the Applicant's sampling, characterization, analysis, and testing program of site soils is critical to the Applicant's demonstration that the site soils are adequate for the proposed foundation loadings and to show an adequate margin of safety against potential failure during an earthquake. See State Findings ¶ 11.

B.9 Although the State contends that a factor of safety of at least 1.1 against the various soils failure modes in an earthquake is "inviolable," the NRC Staff testified that it is not necessary to meet a factor of safety of 1.1 against soils failure to satisfy NRC regulatory requirements in 10 C.F.R. Part 72. Tr. at 6594-96 (Ofoegbu). As explained by the Staff, all that Part 72 requires is that the SSCs important to safety be shown to perform their safety functions when subjected to seismic loadings. Id. The foundation stability analyses performed by PFS demonstrate that this condition will be met, whether or not the factor of safety guidelines are satisfied. Id.

B.10 The Board finds that the Applicant's foundation stability analyses demonstrate that there are sufficient margins against the onset of soil failure. The minimum factors of safety calculated by PFS against sliding and bearing capacity failure of the pads are 1.27 and 1.17 (or 27% and 17%) respectively. Rebuttal Testimony of Paul J. Trudeau to Testimony of State of Utah Witness Dr. Stephen F. Bartlett on Section C of Unified Contention Utah L/QQ (Soils

Characterization) [hereinafter Trudeau Soils Reb.] Post Tr. 11,954, at 2. These factors of safety are well above the Commission's recommended 1.1 factor of safety. Further, the Applicant's foundation stability analyses were performed utilizing conservative assumptions. If those assumptions were replaced with more realistic ones, we believe the analyses would show even larger margins of safety. See id. at 2-5.

3. Importance of the Shear Strength of the Upper Lake Bonneville Clays

B.11 The parties agree that the soils in the upper Lake Bonneville clay layer are the soils of interest for establishing the minimum value of soil strength; however, the parties disagree as to the extent to which an "accurate" computation of the strength of those soils is necessary.

B.12 The State insists that an accurate and adequate characterization of the upper Lake Bonneville clays is essential to the PFS demonstration that the pads and CTB will be supported on a stable foundation during a seismic event. Consequently, the State argues that PFS must show that both the seismic performance and the shear strength characteristics of these soils throughout the pad emplacement area and footprint of the CTB are well-defined and understood. See State Findings ¶ 13.

B.13 PFS focused its soils investigations -- borings, samplings, and laboratory tests -- on the upper Lake Bonneville clay layer. Due to the Applicant's conservative approach in establishing the minimum strength and other characteristics of the facility's site soils, the Board finds that even if there were some inaccuracies in the determination of the strength of the upper Lake Bonneville clays, the conservatisms built into the Applicant's methodology for determining the soils properties and the factors of safety against soil failure are more than sufficient to assure that the soils conditions are adequate to meet the anticipated foundation loadings. See Trudeau Soils Reb. Post Tr. 11,954, at 2-5.

4. Specific State Concerns with the Applicant's Testing of the Subsurface Soils

B.14 The State's position is that there are fatal flaws in the PFS testing program because, inter alia, PFS has not adequately sampled and tested the upper Lake Bonneville clays or established their stress-strain behavior under the range of cyclic strains imposed by the DBE. Bartlett Post Tr. 11,822, at 5. At bottom, the State claims that by not adequately defining the lateral and vertical variability of the upper Lake Bonneville clays through site-specific investigations and laboratory analyses, PFS has not shown that those soils will have the shear strength to resist earthquake loadings that PFS is relying upon in its seismic stability calculations. See State Findings ¶ 15. The Staff and PFS do not share the State's concerns.

a. Density of Soil Borings

B.15 The State claims that in many respects the Applicant's sampling of the pad emplacement area is grossly deficient. See State Findings ¶ 23. One of the State's primary concerns is that the sampling program does not conform to the density of borehole spacings recommended in Regulatory Guide 1.132, Appendix. C. Bartlett Post Tr. 11,822, at 5-7. PFS admitted that it used Regulatory Guide 1.132 to plan its field and laboratory investigations for the CTB. Trudeau Soils Reb. Post Tr. 11,954, at 6. Unlike borehole spacing used in the pad emplacement area, there is no disagreement that PFS has met the density of boreholes recommended in Regulatory Guide 1.132, Appendix C, for the CTB.

B.16 Appendix C of Regulatory Guide 1.132 provides a table of spacing and depth of subsurface explorations for various types of safety-related foundations. For linear structures, such as a row of storage pads, Regulatory Guide 1.132 recommends a spacing of one boring per every 100 linear ft. for favorable, uniform geologic conditions, where continuity of subsurface strata is found. Even though this Regulatory Guide is specific to NPPs, the State argues that it should serve as appropriate guidance at the PFS site unless PFS has devised a more conservative sampling plan. In addition, the State contends that its position is reinforced by the

fact that PFS makes analogies to nuclear power plant guidance in arguing for the grant on its seismic exemption. See State Findings ¶ 24.

B.17 According to the State, PFS drilled nine boreholes (A1, B1, C1, A2, B2, C2, A3, B3, and C3) in or near the pad emplacement area for the purpose of retrieving samples for laboratory testing and analysis. Bartlett Post Tr. 11,822, at 7. The State calculates borings taken together with the CPT soundings result in a spacing of about one boring or sounding every 221 ft. in the pad area. Bartlett Post Tr. 11,822, at 6. Mr. Trudeau claims that seven borings, in addition to the nine cited in Dr. Bartlett's testimony, were drilled in or near the pad emplacement area (i.e., boreholes A4, B4, C4, D1, D2, D3, and D4). Trudeau Soils Reb. Post Tr. 11,954, at 7-8. Reviewing SAR Fig. 2.6-19, the State notes borings A4, B4, and C4 are south of the rail spur and are about 200 ft. from the edge of the southern-most row of pads. Borings D1, D2, and D3 (outside the eastern boundary of the perimeter fence), and D4 (adjacent to the CTB) are about 375 ft. or more from the edge of the eastern-most row of pads. Regulatory Guide 1.132 recommends borings be spaced one every 100 linear ft.; therefore, the State argues that the Board should not consider those borings to meet the intent of Regulatory Guide 1.132. See State Findings ¶ 25.

B.18 The State argues that PFS used an approximate borehole and cone penetrometer spacing of about 221 ft. for the pad area. State Findings ¶ 26.

B.19 NRC Regulatory Guide 1.132, "Site Investigations for Foundations of Nuclear Power Plants," is not a binding regulatory requirement (and not even a guidance document) for ISFSIs, but only a guidance document issued by the NRC Staff with respect to soils investigations for the foundations of nuclear power plants. See Curators of University of Missouri, CLI-95-1, 41 NRC 71, 97-98 (1995). The applicable regulatory guidance document for Part 72 facilities, which is NUREG-1567, does not provide any guidelines on the number or placement of borings for

foundation analyses. See Standard Review Plan for Spent Fuel Dry Storage Facilities, NUREG-1567 (Mar. 2000).

B.20 Nuclear power generation facilities have larger and more heavily loaded foundations than those of the proposed structures at the PFS facility. Trudeau Soils Reb. Post Tr. 11,954, at 5. They also have several categories of interconnected safety-related systems and components, such as buried piping and electrical power and control systems, which are sensitive to movements of the ground and the enclosing structures. By contrast, ISFSIs have no such interconnected systems. Trudeau/Wissa Post Tr. 10,834, at 13-14; Trudeau Soils Reb. Post Tr. 11,954, at 5.

B.21 For the above-cited reasons, the guidance in Regulatory Guide 1.132 is not directly applicable to ISFSIs, such as the PFS facility. Trudeau Soils Reb. Post Tr. 11,954, at 5. In fact, State expert, Dr. Bartlett acknowledged that Regulatory Guide 1.132 is guidance and not strictly applicable to ISFSIs. Surrebuttal of Dr. Steven Barlett to PFS Witness Paul Trudeau's Rebuttal Testimony on Section C of Unified Contention Utah L/QQ [hereinafter Bartlett Soils Surrebuttal] Post Tr. 11,982, at 3.

B.22 Regulatory Guide 1.132 also 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. 11,001, at 5-6. Further, Regulatory Guide 1.132 states:

> Because the details of the actual site investigations program will be highly site dependent, the procedures described herein should be used only as guidance and should 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.

Site Investigations for Foundations of Nuclear Power Plants, Regulatory Guide 1.132 (March 1979) (PFS Exh. 234) [hereinafter Reg. Guide 1.132].

B.23 PFS elected to follow the guidance in Regulatory Guidance 1.132 with respect to the borings in the CTB because that building is somewhat analogous to a nuclear power plant structure. For the storage pads, however, PFS exercised professional judgment and developed a subsurface investigation program that combined the drilling of boreholes with other activities to the extent warranted by site conditions and the size, loading, and isolation of the storage pads. Trudeau Soils Reb. Post Tr. 11,954, at 6.

B.24 The Applicant based its professional judgment upon extensive investigations conducted at the proposed site. The initial geotechnical investigations at the PFS site were performed in late 1996. The results of those initial investigations were reflected in the initial version (Rev. 0) of the SAR for the PFS facility, which was filed in June 1997. Trudeau/Wissa Post Tr. 10,834, at 4-5.

B.25 PFS performed an initial set of borings in 1996 in the pad emplacement area, following a uniform grid-like pattern, with the borings spaced approximately 600 ft. apart and covering the entire area. Trudeau/Wissa Post Tr. 10,834, at 4-6. Such a grid was subject to supplementation with additional borings, if anomalous or irregular conditions were encountered; however, no such conditions were identified. Trudeau Soils Reb. Post Tr. 11,954, at 6.

B.26 This initial set of borings served to establish that the soil properties were reasonably uniform across the pad emplacement area of the PFS site. Trudeau Soils Reb. Post Tr. 11,954, at 6.

B.27 As the initial borings were made, SPTs were performed that provided estimates of soil strength and compressibility and allowed visual inspection of samples and index property testing

of the samples in the laboratory. The "blow count" values required to drive the standard split-spoon sampler into the soil at various depths were consistent across the pad emplacement area, confirming the Applicant's belief that the subsoil characteristics are uniform and consistent across the pad emplacement area. Trudeau Soils Reb. Post Tr. 11,954, at 6-7. Based on these initial results, PFS confirmed that it was sufficient to drill boreholes in a uniform grid across the entire pad emplacement area, so that all sections of the area were covered. Id. at 6.

B.28 After the initial borings, PFS performed additional soil investigations, including borings in the CTB area and a series of CPT soundings to better assess soil strength and compressibility. CPTs are conducted using a device with an instrumented conical tip that is pushed into the soil and which provides an essentially continuous record of the soil strength by tracking the force required to advance the cone through the soil. Tr. at 11,727-29 (Trudeau). The device also has an instrumented sleeve that advances as the cone tip moves downward and measures the force required to overcome the friction acting on the sleeve and move the sleeve into the ground. Id.

B.29 In 1999, PFS drilled and sampled twelve additional borings in the CTB area and performed thirty-nine CPTs (sixteen of which included measurements of pressure and shear wave velocities in addition to the penetration resistance data), and eighteen dilatometer soundings. Trudeau/Wissa Post Tr. 10,834, at 5.

B.30 Subsequent CPTs yielded essentially the same value of tip resistance for comparable depths at various locations across the pad emplacement area, indicating again that the stratigraphy across the site is uniform. Trudeau Soils Reb. Post Tr. 11,954, at 7.

B.31 The results of the geotechnical investigations conducted by PFS are presented in Section 2.6 and Appendix 2A of the SAR, as revised through April 2001 (Rev. 22). That section, which is 219 pages long plus attachments and appendices, presents a comprehensive description of the

various investigations that have been conducted. It includes geologic maps, profiles of the site stratigraphy, and discussions of structural geology, geologic history, and engineering geology. Trudeau/Wissa Post Tr. 10,834, at 5-6.

B.32 Figure 2.6-5 of the SAR includes fourteen sheets of "foundation profiles" that depict the composition of the PFS facility subsoil layers at various locations in the pad emplacement area, and Figures 2.6-20 through 2.6-22 of the SAR present foundation profiles under the CTB. These profiles cover all safety-related structures and encompass all borings made by PFS in the vicinity of those structures. Trudeau/Wissa Post Tr. 10,834, at 6; PFS Exh. 233, 233A.

B.33 The locations of the borings made to study subsurface conditions at the PFS site are summarized in three location plans (Figs. 2.6-2, 2.6-18, and 2.6-19 of the SAR), which permit correlating the locations of the borings with those of the CPTs and the geological samplings performed by Geomatrix. Trudeau/Wissa Post Tr. 10,834, at 6; PFS Exh. 235.

B.34 The composition of the soils at the PFS site has been well established through the investigations performed by PFS. Tr. at 11,835 (Bartlett). It is undisputed that the soils below 30 ft. or so are dense and have significant strength and very low compressibility, so they are of no concern from the geotechnical standpoint. Tr. at 11,832-33 (Bartlett). This underlying layer is identified in the Staff's SER as "Layer 2." Ofoegbu Post Tr. 11,001, at 5-8.

B.35 All parties agree that the upper Lake Bonneville Deposits are, relatively speaking, the least strong and the most compressible soils in the profile. See, e.g., Tr. at 11,749 (Trudeau); Tr. at 11,788-90 (Ofoegbu); Tr. at 11,834-35 (Bartlett). Beneath the upper Lake Bonneville Deposits is a 10 ft. layer referred to as lower Lake Bonneville Deposits or "Layer 1C," which is siltier and less clayey and strong than the upper Lake Bonneville Deposits. See Tr. at 11,748

(Trudeau); Tr. at 11,836 (Bartlett). Underneath Layer 1C is a 3 to 5 ft. layer of silty clay and clay silt layer, similar to but stronger than the layer 1B material. Tr. at 11,748-49 (Trudeau).

B.36 A determination was made by the Applicant after the initial tests that the soil properties at the PFS site are reasonably uniform in the horizontal direction (that is, across the various site locations). Trudeau/Wissa Post Tr. 10,834, at 6; Tr. at 11,772 (Trudeau). Layer 1B is particularly uniform across the site. Tr. at 11,816 (Ofoegbu); Tr. at 11,884-85 (Bartlett).

B.37 The horizontal consistency of the materials at the site was further demonstrated by the CPT data, which show that the upper soil layers have fairly uniform properties across the pad emplacement area and beneath the CTB. Trudeau/Wissa Post Tr. 10,834, at 7.

B.38 A trench was dug by Geomatrix near the center of the pad emplacement area. Data obtained from that trench confirmed that the soils in approximately the upper 30 ft. of the subsoil are fairly uniform and consistent in the horizontal direction across the site. The site investigations conducted by Geomatrix for PFS are described in the Geomatrix report "Fault Evaluation Study & Seismic Hazard Assessment, February 1999." Trudeau/Wissa Post Tr. 10,834, at 7.

B.39 Drawings known as "geological plates" were prepared by Geomatrix based on its site investigations. Data from the geological plates correlate well with the data on subsurface conditions presented in the foundation profiles developed by PFS. Comparison of the Geomatrix plates with the foundation profiles in SAR Figure 2.6-5 demonstrates that the nature, location, and thickness of the various layers of the profile are essentially the same, thus corroborating the foundation profile data. Trudeau/Wissa Post Tr. 10,834, at 7.

B.40 The PFS facility boring program determined that the pad emplacement area subsurface conditions are uniform, so that they conform to the general guidance in Regulatory Guide 1.132, which states:

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

Reg. Guide 1.132 at 1.132-3 (footnote omitted). Because of the uniform site conditions, there is no need for a denser set of borings. Trudeau/Wissa Post Tr. 10,834, at 14. There is no reason to believe that a denser set of borings would have yielded any different results from those that PFS obtained. Id.

B.41 It is therefore appropriate to characterize the PFS site as “uniform” and thus, if the guidance in Regulatory Guide 1.132 is to be followed, a maximum spacing of borings is sufficient for the adequate characterization of the subsurface conditions. Trudeau/Wissa Post Tr. 10,834, at 14.

B.42 The soils investigations performed at the PFS facility are thus sufficient to properly characterize the site from the geotechnical standpoint and demonstrate that the soil conditions at the PFS site are adequate for the proposed foundation loadings.

b. Continuous Soil Sampling

B.43 The State also claims that PFS has not continuously sampled the upper Lake Bonneville clays with depth. See State Findings ¶ 28.

B.44 Again the State starts with Regulatory Guide 1.132, which recommends continuous sampling in a single boring or when that is not possible, then samples should be taken from

adjacent closely spaced borings in the immediate vicinity to represent the material in the omitted depth intervals. See State Findings ¶ 29.

B.45 Although PFS relies upon CPT data in lieu of collecting samples continuously throughout the upper Lake Bonneville clays to confirm that no weak layers are present, the State contends that CPT is an in-situ test that indirectly measures soil property; it is not a technique for obtaining undisturbed samples for laboratory testing. Tr. at 11,868-69 (Bartlett). CPTs measure the resistance to penetration required to advance the cone shaped tip of the instrument through the soils; these measurements are recorded as tip resistance values and are related to the stiffness and density of the soil. Tr. at 11,728 (Trudeau). The State argues, however, that PFS did not conduct any statistical analysis of the CPT data to determine the variability of the upper Lake Bonneville clays. Nor did PFS analyze the range or standard deviation of the tip resistance across the site. State Findings ¶ 32.

B.46 At the PFS site, the CPT measured the relative stiffness of the soils. The State argues that this data would need to be correlated back to obtain engineering soil properties, such as shear strength. Tr. at 11,947-48 (Bartlett). The State's experts believe that the visual representation of the CPT data on PFS Exhibit 233a is a depiction of the relative stiffness of the tip resistance of the cone penetrometer, not of the shear strength of that layer. See id.

B.47 According to the State, in the pad emplacement area, the bore holes and CPTs are not adjacent to each other; they are spaced tens, if not hundreds of feet apart. Tr. at 11,863-66 (Bartlett).

B.48 The State insists that the 3 to 13 ft. thick upper Lake Bonneville clays is the critical layer that may affect the engineering properties that PFS is relying upon. It claims that the intent of Regulatory Guide 1.132 is to determine whether there are relatively thin zones of weak or

unstable soils in the upper Lake Bonneville clays. The State believes that sampling at 5 ft. intervals in a 3 to 13 ft. layer does not constitute continuous sampling. In addition, the CPTs were performed after the laboratory samples had been taken. Therefore, the State argues that the CPT data could not have been used to select the weakest zone for the laboratory shear strength test program. See State Findings ¶ 36. Further, the State contends that PFS has not demonstrated that it has closely spaced staggered borings in the pad area in which PFS has continuously sampled the depth of the upper Lake Bonneville clays. See Tr. at 11,863-66 (Bartlett).

B.49 For the same reasons presented in our previous discussion, the guidance in Regulatory Guide 1.132 with respect to the taking of continuous soil samples is inapplicable to an ISFSI. Even though the recommendations in the guide are not applicable to the PFS facility, the sampling conducted by PFS in the pad emplacement area through the use of CPTs, which are continuous through the upper 30 ft. of the soil profile, was consistent with the guide's recommendations. (The soils below the upper 30 ft. are much stronger and less compressible than those above, consequently continuous sampling of the deeper soils was not required.) Trudeau Soils Reb. Post Tr. 11,954, at 7-9.

B.50 The State asserts that the sampling program conducted by PFS does not meet the guidance in Regulatory Guide 1.132 because the continuous measurements taken by the cone penetrometer are not "sampling" since no soil samples are recovered for laboratory testing. Tr. at 11,868 (Bartlett). There is, however, no basis for making such a distinction. The purpose of the recommendation in Regulatory Guide 1.132 that continuous sampling be conducted is to identify "[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." Reg. Guide 1.132 at 1.132-5. The soils characterizations conducted by PFS, both

through the drilling of boreholes and the performance of CPTs, establish that no such zones of weak or unstable soils exist at the pad emplacement area or under the CTB. Trudeau Soils Reb. Post Tr. 11,954, at 8-9.

B.51 Such layers would have been detected through changes in the cone tip resistance measured during the CPTs, which included readings at 0.2 ft. intervals within the layer-1B soils at thirty-seven locations across the pad emplacement area. Id. at 9; Tr. at 11,773 (Trudeau). Moreover, continuous, undisturbed sampling of layer 1B soils in boreholes drilled within the adjacent CTB area did not reveal any zones of weak or unstable soils. See Trudeau Soils Reb. Post Tr. 11,954, at 9.

B.52 Therefore, the objectives of Regulatory Guide 1.132 with respect to continuous soil sampling have been achieved. Trudeau Soils Reb. Post Tr. 11,954, at 7-9; Ofoegbu Post Tr. 11,001, at 9.

c. Undersampling

B.53 Shear strength is easily established through laboratory testing. Tr. at 11,840 (Bartlett). The State does not object to the manner in which PFS conducted its laboratory tests for determining the soil shear strength parameters. Tr. at 11,840-41 (Bartlett). However, the State asserts that the Applicant's determination of the minimum shear strength of the soil is inadequate due to undersampling, because PFS estimated the minimum horizontal shear strength of the soils in the pad emplacement area by performing laboratory tests on three specimens taken from a single soil sample. See State Findings ¶ 41.

B.54 According to the results of the Applicant's testing, the soil sample used to measure the minimum horizontal strength of the soils in the pad emplacement area was obtained from the weakest portion of the weakest layer (Layer 1B) of the soil profile. This sample also exhibits the

highest void ratio of all the samples tested in the pad emplacement area (signifying lowest density and hence lowest strength), and it was taken from the quadrant in the pad emplacement area that had been determined to have the lowest soil strength. Trudeau Soils Reb. Post Tr. 11,954, at 9.

B.55 PFS contractor ConeTec took continuous measurements of cone penetrometer tip resistance at thirty-seven locations in the pad emplacement area of the PFS site. The results of those measures are presented graphically in the foundation profiles prepared by PFS, such as PFS Exh. 233A. ConeTec also provided tables of numerical values of tip resistance versus depth, which recorded the actual measurements of tip resistance and allow a numerical correlation to be drawn between the measured tip resistance and the undrained shear strength of the soil at the various locations in the soil profile. Tr. at 11,772-73, 11,955-62 (Trudeau); Tr. at 11,789-91, 11,817-18 (Ofoegbu). The value of undrained shear strength that can be derived from the results of the CPTs for the lowest tabulated value of tip resistance corresponds almost exactly to the value of shear strength measured in the laboratory by PFS for the sample it selected for that purpose. PFS Exh. 238; Tr. at 11,960 (Trudeau). Therefore, the CPT results confirm that the value of minimum shear soil strength determined by PFS in laboratory tests is indeed the minimum value of undrained shear strength found in the pad emplacement area.

B.56 Because of the uniformity of the soils in the horizontal direction, the manner in which the test sample used for determining the minimum value of undrained shear strength was selected, and the confirmation provided by the CPT measurements, it is reasonable to conclude that the value of undrained horizontal shear strength used by PFS represents that of the weakest soils found at the pad emplacement area at the PFS site. Tr. at 11,772 (Trudeau).

B.57 According to the State, the sample for the one borehole taken in the pad emplacement area resulted in a shear resistance, for a vertical stress of 2 kips per square foot (ksf), of about

2.1 ksf. Tr. at 11,937 (Bartlett). From one of the samples taken from the CTB area, the shear resistance for a vertical stress of 2 ksf was about 1.75 ksf. Tr. at 11,938 (Bartlett). The State compares the CTB and pad data, and claims that this comparison should strongly suggest to the Board that the 2.1 ksf used for the design of the pad emplacement area is not the lower bound undrained shear strength of the upper Lake Bonneville clays at the PFS site. See State Findings ¶ 42.

B.58 According to the State, the Applicant's demonstration to overcome the State's claim of gross undersampling in the pad area is based on the following sequence: the direct shear test sample came from one borehole in the northeast quadrant of the site; of all the soils specimens tested in the pad area, the northeast quadrant had the highest void ratio; a high void ratio results in low soil density; and low soil density is evidence of a weak soil. See State Findings ¶ 44. Furthermore, the State insists that there is no apparent reason PFS could not have performed additional direct shear testing on other undisturbed samples from some or all of the other five borings in the pad area. See State Findings ¶ 44.

B.59 Dr. Bartlett, the State's expert, argues against using CPT data alone to predict shear strength. Bartlett Post Tr. 11,822, at 9; Tr. at 11,874-81 (Bartlett).

B.60 According to Dr. Bartlett, to accurately predict undrained shear strength from CPT data, a specific N_K factor in the following equation from EPRI, Equation 4-61 (State Exh. 100), must be developed for the site-specific soils. Equation 4-61, $q_c = N_k s_u + \sigma_{vo}$ gives the theoretical relationship for the cone tip resistance in clay, where q_c is the cone tip resistance; σ_{vo} is the total overburden stress; and N_k is the cone bearing factor, which is empirically determined. Published ranges for N_k from various locations for clayey soils vary from 4.5 to 75. State Exh. 100 at 4-55, 4-47.

B.61 The State argues that Mr. Trudeau relied on an N_k factor developed by ConeTec to predict undrained shear strength from the CPT data. See State Findings ¶ 47. This factor was developed from triaxial compression tests which measure shear in a subvertical direction; however, the mode of failure for sliding at the PFS site is horizontal i.e., direct shear. Thus, the State believes that the ConeTec N_K factor is inappropriate for sliding calculations and likely overestimates the available shear resistance provided by the clayey soils. See State Findings ¶ 47 (citing Tr. at 11,955-62, 11,972 (Trudeau)).

B.62 To counter the Applicant's position, Dr. Bartlett developed some hand-drawn plots of the CPT data published in the ConeTec report to show there is horizontal and vertical variability in the upper Lake Bonneville clays. Tr. at 11,891-92 (Bartlett); State Exh. 99. Dr. Bartlett testified that he had been unable to obtain the electronic CPT data from PFS so that he could refine his plots. Tr. at 11,898-99 (Bartlett). Dr. Bartlett, relying on his past experience in testing Lake Bonneville sediments and his composite plots of CPT data, testified that there could potentially be a difference of a factor of two in the tip resistance and undrained shear strength variability of the upper Lake Bonneville clays across the pad emplacement area. Bartlett Post Tr. 11,822, at 9; Tr. at 11,874-81 (Bartlett).

B.63 Drawing upon Dr. Bartlett's conclusions (as established in his prefiled testimony), the State argues that if one were to assume that one undrained shear strength value (2.1 ksf) that PFS has obtained for the pad area, which is an average value, and taking into account the variability in the CPT logs, then the shear strength values in the pad area could range from 1.4 ksf to 2.8 ksf. See State Findings ¶ 49 (citing Bartlett Post Tr. 11,822, at 7). PFS has a 1.27 factor of safety against sliding for the pad based on 2.1 ksf. An undrained shear strength of 1.82 ksf or less decreases the factors of safety below 1.1. State Findings ¶ 49.

B.64 Dr. Bartlett asserts that there can be considerable horizontal variability in the shear strength exhibited by the upper Lake Bonneville soils across the pad emplacement area and that there may be some location at which the shear strength may be considerably below the 2100 pounds per square foot (2.1 ksf) obtained by PFS in its laboratory tests. See Bartlett Post Tr. 11,822, at 9. The evidence provided by Dr. Bartlett in support of his position, however, are some tracings he made with markers of the cone penetration tip resistance plots, taken off enlarged photocopies of the data plotted in the foundation profiles in SAR Figure 2.6-5. State Exh. 99; Tr. at 11,893-99 (Bartlett). We find these drawings to be insufficient evidence to support Dr. Bartlett's claim. See Trudeau Soils Reb. Post Tr. 11,954, at 10-11. Furthermore, resorting to manual plots like Dr. Bartlett's is unnecessary because a report provided by ConeTec includes tabulations of the actual values of cone penetration tip resistance measured in the tests. See PFS Exh. 238. In fact, the foundation profiles show that the measured cone penetrometer tip resistance varies as one moves downwards (even within a given soil layer) and is remarkably uniform for a given depth from one location to another. Trudeau Soils Reb. Post Tr. 11,954, at 10-11.

B.65 Even assuming that such variability existed, there is no basis for asserting that the value used by PFS is not the minimum shear strength of the soils in the pad emplacement area. If soils of lower strength were to exist in the pad emplacement area, the conservatisms incorporated into the PFS analyses and design (discussed below) would more than compensate for the difference in that hypothetical lower strength and that utilized by PFS in its analyses.

d. Additional Tests (Cyclic Triaxial and Triaxial Extension Tests)

B.66 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. 11,822, at 10-12. Of particular concern to the State is that PFS "ensure that there is no significant degradation of 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 below, we find that PFS has satisfactorily addressed the State's concerns.

B.67 According to the State, earthquake loadings are cyclic in nature with several reversals in the direction of loading during a large earthquake. The State claims that PFS has used the peak undrained shear strength determined from a monotonic triaxial compression and direct shear tests (i.e., one directional loading without cycling) to represent the soil's shear resistance for the design of the pads and CTB foundations. Bartlett Post Tr. 11,822, at 10.

B.68 Dr. Bartlett expressed concern that PFS had not tested the strength behavior of the upper Lake Bonneville clays at a range of high strain levels. This is important because if their shear strength degrades due to earthquake cycling, then there is an additional unconversatism introduced into the PFS sliding calculations. Tr. at 11,992-93 (Bartlett). He suggests that to remedy this defect, PFS could conduct strain-controlled cyclic triaxial tests to ensure that there is no significant loss or degradation of shear strength due to cycling. Bartlett Post Tr. 11,822, at 10-12.

B.69 While PFS did not conduct strain-controlled cyclic triaxial tests, it performed resonant column tests, which are a form of strain-controlled cyclic triaxial tests. Indeed, they are the only form of strain-controlled testing that is recommended in Appendix B, "Laboratory Test Methods for Soil and Rock," to NRC Regulatory Guide 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants" for use in developing curves of shear

moduli and damping versus shear strain. Trudeau/Wissa Post Tr. 10,834, at 18. The resonant column test results can be readily extrapolated to establish the behavior of site soils at higher strains than those covered by the tests, so that all the strains potentially of interest are covered. Id. Therefore, strain-controlled cyclic triaxial tests to measure shear moduli and damping at higher levels of strain than those measured in the resonant column tests are not required. Id. at 19; Tr. at 11,736-39 (Trudeau).

B.70 The site response analyses performed by Geomatrix established that the layer of soil exhibiting greatest effective shear strain is Layer 1B. For the soils in that layer, the effective shear strains under design basis seismic loadings are within the range of strains measured directly in the resonant column tests. Trudeau/Wissa Post Tr. 10,834, at 19; Tr. at 11,736 (Trudeau). For that reason, additional strain-controlled cyclic triaxial tests are unnecessary.

B.71 In addition, PFS conducted stress-controlled cyclic triaxial tests to determine the soil's collapse potential. The results of the tests did not show any degradation of the shear strength of the samples throughout 500 cycles of loading at extremely high cyclic stress ratios. The resulting cyclic strains were very small, indicating an essentially elastic response throughout the tests and demonstrating that there is no strength degradation for these soils due to even higher levels of cyclic stress than those experienced during a DBE. Thus, strain-controlled cyclic triaxial tests are unnecessary. Trudeau Soils Reb. Post Tr. 11,954, at 11.

B.72 Although the State attempts to argue to the contrary, their expert Dr. Bartlett agreed that if one can be assured that there is no marked decrease in shear strength at high levels of strain, the concern about characterizing the dynamic properties of the soil at high strain levels is of no consequence. Tr. at 11,992-93 (Bartlett). He characterized the testing that PFS conducted with respect to this issue at a "C-minus" level, meaning that knowledge in this area could be

improved, but the failure to conduct the strain-controlled triaxial tests was not a fundamental flaw in the PFS program. Id.

B.73 According to the State, PFS has primarily used triaxial compression tests to calculate the soil's resistance to bearing capacity failure and has given no consideration to performing triaxial extension tests to determine the degree of anisotropy of the foundation soils. The State contends that if significant anisotropy is present, then the use of triaxial compression tests overestimates the average shear resistance along the potential failure plane. Bartlett Post Tr. 11,822, at 11-12. The State believes that this issue has the greatest significance in analyzing the bearing capacity of the storage pads, due to their relatively narrow width (30 ft.) and the small margin (i.e., 5%) against seismic bearing capacity failure estimated by the Applicant. Bartlett Post Tr. 11,822, at 11-12.

B.74 As previously discussed, the vertical shear strength obtained by PFS in its triaxial compression tests for the pad emplacement area is 2.2 ksf, and the horizontal shear strength as obtained in the direct shear tests is 2.1 ksf. Hence, the degree of anisotropy exhibited by the PFS site soils is slight, if any. Tr. at 11,973 (Trudeau); Tr. at 12,021 (Ofoegbu).

B.75 The soil failure mechanism is a composite of failures along horizontal and vertical surfaces and is adequately represented by either the horizontal or vertical shear strengths determined by laboratory test results and field measurements. Tr. at 12,017-21 (Ofoegbu). Therefore, the effects of anisotropy are insignificant.

B.76 Dr. Bartlett asserts that performing triaxial extension tests is necessary to properly assess the bearing capacity of the soils beneath the storage pads. Bartlett Post Tr. 11,822, at 11-12. However, such tests typically are not performed to assess the bearing capacity of foundations, nor are they mentioned in Appendix B of Regulatory Guide 1.138. Trudeau/Wissa

Post Tr. 10,834, at 19-20. Moreover, the minimum factor of safety against bearing capacity failure of the storage pads was computed by PFS using many conservative assumptions, including among others declining to use, as is customary, the average shear strength of the soil through a depth of 30 ft. below the base of the pads to determine the bearing capacity. Ofoegbu Post Tr. 11,001, at 7. If this and other conservatisms in the analysis were removed, the calculated minimum factor of safety against bearing capacity failure of the storage pads would be well in excess of the required standard. Therefore, the concerns about soil anisotropy are inconsequential and the asserted need for triaxial extension tests does not exist.

C. Use of Soil-cement and Construction

1. Background and Proposed Uses

a. Design Description

C.1 Soil-cement is a material produced by blending, compacting, and curing a mixture of soil, portland cement, other possible admixtures, and water to form a hardened material with specific engineering properties. Trudeau/Wissa Post Tr. 10,834, at 21. Soil-cement typically has far greater strength than that of the soil that is its main constituent, and thus is used to increase soil strength. Id.

C.2 Some soil-cement mixtures are referred to as "cement-treated soils." Referring to a particular mixture as a "soil-cement" or as a "cement-treated soil" is a function of the durability of the mixture of soil, portland cement, and/or other admixtures that has been formulated. Mixtures with greater degrees of stabilization and/or durability are generally referred to as soil-cement; mixtures with lesser such qualities are called cement-treated soil. Soil-cement is typically expected to be able to pass durability tests that measure the ability of the stabilized soil to retain its properties after long periods of exposure to the elements. Trudeau/Wissa Post Tr. 10,834, at 22. Cement-treated soil has less strength than soil-cement and is not expected to pass durability tests.

C.3 PFS intends to use soil-cement and cement-treated soil in three different ways. In the area directly underneath the concrete 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 DBE. The cement-treated soil will provide bonding with the bottom of the concrete pad above it and with the clay soils beneath, so as to transfer the horizontal earthquake forces downwards from the pad and into the underlying clay soils. Soil-cement is to be used in

the area around and between the cask storage pads. There, the function of the soil-cement is to support the weight of the transporter vehicle that is used to deliver storage casks to the pad area. Soil-cement was chosen for this application so that the soil materials would not need to be wasted and replaced with structural fill. Finally, soil-cement is to be placed around the CTB foundation mat, extending outward from the mat a distance equal to the associated mat dimension, to provide additional passive resistance against sliding forces in the event of a DBE. Trudeau/Wissa Post Tr. 10,834, at 23-24.

C.4 Soil-cement is often used for soil stabilization purposes, that is, to improve the compressive strength of the soil so that it becomes more rigid and less compressible, and to increase its resistance to sliding by virtue of its cohesive properties. Tr. at 10,843-44 (Wissa). At the PFS facility, the design relies on the compressive strength of the soil-cement to provide passive resistance to sliding of the CTB, and it relies on the cohesive strength of the cement-treated soil underneath the pads to essentially bond the pads to the underlying stiff clays. Tr. at 10,841-42, 10,845 (Trudeau). While the soil-cement “frame” surrounding the storage pads provides passive resistance against sliding of the pads, PFS conservatively does not take credit for such resistance. Tr. at 11,965-67 (Trudeau).

C.5 The soil-cement and cement-treated soil at the PFS facility have several design requirements. First, the cement-treated soil underlying the pads should have a minimum unconfined compressive strength of 40 pounds per square inch (psi). The cement-treated soil is also required to have a thickness no greater than 2 ft. and a modulus of elasticity or Young’s Modulus (that is, a vertical stress to strain ratio) less than or equal to 75,000 psi. Second, the soil-cement to be placed around and between the cask storage pads is to have a thickness of 28 in. (3 ft. height of the pads, minus the top 8 in., which will be filled with compacted aggregate). The soil-cement adjacent to the pads should have a minimum unconfined compressive strength

of at least 250 psi, in order to meet the durability (wet/dry and freeze/thaw cycle) requirements, since it will be exposed to the detrimental effects of frost. Finally, the soil-cement to be placed around the CTB will have a thickness of 5 ft. (plus 8 in. to be filled with aggregate). It also is expected to have a minimum unconfined compressive strength of at least 250 psi, in order meet the durability requirements (wet/dry and freeze/thaw), since its upper half will be within the frost zone, and to provide the required passive resistance to sliding. Trudeau/Wissa Post Tr. 10,834, at 24-25.

C.6 All parties agree that these design requirements can be met by the use of appropriate soil-cement mixtures. Trudeau/Wissa Post Tr. 10,834, at 31; Tr. at 11,018-19, 11,021 (Ofoegbu); Tr. at 11,088-89 (Mitchell). Indeed, the State soil-cement expert testified that he knew of nothing that would preclude PFS from meeting its design objectives for the soil-cement program. Tr. at 11,211-12 (Mitchell).

b. General State Concerns

C.7 The State raises several problems with the Applicant's proposed use of soil-cement to bolster the foundations of the storage pads and the CTB. According to the State, the Applicant's soil-cement program is part of the demonstration that the Applicant must make in order to satisfy the requirements of 10 C.F.R. § 72.102. Furthermore, the State contends that the Applicant bears the ultimate burden of demonstrating that the soil conditions at the site are adequate with the addition of soil-cement to sustain the proposed foundation loadings. In this regard, the State argues that there are a number of uncertainties in the Applicant's current testing programs and proposed future testing programs to prevent the Board from finding that the Applicant has sustained its burden. In addition, the State asserts that the NRC inspection programs are not designed to detect latent defects or to be a "construction watchdog," and that the Applicant and

Staff cannot rely upon these programs as assurance that the soil-cement will meet its intended goals. See State Findings ¶¶ 125-30.

C.8 Moreover, the State claims that several of the tests already completed by the Applicant demonstrate that the Applicant's proposed precedent-setting use of cement-treated soil will not adequately sustain the proposed foundation loadings. The State argues that the Applicant's sliding stability calculations are not based upon site-specific investigations and laboratory analysis and thus are unreliable. In addition, the State claims that the Applicant has not adequately demonstrated that the shear strength of the cement-treated soil will meet the necessary 1.1 factor of safety required by the NRC Staff. The State also contends that the Applicant has not demonstrated that the proposed facility design will meet the requirements of 10 C.F.R. §§ 72.102(d), 72.90, 72.102(c), and 72.122(b). Finally, the State claims that even if the Applicant can demonstrate that its design concept is adequate, there is evidence to indicate that significant degradation due to cracking, debonding along interface layers, and moisture infiltration will erode its ability to meet the proposed foundation loadings. See State Findings ¶¶ 130-34.

2. Specific State Challenges

a. Potential Problems with the Construction Process

C.9 According to the State, one uncertainty during the construction stage is the effect construction and exposure of the subsurface layer will have on the upper Lake Bonneville clays. See State Findings ¶ 92. The State believes that once the surficial material is removed, the clays may be disturbed by construction activities or become degraded by weather conditions causing the clays to dry out (hot, dry conditions) or gain moisture (wet conditions). See State Findings ¶ 92.

C.10 Another uncertainty, raised by the State, concerns how much of the upper Lake Bonneville clays will be removed along with the eolian silts during excavation of the site. The State contends that the record contains insufficient evidence to establish that the eolian silts have a uniform depth across the site and that the depth is 3 ft. Therefore, the State argues that the Applicant cannot rely on varying the thicknesses of cement-treated soils if the eolian silts are at depths greater than 3 ft. because of Holtec's constraint that the cement-treated soil may only be 1 to 2 ft. thick. See State Findings ¶ 93. If the upper Lake Bonneville clays are excavated or disturbed, PFS intends to replace them with compacted clay fill. Tr. at 10,898-900 (Trudeau). According to the State's experts, however, there is currently no analysis on whether the remolded upper Lake Bonneville clays, consisting of compacted clay fill, will have the same shear strength as the undisturbed upper Lake Bonneville clays that form the basis of the PFS pad sliding analysis. State of Utah Testimony of Dr. Steven F. Bartlett and Dr. James K. Mitchell on Unified Contention Utah L/QQ (Soil-cement) [hereinafter Mitchell/Bartlett] Post Tr. 11,033, at 11-12. The State contends that this has significance to the PFS pad sliding calculations, because remolded or compacted clay will have a decrease in shear strength from the design values PFS is relying upon for the native soils. Id. at 12.

C.11 As described by the Applicant's expert during the hearing, the soil-cement and the cement-treated soil to be used at the PFS facility will be constructed by removing the topmost layer of soil at the PFS site, which is a layer of eolian silt, and mixing it with cement in the appropriate proportions, as construction proceeds across the site. Trudeau/Wissa Post Tr. 10,834, at 23. Soil-cement manufacture will likely involve mixing the soil and the cement at a processing plant onsite to ensure high quality. Tr. at 10,890-91 (Wissa); Tr. at 10,906 (Trudeau).

C.12 The design requires that there be a minimum of 1 ft. and a maximum of 2 ft. of cement-treated soil under each storage pad. There may be an area in the south-eastern corner of the pad emplacement area where the eolian silt extends deep enough that, after its removal, it may be necessary to fill in below one or more of the pads to limit the cement-treated soil thickness to 2 ft. Tr. at 10,898 (Trudeau); 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 Utah L/QQ [hereinafter Trudeau/Wissa Reb.] Post Tr. 11,232, at 7. In any location where this happens, PFS expects to place compacted native soils.

C.13 The State's construction concerns, such as remolding of the upper Lake Bonneville clays, are easily addressed by use of appropriate techniques for the installation of soil-cement and cement-treated soil. Trudeau/Wissa Post Tr. 10,834, at 36-37. The main area of concern with respect to remolding of the native soils is with respect to the cask storage pads, for which the cohesive strength of the clay under the cement-treated soil is required to provide sliding resistance. However, there is construction equipment that can be located on either side of the pads at the placement locations and reach out to make a cut to the final subgrade surface, if necessary. All other construction equipment can be kept off of the exposed subgrade. Through these means, the subgrade can be sufficiently protected during the soil-cement installation. Id. The State's expert, Dr. Mitchell, agreed that the measures proposed by PFS can effectively protect the soils from any adverse effects from disturbance due to construction activities. Tr. at 11,162 (Mitchell). He also agreed that the construction techniques proposed by PFS to avoid remolding of the clay soils are within the state of the art. Dep. of James K. Mitchell (Mar. 15, 2002) (PFS Exh. 228) [hereinafter Mitchell Dep.] at 114-15.

C.14 As we explained above, the State also claims that construction activities or removal of the overlying soils may cause the underlying soils to lose strength, casting doubt on the utility of the undrained shear strength values obtained by the Applicant in its sampling program. For this reason, the State urges us to find that the samples of the upper Lake Bonneville clays that the Applicant has used for testing may not be representative of actual field conditions. We find no basis to do so. We would expect that any major construction project that ensues over a protracted period will encounter field conditions that differ from the conditions experienced during a pre-construction testing program. The Applicant has not discounted that potential; indeed, it has committed to utilize construction practices with field quality control requirements that take this potential into account. See Trudeau/Wissa Post Tr. 10,834, at 34-37; PFS Exh. JJJ at 2.6-118 to -119. The Applicant has also 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. There are a number of construction techniques available to prevent damage to the native soils beneath the pads, and the Applicant intends to use appropriate measures to prevent such damage. Trudeau/Wissa Post Tr. 10,834, at 36-37. With respect to potential damage due to exposure to the elements, this will be minimized through the use of proper construction procedures, scheduling, and measures such as the removal of excess moisture from the soil. See id. at 36.

b. Design Problems Affecting the Native Soil and the Cement Pad

(i) Cracking

C.15 The State has raised particular concerns about cracks in the soil-cement and cement-treated soil, could lead to a loss of tensile strength in those materials. See State Findings ¶ 117. According to the State, the loss of tensile capacity is important, because it will decrease some of the structural competency of the soil-cement layer. Tr. at 11,112-13 (Mitchell). The State

argues that because there are no cases to draw upon that use soil-cement of the depths that PFS intends to use, it is difficult to predict the size and extent of such cracks. Tr. at 11,111 (Mitchell).

C.16 The State raised several factors that could cause the concrete slab to crack. First, the State contends that concrete slabs can crack for unknown reasons (e.g., garage floors, bridges). Tr. at 11,130 (Mitchell). According to the State, the concrete slab may also crack from a cask tip-over or seismic event. Tr. at 11,130-32 (Mitchell); at 11,133 (Bartlett). Finally, the State argues that the pad could be degraded by windblown sulfates and salts that attack and corrode the steel reinforcing bar via shrinkage cracks in the concrete and cause the concrete to spall and crack. Tr. at 11,134-36 (Bartlett) .

C.17 There is a 8 in. layer of aggregate on top of the soil-cement around the pads. SAR Fig. 4.2-7 (State Exh. 212). In the north-south direction there will essentially be a 30 ft. wide gravel trench, and if there is no rapid drainage of water from the aggregate, the State argues that it will create a bathtub effect. The State believes that shrinkage cracks between the soil-cement and the storage pads or debonding of the laminar planes will result in the ingress of standing water as well as snow melt. Tr. at 11,137-40 (Mitchell); at 11,140-44 (Bartlett). This increased moisture, in turn, could lead to the weakening of the upper Lake Bonneville clays. Tr. at 11,147-48 (Mitchell). According to the State, of greater operational significance is whether weakened soil-cement from water infiltration will be capable of supporting the cask transporter used to move the 175-ton storage casks. <u>See</u> State Findings ¶ 120.

C.18 At the outset of its argument, the State notes that the main consequence of crack formation is the potential infiltration of moisture into the soil beneath the soil-cement that surrounds the pads and the CTB. Tr. at 11,147-48 (Mitchell). However, as discussed below (<u>see</u> section ii. moisture), water infiltration -- if occurring -- is not expected to have a significant

adverse impact on the performance in an earthquake of the soil-cement or cement-treated soil, and the underlying soils, or on the behavior of safety-related structures at the PFS facility under seismic loadings.

C.19 The State is also concerned about the potential reduction in the tensile strength of the soil-cement due to crack formation (Mitchell/Bartlett Post Tr. 11,033, at 8-9; Tr. 11,208-09 (Bartlett)); however, any such loss would occur only in the cracked area, and would not constitute a total loss of tensile strength unless the crack went through the entire cross-section of the soil-cement. Tr. at 11,300-01 (Trudeau). This is unlikely to occur. Tr. at 11,110-11 (Mitchell). In any event, the Applicant does not rely on the tensile strength of the soil-cement, so the effect, if any, of such cracking is inconsequential. See Trudeau/Wissa Post Tr. 10,834, at 41; Tr. at 11,296-97 (Trudeau).

(ii) Moisture

C.20 The State claims that water will infiltrate the soil-cement or cement-treated soil layers and will potentially degrade those materials. Tr. at 11,147-49 (Mitchell). Potential pathways for water infiltration include cracks in the concrete slab, shrinkage cracks between the soil-cement and the structure (pads or CTB), and standing water in the rows between the pads. Tr. at 11,137-38 (Mitchell).

C.21 The Board finds that the Applicant has demonstrated that water infiltration will not be a problem at the PFS site, because the storage casks on top of the pads will provide a source of heat that will be transmitted downwards through the concrete pad and the cement-treated soil. Therefore, the area beneath the pads on which casks rest will be warmer than surrounding areas, causing moisture to migrate away from the cement-treated layer beneath the pads to the surrounding areas due to heat gradient effects. Trudeau/Wissa Post Tr. 10,834, at 33; Tr. at

11,012 (Ofoegbu). In addition, there is no mechanism for moisture to migrate towards the upper layer of the soil, given the great depth to the groundwater table at the site and the semiarid conditions in Skull Valley. Trudeau/Wissa Post Tr. 10,834, at 37; Ofoegbu Post Tr. 11,001, at 15-16.

C.22 Moreover, the mechanisms postulated by the State witness under which such infiltration could happen are unlikely (e.g., continuous top-to-bottom cracking of the 3 ft. thick reinforced concrete, see Tr. at 11,134-37 (Bartlett); dropping of a cask on the pad, causing a crack that is not subsequently repaired, see Tr. at 11,130-34 (Mitchell); water accumulating in the permeable "bathtub" created by the 8 in. of aggregate that will be placed on top of the soil-cement and then filtering down through shrinkage cracks, see Tr. at 11,137-39 (Mitchell); snowfall accumulating on top of the aggregate, see Tr. at 11,141-42 (Bartlett); and separation between the CTB and the soil-cement layer adjacent to it due to differential settlement, see Tr. at 11,153-57 (Bartlett)). For example, the Applicant will install berms around the pad emplacement area to direct any surface water away from the pad emplacement area; within the pad emplacement area, the site is generally sloped from south to north and from the center of the site to the edges where there are concrete-lined drainage ditches to transport the surface water to the detention pond at the north. Tr. at 11,233-34 (Trudeau). Accordingly, there is no potential for significant presence of standing water in the pad emplacement area following snow melt, run-off, thunderstorms, or any other mechanism. Tr. at 11,234 (Trudeau).

C.23 Furthermore, any water that enters through a crack in the soil-cement will be unlikely to penetrate all the way down to the underlying soils, because the soil-cement will be constructed in thin lifts, all of which will cure at different times. While each of the lifts may have its own shrinkage cracks, it is very unlikely that the cracks on each lift will line up exactly with the cracks on other lifts. Tr. at 11,234-35 (Trudeau). In addition, the adhesive material used to provide

bonding between successive lifts will serve as a barrier against crack propagation. Tr. at 11,197-98 (Mitchell). And, in the area of the soil-cement frame around the pads, there is a continuous layer of cement-treated soil that extends out beyond the pads that would prevent the downward passage of water. Tr. at 11,236-37 (Trudeau).

C.24 If water does enter the soils beneath the soil-cement through cracks, it just as easily can evaporate through them during dry periods. Tr. at 11,196 (Mitchell). Total precipitation at the PFS site is on the average 9 in. a year. Tr. at 11,139 (Bartlett). The site thus has a semi-arid climate, which would facilitate the evaporation of any accumulated water. Tr. at 11,236 (Trudeau).

C.25 In addition to the unlikelihood of water infiltration through the mechanisms postulated by the State, tests performed on the soils at the PFS site demonstrate that the strength of the soils is only minimally affected by an increase in water content. See SAR at 2.6-42 to 2.6-44b (PFS Exh. 230); Ofoegbu Post Tr. 11,001, at 15-16.

C.26 Finally, any moisture accumulation and attendant potential reduction in the shear strength of the soil would only be a localized phenomenon, which would not have a significant effect on the strength or bearing capacity of the soils underlying the storage pads or the CTB. Ofoegbu Post Tr. 11,001, at 15-16; Tr. at 11,152-53 (Mitchell); Tr. at 11,157-58 (Bartlett).

(iii) Pad-to-Pad Interactions

C.27 The State believes that because the soil-cement and cement-treated soil will not be constructed with steel or other reinforcement, they will be very weak in tension. Mitchell/Bartlett Post Tr. 11,033, at 7-8. Further, the State argues that because the casks, pads, soil-cement, and soils have very difference masses, these masses will have different frequencies of vibration and behave differently during the cyclic forces from an earthquake. Tr. at 11206-07 (Bartlett).

The State maintains that the inertial effect (i.e., the fundamental frequencies at which these different masses want to vibrate) introduces not only compression but tension into the system and creates out-of-phase motion of the various masses. Id.

C.28 According to the State, the Applicant assumes that during an earthquake, the system – pad, soil-cement, cement-treated soil, and soil – will act as an integrated mat keeping each individual pad in place and in phase with the other adjacent pads and will transfer all of the dynamic load down to the underlying native soils. Mitchell/Bartlett Post Tr. 11,033, at 8. Conversely, the State claims that the heavily-reinforced relatively massive pad and the weak soil-cement in between the pads will act out of phase, and the soil-cement which is stiffer than the underlying native clays will act as a strut and pick up the dynamic load and transfer it laterally. Tr. at 11,206-07 (Bartlett).

C.29 Further, the State notes the storage pad has been analyzed to determine its structural suitability for dynamic loading conditions but no similar calculation exists for the underlying cement-treated soil or soil-cement. Mitchell/Bartlett Post Tr. 11,033, at 7-8.

C.30 As we explained in detail above, the State's major concern appears to be that the Applicant has assumed that the different masses of the entire system would move in-phase. According to the record, however, the Applicant has designed the proposed facility to prevent the pads from sliding at all in the event of an earthquake. See, e.g., Testimony of Paul J. Trudeau on Section D of Unified Contention Utah L/QQ [hereinafter Trudeau Section D] Post Tr. 6135, at 5, 9. Thus, the analysis that the State criticizes does not represent the Applicant's anticipated performance of the pads and their foundations during an earthquake.

C.31 During the hearing, the Applicant's expert Paul Trudeau explained that there will be no out-of-phase motion of the pads relative to the underlying soil, so the "push and pull action"

posited by the State will not take place. However, to test the State's hypothesis, Holtec performed an analysis in which it modeled two adjacent pads, 5 ft. apart, one pad fully loaded with eight casks, the other having only a single cask, and included a representation of the soil-cement between the pads. Rebuttal Testimony of Alan I. Soler on Section D of Unified Contention Utah L/QQ [hereinafter Soler Reb.] Post Tr. 10,557, at 2. The configuration in these simulations was set so that the potential for pad-to-pad forces was maximized. No forces were allowed to be absorbed by the soil-cement; no forces were allowed to be transmitted downwards to the cement-treated soil and to the soils beneath; no damping was included in the model; a maximum value of Young's Modulus for the soil-cement was assumed; the pads were not allowed to slide; and no credit was taken for the potential crushing of the soil-cement by the forces going from one pad to the other. Tr. at 10,657, 10,720-24 (Soler).

C.32 Holtec performed two computer simulations for this model: one in which the soil-cement between the pads is assumed to retain its integrity and therefore be able to transmit both tension and compression forces; and another simulation in which the soil-cement is assumed to be cracked and thus able to transmit only compression forces. Soler Reb. Post Tr. 10,557, at 2. Notwithstanding the very conservative assumptions made in running these simulations, the maximum calculated force in the soil beneath the pads was less than that required to initiate pad sliding. Additional Cask Analyses for the PFSF (PFS Exh. 225) [hereinafter Additional Cask Analyses]; Tr. at 10,723 (Soler). Also, while both simulations predicted some interactions between the pads or between the pads and the soil-cement, the forces resulting from those interactions, when added to the seismic loadings, resulted in total cask motions of the same order - inches - as had been obtained in prior simulations that had not expressly accounted for pad-to-pad interaction forces. Soler Reb. Post Tr. 10,557, at 3-4.

3. Testing

a. Adequacy

C.33 The appropriate soil-cement formulation for each of the aforementioned applications will be established through a laboratory testing program. Trudeau/Wissa Post Tr. 10,834, 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) (PFS Exh. GGG) [hereinafter ESSOW]. 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 QA Category I requirements of the ESSOW. SAR at 2.6-109 (PFS Exh. JJJ).

C.34 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. 10,834, at 31. In particular, the ESSOW cites Regulatory Guide 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) (PFS Exh. HHH) [hereinafter ACI Committee 230 Report] with respect to mix proportioning, testing, construction and quality control for soil-cement. Trudeau/Wissa Post Tr. 10,834, at 29.

C.35 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 and degradation cited by the State. Ofoegbu Post Tr. 11,001, at 18-19.

C.36 The ESSOW sets forth a series of tests to be conducted in several phases. Trudeau/Wissa Post Tr. 10,834, 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 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.

C.37 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, and although 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.” Trudeau/Wissa Post Tr. 10,834, at 30.

C.38 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. In particular, these tests establish the

optimum moisture content and maximum density for molding laboratory test specimens for further testing. Id. at 27. AGEC has provided preliminary results for these tests. Id. at 30.

C.39 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 27-28. 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." Trudeau/Wissa Post Tr. 10,834, at 27-28.[36]

C.40 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.

C.41 The next step in the proposed soil-cement testing program is the performance of compressive strength tests in accordance with ASTM D1633 and D558. Trudeau/Wissa Post Tr.

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[36] 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 3 ft. thick concrete pads and is therefore not exposed to the elements. Trudeau/Wissa Post Tr. 10,834, 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.

10,834, 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.

C.42 Finally, as noted above, the cement-treated soil will be subject to direct shear tests to confirm that the bond (1) at the interfaces between the concrete bottom of the cask storage pad and the cement-treated soil; (2) at the interfaces between lifts of cement-treated soil; and (3) 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.

C.43 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; SAR at 2.6-118 (PFS Exh. JJJ). 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. SAR at 2.6-118 (PFS Exh. JJJ).

C.44 To ensure that sufficient bonding is achieved, PFS plans to utilize the techniques described in the ACI Committee 230 Report and the 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. 10,834, at 31. 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 6 in.; (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. SAR at 2.6-118 (PFS Exh. JJJ).

C.45 During the hearing, the Applicant's witness, Dr. Wissa, testified that the Applicant's soil-cement laboratory testing program is adequate, if properly implemented. Trudeau/Wissa Post Tr. 10,834, at 30. The State notes that when asked to elaborate Dr. Wissa further testified that if he were to conduct the PFS soil-cement testing program, he would basically need to start all over so that he could vouch for the quality of his work. Tr. at 10,980 (Wissa). He would use the AGEC test results as a check on his results. Tr. at 10,978-79 (Trudeau). The State points out that at the end of the hearing, Dr. Wissa noted that he was talking with the Applicant about taking over the soil-cement testing program, but at that time it was unknown who will be conducting the testing program. See State Findings ¶ 81.

C.46 The State points out that the quality and success of the PFS demonstration that it can prove and successfully implement its soil-cement design concept depends in significant part on the credentials and experience of the person or entity chosen to conduct and supervise the testing program. See State Findings ¶ 82.

C.47 All parties agree that PFS has developed a suitable program for testing the properties of the soil-cement, which is embodied in the ESSOW. Trudeau/Wissa Post Tr. 10,834, at 25-26; Tr. at 11,089-93 (Mitchell). The program will be effective in establishing whether the properties of the soil-cement specified in the design have been achieved. Tr. at 11,266 (Mitchell).

C.48 The parties also agree that the program is based on appropriate industry standards, including the ACI Committee Report, and that it includes the proper tests and suitable test

methodology. Trudeau/Wissa Post Tr. 10,834, at 29; Ofoegbu Post Tr. 11,001, at 14-15; Tr. at 11,060-61 (Mitchell).

C.49 Finally, the parties agree that the program to which PFS has committed in the SAR (SAR at 2.6-118, 2.6-119 (PFS Exh. JJJ)) is reasonable and should lead to proper soil-cement and cement-treated soil installation. Trudeau/Wissa Post Tr. 10,834, at 31-32; Ofoegbu Post Tr. 11,001, at 12-13; Tr. at 11,088-89 (Mitchell). The program -- including the construction procedures it calls for -- is based on well-accepted, standard practices set forth in manuals issued by organizations such as the U.S. Army Corps of Engineers and the Portland Cement Association. Tr. at 10,973-74 (Trudeau/Wissa).

C.50 There is also a fair degree of flexibility in establishing the acceptance criteria for the soil-cement and cement-treated soil, as well as the tolerances that the specified material content must meet. Tr. at 10,945-47 (Wissa); Tr. at 11,179-81 (Mitchell). If, however, a soil-cement installation failed to meet design requirements, it is most likely that the Applicant would rework or replace the soil-cement rather than attempt to demonstrate its acceptability through analyses. Tr. at 10,938-40 (Trudeau); Tr. at 10,965-67 (Wissa).

C.51 PFS witnesses also testified that appropriate measures will be taken during construction to ensure that the required quality of installation and the requisite properties of the soil-cement are achieved and any non-conformances are corrected. The work would be subject to oversight of both the contractor and the owner, would be subject to NRC approval, and would be required to conform with NRC requirements. Tr. at 10,968-69 (Wissa); Tr. at 10,992-93 (Trudeau).

b. Proof of Design and Timing

C.52 A major point of disagreement between the Applicant and the Staff, on the one hand, and the State, on the other, is that the State believes that the test program to confirm that the soil-

cement will have the requisite properties should be completed before licensing of the facility, whereas the other parties do not think this is either required by NRC regulations or necessary. Compare Mitchell/Bartlett Post Tr. 11,033, at 5 with Trudeau/Wissa Post Tr. 10,834, at 33-34 and Tr. at 11,017 (Ofoegbu).

C.53 During their testimony, State experts pointed out the potentially adverse economic consequences that could befall PFS if, after licensing, it was determined that the use of soil-cement in the manner proposed by PFS was for some reason unworkable. Tr. at 11,096-100; 11,104-07 (Mitchell). In response, the Applicant argued that when questioned, the State was unable to point to a regulatory rule, regulation, or regulatory guidance that requires the soil-cement testing program to proceed in advance of licensing, and the Staff witness testified that, once the design requirements are established and a commitment is made to perform an appropriate testing program to demonstrate compliance with them, an applicant is free to defer testing to the post-licensing phase. Tr. at 11,017-18 (Ofoegbu).

C.54 The State's position, contrary to that of the Staff, is that the Applicant's future soil-cement program is part of the demonstration PFS must make to prove its design concept and to satisfy 10 C.F.R. § 72.102. That demonstration, says the State, must be satisfied before PFS can receive a license. See State Findings ¶ 125.

C.55 In this proceeding, the Applicant bears the ultimate burden of proof. In order to carry that burden, PFS must show by site-specific investigations and laboratory analyses that its soil conditions, including soil-cement, are adequate for the proposed foundation loadings. 10 C.F.R. § 72.102(d). See State Findings ¶ 126.

C.56 According to the State, the items that PFS has yet to demonstrate in its soil-cement program include the following: (1) adequate classification of surficial soils; (2) selection of soils

for further testing including durability (wet-dry, freeze-thaw); (3) testing to determine the correct percentage of cement to add to soils as well as moduli testing to achieve Young's Modulus of less than 75,000 psi; and (4) testing to determine bonding and adhesion between interface layers. Furthermore, the State argues that there are uncertainties about implementing the Applicant's laboratory testing program, whether confirmatory tests can be successfully carried out in the field, and whether construction techniques or activities will degrade the strength of the upper Lake Bonneville clays. See State Findings ¶ 127.

C.57 The State argues that neither the Applicant nor the Staff can rely on the NRC's inspection program to ensure that placement of cement-treated soil under 500 pads at PFS will meet the Applicant's target performance goals. The State argues that the NRC's inspection program is not geared to detect latent defects or to be a watchdog at every step of construction. See State Findings ¶ 130.

C.58 According to the State, the Applicant has not met its burden to show in its sliding analyses, PFS Exhibit UU, that the shear strength of both the upper Lake Bonneville clays and the cement-treated soil will meet a minimum factor of safety of 1.1. See State Findings ¶ 132.

C.59 The State also argues that neither the Applicant nor the Staff have demonstrated that the Applicant can meet the requirements of 10 C.F.R. § 72.102(d). Also, the State contends that PFS has not met the requirements of 10 C.F.R. §§ 72.90, 72.102(c), or 72.122(b). See State Findings ¶ 133.

C.60 The testimony at the hearing shows that the design requirements for the soil-cement and the cement-treated soil have been adequately established. Additionally, the PFS witnesses testified that PFS has committed to developing a soil-cement mix design using standard industry practices, and has further committed to performing a soil-cement testing program in accordance

with appropriate industry standards. Thus, PFS has specified the tests it intends to perform and the acceptance criteria that will be applied to the test results. As stated in the SAR, PFS is also committed to performing field testing during construction to demonstrate that it has, indeed, achieved in the field the bond strengths that are required. Trudeau/Wissa Post Tr. 10,834, at 34-35. Such tests will include obtaining core samples through the pad and the underlying layers of interest, taking them to the laboratory, performing shear tests at the interfaces, and demonstrating that the shear strength along those interfaces exceeds that of the underlying clay. The Applicant believes that this will confirm that a good bond has been achieved and that the shear strength available along those interfaces exceeds the shear strength used in the sliding stability analyses. Tr. at 10,963, 10,971, 10,981-82 (Wissa).

C.61 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. Trudeau/Wissa Post Tr. 10,834, 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. 11,001, at 14.

C.62 The Staff has also reviewed the design submitted by PFS, including the associated calculations and specified material properties, to determine compliance with the applicable regulatory requirements. See Tr. at 11,016 (Ofoegbu). The Staff determined that the analyses submitted by PFS demonstrated that the design would be safe, and that based on information available in the technical literature, the material properties used by PFS in its design are

achievable. See Tr. at 11,021 (Ofoegbu). The Staff's analysis is documented in the SER. Tr. at 11,022 (Ofoegbu). In this regard, the Staff concluded that PFS has "proven its design," even without having completed the soil-cement testing program. Tr. at 11,021 (Ofoegbu).

4. Precedent

C.63 The parties do not agree on whether the Applicant's use of soil-cement to resist sliding during an earthquake is a unique application of soil-cement. The State maintains that it is a unique and untested application to add cement to soil to provide additional seismic sliding resistance and stability to shallowly embedded foundations from strong ground motions. Mitchell/Bartlett Post Tr. 11,033, at 5-6; Tr. at 11,051 (Mitchell).

C.64 The State argues that the weight of the evidence and the direct experience and involvement in many of the projects PFS is relying upon to prove its case weigh strongly in the State's favor. The State also insists that the Applicant cannot rely on the properties of the material as defining precedent. See State Findings ¶ 114.

C.65 The Applicant and the Staff point out that soil-cement has been used for soil stabilization in numerous instances, both in the United States and abroad. Trudeau/Wissa Post Tr. 10,834, at 33; Trudeau/Wissa Reb. Post Tr. 11,232, at 1-3; Ofoegbu Post Tr. 11,001, at 14. For example, soil-cement has been used to provide foundation strength for an office building in Tampa, Florida, a dam spillway foundation mat in Fort Worth, Texas, a number of coal handling and storage facilities throughout the United States, a nuclear power station in Koeberg, South Africa, and a variety of other applications including highways. ACI Committee 230 Report at §§ 2.5,2.6; SAR at 2.6-113, 2.6-114 (PFS Exh. JJJ); Tr. at 10,971-72 (Trudeau). In that respect, the Applicant argues that since all uses of soil-cement rely on the same mechanical properties, all prior uses of soil-cement can be said to constitute precedents for its use at the PFS facility.

Tr. at 11,262-63 (Mitchell). The number of applications for soil-cement and the confidence in its use by the technical community continues to grow over time. Tr. at 11,190-91 (Mitchell).

C.66 In particular, soil-cement was used extensively to resist lateral forces and form permanent foundations for the five highway tunnels for I-90 and I-93 that converge at the Fort Point Channel crossing of Boston's Central Artery/Tunnel Project. This is essentially the same use of soil-cement that is being proposed for the PFS facility. Trudeau/Wissa Reb. Post Tr. 11,232, at 1-2; Tr. at 10,846-47 (Wissa).

C.67 In this instance, we do not need to decide whether there is precedent for the PFS proposed use of the soil-cement, because, as explained by the Staff's experts during the hearing, there is no regulatory requirement that the suitability of soil-cement for its intended use be demonstrated by case history precedent. Ofoegbu Post Tr. 11,001, at 12-13. Instead, what is of significance to the Board is whether the Applicant's proposed design satisfies the regulatory requirements. 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.

5. Young's Modulus

C.68 The State insists that one of the most difficult tasks confronting the Applicant is to find a mix using PFS surficial site soils that will attain a Young's Modulus (i.e., a vertical stress to strain ratio) of less than 75,000 psi for 40 psi compressive strength cement-treated soil. <u>See</u> State Findings ¶ 98. This is, however, not as a difficult task as the State contends. All parties agree that seeking to limit the Young's Modulus to less than 75,000 psi for cement-treated soil having an unconfined compressive strength of 40 psi is achievable, because having a relatively low modulus is consistent with the relatively low strength required. Tr. at 10,914-15 (Wissa); Tr. at

11,023-24 (Ofoegbu); Tr. at 11,159-60 (Mitchell). This is also supported by data reported in the literature. Tr. at 11,023-26 (Ofoegbu).

C.69 Dr. Ofoegbu provided citations to references that demonstrate that a Young's Modulus of no more than 75,000 psi is achievable at a compressive strength of 40 psi. Tr. at 11,025-26 (Ofoegbu). The State attempts to dismiss this testimony by asserting that the tests referenced by Dr. Ofoegbu are based on site-specific soils (State Findings ¶ 99), but a careful review of the record does not support this proposition. Dr. Ofoegbu did not indicate any such limitations when describing the literature, and in fact referred to a paper that provided charts of ranges of values of Young's moduli for use in soils analysis. Tr. at 11,026 (Ofoegbu).

C.70 Another issue raised in the State's proposed findings is that the soil-cement and the cement-treated soil continue to cure with time. See State Findings ¶ 100. The State argues that in order to achieve a Young's Modulus of no more than 75,000 psi, one may need to start with a modulus perhaps as low as 40,000 psi. Tr. at 11,222 (Mitchell). During the hearing, the State's expert on the subject, Dr. Mitchell, testified that he could not specify a starting value of modulus to aim for without test data, and that his only point was that it would not be prudent to start at the 75,000 psi value. Tr. at 11,222 (Mitchell). Our record demonstrates that the greatest increase in Young's Modulus occurs during the first 28 days of curing. Tr. at 11,226-27 (Mitchell); Tr. at 11,251-52 (Wissa). For that reason, the Young's Modulus value used in Holtec's cask drop and tip-over analysis is benchmarked at a curing age of 28 days. Tr. at 11,253 (Trudeau). In other words, the 75,000 psi maximum Young's Modulus value is determined as of the 28-day curing point. Id. Dr. Mitchell was not aware of what benchmark PFS intended to apply, but confirmed that 28-day strength was a commonly used value. Tr. at 11,227-28 (Mitchell). Because he was unaware of the benchmark used by PFS, Dr. Mitchell incorrectly assumed that the 75,000 psi limit applied throughout the life of the facility. See, e.g., Tr. at 11,216-17 (Mitchell). While the

strength of the cement-treated soil increases slowly with time after 28 days as it continues to cure, this process is immaterial because the important data point for which the cask drop and tip-over analyses are performed is after 28 days of curing.

C.71 The State also insists that the Young's Modulus should be measured under dynamic, not static loads. See State Findings ¶ 101-02. State witness Dr. Ostadan, however, noted that the Holtec design intent could be satisfied by formulating a test program that established that the modulus of elasticity of the cement-treated soil did not exceed 75,000 psi at the strain level occurring in the vicinity of cask impact (i.e., 1.93% in the soil directly beneath the cement-treated soil) based on Holtec's analysis. Tr. at 7426-27 (Ostadan). Thus, we find the distinction between dynamic and static loadings immaterial; the important issue is that the proper strain level be achieved in the test. The PFS approach in determining the Young's Modulus is to use soil strain level as the reference parameter for its Young's Modulus testing. Moreover, the Sandia National Laboratories paper that provided experimental data forming the bases for the cask drop analyses uses static moduli of elasticity for the soils underlying the pad, and it indicated substantial agreement between analytical results and experimental results, demonstrating that large-strain moduli are appropriate for such analyses. Tr. at 10,927-28 (Trudeau); Tr. at 10,988 (Wissa).

C.72 For the reasons established above, we find that the State's concerns are unfounded and the Applicant has demonstrated it will be able to attain a Young's Modulus of less than 75,000 psi for 40 psi compressive strength cement-treated soil.

D. Seismic Design and Foundation Stability

1. Overview of the Pad Storage System

a. Proposed Design Concept for the Pad Storage System

D.1 At the PFS facility, the SNF will be stored in large storage casks resting on concrete pads. The storage cask system to be used by PFS is the HI-STORM 100.

D.2 As described in Section 4.2.1.5.2 of the SAR, the HI-STORM 100 storage casks will be placed on a regular array of concrete pads arranged to provide a lateral (edge to edge) spacing of 35 ft. between adjacent pads in the east-west direction and 5 ft. longitudinal spacing in the north-south direction. Each pad will be sized to accommodate a 2 x 4 array of casks with a 15 ft. pitch (the distance between the casks' center points) in the width (or east-west) direction and 16 ft. in the length (north-south) direction. As described in Section 4.2.3.1 of the SAR, the cask storage pads will be independent structural units constructed of reinforced concrete, each pad being 30 ft. wide, 67 ft. long, and 3 ft. thick. Each pad will be capable of supporting eight loaded storage casks. At maximum capacity, the facility would contain 500 such pads, each supporting eight loaded storage casks. Singh/Soler Post Tr. 5750, at 8; Joint Testimony of Robert Youngs and Wen Tseng on Unified Contention Utah L/QQ [hereinafter Youngs/Tseng] Post Tr. 5529, at 9-10. A graphical representation of the cask storage arrangement is shown on PFS Exhibit 84 and Staff Exhibit X (Figure 1.2-1 of the SAR).

b. Overview of State's General Concerns

D.3 The State contends that the entire design and seismic performance of the cask-pad system at the PFS site relies on one design calculation: Holtec's nonlinear cask stability analysis to determine the seismic loading to the pads and foundations. State Findings ¶ 146. According to the State, seismic forces estimated in this one design calculation propagate

throughout other seismic and engineering calculations that the Applicant is relying upon to demonstrate the performance of its ISFSI during an earthquake. Tr. at 7344-45 (Ostadan). Thus the State argues that Holtec nonlinear analyses are also being used as the basis to: (1) predict cask movement atop the pads; (2) predict seismic loads transferred to the soil-cement, cement-treated soil, and soils; (3) perform the structural design of the storage pads; (4) predict pad sliding; and (5) analyze the effects of SSI on the response of the pads and the casks. See State Findings ¶ 146.

D.4 The State argues that the Applicant's design is unconventional and unprecedented. State Findings ¶ 148. According to the State, the unproven features at the PFS site include: (1) unanchored cylindrical casks; (2) acceptance of cask sliding on the pad as a basic design philosophy and taking full credit from cask sliding to reduce the seismic load to the storage pads and their foundations; (3) shallowly embedded storage pads founded on a compressible clay with the potential for several inches of settlement; and (4) untested and precedent-setting use of cement-treated soil and soil-cement as a structural foundation element to resist lateral earthquake forces and to add strength and stiffness to soils. Tr. at 7724-28; 7350-53, 10,286-87 (Ostadan/Bartlett). On top of this, the State insists that there is relatively little margin for error in design. State Findings ¶ 148. For example, if the Applicant has under-predicted dynamic loads by only 20% or over-predicted capacity by 20%, then the State points out that it becomes questionable whether the PFS design will perform during an earthquake. Tr. at 7342-43 (Ostadan).

D.5 In addition to no performance data, the State claims the Applicant has provided no test data it can rely upon to evaluate the performance of the casks. The State notes that the storage casks intended for the PFS site have not been tested either experimentally or during an actual earthquake to determine their performance under earthquake conditions. State Findings ¶ 150.

Holtec entered the dry cask business only in the mid-1990s. Tr. at 5915-16 (Singh). According to the State about twelve HI-STORM 100 casks are in use and about fifteen others have been delivered to nuclear reactor sites. See State Findings ¶ 150.

D.6 The State believes that it was possible for the Applicant to have acquired experimental test data on the performance of the HI-STORM 100 casks. According to the State, the Applicant could have conducted shake table tests on a scaled model cask or may acquire such data by conducting shake table tests on a full-sized cask next spring in the United States. The State points out, however, that the Applicant has chosen not to do so. State Findings ¶ 156.

D.7 The State points out that an assertion of "engineering judgment" without any explanation or reasons for the judgment, is insufficient to support the conclusions of the expert engineering witness. State Findings ¶ 157 (citing Texas Utilities Electric Co. (Comanche Peak Steam Electric Station, Units 1 and 2), LBP-84-10, 19 NRC 509, 518 (1984)). Further, the State argues that where an expert witness states ultimate conclusions on a crucial aspect of the issue being tried, and where that conclusion rests upon a performed analysis, the witness must make available sufficient information pertaining to the details of the analysis to permit the correctness of the conclusion to be evaluated. State Findings ¶ 157 (citing Virginia Electric and Power Co. (North Anna Nuclear Power Station, Units 1 and 2), ALAB-555, 10 NRC 23, 27 (1979)).

D.8 The State also argues that Holtec's design calculation is based entirely on a nonlinear computer program. State Findings ¶ 158. The State insists that nonlinear analyses are well known for being sensitive to the selection of input parameters and have been referred to as obtaining solutions from a "black box." Tr. at 7335-36; 7551-52 (Ostadan). According to the State, small changes in an input parameter, such as contact stiffness or damping, could dramatically change the result of nonlinear analyses. Tr. at 7336, 7352 (Ostadan); State Testimony of Dr. Mohsin R. Khan and Dr. Farhang Ostadan on Unified Contention Utah L/QQ,

Part D (Cask Stability) [hereinafter Khan/Ostadan] Post Tr. 7123, at 11-12. Finally, the State notes that PFS has no performance data or experience data to calibrate its design calculation. State Findings ¶ 158.

2. Specific State Concerns with the Applicant's Pad Stability Analysis

a. Concern with the Applicant's Methodology

D.9 The State maintains that the many disparate pieces of the Applicant's seismic design have evolved, often in response to cost cutting measures, and have not been fully thought out and integrated into a cohesive and rigorous design. State Findings ¶ 171. According to the State, emblematic of this is the lack of independent verification or checks and balances of the input parameters to the various design calculations. State Findings ¶ 171. In addition, the State highlights the fact that many of the input parameters for the design calculations are derived from Holtec -- a company with a large financial stake in the outcome of the proposed project. State Findings ¶ 171.

D.10 Furthermore, the State argues that, the Applicant's design has evolved from contemplation of anchored casks, excavation, and replacement of foundation soils with structural fill, to unanchored casks and removal of the eolian silts to save costs. State Findings ¶ 172; Tr. at 10,293-94 (Bartlett). According to the State, soil-cement was first introduced to stabilize the eolian silts in place and to provide a stable platform for the cask transporter, and only when there was a significant increase in ground motions, did PFS introduce using cement-treated soil as an mechanism to resist seismic loading. State Findings ¶ 172.

D.11 Moreover, the State continues, rather than bypass the weaker and more compressible zone of the upper Lake Bonneville clays either by treating the clays or embedding the pads into deeper, stiffer, and stronger soil, the Applicant has chosen to place cement-treated soil on top of

the relatively soft clays in an application that is precedent setting and whose design concept and requirements are yet to be tested. See State Findings ¶ 173. According to the State, the Applicant's seismic design of the cask-pad system is to transfer the seismic loads from the casks and pads down through the cement-treated soils to the upper Lake Bonneville clays. State Findings ¶ 173.

D.12 The State notes that the foundation design of the CTB also changed in response to an estimated 35% increase in ground motions at the PFS site. Tr. at 7313-14 (Bartlett); State of Utah Testimony of Dr. Steven F. Bartlett and Dr. Farhang Ostadan on Unified Contention Utah L/QQ (Dynamic Analysis) [hereinafter Bartlett/Ostadan] Post Tr. 7268, at 4; CTB Analysis at 5-6. Now, the State continues, existing soils around the footprint of the CTB will be excavated to a depth of about 5 ft., mixed with cement and placed a distance of about 240 ft. to 280 ft. out from the base mat of the building. Testimony of Bruce E. Ebbeson on Section D of Unified Contention L/QQ [hereinafter Ebbeson] Post Tr. 6357, at 5; Tr. at 7313 (Bartlett). A reinforced concrete key will also be constructed around the perimeter of the foundation mat. Ebbeson Post Tr. 6357, at 5.

D.13 The State also points out that the soils at the PFS site have limited capacity to carry loads and thus the Applicant turned to the use of soil-cement surrounding the foundation of the CTB as a means to provide passive resistance to sliding during an earthquake. Tr. at 7313-16 (Bartlett/Ostadan). The State argues that in order for the soft clays to attract the load, there must be some lateral movement of the building to mobilize the peak shear strength of the soil-cement. Tr. at 7316 (Ostadan). If the passive resistance of soil-cement is not used, the State believes the calculated factor of safety against sliding will be less than 1. Tr. at 10,798 (Ebbeson); Bartlett/Ostadan Post Tr. 7268, at 4-5.

D.14 Finally, the State contends that there are complexities in evaluating a foundation design, especially under seismic conditions. State Findings ¶ 177. Unlike fabricated material such as concrete and steel where the boundary conditions in the design are reasonably well-defined, the State argues that is not the case with soils. Tr. at 10,300 (Bartlett). Soils are naturally deposited materials that are heterogenous and isotropic, and are quick to reach a yield and exhibit nonlinear complex behavior. Id. Because of the way in which soils have been laid down by nature, the State asserts that there are huge uncertainties in soil properties that affect their strength and compressibility, both with time and during an earthquake. Id. at 10,301. According to the State, the usual practice in geotechnical engineering is to rely on simple models that are based on basic civil engineering concepts. Id. Because of these uncertainties and the judgment involved, the State argues it is only when there is sufficient precedence and actual experience that geotechnical engineers have confidence in their models. Accordingly, the State believes there should be a hesitancy to model the nonlinear behavior of a soil beneath a foundation system based on an untested design or reliance on a nonlinear analysis, such as the one performed by Holtec. Id. at 10,301-02.

b. Cask Sliding as a Design Concept

D.15 According to the State, PFS relies on cask sliding as a mechanism to reduce seismic loads. Bartlett/Ostadan Post Tr. 7268, at 5. For example, based on his review of the forces that Holtec has provided, Dr. Ostadan insists that at different times during the duration of shaking there is separation in the contact points between the cask and the pad. Tr. at 10,435-36 (Ostadan). The State also points out that nowhere in Holtec's analysis has it presented the forces for the casks analytically anchored to the pad -- in the analyses the casks have always been allowed to slide smoothly on the pads. Tr. at 10,291 (Ostadan). As

such, the State believes the forces transmitted to the pad and the underlying soils will be significantly greater if the casks are not allowed to slide. Tr. at 10,292 (Bartlett).

D.16 The State argues that sliding of SSCs in earthquake resistant design is not common and is rarely used. Tr. at 7345 (Ostadan). The State believes that with no experimental or reliable performance data, it is a bold gesture for PFS (and the NRC Staff) to rely solely on the Holtec nonlinear analysis to predict cask performance and to take full credit for reduction of seismic forces to the foundations resulting from sliding of the casks atop the pads. See State Finding ¶ 186.

D.17 Contrary to the State's argument, the PFS storage cask design does not "control" sliding, but merely allows it to occur. Tr. at 7335-39 (Ostadan). And the fact that sliding may occur in a "uniform and controlled manner" is not a design requirement but, rather, the result predicted by the cask stability analyses conducted by Holtec. Tr. at 7341-42 (Ostadan).

D.18 It is true that, if pad sliding occurs, such sliding has the beneficial effect of reducing the seismic loading to which the cask is subjected. See, e.g., Tr. at 7348-49, 7354 (Ostadan); Tr. at 6633-35 (Pomerening); Tr. at 6155-56 (Trudeau). However, to the extent that such effect occurs, it is again a consequence of the design and not a design feature or mechanism. In any event, any facility that features unanchored casks (such as those at the Hatch and San Onofre plants) resting on a concrete foundation will be subject to potential sliding of the foundation, and will thereby experience a beneficial reduction in the seismic loadings on the casks. Hence, this feature of the PFS facility is also not unique. Tr. at 7306-07 (Ostadan).

c. Flexibility of the Storage Pads

D.19 Foundation damping (also known as radiation damping) is the property of structures to reflect back or radiate into the soil a portion of the energy imparted upon the structures by the seismic excitation. Tr. at 7457-59 (Ostadan). If a structure is rigid, it will be efficient in radiating energy back into the soil. As the flexibility of the structure increases, its ability to radiate energy back into the soil decreases. Tr. at 7455-57 (Ostadan). This reduction in radiation damping is a matter of degree, and is a function of the amount of flexibility exhibited by the structure. Tr. at 7459-60 (Ostadan).

D.20 The State believes that the question for the Board to address regarding pad flexibility is whether the pad is flexible enough for the cask drop and tip-over constraint and rigid enough to produce significant radiation damping and provide a smooth (i.e., undeformed) surface for cask sliding. State Findings ¶ 196.

D.21 According to the State, an initial premise underlying Holtec's design calculations is that the pad will act as a rigid body. State Findings ¶ 189. In that regard, the State notes that the original Holtec Multi-Cask Seismic Response at the PFS ISFSI (at 3-4), dated May 19, 1997 states "the characteristics of the pad are based on the assumption that the 30 ft. by 64 ft. section responds to seismic excitation as a rigid body; this assumption has been based on recommendation of the project architect and engineering group responsible for the ISFSI design of the PFS facility." State Findings ¶ 189 (citing Tr. at 6186-87 (Trudeau)). The State argues that this assumption was not based on the recommendation of either Mr. Trudeau or Dr. Tseng. According to the State, this starting premise has led to Holtec's assumption that there will be no deformations in the pad and that the casks will slide smoothly over the pads. State Findings ¶ 189. The State also insists that the assumption of

pad rigidity has also guided Holtec's selection of soil springs and damping values. Tr. at 7451 (Ostadan).

D.22 According to the State, Holtec has calculated damping that is associated with a rigid pad and the International Civil Engineering Consultants, Inc. (ICEC)[37] has used those respective soil springs and damping values in its design calculation. Bartlett/Ostadan Post Tr. 7268, at 14. The State argues that the Applicant has not shown the pads are rigid, yet it still takes full credit for radiation damping. Tr. at 7459 (Ostadan). The State insists that if the pads are in fact rigid, there would be significant SSI effects with the soils playing a major role in dissipating energy through radiation damping. Id. at 7455-57.

D.23 The State contends that ICEC was never asked to determine the appropriate damping under rigid or flexible conditions. Tr. at 7467 (Ostadan). Instead, according to the State, ICEC was given the dynamic forces that came out of Holtec's nonlinear time history analysis. Id. The State believes that the relative flexibility or rigidity of the pad could have been easily ascertained by using the industry standard computer program for soil structure interaction, SASSI, and conducting a half-day analysis, first by assuming the pad was rigid and then assuming it has the properties of concrete. Id. at 7466, 7471. According to the State, by calculating the amount of damping for these two scenarios, the Applicant would have quantified the appropriate amount of damping for the PFS site. Id. This issue is important because if smaller damping values are used, seismic loads would be higher than those calculated by Holtec. Id. at 7470-71.

D.24 The State argues that the contact condition in Holtec's analytical calculation for cask tip-over and drop also requires that the pad and underlying cement-treated soil be somewhat

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[37] PFS Expert Dr. Wen Tseng is President of ICEC.

flexible to be able to absorb energy from cask impact. Tr. at 7449 (Ostadan). Here, the State insists that PFS is asking that the cement-treated soil be strong enough to carry the horizontal loads and meet the pad sliding requirements but soft enough to satisfy Holtec's cask drop tip-over conditions. Tr. at 7422-23, 7450-54 (Ostadan).

D.25 During the trial, the State's expert, Dr. Ostadan asserted that what is important is not necessarily the amplitude of displacement, as argued by the Applicant's experts, but rather the movement of the different points on the pad with respect to each other. Tr. at 7460, 7465 (Ostadan). Dr. Ostadan believes that the larger the relative movement over the pad, the less damping, and less damping will result in an increase in the calculated seismic loadings. Id. at 7460, 7469-70. The State argues further when evaluating pad stiffness, one needs to look at the entire pad and determine whether it is moving intact together and engaging with the soil (highest damping) or flip flopping (less damping). Where the maximum deformation is repeating in nearby adjacent points, the pad is flexible. Dr. Ostadan, the State's expert in SSI, testified that looking at maximum relative displacement was a very cumbersome way to ascertain pad flexibility when two runs on SASSI would readily produce the answer. Tr. at 7471 (Ostadan).

D.26 As we previously noted, Dr. Ostadan expressed concern over the Applicant's analysis because it did not account for a potential rippling effect of the pad, which could decrease the radiation damping available. Tr. at 7464-65, 7468-71 (Ostadan). However, a plot of vertical displacements on the pad as a function of location on the pad shows that the displacement along the pad is virtually zero for most of the length of the pad and there is one single, gradual, small vertical displacement of the pad at the point of application of the seismic loading, which slowly decreases as one moves away from the point of application of the seismic force. PFS Exh. 227; Rebuttal Testimony of Wen S. Tseng on Section D of Unified

Contention Utah L/QQ [hereinafter Tseng Reb.] Post Tr. 10,727, at 2-3. These results show the absence of “ripples” of the type of concern to Dr. Ostadan, and demonstrate the rigid behavior of the pad under dynamic seismic loadings. Id.

D.27 PFS performed an evaluation of the effects of pad flexibility on the properties of the foundation, based on the methodology described in a recognized technical paper (Iguchi and Luco (1981)) and demonstrated that the effect of flexibility on the foundation damping properties of the pad is insignificant in the frequency range of importance to the cask response. Youngs/Tseng Post Tr. 5529, at 21-22; Tseng Reb. Post Tr. 10,727, at 2. This result is supported by the computer analyses conducted by Sandia Laboratories for the NRC Staff, which incorporated pad flexibility and yielded very small cask displacements under seismic loadings. Tr. at 6788-89 (Luk). Holtec also performed, for another facility, parametric studies that compared the stability of the casks assuming a rigid versus a flexible pad and determined that the differences in the two cases were negligible. Singh/Soler Post Tr. 5750, at 38-39.

D.28 The maximum displacements shown in Table D-1(d) of the ICEC Calculation are on the order of 3/8th of an inch. Youngs/Tseng Post Tr. 5529, at 24; PFS Exh. 85 at 234. These are "very small" displacements. Dep. of Farhang Ostadan (Mar. 8, 2002) (State Exh. 112) at 105. Moreover, Dr. Tseng testified that the maximum displacements in that table include rigid displacements, that is, vertical motions of the entire pad as a rigid body. Youngs/Tseng Post Tr. 5529, at 24. When the rigid displacements are removed, the maximum deviation of local displacements from rigid body motion for the pad is on the order of approximately 1/8 of an inch. Id.; Tr. at 10,754-55 (Tseng). Such a small local displacement would produce only secondary effects on the global dynamic response of the

pad/cask system, and would not affect the stability of the casks. Youngs/Tseng Post Tr. 5529, at 24; Tr. at 5662 (Tseng); Singh/Soler Post Tr. 5750, at 46-47.

D.29 Regarding the State's claim that the Applicant's seismic design entails "conflicting requirements," the Board notes that the State's own experts' testimony opined that these "conflicting" requirements can be resolved by the use of appropriate materials, and testing can show whether or not these requirements have been satisfied. See Tr. at 7451-52, 10,399-401 (Ostadan).

D.30 Furthermore, there is no requirement that the pad be rigid to assure smooth sliding of the cask; in fact, the effects of pad flexibility are only second-order in nature, as Holtec demonstrated in analyses performed for Tennessee Valley Authority's Sequoyah Nuclear Power Plant. Singh/Soler Post Tr. 5750, at 38-39; Tr. at 6014 (Singh). In any case, Holtec used both an upper bound (0.8) and a lower bound (0.2) coefficient of friction between the cask and the pad to account for local irregularities at the interface between the two bodies; in some analyses, Holtec used a random coefficient of friction from 0 to 1 between the cask and the pad. Singh/Soler Post Tr. 5750, at 41; Tr. at 6018-20 (Soler). These variations in the coefficient of friction would account for the potential effects of pad flexibility on cask sliding. Singh/Soler Post Tr. 5750, at 41. We therefore conclude that the effects of pad flexibility on the dynamic behavior of the casks in a seismic event are negligible.

d. Soil-Structure Interaction Analysis

(i) Geomatrix Analysis of the Soil Column

D.31 Geomatrix performed a soil column analysis to obtain the strain compatible soil properties in the free field using a common industry program, SHAKE. Tr. at 7570-72 (Ostadan). Because soil material is highly nonlinear during earthquake shaking, the State

contends that it is common industry practice to perform a soil column analysis without any structures or foundations present and input the design motion at the top of the column, thereby obtaining the properties of the soil as impacted by the design motion. Id. at 7514-15, 7571 (Ostadan). The State insists that, although the SHAKE soil properties are used in SSI analysis, the SHAKE analysis cannot in any way be considered an SSI analysis. See Tr. at 7513 (Ostadan).

D.32 According to the State, the SHAKE analysis is done in the free field and does not take into account the natural frequency of the structure or the foundation or other SSI effects. Tr. at 7515, 7570 (Ostadan). On a relatively soft and layered soil site like the PFS site, the State insists that one needs to account for the following: (1) deformation of the soil; (2) the additional amplification of the seismic motion that could be caused by the soil; (3) radiation damping in the soil; and (4) realistic values for the seismic loads and seismic response of the structure. Tr. at 10,312-13 (Ostadan). The State believes that the seismic response of a structure will be influenced by its natural frequency, which for the pad-foundation system is about 5 to 11 hertz (hz). Id. at 10,418-19 (Bartlett/Ostadan).

D.33 There is no regulatory requirement that the design of cask storage pads include a formal SSI analysis. In addition, there is no claim in this proceeding that the design inputs provided by Geomatrix were inadequate. Nor is there a claim that Holtec's analyses were deficient for failing to expressly quantify soil-structure interaction effects. There is also no claim that the pad is incorrectly designed, except in that long term pad settlement was not considered in the pad's design, which we find to be inconsequential and unrelated to soil-structure interaction. For these reasons, the Board finds the Applicant's SHAKE analysis to be adequate.

(ii) Pad Acceleration

D.34 According to the State, Dr. Ostadan has written and reviewed numerous SSI reports. State Findings ¶ 200. His criticism of Holtec's cask stability report is that it does not discuss or present its results for a reviewer to evaluate and does not quantify any SSI effects. Dr. Ostadan believes that Holtec is focused only on the displacement of the cask and these other questions remain unanswered. Tr. at 7517 (Ostadan).

D.35 Dr. Ostadan contends that Holtec's seismic analysis of the soil properties under the pad is represented by a set of soil springs or soil damping parameters. Tr. at 7565 (Ostadan). The soil properties Holtec used were those from Geomatrix's SHAKE analysis. Id. at 7567.

D.36 According to Dr. Ostadan, the seismic loads from the cask are exerted on the pad and additional seismic loads are due to the mass of the pad, itself. Tr. at 7533 (Ostadan). He notes that the forces exerted on the pad coming from the cask were provided to ICEC by Holtec, but Holtec did not provide the inertial load of the pad itself. By not providing the initial load of the pad itself, Dr. Ostadan believes ICEC and Stone and Webster were forced to make two assumptions. First, ICEC, in the design of the pads, and Stone and Webster, in its pad sliding analysis, had to guess at the inertial load of the pad itself. Second, Stone and Webster also had to guess at the loads coming from the cask. Id. at 7529-36 (Ostadan).

D.37 The State criticizes this analysis because, ICEC was not asked to perform a complete SSI analysis or to analyze damping. Tr. at 7466-67 (Ostadan). ICEC simply applied the dynamic forces obtained from Holtec in the SASSI model to obtain the stresses and moments in the pad for purpose of structural design, and to estimate the amount of steel

reinforcement. Id. at 7467. At best, the State claims that ICEC's analysis constitutes about 10 to 20% of what is needed for a complete SSI analysis. State Findings ¶ 204.

D.38 Dr. Ostadan also argues that to understand the inertial load of the pad requires knowledge of the acceleration of the pad. Tr. at 10,338 (Ostadan). Further, according to the State, from Holtec's analysis it is unknown how much shear force is going to be generated based on the pad itself. In that regard, the State contends that the dynamic loads from Holtec do not include acceleration of the pad or shear forces. State Findings ¶ 205.

D.39 Turning to the dynamic forces for pad stability, the State contends that instead of obtaining the acceleration of the pad from Holtec in the cask stability design calculations contained in PFS Exhibit UU, Mr. Trudeau assumed a number -- peak ground acceleration (0.7g) -- for a design input into the pad sliding analysis. Bartlett/Ostadan Post Tr. 7268, at 18. According to the State, peak ground acceleration (PGA) is the ground motion in the free field and does not account for SSI effects. State Findings ¶ 206. Thus, the State insists that use of PGA for the seismic loads for the pads has nothing to do with the response of the pad. Tr. at 7480-81 (Ostadan). The pad, the soil, and the foundation have their own natural frequency, which ranges from 5 to 11 hz. Id. at 7481. Thus, the State argues that using PGA as the input motion to estimate the seismic loads for the pad is appropriate only for rock sites, which is not the case at the PFS site. Tr. at 7480-81 (Ostadan).

D.40 According to the State, Mr. Trudeau attempts to justify his use of PGA by assuming that there is a tremendous amount of radiation damping -- 48 to 52% -- and with these high damping values, the design motion in the free field (0.7g) is a fairly close fit. Trudeau Section D Post Tr. 6135, at 14-16; Tr. at 6199-6200 (Trudeau); Tr. at 7623 (Ostadan). However, the State argues that Dr. Ostadan, an expert in SSI – compared to Mr. Trudeau

who has little experience in analyzing SSI[38] – found Mr. Trudeau's assertion of high damping values "unusual" for such a foundation system. Tr. at 7623 (Ostadan). Moreover, according to the State, the "simple" calculation that Mr. Trudeau performed to arrive at about 50% damping was not the method he used in his CTB stability calculations. State Findings ¶ 207 (citing Tr. at 6200-01 (Trudeau)). In that regard, although the State has some concerns about the CTB analysis (e.g., damping and treating the CTB mat as rigid), the State believes that Stone and Webster took a logical approach in obtaining the dynamic response of the CTB mat. Tr. at 7530 (Ostadan).

D.41 Because, in his opinion, the report performed by the Staff's expert Dr. Vincent Luk,[39] is about the only place in the record that comes close to discussing pad accelerations, Dr. Ostadan testified that he resorted to Figures 17 and 20b of the Luk Report as an indicator of pad acceleration. Tr. at 10,339, 10,342-44 (Ostadan). Dr. Ostadan contends that Dr. Luk omitted proportional damping from his model and that his analysis tends to over-predict high frequency response, which may be responsible for the 3g acceleration in Figure 17, but Figure 20b does have accelerations beyond 10g. Id. at 10,342-43. In that regard, Dr. Ostadan claims that even though Dr. Luk omitted proportional damping and Figures 17 and 20b only apply to one node, one still can glean high pad acceleration from those figures. Id. at 10,343-44. Moreover, accelerations at low end frequencies in the range of 5 to 7 hertz are still large and indicate high accelerations of the pad. Id. at 10,343.

D.42 For these reasons, the State insists that there is ample evidence to suggest that the acceleration of the pads may be greater than that estimated by PFS. State Findings ¶ 211.

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[38] State Findings ¶ 207 (citing Tr. at 6163 (Trudeau).

[39] For a full description of Dr. Luk's report, see below, Findings E.152-.174.

D.43 Essentially, the question raised by the State is what the correct value of response acceleration for the pad would be and how much of an error would be introduced by using, as PFS did, the PGA (that is, the free-field ground acceleration) as a proxy for the response acceleration of the pad. The short answer is that the horizontal response acceleration computed based on the Holtec analysis would be .79g instead of the .711g used by PFS in its analyses. Rebuttal Testimony of Paul J. Trudeau on Section D of Unified Contention Utah L/QQ [hereinafter Trudeau Section D Reb.] Post Tr. 11,275, at 1, 3-4. Use of the .79g acceleration instead of the PGA employed by PFS would merely result in a slight decrease in the "base case" factor of safety against sliding of the pads from 1.27 to 1.22. Trudeau Section D Reb. Post Tr. 11,275, at 4.

D.44 The Applicant provided ample support for its claim that the horizontal pad response acceleration differs little from the PGA by showing that the radiation damping applicable to the soil/pad/cask system is so high (50% for the "best estimate" soil properties case) that the effects of SSI in terms of amplifying the accelerations imparted on the pad are limited. Therefore, the response acceleration of the pad is essentially equivalent to the free field ground acceleration. Trudeau Section D Post Tr. 6135, at 14-16; PFS Exh. 231; Tr. at 11,280 (Trudeau).

D.45 Dr. Ostadan indicated that he had not seen a calculation that demonstrated the existence of the 50% value of radiation damping estimated by PFS and expressed concern that such a damping level might be unrealistic. Tr. at 7623 (Ostadan). He agreed, however, that if such a level of damping could be established, his concern about the difference between PGA and the response acceleration of the pads would diminish. Tr. at 7624 (Ostadan). PFS subsequently produced a calculation that substantiated the radiation

damping values it used and which was not challenged by the State. See PFS Exh. 231; Tr. at 11,279-81 (Trudeau).

D.46 The Applicant provided additional confirmation of the appropriateness of its use of peak ground acceleration in its cask stability analysis by comparing the factor of safety against sliding of the pads it computed for its base case, 1.27, against the factor of safety that would be obtained using the time history of forces developed by Holtec in its SSI analysis of the pad and casks. The use of this time history of forces at the base of the pad and casks yielded a factor of safety against sliding of 1.25, demonstrating that there is only a very slight reduction in the minimum factor of safety against sliding when these loads are used instead of computing the inertial forces of the pad and cement-treated soil based on the peak horizontal ground accelerations. Trudeau Section D Post Tr. 6135, at 14-16.

D.47 The State's attempt to contrast (1) the horizontal pad acceleration used by the Applicant (.711g) with (2) the response of the mat of the CTB from the SSI analysis performed by PFS for that building (1.047g) is unavailing. As noted by the Applicant's expert, Mr. Trudeau, the CTB is a much taller structure than the pads, hence the SSI effects should be more pronounced for that building than for the pads. Tr. at 6192-93 (Trudeau).

D.48 Similarly, we do not agree with the conclusions Dr. Ostadan drew from his comparison of the Luk Report pad accelerations with the pad accelerations used by Stone and Webster. Bartlett/Ostadan Post Tr. 7268, at 18. During the hearing, Dr. Ostadan acknowledged that at the time he provided the testimony in Answer 37, he had not reviewed the Luk Report in any detail. Tr. at 7781, 7786, 7793, 7798 (Ostadan). Accordingly, he was not aware that the figures from the report on which he relied were obtained by omitting the stiffness proportional damping and were only for a single node, and thus could not be relied upon to be a correct representation of the pad accelerations. Tr. at 7788 (Ostadan); Tr. at

7794-98, 7801-02 (Ostadan). On redirect, Dr. Ostadan reiterated his view that the Luk Report can be read to suggest that high pad response accelerations exist. Tr. at 10,342-44 (Ostadan). However, Dr. Ostadan could not reconcile his assertion that the Luk Report's high accelerations should be given credit with the inconsistent fact that the same report predicts very little displacement of the cask under such accelerations. Tr. at 10,427-28 (Ostadan). In light of this and the rest of the evidence on this point, we find that the State's argument based on the accelerations depicted in the Luk Report to be without merit.

D.49 To summarize, nothing before us indicates the Applicant's use of the PGA to compute the dynamic forces for pad stability is erroneous. In addition, we note that the peak acceleration, whatever its value, will be applied only at a single point in time in the entire time history; therefore, any errors in the computation of that acceleration will be absorbed by the fact that the average factor of safety against sliding is approximately ten throughout the duration of the earthquake. Trudeau Section D Post Tr. 6135, at 16.

D.50 Moreover, as the undisputed testimony of all parties shows, sliding of the pads, if occurring, tends to reduce loading on the casks and is therefore beneficial from the standpoint of cask stability. Therefore, this concern, even if valid, would have no practical impact on the safety of the facility.

e. Pad-to-Pad Interaction

D.51 Dr. Bartlett's and Dr. Ostadan's pre-filed testimony (answer 36) provides a detailed description of the State's concerns with pad-to-pad interaction. Bartlett/Ostadan Post Tr. 7268, at 15-18. They claim that in its sliding stability calculation, Stone and Webster assumed that, for longitudinal column of storage pads, the soil and cement-treated soil under the pads and soil-cement around the pads would move in unison with the pads. In other

words, they contend that Stone and Webster assumed that during an earthquake the different masses of the entire system would be in-phase. Id. at 15-16.

D.52 Next, the State witnesses observed that the storage pads and surrounding soil-cement are not structurally tied together, such as with reinforcing rebar and argued that in Dr. Tseng's deposition testimony, he admitted that the pads and soil-cement would not act as an integrated structure. Bartlett/Ostadan Post Tr. 7268, at 15-16.

D.53 The crux of the State's testimony is that during the cycling of earthquake forces, the following will occur: (1) there will be separation between the soil-cement and the storage pads; (2) the soil-cement and pads will not act as an integrated unit; and (3) the difference in modulus between the very stiff soil-cement and the relative soft upper Lake Bonneville clay will create strain incompatibility and stress concentration in the soil-cement as the gap between the soil-cement and pads attempts to close. As a consequence the State believes that if the soil-cement does not fail in compression, it will act as a strut introducing significant transfer of inertial force through pad-to-pad interaction. Bartlett/Ostadan Post Tr. 7268, at 15-16.

D.54 According to the State, experts the SSI effects will cause the pads, which are only 5 ft. apart from each other, to move differently from the free field motion of the soils. Tr. at 10,380-81 (Ostadan); Bartlett/Ostadan Post Tr. 7268, at 17. The State believes that this phasing of the motion of the pads will create a push and pull action as the pads move towards and away from each other, creating a force transfer that has not been accounted for in the Applicant's pad sliding analysis of the pads and stability analysis of the casks. Tr. at 10,380-81 (Ostadan); Bartlett/Ostadan Post Tr. 7268, at 17.

D.55 In one of the many computer runs that Dr. Soler conducted during the course of the hearing, one scenario involved modeling compression of the soil-cement within a two pad system. Tr. at 10,382 (Ostadan); PFS Exh. 225, at 28. The resulting force transfer from that analysis is reported to be 1900 kips. Tr. at 10,382 (Ostadan). The State argues that this is a large force, which has not been accounted for in the stability analysis of the pads. Tr. at 10,382 (Ostadan). The State has serious concerns about, what it believes is, the already slim margin in the Applicant's pad seismic stability calculation, PFS Exhibit UU, where the base case has only a 27% calculated margin of safety. See State Findings ¶ 217.

D.56 According to the State, the Applicant's attempt to present the pad-to-pad interaction effect is unrealistic because the Applicant insists that the pads will not slide. State Findings ¶ 218 (citing Trudeau Section D Reb Post Tr. 11,275, at 6-7). The State insists that Mr. Trudeau assumes that there will be no sliding because PFS has calculated a 27% factor of safety against sliding and also because of the interface strength and bonding of the soils-cement-treated, soil-pad-soil-cement system. Id. In the State's opinion, however, this does not overcome its concerns, because as Dr. Bartlett testified, pad-to-pad interaction can occur without sliding. Bartlett/Ostadan Post Tr. 7268, at 17. According to the State, the upper Lake Bonneville clay underlying the pads is a relatively deformable body compared to the much stiffer soil-cement plug between the pads. State of Utah Partial Surrebuttal Testimony of Dr. Steven F. Bartlett to Rebuttal Testimony of Paul J. Trudeau on Unified Contention Utah L/QQ (Dynamic Analyses) [hereinafter Bartlett Dynamic Analysis Partial Surrebuttal] Post Tr. 11,306, at 4. Thus, during earthquake cycling there will be SSI effects from the differences in kinematic (stiffness of the soil-cement relative to the deformable clay soil) and inertial (mass differences between the cask-pad system and the soil-cement) properties of the system. Id. An example of these effects, according to the State, is that the relatively stiff

plug of soil-cement will transmit the earthquake force horizontally from pad-to-pad whether or not the pad is sliding. Id. The Holtec Report, discussed in the preceding paragraph, briefly analyzed a simple two-pad system in the longitudinal direction and showed a significant transfer of lateral forces even without initial pad sliding. Id. Certainly, the State insists pad sliding will cause more severe pad to pad interaction effects than calculated by Holtec in PFS Exhibit 225. Id. Furthermore, the State points out that the Holtec calculation did not include the effects of multiple pad interactions. State Findings ¶ 218.

D.57 Testimony by Applicant and Staff witnesses showed that the soils beneath the pad foundations are essentially uniform across the pad emplacement area and have sufficient strength to withstand the DBE loadings without experiencing significant deformation (i.e., strain). Mr. Trudeau testified that the effective shear strain in the clayey soil underlying the soil-cement was only .13%. Trudeau Section D Post Tr. 6135, at 19; Tr. at 6208-09 (Trudeau). While this strain was computed for the free-field, no significant variations in soil strain level would be anticipated if the presence of the pads and the casks was taken into account. Tr. at 6210-12 (Trudeau).

D.58 With respect to soil strength, Dr. Bartlett referred to the upper Lake Bonneville clays as “fairly soft” and “somewhat as a jello.” Tr. at 11,309 (Bartlett). On further examination, however, he acknowledged that the clays have a strength in excess of 2000 pounds psi and are “soft” only when compared with an adjacent soil-cement layer. Tr. at 11,335 (Bartlett). Another description of the strength of these clays was provided by Mr. Trudeau, who stated that the upper Lake Bonneville clays at the PFS site are partially saturated, stiff, and competent. Tr. at 6278 (Trudeau).

D.59 There is also uniformity of properties in the upper Lake Bonneville clay soils across the pad emplacement area. See, e.g., Tr. at 11,726 (Trudeau); Tr. at 11,816-18 (Ofoegbu).

D.60 As we previously noted, to support the Applicant's position, Holtec conducted an analysis in which it modeled two adjacent pads, 5 ft. apart, one pad fully loaded with eight casks, the other having only a single cask, and included a representation of the soil-cement between the pads. Soler Reb. Post Tr. 10,557, at 2. Holtec performed two simulations for this model: one in which the soil-cement between the pads is assumed to retain its integrity and therefore able to transmit both tension and compression forces; and another simulation in which the soil-cement is assumed to be cracked and thus able to transmit only compression forces. Soler Reb. Post Tr. 10,557, at 2.

D.61 The configuration in these cases was set to maximize the potential for pad-to-pad forces. As explained by Dr. Soler: (1) no forces were allowed to be absorbed by the soil-cement; (2) no forces were allowed to be transmitted downwards to the cement-treated soil and to the soils beneath; (3) no damping was included in the model; (4) a maximum value of Young's Modulus for the soil-cement was assumed; and (5) no credit was taken for the potential crushing of the soil-cement by the forces going from one pad to the other. Tr. at 10,657, 10,720-24 (Soler). According to the results of Holtec's analysis, the maximum estimated force in the soil beneath the pads was less than the minimum required to initiate pad sliding. Tr. at 10,723 (Soler). Also, while both cases predicted some interactions between the pads or between the pads and the soil-cement, the forces resulting from those interactions, when added to the seismic loadings, resulted in total cask motions of the same order -- inches -- as had been obtained in prior simulations that had not expressly accounted for pad-to-pad interaction forces. Soler Reb. Post Tr. 10,557, at 3-4.

D.62 Dr. Ostadan made it clear that his concern was not with the effect of pad-to-pad interaction forces on the structural integrity of the pads or the direct effect of these forces on the casks, but only with their potential effect on the foundations. See Tr. at 10,697-700

(Ostadan). Since the results of the Holtec simulation indicate that pad-to-pad interaction forces have essentially no impact on the stability of either the pads or the storage casks, we agree with the Applicant's assessment that pad-to-pad interaction forces have no practical significance.

D.63 State witnesses testified that their concern over pad-to-pad interaction would be magnified if the pads actually were to slide. Tr. at 7520 (Ostadan). The testimony of PFS witnesses, however, is that the design of the cement-treated soil will provide a large margin against the potential sliding of the pads. See, e.g., Trudeau Section D Post Tr. 6135, at 9, 18-19; Trudeau Section D Reb. Post Tr. 11,275, at 6-7.

D.64 The State interpreted the pad-to-pad interaction forces resulting from the two Holtec analyses referenced above as being potentially additive to those included in the PFS sliding stability calculation and resulting in making the forces acting on the pad exceed the available resisting forces and potentially induce pad sliding. Tr. at 10,618-21 (Ostadan). This interpretation is erroneous for two reasons. First, and most significant, the Holtec model is all-inclusive, since it accounts both for the seismic forces acting directly on the pads and for the effects of pad-to-pad interaction. Tr. at 10,618-20 (Soler). In addition, it would be improper to add the maximum seismic forces acting on the pad and the maximum pad-to-pad interaction forces, since they could act at different points in time and, depending on the direction of the pad motion, could be subtractive rather than additive. Id.

D.65 Dr. Ostadan also theorized that there could be configurations in which interaction loads from various pads could accumulate on a single pad and result in potential sliding of the pad, but indicated that without additional analysis he could not specifically postulate any. Tr. at 10,685-91 (Ostadan). Without further support, however, we cannot give credit to his

theory. At any rate, as we discussed above, sliding of the pads is beneficial to the stability of the casks and has no adverse safety consequences.

D.66 In responding to the State's claim concerning effect of interaction between pad and the 5 ft. layer of soil-cement separating the pads, PFS testified that, by virtue of the interface strengths between the concrete pad and the underlying cement-treated soil and between the cement-treated soil and the underlying silty clay/clayey silt, the pads will be bonded to the underlying clayey soils; therefore, because pads will not slide, there will not be interaction between the pad and the soil-cement frame. Trudeau Section D Reb. Post Tr. 11,275, at 6-7. The pads are sufficiently close in the north-south direction that the pads and 5-ft. wide soil-cement plug between them will move in concert with the underlying soils when they deform due to the earthquake loading; and thus, there will be no pad-to-pad interaction. Id.

D.67 In addition, should there be a sliding of the pads leading to a collision with the soil-cement frame across a postulated gap between the two surfaces, the soil-cement will tend to crush under the imparted loading, because there is a significant difference between the compressive strength and modulus of elasticity of the 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). NRC Staff Testimony of Daniel J. Pomerening and Goodluck I. Ofoegbu Concerning Unified Contention Utah L/QQ, Part D [hereinafter Pomerening/Ofoegbu] Post Tr. 6496, at 21-23. The crushing of the soil-cement will limit the magnitude of the force that can be transmitted from one pad to another. Id. at 26. 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 during a collision between the pad and the soil-cement plug will be minor. Id.

D.68 In their direct testimony, PFS experts, Dr. Singh and Dr. Soler provided an answer analogous to the Staff's, indicating that if one postulated the existence of a gap between a pad and the adjacent soil-cement plug, and further postulated that the pads did slide under the design basis seismic event, the closure of the soil-cement to pad gap would lead to horizontal impacts not included in the current analysis. The impact, however, would result in an additional energy absorption by the soil-cement, resulting in minimal changes to the forces on the pad and casks. Singh/Soler Post Tr. 5750, at 44-45.

D.69 In order to test the validity of this hypothesis, Holtec performed an analysis in which it examined the potential effect of a gap between a pad and the adjacent soil-cement layer. The analysis evaluated the impact forces that would be imparted on the pad as a result of its collision with the soil-cement across the gap and the effect of those forces on the stability of the casks on the pad. For this analysis, a single pad fully loaded with eight casks was allowed to slide on the underlying soil and collide with a fixed, rigid soil-cement frame surrounding the entire pad with a clearance gap of approximately 0.6 in. to all edges of the moving pad. Soler Reb. Post Tr. 10,557, at 2. The results of the Holtec analysis for this case indicate that, while there will be impacts between the pad and the surrounding soil-cement, the forces produced by those impacts tend to offset the forces that would be imparted by the gradual application of compression of the pad against the soil-cement, so that the net result is a reduction in the overall forces acting on the pad and the casks, and a reduction by a factor of two in the displacement of the casks. Tr. at 10,564-67 (Soler). In short, the collision between the pad and the soil-cement frame has no discernible adverse impact on the stability of the casks. <u>See</u> PFS Exh. 225.

D.70 The reason for the limited effect of pad-to-pad interactions is that such interactions do not impart forces of sufficient magnitude on the pads to affect the stability of the casks on

the pads. Soler Reb. Post Tr. 10,557, at 4. The speculations by the State witnesses on how those interaction forces may alter the “load path” and cause sliding of individual pads or soil failures do not alter the fact that, as shown by the testimony, pad-to-pad interaction concerns are inconsequential. Again, if the interaction results in sliding of the pads, this effect is beneficial in terms of enhancing the stability of the casks.

f. Pad Settlement

D.71 According to the State, long-term pad settlement was not considered in the Applicant’s structural design of the pads or in Holtec’s cask sliding stability analysis. Tr. at 10,332 (Bartlett). Settlement from differential cask loading could cause dishing or tilting of the pads. Consequently, such an effect would impact Holtec’s cask stability analysis because Holtec assumed a perfectly horizontal planar surface in its cask sliding and stability analyses. Tr. at 10,332-33 (Bartlett).

D.72 The State contends that the PFS estimations of pad settlement have spiraled downward from an initial 5 in. of settlement, to approximately 2 in. to finally, in rebuttal testimony, 0.5 in. Internal Memo from Macie to Trudeau/Georges, Apr. 2, 1997 (State Exh. 211) [hereinafter Macie Memo]; SAR at 2.6-50 (State Exh. 168); Trudeau Reb. Post Tr. 11,275, at 4.

D.73 According to the State’s experts, in 1997 the Applicant predicted total differential pad settlement of 5 in. in one month under full loads. Macie Memo; Tr. at 10,334 (Bartlett). The State also believes that at one time PFS contemplated pre-loading the pads (applying a certain amount of fill to try to take the settlement out before the pads are constructed). Tr. at 10,334-35 (Bartlett). There is no known plan for the Applicant to do any pre-loading. Tr. at 10,335 (Bartlett). While the State admits that the 1997 memo is only a historical document,

not a currently-applicable one, it nonetheless argues that it does point out the long-standing concerns about the settlement of the pads and potential impact such settlement could have upon structural adequacy of the pads. State Findings ¶ 234.

D.74 The State agrees that 2 to 3 in. of settlement is a reasonable estimate of total settlement, but it believes a few inches of settlement is a significant number in foundation design. State Findings ¶ 234. For the case at hand, given Holtec's assumptions of a perfectly smooth surface for point-to-point contact on the bottom of the cask, the State argues that what is important is the relative distribution of the settlement and the angle of inclination of the pad, and how they impact sliding and the inertial forces transferred to the pads and foundation. Tr. at 7763-64 (Ostadan).

D.75 During the hearing, Dr. Ostadan indicated that he expects that, over the long range, the middle of the pad will settle more and the edges will settle less, deforming the pad into a concave shape. According to Dr Ostadan, this deformation may reduce the area of contact between those casks placed on the center of the pad and the surface of the pad, and may influence the rate of cask sliding during a seismic event. Tr. at 7363-65 (Ostadan).

D.76 Pad settlement is due to three mechanisms: immediate settlement similar to elastic settlement as the pad is loaded; consolidation settlement; and long-term, creep-type settlement. Tr. at 7495-96 (Bartlett). Immediate settlement is over in a matter of days; consolidation settlement occurs over a term of months, or at most a few years; and creep settlement takes place over the full design life of the facility. Tr. at 7495-96, 7644-46 (Bartlett).

D.77 The estimated total long-term settlement of the pads was computed in Stone and Webster Calculations 05996.02-G(B)-03, Rev. 3, <u>Estimated Static Settlement of Storage</u>

Pads, and 05996.02-G(B)-21, Rev. 0, Supplement to Estimated Static Settlement of Cask Storage Pads (May 21, 2001). As explained in those calculations, the settlement of the pad is predicted based on conservative assumptions that result in an upper-bound estimate of approximately 1.75 in. for the total long-term settlement of the pads. Trudeau Section D Reb. Post Tr. 11,275, at 4; SAR at 2.6-50 (State Exh. 168).

D.78 Mr. Trudeau testified that, based on the conservatisms incorporated in the pad static settlement analyses, the actual long-term static settlement of the pads that can be reasonably expected to occur would be much less than the 1.75 in. that is predicted in the Stone and Webster calculations -- which is only one fourth to one third of this estimated value, or approximately 0.5 in. Trudeau Section D Reb. Post Tr. 11,275, at 4.

D.79 In his testimony, Dr. Bartlett indicated that he did not contest the 1.75 in. total settlement reflected in the most recent PFS calculation. See Tr. at 11,347-48 (Bartlett). Dr. Ostadan, on the other hand, testified that there would be an additional impact on the settlement experienced by any one pad from the loading of other pads. Tr. at 7765-73 (Ostadan). Such an impact, however, would appear to be at best a second order effect considering the distance between pads and the fact that Dr. Ostadan assumes that the pad deflection will be greatest at the center of the pads and least at the edges, where any impact of the settling of adjacent pads would be experienced. Id.

D.80 We note that, according to State Exhibit 168, the 1.75 in. of maximum, long-term pad settlement is computed assuming that the entire upper layer of subsoil has the same compressibility characteristics as those of the upper Bonneville Lake deposits, which as discussed above are the weakest and most compressible soils at the PFS site. Therefore, we conclude that, as indicated in State Exhibit 168, the 1.75 in. estimate “conservatively overestimates the expected settlements.” In light of this clear overestimation, we regard the

0.5 in. estimate provided by Mr. Trudeau, from which the known conservatisms have been removed, as reasonable. Such settlement levels would raise no significant stability concerns. Tr. at 11,125 (Mitchell).

D.81 The Applicant and the Staff witnesses also testified that, because of the great stiffness contrast between the concrete pad and the underlying clayey soils, the long-term settlement of the pads at the PFS facility will be essentially uniform across the pad. Thus, its effect on the dynamic response of the pads and the casks supported on the pads should be negligible. Trudeau Section D Reb. Post Tr. 11,275, at 6.

D.82 Dr. Bartlett disputed the assessment that the long-term settlements of the pad would be essentially uniform. Bartlett Dynamic Analysis Partial Surrebuttal Post Tr. 11,306, at 3. However, as Dr. Bartlett himself explained, determining the distribution of the estimated maximum settlement of a foundation is difficult and involves choosing between (1) assuming it occurs at the center of the pads to maximize "the dishing effect," (2) assuming it all occurs on one side of the foundation so as to produce some tilting, and (3) distributing the total settlement over the minimum footing width, to emphasize differential settlement with adjacent structures. Tr. at 11,349-50 (Bartlett). In reality, the assumption of uniform pad settlement is the only one supported by physical considerations, as pointed out by Mr. Trudeau and Dr. Ofoegbu.

D.83 The main consequence posited by Dr. Ostadan of the long-term settlement of the pads would be altering the pattern of cask sliding on the pad by giving rise to a "dishing" effect in the middle of the pad that would make it somewhat more difficult for a cask to slide at some points and easier to slide at others. Tr. at 7501-02 (Ostadan). However, assuming that there was a 0.5 in. differential settlement in the center of a pad relative to the pad's edges, the average slope measured along the short end of the pad would be only 0.159

degrees. Soler Reb. Post Tr. 10,557, at 4. Such a slight slope would have no significant impact on the motion of the casks. Id.

D.84 The State witnesses sought to distinguish this effect from the expected local variations in the coefficient of friction between the cask and the pad. Tr. at 7502-06 (Ostadan/Bartlett). However, as discussed above, the cask stability analyses performed by Holtec utilized a variety of friction coefficients, including random variations in such coefficients, and in no case was a substantial amount of cask displacement observed. Therefore, it does not appear likely that the long term settlement phenomenon will induce cask motions that differ significantly from those obtained in the Holtec analyses. Soler Reb. Post Tr. 10,557, at 4.

D.85 Another potential consequence of long term settlement of the pads postulated by the State was a “slight inclination” or tilting of the pads. Tr. at 7500, 11,323-24 (Bartlett). However, the maximum angle of tilting of the pad resulting from such settlement would be on the order of only 0.64 degrees. See Tr. at 7761-63 (Bartlett). That level of tilting could result in effectively changing slightly the coefficient of friction between the cask and the pad. Tr. at 7504 (Bartlett). Pad tilting is accounted for in the Holtec analysis and shown to have only secondary effects on the stability of the casks. Tr. at 6012-14 (Soler/Singh).

D.86 Witnesses for both Applicant and the Staff testified that the anticipated long-term settlement of the pads does not pose a concern in terms of the dynamic stability of the foundations and constitutes, at most, a maintenance issue. Trudeau Section D Reb. Post Tr. 11,275, at 5; Pomerening/Ofoegbu Post Tr. 11,001, at 10-11. Based on the evidence in the record, we agree.

g. CTB Analysis

D.87 The State argues that PFS cannot meet a factor of safety of at least 1.1 without the buttressing effect of soil-cement around the foundation perimeter of the CTB basemat, yet not until some distant future date will PFS acquire any data that it may arguably rely upon to support its use of soil-cement. State Findings ¶ 241. The design calculations for the sliding stability of the CTB under a 2000-year DBE are found at PFS Exhibit UU and as the State notes, there are no sliding calculations for a 10,000-year mean return period earthquake. Tr. at 6348 (Trudeau). The State contends that furthermore, there are no engineering calculations or performance data to support the presumed passive resistance PFS expects to obtain from using such a mass of soil-cement around the perimeter of the CTB mat foundation for the 2000-year DBE. Bartlett/Ostadan Post Tr. 7268, at 21. Nor, continues the State, is there any analysis of the effects of separation and cracking caused by out-of-phase motion of the CTB mat foundation and the soil-cement buttress or how bending and tensile stresses that develop in the soil-cement will resist seismic forces without cracking or separation. Bartlett/Ostadan Post Tr. 7268, at 21. Furthermore, the State notes that the base mat of the CTB is expected to settle 3 in.; however, the State claims that the effects of this settlement of the integrity of soil-cement and its separation from the CTB on the passive resistance have not been considered by PFS. State Findings ¶ 241.

D.88 The State also challenges Mr. Trudeau's statement that he considered SSI in the CTB Analysis contained in PFS Exhibit VV: "[i]nasmuch as the loads came from our structural dynamics people." State Findings ¶ 242 (citing Tr. at 6191 (Trudeau)). According to the State, however, there has been no dynamic analysis of the interaction of the soil-cement with the CTB mat foundation for the 2000-year DBE. Bartlett/Ostadan Post Tr. 7268, at 21. Under the DBE, the maximum horizontal acceleration response of the CTB mat is

1.047g. Tr. at 6192 (Trudeau). The free field peak horizontal ground acceleration response of the adjacent soil-cement buttress is 0.71g. Tr. at 6264 (Trudeau). Consequently, the State argues that there is a 47% difference between the horizontal response of the CTB and the surrounding soil-cement. Id. The soil-cement buttress is not structurally tied to the CTB mat foundation, and given the large differences in horizontal acceleration response between those two masses, the State believes there is a significant potential for out-of-phase motion resulting from this inertial interaction. Bartlett/Ostadan Post Tr. 7268, at 21; Tr. at 6265 (Trudeau). According to the State, the Applicant has not considered the reduction of foundation damping and the concomitant higher seismic loads or the kinematic motion of the CTB caused by the blanket of soil-cement around the CTB foundation. Bartlett/Ostadan Post Tr. 7268, at 21. The State's additional concern here is that soil-cement will not provide passive resistance. All of these factors, the State argues, affect the PFS design calculation in meeting a factor of safety of at least 1.1. Bartlett/Ostadan Post Tr. 7268, at 21.

D.89 Similar to the concerns the State raised on the rigidity or flexibility of the storage pad, the State questions whether the Applicant has appropriately treated the CTB mat as rigid. Bartlett/Ostadan Post Tr. 7268, at 21. Mr. Ebbeson testified that the "potential effect of mat flexibility is accommodated by the factor of safety applied in the seismic stability calculations." Ebbeson Post Tr. 6357, at 14. The State insists, however, that the factor of safety against sliding in Calulation No. G(B) 13 has one case at a minimum of 1.15 and another, which PFS claims to be a conservative analysis, of 1.26. State Findings ¶ 213. Furthermore, the State argues that there are no design calculations to support the Applicant's assumption that the foundation mat is rigid. See Bartlett/Ostadan Post Tr. 7268, at 21.

D.90 Because the use of soil-cement is contributing to the PFS demonstration of meeting a factor or safety of 1.1 against sliding, the State insists that it is essential that there are analyses, data, and engineering calculations to support the claimed resistance to sliding, including whether the soil-cement will, in fact, perform as intended during an earthquake. In that regard, the State believes that PFS has made no such showing. See State Findings ¶ 244.

D.91 The only failure mechanism that is being raised as potentially occurring with respect to the CTB is sliding. See Tr. 7655-56 (Bartlett); Tr. 7663, 7674 (Ostadan). If such sliding were to occur, we note that it would have no safety consequences, since there are no safety-related structures connected to the building that could be adversely affected by the sliding. Tr. 7323-25 (Bartlett, Ostadan); Trudeau Section D Post Tr. 6135, at 21; Ebbeson Post Tr. 6357, at 14.

D.92 Also, as the Staff witnesses testified, it is not necessary to meet a factor of safety of 1.1 against sliding in order to satisfy the regulatory requirements of Part 72. Part 72 requires that the SSCs important to safety be shown to perform their safety functions when subjected to seismic loadings. The sliding analyses performed by PFS indicate that this condition will be met, whether or not the factor of safety recommendations in the Standard Review Plan for nuclear power plants are satisfied. Tr. at 6594-96, 6739-41 (Ofoegbu). Therefore, the claims raised by the State in Section D of Contention L/QQ with respect to the dynamic stability of the CTB have no licensing significance.

h. Transfer Options

D.93 During the hearing, the Applicant's expert Mr. D. Wayne Lewis testified that the process of transferring an MPC containing spent fuel from the shipping cask in which it is

brought to the PFS site to a storage cask located on its pad takes approximately 20 hours. Testimony of Donald Wayne Lewis on Section E of Unified Contention Utah L/QQ [hereinafter Lewis] Post Tr. 8968, at 4. He also testified that, within that period, the total time that the MPC is not completely sealed within either a shipping cask or storage cask is 9 hours per operation (from initiation of the removal of the HI-STAR cask closure plate bolts to completion of the installation of the HI-STORM cask lid and bolts). Lewis Post Tr. 8968, at 4. Finally, he testified that the total time the canister is being lifted directly or in the transfer cask and held by the crane in the transfer cell while being transferred from the shipping cask to the storage cask is approximately 3 hours per transfer operation. Lewis Post Tr. 8968, at 4.

D.94 The gist of the State's arguments is that the testimony of Mr. Lewis provides a potentially inaccurate estimate of the duration of the cask transfer operations in the CTB. The State presented no testimony on cask transfer issues. See State Findings ¶¶ 165-68.

D.95 Counsel for the State sought to cast doubts on the estimates provided by Mr. Lewis and he was asked to explain in detail how each estimate was obtained, the basis for the estimated length of the operations involved, and the manner in which the various lengths were computed. See, e.g., Tr. at 8984-86, 8997-99, 9008-10, 9020-22, 9039-41, 9043-44 (Lewis). Despite intense probing by counsel, the State was unable to elicit any retraction or modification of the estimate.

D.96 The State argues that the evidence does not support Mr. Lewis' claim that the times in the Holtec Table (PFS Exh. AAA) are based on experience in loading Holtec casks. State Findings ¶ 165. To support this argument, the State notes that the CoC for HI-STORM was issued May 31, 2000; the first HI-STORM cask was loaded on June 26, 2001; and the first HI-STAR loaded with a MPC occurred on July 6, 2000. State Findings ¶ 165. Further, the

State notes that Table 5.1-1, Rev. 0, is part of the Applicant's original 1997 license application. The State also argues that the table from which PFS estimates the canister transfer operations time is not derived from actual Holtec cask transfer operations. State Findings ¶ 165.

D.97 Mr. Lewis testified that many of the activities encompassed by the estimate routinely take place at operating nuclear plants such as Hatch and Dresden. Tr. at 9046-47 (Lewis). He also stated it was his understanding that the estimates are based on operational or pre-operational activities at those plants. Tr. at 9059, 9078-80 (Lewis). Mr. Lewis also worked with personnel from utilities that have been actually involved in loading similar casks at the Point Beach and Palisades nuclear power plants to develop his estimates. Tr. at 8982 (Lewis).

D.98 The State tries to undercut the 20-hour total estimate by noting that it is an operational worker exposure time that would occur over a 3-day period. State Findings ¶ 166. Mr. Lewis explained that the entire transfer operation described in PFS SAR Table 5.1-1 (PFS Exh. ZZ) could extend for up to 3 days, but 20 hours would be the duration of the actual operation. Tr. at 9074-76 (Lewis).

D.99 The State attacks the 9-hour estimate of the time that it will take to transfer the MPC from the transportation to the storage cask by pointing out that there are no licensing commitments or regulatory requirements that require PFS to complete the operation within a working day. State Findings ¶ 167. However, Mr. Lewis made it clear that there was no condition under which the MPC would be allowed to remain overnight outside the protection of a shipping or a storage cask. Tr. at 9073-74, 9077 (Lewis).

D.100 Finally, the State tries to undercut Mr. Lewis's testimony that it will take 2.8 hours from the time the MPC is placed in a transfer cask to the time it is placed in a storage cask. The State makes reference to the deposition testimony of Holtec's Dr. Singh that it should be possible to complete the transfer "during the course of the day," with a day meaning a working day, which can be 8 to 12 hours. State Findings ¶ 167. However, Mr. Lewis interpreted this testimony to mean that the transfer should be completed within a day, but not necessarily take the whole day, which was consistent with his testimony. Tr. at 9031 (Lewis).

D.101 Mr. Lewis repeatedly testified that the estimated durations of the various activities involved in the transfer of the MPC from the transportation cask to the storage cask were conservatively high and that some of the steps could be accomplished concurrently, thus further saving time from the overall duration. <u>See</u> Tr. at 8974, 8975, 9011, 9033, 9041, 9044, 9066 (Lewis). Therefore, even if there was some imprecision in the estimated durations of some of the activities, the estimates would still remain conservative and would represent upper limits to the portion of the time during which the safety-related equipment in the CTB would be required to operate, or during which the MPC would be without the protection of a transportation or storage cask.

E. Cask Stability

1. General Overview

E.1 The HI-STORM System consists of a massive cylindrical steel and concrete storage cask surrounding a multi-purpose stainless steel canister in which the SNF is sealed. Each cask is approximately 20 ft. tall (239.5 in.) and approximately 11 ft. in diameter (132.5 in.). When loaded with a spent fuel canister, the casks 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 shielding concrete is installed. These walls are approximately 30 in. thick. The MPC in which the spent fuel is sealed is stored vertically within the storage cask. Singh/Soler Post Tr. 5750, at 7.

E.2 The storage 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. The inner shell of the storage cask has channels attached to its interior surface to guide the MPC during insertion and removal. These channels would also provide a flexible medium to absorb impact loads under postulated, non-mechanistic tip-over events, while allowing cooling air to freely circulate through the cask. Singh/Soler Post Tr. 5750, at 7.

E.3 The HI-STORM System storage cask is designed as a buttressed ASME Section III, Class 3, Subsection NF cylindrical structure. The outer steel shell (which is 0.75in. thick) and the inner steel shell (which is 1.25 in. thick) are both welded to a 2 in. thick steel baseplate, and are joined by four full-length inter-shell radial steel support plates, each 0.75 in. thick and welded to the inner and outer shells. The concrete shielding is placed within this steel weldment. The cask provides an internal cylindrical cavity, 191.5 in. in height and 73.5 in. in diameter, for housing 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. As stated earlier, steel channels are located on the interior surface of the inner shell which act to minimize the loadings that would be imparted to the MPC in a postulated, hypothetical cask tip-over scenario. Singh/Soler Post Tr. 5750, at 10. The circular gap between the channels and the MPC varies from 0.75 to 4.75 in. diametrically. Thus, the effective radial gap between the MPC and the channels which retains the MPC in place over most of its axial extent is 3/8 of an inch. Tr. at 6104-05 (Singh).

E.4 The multi-purpose canister is the component in which the spent fuel is placed. The spent fuel is loaded into the MPC at a nuclear power plant site, after which, the MPC is filled with an inert gas (helium) and welded shut for storage at the plant site or ready for transport off-site. The MPC consists of (1) the stainless steel enclosure vessel and (2i) the 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 in the MPC enclosure vessel. Singh/Soler Post Tr. 5750, at 11.

E.5 Holtec performed seismic analyses for the HI-STORM 100 to be used at the PFS facility using the general design parameters for the HI-STORM 100 together with the site-specific earthquake ground motions for the PFS site and other relevant site-specific parameters. The analyses showed that under DBE conditions for the PFS facility, the loaded HI-STORM 100 casks have large safety margins against overturning or sliding. In no case did the analyses predict that there would be any cask tip-over or any cask-to-cask impacts. Singh/Soler Post Tr. 5750, at 20-24.

E.6 Under the DBE, the Holtec model showed a maximum displacement of the cask on the order of 3 to 4 in. The maximum angle of tilt indicated by the analysis for the 2000-year DBE for an upper bound coefficient of friction of 0.8 is 1.026 degrees. Singh/Soler Post Tr. 5750, at 24-26. This can be compared to the angle of tilt of 29.3 degrees at which a cask would tip if slightly disturbed, due to the moment of its own weight (i.e., the orientation at which the center of gravity of the cask is directly over the edge of the cask). This provides a safety factor against cask tip-over for the PFS facility DBE of 29.3/1.026, or 28.6. Singh/Soler Post Tr. 5750, at 26.

E.7 Holtec also performed an analysis of the performance of a loaded HI-STORM storage cask subject to accelerations from a postulated, beyond-design basis, 10,000-year return period earthquake for the PFS site. The earthquake had a vertical peak ground acceleration (PGA) of 1.33g and horizontal PGAs of 1.25g and 1.23g. Singh/Soler Post Tr. 5750, at 27. The loaded cask exhibited larger rotations relative to the pad (approximately 10.89 degrees from the vertical) than in the earlier analyses using the DBE levels, but the results of this analysis still showed the existence of significant margins against tip-over. Using the same definition of safety factor against cask overturning as before, the safety factor against overturning for the 10,000-year return period earthquake was 2.69 (29.3/10.89). Id.

E.8 Holtec performed a series of additional beyond design basis analyses under a variety of assumptions. Those using the 10,000-year return period earthquake showed maximum rotations on the order of 10 to 12 degrees, confirming the large margins of safety against cask tip-over stated above. Tr. at 5774-76, 5787-88 (Soler).

E.9 Holtec also evaluated the results of a hypothetical cask tip-over event with the attendant impact of the cask on the pad. This tip-over analysis showed that the maximum fuel deceleration is below 45g, which is a licensing limit set by the NRC Staff. Singh/Soler

Post Tr. 5756, at 32-33. As discussed below, staying within the 45g limit ensures that, in reality, a very large safety margin exists against canister breach and potential releases of radioactivity. Therefore, even assuming that a cask were to tip over, the cask tip-over analyses conducted by Holtec show that no breach of the cask or release of radioactivity from the cask would occur.

E.10 To perform its design basis analyses, Holtec used its specially-developed computer code known as DYNAMO. Singh/Soler Post Tr. 5750, at 14. This code has been validated and has been reviewed and accepted by the NRC for the licensing of free-standing spent fuel storage systems. Singh/Soler Post Tr. 5750, at 14-17, 10-20. It has been used by Holtec to perform the seismic analyses in its SAR for the HI-STORM System which supports the CoC that the NRC has issued for the HI-STORM 100 Cask Storage System under 10 C.F.R. Part 72. Singh/Soler Post Tr. 5750, at 14. Holtec has also performed site-specific seismic analyses using DYNAMO for the HI-STORM System for spent fuel systems for Pacific Gas & Electric (Diablo Canyon), Exelon (Dresden), Energy Northwest (Columbia Generating Station), Entergy Nuclear Northeast (J.A. Fitzpatrick) and Tennessee Valley Authority (Sequoyah). All of these analyses have been for storage casks on concrete storage pads. Singh/Soler Post Tr. 5750, at 14.

E.11 In addition, Holtec has extensive experience in using DYNAMO for the seismic analysis of spent fuel racks used to store spent fuel at nuclear power plants. Singh/Soler Post Tr. 5750, at 14. The spent fuel racks are large free-standing rectangular structures of honeycomb construction that sit in the spent fuel pool. These racks are square or rectangular, are supported by four or more stubby legs, and rest on the spent fuel pool floor slab. During a seismic event, the racks may slide, tip, and rotate with respect to the spent fuel pool in a manner similar to the potential motions of a storage cask on a concrete storage

pad. The same non-linear phenomena (sliding and tip-over) are modeled with the additional feature that fluid coupling between racks, and between racks and walls, is also considered. Holtec has employed its wet storage seismic simulation methodology using DYNAMO at numerous nuclear sites (more than forty), both in the U.S. and abroad. Singh/Soler Post Tr. 5750, at 14-17.

E.12 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 class of problems for which it is used in accordance with ASME NQA-2a-1990, Part 2.7, "Quality Assurance Requirements of Computer Software for Nuclear Facility Applications". 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. The problems were chosen to demonstrate all of the features that are built into DYNAMO. In addition, problems that had no simple analytical solutions were also evaluated and shown to give good agreement with numerical solutions using other industry 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 license submittals, DYNAMO was subjected to additional validation at the request of NRC's reviewers. In every case, the DYNAMO code proved capable of providing acceptable solutions to the problem. Thus, DYNAMO has been extensively benchmarked to confirm its adequacy as a non-linear dynamics code. Singh/Soler Post Tr. 5750, at 20.

E.13 In performing the seismic cask stability analysis for the PFS facility, Holtec modeled the casks as free-standing structures on the concrete storage pads with compression-only contact and friction elements modeling the interfaces between casks and the pad. The casks, along with their loaded internals, were modeled as rigid bodies. Singh/Soler Post Tr.

5750, at 20-21. The concrete storage pad was modeled as a rigid rectangular slab, and the effect of the soil/soil-cement foundation was modeled by springs and dampers to characterize the soil resistance in deflection and rotation. Singh/Soler Post Tr. 5750, at 20. Data characterizing the earthquake excitation (acceleration time histories) and the soil response (soil properties used to characterize the soil springs and dampers) were provided to Holtec as design inputs by Geomatrix. Singh/Soler Post Tr. 5750, at 21.

E.14 Specifically, Geomatrix provided Holtec with sets of “Best Estimate,” “Lower Range,” and “Upper Range” soil properties for the soil under the pad, including the effect of soil-cement, as applicable. Holtec then computed the values of the spring constants and damping coefficients for use in its analyses using the soil property values supplied by Geomatrix. This was done in accordance with the formulas provided in ASCE Standard 4-86, “Seismic Analysis of Safety Related Nuclear Structures and Commentary,” Tables 3300-1 and 2, and Figure 3300-3. Singh/Soler Post Tr. 5750, at 21-22. These formulas are derived from a well-recognized technical treatise, Newmark, N. M., and Rosenblueth, E., Fundamentals of Earthquake Engineering, Prentice-Hall, Inc., Englewood Cliffs, N.J., 1971. Tseng Reb. Post Tr. 10,727, at 5.

E.15 Geomatrix also supplied Holtec with the ground motions for the 2000-year return period design basis seismic event in the form of three acceleration time histories entitled “Fault Normal,” Fault Parallel,” and “Vertical.” These seismic ground motions were developed to match a 5%-damped response spectra having the following zero period acceleration (ZPA), also known as the PGA values:

Fault Normal – 0.711g
Fault Parallel – 0.711g
Vertical – 0.695g

The actual time histories used in the dynamic analyses, developed in accordance with Section 3.7.1 of the NRC Staff's Standard Review Plan (NUREG-0800), had the following peak acceleration amplitudes:

Fault Normal – 0.73g
Fault Parallel – 0.71g
Vertical – 0.73g

In the design basis analysis, Holtec applied these acceleration time histories at the base of the soil springs with the spring constants and damping values computed as described above. Singh/Soler Post Tr. 5750, at 21-22.

E.16 For the design basis analysis, Holtec modeled various configurations of one to eight casks on the concrete pad using the lower bound, best estimate and upper range soil properties. To model the effect of friction between the cask and pad, Holtec used an upper bound coefficient of friction of 0.8 at the cask/pad interface (to emphasize or increase the likelihood for cask tipping) and a lower bound coefficient of friction of 0.2 (to emphasize or increase the likelihood of cask sliding). Singh/Soler Post Tr. 5750, at 23. To model the compression contact at the cask/pad interface, Holtec used a vertical contact stiffness of 454,000,000 pounds per in., and to model the loss of energy that would occur should the cask lift up and impact down on the pad Holtec used an impact damping value of 5%. Singh/Soler Post Tr. 5750, at 41, 79. The vertical contact stiffness and impact damping were modeled using springs and dampers at the cask/pad interface with the appropriate values. Singh/Soler Post Tr. 5750, at 41.

E.17 Nine cases were run for the upper bound coefficient of friction of 0.8, and one case was run for a lower-bound coefficient of friction of 0.2 for the configuration that gave the limiting results using upper bound coefficient of friction of 0.8. As explained during the

hearing, the reason only one case was run at the 0.2 coefficient of friction was that previous cask stability analyses that Holtec had performed for the PFS facility for different earthquakes showed that the bounding solution for cask displacement (as measured at the top of the casks) was for a coefficient of friction of 0.8. Singh/Soler Post Tr. 5750, at 24.

E.18 As stated above, for the 2000-year DBE, the Holtec analysis using the upper bound coefficient of friction of 0.8 showed a maximum displacement of the cask on the order of 3 to 4 in. with a corresponding maximum angle of tilt of 1.026 degrees, which provides a factor of safety in the angle of tilt of 28.6 when compared to the angle of tilt at which a cask would tip over from the moment of its own weight. The case evaluated for a coefficient of friction of 0.2 produced a maximum sliding displacement on the order of 2 in. Singh/Soler Post Tr. 5750, at 25-26.

E.19 In addition to its design basis cask stability analyses using DYNAMO, Holtec undertook various beyond-design-basis cask stability analyses. For these analyses, Holtec used the VisualNastran computer code because the beyond-design-basis analyses that Holtec conducted were mostly for the 10,000-year earthquake level. DYNAMO is a small deflection program, which means that it cannot accurately model large cask rotations or displacement, whereas VisualNastran is capable of modeling large rotations of the cask that could occur under the 10,000-year earthquake event. Singh/Soler Post Tr. 5750, at 62.

E.20 Holtec ran various cask configurations under different assumptions to evaluate the response of the casks to a 10,000-year return period earthquake with a vertical PGA of 1.33g and horizontal PGAs of 1.25g and 1.23g and to respond to specific issues raised by the State and its witnesses. The results of these analyses are set forth in the report, “PFSF Beyond Design Basis Scoping Analyses” (Holtec Report No. 2022854) (PFS Exh. 86C) [hereinafter PFS Scoping Analyses], which is supported by PFS Exhibit 86D. The results are

also set forth in the report “Additional Cask Analysis for the PFSF” (Holtec Report No. 2022878) (PFS Exh. 225) which is supported by PFS Exhibits 225B, and 225D. In addition, using VisualNastran, Holtec produced visual simulations from the analyses which are contained in the “movies” collected in PFS Exhibits OO and 225A.

E.21 The simulations of the 10,000 year beyond DBE showed some instances of large cask rotations, on the order of 10 to 12 degrees. Tr. at 5774-76 (Soler). Even with such large rotations, the casks still have a factor of safety in excess of two when compared to the angle of tilt (29.3 degrees) at which a cask would tip over from the moment of its own weight. Singh/Soler Post Tr. 5750, at 26.

2. Singh/Soler

a. Asserted Conflict of Interest for Drs. Singh and Soler

E.22 The State argues that Drs. Singh and Soler have a unique interest in the outcome of this hearing compared to all the other witnesses, in that Drs. Singh and Soler have an extensive financial interest in the Applicant prevailing in this case. Dr. Singh is the president and chief executive officer of Holtec, and Dr. Soler is the executive vice president. Tr. at 5907-08 (Singh). Drs. Singh, Soler, and another individual hold sole interest in the privately owned company, Holtec. Id. at 5917.

E.23 At the time of the hearing, Holtec had only approximately twelve storage casks in use, all of which are HI-STORM 100 casks. Tr. at 5918 (Singh). If the PFS facility attains fruition, Holtec (effectively Drs. Singh and Soler) has the potential to sell 4000 storage casks and other products such as the HI-TRAC canister cask to the PFS project. Id. at 5910-11, 5920. Dr. Singh admitted that sales to PFS could reach hundreds of millions of dollars. Tr. at 5910-11, 5920 (Singh).

E.24 Thus, the State contends the financial rewards from the successful outcome of this proceeding in favor of the Applicant are substantial. If the PFS facility is licensed, the State contends that the financial benefits to Drs. Singh and Soler, as two of three sole owners of the privately owned company Holtec, will pale in comparison to the usual expert witness compensation. The State argues that Drs. Singh and Soler have a substantial interest in both the licensing of the PFS facility and the affirmation by this Board of the Holtec analyses, including those conducted with the DYNAMO code, also owned, in part, by Drs. Singh and Soler. Based on the Licensing Board's decision concerning the propriety of Holtec's codes and methodologies, the State contends that the outcome in this case may have far-reaching effects on Holtec's business. State Findings ¶ 259.

E.25 In response, the Applicant argues that there does not exist a legal doctrine to support the State's claim of a conflict of interest. According to the Applicant, a conflict of interest typically implies that a party has assumed conflicting obligations, a situation which the Applicant contends is not present here. Applicant Reply at 148.

E.26 Bias or interest in the outcome of this case "goes only to the persuasiveness or weight that should be accorded the expert's testimony." Louisiana Power and Light Co. (Waterford Steam Electric Station, Unit 3), ALAB-732, 17 NRC 1076, 1091 (1983) (citing 11 J. Moore & H. Bendix, Moore's Federal Practice ¶ 702.30[1] (2d ed. 1982)). The State alleges that Drs. Singh and Soler have a bias and interest in the outcome of this case. Accordingly, the State requests that the Board to consider those biases and interest in our deliberation of the weight to accord their testimony and other evidence relevant thereto. We have considered the potential biases and interests raised by the State and find no reason to discredit the testimony of either Dr. Singh or Dr. Soler.

b. Experience

E.27 Holtec testified that it performed site-specific cask stability analyses for the PFS site and five other ISFSIs. Singh/Soler Post Tr. 5750 at 14. The State argues that only at three of the five ISFSI sites did Holtec analyze free-standing casks: the Dresden site, where ZPA is 0.2g (State Exh. 121 at 38); the Entergy Northwest (Columbia Generating) site, where the ZPA is about 0.5g (State Exh. 120 at 18); and the Tennessee Valley site, where the ground motion is approximately 0.5 to 0.6g (State Exh. 121 at 38). At the fourth ISFSI, J.A. Fitzpatrick, there is no record evidence of the ground motions. According to the State, other than for the PFS site, the only other site where Holtec has conducted a nonlinear analysis is at Diablo Canyon, a site with the ground motions were as high as the 2000-year earthquake at PFS, but this was on the HI-STORM 100SA, the anchored version of the HI-STORM 100 cask. The State contends that Holtec's analysis of the anchored HI-STORM 100SA is not comparable to an analysis of an unanchored cask. In addition, the State believes that the record does not support the Applicant's claim that Dr. Soler and the other Holtec analysts have any previous experience conducting nonlinear seismic analysis of free-standing casks at ground motions that equal or exceed the 0.7g ground motion for a 2000-year earthquake at the PFS site. State Findings ¶ 262.

E.28 Additionally, according to the State, there are no known or anticipated sites that store or will store the unanchored casks supported by soil-cement or cement-treated soil. State Findings ¶ 263. The State also contends Dr. Soler and other Holtec analysts have no previous experience conducting nonlinear seismic analyses of free-standing casks supported by cement-treated soil or soil-cement foundations.

E.29 Furthermore, regarding Holtec's prior experience analyzing free-standing spent fuel racks, the State claims that in the analysis of spent fuel racks, the racks are submerged in

water and there are very small gaps between the racks; therefore, the nonlinear stability analysis of a cask is "very different" from a free-standing spent fuel rack. Tr. at 7143 (Khan). The State argues that the record lacks sufficient facts to conclude that the analyses of free-standing spent fuel racks are relevant to the experience and training necessary to conduct nonlinear seismic analysis of objects potentially subject to large deformation and rotations at high ground motions. State Findings ¶ 264.

E.30 In its analysis, Holtec modeled the effects of SSI through soil springs (linear and rotational) and dampers. Tr. at 5993 (Soler). Dr. Soler and Chuck Bullard, a Holtec employee, authored the various cask stability reports for the PFS site. Tr. at 5992 (Soler). The State claims that Dr. Soler admitted that neither he nor Mr. Bullard had expertise in analyzing soil dynamics and foundation design (calculating the soil springs and dampers). The State argues that besides the analysis for Tennessee Valley Authority, the only soil dynamic work that Dr. Soler has performed is for this case. State Findings ¶ 265.

E.31 In sum, the State argues that, (1) Holtec has not performed seismic analyses of free-standing casks at sites with ground motions equal to or greater than the 2000-year earthquake at PFS; (2) neither Dr. Soler nor any other identified Holtec analyst are experts in soil mechanics; (3) Dr. Soler and another Holtec analyst have calculated the soil springs and dampers at only one other site; (4) evidence is lacking that Holtec has prior experience analyzing seismic pad-to-pad interaction; and (5) there is a lack of evidence to link the relevance of prior free-standing spent fuel rack seismic analyses to the analysis in this case. Thus, the State concludes that Holtec and its witnesses have limited experience in performing nonlinear cask stability analyses at sites similar to the proposed PFS facility, and asks the Board to consider Holtec's limited experience in the context of the weight given on various issues. State Findings ¶ 267.

E.32 Contrary to the State's arguments, the Board finds that Dr. Singh and Dr. Soler have extensive professional experience in conducting cask stability analyses. Dr. Singh has a Ph.D. in Mechanical Engineering, which he received from the University of Pennsylvania in 1972. He has extensive experience in the design and licensing of nuclear spent fuel systems which extends back to 1979. Over the past 23 years, Dr. Singh has personally led the design and licensing of spent fuel storage systems for over forty nuclear plants, and for Holtec's HI-STAR 100 System and HI-STORM 100. Dr. Soler is responsible for all corporate engineering activities at Holtec, including overseeing the analyses performed to establish the stability of the HI-STORM 100 under postulated seismic events. Dr. Soler has either performed or reviewed all HI-STORM 100 seismic analyses conducted in support of deployment of the HI-STORM 100 at the PFS facility. Likewise he has either performed or reviewed dozens of seismic analyses for freestanding storage casks and storage racks over many years. Together, they have approximately 40 years of experience in dynamic analyses of spent fuel racks, storage, and transportation casks. Singh/Soler Post Tr. 5750, 1-2, 4-5.

3. Reliability of Analysis

a. Dynamo Program

E.33 According to the State, Holtec modified a published "general lumped mass analysis" code to create the predecessor to DYNAMO, the code used to generate the result in the Holtec 2000-year report. State Exh. 120 at 24-28. Thus, the State insists that DYNAMO is a "small deformation code" that is not capable of processing "large" cask rotations. State Findings ¶ 273.

E.34 The State argues that although not quantified, Dr. Soler opines that the maximum angle of rotation for which DYNAMO is capable of accurately processing results, is less than

15 degrees. State Findings ¶ 274. Dr. Soler's opinion is that as long as the deflections predicted by DYNAMO "are not too large," he is confident DYNAMO is generating accurate results. Tr. at 9930-31 (Soler). The State claims there is no evidence to support Dr. Soler's confidence in DYNAMO producing accurate results in this case. State Findings ¶ 274.

E.35 According to the State, except for the PFS site, DYNAMO has not been used to analyze the stability of a free-standing cask where the ground motions are equal to or greater than those for a 2000-year earthquake at the PFS site (0.7 g). Khan/Ostadan Post Tr. 7123, at 5.

E.36 The State argues that by using the same input parameters, DYNAMO failed to predict cask tip-over, when in a Holtec nonlinear seismic analysis of its HI-STAR 100 cask using VisualNastran, Holtec determined the HI-STAR cask would in fact tip over at a ZPA of 0.6g. State Findings ¶ 276. Although the HI-STAR cask has different features than the HI-STORM cask, both were analyzed as free-standing casks. Additionally, the State notes that the Holtec 2000-year report (State Exh. 173) references the methodology described in the HI-STAR technical paper (State Exh. 199) that discusses DYNAMO's failings as compared to VisualNastran. State Findings ¶ 276. In fact, the State contends that Dr. Soler testified the reason, in part, for the technical paper comparison was to be cognizant that "you can't just say, because the computer program says it's so, that means it's so." State Findings ¶ 276 (citing Tr. at 9775 (Soler)).

E.37 Because Holtec holds its DYNAMO code as proprietary information which, the State claims, has not been provided to the Staff or the State, the State insists that the parties have had no opportunity to test the reliability and limits of the DYNAMO code. State Findings ¶ 277. The State argues that "a trier of fact would be derelict in the discharge of its responsibilities were it to rest significant findings on expressions of expert opinion not

susceptible of being tested on examination of the witness." State Findings ¶ 277 (citing Virginia Electric and Power Co. (North Anna Nuclear Power Station, Units 1 and 2), ALAB-555, 10 NRC 23, 26 (1979)).

E.38 Additionally, the State highlights the testimony in which Dr. Soler admitted that the contact spring stiffness computations used in State Exhibit 173 are not all included in that report, but referred to earlier documents that "set forth" the theory. State Findings ¶ 278 (citing Tr. at 9780 (Soler)). These "earlier documents," the State argues, are not in evidence and therefore their reliability, and the reliability of the contact spring stiffness computations, have not been tested. State Findings ¶ 278. We find the State's arguments in this regard to be unpersuasive. As the State has done in analogous circumstances, it could have requested this information during discovery or during the hearing. Thus, we are unpersuaded by the State's argument that it was not given adequate information to challenge Holtec's results.

E.39 The State contends that the question of whether DYNAMO, as a small deformation code, generated accurate results in the Holtec 2000-year report is a "significant finding" in which the opportunity to test the witnesses on cross examination is limited, in part, by the unavailability of the DYNAMO code. In addition, the State claims there is an incomplete computation of input parameters in the record. As a result, the State requests that the Board consider the opposing parties ability to test witnesses on cross examination as a factor in weighing the evidence of the reliability of DYNAMO. State Findings ¶ 279.

E.40 To support its use, Dr. Singh testified that DYNAMO has "been used in over a thousand discrete structures, qualifying them," and is a "well tested program." Tr. at 6099-100 (Singh). The State claims that PFS offered no evidence with respect to the type of

"discrete structures" qualified by DYNAMO and how those DYNAMO analyses are relevant to this case given the unique and unprecedented design posed by PFS. State Findings ¶ 280.

E.41 According to the State, the Staff cites and accepts the results obtained with DYNAMO in the Holtec 2000-year report, but does not specifically refer to the code used to obtain the Holtec results. State Findings ¶ 281 (citing SER at 5-30). Holtec claims that the Staff also reviewed and accepted that DYNAMO performed at other spent fuel storage sites. Singh/Soler, Post Tr. 5750, at 14. The State argues that there is no evidence in the record concerning (1) the basis of the Staff's acceptance of Holtec's use of the DYNAMO code to accurately predict the dynamic behavior of unanchored casks under high seismic ground motions at the PFS site or sites with similar design characteristics; (2) whether the Staff independently validated the results obtained with DYNAMO; and (3) whether the Staff's previous acceptance of DYNAMO results have any direct bearing in this case where the Applicant has proposed to place free-standing dry storage casks on a shallowly embedded foundation supported by cement-treated soil in a seismically active location. Furthermore, the State claims that the Staff did not have access to the DYNAMO code for any purposes, including verifying the input parameters, the model, or results. State Findings ¶ 281.

E.42 During his testimony, Dr. Singh had with him DYNAMO's training manual in which he testified the manual contained over a dozen cases in which DYNAMO simulated a "wide variety of problems [such as] harmonic resonance, bifurcation, [and] dynamic responses of nonlinear structures." Tr. at 9679 (Singh). Dr. Singh testified that the DYNAMO training manual demonstrates that DYNAMO has been validated for both fuel rack and cask stability analyses. Dr. Singh also testified that DYNAMO has been validated for dynamic responses of nonlinear structures. Tr. at 9678-80 (Singh). However, the State argues that it is significant that the Applicant failed to proffer supporting documentation from the DYNAMO

training manual to document the scope and relevance of Dr. Singh's claims in this matter. State Findings ¶ 282.

E.43 Additionally, Drs. Singh and Soler testified "problems that had no simple analytical solutions were also evaluated [with DYNAMO] and shown to give good agreement with numerical solutions using finite element codes such as ANSYS." Singh/Soler Post Tr. 5750, at 20. The State argues that this statement is inconsistent with other statements made by Holtec that indicate that ANSYS was not reliable. State Findings ¶ 283 (citing Tr. at 6099-100 (Singh)). In light of Dr. Singh's testimony as to the credibility of ANSYS to accurately model the seismic behavior of freestanding structures -- the issues at the heart of this case -- the State insists that Holtec's comparison between DYNAMO and ANSYS in this instance is unreliable. State Findings ¶ 283.

E.44 Holtec testified that DYNAMO produced results in "good agreement" with known solutions for a "series of classical problems." Singh/Soler Post Tr. 5750, at 20. According to Holtec, the classical problems demonstrated DYNAMO features such as compression only behavior and friction resistance. Id. The State claims that no evidence was offered that demonstrates the relevance of the classical problems to the unique issues under consideration here. State Findings ¶ 284.

E.45 We find that the record does not support the State's criticisms. First, the results of the Holtec DBE analyses show that in the event of a DBE, the casks will undergo small, not large, rotations. As stated above, the maximum rotation of the casks from the nine configurations evaluated by Holtec for the DBE was 1.026 degrees. Dr. Soler testified that he would consider large rotations to be somewhere on the order of 20 degrees or less. See Tr. at 6101-02 (Soler). Therefore, the rotations obtained through the use of DYNAMO are well within the code's capabilities. Dr. Soler himself has extensive experience in the running

of the DYNAMO code and is well aware of its small deflection limitations, and yet he was comfortable with using DYNAMO for the DBE (Tr. at 9930-9931 (Soler)). In that regard, Dr. Soler decided to use VisualNastran for evaluating cask stability for the 10,000 year beyond DBE because of the potential for large cask rotations in that case. Singh/Soler Post Tr. 5750, at 62.

E.46 Second, contrary to the State's claim, Holtec has validated its DYNAMO results for the 2000-year DBE at the PFS site against another structural analysis code, that is, VisualNastran. As part of its beyond design basis scoping analysis Holtec ran one of the nine configurations of the original design basis analysis using VisualNastran. The VisualNastran run of the DBE predicted cask displacements on the order of several inches, similar to the DYNAMO results, thus showing that the DYNAMO runs were within the capabilities of that code. Singh/Soler Post Tr. 5750, at 65-71.

E.47 Third, as discussed above, DYNAMO has been extensively benchmarked, validated and accepted by the Staff, and it has been shown, to the Staff's satisfaction, to provide valid predictions. Singh/Soler Post Tr. 5750, at 19-20, 77.

E.48 As the State correctly points out, Dr. Soler cautioned against using DYNAMO for predictions of large displacements or cask rotations (on the order of 15 degrees or more), because the results may not be accurate because the code's capability is being exceeded. State Findings ¶ 274.

E.49 The ability of DYNAMO to accurately make predictions for the PFS facility 2000-year DBE was confirmed by Holtec's use of VisualNastran to model one of the cases for 2000-year PFS facility DBE that had initially been modeled using DYNAMO. The results obtained by Holtec using VisualNastran showed cask displacements of only a few inches and

cask rotations on the order of one degree similar to those obtained by DYNAMO. PFS Exh. 86C at 20-21.

E.50 The appropriateness of using DYNAMO for the cask stability analysis of the 2000-year DBE at the PFS facility is further supported by the results of the Dr. Luk's simulations for the PFS facility 2000-year DBE, which similarly predicted maximum cask displacements on the order of a few inches and minimal cask rotations of less than one degree. Vincent K. Luk, et. al., Seismic Analysis Report on HI-STORM 100 Casks at Private Fuel Storage Facility, Rev. 1 (Mar. 31, 2002) (Staff Exh. P) at 30 [hereinafter Luk Report]. Thus, there is no technical merit to the State's claim that DYNAMO gives unreliable predictions of the performance of the HI-STORM storage casks for the PFS facility 2000-year DBE.

b. VisualNastran Results

E.51 In their testimony, Drs. Singh and Soler described additional computer simulations which they conducted, using the VisualNastran code, to address various claims raised by the State in Part D of this contention, including: (1) the failure to consider non-vertically propagating waves; (2) the lack of sufficient time histories; (3) overestimation of soil damping; and (4) failure to consider resonance. 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.[40] 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.

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[40] Drs. Singh and Soler included some analyses using the 2000-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.

Drs. Singh and Soler testified that because these analyses used the 10,000-year return period earthquake, they bound the 2000-year design basis seismic event. Furthermore, by virtue of their increased strength, these analyses would bound any issues raised by the State concerning the appropriateness of the PFS evaluation of the response to the 2000-year DBE. Singh/Soler Post Tr. 5750, at 62-64.

E.52 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. 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 VisualNastran simulation using a 2000-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 VisualNastran 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 VisualNastran more accurately display slightly larger rotations than those predicted from DYNAMO if the rotations reach the upper end of small rotations.

(2) The VisualNastran 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.

E.53 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, and a significant contribution from out-of-phase effects is experienced. Despite the orientations observed, all of the casks concluded the simulation in a vertical orientation, although perhaps in a new location (e.g., cask 1 came to rest approximately 8 in. away from its starting point). Singh/Soler Post Tr. 5750, at 70.

E.54 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 tip-over. Accordingly, these bounding analyses confirmed the Applicant's conclusion that the HI-STORM 100 casks will perform satisfactorily in a DBE at the PFS site. Singh/Soler Post Tr. 5750, at 71.

E.55 The State claims these VisualNastran results are unreliable. See State Findings ¶ 288. According to the State, many times throughout cross examination, Dr. Soler could not identify specific details or results of his various nonlinear analyses, because he either (1) did not personally seek the requested results; (2) he observed the data visually and did not record the results; (3) he did not know the inner workings of VisualNastran; or (4) he needed additional time to locate the details. State Findings ¶ 288. For example, as documented in the transcript, the untracked casks in Dr. Soler's animations appeared to move greater

distances than the tracked cask. Tr. at 5761-62 (Soler). Dr. Soler did not have the ability to identify the actual deflection or angle of rotation of these untracked casks. Tr. at 5779 (Soler). Thus, the State argues that although VisualNastran is a publically available code, the ability of parties, and the Licensing Board to test the reliability of Dr. Soler's testimony based on the nonlinear analyses was severely restricted. State Findings ¶ 288.

E.56 Additionally, the State notes that Dr. Soler admitted that no document in evidence lists every input value for each of his simulations. State Findings ¶ 289. Furthermore, the State points out that Dr. Singh admitted that the Holtec Beyond Design Basis Report does not list "each numerical value." State Findings ¶ 289 (citing Tr. at 5796 (Singh)). Nor could Dr. Soler provide the critical damping used in his analyses of case eleven but relied upon "whatever ASCE 486 would ask you to use for the soil properties given to us, that is what we used." Tr. at 5788-89 (Soler). Furthermore, the State points out that Dr. Soler was unaware of "the equations for equilibrium of rigid bodies [which are] built into the [VisualNastran] code." Tr. at 5968 (Soler).

E.57 The State contends that the reliability of Holtec's opinions which rely on the results generated using VisualNastran is a "significant finding" in which the opportunity to test the witnesses on cross examination is limited, in part, by the parties' inability to test the VisualNastran results through cross examination. State Findings ¶ 291.

E.58 Dr. Singh testified that the Staff's grant of a CoC for Holtec's HI-STAR 100 shipping cask was supported in part by analyses generated with VisualNastran. Tr. at 6111-12 (Singh). The State claims there is no evidence in the record concerning: (1) the basis of the Staff's acceptance of Holtec's use of the VisualNastran code; (2) whether the Staff independently validated the results obtained with VisualNastran; and (3) whether the Staff's previous acceptance of VisualNastran results have any direct bearing in this case where the

Applicant has proposed to place free-standing dry storage casks on a shallowly embedded foundation supported by cement-treated soil in a seismically active location. The State also argues that there is no evidence that VisualNastran has been independently validated with test data for the sliding and uplift of free-standing casks in an area with high seismic ground motions. Based on the lack of supporting evidence, the State argues that the Staff's previous acceptance of VisualNastran with respect to the HI-STAR 100 cask is unpersuasive in this case. State Findings ¶ 292.

E.59 Although the State criticizes the VisualNastran simulations because Dr. Soler did not have the computer track and record the data on cask displacements for each cask, Dr. Soler explained that the evaluations were a "scoping analysis." Furthermore, as the full title of the Report reflects, the primary purpose was to see "whether at the end of the earthquake do we have eight casks still standing." Tr. at 5771 (Soler).

E.60 The focus of the analysis was not on the specific measurements of the displacement of the casks. See Tr. at 5771 (Soler). Accordingly, as Dr. Soler explained, he had initially set the computer to record the measured displacements of cask 1 for all eleven cases, which remained stored in the computer and could be retrieved from the computer. Tr. at 5761-62 (Soler). At the request of the Board, Dr. Soler produced a table showing the cask displacements and angle of rotation for cask 1 in all eleven runs of the Holtec Beyond Design Basis Report. Tr. at 5773-76 (Soler). As Dr. Soler explained, obtaining displacement for all casks would take a significant amount of time, and the Board decided to have Dr. Soler produce the information for cask 1 already stored in the computer, and have more steps taken later to obtain measured displacement data on additional casks if the parties or Board deemed that necessary. Tr. at 5773-76. The State did not pursue this matter and we now find the additional data to be unnecessary.

E.61 Furthermore, we find that the lack of detailed recorded information on the other casks did not restrict the State's ability to cross-examine Holtec on its results. The VisualNastran computer runs were beyond-design basis scoping analyses whose primary function was to determine whether the casks would tip over under the 10,000-year earthquake event given various bounding and worst-case assumptions. Tr. at 5771 (Soler); Singh/Soler Post Tr. 5750, at 62-71. Another purpose was to establish whether the 2000-year runs using VisualNastran would provide responses similar to those obtained with DYNAMO for the 2000-year DBE, i.e., inches of displacement, not feet of displacement. PFS Scoping Analyses at 20-21. Both of these points are evident from the simulations. Thus, the points for which the computer cases were offered did not require detailed results for each cask.

E.62 Regarding the State's claim that Dr. Soler could not provide the percentage of critical damping used in the formulas, we note that Dr. Soler explained that the formulas for damping provided by ASCE Standard 4-86 (which as discussed earlier are based on well-recognized sources) are not developed based on a percent of critical damping. See Tr. at 5788-89 (Soler). Instead, Dr. Soler provided the actual damping input values used in the run in questions and similarly provided the actual damping input values used for the other runs. See PFS Exh. 86D.

E.63 We also find the State's concern that Dr. Soler was "unaware" of the equations for equilibrium for rigid bodies built into the VisualNastran code to be unnecessary. In response to the State's question on how a particular figure showing the casks in motion "was modeled mathematically," Dr. Soler replied "the equations for equilibrium of rigid bodies [are] built into the code. I do not externally model anything mathematically." Tr. at 5967-68 (Soler). Contrary to the State's concern, we find that this statement does not discredit Dr. Soler's testimony. Furthermore, the record reflects that the equations are well known by him.

E.64 For the same reasons, the State's objections to the visual simulations that were raised during the hearing are found to be without merit. During the hearing, the State claimed that without supplemental data the animations merely represent one analyst's simulation of cask behavior. Tr. at 5853-54, 10,532-54. We, do not agree, however, with this characterization. The visual simulations are computer-generated visual representations of the results of the computer analysis. As explained by Dr. Soler, "these videos were not created outside the program, they are part of the program, and they use the results as they are calculated." Tr. at 5756-57 (Soler). Therefore, they are only a visual portrayal of the numerical computations made by the VisualNastran code for the input parameters used, and not some abstract "simulation" produced by Dr. Soler. As stated, the points for which PFS relies on the simulations are (1) to demonstrate that VisualNastran and DYNAMO provide comparable results for the 2000-year DBE; and (2) to demonstrate that the casks do not tip over. Establishing those points, or questioning Holtec on them, does not require the quantitative data on cask displacements.

c. Input Parameters

(i) Contact Stiffness

E.65 Vertical contact stiffness represents the amount of force applied at the interface points of contact between two bodies that would be required to have one of the bodies approach or penetrate the other a unit-distance. The parameter is measured in the pounds of force required to cause one body to approach the second body by one inch. For example, for a pad of undefined material on which a HI-STORM 100 cask weighing 360,000 pounds is placed, which causes a deformation or deflection of the pad of 0.01 in., the contact stiffness would be 360,000 pounds/0.01 in. or 36 x 10^6 pounds per in. Singh/Soler Post Tr. 5750, at 78.

E.66 For its cask stability analysis for the 2000-year DBE, Holtec used a vertical contact stiffness of 454 x 10^6 pounds per in. Singh/Soler Post Tr. 5750, at 79.

E.67 As a result of his study, Dr. Khan argues that nonlinear mathematical models are highly sensitive to the assumed contact stiffness between the cask and the storage pad. See Khan/Ostadan Post Tr. 7123, at 6. Dr. Khan explains that local contact stiffness is needed in a mathematical simulation before any sliding occurs. Id. at 9. After sliding occurs, the horizontal displacement is a function of the inertial forces overcoming the coefficient of friction times the mass. Id. Thus, according to Dr. Khan, displacement of the cask from seismic ground motion should not be very sensitive to the contact stiffness values. Id.

E.68 Additionally, Dr. Khan maintains that in nonlinear analytical solutions, high-contact stiffness values also absorb significant amounts of energy before sliding actually occurs by reducing instantaneous velocities for the next successive iteration in the nonlinear analysis. As a result, Dr. Khan believes high contact stiffness could underestimate vertical displacement of the cask. Khan/Ostadan Post Tr. 7123, at 9.

E.69 The State argues that in its model, Holtec used contact stiffness to "define the stiffness of the vertical-only 'compression springs' at the interface of the cask and the pad." State Findings ¶ 300 (citing Singh/Soler Post Tr. 5750, at 79). Holtec used a single vertical contact stiffness value for its simulations in the Holtec 2000-year report. Id.; Tr. at 6042-44 (Soler). The State claims that notwithstanding its challenge to Holtec's contact stiffness value, Dr. Soler opined that "we got acceptable answers in the 2000-year return earthquake, so there was no incentive for us there to lower the contact stiffness." State Findings ¶ 300 (citing Tr. at 6043) (Soler)). In its simulations of the 10,000-year earthquake, Holtec used a vertical contact stiffness of 18,864,480 pounds per in. Tr. at 9575 (Soler).

E.70 Prior to his parametric study, Dr. Khan had not had occasion to select a contact stiffness value for sliding or tipping. Tr. at 7217 (Khan). Similarly, the State claims that neither Dr. Soler nor Dr. Singh have proffered evidence that they have prior experience selecting a contact stiffness value for a sliding or tipping analysis of a free-standing cask where the ground motions equal to or exceed those for a 2000-year earthquake at PFS. State Findings ¶ 301.

E.71 A vertical contact stiffness of 450 x 10^{-6} pounds per in. for unanchored casks is too high, opines Dr. Khan, because the contact stiffness makes the vertical frequency of the cask too rigid which underestimates the vertical displacement of the cask. Khan/Ostadan Post Tr. 7123, at 11. Dr. Khan testified that "[o]nce [cask] sliding begins, the high [contact] stiffness values artificially treat the solution as linear [e.g., as if the cask is anchored to the pad] without amplifying it in the upward direction and give non-unique or invalid results." Id. at 11. According to Dr. Khan, a high contact stiffness corresponding to a high response spectra frequency will never amplify the cask motion. See Tr. at 7231 (Khan). Holtec notes its contact stiffness corresponds to a frequency in the rigid range of 111 hertz. Tr. at 9634-35 (Soler).

E.72 In the absence of test data, it is Dr. Khan's opinion that to conservatively capture the dynamic behavior of the cask, including cask rotation or rocking, the appropriate contact stiffness for unanchored casks must correlate with a frequency that falls within the amplified range of the response spectra curve. Khan/Ostadan Post Tr. 7123, at 12; Tr. at 9362, 9373-74, 9482-83 (Khan). He believes that the rotational stiffness or rotational springs in the model will move the cask with a certain damping at an associated frequency. Tr. at 9482 (Khan). According to Dr. Khan, if the contact stiffness does not correlate with the frequency

in the amplified region of the response spectra, then the mathematical code will treat the problem as linear as if the cask is anchored to the pad. Khan/Ostadan Post Tr. 7123, at 12.

E.73 Paramount to Dr. Khan's opinion is that the Applicant has offered no test data to support its nonlinear cask stability results. If the real dynamic behavior of the structure is unknown, Dr. Khan is adamant that structural analysis design philosophy mandates that the structure's behavior is analyzed using the "peak of the spectra times the weight and other factors into consideration." Tr. at 7236 (Khan). Thus, for design purposes in the absence of test data, to estimate the dynamic response of the cask, Dr. Khan believes a range of contact stiffness is selected that correlates with the rocking frequencies in the earthquake response spectra that give the maximum dynamic response. Id. at 7208, 7215.

E.74 Dr. Khan opined that contact stiffnesses in the range of 1 x 10^6 pounds per in. and 10 x 10^6 pounds per in. correspond to frequencies in the amplified spectral range of the response spectra. Khan/Ostadan Post Tr. 7123, at 13.

E.75 As explained above, Dr. Khan testified that he would choose the contact stiffness so that the natural vertical frequency of the cask on the pad was in resonance with the amplified spectral range of the earthquake. Tr. at 7215-16 (Khan). Several consequences, however, flow from following the approach suggested by Dr. Khan. First, such an approach artificially maximizes the vertical response of the cask by assuming that the natural frequency of the cask and the earthquake are in resonance. Second, because the amplified spectral range of an earthquake will vary depending on the geology and soils of its location, setting the contact stiffness to artificially cause the cask and the earthquake to be in resonance means that the choice of contact stiffness will vary depending on the geographic location of an ISFSI and the assumed earthquake excitation.

E.76 Contrary to Dr. Khan's position, however, Drs. Singh and Soler testified that contact stiffness is a physical property of the cask-pad interface that can be determined from the physical characteristics of the cask and the pad, and as such it would not change depending on geographic location or earthquake excitation. Tr. at 9618-19 (Soler).

E.77 Drs. Singh and Soler testified that often, in computer modeling, one will chose a value of contact stiffness that is lower than the actual physical contact stiffness to avoid excessive computing time, but one should always avoid using such a low value that the corresponding cask frequencies fall into the amplified spectral range of the earthquake spectra. See Tr. at 9641-45 (Soler). If that was done (as proposed by Dr. Khan), the results of the analysis would be contaminated by introducing an artificial excitation of the cask which does not exist in fact, since the actual physical contact stiffness of the cask-pad interface does not produce cask frequencies in the amplified spectral range of the earthquake. Therefore, choosing a contact stiffness that would cause resonance of the cask with the earthquake should be avoided because it would produce unrealistic results that would not be expected to occur under earthquake conditions. Tr. at 9633-45 (Singh/Soler).

E.78 Drs. Singh and Soler also state that a correct computer model should be able to predict accurately both dynamic and static conditions (Singh/Soler Post Tr. 5750, at 88) and that choosing a contact stiffness of 1×10^6 pounds per in., as suggested by Dr. Khan, would result in a deformation of 3/8 of an inch of the reinforced concrete pad under static conditions, which is an obviously incorrect result that defies reality. Singh/Soler Post Tr. 5750, at 80-81.

E.79 Dr. Khan does not dispute that a deflection of 3/8 of an inch resulting from a HI-STORM 100 cask that just sits on the surface of a concrete pad is contrary to physical

fact (Tr. at 7218-19 (Khan)), but takes exception to the concept that a model should be able to accurately predict both static and dynamic conditions. Id. at 7211-15 (Khan).

E.80 We are presented here with a situation in which experts from opposing parties provide conflicting technical testimony. As we explain below, we agree with the interpretation of Drs. Singh and Soler on the proper application of vertical contact stiffness and decline to follow Dr. Khan's suggested approach.

E.81 Contact stiffness is a physical parameter of contacting objects and their intrinsic material properties. Tr. at 9618-22 (Singh/Soler). The contact stiffness at the interface of two objects can therefore be derived from nature's physical laws, as shown by Heinrich Hertz in 1881. Tr. at 9618-19 (Singh). Holtec computed the vertical contact stiffness of 454×10^6 pounds per in. for its DYNAMO design basis analysis using a well-established methodology developed by Timenshenko and Goodier for calculating the contact stiffness between two objects. Tr. at 9622-24 (Singh); Multi-Cask Seismic Response at the PSF ISFSI (PFS Exh. 226) [hereinafter Multi-Cask Seismic Response]. The approach used by Holtec is in accordance with guidance from the ANSYS Training Manual, which states that the "[h]ertz contact stiffness often provides an appropriate basis" for determining the contact stiffness to be used for modeling bulky objects. ANSYS Training Manual at 3-6; Tr. at 9625-26 (Singh) (PFS Exh. 221). The Hertzian theory of contact is the standard state of the art, technique used to simulate the interface between two bodies. Tr. at 9628-29 (Singh/Soler).

E.82 Similarly, in Sandia's modeling of cask stability for the NRC, contact stiffness was not treated as a "physical behavior." It was determined in accordance with the intrinsic properties of the contacting materials and the applicable physical relationships for determining contact stiffness. Tr. at 6809-11 (Luk).

E.83 Because contact stiffness is an intrinsic property of the contacting bodies, it does not vary from one geographic location to another as the earthquake characteristics change, as it would under the approach espoused by Dr. Khan. Tr. at 9617-19 (Soler).

E.84 Drs. Singh and Soler refer to guidance provided in ANSYS manuals on choosing appropriate contact stiffness for computer modeling to support their position concerning the proper choice of contact stiffness here. Singh/Soler Post Tr. 5750, at 79-82. ANSYS is a recognized, general purpose computer modeling program accepted by Dr. Khan as an authoritative source. Khan/Ostadan Post Tr. 7123, at 8. The ANSYS Training Manual refers to the Hertz contact stiffness theory applied by Holtec as "often provid[ing] an appropriate basis" for choosing a contact stiffness. ANSYS Training Manual 3-6 to 3-7 (PFS Exh. 221). The Training Manual also contains more than 100 pages devoted almost entirely to friction and contact problems. In addition, the ANSYS Verification Manual contains sample problems covering friction and contact issues and related guidance. See ANSYS Training Manual (PFS Exh. SS); Singh/Soler Post Tr. 5750, at 79-80.

E.85 The guidance provided by the ANSYS Training Manual included in PFS Exhibit SS makes it clear that, in order to achieve realistic modeling, the choice of stiffness for the contact springs between two contacting surfaces should not produce analysis results predicting a measurable penetration or deflection of one of the bodies in contact, because such penetration or deflection is contrary to physical fact. In this respect, the guidance states that a contact stiffness that results in minimum penetration or deflection provides the "best accuracy," and "[t]herefore, the contact stiffness should be very great." ANSYS Training Manual at 3-3 (PFS Exh. SS). The guidance goes on to note, however, that in order to avoid convergence difficulties that may arise from "too stiff a value," determining "a good stiffness value usually requires some experimentation" but that "if you can visually detect

penetration in a true-scale displaced plot of the entire model, the penetration is probably excessive." In that case, one should "[i]ncrease the stiffness and restart." ANSYS Training Manual at 3-3, 3-14 (PFS Exh. SS).

E.86 As explained by the Applicant's experts, the selection of contact stiffness is a well-defined and understood problem when dealing with known properties of materials. Tr. at 9628-29, 9639-40 (Singh/Soler). Guidance exists for selecting contact stiffness values in general purpose, validated, and well-established computer modeling programs, such as ANSYS, using tested mathematical solutions. Rather than being an unknown quantity to which dynamic analyses are extremely sensitive, the appropriate setting of a contact stiffness value is relatively straight-forward for an experienced modeler.

E.87 A decisive factor in assessing Dr. Khan's suggested approach is that his choice of contact stiffness produces results that are contrary to physical reality. Using a contact stiffness of 1×10^6 pounds per in., as suggested by Dr. Khan, results in a deflection of 3/8 of an inch in the pad simply from having the cask rest on its surface. Singh/Soler Post Tr. 5750, at 80-81. This is totally unrealistic, since the pressure applied by a fully loaded cask on the reinforced concrete pad is 26 psi, equivalent to a man standing on one foot. Singh/Soler Post Tr. 5750, at 50. Dr. Khan does not dispute that 3/8 of an inch deflection is totally unrealistic. Tr. at 7218-19 (Khan). Drs. Soler and Singh maintain that a model should be able to provide a physically correct answer for all conditions, including the initial static case, and dynamic loading. Singh/Soler Post Tr. 5750, at 88. Dr. Luk agrees. Tr. at 6816-17 (Luk). We also agree.

E.88 Further, Dr. Soler testified that there are simple mathematical relationships between the natural frequency of the cask under dynamic conditions, the static deflection of the pad caused by the cask resting on its surface, and its contact stiffness. Tr. at 9632-34 (Soler).

Those relationships involve the same formula that Dr. Khan cites as the basis for choosing a contact stiffness. According to those relationships, the frequency of the cask vibrating or oscillating on the pad is a function of the static deflection of the pad caused by the cask resting on its surface, or in other words, the static contact stiffness. See Khan/Ostadan Post Tr. 7123, at 12-13; Tr. at 9382-89 (Khan); Additional Cask Analyses at 21; Tr. at 9632-34 (Soler).

E.89 Dr. Khan's assertion that contact stiffness should vary according to geographic location and as an earthquake's characteristics change is without any support and belies the guidance provided by modeling programs, such as ANSYS, that make no mention of this supposedly essential fact. Dr. Khan's concerns about the use of an appropriate contact stiffness are not credible when looked at in terms of the results of Dr. Khan's choice of a vertical contact stiffness of 1×10^6 and a horizontal contact stiffness of 100,000 pounds per in. Not only would the use of these values result in a storage cask literally sinking nearly half an inch into a reinforced concrete storage pad, they would result in a model that predicts displacement of 3/4 of an inch, before actual cask sliding occurred. Neither of these predictions about how the cask and storage pad would behave comports with physical reality.

E.90 Furthermore, in response to Dr. Khan's claims, Holtec performed additional VisualNastran computer simulations using a lower contact stiffnesses than in its analyses using DYNAMO. Holtec ran VisualNastran using a vertical contact stiffness in the middle of the range of values that Dr. Khan claimed should have been used. Even though this brought the model within the spectral range of the earthquake input spectra, and thereby contaminated the results, the program still showed displacements on the order of inches and not feet as claimed by Dr. Khan. Additional Cask Analyses at 29-30; Tr. at 9671-76 (Soler).

Holtec also ran Dr. Khan's model using the unreasonably low values for vertical contact stiffness (1,000,000 pounds/in.) and impact damping (1%) that Dr. Khan had used in his analysis. Even at these values the casks did not tip over or impact each other. Additional Cask Analyses at 24-26; Tr. at 9606-07, 9611-14 (Soler).

E.91 The State attacks Dr. Soler for saying that "we got acceptable answers in the 2000-year return earthquake, so there was no incentive for us there to lower the contact stiffness." State Findings ¶ 300 (citing Tr. at 6043(Soler)). However, the context of Dr. Soler's statement was that, while the "actual contact stiffness is indeed very high," the use of a high contact stiffnesses can lead to excessive computation time. Tr. at 6041 (Soler). Accordingly, as explained by Drs. Singh and Soler, analysts will often use a contact stiffness value which is less than the actual value of the stiffness in order to reduce computing time, but yet not so low as to corrupt the solution. Tr. at 6041-44 (Singh/Soler). This explanation parallels guidance found in the ANSYS training manual on the use of contact stiffness. ANSYS Training Manual at 3-6 to 3-7 (PFS Exh. 221); Tr. at 9641-45 (Soler).

E.92 Thus, for the 2000-year DBE there was "no incentive" or reason for Holtec to use a lower contact stiffness value because Holtec was able to arrive at a converging solution using a high value for the contact stiffness close to its actual value. Tr. at 6042-43 (Soler). With VisualNastran runs, because of the vast amount of data that was being collected, Holtec used a lower contact stiffness in order to decrease the computation time. However, it did test runs to ensure that the use of a lower contact stiffness would not "significantly alter" the results. Tr. at 6043 (Soler). In this respect, Dr. Soler explained that there is a relatively wide range of contact stiffnesses over which the solution does not show a great variation in results. Tr. at 6039-41 (Singh). The objective is to select and use a contact stiffness value

for the analysis that is within this range. Singh/Soler Post Tr. 5750, at 81-82. Thus, the Board finds the Applicant's choice of contact stiffness to be reasonable.

(ii) Damping Values

E.93 In Holtec's cask stability analysis for the 2000-year earthquake using DYNAMO, Holtec used a 5% value for impact damping at the cask-pad interface to represent the dissipation of energy that occurs when the cask and the concrete pad impact each other during an earthquake event. Singh/Soler Post Tr. 5750, at 90-91. In the subsequent analyses using VisualNastran, Holtec used higher impact damping values based on analysis and test data that showed that the dissipation of energy through impact damping between a steel and concrete surface is much greater and would justify impact damping values of 40% or more. Tr. at 6095-98 (Soler), 6098-99 (Singh). Holtec also explained that the "extent of damping is directly related to the severity of the event." Tr. at 9671 (Singh).

E.94 To illustrate the reasonableness of the impact damping values used in Holtec's cask stability analysis, and to further support their analysis in light of the State's criticism, Drs. Singh and Soler also provided computer simulations showing the effect of impact damping on a ball or cask dropped from a height of 18 in. using impact damping values of 1%, 5% and 40%. At 1% damping, which is the value that Dr. Khan would have Holtec use in accordance with structural damping guidelines, the ball or cask would require more than seventy-three bounces before it came to rest; at 5% the ball or cask would come to rest after approximately fourteen bounces and at 40%, the ball or cask would come to rest after two or three bounces. Tr. at 9664-68 (Soler); PFS Exh. 225A.

E.95 In another animation, Dr. Soler replaced the sphere image with a cylinder representing a cask where he dropped the cylinders from a height of 18 in. and each cylinder

had either 1%, 5%, or 40% damping. In this case, the cylinders with 40% damping and the 1% damping bounced three times and more than seventy-three times, respectively, before stopping. Id. at 9665-68 (Soler).

E.96 Holtec also generated a number of additional animations, including an eight-cask animation with a 40-million pounds per in. contact stiffness, 5% damping, and lower bound soil springs, for a 2000-year earthquake. Tr. at 9673 (Soler). Another eight-cask animation used a 40-million pounds per in. contact stiffness, 40% critical damping, lower bound soil springs, for a 2000-year earthquake. Id.

E.97 Dr. Singh also referred to publicly available test data from NRC-sponsored impact experiments that Holtec used to correlate its program and benchmark its calculations. Tr. at 9660-61 (Singh). The State would have this testimony ignored, because neither the Applicant nor the Staff offered any supporting documentation concerning the impact tests, which Holtec program was correlated with NRC data, or how the NRC impact tests relate to damping of HI-STORM casks during a seismic event. State Findings ¶ 342. We note, however, that the State had ample opportunity to pursue this issue on cross-examination, to obtain the publicly available test data, or to have its experts review the data and comment on them.

E.98 Regarding Holtec's animations, Dr. Khan testified that he disagreed that a dropped sphere would be similar to the impact damping between the cask and a pad because the earthquake motion is moving the cask up and down. Tr. at 9400-01 (Khan). Furthermore, the State notes that Dr. Soler also testified that he expected a cask would not simply bounce vertically up and down, but uplift and rock from side to side, depending upon the earthquake. State Findings ¶ 351 (citing Tr. at 9932 (Soler)). Dr. Soler also agreed that during an earthquake, the frequency and peak intensity would change with time. Id. Highlighting these

conclusions, the State argues that the seismic behavior of a cask would not simply bounce in a pure vertical direction, but could also rock from side to side. Therefore, according to the State, the bouncing sphere animation and bouncing cask animation are inconclusive to define the damping experienced by a HI-STORM cask during a seismic event. State Findings ¶ 351.

E.99 A careful review of Dr. Khan's testimony regarding this issue, however, reveals that Dr. Khan did not disagree that there would be impact damping for a sphere dropping on a hard surface; rather he believed that in addition to impact damping other damping mechanisms would also operate under earthquake conditions (such as structural damping and rattling of the casks internals). Tr. at 9400-01 (Khan). When asked to consider only impact damping, Dr. Khan did not disagree that the impact damping of a dropping sphere would be analogous to the impact damping of a cask hitting the pad under earthquake conditions. Tr. at 9402-03 (Khan).

E.100 Moreover, we note that the dropping sphere analogy is simply intended to demonstrate the effect of the choice of impact damping on the behavior of the dropped object (sphere or cask). There is no physical distinction between an earthquake rocking a cask "up and down" and a ball bouncing up and down due to the effect of gravity in terms of loss of energy from the cask or the ball impacting the surface. Tr. at 9910 (Soler).

E.101 Furthermore, we do not agree with the State's concern that no evidence was proffered that the ball or cask with 40% impact damping would "better simulate" cask impact damping under seismic ground motion than the other balls or casks with 5% and 10% damping. During the hearing, the Applicant offered the testimony of Dr. Soler, a witness with extensive background and experience, as to which of the animations' damping ratios best represents the impact damping of the steel and concrete storage casks on the concrete pad.

Dr. Soler testified that, "[o]n the basis of [his] experience, [he] would expect [a cask] to bounce . . . maybe two or three, maybe four times," and thus "in [his] view, a choice of a number around 40 percent of critical damping is correct." Tr. at 9911 (Soler). We find Dr. Soler's testimony to be adequate support for the Applicant's conclusion.

E.102 The State also argues that the Applicant has not produced evidence to support a finding that impact damping increases with an increase in ground motion. State Findings ¶ 339.

E.103 We disagree. As Dr. Singh explained in his testimony, this relationship between the percent of critical damping and the severity of the event is recognized in NRC guidance for structural damping. Tr. at 9670-71 (Singh). In this respect, we note that Regulatory Guide 1.61 concerning structural damping does allow a greater percent critical damping for SSEs than for operating basis earthquakes because energy dissipation during an earthquake depends upon "a number of factors" including the design, the material used, and the "magnitude of the deformations experienced." Tr. at 9670-71 (Singh). And while Regulatory Guide 1.61 refers to structural damping instead of impact damping, the State has provided no evidence to suggest that Dr. Singh is incorrect for relying on it to support his analysis.

E.104 Dr. Khan raised a concern that the dynamic response may be underestimated in a nonlinear horizontal sliding analysis where (1) it is assumed that energy is both dissipated and (2) is absorbed through use of a high damping value. Tr. at 9392-93 (Khan). Dr. Ostadan concurred that the damping has been overestimated which resulted in reducing seismic loads in the dynamic analyses. Tr. at 10,389 (Ostadan).

E.105 We do not agree. In its model, Holtec does not include any dampers that would reduce the effectiveness of the horizontal friction springs. Tr. at 10,639-40 (Soler). Thus, as

acknowledged by Dr. Khan, friction remains the energy dissipation mechanism in a situation involving sliding, and we, therefore, find Dr. Khan's criticism to be unfounded. Tr. at 9399-400 (Khan).

E.106 The State takes issue with Dr. Singh's testimony that the actual magnitude of the impact damping would be greater than 40% based on Holtec having calculated greater than 50% impact damping in a simulation of a cask dropped on a very thick concrete foundation. Tr. at 6098 (Singh). The State challenges this testimony, because PFS neither "proffer[ed] supporting calculations for the impact damping" of greater than 50% for a metal cask on a concrete foundation, nor "explain[ed] the details" of the assumptions and relevance "to the impact damping for the HI-STORM 100" referred to by Dr. Singh as support for use of a 40% impact damping value. State Findings ¶ 341. The State was free to cross-examine Dr. Singh on the relevance and assumptions of the calculation to which he had referred, but chose not to do so.

E.107 Finally, the State highlights Dr. Soler's testimony, which explains that the damping value changes, because critical damping is a function of stiffness. State Findings ¶ 350 (citing Tr. at 9673-74 (Soler)). The State argues, however, that Holtec did not proffer an animation where it simultaneously lowered both damping and stiffness; therefore, Dr. Soler's conclusion with respect to "both" has no basis. The State believes that because the associated damping is a function of stiffness and that the cask behavior during a seismic event is nonlinear, the additional Holtec animations varying either damping or contact stiffness are insufficient to show the cask behavior is not sensitive to both a lower damping and a lower contact stiffness, than the values used in the Holtec simulations (e.g. 40% damping and 18.8 x 10^6 pounds per in. contact stiffness). State Findings ¶ 350.

E.108 The State's concerns are unwarranted for several reasons. In the first place, as discussed above, the input parameter values for contact stiffness and damping used by Holtec are reasonable and appropriate. Second, the State cites no evidentiary support for its claim.

E.109 Moreover, the Holtec sensitivity study used, as its base parameters, reasonable values of contact stiffness and damping, i.e., 40-million pounds per in. for contact stiffness and 40% for damping. Additional Cask Analyses at 29; Tr. at 6046 (Singh); Tr. at 9911 (Soler). The sensitivity analyses performed by Holtec show that a reduction of contact stiffness by a factor of 8 or of damping by a factor of 8 has little impact on the results. Additional Cask Analyses at 29; Tr. at 9676 (Soler). While there are slight increases in the cask displacements, these are minimal, and still in the order of inches and not feet. Tr. at 9676 (Soler). Thus, no adverse results are detectable when one departs from the reasonable values used by Holtec in its cask stability analyses.

E.110 The State's criticism is also unsupported since there are analyses on the record in which Holtec reduced both contact stiffness and damping at the same time from the base case of 40 million pounds per in. for contact stiffness and 40% for damping. Case 1 of the Beyond Design Basis Report, based on the 2000-year earthquake and lower bound soil springs, was run using a contact stiffness of 18.8 million pounds per in. for contact stiffness and 27.5% damping -- approximately a 50% reduction for contact stiffness and a approximately a 30% reduction of damping from the base case. Again, the results show displacements of inches, not feet. PFS Exh. 86D at 13; PFS Exh. OO.

d. Angle of Rotation

E.111 (Utah 354) Without consideration of the effects from SSI, the State argues that Holtec predicted a maximum angle of rotation for a single cask of approximately 10 degrees given a coefficient of friction of 0.8 and a 10,000-year earthquake. Tr. at 6031 (Soler). The HI-STORM 100 cask will "theoretically" tip over if the cask tipped at an angle of approximately 29 degrees from vertical. Tr. at 6033-34 (Soler). However, Dr. Singh agreed that the cask could tip over if there is residual momentum when the cask reaches approximately 29 degrees (point where the center of gravity is over the corner of the cask). Tr. at 6109-10 (Soler).

E.112 The State notes that, in a paper presentation, Drs. Soler and Singh state that "[a]fter a certain threshold value, the response (viz maximum tilting of the cask axis) increases rapidly with increase in the [zero period acceleration] level."[41] Seismic Response Characteristics of HI-STAR 100 Cask System on Storage Pads (Jan. 1998) at 15-16 (State's Exh. 174) [hereinafter HI-STAR 100 Seismic Response Characteristics]. The State also points out that during cross examination in this case, Dr. Soler disagreed with the quote from his publication in that he did not agree that after a certain point, the maximum tilting would "rapidly" increase as the ZPA increased. State Findings ¶ 355 (citing Tr. at 6032 (Soler)). Additionally in the HI-STAR publication, Drs. Soler and Singh recommend that the maximum rotation of the cask be set to 25% of the ultimate cask tip-over value. HI-STAR 100 Seismic Response Characteristics at 15. However, in this case, the State contends that the Applicant did not offer evidence concerning the "ultimate" cask tip-over value. The State notes that 25% of the "theoretical" tip-over value of 29 degrees is 7.25 degrees. It also notes that Dr.

[41]The paper was authored jointly by Dr. Soler, Dr. Singh, and Martin G. Smith.

Singh testified that the HI-STAR cask is more likely to tip over than the HI-STORM cask because of the HI-STAR's lower height-to-diameter ratio. State Findings ¶ 355.

E.113 According to the State, for the HI-STORM 100 cask, to ensure an adequate safety factor to prevent cask tip-over, Dr. Soler opined that the maximum excursion of the top of the cask should not exceed half the radius (33.16 in.). State Findings ¶ 356 (citing Tr. at 6034-35 (Soler)). When considering the maximum excursion of the top of the cask of 33.16 in. with a cask height of 231.25 in., the State argues that to ensure an adequate margin of safety a maximum allowable rotation angle is 8.15 degrees from vertical. State Findings ¶ 356.

E.114 In two reports, Holtec estimated maximum cask rotation angles of approximately 10 degrees for a 10,000-year earthquake assuming a coefficient of friction of 0.8. This estimated maximum cask rotation angle exceeds the maximum rotation angle of 8.15 degrees suggested by Dr. Soler. Tr. at 6031 (Soler); PFS Exh. 86d at 13. In Applicant's Exhibit 86d, the rotation angle from vertical was calculated based on 50% of peak-to-peak excursion instead of the maximum excursion of the top of the cask from the location at the start of the run. Applicant Exh. 86d at 13. The State notes that 50% of the maximum peak-to-peak excursion is lower than the maximum excursion recorded at the top of the cask. Thus, the State argues that the rotation angle calculated in Applicant's Exhibit 86d may not reflect the maximum angle of rotation that occurred during the simulations. State Findings ¶ 357.

E.115 The State notes that Dr. Soler also admitted, based solely on the animation, that in some runs in which he quantified only cask 1's movement, casks other than cask 1 (e.g., cask 5), appear to move more than cask 1. State Findings ¶ 358 (citing Tr. at 5761-62 (Soler)). In an additional eight-cask simulation for a 2000-year earthquake, Dr. Soler actually

quantified the movement for cask 1 and cask 5. In this simulation, he showed that the maximum excursion of the top of the cask for cask 1 was 3.4 in. and for cask 5 was 10.5 in. See Additional Cask Analyses.

E.116 Dr. Singh claims that the "actual" maximum angle of rotation for the cask would be "much less" due to the "huge conservatisms" built into Holtec's model. Tr. at 6035 (Singh). The State points out that, however, that notwithstanding any actual conservatisms, the record is devoid of any evidence quantifying the "huge conservatisms" claimed by Dr. Singh. State Findings ¶ 359.

E.117 We need not decide whether the Applicant must accept 8.15 degrees from vertical as the maximum acceptable angle of rotation as a design basis standard applicable to the 2000-year DBE, because the PFS facility meets that standard. The maximum angle of rotation from the DYNAMO design basis cask stability analysis for the 2000-year DBE is 1.026 degrees. The maximum angle of rotation that Holtec computed for its various 2000-year DBEs using VisualNastran was approximately 2.23 degrees. Likewise, the maximum angle of rotation computed by Dr. Luk for the 2000-year DBE event was 0.40 degrees. Luk Report at 30.

e. Time Histories

E.118 The State alleges as a deficiency the fact that the PFS cask stability calculations use only one set of seismic time histories. The State claims that non-linear analyses are sensitive to the phasing of input motion, more than one set of time histories should be used, and that "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. The concerns are expressed in the direct testimony of State witness Dr. Ostadan. Bartlett/Ostadan Post Tr. 7268, at 22-23.

E.119 Time histories represent the variation of ground acceleration with time during an earthquake. They are used to represent the motions to which the site structures would be subject during the design earthquake. Youngs/Tseng Post Tr. 5529, at 8.

E.120 NRC regulatory guidance (Section 3.7.1 of NUREG-0800 and Section 5 of NUREG-1567) allows the designer a choice between two alternative methods for developing design time histories. One approach is to use multiple sets of time histories that in the aggregate envelop the design response spectra, although any individual time history may fall well below the design spectrum at some frequencies. The second approach is to develop a single set of time histories that envelops the design response spectra and a target power spectral density function. Time histories developed using the second approach are often called spectrum-compatible time histories. Youngs/Tseng Post Tr. 5529, at 8-9.

E.121 PFS elected to use the second approach, that is to utilize a single set of time histories. Its consultant, Geomatrix, developed a set of time histories (consisting of three independent time histories, representing two horizontal and one vertical components of ground motion), in accordance with the methodology specified in the NRC regulatory guidance documents. Youngs/Tseng Post Tr. 5529, at 8. The three components of motion were then modified until their resulting response spectra enveloped the design response spectra following the criteria specified in NUREG-0800. Youngs/Tseng Post Tr. 5529, at 9.

E.122 The methodology used by PFS for developing the time histories for the stability analyses of the casks is appropriate and consistent with NRC Staff guidance. Pomerening/Ofoegbu Post Tr. 6496, at 26-27. The response spectrum envelops the design response spectrum and encompasses the power spectral density of the design spectrum over the requisite frequency range. Tr. at 6507-08 (Pomerening).

f. Cold Bonding

E.123 The State has raised the concern that PFS "has failed to consider the potential for cold bonding between the cask and the pad and its effects on sliding in its calculations." Unified Utah L/QQ, at 4. As State witness Dr. Ostadan explained in his direct testimony, cold bonding occurs when two bodies (cask and pad) with such a large load (the cask) are in contact. Some local deformation and redistribution of stresses may occur over many years at the points of contact, which can create a bond in the form of a welding, which increases the resistance to sliding of the cask on the pad. Bartlett/Ostadan Post Tr. 7268, at 23.

E.124 The Applicant testified that the average pressure at the interface between the pad and the cask is approximately 26 psi. Singh/Soler Post Tr. 5750 at 50. Even assuming that the entire weight of the cask was supported only over a 12 in. wide annulus around the periphery, the static contact pressure would rise only to 40 psi. Id. This pressure is well below the allowable bearing stress of 1785 psi in concrete with a compressive strength of 3000 psi. Pomerening/Ofoegbu Post Tr. 6496, at 25. Indeed, this level of pressure is comparable to a 200-pound man standing on the ball of one foot. Singh/Soler Post Tr. 5750 at 50. Such pressure is clearly insufficient to create a bonding between the steel bottom of the cask and the concrete surface of the pad. Id. at 50-51.

E.125 In order for cold bonding to occur, the pressure applied by the steel cask on the concrete would somehow have to increase significantly from the amount quoted above. Tr. at 5894-95 (Soler). However, Dr. Singh testified he could not visualize how this could occur, because concrete would crush with the increasingly large pressure before it became bonded with the steel cask. Tr. at 6116-17 (Singh). Thus, occurrence of cold bonding between concrete and steel is highly improbable.

E.126 A calculation performed by the Staff established that the initial strain in the concrete caused by the presence of the cask is 8.33 micro-in./in., for a total deformation of 300 micro-in. Pomerening/Ofoegbu Post Tr. 6496, at 25. Long-term creep accounts for an additional deformation of 672 micro-in. Combining the initial and creep deformations gives a total deformation of 972 micro-in. Id. 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 under seismic load conditions. Id.

E.127 The State witnesses testified that the existence of cold bonding would operate to impede the initial sliding motion of the cask under seismic loadings. Tr. at 7720-21 (Ostadan). However, the seismic forces would readily break the bond and the cask would then slide on the pad in accordance with whatever coefficient of friction existed between the cask and the pad. Tr. at 7722-23 (Ostadan). Therefore, assuming the cold bonding phenomenon actually took place, its effect would be very limited both in duration and effect and, as Applicant's witnesses testified, would be subsumed in the variable coefficients of friction assumed in the Holtec analyses. Any small perturbations in the cask response due to irregular sliding would be within the range of results encompassed by the design basis simulations. Singh/Soler Post Tr. 5750, at 51-52.

4. Khan Report

E.128 At the request of the State, Dr. Mohsin Khan conducted a parametric study by modeling aspects of the seismic reaction of the HI-STORM 100 cask to evaluate Holtec's seismic analysis of free-standing casks at the PFS site. Khan/Ostadan Post Tr. 7123, at 7. For his parametric study, Dr. Khan utilized a finite element structural analysis code, SAP2000, to model a single HI-STORM 100 cask as beam elements in which the base of the cask is connected to the storage pad using nonlinear elements. Id. Dr. Khan's

methodology, analysis, results, and conclusions are described in the Analytical Study of HI-STORM 100 Cask System for Sliding and Tip-Over Potential During High-Level Seismic Event, Technical Report No. 01141-TR-000, Revision 0 (Dec. 11, 2001) (State's Exh. 122) [hereinafter Khan Parametric Study].

E.129 Dr. Khan performed case studies using three mathematical single cask models with varying degrees of complexity. Khan/Ostadan Post Tr. 7123, at 7. In the second and third case studies, Dr. Khan varied input parameters such as contact stiffness, the coefficient of friction, and damping. Khan/Ostadan Post Tr. 7123, at 8.

E.130 In the second case, which discounted rocking effects, for a coefficient of friction of 0.8 the horizontal cask displacement varied from 42.74 in. to 0.057 in. with varying contact stiffness from 1 x 10^6 pounds per in. to 454 x 10^6 pounds per in., respectively. Khan Parametric Study, Table 2 at 11. Similarly, in the three-dimensional case, the horizontal and vertical displacement varied with the values of contact stiffness, coefficient of friction, and structural damping. Khan Parametric Study, Table 3 at 13.

E.131 Dr. Khan chose to perform his parametric analysis for different contact stiffnesses and impact damping values using the SAP2000 computer code. He testified that the only reason he chose SAP2000, as opposed to a more general purpose program such as ANSYS, is because SAP2000 has a very efficient solution algorithm that takes less time to run than ANSYS. Tr. at 7171 (Khan).

E.132 SAP2000 is highly focused on the analysis of structures and was originally developed primarily for linear elastic analysis. Tr. at 9343-46 (Khan). It, however, does have the capability for a limited number of pre-defined nonlinear elements and may be used to model

local structural non-linearities such as gaps, isolators, and the like. Tr. at 9346-48 (Khan). Thus, SAP2000, like DYNAMO, is a small deflection program. Tr. at 7173-74 (Khan).

E.133 Dr. Khan insisted on cross examination that the fact that SAP2000 is a small deflection program did not bring into question the validity of the results of his analysis, in particular runs 1 and 3 of his third model, which show casks lifting off the ground by 1 or 2 ft. and moving laterally 30 to 40 ft. Tr. at 9348-60 (Khan). Dr. Kahn explained these results by stating that in these runs, the casks (1) moved essentially straight up by more than 1 or 2 ft., but did not rotate significantly, and (2) moved laterally 30 to 40 ft. in relation to the pad and the ground by bouncing up and down on the pad. Dr. Khan claimed that, because the casks assertedly did not rotate significantly, his analysis did not run afoul of the small deflection limitations of SAP2000. See Tr. at 9354-60, 9512-15 (Khan). Only if the casks showed large rotations would Dr. Khan consider there to be geometric non-linearities that would affect the validity of his SAP2000 results. Tr. at 9512-15 (Khan).

E.134 Dr. Khan's model was also unable to duplicate known classical solutions. Singh/Soler Post Tr. 5750, at 83-85.

E.135 The results obtained from Dr. Khan's model and his choice of contact stiffness are further cast into doubt by Dr. Khan's failure to verify his results. Although Dr. Khan's model was the first model of a large free-standing structure that he ever produced, the evidence indicated it had not been validated, verified, or bench-marked against either any classical problems or against any real world data. Moreover, his code was subjected neither to review by any third party nor to the kind of scrutiny that an NRC submission would ordinarily require to validate his results. Without such independent verification, the Board must view his results with some skepticism. Unlike Dr. Khan's model, Holtec benchmarked DYNAMO in a

manner consistent with ASME NQA-2a-1990, Part 2.7 “Quality Assurance Requirements of Computer Software for Nuclear facility Applications.” Singh/Soler Post Tr. 5750, at 77.

E.136 Holtec performed an analysis of Dr. Khan's model and input parameters for run 3 of his third model using VisualNastran a code which is capable of handling large deflections. Tr. at 9603 (Soler); Additional Cask Analyses at 15-16, 24-26. Specifically, Holtec used the same contact stiffness (1,000,000 pounds per in.) and damping at the cask pad interface (1%) as Dr. Khan had used in his run 3, as shown on Table 3 of the Altran Report. As discussed above, these are the two parameters that Dr. Khan claimed that Holtec did not properly apply in its model and which gave rise to the difference between his results and those obtained by Holtec in its cask stability analyses.

E.137 Holtec could not duplicate Dr. Khan's results using VisualNastran. Even with the unrealistic input parameters used by Dr. Kahn, the VisualNastran simulation did not show bouncing up and down of the cask by 1 to 2 ft. over a lateral distance of 25 or more feet as predicted by Dr. Khan. Rather, there was only a slight bouncing of the casks up and down and although the casks rocked and tipped, they never came close to tipping over. Furthermore, instead of the lateral displacement of 25 ft. or more, Holtec obtained displacements of less than a foot or two. Additional Cask Analyses; PFS Exh. 225A; Tr. at 9602-04, 9610-15 (Soler).

E.138 In addition to the use of SAP2000 beyond its capabilities, the values that Dr. Khan used for vertical contact stiffness of 1×10^6 pounds per in. and damping values of .01% and 1% that produced large cask movements were contrary to well understood physical principles. The use of such parameters would, as a result, lead to totally unrealistic predictions.

5. State's Request for a Shake Table Analysis

E.139 Dr. Khan opines that the only way to validate Holtec's seismic analysis is to benchmark the cask displacement with actual shake table test data. Khan/Ostadan Post Tr. 7123, at 13. Consistent with this opinion, Dr. Khan testified that he could also not claim his parametric study results were correct without first validating his results with test data and calibrating his damping, stiffness, and rocking values. Tr. at 7178-79 (Khan). Moreover, Dr. Khan opined that the nonlinear cask seismic analyses should be validated with test data regardless of the analyst's confidence in his solution. Id. at 9425.

E.140 As an example of what Dr. Khan believed to be the NRC philosophy supporting the need to validate nonlinear seismic analysis with test data, Dr. Khan referred to NRC Regulatory Guide 1.100, Institute of Electrical and Electronic Engineers, Inc., Recommended Practice for Seismic Qualification of Class 1E Equipment for Nuclear Power Generating Stations [hereinafter IEEE Std. 344-1987]. Khan/Ostadan Post Tr. 7123, at 13-14. IEEE Std. 344-1987, section 6 states that "[t]he analysis method is not recommended for complex equipment that cannot be modeled to adequately predict its response. Analysis without testing may be acceptable only if structural integrity alone can ensure the design-intended function." Id. at 14. IEEE Std. 344-1987 provides "good guidelines" for nonlinear seismic analysis which have been applied in the qualification of structures. Tr. at 9431-32 (Khan). IEEE Std. 344-1987 requires test data validation for Class 1E electrical equipment defined as "electrical equipment and system that are essential to emergency reactor shutdown, containment isolation, reactor core cooling, and containment and reactor heat removal, or are otherwise essential in preventing a significant release of radioactivity to the environment." Id. at 9428-29.

E.141 Dr. Khan testified that the IEEE Std. 344-1987 provision that states "analysis without testing may be acceptable only if structural integrity alone can ensure the design intended function" is not applicable to cask analysis because a designer cannot rely on its judgment that its design is adequate. Tr. at 9437-42 (Khan). In this case, shake table data would validate the cask dynamic response or whether the cask tips over under various ground motions.

E.142 The State contends that during the hearing, Dr. Luk confirmed he and the individuals in "his group" view shake table testing as "useful" in confirming his analysis. State Findings ¶ 367 (citing Tr. at 11,569, 11,572 (Luk)). In fact, the State claims that during the June 2002 hearings, Dr. Luk updated his previous testimony that he expected a "true state-of-the-art" shake table test facility that could accommodate a full scale cask would be available at the University of California at San Diego in the Spring of 2003 due to a recent grant from the National Science Foundation. Additionally, Luk testified that the Staff plans to request funding for shake table tests. State Findings ¶ 367 (citing Tr. at 11569-72 (Luk). The State also highlights Dr. Luk's testimony in which he states that he could "almost assure [that] the cask will not be damaged or destroyed on the shake table." State Findings at ¶ 367 (citing Tr. at 7111 (Luk).

E.143 The State also argues that Dr. Cornell agreed that physical test data would reduce the amount of uncertainty in a seismic assessment. State Findings ¶ 369.

E.144 The State argues that although he was opposed to the shake table tests, Dr. Singh did acknowledge that shake table tests are necessary "when you have some ambiguities and some concern, some possible uncertainty with respect to performance," albeit he believes there is no uncertainty with Holtec's analysis. State Findings ¶ 371 (citing Tr. at 9731(Singh)). Moreover, the State pointed out that although both Dr. Singh and Dr. Soler

initially denied ever discussing performing shake table tests with PFS (Tr. at 9732-33 (Singh/Soler)), in November 1997, Dr. Singh sought funding from PFS to verify their analytical work by conducting scale model tests on a shake table (Tr. at 9738-40 (Singh)). See State Exh. 197A. Dr. Singh testified that because the Staff relied upon a "simpleminded static limit," NRC requested that Holtec conduct scale model shake table tests to support the HI-STORM 100 application for a CoC at high earthquake levels. Tr. at 9739-40 (Singh). According to the State, Dr. Singh, in responding to the State's claim, denied ever recommending shake table tests to PFS, claiming instead the letter was "politically correct" and a result of "the guy who has the gold makes the rule," implying that NRC, not Holtec, desired the shake table tests. State Findings ¶ 371 (citing Tr. at 9745-48 (Singh)).

E.145 Although Dr. Singh does not define "high earthquake levels," it appears Holtec sought funding from PFS and Pacific Gas and Electric (PG&E) for shake table tests. See Tr. at 9742 (Singh). Dr. Soler testified that ground motions at the PG&E Diablo Canyon facility are 0.9g. Tr. at 5932 (Soler). According to the State, the peak horizontal ground accelerations at the PFS site are estimated as 1.15g for deterministic, and 0.711g for a 2000-year event. State Findings ¶ 377.

E.146 Moreover, the November 1997 Holtec letter to PFS states "if PFS elects not to support [shake table testing], then we can provide all high seismic material stripped from Revision 1 of the HI-STORM [topical safety evaluation report] for direct incorporation in the Skull Valley site-specific submittal, and we will proceed with only anchored cask certification on this new docket." State Exh. 197A.

E.147 The State argues that no expert disagreed that the results of nonlinear analyses could be validated only with test data. The State asserts that the experts disagreed in various degrees as to the actual need for shake table data. Moreover, the State contends

that given Dr. Luk’s anticipation of a shake table test facility able to conduct full scale tests becoming available, Dr. Luk’s assurances that a cask would not be damaged by shake table tests, and the lack of convincing evidence of the accuracy and reliability of the Holtec 2000-year report, the Board cannot find without such testing that the Applicant has met its burden to demonstrate that free-standing HI-STORM 100 casks will not tip over under at 2000-year earthquake at the PFS facility. State Findings ¶ 376.

E.148 Although the State points to Dr. Luk’s testimony concerning the "useful[ness]" of shake table testing, it neglects to highlight Dr. Luk's extensive discussion on the limitations of shake table testing for cask stability analysis.[42]

E.149 Dr. Cornell, who has extensive knowledge of nonlinear seismic analysis, testified that one “would gain information” from shake table testing that would reduce the amount of uncertainty, but he went on to say that: “In practice, it’s seldom necessary to do so, seldom believed to be necessary to do so when doing nonlinear dynamic analyses of a facility.” Tr. at 7979 (Cornell). He further stated that it is his opinion that “in this case there is sufficient margin enough to demonstrate that this ten to the minus four accident failure probability is easily reached without the need” for shake table tests. Tr. at 8024 (Cornell).

E.150 Dr. Cornell further made the important point that “[a] shake table is another model” with its own uncertainties. While one would gain some information, the test would introduce a different set of uncertainties. Tr. at 8023-25 (Cornell). For example, it was widely

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[42]Dr. Luk pointed to limitations on shake table testing other than size of a facility, such as being able to apply only one horizontal motion. Tr. at 6966-68 (Luk). More importantly, he noted that there are some limitations that cannot be technically overcome. For example, in situ soil conditions cannot be recreated on a shake table test, and as a result, SSI effects cannot be incorporated. Tr. at 6968 (Luk). Moreover, Dr. Luk never agreed with the State’s implicit assumption that shake table testing is necessary to confirm whether a finite element model is appropriately constructed. Indeed, in a complicated model like Dr. Luk’s model that takes into account SSI, the shake table testing cannot, according to Dr. Luk, simulate that SSI.

recognized by all witnesses that it would be impossible to duplicate the PFS site conditions in a shake table test. See, e.g., Tr. at 7982-83 (Cornell); Tr. at 9728-29 (Singh).

E.151 Dr. Singh emphasized the difficulty in "simulating the conditions of a cask on the pad" in a shake table test. Tr. at 9682-83 (Singh). He explained, for example, why it would be not be feasible to experimentally control the coefficient of friction between the cask and the pad so as to be able to obtain meaningful data that one could correlate with the numerical computer models of free-standing casks on a concrete pad. Tr. at 9682-84, 9888-91 (Singh). In other words, one would not be able to design a shake table test "to measure all [the] critical variables that participate in the dynamic behavior" of the cask and then set the input parameters of your computer model accordingly to see how well the results predicted by the program correlate with the test data. Id. at 9892. Absent such correlation, one could not use the data from a shake table test as a reliable benchmark for the numerical program. Id. at 9890-91. We find that, in light of the limitations that would be embodied in shake table testing of this nature will hamper its usefulness from a scientific point of view, such testing would not be particularly beneficial here in establishing reasonable assurance of adequate public health and safety.

6. The Staff-Sponsored Sandia Report Conducted by Dr. Vincent Luk

E.152 To verify the Holtec Analysis, the NRC Staff commissioned Dr. Vincent Luk of Sandia National Laboratories (SNL) to conduct an independent evaluation of the PFS project. Dr. Luk assembled and directed a team of experts in conducting an evaluation of the seismic behavior and stability of the free-standing, cylindrical HI-STORM 100 casks to be installed on concrete storage pads at the proposed PFS facility. 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 SSI,

under prescribed seismic conditions. NRC Staff Testimony of Vincent K. Luk and Jack Guttmann Concerning Unified Contention Utah L/QQ (Geotechnical Issues) [hereinafter Luk/Guttmann] Post Tr. 6760, at 2-3. 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) [hereinafter Luk Report].

E.153 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 SNL 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.

E.154 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 PFS facility, including the SSI effects during a seismic event. Three different sets of seismic conditions were modeled: (1) the 2000-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 2000-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 2000-year event; and ground motions for the PFS 10,000-year event. Luk/Guttmann Post Tr. 6760, at 4.

E.155 The ABAQUS/Explicit code was used to analyze the three-dimensional coupled finite element models, that consist of (1) a single cylindrical HI-STORM 100 cask (with the MPC-68 option); (2) a flexible full-sized concrete pad (30 ft. x 67 ft. x 3 ft.); (3) a shallow surface layer of compact aggregate around the pad; (4) a soil-cement layer adjacent to the pad (2 ft. 4 in. thick); (5) a soil-cement layer under and adjacent to the pad (approximately 2 ft. thick); and (6) an underlying, layered soil foundation. The layout of the entire coupled model is shown in Figure 1 of the Luk Report. Luk Report 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 Luk Report, 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. In other words, the cask and pad were modeled as elastic bodies with zero damping. Luk/Guttmann Post Tr. 6760, at 5-6.

E.156 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 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 SSI effects. Luk/Guttmann Post Tr. 6760, at 5.

E.157 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.

E.158 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 2000-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. Luk/Guttmann Post Tr. 6760, at 6.

E.159 Each set of seismic time histories has one vertical and two horizontal components of statistically independent seismic accelerations. For the 2000-year return period earthquake, the PGAs that were modeled, based on artificial time histories specific to the PFS site, were 0.728g (horizontal, east-west), 0.707g (horizontal, north-south), and 0.721g (vertical); these PGAs envelop the 2000-year design basis response spectra of 0.711g (horizontal) and 0.695g (vertical) stated in the 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

1.25g and 1.23g for the horizontal components, and 1.33g 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 0.641g for the two horizontal components, and 0.433g for the vertical component. The duration for this event was 41.8 seconds. Luk/Guttmann Post Tr. 6760, at 6-7.

E.160 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. Luk/Guttmann Post Tr. 6760, at 7.

E.161 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 2000-year (design basis) event, the best estimate soil profile data, 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 tip-over) were used at the cask/pad interface. Bounding coefficients of friction of either 1.00 or 0.31 were also assumed at the

other two interfaces, as shown in Table 8 of the Luk Report (Best Estimate, Model Type 1). Luk/Guttmann Post Tr. 6760, at 7-8.

E.162 These sensitivity studies showed that the maximum horizontal displacement (sliding) of the cask was obtained when using a coefficient of friction of 0.20 at the cask/pad interface and 0.31 at the pad/soil-cement layer and soil-cement layer/soil foundation interfaces, as shown in Table 8 of the Luk Report (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 Luk Report for the 2000-year event. Luk/Guttmann Post Tr. 6760, at 8; Luk Report at 30.

E.163 Similarly, several studies were conducted for the 1971 San Fernando Earthquake (Pacoima Dam record) and the 10,000-year return period event, using a coefficient of friction of 0.20 at the cask/pad interface, and 0.31 at the other two interfaces, in order to maximize the potential for horizontal displacement (sliding) of the cask. The results of these studies are shown in Tables 9 and 10 of the Luk Report. 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 0.8 at the cask/pad interface and 1.0 at the other two interfaces, in order to maximize the potential for cask tip-over. These results are also shown in Tables 9 and 10 of the Luk Report. Luk/Guttmann Post Tr. 6760, at 8; Luk Report at 31-32.

E.164 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 ft., 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 ft.) to incorporate the depth variation of soil properties such as shear wave velocity and damping profiles. As shown in Section 3.4.1 and Tables 2-7 of the Luk Report, specific soil properties considered include: (1) Young's Modulus, (2)Poisson's ratio, (3) density, (4) damping ratio, and (5) a mass-related damping factor. This foundation modeling and its rationale are discussed in greater detail in sections 3.2.4 to 3.4.1 of the Luk Report. Luk/Guttmann Post Tr. 6760, at 8-9; Luk Report at 7-12.

E.165 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 2000-year return period and the 1971 San Fernando Earthquake, Pacoima Dam record, as shown in Tables 2 to 4 of the Luk Report. 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; in contrast, for seismic events with a 2000-year return period, the low strain shear modulus and damping were used. Luk/Guttmann Post Tr. 6760, at 9; Luk Report at 10-12.

E.166 The results from the Staff's seismic analyses indicate that the maximum horizontal cask sliding displacements are 3.98 in. for the 2000-year return period event; 3.00 in. for the

1971 San Fernando Earthquake, Pacoima Dam record; and 15.94 in. for the 10,000-year return period event. Luk/ Guttmann, Post Tr. 6760, at 9-10.

E.167 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 PFS site. The separation distance between neighboring casks is 47.5 in. Half of this distance, or 23.75 in., is regarded as the cask collision criterion. Inasmuch as maximum displacement under the design basis 2000-year earthquake is 3.98 in., 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 in. Similarly, under 10,000-year seismic conditions, the maximum displacement was 15.94 in., which is less than the collision criterion of 23.75 in. Thus, even under the beyond-design basis 10,000-year event conditions, cask collisions were not found to occur. Luk/Guttmann Post Tr. 6760, at 10.

E.168 Similarly, the model predicts that tip-over of the HI-STORM 100 storage cask is unlikely to occur at the PFS site during a seismic event. In this regard, with respect to the 2000-year return period seismic event, the coupled model analysis predicts that the maximum cask rotation in either horizontal direction with respect to the vertical axis is equal to or less than 0.03 degrees, using a coefficient of friction of 0.20 for the cask/pad interface. Further, using a coefficient of friction of 0.80, in order to maximize the amount of cask rotation, results in a maximum cask rotation of about 0.22 degrees in the east-west direction and about 0.40 degrees in the north-south direction, with respect to the vertical axis, for the 2000-year earthquake. In sum, the maximum cask rotation, with respect to the vertical axis, is equal to or less than 0.40 degrees under 2000-year return period seismic conditions. Luk/Guttmann Post Tr. 6760, at 11.

E.169 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 0.2, 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 0.8, 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). Luk/Guttmann Post Tr. 6760, at 11.

E.170 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. Luk/Guttmann Post Tr. 6760, at 11-12.

E.171 Based on the maximum cask rotation predicted by the model, Dr. Luk concluded that cask tip-over is unlikely to occur during either the 2000-year or 10,000-year return period seismic events at the PFS site. The cask rotation that is associated with a cask tip-over is approximately 29 degrees. A rotation of less than 29 degrees (as is predicted here) would

be insufficient to result in tip-over of a loaded HI-STORM 100 cask. Luk/Guttmann Post Tr. 6760, at 12.

E.172 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 2000-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 in. above the top surface of the pad. Luk/Guttmann Post Tr. 6760, 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 of 0.26 in., 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 in. above the top surface of the pad. Luk/Guttmann Post Tr. 6760, at 12-13.

E.173 In sum, based on its confirmatory analysis, the Staff concluded that excessive cask sliding, cask collisions, and cask tip-over will not occur during either a 2000-year return period or 10,000-year return period seismic event at the PFS site. Luk/Guttmann Post Tr. 6760, at 13.

E.174 The Luk Report performed by Dr. Luk did not attempt to duplicate the Holtec analysis, nor did it do so. The Luk Report utilized a methodology (a state-of-the-art three-dimensional finite element analysis) 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. at 6934, 6937-41 (Luk). 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 tip-over in the event of either the design basis (2000-year return period) earthquake or the 10,000-year return period seismic event. See Luk/Guttmann Post Tr. 6760, at 13.

a. Conflict of Interest Involving Study's Advisory Panel

E.175 A review panel consisting of three NRC Staff and four industry representatives provided technical advice and input to the generic and site specific cask stability studies conducted by Dr. Luk and his associates. Tr. at 6994-6996, 7052-54 (Luk). The hearing testimony revealed that industry representatives included representatives from Southern Company, San Onofre Nuclear Generation Station, the Electric Power Research Institute, and a private consultant, Dr. Robert Kennedy. Id. at 6995. Southern Company and Southern California Edison, owner of San Onofre, are members of the PFS consortium. SER at 17-1. The role of the industry panel members was to provide recommendations concerning the analysis methodology and range of input parameters used by Dr. Luk. Tr. at 7054 (Luk). The advisory panel, including industry representatives, reviewed the generic Luk finite element model and provided their comments and recommendations. Id. at 7076. According to the State, Dr. Luk met with the advisory panel on three occasions, including November 2001, to discuss the details of the PFS model. State Findings ¶ 383 (citing id. at 7077-78). The panel provided comments on the completed 2000-year analysis for PFS at the November 2002 advisory panel meeting. The advisory panel meetings were not open to the public. Id. at 7082-83.

E.176 Although Dr. Luk testified that he had no knowledge that any of the industry representatives were associated with PFS (Tr. at 6995-96, 7053-54, 7081-86 (Luk)), the State contends that the Licensing Board should presume the Staff was aware that representatives from PFS member companies were on the advisory panel. State Findings ¶ 384. The State argues that "fundamental fairness" to the conduct of a licensing proceeding mandates the "disclos[ure] of all potential conflicts of interest," whether or not a party believes them to be material and relevant to a licensing proceeding. Long Island Lighting Co. (Shoreham Nuclear Power Station Unit 1), LBP-82-73, 16 NRC 974, 979 (1982). Furthermore, the State insists that such disclosure is necessary to enable the Licensing Board to determine the materiality of the potential conflict of interest. Due to the lack of disclosure of potential conflicts of interest of the industry representatives on Dr. Luk's advisory panel, compounded by the lateness of the availability of the Luk Report to the State, the State insists that it has had little or no opportunity to probe the backgrounds of the advisory panel and its influence on the Luk methodology and analysis during discovery. Because of the constraints placed on the State to probe the issue and potential inability to raise it themselves, the State argues that the Board cannot find that at least two industry representatives on Dr. Luk's advisory panel have no conflict of interest with the outcome of the Dr. Luk's cask stability analysis for the PFS site. Thus, the State requests that the Board weigh, in assessing the Luk Report, the impact of the potential conflict of interest from the PFS member companies in its assessment of the Luk Report. State Findings ¶ 384.

E.177 Although we agree that there is an appearance of a potential conflict, we find no reason for concern that Dr. Luk's report may have been tainted due to this alleged potential conflict of interest. Based on our observation of Dr. Luk's testimony, including his responses to cross-examination and Board questions, we are fully satisfied as to his candor, objectivity,

and professionalism. Furthermore, Dr. Luk testified that he did not receive any comments from the review panel prior to completing his preliminary PFS site-specific analysis in October 2001; and while the review panel was then given an opportunity to review his site-specific analysis, that review did not result in any changes in the report before it was issued in March 2002, other than the manner in which the results were presented and the removal of irrelevant material. Tr. at 7102-05 (Luk).

E.178 There is no evidence to suggest that the utility representatives gave Dr. Luk incorrect technical advice, or that Sandia followed any such advice, or that any advice provided by the utility representatives adversely affected the validity of the results of Dr. Luk's PFS facility analysis.

E.179 What was gleaned from the extensive examination of Dr. Luk on the review panel was that the panel gave advice on technical issues such as what damping to use, the values of the coefficient of restitution, and the merits of using a single cask on a single pad for the analyses. Tr. at 7084-85 (Luk). At no point did the review panel members provide any advice that Dr. Luk considered inappropriate. Id. This is confirmed by the written minutes of the meetings of the panel, which reveal that the meetings were of a highly technical nature, as would be anticipated for such a panel. See Staff Exh. GG.

b. Dr. Luk's Relative Experience

E.180 According to the State, Dr. Luk's sole experience in modeling the free-standing, dry storage casks includes the site specific analysis for PFS, Hatch, and San Onofre; however, the State contends that the site conditions for Hatch and San Onofre are not similar to those at the proposed PFS site. State Findings ¶ 385. The State argues that although Dr. Luk modeled ground motions in excess of those estimated for Hatch, the Hatch ground motions

are approximately 0.15g horizontal and 0.1g vertical. Furthermore, the State suggests that although the Hatch site stores twelve casks on a concrete pad in a 2 x 6 array, Dr. Luk modeled a square pad with a 2 x 2 array. According to the State, the casks proposed for San Onofre are three unanchored, rectangular, horizontal casks tied together, unlike the individual cylindrical HI-STORM 100 casks. State Findings ¶ 385. Dr. Luk acknowledged that the seismic response of the horizontal casks at San Onofre is “very different” when compared to the cylindrical cask proposed for PFS. Tr. at 7056 (Luk). The State argues that the Hatch site conditions are different from those proposed for PFS. It further argues that the San Onofre site conditions and the facility design are substantially different from those proposed for PFS. Therefore, the State insists that Hatch and San Onofre do not provide relevant modeling experience for the PFS design or site conditions. State Findings ¶ 385.

E.181 The State claims that initially Dr. Luk testified that he has no expertise in SSI, but later recanted his testimony to claim he has such expertise. According to the State, Dr. Luk claims the evaluation of SSI effects is nothing more than a systematic evaluation to address the dynamic coupling between a structure and soil. State Findings ¶ 389 (citing Tr. at 6917, 7036-37 (Luk)).

E.182 The State also claims that Dr. Luk professes to be an expert in SSI based on his work over the past few years in evaluating coupling between components in a nuclear power plant. However, the State insists that Dr. Luk’s own SSI experience is limited to “the past few years” which would encompass the analyses he performed for the Hatch and San Onofre ISFSIs, and requests that we find the record bare in its support that Dr. Luk alone is qualified to model SSI effects. State Findings ¶ 390.

E.183 The State also contends that there is insufficient evidence to find that Dr. Luk's associates are qualified to accurately model the soil dynamics or the SSI effects. State Findings ¶ 391.

E.184 The State claims that Dr. Luk's experience in the nonlinear modeling of the seismic behavior of cylindrical, free-standing casks is limited to his generic study and the Hatch analyses. The State, therefore, argues that Dr. Luk does not have experience in the nonlinear modeling of the seismic behavior of cylindrical, free-standing casks supported by cement-treated soil and a relative soft clay foundation at ground motions equal to or greater to the 2000-year earthquake at PFS. State Findings ¶ 392.

E.185 For the reasons stated previously, we find that Dr. Luk and his team are quite experienced in conducting finite element analyses of the type conducted here, and that their PFS site-specific analyses provide useful, relevant, and reliable evidence as to the stability of the casks at the PFS facility, in the event of either a 2000-year DBE, or a 10,000-year beyond-design basis event.

E.186 Although the State attempts to portray Dr. Luk as having no expertise in soil dynamics, the record does not support this view. Dr. Luk described the members of his research team and their respective areas of expertise related to his PFS site-specific analysis. Tr. at 6765-66 (Luk). Dr. Luk stated that he is personally not an expert in soil mechanics, but he is very familiar with the dynamic coupling process involved in SSI. Further, he stated that he considers himself to be an expert in conducting SSI analyses. In addition, he obtained input for his analytical model from a member of his team (Mr. Po Lam), who developed the soil foundation model and the deconvolution process that is part of the soil foundation modeling. Tr. at 7036-37 (Luk). Based on this testimony, as well as our familiarity with Dr. Luk's ability to respond to questions concerning this aspect of this study,

we are unable to adopt the State's suggestion that neither Dr. Luk nor any member of his research team are qualified to model soil dynamics or SSI effects.

c. Comparison of Dr. Luk's Report and the Holtec Report

E.187 According to the State, the Holtec witnesses agreed that the Luk Report did not confirm Holtec's methodology, but similarly concluded that the cask would not tip over. State Findings ¶ 394. Dr. Soler testified that the Staff's analysis "studies a different problem than [Holtec] simulated either with DYNAMO or VisualNastran." Tr. at 5898 (Soler). Dr. Soler further testified that the Staff's analysis "models certain features of the problem in a different manner than [Holtec]." Id. The State also claims that Dr. Soler did not know why there was a difference in the results between his analysis and the Staff analysis for a 10,000-year earthquake with a coefficient friction of 0.8. State Findings ¶ 394. Although Dr. Soler claims the Luk Report's magnitude of excursions are in the same order with those determined by Holtec (Tr. at 5998 (Soler)), the State claims that Dr. Luk testified that the Holtec and Luk results cannot be compared. State Findings ¶ 394 (citing Tr. at 6949-51 (Luk)).

E.188 The State also asserts that Dr. Luk did not compare his SSI effects for a 2000-year or 10,000-year earthquake with those predicted by PFS. Nor did Dr. Luk compare his deconvoluted time histories for a 2000-year or 10,000-year earthquake with those predicted by PFS at similar depths. State Findings ¶ 396.

E.189 According to the State, when probed about his confidence in the results from a "very complicated model" with large amounts of data, Dr. Luk further testified that the results are dependent on the input parameters. State Findings ¶ 398 (citing Tr. at 6987 (Luk)). The Luk model has 4124 elements: 864 elements model the cask, 384 elements model the storage pad, 28 elements model the aggregate, 848 elements model the soil-cement adjacent and

beneath the pad, and 2000 elements model the soil foundation. Tr. at 7026-27 (Luk). Given the complexity of Dr. Luk's model, at this juncture the State argues that it is appropriate to heed Dr. Cornell's affirmation not to be too enamored with the computer program itself. Furthermore, the State insists that Dr. Luk acknowledges the value in validating nonlinear results with test data. State Findings ¶ 387.

E.190 Due to the lack of test data to validate his results, the State contends that Dr. Luk relies on his sensitivity analyses and the experience he has gained in his NRC related study to substantiate his model. State Findings ¶ 399. In an effort to demonstrate that the accuracy of his model, Dr. Luk points to his seismic analysis at Hatch where the ground motion was increased to demonstrate that his model could show cask tip-over. Tr. at 6988 (Luk). Although Dr. Luk relies on the Hatch analysis to support the accuracy of his PFS model, the State notes that he also testified that the PFS model was modified to simulate the soil-cement layer at the PFS site. State Findings ¶ 399 (citing Tr. at 7026 (Luk)). Moreover, the State contends that the soil-cement (cement-treated soil) layer, with interfaces both above and beneath the layer, "actually caused quite a bit of difficulties in the simulation portion." State Findings (citing Tr. at 7028 (Luk)).

E.191 Based on the apparent complexity of the model, the differences in the PFS and the Hatch model, and the lack of test data to validate any results, the State insists there is insufficient evidence to conclude the PFS model developed by Dr. Luk accurately or conservatively estimates cask response, including displacement, angle of rotation, and tip-over. State Findings ¶ 400.

E.192 We find the State's concerns to be without merit. With respect to the differences in results obtained by the two models, particularly for the 10,000-year beyond-DBE and a coefficient of friction between cask and pad of 0.8, such differences are to be expected. As

Dr. Singh testified, “[w]hen you model a complex problem and you take a different modeling path you're going to have some differences in the final results. But the solutions in the end are essentially in agreement with ours.” Tr. at 5899 (Singh).

E.193 The difference in methodologies between the two analyses is not a basis for questioning either set of results. Indeed, had Dr. Luk used the same methodology and obtained the same results as Holtec, there would be little confirmatory value in the Dr. Luk’s results. It is the fact that Holtec and Dr. Luk used two different methodologies that makes the Luk Report’s result analyses valuable in confirming the Holtec analyses. It is because the Luk analyses differ in many respects from Holtec’s with respect to assumptions or methodological approaches about which the State expressed concern (e.g., choosing a particular contact stiffness value, choosing a particular damping value, not taking into account the effects of non-vertically propagating seismic waves, using soil springs to represent soil dynamic behavior, not modeling SSI fully, etc.), that the analyses are confirmatory, arriving at the same conclusions without using methodological assumptions the State contended were problematic in the Holtec analyses. See Tr. 6827-29 (Guttmann).

E.194 As we have previously discussed above, the State's arguments on the need for shake table testing do not prevail. The State also offers explanation for why the absence of physical benchmarking renders the Luk Report analyses non-confirmatory of the Holtec analyses. The absence of shake table tests does not mean that the adequacy of the Luk model has not been independently established. The ABAQUS code has been benchmarked against a wide variety of classical problems. For the specific case of the PFS facility, Dr. Luk and his team checked the results produced by the analysis against test data to verify the appropriateness of the analytical results. Tr. at 6812-13 (Luk).

d. Luk's Modeling

E.195 In Table 8, column 2, of the Luk Report, μ1 is the interface coefficient of friction between the casks and the storage pad; μ2 represents the coefficient of friction at two different interfaces: between the bottom of the pad and the top of the cement-treated soil; and between the bottom of the cement-treated soil and the top of the upper Lake Bonneville clays. Tr. at 10,347-48 (Bartlett).

E.196 To model the interfaces, including interfaces above and below the cement-treated soil (μ2), Dr. Luk used what he referred to as Coulomb's law of friction, F = μ n, where F is the frictional resistance, μ is the coefficient of friction, and n is the normal stress. Tr. at 11,510 (Luk); Tr. at 11,407 (Bartlett). Dr. Luk observed that "the so-called coefficient of friction at the interface between two bodies is an estimate of the friction in the systems of one body in motion with respect to the other, basically, fitting Coulomb's Law of Friction." Tr at 11,509-10 (Luk). In particular, Dr. Luk testified that "Coulomb's Law of Friction is a description of the frictional resistance at the interface, as material properties at the interface." Tr. at 11,510 (Luk).

E.197 By treating the interface conditions as frictional material, the State is adamant that Dr. Luk's model does not represent the actual PFS design or the PFS site soils. Tr. at 10,375-77 (Bartlett). Of particular concern to the State is the way in which Dr. Luk has modeled the interface conditions, μ2, and also the way in which Dr. Luk's model does not account for the post-yield behavior of the upper Lake Bonneville clays. State Findings ¶ 412.

E.198 The actual design of the storage pad system at the PFS site is undisputed: a 1 to 2 ft. thick cement-treated soil layer, the top of which is bonded to the underside of the concrete storage pads and the bottom of which is bonded to the top of the native soil layer (i.e., upper

Lake Bonneville clay). SAR at 2.6-108 (PFS Exh. JJJ); Trudeau/Wissa Post Tr. 10,834, at 24-25; SER at 2-59. The State claims that unlike structural fill, which derives its resistance to seismic forces from friction, cement-treated soil derives its resistance to seismic forces from cohesion. State Findings ¶ 413.

E.199 The State contends that the PFS design intent to withstand seismic forces from the DBE is to rely on cohesion from bonding at the interface layers (the upper Lake Bonneville clays interface with the laminated cement-treated soil lifts; and the cement-treated soil with the underside of concrete pad) to transfer the horizontal earthquake forces downward from the storage pad to the underlying clay soils. Tr. at 10,375-76 (Bartlett).

E.200 According to the State, the soils characterization at the PFS site was conducted by Stone & Webster but these soil properties are not used in the Luk model. According to the State, Luk's model uses the dynamic soil properties -- upper bound, best estimate, and lower bound for the PFS site -- developed by Geomatrix. State Findings ¶ 415.

E.201 The State claims that clays are not a granular material -- they are a relatively soft plastic material. Tr. at 10,377 (Bartlett). The State further claims that the clays that PFS is relying upon to transfer earthquake forces -- the upper Lake Bonneville clays -- derive their strength from cohesion (i.e., the undrained shear strength) not from friction. Id.

E.202 The State claims that cohesion is a material property -- it is the shear strength or resistance to sliding within the material. Tr. at 11,687 (Bartlett). The State asserts that Dr. Luk indicated that his model does not incorporate cohesive strength of the soils. State Findings ¶ 417.

E.203 The State claims that cohesion is generally not thought of as a dynamic property -- it is a shear strength property that is measured by a static test. Tr. at 11,706 (Bartlett). The

State believes that cohesion is also an interface property -- it is the strength of the bond at the interface between two layers.[43] Tr. at 11,688 (Bartlett).

E.204 Furthermore, the State argues that cohesion was not an inherent property included in the dynamic soil properties that were developed by Geomatrix for the PFS site. Tr. at 10,409; 11,690-91 (Bartlett). The dynamic soil properties given to him by the Staff and apparently developed by Geomatrix are (1) maximum shear modulus, (2) soil density, (3) Poisson's ratio, and (4) an estimation of shear modulus degradation and damping degradation as a function of strain. Tr. at 11,710 (Bartlett). Dr. Bartlett, the State's soils expert, testified that he knows of no theory of obtaining cohesion of the upper Lake Bonneville clays from the dynamic soil properties developed by Geomatrix. Tr. at 11,711 (Bartlett).

E.205 In this regard, the State contends that the Board should give particular deference to Dr. Bartlett. The State claims his expertise in soils is unquestioned. The State asserts that Dr. Luk, on the other hand, admits that he has no expertise in soils and that he relied on a seismologist for the soil input to the numerical model. The State argues that Dr. Luk admits shear strength (i.e., cohesion) is not represented in his model. The State requests that the Board finds that the Luk model does not incorporate cohesion through the use of the dynamic soils properties developed by Geomatrix. State Findings ¶ 420.

E.206 The State also argues that the Luk Report does not model the design PFS intends to employ at the site; does not model the actual interface conditions at the PFS site; and by using a frictional material to represent the behavior of the two µ2 interfaces, employs an inappropriate constitutive relationship to use in the numerical model. State Findings ¶ 425.

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[43]Strictly speaking the term "adhesion" refers to the condition between two dissimilar materials; "cohesion" is the failure within the material itself. Tr. at 11,416-17 (Bartlett).

E.207 In sum, the State believes that the properties described in the Luk Model at the interfaces designated as µ2 do not properly represent the strengths of those interfaces, and therefore, the Luk Model overemphasizes sliding, which potentially could dampen out the seismic energy that is delivered to the cask. State Findings ¶ 426.

E.208 The Applicant, in response to the State's assertions, stated that in modeling the interfaces above and below the cement-treated soil, Dr. Luk used a well-established method of finite element modeling. PFS Reply ¶ 353 (citing Tr. at 11,511-12 (Luk)). The Applicant asserted that the State and its witness Dr. Bartlett misunderstand Dr. Luk's methodology. PFS Reply ¶ 353. It was not, as Dr. Bartlett interpreted it, to treat the underlying upper Lake Bonneville clays as if they were "sand." Tr. at 10,530-35 (Bartlett). In reality, as Dr. Luk testified, modeling the interface using a frictional relationship does not represent the characterization of the properties of the materials, but that of the interface between them. Tr. at 11,510-12, 11573, 11580-81 (Luk). As described by Dr. Luk,

> "Coulomb's Law of Friction is a description of the frictional resistance at the interface, as material properties at the interface. It's also a parameter that has depends on the material, but more on the surface condition of the two bodies."

Tr. at 11,510 (Luk).

E.209 In response to the State request for the Board to give "particular deference" to Dr. Bartlett on this issue, the Applicant insists that Dr. Bartlett is admittedly unqualified to render an opinion as to how a finite element model should be constructed or interpreted. PFS Reply ¶ 354. Furthermore, during the hearing, Dr. Bartlett acknowledged that he could not comment on the appropriateness of the modeling techniques used by Dr. Luk and his Sandia colleagues, but was limiting his comments to the properties of the materials analyzed by the report. Tr. at 10,347 (Bartlett). The Board notes that Dr. Luk testified that his model did not

represent any particular material at the interface, but rather the "physical phenomenon associated with a sliding resistance, based on Coulomb's Law of Friction." Tr. at 11,586 (Luk).

E.210 Much of the State's concern with Dr. Luk's model centers on the State's disagreement with the coefficients of friction used. However, as Dr. Luk testified, the coefficient of friction at the interface does not represent a property of a material. Tr. at 11,573, 11,580-81 (Luk). Thus, the State's attacks are without merit since they rebut assumptions that have not been made and methodologies that have not been applied in Dr. Luk's analyses.

E.211 Contrary to the State's claim that soil cohesion was not taken into account as a property of the underlying soils by the Luk model, we note that during the hearing Dr. Luk testified that his model did take into account the internal cohesion of the materials modeled (Tr. at 11,573-75, 11,580-81 (Luk)).

E.212 Moreover, if there was error in Dr. Luk's model and it indeed treated the soils as cohesionless materials, such an error would maximize the tendency of the pads to slide. See Tr. at 10,535 (Bartlett). However, the results of the Luk analysis show that there was minimal pad displacement under both the DBE and the beyond-design basis 10,000-year seismic event. Tr. at 11,516-29, 11,575-78, 11,586-88, 11,610-11 (Luk); Staff Exh. YY. Thus, if the State is correct and the Luk analysis tends to disregard the cohesive properties of the soil, the actual behavior of the pads in an earthquake should exhibit even less sliding than predicted by Dr. Luk's report.

E.213 The third deficiency with the Luk model of the foundation soils alleged by the State is that it does not account for the post-yield behavior of the upper Lake Bonneville clays.

However, Dr. Luk explained that such effects are not significant and, after a few months of evaluations, he and his team decided that using an elastic model to simulate the soil foundation was adequate. Tr. at 11,548. (Luk).

E.214 We find the State's final claim that Dr. Luk erred by using the soil properties provided by Geomatrix to Holtec rather than the soil properties developed by Stone & Webster, to be without merit for two primary reasons. First, Stone & Webster did not develop dynamic soil properties, but also used those developed by Geomatrix in that it used dynamic loads on the casks obtained from ICEC's design calculation, which in turn used the results of Holtec's dynamic analyses that utilizes the soil properties developed by Geomatrix. See Tr. at 6235-37, 6340 (Trudeau). Second, by using the Geomatrix soils properties as input to the computer model, Dr. Luk incorporated soil cohesion into his analysis. Tr. at 11,573-75 (Luk). While Dr. Bartlett disputes that it is possible to incorporate soil cohesion from the dynamic soil properties provided by Geomatrix, we find Dr. Luk to be more experienced with the ABAQUS code. And Dr. Luk testified that incorporating soil cohesion is among the features of the ABAQUS code. Tr. at 11,574-75 (Luk).

E.215 Dr. Bartlett raised two other concerns about Dr. Luk's modeling of the foundation soils: (1) the assumed thickness of the cement-treated layer, and (2) the value of the Young's Modulus used in the analysis. See Tr. at 11,481-82 (Bartlett). The Young's Modulus issue is discussed separately below. With respect to the thickness of the cement-treated soil layer, Dr. Bartlett expressed a concern that Dr. Luk's model assumed a uniform thickness of 2 ft. for the cement-treated soil layer. Two ft. is a maximum value and the actual thickness can be as little as 1 ft., depending on the amount of aeolian soil that needs to be replaced with a cement-treated soil mixture. Tr. at 11,445-46 (Bartlett). Dr. Luk explained, however, that while his team was aware that the thickness of the cement-treated

soil mixture was variable, the decision was made to use the higher value for the thickness of the cement-treated soil, because with a thicker layer of cement-treated soil, more of the energy that is associated with the ground excitations will go the pad and the cask, so in that sense, more conservative results will be generated in evaluating the dynamic behavior of the cask. Tr. at 11,544 (Luk).

e. Young's Modulus

E.216 The Luk Report uses a 270,000 psi Young's Modulus for the cement-treated soil under the pads. Luk Report at 10-12.

E.217 None of the parties disagree that Holtec has constrained the Young's Modulus of cement-treated soil to less than 75,000 psi. However, the State believes, that meeting a Young's Modulus of 75,000 psi is an integral part of the PFS soil-cement testing that PFS has yet to conduct. State Findings ¶ 428.

E.218 Dr. Luk testified he obtained the 270,000 psi from a member of the NRC Staff, Mr. Mahandra Shah. Tr. at 11,625 (Luk). According to the State, there is nothing in the record to describe Mr. Shah's technical background, qualifications, or experience. State Findings ¶ 429. Counsel for the Staff represented that Mr. Shah conducted a literature review in which 270,000 psi was referenced for soil-cement, and he decided to use that value. Tr. at 11,629 (Turk).

E.219 The State insists that there is no support for Luk's use of a value of 270,000 psi when the actual value is known for the cement-treated soil at the PFS site. Further, the State argues that the Young's Modulus for cement-treated soil is not some obscure technical reference. State Findings ¶ 430.

E.220 According to the State, the question then arises as to the effect of misrepresenting Young's Modulus for cement-treated soil in the Luk Model. The State's expert testified that nonlinear models are extremely sensitive to input parameters and was unwilling to hazard a guess at the effect. The State points out that the issues of changes of input parameters to nonlinear modeling is at the heart of the dispute between the State and PFS in Holtec's cask stability analysis. State Findings ¶ 431.

E.221 In its challenge, however, the State's experts did not state what, if any, difference they believed that the Young's Modulus value may have on the accuracy of Dr. Luk's results. Dr. Luk testified that the higher value of Young's Modulus would conservatively maximize the seismic loads transferred from the underlying soil foundation to the storage pad and cask, and therefore, maximize the potential for horizontal cask displacement due to sliding, cask rotation, and potential tip-over. Tr. at 11,542-44 (Luk). Likewise, as discussed above, using a higher value of Young's Modulus does not affect the dynamic behavior of the cement-treated soil, because the soils have been demonstrated by sensitivity studies to be relatively insensitive to variation in the value of Young's Modulus. Thus, changes in the Young's Modulus value would not likely have significant non-linear effects in a dynamic analysis of cask stability. Tr. at 11,631-32 (Luk).

E.222 We find the State's concerns regarding the modeling of the interface, the allowance for sliding in some cases, and the value of the Young's Modulus for the cement-treated soil layer in Dr. Luk's analyses to be unfounded. Dr. Luk's analyses appropriately model the properties of each of the materials present in the PFS facility design and at the PFS site. The use of a higher Young's Modulus for the cement-treated soil is a conservative design element that addresses, <u>inter alia</u>, the State's concern about underestimating forces transferred to a cask. Likewise, the model appropriately takes into account bounding

conditions at the interfaces between these materials. It also demonstrates that sliding of the pads is beneficial to cask stability, and that even in the absence of any sliding of the pads, cask displacement and rotation remain minimal under any possible conditions at the PFS site.

F. Seismic Exemption Request

1. Background

F.1 Applicable regulations in 10 C.F.R. § 72.102(b) and 10 C.F.R. § 72.102(f), provide for the assessment of design basis seismic ground motions for ISFSIs at sites west of the Rocky Mountains based on the deterministic procedures and criteria formerly used for nuclear power plant seismic design (10 C.F.R. Part 100, Appendix A). In 1996, the Commission changed the seismic design requirements for new NPPs by issuing regulations and guidance documents that provide for use of PSHA methodology. 10 C.F.R. § 100.23; Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion, Regulatory Guide 1.165 (Mar. 1997). The Commission is considering a similar rule change to employ the use of PSHA methodology for the seismic design of ISFSIs. See Geological and Seismological Characteristics for Siting and Design of Dry Cask Independent Spent Fuel Storage Installations and Monitored Retrievable Storage Installations, 67 Fed. Reg. 47,745 (July 22, 2002).

F.2 SECY-98-126 (June 4, 1998), referenced in the State's contention, was the initial rulemaking plan for implementing the change from deterministic methods to PSHA methods for the seismic design of ISFSIs. That SECY document discussed three different rulemaking options for the Commission for incorporating PSHA methods into 10 C.F.R. Part 72. The "preferred" approach set forth in SECY-98-126 proposed a 1000-year mean return period DBE for Category 1 SSCs important to safety (those whose failure would not result in radiological doses exceeding the requirements of 10 C.F.R. § 72.104(a)) and a 10,000-year mean return period DBE for Category 2 SSCs (those whose failure would result in radiological doses exceeding the requirements of 10 C.F.R. § 72.104(a)).

F.3 This initial rulemaking plan, however, was essentially superseded by SECY-01-0178, dated September 26, 2001 in which the NRC Staff recommended to the Commission that the rulemaking plan be modified to add another option, which it identified as the "preferred" one, in lieu of the two-tiered approach identified as the preferred option in SECY-98-126. This new "preferred" option features the use of a 2000-year mean return period earthquake as the design basis for all ISFSI SSCs. In a Staff Requirements Memorandum dated November 19, 2001, the Commission approved the modification to the rulemaking plan proposed by SECY-01-0178, further instructing the NRC Staff that the proposed rule should solicit comments on a range of "exceedance levels" from 5×10^{-4} through 1×10^{-4} to which the failure probability of SSCs should be set.

F.4 On July 22, 2002, the NRC issued a proposed rule to make the Part 72 regulations compatible with the 1996 revision to Part 100 that addressed uncertainties in seismic hazard analysis. 67 Fed. Reg. at 47,745-55. The proposed rule would require a new specific license applicant for a dry cask storage facility located in either the western U.S. or in areas of known seismic activity in the eastern U.S., and not co-located with an NPP, to address uncertainties in seismic hazard analysis by using appropriate analyses, such as a PSHA or suitable sensitivity analyses, for determining the DBE. The new proposed regulation, 10 C.F.R. § 72.103, would eliminate the current requirement to comply with the deterministic methodology of Appendix A to Part 100. Id. at 47,746. As part of the proposed rule, the Commission indicated it is considering using a MAPE value in the range of 5×10^{-4} to 1×10^{-4} for ISFSI applications. Draft Regulatory Guide DG-3021, "Site Evaluations and Determination of Design Earthquake Ground Motion for Seismic Design of Independent Spent Fuel Storage Installations and Monitored Retrievable Storage Installations," has been developed to provide guidelines that are acceptable to the NRC staff for determining the DE

for an ISFSI. Id. at 47,750. The Draft Regulatory Guide currently recommends a MAPE value of 5 x 10^{-4} as an appropriate risk-informed value for the design of a dry cask storage ISFSI. Id. at 47,752.

F.5 On April 2, 1999, PFS filed an exemption request to use PSHA methods for determining the seismic design of the PFS facility using a 1000-year mean return period earthquake as the PSHA design basis. See Request for Exemption to 10 C.F.R. 72.102(f)(1) Seismic Design Requirement Docket No. 72-22/TAC NO. L22462 Private Fuel Storage Facility (PFS Exh. 247). On August 24, 1999, PFS amended its request for an exemption to seek the use of a 2000-year mean return period earthquake as the design basis for the PFS facility. See PFS Exh. 248. In its SER of October 2000, the NRC Staff approved the PFS request to use PSHA methodology for the seismic design of the PFS facility based on a 2000-year mean return period DBE. The final statement of the Staff's reasons for granting the exemption is set forth in the SER issued in March 2002. See SER at 2-50 to 2-51.

2. Legal Standards for Governing the Site-Specific Analysis Necessary to Obtain an ISFSI License

F.6 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. In

addition, 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.

F.7 Pursuant to 10 C.F.R. § 72.122(b)(1), SSCs "must be designed to accommodate the effects of, and to 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.

F.8 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."

F.9 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 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 an [NPP] need be designed to withstand the effects of surface faulting," and it identifies "[o]ther geologic and seismic factors required to be taken into account in the siting and design of [NPPs]." 10 C.F.R. Part 100, App. A. 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. Id.

F.10 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 deterministic methodology would establish the SSE for an 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. See 10 C.F.R. Part 100, App. A.

F.11 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 Part 72] 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.

3. Classification of Hazardous Curves

F.12 As discussed above, Part 72 currently requires PFS to assess the maximum vibratory ground motion that could be experienced at the PFS site using DSHA methodology. Part 72 cross references the standard that formerly applied to NPPs, i.e., 10 C.F.R. Part 100, Appendix A. Under the changes in the NPP requirement, codified at 10 C.F.R. § 100.23, an NPP applicant now refers to NRC guidance (Reg. Guide 1.165) where the "reference probability" for determining the SSE from a PSHA is specified to be that probability which has an annual median probability of 1 x 10^{-5} of exceeding the SSE, which is equivalent to a MAPE of 1 x 10^{-4} (or a return period of 10,000 years) for the Central and Eastern United States (CEUS); there is the option that an applicant may request and justify the use of a higher reference probability for a site not in the CEUS (e.g., in the western United States (WUS)). Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Reg. Guide 1.165 (Mar. 1997), at 1.165-12 (State Exh. 201); Tr. at 8001-02 (Cornell).

F.13 As described in testimony by all parties, if an NPP were to be sited at the PFS site, acceptable design levels would be established using ground motions with a mean annual return period somewhere between 5000 years and 10,000 years. Tr. at 10,111-14, 10,120-24 (Arabasz); Testimony of C. Allin Cornell [hereinafter Cornell] Post Tr. 7856, at 47-48; 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 Stamatakos/Chen/McCann] Post Tr. 8050, at 26-29.

F.14 The State challenges the Staff's claim that based on a survey of five NPPs in the WUS, the reference probability for a hypothetical NPP at the PFS site would be a mean annual exceedance probability of 2×10^{-4} (5000-year MRP). According to the State, two of the five NPPs in the survey are located in California, one is in Arizona, and two are in Washington State. State Findings ¶ 454. For the following reasons, the State does not agree with Dr. McCann's assessment and with Dr. Stamatakos' position that the average MAPE of 2×10^{-4} from the five NPPs represent a value that is applicable to the entire WUS. First, at least three of the five NPPs in the survey are located near tectonic plate boundaries along the western coast, have steep hazard curves, and are not simply representative of the Intermountain area. Second, the Palo Verde site in Arizona is in an area of low seismicity and with a mean exceedance probability corresponding to a 26,000-year return period earthquake, is not only an outlier in the calculation of the sample mean but its MAPE argues against the applicability of a 5000-year MAPE to the entire WUS. Third, the State does not believe that extrapolating an average MAPE from such a small number of NPPs to the Skull Valley site -- or to any other hypothetical NPP site in the WUS away from the plate boundary -- would withstand critical scrutiny in an NPP licensing hearing. Representing that the

sample mean characterizes "nuclear power plants in the Western United States" is defensible only semantically. State Findings ¶ 454.

F.15 The State argues that although the 5000-year MRP may justifiably apply at WUS NPP sites where there are steep hazard curves, such as near tectonic plate boundaries, it does not necessarily apply in the Intermountain west. State Findings ¶ 455. As an example, the State argues that DOE-STD-1020-2002 sets a greater probability of exceedance (i.e., a shorter return period) for sites located near tectonic plate boundaries than other DOE sites. Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities, DOE-STD-1020-2002 (Jan. 2002), at Table C-3 (Staff Exh. QQ) [hereinafter DOE-STD-1020]. For PC-4 facilities -- equivalent to NPPs -- the standard for sites located near tectonic plate boundaries is 2×10^{-4} (i.e., a 5000-year return period) whereas for non-plate tectonic sites the return period is 10,000 years. Id.

F.16 Dr. Arabasz presented a qualitative analysis of nuclear facility sites in the WUS, including the Basin and Range province, and used a literature review and the steepness of hazard curves at some of those sites to ascertain whether the implied probability of exceeding an SSE corresponded to a 5000-year MRP or a 10,000-year MRP. The State contends that Dr. Arabasz is an expert with extensive professional experience in studying and monitoring earthquakes in the Basin and Range province, and that the Board should give substantial deference to his analysis. State Findings ¶ 456.

F.17 According to the State, Dr. Arabasz first looked at the available information. State Findings ¶ 457. SECY-98-071 (Staff Exh. S) documents the grant of an exemption to the Idaho National Engineering and Environmental Laboratory (INEEL) to store Three Mile Island, Unit 2 (TMI-2) fuel, including the following, at page 2: "Based on 10 C.F.R. 100.23 requirements, as described in Regulatory Guide 1.165, 'identification and characterization of

seismic sources and determination of safe shutdown earthquake ground motion,' a future nuclear power plant in the western United States can use as a safe shutdown earthquake the 10,000-year return period mean ground motion." Tr. at 10,093-94 (Arabasz). Thus, according to the State, in the foregoing document, issued in April 1998 -- eight months after August 1997 when DOE published the Yucca Mountain Topical Report YMP/TR-003-NP in which the average MAPE for five NPPs in the WUS was reported (State Exh. 202, Table C-2) -- the Staff accepted a 10,000-year MRP as an appropriate SSE reference standard for an NPP at the INEEL ISFSI site. State Findings ¶ 457.

F.18 Next, Dr. Arabasz turned to DOE's effort to equate a design basis at Yucca Mountain to the SSE reference probability for an NPP. Even though it had calculated an average MAPE of about 2 x 10^{-4} (5000-year MRP) for five NPPs in the WUS, as reported in its Yucca Mountain Topical Report YMP/TR-003-NP, Table C-2, Dr. Arabasz argues that DOE chose not to use 5000 years but 10,000 years as the MRP for the Yucca Mountain DBE. Tr. at 10,120-10,121 (Arabasz).

F.19 For more information relevant to an appropriate SSE reference probability eastward of the plate boundary into the Intermountain area, Dr. Arabasz turned to Kennedy & Short's paper entitled, "Basis for Seismic Provisions of DOE-STD-1020" (Apr. 1994) in which they give an overview of the slopes of the seismic hazard curves and show how they vary across the country. Tr. at 10,099 (Arabasz). State Exh. 203 used a value, A_R, to describe "the ratio of ground motions corresponding to a tenfold reduction in exceedance probability." Tr. at 10,099 (Arabasz). In effect, the ratio is a measure of the increase in ground motions as the annual probability decreases. From seismic hazard curves at several nuclear sites, Kennedy & Short provide ratios for the probability intervals 1 x 10^{-5} to 1 x 10^{-4}, designated as A5/A4, and 1 x 10^{-4} to 1 x 10^{-3} (A4/A3). Tr. at 10,099-100 (Arabasz). As can be seen from State

Exh. 203, eastern sites tend to have the relatively highest ratios, high seismic sites near tectonic plate boundaries tend to have the relatively lowest values, and western sites not near tectonic plate boundaries tend to have intermediate ratios. Armed with this information, Dr. Arabasz added to the Kennedy & Short Table A-2 after determining the value of A5/A4 and A4/A3 for four of the five western sites of DOE Table C-2, State Exh. 202 (Diablo Canyon values were already determined by Kennedy & Short), and for the Yucca Mountain site. Dr. Arabasz concluded that three of the five NPP sites on DOE Table C-2 (Diablo Canyon, San Onofre, and Washington Nuclear Plant 3 near Satsop) are near tectonic plate boundaries and have low ratios of A5/A4 (steep hazard curves) of about 1.5 or less; Palo Verde and Yucca Mountain have A_R ratios more like eastern sites. Tr. at 10,105-07 (Arabasz). From comparing Kennedy and Short's Table A-2, State Exh. 203, with Table C-3 of DOE STD-1020-2002 (State Exh. 207), Dr. Arabasz concluded the following for A5/A4 ratios in the range of 1.5:

> Under the DOE framework using Table C-3, one would achieve large risk reduction ratios that would justify the use of the 5000-year P sub H value [probability of exceeding the seismic hazard]. When we have slopes of the order of 2 in A5/A4 space, for example, under western DOE sites not near tectonic plate boundaries, INEEL, Los Alamos, Hanford, the assumption is that the engineering judgment was made as part of the DOE design approach that these A5/A4 slopes did not justify the 5000-year return period motion.

Tr. at 10,108 (Arabasz).

F.20 (Utah 460) Dr. Arabasz continued his qualitative analysis of non-coastal western sites by using as a proxy for NPP information a review of the 84th percentile deterministic motions for the INEEL, PFS, Yucca Mountain, and Los Alamos sites. He observed, qualitatively, that without exception the ground motion values approach or exceed 10,000 years. Tr. at 10,109-14, 10,120-24 (Arabasz). The State argues that Dr. Arabasz's presentation credibly shows that as you move eastward from the plate boundary to Hanford,

Palo Verde, Yucca Mountain, INEEL, Los Alamos, and the PFS site, the appropriate SSE reference probability for an NPP would not appropriately be pegged at 2×10^{-4} (5000-year MRP) but rather at approximately 1×10^{-4} (10,000-year MRP). State Findings ¶ 460.

F.21 The State contends that the weight of the evidence presented in the hearing is that a technically defensible SSE for an NPP sited in the Intermountain area would have a return period of approximately 10,000 years and, therefore, the upper-end DBE benchmark for the PFS site should be a MAPE of 1×10^{-4}. State Findings ¶ 463. According to the State, a number of regulatory codes are now using a 2500-year return period as the basis for seismic design. For example, DOE-STD-1020-2002 uses a 2500-year ground motion for the design of PC-3 facilities -- those facilities similar to ISFSIs -- not near tectonic plate boundaries. Also, the International Building Code 2000 (IBC 2000) is based on seismic hazard defined in terms of Maximum Considered Earthquake ground motions associated with a 2500-year return period earthquake. State Findings ¶ 464.

F.22 Under the IBC 2000 (the building code currently in force in Utah), the design basis for certain buildings is a 2500-year return period earthquake. According to the Code, one first enters the hazard curve at 2500 years and obtains the ground motions; then one multiplies those ground motions by two-thirds. An Importance Factor is used for certain structures, such as those that contain hazardous materials; in such cases, ground motions obtained after the two-thirds reduction are multiplied by 1.5, resulting in a return to the 2500-year ground motions. Tr. at 7902-05 (Cornell).

F.23 Dr. Bartlett testified that interstate highway bridges in Utah are constructed using a 2500-year DBE. Tr. at 12,807, 12,977 (Bartlett). Such structures must survive a 2500-year event with essentially no structural damage. Id. at 12,977.

F.24 The State believes that at the low end, the DBE benchmark for the PFS site sensibly must be at least 2500 years. Currently, interstate highway bridges in Utah, certain buildings under the IBC 2000 building code, and PC-3 facilities under DOE-STD-1020-2002, all use a 2500-year DBE. The State believes that this raises a public policy concern because by allowing a 2000-year DBE for the PFS facility, it will have a lower DBE than that now required by other standards. State Findings ¶ 467. At a minimum, the State argues that setting the DBE for a nuclear facility lower than that for other non-nuclear structures or DOE PC-3 facilities poses a real public perception problem, as Dr. Arabasz explained, "[a]bsent the coterie of the cognoscenti, who can explain it." Tr. at 9208 (Arabasz). Thus, the State concludes that the evidence presented in this proceeding does not justify a 5×10^{-4} MAPE that PFS has requested and the Staff has accepted. State Findings ¶ 467.

F.25 As previously demonstrated, the State has taken issue with Staff testimony that an appropriate bench-mark for an NPP SSE at the PFS site would be a 5000-year return period earthquake, as opposed to a 10,000-year return period earthquake. The State and the Staff agree that whether a 5000- or 10,000-year earthquake for an NPP at the PFS site is the appropriate benchmark turns on whether the PFS facility is a high-seismicity site.

F.26 In this respect, as testified to by Dr. Stamatakos, the hazard curve produced by Geomatrix for the PFS site is similar to the hazard curves for many high-seismicity sites along the San Andreas Fault. Tr. at 12,753-54 (Stamatakos). From this similarity, Dr. Stamatakos concluded that if the PFS facility is not a high-seismicity site, the real hazard curves should not be as high as those produced by Geomatrix, from which it would follow that the PFS facility has been designed to a significantly higher return period than the 2000-year return period ground motions obtained from the Geomatrix PSHA hazard curves. Tr. at

12,754 (Stamatakos). If that were the case, the design basis ground motions obtained from the Geomatrix PSHA would be overly conservative.

F.27 On the other hand, Dr. Stamatakos testified that if the hazard curve produced by Geomatrix accurately reflects the conditions at the PFS facility, and the 2000-year return period earthquake has a horizontal acceleration in excess of 0.7g, then such a high ground acceleration for a 2000-year return period earthquake would, by definition, classify the PFS facility as a high-seismicity site, and it would be appropriate to use a 5000-year mean return period earthquake as the NPP SSE benchmark. Tr. at 12,754 (Stamatakos).

F.28 The State acknowledges that the Geomatrix investigators who conducted a PSHA for the PFS site, as contractors for the Applicant, are highly competent. Tr. at 9322-23 (Arabasz). Also, there is general agreement among the parties that Geomatrix conducted an adequate PSHA to depict the potential hazard at the PFS site. See, e.g., Tr. at 9119-20 (Arabasz).

F.29 A PSHA typically is an enormous undertaking involving seismic source characterization, ground motion modeling, and hazard calculations. Tr. at 9115-18, 9330 (Arabasz). As such there is a large spectrum of parameters and values to be aggregated into the process of calculating the hazard. Id. at 9878.

F.30 We do not need to resolve the dispute on whether the PFS facility is a high seismicity site such that the appropriate benchmark NNP SSE would be 5000 years. We note that, although the Staff testified to a 5000-year NPP benchmark at the hearing, the SER only concludes that, because the PFS facility's risk is lower than that of an NPP, the PFS facility may have a DBE that has a MAPE greater than 1×10^{-4}. The 2000-year DBE selected for the PFS facility design is consistent with the Staff's determination. See SER.

F.31 Further, we note that the Staff has identified what it considers to be many conservatisms in the Geomatrix PSHA. Therefore, the 2000-year DBE constitutes a conservative prediction of the seismic hazard at the PFS facility. This conservatism is above and beyond the inherent conservatisms embodied in the PFS facility design, and provides additional confidence that the 2000-year DBE for the PFS facility provides sufficient protection of public health and safety.

4. Basis for Applicant's Exemption Request

a. Use of a Risk-Informed Seismic Design

F.32 The Applicant's witness, Dr. Cornell, articulated the PFS position on the appropriateness of using a 2000-year return period earthquake for the seismic design of the PFS facility based on accepted principles of risk-informed seismic design. Dr. Cornell has extensive experience in seismic risk analysis and the development of appropriate seismic codes and standards. He has been involved in seismic PRAs and seismic margin studies for dozens of nuclear projects and is among the foremost experts in seismic risk assessment for nuclear facilities. Cornell Post Tr. 7856, at 1-6. Given Dr. Cornell's recognized expertise and the other parties' general agreement with the risk principles enunciated by Dr. Cornell in his testimony, we will first set forth those general risk-based principles, which we adopt.

F.33 The first general principle of risk-informed seismic design is that there should be a risk-graded approach to seismic safety that allows facilities and structures with lesser consequences of failure to have larger mean annual probabilities of failure than those allowed for facilities for which the consequences of failure would be more severe. In other words, under a risk-graded approach to seismic safety, the less severe the anticipated consequences of failure, the larger the probability of failure that can be tolerated. Examples

of seismic standards that explicitly incorporate a risk-graded approach are the draft International Standards Organization guidelines for offshore structures, Federal Emergency Management Agency guidelines for building assessment, and DOE Standard 1020. Cornell Post Tr. 7856, at 11-12.

F.34 Such a risk-graded approach was implemented in the Staff's approval of the PFS exemption request. The Staff concluded that, because an ISFSI like the PFS facility poses less radiological risk than an NPP, an ISFSI can be subjected to less stringent licensing requirements for seismic safety than those for an operating NPP. SER at 2-50 to 2-51. This conclusion is in accordance with the Commission's acknowledgment that the potential consequences of failure of ISFSIs are much less severe than those for NPPs, and therefore, the licensing standards for ISFSIs need not be as strict as those for operating NPPs. See Cornell Post Tr. 7856, at 12-13.

F.35 The State's expert witness, Dr. Arabasz, agreed that it is appropriate to use a risk-graded approach for the seismic analysis and design of facilities and structures. Tr. at 9122 (Arabasz). Likewise, he agreed with Dr. Cornell and the Staff that it is appropriate to allow a higher probability of seismic failure for ISFSIs, such as the PFS facility, than for NPPs since ISFSIs inherently pose less risk than an operating NPP. Tr. at 9122-24 (Arabasz). Thus, the parties are in full agreement that it is appropriate to use a risk-graded approach to seismic safety for licensing the PFS facility and that under such a risk-graded approach the PFS facility can be subject to less strict seismic safety requirements than those for an operating NPP.

b. Use of Risk Reduction Factors

F.36 The second general principle of risk-informed seismic design articulated by Dr. Cornell is that the adequacy of a DBE to provide the desired level of seismic safety is judged based on two considerations or factors, often referred to as the "two-handed approach." The first factor is the MAPE of the DBE. The second factor is the level of conservatism incorporated into the criteria and procedures for the design of the facility. Cornell Post Tr. 7856, at 11. Following DOE 1020 parlance, this second factor was referred to by PFS and the State as the risk reduction (RR) factor. See, e.g., id. at 16; Tr. at 9131-36 (Arabasz); Tr. at 12,804-05 (Bartlett).

F.37 Underlying this second general principle is the fact that the design procedures and the acceptance criteria (e.g., applicable codes and standards) for seismic design usually include conservatisms that reduce the risk of failure. These conservatisms are not explicitly identified, but are embedded in the design procedures and in the provisions of the various codes and standards pursuant to which seismic design is accomplished. Because of the conservatisms incorporated in seismic design procedures and acceptance criteria, the probability of failure of a seismically-designed facility is virtually always less than the MAPE of the governing DBE. In other words, virtually all facilities designed against a given DBE have a mean return period to failure that is longer than the mean return period of the earthquake for which they are designed. In practical terms, this means that seismically-designed SSCs are able to withstand a more severe, i.e., more infrequent, earthquake than that used as the DBE. Cornell Post Tr. 7856, at 13-15.

F.38 This second principle is of great import here, for it means that the actual probability of failure of a seismically-designed facility, such as the PFS facility, is a function of both the MAPE of the DBE and the level of conservatism incorporated in the design procedures and

the acceptance criteria for seismic design of the facility. This function can be expressed by the simple algorithm MAPE/RR. See Cornell Post Tr. 7856, at 11, 13-15.

F.39 The MAPE is the inverse of the DBE. Tr. at 9145-46 (Arabasz). For example, the MAPE of the PFS facility 2000 year DBE is 5×10^{-4}. Cornell Post Tr. 7856, at 11. Therefore, assuming that the seismic design procedures and acceptance criteria for the PFS facility achieved an RR on the order of 5, the annual probability of seismic failure for the PFS facility would be 1×10^{-4}, or 1 in 10,000. See Tr. at 9134, 9180-81 (Arabasz).

F.40 Therefore, the actual level of seismic safety achieved by the seismic design of a facility, such as the PFS facility, cannot be determined by simply looking at its DBE. Equally important, the comparative level of seismic safety of two facilities cannot be evaluated solely on the basis of their relative DBEs, unless they are also designed to the same procedures and criteria. Rather, both factors -- the MAPE of the DBE as well as the level of conservatism in the design procedures and acceptance criteria -- must be considered when comparing the seismic safety of two facilities or structures. See Cornell Post Tr. 7856, at 13-15.

F.41 For example, the annual probability of seismic failure for a facility or structure with a 2500-year return period earthquake as its DBE (with a corresponding MAPE of 4×10^{-4}) but designed to seismic codes and standards providing an RR of only 2 would be 2×10^{-4}, or 1 in 5000. Therefore, even though the DBE of such a facility would be an earthquake of higher intensity than that for the PFS facility, its annual probability of failure would be twice that for the PFS facility (assuming an RR of 5 for the PFS facility seismic design), because the underlying seismic codes and standards for such a facility would embody significantly less conservatisms than those for the PFS facility. See, Cornell Post Tr. 7856, at 51-53; Tr. at 12,961-63 (Cornell).

F.42 DOE-STD-1020-94, "Natural Phenomena Hazards Design and Evaluation Criteria for Dept. of Energy Facilities," Apr. 1994 (PFS Exh. DDD) [hereinafter DOE-STD-1020-94], is a good example of the application of a risk-graded approach toward seismic design. This standard establishes a set of "performance categories" for seismically designed SSCs with increasing consequences of failure, and thus decreasing probabilities of failure, as their performance goals. DOE-1020-94 established performance goals (reflecting increasingly severe consequences of failure) of 1×10^{-3} for PC-1 structures (designed to protect occupant safety) 5×10^{-4} for PC-2 category structures (essential facilities and buildings, such as hospitals, that should continue functioning after an earthquake with minimal interruption), and 1×10^{-4} and 1×10^{-5} for PC-3 and PC-4 category structures (which correspond to ISFSIs and NPPs respectively). The MAPE for the design basis ground motions under DOE-1020-94 were set as 2×10^{-3}, 1×10^{-3}, 5×10^{-4}, and 1×10^{-4} for PC-1, PC-2, PC-3, and PC-4 structures respectively. See DOE-STD-1020-94.

F.43 To bridge the gap between the performance goals and the DBE MAPEs, DOE 1020 standards call for design procedures and acceptance criteria that vary among the categories, ranging from those "similar to. . . model building codes" for PC-1 and PC-2, to those for PC-4 which "approach the provisions for commercial nuclear power plants." The quantitative effect of applying the conservatisms built into these various design procedures and acceptance criteria is to reduce the risk reflected in the MAPE of the design basis ground motions so that it meets the corresponding performance goals. DOE-STD-1020-94 at 2-2, C-4 to C-5.

F.44 The experts for both the Applicant (Dr. Cornell) and the State (Drs. Arabasz and Bartlett) "emphatically" agreed on the appropriateness of applying this two-factor, or two-handed, approach to evaluating the seismic safety of the PFS facility. Tr. at 9120-21, 9187-89 (Arabasz); Tr. at 12,804-05, 12,859-60, 12,878 (Bartlett); Tr. at 8012-13 (Cornell).

The NRC Staff also agreed in principle with the fact that conservatisms in the PFS facility seismic design would reduce the probability of seismic failure of the PFS facility to be less than the MAPE for the 2000-year DBE, but the Staff's approach in evaluating those conservatisms, which is challenged by the State, differed from that of PFS and the State. The disagreement between the parties centers on the application of these principles.

(i) Risk Reduction Factors at the ISFSI v. at a Nuclear Power Plant

F.45 The State argues that NPP NRC Standard Review Plans (SRPs), cited by the Applicant, do not address the seismic performance requirements of unanchored casks supported by shallowly embedded pad foundations buttressed by cement-treated soil and subject to high levels of strong ground motion. State of Utah Testimony of Dr. Steven Bartlett on Unified Contention Utah L/QQ, Part E (Lack of Design Conservatism) [hereinafter Bartlett Part E] Post Tr. 12,776, at 13. Specifically, the State contends that free-standing storage casks are not typical of SSCs found at commercial NPPs. State Findings ¶ 532. According to the State, reactor pressure vessel and primary coolant systems at NPPs are anchored and not allowed to freely slide, rotate, or uplift under seismic forces. Furthermore, the State asserts that the methods used to analyze the sliding and tipping stability of free-standing casks are not normally encountered in NPP SSC analyses. Thus, the State contends the RR ratios encompassed in SRPs for reactor pressure vessel and primary coolant systems at nuclear power plants cannot be inferred under the PFS "similarity argument" to free-standing dry storage casks. State Findings ¶ 532.

F.46 The State also believes that although not aware of the "details," Dr. Cornell testified that the NPP PRAs discussed in NUREG/CR 6728 accounted for the sliding, overturning, and bearing failures. Thus "infers" Dr. Cornell, NPP foundations have RR ratios at least in the range of five to twenty. Assuming arguendo that the NPP PRAs did demonstrate RR

ratios of between five and twenty for foundations, the State argues that Dr. Cornell has failed to demonstrate that the NPP RR ratios for foundations are applicable in this case. State Findings ¶ 532 (citing Tr. at 12,952-53 (Cornell)). According to the State, Dr. Cornell admitted that none of the NPPs evaluated were supported by cement-treated soil or soil-cement. Moreover, the State claims that when questioned, Dr. Cornell could not identify an NPP site where foundations are supported by soil-cement and relatively soft soils. State Findings ¶ 532 (citing Tr. at 7945-47, 12,968 (Cornell)).

F.47 Compared to the original deterministic standard, the 2000-year DBE results in a substantial reduction in the seismic demand against which PFS has designed its facility. Bartlett Post Tr. 12,776, at 6. Additionally, the State claims that a 2000-year DBE reduces the safety level achieved (or increases the probability of failure) when compared to a deterministic DBE. State Findings ¶ 535.

F.48 The State contends that although a factor of safety may be the same for different DBEs, the amount of actual design margin is different. According to the State's experts, a factor of safety is a function of the capacity divided by the demand. Thus, if the factor of safety is kept constant and the demand is reduced from a 10,000-year DBE to a 2000-year DBE, then the capacity is also is reduced. Although the factor of safety is the same for both earthquakes, the actual capacity -- the design margin -- is larger for the 10,000-year earthquake compared to the 2000-year earthquake. Tr. at 12,837-38 (Bartlett).

F.49 The State insists Dr. Cornell's opinion that SSCs at the PFS facility have RR ratios of "5 to 20 or greater" is inconsistent with his other testimony that the margins are 2 to 3 times the design basis capacity. State Findings ¶ 537. Thus, the State argues that the Applicant has not met its burden of demonstrating that its SSCs meet a supportable performance goal and RR factors for a 2000-year DBE. State Findings ¶ 538.

F.50 As stated, Dr. Cornell's conclusion that the RRs applicable to the SSCs important to radioactive material containment for the PFS facility are 5 to 20 or greater is based on his familiarity with the conservatisms embodied in nuclear codes and standards and evidence of actual conservatisms in the PFS facility seismic design. As we previously noted, Dr. Cornell is a recognized expert in the area of evaluating conservatisms that exist in codes and standards.

F.51 It is well established that the NRC guidelines on design acceptance criteria and procedures for NPPs set forth in the Standard Review Plan (NUREG-0800) contain many conservatisms that result in significant RR for typical NPP components. See Staff Exhs. CC to EE, 64. These conservatisms are introduced through prescribed analysis methods, specification of material strengths, and limits on inelastic behavior. However, unlike DOE-1020, the conservatism levels in the NRC acceptance criteria guidelines are not keyed to specific RRs. Nonetheless, the RRs achieved through the use of NRC guidelines for typical NPP SSCs have been found to be equal to, or higher than, the RR of 10 for PC-4 category facilities designed to DOE-STD-1020-94. Cornell Post Tr. 7856, at 19; DOE-STD-1020-94 at 2-2, C-4 to C-5.

F.52 The significant RR (of 5 to 20, or more) for typical NPP SSCs was established by seismic risk analyses performed at many NPPs. Virtually all the current NPPs in the United States were designed based on the Appendix A "deterministic" design basis ground motion approach, prior to the adoption of PSHA methodologies, and on SRP guidelines that were intentionally more conservative than, for example, corresponding building design standards. Subsequent PSHAs for these NPPs established that the Appendix A design basis ground motions had a mean return period of approximately 10,000 years. Further, numerous seismic probabilistic risk analyses (PRAs) and seismic margin studies were also

subsequently performed for SSCs at existing NPPs which established the beyond-design-basis robustness for SSCs designed to the NPP SRP. The results of these PRAs and margin studies provide the data upon which the general range of RR values of 5 to 20 or more for typical NPP SSCs designed to the NRC's SRPs is based. These conservatisms in the design of NPP SSCs enable NPPs to achieve a performance goal of about 1 x 10^{-5}. Rebuttal Testimony of C. Allin Cornell to the Testimony of State Witness Dr. Walter Arabasz on Section E of Unified Contention Utah L/QQ, [hereinafter Cornell Reb.] Post Tr. 12,951, at 4.

F.53 The NRC's SRPs for ISFSIs, NUREG-1567, and for dry cask storage systems, NUREG-1536 generally provide for use of the same codes and standards employed for NPPs under NUREG-0800. By virtue of this commonality of design procedures and acceptance criteria, similar levels of conservatisms can be expected for SSCs designed to the SRPs for ISFSIs and dry storage systems as for NPP SSCs designed to NUREG-0800. Cornell Post Tr. 7856, at 20-23. Additionally, those responsible for the PFS facility design testified that in designing the facility they generally used the same design criteria and procedures applicable to NPPs and applied the standards and codes applicable for nuclear components. Singh/Soler Post Tr. 5750, at 8-10; Ebbeson Post Tr. 6357, at 3-4; Trudeau Section D Post Tr. 6135, at 3-4; Young/Tseng Post Tr. 5529, at 11. Because SSCs at the PFS facility are designed following the same codes and standards as those for NPPs, the conclusion that the RR factors for typical SSCs designed to the NPP SRP are in the range of 5 to 20 (or greater) would apply to such SSCs at the PFS facility. Cornell Post Tr. 7856, at 23. For these reasons, we find the State's concerns to be without merit.

(ii) Fragility Curves for the SSCs

F.54 A fragility curve describes the design margin and the variability of the design margin (see Tr. at 8020 (Cornell)) as a function of the amplitude of strong ground motion (Bartlett Part E Post Tr. 12,776, at 11). The fragility curve in combination with the seismic hazard curve provides the probability of failure for the SSC for a range of ground motions. Bartlett Part E Post Tr. 12,776, at 9. Importantly, according to the State, fragility curves would provide this Licensing Board better assurance in setting and evaluating the lower DBE sought by PFS because those curves allow as precise as possible an estimate of the actual seismic design margin and its variability for the range of ground motions. State Findings ¶ 507. PFS did not develop fragility curves for the proposed PFS ISFSI. Tr. at 8003 (Cornell).

F.55 Although proffering differing importance to the development of fragility curves in this case, no party disagreed that, absent brittle behavior in the system, the Applicant could still demonstrate an acceptable probability of failure absent fragility curves if the SSCs are shown to meet the established performance goal and RR factors at the specified DBE. Thus, notwithstanding the comfort level that fragility curves would give us, we are reluctant to find that a properly justified DBE mandates fragility curves.

(iii) Risk Reduction Factors of Free-Standing Casks

F.56 The State also claims that the Applicant has not conducted a probabilistic risk assessment to evaluate any design margins or the consequences of casks tipping over. Additionally, the State insists that there are no fragility curves for the HI-STORM 100 casks at the PFS site. State Findings ¶ 512. According to the State, Dr. Cornell's opinion that the storage cask will achieve a performance goal of 1×10^{-4} is based on the cask vendor's

unanalyzed prediction of what will occur from strong ground motions generated by a 10,000-year return period earthquake; the Staff's assessment that "no sliding impact between the casks" will occur under a 10,000-year ground motion; and Holtec's tip-over analysis. State Findings ¶ 512 (citing Cornell Post Tr. 7856, at 39).

F.57 Although Dr. Cornell relies on Holtec's 10,000-year prediction of no cask tip-over, the State claims that he did not "believe" he reviewed Holtec's Beyond Design Basis Scoping Analyses, Rev. 1 (Apr. 19, 2002). Tr. at 7986 (Cornell). Moreover, the State argues that contrary to his opinion that uncertainties must be factored into estimates of safety margins, Dr. Cornell did not quantify the uncertainties in the cask vendor's nonlinear finite element cask stability analysis. Dr. Cornell claims quantification of uncertainties was not necessary because the major source of uncertainty is nonlinear behavior and Holtec performed a nonlinear analysis. State Findings ¶ 513 (citing Tr. at 7972-73 (Cornell)). The State argues Dr. Cornell's opinion -- that quantification of uncertainty in Holtec's nonlinear cask stability analysis is unnecessary because Holtec conducted a nonlinear analysis -- is too tenuous a connection to show that PFS has met its burden of showing conservatism in the facility's SSCs. State Findings ¶ 513.

F.58 With respect to potential effects of cask tip-over or drop, the State notes that oral discussions with the cask vendor were the sole basis to support Dr. Cornell's opinion, because the State contends that he could not recall reviewing Holtec's drop/tip-over analysis entitled, PFS Site Specific HI-STORM Drop/Tipover Analyses, HI-2012653, Rev. 2 (Oct. 31, 2001). State Findings ¶ 514 (citing Tr. at 7975-76 (Cornell)).

F.59 According to the State, Holtec's conclusions that the canister would not be breached are dependent upon its assumption that the angular velocity of a tipping cask is zero. State Findings ¶ 515. If the casks in fact tipped over during a seismic event, however, Dr. Cornell

opined that “[t]he initial [angular] velocity [of the tipping cask] would probably clearly have to be something greater than zero or it would not be moving in that direction.” Tr. at 7978 (Cornell). Thus, the State argues that Dr. Cornell’s testimony is inconsistent with that of the Holtec witnesses. State Findings ¶ 515.

F.60 In Dr. Cornell’s opinion, the uncertainties in the SSI analysis for the PFS site would be comparable to uncertainties for an SSI analysis at an NPP. See Tr. at 8021 (Cornell). The State contends that somewhat contradictory to the foregoing is Dr. Cornell’s admission that he is unaware of any NPP site that is supported by cement-treated soil and a layer of relatively soft soils such as at the PFS site. State Findings ¶ 517.

F.61 The State argues that the slope of the hazard curve for the PFS site may also be impacted by nonlinear soil behavior. According to the State, NUREG/CR-6728 recommends that nonlinear soil effects on the determination of the seismic scale factor be included in the development of the hazard curve slope. The NUREG/CR-6728 concept of accounting for nonlinear behavior is also applicable to any nonlinear behavior, such as a cask sliding on the pad. The State argues that PFS has not considered nonlinear effects, nor has it calculated the seismic scale factor based on considerations of the slope of the hazard curve. Bartlett Part E Post Tr. 12,776, at 12.

F.62 The HI-STORM 100 cask system is designed to the SRP for dry storage systems, NUREG-1536, including SRP-dictated accident conditions, such as hypothetical drop and tip-over events. Singh/Soler Post Tr. 5750, at 29-30. The cask and canister are not, however, “typical” NPP SSCs for which RR factors of 5 to 20 or more have been demonstrated. Therefore, some further analysis is necessary to provide confidence that the desired performance goal for the HI-STORM 100 cask system has been achieved. Both Holtec and Dr. Luk have performed beyond design basis analyses of the HI-STORM 100

cask system which demonstrate that the casks will not tip over during a beyond-design basis 10,000-year return period earthquake and that significant margins still remain against tip-over even at the 10,000 year earthquake event. These analyses demonstrate that the effective RR of the HI-STORM 100 cask system is in excess of 5, because the casks can survive both the 2000 year DBE and the beyond-design-basis 10,000 year earthquake. Accordingly, the design of the HI-STORM 100 provides RR factors comparable to those available for typical NPP SSCs. These demonstrations are in themselves sufficient to provide confidence that a performance goal on the order of 1 x 10^{-4} has been achieved. Cornell Post Tr. 7856, at 24-25, 28-29; Cornell Reb. Post Tr. 12,951, at 3-4.

F.63 Specifically, the Holtec beyond-design basis analyses showed maximum cask rotations for the 10,000-year return period earthquake event of approximately 10 to 12 degrees, still providing a factor of safety against tip-over on the order of 2 to 3, as measured against the center-of-gravity over corner location of 29.3 degrees at which the cask would tip over on its own accord. Further, many of the 10,000-year beyond design bases evaluations performed by Holtec assumed unrealistic, “worst-case” assumptions regarding soil damping and other factors. The demonstration under such worst-case assumptions that the casks would not tip over, with significant factors of safety still remaining, provides confidence that the casks would not tip over during even a 10,000-year earthquake event. Singh/Soler Post Tr. 5750, at 95; Cornell Post Tr. 7856, at 28-29; Cornell Reb. at 3-4; Tr. at 6106-08 (Soler).

F.64 This conclusion is supported by the Dr. Luk’s analyses which used sophisticated modeling techniques. The Luk cask stability analyses showed cask rotations on the order of 1 degree for a 10,000-year return period earthquake event, suggesting even larger margins of safety against tip-over than those demonstrated by Holtec. Luk/Guttman Post Tr. 6760, at 11-12.

F.65 Assuming, however, that the casks were to tip over, it has been demonstrated that no breach of the confinement barrier of the canister containing the SNF would occur. Holtec has performed a hypothetical, non-mechanistic tip-over analysis that demonstrates the decelerations at the top of the canister due to tip-over would remain within the HI-STORM 100's 45g design basis limit. Singh/Soler Post Tr. 5750, at 24. Moreover, as is typical of design basis limits, large conservatisms exist in this analysis. In the first place, the actual "g" limit for the fuel cladding in the fuel assemblies is at least 63g. Additionally, there are large margins in the design of the MPC canister system that would prevent the release of radioactive material under much larger loadings. It has been demonstrated that the canister can withstand a 25 ft. straight drop, unprotected by a cask onto a hard concrete surface, maintaining confinement when subject to forces up to 300g and maintaining significant margins against reaching the failure strain limit of the material. Singh/Soler Post Tr. 5750, at 11-12; Tr. at 12,075 (Singh). These large margins against breach of the radioactive confinement barrier provide additional confidence that a performance objective of 1×10^{-4} has been met with respect to the HI-STORM 100, since the cask will maintain containment of the radioactive matter even if it tips over in a beyond-design-basis earthquake. Cornell Post Tr. 7856, at 28-29; Cornell Reb. 12,951, at 3-4.

F.66 To reasonably assure public health and safety, the State believes that the DBE for the PFS facility must be formally linked to a specific performance goal and RR ratio. In this regard, the State claims that the Staff has not done so in that it has not established a performance goal (failure probability) for this facility or any previous ISFSIs. State Findings ¶ 506. The State, however, ignores two critical points. First, regardless of whether the Staff formally linked the 2000-year DBE to a specific performance goal and associated RR factor, the NRC's seismic design criteria and procedures contain numerous inherent conservatisms

that give effect to the NRC's defense-in-depth policy. Cornell Post Tr. 7856, at Attach. A. Further, the Staff has analytically confirmed by Dr. Luk's analysis (which shows no cask tip-over for the 10,000-year earthquake) that large safety margins exist with respect to the casks. See Seismic Analysis Report. Second, the PFS extensive analyses of the conservatisms in the PFS facility design shows that the RR factor for PFS facility SSCs is at least 5 or greater, and on that basis the PFS facility seismic design meets a performance goal of 1 x 10^{-4}. Cornell Post Tr. 7856, at 29-30. Thus, the fact that the Staff in its analysis did not formally couple a performance goal and RR ratio to the 2000-year DBE for the PFS facility is irrelevant here.

F.67 We do not agree with the State's claim that the Applicant has not met its burden of showing conservatism in SSCs at the proposed facility for the following reasons:

(1) As noted above, Dr. Cornell's reference to factoring uncertainties into estimates of safety margins was in the context of performing a formal seismic PRA, which is not necessary here.

(2) The State incorrectly asserts that Dr. Cornell concluded that quantification of uncertainties was not necessary here because the major source of uncertainty is nonlinear behavior and Holtec performed a nonlinear analysis. State Findings ¶ 513. To the contrary, Dr. Cornell testified -- apart from any such potential uncertainty -- that one could judge that the PFS design was capable of meeting a performance goal of 1 x 10^{-4} based on the results of Holtec's and Dr. Luk's cask stability evaluations for the 10,000-year earthquake "without going into more refined detail as to exactly how large that margin really is and how much uncertainty there is about it." Tr. at 8019-20 (Cornell). Thus, it was not the nature of the analyses that made quantification of

uncertainties unnecessary, but the large margins against failure predicted by the results of the analyses.

(3) The Holtec simulations for the 10,000-year earthquake use a range of bounding, worst-case, and conservative assumptions, which give more than adequate account of potential input parameter uncertainties that could affect the results. These include: (1) using upper and lower bound coefficients of friction of 0.8 and 0.2 as well as random coefficients of friction; (2) using unrealistic radiation soil damping of 1% and 5%; and (3) choosing the stiffness of the soil springs to provide resonance of the cask-pad system with amplified spectral range of earthquake input spectra. Singh/Soler Post Tr. 5750, at 62-71.

(4) Sandia's use of an entirely different methodology to model the 10,000-year earthquake, likewise showing no cask tip-over, accounts for any potential uncertainty due to the use of different modeling techniques.

(5) Even if the casks were to tip over, the record establishes that there would be no release of radioactive material. Therefore, even if alleged uncertainties in Holtec's dynamic cask stability analyses somehow resulted in cask tip-over, the public health and safety would still be adequately protected.

For these reasons, potential uncertainties in the dynamic cask stability analyses have been adequately considered and accounted for by using a wide range of input parameters and different modeling techniques as well. All these analyses show significant margins against cask tip-over still remaining, even for the 10,000-year earthquake. Further, even if the casks were to tip over, no radioactive release would occur. Accordingly, formal quantification of uncertainty, as stated by Dr. Cornell, is unnecessary and the record clearly establishes that

PFS has met its burden of showing sufficient conservatisms exist to meet a performance goal of 1 x 10^{-4}.

(iv) CTB Foundations and Storage Pads

F.68 In challenging the Applicant's analysis, Dr. Bartlett made two arguments to support his position that an RR of 5 to 20 or more typical for NPP SSCs is inapplicable to the storage pad and CTB foundations. Tr. at 12,812-17 (Bartlett). The State argues that Dr. Cornell's opinion that the CTB foundation would have a RR ratio of 5 or greater based upon Mr. Trudeau's and Mr. Ebbeson's estimate that the CTB foundation would be able to withstand 10,000-year ground motions is incorrect, because there are no engineering calculations to support the PFS supposition that its facility can withstand a 10,000-year DBE. If such were the case, the State argues PFS would not need an exemption from the deterministic ground motions requirements. It notes that Geomatrix computed the deterministic ground motions for the PFS site to be approximately 1.15g, and those same ground motions are likely for a 10,000-year DBE. State Findings ¶ 525-26 (citing SER at 2-34).

F.69 The State contends that although Dr. Cornell's opinion relies, in part, on Paul Trudeau's testimony concerning the foundation stability of the storage pad and CTB for a 2000-year DBE, he did not review Mr. Trudeau's foundation stability design calculations, including his methodology or any of his assumptions. State Findings ¶ 528 (citing Tr. at 7989-91 (Cornell)). Moreover, according to the State, neither Mr. Trudeau nor any other witness has performed any foundation stability calculations for a 10,000-year mean return period earthquake and has not shown that the foundations meet a factor of safety of 1.1 for that case. Tr. at 12,874-75 (Bartlett); id. at 6348 (Trudeau). Furthermore, the State notes that in attempting to make a point about no hazardous material release, Dr. Cornell, relying on other PFS witnesses, admitted there would be sliding of the storage pads for a 10,000-

year mean return period earthquake. State Findings ¶ 528 (citing Cornell Post Tr. 7856, at 28).

F.70 Designing a structure in conformance with the NRC's SRP design acceptance criteria is entirely different than evaluating the margins embedded in the design acceptance criteria called for by the two-handed approach endorsed by both the State's and the Applicant's experts. In designing for a 10,000 year earthquake, the Applicant would need to establish that its design meets the SRP acceptance criteria for the 10,000-year ground motions. In so doing, the Applicant would be in effect redesigning the facility so that it can withstand much higher earthquake acceleration, i.e. those from an earthquake with a return period much longer than 10,000-years. See Tr. at 12,963-66 (Cornell). The State's logic would then require the Applicant to design its facility to this much larger earthquake, creating a proverbial "catch 22" situation.

F.71 Similar to the discussion above, to require the performance of design calculations showing a factor of safety of 1.1 against sliding for the 10,000-year earthquake would be to impose the SRP design acceptance criteria for the 10,000-year earthquake which, as explained above, is contrary to the purpose of evaluating the beyond-design-basis margins to determine under what conditions failure would occur. Tr. at 12,954 (Cornell). One could do a beyond-design-basis calculation for the foundations analogous to the Holtec Beyond Design Basis Report for the casks. However, such is not necessary for the foundations. Id. at 12,954-56. PFS has quantified some of the major conservatisms that exist with respect to the storage pad and CTB foundations and has shown by this quantification that there are factors of safety inherent in the design of the foundations that would allow them to withstand loads from the 10,000-year earthquake. Id.

F.72 Specifically, for example, the factor of safety that PFS calculated for the storage pads against sliding was obtained by applying the following conservatisms:

- The calculated factor of safety of the pads against sliding of 1.27 in the east-west direction and 1.36 in the north-south direction did not take into account the passive resistance provided by the soil-cement around the pads. Taking credit for this conservatism would increase the factor of safety from 1.27 to 3.3 in the east-west direction and from 1.36 to 2.35 in the north-south direction without taking other conservatisms into account. Trudeau Section D Post Tr. 6135, at 9-10.

- In addition, the calculation for sliding is based upon the static shear strength of the underlying clay silt soils. Trudeau Soils Reb. Post Tr. 11,954, at 2-5. It is undisputed that the underlying clayey silt soils will exhibit greater strength under the dynamic loadings experienced under an earthquake of at least 30% and potentially up to 100%. Tr. at 11,967-68 (Trudeau); id. at 12,976-77 (Bartlett); Bartlett Soils Surrebutal Post Tr. 11,982, at 2-3. Assuming a 50% increase in strength would increase the factor of safety for the east-west base case from 1.27 to 1.9, again without taking other conservatisms into account. Trudeau Soils Reb. Post Tr. 11,954, at 3.

- PFS computed the minimum 1.27 and 1.36 factors of safety using the lower-bound, worst-case static shear strength for the entire pad storage area. Tr. at 11,960-62, 11,966 (Trudeau). Further, this lower-bound strength was obtained from the weakest layer of soil underlying the pads whereas the pads will be resting in most cases on the soils above this layer which are much stronger than the weakest layer for which the lower bound shear strength was determined. Trudeau Soils Reb. Post Tr. 11,954, at 3.

- Any measurement of the strength of soils will disturb the soils and result in soil strength values that are less than the actual strength that the soils will exhibit in place. Therefore, when the measured value of strength is used in the factor of safety computations, there is a "built-in" conservatism because the actual strength of the soil in place will be higher. Trudeau Soils Reb. Post Tr. 11,954, at 4-5.

- The minimum factor of safety is applicable only when the earthquake reaches its peak magnitude. At all other times, there is considerably more margin available. Trudeau Soils Reb. Post Tr. 11,954, at 2.

- Further, due to the cyclic nature of the seismic loading each of the peak accelerations that impart dynamic loads from the earthquake exist for only one very brief moment of time -- typically less than 0.005 seconds -- and then the seismic loading reverses direction, which minimizes any sliding displacement that would occur. Trudeau Section D Post Tr. 6135, at 4.

F.73 There is similarly a large margin against pad failure due to the loss of soil bearing capacity. The minimum factor of safety of 1.17 against bearing capacity failure for the storage pads was computed using the extremely conservative assumption that 100% of the earthquake loads act in both horizontal directions at the same time. Trudeau Section D Post Tr. 6135, at 11-12. If the load combinations allowed by ASCE 4-86 were used instead, the factor of safety against loss of bearing capacity would be increased to 2.1.

F.74 Another major conservatism in the computation of the factor of safety against loss of bearing capacity is the use of the lower bound static shear strength of the weakest layer of soil underlying the pads. Standard practice for computing bearing capacity is to average the contributions of all soil layers over a depth equal to the shortest dimension of the foundation, in this case the 30 ft. width of the pads. Approximately 2/3 of this depth below the pads

would have soils or cement-treated soils that would be much stronger than the weakest layer of soil from which the lower bound static strength was measured. Using the average strength of the cement-treated soil and soil for the 30 ft. below the pad and the soil's dynamic strength rather than its static strength would have significantly increased the factor of safety against loss of bearing capacity failure. Trudeau Soils Reb. Post Tr. 11,954, at 4. Also, as noted with respect to pad sliding, the laboratory measured strength of the soils would be less than their in situ strength and the maximum earthquake magnitude to which the pads would be subject would be cyclic and of very short duration. Trudeau Soils Reb. Post Tr. 11,724, at 2-5; Trudeau Section D Post Tr. 6135, at 4.

F.75 Taking into account just two of the above many conservatisms (use of the load combinations allowed by ASCE 4-86 and the dynamic strength of the clayey soils) would increase the factor of safety for the pads against loss of bearing capacity to 3.63, which would provide a factor of safety of 1.0 against loss of bearing capacity for vertical and horizontal earthquake accelerations of 1.24g and 1.27g respectively, essentially the same as the 10,000 year earthquake accelerations for the PFS site. Thus, the bearing capacity analysis performed by PFS for the 2000-year return period earthquake is adequately conservative. It provides ample margin to conclude that a RR factor of more than 5 applies with respect to the pads' capability to withstand a loss of bearing capacity. Cornell Post Tr. 7856, at 28.

F.76 The factor of safety against pad overturning is 5.6, without taking into account any conservatism. Trudeau Section D Post Tr. 6135, at 12. Thus, the margins against pad overturning are also sufficient to conclude that a RR factor of more than 5 applies with respect to pad overturning. Cornell Post Tr. 7856, at 28.

F.77 There are also numerous conservatisms included in the CTB foundation design. For example, removing some of the conservatisms in the analysis results in a factor of safety against loss of bearing capacity of the CTB on the order of 10, and the 2000-year return period earthquake accelerations would have to increase by a factor of more than 4 to reduce this factor of safety to 1.0. Trudeau Section D Post Tr. 6135, at 6-8. Similarly, the CTB would not overturn during a 10,000-year earthquake event. Ebbeson Post Tr. 6357, at 8. Therefore, the RR factors applicable to these foundation failure modes would be of 5 or more. Cornell Post Tr. 7856, at 27-28.

F.78 It is not necessary to do a formal 10,000-year return period earthquake evaluation to show a lack of SSC failure in the event of a 10,000 year earthquake. One can determine, as reflected by the discussion above, that sufficient conservatisms exist in the design of the SSCs and their foundations to meet the increase in loadings due to the higher ground accelerations for the 10,000-year event. Indeed, if anything, the demands placed on foundations would be proportionally less for higher earthquake levels due to the higher damping that would be associated with the higher strain levels in the soil for the 10,000-year event so that such an approach would be both appropriate and conservative. Tr. at 12,954-56 (Cornell); Ebbeson Post Tr. 6357, at 9.

F.79 Therefore, RR factors of 5 or more are appropriate for foundation failures associated with overturning, loss of bearing capacity, and sliding of the storage pads. Moreover, foundation failure of the pads would not by itself constitute ultimate failure of the PFS facility resulting in radioactive release, but would be part of a chain of events that one would need to analyze to determine whether the ultimate performance goal had been met. Tr. at 12,802-03 (Bartlett). In this respect, the record shows that the foundation failure mechanism of the pads of most concern to the State -- sliding of the storage pads -- would in fact reduce the

loads transferred to the storage cask on the pad and reduce the likelihood of cask tip-over. Singh/Soler Post Tr. 5750, at 43. Similarly, the RR factors for turnover and loss of bearing capacity of the CTB would be 5 or more, and any potential sliding of the CTB that might occur for a 10,000-year event would result in no adverse health of safety impact.

(v) Time Transfer Estimates

F.80 The State challenges Dr. Cornell's determination of the appropriate RR applicable for the CTB and the seismic struts and cranes inside the CTB, because it disagrees with "the time in which the canister is potentially exposed and SSCs are in use during transfer at the PFS site" and the validity of the specific CTB conservatisms set out in Mr. Ebbeson's testimony relied upon by Dr. Cornell. State Findings ¶¶ 522-24. These challenges, however, ignore the primary rationale for Dr. Cornell's conclusion on the RR applicable to the CTB. Dr. Cornell testified:

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

Cornell Post Tr. 7856, at 26. None of the State's experts took issue with the appropriateness of using a RR of 5 to 20 or more for the CTB and the cranes and struts therein. See Tr. at 9132 (Arabasz); Tr. at 12,786, 12,814 (Bartlett).

F.81 The State also challenges the use of a RR of 5 to 20 or more for the CTB and the cranes and struts therein, focusing on the two ancillary supporting reasons for the determination. The State claims there are serious shortcomings in the Applicant's estimation of the time during which the canister is potentially exposed and the SSCs are in use during the transfer operations between the storage and transportation casks. State Findings

¶¶ 522, 524. As discussed above, the time durations estimated by PFS for various aspects of the transfer operation are reasonable. In addition, and most importantly, the reduction in risk obtained by virtue of the intermittent use of the crane and seismic struts is an additional reduction of the risk above and beyond the RR ratio of 5 to 20 or more that otherwise would apply to these SSCs. Cornell Post Tr. 7856, at 26-27. Therefore, any shortcomings in the PFS estimations of percentage use of SSCs in the CTB would be immaterial. Even if the SSCs were assumed to be constantly in use, their applicable RR ratio would still be 5 to 20 or more.

5. NRC Staff's Justification for Granting Exemption

F.82 To support its evaluation of the PFS exemption request, the Staff requested that the Center for Nuclear Waste and Regulatory Analysis (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: (1) characterization of potential seismic sources, (2) estimation of ground motion attenuation, (3) assessment of probabilistic and deterministic ground motion hazards, (4) assessment of probabilistic surface faulting hazards, and

(5) development of design basis ground motions in compliance with applicable regulations and regulatory guidance. Stamatakos/Chen/McCann Post Tr. 8050, at 9-10.

F.83 Based on the review of the PSHA conducted by Geomatrix, 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.
Stamatakos/Chen/McCann Post Tr. 8050, at 10.

F.84 Following issuance of the CNWRA report, 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} MAPE) 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 SER (see SER §§ 2.1.6.1 and 2.1.6.2) and in the CNWRA report.
Stamatakos/Chen/McCann Post Tr. 8050, at 10.

F.85 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. Stamatakos/Chen/McCann Post Tr. 8050, at 11.

F.86 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 approve this approach in determining the SSE for an NPP, as set forth in 10 C.F.R. § 100.23 and Regulatory Guide 1.165. 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. <u>See</u> 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 INEEL. Stamatakos/Chen/McCann Post Tr. 8050, at 11.

F.87 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. Stamatakos/Chen/McCann Post Tr. 8050, at 11-12.

F.88 As set forth in Section 2.1.6.2 of the Staff's SER, the Staff concluded that the use of the PSHA methodology and a MAPE of 5×10^{-4} (2000-year return period) are acceptable bases to determine the seismic design ground motions of the proposed PFS facility. SER at 2-50 to 2-51. 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) the Staff's 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 Regulatory 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. Stamatakos/Chen/McCann Post Tr. 8050, at 12-13.

F.89 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 SER, 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. SER at 2-48; Stamatakos/Chen/McCann Post Tr. 8050, at 13.

F.90 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, 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 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 CNWRA report) 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. Stamatakos/Chen/McCann Post Tr. 8050, at 13.

F.91 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 SER, these relatively low slip tendency results indicate that the seismic

source characterization of the PSHA study conducted by Geomatrix is conservative. SER at 2-38 to 2-40; Stamatakos/Chen/McCann Post Tr. 8050, at 13-14.[44]

F.92 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 an azimuth of 355 degrees, and a horizontal minimum principal stress (σ_3) with an azimuth of 85 degrees. 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 and optimization of slip tendency values for segments of faults such as the Wasatch that are known to be produce earthquakes. The Staff's slip tendency analysis assumed a normal-faulting regime, with rock density equal to 2.7g/cc, fault dip equal to 60 degrees, water table at a depth of 40 m, and a hydrostatic fluid pressure gradient. Stamatakos/Chen/McCann Post Tr. 8050, at 14.

F.93 The results of the Staff's slip tendency analysis indicate that fault segments with approximately north-south strikes (azimuth = 175 degrees) are optimally oriented for future fault slip. Faults with north, northeast-south, and 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

[44] 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.

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. Stamatakos/Chen/McCann Post Tr. 8050, at 15.

F.94 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, the Staff found this assumption by Geomatrix to be 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. Stamatakos/Chen/McCann Post Tr. 8050, at 15.

F.95 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 the proposed PFS site. In its SAR, the Applicant first developed conceptual models of the physical dimensions of fault rupture -- either rupture area or trace length of surface fault rupture -- based on the geologic record 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. Stamatakos/Chen/McCann Post Tr. 8050, at 15-16.

F.96 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 (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. Stamatakos/Chen/McCann Post Tr. 8050, at 16.

F.97 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 PGA 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 United States Geological Survey (USGS) National Earthquake Hazard Reduction Program (NEHRP) data. This was graphically and clearly shown in Staff Exh. JJ entitled "Comparison of Western U.S. Hazard Curves," prepared by Dr. Stamatakos, in a

highly illustrative scientific notebook entry. Stamatakos/Chen/McCann Post Tr. 8050, at 16-17; Staff Exh. JJ, at 5.

F.98 Similarly, the Staff observed that the 2000-year horizontal PGA for Skull Valley (soil hazard) as estimated by the Applicant, is actually higher than the 2500-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 0.56g and 0.686g, based on a MAPE of 4×10^{-4} (2500-year return period) -- as compared to the Applicant's estimated horizontal PGA of 0.711g, based on a MAPE of 5×10^{-4} (2000-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. In sum, the Staff concluded that because the Applicant's estimate of the seismic hazard is conservative, the proposed ground motions based on the MAPE of 5×10^{-4} (2000-year return period) provides an additional margin of safety in the seismic design. Stamatakos/Chen/McCann Post Tr. 8050, at 17.

F.99 As further stated in the SER, pages 2-48 to 2-49, the Staff found that the Applicant's exemption request was acceptable in that:

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

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

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

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

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

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

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

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

Stamatakos/Chen/McCann Post Tr. 8050, at 17-18.

F.100 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 MAPE of 5×10^{-4} (2000-year return period), and that this provides an acceptable basis for the seismic design of the proposed PFS facility. Stamatakos/Chen/McCann Post Tr. 8050, at 18.

F.101 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 SER on pages 2-49 to 2-51. Stamatakos/Chen/McCann Post Tr. 8050, at 18-19.

F.102 First, as stated in SECY-98-071, the radiological hazard posed by a dry cask storage facility is inherently lower than operating commercial NPP. 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 at 2 (Staff Exh. S). As further stated therein, "the design earthquake for cask and canister technology need not be as high as a nuclear power plant safe shutdown earthquake." Id. (citing 45 Fed. Reg. 74,693 (Nov. 12, 1980)).

F.103 Second, as set forth in the SER, the seismic design for commercial NPPs is based on a determination of the SSE ground motion. SER at 2-50. Previously, this ground motion has been estimated using a deterministic approach in the initial licensing of an 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} MAPE (approximately equivalent to a 100,000-year return period). As the Staff explained, this Reference Probability, which is defined in terms of the median probability of exceedance, corresponds to a MAPE of 1×10^{-4}. That is, the same design ground motion that has a median reference probability of 1×10^{-5}, has a MAPE of 1×10^{-4}. Stamatakos/Chen/McCann Post 8050, at 19.

F.104 Further, analyses of SSEs at NPPs in the WUS (where the proposed PFS facility would be sited), show that the average mean annual probability of exceeding the safe shutdown earthquake is 2.0×10^{-4} -- which is equivalent to an SSE with a 5000-year return period. This is demonstrated in a DOE publication entitled, "Preclosure Seismic Design

Methodology for a Geologic Repository at Yucca Mountain," TR-003-NP, Rev. 2 (Aug. 1997). State Exh. 202 at C-18; Stamatakos/Chen/McCann Post Tr. 8051, at 19-20.[45]

F.105 Based on the foregoing considerations, the Staff determined that the MAPE 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 MAPE of 5×10^{-4} (2000-year return period) as a basis to determine the seismic design ground motions appropriately may be used for the proposed PFS facility. Stamatakos/Chen/McCann Post Tr. 8051, at 20.

F.106 (Staff 6.80) Finally, in addition to the above considerations, the 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} (2000-year return period) were found to be appropriate. These were (a) the Department of Energy's issuance of DOE-STD-1020-94, and (b) the Commission's 1998 approval of a 5×10^{-4} MAPE (2000-year return period) for seismic design ground motions at the TMI-2 ISFSI at INEEL, described in SECY-98-071. Stamatakos/Chen/McCann Post Tr. 8051, at 19-20.

F.107 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

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[45] Specifically, the MAPE for SSEs at the five WUS NPPs listed in the DOE TR-003 report, were reported to be as follows: Diablo Canyon - 1.7×10^{-4}/year (5882-year return period); Palo Verde - 3.8×10^{-5}/year (26,316-year return period); San Onofre - 3.0×10^{-4}/year (3,333-year return period); Washington Nuclear Plant No. 2 - 2.8×10^{-4}/year (3571-year return period); Washington Nuclear Plant No. 3 - 2.2×10^{-4}/year (4545-year return period). State Exh. 202; Tr. 8031, 8327-28.

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 PC-3 SSCs (which are analogous to SSCs at a dry spent fuel storage facility) be designed for ground motions that have a MAPE of 5×10^{-4} (2000-year return period). Stamatakos/ Chen/McCann Post Tr. 8050, at 20-21.

F.108 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 MAPE of 5×10^{-4} (2000-year return period) as the basis for establishing the seismic design ground motions for the TMI-2 ISFSI (designed to passively store SNF debris in dry storage casks), which is discussed in SECY-98-071 and CNWRA-98-007 (Chen and Chowdhury, 1998). In this 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.

F.109 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 MAPE of 5×10^{-4} (2000-year return period) is appropriate for the proposed PFS facility. Stamatakos/Chen/McCann Post Tr. 8051, at 21.

a. DOE Standard

F.110 Contrary to the concerns raised by the State during the hearing, the Staff did not "adopt" DOE STD-1020-94 in approving the 2000-year return period for use in the design of the proposed PFS facility. Rather, the Staff cited the DOE Standard as a reference point, in that it established a mean reference probability (corresponding to a 2000-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 PFS facility, nor did it find any reason to require an NRC license applicant (here, PFS) to justify its seismic exemption request on the type of analysis that DOE might conduct under the DOE Standard, in order to meet all the specified DOE requirements. Stamatakos/Chen/McCann Post Tr. 8051, at 30.

F.111 As the testimony of the Staff's experts explained, the underlying philosophy of DOE-STD-1020-94 is to use a risk-graded approach in establishing the seismic (or other) hazard's MAPE, 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 2000-year return period for the proposed PFS facility also relies on considerations of risk. Thus, as discussed above, the Staff considered (1) 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; (2) the Commission's previous approval of the seismic design ground motion with a 2000-year return period for the TMI-2 ISFSI, which included a quantitative risk assessment; and (3) 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 an NPP). Stamatakos/Chen/McCann Post Tr. 8051, 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.

Stamatakos/Chen/McCann Post Tr. 8051, at 31 (citing SECY-98-071). Thus, contrary to the State's argument, considerations of radiological risk did enter into the Staff's determination to approve the use of a seismic design ground motion with a 2000-year return period, as derived from the Applicant's PSHA for the proposed PFS facility. See Stamatakos/Chen/McCann Post Tr. 8051, at 31.

F.112 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 MAPE of 5×10^{-4} (2000-year return period) to 4×10^{-4} (2500-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 USGS NEHRP maps and thereby result in analytical descriptions of seismic hazards that can be more readily used in conjunction with the USGS NEHRP maps. Stamatakos/Chen/McCann Post Tr. 8051, at 31-32.

F.113 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} (2000-year return period) to 4×10^{-4} (2500-year return period), is inconsequential. This revision results in a small change in the MAPE 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.

F.114 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 2500-year return period ground motion in place of the previous 2000-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 (SF) from of 1.0 to 0.9, thus effectively leaving the seismic design standard for PC-3 facilities unchanged. <u>See</u> DOE-STD-1020-94 at C-10 to C-11. 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 2000-year return period established therein, inasmuch as the design standard has effectively remained the same in DOE-STD-1020-2002 with its use of a nominal 2500-year return period and a 0.9 SF factor.

b. INEEL Exemption for TMI

F.115 The State argues that the Staff also relies on the grant to DOE-INEEL of an exemption from 10 C.F.R. § 72.104(f) for storage in an ISFSI of rubblized fuel debris from the TMI-2 ISFSI to support its decision to grant the Applicant's exemption request.

According to the State, the facts and site conditions at INEEL are different from those at PFS. State Findings ¶ 476. For example the State notes that INEEL is located on federal reservation of vast size -- approximately 800-900 square miles -- and the nearest resident is approximately 50 miles from the site. State Findings ¶ 476 (citing Tr. at 8185, 8187-88 (Chen)). At INEEL, the TMI-2 ISFSI is located on the Idaho chemical processing plant (IPCC) site. The State also points out that ground motions at the IPCC site are 0.30g for a 2000-year MRP and 0.47g for a 10,000-year MRP. The IPCC -- a higher risk facility than the TMI-2 ISFSI -- was designed to peak horizontal accelerations of 0.36g. State Findings ¶ 476; Seismic Ground Motion at Three Mile Island Unit 2 [ISFSI] site in [INEEL] - Final Report (June 1998) at 4-1 (State Exh. 127). Further, the State contends that the TMI-2 ISFSI was also designed to 0.36g horizontal design value which means its ground motions fall somewhere between a 3000- to 4000-year MRP. See Arabasz Post Tr. 9098, at 12. Fuel at INEEL is stored in thirty horizontal concrete modules that under earthquake conditions are not expected to slide. Tr. at 8186-87 (Chen).

F.116 In contrast to the INEEL site, the State contends that the proposed PFS facility is located within 2 miles of the nearest resident, and the land to the north of the site is contiguous with privately owned land. State Findings ¶ 477. Furthermore, the State argues that the Board cannot rule out that someday the land to the north of the PFS site could be developed for residential uses. PFS intends to store 4000 casks containing spent fuel from commercial NPPs, and the design values at PFS are those for a 2000-year MRP. Further, according to the State, PFS uses an unconventional design in which PFS and the Staff consider sliding of the casks and the pads under earthquake conditions to be beneficial because sliding dissipates seismic energy that the casks and foundations would otherwise have to resist. State Findings ¶ 477.

F.117 In responding to the State's claims, Dr. Chen -- the Staff expert 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 (1) "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 (2) although the TMI-2 ISFSI had been designed to a slightly higher standard than the 2000-year return period (5×10^{-4} MAPE) ground motion, the Commission in fact approved the lower 2000-year ground motion as the acceptable seismic design basis for the facility. Stamatakos/Chen/McCann, Post Tr. 8050, at 32-33 (citing SECY-98-071 (Staff Exh. S)).

F.118 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 MAPE of 5×10^{-4} (2000-year return period), with an associated peak horizontal acceleration of 0.30g, as an acceptable design basis for the facility. Thus, SECY-98-071 states, "[g]iven the absence of radiological consequences from any credible seismic event, the Staff finds that the DOE Standard 1020 risk-graded approach of using the 2000-year mean return period ground motion as the DE (design earthquake) is adequately conservative." Stamatakos/Chen/McCann, Post Tr. 8050, at 33 (citing SECY-98-071).

F.119 The Staff also observed that the TMI-2 ISFSI exemption is also pertinent insofar as the Staff's (and the Commission's) approval of a 2000-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. Thus, the TMI-2 ISFSI example also provides a useful analogy, with regard to the issue of relative radiological consequences. Stamatakos/Chen/McCann, Post Tr. 8050, at 33-34.

F.120 While the INEEL ISFSI was designed to a higher ground motion than that for the 2000-year MRP earthquake, the exemption was approved on the basis of the adequacy of the 2000-year MRP earthquake. The safety evaluation for the exemption expressly states that the "DOE Standard 1020 risk graded approach of using the 2000-year return period mean ground motion as the DE is adequately conservative," and concludes that the design earthquake of 0.36g for the INEEL ISFSI is acceptable because it exceeded the 0.30g value for the 2000-year MRP. SECY-98-071 (Staff Exh. S), Att. Final Evaluation of Exemption Request to 10 C.F.R. 72.102(f)(1) Seismic Design Requirement, at 3 (attached to May 28, 1998 letter from NRC to INEEL). The design of the INEEL ISFSI to a higher level than a 2000-year DBE does not change the DBE standard upheld by the NRC there.

c. Geomatrix Probabalistic Seismic Analysis

F.121 There is general agreement among the parties that Geomatrix conducted an adequate PSHA to depict the potential hazard at the PFS site. The Staff, however, goes on to take the view that Geomatrix produced a "conservative" PSHA. Stamatakos/Chen/McCann Post Tr. 8050, at 13-17; Tr. at 8220-21 (Stamatakos). The State believes that the Staff's reliance on the conservative nature of Geomatrix's PSHA to support a grant of a 2000-year MRP to PFS (SER at 2-38 to 2-39) and its assertion that the Applicant's conservative

estimate of hazard provides an additional margin of safety in the seismic design (Stamatakos/Chen/ McCann Post Tr. 8050, at 17) are founded on erroneous premises, questionable speculation about what the relative PSHA outcome should have been, and one-party analyses subject to scientific challenge. State Findings ¶ 479.

F.122 In that regard, the State notes that the Staff did not conduct its own PSHA, but instead, it chiefly reviewed the geological and seismological inputs to Geomatrix's PSHA, evaluated Geomatrix's probabilistic and deterministic hazard results, and performed some independent analysis, notably slip tendency. State Findings ¶ 481 (citing Tr. at 8090-91 (Stamatakos); Stamatakos/Chen/McCann Post Tr. 8050, at 12-18. According to the State, in order to buttress its claim that the Geomatrix PSHA is conservative, the Staff uses the slip tendency analysis conducted by Dr. Stamatakos and his colleagues at Southwest Research Institute and also makes comparisons to PSHA results for sites in and around Salt Lake City. State Findings ¶ 481. The State argues, however, that scrutiny of the Staff's analysis and its PSHA comparisons does not substantiate the Staff's claim that Geomatrix's PSHA results are conservative. State Findings ¶ 481.

F.123 Slip tendency analysis is a modeling technique designed to assess stress states and potential fault activity. The State insists that as used by the Staff, i.e., for the purpose of assessing potential fault activity, the analysis requires as a starting point a specification of the orientation and relative magnitudes of stresses acting on the local geology of Skull Valley. State Findings ¶ 482. As the Staff explains in its prefiled testimony:

> In slip tendency analysis, the underlying assumption is that the regional stress state controls slip tendency and that there are no significant deviations due to local perturbations of the stress conditions. This assumption is supported by a similar slip tendency analysis of the Wasatch fault, which shows the highest slip tendency values for the segments of the fault considered to be most active (Machette et al., 1991) . . . . [The] orientation for the principal stresses was based on recent global positioning satellite information (Martinez et al., 1998a).

Stamatakos/Chen/McCann Post Tr. 8050, at 14. According to the State, because the stress state at Skull Valley is unknown, the Staff had to assume the applicability of regional stress information from elsewhere. State Findings ¶ 482. The Staff reported that it used a horizontal minimum principal stress with an azimuth of 85 degrees, citing Martinez et al., 1998 (State Exh. 184). Stamatakos/Chen/McCann Post Tr. 8050, at 14. The State contends that the cited Martinez paper (State Exh. 184) does not contain this value; rather, the Staff arrived at this value by subjectively "tuning" the regional stress field in the Wasatch Front area to get maximum slip tendency on parts of faults with known paleoseismic (prehistoric) slip like the Wasatch Fault. State Findings ¶ 482.

F.124 Based on the results of its tuned slip tendency analysis, the Staff argues that the East fault has a relatively low slip tendency value and is therefore less likely to slip than faults or fault segments further from the site. Stamatakos/Chen/McCann Post Tr. 8050, at 15. The State claims that this conclusion ignores the Staff's acknowledgment of Geomatrix's finding that, "[i]n all the alternative models and because of the evidence for surface rupture of late Quaternary deposits, the East fault is considered seismogenic and assigned a probability of activity of 1." State Findings ¶ 483 (quoting NWRA Report at 2-17). Based on offsets of those late Quaternary deposits in the immediate vicinity of the PFS site, Geomatrix assessed for the East fault a most likely slip rate of 0.2 mm per year -- the same order of magnitude as the most likely slip rate of 0.4 mm per year for the Stansbury Fault. See State Exh. 185 at Table 6-2. The State argues that the evidence of surface rupture of late Quaternary deposits by the East fault is a far more cogent indicator of the fault's seismogenic potential and of the local stress conditions near the PFS site in Skull Valley than what the Staff guesses them to be from its hypothetical, subjectively tuned modeling. Thus, the State contends that the Staff's interpretation of the stress state in Skull Valley would be one competing opinion in a

PSHA, subject to challenge by other experts. State Findings ¶ 483. Further, the State believes that corresponding inferences the Staff makes from the slip tendency analysis about conservatism in Geomatrix's assessed site-to-source distances and maximum magnitudes (Stamatakos/Chen/McCann Post Tr. 8050, at 15-16) are also arguable and not established conclusions. State Findings ¶ 483.

F.125 Another major line of reasoning the Staff uses to conclude the Geomatrix PSHA is conservative is a comparison to PSHA results in and around Salt Lake City, which leads the Staff to claim that Geomatrix's PSHA may have led to an "overly conservative" hazard result by as much as 50% or more. Stamatakos/Chen/McCann Post Tr. 8050, at 13, 16-17. The State contends that an erroneous premise pervading these comparisons by the Staff is that "fault sources near Salt Lake City are larger and more seismically active than fault sources near the PFS site." State Findings ¶ 485 (quoting Stamatakos/Chen/McCann Post Tr. 8050, at 16).

F.126 In particular, the State attacks the two basic comparisons it claims that the Staff made between Geomatrix's PSHA results and the counterpart hazard results for sites in or near Salt Lake City. First, the State claims from the comparison the Staff concludes that, "[t]he results of the Applicant's PSHA for Skull Valley (Geomatrix Consultants, Inc., 2001a) suggest that it is 1.5 times more likely that a ground motion of 0.5g horizontal peak ground acceleration or greater will be exceeded at the PFS site (assuming hard rock site conditions), than at Salt Lake City, based on the USGS National Earthquake Hazard Reduction Program (Frankel et al., 1997)." State Findings ¶ 487 (quoting Stamatakos/Chen/McCann Post Tr. 8050, at 16). State expert, Dr. Arabasz however, argues in his testimony that there are significant shortcomings in this comparison by the Staff. Tr. at 9864-65 (Arabasz). According to the State, the following facts are relevant to the comparison. <u>See</u> State

Findings ¶ 487. The exact location of the Salt Lake City PSHA calculation is uncertain. See Tr. at 8215-16 (Stamatakos). The hazard calculation for Salt Lake City is based on the USGS NEHRP, whose hazard calculations would not be acceptable for the SAR at the PFS site. See Tr. at 8109-11 (Stamatakos). According to the State, the reason for the latter is that the national hazard mapping is done on a regional scale and includes only major active faults. State Findings ¶ 487. According to the State, Dr. Stamatakos did not know "everything the GS did in [the Salt Lake City] analysis," but presumed that "the Wasatch fault probably controls a lot of what is in that hazard." Id. (quoting Tr. at 8110-11 (Stamatakos)). In the Geomatrix site-specific PSHA for the PFS site, the East fault is only 0.9 km from the CTB, has a mean maximum magnitude of 6.5, and is a major contributor together with the Stansbury and East Cedar Mountain Faults to the total mean hazard; all three faults are within 9 km of the PFS site. SER at 2-47. Given the slip rates of 0.4 mm/yr, 0.2 mm/yr, and 0.07 mm/yr for the Stansbury, East, and East Cedar Mountain Faults, respectively (State Exh. 185 at Table 6-2), the State contends that there is a combined slip rate of 0.67 mm/yr contributing to the annual earthquake activity rate, which is 74% of the Wasatch Fault's slip rate of 1.1 mm/yr. Slip rates, maximum magnitudes, distances, and near-source effects are all part of the complex interplay of parameters in the Geomatrix PSHA. The site-specific Geomatrix PSHA hazard results at the PFS site (for rock site conditions) are an integrated outcome of the seismic source characterization, just as the USGS's regional PSHS hazard results are at Salt Lake City (also for rock site conditions). The State argues that the Staff's claimed conservatism cannot be evaluated by comparing the two bottom lines. Thus, the State believes that without independently performing site-specific PSHAs for the two sites, the Staff's inference that the Geomatrix PSHA is conservative by comparison to sites in or near Salt Lake City is only speculation. State Findings ¶ 487.

F.127 The second comparison made by the Staff that the State attacks relates to hazard calculations at nine sites in the I-15 corridor in the Salt Lake Valley. According to the State, the Staff observes that Geomatrix's 2000-year horizontal PGA (soil hazard) is actually higher than the 2500-year ground motions (also on soil) at the I-15 sites. State Findings ¶ 488 (citing Stamatakos/Chen/McCann Post Tr. 8050, at 17). The State also claims that the Staff explicitly reviewed Geomatrix's revised ground motion modeling in 2001, which involved development of a detailed shear-wave soil profile to calculate site response, and noted: "This change in the shear-wave profile and site response model led to a significant increase in estimated ground motions at the PFS site." State Findings ¶ 488 (quoting SER at 2-41). The State contends that in fact, the 2000-year peak horizontal ground motion increased 35% from 0.528g to 0.711g. State Findings ¶ 488 (quoting SER at 2-41).

F.128 The State also argues that Dr. Stamatakos' bottom-line position is that either Geomatrix provided a very conservative seismic hazard curve or, if the hazard results are accurate, the PFS site deserves to be treated as a tectonic plate boundary site, which would justify a higher reference exceedance probability (lower MRP). State Findings ¶ 491 (citing Tr. at 12,753-54, 12,763 (Stamatakos)). The State believes this to be a false dilemma. State Findings ¶ 491.

F.129 The State concludes that the Applicant's PSHA is adequate. The State argues, however, that the Staff has not sufficiently provided evidence to support its claim that the Applicant's PSHA hazard results are conservative or overly conservative. In sum, the State insists that the evidence does not support a finding that the Staff may rely on claimed conservatisms in the Geomatrix PSHA or a 5000-year benchmark probability as rationale for the PFS 2000-year DBE exemption request. State Findings ¶ 494.

F.130 We do not agree with the State's claim. Although the State points to evidence of surface rupture of Late Quaternary deposits by the East fault, it did not show that the Staff was unaware of such evidence -- and, indeed, the State concedes that Dr. Stamatakos explicitly recognized this factor. State Findings ¶ 483. Based on our review of the evidence, including the Staff's evaluation of the "evidence" referred to in the Geomatrix report, we do not share the State's view that the Staff's slip tendency analysis should be rejected on the grounds that it is based on guesses or hypothetical modeling. Moreover, even if we were to accept the State's claims, they do not address or affect the Staff's conclusion, based on its 3D Stress slip tendency analysis, that the Geomatrix analysis is conservative insofar as it assumes that portions of the East fault near the site have the same tendency to slip as more distant portions of the East fault (whose orientation is more favorable to slip). See SER at 2-38 to 2-39; Stamatakos/Chen/McCann Post Tr. 8050, at 13-16.

F.131 During the hearing, although Dr. Stamatakos recognized that the data in the Geomatrix report reflect a factor of three difference in slip rates between the Wasatch and Stansbury Faults (1.1 mm vs. 0.4 mm) -- he did not retract his stated view that other data (*i.e.*, the Martinez GPS data) show as much as a factor of ten difference in those slip rates, as stated in the Staff's SER. Tr. at 8235-38 (Stamatakos); Stamatakos/Chen/ McCann Post Tr. 8050, at 17. Even if the Wasatch slip rate is only three times greater than the Stansbury Fault's slip rate, that is still a substantial difference -- which should result in a relatively greater earthquake hazard at Salt Lake City due to the Wasatch Fault as compared to the hazard at the PFS site due to the Stansbury Fault, but which is not observed in a comparison of the seismic hazard curves for the PFS site and Salt Lake City. See Staff Exh. JJ, at 5; Stamatakos/Chen/McCann Post Tr. 8050, at 16-17.

F.132 Further, while the State asserts that the results of two different PSHAs cannot be compared, we find no reason why a valid comparison of the resulting seismic hazard curves cannot be made, at least for purposes of examining, even on a crude basis, whether one of those analyses produced seismic hazard curve results which are palpably greater than expected. Moreover, we note that each of the hazard curves in question (for PFS, Salt Lake City, and the I-15 sites) were prepared by established, professional organizations (Geomatrix, the USGS, and Dames & Moore, respectively), and each of their PSHAs were available for review by the parties. See Staff Exh. JJ, at 3-5. Thus, we do not share the State's misgivings about the usefulness of Dr. Stamatakos' comparison of the seismic hazard curves produced by these three PSHA studies.

d. Comparison of the Applicant's Design Proposal with State Standards for Highways and Bridges

F.133 During the hearing, the State insisted that the 2000-year mean return period for the PFS facility does not ensure an adequate level of conservatism because design ground motion levels for certain new Utah building construction and highway bridges are more stringent. The State's conclusion was based on the observation that, for example, the International Building Code 2000 (IBC-2000) will, when in effect, require a MRP of approximately 2500 years for the DBE, which is greater than the 2000-year MRP DBE proposed for PFS. Cornell Post Tr. 7856, at 50-51.

F.134 As we have previously discussed, all parties are in agreement that in order to determine the level of safety achieved by an applicable design one has to take a two-handed approach, addressing both the mean return period of the DBE and the conservatisms embodied in the applicable design procedures and criteria. See Cornell Post Tr. 7856, at 53-54; Tr. at 9120-21 (Arabasz); Tr. at 12,804-05 (Bartlett). Therefore, it would be

inappropriate to compare solely the 2000 mean return period DBE of the PFS facility with the higher MRP DBE of the IBC-2000 or other codes.

F.135 The design procedures and acceptance criteria of the IBC-2000 are much less conservative than those specified by the NRC SRPs. For example, a first step of the IBC-2000 design procedures and criteria is to multiply the DBE by two-thirds, which at the PFS site would reduce the effective IBC-2000 DBE MRP from 2500 years to about 800 years. Cornell Post Tr. 7856, at 51-52, Tr. at 7898-7902 (Cornell). Only in the case of those "essential structures" that merit the IBC-2000 "importance factor" of 1.5 is this two-thirds reduction, in effect, recovered. Cornell Post Tr. 7856, at 51-52.

F.136 Even for those "essential structures" for which this reduction is in effect recovered, the model building codes' design procedures and acceptance criteria are significantly less conservative than those in the SRP. The IBC-2000 and UBC model building codes permit much more liberal allowances for the benefits of post-elastic behavior than either DOE-STD-1020-94 PC-3 and PC-4 criteria, or the NRC SRPs. Cornell Post Tr. 7856, at 51-52. The net effect of the UBC design and acceptance criteria is an RR of only 2 for essential buildings and structures, which is similar to that achieved by the IBC. Cornell Post Tr. 7856, at 52. By contrast, facilities designed to the NRC SRPs typically have RRs of 5 to 20 or more. These differences represent a factor of 2.5 to 10 or more in increased conservatism (as measured by RR) in the design procedures for nuclear facilities versus those in model building codes, even if the multiplier of two-thirds in the IBC-2000 is ignored. Cornell Post Tr. 7856, at 51-52. Thus, the PFS facility structures, even though designed using a lower MRP DBE than the starting point for determining the seismic ground motions under the IBC-2000 or UBC model building codes, would be much stronger and able to

withstand greater ground motions than a structure designed to the ostensibly higher MRP DBE specified in IBC-2000.

F.137 Thus, while the MRP DBE under the IBC-2000 is 25% larger than the proposed MRP for the PFS facility, the more conservative design procedures and criteria of the ISFSIs SRP will ensure that the SSCs at the PFS facility have a mean annual probability of failure that is several times (2 to 8 or more) lower than buildings designed to IBC-2000 standards. Moreover, all PFS facility important-to-safety SSCs have RR factors sufficient to provide a probability of failure of 1×10^{-4} or lower, i.e., at least two times lower than essential facilities designed to the IBC-2000. Additionally, as discussed earlier, a number of key important-to-safety SSCs in the PFS facility have great robustness and/or fractional operating periods that reduce their probabilities of failure even further. Cornell Post Tr. 7856, at 52. Therefore, structures and components important to public health and safety at the PFS facility would be much less likely to fail in an earthquake than would other facilities essential for public health and safety in the event of an earthquake, such as bridges, hospitals, or fire stations.

G. Compliance with the Radiation Dose Limits

G.1 The State contends that the Board is faced with the question of whether the radiation dose limit applicable to the PFS analysis should be based on 10 C.F.R. § 72.104(a) or 10 C.F.R. § 72.106(b). According to the State, under current regulations, PFS must analyze accident dose limits from a deterministic earthquake (similar to a 10,000-year DBE) under 72.106(b). If, in fact, the record shows that PFS has provided supportable analysis for a 10,000-year mean return period event, then the State insists that 72.106(b) is the applicable standard. If, however, PFS is relying on analyzing releases from a 2000-year DBE to satisfy its exemption request, then by allowing PFS to use the 72.106(b) standard, the State insists the Board would be expanding the effect of the PFS request to be exempted from 10 C.F.R. § 72.102 to a dilution of the standard in 10 C.F.R. § 72.106(b). See State Findings ¶ 554.

G.2 A careful review of the Commission's regulations in 10 C.F.R. Part 72, however, demonstrates that there are two sets of requirements pertaining to offsite radiological dose consequences associated with the licensing of an ISFSI. First, the following requirements, established in 10 C.F.R. § 72.104, apply to normal operations and anticipated occurrences:[46]

> (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.

10 C.F.R. § 72.104 (a)(1-3).

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[46] "Anticipated occurrences" are defined as denoting "minor events such as upsets, leaks, and spills," in contrast to "accidents," which "are considered to have a lower probability of occurrence." South Carolina Electric and Gas Co. (Virgil C. Summer Nuclear Station, Unit 1), LBP-79-11, 9 N.R.C. 471, 476 (1979).

G.3 Second, the Commission has established the following requirements in 10 C.F.R. § 72.106(b), with respect to design basis accidents:

> (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).

10 C.F.R. § 72.106(b).

G.4 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.

1. Dose Consequences Analysis Conducted by Applicant

G.5 To examine normal operational doses at the PFS facility, 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 PFS facility capacity of 4000 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. Testimony of Krishna P.Singh and 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 Singh/Soler/Redmond] Post Tr. 12,044, at 7-8.

G.6 In addition, although it has been demonstrated that the casks will not tip over, PFS analyzed a non-mechanistic hypothetical tip-over event in accordance with applicable regulatory guidance. Singh/Soler Post Tr. 5750, at 29; NRC Staff Testimony of Michael D. Waters Concerning Radiological Dose Considerations Related to Unified Contention Utah L/QQ, Part E (Seismic Exemption) [hereinafter Waters] Post Tr. 12,215, at 7. The results of this analysis show that all stresses on the storage cask remain within the allowable values of the HI-STORM 100 CoC, assuring the integrity of the MPC confinement boundary with large margins of safety. Singh/Soler/Redmond Post Tr. 12,044, at 6-7. Therefore, there would be no releases of radioactivity even in the event of a postulated tip-over.

G.7 Holtec qualitatively evaluated the potential radiological consequences of a hypothetical cask tip-over event in its Final SAR (FSAR) for the HI-STORM 100 and determined that the impact of the cask on the pad would only cause localized damage to the concrete and outer shell of the storage cask at the point of impact, reducing somewhat the roundness of the storage cask in the immediate area of impact. Singh/Soler/Redmond Post Tr. 12,044, at 7, 15-16.

G.8 The HI-STORM 100 cask consists of both a radial concrete shield and an outer steel shell. The concrete is fully encased in a steel structure, and four large steel ribs are located between the inner and outer shell. It is physically impossible for the concrete to be lost in the event of impact damage. A local deformation would not significantly affect the shielding performance of the storage cask, since the same mass of steel and concrete would still be present. Singh/Soler/Redmond Post Tr. 12,044, at 15-16. Because radiation shielding is dependent on mass rather than thickness (Tr. at 12,479 (Resnikoff)), rearrangement of the

mass present in the shielding will not result in significant changes in radiation dose levels, since loss of mass in one location of the cask will be offset by an increase in mass in another location. Tr. at 12,148-50 (Soler/Redmond); Tr. at 12,244 (Waters). Additionally, the local deformations would occur at the top of the storage cask, whereas the radiation doses are greater at the middle of the cask. Tr. at 12,551-52, 12,567-68 (Soler/Redmond). Therefore, any increase in the radiological dose levels due to localized deformation of the cask would, at most, be minimal. Singh/Soler/Redmond Post Tr. 12,044, at 15-16.

G.9 Holtec also evaluated the radiological dose consequences resulting from the hypothetical tip-over of multiple casks. Singh/Soler/Redmond Post Tr. 12,044, at 7-12. Hypothetical multiple cask tip-overs would likely result in similar localized damage for each of the casks tipped over, with no significant aggregate effect on radiological doses at the owner-controlled area (OCA) boundary. Singh/Soler/Redmond Post Tr. 12,044, at 8-9, 10; Waters Post Tr. 12,215, at 9-11. The greatest potential for increase in radiological doses at the boundary would not be due to damage to the cask or the MPC, but to the possibility that the bottom of the cask, which has less radiation shielding, might face the OCA boundary. Singh/Soler/Redmond Post Tr. 12,044, at 8-9,10; Waters Post Tr. 12,215, at 14-17; Amended State of Utah Testimony of Dr. Marvin Resnikoff Regarding Unified Contention Utah L/QQ (Seismic Exemption Dose Exposure) [hereinafter Resnikoff] Post Tr. 12,349, at 10.

G.10 Holtec evaluated the effect that 4000 tipped-over casks would have on the radiation dose at the OCA boundary, compared to the doses due to releases from the casks in their normal upright position. In the upright position, the side of the storage cask is in a direct line of sight from all equidistant locations from the cask, the top is not visible from any location, and the bottom is shielded by the ground. In a tipped-over position, the top or bottom of the

cask would be visible from some locations and not from others, while the side of the storage cask cylinder (now horizontal) would also be visible from some locations and not others. Additionally, since the storage cask would be lying on its side, a large portion of the outer radial surface of the cask would be shielded by the ground. From its evaluation of the geometry of the storage cask Holtec concluded that, overall, the decrease in dose rate from the sides of a tipped-over storage cask should more than compensate for the increase in dose rate from the top or bottom of the cask. Further, in the event of multiple casks tipping over, the orientation of the tipped-over casks would be random and the bottoms and tops of many of the casks would be shielded from the OCA boundary by other casks. Singh/Soler/Redmond Post Tr. 12,044, at 8-10.

G.11 Thus, in the event of a beyond-design-basis accident that caused the tip-over of all, or a significant portion of the 4000 casks at the PFS site, the radiological dose levels at the OCA boundary would not be increased from the 5.85 mrem per year for normal operations which had previously been calculated. Thus, there are approximately three orders of magnitude of margin between the expected dose rate at the OCA boundary for 4000 casks in a tipped-over condition compared to the 5 rem accident dose limit in 10 C.F.R. § 72.106(b). Singh/Soler/Redmond Post Tr. 12,044, at 10-11.

G.12 In addition, many conservatisms were included in the PFS calculation of the 5.85 mrem/year dose at the OCA boundary. These included:

- The calculation assumed that all 4000 casks contain fuel with a burnup of 40,000 MWD/MTU and a cooling time of 10 years. This is physically impossible, since the MPCs will be delivered over many years and each additional year of cooling further reduces the radiation source term. A more realistic value of 35,000 MWD/MTU and a cooling time of 20 years has been used in other PFS analyses. These more realistic assumptions result in a

greater than 50% reduction in the calculated normal doses at the site boundary, from 5.85 mrem/year to 2.10 mrem/year.

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

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

Using more realistic assumptions would significantly reduce the calculated radiological dose levels, further decreasing the expected radiation dose consequences of the hypothetical tip over of all 4000 casks at the PFS facility. Singh/Soler/Redmond Post Tr. 12,044, at 11.

a. Time Spent at Boundary

G.13 The PFS site-specific analysis for radiation dose levels uses a 2000 hours per year occupancy time for calculating normal operating dose levels (conservatively based on an assumed worker at the site boundary 40 hours per week for 50 weeks a year), whereas the HI-STORM 100 CoC uses 8760 hours per year to calculate the normal operating dose. Singh/Soler/Redmond Post Tr. 12,044, at 11-12, 13-14. The dose limits established by 10 C.F.R. § 72.104(a) apply to "any real individual who is located beyond the controlled area," not to a hypothetical person at the OCA boundary. Thus, occupancy time for normal operating conditions is determined using a real person standard, which takes into account the site-specific circumstances at a facility. This interpretation is endorsed by NRC regulatory guidance. <u>See</u> Singh/Soler/Redmond Post Tr. 12,044, at 12. Likewise, for

accident conditions, the 5 rem limit would apply to real individuals, and site-specific circumstances would similarly need to be taken into account, including any remedial measures that may be taken during extended accident conditions (e.g., shielding or moving persons away from OCA boundary). See Tr. at 12,072 (Redmond); Tr. at 12,266-67 (Waters).

G.14 Contrary to the PFS analysis, Dr. Resnikoff, who testified in support of the State's claim, assumed that a person was at the fence post 24 hours per day, which totals 8760 hours per year. Dr. Resnikoff, notes that this change in the calculation, by itself, would increase the radiation dose at the controlled area more than fourfold. Resnikoff Post Tr. 12,349, at 6.

G.15 The State argues that the PFS assumption that an individual will be at the controlled area boundary for 2000 hours in a year is not consistent with the language in 10 C.F.R. § 72.106. The regulation refers to "[a]ny individual located on or beyond the nearest boundary of the controlled area." 10 C.F.R. § 72.106. This stands in contrast to section 72.104, which refers to "any real individual." 10 C.F.R. § 72.104. The State insists that the difference in the language is intentional and must be followed. The State contends that while it may be appropriate to assume, for a "real" individual, that a person does not spend 24 hours a day at the controlled area boundary, that assumption is not appropriate for "any individual." According to the State, the term "any individual" must be assumed to include individuals who are present at the fence post all year. State Findings ¶ 559. In fact, in the CoC for the HI-STORM 100, the State argues that the NRC Staff agreed with a comment by Dr. Resnikoff that 8760 hours should be used for estimating the dose at the site boundary. State Findings ¶ 559 (citing 65 Fed. Reg. 25,241, 25,245 (May 1, 2000)). As long as it is possible that some individual will live at the controlled area boundary and spend his or her

days there, the State argues that PFS must base its calculation on radiation exposure of 24 hours per day, or 8760 hours. State Findings ¶ 559.

G.16 The State claims that its position is strengthened by the fact that PFS has no way of excluding anyone from the northern part of the controlled area boundary, because it does not own the property. Moreover, the State notes that it is difficult to predict what conditions will be in 20 years -- or 40 years, when PFS expects its license will terminate. State Findings ¶ 557.

G.17 The Board agrees with the Applicant. Based on the land use surrounding the PFS facility, we find the assumed 2000 hours per year occupancy time to be adequate. Tr. at 12,066-67 (Redmond). The only individuals likely to be present at the OCA boundary would be workers, who are assumed to be present 40 hours per week for 50 weeks in a year to produce an upper bound of 2000 hours per year exposure at the site boundary. Singh/Soler/Redmond Post Tr. 12,044, at 12. Furthermore, as previously explained, the criterion in 10 C.F.R. § 72.106(b) applies to accident dose rates, not to the dose rates for everyday operations.

G.18 Thus, using 2000 hours per year occupancy time is appropriate for normal operations, given the site-specific circumstances at the PFS facility. Singh/Soler/Redmond Post Tr. 12,044, at 12; Tr. at 12,066-67 (Redmond). Such an occupancy time would also be adequate for postulated accident conditions. Our findings are further strengthened by the notion that in addition to measures to limit occupancy of areas of potential radioactive contamination, remedial measures, such as the construction of an earthen berm, could easily be undertaken to assure that radiological dose levels at the boundary of the OCA do not exceed regulatory limits following a beyond-design-basis earthquake. <u>See</u> Tr. at 12,583-84 (Donnell); Tr. at 12,622-23 (Resnikoff); Tr. at 12,266-67 (Waters).

b. Tip-over Analysis

(i) Duration

G.19 The State contends that PFS did not make an estimate of the duration of an accident. State Findings ¶ 560. Further, the State contends that there is no evidence in the record that PFS has any contingency plan for uprighting casks if they tipped over. State Findings ¶ 560. Nevertheless, the State contends that PFS testified that it would be reasonable to assume that an accident lasts for 30 days. According to the State, PFS did not attempt to justify this assumption, but merely relied on a NRC regulatory guidance. State Findings ¶ 560.

G.20 In addition, the State points out that the NRC Staff also assumed that the accident event would last 30 days. State Findings ¶ 561 (citing Tr. at 12,265-66 (Waters)). This assumption was based on NUREG-1567. Standard Review Plan for Spent Fuel Dry Storage Facilities, NUREG-1587 (Mar. 2000) (Staff Exh. 53). While Staff witness Waters testified that he believed 30 days was reasonable, the State argues that there is no evidence that his opinion was based on the existence of any contingency plan or actual knowledge of how long it would take to restore the site to pre-accident conditions. State Findings ¶ 561 (citing Tr. at 12,267 (Waters)). Instead, the State contends that Mr. Waters relied on what he called "fundamental principles of radiological protection" -- time, distance, and shielding. State Findings (quoting Tr. at 12,266 (Waters)). Mr. Waters also expected people near the fence post to have been moved away within 30 days of an accident. Id. at 12267.

G.21 Furthermore, the State notes that, while it may be possible to mitigate radiation doses by installing steel plates around the periphery of the site as an interim measure, such measures do not terminate the accident. The State believes the principles expressed by Mr. Waters, of "time, distance, and shielding," are out of place in this context, and that the Staff's

assumption of a 30-day accident is based on a fundamental misconception and misapplication of 10 C.F.R. § 72.106. State Findings ¶ 569. The State argues that the Staff essentially adopts the assumption that the “accident,” as the term is used in section 72.106, ends when people are removed from the area or some temporary barriers have been erected. State Findings ¶ 569. However, the State believes that the regulation can only be interpreted to mean that the accident has ended when the casks are set upright and restored to their previously designed condition in which the doses they emit are within the limits of 10 C.F.R. § 72.104(a). State Findings ¶ 569.

G.22 The State contends that the clear purpose of section 72.106 is to ensure that the design of a proposed ISFSI is adequate to protect against excessive radiation doses in the event of an accident. The elements of the design consist of the casks and pads themselves, and the size and configuration of the controlled area. The State argues that there is no reference in the standard to contingency measures, whether planned or ad hoc, and insists that such measures are not part of the design of the facility. State Findings ¶ 570.

G.23 The State argues it would violate the principle of defense-in-depth if contingency measures may be relied on as a substitute for an adequately designed ISFSI and controlled area. The State believes that the physical design of a facility must be evaluated on its own merit against NRC standards for safe facility designs. According to the State, contingency measures constitute additional, independent steps for protecting the public in the event of an accident, not substitutes for an adequate design. Otherwise, the State argues that the design requirements of 10 C.F.R. § 72.106 could be diluted merely by listing post-accident measures that could or would be taken to mitigate doses to the public. State Findings ¶ 571.

G.24 We do not agree with the State’s concerns. Even assuming a worst-case cask tip-over and loss of all hydrogen shielding event as postulated by the State, the 5 rem

radiological dose limits set by 10 C.F.R. § 72.106(b) would not be exceeded within at least 36 years of a beyond design basis seismically induced accident. See Tr. at 12,618 (Resnikoff). Thus, we find the State's attack upon the Applicant's calculations to be without merit.

(ii) Multiple Cask Tip-over v. Single Cask Tip-over

G.25 According to the State, in estimating radiation doses at the site boundary in a cask tip-over event, it is necessary to make some assumptions about how the casks will fall. The State believes that the orientation of the casks, whether they have fallen onto each other, and whether they are stretched or flattened by the force of falling on each other, will have an effect on the dose that is calculated. State Findings ¶ 575.

G.26 Dr. Resnikoff performed an analysis assuming that the bottoms of a row of casks face the fence post. He assumed this configuration because, in his opinion, it was conservative. Tr. at 12,428 (Resnikoff).

G.27 The NRC Staff performed calculations, assuming essentially the same configuration as assumed by Dr. Resnikoff. Tr. at 12,243 (Waters). Mr. Waters assumed that 50 casks would be tipped over facing a northern direction. He considered that this would be the bounding case. Id. at 12,257.

G.28 In that regard, the State requests that the Applicant either prepare a defensible model of the configuration of tipped-over casks, or to make conservative assumptions about their configuration. Otherwise, the State argues that the Board cannot find the reasonable assurance of safety that the regulations require. State Findings ¶ 579.

G.29 We find both the predicate for the State's claim and its substance to be groundless. During the hearing, all the witnesses agreed that if a beyond design basis accident were to

cause HI-STORM 100 storage casks at the PFS facility to tip over, the orientation of such casks would be random. Tr. at 12,428 (Resnikoff); Singh/Soler/Redmond Post Tr. 12,044, at 8-10. PFS analyzed a realistic scenario where the casks were presumed to be randomly oriented and determined that there would be no effective change in radiological dose levels from normal operating levels. Singh/Soler/Redmond Post Tr. 12,044, at 7-12.

G.30 However, even if we were to assume all eighty casks in an outside row were to tip with their bottoms perpendicular to the OCA boundary, as Dr. Resnikoff hypothesized during his testimony, the radiological dose limit of 10 C.F.R. § 72.106(b) would never be exceeded during such a tip-over event. Mr. Waters agreed with the conclusions reached by Holtec's analysis of a multiple cask tip-over noting that the dose rates from the sides of the casks would be diminished in a tipped over condition and that overall "one would not expect to see a significant increase . . . in off-site dose rates at any point of the OCA boundary." Waters Post Tr. 12,215, at 15. In addressing a worst-case, cask tip-over hypothetical, Mr. Waters concluded that the increased dose rates under that scenario were well below the radiological dose limits of 10 C.F.R. § 72.106(b). Waters Post Tr. 12,215, at 17. For these reasons, we find the State's concerns to be without merit.

(iii) Angular Velocity

G.31 Based on the Altran Report, Dr. Resnikoff postulated that the Holtec analysis of cask tip-over was inadequate because, contrary to the assumptions made by Holtec, the initial angular velocity of a falling cask may be greater than zero. Resnikoff Post Tr. 12,349 at 8. However, Dr. Resnikoff has never calculated an initial angular velocity for any storage cask tip-over. Tr. at 12,403-04 (Resnikoff). Instead, Dr. Resnikoff testified that he asked "[the State's] other experts what is the angular velocity and is zero correct, and their opinion [was] that the zero initial angular velocity could be greater than zero." Tr. at 12,403 (Resnikoff).

G.32 There is no testimony by any State witness that supports the conclusion that an initial angular velocity greater than zero would be either realistic or more appropriate for a cask tip-over at the PFS facility. State soils expert Dr. Bartlett summarily asserted, in reference to the Holtec non-mechanistic cask tip-over analysis, that "the tipover event postulated that . . . the cask would be perched on its edge with zero angular velocity. During an earthquake, that's not true. If we go to tip-over, we have some angular velocity." Tr. at 12,870-71 (Bartlett). However, Dr. Bartlett admitted that he had not been involved in any calculations of cask stability or the results of a tip-over event. Tr. at 12,870 (Bartlett).

G.33 The Holtec analyses of dynamic cask behavior have shown that the behavior of the cask is characterized by tilting from the vertical resulting in a plane of precession for a certain duration in the course of the earthquake event, resulting in an oscillatory rocking motion with limited return to the vertical position until the rocking finally ends when the earthquake subsides. Singh/Soler/Redmond Post Tr. 12,044, at 16. If the earthquake ground motions were assumed to be increased to the point at which a cask would tip over, the initiating angular velocity propelling the cask towards the ground would be quite small. Singh/Soler/Redmond Post Tr. 12,044, at 16-17.

G.34 Furthermore, the precessionary motion of the cask enables it to remain stable after the center of gravity of the cask is well past the "center-of-gravity-over-corner" position. As a result of this precessionary motion, the location of the cask's center of gravity is likely to be much lower than in the static tip-over scenario (where tip-over begins as soon as the center of gravity crosses the vertical plane containing the axis of overturning rotation). The combination of a shorter distance to fall and a negligible initial angular velocity propelling the tip-over further supports the assumption of an initial angular velocity of zero, because a cask tipping away from precessionary motion is expected to have substantially less kinetic energy

of collision than one tipping from a zero velocity with the center of gravity over corner. Singh/Soler/Redmond Post Tr. 12,044, at 17. Thus, we find the Applicant's assumption of an initial angular velocity of zero is appropriate.

(iv) Deceleration

G.35 Dr. Resnikoff's pre-filed testimony indicated that his concern regarding the possibility of the top of the cask decelerating at a rate in excess of 45g was premised on the initial angular velocity being greater than zero. See Resnikoff Post Tr. 12,359, at 8. He changed his testimony at the hearing and acknowledged that damage to the cladding on fuel rods contained in the fuel assemblies within the storage cask would not be an issue unless the assemblies were subjected to an acceleration of at least 63g. Tr. at 12,409-10 (Resnikoff). Dr. Resnikoff did not know how large an initial angular velocity would be required to exceed the 63g limit, but conceded that an initial angular velocity of greater than zero would be required. Tr. at 12,410-12 (Resnikoff).

G.36 The HI-STORM 100 FSAR places a 45g limit on the deceleration for the top of the HI-STORM 100 storage cask in the event of a cask tip-over event. This is a licensing limit that does not represent the actual ability of the storage cask, the MPC, or the fuel assemblies to maintain both containment and radiation shielding. Singh/Soler/Redmond Post Tr. 12,044, at 17-18; Tr. at 12,158 (Singh). The spent fuel assemblies have design margins that allow them to withstand accelerations up to at least 63g. Singh/Soler/Redmond Post Tr. 12,044, at 17-18; Tr. at 12,158 (Singh). There has been no analysis of postulated beyond-design-basis accidents that resulted in decelerations greater than the 45g limit in the HI-STORM 100 FSAR, let alone the 63g design limit. See Tr. at 12,411 (Resnikoff).

G.37 The MPC also has substantial design margins beyond the 45g level. A hypothetical 25 ft. end drop of a loaded canister on a hard concrete foundation resulted in a computed strain in the confinement boundary of 41% of the failure strain limits for the canister material. Singh/Soler/Redmond Post Tr. 12,044, at 18. The computed strain showed that the MPC could experience a maximum deceleration of approximately 300g without loss of confinement. Tr. at 12,075 (Singh).

G.38 Thus, exceeding the 45g deceleration limit imposed on the top of the canister in the HI-STORM 100 FSAR would not result in increased radiological dose consequences. Decelerations would have to exceed 63g before there was a concern regarding the possible effect of such decelerations on the fuel assemblies contained in the MPC. Tr. at 12,409-11 (Resnikoff). Moreover, due to the large margins of safety built into the design of the MPC, much larger decelerations than 45g would be required before the containment function of the MPC was compromised. Singh/Soler/Redmond Post Tr. 12,044, at 18.

c. Dose Calculations

G.39 Dr. Resnikoff's prefiled testimony contained two radiation dose calculations: an estimation of the gamma dose from the bottom of eighty storage casks, with their bottoms facing the OCA boundary (State Exh. 141), and an estimation of the neutron dose from a cask based on the amount of "water evaporated" from the concrete shielding (State Exh. 143). Beginning with amended State Exh. 141A, Dr. Resnikoff combined both scenarios -- cask tip-over and loss of hydrogen shielding -- to portray a total, worst-case radiological dose at the OCA boundary. The Board, however, does not agree with these calculations.

G.40 The dose exposure that Dr. Resnikoff ultimately calculated at the OCA boundary was less than 150 mrem for the first year, assuming a hypothetical person were at the OCA

boundary for the entire year (which, as discussed above, is not realistic). Radioactive decay would reduce this dose exposure in subsequent years. Thus, assuming that the casks remained on the ground indefinitely with no remedial actions being taken, the 5 rem limit would not be exceeded for a person continuously stationed at the OCA boundary. Tr. at 12,619-20 (Resnikoff).

G.41 Dr. Resnikoff's neutron dose calculation, State Exh. 143, is intended to represent the "increased neutron dose due to reduced shielding" in order to estimate "the increase in dose to workers due to neutrons . . . 1 meter from the cask mid-height if all of the water evaporates from a HI-STORM cask." Resnikoff Post Tr. 12,349, at 12. In this calculation, Dr. Resnikoff assumed that there is some unspecified temperature at which no hydrogen is present in the concrete or the aggregate material contained in the concrete. Tr. at 12,420-23 (Resnikoff). Dr. Resnikoff did not try to calculate the actual amount of hydrogen loss that would take place if a HI-STORM 100 cask tipped over, nor did he have any idea how to calculate the thermal degradation of the cask's concrete over time (Dep. of Marvin Resnikoff (Oct. 29, 2001) (PFS Exh. 240) at 90-93; PFS Exh. 240 at 90-93); nor had he ever used computer programs that computed the temperature of concrete over time (id.). He also did not know how to estimate the reduction in shielding due to concrete heating up over time (id. at 93). Indeed, this was his first attempt to examine thermal degradation in concrete and quantify the loss of radiation shielding that may result. Tr. at 12,418-20 (Resnikoff).

G.42 The premise of Dr. Resnikoff's calculation of the lack of any hydrogen in the concrete due to evaporation of water is unrealistic. It is not easy to evaporate water within concrete because it is in a confined space, and as the water evaporates the air pressure increases. In turn, the increased air pressure will convert the water vapor back to liquid. Likewise, concrete does not lose its moisture content as easily as water might evaporate from a free

surface. In order for large, extensive, sustained water evaporation from the concrete to occur, exposure to high temperatures for a period of months will be necessary. Moreover, it is physically impossible for cask heat-up to release hydrogen contained in the aggregate within the concrete. Singh/Soler/Redmond Post Tr. 12,044, at 26. In an actual simulation of the worst case scenario for heat degradation of the HI-STORM 100 cask, the Staff indicated that neutron dose rates due to thermal degradation would result in a much smaller increase of computed neutron dose rates than those predicted using the unrealistic assumptions in Dr. Resnikoff's analysis. Waters Post Tr. 12,215, at 11-14. In addition to the erroneous assumptions made by Dr. Resnikoff, his neutron dose calculation was also in error because he used the wrong neutron dose from the SAR, which inflated his calculated neutron dose. Tr. at 12,607-08 (Resnikoff).

G.43 Dr. Resnikoff's gamma dose calculation at the OCA boundary was premised on the bottoms of eighty storage casks lined up in a row all facing the OCA boundary. State Exh. 141 at 3-5, 6-8. Such an arrangement is "highly unrealistic." Singh/Soler/Redmond Post Tr. 12,044, at 21.

G.44 Dr. Resnikoff made a total of nine different corrections or changes to his overall dose calculation at four different points in the proceeding. These changes are identified in the testimony of the PFS witness by Dr. Redmond as well as by Dr. Resnikoff in the amendments to his pre-filed direct testimony and in oral testimony at the hearing. See, e.g., Singh/Soler/Redmond Post Tr. 12,044, at 23; Tr. at 12,374-75, 12,428-30 (Resnikoff); State Exh. 141A).

G.45 We choose not to engage here in a detailed analysis of the impact of the various changes in Dr. Resnikoff's dose rate calculations. It suffices to indicate that the number and nature of those changes undercuts confidence in the accuracy of his analyses.

G.46 An important error in Dr. Resnikoff's dose calculations is that he did not consider the effect of radioactive decay. The majority of the gamma radiation from the SNF comes from the radioactive decay of Cobalt-60 and Cesium-137, with Cobalt-60 being the main gamma emitter for radiation emanating off the bottom of the cask, accounting for 90% of the total gamma dose calculated by Dr. Resnikoff. Tr. at 12,619-20, 12,624-25 (Resnikoff). Although the half-life of Cobalt-60 is approximately 5 years, Dr. Resnikoff neglected to take radioactive decay into account when arriving at his dose estimates. Tr. at 12,617-20 (Resnikoff).

G.47 Taking into account only the radioactive decay of just the Cobalt-60 and ignoring the decay of other radioisotopes will result in a total radiation dose over 50 years of 2582.1 mrem, or 2.58 rem. In fact, as Dr. Resnikoff admitted, taking into account radioactive decay the 5 rem accident limit specified in 10 C.F.R. § 72.106(b) will not be reached (Tr. at 12,620 (Resnikoff)) no matter how long one assumes that the casks remain in a worst case tip-over and total loss of hydrogen shielding condition, and disregarding any remedial actions that might be taken in the intervening period by PFS or others.

2. Complete Air Inlet Blockage Under HI-STORM 100 Certificate of Compliance

G.48 A thermal analysis was conducted by the Applicant to support the HI-STORM 100 CoC. Staff Exh. FF. The thermal analysis makes a very conservative assumption that no heat transfer to the surrounding air will occur. In effect, the calculation assumes that the cask not only has all its air inlet ducts completely blocked, but that it is shrouded in a "heavy blanket" that prevents heat transfer. Tr. at 12,152-53 (Singh). Under these extreme conditions, the short-term temperature limit of the concrete would be reached in 33 hours. Id.

G.49 Due to the way the air inlets are configured, it is physically impossible for all the air inlet vents of a cask to be blocked due to a cask tip-over. Singh/Soler/Redmond Post Tr. 12,044, at 24-25. Therefore, even in a tipped-over condition, heat transfer continues to take place and the air inlet ducts continue to dissipate heat. Thus, the concrete temperature of the HI-STORM cask would be expected to remain below the short term limit. Singh/Soler/Redmond Post Tr. 12,044, at 25-27. More importantly, the spent fuel canister is not expected to melt. See Tr. at 12,205-07 (Singh).

G.50 Even assuming all inlet vents were blocked, the bounding steady state temperature for the concrete would be well below the 600 degrees Fahrenheit necessary for extensive water evaporation. Singh/Soler/Redmond Post Tr. at 12,044, at 25-27. Both conduction and radiation of heat will still occur from a storage cask that has all its air inlet vents blocked. Tr. at 12,300-01 (Waters). Therefore, the evaporation of water from the concrete of a tipped-over cask would be small, even if the cask remained in a tipped-over position for a period of months. Singh/Soler/Redmond Post Tr. 12,044, at 26.

G.51 Exceedance of the short-term temperature limit of the concrete does not materially affect public health and safety because it has no effect on the containment of the spent fuel within the storage cask; and there would be no significant reduction in the shielding effectiveness of the system. Tr. at 12,154-55 (Singh).

X. CONCLUSIONS OF LAW

The Licensing Board has considered all of the material presented by the parties on Contention Utah L/QQ (Geotechnical). Based upon a review of the entire evidentiary record in this proceeding and the proposed findings of fact and conclusions of law submitted by the parties, and in accordance with the views set forth in Parts I through IX above -- which we believe are supported by a preponderance of the reliable, material and probative evidence in the record -- the Board has decided the matters in controversy concerning this contention and reaches the following legal conclusions in favor of the Applicant:

There is reasonable assurance the spent fuel casks would not tip over during a design basis seismic accident. See Findings E.5-.8, .21, .52, .54, .168, .171-.74; F.62-.64.

If a spent-fuel storage cask were to tip over, there is reasonable assurance the spent fuel canister inside would not break or melt. See Findings E.9; F.65; G.6-.9, .36-.38, .49-50.

In any event, as has been previously noted by the Commission and the Board, even if the spent fuel canister were to break or melt, the absence of significant dispersive forces would mitigate the consequences of the accident. CLI-00-13, 52 NRC 23, 31 (2000). See also Findings F.34,.102.

Additionally we find the Applicant has met its burden in each of the following aspects of the contention:

Section C of Contention Utah L/QQ. Pursuant to 10 C.F.R. §§ 72.102(c) and (d), the Applicant has demonstrated that the program it implemented to determine the characteristics of the soils at the site provides reasonable assurance that the soil conditions are adequate for the proposed foundation loading.

Section D of Contention Utah L/QQ. Pursuant to 10 C.F.R. § 72.122(b)(2), the Applicant has demonstrated that the design of the structures, systems, and components important to safety at the facility provides reasonable assurance that anticipated earthquake phenomena will not impair their capability to perform their intended safety functions.

Section E of Contention Utah L/QQ. Pursuant to 10 C.F.R. § 72.7, both the Applicant and the Staff have provided adequate justification to support the conclusion that the Staff's grant of the Applicant's exemption request -- i.e., to use a PSHA methodology and a 2000-year design basis earthquake -- was authorized by law, will not endanger life or property or the common defense and security, and is otherwise in the public interest.

For the reasons set forth herein, it is, this 22nd day of May, 2003, ORDERED that:

1. Contention Utah L/QQ (Geotechnical) is RESOLVED in favor of the Applicant PFS.

2. Pursuant to 10 C.F.R. § 2.760(a), this Partial Initial Decision will constitute the FINAL ACTION of the Commission within forty (40) days of this date unless a petition for review is filed in accordance with 10 C.F.R. § 2.786(b), or the Commission directs otherwise.

3. Within fifteen (15) days after service of this partial initial decision (which shall be considered to have been served by regular mail for the purpose of calculating that date), any party may file a petition for review with the Commission on the grounds specified in 10 C.F.R. § 2.786(b)(4). The filing of a petition for review is mandatory in order for a party to have exhausted its administrative remedies before seeking judicial review. Within ten (10) days after service of a petition for review, any party to the proceeding may file an answer supporting or opposing Commission review. The petition for review and any answers shall conform to the requirements of 10 C.F.R. § 2.786(b)(2)-(3).

THE ATOMIC SAFETY
AND LICENSING BOARD*

/RA/

Michael C. Farrar, Chairman
ADMINISTRATIVE JUDGE

/RA/

Jerry R. Kline
ADMINISTRATIVE JUDGE

/RA/

Peter S. Lam
ADMINISTRATIVE JUDGE

Rockville, Maryland

May 22, 2003

* Copies of this Partial Initial Decision were sent this date by Internet e-mail transmission to counsel for (1) Applicant PFS; (2) Intervenors Skull Valley Band of Goshute Indians, OGD, Confederated Tribes of the Goshute Reservation, Southern Utah Wilderness Alliance, and the State of Utah; and (3) the NRC Staff.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

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 the foregoing LB PARTIAL INITIAL DECISION (REGARDING GEOTECHNICAL ISSUES) (LBP-03-08) have been served upon the following persons by deposit in the U.S. mail, first class, or through NRC internal distribution.

Office of Commission Appellate Adjudication
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Administrative Judge
Michael C. Farrar, Chairman
Atomic Safety and Licensing Board Panel
Mail Stop - T-3 F23
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Administrative Judge
Jerry R. Kline
Atomic Safety and Licensing Board Panel
Mail Stop - T-3 F23
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Administrative Judge
Peter S. Lam
Atomic Safety and Licensing Board Panel
Mail Stop - T-3 F23
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Sherwin E. Turk, Esquire
Catherine L. Marco, Esquire
Office of the General Counsel
Mail Stop - 0-15 D21
U.S. Nuclear Regulatory Commission
Washington, DC 20555-0001

Diane Curran, Esquire
Harmon, Curran, Spielberg & Eisenberg, L.L.P.
1726 M Street, NW, Suite 600
Washington, DC 20036

Joro Walker, Esquire
Director, Utah Office
Land and Water Fund of the Rockies
1473 South 1100 East, Suite F
Salt Lake City, UT 84105

Martin S. Kaufman, Esquire
Atlantic Legal Foundation
205 E. 42nd St.
New York, NY 10017

Docket No. 72-22-ISFSI
LB PARTIAL INITIAL DECISION (REGARDING
GEOTECHNICAL ISSUES) (LBP-03-08)

Denise Chancellor, Esquire
Assistant Attorney General
Utah Attorney General's Office
160 East 300 South, 5th Floor
P.O. Box 140873
Salt Lake City, UT 84114

Jay E. Silberg, Esquire
D. Sean Barnett, Esquire
Shaw Pittman
2300 N Street, NW
Washington, DC 20037-1128

John Paul Kennedy, Sr., Esquire
David W. Tufts, Esquire
Confederated Tribes of the Goshute
Reservation and David Pete
Durham Jones & Pinegar
111 East Broadway, Suite 900
Salt Lake City, UT 84105

Richard Wilson
Department of Physics
Harvard University
Cambridge, MA 02138

Tim Vollmann, Esquire
3301-R Coors Road N.W., #302
Albuquerque, NM 87120

Paul C. EchoHawk, Esquire
ECHOHAWK LAW OFFICES
151 North 4th Avenue, Suite A
P.O. Box 6119
Pocatello, ID 83205-6119

Marlinda Moon, Chairman
Sammy Blackbear, Sr., Vice-Chairman
Miranda Wash, Secretary
Skull Valley Band of Goshute Indians
P.O. Box 511132
Salt Lake City, UT 84151-1132

Stephen L. Simpson, Esquire
Office of the Solicitor
Department of the Interior
Division of Indian Affairs
1849 C Street, NW, Mailstop 6456-MIB
Washington, DC 20240

[Original signed by Evangeline S. Ngbea]

Office of the Secretary of the Commission

Dated at Rockville, Maryland,
this 22nd day of May 2003